Technical Report CERC-94-8
May 1994

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of Engineers
Waterways Experiment

Wave Conditions for Pier 400 Dredging and Landfill Project, Los Angeles Outer Harbor, Los Angeles, California

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

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Prepared for Port of Los Angeles, Harbor Department

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Waterways Experiment Station
3909 Halls Ferry Road
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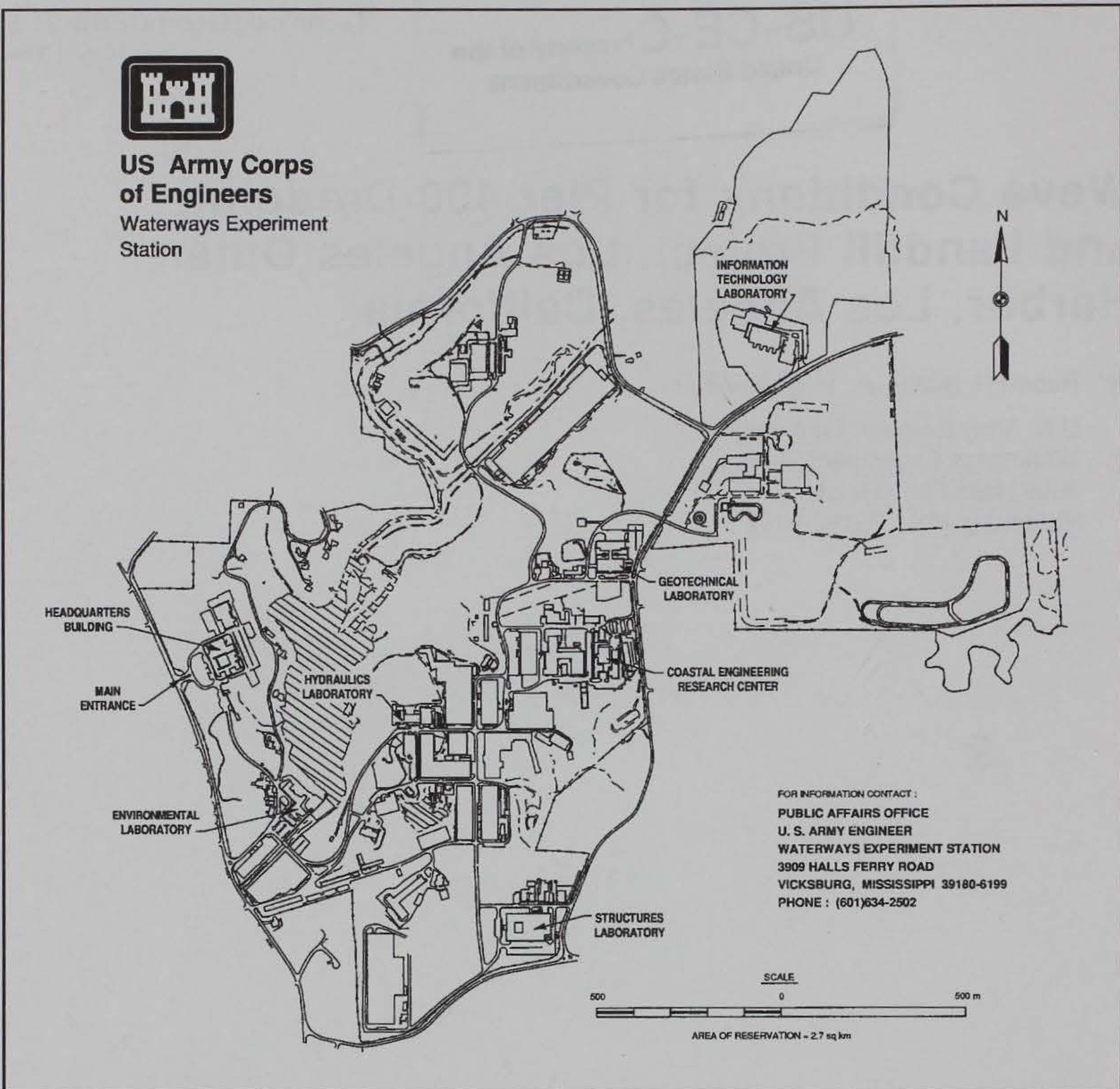
San Pedro, CA 90733-0151



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Preface

This report presents the results of a physical model study, with respect to short-period storm wave conditions, for the Pier 400 Dredging and Landfill Project in Los Angeles Outer Harbor. The work was authorized by a cooperative agreement between the U.S. Army Engineer Waterways Experiment Station (WES), and the City of Los Angeles, California, after approval was granted by the U.S. Army Corps of Engineers Directorate of Research and Development. Funds were provided by the City of Los Angeles on 1 February, 15 March, and 14 June 1993.

Model testing was conducted at WES during the period April-August 1993 by personnel of the Coastal Engineering Research Center (CERC) under the direction of Dr. James R. Houston and Mr. Charles C. Calhoun, Jr., Director and Assistant Director, CERC, respectively; and under the direct supervision of Messrs. C. E. Chatham, Jr., Chief, Wave Dynamics Division, and Dennis G. Markle, Chief, Wave Processes Branch. The tests were conducted by Messrs. Hugh F. Acuff, Civil Engineering Technician, and William G. Henderson, Computer Technician, under the supervision of Mr. Robert R. Bottin, Jr., Project Manager. This report was prepared by Messrs. Bottin and Acuff.

During the course of the investigation, liaison was maintained by means of conferences, telephone conversations, and monthly progress reports. The following personnel visited WES to participate in conferences and/or observe model operation during the course of the study:

• Mr. John Warwar	Port of Los Angeles
• Mr. Dick Wittkop	Port of Los Angeles
• Mr. John Foxworthy	Port of Los Angeles
• Mr. Ron Reddick	Port of Los Angeles
• Mr. Doug Thiessen	Port of Los Angeles
• Mr. Jamie Merino	U.S. Army Engineer Division, South Pacific
• Mr. Ark Shak	U.S. Army Engineer District, Los Angeles
• Ms. Jane Grandon	U.S. Army Engineer District, Los Angeles
• Mr. Robert Michaels	U.S. Army Engineer District, Los Angeles
• Mr. Don Spencer	U.S. Army Engineer District, Los Angeles
• Mr. Brian Moore	U.S. Army Engineer District, Los Angeles
• Mr. Thomas Leung	U.S. Army Engineer District, Los Angeles

- Dr. Kimo Walker — Moffatt and Nichol, Engineers - Pier 400 Design Consultants
- Mr. Russ Boudreau — Moffatt and Nichol, Engineers - Pier 400 Design Consultants
- Mr. Jal Birdy — Moffatt and Nichol, Engineers - Pier 400 Design Consultants
- Mr. Henry Steinorth — Fredrick R. Harris - Pier 400 Design Consultants
- Mr. Ed Schmeltz — Frederick R. Harris - Pier 400 Design Consultants
- Mr. Mark Savore — Kinnetic Labs - Pier 400 Design Consultants
- Mr. Julius Kerenyi — American President Lines
- Mr. Saunders Jones — American President Lines
- Mr. Bob Johansen — American President Lines

Dr. Robert W. Whalin was Director of WES during model testing and the preparation and publication of this report. COL Leonard G. Hassell, EN, and COL Bruce K. Howard, EN, were Commanders.

Conversion Factors, Non-SI to SI Units of Measurement

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

Multiply	By	To Obtain
acres	4046.873	square meters
degrees (angle)	0.01745329	radians
feet	0.3048	meters
miles (U.S. statute)	1.609347	kilometers
square feet	0.09290304	square meters
square miles (U.S. statute)	2.589998	square kilometers

1 Introduction

Background

The ports of Los Angeles and Long Beach are located in San Pedro Bay along the southern coast of California (Figure 1). Historically, they have experienced long-period surge activity, which occasionally results in mooring difficulties for ships berthed in various locations within the harbors complex. In coordination with the U.S. Army Corps of Engineers, the Ports of Los Angeles and Long Beach are conducting studies for harbor development and expansion to accommodate future needs. Descriptions of the existing breakwaters may be found in Bottin (1988).

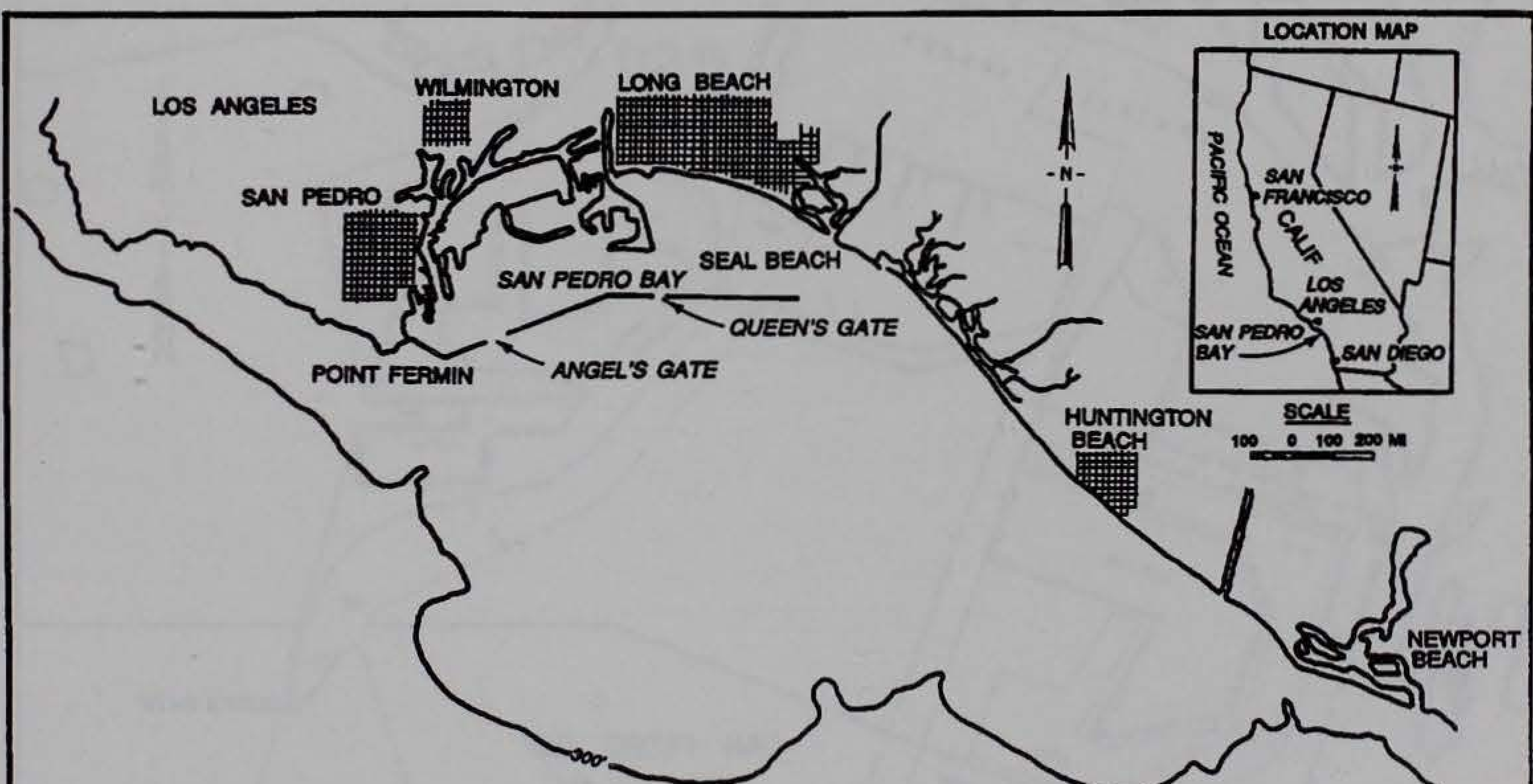

Figure 1. Project location

A distorted model (scale, 1:400 horizontal, 1:100 vertical) of the Los Angeles-Long Beach Harbors complex was designed and constructed at the U.S. Army Engineer Waterways Experiment Station (WES) in the early 1970's and has been used since that time to determine the effects of long-period waves (30 to 400 sec), which lead to resonant harbor oscillations that can cause ship loading-unloading problems and downtime. The model distortion and scales, however, make the model inappropriate for short-period (4- to 25-sec) wind wave testing.

Model Study Objectives

At the request of the Port of Los Angeles an existing undistorted hydraulic model, which includes a portion of the Los Angeles Outer Harbor (Figure 2), was reactivated by the WES Coastal Engineering Research Center to:

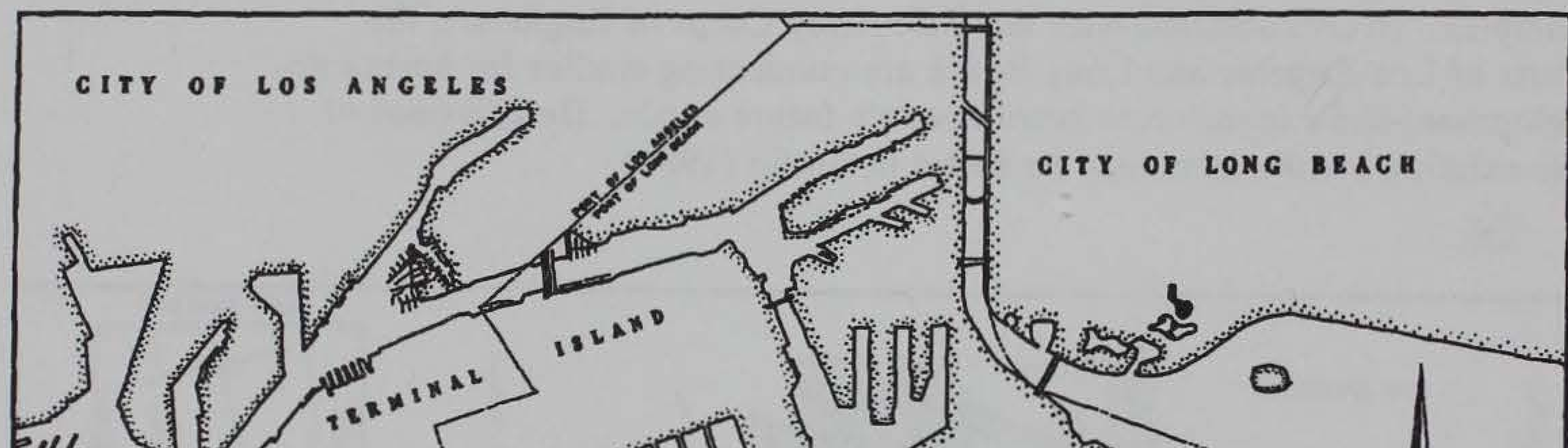

Figure 2. Approximate limits of proposed model relative to harbor

a. Determine design wave conditions along the seaward perimeter of the proposed Pier 400 landfill.

b. Determine short-period wave conditions at various locations in the Los Angeles Outer Harbor for two stages of construction of the Pier 400 dredging and landfill project.

Previously Reported Model Test Results

The original purpose of the undistorted Los Angeles Outer Harbor model was to investigate short-period wave conditions for slightly different proposed harbor development located near the Angel's Gate entrance (Bottin and Tolliver 1989). Additional testing was conducted to determine wave conditions for the optimum plan for protection of the southern container slip from locally generated wind waves from within the harbors complex (Bottin and Acuff 1991). Another series of tests was then conducted to determine wave conditions and the optimum plan for protection for various berthing areas during the two construction phases of the proposed harbor expansion (Bottin and Acuff 1992).

2 The Model

Design of Model

The Los Angeles Outer Harbor Model (Figure 3) was constructed to an undistorted linear scale of 1:100, 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 short-period wave patterns including the effects of wave refraction, diffraction, and reflection. 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:

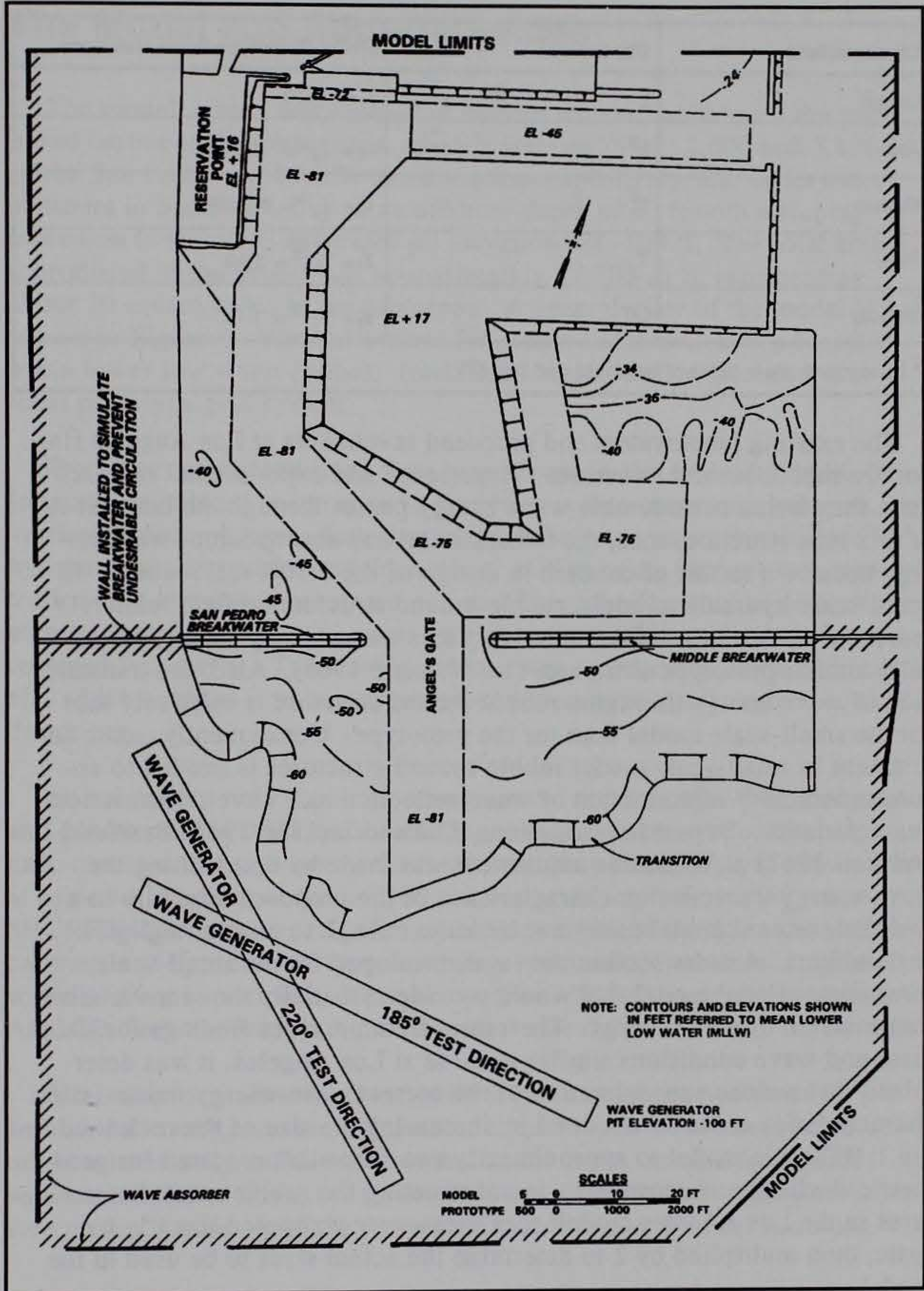

Figure 3. Model layout

Characteristic	Dimension[1]	Model-Prototype Scale Relations
Length	L	$L_r = 1:100$
Area	L^2	$A_r = L_r^2 = 10,000$
Volume	L^3	$V_r = L_r^3 = 100,000$
Time	T	$T_r = L_r^{1/2} = 1:10$
Velocity	L/T	$V_r = L_r^{1/2} = 1:10$

[1] Dimensions are in terms of length (L) and time (T).

The existing breakwaters and proposed revetments at Los Angeles Harbor 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 1:100-scale model. In small-scale hydraulic models, rubble-mound structures reflect relatively more and absorb or dissipate relatively less wave energy than geometrically similar prototype structures (Le Méhauté 1965). Also, the transmission of wave energy through a rubble-mound structure is relatively less for the small-scale model than for the prototype. Consequently, some adjustment in small-scale model rubble-mound structures is needed to ensure satisfactory reproduction of wave-reflection and wave-transmission characteristics. In past investigations (Dai and Jackson 1966, Brasfeild and Ball 1967) at WES, this adjustment was made by determining the wave-energy transmission characteristics of the proposed structure in a two-dimensional model using a scale large enough to ensure negligible scale effects. A cross section then was developed for the small-scale, three-dimensional model that would provide essentially the same relative transmission of wave energy. Therefore, from previous findings for structures and wave conditions similar to those at Los Angeles, it was determined that a close approximation of the correct wave-energy transmission characteristics could be obtained by increasing the size of the rock used in the 1:100-scale model to approximately two times that required for geometric similarity. Accordingly, in constructing the rubble-mound structures in the Los Angeles model, rock sizes were computed linearly by scale, then multiplied by 2 to determine the actual sizes to be used in the model.

The Model and Appurtenances

The model, which was molded in cement mortar, reproduced the proposed harbor expansion stages, Angel's Gate entrance, 2,800 and 5,100 ft[1] of the San Pedro and Middle Breakwaters, respectively, and underwater contours in San Pedro Bay to an offshore depth of 60 ft with a sloping transition to the wave generator pit elevation[2] of -100 ft. The total area reproduced in the model was approximately 27,500 sq ft, representing about 10 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.

Prototype wave conditions were reproduced in the model by an 80-ft-long, unidirectional spectral wave generator with a trapezoidal-shaped, vertical motion plunger. The electrohydraulic wave generator utilized a hydraulic power supply, and the vertical motion of its plunger was controlled by a computer-generated command signal. The controlled movement of the plunger caused water displacements, which reproduced the required test waves. The wave generator was mounted on retractable casters, which enabled it to be positioned to generate waves from the required directions.

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

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

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1 A table of factors for converting non-SI units of measurement to SI units is presented on page vii.

2 All elevations cited herein are in feet referred to mean lower low water (mllw) unless otherwise noted.

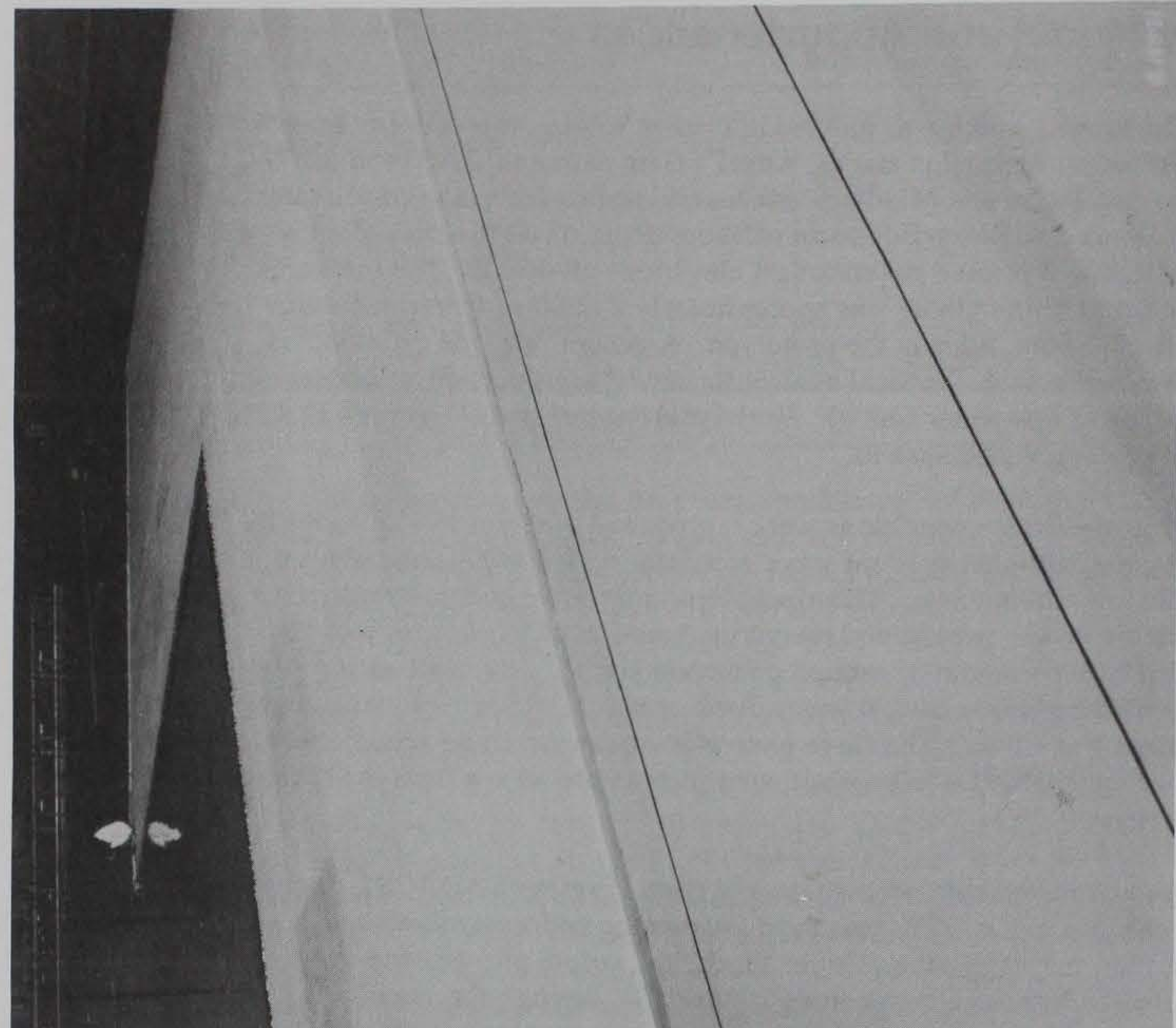

Figure 4. General view of model

3 Test Conditions and Procedures

Selection of Test Conditions

Still-water level

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

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

a. The maximum amount of wave energy reaching a coastal area normally occurs during the higher water phase of the local tidal cycle.

b. Most storms moving onshore are characteristically accompanied by a higher water level due to wind 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.

Swl's of +5.5 and +8.0 ft were selected by the Port of Los Angeles for use during model testing. This lower value (+5.5) represents mean higher high water in Los Angeles Outer Harbor and was used while testing operational wave conditions. The +8.0-ft swl represents an extreme high tide that contains storm surge and high astronomical tide. It was used while testing extreme wave conditions for design purposes.

Factors influencing selection of test wave characteristics

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 distance over the water (fetch) that 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 and island sheltering

When wind waves move into water of gradually decreasing depth, transformations take place in all wave characteristics except wave period (to the first order of approximation). The most important transformations with respect to the selection of test wave characteristics are the changes in wave height and direction of travel due to the phenomenon referred to as wave refraction. The change in wave height and direction may be determined by conducting a wave-refraction analysis. The shoaling coefficient, a function of wave length and water depth, can be obtained from the *Shore Protection Manual* (1984). When the refraction coefficient is determined, it is multiplied by the shoaling coefficient and gives a conversion factor for transfer of deepwater wave heights to shallow-water values.

As deepwater waves approach Los Angeles Harbor from the west counterclockwise through the south, wave propagation is inhibited due to offshore islands, which partially shelter the harbor.

The wave climate approaching the project site was analyzed by Port of Los Angeles consultants considering wave refraction, shoaling, and island sheltering. Approaching waves were grouped into two directions at the -60-ft contour. Extratropical storms are represented by a 220-deg azimuth (210- to 230-deg band), and tropical cyclones and pre-frontal seas are represented by a 185-deg azimuth (170- to 190-deg band). Operational waves are expected from either of these two directions.

Selection of test waves

Based on wave analysis by consultants of the Port of Los Angeles, the test wave characteristics shown in the tabulation below were selected for use in the model investigation. Unidirectional wave spectra based on Joint North Sea Wave Project (JONSWAP) parameters for the test waves listed were reproduced for tests throughout the investigation. Note the JONSWAP gamma "γ" parameters included in the tabulation. These values relate to the energy distribution in the spectral curves. Larger gamma values produce sharper peaks in the spectral energy distribution curve.

Analysis of Model Data

The conditions tested were evaluated by a comparison of wave heights at selected locations in the model, visual observations, and wave pattern photographs. In the wave-height analysis, the average height of the highest one third of the waves (H_s) recorded at each gage location was computed. All wave heights then were adjusted to compensate for excessive wave-height attenuation due to viscous model bottom friction, by application of Keulegan's equation (Keulegan 1950).[1] 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.

1 G. H. Keulegan. (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, WES, Vicksburg, MS, by letter of 2 May 1950.

Selected Test Waves[1]

Direction, deg	Period, sec	Height, ft	γ	Condition
swl = +5.5 ft				
185	8	4, 6, 8	3.3	operational sea
185	12	4, 6, 8	5.0	operational sea
185	16	4, 6, 8	7.0	operational swell
220	8	4, 8, 12	3.3	operational sea
220	12	4, 8, 12	5.0	operational sea
220	16	4, 8, 12	7.0	operational swell
swl = +8.0 ft				
185	8	10	3.3	25 yr sea
185	12	12	3.3	10 yr TS[2]
185	12	12	5.0	50 yr sea
185	14	14	5.0	25 yr TS
185	14	16	3.3	100 yr sea
185	16	18	3.3	200 yr sea
185	16	18	5.0	100 yr TS
185	16	20	5.0	200 yr TS
220	14	14	7.0	25 yr ETS[3]
220	16	16	7.0	50 yr ETS
220	18	18	7.0	100 yr ETS
220	20	20	7.0	200 yr ETS

[1] All selected waves were defined at approximately the -60 ft contour.
[2] TS - tropical storm
[3] ETS - extratropical storm

4 Tests and Results

Tests

Tests were conducted for two stages of construction for the Port of Los Angeles (POLA) Outer Harbor expansion plan. Brief descriptions of the construction stages are presented below, with dimensional details shown in Plates 1 and 2.

a. POLA Stage 1 (Plate 1) consisted of a 63-ft-deep, approximately 1,200-ft-wide dredged channel extending through Angel's Gate and northerly along Reservation Point with a turning basin south of Fish Harbor. From the turning basin, a 45-ft-deep channel extended easterly adjacent to Pier 300. A landfill (Pier 400) and causeway were constructed north of Angel's Gate entrance with the dredged material, which provided wave protection to the inner berthing areas formed adjacent to Pier 300. The landfill totaled approximately 225 acres.

b. POLA Stage 2 (Plate 2) consisted of an 81-ft-deep, approximately 1,200-ft-wide dredged channel extending through Angel's Gate and northerly along Reservation Point with a turning basin south of Fish Harbor. From the turning basin, a 45-ft-deep channel extended easterly adjacent to Pier 300. An additional turning basin and slip dredged to a -75-ft depth was included south and east of the Pier 400 landfill inside Angel's Gate. The area of the landfill (Pier 400) was increased to approximately 580 acres with the dredged material.

Prior to collecting data for POLA Stages 1 and 2, design wave information was obtained with the -63- and -81-ft channel configurations without the landfill in place. These configurations are shown in Plates 3 and 4. The purpose of these tests was to obtain design wave heights along the toe of the proposed Pier 400 landfill for design of the revetment. Had the landfill been installed, wave measurements would have been contaminated due to reflections from the landfill.

Wave height tests were conducted for both POLA Stages 1 and 2 and the -63- and -81-ft channel configurations with no landfills installed for the test waves shown in Chapter 3. Wave gage locations are shown in Plates 1-4. Wave pattern photographs were secured for representative test waves for the various configurations to provide documentation of test results. In addition, videotape footage was secured for POLA Stage 2 and furnished to the Port of Los Angeles for use in briefings, public meetings, etc.

Test Results

In evaluating test results, the relative merits of the proposed configurations tested were based on an analysis of measured wave heights in selected locations in the harbor. Model wave heights (significant wave height or $H_{1/3}$) were tabulated to show measured values at the selected locations. The -81-ft channel configuration (no landfill), followed by the -63-ft channel configuration (no landfill), were tested initially. Then, POLA Stages 1 and 2 were tested. The tests were conducted in this sequence to initially provide design wave data for the Pier 400 revetment and to minimize construction costs.

Dredged channels without landfills

Results of wave height tests conducted with the -81-ft channel configuration with no landfill are presented in Table 1. For operational wave conditions with the +5.5-ft swl, maximum wave heights were 12.3 ft along the toe of the proposed Pier 400 revetment (gage 5) for 8-sec, 12-ft test waves from 220 deg; and 4.6 ft adjacent to Pier 300 (gage 12) for 8-sec, 8-ft test waves from 185 deg. For extreme wave conditions with the +8.0-ft swl, maximum wave heights were 22.3 ft at the toe of the proposed Pier 400 revetment (gage 6) and 10.4 ft adjacent to Pier 300 (gage 11), both for 16-sec, 20-ft test waves from 185 deg. Wave pattern photographs were obtained from overhead to determine the angle of wave approach at the toe of the proposed landfill for the various test wave conditions. Examples of wave patterns obtained at the toe of the proposed landfill with the -81-ft channel configuration are shown in Photo 1.

To determine the sensitivity of wave heights along the proposed Pier 400 landfill for direction of wave approach, additional tests were conducted for extreme test conditions from 180 and 190 deg with the -81-ft channel configuration installed. Additional gages also were installed to increase gage density. Results of these tests are shown in Table 2. Maximum wave heights at the toe of the proposed landfill were 20.6 ft (gage 6) for 16-sec, 20-ft test waves from 180 deg; and 20.5 ft (gage 5A) for 16-sec, 20-ft test waves from 190 deg.

Results of wave height tests conducted with the -63-ft channel configuration with no landfill are presented in Table 3. For operational wave conditions with the +5.5-ft swl, maximum wave heights were 12.3 ft along the toe of the proposed Pier 400 revetment (gage 5) for 8-sec, 12-ft test waves from 220 deg; and 4.4 ft adjacent to Pier 300 (gage 12) for 12-sec, 12-ft test waves from 220 deg. For extreme wave conditions with the +8.0-ft swl, maximum wave heights were 21.8 ft at the toe of the proposed revetment (gage 5) for 16-sec, 18-ft test waves from 185 deg; and 10.0 ft adjacent to Pier 300 (gage 12) for 16-sec, 20-ft test waves from 185 deg.

Analysis of test results obtained for the -81- and -63-ft channel configurations (without the proposed landfill) reveals that the -63-ft configuration bathymetry tended to focus wave energy slightly more to the east inside the harbor. Referring to the wave height data in Tables 1 and 3, and the gage locations in Plates 3 and 4, it is noted that maximum wave heights occur at gage 6 for the -81-ft configuration and at gage 5 for the -63-ft configuration for the 185-deg test directions. Also, wave heights at gage 4 are significantly higher for the -63-ft configuration for the 185-deg direction, which indicates focusing of more energy to the east. Gage 5 is located on a corner of the landfill and gage 6 is on a straight section. Designers of the revetment armor units should take these results into consideration since the armor on the corner would generally be less stable than that on the straight section, considering the angle of wave attack at Pier 400. In summary, the revetment armor units at the southeast corner of Pier 400 will be more susceptible to damage during the interim POLA Stage 1 phase of construction (-63-ft configuration) than after the final POLA Stage 2 phase (-81-ft configuration) has been completed.

POLA Stages 1 and 2

Results of wave height tests with POLA Stage 1 installed are presented in Table 4. For operational wave conditions with the +5.5-ft swl, maximum wave heights were 0.4 ft adjacent to Pier 300 (gage 12) for 16-sec, 12-ft test waves from 220 deg and 1.0 ft in the channel west of Pier 400 (gage 9) for 16-sec, 8-ft test waves from 185 deg. For extreme wave conditions with the +8.0-ft swl, maximum wave heights were 1.4 ft adjacent to Pier 300 (gage 12) and 3.7 ft in the channel west of Pier 400 (gage 9) for 16-sec, 20-ft test waves from 185 deg. Representative wave patterns for POLA Stage 1 at both Piers 300 and 400 are shown in Photos 2-16. Visual observations during the conduct of tests revealed overtopping of the landfill in an area approximately 1,500 ft west of the causeway for some of the extreme wave conditions from 185 deg.

Wave height test results for POLA Stage 2 are presented in Table 5. For operational waves with the +5.5-ft swl, maximum wave heights were 0.3 ft adjacent to Pier 300 (gage 13) for 16-sec, 6- and 8-ft test waves from 185 deg and 16-sec, 12-ft test waves from 220 deg; 1.1 ft in the channel west of Pier 400 (gage 9) for 16-sec, 12-ft test waves from 220 deg;

and 2.4 ft in the dredged berth east of Pier 400 (gage 6) for 8-sec, 12-ft and 12-sec, 12-ft test waves from 220 deg. For extreme wave conditions with the +8.0-ft swl, maximum wave heights were 1.3 ft adjacent to Pier 300 (gages 13 and 15); 3.1 ft in the channel west of Pier 400 (gage 9); and 3.9 ft in the dredged berth east of Pier 400 (gage 6) all for 16-sec, 20-ft test waves from 185 deg. Typical wave patterns at both Piers 300 and 400 for POLA Stage 2 are shown in Photos 17-31. Overtopping of the landfill approximately 2,000 ft west of the causeway (at approximately the gage 4 location) was observed for some of the extreme wave conditions from 185 deg.

5 Conclusions

Based on results of the Los Angeles Outer Harbor coastal hydraulic model investigation, conclusions relative to the dredged channel configurations (without landfills) and POLA Stages 1 and 2 landfill configurations are as follows:

Dredged Channel Configurations with No Landfills

a. Both the -63- and -81-ft channel configurations resulted in large wave heights at the toe of the proposed Pier 400 landfill. Maximum wave heights of 21.8 and 22.3 ft will occur for the -63- and -81-ft channels, respectively, for extreme wave conditions with a +8.0-ft swl. The -63-ft channel bathymetry focuses wave energy slightly more to the east inside the outer harbor than the -81-ft channel bathymetry.

b. For operational wave conditions with the +5.5-ft swl, maximum wave heights of 4.4 and 4.6 ft will occur adjacent to Pier 300 for the -63- and -81-ft channel configurations, respectively, with no landfills installed.

c. For extreme wave conditions with the +8.0-ft swl, maximum wave heights of 10.0 and 10.4 ft will occur adjacent to Pier 300 for the -63- and -81-ft channel configurations, respectively, with no landfills installed.

POLA Stages 1 and 2

a. Both the POLA Stage 1 and Stage 2 landfill configurations provide excellent wave protection to the Pier 300 berthing areas. For operational wave conditions with the +5.5-ft swl, maximum wave heights will not exceed 0.4 ft; and for extreme wave conditions with the +8.0-ft swl, wave heights will not exceed 1.4 ft for either stage of construction.

b. The berth in the channel west of Pier 400 will experience maximum wave heights of 1.0 and 1.1 ft for operational wave conditions with the +5.5-ft swl; and 3.1 and 3.7 ft for extreme wave conditions with the +8.0-ft swl for POLA Stages 1 and 2, respectively.

c. The dredged berth east of Pier 400, included in the POLA Stage 2 configuration, will experience maximum wave heights of 2.4 ft and 3.9 ft for operational and extreme wave conditions, respectively.

d. Overtopping of the landfill in an area approximately 1,500 to 2,000 ft west of the causeway may occur for both POLA Stages 1 and 2 for extreme test wave conditions with the +8.0-ft swl.

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Table 1
Design Wave Information with the -81-ft Channel Depth (No Pier 400 Landfill)

Test Wave				Wave Height at Indicated Gage Location (ft)											
Direction deg	Period sec	Height ft	γ	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12
swl = +5.5 ft															
185	8	4	3.3	4.5	5.5	0.4	1.5	2.8	5.6	1.3	1.1	0.9	1.2	0.9	2.4
		6	3.3	6.3	7.5	0.6	2.2	4.1	8.0	2.2	1.6	1.3	1.8	1.4	3.7
		8	3.3	8.1	9.5	0.8	2.9	5.1	10.3	3.4	2.0	1.7	2.2	1.8	4.6
185	12	4	5.0	4.1	5.3	0.9	1.2	1.8	4.5	2.1	1.9	1.2	2.9	2.3	1.1
		6	5.0	6.4	8.3	1.3	1.7	2.8	7.0	3.0	2.7	1.7	4.2	3.0	1.8
		8	5.0	8.4	10.9	1.6	2.2	3.9	9.2	3.9	3.4	2.2	5.0	3.7	2.6
185	16	4	7.0	3.9	5.5	1.1	2.0	2.2	4.3	2.5	2.0	1.9	1.6	1.5	0.9
		6	7.0	5.9	8.4	1.6	2.7	3.4	6.6	3.9	2.9	2.6	2.5	2.3	1.4
		8	7.0	8.0	11.2	2.1	3.7	4.9	9.1	5.7	3.6	3.3	3.3	3.5	2.3
220	8	4	3.3	5.1	3.5	0.1	2.9	3.4	1.8	0.8	1.1	0.2	0.2	0.5	1.0
		8	3.3	9.2	6.8	0.4	5.5	6.5	3.3	1.9	2.1	0.7	0.4	0.9	1.6
		12	3.3	14.7	11.5	1.0	8.6	12.3	6.2	3.2	3.4	1.4	1.1	1.4	3.0
220	12	4	5.0	6.5	3.1	0.3	2.6	3.8	1.4	1.6	1.5	0.7	1.5	0.8	1.2
		8	5.0	10.9	6.9	0.8	5.4	8.3	2.6	3.2	2.7	1.3	3.2	1.6	1.8
		12	5.0	15.7	10.7	1.5	7.7	11.9	3.6	4.6	3.9	2.1	4.6	2.5	2.6
220	16	4	7.0	4.0	2.2	0.5	1.9	2.2	1.4	1.1	1.4	0.8	0.8	0.7	0.5
		8	7.0	8.1	4.5	1.2	3.7	4.5	2.9	2.2	2.8	1.6	1.8	1.3	1.1
		12	7.0	12.2	7.0	1.7	5.7	7.2	4.6	3.3	3.8	2.7	2.6	2.0	1.7

(Continued)

Table 1 (Concluded)

Test Wave				Wave Height at Indicated Gage Location (ft)											
Direction deg	Period sec	Height ft	γ	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12
swl = +8.0 ft															
185	8	10	3.3	9.8	12.4	0.9	3.6	6.7	11.9	3.2	2.6	2.1	3.0	2.2	5.8
185	12	12	3.3	12.4	16.9	1.9	4.8	7.6	14.4	5.9	4.8	3.3	7.1	4.1	5.5
		12	5.0	12.6	16.6	1.9	4.7	7.8	14.1	5.9	4.9	3.4	7.5	3.5	5.4
185	14	14	5.0	15.0	19.7	3.0	6.7	8.9	15.7	7.9	5.5	4.1	6.3	3.9	4.9
		16	3.3	16.7	21.5	3.5	9.7	10.9	18.3	9.5	6.7	4.9	6.4	6.8	5.8
185	16	18	3.3	17.8	24.9	4.1	12.9	12.2	21.0	11.3	7.7	6.6	6.1	8.7	6.7
		18	5.0	19.0	25.3	4.0	14.0	12.3	20.5	12.3	7.7	6.9	5.6	9.3	6.8
		20	5.0	20.9	27.4	4.4	15.0	13.5	22.3	13.1	8.6	7.7	6.1	10.4	7.3
220	14	14	7.0	17.9	10.4	2.1	8.3	11.4	4.9	5.3	4.7	3.2	3.1	2.6	3.3
220	16	16	7.0	18.6	10.2	2.8	8.6	10.8	5.9	5.4	5.5	3.8	3.3	2.9	3.5
220	18	18	7.0	17.1	12.1	3.4	7.0	12.0	8.3	4.6	6.0	5.1	5.3	4.2	4.3
220	20	20	7.0	17.2	13.6	3.8	7.1	13.5	10.1	4.3	7.7	4.6	6.9	6.4	5.9

Table 2
Design Wave Information for Test Waves from 180 and 190 Deg (-81-Ft Channel Depth) (No Pier 400 Landfill)

swl = +8.0 ft													
Test Wave			Wave Height at Indicated Gage Location (ft)										
Period, sec	Height, ft	γ	Gage 1	Gage 2	Gage 5	Gage 5A	Gage 5B	Gage 6	Gage 6A	Gage 7	Gage 7A	Gage 8	Gage 9
180 Degrees													
8	10	3.3	9.7	9.2	6.0	7.0	6.3	12.5	11.0	6.5	3.4	2.8	3.2
12	12	3.3	10.1	11.8	4.1	5.3	6.9	12.7	14.0	10.0	6.9	5.3	5.1
	12	5.0	10.2	11.8	3.7	4.7	6.5	12.5	14.0	10.2	6.9	5.1	5.3
14	14	5.0	14.2	14.8	4.4	5.9	9.5	14.9	15.3	12.4	9.8	6.3	7.9
	16	3.3	15.4	16.6	5.9	7.6	11.2	17.1	17.7	14.8	11.6	7.1	9.3
16	18	3.3	19.8	20.8	6.2	9.6	14.4	18.9	19.0	16.5	13.8	9.2	10.4
	18	5.0	21.2	22.3	6.8	10.9	14.8	19.5	18.4	16.9	15.5	9.5	10.8
	20	5.0	23.4	24.9	7.9	12.4	17.1	20.6	20.2	18.4	16.7	9.6	11.5
190 Degrees													
8	10	3.3	9.5	13.8	12.1	13.7	12.5	6.3	3.3	3.7	2.0	2.6	1.5
12	12	3.3	16.6	16.0	12.1	13.5	13.1	8.7	5.2	4.7	5.1	3.9	3.8
	12	5.0	17.1	15.8	12.0	12.9	12.2	8.6	4.8	4.4	5.3	4.2	3.7
14	14	5.0	17.1	19.2	13.3	14.0	13.7	11.1	7.1	4.4	5.8	5.2	4.0
	16	3.3	19.7	21.8	16.2	17.2	16.9	14.1	8.8	5.1	6.7	5.8	4.9
16	18	3.3	18.9	24.1	17.7	18.7	18.9	15.5	10.6	6.5	7.6	6.8	5.3
	18	5.0	20.8	25.0	17.5	18.7	18.6	16.2	11.7	6.3	8.1	6.7	5.7
	20	5.0	22.9	27.1	19.8	20.5	20.2	18.0	12.6	7.2	8.3	7.4	6.2

Table 3
Design Wave Information with the -63-ft Channel Depth (No Pier 400 Landfill)

Test Wave				Wave Height at Indicated Gage Location (ft)											
Direction deg	Period sec	Height ft	γ	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12
swl = +5.5 ft															
185	8	4	3.3	4.2	4.3	0.2	3.4	4.4	3.1	0.8	0.7	0.8	0.1	0.3	0.4
		6	3.3	6.2	6.6	0.3	5.3	6.5	4.3	1.2	1.2	1.2	0.1	0.4	0.9
		8	3.3	8.0	8.6	0.4	6.5	8.8	5.6	1.7	1.7	1.6	0.1	0.8	1.7
185	12	4	5.0	4.3	4.4	0.3	4.2	4.9	1.5	0.7	1.5	1.5	0.1	0.4	0.5
		6	5.0	6.6	6.8	0.5	6.0	7.6	2.7	1.0	2.1	2.1	0.2	0.7	0.9
		8	5.0	8.5	9.1	0.7	7.5	9.8	3.9	1.2	2.6	2.4	0.2	0.9	1.7
185	16	4	7.0	5.0	4.8	0.3	3.9	4.9	2.7	0.9	1.6	1.6	0.1	0.6	1.0
		6	7.0	7.6	7.4	0.8	5.8	7.5	4.3	1.3	2.1	2.0	0.2	1.2	1.7
		8	7.0	10.1	9.9	1.1	7.5	9.8	5.9	1.8	2.7	2.7	0.4	1.7	2.7
220	8	4	3.3	4.8	3.0	0.2	2.4	3.8	1.6	0.9	1.1	0.4	0.1	0.1	0.9
		8	3.3	8.9	6.0	0.5	4.9	7.3	3.0	1.9	1.8	0.8	0.1	0.4	1.7
		12	3.3	15.0	10.4	0.9	8.1	12.3	5.3	2.9	3.1	1.4	0.2	1.0	2.9
220	12	4	5.0	6.1	2.2	0.4	1.9	2.2	1.1	1.4	1.7	0.5	0.2	0.3	2.1
		8	5.0	12.5	5.3	0.7	4.0	6.1	2.1	2.9	3.0	0.9	0.3	0.8	3.4
		12	5.0	16.8	8.6	1.0	6.0	9.8	2.9	4.3	4.4	1.8	0.8	1.3	4.4
220	16	4	7.0	3.9	1.3	0.2	1.1	1.4	0.5	1.0	1.3	0.5	0.1	0.3	0.8
		8	7.0	8.0	2.8	0.7	2.1	3.5	1.2	2.0	2.4	1.1	0.2	0.5	1.5
		12	7.0	12.2	4.6	1.2	3.1	6.1	2.1	3.1	3.2	1.9	0.4	1.2	2.4

(Continued)

Table 3 (Concluded)

Test Wave				Wave Height at Indicated Gage Location (ft)											
Direction deg	Period sec	Height ft	γ	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12
swl = +8.0 ft															
185	8	10	3.3	10.0	11.0	0.4	8.0	10.0	7.4	2.0	2.1	2.0	0.1	1.0	2.3
185	12	12	3.3	13.0	14.4	1.1	11.8	14.1	7.1	1.7	3.8	3.6	0.6	1.7	3.1
		12	5.0	13.1	14.5	1.1	12.2	14.3	7.1	1.7	3.9	3.7	0.7	1.8	3.6
185	14	14	5.0	17.1	16.4	1.3	13.5	15.8	9.2	2.5	4.4	4.5	1.0	1.8	3.8
		16	3.3	18.3	18.1	1.6	15.4	17.9	9.9	3.4	5.1	5.5	1.4	2.2	4.5
185	16	18	3.3	20.3	20.9	2.0	16.8	21.8	12.9	4.0	5.7	6.3	1.9	3.7	7.7
		18	5.0	21.8	21.1	2.3	16.5	21.1	13.1	4.5	6.0	6.6	2.1	4.3	9.5
		20	5.0	24.6	22.9	2.4	17.1	21.7	15.5	5.2	7.1	7.3	2.8	5.7	10.0
220	14	14	7.0	16.2	7.2	1.4	5.4	8.0	2.8	4.9	4.5	2.4	0.8	1.4	4.2
220	16	16	7.0	18.6	7.4	1.6	7.6	8.7	3.4	5.0	4.6	3.4	1.2	1.7	4.3
220	18	18	7.0	17.9	7.5	2.0	8.4	10.5	4.6	5.0	6.0	3.9	1.5	2.2	4.3
220	20	20	7.0	18.1	8.3	2.7	9.0	11.3	4.6	6.0	7.0	4.8	1.9	3.1	6.2

Table 4
Wave Heights for POLA Stage 1

Test Wave				Wave Height at Indicated Gage Location (ft)													
Direction deg	Period sec	Height ft	γ	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14
swl = +5.5 ft																	
185	8	4	3.3	4.6	4.9	0.3	2.2	0.2	0.5	4.6	0.9	0.1	0.1	0.1	0.1	0.1	0.1
		6	3.3	6.4	7.0	0.6	3.5	0.2	0.7	6.8	1.3	0.2	0.1	0.1	0.1	0.1	0.1
		8	3.3	8.4	9.1	0.8	4.7	0.3	0.9	8.7	1.6	0.3	0.1	0.1	0.1	0.1	0.1
185	12	4	5.0	4.4	4.8	0.6	3.3	0.2	0.6	5.1	2.0	0.4	0.1	0.1	0.1	0.1	0.1
		6	5.0	6.8	7.3	0.9	4.7	0.4	0.9	7.6	2.8	0.6	0.2	0.2	0.1	0.1	0.1
		8	5.0	8.9	9.6	1.1	5.8	0.7	1.1	9.7	3.4	0.7	0.4	0.3	0.1	0.1	0.1
185	16	4	7.0	4.9	4.9	1.1	1.9	0.7	0.8	3.3	1.9	0.5	0.3	0.2	0.2	0.1	0.2
		6	7.0	7.4	7.5	1.6	2.9	1.2	1.3	4.8	2.6	0.8	0.5	0.3	0.2	0.1	0.3
		8	7.0	10.1	10.3	2.0	4.0	1.8	1.7	6.5	3.2	1.0	0.6	0.4	0.3	0.1	0.4
220	8	4	3.3	4.7	2.9	0.3	1.6	0.4	0.4	3.7	0.4	0.1	0.1	0.1	0.1	0.1	0.1
		8	3.3	9.1	6.1	0.6	3.1	0.7	0.6	7.0	0.8	0.1	0.1	0.1	0.1	0.1	0.1
		12	3.3	15.2	11.3	1.1	5.4	0.8	0.9	12.8	1.5	0.2	0.2	0.2	0.1	0.1	0.1
220	12	4	5.0	6.1	2.4	0.6	1.8	0.5	0.6	2.4	0.7	0.3	0.2	0.2	0.1	0.1	0.1
		8	5.0	12.5	5.4	1.1	3.2	0.7	1.0	5.6	1.4	0.3	0.4	0.4	0.1	0.1	0.2
		12	5.0	17.6	8.3	1.6	3.9	1.4	1.5	7.7	2.7	0.6	0.7	0.6	0.3	0.1	0.3
220	16	4	7.0	4.0	1.2	0.7	0.7	0.7	0.5	1.0	0.6	0.2	0.2	0.2	0.1	0.1	0.2
		8	7.0	8.2	2.8	1.3	1.4	1.0	1.0	2.3	1.3	0.3	0.4	0.3	0.2	0.1	0.3
		12	7.0	12.5	4.5	1.9	1.8	1.7	1.7	3.3	2.1	0.7	0.7	0.5	0.4	0.1	0.5

(Continued)

Table 4 (Concluded)																	
Test Wave				Wave Height at Indicated Gage Location (ft)													
Direction deg	Period sec	Height ft	γ	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14
swl = +8.0 ft																	
185	8	10	3.3	10.5	11.1	0.9	6.1	0.7	1.0	9.7	2.1	0.4	0.2	0.1	0.1	0.1	0.1
185	12	12	3.3	13.6	15.3	1.7	9.9	1.4	1.5	13.5	4.7	1.3	0.6	0.6	0.2	0.1	0.3
		12	5.0	13.6	15.4	1.8	10.1	1.5	1.5	13.3	4.9	1.5	0.8	0.9	0.1	0.1	0.3
185	14	14	5.0	17.4	17.6	2.6	8.4	2.2	2.3	12.2	5.8	1.9	1.1	0.9	0.2	0.2	0.5
		16	3.3	19.1	19.5	3.3	10.0	3.1	3.0	14.3	6.9	2.4	1.5	1.4	0.4	0.3	0.7
185	16	18	3.3	20.5	22.7	4.0	10.0	3.7	3.9	16.4	7.1	2.8	1.8	1.6	1.0	0.7	1.0
		18	5.0	22.5	23.5	4.3	10.8	4.2	4.2	15.8	7.1	3.1	2.0	1.9	1.1	0.8	1.1
		20	5.0	24.9	26.0	4.7	12.3	4.8	4.5	17.8	7.8	3.7	2.4	2.3	1.4	1.1	1.4
220	14	14	7.0	16.6	8.0	2.0	4.6	2.0	2.5	8.7	3.3	1.1	0.9	0.7	0.5	0.4	0.4
220	16	16	7.0	18.5	8.1	2.5	4.8	2.6	3.1	8.2	3.7	1.3	1.1	0.8	0.4	0.3	0.5
220	18	18	7.0	18.0	8.1	2.8	5.5	3.2	3.2	9.0	2.9	1.5	1.2	0.8	0.9	0.8	0.9
220	20	20	7.0	18.0	9.1	3.9	6.3	4.1	4.0	10.9	3.0	1.7	1.7	1.0	1.1	1.2	1.1

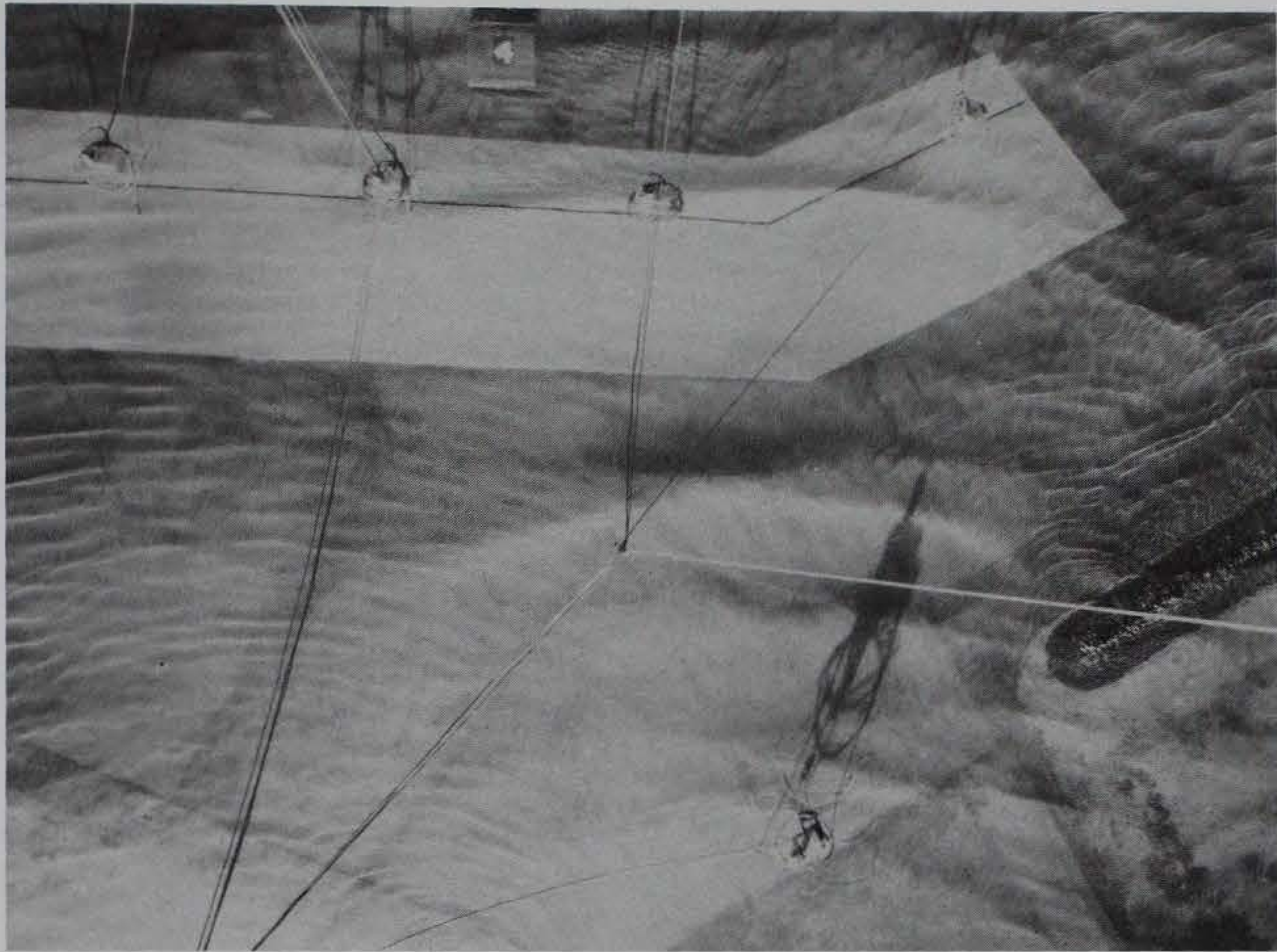
Table 5
Wave Heights for POLA Stage 2

Test Wave				Wave Height at Indicated Gage Location (ft)															
Direction deg	Period sec	Height ft	γ	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14	Gage 15	Gage 16
swl = +5.5 ft																			
185	8	4	3.3	4.3	6.2	0.6	1.0	0.2	0.6	2.3	1.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
		6	3.3	6.1	8.7	0.8	1.5	0.3	0.8	3.6	1.5	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1
		8	3.3	8.1	10.7	0.8	2.0	0.4	1.0	4.9	1.8	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1
185	12	4	5.0	4.4	5.8	1.1	0.8	0.2	0.3	1.7	1.6	0.2	0.2	0.1	0.1	0.1	0.1	0.1	0.1
		6	5.0	6.7	8.9	1.8	1.3	0.6	0.7	2.7	2.2	0.6	0.5	0.2	0.3	0.1	0.2	0.1	0.2
		8	5.0	8.7	11.7	2.3	1.7	0.8	0.9	3.6	2.6	0.7	0.6	0.3	0.3	0.1	0.1	0.1	0.2
185	16	4	7.0	4.2	5.8	2.1	0.7	0.4	0.4	1.6	1.3	0.3	0.4	0.2	0.2	0.1	0.1	0.1	0.3
		6	7.0	6.4	8.9	2.9	1.0	0.6	0.6	2.3	1.8	0.6	0.6	0.4	0.4	0.3	0.3	0.2	0.4
		8	7.0	8.5	11.9	3.9	1.5	0.8	0.8	3.1	2.3	0.8	0.8	0.5	0.6	0.3	0.4	0.2	0.5
220	8	4	3.3	4.6	3.4	0.1	2.5	0.2	0.8	4.4	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
		8	3.3	8.8	6.8	0.3	4.4	0.7	1.3	7.4	0.5	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
		12	3.3	14.8	11.6	0.8	7.3	1.3	2.4	13.6	1.4	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1
220	12	4	5.0	6.2	2.5	0.4	2.6	0.6	0.8	3.3	0.6	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1
		8	5.0	13.1	5.7	1.2	5.4	1.3	1.7	7.4	1.4	0.5	0.5	0.3	0.1	0.2	0.2	0.2	0.2
		12	5.0	18.8	9.1	1.8	7.5	1.7	2.4	11.2	2.5	0.9	0.9	0.3	0.3	0.2	0.2	0.2	0.2
220	16	4	7.0	4.0	1.7	0.5	1.1	0.2	0.3	1.4	0.4	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1
		8	7.0	8.1	3.7	1.2	2.4	0.8	1.1	3.2	1.0	0.6	0.3	0.1	0.1	0.1	0.1	0.1	0.1
		12	7.0	12.5	5.9	1.8	3.6	1.3	1.7	4.9	1.6	1.1	0.6	0.3	0.3	0.3	0.2	0.2	0.2

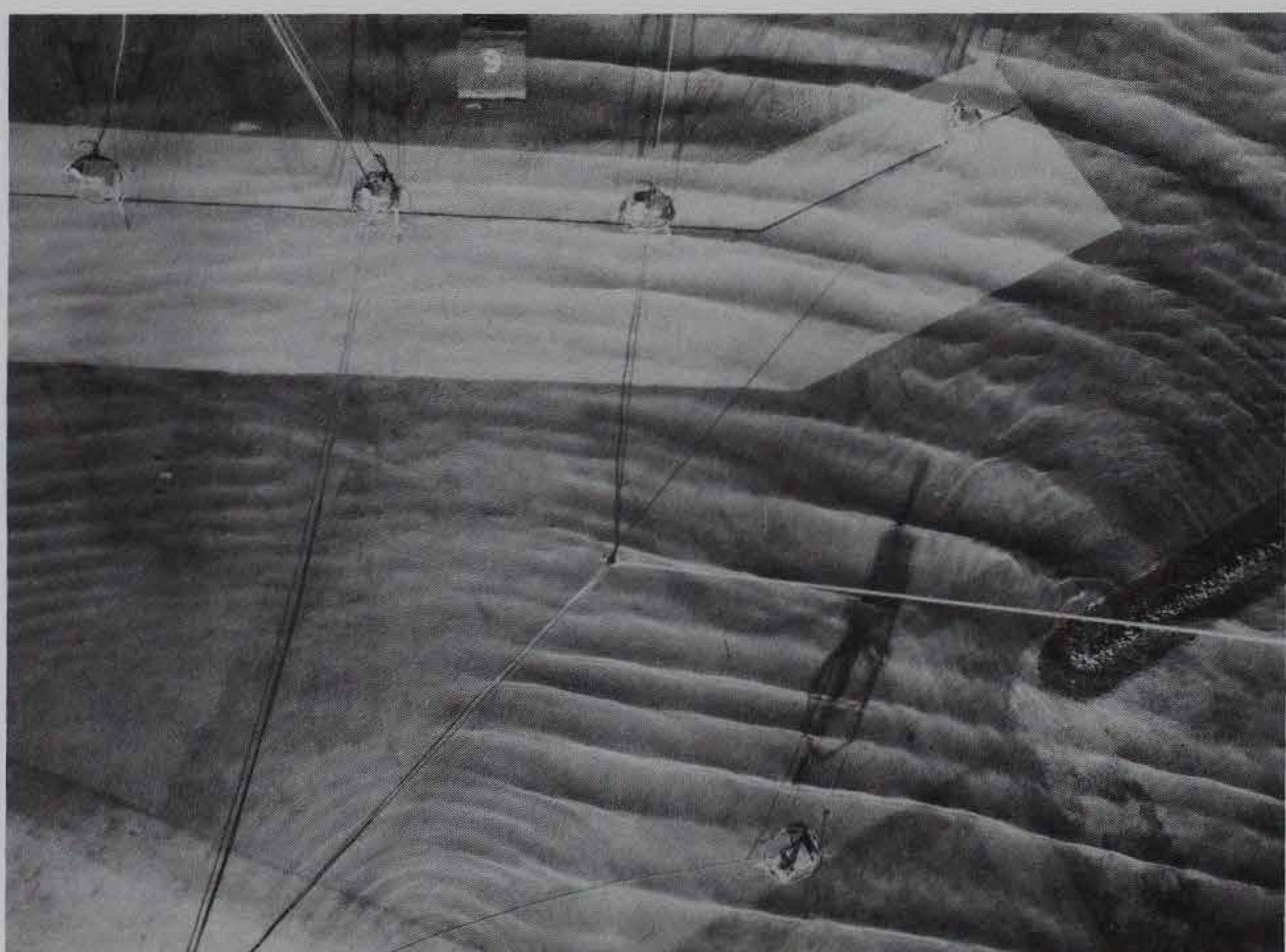
(Continued)

Table 5 (Concluded)

Test Wave				Wave Height at Indicated Gage Location (ft)															
Direction deg	Period sec	Height ft	γ	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14	Gage 15	Gage 16
swl = +8.0 ft																			
185	8	10	3.3	10.1	13.3	1.1	2.6	0.7	1.5	5.7	2.2	0.3	0.2	0.1	0.2	0.1	0.1	0.1	0.1
185	12	12	3.3	13.2	17.8	3.2	4.3	1.5	2.1	7.5	4.3	1.2	1.1	0.6	1.0	0.3	0.3	0.4	0.4
		12	5.0	13.3	17.7	3.1	4.5	1.7	2.0	7.4	4.3	1.3	1.2	0.6	1.1	0.3	0.3	0.3	0.3
185	14	14	5.0	14.9	20.6	4.9	5.0	1.6	2.1	6.9	4.7	1.7	1.6	0.9	1.6	0.5	0.7	0.5	0.7
		16	3.3	16.1	22.3	5.8	6.6	1.9	2.7	8.8	5.6	2.0	1.8	1.0	2.1	0.6	0.8	0.9	0.8
185	16	18	3.3	18.4	25.0	7.5	6.9	2.2	2.9	9.4	5.7	2.4	2.0	1.4	2.2	1.1	0.9	0.8	1.2
		18	5.0	19.5	26.4	7.8	7.7	2.2	3.4	9.8	5.8	2.7	2.1	1.4	2.5	1.0	0.9	1.1	1.3
		20	5.0	21.9	27.2	8.1	9.2	2.8	3.9	11.3	6.3	3.1	2.5	1.6	2.9	1.3	1.2	1.3	1.5
220	14	14	7.0	18.0	9.0	1.7	7.6	1.9	1.9	8.9	3.0	1.1	1.0	0.5	0.7	0.2	0.3	0.2	0.2
220	16	16	7.0	18.9	9.2	2.3	6.7	1.9	2.0	8.2	3.1	1.2	0.9	0.4	0.6	0.3	0.3	0.2	0.2
220	18	18	7.0	17.1	11.2	3.1	6.1	2.2	2.4	8.5	2.4	1.1	0.9	0.8	0.5	0.8	0.5	0.5	0.7
220	20	20	7.0	17.5	12.9	4.3	6.1	2.6	2.9	11.1	2.8	1.4	0.9	0.9	0.8	1.0	0.6	0.8	0.7

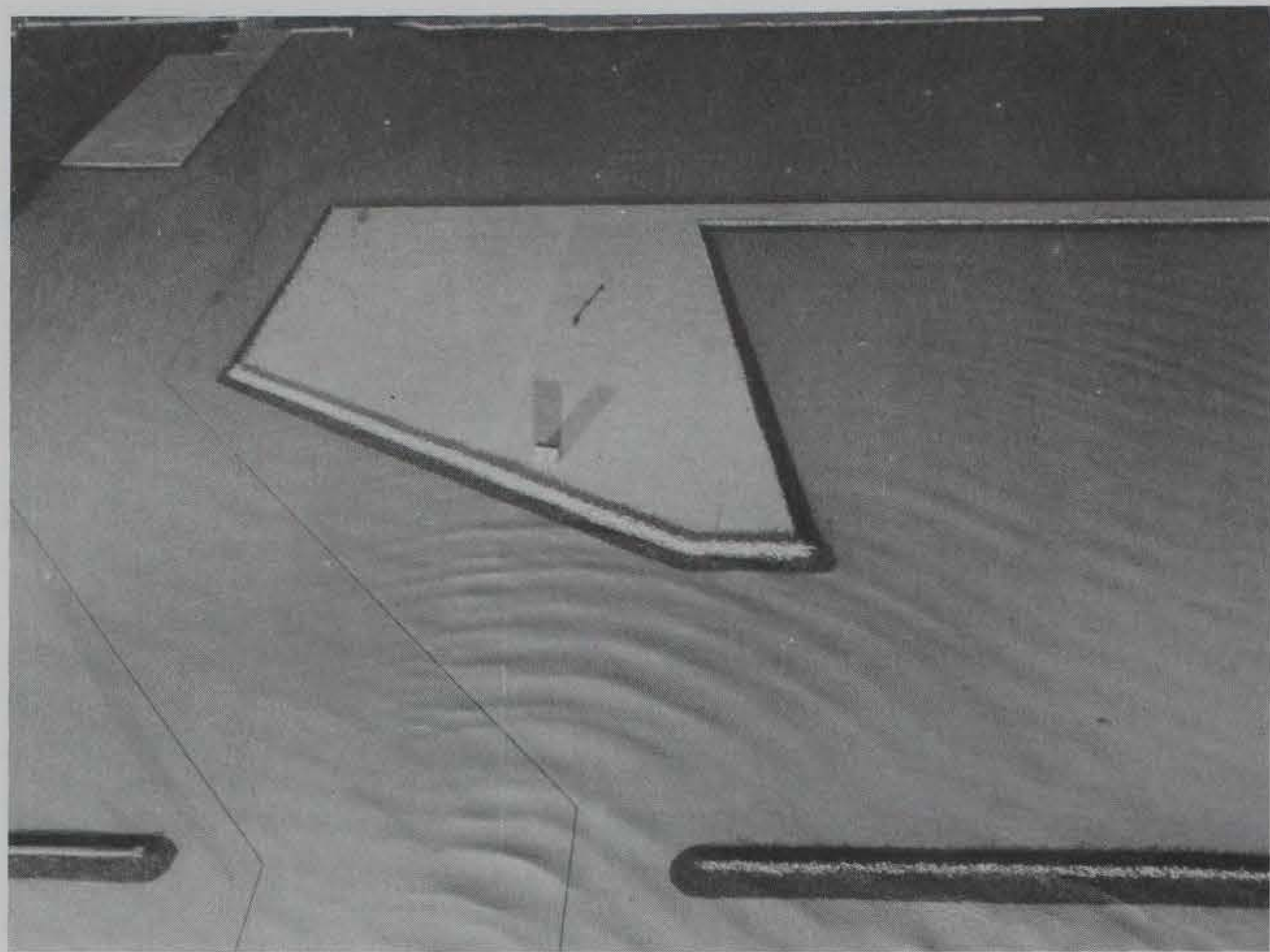


a. 8-sec, 10-ft test waves from 185 deg; swl = +8.0 ft

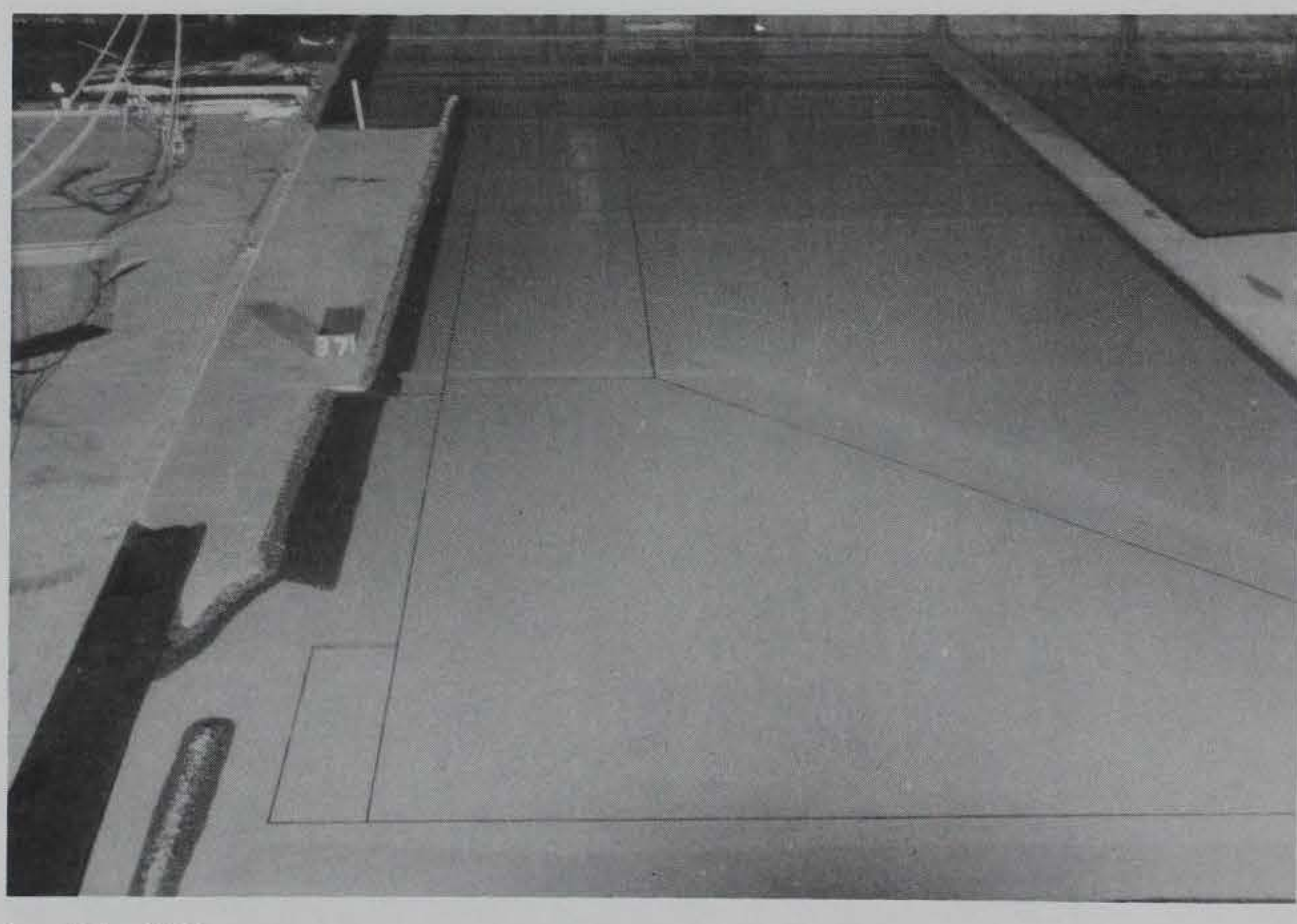


b. 8-sec, 12-ft test waves from 220 deg; swl = +5.5 ft

Photo 1. Typical wave patterns at toe of proposed Pier 400 landfill

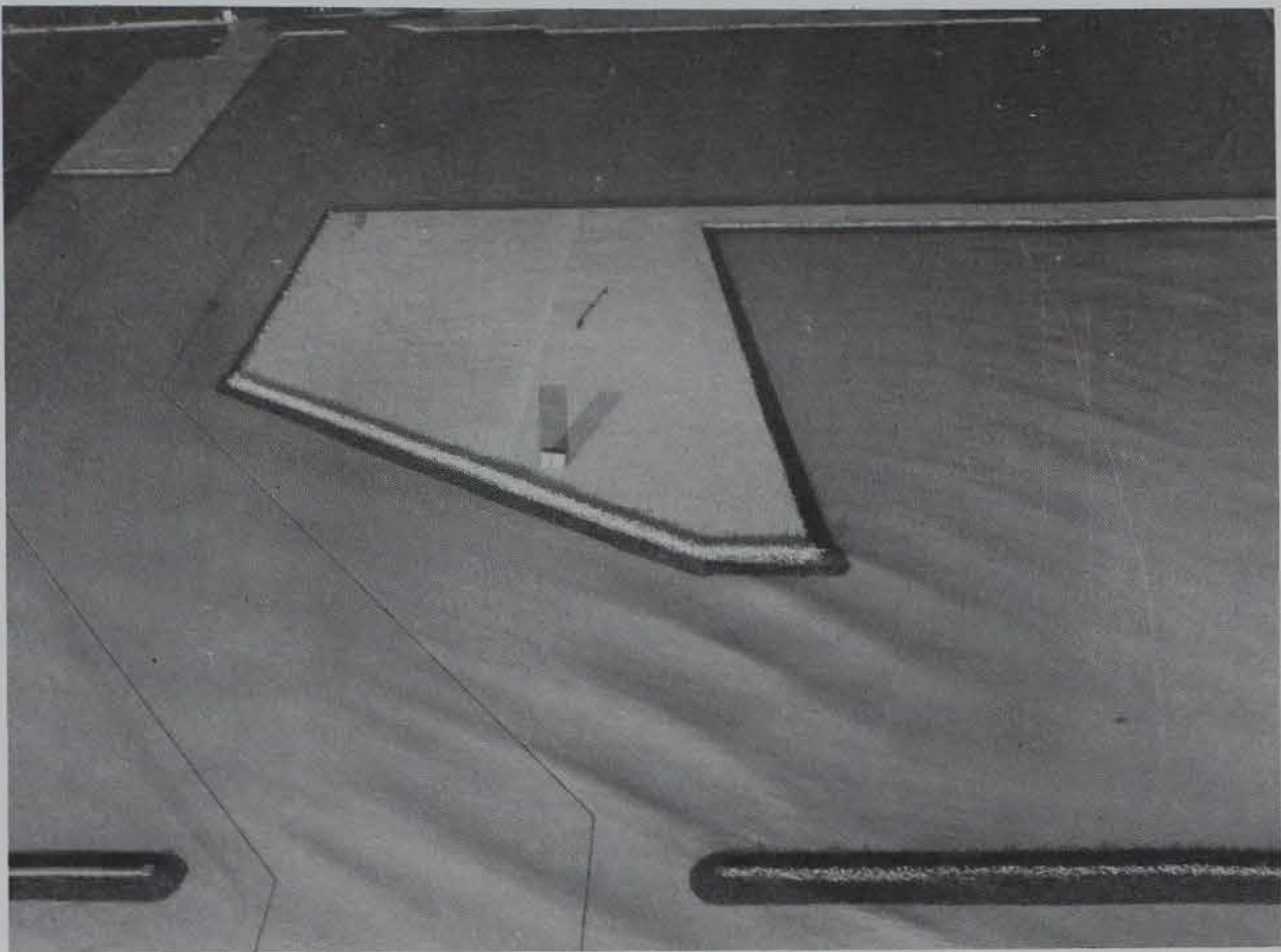


a. Pier 400

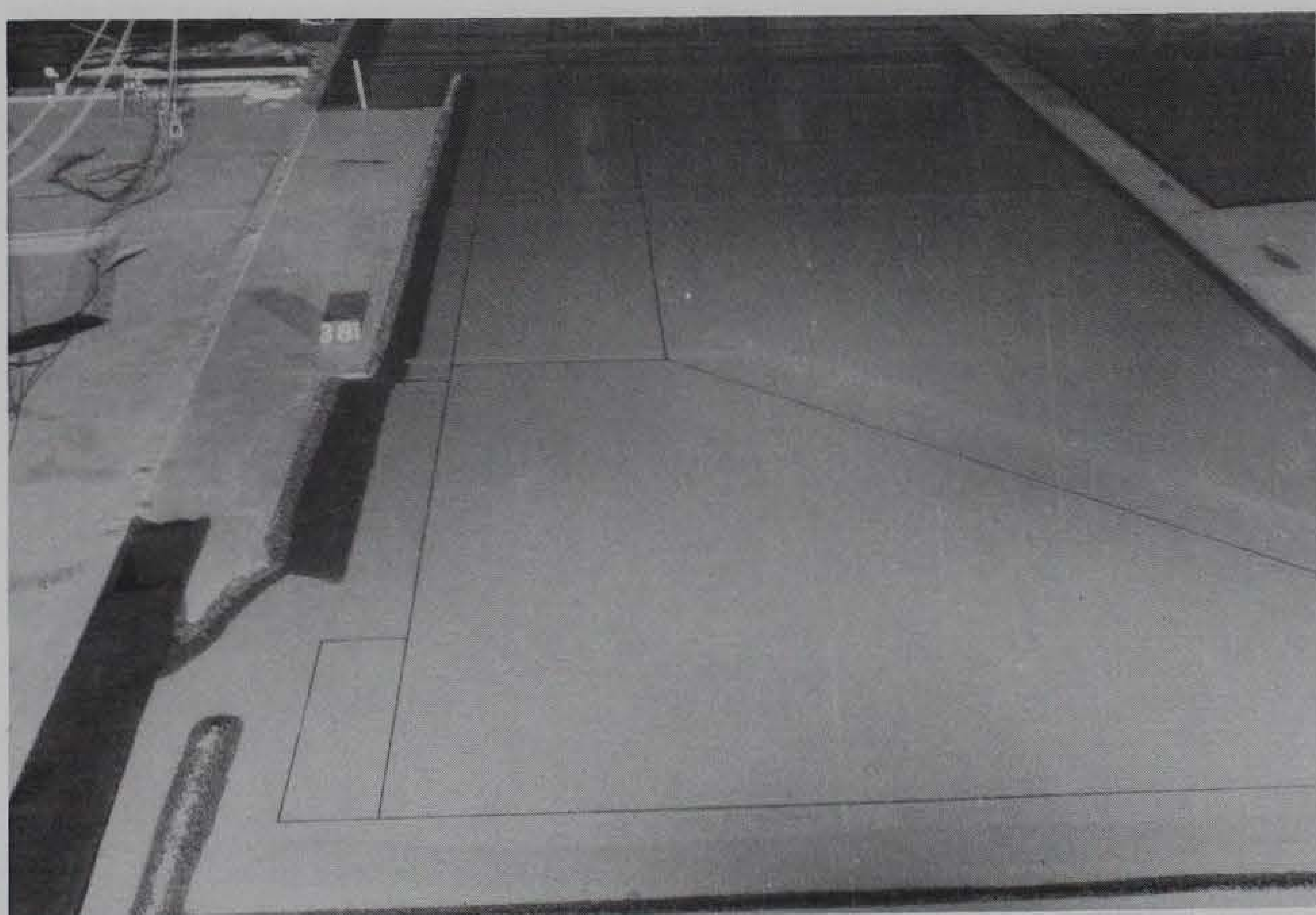


b. Pier 300

Photo 2. Typical wave patterns for POLA Stage 1; 8-sec, 4-ft test waves from 185 deg; $\gamma = 3.3$; swl = +5.5 ft

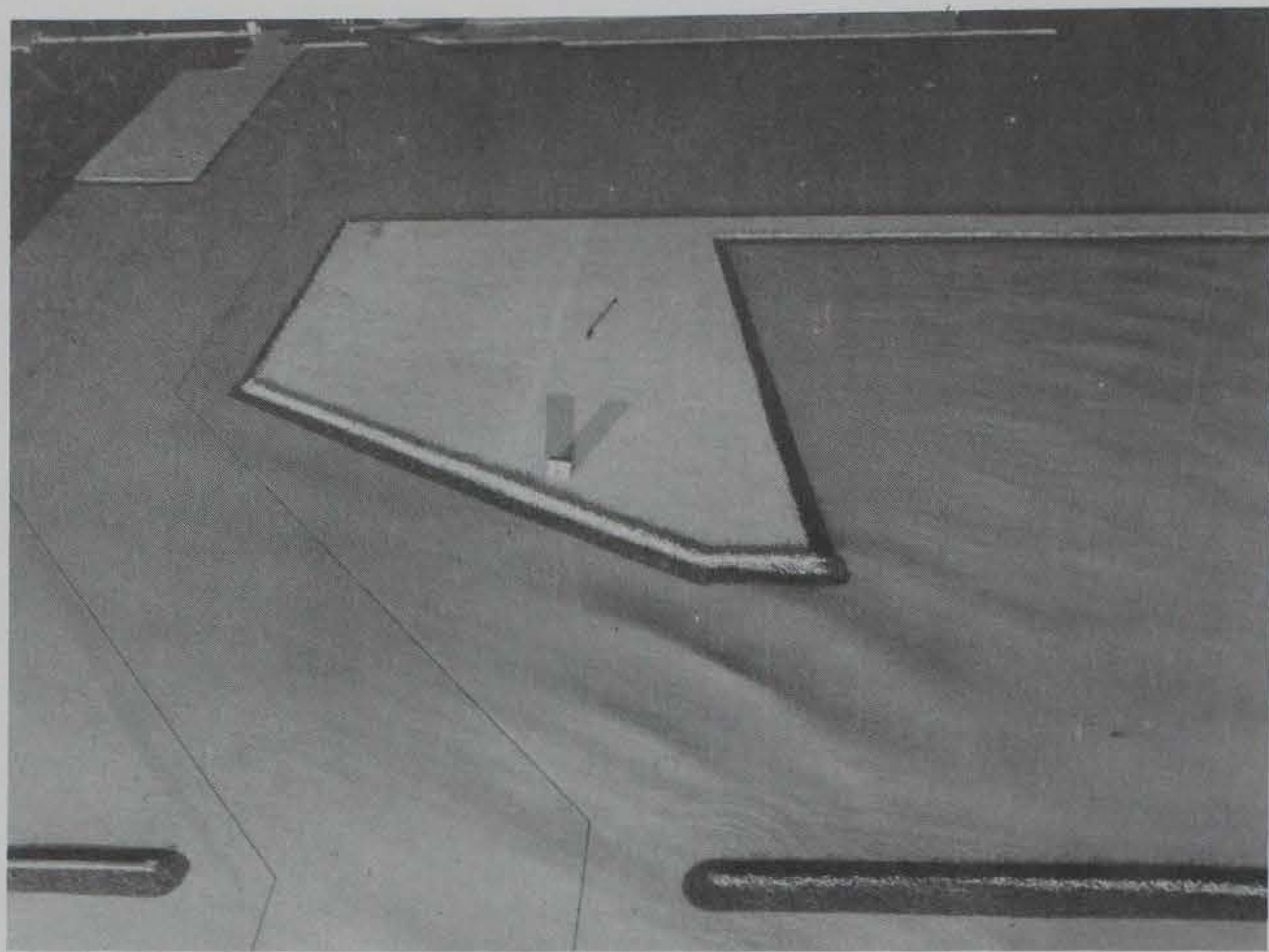


a. Pier 400

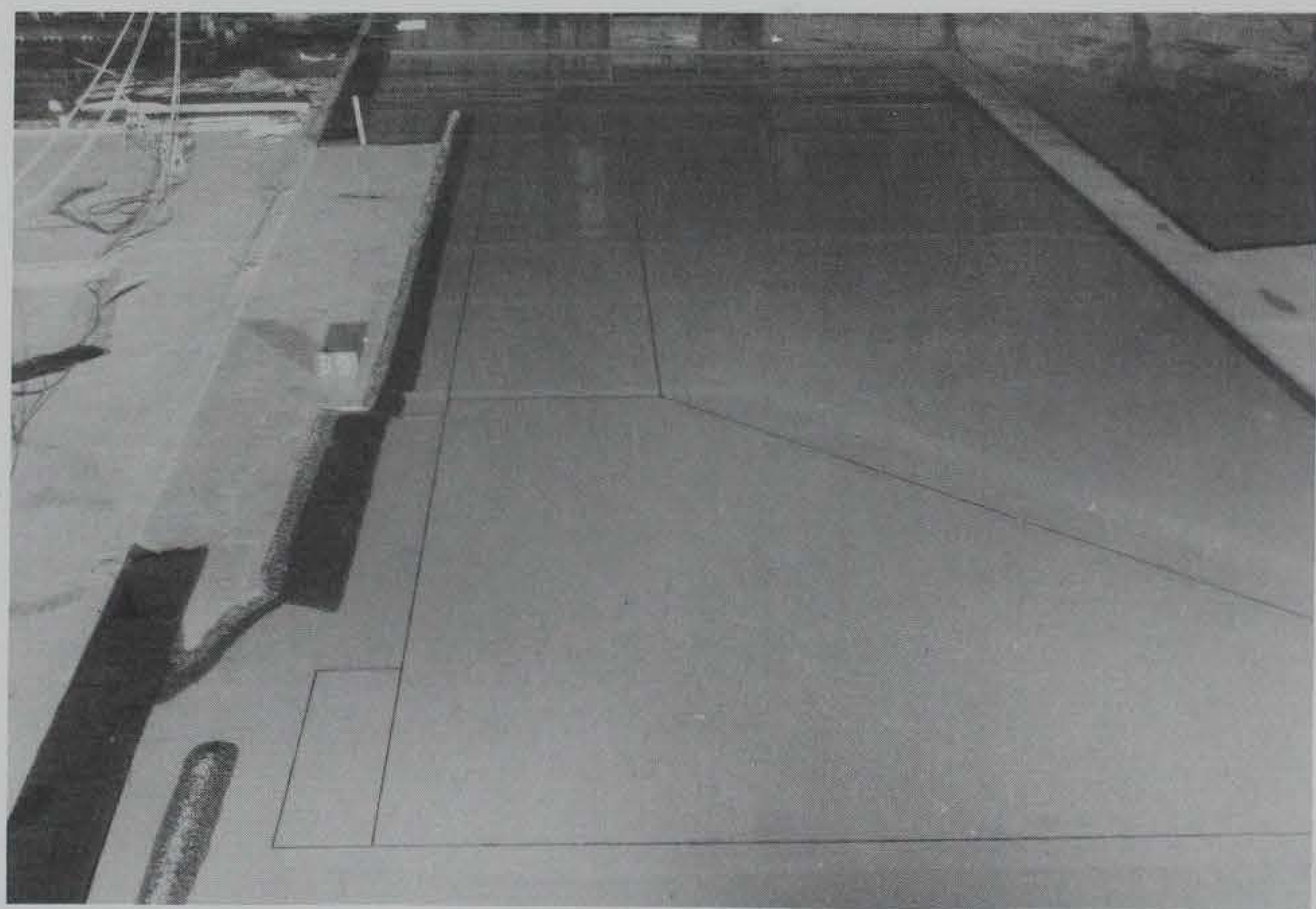


b. Pier 300

Photo 3. Typical wave patterns for POLA Stage 1; 12-sec, 6-ft test waves from 185 deg; $\gamma = 3.3$; swl = +5.5 ft

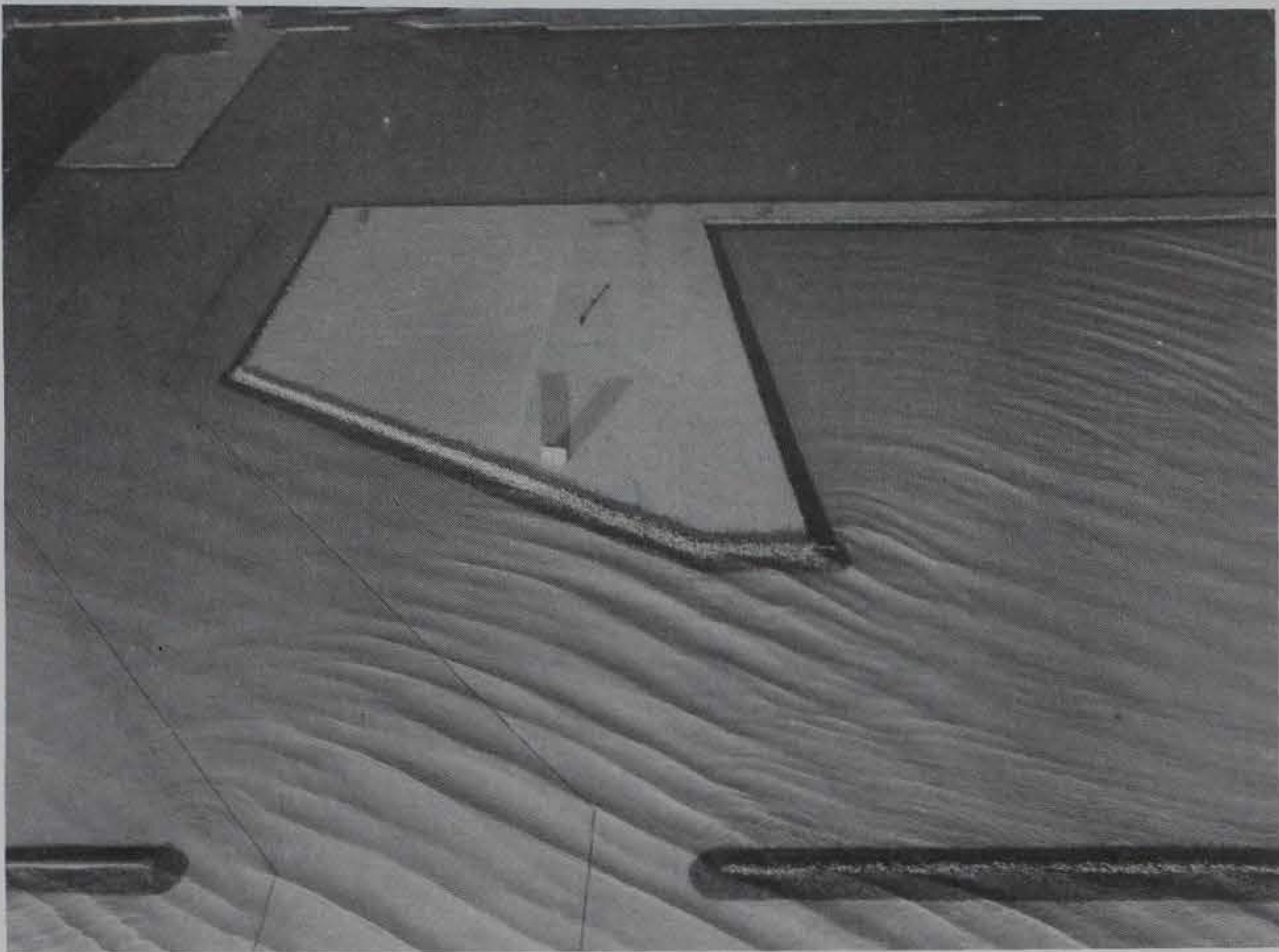


a. Pier 400



b. Pier 300

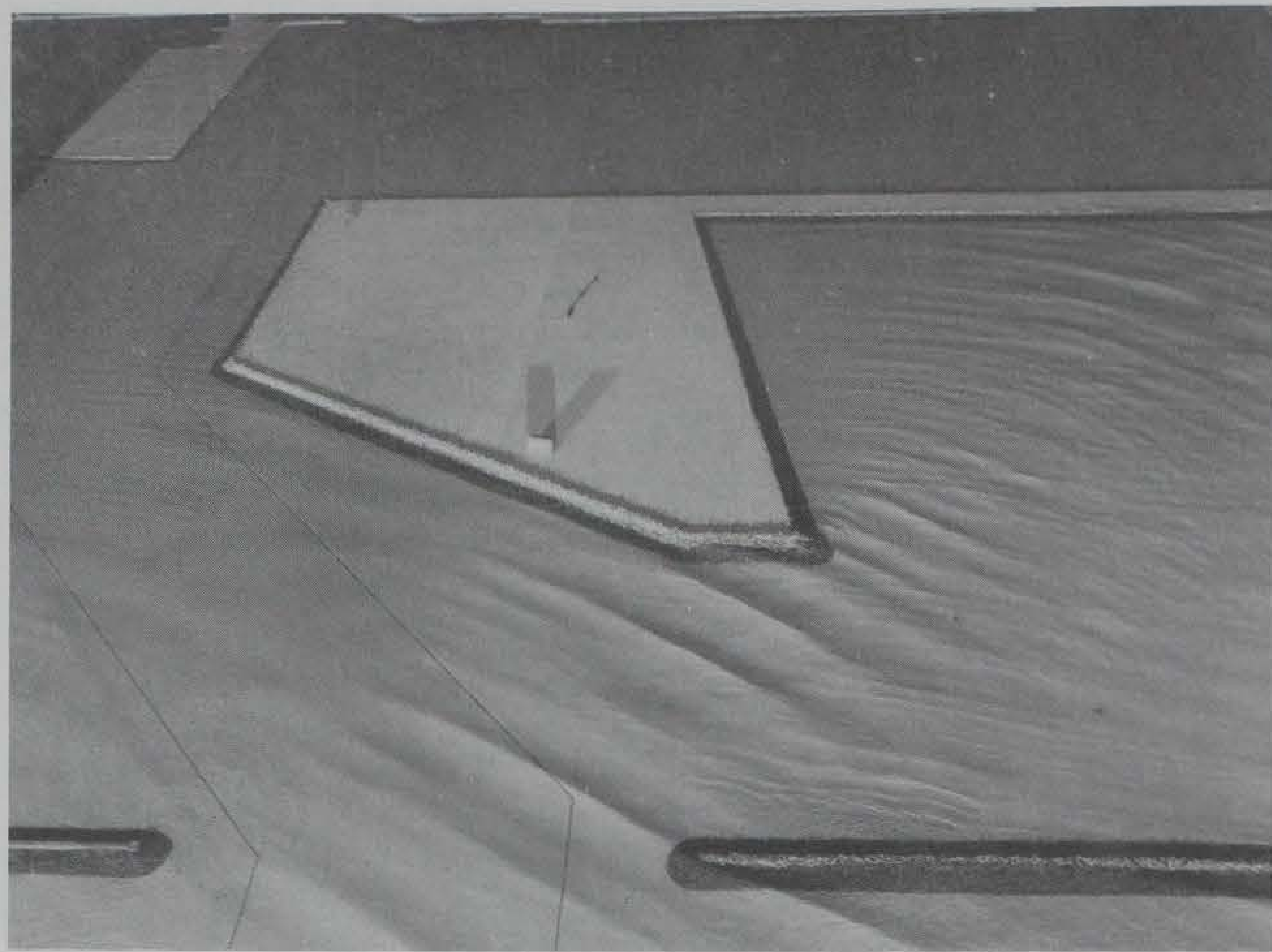
Photo 4. Typical wave patterns for POLA Stage 1; 16-sec, 8-ft test waves from 185 deg; $\gamma = 3.3$; swl = +5.5 ft



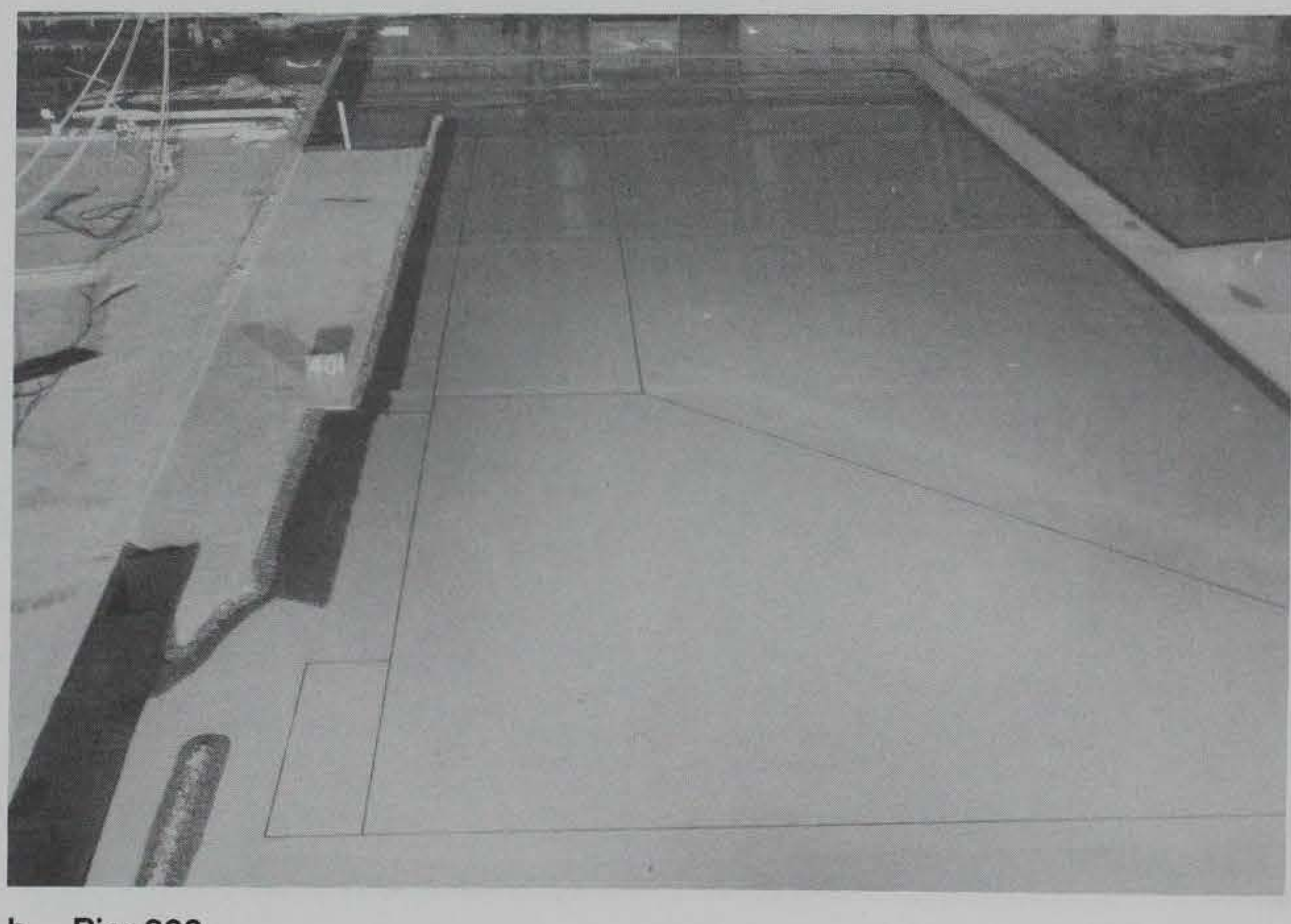
a. Pier 400

b. Pier 300

Photo 5. Typical wave patterns for POLA Stage 1; 8-sec, 10-ft test waves from 185 deg; $\gamma = 3.3$; swl = +8.0 ft

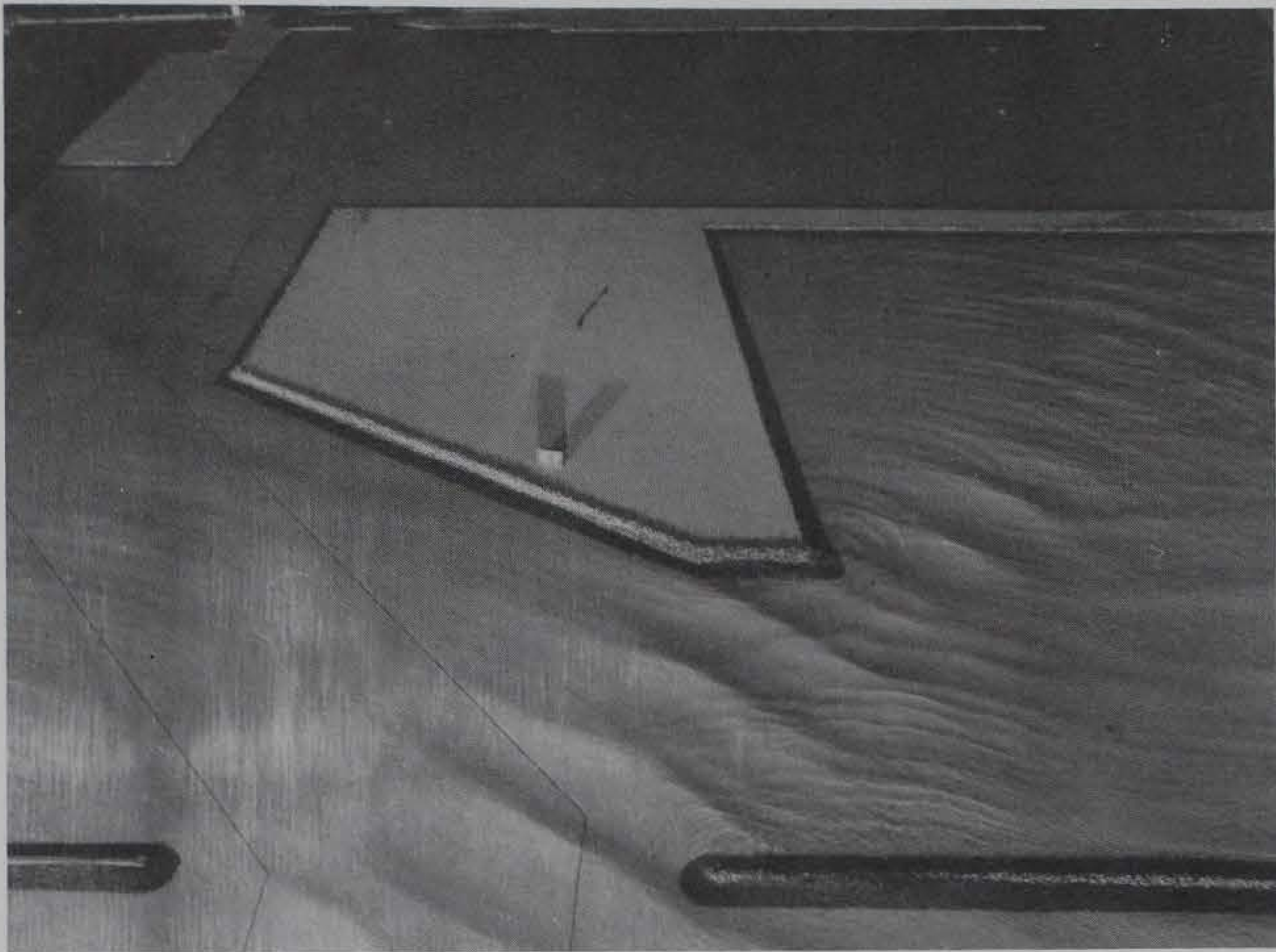


a. Pier 400

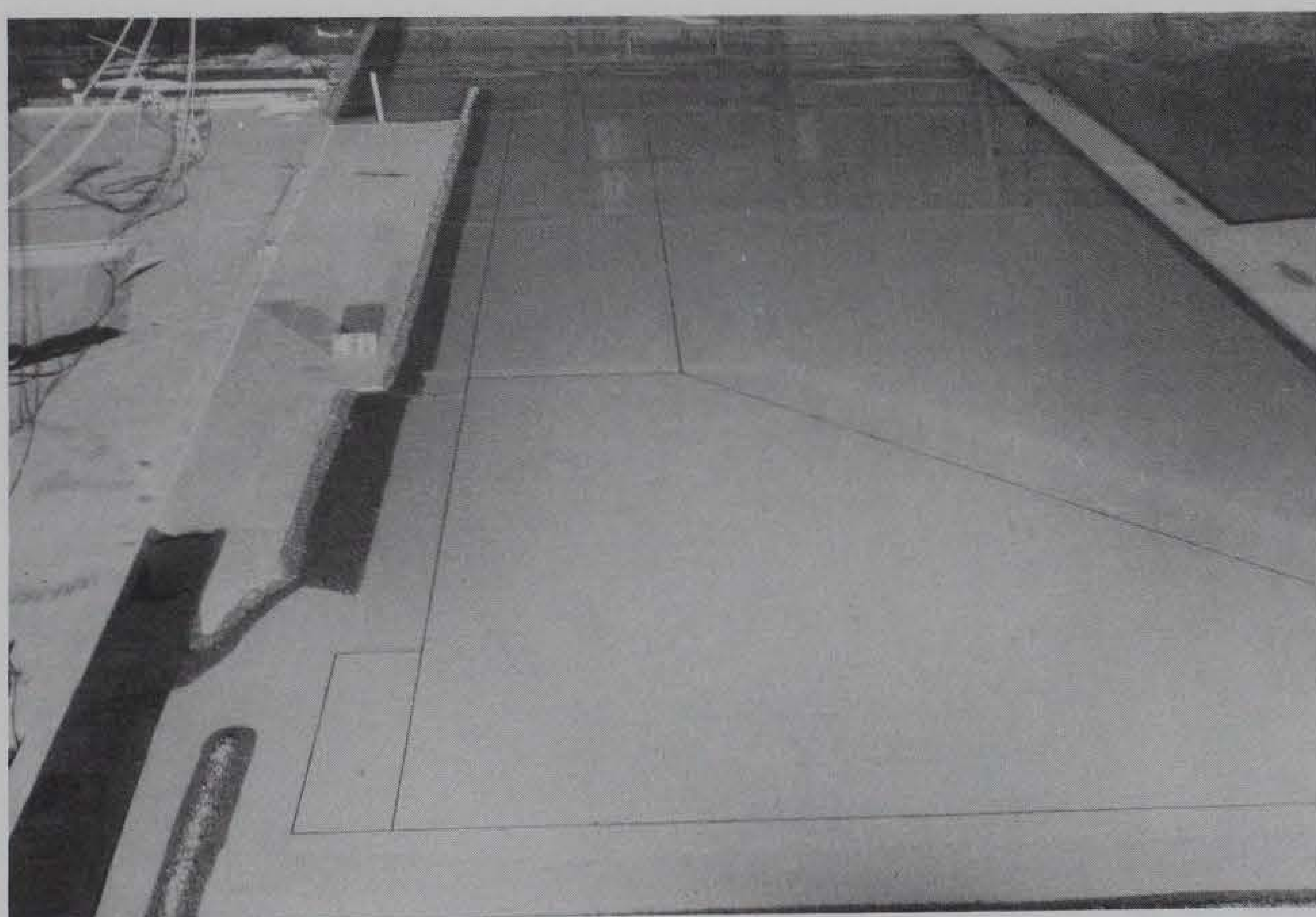


b. Pier 300

Photo 6. Typical wave patterns for POLA Stage 1; 12-sec, 12-ft test waves from 185 deg; $\gamma = 3.3$; swl = +8.0 ft

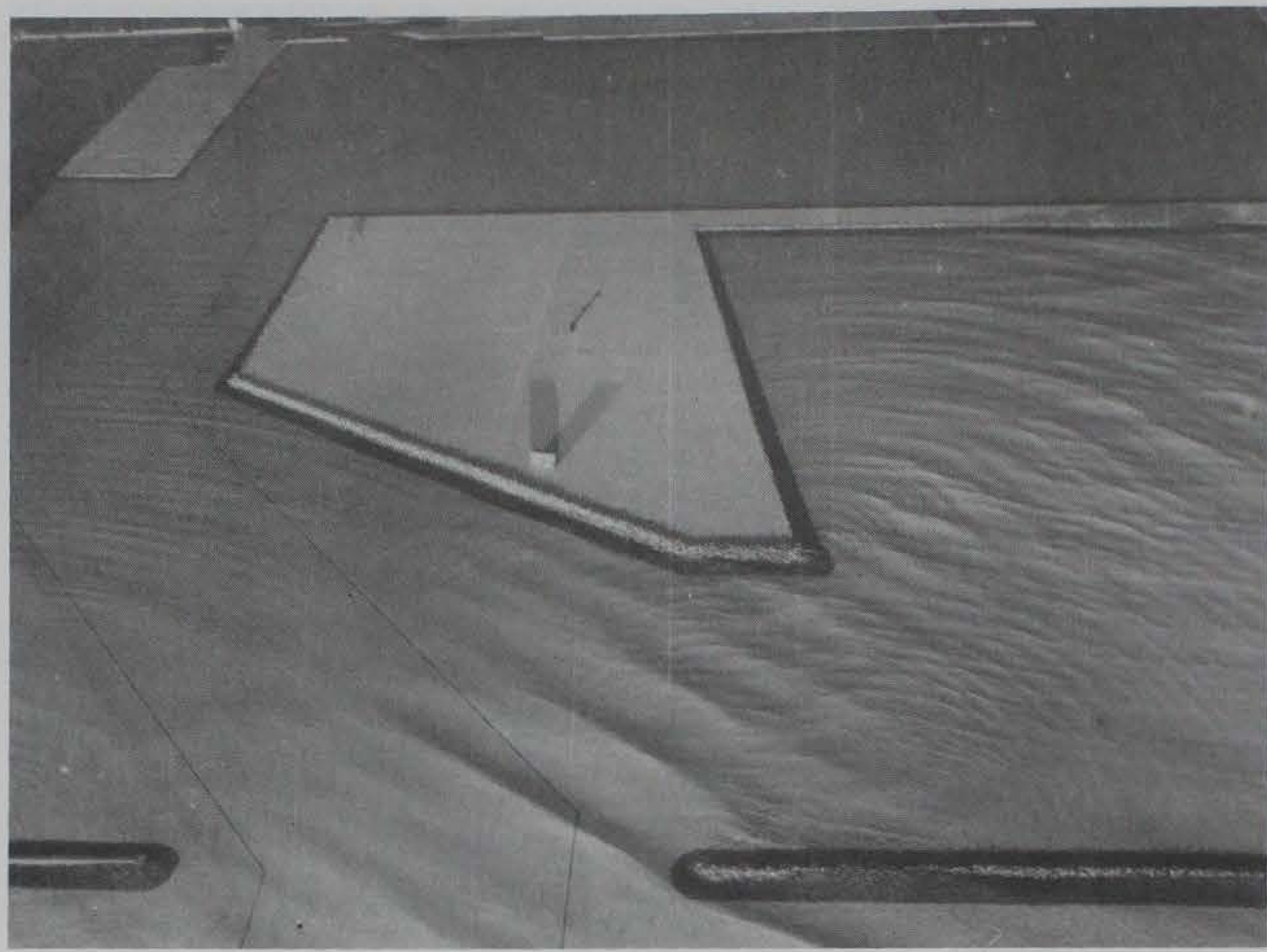


a. Pier 400

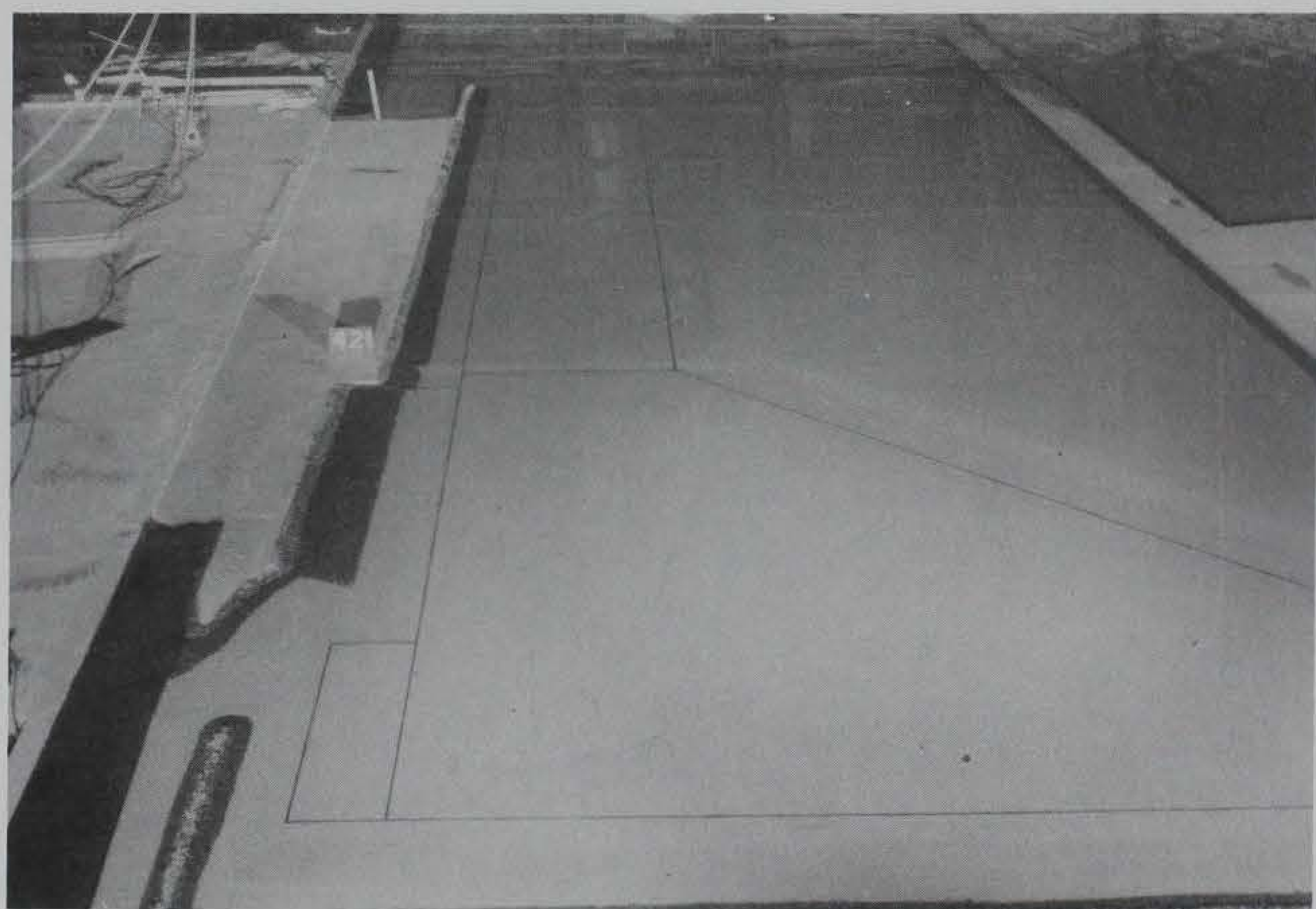


b. Pier 300

Photo 7. Typical wave patterns for POLA Stage 1; 14-sec, 14-ft test waves from 185 deg; $\gamma = 5.0$; swl = +8.0 ft

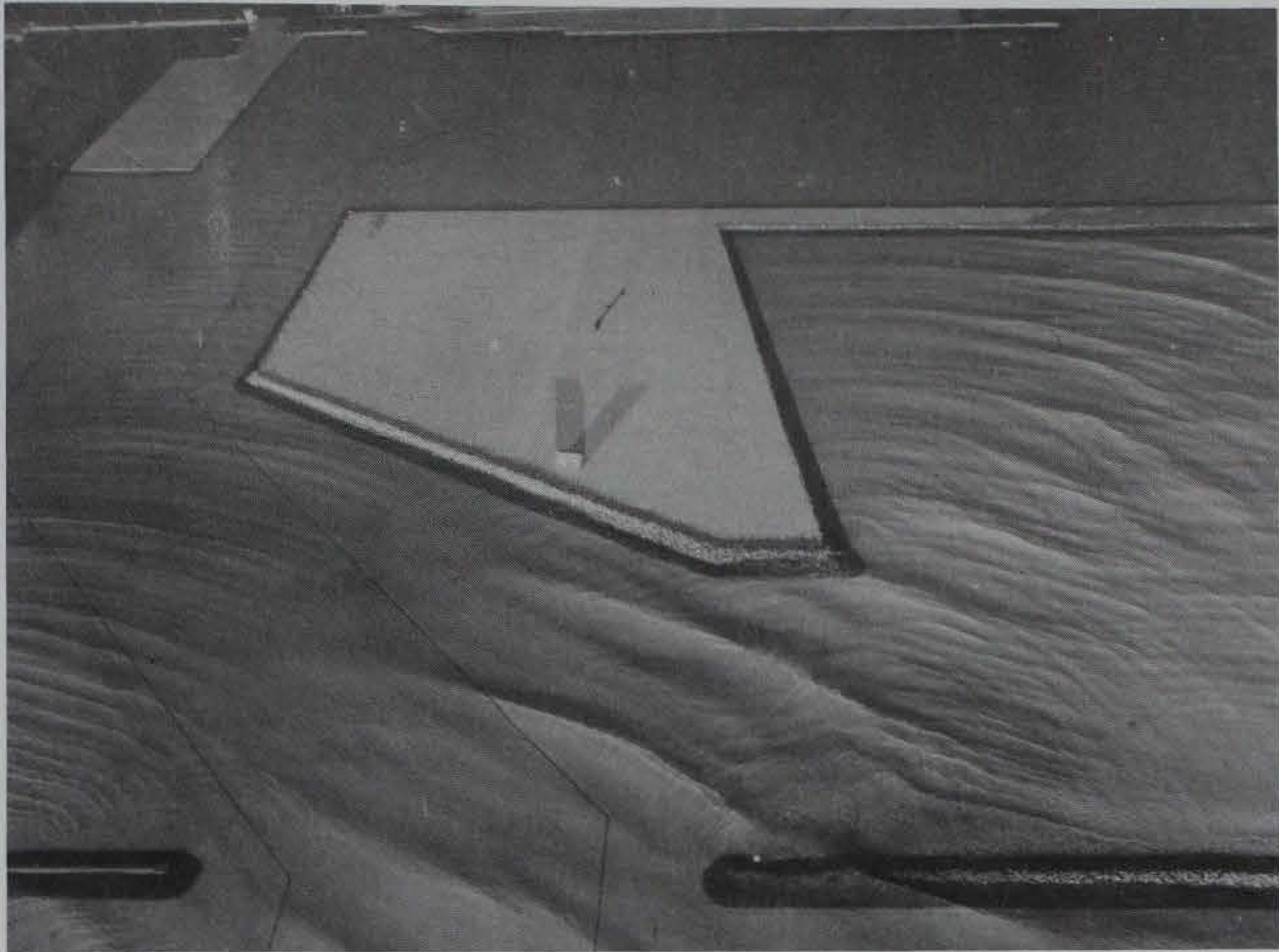


a. Pier 400



b. Pier 300

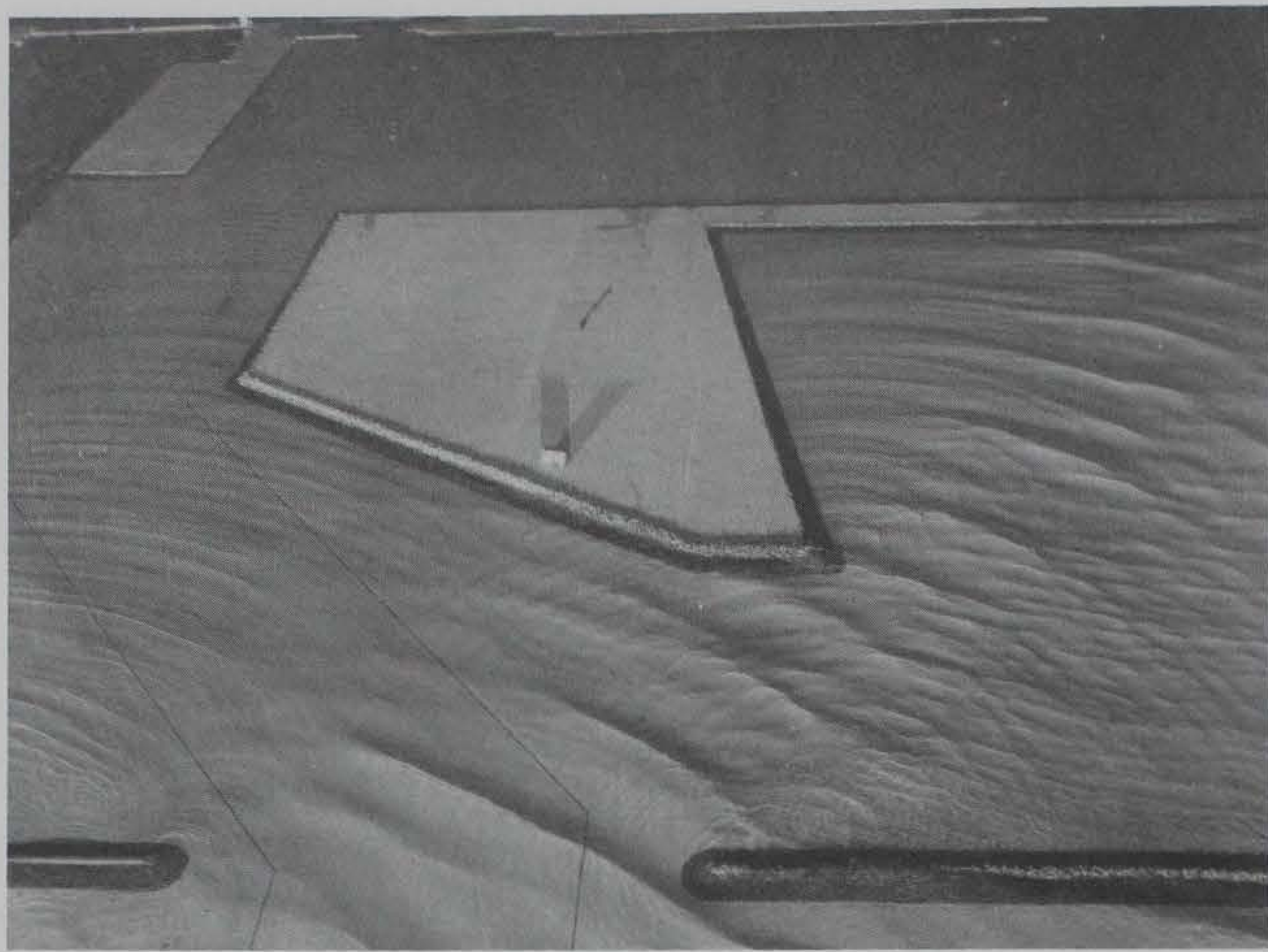
Photo 8. Typical wave patterns for POLA Stage 1; 14-sec, 16-ft test waves from 185 deg; $\gamma = 3.3$; swl = +8.0 ft



a. Pier 400

b. Pier 300

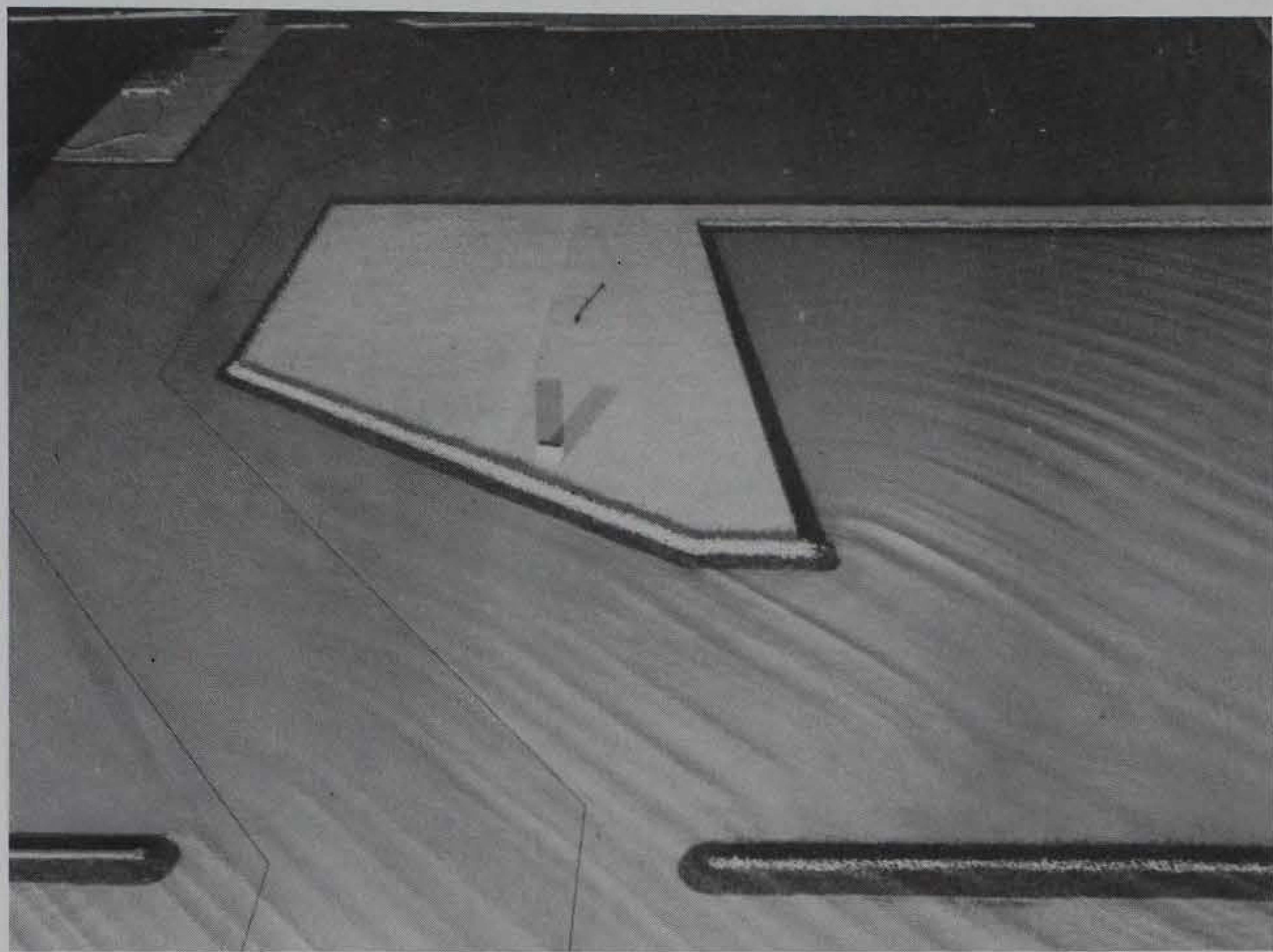
Photo 9. Typical wave patterns for POLA Stage 1; 16-sec, 18-ft test waves from 185 deg; $\gamma = 3.3$; swl = +8.0 ft



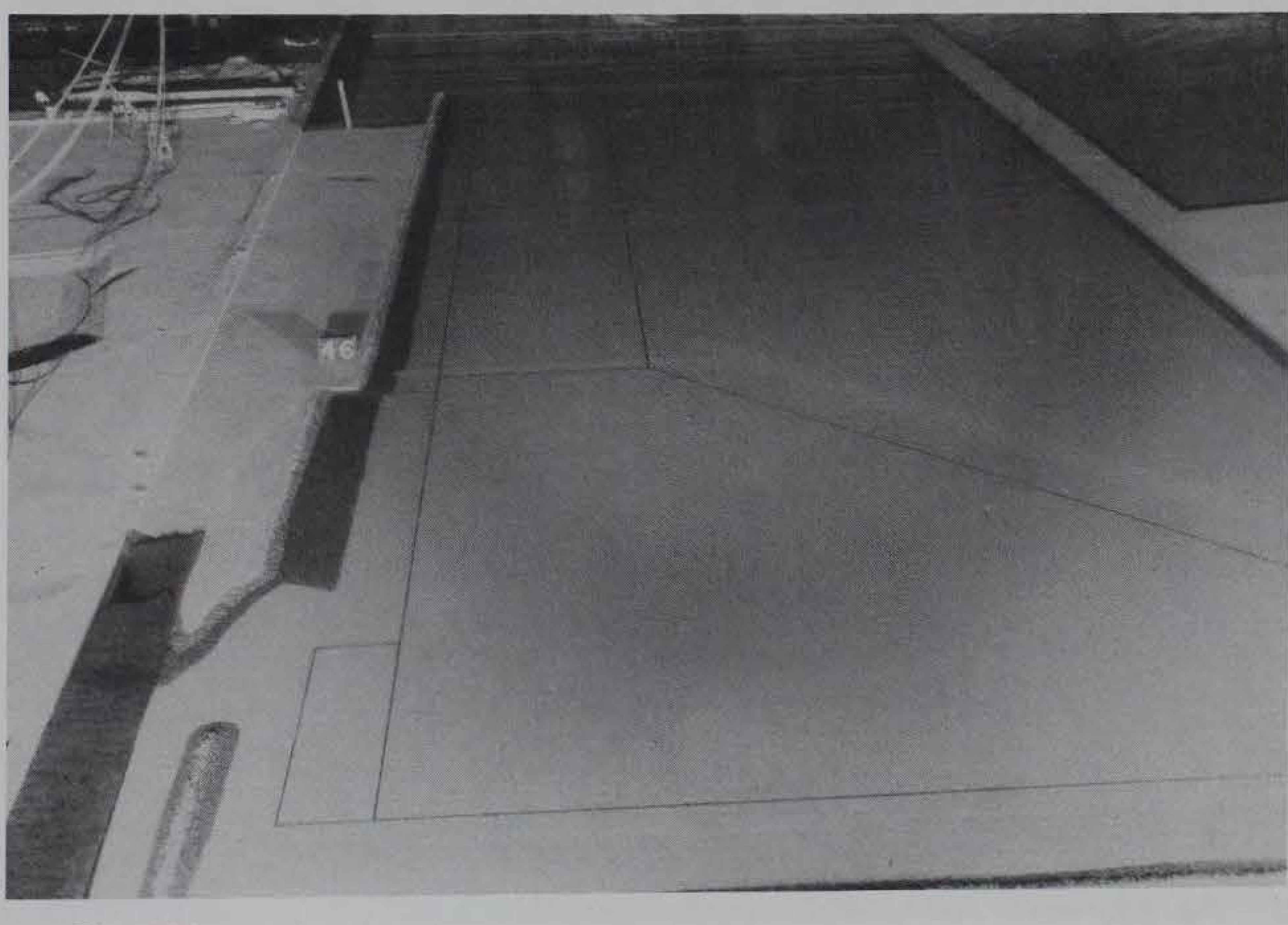
a. Pier 400

b. Pier 300

Photo 10. Typical wave patterns for POLA Stage 1; 16-sec, 20-ft test waves from 185 deg; $\gamma = 5.0$; swl = +8.0 ft

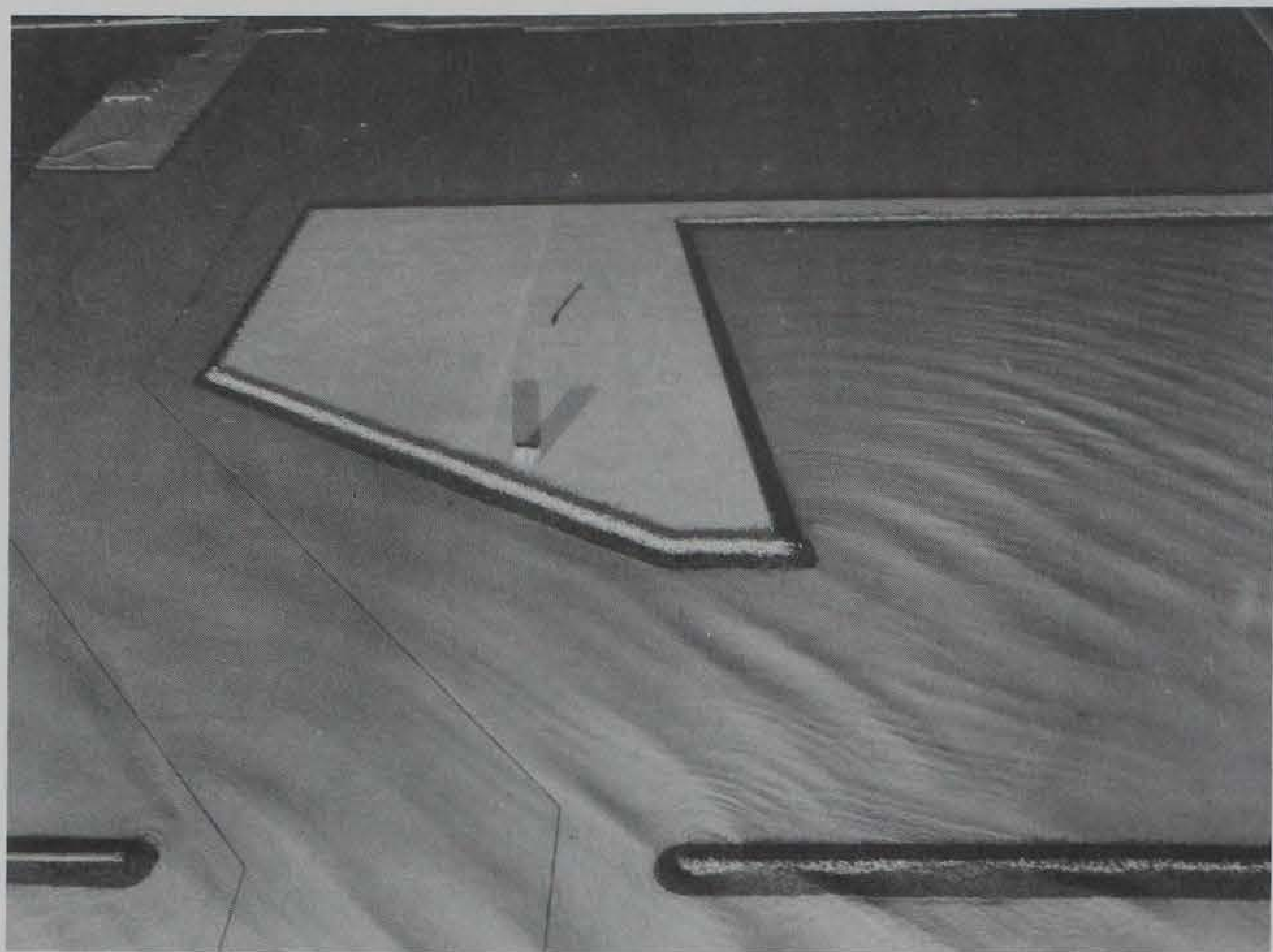


a. Pier 400



b. Pier 300

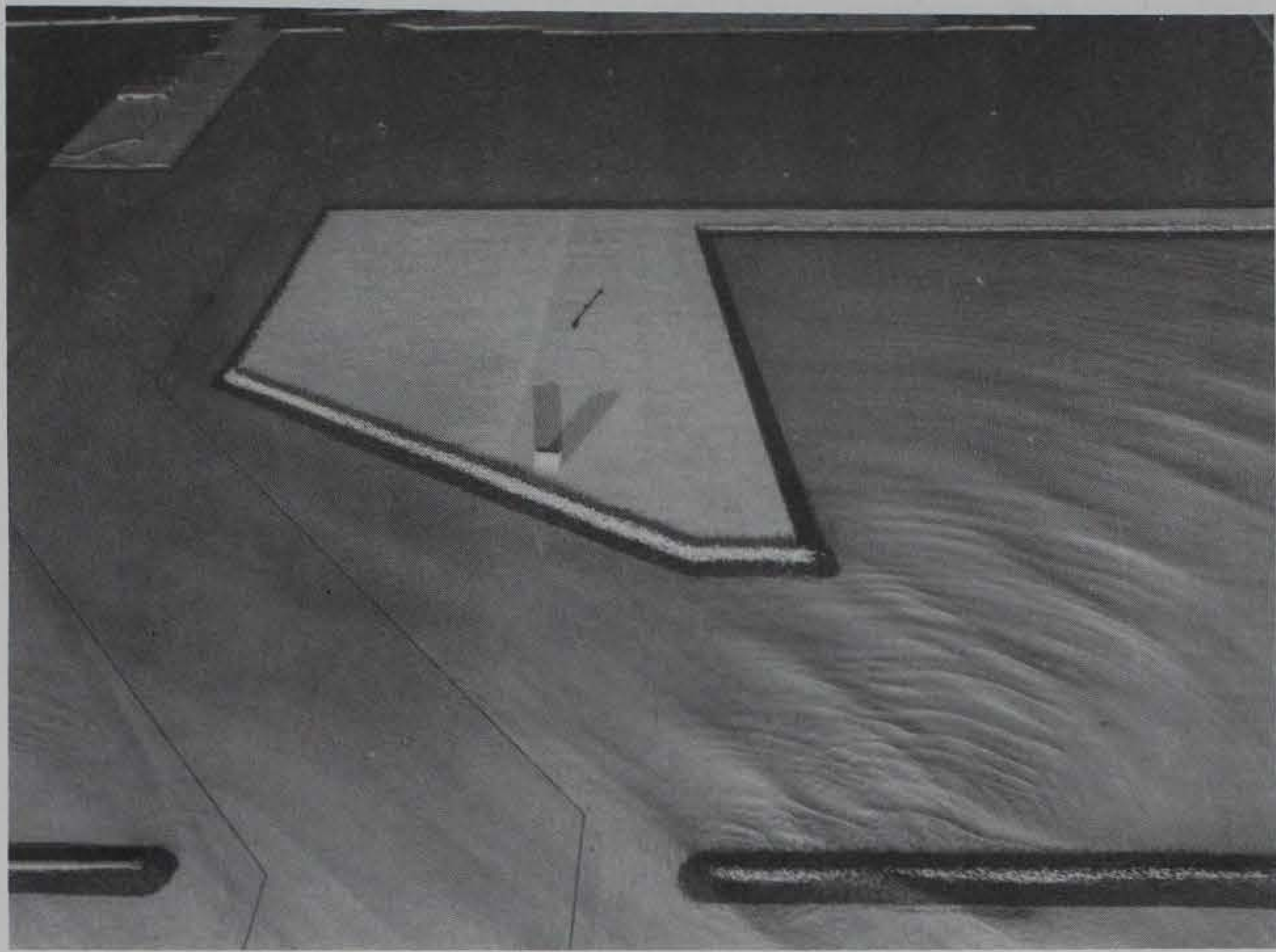
Photo 11. Typical wave patterns for POLA Stage 1; 8-sec, 4-ft test waves from 220 deg; $\gamma = 3.3$; swl = +5.5 ft



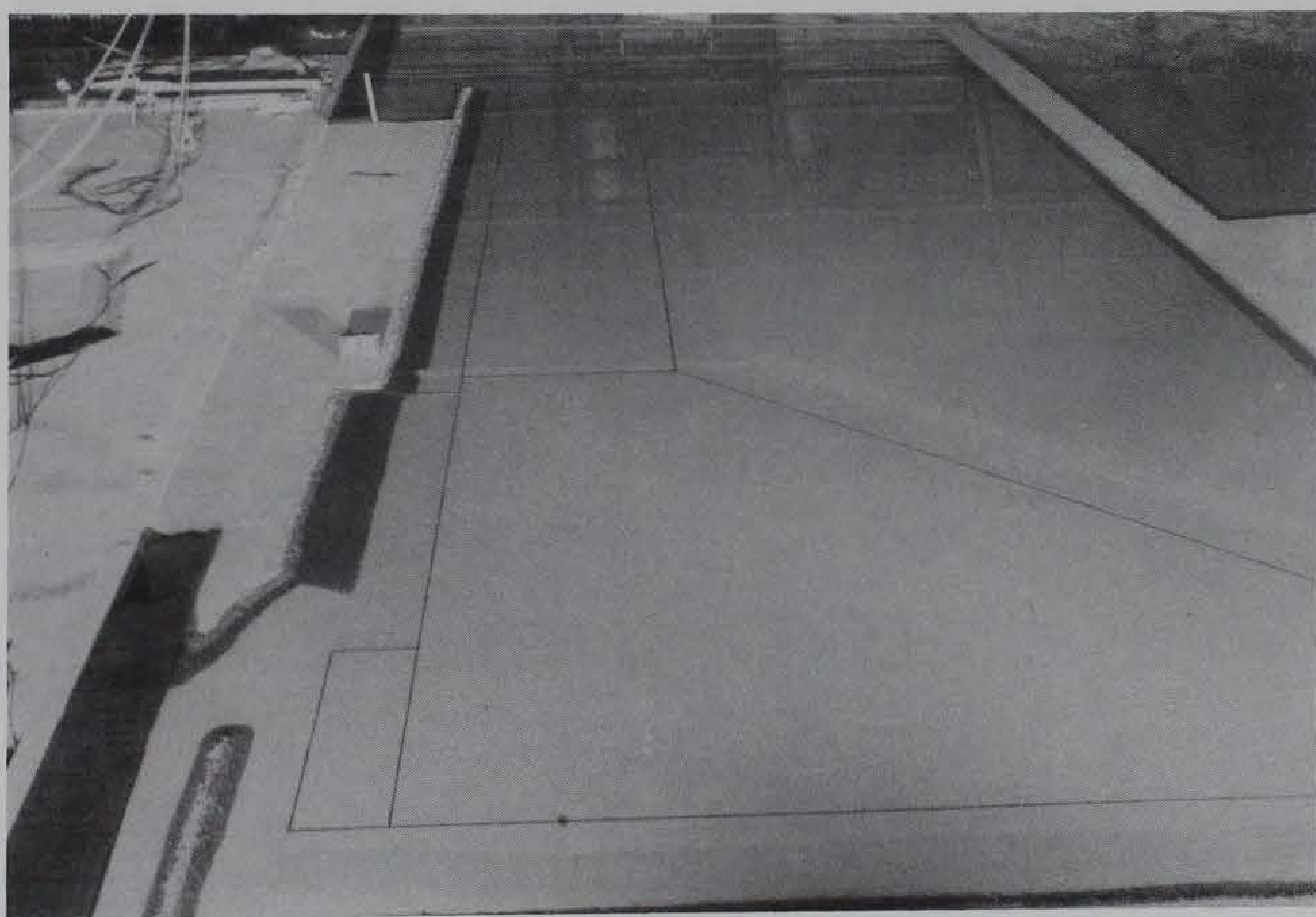
a. Pier 400

b. Pier 300

Photo 12. Typical wave patterns for POLA Stage 1; 12-sec, 8-ft test waves from 220 deg; $\gamma = 5.0$; swl = +5.5 ft

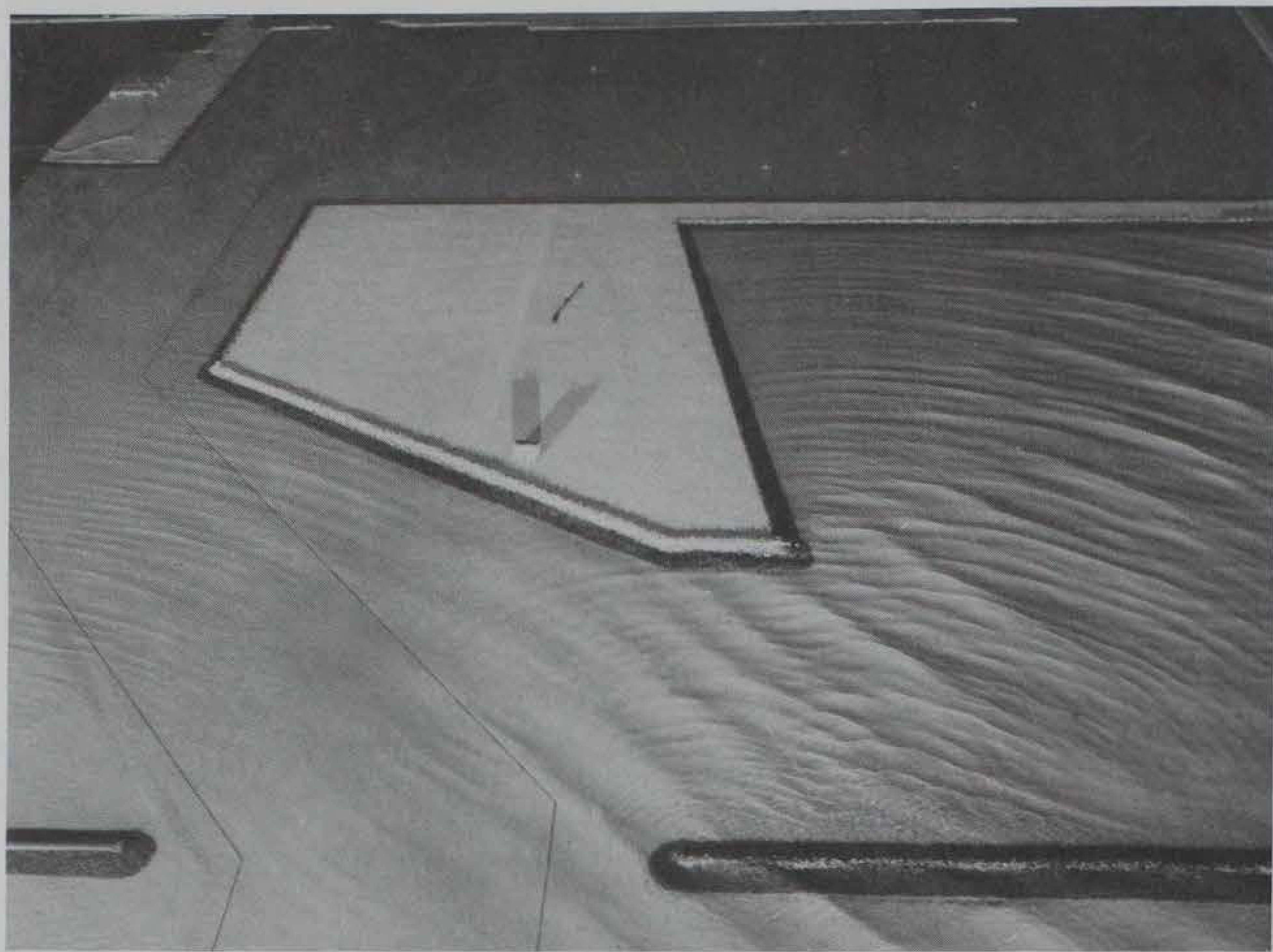


a. Pier 400



b. Pier 300

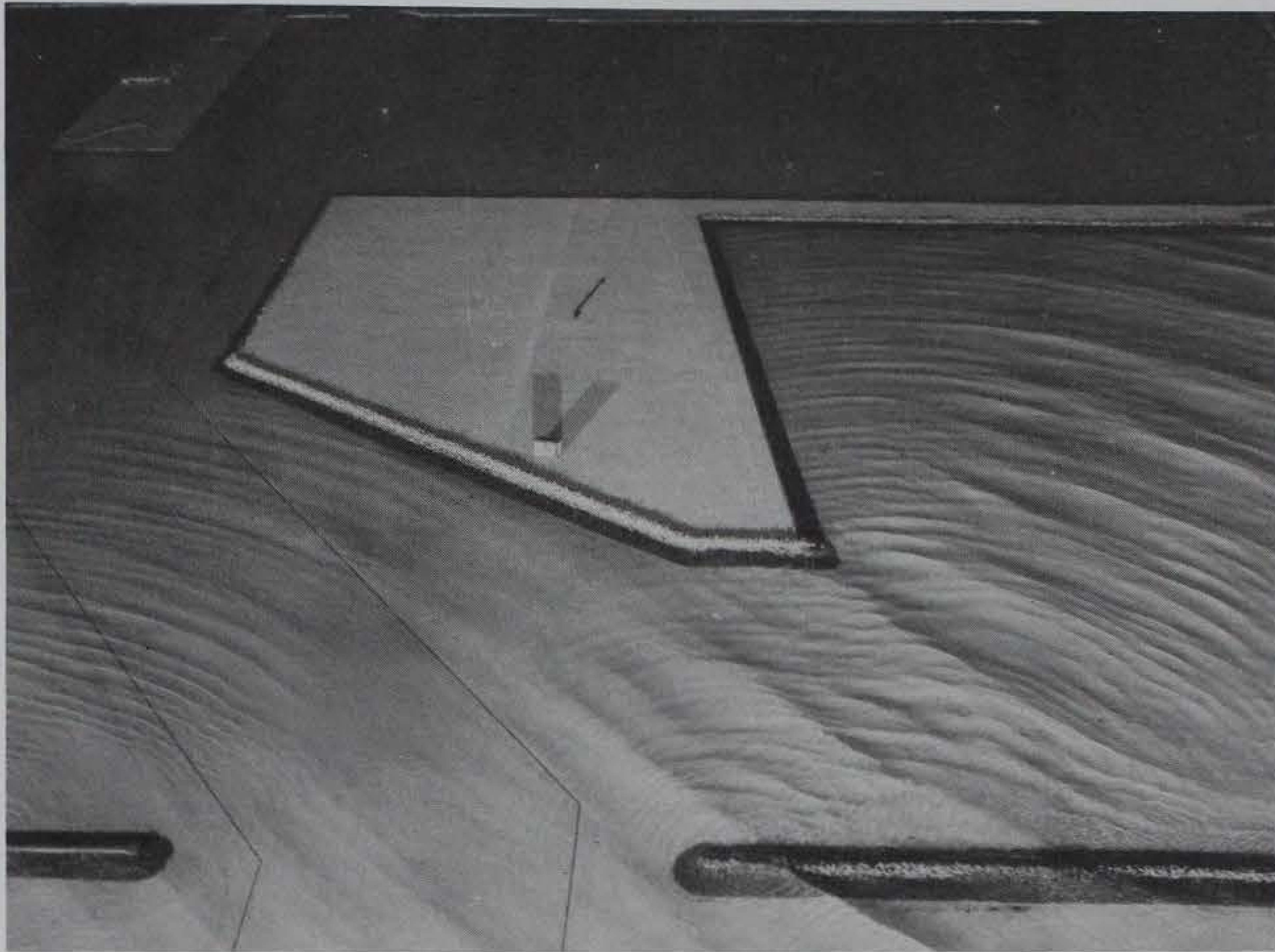
Photo 13. Typical wave patterns for POLA Stage 1; 16-sec, 12-ft test waves from 220 deg; $\gamma = 7.0$; swl = +5.5 ft



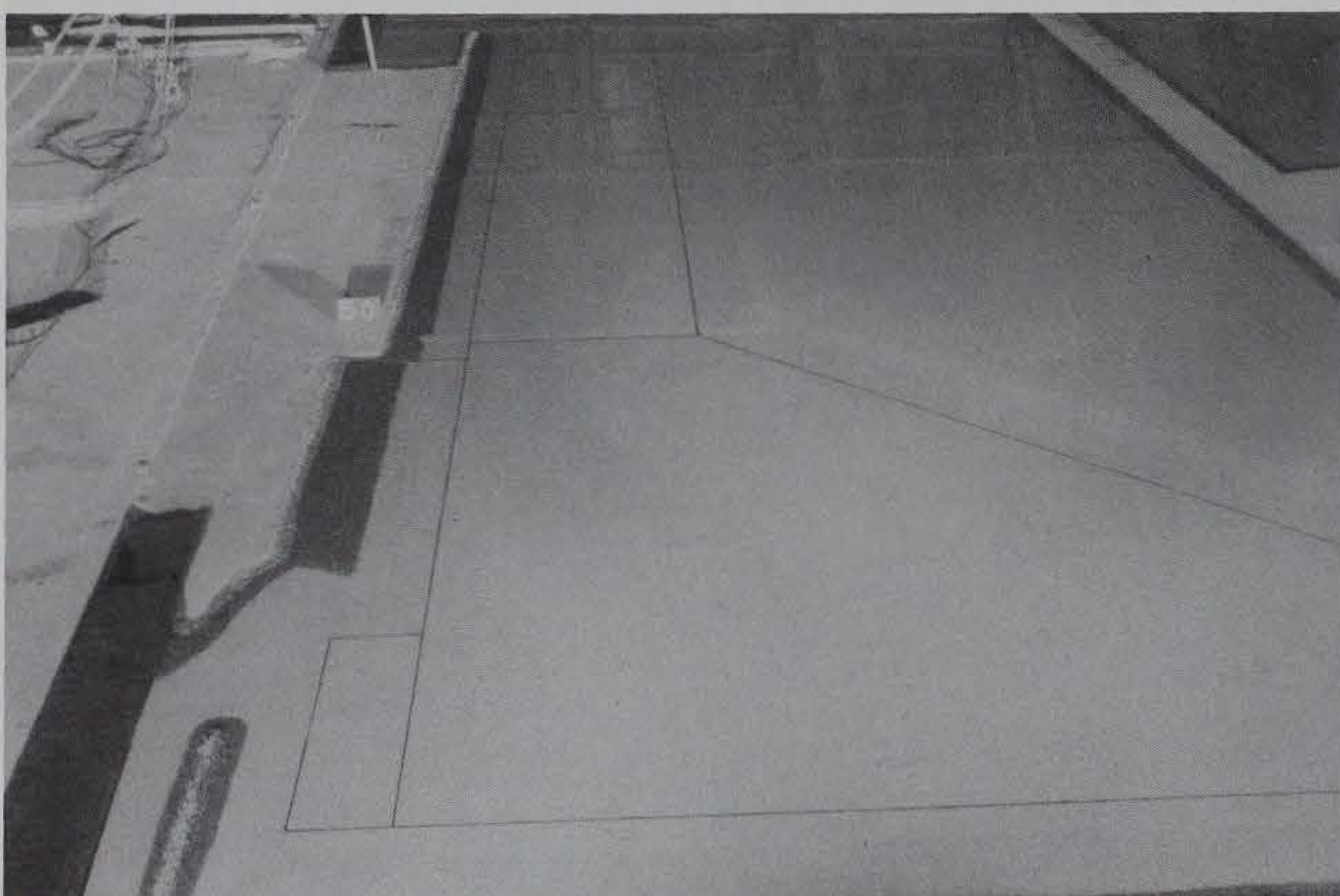
a. Pier 400

b. Pier 300

Photo 14. Typical wave patterns for POLA Stage 1; 14-sec, 14-ft test waves from 220 deg; $\gamma = 7.0$; swl = +8.0 ft

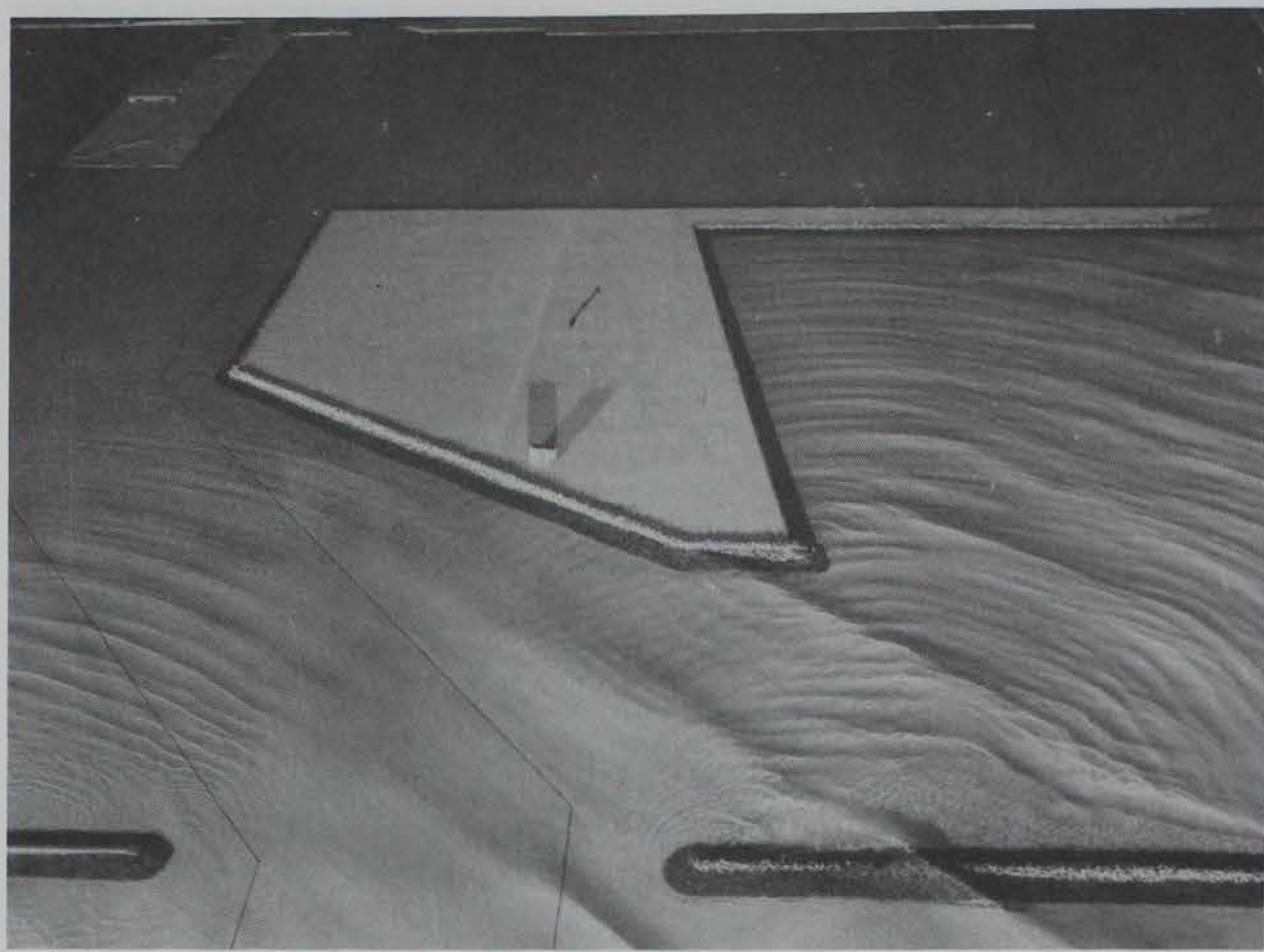


a. Pier 400

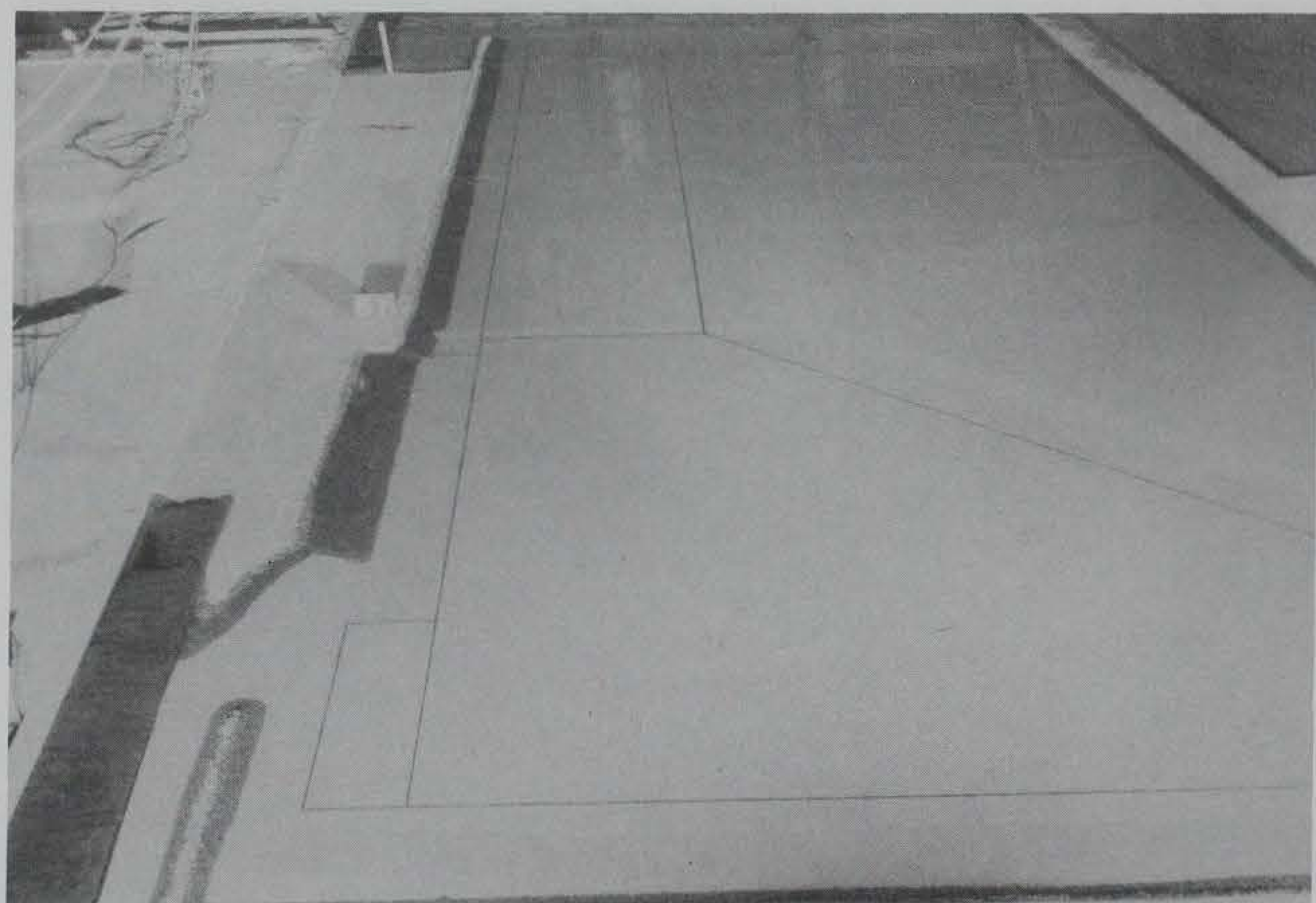


b. Pier 300

Photo 15. Typical wave patterns for POLA Stage 1; 16-sec, 16-ft test waves from 220 deg; $\gamma = 7.0$; swl = +8.0 ft

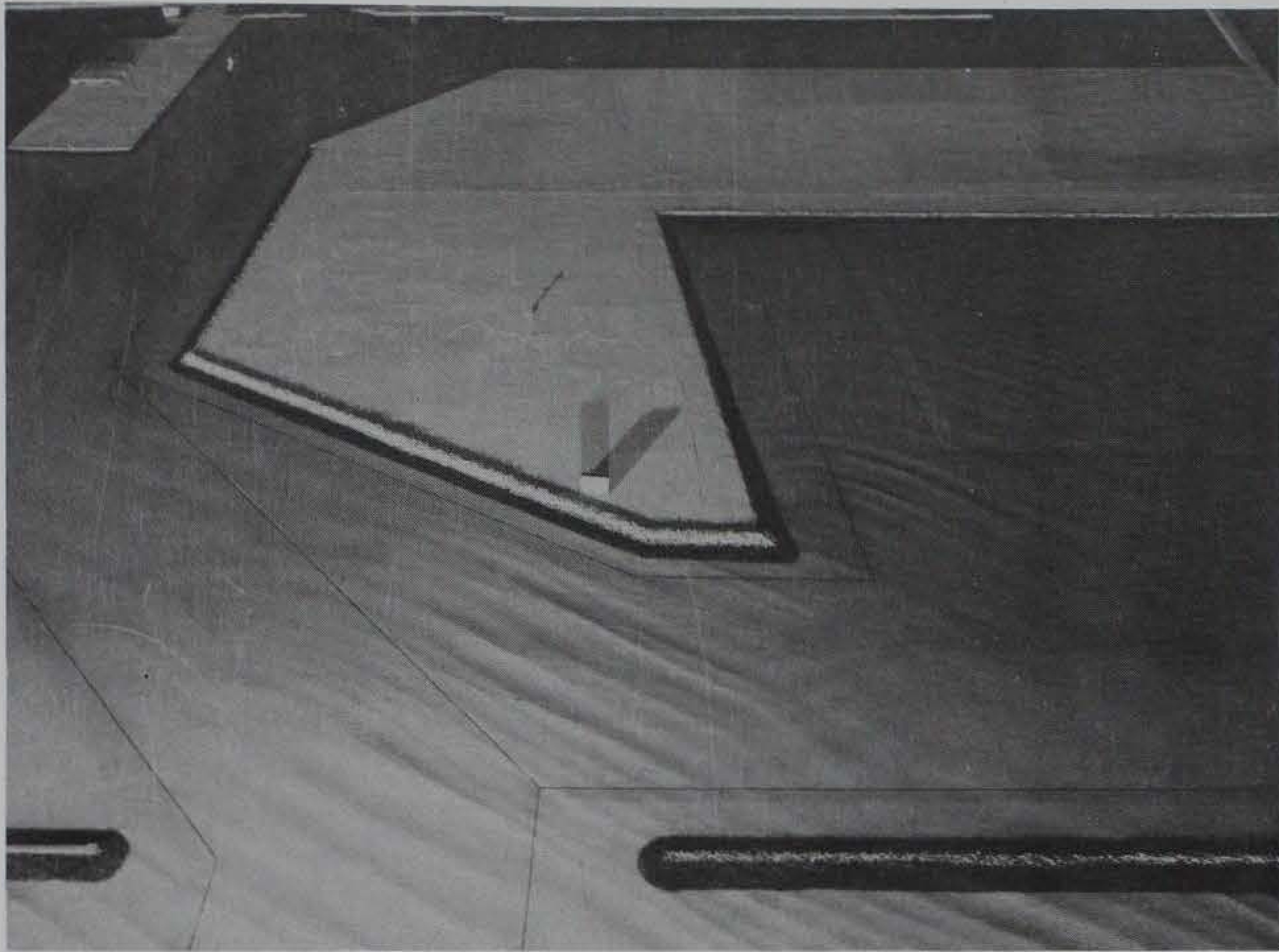


a. Pier 400

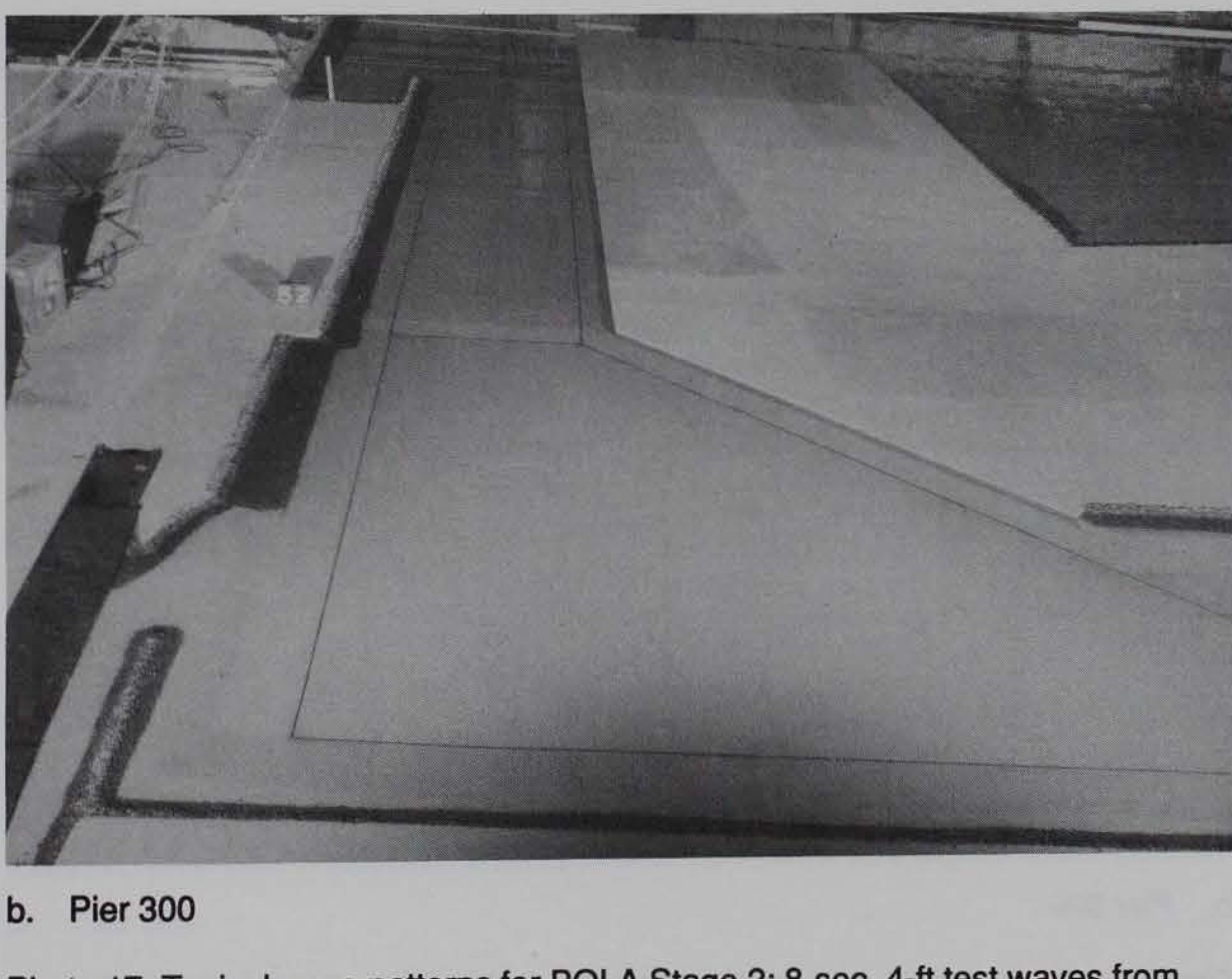


b. Pier 300

Photo 16. Typical wave patterns for POLA Stage 1; 20-sec, 20-ft test waves from 220 deg; $\gamma = 7.0$; swl = +8.0 ft

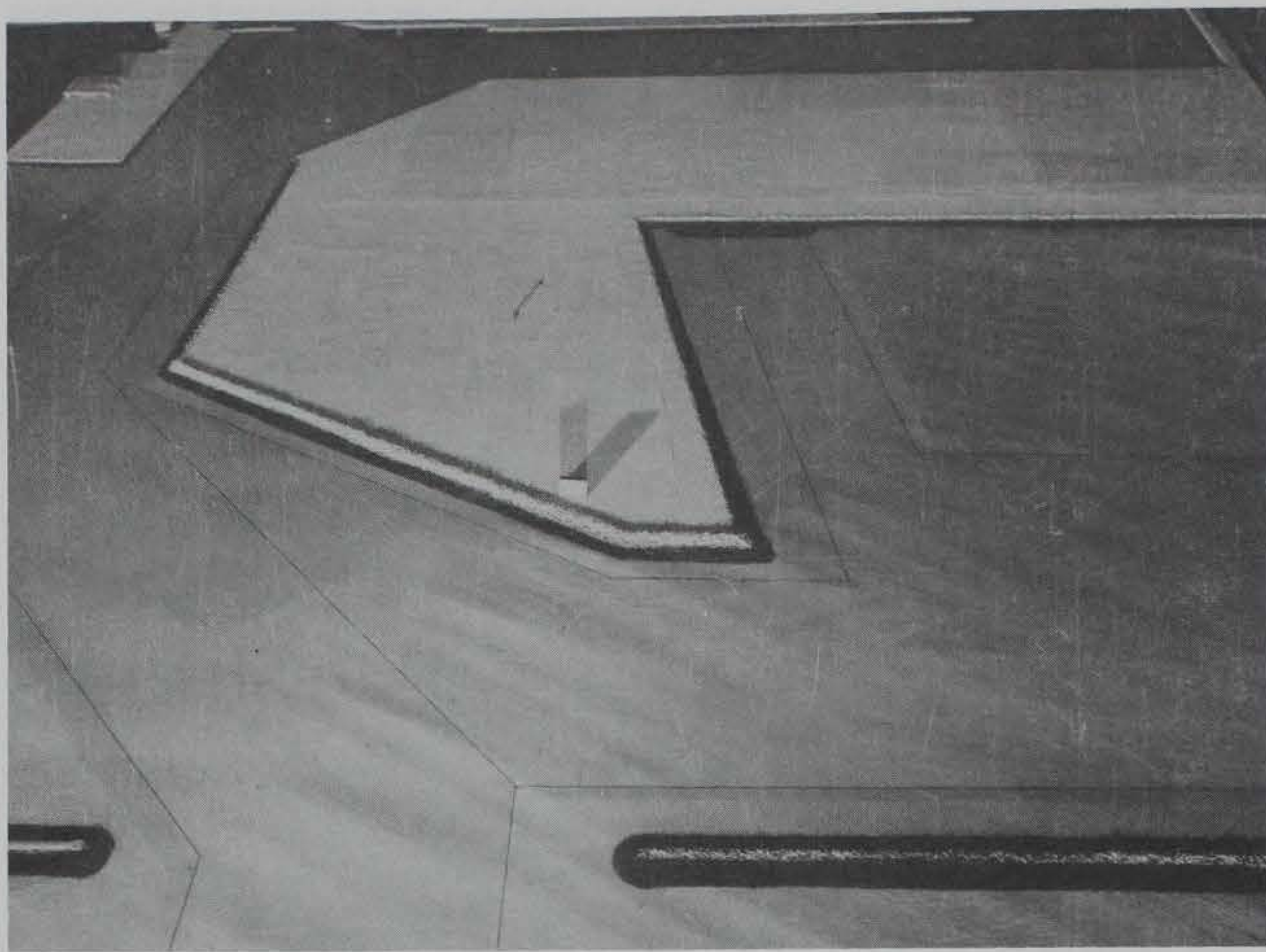


a. Pier 400

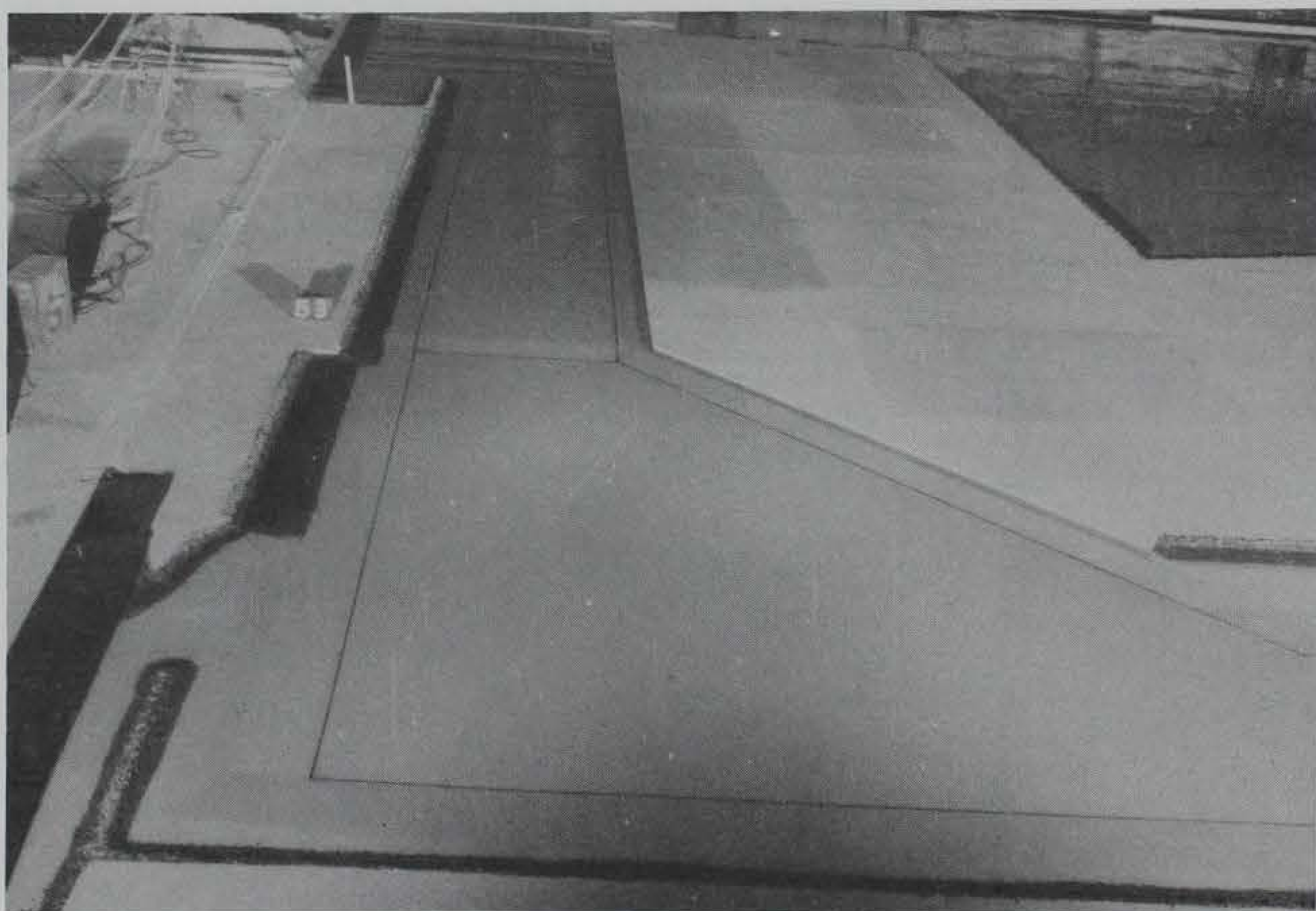


b. Pier 300

Photo 17. Typical wave patterns for POLA Stage 2; 8-sec, 4-ft test waves from 185 deg; $\gamma = 3.3$; swl = +5.5 ft

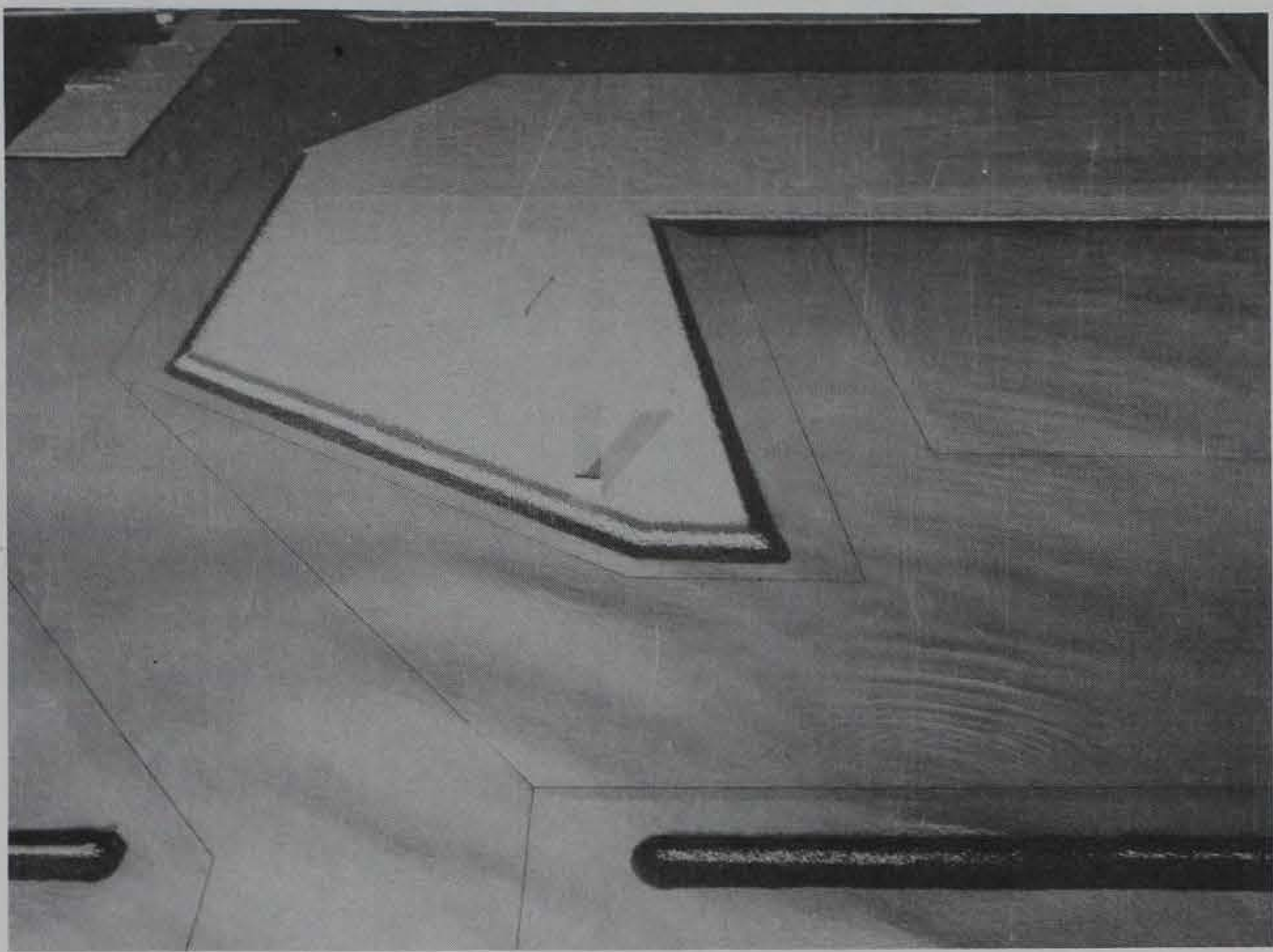


a. Pier 400

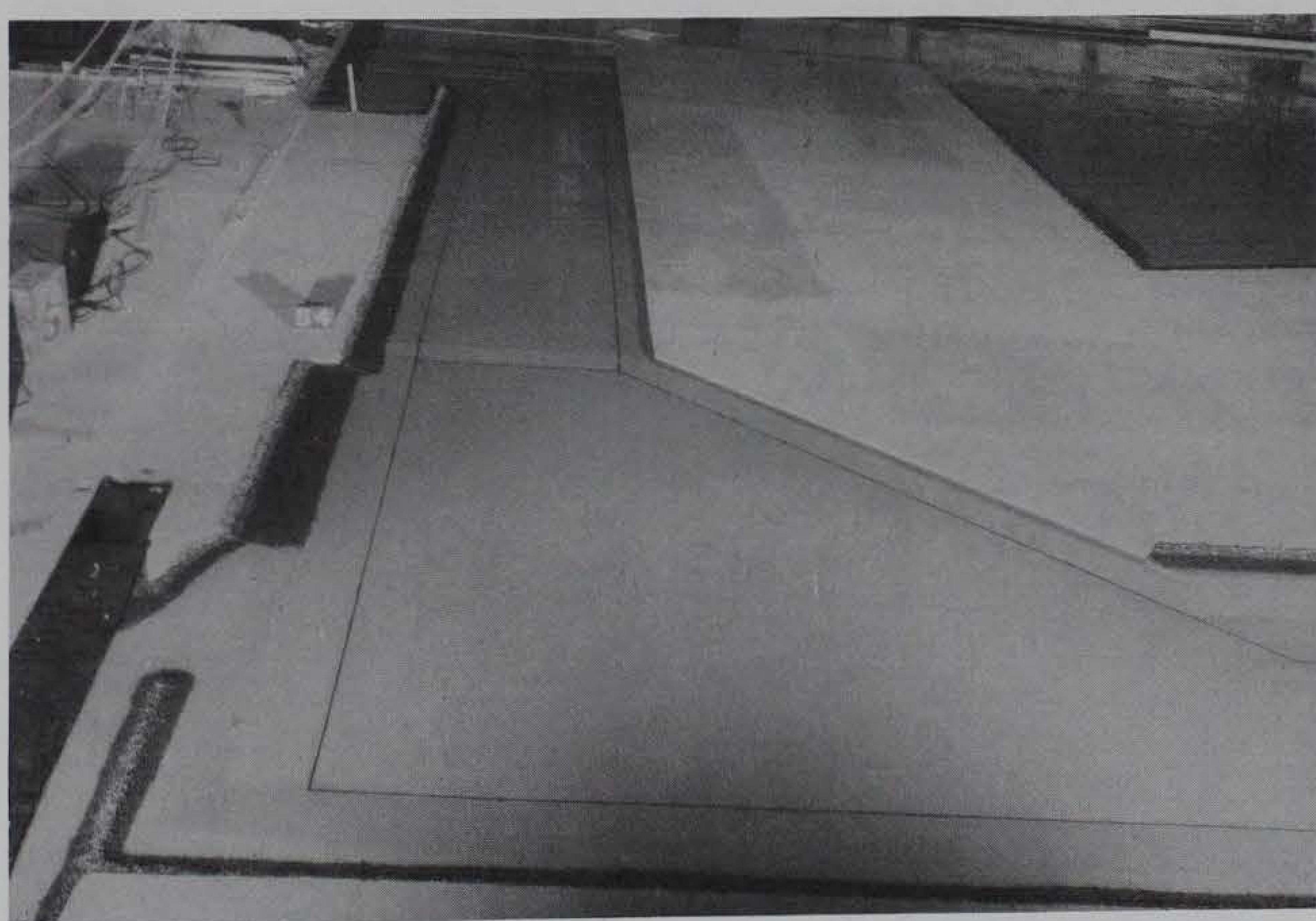


b. Pier 300

Photo 18. Typical wave patterns for POLA Stage 2; 12-sec, 6-ft test waves from 185 deg; $\gamma = 5.0$; swl = +5.5 ft

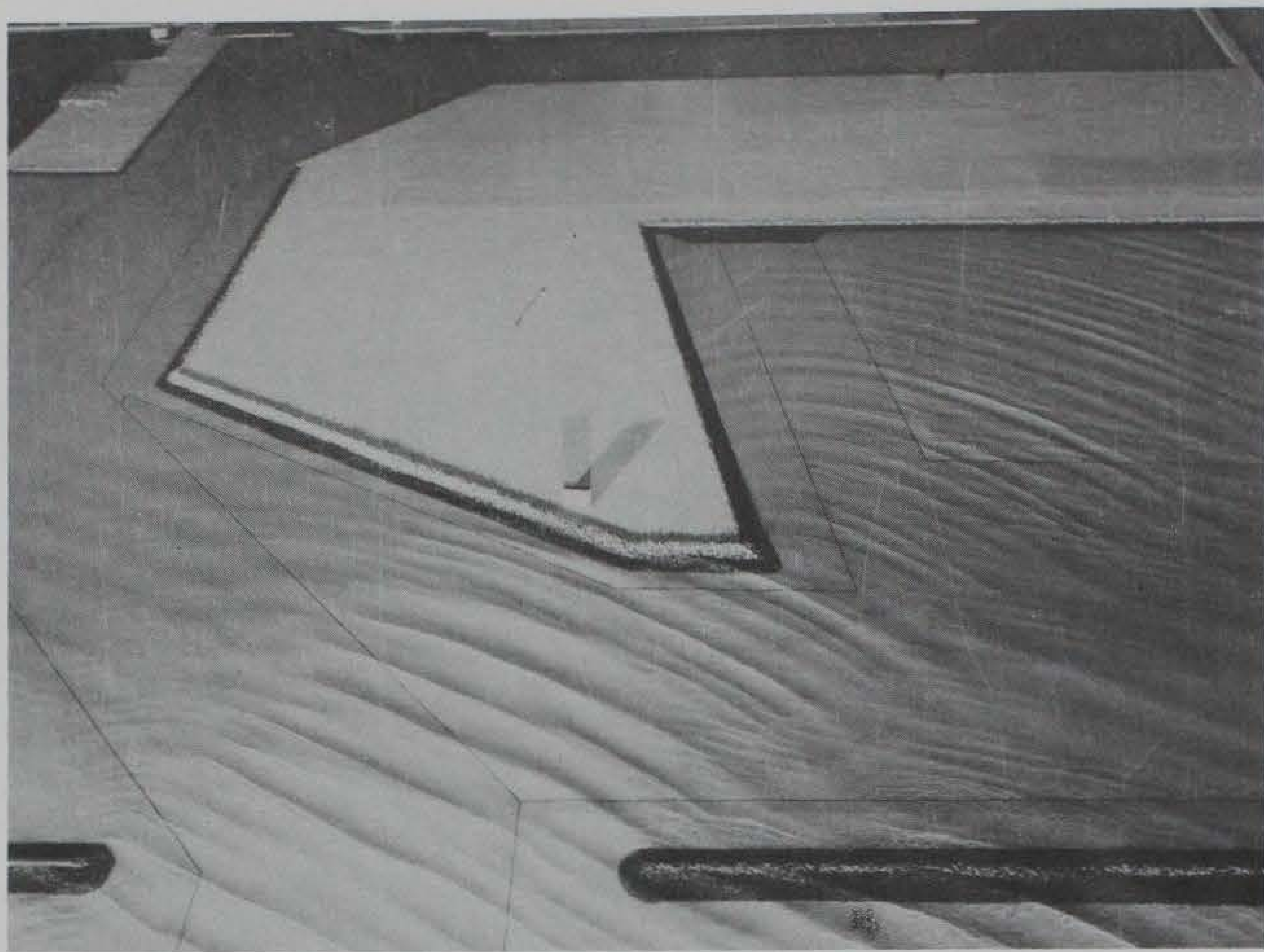


a. Pier 400



b. Pier 300

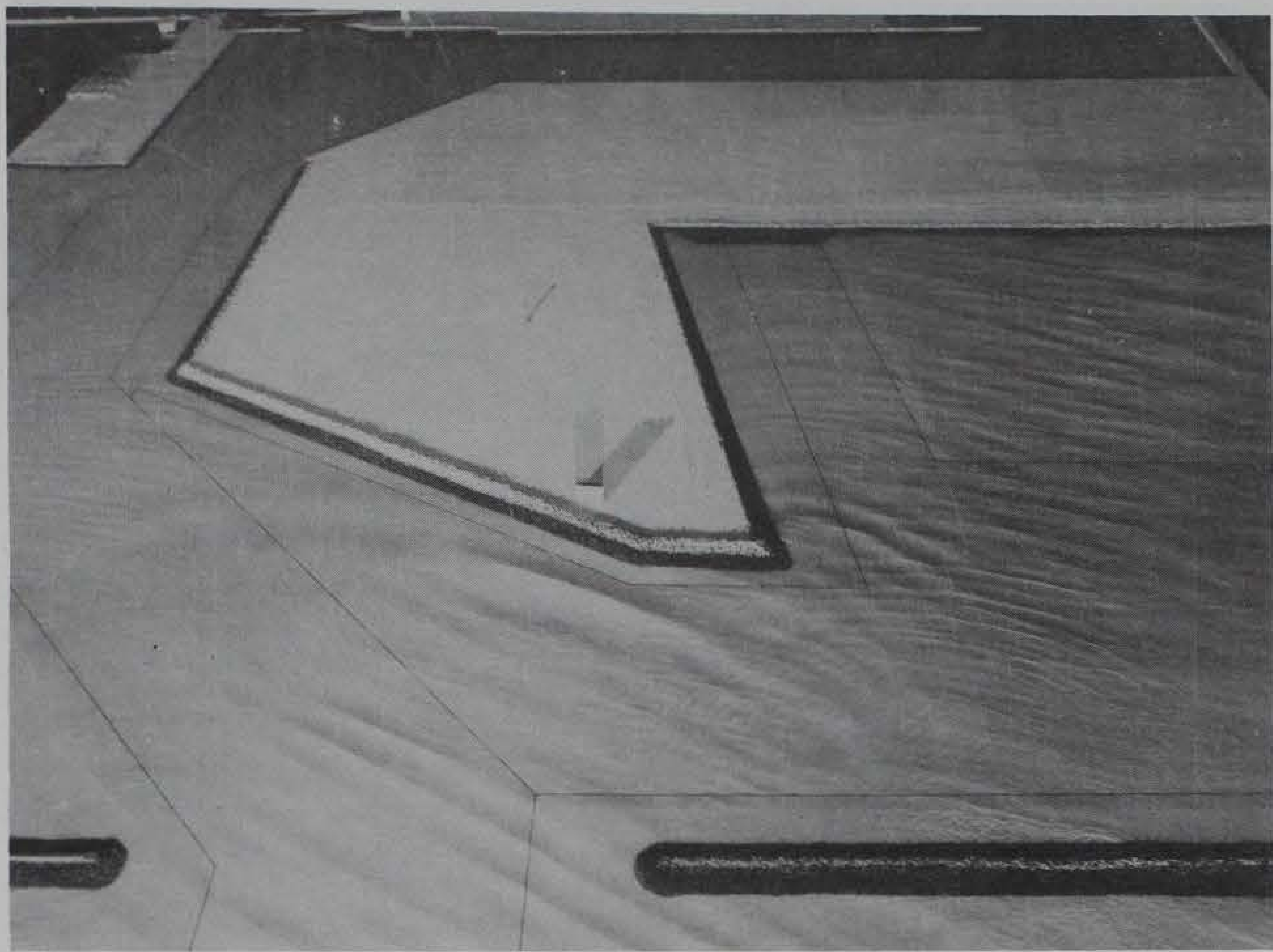
Photo 19. Typical wave patterns for POLA Stage 2; 16-sec, 8-ft test waves from 185 deg; $\gamma = 7.0$; swl = +5.5 ft



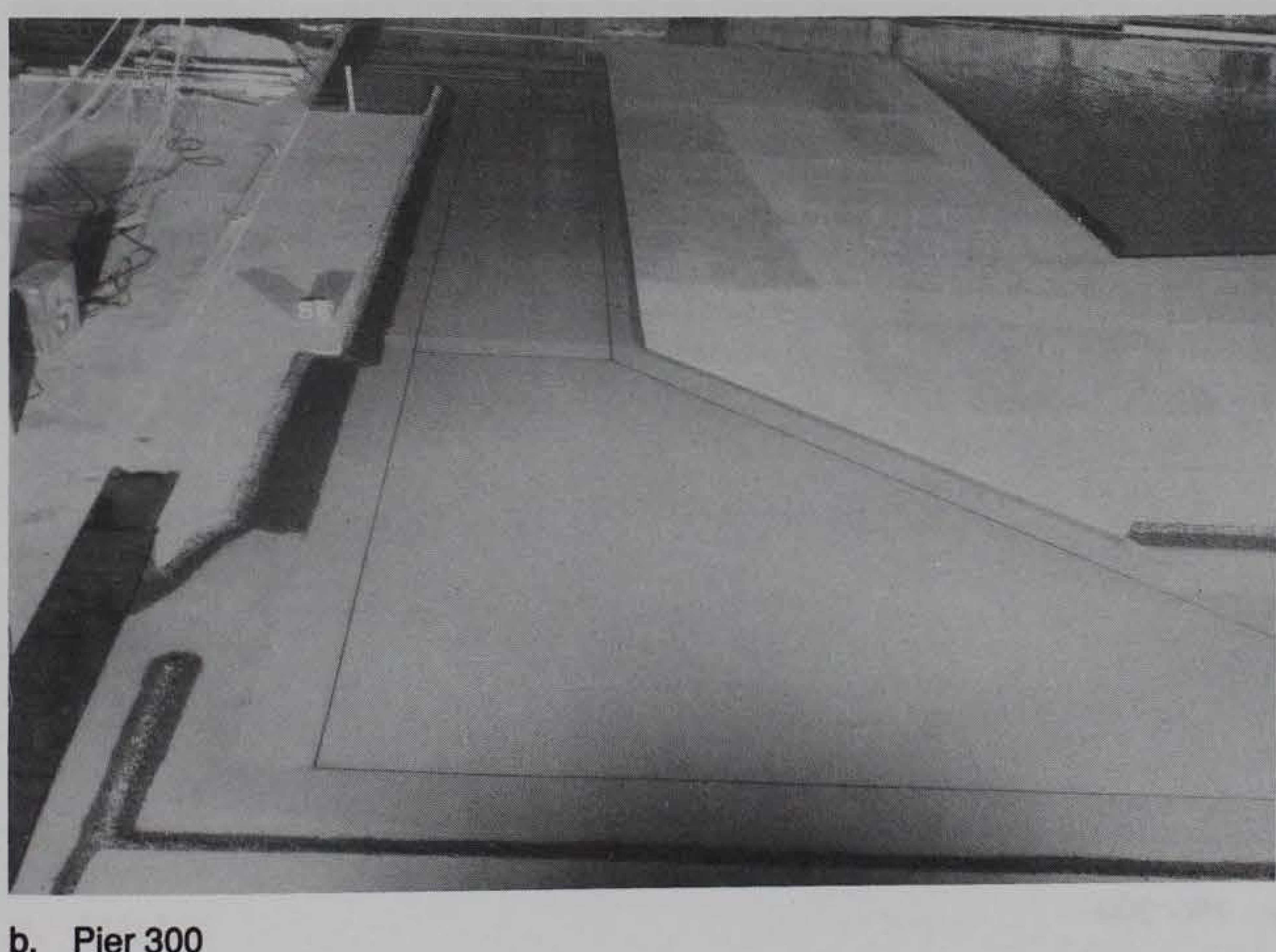
a. Pier 400

b. Pier 300

Photo 20. Typical wave patterns for POLA Stage 2; 8-sec, 10-ft test waves from 185 deg; $\gamma = 3.3$; swl = +8.0 ft

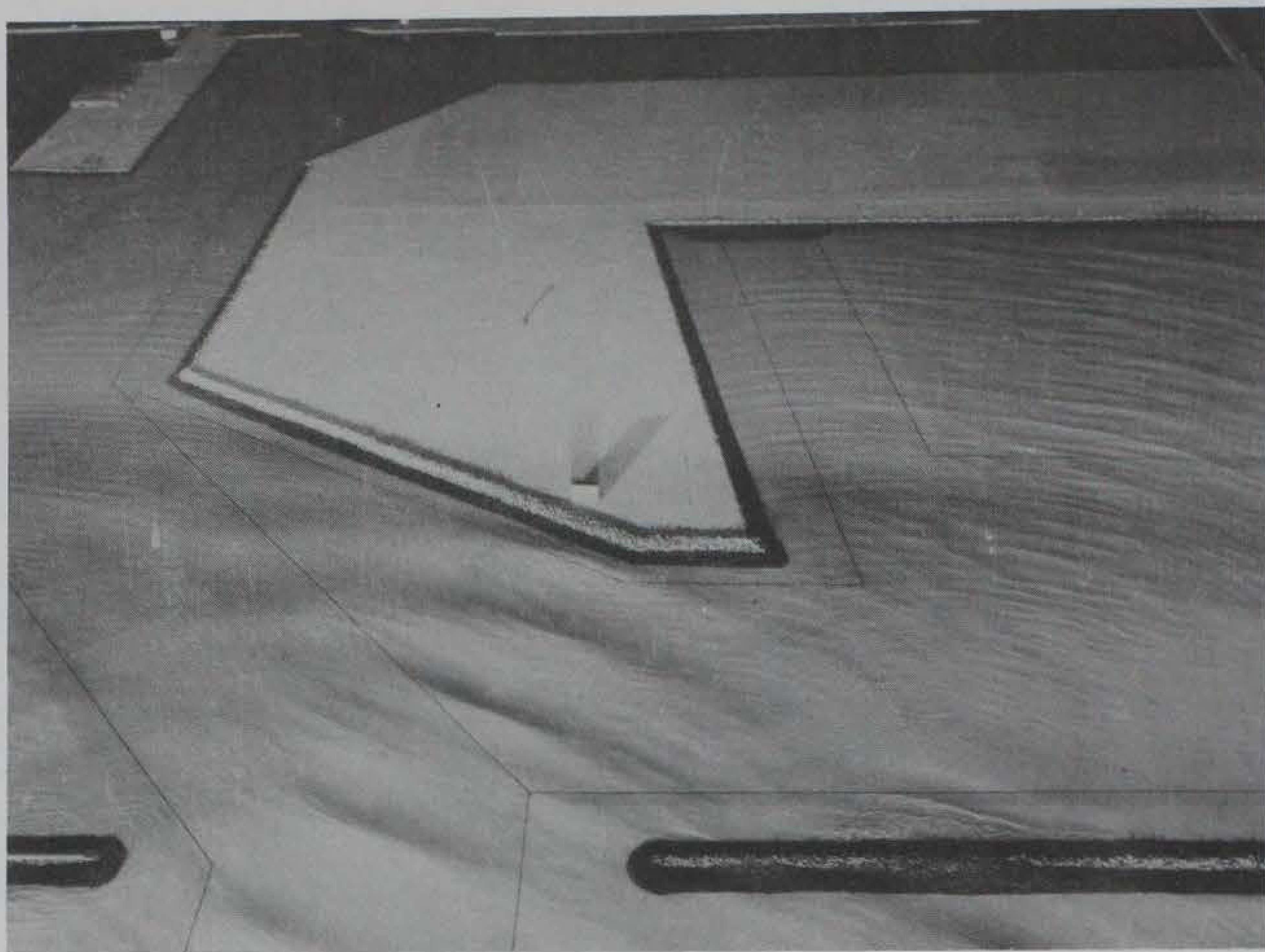


a. Pier 400

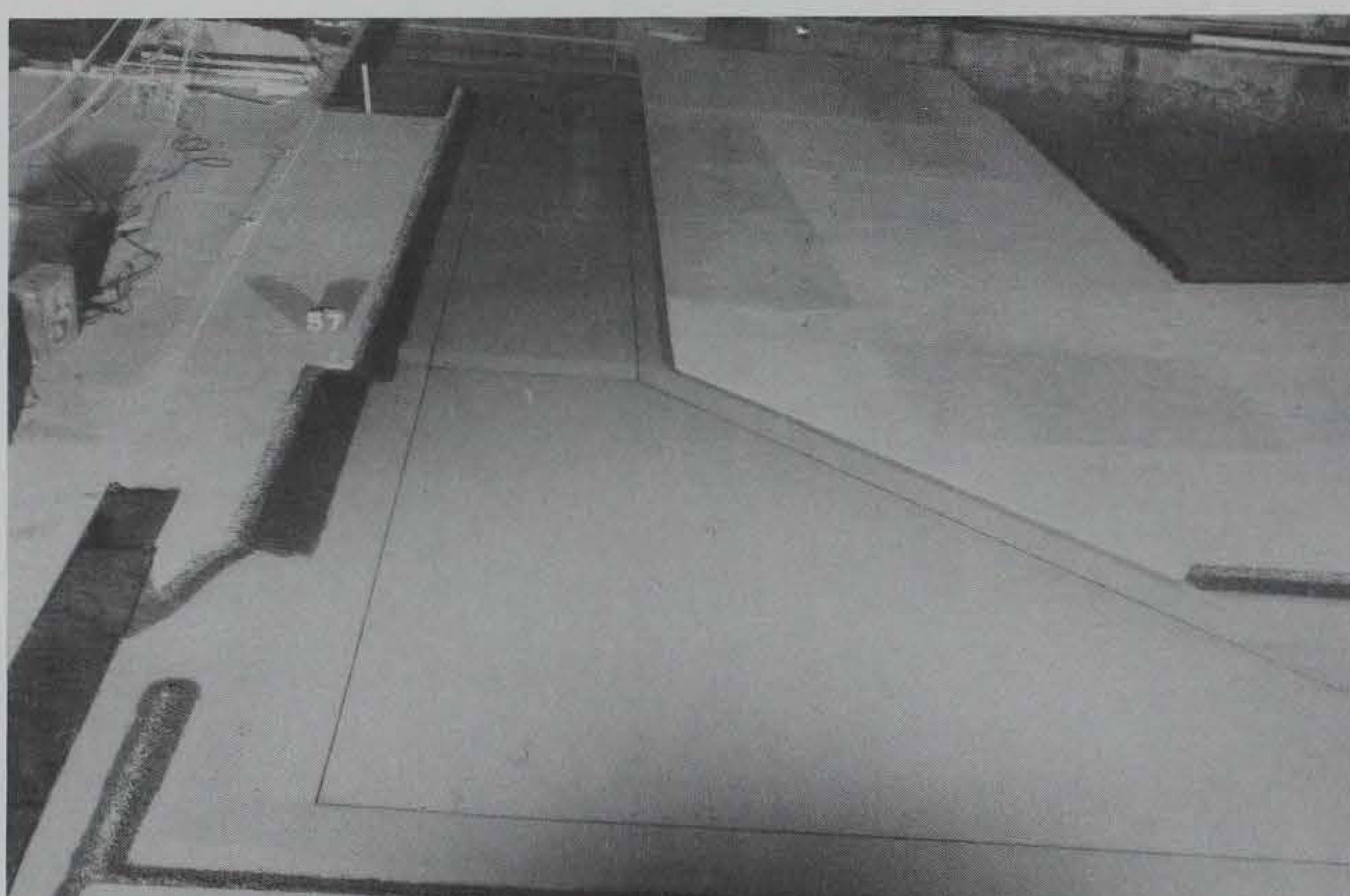


b. Pier 300

Photo 21. Typical wave patterns for POLA Stage 2; 12-sec, 12-ft test waves from 185 deg; $\gamma = 3.3$; swl = +8.0 ft

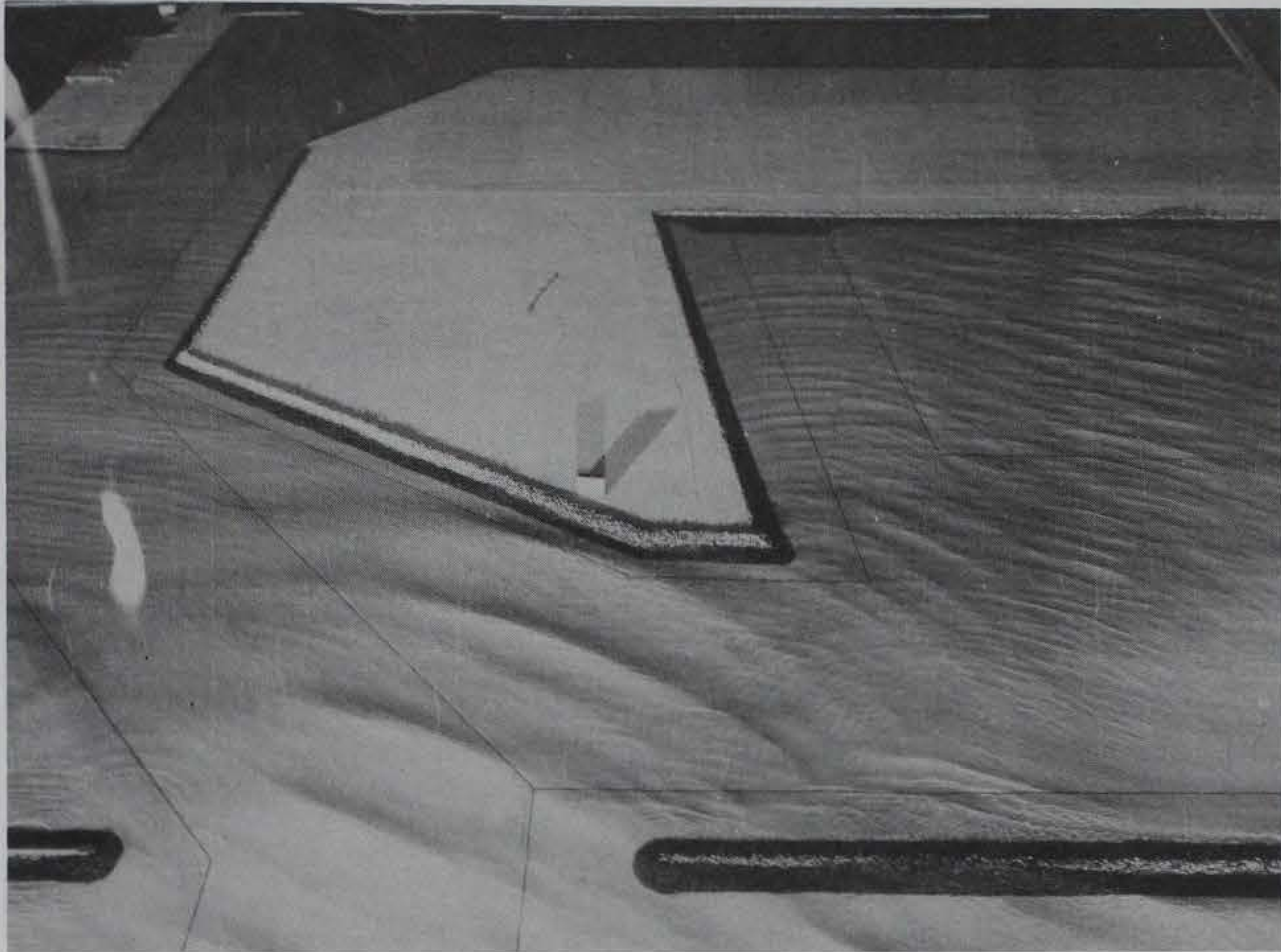


a. Pier 400

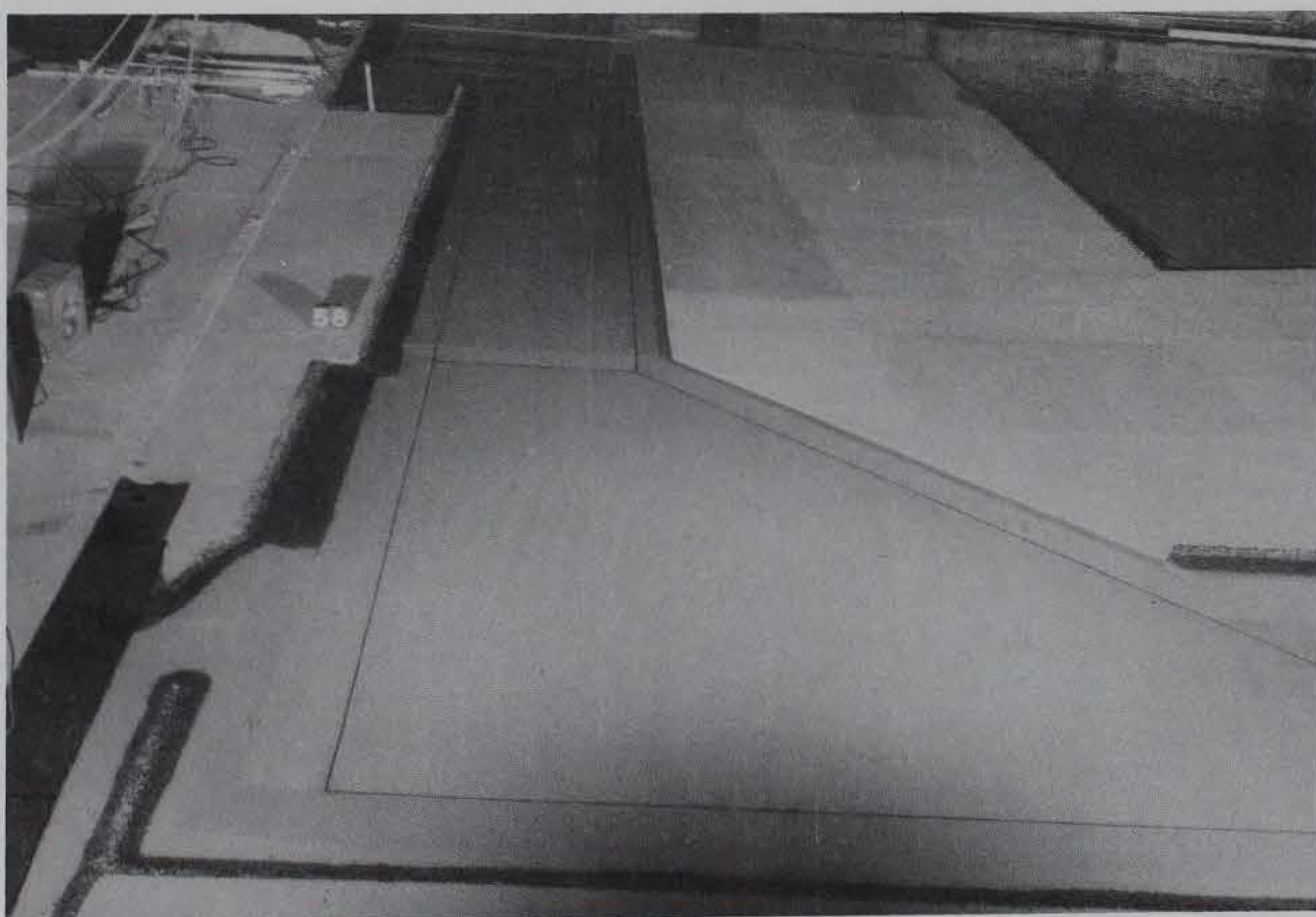


b. Pier 300

Photo 22. Typical wave patterns for POLA Stage 2; 14-sec, 14-ft test waves from 185 deg; $\gamma = 5.0$; swl = +8.0 ft

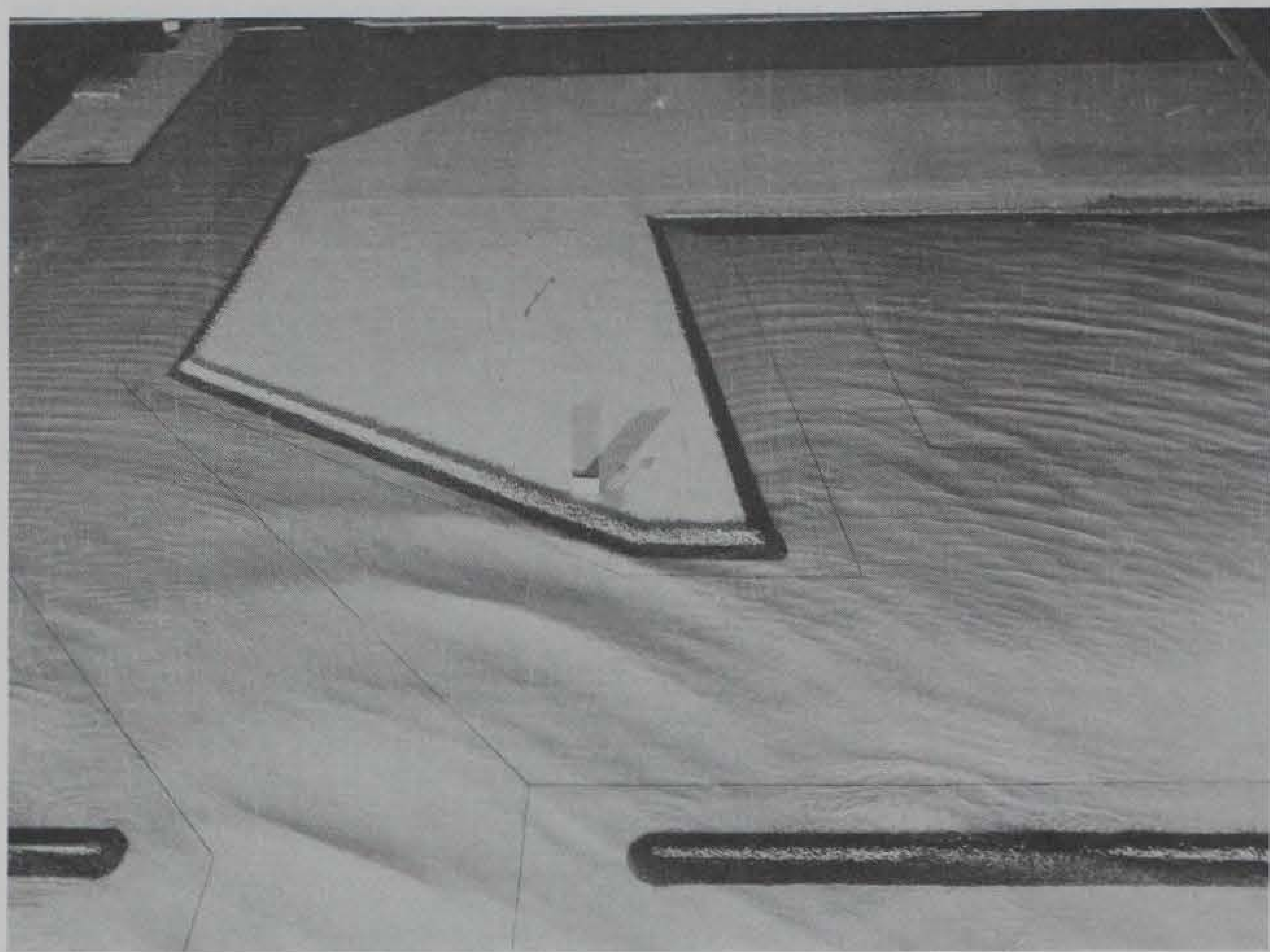


a. Pier 400

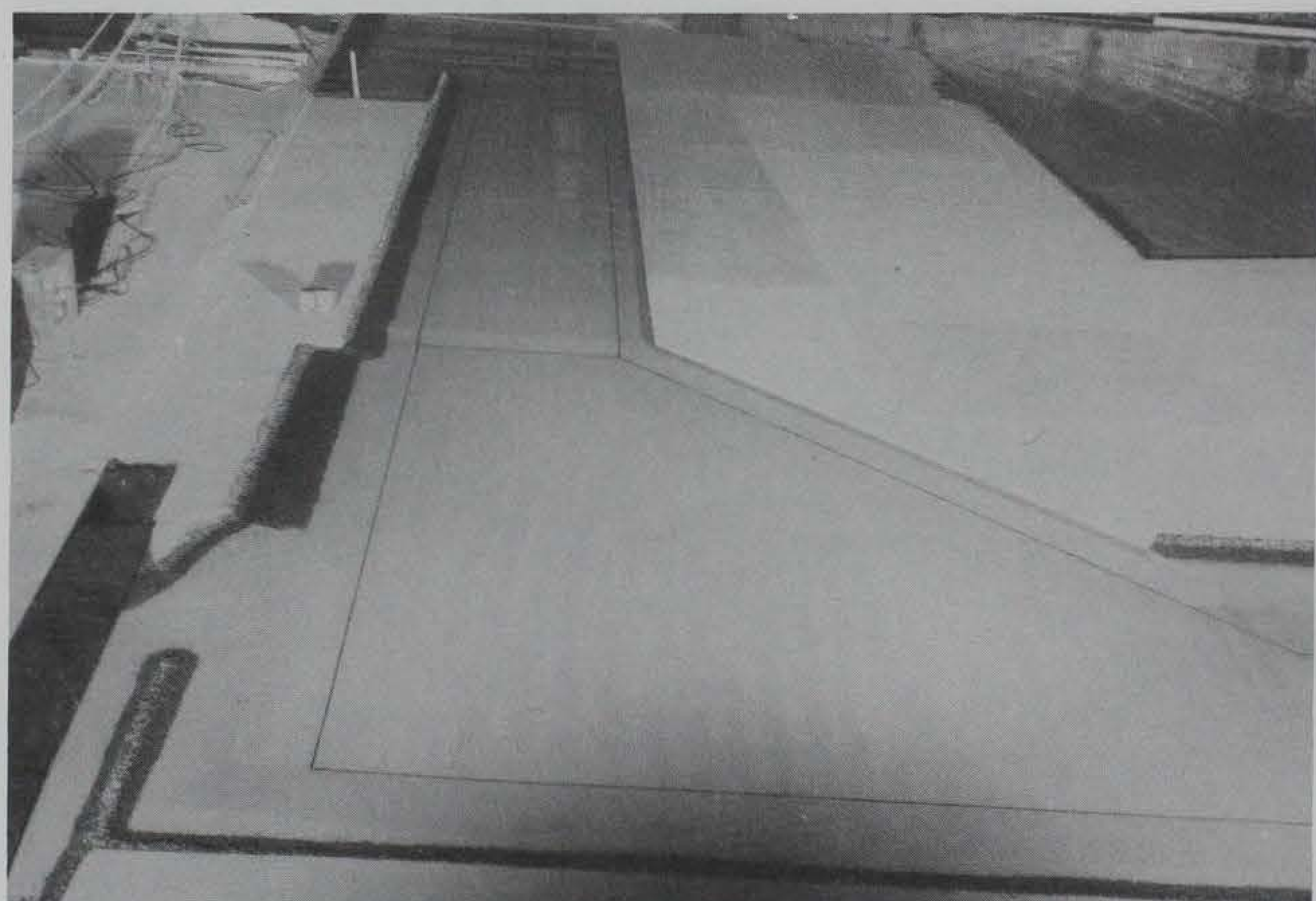


b. Pier 300

Photo 23. Typical wave patterns for POLA Stage 2; 14-sec, 16-ft test waves from 185 deg; $\gamma = 3.3$; swl = +8.0 ft

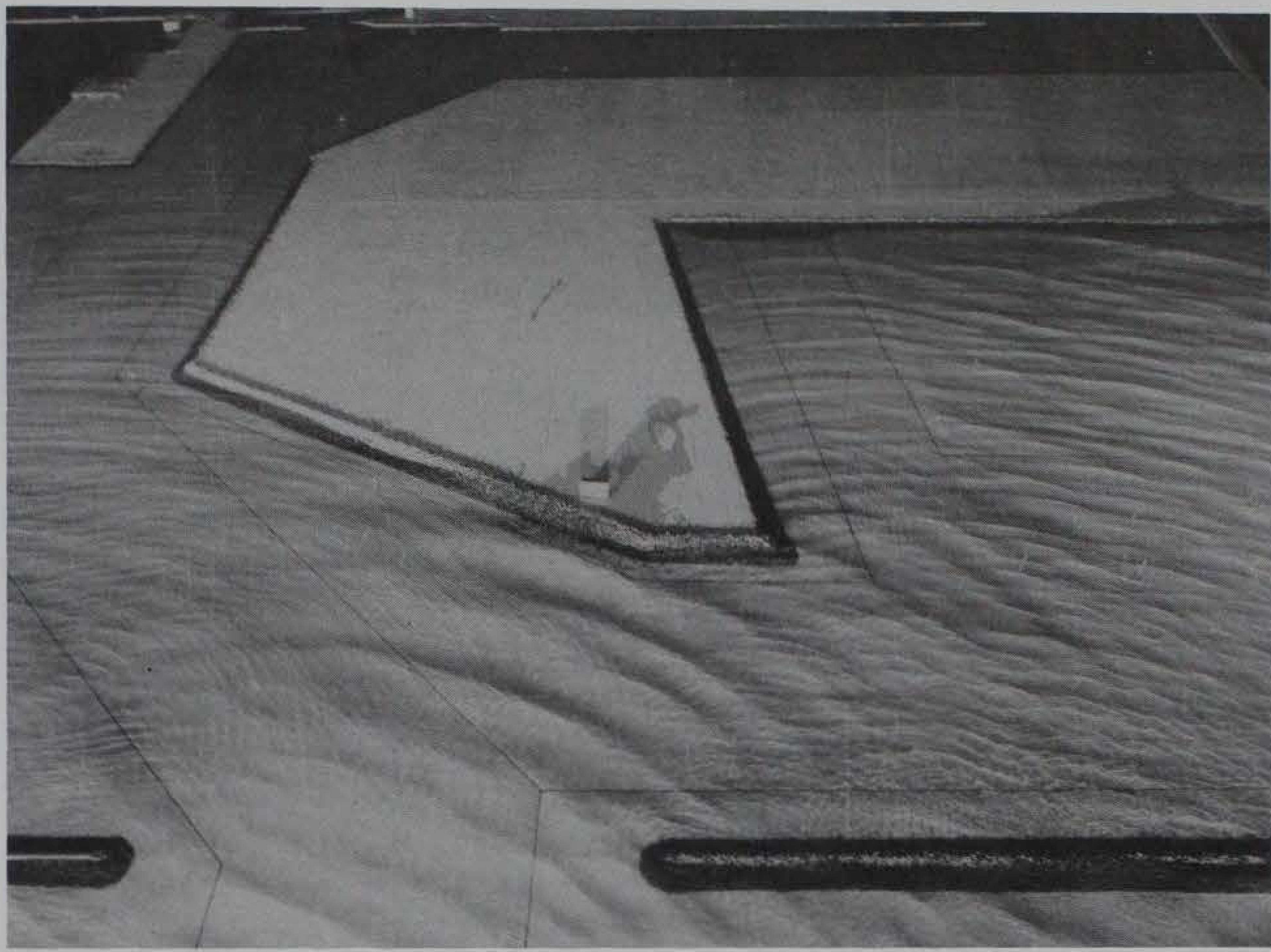


a. Pier 400

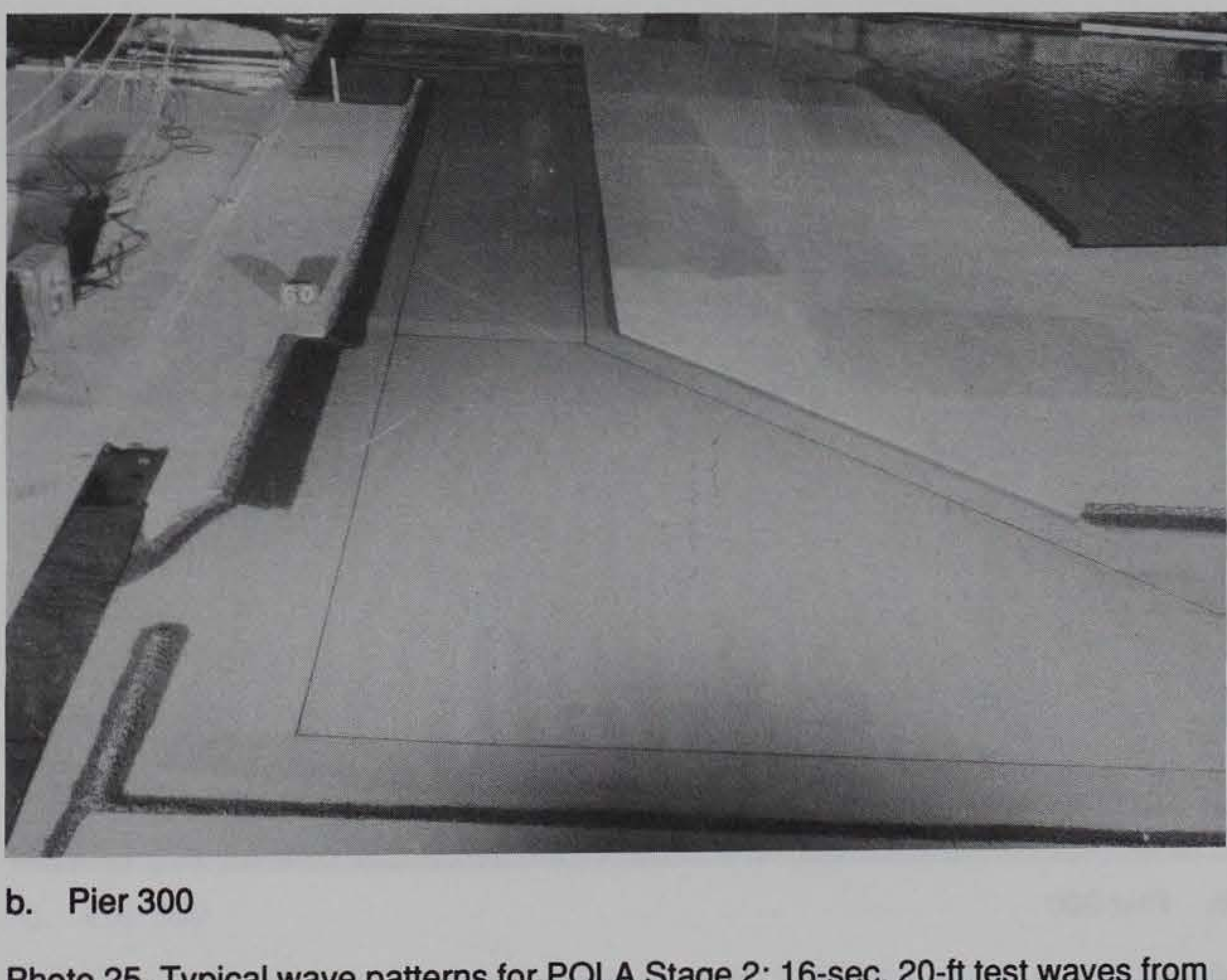


b. Pier 300

Photo 24. Typical wave patterns for POLA Stage 2; 16-sec, 18-ft test waves from 185 deg; $\gamma = 5.0$; swl = +8.0 ft

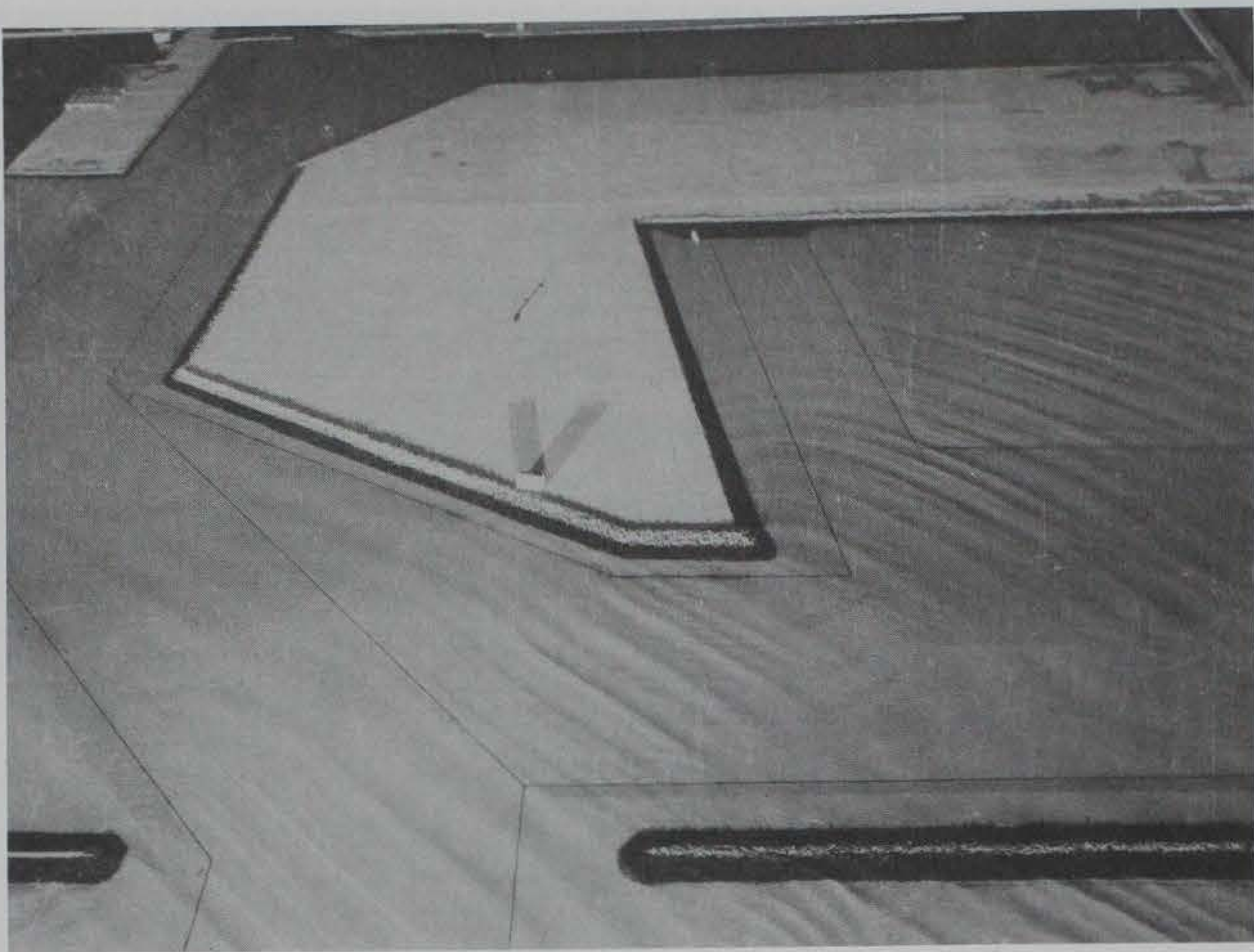


a. Pier 400

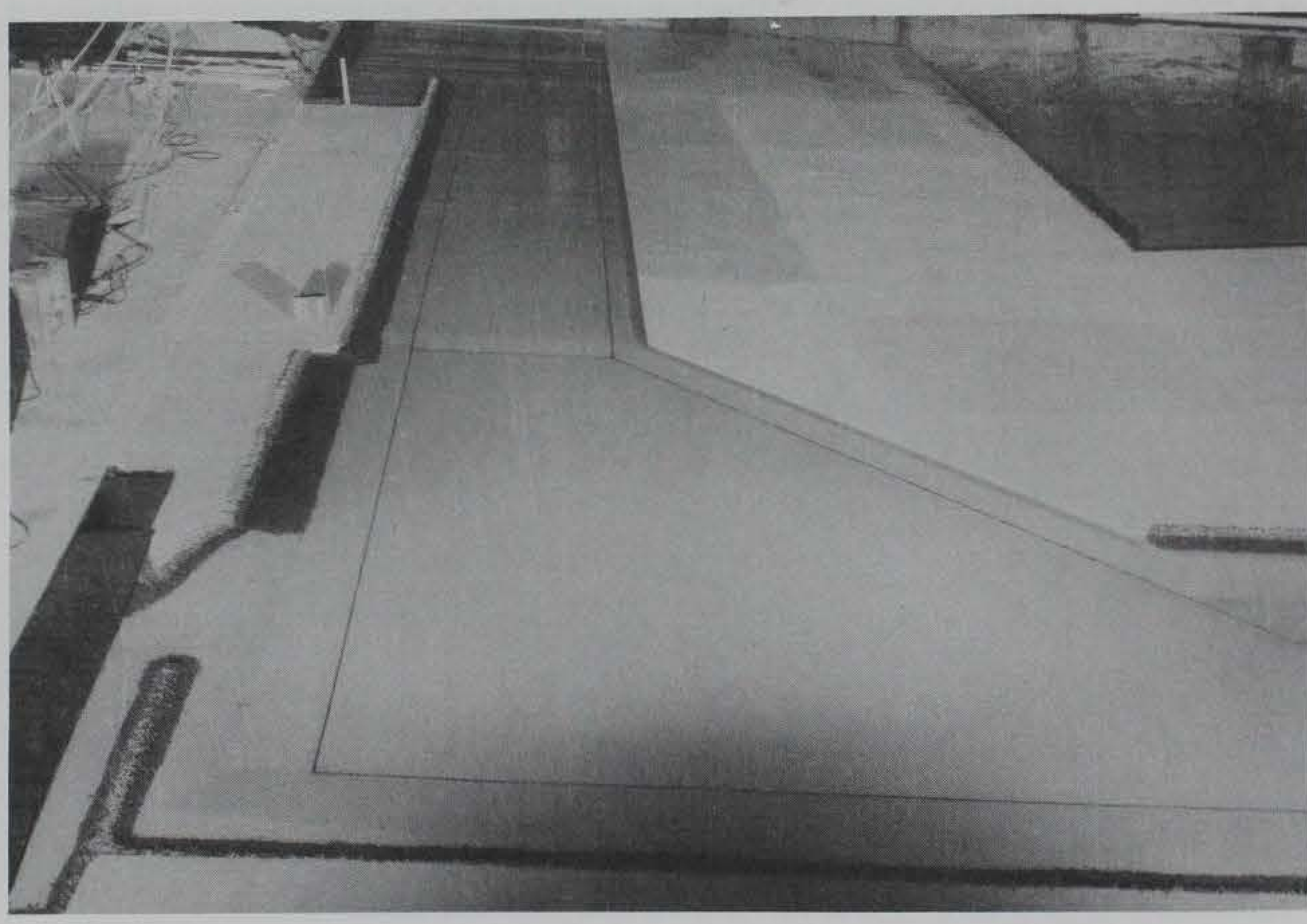


b. Pier 300

Photo 25. Typical wave patterns for POLA Stage 2; 16-sec, 20-ft test waves from 185 deg; $\gamma = 5.0$; swl = +8.0 ft

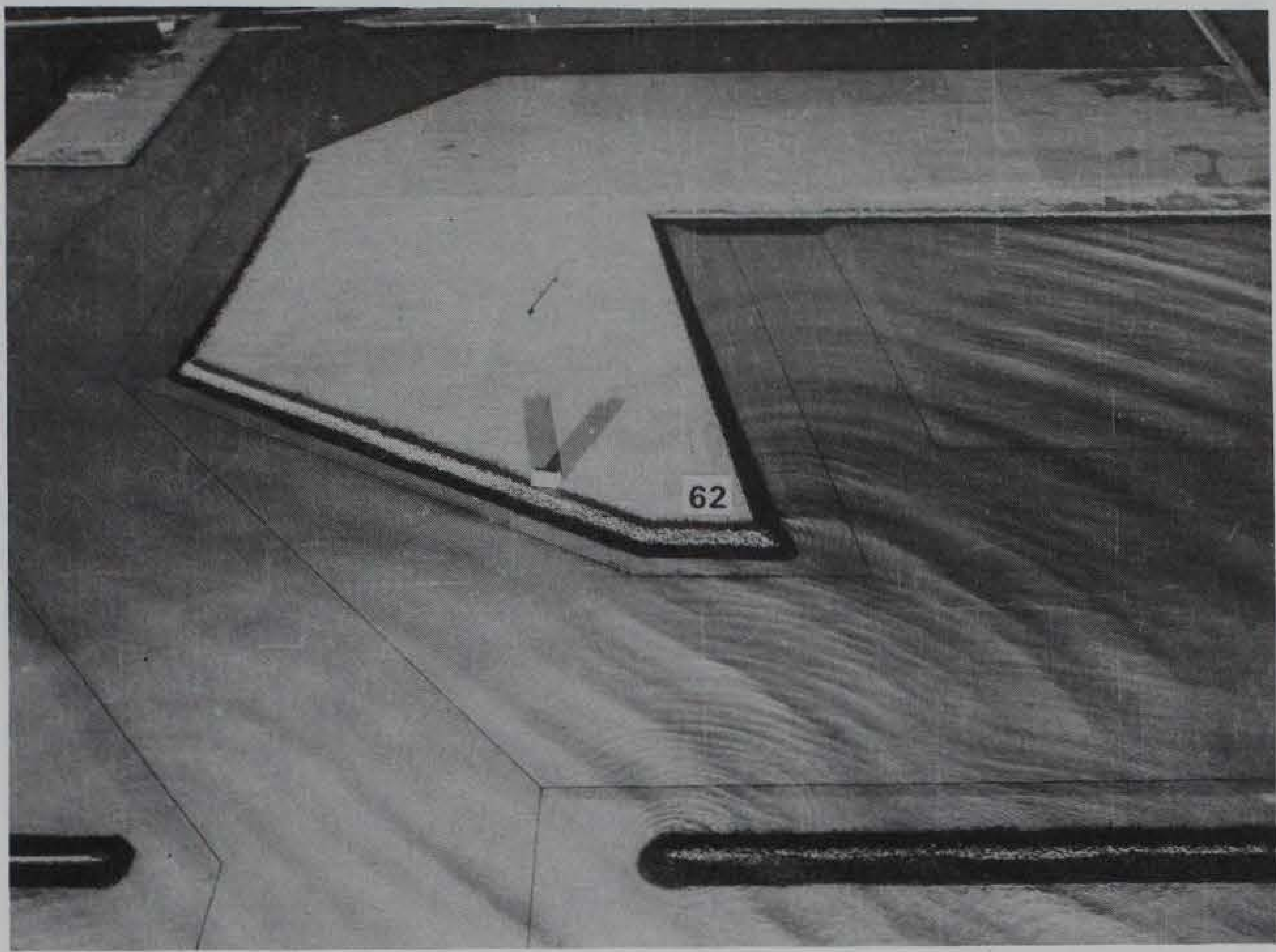


a. Pier 400

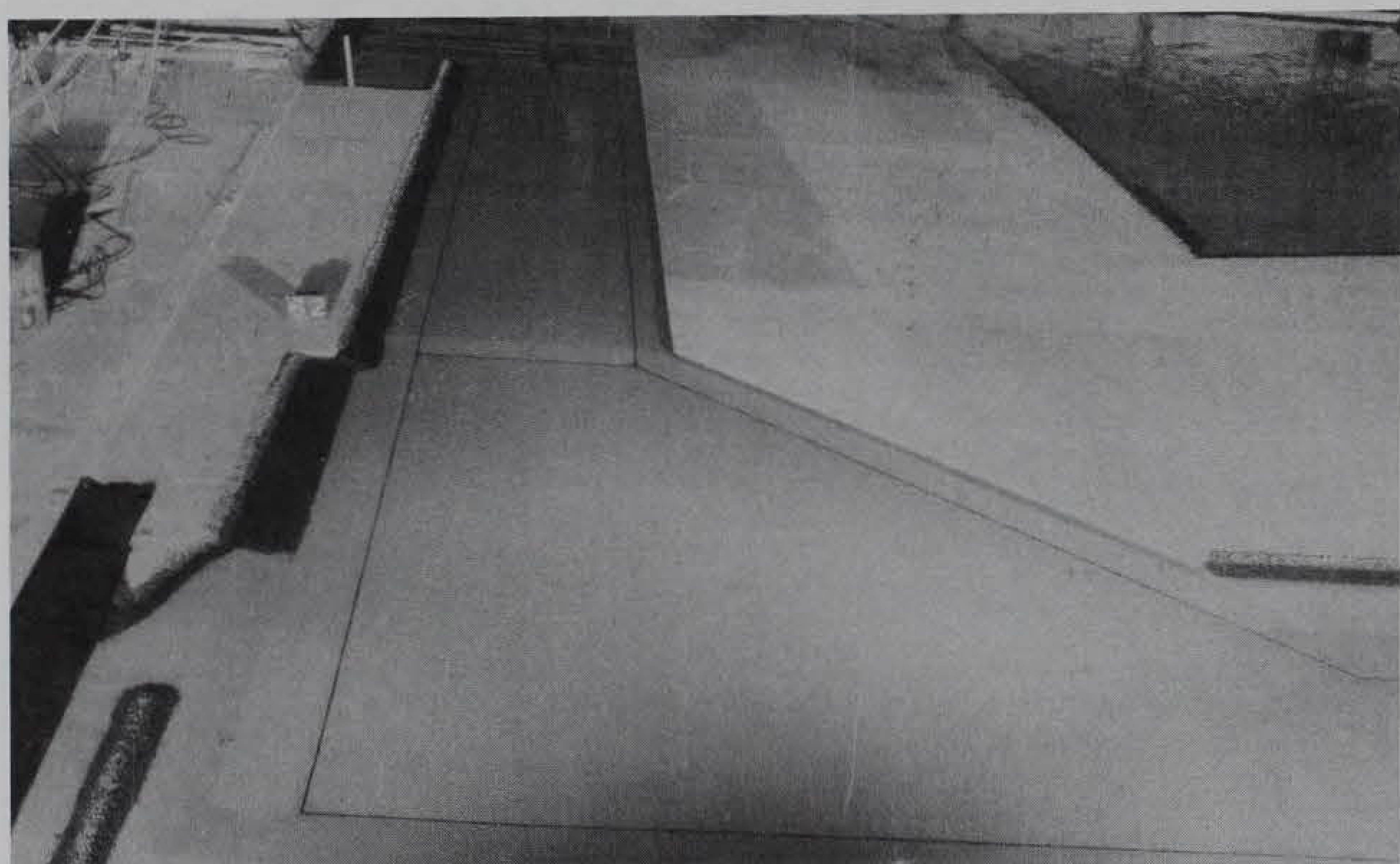


b. Pier 300

Photo 26. Typical wave patterns for POLA Stage 2; 8-sec, 4-ft test waves from 220 deg; $\gamma = 3.3$; swl = +5.5 ft

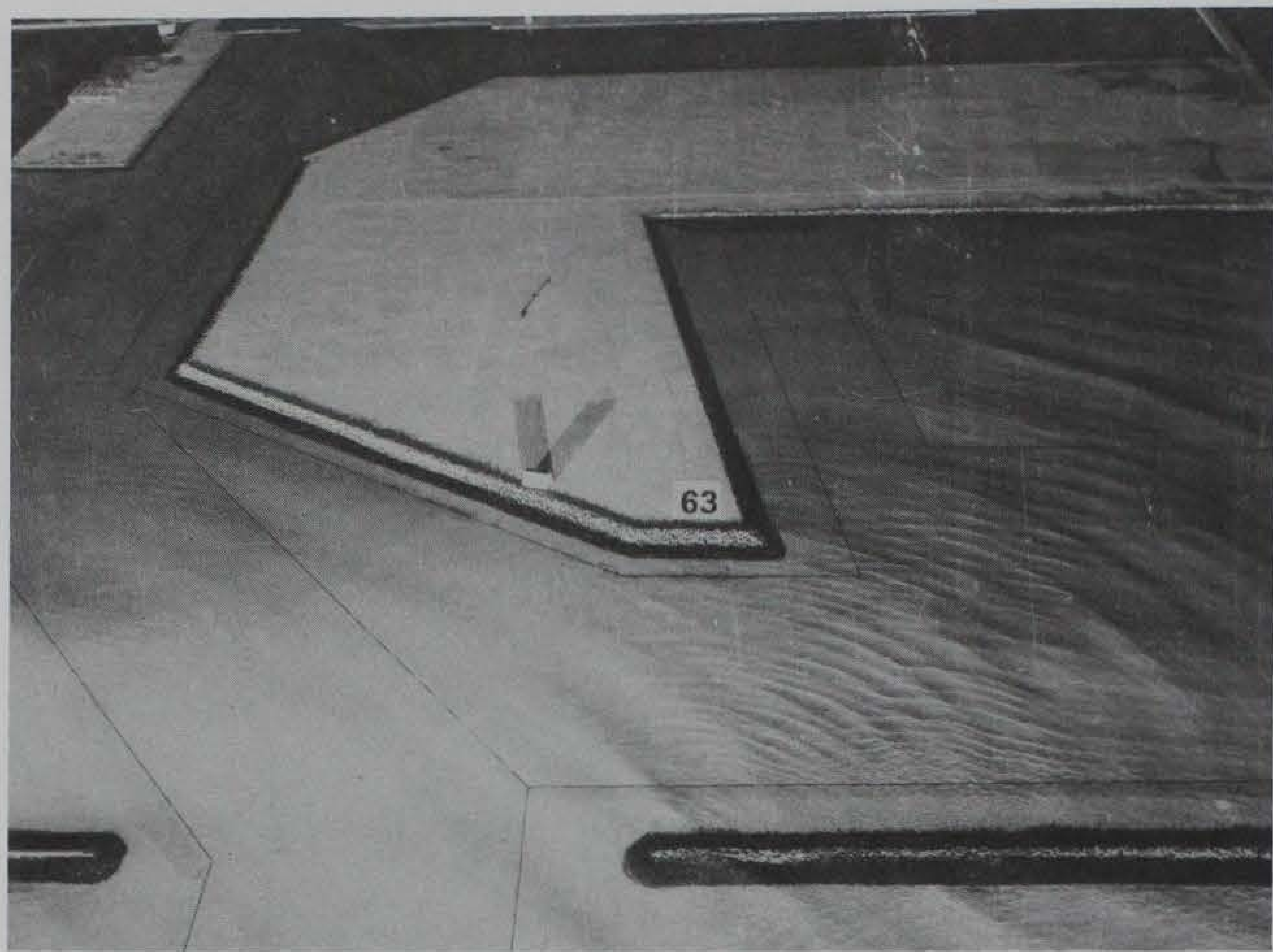


a. Pier 400

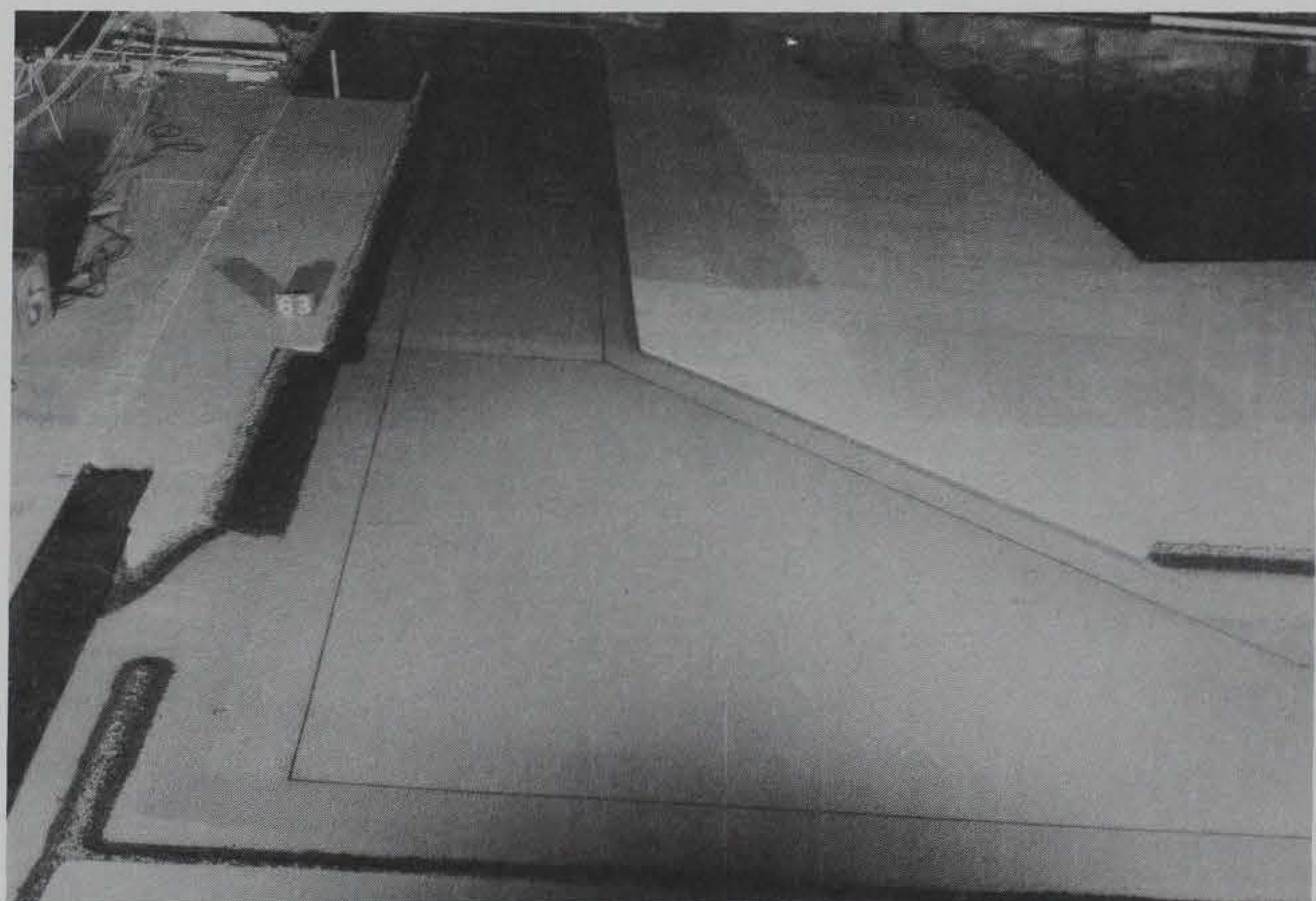


b. Pier 300

Photo 27. Typical wave patterns for POLA Stage 2; 12-sec, 8-ft test waves from 220 deg; $\gamma = 5.0$; swl = +5.5 ft

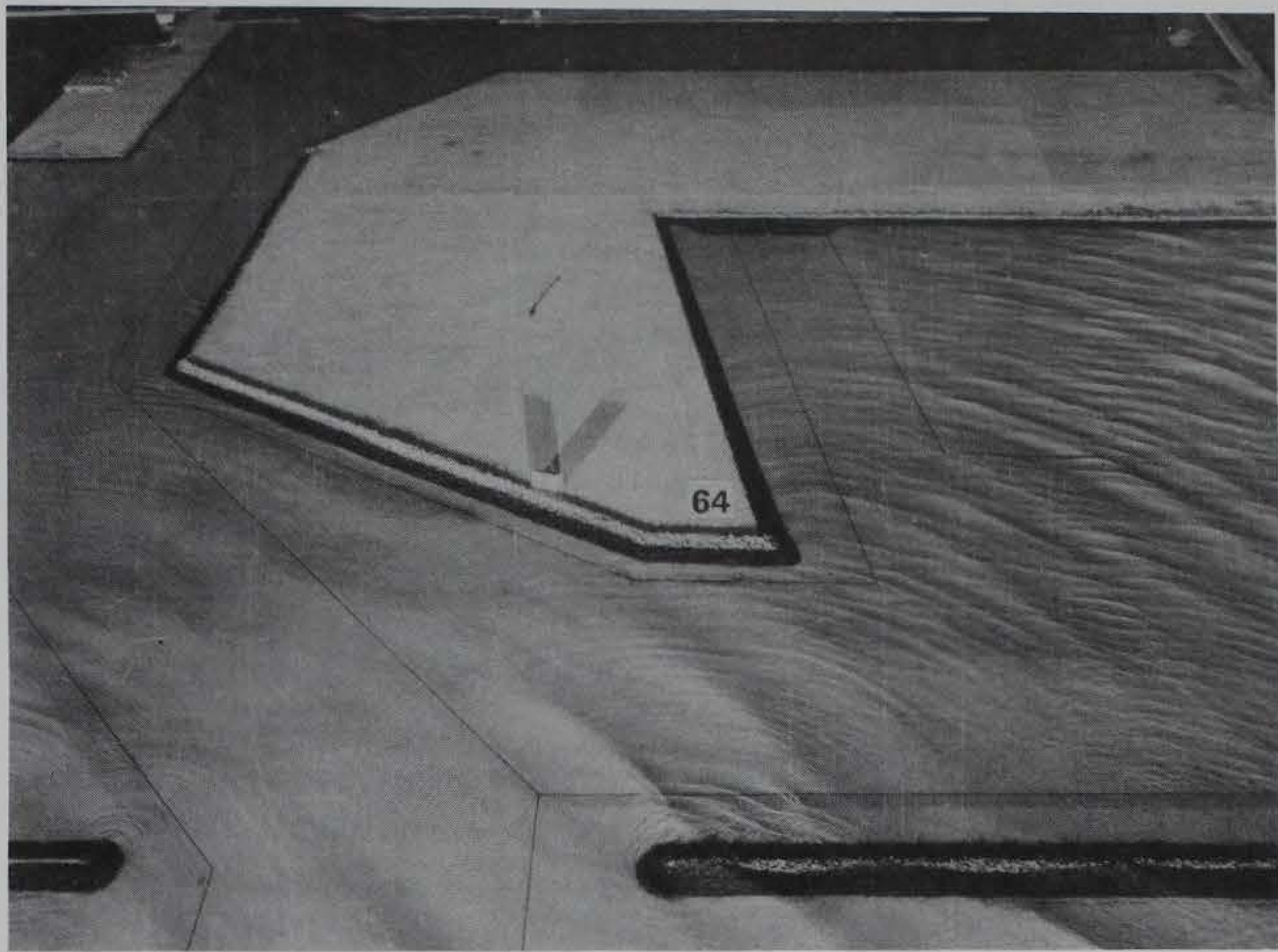


a. Pier 400

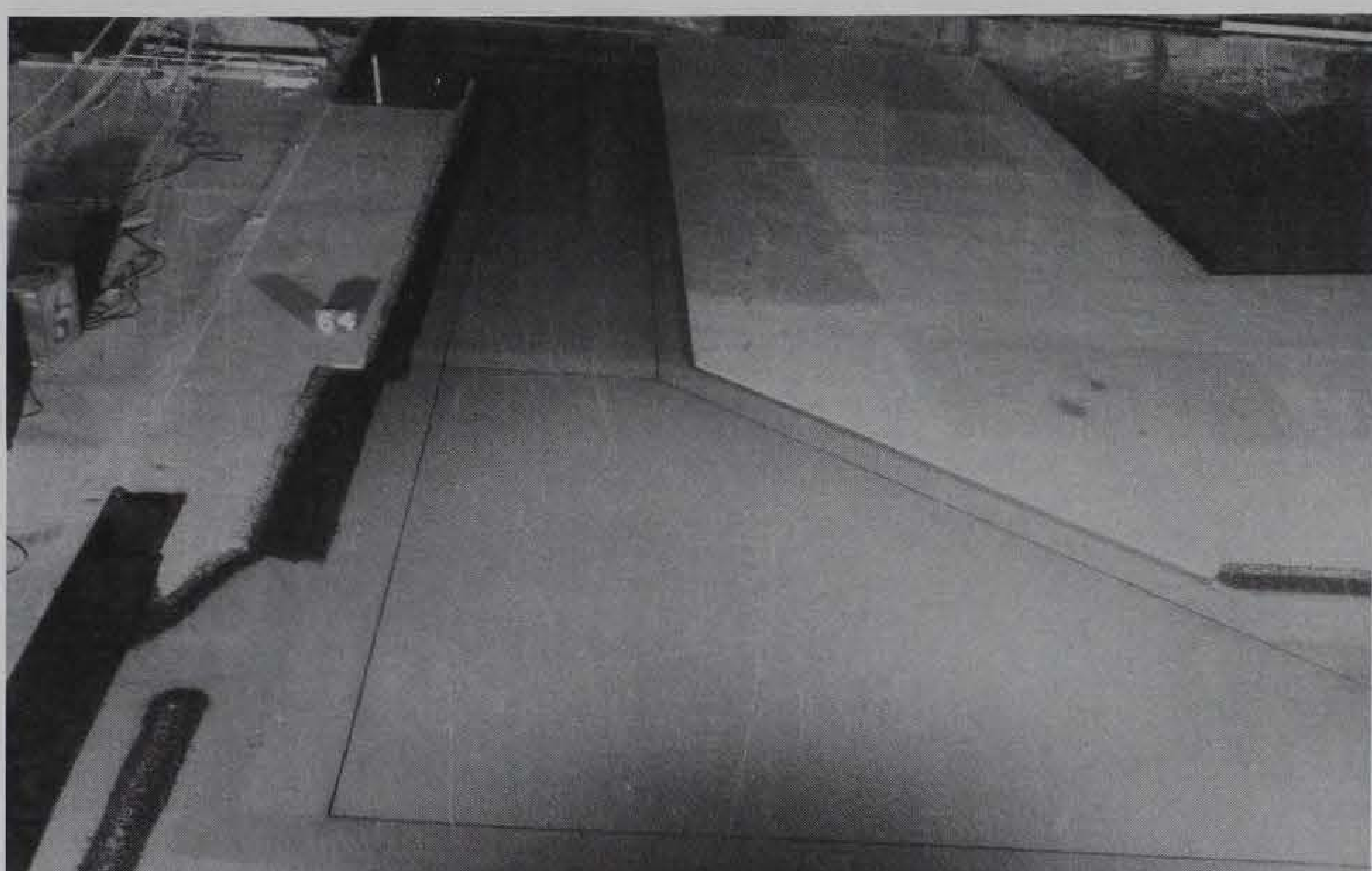


b. Pier 300

Photo 28. Typical wave patterns for POLA Stage 2; 16-sec, 12-ft test waves from 220 deg; $\gamma = 7.0$; swl = +5.5 ft

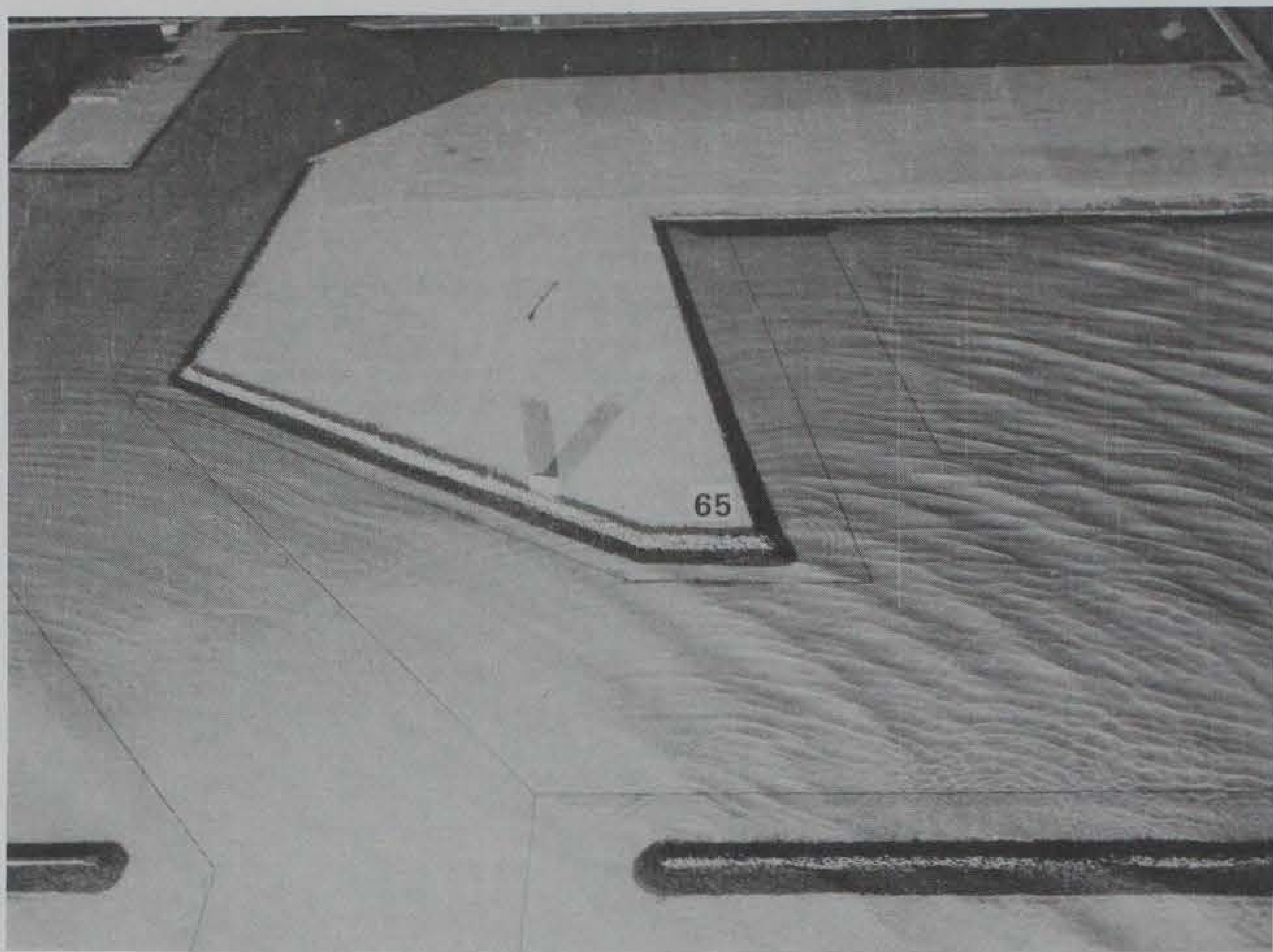


a. Pier 400

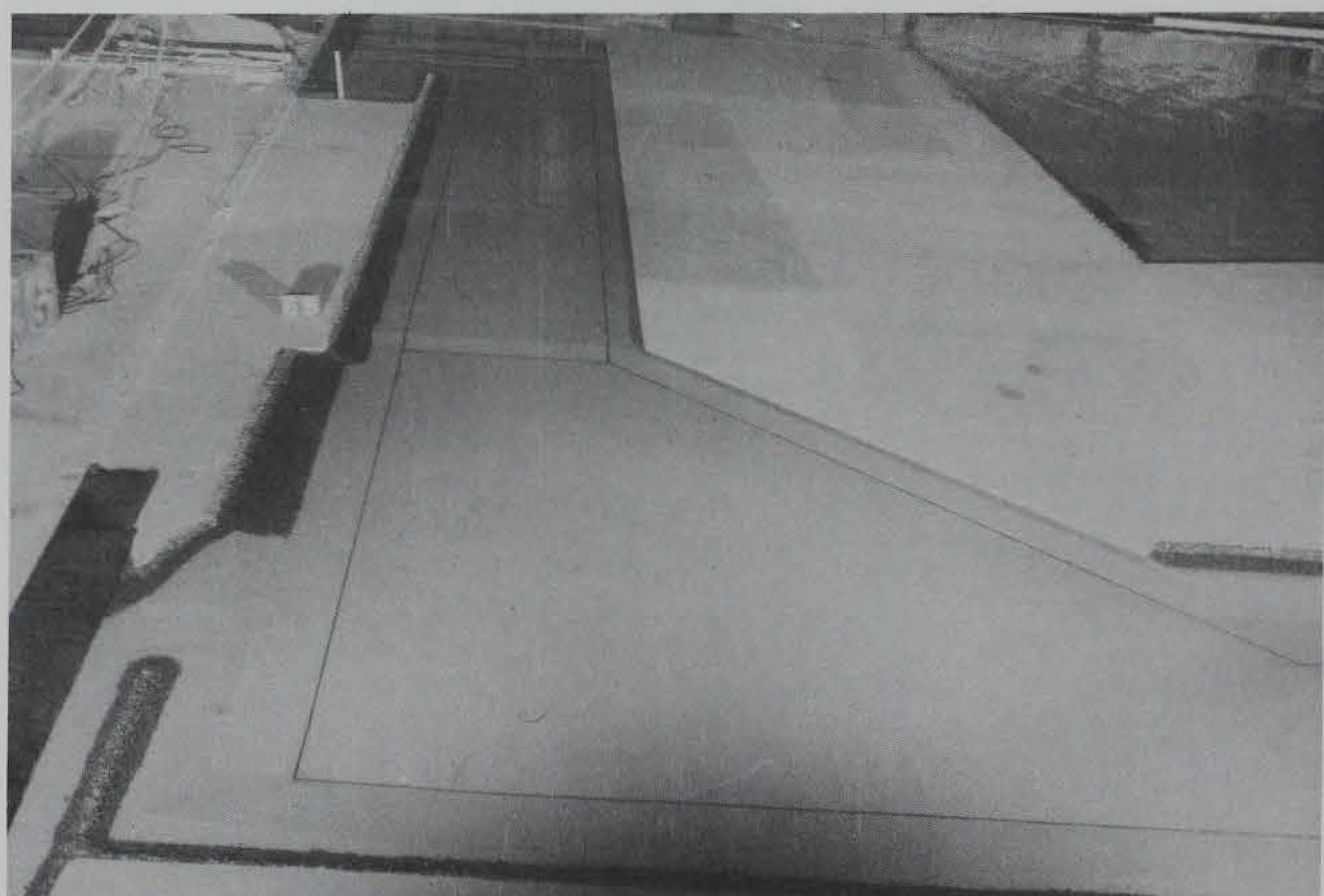


b. Pier 300

Photo 29. Typical wave patterns for POLA Stage 2; 14-sec, 14-ft test waves from 220 deg; $\gamma = 7.0$; swl = +8.0 ft



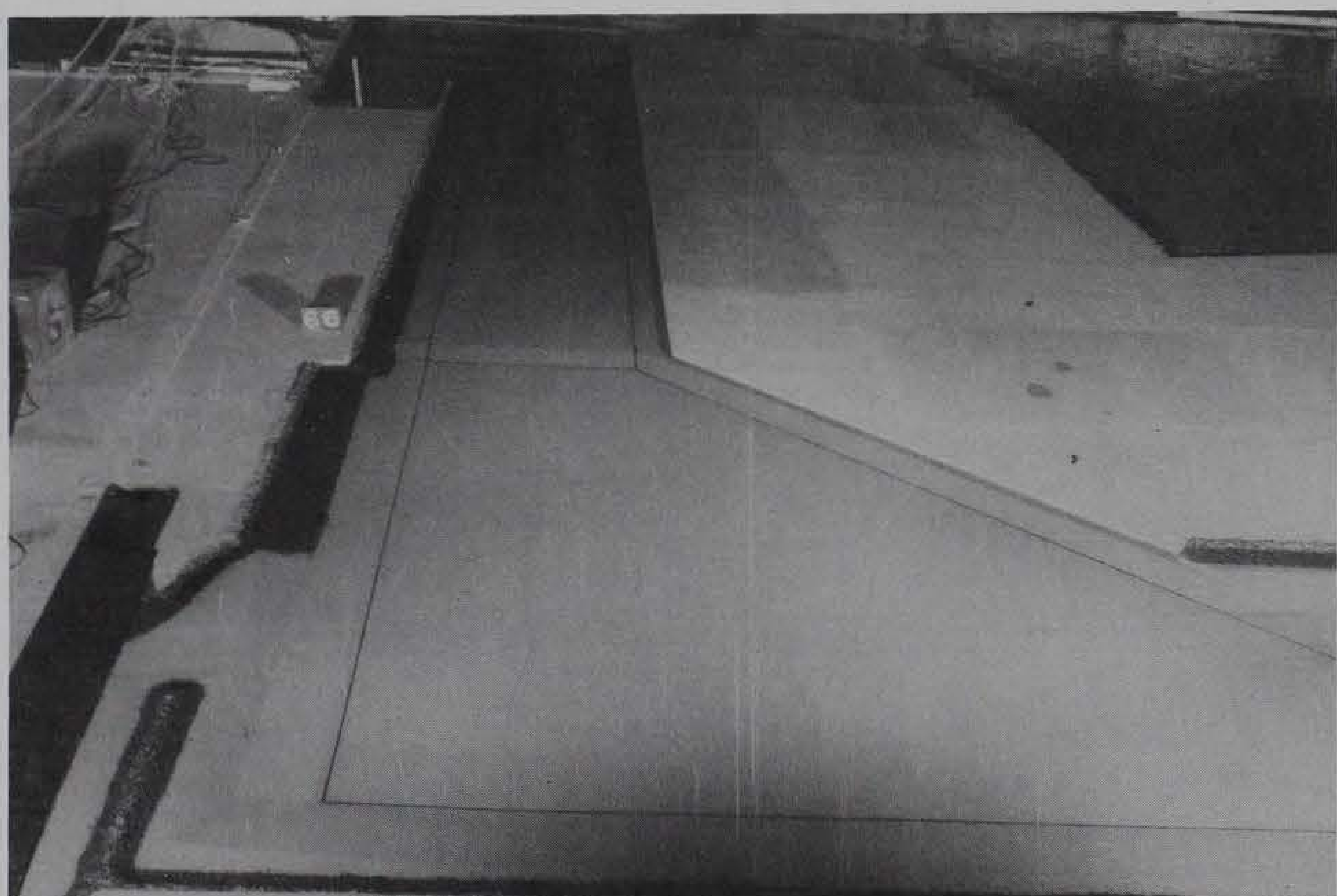
a. Pier 400



b. Pier 300

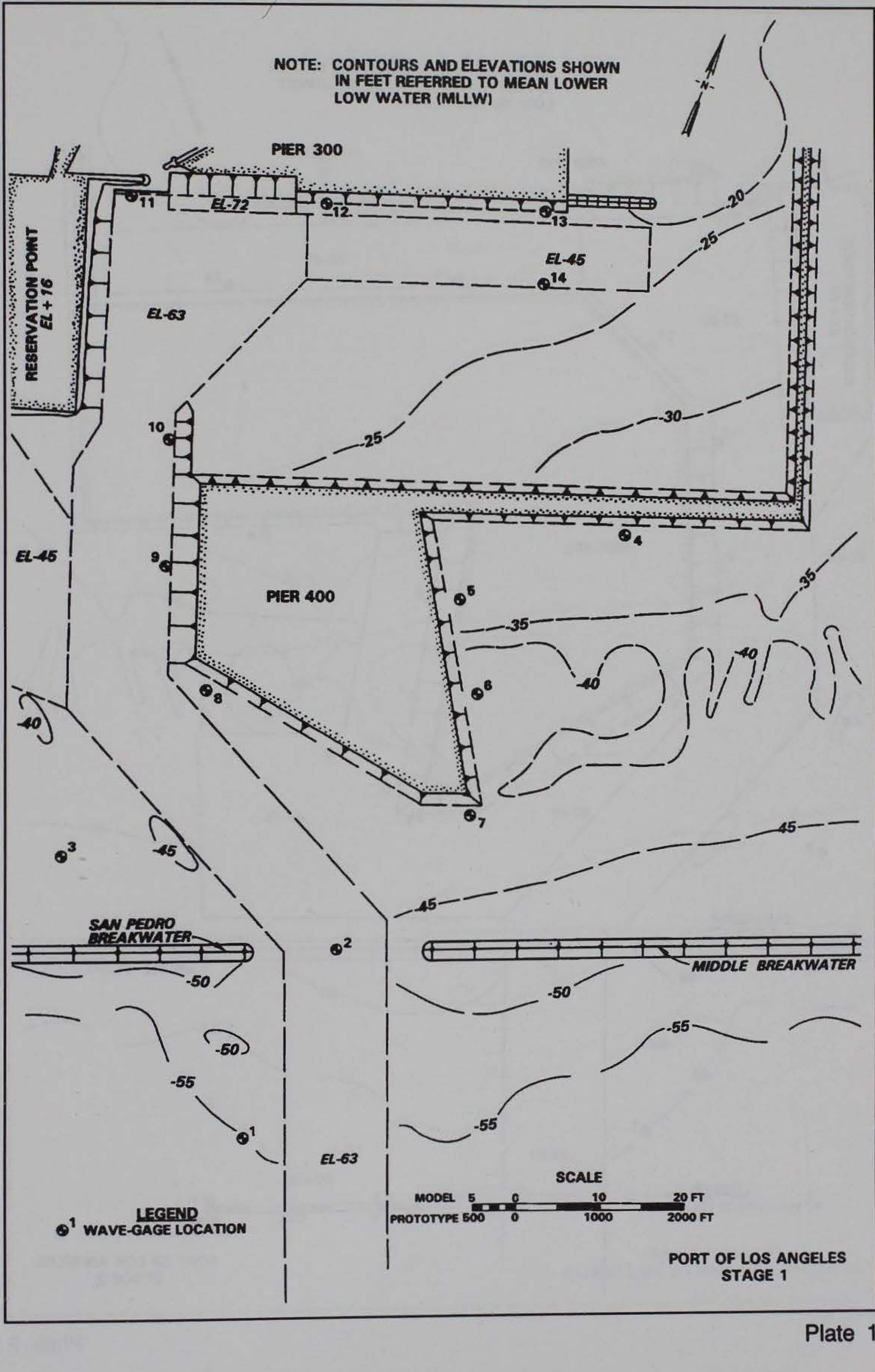
Photo 30. Typical wave patterns for POLA Stage 2; 16-sec, 16-ft test waves from 220 deg; γ = 7.0; swl = +8.0 ft

a. Pier 400



b. Pier 300

Photo 31. Typical wave patterns for POLA Stage 2; 20-sec, 20-ft test waves from 220 deg; $\gamma = 7.0$; swl = +8.0 ft


NOTE: CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
N
PIER 300
EL-72
EL-45
EL-63
RESERVATION POINT EL+16
PIER 400
EL-45
-20
-25
-25
-30
-35
-35
-40
-40
-40
-45
-45
-45
SAN PEDRO BREAKWATER
MIDDLE BREAKWATER
-50
-50
-50
-55
-55
-55
EL-63
SCALE
MODEL 5 0 10 20 FT
PROTOTYPE 500 0 1000 2000 FT
LEGEND
WAVE-GAGE LOCATION
PORT OF LOS ANGELES
STAGE 1

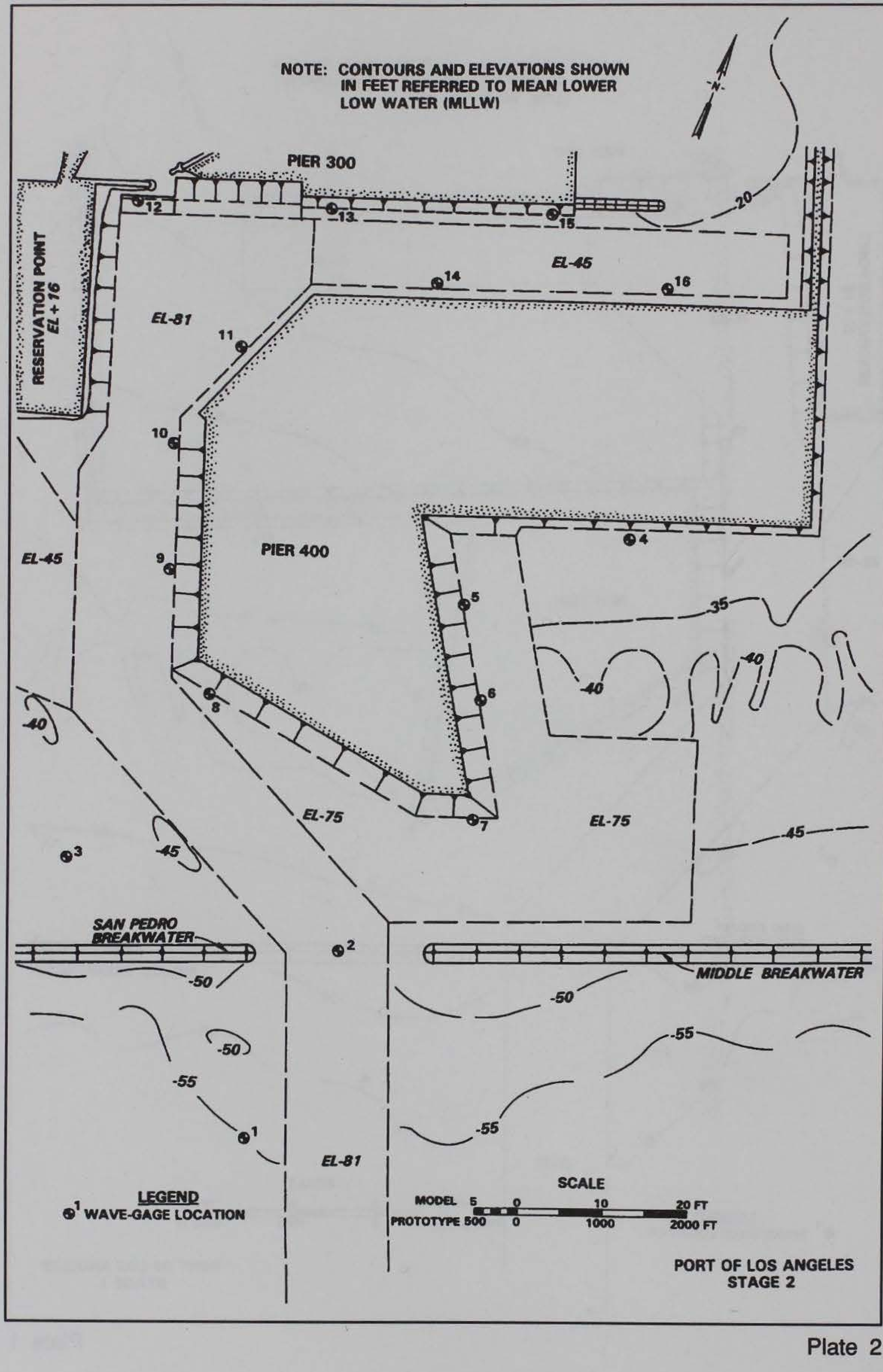

NOTE: CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
N
PIER 300
RESERVATION POINT
EL + 16
12
13
15
20
EL-45
14
16
EL-81
11
10
9
PIER 400
4
5
35
-40
-40
EL-45
6
8
-40
EL-75
7
EL-75
45
3
-45
SAN PEDRO BREAKWATER
2
MIDDLE BREAKWATER
-50
-50
-55
-50
-55
-55
1
EL-81
LEGEND
WAVE-GAGE LOCATION
SCALE
MODEL 5 0 10 20 FT
PROTOTYPE 500 0 1000 2000 FT
PORT OF LOS ANGELES
STAGE 2

Plate 2

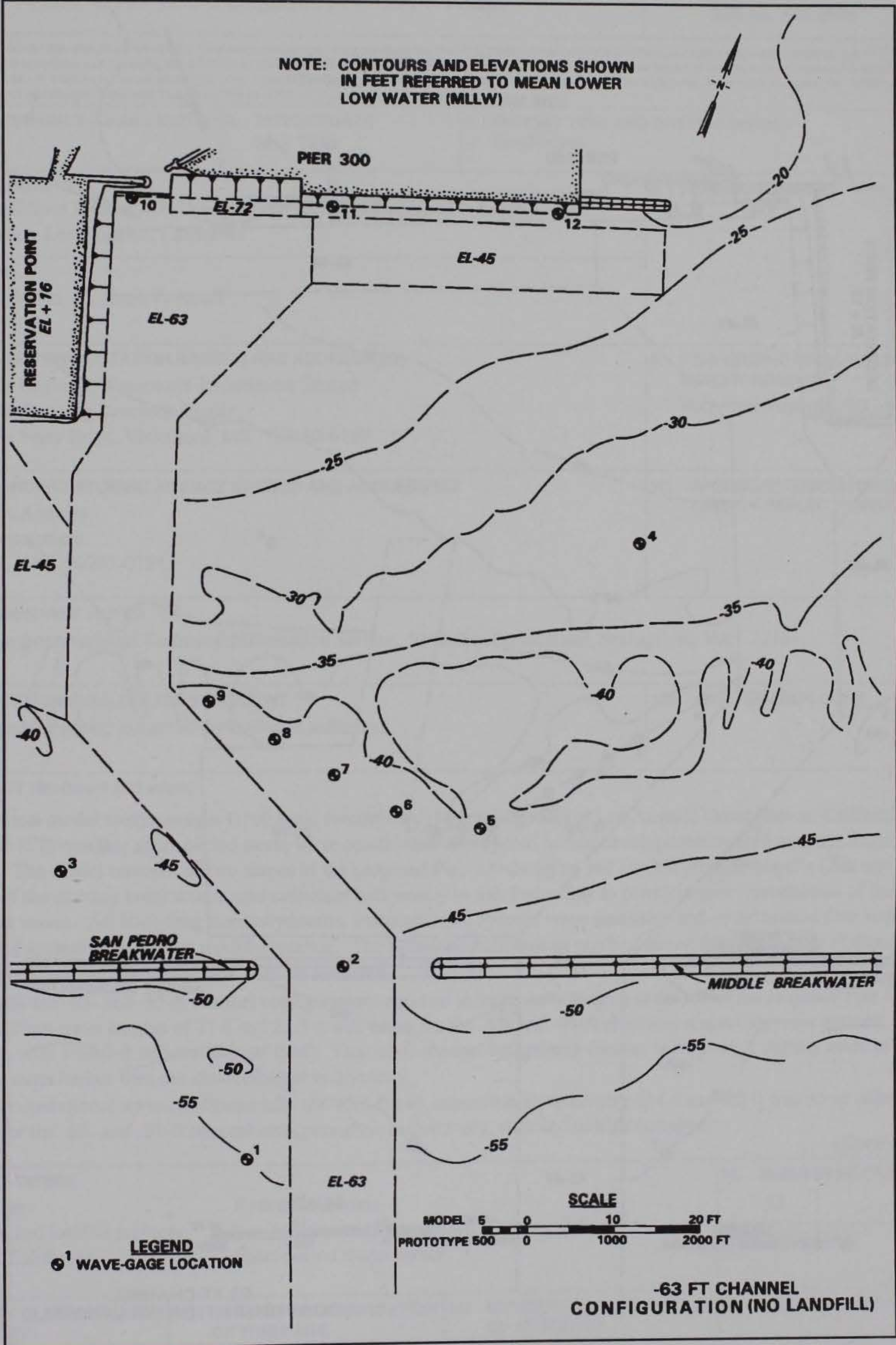
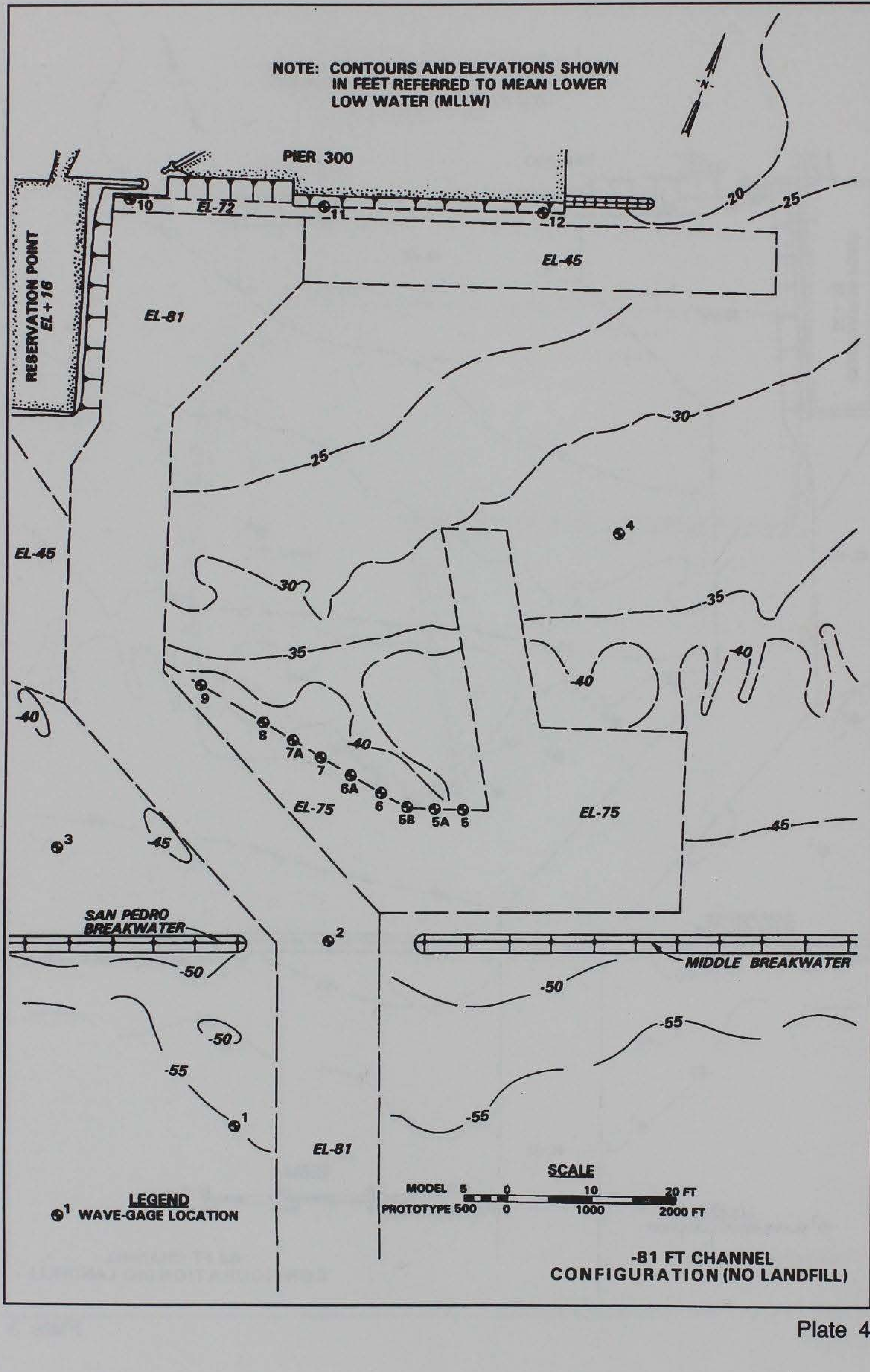

NOTE: CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
PIER 300
RESERVATION POINT EL + 16
EL-72
EL-45
EL-63
EL-45
-20
-25
-30
-35
-40
-45
-50
-55
SAN PEDRO BREAKWATER
MIDDLE BREAKWATER
EL-63
SCALE
MODEL 5 0 10 20 FT
PROTOTYPE 500 0 1000 2000 FT
LEGEND
1 WAVE-GAGE LOCATION
-63 FT CHANNEL CONFIGURATION (NO LANDFILL)

Plate 3


NOTE: CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOWER LOW WATER (MLLW)
N
PIER 300
RESERVATION POINT EL+16
EL-72
EL-45
EL-81
EL-45
EL-75
EL-75
EL-81
SAN PEDRO BREAKWATER
MIDDLE BREAKWATER
-20
-25
-30
-35
-40
-45
-50
-55
1
2
3
4
5
5A
5B
6
6A
7
7A
8
9
10
11
12
LEGEND
WAVE-GAGE LOCATION
SCALE
MODEL 5 0 10 20 FT
PROTOTYPE 500 0 1000 2000 FT
-81 FT CHANNEL CONFIGURATION (NO LANDFILL)

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13. ABSTRACT *(Maximum 200 words)*

A physical model study, using a 1:100 scale (undistorted) hydraulic model of Los Angeles Outer Harbor, California, was conducted to investigate short-period storm wave conditions for proposed harbor development located near the Angel's Gate entrance. The model reproduced two stages of the proposed Pier 400 dredging and landfill project, Angel's Gate entrance, portions of the existing breakwaters, and sufficient bathymetry in San Pedro Bay to permit proper reproduction of the required test waves. An 80-ft-long electrohydraulic, unidirectional, spectral wave generator and an automated data acquisition and control system were used in model operation. The following conclusions can be derived from the results of these tests:

Dredged channel configurations with no landfills

a. Both the -63- and -85-ft channel configurations resulted in large wave heights at the toe of the proposed Pier 400 landfill. Maximum wave heights of 21.8 and 22.3 ft will occur for the -63- and -81-ft channels, respectively, for extreme wave conditions with a +8.0-ft still-water level (swl). The -63-ft channel bathymetry focuses wave energy slightly more to the east inside the outer harbor than the -81-ft channel bathymetry.

b. For operational wave conditions with the +5.5-ft swl, maximum wave heights of 4.4 and 4.6 ft will occur adjacent to Pier 300 for the -63- and -81-ft channel configurations, respectively, with no landfills installed.

(Continued)

14. SUBJECT TERMS

Breakwaters	Hydraulic models
Dredging and landfill projects	Los Angeles and Long Beach Harbors, California
Harbors, California	Short-period storm waves

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16. PRICE CODE

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UNCLASSIFIED	UNCLASSIFIED		

13. (Concluded).

c. For extreme wave conditions with the +8.0-ft swl, maximum wave heights of 10.0 and 10.4 ft will occur adjacent to Pier 300 for the -63- and -81-ft channel configurations, respectively, with no landfills installed.

Port of Los Angeles (POLA) Stages 1 and 2

a. Both the POLA Stage 1 and Stage 2 landfill configurations provide excellent wave protection to the Pier 300 berthing areas. For operational wave conditions with the +5.5-ft swl, maximum wave heights will not exceed 0.4 ft; and for extreme wave conditions with the +8.0-ft swl, wave heights will not exceed 1.4 ft for either stage of construction.

b. The berth in the channel west of Pier 400 will experience maximum wave heights of 1.0 and 1.1 ft for operational wave conditions with the +5.5-ft swl; and 3.1 and 3.7 ft for extreme wave conditions with the +8.0-ft swl for POLA Stages 1 and 2, respectively.

c. The dredged berth east of Pier 400, included in the POLA Stage 2 configuration, will experience maximum wave heights of 2.4 ft and 3.9 ft for operational and extreme wave conditions, respectively.

d. Overtopping of the landfill in an area approximately 1,500 to 2,000 ft west of the causeway may occur for both POLA Stages 1 and 2 for extreme test wave conditions with the +8.0-ft swl.