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NOYO RIVER AND HARBOR, CALIFORNIA DESIGN FOR WAVE AND SURGE PROTECTION

Coastal Model Investigation

by

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A 1:75-scale undistorted hydraulic model was used to determine both short- and long-period wave conditions and river flow conditions in Noyo River and Harbor as a result of various breakwater configurations at the entrance. The model reproduced the river from its mouth to a point approximately 15,000 ft upstream, both Noyo Harbor and Dolphin Marina located on the south bank, approximately 3,500 ft of the California shoreline on each side of the river mouth, Noyo Cove, and sufficient offshore area in the Pacific Ocean to permit generation of the required test waves. Some improvement plans included the use of dolosse armor units for breakwaters constructed in Noyo Cove seaward of the river entrance. A 45-ft-long wave generator, crushed coal sediment tracer material, and an automated data acquisition and control system were utilized in model operation. It was concluded from the model investigation that:

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19. ABSTRACT (Continued).

a. Existing conditions are characterized by very rough and turbulent wave conditions in the Noyo River entrance during periods of storm wave attack.

b. Deepening of the entrance channel will not significantly improve wave conditions in the existing river entrance, considering all test conditions.

c. The originally proposed breakwater location (Plan 3) resulted in excessive wave heights (8.8 ft) in the river entrance.

d. Of the 40 expedient rubble-mound (stone) breakwater plans (Plans 5-42) tested, the alignment of the 637-ft-long breakwater of Plan 39 appeared to be optimum with regard to wave protection, navigation, and economics.

e. The 637-ft-long dolosse breakwater of Plan 43 (same alignment as Plan 39) was selected as the optimum improvement plan for protection of the Noyo River entrance.

f. The breakwater configuration of Plan 43 will result in improved surge conditions due to long-period wave energy in Noyo River and Harbor.

g. The breakwater configuration of Plan 43 will not interfere with the movement of riverine sediment seaward into Noyo Cove; however, the structure will direct sediment to the northern portion of the cove.

PREFACE

A request for a model investigation of Noyo River and Harbor, California, was initiated by the US Army Engineer District, San Francisco (SPN), in a letter to the US Army Engineer Division, South Pacific (SPD), dated 16 February 1979. Authorization for the US Army Engineer Waterways Experiment Station (WES) to perform the study subsequently was granted and funds were authorized by SPN on 15 June 1981, 20 October 1981, 14 February 1984, 5 February 1987, and 28 January 1988. The Noyo River and Harbor project was under the jurisdiction of SPN with engineering support provided by the US Army Engineer District, Los Angeles (SPL).

Model testing was conducted at WES intermittently during the period August 1984-August 1986 by personnel of the Wave Processes Branch (WPB), Wave Dynamics Division (WDD), Coastal Engineering Research Center (CERC) under the direction of Drs. R. W. Whalin and J. R. Houston, Chiefs, CERC; and Messrs. C. C. Calhoun, Jr., Assistant Chief, CERC; C. E. Chatham, Jr., Chief, WDD; and D. G. Outlaw, Chief, WPB. The tests were conducted by Messrs. H. F. Acuff and M. G. Mize, Civil Engineering Technicians, under the supervision of Mr. R. R. Bottin, Jr., Project Manager. Two-dimensional flume tests conducted to design the 1:75-scale breakwater cross section for the study were performed by Mr. Acuff under the supervision of Mr. D. G. Markle, Research Hydraulic Engineer, assigned to the Wave Research Branch, WDD, CERC. The main text of this report was prepared by Messrs. Bottin and Acuff. Appendix A was prepared by Mr. Markle. This report was edited by Mr. Bobby Odom, Information Products Division, Information Technology Laboratory.

Prior to the model investigation, Messrs. Outlaw and Bottin met with representatives of SPN and visited Noyo River and Harbor to inspect the prototype site. During the course of the investigation, liaison was maintained by means of conferences, telephone communications, and monthly progress reports.

Mr. Hugh Converse, SPD; Messrs. Bill Brick, Mark Dettle, and Herb Cheong, SPN; Messrs. Dee Gonzales, Alan Alcorn, Angel Fuertes, and Tad Nizinski, SPL; Mr. Howard Merritt, Harbor Master, Noyo Harbor; and Mr. Don Bradley, Harbor Commissioner, Noyo Harbor, visited WES to observe model operation and participate in conferences during the course of the study.

This investigation was the second model study of wave action at Noyo

Harbor conducted by WES. The first was completed in 1966 and reported in WES Technical Report No. 2-799, "Wave Action and Breakwater Location, Noyo Harbor, California," dated November 1967.

COL Dwayne G. Lee, EN, was Commander and Director of WES during the preparation and publication of this report. Dr. Robert W. Whalin was Technical Director.

CONTENTS

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
cubic feet	0.02831685	cubic metres
degrees (angle)	0.01745329	radians
feet	0.3048	metres
inches	2.54	centimetres
miles (US statute)	1.609347	kilometres
pounds (force)	4.448222	newtons
pounds per cubic foot	16.01846	kilograms per cubic metre
square feet	0.09290304	square metres
square miles (US statute)	2.589998	square kilometres

NOYO RIVER AND HARBOR, CALIFORNIA

DESIGN FOR WAVE AND SURGE PROTECTION

Coastal Model Investigation

PART I: INTRODUCTION

Prototype

1. Noyo River and Harbor are located on the California Coast in Mendocino County, approximately 135 miles* north of San Francisco and 87 miles south of Eureka (Figure 1). The shoreline in the locality consists of broken, irregular cliffs about 40 to 80 ft high with numerous rocks extending several hundred yards offshore. Small pocket beaches are found at the heads of coves in the immediate vicinity. The Noyo River empties into Noyo Cove which is approximately 1,800 ft wide, north to south, and 2,000 ft long, east to west.

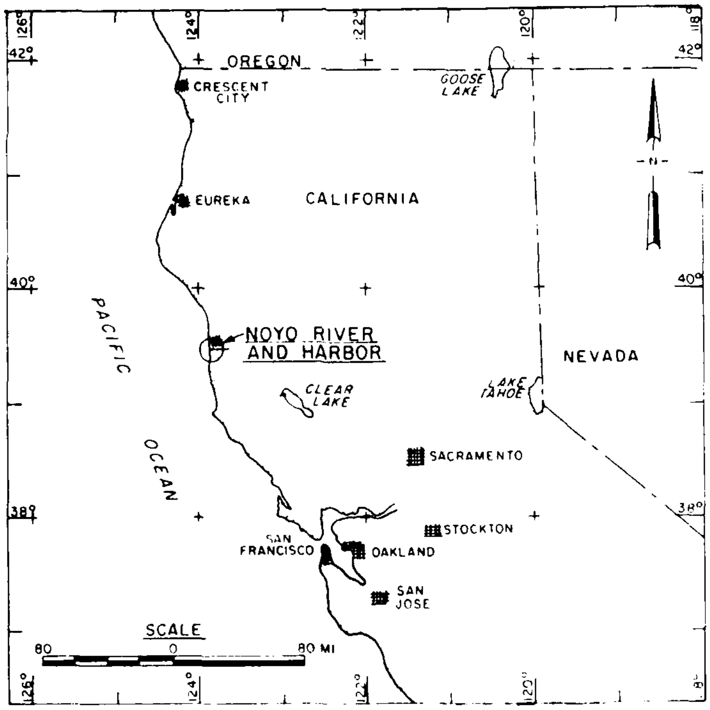


Figure 1. Project location

* A table of factors for converting non-SI units of measurements to SI (metric) units is presented on page 4.

2. The existing Noyo River and Harbor project was authorized by the River and Harbor Act of 1930 (US Army Engineer District (USAED), San Francisco 1979), and construction was completed in 1961. It consists of a jettied entrance at the river mouth; a 10-ft-deep, 100-ft-wide entrance channel; and a 10-ft-deep, 150-ft-wide river channel extending upstream about 0.6 miles. Noyo Harbor is located on the south bank of the river at the upstream limit of the dredged river channel. Also, further upstream, approximately 1.1 miles from the river mouth, a privately owned harbor, Dolphin Marina, is located on the south bank. An aerial photograph of the area is shown in Figure 2.

Problem

3. Noyo Cove is open to the Pacific Ocean and exposed to large waves generated by local coastal storms accompanied by strong winds (sea) and distant ocean storms without local winds (swell). Waves in excess of 20 ft in height approach the cove from the southwest clockwise through northwest directions. Heavy seas sweep across the cove and through the jettied river entrance, making it impassable for entry or departure during these periods. In addition to these adverse wave conditions, the harbor has experienced strong surging problems due to long-period wave energy resulting in damages to small craft moored there. Shoaling in the river channel is also experienced due to the deposition of material brought down the river during the winter rainy season. This shallow river channel results in navigational difficulties, particularly upstream of Noyo Harbor. Vessels are subject to damage by grounding and are forced to wait for favorable tide conditions to provide adequate depths.

4. Improvements at Noyo River and Harbor would result in prevention of boat damage, a harbor of refuge for vessels during storm activity, increased recreational boating, and area redevelopment. Potential commercial benefits would include increased lumber processing (barging of wood chips to Eureka and barging of finished lumber to Los Angeles) and commercial fishing (increased fish catch).

Proposed Improvements

5. Authorization for improvements at Noyo River and Harbor was granted by the River and Harbor Act of 1962. Under this authorization, however,

Figure 2. Aerial view of prototype site

breakwaters were proposed to protect the outer cove for development. The breakwaters required were not economically feasible (due to the high cost of construction and maintenance) resulting in the project being transferred to an inactive category. The Water Resources Development Act of 1976 modified the 1962 project to provide for construction of up to two breakwaters without a specific location to protect the harbor entrance (USAED, San Francisco 1979). The location of breakwaters in more shallow water would reduce construction cost significantly. The 1976 Act also included additional channel improvements (deepening, widening, and extending) as deemed necessary to meet applicable economic and environmental criteria.

Purpose of Model Study

6. At the request of the US Army Engineer District, San Francisco (SPN) and the US Army Engineer District, Los Angeles (SPL), a hydraulic model investigation was initiated by the US Army Engineer Waterways Experiment Station's (WES) Coastal Engineering Research Center (CERC) to:

- a. Study short- and long-period wave conditions and river flow conditions in Noyo River and Harbor.
- b. Determine the most economical breakwater configuration that would provide adequate wave protection to the entrance.
- c. Provide qualitative information on the effects of the breakwaters on sediment moving down the river.
- d. Develop remedial plans for the alleviation of undesirable conditions as found necessary.

Wave-Height Criteria

7. 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, however, SPL initially specified that for an improvement plan to be acceptable, maximum wave heights were not to exceed 4.0 ft in the existing Noyo River entrance. During the course of the investigation, however, the maximum wave height that could be tolerated in the existing entrance was increased to approximately 6.0 ft (provided the wave was nonbreaking). This value was selected at a meeting in Fort Bragg, CA, attended by representatives of SPL, SPN, WES, US Coast Guard, and local harbor users.

PART II: MODEL

Design of Model

8. The Noyo River and 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 follows:

Characteristic	Dimension*	Scale Relations Model:Prototype
Length	L**	$L_r = 1:75$
Area	L^2	$A_r = L_r^2 = 1:5,625$
Volume	L^3	$V_r = L_r^3 = 1:421,875$
Time	T	$T_r = L_r^{1/2} = 1:8.66$
Velocity	L/T	$V_r = L_r^{1/2} = 1:8.66$
Roughness (Manning's coefficient n)	$L^{1/6}$	$n_r = L_r^{1/6} = 1:2.054$
Discharge	L^3/T	$Q_r = L_r^{5/2} = 1:48,714$

* Dimensions are in terms of length and time.
** For convenience, symbols and unusual abbreviations are listed and defined in the Notation (Appendix B).

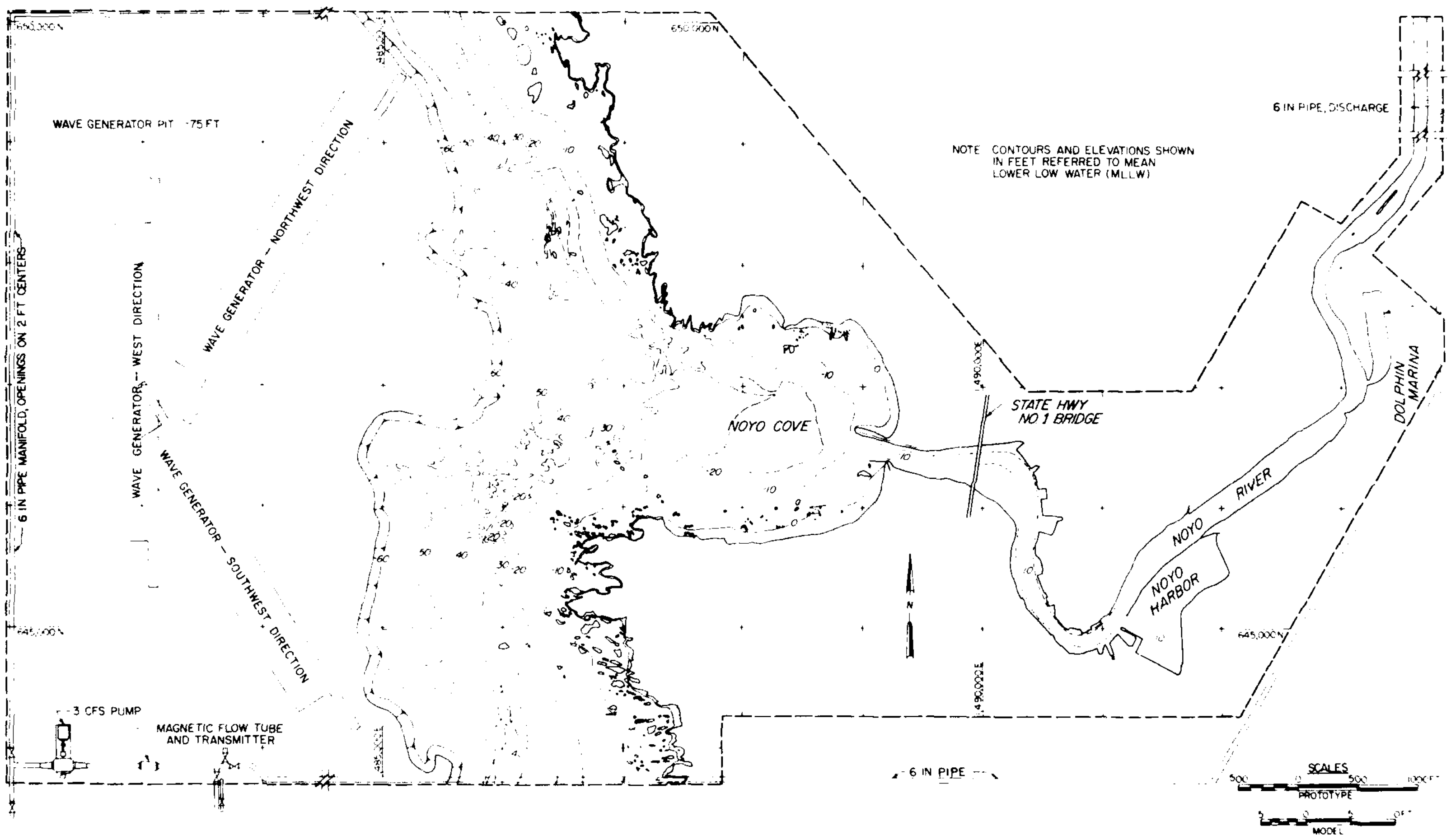


Figure 3. Model layout

9. The proposed breakwaters at Noyo included the use of concrete armor units (dolos). Since the porosity of these armor units differs from that of rock and since the units could not be reproduced to scale (due to cost and time requirements), two-dimensional wave transmission tests were conducted at a scale large enough to have negligible scale effects (i.e. 1:31) to determine the correct transmission through the proposed structures. This transmission was then duplicated at a scale of 1:75 using small dolos and rock cross sections, and the three-dimensional model structures were built accordingly. These tests are detailed in Appendix A.

10. Parts of the existing jetties at Noyo River entrance are 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 (LeMéhauté 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 was then 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 Noyo, it was determined that a close approximation of the correct wave-energy transmission characteristics would be obtained by increasing the size of the rock used in the 1:75-scale model to approximately 1-1/2 times that required for geometric similarity. Accordingly, in constructing the rubble-mound structures in the Noyo River and 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.

11. The values of Manning's roughness coefficient n used in the design of the main river channel were calculated from water-surface profiles

of known discharges in the prototype. From these computations and experience, an n value of 0.030 was selected for use in the main river channel. In addition, based on experience, an n value of 0.050 was selected for overbank roughness. Therefore, based on previous WES investigations (Miller and Peterson 1953, Cox 1973), the various model areas from the Noyo Harbor entrance extending upstream were given finishes that would represent prototype n values of 0.030 and 0.050.

12. Ideally, a quantitative, three-dimensional, movable-bed model investigation would best determine the impacts of the proposed structures with regard to the deposition of sediment at the river mouth. 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. In view of the complexities involved in conducting movable-bed model studies and due to limited funds and time for the Noyo River and Harbor project, the model was molded in cement mortar (fixed-bed) at an undistorted scale of 1:75, and a tracer material was obtained to qualitatively determine the deposition of riverine sediment (degree of accretion, etc.) at the river mouth for existing conditions and the optimum improvement plan.

Model and Appurtenances

13. The model reproduced the lower 15,000 ft of Noyo River, both Noyo Harbor and Dolphin Marina (located on the south bank), Noyo Cove, approximately 5,500 ft of the California shoreline on each side of the river mouth, and underwater topography in the Pacific Ocean to an offshore depth of 60 ft with a sloping transition to the wave generator pit elevation of -75 ft. The total area reproduced in the model was approximately 12,000 sq ft, representing about 2.4 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 (mllw).* Horizontal control was referenced to a local prototype grid system.

14. Model waves were generated by a 45-ft-long piston-type generator. The horizontal movement of the piston plate caused a periodic displacement of

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

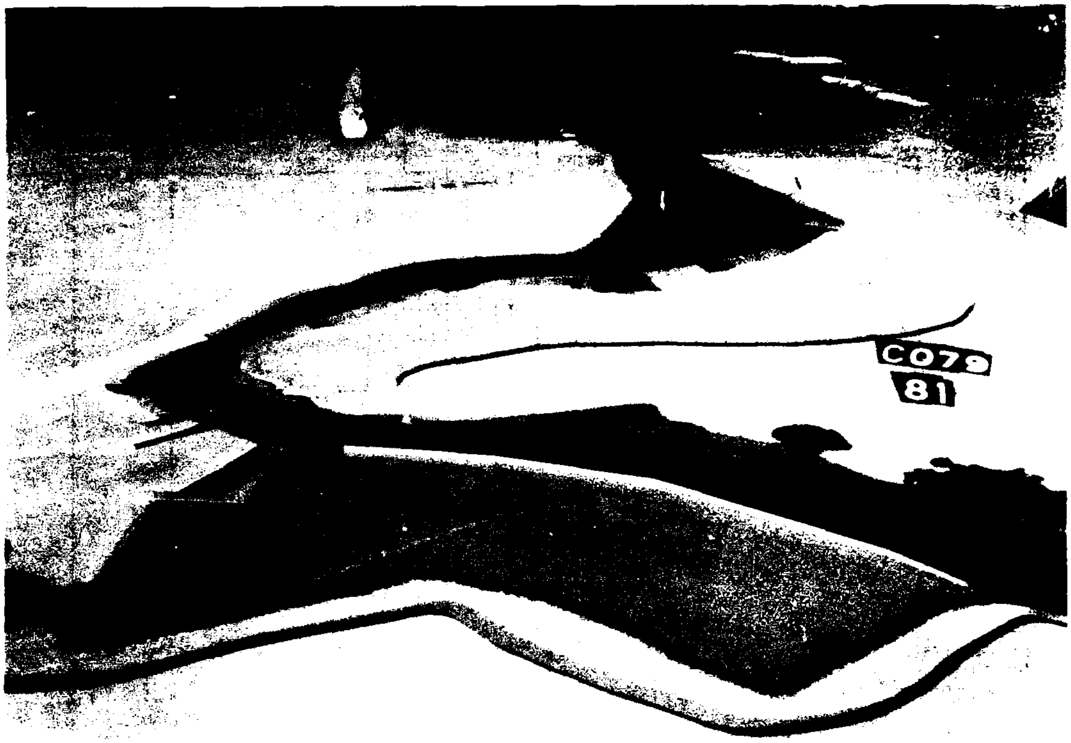


Figure 4. General view of model

water incident to this motion. The length of the stroke and the frequency of the piston plate movement were variable over the range necessary to generate waves with the required characteristics. In addition, the wave generator was mounted on retractable casters which enabled it to be positioned to generate waves from the required directions.

15. A water circulation system (Figure 3) consisting of a 6-in. perforated-pipe water-intake manifold, a 3-cfs pump, and a magnetic flow tube and transmitter, was used in the model to reproduce steady-state flows through the river channel that corresponded to selected prototype river discharges.

16. An Automated Data Acquisition and Control System (ADACS), designed and constructed at WES (Figure 5), was used to secure 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 was then analyzed to obtain the wave-height data.

17. A 2-ft (horizontal) solid layer of fiber wave absorber was placed

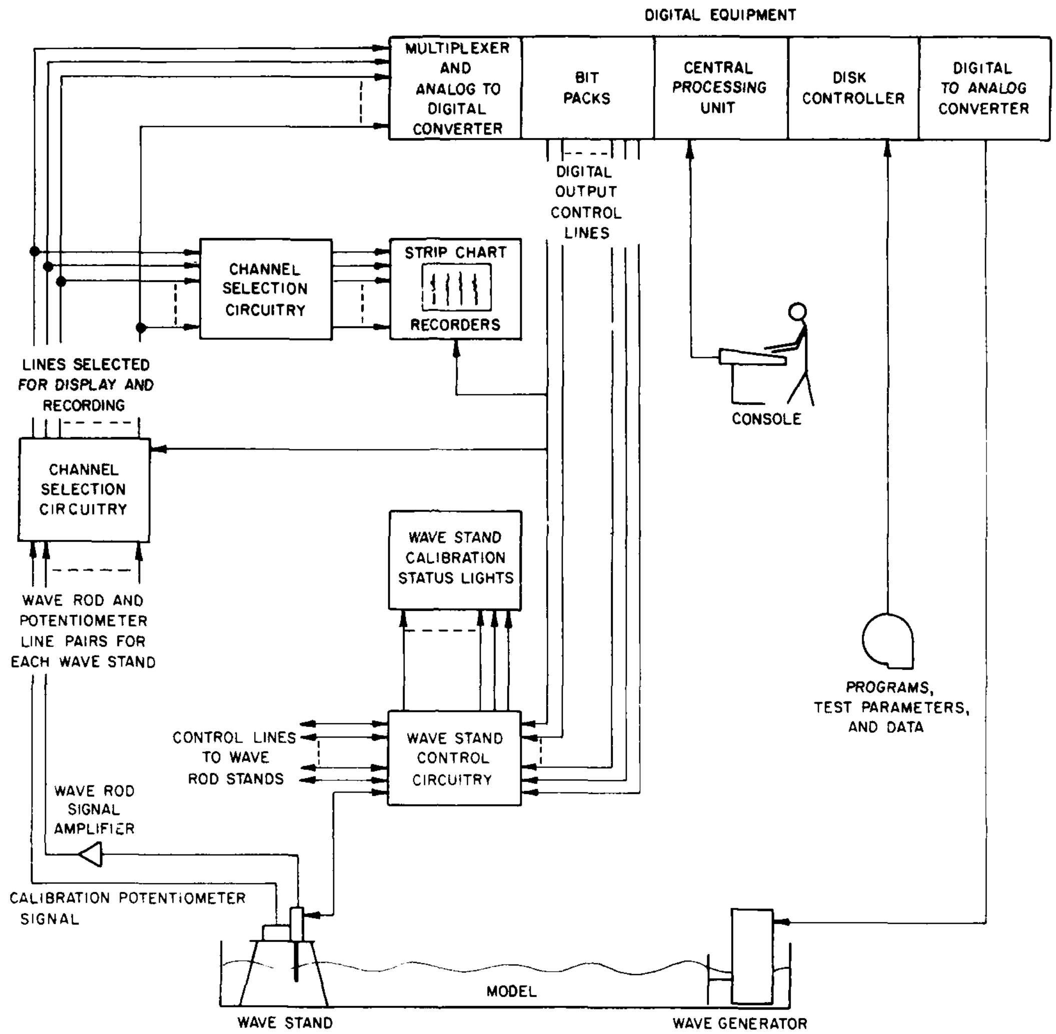


Figure 5. Automated Data Acquisition and Control System (ADACS)

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.

18. As discussed previously in paragraph 12, a fixed-bed model was constructed and a tracer material was selected to qualitatively determine the deposition of sediment at the river mouth. Using the prototype sand characteristics (median diameter, D_{50} = 0 25 mm, specific gravity = 2.69), 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 . These relations were determined experimentally using a wide range of conditions and bottom materials. 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} = 0.76 mm) were selected for use as a tracer material. Tests involving the effect of various structures on the movement of bed-load sediment through the lower reaches of a river (similar to the Noyo River) were conducted recently at WES for Rogue River, OR (Bottin 1982).

PART III: TEST CONDITIONS AND PROCEDURES

Selection of Test Conditions

Still-water level

19. Still-water levels (swl) for 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 structures by the waves, the reflection of wave energy from various structures, and the transmission of wave energy through porous structures.

20. In most cases, it is desirable to select a model swl that closely approximates the higher water stages which 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 tide and shoreward mass transport.
- 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.

21. A swl of +6.2 ft was initially selected by SPL for use during model testing. This value (+6.2) represented mean higher high water (mhhw). During the conduct of model testing; however, the swl was revised to +7.0 ft, which represents a monthly occurrence at the site.

Factors influencing selection of test-wave characteristics

22. 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

23. 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 are determined by using the numerical Regional Coastal Processes Wave Transformation Model (RCPWAVE) developed by Ebersole (1984). 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 which comprise the domain of interest. Much of the early work in this area during the 1950's was based on wave-ray methods and manual construction of refraction diagrams using linear gravity-wave theory. During the 1960's and early 1970's, 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.

24. 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.

25. 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 wave length and water depth, can be obtained from the Shore Protection Manual (1984).

26. Refraction and shoaling coefficients and shallow-water directions were obtained at Noyo for various wave periods from five deepwater wave directions (northwest counterclockwise through southwest) and are presented in Table 1. Shallow-water wave directions and refraction coefficients represent an average of the values in the immediate vicinity of the Noyo site (approximately the location of the wave generator in the model). Shoaling coefficients were computed for an 81-ft water depth (75-ft pit elevation with 6-ft tide conditions superimposed) corresponding to the simulated depth at the model wave generator. The wave-height adjustment factor $K_r \times K_s$ can be applied to any deepwater wave height to obtain the corresponding shallow-water value. Based on the refracted directions secured at the approximate locations of the wave generator in the model for each wave period, the following test directions (deepwater direction and corresponding shallow-water direction) were selected for use during model testing.

Deepwater Direction deg	Selected Shallow-Water Test Direction deg
Northwest, 315	300
West-northwest, 292.5	288
West, 270	270
West-southwest, 247.5	254
Southwest, 225	238

Prototype wave data and selection of test waves

27. Measured prototype wave data on which a comprehensive statistical analysis of wave conditions could be based were unavailable for the Noyo

Harbor area. However, statistical deepwater wave hindcast data representative of this area were obtained from the Sea-State Engineering Analysis System (SEAS) by Corson (1985). Deepwater SEAS data are summarized in Table 2. These data were converted to shallow-water values by application of refraction and shoaling coefficients and are shown in Table 3. Characteristics of test waves used in the model (selected from Table 3) are shown in the following tabulation:

Deepwater Direction	Selected Test Waves Period, sec	Selected Test Waves Height, ft
Northwest	7	8, 14, 20
	9	6, 12, 20
	11	6, 12, 24
	13	6, 12, 20
	15	10, 20
	17	6, 12, 22
	19	12
West-northwest	7	8, 16
	9	6, 10, 18
	11	6, 12, 24
	13	6, 14, 22
	15	10, 20, 30
	17	10, 20, 28
	19	12, 22
West	7	8, 14, 20
	9	6, 12, 22
	11	6, 12, 18, 30
	13	6, 12, 20, 30
	15	10, 20, 30
	17	10, 20, 28
West-southwest	7	8, 14, 20
	9	6, 12, 22
	11	10, 20, 30
	13	10, 20, 32
	15	10, 20, 32
	17	14, 20, 28
Southwest	7	8, 14, 20
	9	10, 16, 22
	11	6, 14, 20, 30
	13	10, 20, 32
	15	10, 20, 32
	17	22

River discharges

28. The Noyo River drains an area of approximately 106 square miles. River discharge data obtained from water discharge records during the period

1952-1981 were available from a water-stage recorder gage located 3.5 miles east of the river mouth. Based on these data, the following river discharges and recurrence intervals were projected by SPL and simulated in the model.

Discharge, Q cfs	Recurrence Interval years
7,000	2
20,000	10
27,000	25
33,000	50
41,000	100

Analysis of Model Data

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

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

   b. Comparison of riverine sediment tracer movement and subsequent deposits.

   c. Visual observations and wave pattern photographs.

In the wave-height data analysis, the average height of the highest one-third of the waves recorded at each gage location was computed. All wave heights were then 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.

PART IV: TESTS AND RESULTS

Tests

Existing conditions

30. Prior to testing of the various improvement plans, comprehensive tests were conducted for existing conditions (Plate 1). Wave-height data were obtained in the lower reaches of the river (including Noyo Harbor and Dolphin Marina) and along the center lines of the proposed breakwaters (for design wave information) for the selected test waves and directions listed in paragraph 27. Wave-pattern photographs were secured for representative test waves from the five test directions, and riverine sediment tracer patterns were obtained at the river mouth for various river discharges.

Improvement plans

31. Wave-height tests were conducted for 46 test plans. Two of these improvement plans consisted of channel deepening only, and the remaining alternatives included one or more breakwaters installed in the cove west of the entrance with variations in the lengths, alignments, and locations of the structures. Wave-pattern photographs were obtained for several test plans while riverine sediment tracer pattern tests and long-period wave tests were secured only for the optimum improvement plan. Dimensional details are presented in Plates 2-22; brief descriptions of the improvement plans are presented in the following subparagraphs:

- a. Plan 1 (Plate 2) entailed deepening of the entrance channel to -20 ft from the highway bridge seaward to the 20 ft contour in Noyo Cove.
- b. Plan 2 (Plate 2) involved deepening of the entrance channel to -15 ft from the highway bridge seaward to the 15 ft contour in Noyo Cove.
- c. Plan 3 (Plate 3) consisted of the installation of a 370-ft-long dolosse breakwater in Noyo Cove west of the river entrance.
- d. Plan 3A (Plate 3) entailed the elements of Plan 3 with a 75-ft extension at the north end of the breakwater resulting in a 445-ft-long structure.
- e. Plan 4 (Plate 4) included the 370-ft-long breakwater of Plan 3, with an additional 300-ft-long dolosse breakwater installed to the north and shoreward of the original structure.
- f. Plan 5 (Plate 5) involved the 370-ft-long breakwater of Plan 3, but the structure was constructed entirely of stone.

g. Plan 6 (Plate 5) included the 370-ft-long breakwater of Plan 5 with an additional 300-ft-long rubble-mound breakwater installed to the north and seaward of the original structure.

h. Plan 7 (Plate 6) consisted of a 450-ft-long offshore rubble-mound breakwater located approximately 800 ft west of the existing entrance and a 187-ft-long shore-connected rubble-mound breakwater installed south of the offshore structure.

i. Plan 8 (Plate 6) entailed the elements of Plan 7 with a 75-ft-long extension at the south end of the offshore breakwater resulting in a cumulative breakwater length of 712 ft.

j. Plan 9 (Plate 6) involved the 450-ft-long offshore breakwater of Plan 7 with a 150-ft-long offshore breakwater installed approximately 400 ft from and southerly of the existing entrance.

k. Plan 10 (Plate 7) included a 638-ft-long rubble-mound north breakwater and a 187-ft-long rubble-mound south breakwater. Both these structures were located offshore and seaward of the existing entrance.

l. Plan 11 (Plate 7) entailed the 638-ft-long north breakwater of Plan 10 with a 487-ft-long shore-connected, rubble-mound breakwater installed south of the offshore structure.

m. Plan 12 (Plate 8) consisted of a 450-ft-long offshore, rubble-mound breakwater located in Noyo Cove approximately 500 ft west of the existing entrance.

n. Plan 12A (Plate 8) included the 450-ft-long breakwater of Plan 12 with a 75-ft-long extension at the south end of the structure.

o. Plan 13 (Plate 9) included a 450-ft-long offshore, rubble-mound breakwater in Noyo Cove on the same alignment as the Plan 12 structure and a 300-ft-long rubble-mound, shore-connected breakwater installed south of the offshore structure.

p. Plan 14 (Plate 9) entailed the 450-ft-long offshore, rubble-mound breakwater of Plan 13 and an additional 300-ft-long rubble-mound offshore structure installed southwesterly of the original breakwater.

q. Plan 15 (Plate 10) consisted of an 825-ft-long offshore, rubble-mound breakwater located in Noyo Cove approximately 750 ft seaward of the existing entrance.

r. Plan 16 (Plate 11) involved a 675-ft-long offshore, rubble-mound breakwater located about 550 ft seaward of the existing entrance in Noyo Cove.

s. Plan 16A (Plate 11) included the 675-ft-long breakwater of Plan 16 with a 337-ft-long rubble-mound, offshore breakwater installed north-northwest of the existing entrance.

t. Plan 17 (Plate 12) consisted of a 712-ft-long offshore, rubble-mound breakwater located in Noyo Cove approximately 650 ft seaward of the existing entrance.

u. Plan 18 (Plate 12) involved the 712-ft-long breakwater of Plan 17 with a 413-ft-long extension at the south end of the structure.

v. Plan 19 (Plate 12) entailed the offshore breakwater of Plan 17 with 75 ft of structure removed from the north end of the breakwater resulting in a 637-ft-long structure.

w. Plan 20 (Plate 12) included the 637-ft-long breakwater of Plan 19 with a 75-ft-long entension at the south end of the structure.

x. Plan 21 (Plate 13) consisted of a 712-ft-long offshore, rubble-mound breakwater located in Noyo Cove about 600 ft seaward of the existing entrance.

y. Plan 22 (Plate 13) included the offshore breakwater of Plan 21 with 75 ft of structure removed from the north end of the breakwater resulting in a 637-ft-long structure.

z. Plan 23 (Plate 13) involved the offshore breakwater of Plan 21 with 150 ft of structure removed from the north end of the breakwater resulting in a 562-ft-long structure.

aa. Plan 24 (Plate 14) consisted of a 525-ft-long offshore rubble-mound breakwater (same alignment as Plan 7) installed in Noyo Cove west of the entrance.

bb. Plan 25 (Plate 14) entailed the offshore breakwater of Plan 24 with 37 ft of structure removed from the south end of the breakwater and installed on the north end. The breakwater length remained 525 ft.

cc. Plan 26 (Plate 14) involved the elements of Plan 25 but 75 ft of structure was removed from the south end of the breakwater resulting in a 450-ft-long structure.

dd. Plan 27 (Plate 15) included the 450-ft-long offshore breakwater of Plan 26 with a 187-ft-long shore-connected, rubble-mound breakwater installed south of the offshore structure.

ee. Plan 28 (Plate 15) included the 450-ft-long offshore breakwater of Plan 26 with a 150-ft-long offshore rubble-mound breakwater installed about 400 ft from and southwesterly of the existing entrance.

ff. Plan 29 (Plate 15) included the elements of Plan 28, but 37 ft of structure was removed from the north end of the most westerly offshore breakwater resulting in a cumulative length of 562 ft of structure.

gg. Plan 30 (Plate 15) involved the elements of Plan 29 but 75 ft of structure was removed from the south end of the most westerly offshore breakwater resulting in a cumulative structure length of 487 ft.

hh. Plan 31 (Plate 16) consisted of a rubble-mound breakwater originating at the head of the existing south jetty and extending about 600 ft parallel to an extension of the south channel line. From this point, the breakwater was extended

260 ft in a northwesterly direction (same alignment as Plan 16).

ii. Plan 32 (Plate 17) entailed a rubble-mound breakwater originating at the head of the existing south jetty and extending approximately 737 ft parallel to an extension of the south channel line. From this point, the breakwater extended northerly for a distance of 200 ft (same alignment as Plan 23).

jj. Plan 33 (Plate 17) included the elements of Plan 32 but the shoreward 447 ft of the breakwater was removed resulting in a structure length of 490 ft.

kk. Plan 34 (Plate 18) involved a 390-ft-long offshore rubble-mound breakwater in Noyo Cove installed approximately 500 ft west of the existing entrance.

ll. Plan 35 (Plate 18) entailed the elements of Plan 34, but the breakwater was extended 160 ft southeasterly resulting in a 550-ft-long structure.

mm. Plan 36 (Plate 18) consisted of a 585-ft-long rubble-mound breakwater originating at the large rock south of the south jetty and extending northwesterly.

nn. Plan 37 (Plate 19) involved a 525-ft-long rubble-mound breakwater that originated at the large rock south of the south jetty and curved across the entrance terminating approximately 200 ft from the existing north jetty head.

oo. Plan 38 (Plate 19) included the elements of Plan 37 with a 60-ft northerly extension of the structure resulting in a 585-ft-long breakwater.

pp. Plan 39 (Plate 19) entailed the elements of Plan 37 but the head of the breakwater was moved approximately 150 ft seaward and the length was extended resulting in a total structure length of 637 ft.

qq. Plan 40 (Plate 20) consisted of a 1,087-ft-long rubble-mound breakwater extending from the south shore of Noyo Cove (seaward portion of the cove) northwesterly along the existing shallow contours.

rr. Plan 41 (Plate 20) entailed an 800-ft-long submerged offshore rubble-mound breakwater approximately 1,200 ft seaward of the existing entrance. The crest elevation of this structure was -20 ft, and it extended from the -20 ft contour in the southern portion of the cove to the -20 ft contour in the northern portion of the cove.

ss. Plan 42 (Plate 21) involved two offshore rubble-mound breakwaters with a cumulative length of 870 ft. These structures were located seaward of the entrance to the cove.

tt. Plan 43 (Plate 22) consisted of a 637-ft-long dolosse breakwater installed on the same alignment as Plan 39.

Short-period wave-height tests

32. Wave-height tests for the various improvement plans were conducted using test waves from one or more of the directions listed in Paragraph 27. Tests involving certain proposed improvement plans were limited to the most critical direction of wave approach (i.e. west-northwest). The most promising plan of improvement (Plan 43) was tested comprehensively for waves from all five test directions. Wave-gage locations for each improvement plan are shown in the referenced plates.

Riverine sediment tracer tests

33. Riverine sediment tracer tests were limited to only the most promising breakwater plan (Plan 43) using river discharges ranging from 7,000 to 41,000 cfs. Tracer material was introduced into the model in the lower reaches of the river to represent bed-load sediment.

Long-period wave tests

34. Long-period (60 to 200 sec) wave tests were conducted for existing conditions and the best breakwater plan (Plan 43) with respect to short-period wave protection, using waves from the west direction. The two types of tests involved with investigating long-period waves are as follows:

- a. Frequency response tests involved the placement of wave sensors at strategic locations throughout the harbor (Plate 23) to measure the amplitude of the oscillations. By plotting the ratio of the measured wave height at each gage to the incident wave height (response factor) versus the wave periods tested, frequency response curves showing resonant peaks were obtained (Plates 24-33).
- b. Surface-float tests were conducted using small white squares of styrofoam confetti to determine oscillation patterns. The confetti was spread over the surface of the channel and basins, and subsequent movement by each wave period was observed. Through visual observations, the oscillation patterns and location of nodes and antinodes were determined.

Videotape

35. Videotape footage of the Noyo River and Harbor model was secured for existing conditions and Plan 43 showing the area under attack by storm waves approaching from the west-northwest test direction. This footage was furnished to SPL and SPN for use in briefings, public meetings, etc.

Results

36. In evaluating test results, the relative merits of the initial improvement plans were based on an analysis of measured wave heights at the river entrance and in the lower reaches of the river. Model wave heights (significant wave height or $H_{1/3}$) were tabulated to show measured values at selected locations. From these data the optimum improvement plan was selected and then subjected to riverine sediment tracer tests and long-period wave tests. These test results were compared to those of existing conditions to determine their merit or the impact of the improvement plan with respect to these conditions. The general movement of riverine sediment tracer material and subsequent deposits were shown in photographs. Arrows were superimposed onto these photographs to depict sediment movement patterns.

Existing conditions

37. Results of wave-height tests conducted in the lower reaches of the river for existing conditions are presented in Table 4 for test waves from the five directions with the +6.2 ft swl. Maximum wave heights were 15.0 ft in the entrance (Gage 1); 3.8 ft between the entrance and the first bend in the river (Gage 2); and 2.1 ft in the first bend of the river (Gage 3). All were for 19-sec, 22-ft test waves from west-northwest. Wave heights in the navigation channel upstream of the first bend in the river (Gages 4-6) ranged from <0.1 ft to 1.0 ft, and wave heights ranged from <0.1 ft to 0.2 ft in both Noyo Harbor (Gages 7 and 8) and Dolphin Marina (Gage 9). Typical wave patterns obtained for existing conditions are shown in Photos 1-5.

38. Design wave heights obtained along the center lines of the proposed breakwaters are presented in Table 5 for test waves from the five directions using the +6.2 ft swl. Maximum wave heights were 21.5 ft along the center line of the inner breakwater (Gage 11A) for 17-sec, 20-ft test waves from west-northwest; 31.6 ft along the center line of the outer north breakwater (Gage 2A) for 15-sec, 30-ft test waves from west; and 25.1 ft along the center line of the outer south breakwater (Gage 6A) for 19-sec, 22-ft test waves from west-northwest.

39. Additional design wave heights were secured along the center lines of the inner breakwaters (Gages 10A-17A) for test waves from west-northwest with the +7.0 ft swl and are presented in Table 6. Maximum wave heights along the center line of the easternmost breakwater location (Gages 10A-13A) were

20.4 ft, and maximum wave heights along the center line of the westernmost breakwater location (Gages 14A-17A) were 20.8 ft, both for 19-sec, 22-ft test waves.

40. Riverine sediment patterns secured with existing conditions installed for river discharges ranging from 7,000-41,000 cfs are shown in Photos 6-10. The 7,000 cfs discharge did not move the tracer material (Photo 6), but each sucessively larger flow resulted in sediment tracer deposits further seaward in Noyo Cove.

41. Long-period (60 to 200 sec) wave tests were conducted for existing conditions using waves from the west direction with a +7.0 ft swl. The gage arrangement for these tests is shown in Plate 23. To ensure accurate determination of incident wave heights, at the river entrance, the first 10 gages were placed in an array to measure nodes and antinodes of possible standing waves. The incident wave height was then calculated from the following relationship:

$$H_i = \frac{H_a + H_n}{2}$$

where

H_i = incident wave height

H_a = wave height at antinode

H_n = wave height at node

The test results obtained with gage array were used to determine incident wave heights in the entrance and corresponding wave-machine stroke settings. During the conduct of these tests, squares of styrofoam confetti were spread over the water surface and observed over the 60- to 200-sec period range. Areas of maximum horizontal movement (nodes) and minimum horizontal movement (antinodes) were identified through this series of visual observations. Wave gages were placed in antinodal areas. Measured wave heights at a particular gage location were divided by the incident wave height for that period to obtain the response factor or $R = H/H_i$. Frequency response (response factor versus wave period) curves were subsequently plotted for Gages 11-20.

42. Frequency response curves for existing conditions are shown in Plates 24-33. These test results indicate that resonant peaks (with amplification factors in excess of 1.0) will occur at various stations in Noyo River (Gages 11-15 and 19) for wave periods of 60, 90, 95, 110, 115, 130, 150, 155,

165, and 185 sec. Resonant peaks (with amplification factors in excess of 1.0) will occur in Noyo Harbor (Gages 16-18) for wave periods of 75, 95, 102.5, 115, and 155 sec. The maximum peak in Dolphin Marina (Gage 20) occurred for a 110-sec wave period with an amplification factor of 0.95.

Improvement plans

43. Wave-height data obtained with the -20 ft entrance channel installed (Plan 1) are presented in Table 7 for test waves from the five directions with the +6.2 ft swl. Maximum wave heights were 14.8 ft in the entrance (Gage 1) for 13-sec, 22-ft test waves from west-northwest; 2.8 ft between the entrance and the first bend in the river (Gage 2) for 13-sec, 22-ft test waves from west-northwest; and 2.7 ft in the first bend of the river (Gage 3) for 17-sec, 20-ft test waves from west-northwest. Wave heights in the navigation channel upstream of the first bend in the river (Gages 4-6) ranged from <0.1 ft to 1.0 ft; and wave heights ranged from <0.1 ft to 0.3 ft in both Noyo Harbor (Gages 7 and 8) and Dolphin Marina (Gage 9). Typical wave patterns with Plan 1 installed are shown in Photos 11-13.

44. Results of wave-height tests for Plan 2 (-15 ft entrance channel depth) are presented in Table 8 for test waves from all five directions and the +6.2 ft swl. Maximum wave heights were 14.5 ft in the entrance (Gage 1); 3.1 ft between the entrance and the first bend in the river (Gage 2); and 1.6 ft in the first bend of the river (Gage 3). All were for 19-sec, 22-ft test waves from west-northwest. Wave heights in the navigation channel upstream of the first bend in the river (Gages 4-6) ranged from <0.1 ft to 1.0 ft, and wave heights ranged from <0.1 ft to 0.2 ft in both Noyo Harbor (Gages 7 and 8) and Dolphin Marina (Gage 9). Representative wave patterns for Plan 2 are shown in Photos 14-16.

45. Wave-height data secured for Plans 3 and 3A for test waves from the west-northwest direction with the +7.0 ft swl are presented in Table 9. Maximum wave heights obtained for Plan 3 were 8.8 ft in the river entrance (Gage 1) for 15-sec, 30-ft test waves; 1.6 ft between the entrance and the first bend in the river (Gage 2) for 15-sec, 30-ft test waves; and 1.0 ft in the first bend of the river (Gage 3) for 17-sec, 28-ft test waves. For Plan 3A, maximum wave heights were 8.0 ft in the river entrance for 17-sec, 20-ft test waves; 1.4 ft between the entrance and the first bend in the river for 15-sec, 30-ft and 17-sec, 28-ft test waves; and 1.1 ft in the first bend of the river for 17-sec, 20-ft test waves. Wave heights in the navigation

channel upstream of the first bend in the river (Gages 4-6) ranged from <0.1 ft to 0.6 ft; and wave heights ranged from <0.1 ft to 0.2 ft in both Noyo Harbor (Gages 7 and 8) and Dolphin Marina (Gage 9) for both Plans 3 and 3A. Typical wave patterns secured for Plans 3 and 3A are shown in Photos 17 and 18.

46. Wave heights obtained for Plan 4 are presented in Table 10 for test waves from west-northwest with the +7.0 ft swl. Maximum wave heights obtained were 9.6 ft in the river entrance (Gage 1) for 17-sec, 20-ft test waves; 1.6 ft between the entrance and the first bend in the river (Gage 2) for 15-sec, 30-ft test waves; and 0.9 ft in the first bend of the river (Gage 3) for 17-sec, 28-ft test waves. Wave heights in the navigation channel upstream of the first bend in the river (Gages 4-6) ranged from <0.1 ft to 0.5 ft. Wave heights did not exceed 0.2 ft in Noyo Harbor (Gages 7 and 8) or 0.1 ft in Dolphin Marina (Gage 9). Wave patterns for Plan 4 are shown in Photo 19.

47. The improvement plans tested to this point were not successful in reducing wave heights to the established 4-ft criterion in the existing entrance. It was also noted that the installation/modification of the dolosse breakwater was difficult and time-consuming. As an expedient, until a promising plan was developed, breakwaters from this point were constructed with rock and had similar transmission coefficients as the dolosse structures. A comparison of the test results for dolosse and rock structures initially indicated the wave height at Gage 1 would vary only about 0.1 ft.

48. Wave-height tests were conducted with 21 rubble-mound breakwater configurations installed (Plans 5-23) for 15-sec, 30-ft test waves from west-northwest using the +7.0 ft swl. Results of these tests along with cumulative breakwater lengths are presented in Table 11. Wave heights obtained in the entrance (Gage 1) ranged from 3.4 to 8.7 ft. Cumulative breakwater lengths ranged from 370 to 1,125 ft in length. Some of these breakwater lengths were promising in regard to wave protection; however, it appeared that navigational difficulties may be experienced.

49. A meeting was held at Fort Bragg, CA, attended by representatives of SPL, SPN, CERC, US Coast Guard, and Noyo Harbor users, on 10 October 1985. At this meeting, Noyo Harbor users and Coast Guard representatives indicated that they preferred an entrance to the north of the proposed offshore breakwater as opposed to an entrance south of the structure. They also indicated that during extreme wave conditions they could tolerate a 6-ft wave between

the existing jetties, provided it was nonbreaking. As a result the wave height criterion at Gage 1 was increased. Considering the results of improvement plans tested to date, it was requested that additional breakwater configurations be investigated briefly before a plan was selected for detailed study.

50. Wave heights were obtained for 19 additional rubble-mound breakwater configurations (Plans 24-42) for 15-sec, 30-ft test waves from west-northwest with the +7.0 ft swl. Test results and cumulative breakwater lengths for these plans are presented in Table 12. Wave heights in the entrance (Gage 1) ranged from 4.3 to 12.0 ft, and cumulative breakwater lengths ranged from 390 to 1,087 ft in length.

51. Wave heights secured for Plan 43 are presented in Table 13 for test waves from all five directions and the +7.0 ft swl. Maximum wave heights were 6.4 ft in the river entrance (Gage 1) for 17-sec, 28-ft test waves from west-northwest and 17-sec, 20-ft test waves from west; 1.2 ft between the entrance and the first bend in the river (Gage 2) for 15-sec, 30-ft test waves from west-northwest; and 0.6 ft in the first bend of the river (Gage 3) for several test waves. Wave heights in the navigation channel upstream of the first bend in the river (Gages 4-6) ranged from <0.1 ft to 0.4 ft, and wave heights ranged from <0.1 ft to 0.1 ft in both Noyo Harbor (Gages 7 and 8) and Dolphin Marina (Gage 9). Typical wave patterns obtained for Plan 43 are shown in Photos 20-29.

52. Frequency response curves, based on long-period wave tests for Plan 43, are plotted on Plates 24-33. These tests indicate that resonant peaks (with amplification factors in excess of 1.0) will occur at various stations in Noyo River (Gages 11-15 and 19) for wave periods of 60, 80, 95, 105, 110, 125, 140, 150, 170, 180, and 185 sec. Resonant peaks (with amplification factors in excess of 1.0) will occur in Noyo Harbor (Gages 16-18) for wave periods of 95 and 110 sec. The maximum peak in Dolphin Marina (Gage 20) had an amplification factor of 0.8 and occurred for a 110-sec wave period.

53. Riverine sediment patterns with Plan 43 installed are shown in Photos 30-34 for river discharges ranging from 7,000-41,000 cfs. The 7,000-cfs river discharge did not move the tracer material (Photo 30), but each sucessively larger flow resulted in sediment tracer deposits further seaward in Noyo Cove. The presence of the Plan 43 structure directed the flow and sediment to the northern portion of the cove.

Discussion of test results

54. Results of wave-height tests for existing conditions indicated very rough and turbulent wave conditions in the entrance to Noyo River for storm waves from all directions. Wave heights up to 15 ft were recorded between the existing breakwaters. Also, many incident wave conditions resulted in breaking waves in the river entrance.

55. Deepening of the entrance channel did not prevent waves from breaking in the river entrance, considering all test wave conditions. Wave heights of 14.5 and 14.8 ft were secured for the -15 and -20 ft channel depths, respectively.

56. Results of wave-height tests for the initial improvement plan (Plan 3) revealed excessive wave heights in the river entrance (8.8 ft). Modifications to the original structure layout (Plans 3A and 4) were not effective in substantially reducing wave heights in the entrance. The 75-ft-long breakwater extension of Plan 3A resulted in 8.0-ft wave heights, and the additional 300-ft-long breakwater of Plan 4 produced 9.6-ft wave heights in the river entrance.

57. Results of wave-height tests conducted with the 40 expedient rubble-mound breakwater configurations (Plans 5-42) indicated that several of the test plans met the criteria with regard to wave heights in the entrance. However, some of the breakwater lengths were excessive, and they were not economically feasible to construct. Other test plans appeared to potentially create navigational hazards due to their close proximity to the existing structures. Considering all the rubble-mound test plans, the breakwater alignment of Plan 39 appeared to be optimum with regard to wave heights obtained in the entrance, economics, and navigation.

58. Results of wave-height tests for the 637-ft-long dolosse breakwater of Plan 43 (same alignment as Plan 39) indicated a maximum wave height of 6.4 ft in the river entrance. Visual observations revealed the waves were nonbreaking. These conditions were observed by representatives of the Noyo Harbor District and SPN during a conference at WES, and Plan 43 was selected as the optimum improvement plan tested with respect to wave protection, navigation, breakwater stability, and costs.

59. A comparison of long-period wave test results for existing conditions and Plan 43 indicates that the breakwater, in most cases, reduced long period wave energy in Noyo River and Harbor. Response peaks in general were

reduced slightly in both magnitude and width. The breakwater of Plan 43 should result in improved long-period wave conditions in the area.

60. A comparison of riverine sediment patterns obtained for existing conditions and Plan 43 indicates that the breakwater will not interfere with the movement of sediment seaward into Noyo Cove. The breakwater did, however, direct sediment to the northern portion of the cove as opposed to the center of the cove as was the case for existing conditions.

PART V: CONCLUSIONS

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

a. Existing conditions are characterized by very rough and turbulent wave conditions in the Noyo River entrance during periods of storm wave attack.

b. Deepening of the entrance channel will not significantly improve wave conditions in the existing river entrance, considering all test conditions.

c. The originally proposed breakwater location (Plan 3) resulted in excessive wave heights (8.8 ft) in the river entrance.

d. Of the 40 expedient rubble-mound (stone) breakwater plans (Plans 5-42) tested, the alignment of the 637-ft-long breakwater of Plan 39 appeared to be optimum with regard to wave protection, navigation, and economics.

e. The 637-ft-long dolosse breakwater of Plan 43 (same alignment as Plan 39) was selected as the optimum improvement plan for protection of the Noyo River entrance.

f. The breakwater configuration of Plan 43 will result in improved surge conditions due to long-period wave energy in Noyo River and harbor.

g. The breakwater configuration of Plan 43 will not interfere with the movement of riverine sediment seaward into Noyo Cove; however, the structure will direct sediment to the northern portion of the cove.

REFERENCES

Bottin, R. R., Jr. 1982. "Design for Flood Control, Wave Protection, and Prevention of Shoaling, Rogue River, Oregon; Hydraulic Model Investigation," Technical Report HL-82-18, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Bottin, R. R. Jr., and Chatham, C. E., Jr. 1975. "Design for Wave Protection, Flood Control, and Prevention of Shoaling, Cattaraugus Creek Harbor, New York; Hydraulic Model Investigation," Technical Report H-75-18, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Brasfeild, C. W., and Ball, J. W. 1967. "Expansion of Santa Barbara Harbor, California; Hydraulic Model Investigation," Technical Report No. 2-805, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Corson, W. D. 1985. "Pacific Coast Hindcast Deepwater Wave Information," WIS Report 14, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Cox, R. G. 1973. "Effective Hydraulic Roughness for Channels Having Bed Roughness Different from Bank Roughness; A State-of-the-Art Report," Miscellaneous Paper H-73-2, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Dai, Y. B., and Jackson, R. A. 1966. "Design for Rubble-Mound Breakwaters, Dana Point Harbor, California; Hydraulic Model Investigation," Technical Report No. 2-725, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Ebersole, B. A. 1984. "Refraction-Diffraction Model for Linear Water Waves," Journal of Waterway, Port, Coastal and Ocean Engineering, American Society of Civil Engineers.

Giles, M. L., and Chatham, C. E., Jr. 1974. "Remedial Plans for Prevention of Harbor Shoaling, Port Orford, Oregon; Hydraulic Model Investigation," Technical Report H-74-4, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Keulegan, G. H. 1950. "The Gradual Damping of a Progressive Oscillatory Wave with Distance in a Prismatic Rectangular Channel," (unpublished data), National Bureau of Standards, Washington, DC; prepared at request of Director, US Army Engineer Waterways Experiment Station, Vicksburg, MS, by letter dated 2 May 1950.

__________. 1973 (Feb). "Wave Transmission Through Rock Structures," Research Report H-73-1, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

LeMéhauté, B. 1965. "Wave Absorbers in Harbors," Contract Report No. 2-122, US Army Engineer Waterways Experiment Station, Vicksburg, MS, prepared by National Engineering Science Company, Pasadena, CA, under Contract No. DA-22-079-CIVENG-64-81.

Miller, I. E., and Peterson, M. S. 1953. "Roughness Standards for Hydraulic Models: Study of Finite Boundary Roughness in Rectangular Flumes," Technical Memorandum No. 2-364, Report 1, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Noda, E. K. 1972. "Equilibrium Beach Profile Scale-Model Relationship," Journal, Waterways, Harbors, and Coastal Engineering Division, American Society of Civil Engineers, Vol 98, No. WW4, pp 511-528.

Shore Protection Manual. 1984. 4th ed., 2 Vols, US Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, US Government Printing Office, Washington, DC.

Stevens, J. C., et al. 1942. "Hydraulic Models," Manuals of Engineering Practice No. 25, American Society of Civil Engineers, New York.

US Army Engineer District, San Francisco. 1979. "Plan of Study for Advance Engineering and Design, Noyo River and Harbor, Mendocino County, California," San Francisco, CA.

Table 1

Summary of Refraction and Shoaling Analysis for Noyo Harbor

Wave Period sec	Shallow-Water Azimuth, deg	Refraction* Coefficient	Shoaling** Coefficient	Wave-Height Adjustment Factor
		Northwest, 315 deg		
7	312.2	0.981	0.956	0.938
9	307.3	0.950	0.917	0.871
11	302.8	0.926	0.917	0.849
13	299.3	0.912	0.938	0.855
15	296.2	0.897	0.971	0.871
17	293.1	0.889	1.009	0.897
19	290.9	0.885	1.044	0.924
		West-Northwest, 292.5 deg		
7	292.5	0.998	0.956	0.954
9	291.3	0.992	0.917	0.910
11	289.8	0.993	0.917	0.911
13	288.4	0.996	0.938	0.934
15	287.1	1.006	0.971	0.977
17	285.7	1.003	1.009	1.012
19	284.5	1.010	1.044	1.054
		West, 270 deg		
7	270.0	1.000	0.956	0.956
9	270.2	0.995	0.917	0.912
11	270.0	0.992	0.917	0.910
13	270.1	0.981	0.938	0.920
15	270.4	0.973	0.971	0.945
17	270.5	0.972	1.009	0.981
19	270.6	0.975	1.044	1.018
		West-Southwest, 247.5 deg		
7	247.5	0.999	0.956	0.955
9	249.5	0.990	0.917	0.908
11	251.8	0.988	0.917	0.906
13	254.1	0.989	0.938	0.928
15	255.9	0.996	0.971	0.967
17	257.7	1.002	1.009	1.001
19	259.1	1.011	1.044	1.055
		Southwest, 225 deg		
7	225.8	0.988	0.956	0.945
9	229.5	0.953	0.917	0.874
11	234.2	0.929	0.917	0.852
13	238.4	0.919	0.938	0.862
15	242.4	0.903	0.971	0.877
17	245.7	0.891	1.009	0.899
19	248.4	0.882	1.044	0.921

* At approximate locations of wave generator in model.
** At 81-ft depth (75-ft pit elevation with 6-ft storm tide conditions superimposed).

Table 2

Estimated Magnitude of Deepwater Waves (Sea and Swell)

Approaching Noyo Harbor from the Directions Indicated

Wave Height ft	Occurrences* per Wave Period, sec 4.4-8.0	8.1-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	>18.2	Total
Northwest								
0-3.1	19	6						25
3.1-6.4	1,052	775	366	17		6		2,216
6.4-9.7	3,368	1,038	2,386	158	23	1		6,974
9.7-13.0	1,465	223	1,025	819	145	11	2	3,690
13.0-16.3	135	255	43	347	184	12		976
16.3-19.6	1	68	11	20	82	9		191
19.6-22.9		10	12	1	11	13		47
22.9-26.2			2					2
Total	6,040	2,375	3,845	1,362	445	52	2	14,121
West-Northwest								
0-3.1	5	29						34
3.1-6.4	206	880	925	60	8	5		2,084
6.4-9.7	302	656	4,446	1,578	182	16	9	7,189
9.7-13.0	243	79	1,870	3,649	1,190	78		7,109
13.0-16.3	18	138	347	1,746	1,629	172	1	4,051
16.3-19.6		46	65	301	944	289	1	1,646
19.6-22.9			14	26	157	202		399
22.9-26.2			1		45	72		118
26.2-29.5					2	8		10
29.5-32.8						9		9
>32.8						4		4
Total	774	1,828	7,668	7,360	4,157	855	11	22,653

(Continued)

* Occurrences compiled for period 1956-1975.

(Sheet 1 of 3)

Table 2 (Continued)

Wave Height ft	Occurrences per Wave Period, sec							
	4.4-8.0	8.1-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	>18.2	Total
				West				
0-3.1								
3.1-6.4	51	135	157	22				365
6.4-9.7	52	218	927	358	34	5		1,594
9.7-13.0	80	63	803	1,256	356	19		2,577
13.0-16.3	20	55	289	1,111	834	46		2,355
16.3-19.6	1	30	70	286	704	48		1,139
19.6-22.9		1	14	65	258	92		430
22.9-26.2			2	3	52	65		122
26.2-29.5			2	4	2	4		12
29.5-32.8			3		2	10		15
Total	204	502	2,267	3,105	2,242	289		8,609
				West-Southwest				
0-3.1								
3.1-6.4	14	34	1					49
6.4-9.7	64	113	127	9	2			315
9.7-13.0	97	65	297	141	25	2		627
13.0-16.3	33	89	255	285	88	3		753
16.3-19.6	2	72	104	182	148	5		513
19.6-22.9		8	63	79	124	8		282
22.9-26.2			13	26	31	12		82
26.2-29.5			5	4	10			19
29.5-32.8			1	4	9			14
>32.8				1				1
Total	210	381	866	731	437	30		2,655

(Continued)

Table 2 (Concluded)

Wave Height ft	Occurrences per Wave Period, sec 4.4-8.0	8.1-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	>18.2	Total
				Southwest				
0-3.1								
3.1-6.4	1		3					4
6.4-9.7	47	11	9	1				68
9.7-13.0	92	40	71	32	1			236
13.0-16.3	57	112	39	44	17			269
16.3-19.6	4	105	77	48	18			252
19.6-22.9		19	117	41	33	2		212
22.9-26.2			47	11	11			69
26.2-29.5			9	3	5			17
29.5-32.8			2	6	3			11
>32.8			1	6	1			8
Total	201	287	375	192	89	2		1,146

(Sheet 3 of 3)

Table 3

Estimated Magnitude of Shallow-Water Waves (Sea and Swell)

Approaching Noyo Harbor from the Directions Indicated

Wave Height ft	Occurrences* per Wave Period, sec 4.4-8.0	8.1-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	>18.2	Total
				Northwest				
1-4	19	6						25
4-6		775	366	17		6		1,164
6-8	1,052							1,052
8-10	3,368	1,038	2,386	158	23	1		6,974
10-12		223	1,025	819	145	11	2	2,225
12-14	1,465		43	347				1,855
14-16	135	255			184	12		586
16-18		68	11	20	82	9		190
18-20	1	10	12	1	11			35
20-22						13		13
22-24			2					2
Total	6,040	2,375	3,845	1,362	445	52	2	14,121

(Continued)

* Occurrences compiled for period 1956-1975.

Table 3 (Continued)

Wave Height ft	Occurrences per Wave Period, sec 4.4-8.0	8.1-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	>18.2	Total
				West-Northwest				
1-4	5	29						34
4-6		880	925	60				1,865
6-8	206				8	5		219
8-10	302	656	4,446	1,578	182	16		7,180
10-12		79	1,870				9	1,958
12-14	243			3,649	1,190	78		5,160
14-16	18	138	347	1,746	1,629			3,878
16-18		46	65			172	1	284
18-20				301	944	289		1,534
20-22			14	26			1	41
22-24			1		157	202		360
24-26					45			45
26-28						72		72
28-30					2	8		10
30-32								0
32-34						9		9
>34						4		4
Total	774	1,828	7,668	7,360	4,157	855	11	22,653

(Continued)

(Sheet 2 of 5)

Table 3 (Continued)

Wave Height ft	Occurrences per Wave Period, sec 4.4-8.0	8.1-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	>18.2	Total
				West				
4-6		135	157	22				314
6-8	51							51
8-10	52	218	927	358	34	5		1,594
10-12		63	803	1,256				2,122
12-14	80				356	19		455
14-16	20	55	289	1,111	834	46		2,355
16-18		30	70					100
18-20	1			286	704	48		1,039
20-22		1	14	65	258			338
22-24			2			92		94
24-26				3	52	65		120
26-28			2		2			4
28-30			3	4	2	4		13
30-32								
32-34						10		10
Total		204	502	2,267	3,105	2,242	289	8,609

(Continued)

Table 3 (Continued)

Wave Height (ft)	Occurrences per Wave Period, sec 4.4-8.0	8.1-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	>18.2	Total
				West-Southwest				
4-6		34	1					35
6-8	14							14
8-10	64	113	127	9	2			315
10-12		65	297					362
12-14	97			141	25	2		265
14-16	33	89	255	285	88			750
16-18		72	104			3		179
18-20	2			182	148	5		337
20-22		8	63	79				150
22-24			13		124	8		145
24-26				26	31			57
26-28			5	4		12		21
28-30			1		10			11
30-32				4	9			13
32-34				1				1
Total		210	381	866	731	437	30	2,655

(Continued)

(Sheet 4 of 5)

Table 3 (Concluded)

Wave Height ft	Occurrences per Wave Period, sec 4.4-8.0	8.1-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	>18.2	Total
				Southwest				
4-6			3					3
6-8	1							1
8-10	47	11	9	1	1			69
10-12		40	71	32				143
12-14	92		39					131
14-16	57	112		44	17			230
16-18		105	77	48				230
18-20	4		117	41	18			180
20-22		19			33	2		54
22-24			47	11	11			69
24-26			9	3	5			17
26-28			2					2
28-30			1	6	3			10
30-32				6	1			7
Total	201	287	375	192	89	2		1,146

(Sheet 5 of 5)

Table 4

Wave Heights for Existing Conditions, swl = +6.2 ft

Test Wave Period sec	Test Wave Height ft	Wave Height, ft, at Indicated Gage Location: Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9
					West					
7	8	2.4	0.4	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	14	4.0	0.7	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	7.4	1.5	0.6	0.3	0.2	0.1	<0.1	<0.1	<0.1
9	6	1.8	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	4.8	0.4	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	5.3	0.7	0.2	0.2	0.2	0.1	<0.1	<0.1	<0.1
11	6	2.0	0.3	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	3.8	0.5	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	24	7.2	1.0	0.3	0.2	0.1	<0.2	0.1	0.2	0.1
13	6	3.3	0.5	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	6.7	1.3	0.4	0.3	0.2	<0.1	<0.1	<0.1	<0.1
	20	8.8	1.8	0.7	0.6	0.3	0.2	<0.1	<0.1	0.1
15	10	3.6	0.6	0.3	0.1	0.1	<0.1	<0.1	<0.1	<0.1
	20	10.1	2.8	1.4	0.6	0.3	0.1	<0.1	<0.1	0.1
17	6	4.9	1.0	0.6	0.3	0.3	0.2	0.2	0.2	0.1
	12	7.0	1.4	1.1	0.4	0.4	0.3	0.2	0.3	0.3
	22	8.6	1.4	0.8	0.5	0.3	0.2	0.1	0.1	0.1
19	12	4.0	0.8	0.4	0.3	0.2	0.1	<0.1	<0.1	0.1
					West-Northwest					
7	8	1.9	0.3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	16	13.0	1.7	0.7	0.3	0.1	<0.1	<0.1	<0.1	<0.1
9	6	1.9	0.3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	10	3.9	0.6	<0.1	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	18	10.1	1.4	0.4	0.3	0.2	<0.1	<0.1	<0.1	<0.1
11	6	1.7	0.3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	4.9	0.6	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	24	11.2	1.5	0.5	0.4	0.2	0.1	<0.1	<0.1	<0.1
13	6	2.4	0.4	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	14	7.8	1.7	0.6	0.4	0.2	<0.1	<0.1	<0.1	<0.1
	22	9.8	1.8	0.8	0.3	0.2	0.1	<0.1	0.1	0.1
15	10	4.5	0.9	0.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	20	12.3	2.6	1.3	0.5	0.3	0.1	<0.1	<0.1	<0.1
	30	13.0	2.8	1.6	0.7	0.4	0.1	0.1	0.1	0.1
17	10	7.2	1.5	0.9	0.8	0.3	0.1	<0.1	<0.1	<0.1
	20	11.4	2.3	1.1	0.6	0.4	0.2	0.1	0.2	0.2
	28	12.4	2.6	1.7	0.8	0.5	0.3	0.2	0.2	0.2
19	12	7.0	1.8	0.7	0.3	0.3	0.1	<0.1	0.1	0.1
	22	15.0	3.8	2.1	1.0	0.3	0.2	0.1	0.1	0.1

(Continued)

Table 4 (Continued)

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
		Wave Height, ft, at Indicated Gage Location								
					West					
7	8	6.2	1.2	0.4	0.2	<0.1	<0.1	<0.1	<0.1	<0.1
	14	6.9	1.3	0.4	0.3	0.2	0.1	0.1	0.1	0.1
	20	7.2	0.8	0.2	0.1	0.2	0.1	0.1	0.1	0.1
9	6	2.3	0.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	8.0	1.2	0.5	0.4	0.2	<0.1	<0.1	<0.1	<0.1
	22	8.1	0.8	0.2	0.2	0.2	0.1	<0.1	0.1	0.1
11	6	3.4	0.7	0.2	0.1	0.1	<0.1	<0.1	<0.1	<0.1
	12	9.8	1.8	0.7	0.3	0.2	<0.1	<0.1	<0.1	<0.1
	18	11.0	1.7	0.7	0.3	0.3	0.1	<0.1	0.1	0.1
	30	8.1	0.9	0.3	0.2	0.2	0.3	0.1	0.1	<0.1
13	6	3.4	0.6	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	7.9	1.5	0.6	0.4	0.3	0.1	<0.1	<0.1	<0.1
	20	12.3	2.2	0.9	0.5	0.3	0.1	<0.1	0.1	<0.1
	30	12.6	2.4	1.1	0.4	0.3	0.2	0.1	<0.1	0.1
15	10	3.8	0.7	0.3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	11.1	2.0	0.8	0.4	0.3	0.1	<0.1	<0.1	<0.1
	30	11.9	2.4	1.2	0.5	0.4	0.2	0.1	0.1	0.1
17	10	4.6	1.0	0.6	0.2	0.2	<0.1	<0.1	<0.1	<0.1
	20	11.5	3.4	1.7	0.6	0.4	0.2	0.1	0.1	0.1
	28	11.2	2.4	1.1	0.6	0.4	0.2	0.1	0.1	0.2
					West-Southwest					
7	8	7.5	1.3	0.4	0.2	<0.1	<0.1	<0.1	<0.1	<0.1
	14	4.6	0.6	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	8.1	1.4	0.4	0.2	0.2	0.1	<0.1	0.1	0.1
9	6	2.8	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	6.6	1.0	0.4	0.4	0.2	<0.1	<0.1	<0.1	<0.1
	22	10.6	1.6	0.5	0.4	0.2	0.1	<0.1	0.1	0.2
11	10	7.0	1.2	0.4	0.2	0.1	<0.1	<0.1	<0.1	0.1
	20	11.5	1.3	0.5	0.2	0.2	0.1	<0.1	0.1	0.1
	30	9.4	1.3	0.5	0.4	0.2	0.2	0.1	0.2	0.2
13	10	9.0	1.8	0.8	0.4	0.3	0.1	<0.1	<0.1	<0.1
	20	9.0	1.4	0.5	0.3	0.2	0.1	<0.1	<0.1	<0.1
	32	9.6	1.9	0.7	0.4	0.3	0.2	0.1	0.2	0.2
15	10	8.0	1.8	0.7	0.5	0.2	0.1	<0.1	<0.1	<0.1
	20	11.6	2.1	0.8	0.6	0.3	0.1	<0.1	0.1	0.1
	32	10.8	2.2	1.0	0.7	0.4	0.2	0.1	0.2	0.2
17	14	11.0	2.9	1.2	0.5	0.3	0.1	0.1	0.1	0.1
	20	9.3	2.0	0.9	0.4	0.3	0.1	0.1	0.1	<0.1
	28	10.9	2.1	1.1	0.6	0.4	0.2	0.2	0.1	0.2

(Continued)

Table 4 (Concluded)

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
					Southwest					
7	8	4.4	1.1	0.3	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	14	5.7	1.0	0.3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	5.1	0.7	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
9	10	6.7	0.9	0.3	0.3	0.2	<0.1	<0.1	<0.1	<0.1
	16	8.7	1.2	0.3	0.3	0.2	0.1	0.1	0.1	0.2
	22	9.6	1.3	0.4	0.3	0.2	<0.1	<0.1	<0.1	<0.1
11	6	3.3	0.5	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	14	9.6	1.5	0.4	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	20	6.5	1.1	0.3	0.2	0.1	0.1	<0.1	<0.1	<0.1
	30	9.1	1.1	0.4	0.3	0.2	0.2	<0.1	0.1	0.1
13	10	9.5	1.8	0.5	0.3	0.2	<0.1	<0.1	<0.1	<0.1
	20	9.3	1.8	0.6	0.3	0.2	0.1	<0.1	<0.1	0.1
	32	7.9	1.3	0.5	0.3	0.2	0.2	<0.1	0.1	0.1
15	10	8.0	1.6	0.6	0.5	0.2	<0.1	<0.1	<0.1	<0.1
	20	9.5	1.9	0.7	0.5	0.3	0.1	<0.1	<0.1	0.1
	32	10.5	2.3	1.0	0.6	0.4	0.2	0.1	0.1	0.1
17	22	9.7	2.3	1.1	0.5	0.4	0.2	0.1	0.1	0.1

(Sheet 3 of 3)

Table 5

Wave Heights Obtained at Various Locations Along Center Lines of Proposed

Breakwaters (Structures not in Place), swl = +6.2 ft

Test Wave Period sec	Test Wave Height ft	Gage 1A	Gage 2A	Gage 3A	Gage 4A	Gage 5A	Gage 6A	Gage 7A	Gage 8A	Gage 9A	Gage 10A	Gage 11A	Gage 12A	Gage 13A
							Northwest							
7	8	7.6	4.4	5.5	6.6	5.9	4.2	6.8	6.8	6.6	2.2	3.1	3.4	4.1
	14	12.8	11.5	8.9	11.7	9.8	17.0	14.3	10.4	6.2	4.3	7.2	7.1	9.5
	20	13.4	13.0	16.1	21.2	19.5	15.3	11.4	11.3	8.4	14.0	12.4	12.9	12.5
9	6	4.8	3.7	2.8	1.8	1.4	1.9	3.8	4.1	6.4	1.4	1.9	2.1	1.3
	12	10.0	8.2	10.5	10.2	8.3	10.0	19.0	15.0	7.6	6.9	7.1	9.6	9.1
	20	16.4	14.9	12.2	13.8	19.6	18.9	15.2	12.3	8.3	13.7	11.6	13.2	14.4
11	6	3.3	1.6	2.8	2.6	1.8	2.5	2.4	4.6	6.9	1.3	2.2	2.2	1.3
	12	9.9	5.9	6.9	8.3	8.1	10.9	9.7	11.0	7.9	3.8	5.6	4.6	4.2
	24	14.9	17.1	15.1	22.6	19.5	17.5	14.9	12.7	8.2	14.6	17.0	14.9	12.9
13	6	6.9	5.2	4.9	4.6	4.6	5.4	7.1	7.4	10.4	2.7	2.2	2.7	3.2
	12	11.2	10.1	7.3	7.8	12.0	12.6	16.9	17.5	7.1	6.6	6.3	6.4	7.4
	20	16.7	19.4	13.0	15.5	21.2	17.9	18.3	11.7	7.1	16.6	13.2	15.7	14.9
15	10	5.7	7.7	4.2	4.8	5.7	7.4	12.5	14.1	8.4	3.3	3.1	3.9	5.8
	20	14.7	16.1	11.7	14.7	15.1	17.8	17.6	13.8	6.9	11.2	10.3	11.7	13.6
17	6	6.9	6.7	5.1	6.3	5.3	6.2	9.4	12.9	10.1	4.5	4.7	4.5	3.5
	12	10.0	11.8	8.3	11.5	8.4	14.1	16.8	13.1	11.1	7.7	7.1	8.7	12.6
	22	16.7	15.5	16.6	19.1	18.7	18.4	17.2	14.3	8.1	14.8	14.3	14.5	15.6
19	12	10.0	9.2	9.7	10.5	9.8	11.3	13.8	12.3	7.5	6.9	6.6	7.6	10.6

(Continued)

Table 5 (Continued)

Test Wave		Wave Height, ft, at Indicated Gage Location												
Period sec	Height ft	Gage 1A	Gage 2A	Gage 3A	Gage 4A	Gage 5A	Gage 6A	Gage 7A	Gage 8A	Gage 9A	Gage 10A	Gage 11A	Gage 12A	Gage 13A
						West-Northwest								
7	8	3.8	3.4	5.1	5.7	4.3	5.9	11.5	9.0	7.1	5.0	3.9	5.2	7.5
	16	15.1	12.3	14.4	15.1	16.5	17.4	15.1	13.0	6.1	10.6	13.1	15.2	12.6
9	6	2.1	2.7	2.3	3.8	3.5	5.5	7.1	5.7	7.3	2.3	2.0	2.5	1.8
	10	6.5	4.5	4.1	7.4	8.7	12.6	15.4	17.2	5.8	4.9	4.9	6.4	7.4
	18	18.6	14.4	14.7	21.9	23.0	17.3	13.9	14.3	7.4	13.8	12.2	13.8	13.3
11	6	3.1	2.1	2.4	2.2	3.2	5.5	5.2	7.4	8.6	1.1	2.6	2.1	1.6
	12	9.3	5.9	7.0	8.3	9.8	12.6	16.8	12.2	6.1	5.7	7.2	6.5	10.8
	24	21.2	17.2	25.2	23.0	15.8	17.0	13.6	11.7	7.2	16.9	18.3	15.9	13.8
13	6	3.2	3.4	2.8	4.3	5.5	5.9	9.7	10.3	8.4	1.9	1.9	2.3	2.8
	14	9.6	11.6	10.9	17.0	19.2	12.9	12.0	14.8	6.3	11.7	10.3	11.6	13.5
	22	24.3	20.6	24.5	24.0	16.5	16.2	13.6	13.0	8.2	14.4	13.0	13.7	14.8
15	10	3.7	5.7	5.0	6.5	5.9	10.2	12.9	10.3	9.5	3.0	4.4	6.1	5.6
	20	14.2	15.5	10.8	17.9	14.8	16.1	18.4	14.5	6.8	14.0	12.2	14.6	15.4
	30	17.5	17.9	18.2	23.3	20.4	21.2	16.7	13.2	8.7	16.7	18.5	17.8	13.8
17	10	11.2	10.9	9.7	10.3	9.1	12.4	11.7	13.1	9.2	9.3	8.2	6.3	7.4
	20	16.5	16.2	19.4	23.4	20.7	19.6	19.0	11.6	7.8	20.1	21.5	18.3	14.0
	28	17.8	22.0	24.4	24.0	21.8	22.2	19.8	12.8	8.7	21.1	18.9	18.6	15.3
19	12	10.4	11.2	10.8	9.9	8.9	10.6	12.7	14.3	5.9	7.2	6.7	8.0	9.6
	22	22.3	22.0	21.6	22.3	21.4	25.1	19.3	12.8	8.7	14.4	18.1	20.8	15.6

(Continued)

Table 5 (Continued)

Test Wave		Wave Height, ft, at Indicated Gage Location												
Period sec	Height ft	Gage 1A	Gage 2A	Gage 3A	Gage 4A	Gage 5A	Gage 6A	Gage 7A	Gage 8A	Gage 9A	Gage 10A	Gage 11A	Gage 12A	Gage 13A
							West							
7	8	6.9	9.0	6.4	7.4	8.0	8.8	9.6	9.8	4.9	3.2	4.9	5.4	7.4
	14	10.5	13.8	15.7	16.9	8.3	10.0	9.3	9.8	7.9	11.7	10.9	11.0	12.0
	20	18.1	18.2	21.9	16.4	13.9	12.8	11.0	10.3	6.3	12.0	10.9	12.1	12.2
9	6	5.3	4.0	3.3	4.4	3.3	5.6	4.3	4.6	6.5	2.7	2.4	2.8	2.6
	12	14.4	14.1	14.0	11.7	15.7	14.5	10.6	8.3	8.2	12.7	10.7	12.2	12.2
	22	16.7	22.7	26.6	17.3	15.5	12.4	14.1	11.6	7.5	12.4	12.0	15.3	13.2
11	6	5.6	3.8	4.1	3.9	4.3	4.9	5.1	5.0	6.5	3.0	3.3	3.7	3.3
	12	14.9	10.9	12.3	13.4	7.4	14.3	17.8	10.3	6.3	8.7	11.2	11.6	11.6
	18	20.5	23.0	28.9	22.7	18.3	16.5	12.3	12.0	8.2	13.9	14.2	12.8	13.6
	30	25.7	23.5	21.0	19.5	15.3	14.5	13.3	14.3	8.2	9.6	10.4	11.1	11.8
13	6	5.2	3.2	4.2	5.8	5.9	5.4	6.3	12.6	7.7	3.4	3.5	4.5	6.1
	12	9.3	8.7	8.4	10.3	9.6	13.8	15.4	13.4	6.0	6.5	7.1	8.7	12.1
	20	17.4	15.7	19.2	19.6	16.3	15.5	13.5	10.5	7.5	12.4	12.4	13.0	12.6
	30	22.6	28.7	30.9	17.0	14.8	15.2	13.1	14.3	8.1	11.9	8.8	10.1	11.4
15	10	9.7	9.0	8.2	10.5	8.3	13.3	15.5	12.9	8.4	4.7	6.3	8.6	11.3
	20	16.1	28.8	26.5	19.3	18.5	17.3	15.0	12.6	7.8	11.8	11.0	13.8	12.4
	30	23.9	31.6	27.3	21.0	17.7	17.2	16.1	12.9	7.5	13.0	11.0	13.6	13.7
17	10	7.3	9.6	12.4	7.2	11.0	13.3	15.9	12.3	7.0	7.5	8.3	9.2	10.0
	20	18.6	26.9	22.6	15.4	17.7	17.5	14.9	12.0	7.2	10.8	11.3	11.9	11.8
	28	21.7	22.0	20.1	21.0	20.5	19.3	16.2	14.0	7.8	16.0	14.7	16.3	14.6

(Continued)

Table 5 (Continued)

Test Wave		Wave Height, ft, at Indicated Gage Location												
Period sec	Height ft	Gage 1A	Gage 2A	Gage 3A	Gage 4A	Gage 5A	Gage 6A	Gage 7A	Gage 8A	Gage 9A	Gage 10A	Gage 11A	Gage 12A	Gage 13A
							West-Southwest							
7	8	4.3	4.3	6.2	7.7	8.5	7.4	6.0	8.4	6.1	5.2	5.2	4.7	4.8
	14	10.9	9.1	19.3	13.4	8.5	12.9	7.9	9.1	6.9	7.6	6.0	5.3	6.1
	20	15.4	20.9	18.3	16.2	12.4	10.7	9.2	10.5	7.9	13.2	14.0	10.0	8.1
9	6	4.1	3.6	4.4	2.3	3.1	6.7	6.9	8.0	8.7	1.5	2.5	2.1	3.5
	12	16.5	10.1	8.4	14.1	13.0	14.9	9.5	8.3	6.8	10.3	10.9	9.6	9.4
	22	20.7	18.7	19.5	19.9	13.2	14.2	12.5	9.8	7.3	16.8	17.3	14.9	11.6
11	10	11.3	7.1	4.7	7.7	11.2	12.7	14.3	11.4	5.8	7.6	8.5	9.9	11.7
	20	20.1	24.3	18.7	18.1	18.6	16.8	13.7	10.1	7.7	12.7	15.2	12.0	11.9
	30	21.4	20.8	22.0	18.5	13.7	11.4	12.4	13.4	8.1	9.8	12.1	12.1	13.2
13	10	6.7	6.9	9.7	10.5	13.8	14.8	13.5	11.6	6.0	10.0	9.0	10.9	13.0
	20	18.4	22.9	21.9	15.5	15.2	14.4	13.8	11.8	7.8	11.2	8.6	8.0	10.5
	32	24.1	20.6	23.3	18.2	14.7	14.3	12.1	11.4	7.6	15.1	16.2	14.1	13.5
15	10	4.5	6.9	7.3	7.8	11.0	16.7	12.6	10.5	6.5	8.9	7.7	8.8	10.5
	20	17.9	22.3	17.3	17.5	15.6	17.1	13.5	10.5	7.4	9.5	8.6	9.1	9.2
	32	23.7	23.4	24.0	17.8	16.0	14.0	14.5	14.3	8.0	12.4	11.1	11.2	11.3
17	14	112.3	18.2	15.1	20.9	18.8	15.5	11.0	9.7	8.1	10.8	10.6	11.0	10.9
	20	21.5	22.2	19.4	16.8	14.3	15.0	13.9	13.0	7.8	10.2	11.7	11.5	11.2
	28	21.5	23.1	19.1	16.0	14.4	13.4	11.3	11.9	7.7	11.1	11.2	11.9	12.9

(Continued)

Table 5 (Concluded)

Test Wave		Wave Height, ft, at Indicated Gage Location												
Period sec	Height ft	Gage 1A	Gage 2A	Gage 3A	Gage 4A	Gage 5A	Gage 6A	Gage 7A	Gage 8A	Gage 9A	Gage 10A	Gage 11A	Gage 12A	Gage 13A
							Southwest							
7	8	5.8	7.2	8.0	6.2	8.6	8.3	7.2	6.1	8.4	5.1	5.1	5.0	5.7
	14	11.7	13.4	15.5	12.1	10.5	11.2	6.3	6.3	5.1	6.7	6.2	6.1	6.1
	20	14.6	15.6	14.3	11.1	11.3	9.7	9.2	9.5	6.4	8.7	6.2	6.1	6.1
9	10	6.3	7.9	7.5	8.8	11.8	12.3	6.9	6.0	7.2	8.9	8.3	10.6	7.4
	16	16.2	22.0	19.0	16.1	14.5	11.9	9.7	10.0	8.3	10.4	12.6	11.2	9.6
	22	17.0	21.7	18.5	17.0	14.4	13.4	10.3	8.1	7.1	13.0	12.4	11.9	10.8
11	6	1.9	1.3	3.2	7.4	5.9	6.0	9.0	8.0	5.0	4.5	4.5	5.7	7.4
	14	21.7	20.4	21.9	13.2	11.8	13.3	12.0	9.5	7.7	8.6	9.2	7.5	7.1
	20	22.0	21.4	16.3	12.1	11.8	12.6	10.8	12.9	8.1	11.5	11.7	10.0	9.9
	30	22.7	19.9	20.9	13.1	11.9	11.2	11.6	11.5	8.1	10.6	11.2	8.9	10.5
13	10	10.5	7.5	9.8	14.7	12.6	12.8	10.5	8.5	6.1	11.9	9.1	8.9	10.3
	20	23.1	18.8	20.7	17.5	14.3	13.0	10.1	8.8	7.3	12.6	10.6	10.4	12.1
	32	22.8	20.4	20.3	15.9	12.2	11.7	11.3	12.3	8.5	14.6	13.3	12.0	12.2
15	10	8.7	8.7	11.5	11.5	15.6	15.9	12.1	10.3	6.6	9.7	8.0	9.7	9.8
	20	18.8	19.7	29.9	21.2	23.2	19.9	16.1	13.9	8.5	14.1	12.7	10.3	8.8
	32	20.4	15.2	16.7	17.7	16.6	15.8	11.1	9.2	8.3	11.7	11.8	13.1	12.4
17	22	21.7	16.4	17.0	14.4	14.4	12.9	11.8	14.2	8.1	10.1	9.7	9.2	9.3

Table 6

Wave Heights Obtained Along Center Lines of Proposed Inner Breakwaters From West-Northwest, swl = +7.0 ft

Test Wave		Wave Height, ft, at Indicated Gage Location							
Period sec	Height ft	Gage 10A	Gage 11A	Gage 12A	Gage 13A	Gage 14A	Gage 15A	Gage 16A	Gage 17A
7	8	3.1	3.0	5.2	5.3	2.5	3.6	6.3	5.4
	12	7.7	9.3	9.9	10.1	8.3	9.7	10.1	11.2
	16	12.8	12.1	12.4	13.7	11.3	11.5	13.2	15.4
9	6	2.6	2.1	2.5	2.9	3.0	2.3	2.8	3.1
	10	3.9	4.6	6.1	7.5	3.2	5.5	6.5	8.9
	14	9.8	11.3	11.8	13.2	12.3	11.2	12.1	13.9
	18	19.5	16.2	16.2	13.1	17.9	19.0	15.5	13.2
11	6	2.4	2.8	3.6	3.4	2.9	2.9	2.3	3.2
	12	7.8	8.8	6.9	12.8	7.8	7.9	8.2	14.0
	18	13.7	15.6	13.7	13.2	14.2	13.9	14.8	12.5
	24	19.7	17.5	15.4	13.1	17.3	17.3	17.1	17.4
13	6	2.3	2.8	3.1	3.0	2.5	3.1	2.8	2.8
	10	5.5	6.2	6.4	6.2	6.7	6.5	8.0	7.7
	14	13.6	11.7	12.2	12.8	15.8	13.7	11.1	10.1
	18	15.9	14.8	15.3	16.2	14.8	18.0	16.4	16.0
	22	18.2	17.9	18.2	15.4	18.7	18.4	18.2	16.1
15	10	5.5	5.3	5.6	5.2	5.6	5.0	6.3	4.8
	15	9.6	9.1	12.1	12.5	10.8	10.2	11.0	11.5
	20	16.7	14.6	17.8	15.3	15.1	16.7	16.2	15.6
	25	17.1	20.3	20.0	15.4	16.4	19.4	20.2	18.3
	30	15.9	15.6	17.9	15.5	16.5	17.7	20.0	18.8
17	10	7.5	7.2	7.1	10.4	8.2	8.3	7.9	10.5
	16	13.2	15.0	16.0	19.0	14.0	14.7	13.7	17.3
	20	20.3	19.1	17.5	18.5	18.2	16.6	16.8	18.1
	24	20.0	17.0	17.8	17.7	17.9	19.3	17.5	17.1
	28	19.5	17.9	19.1	18.5	18.6	18.8	18.1	18.1
19	12	9.0	7.9	8.7	11.6	9.6	8.2	7.7	12.9
	17	13.0	10.4	13.1	16.8	12.1	11.2	12.4	19.5
	22	19.5	20.4	20.2	18.8	19.2	20.3	20.1	20.8

Table 7

Wave Heights for Plan 1, swl = +6.2 ft

Test Wave		Wave Height, ft, at Indicated Gage Location								
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9
						Northwest				
7	8	1.5	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	14	3.7	0.3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	6.6	0.7	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
9	6	0.7	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	4.3	0.4	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	7.0	0.9	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
11	6	1.4	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	3.5	0.5	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	24	9.6	1.4	0.4	0.3	0.2	0.1	0.1	0.2	<0.1
13	6	2.6	0.4	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	5.2	0.7	0.2	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	20	9.6	1.7	0.9	0.4	0.3	0.1	<0.1	<0.1	0.1
15	10	3.8	0.8	0.4	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	20	10.2	1.8	1.5	0.6	0.4	0.1	<0.1	<0.1	0.1
17	6	5.0	0.7	0.3	0.2	0.2	<0.1	<0.1	<0.1	<0.1
	12	6.3	0.8	0.4	0.2	0.2	<0.1	<0.1	<0.1	0.1
	22	7.8	0.9	0.9	0.4	0.3	0.2	0.2	0.1	0.1
19	12	4.0	0.6	0.3	0.2	0.1	<0.1	<0.1	<0.1	0.1
						West-Northwest				
7	8	4.1	0.7	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	16	7.9	1.3	0.4	0.2	0.1	<0.1	<0.1	<0.1	<0.1
9	6	1.7	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	10	4.0	0.5	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	18	9.5	1.1	0.3	0.2	0.2	<0.1	<0.1	<0.1	<0.1
11	6	1.7	0.3	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	4.5	0.7	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	24	9.8	1.5	0.6	0.3	0.2	<0.1	<0.1	0.1	0.1
13	6	2.4	0.5	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	14	7.7	1.5	0.6	0.3	0.3	0.1	<0.1	<0.1	<0.1
	22	14.8	2.8	1.8	0.7	0.5	0.1	0.1	0.1	0.1
15	10	4.5	1.1	0.5	0.3	0.2	<0.1	<0.1	<0.1	<0.1
	20	9.5	1.8	1.3	0.5	0.3	0.1	<0.1	0.1	0.2
	30	10.9	2.0	1.4	0.6	0.4	0.1	0.1	0.1	0.1
17	10	6.6	0.8	0.7	0.3	0.2	<0.1	<0.1	<0.1	<0.1
	20	13.9	2.5	2.7	0.8	0.5	0.2	0.2	0.2	0.1
	28	11.0	1.6	1.8	0.7	0.4	0.2	0.3	0.2	0.2
19	12	5.3	1.0	0.5	0.2	0.2	<0.1	<0.1	<0.1	<0.1
	22	12.8	2.2	1.3	1.0	0.4	0.2	0.2	0.2	0.1

(Continued)

Table 7 (Continued)

Test Wave		Wave Height, ft, at Indicated Gage Location								
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9
					West					
7	8	3.6	0.3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	14	9.0	0.8	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	5.2	0.4	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
9	6	1.4	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	6.5	0.7	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	22	5.6	0.4	0.2	0.1	0.1	<0.1	<0.1	<0.1	<0.1
11	6	2.6	0.4	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	6.3	1.2	0.3	0.2	<0.1	<0.1	<0.1	<0.1	<0.1
	18	9.3	1.3	0.5	0.2	0.1	<0.1	<0.1	0.1	0.1
	30	6.9	0.8	0.4	0.2	0.2	<0.1	0.1	0.2	0.2
13	6	2.5	0.4	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	10.4	1.9	0.5	0.4	0.3	<0.1	<0.1	<0.1	<0.1
	20	7.4	1.0	0.4	0.3	0.2	0.1	<0.1	0.1	<0.1
	30	8.2	1.2	0.6	0.3	0.2	0.1	<0.1	0.1	0.1
15	10	3.1	0.4	0.3	0.1	0.1	<0.1	<0.1	<0.1	<0.1
	20	10.6	2.0	1.3	0.5	0.3	0.1	0.1	0.1	0.2
	30	7.6	1.4	0.9	0.3	0.2	0.1	0.1	0.2	0.3
17	10	6.5	0.7	0.7	0.2	0.2	<0.1	<0.1	<0.1	<0.1
	20	10.3	1.3	1.6	0.6	0.4	0.2	0.1	0.1	<0.1
	28	10.8	1.2	1.7	0.5	0.3	0.2	0.2	0.1	0.2
					West-Southwest					
7	8	3.7	0.4	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	14	4.1	0.3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	7.7	0.8	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
9	6	1.8	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	4.2	0.7	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	22	6.2	0.8	0.3	0.2	0.1	<0.1	<0.1	<0.1	0.1
11	10	6.2	1.0	0.2	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	20	8.3	1.4	0.4	0.2	0.2	0.1	<0.1	0.1	<0.1
	30	7.6	0.9	0.3	0.2	0.2	0.1	<0.1	0.1	0.1
13	10	6.6	1.4	0.4	0.2	0.2	<0.1	<0.1	<0.1	<0.1
	20	6.5	1.1	0.4	0.3	0.2	<0.1	<0.1	0.1	<0.1
	32	7.0	1.2	0.8	0.4	0.3	0.1	0.1	0.1	0.2
15	10	6.2	1.3	0.7	0.2	0.2	<0.1	<0.1	<0.1	<0.1
	20	8.9	1.4	0.8	0.4	0.2	<0.1	<0.1	<0.1	<0.1
	32	12.3	1.3	0.8	0.3	0.2	0.1	0.1	0.1	0.1
17	14	7.6	1.1	1.2	0.4	0.3	0.1	<0.1	0.1	0.1
	20	10.2	1.3	1.6	0.6	0.4	0.1	0.2	0.1	0.1
	28	9.0	1.0	1.2	0.5	0.3	0.1	0.1	0.1	0.1

(Continued)

Table 7 (Concluded)

Test Wave		Wave Height, ft, at Indicated Gage Location								
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9
					Southwest					
7	8	4.5	0.3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	14	3.7	0.3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	4.5	0.5	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.1
9	10	5.7	0.6	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	16	8.1	1.0	0.2	0.1	<0.1	<0.1	<0.1	<0.1	0.1
	22	8.0	0.8	0.2	0.2	0.1	<0.1	<0.1	<0.1	0.1
11	6	2.2	0.4	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	14	7.2	1.9	0.7	0.3	0.2	<0.1	<0.1	<0.1	<0.1
	20	8.9	1.2	0.4	0.2	0.2	0.1	<0.1	0.1	0.1
	30	8.4	1.0	0.3	0.2	0.1	0.1	<0.1	<0.1	0.1
13	10	6.8	1.2	0.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	20	6.7	1.1	0.4	0.3	0.2	<0.1	<0.1	0.1	<0.1
	32	7.1	1.0	0.4	0.3	0.2	0.1	<0.1	0.1	<0.1
15	10	5.4	1.2	0.8	0.3	0.2	<0.1	<0.1	<0.1	<0.1
	20	7.9	1.3	0.7	0.2	0.2	<0.1	<0.1	<0.1	0.1
	32	7.6	1.4	0.9	0.4	0.2	0.1	0.1	0.1	0.1
17	22	6.2	0.7	0.5	0.3	0.2	0.1	<0.1	0.1	<0.1

Table 8

Wave Heights for Plan 2, swl = +6.2 ft

Test Wave		Wave Height, ft, at Indicated Gage Location								
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9
				Northwest						
7	8	1.8	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	14	3.1	0.4	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	8.8	1.2	0.4	0.2	0.1	<0.1	<0.1	<0.1	<0.1
9	6	1.5	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	3.7	0.5	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	11.5	1.4	0.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1
11	6	1.0	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	3.3	0.4	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	24	8.9	1.2	0.4	0.3	0.2	0.1	0.1	0.2	0.1
13	6	2.5	0.4	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	5.8	0.9	0.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	20	11.2	2.1	0.9	0.4	0.3	0.1	<0.1	<0.1	0.1
15	10	4.5	0.6	0.4	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	20	5.7	1.2	0.8	0.4	0.3	0.1	<0.1	<0.1	0.1
17	6	5.4	0.8	0.7	0.2	0.2	<0.1	<0.1	<0.1	<0.1
	12	6.5	1.0	0.6	0.2	0.2	<0.1	<0.1	<0.1	0.1
	22	7.8	1.3	1.1	0.5	0.3	0.2	0.1	0.2	0.1
19	12	3.1	0.7	0.3	0.3	0.2	<0.1	<0.1	<0.1	0.1
				West-Northwest						
7	8	2.8	0.4	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	16	8.5	1.5	0.5	0.3	0.2	<0.1	<0.1	<0.1	<0.1
9	6	1.8	0.3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	10	3.4	0.5	0.1	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	18	13.1	2.2	0.7	0.5	0.2	<0.1	<0.1	<0.1	<0.1
11	6	1.9	0.3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	4.3	0.6	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	24	12.9	1.8	0.9	0.5	0.3	0.1	<0.1	0.1	<0.1
13	6	3.5	0.5	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	14	9.9	1.6	0.8	0.4	0.3	<0.1	<0.1	<0.1	<0.1
	22	11.1	1.6	0.8	0.4	0.2	0.1	<0.1	<0.1	0.1
15	10	4.7	0.9	0.5	0.1	0.1	<0.1	<0.1	<0.1	<0.1
	20	8.0	1.9	1.1	0.6	0.4	0.1	<0.1	<0.1	0.1
	30	12.4	2.2	1.3	0.7	0.3	0.2	0.1	0.2	0.1
17	10	6.9	1.2	1.1	0.4	0.3	0.3	<0.1	<0.1	<0.1
	20	9.0	1.4	1.1	0.7	0.4	0.2	0.1	0.1	0.1
	28	12.4	1.5	1.2	1.0	0.6	0.3	0.2	0.2	0.2
19	12	6.5	1.0	0.4	0.3	0.1	0.1	<0.1	<0.1	<0.1
	22	14.5	3.1	1.6	0.8	0.5	0.5	0.1	0.2	0.1

(Continued)

Table 8 (Continued)

Test Wave		Wave Height, ft, at Indicated Gage Location								
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9
					West					
7	8	3.2	0.4	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	14	8.7	1.3	0.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	20	5.7	0.5	0.2	0.1	0.1	<0.1	<0.1	<0.1	<0.1
9	6	2.0	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	6.7	1.0	0.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	22	5.7	0.5	0.2	0.2	0.1	<0.1	<0.1	<0.1	0.1
11	6	2.8	0.5	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	8.0	1.2	0.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	18	11.9	1.7	0.8	0.4	0.2	<0.1	<0.1	<0.1	0.1
	30	7.6	0.8	0.3	0.2	0.2	0.1	<0.1	<0.1	0.1
13	6	4.7	0.8	0.2	0.2	<0.1	<0.1	<0.1	<0.1	<0.1
	12	8.1	1.8	0.9	0.4	0.3	<0.1	<0.1	<0.1	<0.1
	20	8.6	1.5	0.8	0.4	0.2	<0.1	<0.1	0.1	<0.1
	30	9.1	1.2	0.6	0.2	0.2	<0.1	<0.1	<0.1	0.1
15	10	3.5	0.5	0.3	<0.1	0.1	<0.1	<0.1	<0.1	<0.1
	20	7.2	1.5	0.9	0.4	0.3	0.1	<0.1	<0.1	<0.1
	30	8.7	1.4	0.9	0.3	0.2	0.1	0.1	0.1	<0.1
17	10	3.9	0.5	0.2	0.1	0.1	<0.1	<0.1	<0.1	<0.1
	20	8.3	1.2	1.3	0.6	0.4	0.2	0.1	0.1	<0.1
	28	11.2	1.5	1.3	0.6	0.3	0.1	0.1	0.1	0.2
					West-Southwest					
7	8	3.7	0.4	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	14	3.1	0.3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	7.4	1.1	0.3	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
9	6	1.5	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	6.0	0.8	0.3	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	22	6.4	0.9	0.2	0.2	0.1	<0.1	<0.1	<0.1	0.1
11	10	6.5	1.1	0.2	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	20	9.0	0.9	0.3	0.2	0.2	<0.1	<0.1	<0.1	<0.1
	30	7.8	0.8	0.3	0.3	0.2	0.1	0.1	0.1	0.2
13	10	7.6	1.6	0.7	0.3	0.3	0.1	<0.1	<0.1	<0.1
	20	7.8	1.3	0.5	0.3	0.2	<0.1	<0.1	<0.1	<0.1
	32	9.1	1.4	0.8	0.4	0.2	0.1	0.1	0.1	0.2
15	10	5.2	0.9	0.3	0.1	0.1	<0.1	<0.1	<0.1	<0.1
	20	9.1	1.2	1.0	0.5	0.3	<0.1	<0.1	<0.1	<0.1
	32	7.7	1.1	0.8	0.3	0.2	0.1	0.1	0.2	0.1
17	14	10.5	1.2	1.1	0.4	0.3	<0.1	<0.1	<0.1	0.1
	20	9.8	1.0	1.0	0.4	0.2	0.1	0.1	0.1	<0.1
	28	10.7	1.2	0.9	0.5	0.3	0.2	0.1	0.1	0.1

(Continued)

Table 8 (Concluded)

Test Wave		Wave Height, Ft, at Indicated Gage Location								
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9
					Southwest					
7	8	5.2	0.9	0.2	0.2	<0.1	<0.1	<0.1	<0.1	<0.1
	14	5.4	0.8	0.2	0.1	<0.1	<0.1	<0.1	<0.1	0.1
	20	4.8	0.5	0.2	0.1	0.1	<0.1	<0.1	<0.1	0.1
9	10	5.7	1.4	0.4	0.2	0.2	<0.1	<0.1	<0.1	<0.1
	16	6.6	1.0	0.3	0.2	0.1	<0.1	<0.1	<0.1	0.1
	22	7.6	1.1	0.4	0.2	0.2	0.1	0.1	0.1	<0.1
11	6	2.8	0.5	0.1	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	14	7.8	1.3	0.5	0.3	0.2	<0.1	<0.1	<0.1	0.1
	20	7.3	0.9	0.3	0.2	0.2	<0.1	<0.1	<0.1	0.1
	30	7.3	0.8	0.3	0.3	0.2	0.1	0.1	0.1	0.2
13	10	4.5	0.9	0.4	0.2	0.2	<0.1	<0.1	<0.1	<0.1
	20	7.5	1.5	0.6	0.4	0.2	<0.1	<0.1	<0.1	<0.1
	32	6.4	1.1	0.4	0.3	0.2	0.1	<0.1	0.1	<0.1
15	10	5.0	1.2	0.6	0.2	0.2	<0.1	<0.1	<0.1	<0.1
	20	9.9	1.6	1.0	0.6	0.3	0.1	<0.1	<0.1	0.1
	32	11.1	1.6	1.0	0.5	0.4	0.2	0.1	0.1	0.1
17	22	6.7	0.7	0.5	0.3	0.2	0.1	0.1	0.1	0.1

(Sheet 3 of 3)

Table 9

Wave Heights Obtained for Plans 3 and 3A From West-Northwest, swl = +7.0 ft

Test Wave		Wave Height, ft, at Indicated Gage Location									
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10
					Plan 3						
7	8	0.9	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	1.1
	16	3.8	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	6.4
9	6	1.3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	1.1
	10	2.8	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	2.4
	18	6.1	0.4	0.1	0.2	0.1	<0.1	<0.1	<0.1	<0.1	7.6
11	6	0.7	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	1.0
	12	2.5	0.3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	3.0
	24	5.8	0.7	0.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1	7.1
13	6	1.0	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	1.4
	14	6.4	0.8	0.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1	7.3
	22	7.9	0.9	0.4	0.2	0.2	<0.1	<0.1	<0.1	0.1	<8.9
15	10	2.0	0.4	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	2.4
	20	5.1	0.9	0.4	0.1	0.1	<0.1	<0.1	<0.1	0.1	7.6
	30	8.8	1.6	0.7	0.3	0.3	0.1	0.1	0.1	0.2	10.5
17	10	4.1	0.6	0.2	0.1	0.1	<0.1	<0.1	<0.1	<0.1	5.3
	20	8.4	1.2	0.8	0.4	0.2	0.1	0.1	0.1	0.2	11.1
	28	7.5	1.4	1.0	0.6	0.3	0.2	0.2	0.2	0.1	11.7
19	12	4.2	0.7	0.2	0.3	0.2	0.1	<0.1	<0.1	<0.1	7.1
	22	7.1	0.9	0.5	0.4	0.2	0.1	0.1	0.1	0.1	8.8
					Plan 3A						
7	8	0.9	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	1.3
	16	3.3	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	4.1
9	6	1.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	1.0
	10	1.9	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	1.2
	18	5.4	0.6	0.2	0.2	0.1	<0.1	<0.1	<0.1	0.1	7.2
11	6	0.4	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.7
	12	2.4	0.4	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	4.8
	24	4.3	0.7	0.2	0.2	0.2	<0.1	<0.1	<0.1	0.1	9.4

(Continued)

Table 9 (Concluded)

Test Wave		Wave Height, ft, at Indicated Gage Location									
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10
		Plan 3A (Continued)									
13	6	0.7	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	1.3
	14	4.5	0.7	0.2	0.2	0.1	<0.1	<0.1	<0.1	<0.1	6.0
	22	6.4	0.8	0.4	0.1	0.2	<0.1	<0.1	<0.1	0.1	9.4
15	10	2.2	0.5	0.3	0.1	0.1	<0.1	<0.1	<0.1	<0.1	2.2
	20	4.0	0.8	0.3	0.1	0.1	<0.1	<0.1	<0.1	0.1	6.7
	30	6.4	1.4	0.6	0.2	0.2	0.1	<0.1	0.1	0.1	9.3
17	10	3.2	0.5	0.2	0.1	<0.1	<0.1	<0.1	<0.1	0.1	4.4
	20	8.0	1.2	1.1	0.4	0.2	0.1	0.1	0.1	0.2	11.3
	28	7.4	1.4	0.9	0.6	0.3	0.2	0.2	0.2	0.1	10.3
19	12	3.7	0.6	0.2	0.3	0.2	0.1	0.1	<0.1	<0.1	6.0
	22	5.7	0.7	0.5	0.4	0.2	0.1	0.1	0.1	0.1	8.1

(Sheet 2 of 2)

Table 10

Wave Heights Obtained for Plan 4 from West-Northwest, swl = +7.0 ft

Test Wave		Wave Height, ft, at Indicated Gage Location									
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10
7	8	1.1	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	1.2
	16	3.9	0.3	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	14.5
9	6	1.4	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	1.6
	10	1.7	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	2.3
	18	6.2	0.9	0.2	0.2	0.1	<0.1	<0.1	<0.1	0.1	16.8
11	6	0.9	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	1.8
	12	2.5	0.4	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	7.2
	24	5.7	0.7	0.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1	11.6
13	6	0.7	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	1.6
	14	6.1	0.9	0.2	0.3	0.1	<0.1	<0.1	<0.1	<0.1	10.8
	22	6.2	0.9	0.3	0.2	0.1	<0.1	<0.1	<0.1	0.1	11.8
15	10	2.3	0.4	0.3	0.1	0.1	<0.1	<0.1	<0.1	<0.1	3.4
	20	4.8	1.0	0.4	0.2	0.2	<0.1	<0.1	<0.1	<0.1	12.2
	30	7.8	1.6	0.7	0.3	0.2	<0.1	<0.1	<0.1	<0.1	15.2
17	10	4.3	0.8	0.4	0.2	0.1	0.2	<0.1	<0.1	<0.1	8.1
	20	9.6	1.4	0.7	0.3	0.2	0.1	0.1	0.1	0.1	17.8
	28	6.4	1.2	0.9	0.5	0.3	0.2	0.2	0.1	0.1	14.8
19	12	3.9	0.7	0.3	0.3	0.1	<0.1	<0.1	<0.1	<0.1	8.8
	22	6.9	0.9	0.6	0.4	0.2	0.1	<0.1	<0.1	<0.1	12.1

Table 11

Wave Heights Obtained for Rubble-Mound Breakwaters of Plans 5-23 for 15-sec, 30-ft Waves from West-Northwest, swl = +7.0 ft

Plan Number	Gage 1 Wave Height, ft	Cumulative Breakwater Length, ft
5	8.7	370
6	6.2	670
7	5.5	637
8	4.2	712
9	5.0	600
10	7.3	825
11	6.2	1,125
12	5.5	450
12A	4.7	525
13	4.5	750
14	5.0	750
15	4.3	825
16	8.5	675
16A	8.5	1,012
17	3.4	712
18	4.3	1,125
19	4.0	637
20	3.9	712
21	3.6	712
22	3.9	637
23	5.4	562

Table 12

Wave Heights Obtained for Rubble-Mound Breakwaters of Plans 24-42 for 15-sec, 30-ft Waves from West-Northwest, swl = +7.0 ft

Plan Number	Gage 1 Wave Height, ft	Cumulative Breakwater Length, ft
24	6.9	525
25	5.7	525
26	6.9	450
27	5.8	637
28	5.4	600
29	5.9	562
30	6.3	487
31	9.2	860
32	6.4	937
33	6.1	490
34	7.2	390
35	6.5	550
36	6.5	585
37	5.8	525
38	4.3	585
39	7.6	637
40	12.0	1,087
41	10.5	800
42	11.4	870

Table 13

Wave Heights for Plan 43

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
					Northwest					
7	8	0.7	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	14	2.1	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	1.6	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
9	6	0.8	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	1.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	2.1	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
11	6	0.6	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	1.2	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	24	3.1	0.3	0.2	0.1	0.1	0.1	<0.1	<0.1	<0.1
13	6	1.8	0.4	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	2.8	0.5	0.2	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	20	4.8	0.7	0.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1
15	10	1.7	0.4	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	3.7	0.8	0.4	0.2	0.1	<0.1	<0.1	<0.1	<0.1
17	6	2.7	0.5	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	3.6	0.5	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	22	5.1	0.6	0.4	0.2	0.1	<0.1	<0.1	<0.1	<0.1
19	12	2.0	0.5	0.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1
					West-Northwest					
7	8	0.5	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	16	1.7	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
9	6	1.3	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	10	1.3	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	18	5.2	0.3	0.1	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
11	6	0.4	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	1.7	0.4	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	24	4.4	0.4	0.2	0.1	0.1	<0.1	<0.1	<0.1	<0.1
13	6	0.6	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	14	4.4	0.6	0.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	22	3.8	0.5	0.2	0.2	0.1	<0.1	<0.1	<0.1	<0.1
15	10	2.7	0.5	0.3	0.1	0.1	<0.1	<0.1	<0.1	<0.1
	20	3.5	0.7	0.3	0.1	0.1	<0.1	<0.1	<0.1	<0.1
	30	6.3	1.2	0.6	0.2	0.2	<0.1	<0.1	<0.1	<0.1
17	10	3.6	0.5	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	6.1	0.8	0.6	0.2	0.2	0.1	<0.1	<0.1	<0.1
	28	6.4	1.0	0.6	0.3	0.2	0.1	0.1	0.1	0.1
19	12	2.7	0.5	0.4	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	22	5.2	0.8	0.5	0.4	0.2	0.1	0.1	0.1	<0.1

(Continued)

Table 13 (Continued)

Test Wave		Wave Height, ft, at Indicated Gage Location								
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9
					West					
7	8	1.3	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	14	2.8	0.5	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	1.9	0.3	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
9	6	1.3	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	3.4	0.3	0.1	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	22	3.5	0.3	0.1	0.1	0.1	<0.1	<0.1	<0.1	<0.1
11	6	0.5	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	1.7	0.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	18	3.0	0.3	0.1	0.1	0.1	<0.1	<0.1	<0.1	<0.1
	30	4.4	0.5	0.2	0.1	0.1	0.1	<0.1	<0.1	<0.1
13	6	1.7	0.4	0.2	0.2	<0.1	<0.1	<0.1	<0.1	<0.1
	12	3.2	0.6	0.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	20	3.5	0.5	0.2	0.1	0.1	<0.1	<0.1	<0.1	<0.1
	30	5.6	0.6	0.3	0.2	0.2	0.1	<0.1	<0.1	<0.1
15	10	2.0	0.5	0.3	0.1	0.1	<0.1	<0.1	<0.1	<0.1
	20	3.4	0.7	0.3	0.1	0.1	0.1	<0.1	<0.1	<0.1
	30	5.5	0.8	0.3	0.2	0.2	0.1	0.1	0.1	<0.1
17	10	2.7	0.5	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	6.4	0.7	0.6	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	28	5.4	0.6	0.4	0.2	0.2	0.1	0.1	0.1	0.1
					West-Southwest					
7	8	1.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	14	3.9	0.3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	3.9	0.3	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
9	6	0.5	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	12	2.2	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	22	4.7	0.3	0.1	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
11	10	1.4	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	3.5	0.3	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	30	3.3	0.4	0.2	0.1	0.1	0.1	<0.1	<0.1	<0.1
13	10	2.2	0.4	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	5.0	0.6	0.2	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	32	4.8	0.4	0.2	0.1	0.1	0.1	<0.1	<0.1	<0.1
15	10	2.2	0.4	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	3.7	0.5	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	32	4.5	0.7	0.3	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
17	14	4.2	0.6	0.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	20	5.0	0.7	0.4	0.2	0.1	0.1	<0.1	<0.1	<0.1
	28	6.2	0.8	0.4	0.2	<0.1	0.1	0.1	<0.1	<0.1

(Continued)

Table 13 (Concluded)

Test Wave		Wave Height, ft, at Indicated Gage Location								
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9
					Southwest					
7	8	1.4	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	14	1.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	3.0	0.4	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
9	10	2.1	0.2	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	16	2.5	0.3	0.1	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	22	4.1	0.4	0.2	0.2	<0.1	<0.1	<0.1	<0.1	<0.1
11	6	1.1	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	14	2.4	0.3	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	3.5	0.4	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	30	4.6	0.5	0.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1
13	10	2.3	0.5	0.2	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	20	3.7	0.6	0.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1
	32	4.3	0.6	0.3	0.2	0.1	<0.1	<0.1	<0.1	<0.1
15	10	2.2	0.5	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	20	3.4	0.7	0.4	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
	32	4.8	0.7	0.3	0.1	<0.1	<0.1	<0.1	<0.1	<0.1
17	22	4.5	0.7	0.5	0.2	0.1	<0.1	<0.1	<0.1	<0.1

(Sheet 3 of 3)

Photo 1. Typical wave patterns for existing conditions; 13-sec, 20-ft waves from northwest; swl = +6.2 ft

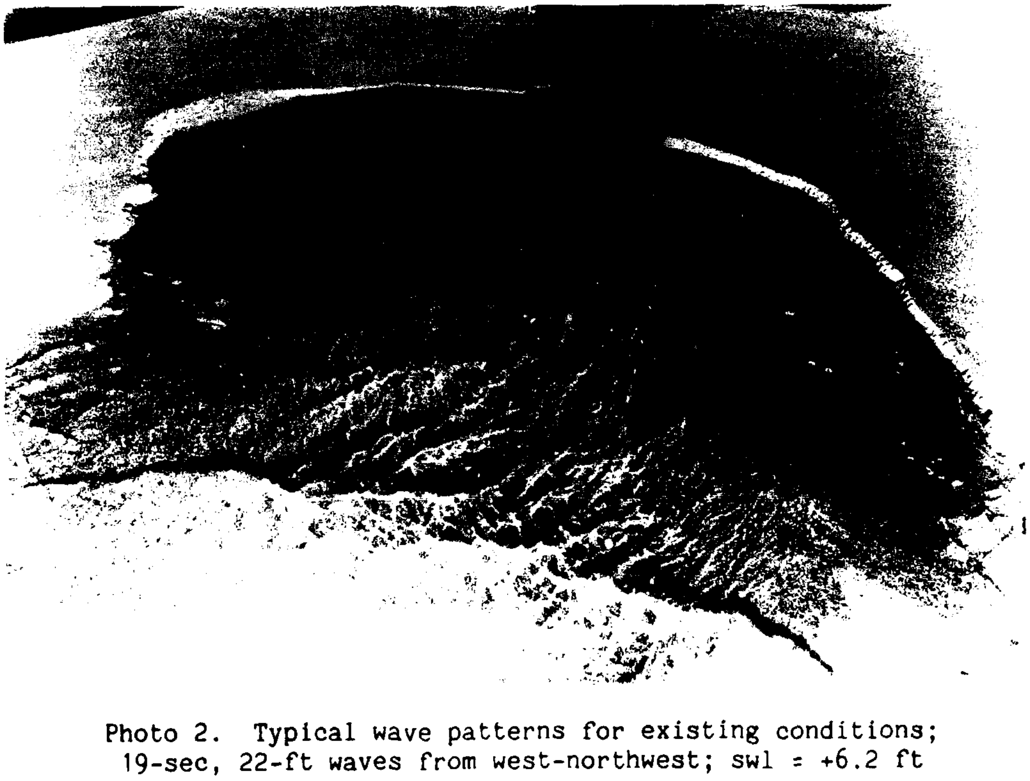


Photo 2. Typical wave patterns for existing conditions; 19-sec, 22-ft waves from west-northwest; swl = +6.2 ft

Photo 3. Typical wave patterns for existing conditions;
11-sec, 12-ft waves from west; swl = +6.2 ft

Photo 4. Typical wave patterns for existing conditions;
15-sec, 20-ft waves from west-southwest; swl = +6.2 ft

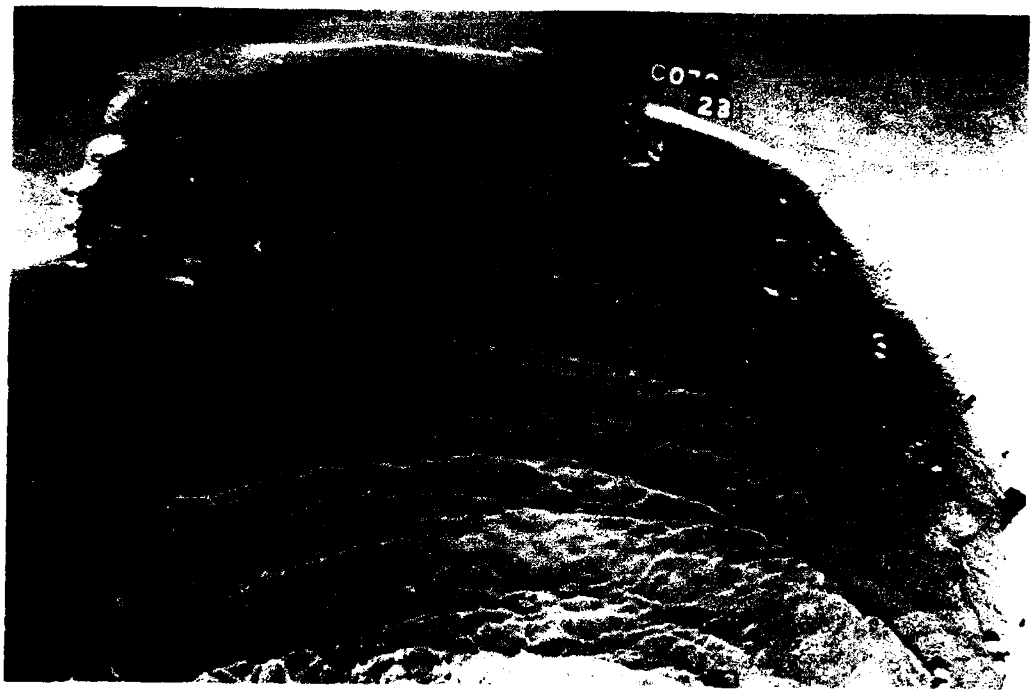


Photo 5. Typical wave patterns for existing conditions; 11-sec, 14-ft waves from southwest; swl = +6.2 ft

Photo 6. Riverine sediment patterns for existing conditions; 7,000-cfs river discharge

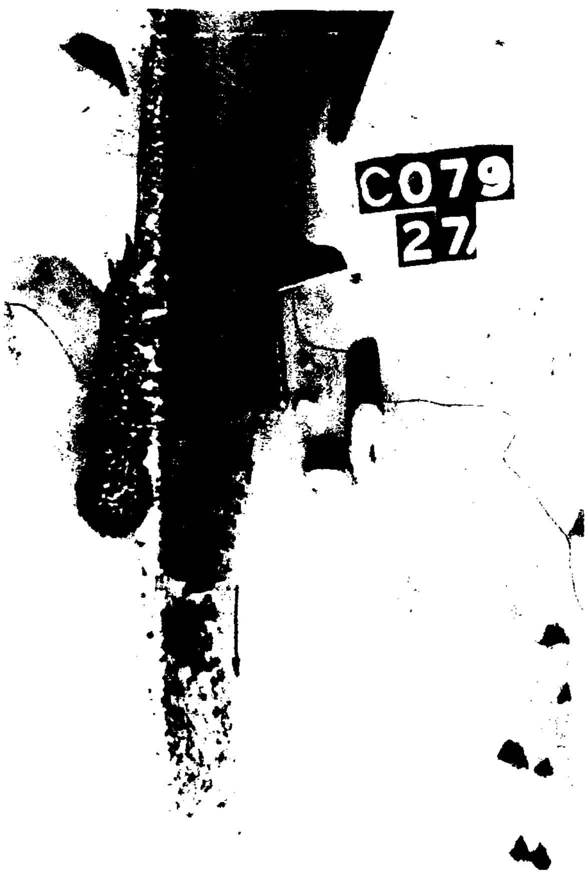


Photo 7. Riverine sediment patterns for existing conditions; 20,000-cfs river discharge

Photo 8. Riverine sediment patterns for existing conditions; 27,000-cfs river discharge

Photo 9. Riverine sediment patterns for existing conditions; 33,000-cfs river discharge

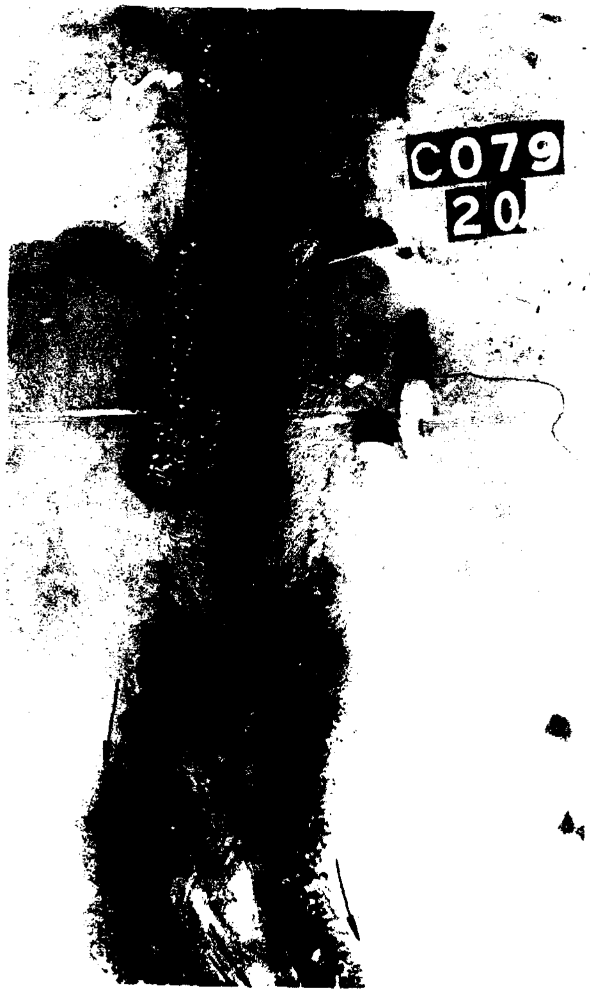


Photo 10. Riverine sediment patterns for existing conditions; 41,000-cfs river discharge

Photo 11. Typical wave patterns for Plan 1, 13-sec, 20-ft waves from northwest; swl = +6.2 ft

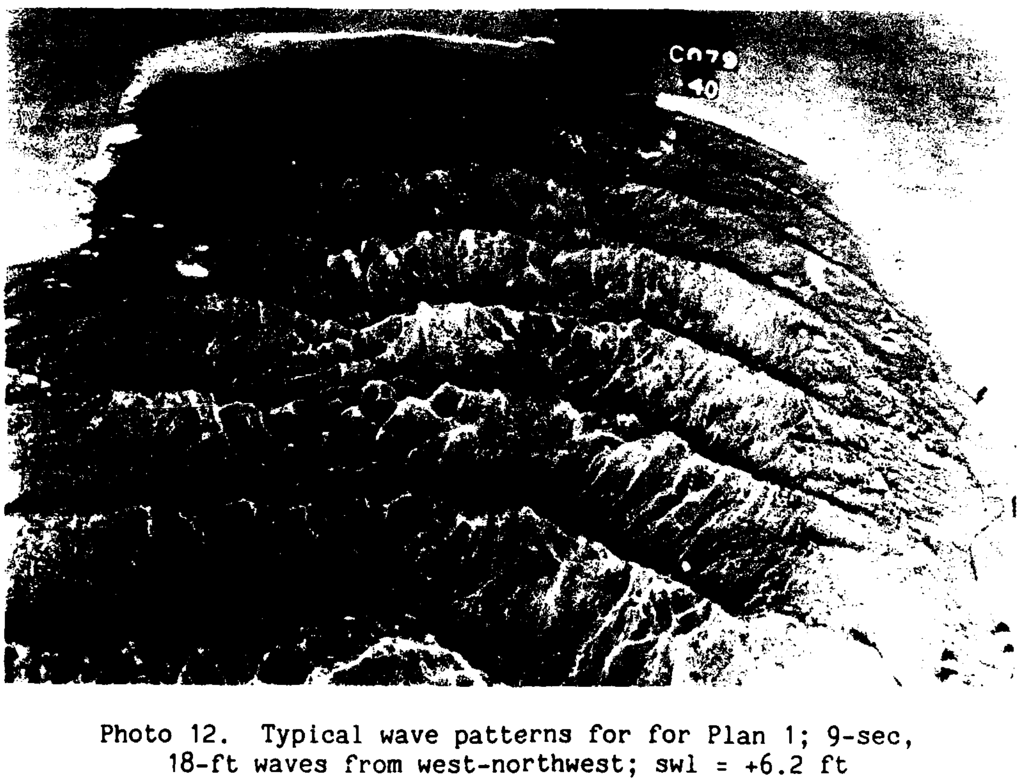


Photo 12. Typical wave patterns for for Plan 1; 9-sec, 18-ft waves from west-northwest; swl = +6.2 ft

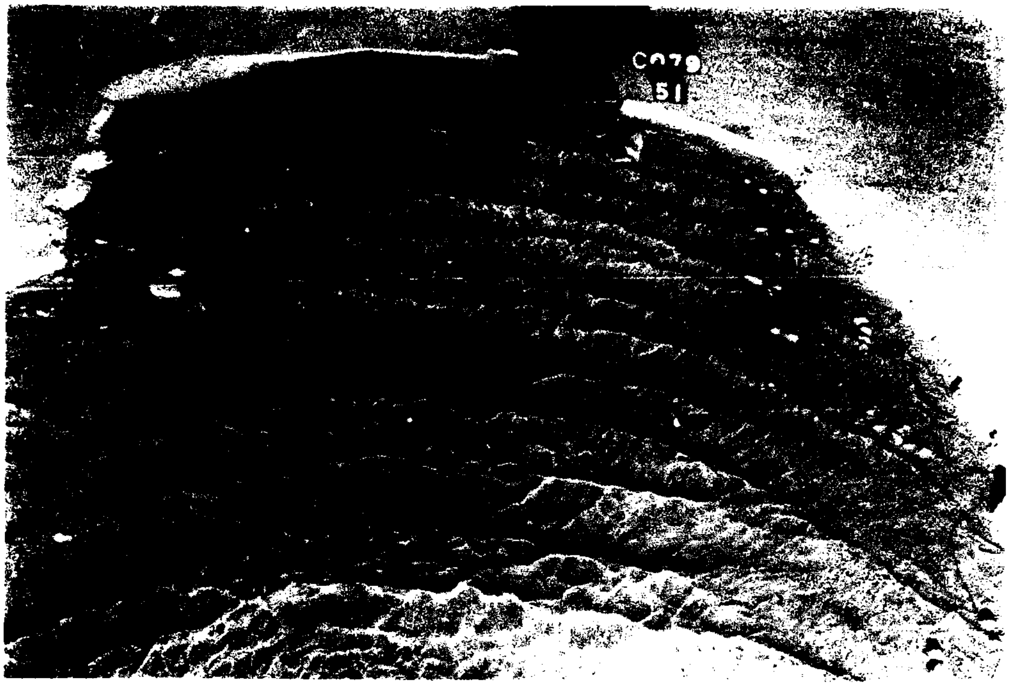


Photo 13. Typical wave patterns for Plan 1; 11-sec, 20-ft waves from southwest; swl = +6.2 ft

Photo 14. Typical wave patterns for Plan 2; 9-sec, 20-ft waves from northwest; swl = +6.2 ft

Photo 15. Typical wave patterns for Plan 2; 13-sec, 22-ft waves from west-northwest; swl = +6.2 ft

Photo 16. Typical wave patterns for Plan 2; 13-sec, 32-ft waves from west-southwest; swl = +6.2 ft

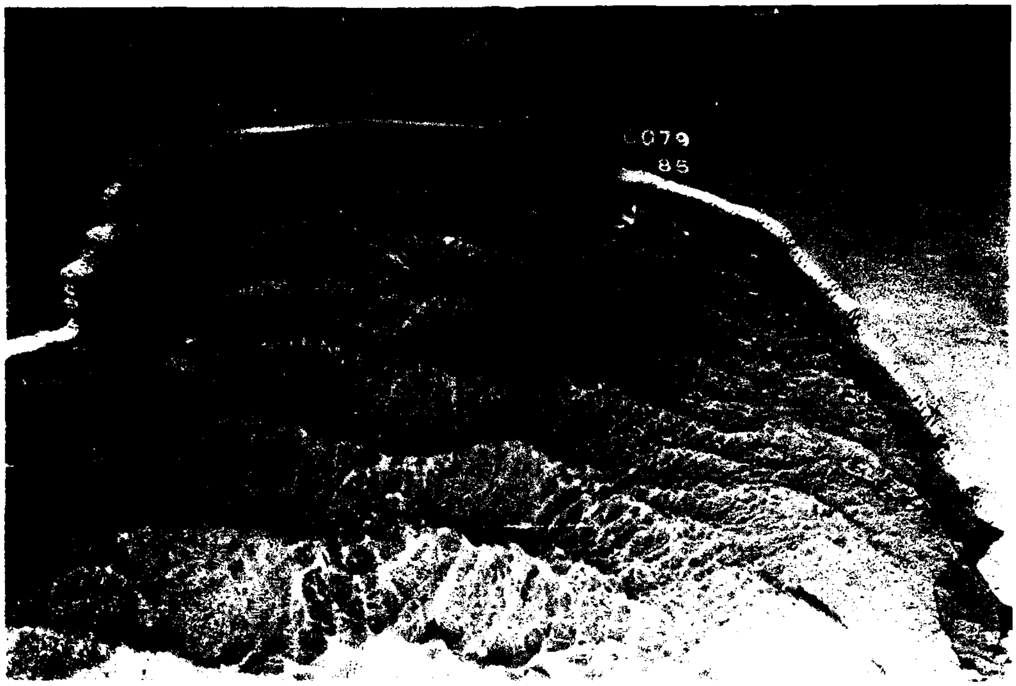


Photo 17. Typical wave patterns for Plan 3; 9-sec, 18-ft waves from west-northwest; swl = +7.0 ft

Photo 18. Typical wave patterns for Plan 3A; 17-sec, 20-ft waves from west-northwest; swl = +7.0 ft

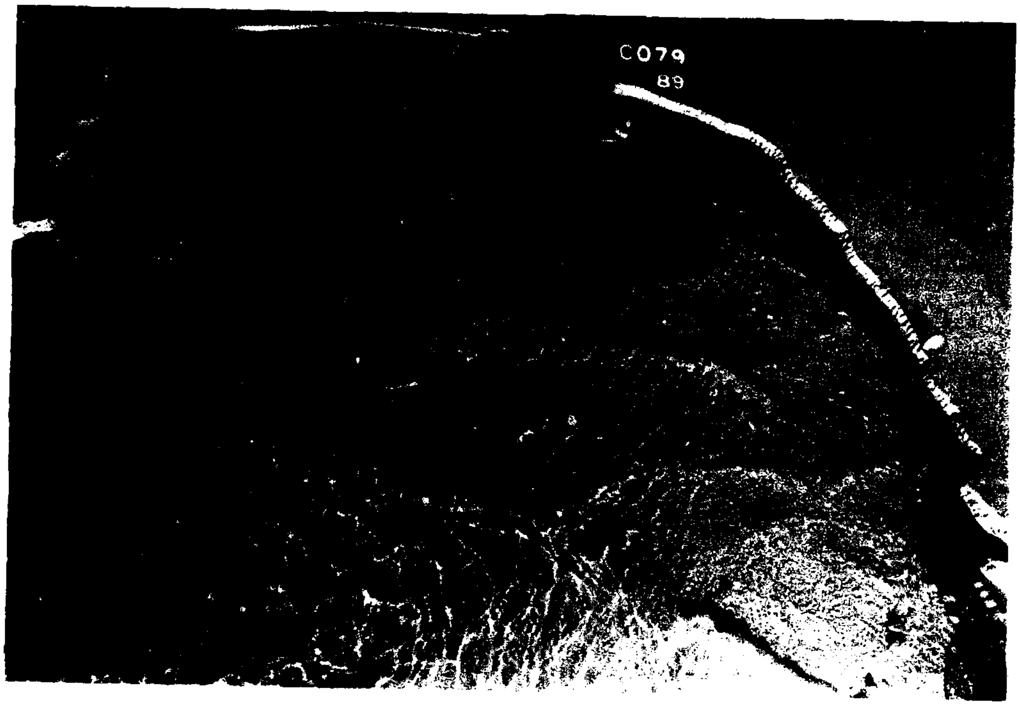


Photo 19. Typical wave patterns for Plan 4; 15-sec, 30-ft wave from west-northwest; swl = +7.0 ft

Photo 20. Typical wave patterns for Plan 43; 13-sec, 12-ft waves from northwest; swl = +7.0 ft

Photo 21. Typical wave patterns for Plan 43; 17-sec, 22-ft waves from northwest; swl = +7.0 ft

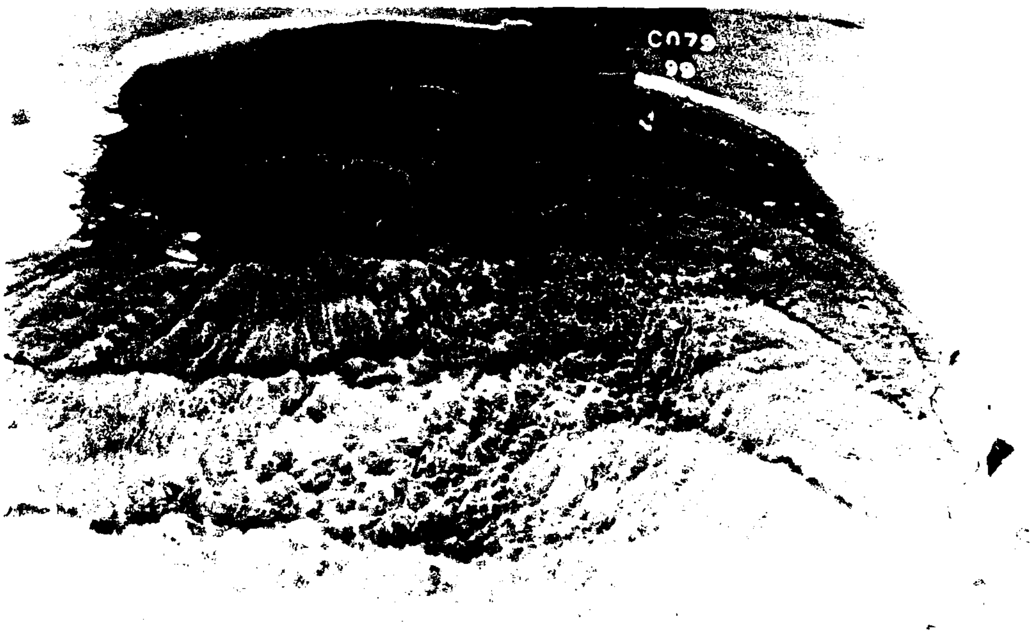


Photo 22. Typical wave patterns for Plan 43; 13-sec, 14-ft waves from west-northwest; swl = +7.0 ft

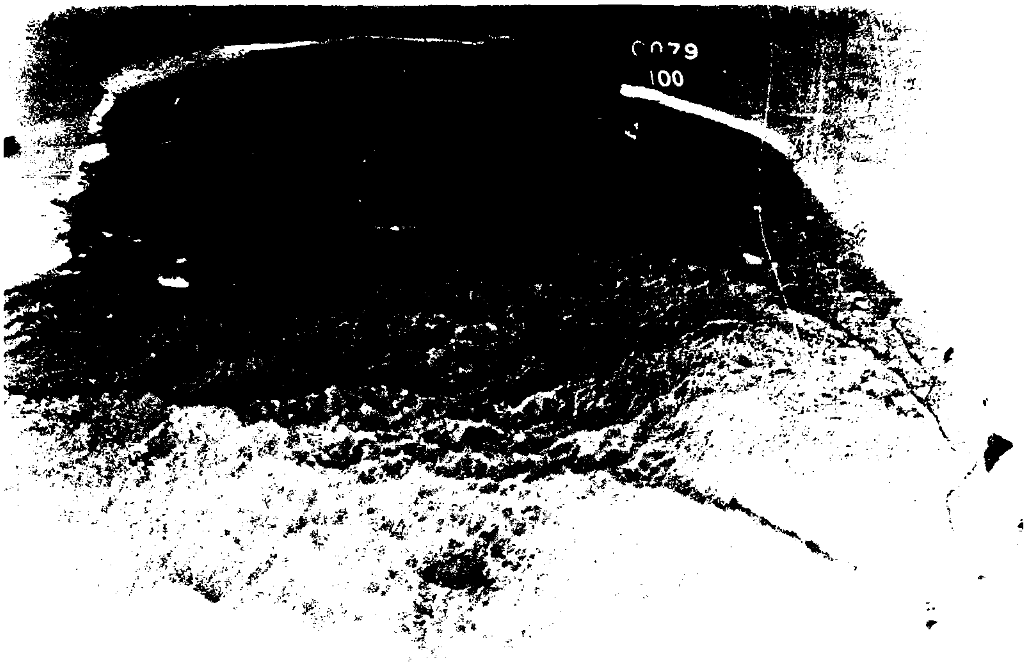


Photo 23. Typical wave patterns for Plan 43; 17-sec, 20-ft waves from west-northwest; swl = +7.0 ft

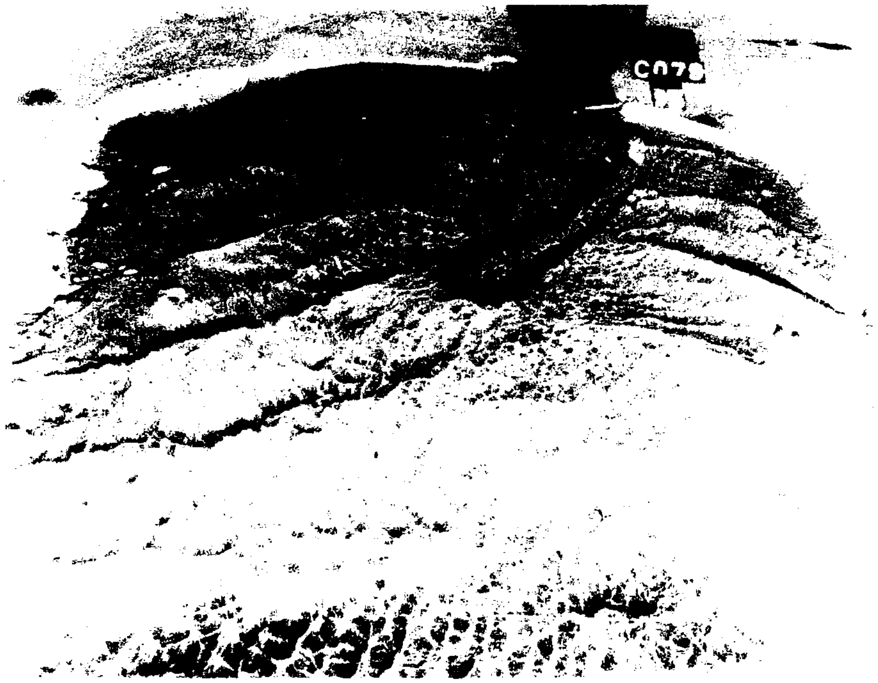


Photo 24. Typical wave patterns for Plan 43; 13-sec, 14-ft waves from west; swl = +7.0 ft

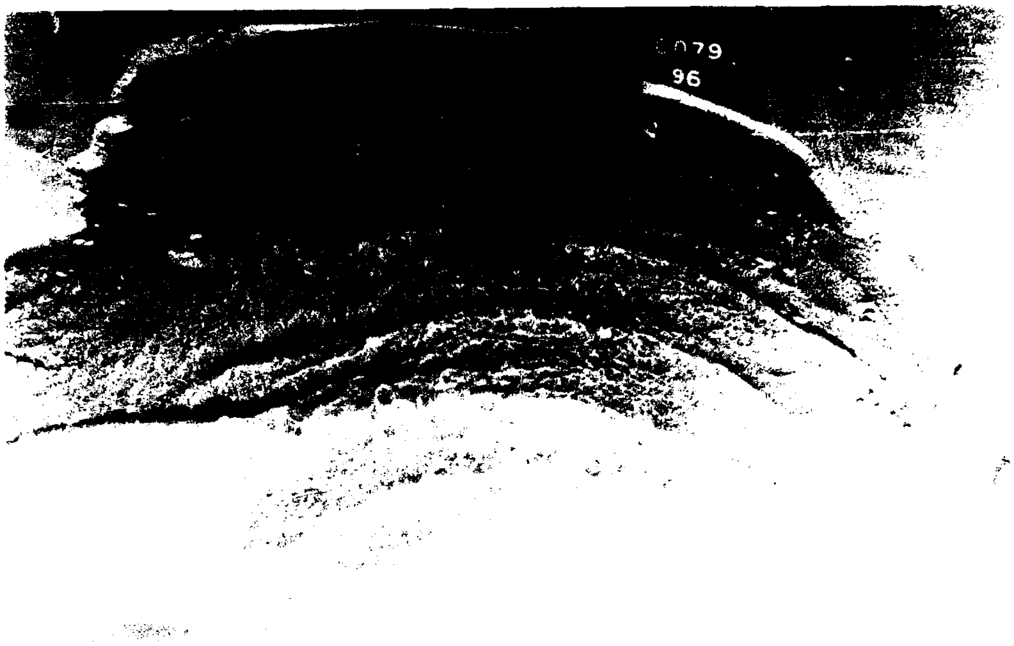


Photo 25. Typical wave patterns for Plan 43; 17-sec, 20-ft waves from west; swl = +7.0 ft

Photo 26. Typical wave patterns for Plan 43; 9-sec, 12-ft from west-southwest; swl = +7.0 ft

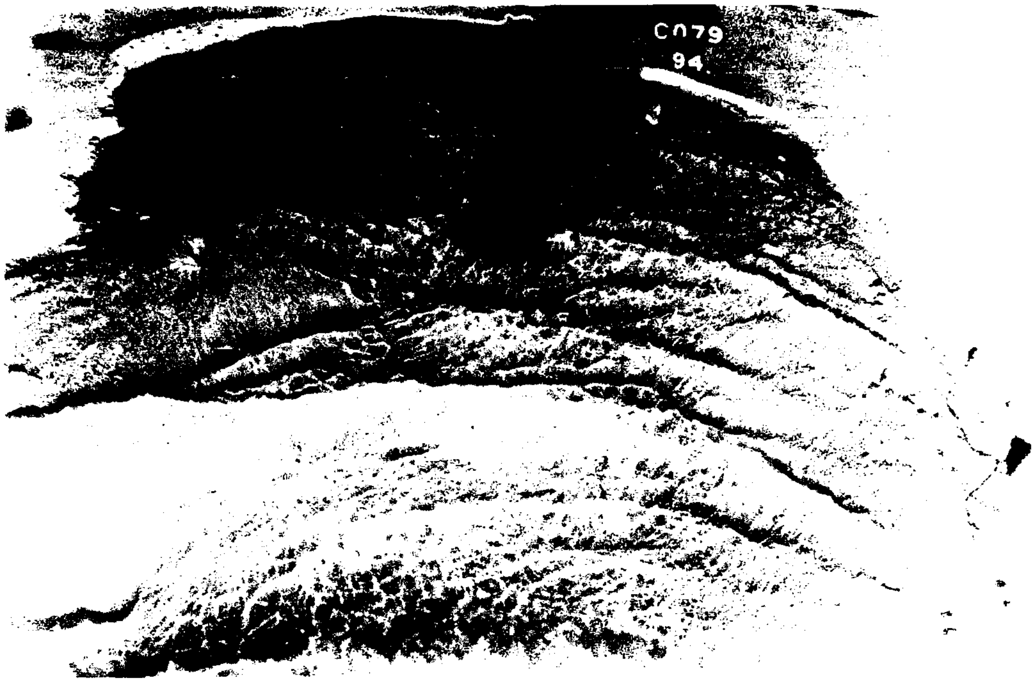


Photo 27. Typical wave patterns for Plan 43; 17-sec, 28-ft waves from west-southwest; swl = +7.0 ft

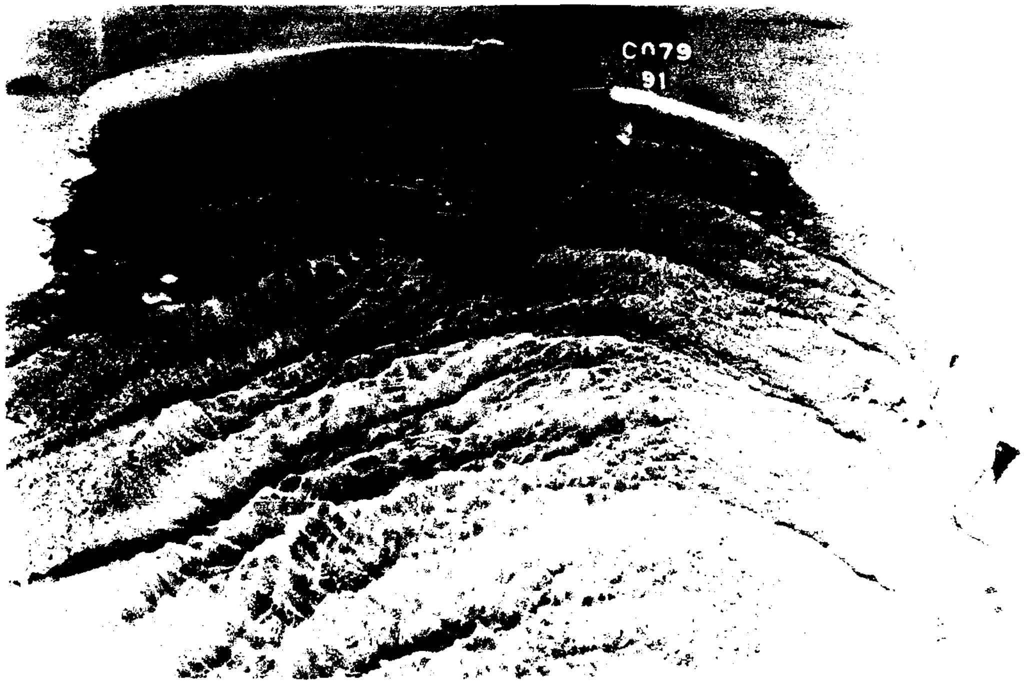


Photo 28. Typical wave patterns for Plan 43; 11-sec, 14-ft waves from southwest; swl = +7.0 ft

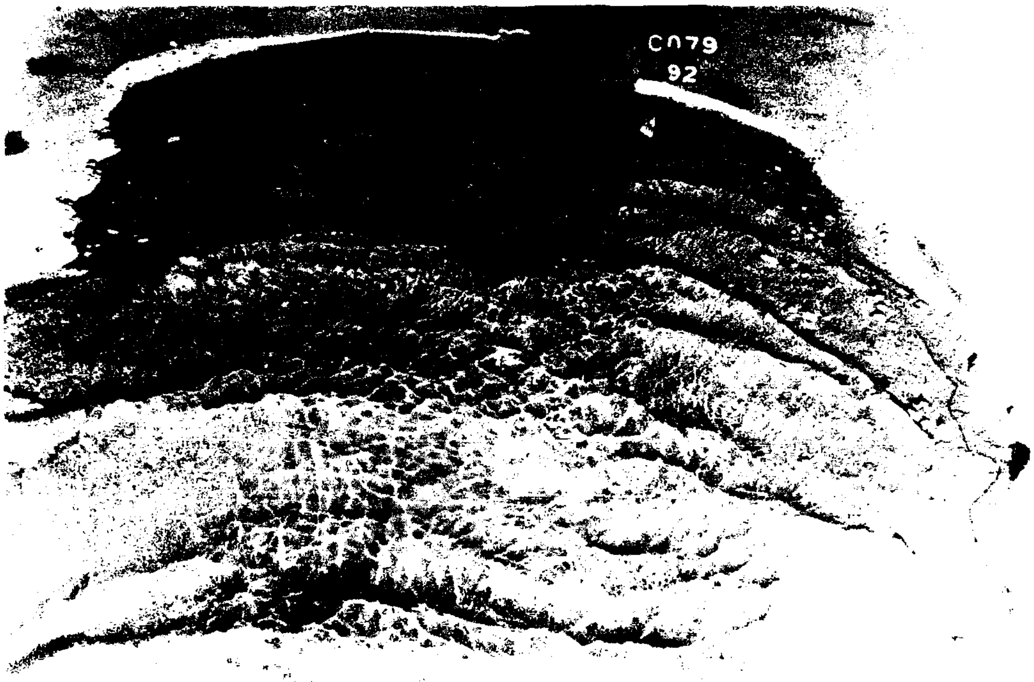


Photo 29. Typical wave patterns for Plan 43; 15-sec, 32-ft waves from southwest; swl = +7.0 ft

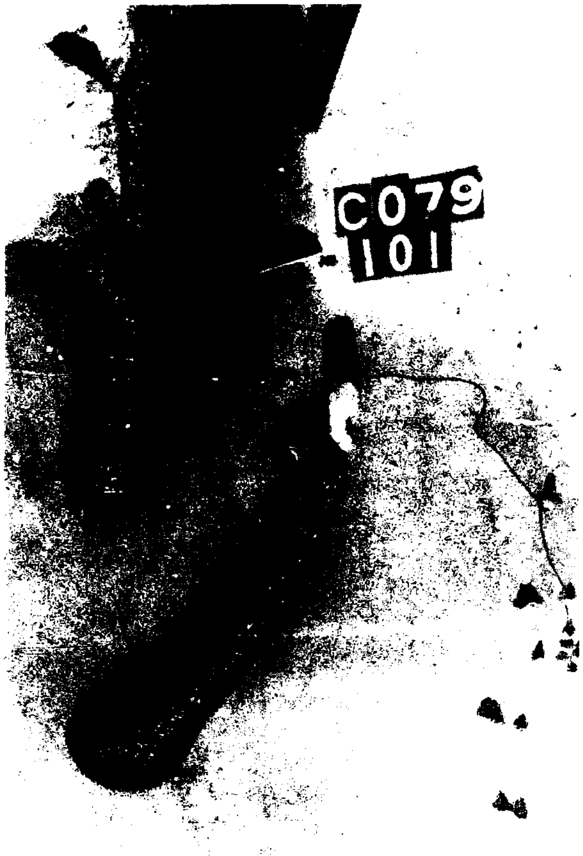


Photo 30. Riverine sediment patterns for Plan 43; 7,000-cfs river discharge

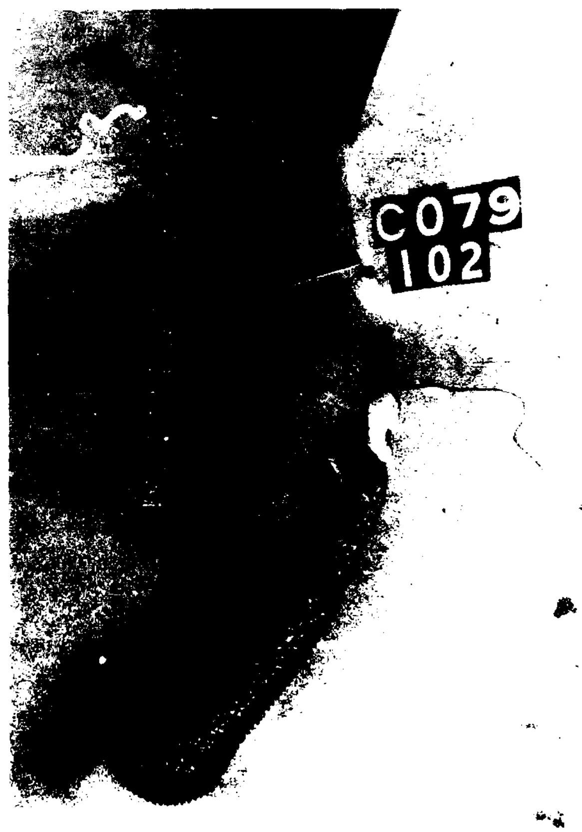
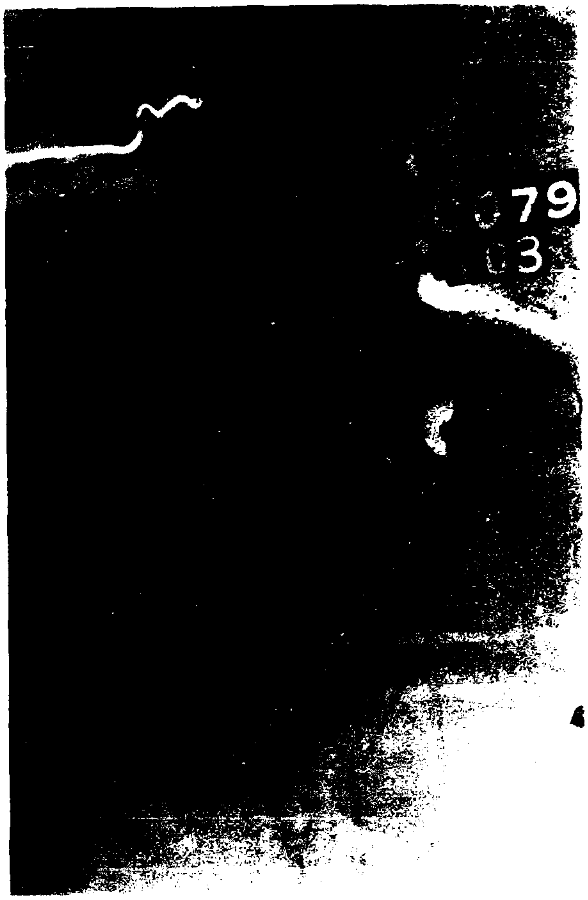


Photo 31. Riverine sediment patterns for Plan 43; 20,000-cfs river discharge

Photo 32. Riverine sediment patterns for Plan 43; 27,000-cfs river discharge



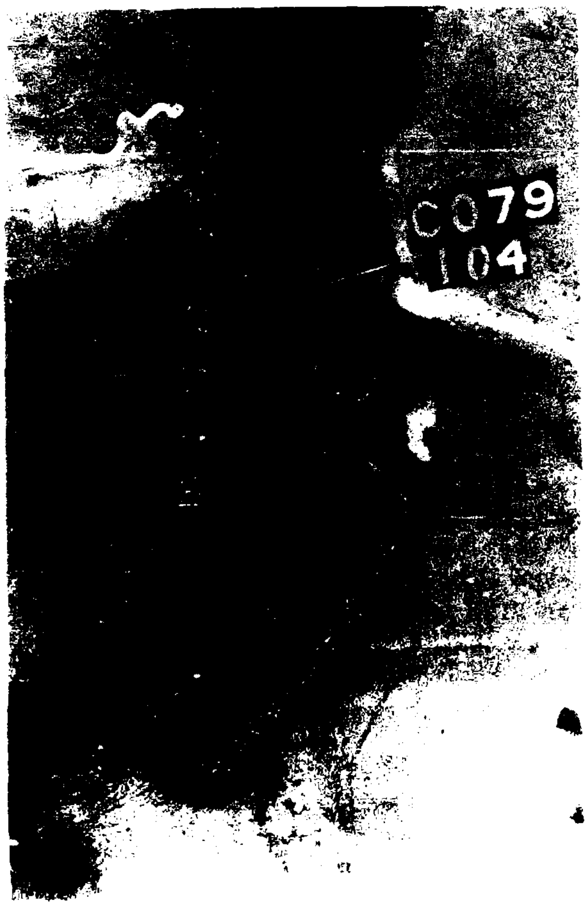


Photo 33. Riverine sediment patterns for Plan 43; 33,000-cfs river discharge

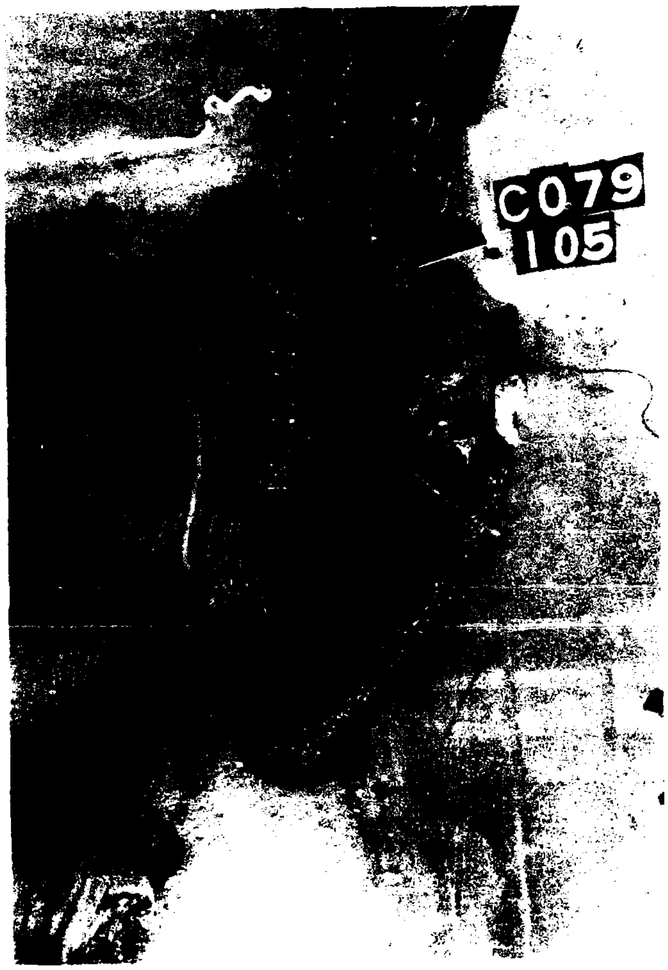


Photo 34. Riverine sediment patterns for Plan 43; 41,000-cfs river discharge

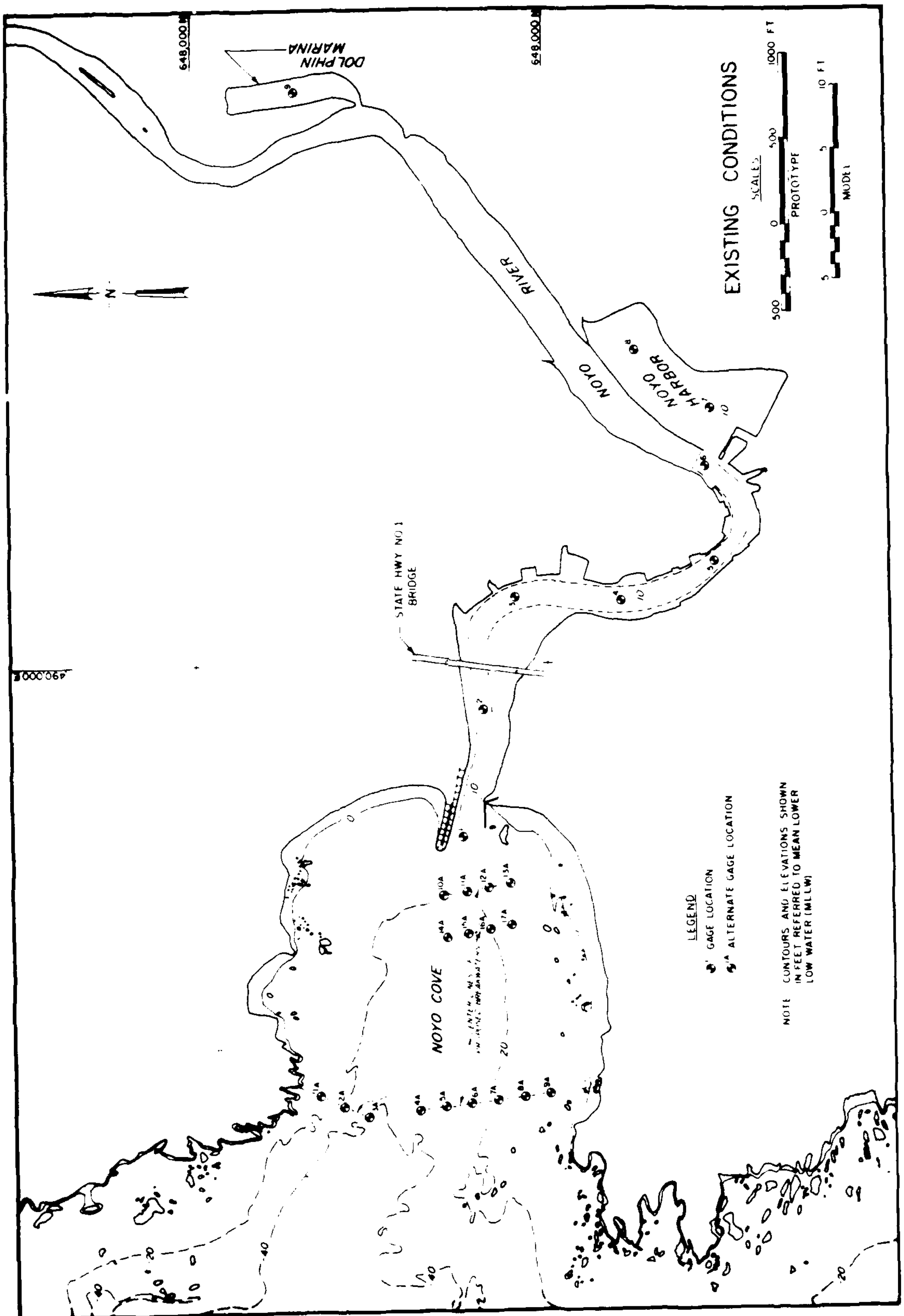
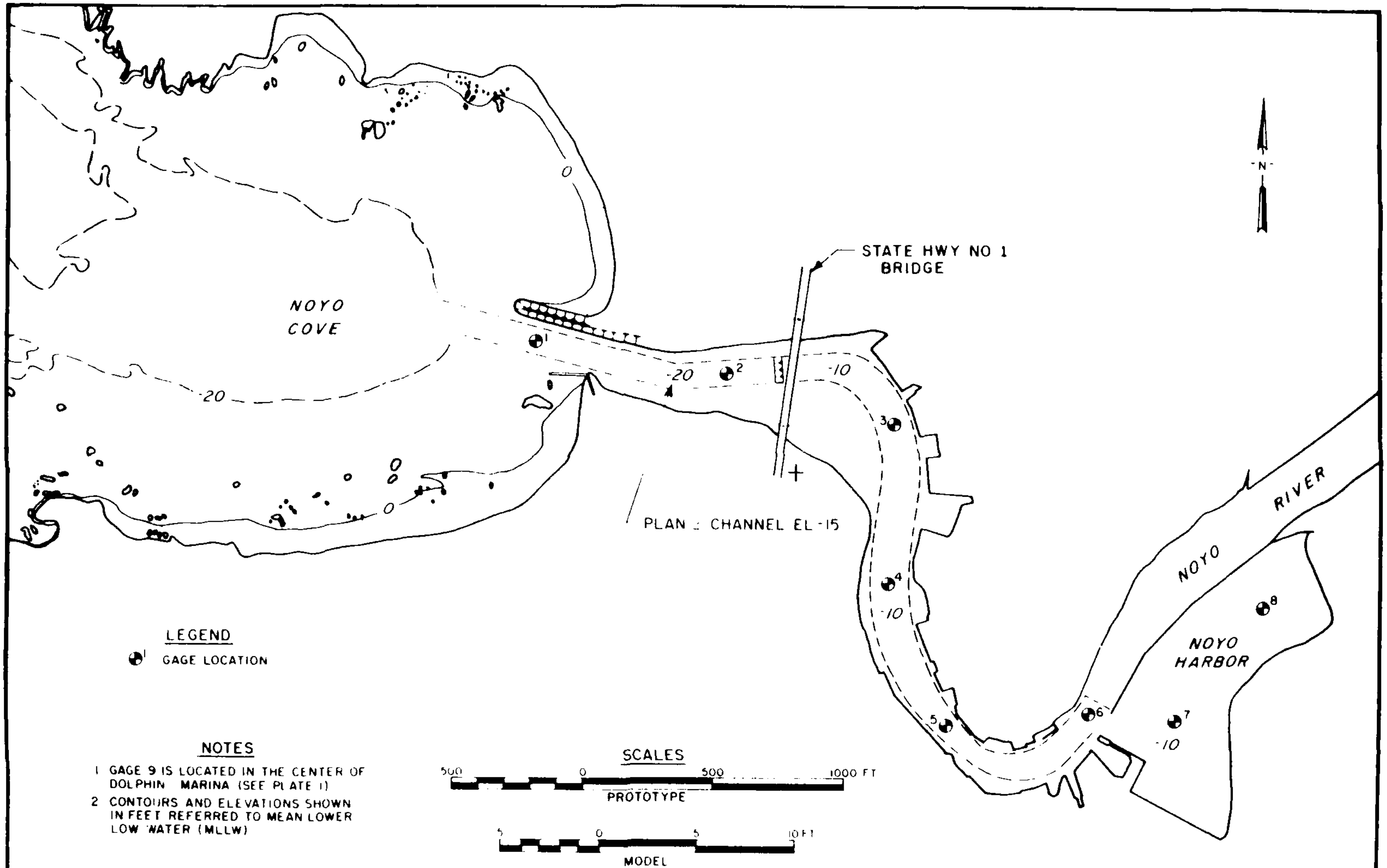
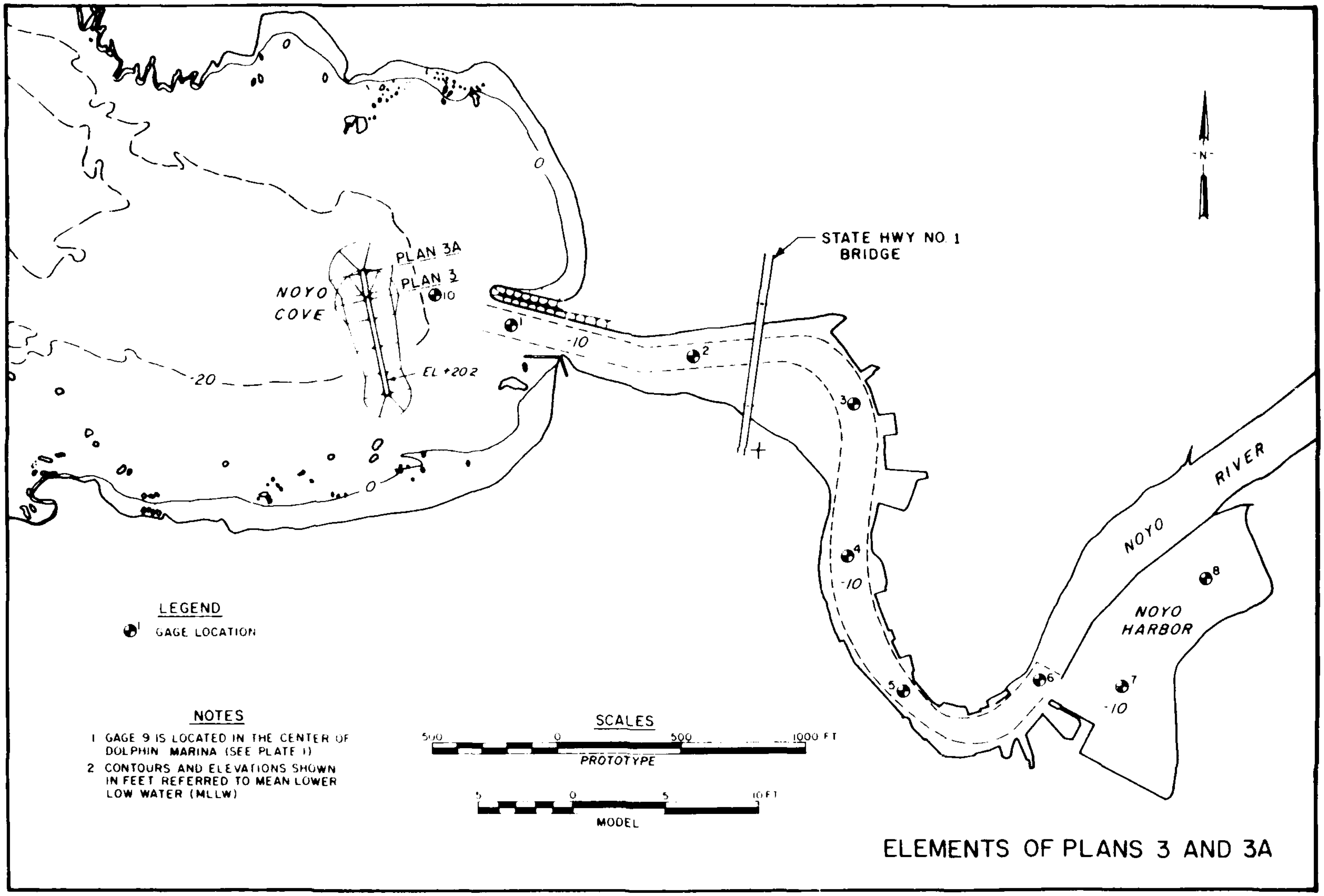

EXISTING CONDITIONS
SCALES
0 500 1000 FT
PROTOTYPE
0 5 10 FT
MODEL
DOLPHIN MARINA
NOYO RIVER
NOYO HARBOR
STATE HWY NO 1 BRIDGE
NOYO COVE
648,000 N
LEGEND
GAGE LOCATION
ALTERNATE GAGE LOCATION
NOTE CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)

PLATE 1



ELEMENTS OF PLANS 1 AND 2

PLATE 2



ELEMENTS OF PLANS 3 AND 3A

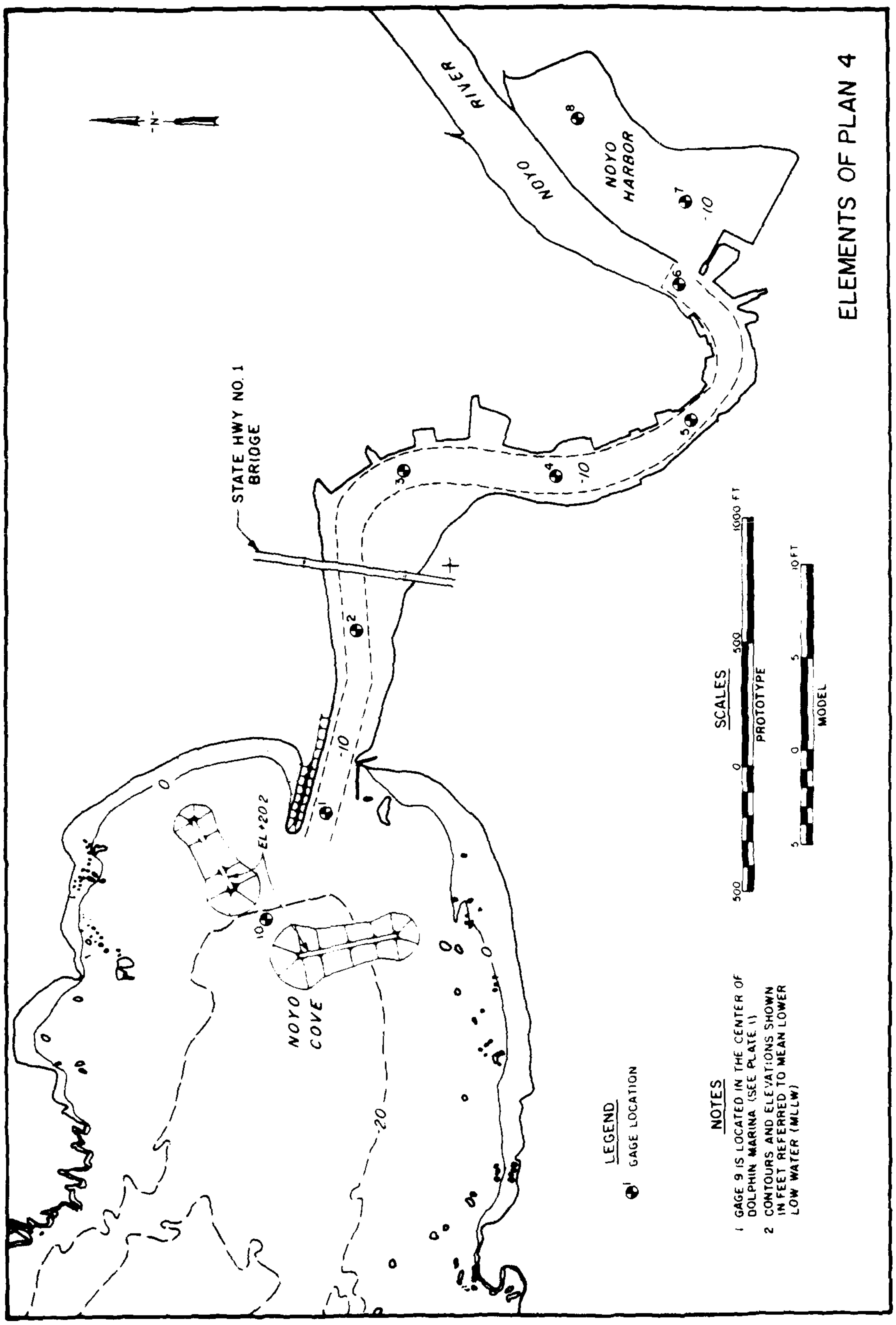
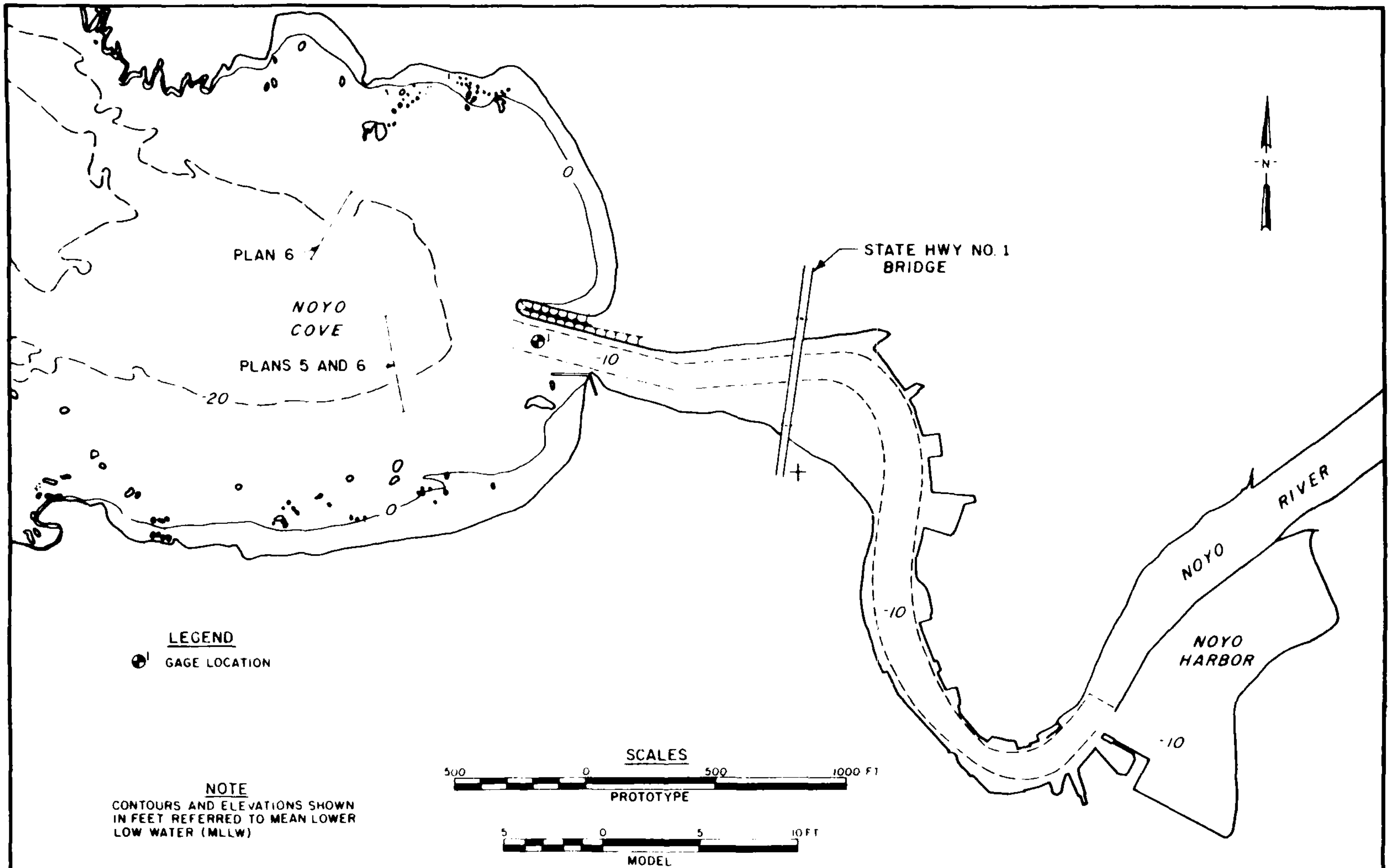
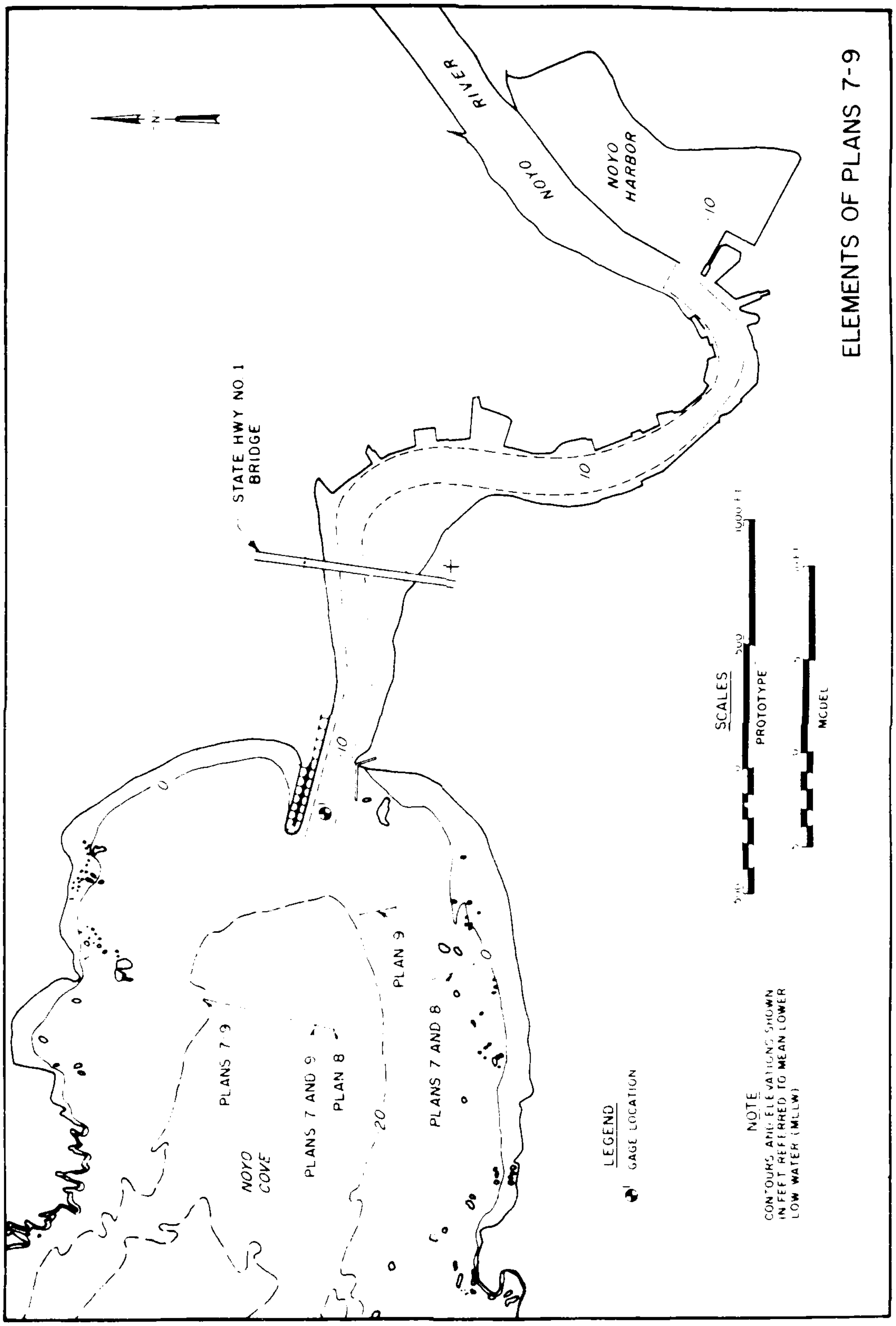
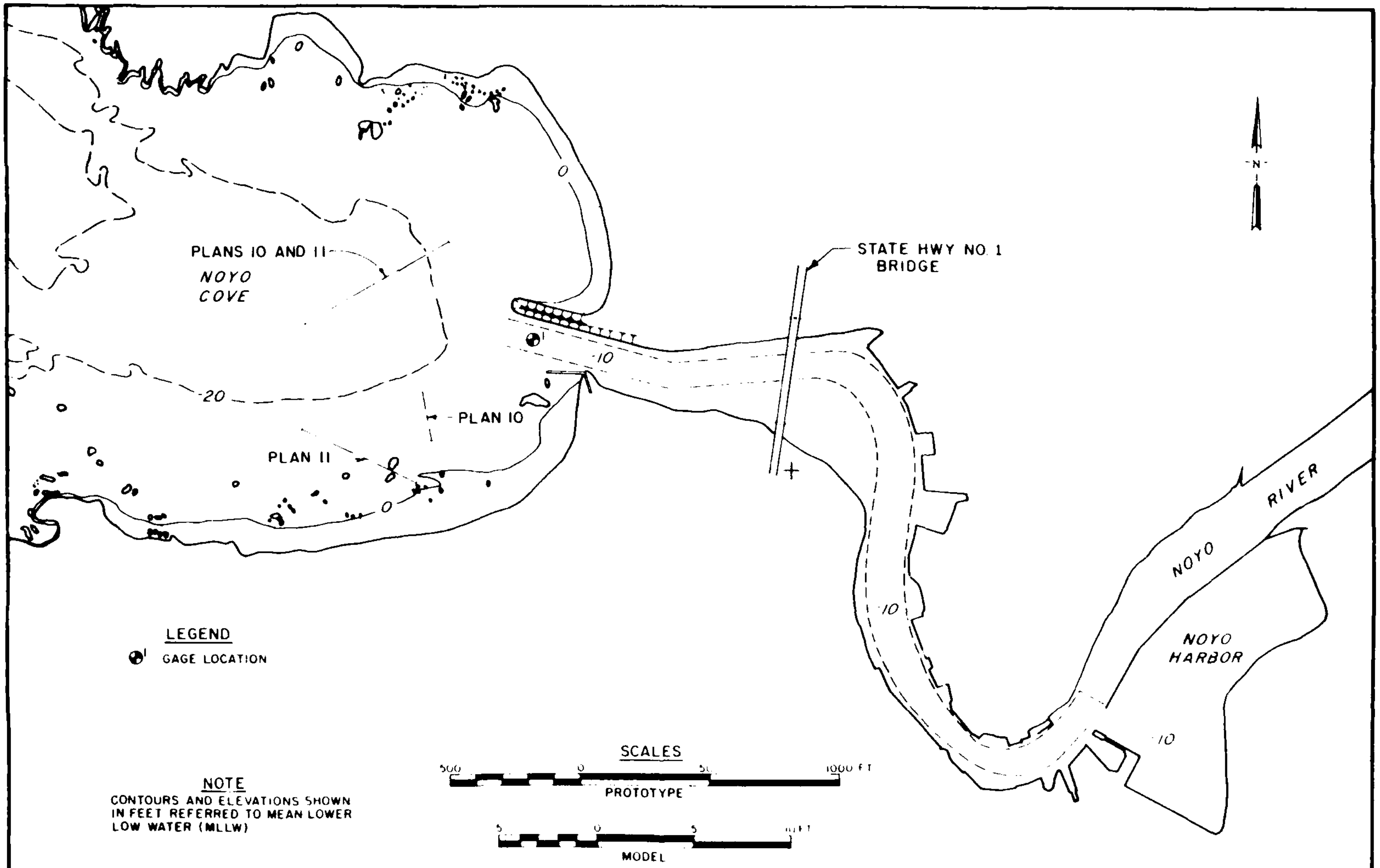

ELEMENTS OF PLAN 4
N
NOYO RIVER
NOYO HARBOR
STATE HWY NO. 1 BRIDGE
NOYO COVE
EL +20.2
SCALES
500 0 500 1000 FT
PROTOTYPE
5 0 5 10 FT
MODEL
LEGEND
GAGE LOCATION
NOTES
1 GAGE 9 IS LOCATED IN THE CENTER OF DOLPHIN MARINA (SEE PLATE 1)
2 CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)

PLATE 4

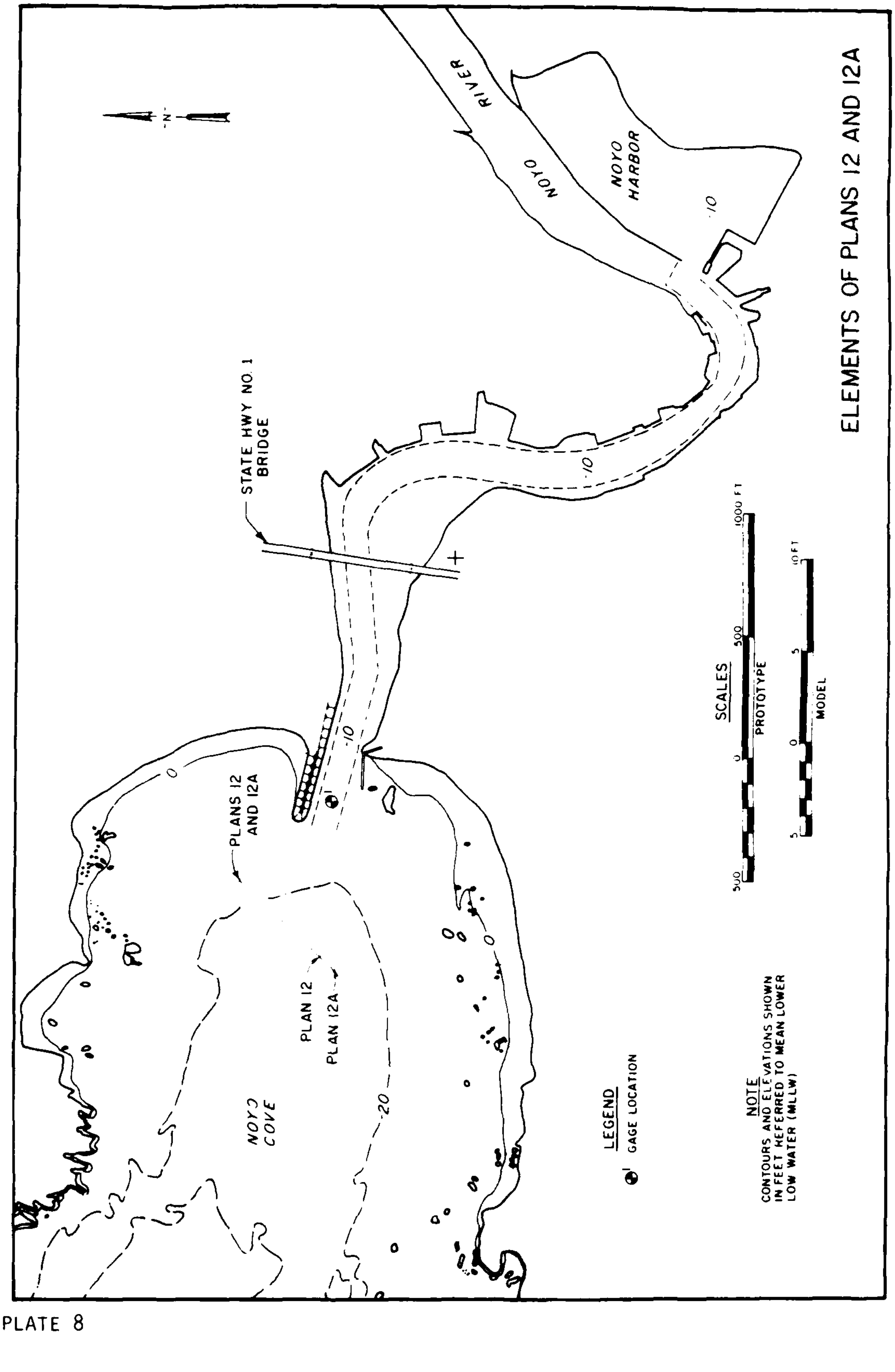


ELEMENTS OF PLANS 5 AND 6

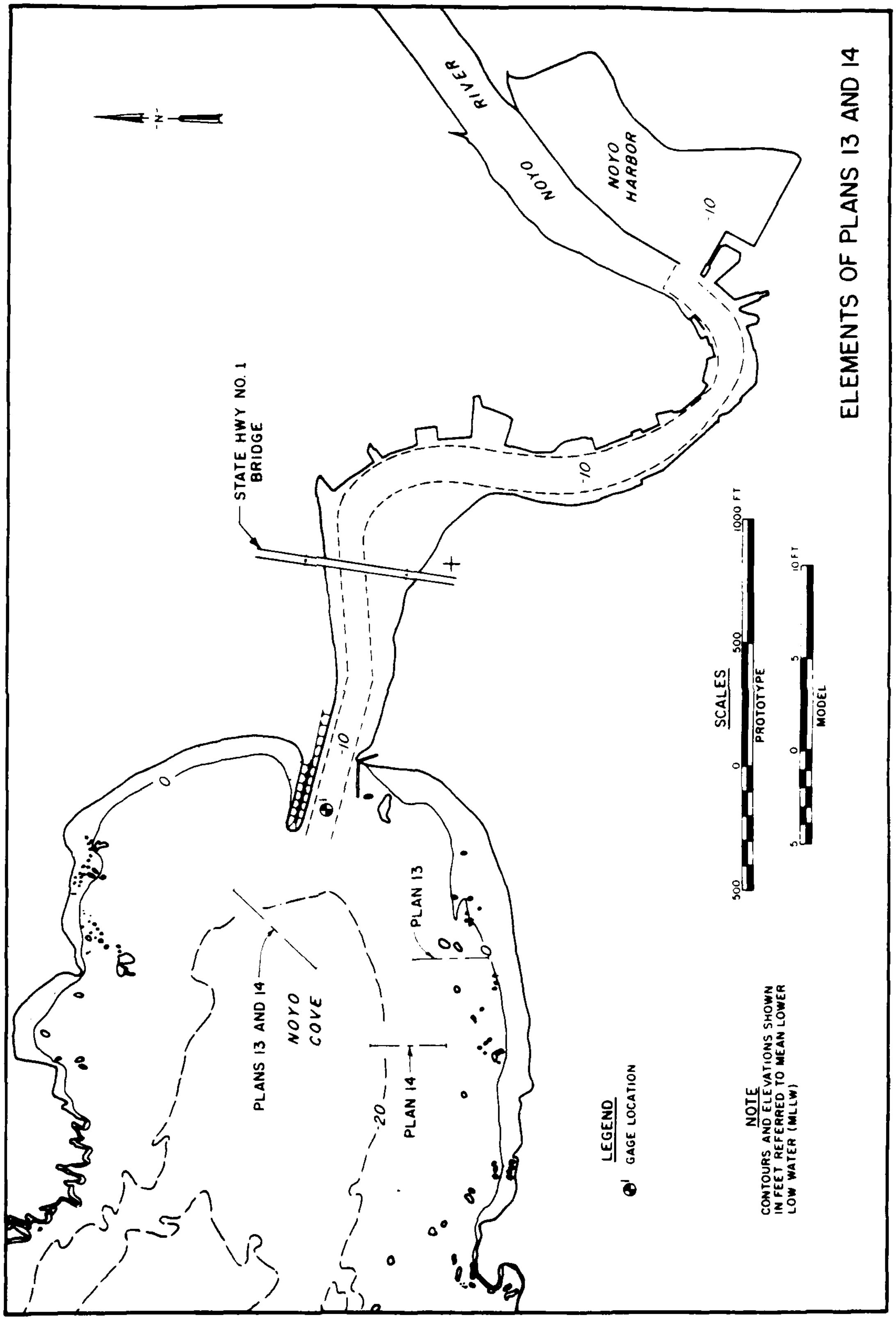

ELEMENTS OF PLANS 7-9
NOYO RIVER
NOYO HARBOR
STATE HWY NO 1 BRIDGE
NOYO COVE
PLANS 7-9
PLANS 7 AND 9
PLAN 8
PLAN 9
PLANS 7 AND 8
SCALES
PROTOTYPE
MODEL
LEGEND
GAGE LOCATION
NOTE
CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)



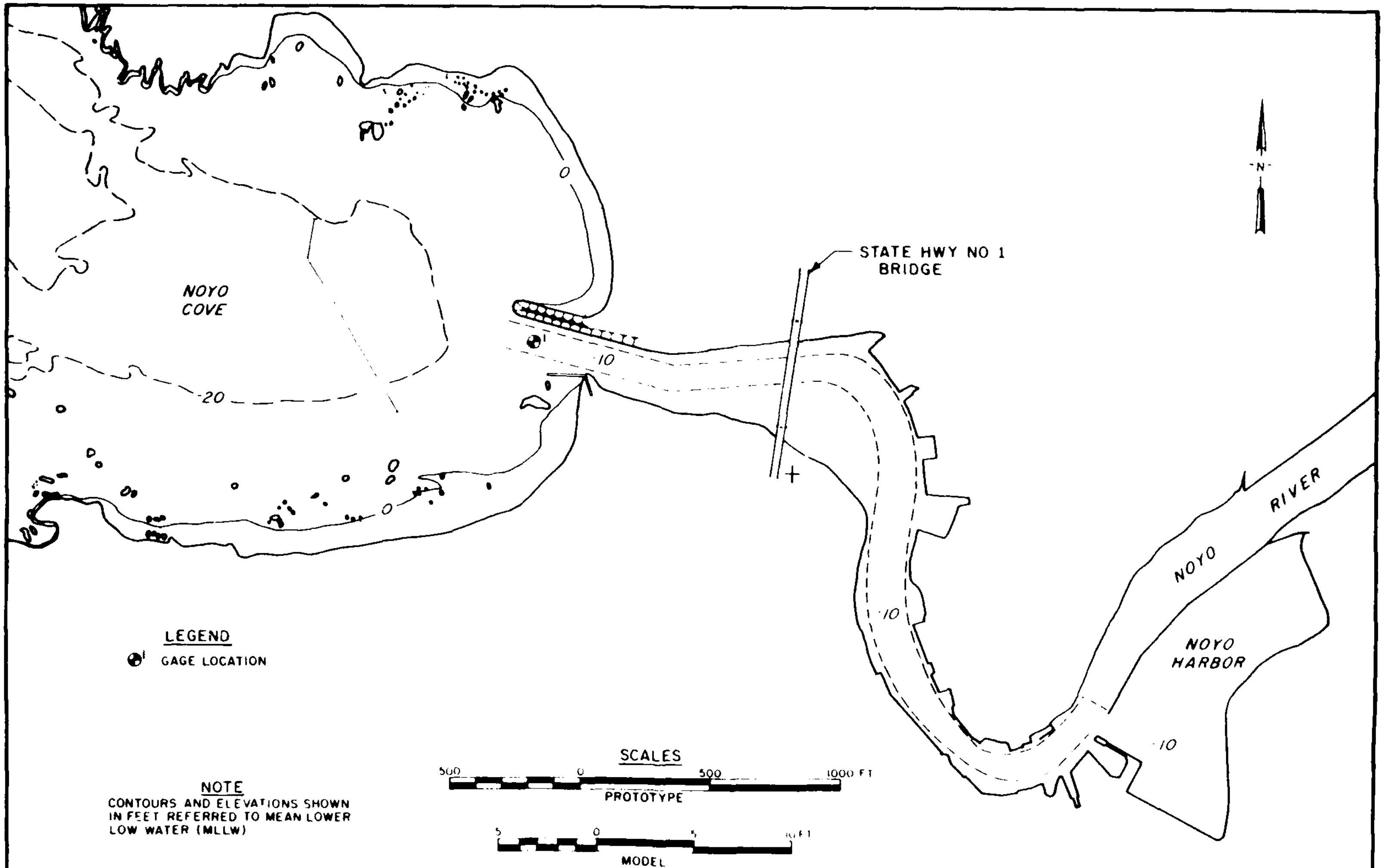
ELEMENTS OF PLANS 10 AND 11



ELEMENTS OF PLANS 12 AND 12A

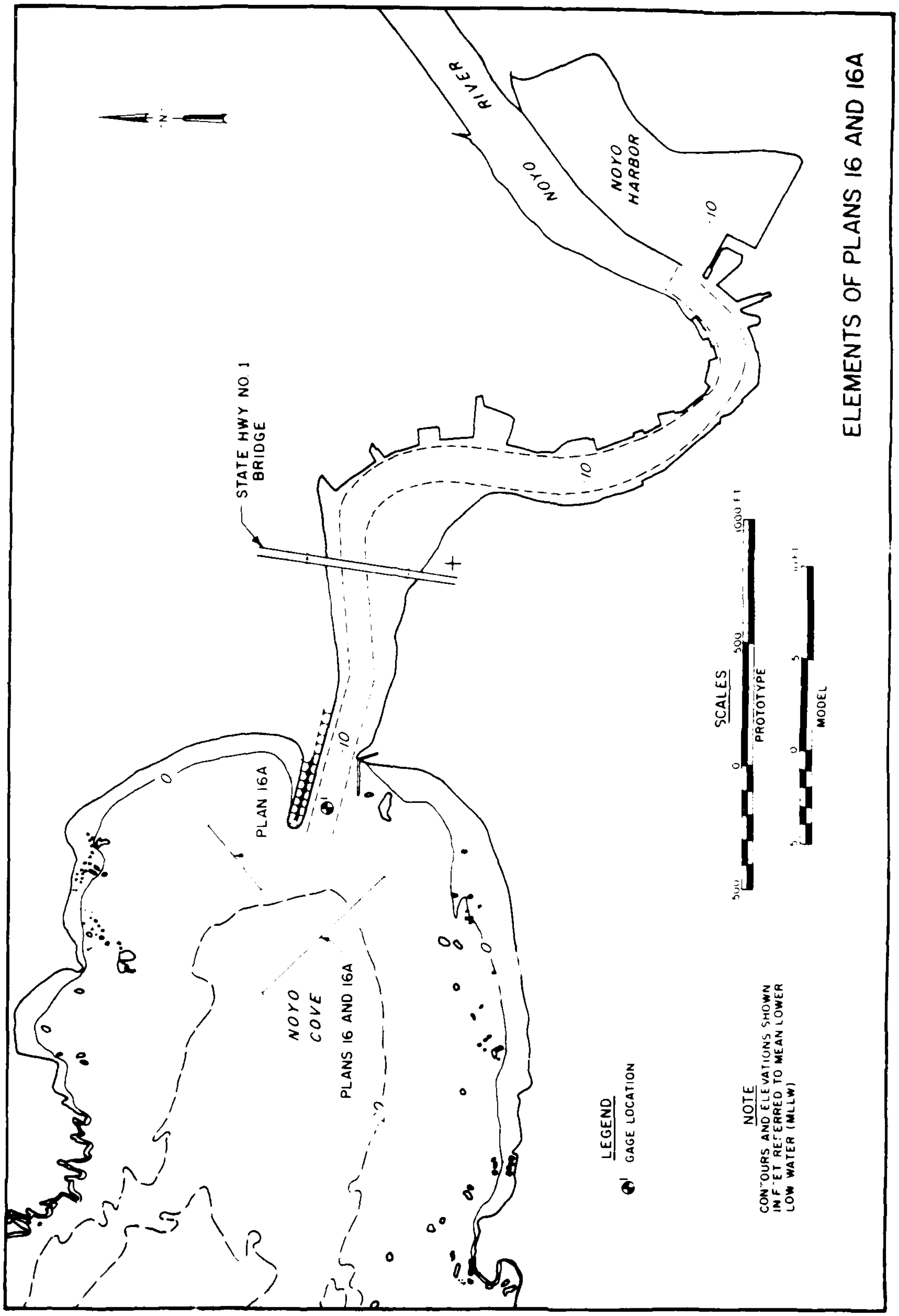


ELEMENTS OF PLANS 13 AND 14
N
NOYO RIVER
NOYO HARBOR
-10
STATE HWY NO. 1 BRIDGE
-10
0
NOYO COVE
PLANS 13 AND 14
PLAN 13
PLAN 14
-20
SCALES
500 0 500 1000 FT
PROTOTYPE
5 0 5 10 FT
MODEL
LEGEND
GAGE LOCATION
NOTE
CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)

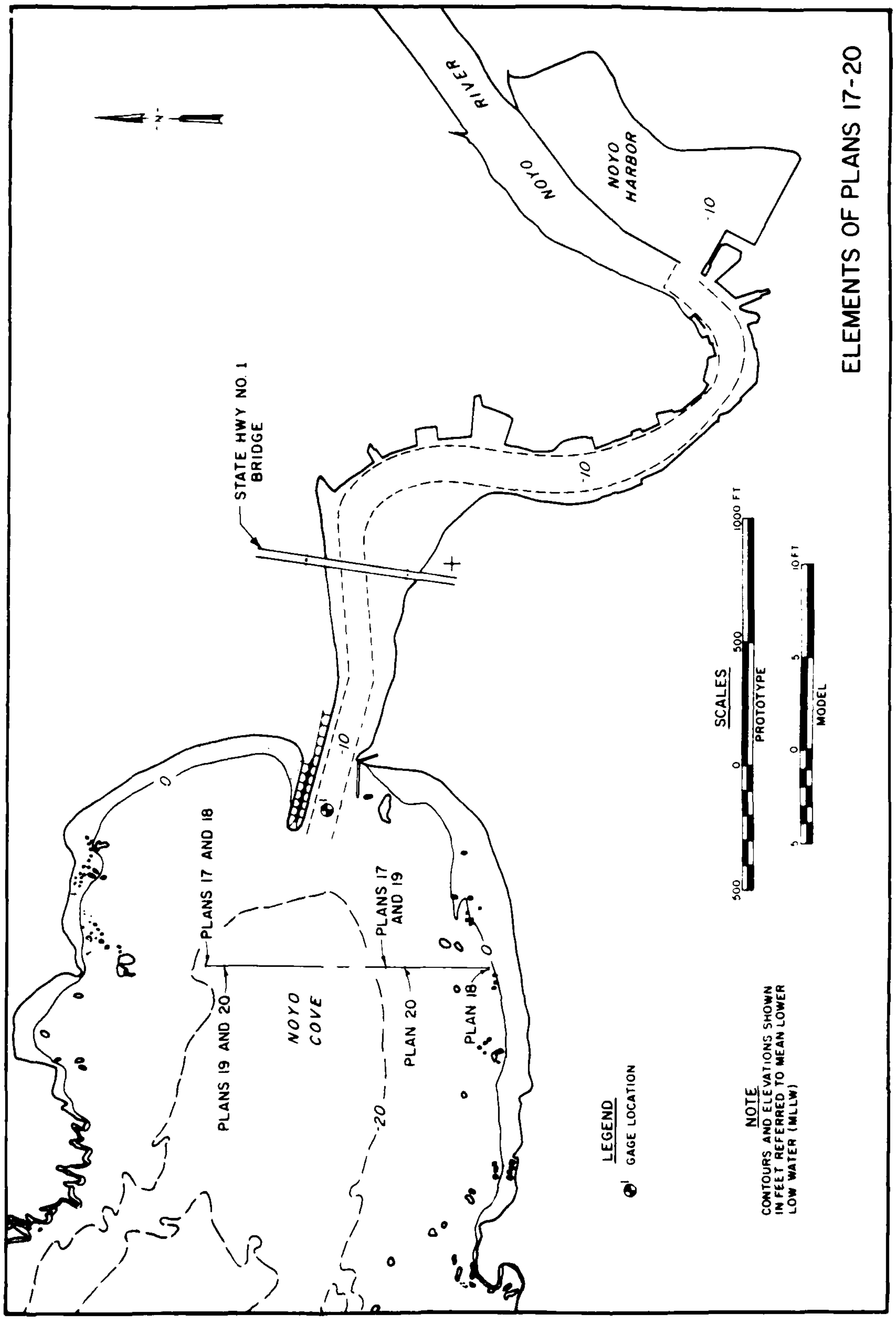


ELEMENTS OF PLAN 15

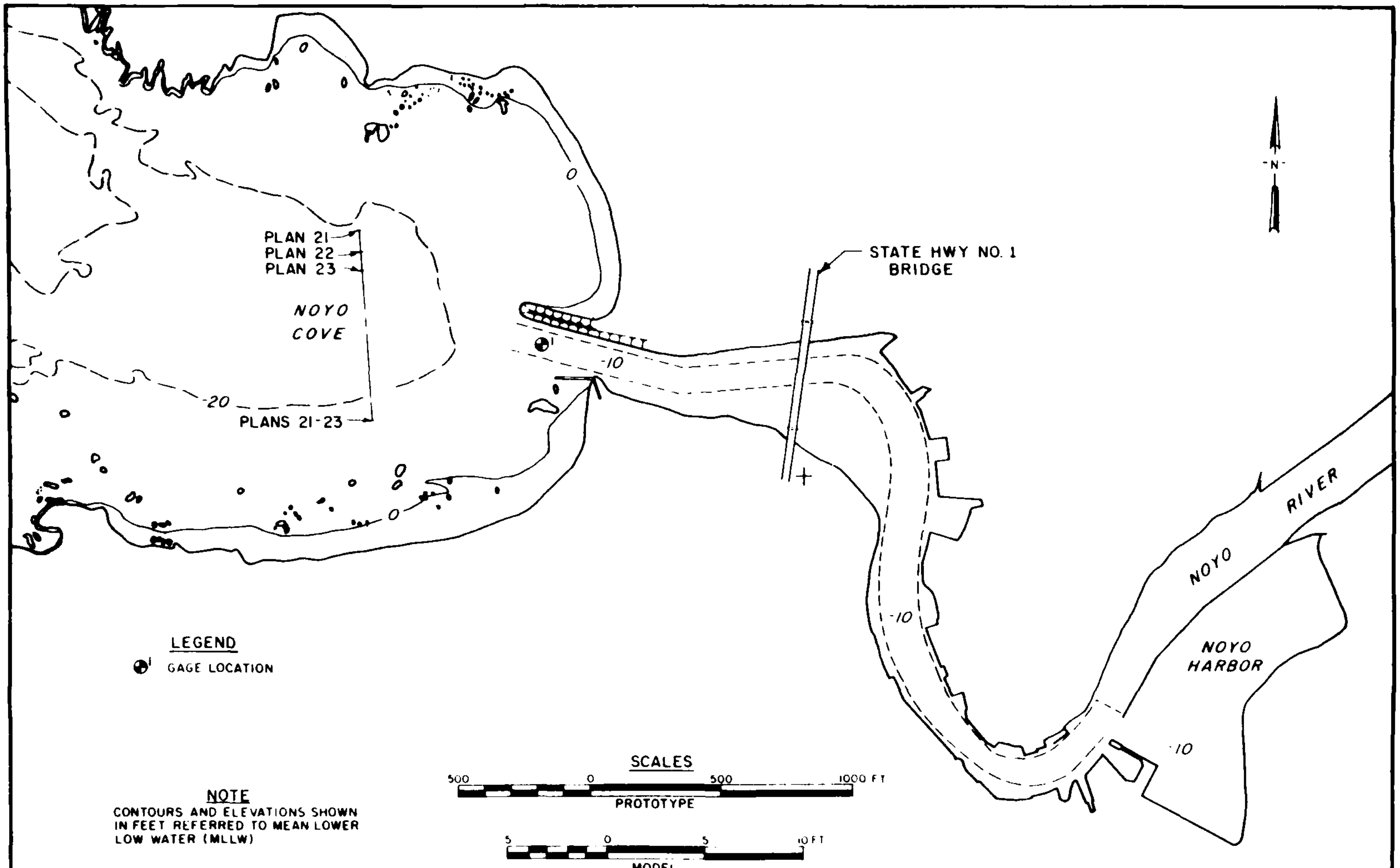
PLATE 10



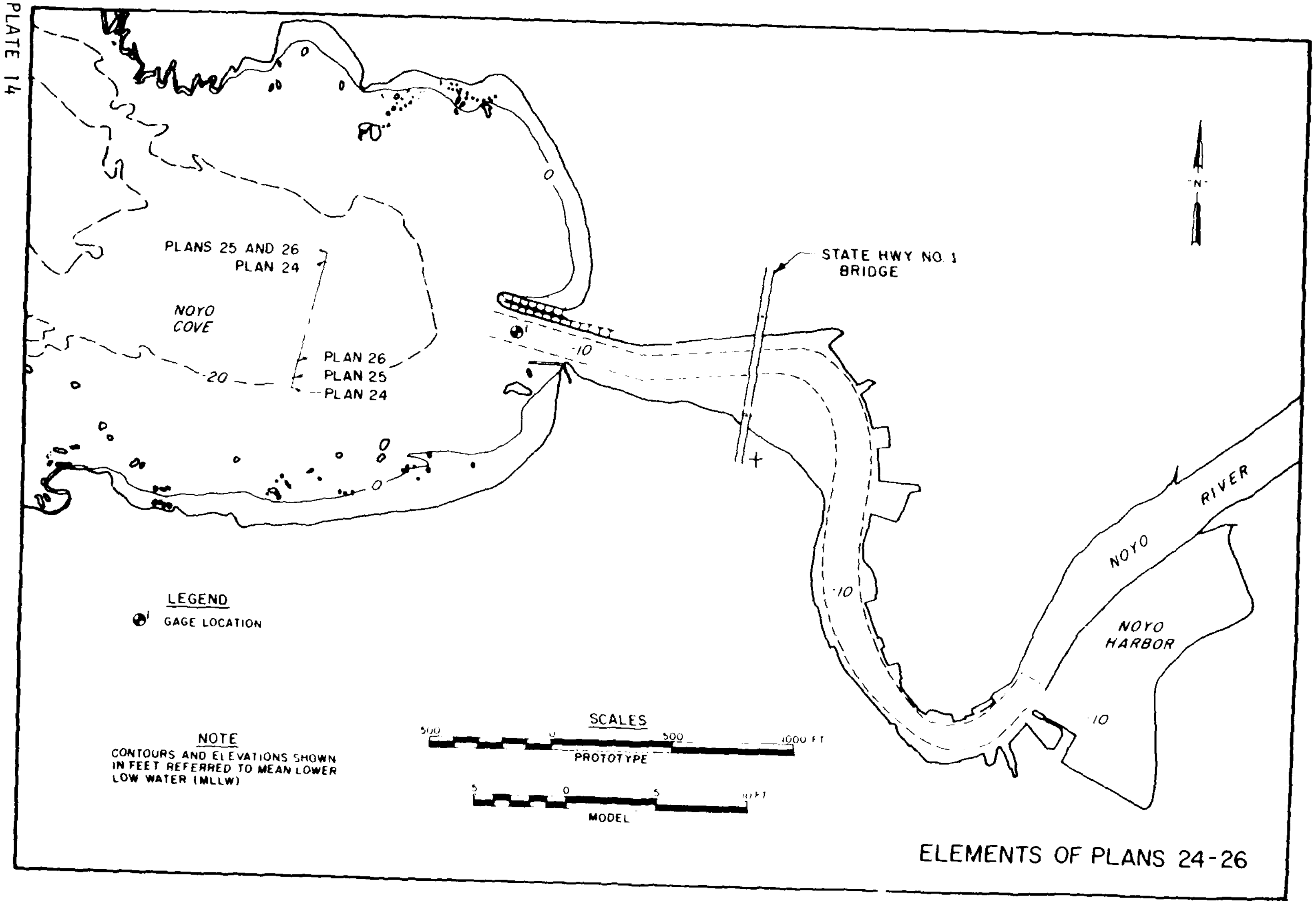
ELEMENTS OF PLANS 16 AND 16A



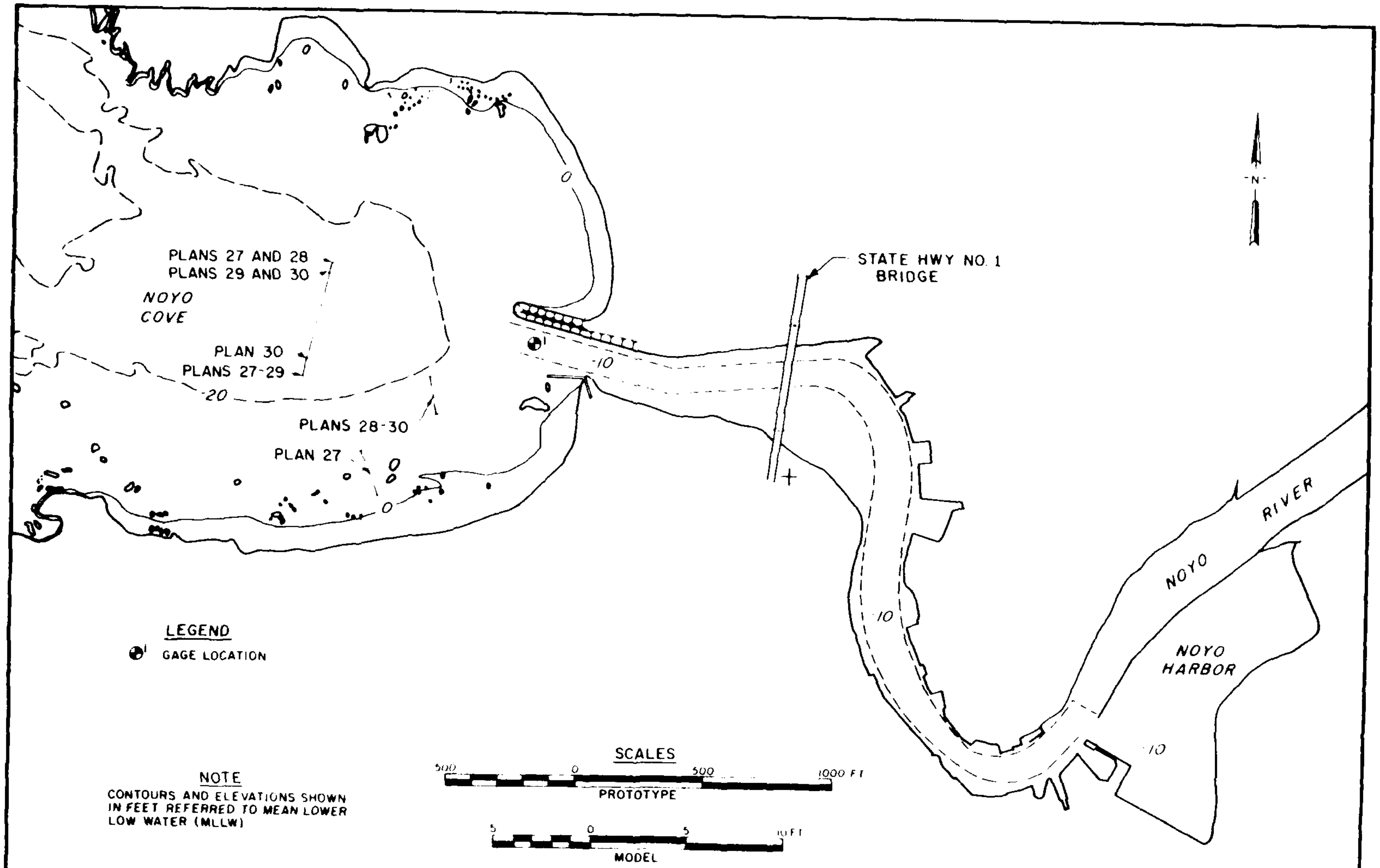
ELEMENTS OF PLANS 17-20



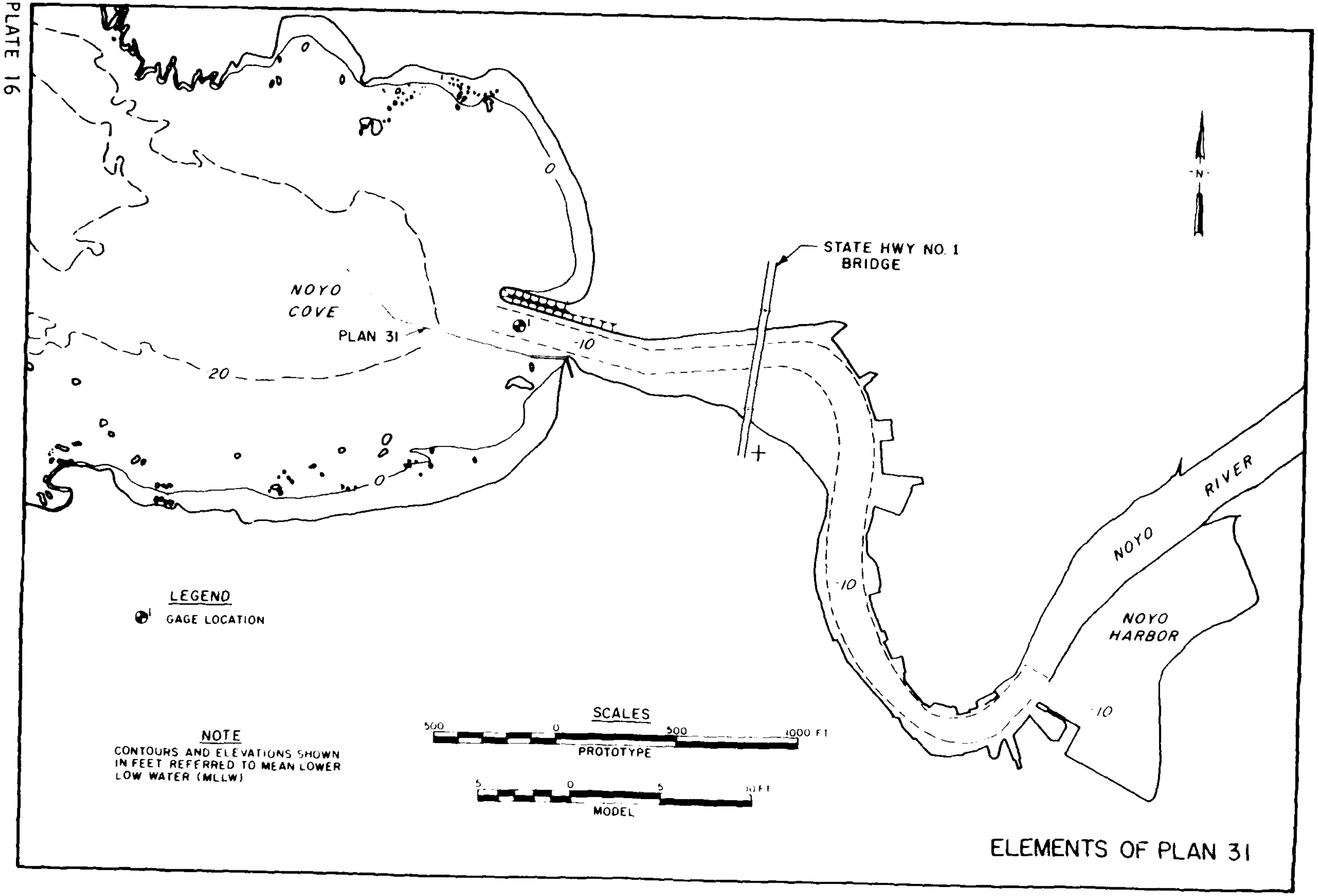
ELEMENTS OF PLANS 21-23



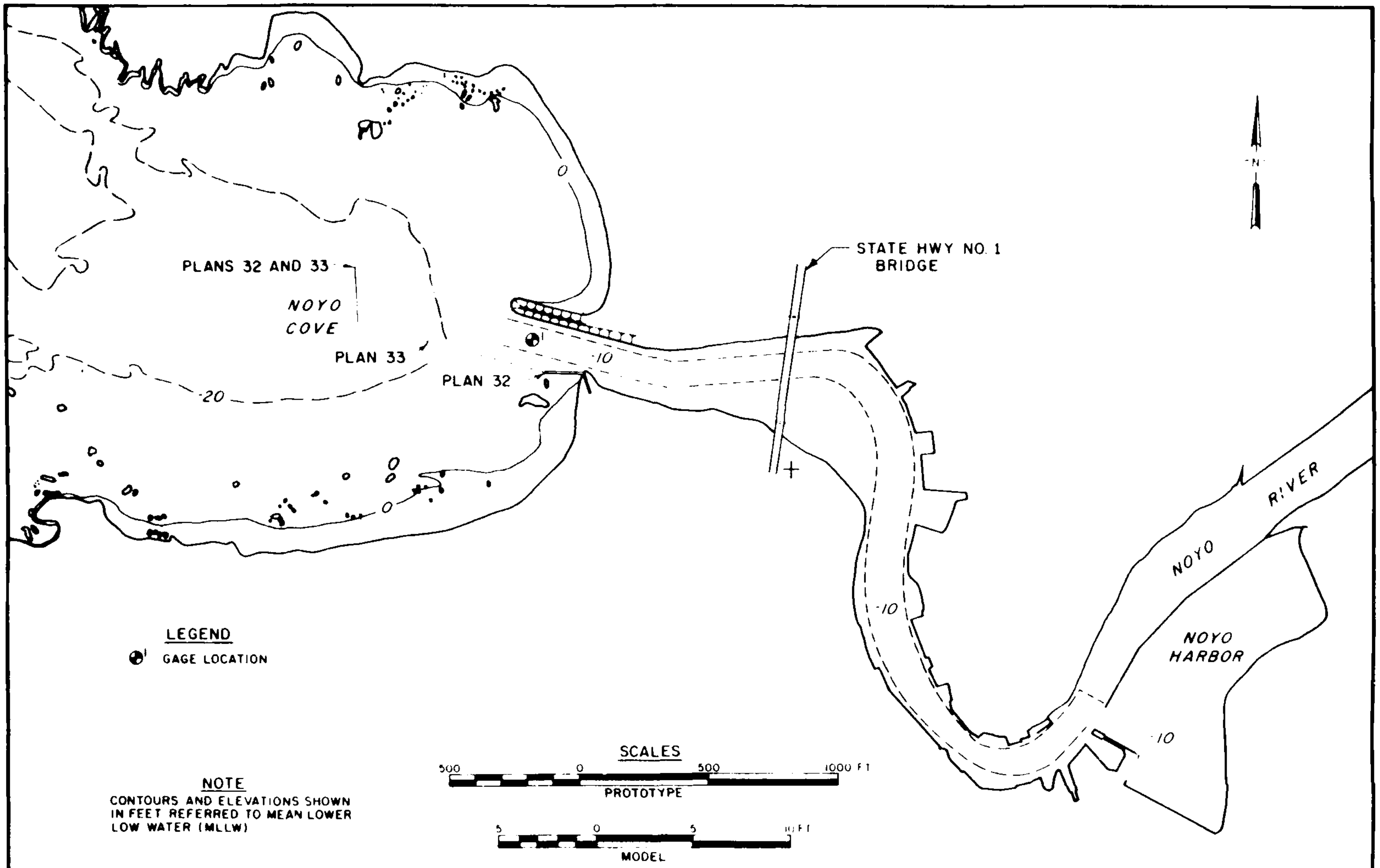
ELEMENTS OF PLANS 24-26



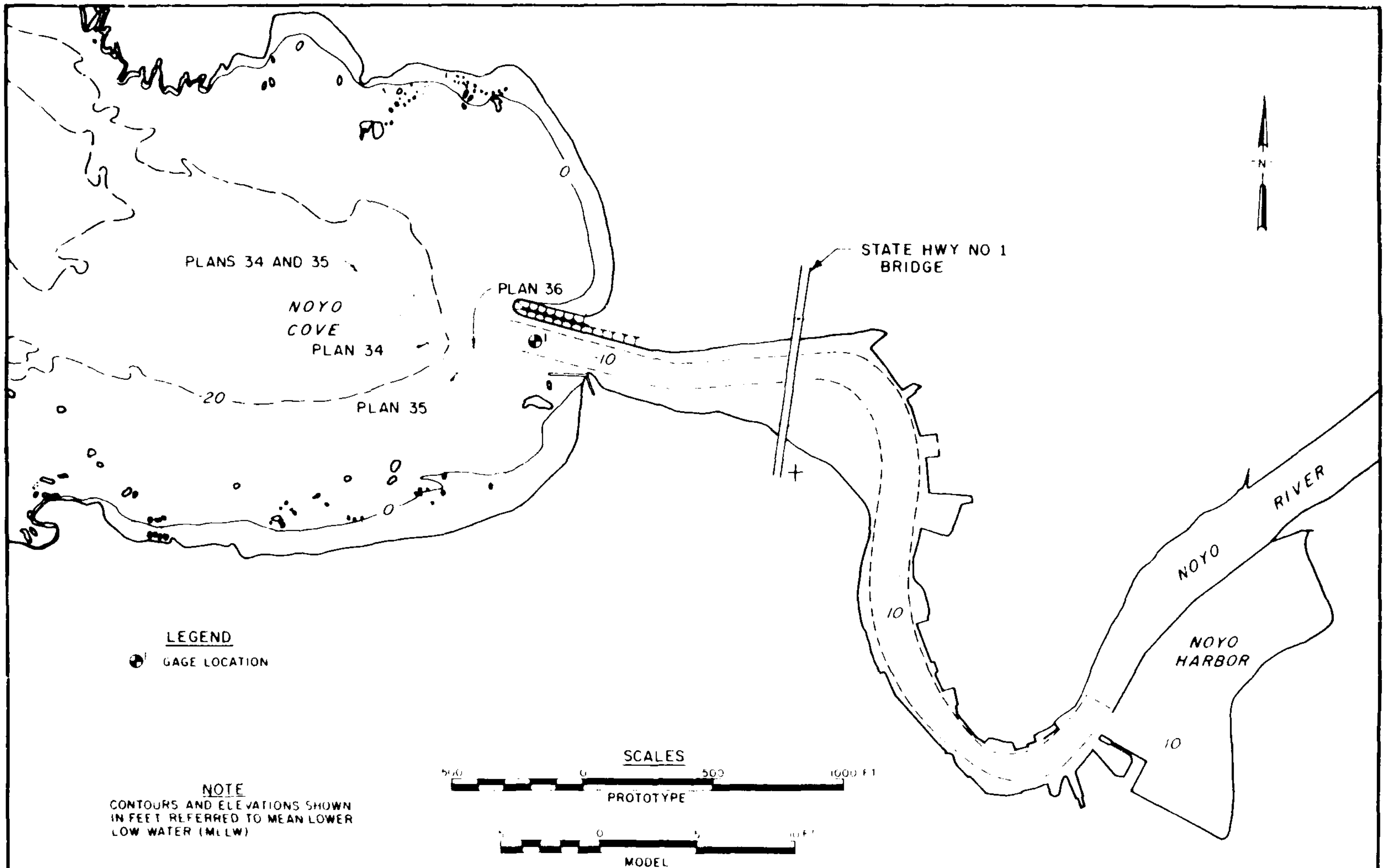
ELEMENTS OF PLANS 27-30



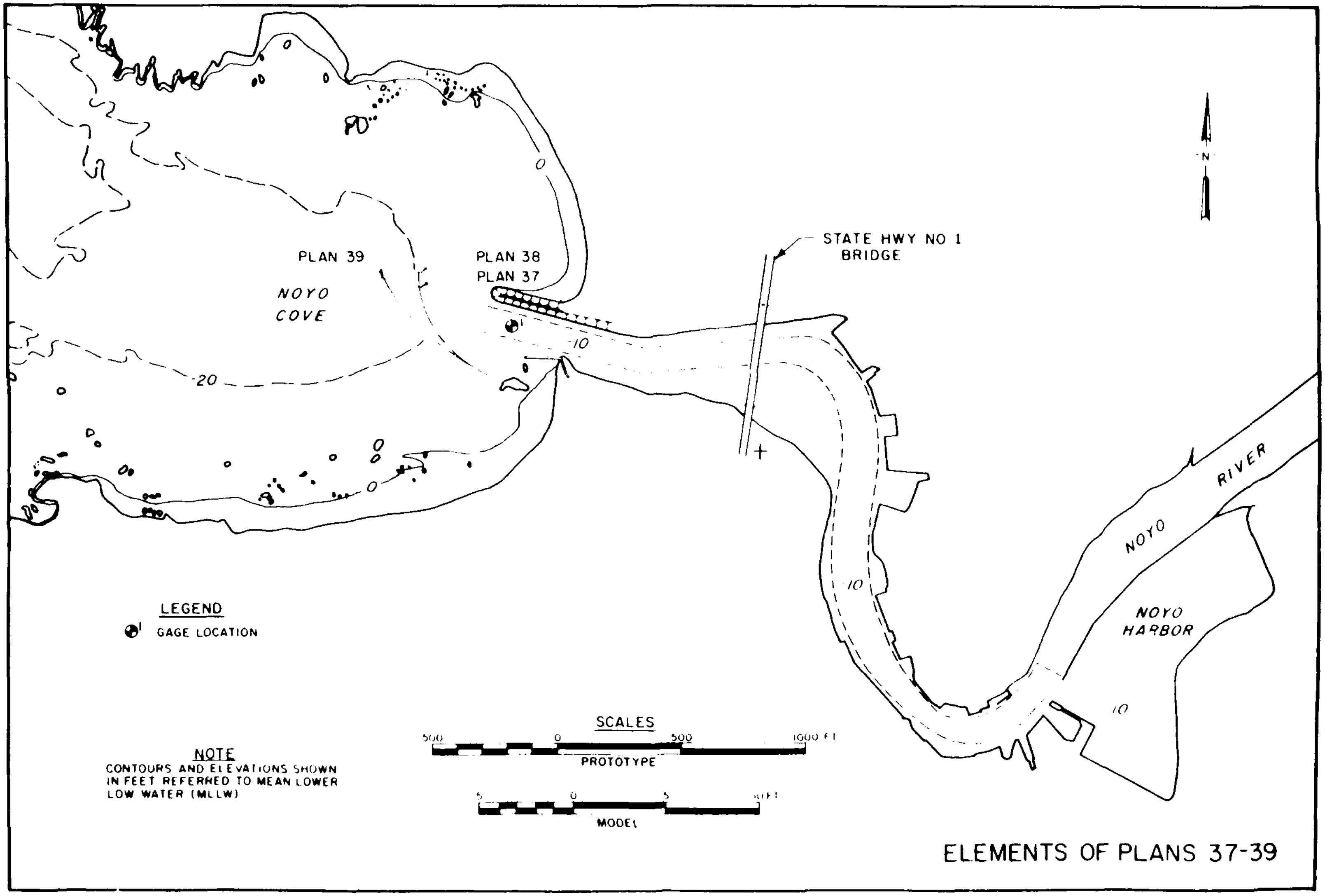
ELEMENTS OF PLAN 31



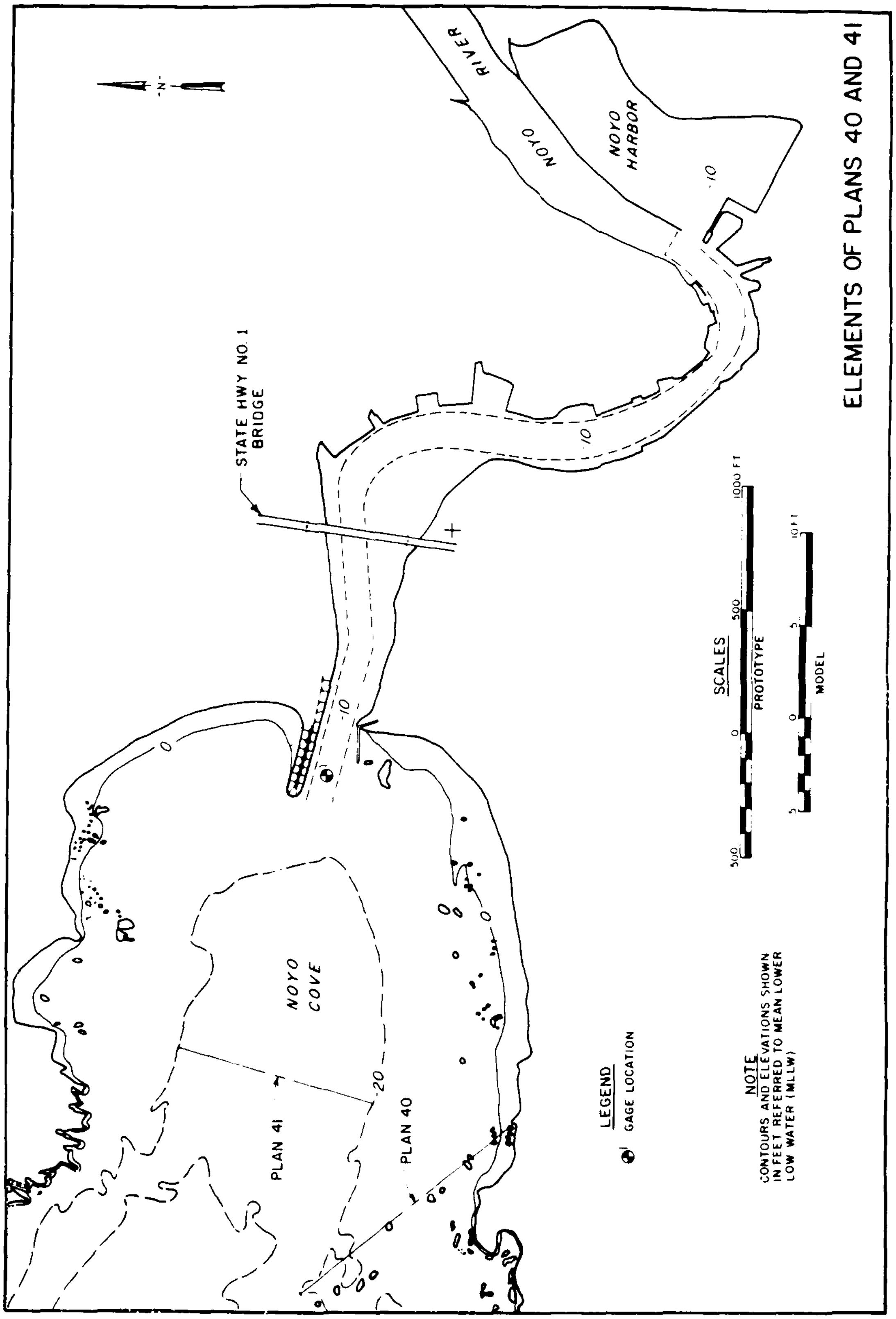
ELEMENTS OF PLANS 32 AND 33



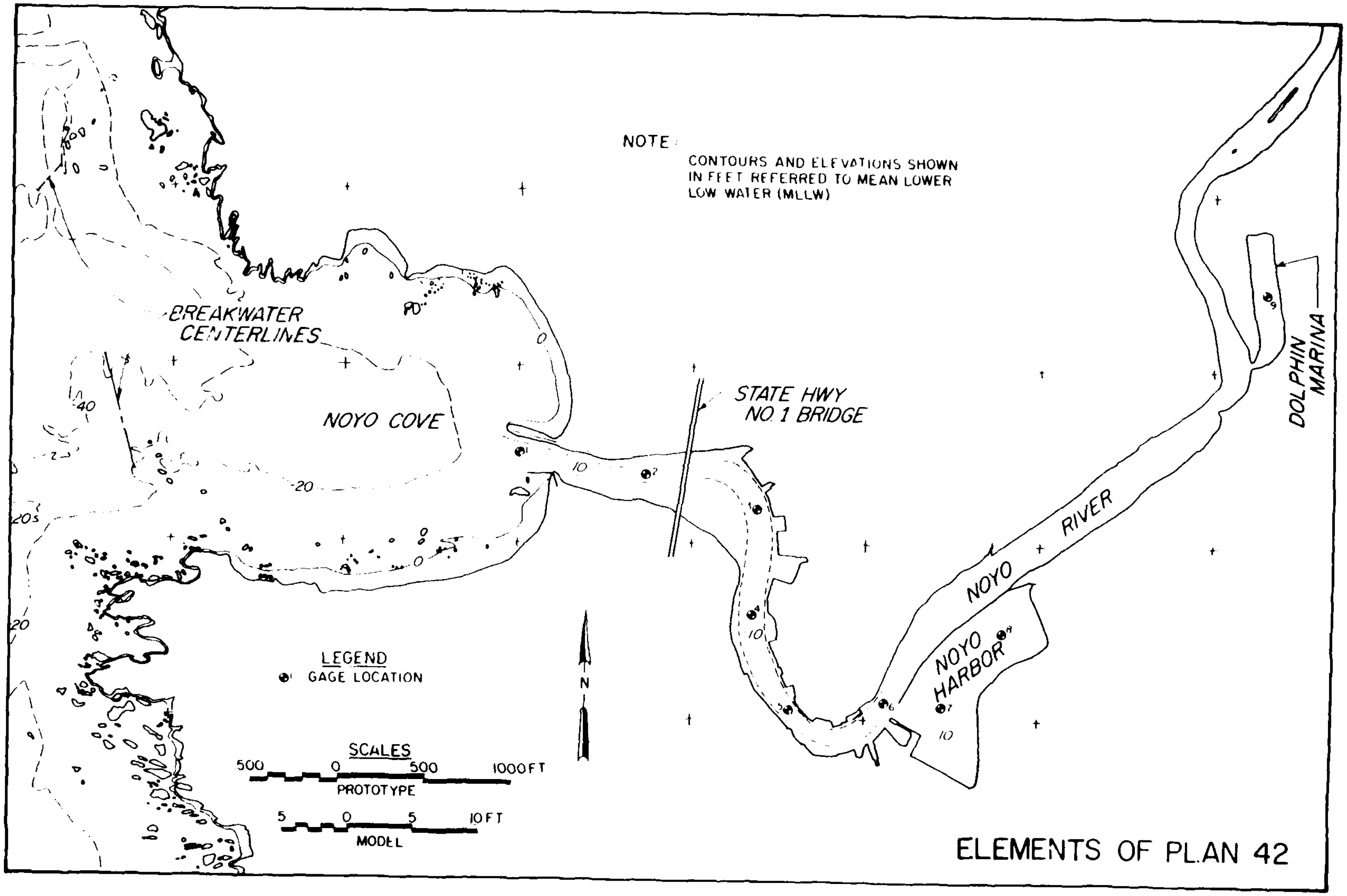
ELEMENTS OF PLANS 34-36



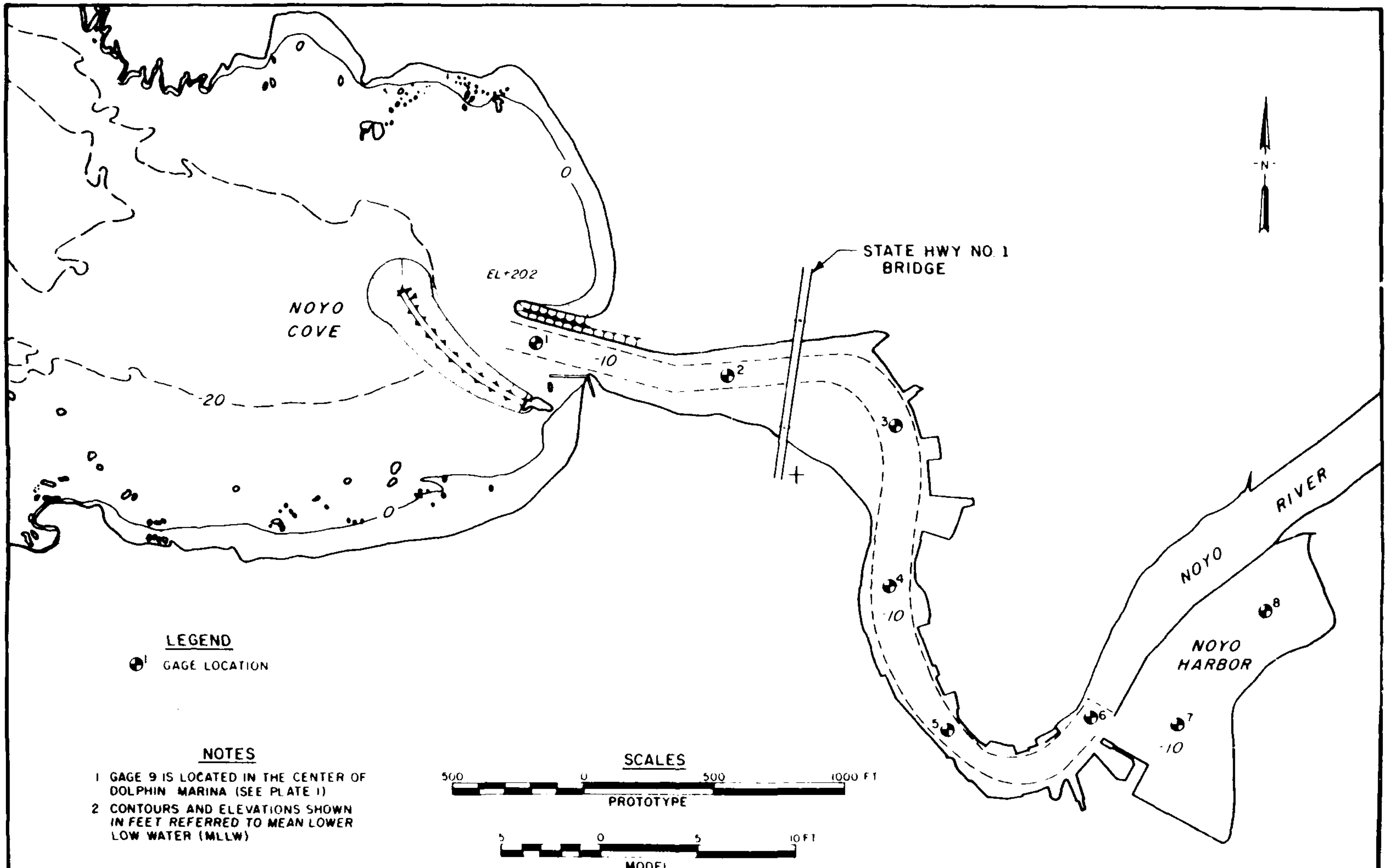
ELEMENTS OF PLANS 37-39



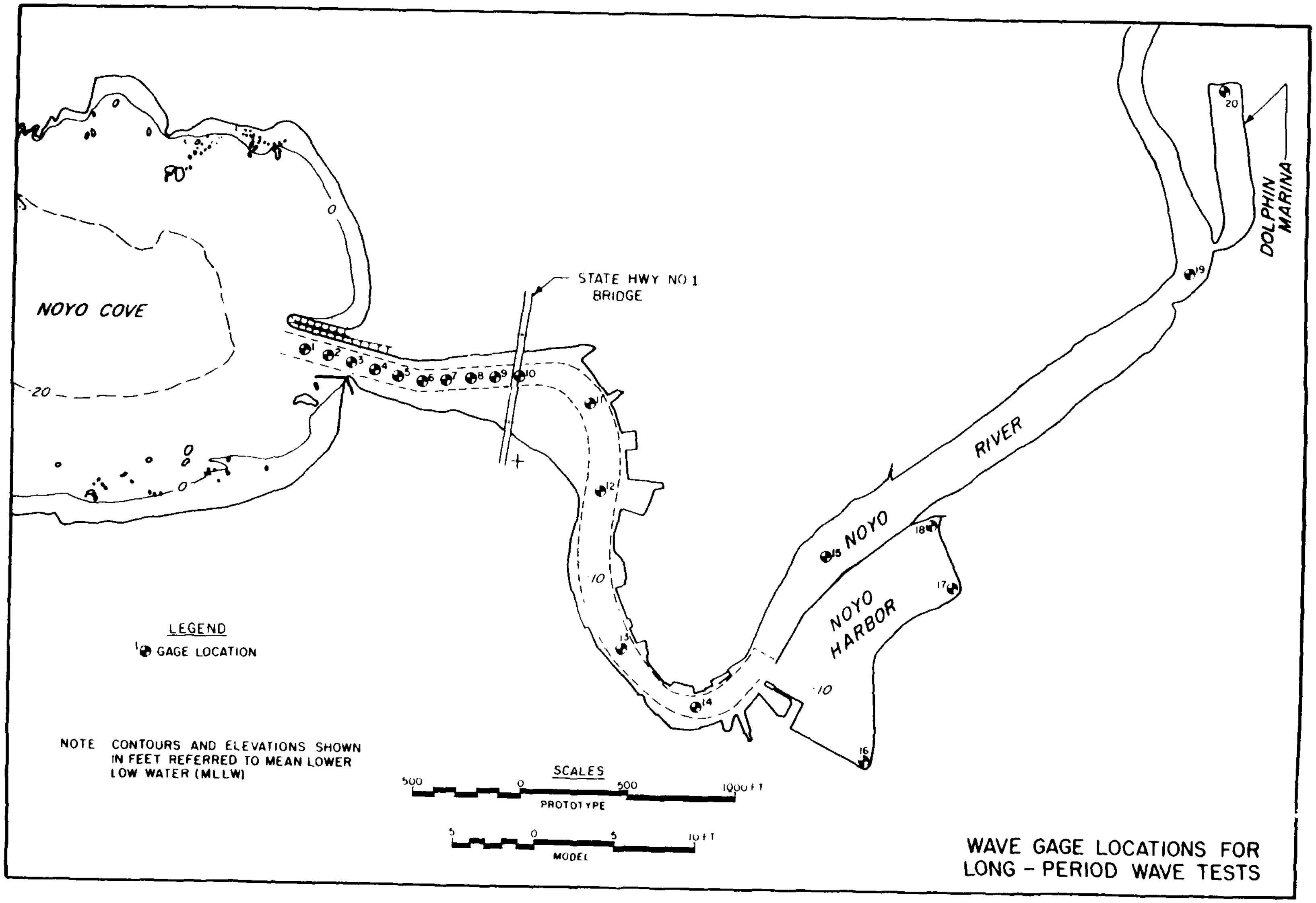
ELEMENTS OF PLANS 40 AND 41



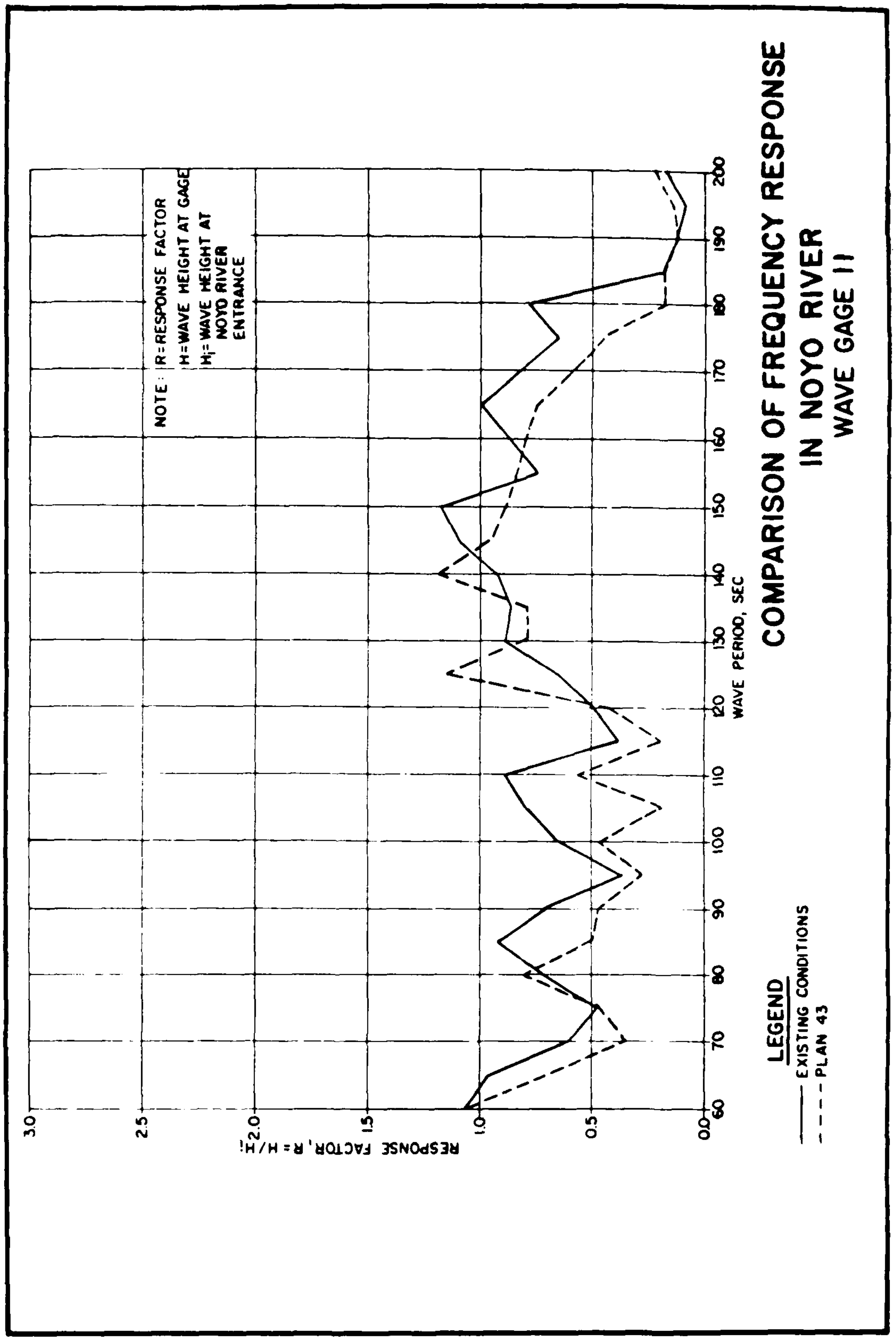
ELEMENTS OF PLAN 42



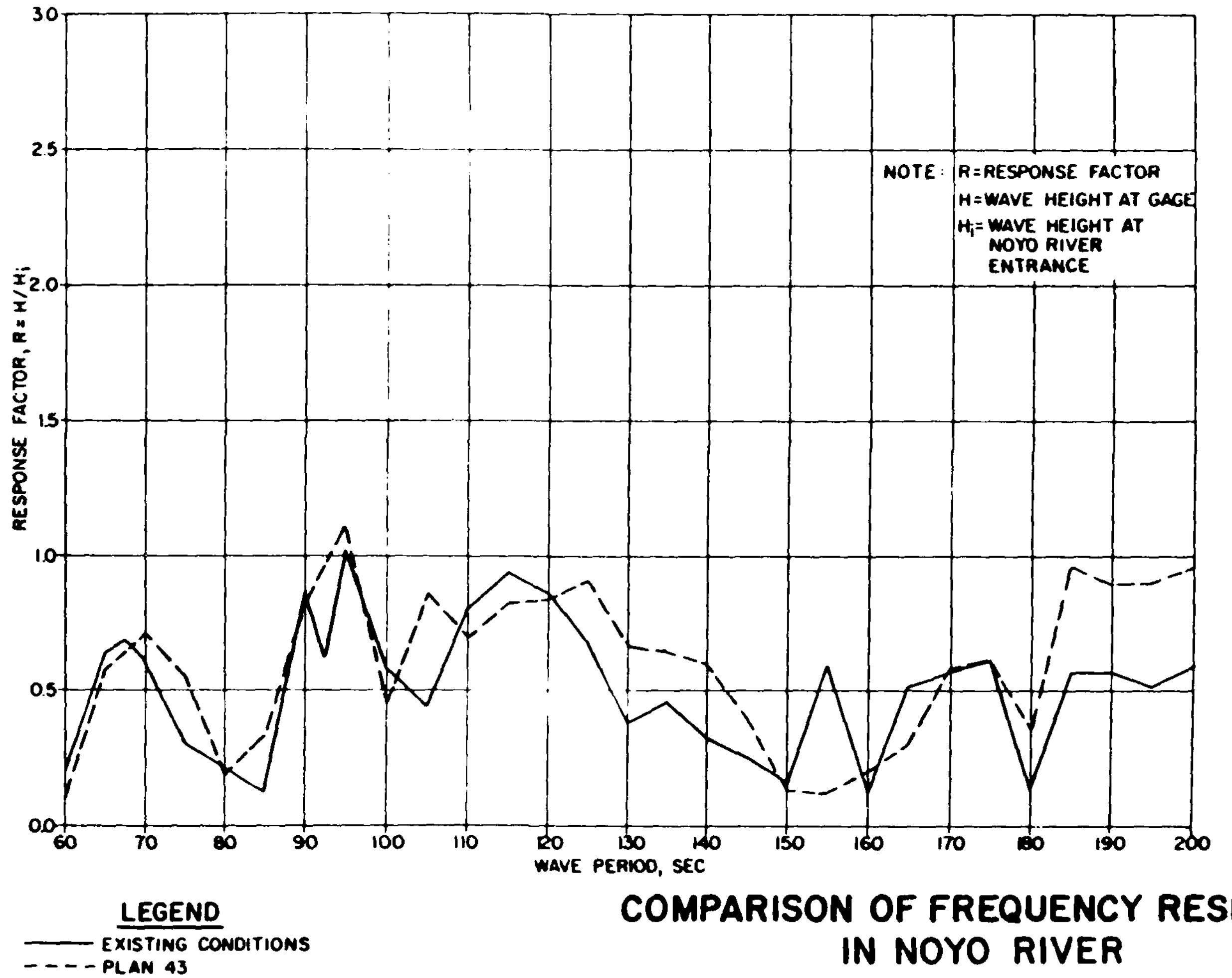
ELEMENTS OF PLAN 43



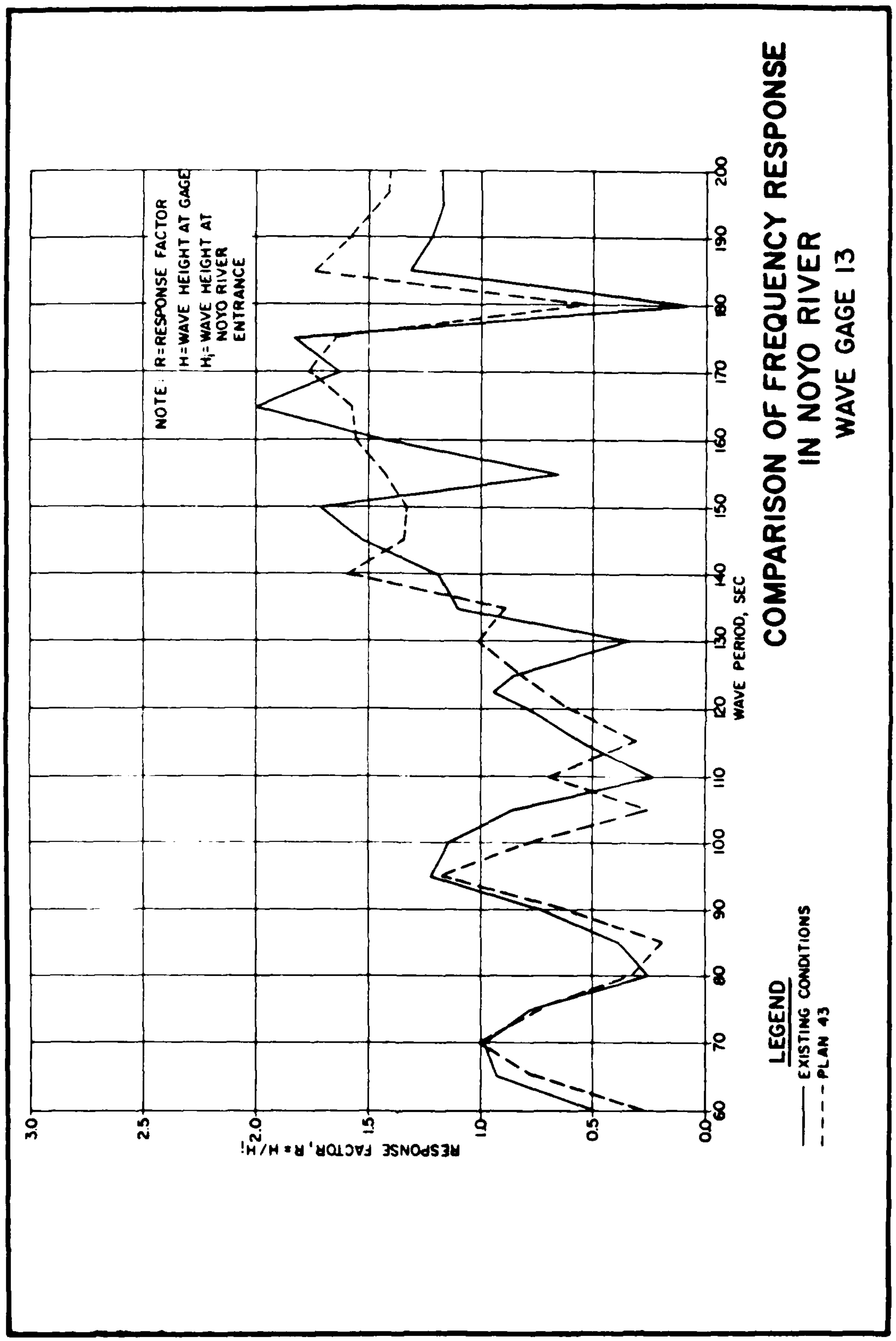
WAVE GAGE LOCATIONS FOR LONG - PERIOD WAVE TESTS

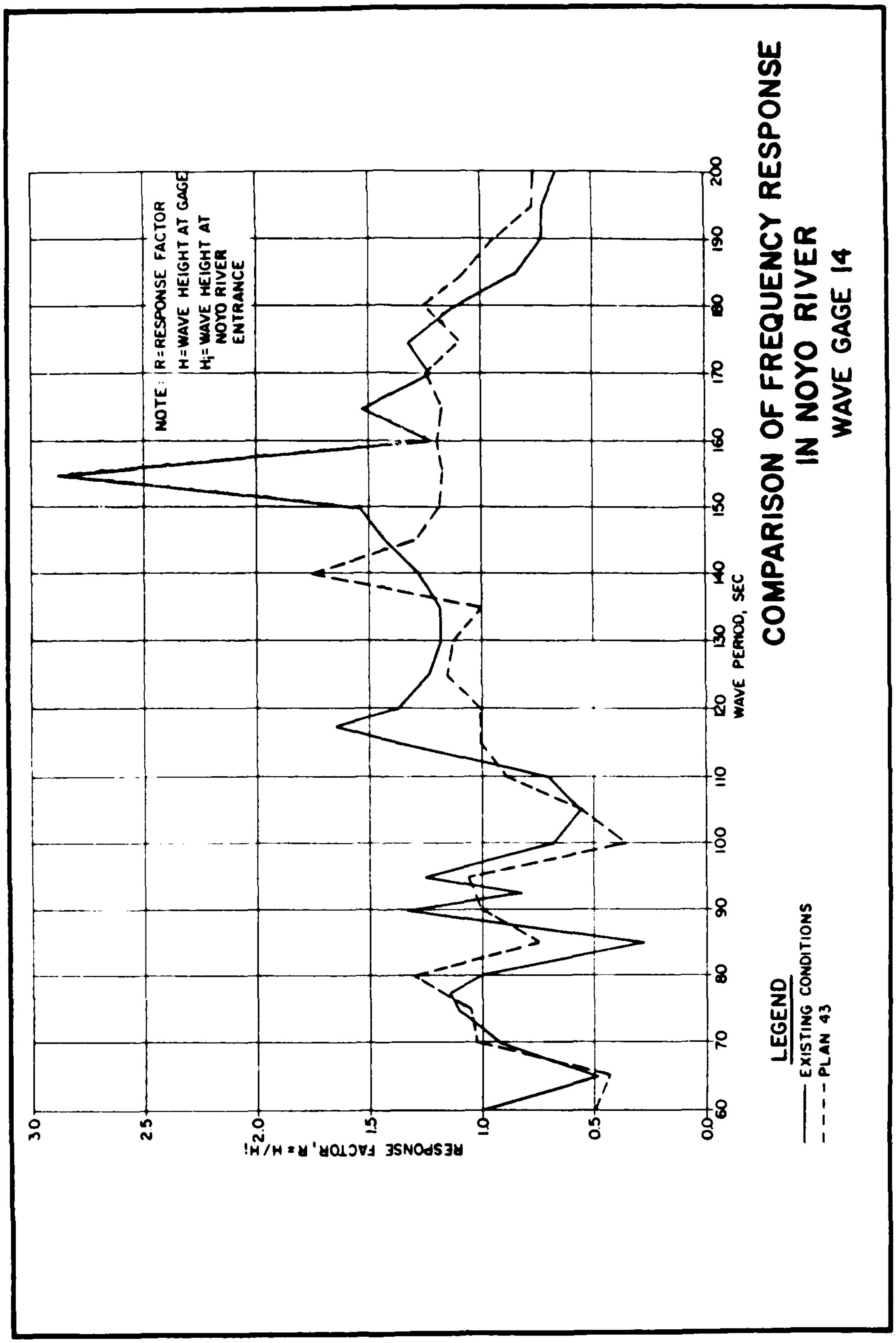


COMPARISON OF FREQUENCY RESPONSE
IN NOYO RIVER
WAVE GAGE 11
NOTE: R=RESPONSE FACTOR
H=WAVE HEIGHT AT GAGE
H_i= WAVE HEIGHT AT NOYO RIVER ENTRANCE
LEGEND
EXISTING CONDITIONS
PLAN 43
RESPONSE FACTOR, R = H/H_i
WAVE PERIOD, SEC
0.0
0.5
1.0
1.5
2.0
2.5
3.0
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200

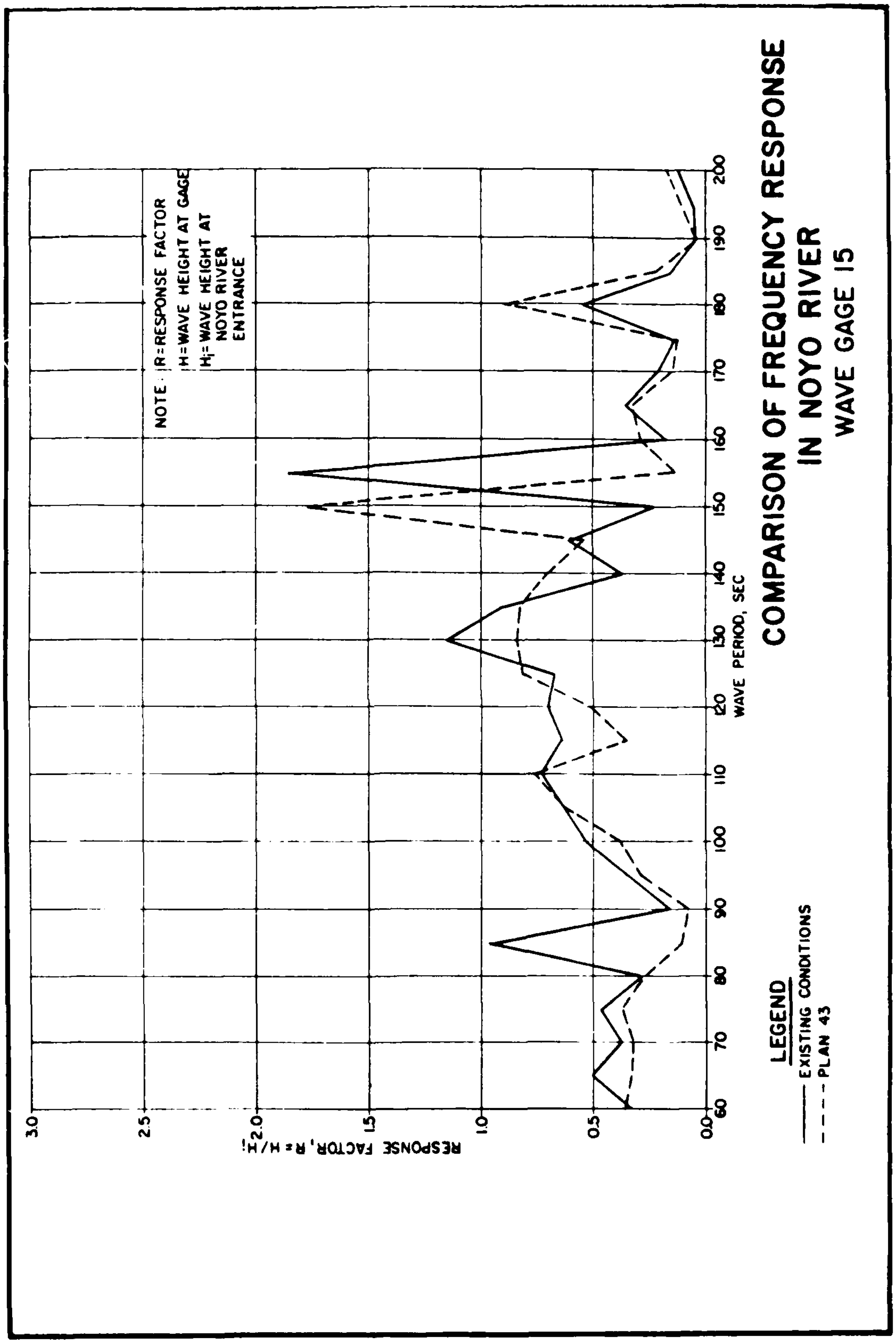


COMPARISON OF FREQUENCY RESPONSE
IN NOYO RIVER
WAVE GAGE 12


COMPARISON OF FREQUENCY RESPONSE
IN NOYO RIVER
WAVE GAGE 13
NOTE: R=RESPONSE FACTOR
H=WAVE HEIGHT AT GAGE
H_i=WAVE HEIGHT AT NOYO RIVER ENTRANCE
RESPONSE FACTOR, R = H/H_i
WAVE PERIOD, SEC
0.0
0.5
1.0
1.5
2.0
2.5
3.0
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
LEGEND
EXISTING CONDITIONS
PLAN 43

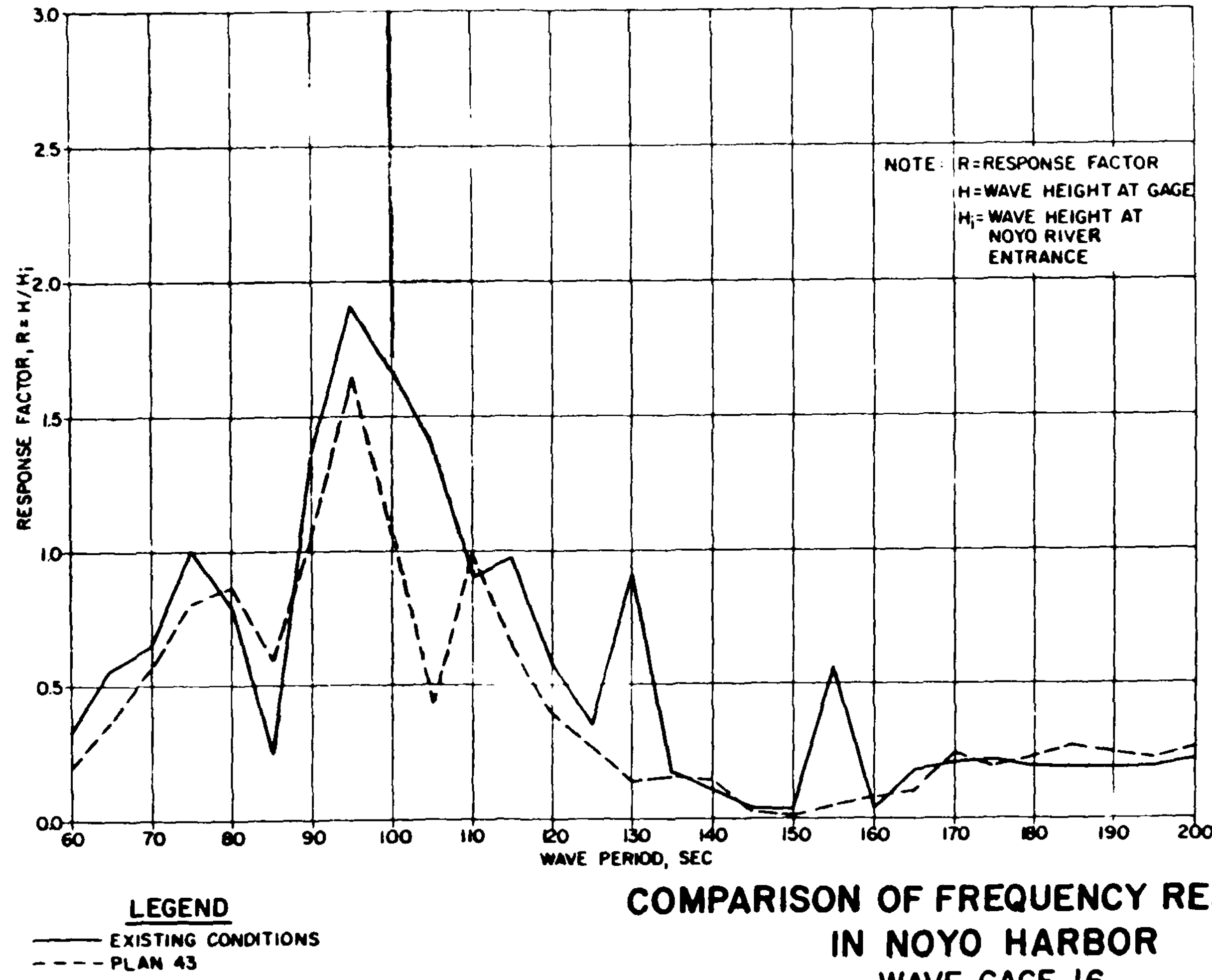


COMPARISON OF FREQUENCY RESPONSE
IN NOYO RIVER
WAVE GAGE 14

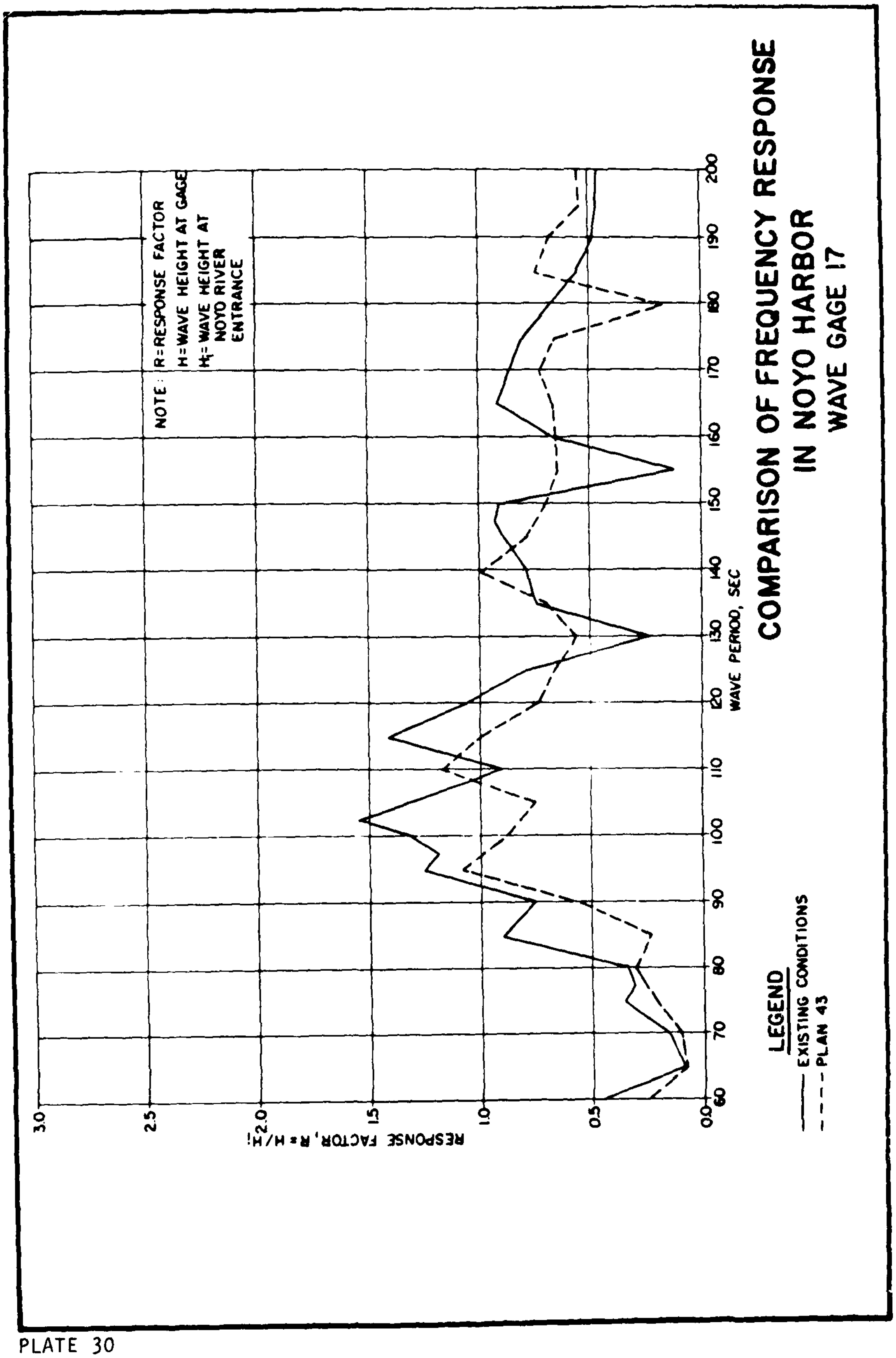


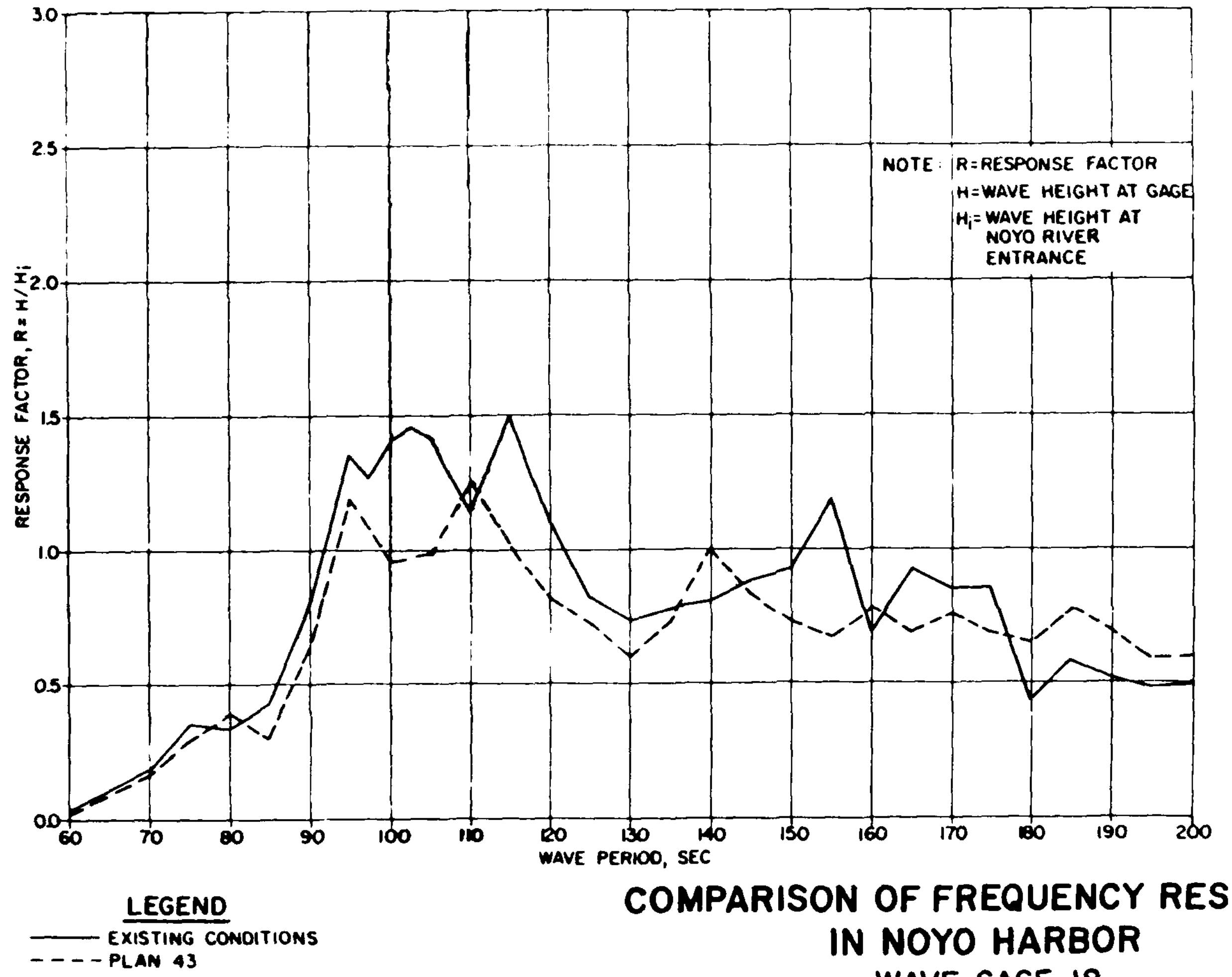
COMPARISON OF FREQUENCY RESPONSE
IN NOYO RIVER
WAVE GAGE 15
NOTE: R=RESPONSE FACTOR
H=WAVE HEIGHT AT GAGE
Hi=WAVE HEIGHT AT NOYO RIVER ENTRANCE
LEGEND
EXISTING CONDITIONS
PLAN 43
RESPONSE FACTOR, R=H/Hi
WAVE PERIOD, SEC
0.0
0.5
1.0
1.5
2.0
2.5
3.0
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200

PLATE 28



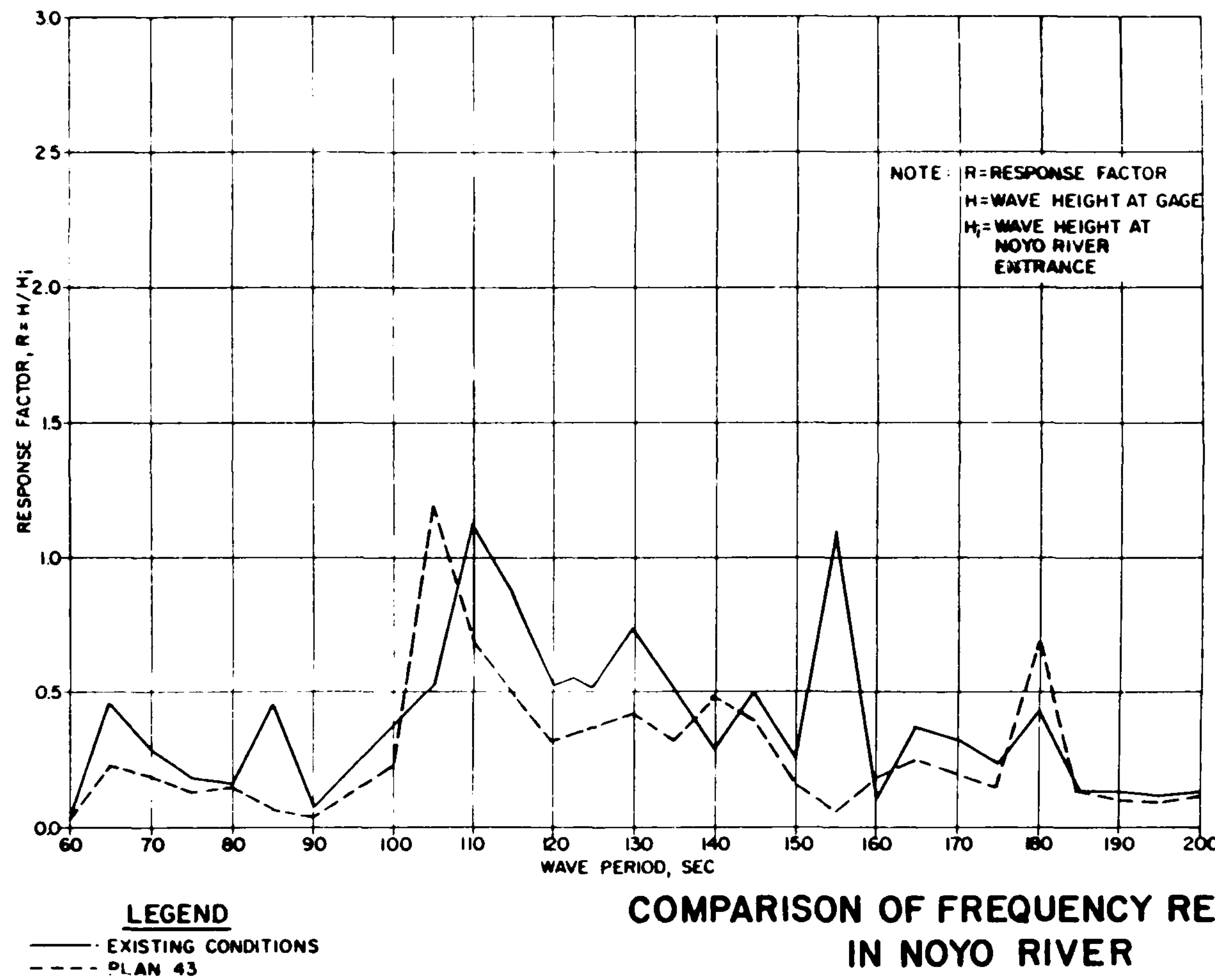
COMPARISON OF FREQUENCY RESPONSE
IN NOYO HARBOR
WAVE GAGE 16


COMPARISON OF FREQUENCY RESPONSE
IN NOYO HARBOR
WAVE GAGE 17
NOTE: R=RESPONSE FACTOR
H=WAVE HEIGHT AT GAGE
Hi=WAVE HEIGHT AT NOYO RIVER ENTRANCE
LEGEND
EXISTING CONDITIONS
PLAN 43
RESPONSE FACTOR, R=H/Hi
WAVE PERIOD, SEC
0.0
0.5
1.0
1.5
2.0
2.5
3.0
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200



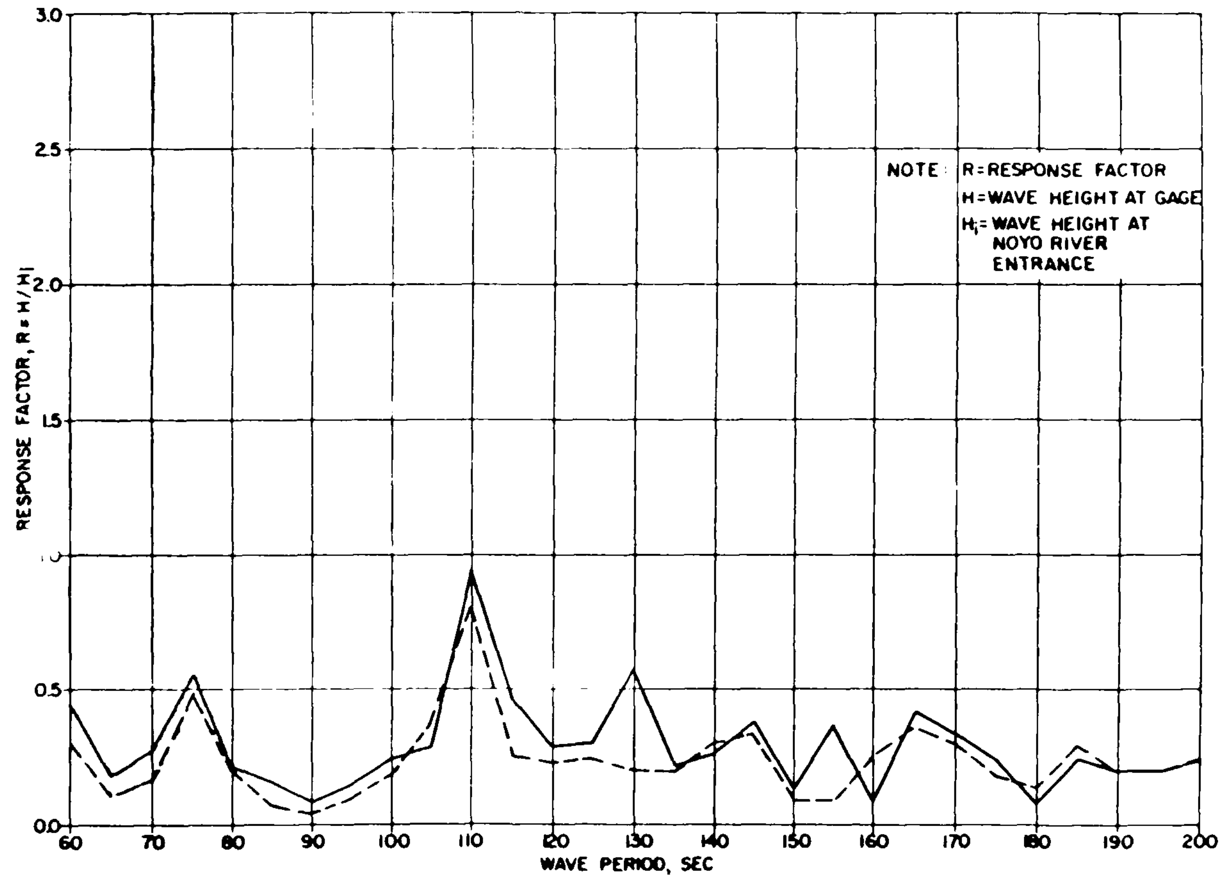
COMPARISON OF FREQUENCY RESPONSE IN NOYO HARBOR

WAVE GAGE 18



COMPARISON OF FREQUENCY RESPONSE IN NOYO RIVER

WAVE GAGE 19



LEGEND

—— EXISTING CONDITIONS

- - - PLAN 43

COMPARISON OF FREQUENCY RESPONSE IN DOLPHIN MARINA

WAVE GAGE 20

APPENDIX A: WAVE-TRANSMISSION TESTS FOR NOYO RIVER AND HARBOR BREAKWATER

1. The Noyo Harbor Wave Action model was constructed at a scale of 1:75 (model:prototype) based on Froude model laws (Stevens et al. 1942).* A dolos armored rubble-mound breakwater has been proposed to improve navigation conditions into and out of the harbor. The harbor model will be used to optimize the location, length, alignment, and overall geometry of the breakwater that is needed to create the desired wave conditions on the harbor side of the breakwater. Due to the small scale of the harbor model and the dependency of wave transmission characteristics on the Reynolds number, care must be taken to ensure that the 1:75 model breakwater reproduces the correct wave transmission characteristics.

2. Model tests described in this appendix were conducted at a scale (1:31) large enough to preclude transmission scale effects. Results of these tests were used to design a 1:75 scale breakwater to ensure that the proper wave transmission characteristics were reproduced in the smaller scale model.

3. The dolos armored, rubble-mound breakwater (Plate A-1) proposed for Noyo Harbor, was reproduced at undistorted linear scales of 1:31 and 1:75 based on Froude model laws (see paragraph 8 of the main text). The specific weight of the water used in the model was assumed to be 62.4 pcf and that of seawater is 64 pcf. In some instances the specific weight of the model construction material differed from that of the prototype. These variables were related using the following transference equation:

$$\frac{\left(W_a\right)_m}{\left(W_a\right)_p} = \frac{\left(\gamma_a\right)_m}{\left(\gamma_a\right)_p} \left[\frac{(L_m)}{(L_p)}\right]^3 \left[\frac{S_{a_p} - 1}{S_{a_m} - 1}\right]^3 \qquad \text{(A1)}$$

where

W_a = weight of individual armor unit or stone, lb

subscripts m,p = model and prototype quantities, respectively

γ_a = specific weight of individual armor unit or stone, pcf

L_m/L_p = linear scale of the model

S_a = specific gravity of individual armor unit or stone relative to the water in which the breakwater is constructed

γ_w = specific weight of water, pcf

* References cited in this appendix are included in the References at the end of main text.

Since the models were constructed using Froude model law and wave transmission is highly dependent upon viscous forces and hence dependent upon the Reynolds number, corrections had to be made in the sizes of various construction materials at both model scales. These corrections were made by the guidance provided by Keulegan (1973).

4. All of the two-dimensional wave transmission tests were conducted in a 150-ft-long, 1.5-ft-wide, and 3-ft-deep glass walled flume. The flume was equipped with a horizontal displacement wave generator capable of producing both monochromatic and spectral wave conditions.

5. The bathymetry seaward of the proposed breakwater toe is quite flat. The nature of the wave transmission tests did not require that the maximum depth-limited breaking wave be created at the toe of the structure. Therefore, it was decided to test the structure with a flat bottom seaward of the test section.

6. Prior to installation of the first breakwater section, the flume was calibrated for the selected wave periods and water depths. All tests were conducted with monochromatic waves. Test waves of the required characteristics were generated by varying the frequency and amplitude of the wave generator paddle. Changes in water-surface elevations as a function of time were measured by an electrically operated, parallel rod resistance wave gage. The gage was positioned in the flume at the point where the sea-side toe of the breakwater would be situated. Therefore, the flume was calibrated for the wave conditions that would reach that point in the flume and were not influenced by the presence of a breakwater structure.

7. Model breakwater sections were constructed to reproduce, as closely as possible, the results of usual methods of prototype construction. Core material was dumped by bucket or shovel, smoothed to grade, and compacted with hand trowels to simulate natural consolidation resulting from wave action during prototype construction. The underlayer stone was added and smoothed to grade but was not compacted. The berm was then constructed in the same manner as the underlayer. The structure was then covered from the sea-side berm to the harbor-side berm with two layers of dolos armor units. The dolos toes were constructed using special placement while the remainder of the dolosse were placed in a random manner, i.e. placed in such a way that no intentional interlocking of the armor units was obtained. Photo A1 shows a comparison between random and special placement of dolos toe units.

8. Based on prototype data, guidance from US Army Engineer District, Los Angeles, and wave height measurements made in the three-dimensional harbor wave action model, a still-water level (swl) of +7.0 ft mean lower low water (mllw), a -24.5 ft mllw sea-side toe elevation, and the following incident wave conditions were selected for use in the wave transmission tests conducted at the 1:31 and 1:75 scales:

Wave Period sec	Wave Height ft
9.0	5.0, 8.0, and 11.3
13.0	5.0, 10.0, and 13.7
17.0	5.0, 10.0, and 15.0

9. During the wave transmission tests at both model scales, a wave gage was positioned a distance shoreward of the breakwater center line that was equal to one-half wavelength, $L/2$, for a 13.0-sec wave in a 31.5-ft water depth D . The structure was exposed to the incident wave conditions described in the previous paragraph, and the data collected at the wave gage was analyzed to determine the average transmitted wave height. The breakwater sections were exposed to each incident wave condition for approximately 30-sec model time; the flume was allowed to still out; and the test was repeated two more times. Thus, the average transmitted wave height reported herein for each incident wave condition is the average of the average wave height measured for each of three tests. For all tests conducted, the average transmitted wave heights measured for repeated test conditions did not vary more than ±0.2ft (prototype).

10. The results of the tests conducted with the 1:31 and 1:75 scale breakwater sections (Plate A1 and Photos A2 and A3, respectively) are presented in Table A1. The wave transmission coefficient C_t is a nondimensional measure of transmitted wave height and was obtained by dividing the average transmitted wave height H_t by the incident wave height H_i measured without the structure in place, i.e. $C_t = H_t/H_i$. The wave transmission coefficient was plotted against incident wave height (Plate A2), incident wave steepness H_i/L (Plate A3), and relative depth D/L (Plate A4) to determine data trends, if any existed, and how they compared for the two model breakwater sections.

11. The transmitted heights measured for the 13-sec incident waves were almost identical for the two breakwater sections while the 1:75 scale

breakwater section had slightly higher and lower wave transmission for the 9- and 17-sec wave periods, respectively, than did the 1:31 scale breakwater section. Both breakwater sections showed the same trends of decreasing C_t with increasing incident wave height and wave steepness, but no trend was obvious with changes in relative depth.

12. The maximum breaking condition that could be produced and controlled at the 1:31 scale with the flat bottom flume was a 17.0-sec, 16.3-ft wave. Very limited stability tests were conducted using this incident wave condition to see if the proposed breakwater section showed any indication of stability problems. The structure was exposed to 1 hr (prototype time) of this incident wave condition, and the structure accrued no damage and exhibited only occasional minor rocking of two or three dolos units.

13. The underlayer and core material sizes of the 1:75-scale model breakwater section could have been changed to improve the comparison with either the 9.0- and 17.0-sec wave transmission characteristics exhibited by the 1:31-scale model. However, to match the transmission characteristics for all three wave periods would have required three different 1:75-model scale sections. Since the 1:75-model test section satisfactorily reproduced the transmission characteristics for the midrange wave periods, it was used for all the tests. The periods outside the midrange are not expected to significantly influence the breakwater configuration but some judgement should be used when looking at the transmitted wave heights measured in the 3-dimensional model for the longer and shorter wave periods.

14. Based on the tests and results reported herein, it appears that the 1:75-scale model breakwater section (Plate A2 and Photo A2) should adequately reproduce the wave transmission characteristics of its prototype counterpart.

15. Based on the very limited wave stability tests conducted, the 24,000-lb dolos proposed for the 1V on 2H slope of the breakwater trunk should be an adequate design for wave heights up to and including 16.3 ft. Additional stability tests are needed to check the dolos stability when exposed to the larger depth-limited breaking waves that will occur when the correct bathymetry is represented seaward of the breakwater toe.

Table A1

Wave Transmission Data for Noyo Breakwater, Plan A

swl = +7.0 ft, mllw					
Incident Wave			Water Depth, $D = 31.5$ ft		
Period T, sec	Height H_i, ft	Relative Depth D/L	Wave Steepness H_i/L	Average Transmitted* Wave Height H_t, ft	Wave Transmission Coefficient $C_t = H_t/H_i$
			Model Scale 1:31		
9.0	5.0	0.12	0.019	0.77	0.15
9.0	8.0	0.12	0.030	0.93	0.12
9.0	11.3	0.12	0.043	1.14	0.10
13.0	5.0	0.08	0.013	1.13	0.23
13.0	10.0	0.08	0.025	1.47	0.15
13.0	13.7	0.08	0.034	1.94	0.14
17.0	5.0	0.06	0.009	1.17	0.23
17.0	10.0	0.06	0.019	1.60	0.16
17.0	15.0	0.06	0.028	1.95	0.13
			Model Scale 1:75		
9.0	5.0	0.12	0.019	0.81	0.16
9.0	8.0	0.12	0.030	1.06	0.12
9.0	11.3	0.12	0.043	1.26	0.11
13.0	5.0	0.08	0.013	1.15	0.23
13.0	10.0	0.08	0.025	1.47	0.14
13.0	13.7	0.08	0.034	1.78	0.13
17.0	5.0	0.06	0.009	1.00	0.20
17.0	10.0	0.06	0.019	1.30	0.13
17.0	15.0	0.06	0.028	1.66	0.11

* Average of three tests conducted for each incident wave condition.

Photo A1. Comparison of random and special dolos toe construction.

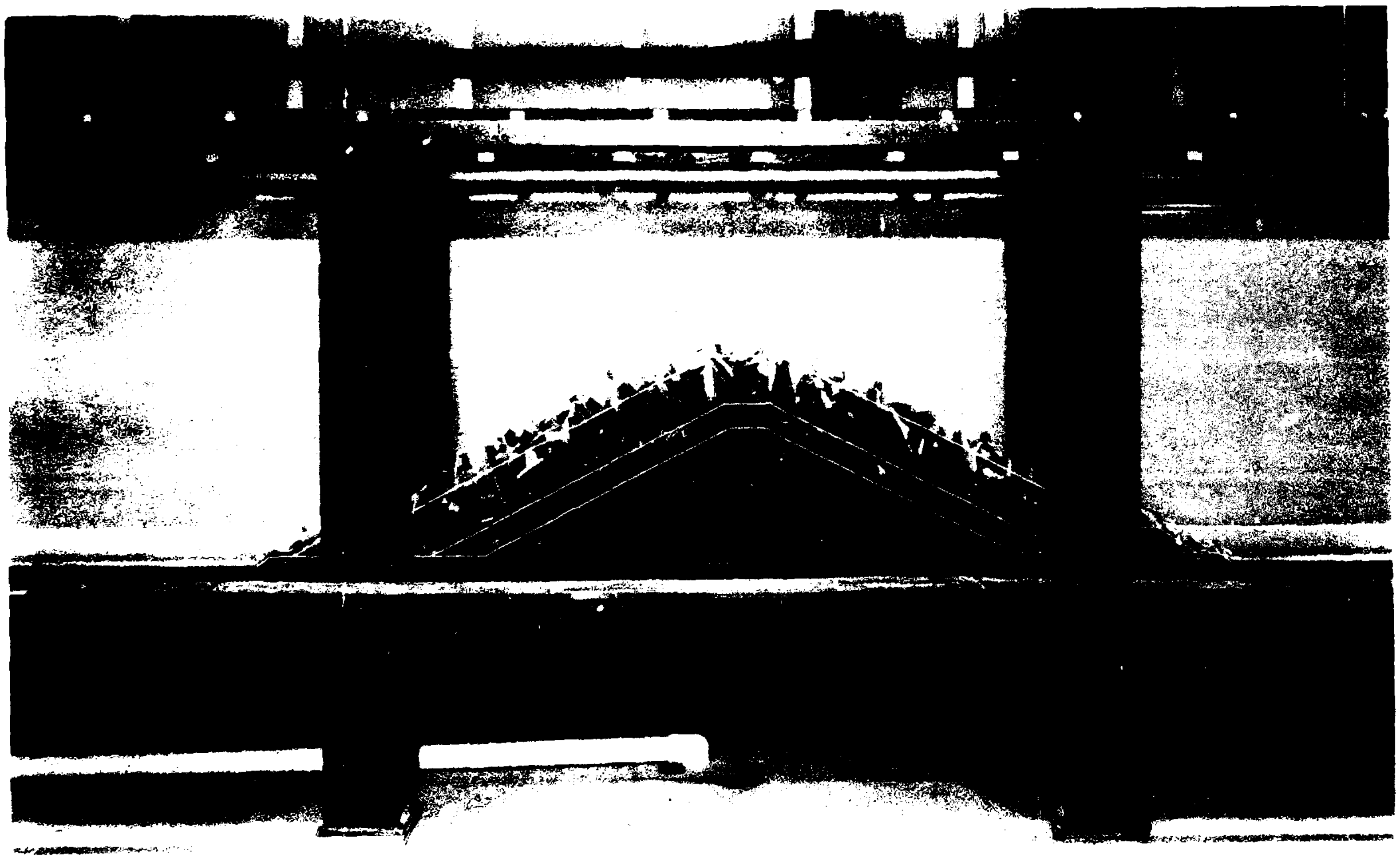


Photo A2. Side view of the 1:31 scale dolos breakwater section, Plan A

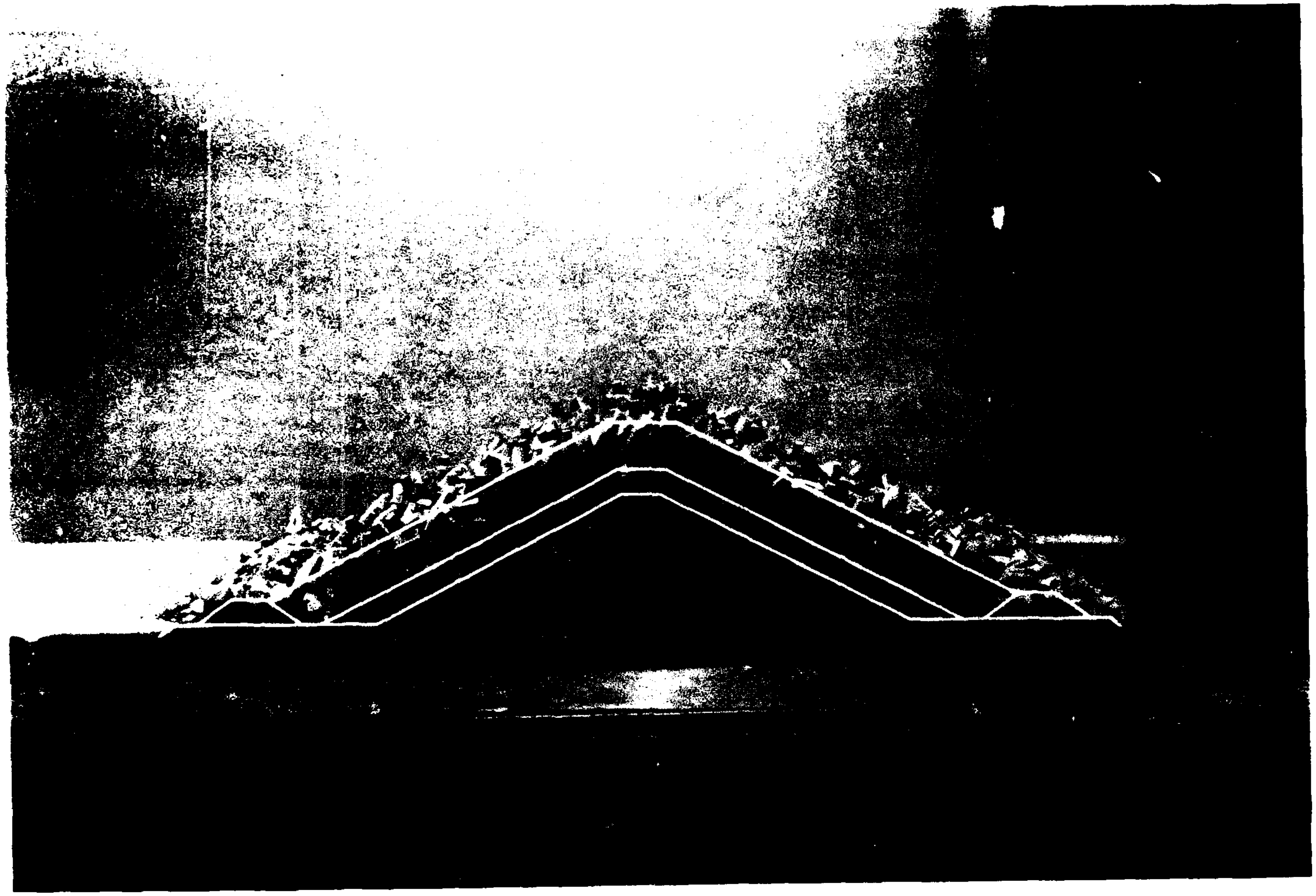
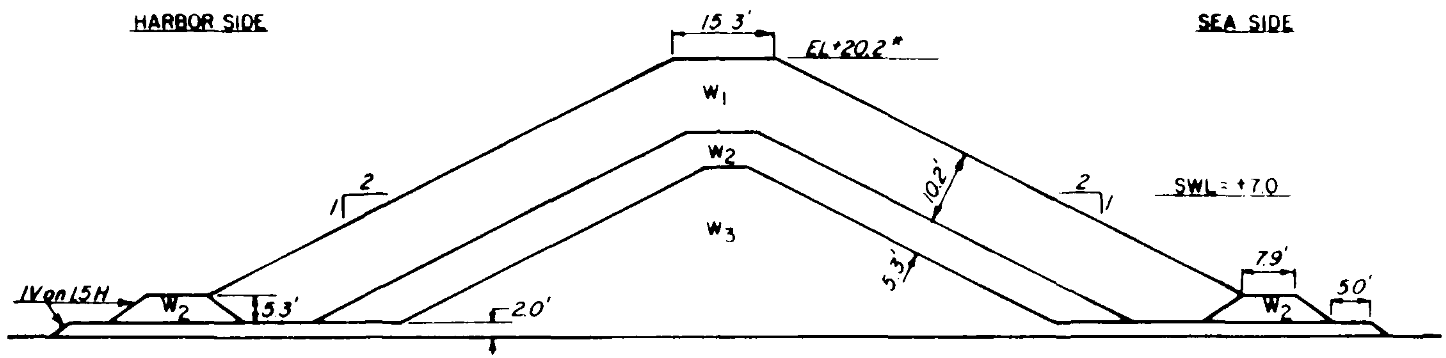


Photo A3. Side view of the 1:75 scale dolos breakwater section, Plan A



MATERIAL CHARACTERISTICS

PROTOTYPE

** W_1 = 24,000 LB DOLOSSE @ 150 PCF
W_2 = 1,200 TO 4,800 LB STONE @ 165 PCF
W_3 = 6 TO 120 LB STONE @ 165 PCF

MODEL SCALE 1:31

W_1 = 0.882 LB DOLOSSE @ 142.7 PCF
W_2 = 0.036 TO 0.142 LB STONE @ 165 PCF
† W_3 = 0.049 TO 0.072 LB STONE @ 165 PCF

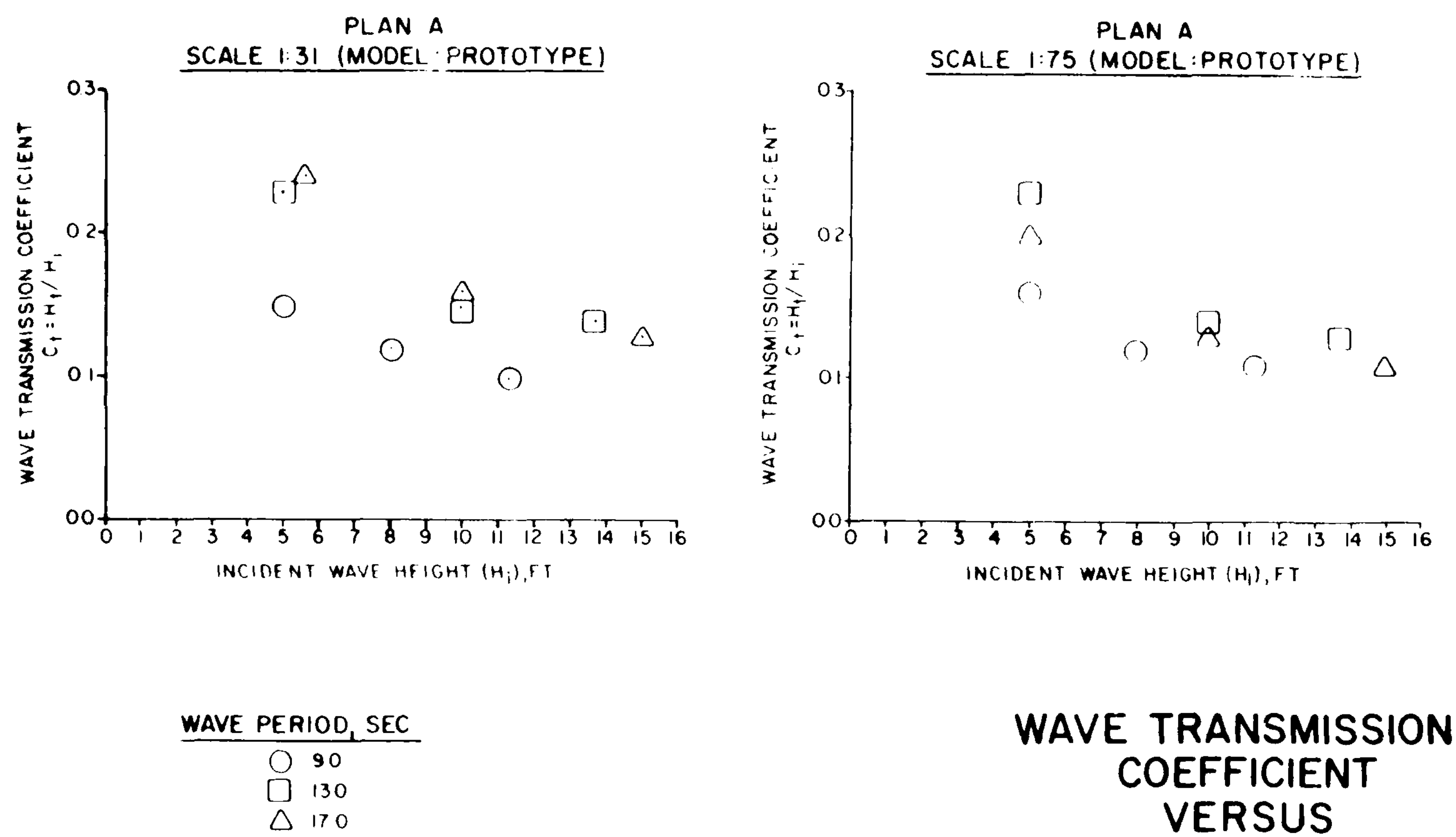
† MODEL SCALE 1:75

W_1 = 0.077 LB DOLOSSE @ 146.8 PCF
W_2 = 0.009 TO 0.031 LB STONE @ 165 PCF
W_3 = 0.004 TO 0.009 LB STONE @ 165 PCF

NOTES

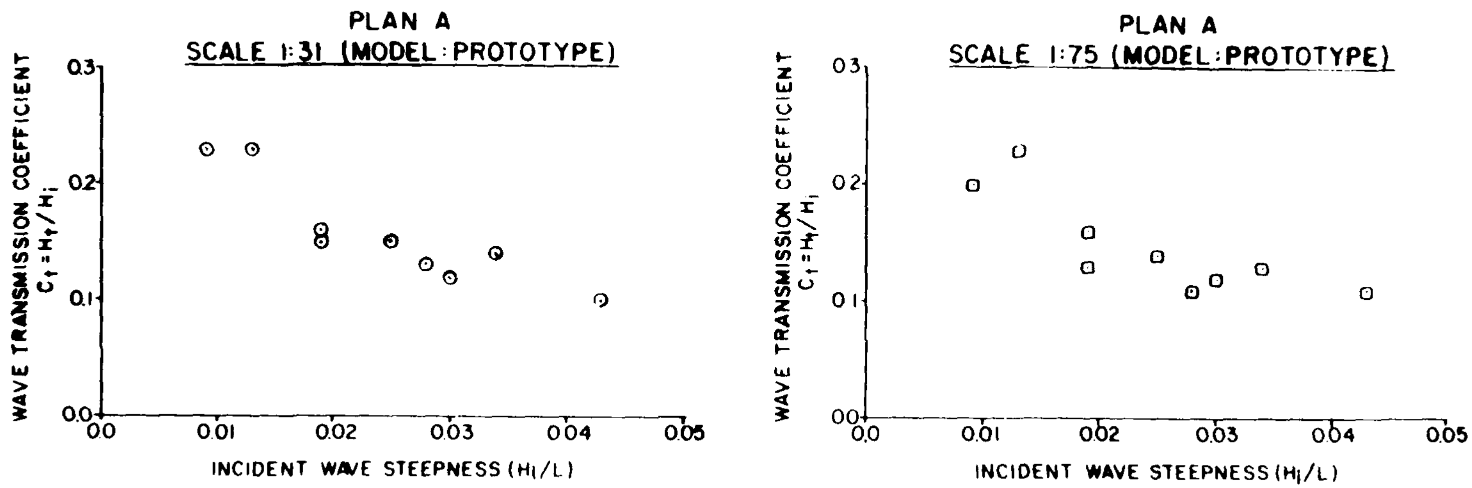
* ELEVATIONS IN FEET REFERRED TO MLLW
** TWO LAYERS RANDOM PLACED EXCEPT FOR SPECIAL PLACED TOE DOLOSSE
† OVERSIZED IN ORDER TO REPRODUCE CORRECT TRANSMISSION CHARACTERISTICS

NOYO BREAKWATER PLAN A

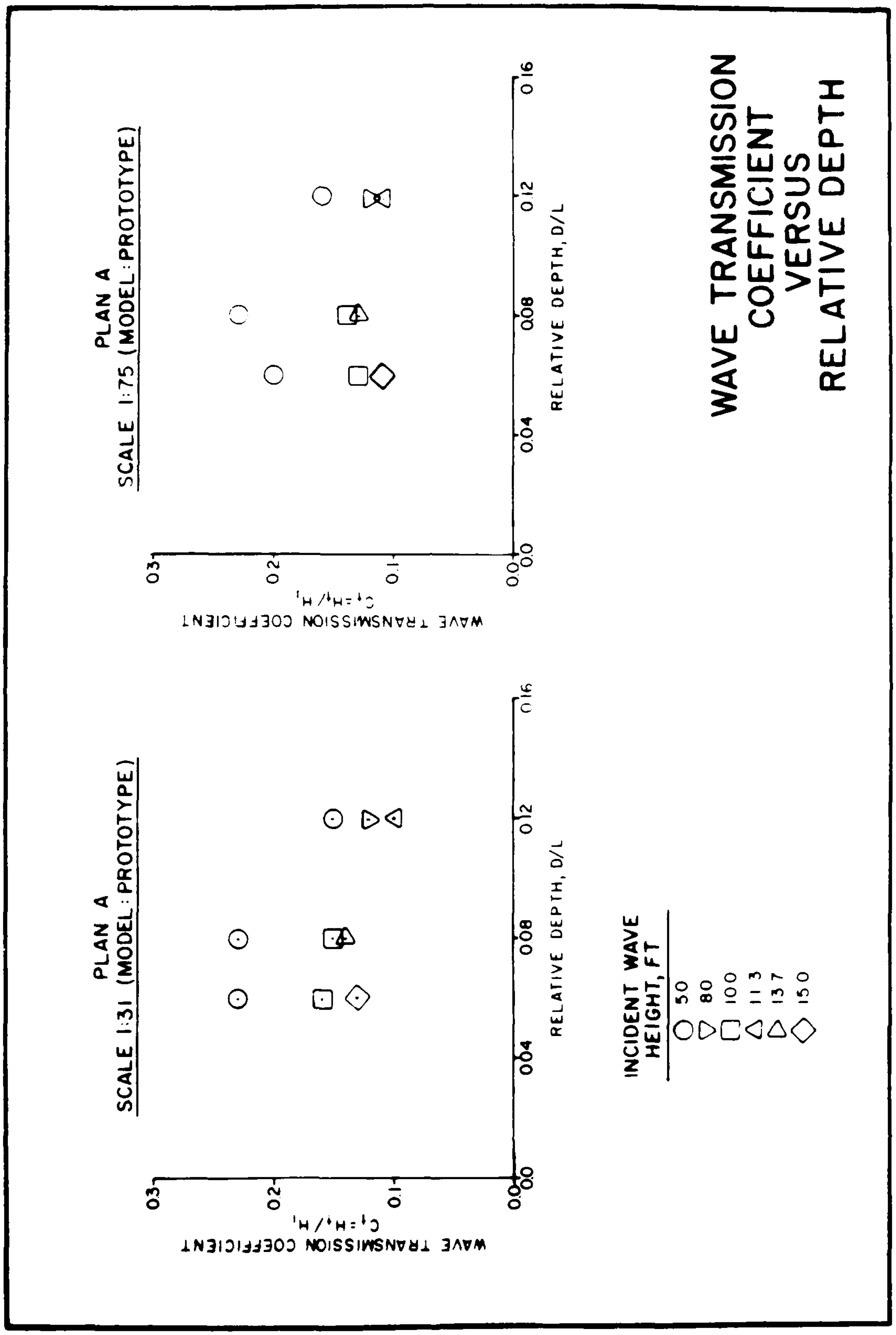


WAVE TRANSMISSION COEFFICIENT VERSUS INCIDENT WAVE HEIGHT

PLATE A2



WAVE TRANSMISSION COEFFICIENT VERSUS INCIDENT WAVE STEEPNESS



PLAN A
SCALE 1:31 (MODEL:PROTOTYPE)
PLAN A
SCALE 1:75 (MODEL:PROTOTYPE)
WAVE TRANSMISSION COEFFICIENT
$C_t = H_t/H_i$
0.0
0.1
0.2
0.3
RELATIVE DEPTH, D/L
0.04
0.08
0.12
0.16
INCIDENT WAVE HEIGHT, FT
50
80
100
113
137
150
WAVE TRANSMISSION COEFFICIENT VERSUS RELATIVE DEPTH

PLATE A4

APPENDIX B: NOTATION

A	Area
C_t	Wave transmission coefficient
D	Water depth
H	Wave height
H_i	Incident wave height
H_a	Wave height at antinode
H_n	Wave height at node
H_t	Average transmitted wave height
$H_{1/3}$	Significant wave height
K_r	Refraction coefficient
K_s	Shoaling coefficient
L	Length, wave length
L_m/L_p	Linear scale of the model
m	Model construction material quantity
N_D	Sediment size ration
N_γ	Relative specific weight ratio
p	Prototype construction material quantity
R	Frequency response factor
S_a	Specific gravity of individual armor or stone relative to the water in which the breakwater is constructed
T	Time, wave period
V	Velocity
$\forall$	Volume
W_a	Weight of individual armor unit or stone, lb
θ	Direction of wave approach
γ_a	Specific weight of individual armor unit or stone, pcf
γ_w	Specific weight of water, pcf
μ	Vertical scale
λ	Horizontal scale