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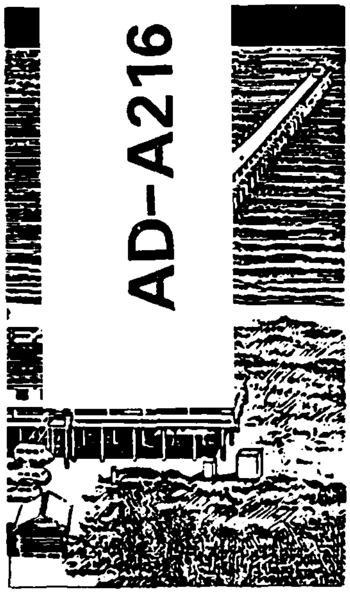

MISCELLANEOUS PAPER CERC-89-17

BOLSA BAY, CALIFORNIA, PROPOSED OCEAN ENTRANCE SYSTEM STUDY

Report 4

PHYSICAL MODEL

by

Robert R. Bottin, Jr., Hugh F. Acuff

Coastal Engineering Research Center

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

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A 1:75-scale (undistorted) hydraulic model of a proposed ocean entrance at Bolsa Bay, California, was used to investigate wave conditions in the entrance and interior basins of the proposed marina and sediment patterns along the coast as a result of the proposed jetties and breakwater. The model reproduced approximately 8,000 ft of the California shoreline, the proposed interior basins of the marina complex, a portion of the Wintersburg Flood-Control Channel, and sufficient offshore bathymetry in the Pacific Ocean to permit generation of the required test waves. An 80-ft-long unidirectional, spectral wave generator, an automated data acquisition system, a circulation system to generate steady-state flood and ebb tidal flows, and a crushed coal tracer material were utilized in model operation. It was concluded from test results that the originally proposed improvement plan with the navigable ocean entrance and connector channel to Huntington Harbour will not meet the established wave height criteria in the interior
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basins unless revetments along the interior channels and a 300-ft-long spur across the opening of the northwest basin are installed, in conjunction with raising the crest elevation of a 900-ft-long portion of the offshore breakwater from +18 to +22 ft. The lengths of the north and south wings of the offshore breakwater were adequate to prevent the movement of sediment into the entrances of the marina. It was also determined that discharges from Wintersburg Channel should have minimal impacts in the interior channels and basins of the marina complex. [illegible]

The originally proposed improvement plan with the navigable ocean entrance without the connector channel to Huntington Harbour also requires installation of revetments along the interior channels and a 300-ft-long spur across the opening of the northwest basin to achieve the established wave height criteria. Removal of a 750-ft portion of the north wing of the offshore breakwater resulted in a slight increase in wave heights in the interior basins and sediment deposits in the entrance between the north jetty and the north breakwater for test waves from the west.

For the non-navigable ocean entrance plan, sediment along the shoreline and in the breaker zone will move either north or south depending on the direction of wave approach. For waves from all directions, some material will bypass the new entrance and some will penetrate into the entrance channel. Deepest penetration will occur for waves from west, west-southwest, and southwest.

PREFACE

This report presents the results of a physical model study for proposed modifications at Bolsa Bay, California. The study was sponsored by the California State Lands Commission (SLC) through a Memorandum of Agreement between SLC and the Department of the Army signed July 2, 1987. This investigation was conducted by the US Army Engineer Waterways Experiment Station's (WES) Coastal Engineering Research Center (CERC) under authority of Title III of the Intergovernmental Cooperation Act of 1968.

The model study was conducted during the period December 1988 - April 1989 by personnel of the Wave Processes Branch (CW-P), Wave Dynamics Division (CW), CERC, under the direction of Dr. J. R. Houston, Chief of CERC; Mr. C. C. Calhoun, Jr., Assistant Chief of CERC; Mr. C. E. Chatham, Jr., Chief of CW; Mr. D. G. Outlaw, Chief of CW-P; and Dr. S. A. Hughes, Project Manager, CW. The tests were conducted by Mr. R. R. Bottin, Jr., Principal Investigator; Mr. H. F. Acuff, Civil Engineering Technician; Mr. D. M. Bell-Winston, Contract Student; and Mr. W. M. Henderson, Computer Technician. Messrs. Bottin and Acuff authored the technical sections of this report. Dr. L. Z. Hales, Wave Research Division, CERC, and Dr. Hughes prepared the background information common to all the Bolsa Chica study reports. During the course of the investigation, liaison was maintained by means of conferences, telephone communications, and monthly progress reports. Mr. Daniel Gorfain was project manager for SLC during the course of the study.

COL Dwayne G. Lee, EN, was Commander and Director of Waterways Experiment Station (WES) during the conduct of this investigation and COL Larry B. Fulton, EN, was Commander and Director during the preparation and publication of this report. Dr. Robert W. Whalin was Technical Director.

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CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENTS

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
acres	4,046.873	square metres
cubic feet	0.02831685	cubic metres
cubic yards	0.7645549	cubic metres
feet	0.3048	metres
inches	2.54	centimetres
knots (international)	0.5144444	metres per second
miles (US statute)	1.609347	kilometres
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square feet	0.09290304	square metres
square miles (US statute)	2.589998	square kilometres

BOLSA BAY, CALIFORNIA, PROPOSED

OCEAN ENTRANCE SYSTEM STUDY

PHYSICAL MODEL

PART I: INTRODUCTION

Bolsa Chica Modeling Studies

1. The State of California, State Lands Commission (SLC), is reviewing a plan for a new ocean entrance system as part of a multi-use project. This project involves both State and private property in the proposed development by the SLC, Signal Landmark, Inc., and others. The project, located in the Bolsa Chica area of the County of Orange, California, includes navigational, commercial, recreational, and residential uses, along with major wetlands restoration. The County of Orange has approved a Land Use Plan (LUP) in 1985 as part of the Local Coastal Program for Bolsa Chica in accordance with the California Coastal Act of 1976. This same LUP was certified by the California Coastal Commission (CCC) with conditions in 1986. Part of the LUP certification requirement to satisfy those conditions include confirmation review of modeling studies of a navigable and a non-navigable ocean entrance at Bolsa Chica.

2. In order to satisfy the CCC requirements for confirmation of the LUP, the SLC requested the US Army Engineer Waterways Experiment Station (WES), through a Memorandum of Agreement executed July 2, 1987, to conduct engineering studies on the technical and environmental assessment of a navigable ocean entrance system and a non-navigable ocean entrance system, as conditionally approved in the LUP. Results of these studies will assist SLC and other parties which are formulating reports and plans for the proposed Bolsa Bay project that meet the criteria set forth in Policies 23 through 26 of the LUP. These services were provided to SLC by WES under authority of Title III of the Intergovernmental Cooperation Act of 1968. As such, resultant study products are based on specific technical expertise only and should not be inferred to indicate support or non-support by the Corps of Engineers for either project involving a navigable or non-navigable ocean entrance, or for the environmental or economic aspects of these or any

other subsequent project.

3. Modeling studies of the Bolsa Chica area conducted by WES fall into four general categories:

a. Numerical modeling of long-term shoreline response as influenced by placement of entrance channel stabilization structures, including sand management concepts.

b. Physical modeling of the proposed entrance channel, interior channels, and marina with regard to wave penetration, harbor oscillation, and qualitative sediment movement paths.

c. Numerical modeling of tidal circulation, including transport and dispersion of conservative tracers, in the Bolsa Bay, Huntington Harbour, and Anaheim Bay complex.

d. Potential impacts of various ocean entrance designs on the local wave climate and, consequently, the potential impacts on recreational surfing activities at the proposed ocean entrance.

4. Detailed results of the modeling studies are reported in four separate reports. The report titles and a brief description of each report scope are given below.

Report 1: Preliminary Shoreline Response Computer Simulation

5. This report describes numerical model simulations of long-term shoreline position change as a result of longshore movement of sediment. Shoreline change simulations covering a 10-year period over the reach of coast from Anaheim entrance southward to the Santa Ana River are compared for a variety of conditions, including a non-navigable entrance, a structured navigable entrance without sand management, and a structured navigable entrance with sand management techniques. This study was conducted to determine a reasonable range of shoreline response to construction of an entrance system, and to evaluate the potential for mitigation of any adverse effects induced by the entrance.

Report 2: Comprehensive Shoreline Response Computer Simulation

6. This report describes numerical model simulations of long-term shoreline change under the same conditions as tested in the

modeling described in Report 1. The comprehensive modeling effort utilizes hindcast wave data obtained from the Wave Information Study (WIS) of the Corps of Engineers. These hindcast data represent the best available wave estimates for use in the shoreline model. Partial funding of the WIS hindcast at Bolsa Chica was provided by Signal Landmark as part of the overall Bolsa Chica Study. This report also contains a stability analysis of the proposed non-navigable entrance channel.

Report 3: Tidal Circulation and Transport Computer Simulation, and Water Quality Assessment

7. This report describes numerical model simulations of tidal circulation constituent transport in the Bolsa Bay, Huntington Harbour, and Anaheim Bay complex. A link-node model was calibrated and verified using data from the present configuration of the tidally-subjected region. The calibrated numerical model was then used to simulate a variety of proposed area developments, including increased wetlands, muted tidal areas, marinas, and navigation channels. Modeling provided results for the proposed navigable and non-navigable entrance alternatives, with and without a navigable connector channel to Huntington Harbour from Outer Bolsa Bay. Water Quality assessment is provided based on existing conditions and data, coupled with constituent transport modeling results. The transport modeling results provide estimates of water flushing and residence times which are used to project water quality parameters expected in the new wetlands configuration.

Report 4: Physical Model Simulation

8. This report describes results obtained from tests conducted in a 1-to-75 model-to-prototype scale physical model of the proposed Bolsa Bay entrance channel and marina complex. The purpose of the testing was to examine wave penetration into the marina basin and the resulting harbor wave conditions, to qualitatively study current circulation and sediment transport paths in the vicinity of the structures, and to assess the entrance channel and jetty design configuration. Physical model inputs included unidirectional irregular waves, steady-state flood and ebb tidal currents, and flood flows from the East Garden Grove-Wintersburg Flood Control Channel.

Purpose of the Physical Model Study

9. The most reliable means of determining short-period wave conditions

and wave-induced current patterns in a hydrodynamic environment is through the use of a physical hydraulic model. Physical models normally are used to determine conditions at a specific site and are not normally used for regional studies due to costs involved and the large model area required. An undistorted physical hydraulic model was designed and constructed at CERC to:

a. Determine wave conditions in the entrance and mooring areas of the proposed marina complex with the navigable entrance channel.

b. Determine wave conditions in the entrance of the proposed nonnavigable entrance channel.

c. Provide qualitative information on the effects of the breakwater and/or jetties on the deposition of sediment material moving alongshore in the vicinity of the project for both the navigable and nonnavigable entrance.

d. Determine the effects of flood flows from Wintersburg Channel as it enters into the marina complex.

e. Develop remedial plans for the alleviation of undesirable conditions as necessary.

Scope

10. The scope of work for this task as outlined in the Management Plan for the Proposed Bolsa Bay, California, New Ocean Entrance System Study includes the following:

a. Construct, and conduct studies by use of, a physical hydraulic model of the proposed navigable entrance channel and structures (jetties and breakwater), and non-navigable entrance channel (jetties, no breakwater) to determine if the jetty and/or breakwater locations, orientation, type, and dimensions are acceptable. The navigable entrance channel alternative will be evaluated with and without a navigable connector channel to Huntington Harbour.

b. Develop a plan that has acceptable wave climate in the channels and basins.

c. Qualitatively study the tidal (and flood flows from Wintersburg Channel) current circulation patterns and sediment transport paths in the vicinity of the structures.

d. Conduct the model study at an undistorted scale ranging between 1:50 and 1:75.

e. Develop interior wave climates by inputting wave data developed through the WIS program.

f. WES shall conduct tours of the model and shall maintain the model ready for additional testing for a period of one year after completion of testing, and thereafter, upon a request from SLC, maintain the model at a cost to SLC of approximately $10,000 per year until the space is required for Corps studies.

g. WES shall document its procedures, analyses, and findings in a report and shall support the report in connection with any local and state agency proceedings.

PART II: BACKGROUND

Description of the Bolsa Chica Area

11. Bolsa Chica is an unincorporated area of Orange County, California, located along the coastline approximately 9 miles* south of Long Beach and surrounded by the City of Huntington Beach (Figure 1). The Bolsa Chica project area (Figure 2) comprises approximately 1,645 acres, which includes the Bolsa Mesa and adjacent lowlands, and the shoreline adjacent to the Bay from the intersection of Warner Avenue and the Pacific Coast Highway (PCH) to the Huntington Mesa, located to the north of the intersection of Golden West Boulevard and the PCH. As discussed by the US Army Engineer District, Los Angeles (1987), the project area is bordered by bluffs on the northwest and southeast, and by the Pacific Coast Highway and Bolsa Chica Beach State Park on the southwest. Urban lands lie north and east of the project area.

12. The Bolsa lowland area is a remnant of a once-extensive tidal and river wetlands system of the mouth of the Santa Ana River which extended inland across the coastal plain to the surrounding mountains. Historically, the lowlands were frequently inundated by tidal flows through a direct natural connection to the ocean, and received fresh water from artesian wells and from local storm-water runoff. In 1899 tidal flow into the Bolsa Chica area was modified by construction of tide gates, and the natural channel to the ocean was eventually closed. The Bolsa Chica area was further modified in the 1920s by oil and gas interests, and construction of PCH. Subsequently, construction of the East Garden Grove-Wintersburg Flood Control Channel bisected the area, and its flow discharged into Outer Bolsa Bay and then into Huntington Harbour.

13. At present, tidal flow enters Outer Bolsa Bay and Inner Bolsa Bay (Figure 3) only through Huntington Harbour and Anaheim Bay (Orange County Environmental Management Agency 1985). Local runoff and precipitation provide the freshwater inflow. Dirt roads and dikes criss-cross the lowland connecting drill pads, oil pumping rigs, related structures, and pipe networks. Other existing improvements include the East Garden Grove-Wintersburg Flood Control Channel, bridges that cross the channel, tide gates at the confluence

* A table of factors for converting non-SI units of measurements to SI (metric) units is presented on page 3.

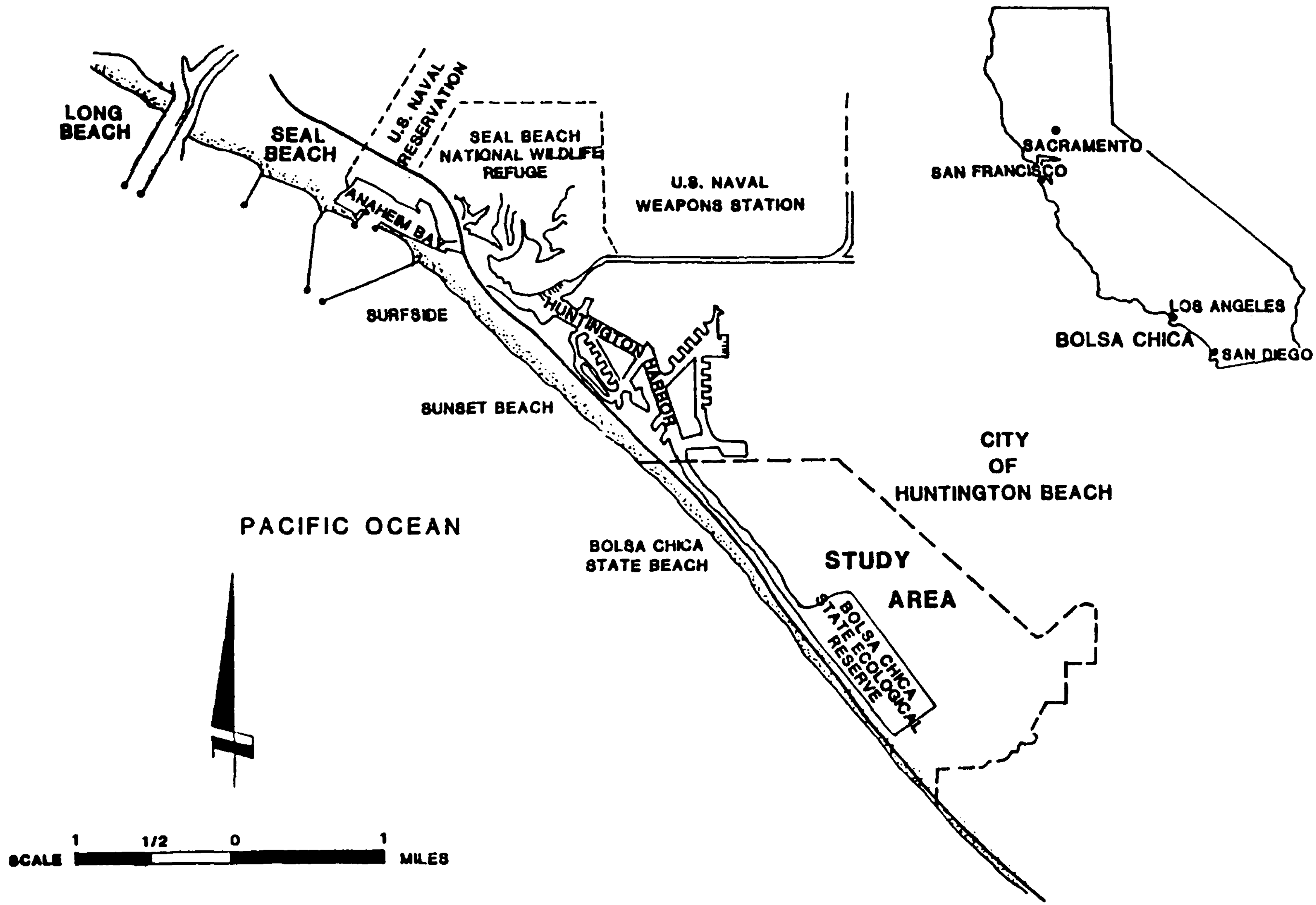

Figure 1. Bolsa Chica study region location

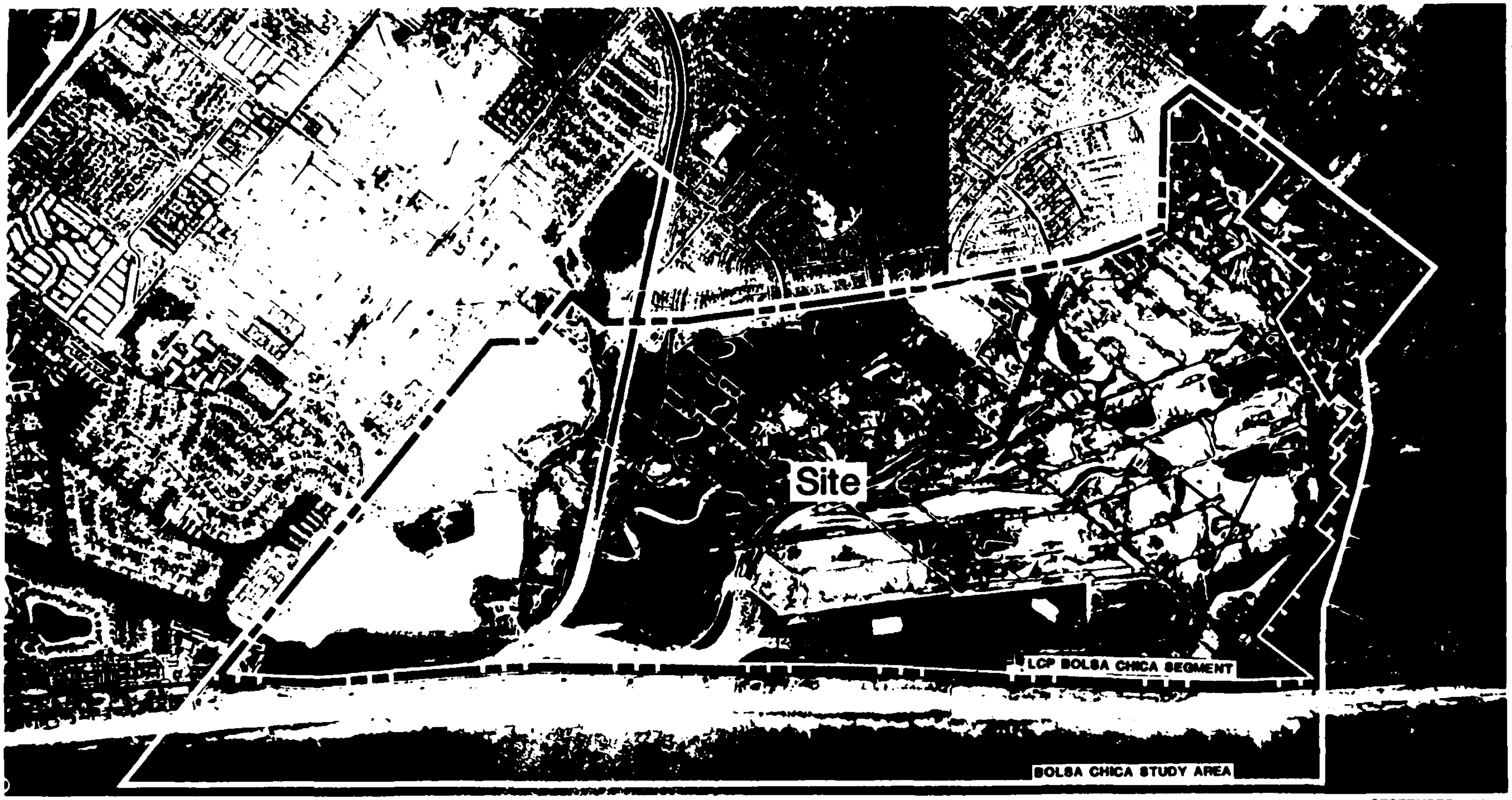

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Figure 2. Bolsa Chica area of interest

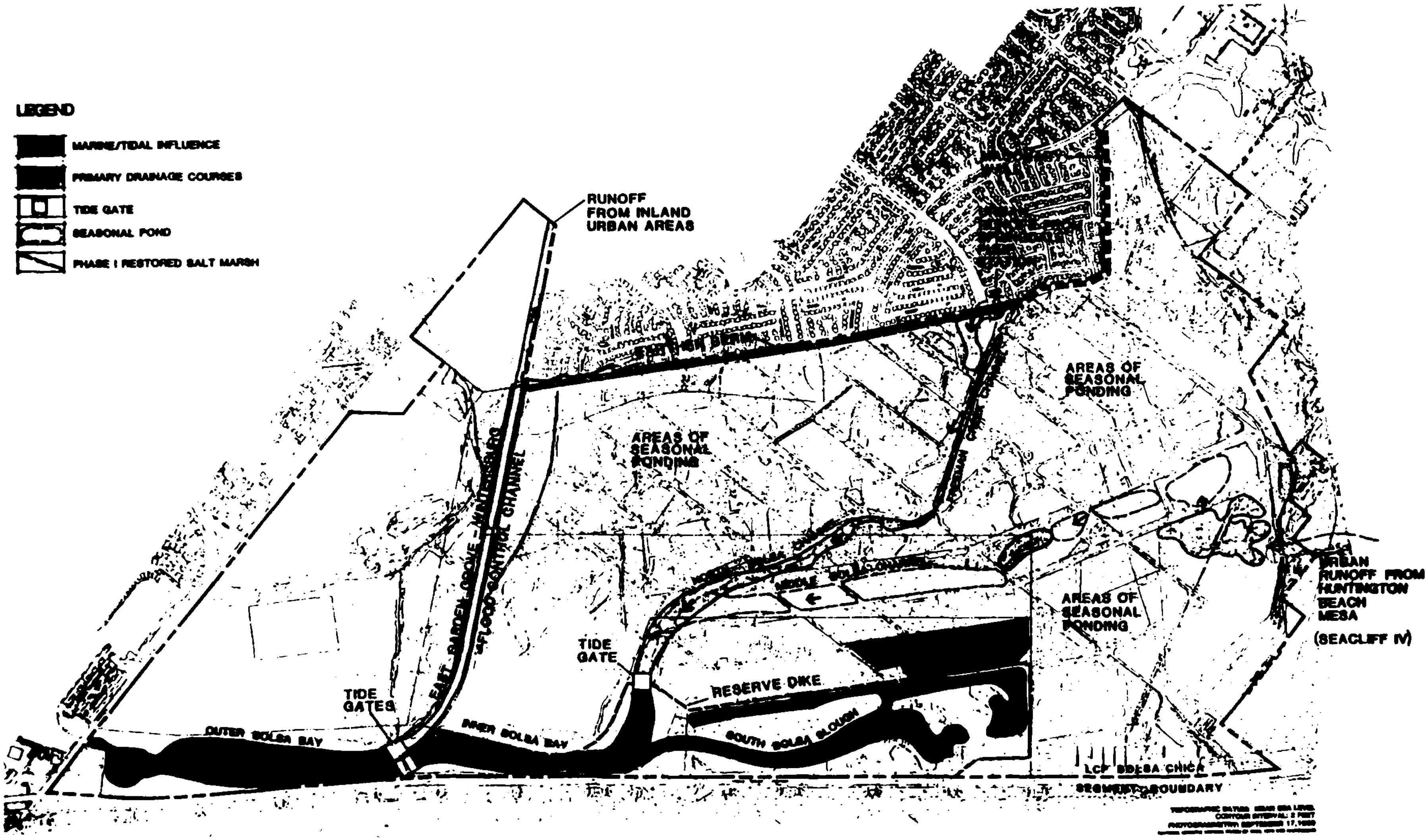


Figure 3. Present tidal inundation, Bolsa Chica

of the flood control channel and Outer Bolsa Bay, and a pedestrian walkway and footpath to the Bolsa Chica State Ecological Reserve from a public parking lot adjacent to PCH.

14. The community surrounding Bolsa Chica (the City of Huntington Beach), is predominantly a medium-density residential community. Bolsa Chica State Beach, on the ocean side of Bolsa Chica across the PCH, is utilized by both residents and visitors from outside the area. Recreational beach uses include sunbathing, swimming, picnicking, surfing, and hiking and bicycling along trails located along the seaward side of the beach parking areas. There is also a private equestrian facility with training facilities located in the northerly corner of the lowland. Recreational boating opportunities in the immediate area are located in the marina at Huntington Harbour, with ocean access being provided by the entrance to Anaheim Bay.

15. A 300-acre State-owned Ecological Reserve, of which 173 acres have been restored to high quality wetlands habitats, contains a limited amount of public footpaths for nature study. Public access into the majority of the Reserve is restricted to preclude unnecessary disruptions to wildlife values and use. An additional 230 acres adjacent to the Reserve is leased to the State of California by the major landowner of the area, Signal Bolsa Corporation (Figure 4). These lands would be conveyed to the State provided that the State causes the construction of a navigable ocean entrance and channel connecting to Signal lands, as part of the Bolsa Chica Land Use Plan. The Bolsa Chica lowland and existing wetlands in the Reserve provide important habitat both for migratory birds which nest, rest, and/or feed in the area, as well as resident shorebirds, waterfowl, and other vertebrate and invertebrate wildlife.

16. The County of Orange has adopted a Land Use Plan for the Bolsa Chica Project pursuant to State requirements under the California Coastal Act of 1976. The plan was certified by the Coastal Commission in January 1986, subject to review and confirmation of five elements. The certified Land Use Plan contains both urban and wildlife uses that yield more than 75 percent of the area as public use and other public open space. This certified Land Use Plan includes 915 acres of restored wetlands, 86.8 acres of additional environmentally sensitive habitats, a 1300-slip public marina with land provided for an additional 400 dry-stored boats, public launch ramps, and commercial

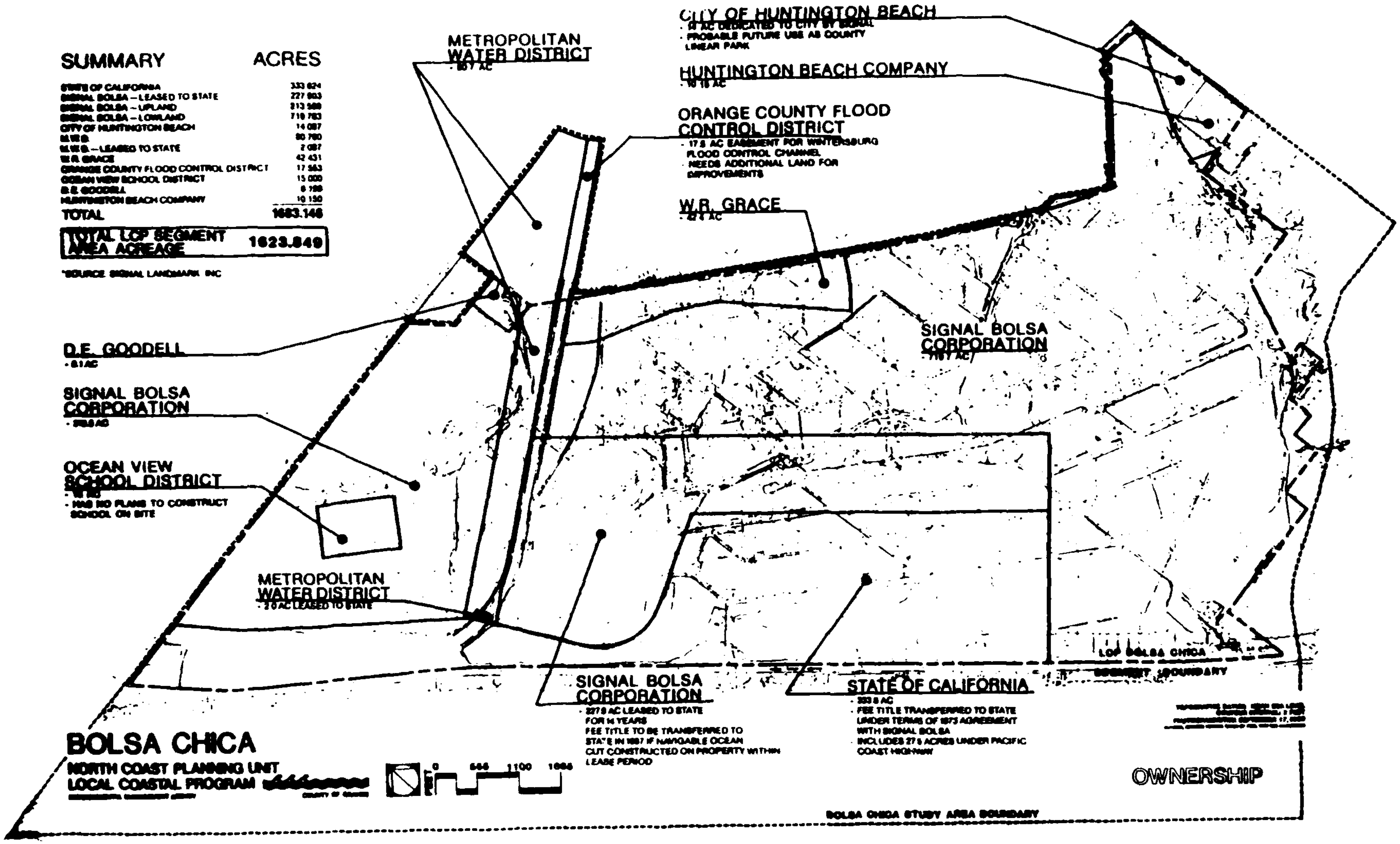


Figure 4. Land ownership, Bolsa Chica, California, region

areas providing visitor-serving uses and amenities. More than 100 acres of navigable waters also will be created to serve the marina-commercial complex, and to provide delivery of ocean waters to the restored wetlands areas. Flood control improvements, new public roads, hiking, bicycling and equestrian trails, public parks, and other major infrastructure are also planned. Finally, the Plan will contain residential uses, including waterfront and off-water dwelling units.

Historical Perspective

17. Involvement of the Federal government in the Bolsa Chica region was directed by Congressional resolutions in 1964 and 1976, and reaffirmed by the Water Resources Development Acts of 1986 and 1988. (The use of the phrase "Sunset Harbor" in those authorizing documents is incorrect, as no such location exists.) The 1964 resolution requested a study to determine the need for a light-draft vessel harbor at Bolsa Chica. The 1976 resolution expanded the study scope to include determination of the feasibility and desirability of providing and maintaining tidal waters and re-creating a tidal marsh. Several studies and surveys have been conducted by the US Army Corps of Engineers, South Pacific Division, Los Angeles (SPL), and non-Corps interests. In addition, a Corps feasibility study had been initiated in response to the 1976 Congressional authority, but has not been completed at the present time.

Congressional Resolution of 1964

18. This resolution, requested by Congressman Richard T. Hanna and adopted April 11, 1964, states:

> "...Resolved by the Committee on Public Works of the House of Representatives, United States, that the Board of Engineers for Rivers and Harbors is hereby requested to review the reports on the coast of southern California, with a view to determining the need for a harbor for light-draft vessels in the Bolsa Chica-Sunset Bay area, California..."

Congressional Resolution of 1976

19. This resolution, requested by Congressman Mark W. Hannaford and adopted September 23, 1976, states:

> "...Resolved by the Committee on Public Works and Transportation of the House of Representatives, United States, that the Board of Engineers for Rivers and Harbors is hereby requested to review the reports on the Coast of Southern California for Light Draft Vessels with a view to determining whether any modifications therein are

warranted in the Bolsa Chica-Sunset Bay area, California, and to conduct a study to determine the feasibility and desirability of re-creating a tidal marsh upon the State-controlled lands in Bolsa Chica Bay for increasing its value for fish and wildlife. This study is to include evaluation and investigation of levees, jetties, breakwaters, and other works needed to provide and maintain tidal waters within the proposed marsh..."

Water Resources Development Act of 1986 (PL 99-662)

20. The following excerpt from the Water Resources Development Act of 1986 pertains to the Bolsa Chica area, although the Corps has not at present interpreted pertinent sections of the Act, nor determined how best to implement such sections thereof:

SEC. 1119: SUNSET HARBOR, CALIFORNIA

a. "...The Secretary is directed to expedite completion of the feasibility study of the navigation project for Sunset Harbor, California,...and to submit a report to Congress on the results of such study...

b. ...Upon execution of agreements by the State of California or Local sponsors, or both, for preservation and mitigation of wetlands areas and appropriate financial participation, the Secretary is authorized to participate with appropriate non-Federal sponsors in a project to demonstrate the feasibility of non-Federal cost sharing under provisions of Section 916 of this Act..."

21. Any and all provisions of the Water Resources Development Act of 1986 (PL 99-662) should be read with the understanding that the Department of the Army has not, at present, made any determination or interpretation with respect to this Act.

Water Resources Development Act of 1988 (PL 100-676)

22. The following excerpt from the Water Resources Development Act of 1988 pertains to the Bolsa Chica area.

SEC. 4: SUNSET HARBOR, CALIFORNIA

f. "...The demonstration project at Sunset Harbor, California, authorized by Sec. 1119(b) of the Water Resources Development Act of 1986 (100 Stat. 4238), is modified to include wetland restoration as a purpose of such demonstration project. All costs allocated to such wetland restorations shall be paid by non-federal interests in accordance with Sec. 916 of such Act..."

Settlement Agreement of 1973

23. During preparation of this report, Signal Bolsa Corporation was the major landowner in the Bolsa Chica study area, having title to 1,200 acres.

W. R. Grace Properties, Inc. owned 42 acres adjacent to the East Garden Grove-Wintersburg Flood Control Channel and the northerly boundary of the site. Slightly more than 100 acres were owned by other interests which include the Metropolitan Water District of Southern California, the Huntington Beach Company, the Ocean View School District, and Donald Goodell. The State of California holds title to 327.5 acres in addition to 230 acres that it holds pursuant to a lease with an option to acquire, subject to the provisions of the 1973 "Boundary Settlement and Land Exchange Agreement Regarding Lands in the Bolsa Chica Area, Orange County, California."

24. Under the 1973 Settlement Agreement between the State and Signal Landmark, Inc., which was signed by the governor of California on March 15, 1973, the State acquired title to a 327.5-acre parcel in the Bolsa Chica lowland. The State also acquired a lease for an additional 230 acres adjacent to the 327.5-acre parcel for a period of 14 years, which was extended to 17 years by the parties in 1984. The State has an option to acquire title to the 230-acre lease parcel if (among other conditions) a navigable ocean entrance system is constructed within a specified time period. Such a system is to consist of a navigable waterway between the Pacific Ocean and land owned by Signal Bolsa Corporation in the Bolsa Chica area.

Proposed Improvements

25. The County of Orange has adopted a Land Use Plan (LUP) as part of the Local Coastal Program for the Bolsa Chica area in accordance with the California Coastal Act of 1976. This LUP includes a navigable ocean entrance system (Preferred Alternative), and a non-navigable ocean entrance system (Secondary Alternative). The principal landowner of the region, Signal Bolsa Corporation, desires to implement the Preferred Alternative.

Preferred Alternative

26. The Preferred Alternative of the LUP, as depicted in Figure 5, contains the following features and acreage allocations:

a. 915 acres of restored, high quality, fully-functioning full tidal, muted tidal, fresh, and brackish water wetlands within the study area, with emphasis on diversity of habitat and protection and recovery of endangered species.

b. 86 acres of existing or newly created environmentally sensitive habitat within the study area.

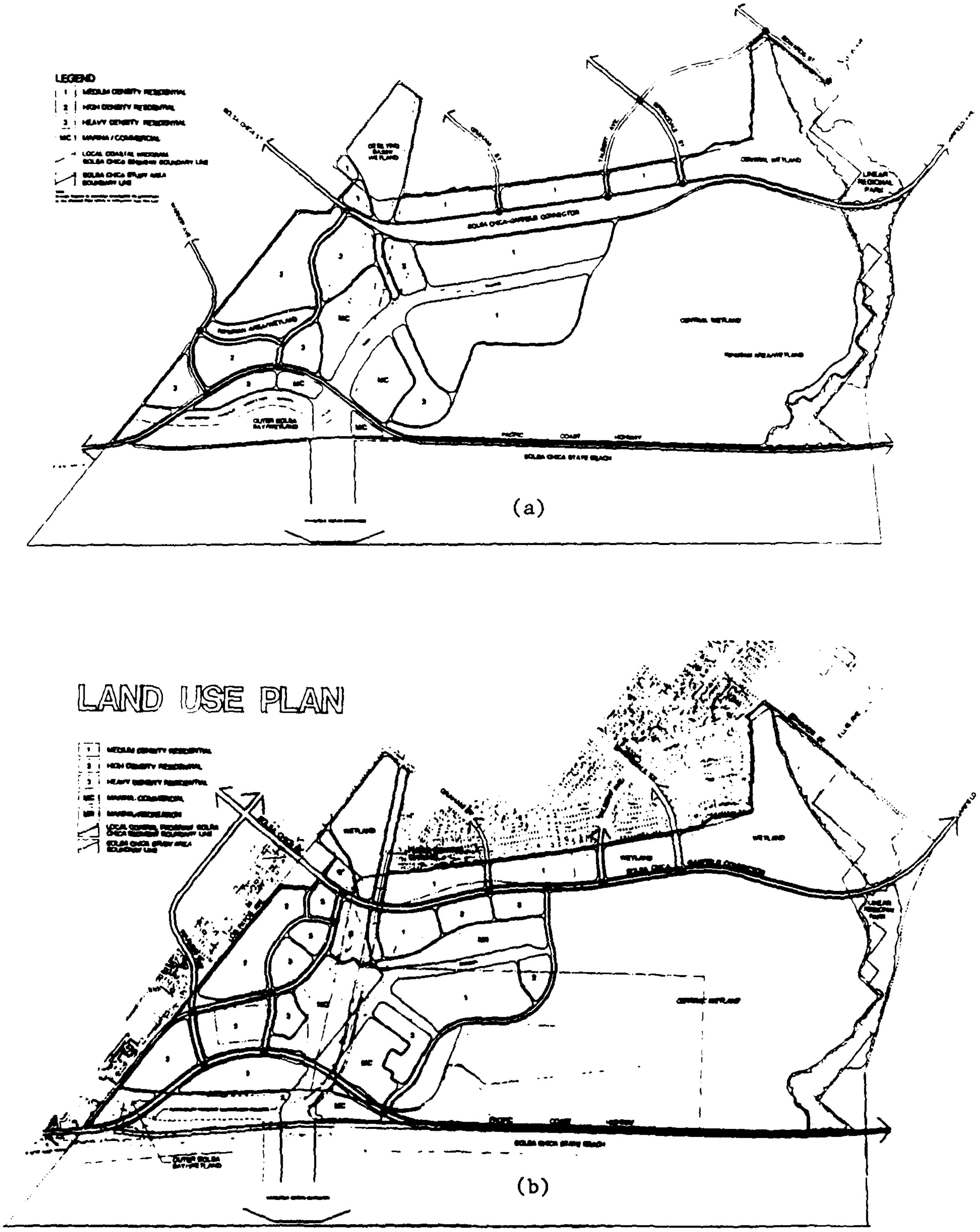


Figure 5. Bolsa Bay Preferred Alternative;
(a) adopted land use plan, and (b) revised land use plan

c. Buffer areas between wetlands and urban development to protect environmentally sensitive habitats.

d. A fully-navigable ocean entrance to provide a continuous, assured source of water for tidal wetlands and interior water ways, and for recreational boating ocean access from both the Bolsa Chica area and Huntington Harbour.

e. Interior navigable waterways providing navigable connections to the Bolsa Bay marina, waterfront residential housing, and Huntington Harbour.

f. At least 75 acres of mixed-use, marina and commercial area providing in-water berthing and dry storage for at least 1,700 boats.

g. A realignment of the Pacific Coast Highway (PCH) from the existing PCH-Warner Avenue intersection, across Outer Bolsa Bay, Bolsa Chica Mesa, and the main entrance channel to the proposed marina.

h. An internal roadway system connecting Bolsa Chica Street with Garfield Avenue within a corridor between 500 and 950 ft from adjacent existing neighborhoods.

i. Creation of a 130-acre Bolsa Chica Linear Regional Park on Huntington Mesa.

j. Approximately 500 gross acres of medium-, high-, and heavy-density residential development in the lowland and on Bolsa Chica Mesa.

Secondary Alternative

27. In certifying the LUP, the California Coastal Commission (CCC) also certified an alternative plan (Secondary Alternative), shown in Figure 6, with a non-navigable ocean entrance and different internal use configurations than the Preferred Alternative. This alternative contains 915 acres of wetlands, a non-navigable ocean entrance, and a marina along the present Warner Avenue alignment on Bolsa Chica Mesa. The CCC indicated that the Secondary Alternative could be certified as the LUP without further hearings if the proposed navigable ocean entrance were found to be infeasible pursuant to performance standards contained in the November 1984 staff report and the January 1986 certified LUP, and if the Secondary Alternative were adopted by the County of Orange as its Land Use Plan.

Previous Studies

28. The Bolsa Chica area is located immediately adjacent to Huntington Harbour, from which navigation vessels exit to the Pacific Ocean through

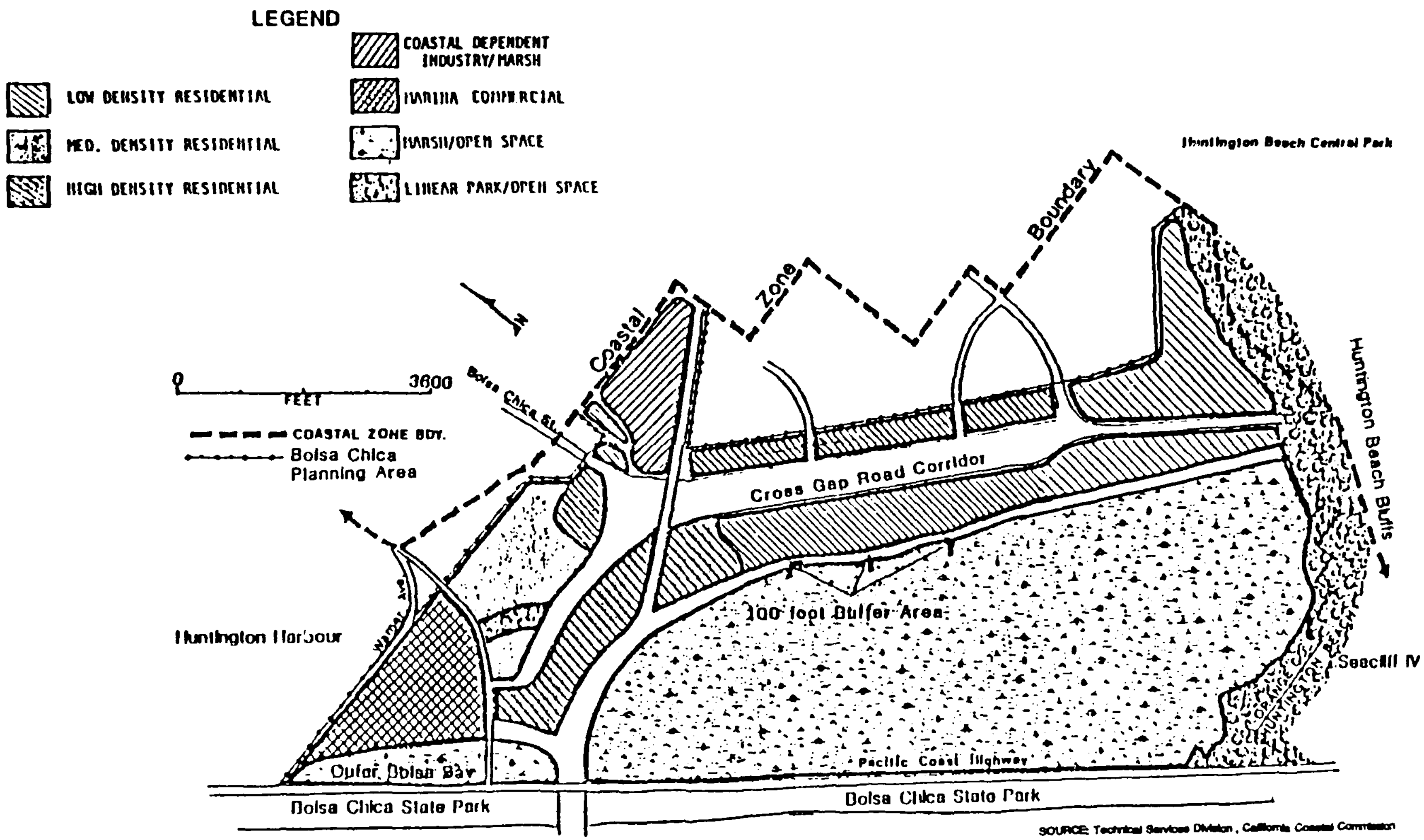


Figure 6. Bolsa Bay Secondary Alternative

Anaheim Bay. The Anaheim Bay entrance is heavily utilized by Seal Beach Naval Weapons Station, and concern has existed for many years about accidental encounters between civilian and military craft in this area, where ammunition off-loading and storage are common practices. Local interests have previously requested the US Army Engineer District, Los Angeles, to investigate the practicality of the construction of a new entrance channel connecting Bolsa Chica with the Pacific Ocean.

29. The Bolsa Chica and Huntington Harbour regions are separated from the Pacific Ocean by Surfside, Sunset Beach, and Bolsa Chica State Beach. The west jetty at Anaheim Bay effectively creates a littoral cell boundary at Seal Beach for the region of coast to the north, and the east jetty is a boundary for the littoral cell between the Anaheim jetties and Newport to the south. Rivers no longer contribute significant sediment into the littoral cell between Anaheim and Newport Beach. Artificial beach nourishment at Surfside-Sunset, in amounts that average approximately 350,000 cu yd per year, has provided a feeder beach for the littoral cell that extends down the coast toward Newport Beach. Much of the nourishment is due to disposal of material excavated from the Navy channel at Anaheim and has been dictated by funds available, rather than by the optimum requirements for beach nourishment.

30. A new entrance channel to Bolsa Chica will require stabilization by a jetty system. Furthermore, interruption of downcoast movement of littoral material may require a sand bypassing system. Tidal flow through a new entrance channel also may affect tidal circulation through Huntington Harbour. These concerns are multifaceted and interrelated, and have given rise to many studies of beach processes and tidal circulation evaluations in recent years.

State of California studies

31. Following completion of the boundary settlement and land exchange agreement between the State of California and Signal Landmark, Inc., it became apparent that a plan should be developed depicting the interests of all concerned State agencies. A plan, entitled "Bolsa Chica Marsh Re-Establishment Project" (State of California, 1974), was presented by The Resources Agency. Alternative methods were evaluated for obtaining the greatest benefits for the use of public lands in

Bolsa Chica and fulfilling the land settlement commitments. Each alternative included the following:

- a. Development of an additional area to provide a total of approximately 350 acres of marsh.
- b. Construction of interpretive and visitor-use facilities.
- c. Construction of a channel to the ocean to provide tidal waters to the marsh and ocean access for boats.
- d. Construction of an 1800-boat marina and small boat launching ramp.
- e. Provisions for a 300-ft wide channel connection between Signal properties and State lands.
- f. Integrated development between Bolsa Chica State Beach and the marina-ecological reserve complex.
- g. Transportation alternatives for the beach-marina-marsh complex.

Orange County studies

32. In addition to continuous water quality monitoring studies, the "Bolsa Chica Local Coastal Program Land Use Plan" was adopted by the Orange County Environmental Management Agency (1985), and it contains all suggested modifications approved by the CCC on October 23, 1985. The LUP includes the following features:

- a. 915 acres of productive and diverse wetlands and 86 acres of environmentally sensitive habitat areas.
- b. A navigable ocean entrance to provide high-quality tidal flow to the wetlands and navigable access to the ocean, new navigable waterways, a 75-acre or larger marina and commercial area with berthing and dry storage for at least 1,700 boats, launch ramps, and coastal-dependent, visitor-serving commercial facilities.
- c. An optional navigable interior waterway connection to Huntington Harbour.

US Army Engineer District, Los Angeles, studies

33. The Corps study of the Bolsa Chica/Sunset Bay area, California, was authorized by Congressional resolutions in 1964 and 1976, and reaffirmed in the Water Resources Development Acts of 1986 and 1988. Several studies and surveys have been initiated, but a Corps feasibility study in response to the study authority has not been completed at the present time. Preliminary studies, and current indications of the desirability for both recreational

boating and wetland restoration within the local community, suggest that achievement of both may be feasible. However, additional study is needed to determine (a) the engineering, economic, and environmental feasibility of specific plans for small-craft harbor development, and wetland preservation, enhancement, and restoration, and (b) the extent of Federal participation, if any, in any plan implementation.

Previous tidal circulation studies

34. Waterways Experiment Station (1981). The first hydrodynamic modeling of the tidal circulation characteristics of existing Bolsa Chica tidal areas was conducted for SPL by WES in 1981 to compare tidal elevations, velocities, and volumes of flow at specific prototype gage locations in Anaheim Bay, Huntington Harbour, Warner Avenue Bridge, Outer Bolsa Bay, and Inner Bolsa Bay (US Army Engineer Waterways Experiment Station 1981). The hydrodynamic model used in this study was a two-dimensional, depth-averaged, finite-difference approximation model developed at WES. Comparisons were made for existing conditions and seven proposed alternative plans. Prototype field data for numerical model calibration and comparison with alternatives had been obtained by Meridian Ocean Systems, Inc., at data stations during a 25-hr period over April 24-25, 1980. The primary objective of the study was to identify any impacts to the existing channel system in Huntington Harbour resulting from a new ocean entrance, marina, and wetland areas in Bolsa Chica. The tidal characteristics of the existing wetlands and new wetlands under the proposed plans, however, were not considered in that study. The conclusion reached from the study was that tidal amplitudes were not significantly altered in Anaheim Bay, Huntington Harbour, or Outer Bolsa Bay by any of the plans evaluated. Direction of flood flow under Warner Avenue Bridge with the proposed new entrance channel in place changed flow direction such that flood flow was into Huntington Harbour. Hence, a region of reduced tidal velocity was indicated in Huntington Harbour.

35. Philip Williams & Associates (1984). A study of the tidal characteristics of the existing Huntington Harbour area and seven proposed alternative designs for Bolsa Chica, and an evaluation of a self-maintained ocean entrance at Bolsa Chica, were conducted by Philip Williams & Associates (1984). Because of the significant channelization throughout the flow system, this study utilized a one-dimensional link-node model that uses the method of

characteristics to solve the equations of water motion within each link. Field data previously obtained by Meridian Ocean Systems, Inc., during a 25-hr period over April 24-25, 1980, were also used in this study for calibration and comparison of results. The purpose of the study was to evaluate the impacts of proposed plans on tidal velocities in Huntington Harbour, and to determine the tidal range in the restored wetland. The study concluded that, for the case of no new ocean entrance, tidal velocities in Huntington Harbour would increase with the addition of fully tidal wetlands in Bolsa Chica. With a new ocean entrance, however, the velocities would not generally increase. The analysis of tidal range in the restored wetlands consisted of a qualitative comparison between simulated conditions with and without the new ocean entrance. The results from the analysis indicated that a small dampening and phase lag would occur to the tide in Bolsa Chica if the area were opened to full tidal action with no new ocean entrance. A maximum reduction in tidal range of about 25 percent would occur during very high spring tides. These studies also concluded that proposed restoration designs for Bolsa Chica would have sufficient tidal prism to maintain a natural channel of between 1,400 and 3,700 sq ft, if the channel sides were stabilized. The channel could have widths of 200 to 450 ft, with depths from 10 to 12 ft.

36. Moffatt & Nichol, Engineers (1987). A hydraulic analysis of the Bolsa Chica wetlands was performed by Moffatt & Nichol, Engineers (1987) using a one-dimensional link-node model that was calibrated to existing conditions using field measurements taken over a 3-week period from August 16 through September 5, 1986. The study was performed to:

a. Provide an understanding of the hydraulic response of coastal wetlands, and wetlands with a muted tide regime that is applicable to Bolsa Chica wetlands.

b. Model the hydraulics of the existing Bolsa Chica wetlands and the tidal cell added by the California Department of Fish and Game.

c. Develop a wetland model that is calibrated to existing conditions, and that can be used to analyze proposed wetland configurations.

The scope of the work required that the study:

a. Describe the hydraulics of coastal wetlands as well as tide control structures that are applicable to Bolsa Chica.

b. Outline the design approach used in the hydraulic analysis of wetlands.

c. Modify and calibrate a numerical model to analyze the existing conditions in the Bolsa Chica wetlands, and

d. Perform a sensitivity analysis to identify the relative effect that each input value has on the results in order to indicate confidence intervals.

37. The calibrated model will be used to further analyze proposed wetland configurations for Bolsa Chica. Since results obtained for proposed configurations cannot be compared with measurements to assess accuracy, a sensitivity analysis was performed to estimate the range in which the results are most likely to fall. It was determined by this study that tide range in the wetlands is greatly affected by the type of tide control structure used. Tide control structures can be designed to provide the required tidal range and mean water level in the wetlands. This is important to achieve the desired mix of habitats. The hydraulic design comprises a large part of the wetland design. The complex calculations involved are readily solved by this numerical model in a timely and economical fashion.

Previous beach sand movement studies

38. Beach Erosion Board (1956). The Anaheim Bay jetties were completed in 1944 and serve as an effective barrier to littoral sand transport along the shore to a depth of about 20 ft. The construction of the jetties was followed by severe erosion of the beach immediately to the south of the east jetty. The eroded sand was apparently transported in a southerly direction by the dominant wave action. Erosion progressed to such a degree that extensive property damage was imminent and, late in 1947, a beach fill was placed to restore the shore. Subsequently, this reach of shoreline has been periodically renourished with an average annual volume of approximately 350,000 cu yd of sand made available from channel maintenance operations at Anaheim. Sand movement along the coast was correlated with dominant wave energy by this study (Caldwell 1956).

39. US Army Engineer District, Los Angeles (1978). Because of the continuing necessity to rehabilitate the Surfside-Sunset Beach region of coastline due to severe beach erosion, SPL established a monitoring program to evaluate the effectiveness of the placement procedures. One of the purposes of the effort was to determine if portions of the material disappearing from the beach was moving offshore where it would be recycled periodically to the beach. Results of the overall monitoring program were inconclusive.

40. **Waterways Experiment Station (1984).** The potential effects of a new entrance channel to Bolsa Chica on unstabilized adjacent shorelines was considered by WES in 1984 (Hales 1984). That study utilized a one-line numerical model for longshore sediment transport and an equivalent monthly wave climate deduced from frequency of occurrence of waves from a 3-year hindcast (1956 to 1958) by National Marine Consultants (1960) and Marine Advisors (1961). Evaluations were performed for uniform bypassing placement distributions of 300, 500, 1,000, and 2,000 ft from the east jetty at Anaheim Bay. As the distribution of the bypassed material was extended farther down coast, those computational cells nearer the east jetty experienced an increased depletion of material. The actual equilibrium shoreline orientation that develops will be in response to the effectiveness of the bypassing program and the actual wave climate.

Regional Geology

41. As discussed in House Document No. 349 (US Congress 1954), Bolsa Chica is on the edge of San Pedro Bay, approximately in the center of the Los Angeles coastal plain. This low plain is bordered on the north by the eastern Santa Monica Mountains and the Repetto Hills, on the east by the Puente Hills and the Santa Ana Mountains, on the southeast by the San Joaquin Hills, and on the south and west by the Pacific Ocean. Many of the structural features surrounding the Los Angeles coastal plain are extremely young, and the present relief and alignment of geographic units are, to a large extent, the product of a mountain-building epoch. The gently curving arc of shoreline extending from Point Fermin on the west to the bluffs of Corona del Mar on the east is composed, in part, of disconnected stretches of barrier beach fronting slowly rising tidal marsh areas. Separating these lowlands are the friable wave-cut cliffs or bluffs at Long Beach, Seal Beach, Huntington Beach, and Newport Beach. The character of these wave-cut bluffs, and the uniform plain to which they have been shaped by the sea, indicate that each headland formerly extended seaward of the present shoreline.

42. Under natural conditions that existed over 100 years ago, the Los Angeles and San Gabriel Rivers deposited most of their sediment loads on the ocean bars at their mouths where this material became available for

nourishment of the beaches. Flood-control structures in the upper reaches of these rivers, constructed during the past century, now have nearly eliminated sediment from being delivered to the beaches by the rivers.

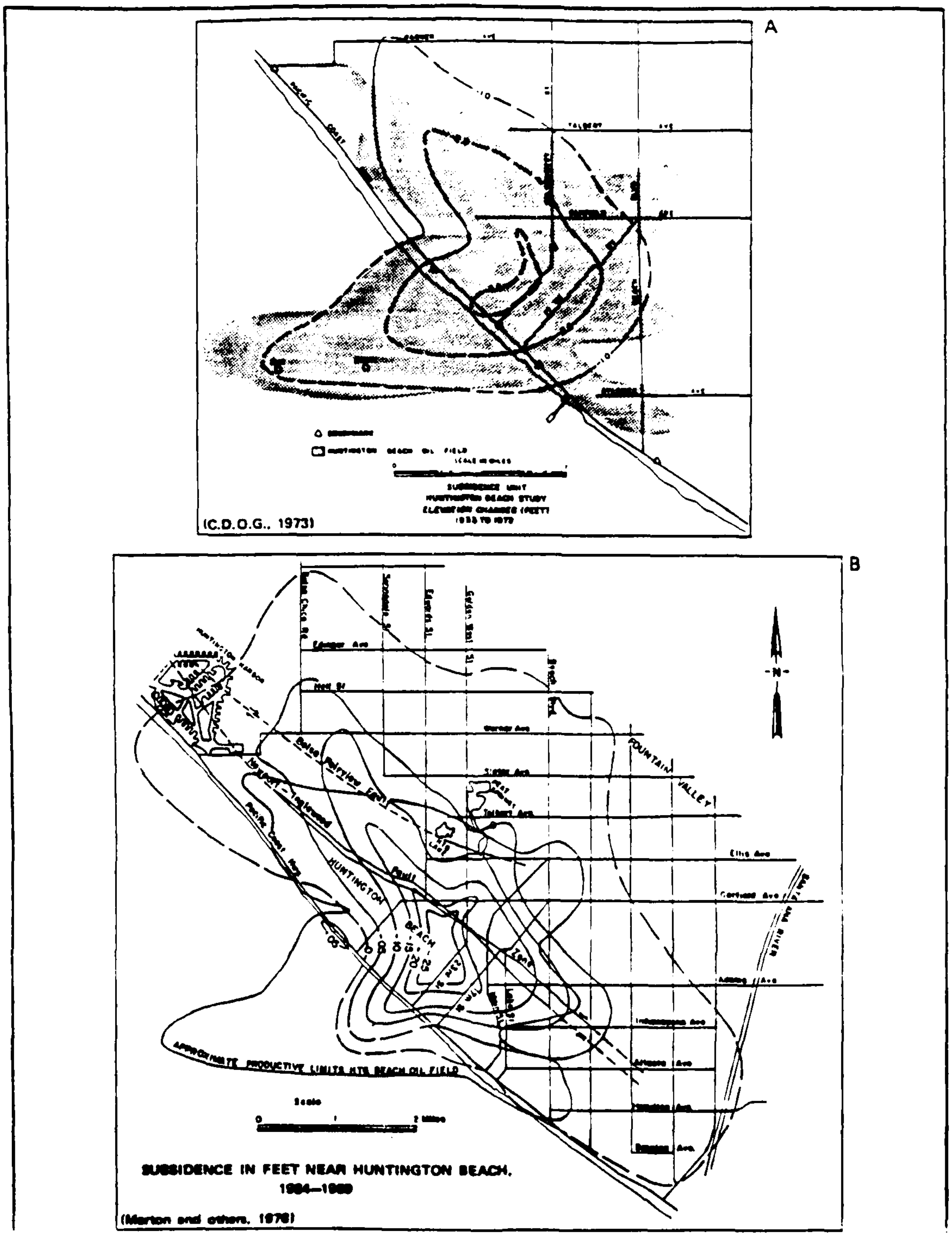
43. The significant findings resulting from a review of the geologic history of the area under investigation may be summarized as follows:

- a. Prior to historic time, uplift and erosion of the headlands, together with subsidence and fill of low area, developed the early shoreline into a semblance of the present shore.
- b. The shoreline appears to have become relatively stable at about the beginning of historic time, and further erosion of the headlands was dependant on the balance between losses of beach material by marine erosion and wind, and the periodic supply of new material brought to the shore by streams.
- c. During historic time, the beaches adjacent to Long Beach, Seal Beach, and Huntington Beach bluffs have remained comparatively narrow, which indicates that a very close balance between loss and supply existed in these areas.

Subsidence in the Bolsa Chica Area

44. The Local Coastal Plan has identified ground subsidence as one of the geologic hazards that must be addressed in planning the Bolsa Chica development. Subsidence in the Bolsa Chica area has been evaluated by Woodward-Clyde Consultants (1984, 1986). Subsidence refers to broad scale, gradual downward changes in elevation of the land surface. Such subsidence can occur naturally and from influences by man. The natural causes could be tectonic structural flexure of faulting, consolidation of sedimentary rocks, or highly compressible peat deposits. Man-induced subsidence has been attributed to oil and water withdrawal in many of California's oil fields and ground-water basins.

45. The major subsidence area has coincided with the limits of the Huntington Beach oil field. Historical subsidence patterns from 1933 to 1972, and from 1964 to 1969 are shown in Figure 7 (Woodward-Clyde Consultants 1984). The decrease in the subsidence has been attributed to water injection of oil producing zones which was initiated in 1959. Estimates of the maximum amount of subsidence have ranged up to 5 ft since 1920 when oil production began. The maximum range of subsidence from 1955 to 1968 was reported as 0.15 ft (1.8 in.) per year, but this rate decreased to 0.05 ft (0.6 in.) per year



Project: BOLSA CHICA DEVELOPMENT Project No. 41602A	MAPS SHOWING HISTORICAL SUBSIDENCE NEAR HUNTINGTON BEACH	

Figure 7. Historical subsidence near Huntington Beach, California

from 1968 to 1972 (California Division of Oil and Gas 1973).

46. Subsidence rates from 1976 to 1985 have been calculated by analyzing precise leveling data of benchmarks in the area obtained from the Orange County Surveyor's Office. The history of subsidence in the areas was presented for the periods from 1976 to 1982, 1976 to 1985, and 1982 to 1985. The average annual subsidence rates for these periods are presented in Figures 8 through 10, respectively (Woodward-Clyde Consultants 1984, 1986). Review of these figures indicate that although subsidence is continuing across the site, it appears that in the last several years it is occurring at a lower rate. The annual subsidence over the site is estimated to continue at an average rate of 0.01 ft per year, based on the rates from 1982 to 1985. However, the subsidence in the area is considered to be primarily due to hydrocarbon withdrawal, and the rate should respond closely to oil extraction and water injection.

Sea Level Rise in the Bolsa Chica Area

47. The annual average rate of mean sea level rise along the California coast is approximately 0.005 ft per year, based on available tide gage records. A 0.5 ft per century rate is also considered the global average of sea level increase over the past century (Revelle 1983).

48. Various projections of future sea level rise have been proposed, and are illustrated in Figure 11. Work summarized by Hoffman, Keyes, and Evans (1983) and Hoffman (1983) foresees the possibility of rates of increase with upper limits exceeding an average of 9 ft per century over the next 120 years. These projections are based on fundamentally unverifiable computer models of global warming given past and projected increases in atmospheric carbon dioxide and other greenhouse gases, including methane and chlorofluorocarbons. These scenarios contain a large amount of uncertainty, as reflected in the wide range of estimates shown in Figure 11 (Seidel and Keyes 1983). The most recent study by the Marine Board (1987) predicts a rate of increase of 1.3 ft per century (0.013 ft per year), and is recommended for 25-year design projects. However, the historical rate of sea level rise has been only approximately 0.5 ft per century.

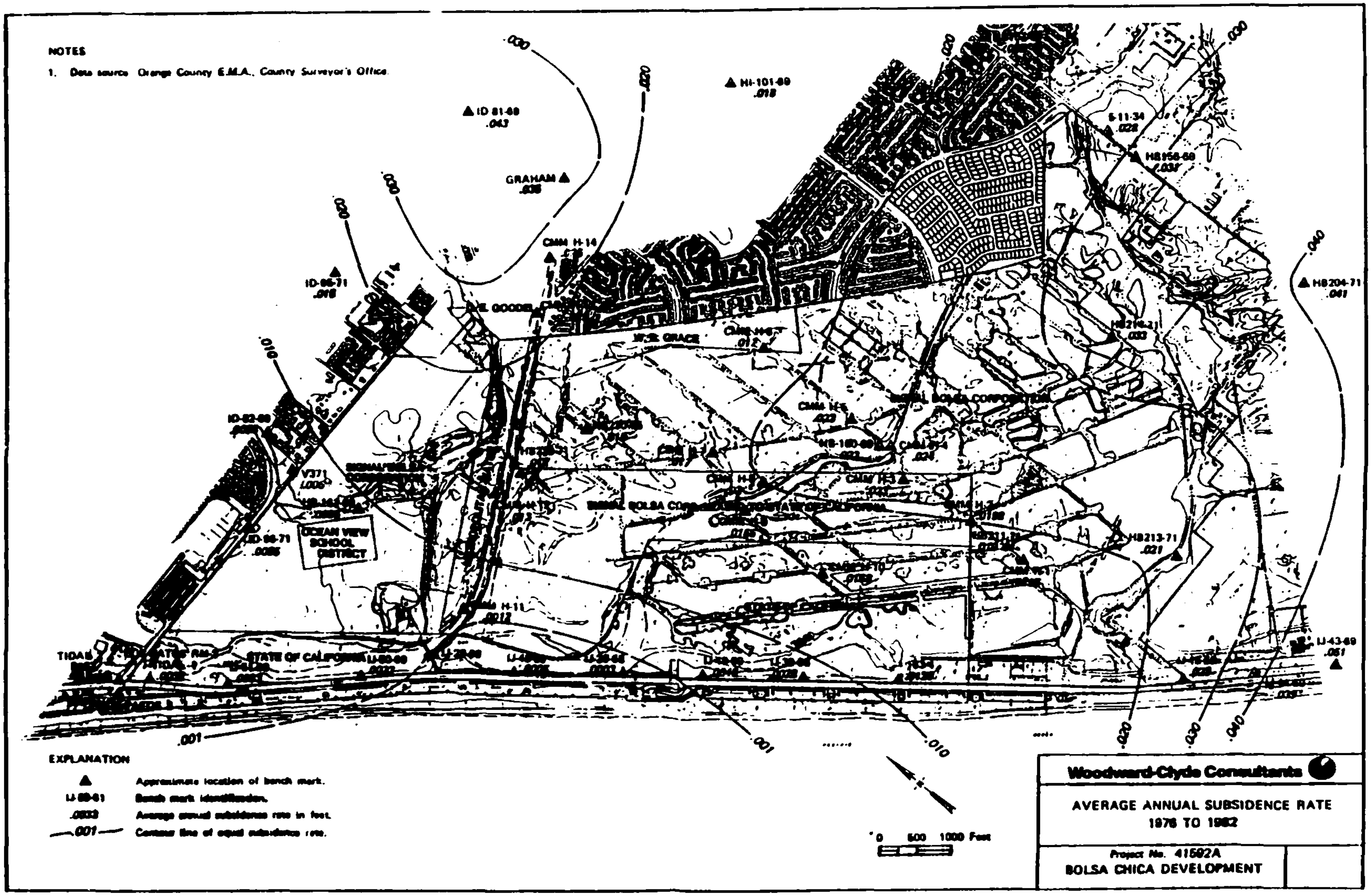


Figure 8. Average annual subsidence rate, 1976 to 1982, Bolsa Chica region

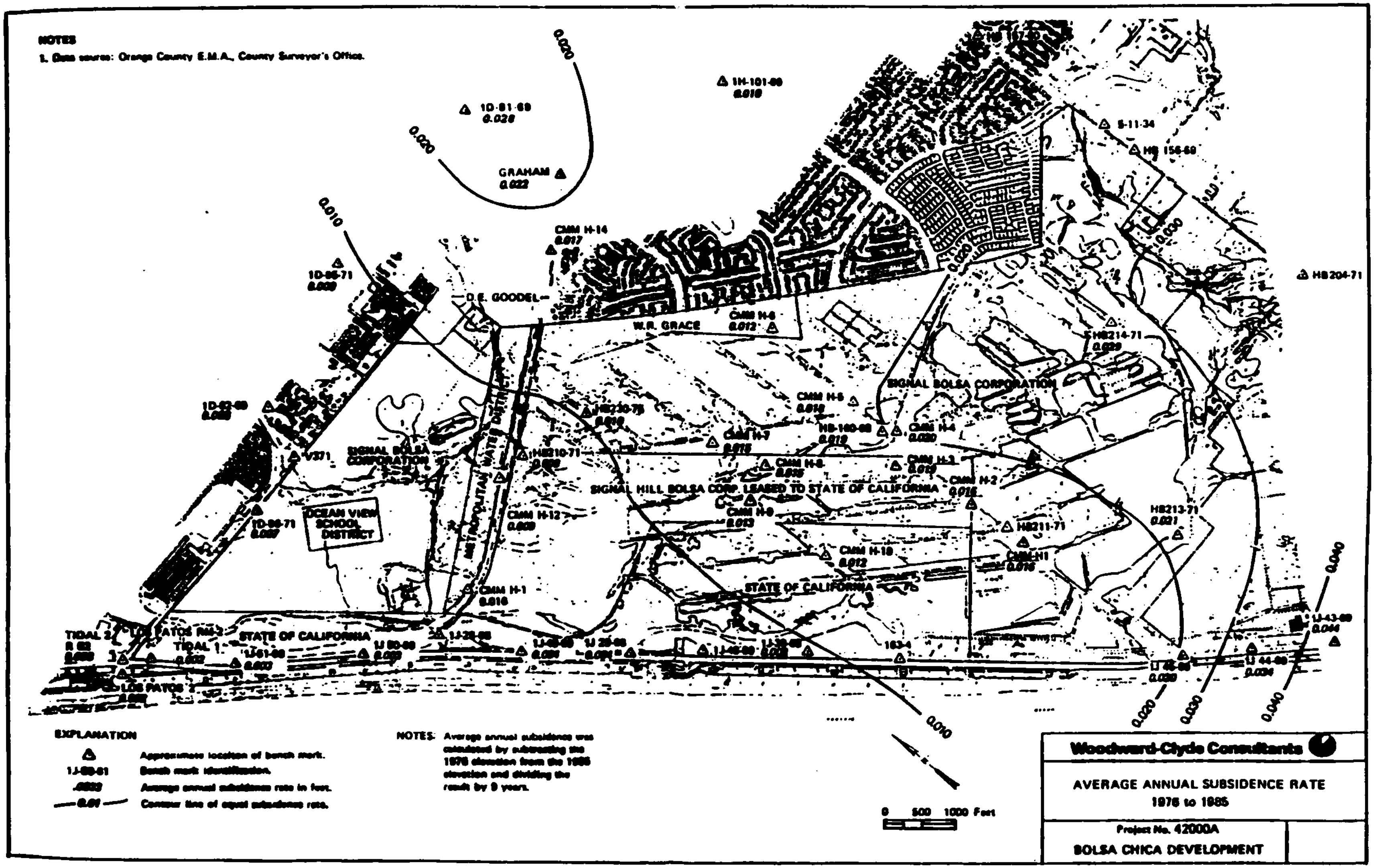


Figure 9. Average annual subsidence rate, 1976 to 1985, Bolsa Chica region

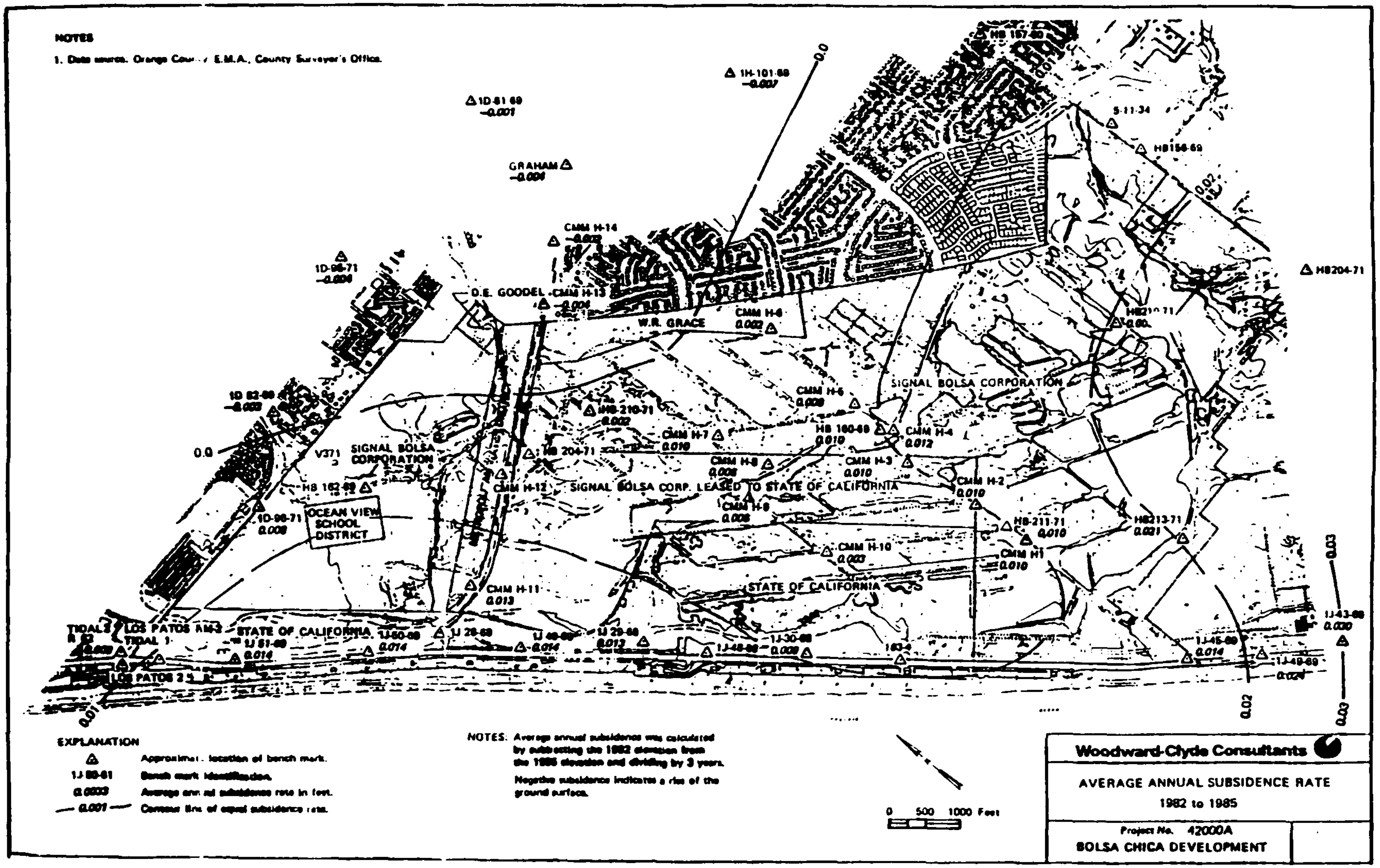


Figure 10. Average annual subsidence rate, 1982 to 1985, Bolsa Chica region

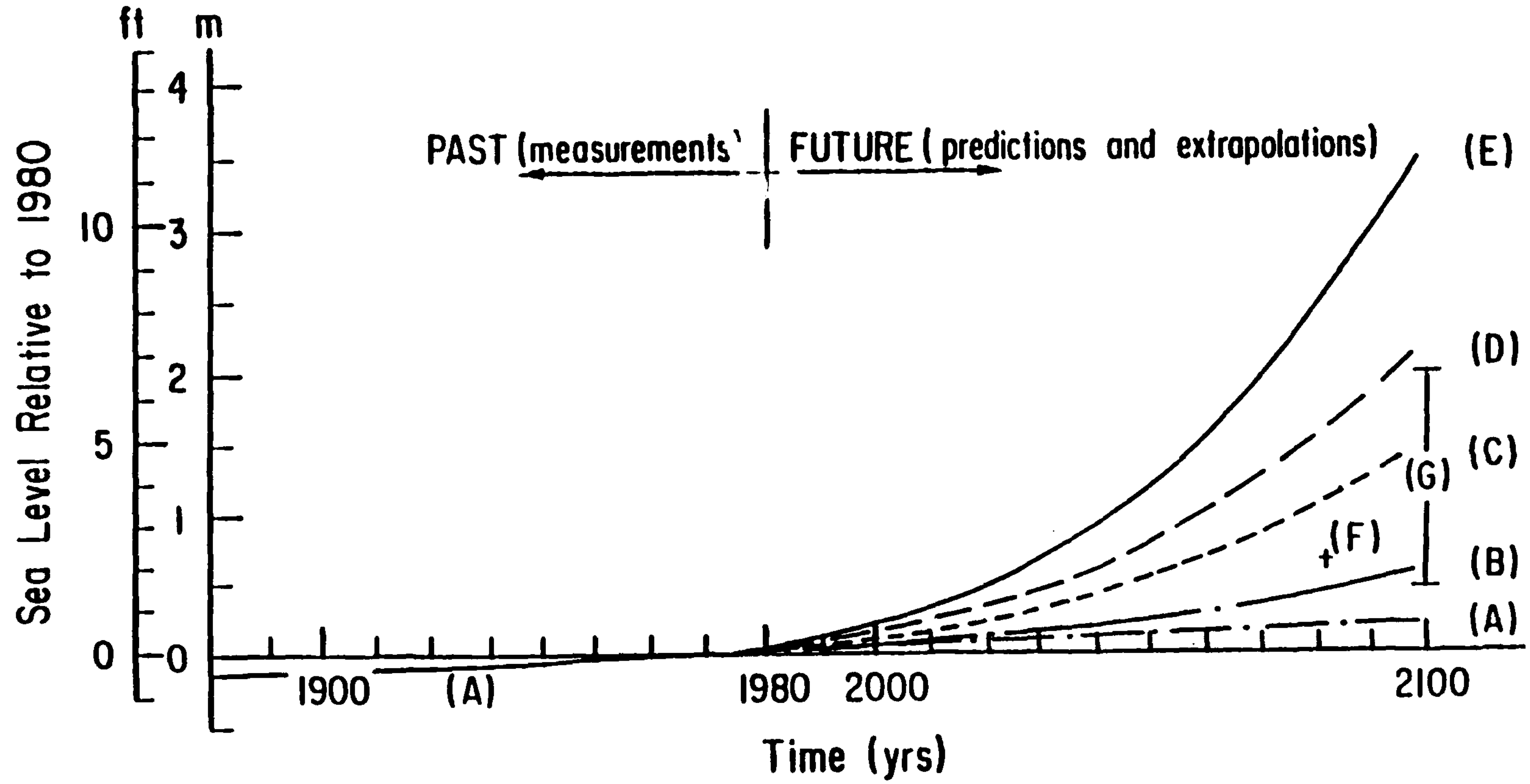


Figure 11. Schematic of eustatic sea level rise curves, (A) Rate of rise over last century projected into the future, (B), (C), (D), and (E) Hoffman et al. (1983) estimates respectively for conservative, mid-range low, mid-range high, and high rates of increase, (F) Revelle (1983), (G) Polar Research Board estimate augmented for thermal expansion (Revelle 1983)

PART III: THE MODEL

Design of Model

49. The Bolsa Chica Model (Figure 12) was constructed to an undistorted linear scale of 1:75, model to prototype. Scale selection was based on such factors as:

- a. Depth of water required in the model to prevent excessive bottom friction.
- b. Absolute size of model waves.
- c. Available shelter dimensions and area required for model construction.
- d. Efficiency of model operation.
- e. Available wave-generating and wave-measuring equipment.
- f. Model construction costs.

A geometrically undistorted model was necessary to ensure accurate reproduction of both wave refraction and wave diffraction, simultaneously. Following selection of the linear scale, the model was designed and operated in accordance with Froude's model law (Stevens, et al., 1942). The scale relations used for design and operation of the model were as follows:

Characteristic	Dimension*	Model-Prototype Scale Relations
Length	L	$L_r = 1:75$
Area	L^2	$A_r = L_r^2 = 1:5,625$
Volume	L^3	$\cancel{V}_r = L_r^3 = 1:421,875$
Time	T	$T_r = L_r^{1/2} = 1:8.66$
Velocity	L/T	$V_r = L_r^{1/2} = 1:8.66$
Discharge	L^3/T	$Q_r = L_r^{5/2} = 1:48,714$

* Dimensions are in terms of length and time.

50. The proposed navigable ocean entrance at Bolsa Chica included the use of rubble-mound structures. Experience and experimental research have shown that considerable wave energy passes through the interstices of this type structure; thus, the transmission and absorption of wave energy became a matter of concern in design of the 1:75-scale model. In small-scale hydraulic models, rubble-mound structures reflect relatively more and absorb or dissipate relatively less wave energy than geometrically similar prototype structures (Le Mehaute 1965). Also, the transmission of wave energy through a

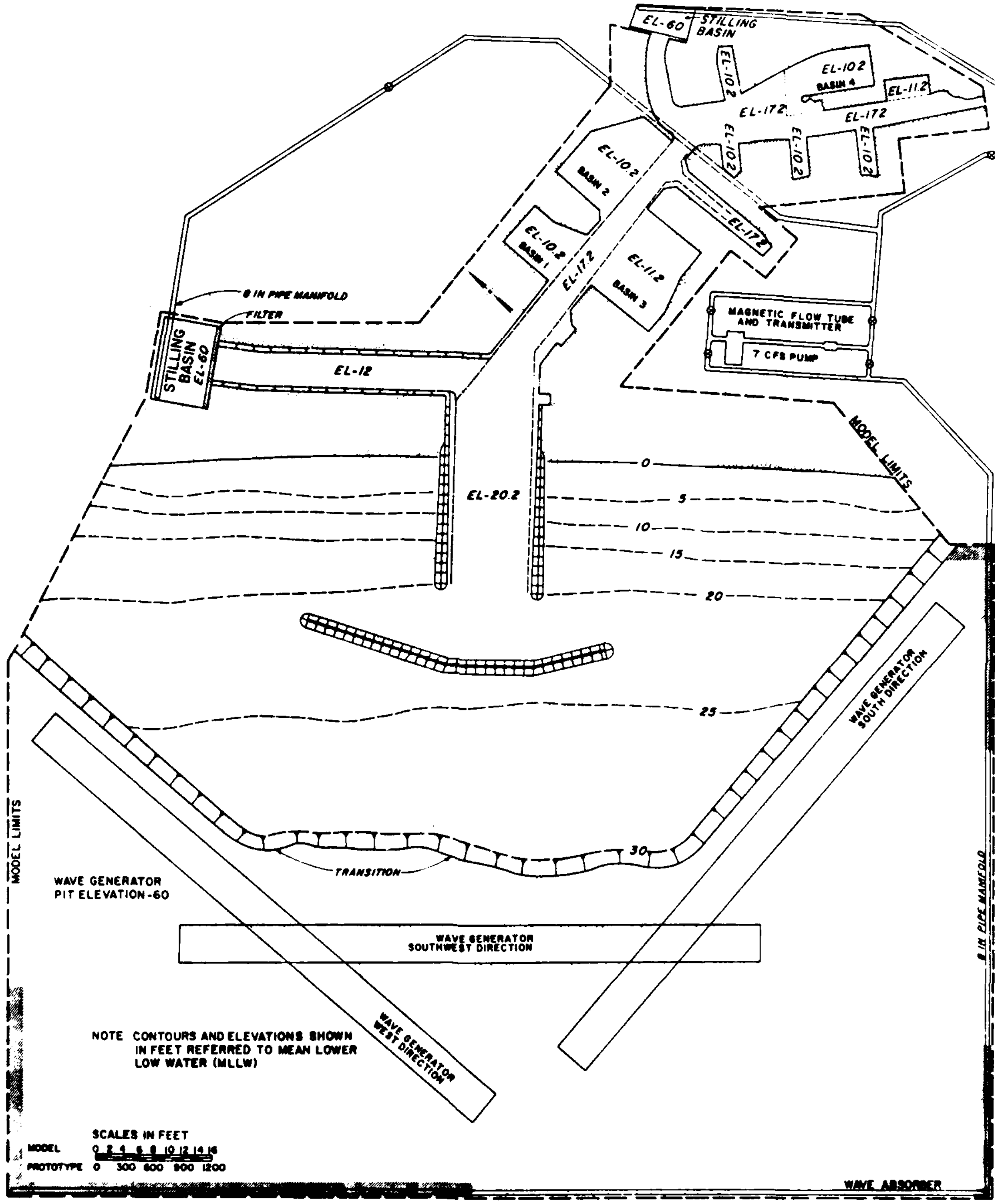


Figure 12. Model layout

rubble-mound structure is relatively less for the small-scale model than for the prototype. Consequently, some adjustment in small-scale model rubble-mound structures is needed to ensure satisfactory scaled reproduction of wave-reflection and wave-transmission characteristics. In past investigations (Dai and Jackson 1966, Brasfeild and Ball 1967) at WES, this adjustment was made by determining the wave-energy transmission characteristics of the proposed structure in a two-dimensional model using a scale large enough to ensure negligible scale effects. A section then was developed for the small-scale, three-dimensional model that would provide essentially the same relative transmission of wave energy. Therefore, from previous findings for structures and wave conditions similar to those at Bolsa Chica, it was determined that a close approximation of the correct wave-energy transmission characteristics would be obtained by increasing the size of the stone used in the 1:75-scale model to approximately one-and-one-half times that required for geometric similarity. Accordingly, in constructing the rubble-mound structures in the Bolsa Chica model, the rock sizes were computed linearly by scale, then multiplied by 1.5 to determine the actual sizes to be used in the model. Consequently, rubble mound stability is not tested due to model scale effects.

51. Ideally, a quantitative, three-dimensional, movable-bed model investigation would best determine the effectiveness of various project plans at Bolsa Chica with regard to the deposition of sediment of the harbor entrance. However, this type of model investigation is difficult and expensive to conduct, and each area in which such an investigation is contemplated must be carefully analyzed. The following computations and prototype data are considered essential for such investigations (Chatham, Davidson, and Whalin, 1973):

- a. A computation of the littoral transport, based on the available wave statistics.
- b. An analysis of the sand size distribution over the entire project area (offshore to a point well beyond the breaker-zone).
- c. Simultaneous measurements of the following items over a period of erosion and accretion of the shoreline (this measurement period should be judiciously chosen to obtain the maximum probability of both erosion and accretion during as short a time span as possible):

(1) Continuous measurements of the incident wave characteristics. Such measurements would entail placing enough redundant sensors to accurately estimate the directional spectrum over the entire project area, and in addition, would include conducting a rather sophisticated analysis of these data.

(2) Bottom profiling of the entire project area using the shortest time intervals possible.

(3) Nearly continuous measurements of both littoral and onshore-offshore transport of sand. These measurements would be especially important over the erosion-accretion period. A wave-forecast service would be essential to this effort to prepare for full operation during the erosion period.

In view of the complexities involved in conducting movable-bed model studies and time and funding constraints for the Bolsa Chica project, the model was molded in cement mortar (fixed bed) at an undistorted scale and a tracer material was utilized to determine qualitatively the degree of erosion and accretion at the entrance for various improvement plans.

The Model and Appurtenances

52. The model reproduced approximately 8,000 ft of the California shoreline, the proposed interior basins of the marina complex, a portion of Wintersburg Flood-Control Channel, and underwater contours in the Pacific Ocean to an offshore depth of 30 ft with a sloping transition to the wave generator water depth of -60 ft. The total area reproduced in the model was approximately 15,000 sq ft, representing about 3.0 sq miles in the prototype. A general view of the model looking from the marina to the offshore is shown in Figure 13. Vertical control for model construction was based on mean lower low water (mllw).* Horizontal control was referenced to a local prototype grid system.

53. Model waves were generated by an 80-ft-long, unidirectional spectral, electrohydraulic wave generator with a trapezoidal-shaped, vertical-motion plunger. The wave generator utilized a hydraulic power supply. The vertical motion of the plunger caused a time varying displacement of water which generated the required irregular test waves. The wave generator also was mounted on retractable casters which enabled it to be positioned to generate waves from the different directions.

* All elevations (el) cited herein are in ft referred to as mean lower low water (mllw) unless otherwise noted.

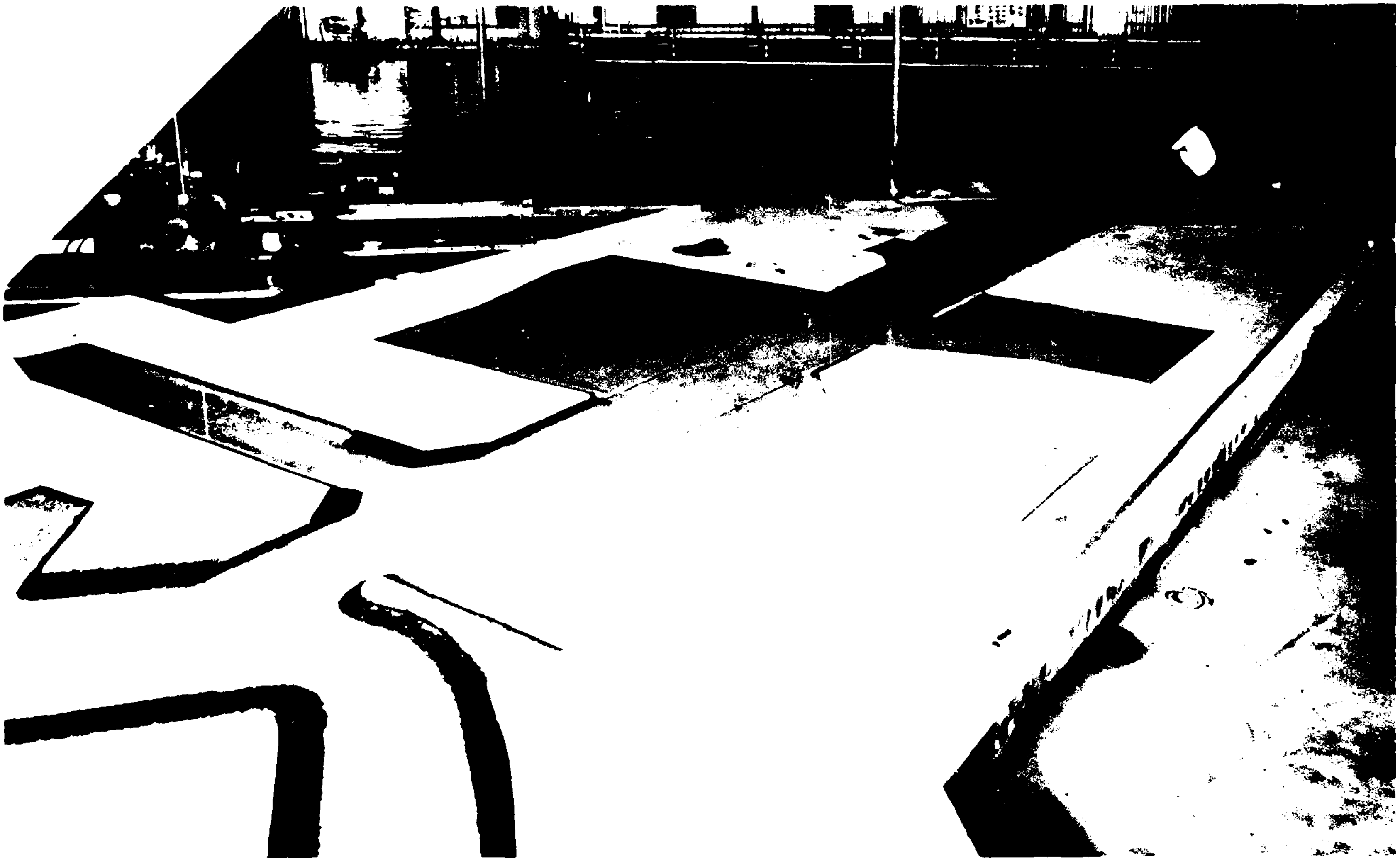


Figure 13. General view of model

54. A water circulating system (Figure 12) consisting of 8-in, perforated pipe, water intake and discharge manifolds, a 7-cfs pump, and a magnetic flow tube and transmitter, was used in the model to reproduce steady-state tidal flows. The flows corresponded to maximum ebb and flood tidal discharges as determined by the results of the numerical tidal circulation model (Hales, in preparation). The circulation system was also used to reproduce flows in the Wintersburg Channel that corresponded to specified prototype discharges.

55. An Automated Data Acquisition and Control System (ADACS), designed and constructed at WES (Figure 14), was used to record and analyze wave-height data at selected locations in the model. Basically, through the use of a minicomputer, ADACS recorded onto magnetic tape the electrical output of parallel-wire, resistance-type wave gages that measured the change in water-surface elevation with respect to time. The magnetic tape output of ADACS then was analyzed to obtain the wave-height data.

56. A 2-ft (horizontal) solid layer of fiber wave absorber was placed around the inside perimeter of the model to dampen wave energy that might otherwise be reflected from the model walls. In addition, guide vanes were placed along the wave generator sides in the flat pit area to ensure proper formation of the wave train incident to the model contours.

Selection of Tracer Material

57. As discussed previously in paragraph 51, a fixed-bed model was constructed and a tracer material selected to determine qualitatively the deposition of sediment at the proposed entrance for the various improvement plans. The tracer was chosen in accordance with the scaling relations of Noda (1972), which indicate a relation or model law among the four basic scale ratios, i.e. the horizontal scale, λ; the vertical scale, μ; the sediment size ratio, n_D; and the relative specific weight ratio, n_γ (Figure 15). These relations were determined experimentally using a wide range of wave conditions and bottom materials and they are valid mainly for the breaker zone.

58. Noda's scaling relations indicate that movable-bed models with scales in the vicinity of 1:75 (model to prototype) should be distorted (i.e., they should have different horizontal and vertical scales). Since the fixed-bed model of Bolsa Chica was undistorted to allow accurate reproduction of

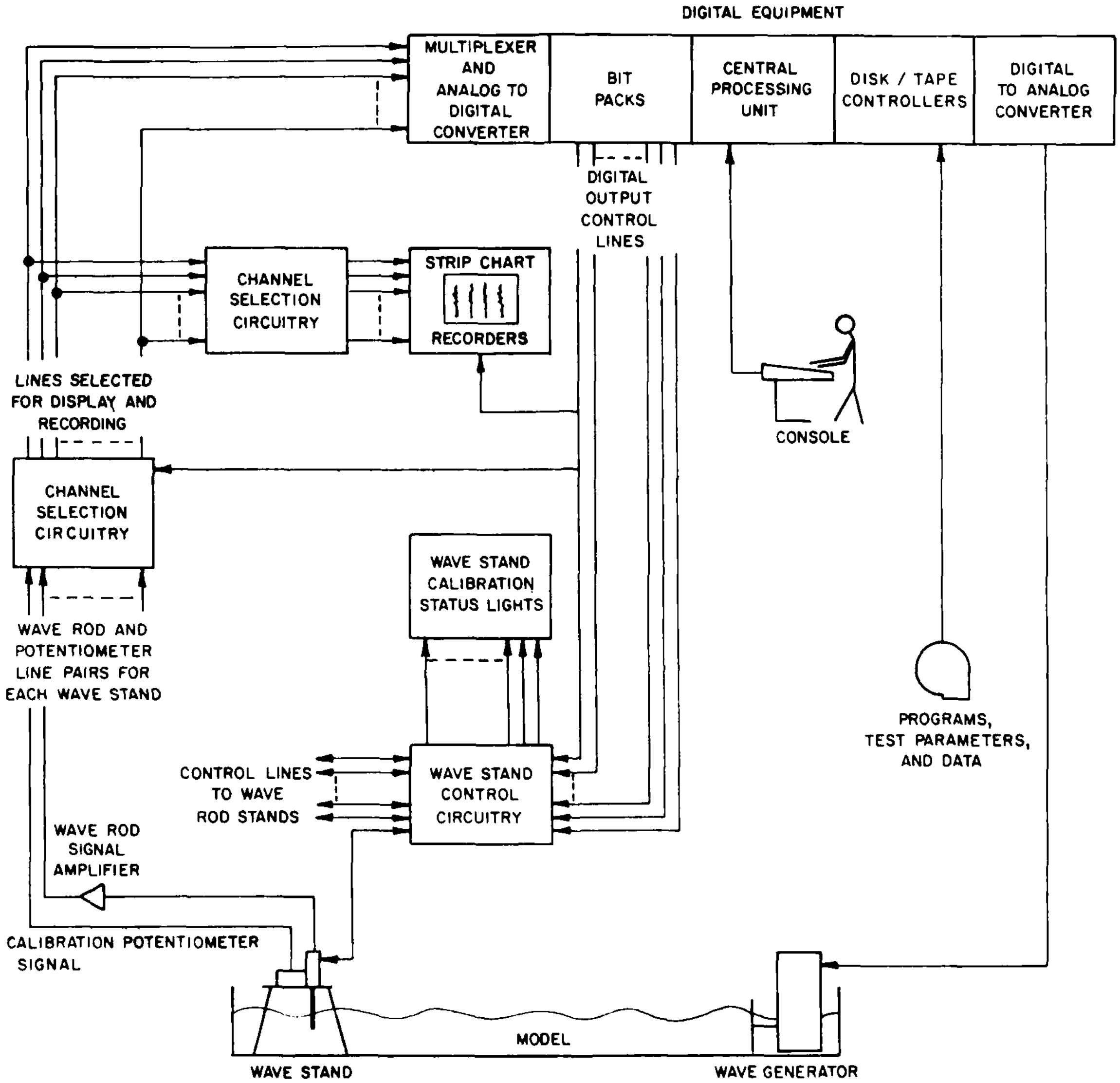


Figure 14. Automated Data Acquisition and Control System (ADACS).

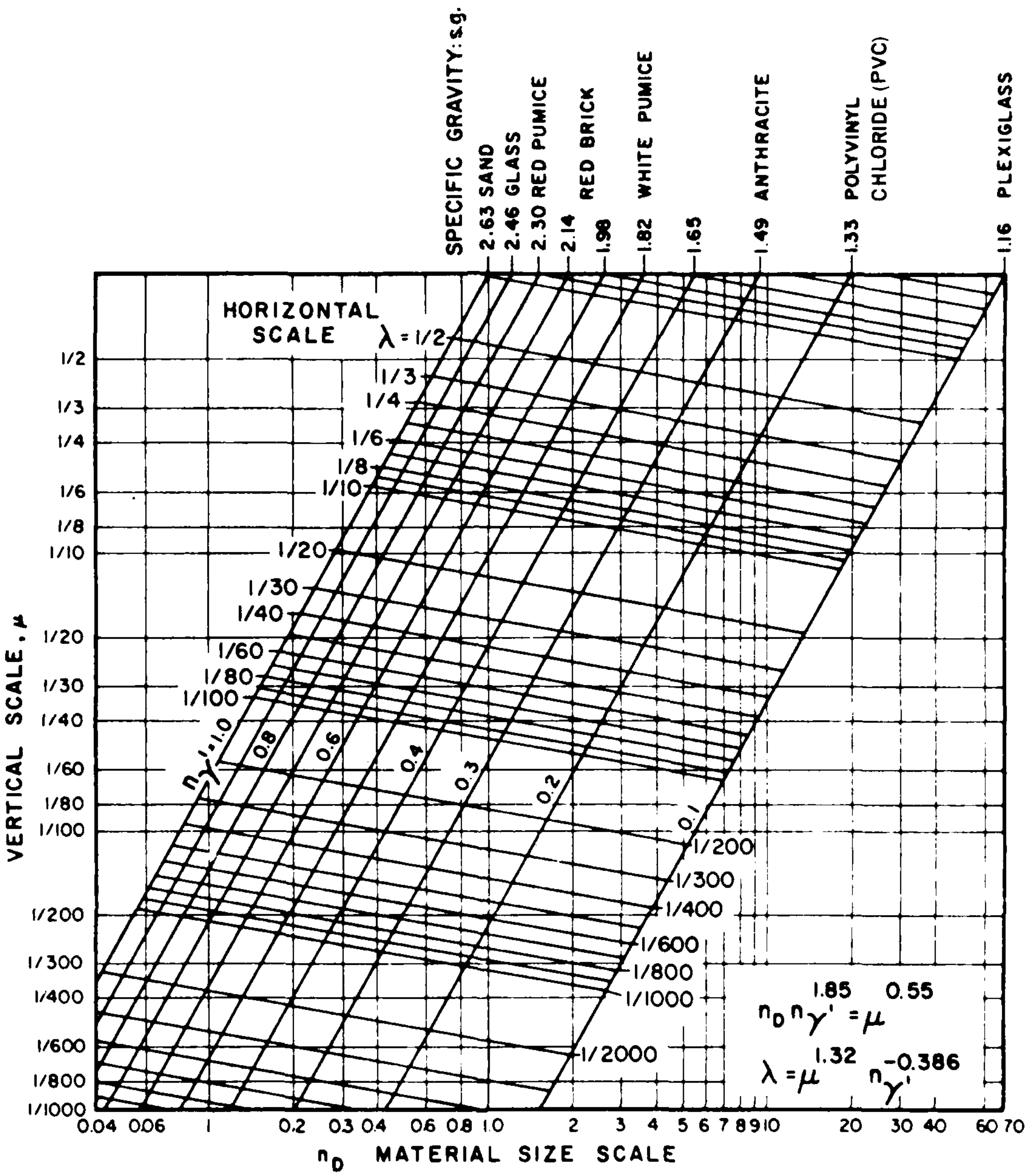


Figure 15. Graphic representation of model law (Noda, 1972).

short-period wave and current patterns, the following procedure was used to select a tracer material. Using the prototype sand characteristics (median diameter, D_{50} = .33 mm; specific gravity = 2.7) and assuming the horizontal scale to be in similitude (i.e. 1:75), the median diameter for a given specific gravity of tracer material and the vertical scale were computed. The vertical scale was then assumed to be in similitude and the tracer median diameter and horizontal scale were computed. This resulted in a range of tracer sizes for given specific gravities that could be used. Although several types of movable-bed tracer materials were available at WES, previous investigations (Giles and Chatham 1974, Bottin and Chatham 1975) indicated that crushed coal tracer more nearly represented the movement of prototype sand. Therefore, quantities of crushed coal (specific gravity = 1.30; median diameter, D_{50} = 0.84 mm) were selected for use as a tracer material throughout the model investigation.

PART IV: TEST CONDITIONS AND PROCEDURES

Selection of Test Conditions

Still-water level

59. Still-water levels (swl's) for wave action models are selected so that various wave-induced phenomena that are dependent on water depths are accurately reproduced in the model. These phenomena include refraction of waves in the project area, overtopping of structures by the waves, reflection of wave energy from various structures, and transmission of wave energy through porous structures.

60. In evaluating harbor response, it is desirable to select a model swl that closely approximates the higher water stages which normally occur in the prototype because this swl will likely correspond to the most severe conditions in the interior channels and basins. Also, the maximum amount of wave energy reaching a coastal area normally occurs during the higher water phase of the local tidal cycle, and most storms moving onshore are characteristically accompanied by a higher water level due to wind tide and shoreward mass transport. Wave penetration tests were conducted primarily at a water elevation corresponding to +7.0 ft (expected annual occurrence during the storm season), and results were checked at lower elevations. In addition, extreme high water conditions (+8.0 ft) were used in the model to simulate measured 1983 storm conditions at the site.

61. The numerical model of tidal circulation (Report 3 of the Bolsa Chica studies; Hales, 1989) indicated that maximum velocities will occur during the flood and ebb phases of the tidal cycle at a tide level of +2.8 ft for the navigable ocean entrance plans. For the nonnavigable ocean entrance plan, maximum flood and ebb phases occurred at swl's of 3.0 and 0.9 ft, respectively. Therefore, swl's of +0.9, +2.8 and +3.0 ft were selected for model testing of flood and ebb tidal flow conditions, depending on the ocean entrance plan tested. Tests were also conducted with swl's of 0.0 ft (mllw).

Factors influencing selection of test wave characteristics

62. In planning the testing program for a model investigation of harbor

wave-action problems, it is necessary to select dimensions and directions for the test waves that will allow a realistic test of proposed improvement plans and an accurate evaluation of the elements of the various proposals. In nature, surface-wind waves are generated primarily by the interactions between tangential stresses of wind flowing over water, resonance between the water surface and atmospheric turbulence, and interaction between individual wave components. The height and period of the maximum wave that can be generated by a given storm depend on the wind speed, the length of time that wind of a given speed continues to blow, and the water distance (fetch) over which the wind blows. Generally, selection of test wave conditions entails evaluation of such factors as:

a. The fetch and decay distances (the latter being the distance over which waves travel after leaving the generating area) for various directions from which waves can attack the problem area.

b. The frequency of occurrence and duration of storm winds from the different directions.

c. The alignment, size, and relative geographic position of the navigation entrance to the harbor.

d. The alignments, lengths, and locations of the various reflecting surfaces inside the harbor.

e. The refraction of waves caused by differentials in depth in the area seaward of the harbor, which may create either a concentration or a diffusion of wave energy at the harbor site.

Prototype wave data and selection of test waves

63. Measured prototype wave data over a sufficiently long time period on which to base a comprehensive statistical analysis of wave conditions for the Bolsa Chica area were not available. However, statistical wave hindcast estimates representative of this area, were developed as a wave information task (Jensen, in preparation) during the Bolsa Chica studies. Wave estimates were developed with a numerical grid spacing of five nautical miles in San Pedro Bay. Grid point 13, located in a 66-ft depth seaward of the proposed Bolsa Chica entrance, was the wave hindcast station used for this study. Hindcast data are summarized in Table 1.

Wave refraction

64. When wind waves move into water of gradually decreasing depth, transformations take place in all wave characteristics except wave period (to

the first order of approximation). The most important transformations with respect to the selection of test wave characteristics are the changes in wave height and direction of travel due to the phenomenon referred to as wave refraction. For regular waves, the change in wave height and direction are determined by calculating refraction coefficients (K_r) from deepwater to shallow-water, and multiplying them by the shoaling coefficient (K_s) to give conversion factors for transfer of deepwater wave heights to shallow water heights. The shoaling coefficient is a function of wave length and water depth and can be obtained from the Shore Protection Manual (1984).

65. Computations were performed to determine the variation of refraction coefficients from the selected wave hindcast station to the approximate location of the wave generator in the model. Shoaling coefficients were computed for a depth corresponding to the simulated depth in the wave generator pit. K_r multiplied by K_s gave conversion factors for transfer of wave conditions at the selected wave hindcast station to shallow water values (location of wave generator in model).

66. Refraction and shoaling coefficients and shallow-water directions were obtained at Bolsa chica for various wave periods from five directions (west counterclockwise to south). Based on refracted directions determined at the approximate locations of the wave generator in the model for each wave period, the following test directions (wave hindcast station direction and corresponding shallow-water direction) were selected for use during model testing. Directions pertain to direction of wave approach relative to true north, and they relate to the actual orientation of the Bolsa Chica site.

Wave Hindcast Direction deg	Selected Shallow-Water Test Direction deg
West, 270	259
West-Southwest, 247.5	242
Southwest, 225	225
South-Southwest, 202.5	207
South, 180	188

Hindcast wave estimates were converted to shallow-water values by application of refraction and shoaling coefficients and are shown in Table 2. Characteristics of test waves used in the model (selected from Table 2) are shown in the following tabulation:

Directions Represented at Hindcast Station	Selected Test Waves Period, sec	Height, ft
West	5	7
	7	8
	9	7,9
	11	7,9
	13	5,9
	15	8
West-Southwest	5	7
	7	7,9
	9	9,12
	11	7,10,13
	13	9,12,15
	15	7,10,12,15
	17	7,9,12,15
Southwest	5	7
	7	7,9
	9	7,10
	11	7,10
	13	7,10
	15	10
	17	5
South-Southwest	7	8
	9	8
	11	8
	13	8
South	5	7
	7	8
	9	8

67. Based on 20-year statistics at wave hindcast grid point 13 (Jensen, in preparation), a return period table was generated. This table indicated that waves with significant heights of 11.7 ft and 14.6 ft were representative of 1- and 21-year return periods, respectively. Wave periods of 14.3 sec were associated with both these wave conditions along with a direction corresponding to west-southwest. Therefore, 14.3-sec, 11.4- and 14.6-ft waves were generated in the model from the west-southwest direction and represented the most critical test conditions.

68. In addition, waves were generated in an effort to characterize 1983 storm conditions for various test plans. Based on limited data, it was estimated that waves with periods of 22 seconds and significant heights close to 20 ft occurred during this storm. Irregular waves with these characteristics could not be generated due to mechanical limitations of the wave generator, however; 22-sec, 17-ft monochromatic waves were within the capabilities of the wave machine and, consequently, were used in the model for these tests. These severe wave conditions were associated with the dominant (west-southwest) direction.

69. With the exception of the monochromatic test waves listed in paragraph 68, unidirectional irregular waves for the selected test conditions (based on JONSWAP spectral parameters) were generated and used throughout the model investigation. Plots of typical wave spectra are shown in Figure 16. The dashed line represents the desired spectra while the solid line represents the spectra generated by the wave generator. A typical wave train time-history is shown in Figure 17.

<u>Tidal flows and velocities</u>

70. The numerical tidal circulation studies conducted for Bolsa Chica (Hales, in preparation) indicated that tidal velocities through the proposed entrance and channel of the marina complex ranged from 0.3 to 0.65 fps for flood flow conditions and from 0.4 to 0.75 fps for ebb flows for navigable entrance conditions. For nonnavigable ocean entrance conditions maximum flood and ebb velocities through the entrance were 1.0 and 1.45 fps, respectively These values were reproduced in the model and verified prior to testing of flood and ebb tidal flow conditions.

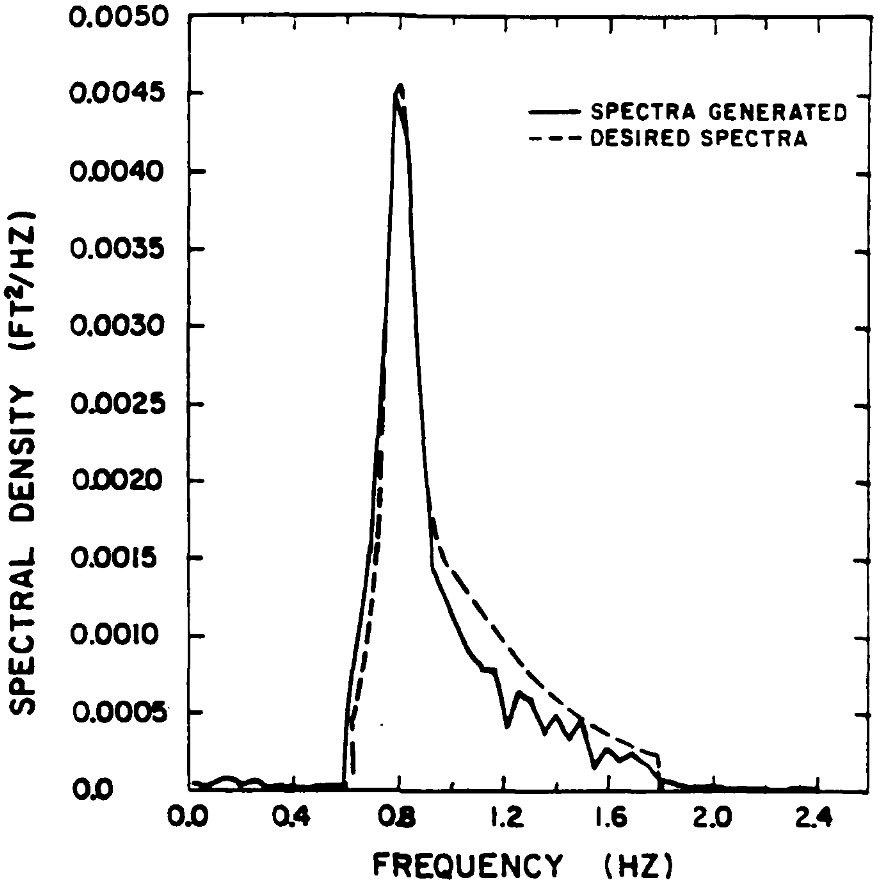


Figure 16. Typical wave-spectra plot, 11-sec, 10-ft waves

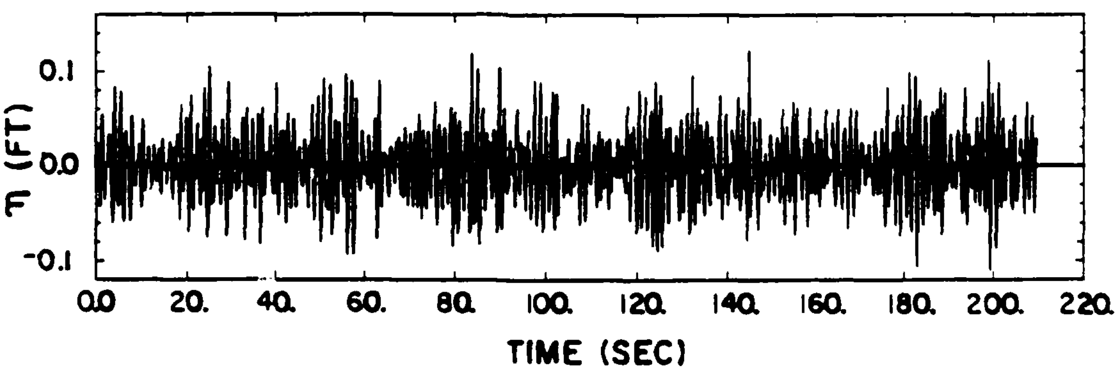


Figure 17. Typical wave train time-history, 11-sec, 10-ft test waves

Wintersburg Channel discharges

71. Proposed plans indicate that the existing Wintersburg flood-control channel will discharge into the eastern portion of the marina complex. The channel, which drains an area of about 18,000 acres, is earth-lined and trapezoidal in shape. Based on recent hydrology study (Moffatt and Nichol, Engineers 1986) for the water shed, channel discharges of 1760- and 9710-cfs were projected for recurrence intervals of 2 and 100 years, respectively. These discharge values were simulated during model testing of the Wintersburg Channel.

Analysis of Model Data

72. Relative merits of various improvement plans tested were evaluated by:

a. Comparison of wave heights at selected locations in the model.

b. Comparison of sediment tracer movement and subsequent deposits.

c. Visual observations, wave pattern photographs, and videotape footage.

In analyzing wave-height data the energy-based wave (H_{mo} - energy based on four times the standard deviation of the surface elevation data at each gage, usually equivalent in deep water to the significant wave heights, $H_{1/3}$) were tabulated to show values at selected gage locations. Wave heights analyzed included energy in the .05-.2 hz frequency band (approximately 4-22 sec prototype) in the wave spectra. All wave heights then were adjusted to compensate for excessive model wave height attenuation due to viscous bottom friction by application of Keulegan's equation (Keulegan 1950). From this equation reduction of wave heights in the model (relative to the prototype) can be calculated as a function of water depth, width of wave front, wave period, water viscosity, and distance of wave travel. Current magnitudes for Wintersburg Channel discharges were determined by timing the progress of a weighed float over known distances on the model floor.

Wave-Height Criteria

73. Completely reliable criteria have not yet been developed for ensuring satisfactory navigation and mooring conditions in small-craft harbors during attack by waves. For this study, SLC specified that for an improvement

plan to be acceptable, maximum wave heights were not to exceed 4.0 ft in the entrance (between the jetty heads); 1.5 ft in the interior basins for incident test waves with a 21 year recurrence interval; and 1.0 ft in the interior basins for incident wave conditions with an annual reoccurrence.

PART V: TEST AND RESULTS

The Tests

Test plans

74. Wave height tests were conducted for 18 variations in the design elements of three basic improvement plans. The plans consisted of a proposed navigable entrance, both with and without a connector channel to Huntington Harbour, and a non-navigable entrance in the vicinity of Bolsa Bay. Variations entailed changes in the lengths and/or cross-sections of the proposed breakwater and jetties, and the addition of revetments and/or baffles in the interior channels and basins of the marina complex. Sediment tracer tests, wave pattern photographs, and/or videotape footage also were obtained for selected test plans. Brief descriptions of the improvement plans are presented in the following subparagraphs; dimensional details are presented in Plates 1-10.

Navigable Ocean Entrance With Connector Channel

a. Plan 1 (Plate 1) consisted of the proposed marina complex with parallel jetties (el +14 ft) spaced 1,000 ft apart and extending seaward to the -20 ft contour. A 3,150-ft-long offshore breakwater (el +18 ft) was located about 750 ft seaward of the jetty heads. The plan also included an 800-ft-wide entrance channel (el -20.2 ft) and a 350-ft-wide connector channel (el -12 ft) that would extend northwesterly to Huntington Harbour.

b. Plan 2 (Plate 2) involved the elements of Plan 1 but approximately 950 ft of revetment was installed on the north side of the interior channel and about 1,050 ft on the south side.

c. Plan 3 (Plate 2) included the elements of Plan 1 and the revetment of Plan 2 but 900 ft of the offshore breakwater seaward of the entrance channel was raised and sealed to prevent wave transmission and overtopping of this portion of the structure.

d. Plan 4 (Plate 3) entailed the elements of Plan 1; the revetment of Plan 2; the raised and sealed breakwater section of Plan 3; and a 300-ft-long solid baffle across the opening to the northwest basin (Basin 1) of the marina.

e. Plan 5 (Plate 3) consisted of the elements of Plan 4 but the 300-ft-long solid baffle across the opening to the northwest basin was replaced with a rubble baffle.

f. Plan 6 (Plate 4) involved the elements of Plan 1; the revetment of Plan 2; and the rubble baffle of Plan 5. The 900 ft raised and sealed breakwater section of Plan 3 was restored to its original crest el of +18 ft.

g. Plan 7 (Plate 5) entailed the elements of Plan 1; the revetment of Plan 2; and the rubble baffle of Plan 5. The crest el of the 900 ft breakwater section seaward of the entrance channel, however, was raised to +22 ft.

h. Plan 8 (Plate 5) included the elements of Plan 7 but the 300-ft-long rubble baffle was removed.

Navigable Ocean Entrance Without Connector Channel

i. Plan 9 (Plate 6) consisted of the proposed marina complex with parallel jetties (el +14 ft) spaced 700 ft apart and extending seaward to the -20 ft contour. A 2,850-ft-long offshore breakwater was located about 750 ft seaward of the jetty heads. The crest el of a 600 ft section of the offshore breakwater was installed at an el of +22 ft with the winged ends installed at an +18 ft el. The plan also included a 500-ft-wide entrance channel and a revetted berm across Outer Bolsa Bay.

j. Plan 10 (Plate 7) included the elements of Plan 9 with approximately 825 ft of revetment on the north side of the interior channel and about 1,050 ft on the south side.

k. Plan 11 (Plate 7) entailed the elements of Plan 9; the revetment of Plan 10; and a 150-ft-long rubble baffle across the opening to the northwest basin (Basin 1) of the marina.

l. Plan 12 (Plate 7) involved the elements of Plan 11 but the rubble baffle was increased from 150 to 300 ft in length.

m. Plan 13 (Plate 8) consisted of the elements of Plan 9; the revetment of Plan 10; and the baffle of Plan 12 but the north wing of the offshore breakwater was reduced in length from 1,500 to 750 ft resulting in a 2,100-ft-long breakwater.

n. Plan 14 (Plate 8) included the elements of Plan 13 but the north jetty was raised and sealed to prevent wave transmission and overtopping of the structure.

o. Plan 15 (Plate 9) entailed the elements of Plan 9; the revetment of Plan 10; the baffle of Plan 12; and the 2,100-ft-long offshore breakwater of Plan 13 (750 ft reduction of north wing), but the crest el of the north jetty was raised from +14 to +18 ft.

p. Plan 16 (Plate 9) consisted of the elements of Plan 9; the revetment of Plan 10; the baffle of Plan 12; and the +18 ft north jetty crest el of Plan 15; but the offshore breakwater wings were reduced by 750 ft on the north (Plan 13) and 250 ft on the south resulting in an 1,850-ft-long structure.

q. Plan 17 (Plate 9) involved the elements of Plan 16 with an additional 150 ft of the north wing of the offshore breakwater removed resulting in a 1700-ft-long structure.

Nonnavigable Ocean Entrance

r. Plan 18 (Plate 10) consisted of a 160-ft-wide non-navigable entrance channel (el -2.2 ft) extending from the coastline through the southeastern portion of Outer Bolsa Bay. Jetties (el +14.8 ft) also paralleled the new entrance channel. They originated shoreward of the existing Pacific Coast Highway and terminated on the beach at mean high tide (el +4.6 ft). Contours upstream of the entrance were not reproduced, however, the approximate tidal prism was included.

Wave-height tests

75. Wave-height tests for the various improvement plans were conducted using test waves from one or more of the directions listed in paragraph 66. Tests involving certain proposed plans of improvement were limited to the most critical directions of wave approach (i.e. west-southwest, west, and/or south). Some plans were tested for waves from all five test directions. Wave gage locations for each improvement plan are shown in the referenced plates.

Sediment tracer tests

76. Sediment tracer tests were conducted for the most promising improvement plans using representative test waves and swl's for the various test directions. Some of the improvement plans were limited to test waves from the most critical direction (i.e. west or south) with respect to accretion in the entrance. Tracer material was introduced into the model north of the north jetty and south of the south jetty to represent sediment from those shorelines, respectively, as shown in Photos 11 and 12.

Wave patterns and videotape

77. Wave pattern photographs and slides and videotape footage were secured for various test plans showing the area under attack by storm waves and/or the location of sediment deposits. Videotape footage also was obtained which depicted various testing procedures. These data were furnished to SLC for use in briefings, public meetings, etc.

Wintersburg Channel current tests

78. Current patterns and magnitudes resulting from discharges from the Wintersburg Channel were obtained in the area where the channel entered the marina complex. These data were secured for discharges with 2- and 100-yr recurrence intervals.

Test Results

79. In evaluating test results, relative merits of various plans were based on an analysis of measured wave heights in the marina entrance and mooring areas, and/or movement of sediment tracer material and subsequent deposits. Model wave heights were tabulated in prototype units to show measured values at selected locations. The general movement of tracer material and subsequent deposits were shown in photographs. Arrows were superimposed on these photos to depict sediment movement patterns. Current patterns and magnitudes from the Wintersburg Channel also were superimposed on to photographs for the channel discharges tested.

Navigable ocean entrance with connector channel

80. Results of wave height tests conducted for Plans 1-8 are presented in Table 3 for test waves from west-southwest with the 7.0 ft swl. Wave heights in the entrance (between the jetty heads, gage 1) ranged from 2.1 to 3.1 ft for 14.3-sec, 11.7-ft test waves (1-yr recurrence) and from 2.4 to 3.7 ft for 14.3-sec, 14.6-ft test waves (21-yr recurrence) for Plans 1-8. All these values were within the established 4.0 ft wave height criterion at this location. Maximum wave heights in the interior basins (gages 6-13) were 2.0, 1.5, 1.0, 0.9, 0.8, 1.1, 0.9, and 1.2 ft for Plans 1-8 respectively, for 14.3-sec, 11.7-ft test waves. For 14.3-sec, 14.6-ft test waves, maximum wave heights were 2.3, 1.7, 1.0, 0.9, 0.8, 1.2, 1.0, and 1.3 ft in the interior basins for Plans 1-8, respectively. While Plans 3-8 met the established wave height criterion of 1.5 ft for 21-yr recurrence waves, only Plans 3, 4, 5, and 7 met the 1.0 ft wave height criterion for incident waves with a 1-yr recurrence interval. Some of the structural elements of Plans 1-8 are shown in Photos 1-5.

81. After evaluation of wave-height test results for Plans 1-8, Plan 7 (+22 ft raised offshore breakwater section, absorber, and rubble baffle) was considered the best plan to this point and selected for further testing.

82. Wave heights obtained for Plan 7 for test waves from all five directions with the +7.0 ft swl are presented in Table 4. Maximum wave heights obtained in the entrance (gage 1) were 2.6 ft for 13-sec, 15-ft and 15-sec, 15-ft test waves from west-southwest; and maximum wave heights in the interior basins were 1.2 ft (gage 9) for 15-sec, 10-ft test waves from southwest.

Typical wave patterns obtained for Plan 7 with the +7.0 ft swl are shown in Photos 6-10.

83. Wave heights obtained with Plan 7 installed for the +2.8 ft swl with maximum flood tidal conditions are presented in Table 5 for test waves from all five directions. Maximum wave heights were 1.6 ft in the entrance (gage 1) for 13- and 15-sec, 15-ft waves from west-southwest and 11-sec, 8-ft waves from south-southwest; and 0.6 ft in the interior basins (gages 9 and 11) for 15-sec, 15-ft waves from west-southwest and 15-sec, 10-ft waves from southwest.

84. Wave height test results for Plan 7 for the +2.8 ft swl with maximum ebb tidal conditions are presented in Table 6 for test waves from the five test directions. Maximum wave heights in the entrance (gage 1) were 1.8 ft for 11-sec, 8-ft waves from south-southwest; and maximum wave heights in the interior basins (gage 9) were 0.5 ft for 15-sec, 15-ft test waves from west-southwest and 11- and 13-sec, 8-ft waves from south-southwest.

85. Results of wave height tests with Plan 7 installed for test waves with the 0.0 ft swl for all five directions are presented in Table 7. Maximum wave heights were 1.7 ft in the entrance (gage 1) for 11-sec, 8-ft waves from south-southwest and 0.3 ft in the interior basins (gage 9) for several of the test waves.

86. The general movement of tracer material and subsequent deposits obtained for Plan 7 for representative test waves from the west are shown in Photos 13-16. The tracer initially placed in the model (Photos 11-12) was subjected to a range of wave conditions (9-sec, 7-ft; 11-sec, 9-ft; 13-sec, 9-ft; and 15-sec, 8-ft test waves) with the 0.0 ft swl. Progressively, the deposits were subjected to these wave conditions with the +7.0 ft swl, and then the +2.8 ft swl (first with maximum flood flow conditions and finally for maximum ebb flow conditions). Sediment tracer moved southerly along the shoreline and in the breaker zone. Some material deposited along the seaward side of the breakwater, however, it did not enter the entrance between the offshore breakwater and the north jetty. It was noted that some tracer penetrated the voids of the jetty at the shoreline and deposited on the channel side of the structure.

87. The movement of tracer material and deposits resulting from representative test waves from west-southwest with Plan 7 installed are shown

in Photos 17-20 for the various swl's and maximum tidal flow conditions. Waves selected for use with this test direction were 9-sec, 9-ft; 13-sec, 12-ft; and 15-sec, 15-ft. Tracer movement was similar to that for test waves from west. Sediment moved southerly along the shoreline and in the breaker zone accumulating along the north jetty. It did not enter the entrance, and some tracer penetrated the voids of the jetty at the shoreline depositing on the inside (channel side) of the jetty.

88. The general movement of tracer material and deposits resulting from representative test waves (7-sec, 9-ft; 11-sec, 10-ft; and 15-sec, 10-ft) from southwest for the various swl's and tidal flows with Plan 7 installed are shown in Photos 21-28 for both the north and south shorelines. On the north shoreline, sediment in general, moved southerly with the majority accumulating along the shoreline. Small deposits occurred along the north jetty, but not to the extent as it occurred for test waves from west and west-southwest. Sediment on the south shoreline moved northerly and then in a counterclockwise eddy along the south jetty. Material deposited alongside the jetty but did not migrate into the entrance between the offshore breakwater and the head of the south jetty. Material along the shoreline accreted against the south jetty, migrated through the voids of the structure, and deposited on its channel side.

89. The movement of tracer material and deposits resulting from test waves from the south-southwest for the swl's and maximum flow conditions are shown in Photos 29-32 for Plan 7. Representative test waves from this direction were 9-sec, 8-ft and 13-sec, 8-ft waves. Tracer material moved northerly and deposited along the shoreline and the south jetty. Some penetrated the south jetty at the shoreline and deposited along the channel side of the structure.

90. The general movement of tracer and subsequent deposits obtained for Plan 7 for representative test waves from south (5-sec, 7-ft and 9-sec, 8-ft waves) are shown in Photos 33-36 for the selected swl's and maximum tidal flow conditions. Material for this direction moved northerly and deposited along the shoreline and adjacent to the south jetty. No sediment moved into the entrance, however, some penetrated the jetty at the shoreline and deposited on the channel side of the structure.

91. Current patterns and magnitudes throughout the interior channels of the marina complex due to discharges from the Wintersburg Channel are shown in Photos 37 and 38. Current magnitudes in the interior channels ranged from 0.4 to 0.6 fps for the 1,760-cfs discharge and from 1.4 to 1.9 fps for the 9,710-cfs discharge. In general, currents flowed from the Wintersburg Channel through the interior basin channels toward the navigable entrance. Currents in the northern basins eddied clockwise while currents south of the interior channel eddied in a counterclockwise direction. Magnitudes of the currents in the basins were negligible. Coal tracer material placed in the Wintersburg Channel, to represent bed load riverine sediment, did not move into the interior channel even for the 100-yr discharge.

Navigable ocean entrance without connector channel

92. Results of wave height tests conducted for Plans 9-12 for test waves from west-southwest with the +7.0 ft swl are presented in Table 8. Wave heights in the entrance between the jetty heads (gage 1) ranged from 2.1 to 2.3 ft for 14.3-sec, 11.7-ft test waves (1-yr recurrence) and from 2.3 to 2.6 ft for 14-sec, 14.6-ft test waves (21-yr recurrence) for Plans 9-12. These values were all within the established 4.0 ft wave height criterion at this location. Maximum wave heights in the interior basins (gages 6-13) were 1.5, 1.2, 0.9, and 0.9 ft for Plans 9-12, respectively, for 14.3-sec, 11.7-ft test waves. For 14.3-sec, 14.6-ft test waves, maximum wave heights were 1.7, 1.2, 1.0, and 1.0 ft in the interior basins for Plans 9-12, respectively. Plans 10-12 met the established wave height criterion of 1.5 ft for 21-yr recurrence waves, while Plans 11 and 12 met the 1.0 ft criterion for incident waves with a 1-yr recurrence interval.

93. Results of wave height tests for Plans 11 and 12 for additional test waves from west-southwest with the +7.0 ft swl are presented in Table 9. Maximum wave heights were 2.9 ft in the entrance for both plans. In the interior basins, maximum wave heights were 1.4 and 1.2 ft, respectively, for Plans 11 and 12.

94. Wave heights measured for Plan 12 for test waves from the southwest and west directions are presented in Table 10 using the +7.0 ft swl. Maximum wave heights were 2.4 ft in the entrance for 13-sec, 9-ft test waves from west; and 0.8 ft in the interior basins for 15-sec, 10-ft test waves from southwest and 13-sec, 9-ft test waves from west. Typical wave patterns for

Plan 12 are shown in Photos 39-41 for the west, west-southwest, and southwest directions.

95. Wave height test results for Plans 13 and 14 are presented in Table 11 for representative test waves from west with the +7.0 ft swl. Maximum wave heights were 2.9 and 2.7 ft in the entrance; and 1.1 and 0.7 ft in the interior basins for Plans 13 and 14, respectively. Wave patterns for the reduced north wing breakwater length of Plan 13 are shown in Photo 42.

96. Results of wave height tests for representative test waves from west-southwest with Plans 13 and 14 installed are presented in Table 12 for the +7.0 ft swl. Maximum wave heights obtained in the entrance were 3.6 and 3.3 ft, and maximum wave heights in the interior basins were 1.5 and 0.9 ft for Plans 13 and 14, respectively.

97. Wave heights obtained for Plans 15-17 for representative test waves from west-southwest with the +7.0 ft swl are presented in Table 13. Maximum wave heights were 3.7, 3.9, and 3.6 ft in the entrance; and 1.4, 1.5, and 1.8 ft in the interior basins for Plans 15-17, respectively. Typical wave patterns for Plans 15-17 are shown in Photos 43-45 for test waves from west-southwest.

98. Wave height test results obtained for Plan 16 for test waves from west and for Plan 17 for test waves from west and south are shown in Table 14. For test waves from west, maximum wave heights were 2.9 and 3.1 ft in the entrance and 1.1 and 1.6 ft in the interior basins for Plans 16 and 17, respectively. Test waves from south resulted in maximum wave heights of 3.6 ft in the entrance and 0.6 ft in the interior basins for Plan 17. Typical wave patterns for Plans 16 and 17 from test waves from west and/or south are presented in Photos 46-48.

99. The general movement of tracer material and subsequent deposits obtained for representative test waves from west for the 750 ft and 1,000 ft reduced north breakwater wings of Plans 15 and 17, respectively, are shown in Photos 49-52 for the 0.0 and +7.0 ft swl's. Tracer material moved southerly along the shoreline and in the breaker zone. Some material moved seaward along the north jetty and into the entrance opening between the head of the jetty and the offshore breakwater. With the 750 ft reduction (Plan 15) less material migrated into the entrance and it appeared that only the finer particles of sediment moved to this location as opposed to the 1,000 ft wing

reduction of Plan 17. For both plans tracer material penetrated the voids of the rubble-mound jetty at the shoreline and deposited on the channel side of the structure. Maximum flood and ebb tidal flow conditions with the +2.8 ft swl indicated little or no effect on sediment deposits in the entrance; and therefore, these tests were not documented and tracer tests were discontinued for this test plan series.

100. The movement of tracer material and deposits resulting from representative test waves from south for the original south breakwater wing length (Plan 15) and the 250 ft south wing reduction (Plan 16) are shown in Photos 53-56 for the 0.0 and +7.0 ft swls. Tracer material moved northerly along the shoreline with some moving seaward along the south breakwater. The original length of the south wing (Plan 15) resulted in no deposits in the entrance; however, the reduced wing length of Plan 16 did result in deposits in the entrance between the offshore breakwater and the south jetty head. Sediment along the shoreline penetrated the voids of the south jetty and deposited on the channel side of the structure for both plans.

Non-navigable ocean entrance

101. Results of wave height tests for the non-navigable ocean entrance plan (Plan 18) are presented in Table 15 for representative test waves from all directions with all swl's. Maximum wave heights obtained in the entrance (gage 1) were 2.3, 2.8, 3.5, and 6.8 ft for the 0.0-, +0.9- (maximum ebb), +3.0- (maximum flood) and +7.0-ft swl's, respectively. Representative wave patterns for Plan 18 with the +7.0 ft swl are shown in Photos 57-61 for all five directions.

102. The general movement of tracer material and deposits resulting from representative test waves with the 0.0 and +7.0 ft swl's are presented in Photos 62-71 for various test directions with Plan 18 installed. Tracer material moved southerly for test waves from west and west-southwest and northerly for test waves from southwest, south-southwest, and south. Test waves from all directions indicated that some tracer material would enter the non-navigable ocean entrance channel and some would bypass the entrance. Tidal flow conditions and associated swl's had little or no effect on the movement of tracer material in the new entrance.

Severe storm conditions

103. Results of wave height tests for Plans 7, 12, and 18 for 22-sec, 17-ft test waves from west-southwest with the +8.0 ft swl (simulated 1983 storm conditions) are presented in Table 16. Maximum wave heights were 3.7, 3.4, and 5.6 ft in the entrance (gage 1) for Plans 7, 12, and 18, respectively; and 1.4 and 2.1 ft in the interior basins (gages 6-13) for Plans 7 and 12, respectively.

Discussion of Test Results

Navigable ocean entrance with connector channel

104. Results of wave height tests for the navigable ocean entrance with the connector channel to Huntington Harbour revealed that the originally proposed plan (Plan 1) failed to meet the established wave height criteria in the interior basins of the marina complex. The 1.0 ft criterion for 1-yr recurrence waves was exceeded by 1.0 ft, and the 1.5 ft criterion for 21-yr recurrence waves was exceeded by 0.8 ft. The revetment installed along each side of the interior channel (Plan 2) reduced wave heights to 1.5 and 1.7 for 1- and 21-yr recurrence waves, respectively. The raised breakwater section of Plans 3-5 was installed to prevent overtopping and transmission through the structure. All these plans met the desired wave height criteria which indicated that wave energy was entering over and/or through this portion of the +18 ft breakwater for the previous tests (Plans 1 and 2). The 300-ft-long rubble baffle of Plan 5 appeared to further reduce wave heights in the interior basin with the sealed outer breakwater. The baffle, along with the original (+18 ft) breakwater (Plan 6), however, resulted in wave heights that exceeded the criterion in the interior basins. Installation of the +22 ft breakwater section with the revetment and baffle (Plan 7) met the wave height criteria, however, when the baffle was removed wave heights in one of the interior basins increased and exceeded the established criterion. The Plan 7 test plan configuration (+22 ft breakwater section, revetment along each side of the interior channel, and 300 ft rubble baffle) was determined to be an acceptable plan based on wave heights obtained in the entrance and interior basins for incident waves with 1- and 21-yr recurrence intervals.

105. Initially, during model wave-height testing, plans were developed to meet the established wave height criteria based on the 1- and 21-yr recurrence intervals. The plan selected then was subjected to the range of wave conditions based on hindcast data. The larger hindcasted waves would occur more frequently than 21 yrs (based on number of occurrences in tables), and therefore should be less than 1.5 ft for the test plan to be acceptable.

106. Based on wave height data obtained with Plan 7 installed 0.0-, +2.8- (with the maximum flood and ebb tidal flows), and +7.0-ft swl's, the +7.0 ft swl resulted in significantly larger wave heights in the entrance channel and interior basins. All swl's resulted in wave heights well within the established criteria in the entrance (4 ft). In the interior basins, the +7.0 ft swl resulted in maximum wave heights of 1.2 ft, while maximum wave heights for the other swl's ranged from 0.3 to 0.6 ft. Maximum wave heights, therefore, expected in the entrance and interior basins of the marina complex are 2.6 and 1.2 ft, respectively, for hindcast wave conditions with Plan 7 installed. These will occur at the +7.0 ft swl.

107. During preliminary wave height testing, visual observations revealed long period surging of the interior basins. These surges oscillated with periods in the 100-sec range. Several structural alternatives (absorbers, baffles, etc) were installed in the interior basins in an effort to reduce surge conditions, and it was noted that baffles across portions of the interior basins were the most effective alternatives. The scope of this study did not include testing long period wave conditions, however, oscillations of these periods normally do not impact operation of small boat harbors. Prior to construction of an improvement plan at Bolsa Chica, it is recommended that long-period wave testing be conducted to check/optimize interior harbor alternatives.

108. Results of tracer tests for Plan 7 indicated that sediment would not deposit in the entrance (between the offshore breakwater and the jetty heads) for any of the test conditions. For test waves from west, west-southwest, and southwest tracer material on the north shoreline moved south and deposited on the shoreline and adjacent to the north jetty. Test waves from southwest, south-southwest, and south resulted in a northerly movement of tracer from the south shoreline. Sediment deposited on the shoreline and adjacent to the south jetty. Sediment tracer material on the shorelines

adjacent to the jetties penetrated through the rubble mound structures and resulted in deposits on their channel sides. Also, the flood and ebb tidal flows appeared to have little or no effect on sediment deposited near the jetty heads.

109. Test results for discharges from Wintersburg Channel indicated that current patterns and magnitudes should have minimal impacts in the interior channels and basins of the marina for Plan 7. Maximum velocities ranged from 0.4 to 0.6 fps and 1.4 to 1.9 fps in the interior channels of the marina for the 2- and 100-yr discharges, respectively. The model also indicated that the movement of bed-load sediment into the interior channels would not occur for the conditions tested. Suspended sediment may deposit, however, in areas where current magnitudes are reduced (i.e., in areas where eddies are formed).

Navigable ocean entrance without connector channel

110. Results of wave height tests for the navigable ocean entrance without the connector channel to Huntington Harbour indicated that the originally proposed plan (Plan 9) failed to meet the wave height criteria in the interior basins of the marina complex. The 1.0 ft criterion for 1-yr recurrence waves was exceeded by 0.5 ft and the 1.5 ft criterion for 21-yr recurrence waves by 0.2 ft. The revetment installed along the sides of the interior channel (Plan 10) reduced wave heights to within 0.2 ft of the criterion for 1-yr waves and met the criterion for 21-yr recurrence waves. Both the 150- and 300-ft-long rubble baffles of Plans 11 and 12, respectively, resulted in wave conditions within the established criterion. The 300-ft-long baffle (Plan 12), however, reduced wave heights in the corner of the northwest basin to 0.6 ft as opposed to 0.9 ft for the 150-ft-long baffle Plan 11.

111. A comparison of wave heights for Plans 11 and 12 for hindcast waves from west-southwest indicated that maximum wave heights in the interior basins were 0.2 ft less for the 300-ft-long baffle (Plan 12) as opposed to the 150-ft-long baffle (Plan 11). Plan 12, therefore, was selected for additional testing. Based on test results for the navigable ocean entrance plan with the connector channel (Plan 7), wave characteristics, wave directions, and swl's were limited to the most severe conditions.

112. Wave heights obtained with Plan 12 installed revealed that the west-southwest direction was the most critical with regard to wave conditions

in the interior basins. Maximum wave heights expected in the entrance and interior basins of the marina complex are 2.9 ft and 1.2 ft, respectively, for hindcast wave conditions.

113. The removal of 750 ft of the north wing of the offshore breakwater (Plan 13) resulted in an increase in maximum wave heights in the interior basins by 0.3 ft for test waves from both the west and west-southwest directions. Raising and sealing of the north jetty to prevent overtopping and transmission through it (Plan 14), however, substantially reduced wave heights in the basins, indicating that wave energy was entering through the jetty. The 750-ft reduction of the north breakwater wing with an increase in the crest el of the north jetty from +14 ft to +18 ft (Plan 15) resulted in maximum wave heights of 1.4 ft in the interior basins (0.2 ft higher than Plan 12 and 0.1 ft lower than the Plan 13 configuration). The 750-ft reduction of the north wing and the +18 ft north jetty with a 250-ft reduction of the south wing of the offshore breakwater (Plan 16) will result in maximum wave heights of 1.5 ft in the interior basins; and an additional 250-ft reduction of the north wing (Plan 17) will increase maximum wave heights to 1.8 ft in the basins. The 1,000 ft reduced north breakwater wing length of Plan 17 will also result in wave heights of 1.6 ft in the interior basins for test waves from west.

114. Based on the results of sediment tracer tests conducted for the navigable ocean entrance with the connector channel (Plan 7), sediment deposits on the north side of the entrance should be identical for Plan 12 since the location of the north wing of the offshore breakwater and the north jetty had not changed and test conditions were the same. These tests indicated that tracer material would deposit along the shoreline and on the seaward side of the north jetty, but would not result in sediment deposits in the entrance between the north jetty and the offshore breakwater.

115. Removal of 750 ft or more of the north wing of the offshore breakwater (Plans 15 and 17) indicated that sediment would migrate along the north jetty and deposit in the entrance between the breakwater and the head of the north jetty for test waves from west. For test waves from south the original south breakwater wing (Plan 15) was effective in preventing deposits in the entrance. The 250 ft reduction of the south wing of the breakwater (Plan 16), however, resulted in sediment deposits between the offshore

breakwater and the south jetty head. Tracer material on the shorelines migrated adjacent to the jetties and penetrated the voids of the structures resulting in deposits on their channel sides. The flood and ebb tidal flows appeared to have little or no effect on sediment deposits in the entrance and test conditions with the +2.8 ft swl were not tested comprehensively for this test series.

116. Even though the navigable ocean entrance without the Huntington Harbour connector channel (Plans (9-17) was reduced in width (from 800 to 500 ft), discharges from the Wintersburg Channel should be similar to the plan tested previously (Plan 7) since the cross-sectional area in the entrance is far greater than in the interior channels. The cross-sectional area and configuration of the interior channels was the controlling factor with regard to discharge profiles and velocities and remained unchanged for the test plans without the connector channel.

Non-navigable ocean entrance

117. Wave height tests for the non-navigable ocean entrance of Plan 18 indicated that wave heights in the entrance increased for the higher swl's tested. Maximum wave heights ranged from 5.0 to 6.8 ft for the +7.0 ft swl for all five test directions, however, they decreased to maximums ranging from 1.4 to 2.2 ft at a point about 750 ft upstream (gage 2) and from 0.5 to 1.0 ft approximately 1,200 ft upstream (gage 3).

118. Results of tracer tests for the non-navigable ocean entrance indicated that sediment would move either northerly or southerly along the shoreline depending on the direction of incident wave approach. For all test directions some sediment tracer by-passed the non-navigable ocean entrance while some entered the entrance with deepest penetration for test waves from west, west-southwest, and southwest. Maximum flood and ebb tidal flow conditions appeared to have little effect on the limited amount of tracer injected into the model.

Severe storm conditions

119. Results of wave height tests for severe wave conditions from west-southwest with an extreme swl (+8.0 ft), ich simulated estimated 1983 storm

conditions, indicated that the navigable ocean entrance with the Huntington Harbor connector channel (Plan 7) afforded more wave protection than the navigable ocean entrance without the connector channel (Plan 12). Undesirable wave conditions in the interior basins of Plan 12, however, could probably be alleviated by the installation of additional baffles across portions of the basin entrances, or possibly a realignment of the revetted berm across outer Bolsa Bay. Wave heights in the non-navigable ocean entrance channel (Plan 18) for severe storm conditions did not appear to differ from the hindcasted wave conditions tested.

PART VI: CONCLUSIONS

120. Based on the results of the hydraulic model investigation reported herein, it is concluded that:

Navigable ocean entrance with connector channel

a. The originally proposed improvement plan (Plan 1) will not meet the established wave height criteria in the interior basins (wave heights not to exceed 1.0 ft for wave conditions with 1-year recurrence interval and 1.5 ft for waves with 21-year interval).

b. Wave conditions in the interior basins that will meet the established criteria may be achieved by the installation of revetments along the interior channels and a 300-ft-long spur across the opening of the northwest basin, in conjunction with raising the crest elevation of a 900-ft-long portion of the offshore breakwater from +18 to +22 ft (Plan 7).

c. The +7.0 ft still water level (swl) resulted in significantly larger waves in the entrance and interior channels than the +2.8 ft swl with the maximum flood and ebb tidal flows and the 0.0 ft swl for Plan 7.

d. Maximum wave heights that may be expected in the entrance and interior basins of the marina complex are 2.6 ft and 1.2 ft, respectively, for Plan 7 using hindcast wave conditions.

e. The lengths of the north and south wings of the Plan 7 offshore breakwater were adequate to prevent the movement of sediment into the entrances of the marina. Sediment will accumulate on the seaward sides of the north and south jetties. Along the shoreline, sediment will penetrate through voids in the rubble mound jetties and deposit along the channel sides of the structures.

f. Discharges from Wintersburg Channel should have minimal impacts in the interior channels and basins of the marina complex for Plan 7.

Navigable ocean entrance without connector channel

g. The originally proposed improvement plan (Plan 9) will not meet the established wave height criteria in the interior basins of the marina complex.

h. Wave conditions that will meet the established wave height criteria in the interior basins may be achieved by the installation of revetments along the interior channels and a 300-ft-long spur across the opening of the northwest basin (Plan 12).

i. Maximum wave heights that may be expected in the entrance and interior basins of the marina complex are 2.9 ft and 1.2 ft, respectively, for hindcast wave conditions with Plan 12 installed.

j. Removal of a 750-ft portion of the north wing of the offshore breakwater (Plan 13) will result in increased maximum wave heights (1.5 ft) in the interior basins for hindcast wave conditions.

k. Removal of a 750-ft portion of the north wing of the offshore breakwater in conjunction with raising the north jetty from +14 to +18 ft (Plan 15) will result in maximum wave heights of 1.4 ft in the interior basins for hindcast wave conditions. Wave heights will increase to 1.5 ft with the removal of a 250-ft portion of the south wing (Plan 16) and to 1.8 ft with the removal of an additional 250 ft of the north wing (Plan 17).

l. Removal of 750 ft or more of the north wing of the offshore breakwater (Plans 15 and 17) will result in sediment deposits in the entrance between the north jetty and the north breakwater for test waves from the west direction. Removal of 250 ft of the south wing (Plan 16) will result in sediment deposits in the entrance between the south jetty and the south breakwater for test waves approaching from the south. Penetration of sediment through voids in the rubble mound jetties will occur as was noted for previous test plans.

Non navigable ocean entrance

m. For the non-navigable ocean entrance plan (Plan 18) wave heights in the entrance during storm wave conditions will range from 5.0 to 6.8 ft (depending on the direction of wave approach) for the +7.0 ft swl. These waves, however, will be reduced in height to 2.2 ft or less at a point approximately 750 ft upstream and to 1.0 ft or less about 1,200 ft upstream. Also, wave heights in the entrance will be reduced significantly for lower swl's.

n. Sediment along the shoreline and in the breaker zone will move either north or south depending on the direction of wave approach. For waves from all directions some material will bypass the new entrance and some will penetrate into the entrance channel. Deepest penetration will occur for waves from west, west-southwest, and southwest.

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Table 1

Estimated Magnitude of Wave Conditions Approaching Bolsa Chica from the Directions Indicated

Wave Height (ft)	Occurrences* per Wave Period (sec)							
	1.5-6.0	6.1-8.0	8.1-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	TOTAL
West								
0.0 - 3.3	1069	5240	9794	1182	140	2	---	17427
3.4 - 4.9	79	3718	5205	5404	611	10	---	15027
5.0 - 6.6	5	270	2037	1836	607	3	---	4758
6.7 - 8.2	----	1	372	254	102	2	---	731
8.3 - 9.8	----	----	9	2	1	----	---	12
TOTAL	1153	9229	17417	8678	1461	17	---	37955
West-Southwest								
0.0 - 3.3	48	120	349	174	99	36	---	826
3.4 - 4.9	12	93	386	1309	1783	209	12	3804
5.0 - 6.6	2	15	466	1057	3516	756	43	5855
6.7 - 8.2	----	3	511	885	2625	1373	92	5489
8.3 - 9.8	----	----	187	605	1063	875	65	2795
9.9 - 11.5	----	----	18	182	410	317	32	959
11.6 - 13.1	----	----	-----	26	169	136	15	346
13.2 - 14.7	----	----	-----	----	47	39	6	92
14.8 - 16.4	----	----	-----	----	----	5	---	5
TOTAL	62	231	1917	4238	9712	3746	265	20171
Southwest								
0.0 - 3.3	8	10	39	6	----	2	---	65
3.4 - 4.9	----	----	7	11	2	8	1	29
5.0 - 6.6	1	2	25	8	11	----	---	47
6.7 - 8.2	----	3	11	3	11	----	---	28
8.3 - 9.8	----	----	4	3	7	2	---	16
TOTAL	9	15	86	31	31	12	1	185
South-Southwest								
0.0 - 3.3	----	11	40	----	----	----	---	51
3.4 - 4.9	5	1	2	----	----	----	---	8
5.0 - 6.6	----	7	14	1	----	----	---	22
6.7 - 8.2	----	8	10	2	1	----	---	21
TOTAL	5	27	66	3	1	----	---	102
South								
0.0 - 3.3	2	----	3	----	----	----	---	5
3.4 - 4.9	2	----	-----	----	----	----	---	2
5.0 - 6.6	2	----	2	----	----	----	---	4
6.7 - 8.2	----	2	7	----	----	----	---	9
TOTAL	6	2	12	----	----	----	---	20

*Occurrences compiled for period 1956-1975. Each occurrence represents a 3-hr duration.

Table 4

Wave Heights for Plan 7 for Test Waves from All Directions

swl = +7.0 ft

Test Wave		Wave Height, ft														
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 15	Gage 16
								West								
5	7	1.0	0.3	0.2	0.2	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	3.1
7	8	1.5	0.6	0.4	0.6	0.5	0.2	0.2	0.2	0.3	0.2	0.1	0.1	0.2	0.1	3.9
9	7	1.5	0.7	0.5	0.7	0.6	0.3	0.2	0.3	0.4	0.2	0.3	0.1	0.2	0.1	4.5
	9	1.6	0.9	0.7	0.9	0.9	0.4	0.3	0.3	0.5	0.3	0.4	0.2	0.3	0.1	5.2
11	7	1.7	0.9	0.9	0.9	0.9	0.4	0.3	0.3	0.6	0.3	0.3	0.2	0.3	0.1	5.3
	9	1.8	1.1	1.0	1.1	1.1	0.5	0.4	0.4	0.7	0.4	0.4	0.2	0.3	0.1	5.7
13	5	1.5	0.8	0.7	0.9	0.7	0.4	0.3	0.3	0.5	0 3	0.3	0.2	0.3	0.1	5.0
	9	2.2	1.3	1.0	1.3	1.0	0.7	0.4	0.5	0.9	0.5	0.6	0.3	0.5	0.2	6.4
15	8	2.2	1.3	1.0	1.3	1.0	0.7	0.4	0.5	0.9	0.5	0.6	0.3	0.5	0.2	6.5
								West-Southwest								
5	7	0.6	0.2	0.2	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	3.1
7	7	0.9	0.3	0.3	0.3	0.4	0.2	0.2	0.1	0.2	0.2	0.1	0.1	0.1	0.1	3.4
	9	1.0	0.5	0.4	0.5	0.5	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.2	0.1	3.9
9	9	1.4	1.0	0.6	0.8	0.8	0.4	0.3	0.3	0.5	0.3	0.3	0.2	0.2	0.1	3.8
	12	1.8	1.2	0.7	0.9	1.0	0.5	0.4	0.4	0.6	0.4	0.4	0.2	0.3	0.1	4.7
11	7	1.5	1.0	0.8	1.1	0.8	0.5	0.3	0.4	0.6	0.4	0.4	0.2	0.3	0.1	4.2
	10	1.7	1.2	0.9	1.0	0.9	0.5	0.4	0.5	0.7	0.4	0.5	0.2	0.5	0.2	4.6
	13	1.9	1.4	1.1	1.2	1.1	0.6	0.4	0.5	0.8	0.5	0.6	0.3	0.5	0.2	5.4
13	9	1.6	1.1	0.9	1.0	0.8	0.7	0.4	0.4	0.7	0.4	0.5	0.2	0.4	0.2	5.2
	12	2.2	1.5	1.0	1.2	1.0	0.8	0.5	0.6	1.0	0.6	0.7	0.4	0.7	0.2	5.9
	15	2.6	1.8	1.3	1.5	1.2	0.9	0.5	0.7	1.1	0.7	0.8	0.4	0.7	0.2	7.2

(Continued)

Table 4 (Continued)

Test Wave		Wave Height, ft														
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 15	Gage 16
							West-Southwest (Continued)									
15	7	1.9	1.1	0.8	1.1	0.6	0.6	0.4	0.6	0.8	0.5	0.6	0.3	0.5	0.2	4.5
	10	2.4	1.3	1.1	1.3	0.8	0.8	0.6	0.6	0.9	0.6	0.7	0.3	0.7	0.2	6.4
	12	2.4	1.6	1.1	1.4	0.9	0.8	0.6	0.6	1.0	0.6	0.8	0.4	0.7	0.3	6.0
	15	2.6	1.7	1.3	1.5	1.1	0.9	0.6	0.7	1.1	0.7	0.9	0.4	0.8	0.3	6.8
17	7	1.8	1.1	0.8	0.9	0.6	0.6	0.4	0.6	1.0	0.7	0.6	0.3	0.6	0.2	4.8
	9	2.0	1.2	0.9	1.1	0.7	0.8	0.5	0.6	1.1	0.7	0.7	0.3	0.7	0.3	5.7
	12	2.3	1.4	0.9	1.2	0.5	0.8	0.5	0.6	1.1	0.7	0.7	0.4	0.7	0.3	6.4
	15	2.5	1.7	1.2	1.5	1.1	0.7	0.5	0.7	1.1	0.7	0.7	0.4	0.7	0.3	7.7
								Southwest								
5	7	0.6	0.3	0.1	0.2	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	2.0
7	7	0.8	0.8	0.3	0.4	0.5	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.1	2.1
	9	0.9	0.9	0.3	0.6	0.5	0.3	0.2	0.2	0.3	0.2	0.2	0.1	0.2	0.1	3.0
9	7	1.0	0.9	0.5	0.5	0.8	0.4	0.3	0.3	0.4	0.2	0.3	0.1	0.2	0.1	2.9
	10	1.3	1.1	0.8	0.[illegible]	0.9	0.4	0.3	0.3	0.5	0.3	0.3	0.2	0.3	0.1	4.7
11	7	1.2	1.1	0.8	0.6	0.8	0.5	0.3	0.4	0.6	0.3	0.4	0.2	0.3	0.1	3.7
	10	1.5	1.3	1.1	0.9	1.0	0.7	0.4	0.5	0.7	0.4	0.6	0.3	0.5	0.2	4.9
13	7	1.4	1.3	1.1	0.9	0.9	0.8	0.4	0.6	0.8	0.4	0.5	0.2	0.4	0.2	4.0
	10	1.6	1.5	1.2	1.3	1.1	0.9	0.4	0.6	0.8	0.6	0.6	0.3	0.5	0.2	4.8
15	10	2.0	1.8	1.0	1.5	1.0	1.0	0.7	0.7	1.2	0.7	0.9	0.4	0.6	0.2	4.9
17	5	1.4	1.1	0.6	1.0	0.6	0.8	0.5	0.5	0.8	0.6	0.6	0.3	0.5	0.2	3.4

(Continued)

Table 4 (Concluded)

Test Wave		Wave Height, ft														
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 15	Gage 16
								South-Southwest								
7	8	1.2	0.7	0.3	0.6	0.6	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.2	0.1	0.9
9	8	1.8	1.0	0.8	0.9	0.8	0.5	0.3	0.3	0.4	0.3	0.3	0.2	0.3	0.1	1.2
11	8	2.2	1.6	1.1	1.5	1.1	0.6	0.4	0.5	0.8	0.5	0.6	0.2	0.5	0.2	1.5
13	8	2.1	1.3	1.3	1.2	1.1	0.7	0.3	0.6	0.8	0.5	0.6	0.3	0.5	0.2	1.6
								South								
5	7	1.0	0.4	0.2	0.3	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.8
7	8	1.5	0.8	0.4	0.7	0.5	0.2	0.2	0.2	0.3	0.2	0.2	0.1	0.2	0.1	1.0
9	8	1.8	1.2	0.8	1.0	0.9	0.5	0.5	0.5	0.5	0.4	0.3	0.2	0.3	0.1	1.2

(Sheet 3 of 3)

Table 5

Wave Heights for Plan 7 for Test Waves from All Directions

with Maximum Flood Tidal Conditions, swl = +2.8 ft

Test Wave		Wave Height, ft														
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 15	Gage 16
								West								
5	7	0.4	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	2.0
7	8	0.6	0.3	0.2	0.3	0.3	0.1	0.1	0.1	0.2	0.1	0.1	0.1	0.1	0.1	2.9
9	9	1.0	0.4	0.3	0.5	0.4	0.2	0.1	0.1	0.4	0.2	0.2	0.1	0.1	0.1	4.6
11	9	1.0	0.5	0.4	0.5	0.5	0.2	0.2	0.2	0.3	0.2	0.3	0.1	0.2	0.1	4.4
13	9	1.1	0.6	0.4	0.6	0.5	0.2	0.2	0.2	0.4	0.3	0.2	0.1	0.2	0.1	6.1
15	8	1.0	0.5	0.4	0.5	0.4	0.2	0.2	0.2	0.3	0.2	0.3	0.1	0.2	0.1	5.5
								West-Southwest								
5	7	0.4	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	1.6
7	9	0.7	0.3	0.2	0.3	0.2	0.1	0.1	0.1	0.2	0.1	0.1	0.1	0.1	0.1	2.1
9	12	1.0	0.5	0.3	0.5	0.3	0.2	0.2	0.2	0.4	0.2	0.2	0.1	0.2	0.1	4.4
11	13	1.1	0.5	0.4	0.5	0.4	0.2	0.2	0.2	0.4	0.3	0.2	0.1	0.2	0.1	4.9
13	15	1.6	0.7	0.5	0.7	0.5	0.4	0.3	0.3	0.5	0.3	0.3	0.2	0.2	0.1	6.3
15	15	1.6	0.7	0.5	0.6	0.5	0.3	0.2	0.3	0.6	0.3	0.3	0.1	0.2	0.1	6.3
17	15	1.5	0.8	0.6	0.7	0.5	0.2	0.2	0.3	0.4	0.2	0.3	0.2	0.2	0.1	6.6

(Continued)

Table 5 (Concluded)

Test Wave		Wave Height, ft														
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 15	Gage 16
								Southwest								
5	7	0.4	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	1.0
7	9	0.7	0.4	0.2	0.3	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	1.4
9	10	0.9	0.6	0.4	0.5	0.4	0.4	0.2	0.2	0.3	0.2	0.2	0.1	0.2	0.1	2.2
11	10	1.0	0.7	0.5	0.5	0.4	0.3	0.2	0.3	0.4	0.2	0.3	0.2	0.2	0.1	2.4
13	10	1.1	0.8	0.6	0.6	0.4	0.4	0.2	0.3	0.5	0.3	0.4	0.2	0.2	0.1	3.0
15	10	1.3	0.9	0.5	0.7	0.6	0.5	0.3	0.4	0.6	0.4	0.6	0.2	0.4	0.1	3.3
17	5	0.7	0.6	0.3	0.4	0.3	0.3	0.2	0.3	0.4	0.2	0.3	0.1	0.2	0.1	1.9
								South-Southwest								
7	8	0.9	0.4	0.2	0.4	0.2	0.1	0.1	0.1	0.2	0.1	0.1	0.1	0.1	0.1	0.9
9	8	1.2	0.6	0.3	0.6	0.4	0.2	0.2	0.2	0.3	0.2	0.2	0.1	0.2	0.1	1.3
11	8	1.6	1.0	0.5	0.9	0.7	0.3	0.3	0.3	0.5	0.3	0.3	0.2	0.2	0.1	1.4
13	8	1.4	0.8	0.5	0.8	0.6	0.3	0.2	0.3	0.5	0.3	0.3	0.2	0.3	0.1	1.4
								South								
5	7	0.7	0.3	0.2	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.5
7	8	1.3	0.5	0.2	0.5	0.3	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.6
9	8	1.5	0.8	0.4	0.7	0.5	0.2	0.2	0.2	0.3	0.2	0.2	0.1	0.1	0.1	0.8

(Sheet 2 of 2)

Table 6

Wave Heights for Plan 7 for Test Waves from All Directions

with Maximum Ebb Tidal Conditions, swl = +2.8 ft

Test Wave		Wave Height, ft														
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 15	Gage 16
								West								
5	7	0.5	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	2.3
7	8	0.6	0.2	0.2	0.3	0.2	0.1	0.1	0.1	0.2	0.1	0.1	0.1	0.1	0.1	3.2
9	9	1.0	0.4	0.3	0.5	0.3	0.2	0.1	0.1	0.3	0.2	0.2	0.1	0.2	0.1	4.9
11	9	1.0	0.5	0.4	0.6	0.4	0.2	0.1	0.2	0.3	0.2	0.2	0.1	0.2	0.1	4.9
13	9	1.1	0.5	0.4	0.7	0.4	0.2	0.1	0.2	0.3	0.2	0.2	0.1	0.2	0.1	5.3
15	8	1.0	0.5	0.4	0.6	0.4	0.2	0.2	0.2	0.3	0.2	0.2	0.1	0.2	0.1	5.4
								West-Southwest								
5	7	0.4	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	1.4
7	9	0.6	0.2	0.2	0.2	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	1.7
9	12	1.0	0.5	0.3	0.5	0.3	0.2	0.2	0.2	0.3	0.2	0.2	0.1	0.2	0.1	3.4
11	13	1.1	0.5	0.4	0.5	0.4	0.2	0.2	0.2	0.3	0.2	0.2	0.1	0.1	0.1	3.7
13	15	1.5	0.7	0.5	0.8	0.6	0.3	0.2	0.3	0.4	0.3	0.3	0.1	0.2	0.1	5.8
15	15	1.6	0.7	0.6	0.9	0.6	0.3	0.2	0.3	0.5	0.3	0.3	0.1	0.2	0.1	5.1
17	15	1.5	0.8	0.6	0.8	0.6	0.2	0.2	0.2	0.4	0.2	0.3	0.1	0.2	0.1	5.2

(Continued)

(Sheet 1 of 2)

Table 6 (Concluded)

Test Wave		Wave Height, ft														
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 15	Gage 16
								Southwest								
5	7	0.3	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.8
7	9	0.5	0.3	0.2	0.2	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	1.1
9	10	0.9	0.5	0.3	0.4	0.4	0.3	0.1	0.2	0.3	0.2	0.2	0.1	0.1	0.1	1.9
11	10	0.9	0.6	0.4	0.5	0.5	0.3	0.2	0.2	0.3	0.2	0.3	0.1	0.2	0.1	1.9
13	10	1.2	0.8	0.5	0.5	0.5	0.3	0.2	0.3	0.4	0.2	0.3	0.1	0.2	0.1	2.4
15	10	1.3	0.9	0.5	0.7	0.5	0.4	0.2	0.3	0.4	0.3	0.4	0.2	0.3	0.1	2.7
17	5	0.6	0.5	0.3	0.5	0.3	0.2	0.1	0.2	0.3	0.2	0.2	0.1	0.2	0.1	1.5
								South-Southwest								
7	8	0.9	0.4	0.2	0.3	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.6
9	8	1.3	0.6	0.4	0.5	0.3	0.2	0.1	0.1	0.3	0.1	0.2	0.1	0.1	0.1	0.9
11	8	1.8	0.9	0.6	0.8	0.6	0.3	0.2	0.2	0.5	0.3	0.3	0.1	0.2	0.1	1.1
13	8	1.6	0.8	0.6	0.7	0.6	0.3	0.1	0.2	0.5	0.2	0.3	0.1	0.2	0.1	1.0
								South								
5	7	0.9	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.5
7	8	1.4	0.5	0.2	0.5	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.8
9	8	1.6	0.9	0.4	0.7	0.4	0.2	0.2	0.2	0.3	0.2	0.2	0.1	0.1	0.1	1.2

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Table 7

Wave Heights for Plan 7 for Test Waves from All Directions

swl = 0.0 ft

Test Wave Period sec	Test Wave Height ft	Wave Height, ft Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 15	Gage 16
								West								
5	7	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	1.5
7	8	0.3	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	2.5
9	9	0.5	0.2	0.2	0.3	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	3.9
11	9	0.6	0.2	0.2	0.3	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	4.1
13	9	0.6	0.2	0.3	0.4	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	5.6
15	8	0.6	0.2	0.2	0.3	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	5.0
								West-Southwest								
5	7	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	1.0
7	9	0.5	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	1.7
9	12	0.8	0.3	0.2	0.3	0.2	0.1	0.1	0.1	0.2	0.1	0.2	0.1	0.1	0.1	3.6
11	13	0.9	0.3	0.3	0.3	0.3	0.2	0.1	0.1	0.2	0.1	0.2	0.1	0.1	0.1	4.6
13	15	1.1	0.4	0.4	0.5	0.4	0.2	0.1	0.2	0.3	0.1	0.2	0.1	0.1	0.1	5.8
15	15	1.1	0.4	0.4	0.5	0.4	0.2	0.1	0.2	0.3	0.1	0.2	0.1	0.1	0.1	5.6
17	15	1.2	0.5	0.4	0.5	0.4	0.2	0.2	0.2	0.3	0.1	0.2	0.1	0.2	0.1	4.6

(Continued)

Table 7 (Concluded)

Test Wave		Wave Height, ft														
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 15	Gage 16
								Southwest								
5	7	0.4	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.5
7	9	0.7	0.2	0.1	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	1.0
9	10	0.8	0.3	0.2	0.3	0.2	0.1	0.1	0.2	0.2	0.1	0.2	0.1	0.1	0.1	1.9
11	10	0.8	0.4	0.2	0.3	0.2	0.2	0.1	0.2	0.2	0.1	0.2	0.1	0.1	0.1	2.1
13	10	1.0	0.5	0.3	0.3	0.2	0.2	0.1	0.2	0.2	0.2	0.2	0.1	0.1	0.1	2.9
15	10	1.0	0.5	0.3	0.3	0.2	0.2	0.1	0.2	0.3	0.2	0.2	0.1	0.1	0.1	2.7
17	5	0.6	0.4	0.2	0.3	0.2	0.1	0.1	0.1	0.2	0.1	0.2	0.1	0.1	0.1	1.4
								South-Southwest								
7	8	0.8	0.2	0.1	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.4
9	8	1.2	0.4	0.2	0.4	0.3	0.1	0.1	0.1	0.2	0.1	0.1	0.1	0.1	0.1	0.6
11	8	1.7	0.7	0.4	0.6	0.4	0.2	0.1	0.2	0.3	0.2	0.2	0.1	0.1	0.1	0.9
13	8	1.3	0.5	0.3	0.5	0.4	0.1	0.1	0.2	0.2	0.1	0.1	0.1	0.1	0.1	0.7
								South								
5	7	0.7	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.4
7	8	1.1	0.3	0.2	0.3	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.6
9	8	1.4	0.6	0.3	0.5	0.3	0.1	0.1	0.1	0.2	0.1	0.1	0.1	0.1	0.1	0.7

(Sheet 2 of 2)

Table 8

Wave Heights for Plans 9-12 for Test Waves from West-Southwest

swl = +7.0 ft

Test Wave		Wave Height, ft														
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 15	Gage 16
								Plan 9								
14.3	11.7	2.1	1.2	1.0	1.2	1.0	1.5	0.9	0.9	1.3	0.7	0.9	0.5	0.8	0.3	6.2
	14.6	2.3	1.4	1.2	1.4	1.2	1.7	1.1	1.0	1.4	0.7	1.0	0.5	0.9	0.3	6.5
								Plan 10								
14.3	11.7	2.1	1.2	1.0	1.2	0.9	1.2	0.8	0.6	1.0	0.5	0.8	0.3	0.6	0.2	6.5
	14.6	2.3	1.3	1.1	1.3	1.0	1.2	0.7	0.6	1.1	0.5	0.8	0.3	0.6	0.2	7.1
								Plan 11								
14.3	11.7	2.1	1.2	0.9	1.2	0.8	0.9	0.6	0.5	0.9	0.5	0.7	0.3	0.6	0.2	6.5
	14.6	2.4	1.4	1.1	1.5	1.0	1.0	0.7	0.6	1.0	0.5	0.7	0.3	0.6	0.2	6.8
								Plan 12								
14.3	11.7	2.3	1.3	0.9	1.3	0.9	0.6	0.4	0.5	0.9	0.5	0.6	0.3	0.6	0.2	5.8
	14.6	2.6	1.5	1.1	1.6	1.1	0.7	0.5	0.6	1.0	0.5	0.7	0.3	0.7	0.2	6.8

Table 9

Wave Heights for Plans 11 and 12 for Test Waves from West-Southwest

swl = +7.0 ft

Test Wave		Wave Height, ft														
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 15	Gage 16
								Plan 11								
5	7	1.0	0.3	0.1	0.3	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	3.6
7	7	1.2	0.4	0.4	0.4	0.3	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.2	0.1	4.0
	9	1.4	0.5	0.5	0.5	0.4	0.3	0.2	0.2	0.3	0.2	0.2	0.1	0.2	0.1	4.6
9	9	1.6	0.8	0.6	0.8	0.6	0.5	0.4	0.3	0.5	0.3	0.4	0.2	0.3	0.1	4.5
	12	1.8	1.0	0.7	1.0	0.7	0.7	0.8	0.4	0.6	0.4	0.4	0.2	0.4	0.1	5.4
11	7	1.7	0.8	0.8	0.9	0.5	0.5	0.3	0.3	0.5	0.3	0.3	0.2	0.3	0.1	4.7
	10	1.9	1.1	1.0	1.1	0.7	0.7	0.5	0.4	0.7	0.4	0.5	0.2	0.4	0.1	5.0
	13	2.1	1.2	1.0	1.3	0.8	0.8	0.5	0.5	0.8	0.5	0.6	0.3	0.5	0.2	5.9
13	9	2.1	1.1	0.9	1.0	0.7	0.7	0.5	0.4	0.8	0.4	0.5	0.3	0.4	0.2	5.3
	12	2.5	1.3	1.1	1.3	0.8	1.0	0.6	0.6	0.9	0.5	0.6	0.3	0.5	0.2	5.9
	15	2.9	1.5	1.2	1.7	1.1	1.2	0.7	0.7	1.1	0.6	0.8	0.4	0.6	0.2	7.9
15	7	1.9	0.9	1.0	1.1	0.7	0.7	0.5	0.4	0.7	0.4	0.5	0.2	0.4	0.2	4.6
	10	2.4	1.2	1.1	1.4	0.9	0.9	0.6	0.6	1.0	0.5	0.7	0.3	0.5	0.2	6.2
	12	2.6	1.4	1.1	1.7	1.1	1.0	0.7	0.6	1.0	0.6	0.8	0.4	0.5	0.3	6.2
	15	2.9	1.6	1.2	2.0	1.2	1.2	0.9	0.7	1.2	0.6	0.9	0.5	0.6	0.3	6.9
17	7	1.9	0.9	0.9	1.0	0.8	0.7	0.5	0.4	0.6	0.3	0.5	0.2	0.4	0.1	5.3
	9	2.0	1.1	1.1	1.4	0.9	1.0	0.6	0.6	0.9	0.5	0.7	0.3	0.5	0.2	6.0
	12	2.3	1.4	0.8	1.5	1.1	1.2	0.6	0.7	1.0	0.6	0.8	0.4	0.7	0.3	6.6
	15	2.9	1.5	1.2	1.9	1.3	1.3	0.9	0.8	1.4	0.6	0.9	0.5	0.8	0.3	8.4

(Continued)

Table 9 (Concluded)

Test Wave		Wave Height, ft														
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 15	Gage 16
								Plan 12								
13	15	2.9	1.6	1.0	1.5	1.1	1.0	0.6	0.6	1.1	0.6	0.8	0.4	0.7	0.2	8.0
15	15	2.9	1.7	1.1	1.8	1.2	1.0	0.6	0.6	1.1	0.6	0.8	0.5	0.7	0.3	7.5
17	12	2.4	1.4	0.9	1.5	1.0	1.0	0.6	0.7	1.0	0.5	0.8	0.4	0.7	0.3	7.0
17	15	2.9	1.6	1.1	1.7	1.3	1.1	0.7	0.8	1.2	0.6	0.9	0.5	0.8	0.3	8.1

Table 10

Wave Heights for Plan 12 for Test Waves from Southwest and West

swl = +7.0 ft

Test Wave Period sec	Test Wave Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 15	Gage 16
								Southwest								
5	7	0.7	0.3	0.1	0.4	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	2.4
7	7	1.0	0.5	0.2	0.5	0.4	0.1	0.1	0.1	0.2	0.1	0.1	0.1	0.1	0.1	2.4
	9	1.2	0.6	0.3	0.6	0.5	0.2	0.2	0.1	0.2	0.2	0.2	0.1	0.1	0.1	3.1
9	7	1.4	0.7	0.4	0.6	0.6	0.2	0.3	0.2	0.3	0.2	0.2	0.1	0.2	0.1	3.2
	10	1.5	0.9	0.5	0.8	0.7	0.3	0.3	0.3	0.4	0.3	0.3	0.2	0.2	0.1	4.6
11	7	1.5	0.8	0.6	0.8	0.5	0.3	0.3	0.3	0.4	0.2	0.2	0.2	0.2	0.1	3.8
	10	1.8	1.0	0.7	1.1	0.7	0.5	0.4	0.4	0.5	0.3	0.4	0.2	0.4	0.1	5.3
13	7	1.5	1.0	0.8	1.1	0.6	0.4	0.3	0.3	0.5	0.3	0.3	0.2	0.3	0.1	4.5
	10	1.7	1.1	0.8	1.3	0.7	0.4	0.3	0.4	0.6	0.3	0.4	0.2	0.4	0.2	5.2
15	10	1.8	1.2	0.8	1.9	0.9	0.6	0.5	0.4	0.8	0.5	0.6	0.3	0.5	0.2	5.2
17	5	1.3	0.8	0.5	1.0	0.7	0.4	0.4	0.4	0.6	0.4	0.4	0.2	0.3	0.1	3.0
								West								
5	7	1.1	0.3	0.1	0.2	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	3.3
7	8	1.4	0.4	0.3	0.4	0.3	0.1	0.2	0.1	0.2	0.2	0.1	0.1	0.1	0.1	4.2
9	7	1.6	0.6	0.5	0.5	0.3	0.2	0.2	0.2	0.3	0.2	0.2	0.1	0.2	0.1	4.7
	9	1.9	0.8	0.6	0.8	0.5	0.4	0.3	0.3	0.5	0.3	0.4	0.2	0.3	0.1	5.4
11	7	2.0	0.8	0.7	0.8	0.4	0.3	0.3	0.3	0.4	0.3	0.3	0.1	0.3	0.1	5.6
	9	2.1	1.0	0.7	1.0	0.6	0.4	0.3	0.3	0.6	0.3	0.3	0.2	0.3	0.1	5.8
13	5	1.6	0.7	0.6	0.7	0.4	0.2	0.2	0.2	0.3	0.2	0.2	0.1	0.2	0.1	5.3
	9	2.4	1.1	0.8	1.1	0.7	0.5	0.4	0.4	0.8	0.4	0.4	0.2	0.5	0.1	6.6
15	8	2.1	0.9	0.6	1.1	0.6	0.6	0.4	0.4	0.7	0.4	0.5	0.2	0.4	0.1	6.6

Table 11

Wave Heights for Plans 13 and 14 for Test Waves from West

swl = +7.0 ft

Test Wave		Wave Height, ft														
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 15	Gage 16
								Plan 13								
9	9	2.5	1.1	0.9	0.9	1.0	0.5	0.5	0.4	0.7	0.4	0.4	0.2	0.4	0.2	12.0
11	9	2.6	1.3	1.5	1.3	1.1	0.7	0.5	0.5	0.9	0.5	0.6	0.3	0.5	0.2	12.7
13	9	2.9	1.7	1.8	1.7	1.4	0.9	0.6	0.7	1.1	0.7	0.8	0.3	0.7	0.3	12.9
15	8	2.3	1.7	1.8	1.4	1.4	1.0	0.6	0.7	1.1	0.7	0.7	0.4	0.7	0.2	11.6
								Plan 14								
11	9	2.7	0.8	0.9	0.9	0.7	0.4	0.3	0.3	0.6	0.3	0.4	0.2	0.3	0.1	14.1
13	9	2.7	1.0	1.0	1.1	0.8	0.5	0.4	0.4	0.7	0.4	0.5	0.2	0.5	0.1	14.1
15	8	2.3	0.9	1.0	1.0	0.7	0.6	0.3	0.4	0.6	0.4	0.5	0.2	0.4	0.1	14.6

Table 12

Wave Heights for Plans 13 and 14 for Test Waves from West-Southwest

swl = +7.0 ft

Test Wave		Wave Height, ft														
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 15	Gage 16
								Plan 13								
13	15	3.3	2.0	1.7	2.0	1.5	1.0	0.7	0.7	1.2	0.7	1.0	0.5	1.0	0.3	12.6
15	15	3.4	2.1	1.8	1.9	1.5	1.2	0.7	1.0	1.4	0.9	0.8	0.4	0.7	0.3	12.3
17	12	2.8	1.9	1.4	1.8	1.4	1.1	0.6	0.7	1.2	0.7	0.8	0.4	0.7	0.3	12.9
17	15	3.6	2.1	1.6	2.2	1.7	1.2	0.7	0.9	1.5	0.8	0.9	0.5	0.9	0.3	15.3
14.3	11.7	2.5	1.7	1.4	1.5	1.2	1.0	0.5	0.7	1.1	0.5	0.8	0.4	0.6	0.2	12.5
	14.6	2.9	2.0	1.7	1.8	1.4	1.1	0.5	0.8	1.3	0.7	0.9	0.4	0.6	0.3	13.8
								Plan 14								
13	15	3.2	1.5	1.0	1.5	0.9	0.6	0.4	0.4	0.8	0.4	0.6	0.3	0.6	0.2	11.2
15	15	3.3	1.5	1.1	1.4	0.9	0.7	0.5	0.6	0.9	0.5	0.6	0.3	0.5	0.2	11.2
17	12	2.7	1.2	0.9	1.2	0.8	0.8	0.5	0.5	0.8	0.5	0.5	0.3	0.4	0.2	12.4
17	15	3.1	1.4	0.9	1.4	1.0	0.9	0.5	0.6	0.9	0.5	0.5	0.3	0.5	0.2	15.0

Table 13

Wave Heights for Plans 15 - 17 for Test Waves from West-Southwest

swl = +7.0 ft

Test Wave		Wave Height, ft														
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 15	Gage 16
								Plan 15								
13	15	3.1	1.6	1.3	1.7	1.2	0.8	0.6	0.7	1.1	0.6	0.8	0.4	0.8	0.3	12.1
15	15	3.7	2.0	1.5	1.7	1.3	1.0	0.6	0.8	1.4	0.8	0.9	0.4	0.7	0.3	13.0
17	12	2.9	1.5	1.1	1.6	1.2	0.9	0.6	0.6	1.1	0.6	0.7	0.4	0.6	0.3	12.8
	15	3.4	1.8	1.3	1.9	1.4	1.0	0.6	0.7	1.4	0.7	0.9	0.4	0.8	0.3	14.0
14.3	11.7	2.6	1.5	1.2	1.4	1.0	0.9	0.5	0.6	1.0	0.6	0.7	0.3	0.6	0.2	12.9
	14.6	3.1	1.7	1.4	1.6	1.2	1.0	0.5	0.7	1.2	0.7	0.8	0.4	0.7	0.3	13.1
								Plan 16								
13	15	3.5	1.7	1.3	1.7	1.1	0.8	0.6	0.6	1.3	0.8	0.9	0.5	0.9	0.3	13.7
15	15	3.6	1.9	1.5	1.7	1.2	1.1	0.6	0.8	1.5	0.9	0.8	0.4	0.7	0.3	13.3
17	12	3.3	1.6	1.0	1.5	1.0	0.9	0.6	0.6	1.1	0.7	0.8	0.4	0.8	0.2	13.5
	15	3.9	1.7	1.1	1.6	1.2	0.9	0.7	0.7	1.2	0.7	0.8	0.4	0.8	0.3	14.6
14.3	11.7	2.7	1.6	1.1	1.3	1.0	0.9	0.6	0.6	1.2	0.6	0.7	0.4	0.6	0.2	13.6
	14.6	3.1	1.8	1.1	1.6	1.1	0.9	0.6	0.7	1.4	0.7	0.8	0.4	0.6	0.2	13.9
								Plan 17								
13	15	3.6	1.6	1.1	1.8	1.2	1.0	0.7	0.8	1.4	0.9	1.1	0.6	1.1	0.3	12.6
15	15	3.6	1.7	1.1	1.9	1.3	1.2	0.7	0.9	1.8	1.0	1.0	0.5	0.8	0.3	12.8
17	12	2.8	1.5	0.9	1.9	1.2	1.1	0.6	0.7	1.4	0.7	1.0	0.4	0.9	0.3	12.7
	15	3.5	1.6	1.0	2.0	1.3	1.0	0.7	0.8	1.3	0.8	1.1	0.5	1.0	0.4	13.7
14.3	11.7	2.5	1.4	0.9	1.5	1.0	0.9	0.5	0.6	1.2	0.6	0.7	0.4	0.6	0.2	13.3
	14.6	2.9	1.5	1.1	1.8	1.1	1.0	0.6	0.7	1.3	0.7	0.9	0.5	0.7	0.3	13.1

Table 14

Wave Heights for Plans 16 and 17 for Test Waves from West and/or South

swl = +7.0 ft

	Test Wave		Wave Height, ft														
Direction	Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 15	Gage 16
							Plan 16										
West	11	9	2.7	1.1	1.2	1.1	0.8	0.7	0.5	0.5	0.7	0.5	0.5	0.3	0.5	0.2	12.1
	13	9	2.9	1.4	1.4	1.6	1.1	0.9	0.7	0.7	1.1	0.6	0.9	0.3	0.7	0.3	12.9
	15	8	2.6	1.5	1.5	1.2	1.0	0.9	0.6	0.6	1.1	0.6	0.7	0.3	0.6	0.2	12.7
							Plan 17										
	11	9	2.6	1.1	1.0	1.1	0.9	0.7	0.5	0.5	0.8	0.5	0.6	0.3	0.6	0.2	13.4
	13	9	3.1	1.5	1.2	1.8	1.2	1.0	0.7	0.8	1.4	0.7	0.9	0.4	0.8	0.3	13.4
	15	8	2.5	1.3	1.3	1.5	1.1	1.1	0.8	0.7	1.6	0.7	1.0	0.4	0.8	0.3	14.3
South	5	7	2.4	0.5	0.2	0.4	0.4	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.2	0.1	1.6
	7	8	2.6	0.9	0.8	0.6	0.8	0.2	0.3	0.2	0.3	0.3	0.2	0.2	0.2	0.1	2.8
	9	8	3.6	1.5	1.1	1.1	1.2	0.5	0.6	0.5	0.6	0.4	0.4	0.3	0.3	0.1	4.3

Table 15

Wave Heights for Plan 18 for Representative Test Waves from All Directions

Direction	Test Wave Period sec	Test Wave Height ft	Wave Height, ft Gage 1	Gage 2	Gage 3	Gage 1	Gage 2	Gage 3	Gage 1	Gage 2	Gage 3	Gage 1	Gage 2	Gage 3
			swl = 0.0 ft			maximum ebb tidal conditions swl = +0.9 ft			maximum flood tidal conditions swl = +3.0 ft			swl = +7.0 ft		
West	9	7	1.3	0.5	0.3	1.8	0.6	0.3	2.7	0.6	0.4	5.0	1.1	0.5
	13	9	1.6	0.6	0.3	1.9	0.6	0.3	2.9	0.8	0.5	4.7	1.5	0.6
	15	8	1.5	0.5	0.3	1.7	0.7	0.4	2.4	0.9	0.4	4.1	1.4	0.6
West-Southwest	9	9	2.1	0.7	0.5	2.7	0.8	0.3	3.5	1.2	0.8	5.6	1.4	0.6
	13	12	2.2	0.8	0.5	2.6	0.8	0.3	3.3	1.2	0.7	5.0	1.8	0.8
	17	15	2.0	0.3	0.5	2.3	0.8	0.4	3.0	1.0	0.6	4.6	1.5	0.7
Southwest	7	9	1.6	0.5	0.4	2.5	0.6	0.3	3.4	0.6	0.4	6.8	1.5	0.8
	11	10	2.3	0.7	0.5	2.8	0.9	0.5	3.3	1.2	0.7	6.0	2.2	1.0
	15	10	2.2	0.8	0.5	2.6	1.0	0.5	3.1	1.2	0.7	5.5	2.0	0.8
South-Southwest	9	8	2.0	0.6	0.4	2.3	0.7	0.5	3.2	0.9	0.7	6.2	1.4	0.7
	13	8	1.9	0.6	0.4	2.2	0.7	0.4	2.8	0.8	0.5	5.3	1.5	0.6
South	5	7	1.1	0.1	0.1	1.7	0.2	0.1	3.3	1.2	0.7	5.1	0.7	0.3
	9	8	1.8	0.7	0.4	2.1	0.8	0.5	2.9	1.0	0.7	6.1	1.4	0.6

Table 16

Wave Heights for Plans 7, 12 and 18 for Test Waves from West-Southwest

swl = +8.0 ft

	Test Wave		Wave Height, ft														
Plan	Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 15	Gage 16
							Plan 16										
7	22	17	3.7	2.2	1.4	1.8	1.9	1.2	0.9	0.7	1.4	0.8	0.7	0.5	0.6	0.3	6.8
12	22	17	3.4	2.3	1.0	2.2	1.4	1.6	1.1	1.0	2.1	0.8	1.0	1.0	0.9	0.5	7.7
18	22	17	5.6	1.4	0.5	---	---	---	---	---	---	---	---	---	---	---	---

Photo 1. View of the offshore breakwater and jettied entrance of Plan 1

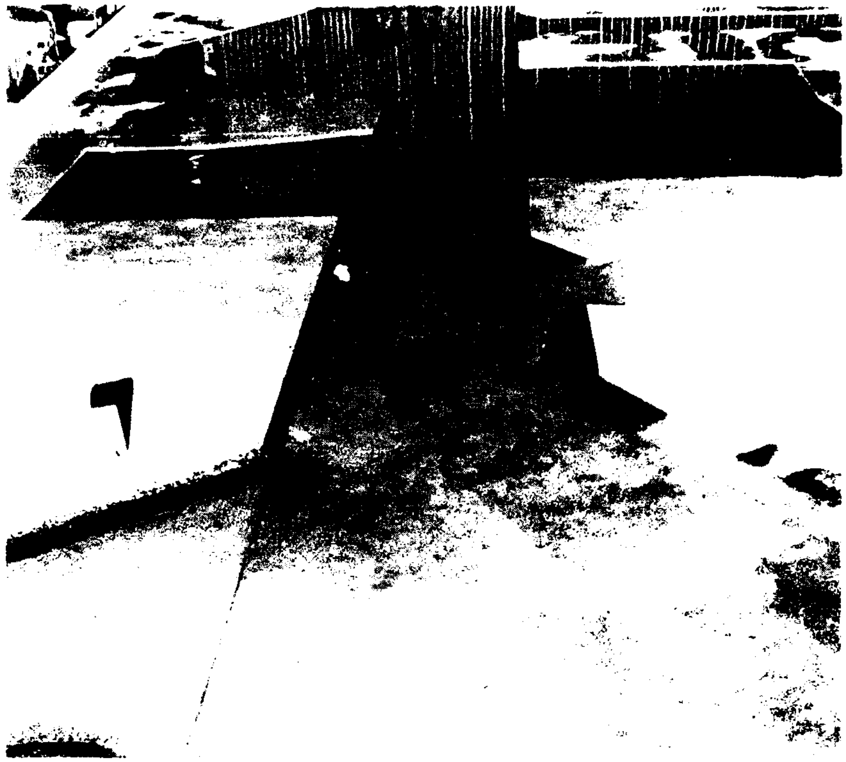


Photo 2. View of the interior channel and some of the basins of Plan 1 (Note the stone adjacent to the interior channel is submerged and serves as toe protection as opposed to an absorber). The Huntington Harbour connector channel extends to the left in the photograph

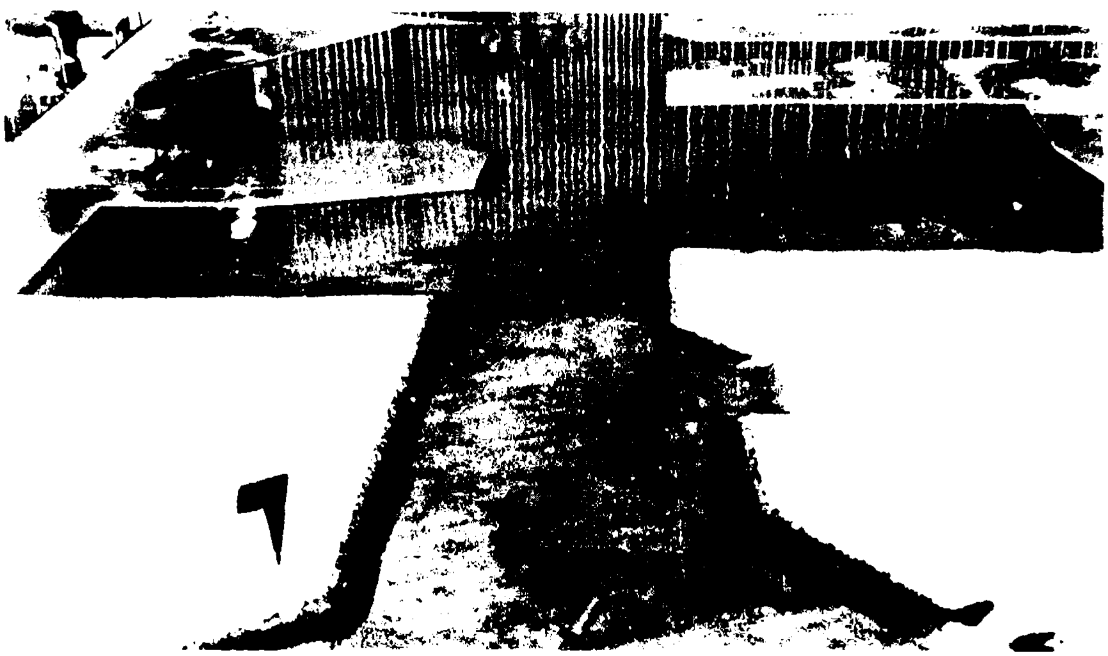


Photo 3. View of absorber placed adjacent the interior channel. Absorber was used for Plans 2-8

Photo 4. View of the 900 ft raised and sealed portion of the offshore breakwater used for Plans 3-5. Metal was placed along this section of breakwater to prevent overtopping and transmission through the structure. Wave absorbor was then placed over the metal to minimize reflections back toward the incoming wave train

Photo 5. View of rubble baffle across a portion of the opening of the northwest basin which was used in Plans 5-7

Photo 6. Typical wave patterns for Plan 7; 13-sec, 9-ft waves from west; +7.0 ft swl

Photo 7. Typical wave patterns for Plan 7; 15-sec, 12-ft waves from west-southwest; +7.0 ft. swl

Photo 8. Typical wave patterns for Plan 7; 15-sec, 10-ft waves from southwest; +7.0 ft swl

Photo 9. Typical wave patterns for Plan 7; 13-sec, 8-ft waves from south-southwest; +7.0 ft swl

Photo 10. Typical wave patterns for Plan 7; 9-sec, 8-ft, waves from south; +7.0 ft swl

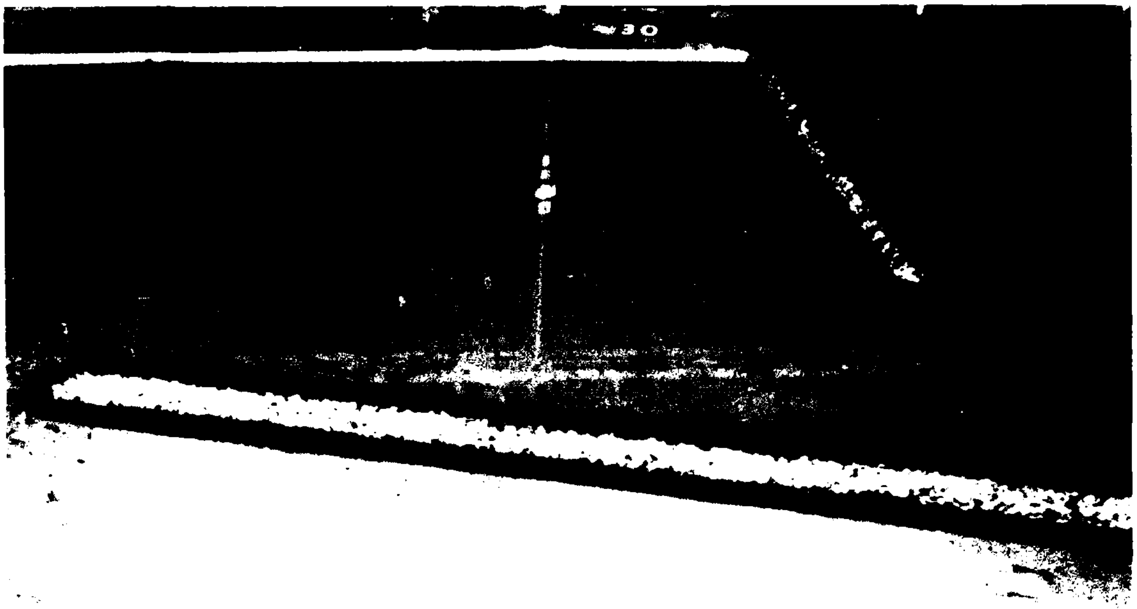


Photo 11. Placement of tracer material north of the entrance prior to testing

Photo 12. Placement of tracer material south of the entrance prior to testing

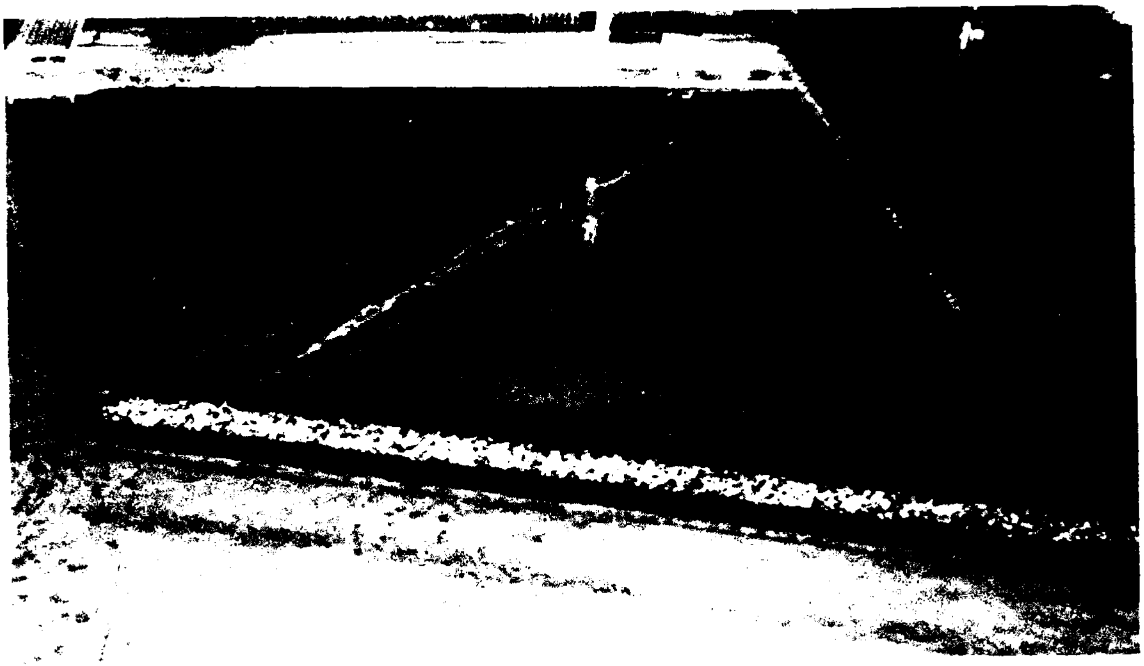


Photo 13. General movement of tracer material and subsequent deposits for Plan 7 for test waves from west after testing of the 0.0 ft swl

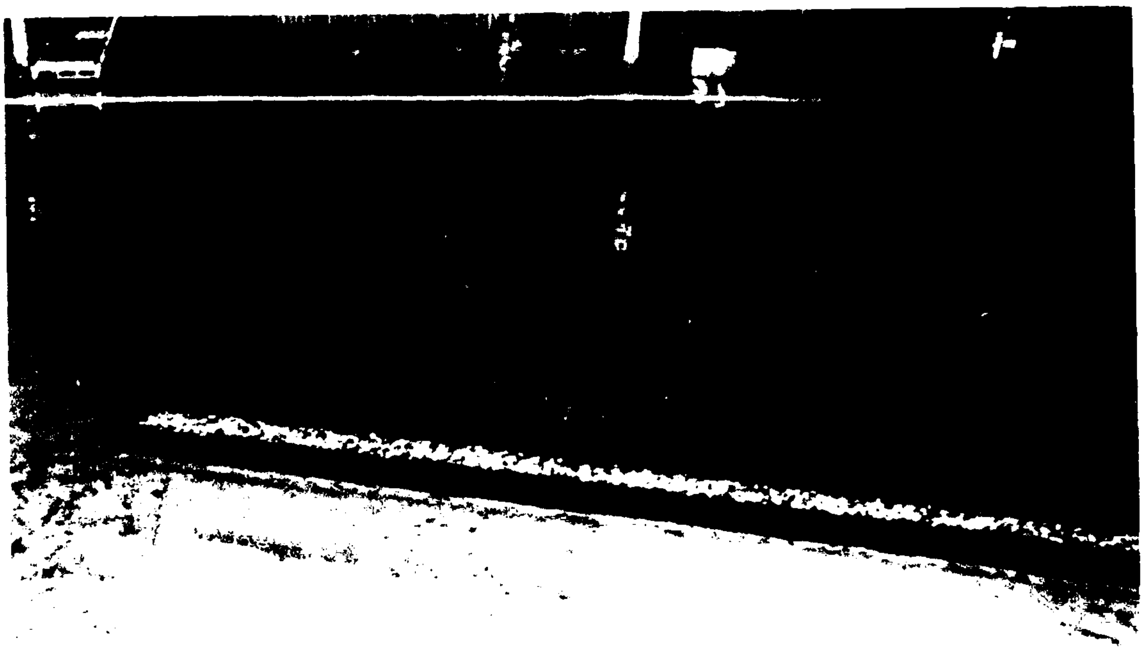


Photo 14. General movement of tracer material and subsequent deposits for Plan 7 for test waves from west after testing of the 0.0 ft and +7.0 ft swl's

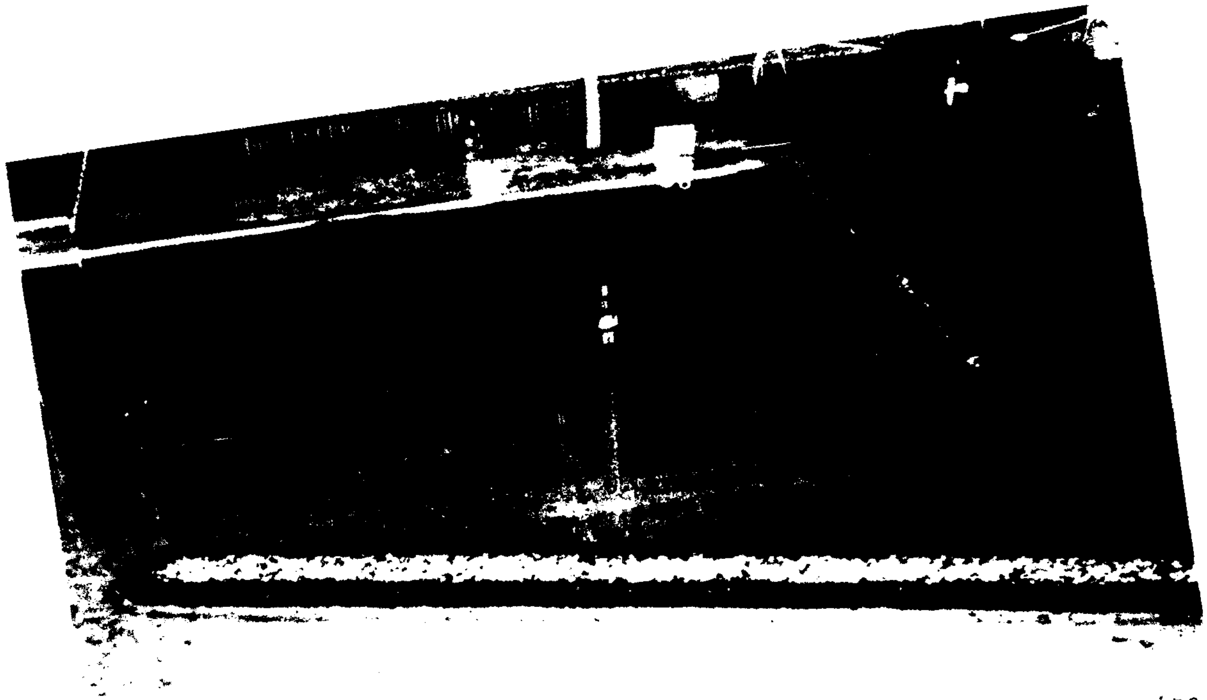


Photo 15. General movement of tracer material and subsequent deposits for Plan 7 for test waves from west after testing of the 0.0, +7.0 and +2.8 ft swl's (maximum flood flow conditions)

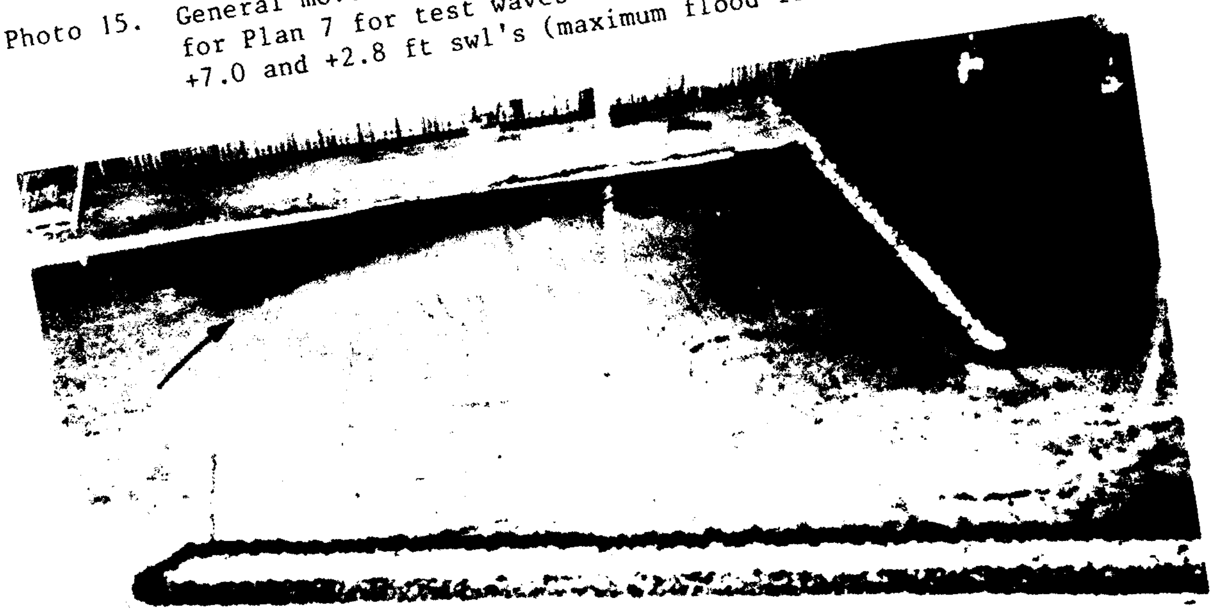


Photo 16. General movement of tracer material and subsequent deposits for Plan 7 for test waves from west after testing of the 0.0, +7.0 and +2.8 ft swl's (maximum flood and ebb flow conditions)

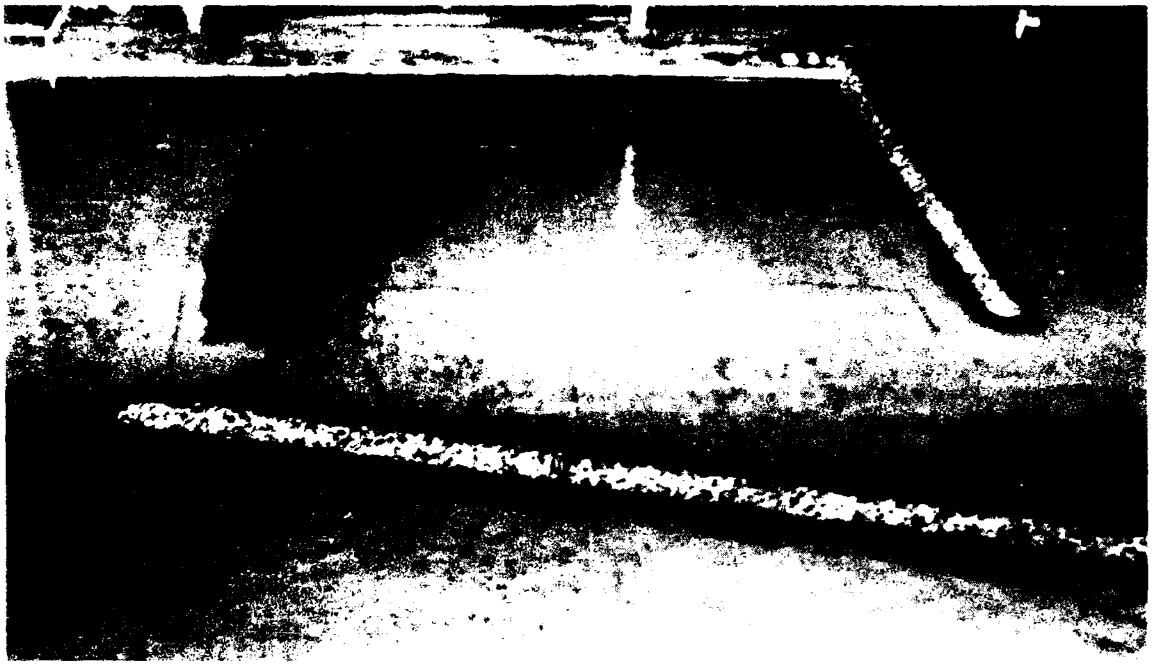


Photo 17. General movement of tracer material and subsequent deposits for Plan 7 for test waves from west-southwest after testing of the 0.0 ft swl

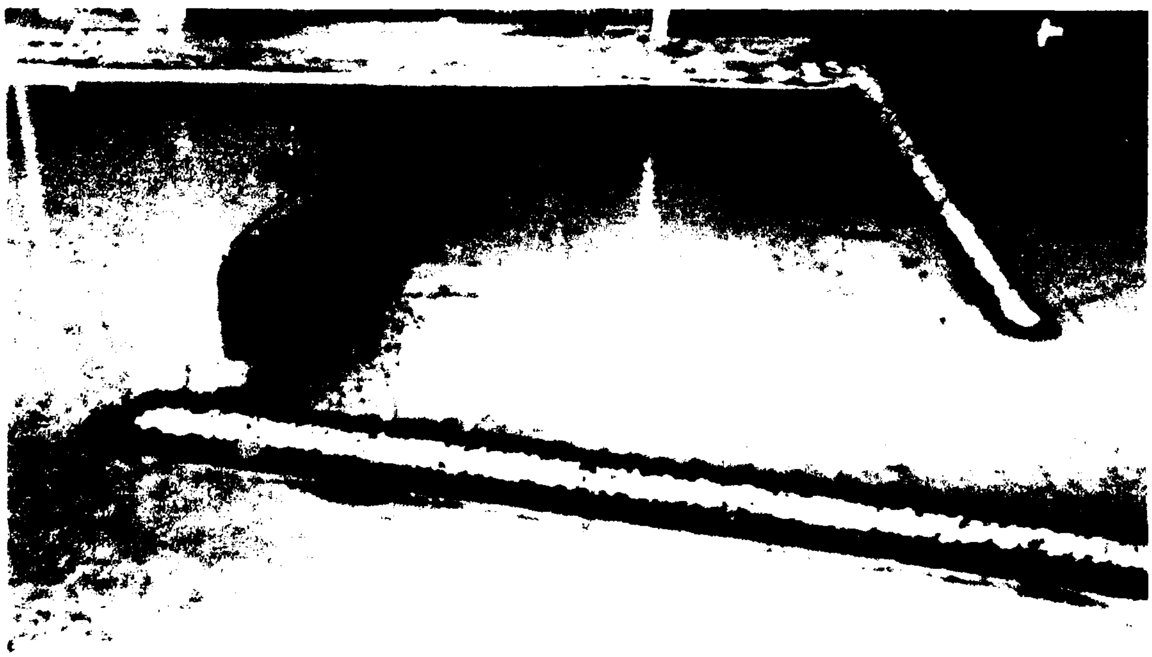


Photo 18. General movement of tracer material and subsequent deposits for Plan 7 for test waves from west-southwest after testing of the 0.0 and +7.0 ft swl's

Photo 19. General movement of tracer material and subsequent deposits for Plan 7 for test waves from west-southwest after testing of the 0.0, +7.0 and +2.8 ft swl's (maximum flood flow conditions)

Photo 20. General movement of tracer material and subsequent deposits for Plan 7 for test waves from west-southwest after testing of the 0.0, +7.0 and +2.8 ft swl's (maximum flood and ebb flow conditions)

Photo 21. General movement of tracer material and subsequent deposits on the north shore for Plan 7 for test waves from southwest after testing of the 0.0 ft swl

Photo 22. General movement of tracer material and subsequent deposits on the north shore for Plan 7 for test waves from southwest after testing of the 0.0 and +7.0 ft swl's

Photo 23. General movement of tracer material and subsquent deposits on the north shore for Plan 7 for test waves from southwest after testing of the 0.0, +7.0 and +2.8 ft swl's (maximum flood flow conditions)

Photo 24. General movement of tracer material and subsquent deposits on the north shore for Plan 7 for test waves from southwest after testing of the 0.0, +7.0 and +2.8 ft swl's (maximum flood and ebb flow conditions)

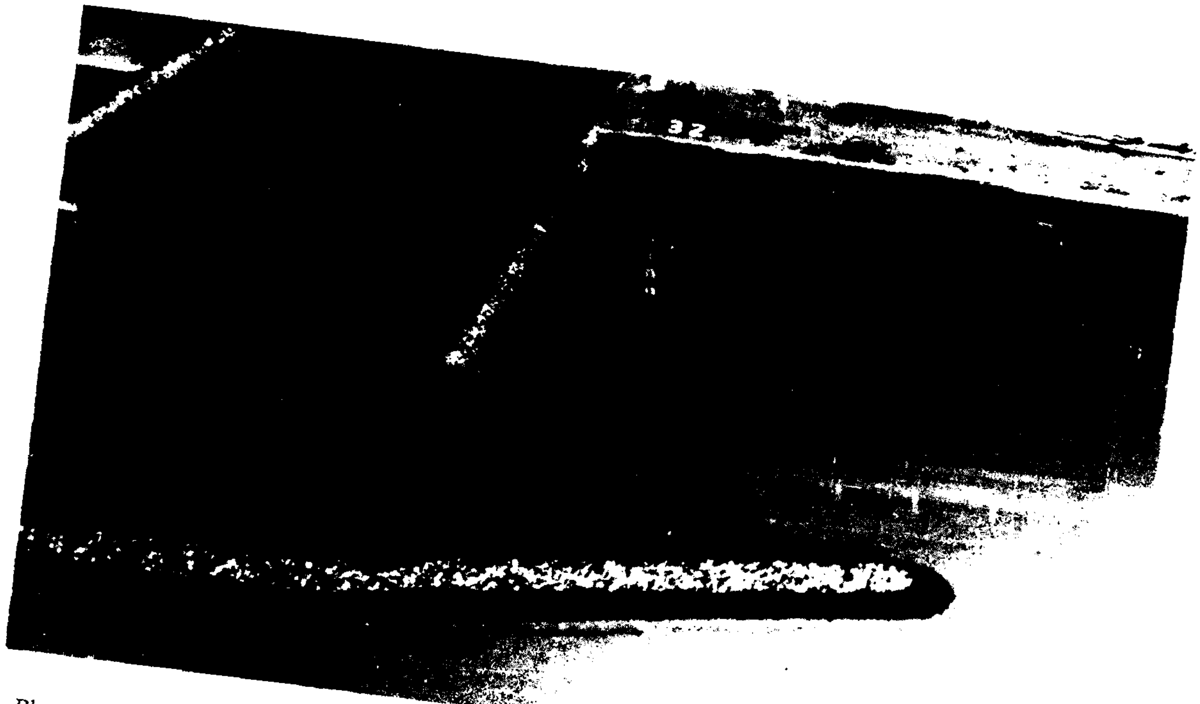


Photo 25. General movement of tracer material and subsequent deposits on the south shore for Plan 7 for test waves from southwest after testing of the 0.0 swl

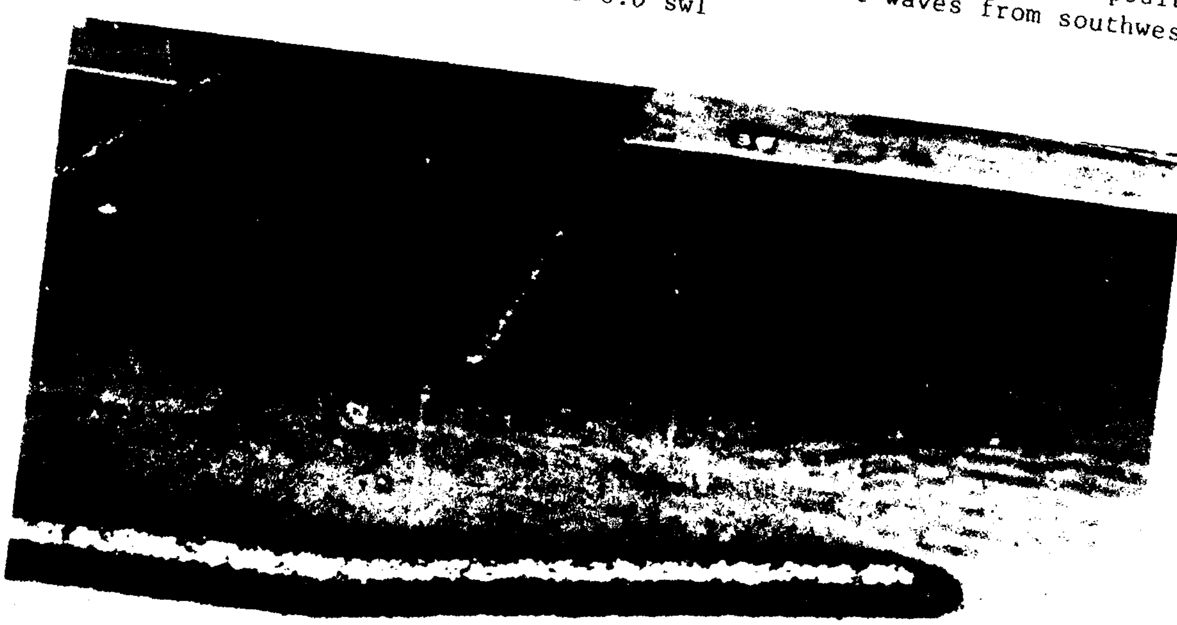


Photo 26. General movement of tracer material and subsequent deposits on the south shore for Plan 7 for test waves from southwest after testing of the 0.0 and +7.0 ft swl's

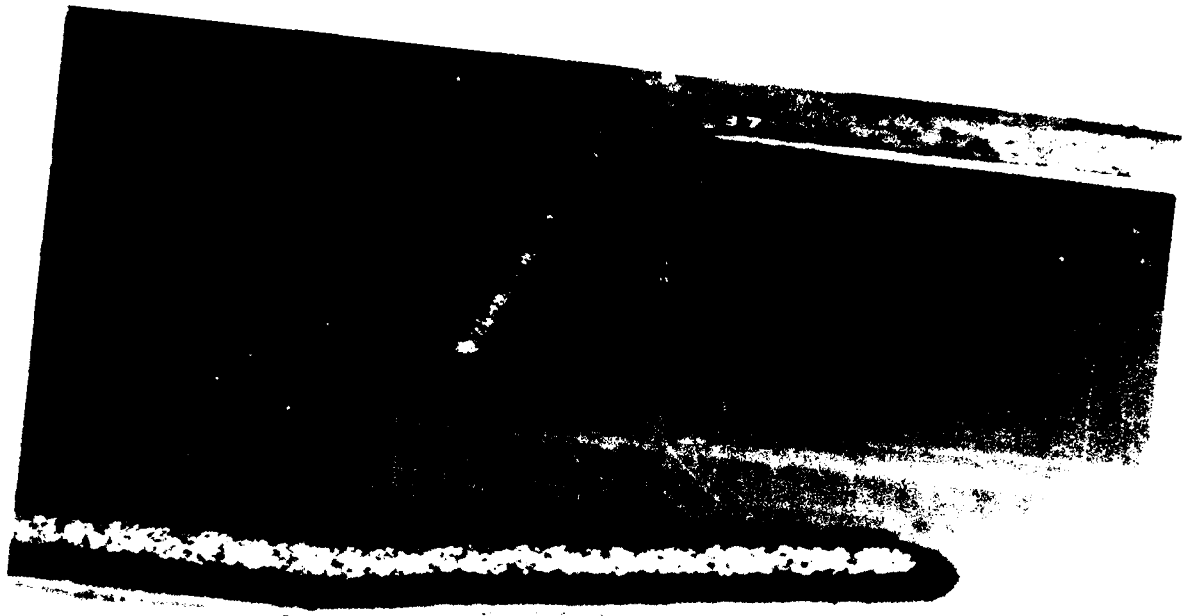


Photo 27. General movement of tracer material and subsquent deposits on the south shore for Plan 7 for test waves from southwest after testing of the 0.0, +7.0 and +2.8 ft swl's (maximum flood flow conditions)

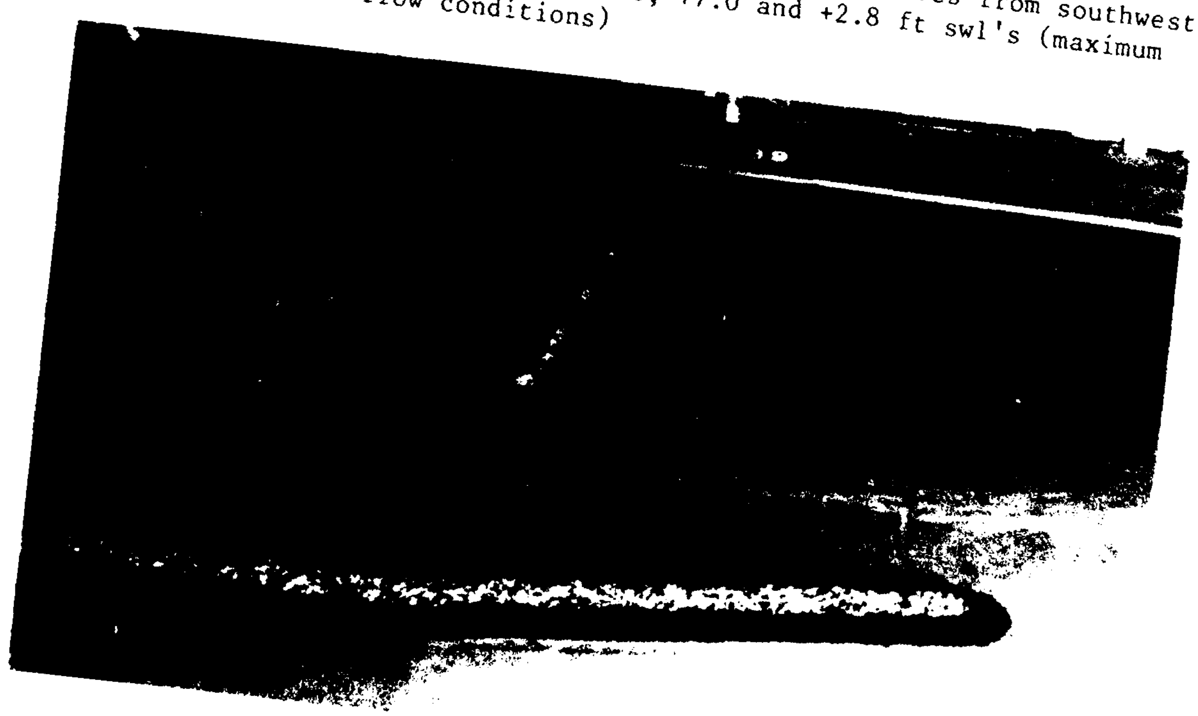


Photo 28. General movement of tracer material and subsequent deposits on the south shore for Plan 7 for test waves from southwest after testing of the 0.0, +7.0 and +2.8 ft swl's (maximum flood and ebb flow conditions)

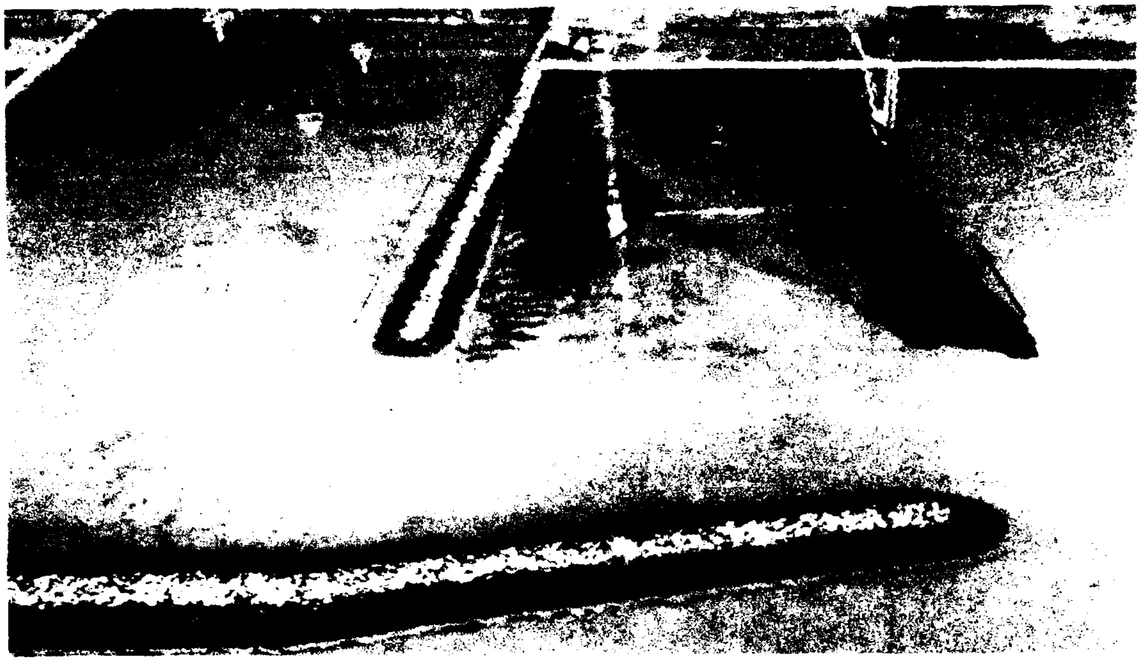


Photo 29. General movement of tracer material and subsequent deposits for Plan 7 for test waves from south-southwest after testing of the 0.0 ft swl

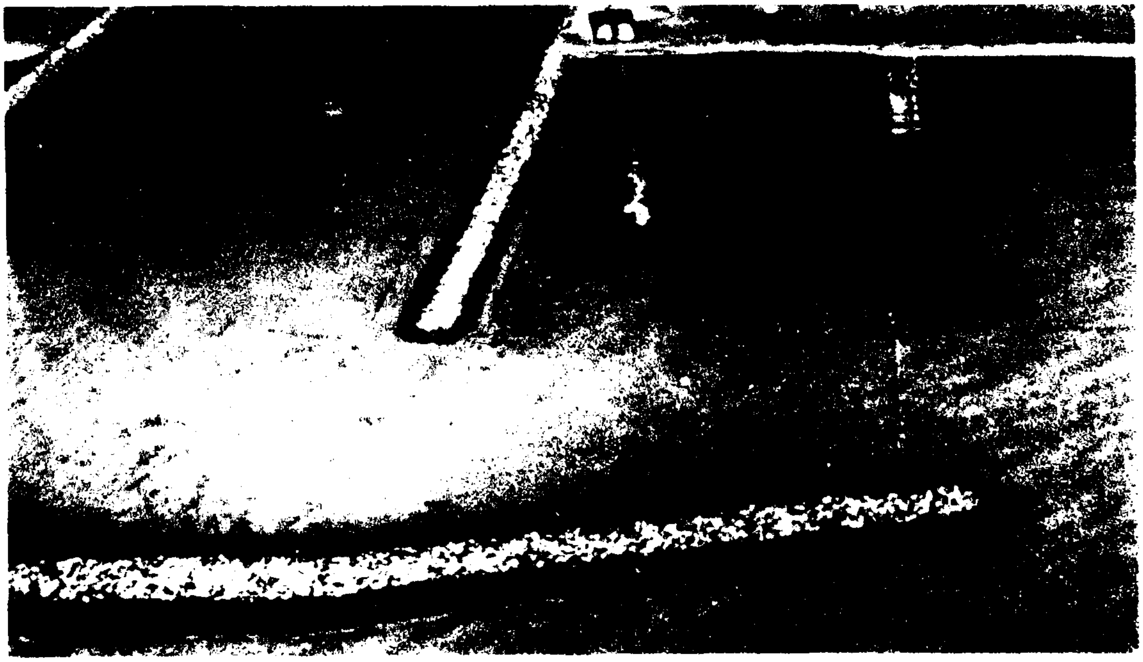


Photo 30. General movement of tracer material and subsquent deposits for Plan 7 for test waves from south-southwest after testing of the 0.0 +7.0 ft swl's

Photo 31. General movement of tracer material and subsquent deposits for Plan 7 for test waves from south-southwest after testing of the 0.0, +7.0 and +2.8 ft swl's (maximum flood flow conditions)

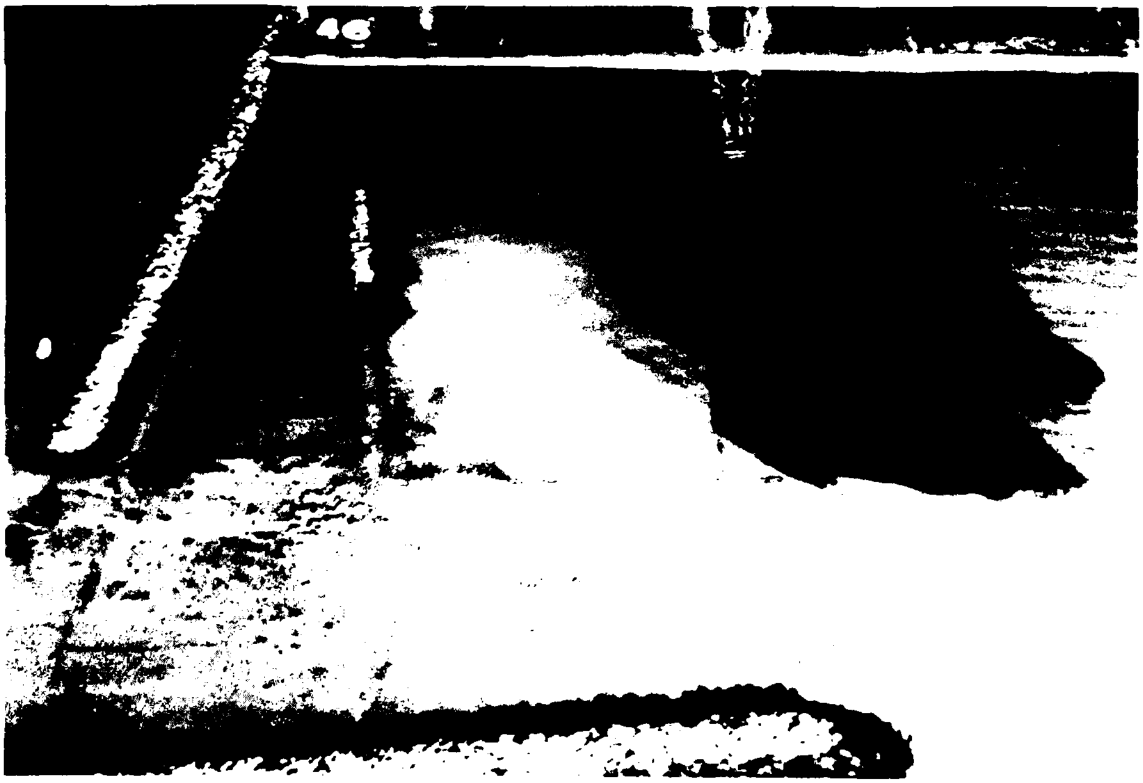


Photo 32. General movement of tracer material and subsquent deposits for Plan 7 for test waves from south-southwest after testing of the 0.0, +7.0 and +2.8 ft swl's (maximum flood and ebb flow conditions)

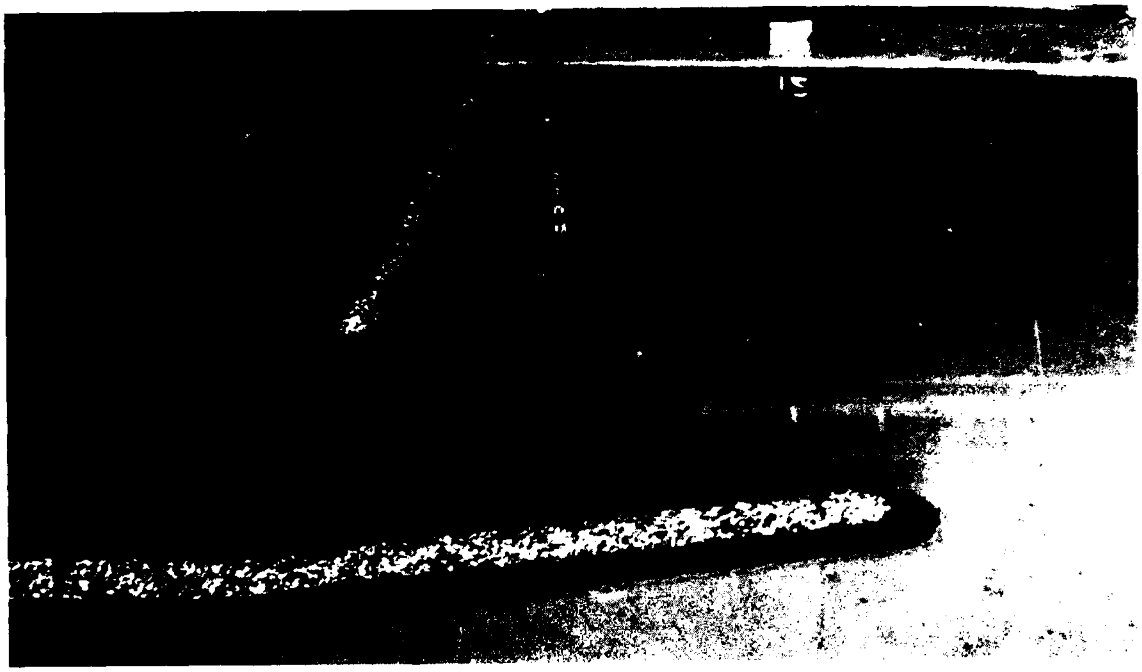


Photo 33. General movement of tracer material and subsequent deposits for Plan 7 for test waves from south after testing of the 0.0 ft swl

Photo 34. General movement of tracer material and subsequent deposits for Plan 7 for test waves from south after testing of the 0.0 and +7.0 ft swl's

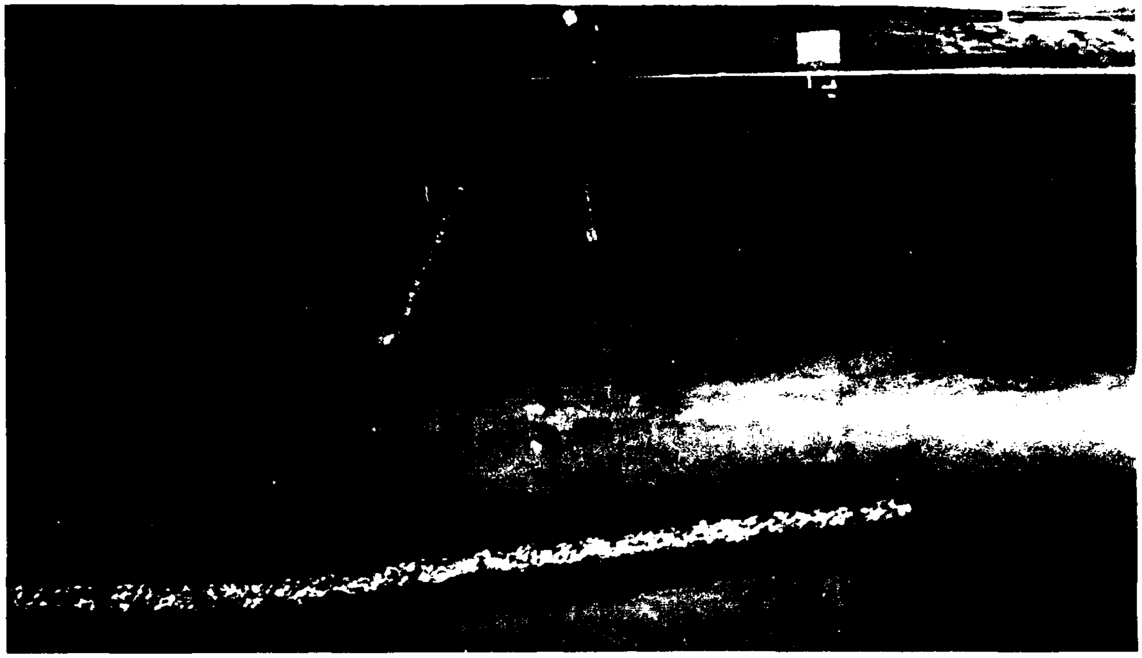


Photo 35. General movement of tracer material and subsequent deposits for Plan 7 for test waves from south after testing of the 0.0, +7.0 and +2.8 ft swl's (maximum flood flow conditions)

Photo 36. General movement of tracer material and subsequent deposits for Plan 7 for test waves from south after testing of the 0.0, +7.0 and +2.8 ft swl's (maximum flood and ebb flow conditions)

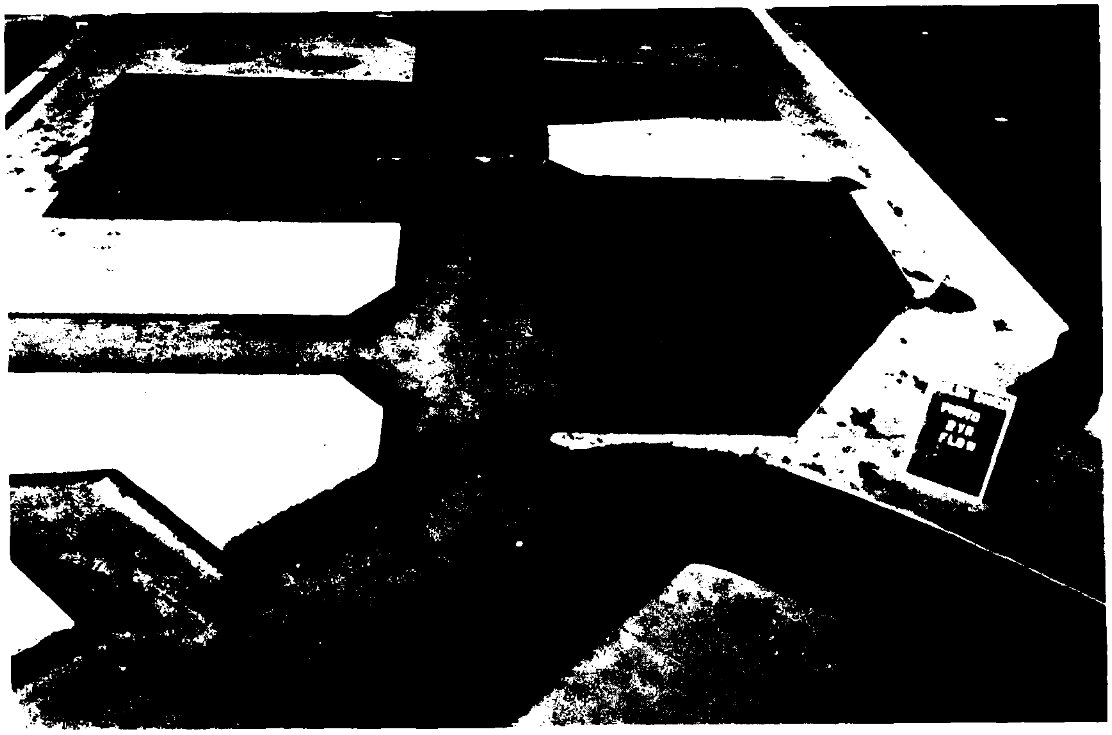


Photo 37. Current patterns and magnitudes (prototype fps) in interior channels of Plan 7 due to 1,760-cfs Wintersburg Channel discharge

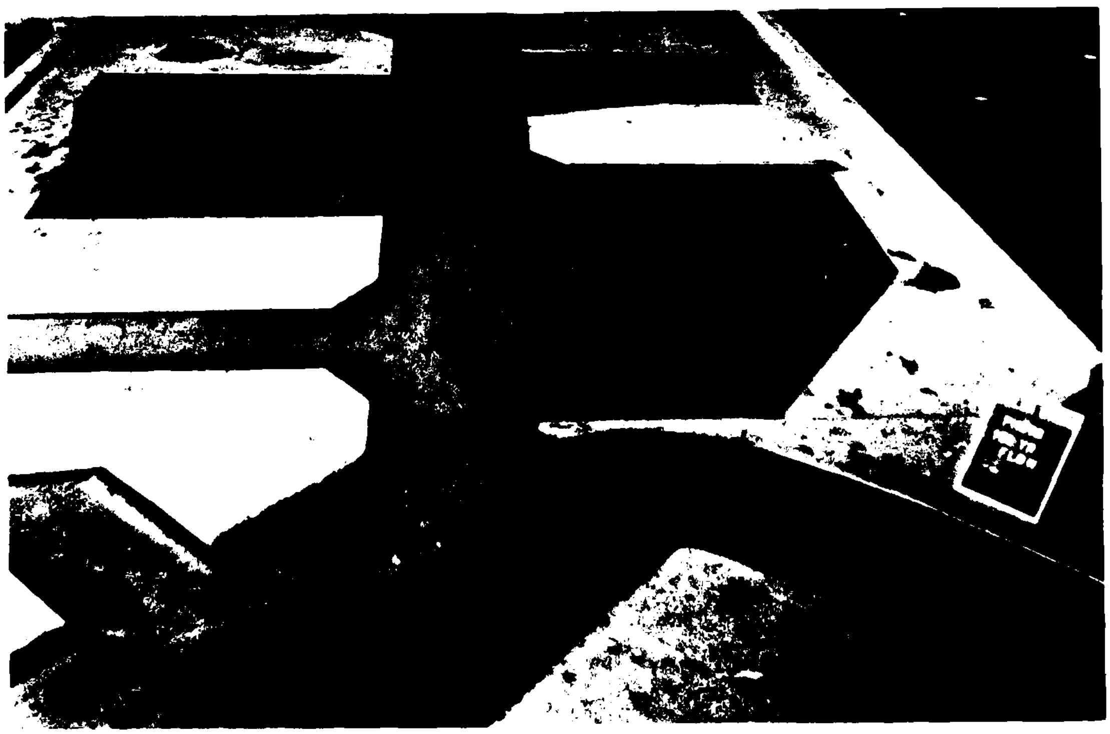


Photo 38. Current patterns and magnitudes (prototype fps) in interior channels of Plan 7 due to 9,710-cfs Wintersburg Channel discharge

Photo 39. Typical wave patterns for Plan 12; 13-sec, 9-ft waves from west; +7.0 ft swl

Photo 40. Typical wave patterns for Plan 12; 14.3-sec, 11.7-ft waves from west-southwest; +7.0 ft swl

Photo 41. Typical wave patterns for Plan 12; 15-sec, 10-ft waves from southwest; +7.0 ft swl

Photo 42. Typical wave patterns for Plan 13; 13-sec, 9-ft waves from west; +7.0 ft swl

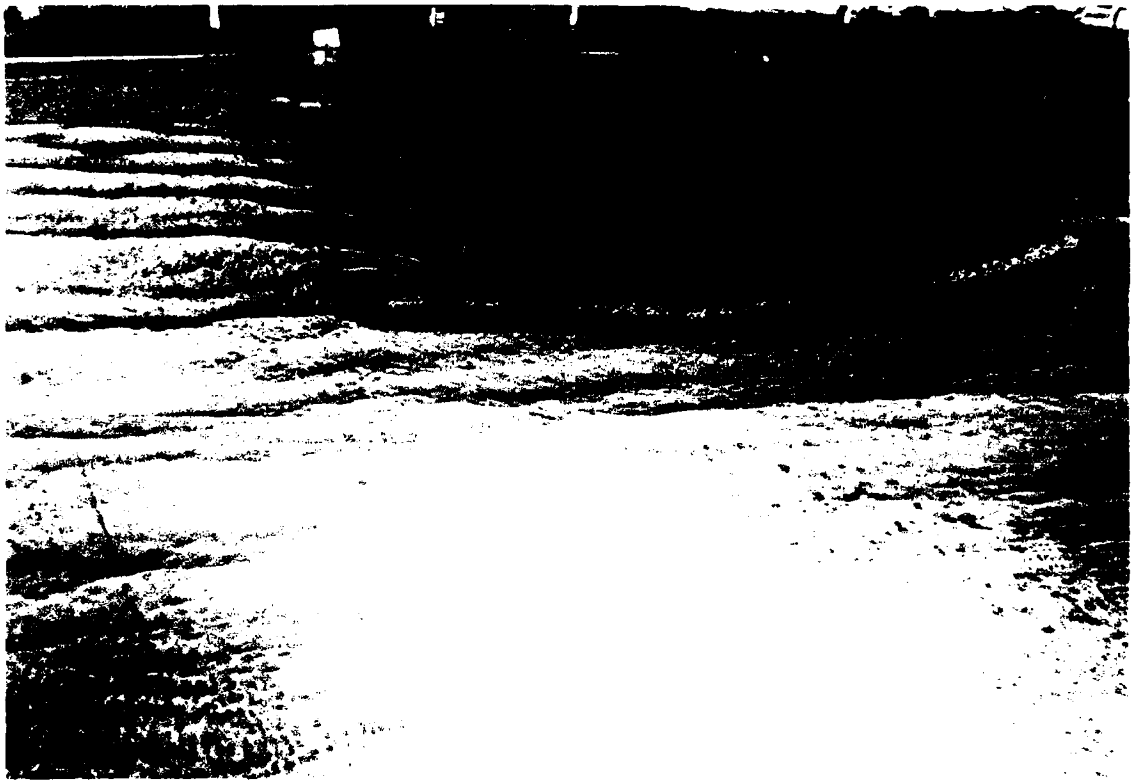


Photo 43. Typical wave patterns for Plan 15; 14.3-sec, 11.7-ft waves from west-southwest; +7.0 ft swl

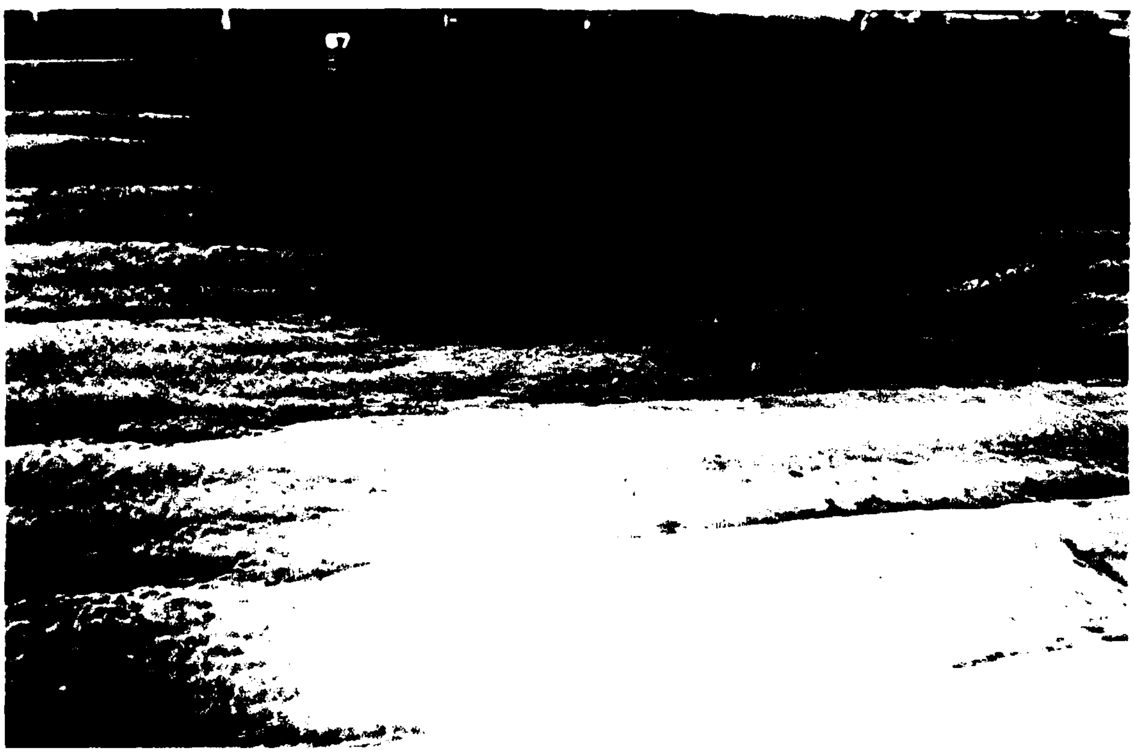


Photo 44. Typical wave patterns for Plan 16; 14.3-sec, 11.7-ft waves from west-southwest; +7.0 ft swl

Photo 45. Typical wave patterns for Plan 17; 14.3-sec, 11.7-ft waves from west-southwest; +7.0 ft swl

Photo 46. Typical wave patterns for Plan 16; 13-sec, 9-ft waves from west; +7.0 ft swl

Photo 47. Typical wave patterns for Plan 17; 13-sec, 9-ft waves from west; +7.0 ft swl

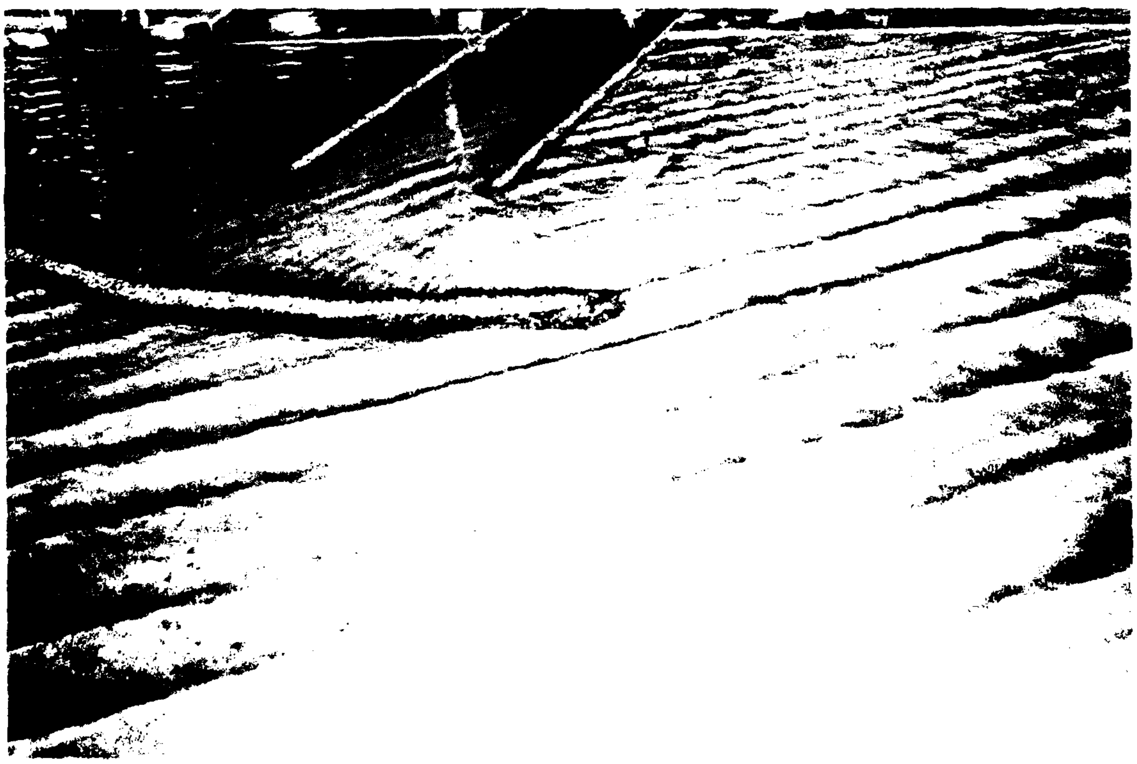


Photo 48. Typical wave patterns for Plan 17; 9-sec, 8-ft waves from south; +7.0 ft swl

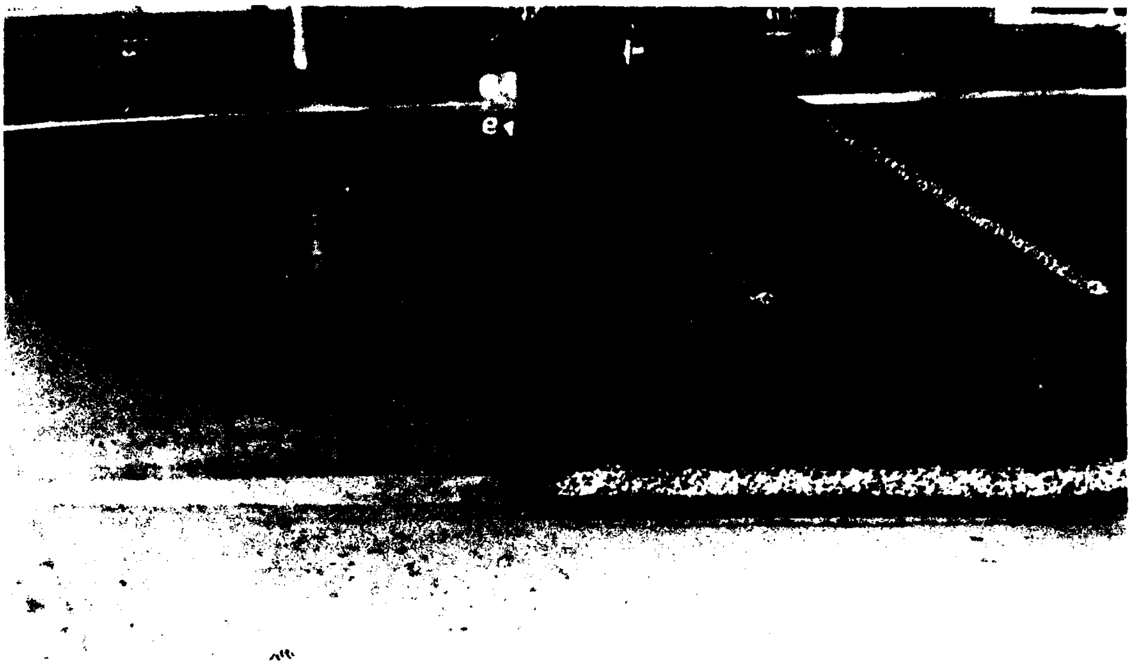


Photo 49. General movement of tracer material and subsequent deposits for Plan 15 for test waves from west after testing of the 0.0 ft swl

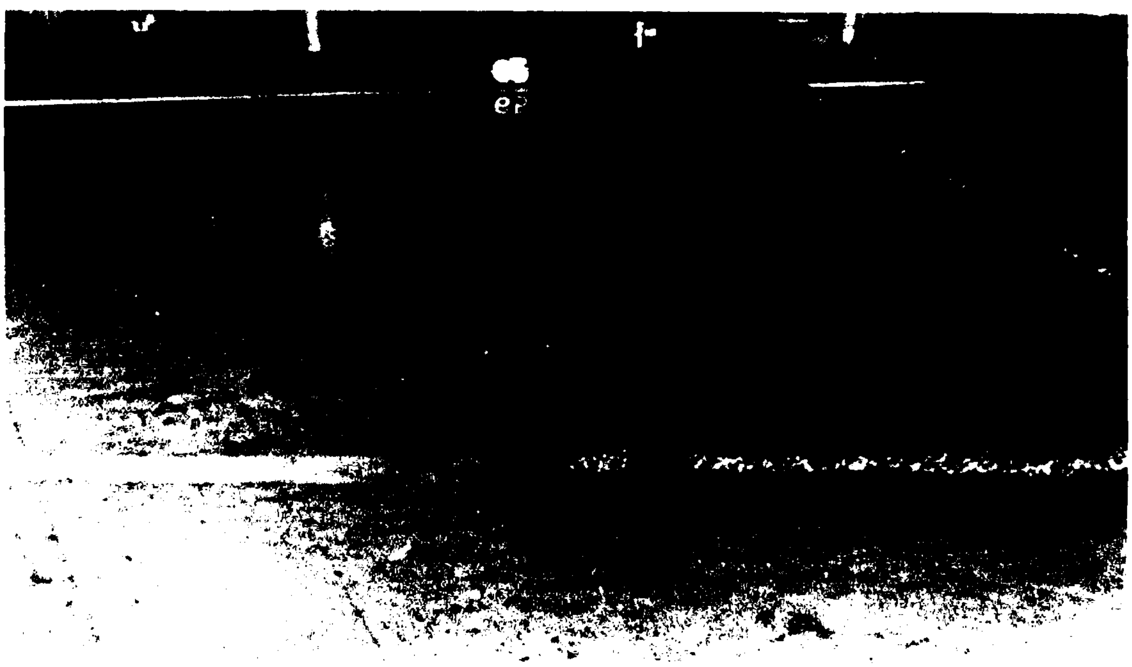


Photo 50. General movement of tracer material and subsequent deposits for Plan 15 for test waves from west after testing of the 0.0 and +7.0 ft swl's

Photo 51. General movement of tracer material and subsequent deposits for Plan 17 for test waves from west after testing of the 0.0 ft swl

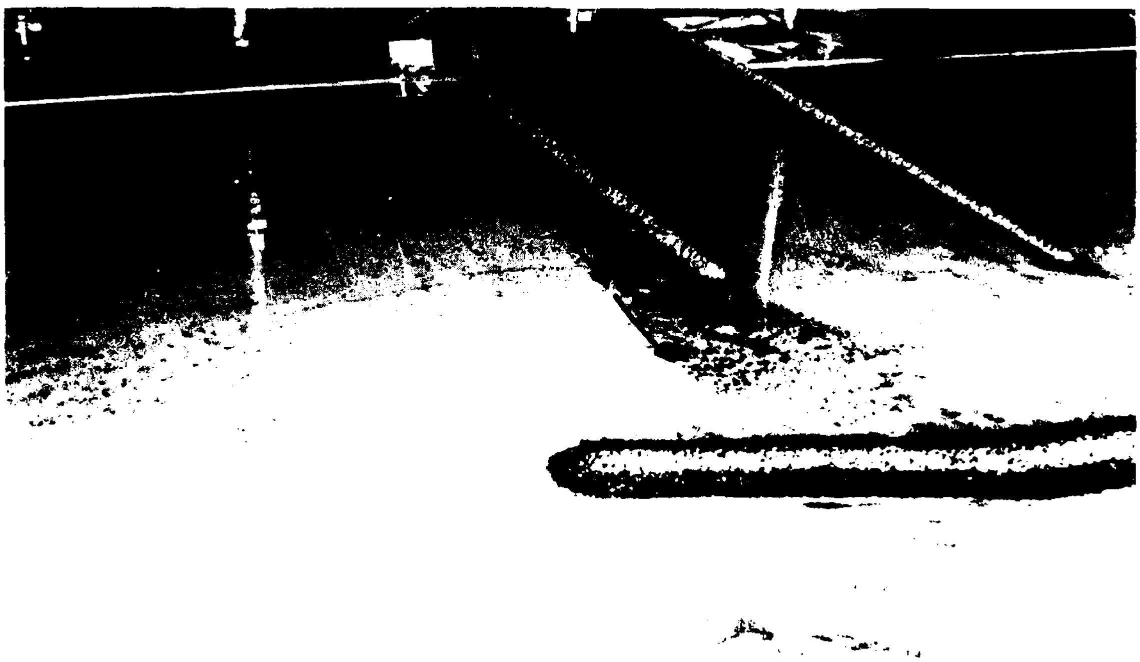


Photo 52. General movement of tracer material and subsequent deposits for Plan 17 for test waves from west after testing of the 0.0 and +7.0 ft swl's

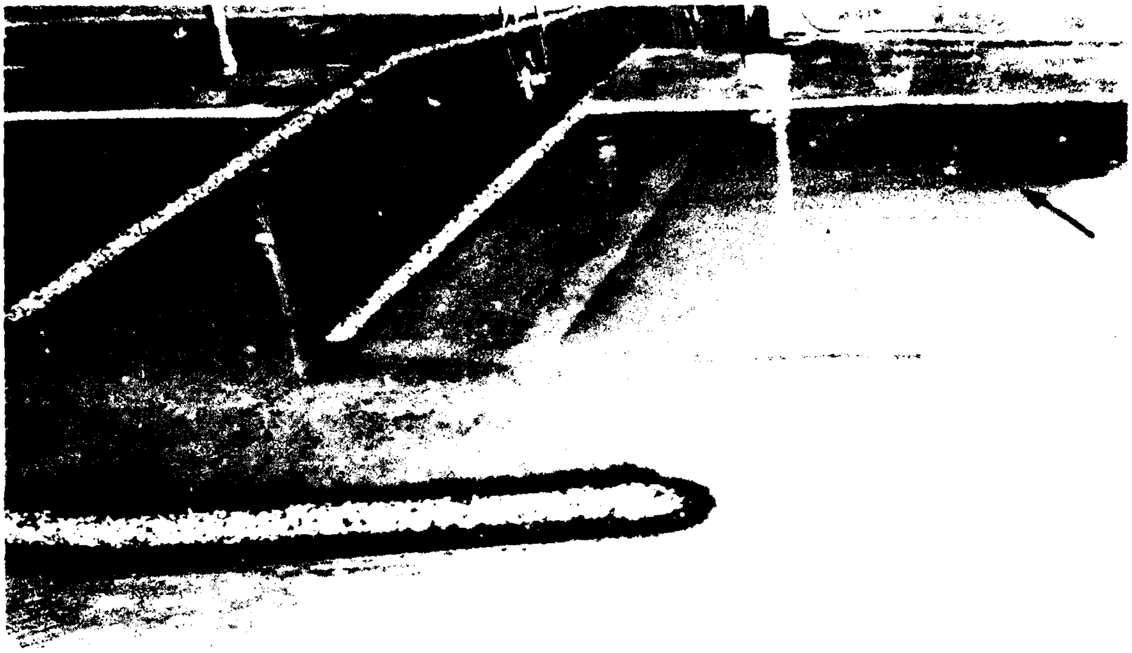


Photo 53. General movement of tracer material and subsequent deposits for Plan 15 for test waves from south after testing of the 0.0 ft swl

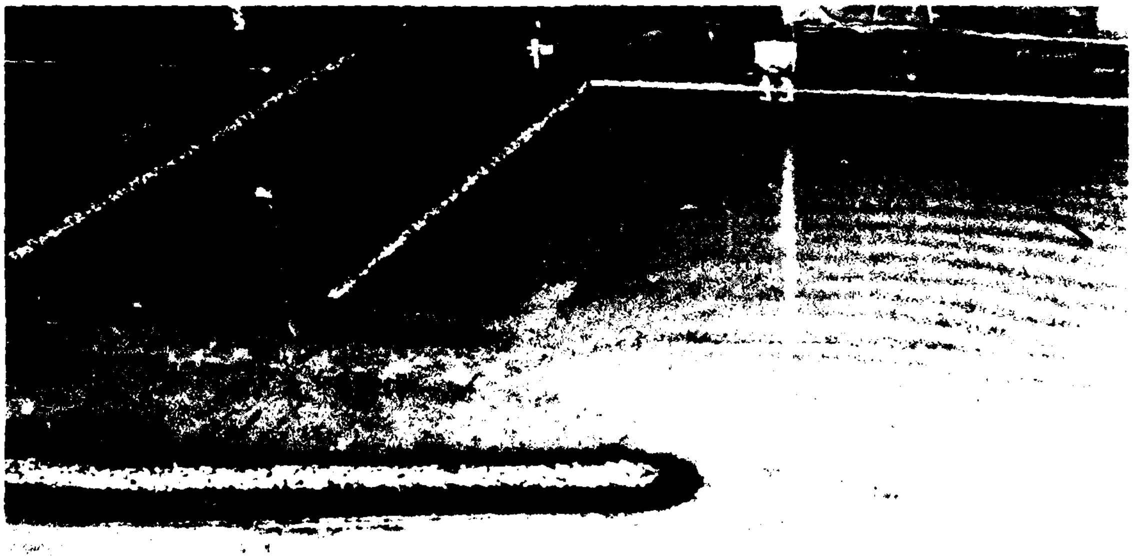


Photo 54. General movement of tracer material and subsequent deposits for Plan 15 for test waves from south after testing of the 0.0 and +7.0 ft swl's

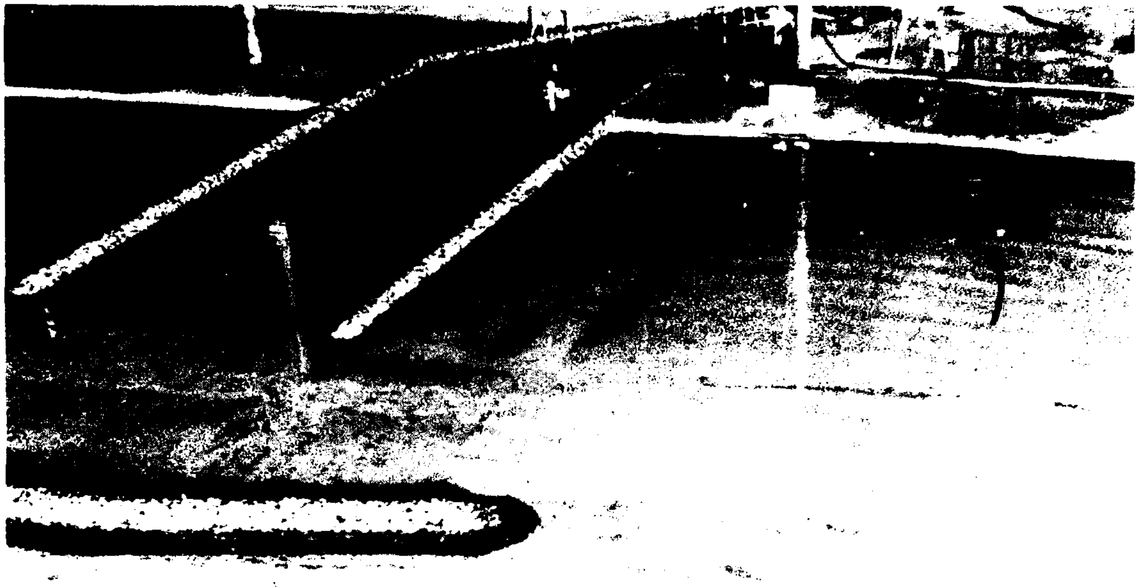


Photo 55. General movement of tracer material and subsequent deposits for Plan 16 for test waves from south after testing of the 0.0 ft swl

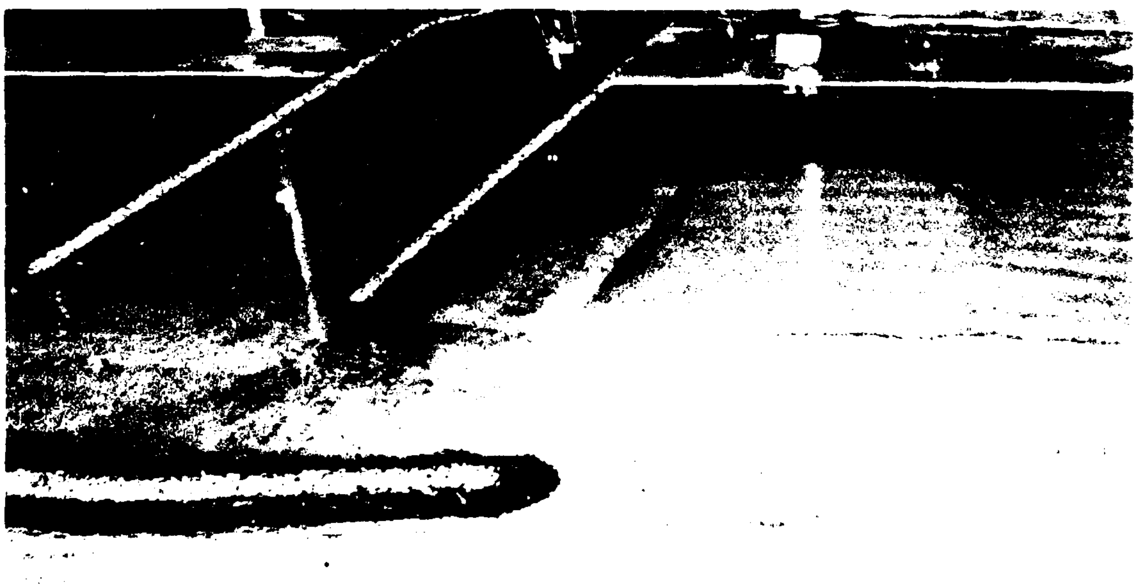


Photo 56. General movement of tracer material and subsequent deposits for Plan 16 for test waves from south after testing of the 0.0 and +7.0 ft swl's

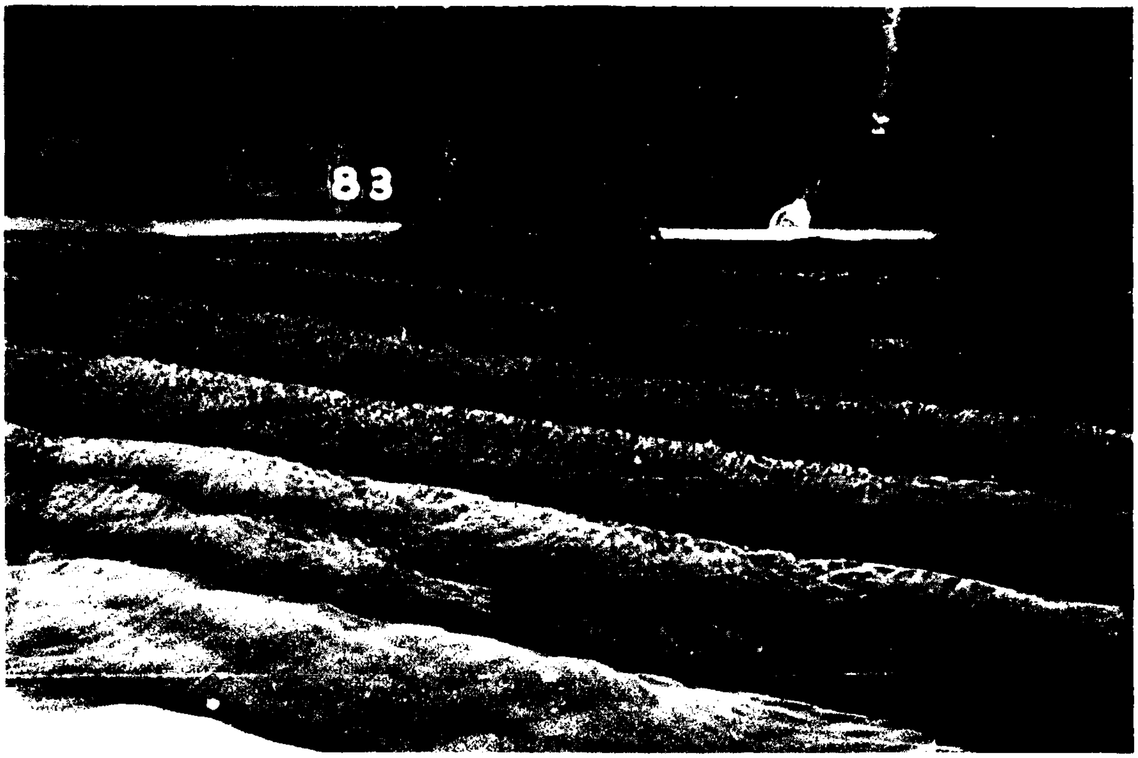


Photo 57. Typical wave patterns for Plan 18; 9-sec, 7-ft waves from west; +7.0 ft swl

Photo 58. Typical wave patterns for Plan 18; 9-sec, 9-ft waves from west-southwest; +7.0 ft swl

Photo 59. Typical wave patterns for Plan 18; 7-sec, 9-ft waves from southwest; +7.0 ft swl

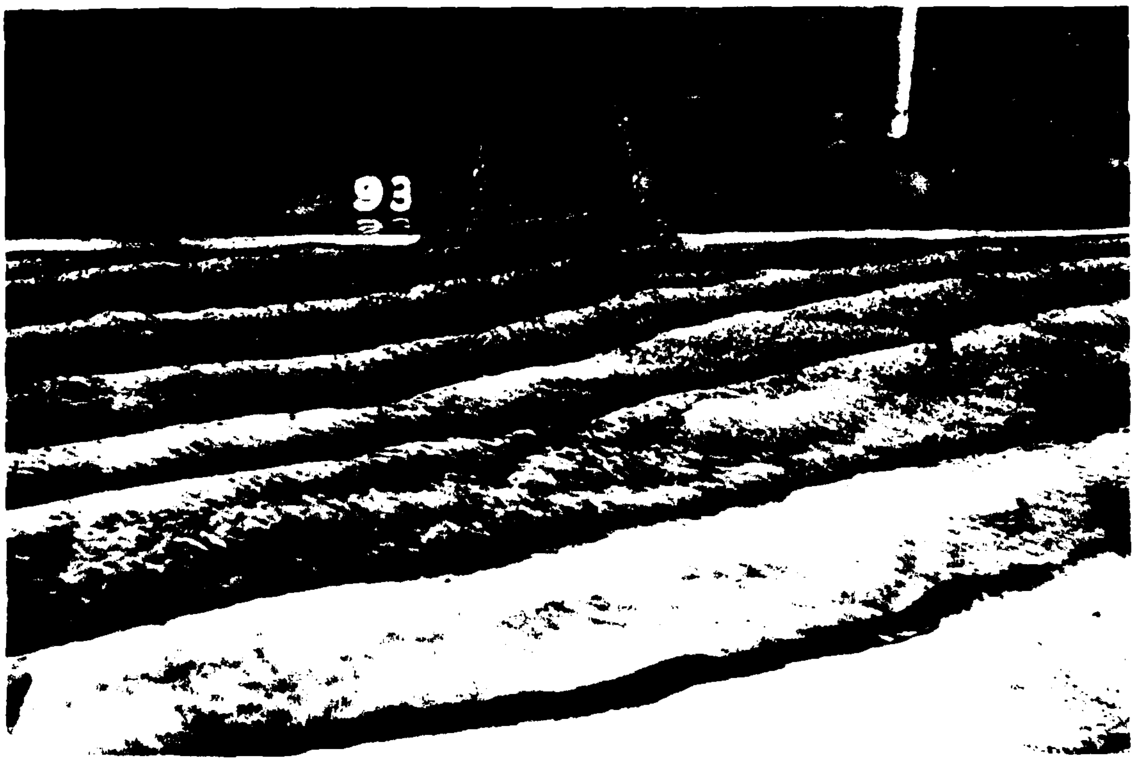


Photo 60. Typical wave patterns for Plan 18; 9-sec, 8-ft waves from south-southwest; +7.0 ft swl

Photo 61. Typical wave patterns for Plan 18; 9-sec, 8-ft waves from south; +7.0 ft swl

Photo 62. General movement of tracer material and subsequent deposits for Plan 18 for test waves from west after testing of the 0.0 ft swl

Photo 63. General movement of tracer material and subsequent deposits for Plan 18 for test waves from west after testing of the 0.0 and +7.0 ft swl's

Photo 64. General movement of tracer material and subsequent deposits for Plan 18 for test waves from west-southwest after testing of the 0.0 ft swl

Photo 65. General movement of tracer material and subsequent deposits for Plan 18 for test waves from west-southwest after testing of the 0.0 and +7.0 ft swl's

Photo 66. General movement of tracer material and subsequent deposits for Plan 18 for test waves from southwest after testing of the 0.0 ft swl

Photo 67. General movement of tracer material and subsequent deposits for Plan 18 for test waves from southwest after testing of the 0.0 and +7.0 ft swl's

Photo 68. General movement of tracer material and subsequent deposits for Plan 18 for test waves from south-southwest after testing of the 0.0 ft swl

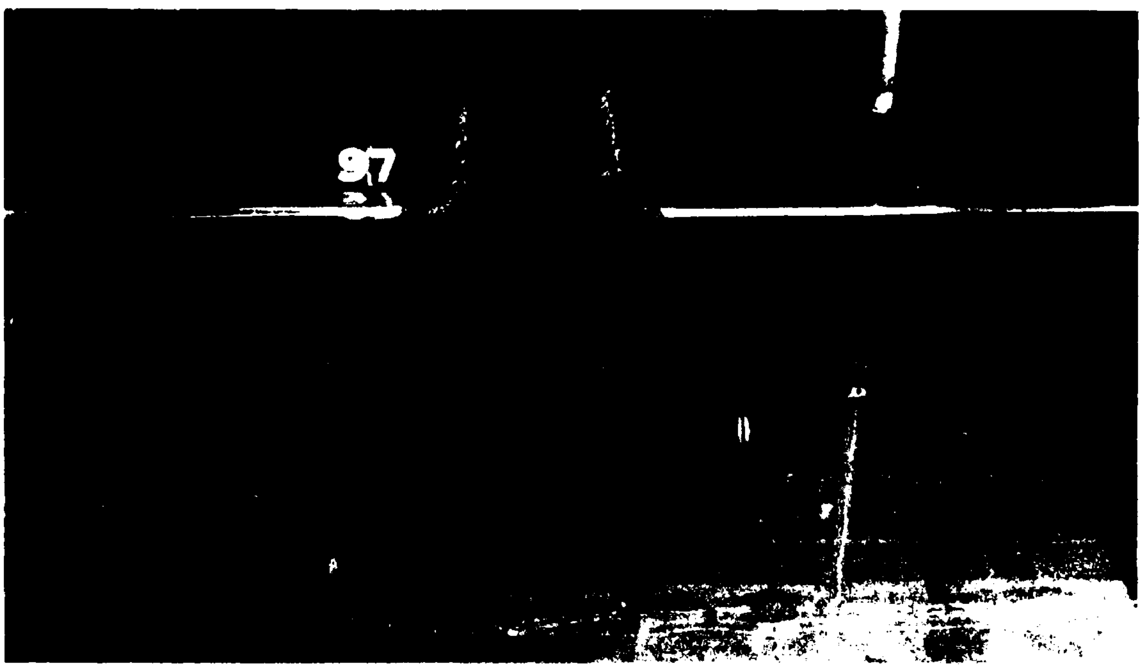
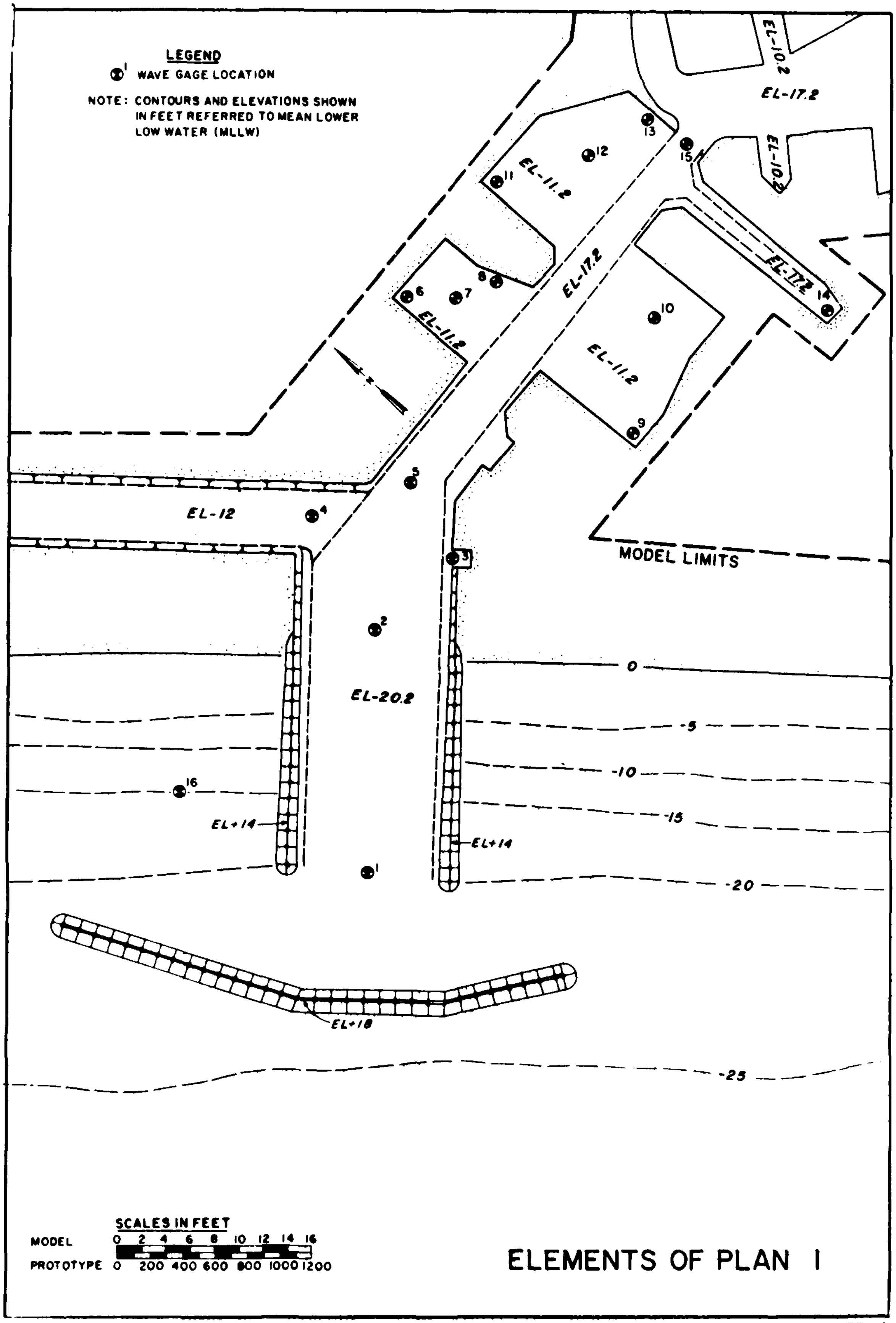


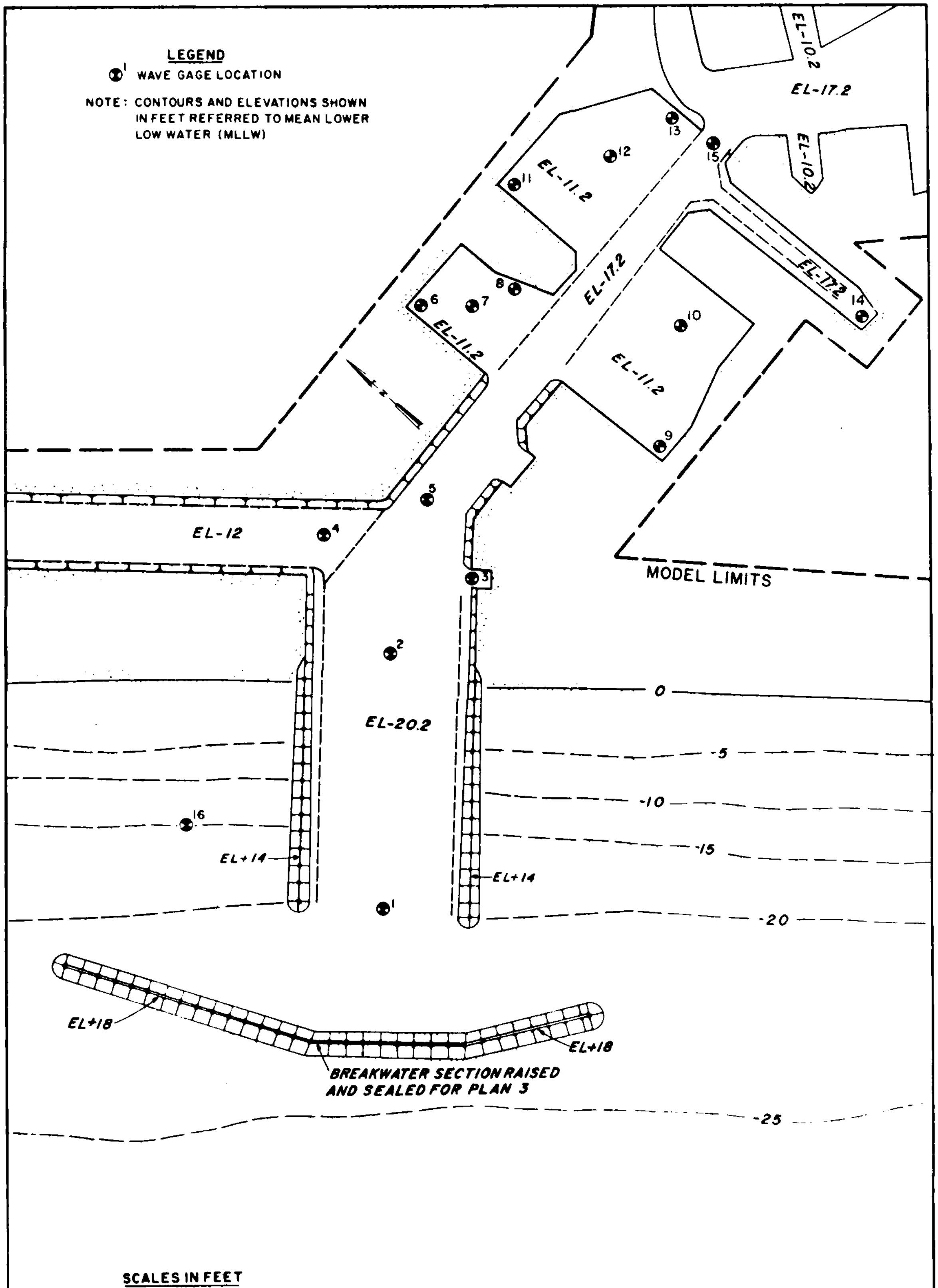
Photo 69. General movement of tracer material and subsequent deposits for Plan 18 for test waves from south-southwest after testing of the 0.0 and +7.0 ft swl's

Photo 70. General movement of tracer material and subsequent deposits for Plan 18 for test waves from south after testing of the 0.0 ft swl

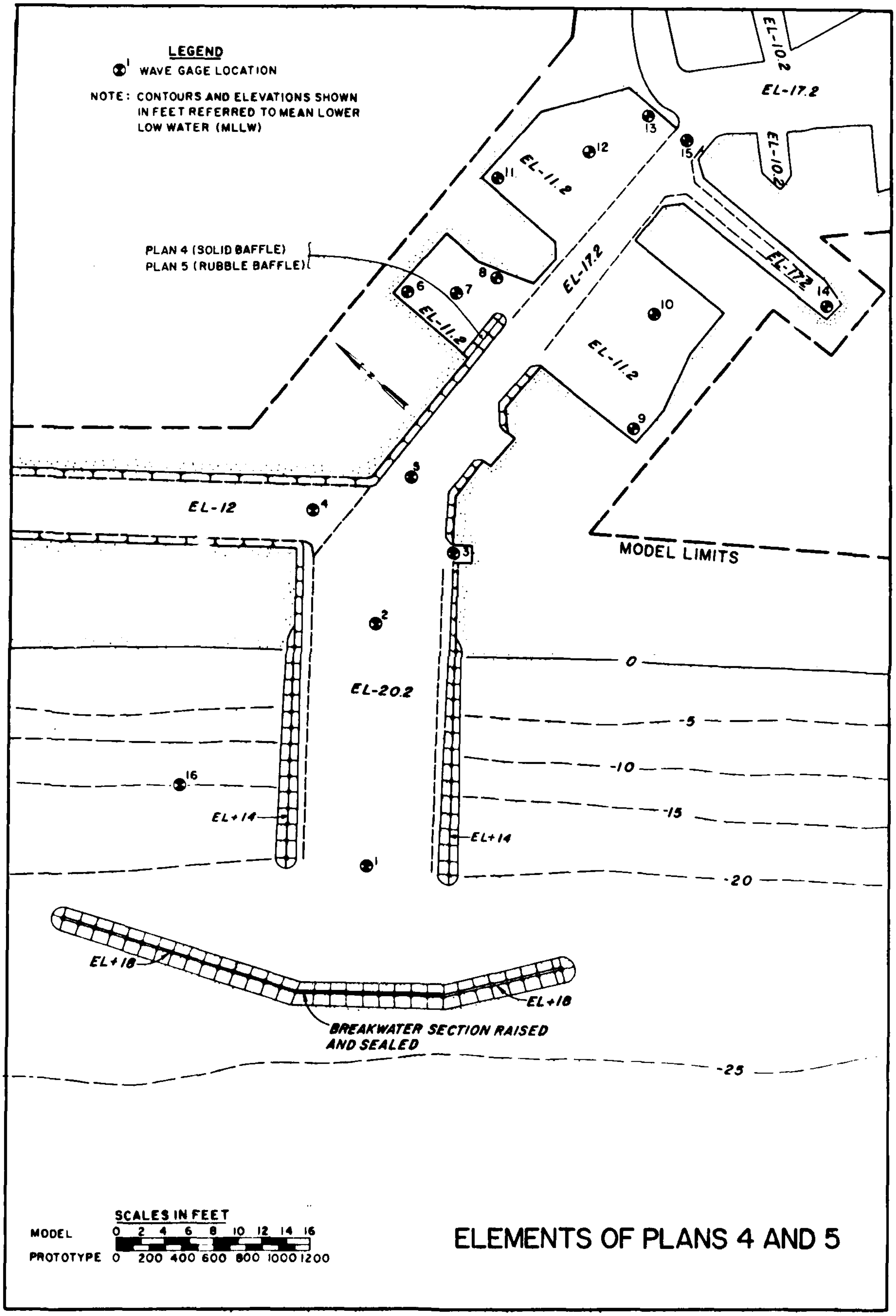
Photo 71. General movement of tracer material and subsequent deposits for Plan 18 for test waves from south after testing of the 0.0 and +7.0 ft swl's



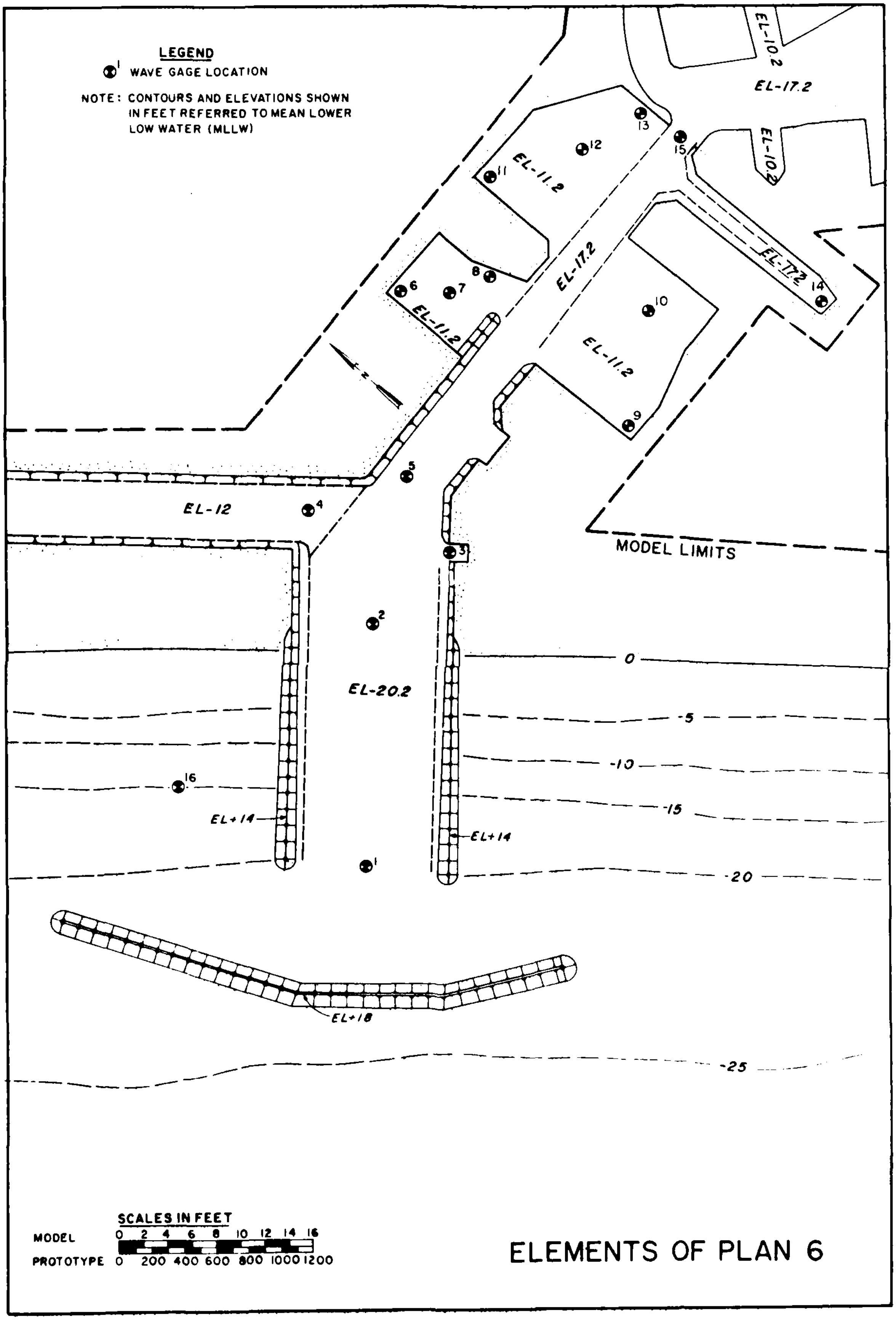
ELEMENTS OF PLAN 1



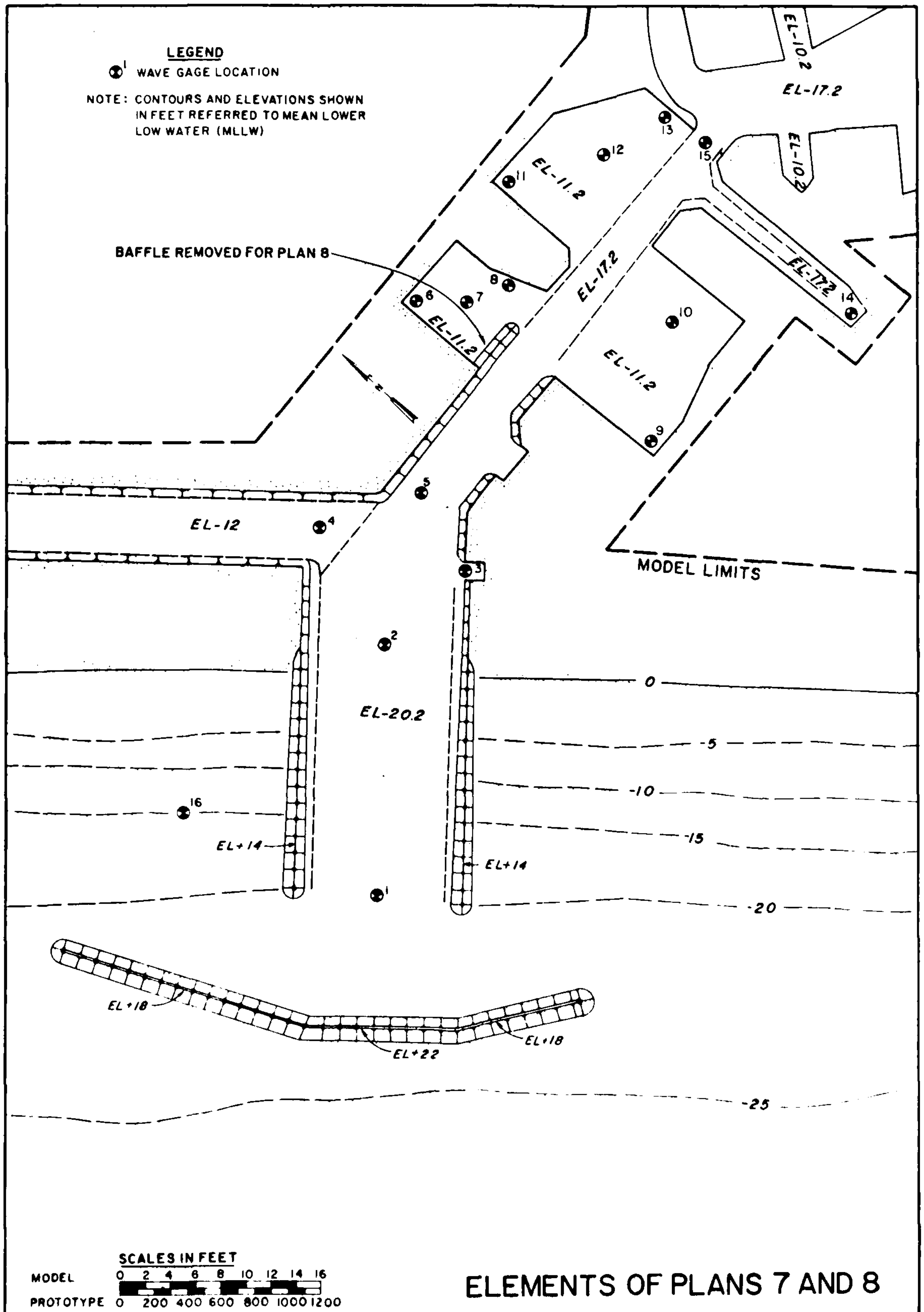
ELEMENTS OF PLANS 2 AND 3



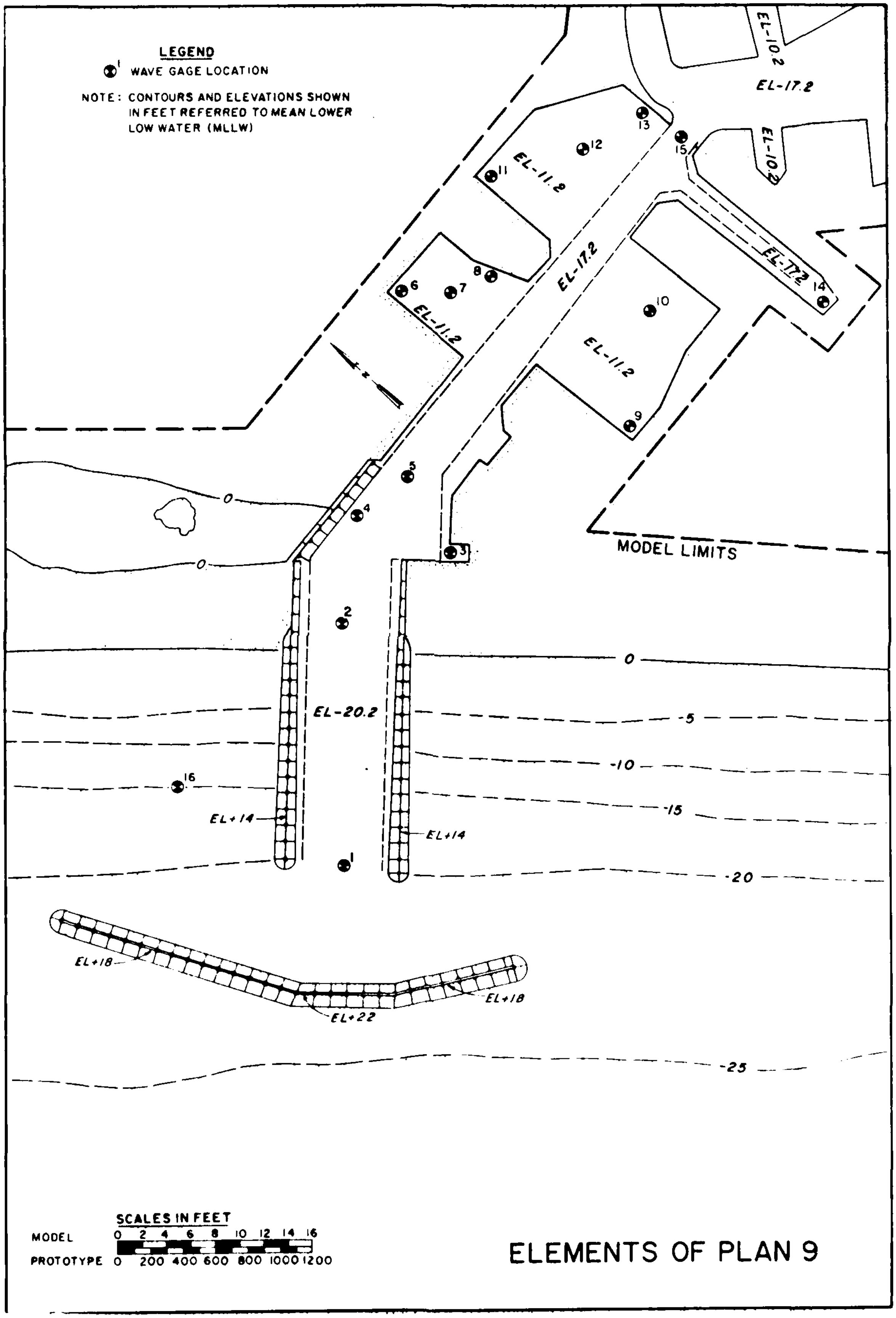
ELEMENTS OF PLANS 4 AND 5



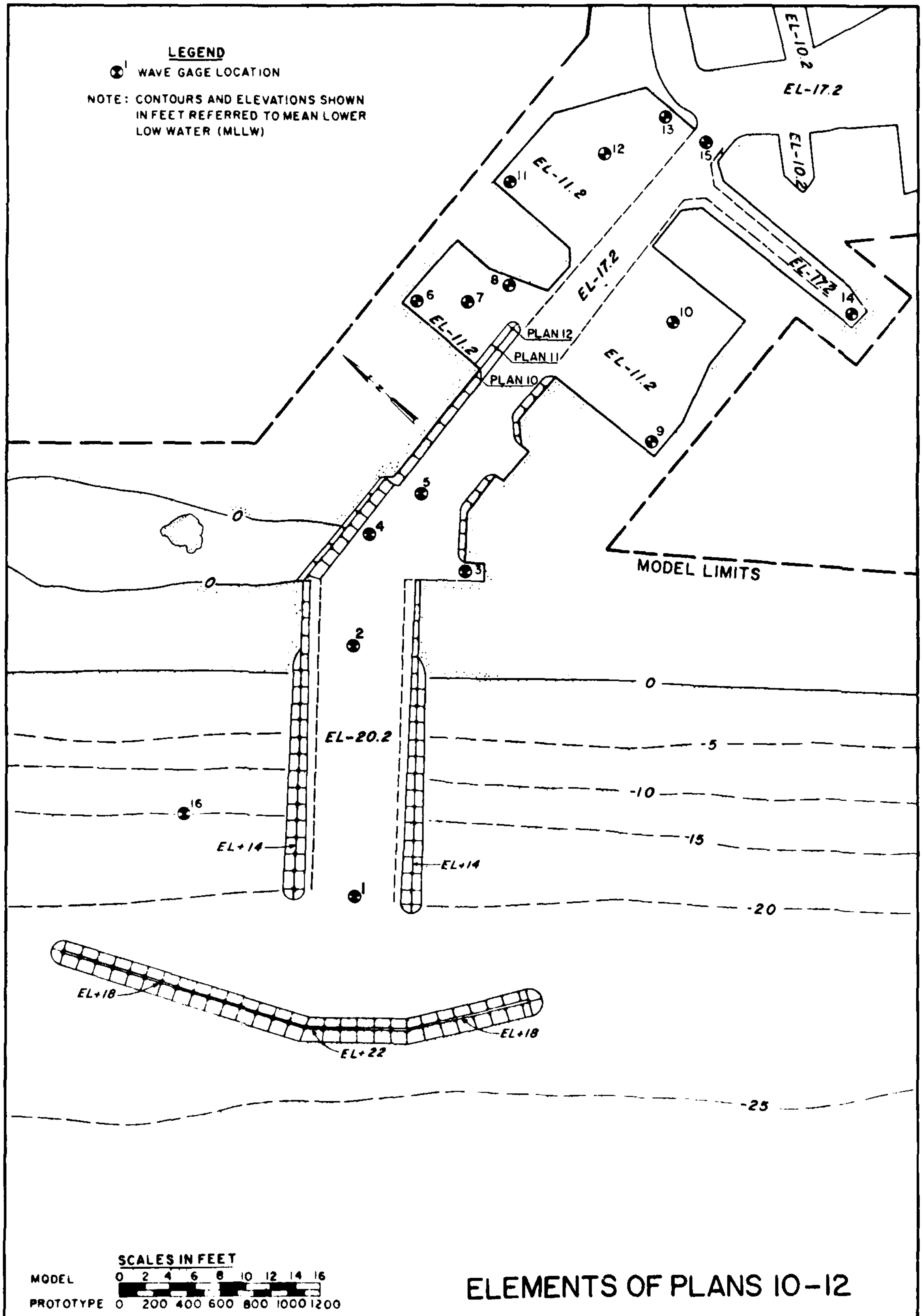
ELEMENTS OF PLAN 6



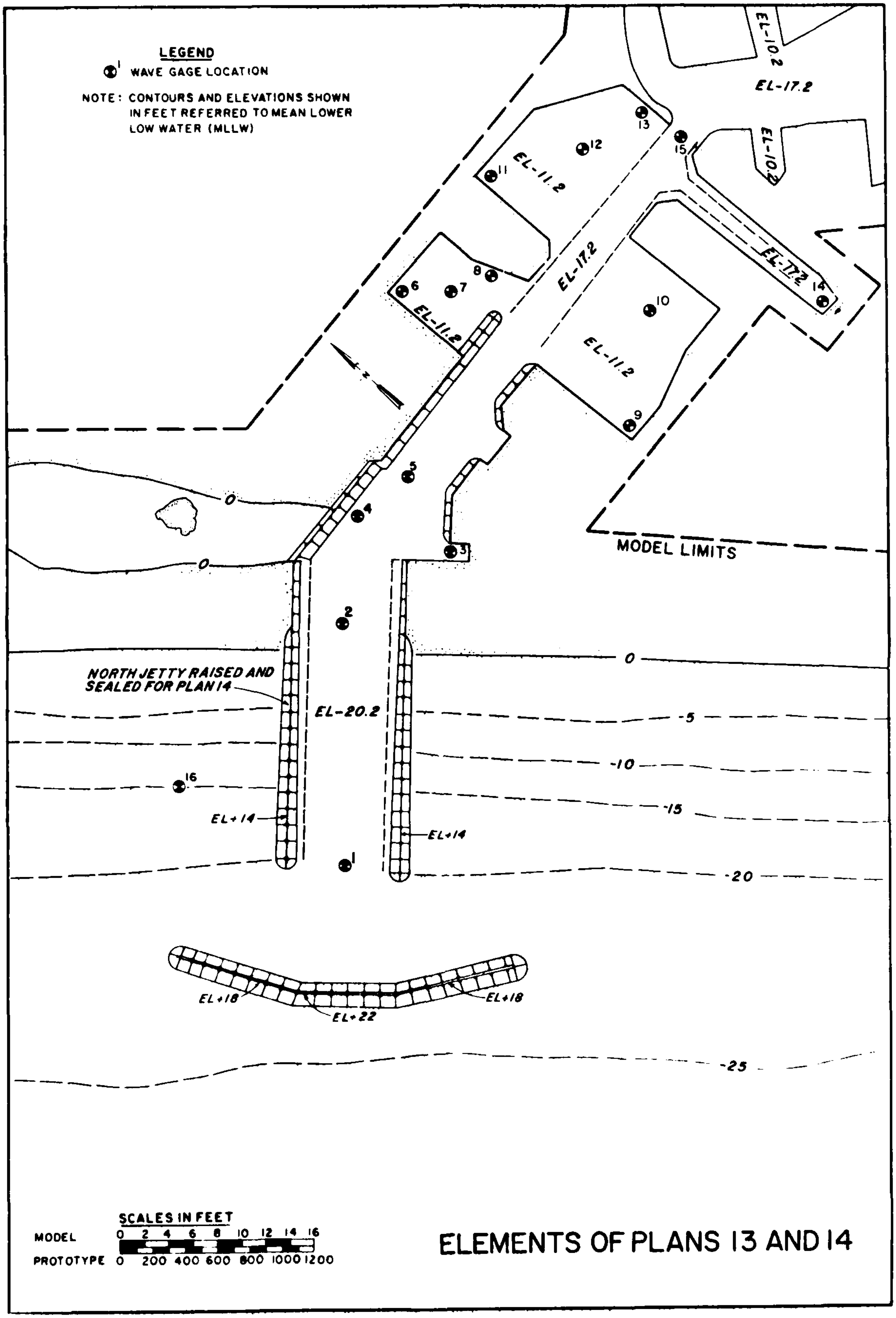
ELEMENTS OF PLANS 7 AND 8



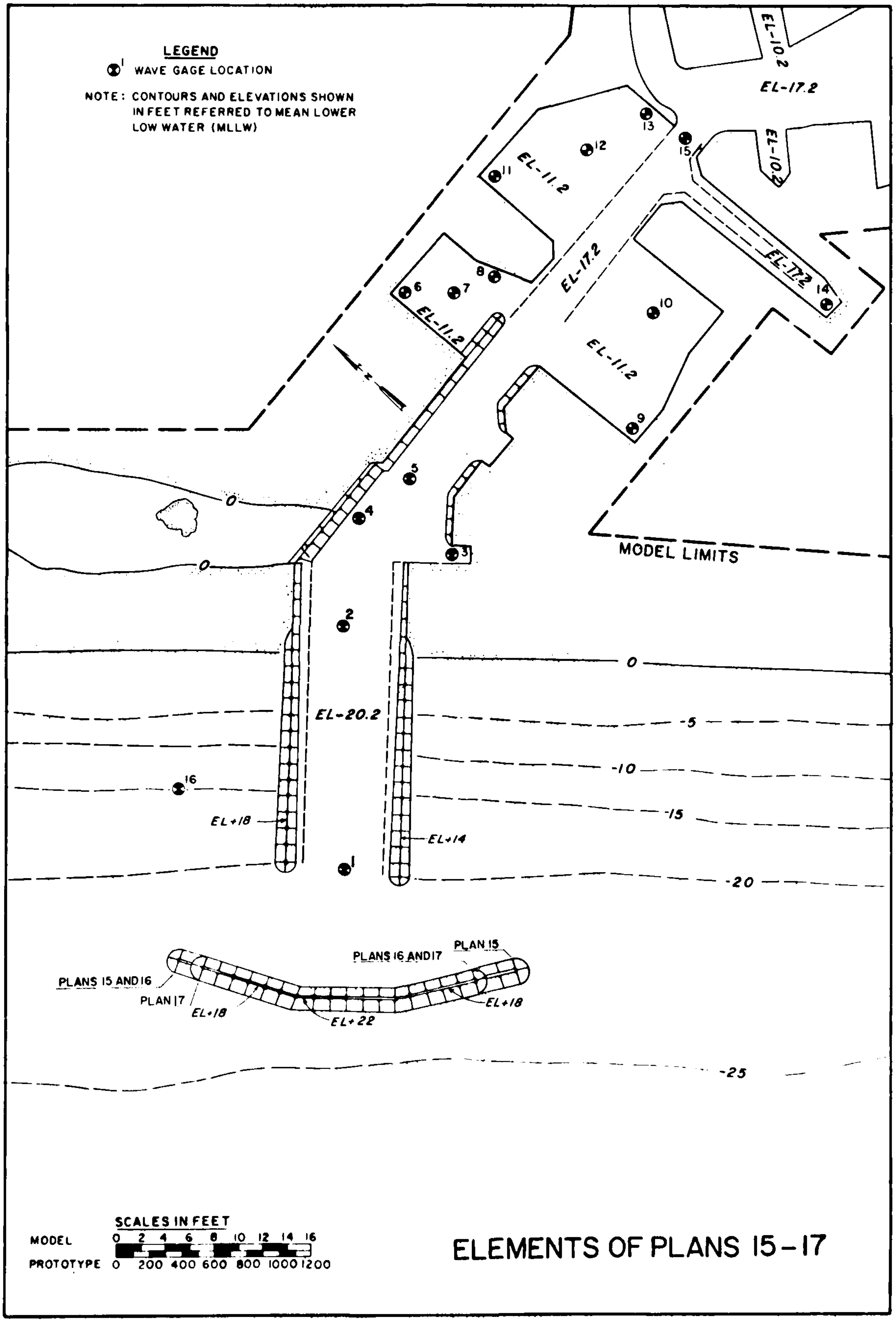
ELEMENTS OF PLAN 9



ELEMENTS OF PLANS 10-12



ELEMENTS OF PLANS 13 AND 14



ELEMENTS OF PLANS 15-17

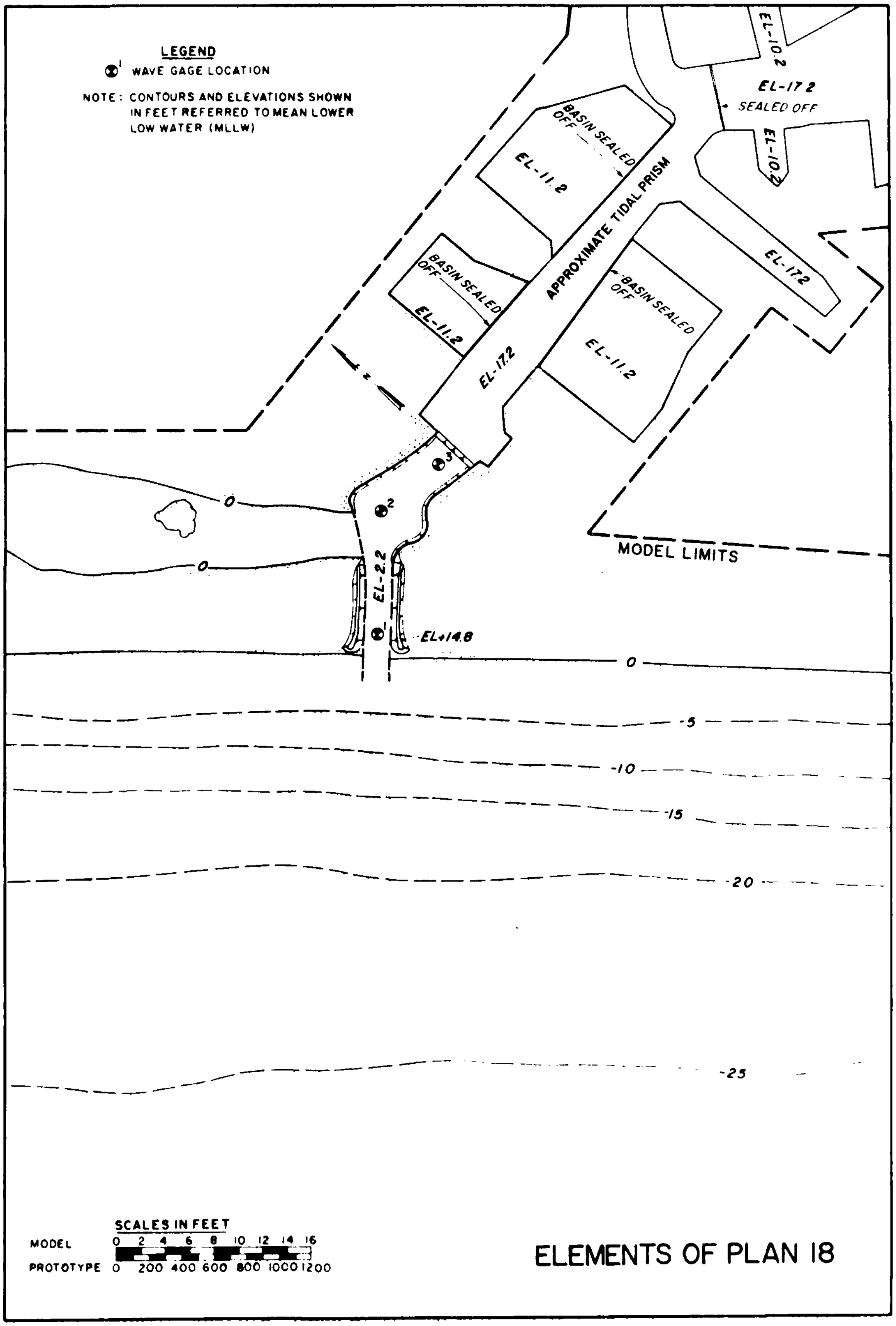

LEGEND
WAVE GAGE LOCATION
NOTE: CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
BASIN SEALED OFF
EL-11.2
APPROXIMATE TIDAL PRISM
EL-17.2
EL-10.2
SEALED OFF
EL-2.2
EL+14.8
MODEL LIMITS
0
-5
-10
-15
-20
-25
SCALES IN FEET
MODEL 0 2 4 6 8 10 12 14 16
PROTOTYPE 0 200 400 600 800 1000 1200
ELEMENTS OF PLAN 18

PLATE 10