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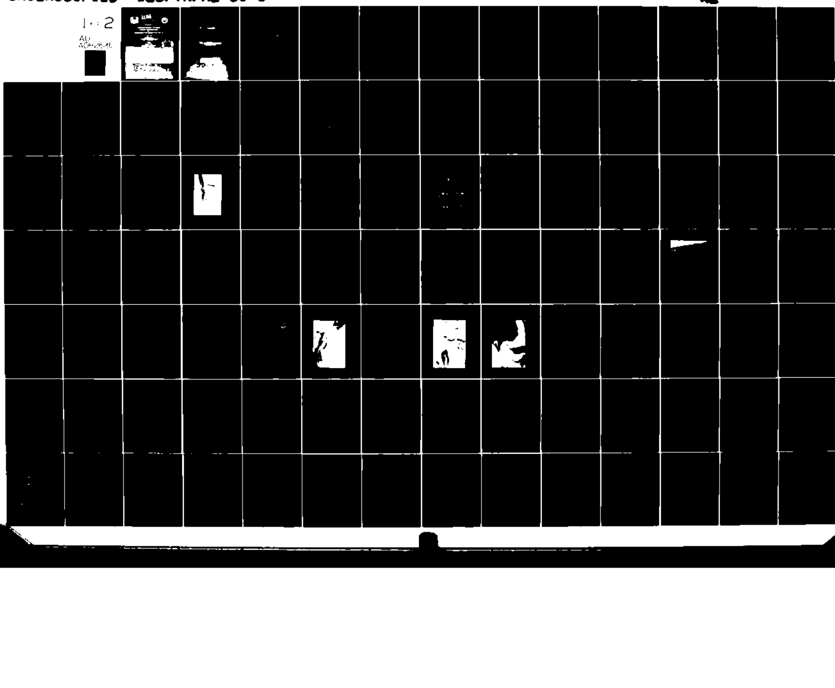

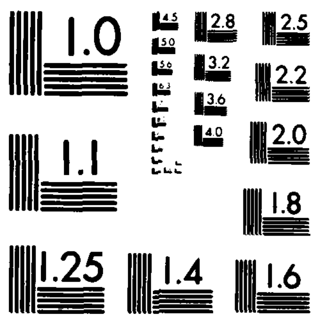

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→ It is frequently necessary to construct large engineering works of improvement in the surf and nearshore zone to protect harbor entrances, recreational beaches, and navigation channels. Shallow-water surface-gravity waves breaking on the structure during construction will cause bottom material to be suspended and transported from the region by longshore or other currents that may exist. This removal of material is often not compensated by an influx of

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20. ABSTRACT (Continued).

additional material, and the result is a scour hole, or erosion, which usually develops along the toe of the structure. In order to ensure structural stability and functional adequacy of the works of improvement, any scour area must be filled with nonerosive material (sufficiently stable to withstand the environmental forces to which it will be subjected). This may result in additional quantities of material being required during construction which can potentially lead to substantial cost overruns. The objective of this study is to develop techniques to minimize and control scour during nearshore construction, and to predict the probable magnitude of scour that may result as a function of the wave climate.

Most major stone structures require a foundation blanket for bearing surfaces to support the mass of the structure above, and to serve as scour protection during the actual construction. The thickness and design features of the blanket vary with location, but in general are on the order of 2 to 5 ft thick, extend beyond the toe of the structure from 5 to 25 ft, and are composed of quarry-run spoils. In recent years, a wide variety of plastic filter fabrics have been used on unconsolidated materials to prevent the settlement of heavier stone into the foundation. A layer of crushed stone or shell should be applied next in order to prevent puncture or tearing of the cloth by heavier stone. Foundation bedding materials should be placed ahead of the core construction at least 50 ft to prevent temporal scouring to undermine the working section. In those regions where, historically, vertical-walled structures have been built (sheet-steel pile, concrete pile, sheet-steel cellular units, bulkheads, etc.), these should be rehabilitated or rebuilt with sloped rubble-mound stone to dissipate wave energy along the structure.

"Accelerated core placement" has been utilized successfully in crossing scour holes susceptible to continuing scour. On major structures where the work will extend over more than a single construction season, no more stone should be placed than can be armored that season. Gabion units have been fabricated and placed in a continuous layer as foundation bedding material instead of a loose layer of crushed stone to ensure that the bedding material will be evenly distributed even after structure settlement. In emergency situations, scour has been minimized by filling ebb or flood channels with dredged material to allow construction operations to continue. Unique construction procedures have been applied in the State of Alaska which include working when soils are frozen or using ice block covered with soil to temporarily prohibit currents from scouring the work area during construction.

The purpose of this report is to document the location and magnitude (regionally) of the problem of scour and erosion during construction of major structures in the coastal zone, and to review the present design and construction practices which are being applied in this regard.

PREFACE

The study reported herein was authorized as a part of the Civil Works Research and Development Program by the Office, Chief of Engineers (OCE). This particular work unit, Erosion Control of Scour During Construction, is part of the Improvement of Operations and Maintenance Techniques (IOMT) program. Mr. Milt Millard and Mr. William Godwin were the OCE Technical Monitors of the IOMT program during preparation and publication of this report.

This study was conducted during the period 1 October 1977 through 31 May 1979 by personnel of the Hydraulics Laboratory of the U. S. Army Engineer Waterways Experiment Station (WES) under the general supervision of Messrs. H. B. Simmons, Chief of the Hydraulics Laboratory; F. A. Herrmann, Jr., Assistant Chief of the Hydraulics Laboratory; R. A. Sager, Chief of the Estuaries Division; Dr. R. W. Whalin, Chief of the Wave Dynamics Division; Mr. D. D. Davidson, Chief of the Wave Research Branch; and Dr. J. R. Houston, Research Engineer and Principal Investigator for the Erosion Control of Scour During Construction work unit. Dr. L. Z. Hales, Research Hydraulic Engineer, performed the investigations described herein and prepared this report.

During the conduct of this study, numerous conferences were held with personnel of the various U. S. Army Corps of Engineers coastal and Great Lakes District and Division offices. Special acknowledgment is made to the following individuals without whose assistance the project objectives could not have been successfully met:

North Central Division: Mr. C. Johnson
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Pacific Ocean Division: Messrs. E. Nagasawa, J. Hatashima, and S. Sullivan.

The conclusions reached in this report were based in part on conference discussions and should not necessarily be regarded as Corps policy.

Directors of WES during the conduct of the investigation and the preparation and publication of this report were COL John L. Cannon, CE, and COL Nelson P. Conover, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
acres	0.4047	hectares
cubic feet	0.02831685	cubic metres
cubic yards	0.7645549	cubic metres
feet	0.3048	metres
feet per second	0.3048	metres per second
inches	25.4	millimetres
knots (international)	0.5144444	metres per second
miles (U. S. statute)	1.609344	kilometres
mils	0.0254	millimetres
pounds (force) per square inch	6894.757	pascals
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
squares	9.290304	square metres
square miles (U. S. statute)	2.589988	square kilometres
square yards	0.8361274	square metres
tons (2000 lb, mass)	907.1847	kilograms
yards	0.9144	metres

EROSION CONTROL OF SCOUR DURING CONSTRUCTION

PRESENT DESIGN AND CONSTRUCTION PRACTICE

PART I: INTRODUCTION

Statement of the Problem

1. It is frequently necessary to construct large engineering works of improvement in the surf and nearshore zone to protect harbor entrances, recreational beaches, and navigation channels. These structures, usually built from quarried rock or precast concrete, are placed in position by crane or barge. When these major structures are erected in the coastal zone they alter the existing currents that normally occur at a particular location due to the unique topography and the wave climate peculiar to that specific region. Shallow-water surface gravity waves breaking on the new structure will cause bottom material to be suspended and transported from the region by longshore or other currents that may exist. This removal of material is often not compensated by an influx of additional material; and the result is a scour hole, or erosion, that usually develops along the toe of the structure.

2. In order to ensure structural stability and functional adequacy of the works of improvement, any scour area must be filled with nonerodible material (sufficiently stable to withstand the environmental forces to which it will be subjected). This may result in additional quantities of material being required during construction that can potentially lead to substantial cost overruns. To minimize potential cost increases due to scour during construction, it is necessary to quantify the probability and ultimate extent of potential scour during the scheduled construction period. This is an extremely complex problem and quantification of the probability of potential scour will always be site-specific.

3. Some problems associated with scour and erosion around major structures may develop and evolve over a long period of time. That is

to say, erosion which is of an extent sufficient to endanger the structural stability of the engineering work may not occur during the initial construction process, but may dramatically occur at a later date when a unique combination of wave characteristics and approach directions exists to produce scour and erosion of sufficient magnitude to cause damage. These conditions will be referred to as "secondary construction effects" as contrasted to the scour and erosion that takes place during the actual construction process which will be referred to as "primary construction effects."

Objectives of the Study

4. Effective and comprehensive procedures do not exist for eliminating scour during construction in the nearshore environment. In cases where low currents may be expected, scour can be minimized by laying a base layer, or foundation blanket, some considerable distance ahead of the construction of the upper portions of the structure. In the expected presence of higher currents, placement of larger stone may be required. Quantification of the most appropriate procedure is seriously hampered by the inability to predict the extent of potential scour.

5. The objectives of this study are to develop techniques to minimize and control scour during nearshore construction and to predict the probable magnitude of scour that may result as a function of the wave climate. Research planned is directed at two major efforts. One phase of the study will be the development of numerical techniques (incorporating both refraction and diffraction effects near the structure) for computing wave-induced velocities and tidal currents in the vicinity of structures and applying these results to the sediment transport characteristics of the bottom material at the particular site. This should result in an accounting of the flux of material around the structure, and thus knowledge of the extent of erosion or accretion to be expected as a function of wave climate, local topography, and structure design. Another phase of the overall study was an investigation of techniques currently being used by the Corps of Engineers (CE) to combat the scour problem.

6. This report is directed toward an understanding of present techniques used by the CE to combat scour during construction. This information was obtained from conferences and personal interviews with those responsible individuals in all CE coastal and Great Lakes districts. A draft of this report was transmitted to these districts, and the review comments and suggestions from each have been incorporated into this final report. Report 2 will be an in-depth literature review of experimental and theoretical works which may be adaptable to coastal zone erosion studies.

Synopsis of Findings

7. It was determined that just as there are a wide variety of climatic regions around the nation, also there is an assortment of design and construction techniques presently being used to overcome the problem of scour and erosion during nearshore construction (both new construction and maintenance construction). These differing techniques have evolved through years of experience in working under varying wave and soil conditions and reflect the accumulated knowledge of many individuals gained over many construction seasons. The extremes in wave climate are represented by the high energy environment of the north Pacific coastal region (particularly northern California, Oregon, and Washington, excluding Puget Sound) and the relatively low energy levels of the Great Lakes because of the limited fetches. The relatively mild wave climate of the Gulf of Mexico is partially offset by the adverse foundation characteristics of the Mississippi River deltaic formations on the Louisiana coast. One area of the south Atlantic has historically been required to construct only a minimal number of nearshore structures, and such foreseeable work also appears to be limited; hence, except from an academic standpoint, this problem is considered only peripherally by those working in this region of the nation.

8. It is concluded that four fundamentally different materials are presently being used to combat scour from wave-induced erosion around major stone structures. These are:

a. A layer of crushed or quarry-run stone (graded or ungraded) placed as a foundation blanket on sandy or otherwise semi-consolidated foundations to prevent upward migration of loose materials and settlement of larger stone sizes.

b. Fabricated gabion units placed underneath stone structures to form a continuous layer in lieu of a crushed stone foundation blanket.

c. A wide assortment of synthetic filter fabrics that are placed underneath rock structures to prevent settlement into otherwise unconsolidated foundations.

d. To a lesser extent, the use of Gobimats, particularly for toe protection of shore-connected structures such as sea-walls or slope revetments.

For those situations where the extension of an existing structure is required across a large scour hole, the technique of "accelerated core placement" is applied. If the hole can effectively be traversed, normal construction procedures are resumed. Nonstructural techniques for estimating additional material to be required as a result of scour may be lumped all-inclusively into "experience factors," such as the estimated equivalent uniform scour computations of the U. S. Army Engineer District, Galveston, or the void ratio adjustment determination of the U. S. Army Engineer District, Los Angeles.

Geographic Locations

9. Due to the large number and wide variety of nearshore construction projects, both completed and planned, it is convenient to consider a regional breakdown or geographic analysis of the various structures and construction techniques. The following seven geomorphic or physiographic regions have been selected:

a. The Great Lakes Region

b. North Atlantic Region, extending to approximately the Virginia-North Carolina border

c. South Atlantic Region, extending to Key West, Florida

d. Gulf of Mexico

e. South Pacific Region, extending to approximately the California-Oregon border *f*. North Pacific Region

g. Alaska

Additional subregional considerations will be used where appropriate and convenient, such as the Great Lakes and the Gulf of Mexico.

PART II: REGIONAL ANALYSIS OF SCOUR PROBLEMS

The Great Lakes Region

10. The Great Lakes region of the United States and Canada (Figure 1) comprises 299,000 square miles,* covering northeastern Minnesota, essentially all of Michigan, and parts of six other states, with 4,000 miles of mainland shores and 1,500 miles of island shores. The international boundary passes through these lakes and their connecting channels, with the exception of Lake Michigan which lies entirely within the United States.

11. The origin of the Great Lakes basins has been a subject of interest to geologists for more than a century. Shepard (1937) concluded that glacial erosion was the principal cause of their existence. The lakes themselves originated during the Pleistocene age. The continental ice cap developed to a thickness of several thousand feet over all of Canada and spread southward, completely covering what is now the Great Lakes region. The close relationship between topography of the lake basins and bedrock geology leaves little doubt that the Great Lakes were excavated by erosional processes and that they are not primarily the result of diastrophic action (the process of deformation that produces in the Earth's crust its continents and ocean basins), according to Hough (1958). The relative importance of preglacial stream erosion and of glacial scour cannot be evaluated precisely; however, the fact that upwarp of the land to the northeast has occurred does not disprove the probable occurrence of glacial scour in the development of the basins.

12. As the ice cover receded, the patterns and the levels of the lakes were repeatedly changed as new lower outlets were uncovered. This effect is illustrated by such features as the perched wave-cut cliffs of Mackinac Island, the lake-deposited clay flats of Chicago and

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 4.

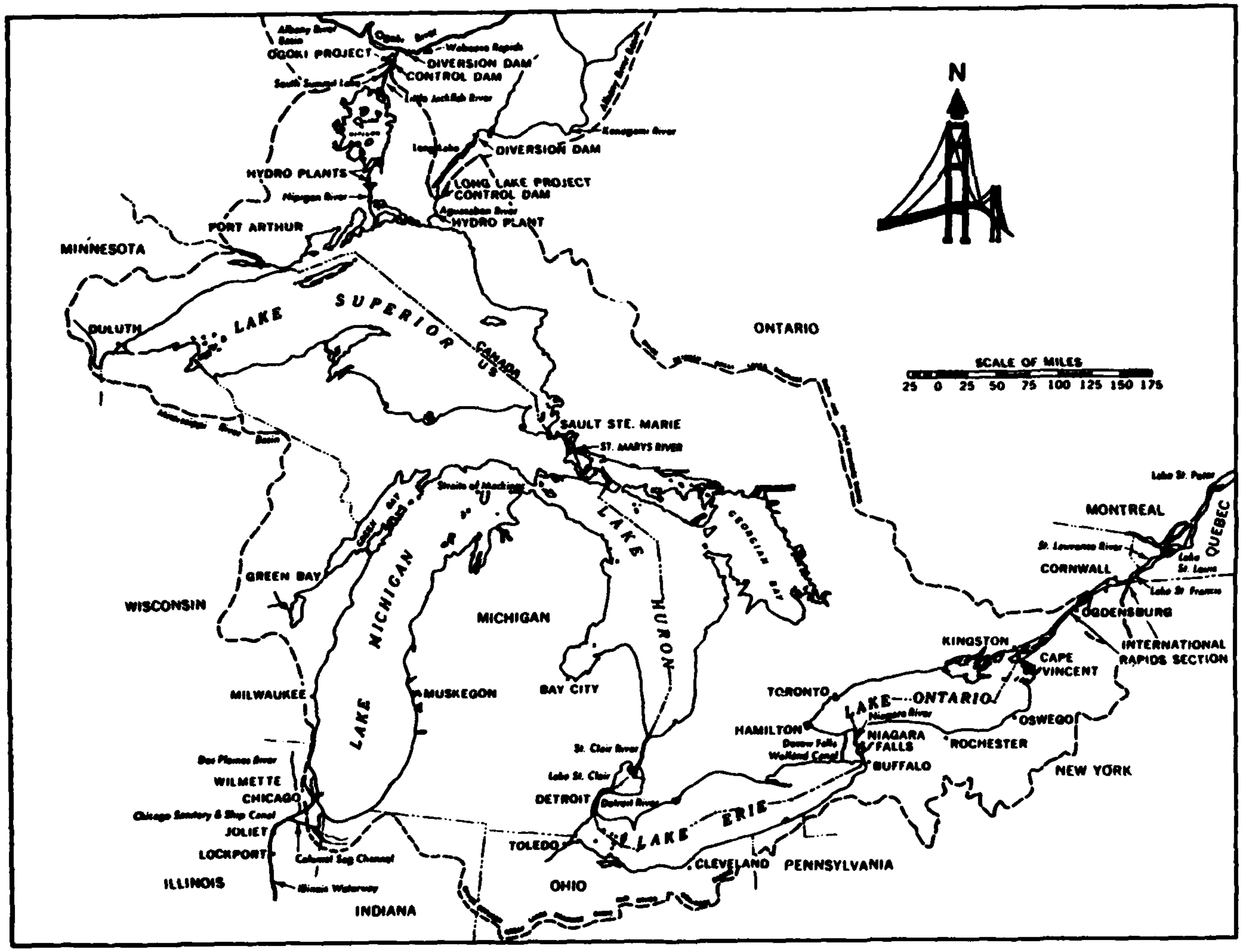


Figure 1. The Great Lakes region

Toledo, and the sand tracts of the dune areas. Stream and shoreline erosion processes have made only slight changes in the original topography of the lakes, although present shoreline erosion is of great economic significance.

13. Enormous quantities of water are required to effect even small changes in the lake levels. Therefore, comparatively large variations in supplies to the lakes have little immediate effect on lake levels, although variations up to 2 ft occur occasionally on Lake Erie during the rainy seasons. Flow rates in the outlet rivers are remarkably steady in comparison with the range of flows observed in other large rivers of the world. Where suitable head exists, these large steady flows make generation of electric power economically feasible.

14. The erosion potential of a surface gravity wave is a function of wave height and water depth. The greatest effect of the wave occurs in the surf zone from the inception of breaking to the limit of runup, or until longshore currents are deflected by a structural measure at which time scouring potential of the velocity field becomes important. Quantitative knowledge of maximum wave conditions in the Great Lakes is essential for the design of shore protection features. A systematic accumulation of wave statistics from measured data does not exist. Deep-water wave conditions are determined from synoptic meteorological data and transferred to a specific site by applying appropriate wave transformations. Resio and Vincent (1976-1978) have developed, through hindcasting, a storm climatology for the Great Lakes that is applicable for many planning, design, operation, and maintenance functions on the Great Lakes. Historical wind data from stations along the lakes served as input to the numerical hindcast model, and statistical estimates of the storm wave climatology were calculated for 5-, 10-, 20-, 50-, and 100-year return periods. The mean significant period for each of these heights is provided for four seasons of the year and is separated into three approach directions relative to shore. From a general standpoint, the probable once-a-year significant wave heights for selected locations in each of the lakes is given in Table 1 by the Great Lakes Basin Commission (1975).

15. Wave-generated currents, which transport beach material in the surf zone, are important factors in beach stability. The direction of littoral drift is determined by the angle of approach of the wave which ultimately determines the direction of the longshore current. Wave-generated currents carry some particles along the bottom as bed load while other particles are carried some height above the bottom as suspended load. Finer materials such as silt and clay need little energy to keep them in suspension, and they can escape from the local area by relatively low-energy lake currents. When the natural dynamic balance of forces moving material along the shore is altered, the various forces will tend to reestablish a new dynamic equilibrium condition. For example, a groin in its early stages interrupts longshore drift by causing particles to be deposited on the updrift side and by contributing to erosion on the downdrift side. Increased wave action on the beach increases the rate of littoral transport, and rising lake levels cause bluffs and beaches to erode at a more rapid rate. The estimated direction of net littoral drift along the shores of the Great Lakes is illustrated in Figure 2.

16. Long-term lake level fluctuations and seasonal variations have been documented by the U. S. Department of Commerce, National Ocean Survey, at stations throughout the Great Lakes. These data, consisting of monthly mean, hourly, and instantaneous water levels, have been utilized by the Detroit District (1979) to develop standardized frequency curves for design water-level determination on the Great Lakes. These design water levels are intended to be used for the design of piers, shore protection measures, or other coastal structures where a water-level requirement is dictated by project constraints. These lake level exceedance frequency curves for Lakes Superior, Michigan-Huron, Erie, and Ontario are shown in Figures 3, 4, 5, and 6, respectively.

17. In addition to long-term lake level fluctuations and seasonal variations, the Great Lakes are subject to irregular oscillations caused by changes in barometric pressure ranging from a few inches to several feet. These temporary fluctuations may extend over a few minutes to

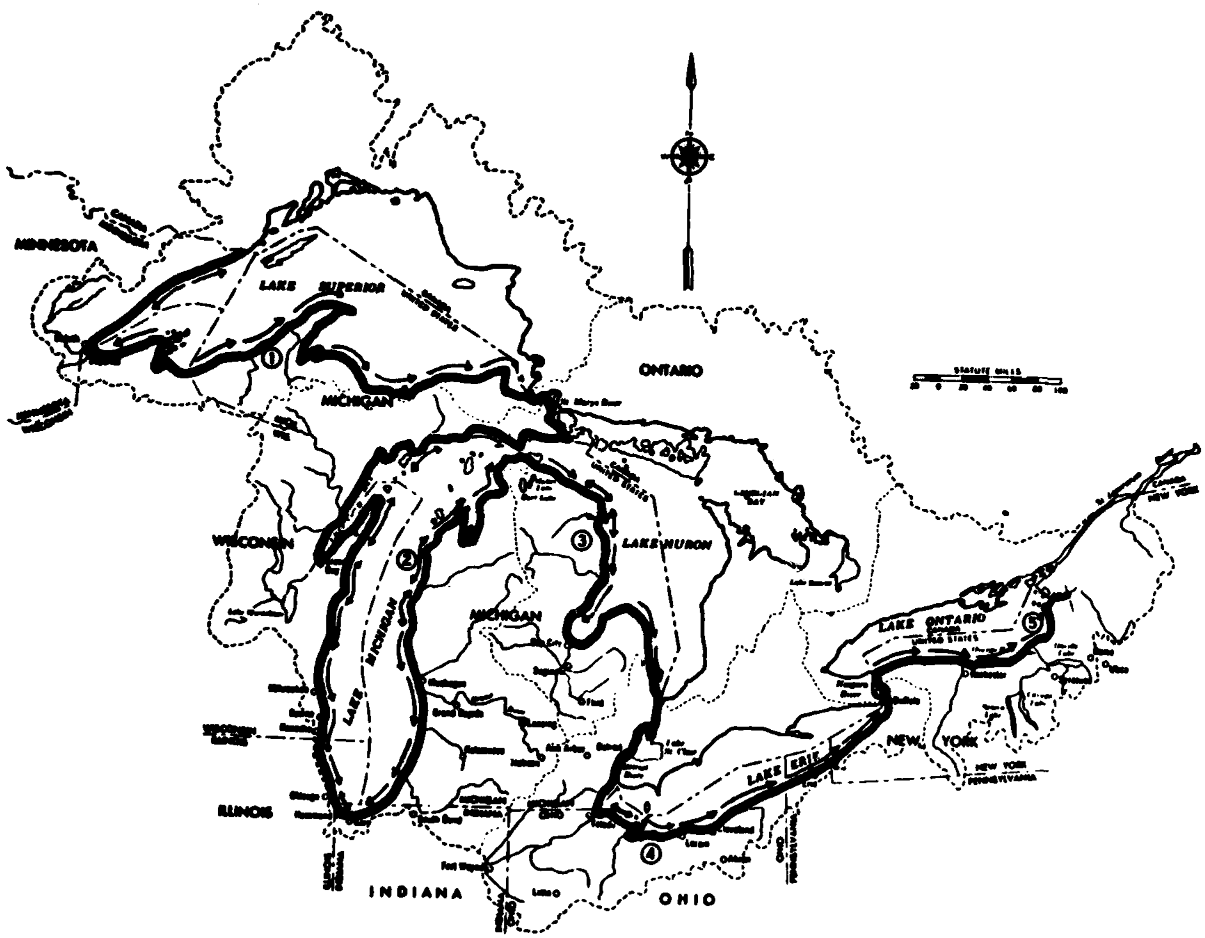


Figure 2. Estimated direction of net littoral drift along the Great Lakes (after Great Lakes Basin Commission 1975)

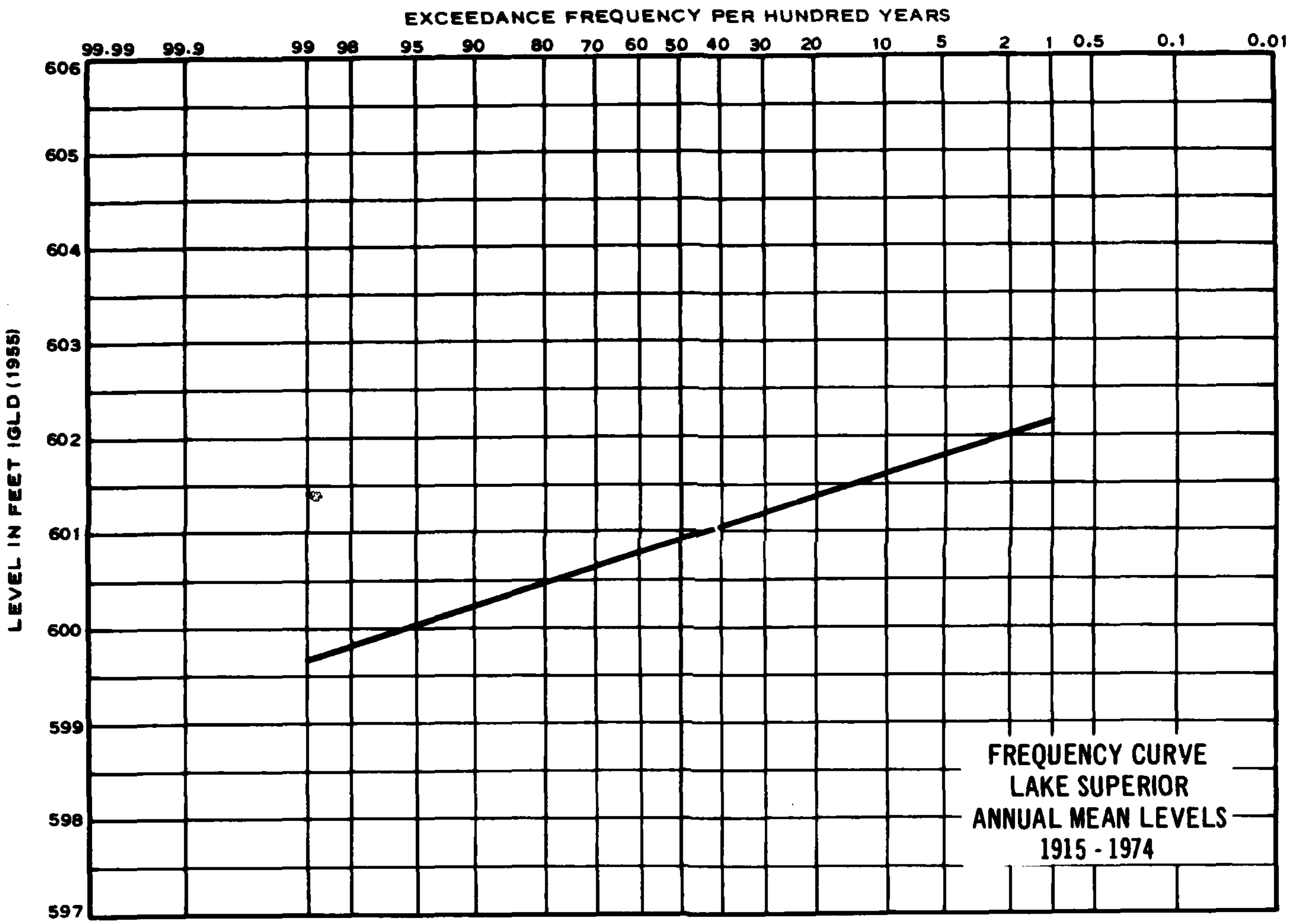


Figure 3. Lake Superior long-term water-level exceedance frequency (after U. S. Army Engineer District, Detroit, 1979)

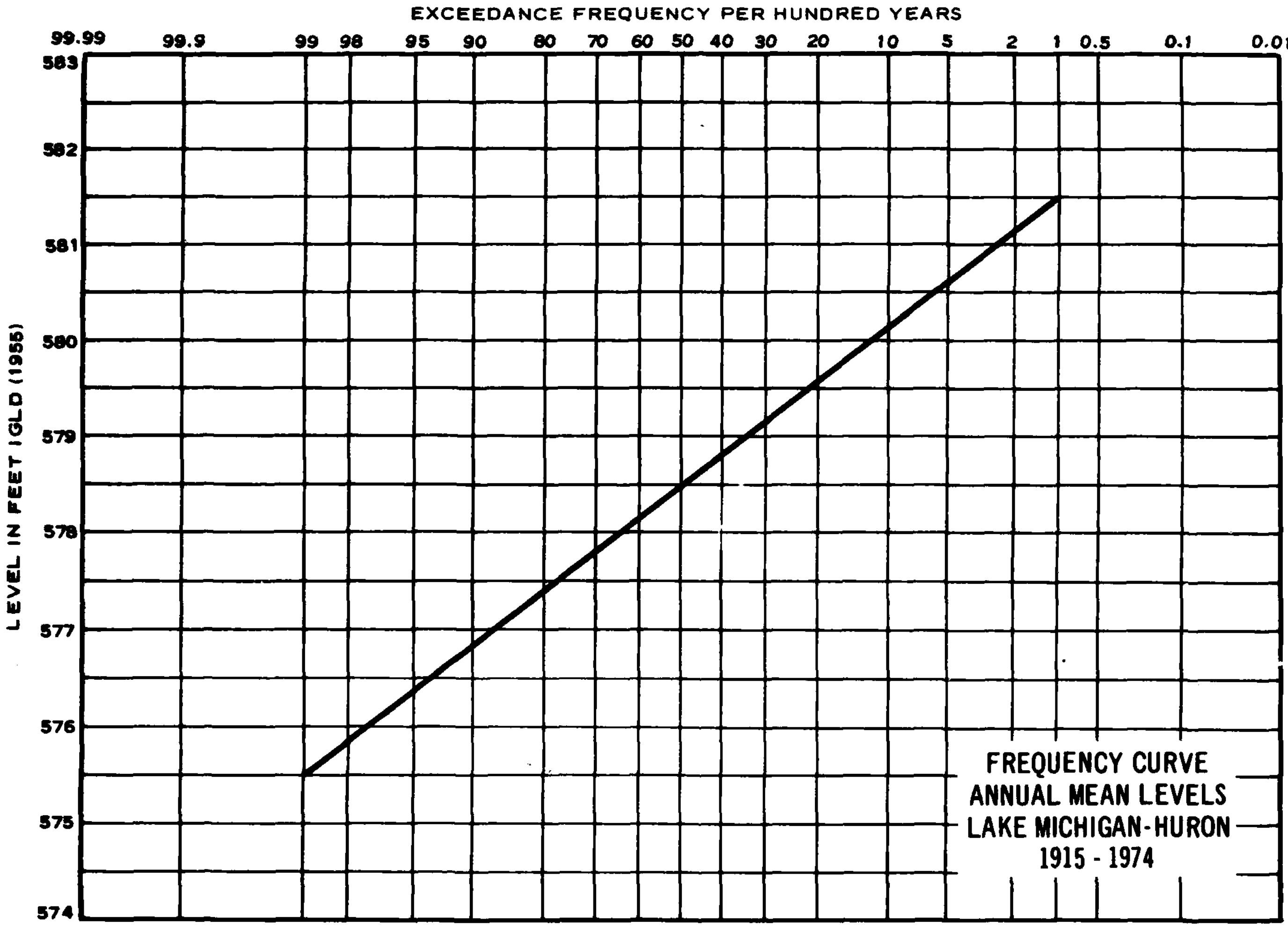


Figure 4. Lake Michigan-Huron long-term water-level exceedance frequency (after U. S. Army Engineer District, Detroit, 1979)

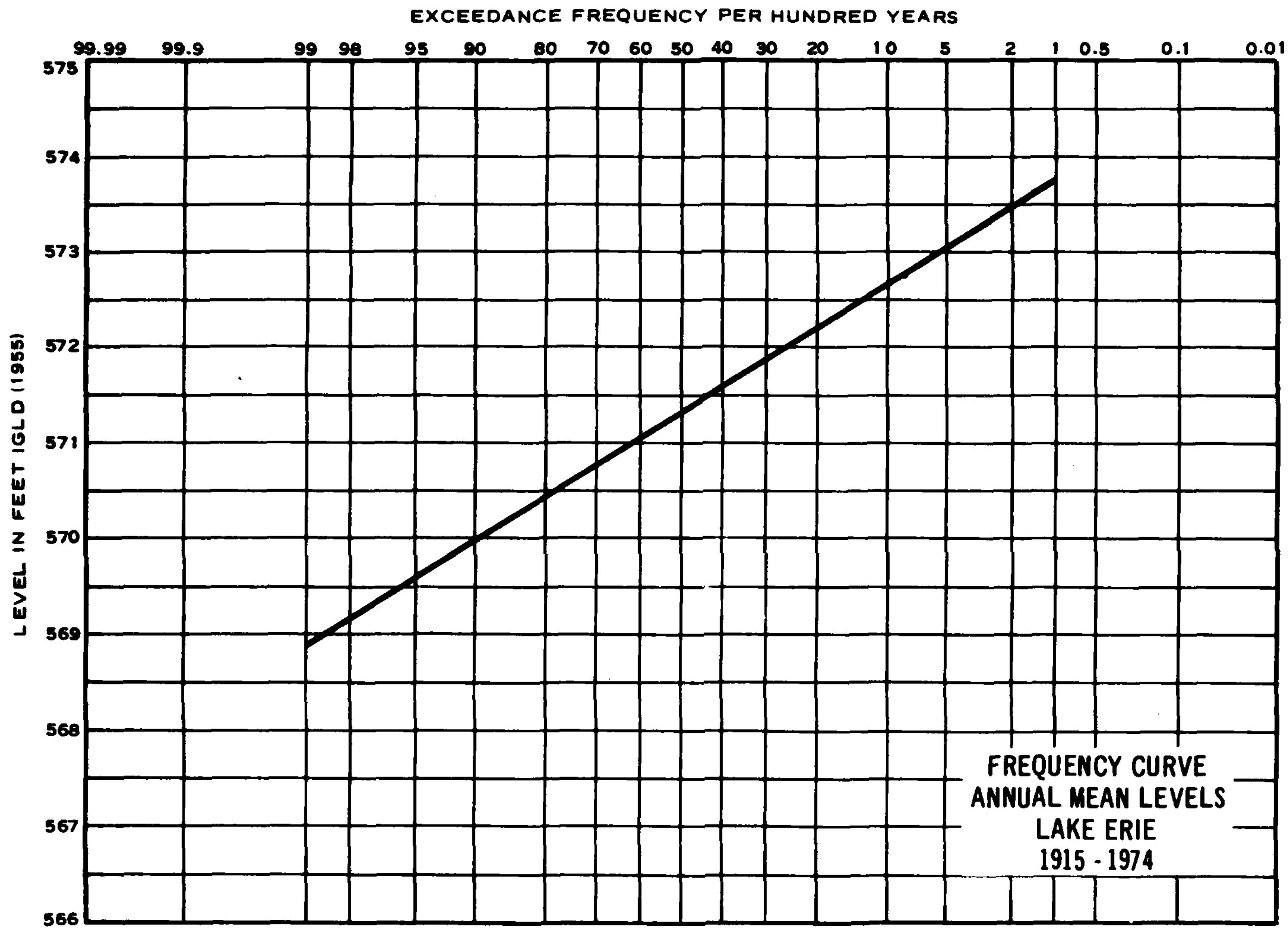


Figure 5. Lake Erie long-term water-level exceedance frequency (after U. S. Army Engineer District, Detroit, 1979)

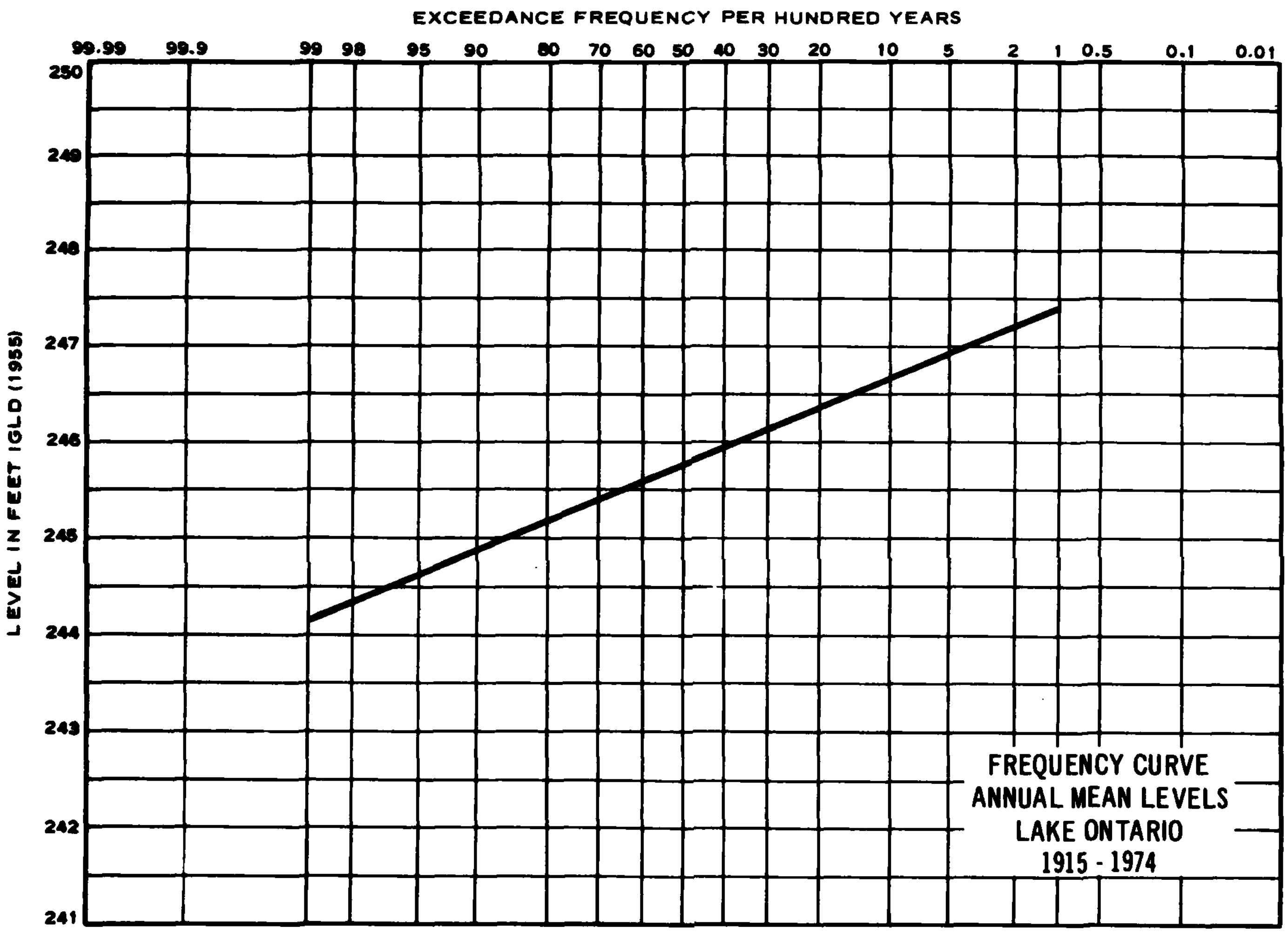


Figure 6. Lake Ontario long-term water level exceedance frequency (after U. S. Army Engineer District, Detroit, 1979)

several days. At times the lake levels are also affected by stresses of wind blowing over the water. Sufficient velocity drives the water forward in greater volume than it can be carried by return currents, raising the lake level on the downwind side. When a moving low-pressure weather system moves along the major axis or across one of the lakes at a speed conducive to excitation of a mode of oscillation of the lake, significant seiche action can result. The magnitude of these short-period fluctuations depends on local conditions. The maximum rises recorded on the lakes, and their frequency of occurrence, are given in Table 2.

18. The design of breakwaters or jetties on the Great Lakes requires provisions for stability against wave forces comparable to the maximum probable pressure that might be developed by an ice sheet. Since the maximum wave forces and ice thrust cannot occur at the same time, no special allowance for overturning stability to resist ice thrust need be made. However, adequate precautions must be observed to ensure that the structure is secure against sliding on its base. This is usually accomplished by heavy stone placement if the structure is of the cellular sheet-steel pile variety. Interior harbor facilities such as piers must be designed to resist considerable ice thrust.

Lake Superior

19. Background. The Lake Superior basin is a long narrow watershed extending 350 miles from east to west and 150 miles from its northernmost reach to its southernmost boundary. Lake Superior is the largest freshwater lake in the world with a surface area of 31,820 square miles. The basin experiences a typical midcontinental climate that is modified considerably by the waters of Lake Superior. Because of its effective heat storage capacity, the lake may warm winter air masses moving over the region by as much as 15° to 20°F.

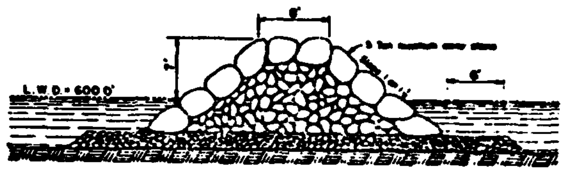
20. Prevailing winds and storms are from the west and southwest, which may cause great extremes of weather conditions and temperatures over the basin. The Keweenaw Peninsula serves to deflect storms crossing the region from the west. The lake is so large that there are appreciable differences in climate between the north and the south

shores, and also between the western and eastern ends of the basin.

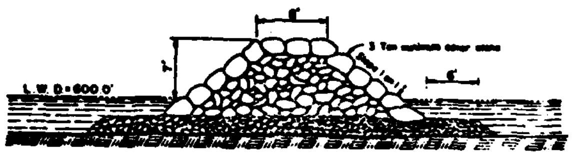
21. Excellent fishing and boating exist on Lake Superior as the water quality is exceptionally good. Due to the great depth and size of the lake and the normal low temperature of the water in the lake, as well as the prevalent long winter season in the locality, ice forms early and remains late in the spring. Ice becomes very thick in sheltered areas and piles up in great windrows on the more exposed portions of the lake, particularly near the west and east ends. Ice thicknesses of 30 to 36 in. are not uncommon in harbor slips and 20 to 24 in. of ice frequently form in the lake, with the ice sheet often extending beyond view from shore. Airline pilots have reported solid ice all the way to Canada; however, in most winters there is usually some open water in eastern Lake Superior.

22. In the late 1800's, harbors for the protection of fishing fleets were constructed at many locations along the coast of Lake Superior. At that time timber was plentiful and relatively inexpensive, and it was not uncommon to find this and other readily available material being used in the fabrication of wave protection measures. Timber cribs were often formed and filled with whatever stone sizes could be obtained locally. Over the years, these structures gradually deteriorated, and maintenance or replacement procedures had to be developed. With a dramatically rising cost of both labor and timber, it was no longer practical to use the earlier techniques. For those locations where the wave climate was sufficiently quiet to permit their installation, sheet-steel pile breakwaters became the accepted method of offering harbor protection. Representative examples of the various types of breakwater construction which have been used in Lake Superior are shown in Figures 7-10. The sheet-steel pile construction is usually built in conjunction with cellular modules that provide stability against sliding, as the sheet steel which forms the outer casing is driven several feet into the foundation and the entire unit is filled with sand and/or rock.

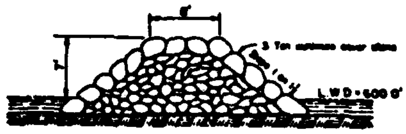
23. It became apparent in the 1940's that the spacing of the small-craft harbors for fishing or recreation was not entirely satisfactory in the event of the occurrence of sudden storms which can arise quite



Lakeward End to -5 Foot Contour



-5 Foot Contour to +2 Foot Contour



+2 Foot Contour to Landward End

RUBBLE MOUND - BUILT 1957

EAST AND WEST BREAKWATERS

Total Lengths { East Breakwater - 825 Feet ±
West Breakwater - 555 Feet ±

Figure 7. Typical breakwater construction, Black River Harbor, Michigan, Lake Superior

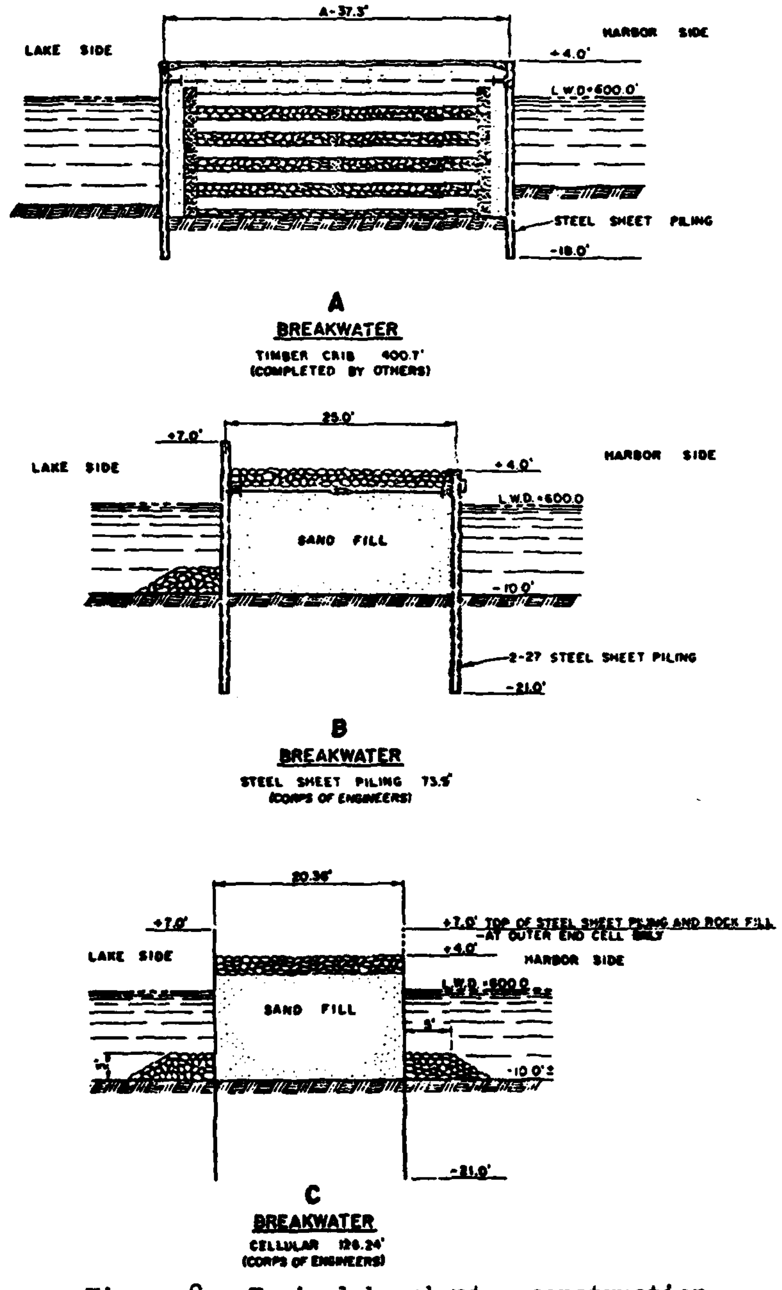


Figure 8. Typical breakwater construction, La Pointe Harbor, Wisconsin, Lake Superior

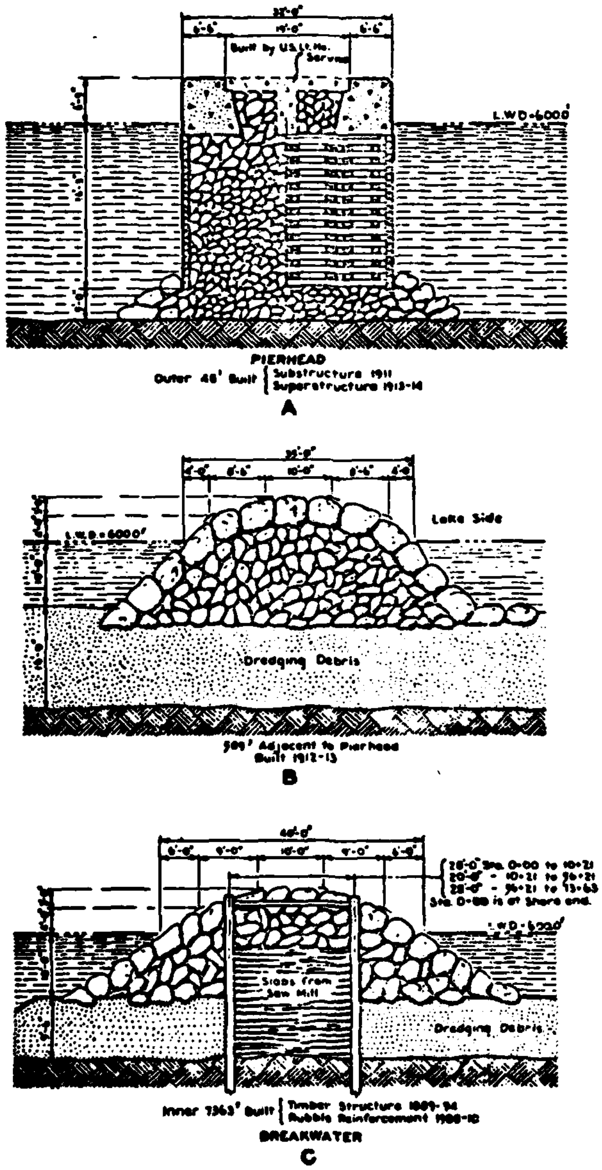


Figure 9. Typical breakwater construction, Ashland Harbor, Wisconsin, Lake Superior

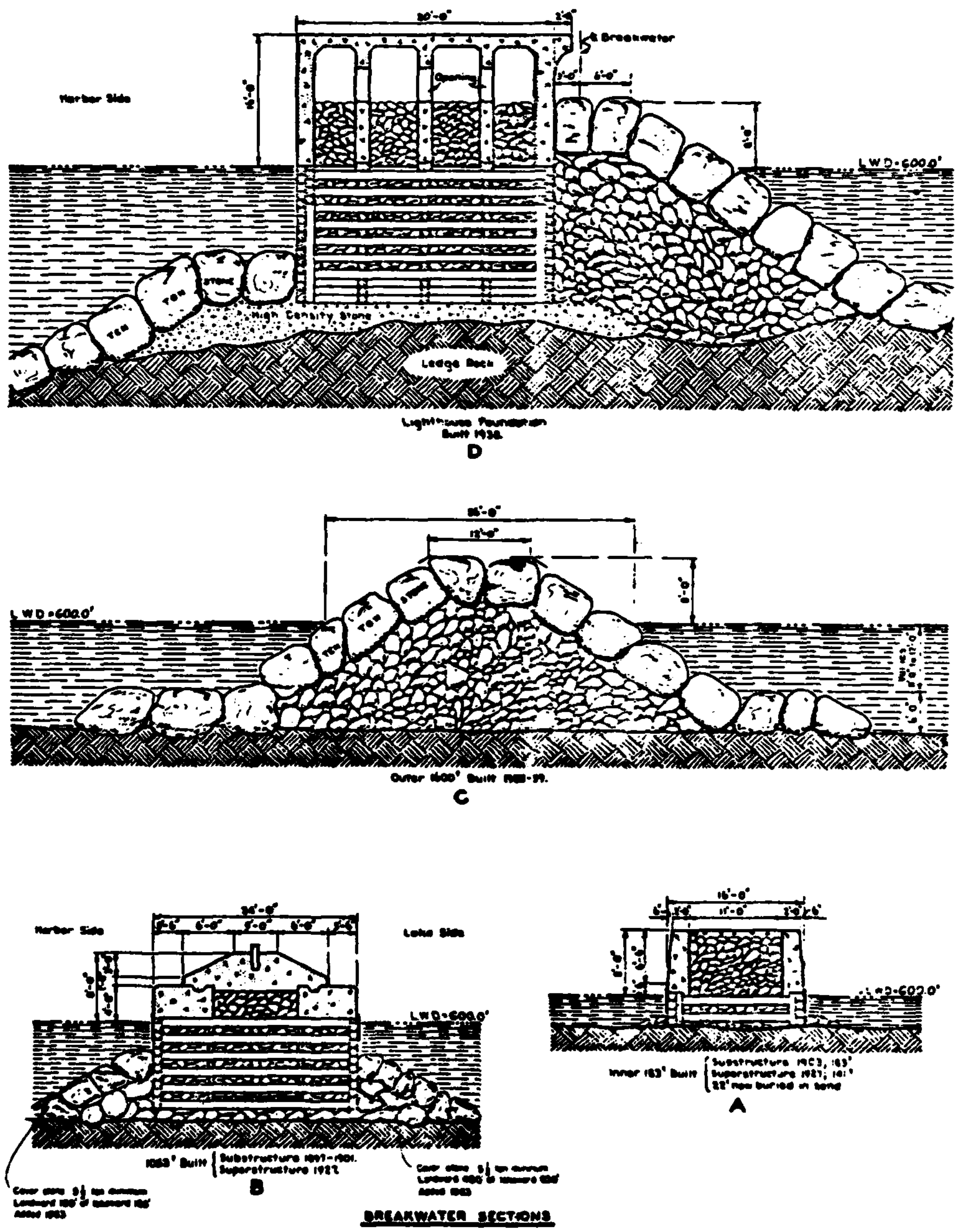


Figure 10. Typical breakwater construction, Presque Isle Harbor, Michigan, Lake Superior

rapidly on the lakes. The concept of the harbor of refuge was developed with the idea of offering a place of shelter where boats of any size navigating the lakes may seek protection from storms. The practice of using vertical sheet-steel pile for the construction of all or part of the features of these harbors of refuge was widely accepted. A typical example of such construction is Whitefish Point Harbor, Michigan (Figure 11), which is the only harbor of refuge on Whitefish Bay. It was envisioned that the harbors of refuge would be spaced on the order of 50 miles apart, although this has not been accomplished in its entirety.

24. Scour problems. Big Bay Harbor, Michigan, constitutes a vivid example of the "secondary construction effects" mentioned previously, although there is evidence of the same phenomena occurring at several other locations on Lake Superior (i.e., Grand Traverse Bay Harbor, the upper entrance to the Keeweenaw Waterway, Black River Harbor, etc.). The problem arises when the wave approach is such that the wave crest propagates along the section of vertical sheet-steel pile on the upcoast side of the harbor, increases in amplitude because of refraction effects and the Mach-stem phenomenon, and terminates as a geyser of water plunging over the crest of the breakwater at the shore end. Depending on the intensity of the storm, the water plume may reach 10 to 12 ft in height, and the resulting dynamic forces are sufficient to severely damage or destroy the structure. It has become necessary to rehabilitate and maintain these types of structures with huge amounts of stone riprap.

25. The procedures for combating the problem of scour and damage to the structures on Lake Superior include repairing the vertical sheet-steel pile with sloping rubble-mound structures which absorb or dissipate a large part of the wave energy as the crest travels along the breakwater. This also assists in the problem of ice damage, as the ice breaks up and overlaps, pushing against the vertical walls. If breakwaters were constructed of rubble-mound stone instead of vertical sheet-steel piles, the initial construction cost would probably increase; however, the maintenance cost of repairing rubble-mound structures should be significantly less if adequate size stone were used in the initial construction.

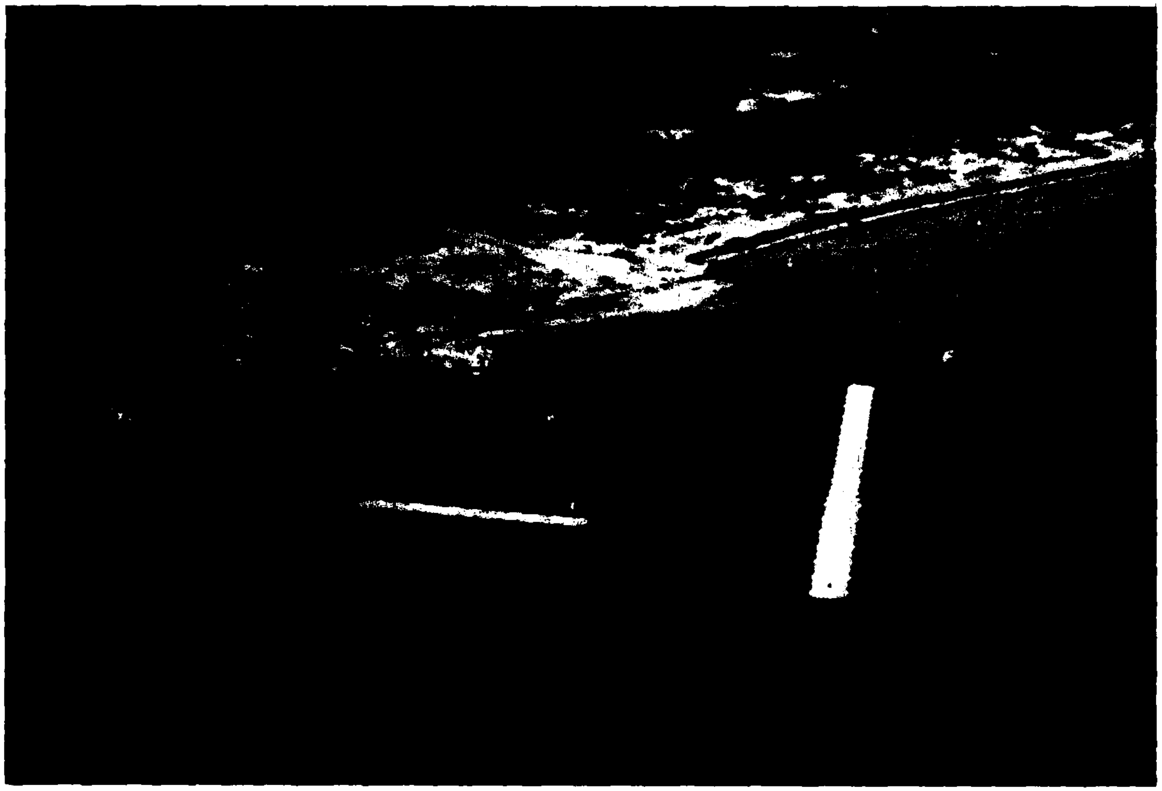


Figure 11. Combination sheet-steel pile and cellular breakwater construction, Whitefish Point Harbor of Refuge, Michigan, Lake Superior. (Additional construction has occurred in harbor since photograph was made.)

Lake Michigan

26. Background. Lake Michigan lies in the west central portion of the Great Lakes region, south of Lake Superior. It has a maximum depth of 923 ft, has a surface area of 22,300 square miles, and is connected to Lake Huron by the Straits of Mackinac. The directions of currents in the Straits alternate from east to west depending upon barometric pressure and wind conditions; the net flow, however, is eastward into Lake Huron.

27. The bed of Lake Michigan is divided into four regions; a smooth basin to the south, a divide, a northern basin, and a submarine ridge and valley province to the northeast. Bottom materials consist of sand along the shore, gravel between 50- and 100-ft contours, and mud at depths in excess of 100 ft. These sediments fill depressions and smooth the lake bottom. It is believed that the direction of net littoral drift is southward for both the east and west sides of the lake, at least for the lake's south half. Enormous quantities of beach sand and sand dunes have accumulated along the shore at the south end of the lake, and the littoral transport southward may be a contributing factor.

28. Lake Michigan is the only one of the Great Lakes situated entirely within the United States. Its total shoreline length is 1,362 miles, parts of which are located in Wisconsin, Illinois, Indiana, and Michigan. It is distinctive from the other Great Lakes in that it is the only lake which extends from north to south. This makes it the most significant transportation barrier in the midwest. Lake Michigan contains the largest embayments of any of the Great Lakes and has the least number of islands, all of which are located in the northern third of the lake.

29. One of the most impressive natural shore types of the Great Lakes is the large expanse of sand dunes along Lake Michigan's shore. These dunes extend almost continuously from the Indiana Dunes National Lakeshore northward to the tip of the Leelanau Peninsula in Michigan. They result from the prevailing westerly winds that cause an almost continuous washing and separation of shore soil materials by wave action.

Often associated with the dune areas, especially during years of low water levels on the Great Lakes, are wide sandy beaches that are heavily used for recreation. Particularly significant from the erosion standpoint are the vulnerable erodible bluff areas found along many shoreland reaches. Often used as building sites because of their scenic views, the erodible bluffs of Michigan and Wisconsin are being continuously threatened and damaged by erosion. Erosion control plans are continually being formulated and updated.

30. Scour problems. Problems previously mentioned regarding Lake Superior are found to exist in Lake Michigan also. Original timber crib breakwaters which were built when lumber was relatively plentiful and inexpensive, have been rehabilitated over the years with sheet-steel piles. The results have been, for example, that at Manistee Harbor or Leland Harbor waves propagate along the breakwater, increase in amplitude, and terminate by plunging and surging over the breakwater at the beach. At Grand Haven Harbor, waves propagate through the entrance channel and overtop the parallel jetties that stabilize the location of the inlet. The jetties are approximately 300 ft apart; hence, wave energy of a magnitude sufficient to scour the shore of the inner harbor is able to penetrate the basin for certain directions of approach of storm waves.

31. At Leland Harbor, Michigan "primary" scour problems arose during construction of the rubble-mound breakwater. Since Lake Michigan completely freezes over in the winter, it is necessary to cease construction operations until spring on those projects that cannot be completed before ice begins to form on the lake. The existing project provides for a harbor of refuge consisting of a breakwater about 1,200 ft long running parallel to the shore for some distance and then angling toward the shore at its northern end. During the fall construction season, half of the breakwater was completed. During the next construction season, the second half of the breakwater was built, starting at the opposite end of the structure and joining the two halves in the middle. As work progressed toward the existing first half, it was discovered that a tremendous scour hole had developed off the end of

the first half of the jetty. Since tidal currents are of no significance in the Great Lakes, the scouring action must be attributable to wind- and wave-generated currents that were deflected by the massive rubble-mound structure. The only practical method for combating this situation was "rapid placement" of construction materials; i.e., fill the scour hole as rapidly as possible with whatever amount of material was required to bring the structure up to design grade.

32. An example of a slightly different structural design of breakwater configurations (Figure 12) is Port Washington Harbor, Wisconsin. This illustrates the placement of massive caisson sections on a foundation blanket of small stone in order to distribute the weight of the caisson so that settlement of the breakwater crest will not be as probable as it would otherwise.

33. As in Lake Superior, it is envisioned that the foreseeable work load in Lake Michigan will be rehabilitation and maintenance of existing facilities where breakwater and harbor facilities are involved. Public Law 91-611 (Rivers and Harbors Act of 1970) authorized the CE to construct, operate, and maintain structures to confine polluted dredged materials for a period of 10 years to protect and improve the quality of water in the Great Lakes. During this 10-year period, the states and cities are expected to cease discharging pollutants into the waterways. At present, 11 Wisconsin harbors on Lake Michigan have sediments classified as polluted by the U. S. Environmental Protection Agency (EPA). These sediments therefore require confinement. The confinement facilities are complete at Milwaukee, Manitowoc, and Kenosha. A facility was under construction in January 1979 to serve Green Bay, with sites selected and being designed to serve Sheboygan and Kewaunee. Sites are also being analyzed for Sturgeon Bay, Wisconsin, and Menominee-Marinette, Michigan and Wisconsin. There exists potential for a like number of sites to be developed on the east side of Lake Michigan; hence, about a dozen diked disposal sites will probably evolve for this lake. When sited in open water, these diked facilities should be of sound design and should be built with construction techniques that will minimize scour or erosion.

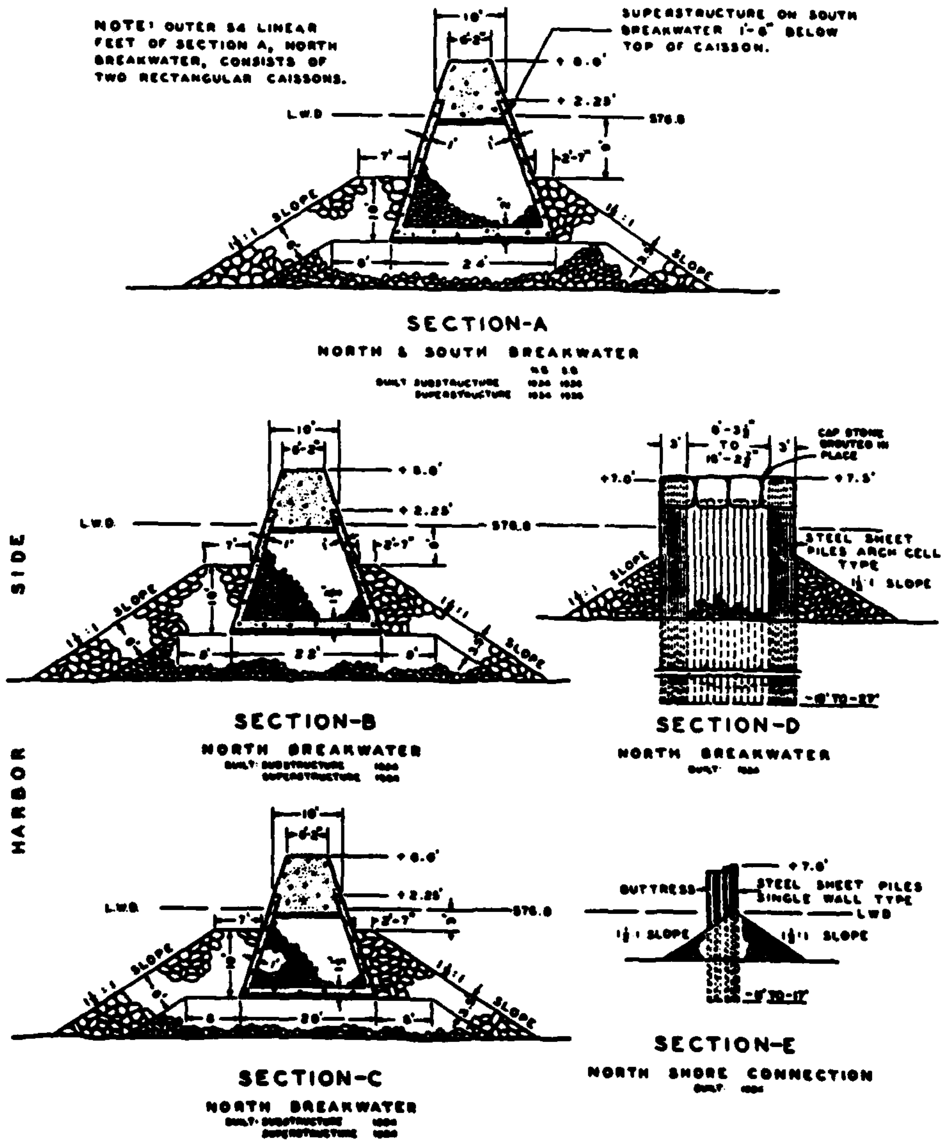


Figure 12. Weight distributing foundation sublayer, Port Washington Harbor, Wisconsin, Lake Michigan

Lake Huron

34. Background. The United States portion of the Lake Huron basin lies entirely within the State of Michigan and comprises approximately 14 percent of the Great Lakes drainage area. The basin is characterized by its hilly areas in the western and southern areas and flat glacial-like plains in the east. The climate of the Lake Huron basin is a humid continental type. Prevailing westerly winds passing over adjacent Lake Michigan have a moderating effect on summer and winter temperatures. In addition, the presence of Lake Huron moderates the lake shore to a significant extent.

35. Lake Huron receives inflow from Lake Superior through St. Marys River, and from Lake Michigan through the Straits of Mackinac. Its maximum depth is 752 ft. The outflow from Lake Huron passes through an outlet channel composed of the St. Clair and Detroit Rivers and Lake St. Clair. There are no navigation locks in the channel between Lakes Huron and Erie, and dredging operations in this waterway over the years have made a deeper channel. Because of this, a substantial lowering of the water levels of Lakes Michigan and Huron would occur except for compensating dikes which were constructed to maintain lake water levels at preproject elevations. The water quality of Lake Huron is good except for an isolated problem in Saginaw Bay.

36. Scour problems. Engineering works of improvement in Lake Huron are exposed to the same potentials for scour during construction and secondary construction effects as those structures in Lakes Michigan and Superior, which have already been discussed. Additionally, a scour-erosion problem developed in late 1978 at the open-water diked disposal site in Saginaw Bay that may possibly be attributed to a lack of historical wave data on which to base the structure design.

37. Because of the polluted sediments of the Saginaw River near Saginaw, Michigan, and since it is necessary to dredge a portion of this river and a navigation channel for some 3 miles into Saginaw Bay, it was necessary to construct a diked disposal site to confine these polluted dredgings. The facility was constructed without incident, with cover stone extending part way up the face of the structure to an elevation

determined to be the maximum extent of the runup for the design wave. The remainder of the structure was exposed, small crushed rock that constituted the major volume of the fill.

38. The diked disposal site is situated near the south end of Saginaw Bay; and in September 1978, a northeasterly storm developed over Lake Huron, generated waves propagating toward Saginaw Bay that penetrated the bar section at the mouth of the bay, re-formed waves that combined with those waves being generated in Saginaw Bay proper, and caused significant damage to occur to the diked structure. It is not known if the visible erosion at the top of the structure (as a result of wave uprush and unup) is matched by corresponding erosion at the toe of the structure as a result of downrush. No toe structural damage is apparent. In May 1979, a similar event occurred that caused even greater erosion and scour. As a remedial measure, the cover is being extended to a higher elevation. Efforts are under way to ascertain and quantify the physical processes responsible for damaging the retaining dike.

Lake Erie

39. Background. Being the second smallest of the Great Lakes, Lake Erie is only 58 miles wide at the widest point; and it has the shallowest maximum depth, 210 ft. It is the only lake in the system whose point of greatest depth is above sea level. The surface area of Lake Erie is 9,910 square miles, approximately one-thirteenth the area of Lake Superior. The 30-ft depth contour is approximately 1 mile offshore all around the shoreline, which contributes to the great fluctuations in water level, fluctuations that are greater than any of the other Great Lakes. Strong winds blowing along the axis of the lake can create seiches that have been known to lower the water level at one end of the lake by 8 ft or more, while the water depths of harbors at the other end of the lake can rise a comparable amount.

40. The Lake Erie basin is divided into three areas. The western basin is relatively shallow and covered with fine sediments. The Detroit River discharge produces a flow pattern that penetrates far south into Lake Erie's western basin, and is traceable eastward through the northern island area into the central basin. The central basin of

Lake Erie is the largest of the three basins; it has a smooth, flat bottom. The eastern basin is deeper than, and is separated from, the central basin by the Long Point Erie Moraine. Glacial erosion deposits cover the adjacent shorelands and deeper lake bottom areas. Lake Erie discharges primarily at its eastern end through the Niagara River into Lake Ontario.

41. Scour problems. The situations regarding secondary construction effects encountered in the other lakes are equally applicable in Lake Erie. Additionally, a primary construction scour problem arose during construction of the Buffalo Disposal Dike No. 4, when it was found that the fine underlayer material would not remain at the design slopes. Dike No. 4 was built in the east end of the lake and the bottom is composed of loose silts and soft silty clays. In order to keep the core stone from settling into the unconsolidated foundation, the soft sediments were intended to be bridged with a 6-ft-thick layer of sand composed of minus 3/8 in. to No. 200 sieve size material. When storm conditions arose, large waves would cause mass transport of water to pile up against adjacent existing breakwaters, and from continuity, the return flow to the main lake would create currents sufficient to scour this sand underlayer from the designed 1V-on-5H slope. Bottom lake return currents had not been accounted for in the original design, and the solution to this problem was the placement of additional volumes of sand until the stable slope was formed between 10:1 and 20:1 horizontal to vertical dimension. This was indeed a boundary condition problem caused by setup of waves that developed return flow currents resulting in scour of the foundation. Approximately 20 percent additional material was necessary to stabilize the foundation layer.

42. When the Cleveland Disposal Dike No. 14 was being constructed, a slightly different problem arose that may more properly be attributed to construction techniques rather than to scour and erosion as normally considered. The self-unloading container ships that were transporting the core material to the work site would place large sections of the dike core above the waterline for reshaping later. Material less than

about 100 lb simply could not be retained in the wave climate occurring at the time without additional protection by cover stone.

43. The winter storm wave conditions begin rather abruptly in early October and continue through most of December, at which time the lake freezes over. Storms also occur frequently in early spring. The ideal situation would be to award construction contracts in January so the work force would have a full construction season in which to operate. However, these diked disposal sites are very large jobs, taking approximately 3 years to complete at a cost of up to 30 million dollars; hence, the fact that contracts are awarded when construction funds become available probably does not alter the completion of the final product.

44. Early pilot projects of construction in Lake Erie in the early 1960's used core stone as a foundation layer, but it was found that settlement of the structures occurred as the material penetrated into the lake bottom. The material that is ultimately designed as the bedding layer depends on both the consolidation of the bottom material and the currents that are expected to exist in the local region.

45. Essentially three placement methods are used for positioning material at the structure site in Lake Erie. They are:

- a. Self-unloaders, which are modified ore-carrying ships whose compartments are filled at the quarry with minus 8-in. material to be placed at the site. Conveyor belts carry the material from inside the ship and dump over the side of the vessel.
- b. Bottom-dump scows or large barges fabricated in such a manner that the bottom sections can be opened to allow the transported material to free-fall to the underwater parts of the structure.
- c. Crane and bucket operations for placing large cover stone and forming final grade lines.

The methods of transporting the material to the site really depend largely on the type of material being moved and the location of the quarry. Bedding and core material usually comes by boat or barge. The larger cover stone may be transported by barge or may be moved near the site by rail and trucked the remaining distance to the construction area.

Lake Ontario

46. Background. Lake Ontario, the smallest of the Great Lakes, has the shortest shoreline within the United States. Lying entirely within the State of New York, it extends 290 miles from the mouth of the Niagara River to the head of the St. Lawrence River. The shoreline is fairly regular, running in an east-west direction from the Niagara River for approximately 160 miles, then diverting to a north-south direction and becoming irregular with several large bays in the northern half. The lake is approximately 804 ft deep at the deepest point, where the bottom is 561 ft below sea level. This is lower than the bottom of any of the other lakes except Lake Superior. The lake bottom slopes gradually southward from the north shore, across more than two-thirds of the lake. The bottom formation then rises abruptly to the south shores. The retreating glaciers deposited sediments in the lake and along its shoreline.

47. Sediments in Lake Ontario are derived mainly from the rivers and erosion of the shoreline bluffs. Suspended particles from the Niagara River discharges are carried considerable distances into the lake. The large accumulation of sediment at the eastern end of the lake indicates that a major part of the suspended input moves in a west-to-east direction, paralleling the direction of the flow of the lake and the prevailing westerly winds, and constitutes a modern process. Littoral drift directions are relatively easy to infer from patterns around shore features and from textural gradients.

48. Littoral drift patterns reflect the general wind conditions for the lake with the prevailing westerly winds being responsible for the net eastward drift from the Niagara River along both the south and north shores. Periodic easterly storms can account for the net westward drift from the Welland Canal toward Hamilton, while surface currents, upwelling, and easterly storms will account for a net westerly drift toward Hamilton from the northern shore. In the past, offshore disposal of dredged material was a minor source of sedimentation for the lake.

49. Scour problems. Secondary construction effects experienced

in other lakes regarding the propagation of wave crests along vertical sheet-steel breakwaters and scouring of the shore section to collapse of the structures have also been observed in Lake Ontario. In addition to this phenomenon, at Great Sodus Bay, New York, another separate and distinct problem existed in which surface wind stress and other meteorological conditions caused a setup of elevation of the water surface in the bay. The resulting seiches, acting as short-period low-amplitude tides, caused flow to occur into and out of Great Sodus Bay. The outflow currents, or ebb flows, would tend to meander and cause the location of the navigation channel between the 450-ft-wide jetties to shift temporally. Eventual meandering brought the channel in close proximity of the south jetty, and scouring and erosion at the toe resulted. During the mid-1960's, it was necessary to rehabilitate this structure with sloped rubble-mound construction in order to provide stability to both the jetty and the navigation channel.

North Atlantic Region

50. The Atlantic coast of the United States north of New York City was formed by glacial action during the Pleistocene ice age, as was the Great Lakes region, with the continental glaciers extending as far south and east as Nantucket Island, Massachusetts. Hard, enduring rock symbolizes the area, and most of the beaches reflect the nature of their development. The New England coastline has had a complex history of glacial scour and this has resulted in the most irregular shoreline in the nation. Sand is not nearly as plentiful on most parts of this region as elsewhere because the hard rocks of the area do not produce much sand by weathering, and the rivers generally discharge into drowned valleys.

51. South of New York City the coastline is deeply embayed, with the coastal plain being bordered by long sandy barrier island beaches. These barrier beaches are, in general, separated from the mainland by partially filled lagoons. The coasts of this region are exposed to the full force of the Atlantic hurricane, as well as the local "northeasters"

which consistently plague the regions north of Cape Cod and are occasionally felt all the way south to Cape Hatteras.

New England region

52. Background. The entire New England region, from the western Connecticut border to Canada, is serviced by the U. S. Army Engineer Division, New England. As such, this one unit has responsibility for protection and maintenance of the beaches, coasts, navigation channels, and other works of improvement along this portion of the United States coastline (Figure 13). Beach and shore erosion is one of this area's serious problems, and this concern has led to increasing involvement

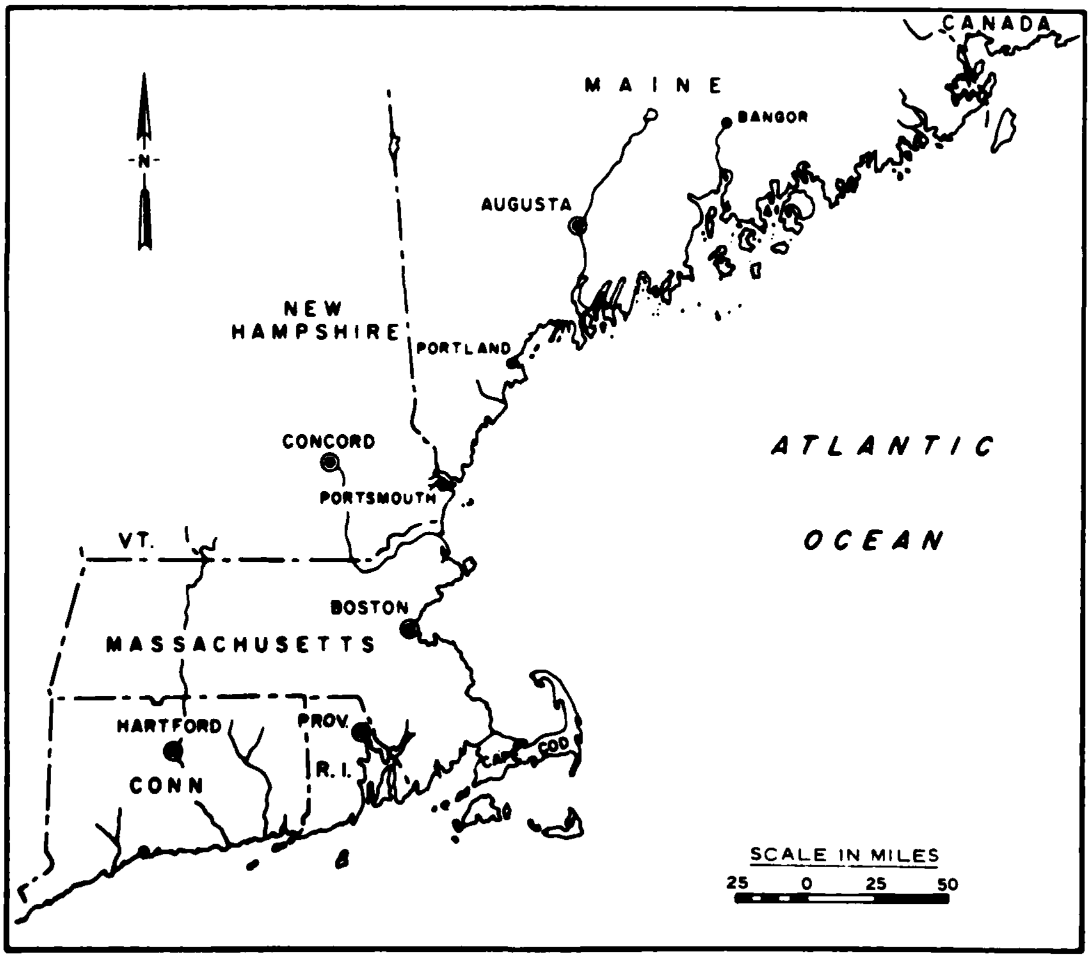


Figure 13. New England region

in shore protection of the coastal states by the CE. Over the years, the CE has been charged with the study of publicly owned shores; in 1956, this authority was broadened to include the protection of shores other than public if such protection is incidental to the protection of the publicly owned shores.

53. Closely related to shore erosion control is the associated problem of hurricane protection. Severe damage has been sustained along the southern coastal areas of the United States and along the eastern shores as far north as Long Island, Rhode Island, and the southern shore of Cape Cod. The north shore of Cape Cod and the remaining New England coastline have not experienced serious hurricane damage in the past; however, these areas are vulnerable to the much more frequent but lesser magnitude winter storms commonly referred to as "northeasters."

54. The northeaster is much like the hurricane in that it is typically generated in the tropical area of the Caribbean and follows a gentle curving, clockwise coastal route toward New England. It usually has accompanying high winds, though not normally of hurricane force, and is not as well defined a storm as a hurricane. The storm is usually associated with heavy precipitation in the form of rain and/or snowfall. The slow forward movement of these storms, sometimes causing them to remain in the New England area for three or more days, increases the coastal damage associated with them since several tidal cycles may occur with gale wind conditions and high tides ranging between 3 to 5 ft above normal. Along the New England coast, two recent peaks of storm activity have been detected, one in December and the other in February 1978. These two maxima are not really pronounced, however, since cyclonic activity is high from November through March.

55. Scour problems. In the New England area, it is standard practice to recommend to coastal work contractors that sand placement and other structures be built only at certain times of the year. It is probably impossible to work at all during the period December-February because of inclement weather throughout the region and additional ice problems. However, in the summer months, the beaches are crowded with

bathers, and the use of heavy equipment can be problematical. Hence, it is virtually necessary to perform construction in the spring and fall.

56. The ordinary wave climate varies considerably along the New England coast. Lower Cape Cod to the shores of Connecticut along the Long Island Sound usually has waves about 2 to 3 ft in height. The northeastern shore of Cape Cod through the Boston area has essentially an unlimited fetch distance, and therefore the waves in this region are frequently in the range of 5 to 7 ft. In addition, the spring tide levels are higher than the average tide range, and the wave climate is superimposed on top of already high-water levels. Hence, the potential for wave damage and erosion varies from relatively moderate along the Connecticut coast to potentially high north of the cape.

57. Between 19 and 27 February 1969, three very large storms entered the Merrimack Embayment and caused extensive damage to the shore side of the south jetty (as wave energy penetrated the jettied entrance). Waves overtopping the north jetty eroded approximately 260 ft of beach sand from the area. In an attempt to halt the erosion process, a rock revetment structure was constructed. The purpose of this revetment was to prevent flanking of the south jetty structure. The U. S. Army Engineer Waterways Experiment Station (WES) was requested to conduct a physical model study to investigate, among other things, the mechanism by which scour and erosion are taking place inside the south jetty. Based on results of the three-dimensional model investigation by Curren and Chatham (1979), it was determined that of the improvement plans tested involving the north jetty, a 900-ft extension was optimum with respect to wave protection, prevention of erosion on Plum Island, and construction costs. Hence, the secondary construction effects produced by wave energy passing between the jetties can be alleviated by an extension and a curvature of the north jetty. It remains to be seen whether or not this plan is economically justifiable.

58. It is customary in this region to put a filter blanket layer of gravel or small rock under new stonework, but this is usually not necessary in the rehabilitation of existing structures. In order to prevent scour around the toe during construction, the filter layer is

placed ahead of the core and cover stone approximately 50 ft from the entire width of the structure base. Gabion units have been used as riverbank stabilization measures but have not been utilized beneath breakwaters or jetties in the New England region.

59. Scour may not in all cases be considered a detriment. For example, the functional design of jettied navigation entrances to harbors considers the application of tidal currents to keep the channel open by removing sediment and littoral drift, and thus minimize dredging. In general, the navigation channels in the New England area do not migrate a sufficient amount to cause undermining of the structures. This is reflected by the fact that the New England Division does not have to spend an unusually large amount of time or money rehabilitating the jetties. Because the scour control methods appear to be working and because of the physiography of the glaciated coastal region, scour is not a pressing high-priority problem in the New England area. Future work on the exposed shores of the Cap Cod islands may encounter greater problems with scour.

Central North Atlantic region

60. Background. The central North Atlantic region of the United States, extending from Long Island, New York, to the northern part of North Carolina (Figure 14) is the oldest major population area in the nation and is also one of the oldest in geologic terms. Its age is reflected in the relative straightness of the shore all the way from the mouth of the Hudson River to Cape Hatteras. There are many estuaries and bays of varying size along this reach of coast, but the sand beach itself curves in a smooth regular pattern.

61. All along this east coast from just below New York Harbor lies a string of long narrow barrier islands. These islands range from 1 to 3 miles in width and are separated from the mainland by estuaries that vary in width from river size to over 5 miles. The inland bays are connected to the ocean by tidal inlets of various sizes that separate the barrier beaches into finite islands. The inlets may be fairly close to each other in places, or they may be as much as 30 miles apart. Two of the nation's major bays, Chesapeake Bay and Delaware Bay, are

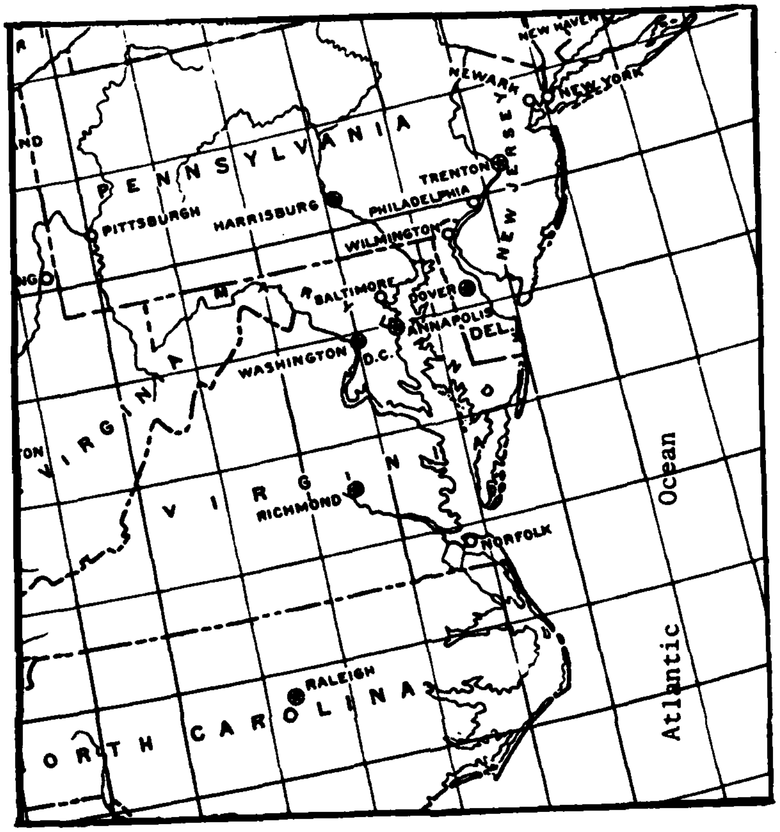


Figure 14. Central North Atlantic region

located along this section of coastline.

62. The formation of the barrier beaches is related to the gentle sloping shores of the area. The sand originally carried into the sea by the rivers and streams was dropped a little at a time by the longshore currents in relatively straight lines. These depositions continued to build up, first as longshore bars and later as protruding island masses. The process was aided by windblown sand carried from the mainland out to the long thin barrier beaches. While there are places where the coast is exposed to the open ocean, the barrier islands are in effect the Atlantic shore.

63. Scour problems. The south shore of Long Island is also characterized by a series of barrier beaches, which are separated from the mainland by tidal bays that vary in width up to 5 miles. The width

of the barrier beach is generally less than one-half mile and in some locations is less than 100 ft. The islands are interrupted by inlets at several locations, and navigation improvements such as dredging and stabilization structures are maintained as well as shore protection features such as groins. The characteristics of the incoming surface gravity waves in this area are, generally, such that the direction of net longshore sediment transport is to the west along Long Island and then south along the New Jersey, Delaware, Maryland, and Virginia coasts. The inlets along the south shore of Long Island that are exposed to the full forces of the north Atlantic storms have previously been stabilized by major jetties. A significant amount of maintenance dredging is required to keep the inlets navigable as littoral drift on the order of 200,000 cu yd net annually is transferred past the east jetties and into the channels. While rehabilitation work to the existing structures is performed as required, major new stoneworks are not anticipated in the foreseeable future except for the construction of one groin at Rockaway Beach in the early 1980's. The rehabilitation work which has been performed to the existing structures in this region was such that additional foundation stabilization measures were not necessary.

64. Barnegat Inlet, New Jersey, approximately 55 miles south of Sandy Hook, is an example of a rehabilitation work of sufficient magnitude to require specific foundation design measures. The project consists of a channel protected by two converging stone jetties. In order to allow for successful maintenance dredging of the inlet channel, raising of the north jetty was begun in August 1972 and completed one year later at a cost of approximately 1.4 million dollars. The purpose of the foundation mat (Figure 15) is not only to distribute the load over a wider base but also to prevent shear failure and erosion of the underlying soil at the toe of the rock mound. When load is placed on a soil exceeding its bearing strength, the soil will fail by shearing along a curved plane, cutting the bottom at some distance beyond the toe of the superimposed load. Therefore, a base of material stronger than the soil but smaller than the core stone should be laid which extends beyond the toe and the plane of failure. A mat layer of material

varying in size from 3 to 50 lb and approximately 2 ft thick was required to protect the structure from settlement and scour failure.

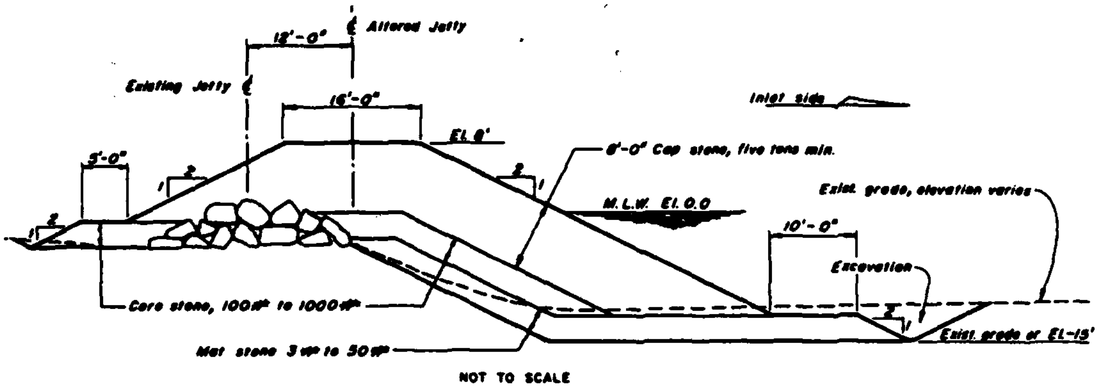


Figure 15. Typical jetty section, Barnegat Inlet, New Jersey

65. The Delaware Coast Hurricane and Flood Protection Study was a major analysis of proposed interrelated structures such as levees, dikes, seawalls, bulkheads, and groins to protect this region of coastline from a design storm. Knowledge of the magnitude of the amount of scour to be expected from the design storm was imprecise. Guidance along this line was obtained from studying the effects of known storms at specific locations. It was ultimately determined that an expected scour of 6-ft depth in front of concrete seawalls would not be unrealistic from the design storm. Therefore, seawall toe protection of a graded layer topped by cover stone was developed, although serious questions arose regarding the size of the cover stone, as it had previously been observed that wooden bulkheads had been completely destroyed by stone being tossed like missiles under severe wave attack. The project has not been implemented to date.

66. Ocean City, Maryland, is the principal seaside resort on the Atlantic coast of Maryland and is situated on the barrier island. A hurricane opened an inlet across the island immediately south of the city in 1933, and the inlet was stabilized in 1935 when jetties were built on each side of the inlet. A strong littoral current flows southward in this region; and in order to inhibit beach erosion, a series of

groins was constructed in the mid-1950's. This coast has felt the full intensity of winter Atlantic storms and is believed to have a net southerly transport of littoral material of approximately 150,000 cu yd per year. Frequent dredging of the inlet is required. It is believed that the south jetty is not sandtight, and some littoral material may be passing through the south jetty into the inlet.

67. An intense Atlantic storm of 1962 seriously eroded the barrier island away from the south jetty at Ocean City, Maryland, and cut back beyond the landward end. Major rehabilitation plans are being developed to extend the jetty landward to stop the breach of the island. This construction will take place in the presence of both tides and wind waves. Van Heemskerck (1963) recommends specific construction methods for such cases and discusses foundation and scour problems of closures.

68. In the region immediately south of Chesapeake Bay, the Rudy Inlet-Virginia Beach Hurricane Protection and Beach Erosion Control Project is considering the extension of the north and south jetties at Rudy Inlet, and a possible offshore beakwater for beach protection. A system of groins will be constructed if maintenance experience indicates the need and justification thereof. Rudy Inlet is significantly smaller than the majority of tidal inlets; and therefore, the structure extensions probably will not incorporate additional elaborate features.

South Atlantic Region

69. Background. The Atlantic coasts of both North and South Carolina are uniquely characterized by a series of seaward projecting barriers called cuspate forelands. On the north, the curving barrier islands enclose Albemarle and Pamlico Sounds, large lagoons that are combined with estuaries. These and other coastal bays of the area are connected to the ocean by about 50 relatively permanent tidal inlets, and many others are opened temporarily during major storms. The barrier islands along the South Carolina and Georgia coasts are short and thick, rather than being long and narrow as previously encountered. Along the Florida Atlantic coast, the lagoons are mostly very narrow

and would have been filled entirely except for having been kept open by the Atlantic Intracoastal Waterway. The wide lagoons south of Miami are protected by an arc of coral reefs that appear in planform to be an extension of the sandy barriers to the north. This area is shown in Figure 16.

70. Scour problems. Masonboro Inlet, a natural inlet through the coastal barrier beaches of North Carolina, is located in the southern portion of the state, approximately 8 miles northeast of Wilmington, North Carolina. Evidently, the inlet has been open almost continuously since around 1733, although it has migrated extensively to its present location. Improvements for the inlet, authorized in 1949, included two jetties, an ocean entrance channel between the jetties, and interior navigation channels to the Atlantic Intracoastal Waterway. Due to funding limitations, it was proposed to construct the north jetty initially since it was on the apparent updrift side of the inlet. The plan of improvement consisted of a north jetty with a low interior weir, a deposition basin adjacent to the north jetty, and the reestablishment of the navigation channel. Construction of this plan was completed in June 1966. Within three years, the navigation channel had migrated through the deposition basin and was endangering the stability of the north jetty. WES (Seabergh 1976) was requested to conduct a physical model study of the inlet to investigate, among other things, the optimum length and alignment of the south jetty. It was determined that the south jetty should be located on a sandy shoal region where the mean water depth is typically about 5 to 6 ft. Because of this, the project design included foundation scour control blankets and construction was under way during the spring and summer of 1979. Typical jetty cross sections are shown in Figure 17 where it can be observed that a gabion foundation blanket has been specified.

71. Gabions are relative newcomers to American construction but have been installed in their present form in Europe since the late 1800's when the metal gabion was developed. This form (Figure 18) provided the necessary structural strength; and galvanizing added durability with economy, thus broadening the field of applicability. Gabions

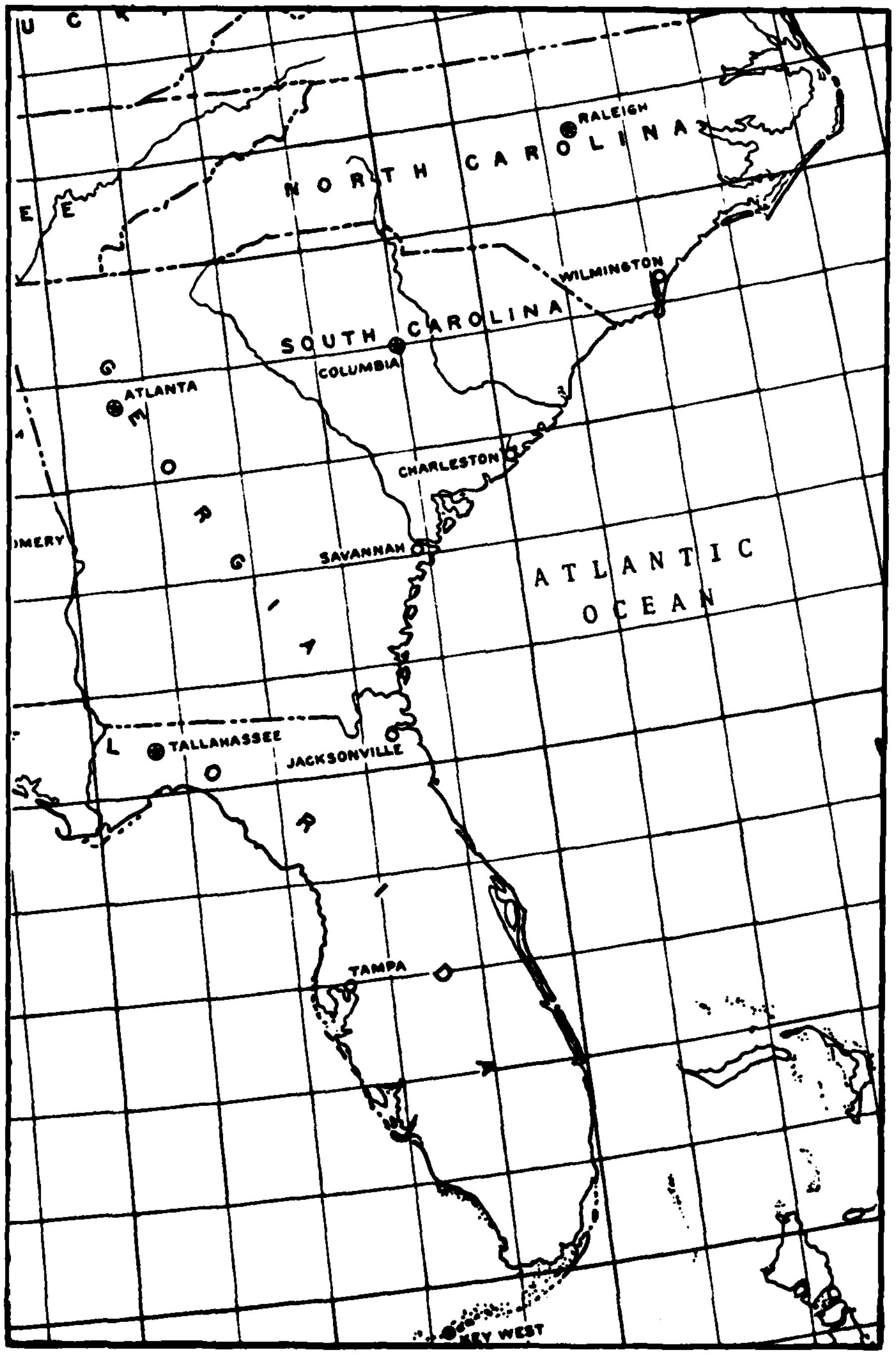


Figure 16. South Atlantic region

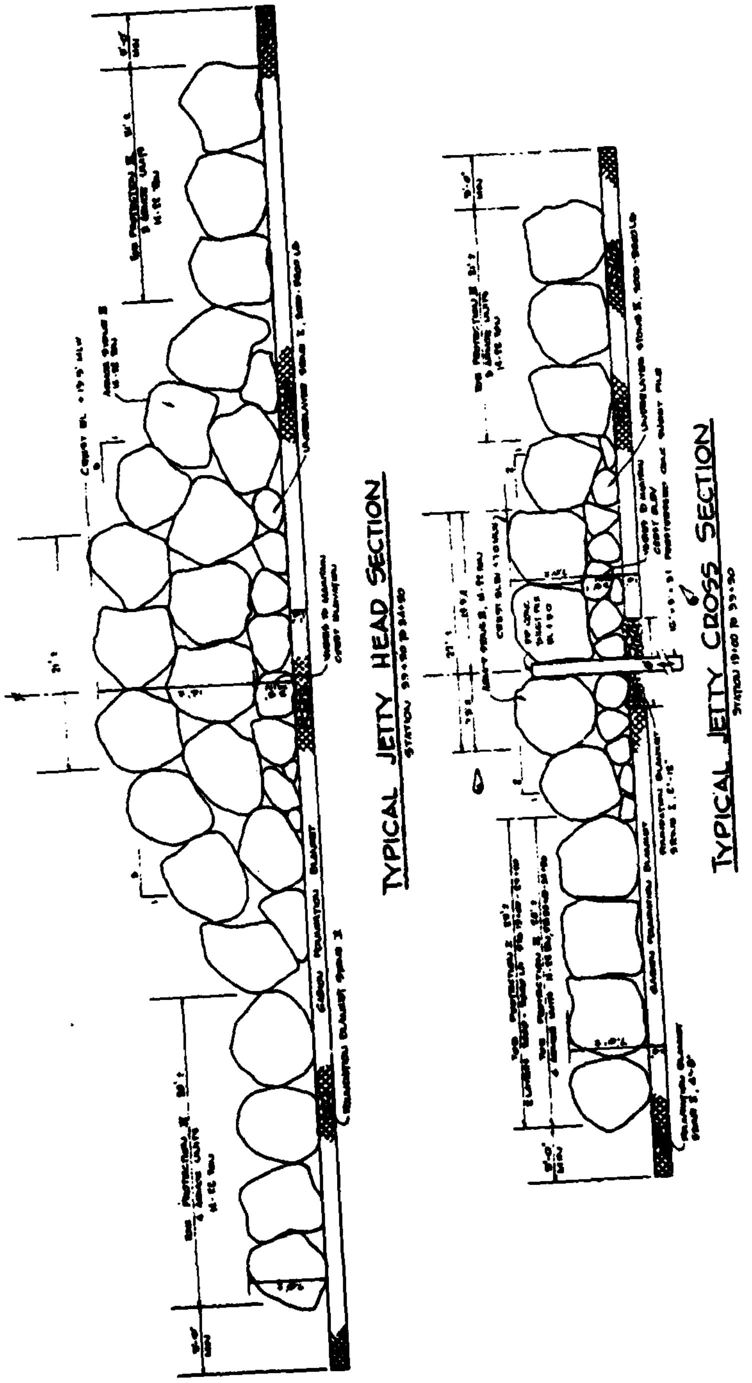


Figure 17. Typical jetty cross sections, Masonboro Inlet, North Carolina

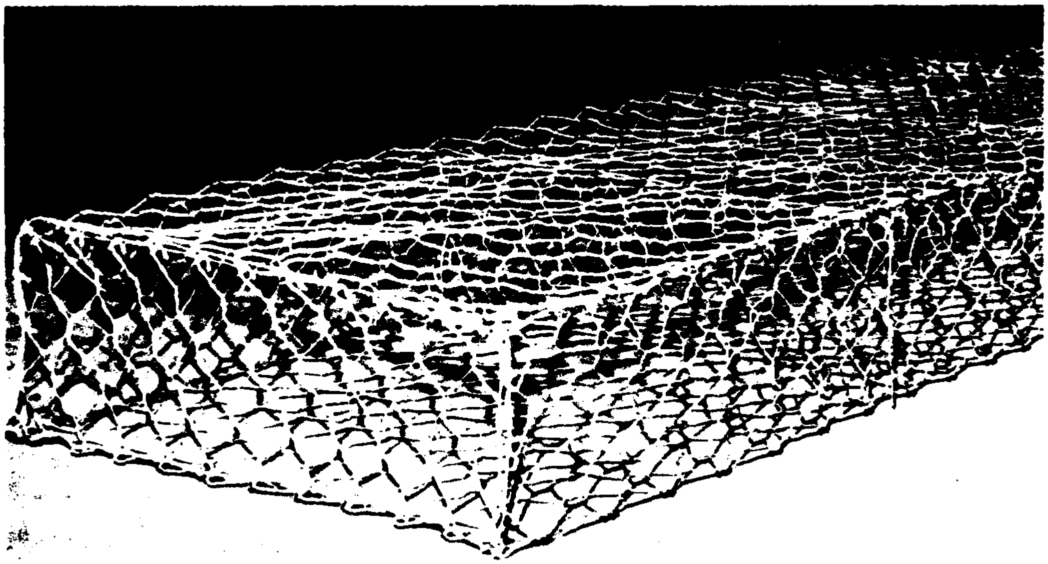


Figure 18. Gabion unit prior to filling with stone (after Terra Aqua Conservation, Bekaert Steel Wire Corp., 1977)

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

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

73. The foundation blanket specified for Masonboro Inlet will

consist of an 18-in.-thick foundation layer of Stone I (ranging in size from 2 to 12 in.) between sta 0+00 and 13+00, and shall consist of 12-in.-deep gabion baskets lined with synthetic filter fabric and filled with foundation blanket Stone II (ranging in size from 4 to 8 in.) between sta 13+00 and 34+50. The foundation blanket shall be composed of tough, durable stone reasonably free from dirt or other foreign particles. The blanket stone shall consist of either shell (marine) limestone having a unit weight not less than 118 lb per cu ft saturated surface dry (specific gravity 1.9) or granite stone having a unit weight not less than 160 lb per cu ft saturated surface dry (specific gravity 2.56). The stone shall be graded as follows:

Foundation Blanket Stone I (sta 0+00 to 13+00)

Percent by Weight Passing	Sieve Size, in.
95-100	12
50-70	9
25-45	6
10-20	3
0-5	2

Foundation Blanket Stone II (sta 13+00 to 34+50)

Percent by Weight Passing	Sieve Size, in.
95-100	8
40-60	6
0-5	4

74. The gabions shall be made of wire mesh with heavily zinc-coated steel wire. The mesh opening shall be of a size that will contain a 4-in. rock measured in any dimension. Gabions shall be supplied with dimensions 12 ft long, 3 ft wide, and 1 ft deep and shall be fabricated in such a manner that the sides, ends, lid, and diaphragms can be assembled at the construction site into a rectangular basket of the specified size. Lacing wire shall be supplied in sufficient quantity to securely fasten all edges of the gabion. The wire mesh shall have a minimum diameter of 0.114 in. and a tensile strength of at least 70,000 psi. The wire mesh shall be fabricated in such a manner as to be nonraveling, and the zinc coating and tensile strength shall meet

all requirements of Federal Specification QQ-W-461. The interior of the baskets will be lined with plastic filter fabric.

75. Between sta 0+00 and 13+00, the foundation blanket stone shall be placed by methods that will tend to prevent segregation of stone sizes. The placing device shall be lowered to rest before releasing the stone. An allowable tolerance of plus 6 or minus 3 in. will be allowed in the thickness of the completed foundation blanket. Compaction of the stone will not be required, but it shall be spread and finished to a reasonably even surface free from mounds or windrows. The foundation blanket stone shall be used as a filler to compensate for ground variations.

76. Between sta 13+00 and 34+50, the foundation to receive the gabions shall be reasonably smooth, free of irregular depressions or mounds. Irregularities in the foundation shall be corrected, or in the case of natural depressions or holes, by filling with foundation blanket Stone I. The gabions shall be placed as close to the theoretical side of the sheet-pile wall as practicable; however, the gap between gabions placed on each side of the center line shall not exceed 8 ft to allow for sheet-pile wall construction. Following the sheet-pile wall construction, the void between the wall and the gabion shall be filled with foundation blanket Stone II. The gabions may be placed either perpendicular or parallel with the axis of the jetty to best fit the minimum dimensions, except that the peripheral two rows shall consist of gabions having a minimum length of 6 ft placed perpendicular to the axis of the jetty.

77. During the placement of the stone and gabion units, care shall be taken to ensure that the various classifications will make a compact mass and form as nearly as practicable a cross section of uniform height, width, and slopes. All stones shall be placed so as to leave no large voids between them. Stone placement shall commence at the end nearest shore and continue seaward. In placing the sheet pile, scour may develop around the edge and end of the jetty as the work progresses. The contractor shall conduct his operations to minimize

this scour by placement of the foundation blanket at least 200 ft ahead of the underlayer and cover stone. The placement of the underlayer and armor stones along both sides of the sheet-pile wall shall follow as closely behind the placement of the concrete sheet piles as practicable.

78. Toe protection measures to eliminate scour and undermining of the structure shall consist of stone weighing from 3,000 to 5,600 lb (sta 0+00 to 25+00) with at least 60 percent of the stone weighing 4,600 lb or more. Between sta 8+00 and 13+00, right side, the stone shall consist of stones weighing from 5 to 8 tons with at least 75 percent of the stones weighing 6.5 tons or more. Between sta 13+00 and 34+50, right side, the stone shall consist of stones weighing from 14 to 22 tons with at least 75 percent of the stones weighing 18 tons or more.

79. The synthetic filter fabric used to line the gabion units shall be a pervious sheet of plastic yarn with an Equivalent Opening Size (EOS) no finer than the U. S. Standard Sieve No. 80, and no coarser than the U. S. Standard Sieve No. 30. The plastic yarn shall consist of a long-chain synthetic polymer composed of at least 85 percent by weight of propylene, ethylene, ester, amid, or vinylidene chloride, and shall contain stabilizers and/or inhibitors added to the base plastic if necessary to make the filaments resistant to deterioration due to ultraviolet and heat exposure. The fabric shall conform to the physical strength requirements as follows: (a) tensile strength, 200 lb minimum in any principal direction, ASTM D 1682 Grab Test method using 1-in. square jaws and a travel rate of 12 in. per min; (b) puncture strength, 80 lb minimum, ASTM D 751 Tension Testing Machine with Ring Clamp, steel ball replaced with a 5/16-in.-diam solid steel cylinder with a hemispherical tip centered within the ring clamp; and (c) abrasion resistance, 55 lb minimum in any principal direction, ASTM D 1682 as in tensile strength, after abraded as in ASTM D 1175 Rotary Platform, Double Head Method, rubberbase abrasive wheels equal to CS-17 "Calibrase" by Taber Instrument Co.,

1 kilogram load per wheel, 1,000 revolutions.

80. The fabric shall be fixed so that the yarns will retain their relative position with respect to each other. The edges of the fabric shall be finished to prevent the outer yarn from pulling away from the fabric. The seams of the fabric shall be sewn with thread of a material meeting the chemical requirements given above for plastic yarn or shall be bonded by cementing or by heat. The sheets of filter fabric shall be attached at the factory or another approved location to form sections not less than 6 ft wide. Seams shall be tested in accordance with method ASTM D 1683, using 1-in. square jaws and 12 in. per min constant rate of traverse. The strengths shall be not less than 90 percent of the required tensile strength of the unaged fabric in any principal direction.

81. According to the work of Keown and Dardeau (in preparation), in 1956, as the result of severe North Sea storm and tidal damages, the Dutch Government initiated an extensive construction program to minimize the future deterioration of stream and coastal protective works. This activity was the first known application of filter fabric material as a component of a major hydraulic structure. The use of petrochemical-based synthetic materials did not find immediate acceptance in the American engineering community. As late as 1967, there were only two domestic sources of filter fabric, although at the present time there are at least 20. In addition to the problem of fabric availability prior to about 1970, no direct cost comparisons were documented and widely distributed that encouraged the use of filter fabric as a substitute for granular filters. As nonwoven or random fiber fabrics became available, it was clear that an examination of fabrics and methods of evaluating their engineering properties was necessary. Laboratory and field investigations were conducted by Calhoun (1972) to obtain data for the development of design criteria and acceptance specifications for plastic filter cloths used as replacement for granular filter materials. Laboratory tests were conducted at WES during the period 1974-1976 to refine existing test methods for woven

fabric and to develop new methods for the evaluation of nonwoven fabrics. Results of this effort and further field experience provided the basis for new OCE Guidelines (1977) which made provisions for field use of woven and nonwoven synthetic filter fabrics.

82. Improvements to Oregon Inlet, North Carolina, are in the planning stage and consist of dual stone or dolos jetties extending approximtely 6,000 ft from shore. These structures will be constructed on sandy shoal regions, and present plans envision the use of gabions as foundation bedding material. WES was requested to perform both a three-dimensional model investigation to aid in determining the optimum length and alignment of the jetties, and a two-dimensional and three-dimensional investigation of the stability of the proposed jetties against wave attack. These studies are being conducted at this time.

83. Based on the hydrography and physiography of the region, and on historical data from near localities, conclusions were reached by the U. S. Army Engineer District, Wilmington, regarding the stability test of the head sections of the jetties. It is believed that after construction of the jetties, the flow fields and current patterns over the bar region will be altered to such an extent that the shoal region will readjust itself to more nearly conform to the offshore topography upcoast and downcoast. This will result in the removal of material for a depth of about 11 ft in the vicinity of the structure head section, and the conclusion was reached that the readjustment will develop on a slope of around 1V on 5H. Hence, the stability tests are being conducted as if the hydrography in the near region of the structure head were readjusted for a depth of around 11 ft, with a resulting 1V-on-5H slope away from the structure to deeper water.

84. An analysis of past wave conditions in the area suggests that a 17.6-ft wave height would be appropriate for a 1-in-50-year stability analysis. Additionally, because of the relatively frequent large storm and hurricane probability for the region, 8-ft storm surges are not uncommon. Therefore, the design wave height of 17.6 ft may arrive at either a low water elevation or any other elevation up to the maximum

storm surge height. At the maximum storm surge elevation, the design wave will not break directly on the outer (head) section of the structure; however, at a low water datum, the dynamics of the wave motion are such that the structure may experience the full force of waves breaking directly on the structure. Therefore, the stability tests of the jetty head are being conducted for both breaking and nonbreaking wave conditions using a design wave of 17.6 ft.

85. Until 1978, Murrells Inlet, South Carolina was an unimproved inlet about 19 miles northeast of the city of Georgetown. As part of the Grand Strand, a resort area consisting of sandy beaches, sand dunes, marshlands, and maritime forests stretching from North Carolina to Winyah Bay (Georgetown), South Carolina, this inlet provides access to a well-mixed tidal lagoon of ocean salinity that has no source of freshwater inflow other than local surface runoff. The inlet maintains its existence due to tidal currents (mean ocean tide range 4.8 ft) which produce a tidal prism of 253 million cu ft flowing through the inlet during a tidal cycle. In opposition to the tidal currents that tend to maintain an open inlet are littoral currents generated by wind waves carrying sand along the shoreline into the vicinity of the inlet, causing the formation of sand shoals. These shoals prevent easy ingress and egress through the inlet due to their ephemeral nature with respect to location and depth. Waves breaking on these shallow shoal areas contribute to difficult and sometimes hazardous navigation conditions.

86. Physical model tests were performed by Perry, Seabergh, and Lane (1978) to aid in determining the optimum solution to navigation problems at Murrells Inlet. The final plan which evolved with the aid of the model is illustrated in Figure 19. The plan included construction of a sand dike at a +10.0 ft mlw elevation parallel to the south shore to provide the landward end of the rubble-mound south jetty which would be constructed across the existing ebb current outflow channel and onto the shoal area.

87. The Murrells Inlet north jetty was completed in early 1979 and included a foundation blanket of filter material for bearing

strength and erosion control. Scour was minimal to nonexistent during construction of the north jetty.

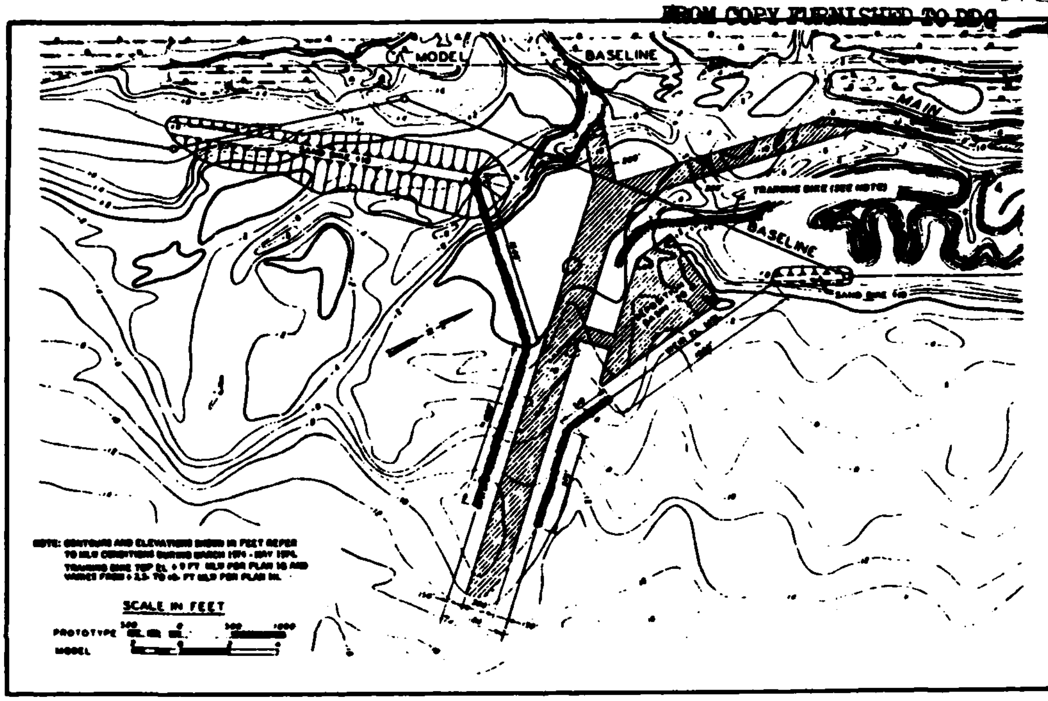


Figure 19. Murrells Inlet, South Carolina, model layout

88. Construction of the south jetty was initiated upon completion of the north jetty in February 1979. Since the south jetty was planned to be constructed across the offshore shoal region, it was necessary to excavate some areas in order to reach the design grade for the structure foundation. The operational procedure for keeping a trench open long enough to physically place two or three grades of stone (developed from the best available design guidance) is difficult at best; hence, it was anticipated (desired) that as construction of the south jetty proceeded from the south shoreline across the ebb channel, beneficial scour would result. A general view is provided of the area in Figure 20 which shows the completed north jetty, the beginning phase of the south jetty construction, and dredge performing maintenance operations,in the navigation channel.

Figure 20. Murrells Inlet, South Carolina, after construction of north jetty, 7 May 1979

89. A 2-ft-thick foundation blanket was designed to support the core stone, and the initial placement was uneventful (Figure 21, 27 February 1979). It was expected that as closure of the channel proceeded, the reduced cross-sectional area for flow would experience an increased current velocity sufficient to initiate sediment movement and assist in the necessary excavation of the shoal. Since the shoal is visible at low tide but is not exposed at high water, it would be necessary to excavate about 2 ft of material to reach the design grade for the south jetty. It was estimated that as construction proceeded across the channel, shoal erosion of 8 to 10 ft might occur on the shoal slope until the ebb discharge could redistribute and become concentrated in the new relocated navigation channel.

90. Scouring action that resulted from partial closure of the ebb channel was more severe and intense than anticipated, and the channel bottom scoured to approximately 5 ft below the design grade for the structure. More importantly, the ebb current was so intense that the foundation bedding material, which was being placed in the channel, would not remain long enough for the stabilizing core stone to be placed. Figure 22 (7 May 1979) indicates the manner in which the scour channel was relocating itself in front of the new jetty construction. Emergency operation procedures were initiated when it became apparent that ordinary construction techniques would not be sufficient to permit further placement of structure material. It was necessary to divert the dredging operation to provide fill material by pumping sand along and in front of the new jetty section. This action has been successful to the extent that the contractor is able to proceed with construction as the pumping and filling operations are able to stay ahead of the stone placement.

91. The problem of crossing and closing a channel containing a strong tidal current is indeed a unique construction operation, and one for which the usual scour control techniques may not be entirely successful. This situation is somewhat similar to the Netherlands problems of closure of large estuaries on the North Sea. However, those dike construction projects are enormous in magnitude; and that nation is

Figure 21. Murrells Inlet, South Carolina, showing initial phases of south jetty construction, 27 February 1979

Figure 22. Murrells Inlet, South Carolina, showing ebb channel relocation and scouring of shoal region in front of south jetty construction, 7 May 1979

mobilized sufficiently to combat the problem whereas smaller contractors working on smaller individual projects may be required to devise new techniques on a site-to-site basis as the need arises, based on the recommendations of Van Heemskerck (1963).

92. Further south, along the Georgia coast, major structures in the coastal zone are minimal. Although the Savannah River entrance is jettied and there has been one groin constructed on Tybee Island, the measures are essentially problem-free. On the ocean side of St. Simon Island, private interests have built three groins and have requested permits to build five more with accompanying beach nourishment; however, this activity is not being monitored at the present time as it remains a private enterprise.

93. The western portion of the southern part of the Gulf Stream originates in the Straits of Florida between southern Florida and Cuba, flows north very near the coastline, and has come to be known as the Florida Current. Relative to other open-ocean currents, the Florida Current is fairly strong, approaching a magnitude of 2 m per sec a few kilometres offshore. The strength of the current decays nearer to the coast, however; and this magnitude is probably not sufficient to account for the transport of sediments. At the same time, locally generated currents may interact with the nearshore extremities of the Florida Current to create zones of currents adequate to move sandy material. It is known from observation that northward flowing nearshore ocean currents exist, and the scour problems experienced by the U. S. Army Engineer District, Jacksonville, may be related to current conditions as much as to wave action effects.

94. Inside the littoral zone, the biggest problems seem to be associated with the effects of jetties and groins on the adjoining shores. While the origin of the problem of shoreline readjustment is well known, the specific solution for an individual site is not always readily apparent. The manner in which the adjacent shoreline readjusts itself to the man-made features is not always identical under apparently similar circumstances, because the phenomena causing the effect may be

obscured by more dramatic features. Indeed, the wave climate itself changes significantly from northern to southern Florida, as the sheltering Bahama Islands intercept a large percentage of wave energy arriving from the southeastern direction. The net littoral drift direction is from north to south all along the Florida Atlantic coast except for small isolated situations, according to Walton and Dean (1973).

95. The most recent jettied structures constructed along the Florida east coast were the north and south jetties at Ponce de Leon Inlet, approximately 100 miles south of Jacksonville, completed in 1972. Construction was initiated without design of a foundation blanket. Scour on the order of 10 ft occurred as construction proceeded seaward, and material overruns as much as 300 percent were experienced along some sections of the jetties. A foundation blanket design was subsequently added to the contract and alleviated most of the scour problems.

96. Actually, scour was not an uncommon occurrence when the first jetty construction projects were undertaken in this region. Contractors were accustomed to the necessity of having to place additional material to account for development of a scour hole. Originally, cypress mats were fabricated and secured on the bottom with large stone. The graded rock filter foundation blanket was introduced later. When a tidal hydraulic head of 2 to 3 ft exists across an inlet connecting a fairly large estuary to the ocean, velocities on the order of 6 to 9 fps are not uncommon, and observations indicate that scour holes may develop which approach 12 to 15 ft in depth. Early contractors were believed to have unbalanced their contract bids on jetty projects; i.e., they appeared to bid a relatively low unit price on the core and cover stone, but the bid price of the foundation bedding layer would be unusually large in the anticipation that large quantity overruns would be necessary to fill scour holes. However, the philosophy of the time was that the additional material required to fill scour holes actually contributed to a much stronger and more stable structure.

97. A "rule of thumb" to estimate the expected scour over a long period of time near reflecting structures (CERC 1975) is that the

maximum depth of a scour trough below the natural bed is about equal to the height of the maximum unbroken wave that can be supported by the original depth of water at the toe of the structure. This is occasionally interpreted as the maximum scour depth to be expected should be of the same order of magnitude as the significant wave height that occurs during storms at the local area.

Gulf of Mexico

98. The Gulf of Mexico is a relatively shallow oceanic-type basin with the greatest depth slightly more than 12,000 ft and covers an area of approximately 619,000 square miles (Figure 23). Early researchers

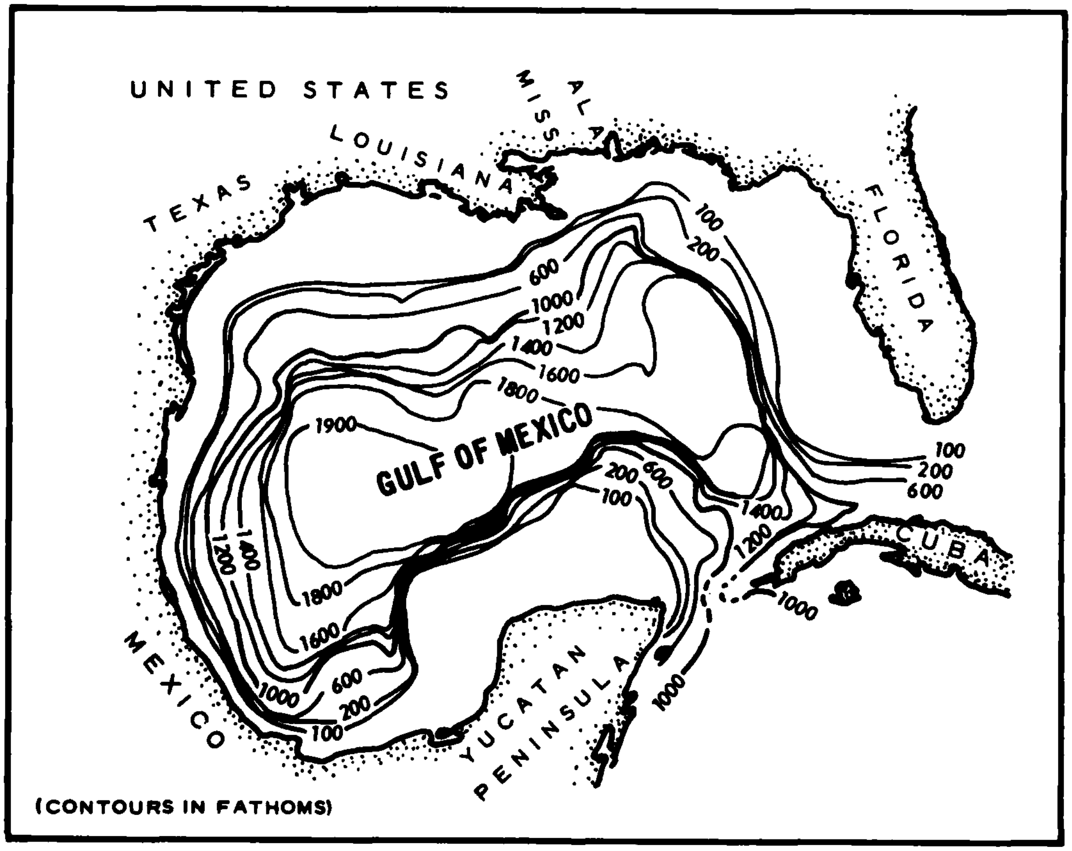


Figure 23. Gulf of Mexico

believed that the central portion of the gulf was originally an extension of the coastal plain of the United States and that the present basin resulted from the collapse of this extension sometime during the Cenozoic era. Recent geophysical information suggests that at least the central portion of the gulf is underlain by a typically oceanic crust, implying that it has always been a basin. This is supported by the theories of plate tectonics, continental drift, and seafloor spreading, which indicate that the Earth's crust is composed of a series of relatively rigid plates floating on the Earth's mantle and which are continually drifting apart. Although there is no complete agreement on the number, six major plates are recognized and about twenty small plates are known to exist. The continental drift theories are intuitively supported by the relatively good manner in which the continents can be fitted together in one unit. Except for slight overlaps and a few gaps, the continents of North and South America and Africa appear at one time to have been joined. Upon their separation by drifting plates, the Gulf of Mexico and other depressions of the Caribbean were probably formed (Shepard 1973).

99. Sediments are contributed to the northern Gulf of Mexico primarily by two large rivers, the Mississippi and the Rio Grande, with the former being far more important. Sediments from the Mississippi River are swept westward by the littoral currents and seasonal changes, resulting in a wide dispersion pattern. Smaller rivers and streams between these major rivers flow into bays, so that the major part of their load never reaches the open gulf waters (Harding and Nowlin 1966).

100. The principal inflow of water to the Gulf of Mexico is through the Yucatan Channel. Most of the outflowing water passes directly into the North Atlantic through the Florida Straits. The most prominent feature of the surface circulation of the Gulf of Mexico is the large clockwise current in the eastern gulf. The western portion of this loop is formed by the Yucatan Current, which turns eastward and then southeastward, flowing along the western coast of Florida. The average tide range in the gulf is small, being only 1 to 2 ft at most coastal

locations on the average. Generally the type of tide in the gulf is diurnal; i.e., only one high and one low water occurring during a lunar day. Wind waves generated on the gulf are usually not large as the maximum waves encountered in this region rarely attain heights over 18 ft. The principal potential danger to the inhabitants of low-lying coastal areas surrounding the gulf is from inundation due to hurricane or storm surges. The region is susceptible to hurricanes originating in the low latitudes of the Atlantic that travel along large clockwise tracks into the gulf. Hurricanes entering the gulf generally pass through the Yucatan Channel and thus tend to maintain northerly courses. Consequently, storm surges occur more frequently along the northern than along the southern or western gulf coasts.

Eastern gulf coast region

101. Background. The eastern part of the Gulf of Mexico (Figure 24) from the Florida Keys to the Mississippi River Delta, is generally fringed by islands of varying nature. An indented swampy coastal section extends from Cape Sable in southern Florida to Cape Romano, from which point the sandy barrier beaches extend as far north as Tarpon Springs. Another reach of swampy coastal region exists to a point around the coastal bend of northwestern Florida, and then more sandy barriers extend intermittently all the way to the Mississippi River Delta. All this region is low and relatively flat; however, the average wave energy is significantly less than along the Atlantic coast because of the protection from the predominant easterly winds and because of the relatively short fetches of the occasional westerlies. Nevertheless, intense hurricanes have struck nearly all parts of this coast, including the worst hurricane in history (Hurricane Camille) in 1969.

102. Wave data summaries indicate that moderate seas exist over the eastern gulf for most of the year, with around 60 percent of the observations indicating wave heights less than 3 ft, only 15 percent of the heights greater than 5 ft, and about 1 percent with heights greater than 12 ft. Wave periods greater than 9 sec are indicated in only 6 percent of the observations, with periods shorter than 5 sec noted in

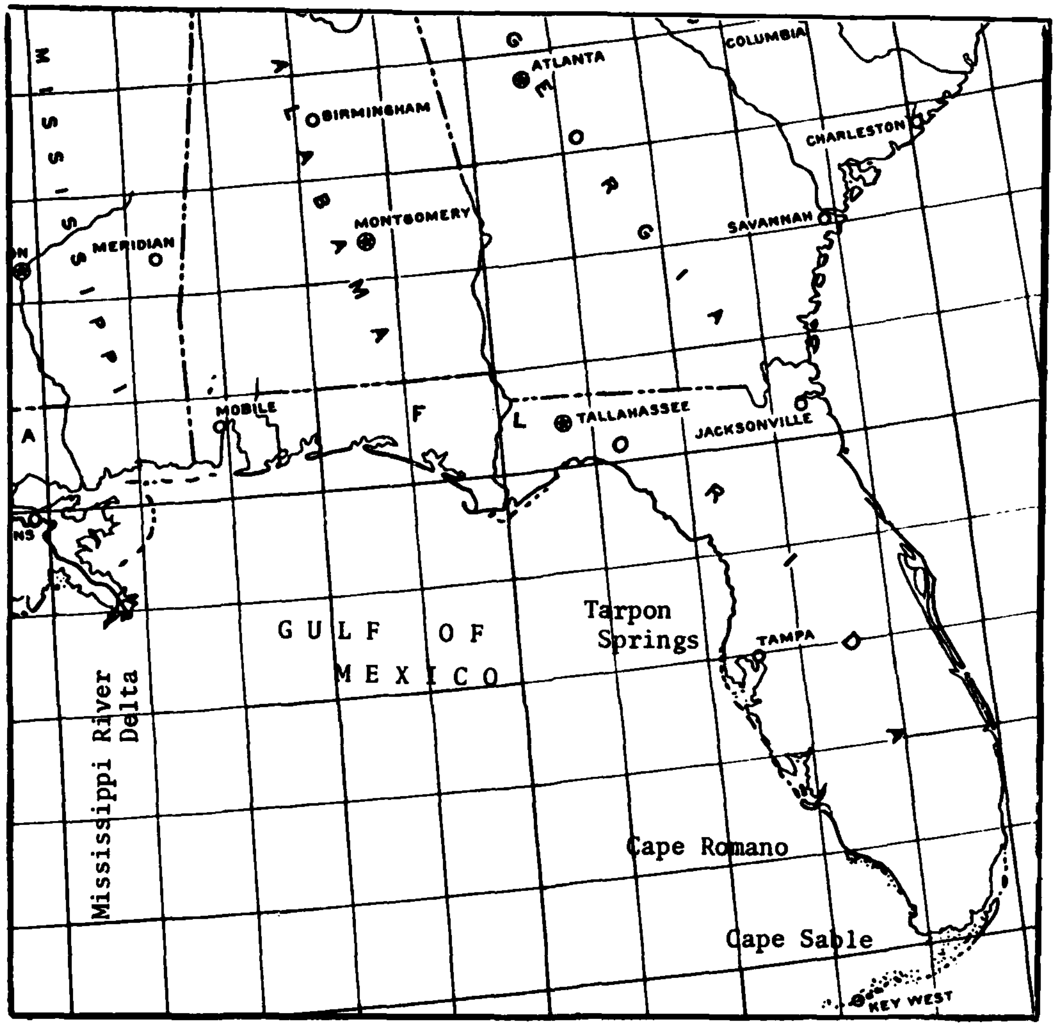


Figure 24. Eastern Gulf of Mexico

70 percent of the observations.

103. Hurricanes affecting this region form in the Atlantic or Caribbean, and there is normally advance warning in order to take precautions. The rise in water level and resulting attack by unusually high waves is responsible for much of the damage experienced during hurricanes. High winds and flooding from the storm surge account for most of the remaining damage. The probability of a tropical storm or hurricane influencing the eastern gulf coast during any given year is

about 50 percent, and the probability of two hurricanes or tropical storms during a given season is about 15 percent. As a severe storm crosses the continental shelf and moves shoreward, the mean water level may increase 15 ft or more. Hurricane Camille produced a hurricane surge of almost 25 ft above mean sea level in portions of Mississippi and Louisiana. The storm surge is superimposed on the normal tides, and wind waves are then superimposed on the surge. Currents set up along the coast by the gradient in storm surge heights can combine with the waves to weaken and undermine coastal structures.

104. Along the coast of the extreme southwestern part of Florida, mangrove growth sticks up through the marine shell layer that forms what should be a beach. Many swampy islands lie behind the red mangroves, and this semiburied forest indicates that formerly the mangrove swamp extended farther seaward than at the present. No sand or clay is available here, so pure shell deposits overlie pure peat. The entire Everglades and Ten Thousand Island coastal region developed from small reefs, which in turn supported mangrove trees, thus developing into the present swampy coastal configuration south of Cape Romano. At the cape, the coast makes a sharp bend northward, and the southerly drifting littoral material continues seaward and never reaches the mangrove swamps.

105. North of Cape Romano a long chain or narrow barrier islands have developed along the coast of central west Florida; and these islands are broken intermittently by natural passes and inlets which in practically all cases are unstructured and, except for specific dredging projects, are left to respond to the natural dynamic forces of the area. In the Homosassa Bay area, north of Tampa Bay, the offshore shielding island chain is broken by a very flat, highly irregular coastline where the rivers carry very little sediment. The resulting shoreline is a series of small islands stabilized by growing vegetation and receives very low wave energy. From Homosassa Bay north to the point where the coastline of Florida makes a very abrupt westward turn, there is no maze of small offshore islands except for the Cedar Keys area. This region has been classified as a zero-energy coastline, meaning that the waves have virtually no effect, although occasionally hurricanes or other

violent storms may produce significant changes to the littoral regime.

106. Scour problems. Several project studies are being evaluated for this area that may require jetties or other major stone structures; but in general, the low wave energy and the undeveloped nature of much of the region historically has not required such works.

107. Construction and continued maintenance of jetties and weir-jetty systems have been required along the coast of northwest Florida from Apalachicola Bay to Perdido Bay, Alabama. Rehabilitation is required as periodic inspections reveal that the continual wave attack, even though the waves are relatively mild, causes the cover and core stone to shift and settle; and this combined with foundation movement produces damage factors sufficient to require maintenance procedures to be employed. Most of this work is conducted with barge-mounted draglines.

108. During rehabilitation work where required, and on all new stone construction projects, the procedure and technique developed by the U. S. Army Engineer District, Mobile, is first to place approximately one-half of the foundation blanket material thickness, and subsequently to place a second layer comprising the second half of the blanket. The purpose for this procedure is to initially stabilize the sand bottom by reducing the amount of time that the end section of the foundation layer is exposed to wave attack; e.g., scour is always expected at the tip end of the foundation blanket and the intent is to minimize the time required to lay a specified length of blanket, usually 50 ft. Associated with this practice is the philosophy that successful contractors simply do not try to work in adverse wave conditions. The foundation blanket material usually consists of quarry-run stone which varies in size from 5 to 200 lb.

109. Similar weir-jetty systems have been constructed at Perdido Pass, Alabama (Figure 25) and at Destin East Pass, Florida (Figure 26). These are both converging, arrowhead type jetty configurations, but the weir section is in the east jetty at Perdido Pass and is located in the west jetty at East Pass. The wave climate of the region is such that there is believed to be a reversal with season in the direction of littoral transport, and the direction and magnitude of the net annual drift are subject to considerable debate. The consensus of opinion in

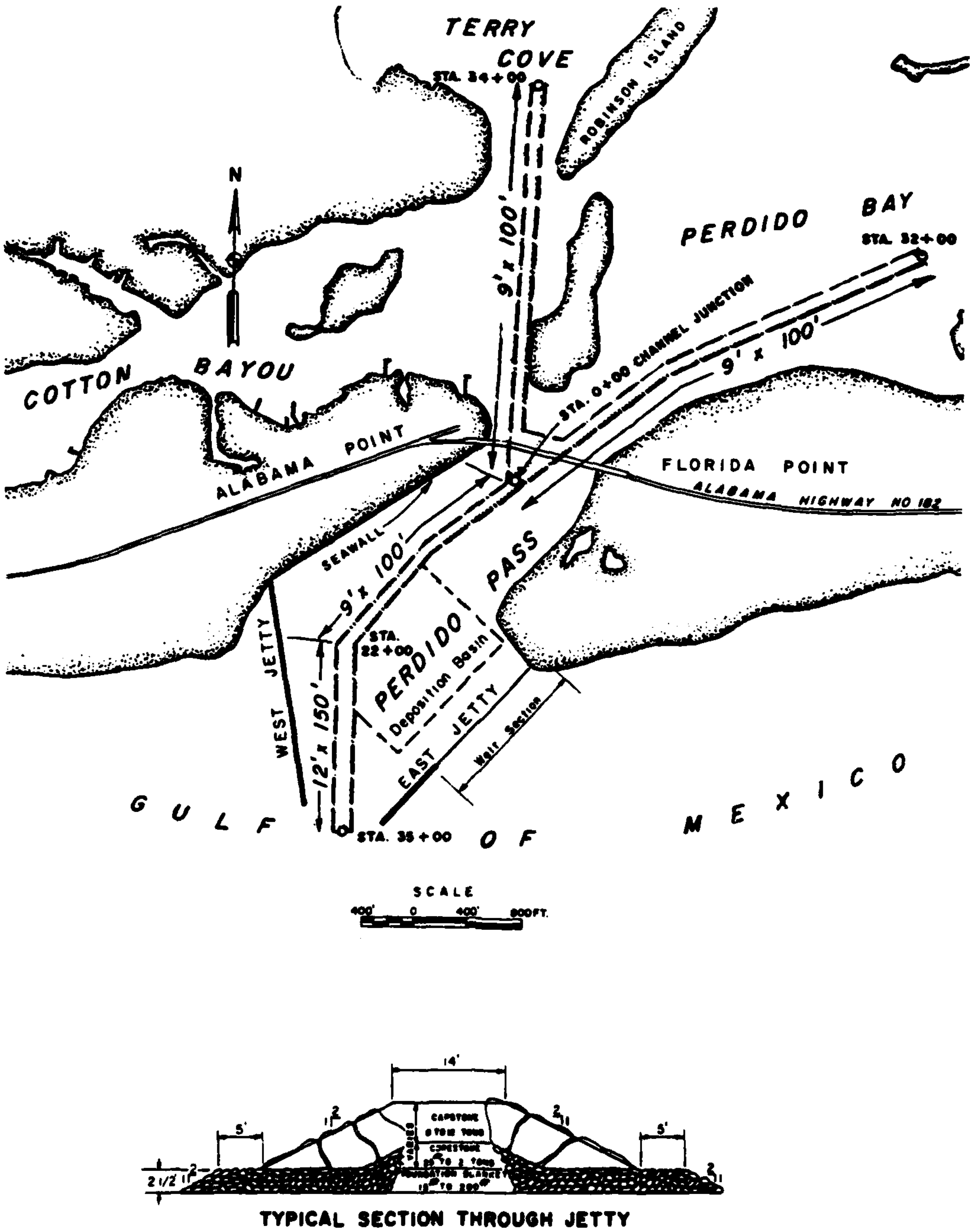


Figure 25. Perdido Pass, Alabama, jetty alignment and typical jetty cross section

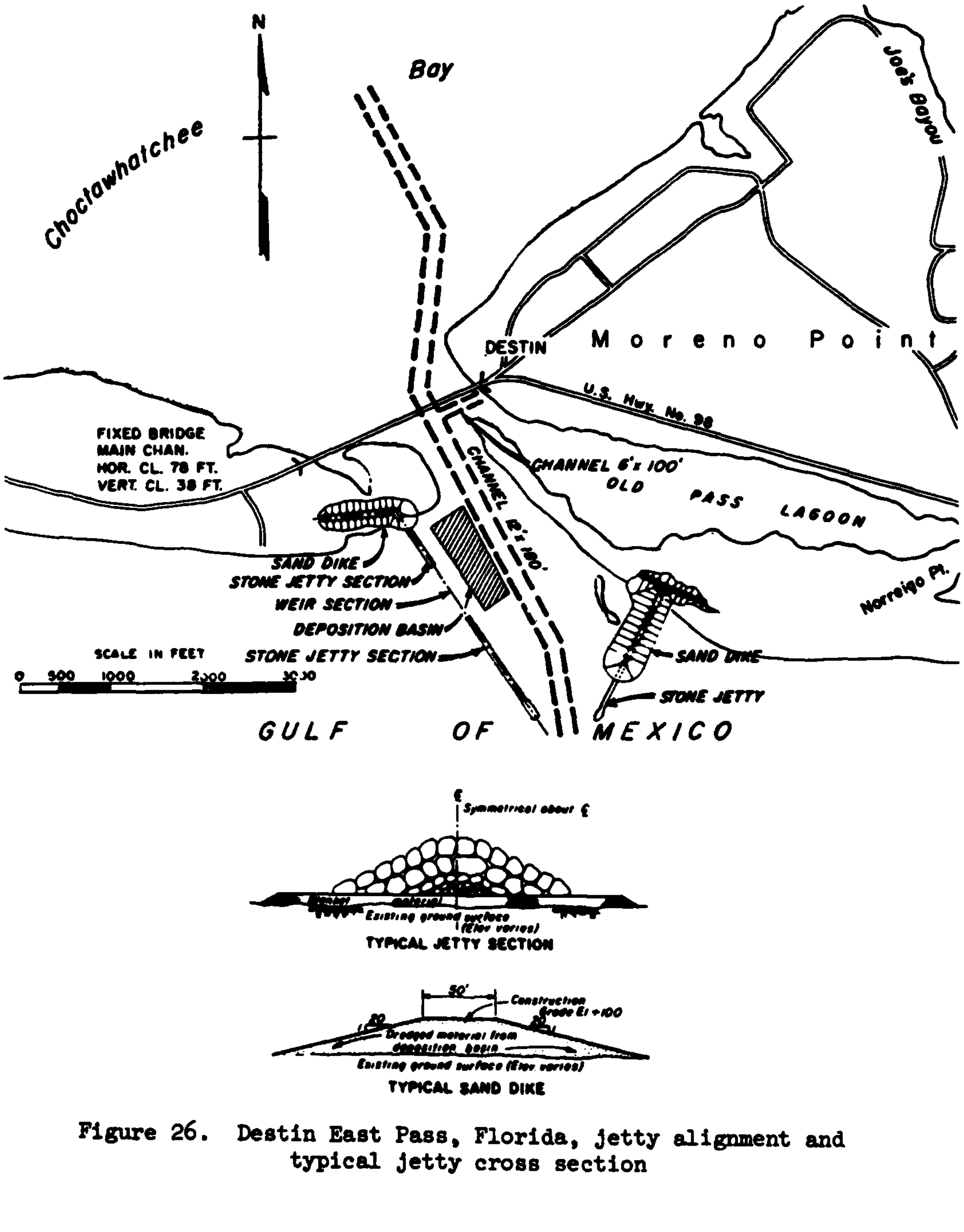


Figure 26. Destin East Pass, Florida, jetty alignment and typical jetty cross section

the engineering community is that the direction of net transport, on an annual basis, is probably west at Perdido Pass and is probably east at East Pass. Hence, the purpose of the weir location is to intercept a significant portion of the net transport and prevent it from penetrating the navigation channel, and the converging jetty alignment is designed to permit flow of littoral material around the end of the jetties (which does not pass over the weir). That sand which passes over the weir section is intended to be collected in a deposition basin so the dredge can operate in a relatively calm area.

110. No unusual scour problems occurred during construction of the jetty system at Perdido Pass. However, problems of a significant nature existed at Destin East Pass; and they may be attributed in part to scour and erosion effects and in part to improper construction techniques on the part of the contractor. The weir design specifications required the erection of concrete sheet piles driven 8 ft into the foundation and stabilized by massive timber wales that were bolted together. Construction problems began to occur when it appeared some of the connecting bolts were not threaded enough to provide a rigid member. Waves breaking onto the sheet piling caused separation of some of the piles sufficient to allow currents and littoral material to pass, and scour holes up to 14 ft deep developed at the weir. Ebb currents on the order of 6 to 10 fps were not uncommon. Excessive torque developed in the wales, bolts sheared, and 100 ft of the weir section collapsed into the scour hole. The damaged section of the weir jetty was repaired by construction of a rubble-mound section instead of the originally designed concrete sheet piling.

111. Desirable features of the stone-type structure include the fact that small portions of the structure may be placed individually instead of large portions with a large surface area on which wave forces can be expended. The uneven rubble mound assists in dissipation of wave energy, and the elevation of the crest of the weir can be "tuned"; i.e., adjustments to elevation may be easily and relatively quickly performed during construction should it become apparent that the weir is not performing as anticipated.

112. Dredging of the deposition basins at these two locations has not been performed as originally envisioned for a number of reasons. First, the priority dredging requirements apply to keeping the navigation channel open; and then if time and funds are available, the sediment accumulation in the deposition basin will be removed. Another factor which enters into the decision-making process is the fact that local landowners simply do not wish to have dredged material deposited on their downdrift beaches. Reasons for this may be connected to the fact that in Alabama and Florida, when a beach is nourished by artificial fill, the newly formed land area becomes State property, whereas if the beach widening is by natural processes the increased area remains private property. Many landowners would thus prefer to wait for natural forces to potentially nourish their beaches.

113. The St. George Island Channel project has experienced secondary construction effects in that the stabilizing jetties do not extend all the way through the island into Apalachicola Bay. The resulting scour and erosion occurred when wave energy propagated through the jetties and agitated the unprotected island areas. Fairly strong ebb currents were adequate to transport the material out of the inlet. A similar phenomenon has also been observed at several Great Lake harbors; however, in that region it is not due to tidal-generated ebb or flood currents, but currents due to long-period wave energy which excites one or more modes of oscillation.

Louisiana coastal region

114. Background. Coastal problems of the State of Louisiana are so distinctly different from those of other regions that they merit special investigations and unique solution processes (Figure 27). This is the region which has produced the greatest change in planform in recent times, and these changes are directly the result of the Mississippi River. This is one of the world's largest river systems and transports a tremendous sediment load. When this huge river reaches the Gulf of Mexico, the sediments carried as suspended and bed load are deposited. Much of this deposition takes place in the form of a bar at the mouth of the river, but many of the fine materials in suspension are swept away

Figure 27. Louisiana coastal region

by the gulf currents and carried many miles before settling out. The delta area thus being formed extends into the Gulf of Mexico beyond the general coastline. The older deltas on either side of the present Mississippi River outlets are deeply embayed, and sand barrier islands have formed along parts of their margins. Marshy and swampy areas cover much of this coastal region and the materials are generally highly unconsolidated. In addition to the obvious delta regions, coastal Louisiana has some 75 miles of smooth, gently curving shoreline west of the deltaic complex known as the Chenier plain. Two necessary requirements for the development of the Chenier area are (a) proximity to a large delta where great quantities of clay and silt are introduced into the gulf, and (b) littoral currents that distribute much of the clay along the coast in the direction of downdrift from the delta.

115. Louisiana's coastline is a region where land and sea are intricately intertwined. The effect of the many bays and coastal inlets is a tidal shoreline of marsh grass and unstable deposits of mud and silt with minimal human habitation of roughly a 30-mile strip of land parallel with the gulf. This sparce population is one reason for the absence of a larger number of coastal engineering works. The major structures that do exist are for the primary purpose of stabilizing navigation entrances for commercial transportation to the more populated inland cities.

116. The most unstable part of the Louisiana coast is the stretch from the shores of Lake Borgne to Timbalier Bay, as the shore is a mixture of green marsh grass and water. From Atchafalaya Bay to Vermilion Bay, the shore is swampy but more solid. Atchafalaya Bay seems to be decreasing in volume as an active delta building process is occurring in the bay. Beyond Marsh Island, the shore becomes smooth and regular, curving gently to the Texas border at the mouth of the Sabine River. The swampland in this region gives way to some stretches of firm beach, although it is not hard-packed quartz or coral, but one closely related to the swamplands of the east. Because of the inherent nature of the entire region, structural erosion and many rehabilitation requirements can be all-inclusively considered as "scour-settlement" problems.

117. Scour problems. Maintenance of sufficient navigation depths on the Mississippi River from Baton Rouge to the gulf is an undertaking of major importance. The Port of New Orleans, which is about 95 miles above the Head of Passes on the Mississippi River, is the second largest port in the United States in waterborne commerce. Authorized channel dimensions from New Orleans to the gulf are: New Orleans to Head of Passes, 40 ft by 1,000 ft; Southwest Pass, 40 ft by 800 ft; Southwest Pass bar channel, 40 ft by 600 ft; South Pass, 30 ft by 450 ft; and South Pass bar channel, 30 ft by 600 ft. A seaway canal, the Mississippi River Gulf Outlet (MRGO), from New Orleans to the gulf along a shorter route has also been constructed with authorized dimensions of 36 ft by 500 ft. All these passes require extensive and significant structural

measures for their stability and functional effectiveness, including headland dikes, spur dikes, bulkheads, revetments, and jetties.

118. Additional outlets south of Venice, Louisiana, will be provided by enlargment of the existing channels of Baptiste Collette Bayou and Grand and Tiger Passes. Channel dimensions will be 14 ft by 150 ft except for entrance channels which will be 16 ft by 250 ft. Jetties to reduce the cost of maintenance dredging will be constructed to the -6 ft contour to prevent silt from settling into the navigation channel from lateral directions. The purpose of these additional outlets is to provide navigation access between Venice and the adjacent areas of the Gulf of Mexico, and to provide a shorter navigation route between the east and west waters of the gulf. All navigation will realize considerable savings in transportation cost using these channels rather than South or Southwest Pass of the Mississippi River. Commercial and sport fishermen alike will derive similar time and distance benefits.

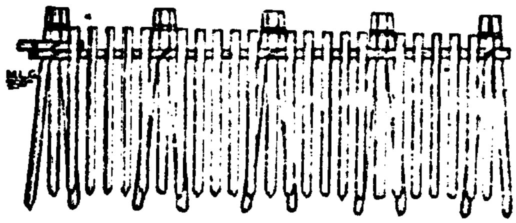
119. The structure-foundation problems of the U. S. Army Engineer District, New Orleans, are in many cases associated with current and wave conditions occurring at the time of construction. In other situations, unidirectional river currents combined with ship waves, and occurring over long periods of time, gradually undermine many stabilizing structures and extensive rehabilitation is necessary. The District is actively engaged in experimental programs of significant magnitude to evaluate different scour control and foundation settlement reduction techniques. Among these are foreshore revetment materials that have not previously been used and combinations of filter fabrics and placement of filter fabrics at different locations within rock structures. To ascertain the effectiveness of these prototype experimental studies, the District will conduct annual surveys on all foreshore rock work presently under construction or which will be constructed in the future where there are test sections of different foundation techniques. Profiles and cross sections of this and all offshore jetties also will be obtained.

120. Foreshore dikes are constructed in conjunction with revetment work to prevent runoff of rainfall from scouring streambanks. Headland dikes are required at the Head of Passes to ensure separation of the

flow channels. Both these structures are subject to waves generated by tows and ships that break over the top of the dikes. Scour occurs behind the dikes, and the filter material composed of shell bedding or limestone rock keeps settling. Construction techniques using different placement of filter fabrics and limestone rock sizes are being evaluated. Settlement of foreshore dikes is not uniform in most cases, and it is desired to slow the settlement rate by applying a layer of synthetic filter fabric with optimum size limestone rock.

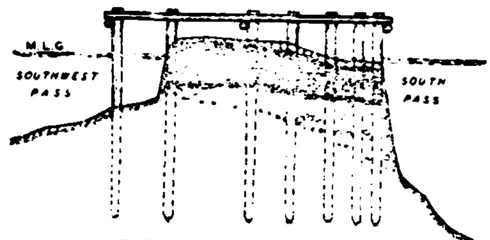
121. Along the river passes, bulkheads parallel to the channel banks and permeable timber pile spur dikes are constructed to stabilize the navigation channel, and to assist in keeping the channel self-scouring. The nature of the function of these structures is conducive to uncontrollable scour and erosion. Scour around vertical piles in unidirectional flow has been the subject of much investigation, and while the phenomena are probably related to shedding of eddies and vortices, the problem still is not sufficiently well understood to confidently provide design guidance with a high probability of success. Likewise, the vertical impermeable bulkheads are almost total reflectors so that a 3-ft wave develops a clapotis of 6 ft, and scouring of the sand and silt foundation underneath the bulkhead is accelerated by the clapotis effect. Rehabilitation of these structures is a continuing feature of the operation and maintenance work in this region. Present design and construction technqiues are not entirely satisfactory, and research aimed at developing improved construction techniques would be quite valuable. Typical sections of these aforementioned structures are shown in Figure 28.

122. East Timbalier Island is an area of much concern by various private interests. The stability efforts comprised of rock placement along the shoreline is being conducted by private interests to ensure the safety of extensive machinery on the island and is not really for beach building purposes. Massive concrete blocks and huge quantities of rock have been placed on relatively firm sand in the surf zone; but because of the turbulence associated with breaking waves, sand is tossed into suspension and transported out of the area, allowing settlement to



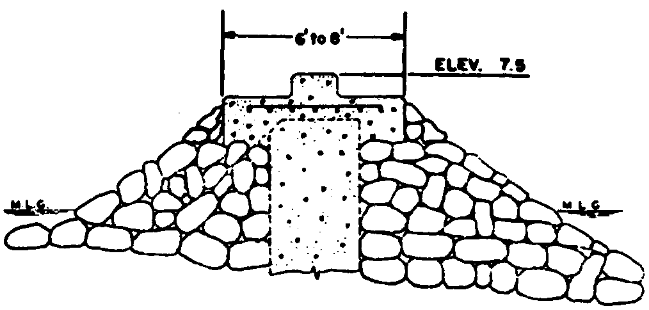
TYPICAL SPUR DIKE
MISSISSIPPI RIVER
VICINITY MILE 2 (AHP)

a.



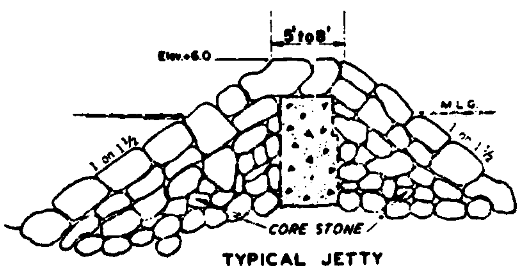
TYPICAL CROSS SECTION
HEADLAND DIKES

b.



TYPICAL CROSS SECTION
INNER EAST JETTY
SOUTH PASS

c.



TYPICAL JETTY
CROSS SECTION
SOUTHWEST PASS

d.

Figure 28. Typical sections of structures along the Mississippi River passes

occur. Over a period of months, this operation has taken on the atmosphere of a low-intensity, seemingly never-to-end effort, as the contractors are paid for quantity of rock dumped into the scour areas.

123. At the Sabine River outlet along the Louisiana-Texas border, vertical bulkheads have been constructed for stabilization of the narrow navigation channel in order to provide a relatively self-scouring and self-maintaining condition. Waves propagating between the parallel jetties have reflected off the bulkheads and caused scour to such an extent that some sections have collapsed. Expensive rehabilitation has been conducted with rock being placed in front of the reconstructed sections.

124. Grand Isle is located on the gulf, and is one of the many low, irregular islands separated by bays, lagoons, and bayous that form such a large part of the shoreline of Louisiana. It is a base of operation for large offshore petroleum and sulphur industries and is a commercial fishing and sport fishing center. Because of Grand Isle's location and topography, improvements on the island are subject to damage from erosion along its gulf shore and from the combined effect of winds and surges generated by hurricanes. In recent years, severe scour has occurred on the west end of the island. The first effort to solve this problem was construction of vertical sheet-steel pile bulkheads to stabilize the location of the shoreline. These reflecting walls agitated the situation and the scour and erosion became much more severe. Finally, a rock jetty structure was built on the west end of the island which has minimized the excessive erosion.

125. Baptiste Collette Bayou is located east of Venice, Louisiana, and extends from the Mississippi River to Breton Sound. The navigation channel had been previously constructed and the work submitted for bids in late 1978 consisted of furnishing and placing approximately 41,000 cu yd of shell and approximately 83,000 tons of stone at specified marine locations in Baptiste Collette Bayou, including the designated test areas where the addition of approximately 15,000 sq yd of synthetic filter fabric was to be placed for test purposes. Soil borings of the area indicated that the foundation is almost entirely gray, soft to very

soft, oxidized, fat, inorganic clay of high plasticity. Six different tests are being conducted in four different test sections, two sections in each of the east and west jetties. The configurations of the elements that constitute the construction features of the two jetties are shown in Figure 29, as well as a typical cross section of the entire navigation channel and jetty system. The purpose of this experimental prototype testing is to determine which combination of filter fabric, location of filter fabric, and stone sizes successfully and significantly reduces the amount of settlement into the relatively unconsolidated foundation. Jetty profiles and cross sections are shown in Figures 30 and 31 for the east and west jetty, respectively.

126. Technical specifications for the filter fabric are essentially those given in paragraph 79, with the exception of the variation in tensile strength and equivalent opening size. Requirements of these parameters for the test sections were:

Stations	Equivalent Opening Size	Tensile Strength
	West Jetty	
360+00 to 365+00	70	200 lb any direction
385+00 to 387+50	70	200 lb any direction
387+50 to 390+00 (nonwoven)	70	200 lb any direction
	East Jetty	
360+00 to 365+00	100	400 lb any direction
403+00 to 405+50	100	400 lb any direction
405+50 to 408+00	35	1000 lb any direction

The physical strength requirements are as previously noted. The material was placed in such a manner as to avoid defects or damage during installation.

127. The shell noted on the cross sections of Figure 29 was either live or dead clams, reef shell, or cannery shell, and was free of sticks, mud, or other foreign matter. The three stone sizes of Figures 29, 30, and 31 conformed to the following:

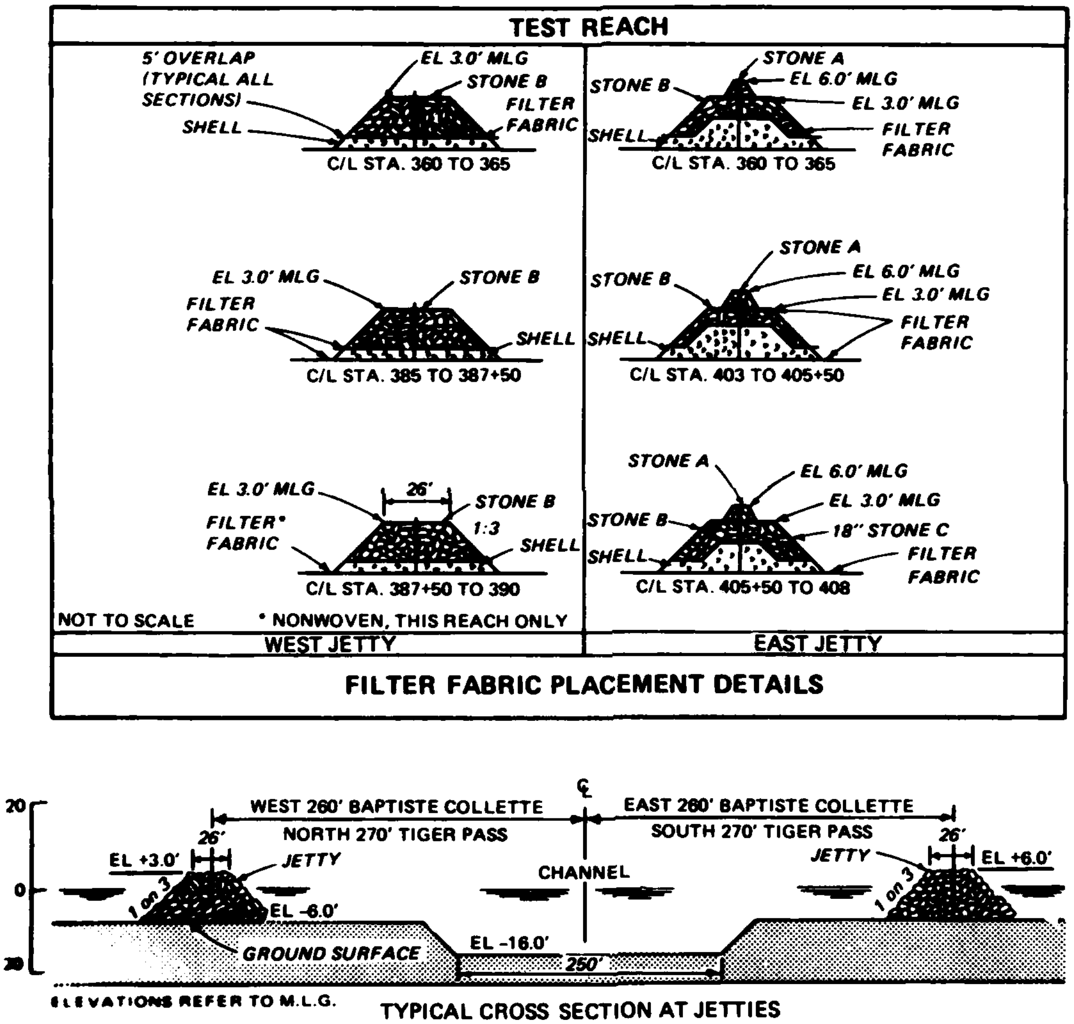


Figure 29. Baptiste Collette Bayou, experimental prototype tests

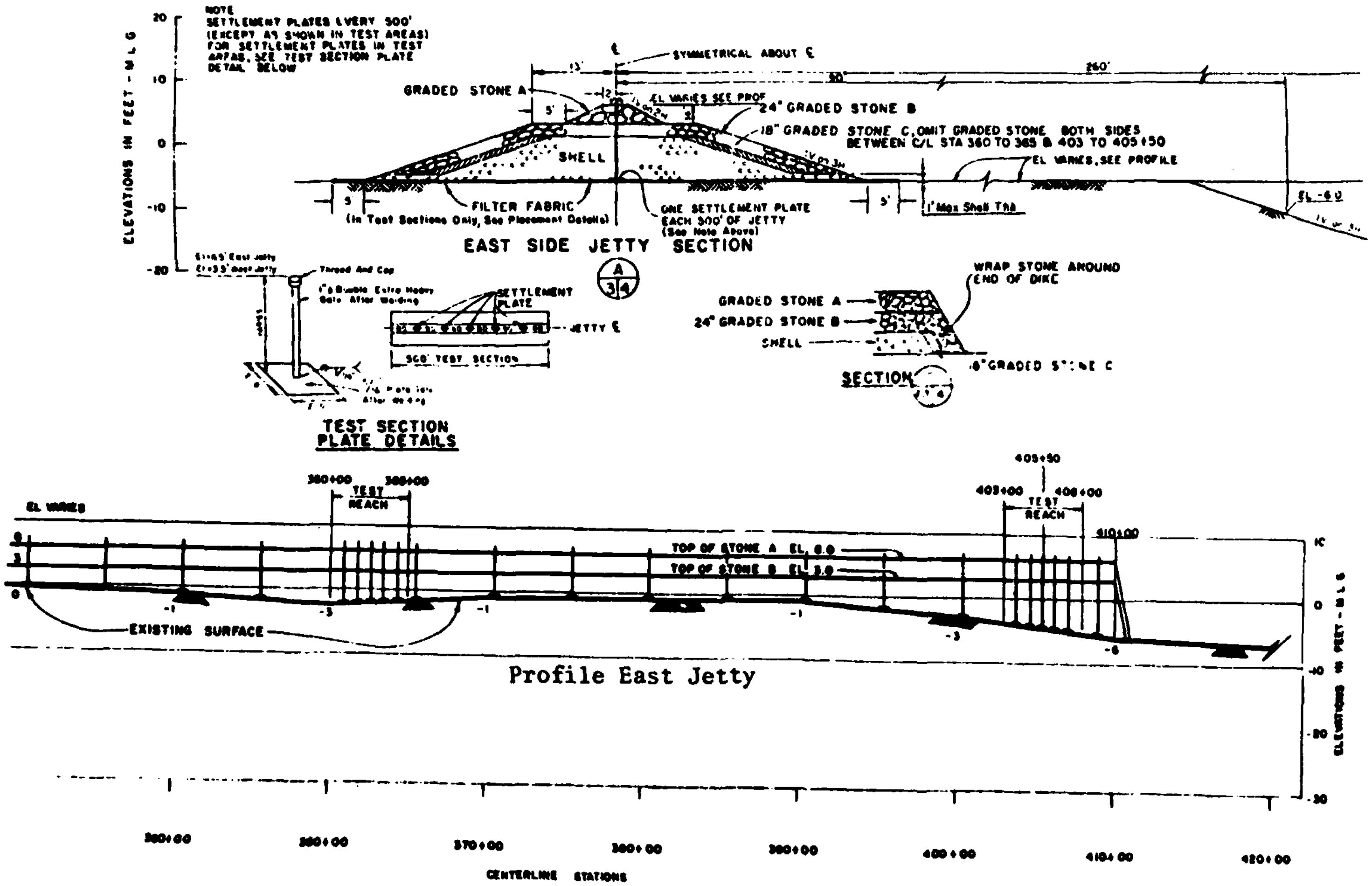


Figure 30. Baptiste Collette Bayou, east jetty test sections

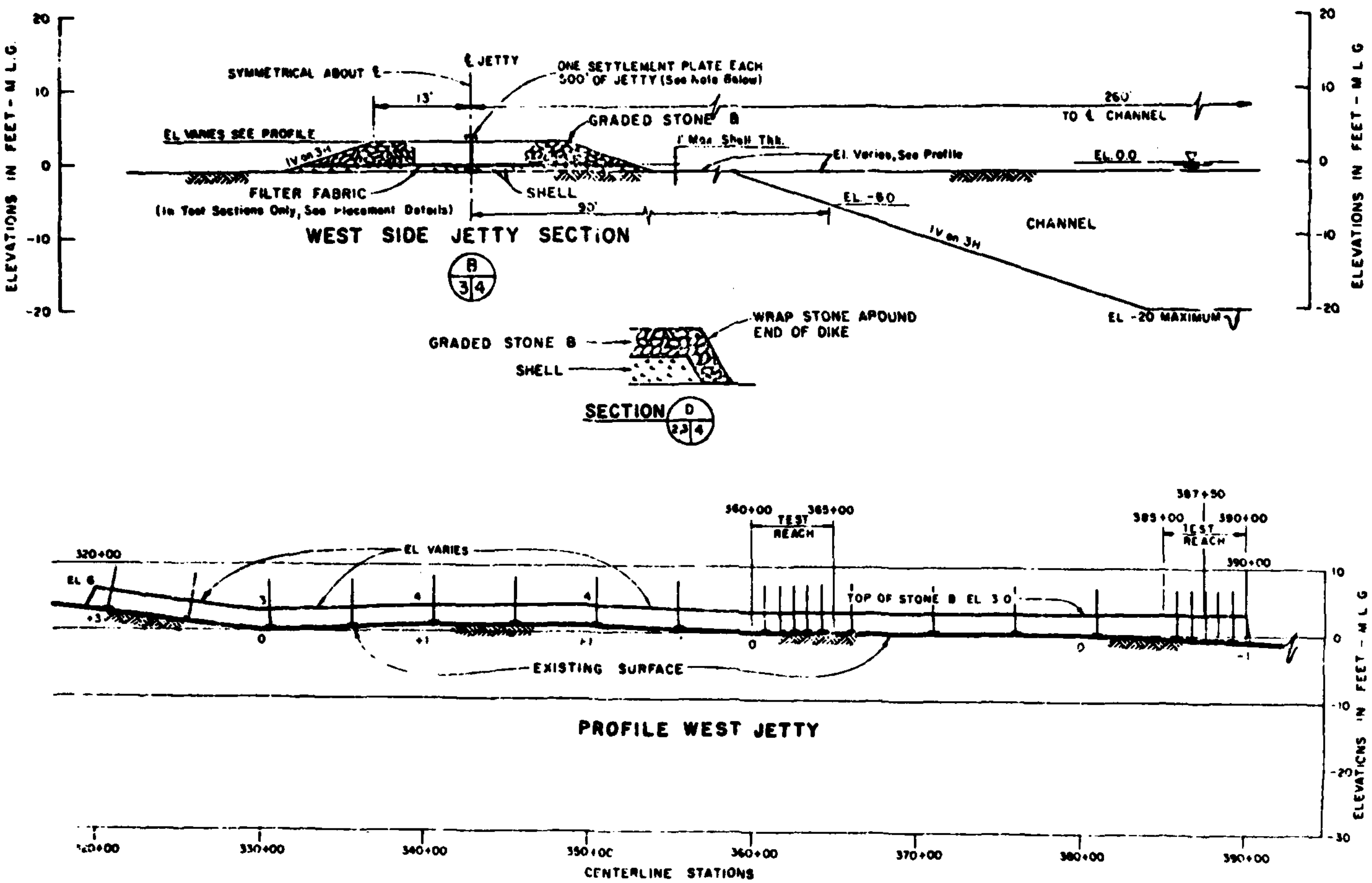


Figure 31. Baptiste Collette Bayou, west jetty test sections

Stone Weight, lb	Cumulative percent Finer by Weight
Stone A Gradation	
5000	100
2500	70-100
500	40-65
100	20-45
5	0-15
1	0-5
Stone B Gradation	
1200	100
750	72-100
200	40-65
50	20-38
10	5-22
5	0-15
1	0-5
Stone C Gradation	
400	100
250	70-100
100	50-80
30	32-58
5	15-34
1	2-20
less than 1/2 in. (max size)	0-5

All stone was placed by 2-1/2-cu-yd dragline bucket.

128. The largest overrun of quantities was experienced for the main armor protection of Graded Stone B at Baptiste Collette Bayou and Grand and Tiger Passes which was 2 ft thick on the typical cross sections shown on the plans. The specifications allowed for a 12-in. tolerance in placing stone beyond the cross section. This alone could account for a 50 percent increase in the Stone B quantities. Detailed records of settlement during construction are not available; however, field inspection indicated that the jetty exceeded the minimum elevation in several reaches, but also experienced significant subsidence up to 7 ft in some reaches. It is the opinion of the New Orleans District that portions of the overrun may be attributed to rock tolerance

specifications. The large armor rocks could have caused settlement although no data are available from settlement plates during construction. In addition to the test sections shown on the profiles of the jetties, settlement plates had been installed in the center line of both jetties approximately every 500 ft for the purpose of accurately determining the effectiveness of each variation in test section of the Baptiste Collette Bayou experimental prototype jetty investigations.

129. Erosion protection had long been needed at Holly Beach, Louisiana, and in 1970 the State Highway Department constructed a 3-mile revetment project at this location to stabilize a section of Louisiana Highway 82 which lies directly adjacent to the beach. The importance of this coastal highway which connects Louisiana and Texas is related to the fact that it serves as a hurricane evacuation route. This highway had been seriously damaged in the 1950's and 1960's by winter storms passing directly over the area, with wind-driven water from the gulf overtopping the highway which is approximately 7 ft above mean sea level. Major storms in this region are capable of generating significant waves at the coastline of 3 to 4 ft with maximum waves approaching 6 ft. Wave periods are estimated to be from 5 to 8 sec. This section of the State highway is only 50 ft from the Gulf of Mexico.

130. A 200-ft portion of the revetment project was constructed of Gobi blocks for test purposes (Figure 32). This prototype test was divided into two reaches each 100 ft long. Both sections consisted of hand-placed blocks and differed only in the type of filter cloth used in the underlayer. The Gobi blocks are approximately 4 in. by 8 in. by 8 in., weigh 13-1/2 lb each, and have an open area of 35 percent. For a design wave of 3.6 ft, and placed on a 1V-on-3H slope, the 13-lb Gobi block provides a stability coefficient exceeding 50, using the Hudson (1974) approach. This appears to be greater than other known methods of erosion protection, as it corresponds to a W_{50} size of riprap stone which varies from 150 to 360 lb.

131. The prototype test section was put through a very severe test only two weeks after completion when a winter storm inundated the highway and caused extensive damage along parts of the road. Prior to

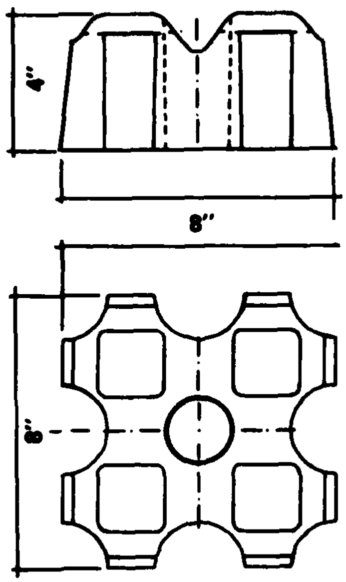


Figure 32. Gobi block typical dimensions and configuration

placement of the blocks, the filter cloth had been anchored by excavating a trench to the -3 ft mean low gulf elevation, which is approximately 2 ft below mean sea level. The bottom row of Gobi blocks had been set against a 3-in. by 12-in. timber header. Currents running parallel to the shore eroded sand at the toe but did not displace any of the blocks. The experience at Holly Beach clearly proved that a filter cloth used in conjunction with Gobi blocks in a structure subject to wave attack should have a high retention of the local soil while having a high ratio of open area in relation to the bank material; i.e., the percent open area should exceed the porosity of the bank material to relieve hydrostatic pressure. A secondary filter of coarse beach sand and shell fragments formed under the filter cloth which prevented the very fine sand from leaching out, but there was no clogging of the filter cloth at this installation.

132. Success in this initial effort led to successive installations along with pertinent fundamental research of the subject by McCartney and Ahrens (1975). The Gobimat system of scour and erosion control is now commercially available, and consists of various sizes of

custom-designed concrete Gobi blocks bonded to a hydraulic filter cloth that can be hoisted very simply by a small crane and laid like a blanket on any location where scour is a problem. This, in many cases, can eliminate the necessity for bulkhead construction and the riprap necessarily associated with bulkheads. When placed above the water line on revetment areas, the mattress will be completely covered with natural or planted grass within a few weeks. While Gobimat was developed in the Netherlands and Louisiana to combat the unique scour problems inherent to delta country, the system is now being widely used by oil companies, state and local municipalities, and levee boards.

Western gulf coast region

133. Background. West of the Sabine Pass, the shoreline changes relatively abruptly from desolate-looking swamps to that of beach areas adjacent to flat grasslands, stretching to the north as far as the eye can see. Beach sands become firmer and take on a whiter tint; and the long, thin chain of barrier islands that border approximately 320 miles of the Texas coast commence along Galveston East Bay (Figure 33). Most of the barrier islands along the Texas coast are wider than those along other coasts, having widths of 5 miles or more in several places. The islands form an interconnecting series of bays and lagoons linked to a few large estuaries that extend for most of the 400-mile length of the coast. Laguna Madre, on the southern shore adjacent to Padre Island, contains salinities well above the open ocean. This is a result of heavy evaporation and a scarcity of streams entering all except the southern end of Laguna Madre.

134. Galveston Bay is the largest and most important estuary along the Texas coast because it reaches almost to Houston, the largest city in the South. The city of Galveston, however, is the only major center directly fronting on the gulf. Farther south, Corpus Christi Bay extends inland to the city of Corpus Christi. This is the deepest bay on the Texas coast and is blocked by the barrier Mustang Island, with no direct pass into the gulf, and only one entering river.

135. Hurricanes have had significant effects on development of the Texas coast. The greatest hurricane disaster in United States

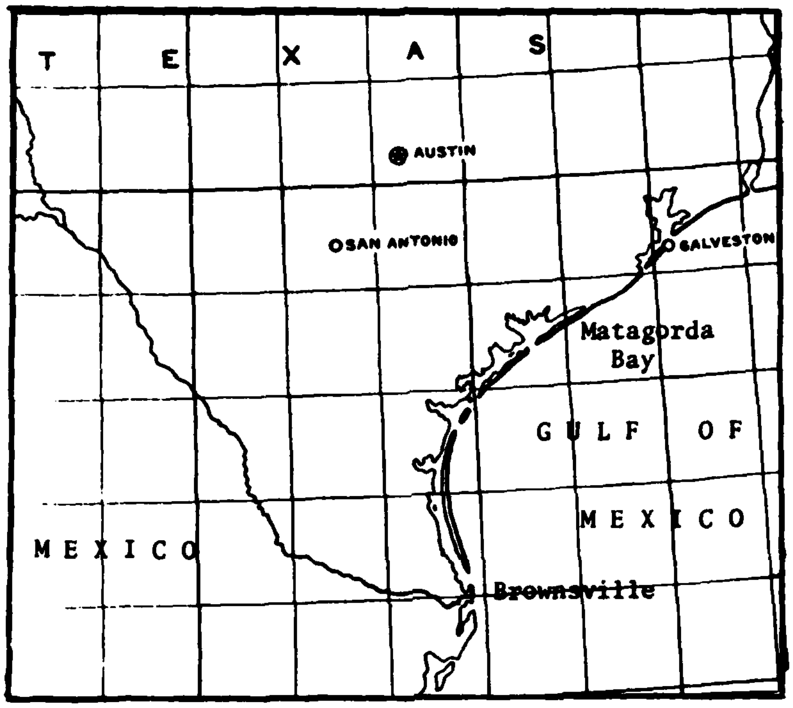


Figure 33. Western Gulf of Mexico

history occurred at Galveston in 1900 with 15-ft-high tides and winds in excess of 120 mph. All parts of the island were flooded and 6,000 people were killed. The hurricane cut the gulf shoreline back several hundred feet, and much of the eroded sand formed massive offshore bars; however, most of this material was shifted back to the beach within a year following the hurricane (Shepard and Wanless 1971).

136. The prevailing winds along the Texas shore throughout most of the year are from the south and southeast. From the Louisiana border, the coastline extends generally southwest to the coastal bend near Corpus Christi, and from that point turns generally south to the Mexican border. Because of this shoreline configuration, wind waves produce a net littoral transport from northeast to southwest along the upper coast, and from south to north along the lower coast, although changes in wind directions reverse the direction of littoral transport for short periods of time.

137. The general littoral movements of beach and shore material

along the gulf are interrupted both by artificial structures and by tidal currents through passes between the gulf and the inland bays. Longshore transport accumulates in the navigation channel entrances and must be removed periodically by hopper dredge.

138. Other factors affecting the rate and volume of littoral transport include the character of the sediments carried to the gulf by the major rivers along the coast and the temporal strength of the longshore current affecting movement of material in the littoral zone. Along the upper coast, the Sabine, Neches, and Trinity Rivers carry mostly fine sediments to the coast and do not supply significant volumes of sands to the beaches. The Brazos, Colorado, and intervening rivers southward to the Rio Grande carry larger percentages of sandy materials and, during flood periods, contribute considerable sand to the gulf and its shore processes. In the coastal bend area between Pass Cavallo and Arkansas Pass, littoral transport is subject to reversal more frequently than along the remainder of the coast, as this is probably near a nodal point.

139. The physiography of the Texas coastline is such that a wide continental shelf exists along the northern sections of the State, decreasing in width in a southerly direction. The effect of this variation in shelf topography is to allow the deepwater wave energy to propagate closer to the coastline in a relatively undiminished state. At the same time, the sediments comprising the foundation material in the northern parts of the State are much finer and more unconsolidated than the sandier deposits that exist farther south. These situations are compensating to the extent that while the scouring may be less in the northern regions of the State, the displacement due to settlement will probably be greater.

140. <u>Scour problems.</u> The Galveston District has found from many years of experience that the additional materials needed to provide final grade to massive rock structures is the result of a combination of "displacement and scour." On the average, the determination of this combination from historical records has provided the approximation factor to estimate quantities of structure material

along the Texas coast without the necessity of decomposing the material overruns from past work into "displacement effects" and "scour effects." The application of this "displacement and scour" adjustment factor is then applied to the template estimates.

141. The procedure developed for this region is to compute the quantities of material to be advertised for bids as that which would exist above a hypothetical groundline uniformly 3 ft below the actual groundline, and extending from the surf zone to the end of the structure in a seaward direction. Because this additional 3 ft of height will require the structure side slopes to generate a wider base, all quantities will theoretically be affected. Hence, the core stone and armor stone estimates are increased accordingly. It has been learned that, when working conditions are good, the rate of construction progress will be rapid enough so that the amount of scour or displacement in front of the structure will not exceed 3 ft. However, when wave conditions are inclement or contractor construction procedure contributes to less than satisfactory rates of progress, there may be sufficient time for larger scour holes to develop.

142. Foundation blanket stone is always designed for placement under stone structures for bearing and scour control. The blanket is composed of quarry-run stone, reasonably well graded and varying in size from 1/2 in. to 200 lb. The thickness of the blanket is designed to be 3 ft under the trunk section of the jetties or breakwaters, increasing in thickness to 5 ft under the head section.

143. There has been no experience with using synthetic filter fabric on the open coast by the Galveston District. Filter cloth has been used, however, in Galveston Bay during the construction of a marsh habitat as a part of the Dredged Material Research Program (DMRP) by Allen et al. (1978). Dike construction was predicated on the use of sand-filled bags, and scour holes developed under some sections of the dike and at the end section. The sediments were very fine, and a combination of wind and ship waves from the Houston Ship Channel interacted with tidal currents to cause bottom material to move. As the open section of the dike was being closed, scour occurred and a portion of the bag

construction collapsed. Initial placement was conducted in the absence of filter material for foundation stabilization; however, cloth filter fabric was introduced in an attempt to minimize further deterioration of the dike. This procedure was found to be highly successful from a scour control standpoint, although the physical placement of the filter fabric on the bottom in 3 ft of water was accomplished with some considerable degree of difficulty. The filter cloth did not solve all problems, but it is believed to have helped substantially in reducing the foundation scour.

144. Since the wave climate in the Gulf of Mexico is relatively mild, physical model tests for structural stability of major stone works are not ordinarily considered necessary. Guidance obtained from years of construction experience, and published literature, has proven adequate, although some of the older structures have developed internal cavities over the years that necessitate rehabilitation. Some littoral material probably penetrates these structures, but distinct shoal regions in the navigation channels are not discernible.

145. The basic underlying philosophy of the region concerning the dredging of the navigation channels is to balance the dredging requirements with the littoral drift transport rates to maintain open channels and reduce downdrift beach erosion. One avenue of approach is to rehabilitate the existing structures to a degree sufficient to preclude penetration of the jetties by any littoral material, thus requiring the littoral drift to form a fillet on the updrift side of the jetty and travel around the tip into deeper water. While the navigation channel would remain essentially open, any required dredging would necessarily have to be performed in a rough wave climate. A second approach is to permit littoral material to penetrate the jetties into the navigation channel, and remove the shoal region by dredging in the relatively calm waters which are protected by the jetties. Rehabilitation of existing structures is not nearly so critical from an erosion and scour control standpoint as is construction of a major new stone structure. Foundation blanket material will be lost, however, if it is not rapidly covered

with core stone. This is a construction problem more than a design problem.

146. The construction of major stone jetties is being considered at three locations along the Texas gulf coast: (a) two new parallel jetties to stabilize the mouth of the Colorado River, and to be operated in conjunction with a weir section located in the east jetty; (b) removal of the north jetty at Freeport Harbor, widening of the navigation entrance channel, and construction of a new north jetty; and (c) a major rehabilitation and extension of the north jetty at Brazos Island Harbor.

147. The design features of the jetties proposed at the mouth of the Colorado River are shown in Figure 34, where the 3-ft foundation blanket stone, toe protection features, and the assumed 3 ft of scour and/or displacement can be noted. Although the concept of the weir jetty is relatively new, and experience with existing weir jetty installations may be inconclusive according to Seabergh and Lane (in preparation), it has apparent advantages in initial cost, and maintenance dredging can be accomplished by pipeline dredge.

South Pacific Region

148. The coastline of the State of California can effectively be considered as an entity, although the physiographic and climatological characteristics begin to change north of Cape Mendocino, approximately 100 miles south of the California-Oregon border. In this northern extremity, the dry summers come to an end and the annual rainfall increases to around 10 times that which falls in San Diego, and the mountainous coast becomes covered with a heavy growth of trees.

149. Along the Pacific coast of the United States, there are problems quite unlike those that dominate the Atlantic and gulf regions. The shoreline is largely cliffed coasts with rocky promontories, and cliff erosion which was practically unknown in other localities becomes a significant source of beach material. At the same time, a problem of at least the same magnitude of severity is that of the effects of man-made structures on the littoral regime. The major natural harbor

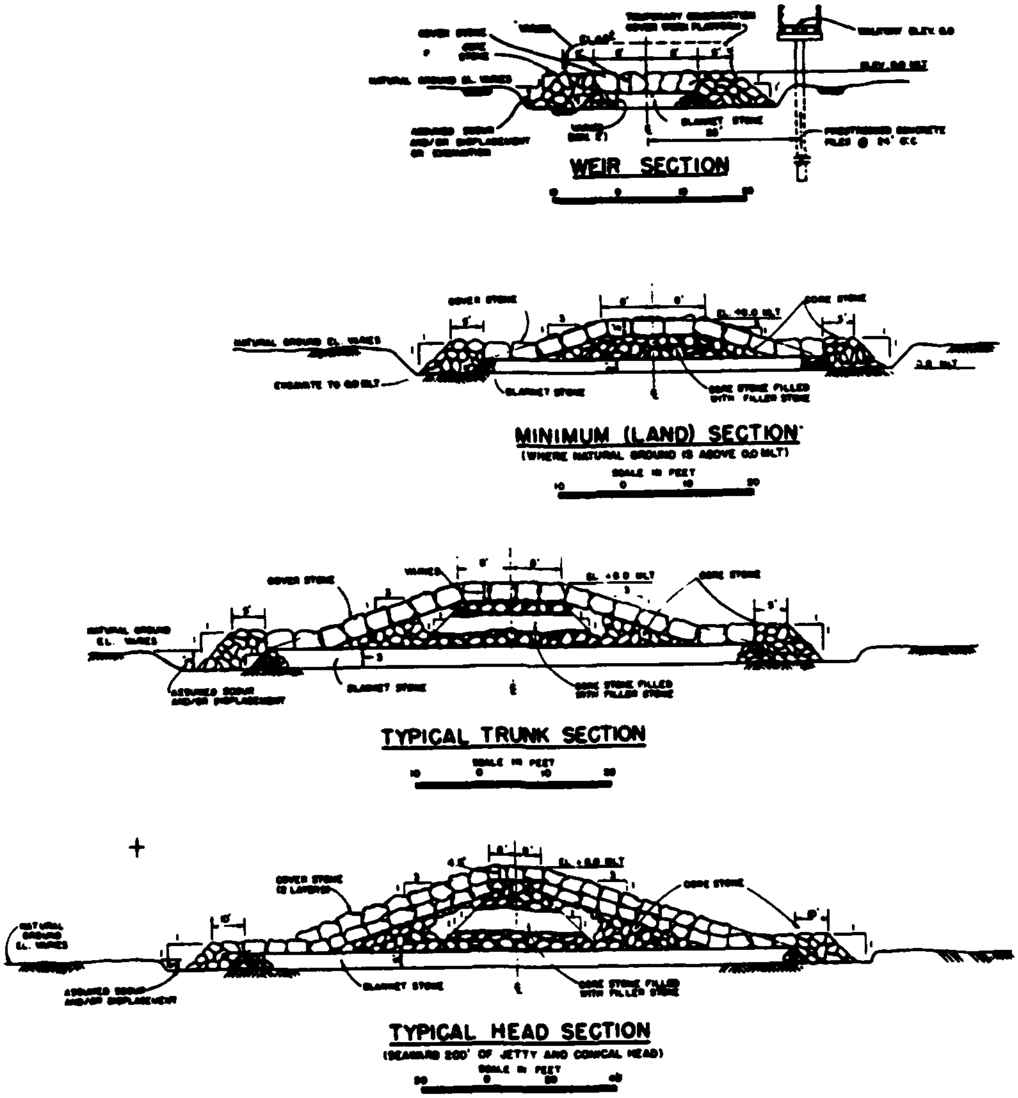


Figure 34. Jetties at mouth of Colorado River, typical design features

on the southern California coast is located at San Diego, but the growth of pleasure boating in the 1950's and 1960's led to the construction of many new marinas and small-craft harbors. The associated breakwaters tended to interrupt the drift of littoral material and this, coupled with floodwater retarding structures for flood-control purposes on the rivers, significantly reduced the amount of sand to nourish downdrift beaches. The area is illustrated in Figure 35.

150. The Pacific anticyclone plays an important role in the generation of waves along the California coast, particularly during the summer months when it is the dominant feature of the meteorological circulation in the eastern Pacific. At this time the predominant wave action, particularly in southern California, is usually generated by the prevailing west-northwest to northwest winds along the Pacific coast of the United States. These wind strengths are dependent upon the offshore pressure gradients, which in turn interact with the thermal trough over central California and Nevada (Meteorology International, Inc., 1977).

151. Extratropical cyclones that originate in the vicinity of Japan represent the most important source of severe wave reaching the California coast. These waves propagate across the Pacific Ocean and show a steady decrease in energy intensity southward along the coast. It is possible, however, with some storms, for the maximum wave energy to be centered upon central or southern California. This area usually experiences its most severe waves when storms develop between Hawaii and California and move with very strong westerly winds toward the east.

152. Southern hemisphere swell often arrives along this coast with very long periods (13 to 20 sec), and indications are that these waves are generated by extratropical cyclones proceeding from west to east across the south Pacific Ocean. Tropical cyclones off the west coast of Mexico usually occur once or twice during the summer and early fall and can generate swell sufficiently high to require consideration in coastal design work.

153. The State of California consists of 11 geomorphic provinces, each of which is distinctive from the others (South Pacific Division 1977). Along the coastlines of the State, the south coastal basin, central coastal basin, San Francisco Bay area, and the north coastal basin contain almost 1,200 miles of shoreline.

Southern California

154. Background. The coast of southern California south of Point Conception is shielded to a considerable degree by the group of eight offshore islands that are separated from the mainland by deep water.

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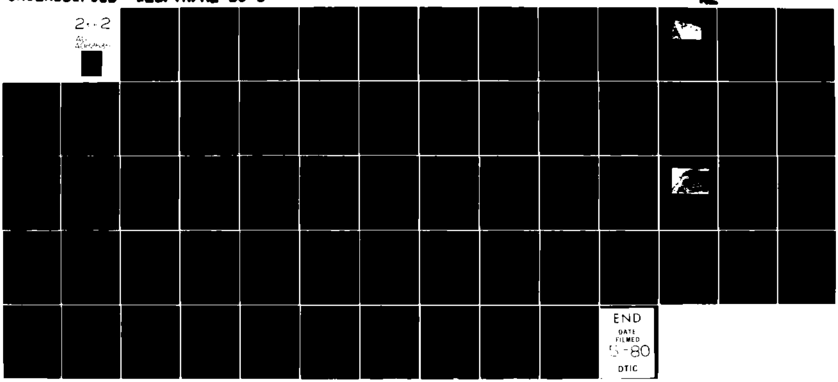

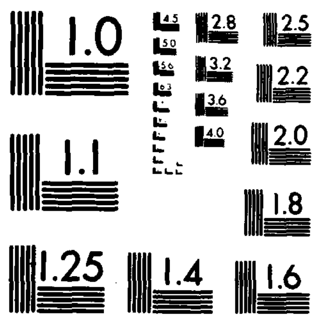

MICROCOPY RESOLUTION TEST CHART

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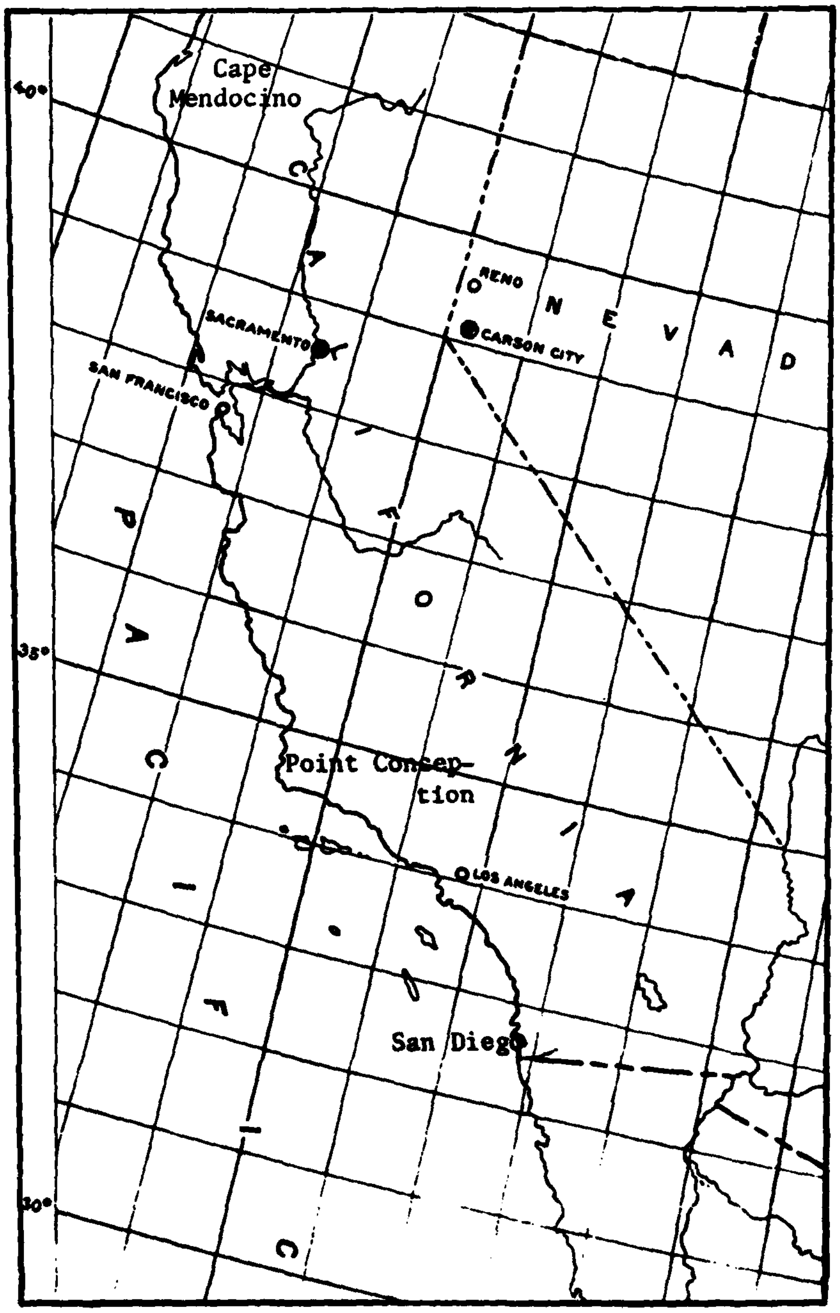

Figure 35. South Pacific region

The sheltering afforded by these islands protects the mainland coast from a portion of the deepwater wave energy propagating toward shore; thus, the wave climate on the beach is significantly less than for those regions north of the Point. The allure of this land of eternal summer is reflected by the fact that following World War II, the trickle of newcomers turned into a floodtide so that at the present time, the south coastal basin of California containing about 7 percent of the State's land area is home for about 50 percent of the State's population.

155. The south coastal basin extends from the Mexican border to just south of Santa Barbara, a distance of approximately 230 miles. The topography is varied, including gently sloping coastal plains that comprise about one-third of the area. White sandy ocean beaches and steep coastal cliffs rising from the sea make it a land of great contrasts. The coastline climate is characterized by light precipitation and mild temperatures that have small daily and annual ranges. Because ocean waters are warm enough for water-contact sports throughout the year, ocean-oriented sports are a way of life for a large part of the population. Eight small-craft harbors have been constructed in the region, although some reaches of the coastline between all-weather harbors exceed 35 miles, which is the spacing considered desirable by the State for small-craft harbors of refuge. Erosion along this region is a continuing problem. Only about 27 miles of the 233 miles of shoreline is considered stable by the South Pacific Division (1977), with the remaining shoreline being eroded at varying rates. Critical erosion is taking place along 160 miles of the shore and threatens highways, homes, businesses, and recreational beaches.

156. Scour problems. The Los Angeles District encompasses essentially all of the south coastal basin, where major stone structures consist of jetties, attached and offshore breakwaters, and groins for beach stabilization. The problems associated with surf zone and open-ocean construction have been known to include scour and erosion effects ever since the necessity arose for this type of endeavor. In past years, foundation bedding material was not used, and the core stone would slump and settle into the sandy foundation. Since each

individual project was unique in this respect, the technique of on-the-job problem solving was considered a requirement for doing business. The successful contractors developed a significant amount of expertise, and it has been determined that the equipment operators make a considerable difference regarding the degree of difficulty encountered in construction operations. Over the years, the use of foundation bedding material evolved, so that it is now standard procedure to incorporate this feature into the design requirements. Historically, filter fabric has not been used, although it is now being considered in the design of pending structures; i.e., the offshore detached breakwaters for beach protection at Imperial Beach. It is recommended that when filter fabric is used, a layer of quarry-run stone be laid on top of the fabric to minimize the probability of large stones tearing the membrane.

157. Since it is known that scour during construction will increase the volume of stone required relative to that determined by template projections, the District compensates for this increase by using a correction factor based on an analysis of past stone construction contracts. It has been observed that the final payment quantities are, in general, a function of the void ratio of the structure and of specific gravity or unit weight of the rock. Quantities of material are usually computed based on a void ratio of 35 percent and a unit weight of 165 lb per cu ft. This consistently results in final quantities approximately 10 percent greater than that estimated. It has been determined that estimates based on a void ratio of 26 percent and a unit weight of 170 lb per cu ft will predict final quantities approximately equal to that actually placed. While this procedure does not solve the problem of scour during construction, it certainly acknowledges the existence of a problem and is an operational solution of avoiding unanticipated cost overruns. In fact, it is apparently a very good operational solution for avoiding unanticipated cost overruns due to scour during construction.

158. Rip currents resulting from wave energy typically develop along the toe of groins or breakwaters, and frequently a 2- to 4-ft-deep scour area develops overnight and can be observed in the mornings

before construction resumes. To minimize this problem, an operational procedure has evolved that consists of placing an additional 20 to 40 ft of foundation bedding material in front of the core stone and cover stone at the end of the day. The specific distance is usually determined by the size and reach of crane available for the construction job. Larger cranes working from the structure crest can place a 30-ft section of foundation bedding material ahead of the core. This procedure tends to reduce the amount of scour that would otherwise occur. Discussions with District personnel seem to indicate that the additional material required to fill scour holes adds to stability of the structure by providing a better foundation.

159. The philosophy of the District is that the bedding material is the foundation of the whole structure; therefore, it is desirable to have plenty of bedding stone, probably 2 ft thick and extending beyond the toe of the structure on each side for approximately 5 ft. Side slopes of the structure are dependent on the precise design and type of cover material. The contractor should be encouraged to use an adequate amount of stone to make sure that the template projections are satisfied.

160. Rip current locations depend on incident wave conditions and bottom topography. Rip currents up to 5 knots in magnitude have been observed, and this is certainly a sufficient velocity to move bed material. Spar-buoy markers are used by the District to locate the lateral extent of the foundation bedding material to ensure that the toe of the structure is adequately protected by the bedding layer. If a scour hole develops and bedding material is lost, it should be replaced and promptly covered with protective stone. The side slopes should not, however, become so steep as to encourage rolling of material. Rip currents occurring along the toe of the structure may be further combated by the construction of spur dikes perpendicular to the axis of the main structure. Prestressed concrete pile jetties at Doheny Beach (San Juan Creek) have experienced scour along the toe of 6 to 8 ft due to flood flows. The smooth surface of the concrete piles apparently encouraged the flow concentration. This region was rehabilitated with rubble, and the problem has not reoccurred. It is imperative that when filling scour holes,

stone be used which is large enough so it won't be moved immediately, but small enough so turbulence will not move sand from underneath the stone and permit settlement. This is, in effect, the purpose of the foundation blanket; it not only acts as a bearing surface for the structure but also serves as a filter to prevent migration of material.

161. During construction of offshore detached breakwaters in the Los Angeles District, bottom-dump scows are often used to transport the material to the site and are unloaded by emptying directly over the construction site. In this procedure, it has been found that better unit costs can be obtained by specifying quarry-run core stone instead of a specified rock size. It is more economical to ask for a gradation that consists of up to 80 percent of the quarry product (saving the cost of the contractor separating the stones). This also seems to provide for better keying of the stones, apparently results in a stronger structure, and in many cases produces a more economical product.

162. In southern California it is usually necessary to work on coastal construction projects after the major tourist season. While most observers like to watch construction machinery work, the problem arises when accident potentials exist by people getting in the contractor's way. When advancing from shore, it is also expedient to work with the tide, at least through the surf zone. If the foundation blanket material is placed at low tide, the cover stone can be placed before significant movement occurs.

Central and northern California

163. Background. The central coastal basin extends northward from Santa Barbara to Santa Cruz, and except for river valleys, there is little or no coastal plain. Mountainous terrain and rolling hills extend to the shoreline, producing a rugged coast considered one of the most scenic in the United States. Of the approximately 360 miles of mainland shoreline in this area, about 50 miles is considered stable, with the remaining 310 miles in varying degrees of erosion (South Pacific Division 1977). About 90 miles is critically eroded to the extent that highways, urban properties, and recreational beaches are threatened.

164. The San Francisco Bay consists of four separate bays: Suisun,

San Pablo, Lower San Francisco, and San Francisco Bay proper. The bay area contains about 280 miles of bayshore and 150 miles of scenic coastline. The bay is a vast landlocked estuarine complex through which drains runoff from the entire Central Valley to the Pacific Ocean. San Francisco Bay has decreased in size by about 40 percent since 1850, as a result of dredging and filling of shallow areas. About 80 percent of the bay is less than 30 ft deep, and a large quantity of fine sediments have accumulated in the bay.

165. The north coastal basin extends northward from the Russian River to the California-Oregon border. Mountains and rolling hills extend to the ocean and create impressive coastal scenery. Commercial navigation facilities include Humboldt and Crescent City Harbors, where the principal waterborne commerce consists of lumber, fishing, and petroleum. The cliffs and jutting promontories of the shoreline are frequently attacked by severe storms and strong winds. Heavy waves generated by storms in the North Pacific strike the coast during summer as well as winter. Damage from other types of sea waves (tsunamis) also has occurred in the Crescent City area where major damage was experienced by the tsunami of 1964. Other nondamaging tsunamis have occurred since then.

166. Scour problems. The central coastal basin, San Francisco Bay, and north coastal basin lie within the U. S. Army Engineer District, San Francisco. While the coastline in this combined region is greater than that within the south coastal basin, navigation projects requiring major engineering works of improvement number only 10, compared with 12 in southern California. The concentration of people in southern California, more severe wave climate north of Point Conception, and the greater frequency of drizzle and fog blankets contribute to a lesser density of small-craft harbors. Another reason for the smaller requirements of beach activities, according to Stambler (1972), is the coldness of the shore waters produced by the phenomenon of upwelling. The prevailing winds along the northern California coast are from the northwest and blow roughly parallel to the shore. The Coriolis effect is to cause the surface layer of ocean water to be driven westward, approximately at

right angles to the direction of the wind. As the surface waters move away from the shore, they are replaced by deeper water which is several degrees colder. The result is that northern California has some of the coldest sea temperatures, at that latitude, of any place on earth. There are, additionally, four harbors inside San Francisco Bay which have required breakwaters or jetties.

167. While the number of structures in this District are relatively few, because of the more severe wave attack, the problems arising from scour and erosion during and after construction may be more significant than those in southern California. The large wave heights and long periods are capable of disturbing bottom particles at greater depths than that experienced elsewhere, and the strong tidal currents, particularly at Humboldt Bay, can displace the material and undermine the toes of structures. In order to properly design for the scour effects, it is necessary to know both the wave and current energy fields on both sides of breakwaters and jetties.

168. Humboldt Harbor is located on a land-locked bay at Eureka, California, about 225 miles north of San Francisco. Improvements to the harbor were first authorized in 1881, and the project consists of nearly 2 miles of jetties, assorted channels, and turning basins. The jetties are exposed to storm waves originating from the south-southwest to north. Deepwater wave hindcast statistics for this region indicate that a significant wave height of 34 ft and a period of 13 sec will occur occasionally. Because of the effects of refraction by the underwater topography, this significant wave height may be amplified to around 40 ft at the structure.

169. The north and south jetties at Humboldt Bay have been damaged and repaired several times since their initial construction. Large monolith concrete features were added to the north and south jetties in 1961 and 1963, respectively. Limited amounts of armor protection were provided to the south jetty with the placement of 256 unlinked 100-ton concrete cubes. The north monolith was undermined by scour, broke into several large irregular blocks, and gradually deteriorated due to the forces of the wave action. The south monolith also became badly damaged

and the 100-ton cubes were displaced. The water depths at the head of the jetties are such that 40-ft-high waves can break directly on the structure.

170. The head section of the jetties deteriorated severely in the mid-1960's, and WES (Davidson 1971) was requested to perform a physical hydraulic model study to determine, among other things, the dimensions of the largest waves that can attack the proposed structure so that design-wave conditions can be selected, and the optimum shape of the armor units that will be stable for the selected design wave conditions. The design waves selected for the north jetty head at still-water levels of +7.0 and 0.0 ft mllw were 40 and 31 ft, respectively. These design wave heights were determined from model tests and refraction analysis of the existing statistical deepwater wave data by National Marine Consultants, Inc. (1960). The no-damage wave height was found to be 36 ft for 33-ton tribars if the armor units in the top layer of the section were linked in clusters of units placed on radials from the concrete monolith. While the model tests indicated that the armor units in the top layer should be linked, suitable linking materials could not be found during design. Further model studies were conducted and the final section used for the repair consisted of 42-ton dolosse forward and 43-ton dolosse toward the trunks of the heads.

171. Uncertainty exists as to the desirability and degree of reinforcement to be placed in dolos units. Three kinds of dolosse (unreinforced, conventionally reinforced, and fiber reinforced) were placed in the Humboldt jetties and were color-coded for study purposes. These kinds and locations of dolosse were as follows, according to Magoon.*

* O. T. Magoon, Chief, Coastal Engineering Branch, U. S. Army Engineer Division, San Francisco, Memorandum for Chief, Planning Division, 15 August 1976.

	Humboldt Jetties Dolos Units	
	North Jetty	South Jetty
Unreinforced Dolosse		
Number Placed	4*	22
Number Broken	0	9**
Conventionally Reinforced Dolosse, 75 lb/cu ft		
Number Placed	2238	2513
Number Broken	18	9
Fiber Reinforced Dolosse		
Number Placed	17†	0
Number Broken	0	0

* Located in relatively protected areas.
** Placed in open areas at completion of work. Not integrated with other dolosse.
† 7 dolosse reinforced with 80 lb of fiber per cubic yard of concrete. 10 dolosse reinforced with 200 lb of fiber per cubic yard of concrete.

Figure 36 shows a section of the Humboldt Bay north jetty, and reveals the settled pattern of 42-ton dolos units.

172. Concern has arisen over the years regarding the stability of the south jetty. Deep scour pockets occasionally develop along the inside as a result of wave energy penetration through the entrance and a strong ebb current regime. Scour holes up to 50 ft deep have been detected, and past endeavors during rehabilitation efforts have resulted in extension of the toe and the base slope has become flattened. As a result of these actions, the scour appears to have ceased and channel migration is now at an acceptable magnitude. The toe has been armored, and it is believed the wave energy that continues to be dissipated on the interior of the structure will not cause further toe erosion sufficient to endanger the integrity of the jetty.

173. Moss Landing Harbor is located near the center of Monterey Bay, at the head of Monterey Canyon. The canyon comes almost to the entrance of the jetties, and some of the material transported alongshore is lost down the canyon. The direction of net longshore transport south

Figure 36. Humboldt Bay Harbor north jetty showing settled condition of 42-ton dolos armor units. 1978 photograph by WES

of Cape Mendocino is southerly, except for isolated locations. Wave-induced currents from the north cause strong rips to develop on the outside of the north jetty, and ebb currents on the inside are sufficient to move any sand that may pass the tip of the jetty. Divers studying the stability of the region report the canyon is migrating toward the north jetty. A potential problem of undermining of the north jetty by continued migration of Monterey Canyon may exist. Since canyon regulation is beyond present ability, close reconnaissance of potential hazards to the north jetty must be continued.

174. Half Moon Bay structures consist of two breakwaters that form a protected harbor for commercial fishing vessels and recreational craft (Figure 37). This bay is located approximately 15 miles south of San Francisco. As a remedial measure to eliminate excessive surge, construction of a 1,050-ft extension of the west breakwater was completed in 1967. Fifteen to twenty-foot waves that occurred at the entrance caused a Mach-stem effect which was felt all the way to the bottom of the bay along the breakwater. The 4-ft-high waves that existed adjacent to the

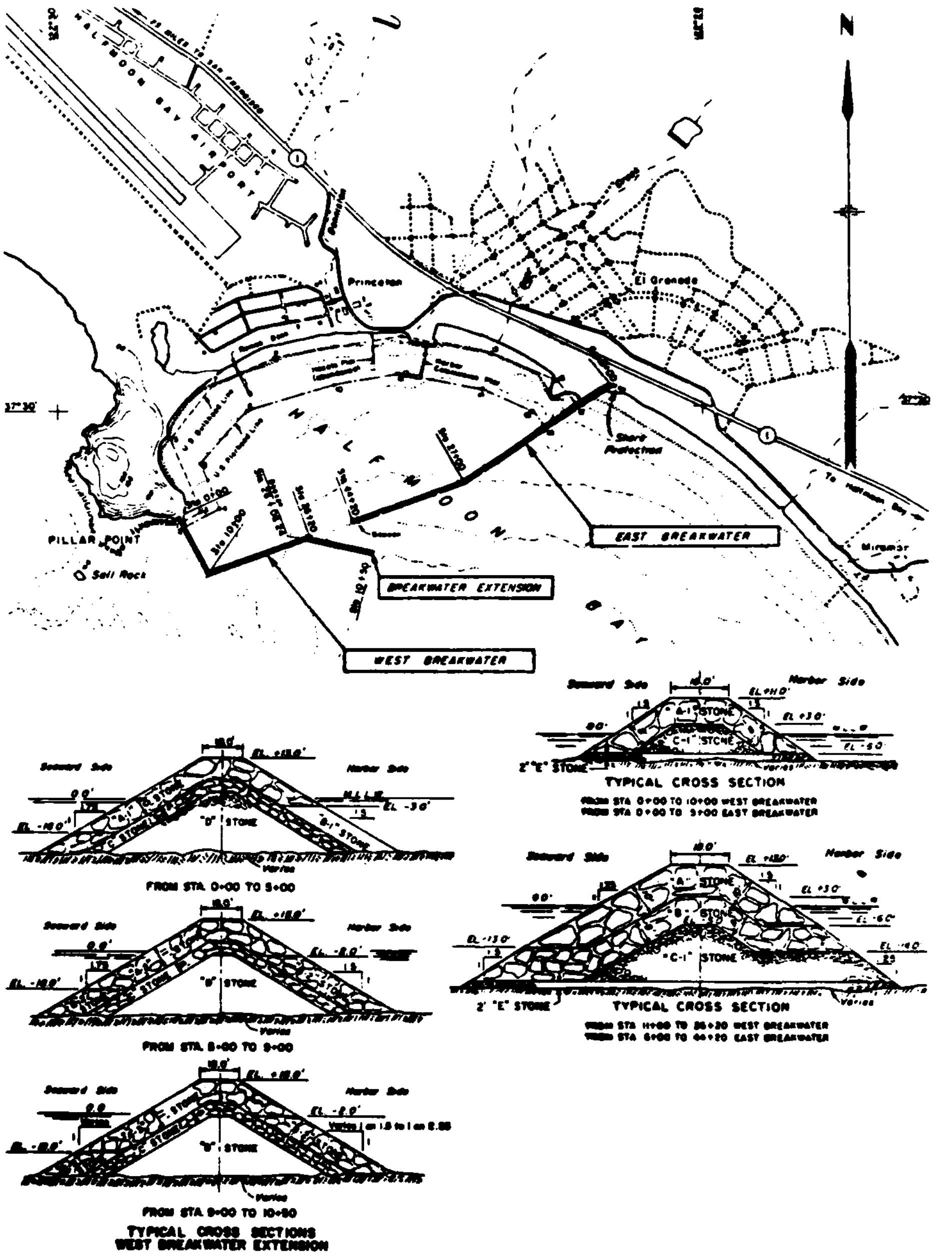

Figure 37. Half Moon Bay, California,
typical breakwater cross sections

breakwater were sufficient to cause scour of the toe of the structure, and the side slopes have steepened as armor stone has settled into the holes. Core stone have settled and large cavities up to 13 ft deep have developed along the center line of the structure with the shell remaining essentially intact by arching.

175. Rehabilitation of the toe scouring problems at Half Moon Bay was accomplished in 1978 by the placement of foundation bedding material where necessary and 6- to 20-ton cover stone where appropriate. The bedding material size varied from a No. 4 sieve to 6-in. size particles. The irregular holes in the breakwater crest were from 3 to 13 ft deep, with widths from 2 to 4 ft, and lengths from 2 to 6 ft. The holes were repaired with concrete, cast in place in synthetic filter cloth membrane bags. In holes shallower than 6 ft, one unit was cast; in holes deeper than 6 ft, two units of approximately equal volume were cast, one on top of the other. In order to adequately anticipate scour and erosion problems that may occur at Half Moon Bay in the future, knowledge of the wave and current conditions inside the channel and harbor after crossing the bar is desirable.

176. The philosophy of the San Francisco District regarding the control of scour and erosion during and after coastal construction may be summarized in the following two statements: (a) on foundations of thick, loose, sandy material, always use foundation blanket material (graded) extending for a distance of 5 ft beyond the toe of the structure; (b) on foundations of thin, loose, sandy material, excavate any loose material down to a firm foundation before the placement of any stone.

Hawaiian Islands

177. Background. The Hawaiian Island chain was originally built by volcanic eruptions, although vulcanism now continues only on the largest island, Hawaii. Because of this origin, a considerable part of the Hawaiian coasts can be described best as volcanic. Most of the shores have been modified by wave erosion and by the building of coral reefs along the margins of old volcanoes. Coastal plains have been built around some of the volcanoes and are partly alluvial and partly raised

coral reefs (Shepard and Wanless 1971).

178. The Hawaiian coastline varies dramatically in physical character from island to island, and from one area to another on each island. A large part of the coast consists of precipitous sea cliffs, although there are also long, extensive, sandy beaches on Kauai, Oahu, Maui, and Molokai. Pocket beaches are found on the Islands of Hawaii, Lanai, and Niihau. The best beaches lie on the western sides, protected from the waves of the northeast trade wind. The beaches are composed primarily of remains of organisms that lived in the sea, and are often called coral sand beaches, although coral is only one constituent and basaltic material is part of the composite. This beach sand has a lighter unit weight and the grain size is more rapidly reduced by abrasion than the quartz and feldspar sands that are commonly found on most other beaches of the world. Most of the Hawaiian beaches are subject to serious erosion and the most famous, Waikiki Beach, is at present largely artificial. The big island, Hawaii, has experienced disasters from both volcanic eruptions and tsunamis.

179. Coral reefs play a major role in the Hawaiian beach system by providing the major source of beach material and by protecting the shoreline against wave attack. Several types of reefs are found in the islands, although the fringing reef which grows directly along the shore and connects to the land mass is the most common. Kaneohe Bay has the only barrier reef in Hawaii. The coral reefs are constructed by shallow-water organisms comprising corals and coralline algae. All the reefs have passes or channels of deeper water leading through them. These channels are often opposite the mouths of rivers where both the fresh water and the sediment brought into the ocean by the river inhibits the growth of the corals. At Waikiki, the outer edge of the fringing reef determines where oscillatory waves change to waves of translation which are used for surfing.

180. Scour problems. Two deep-draft harbors and two small-boat harbors are located on the Island of Hawaii, and all have required the construction of breakwaters or other measures for their protection. Hilo Harbor was constructed in 1930, and consists of a rubble-mound breakwater

10,080 ft long. This project was intended to improve navigation conditions in the harbor by reducing surge. The Corps of Engineers (Palmer et al. 1967) conducted hydraulic model tests of various structural schemes which showed that conditions in the harbor could be improved by raising and lengthening the existing breakwater, constructing a 4,000-ft long west breakwater, and a 6,600-ft-long land dike. Further work on this project has been deferred at the present time. Hilo Harbor is located on the eastern side of the island, and Kawaihae deep-draft and Kawaihae small-boat harbors are situated on the northwest. These harbors were created by dredging part of an extensive coral reef. Honokohau small-boat harbor, near Kailua-Kona, Hawaii, on the western side of the island, was constructed in 1970 by the Corps of Engineers by excavating inland. No breakwaters were required, but rubble-mound wave absorbers were necessary along the entrance channel.

181. Kahului Harbor is on the north shore of Maui and has two breakwaters. In 1966, sections of the breakwaters were rehabilitated to withstand larger storm waves by using 35- and 50-ton tribars. The breakwaters were again repaired in 1977 by using 20- and 30-ton steel reinforced dolosse on the west breakwater trunk and on the seaward quadrants of both breakwater heads, respectively. Architectural, engineering, and design studies are currently under way for navigation improvements to the State-constructed Maalaea boat harbor on the south shore. Lahaina small-boat harbor is located on the west coast of the island, and construction of a small-boat harbor immediately north of the existing harbor has been authorized.

182. Manele is a small-boat harbor on the southern coast of Lanai, and was constructed by the State of Hawaii to provide limited protection against wave action. At the request of the State, the Corps of Engineers constructed a 470-ft-long extension to a 100-ft-long existing rubble-mound breakwater for additional protection.

183. The small town of Kalaupapa is located on the northern coast of the Island of Molokai. This coast is exposed to severe winter storms, and the Corps of Engineers has improved the existing barge harbor by

lengthening the rubble-mound breakwater and protecting it with armor stone.

184. Haleiwa Beach Park is on the north shore of Oahu, immediately east of the Haleiwa small-boat harbor, and is also exposed to storms from the north. In 1969, a severe storm damaged the groin and offshore breakwater used to stabilize the beach. The breakwater was rehabilitated with Corps assistance; however, since that time, the beach structures and the harbor breakwater have been damaged several times by severe winter storm waves.

185. Honolulu Harbor is located on the southeast coast of the Island of Oahu, is the only commercial deep-draft harbor on the island, and is the largest civil port in Hawaii. The harbor was originally developed in the late 1700's in a natural protected embayment created by the flows of the Nuuanu Stream. The harbor is roughly crescent-shaped, is approximately 2 miles long, and varies in width from 600 to 2,900 ft. The existing harbor has been extensively expanded from its original configuration by the dredging of berthing areas into the natural shoreline and construction of piers and other harbor-related structures along the entire shoreline fronting the harbor complex. Dredged material was used to create an island seaward of the harbor for protection of shoreside facility developments.

186. Waianae is located on the west coast of Oahu, approximately 30 miles from Honolulu. The Waianae coast is an excellent boating area, and the waters offshore provide some of the best fishing in the Hawaiian Islands. The area is presently served by the existing Pokai Bay Boat Harbor. A State-constructed breakwater extends into Pokai Bay forming a protected basin that is used as a berthing area for a maximum of about 90 boats. This harbor and its facilities are severely overcrowded. In addition, littoral material from the north is trapped within the Pokai Bay boat basin, causing shoaling and requiring frequent sand removal. For these reasons, WES (Bottin et al. 1976) was requested to perform a physical hydraulic model study of a proposed new harbor site to aid in the development of a satisfactory harbor configuration and

entrance channel location and alignment. The Corps of Engineers completed construction of this new harbor at Waianae in 1979. The project consists of a 1,690-ft-long breakwater armored with 2-ton dolosse, a 220-ft-long stub breakwater, an entrance channel, turning basin, and main access channel. The new harbor is designed to accommodate about 300 recreational boats.

187. The existing barge harbor at Barbers Point is located at the southern extent of the west coast of the Island of Oahu, and is approximately 16 miles west of Honolulu. The existing harbor is an L-shaped area that was dredged in the coral formation of the flat coastal plain by private interest in 1960 and is approximately 8 acres in size. The authorized plan of improvement provides for a 450-ft-wide, 42-ft-deep entrance channel, and a 46-acre harbor basin dredged 38 ft deep. A hydraulic model investigation of the project was conducted in 1967-68 by the Look Laboratory (1970). This model study indicated that a nearly trapezoidal-shaped, 77-acre basin was a most effective configuration.

188. Because of the long time interval between authorization of the project in 1965 and changes in the economic and environmental conditions of the project area as well as in Corps water resource planning policies, a postauthorization study was conducted by the U. S. Army Engineer District, Honolulu, in 1976 to assess the past basic planning decisions for Barbers Point deep-draft harbor and to provide a harbor project which could respond to these changes. This postauthorization study indicated a need for a basin that is larger than the original design. This proposed plan of improvement consists of an entrance channel 4,480 ft long, 450 to 650 ft wide, and 38 to 42 ft deep; an inshore basin with an area of about 92 acres; and 4,700 ft of wave absorbers. WES (Durham 1978) performed a numerical analysis of harbor oscillations to determine the effects of the basin enlargement on surging in the proposed deep-draft harbor, and other WES studies are still ongoing.

189. Fronting Ala Moana Park in the city of Honolulu, on the Island of Oahu, is a shallow coral reef extending about 1,500 ft offshore and lying between the existing channels of Ala Wai Boat Harbor on the east

and Kewalo Basin on the west. Ala Wai Boat Harbor is a pleasure craft harbor, and Kewalo Basin is the home port of a large portion of Hawaii's commercial fishing fleet and research and charter boat service. WES (Brasfeild and Chatham 1967) conducted a physical hydraulic model study to: (a) develop a satisfactory circulation system through the inner lagoon, (b) study wave action in Kewalo Basin for existing conditions and with proposed breakwater and wave-absorber plans, and (c) investigate the effects of proposed construction plans on wave conditions at Kewalo Basin. It was determined that the construction of the proposed Kewalo Peninsula would aggravate the unfavorable wave conditions presently existing in Kewalo Basin unless remedial action is taken.

190. Two deep-draft harbors and one small-boat harbor have been constructed on the Island of Kauai; i.e., Port Allen deep-draft harbor, Nawiliwili deep-draft harbor, and Nawiliwili small-boat harbor. The existing project of Port Allen consists of a breakwater 1,200 ft long, an entrance channel, and a harbor basin. The breakwater has required no maintenance or other work since it was constructed. Nawiliwili deep-draft harbor was first developed in 1930 by the construction of a rubble-mound breakwater, and was enlarged in 1956 using a single layer of 17.8-ton tribars. The breakwater is 2,150 ft long and was rehabilitated in 1977 with two layers of random-placed 12-ton dolosse. The adequacy of the breakwater repair sections had been investigated by model tests performed by WES (Davidson 1978). These model tests were deemed necessary because there are limited design data for dolosse in breaking wave conditions, since the crown elevation of the Nawiliwili breakwater is such that the structure can be overtopped by the selected design wave, and because the water depth at the toe of the breakwater is such that waves of height equal to or slightly less than the design wave will break directly on the structure. Federal navigation improvements to a fourth harbor on Kauai, the State-constructed Kikiaola Boat Harbor, have been authorized and are presently in the architectural, engineering, and design stages.

191. The problem of scour and erosion of sandy or otherwise unconsolidated material has been a continuing (although not extremely serious)

problem at all of the previously mentioned sites where breakwaters and jetties have been constructed, from the standpoint of both design procedures and construction operations. Scour is a serious problem at shore protection project sites in Hawaii, American Samoa, Guam, and the Northern Mariana Islands. While the effects of flood and ebb currents discharging through estuary inlets are virtually nonexistent in the Hawaiian Islands, exceedingly severe wave climates frequently occur; and the accompanying structural damage combined with undermining and erosion of the toe by wave-induced currents can produce complete failure of the structure. The construction procedure practiced by the U. S. Army Engineer Division, Pacific Ocean, is to essentially bury the toe of the structure. This is accomplished by excavating a trench in those situations where the material is not being moved by the naturally existing forces, usually 2 to 6 ft deep, and building the base of the structure in this excavation. In some cases this can be difficult and costly as the newly excavated material may try to return to the cavity; in other situations the current-induced scour tends to assist the operation by removing portions of the loose material. The Division philosophy has been summarized by Cheung* as:

> ".....In Hawaii, as well as other Pacific Islands, we generally attempt to place the toe on solid reef rock or basalt. Where the shoreline is sandy and no solid substrate is near the surface, we excavate as deep as practicable (about 6 ft in sand) to place the toe of the structure below the anticipated scour depth. In addition, the toe stone is extended seaward to reduce the impact of toe scour on the stability of the structure....."

192. A good example of burying the toe of a structure is the Kekaha shore protection project on the Island of Kauai, presently under construction. This project consists of 5,900 ft of rubble revetment

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* K. Cheung, Chief, Engineering Division, U. S. Army Engineer Division, Pacific Ocean, Honolulu, Hawaii, personal communication, 17 July 1979.

placed on a sandy foundation with a history of serious erosion. The revetment crest elevation is +12 ft mllw, and is being constructed to a 1V-on-1.5H slope with two layers of 2- to 4-ton armor over a bedding underlayer of spalls which range up to 500-lb stone. The toe is being excavated to -6 ft mllw with a 12-ft-wide bench of armor stone and underlayer extending seaward.

193. Often only localized pockets of sand exist in the vicinity of coral reefs, or perhaps a thin veneer of sand covers the reef. Under these conditions, the sandy material is removed all the way to the reef or otherwise solid foundation. It is desirable that guidance be provided regarding the depth of penetration necessary for structures on thick sand, or the degree and extent of toe protection on thin layers of sandy material in order to provide adequately for the structure safety, but at the same time to avoid excessive and costly overbuilding.

North Pacific Region

194. The coastlines of the States of Oregon and Washington probably come closer to being in their natural state than any other section of coastline in the continental United States. No large cities and only a few small towns overlook the shore as the major cities have been situated farther inland on the rivers such as the Columbia, or on the shores of Puget Sound. The reason for this, according to Stambler (1972), is the trend toward colder temperatures, damp overcast skies for a large portion of the time, and fierce storms in winter. This region is, therefore, less attractive for both large resort towns and seaports than the gentler regions to the south.

195. The relatively straight coastline north of the California-Oregon border alternates between large river lowlands and mountainous tracts. The lowlands are usually bordered by long barrier spits and partially filled lagoons. The mountain regions are bordered by a rugged coast with many prominent points.

196. The winds in this area are significantly different from those off California and the winter storms have a preponderance of southerly

direction. Northwest winds are predominant in the summer. The result is that winter storms transport sand north along the beaches, and the smaller waves of summer return it to the south. The apparent resulting net transport of longshore material to the north is reflected by sand accumulations on the southern side of jetties at the river mouths in excess of that which has accumulated on the northern side. However, researchers at Oregon State University have advanced the theory that the coastline is made up of a series of segments, and within each segment the net transport is essentially zero. Entrances to most of the rivers and estuaries have been stabilized by jetties to maintain navigation channels.

197. The character of the coastal region completely changes beyond Cape Flattery, at the entrance to the Strait of Juan de Fuca. This strait is the southernmost of the deep glacial troughs that continue northward toward Alaska. While the strait separating northern Washington State from Vancouver Island is quite uniform, the eastern reaches known as Puget Sound consist of a series of intricate embayments and numerous passages between the San Juan Islands, and this produces a coast more irregular than encountered elsewhere. This glaciated area has an abundance of natural shelters for all boats, unlike the absence of many natural harbors on the Pacific Ocean coasts.

Northwest coastal area

198. Background. The coastal region of the Pacific northwest, (Figure 38) may best be described as erosional tectonic with uplifted coastal terraces. Steep and often unstable cliffs are interspersed between sandy beaches and river mouths, with rock outcrops appearing frequently near headlands. There are no known canyons extending within 3 miles of the shore. Surface sediments of the nearshore zone are primarily sands consisting of quartz and feldspar. This sandy region extends out to a water depth of approximately 300 ft off the northern and central Oregon coast, varying in thickness up to 90 ft. Off the Washington coast, the sand extends to a water depth of at least 200 ft.

199. The seasonal cycle of winds on the Pacific Northwest coast is largely determined by the circulation about the North Pacific

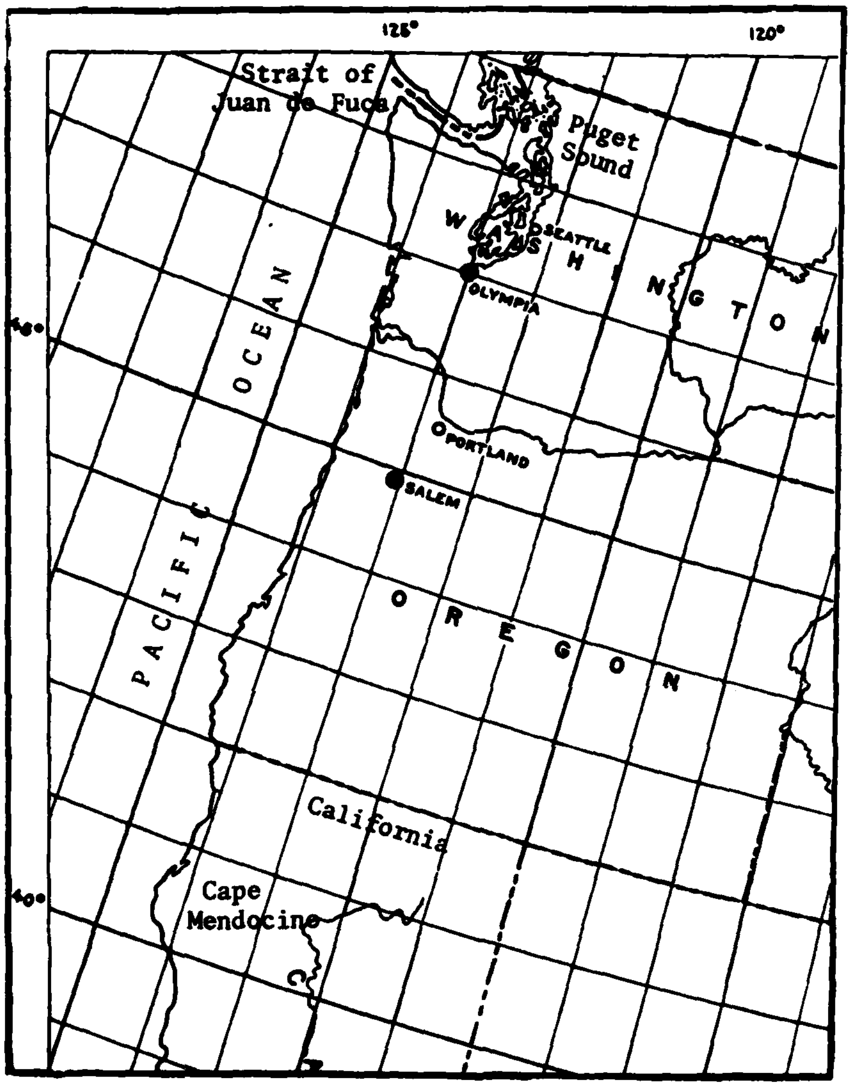

Figure 38. North Pacific region

high-pressure area and the Aleutian low-pressure area. The interaction of these pressure zones favors the development of summer winds generally from northwest to north over the nearshore and coastal areas of Oregon and Washington. During winter, the North Pacific high weakens and shifts, while the Aleutian low intensifies. The resulting gale-force winds approach the coast from a southwesterly direction. The barriers presented by the mountains of the Coast Range deflect the winds so that they tend to align with the general orientation of the coast. Waves thus produced transport a tremendous amount of littoral material northward in this region.

200. Scour problems. The Corps of Engineers has been responsible for the maintenance of Pacific coast harbors since the mid-1860's. At the present time, CE work consists of maintaining or improving channels, and constructing, maintaining, or improving jetties and breakwaters. The U. S. Army Engineer District, Portland, has responsibility for 15 coastal projects in Oregon, in addition to the Columbia River 48-ft-deep channel shared by Oregon and Washington. It is necessary to dredge about 3.5 million cu yd of bottom material each year to keep the entrance channel to the Columbia River at the depth intended for navigation. This river transports a vast quantity of sand to the ocean, and due to strong southwest winter winds, much of this material is carried northward by littoral currents. There is evidence to suggest that sediments also bypass the river jetties in large quantities. Only two major navigation projects of the Pacific coast are maintained by the U. S. Army Engineer District, Seattle, at the present time. These are Quillayute River at La Push, and Grays Harbor.

201. The use of filter fabric as a foundation blanket material has not been seriously considered by the Portland District because of the severe wave climate and the anticipated problem of keeping the fabric in place until a stabilizing layer of cover stone can be applied. Crushed rock blankets are routinely designed and constructed which consist of material varying in size (graded) up to 500 lb. This material is placed in a layer at least 3 ft thick extending beyond the toe of the structure for approximately 25 ft. The breakwater or jetty is constructed by a crane working from the crest of the structure placing each stone individually for keying and stability. During normal jetty construction operations, approximately a 50-ft section of structure is constructed as a unit. In addition to armoring the side slopes, the nose of the core is armored with cover stone for a minimum length of 30 ft when work is suspended for the winter season. The stone thus placed will be rearranged during the next construction season to conform to the original

specifications. Progress on the order of 50 to 100 ft per week is not uncommon.

202. Probably the most severe case of scour during construction ever experienced occurred while building the south jetty at Tillamook Bay entrance channel. This configuration is quite similar to that at Murrells Inlet, South Carolina, previously discussed, in that the south jetty would extend from the shore across a flood channel and onto a large shoal region (Figure 39). It had been anticipated that 3 to 5 ft of scour could result from the wave climate known to exist in the area and because of the effect of the strong tidal current. As jetty construction progressed across the flood channel, scour and erosion began to undermine the end of the structure; and the channel proceeded to migrate at the same rate as the new jetty construction. The result was that a 40-ft-deep scour hole (unaccounted for in cost estimates) had to be filled. During the first year of construction, substantial overruns of stone quantities were experienced, reflected by the fact that approximately 5,000 ft of structure had been expected to be built and less than 3,000 ft was actually constructed.

203. During the second construction year of the Tillamook Bay south jetty, the technique of "accelerated core placement" was developed by the District. To minimize scour of the ocean bottom ahead of the jetty, rapid placement of the core stone was essential. During construction, the contractor was required to work all daylight hours, seven days per week, except as prevented by weather or sea conditions, and was required to place the core stone at a minimum rate of 650 tons per hour. Sufficient equipment and personnel were available for this rate of placement. Core stone placement was accomplished only when predicted tides for the 10-day period following were not expected to exceed a 9-ft differential between lower low water and higher high water. It was the intent to place stone during a period when scouring velocities ahead of the core construction were lowest and most favorable for the work. Prior to the start of each season's core construction, core stone was stockpiled in amounts sufficient to ensure uninterrupted placement at the above specified rate. During a construction season, the contractor

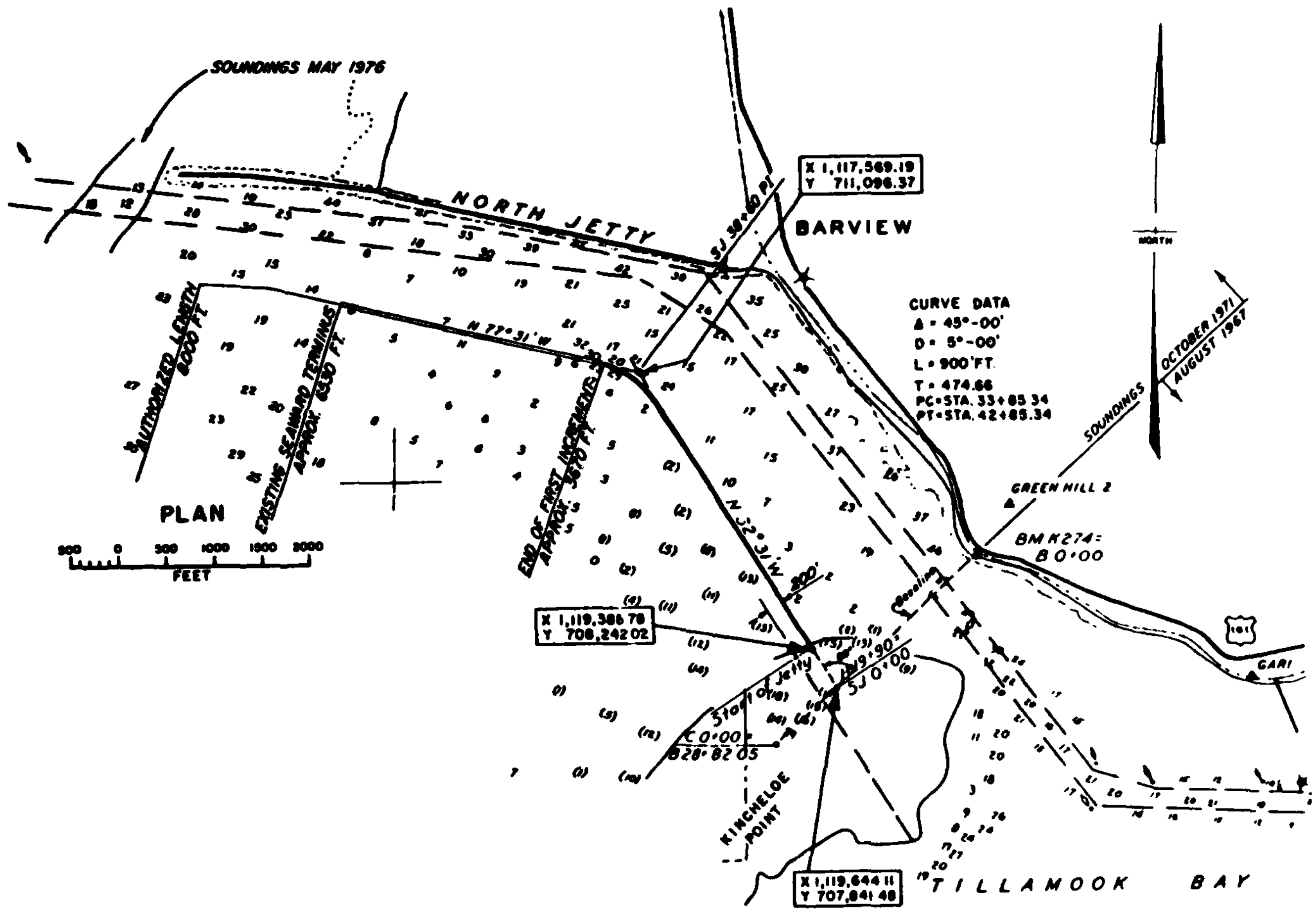

Figure 39. Tillamook Bay, Oregon, jetty alignment

was instructed to place no more core stone than could be armored that season. This accelerated placement technique reduced the depth of the scour hole by about 50 percent, from a 40-ft-deep hole during the first construction year to around 20 ft for the subsequent construction periods.

204. Most of the major offshore stone construction work being conducted by the Portland District consists of extensions and rehabilitation of existing jetties and breakwaters, so much so that new work has come to be thought of as simply the foundation for the first rehabilitation. As part of the rehabilitation efforts for the Umpqua River entrance structures, a training jetty between the existing jetties has been proposed to stabilize the location of the navigation channel. Located in the curving lower section of the channel, unnatural accelerations will exist and be reflected as scouring forces on the training jetty. The anticipated scour is approximately 2 ft, as shown in Figure 40. In this District, it is more economical to quarry stone than to cast armor units.

205. Grays Harbor, Washington, is located about 45 miles north of the mouth of the Columbia River. The natural entrance to the harbor was about 2.5 miles wide and is now protected by two converging rubble-mound jetties that are 6,500 ft apart at their outer ends. The top elevation of both jetties as originally constructed was +8.0 ft mllw. The outer bar and entrance channel are actually self-maintaining for the authorized dimensions and thus require no maintenance dredging. Controlling depths on the outer bar are about -35 ft mllw and maximum depths in the channel are about -70 ft mllw. Following the rehabilitation of the south and north jetties in the late 1930's and early 1940's to +20 ft mllw, the entrance channel migrated south to a location adjacent to the south jetty. Scour along this jetty caused fear that the structure would eventually be undermined unless the scour could be controlled. As a result of a physical hydraulic model study by WES (1955) in 1952, and based on recommendations of the U. S. Army Engineer Committee on Tidal Hydraulics (1963), it was decided that the outer 6,000 ft of the south jetty should be allowed to deteriorate to about elevation 0.0 ft mllw to

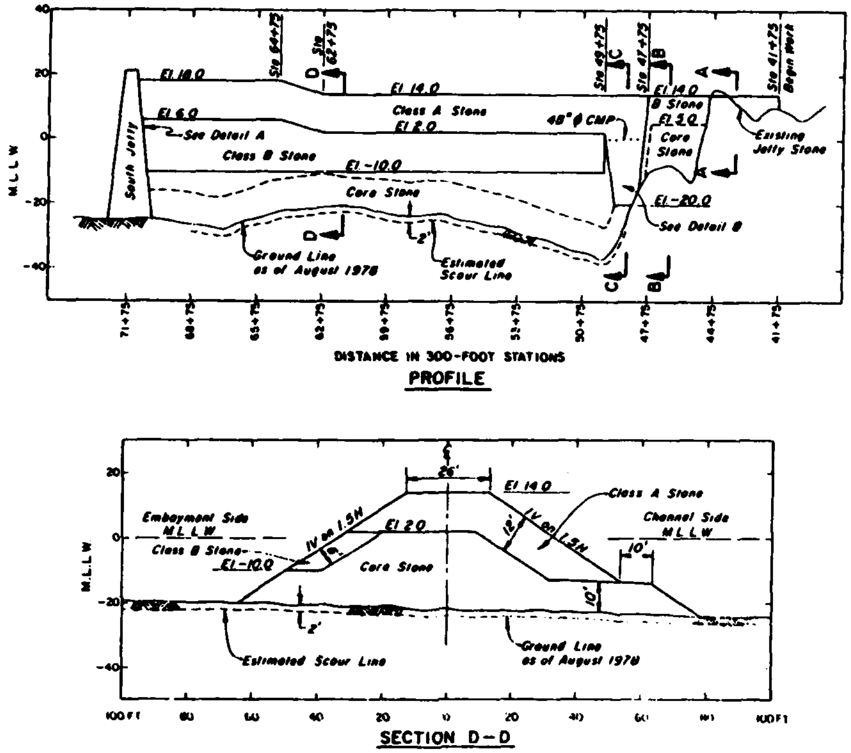

Figure 40. Umpqua River, Oregon, training jetty profile and section

alleviate the very serious erosion problem at Point Chehalis.

206. The inner 4,000 ft of the south jetty was rehabilitated in 1966 to elevation +20 ft mllw and a blanket of quarry rock was placed along the channel side slope of the jetty to prevent undermining. Disposition of the blanket material was a special concern during construction because strong tidal currents, heavy wave action, and placement to depths of -60 ft mllw prevented assurance that adequate slope protection was being attained. However, the jetty is still at project height and apparently the slope protection has successfully prevented undermining to date. The outer 6,000 ft of jetty continues to subside and is presently well below mllw. The outer end of the submerged jetty is retreating landward as scour to a depth of -50 ft mllw progressively undermines the end.

207. Based on experimental studies by Brogdon and Fisackerly (1973),

in 1975 the Seattle District rehabilitated all but 300 ft of the outer 6,300 ft of north jetty to evelation +20 ft mllw. The jetty had deteriorated to at or below the high-water line, allowing sand to pass over the jetty and into the entrance channel. Continued deterioration would have caused a significant increase in maintenance dredging, compounded the scour potential along the south jetty, and caused major shoreline erosion of lands north of the jetty. The 1975 center line was set slightly landward of previous works so that the seaward toe of the stone placement was above the midtide level and on a base of existing rock. The landward toe of the new section would catch on existing jetty rock in places but would usually lie on sands along the inside of the jetty. Either a scour hole as a result of waves overtopping the low jetty or an accumulation of sand existed along the inside toe. As the contractor proceeded with construction, he was required to either fill the scour hole with bedding rock or partially excavate the sands and backfill with excess bedding material. The bedding rock then provided a base for placement of the large capstones and/or was of sufficient excess quantity to fill scour holes that might subsequently develop.

208. In estimating the quantities to be required in the rehabilitation of the north jetty, a fairly substantial contingency factor had been applied in anticipation of the necessity to fill scour holes. The philosophy of this District regarding stability of the structure may be considered as: (a) either excavate the anticipated scour region if it does not occur naturally during construction and fill with sufficient stone, or (b) provide excess stone on the toe of the structure to adequately fill the scour hole should it develop at some time following construction.

209. During the historical operation of the jetties, the north and south beach shoreline erosion/accretion trends can be directly related to conditions of the jetties. During years just prior to rehabilitation, recession of both shorelines has been extensive and on the order of 2 million cu yd per year. Just prior to reconstruction of the north jetty in 1941-42, the shoreline had receded almost 1 mile from its position in

1935. Upon each rehabilitation of a jetty, the accretion was immediately evident and initially was on the order of 2 million cu yd per year. With both jetties presently in good condition, the north and south shorelines are in a relatively stable position and appear to be only undergoing expected seasonal beach changes.

Strait of Juan de
Fuca and Puget Sound

210. Background. The Strait of Juan de Fuca separates the southern shore of Vancouver Island, Canada, from the northern coast of the State of Washington. This important body of water, clearly the result of glacial erosion, is the connecting channel between the Pacific Ocean and the interisland passages extending southeastward into Puget Sound and northeastward to the inland waters of British Columbia. The strait has very steep sides, and the water depths descend rapidly. The 100-fathom contour penetrates up the strait for almost 40 miles.

211. The coastline along the strait is generally steep with narrow beaches of coarse sand and large boulders, except in the more protected bays. West of Port Angeles there are large sweeping sandy beaches, but to the east the shoreline is more gravelly except for the mud flats of Dungeness. Waves generated in the North Pacific enter the strait as swell. In addition, shorter period waves may be generated in the strait due to the relatively long fetch and predominant west-northwest winds.

212. East of Dungeness Spit, the comparatively smooth southern shore of the Strait of Juan de Fuca is interrupted by many bays and indentions, clearly an indication of north-to-south ice movement during the glacial age. Puget Sound proper is connected to the strait by a series of waterways, channels, and inlets. The width of these deep waterways decreases in a northward direction, but shoals and tidewater flats frequently give a total width of as much as 5 miles.

213. The shoreline of Puget Sound is very dissimilar in character from that of other coastal areas of the continental United States. Of the total of over 2,000 miles, the greater portion is faced with bluffs ranging from 50 to 500 ft composed of glacial till. A generally narrow beachline is found at the base of these bluffs, and Puget Sound has,

surprisingly, a considerable number of beaches. Puget Sound beaches consist either of gray coarse sand or a mixture of sand with small gravel. The climate of the area is typically maritime. Winds may exceed 60 knots during winter storms, but because of the limited fetches, waves rarely exceed 6 ft in height, even in the larger open areas of the sound.

214. Scour problems. Because of the climatic and geophysical features of the region, wave-induced currents sufficient to cause scour around structures are rare, although river currents are capable of transporting materials into Puget Sound. Foundation settlement problems occur occasionally as there are sections of mud-flat bottom conditions. Early construction work utilized the practice of building stone structures on brush mattresses that had been placed on unconsolidated foundations. The mattresses eventually deteriorated and the structures generally experienced some degree of settlement, although probably not from either wave action or wave-induced scour. The brush mattress bedding material was replaced with quarry-run stone when this practice became routine elsewhere.

215. The sandy spit at Port Angeles, Washington, which extends out into the Strait of Juan de Fuca, provides great protection to the town and harbor from wind-generated waves and the swell propagating from the North Pacific. The spit, Ediz Hook, approximately 3.5 miles in length, built eastward by material from erosion of the bluffs immediately to the west (Figure 41). This spit is owned by the U. S. Government and the City of Port Angeles; parts of the Government portion are leased on a long-term basis to the city, and portions are, in turn, subleased to large pulp mill operations. The pulp mills use tremendous quantities of water, and have constructed reservoirs in the mountains near Port Angeles for supply purposes. The pulp mills have also constructed a pipeline to transport the fresh water to the mills. The pipeline was placed at the base of the cliffs which fed the spit. To protect the pipeline, the pulp mills originally erected steel-sheet pile walls that failed by erosion at the toe. The walls were rebuilt and stabilized with rock riprap, and are being maintained by the pulp companies at a cost of

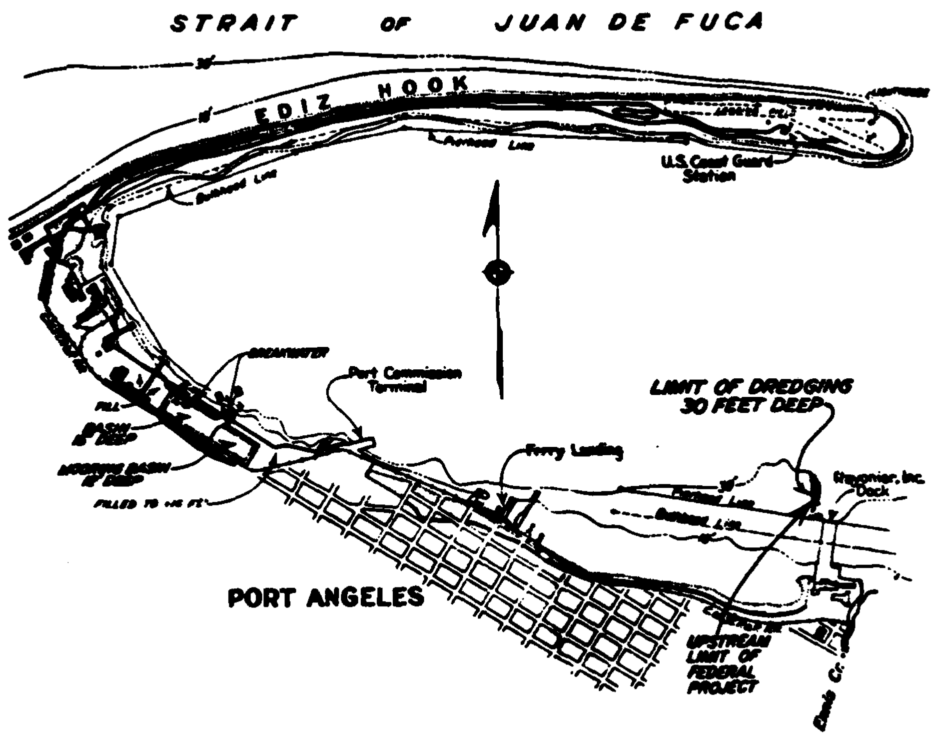

Figure 41. Ediz Hook, Washington, shore erosion control structure

about one-half million dollars per year. With the cliff erosion under control, serious scour of the west side of the spit began to occur.

216. To stabilize the spit, the Seattle District developed shore revetment plans to be constructed and operated in conjunction with beach nourishment techniques. Some previous revetment works had been built in the past, and the necessity existed for excavating portions of the old structure. Parts of these former works were constructed with little or no toe or filter bed material and had eroded and scoured underneath the structure; thus, insight was provided into the necessity of designing adequate toe and filter bed erosion protection features. The construction specifications required that the old structure be excavated adequately to receive the foundation blanket material for the new work, that the blanket be composed of Class C stone with a thickness of 3 ft, and that the blanket extend beyond the toe of the new revetment structure for a distance of 20 ft. Following completion of the revetment, 170,000

tons of cobble beach nourishment was placed, with the material ranging in size from 1 to 12 in., and with 50 percent of the material larger than 3 in. The material was placed parallel to the shoreline along the seaward face of the revetment and extended out into the tide zone, averaging 15 cu yd per foot of beach. The majority of the material was placed along the base of the spit, and the remaining portion along the western half of the spit. The material was obtained from upland sources and was processed at the quarry to yield the heavy gradation.

217. During the revetment construction process, it was desired that the newly excavated areas not be refilled with littoral drift material; therefore, it was specified that the excavation of existing structures and reconstruction of revetment should be conducted concurrently, except for the placement of the beach nourishment material. The contractor was further required to conduct his operations in such a manner that no section of the north side of Ediz Hook adjacent to the work area be left without protection during nonworking hours.

218. The construction of the shore revetment and beach nourishment work on Ediz Hook was considered as a last line of defense. The source of beach-building material had effectively been eliminated from the system, and the scouring potential of the incoming wave trains continued to transport material in an easterly direction. The beach nourishment section extended from an elevation of +12 ft mllw to a lowest elevation of -5 ft mllw. Periodic nourishment estimated at an equivalent volume every five years is authorized.

Alaska

219. Background. The largest of the United States, Alaska has more miles of coastline than all the other states combined. Not considering the small bays, there are around 7,000 miles of shore situated in three climatic zones (temperate, subarctic, and arctic). Alaska is bounded on the south by the North Pacific Ocean, on the west by the Bering Sea, and on the north by the Arctic Ocean. The State's eastern neighbor is Canada, including a narrow 350-mile-long strip of land which

constitutes the coast of British Columbia. This Panhandle region consists of elongated island land fjords known as the Inland Passage, a well-protected series of inlets and sheltered stretches of deep water utilized by cruise ships, commercial vessels, and the Alaska Marine Highway system. The Panhandle is located in the temperate zone. The remainder of the southern coast of Alaska, including both sides of the Aleutian Island chain, lies in the subarctic climate zone. Essentially all of the western coast and all of the northern coastline lie in the arctic zone (Figure 42).

220. Water navigation and transportation in Alaska are impacted by three types of ice: river ice, coastal ice, and sea ice. Each type has its own season and its own importance. The arctic zone is that region in which there is essentially 100 percent sea ice coverage in the winter. Coastal ice forms and melts earlier than sea ice, forming in early November and melting in early May at Bristol Bay. Because of the limited navigation season, there are only four navigation projects by the Corps north of Bristol Bay, Nome being one of these. Accordingly, attention will be directed toward the southern subarctic and temperate zones of Alaska, including the Aleutian Islands, the Alaska Peninsula, Gulf of Alaska, and the Panhandle.

221. The curving coast of southern Alaska is composed of mountains which are still growing, thus accounting for the numerous severe earthquakes that have caused great catastrophes along this coast and that sometimes change the coastline radically. The Alaskan earthquake of 1964 was accompanied by uplifts of land greater than ever recorded elsewhere. The effects of Pleistocene glaciation are seen in the inland passages and islands, which dominate the Panhandle, and in the numerous fjords which indent the Alaska Peninsula. Coastal islands trend mostly in a northwest direction, thus permitting ships to traverse this entire region with scarcely any exposure to the open ocean.

222. The Copper River drains a large interior lowland area and the surrounding mountains. The river delta includes three zones parallel to the coast: the inner marsh, the tidal flat, and the barrier islands. The sand and silt from the Copper River meltwater have been deposited in

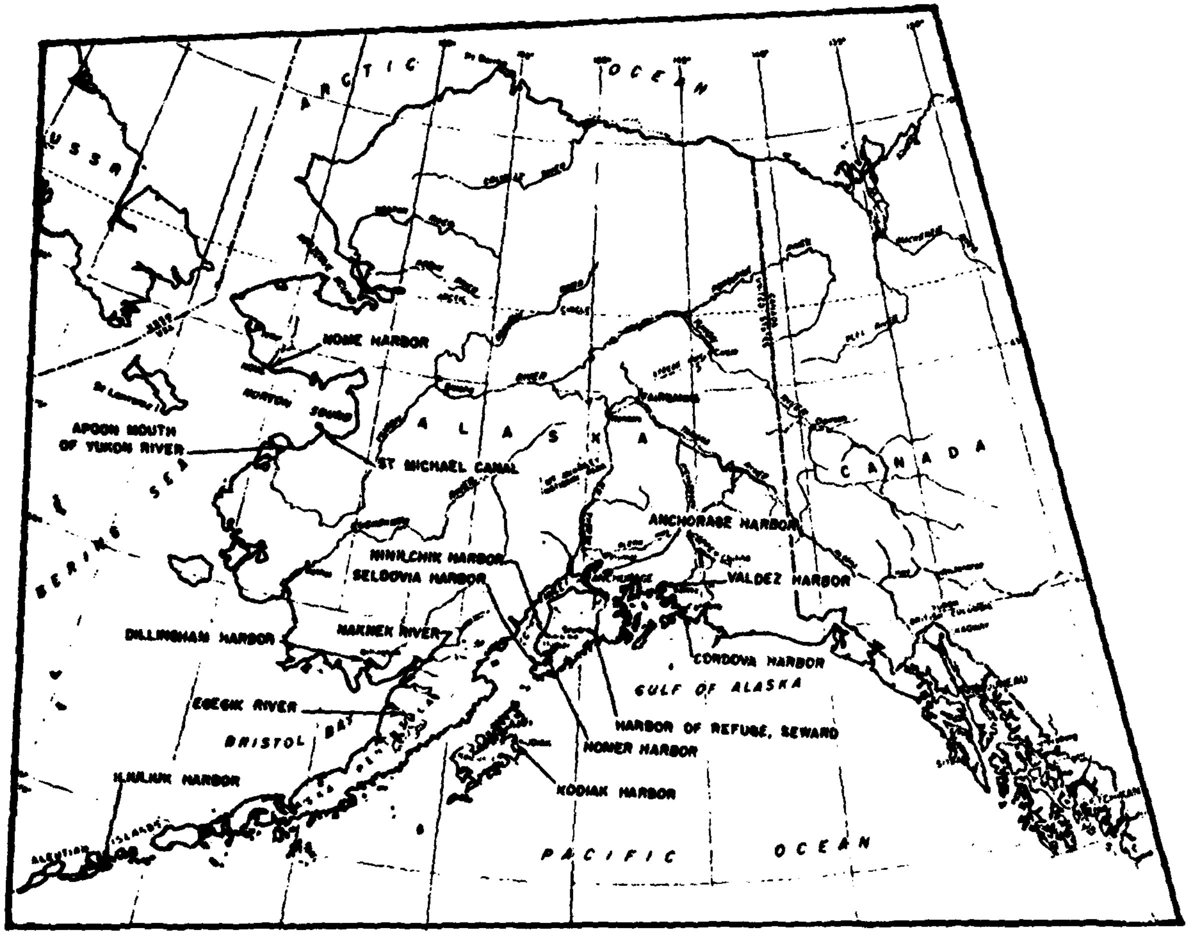

Figure 42. Alaska

what was a lagoon inside the barrier islands but is now largely a tidal flat with surface sediment of sand. This river delta is near the eastern end of the area affected by the 1964 earthquake and experienced immediate erosion and scour of uplifted mud flats. Anchorage is located on Cook Inlet approximately 80 miles west of the epicenter of the earthquake, and despite its being above the effects of the waves, it was severely damaged. The chief reason for the destruction at Anchorage was the layer of finely laminated clay deposit underlying the outwash gravels. This clay has a property which makes it flow like a liquid when vibrated.

223. The Alaska Range on the northwest side of Cook Inlet merges with the Aleutian Range to form the Alaska Peninsula, which separates the Pacific Ocean from Bristol Bay on the Bering Sea. All of the peninsula was covered by glaciers during the Pleistocene ice age. The southern coast of the peninsula is very rugged because of volcanic cones and sculpturing by the glaciers.

224. The Alaska Peninsula terminates in the western direction in the island arc system known as the Aleutian Islands. The islands were originally built by volcanoes in the Pleistocene age and postglacial time on foundations of older rock, mostly volcanic. Starting with a single cone rising out of the ocean, successive eruptions formed a series of new cones. Ash blew to the shore and was washed by longshore currents to form tombolos, tying the islands together. Forest vegetation is almost wholly lacking in the Aleutians, because glacial erosion removed nearly all soil and weathered rock.

225. Ninety percent of the world's earthquakes occur in the Pacific Earthquake Zone, which includes on the north the southcentral part of Alaska, the Alaska Peninsula, and the Aleutian Islands. A small earthquake zone follows the coast range from Vancouver Island through southeastern Alaska. A major Alaskan earthquake zone runs from the Copper River valley through the Aleutian Islands, and most Alaskan earthquakes occur in this zone. Tidal waves often accompany earthquakes in the coastal areas of Alaska and tidal waves generated by Alaskan earthquakes often affect coastal communities in Japan, Hawaii, California, and Oregon.

226. Scour Problems. There are approximately 35 navigaton projects under the jurisdiction of the U. S. Army Engineer District, Alaska, 20 of which have required some type of major structure in the coastal zone or in semiprotected inlet waters. Most of these basins are small-boat harbors which are used for individual transport, for subsistence and commercial fishing, and for lighter moorage. These are shallow-draft transporters that are used to transfer cargo from oceangoing vessels which can anchor in deep water nearby, at most locations.

227. Tides in Alaskan waters vary from very high in parts of southeastern Alaska, Cook Inlet, and Bristol Bay to essentially zero in the Arctic Ocean. Most of Alaska has two tides per day. In addition to lunar tides, some parts of western Alaska also are subjected to wind tides caused by strong westerlies that can raise the sea level several feet on shallow shelving coasts. Strong tidal currents persist during maximum ebb and maximum flood flow.

228. Unique problems that do not exist elsewhere are associated with construction in the Alaskan coastal waters, and their solutions also are unique. However, these problems are well recognized and well respected by those responsible for operations in this region. These problems include large waves, strong tidal currents, very large tide ranges, extreme cold, many types of ice problems, and high sedimentation rates. Because of the potential for disaster associated with working in such adverse conditions, construction safeguards are clearly spelled out in contract specifications. Among the routine restrictions are limitations on the hours of the day when placement of stone or other material will be permitted due to the probability of strong tidal currents. Also, limitations are sometimes imposed because of wave characteristics. It is common practice to specify a 3-ft-thick foundation blanket composed of quarry-run stone both to serve as a bearing surface for the mass of the structure and to combat the scour potential of the anticipated strong tidal and wave-induced currents. Because of the prior planning and other precautions that are absolutely essential under these environmental conditions, damage to major structures while under construction has been minimal.

229. A typical example of the design and magnitude of a representative small-boat harbor is that of Ketchikan Harbor (Figure 43). This is the first major city reached in Alaska when sailing north from Seattle. It is located on the inside passage of the Panhandle, about 650 miles north of Seattle and about 250 miles south of Juneau, the capital of Alaska. Ketchikan Harbor, a major fishing port, site of a wood pulp mill and attendant logging operations, and a distribution and transportation center for southeastern Alaska, is located in Tongass

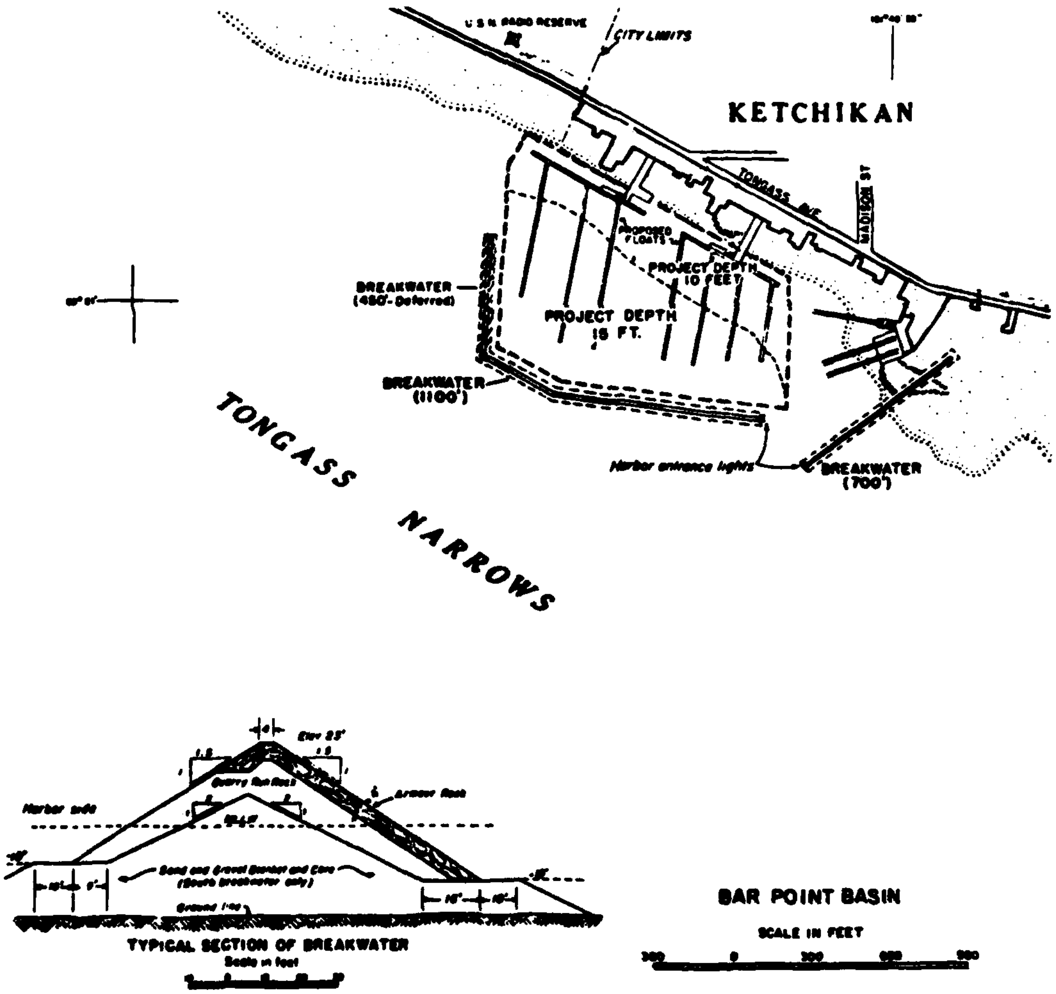

Figure 43. Bar Point Basin, Ketchikan Harbor, Alaska

Narrows off the southerly end of Revillagigedo Island. Despite the adequacy of port facilities for oceangoing vessels, a definite need existed for sheltered moorage for small craft as a result of extensive fishing industries in the area.

230. The project at Thomas Basin, Ketchikan Harbor, adopted in 1930, provided for a small-boat basin approximately 13 acres in area dredged to a depth of 10 ft at mllw. The basin was protected on the south by a concrete-capped rock breakwater 950 ft long and 23 ft above mllw. Subsequent to 1933, the completion date, shoaling induced by the flow of the Ketchikan Creek into the basin has required maintenance dredging. In 1954 the project was modified to include a second basin at Bar Point with three breakwaters, 700, 450, and 1,100 ft in length. Construction, except for the 450-ft west breakwater which is deferred indefinitely, was completed in 1958. Present status of the Bar Point Basin, Ketchikan Harbor, is shown in Figure 44. Sand and gravel were used not only for the foundation blanket material but also as the central core element for a large portion of the structure.

231. On the other extreme end of southern Alaska, in the Aleutian Islands, a small-boat harbor was constructed in 1973-74, which included an 11-acre anchorage basin and a 100-ft-wide entrance channel about 450 ft long, a 200-ft rock groin to protect the entrance, and the development of a 1,500-ft-long training dike to deflect tidal flows away from the area. Tides on the order of 12 ft occur regularly, and serious problems were anticipated in constructing the training dike in the strong current field generated by this tidal flow. Construction was carried out under frosty and light ice conditions. A 3-ft-thick foundation blanket of quarry-run stone was placed under the structure. Advancement would continue until the tidal current became so strong that material began to move from the area; work would then cease until the tidal flow change would cause a decay of the magnitude of the current, at which time construction would resume and work would continue as far as possible before an increased tidal current again made construction advancement impossible. By scheduling the hours of contractor operation in this manner, it was possible to extend the training dike across an

Figure 44. Present status of Bar Point Basin, Ketchikan Harbor, Alaska. (1975 photograph by U. S. Army Engineer District, Alaska)

area of strong tidal currents with only minimal loss of material due to scour or erosion during the actual construction operation.

232. Other unique construction elements have been utilized as temporary solutions until the permanent situation could be finalized. For example, operations are frequently conducted on frozen soil or under icy conditions where a quagmire would otherwise prevent any progress from being accomplished. Under other circumstances at Fairbanks, it was required to divert the river into a new channel. Problems arose with preventing the flow from continuing down the old channel even after the cutoff plug had been removed. In this situation, large chunks of ice were placed into the old existing channel and covered with mud and soil and successfully served as channel blockage temporarily until the permanent levee could be installed. These solutions are unique to the State of Alaska, where the problems are dissimilar from those of the other

states; hence, these techniques probably cannot be extrapolated elsewhere, nor can solutions to problems in the other states necessarily be expected to be viable in Alaska.

PART III: QUANTITATIVE ESTIMATE OF SCOUR EFFECTS

233. An indication of the magnitude of the erosion and scour problem can be obtained by analyzing records of both ongoing construction and previously completed projects. Not only will operation time be lost while scour holes are filled with material quantities usually unaccounted for in original estimates, but also the prime stonework contractor may require downtime compensation for those days when stone placement is not possible due to excessive scour and erosion. The following examples serve to illustrate typical magnitudes of representative scour and erosion problems that have occurred during construction in the surf zone.

Murrells Inlet South Jetty

234. Construction of the south jetty at Murrells Inlet, South Carolina, discussed previously, involved crossing and closing an ebb channel. Tidal currents are relatively low at Murrells Inlet, and no scour problems were encountered during construction of the north jetty. Scour was more severe than anticipated at the south jetty, and the circumstance giving rise to this situation has been documented by Lesemann*:

> "A low pressure area passed through the coastal South Carolina area on 25 April 1979, accompanied by high winds and higher than predicted tides. The event coincided with the occurrence of spring tides, scheduled to be 1.2 ft above normal and 1.0 ft below normal. The actual still water elevation reached +6.8 ft or 2.3 ft above normal due to the high winds.
>
> Considerable beach erosion occurred during the period of 24-25 April and there was significant shifting of the sandbars and shoals within the inlet complex. A portion of the sand fillet that had built up on the north side of the north jetty was lost. The sand berm inside of the weir was also decreased in size. One of the weir warning signs was lost. The two large

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* J. J. Lesemann, Chief, Engineering Division, U. S. Army Engineer District, Charleston, Memorandum for Record, 4 May 1979.

creosoted posts snapped off at the sand line apparently due to waves hitting the sign (bottom elevation +11.0 ft mlw) and the supporting posts. The seaward side of the land cut at inner channel A moved into the channel about 15 to 30 ft, actually moving the channel by about that much. Sand was also lost from the seaward side of the south sand dike. The south jetty at that time extended to about station 36+50.

The south jetty construction had been progressing at a rate of about 6.4 ft per working day in a wide area with a bottom elevation of about -8.0 ft mlw. Construction had reached to about two thirds of the way across the old channel area which was approximately 600 ft wide with depths ranging from 8 to 10 ft at mean low water.

The highest tides and winds occurred in the late afternoon tide on 25 April. Very strong ebb tides were noted on the 25th and 26th and investigations indicated that a scour hole about 15 ft deep mlw had developed at the end of the south jetty. It was noted that the ebb tide was impinging on the outer end of the sand dike and the inner portion of the jetty, then running parallel to the jetty and around the end. This action had actually been occurring ever since construction of the south jetty had reached about midpoint in the former 600 ft wide channel area. It had been causing the shoal opposite the outer end of the jetty to retreat, but no significant scour hole had developed and progress was being made on narrowing the opening. After 25 April, forward progress was greatly slowed due to the considerable excess stone now necessary due to the 15 ft scour hole. The reduced advancement of the jetty longitudinally allowed more time for the shoal beyond the jetty to retreat, resulting in little progress in making closure.

The pilot channel, about midway between inner channel A and the 10 ft ocean contour, had shoaled to a -4.0 ft mlw depth prior to the storm; however, the extent of shoaling was somewhat greater after the storm. Maintenance dredging of the pilot channel began on 30 April, using a 16 in. pipeline dredge and the material is being disposed of along the upcoast nourishment area. A decision was made to excavate a 125 ft bottom width rather than the previous 90 ft width in order to increase the flow through this area, and hopefully decrease the volume and velocity of ebb flow against and around the south jetty. Dredging of the outer pilot channel should be completed by 9 May.

Placement of jetty stone was stopped on 4 May. A decision was made that sand would be pumped into the area between the end of the jetty and the shoal (dries at low water) now existing about 250 ft from the end of the jetty. Foundation stone will be placed in a 4 ft blanket at the end of the jetty up to elevation -6.0 ft mlw. Starting about 9 May, the dredge will move to an inner portion of the pilot channel, in an area to be widened, where it will be cutting into a solid bank and therefore pumping at its highest rate. This sand will be placed in the area between the dry shoal and the jetty end, up to an elevation of about -6.0 ft mlw. When this is completed, the foundation stone blanket will be extended 50 ft in front of the jetty, at which time the placement of cover and armor stone will resume. The foundation blanket will be kept as far ahead of the jetty stone as possible."

235. Three proposals were solicited from the dredging contractor to determine an equitable payment for this unorthodox effort: (a) cubic yard rate, (b) rental rate per day, and (c) lump sum quantity. It was decided that a rental rate of $6,000 per day of effective pumping time (10 hr) plus an additional lump sum of $10,800 for mobilization and demobilization of the dredge plant which was already in the vicinity was reasonable. Subsequent performance indicated four working days were necessary to bring the scour hole elevation up to about -6.0 ft mlw. Additionally, since the stone contractor was being subjected to a stop-work order due to circumstances beyond his control, negotiated compensatory settlement was reached at $2,000 per weekday and $1,100 per Saturday while the stopwork order remained in effect (15 weekdays and 4 Saturdays). While the stone contractor was quite willing to place all the stockpiled stone into the scour hole, such a procedure would have quickly resulted in the stockpile becoming exhausted, and the contractor would have then had to wait for additional material. Thus, it was in the interest of both the contractor and the Government to negotiate a settlement for the period of time necessary to fill the scour hole.

236. The cost necessary to conduct this emergency construction procedure was approximately $70,500, although this effort only brought the groundline up to 4 to 5 ft below the original survey from which

bids were solicited. At this particular section, stone quantities in excess of the estimate by approximately 70 percent were placed.

237. The elevation of the groundline in front of, and adjacent to, the jetty construction continued to remain a few feet below the original survey as work resumed and progressed through mid-July 1979. At that time it was determined that the degradation was excessive, with scour holes about 10 to 12 ft deep. A stopwork order was issued on 18 July 1979 so that the scour holes could again be pumped up with sand from the dredge. This was considered essential because of the extremely high cost of stone. Stone placement was resumed in mid-August after additional expenditures of approximately $70,000 were necessary to bring the bottom elevation up to a satisfactory working elevation.

238. Thus, it appears that as of August 1979, approximately $140,000 has been spent filling unanticipated scour holes with sand between sta 36+50 and 42+50, a distance of about 600 ft. Additional stone quantities (in excess of the original estimate by about $150,000) also have been necessary. Hence, direct costs attributable to scour and erosion during construction of the Murrells Inlet south jetty approximate $300,000 at the present time (an additional 2,000 ft of jetty remains to be constructed). The total cost of this scour problem will not be known for several months.

Baptiste Collette Bayou Experimental Jetty Construction

239. The coastal region of the State of Louisiana is so distinctly different from other areas of the nation that special investigations are often necessary. The tremendous sediment load carried to the Gulf of Mexico by the Mississippi River results in a highly unconsolidated foundation upon which structures must be erected. The most unstable part of this coast is the region from the shores of Lake Borgne to Timbalier Bay, including the Mississippi River outlet passes. Because of the inherent nature of the foundation characteristics of the entire region, erosion and scour near structures can be all-inclusively considered as "scour-settlement" problems. Outlets south of Venice, Louisiana, both in an

easterly and westerly direction, will provide stable navigation channels protected by parallel jetties to reduce maintenance dredging.

240. Baptiste Collette Bayou connects the Mississippi River with Breton Sound and is located directly east of Venice, Louisiana. The contract for this experimental jetty construction was awarded in late 1978, and provided for the placement of 83,000 tons of stone in a specified manner in conjunction with various filter fabrics. The purpose of this experimental prototype testing is to determine which combination of filter fabric, location of filter fabric, and stone sizes successfully and significantly reduce the amount of settlement into the relatively unconsolidated foundation. The contract was completed in late May 1979, with the following stone quantities placed:

	Estimated Quantity, tons	Placed Quantity, tons
Graded stone A	14,000	14,054
Graded stone B	61,600	95,086
Graded stone C	7,500	7,866
Total	83,100	117,006

This resulted in a 41 percent increase in stone quantity beyond that originally estimated. At a unit cost of $13.25 per ton for all stone sizes, the estimated cost of the stone (originally $1,101,075) was increased to $1,550,324. This variance of $449,249 can be directly attributable to the purchase of additional quantities of stone necessary to fill scour and erosion areas and to replace stone that settled into the foundation before final grade was attained.

Tiger Pass Jetty Construction

241. Tiger Pass connects the Mississippi River with the Gulf of Mexico and is located directly west of Venice, Louisiana. The contract for this jetty construction was awarded in September 1978, and provided for the placement of 82,000 tons of graded stone with filter fabric installation to reduce settlement into the unconsolidated foundation. The contract was completed in March 1979, with the following quantities of stone actually placed.

	Estimated Quantity, tons	Placed Quantity, tons
Graded stone A	2,000	2,030
Graded stone B	63,000	86,454
Graded stone C	17,000	16,719
Total	82,000	105,203

This 28 percent increase in stone quantity beyond the original estimate, at a unit cost of $12.75 per ton, resulted in increased construction costs of $295,838. This increased cost is attributable to quantities of material necessary to fill scour areas and to provide additional material to replace that which settled into the unconsolidated foundation prior to attainment of final grade.

Ventura Marina Breakwater

242. Ventura Marina, a man-made harbor located 6 miles northwest of Channel Islands Harbor on the coast of southern California, was completed by local interests in 1963. The harbor was plagued with disasters or near-disasters for the next six years which closed the facility an average of 66 days each year. During this time, hazardous conditions were caused by breaking waves in the harbor entrance and excessive buildup of sand. Harbor improvements authorized in 1968 and completed in 1972 consisted of a new detached breakwater and dredging to form a sand trap.

243. The nearshore sediment in this region is predominantly sand. There is no bulge in the shoreline at the mouth of the Santa Clara River because after major floods most of the new sediment drifts southeastward along the shore, and the finer silts and clays are mostly carried beyond the 30-ft depth. Hence, the resultant nearshore physiography is essentially firm sand.

244. As previously discussed, since it is known a priori that a larger amount of stone will be necessary than that determined by template projections, the philosophy of the Los Angeles District is to apply a computational correction factor based on an analysis of past stone construction contracts. This is equivalent to using a void ratio of 26

percent instead of 35 percent, and a unit weight of stone of 170 lb per cu ft instead of 165 lb per cu ft, and effectively advertises an after-scour quantity for bids. In order to provide the same basis for comparison, the following estimated quantities were developed assuming no scour would occur.

	Estimated Quantity, tons	Placed Quantity, tons
Capstone A-13	99,057	116,489
Capstone A-7	13,208	14,020
Toe stone B-1	15,094	7,875
Core stone B	131,709	108,701

The increase of $62,159 above the original contract estimate of $1,638,065 for the stone work of this project represents approximately a 4 percent increase in cost.

Dana Point Harbor Breakwaters

245. Dana Point Harbor, completed in 1970, consists of breakwaters, entrance and interior channels, an anchorage area, and a turning basin. The harbor was planned and coordinated with redevelopment of Doheny State Beach. It provides an important link in the chain of harbors of refuge along the Pacific coast. Mooring spaces were available for 2,200 small craft in 1977. The project was model-tested in a physical hydraulic model at WES (Dai and Jackson 1966), and later the resulting prototype received the 1972 Chief of Engineers Distinguished Design Award for Civil Works.

246. Dana Point appears to be located at the northern end of the Oceanside Littoral Cell, and there seems to be no indication of littoral material moving around the point. There is evidence, however, of a general net southerly movement of sand farther south along the beach. Hence, there is probably no accumulation of fine material in this region, and the nearshore area is composed of firm sand. The following estimated quantities were again developed, assuming no scour activity would occur.

	Estimated Quantity, tons	Placed Quantity, tons
Capstone A-12	367,925	428,800
Capstone A-6	148,113	180,540
Capstone A-3	29,245	34,325
Core stone B	658,546	593,908
Core stone C	294,624	290,062

The unit cost for all stone quantities of this contract was $7.65; hence, the increased cost of $515,818 represents both a 6.5 percent excess quantity and excess cost beyond the original estimate.

Ponce de Leon Jetty Construction

247. Ponce de Leon Inlet is located on the east coast of Florida about 65 miles south of St. Augustine Harbor and about 57 miles north of Canaveral Harbor. The spring tidal ranges are 4.9 ft in the ocean and 2.7 ft just inside the inlet; therefore, significant tidal inlet currents can be established sufficient for scour and sediment transport. Prior to the improvement plan initiated in mid-1968, engineering operations to improve navigation conditions in the inlet had been minimal.

248. The inlet stabilization plan has been documented by Partheniades and Purpura (1972), and consists of an entrance channel, north and south jetties, and an impoundment basin adjacent to the north jetty. It is estimated that the wave climate in this region is sufficient to transport a gross drift rate annually of 600,000 cu yd to the south and 100,000 cu yd to the north (Walton and Dean 1973). The beach sand is clean, fine, relatively uniform, with a mean grain size of around 0.2 mm; hence, the material is highly susceptible to erosive forces. The north jetty consists of 500 ft of concrete sheet piling near the beach, an 1,800-ft weir section, and 1,750 ft of rubble-mound breakwater. The south jetty is 3,800 ft long and is composed entirely of rubble-mound stone of various sizes.

249. Severe scour on the order of 10 ft occurred as construction proceeded seaward, and material overruns of as much as 300 percent were experienced along some sections of the jetties. The original estimated quantities for the north and south jetties were:

	Estimated Quantity	
	North Jetty	South Jetty
Foundation stone	11,600 tons	18,600 tons
Core stone	300 tons	10,000 tons
Capstone (+8 tons)	29,100 tons	39,900 tons
Capstone (1/2 to 1 ton)	0	12,600 tons
Filter stone	0	1,600 tons
Concrete sheet piles	3,600 lin ft	0

Based on the low-bid unit prices, the estimated cost of this project (including dredging the navigation channel) was $2,304,600. Because of the severe scour and erosion that occurred during the construction, the final cost escalated to $3,980,740, an increase of 73 percent, or $1,676,140. It is considered by some authorities, however, that the additional stone creates a much more stable structure (i.e., possibly resulting in some benefit, namely less or no maintenance, because of the initial cost overrun).

Buffalo Disposal Dike No. 4

250. A primary construction scour problem arose during the construction of Disposal Dike No. 4 at Buffalo, New York. In order to keep the core stone from settling into the unconsolidated foundation, the soft sediments were to be bridged with a 6-ft thick layer of sand composed of minus 3/8 in. to No. 200 sieve size material. Storm conditions arose, and large waves caused mass transport of water to pile up against adjacent existing breakwaters; and from continuity, the return flow to the main lake created currents sufficient to scour this sand underlayment from the designed area. The only apparent solution to this problem was the rapid placement of additional volumes of sand until the stable slope was developed. Not only was the placed sand being carried from the region, but scouring of undisturbed sections of the dike location also occurred. Thus, in addition to the excess sand required to replace that which was transported away (Granular Fill G), supplemental stone was necessary in some other sections.

	Estimated Quantity, tons	Placed Quantity, tons
Stone A	127,000	142,929
Stone B	151,000	147,503
Stone C	69,000	81,067
Blast furnace slag	223,000	180,763
Stone K	22,603	21,234
Slag type F	48,600	55,794
Granular fill G	408,101	471,081

The final contract payment for the completed project was $15,291,126. This represented an increase of $699,776 above the estimated construction cost of $14,591,000, or approximately a 5 percent increase that can be directly attributable to scouring effects of lake bottom currents which could not be accounted for in the original design. This was indeed a boundary condition problem caused by setup of waves that developed return flow currents resulting in scour of the foundation.

Jetties at Mouth of Colorado River, Texas

251. The proposed plan of improvement for the navigation and recreation features of the Mouth of the Colorado River, Texas, Project (Galveston District 1977) to be constructed in FY 1980, provides for a jettied entrance channel 15 ft deep and 200 ft wide, a navigation channel 12 ft deep and 100 ft wide along the alignment of the existing Colorado River flood discharge channel to the Gulf Intracoastal Waterway near Matagorda, and a harbor and turning basin. The jetty system for the project includes a weir jetty on the eastern side of the entrance channel and an impoundment basin adjacent to the weir to trap westerly moving littoral material. The east jetty will be 2,650 ft long, and the west jetty will be 1,450 ft long.

252. The procedure developed by the Galveston District from an analysis of historical construction contract records for the region is to compute the quantities of material that would exist above a hypothetical groundline uniformly 3 ft below the actual groundline, and extending from the surf zone to the end of the structure in the seaward direction. It has been observed that when working conditions are good,

the rate of construction progress will be rapid enough so that the amount of scour or displacement in front of the structure does not exceed this 3-ft approximation.

253. The quantities of materials that will comprise the jetties in this location have been estimated by the Galveston District, including the assumed scour quantities, and consist of the following:

	Estimated Quantity, tons
Foundation blanket stone	61,600
Core stone	97,600
Filler stone	11,700
Cover stone	76,900

Based on 1977 price levels, the estimated cost for placing these amounts of material was $5,832,840. The effect of the potential scour development has been to add $1,477,433, or approximately 25.1 percent to this estimate.

Destin East Pass Jetties

254. The problems which arose during the construction of the jetties at Destin East Pass, Florida, may be attributed in part to scour and erosion effects, and in part to poor construction techniques on the part of the contractor. Construction problems began to appear when improperly threaded bolts connecting timber wales failed, and waves breaking onto the sheet piling caused separation of some of the piles sufficient to allow currents and littoral material to pass. Scour holes up to 14 ft deep developed at the weir, and ebb currents on the order of 6 to 10 fps were not uncommon. Additional scouring occurred all along the jetties. The estimated and placed quantities of material on this project were as follows:

	Estimated Quantity, tons	Placed Quantity, tons
Capstone + core stone	42,200	61,039
Blanket material	36,500	24,196

255. Because a significant amount of foundation blanket material

was being swept from the project area before it could be stabilized with core stone, the procedure of rapid placement of core material was utilized, and some of the foundation material was not applied. It was replaced, however, with core and cover stone that cost about 20 percent more per unit volume. The value of the placed material exceeded the estimated cost by 13 percent, or $92,623. An additional $40,117 was exceeded on other contract items which, coincidentally, produced the same percentage increase in the total contract payment of $1,123,155.

Tillamook South Jetty

256. One of the most severe cases of scour during construction ever experienced occurred during the construction of Phases I and II of the south jetty at Tillamook Bay entrance channel. The configuration is similar to that at Murrells Inlet in that the south jetty would extend from the shore across a flood channel and onto a large shoal area. The original basic justification for the south structure was that it should act as a breakwater to protect the bay from southerly waves. Uncertainty existed as to the extent of this feature; i.e., it was not known how far into the ocean the jetty or breakwater should extend in order to have optimum effectiveness. The decision was made that "incremental construction" would be employed, with the end of the structure being located when sufficient protection had been obtained.

257. The initial authorization provided for up to 8,000 ft of jetty, and construction of Phase I was started in 1969, providing for a 5,000-ft section of this initial effort. However, local interests indicated the belief that the spacing was too wide for the relative length and therefore would not assist in maintaining channel dimensions. The problem was investigated by physical model testing at WES (Fisackerly 1974), and the Phase II extension was recommended and started essentially upon completion of Phase I. This provided for an additional 1,300 ft of jetty, which was completed in 1972. Local interests were again concerned about the excessive amount of wave energy that was being transmitted into the navigation channel. A Phase III jetty extension effort

was initiated in FY 78 to extend the jetty to its authorized length of 8,000 ft, thus providing for an extension of 1,700 ft, and is scheduled for completion in the fall of 1979.

258. During the Phase I construction, it had been anticipated that 3 to 5 ft of scour could result from the wave climate known to exist in the area and because of the contribution to erosional effects by the strong tidal currents known to exist through the inlet. Scour of an unexpected magnitude began to undermine the end of the structure, and the scour hole proceeded to migrate at the same rate as the new jetty construction. A scour hole about 40 ft deep developed, and heavy overruns of material were required to fill this hole. The technique of "accelerated core placement" was employed, and this reduced the depth of the scour hole by about 50 percent, and then later to about 6 to 8 ft. The quantities of stone estimated and actually placed were:

	Phase I	
	Estimated Quantity, tons	Placed Quantity, tons
Class A stone	227,249	326,139
Bedding material	198,688	198,888
Core stone	0	130,022
Total	425,937	655,029

The estimated quantity of material in Phase I was exceeded by 53.8 percent. However, toward the end of the Phase I effort, it was determined that the technique of rapid placement of the core stone modification to the contract had successfully reduced the scour overrun quantities by about 50 percent. The Phase II contract was awarded with estimated quantities reflecting the anticipated stone to be required to fill the holes caused by scour and erosion. Excluding these additional scour quantities, the estimated and placed quantities of Phase II were:

	Phase II	
	Estimated Quantity, tons	Placed Quantity, tons
Class A stone	266,250	266,529
Core stone	316,500	482,568
Select A stone	54,750	74,847
Total	637,500	783,944

In this phase of construction, the estimated quantities were exceeded by only about 23 percent. It should be noted that "accelerated core placement" is based on the calculated risk that a big storm will not occur before the armor stone is placed. The successful "accelerated core placement" technique at Tillamook occurred in the absence of any major storm. Otherwise substantial damage (possibly even total loss) would have occurred to the core placed ahead of the unarmored section.

259. Considering both Phase I and Phase II of the Tillamook south jetty construction, the estimated quantities were exceeded by 21.1 percent due to scour effects, or approximately $1,844,090.

Summation

260. The extent of the scour problem at any particular location is a complex functional relationship between many variables which interact and tend to obscure a complete understanding of the phenomena. Among the site-specific characteristics involved are soil-foundation parameters, hydrography, type and orientation of structures, and the wave climate. Hence, scouring action varies over a wide range, depending upon the predominance of certain of these pertinent variables, and is displayed by the aforementioned examples, summarized as follows:

Summary of Estimated Scour Effects at Typical Locations

Project	Estimated Cost due to Scour	Estimated Percent Increase due to Scour	Price Level	Estimated Cost due to Scour in 1979 Dollars*
Murrells Inlet South Jetty	$ 300,000**	**	1979	$ 300,000
Baptiste Collette Jetties	449,249†	41.0†	1979	449,249
Tiger Pass Jetties	295,838†	28.0†	1979	295,838
Ventura Marina Breakwater	62,159	4.0	1972	102,252
Dana Point Harbor Breakwaters	515,818	6.5	1970	909,387
Ponce de Leon Jetties	1,676,140	73.0	1971	2,837,705
Buffalo Disposal Dike No. 4	699,776	5.0	1977	815,939
Colorado River Jetties, Texas	1,477,433††	25.1††	1977	1,722,687
Destin East Pass Jetties	92,623	13.0	1970	163,294
Tillamook South Jetty	1,844,090‡	21.1‡	1972	3,033,528
Total				$10,629,879

* Based on Consumer Price Index referenced to 1967; The World Almanac and Book of Facts 1979, 1970--0.860; 1971--0.824; 1972--0.799; 1973--0.752; 1974--0.677; 1975--0.620; 1976--0.587; 1977--0.551; 1978--0.517; and assumed inflation rate of 10 percent for 1979.
** Construction still in progress; ultimate effect unknown at this time.
† Includes both scour and foundation settlement effects.
†† Construction expected to begin in late 1979; scour effects estimated.
‡ Phase I and Phase II.

PART IV: SUMMARY

261. Scour and erosion of foundation material around major structures constructed in the coastal zone have been well known and continuing problems since surf-zone work commenced. Over the years, those responsible for the integrity of such structures have developed construction techniques to minimize quantity and cost overruns, although in most cases these procedures are regional in nature. Because of varying wave and current conditions from one locality to another, those techniques which are optimum for one location may not be strictly applicable to another region. The procedures presently being used to combat the problem of scour in the coastal zone during construction, and to eliminate "secondary construction effects" which develop some time after completion of the initial construction work, and the underlying philosophy on which these procedures are based, are enumerated as follows:

a. It is universally accepted that most major stone structures require a foundation blanket for bearing surfaces to support the mass of the structure above, and to serve as scour protection during the actual construction. The thickness and design features of the blanket vary with location, but in general are on the order of 2 to 3 ft thick and extend on either side of the structure from 5 to 25 ft beyond the toe. In parts of the Great Lakes, a layer of sand has been placed initially on soft unconsolidated muds, and then covered with quarry-run stone. Off the coast of Louisiana, where shell is plentiful, a layer of this material is frequently used.

b. In recent years a wide variety of plastic filter fabrics have come into vogue, and these are gaining acceptance as the initial item to be placed on unconsolidated materials to prevent migration of the fine materials up into the voids of the larger stones and a resulting hole into which the larger stone may settle. A layer of crushed stone or shell should be applied next in order to prevent puncture or tearing of the cloth by heavier stone. The cloth may be layered in sheets for the usual application, or fabricated into bags or tubes for specialized features.

c. Foundation bedding materials should be placed ahead of the core construction at least 50 ft to prevent temporal scouring to undermine the working section. Some have found it expedient to place one-half the thickness of the bedding initially, and then to make a second application with the second half of the material. At the end of the construction day, a 30- to 50-ft section of bedding material should be applied to minimize overnight scouring. In the heavy wave climate of the Oregon coast, the foundation bedding layer is designed 5 ft thick and intended to extend beyond the toe of the structure for 25 ft.

d. "Accelerated core placement" has been utilized successfully in crossing scour holes susceptible to continuing scour. On major structures where the work will continue over more than a single construction season, no more core stone should be placed than can be armored that season. Core stone of quarry-run rock instead of sorted stone has been determined to provide better keying action and a more stable, overdesigned structure, and at lower unit cost as the contractor does not have to separate up to 80 percent of the materials.

e. Gabion units have been fabricated and placed in a continuous layer as foundation bedding material instead of a loose layer of crushed stone, to ensure that the bedding material will be evenly distributed even after structure settlement.

f. To a lesser extent, the use of Gobimat revetment material has been utilized in special cases of nearshore coastal structures such as seawalls, slope protection structures, and foreshore dike work.

g. On high wave energy coasts where erosion may occur some time after the initial construction, the extent of the ultimate scour is estimated, and the foundation is excavated to that depth (usually 2 to 6 ft in sand). When conditions prohibit the foundation excavation, an excess quantity of rock is placed on the lower slope and toe to fill any scour hole that may develop later. On the Hawaiian coast, it is frequently possible to excavate down to a firm coral foundation.

h. In emergency construction situations, scour has been minimized by filling ebb or flood channels with dredged material to allow construction operations to continue unabated.

i. The expeditious selection of the construction season, as well as the specification of a select series of days when tide predictions indicate favorable working conditions, has been found to contribute significantly to the successful completion of coastal structure work.

j. Many construction technique problems arise because of poor contractor procedures such as trying to work in adverse weather or wave condition of excessive height, or failing to have stock-piled an adequate amount of core material for rapid placement situations.

<u>k</u>. Weir sections in jetties normally are constructed of stone instead of sheet steel or concrete pile, as the pile sections are susceptible to scouring and undermining which can result in complete failure. Stone or rubble-mound weirs are more stable and, furthermore, can be "fine-tuned" to allow for changing characteristics near an inlet.

<u>l</u>. From an analysis of past construction records, some regions have developed procedures for estimating the magnitude of the overruns due to scour, on an over-all basis, and thus additional project quantities to be required because of the scour phenomenon. The void ratio or unit weight of the stone being used in the structure may be updated, or a uniform depth of scour for the length of the structure may be equivalently computed.

<u>m</u>. Unique construction operations have been applied in the State of Alaska that are not applicable elsewhere. Temporary solutions to scour problems have been developed which include working when soils are frozen, or using ice blocks covered with soil to temporarily prohibit currents from scouring the work area during construction.

262. Most of the above procedures are seriously hampered by the inability to predict the extent of scour to be expected. Development of effective predictive capabilities will greatly enhance the effectiveness of present procedures as well as provide a basis for the development of alternate procedures for controlling scour and erosion during construction, and the minimization of scour after the initial construction.

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Table 1

Probable Once-A-Year Significant Wave Height

at Selected Great Lakes Locations*

Locality	Probable Once-A-Year Maximum Wave Height, H_o ft	Period Range sec	Probable Direction of Approach	Probable Duration hr
	Lake Superior			
Brule River	20	9-11	NE	6
Carver's Bay	27	11-13	NE	6
Little Lake	22	10-12	NW	8
North Shore	15	7-9	E or NE	6
Grand Marais (Mich.)	25	11-13	NE	6
Eagle Harbor	29	13-15	N or NE	8
	Lake Michigan			
North Bay	9	4-5	NE or S	6
Milwaukee	13	5-6	E	5
Chicago	8.5	4-7	N	9
Muskegon	15	5-7	SW	10
Frankfort	17	4-7	SW or WSW	9
Kenosha	13	7-9	E	5
Manitowoc	11	7-8	E	5
Berrien County	11	7-8	W or NW	5
Indiana	12	7-8	N or E	6
	Lake Huron			
North Point	9	5-6	NE or SE	6
Harbor Beach	13	5-7	E	5
Port Huron	8	4-6	N	9
	Lake Erie			
Cleveland	9	5-6	W or WNW	6
Erie	9	5-6	W or WNW	6
Buffalo	11	6-7	W	8
Huron	11	6-7	W or WNW	6
Monroe	8	5-6	E or ENE	6
Reno Beach	5	4-5	E or ENE	6
	Lake Ontario			
Olcott	9	5-6	W or WNW	6
Oswego	11	6-7	W or WNW	8
Fair Haven State Park	11	6-7	E or ENE	6
Fort Niagara State Park	12	6-7	E or ENE	6

* After Great Lakes Basin Commission, 1975.

Table 2

Frequency of Maximum Short Period Fluctuations

at Selected Great Lakes Locations*

Lake	Gage Location	Maximum Rise ft	Frequency of One Such Rise, in Years
Superior	Two Harbors**	2.1	10.00
Superior	Marquette	2.8	43.50
Superior	Point Iroquois**	2.3	8.50
Michigan-Huron	Mackinaw City	1.7	13.50
Michigan	Ludington-White Lake**	1.2	4.25
Michigan	Calumet Harbor	2.8	29.00
Michigan	Milwaukee	2.3	36.75
Michigan	Sturgeon Bay Canal**	1.7	3.25
Huron	Harbor Beach	2.1	45.75
Huron	Fort Gratiot (Port Huron)**	2.5	9.50
Erie	Gibralter**	3.0	9.75
Erie	Toledo	4.5	10.00
Erie	Put-In-Bay**	2.4	8.50
Erie	Cleveland	2.7	46.50
Erie	Buffalo	8.4	48.25
Ontario	Oswego	2.1	9.50
Ontario	Tibbetts Point	2.9	15.75

* After Great Lakes Basin Commission, 1975.
** Comparatively short records.

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

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