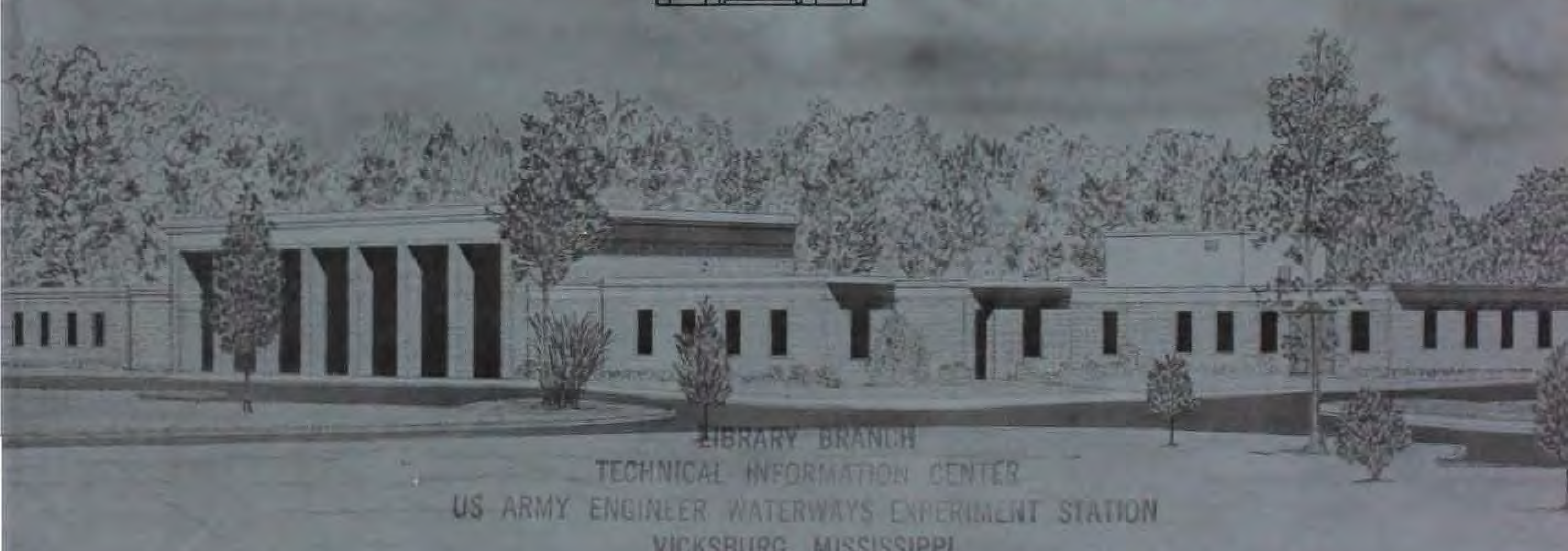
TECHNICAL REPORT H-73-8

STUDY OF BEACH WIDENING BY THE PERCHED BEACH CONCEPT SANTA MONICA BAY, CALIFORNIA

Hydraulic Model Investigation

by

C. E. Chatham, Jr., D. D. Davidson, R. W. Whalin



June 1973

Sponsored by U. S. Army Engineer District, Los Angeles

Conducted by U. S. Army Engineer Waterways Experiment Station
Hydraulics Laboratory
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ARMY-MRC VICKSBURG MISS

FOREWORD

A request for the U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi, to conduct hydraulic model investigations of the perched beach concept was made by the U. S. Army Engineer District, Los Angeles (LAD), during a conference held at the WES on 12-14 August 1969. Authorization for an agreement between the Corps of Engineers (represented by the LAD) and the Division of Highways (DOH) of the State of California for the purpose of studying the technical feasibility of the perched beach concept had been granted by the Office, Chief of Engineers, on 6 January 1969. Subsequently, a formal contract between the Corps of Engineers and the State of California was signed on 14 May 1969. This contract was amended on 20 May 1970 to broaden the scope of work to be performed by the Corps of Engineers.

During October 1971, the State of California requested cessation of the contract work before its completion. Hence, it was not possible to complete the scheduled series of tests. The DOH had been interested in the possible use of the perched beach concept in constructing the proposed Route 1 freeway from Santa Monica to Topanga Canyon. In June 1971, after model studies had begun, the California Legislature deleted the study area from the California Freeway and Expressway System, thereby ruling out direct application of the studies; subsequently, the State of California requested contract cessation after a convenient stopping point had been reached.

The model studies were conducted at the WES during the period May 1970-October 1971, in the Wave Dynamics Division of the Hydraulics Laboratory, under the direction of Messrs. E. P. Fortson, Jr., and H. B. Simmons, Retired Chief and Chief, respectively, of the Hydraulics

Laboratory, and Mr. R. Y. Hudson and Dr. R. W. Whalin, Retired Chief and Chief, respectively, of the Wave Dynamics Division. The tests were conducted by Messrs. C. E. Chatham, Jr., Chief, Harbor Wave Action Branch, and D. D. Davidson, Chief, Wave Research Branch, with the assistance of Messrs. G. G. Stout and L. A. Barnes, SP4 T. M. Anderson, and SP4 J. E. Dick. This report was prepared by Mr. Chatham, Mr. Davidson, and Dr. Whalin. Appendix A was prepared by Dr. D. L. Inman, Consultant for DOH.

During the course of the investigation, liaison was maintained among the LAD, DOH, and WES by means of conferences, telephone communications, and monthly progress reports.

The following personnel visited the WES to observe model operations and participate in conferences: Dr. Inman and Professor J. W. Johnson, Consultants for the DOH; Mr. O. F. Weymouth of the U. S. Army Engineer Division, South Pacific; Mr. Thorndike Saville, Jr., of the U. S. Army Coastal Engineering Research Center; and Messrs. C. H. Fisher, F. J. Buchholz, and H. D. Converse of the LAD.

Directors of the WES during the conduct of the studies and the preparation of this report were COL Levi A. Brown, CE, and COL Ernest D. Peixotto, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

NOTATION

a	Wave amplitude
a_n	Wave amplitude at the shoreline
A	Area
b	Model width
D	Median particle diameter
e	Exponential function
g	Gravitational constant
h	Water depth
h_o	Initial water depth
H	Shallow-water wave height
H_o	Deepwater wave height
k	Wave number = $2\pi/L$
K	Shoaling coefficient
L	Wavelength
L_c	Wavelength for cutoff mode
$L_n(2\lambda X)$	Laguerre polynomial of the order n
L_o	Deepwater wavelength
m	Longshore modal number
n	Offshore modal number
s	Shallow-water orthogonal spacing
s_o	Deepwater orthogonal spacing
S_r	Specific gravity of armor unit (γ_r/γ_w)
t	Time
T	Wave period
v	Longshore orbital velocity
V	Velocity

$\forall$	Volume
W_r	Weight of armor unit, lb
x_b	Width of the surf zone
X	Horizontal coordinate in the offshore direction
X_d	Horizontal coordinate in the offshore direction for a modal decay depth
Y	Horizontal coordinate in the longshore direction
Z	Vertical coordinate
α	Coefficient
$\tan \beta$	Beach slope
γ_c	Specific weight of coal
γ_c'	Apparent specific weight of coal
γ_r	Specific weight of rock, pcf
γ_s	Specific weight of prototype sand
γ_s'	Apparent specific weight of prototype sand
γ_w	Specific weight of water
η_D	Ratio of median particle diameters
η_γ'	Ratio of apparent specific weights
λ	Longshore wave number
μ	Vertical scale
σ	Wave frequency $(2\pi/T)$
$\emptyset$	Velocity potential
ψ	Horizontal scale
ω	Angular (or radian) frequency of the waves
Ω	Model distortion

CONVERSION FACTORS, BRITISH TO METRIC UNITS OF MEASUREMENT

British units of measurement used in this report can be converted to metric units as follows:

Multiply	By	To Obtain
inches	2.54	centimeters
feet	0.3048	meters
square feet	0.0929	square meters
miles (U. S. statute)	1.6093	kilometers
square miles	2.5900	square kilometers
feet per second	0.3048	meters per second
pounds (mass)	0.4536	kilograms
pounds per cubic foot	16.0185	kilograms per cubic meter

SUMMARY

Hydraulic model studies were conducted at the U. S. Army Engineer Waterways Experiment Station to aid in determining the technical feasibility and optimum design factors of the perched beach concept for widening the existing beach to provide right-of-way for a freeway along a portion of the Santa Monica Bay coastline (see location map and typical cross section of perched beach in figs. 1 and 2, respectively). The original study proposal consisted of six main parts. However, during the course of the model studies, the California Legislature deleted this section of the freeway from the California Freeway and Expressway System. As a result, the Division of Highways terminated their freeway location project and canceled further model testing. Consequently, only the following three parts of the study were completed:

a. An undistorted, three-dimensional, fixed-bed model (scale 1:100) was used to determine the effect of the perched beach on rip currents. If adverse interactions were found to be present, the model was to be used to determine means of minimizing them.

b. A distorted-scale (1:100 horizontal, 1:50 vertical), two-dimensional, movable-bed model was used to estimate the amount of sand which might be lost seaward over the toe structure due to normal and storm wave actions and to determine the optimum crown elevation of the submerged structure and the length of stone riprap apron required to reduce the seaward migration of sand to a minimum.

c. An undistorted, two-dimensional model (scale 1:30) was used to determine the structural design of the proposed rubble-mound toe structure for various depths.

The following three parts of the study were not completed:

a. An office data (environmental data) analysis, including a hydrographic survey with sand sampling, a reevaluation of geographical information, and a computation of littoral transport.

b. The construction and testing of a surf beat model to evaluate the wave runup characteristics with and without a perched beach.

c. An office study to determine the feasibility of constructing a three-dimensional, movable-bed model.

This report describes the testing and results up to the premature termination of the model studies. It was concluded from test results that:

a. Installation of the perched beach in the model had no adverse effect on rip currents and reduced rip current velocities about 20 percent for the configuration tested (see plates 1 and 2).

b. Normal wave action (waves expected to occur a high percentage of the time) on the perched beach will cause no appreciable loss of beach fill (see plate 3 for details of plans tested).

c. For the larger storm waves of any significant duration, a large net seaward loss of fill material can be expected.

d. The installation of a 100-ft stone apron in conjunction with a 350-ft-wide (measured from toe structure to 0.0-ft-mllw contour) perched beach (plan 2A) will have little or no effect on the loss of fill material.

e. The installation of a 100-ft stone apron in conjunction with a 700-ft-wide (measured from toe structure to 0.0-ft-mllw contour) perched beach (plan 1A) will significantly reduce the amount of beach fill lost seaward of the toe structure.

f. If the beach fill is extended as far as 1100 ft seaward of the 0.0-ft-mllw contour (plans 3 and 3A), the toe structure will have little or no beneficial effect on reducing the amount of beach fill lost.

g. Of all plans tested, plan 1A appears to offer the greatest degree of protection against seaward loss of beach fill material.

h. Five-thousand-lb armor stone will be needed if the rubble-mound toe structure is located in water depths ranging from 20 to 35 ft; 3000-lb armor stone will be needed in depths from 35 to 45 ft; and 1000-lb armor stone will be needed in depths greater than 45 ft.

i. Quarry-run stone whose 50 percent weight is about 100 lb will be adequate bed material for the toe structure.

j. Two-hundred-fifty-lb stone will be stable for the riprap apron shoreward of the toe structure.

k. Additional wave flume tests will be needed with armor units in the specific gravity range of 1.3 to 1.6 prior to performing a three-dimensional, movable-bed model investigation.

l. Provided adequate prototype data become available for use in model verification, a three-dimensional, movable-bed model investigation appears to be feasible and should result in a valid indication of the relative merits of various project designs.

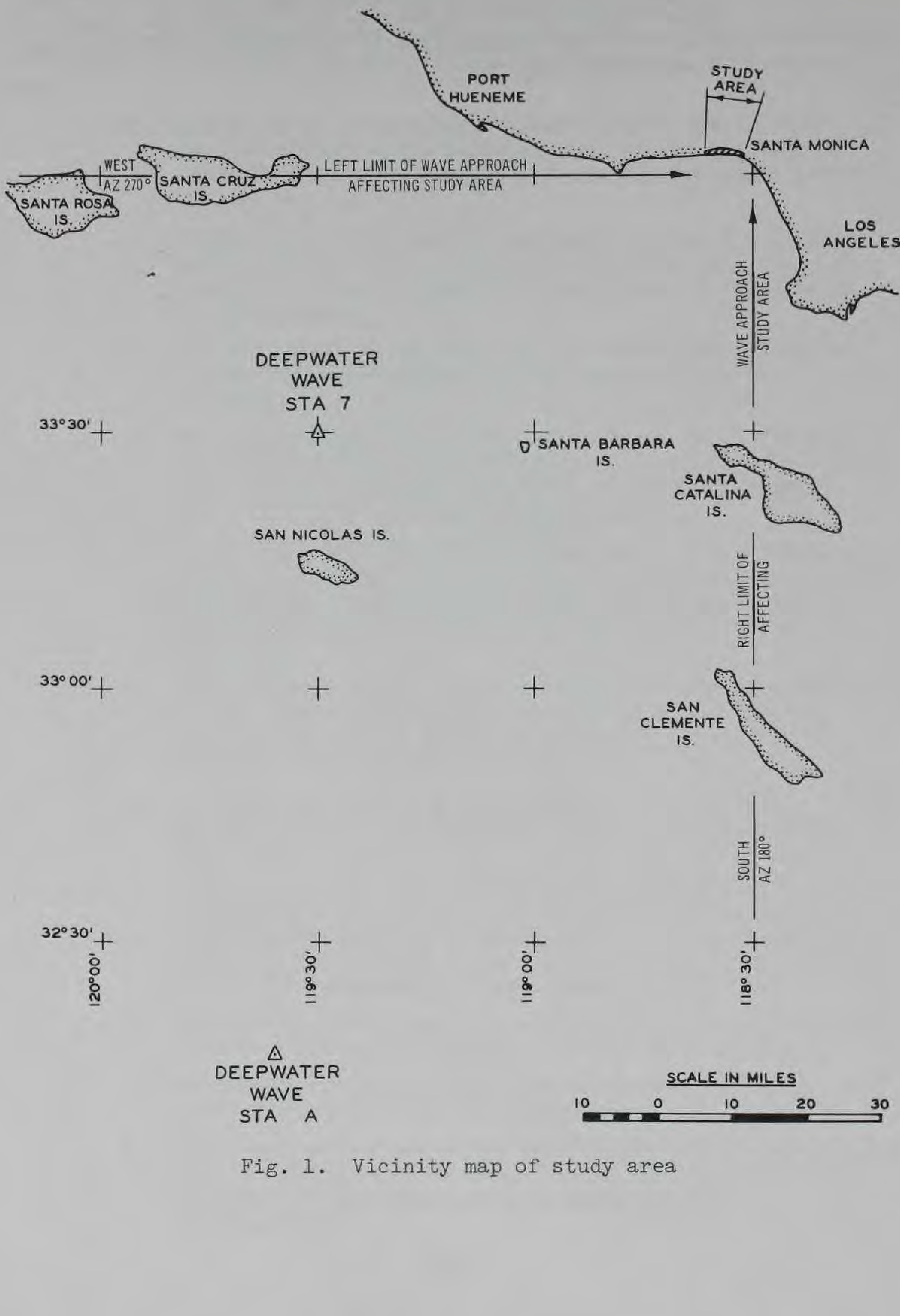

Fig. 1. Vicinity map of study area

STUDY OF BEACH WIDENING BY THE PERCHED BEACH CONCEPT

SANTA MONICA BAY, CALIFORNIA

Hydraulic Model Investigation

PART I: INTRODUCTION

Background

1. Because of topographical, environmental, and urban constraints along a particular stretch of the Santa Monica Bay coastline (fig. 1), the Division of Highways (DOH) of the State of California wanted to evaluate the possibility of locating a proposed freeway along the present shoreline by artificially widening the existing beach. Although the groin method of beach widening was available, it was not considered acceptable in this instance for aesthetic reasons.

2. The concept of a perched beach is based on the natural occurrence of a toe structure offshore from a beach at Algodones in the Gulf of California. This beach was surveyed in April 1960, and it was found that the rock outcrop that formed the natural toe permitted the beach to extend farther seaward than was normal for a beach of this type. These 1960 investigations were reported in a paper by Inman and Frautschy.[1] Subsequent discussions with the Corps of Engineers resulted in the inclusion of the perched beach concept in a 1963 exploratory study[2] by the U. S. Army Engineer District, Los Angeles, to investigate freeway locations both offshore and onshore. This investigation included studies of locating a causeway and trestle offshore and of beach widening by the groin method and the perched beach concept to locate a right-of-way onshore. At that time, the Corps of Engineers stated that more detailed studies were necessary in order to determine the feasibility of the perched beach widening concept. A request by the Los Angeles District in 1969, that a hydraulic model investigation of the perched beach concept be undertaken at the U. S. Army Engineer Waterways

Experiment Station (WES), resulted in the conduct of the studies reported herein.

3. In cases where the berm of an existing beach has been shifted seaward of its original position, the natural profile of the beach before relocation cannot be reproduced, since such a profile would not intersect the deeper bottom. Consequently, moving a beach seaward results in a steeper profile, unless a toe wall intersects the new beach at some point to hold it in a perched position.

4. The proposed 6-mile-long* relocated beach at Santa Monica Bay was to be constructed using approximately the same profile as the old beach from the berm to about the 25-ft depth contour,** where a submerged, rubble-mound, breakwater-type structure extending from the deeper ocean bottom was to be constructed to hold the new beach in a perched position (see fig. 2). Since the turbulence induced by oncoming waves as they travel over the submerged toe structure could transport quantities of beach material seaward, a stone riprap apron would probably be needed along the shoreward edge of the breakwater crest to reduce seaward migration of sand. In order to be effective, a perched beach must allow the natural littoral processes to continue without being supplemented by excessive beach replenishment. Fig. 3 shows the coastal zone terminology used in this report.

Objectives of the Model Studies

5. Hydraulic model studies were conducted at the WES to aid in determining the technical feasibility and optimum design factors of the perched beach concept. These studies and their primary objectives were as follows:

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* A table of factors for converting British units of measurement to metric units is presented on page ix.

** All elevations (el) cited herein are referred to mean lower low water (mllw), and all units are given in prototype dimensions unless otherwise noted.

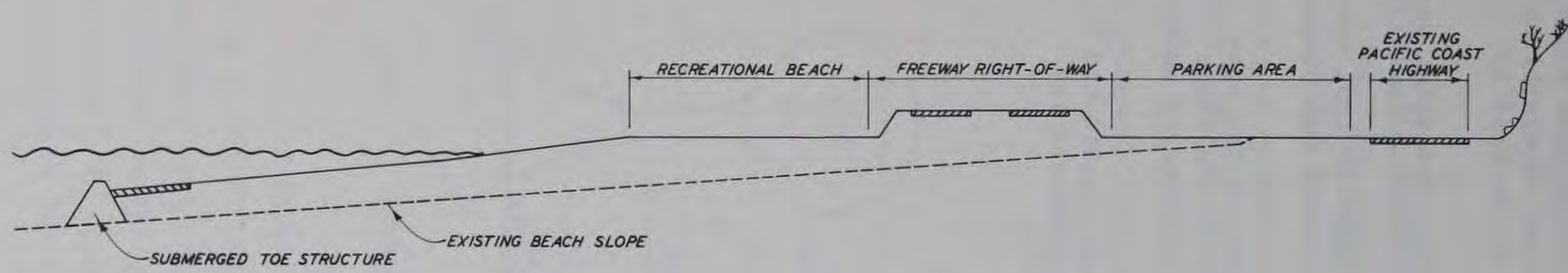

Fig. 2. Typical section of perched beach

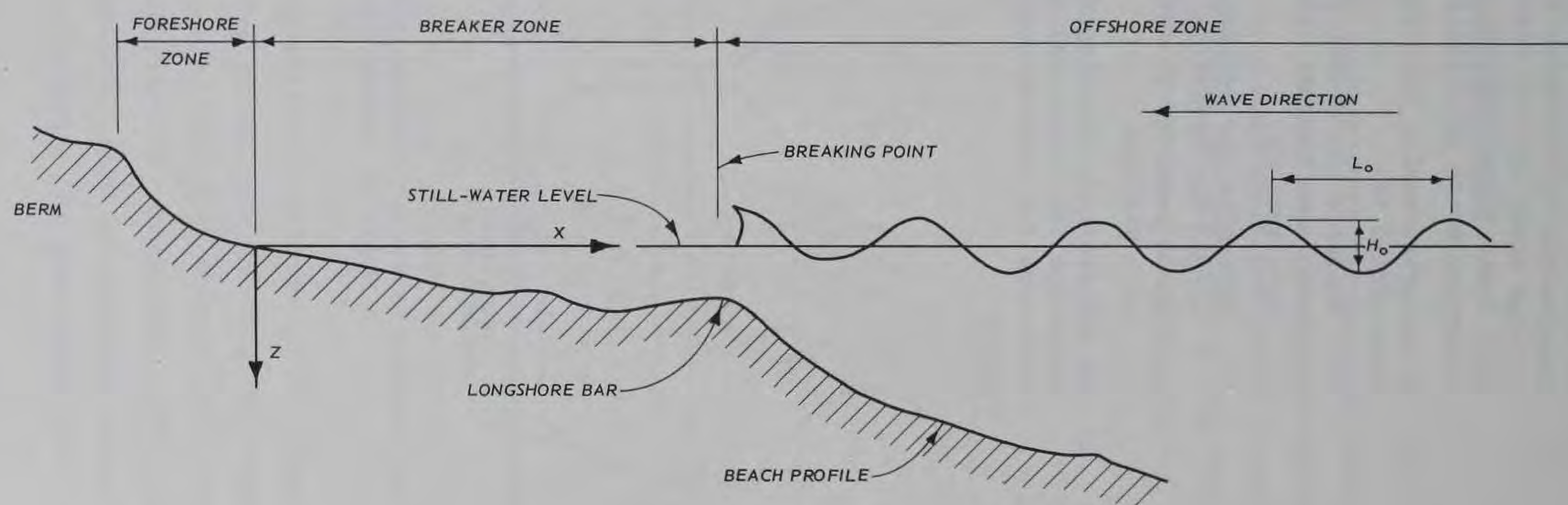

Fig. 3. Coastal zone terminology

a. An undistorted, three-dimensional, fixed-bed model was constructed to determine the effect of the perched beach on edge waves and rip currents. If adverse interactions were found to be present, the model was to be used to determine means of minimizing them.

b. A distorted-scale, two-dimensional, movable-bed model was constructed to:

    (1) Estimate the amount of sand which might be lost seaward over the toe structure due to normal and storm wave actions.

    (2) Determine the optimum crown elevation of the submerged toe structure and the length of the stone apron required to reduce seaward migration of sand to a minimum.

c. An undistorted, two-dimensional model was constructed to determine the structural design of the submerged toe structure for various depths.

PART II: DESIGN OF THE MODELS

The Rip Current Model

Theoretical description

6. Rip currents are strong, narrow currents that flow seaward from a beach through the breakers.[3] These flows are part of the general nearshore circulation of water (i.e., nearshore circulation cells), which consists of an onshore flow toward the breakers, a longshore current in the surf zone, and an offshore flow in relatively strong, narrow rip currents (see fig. 4).

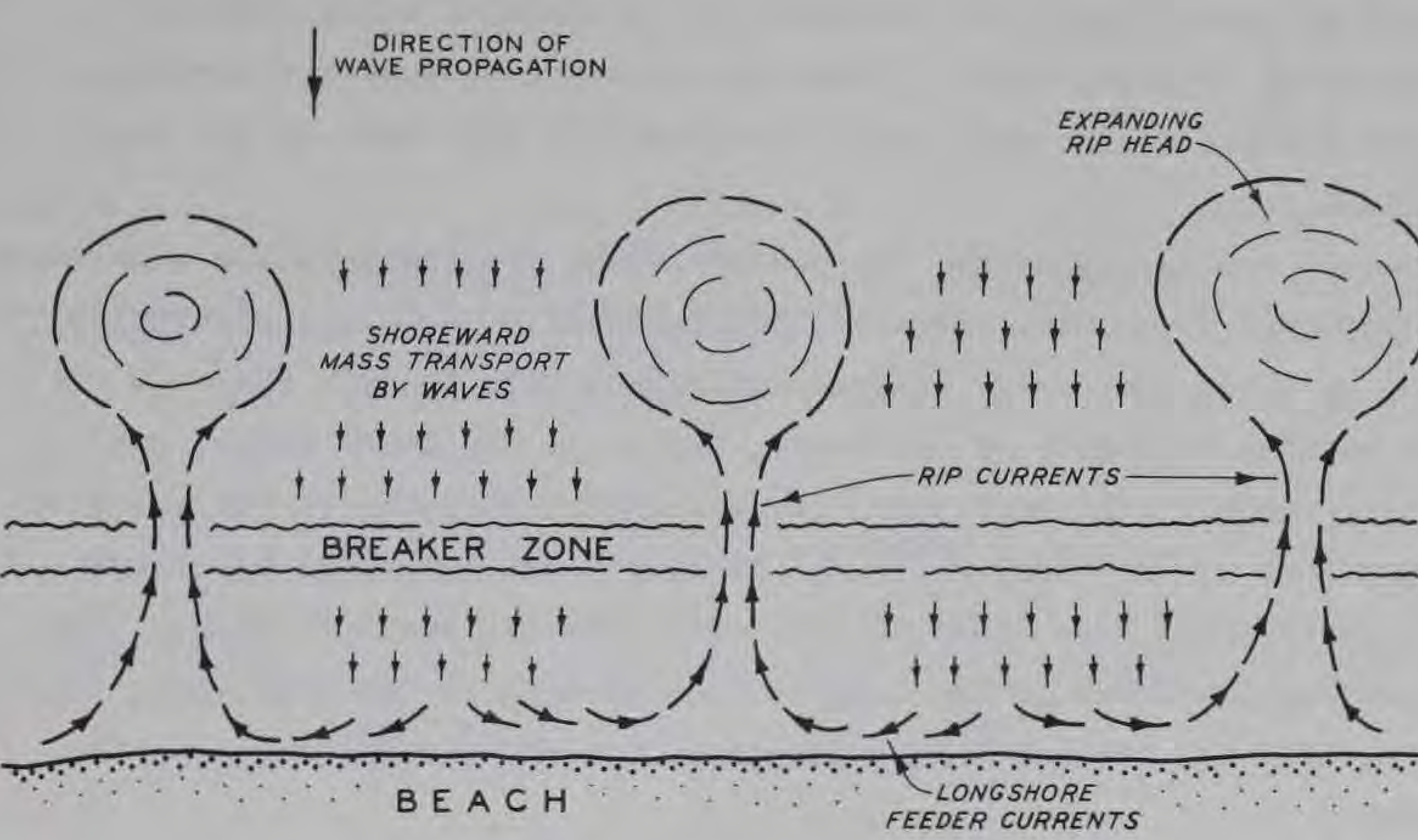

Fig. 4. Nearshore circulation pattern for waves approaching from direction normal to beach

7. If these motions are essentially confined to two dimensions, as in a wave flume, the equilibrium between the incoming waves and the mean water level near the shore can be described theoretically using the concept of radiation stress to describe the second-order effects of the waves. It appears that second-order wave interactions can provide driving terms for a steady flow only inside the surf zone, the

divergence of the radiation stress being zero outside the surf zone. A circulation pattern is thus produced by a longshore variation of the radiation stress in the surf zone. In shallow water, the radiation stress is proportional to the square of the wave height. Therefore, the nearshore circulation is related to the longshore variation in breaker height, with rip currents occurring where the breaker height is lowest.

8. Topographically controlled rip currents occur when a steady longshore variation in wave height is produced by wave refraction or diffraction patterns. In the absence of steady, longshore disturbances, as on a plane beach, the incoming waves generate a set of edge waves of their own frequency.[4,5] These edge waves interact with the incoming waves to give a longshore variation in the breaker height that is stationary in time. Fig. 5 shows the interaction near the breaking point between an incoming wave and a standing edge wave of the same period.

9. An incoming wave may interact with all the possible edge waves of the same frequency. However, the interaction with one particular mode is often dominant. The parameter $\left(\omega^2 x_b\right)/\left(g \tan \beta\right)$, where ω is the angular frequency of the waves, $\tan \beta$ is the beach slope, and x_b is the width of the surf zone, gives a useful estimate of the relative importance of the modes. If x_b is large, the longshore wavelength of the predominant mode tends to be large. The rip currents have a longshore spacing equal to the longshore wavelength of the edge wave; therefore, when the breaker height is large, the rip currents will be far apart.

10. Using shallow-water theory, Eckart[6] has shown that an edge wave with a longshore wave number λ of $2\pi/(\text{wave length})$ and a radian frequency ω of $2\pi/(\text{wave period})$ has a velocity potential $\emptyset$ which is given by

$$\emptyset = \frac{ga_n}{\omega} \cos \lambda Y \left(L_n(2\lambda X)e^{-\lambda X} \cos \omega t\right) \tag{1}$$

where

g = gravitational constant

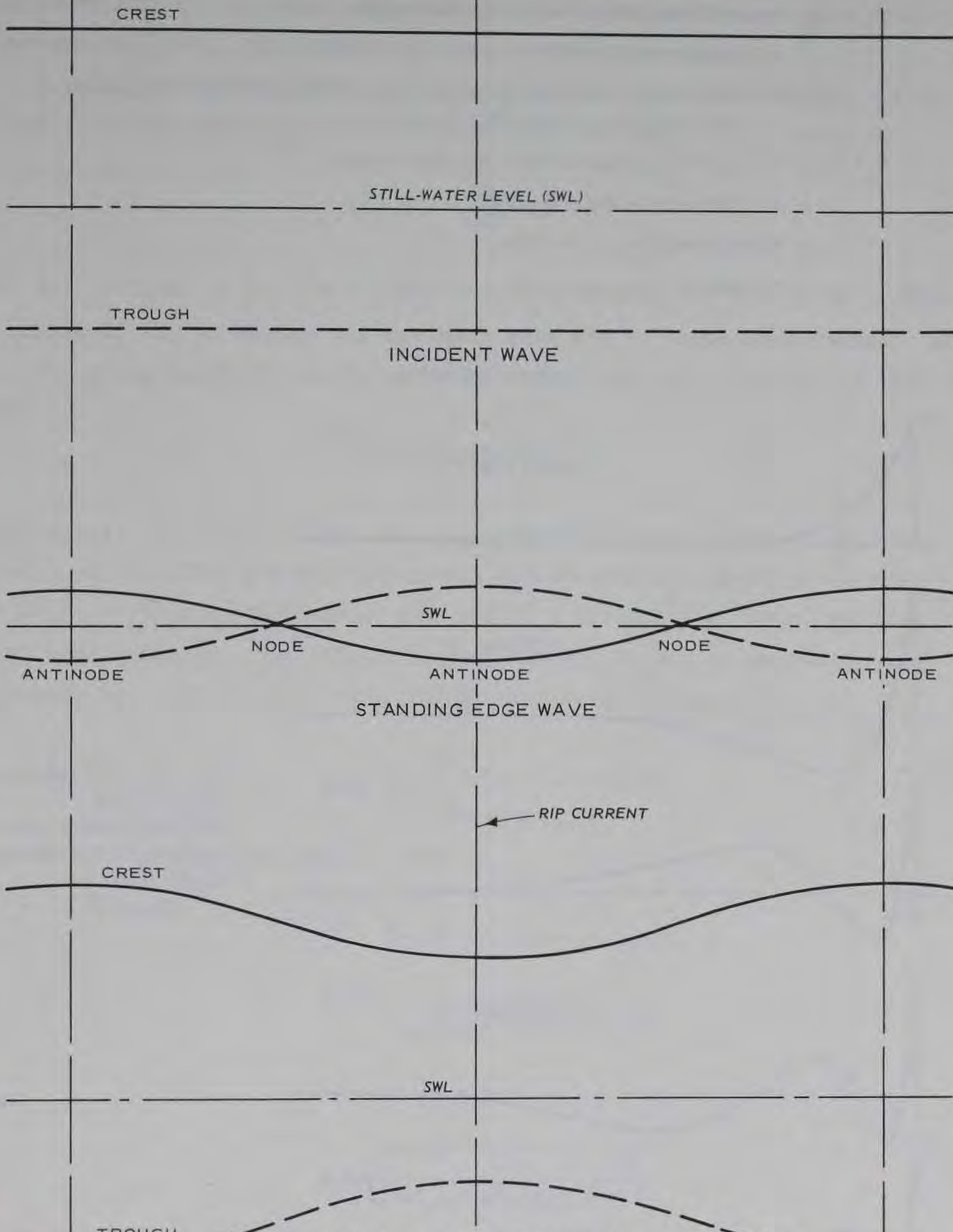

Fig. 5. Interaction near breaking point between incoming wave and standing edge wave of same period (after Bowen and Inman[4])

a_n = wave amplitude at the shoreline ($x = 0$)

$\omega^2 = g\lambda(2n + 1) \tan \beta$

X,Y = horizontal coordinates in the offshore and longshore directions, respectively

$L_n(2\lambda X)$ = Laguerre polynomial of the order n

n = offshore modal number

e = exponential function

t = time

The offshore dependence of the wave form for low values of n is shown in fig. 6. Since n is the number of zeros of the function in an

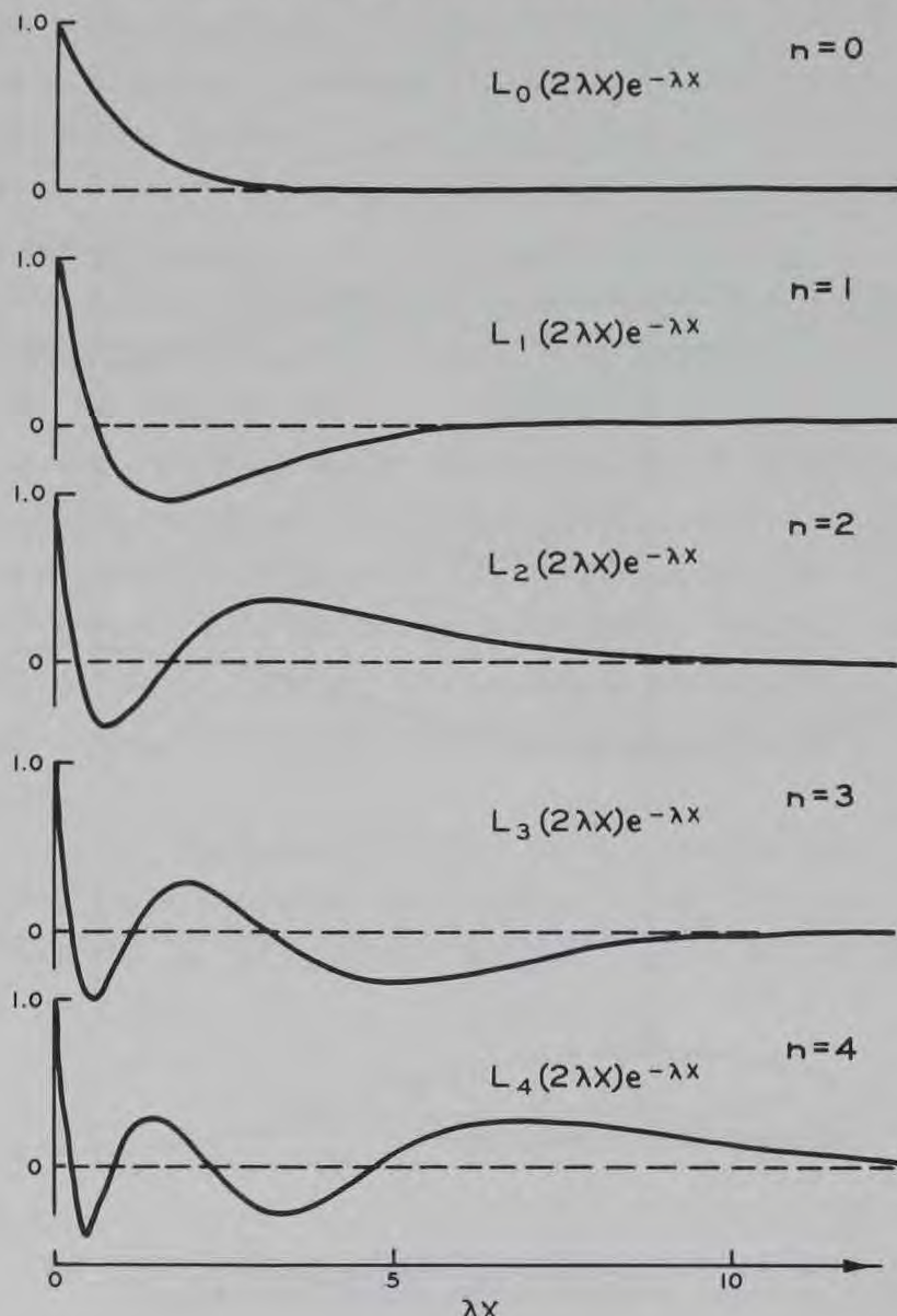

Fig. 6. Offshore dependence of shallow-water solution for edge waves (after Eckart[6])

offshore direction, for higher values of n the curves are more oscillatory, and their amplitudes decrease seaward more slowly.

11. In a model, the solid walls provide an extra boundary condition. At each wall, the longshore orbital velocity v is given by

$$v(X,Y) = \frac{\partial \phi}{\partial Y} \sim \sin \lambda Y$$

If one barrier is at $Y = 0$ and the other barrier is at $Y = b$, then

$$v(X,0) = 0$$

and

$$v(X,b) \sim \sin \lambda b = 0$$

Therefore, $\lambda b = m\pi$, where m is a positive integer and b is the model width. The possible longshore wavelengths are given by $L = 2\pi/\lambda = 2b/m$. The edge waves that can occur in this region therefore have two modal numbers: an offshore modal number n and a longshore modal number m. If the wavelength of an edge wave is given by

$$L = \frac{g}{2\pi} T^2 \sin [(2n + 1)\beta] \tag{2}$$

where T is the wave period, then

$$\frac{2b}{m} = \frac{g}{2\pi} T^2 \sin [(2n + 1)\beta]$$

$$T^2 = \frac{2b(2\pi)}{mg \sin [(2n + 1)\beta]}$$

and

$$T = \sqrt{\frac{4\pi b}{mg \sin [(2n + 1)\beta]}} \tag{3}$$

The possible resonant periods for a given beach slope, model width, and various values of m and n can therefore be calculated using equation 3.

Model construction

12. The rip current model was molded in cement mortar to an undistorted 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 effects.
- b. Absolute size of model waves.
- c. Available shelter dimensions and the area required for constructing the model.
- d. Efficiency of model operation.
- e. Capabilities of available wave-generating and wave-measuring equipment.
- f. Cost of model construction.

Following selection of the linear scale, the model was designed and operated in accordance with Froude's model law.[7] The scale relations used for design and operation of the model were as follows:

Characteristic	Dimension*	Scale Relation (Model:Prototype)
Length	L	$L_r = 1:100$
Area	L^2	$A_r = L_r^2 = 1:10,000$
Volume	L^3	$\forall_r = L_r^3 = 1:1,000,000$
Time	T	$T_r = L_r^{1/2} = 1:10$
Velocity	L/T	$V_r = L_r^{1/2} = 1:10$

* Dimensions are in terms of length and time.

The model reproduced a typical 6040-ft-wide section of the existing beach and underwater contours to an offshore depth of 60 ft. The total area reproduced in the model was approximately 5000 sq ft representing about 1.8 square miles in nature. Waves in the model were generated by a 60.4-ft-long wave machine.

The Movable-Bed Model

13. Scale models of hydraulic phenomena are essentially a means

of replacing the analytical integration of the differential equations governing the processes, including very complicated initial and boundary conditions.[8] Before reliable information can be derived from scale models however, the physical laws which cause the processes must be understood so that the relative magnitudes of the forces involved will remain the same. Unfortunately, in many fluid phenomena, the processes are inherently complicated by nonlinear and turbulent effects and by boundary conditions. The governing differential equations are generally not known; hence, the estimations of the ratios of the forces involved may be of unknown reliability. Thus, the investigator confronted with a physical problem must attempt to build a model which reproduces the dominant processes, with the hope that the influence of the other forces is small. In some instances, when it is not feasible or possible to reproduce all dominant processes, an attempt can be made to reproduce some other parameter, such as bottom evolution; however, extreme care must be taken in such cases to ensure that valid data are acquired for the proposed plans. Movable-bed models of the coastal zone involve problems in which caution in judgment must be exercised.

14. A movable-bed scale model study guided by proper similitude relations and procedures can offer quantitative results which are vitally important in seeking an efficient engineering solution. In addition, by observing the display of the model, an investigator can develop a more concrete feeling of the nature of the problems than might otherwise be achieved.

15. A movable-bed scale model must fulfill the following conditions:

- a. It must be exact; i.e., it must reproduce with exactness the natural phenomenon under study.
- b. It must be consistent; i.e., it must always give reproducible results under the same conditions.
- c. It must be sensitive enough to reproduce the phenomenon under investigation.
- d. It must be economical, of reasonable size, and capable of being completed within a reasonable time interval.

In a movable-bed scale model, the basic similitude requirement is the reproduction of bottom evolution observed in the field, even if it is not achieved through exact similitude of water motion.[8]

16. The perched beach movable-bed model was designed in accordance with the scaling relations of Noda,[9] which indicate a relationship or model law among the four basic scale ratios; i.e., the horizontal scale, the vertical scale, the sediment size ratio, and the relative specific-weight ratio (see fig. 7). These relations were determined experimentally using a wide range of wave conditions and beach materials and are valid mainly for the breaker zone.

Selection of model beach material

17. A search was made of all readily available beach materials, and possible model scales based on the characteristics of these materials were computed. Preliminary model tests were conducted using polystyrene (specific weight = 1.05); but this material proved to be too light, and serious operational problems were encountered. Some of the particles floated on the surface, and some were held in suspension by the waves. The polystyrene material absorbed a large percentage of the wave energy so that the waves never broke. They simply disappeared into a "mush" at the shoreline. Use of the polystyrene was therefore abandoned, and a quantity of crushed coal (specific weight = 1.30) was obtained (see fig. 8).

Selection of model scales

18. Using the scaling relations of Noda[9] and the physical characteristics of the prototype sand and the available crushed coal, the model scales were computed as follows:

a. Ratio of apparent specific weights.

$\gamma_w = 1.00$ = specific weight of water

$\gamma_s = 2.65$ = specific weight of prototype sand

and

$\gamma_c = 1.30$ = specific weight of coal

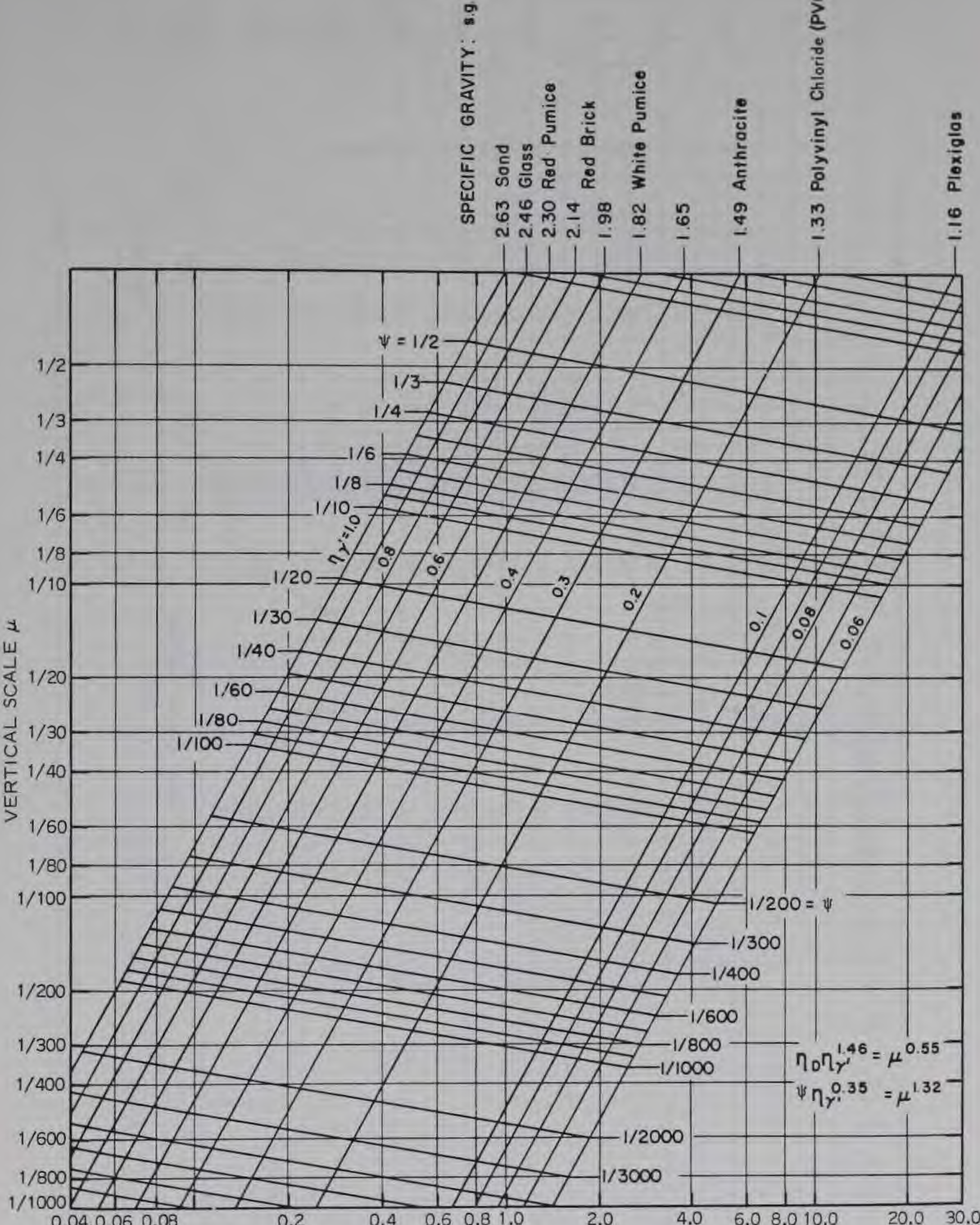

Fig. 7. Graphic representation of model law (after Noda[9])

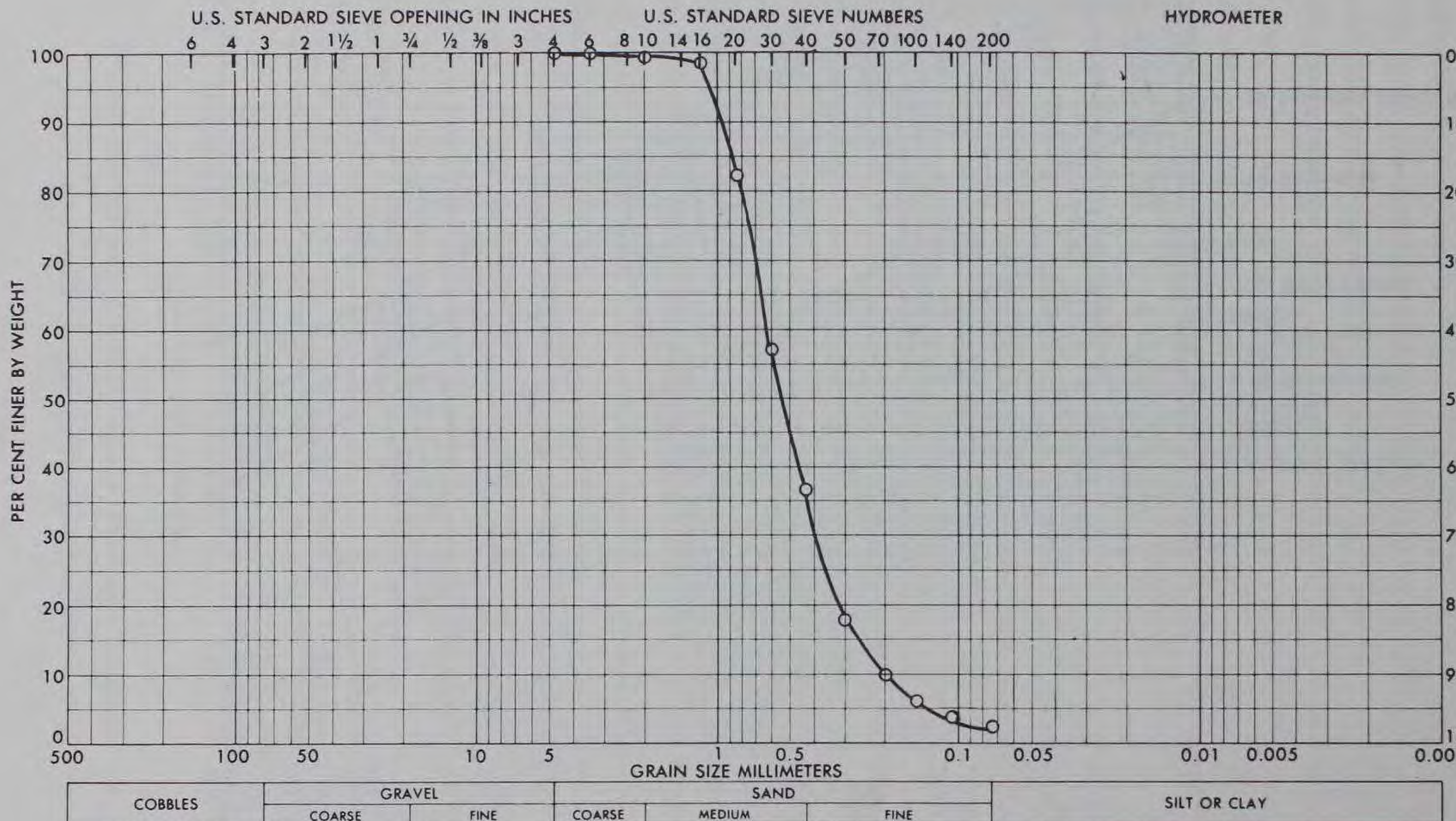

Fig. 8. Grain-size distribution for crushed coal used in movable-bed tests

Now, the apparent specific weight of prototype sand

$$\gamma_s' = \frac{\gamma_s - \gamma_w}{\gamma_w} = \frac{2.65 - 1}{1} = 1.65$$

and the apparent specific weight of coal

$$\gamma_c' = \frac{\gamma_c - \gamma_w}{\gamma_w} = \frac{1.30 - 1}{1} = 0.30$$

Thus, the ratio of apparent specific weights

$$\eta_\gamma' = \frac{\gamma_c'}{\gamma_s'} = \frac{0.30}{1.65} = 0.182$$

b. Ratio of median particle diameters.

D_{proto} = 0.40 mm = median diameter of prototype sand[10]

and

D_{model} = 0.55 mm = median diameter of available coal

Thus, the ratio of median particle diameters

$$\eta_D = \frac{D_{model}}{D_{proto}} = \frac{0.55}{0.40} = 1.375$$

c. Model scales and distortion.

ψ = horizontal scale

and

μ = vertical scale

$$\mu^{0.55} = \eta_D \eta_\gamma'^{1.46} = 1.375\ (0.182)^{1.46}$$

$\mu = 0.0192 = 1/52$, say $1/50$

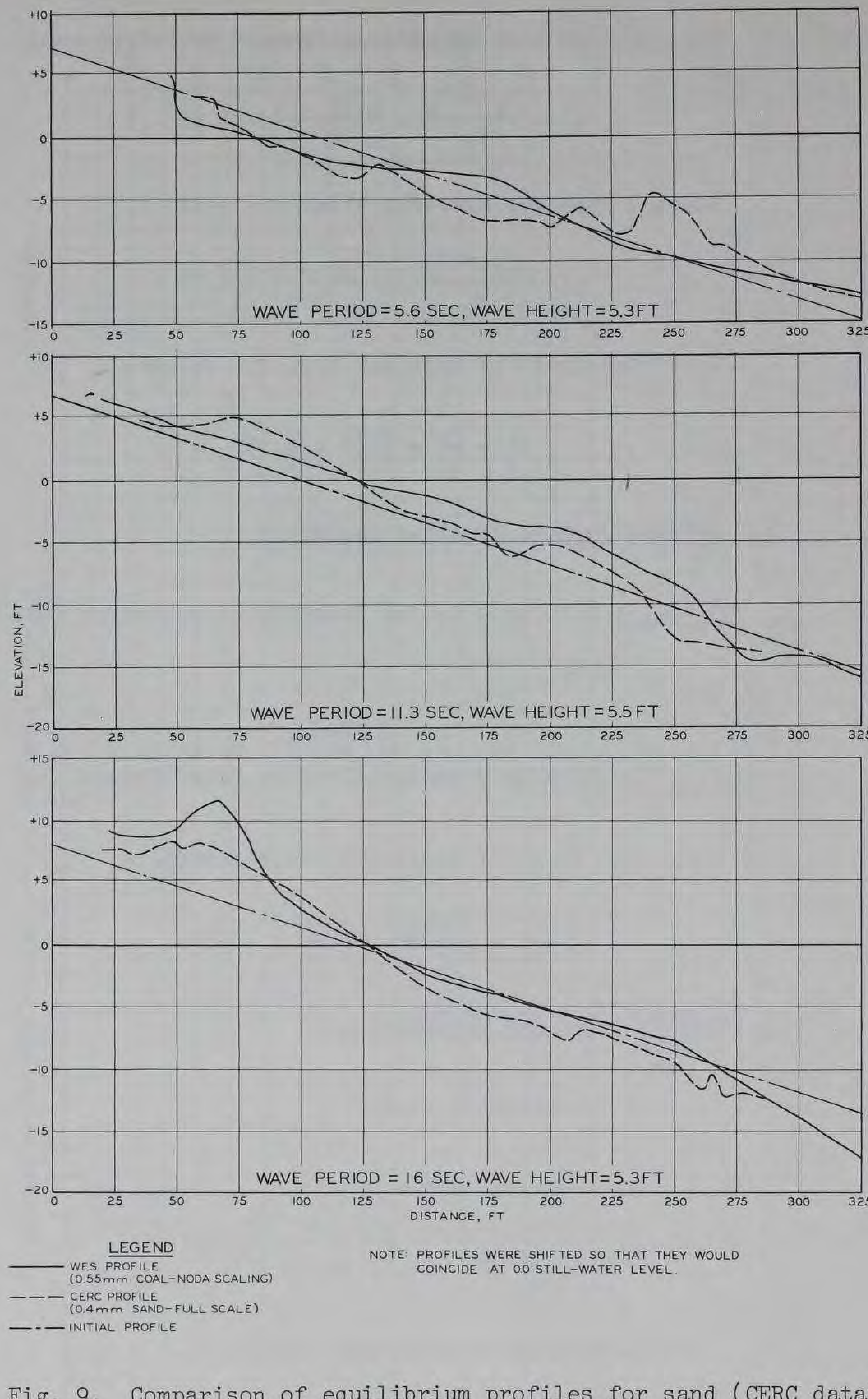

Fig. 9. Comparison of equilibrium profiles for sand (CERC data) and coal (WES data)

$$\psi = \mu^{1.32}\, \eta_{\gamma}'^{-0.35} = (0.0192)^{1.32}(0.182)^{-0.35}$$

$$= 0.0098 = 1/102, \text{ say } 1/100$$

Thus, the model distortion using coal

$$\Omega = \frac{1/50}{1/100} = 2$$

To check the validity of the computed scales, equilibrium profile tests were conducted using coal as the beach material. The test results were then compared with data from full-scale tests conducted at the U. S. Army Coastal Engineering Research Center (CERC) using natural sand as the beach material (fig. 9). While these data are not in exact agreement, they agree more closely than data for any of the other scaling relations tried. The scales computed using Noda's scaling relations were therefore considered adequate for the movable-bed tests.

19. The two-dimensional, movable-bed model was constructed in a 2-ft-wide, 4.5-ft-deep, 148-ft-long wave flume. A flap-type wave generator capable of generating waves with the required characteristics was positioned at the seaward end of the flume. Using coal as the beach material and the scales computed in paragraph 18, average beach slopes were constructed between elevations of +20 and -65 ft. These average beach slopes were taken from a 1961 survey by the Los Angeles District.

The Stability Model

20. Stability tests were conducted first on section models of the submerged toe structure and later on the stone riprap apron. These tests were conducted in a concrete flume that was 119 ft long, 5 ft wide, and 4 ft deep, with a vertical-displacement-type wave generator installed at one end and the test section installed at the other end. The flume bottom seaward of the test sections was molded in concrete to an average slope of 1:170, as dictated by the prototype contours. Test sections of the toe structure and riprap apron were constructed of

limestone rock at a linear scale of 1:30. Selection of this model scale was based on the size of the model armor units available as compared with the estimated size of the prototype armor required for stability, the depth of water to be tested, and the capabilities of the wave generator. Based on Froude's model law and a linear scale of 1:30, the following model-to-prototype scale relations were derived:

Characteristic	Dimension*	Scale Relation (Model:Prototype)
Length	L	$L_r = 1:30$
Area	L^2	$A_r = L_r^2 = 1:900$
Volume	L^3	$\forall_r = L_r^3 = 1:27,000$
Time	T	$T_r = L_r^{1/2} = 1:5.48$
Velocity	L/T	$V_r = L_r^{1/2} = 1:5.48$

* Dimensions are in terms of length and time.

21. The specific weight of the water used in the model was 62.4 pcf, and that of sea water is approximately 64.0 pcf. Also, the specific weight of some of the model stone was not the same as that of the stone to be used in the prototype. Thus, the relations between these variables, model to prototype, were determined from the following transference equation:

$$\frac{(W_r)_m}{(W_r)_p} = \frac{(\gamma_r)_m}{(\gamma_r)_p}\left(\frac{L_m}{L_p}\right)\left[\frac{(S_r)_p - 1}{(S_r)_m - 1}\right]^3$$

where subscripts m and p refer to model and prototype quantities, respectively, W_r is the weight of the individual stones in pounds, γ_r is the specific weight of the rock in pounds per cubic foot, L_m/L_p is the scale of the model, and S_r is the specific gravity of the stones relative to the water in which the stones are placed; i.e., $S_r = \gamma_r/\gamma_w$, where γ_w is the specific weight of water in pounds per cubic foot.

22. Most of the stones used in the model for the armor layers were handpicked from specially sieved stone to conform to a somewhat rectangular shape and individually weighed to conform to a given weight. While the shape and surface characteristics of stone that would be used in the prototype were not known, it was assumed that the armor layers would be composed of individual stones approximately rectangular in shape and that the underlayers and apron would be composed of different sizes of quarry-run material. The smallest sizes of stone used in the model were crushed limestone and basalt, which were sized by sieving.

23. The weights and specific weights of the stone used in the model tests and their corresponding prototype equivalents, based on the scale relations listed in paragraph 20 and the transference equation in paragraph 21, were:

Material	Weight, lb Prototype	Weight, lb Model*	Specific Weight pcf Prototype	Specific Weight pcf Model
		Armor Layer Stone		
A	8300	0.25	148	150
B	6600	0.20	148	150
C	3300	0.10	148	150
D	1800	0.05	148	150
E	5000	0.15	148	150
F	1000	0.03	148	150
G	365-1220 (avg 700)	0.02	148	150
H	152-365 (avg 250)	0.008	148	150
		Underlayer Stone		
A2	<100	<0.001	148	165
B2			148	165
C2			148	165
D2			148	165
E2			148	165
F2			148	165

* Model weights are averages of 100-200 pieces of model stone.

Weights of underlayer stone

24. It was first assumed that the weight of the first underlayer would be $W_r/10$, where W_r is the weight of stone in the armor layer, and that the weight of the second underlayer would be $W_r/200$. However, since the physical geometry of the toe structure would not allow space for two layers of the first underlayer, the second underlayer material was used as the core (when needed) and as a blanket material over the bottom. It was observed in the model that none of the small underlayer material leached through the armor stone.

Placement of stone

25. The stability of an armor cover layer is dependent, to some extent, on the method used for placing the stone. In this investigation, all the stones were placed with the flume dewatered. The underlayer stone was dumped from a shovel and smoothed out by hand. The larger armor-layer stone (A, B, C, and E) was initially placed by hand to simulate loose placement in the prototype. However, later in the testing program and before final selection of armor size for a given depth, the sections were constructed by dumping a few stones at a time from a shovel to assure reproduction of the random placement to be used in the prototype.

PART III: TEST CONDITIONS AND PROCEDURES

The Rip Current and Movable-Bed Models

Selection of still-water levels

26. Still-water levels (swl) were selected so that the various wave-induced phenomena that are dependent upon water depths could be accurately reproduced in the models. These phenomena include the refraction of waves, the overtopping of structures by waves, the reflection of wave energy, and the transmission of wave energy through porous structures.

27. Rip current model. One of the primary purposes of the rip current model was to determine the magnitude of rip current velocities over the submerged toe structure. Since current velocities would probably be greater for shallower depths over the toe structure, a model swl of 0.0 ft mllw representing a low-water stage was selected for these tests. As a check of what effect the depth had on current velocities, limited tests were also conducted with a swl of +5.4 ft mllw (representing mean higher high water).

28. Movable-bed model. The primary purpose of the movable-bed model was to determine the amount of beach fill material that would be lost seaward over the toe structure. Since the waves would break farther seaward for lesser depths, probably causing greater losses of material, a model swl of 0.0 ft mllw representing a low-water stage was selected for these tests.

Selection of wave dimensions and directions

29. Factors influencing selection of test wave characteristics. In planning the test program for a model investigation of wave-action problems, it is necessary to select dimensions and directions for the test waves that will afford a realistic test of the proposed improvement plans and allow an accurate evaluation of the elements of the various proposals. Surface-wind waves are generated by the tangential shear stress of the wind blowing along the water surface and the normal

force of the wind against the wave crests. The magnitude of the maximum wave that can be generated by a given storm depends upon 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 the various directions from which waves can attack the problem area.
- b. The frequency of occurrence and durations of storms from the different directions.
- c. The relative geographic position of the problem area.
- d. The alignments, lengths, and locations of various reflecting surfaces.
- e. The refraction of waves caused by differentials in depth in the area seaward of the study area, a factor which may create either a convergence or a divergence of wave energy.

30. Wave refraction. When wind waves move into water of gradually decreasing depth, transformations take place in all wave characteristics except the wave period. The most important transformations with respect to 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 changes in wave height and direction can be determined by plotting refraction diagrams and calculating refraction coefficients. For this study, refraction diagrams were taken from an investigation by the Los Angeles District[2] for representative wave periods for the critical directions of approach. These diagrams were constructed by plotting the positions of wave orthogonals (lines drawn perpendicular to wave crests) from deep water into shallow water. If it is assumed that there is no energy dissipation, that there is no lateral flow of energy, that no energy is reflected by the variable bottom topography, and that nonlinear effects are negligible, the ratio of the wave heights in deep water H_o to the wave heights in shallow water H will be inversely proportional to the square root of the ratio of the corresponding orthogonal spacings s_o and s, or

$H/H_o = K(s_o/s)^{1/2}$. The quantity $(s_o/s)^{1/2}$ is the refraction coefficient, and K is the shoaling coefficient. Thus, the refraction coefficient multiplied by the shoaling coefficient gives a conversion factor for transferring deepwater wave heights to shallow-water wave heights. The shoaling coefficient, which is a function of wavelength and water depth, can be obtained from reference 11.

31. Prototype wave data. Measured wave data on which a comprehensive statistical analysis of wave conditions could be based were unavailable for the Santa Monica Bay area. However, hindcast wave data were secured from deepwater stations 7 (National Marine Consultants)[12] and A (Marine Advisers, Inc.).[13] The locations of these stations are shown in fig. 1. The data prepared by National Marine Consultants were computed in accordance with the theory of wave spectra and statistics, as presented by Pierson, Neumann, and James.[14] The data prepared by Marine Advisers, Inc., were in accordance with Bretschneider's modification of the Sverdrup-Munk theory.[11] Data from deepwater wave stations 7 and A were analyzed to establish the characteristics and estimated duration of deepwater sea and swell approaching the problem area from all directions (clockwise from south to west). The results of this analysis are presented in tables 1 and 2.

32. The deepwater wave data in tables 1 and 2 were converted to shallow-water data for use in the models by application of refraction and shoaling coefficients. The results of this conversion are presented in tables 3 and 4 for a depth of 60 ft and in tables 5 and 6 for a depth of 24 ft. Since in all of the models waves were to be generated from one direction (that normal to the beach), the data in tables 3-6 were combined in tables 7-10 to show the estimated duration and magnitude of shallow-water waves approaching the problem area from all directions combined.

33. Selection of test waves for the rip current model. The data in tables 7 and 8 were combined, and the largest significant wave heights from either station 7 or station A for wave periods from 6 to 19 sec were selected for testing. In addition to the largest significant wave height for each period, somewhat smaller wave heights for

several representative wave periods were also selected. The selected test waves for the rip current model are presented in the following tabulation:

Selected Test Waves (Characteristics at 60-ft Depth)		Selected Test Waves (Characteristics at 60-ft Depth)	
Period sec	Height ft	Period sec	Height ft
6	10	13	10
7	10	13	13
8	13	14	11
9	7	15	11
9	10	16	11
9	13	17	8
10	14	17	11
11	14	18	10
12	13	19	10
13	7		

34. Selection of test waves for the movable-bed model. Again using the data from tables 7 and 8, wave periods ranging from 7 to 16 sec and wave heights ranging from 3 to 14 ft were selected as being representative of those approaching the problem area. Since data were available from tests with the perched beach installed in the large wave tank at CERC for 7.9-sec, 10-ft waves and 11.3-sec, 12-ft waves, these waves were included in the testing program to allow for a comparison of the data. The selected test waves for the movable-bed model are presented in the following tabulation:

Selected Test Waves (Characteristics at 60-ft Depth)	
Period sec	Height ft
7	5
7.9	10
10	3
10	8
10	14
11.3	12
16	4
16	8

The Stability Model

Selection of test conditions

35. Since the scope of the perched beach project involved prototype construction over an extended period, it was assumed that the toe structure would be constructed in stages so that there would be times when there was no beach fill behind the toe structure. Therefore, the structure was to be designed so that it could stand alone. Also, based on the alignment of the perched beach along the shore and the proposed beach fill slopes, the toe structure could be positioned in water depths ranging from 20 to 60 ft, and the depth of beach fill immediately behind the toe structure could vary from 6 to 8 ft. Therefore, to allow for adequate freeboard to keep the beach fill from escaping over the top of the structure and to provide a consistent structure for construction purposes, a constant 10-ft-high structure was selected for testing in the various water depths. The specific water depths in which the toe structure was to be tested were 20, 25, 30, 45, and 60 ft (see fig. 10). Based on a selected swl of 0.0 ft mllw, the crown elevation of a 10-ft-high structure in the above water depths would be -10, -15, -20, -35, and -50 ft, respectively.

36. Based on the prototype wave data presented in tables 1-10, three of the largest significant wave heights from either station 7 or station A were selected for the stability tests. Test waves selected for the various depths are presented in the following tabulation:

Selected Test Waves for Stability Tests

Water Depth ft	Wave Period sec	Maximum Predicted Wave Height* ft	Maximum Nonbreaking Wave Height in Test Flume ft	Selected Test Wave Height ft
20	11		11	11
	13		13	13
	17		8	8

(Continued)

* Based on tables 1-10.

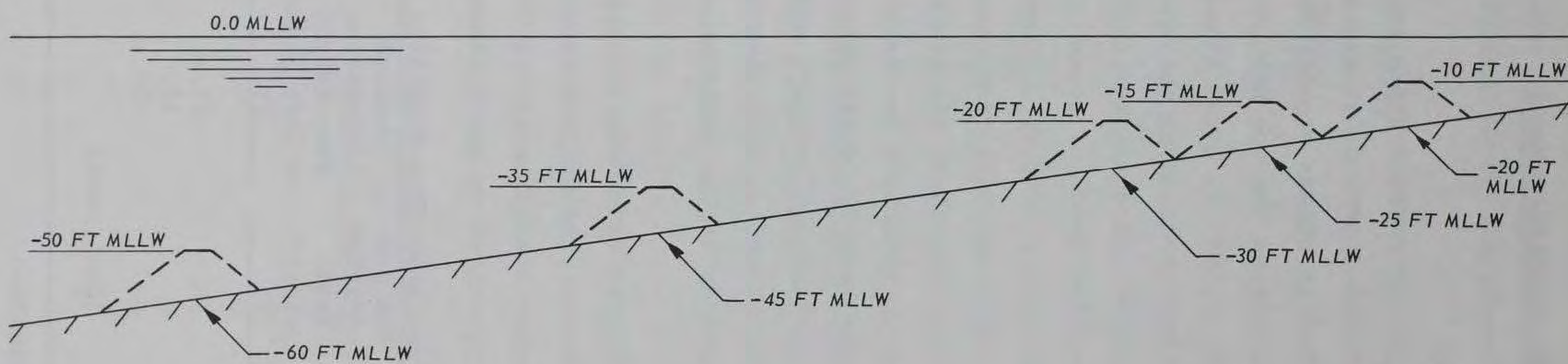

Fig. 10. Proposed test positions for toe structure stability tests

Selected Test Waves for Stability Tests

Water Depth ft	Wave Period sec	Maximum Predicted Wave Height ft	Maximum Nonbreaking Wave Height in Test Flume ft	Selected Test Wave Height ft
25	11	14	13.5	13.5
	13	14	14	14
	17	12	12	12
30	11	14	13.5	13.5
	13	14	14	14
	17	12	12	12
45	11	14	13.5	13.5
	13	14	14	14
	17	12	12	12
60	11	14	13.5	13.5
	13	14	14	14
	17	12	12	12

Since tables 1-10 only convert deepwater waves to shallow-water waves for depths down to 24 ft, the model was used to indicate the maximum nonbreaking wave that could occur in the 20-ft depth. Also, 11-sec waves higher than 13.5 ft were unstable in the model and broke due to wave steepness. A 13.5-ft wave height was therefore selected for testing with the 11-sec wave period.

37. It was anticipated that the most critical waves on the toe structure at the 20-ft depth would be breaking rather than nonbreaking; therefore, tests were conducted on one of the critically stable toe structures to determine the most severe wave effect. It was found that the nonbreaking waves caused slightly more damage to the structure than the breaking waves; thus the nonbreaking waves were selected for testing in the 20-ft depth. It was surmised that the water over the submerged structure acted somewhat as a cushion for the breaking wave condition.

38. Since it was desirable to use a minimum of prototype material, the test sections selected generally had a front slope of 1:1.5, a backslope of 1:1.25, and a crown width of two stone thicknesses.

Test procedures

39. A typical stability test consisted of subjecting a given test section, installed at one end of the wave flume in a given water depth, to attack by test waves of specific height and period for a duration of at least 35 min (prototype) or until damage to the test section stabilized. If no damage to the test section occurred during the 35 min, the section was subjected to the next test wave until the structure was determined to be stable or unstable (for any one or all of the test wave conditions at that depth). If no damage occurred to a given plan at a selected depth, the flume was dewatered and that particular plan was reconstructed before testing at another water depth. Even though this was a time-consuming procedure, it was often easier to rebuild the test sections and change water depths than to move new material into the flume for a given water depth. The behavior of the test sections during wave attack, including the extent of damage, was determined by visual observation. Test waves did at times remove a few loose stones without causing significant damage.

40. Wave heights were measured without the test sections in place with electrical wave height gages in conjunction with a recording oscillograph. Photos were made of all stability test sections that provided significant results, and 14 stability photos are presented herein.

PART IV: TESTS AND RESULTS

Description of Tests

Rip current model

41. Comprehensive tests to determine the location, spacing, and magnitude of rip currents were conducted with the existing beach slopes installed (plate 1) and with the perched beach installed for the test waves listed in paragraph 33. The perched beach was 700 ft wide (measured from the toe structure center line to the 0.0-ft-mllw contour) and the submerged toe structure was located at the -25-ft contour with a crown elevation of -15 ft (plate 2).

42. Initial tests were conducted in the following manner. Dye was introduced at the shoreline across the full width of the model (60.4 ft). The wave generator was then started, and the movement of the dye relative to the model grid system was recorded by a series of time-lapse photos. Since the time interval between successive photos was known, the current velocities were computed directly from the photos.

43. Upon examination of the initial test results and after discussions with Dr. D. L. Inman, Consultant for the California DOH, it was decided to change the testing procedure. Since edge waves have a longshore mode, it appeared that the beach length could be tuned to give maximum incident wave/edge wave interaction. This tuning could be accomplished by adjusting the width of the model so that the longshore width was an even multiple of the preferred edge-wave wavelength. In addition, since the fully developed nearshore circulation might not occur for several minutes after the waves started to break, it was decided to wait several minutes before introducing the dye into the surf zone.

44. Tuning tests were conducted with and without the perched beach installed using the following test waves (selected as being representative and/or most critical from the previous tests):

Wave Period sec	Wave Height ft
9	7
9	13
11	8
11	14
13	7
13	13

For each test wave, the model width was adjusted in either 1/2- or 1-ft increments (depending on the degree of response noted) through one full wavelength for the cutoff mode ($L_c = 5.12T^2$). Since the wavelength of the preferred response mode for the model topography should be less than or equal to the wavelength for the cutoff mode (i.e., $L \leq L_c = 5.12T^2$), an even number of wavelengths of the response mode should have occurred somewhere in the range of widths tested. The dye was injected into the model approximately 3 min (30 min prototype) after the waves started to break and 9 photos were then secured at 6-sec (1 min prototype) intervals.

Movable-bed model

45. The term "base test" as used herein denotes a test performed with existing prototype conditions installed in the model. Movable-bed tests were conducted for base test conditions and for eight variations in design elements of the proposed perched beach. Brief descriptions of the plans are given below, and details are presented in plate 3.

a. Plan 1 consisted of a 700-ft-wide perched beach (measured from the toe structure center line to the 0.0-ft-mllw contour), and the submerged toe structure was located at the -25-ft contour with a crown elevation of -15 ft.

b. Plan 1A was the same as plan 1, with the addition of a 100-ft stone apron installed shoreward of the toe structure.

c. Plan 2 consisted of a 350-ft-wide perched beach, and the submerged toe structure was located at the -22-ft contour with a crown elevation of -10 ft.

d. Plan 2A was the same as plan 2, with the addition of a 100-ft stone apron installed shoreward of the toe structure.

e. Plan 3 consisted of an 1100-ft-wide perched beach, and the submerged toe structure was located at the -28-ft contour with a crown elevation of -20 ft.

f. Plan 3A was the same as plan 3, with the addition of a 100-ft stone apron installed shoreward of the toe structure.

g. Plan 4 consisted of a 700-ft-wide perched beach with no toe structure.

h. Plan 4A consisted of an 1100-ft-wide perched beach with no toe structure.

46. The movable-bed tests were conducted in the following manner. The movable-bed material (coal) was installed in the flume to correspond to one of the plans listed in paragraph 45 (see plate 3). The wave generator was then started and allowed to run continuously until the beach profile had reached an equilibrium condition. The length of time required to reach equilibrium varied from about 3 hr (model time) for the smaller waves to as much as 13 hr (model time) for some of the larger waves. Profiles were measured at regular intervals (usually hourly) to determine the bed evolution. Since initial tests indicated that considerable wave energy was reflected from some of the steeper profiles, a wave filter was installed in front of the flap-type wave generator to dampen rereflected waves.

Stability model (test series 1)

47. Test series 1 was concerned with the design of a 10-ft-high, rubble-mound toe structure in water depths of 20 to 60 ft. The primary purpose of test series 1 was to determine, for the selected design waves, the optimum size stone for toe structure stability in water depths of 20, 25, 30, 45, and 60 ft. The toe structure test plans are designated by three terms. The prefix "T" denotes toe structure test, a number denotes the water depth in which the test was being conducted (1 for 20-ft depth, 2 for 25-ft depth, 3 for 30-ft depth, and 5 for 40-ft depth), and a letter denotes the size of armor stone being tested (see tabulation in paragraph 23.)

Stability model (test series 2)

48. Test series 2 was the second phase of the stability study

in which it was desired to determine, for the selected design waves, the optimum size riprap for the stone apron placed on the beach fill shoreward of the toe structure to prevent material escaping over the toe structure. The stone apron test plans are designated by two terms. The prefix "A" denotes apron tests, and a number denotes a certain toe structure combined with a certain apron material (i.e., 1 denotes 5000-lb stone in the toe structure and 1000-lb stone in the apron, 2 denotes 5000-lb stone in the toe structure and 700-lb stone in the apron, 3 denotes 5000-lb stone in the toe structure and 250-lb stone in the apron, and 4 denotes 3300-lb stone in the toe structure and 250-lb stone in the apron).

Test Results

Rip current model

49. The results of tests to determine the location, spacing, and magnitude of rip currents with the existing beach slopes and with the perched beach installed are presented in tables 11-17. Tables 11-13 present the results of tests using the full model width, initial dye injection, and swl's of 0.0 and +5.4 ft mllw. These data indicate that installation of the perched beach effected an average reduction in rip current velocities of about 20 percent. The maximum velocities observed were 3.0 fps (prototype) with the existing beach slopes installed and 2.8 fps (prototype) with the perched beach installed. A comparison of the test data for swl's of 0.0 and +5.4 ft reveals that the water level had little effect on rip current velocities with the existing beach slopes installed, but, with the perched beach installed, rip current velocities were about 13 percent higher for the lower swl. A swl of 0.0 mllw was therefore used in all subsequent testing.

50. It is difficult to determine any correlation between the number and spacing of rip currents and the various test conditions presented in tables 11-13. Generally speaking, the spacing of the rip currents increased as the wave period increased, but changes in wave height and water level did not produce a clear-cut pattern. During model testing, it was noted that many of the rip current systems were

unstable, and the rips tended to meander or change location with time. For this reason, it was decided that attempts should be made to "tune" the model width as discussed previously (paragraphs 43 and 44). Typical rip current patterns for the full model width using initial dye injection are shown in photos 1-3 with the existing beach slopes installed and in photos 4-6 with the perched beach installed.

51. The results of tests with the width of the rip current model adjusted to search for a "tuned" or resonant condition and using a delayed dye injection are presented in tables 14-16. These data indicate that some of the longshore beach widths were more responsive than others, but no strong resonant conditions were noted. With the existing beach slopes installed (table 14), maximum rip current velocities occurred 200 to 800 ft offshore and reached magnitudes of 7.8 fps (prototype), while, with the perched beach installed (table 15), maximum velocities occurred 200 to 600 ft offshore and reached magnitudes of 5.8 fps (prototype). Maximum currents over the submerged toe structure (700 ft offshore) were 5.0 fps (prototype). Table 17 shows observed surf-zone widths for various test conditions. With the existing beach slopes installed, the waves in many cases started to spill and break a considerable distance offshore (due to the flat bottom slope, currents, etc.) making determination of exact surf-zone widths difficult.

52. Average rip current velocities for the existing beach slopes and the perched beach are compared for each test wave and longshore beach width in table 16. These data show that, in most cases, rip current velocities in the model were higher (an average of about 20 percent) with the existing beach slope installed. Photos 7-10 show typical rip current patterns for various longshore beach widths, with and without the perched beach installed. A review and discussion of the rip current study by Dr. Inman are presented in Appendix A.

Movable-bed model

53. The results of movable-bed tests with the base test and plans 1 through 4A installed are presented in plates 4-11. These data indicate that normal wave action (waves that occur a high percentage of the time) on the perched beach caused no appreciable loss of beach fill but

that for some of the larger storm waves severe loss of the perched beach fill did occur.

54. Tests to determine the effect of a 100-ft stone apron installed shoreward of the toe structure revealed that installation of such a structure in conjunction with a 700-ft perched beach (plan 1A) would significantly reduce the amount of beach fill lost seaward. When installed with a 350-ft perched beach (plan 2A), however, the stone apron had little or no effect on reducing loss of beach fill. When installed with an 1100-ft perched beach (plan 3A), the stone apron had a slightly beneficial effect on reducing the amount of beach fill lost; however, from a comparison of the data for the base test, plan 3, and plan 3A, it appeared questionable whether the toe structure itself was beneficial in this location. Tests were therefore conducted with the toe structure removed (plan 4A), and these test results, when compared with those of plans 3 and 3A, indicated that the toe structure would have little or no beneficial effect on reducing the amount of beach fill lost when located 1100 ft seaward of the 0.0-ft mllw contour. Tests with the toe structure removed for the 700-ft perched beach (plan 4) revealed that the loss of beach fill increased significantly without the toe structure.

55. From a comparison of all the data in plates 4-11, it appears that plan 1A (the 700-ft perched beach, with a toe structure crown elevation of -15 ft and a 100-ft stone apron) would offer the greatest degree of protection to the perched beach fill of any of the plans tested.

56. While the overall test results using coal as a movable-bed material appear satisfactory, there are two inconsistencies which bear further discussion. The first of these is the behavior of the coal in the area above the swl. As can be seen from plates 4-11, the amount of shoreward erosion between the 0.0- and +20-ft contours is inconsistent for several of the plans tested (especially for the larger waves). In addition, in some cases the profile at the shoreward limit of erosion is characterized by a vertical (or possibly undercut) bank as much as 20 ft high. This erosion is probably due to a discrepancy in the

moisture content of the coal above the water line as compared with that of the prototype sand. (The coal appears to be more cohesive in this area. It is also more permeable and therefore can stand on a steeper slope.) This inconsistency should be kept in mind when examining the data in plates 4-11, and a more accurate evaluation can probably be made by considering only those portions of the profiles which are below the swl. The second inconsistency is the formation of a series of bars seaward of the toe structure for the 16-sec test waves (plates 10 and 11). From observations made during model testing, these bars appeared to be related to waves reflected from the toe structure and from the steep profile shoreward of this structure, rather than representing a natural bar formation. As mentioned in paragraph 46, a wave filter was installed in front of the wave generator so rereflected waves or oscillations of the wave basin itself would be negligible. Whether the formation of these bars is a natural condition which would be reproduced in the prototype is questionable, but if so, their magnitude would probably not be proportional to those measured in the model.

Stability model (test series 1)

57. 20-ft depth. Tests were conducted at the 20-ft water depth for plans T1A, T1B, T1C, T1E, and T1E-1. The elements of these plans are shown in plates 12 and 13. Test waves conducted on all the plans for the 20-ft depth are given in paragraph 36. The first few waves of the 13- and 17-sec periods displaced a few loose armor stones from the crown of plans T1A, T1B, T1E, and T1E-1, but no other damage occurred, and these plans were considered stable. Plan T1C, however, was unstable. Several armor stones were displaced from the crown and backslope of plan T1C, the crown was lowered significantly in places, and the remaining armor stone rocked in place as waves passed over the structure. Photos 11 and 12 show plan T1C before and after testing.

58. During all of the tests conducted at the 20- to 30-ft water depths, a slight scour hole developed shoreward of the stone underlayer berm that extends from beneath the toe structure. Although the model structure was constructed on sand which was not to scale, the development of a scour hole indicated qualitatively that scour might occur in

the prototype. Thus, a special test, T1E-1, was conducted in which the shoreward side of the underlayer berm was extended to a width of 10 ft (plate 13) to show the effect of the berm on the scour hole. It was observed that a scour hole still occurred at the end of the berm, but the underlayer stone migrated into the hole and stabilized. Further observations showed that a 5-ft-wide berm also stabilized the scour hole, and it was concluded that the toe structure would not be undermined with a 5-ft-wide berm. At no time during any of the tests was there any indication of scour on the ocean side of the toe structure; therefore, a 5-ft-wide berm should be sufficient on the ocean side as well.

59. The results of the tests at the 20-ft depth indicate that the 5000-lb stone (plan T1E, plate 13) should be used for the toe structure design in 20 ft of water. Photo 13 shows plan T1E after testing with the selected wave conditions.

60. 25-ft depth. Tests were conducted at the 25-ft depth for plans T2A, T2B, T2C, T2C-1, and T2E. Test waves used at the 25-ft depth are given in paragraph 36. Plans T2A, T2B, and T2E were completely stable, except for a few loose stones displaced by the first few waves. A significant number of armor stones were displaced from the crown of plan T2C into the backslope, making a flatter slope. Armor stone left at the crown of plan T2C rocked in place and seemed to be on the verge of toppling; therefore, plan T2C was considered unstable.

61. Since the crown stone of plan T2C (backslope 1:1.25) was displaced and seemed to form a flatter backslope, another test section (plan T2C-1, plate 12) was constructed with a backslope of 1:1.5 to see if flattening the slope would allow the use of smaller armor stone. Test results indicated that, while this structure was not completely stable, it was more stable than plan T2C. A large amount of C-armor stone was again displaced from the crown; thus, the crown elevation of the toe structure was lowered. Considering the close range of armor stone weights being tested, it is not believed that flattening the backslope will offset the use of the next larger stone. Thus, 5000-lb stone (plan T2E, plate 13) should be used for the toe structure in 25 ft

of water. Photo 14 shows plan T2E after testing with the selected wave conditions.

62. 30-ft depth. Plans tested at the 30-ft depth were plans T3C, T3D, and T3E. Test waves used at the 30-ft depth are given in paragraph 36. Plans T3C and T3D (plates 12 and 13) were unstable. Photos 15 and 16 show plans T3C and T3D, respectively, after testing. Several armor stones were displaced during each of these tests leaving high and low places in the crown. Plan T3E (plate 13), however, was completely stable. An after-test view of plan T3E is shown in photo 17. Results of the above tests indicate that the 5000-lb stone should be used to construct the toe structure in 30 ft of water.

63. 45-ft depth. Tests were conducted at a water depth of 45 ft for plans T5C, T5D, and T5F (plates 12 and 13), and test waves for this depth are given in paragraph 36. All of the plans tested at this depth were stable. A few loose stones were displaced during the first few waves for plan T5F, but there was no movement thereafter. Photo 18 shows plan T5F after testing with the selected wave conditions.

64. Armor stones of less than 1000 lb were not tested in the model, because specifications from the prototype quarry indicated that the 1000-lb stone would be well within the quarry-run size. Thus, the above tests and expected prototype stone sizes indicate that the 1000-lb stone can be used for the toe structure in water depths of 45 ft and greater.

65. From all tests conducted in test series 1, it is indicated that 5000-lb armor stone can be used in water depths of up to 35 ft; 3000-lb armor stone, in depths of 35 to 45 ft; and 1000-lb armor stone, in depths greater than 45 ft. The weight of underlayer stone used in the model averaged just under 100 lb (prototype). Thus, quarry stone whose 50 percent weight is about 100 lb should be adequate bed material for the toe structure.

Stability model (test series 2)

66. Stability tests of the stone apron shoreward of the toe structure were conducted for water depths of 20, 25, and 30 ft for plans A1, A2, A3, and A4 (plate 14). At the beginning of the perched

beach model studies, it was planned that the movable-bed model would indicate the shoreward distance behind the toe structure for which the stone apron would need to be tested in the stability model. However, due to an unavoidable delay in the movable-bed model, an apron length of 90 ft (measured shoreward from the center line of the toe structure) was arbitrarily selected so that the stability tests could be continued. Plans A1, A2, and A3 were tested with the 90-ft apron. Plan A1 contained 1000-lb apron stone; plan A2 contained apron stone whose average weight was 700 lb; and plan A3 contained apron stone whose average weight was about 250 lb. Test waves for the various depths are given in paragraph 36. Plans A1, A2, and A3 were all stable for the selected wave conditions and water depths. Photos 19, 20, and 21 show plan A3 after attack by the selected wave conditions at water depths of 20, 25, and 30 ft, respectively. The results of these tests indicate that the 250-lb stone should be suitable apron material for a shoreward distance of 90 ft.

67. It was noted during the tests of plans A1, A2, and A3 that the model test waves did not break on the 90-ft stone apron, but they did break at distances 120 to 240 ft shoreward of the center line of the toe structure, depending upon the water depth being tested. Since it was not known at the time just how far shoreward the stone apron would need to be extended and since the waves were not breaking on the 90-ft apron, plan A4 was tested to determine an adequate stone size if the apron were extended into the breaker zone. Plan A4 consisted of 250-lb stone (average) in the area 90 to 210 ft shoreward of the toe structure, 1000-lb stone in the 90 ft immediately shoreward of the toe structure, and 3300-lb armor stone in the toe structure. The 1000-lb stone was used as a fill material in this particular test due to a shortage of 250-lb stone, and the 3300-lb armor stone was used in the toe structure to see if the beach fill would make any difference in the stability of the toe structure. Results of this test show that an average stone size of 250 lb will provide adequate protection for the beach fill, even if the stone apron is extended into the breaker zone. Photos 22, 23, and 24 show plan A4 after testing with the selected wave

conditions at the 20-, 25-, and 30-ft depths, respectively. Stone sizes less than the 250-lb average were not tested, because this size material was well within the prototype quarry run.

68. Results of Plan A4 also showed that a significant number of 3300-lb armor stones were displaced from the crown of the toe structure, resulting in a slight lowering of the entire crown. In some places, the waves lowered the crown to an elevation just above the elevation of the stone apron. Thus, the test results indicate that 3300-lb armor stone is not adequate to maintain a 10-ft-high toe structure in the 20- to 30-ft range of water depth. This determination is in agreement with the results of test series 1.

PART V: FEASIBILITY OF CONDUCTING A QUANTITATIVE, THREE-DIMENSIONAL, MOVABLE-BED MODEL INVESTIGATION

69. The primary problems associated with conducting a quantitative, three-dimensional, movable-bed model investigation of the stability of the perched beach are understanding the processes involved and developing model laws that include three-dimensional considerations such as reflection, porosity, etc. It is then possible to proceed with the selection of correct movable-bed material (size and specific gravity) and model scales (i.e., with the determination of the applicable scale relations). A review of similitude relations for movable-bed models will not be presented herein, since excellent discussions of the subject are presented in references 8 and 15. An appreciation of the problems involved in such an investigation and the approach that must be taken to this type of study are expressed rather eloquently by Le Méhauté.[16] The remainder of this report is devoted to a discussion of the problems which must be seriously considered and recommendations of an approach to the solution of these problems.

70. It appears, after considerable review and two-dimensional testing, that the scale relations which have been utilized in the past may not be generally applicable for the conduct of a three-dimensional, movable-bed, scale-model investigation. Tests were conducted using practically all scale relations recommended, and, after analysis of the data, it was concluded that Noda's[9] empirical scale relations were the most applicable over the largest portion of the beach profile, especially in the vicinity of the breaker zone. However, it is felt, as has been previously indicated, that these relations are too sensitive to the particle diameter and may not be representative of the true profile in the vicinity of the shoreline. These difficulties could be a consequence of the fact that the data from which these scale relations were derived should not be extrapolated to the extent indicated in fig. 7 (paragraph 16). After termination of the perched beach studies and just prior to publication of this report, Noda[17] published revised scaling

relations in which his empirically derived coefficients were changed as follows:

$$\mu^{0.55} = \eta_D \eta_\gamma'^{1.85} \qquad (\text{formerly} \quad \mu^{0.55} = \eta_D \eta_\gamma'^{1.46})$$

$$\psi = \mu^{1.32} \; \eta_\gamma'^{-0.386} \qquad (\text{formerly} \quad \psi = \mu^{1.32} \; \eta_\gamma'^{-0.35})$$

For the movable-bed model scales of $\mu = 1/50$ and $\psi = 1/100$, these relations indicate that a sediment specific weight of 1.39 (as compared with 1.30 used) and a sediment median diameter of 0.67 mm (as compared with 0.55 mm used) are needed. Assuming that these later scaling relations (1972) are more reliable than those published previously (1971), the movable-bed test results reported herein are somewhat conservative (i.e., there was more beach erosion for the lighter and smaller model material used).

71. Although coal was utilized as the movable-bed material in the flume investigations of this project, it is felt that a more suitable material could be found for additional flume tests or three-dimensional, movable-bed studies. It was known, prior to its use in the present study, that coal has a tendency to form unrealistic ripples or waves for higher velocity oscillatory flow (longer period, larger amplitude waves); however, the wave statistics at the Santa Monica location revealed a relatively mild wave climate, and, since coal was available, it was an expedient choice for the present study. It was found that coal would be a satisfactory material except for the longer period waves. Such waves have a low frequency of occurrence.

72. It is therefore concluded that, prior to performing a three-dimensional, movable-bed model investigation, additional wave-flume tests are needed using materials with a specific gravity range of 1.3 to 1.6. Such tests are needed to aid in the selection of material for a three-dimensional model study. Another problem which must be analyzed during the two-dimensional tests is the size distribution of the model material. If Noda's[9] scale relations are accepted without modification, any appreciable size distribution in the prototype will be practically

impossible to scale correctly in the model (at certain scales), due to the sensitivity of the calculated model scales to variations in particle size. This means that attempts must be made to show either that the size distribution is extremely well sorted or that the particle size distribution in the prototype has a negligible effect on the onshore-offshore transport as well as on the littoral transport. It is believed that the answer to the problem lies somewhere between these two elements. Subsequent to solving the problems discussed above, an evaluation of the feasibility of conducting a quantitative, three-dimensional, movable-bed model investigation can be undertaken.

73. It is well known that quantitative, three-dimensional, movable-bed model investigations are, at best, difficult to conduct, and each area in which such an investigation is contemplated must be carefully analyzed. The following computations and prototype data are considered essential for such investigations:

a. A computation of the littoral transport based on the best available wave statistics.

b. An analysis of the sand size distribution over the entire project area (offshore to a point well beyond the breaker zone).

c. Simultaneous measurement of the following items over a period of erosion and accretion of the shoreline (this measurement period should be judiciously chosen to obtain the maximum probability of both erosion and accretion during as short a time span as possible; a reasonable time period to expect would be from 6 to 12 weeks):

    (1) Continuous measurements of the incident wave characteristics. Such measurements would mean placing enough redundant sensors to obtain accurate estimates of the directional spectrum over the entire project area, and, in addition, would mean conducting a rather sophisticated analysis of all these data.

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

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

74. Upon verification of the model, based on the data acquired, an evaluation can then be made of the effectiveness of various project plans. A quantitative, three-dimensional, movable-bed model investigation of littoral transport appears to be feasible and should be successful (at least for certain areas such as Santa Monica, California), provided it is approached in the manner prescribed and provided adequate prototype data are acquired. A scale-model investigation based on the above philosophy reverts to the category in which there is not necessarily an attempt to reproduce all dominant processes, but rather an attempt is made to reproduce (for verification purposes) the parameter of bottom evolution. In this case, care should be exercised to ensure that installation of various plans does indeed result in valid data. Should there still be any doubts relative to the validity of such an approach, then a small prototype experiment (such as installation of a temporary groin) could be conducted and, if the model reproduced the measured prototype changes, then confidence in the scale model would be justified. Insofar as is known, prototype data to the extent described above have never been acquired; however, it is certainly within the state-of-the-art to obtain such data. Admittedly, the measurement of littoral and onshore-offshore transport will be most difficult, but it is felt that such measurements can be made with sufficient accuracy for model verification. It should be noted that the prototype data acquisition would include measurements of the size and intensity of rip currents in the project area, and, if they persist to any appreciable degree, measurements of the amount of material moved seaward in the rips would be obtained as a part of the overall data.

75. Due to the termination of this project prior to prototype data acquisition and the conduct of a three-dimensional, movable-bed model investigation, the prototype measurement program enumerated above will undoubtedly be expanded upon initiation of such a program. It is anticipated that the essential elements of such a field study have been mentioned.

PART VI: CONCLUSIONS

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

- a. Installation of a 700-ft-wide perched beach in the model had no adverse effect on rip currents and reduced rip current velocities about 20 percent.
- b. Normal wave action (waves expected to occur a high percentage of the time) on the perched beach will cause no appreciable loss of beach fill.
- c. For the larger storm waves of any significant duration, a large seaward loss of the perched beach fill can be expected.
- d. The installation of a 100-ft stone apron in conjunction with a 350-ft-wide (measured from toe structure to 0.0-ft-mllw contour) perched beach (plan 2A) will have little or no effect on the loss of fill material.
- e. The installation of a 100-ft stone apron in conjunction with a 700-ft-wide (measured from toe structure to 0.0-ft-mllw contour) perched beach (plan 1A) will significantly reduce the amount of beach fill lost seaward of the toe structure.
- f. If the beach fill is extended as far as 1100 ft seaward of the 0.0-ft-mllw contour, the toe structure itself will have little or no beneficial effect on reducing the amount of beach fill lost.
- g. Of all plans tested, plan 1A (700-ft perched beach with -15-ft toe structure crown elevation and 100-ft stone apron) appears to offer the greatest degree of protection against seaward loss of beach fill material.
- h. Five-thousand-lb armor stone will be needed if the rubble-mound toe structure is located in water depths ranging from 20 to 35 ft; 3000-lb armor stone will be needed in depths from 35 to 45 ft; and 1000-lb stone will be needed in depths greater than 45 ft.
- i. Quarry-run stone whose 50 percent weight is about 100 lb will be adequate bed material for the toe structure.
- j. Two-hundred-fifty-lb stone will be stable for the stone apron shoreward of the toe structure.
- k. Additional wave flume tests will be needed in the specific gravity range of 1.3 to 1.6 prior to performing a three-dimensional, movable-bed model investigation.

1. Provided adequate prototype data become available for use in model verification, a three-dimensional, movable-bed model investigation appears to be feasible and should result in a valid indication of the relative merits of various project designs.

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Table 1

Estimated Duration and Magnitude of Deepwater Waves (Sea and Swell at Station 7)
Approaching Santa Monica Bay from Various Directions (South to West)

Wave Height ft	Duration, hr/yr, for Various Wave Periods, sec								
	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18+	Total
				South					
1-1.9	32		2						34
2-2.9	53	2							55
3-3.9	12	10	2	2					26
4-4.9		20		2		2			24
5-5.9		2							2
6-6.9		6							6
7-8.9			11						11
9-10.9			6						6
Total	97	40	21	4		2			164
				South-Southwest					
1-1.9									
2-2.9	8	2							10
3-3.9	4	2							6
4-4.9		6							6
5-5.9									
6-6.9		2							2
7-8.9			11						11
9-10.9			2						2
Total	12	12	13						37
				Southwest					
1-1.9	28								28
2-2.9	51	6	4						61
3-3.9	12	14							26
4-4.9		15	2						17
5-5.9		2							2
6-6.9		2							2
7-8.9			4						4
Total	91	39	10						140
				West-Southwest					
1-1.9	9		19						28
2-2.9	25	22	11	10	2				70
3-3.9	6	20	8	2					36
4-4.9		18	8		2				28
5-5.9		4	8	8					20
6-6.9		2	6						8
7-8.9			2	11	4	2			19
9-10.9				3					3
Total	40	66	62	34	8	2			212
				West					
1-1.9	37	2	42	20	2	4			107
2-2.9	65	83	181	93	54	31	10	2	519
3-3.9	15	43	76	44	31	2	8	2	221
4-4.9		41	42	20	8	11	2		124
5-5.9		13	25	28	12	9			87
6-6.9		8	27	28	11	6	4		84
7-8.9			25	28	18	2			73
9-10.9			2	20	14	9			45
11-12.9				15	2	6	2		25
13-14.9				4	8				12
Total	117	190	420	300	160	80	26	4	1297

Table 2

Estimated Duration and Magnitude of Deepwater Waves (Sea and Swell at Station A)

Approaching Santa Monica Bay from Various Directions (South to West)

Wave Height ft	Duration, hr/yr, for Various Wave Periods, sec 2-4	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20	20-22	Total
						South					
0-0.9	70					289	210	97	10	10	686
1-1.9	53			18	9	456	307	123	19	18	1003
2-2.9		26			9	72	62	10		1	180
3-3.9		26			1	10	9	1			47
4-4.9		1				1	9				11
5-5.9			1								1
6-7.9			1					1			2
8-9.9			1								1
10-11.9				1							1
12-13.9				1							1
Total	123	53	3	20	19	828	597	232	29	29	1933
						South-Southwest					
0-0.9	35					201	88	2	19	1	346
1-1.9	9					387	289	44	1		730
2-2.9		9			9	88	105	36	9	1	257
3-3.9		9				9	36	36	10		100
4-4.9		1									1
5-5.9			1								1
6-7.9			1								1
Total	44	19	2		9	685	518	118	39	2	1436
						Southwest					
0-0.9	79			1		122	44	1			247
1-1.9	26			1		517	377	27	10		958
2-2.9		18	1	1		369	298	62	1		750
3-3.9		18			1	70	88	53	2		232
4-4.9		9					9				18
5-5.9			1								1
6-7.9			18						1		19
Total	105	45	20	3	1	1078	816	143	14		2225
						West-Southwest					
0-0.9	44			1							45
1-1.9	26		1		1						28
2-2.9		9				9					18
3-3.9		18					9				27
4-4.9		9				9	10	1			29
5-5.9			1				9				10
6-7.9			18			1	1				20
8-9.9			9						1		10
10-11.9				1							1
12-13.9				1							1
Total	70	36	29	3	1	19	29	1	1		189
						West					
0-0.9	167		27	45	10						249
1-1.9	131		1	9	45	27	9				222
2-2.9		70			9	45	27	10			161
3-3.9		88			1	2	9				100
4-4.9		44				2	9	9			64
5-5.9			18			1		9			28
6-7.9			53		9		2	18	1		83
8-9.9			9					9			18
10-11.9				1					1		2
12-13.9				1							1
Total	298	202	108	56	74	77	56	55	2		928

Table 3

Estimated Duration and Magnitude of Shallow-Water (60-ft Depth) Waves (Sea and Swell at Station 7)

Approaching Santa Monica Bay from Various Directions (South to West)

Wave Height ft	Duration, hr/yr, for Various Wave Periods, sec								
	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18+	Total
				South					
1-1.9	32		2						34
2-2.9	53	2							55
3-3.9	12	10	2	2					26
4-4.9		20		2		2			24
5-5.9		2							2
6-6.9		6							6
7-7.9									
8-8.9			11						11
9-9.9			6						6
Total	97	40	21	4		2			164
				South-Southwest					
1-1.9									
2-2.9	8	2							10
3-3.9	4	2							6
4-4.9		6							6
5-5.9									
6-6.9		2							2
7-7.9									
8-8.9			11						11
9-9.9			2						2
Total	12	12	13						37
				Southwest					
1-1.9	28								28
2-2.9	51	6	4						61
3-3.9	12	14							26
4-4.9		15	2						17
5-5.9		2							2
6-6.9		2							2
7-7.9									
8-8.9			4						4
Total	91	39	10						140
				West-Southwest					
1-1.9	9		19						28
2-2.9	25	22	11	10	2				70
3-3.9	6	20	8	2					36
4-4.9		18	8		2				28
5-5.9		4	8	8					20
6-6.9		2	6						8
7-7.9			2	11	4	2			19
8-8.9									
9-9.9				3					3
Total	40	66	62	34	8	2			212
				West					
1-1.9	37	2	42	20	2	4			107
2-2.9	65	83	181	93	54	31	10	2	519
3-3.9	15	43	76	44	31	2	10	2	223
4-4.9		41	42	20	20	20			143
5-5.9		13	25	28	11	6	4		87
6-6.9		8	27	28					63
7-7.9			25	28	18	2			73
8-8.9					14	9			23
9-9.9			2	20					22
10-10.9					2	6	2		10
11-11.9				15					15
12-12.9					8				8
13-13.9				4					4
Total	117	190	420	300	160	80	26	4	1297

Table 4

Estimated Duration and Magnitude of Shallow-Water (60-ft Depth) Waves (Sea and Swell at Station A) Approaching Santa Monica Bay from Various Directions (South to West)

Wave Height ft	Duration, hr/yr, for Various Wave Periods, sec										
	2-4	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20	20-22	Total
						South					
0-0.9	70					289	210				569
1-1.9	53			18	9	456	307	97	10	10	960
2-2.9		26			9	72	62	123	19	18	329
3-3.9		26			1	10	9	10		1	57
4-4.9		1				1	9	1			12
5-5.9			1								1
6-6.9											
7-7.9			1								1
8-8.9								1			1
9-9.9			1								1
10-10.9				1							1
11-11.9											
12-12.9				1							
Total	123	53	3	20	19	828	597	232	29	29	1933
						South-Southwest					
0-0.9	35					201	88				324
1-1.9	9					387	289	2	19	1	707
2-2.9		9			9	88	105	44	1		256
3-3.9		9				9	36	36	9	1	100
4-4.9		1						36	10		47
5-5.9			1								1
6-6.9											
7-7.9			1								1
Total	44	19	2		9	685	518	118	39	2	1436
						Southwest					
0-0.9	79			1		122	44				246
1-1.9	26			1		517	377	1			922
2-2.9		18	1	1		369	298	27	10		724
3-3.9		18			1	70	88	62	1		240
4-4.9		9					9	53	2		73
5-5.9			1								1
6-6.9											
7-7.9			18								18
8-8.9									1		1
Total	105	45	20	3	1	1078	816	143	14		2225
						West-Southwest					
0-0.9	44			1							45
1-1.9	26		1		1						28
2-2.9		9				9					18
3-3.9		18					9	1			28
4-4.9		9				9	19				37
5-5.9			1								1
6-6.9			18			1	1				20
7-7.9											
8-8.9			9						1		10
9-9.9											
10-10.9				1							1
11-11.9				1							1
Total	70	36	29	3	1	19	29	1	1		189
						West					
0-0.9	167		27	45	10						249
1-1.9	131		1	9	45	27	9				222
2-2.9		70			9	45	27	10			161
3-3.9		88			1	2	9	9			109
4-4.9		44				3	9	9			65
5-5.9			18								18
6-6.9			53		9		2	18	1		83
7-7.9								9			9
8-8.9			9								9
9-9.9									1		1
10-10.9				1							1
11-11.9				1							1
Total	298	202	108	56	74	77	56	55	2		928

Table 5

Estimated Duration and Magnitude of Shallow-Water (24-ft Depth) Waves (Sea and Swell at Station 7) Approaching Santa Monica Bay from Various Directions (South to West)

Wave Height ft	Duration, hr/yr, for Various Wave Periods, sec								
	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18+	Total
South									
1-1.9	32		2						34
2-2.9	53	2							55
3-3.9	12	10	2	2					26
4-4.9		20		2					22
5-5.9		2				2			4
6-6.9		6							6
7-7.9									
8-8.9			11						11
9-9.9									
10-10.9			6						6
Total	97	40	21	4		2			164
South-Southwest									
1-1.9									
2-2.9	8	2							10
3-3.9	4	2							6
4-4.9		6							6
5-5.9									
6-6.9		2							2
7-7.9									
8-8.9			11						11
9-9.9									
10-10.9			2						2
Total	12	12	13						37
Southwest									
1-1.9	28								28
2-2.9	51	6	4						61
3-3.9	12	14							26
4-4.9		15	2						17
5-5.9		2							2
6-6.9		2							2
7-7.9									
8-8.9			4						4
Total	91	39	10						140
West-Southwest									
1-1.9	9		19						28
2-2.9	25	22	11	10	2				70
3-3.9	6	20	8	2					36
4-4.9		22	8		2				32
5-5.9		2	8	8					18
6-6.9			6						6
7-7.9			2						2
8-8.9				11	4	2			17
9-9.9									
10-10.9				3					3
Total	40	66	62	34	8	2			212
West									
1-1.9	37	2	42	20	2	4			107
2-2.9	65	83	181	93	54	31	10	2	519
3-3.9	15	43	76	44	31	2	8	2	221
4-4.9		54	42	20	8	11	2		137
5-5.9		8	25	28	12	9			82
6-6.9			27	28	11	6	4		76
7-7.9			25						25
8-8.9				28	18	2			48
9-9.9			2		14				16
10-10.9				20		9			29
11-11.9					2	6	2		10
12-12.9				15					15
13-13.9				4	8				12
Total	117	190	420	300	160	80	26	4	1297

Table 6

Estimated Duration and Magnitude of Shallow-Water (24-ft Depth) Waves (Sea and Swell at Station A) Approaching Santa Monica Bay from Various Directions (South to West)

Wave Height ft	Duration, hr/yr, for Various Wave Periods, sec 2-4	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20	20-22	Total
						South					
0-0.9	70										70
1-1.9	53			18	9	289	210	97	10	10	696
2-2.9		26			9	456	307	123	19	18	958
3-3.9		26			1	72	62	10		1	172
4-4.9		1				10	9	1			21
5-5.9			1			1	9				11
6-6.9											
7-7.9			1								1
8-8.9			1								1
9-9.9								1			1
10-10.9											
11-11.9				1							1
12-12.9				1							1
Total	123	53	3	20	19	828	597	232	29	29	1933
						South-Southwest					
0-0.9	35										35
1-1.9	9					201	88	2	19	1	320
2-2.9		9				387	289	44	1		730
3-3.9		9			9	88	105	36	9	1	257
4-4.9		1				9	36	36			82
5-5.9			1						10		11
6-6.9											
7-7.9			1								1
Total	44	19	2		9	685	518	118	39	2	1436
						Southwest					
0-0.9	79			1							80
1-1.9	26			1		122	44	1			194
2-2.9		18	1	1		517	377	27	10		951
3-3.9		18				369	298	62	1		748
4-4.9		9			1	70	88	53	2		223
5-5.9			1				9				10
6-6.9											
7-7.9			18								18
8-8.9											
9-9.9									1		1
Total	105	45	20	3	1	1078	816	143	14		2225
						West-Southwest					
0-0.9	44			1							45
1-1.9	26		1		1						28
2-2.9		9				9					18
3-3.9		27					9				36
4-4.9			1			9	10	1			21
5-5.9							9				9
6-6.9			18								18
7-7.9						1	1				2
8-8.9			9								9
9-9.9									1		1
10-10.9				1							1
11-11.9				1							1
Total	70	36	29	3	1	19	29	1	1		189
						West					
0-0.9	167		27	45	10						249
1-1.9	131		1	9	45	27	9				222
2-2.9		70			9	45	27	10			161
3-3.9		132			1	2	9				144
4-4.9			18			2	9	9			38
5-5.9						1		9			10
6-6.9			53								53
7-7.9					9		2	18	1		30
8-8.9			9					9			18
9-9.9											
10-10.9				1					1		2
11-11.9				1							1
Total	298	202	108	56	74	77	56	55	2		928

Table 7

Estimated Duration and Magnitude of Shallow-Water (60-ft Depth) Waves (Sea and Swell at Station 7) Approaching Santa Monica Bay from All Directions (South to West)

Wave Height ft	Duration, hr/yr, for Various Wave Periods, sec								
	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18+	Total
1-1.9	106	2	63	20	2	4			197
2-2.9	202	115	196	103	56	31	10	2	715
3-3.9	49	89	86	48	31	2	10	2	317
4-4.9		100	52	22	22	22			218
5-5.9		21	33	36	11	6	4		111
6-6.9		20	33	28					81
7-7.9			27	39	22	4			92
8-8.9			26		14	9			49
9-9.9			10	23					33
10-10.9					2	6	2		10
11-11.9				15					15
12-12.9					8				8
13-13.9				4					4
Total	357	347	526	338	168	84	26	4	1850

Table 8

Estimated Duration and Magnitude of Shallow-Water (60-ft Depth) Waves (Sea and Swell at Station A) Approaching Santa Monica Bay from All Directions (South to West)

Wave Height ft	Duration, hr/yr, for the Various Wave Periods, sec										
	2-4	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20	20-22	Total
0-0.9	395		27	47	10	612	342				1433
1-1.9	245		2	28	55	1387	982	100	29	11	2839
2-2.9		132	1	1	27	583	492	204	30	18	1488
3-3.9		159			3	91	151	118	10	2	534
4-4.9		64				13	46	99	12		234
5-5.9			22								22
6-6.9			71		9	1	3	18	1		103
7-7.9			20					9			29
8-8.9			18					1	2		21
9-9.9			1						1		2
10-10.9				3							3
11-11.9				2							2
12-12.9				1							1
Total	640	355	162	82	104	2687	2016	549	85	31	6711

Table 9

Estimated Duration and Magnitude of Shallow-Water (24-ft Depth) Waves (Sea and Swell at Station 7) Approaching Santa Monica Bay from All Directions (South to West)

Wave Height ft	Duration, hr/yr, for Various Wave Periods, sec								
	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18+	Total
1-1.9	106	2	63	20	2	4			197
2-2.9	202	115	196	103	56	31	10	2	715
3-3.9	49	89	86	48	31	2	8	2	315
4-4.9		117	52	22	10	11	2		214
5-5.9		14	33	36	12	11			106
6-6.9		10	33	28	11	6	4		92
7-7.9			27						27
8-8.9			26	39	22	4			91
9-9.9			2		14				16
10-10.9			8	23		9			40
11-11.9					2	6	2		10
12-12.9				15					15
13-13.9				4	8				12
Total	357	347	526	338	168	84	26	4	1850

Table 10

Estimated Duration and Magnitude of Shallow-Water (24-ft Depth) Waves (Sea and Swell at Station A) Approaching Santa Monica Bay from All Directions (South to West)

Wave Height ft	Duration, hr/yr, for the Various Wave Periods, sec										
	2-4	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20	20-22	Total
0-0.9	395		27	47	10						479
1-1.9	245		2	28	55	639	351	100	29	11	1460
2-2.9		132	1	1	18	1414	1000	204	30	18	2818
3-3.9		212			11	531	483	108	10	2	1357
4-4.9		11	19		1	100	152	100	2		385
5-5.9			3			2	27	9	10		51
6-6.9			71								71
7-7.9			20		9	1	3	18	1		52
8-8.9			19					9			28
9-9.9								1	2		3
10-10.9				2					1		3
11-11.9				3							3
12-12.9				1							1
Total	640	355	162	82	104	2687	2016	549	85	31	6711

Table 11

Rip Current Observations for Full Model Width, Existing Conditions Installed

Test Wave		Still-water	Characteristics of Rip Currents Prototype Dimensions			
Period sec	Height ft	Level, ft mllw	Number Observed	Average Spacing ft	Average* Velocity fps	Maximum** Velocity fps
6	10	0.0	14	450	1.3	1.7
7	10	0.0	13	470	1.2	1.6
8	13	0.0	14	450	1.7	2.0
9	13	0.0	9	660	1.9	2.4
10	14	0.0	9	680	1.9	3.0
11	14	0.0	6	980	1.8	2.4
12	13	0.0	7	920	2.1	2.6
13	13	0.0	6	1050	2.1	2.2
14	11	0.0	5	1250	2.3	2.7
15	11	0.0	7	1000	1.4	1.7
16	11	0.0	6	1120	1.4	1.9
17	11	0.0	7	1000	1.0	1.3
18	10	0.0	6	1200	1.0	1.8
19	10	0.0	6	1120	1.1	1.4
7	10	+5.4	7	850	1.3	1.7
9	13	+5.4	9	690	1.2	1.5
11	14	+5.4	6	1060	1.5	2.2
13	13	+5.4	6	1040	2.4	3.0
15	11	+5.4	6	920	1.3	2.2
17	11	+5.4	7	970	1.3	1.7
9	7	0.0	10	530	1.3	1.8
9	10	0.0	9	660	1.5	1.8
9	13	0.0	9	660	1.5	1.9
13	7	0.0	5	1200	1.0	1.4
13	10	0.0	7	950	1.3	1.8
13	13	0.0	6	1100	2.3	3.0
17	8	0.0	9	740	1.2	1.5
17	11	0.0	7	1000	1.3	1.7

* Average of all rips observed.
** Maximum rip observed.

Table 12

Rip Current Observations for Full Model Width, Perched Beach Installed

Test Wave		Still-water	Characteristics of Rip Currents Prototype Dimensions			
Period sec	Height ft	Level, ft mllw	Number Observed	Average Spacing ft	Average* Velocity fps	Maximum** Velocity fps
6	10	0.0	10	620	1.0	1.3
7	10	0.0	6	840	1.1	1.5
8	13	0.0	8	830	1.3	1.9
9	13	0.0	12	510	1.0	1.2
10	14	0.0	9	650	1.6	1.9
11	14	0.0	9	680	1.9	2.8
12	13	0.0	6	1040	1.9	2.3
13	13	0.0	7	920	1.7	2.0
14	11	0.0	7	850	1.3	1.7
15	11	0.0	8	740	1.0	1.3
16	11	0.0	7	920	0.9	1.1
17	11	0.0	6	1180	1.0	1.2
18	10	0.0	6	1080	1.1	1.3
19	10	0.0	7	900	1.0	1.4
7	10	+5.4	8	800	0.8	1.1
9	13	+5.4	5	1030	1.4	1.6
11	14	+5.4	7	1000	1.4	1.8
13	13	+5.4	7	920	1.3	1.8
15	11	+5.4	5	1130	0.8	0.9
17	11	+5.4	4	1400	1.0	1.4
9	7	0.0	6	1000	1.1	1.5
9	10	0.0	11	580	1.5	1.9
9	13	0.0	12	510	1.0	1.2
13	7	0.0	7	960	1.0	1.2
13	10	0.0	8	770	1.3	1.6
13	13	0.0	7	920	1.7	2.0
17	8	0.0	8	830	1.1	1.2
17	11	0.0	6	1180	1.0	1.2

* Average of all rips observed.
** Maximum rip observed.

Table 13

Comparison of Rip Current Velocities for Full Model Width

Test Wave Period sec	Test Wave Height ft	Still-water Level ft mllw	Average Velocity* fps: Existing Conditions	Average Velocity* fps: Perched Beach Installed	Maximum Velocity** fps: Existing Conditions	Maximum Velocity** fps: Perched Beach Installed
			Characteristics of Rip Currents, Prototype Dimensions			
6	10	0.0	1.3	1.0	1.7	1.3
7	10	0.0	1.2	1.1	1.6	1.5
8	13	0.0	1.7	1.3	2.0	1.9
9	7	0.0	1.3	1.1	1.8	1.5
9	10	0.0	1.5	1.5	1.9	1.9
9	13	0.0	1.9	1.0	2.4	1.2
10	14	0.0	1.9	1.6	3.0	1.9
11	14	0.0	1.8	1.9	2.4	2.8
12	13	0.0	2.1	1.9	2.6	2.3
13	7	0.0	1.0	1.0	1.4	1.2
13	10	0.0	1.3	1.3	1.8	1.6
13	13	0.0	2.3	1.7	3.0	2.0
14	11	0.0	2.3	1.3	2.7	1.7
15	11	0.0	1.4	1.0	1.7	1.3
16	11	0.0	1.4	0.9	1.9	1.1
17	8	0.0	1.2	1.1	1.5	1.2
17	11	0.0	1.3	1.0	1.7	1.2
18	10	0.0	1.0	1.1	1.8	1.3
19	10	0.0	1.1	1.0	1.4	1.4
7	10	+5.4	1.3	0.8	1.7	1.1
9	13	+5.4	1.2	1.4	1.5	1.6
11	14	+5.4	1.5	1.4	2.2	1.8
13	13	+5.4	2.4	1.3	3.0	1.8
15	11	+5.4	1.3	0.8	2.2	0.9
17	11	+5.4	1.3	1.0	1.7	1.4

* Average of all rips observed across width of model.
** Maximum one rip observed.

Table 14

Rip Current Observations for Various Longshore Beach Widths

With Existing Beach Slopes Installed

Test Wave Period sec	Test Wave Height ft	Longshore Beach Width ft	Current Velocity, fps, at Various Distances Offshore, ft: 200-400	400-600	600-800	800-1000	>1000	Average	No. of Rips	Approximate Spacing Between Rips, ft
9	7	4980	3.2	3.2	2.2	2.0	2.0	2.5	6	1100, 800, 900, 1200, 600
		4880	2.2	2.3	2.2	1.8	--	1.7	5	1100, 1000, 1400, 1200
		4780	2.8	2.0	2.2	2.3	1.7	2.2	4	1500, 1500, 1600
		4730	4.8	4.8	3.2	3.2	3.2	3.8	5	900, 1200, 1500, 900
		4680	2.7	2.7	2.2	2.2	--	2.0	6	900, 1000, 700, 700, 1000
		4630	4.0	4.0	3.5	3.2	3.2	3.6	3	2100, 2300
		4580	3.3	3.3	1.7	1.7	1.0	2.2	3	1600, 1900
9	13	4980	4.3	4.3	3.2	3.2	3.2	3.6	3	1500, 1600
		4880	2.8	2.8	4.0	5.0	5.0	3.9	4	1000, 1400, 800
		4780	3.0	4.5	3.3	3.5	3.5	3.6	4	1400, 1400, 1300
		4730	4.0	4.0	4.5	4.5	5.0	4.4	3	1600, 1700
		4680	3.3	3.2	3.2	3.7	3.7	3.4	3	1100, 1400
		4630	4.3	4.3	3.3	2.7	2.5	3.4	4	1600, 1000, 900
		4580	5.0	5.0	4.8	4.8	4.2	4.8	4	1100, 1100, 1400
11	8	4960	2.7	4.0	4.0	2.5	2.5	3.1	5	1100, 1100, 1400, 1300
		4860	3.3	3.3	2.0	1.7	1.0	2.3	4	1100, 1100, 1200
		4760	3.8	2.8	2.8	1.2	--	2.1	4	1900, 900, 900
		4660	3.7	3.7	3.3	2.5	2.3	3.1	3	1800, 1400
		4610	3.7	3.7	2.8	1.0	--	2.2	4	1300, 1600, 1600
		4560	2.7	3.5	3.5	2.3	2.2	2.8	4	1400, 1600, 1500
		4510	3.5	3.5	3.0	2.0	1.8	2.8	4	1600, 1400, 1400
		4460	3.5	3.5	2.5	1.7	--	2.2	4	1500, 1400, 1500
		4360	2.0	2.3	1.5	1.3	--	1.4	3	1600, 1500
11	14	4960	5.2	5.2	5.2	2.8	2.8	4.2	5	1400, 1000, 1000, 900
		4860	4.3	3.8	3.8	2.0	2.3	3.2	3	2000, 1600
		4760	4.2	4.2	4.2	4.2	2.2	3.8	4	1500, 1600, 1400
		4660	4.0	4.0	4.0	3.8	4.3	4.0	3	1800, 1600
		4610	3.3	3.3	2.3	2.3	1.7	2.6	4	1600, 1000, 900
		4560	4.2	4.2	3.7	3.7	4.3	4.0	5	1600, 800, 900, 1000
		4510	5.3	5.3	3.8	3.8	3.8	4.4	4	1200, 1700, 1400
		4460	2.7	2.7	3.0	2.7	2.7	2.8	4	1400, 1600, 800
		4360	4.3	4.3	3.0	3.0	2.7	3.5	3	1400, 1500
13	7	5190	3.3	3.0	1.7	2.2	--	2.0	4	1900, 1900, 1300
		5090	3.8	3.0	2.5	1.8	1.2	2.5	4	1850, 1850, 1350
		5040	3.2	2.5	2.3	1.8	1.2	2.2	4	900, 2100, 1900
		4990	6.3	6.3	2.7	1.3	--	3.3	4	1700, 1900, 1300
		4940	4.0	4.0	2.0	1.7	1.7	2.7	4	1200, 1900, 1700
		4890	3.5	3.5	2.3	2.3	1.3	2.6	4	1500, 1900, 1400
		4790	3.7	2.7	2.5	2.5	1.5	2.6	4	1500, 1800, 1400
		4690	3.5	3.3	4.0	4.0	2.3	3.4	4	2100, 1000, 1500
		4590	2.5	2.5	2.7	2.7	1.7	2.4	4	1100, 2000, 1400
		4490	3.2	2.7	1.7	1.5	--	1.8	4	1400, 1400, 1600
		4390	2.8	2.0	2.0	1.7	1.7	2.0	4	1000, 2000, 1300
13	13	5190	5.0	5.0	3.3	2.8	3.8	4.0	3	1800, 1800
		5140	7.3	7.3	7.3	3.0	3.0	5.6	3	1500, 2100
		5090	3.8	4.2	4.3	4.3	4.0	4.1	3	2100, 1900
		4990	3.7	4.5	4.5	2.2	1.8	3.3	4	900, 1200, 2200
		4890	4.2	4.8	4.8	3.5	3.2	4.1	3	1700, 2700
		4790	5.2	5.2	5.2	5.0	3.3	4.8	3	1800, 1600
		4740	4.8	4.8	4.7	4.7	2.7	4.3	4	1300, 1300, 2000
		4690	5.0	5.0	3.8	5.0	5.0	4.8	3	1500, 1500
		4590	5.3	5.3	5.5	5.5	4.2	5.2	3	2200, 1800
		4540	7.8	7.8	7.8	3.2	1.3	5.6	3	1500, 2200
		4490	5.2	5.2	6.5	6.5	6.5	6.0	3	1000, 1100
		4390	4.3	4.3	4.7	4.7	3.0	4.2	4	700, 1100, 1500

Table 15

Rip Current Observations for Various Longshore Beach Widths

With Perched Beach Installed

Test Wave Period sec	Test Wave Height ft	Longshore Beach Width ft	Current Velocity, fps, at Various Distances Offshore, ft: 200-400	400-600	600-800	800-1000	>1000	Average	No. of Rips	Approximate Spacing Between Rips, ft
9	7	4980	3.2	2.8	2.5	1.5	0.8	2.2	5	1100, 700, 1900, 1100
		4880	2.3	2.0	2.5	1.2	--	1.6	5	900, 1300, 900, 900
		4780	2.3	2.2	1.7	1.0	--	1.4	5	1000, 900, 500, 1000, 1100
		4730	2.7	1.7	1.7	0.8	--	1.4	5	1200, 1300, 900, 900
		4680	3.0	2.7	2.3	1.8	1.3	2.2	4	1200, 2200, 1100
		4630	2.8	2.5	1.7	1.7	--	1.7	4	850, 1500, 1500
		4580	3.3	3.3	2.2	2.2	1.3	2.5	4	750, 1500, 1150
9	13	4980	3.8	3.3	2.7	1.3	1.3	2.5	5	1150, 850, 850, 950
		4880	4.2	5.0	2.8	2.5	1.7	3.2	7	600, 500, 650, 650, 450, 650
		4780	4.0	3.3	3.7	2.5	1.7	3.0	4	1000, 1300, 900
		4730	5.0	3.3	2.8	2.8	1.3	3.0	5	1150, 1050, 1000, 1200
		4680	5.3	5.3	3.3	1.7	1.3	3.4	6	750, 700, 900, 650, 600
		4630	5.5	5.5	3.3	3.3	3.3	4.2	6	700, 900, 850, 800, 750
		4580	5.0	5.0	3.3	2.0	2.0	3.5	5	900, 800, 1100, 900
11	8	4960	3.3	1.3	1.7	0.8	--	1.4	3	2100, 2000
		4860	3.8	2.0	2.0	--	--	1.6	3	1500, 1600
		4760	3.2	2.8	2.0	1.5	0.8	2.1	3	2100, 1900
		4710	2.8	2.0	2.0	1.7	--	1.7	6	600, 650, 750, 700, 800
		4660	3.2	3.2	1.5	0.5	--	1.7	3	1850, 1850
		4610	3.7	2.8	2.5	2.5	1.7	2.6	5	1050, 850, 900, 1000
		4560	3.8	2.2	1.3	1.5	--	1.8	5	900, 900, 1000, 1000
		4460	3.7	1.8	1.2	0.8	--	1.5	3	1700, 1700
		4360	2.7	2.1	1.8	1.5	--	1.6	3	1600, 1800
11	14	4960	5.4	5.4	3.3	3.3	2.5	4.0	6	1200, 850, 750, 700, 800
		4860	4.2	5.0	3.3	2.5	2.5	3.5	6	800, 1200, 800, 800, 800
		4810	4.2	4.2	4.2	2.8	1.2	3.3	4	800, 1150, 1500
		4760	5.0	5.0	3.3	1.7	1.7	3.3	5	1100, 1100, 1050, 1050
		4710	3.3	4.2	3.3	2.5	1.7	3.0	5	800, 1100, 1000, 1150
		4660	5.8	5.8	4.2	1.7	1.7	3.8	6	800, 1250, 850, 800, 800
		4560	5.0	5.0	5.0	2.5	1.7	3.8	5	900, 600, 650, 700
		4460	4.2	4.2	3.3	1.7	1.7	3.0	6	1050, 950, 650, 750, 1000
		4360	5.8	5.8	2.5	1.7	1.7	3.5	5	1000, 900, 750, 800
13	7	5190	3.3	3.3	2.2	0.8	--	1.9	5	900, 900, 1700, 1300
		5090	4.2	3.3	3.3	1.7	0.8	2.7	4	2000, 1700, 1300
		4990	5.0	3.3	3.3	1.7	1.7	3.0	4	900, 1150, 1450
		4890	4.2	4.2	3.3	1.7	0.8	2.8	4	1000, 900, 1550
		4790	4.2	3.3	3.3	1.7	0.8	2.7	4	1400, 1400, 1800
		4690	4.2	4.2	2.5	2.5	1.7	3.0	4	1600, 1600, 1400
		4590	4.2	4.2	3.3	2.5	1.7	3.2	4	750, 1050, 1400
		4490	4.2	4.2	2.5	1.7	1.7	2.9	3	1500, 1500
		4440	3.7	3.7	3.3	2.5	1.2	2.9	3	1500, 1500
		4390	5.0	5.0	3.3	2.5	1.7	3.5	3	1400, 1500
		4340	4.2	3.3	2.0	2.0	1.2	2.5	3	1350, 1450
13	13	5190	3.3	3.3	1.7	0.8	--	1.8	5	1200, 1550, 1050, 1000
		5090	4.2	4.2	2.5	0.8	0.8	2.5	3	1600, 2000
		5040	5.0	5.0	4.2	1.7	0.8	3.3	5	900, 1050, 850, 1000
		4990	5.0	5.0	3.3	2.5	2.5	3.7	3	1800, 1900
		4940	5.8	5.8	3.3	1.7	1.7	3.7	4	1700, 900, 1000
		4890	5.0	5.0	3.3	1.7	1.7	3.3	4	850, 1100, 1700
		4790	5.0	5.0	3.3	1.7	1.7	3.3	4	1000, 900, 1950
		4740	5.8	5.8	4.2	1.7	1.7	3.8	3	1950, 1800
		4690	5.0	5.0	4.2	1.7	1.7	3.5	3	1800, 1800
		4640	5.0	5.0	4.2	1.7	1.7	3.5	5	1000, 900, 1000, 900
		4590	4.5	4.5	2.5	1.7	1.7	3.0	3	1700, 1550
		4490	5.0	5.0	3.3	1.7	1.7	3.3	4	1300, 700, 1850
		4390	5.5	5.5	2.8	1.7	0.8	3.3	4	1050, 1000, 1400

Table 16

Comparison of Average Rip Current Velocities for Various Longshore Beach Widths with Existing Slope and Perched Beach Installed

Test Wave		Longshore Beach	Average Velocity, fps	
Period, sec	Height, ft	Width, ft	Existing Slope	Perched Beach
9	7	4980	2.5	2.2
		4880	1.7	1.6
		4780	2.2	1.4
		4730	3.8	1.4
		4680	2.0	2.2
		4630	3.6	1.7
		4580	2.2	2.5
9	13	4980	3.6	2.5
		4880	3.9	3.2
		4780	3.6	3.0
		4730	4.4	3.0
		4680	3.4	3.4
		4630	3.4	4.2
		4580	4.8	3.5
11	8	4960	3.1	1.4
		4860	2.3	1.6
		4760	2.1	2.1
		4710	--	1.7
		4660	3.1	1.7
		4610	2.2	2.6
		4560	2.8	1.8
		4510	2.8	--
		4460	2.2	1.5
		4360	1.4	1.6
11	14	4960	4.2	4.0
		4860	3.2	3.5
		4810	--	3.3
		4760	3.8	3.3
		4710	--	3.0
		4660	4.0	3.8
		4610	2.6	--
		4560	4.0	3.8
		4510	4.4	--
		4460	2.8	3.0
		4360	3.5	3.5

(Continued)

Table 16 (Concluded)

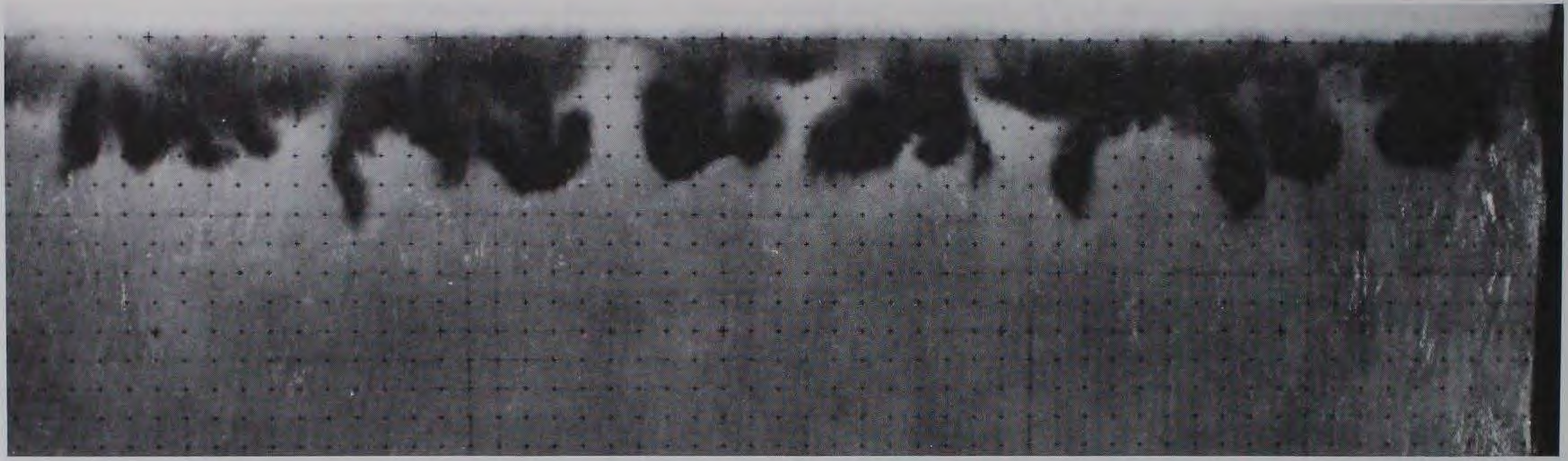
Test Wave		Longshore Beach	Average Velocity, fps	
Period, sec	Height, ft	Width, ft	Existing Slope	Perched Beach
13	7	5190	2.0	1.9
		5090	2.5	2.7
		5040	2.2	--
		4990	3.3	3.0
		4940	2.7	--
		4890	2.6	2.8
		4790	2.6	2.7
		4690	3.4	3.0
		4590	2.4	3.2
		4490	1.8	2.9
		4440	--	2.9
		4390	2.0	3.5
		4340	--	2.5
13	13	5190	4.0	1.8
		5140	5.6	--
		5090	4.1	2.5
		5040	--	3.3
		4990	3.3	3.7
		4940	--	3.7
		4890	4.1	3.3
		4790	4.8	3.3
		4740	4.3	3.8
		4690	4.8	3.5
		4640	--	3.5
		4590	5.2	3.0
		4540	5.6	--
		4490	6.0	3.3
		4390	4.2	3.3

Table 17

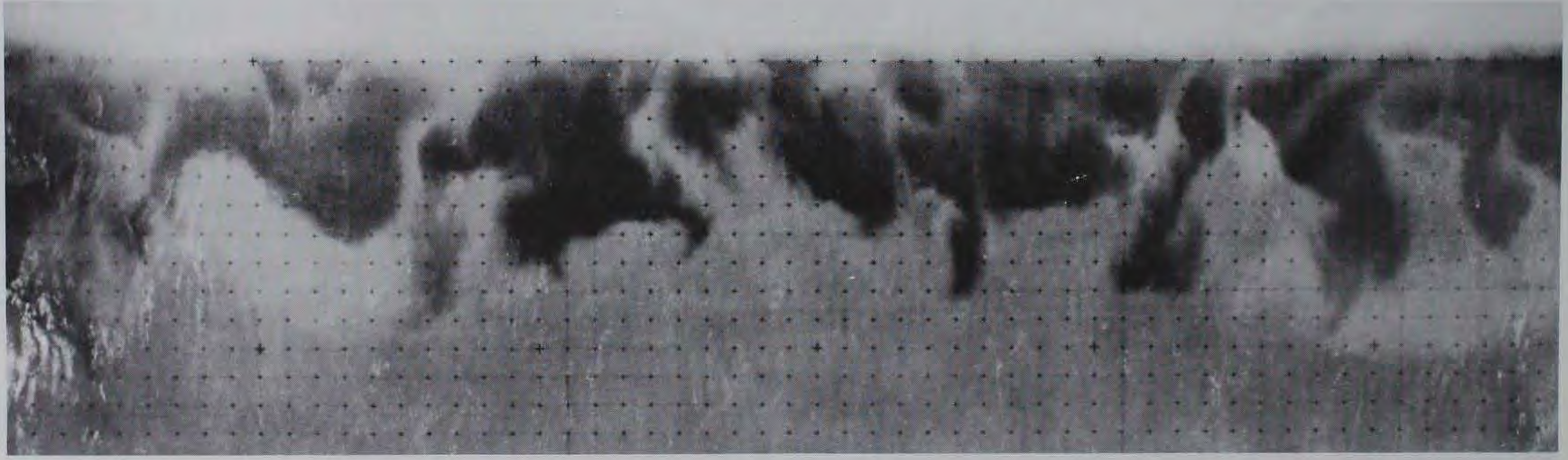
Perched Beach Rip Current Model, Visual Observations

of Surf-Zone Width

Test Wave Period sec	Test Wave Height ft	Still-Water Level ft mllw	Width of Surf Zone ft	Remarks
		Existing Conditions Installed		
7	10	0.0	400	In many cases the waves started to spill and break a considerable distance offshore due to currents, steepness of waves, etc.
9	7	0.0	250-350	
9	13	0.0	1000-1200	
11	8	0.0	200-300	
11	14	0.0	1100-2000	
13	7	0.0	200-300	
13	13	0.0	1500-2000	
15	11	0.0	1000	
17	11	0.0	1000-1200	
		Perched Beach Installed		
6	10	0.0	300-600	Waves broke more uniformly
7	10	0.0	600	
8	13	0.0	600	
9	7	0.0	300	
9	10	0.0	500	
9	13	0.0	700	
10	14	0.0	700	
11	14	0.0	700-1300	
12	13	0.0	700	
13	7	0.0	400	
13	10	0.0	500	
13	13	0.0	700	
14	11	0.0	700	
15	11	0.0	500	
16	11	0.0	700-800	
17	8	0.0	500	
17	11	0.0	600	
18	10	0.0	500	
19	10	0.0	500-600	
7	10	+5.4	350	
9	13	+5.4	500-600	
11	14	+5.4	700	
13	13	+5.4	500	
15	11	+5.4	300-400	
17	11	+5.4	500	

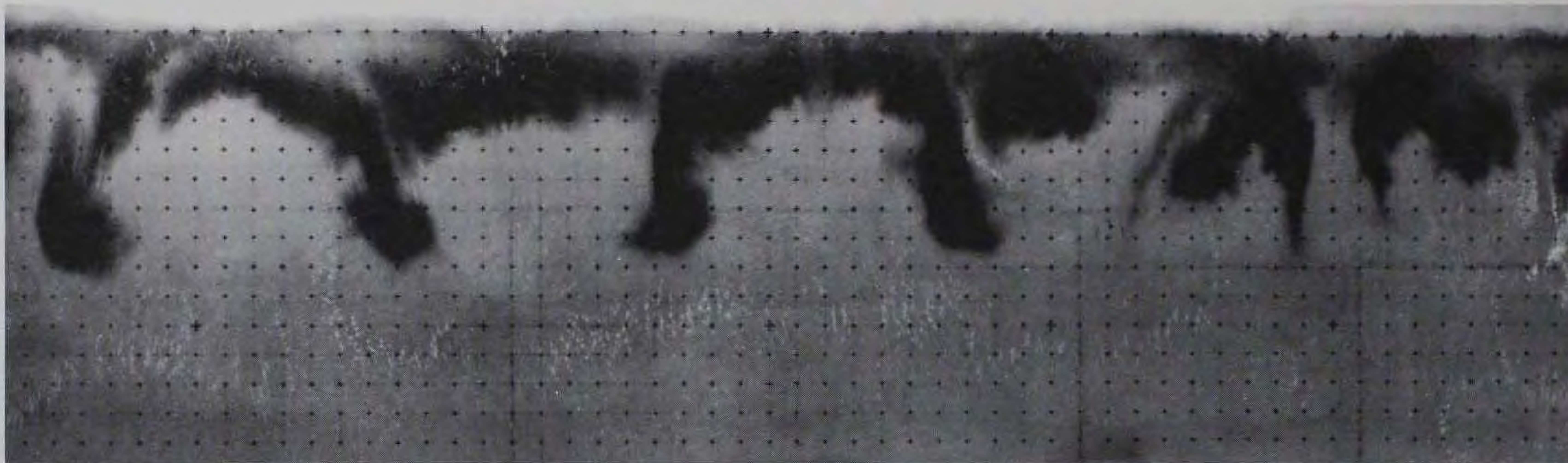


a. Elapsed time = 2.8 min



b. Elapsed time = 5.8 min

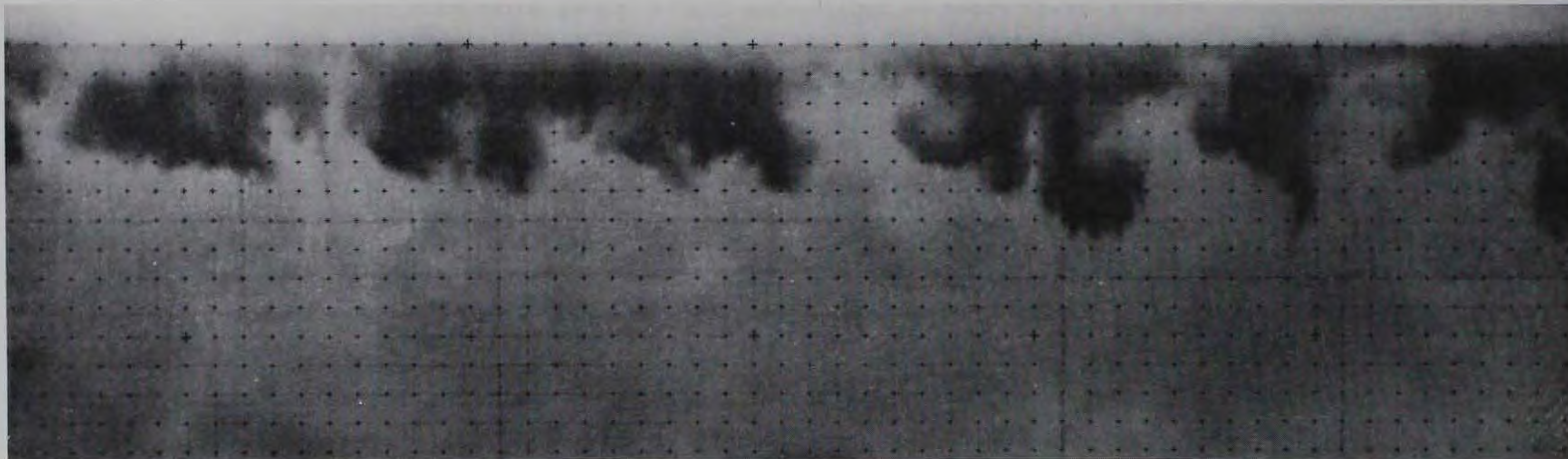
Photo 1. Rip current patterns with existing conditions installed; longshore beach width = 6040 ft; 9-sec, 13-ft waves



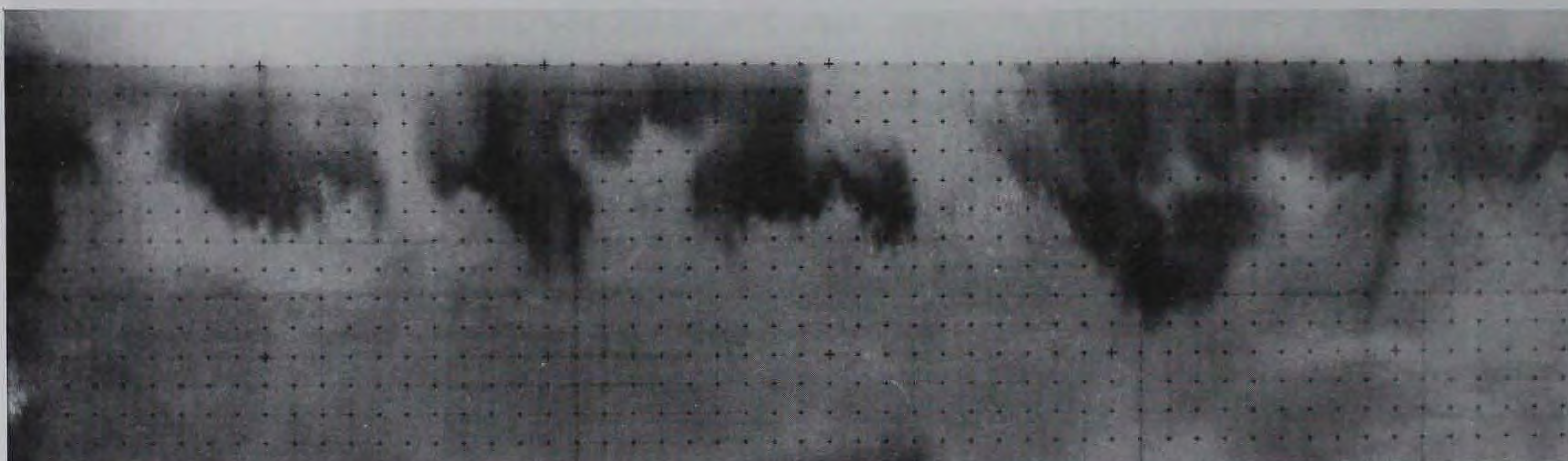
a. Elapsed time = 3.7 min

b. Elapsed time = 6.8 min

Photo 2. Rip current patterns with existing conditions installed; longshore beach width = 6040 ft; 13-sec, 13-ft waves

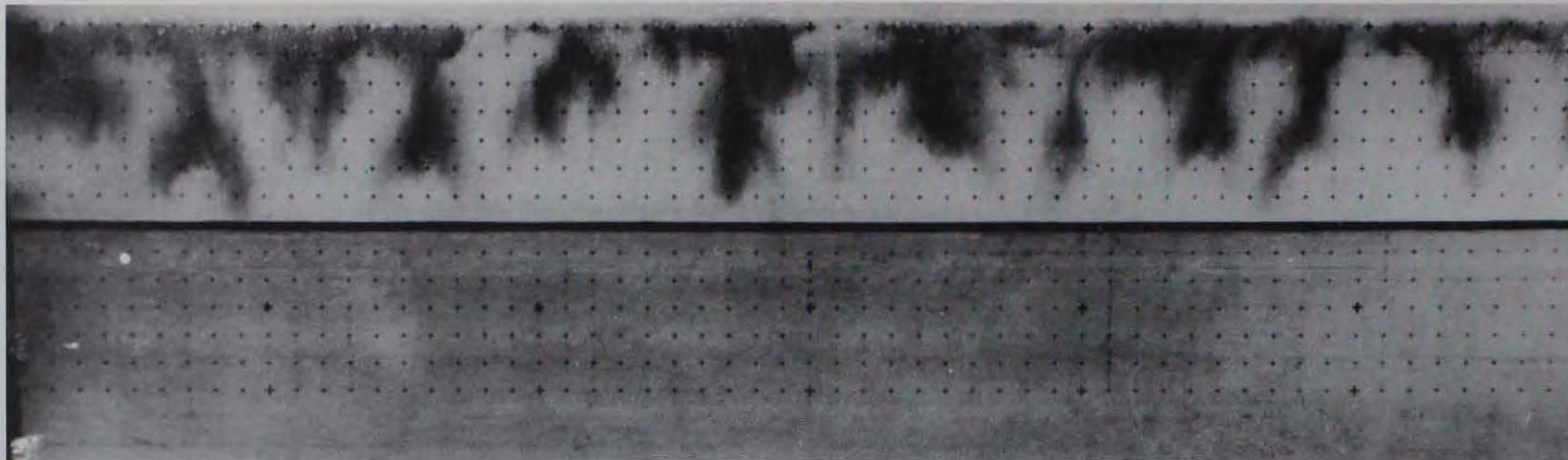


a. Elapsed time = 5.0 min

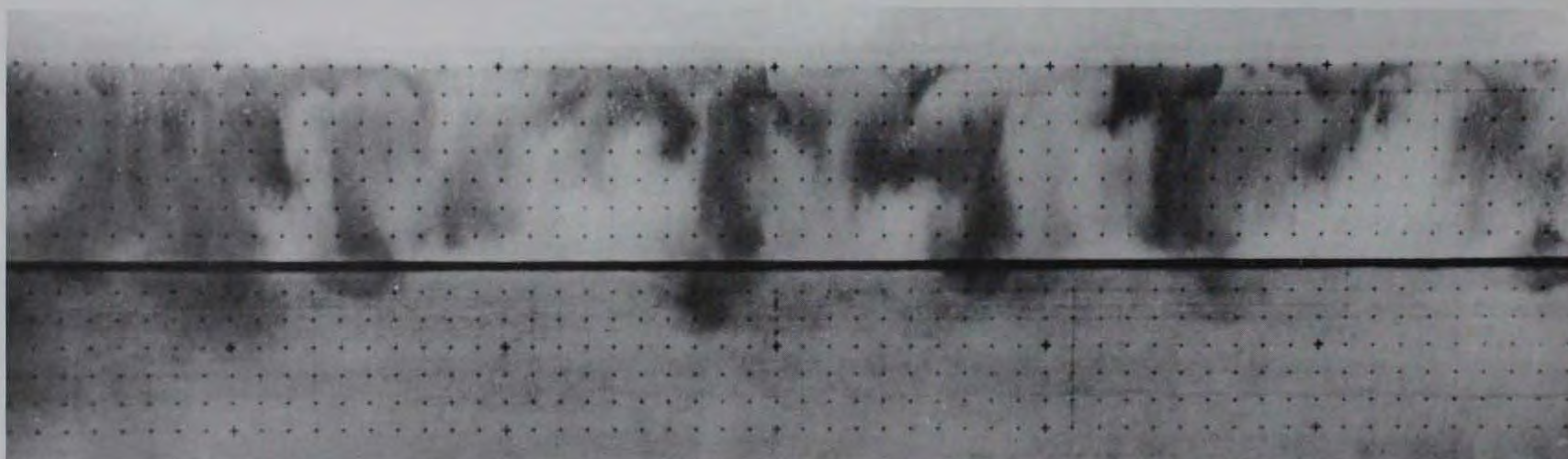


b. Elapsed time = 8.0 min

Photo 3. Rip current patterns with existing conditions installed; longshore beach width = 6040 ft; 17-sec, 11-ft waves



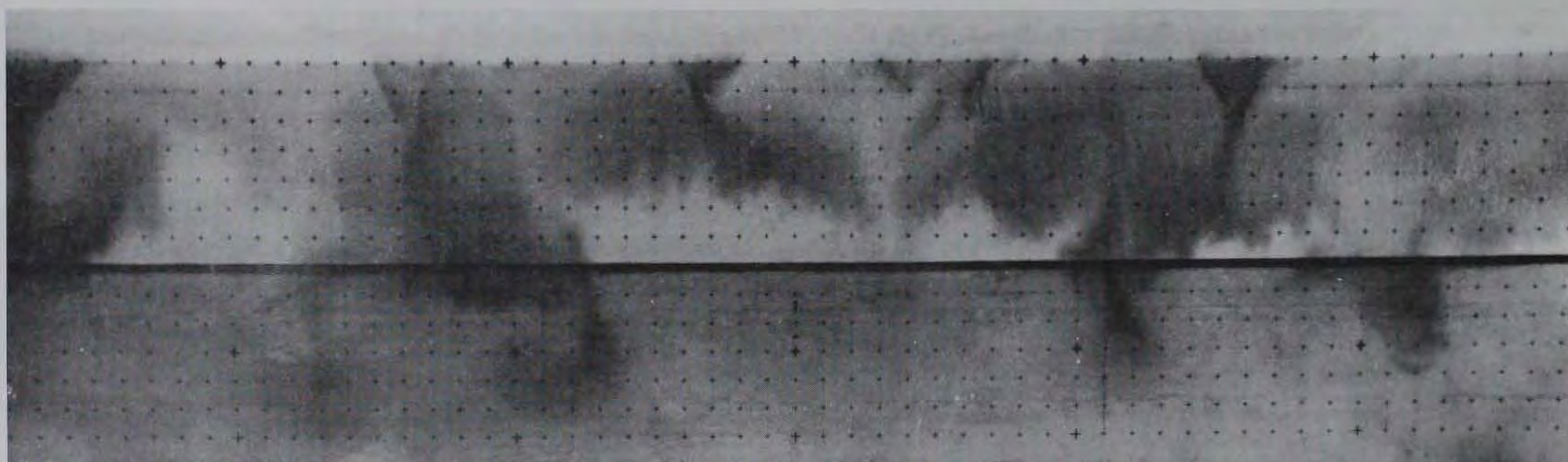
a. Elapsed time = 5.2 min



b. Elapsed time = 10.0 min

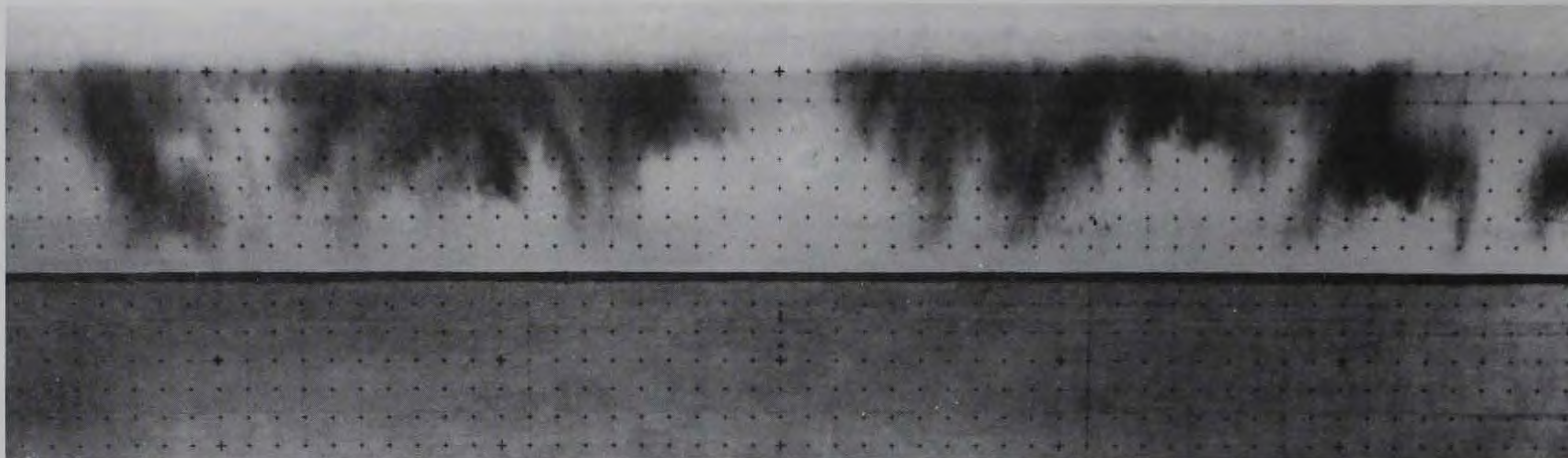
Photo 4. Rip current patterns with perched beach installed; longshore beach width = 6040 ft; 9-sec, 13-ft waves

a. Elasped time = 5.8 min

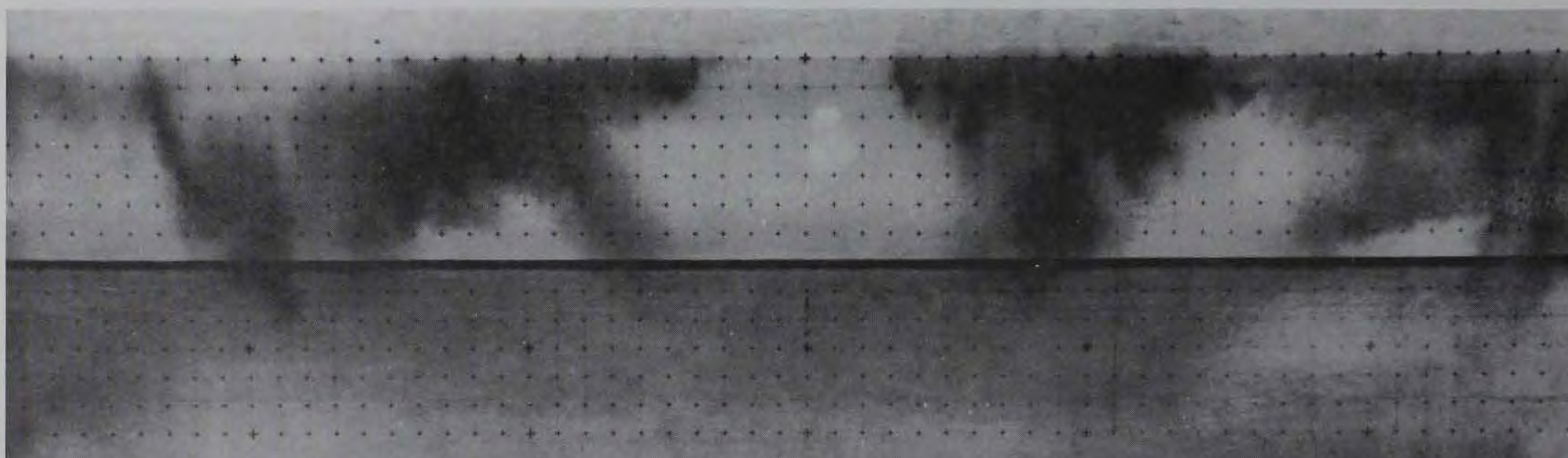


b. Elapsed time = 8.7 min

Photo 5. Rip current patterns with perched beach installed; longshore beach width = 6040 ft; 13-sec, 13-ft waves



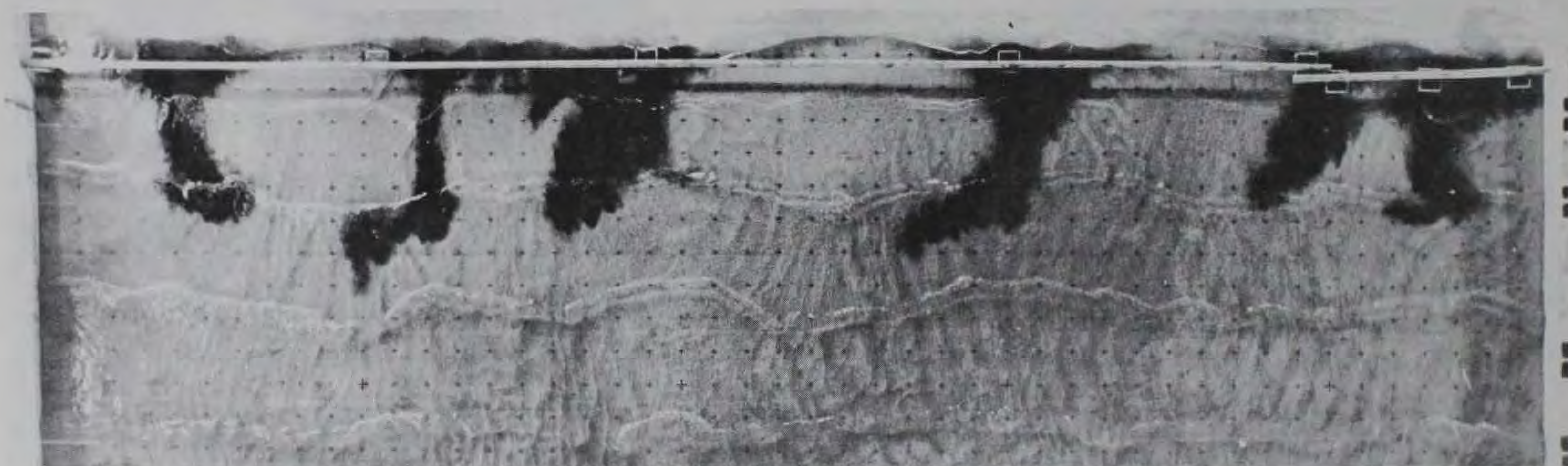
a. Elapsed time = 5.2 min



b. Elapsed time = 10.0 min

Photo 6. Rip current patterns with perched beach installed; longshore beach width = 6040 ft; 17-sec, 11-ft waves

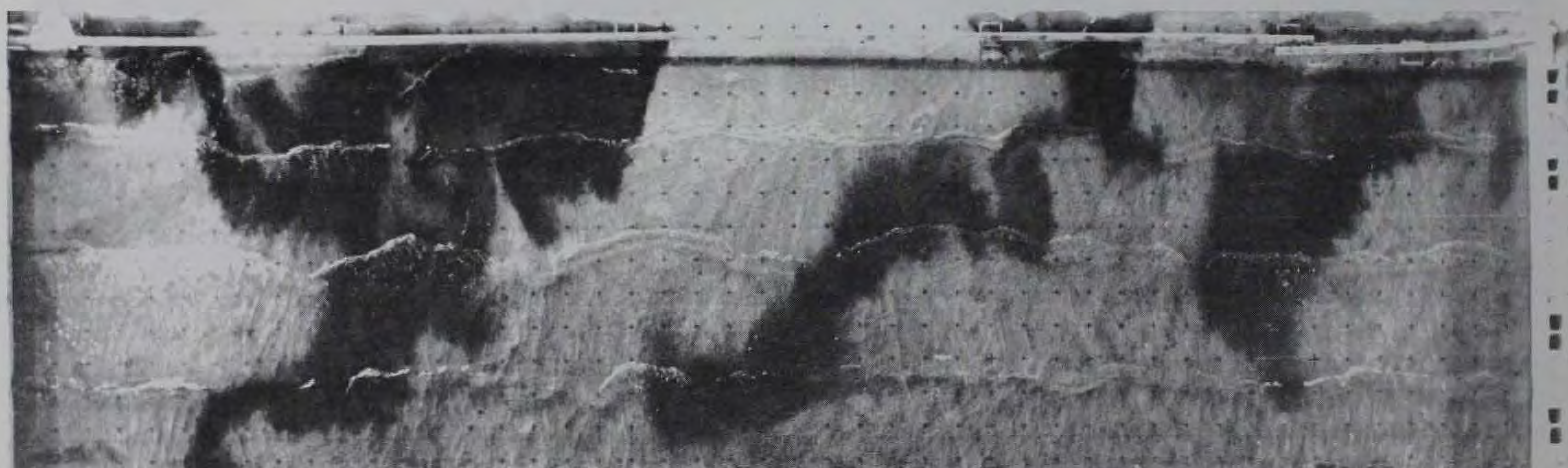
a. Elapsed time = 30 min



b. Elapsed time = 32 min

Photo 7. Rip current patterns with existing conditions installed; longshore beach width = 4690 ft; 13-sec, 13-ft waves (sheet 1 of 2)

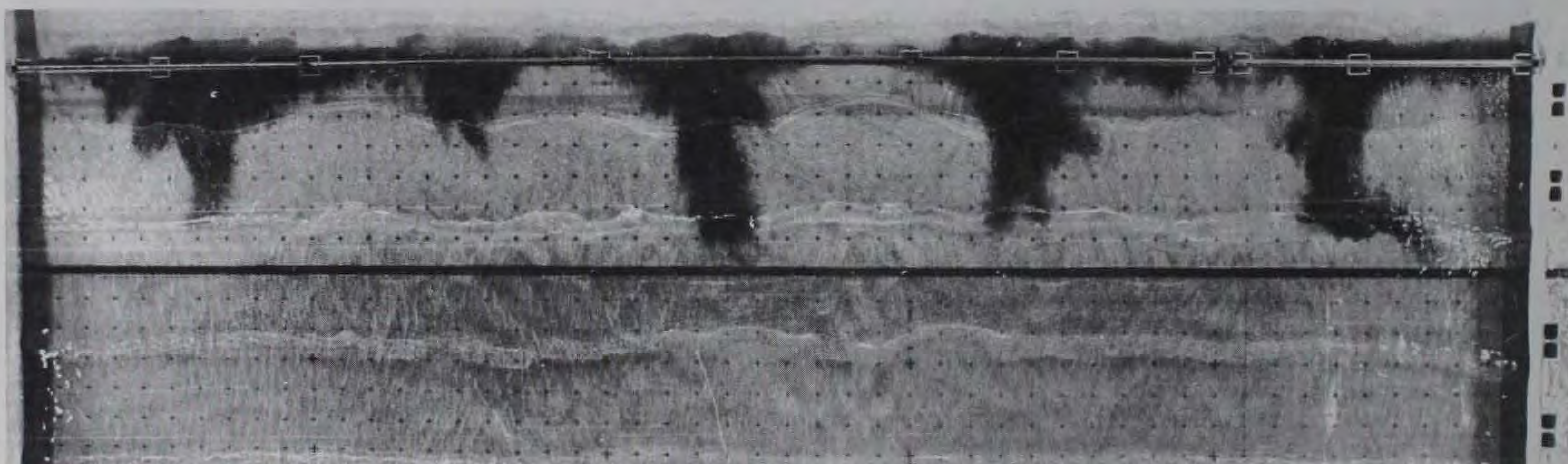
c. Elapsed time = 34 min



d. Elapsed time = 36 min

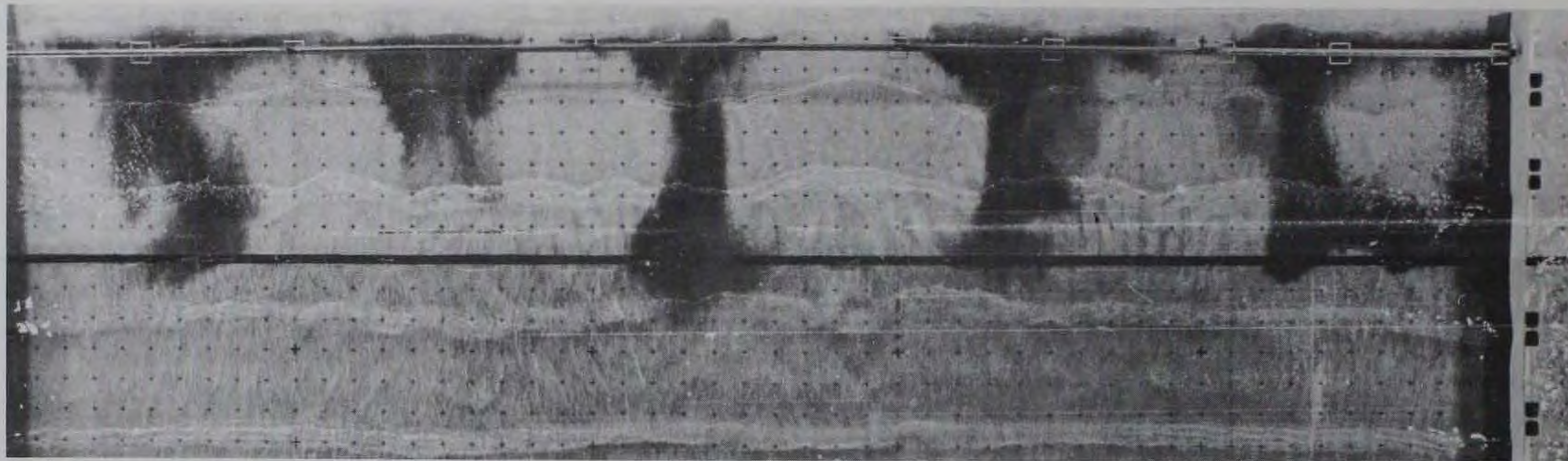
Photo 7. (sheet 2 of 2)

a. Elapsed time = 31 min

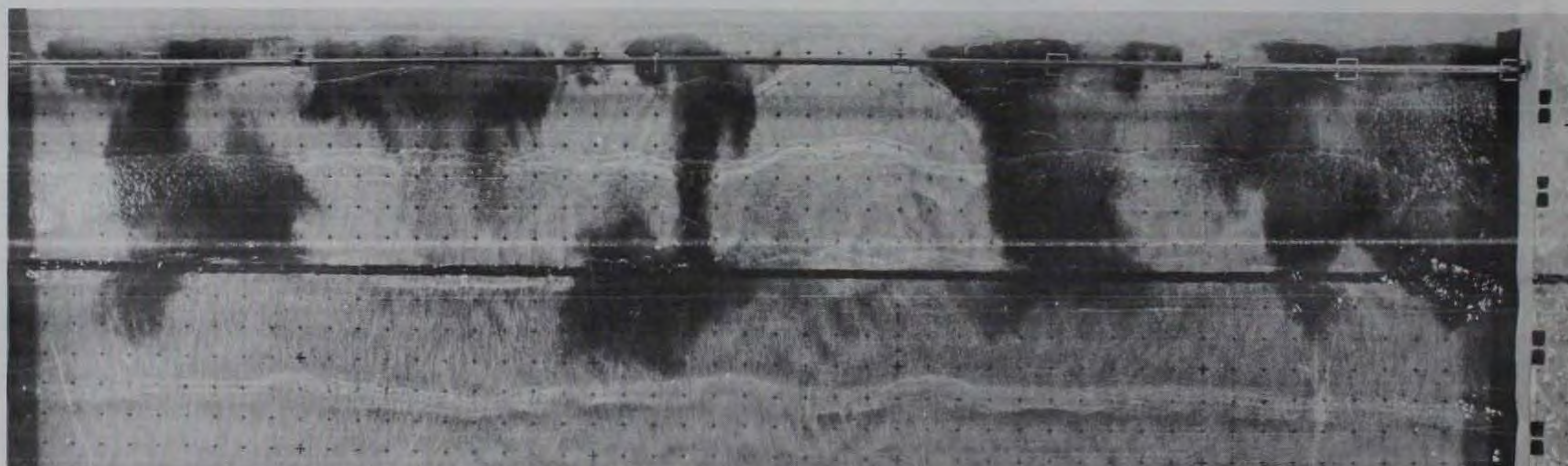


b. Elapsed time = 32 min

Photo 8. Rip current patterns with perched beach installed; longshore beach width = 5040 ft; 13-sec, 13-ft waves (sheet 1 of 2)

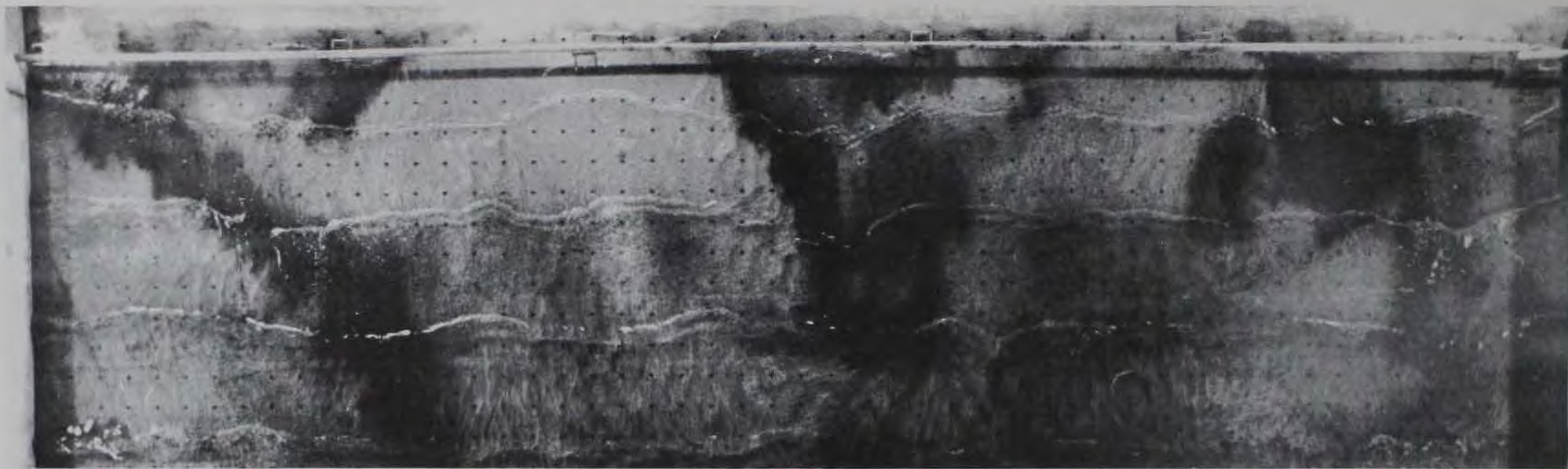


c. Elapsed time = 34 min



d. Elapsed time = 36 min

Photo 8. (sheet 2 of 2)

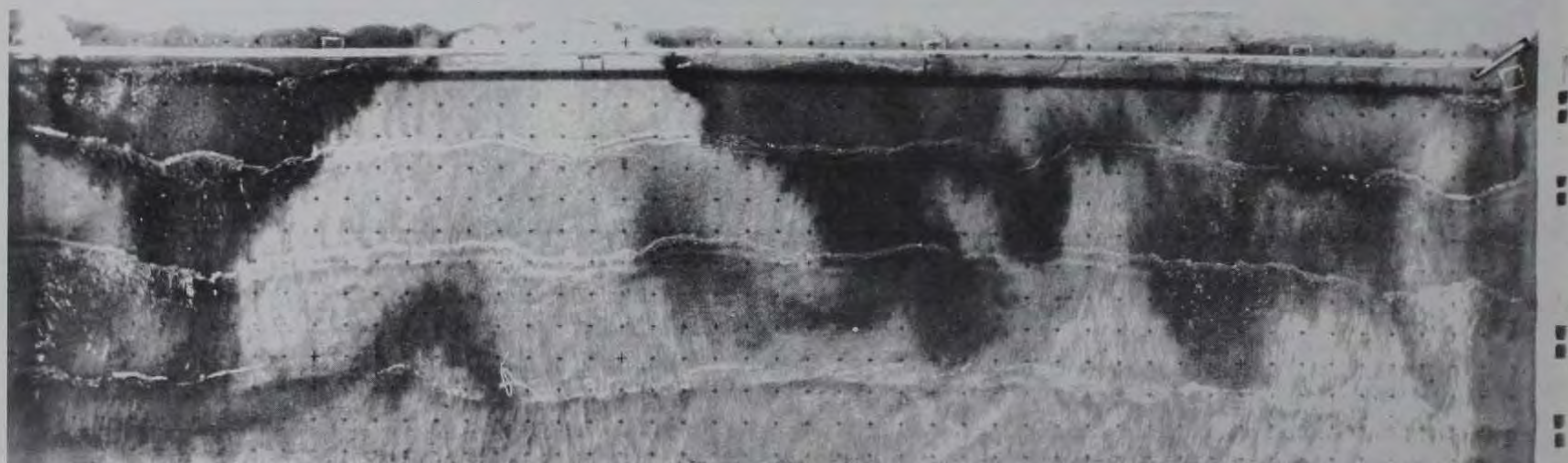


a. Longshore beach width = 5190 ft; elapsed time = 37 min

b. Longshore beach width = 5140 ft; elapsed time = 37 min

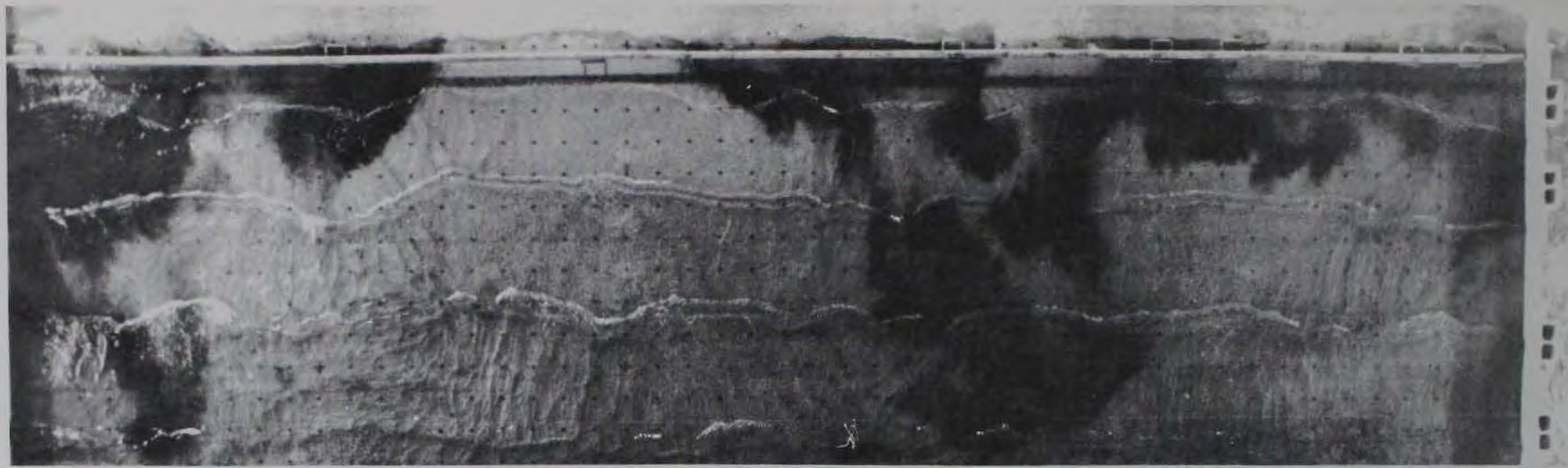
Photo 9. Rip current patterns with existing conditions installed; 13-sec, 13-ft waves (sheet 1 of 6)

c. Longshore beach width = 5090 ft; elapsed time = 38 min

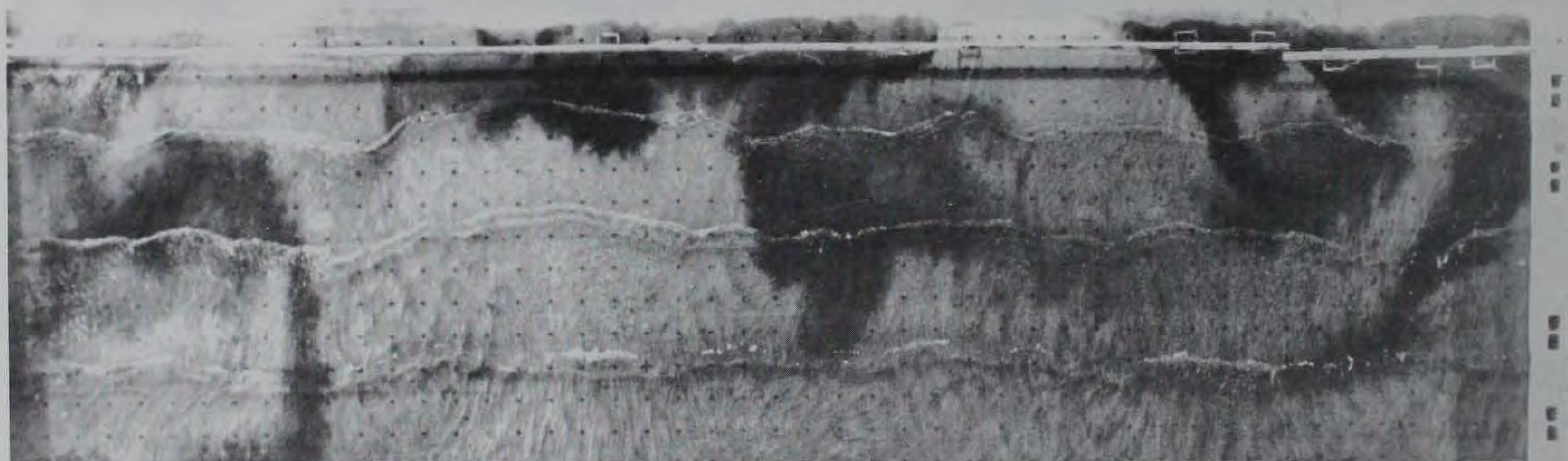


d. Longshore beach width = 4990 ft; elapsed time = 38 min

Photo 9. (sheet 2 of 6)

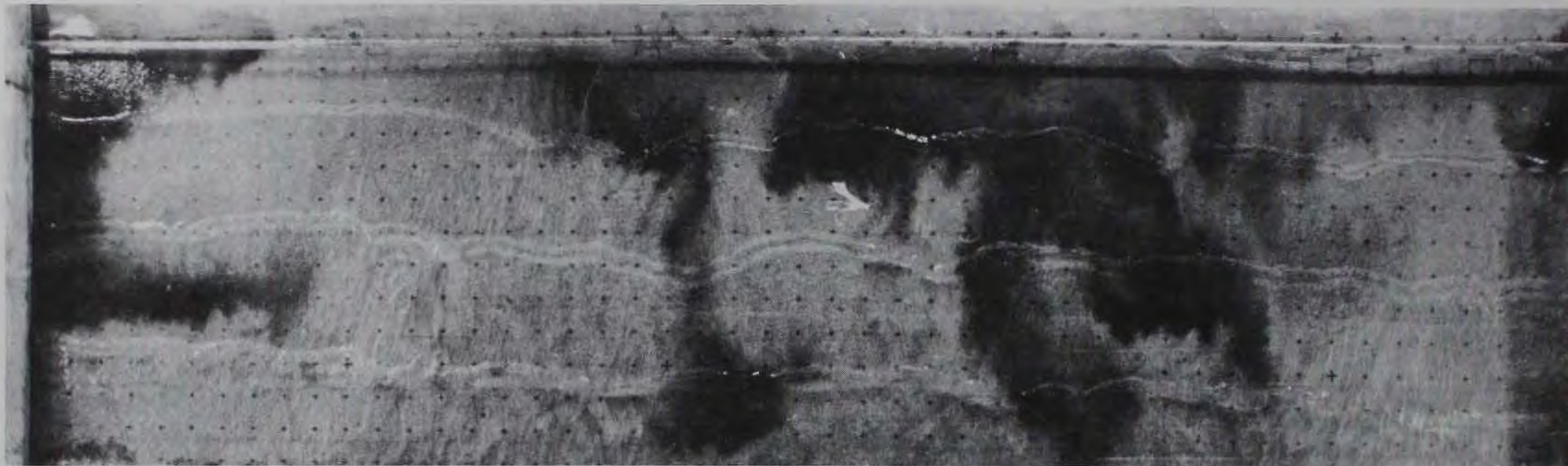


e. Longshore beach width = 4890 ft; elapsed time = 36 min



f. Longshore beach width = 4790 ft; elapsed time = 35 min

Photo 9. (sheet 3 of 6)

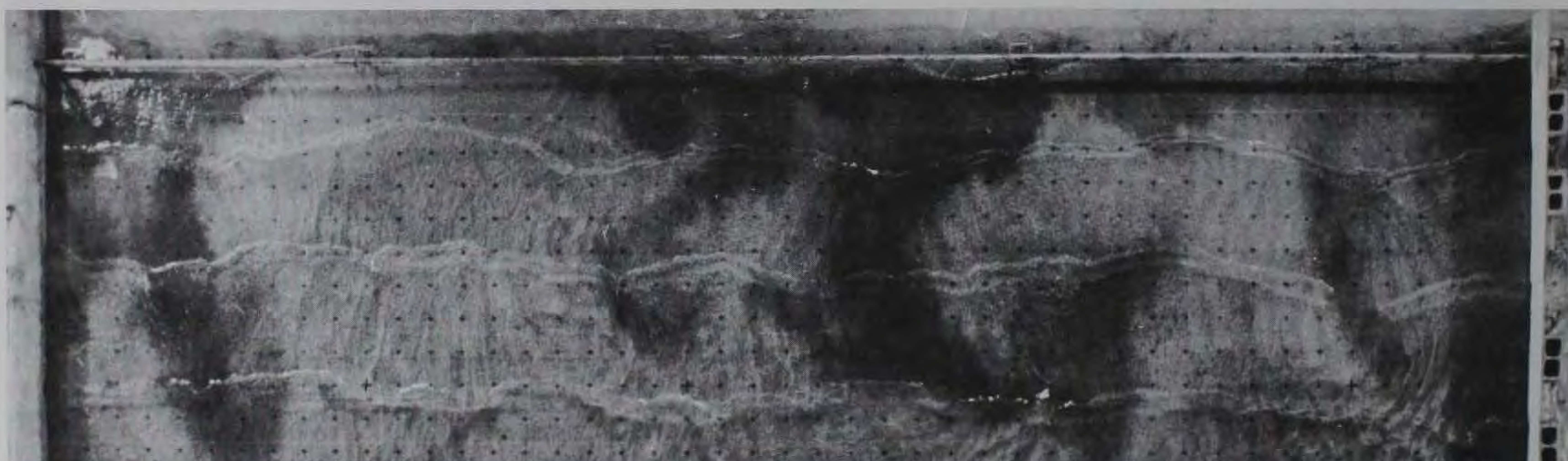


g. Longshore beach width = 4740 ft; elapsed time = 37 min

h. Longshore beach width = 4690 ft; elapsed time = 35 min

Photo 9. (sheet 4 of 6)

i. Longshore beach width = 4590 ft; elapsed time = 36 min



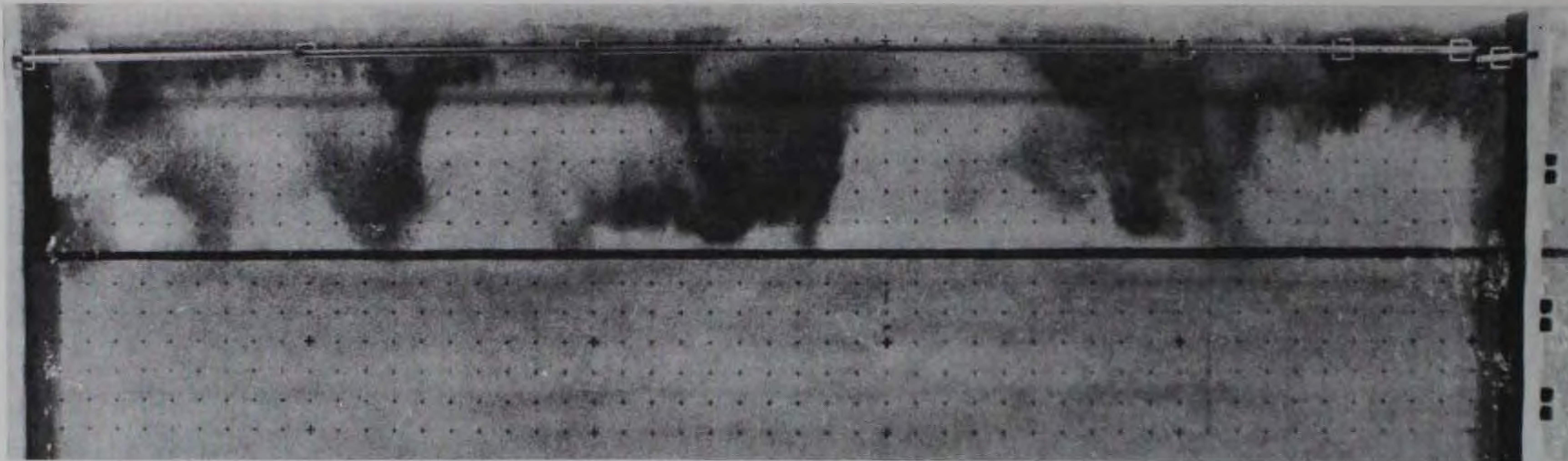
j. Longshore beach width = 4540 ft; elapsed time = 37 min

Photo 9. (sheet 5 of 6)

k. Longshore beach width = 4490 ft; elapsed time = 37 min

l. Longshore beach width = 4390 ft; elapsed time = 34 min

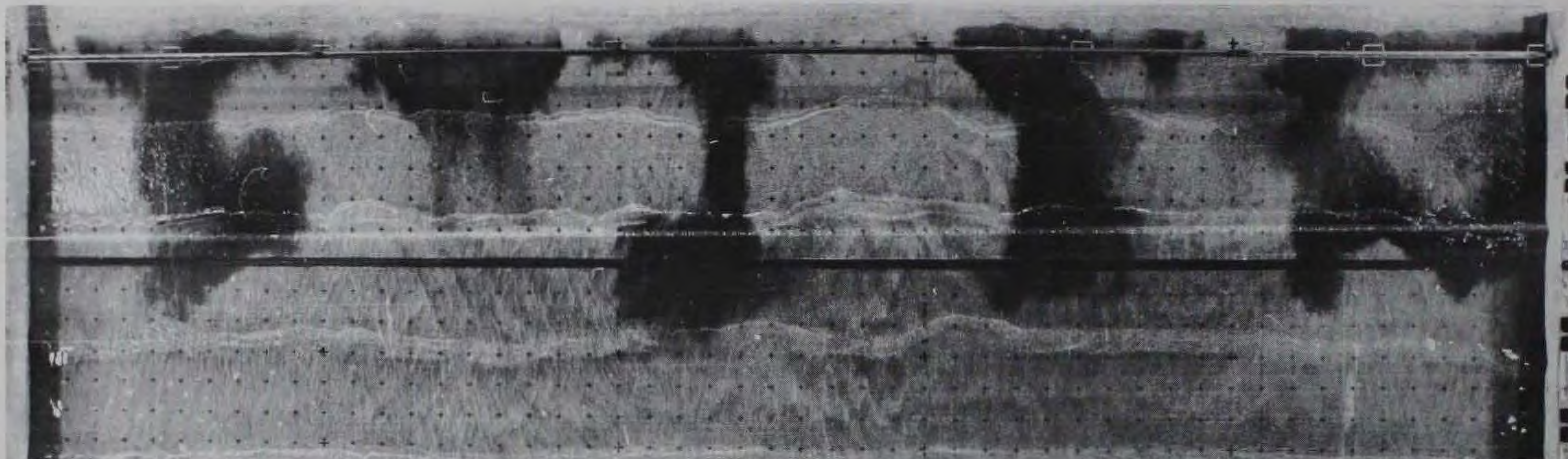
Photo 9. (sheet 6 of 6)



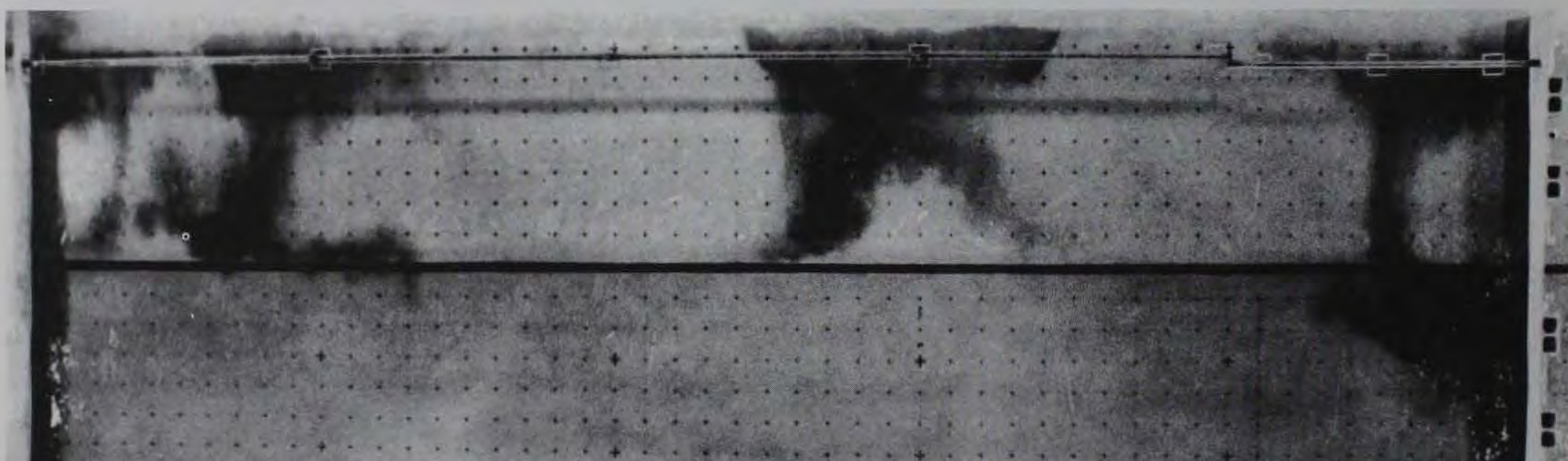
a. Longshore beach width = 5190 ft; elapsed time = 35 min

b. Longshore beach width = 5090 ft; elapsed time = 33 min

Photo 10. Rip current patterns with perched beach installed; 13-sec, 13-ft waves (sheet 1 of 6)

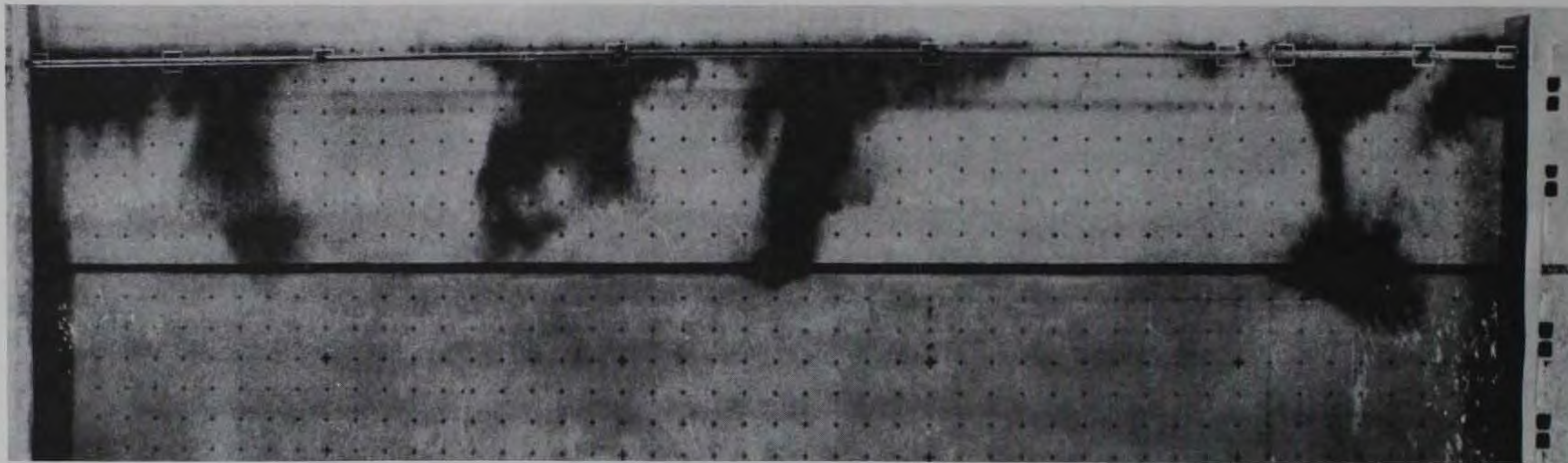


c. Longshore beach width = 5040 ft; elapsed time = 35 min

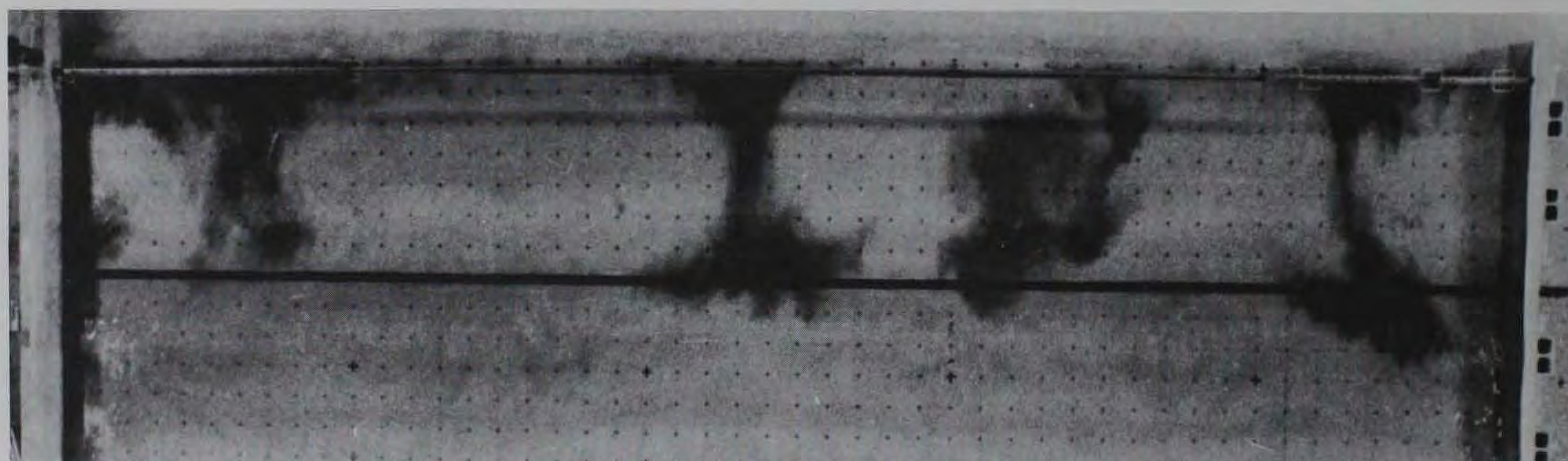


d. Longshore beach width = 4990 ft; elapsed time = 34 min

Photo 10. (sheet 2 of 6)



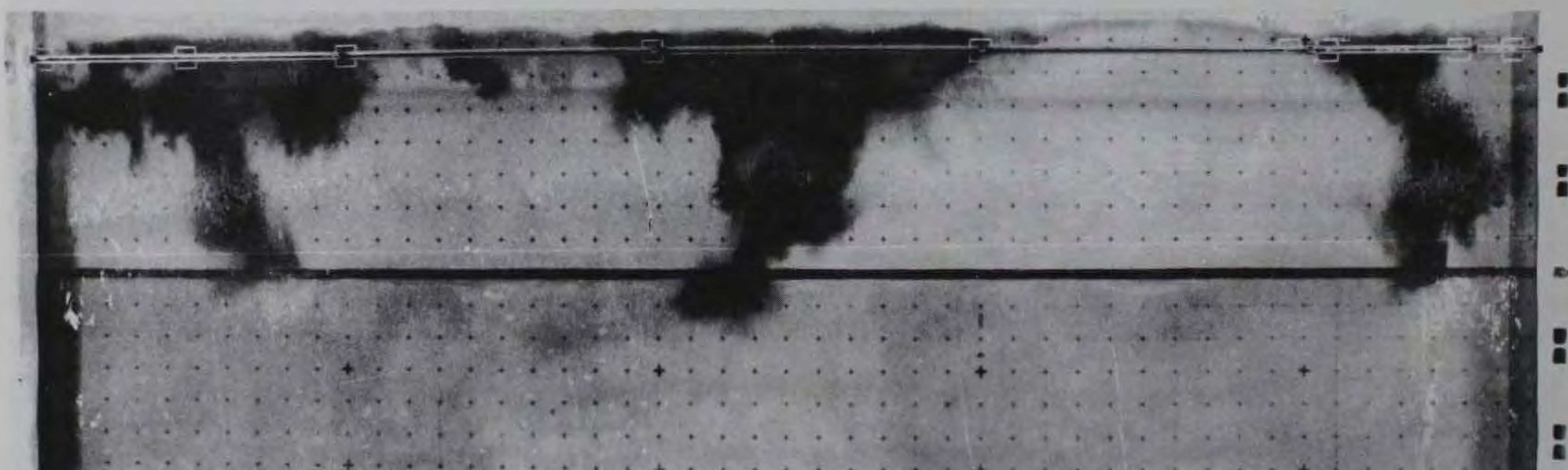
e. Longshore beach width = 4940 ft; elapsed time = 33 min



f. Longshore beach width = 4890 ft; elapsed time = 34 min

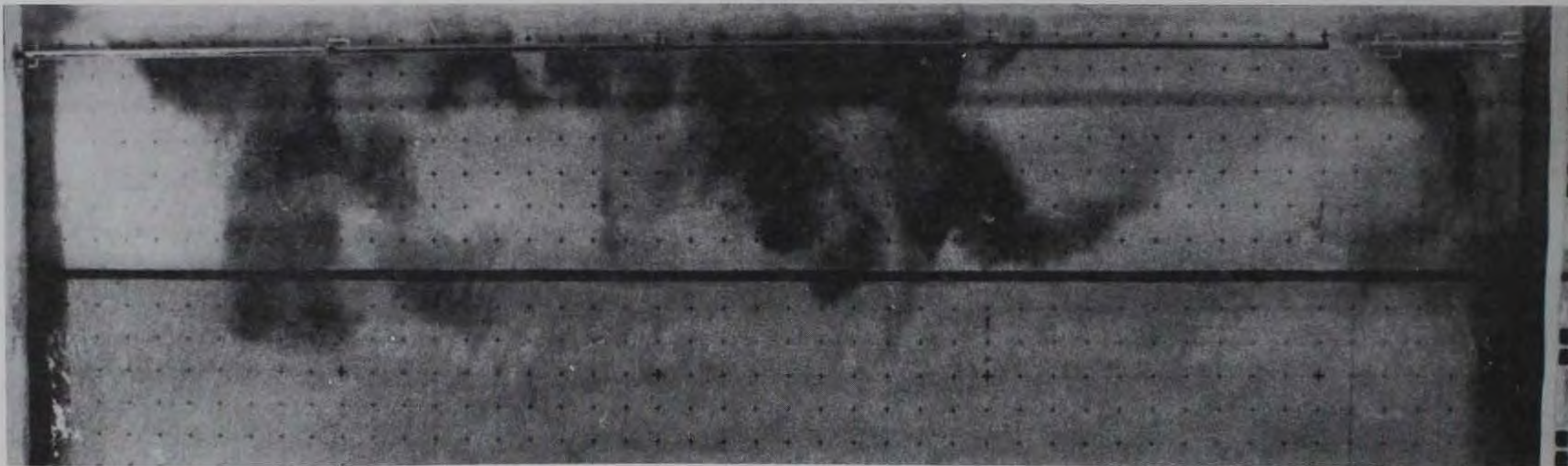
Photo 10. (sheet 3 of 6)

g. Longshore beach width = 4790 ft; elapsed time = 34 min

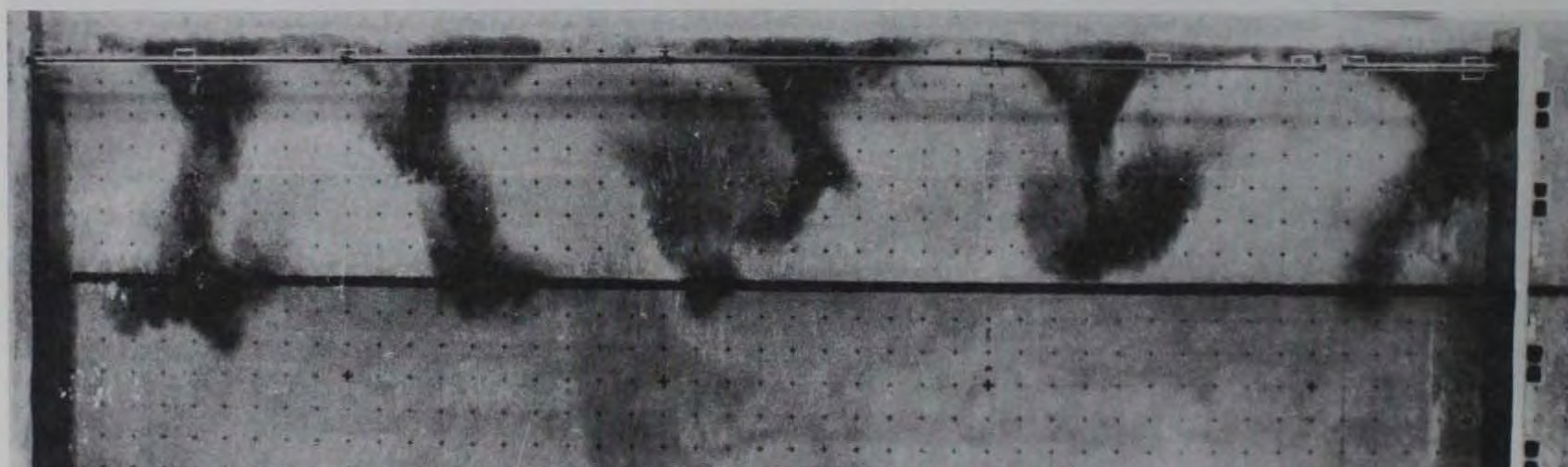


h. Longshore beach width = 4740 ft; elapsed time = 33 min

Photo 10. (sheet 4 of 6)

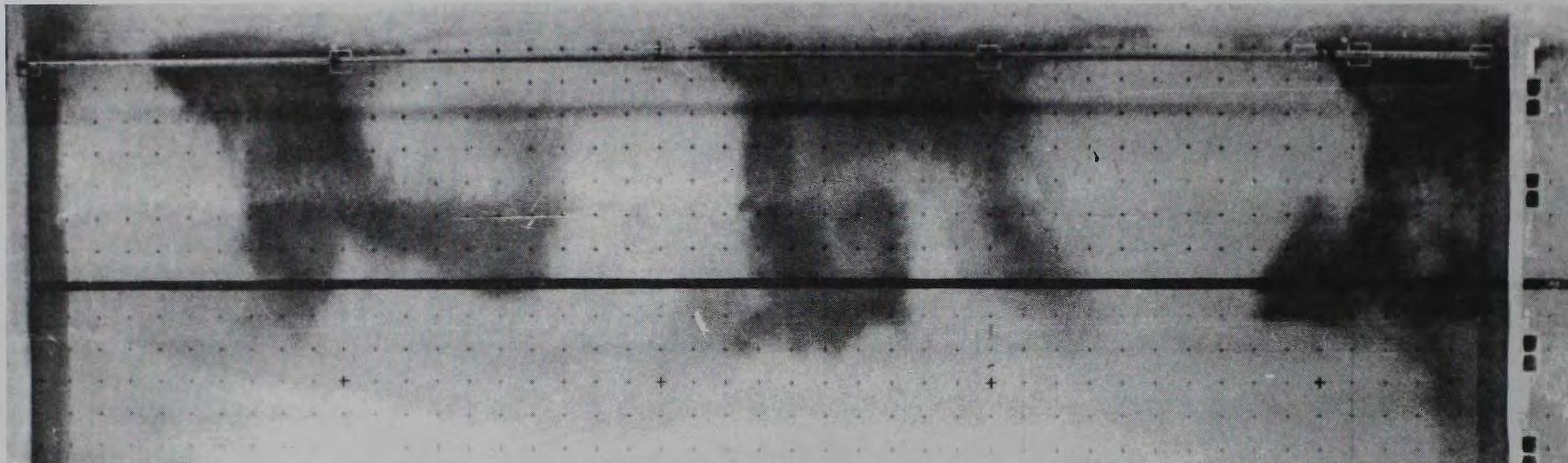


i. Longshore beach width = 4690 ft; elapsed time = 36 min

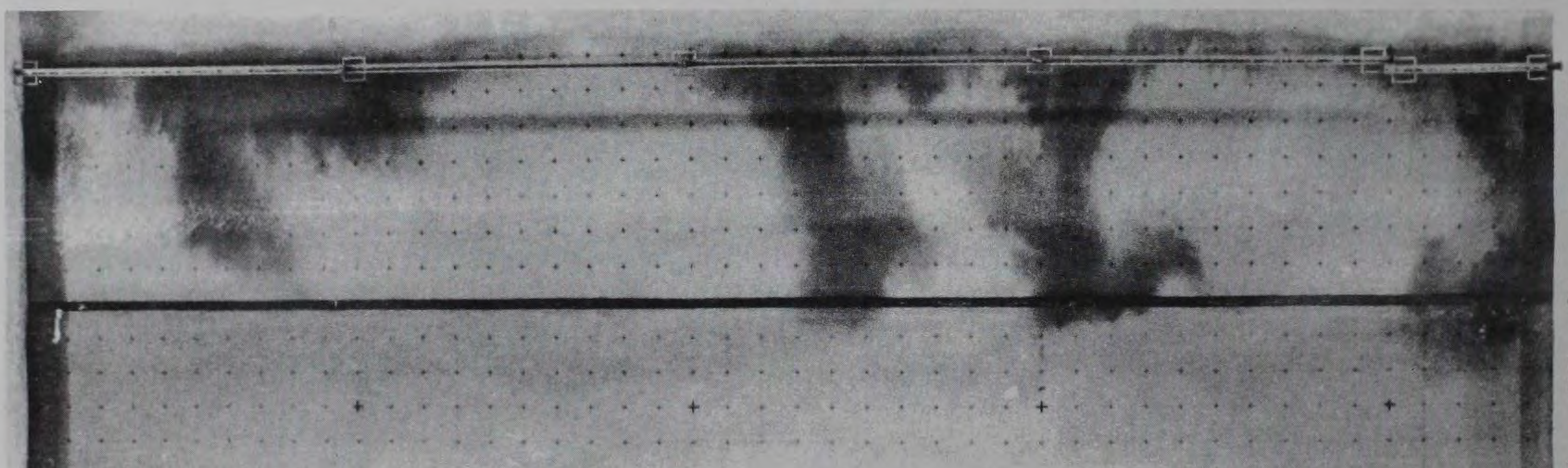


j. Longshore beach width = 4640 ft; elapsed time = 34 min

Photo 10. (sheet 5 of 6)



k. Longshore beach width = 4590 ft; elapsed time = 36 min



l. Longshore beach width = 4490 ft; elapsed time = 33 min

Photo 10. (sheet 6 of 6)

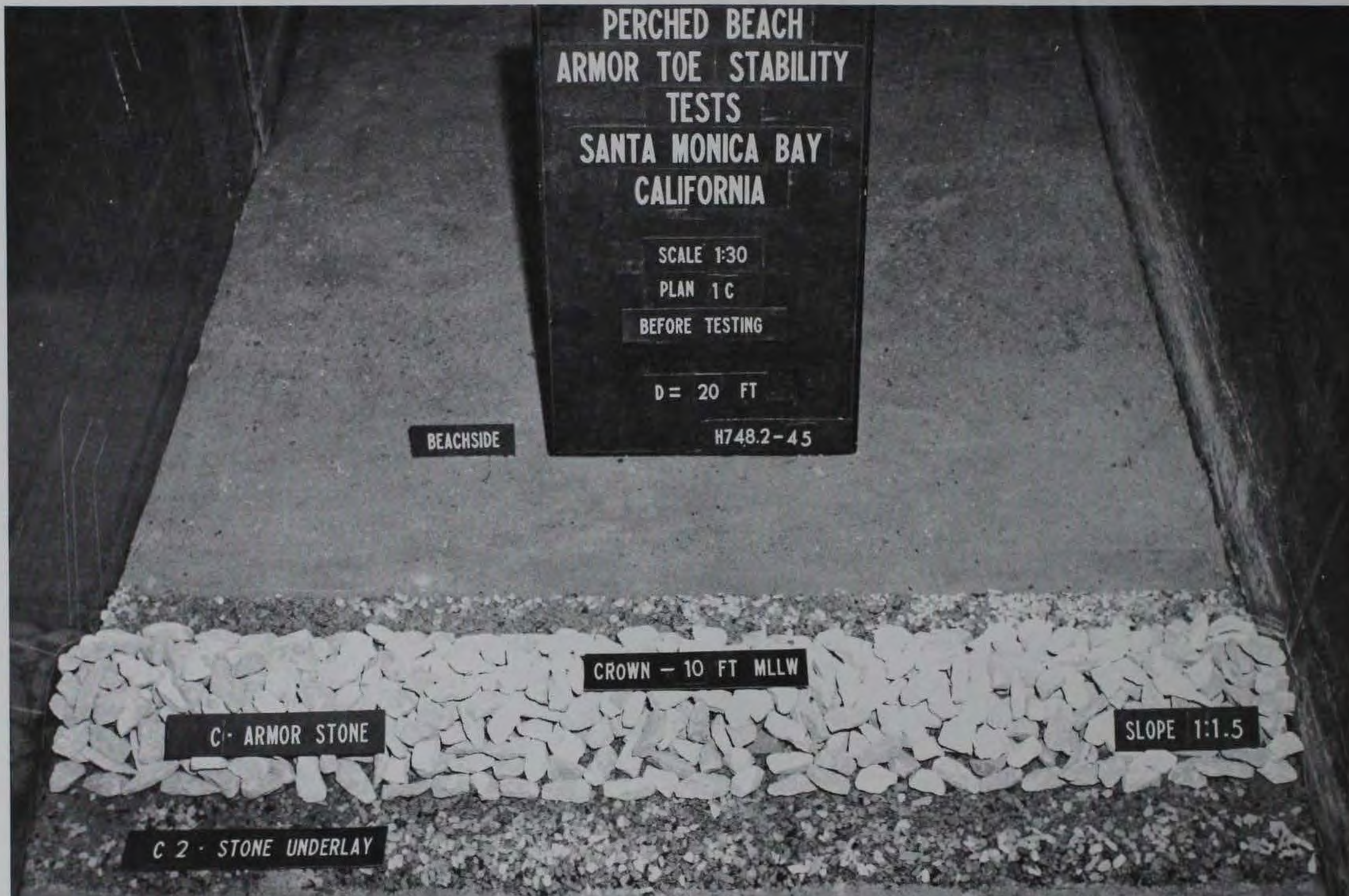


Photo 11. Plan T1C prior to wave attack at 20-ft depth

Photo 12. Plan T1C after attack by 11-sec, 11-ft; 17-sec, 8-ft; and 13-sec, 13-ft waves at 20-ft depth

Photo 13. Plan T1E after attack by 11-sec, 11-ft; 17-sec, 8-ft; and 13-sec, 13-ft waves at 20-ft depth

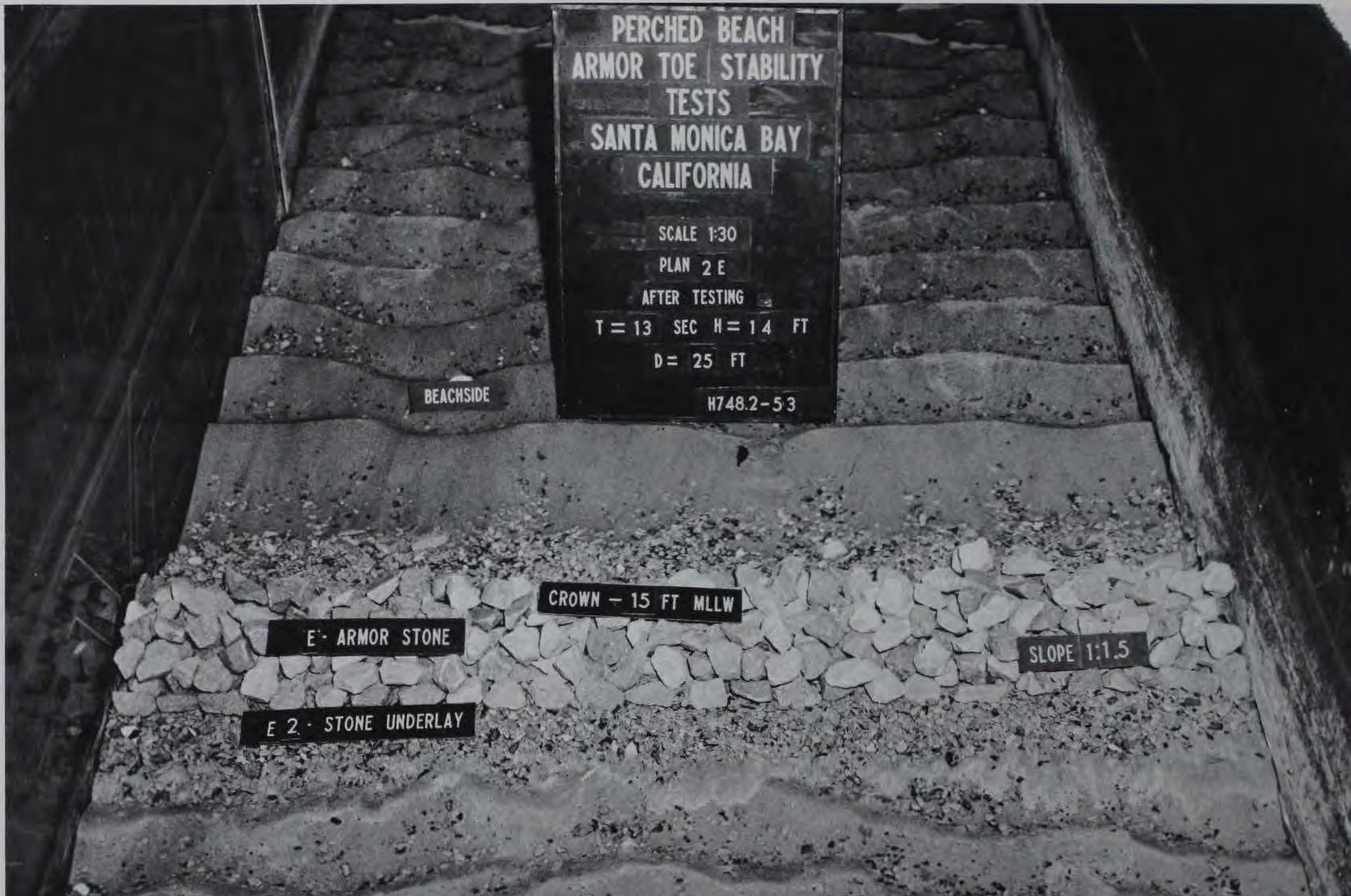


Photo 14. Plan T2E after attack by 11-sec, 13.5-ft; 17-sec, 12-ft; and 13-sec, 14-ft waves at 25-ft depth

Photo 15. Plan T3C after attack by 11-sec, 13.5-ft; 17-sec, 12-ft; and 13-sec, 14-ft waves at 30-ft depth

Photo 16. Plan T3D after attack by 11-sec, 13.5-ft; 17-sec, 12-ft; and 13-sec, 14-ft waves at 30-ft depth

Photo 17. Plan T3E after attack by 11-sec, 13.5-ft; 17-sec, 12-ft; and 13-sec, 14-ft waves at 30-ft depth

Photo 18. Plan T5F after attack by 11-sec, 13.5-ft; 17-sec, 12-ft; and 13-sec, 14-ft waves at 45-ft depth

Photo 19. Plan A3 after attack of 11-sec, 11-ft; 17-sec, 8-ft; and 13-sec, 13-ft waves at 20-ft depth

Photo 20. Plan A3 after attack of 11-sec, 13.5-ft; 17-sec, 12-ft; and 13-sec, 14-ft waves at 25-ft depth

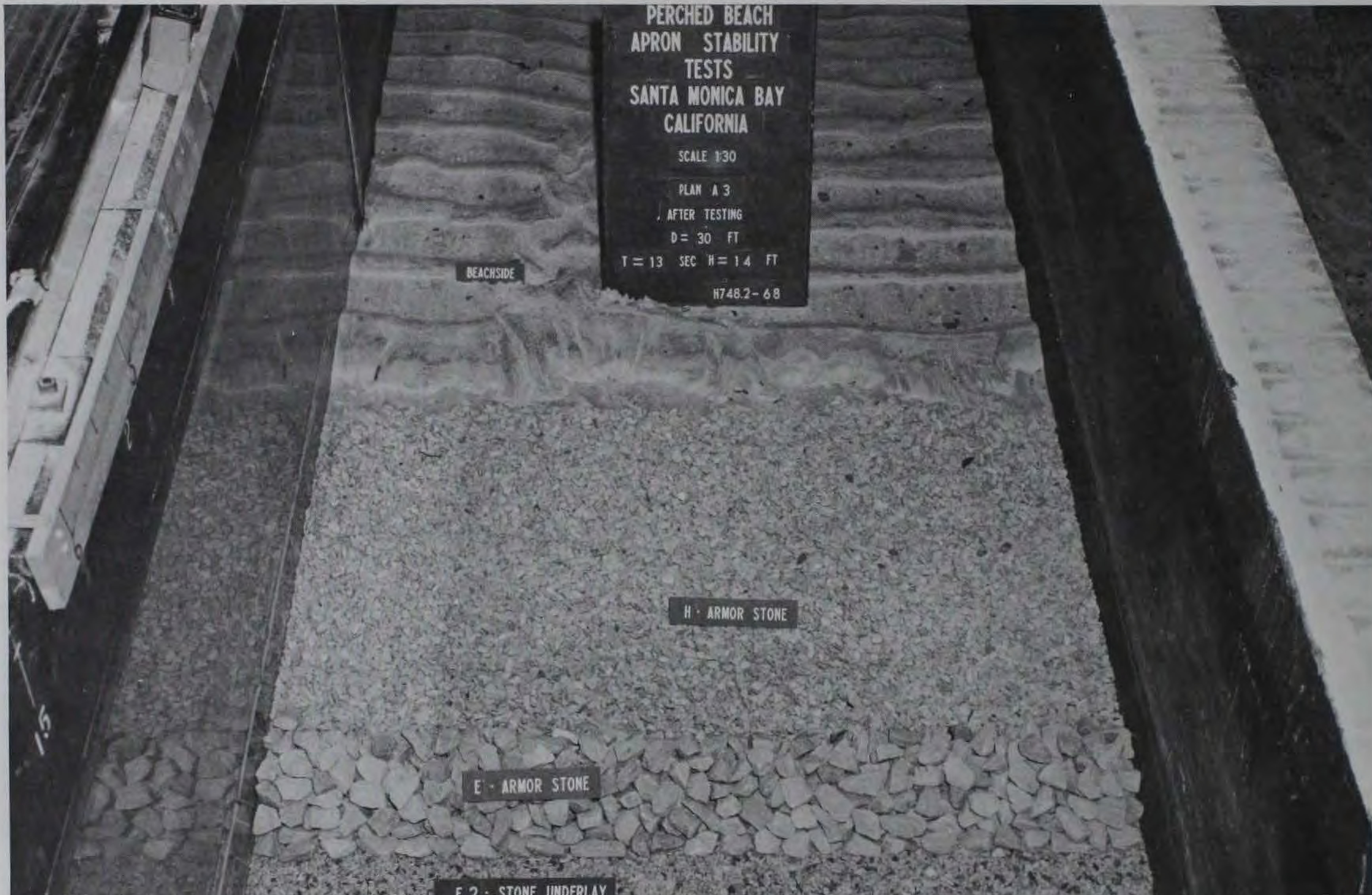


Photo 21. Plan A3 after attack of 11-sec, 13.5-ft; 17-sec, 12-ft; and 13-sec, 14-ft waves at 30-ft depth

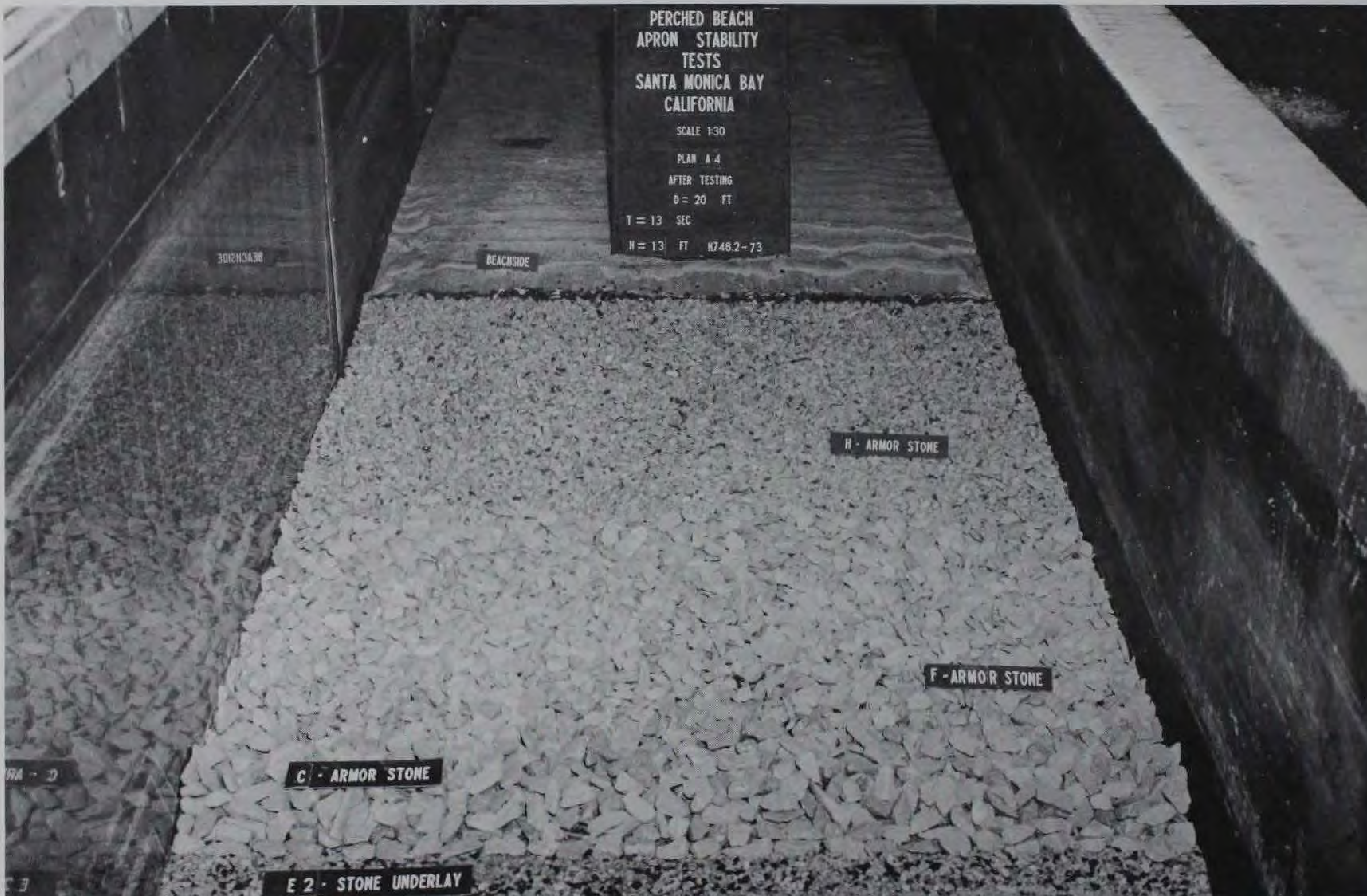


Photo 22. Plan A4 after attack of 11-sec, 11-ft; 17-sec, 8-ft; and 13-sec, 13-ft waves at 20-ft depth

Photo 23. Plan A4 after attack of 11-sec, 13.5-ft; 17-sec, 12-ft; and 13-sec, 14-ft waves at 25-ft depth

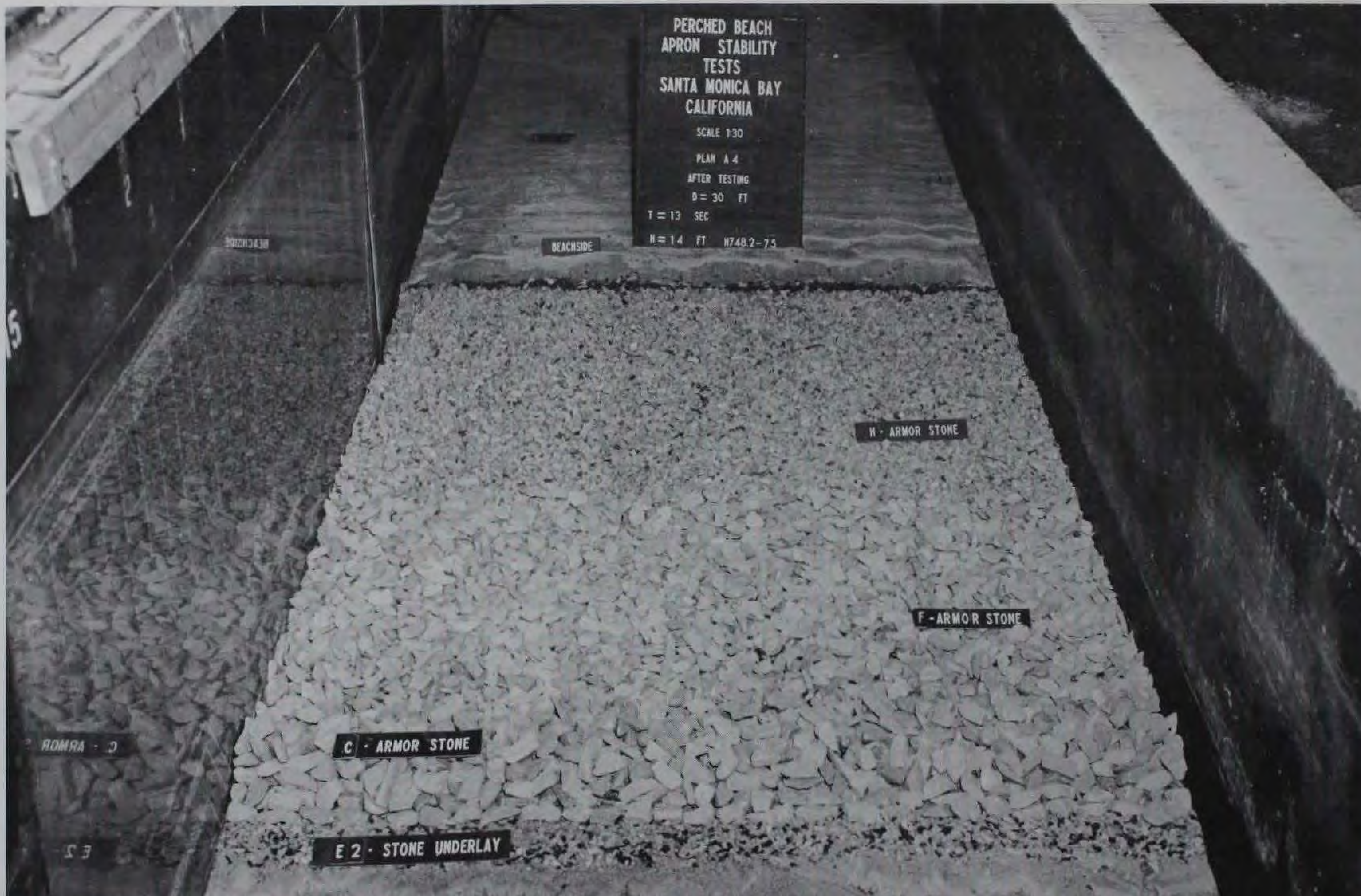
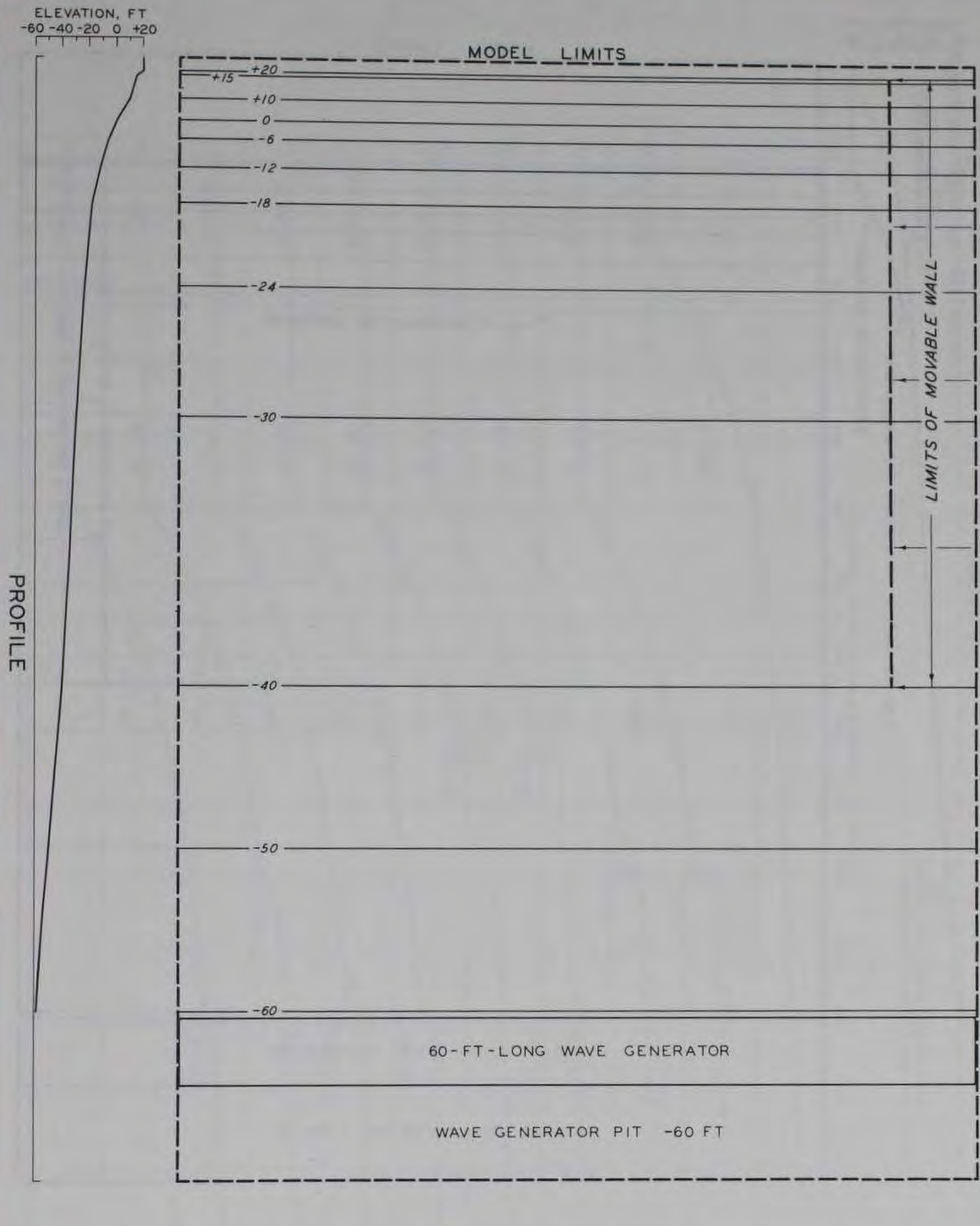


Photo 24. Plan A4 after attack of 11-sec, 13.5-ft; 17-sec, 12-ft; and 13-sec, 14-ft waves at 30-ft depth



NOTE: CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOWER LOW WATER

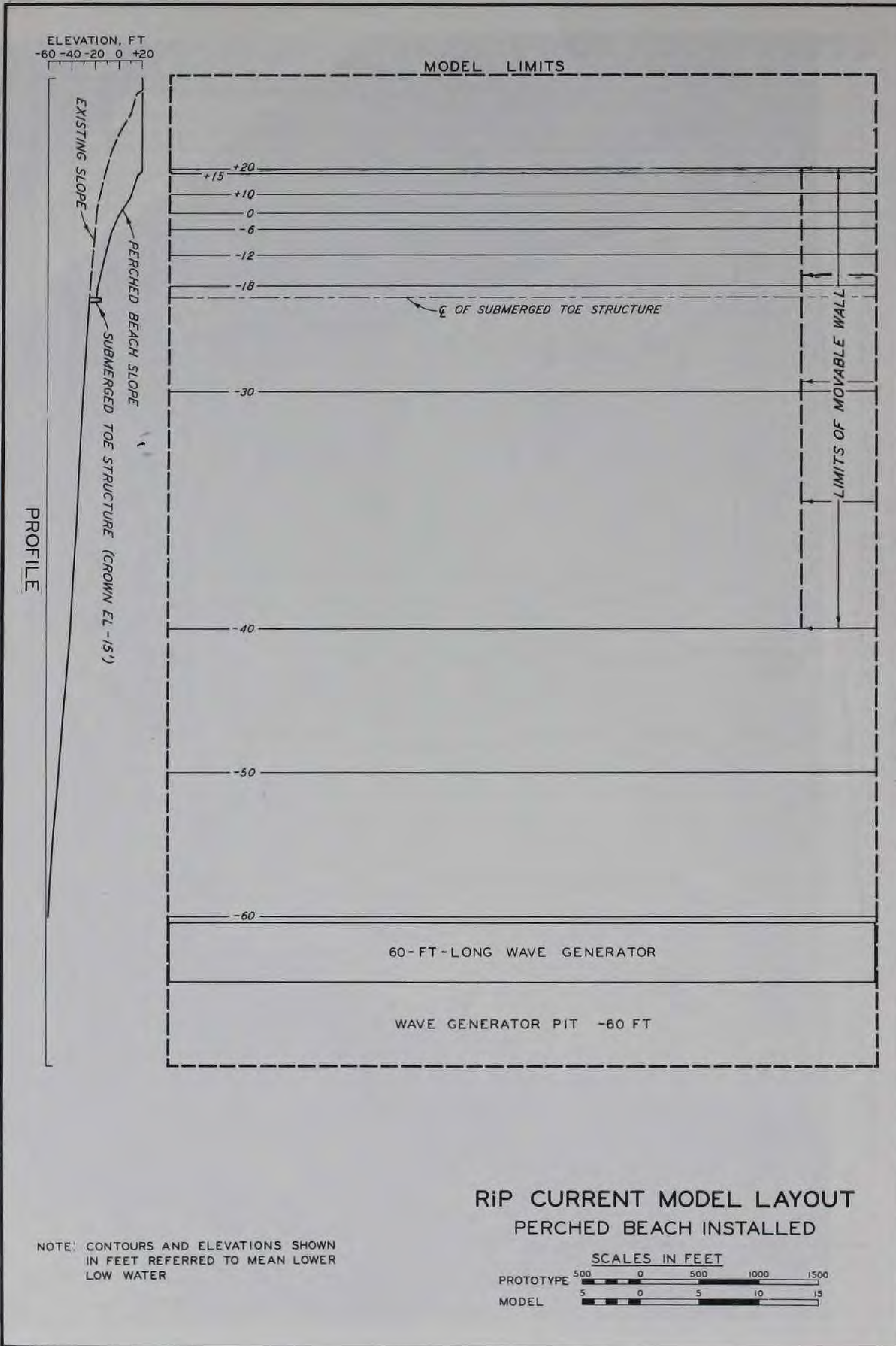
RIP CURRENT MODEL LAYOUT

EXISTING CONDITIONS INSTALLED

SCALES IN FEET

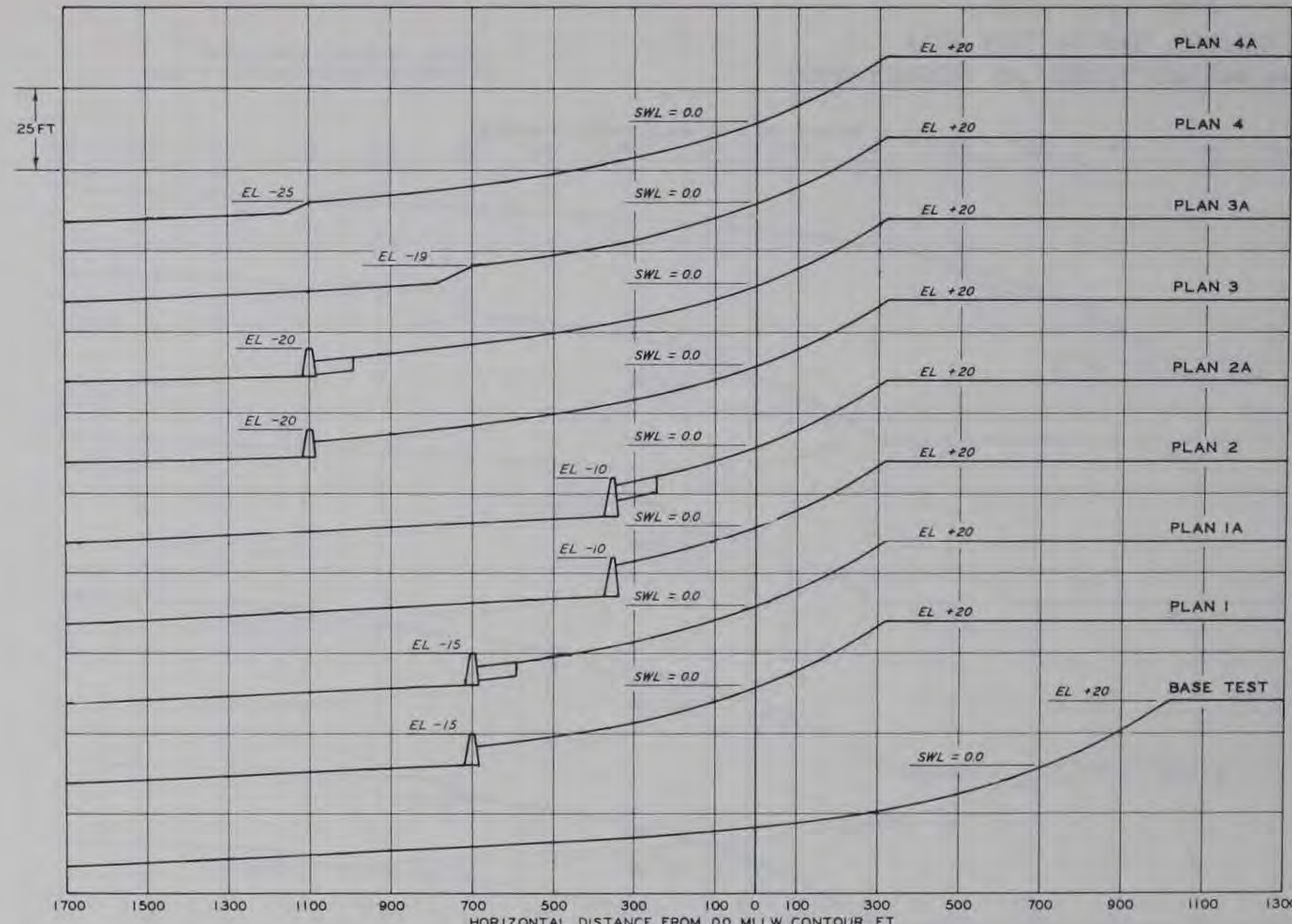
PROTOTYPE 500 0 500 1000 1500

MODEL 5 0 5 10 15



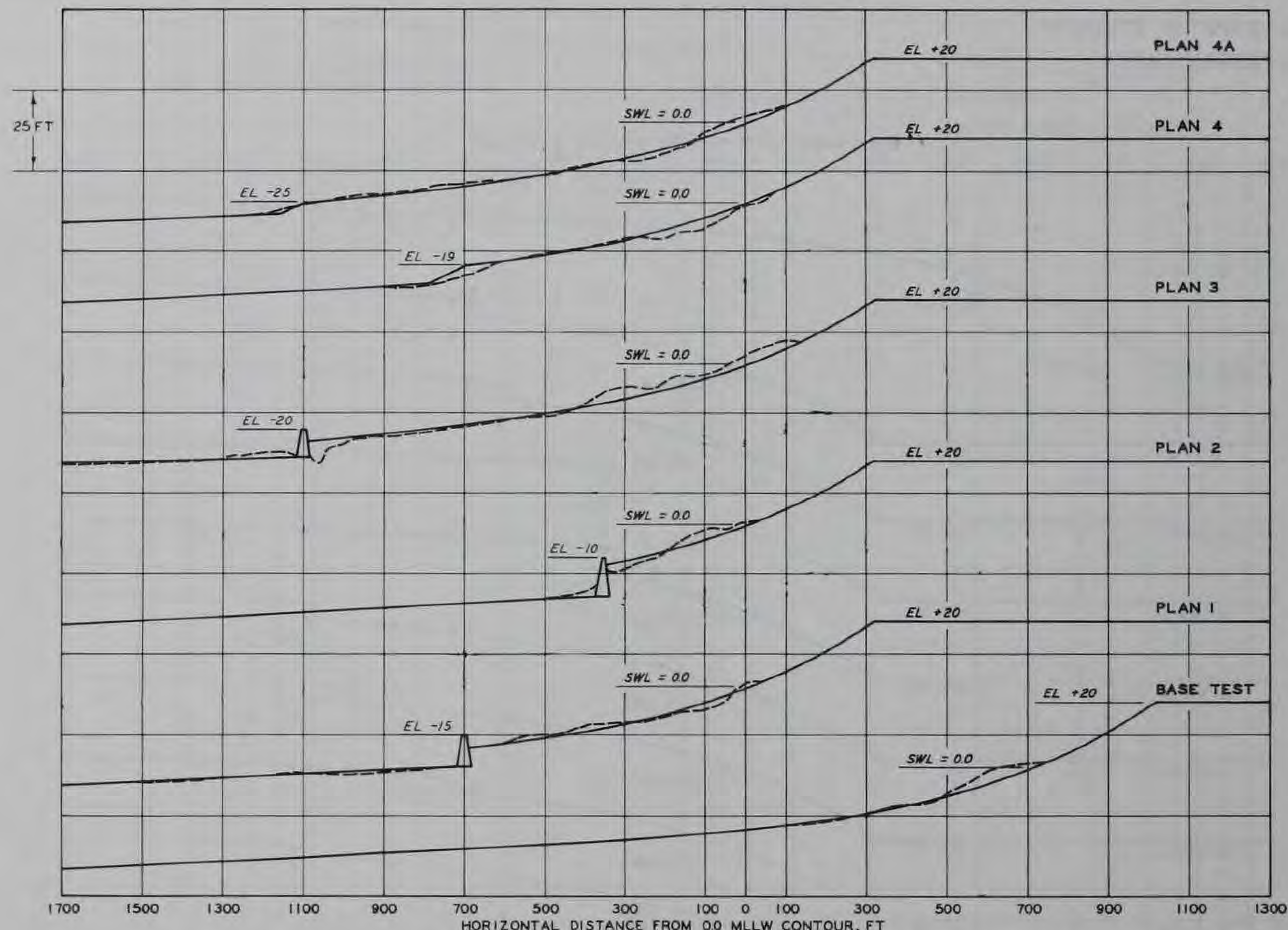
ELEVATION, FT
-60 -40 -20 0 +20
EXISTING SLOPE
PERCHED BEACH SLOPE
SUBMERGED TOE STRUCTURE (CROWN EL -15')
PROFILE
MODEL LIMITS
+20
+15
+10
0
-6
-12
-18
℄ OF SUBMERGED TOE STRUCTURE
-30
-40
-50
-60
LIMITS OF MOVABLE WALL
60-FT-LONG WAVE GENERATOR
WAVE GENERATOR PIT -60 FT
RIP CURRENT MODEL LAYOUT
PERCHED BEACH INSTALLED
NOTE: CONTOURS AND ELEVATIONS SHOWN IN FEET REFERRED TO MEAN LOWER LOW WATER
SCALES IN FEET
PROTOTYPE 500 0 500 1000 1500
MODEL 5 0 5 10 15

PLATE 2



NOTE: ALL ELEVATIONS (EL) REFERRED TO MEAN LOWER LOW WATER (MLLW)

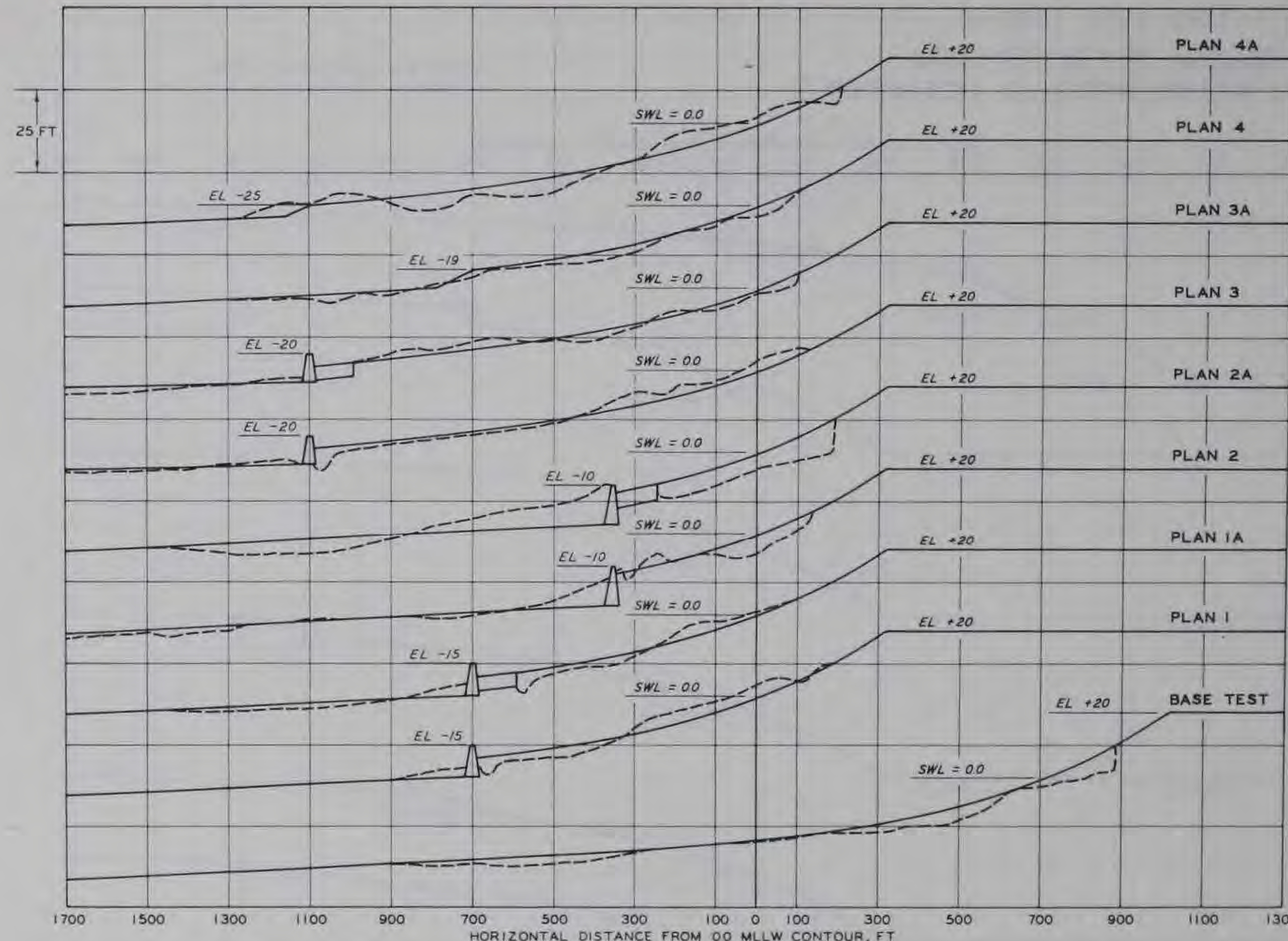
COMPARISON OF VARIOUS PLANS TESTED



NOTE: ALL ELEVATIONS (EL) REFERRED TO MEAN LOWER LOW WATER (MLLW)

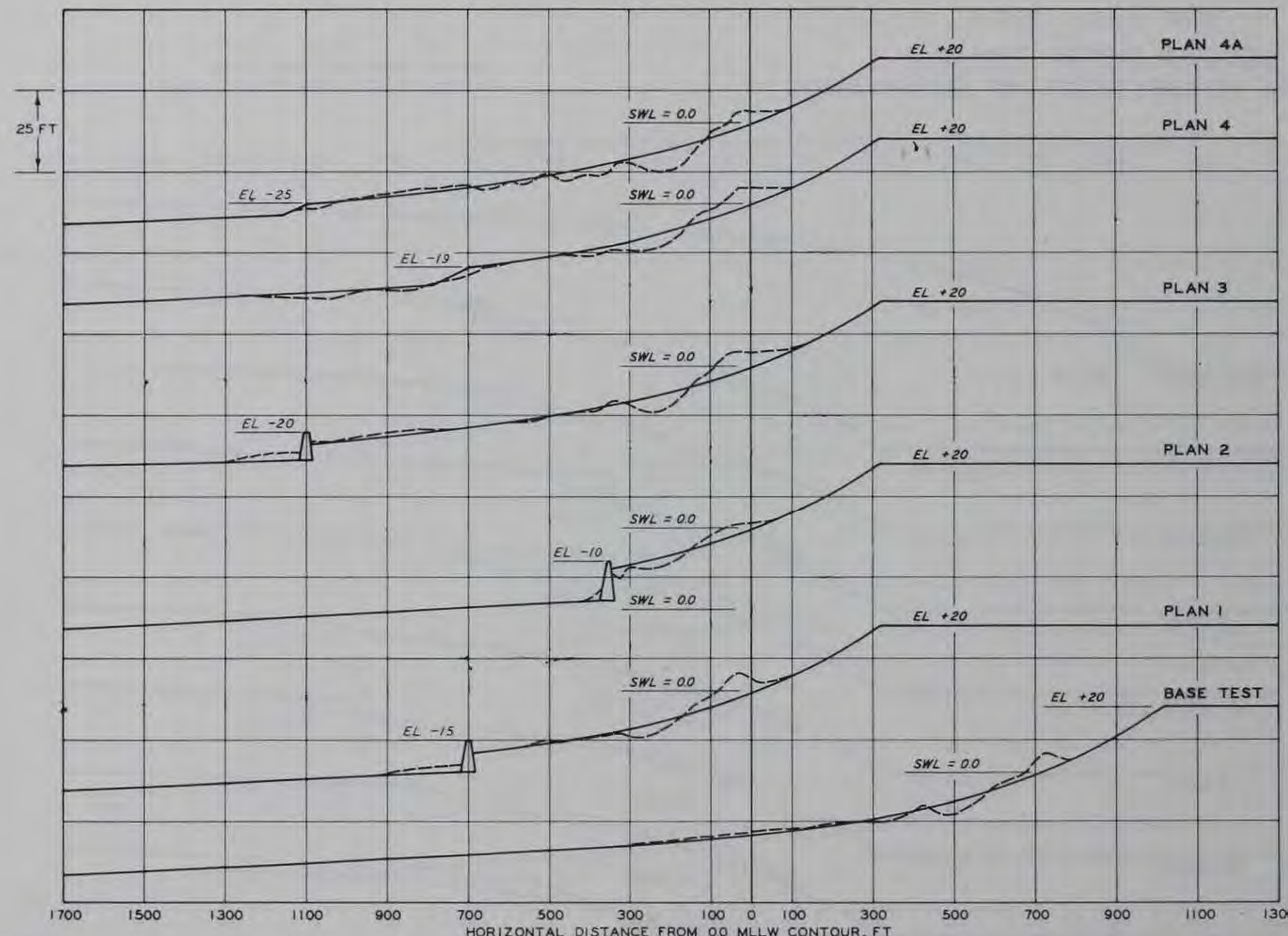
COMPARISON OF EQUILIBRIUM PROFILES FOR ALL PLANS TESTED

7-SEC, 5-FT WAVE



NOTE: ALL ELEVATIONS (EL) REFERRED TO MEAN LOWER LOW WATER (MLLW)

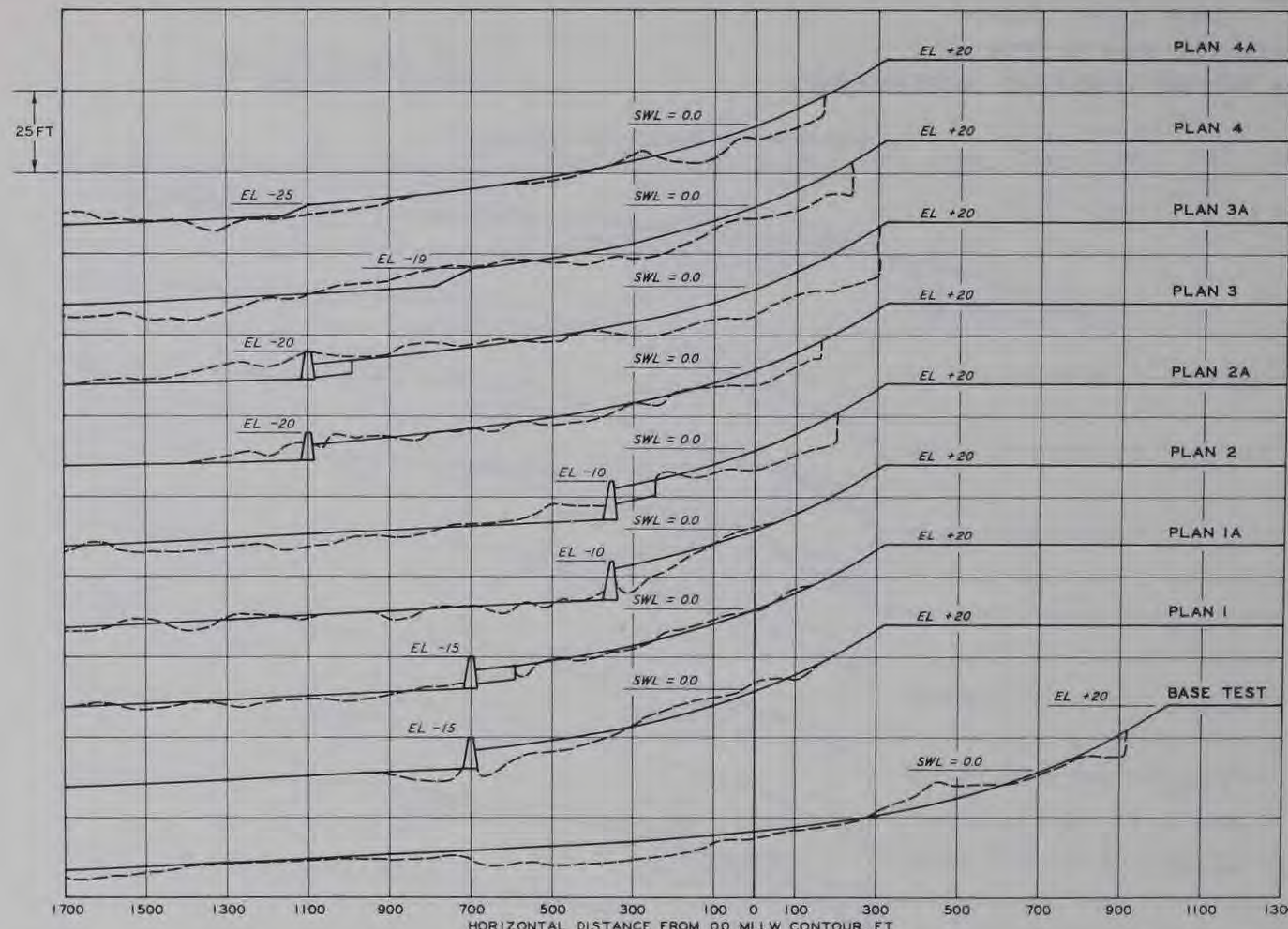
COMPARISON OF EQUILIBRIUM PROFILES
FOR ALL PLANS TESTED
7.9-SEC, 10-FT WAVE



NOTE: ALL ELEVATIONS (EL) REFERRED TO MEAN LOWER LOW WATER (MLLW)

COMPARISON OF EQUILIBRIUM PROFILES FOR ALL PLANS TESTED

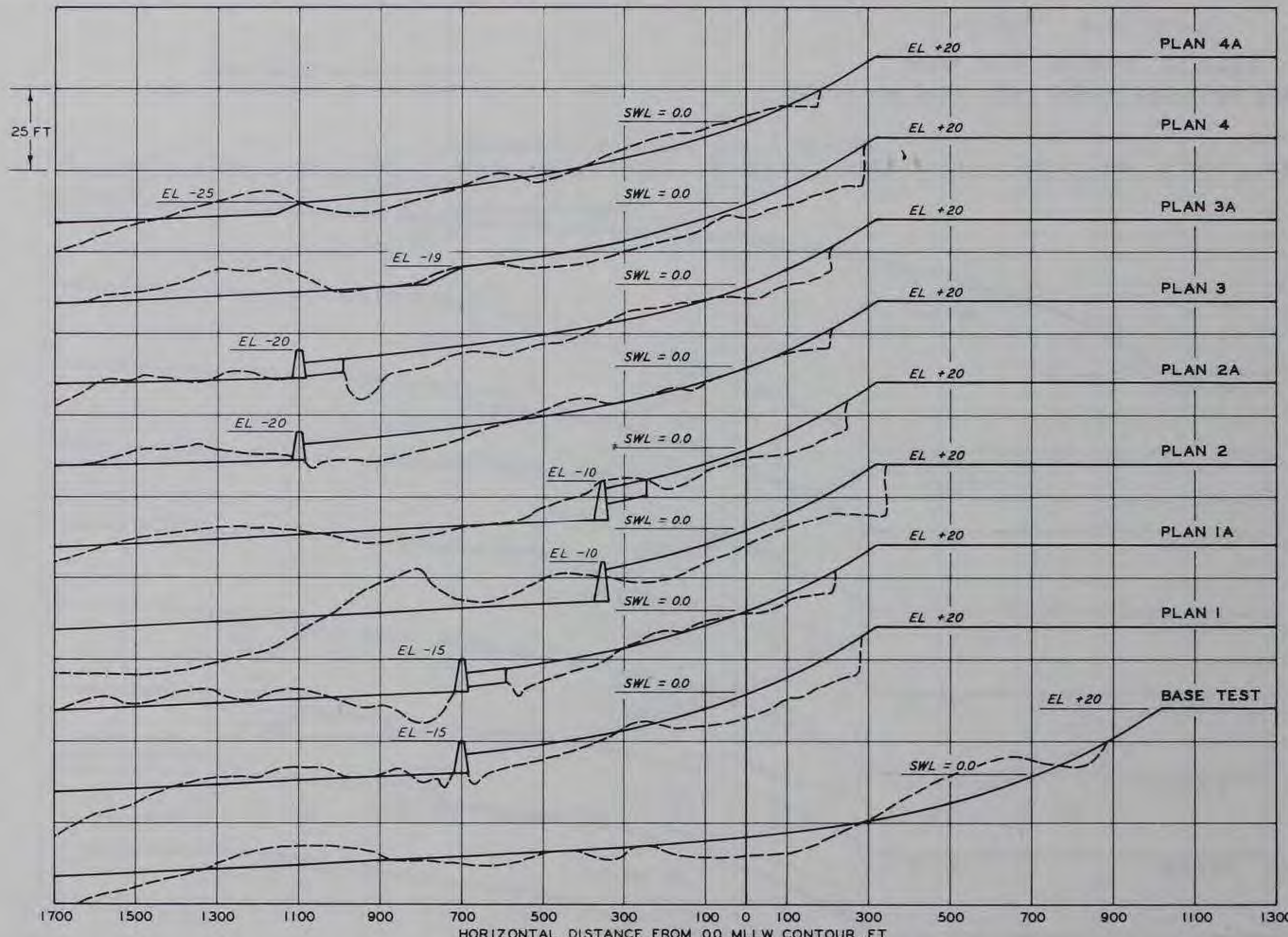
10-SEC, 3-FT WAVE



NOTE: ALL ELEVATIONS (EL) REFERRED TO MEAN LOWER LOW WATER (MLLW)

COMPARISON OF EQUILIBRIUM PROFILES FOR ALL PLANS TESTED

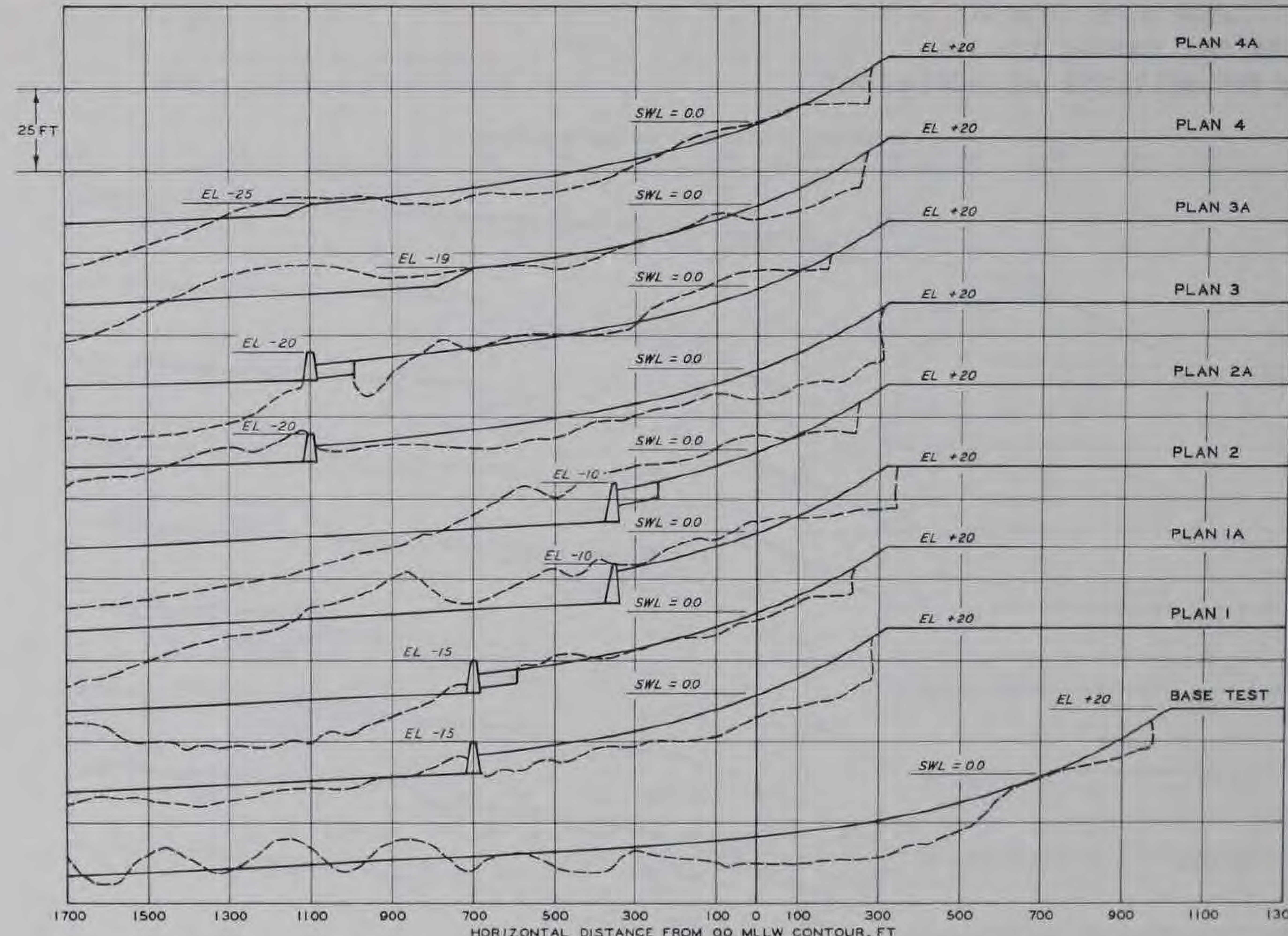
10-SEC, 8-FT WAVE



NOTE: ALL ELEVATIONS (EL) REFERRED TO MEAN LOWER LOW WATER (MLLW)

COMPARISON OF EQUILIBRIUM PROFILES FOR ALL PLANS TESTED

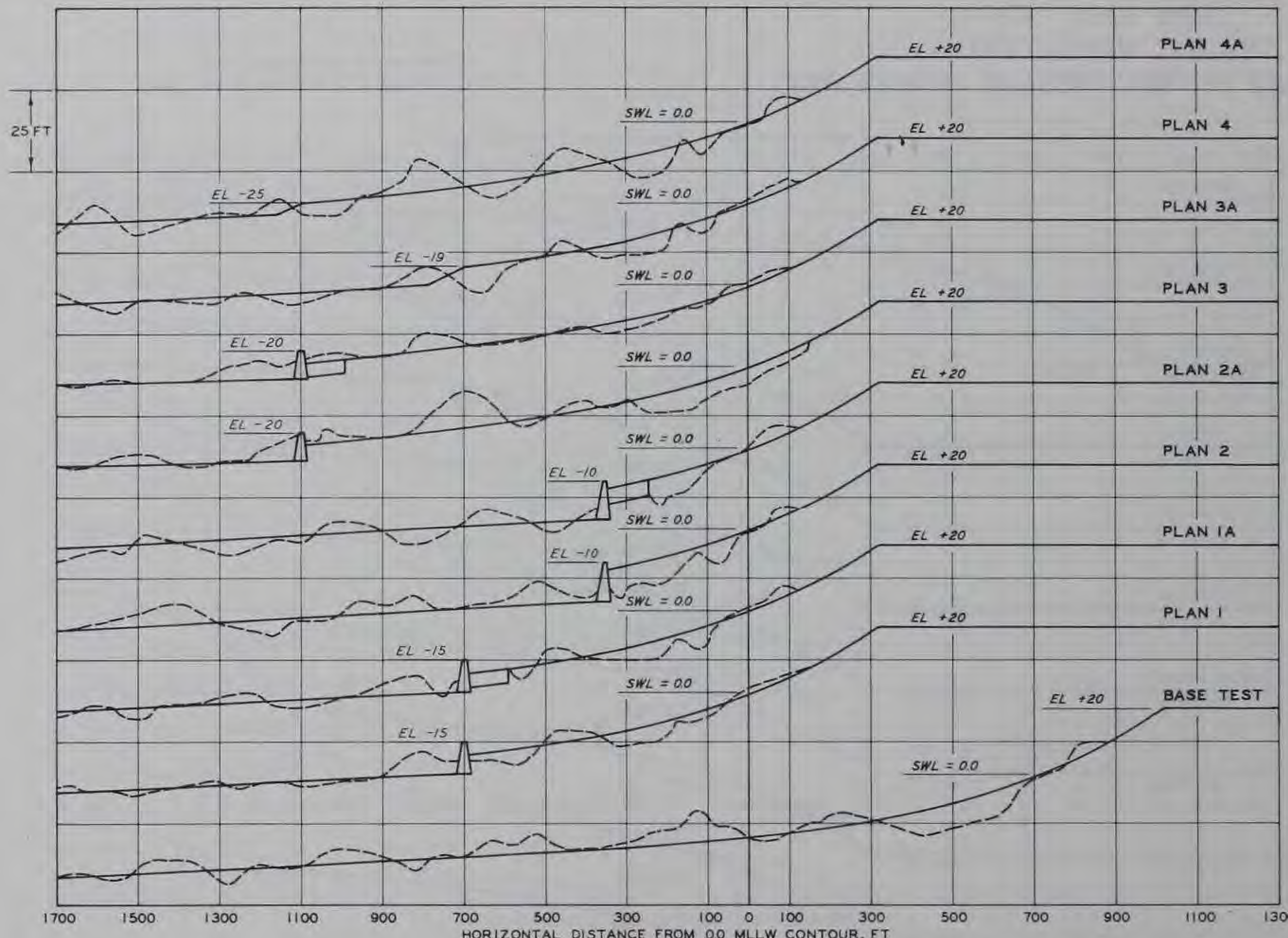
10-SEC, 14-FT WAVE



NOTE: ALL ELEVATIONS (EL) REFERRED TO MEAN LOWER LOW WATER (MLLW)

COMPARISON OF EQUILIBRIUM PROFILES FOR ALL PLANS TESTED

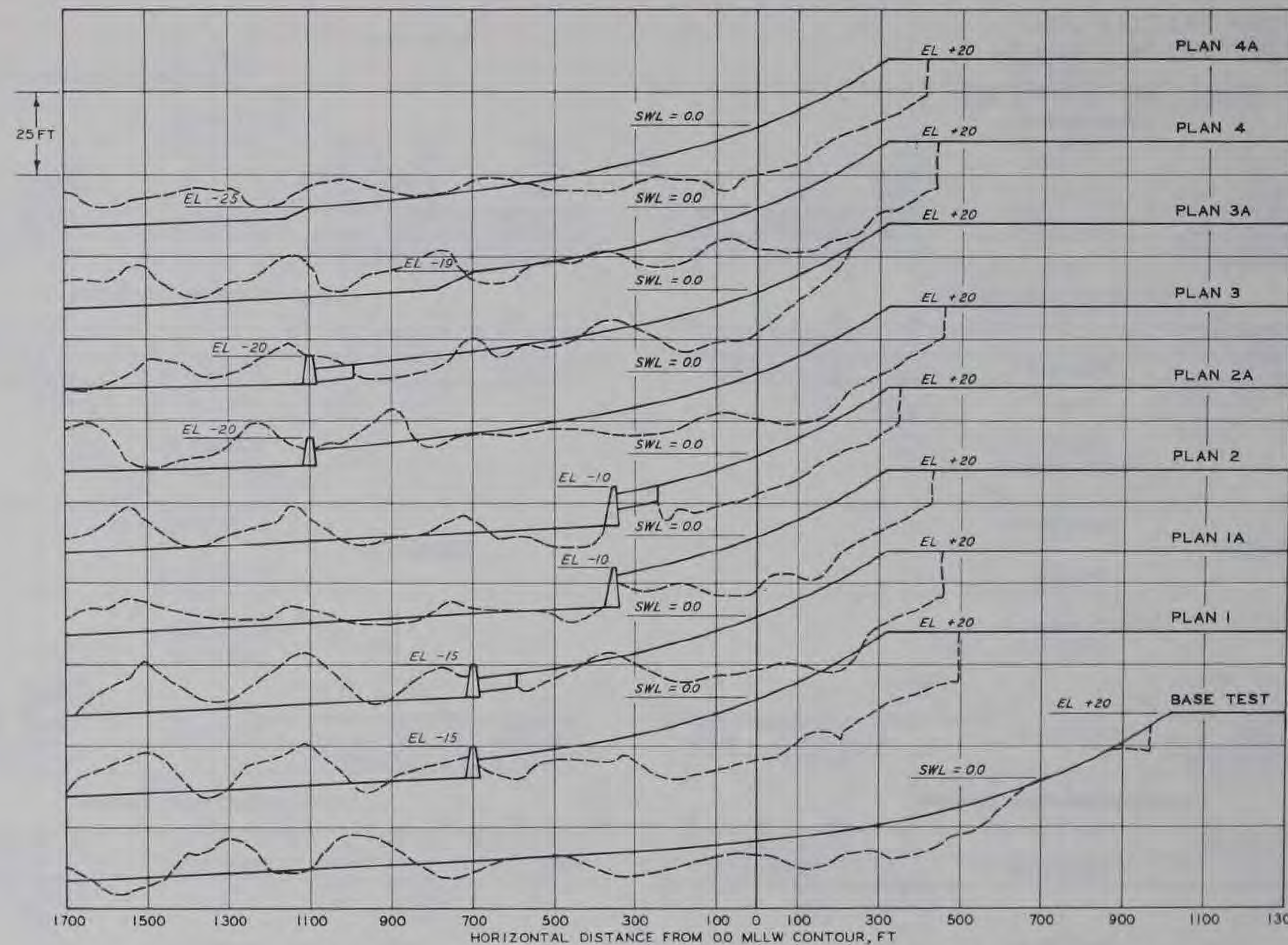
11.3-SEC, 12-FT WAVE



NOTE: ALL ELEVATIONS (EL) REFERRED TO MEAN LOWER LOW WATER (MLLW)

COMPARISON OF EQUILIBRIUM PROFILES FOR ALL PLANS TESTED

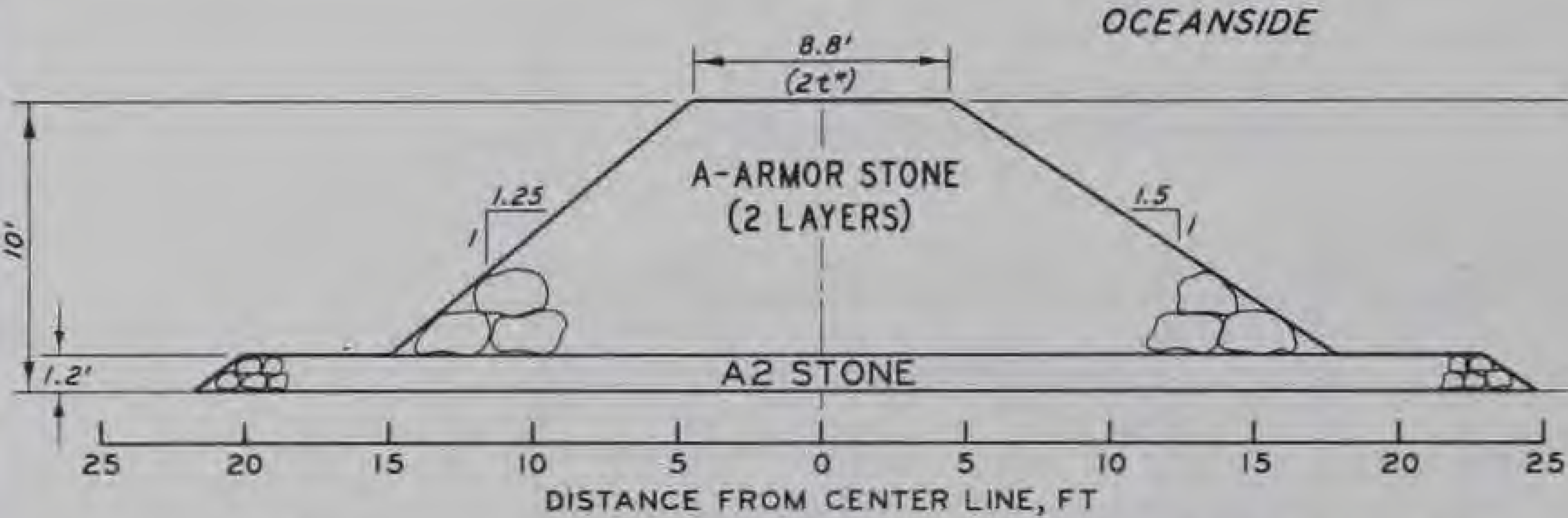
16-SEC, 4-FT WAVE



NOTE: ALL ELEVATIONS (EL) REFERRED TO MEAN LOWER LOW WATER (MLLW)

COMPARISON OF EQUILIBRIUM PROFILES FOR ALL PLANS TESTED

16-SEC, 8-FT WAVE

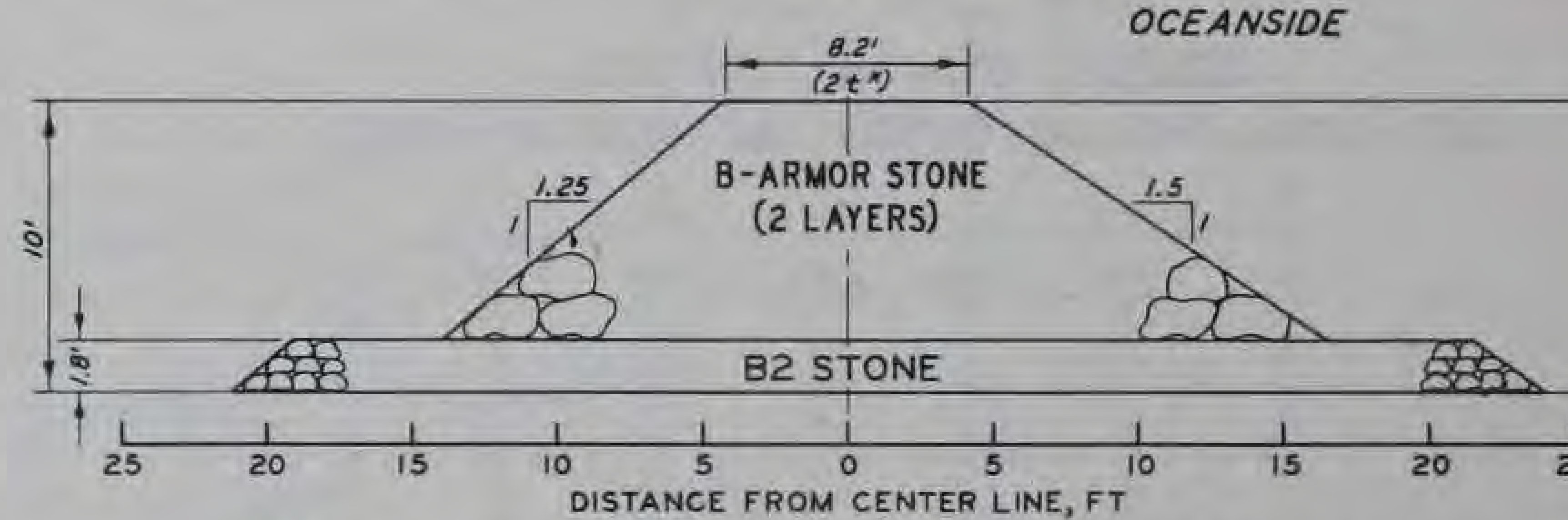


LEGEND

STONE SIZE	WEIGHT, LB PROTOTYPE	MODEL
A	8300	0.25
A2	<100	0.00124

ELEMENTS OF PLANS

PLAN	CREST EL FT MLLW	BOTTOM EL FT MLLW
T1A	-10	-20
T2A	-15	-25

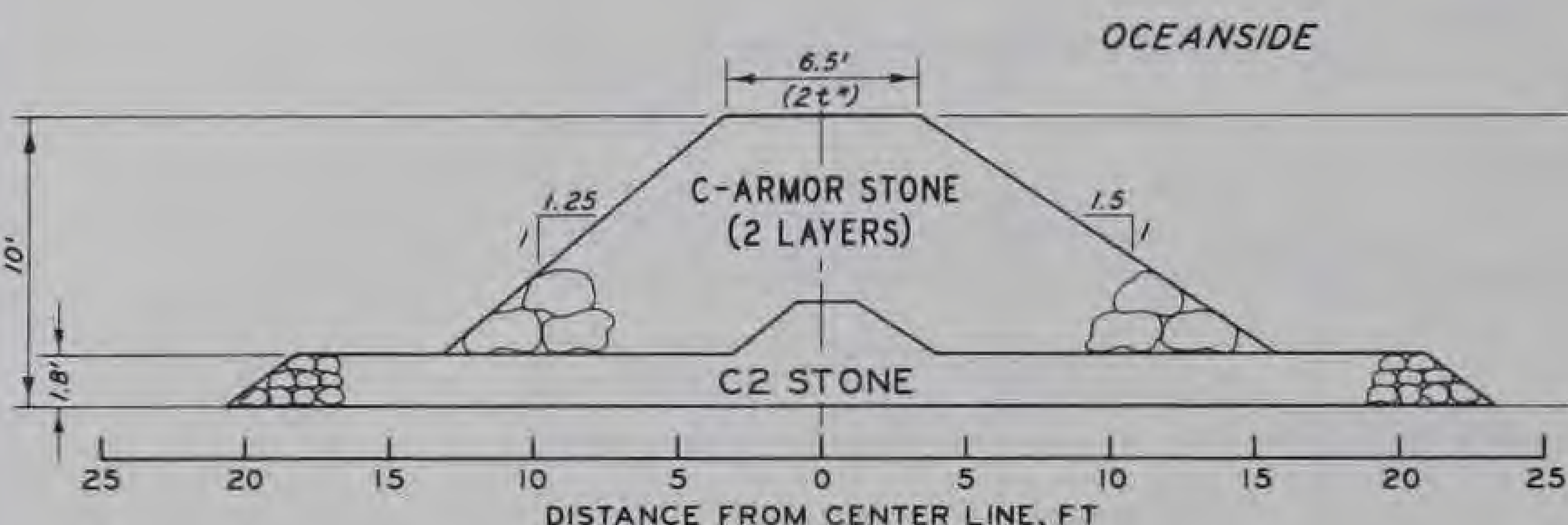


LEGEND

STONE SIZE	WEIGHT, LB PROTOTYPE	MODEL
B	6600	0.20
B2	<100	0.00099

ELEMENTS OF PLANS

PLAN	CREST EL FT MLLW	BOTTOM EL FT MLLW
T1B	-10	-20
T2B	-15	-25

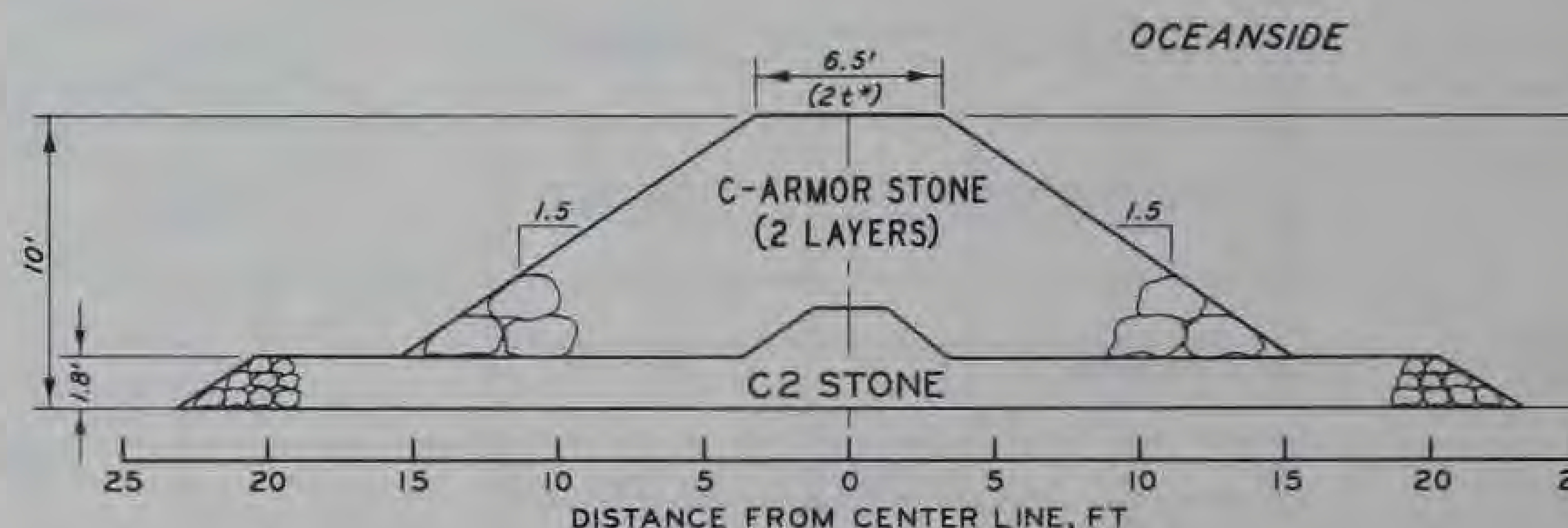


LEGEND

STONE SIZE	WEIGHT, LB PROTOTYPE	MODEL
C	3300	0.10
C2	<100	0.00048

ELEMENTS OF PLANS

PLAN	CREST EL FT MLLW	BOTTOM EL FT MLLW
T1C	-10	-20
T2C	-15	-25
T3C	-20	-30
T5C	-35	-45



LEGEND

STONE SIZE	WEIGHT, LB PROTOTYPE	MODEL
C	3300	0.16
C2	<100	0.00048

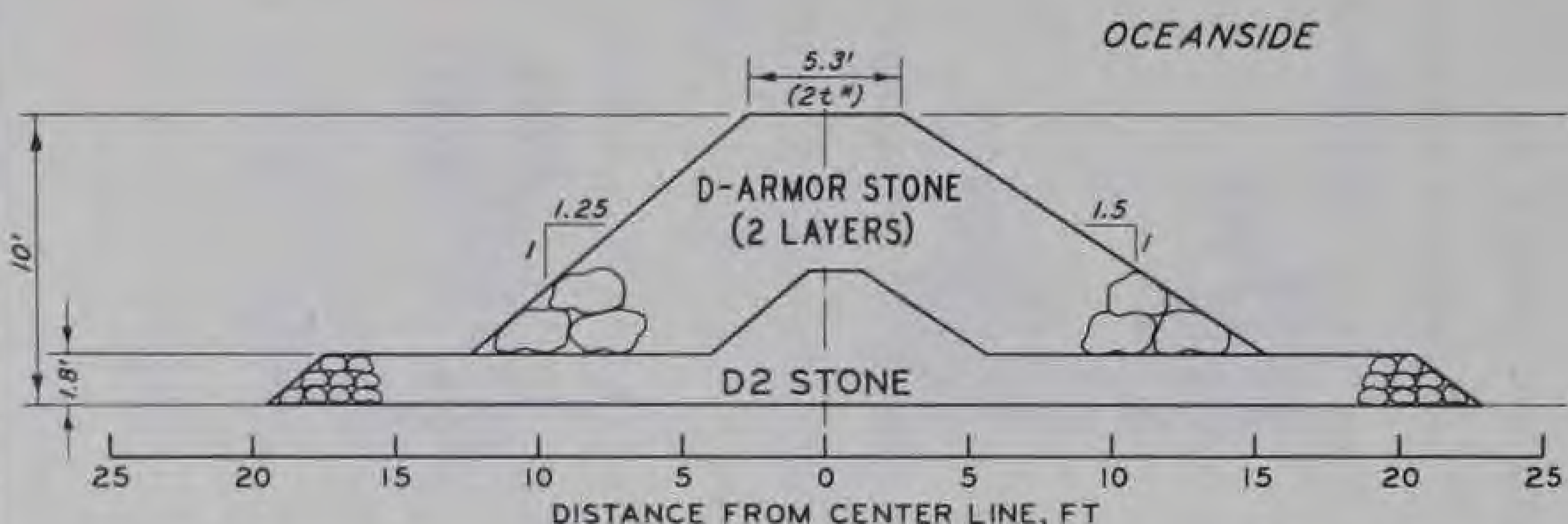
ELEMENTS OF PLANS

PLAN	CREST EL FT MLLW	BOTTOM EL FT MLLW
T2C-1	-15	-25

NOTE: *t IS ONE THICKNESS OF STONE SIZE.

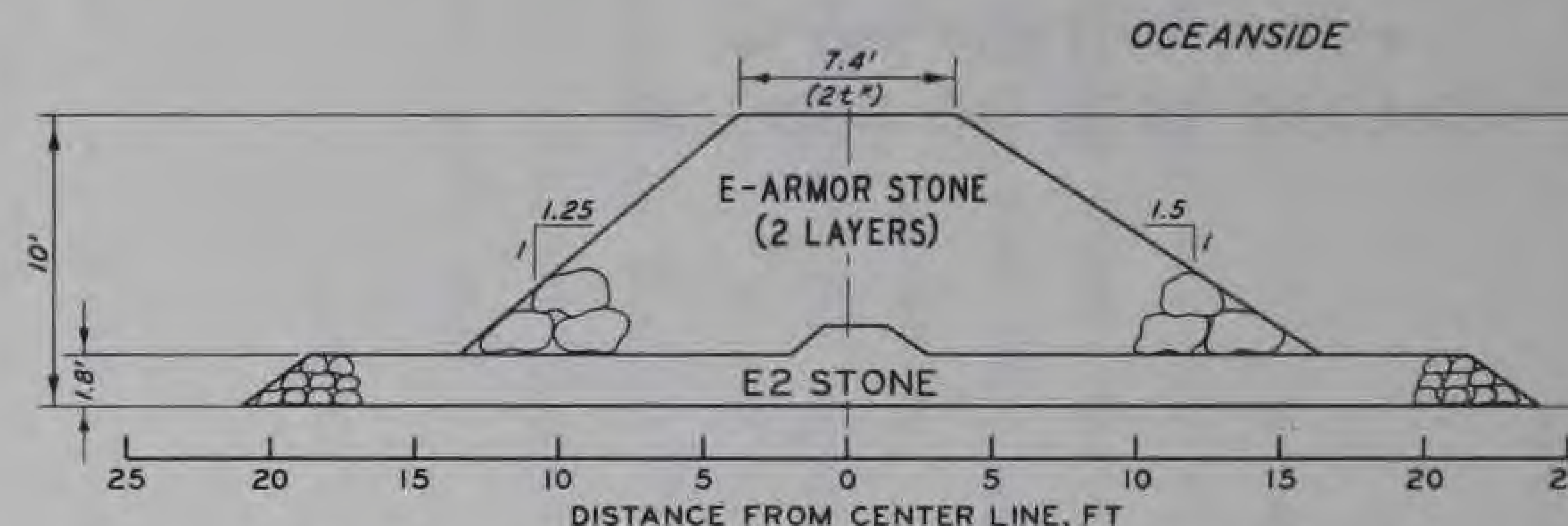
DETAILS OF TEST SECTION

PLANS T1A, T2A, T1B, T2B, T1C, T2C, T3C, T5C, AND T2C-1



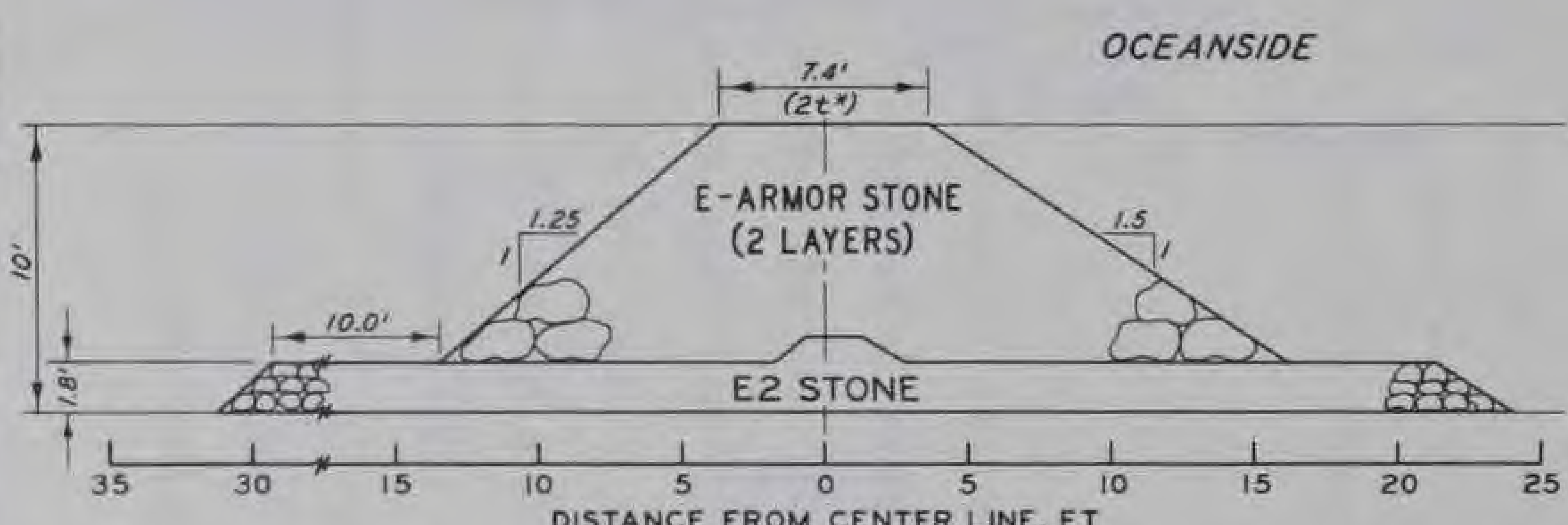
LEGEND		
STONE SIZE	WEIGHT, LB PROTOTYPE	MODEL
D	1800	0.05
D2	<100	

ELEMENTS OF PLANS		
PLAN	CREST EL FT MLLW	BOTTOM EL FT MLLW
T3D	-20	-30
T5D	-35	-45



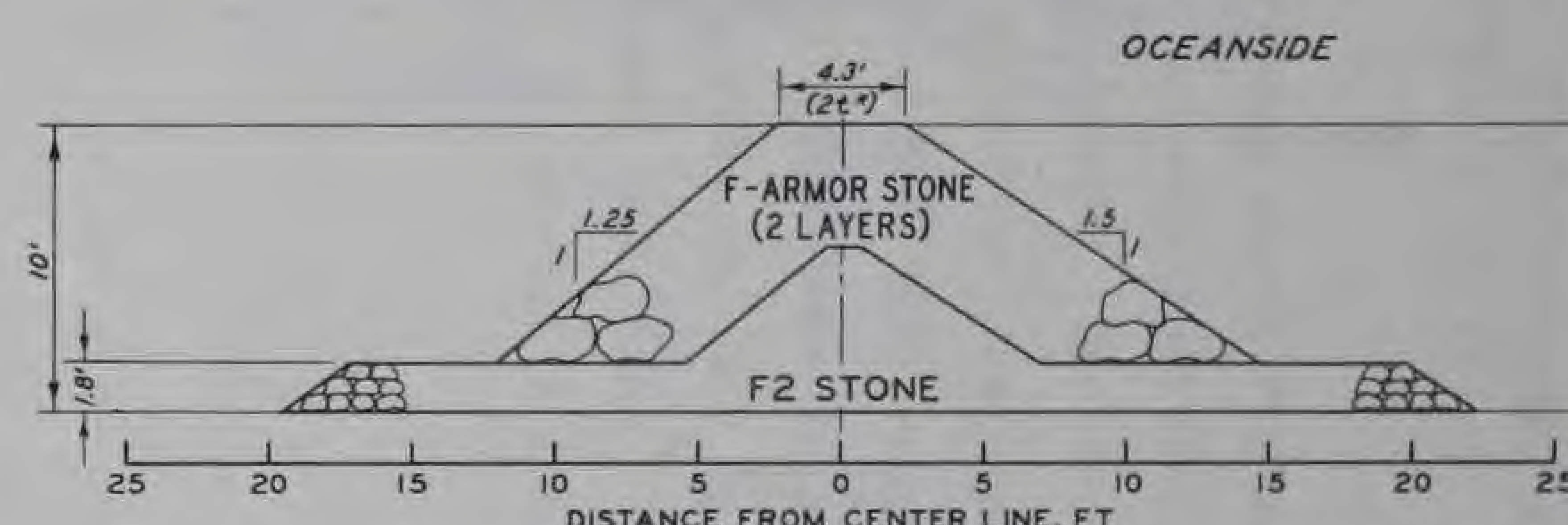
LEGEND		
STONE SIZE	WEIGHT, LB PROTOTYPE	MODEL
E	5000	0.15
E2	<100	0.00076

ELEMENTS OF PLANS		
PLAN	CREST EL FT MLLW	BOTTOM EL FT MLLW
T1E	-10	-20
T2E	-15	-25
T3E	-20	-30



LEGEND		
STONE SIZE	WEIGHT, LB PROTOTYPE	MODEL
E	5000	0.15
E2	<100	0.00076

ELEMENTS OF PLANS		
PLAN	CREST EL FT MLLW	BOTTOM EL FT MLLW
T1E-1	-10	-20



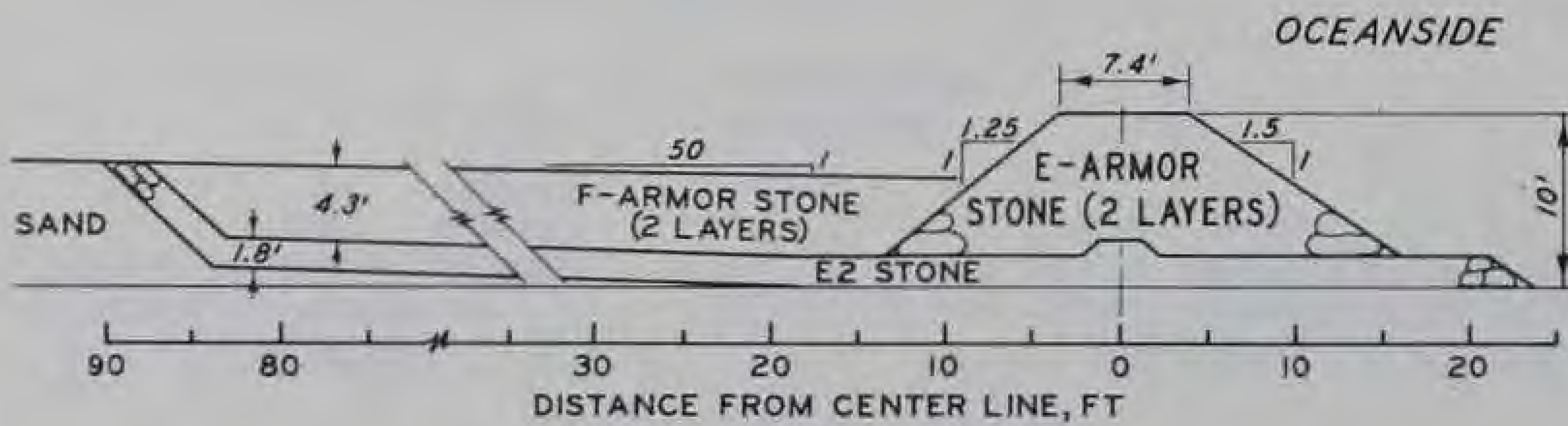
LEGEND		
STONE SIZE	WEIGHT, LB PROTOTYPE	MODEL
F	1000	0.03
F2	<100	

ELEMENTS OF PLANS		
PLAN	CREST EL FT MLLW	BOTTOM EL FT MLLW
T5F	-35	-45

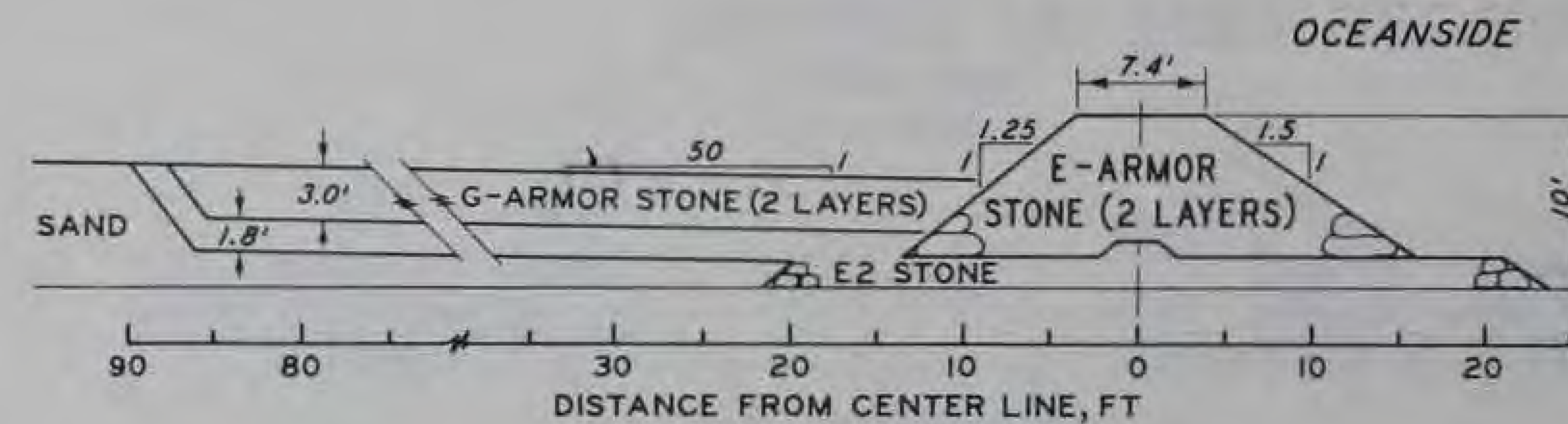
NOTE: *t IS ONE THICKNESS OF STONE SIZE.

DETAILS OF TEST SECTION

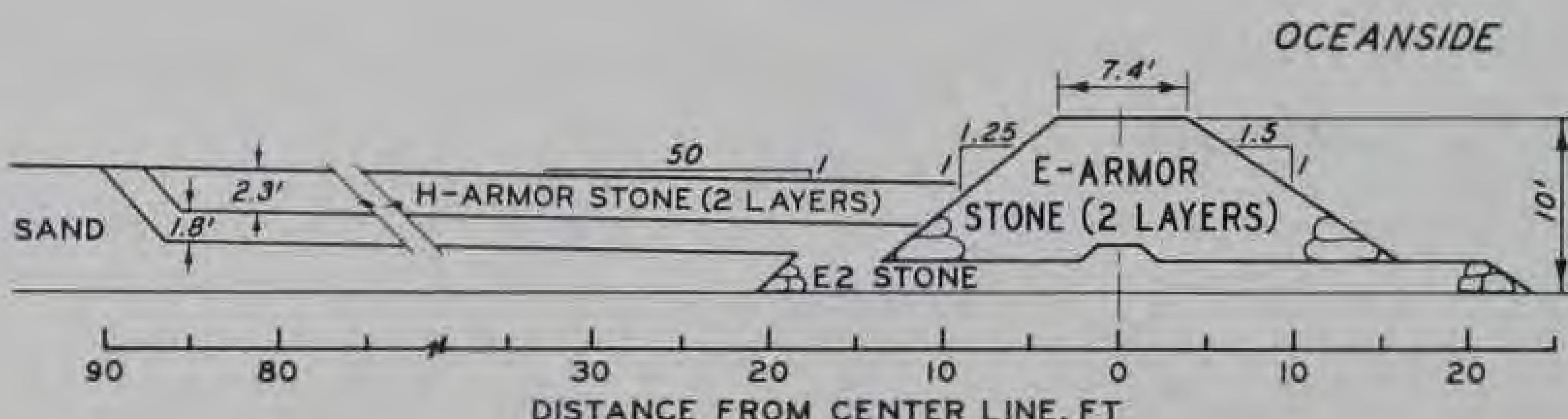
PLANS T3D, T5D, T1E, T2E, T3E, T1E-1, AND T5F



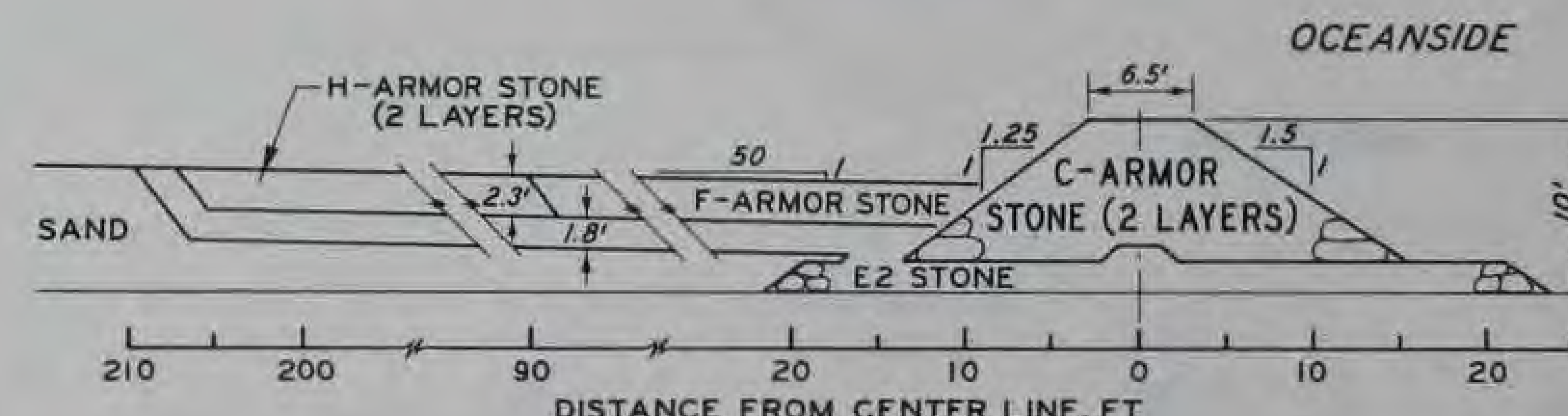
PLAN A1



PLAN A2



PLAN A3



PLAN A4

LEGEND

STONE SIZE	WEIGHT LB
C	3300
E	5000
E2	<100
F	1000
G	385-1220 (AVG 700)
H	152-365 (AVG 250)

ELEMENTS OF PLANS A1, A2, A3, AND A4

CREST EL FT MLLW	BOTTOM EL FT MLLW
-10	-20
-15	-25
-20	-30

DETAILS OF TEST SECTION
PLANS A1, A2, A3, AND A4

APPENDIX A: REVIEW OF RIP CURRENT STUDY
IN THE WAVE BASIN AT WES

1. The present state of knowledge about rip currents indicates that they are due to an interaction between incident (incoming) waves and edge waves that travel along the beach. The interaction formalism requires the edge wave to have the same time frequency as the incoming wave.[4] In nature, the spatial form of the edge waves is determined by a differential equation and boundary conditions in the on-offshore direction; at the beach ($x = 0$) and offshore ($x = \infty$). The problem as treated by Eckart,[6] Ursell,[18] and Bowen and Inman[4] for a plane beach with constant gentle offshore slope β yields a family of possible edge waves such that

$$L = \frac{g}{2\pi} T^2 (2n + 1) \tan \beta \tag{A1}$$

where L is the longshore wavelength of the edge wave, T is the wave period, and $n = 0, 1, 2, 3, \ldots (2n + 1) \beta = \pi/2$ is the modal number, which is equal to the number of zero crossings of the edge wave in the offshore direction. The relation $(2n + 1) \beta = \pi/2$ is referred to as the cutoff mode and gives the greatest possible longshore wavelength

$$L = \frac{g}{2\pi} T^2$$

2. This problem has also been discussed for shallow-water waves on beaches with exponential slopes.[19] In both cases, the boundary condition at infinity is necessary to obtain the usual dispersion relationships. The rip current experiments were performed in a wave basin with a length of 60 ft from the wave maker to swl and a width of 60 ft. The bathymetry was obtained from a profile off Santa Monica Bay and reduced to a scale of 1:100, with no vertical exaggeration. The following discussion refers to the model dimensions. The slope was steepest at the beach, and the depth increased continuously, attaining a maximum still water depth of 0.6 ft (model) at the wave maker. Broken into

segments, the profile had the following slopes:

Segment	Slope
Beach face above swl	0.064
swl (0.0 ft mllw) to -6.0 ft	0.043
-6.0 ft to -12.2 ft	0.028
-12.2 ft to wave maker (-58.8 ft)	0.0076

3. From the above tabulation, it can be seen that the model beach profile had a nearly constant slope of about 0.008 for distances 12.2 ft < x < 60 ft. Also, it was possible to fit approximately the entire slope with a single exponential function, as shown in fig. A1. The first approximation was an attempt to fit the constant slope modes of the edge wave relation to the slope $\tan \beta = 0.008$. Assuming that the edge waves extend most of the distance from the beach to the wave maker, it is not unreasonable to expect their behavior to be governed by the topography which exists over the major portion of their extent. Combining Eckart's work with the three-dimensional analysis of Ursell,[18] it is simple to show that

$$\sigma^2 \leq gk$$

where the wave frequency $\sigma = (2\pi/T)$ and the wave number $k = (2\pi/L)$ or, in feet per second,

$$L \leq 5.12T^2$$

If $T = 0.9$ sec, then

$$L \leq 4.13 \text{ ft}$$

The data in table A1 (from the WES perched beach experiments) indicate that the observed spacing is too large to be explained with this model.

4. Next, the possibility of waves trapped against an exponential slope is considered. First, let the depth of water

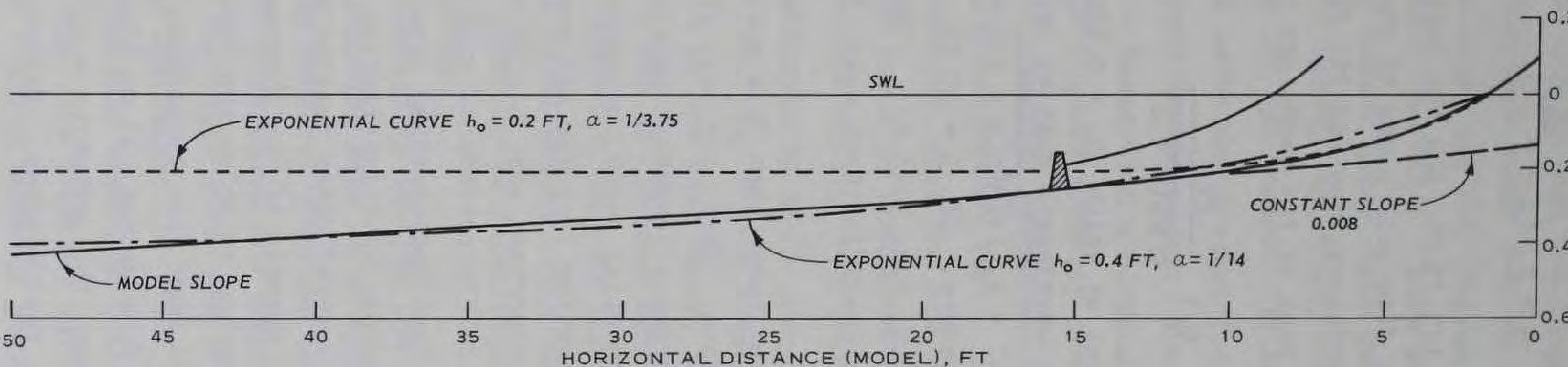


Fig. A1. Beach profile of WES rip current model

$$h = h_o \left(1 - e^{-\alpha x}\right)$$

where the coefficient $\alpha = 1/3.75$ ft and the initial water depth $h_o = 0.2$ ft. This relation fits well close to shore, and, for edge waves which decay rapidly offshore, the bad fit farther out is unimportant. Ball[18] has shown that if the waves are trapped, then

$$L < \frac{2\pi}{\alpha \sqrt{n(n + 1)}}$$

Since $1/\gamma = X_d$ is the point at which $h = 0.63h_o$, then except for the zero mode the longshore wavelength must be less than some characteristic "modal decay depth" = $X_d \, [\pi/\sqrt{n(n + 1)}]$ where n is the modal number, 0, 1, 2, etc., as before. If $L = 10$ ft, then $n = 0, 1, 2$. Unfortunately then, $T = 6.5, 4.2, 3.75$ sec. These periods are too long to interact with the primary incident wave $T \sim 1$ sec to force rip currents.

5. For the exponential curve that fits most of the profile, a similar result can be produced. Here $\alpha = 1/14$ ft, $h_o = 0.4$ ft, and the shortest possible period having a longshore wavelength of 10 ft is $T = 2.8$ sec. For both exponential cases, a longshore wavelength of 10 ft gives periods which are too long for all possible trapped modes. For the larger wavelengths reported, say $L = 20$ ft, there is even worse agreement, and the possible periods of such edge waves are even longer. The conclusion is that the observed rips are probably not due to the usual "incident wave/edge wave interaction."

6. There seem to be several other possibilities. It is possible that this development is an example of the phenomenon occasionally observed on gently sloping beaches in which the "beat wave" of long period interacts with a long-period edge wave to produce large rip spacing.[4] Such a "beat" could result from mechanical or electrical variations in the wave maker. In this case, the prototype would be reproducing what presumably can occur on the real beaches. Unfortunately, little work, either laboratory or field, has been done on such gently sloping systems.

7. Another possibility is that the large spacings are related to cross waves excited by the wave maker[20] or to normal cross-tank modes. In these cases, the edge waves are not trapped but rather are like the normal modes of the basin. Garrett's[20] work was on a constant depth basin and dealt with the lowest order cross wave, which had a period of one-half that of the wave maker. He noted, however, that the next order is of the same period as the wave maker. It is possible that something analogous to these cross waves may account for the wide spacing observed between rip currents in the model. The dispersion relationship for the normal modes in a basin with a sloping bottom are not known, but a theoretical investigation would probably resolve the question of whether edge waves of the observed wavelength and period of the incoming wave can exist at all, trapped or free. If they can and are responsible for the observations, then the model is not reproducing the type of rip currents that occur on real beaches but is being dominated by the end effect of the wave maker. Previous experimental workers may have avoided such contamination by their experimental setup (i.e., Tait[5] and Bowen and Inman[4]). (See fig. 2 from Bowen and Inman.) Since the cross waves are generated at the wave maker, their wavelength is probably determined by the basin width at the wave maker. The actual working area would be much less affected, because the barriers generally would interfere with the basin oscillations. The edge waves found inside the working area would then probably be locally generated trapped modes like the ones occurring naturally.

8. Yet another possibility is that the observed rip current spacing is produced by standing edge waves within the basin that are relatively long in period compared with the incident waves from the wave maker. In this case, the edge waves are trapped modes with spacings that are dependent upon the basin width (60 ft). Since any antinodal point remains relatively stationary for times that are long, compared with the period of the incident waves, an initial circulation pattern is set up in which the rips flow from the antinodal areas of positive water level displacement towards those of negative displacement. Of course, one-half (edge wave) period later, the situation is revised, and

the locations of the rip currents would also reverse, except that the longshore and rip currents just established have momentum and resist change. If they are sufficiently well established to endure (although in decreased intensity) until then, a viable system would prevail. Little is known about the probability of this type of interaction. Its existence depends upon the rate of establishment and damping of the rip currents. In this case, it is to be expected that the rip current velocity should pulsate with the period of the edge waves.

9. Whether the model properly portrays what happens on the real beach is uncertain. Perhaps rip spacing measurements could be made on the real beach. If they are "oversized" then the model may be valid. If they are normal size then perhaps a semienclosed working area is necessary to test the effect of the perched beach on rips.

Table A1

Perched Beach Rip Current Model, Rip Current Observations for Various Model Widths with Existing Beach Slopes Installed

Test Wave Period sec	Test Wave Height ft	Model Width ft	Current Velocity, fps, at Various Distances Offshore, ft 2-4	4-6	6-8	8-10	>10	Average	No. of Rips	Approximate Spacing Between Rips, ft
0.9	0.07	49.8	0.32	0.32	0.22	0.20	0.20	0.25	6	11, 8, 9, 12, 6
		48.8	0.22	0.23	0.22	0.18	--	0.17	5	11, 10, 14, 12
		47.8	0.28	0.20	0.22	0.23	0.17	0.22	4	15, 15, 16
		47.3	0.48	0.48	0.32	0.32	0.32	0.38	5	9, 12, 15, 9
		46.8	0.27	0.27	0.22	0.22	--	0.20	6	9, 10, 7, 7, 10
		46.3	0.40	0.40	0.35	0.32	0.32	0.36	3	21, 23
		45.8	0.33	0.33	0.17	0.17	0.10	0.22	3	16, 19
0.9	0.13	49.8	0.43	0.43	0.32	0.32	0.32	0.36	3	15, 16
		48.8	0.28	0.28	0.40	0.50	0.50	0.39	4	10, 14, 8
		47.8	0.30	0.45	0.33	0.35	0.35	0.36	4	14, 14, 13
		47.3	0.40	0.40	0.45	0.45	0.50	0.44	3	16, 17
		46.8	0.33	0.32	0.32	0.37	0.37	0.34	3	11, 14
		46.3	0.43	0.43	0.33	0.27	0.25	0.34	4	16, 10, 9
		45.8	0.50	0.50	0.48	0.48	0.42	0.48	4	11, 11, 14
1.1	0.08	49.6	0.27	0.40	0.40	0.25	0.25	0.31	5	11, 11, 14, 13
		48.6	0.33	0.33	0.20	0.17	0.10	0.23	4	11, 11, 12
		47.6	0.38	0.28	0.28	0.12	--	0.21	4	19, 9, 9
		46.6	0.37	0.37	0.33	0.25	0.23	0.31	3	18, 14
		46.1	0.37	0.37	0.28	0.10	--	0.22	4	13, 16, 16
		45.6	0.27	0.35	0.35	0.23	0.22	0.28	4	14, 16, 15
		45.1	0.35	0.35	0.30	0.20	0.18	0.28	4	16, 14, 14
		44.6	0.35	0.35	0.25	0.17	--	0.22	4	15, 14, 15
		43.6	0.20	0.23	0.15	0.13	--	0.14	3	16, 15

Note: Units are expressed in terms of model equivalents.

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13. ABSTRACT

Hydraulic model studies were conducted at the U. S. Army Engineer Waterways Experiment Station to aid in determining the technical feasibility and optimum design factors of the perched beach concept for widening the existing beach to provide right-of-way for a freeway along a portion of the Santa Monica Bay coastline. The original study proposal consisted of six main parts. However, during the course of the model studies, the California Legislature deleted this section of the freeway from the California Freeway and Expressway System. As a result, the Division of Highways terminated their freeway location project and canceled further model testing. Consequently, only the following three parts of the study were completed: (a) an undistorted, three-dimensional, fixed-bed model (scale 1:100) was used to determine the effect of the perched beach on rip currents. If adverse interactions were present, the model was used to determine means of minimizing them; (b) a distorted-scale (1:100 horizontal, 1:50 vertical), two-dimensional, movable-bed model was used to estimate the amount of sand which might be lost seaward over the toe structure due to normal and storm wave actions and to determine the optimum crown elevation of the submerged structure and the length of stone riprap apron required to reduce the seaward migration of sand to a minimum; and (c) an undistorted, two-dimensional model (scale 1:30) was used to determine the structural design of the proposed rubble-mound toe structure for various depths. The following three parts of the study were not completed: (a) an office data (environmental data) analysis, including a hydrographic survey with sand sampling, a reevaluation of geographical information, and a computation of littoral transport; (b) the construction and testing of a surf beat model to evaluate the wave runup characteristics with and without a perched beach; and (c) an office study to determine the feasibility of constructing a three-dimensional, movable-bed model. This report describes the testing and results up to the premature termination of the model studies. It was concluded from test results that: (a) installation of the perched beach in the model had no adverse effect on rip currents and reduced rip current velocities about 20 percent for the configuration tested; (b) normal wave action (waves expected to occur a high percentage of the time) on the perched beach caused no appreciable loss of beach fill; (c) for the larger storm waves of any significant duration, a large net seaward loss of fill material can be expected; (d) the installation of a 100-ft stone apron in conjunction with a 350-ft-wide (measured from toe structure to 0.0-ft-mllw contour) perched beach (plan 2A) will have little or no effect on the loss of fill material; (e) the installation of a 100-ft stone apron in conjunction with a 700-ft-wide (measured from toe structure to 0.0-ft-mllw contour) perched beach (plan 1A) will significantly reduce the amount of beach fill lost seaward of the toe structure; (f) if the beach fill is extended as far as 1100 ft seaward of the 0.0-ft-mllw contour (plans 3 and 3A), the toe structure will have little or no beneficial effect on reducing the amount of beach fill lost; (g) of all plans tested, plan 1A appears to offer the greatest degree of protection against seaward loss of beach fill material; (h) 5000-lb armor stone will be needed if the rubble-mound toe structure is located in water depths ranging from 20 to 35 ft; 3000-lb armor stone will be needed in depths from 35 to 45 ft; and 1000-lb armor stone will be needed in depths greater than 45 ft; (i) quarry-run stone whose 50 percent weight is about 100 lb will be adequate bed material for the toe structure; (j) 250-lb stone will be stable for the riprap apron shoreward of the toe structure; (k) additional wave flume tests will be needed with armor units in the specific gravity range of 1.3 to 1.6 prior to performing a three-dimensional, movable-bed model investigation; and (l) provided adequate prototype data become available for use in model verification, a three-dimensional, movable-bed model investigation appears to be feasible and should result in a valid indication of the relative merits of various project designs.

14. KEY WORDS	LINK A		LINK B		LINK C	
	ROLE	WT	ROLE	WT	ROLE	WT
Beaches						
Hydraulic models						
Perched beaches						
Santa Monica Bay						