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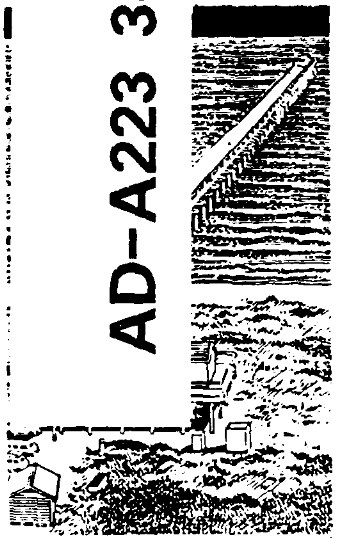

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TECHNICAL REPORT CERC-90-6

REDONDO BEACH KING HARBOR, CALIFORNIA, DEVELOPMENT OF DESIGN DATA FOR HARBOR IMPROVEMENTS

Coastal Model Investigation

by

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Final Report

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A 1:75-scale three-dimensional hydraulic model was used to investigate the design of proposed modifications in various areas of Redondo Beach King Harbor. The model (originally used to develop plans for wave protection in the southern portion of the harbor in the lee of a low-crested breakwater section) reproduced approximately 8,800 ft of the California shoreline and included the existing harbor and offshore bathymetry in the Pacific Ocean to a depth of -60 ft. Improvements consisted of raising portions of the north and south breakwaters, flattening the slope of the existing north breakwater in the vicinity of an adjacent mole, and installing a spur on the inside of the north breakwater. An 80-ft-long unidirectional, spectral wave generator and an automated data acquisition and control system were utilized in model operation. It was concluded from test results that:

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19. ABSTRACT (Continued).

a. For test waves from 260 deg, test results for existing conditions indicated severe overtopping of the breakwater adjacent of Mole A and subsequent flooding of the mole. The proposed improvement plan (Plan 1) with additional stone placed on a 100-ft-long section at the outer end of the Galveston seawall (Plan 1A) will minimize overtopping of the breakwater and flooding of the mole.

b. For test waves from 240 deg, the proposed improvements (Plan 2) required modification to minimize overtopping of the breakwater and subsequent flooding of Mole A. Additional stone placed on a 150-ft-long section of the outer end of the Galveston seawall (Plan 2A) was required. The Plan 2 spur, it appeared, could be reduced in elevation (Plan 2A) and minimize wave energy reaching Mole A due to spilling waves propagating northerly over the breakwater.

c. For test waves from 220 deg, existing conditions revealed severe overtopping of the south breakwater and Mole D with subsequent flooding of the mole and adverse wave conditions in Basin 3. The proposed improvement plans (Plans 3 and 3A) reduced wave heights in the Mole D/Basin 3 vicinity; however, overtopping of the south breakwater and Mole D still occurred, only not to as great a degree. Data obtained should aid in the design of structures proposed along the waterfront in the Mole D/Basin 3 area.

PREFACE

Use of an existing model of Redondo Beach King Harbor to investigate wave conditions in various portions of the harbor was authorized by a letter agreement between the US Army Engineer Waterways Experiment Station (WES) and the City of Redondo Beach, California, dated 15 November 1989.

The Redondo Beach King Harbor model was initially constructed and tested for the US Army Engineer District, Los Angeles (SPL), during the period February through August 1989 and reported in Technical Report CERC-90-4, "Redondo Beach King Harbor, California, Design for Wave Protection; Coastal Model Investigation," dated April 1990. The Corps-sponsored investigation involved providing wave protection principally in the southern portion of the harbor in the lee of a low-crested breakwater section. The test results, reported herein, involved the acquisition of design wave data for protective structures located at Mole D, near the entrance to the harbor, and testing of a proposed protective system at Mole A, in the northern portion of the harbor complex.

Model testing was conducted at WES during the period January through February 1990 by personnel of the Wave Dynamics Division (WDD), Coastal Engineering Research Center (CERC), under the general direction of Dr. James R. Houston, Chief of CERC; Mr. Charles C. Calhoun, Jr., Assistant Chief of CERC; and Mr. Claude E. Chatham, Jr., Chief of WDD. The tests were conducted by Mr. Marvin G. Mize, under the direct supervision of Mr. Robert R. Bottin, Jr., Wave Processes Branch, WDD. Dr. Richard E. Kent, consultant to the City of Redondo Beach, visited WES and was present during most of the testing. This report was prepared by Mr. Bottin and Dr. Kent and typed by Ms. Debbie S. Fulcher, WDD. Ms. Sheila Schoettger, Harbor Director, had authority to act under this agreement for the City of Redondo Beach, and Mr. Bottin, for WES.

COL Larry B. Fulton, EN, was Commander and Director of WES during model testing and the preparation and publication of this report. Dr. Robert W. Whalin was Technical Director.

CONTENTS

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

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

Multiply	By	To Obtain
acres	4,046.856	square metres
degrees (angle)	0.01745329	radians
feet	0.3048	metres
inches	25.4	millimetres
miles (US statute)	1.609347	kilometres
pounds (mass)	0.4535924	kilograms
square feet	0.09290304	square metres
square miles (US statute)	2.589988	square kilometres
tons (2,000 pounds mass)	907.1847	kilograms

REDONDO BEACH KING HARBOR, CALIFORNIA, DEVELOPMENT OF DESIGN DATA FOR HARBOR IMPROVEMENTS

Coastal Model Investigation

PART I: INTRODUCTION

The Prototype

1. Redondo Beach King Harbor (formerly Redondo Beach Harbor), California, is a small craft harbor located on the Pacific coast at the southern end of Santa Monica Bay (Figure 1). It lies within the City of Redondo Beach, about 17 miles* southwest of the business center of Los Angeles. The harbor is entirely man-made and serves as a port of call for visiting craft from the entire Pacific coast. Commercial, recreational, and sport fishing vessels, and boats for hire serve local residents and tourists from throughout the Nation. The harbor is situated near productive fishing areas favorable to both sport and commercial fishing. It consists of about 55 acres of land and 112 acres of water. The harbor provides about 1,600 boat slips in three basins with a 77-acre mooring anchorage area. The commercial and recreational facilities at Redondo Beach King Harbor attract approximately 8,000,000 visitors annually (US Army Engineer District (USAED), Los Angeles 1988).

2. Development of the harbor started in 1937 when a 1,470-ft-long stone breakwater was constructed. The harbor has undergone several modifications, improvements, repairs, etc., since initial construction (USAED, Los Angeles 1988; Bottin 1988) and currently consists of two permeable rubble-mound breakwaters that total 4,885 ft in length, three boat basins enclosed by moles, an entrance channel, and boat mooring area. An aerial photograph of the harbor is shown in Figure 2.

3. The south breakwater is 600 ft long and has an authorized crest elevation (el) of +12 ft.** The north breakwater is 4,285 ft long and has an authorized crest el of +14 ft for its outer 1,600 ft (sta 36+00 - 52+00), and +22 ft between sta 15+50 and 36+00. Actual elevations for the two sections

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

** All elevations (el) cited herein are in feet referred to mean lower low water (mllw).

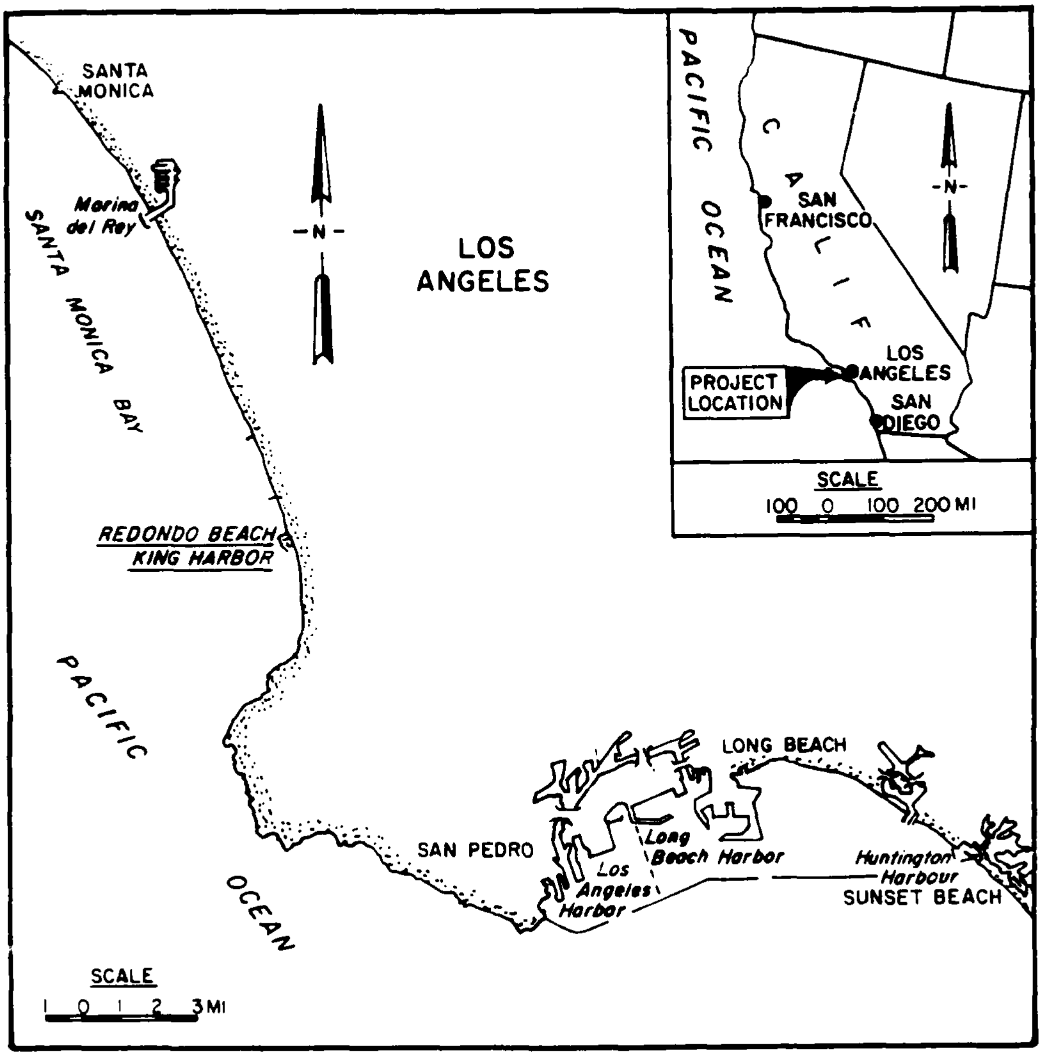

Figure 1. Project location

average approximately +16 and +20 ft, respectively. The shoreward end of the north breakwater has a rubble-mound section (el +14 ft) with a concrete Galveston seawall (el +20 ft). Wave protection baffles to the two northernmost basins (Basins 1 and 2) also have been constructed by the Federal government. Maintenance of the breakwaters is a Federal responsibility, whereas, the City of Redondo Beach is responsible for maintenance of the wave protection baffles and the concrete Galveston seawall.

4. The City of Redondo Beach constructed and maintains the interior harbor, which consists of three boat basins enclosed by moles, all with revetted slopes. The harbor entrance is formed by a 600-ft-wide opening

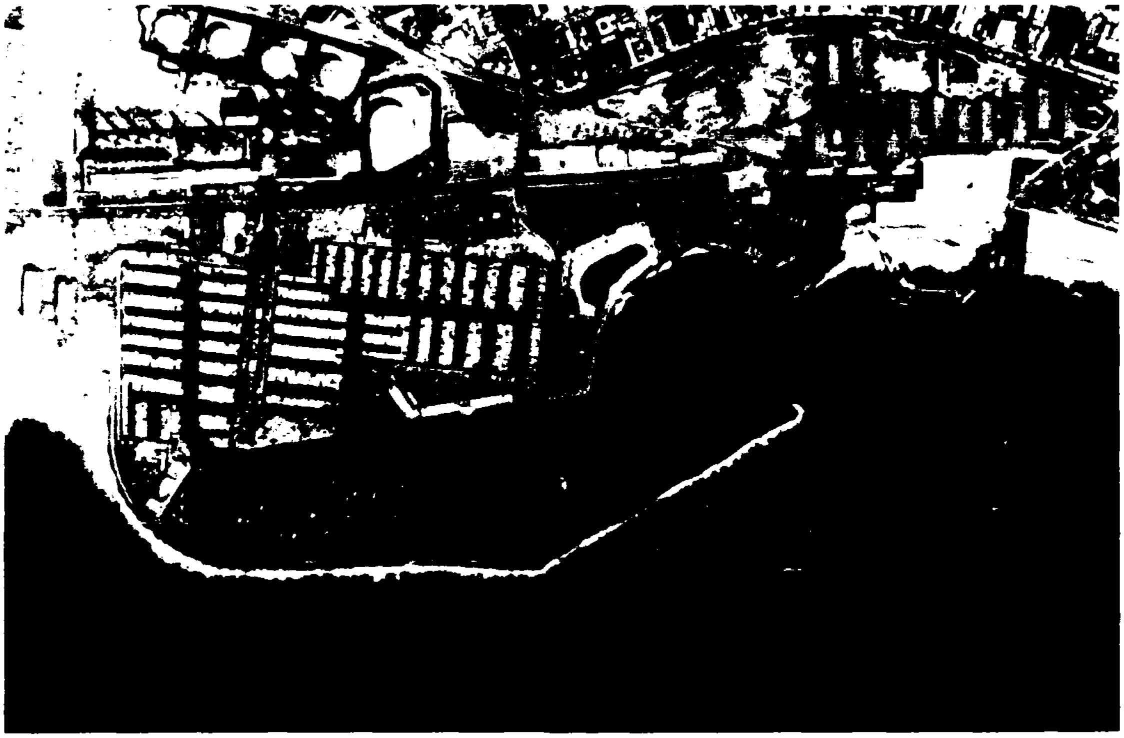

Figure 2. Aerial view of Redondo beach King Harbor

between the breakwaters for small craft navigation. Natural depths through the entrance vary from -34 to -40 ft.

The Problem

5. Frequently, Redondo Beach King Harbor is susceptible to damage when large winter storm waves occur in conjunction with high-water levels. The low-crested portion of the north breakwater is not adequate to dissipate wave energy for these storm events. The energy of overtopping waves, waves passing through the harbor entrance, and wave transmission through the rubble-mound structures result in adverse wave conditions in the harbor. Waves run up the revetment along the moles and result in revetment damage, land erosion, flooding, and structural failure of facilities bordering the water. Some of these facilities include hotels, restaurants, recreational complexes, and public and commercial buildings. Wave energy also passes through the mooring area and into the boat basins, causing damage to boat hulls, mooring lines, and docking and launching facilities. Because of the frequency of these conditions, the City of Redondo Beach has been unable to increase mooring

space in the lee of the low-crested north breakwater. Waves also overtop the higher section of the breakwater during extreme storms and high tides; however, much of this energy is lost, and damage behind this portion is significantly less than the storm damage that occurs behind the low-crested breakwater segment. These adverse conditions make Redondo Beach King Harbor an unsafe port of refuge during times of high tides and large storm waves.

6. Storm damage potential ranges from damage to revetment and from flooding that occurs annually to catastrophic damage from storms having estimated recurrence intervals of 50 to 100 years. Average annual damage cost at the harbor is estimated at $962,300, while costs associated with a 100-year event are estimated to total $10,600,000 (USAED, Los Angeles 1988). The most damaging storm to date at Redondo Beach King Harbor occurred in January 1988 with damage estimates of $14,000,000. Some of this damage included destruction of substantial portions of three buildings; undermining of significant portions of revetment along the moles; sinking of six boats; damage to many other boats and piers; erosion of substantial land along the moles; damage to public parking areas, utilities, and fencing; and the loss of fueling facilities.

Corps-Sponsored Investigation and Conclusions

7. The Redondo Beach King Harbor model was initially constructed and tested for the USAED, Los Angeles, to investigate wave conditions in the southern portion of the harbor in the lee of the outer low-crested north breakwater and the south breakwater. Improvement plans consisted of raising the crest elevation of portions of the north breakwater both with and without installing a transition layer of small stone and extending the length and increasing the crest elevation of the south breakwater. Details of the investigation have been published (Bottin and Mize 1990). Conclusions derived from results of these tests are shown in the following section. Plan numbers in the following subparagraphs refer to the previous investigation.

a. Existing conditions are characterized by very rough and turbulent wave conditions with wave heights up to 8 ft along the moles for 50-year conditions.

b. Of the original improvement plans tested with the seaward wing of the north breakwater raised to an elevation of +20 ft (Plans 1-7), Plan 6 provided the greatest wave protection within the harbor. Wave heights along the moles exceeded the criteria,

however, by 1.0 ft for 50-year conditions.

c. Of the improvement plans tested with portions of the north breakwater raised to elevations of +24 and +20 ft (Plans 8-10), Plan 9 provided the greatest wave protection within the harbor, but wave heights exceeded the criteria along the moles by 0.7 ft for 50-year wave conditions.

d. Of the improvement plans tested with the seaward wing of the north breakwater sealed with small stone and raised to an elevation of +20 ft (Plans 10-14), Plan 12 provided the greatest degree of wave protection to the harbor. For 50-year wave conditions, wave heights met the established wave-height criterion along the moles within the harbor.

e. Of all the improvement plans tested (Plans 1-14), Plan 14 was optimal, considering wave protection and construction costs.

f. Comprehensive wave-height tests conducted for Plan 14 indicated that the established wave-height criteria in the harbor would be met or only slightly exceeded for waves up to a 100-year recurrence from 240 and 260 deg. Waves in excess of 10 ft in height from 220 deg, however, in some cases, will significantly exceed the criteria, particularly at Mole D and the entrance to Basin 3.

Purpose of the Current Investigation

8. At the request of the City of Redondo Beach, the hydraulic model of Redondo Beach King Harbor was used by the US Army Engineer Waterways Experiment Station's (WES's) Coastal Engineering Research Center (CERC) to (a) determine wave conditions in the existing northern portion of the harbor in the vicinity of Mole A for test waves approaching from 260 and 240 deg; (b) determine the adequacy of proposed improvement plans with regard to storm wave protection levels and develop remedial plans, if necessary, for the alleviation of undesirable wave conditions in the vicinity of Mole A; and (c) determine wave conditions at Mole D and in Basin 3 in the southern portion of the harbor for test waves approaching from 220 deg.

PART II: THE MODEL

Design of Model

9. The Redondo Beach King Harbor model (Figure 3) 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 wave and current patterns. 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 follow:

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	$\forall_r = L_r^3 = 1:421,875$
Time	T	$T_r = L_r^{1/2} = 1:8.66$
Velocity	L/T	$V_r = L_r^{1/2} = 1:8.66$

*Dimensions are in terms of length and time.

10. The existing breakwaters and revetments at Redondo Beach King Harbor, as well as proposed improvements, 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

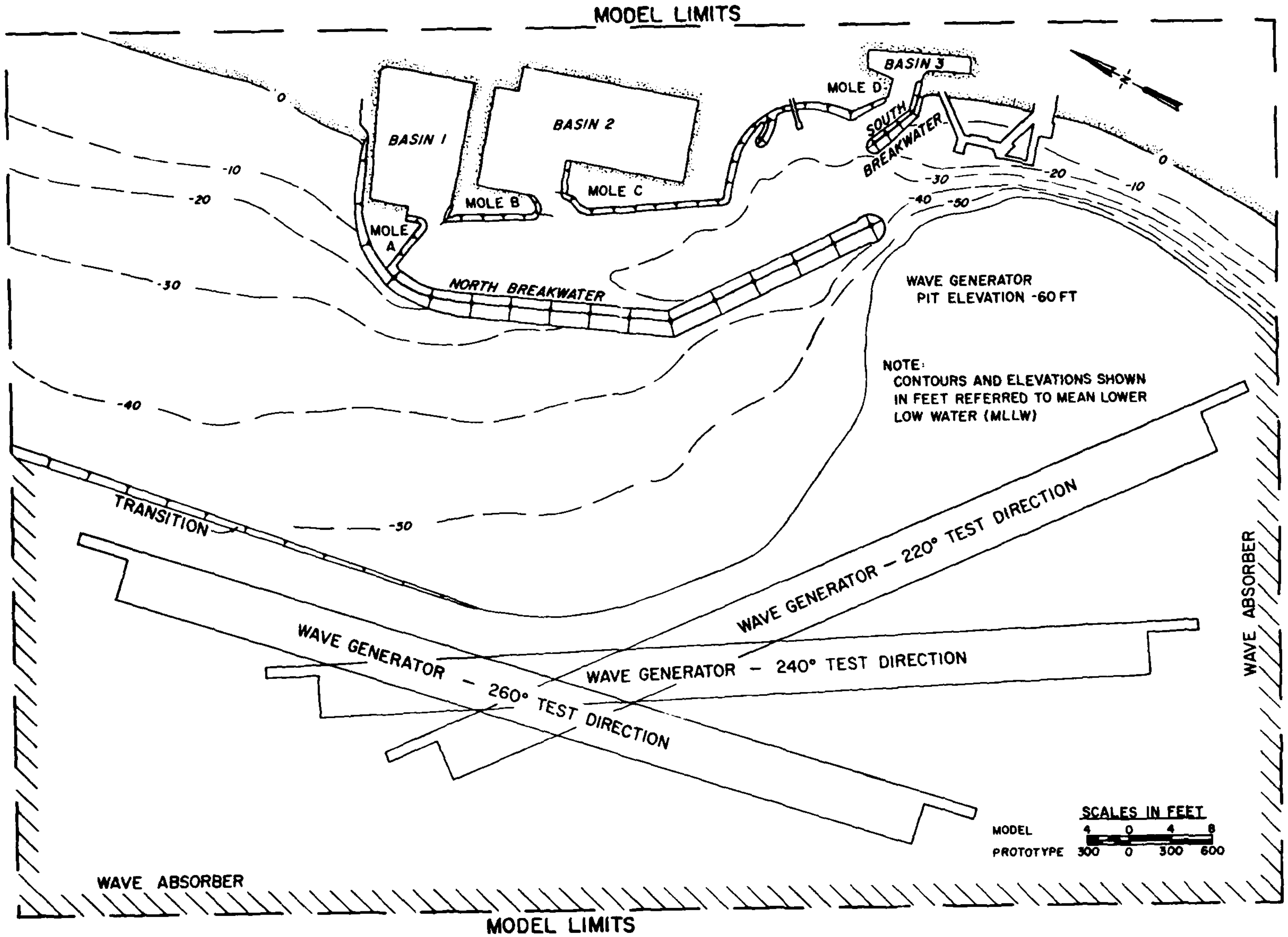

Figure 3. Model layout

structures reflect relatively more and absorb or dissipate relatively less wave energy than geometrically similar prototype structures (Le Méhauté 1965). Also, the transmission of wave energy through a 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 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 Redondo Beach, it was determined that a close approximation of the correct wave-energy transmission characteristics could be obtained by increasing the size of the rock 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 Redondo Beach King Harbor model, the rock sizes were computed linearly by scale and then multiplied by 1.5 to determine the actual sizes to be used in the model.

The Model and Appurtenances

11. The model reproduced about 8,800 ft of the California shoreline and included the harbor and underwater topography in the Pacific Ocean to an offshore depth of 60 ft. The total area reproduced in the model was approximately 10,300 sq ft, representing about 2.1 square miles in the prototype. A general view of the model is shown in Figure 4. Vertical control for model construction was based on mean lower low water. Horizontal control was referenced to a local prototype grid system.

12. 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 was controlled by a computer-generated command signal, and the movement of the plunger caused a periodic displacement of water that generated the required test waves. The wave

Figure 4. General view of model

generator also was mounted on retractable casters that enabled it to be positioned to generate waves from the required directions.

13. An automated data acquisition and control system (ADACS), designed and constructed at WES (Figure 5), was used to generate and transmit control signals, monitor wave generator feedback, and secure and analyze wave-height data at selected locations in the model. Basically, through the use of a Vax 750 computer, ADACS recorded onto magnetic disks the electrical output of parallel-wire, resistance-type wave gages that measured the change in water-surface elevation with respect to time. The magnetic disk output of ADACS then was analyzed to obtain the wave-height data.

14. A 2-ft (horizontal) solid layer of fiber wave absorber was placed around the inside perimeter of the model to dampen any 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.

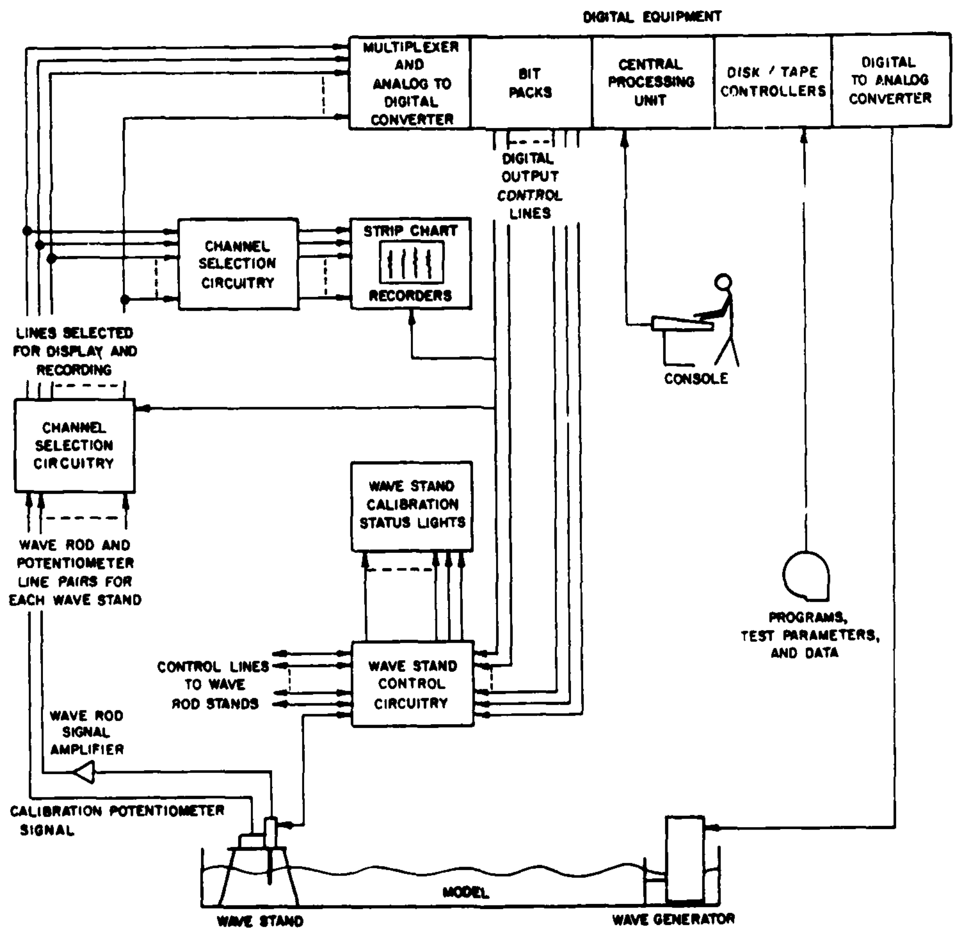

Figure 5. Automated data acquisition and control system

PART III: TEST CONDITIONS AND PROCEDURES

Selection of Test Conditions

Still-water level

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

16. In most cases, it is desirable to select a model swl that closely approximates the higher water stages that normally occur in the prototype for the following reasons:

- a. The maximum amount of wave energy reaching a coastal area normally occurs during the higher water phase of the local tidal cycle.
- b. Most storms moving onshore are characteristically accompanied by a higher water level due to wind-induced mass transport, atmospheric pressure fluctuations, and wave setup.
- c. The selection of a high swl helps minimize model scale effects due to viscous bottom friction.
- d. When a high swl is selected, a model investigation tends to yield more conservative results.

17. Based on a review of 63 years of tide data from a gage located in Los Angeles Harbor, the annual and the 100-year-return probability water levels at the site are +7.0 and +8.0 ft, respectively (USAED, Los Angeles 1988). Extreme water-level predictions for Redondo Beach King Harbor are shown below. The data include periods of storm activity when water level was elevated above the astronomical level due to surge components.

Return Period years	Water el ft above mllw
100	8.0
50	7.9
25	7.8
10	7.6
1	7.0

An swl of +7.0 ft was selected by the city of Redondo Beach for use during model testing for existing conditions and improvement plans.

Factors influencing selection
of test wave characteristics

18. 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. 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 interactions 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. 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.

Wave refraction

19. 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. The change in wave height and direction may be determined by using the numerical Regional Coastal Processes Wave Transformation Model (RCPWAVE) developed by Ebersole (1985). This model predicts the transformation of monochromatic waves over complex bathymetry and includes refractive and diffractive effects. Diffraction becomes increasingly important in regions with complex bathymetry. Finite difference approximations are used to solve the governing equations, and the solution is obtained for a finite

number of grid cells that comprise the domain of interest. Much of the early work in this area during the 1950s was based on wave ray methods and manual construction of refraction diagrams using linear, gravity wave theory. During the 1960s and early 1970s, the linear wave refraction problem was solved in a more efficient way through the use of the digital computer. All of these methods, however, addressed the refraction problem only.

20. The solution technique employed by RCPWAVE is a finite difference approach; thus, the wave climate in terms of wave height, H, wave period, T, and wave direction-of-approach, θ, is available at a large number of computational points throughout the region of interest, and not just along wave rays. Computationally, the model is very efficient for modeling large areas of coastline subjected to widely varying wave conditions and, therefore, is an extremely useful tool in the solution of many types of coastal engineering problems.

21. When the refraction coefficient (K_r) is determined, it is multiplied by the shoaling coefficient (K_s) and gives a conversion factor for transfer of deepwater wave heights to shallow-water values. The shoaling coefficient, a function of wavelength and water depth, can be obtained from the Shore Protection Manual (1984).

22. During the past several years, several wave refraction/diffraction/shoaling analyses have been conducted to establish the local storm wave climate at Redondo Beach King Harbor. Several approaches have been used, among other methods, those originated by Munk and Traylor (1947), Longuet-Higgins (1957), and Dalrymple (1988). Because the bathymetry is so complex offshore of the harbor, in particular at the submarine canyon, the various wave modification results are not always concordant. Basically, the data are in reasonable agreement until the canyon is closely impinged. For example, an extensive modification analysis conducted by Hales (1987) indicates convergence of wave energy along the north breakwater proper with marked divergence at the entrance. O'Reilly (1989), using the Dalrymple approach, found considerable variation over the entire perimeter with areas of convergence at or near the entrance. Strange* found only moderate divergence at the entrance. Even though the wave entry windows vary in azimuth, it is possible,

* Personal Communication, 1988, R. R. Strange, Pacific Weather Analysis Corporation, Santa Barbara, California.

given the correct wave periods, for deepwater waves from as far north as 280 deg to refract into the harbor entrance at about 235 deg. Deepwater waves from 240 deg may refract to 220 deg, and deepwater waves from 230 deg may refract to less than 215 deg (Hales 1987). In addition, wave diffraction at the entrance is such that severe storm waves from deepwater approach angles in the 240- to 270-deg sector will produce diffracted energy inside the entrance approaching from 220 deg. Thus, wave energy propagating directly into the harbor entrance is not an uncommon condition. In general, however, storm waves seaward of the entrance are lower than those impinging on the north breakwater, including the segment extending northerly from the dogleg to the Galveston seawall section adjacent to Mole A.

Prototype storm wave data

23. Deepwater storm waves generated by anti-cyclones in the North Pacific approach the outer continental shelf of the southern California coast from the northwest through west-southwest directions. Moderately high waves generated by hurricanes and Southern Hemisphere disturbances occasionally approach from the southwesterly and southerly quadrants (USAED, Los Angeles 1988). However, due to the shadow effects of the offshore Channel Islands, storm wave exposure for Redondo Beach King Harbor is limited to energy propagated eastward through three windows bounded by azimuths (a) 205 through 235 deg, (b) 240 through 272 deg, and (c) 283 through 290 deg (Figure 6).

24. As seen in Figure 6, the 240- through 272-deg window is the largest of the three; consequently, the most severe storm waves at Redondo Beach usually approach from this sector. However, during prefrontal and offshore stationary low conditions, fairly large waves can approach from the 205- through 235-deg sector. Also, even though the protective shadows of Santa Rosa and Santa Cruz Islands constrain northwesterly wave energy to a narrow approach window (283 through 290 deg), very strong postfrontal winds blowing down the Santa Barbara Channel produce moderately high, relatively short-period waves that occur simultaneously with westerly swell conditions, the sum effect of which causes overtopping of the northwest portion of the Redondo breakwater (Mole A). Waves from all three of these windows are modeled in this investigation in terms of their impacts on Mole A, Mole D, and Basin 3.

25. Deepwater unsheltered storm events occurring in southern California waters since 1900 have been analyzed by Moffatt and Nichol (1983), Seymour et al. (1984), and Walker et al. (1984). In addition, statistically analyzed

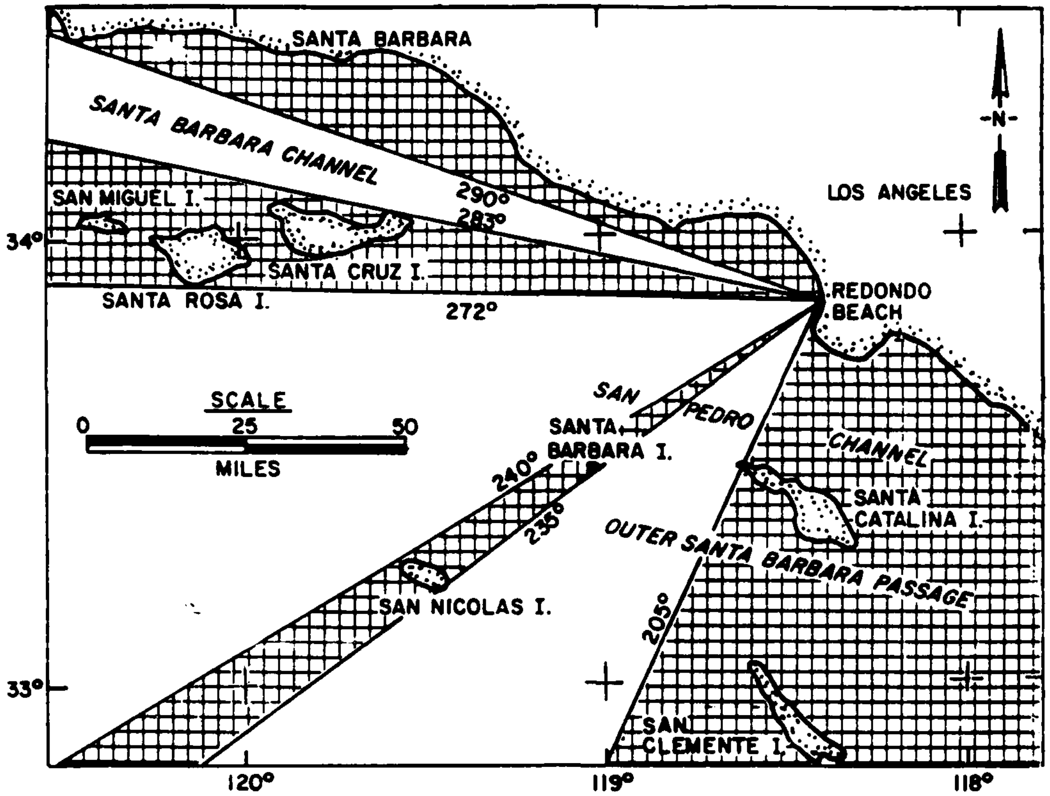

Figure 6. Redondo Beach King Harbor storm wave exposure windows

hindcast results that provide annual sea and swell wave heights at intermediate water depths along the coast of southern California are available in the Sea-State Engineering Analysis System (SEAS) of the Corps of Engineers (Ragsdale 1983). From these data, unsheltered deepwater storm events may be summarized. However, as stated previously, since Redondo Beach King Harbor is sheltered by the offshore islands, waves from various directions of approach are blocked. This blocking action depends on both water depth and wave period, with long-period waves requiring deeper water for passage than short-period waves. With the aid of precise bottom contour charts, all such avenues of approach were determined for Redondo Beach using a numerical program developed by USAED, Los Angeles. The results of these integrations provided sheltered storm wave characteristics on the shoreward side of the islands but still in deep water. Table 1 provides unsheltered deepwater wave characteristics and approach azimuths as well as island sheltering coefficients and sheltered deepwater wave characteristics and approach angles seaward of the harbor for various storm events. More detailed information on the island sheltering theory may be obtained from Hales (1987). These sheltered deepwater storm events still must be propagated to the harbor over the complex

nearshore bathymetry and the Redondo Submarine Canyon.

Selection of test waves

26. Based on all data available, wave conditions in the Mole A and Mole D/Basin 3 areas of the harbor were estimated by the City of Redondo Beach. The following sheltered wave parameter were selected for testing at various locations for the several harbor modifications.

Mole A

Direction deg	Wave Period sec	Approximate Wave Height Seaward of Breakwater at Mole A, ft	Estimated Recurrence Interval, year
260	8	8.0	1-5
		10.0	10<25
	10	8.0	10<25
		10.0	10<25
		12.0	25
	12	10.0	25
		12.0	25
		14.0	50
	14	10.0	50
		12.0	50
		15.5	50+

Direction deg	Wave Period sec	Approximate Wave Height at Wave Generator Location, ft	Estimated Recurrence Interval, year
240	15	12.5	10
		15.0	25
		16.5	50
		18.5	100

Mole D/Basin 3

Direction deg	Wave Period sec	Approximate Wave Height at Harbor Entrance, ft	Estimated Recurrence Interval, year
220	8	8	5<10
	10	8.5	10<25
	12	11.5	25<50
	15	9	25<50
		11.5	50
		13.0	100

27. Unidirectional wave spectra for most of the selected test waves were generated (based on JONSWAP parameters) and used throughout the model investigation. Plots of typical wave spectra are shown in Figure 7. The dashed line represents the desired spectra while the solid line represents the spectra generated by the wave machine. A typical wave train time-history plot, which depicts water-surface elevation (η) versus time is shown in Figure 8. Due to limitations of the model wave generator, some wave conditions used in the study were monochromatic (i.e., constant wave height and period). Monochromatic wave conditions were generated for the 15-sec, 16.5- and 18.5-ft wave characteristics.

Analysis of Model Data

28. Relative merits of the various plans tested were evaluated by:

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

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

In the wave-height data analysis, the average height of the highest one-third of the waves (H_s) recorded at each gage location was computed. 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.* 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.

* G. H. Keulegan, 1950, "The Gradual Damping of a Progressive Oscillaroty Wave with Distance in a Prismatic Rectangular Channel," unpublished data, prepared by National Bureau of Standards, Washington, DC, at the request of the Director, WES, Vicksburg, MS, by letter of 2 May 1950.

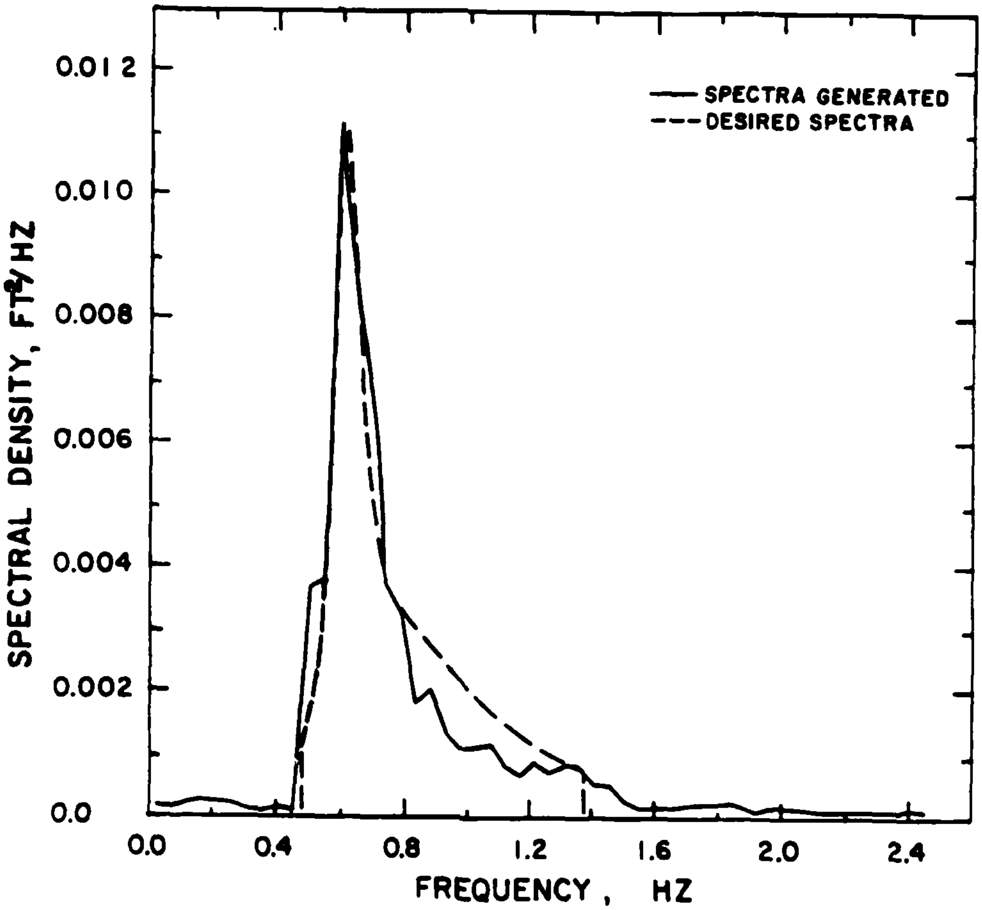

Figure 7. Typical wave spectra plot

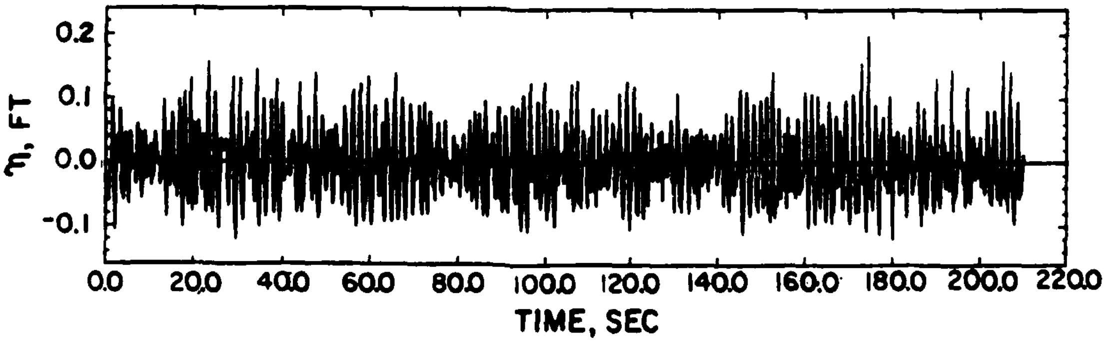

Figure 8. Typical wave train time-history
(14-sec, 15.5-fot test waves)

PART IV: TESTS AND RESULTS

The Tests

Existing conditions

29. Prior to testing of the various improvement plans, tests were conducted for existing conditions (Plate 1) to establish a base from which to evaluate the effectiveness of the plans. Wave height data were secured at various locations throughout the harbor for the selected test waves from 260 and 220 deg. In addition, wave pattern photographs and videotape footage were obtained for representative test waves from three test directions.

Improvement plans

30. Wave heights were secured for three test plan configurations, and wave pattern photographs and videotape footage were secured for several test plans. Variations entailed changes to the north breakwater in the vicinity of Mole A and modifications to the south breakwater. Brief descriptions of the improvement plans are presented in the following subparagraphs; dimensional details are presented in Plates 2-4.

a. Plan 1 (Plate 2) consisted of raising a 200-ft-long portion of the north breakwater from +21 to +27 ft. The raised portion of the breakwater originated at the south end of Mole A and extended southerly. The structure was raised by placing 16-ton stone on the top of the breakwater, and the seaward slope was increased in thickness by 10 ft by the placement of 16-ton stone. The existing slope on the sea side of Mole A was flattened by the installation of 6- to 10-ton stone on a 1V:6.5H slope from an elevation of +12 to +7 ft. From the +7.0 ft el to the existing bottom, the slope then changed to 1V:1.5H. From elevations of +5 to -5 ft, 1,500-lb seal stone was placed adjacent to the existing structure. This layer was 4 ft thick and was covered by the 6- to 10-ton armor.

b. Plan 1A (Plate 2) entailed the elements of Plan 1, but 16-ton stone was placed on the flattened slope adjacent to the seaward 100 ft of the Galveston seawall. The elevation of the stone sloped from +27 ft at the outer end of the seawall to +20 ft at a point 100 ft shoreward.

c. Plan 2 (Plate 3) included the elements of Plan 1A, but a 75-ft-long spur was installed that originated at the outer end of the +27 ft breakwater section and extended into the inner harbor perpendicular to the structure. The spur had a crest elevation of +27 ft, 1V:1.5H side slopes, and a 10-ft crest width. It was constructed with 6- to 10-ton stone.

d. Plan 2A (Plate 3) involved the elements of Plan 2, but the elevation of the spur decreased from +27 ft at its junction with the breakwater to +12 ft at its head. The stone placed on the slope adjacent to the Galveston seawall was increased in elevation to +27 ft and extended to a point 150 ft shoreward of the outer end of the seawall.

e. Plan 3 (Plate 4) consisted of a 150-ft-long south breakwater extension that had a crest elevation of +12 ft and 1V:2H and 1V:1.25H side slopes on the sea and shore sides, respectively. A 300-ft-long portion of the existing south breakwater was also raised to an elevation of +16 ft. The raised section of the breakwater extended 125 ft shoreward and 175 ft seaward from the dogleg in the south structure. Stones were placed on top of the breakwater and along the seaward face of the structure. The breakwater extension and raised section utilized stones ranging from 5 to 13 tons.

f. Plan 3A (Plate 4) included the elements of Plan 3, but 75 ft of the south breakwater extension was removed, resulting in a 75-ft-long extension.

Wave-height tests

31. Wave-height tests were conducted for test waves from 260 and 220 deg. Improvement plans involving modifications at Mole A were evaluated with wave conditions from 260 deg, and waves from 220 deg were used to evaluate the proposed plans at Mole D and Basin 3. Wave gage locations for the improvement plans are shown in Plates 2 and 4.

Wave patterns and videotape footage

32. Wave pattern photographs and videotape footage were obtained in the model for representative test waves for various improvement plans from all three test directions. This documentation of test results was furnished to the City of Redondo Beach.

Test Results

33. In evaluating test results, the relative merits of the various plans were based on visual observations and measured wave-height data in the harbor. Model wave heights (significant wave height, H_s) were tabulated to show measured values at selected locations.

Existing conditions

34. Results of wave-height tests conducted for existing conditions are presented in Table 2 for test waves from 260 deg. Maximum wave heights were 1.8 ft at Mole A (Gage 4) and 3.0 ft in the harbor seaward of Mole B (Gage 5).

Visual observations, however, indicated significant overtopping of the breakwater with extensive flooding of Mole A. These conditions occurred most severely for wave conditions with 25- to 50-year recurrence intervals. Typical wave patterns obtained for existing conditions for test waves from 260 deg are shown in Photos 1-3.

35. Results of wave-height tests for existing conditions are presented in Table 3 for test waves from 220 deg. For estimated 50-year wave conditions, maximum wave heights were 7.9 ft at Mole D (Gage 10), 4.3 ft at the entrance to Basin 3 (Gage 12), and 2.7 ft in the southern end of Basin 3 (Gage 14). Visual observations indicated overtopping of the south breakwater and flooding of Mole D for test waves with recurrence intervals ranging from 10 to 100 years. More significant overtopping and flooding occurred with the more severe test conditions (those with the greater recurrence intervals). Typical wave pattern photos for existing conditions for test waves from 220 deg are presented in Photos 4-6.

Improvement plans

36. Visual observations of test waves from 260 deg with Plan 1 installed indicated that the raised breakwater section and flattened slope were very effective in preventing overtopping and subsequent flooding of Mole A. For 50-year conditions, however, slight overtopping occurred at the southern end of the Galveston seawall. The installation of 100 ft of stone at this location (Plan 1A) revealed substantial improvement, and results obtained were excellent. Wave-height data obtained for Plan 1A are presented in Table 4 for test waves from 260 deg. Maximum wave heights were 1.7 ft at Mole A (Gage 4) and 2.9 ft in the harbor seaward of Mole B (Gage 5). Typical wave patterns obtained for Plan 1A are presented in Photos 7-9.

37. Visual observations with Plan 2 installed, for test waves from 240 deg, indicated the raised breakwater and flattened slopes were effective for 10- to 25-year wave conditions. For 50-year wave conditions, however, excessive overtopping occurred at the southern end of the Galveston seawall, which resulted in flooding of Mole A. A convergence of wave energy occurred at this location. The spur appeared to reduce energy along the south perimeter of Mole A; however, its crest elevation appeared to be excessive. The installation of 150 ft of stone at the southern end of the Galveston seawall and the spur configuration of Plan 2A resulted in a very effective plan of improvement for 50-year wave conditions. Only slight splashover occurred

along the revised portion of the Galveston seawall. When subjected to 100-year wave conditions, the plan was also very effective. Essentially, these waves broke seaward of the breakwater and were less severe than the 50-year waves that converged on the structure. Typical wave patterns obtained for Plan 2A for test waves from 240 deg are shown in Photo 10.

38. Wave heights obtained for Plans 3 and 3A for test waves from 220 deg are presented in Tables 5 and 6, respectively. For 50-year wave conditions, maximum wave heights were 7.5 ft at Mole D (Gage 10); 2.5 ft at the entrance to Basin 3 (Gage 12), and 2.2 ft in the southern end of Basin 3 (Gage 14) for Plan 3. With Plan 3A installed, maximum wave heights for 50-year wave conditions were 7.4 ft at Mole D; 3.2 ft at the entrance to Basin 3; and 2.2 ft in the southern end of Basin 3. Overall conditions throughout the Mole D/Basin 3 area were improved by the test plans considering all test waves. Visual observations, however, indicated overtopping and flooding of portions of Mole D for waves with recurrence intervals ranging from 25 to 100 years. These adverse wave conditions, however, were less severe than those for existing conditions. Plan 3 resulted in slightly less severe conditions than Plan 3A. Typical wave patterns obtained for Plans 3 and 3A are shown in Photos 11-15.

Discussion of test results

39. Test results for existing conditions revealed significant overtopping of the breakwater in the vicinity of Mole A for test waves from 260 deg and subsequent flooding of the mole. The raised breakwater section and the flattened slope seaward of the mole (Plan 1) prevented overtopping with the exception of a 100-ft-long section at the seaward end of the Galveston seawall. By installing additional stone in this area (Plan 1A), overtopping of the structure and flooding of the mole were minimized. Wave-height data indicated that wave heights were only slightly reduced in the outer harbor in the vicinity of Moles A and B for the improvement plans, but damage to Mole A from overtopping of the breakwater (based on visual observations) should be drastically reduced for the improvements.

40. Test results for 240 deg indicated that the installation of additional stone over a 150-ft section at the southern end of the Galveston seawall would minimize overtopping of the structure and subsequent flooding of Mole A for 50-year wave conditions. These 50-year waves appeared to be the worst case since they converged and broke on the structure at this location.

Waves with a 100-year recurrence interval broke seaward of the breakwater and expended most of their energy before getting to the structure. The spur appeared to reduce the severity of conditions along the inner portion of Mole A for 50- and 100-year waves that spilled over the breakwater and progressed to the north. The +27 ft crest el of Plan 2, however, was excessive, and the variable crest elevation of Plan 2A (+27 to +12 ft) was adequate to achieve the desired results. Considering the elements of Plan 2A, however, the additional 150 ft of stone along the Galveston seawall was far more effective than the spur, based on visual observations. In the model, stone was placed adjacent to the seawall to an elevation of +27 ft for a 150-ft distance for Plan 2A. Caution should be exercised prior to the actual placement of these stones in the prototype to ensure structural stability.

41. Test results for existing conditions for test waves from 220 deg revealed significant overtopping of the south breakwater and Mole D, and subsequent flooding of Mole D. The installation of improvement Plans 3 and 3A, in general, reduced wave heights throughout this region; however, overtopping of the breakwater and Mole D still occurred, although not to as great an extent. Plan 3 resulted in less severe wave conditions than Plan 3A. The model provided an excellent data set in the Mole D/Basin 3 area for the design of proposed structures adjacent to Mole D and Basin 3.

PART V: CONCLUSIONS

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

<u>a</u>. For test waves from 260 deg, results for existing conditions indicated severe overtopping of the breakwater adjacent to Mole A and subsequent flooding of the mole. The proposed improvement plan (Plan 1) with additional stone placed on a 100-ft-long section at the outer end of the Galveston seawall (Plan 1A) will minimize overtopping of the breakwater and flooding of the mole.

<u>b</u>. For test waves from 240 deg, the proposed improvements (Plan 2) required modification to minimize overtopping of the breakwater and subsequent flooding of Mole A. Additional stone placed on a 150-ft-long section of the outer end of the Galveston seawall (Plan 2A) was required. The Plan 2 spur, it appeared, could be reduced in elevation (Plan 2A) and minimize wave energy reaching Mole A due to spilling waves propagating northerly over the breakwater.

<u>c</u>. For test waves from 220 deg, existing conditions revealed severe overtopping of the south breakwater and Mole D with subsequent flooding of the mole and adverse wave conditions in Basin 3. The proposed improvement plans (Plans 3 and 3A) reduced wave heights in the Mole D/Basin 3 vicinity; however, overtopping of the south breakwater and Mole D still occurred, only not to as great a degree. Data obtained should aid in the design of structures proposed along the waterfront in the Mole D/Basin 3 area.

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Table 1

Unsheltered and Sheltered Deepwater Wave Characteristics Seaward of Redondo Beach King Harbor for Various Storm Wave Events

Date of Storm	Unsheltered Deepwater Significant Wave Height, H_s, ft	Wave Period T, sec	Unsheltered Deepwater Approach Azimuth, deg	Island Sheltering Coefficient, K_I	Sheltered Deepwater Significant Wave Height, H_s, ft	Sheltered Deepwater Approach Azimuth, deg
January 1988	33.1	15.0	270	0.79	25.9	258
September 1939	26.9	14.0	205	0.67	18.0	224
April 1958	25.1	17.5	293	0.57	14.3	268
March 1983	23.6	18.5	263	0.80	18.9	253
January 1981	21.5	15.5	269	0.76	16.3	257
January 1983	21.0	20.5	283	0.66	13.9	264
November 1982	20.4	10.5	293	0.57	11.6	268
February 1963	19.5	13.5	269	0.76	14.8	257
January 1978	18.6	16.5	284	0.65	12.1	264
February 1960	18.3	18.5	294	0.56	10.3	269
January 1958	18.1	13.5	270	0.75	13.6	258
March 1904	17.9	12.0	225	0.83	14.9	235
March 1912	17.5	11.5	270	0.75	13.1	258
February 1983	17.1	16.5	275	0.71	12.1	260
February 1915	16.5	12.4	280	0.67	11.1	263
January 1915	16.3	11.8	205	0.67	10.9	224

(Continued)

Table 1 (Concluded)

Date of Storm	Unsheltered Deepwater Significant Wave Height, H_s, ft	Wave Period T, sec	Unsheltered Deepwater Approach Azimuth, deg	Island Sheltering Coefficient, K_I	Sheltered Deepwater Significant Wave Height, H_s, ft	Sheltered Deepwater Approach Azimuth, deg
January 1943	16.2	10.8	180	0.43	7.0	214
January 1953	16.0	19.2	260	0.82	13.1	251
February 1969	15.6	14.5	284	0.65	10.1	264
February 1980	15.6	14.5	255	0.86	13.4	248
January 1981	15.4	17.5	265	0.79	12.2	255
December 1969	14.4	20.5	276	0.70	10.1	261
January 1916	14.0	9.6	250	0.88	12.3	245
December 1914	13.0	9.9	180	0.43	5.6	214
February 1926	12.6	16.0	260	0.82	10.3	251
April 1926	11.8	13.8	270	0.75	8.9	259
March 1952	11.7	11.7	250	0.88	10.3	245
December 1937	11.6	16.4	270	0.75	8.7	258
August 1972	11.6	17.5	156	--*	--*	--*
September 1963	10.3	14.5	167	0.23	2.4	208
September 1982	10.1	17.5	158	--*	--*	--*

* Wave energy for this deepwater unsheltered direction of approach could not reach the structure.

(Sheet 2 of 2)

Table 2

Wave Heights for Existing Conditions for Test Waves

from 260 degrees

Test Wave		Wave Height at Indicated Gage Location, ft						
Period	Approximate Wave Height Seaward of Breakwater, ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7
8	8.0	8.1	8.5	8.0	0.9	0.9	0.4	0.7
	10.0	9.9	10.6	9.9	0.9	1.1	0.4	0.9
10	8.0	7.2	8.3	8.4	1.1	1.2	0.7	1.7
	10.0	9.8	10.7	10.9	1.3	1.4	0.9	1.9
	12.0	11.3	12.1	12.9	1.4	1.5	1.0	1.8
12	10.0	9.7	10.5	10.7	1.3	1.5	1.2	1.9
	12.0	11.4	12.6	12.6	1.4	1.8	1.3	2.2
	14.0	13.1	14.6	14.6	1.5	2.1	1.4	2.4
14	10.0	9.8	9.9	10.4	1.2	2.2	1.4	1.8
	12.0	11.8	12.0	12.3	1.3	2.4	1.6	2.0
	15.5	15.0	15.4	16.3	1.8	3.0	2.0	2.5

Table 3

Wave Heights for Existing Conditions for Test Waves

from 220 degrees

Test Wave		Wave Height at Indicated Gage Location, ft								
Period	Approximate Wave Height at Entrance, ft	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14	Gage 15	Gage 16
8	8.0	1.6	2.2	4.4	3.4	2.8	1.3	1.2	4.9	7.9
10	8.5	2.7	3.5	4.7	4.7	3.4	1.7	1.5	4.9	8.5
12	11.5	3.9	4.3	6.1	5.0	4.0	1.7	2.3	6.4	11.6
15	9.0	2.3	2.7	7.2	3.8	3.5	1.9	2.2	6.0	9.2
	11.5	3.1	3.3	7.9	4.7	4.3	2.4	2.7	7.4	11.5
	13.0	3.8	3.7	8.3	5.1	4.7	2.4	3.3	8.2	13.1

Table 4

Wave Heights for Plan 1A for Test Waves for 260 degrees

Test Wave		Wave Height at Indicated Gage Location, ft						
Period	Approximate Wave Height Seaward of Breakwater, ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7
8	8.0	6.8	7.6	8.2	0.7	0.9	0.4	0.6
	10.0	8.4	9.5	10.0	0.7	1.0	0.4	0.7
10	8.0	6.3	7.5	8.5	0.9	1.1	0.6	1.4
	10.0	8.1	9.9	10.9	1.0	1.3	0.7	1.7
	12.0	9.6	11.7	13.0	1.1	1.4	0.8	1.8
12	10.0	8.6	10.3	10.9	1.3	1.6	1.1	1.9
	12.0	10.1	12.3	12.8	1.4	1.8	1.2	2.2
	14.0	12.8	14.6	14.6	1.5	2.1	1.4	2.4
14	10.0	9.2	10.0	10.2	1.2	2.4	1.4	1.8
	12.0	10.7	12.0	11.9	1.4	2.6	1.5	2.0
	15.5	13.2	15.1	15.0	1.7	2.9	1.7	2.2

Table 5

Wave Heights for Plan 3 for Test Waves

from 220 degrees

Test Wave		Wave Height at Indicated Gage Location, ft								
Period	Approximate Wave Height at Entrance, ft	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14	Gage 15	Gage 16
8	8.0	1.1	1.5	3.6	2.2	1.1	0.7	0.8	4.8	7.3
10	8.5	1.7	2.5	4.1	3.3	1.6	1.1	1.1	5.2	7.8
12	11.5	3.6	3.3	5.4	4.9	2.5	1.6	2.1	8.3	12.2
15	9.0	2.0	2.1	6.7	3.0	1.9	1.6	1.4	6.1	8.7
	11.5	2.9	2.6	7.5	3.6	2.5	1.9	2.2	7.2	10.3
	13.0	3.5	2.9	8.1	4.2	3.0	2.2	2.9	8.5	12.1

Table 6

Wave Heights for Plan 3A for Test Waves

from 220 degrees

Test Wave		Wave Height at Indicated Gage Location, ft								
Period	Approximate Wave Height at Entrance, ft	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14	Gage 15	Gage 16
8	8.0	1.6	2.2	3.6	2.7	2.1	1.1	0.8	4.9	7.0
10	8.5	2.7	3.5	3.9	4.4	2.9	1.7	1.4	5.7	8.1
12	11.5	3.6	4.2	5.0	5.0	3.4	1.8	1.9	7.9	11.6
15	9.0	2.3	2.7	6.5	3.5	2.6	1.8	2.1	6.2	8.9
	11.5	2.6	3.0	7.4	4.0	3.2	2.2	2.2	7.3	10.8
	13.0	3.3	3.6	7.7	4.9	4.1	2.5	2.6	8.3	12.6

Photo 1. Typical wave patterns for existing conditions; 10-sec, 12-ft waves from 260 deg

Photo 2. Typical wave patterns for existing conditions; 12-sec, 14-ft waves from 260 deg

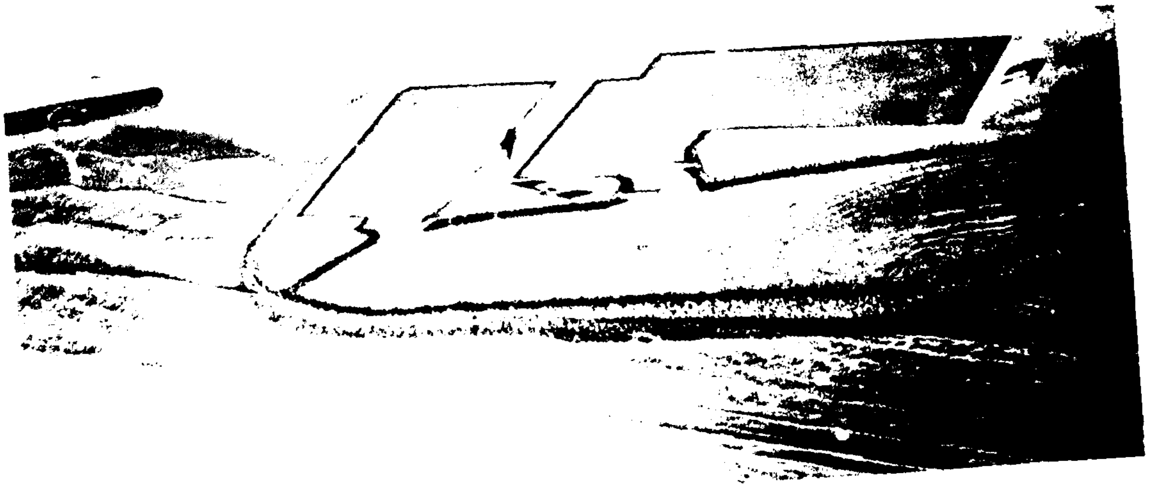


Photo 3. Typical wave patterns for existing conditions; 14-sec, 12-ft waves from 260 deg

Photo 4. Typical wave patterns for existing conditions; 10-sec, 8.5-ft waves from 220 deg

Photo 5. Typical wave patterns for existing conditions; 12-sec, 11.5-ft waves from 220 deg

Photo 6. Typical wave patterns for existing conditions; 15-sec, 13-ft waves from 220 deg

Photo 7. Typical wave patterns for Plan 1A; 10-sec, 12-ft waves from 260 deg

Photo 8. Typical wave patterns for Plan 1A; 12-sec, 14-ft waves from 260 deg

Photo 9. Typical wave patterns for Plan 1A; 14-sec, 12-ft waves from 260 deg

Photo 10. Typical wave patterns for Plan 2A; 15-sec, 15-ft waves from 240 deg

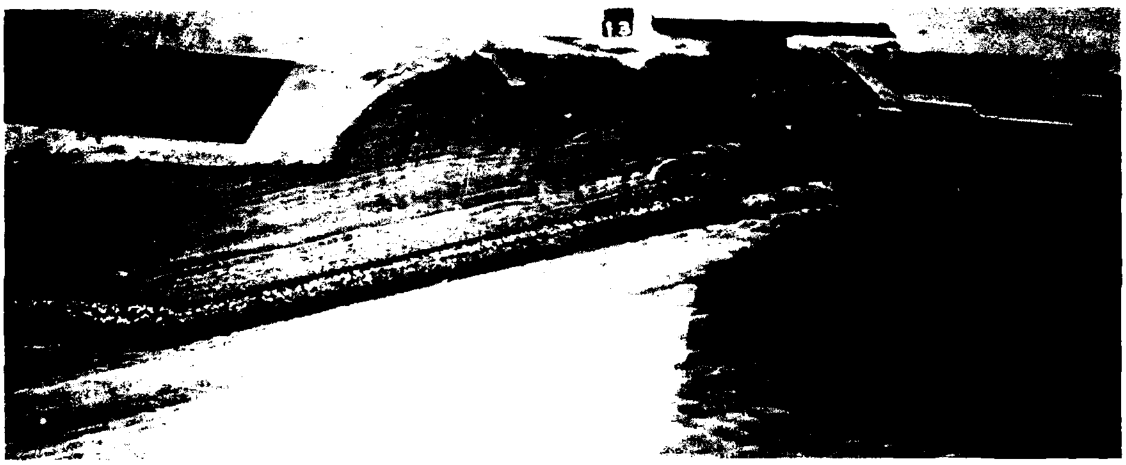


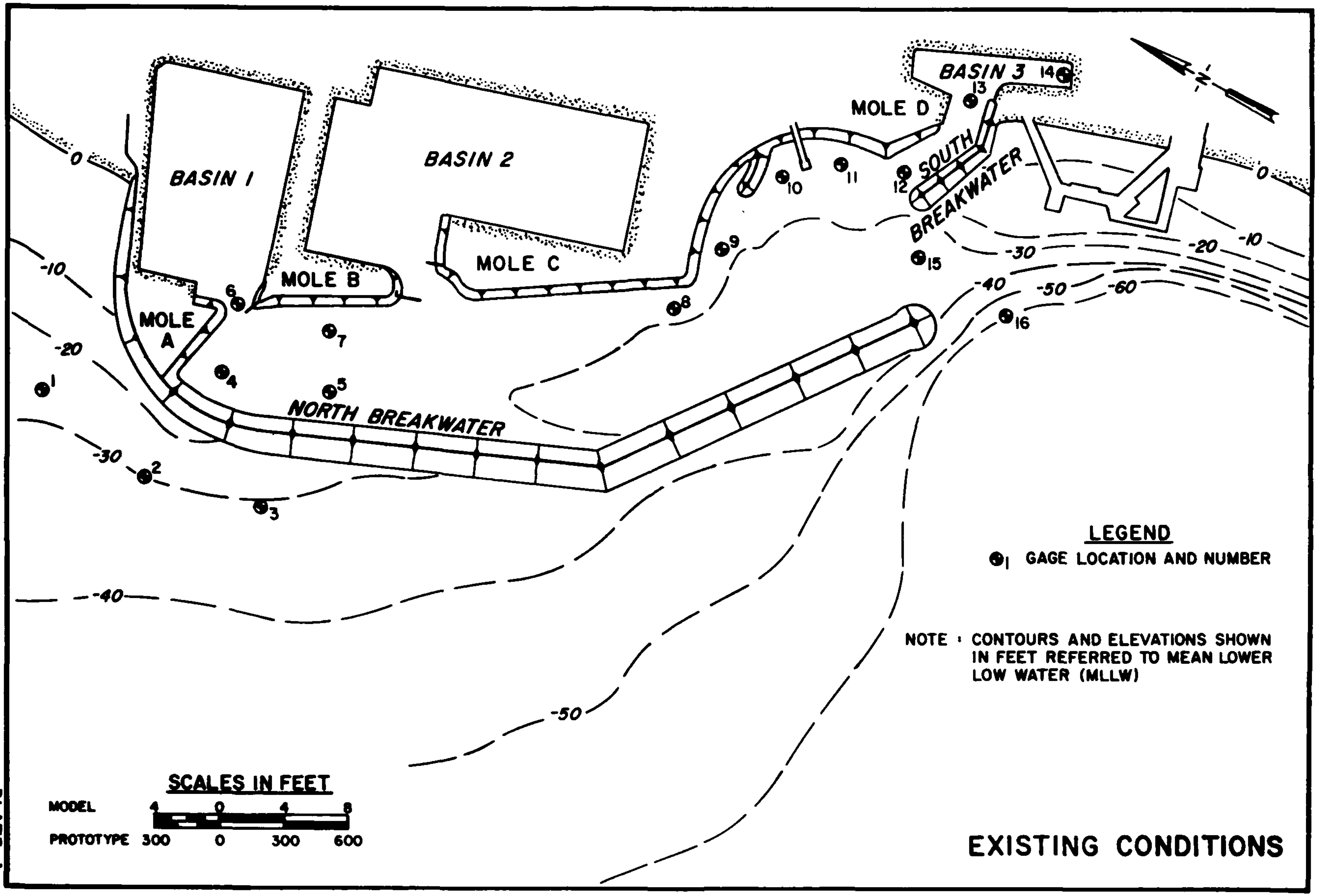
Photo 11. Typical wave patterns for Plan 3; 10-sec, 8.5-ft waves from 220 deg

Photo 12. Typical wave patterns for Plan 3; 12-sec, 11.5-ft waves from 220 deg

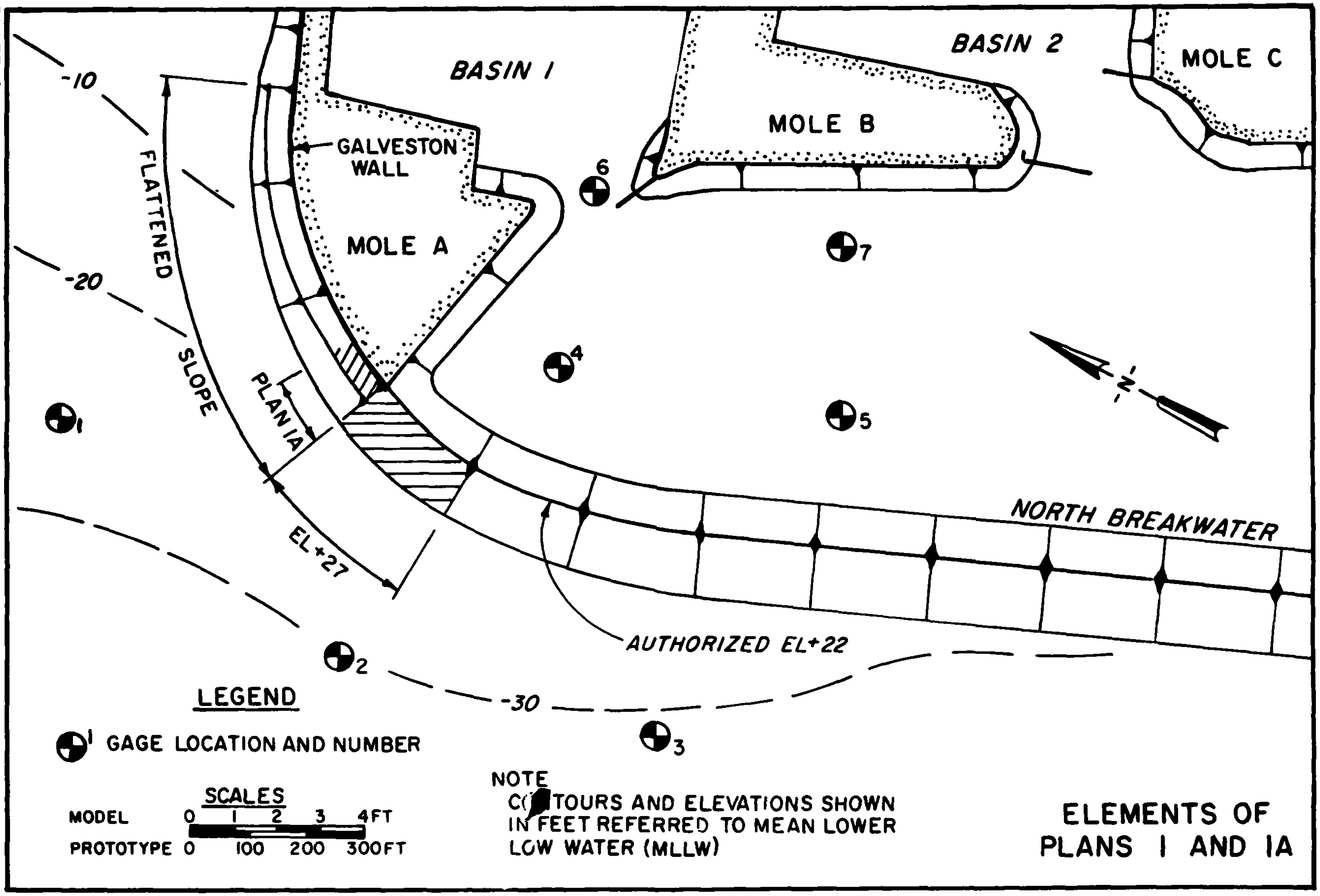
Photo 13. Typical wave patterns for Plan 3; 15-sec, 13-ft waves from 220 deg

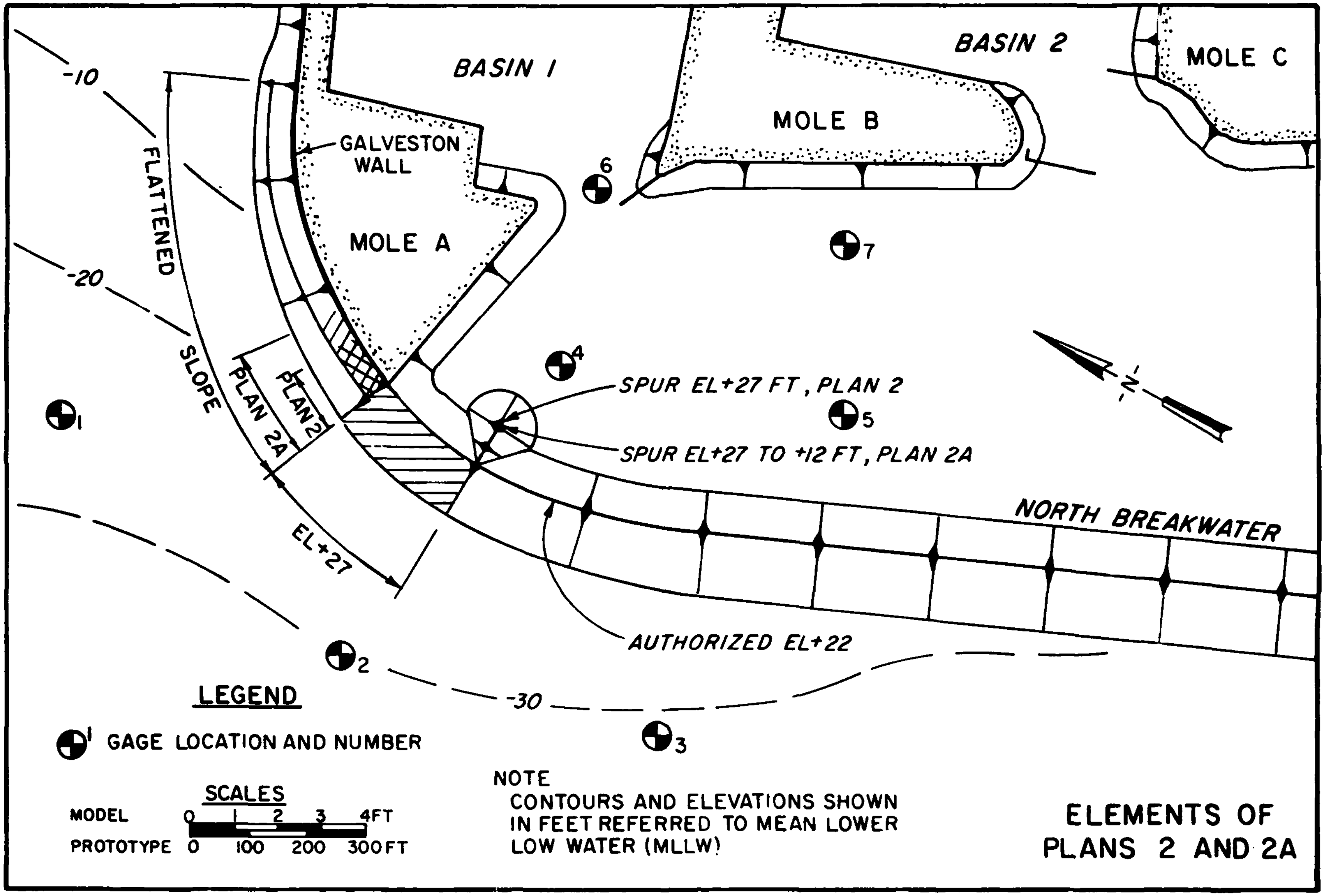
Photo 14. Typical wave patterns for Plan 3A; 12-sec, 11.5-ft waves from 220 deg

Photo 15. Typical wave patterns for Plan 3A; 15-sec, 13-ft waves from 220 deg

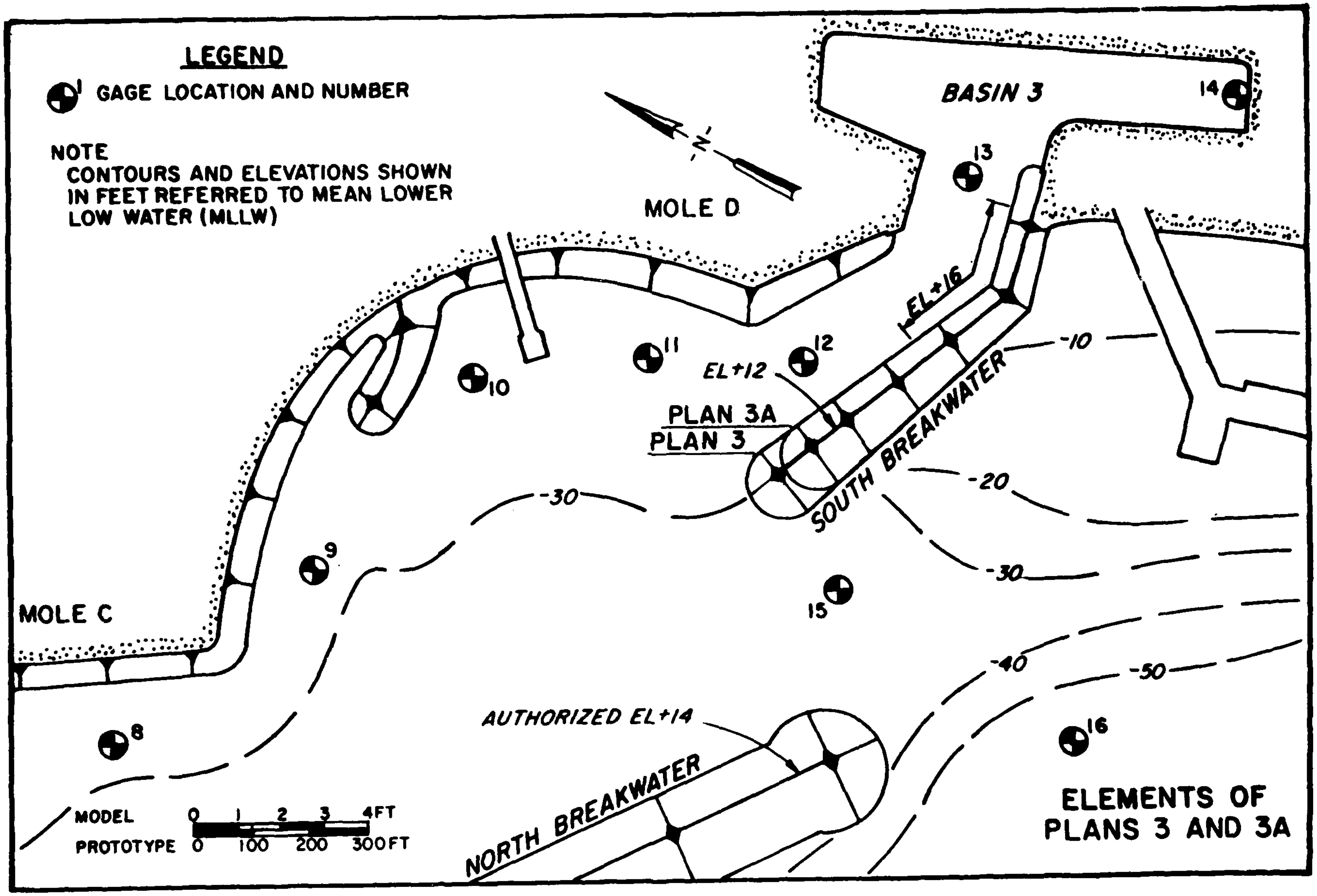


EXISTING CONDITIONS


BASIN 1
BASIN 2
MOLE A
MOLE B
MOLE C
GALVESTON WALL
NORTH BREAKWATER
N
FLATTENED SLOPE
PLAN 1A
EL+27
AUTHORIZED EL+22
-10
-20
-30
1
2
3
4
5
6
7
LEGEND
GAGE LOCATION AND NUMBER
SCALES
MODEL 0 1 2 3 4 FT
PROTOTYPE 0 100 200 300 FT
NOTE
CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
ELEMENTS OF PLANS 1 AND 1A



ELEMENTS OF PLANS 2 AND 2A


LEGEND
1 GAGE LOCATION AND NUMBER
NOTE
CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
N
MOLE D
MOLE C
BASIN 3
EL+16
EL+12
PLAN 3A
PLAN 3
SOUTH BREAKWATER
NORTH BREAKWATER
AUTHORIZED EL+14
-10
-20
-30
-30
-40
-50
8
9
10
11
12
13
14
15
16
MODEL 0 1 2 3 4FT
PROTOTYPE 0 100 200 300FT
ELEMENTS OF PLANS 3 AND 3A