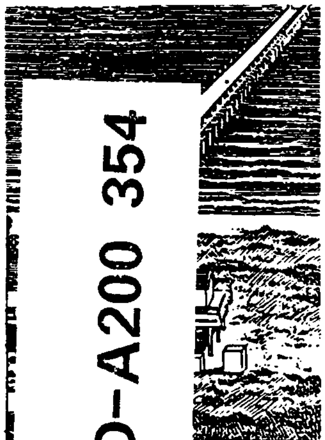
FILE COPY

TECHNICAL REPORT CERC-88-14

US Army Corps of Engineers



JETTY REHABILITATION STABILITY STUDY YAQUINA BAY, OREGON

by

Peter J. Grace, Willie G. Dubose

Coastal Engineering Research Center

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39181-0631

September 1988

Final Report

Approved For Public Release; Distribution Unlimited

Prepared for US Army Engineer District, Portland
Portland, Oregon 97208-2946

Unclassified
SECURITY CLASSIFICATION OF THIS PAGE

REPORT DOCUMENTATION PAGE

Form Approved OMB No 0704-0188 Exp Date Jun 30 1986

1a REPORT SECURITY CLASSIFICATION: Unclassified	1b RESTRICTIVE MARKINGS
2a SECURITY CLASSIFICATION AUTHORITY	3 DISTRIBUTION/AVAILABILITY OF REPORT: Approved for public release; distribution unlimited.
2b DECLASSIFICATION/DOWNGRADING SCHEDULE	
4 PERFORMING ORGANIZATION REPORT NUMBER(S): Technical Report CERC-88-14	5 MONITORING ORGANIZATION REPORT NUMBER(S)
6a NAME OF PERFORMING ORGANIZATION: USAEWES, Coastal Engineering Research Center; 6b OFFICE SYMBOL (If applicable): CEWES-CW-R	7a NAME OF MONITORING ORGANIZATION
6c ADDRESS (City, State, and ZIP Code): PO Box 631, Vicksburg, MS 39181-0631	7b ADDRESS (City, State, and ZIP Code)
8a NAME OF FUNDING/SPONSORING ORGANIZATION: USAED, Portland; 8b OFFICE SYMBOL (If applicable): CENPP-EN	9 PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER
8c ADDRESS (City, State, and ZIP Code): PO Box 2946, Portland, OR 97208-2946	10 SOURCE OF FUNDING NUMBERS: PROGRAM ELEMENT NO / PROJECT NO / TASK NO / WORK UNIT ACCESSION NO

11 TITLE (Include Security Classification)
Jetty Rehabilitation Stability Study, Yaquina Bay, Oregon

12 PERSONAL AUTHOR(S)
Grace, Peter J.; Dubose, Willie G.

13a TYPE OF REPORT	13b TIME COVERED	14 DATE OF REPORT (Year, Month, Day)	15 PAGE COUNT
Final report	FROM Oct 87 TO Jan 88	September 1988	130

16 SUPPLEMENTARY NOTATION
Available from National Technical Information Service, 5295 Port Royal Road, Springfield, VA 22161.

17 COSATI CODES			18 SUBJECT TERMS (Continue on reverse if necessary and identify by block number)
FIELD	GROUP	SUB-GROUP	Hydraulic models (LC); Yaquina Bay, Oregon; Hydraulic structures (LC); Jetties (LC)

19 ABSTRACT (Continue on reverse if necessary and identify by block number)

A three-dimensional, physical, hydraulic model investigation was performed to establish a stable jetty design for use in the proposed rehabilitation of the north jetty at Yaquina Bay, Oregon. The study was conducted at a geometrically undistorted linear scale of 1:45, model to prototype. Unidirectional, spectral test waves were generated from three directions. Wave conditions were selected based on six hindcasted storms which covered the range of historical and probable worst wave conditions. Tests were conducted at still-water levels of 0.0 ft and +10.0 ft mean lower low water. The seaward end of the north jetty has undergone rapid deterioration since the previous 1977-1978 rehabilitation. The loss of approximately 400 ft of the jetty has resulted in increased deposition of littoral material in the entrance channel and more severe wave conditions at the mouth. Three mean armor-stone weights (16, 23, and 29 tons) were used in the rehabilitation plan tested. This plan

(Continued)

20 DISTRIBUTION/AVAILABILITY OF ABSTRACT: ☒ UNCLASSIFIED/UNLIMITED ☐ SAME AS RPT ☐ DTIC USERS	21 ABSTRACT SECURITY CLASSIFICATION: Unclassified
22a NAME OF RESPONSIBLE INDIVIDUAL	22b TELEPHONE (Include Area Code) / 22c OFFICE SYMBOL

DD FORM 1473, 84 MAR — 83 APR edition may be used until exhausted. All other editions are obsolete.

SECURITY CLASSIFICATION OF THIS PAGE
Unclassified

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE

19. ABSTRACT (Continued).

would restore the jetty to its original length and would utilize the "placed-stone" construction technique. The model tests indicated that the recommended armor-stone weights were adequate and that the plan to employ "placed-stone" construction of the jetty on existing deteriorated jetty stone was acceptable. Additional tests were performed with monochromatic wave conditions in an effort to compare wave heights measured in the physical model with wave heights predicted in an earlier study by the numerical model, RCPWAVE. A discussion of those test results is included.

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE

PREFACE

The model investigation reported herein was requested by the US Army Engineer District, Portland (NPP), and conducted at the Coastal Engineering Research Center (CERC) of the US Army Engineer Waterways Experiment Station (WES). Funding authorization was granted by NPP through Intra-Army Order E86870122, dated 8 April 1987, and in Change Order No. R-1, dated 30 November 1987.

Physical model tests and report preparation were performed at WES during the period October 1987 through January 1988 under the general direction of Dr. J. R. Houston, Chief, CERC; Mr. C. C. Calhoun, Jr., Assistant Chief, CERC; Mr. C. E. Chatham, Jr., Chief, Wave Dynamics Division (WDD); and Mr. D. D. Davidson, Research Hydraulic Engineer, WDD.

Many individuals within WES made significant contributions throughout this modeling effort. Successful duplication of detailed bathymetric features in the test basin was accomplished by the Model Construction Section of the Engineering and Construction Services Division under supervision of Mr. M. J. Wooley. Mr. W. D. Corson, Research Division (RD), was responsible for selection and compilation of pertinent wave hindcast data. Dr. R. E. Jensen, RD, performed the transformation of deepwater hindcasted wave conditions into the shallower depths represented in the model. Mr. M. J. Briggs, WDD, contributed in many ways, including generation and calibration of the directional spectral wave board control signals. Specific information, results, and assistance relative to the previous numerical modeling investigation were provided by Messrs. B. A. Ebersole and D. P. Simpson and Ms. M. A. Cialone, all of RD. Testing in the physical model was conducted by Messrs. P. J. Grace, WDD, W. G. Dubose, WDD, and D. A. Dailey, Instrumentation Services Division. Mr. R. D. Carver, WDD, provided technical assistance throughout the test program, and Mr. J. M. Heggins, WDD, contributed significantly during data analysis. This report was prepared by Messrs. Grace and Dubose and edited by Ms. N. Johnson, Information Technology Laboratory, under the Inter-Governmental Personnel Act.

During the course of this investigation, liaison was maintained with CERC by Mr. J. Oliver, US Army Engineer Division, North Pacific, and Messrs. David Illias and Harold Herndon, NPP.

COL Dwayne G. Lee, EN, is the Commander and Director of WES. Dr. Robert W. Whalin is the Technical Director.

CONTENTS

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

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

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
feet	0.3048	metres
horsepower (550 foot-pounds (force) per second)	745.6999	watts
inches	2.54	centimetres
miles (US statute)	1.6093	kilometres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square miles (US statute)	2.589998	square kilometres
tons (2,000 pounds, mass)	907.1847	kilograms

JETTY REHABILITATION STABILITY STUDY, YAQUINA BAY, OREGON

PART I: INTRODUCTION

Prototype

1. Yaquina Bay is an estuary located on the Oregon Coast approximately 110 miles* south of the mouth of the Columbia River (Figure 1). The bay is fed by Yaquina River which drains a predominantly forested watershed of approximately 250 square miles. Elements of the existing project at Yaquina Bay controlled by the US Army Corps of Engineers include two rubble-mound jetties

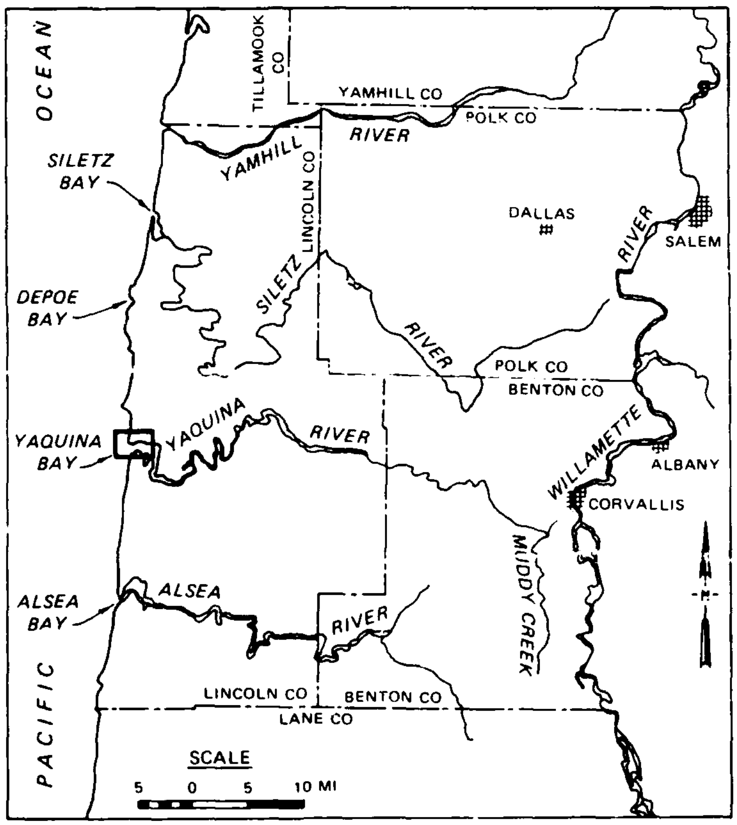


Figure 1. Project location map

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

at the entrance and a 40-ft-deep by 400-ft-wide entrance channel. The jetties, entrance channel, and other project features were constructed to provide safer access for vessels serving the Yaquina River ports of Newport and Toledo, Oregon. Commercial products handled at these ports include lumber, pulp, paperboard, petroleum, and seafood. The Yaquina Bay area is also frequently used by individuals who enjoy recreational fishing and boating.

Problem

2. Vessels navigating the entrance to Yaquina Bay have always been influenced by the presence of a narrow basaltic offshore reef. The reef lies approximately 3,500 ft seaward of river mile 0.0 and extends, from a point about 2,500 ft south of the channel, northward for approximately 17 miles. The entrance channel passes through a narrow opening in the reef directly offshore of the bay. The parallel jetties were constructed on an approximate azimuth of S62 W to permit navigation through this opening; therefore, the jetties offered excellent protection against waves from the west and northwest.

3. The north jetty at Yaquina Bay was originally constructed in 1895 to a length of 2,300 ft. In 1930, efforts to restore the jetty and extend the length of 3,700 ft were completed. This effort was followed by additional reconstruction projects performed in 1933 and 1934. Six years later, a 1,000-ft extension was completed and, in 1958, the present design length of 7,000 ft was authorized. This construction was completed in 1966, at which time the jetty extended the entire distance from shore to the edge of the basaltic reef. By 1970, winter storms had damaged the jetty to such an extent that the outer 330 ft was submerged. A rehabilitation project was authorized in 1976, and this work was completed in 1978. One year after rehabilitation, 60 ft of material had been lost from the jetty end, and after two years, the outer 250-ft section was gone. Aerial photographs taken in 1985 indicated that more than 400 ft of the north jetty's seaward end had been damaged. As the above summary indicates, the north jetty has been plagued with a history of unusually rapid deterioration when compared with similar North Pacific jetties which were built with the same design criteria and construction techniques. Probable causes of this deterioration are foundation scour caused by wave-induced currents during storm events and use of undersized armor stone on the jetty's seaward end. The 1978 rehabilitation specified armor stones

having an average weight of 17.6 tons. This stone size was determined based on a 21.6-ft design wave. In 1985, design wave conditions were reevaluated by Ebersole* with greater consideration of shoaling and refraction effects. This wave propagation analysis resulted in a 28-ft design wave at the jetty head, thereby increasing the mean armor-stone weight at the head to 29 tons (Cialone 1986).

4. Deterioration of the jetty has progressed to such an extent that the crest beyond sta 87+50 lies below mean lower low water (mllw)**; therefore, a portion of the entrance channel behind this section has been left unsheltered. The damaged section also trips passing waves, which at times creates difficult wave and current conditions for small vessels entering or leaving the harbor. This is especially true just before summer dredging is performed. During the winter and spring, shoals form on the south side of the channel forcing boats to enter further northward. Eventually, pilots are forced to follow a narrow entrance path between waves tripped by the damaged jetty and those breaking on the south shoals (US Army Engineer District (USAED), Portland, 1987).

Purpose of Model Study

5. The primary purpose of the physical model study was to determine the adequacy of the rehabilitated jetty design proposed for construction. Due to the repeated past deterioration of the seaward end of the north jetty, designers proposed usage of the placed-stone construction techniques and greater armor-stone weights in an effort to increase the stability of the jetty. This model investigation was performed to determine if the proposed plan was acceptable, and if necessary, to develop alternate designs from which an optimum plan could be chosen based on jetty stability and economic factors. Specific details of the design and construction techniques are presented in Part III.

6. This study also provided an opportunity to compare results of a numerical model (Cialone 1986) which predicts wave heights with corresponding results measured in the physical model. This phase of the investigation is discussed in Part IV.

---

* B. A. Ebersole. 1985 (Feb). "Wave Propagation Analysis for the Yaquina Bay, Oregon, North Jetty Rehabilitation," Memorandum for Record, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

** All elevations (el) cited herein are in feet referred to mllw.

PART II: MODEL

Design of Model

7\. This hydraulic model study was conducted at an undistorted linear scale of 1:45, model to prototype. Selection of the 1:45 scale was based on several factors including: (a) volume of the available wave basin, (b) boundaries of the bathymetric area to be modeled, (c) capabilities of the directional spectral wave generator (DSWG), (d) availability of required model armor-stone sizes, and (e) preclusion of stability scale effects (Hudson 1975). Based on Froude's Model Law (Stevens 1942) and the linear scale of 1:45, the following model-to-prototype relations were derived. Dimensions are in terms of length (L) and time (T).

Characteristic	Dimension	Model-to-Prototype Scale Relation (r)
Length	L	$L_r = 1:45$
Area	L^2	$A_r = L_r^2 = 1:2,025$
Volume	L^3	$V_r = L_r^3 = 1:91,125$
Time	T	$T_r = L_r^{0.5} = 1:6.7$

8\. The specific weights of water used in the model and of seawater were assumed to be 62.4 and 64.0 pcf, respectively. Likewise, specific weights of the construction materials used in the model (165 pcf) were not identical to their prototype counterparts (170 pcf). These variables were related using the following transference equation:

$$\frac{(W_a)_m}{(W_a)_p} = \frac{(\gamma_a)_m}{(\gamma_a)_p}\left(\frac{L_m}{L_p}\right)^3\left[\frac{(S_a)_p - 1}{(S_a)_m - 1}\right]^3 \quad (1)$$

where

W_a = weight of an individual armor unit, lb

m, p = model and prototype quantities, respectively

γ_a = specific weight of an individual armor unit, pcf

L_m/L_p = linear scale of the model

S_a = specific gravity of an individual armor unit relative to the water in which it was placed, i.e., $S_a = \gamma_a/\gamma_w$

γ_w = specific weight of water, pcf

9. Due to the limited area of the test basin, it was impossible to model the entire length of both jetties at the selected scale (1:45). It was essential that the offshore bathymetric features be duplicated to the extent that wave transformation into shallower water was properly modeled. This placed the wave board in a water depth corresponding roughly to the -58 ft mllw contour. This, in turn, allowed model construction of approximately 32 ft (1,440-ft prototype) of the north jetty and 21 ft (950-ft prototype) of the south jetty. For the purpose of this investigation, these lengths were sufficient since stability testing was required only on the outer 450 ft of the north jetty. The head of the north jetty was also positioned in the basin in such a way that it could be subjected to wave attack from any direction within a 50-deg window without substantial loss of wave energy off the ends of the unidirectional waves (Figure 2).

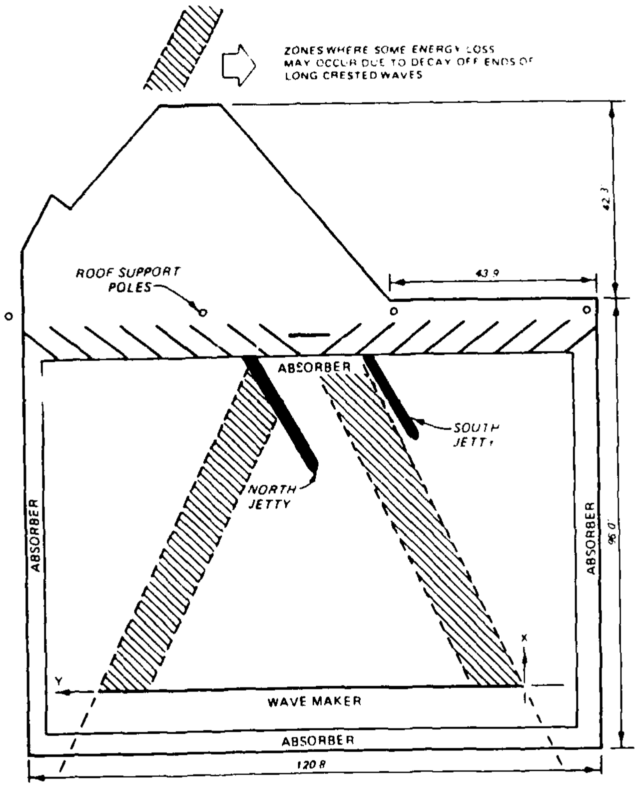


Figure 2. Plan view of directional spectral wave basin

Test Facilities and Equipment

10. This study was conducted in a 96-ft-long by 121-ft-wide wave basin (Figure 2). The concrete floor of the basin was carefully molded to ensure that the complex prototype bathymetry was reproduced throughout the model. All basin walls were lined with a unique wave absorption system to minimize contamination of the desired wave field by reflected wave energy.

11. Test waves were produced by the DSWG, an electronically controlled, electromechanical system consisting of four modules (Figure 3). Each module

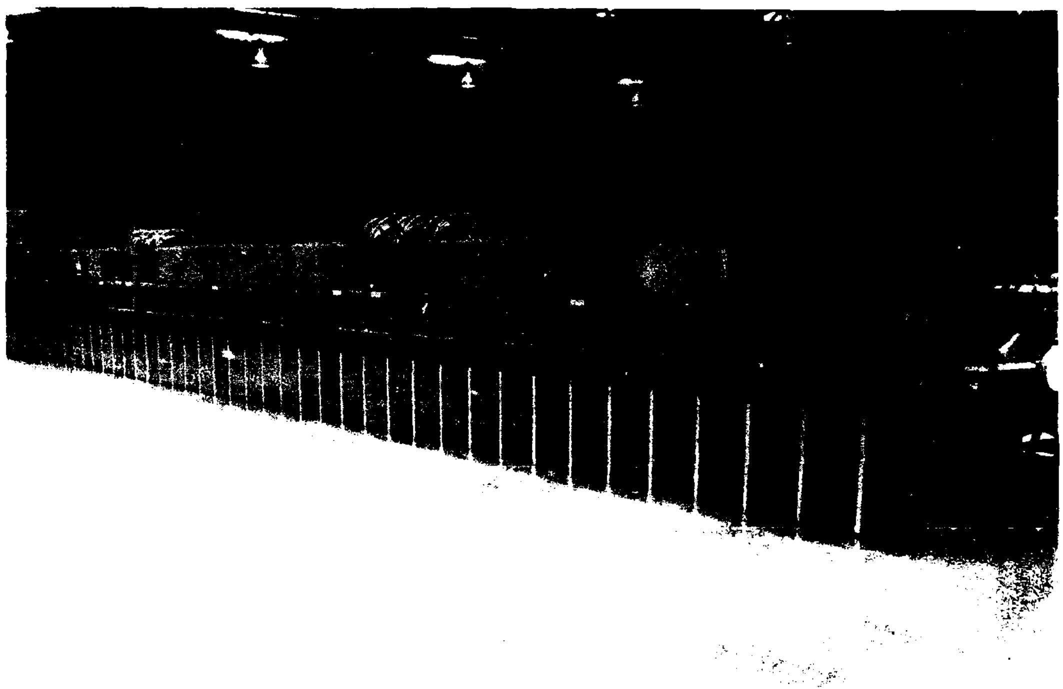


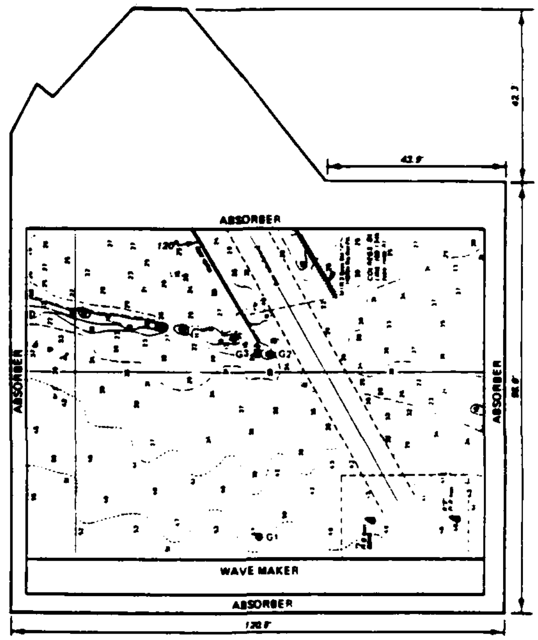
Figure 3. Directional spectral wave generator

contains fifteen 1.5-ft-wide by 2.5-ft-high paddles; therefore, the entire 90-ft-long system consists of 60 paddles, each of which is independently driven by a 0.75-hp electric motor. Adjacent paddles are connected with a flexible-plate seal to provide continuity over the face of the wave board and minimize the introduction of spurious waves (Outlaw and Briggs 1986).

12. Wave heights were measured in the model using capacitance type wave gages at a sampling rate of 10 Hz. During stability tests, three wave gages were used to measure water-surface elevations at the wave board and just seaward of the north jetty head (Figure 4). The numerical/physical model

THREE GAGE ARRAY

GAGE NO.	GAGE X	COORDINATES Y
1	5.0	45.0
2	46.0	42.0
3	46.0	45.0



TEN GAGE ARRAY

GAGE NO.	GAGE X	COORDINATES Y
1	5.0	43.5
2	15.0	43.5
3	42.4	43.5
4	46.4	43.5
5	48.4	43.5
6	50.4	43.5
7	53.0	41.2
8	55.2	42.3
9	57.7	44.0
10	60.5	45.3

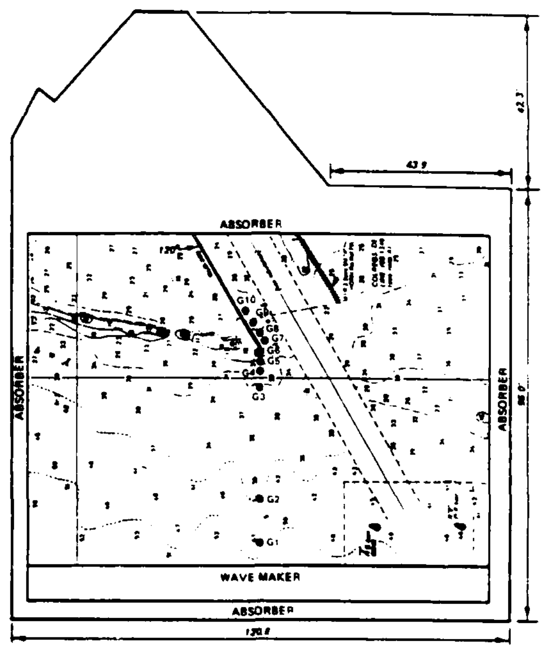


Figure 4. Wave-gage locations

wave-height comparison tests were performed with 10 gages in an effort to more thoroughly monitor the changes in wave heights as the waves progressed into shallower depths (Figure 4).

13. The wave board control signals were generated and transmitted to the DSWG by a Digital Equipment Corporation VAX 11/750 computer. This same computer was used to collect, store, and analyze wave-height data. Some analysis of the monochromatic wave information was performed on an International Business Machines Corporation Personal Computer XT.

Method of Constructing Test Sections

14. Jetty sections in the model were constructed to reproduce, as closely as possible, the results of prototype jetty construction. No information was available on the present condition of the bedding layer; therefore, bedding and core materials in the undamaged jetty sections were dumped by bucket or shovel and leveled to grade lines which corresponded to as-built conditions. These materials were compacted and smoothed to grade with hand trowels in an effort to simulate the natural consolidation that would occur during prototype construction. The primary armor consisted of two layers of parallelepiped-shaped stones, long slab-like stones with a length between two and three times their shortest dimension. Three sizes of the special-shaped stones were handmade to Portland District (NPP) specifications and placed with their long axes perpendicular to the jetty slope. The armor-stone toe was placed in similar fashion with each stone's long axis normal to the longitudinal axis of the jetty; however, stone placement below mllw was more random than placement on the upper slopes. Before actual stability testing was begun, representatives of North Pacific Division (NPD) and NPP were given the opportunity to inspect the model structures. Modifications to the north jetty were made based on their field experience and knowledge of stone-handling capabilities in the prototype.

Selection of Test Conditions

15. Due to the frequent stability problems experienced at the north jetty in recent history, it was essential that this investigation utilize laboratory waves representative of severe conditions which might occur during

the service life of the structure. Both monochromatic and spectral waves were used during various phases of the study. The purpose of this section is to outline the methods employed in establishing those test conditions.

Spectral wave conditions

16. The best available wave information at this site was obtained from the 20-year wave hindcast results of the Wave Information Study (WIS) under way at the Coastal Engineering Research Center (CERC) (Corson et al. 1987). The WIS program was begun in 1976 to produce a data base of wave information for all US coastal waters. For determination of spectral conditions, wave height, period, and direction, statistics were computed for WIS Pacific Phase II Sta 42, located at 44.82° N latitude and 125.01° W longitude. From this 20-year hindcast, the deepwater wave spectra characteristics of the five worst storms were chosen for further analysis. This selection was made based solely on significant wave heights. In addition to these five storms, the spectrum corresponding to the January 1983 storm was also obtained based on estimated wind speeds and directions. It was included as a possible test condition due to recollection of its severity by NPP personnel. The characteristics of those six spectra are listed in Tables 1-6. This information was then used as input into the computer program SHALWV (Hughes and Jensen 1986; Jensen, Vincent, and Abel 1986), which transforms the deepwater wave conditions through shoaling and refraction into shallower depths based on local winds, bathymetric features, and fetch length. For the purpose of this investigation, specific wave conditions were needed at a depth of approximately 58 ft, which corresponded to the location of the wave board in the physical model using a 0.0 ft mllw still-water level (swl). Energy density versus frequency plots (Figures 5-10) were generated from the SHALWV results, and these plots served as the basis for calibration of the wave board control signals for each storm. No directional spreading functions were incorporated into the signals since all wave conditions were to be unidirectional. The hindcasted data, even after transformation into shallow water, indicated that the most severe wave attack was, in all cases, approaching from the southwest quadrant (Figure 11); however, for conservatism, a joint decision was made by CERC and NPP to test with storm signals generated from the west and northwest, as well as the actual direction indicated by the wave propagation analysis.

17. The Texel Marsen Arsloe (TMA) shallow-water spectral form (Hughes 1984; Vincent and Briggs 1987) was selected as the target frequency spectrum.

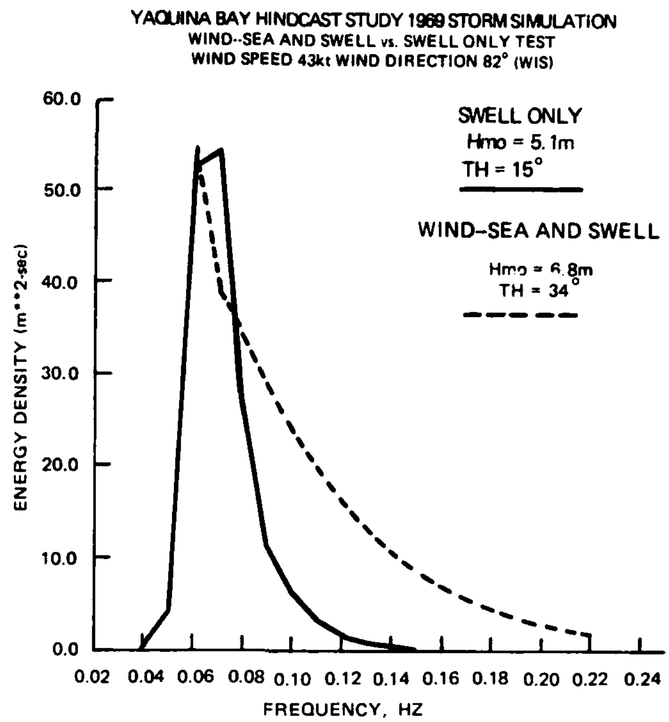


Figure 5. 1969 storm spectrum at 58-ft depth

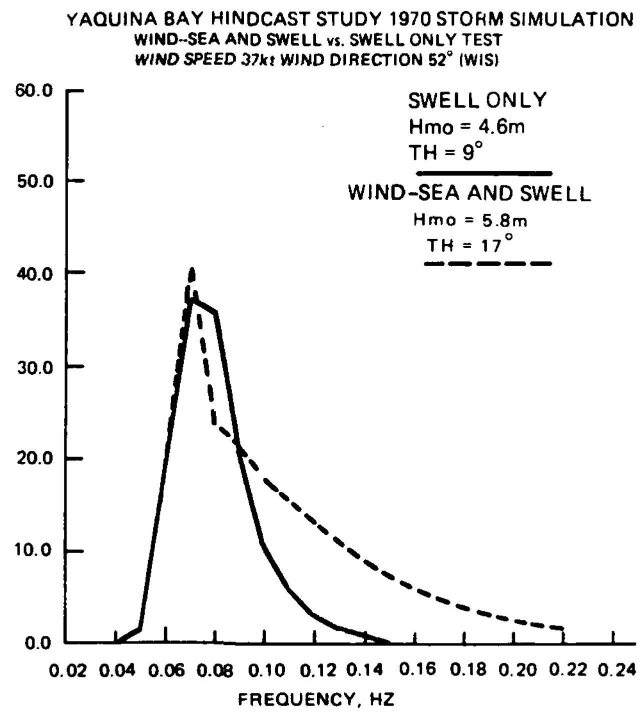


Figure 6. 1970 storm spectrum at 58-ft depth

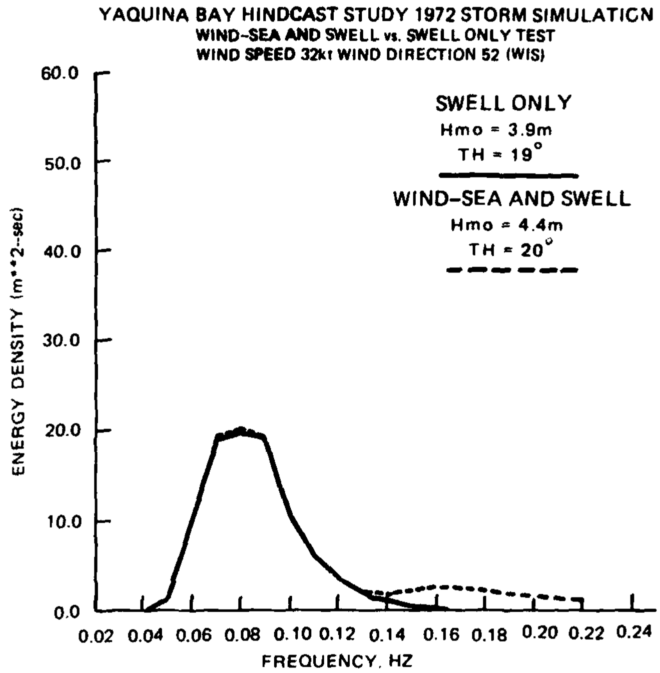


Figure 7. 1972 storm spectrum at 58-ft depth

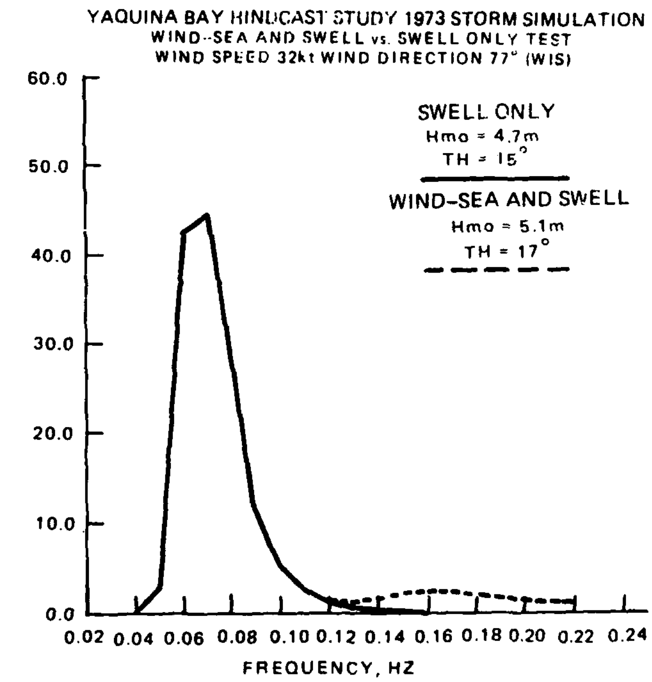


Figure 8. 1973 storm spectrum at 58-ft depth

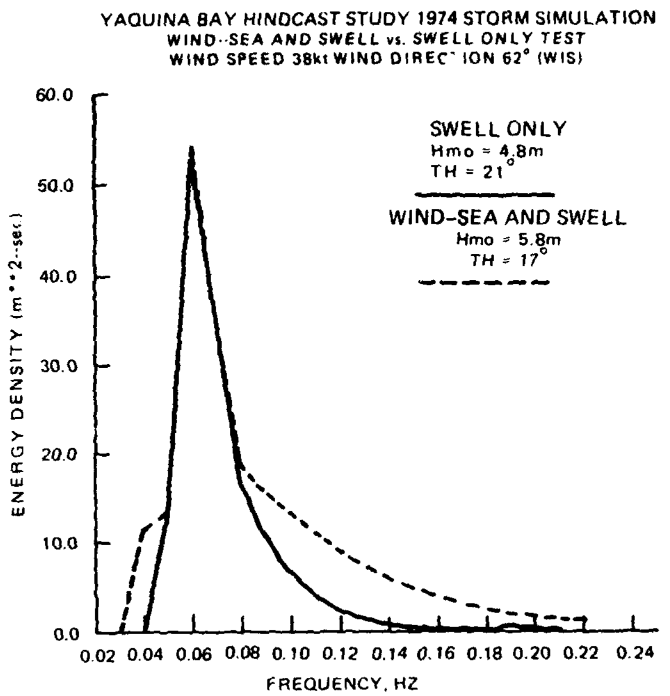


Figure 9. 1974 storm spectrum at 58-ft depth

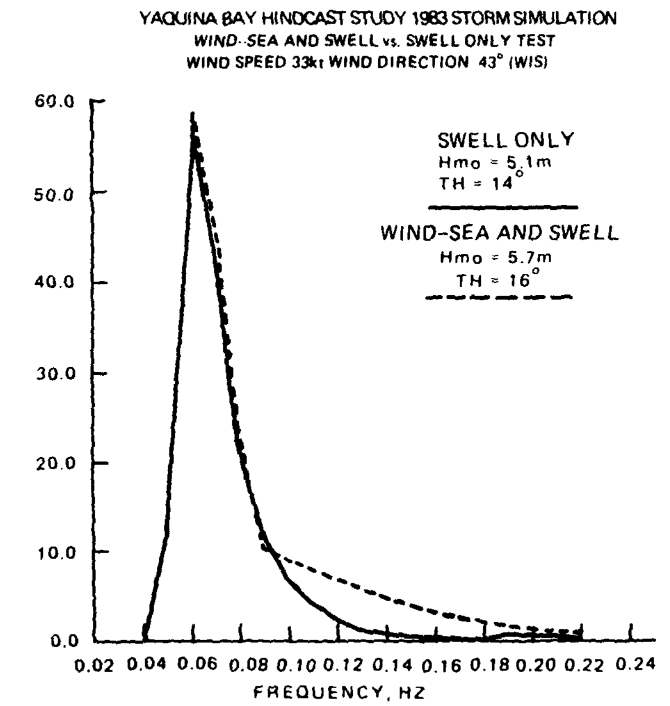


Figure 10. 1983 storm spectrum at 58-ft depth

STORM	PEAK DIRECTION
69	228°
70	264°
72	244.5°
73	241.5°
74	204°
83	247.5°

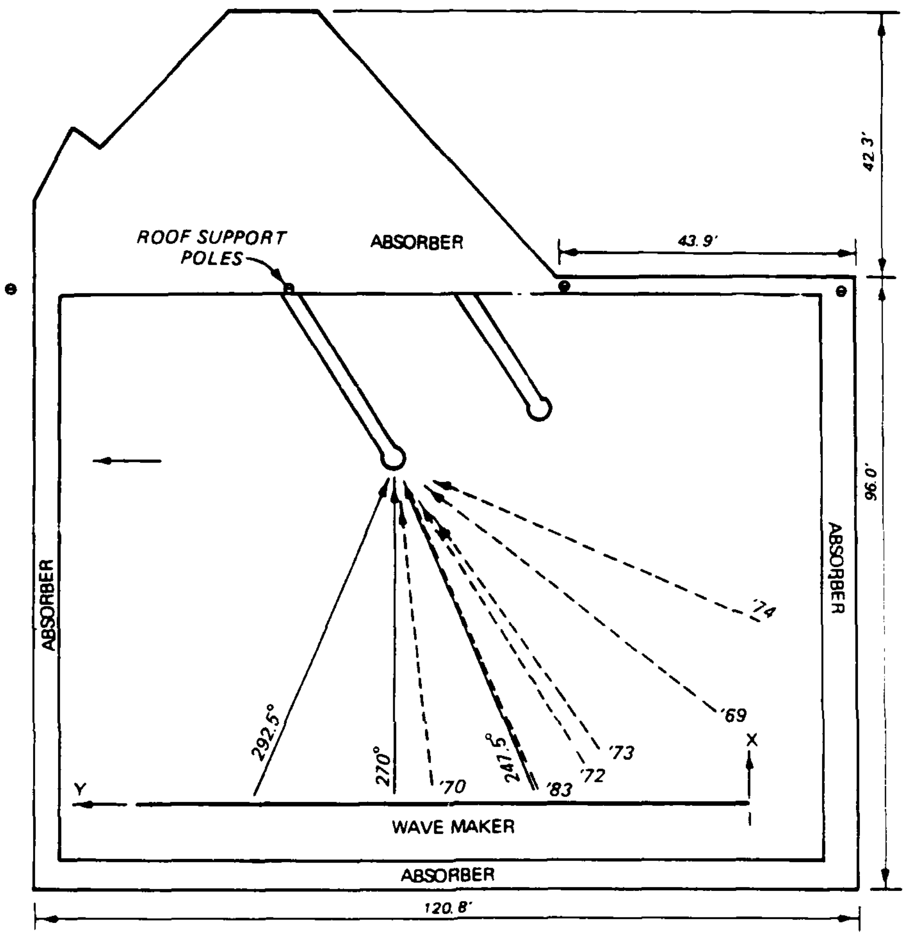


Figure 11. Hindcasted directions of wave attack

This spectrum was named by combining the first three letters of the three data sets used for field verification, i.e., texel, marsen, and arsloe. The TMA spectrum is a function of five basic parameters: water depth h, peak frequency fp, spectral width parameter σ, spectral peak enhancement parameter γ, and spectral alpha constant α. Tests were performed at two swl's, 0.0 and +10.0 ft mllw. The higher level represented an 8-ft tide with an added 2-ft storm surge. The peak frequency for each storm event was obtained directly from SHALWV results. In most cases, the spectral width parameters, σ_a and σ_b, were assigned values 0.07 and 0.09, respectively; however, other

values of σ were used to achieve proper fit of the 1972 and 1973 storm spectra. The peak enhancement parameter and alpha constant were varied as necessary by trial and error until an acceptable fit was accomplished relative to the target storm spectra. Plots depicting the computed wave generator control spectra and the predicted target storm spectra at the 58-ft water depth (0.0-ft swl) are shown in Figures 12-17.

18. For many years, stability tests at WES were performed solely with monochromatic wave conditions. Normal procedure in such a study was to identify the worst wave conditions by holding the swl and wave period constant while varying the wave generator stroke length. This resulted in the determination, for each wave period, of the particular wave height which broke directly on the structure. Using spectral waves, a similar approach was attempted at the beginning of this study to investigate the effect of offshore wave height on breaking wave severity at the structure. Thus, the first tests with the jetties in place were conducted to identify worst wave conditions to be used in subsequent stability tests. This was done by slightly varying the zero-moment wave height H_{mo} of the spectra at the generator while observing the jetty response and measuring wave heights just shoreward of the wave board and seaward of the jetty head. From this search for worst waves, six severe conditions were chosen for the long duration stability tests (Table 7). These six conditions represented the most severe runs observed for each combination of swl and angle of wave attack.

19. Between 1971 and 1981, the Oregon State University (OSU) Sea Grant Program sponsored an investigation of the nearshore wave climatology at Yaquina Bay (Creech 1981). Wave heights were measured at a location approximately 2,200 ft seaward of the north jetty head. The decade of testing yielded a maximum significant wave height of 24 ft with an associated period of 17 sec recorded during a storm in December 1972. This corresponded relatively well with the hindcasted results after shoaling and refraction into similar depths. In the physical model, the wave board was located at a point approximately 2,750 ft seaward of the north jetty head. Results of the wave hindcast and shallow water transformation undertaken during this investigation yielded a maximum H_{mo} of 24.5 ft and a peak period of 17 sec at the wave board. A cumulative distribution of significant wave heights resulting from the OSU program indicated that this was approximately equivalent to a 50-year event (Figure 18).

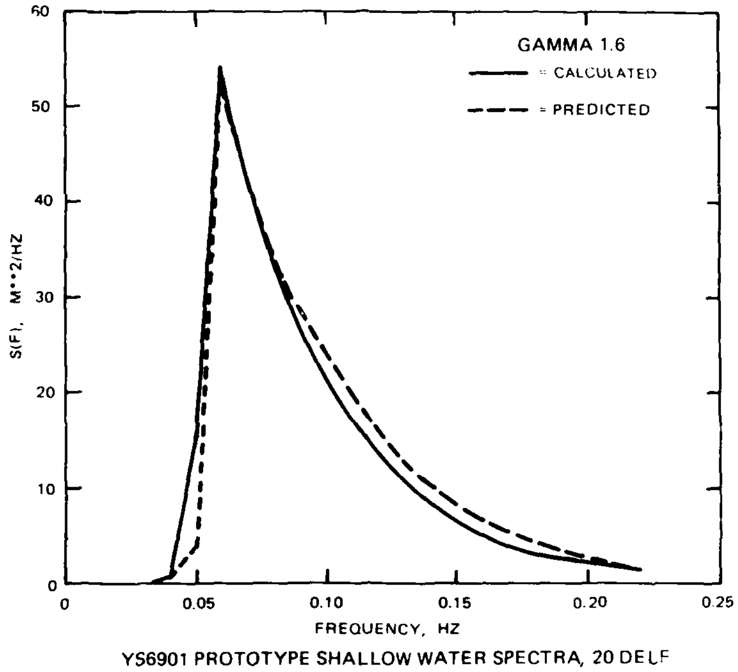


Figure 12. Spectral fit S(f) of 1969 storm at 58-ft depth

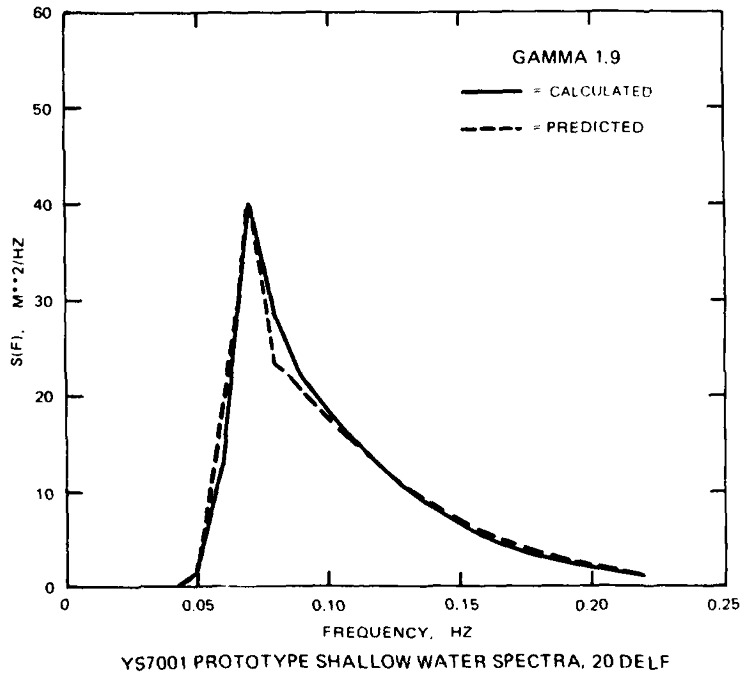


Figure 13. Spectral fit S(f) of 1970 storm at 58-ft depth

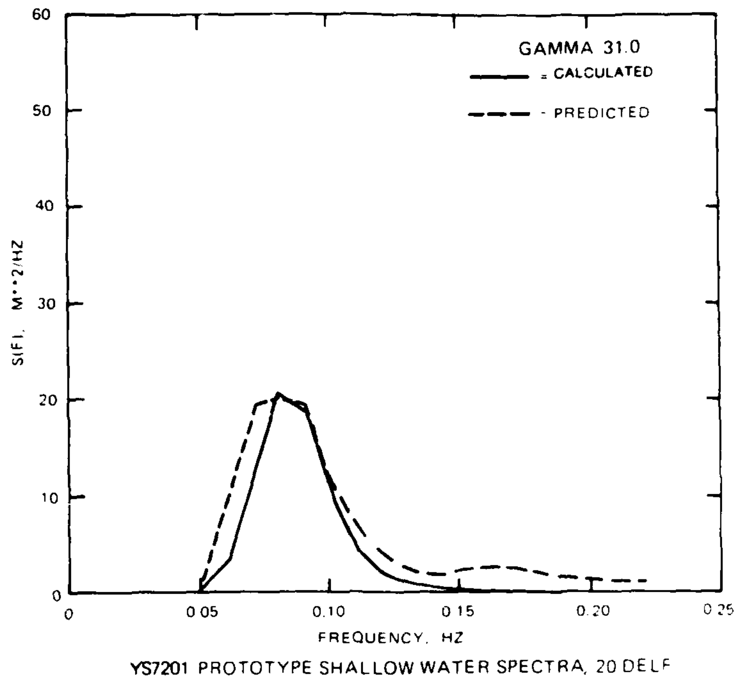


Figure 14. Spectral fit S(f) of 1972 storm at 58-ft depth

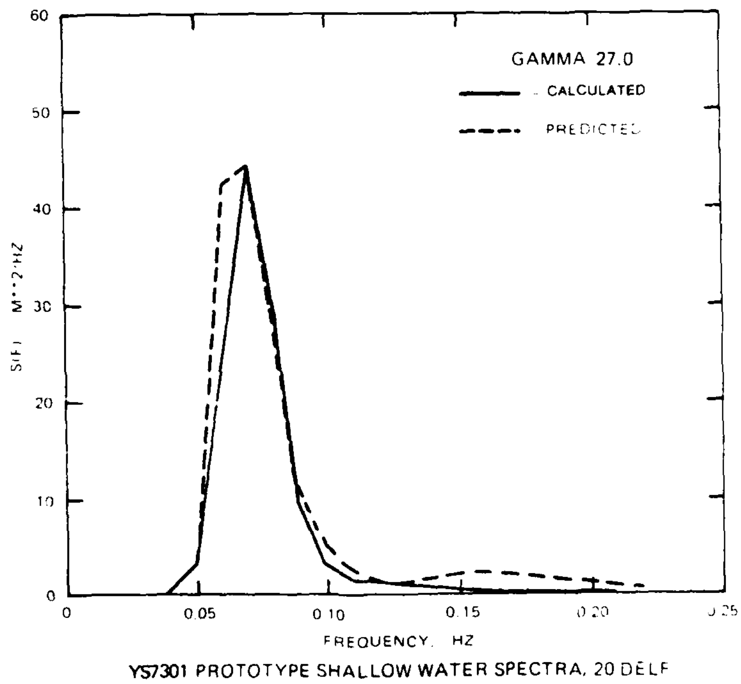


Figure 15. Spectral fit S(f) of 1973 storm at 58-ft depth

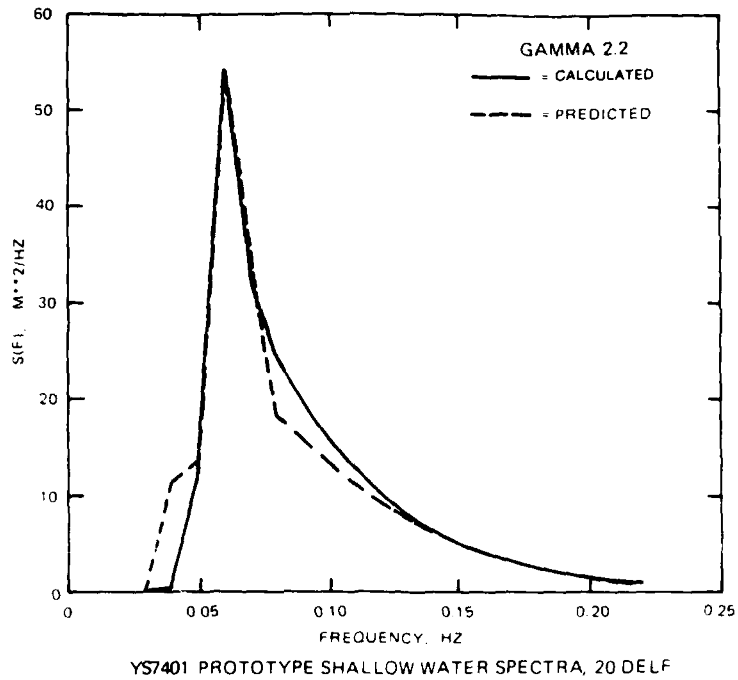


Figure 16. Spectral fit S(f) of 1974 storm at 58-ft depth

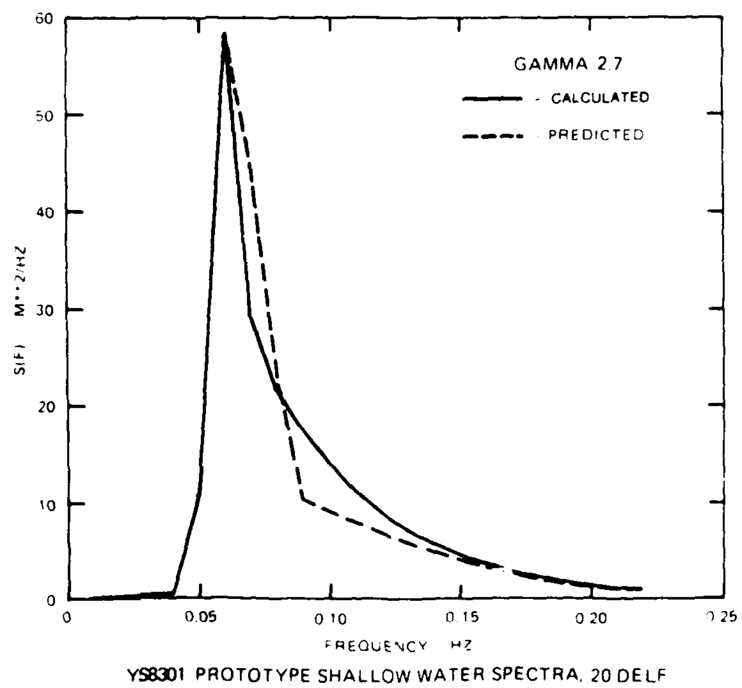


Figure 17. Spectral fit S(f) of 1983 storm at 58-ft depth

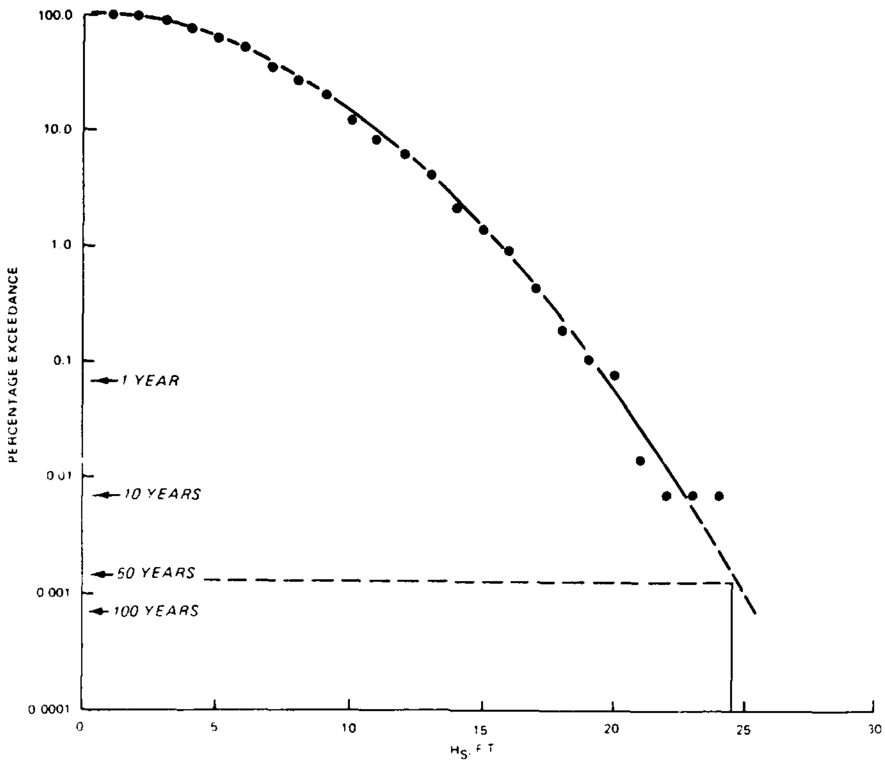


Figure 18. Cumulative distribution of significant wave heights H_s (October 1971-March 1981)

Monochromatic wave conditions

20. Monochromatic wave conditions used in the physical model study were first established during a previously completed numerical wave propagation analysis performed by CERC (Cialone 1986). The purpose of that effort was to determine coastal currents in the vicinity of the entrance to Yaquina Bay. Initial work by Ebersole* was used to identify deepwater wave conditions resulting from the WIS 20-year wave hindcast. From the hindcasted data, investigators chose to simulate wave conditions characterized by combinations of three wave periods and four directions of attack. Cialone (1986) used this information as input into the Regional Coastal Processes Wave model (RCPWAVE), a numerical model which can be used to solve monochromatic wave propagation problems over an arbitrary bathymetry (Ebersole, Cialone, and Prater 1985). For the physical model/numerical model wave comparison, the results of the RCPWAVE study by Cialone (1986) at the locations corresponding to the wave

* Op. cit.

board position in the physical model were chosen for creating monochromatic wave board control signals. The resulting test conditions are listed in Table 8.

PART III: JETTY STABILITY TESTS

Description of Proposed Rehabilitation Plan

21. During the period February to August 1985, CERC conducted the two numerical modeling investigations cited earlier to provide NPP with design information and guidance relative to the proposed north jetty rehabilitation: one to simulate the current regime (Cialone 1986), and the other to predict design wave heights.* The primary objective of the study by Cialone (1986) was to simulate the coastal current regime in the vicinity of the Yaquina Bay entrance. As a result, investigators were able to predict the velocities of both tidal and wave-induced currents at the head of the north jetty. Coastal current regimes were determined for five individual construction alternatives; therefore, final design guidance was based on the proposed rehabilitation plan which, when numerically modeled, corresponded with the optimum current field. Comparison of wave heights calculated in Cialone's (1986) study and those measured in the physical model will be discussed in Part IV.

22. The numerical investigation by Ebersole* involved the prediction of monochromatic design wave heights in the vicinity of the jetty head. This investigation was accomplished by means of a three-step procedure. Deepwater wave conditions were determined based on data gathered during WIS. These wave conditions were used as input for the second step, the numerical model RCPWAVE, which was used to bring the waves into the 41-ft contour area. From the 41-ft contour, the one-dimensional wave shoaling and breaking model (Dally, Dean, and Dalrymple 1984) was used to propagate the wave shoreward over the complex bathymetry of the reef and into the vicinity of the jetties and entrance channel. This procedure resulted in a predicted design wave height of 28 ft at the head of the jetty. Proceeding along the jetty 50 ft shoreward, the predicted design wave height was 25 ft, and at a point 150 ft shoreward of the jetty head, the predicted design wave height was 22 ft (USAED, Portland, 1987). These design waves were calculated based on the +10.0 ft mllw swl.

23. Armor-stone weights were determined using the Hudson Stability Formula (Hudson 1958) as follows:

---

* Ebersole, op. cit.

$$W_a = \frac{\gamma_a (H^3)}{Kd(S_a - 1)^3 \cot \alpha} \qquad (2)$$

where

H = wave height, ft

Kd = dimensionless stability coefficient

α = angle of the jetty's side slope measured from the horizontal

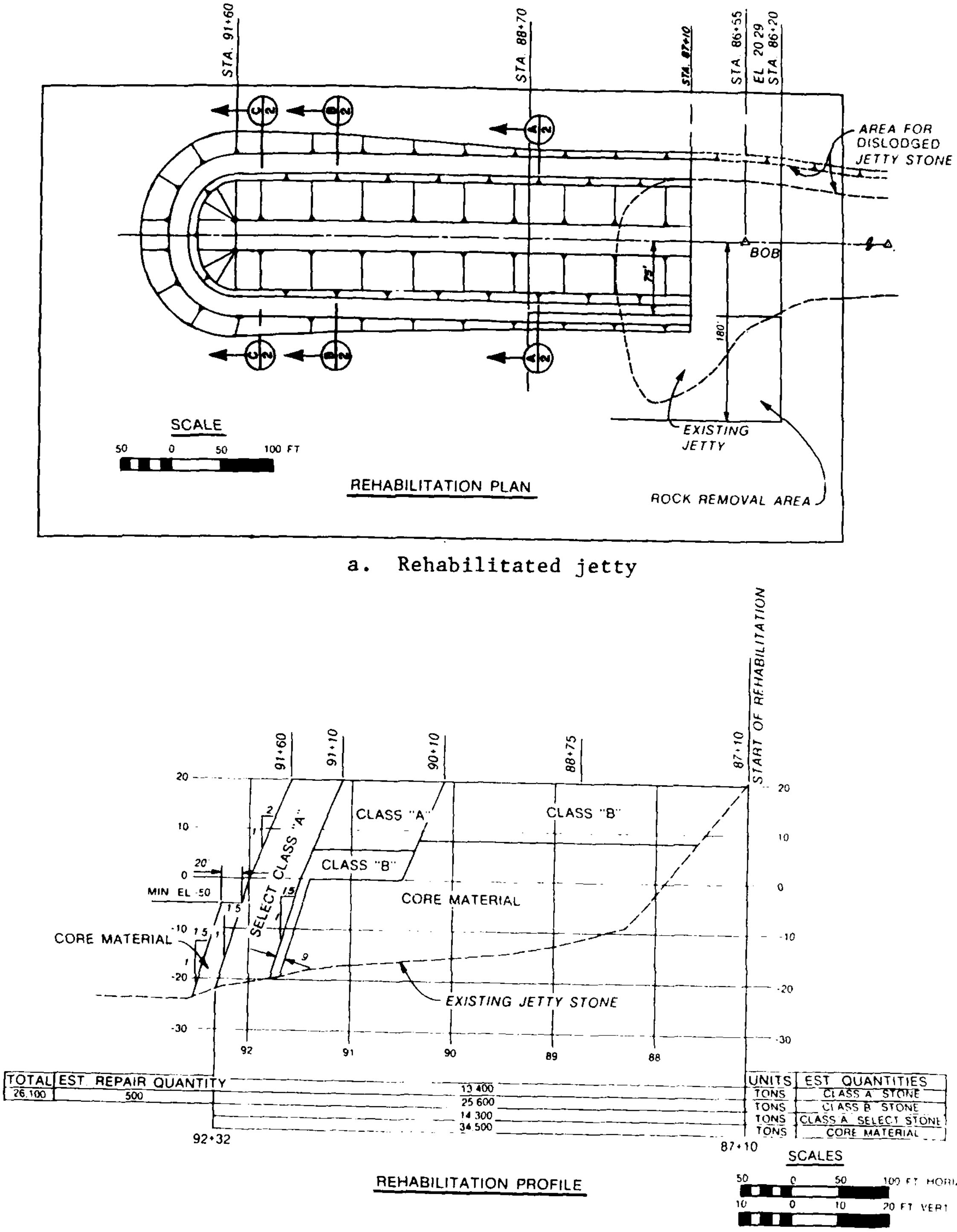
Armor stone placed above and below the low-water elevation were assigned stability coefficients of 8 and 5, respectively. These coefficients were chosen based on previous testing of the stability of special stone placement at Tillamook, Oregon (Markle and Davidson 1979). The Tillamook study indicated that where placed-stone construction can be achieved, a stability coefficient of 10 would be a conservative value. Jetty side slopes of 1V on 2H above 0.0 ft mllw and 1V on 1.5H below 0.0 ft mllw were used. Based on these parameters, a stone specific weight of 170 pcf, and the design wave heights mentioned earlier, the following stone weight requirements were determined:

Stone Classification	Minimum Weight tons	Average Weight tons	Maximum Weight tons
Select A-stone	26.5	29.0	None
A-stone	18.4	23.0	26.4
B-stone	12.0	16.0	18.3

These specifications reflect an arbitrary weight classification which specifies that the minimum weight for a given classification must be at least 15 percent greater than the average weight of the next lower stone classification.

24. Jetty design plan, profile, and cross sections are presented in Figures 19 and 20. The proposed rehabilitation plan was characterized by a jetty crest elevation of +20 ft mllw and a crest width of 30 ft. Armor sections were typically composed of two layers of stone. Below the armor layers was a core of graded material with weights ranging from 0.5 to 12 tons. No bedding material was proposed for use in the rehabilitation; however, provisions to remove portions of the existing deteriorated jetty stone were included in the initial specifications (Figure 20).

25. One of the most important aspects of this proposed rehabilitation



a. Rehabilitated jetty

b. Stability berm

Figure 19. Profile views of proposed rehabilitation plan and stability berm

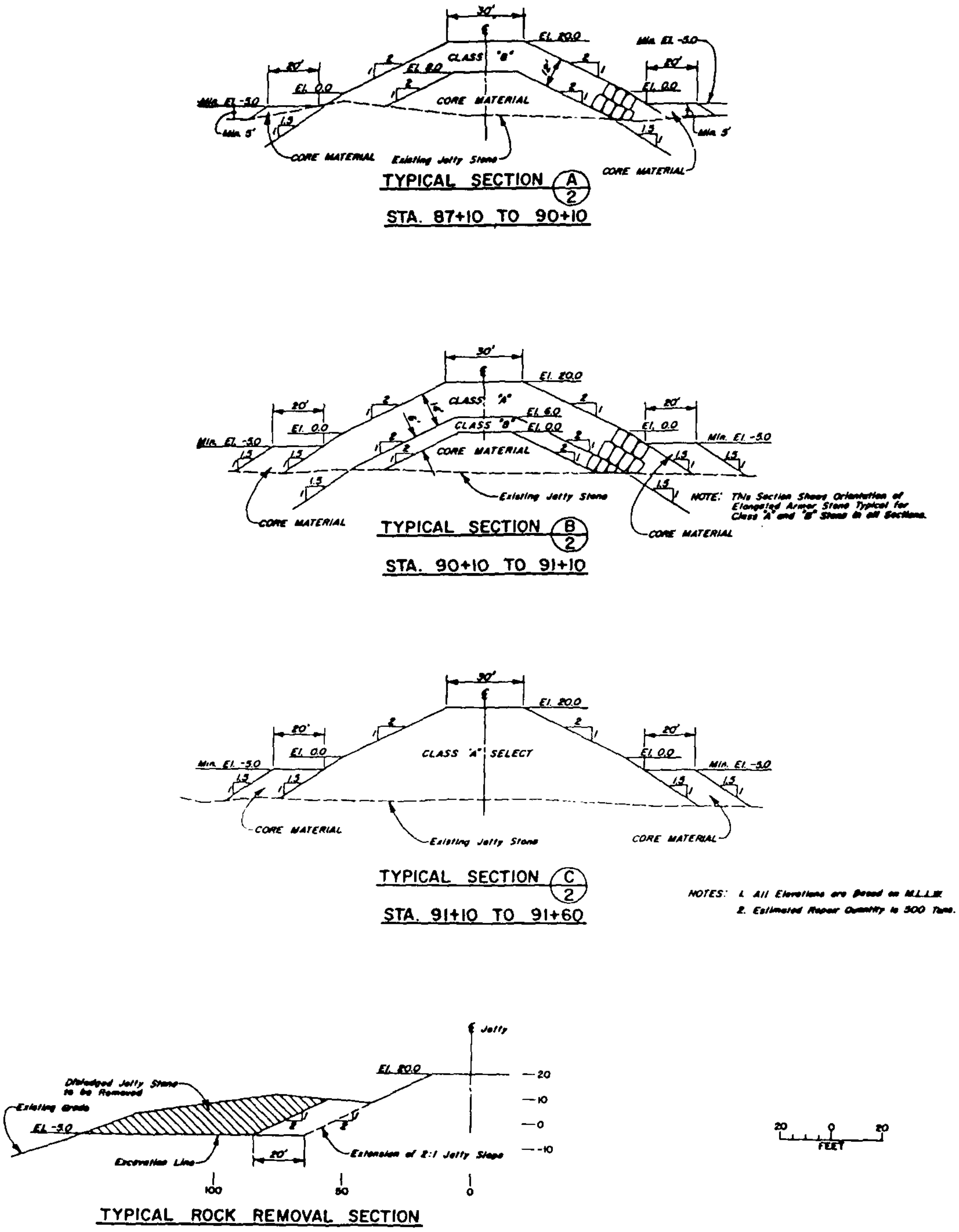


Figure 20. Jetty design cross sections

plan was the use of the placed-stone construction technique. This procedure requires the use of parallelepiped-shaped stone with each special shaped stone placed with its long axis normal to the jetty slope. Past field experience (USAED, Buffalo, 1946) and laboratory studies (US Army Engineer Waterways Experiment Station 1963, Debok and Sollitt undated, Markle and Davidson 1979) have indicated that the use of placed-stone construction techniques results in increased stability of rubble-mound structures when compared with similar structures armored with angular stone.

Storm Conditions Tested and Results

26. As mentioned in Part II, an extensive series of investigative wave tests were initially performed to establish the worst spectral conditions relative to energy dissipation on or near sections of the jetty rehabilitation. The objective of the selection process was to choose the most severe storm condition for each possible combination of swl and direction of wave attack. The six chosen conditions then served as the basis for subsequent long duration stability tests. The structure was rebuilt prior to initiation of the long duration tests and, in most cases, damages incurred during a test sequence were repaired before tests with a new storm condition were begun. Exceptions to this procedure will be noted and explained later in the text.

27. The maximum length of time during which wave generation and simultaneous wave data collection could be accomplished was 9.25 min; therefore, a complete test sequence consisted of six 9.25-min cycles of each condition resulting in a total duration of 55.5 min in the model. This corresponds to subjecting the structure to over 6 hr of peak storm conditions in the prototype. During spectral stability tests, wave data were collected at three locations as shown in Figure 4. This gage arrangement yielded wave height and period information very near the wave board and just seaward of the jetty head. The wave data collected during all spectral stability tests are presented in detail in Appendix A.

Still-water level = +10.0 ft, direction = WSW, storm = 1969 at 100-percent gain

28. Stability testing with unidirectional spectral waves was begun at the +10.0 ft swl with waves approaching from the west-southwest direction. The most severe storm corresponding to those conditions was the 1969 storm at

100-percent gain. Photos 1-4 were taken of the structure before testing. Wave heights measured during the test sequence (Table A1) indicated that the structure was subjected to the following conditions:

a. At wave board (prototype depth = 68 ft):

$$H_{mo} = 24.2 \text{ ft}, \quad T_p = 16.9 \text{ sec}$$

b. At jetty head:

$$H_{mo} = 19.8 \text{ ft}, \quad T_p = 17.0 \text{ sec}$$

Results of the test sequence are summarized below:

a. Cycle 1. No stones were displaced. No movement was noticed in the 16-ton section. One 29-ton parallelepiped stone was rocking in place on the crown. One 23-ton stone on the crown was rocking in place.

b. Cycle 2. Rocking in place of the 23-ton stone continued. The 29-ton stone which was rocking became stable late in the cycle. One 29-ton stone at swl on the head was rocking slightly during the most severe waves. One 16-ton stone just above swl on the south side was pulled out slightly from the matrix after rocking in place for the first half of cycle 2. The stone immediately above it dropped down slightly into the resulting void space. After this occurred, no further movement of either 16-ton stone was noticed.

c. Cycle 3. Rocking in place of the 29-ton stone at swl and the 23-ton stone on the crown continued. No movement was detected in the 16-ton section.

d. Cycle 4. Previously noted rocking continued. One additional stone on the crown at the 23-ton/29-ton transition began rocking in place slightly.

e. Cycle 5. The 23-ton stone which began rocking in cycle 4 was pushed shoreward to lodge against an adjacent stone. No further movement of this stone was noticed. Previously noted rocking of other stones continued.

f. Cycle 6. Previously noted rocking continued. One 16-ton stone at swl on the south side was displaced approximately 2 in. (7.5 ft prototype) downslope and 2 in. (7.5 ft prototype) shoreward.

Since the extent of jetty damage was so minor, photographic documentation of after-testing conditions was not deemed necessary, and further testing was resumed.

Still-water level = +10.0 ft, direction = W, storm = 1974 at 100-percent gain

29. The next sequence of tests was performed with the 1974 storm conditions at 100-percent gain. Waves were approaching from the west and the swl remained at +10.0 ft. Measured wave heights (Table A2) indicated that the structure was subjected to the following conditions:

a. At wave board (prototype depth = 68 ft)

$$H_{mo} = 21.9 \text{ ft}, \quad T_p = 16.4 \text{ sec}$$

b. At jetty head:

$$H_{mo} = 20.1 \text{ ft}, \quad T_p = 16.5 \text{ sec}$$

Observed results of this test series are summarized below:

a. Cycle 1. No stones were displaced in cycle 1. Two 29-ton stones near swl on the face of the head were rocking strongly in place.

b. Cycle 2. One angular 29-ton stone on the face of the head was pushed upslope approximately 3 in. (11.3 ft prototype). Another 29-ton angular stone on the face of the head was rocking slightly. One small (5- to 12-ton) stone from the deteriorated base section was pushed up on the south side of the 29-ton toe.

c. Cycle 3. Rocking continued in the 29-ton area. One 23-ton crown stone at the 23-ton/29-ton transition was rocking strongly. One 23-ton crown stone near the 16-ton/23-ton transition was displaced shoreward down the north side of the jetty to rest on the toe of the 16-ton area.

d. Cycle 4. One 29-ton stone slightly below swl on the south side of the head rolled down to the toe. Surrounding stones shifted slightly to fill the void. Previously mentioned rocking continued.

e. Cycle 5. One large (12- to 18-ton) stone from the south side of the deteriorated base section was pushed up approximately 4 in. (15 ft prototype) onto the toe of the 29-ton section. Previously noted rocking continued.

f. Cycle 6. No additional stone movement was detected.

On completion of this test series, water in the basin was drained and the after-testing condition of the jetty was photographed (Photos 5-8). With the basin dry, two stones which had been displaced unnoticed by observers during

testing were detected. One 29-ton toe stone at the jetty's center line was kicked up to rest on the next stone upslope. Also, one 16-ton stone, presumably from the crown, was pushed down the south side of the jetty to rest at the toe near sta 87+00. At this point, damaged portions of the jetty were repaired.

Still-water level = +10.0 ft, direction = WNW, storm = 1969 at 100-percent gain

30. The third test series at the +10.0 ft swl was performed with the 1969 storm conditions at a 100-percent gain setting. Wave attack was from the west-northwest direction. Wave measurements during testing (Table A3) identified the following conditions:

a. At wave board (prototype depth = 68 ft):

$$H_{mo} = 24.2 \text{ ft}, \quad T_p = 16.9 \text{ sec}$$

b. At jetty head:

$$H_{mo} = 20.4 \text{ ft}, \quad T_p = 17.0 \text{ sec}$$

Before-testing conditions are shown in Photos 9-12. Test results are summarized below:

a. Cycle 1. One 29-ton stone in center of the head section was rocking strongly.

b. Cycle 2. No new stone movement was observed. Previously noted rocking of the 29-ton stone continued.

c. Cycle 3. The rocking 29-ton stone shifted position slightly and stabilized.

d. Cycle 4. No new movement was observed. After the test cycle, it was noticed that one 16-ton crown stone near sta 89+50 had been pulled down to the toe area on the north side of the jetty.

e. Cycle 5. One 29-ton toe stone at the jetty center line was pulled out and moved south approximately 5 in. (18.7 ft prototype). One 29-ton stone near swl on the south side of the jetty head was displaced downslope to the toe. Another slightly higher 29-ton stone dropped into the remaining void.

f. Cycle 6. No new stone movement was observed.

Since some armor-stone movement had occurred in the latter stages of this test series, investigators decided to subject the jetty to further testing with the

second most severe storm condition corresponding to this water level and direction of wave attack.

Still-water level = +10.0 ft, direction = WNW, storm = 1983 at 85-percent gain

31. This additional unscheduled test series was executed at the +10.0 ft swl, again with waves from the west-northwest. The storm chosen was the 1983 condition at an 85-percent gain. Wave measurements (Table A4) verified that the following wave conditions were generated:

a. At wave board (prototype depth = 68 ft):

$$H_{mo} = 19.2 \text{ ft}, \quad T_p = 17.2 \text{ sec}$$

b. At jetty head:

$$H_{mo} = 19.3 \text{ ft}, \quad T_p = 17.1 \text{ sec}$$

Observations made during testing are summarized below:

a. Cycle 1. There was no noticeable stone movement in any area.

b. Cycle 2. One angular 29-ton stone near the center line just below swl was displaced to a point slightly seaward of the toe. The 29-ton stone which had filled the void space in cycle 5 of the prior test series was pulled out and moved to the toe area on the jetty's south side.

c. Cycle 3. One 29-ton stone was rocking on the north side of the head near the swl. The 29-ton stone which had stabilized in cycle 3 of the previous test series shifted approximately 1 in. seaward and began rocking again.

d. Cycle 4. One angular 29-ton crown stone near the 23-ton/29-ton transition moved approximately 1 in. southward and began rocking in place slightly.

e. Cycle 5. No new stone movement was observed.

f. Cycle 6. No new stone movement was observed.

At this point, the basin was drained and after-testing photographs of the structure were taken (Photos 13-15).

Still-water level = 0.0 ft, direction = WSW, storm = 1969 at 100-percent gain

32. Stability tests at the 0.0-ft swl were also begun with wave attack from the west-southwest. The 1969 storm at a 100-percent gain had been chosen

as the most severe condition. This corresponded to the following test wave conditions (Table A5):

a. At wave board (prototype depth = 58 ft):

$$H_{mo} = 21.0 \text{ ft}, \quad T_p = 16.9 \text{ sec}$$

b. At jetty head:

$$H_{mo} = 15.1 \text{ ft}, \quad T_p = 17.0 \text{ sec}$$

The condition of the jetty prior to testing is shown in Photos 16-18. Test results are summarized below:

a. Cycle 1. One 29-ton crown stone demonstrated minor in-place rocking.

b. Cycle 2. Another 29-ton stone just above the swl was rocking slightly.

c. Cycle 3. The stone movement documented in cycle 2 stopped. No other new movement was observed.

d. Cycle 4. No new stone movement was observed.

e. Cycle 5. No new stone movement was observed.

f. Cycle 6. No new stone movement was observed.

Throughout this series of tests, the proposed rehabilitation sections remained in very stable condition; therefore, after-testing photographs were cancelled and the next test series was begun.

Still-water level = 0.0 ft, direction = W, storm = 1969 at 100-percent gain

33. Testing at the 0.0-ft swl continued with the 1969 storm and 100-percent gain wave conditions generated from the west. Wave measurements (Table A6) indicated that the jetty was subjected to the following conditions:

a. At wave board (prototype depth = 58 ft):

$$H_{mo} = 21.7 \text{ ft}, \quad T_p = 16.9 \text{ sec}$$

b. At jetty head:

$$H_{mo} = 15.9 \text{ ft}, \quad T_p = 17.1 \text{ sec}$$

Results of this test series are listed below:

a. Cycle 1. The 29-ton stone noted in cycle 1 of the prior series continued rocking slightly.

b. Cycle 2. No new stone movement was observed.

c. Cycle 3. No new stone movement was observed.

d. Cycle 4. No new stone movement was observed.

e. Cycle 5. One 29-ton stone just below swl on the north side of the head showed minor in-place rocking.

f. Cycle 6. One 23-ton stone at swl on the north side was displaced. This stone was originally just seaward of the 16-ton/23-ton transition and it came to rest on the 16-ton toe near sta 90+00. Another 23-ton stone slightly above swl on the north side shifted approximately 2 in. (7.5 ft prototype) into a new position.

Again, since jetty damage was very minor at this point, after-testing photographs were not taken and testing was continued.

Still-water level = 0.0 ft, direction = WNW, storm = 1974 at 100-percent gain

34. The final test series at the 0.0-ft swl consisted of 100-percent gain, 1974 storm conditions approaching from the west-northwest. Wave measurements (Table A7) yielded the following heights and periods:

a. At wave board (prototype depth = 58 ft):

$$H_{mo} = 18.4 \text{ ft}, \quad T_p = 16.8 \text{ sec}$$

b. At jetty head:

$$H_{mo} = 16.1 \text{ ft}, \quad T_p = 17.1 \text{ sec}$$

The following observations summarize the results of this test series:

a. Cycle 1. Minor rocking of the previously noted 29-ton crown stone continued.

b. Cycle 2. No new stone movement was observed.

c. Cycle 3. One 29-ton toe stone just south of the center line shifted approximately 2 in. (7.5 ft prototype) seaward. Another 29-ton stone just below swl on the north side showed minor rocking in place.

d. Cycle 4. No new stone movement was observed.

e. Cycle 5. One 29-ton stone just above swl on the north side was rocking slightly in place.

f. Cycle 6. No new stone movement was observed.

Photos 19-21 show the condition of the jetty after testing at the 0.0-ft swl.

35. As the above summaries of test observations indicate, some minor damage was incurred at the 0.0-ft swl. At this point, the investigators wished to observe the response of the slightly damaged structure when subjected to high-water storm conditions. This series of tests were designed to simulate a situation in which a storm resulted in minor jetty damage which was not repaired prior to the next severe storm. The storm conditions at the +10.0 ft swl were chosen based on their relatively greater severity. Wave data were collected at various points throughout the basin during this series of tests (Figure 4). A comprehensive tabulation of this data is also included in Appendix A.

Still-water level = +10.0 ft, direction = WSW, storm = 1969 at 100-percent gain

36. This series of tests were initiated with the 100-percent gain, 1969 wave conditions approaching from the west-southwest. Wave measurements (Table A8) indicated that the following conditions were generated:

a. At wave board (prototype depth = 68 ft):

$$H_{mo} = 24.4 \text{ ft}, \quad T_p = 16.9 \text{ sec}$$

b. At jetty head:

$$H_{mo} = 19.6 \text{ ft}, \quad T_p = 17.0 \text{ sec}$$

Results of the tests from the west-southwest direction are summarized below:

a. Cycle 1. Minor rocking of the previously noted 29-ton crown stone continued early in the cycle and then stabilized. One 29-ton crown stone at the 23-ton/29-ton transition demonstrated strong in-place rocking.

b Cycle 2. Strong rocking of one 29-ton crown stone continued.

c. Cycle 3. No new stone movement was observed.

d. Cycle 4. No new stone movement was observed.

e. Cycle 5. No new stone movement was observed.

f. Cycle 6. Previously noted rocking continued. One 16-ton stone at swl on the north side of the jetty near sta 88+00 was rocking in place. One 23-ton crown stone near the 16-ton/23-ton transition was rocking slightly.

Still-water level = +10.0 ft, direction = W, storm = 1974 at 100-percent gain

37. Testing continued at the +10.0 ft swl with 100-percent gain, 1974 storm conditions approaching from the west. Wave height measurements (Table A9) yielded the following incident conditions:

a. At wave board (prototype depth = 68 ft):

$$H_{mo} = 21.5 \text{ ft}, \quad T_p = 16.4 \text{ sec}$$

b. At jetty head:

$$H_{mo} = 19.1 \text{ ft}, \quad T_p = 16.2 \text{ sec}$$

Results of this test series are listed below:

a. Cycle 1. One 16-ton crown stone near sta 87+50 began moderate in-place rocking. The 16-ton stone noted previously near sta 88+00 showed moderate rocking throughout the test. Rocking of the 29-ton stone noted in cycles 1 and 2 of the prior test series also continued.

b. Cycle 2. Previously noted rocking continued. One 29-ton stone just above swl on the north side near sta 91+50 began rocking slightly.

c. Cycle 3. One 29-ton angular crown stone near sta 91+50 was moved approximately 5 in. (18.7 ft prototype) southward to rest on the slope just above swl. Another 29-ton crown stone rocked slightly early in the test but later stabilized.

d. Cycle 4. One 16-ton stone at swl on the north side near sta 89+00 began minor rocking. Previously noted rocking continued.

e. Cycle 5. The 29-ton stone which stabilized in cycle 1 of the previous test series resumed minor rocking.

f. Cycle 6. One 16-ton crown stone near sta 89+90 began strong in-place rocking. One 16-ton stone just above swl on the south side near sta 90+00 was displaced to the toe area.

Still-water level = +10.0 ft, direction = WNW, storm = 1969 at 100-percent gain

38. The final series of tests were performed with 100-percent gain, 1969 storm conditions approaching from the west-northwest. Measured wave

heights (Table A10) indicated that the structure was subjected to the following conditions:

a. At wave board (prototype depth = 68 ft):

$$H_{mo} = 24.2 \text{ ft}, \quad T_p = 16.7 \text{ sec}$$

b. At jetty head:

$$H_{mo} = 21.2 \text{ ft}, \quad T_p = 17.0 \text{ sec}$$

Observed results of the test series are summarized below:

a. Cycle 1. One 16-ton stone slightly above swl on the north side near sta 88+00 rocked mildly early in the test, then flipped over and stabilized. Previously noted rocking continued.

b. Cycle 2. One 29-ton stone on the 1:2 slope just north of the jetty center line was displaced to a point seaward of the tow and then pushed approximately 1 ft (45 ft prototype) southward.

c. Cycle 3. No new stone movement was observed.

d. Cycle 4. One 23-ton stone just below swl on the north side began minor in-place rocking.

e. Cycle 5. Previously noted rocking continued. No new stone movement was observed.

f. Cycle 6. Previously noted rocking continued. No new stone movement was observed.

PART IV: PHYSICAL MODEL/NUMERICAL MODEL COMPARISON TESTS

Description of Numerical Modeling Investigation

39. As noted earlier, NPP authorized CERC to perform an extensive numerical current modeling investigation in 1985 (Cialone 1986). The primary objective of the study was to model coastal currents in the vicinity of the entrance to Yaquina Bay, thereby determining the velocities of the tidal and wave-induced currents near the north jetty. The resulting information on current patterns and intensities was used to evaluate stability of the jetty foundation. The process of determining wave conditions and water current patterns at the Yaquina Bay entrance involved three individual numerical models. These were the WES Implicit Flooding Model (WIFM), the wave-induced current model (CURRENT), and RCPWAVE, which was mentioned in Parts II and III.

40. The RCPWAVE (Ebersole, Cialone, and Prater 1985) model was used to provide wave characteristics necessary for input into the wave-induced current model. RCPWAVE uses finite difference approximations of the governing equations to predict wave propagation outside the surf zone. Wave transformation inside the surf zone is predicted by an empirical method which is based on a hydraulic jump representation of the entire surf zone. The input information required by RCPWAVE includes bathymetric data at each grid cell and the deepwater monochromatic wave height, period, and direction. These deepwater wave conditions were obtained from the 20-year wave hindcast data base (WIS) used during Ebersole's* earlier study. The model computes the corresponding wave height, wave length, and direction at each cell as the monochromatic wave propagates shoreward and responds to the irregular bathymetric features over which it passes. Since Ebersole* had already used RCPWAVE to calculate waves in the -41 ft mllw contour area, Cialone's (1986) inshore grid started at approximately the -96 ft mllw contour and used RCPWAVE to propagate the wave shoreward over the complex bathymetry of the reef and into the vicinity of the jetties and entrance channel. The model was based on the concept that the energy in a breaking wave seeks to attain some stable level, and the rate of decay of energy is related to the deviation in wave energy from its stable level.

---

* Op. cit.

41. Tidal current predictions were obtained with the WIFM model (Butler 1980). This finite difference model was used to compute tidal elevations and currents around the jetties and in the entrance channel.

42. Another finite difference model, CURRENT (Vemulakonda 1984), was used for predicting wave-induced currents. This model computes the horizontal velocity components of longshore and cross-shore currents due to waves only. The current model employs a variably spaced finite difference grid and requires, as input information, the wave number, wave angle, and wave height at each grid cell; therefore, for each deepwater incident wave condition investigated in the current-related study, a corresponding set of wave characteristics for each grid cell had to be determined. As mentioned earlier, this set of wave characteristics was furnished by RCPWAVE (Cialone 1986).

Conditions Tested and Results

43. The monochromatic wave conditions simulated in the physical model were established based on the RCPWAVE model wave conditions used during the numerical investigation (Cialone 1986). Those conditions are listed in Table 8. The wave board conditions were extracted from detailed listings of RCPWAVE results. They are the averaged heights and wave directions which were predicted at the grid cells corresponding to the location of the wave board in the physical model (approximately -58 ft mllw contour). All monochromatic tests were performed at the +10.0 ft mllw swl resulting in a total water depth of 68 ft at the wave board.

44. Documentation of the earlier numerical model investigation indicated that armor-stone weights were based on three wave heights predicted at various locations along the jetty (Cialone and Simpson 1987; USAED, Portland, 1987). These design wave heights were 28 ft at the jetty head, 25 ft at sta 91+10, and 22 ft at and shoreward of sta 90+10. This documentation also indicated that RCPWAVE input wave conditions which resulted in the design wave predictions included wave periods of 12.5, 14.3, and 16.7 sec (Cialone and Simpson 1987, Ebersole*); however, detailed listings of RCPWAVE output related to these specific conditions were not available. Detailed listings which were available corresponded to wave periods of 11.0, 14.0, and 16.0 sec. Although

---

* Ebersole, op. cit.

the predicted wave heights resulting from these runs were not as great as the design values mentioned above, duplication of those conditions in the physical model resulted in measured wave heights approximating the design heights at the jetty head (27.2 ft).

45. The model time duration of each monochromatic test case was 3 min; therefore, the number of waves generated during a particular test was dependent on the wave period as follows:

Wave Period, sec	Number of Waves per Test
11	110
14	86
16	75

Wave heights were measured at 10 locations as shown in Figure 4. A 10-Hz sampling rate was used in all cases.

46. After testing, the data were subjected to a downcrossing analysis. Results of this analysis included significant and average wave periods, and significant, average, and maximum wave heights. A portion of a typical time series record is presented in Figure 21. Tabulated results of the monochromatic wave data are included in Appendix B. During verification of the RCPWAVE model, significant wave heights, peak periods, and wave directions estimated from radar imagery were determined from field data for comparison with the predicted values. For the purpose of this investigation, RCPWAVE predictions were compared with significant wave periods and significant, average, and maximum wave heights measured in the physical model. A wave directionality comparison is not presented since the scope of this physical model study did not permit such measurements.

47. The comparison between numerically predicted wave heights and those measured in the physical model indicated that the RCPWAVE predictions correlated best with the average measured wave heights; however, in the area near the reef crest and jetty head, the measured heights were consistently greater than the RCPWAVE predictions. On initiation of the comparative analysis, it was assumed that a linear relationship existed between the numerical predictions and the corresponding measured heights. In an effort to estimate the strengths of those relationships, correlation analyses were performed on each of the three sets of measured data (i.e., significant wave heights H_{sig}, maximum wave heights H_{max}, and average wave heights H_{avg}) relative to the numerical predictions. The results of all 18 test runs were subjected to such

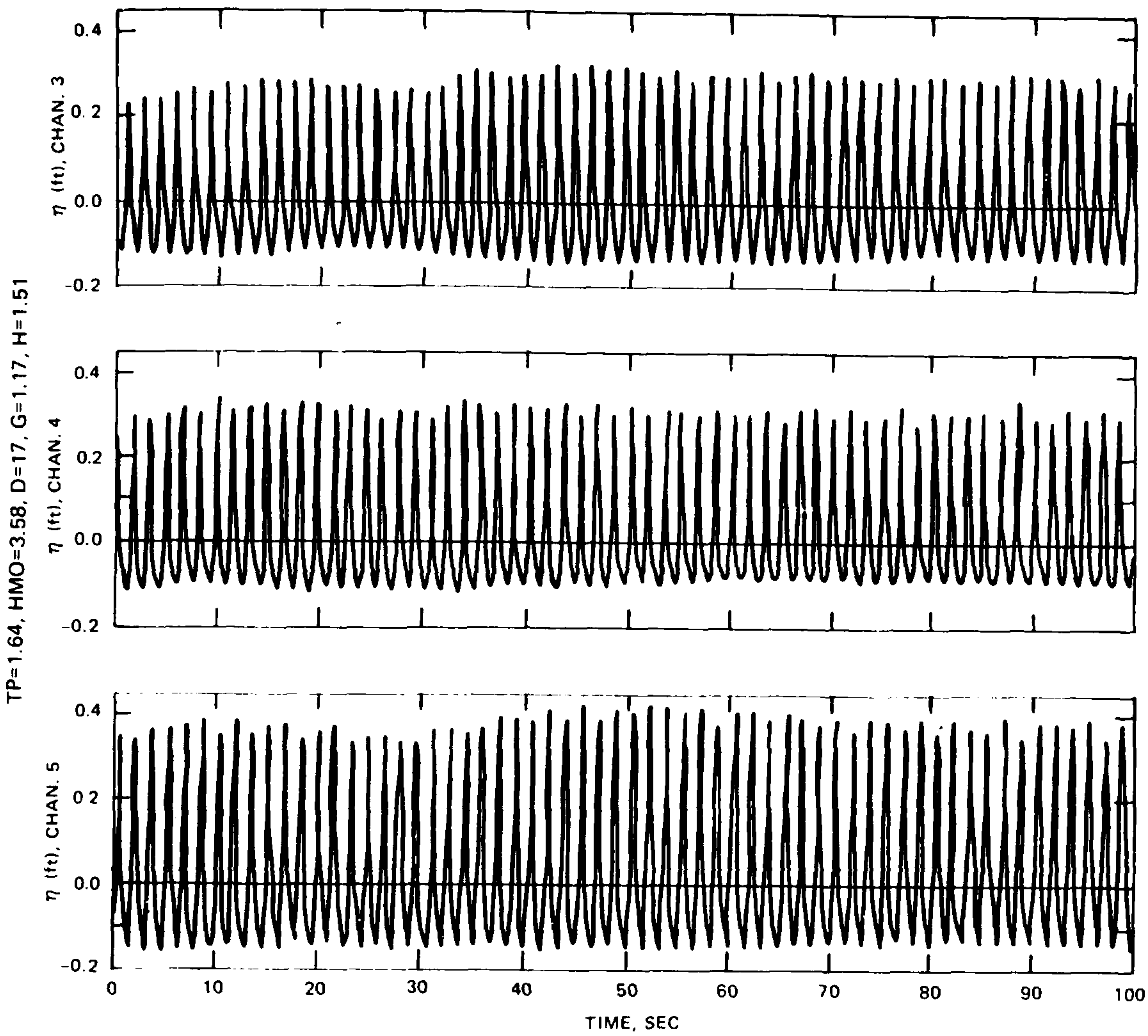


Figure 21. Typical monochromatic wave record

an analysis. After a linear regression for each data set was performed, a correlation coefficient r , was calculated as an indicator of the strength of the relationship. For the average measured wave heights, the 18 correlation coefficients ranged between 0.12 and 0.93, with a mean value of 0.60. The significant measured wave heights demonstrated the second best correlation with r values ranging from 0.10 to 0.93, with a mean value of 0.54. Coefficients for the maximum measured wave heights ranged from 0.00 to 0.93, with a mean value of 0.49. Data resulting from the wave height comparison tests are tabulated and graphically presented in Appendix B. In Figures B1-B18, values on the abscissa are more clearly understood when compared with Figure 4 which shows the wave-gage locations.

48. A major concern resulting from the numerical study was related to the accuracy of the predicted wave heights along the outer 400 ft of the jetty. These were the heights used for the armor-stone weight computations and, as mentioned previously, they indicated a substantial decrease in wave height as the wave progressed shoreward from the head. The measured wave heights also demonstrated this falloff and, in some cases, corresponded fairly well to the predicted values shoreward of the head section (Figures B1-B18). The numerical study did not seem to make accurate wave height predictions in the areas immediately seaward and shoreward of the head section.

PART V: DISCUSSION AND CONCLUSIONS

49. The design sections for the proposed jetty rehabilitation were constructed in the the physical model and subjected to various unidirectional spectral wave conditions. Based on results of those stability tests, it was concluded that:

a. The recommended 29-, 23-, and 16-ton mean armor-stone weights proved adequate when subjected to wave conditions representative of the most severe storms selected using 20-year wave hindcast results. As mentioned previously, those armor weights were calculated using the Hudson Stability Formula (Hudson 1958). The computations were executed using stability coefficients of 8.0 and 5.0 for armor stone placed above and below 0.0-ft mllw, respectively (USAED, Portland, 1987). The test results indicated that the use of these proposed Kd values was justified. Although, at some point during the tests, each of the three armored sections underwent some damage in the form of displaced armor stones, the level of damage was in all cases slight.

b. The proposed plan to construct the rehabilitated jetty on the existing deteriorated jetty stone is acceptable. The bathymetric features created by the 5- to 19-ton deteriorated jetty stone were simulated in the model using properly scaled angular stone. Cross sections of the area were obtained from a 1987 bathymetric survey performed by NPP. Test results indicated that problems with jetty toe stability were minimal when armor toe stones were placed using the "NPD toe placement" technique. This placement method involves positioning of the armor stones with their long axes perpendicular to the longitudinal axis of the jetty (Markle and Davidson 1979). However, stone placement at the toe and on the 1:1.5 slope is more random than in areas on the upper slope simply because of placement difficulties encountered below the water surface. The only displacements of toe stones observed occurred on the extreme seaward portion of the jetty head where the tip of the jetty rested on the offshore reef; however, areas in which these displacements occurred immediately stabilized and no further problems developed. For design conservatism, the outer head section was constructed using a mixture of approximately 33-percent angular and 67-percent parallelepiped 29-ton stone. Greater toe stability would probably be achieved if all toe stones were selected to more closely adhere to the parallelepiped shape criteria.

c. The displaced armor berm which lies near the existing end of the jetty should be excavated during jetty rehabilitation. In the model, tests were run with the berm in place and later with all material above -5.0 ft mllw removed. Video recordings and observations made during each test indicated that wave severity in this area was lessened with removal of the berm, which is as

high as +11.8 ft mllw at its crest. Removal of the stone may alleviate some difficulties encountered by pilots of smaller vessels navigating the north side of the entrance channel during inclement weather conditions. The proposed rehabilitation plan included measures which would be taken to resist current-induced erosion forces. One such measure was to reinforce the submerged channel-side slope along the outer 300 ft of the jetty by placing a toe protection berm constructed of core material below -5.0 ft mllw. If specifications would allow the use of larger stones in construction of the toe berm, some of the material obtained during removal of the existing berm could be used in this capacity.

50. The monochromatic wave tests were included in the investigation primarily to compare wave heights measured in the physical model with wave heights predicted by the numerical model, RCPWAVE, and the one-dimensional shoaling and breaking model. However, during all monochromatic tests, the jetty stability response was observed and recorded. Monochromatic wave heights as great as 27.2 ft at the head of the jetty were generated. Throughout the monochromatic tests, the only damage observed was minor displacement of two 29-ton armor stones. This occured in response to the above-mentioned wave condition (deepwater H = 14.0 ft, T = 14.0 sec, direction = WSW).

51. When monochromatic wave heights measured in the physical model were compared with corresponding wave heights predicted by the numerical models (Appendix B), the following conclusions were drawn:

- a. The numerically and physically modeled wave heights agreed reasonably well in the vicinity of the mildly sloped bathymetry seaward of the complex reef area and jetty head.
- b. Both sets of data indicated that waves increased in height just prior to reaching the jetty head; however, the physically modeled wave heights were generally much higher.
- c. Both sets of data demonstrated that a decrease in wave height occurred as the wave progressed shoreward from the jetty head.
- d. In general, the numerical predictions and physical model measurements did not agree well in the areas immediately seaward and shoreward of the structure head.

52. Based on the data presented in Appendix B, measurements from Gages 1 and 2 suggest that the numerical model, RCPWAVE, yields reasonable wave height magnitudes for bathymetry with relatively gentle slopes; however, Gages 3-6 indicate that the RCPWAVE model results in the vicinity of wave shoaling and breaking do not compare well with wave heights measured in: (a) the surf zone of the physical model where changes in bathymetry were abrupt and severe, or (b) the vicinity of the rubble-mound structures.

53. Although the numerical predictions and physical measurements did not compare well in areas near shore or structures, it should be noted that the numerical modeling investigation was a state-of-the-art effort that combined various individual models and the inherent assumptions on which they were based. From this comparison, it may be concluded that further efforts of this type are needed to ensure future developments relative to numerical simulation of wave dynamics in the surf zone and in the vicinity of coastal structures.

54. As stated earlier, final results of the stability tests indicated that the recommended armor-stone weights were adequate. At that point, additional tests were suggested to investigate reducing the armor weights in an effort to determine a smaller, adequate stone weight. Characteristics of the quarrying and stone-transporting operations indicated that economic benefits from such a reduction in stone size would be minimal; therefore, the proposed additional stone size optimization tests were not undertaken.

PART VI: RECOMMENDATIONS

55. Based on the physical model investigation documented herein, the following recommendations are provided:

a. Armor stones used in the rehabilitation plan should meet the following weight specifications:

Stone Classification	Minimum Weight tons	Average Weight tons	Maximum Weight tons
Select A-stone	26.5	29.0	None
A-stone	18.4	23.0	26.4
B-stone	12.0	16.0	18.3

The specific weight of all armor materials should not be less than 165.0 pcf. Parallelepiped-shaped stone, with their longest dimension between two and three times their least dimension, should be used in the prototype armor layers.

b. The stability of the proposed jetty rehabilitation is dependent on achieving successful stone placement. Detailed specifications concerning stone placement methods should be provided to the contractor. Armoring over the entire rehabilitated section should be a two-layer matrix of stones. All armor stones should be placed with their long axes perpendicular to the jetty slopes. Whenever possible, the outer layer of armor stones should be placed with efforts to attain maximum surface contact between adjacent stones. When placing toe stones, efforts should again be made to arrange stones, as possible, with their long axes normal to the jetty slopes.

c. Although removal of the deteriorated jetty stone berm near sta 87+60 had no noticeable effect on jetty stability, its removal could improve the navigability of small vessels maneuvering near the north jetty in rough conditions. If excavation of deteriorated jetty stones is needed for slope reinforcement on the jetty's outer end, this berm area should be the primary source of material.

REFERENCES

Butler, H. L. 1980. "Evolution of a Numerical Model for Simulating Long Period Wave Behavior in Ocean-Estuarine Systems," Estuarine and Wetlands Processes with Emphasis on Modeling, Marine Science Series, Vol 2, Plenum Press, New York.

Cialone, M. A. 1986. "Yaquina Bay, Oregon, Tidal and Wave-Induced Currents Near the Jettied Inlet," Miscellaneous Paper CERC-86-14, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Cialone, M. A., and Simpson, D. P. 1986. "Numerical Modeling of the Coastal Current Regime at Yaquina Bay, Oregon," Proceedings, Fifth Symposium on Coastal and Ocean Management, American Society of Civil Engineers, Vol 4, pp 3917-3931.

Corson, W. D., Abel, C. E., Brooks, R. M., Farrar, P. D., Groves, B. J., Jensen, R. E., Payne, J. B., Ragsdale, D. S., and Tracy, B. A. 1987. "Pacific Coast Hindcast Phase II Wave Information," WIS Report 16, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Creech, C. 1981. "Nearshore Wave Climatology Yaquina Bay, Oregon (1971-1981)," Ads 418, Oregon State University Sea Grant College Program, Corvallis, OR.

Dally, W. R., Dean, R. G., and Dalrymple, R. A. 1984. "Modeling Wave Transformation in the Surf Zone," Miscellaneous Paper CERC-84-8, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Debok, D. H., and Sollitt, C. K. "Large-Scale Model Study of Placed Stone Breakwaters," Ocean Engineering, Oregon State University, Corvallis, OR.

Ebersole, B. A., Cialone, M. A., and Prater, M. D. 1986. "Regional Coastal Processes Numerical Modeling System; Report I, RCPWAVE - A Linear Wave Propagation Model for Engineering Use," Technical Report CERC-86-4, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Hudson, R. Y. 1958. "Design of Quarry-Stone Cover Layers for Rubble-Mound Breakwaters," Research Report No. 2-2, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

__________. 1975. "Reliability of Rubble-Mound Breakwater Stability Models," Miscellaneous Paper HL-75-5, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Hughes, S. A. 1984. "The TMA Shallow-Water Spectrum Description and Applications," Technical Report CERC-84-7, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Hughes, S. A., and Jensen, R. E. 1986. "A User's Guide to SHALWV: Numerical Model for Simulation of Shallow-Water Wave Growth, Propagation, and Decay," Technical Report CERC-86-2, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Jensen, R. E., Vincent, C. L., and Abel, C. E. 1986. "A User's Guide to SHALWV: Numerical Model for Simulation of Shallow-Water Wave Growth, Propagation, and Decay, Report 2," Technical Report CERC-86-2, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Markle, D. G., and Davidson, D. D. 1979. "Placed-Stone Stability Tests Tillamook, Oregon," Technical Report HL-79-16, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Outlaw, D. G., and Briggs, M. J. 1986. "Directional Irregular Wave Generator for Shallow Water Basins," Presented at 21st American Towing Tank Conference, 5-7 Aug, Washington, DC.

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

US Army Engineer District, Buffalo. 1946. "Project Maps," Buffalo, NY.

US Army Engineer District, Portland. 1987 (Mar). "Yaquina Bay and Harbor, Oregon North Jetty Repair," Design Memorandum, Portland, OR.

US Army Engineer Waterways Experiment Station. 1963. "Stability of South Jetty Siuslaw River, Oregon," Technical Report No. 2-631, Vicksburg, MS.

Vemulakonda, S. A. 1984. "Erosion Control of Scour During Construction; Report 7, CURRENT - A Wave-Induced Current Model," US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Vincent, C. L., and Briggs, M. J. 1987. "Refraction - Diffraction of Irregular Waves Over a Mound," Journal, Waterway, Port, Coastal, and Ocean Engineering, American Society of Civil Engineers, New York.

Table 1

Hindcasted Deepwater Spectral Characteristics 1969 Storm

* Note - Date is divided into four two-digit sections which relate the year, month, day and Greenwich Mean hour, respectively.

* DATE	I	J	HS	PP	PD	MD	WS	WD	HFE
69121100	9	10	6.6	11.1	270.0	238.8	32	180	668.
69121103	9	10	7.1	12.5	270.0	232.9	37	181	707.
69121106	9	10	7.6	12.5	270.0	232.1	42	182	821.
69121109	9	10	8.1	12.5	247.5	227.5	44	180	846.
69121112	9	10	8.8	14.3	202.5	222.7	45	178	837.
69121115	9	10	9.8	14.3	202.5	218.2	45	181	803.
69121118	9	10	10.4	14.3	202.5	217.4	45	184	784.
69121121	9	10	10.5	14.3	202.5	218.5	43	188	744.
69121200	9	10	10.4	14.3	202.5	222.4	41	191	713.
69121203	9	10	10.3	14.3	202.5	225.3	38	196	662.
69121206	9	10	10.3	14.3	202.5	227.7	35	200	611.
69121209	9	10	10.2	14.3	202.5	230.2	34	199	596.
69121212	9	10	10.1	14.3	202.5	232.7	33	197	581.
69121215	9	10	10.0	14.3	270.0	235.6	31	199	549.
69121218	9	10	9.9	14.3	270.0	238.3	29	201	515.

LEGEND

HS = SIGNIFICANT WAVE HEIGHT (M)
PP = PEAK PERIOD BAND (SEC)
PD = PEAK DIRECTION BAND (DEG)
MD = WEIGHTED AVERAGE DIRECTION BAND (DEG)
WS = WIND SPEED (KNOTS)
WD = WIND DIRECTION (DEG)
HFE = HIGH FREQUENCY ENERGY (SQ CM)
DATE = DATE
I = I STATION LOCATION
J = J STATION LOCATION
WIS PACIFIC PII STATION 42
44.82 DEG N. 125.01 DEG W
AVE EN = AVERAGE 1-D SPECTRUM (SQ CM)
AVE DIR EN = AVERAGE 2-D SPECTRUM (SQ CM)
AHFE = AVERAGE HIGH FREQUENCY ENERGY (SQ CM)

AVE EN (SQ CM)	PER	FRQ	AVE DIRECTIONAL EN (SQ CM) 0.0	22.5	45.0	67.5	90.0	112.5	135.0	157.5	100.0	202.5	225.0	247.5	270.0	292.5	315.0	337.5
7.	33.3	0.030	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
71.	25.0	0.040	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
568.	20.0	0.050	0	0	0	0	0	0	0	0	0	0	21	6	539	0	0	0
7384.	16.7	0.060	0	0	0	0	0	0	0	0	110	505	1398	1925	3187	255	0	0
14731.	14.3	0.070	0	0	0	0	0	0	0	0	442	2205	3529	4293	3288	970	0	0
10383.	12.5	0.080	0	0	0	0	0	0	3	9	704	2809	2553	2159	1341	801	0	0
6205.	11.1	0.090	0	0	0	0	0	0	11	36	726	2184	1513	930	514	285	1	0
4301.	10.0	0.100	0	0	0	0	0	0	24	104	710	1749	939	447	223	99	1	0
3011.	9.1	0.110	0	0	0	0	0	1	34	160	630	1286	550	218	98	30	0	0
2119.	8.3	0.120	0	0	0	0	0	2	41	199	538	895	300	82	45	13	0	0
1600.	7.7	0.130	0	0	0	0	0	3	43	214	421	644	205	34	22	9	0	0
1212.	7.1	0.140	0	0	0	0	0	2	36	189	337	466	141	20	10	5	0	0
887.	6.7	0.150	0	0	0	0	0	2	30	154	251	321	100	16	6	3	0	0
655.	6.3	0.160	0	0	0	0	0	1	23	119	187	235	67	13	4	1	0	0
486.	5.9	0.170	0	0	0	0	0	1	19	92	140	169	49	10	2	1	0	0
365.	5.6	0.180	0	0	0	0	0	1	15	74	105	123	35	7	1	0	0	0
279.	5.3	0.190	0	0	0	0	0	2	15	59	73	87	30	8	0	0	0	0
216.	5.0	0.200	0	0	0	0	0	2	13	47	55	66	23	6	0	0	0	0
169.	4.8	0.210	0	0	0	0	0	1	10	37	42	51	19	5	0	0	0	0
134.	4.5	0.220	0	0	0	0	0	1	8	29	33	40	15	4	0	0	0	0

AHFE (SQ CM) = 696.

Table 2

Hindcasted Deepwater Spectral Characteristics 1970 Storm

DATE	I	J	HS	PP	PD	MD	WS	WD	HFE
70122918	9	10	6.5	12.5	292.5	266.0	21	246	442.
70122921	9	10	6.6	12.5	270.0	265.6	24	233	500.
70123000	9	10	6.7	12.5	270.0	264.7	27	220	560.
70123003	9	10	7.0	12.5	270.0	262.7	32	214	651.
70123006	9	10	7.6	12.5	270.0	255.4	37	208	731.
70123009	9	10	8.2	12.5	270.0	247.2	37	213	707.
70123012	9	10	8.7	12.5	270.0	245.8	37	217	691.
70123015	9	10	9.7	12.5	247.5	245.3	37	218	11303.
70123018	9	10	8.8	12.5	247.5	243.7	36	219	669.
70123021	9	10	8.5	12.5	247.5	244.2	32	228	605.
70123100	9	10	8.0	12.5	247.5	246.5	27	238	522.
70123103	9	10	7.7	12.5	247.5	249.7	25	248	484.
70123106	9	10	7.4	14.3	270.0	252.2	22	258	436.
70123109	9	10	7.3	14.3	270.0	254.4	21	269	421.
70123112	9	10	7.1	14.3	270.0	256.3	20	279	410.

LEGEND

HS = SIGNIFICANT WAVE HEIGHT (M)
PP = PEAK PERIOD BAND (SEC)
PD = PEAK DIRECTION BAND (DEG)
MD = WEIGHTED AVERAGE DIRECTION BAND (DEG)
WS = WIND SPEED (KNOTS)
WD = WIND DIRECTION (DEG)
HFE = HIGH FREQUENCY ENERGY (SQ CM)
DATE = DATE
I = I STATION LOCATION
J = J STATION LOCATION
WIS PACIFIC PII STATION 42
44.82 DEG N. 125.01 DEG W
AVE EN = AVERAGE 1-D SPECTRUM (SQ CM)
AVE DIR EN = AVERAGE 2-D SPECTRUM (SQ CM)
AHFE = AVERAGE HIGH FREQUENCY ENERGY (SQ CM)

AVE EN (SQ CM)	PER	FRQ	AVE DIRECTIONAL EN (SQ CM) 0.0	22.5	45.0	67.5	90.0	112.5	135.0	157.5	180.0	202.5	225.0	247.5	270.0	292.5	315.0	337.5
0.	33.3	0.030	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3.	25.0	0.040	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
294.	20.0	0.050	0	0	0	0	0	0	0	0	0	0	0	0	294	0	0	0
1337.	16.7	0.060	0	0	0	0	0	0	0	0	0	0	14	115	371	836	0	0
6349.	14.3	0.070	0	0	0	0	0	0	0	0	4	46	265	1462	3031	1537	1	0
8581.	12.5	0.080	0	0	0	0	0	0	0	0	26	268	756	2126	3305	2064	31	0
5908.	11.1	0.090	0	0	0	0	0	0	0	3	61	528	1045	1629	1604	984	49	0
4251.	10.0	0.100	0	0	0	0	0	0	0	9	92	620	1021	1154	845	467	39	0
2879.	9.1	0.110	0	0	0	0	0	0	0	12	109	588	792	747	400	202	25	0
1994.	8.3	0.120	0	0	0	0	0	0	0	13	119	532	585	447	196	84	13	0
1356.	7.7	0.130	0	0	0	0	0	0	0	8	90	387	423	289	111	37	6	0
962.	7.1	0.140	0	0	0	0	0	0	0	7	73	289	297	193	75	21	2	0
694.	6.7	0.150	0	0	0	0	0	0	0	6	57	218	215	120	54	17	2	0
508.	6.3	0.160	0	0	0	0	0	0	0	5	46	166	152	76	39	16	3	0
378.	5.9	0.170	0	0	0	0	0	0	0	5	34	121	112	56	31	13	3	0
286.	5.6	0.180	0	0	0	0	0	0	0	4	25	89	82	42	26	12	2	0
219.	5.3	0.190	0	0	0	0	0	0	0	7	20	52	56	38	24	12	4	0
170.	5.0	0.200	0	0	0	0	0	0	0	6	16	39	42	29	20	11	3	0
133.	4.8	0.210	0	0	0	0	0	0	0	5	12	30	32	23	16	9	3	0
106.	4.5	0.220	0	0	0	0	0	0	0	4	10	24	26	18	12	6	2	0

AHFE (SQ CM) = 1275.

Table 3

Hindcasted Deepwater Spectral Characteristics 1972 Storm

DATE	I	J	HS	PP	PD	MD	WS	WD	HFE
72012000	9	10	7.0	11.1	270.0	240.8	29	216	593.
72012003	9	10	7.2	11.1	270.0	238.9	30	216	603.
72012006	9	10	7.5	11.1	270.0	236.5	31	216	614.
72012009	9	10	7.8	11.1	270.0	235.2	32	218	624.
72012012	9	10	8.1	12.5	202.5	234.4	33	219	633.
72012015	9	10	9.0	12.5	225.0	233.1	37	220	680.
72012018	9	10	10.1	14.3	225.0	232.3	40	220	705.
72012021	9	10	10.3	14.3	225.0	231.7	40	227	698.
72012100	9	10	10.2	14.3	225.0	231.9	39	234	683.
72012103	9	10	9.9	14.3	247.5	233.0	36	235	637.
72012106	9	10	9.5	14.3	247.5	235.4	33	237	594.
72012109	9	10	9.1	14.3	247.5	237.5	29	246	533.
72012112	9	10	8.8	14.3	247.5	240.3	26	256	484.
72012115	9	10	8.5	14.3	247.5	242.4	24	256	446.
72012118	9	10	8.3	14.3	247.5	244.3	23	256	416.

LEGEND

HS = SIGNIFICANT WAVE HEIGHT (M)
PP = PEAK PERIOD BAND (SEC)
PD = PEAK DIRECTION BAND (DEG)
MD = WEIGHTED AVERAGE DIRECTION BAND (DEG)
WS = WIND SPEED (KNOTS)
WD = WIND DIRECTION (DEG)
HFE = HIGH FREQUENCY ENERGY (SQ CM)
DATE = DATE
I = I STATION LOCATION
J = J STATION LOCATION
WIS PACIFIC PII STATION 42
44.82 DEG N. 125.01 DEG W
AVE EN = AVERAGE 1-D SPECTRUM (SQ CM)
AVE DIR EN = AVERAGE 2-D SPECTRUM (SQ CM)
AHFE = AVERAGE HIGH FREQUENCY ENERGY (SQ CM)

AVE EN (SQ CM)	PER	FRQ	AVE DIRECTIONAL EN (SQ CM) 0.0	22.5	45.0	67.5	90.0	112.5	135.0	157.5	180.0	202.5	225.0	247.5	270.0	292.5	315.0	337.5
1.	33.3	0.030	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9.	25.0	0.040	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
379.	20.0	0.050	0	0	0	0	0	0	0	0	0	0	0	0	378	0	0	0
3392.	16.7	0.060	0	0	0	0	0	0	0	0	21	198	164	451	2555	0	0	0
10281.	14.3	0.070	0	0	0	0	0	0	0	0	174	1455	1595	2916	4101	32	2	0
10770.	12.5	0.080	0	0	0	0	0	0	0	2	304	2218	2683	3397	1932	198	31	0
7702.	11.1	0.090	0	0	0	0	0	0	0	6	289	1874	2261	2349	734	150	34	0
4990.	10.0	0.100	0	0	0	0	0	0	0	9	230	1369	1548	1351	369	87	23	0
3254.	9.1	0.110	0	0	0	0	0	0	0	10	171	903	1019	836	240	57	15	0
2159.	8.3	0.120	0	0	0	0	0	0	0	9	159	648	642	494	154	38	9	0
1474.	7.7	0.130	0	0	0	0	0	0	0	6	119	466	426	307	108	33	5	0
1029.	7.1	0.140	0	0	0	0	0	0	0	1	82	338	296	202	79	23	4	0
735.	6.7	0.150	0	0	0	0	0	0	0	0	60	245	213	139	57	16	2	0
536.	6.3	0.160	0	0	0	0	0	0	0	0	41	174	153	100	46	15	2	0
397.	5.9	0.170	0	0	0	0	0	0	0	0	28	127	113	75	37	11	1	0
300.	5.6	0.180	0	0	0	0	0	0	0	0	20	95	85	57	29	9	1	0
229.	5.3	0.190	0	0	0	0	0	0	0	4	18	61	62	47	25	7	1	0
178.	5.0	0.200	0	0	0	0	0	0	0	3	13	46	48	37	20	6	0	0
139.	4.8	0.210	0	0	0	0	0	0	0	3	10	35	37	29	16	4	0	0
111.	4.5	0.220	0	0	0	0	0	0	0	2	8	28	30	23	13	3	0	0

AHFE (SQ CM) = 596.

END OF 52

Table 4

Hindcasted Deepwater Spectral Characteristics 1973 Storm

DATE	I	J	HS	PP	PD	MD	WS	WD	HFE
73121203	9	10	7.5	11.1	225.0	233.1	34	221	674.
73121206	9	10	7.7	12.5	225.0	234.1	33	217	647.
73121209	9	10	8.3	12.5	202.5	232.3	36	200	686.
73121212	9	10	8.8	12.5	202.5	229.1	38	184	705.
73121215	9	10	9.0	14.3	202.5	228.1	37	182	680.
73121218	9	10	9.3	14.3	202.5	227.7	37	180	671.
73121221	9	10	9.6	14.3	202.5	229.9	34	186	3256.
73121300	9	10	9.8	14.3	202.5	233.7	31	193	3418.
73121303	9	10	9.6	14.3	202.5	238.9	28	205	496.
73121306	9	10	9.5	14.3	292.5	243.9	24	217	432.
73121309	9	10	9.4	14.3	292.5	247.1	25	219	455.
73121312	9	10	9.4	14.3	292.5	249.9	26	220	471.
73121315	9	10	9.5	14.3	292.5	252.2	26	213	471.
73121318	9	10	9.6	16.7	292.5	254.1	26	206	469.
73121321	9	10	9.7	16.7	292.5	256.5	25	210	449.

LEGEND

HS = SIGNIFICANT WAVE HEIGHT (M)
PP = PEAK PERIOD BAND (SEC)
PD = PEAK DIRECTION BAND (DEG)
MD = WEIGHTED AVERAGE DIRECTION BAND (DEG)
WS = WIND SPEED (KNOTS)
WD = WIND DIRECTION (DEG)
HFE = HIGH FREQUENCY ENERGY (SQ CM)
DATE = DATE
I = I STATION LOCATION
J = J STATION LOCATION
WIS PACIFIC PII STATION 42
44.82 DEG N, 125.01 DEG W
AVE EN = AVERAGE 1-D SPECTRUM (SQ CM)
AVE DIR EN = AVERAGE 2-D SPECTRUM (SQ CM)
AHFE = AVERAGE HIGH FREQUENCY ENERGY (SQ CM)

AVE EN	PER	FRQ	AVE DIRECTIONAL EN (SQ CM)															
(SQ CM)			0.0	22.5	45.0	67.5	90.0	112.5	135.0	157.5	180.0	202.5	225.0	247.5	270.0	292.5	315.0	337.5
3.	33.3	0.030	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
30.	25.0	0.040	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
426.	20.0	0.050	0	0	0	0	0	0	0	0	0	0	0	0	83	0	343	0
7657.	16.7	0.060	0	0	0	0	0	0	0	0	23	176	328	381	409	6333	3	0
11747.	14.3	0.070	0	0	0	0	0	0	0	0	227	1660	2362	2963	1221	3309	2	0
11503.	12.5	0.080	0	0	0	0	0	0	0	0	389	2698	2870	2668	1459	1415	1	0
7185.	11.1	0.090	0	0	0	0	0	0	0	2	402	2271	1953	1437	710	405	0	0
3807.	10.0	0.100	0	0	0	0	0	0	4	13	437	1487	947	605	231	79	0	0
2370.	9.1	0.110	0	0	0	0	0	0	6	32	386	1035	537	287	64	20	0	0
1700.	8.3	0.120	0	0	0	0	0	0	8	57	299	687	369	227	44	6	0	0
1307.	7.7	0.130	0	0	0	0	0	0	10	74	249	506	270	157	33	3	0	0
969.	7.1	0.140	0	0	0	0	0	0	8	58	194	370	197	108	26	4	0	0
696.	6.7	0.150	0	0	0	0	0	0	8	50	137	254	144	76	21	2	0	0
513.	6.3	0.160	0	0	0	0	0	0	7	44	104	187	100	49	15	2	0	0
382.	5.9	0.170	0	0	0	0	0	0	6	37	79	135	71	36	12	3	0	0
288.	5.6	0.180	0	0	0	0	0	0	5	28	59	99	53	28	10	2	0	0
220.	5.3	0.190	0	0	0	0	0	1	7	24	42	60	45	27	9	1	0	0
171.	5.0	0.200	0	0	0	0	0	1	6	19	32	45	35	21	7	1	0	0
134.	4.8	0.210	0	0	0	0	0	0	4	15	25	35	27	17	5	1	0	0
106.	4.5	0.220	0	0	0	0	0	0	3	12	20	28	21	13	4	0	0	0

AHFE (SQ CM) = 932.

Table 5

Hindcasted Deepwater Spectral Characteristics 1974 Storm

DATE	I	J	HS	PP	PD	MD	WS	WD	HFE
74011421	9	10	8.3	14.3	202.5	218.4	32	216	593.
74011500	9	10	8.4	14.3	202.5	216.7	32	204	605.
74011503	9	10	8.8	14.3	202.5	214.8	34	197	633.
74011506	9	10	9.1	14.3	202.5	213.7	37	191	677.
74011509	9	10	9.6	14.3	202.5	212.2	39	196	699.
74011512	9	10	10.1	16.7	202.5	211.3	40	202	704.
74011515	9	10	10.6	16.7	202.5	210.4	39	205	673.
74011518	9	10	10.9	16.7	202.5	210.4	38	208	649.
74011521	9	10	10.8	16.7	202.5	211.5	35	212	600.
74011600	9	10	10.4	14.3	202.5	213.1	31	216	539.
74011603	9	10	10.1	14.3	202.5	213.7	30	210	528.
74011606	9	10	9.7	14.3	202.5	214.3	28	203	501.
74011609	9	10	9.5	14.3	202.5	214.8	29	196	524.
74011612	9	10	9.6	14.3	202.5	214.8	30	190	4661.
74011615	9	10	9.4	14.3	225.0	215.8	29	195	5581.

LEGEND

HS = SIGNIFICANT WAVE HEIGHT (M)
PP = PEAK PERIOD BAND (SEC)
PD = PEAK DIRECTION BAND (DEG)
MD = WEIGHTED AVERAGE DIRECTION BAND (DEG)
WS = WIND SPEED (KNOTS)
WD = WIND DIRECTION (DEG)
HFE = HIGH FREQUENCY ENERGY (SQ CM)
DATE = DATE
I = I STATION LOCATION
J = J STATION LOCATION
WIS PACIFIC PII STATION 42
44.82 DEG N, 125.01 DEG W
AVE EN = AVERAGE 1-D SPECTRUM (SQ CM)
AVE DIR EN = AVERAGE 2-D SPECTRUM (SQ CM)
AHFE = AVERAGE HIGH FREQUENCY ENERGY (SQ CM)

AVE EN	PER	FRQ	AVE DIRECTIONAL EN (SQ CM)															
(SQ CM)			0.0	22.5	45.0	67.5	90.0	112.5	135.0	157.5	180.0	202.5	225.0	247.5	270.0	292.5	315.0	337.5
2.	33.3	0.030	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
26.	25.0	0.040	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1360.	20.0	0.050	0	0	0	0	0	0	0	0	9	66	330	9	945	0	0	0
10317.	16.7	0.060	0	0	0	0	0	0	0	0	691	4486	4480	333	324	0	0	0
15347.	14.3	0.070	0	0	0	0	0	0	0	2	1088	7423	4697	1757	333	44	0	0
10149.	12.5	0.080	0	0	0	0	0	0	0	16	907	4868	2845	1221	170	117	0	0
6391.	11.1	0.090	0	0	0	0	0	0	1	39	761	3053	1771	630	75	56	0	0
4088.	10.0	0.100	0	0	0	0	0	0	3	59	643	1901	1022	381	48	29	0	0
2844.	9.1	0.110	0	0	0	0	0	0	4	65	496	1296	670	268	32	9	0	0
2012.	8.3	0.120	0	0	0	0	0	0	5	64	389	865	453	205	25	2	0	0
1439.	7.7	0.130	0	0	0	0	0	0	5	57	300	614	310	129	18	1	0	0
1060.	7.1	0.140	0	0	0	0	0	0	4	41	224	445	232	95	14	1	0	0
766.	6.7	0.150	0	0	0	0	0	0	3	31	163	317	171	65	11	1	0	0
557.	6.3	0.160	0	0	0	0	0	0	2	21	115	228	127	50	10	1	0	0
413.	5.9	0.170	0	0	0	0	0	0	1	15	84	167	96	37	8	1	0	0
311.	5.6	0.180	0	0	0	0	0	0	1	11	63	125	71	28	6	1	0	0
238.	5.3	0.190	0	0	0	0	0	0	5	24	48	78	52	22	5	1	0	0
184.	5.0	0.200	0	0	0	0	0	0	4	19	37	59	40	17	4	1	0	0
145.	4.8	0.210	0	0	0	0	0	0	3	15	29	46	31	13	3	1	0	0
115.	4.5	0.220	0	0	0	0	0	0	2	12	23	36	24	10	2	0	0	0

AHFE (SQ CM) = 1211.

Table 6

Hindcasted Deepwater Spectral Characteristics 1983 Storm

DATE	I	J	HS	PP	PD	MD	WS	WD	HFE
83012521	9	10	8.1	14.3	247.5	223.5	42	162	3017.
83012600	9	10	8.2	14.3	247.5	220.5	41	162	765.
83012603	9	10	8.5	14.3	247.5	218.8	37	168	678.
83012606	9	10	8.8	14.3	247.5	219.4	33	174	600.
83012609	9	10	8.9	14.3	247.5	219.2	31	164	559.
83012612	9	10	9.0	14.3	247.5	222.0	29	155	2679.
83012615	9	10	8.8	14.3	247.5	224.5	31	153	570.
83012618	9	10	9.1	16.7	247.5	227.0	33	152	3151.
83012621	9	10	8.9	16.7	247.5	227.4	35	162	642.
83012700	9	10	9.0	16.7	247.5	226.8	38	173	698.
83012703	9	10	9.0	16.7	247.5	228.3	29	179	522.
83012706	9	10	8.8	16.7	247.5	231.3	20	185	376.
83012709	9	10	8.5	16.7	247.5	233.5	18	216	343.
83012712	9	10	8.3	16.7	247.5	235.3	16	248	301.
83012715	9	10	8.1	16.7	247.5	236.8	16	264	301.

LEGEND

HS = SIGNIFICANT WAVE HEIGHT (M)
PP = PEAK PERIOD BAND (SEC)
PD = PEAK DIRECTION BAND (DEG)
MD = WEIGHTED AVERAGE DIRECTION BAND (DEG)
WS = WIND SPEED (KNOTS)
WD = WIND DIRECTION (DEG)
HFE = HIGH FREQUENCY ENERGY (SQ CM)
DATE = DATE
I = I STATION LOCATION
J = J STATION LOCATION
WIS PACIFIC PII STATION 42
44.82 DEG N. 125.01 DEG W
AVE EN = AVERAGE 1-D SPECTRUM (SQ CM)
AVE DIR EN = AVERAGE 2-D SPECTRUM (SQ CM)
AHFE = AVERAGE HIGH FREQUENCY ENERGY (SQ CM)

AVE EN (SQ CM)	PER	FRQ	AVE DIRECTIONAL EN (SQ CM) 0.0	22.5	45.0	67.5	90.0	112.5	135.0	157.5	180.0	202.5	225.0	247.5	270.0	292.5	315.0	337.5
2.	33.3	0.030	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
16.	25.0	0.040	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1466.	20.0	0.050	0	0	0	0	0	0	0	0	0	0	44	1310	111	0	0	0
11387.	16.7	0.060	0	0	0	0	0	0	0	0	29	121	733	8146	2357	0	0	0
11702.	14.3	0.070	0	0	0	0	0	0	0	0	278	1405	2086	6787	1142	0	0	0
6882.	12.5	0.080	0	0	0	0	0	0	1	0	619	2644	1857	1451	308	0	0	0
4505.	11.1	0.090	0	0	0	0	0	0	2	0	616	2200	1064	512	107	0	0	0
2633.	10.0	0.100	0	0	0	0	0	0	8	9	515	1328	533	200	35	0	0	0
1841.	9.1	0.110	0	0	0	0	0	2	19	31	437	925	295	107	22	0	0	0
1303.	8.3	0.120	0	0	0	0	0	4	38	74	394	576	148	53	12	0	0	0
981.	7.7	0.130	0	0	0	0	0	8	61	131	333	339	70	26	8	0	0	0
800.	7.1	0.140	0	0	0	0	0	10	64	152	262	239	52	14	3	0	0	0
612.	6.7	0.150	0	0	0	0	1	13	61	154	188	148	34	8	2	0	0	0
511.	6.3	0.160	0	0	0	0	1	13	52	138	142	123	31	6	1	0	0	0
385.	5.9	0.170	0	0	0	0	1	11	44	116	100	82	22	4	0	0	0	0
290.	5.6	0.180	0	0	0	0	0	8	34	92	75	58	16	2	1	0	0	0
222.	5.3	0.190	0	0	0	0	3	15	34	54	51	40	17	3	0	0	0	0
173.	5.0	0.200	0	0	0	0	2	12	26	42	39	31	14	2	0	0	0	0
136.	4.8	0.210	0	0	0	0	2	9	21	33	30	23	11	3	1	0	0	0
109.	4.5	0.220	0	0	0	0	1	8	17	26	22	17	9	3	1	0	0	0

AHFE (SQ CM) = 1013.

Table 7

Worst Wave Conditions For Long Duration Stability Tests

SWL (FT. MLLW)	DIRECTION	YEAR OF STORM	GAIN (%)	HMO * (FT)	TP * (SEC)
+10.0	WSW	1969	100	24.2	16.9
+10.0	W	1974	100	21.9	16.4
+10.0	WNW	1969	100	24.2	16.9
0.0	WSW	1969	100	21.0	16.9
0.0	W	1969	100	21.7	16.9
0.0	WNW	1974	100	18.4	16.8

* - CONDITIONS MEASURED AT THE WAVEBOARD

Table 8

Monochromatic Wave Conditions

CASE NO.	DEEPWATER CONDITIONS H (ft)	T (sec)	DIR (deg)	WAVEBOARD CONDITIONS H (ft)	T (sec)	DIR (deg)
1	14.0	11.0	292.5 WNW	13.4	11.0	287
2	14.0	14.0	292.5 WNW	14.5	14.0	286
3	14.0	16.0	292.5 WNW	15.5	16.0	286
4	14.0	11.0	270.0 W	14.5	11.0	273
5	14.0	14.0	270.0 W	15.5	14.0	277
6	14.0	16.0	270.0 W	13.3	16.0	278
7	14.0	11.0	247.5 WSW	13.9	11.0	259
8	14.0	14.0	247.5 WSW	16.5	14.0	263
9	14.0	16.0	247.5 WSW	15.5	16.0	265
10	10.0	11.0	292.5 WNW	9.6	11.0	287
11	10.0	14.0	292.5 WNW	10.3	14.0	286
12	10.0	16.0	292.5 WNW	11.1	16.0	286
13	10.0	11.0	270.0 W	10.4	11.0	273
14	10.0	14.0	270.0 W	11.1	14.0	277
15	10.0	16.0	270.0 W	9.5	16.0	278
16	10.0	11.0	247.5 WSW	9.9	11.0	259
17	10.0	14.0	247.5 WSW	11.8	14.0	263
18	10.0	16.0	247.5 WSW	11.1	16.0	265

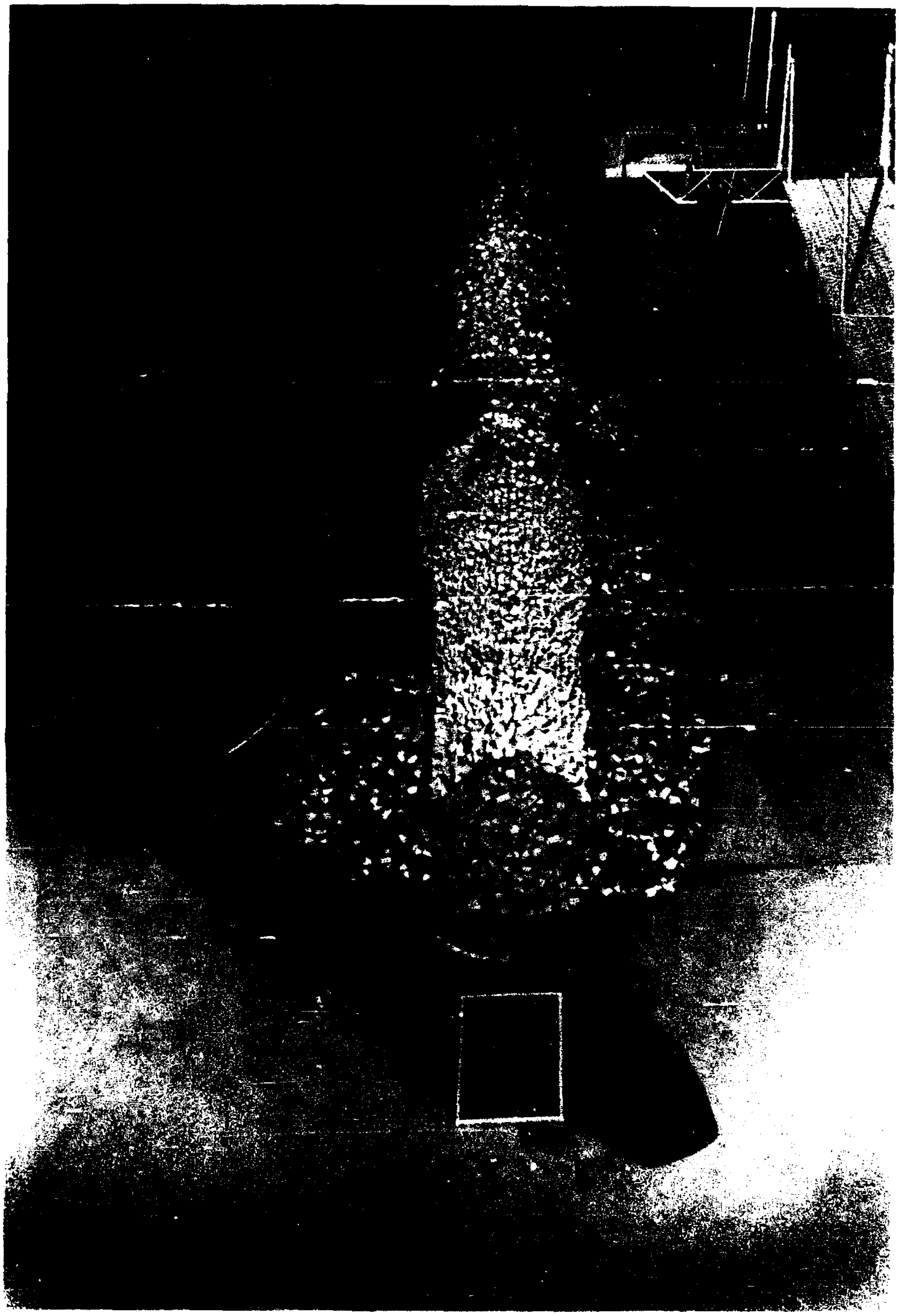


Photo 1. Overhead view of structure before testing with 1969 storm; swl = +10.0 ft; direction of wave attack = west-southwest

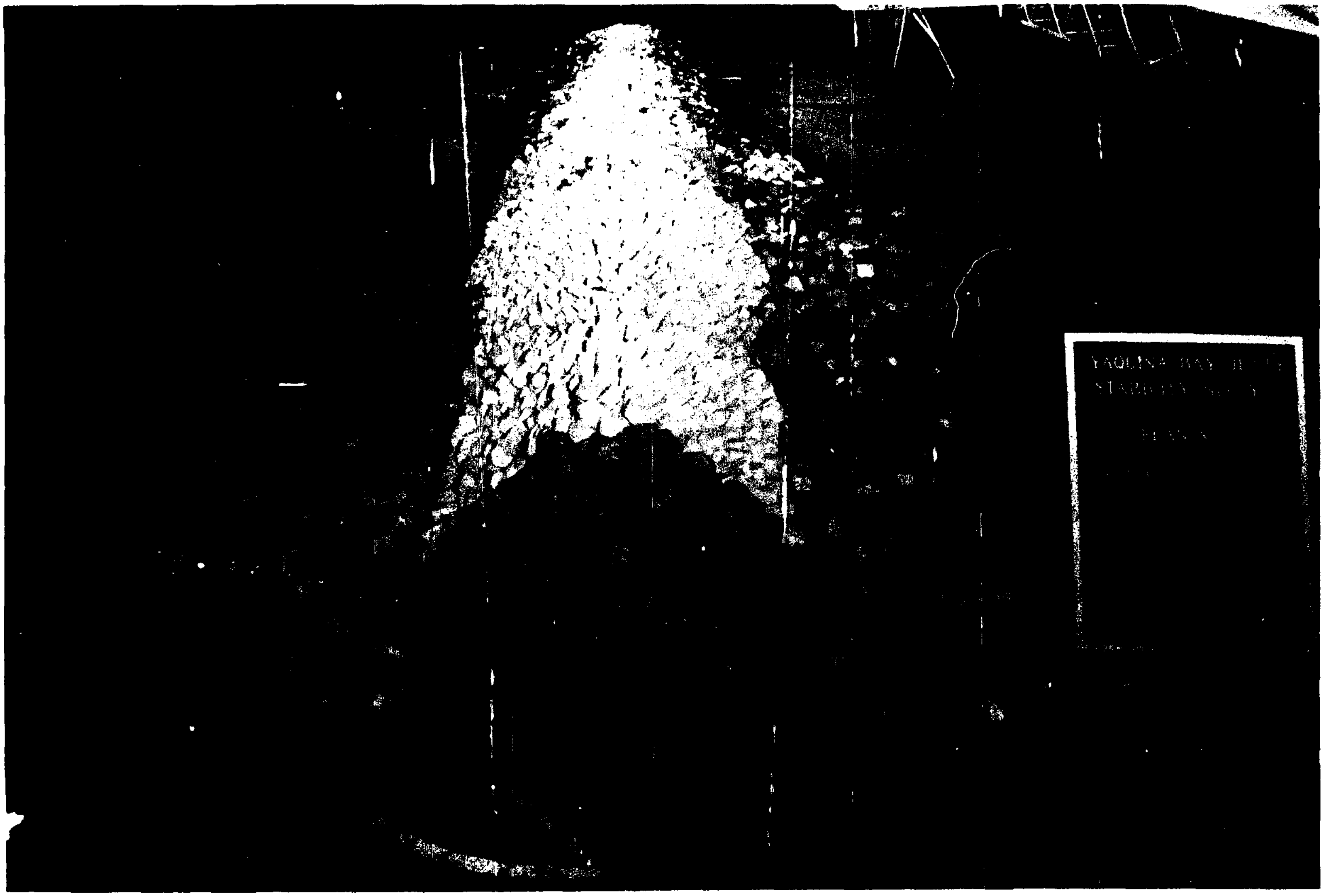


Photo 2. Long-axis view of structure before testing with 1969 storm; swl = +10.0 ft; direction of wave attack = west-southwest

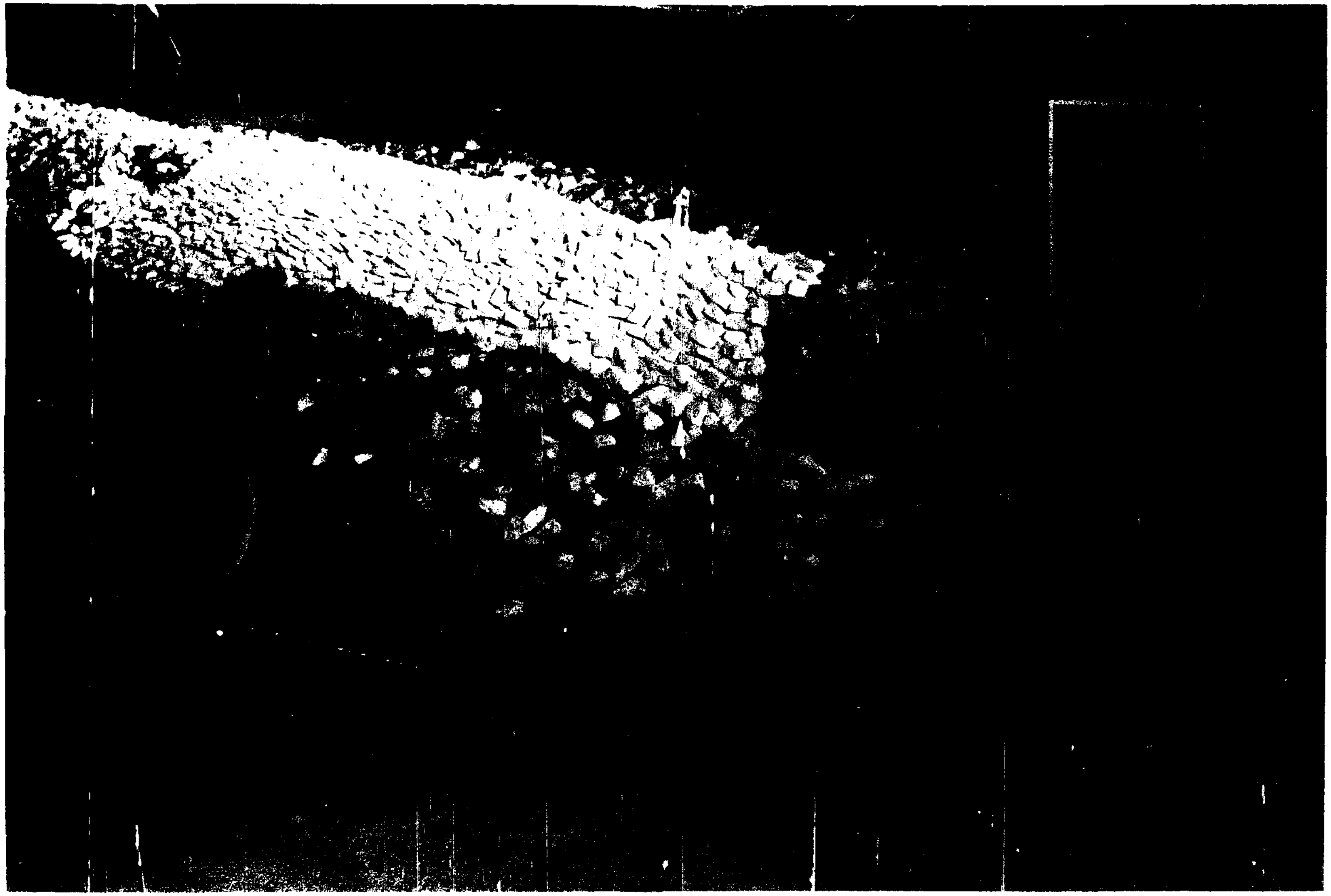


Photo 3. Sea-side view of structure before testing with 1969 storm; swl = +10.0 ft; direction of wave attack = west-southwest

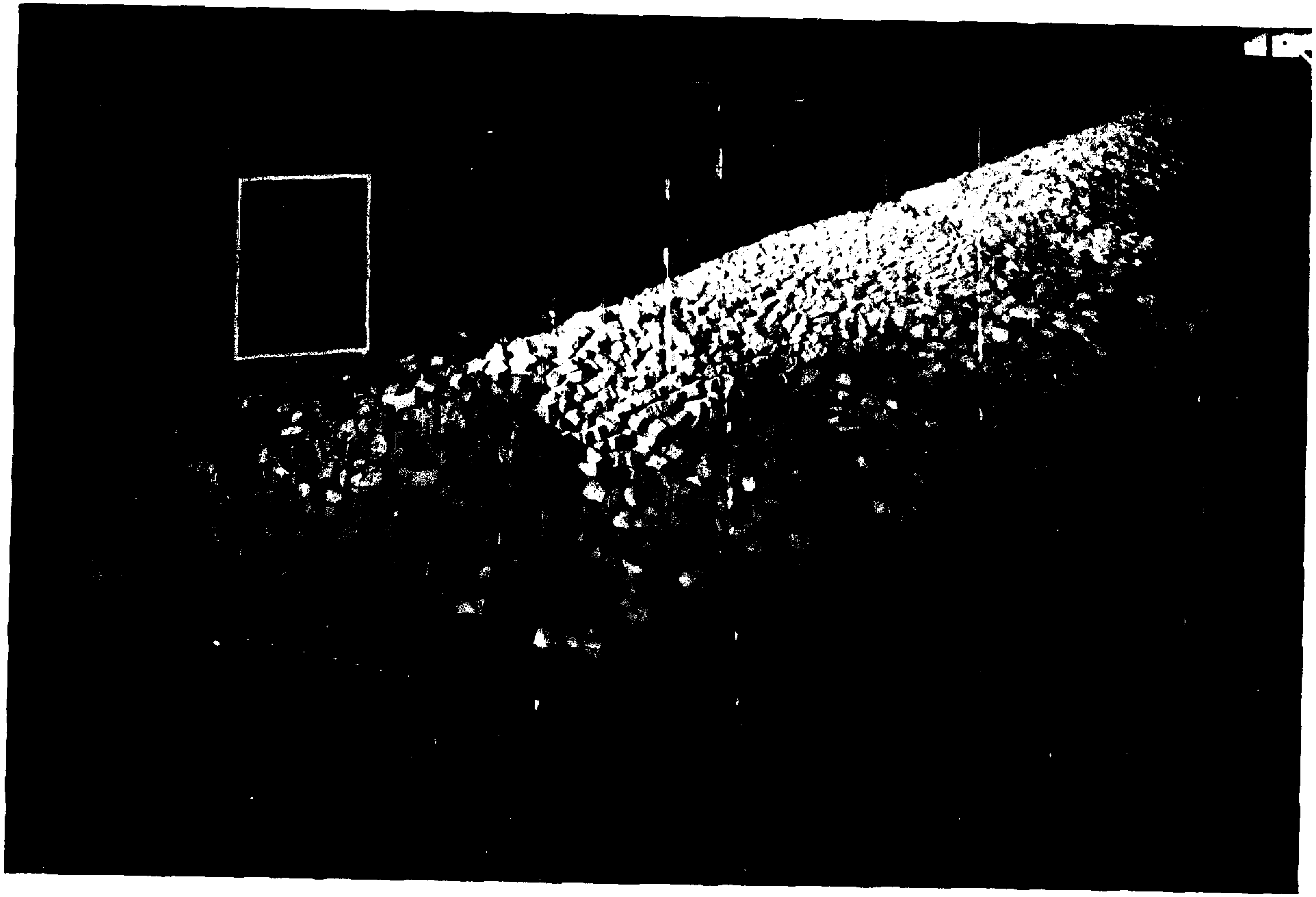


Photo 4. Channel-side view of structure before testing with 1969 storm; swl = +10.0 ft; direction of wave attack = west-southwest

Photo 5. Overhead view of structure after testing with 1969 and 1974 storms; swl = +10.0 ft; directions of wave attack = west-southwest and west

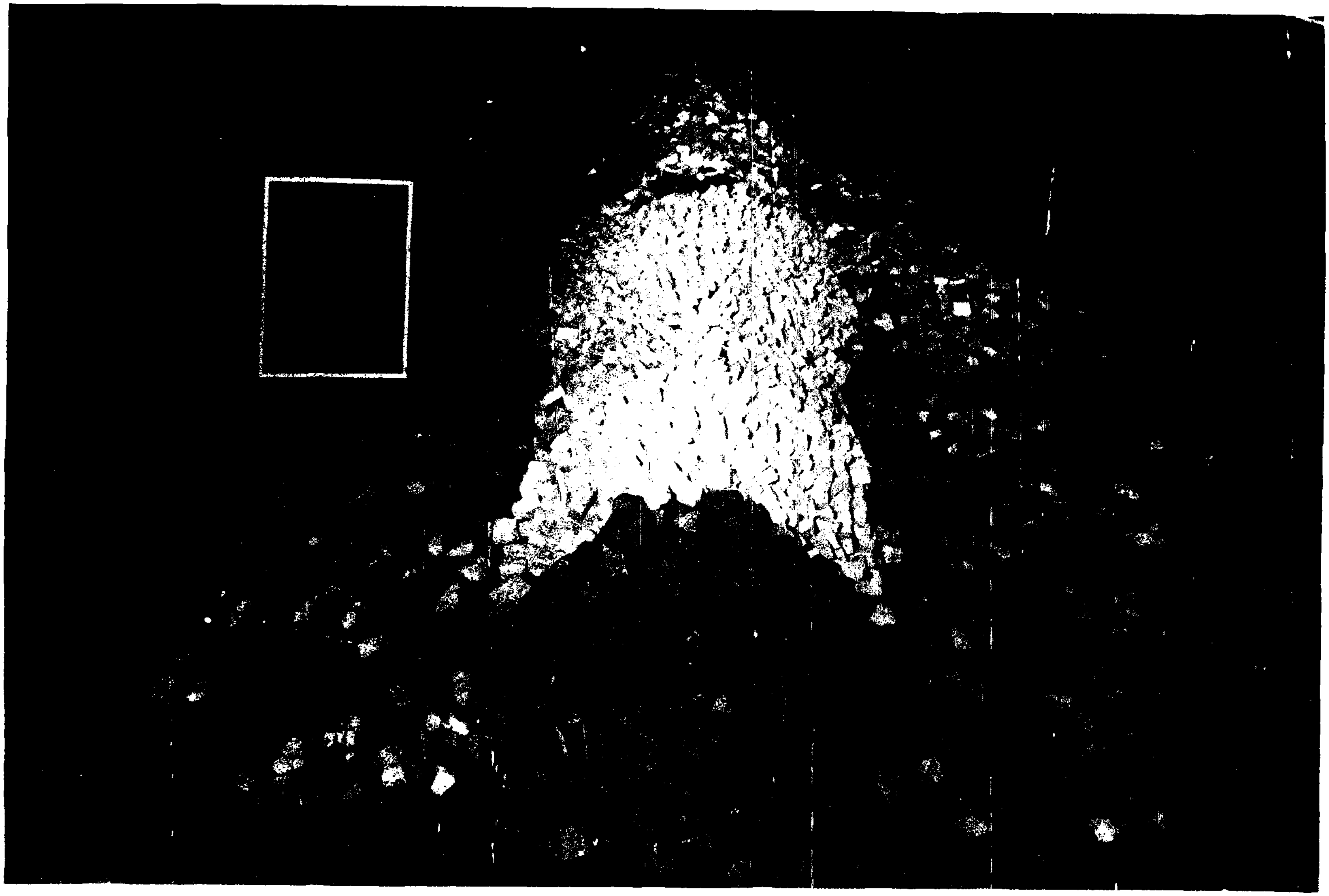


Photo 6. Long-axis view of structure after testing with 1969 and 1974 storms; swl = +10.0 ft; directions of wave attack = west-southwest and west

Photo 7. Sea-side view of structure after testing with 1969 and 1974 storms; swl = +10.0 ft; directions of wave attack = west-southwest and west

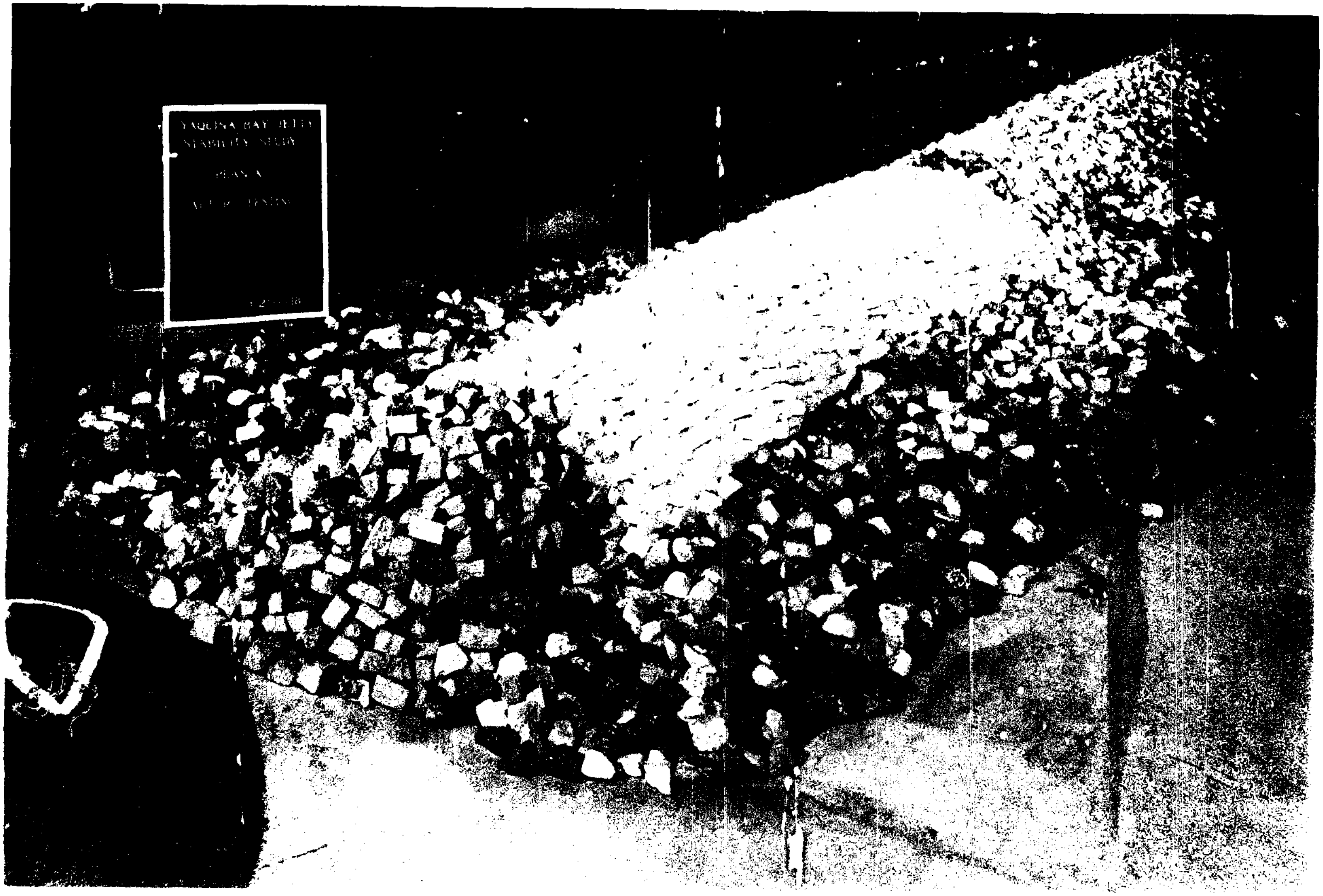


Photo 8. Channel-side view of structure after testing with 1969 and 1974 storms; swl = +10.0 ft; directions of wave attack = west-southwest and west

Photo 9. Overhead view of structure before testing with 1969 storm; swl = +10.0 ft; direction of wave attack = west-northwest

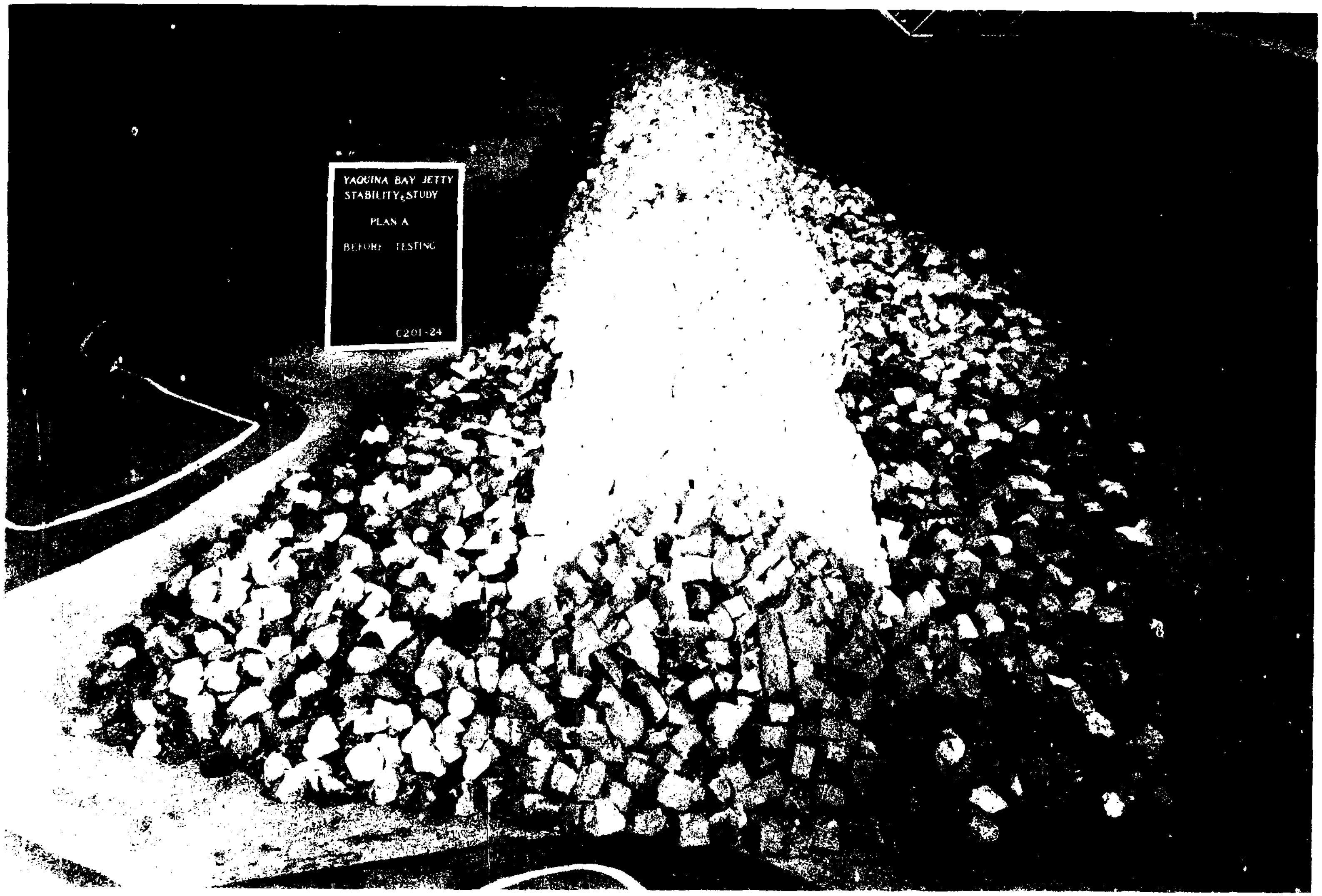


Photo 10. Long-axis view of structure before testing with 1969 storm; swl = +10.0 ft; direction of wave attack = west-northwest

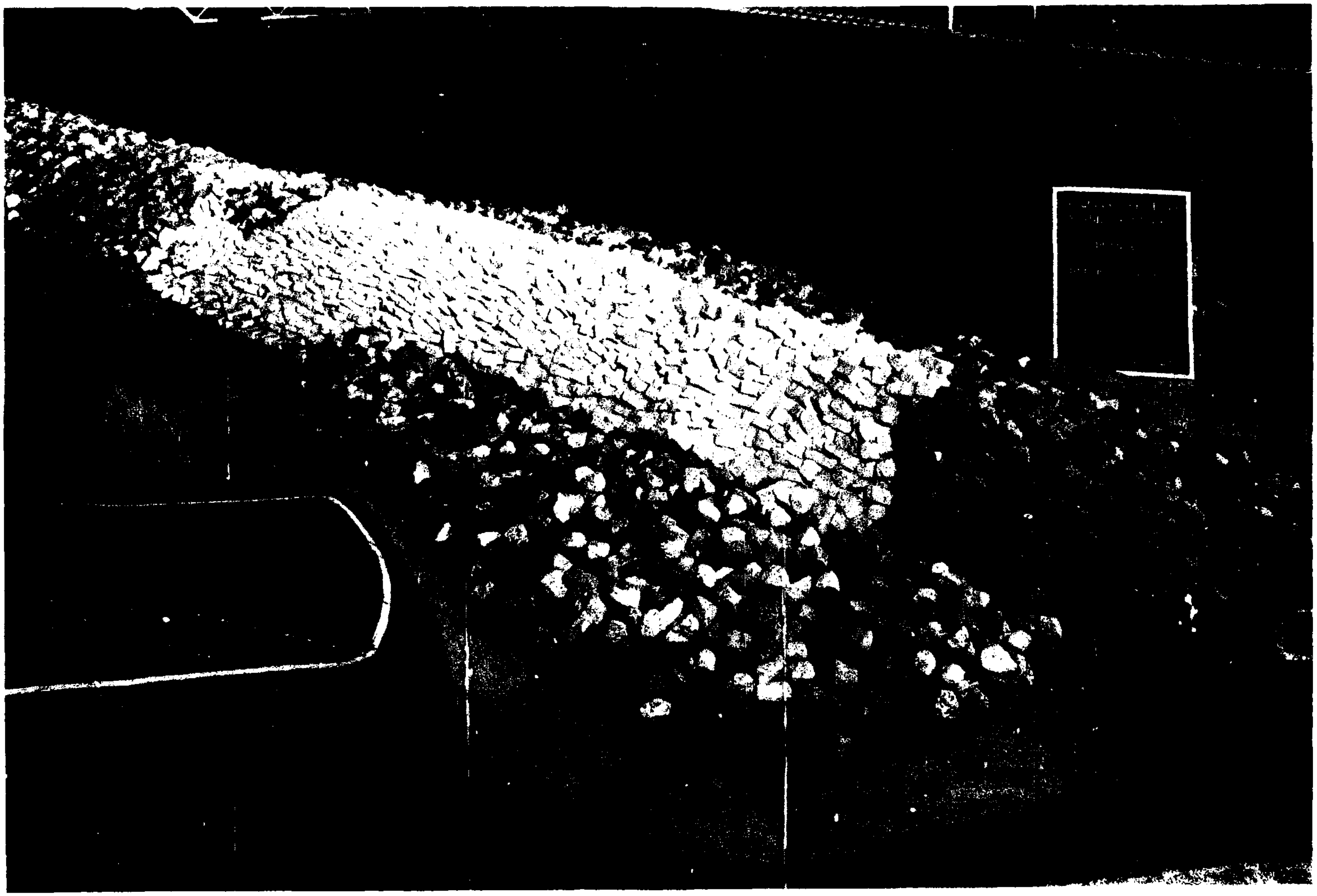


Photo 11. Sea-side view of structure before testing with 1969 storm; swl = +10.0 ft; direction of wave attack = west-northwest

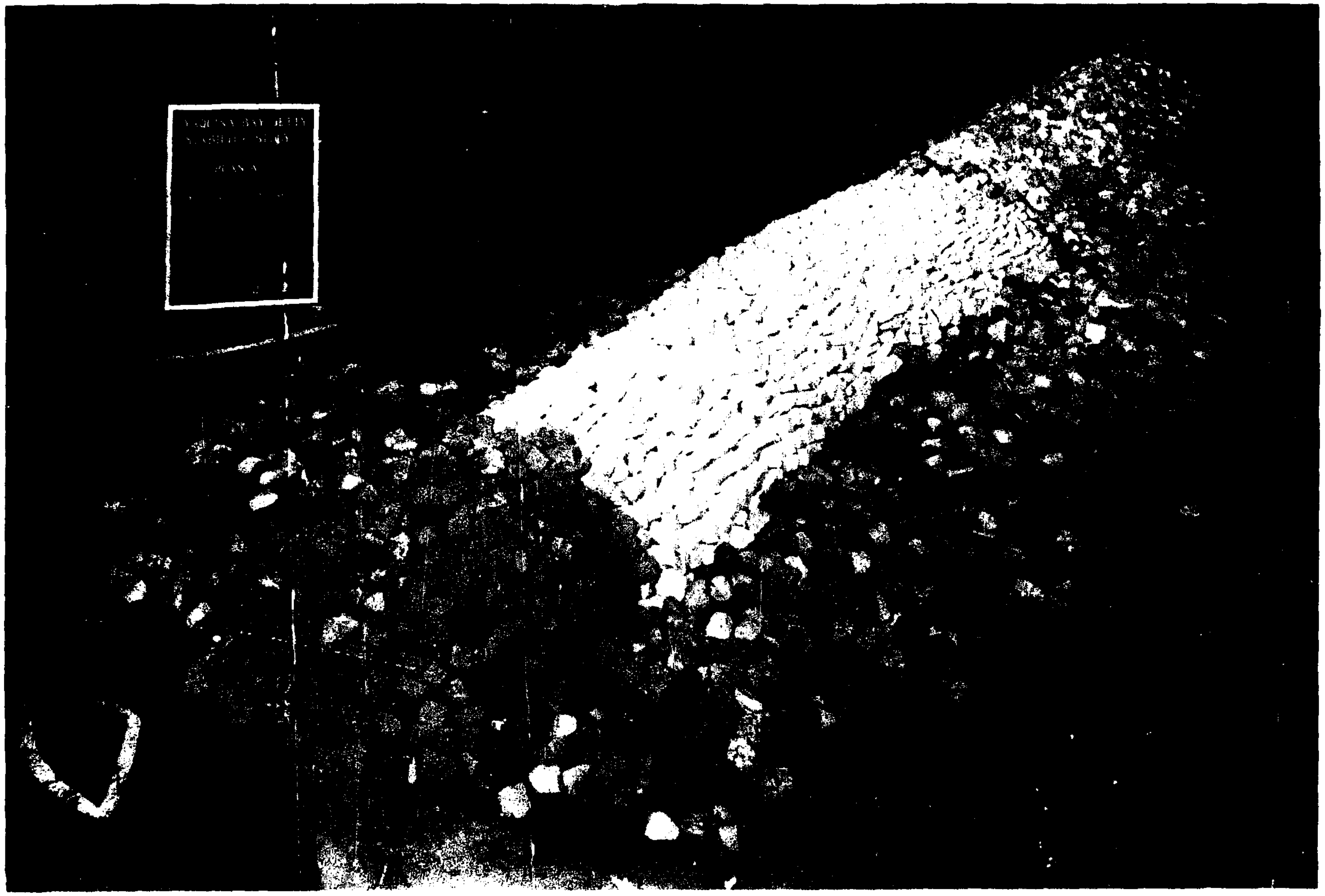


Photo 12. Channel-side view of structure before testing with 1969 storm; swl = +10.0 ft; direction of wave attack = west-northwest

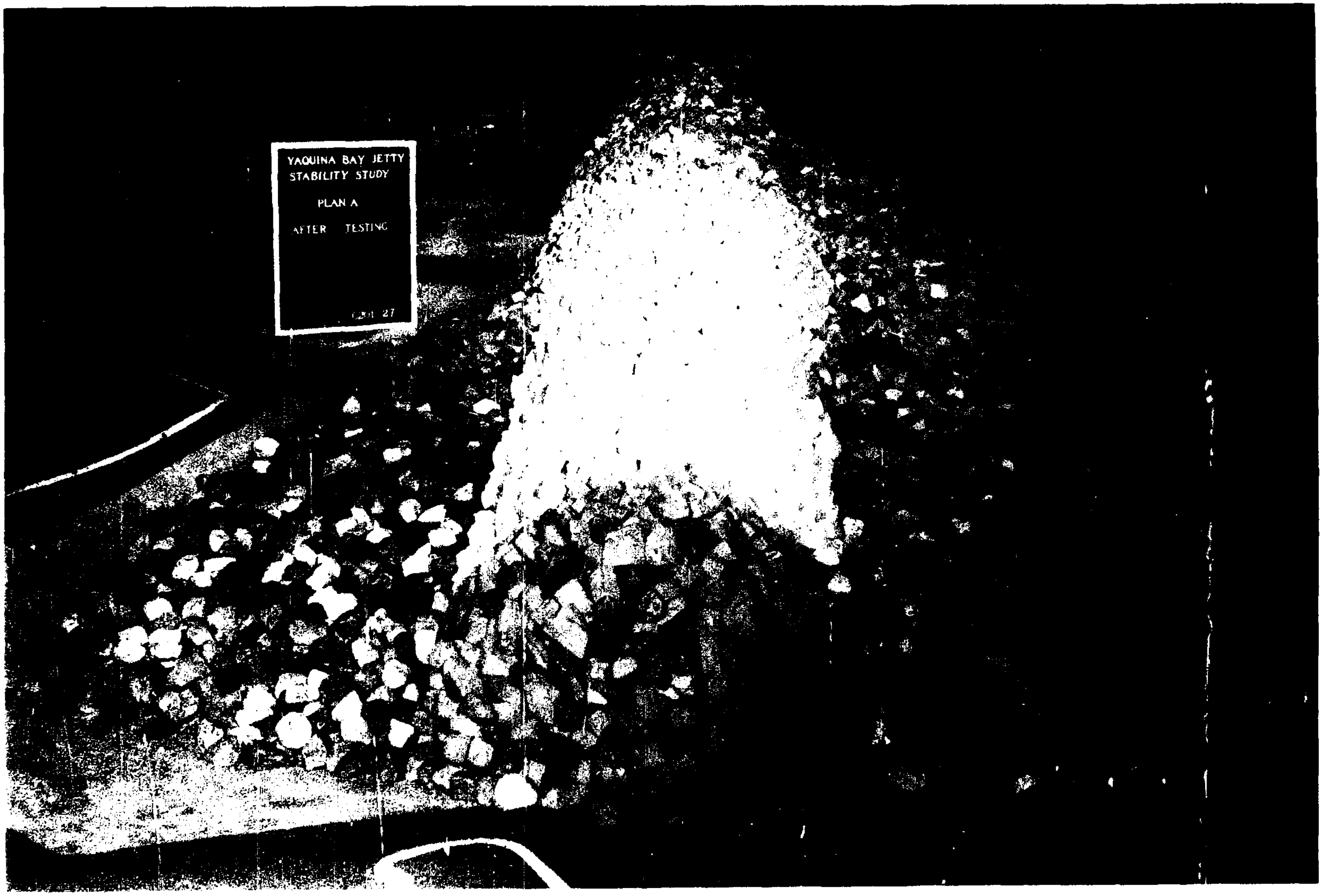


Photo 13. Long-axis view of structure after testing with 1969 and 1983 storms; swl = +10.0 ft; direction of wave attack = west-northwest

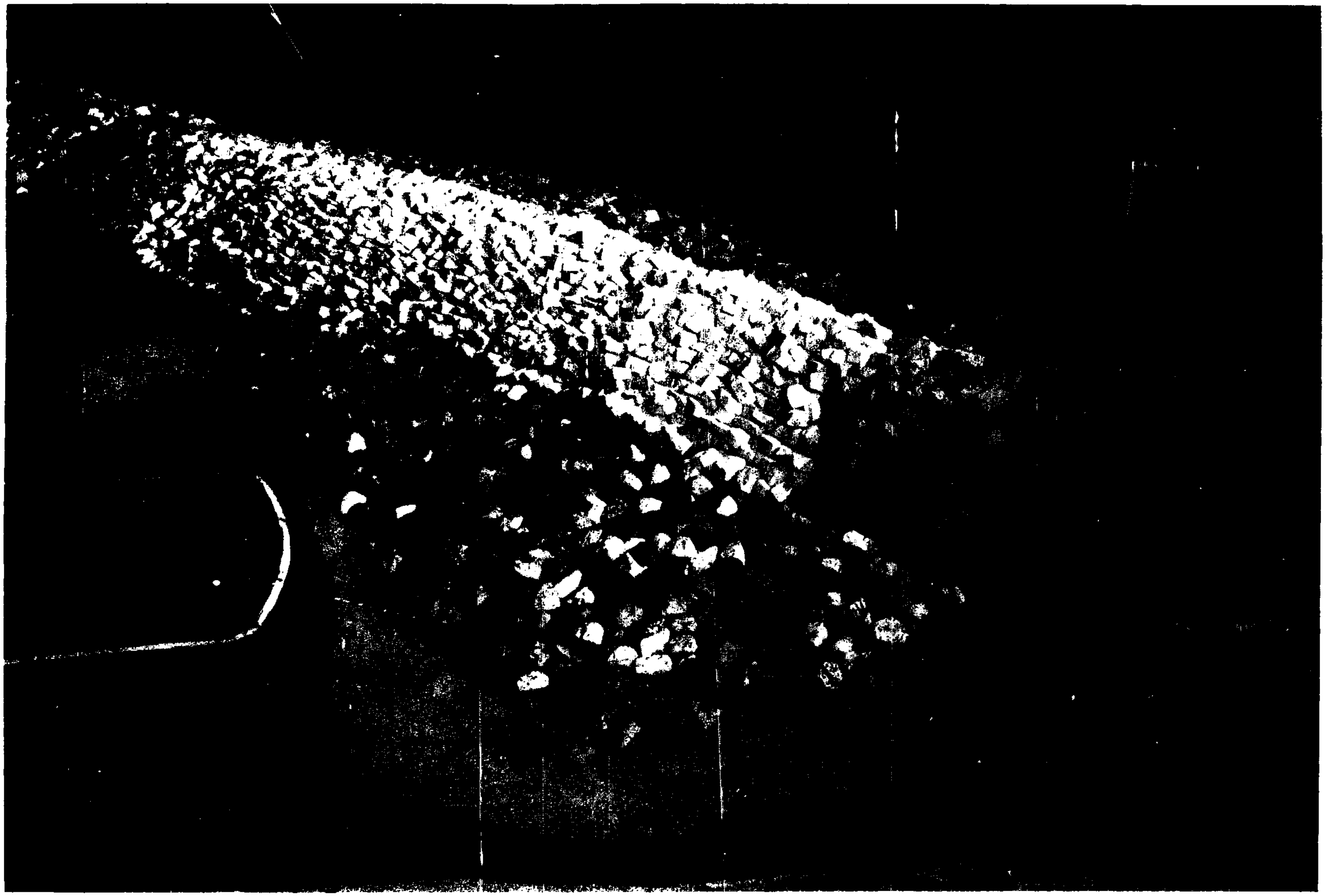


Photo 14. Sea-side view of structure after testing with 1969 and 1983 storms; swl = +10.0 ft; direction of wave attack = west-northwest

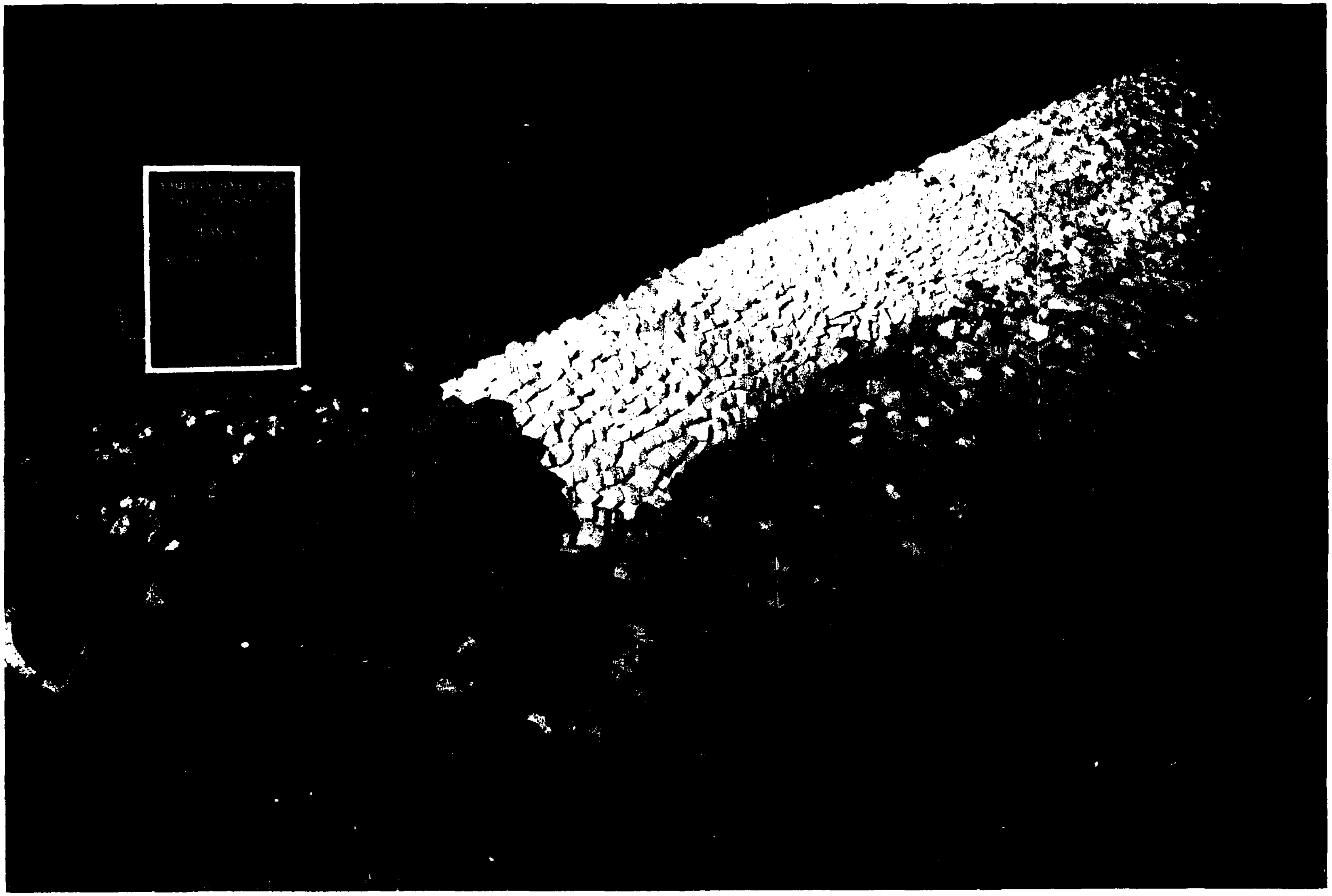


Photo 15. Channel-side view of structure after testing with 1969 and 1983 storms; swl = +10.0 ft; direction of wave attack = west-northwest

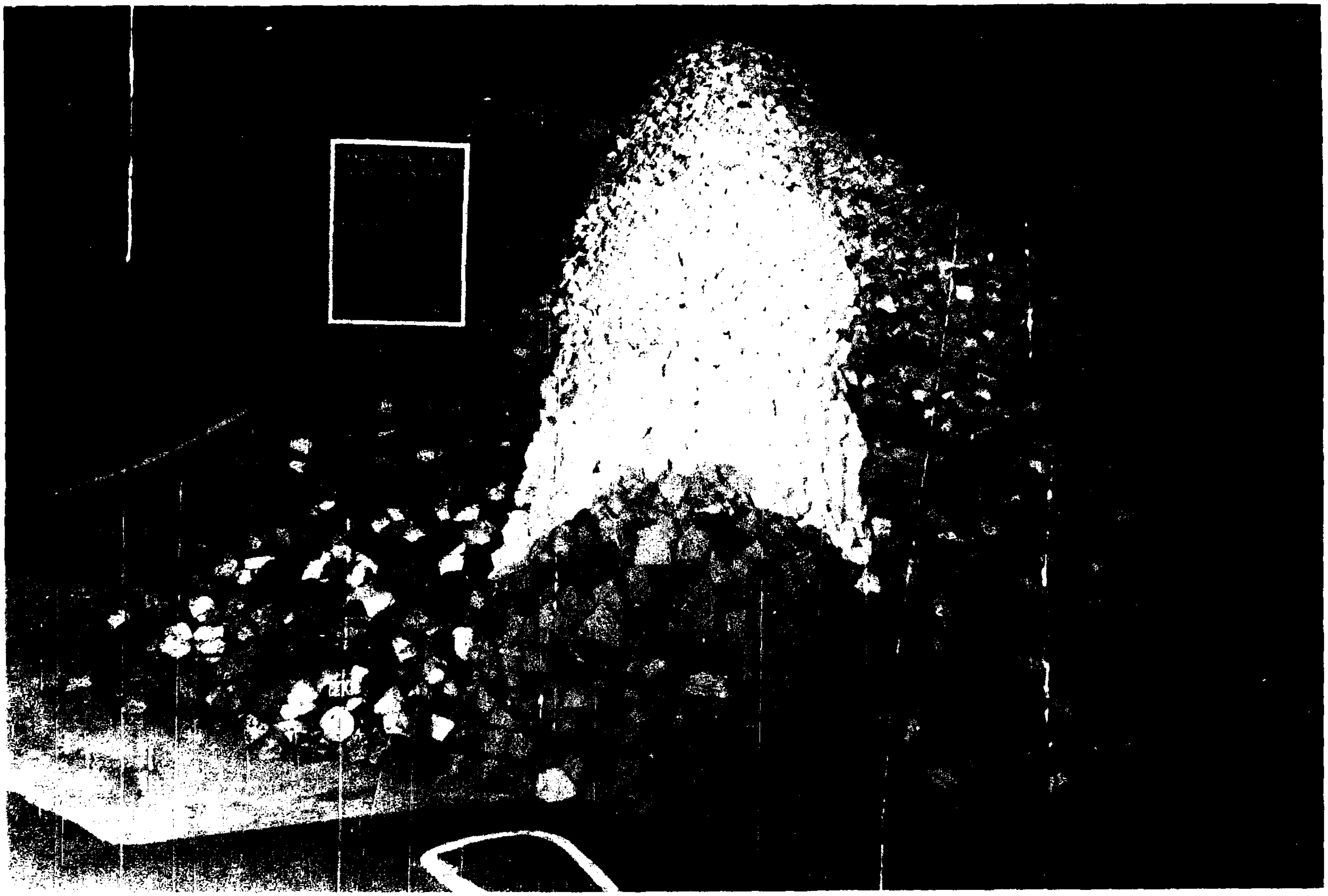


Photo 16. Long-axis view of structure before testing with 1969 storm; swl = 0.0 ft; direction of wave attack = west-southwest

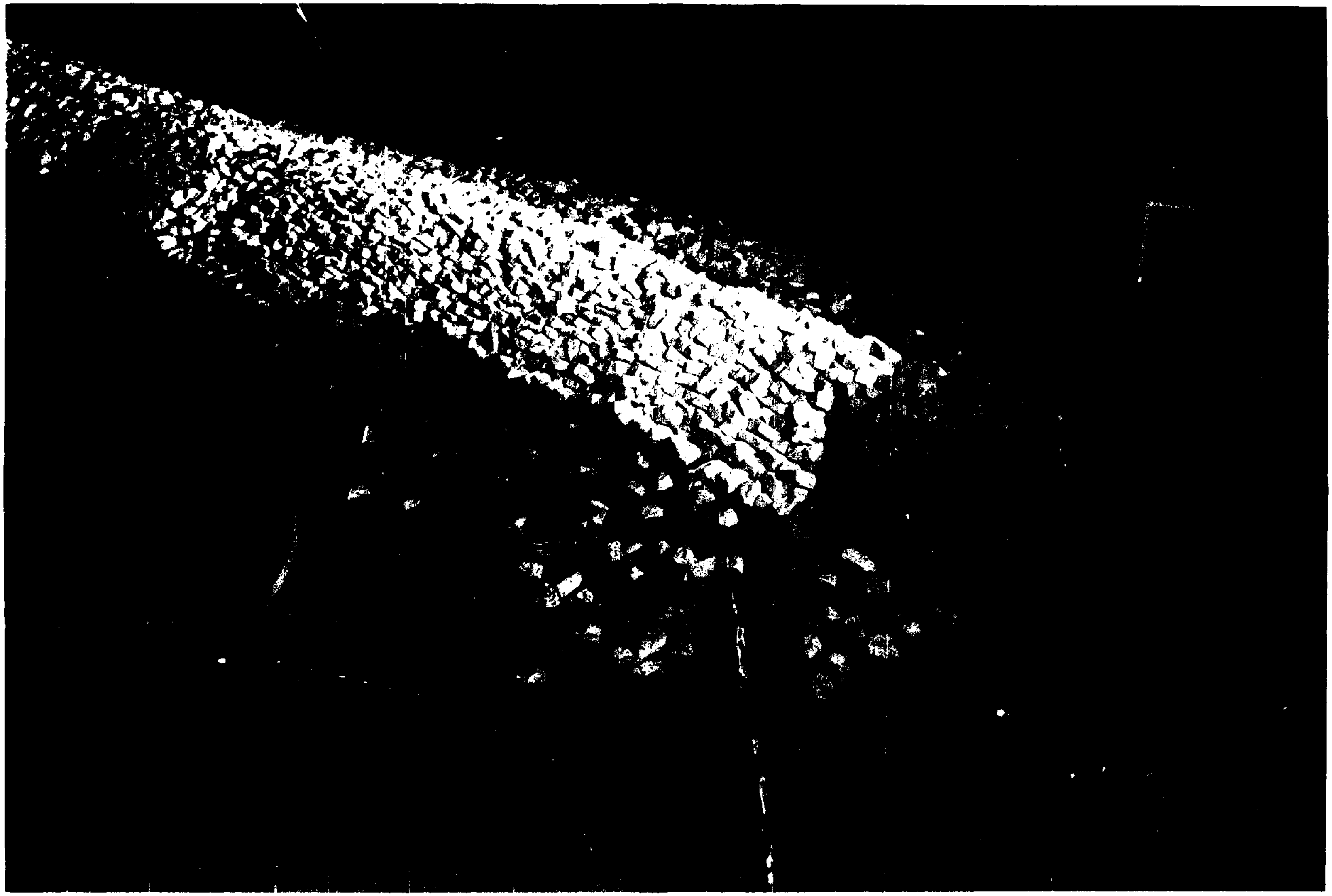


Photo 17. Sea-side view of structure before testing with 1969 storm; swl = 0.0 ft; direction of wave attack = west-southwest

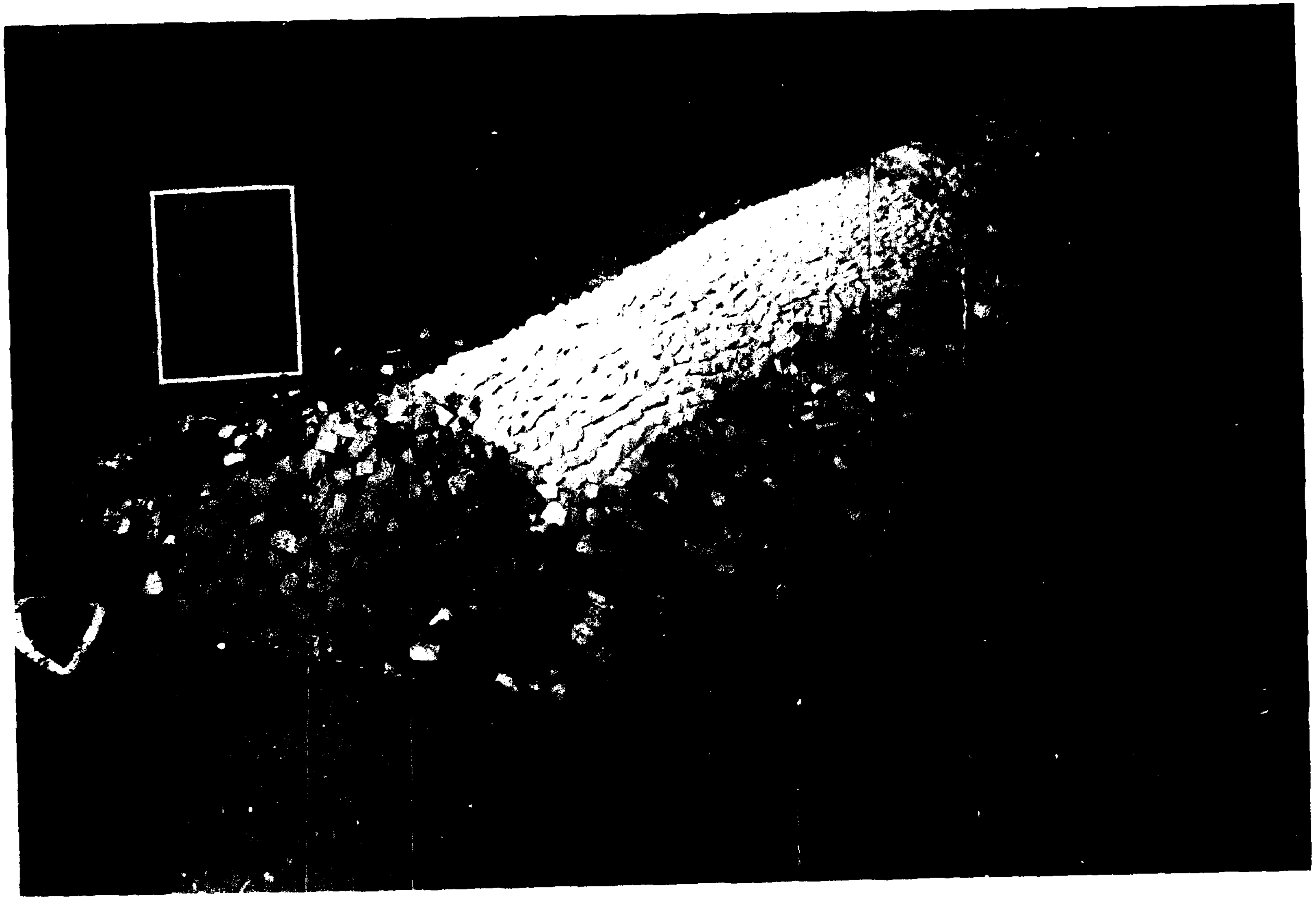


Photo 18. Channel-side view of structure before testing with 1969 storm; swl = 0.0 ft; direction of wave attack = west-southwest

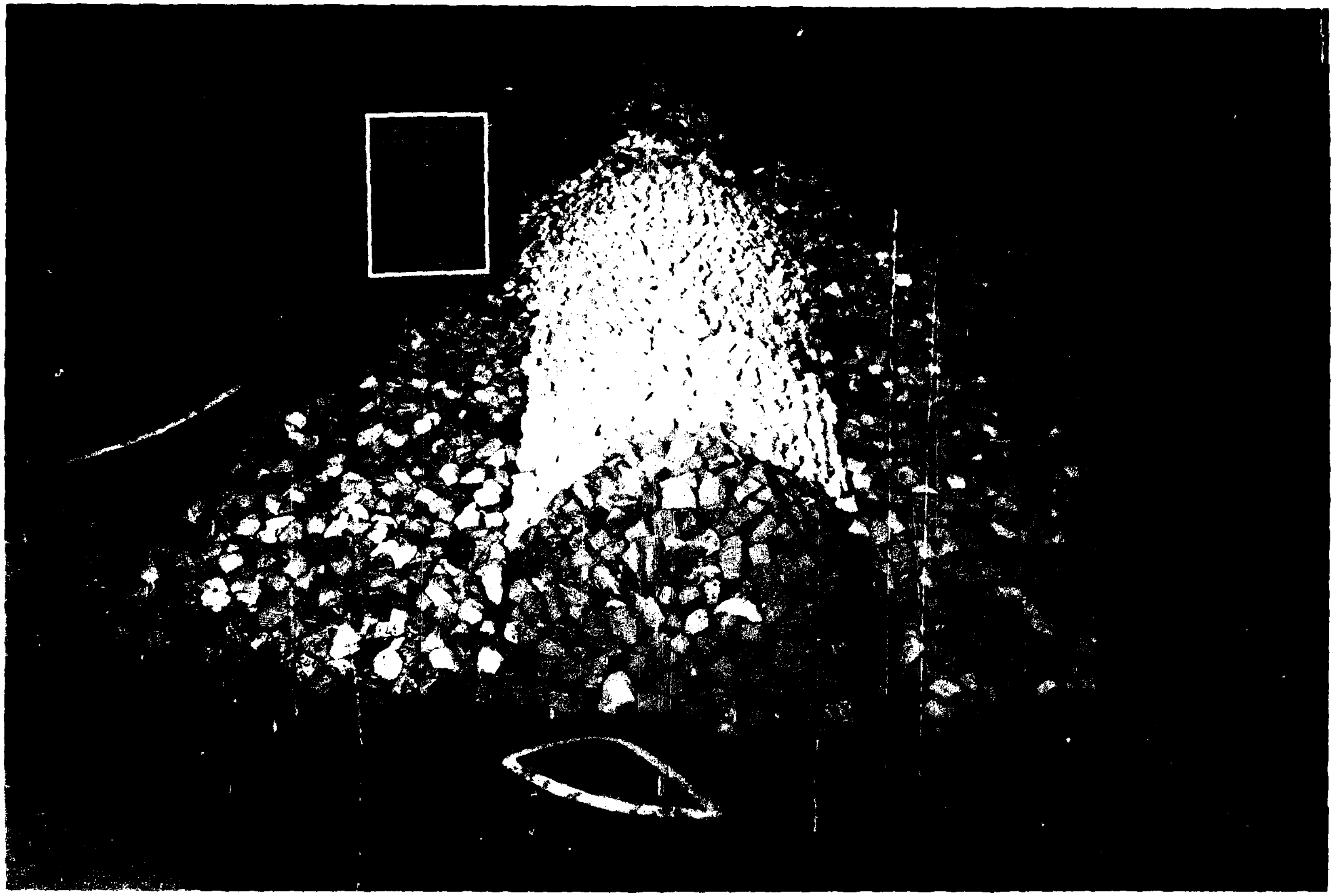


Photo 19. Long-axis view of structure after all testing at 0.0-ft swl

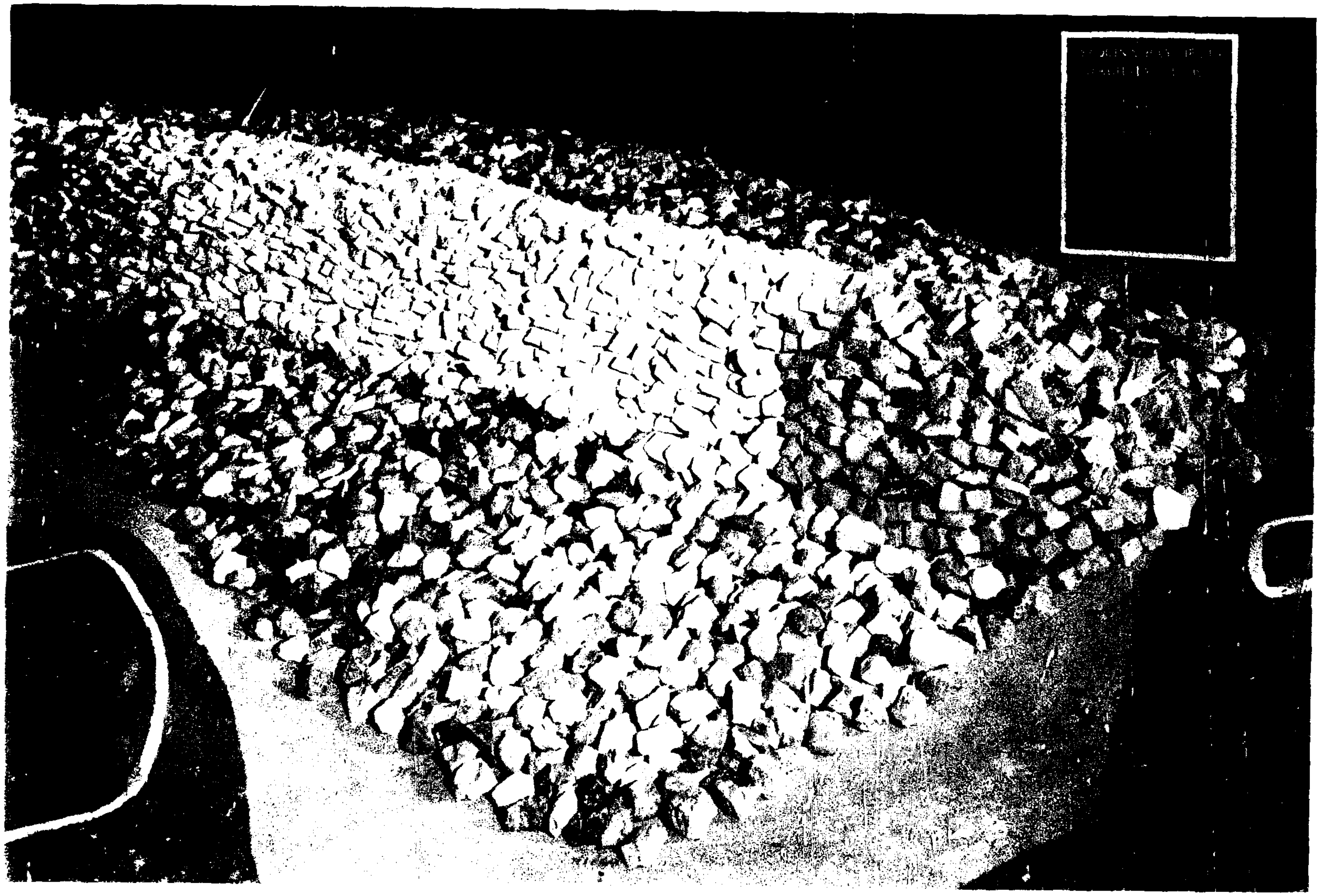


Photo 20. Sea-side view of structure after all testing at 0.0-ft swl

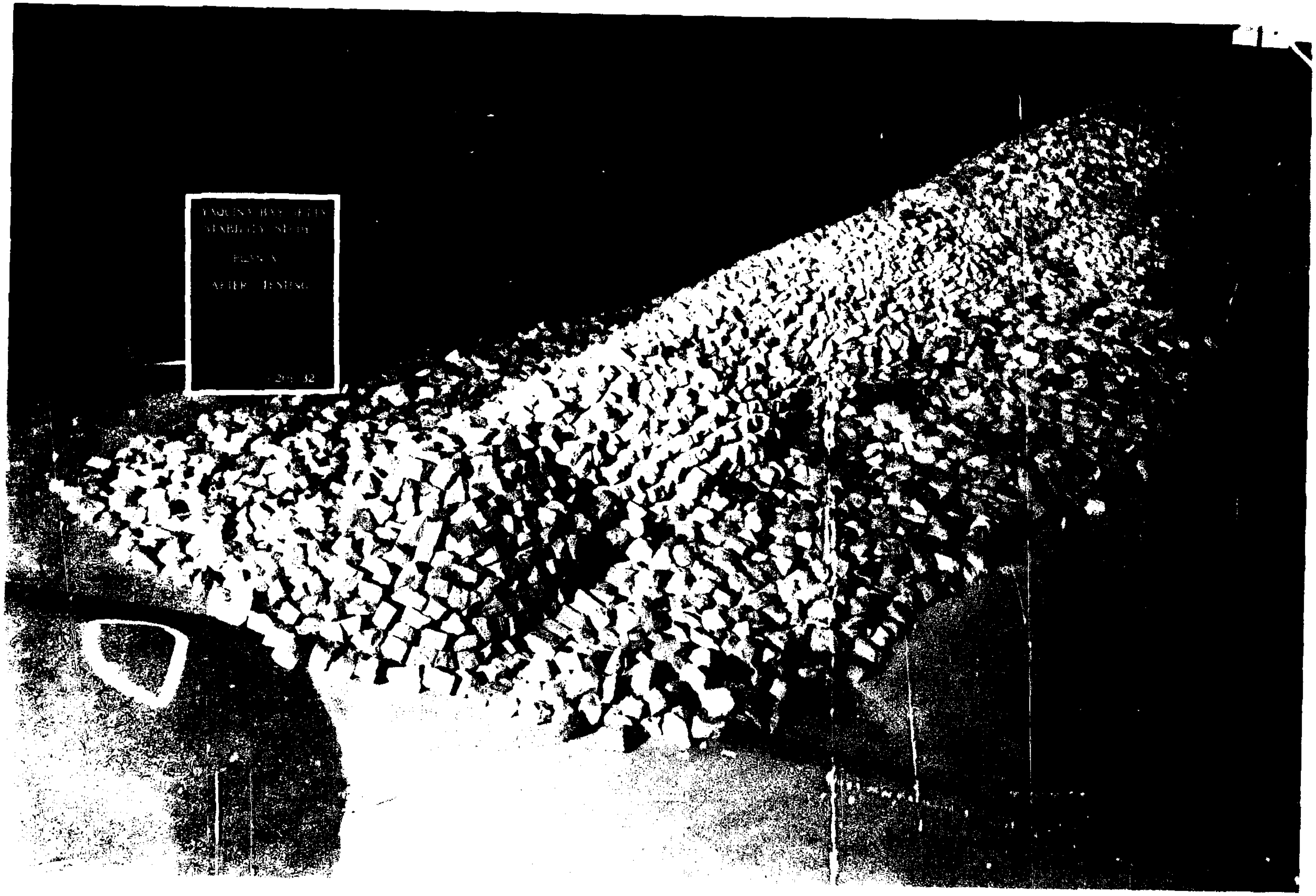


Photo 21. Channel-side view of structure after all testing at 0.0-ft swl

APPENDIX A: A WAVE DATA RESULTING FROM SPECTRAL STABILITY TESTS

Table A1

Measured Wave Conditions, Yaquina Bay Worst Waves Long Duration

Still-Water Level = +10.0 ft, Wave Direction = WSW, 1969 Storm

RUN NO.	GAGE 1 Hmo (ft)	GAGE 1 Tp (Sec)	GAGE 2 Hmo (ft)	GAGE 2 Tp (Sec)	GAGE 3 Hmo (ft)	GAGE 3 Tp (Sec)	AVERAGE Hmo (ft)	AVERAGE Tp (Sec)
6903A	24.4	16.9	19.1	17.0	20.3	17.0	19.7	17.0
6903B	24.2	16.9	19.6	17.0	20.5	17.0	20.1	17.0
6903C	24.1	16.9	19.3	17.0	20.4	17.0	19.9	17.0
6903D	24.2	16.9	19.0	17.0	20.0	17.0	19.5	17.0
6903E	24.1	16.9	19.8	17.0	20.5	17.0	20.2	17.0
6903F	24.1	16.9	19.3	17.0	20.2	17.0	19.8	17.0
AVG =	24.2	16.9			TOTAL AVG.		19.8	17.0

Table A2

Measured Wave Conditions, Yaquina bay Worst Waves Long Duration

Still-Water Level = +10.0 ft, Wave Direction = W, 1974 Storm

RUN NO.	GAGE 1 Hmo (ft)	GAGE 1 Tp (Sec)	GAGE 2 Hmo (ft)	GAGE 2 Tp (Sec)	GAGE 3 Hmo (ft)	GAGE 3 Tp (Sec)	AVERAGE Hmo (ft)	AVERAGE Tp (Sec)
7413A	21.9	16.4	19.7	16.7	20.2	16.2	[illegible]	16.5
7413B	21.8	16.4	20.0	16.7	[illegible]	16.2	20.1	16.5
7413C	21.9	16.4	[illegible]	16.7	20.5	16.2	20.4	16.5
7413D	[illegible]	16.4	19.7	16.7	[illegible]	16.2	20.0	16.5
7413E	21.8	16.4	19.9	16.7	[illegible]	16.2	20.1	16.5
7413F	21.8	16.4	19.6	16.7	[illegible]	16.2	20.0	16.5
AVG =	21.9	16.4			TOTAL AVG.		[illegible]	16.5

Table A3

Measured Wave Conditions, Yaquina Bay Worst Waves Long Duration

Still-Water Level = +10.0 ft, Wave Direction = WNW, 1969 Storm

RUN NO.	GAGE 1 Hmo (ft)	GAGE 1 Tp (Sec)	GAGE 2 Hmo (ft)	GAGE 2 Tp (Sec)	GAGE 3 Hmo (ft)	GAGE 3 Tp (Sec)	AVERAGE Hmo (ft)	AVERAGE Tp (Sec)
6923A	24.2	16.9	20.5	17.0	19.6	17.0	20.1	17.0
6923B	24.3	16.9	20.7	17.0	20.1	17.0	20.4	17.0
6923C	24.2	16.9	20.6	17.0	19.7	17.0	20.2	17.0
6923D	24.1	16.9	20.8	17.0	20.4	17.0	20.6	17.0
6923E	24.2	16.9	20.9	17.0	20.5	17.0	20.7	17.0
6923F	24.2	16.9	20.5	17.0	20.0	17.0	20.3	17.0
AVG =	24.2	16.9			TOTAL AVG.		20.4	17.0

Table A4

Measured Wave Conditions, Yaquina Bay Worst Waves Long Duration

Still-Water Level = +10.0 ft, Wave Direction = WNW, 1983 Storm

RUN NO.	GAGE 1 Hmo (ft)	GAGE 1 Tp (Sec)	GAGE 2 Hmo (ft)	GAGE 2 Tp (Sec)	GAGE 3 Hmo (ft)	GAGE 3 Tp (Sec)	AVERAGE Hmo (ft)	AVERAGE Tp (Sec)
8323A	19.2	17.2	19.4	16.9	19.0	16.9	19.2	16.9
8323B	19.1	17.2	19.3	17.2	19.1	17.2	19.2	17.2
8323C	19.3	17.2	19.6	17.2	19.1	17.2	19.4	17.2
8323D	19.1	17.2	19.4	16.9	19.0	16.9	[illegible]	16.9
8323E	19.1	17.2	19.4	17.2	19.2	17.2	19.3	17.2
8323F	19.2	17.2	19.4	17.2	19.1	16.9	[illegible]	[illegible]
AVG =	19.2	17.2			TOTAL AVG.		19.3	17.1

Table A5

Measured Wave Conditions, Yaquina Bay Worst Waves Long Duration

Still-Water Level = +0.0 ft, Wave Direction = WSW, 1969 Storm

RUN NO.	GAGE 1 Hmo (ft)	GAGE 1 Tp (Sec)	GAGE 2 Hmo (ft)	GAGE 2 Tp (Sec)	GAGE 3 Hmo (ft)	GAGE 3 Tp (Sec)	AVERAGE Hmo (ft)	AVERAGE Tp (Sec)
6903G	21.0	16.9	14.9	17.0	15.2	17.0	15.1	17.0
6903H	21.0	16.9	14.8	17.0	15.5	17.0	15.2	17.0
6903I	20.9	16.9	14.6	17.0	15.3	17.0	15.0	17.0
6903J	21.0	16.9	14.7	17.0	15.5	17.0	15.1	17.0
6903K	21.0	16.9	14.6	17.0	15.5	17.0	15.1	17.0
6903L	20.9	16.9	15.0	17.0	15.0	17.0	15.0	17.0
AVG =	21.0	16.9			TOTAL AVG.		15.1	17.0

Table A6

Measured Wave Conditions, Yaquina Bay Worst Waves Long Duration

Still-Water Level = +0.0 ft, Wave Direction = W, 1969 Storm

RUN NO.	GAGE 1 Hmo (ft)	GAGE 1 Tp (Sec)	GAGE 2 Hmo (ft)	GAGE 2 Tp (Sec)	GAGE 3 Hmo (ft)	GAGE 3 Tp (Sec)	AVERAGE Hmo (ft)	AVERAGE Tp (Sec)
6913G	21.7	16.9	15.4	17.0	16.4	17.0	15.9	17.0
6913H	21.7	16.9	15.5	17.0	16.4	17.3	16.0	17.2
6913I	21.7	16.9	15.4	17.0	16.6	17.0	16.0	[illegible]
6913J	21.[illegible]	16.9	15.4	17.0	16.0	17.2	15.[illegible]	[illegible].1
6913K	21.8	16.9	15.1	17.0	16.3	17.0	15.[illegible]	17.0
6913L	21.7	16.9	15.3	[illegible]	16.4	17.3	15.9	17.2
AVG =	21.7	16.9			TOTAL AVG.		15.9	[illegible].1

Table A7

Measured Wave Conditions, Yaquina Bay Worst Waves Long Duration

Still-Water Level = +0.0 ft, Wave Direction = WNW, 1974 Storm

RUN NO.	GAGE 1 Hmo (ft)	GAGE 1 Tp (Sec)	GAGE 2 Hmo (ft)	GAGE 2 Tp (Sec)	GAGE 3 Hmo (ft)	GAGE 3 Tp (Sec)	AVERAGE Hmo (ft)	AVERAGE Tp (Sec)
7423G	18.5	16.8	15.6	17.7	14.9	16.6	15.3	17.2
7423H	18.3	16.8	16.6	16.9	15.9	16.5	16.3	16.7
7423I	18.3	16.8	16.5	16.8	15.9	17.7	16.2	17.3
7423J	18.3	16.8	16.9	16.9	16.3	16.9	16.6	16.9
7423K	18.4	16.8	16.5	17.7	15.9	18.2	16.2	18.0
7423L	18.3	16.8	16.5	16.5	15.8	17.3	16.2	16.9
AVG =	18.4	16.8				TOTAL AVG.	16.1	17.1

Table A8

Measured Wave Conditions, Yaquina Bay Long Duration Stability Tests, Still-Water Level = +10.0 ft, Wave Direction = WSW, 1969 Storm

RUN NO.	GAGE NO.	Hmo (ft)	Tp (Sec)	RUN NO.	GAGE NO.	Hmo (ft)	Tp (Sec)
6903M	1			6903P	1	24.3	16.9
	2				2	24.8	16.1
	3				3	20.9	16.0
	4	UNSUCCESSFUL			4	20.6	17.0
	5	DATA COLLECTION			5	20.9	16.9
	6				6	19.5	17.0
	7				7	18.5	17.0
	8				8	16.7	17.7
	9				9	14.7	17.7
	10				10	13.5	17.0
6903N	1			6903Q	1	24.5	16.9
	2				2	24.5	16.1
	3				3	20.8	16.0
	4	UNSUCCESSFUL			4	20.9	17.0
	5	DATA COLLECTION			5	21.1	16.9
	6				6	19.6	17.0
	7				7	19.4	17.0
	8				8	17.2	17.7
	9				9	14.8	17.7
	10				10	13.5	17.7
6903O	1	24.3	16.9	6903R	1	24.4	16.9
	2	24.8	16.1		2	24.6	16.1
	3	20.8	16.0		3	20.8	16.0
	4	20.8	17.0		4	20.9	17.0
	5	21.1	16.9		5	21.1	16.9
	6	19.6	17.0		6	19.7	17.0
	7	18.9	17.0		7	19.4	17.0
	8	16.9	17.7		8	17.3	17.7
	9	14.8	17.7		9	14.9	17.7
	10	13.5	17.4		10	13.6	17.7

Table A9

Measured Wave Conditions, Yaquina Bay Long Duration Stability Tests, Still-Water Level = +10.0 ft, Wave Direction = W, 1974 Storm

RUN NO.	GAGE NO.	Hmo (ft)	Tp (Sec)	RUN NO.	GAGE NO.	Hmo (ft)	Tp (Sec)
7413M	1	21.5	16.4	7413P	1	21.4	16.4
	2	22.1	16.4		2	22.1	16.4
	3	21.4	16.2		3	21.5	16.2
	4	20.7	16.2		4	20.7	16.2
	5	20.8	16.2		5	20.5	16.2
	6	19.1	18.2		6	19.3	18.2
	7	18.0	18.2		7	18.1	18.2
	8	14.9	18.2		8	14.7	18.2
	9	13.4	17.7		9	13.4	18.2
	10	12.6	17.7		10	12.5	17.7
7413N	1	21.5	16.4	7413Q	1	21.4	16.4
	2	22.1	16.4		2	21.9	16.4
	3	21.5	16.2		3	21.3	16.2
	4	20.6	16.2		4	20.6	16.2
	5	20.7	16.2		5	20.4	16.2
	6	18.7	16.2		6	19.3	16.2
	7	17.2	18.2		7	17.3	18.2
	8	14.1	18.2		8	14.0	18.2
	9	12.4	18.2		9	12.5	18.2
	10	11.7	18.7		10	11.6	18.2
7413O	1	21.6	16.4	7413R	1	21.5	16.4
	2	21.9	16.4		2	22.0	16.4
	3	21.4	16.2		3	21.3	16.2
	4	20.5	16.2		4	20.5	16.2
	5	20.6	16.2		5	20.4	16.2
	6	18.9	16.2		6	19.3	16.2
	7	17.2	18.2		7	17.4	16.2
	8	14.1	18.2		8	14.4	16.2
	9	12.3	18.2		9	12.9	16.2
	10	11.6	18.5		10	12.0	16.2

Table A10

Measured Wave Conditions, Yaquina Bay Long Duration Stability Tests, Still-Water Level = +10.0 ft, Wave Direction = WNW, 1969 Storm

RUN NO.	GAGE NO.	Hmo (ft)	To (Sec)	RUN NO.	GAGE NO.	Hmo (ft)	To (Sec)
6923M	1	24.2	16.7	6923P	1	24.0	16.7
	2	23.4	16.2		2	23.5	15.7
	3	22.0	17.0		3	21.6	17.0
	4	21.7	17.0		4	21.7	17.0
	5	21.2	17.0		5	21.3	17.0
	6	20.9	17.0		6	21.1	17.0
	7	15.8	16.9		7	15.7	17.0
	8	10.5	16.8		8	10.2	16.8
	9	8.4	16.2		9	8.2	16.1
	10	7.2	17.4		10	7.0	17.5
6923N	1	24.2	16.7	6923Q	1	24.1	16.7
	2	23.5	16.2		2	23.5	16.2
	3	21.7	17.0		3	21.9	17.0
	4	21.5	17.0		4	22.0	17.0
	5	21.2	17.0		5	21.5	17.0
	6	21.2	17.0		6	21.3	17.0
	7	15.6	16.8		7	15.6	16.8
	8	10.2	16.2		8	10.3	16.3
	9	8.1	16.0		9	8.2	16.0
	10	7.0	17.5		10	7.0	17.5
6923O	1	24.2	16.7	6923R	1	24.2	16.7
	2	23.5	16.2		2	23.5	16.2
	3	21.8	17.0		3	21.8	17.0
	4	21.7	17.0		4	22.0	17.0
	5	21.3	17.0		5	21.5	17.0
	6	21.4	17.0		6	21.3	17.0
	7	15.8	17.0		7	15.7	16.8
	8	10.4	16.3		8	10.3	16.3
	9	8.3	16.1		9	8.1	16.0
	10	7.3	17.5		10	7.0	17.5

APPENDIX B: RESULTS OF NUMERICAL/PHYSICAL MODEL WAVE HEIGHT COMPARISON

Monochromatic wave data collected in the physical model and corresponding numerical predictions are presented in Table B1. The physically modeled results were subjected to a downcrossing analysis from which significant, maximum, and average wave heights were obtained. An initial comparison was performed by plotting these values and the related numerical predictions versus distance from the jetty head. The results are depicted in Figures B1-B18. For each of the 18 test runs, the three individual measured heights were also plotted versus the corresponding numerically predicted height. This information is presented in Figures B19-B36. The solid diagonal line simply represents the ideal condition which would result from a perfect correlation between measured and predicted wave heights. A linear regression performed on each data set yielded the equation of the shorter solid line present in the figures. The results of the regression analyses are listed in Table B2.

Table B1

Monochromatic Wave Data; Comparison of Physical and Numerical Models

CASE NO.	GAGE NO.	DIST. FROM JETTY HEAD (ft)	PHYSICAL MODEL DATA HSIGp (ft)	HMAXp (ft)	HAVGp (sec)	Tp (sec)	NUMERICAL MODEL DATA Hn (ft)	Tn (sec)	HSIGp/Hn	HMAXp/Hn	HAVGp/Hn	Tp/Tn
1	1	2102	14.49	14.89	13.99	11.0	12.84	11	1.13	1.16	1.09	1.00
	2	1652	14.56	14.97	13.82	11.0	13.46	11	1.08	1.11	1.03	1.00
	3	418	16.01	16.69	15.14	11.0	14.11	11	1.13	1.18	1.07	1.00
	4	238	15.53	16.13	14.85	11.0	15.93	11	0.97	1.01	0.93	1.00
	5	148	19.90	20.84	18.94	11.0	16.13	11	1.23	1.29	1.17	1.00
	6	58	19.67	20.11	18.88	11.0	15.67	11	1.26	1.28	1.20	1.00
	7	-58	15.15	16.72	13.78	11.0	13.23	11	1.15	1.26	1.04	1.00
	8	-158	10.88	12.52	12.52	11.0	12.40	11	0.88	1.01	1.01	1.00
	9	-270	8.88	9.54	9.19	11.0	7.86	11	1.13	1.21	1.17	1.00
	10	-396	5.94	7.20	4.17	10.8	7.75	11	0.77	0.93	0.54	0.98
2	1	2102	11.88	12.39	11.63	14.0	13.96	14	0.85	0.89	0.83	1.00
	2	1652	15.56	16.30	15.07	14.0	14.65	14	1.06	1.11	1.03	1.00
	3	418	21.35	21.63	20.77	14.0	15.13	14	1.41	1.43	1.37	1.00
	4	238	21.14	22.02	20.49	14.0	17.09	14	1.24	1.29	1.20	1.00
	5	148	23.09	23.70	22.23	14.0	14.86	14	1.55	1.59	1.50	1.00
	6	58	21.71	21.79	21.56	14.0	13.48	14	1.61	1.62	1.60	1.00
	7	-58	16.92	18.92	9.33	14.0	11.28	14	1.50	1.68	0.83	1.00
	8	-158	11.62	13.34	8.14	14.0	10.58	14	1.10	1.26	0.77	1.00
	9	-270	8.32	9.30	7.23	14.0	6.71	14	1.24	1.39	1.08	1.00
	10	-396	8.59	9.86	7.49	14.0	6.61	14	1.30	1.49	1.13	1.00
3	1	2102	13.57	13.84	13.36	15.9	15.00	16	0.90	0.92	0.89	0.99
	2	1652	16.24	17.65	15.49	15.9	15.72	16	1.03	1.12	0.99	0.99
	3	418	19.50	20.90	18.64	15.9	16.20	16	1.20	1.29	1.15	0.99
	4	238	21.76	22.76	20.82	15.9	15.58	16	1.40	1.46	1.34	0.99
	5	148	22.17	24.04	21.08	15.9	13.64	16	1.63	1.76	1.55	0.99
	6	58	20.65	20.84	20.45	15.9	12.87	16	1.60	1.62	1.59	0.99
	7	-58	18.45	22.62	12.82	16.1	11.04	16	1.67	2.05	1.16	1.01
	8	-158	12.79	14.52	10.03	15.9	10.35	16	1.24	1.40	0.97	0.99
	9	-270	6.74	8.49	4.67	15.7	6.57	16	1.03	1.29	0.71	0.98
	10	-396	3.98	5.25	2.50	16.0	6.48	16	0.61	0.81	0.39	1.00
4	1	2102	13.70	14.05	13.05	11.0	17.55	11	0.78	0.80	0.74	1.00
	2	1652	16.57	17.13	16.04	11.0	18.30	11	0.91	0.94	0.88	1.00
	3	418	11.70	12.56	11.12	11.0	17.38	11	0.67	0.72	0.64	1.00
	4	238	12.58	13.56	11.78	11.0	16.36	11	0.77	0.83	0.72	1.00
	5	148	15.15	16.65	13.15	11.0	13.86	11	1.09	1.20	0.95	1.00
	6	58	16.13	17.49	14.30	11.0	12.78	11	1.26	1.37	1.12	1.00
	7	-58	17.22	19.41	15.74	11.0	11.56	11	1.49	1.68	1.36	1.00
	8	-158	12.63	14.38	11.04	11.0	11.18	11	1.13	1.[illegible]9	0.99	1.00
	9	-270	11.15	11.89	10.36	11.0	7.71	11	1.45	1.54	1.34	1.00
	10	-396	9.92	10.61	9.10	11.0	7.88	11	1.26	1.35	1.15	1.00

(Continued)

(Sheet 1 of 5)

Table B1 (Continued)

CASE NO.	GAGE NO.	DIST. FROM JETTY HEAD (ft)	HSIGp (ft)	PHYSICAL MODEL DATA HMAXp (ft)	HAVGp (sec)	Tp (sec)	NUMERICAL MODEL DATA Hn (ft)	Tn (sec)	HSIGp/Hn	HMAXp/Hn	HAVGp/Hn	Tp/Tn
5	1	2102	17.00	17.57	16.46	14.0	18.12	14	0.94	0.97	0.91	1.00
	2	1652	15.05	15.62	14.44	14.0	19.07	14	0.79	0.82	0.76	1.00
	3	418	17.64	18.99	16.73	14.2	17.30	14	1.02	1.10	0.97	1.01
	4	238	14.68	15.84	13.86	14.0	16.41	14	0.89	0.97	0.84	1.00
	5	148	17.83	19.40	16.74	14.0	13.92	14	1.28	1.39	1.20	1.00
	6	58	17.05	17.97	16.09	14.2	12.84	14	1.33	1.40	1.25	1.01
	7	-58	23.15	25.02	21.61	14.0	11.42	14	2.03	2.19	1.89	1.00
	8	-158	18.28	19.48	16.25	14.0	10.96	14	1.67	1.78	1.48	1.00
	9	-270	13.10	14.61	11.83	14.0	7.37	14	1.78	1.98	1.61	1.00
	10	-396	11.34	12.14	10.42	14.0	7.46	14	1.52	1.63	1.40	1.00
6	1	2102	12.67	13.13	12.30	15.9	14.79	16	0.86	0.89	0.83	0.99
	2	1652	16.23	16.55	15.83	15.7	15.56	16	1.04	1.06	1.02	0.98
	3	418	13.24	13.55	12.94	15.7	14.05	16	0.94	0.96	0.92	0.98
	4	238	15.11	15.59	14.54	15.9	15.89	16	0.95	0.98	0.92	0.99
	5	148	15.48	16.02	14.72	15.9	16.15	16	0.96	0.99	0.91	0.99
	6	58	17.86	18.40	16.11	15.9	16.29	16	1.10	1.13	0.99	0.99
	7	-58	19.07	21.05	17.69	15.9	14.99	16	1.27	1.40	1.18	0.99
	8	-158	13.29	14.68	11.78	15.9	14.35	16	0.93	1.02	0.82	0.99
	9	-270	11.25	12.00	10.49	15.9	9.57	16	1.18	1.25	1.10	0.99
	10	-396	10.58	11.79	7.41	15.9	9.65	16	1.10	1.22	0.77	0.99
7	1	2102	11.37	12.35	10.60	11.0	15.17	11	0.75	0.81	0.70	1.00
	2	1652	14.88	15.86	14.31	11.0	15.13	11	0.98	1.05	0.95	1.00
	3	418	15.39	16.28	14.79	11.0	17.11	11	0.90	0.95	0.86	1.00
	4	238	15.13	15.47	14.59	11.0	16.24	11	0.93	0.95	0.90	1.00
	5	148	19.84	20.58	18.69	11.0	14.59	11	1.36	1.41	1.28	1.00
	6	58	18.19	19.58	16.73	10.8	14.22	11	1.28	1.38	1.18	0.98
	7	-58	16.95	19.38	15.51	11.0	14.42	11	1.18	1.34	1.08	1.00
	8	-158	13.79	14.58	11.79	10.8	14.38	11	0.96	1.01	0.82	0.98
	9	-270	9.98	10.73	9.12	10.8	11.72	11	0.85	0.92	0.78	0.98
	10	-396	8.96	10.35	8.17	10.8	12.35	11	0.73	0.84	0.66	0.98
8	1	2102	17.29	17.75	16.95	14.0	19.25	14	0.90	0.92	0.88	1.00
	2	1652	15.32	15.62	14.90	14.0	19.49	14	0.79	0.80	0.76	1.00
	3	418	22.14	23.06	21.03	14.0	21.23	14	1.04	1.09	0.99	1.00
	4	238	18.41	19.81	17.29	14.0	19.58	14	0.94	1.01	0.88	1.00
	5	148	23.66	24.60	22.72	14.0	15.07	14	1.57	1.63	1.51	1.00
	6	58	21.39	21.96	20.93	13.8	13.82	14	1.55	1.59	1.51	0.99
	7	-58	25.71	27.23	22.48	14.0	13.23	14	1.94	2.06	1.70	1.00
	8	-158	21.60	25.30	18.07	14.0	13.10	14	1.65	1.93	1.38	1.00
	9	-270	13.56	14.71	11.98	14.0	10.05	14	1.35	1.46	1.19	1.00
	10	-396	12.06	13.95	10.44	14.0	10.53	14	1.15	1.32	0.99	1.00

(Continued)

(Sheet 2 of 5)

Table B1 (Continued)

CASE NO.	GAGE NO.	DIST. FROM JETTY HEAD (ft)	HSIGp (ft)	PHYSICAL MODEL DATA HMAXp (ft)	HAVGp (sec)	Tp (sec)	NUMERICAL MODEL DATA Hn (ft)	Tn (sec)	HSIGp/Hn	HMAXp/Hn	HAVGp/Hn	Tp/Tn
9	1	2102	14.19	14.47	13.89	15.9	18.41	16	0.77	0.79	0.75	0.99
	2	1652	18.50	18.84	18.06	15.9	18.86	16	0.98	1.00	0.96	0.99
	3	418	16.63	17.51	16.05	15.9	19.85	16	0.84	0.88	0.81	0.99
	4	238	19.53	20.37	18.82	15.9	18.42	16	1.06	1.11	1.02	0.99
	5	148	21.24	22.15	19.75	15.9	14.41	16	1.47	1.54	1.37	0.99
	6	58	21.72	22.33	13.39	15.9	13.29	16	1.63	1.68	1.01	0.99
	7	-58	20.95	22.49	18.67	15.9	12.62	16	1.66	1.78	1.48	0.99
	8	-158	17.82	19.98	14.85	15.9	12.43	16	1.43	1.61	1.19	0.99
	9	-270	13.61	15.07	11.53	15.9	9.25	16	1.47	1.63	1.25	0.99
	10	-396	10.84	12.40	9.90	15.9	9.64	16	1.12	1.29	1.03	0.99
10	1	2102	9.95	10.13	9.71	11.0	9.17	11	1.09	1.10	1.06	1.00
	2	1652	9.27	9.56	9.00	11.0	9.62	11	0.96	0.99	0.94	1.00
	3	418	8.74	8.97	8.49	11.0	10.09	11	0.87	0.89	0.84	1.00
	4	238	9.50	9.64	9.13	11.0	11.38	11	0.83	0.85	0.80	1.00
	5	148	11.85	12.36	11.52	11.0	11.52	11	1.03	1.07	1.00	1.00
	6	58	11.47	11.89	11.08	11.0	11.56	11	0.99	1.03	0.96	1.00
	7	-58	14.87	16.33	14.03	11.0	10.06	11	1.48	1.62	1.39	1.00
	8	-158	9.17	9.60	8.67	11.0	9.41	11	0.97	1.02	0.92	1.00
	9	-270	7.16	7.89	4.84	11.0	5.95	11	1.20	1.33	0.81	1.00
	10	-396	5.01	5.28	4.56	11.0	5.85	11	0.86	0.90	0.78	1.00
11	1	2102	9.43	9.57	9.25	14.0	9.97	14	0.95	0.96	0.93	1.00
	2	1652	12.09	12.69	11.72	14.0	10.46	14	1.16	1.21	1.12	1.00
	3	418	12.58	12.91	12.24	14.0	11.29	14	1.11	1.14	1.08	1.00
	4	238	12.67	12.96	12.38	14.0	12.29	14	1.03	1.05	1.01	1.00
	5	148	14.28	14.62	13.84	14.0	12.42	14	1.15	1.18	1.11	1.00
	6	58	16.04	16.62	15.60	14.0	11.86	14	1.35	1.40	1.32	1.00
	7	-58	16.17	17.15	14.97	14.0	10.21	14	1.58	1.68	1.47	1.00
	8	-158	10.69	11.62	9.80	14.0	9.77	14	1.09	1.19	1.00	1.00
	9	-270	8.35	9.46	7.75	14.0	6.44	14	1.30	1.47	1.20	1.00
	10	-396	7.40	8.18	4.51	14.0	6.38	14	1.16	1.28	0.71	1.00
12	1	2102	10.62	10.76	10.42	15.9	10.71	16	0.99	1.00	0.97	0.99
	2	1652	12.14	12.77	11.78	15.9	11.23	16	1.08	1.14	1.05	0.99
	3	418	12.05	12.80	11.73	15.9	11.58	16	1.04	1.11	1.01	0.99
	4	238	13.66	14.56	13.23	15.9	13.09	16	1.04	1.11	1.01	0.99
	5	148	13.57	14.51	13.06	15.9	13.27	16	1.02	1.09	0.98	0.99
	6	58	15.83	17.03	15.33	15.9	13.32	16	1.19	1.28	1.15	0.99
	7	-58	16.41	17.92	10.15	15.9	11.68	16	1.40	1.53	0.87	0.99
	8	-158	9.17	10.44	7.59	15.9	10.97	16	0.84	0.95	0.69	0.99
	9	-270	8.25	9.12	6.42	15.9	6.98	16	1.18	1.31	0.92	0.99
	10	-396	4.49	5.44	2.54	15.9	6.89	16	0.65	0.79	0.37	0.99

(Continued)

(Sheet 3 of 5)

Table B1 (Continued)

CASE NO.	GAGE NO.	DIST. FROM JETTY HEAD (ft)	PHYSICAL MODEL DATA HSIGp (ft)	HMAXp (ft)	HAVGp (sec)	Tp (sec)	NUMERICAL MODEL DATA Hn (ft)	Tn (sec)	HSIGp/Hn	HMAXp/Hn	HAVGp/Hn	Tp/Tn
13	1	2102	10.52	10.67	10.17	11.0	12.52	11	0.84	0.85	0.81	1.00
	2	1652	12.66	12.99	12.24	11.0	13.06	11	0.97	0.99	0.94	1.00
	3	418	9.75	10.02	9.51	11.0	12.40	11	0.79	0.81	0.77	1.00
	4	238	9.09	9.31	8.81	11.0	13.90	11	0.65	0.67	0.63	1.00
	5	148	10.01	10.43	9.66	11.0	14.08	11	0.71	0.74	0.69	1.00
	6	58	9.64	9.96	9.25	11.0	14.18	11	0.68	0.70	0.65	1.00
	7	-58	15.64	16.33	14.82	11.0	13.36	11	1.17	1.22	1.11	1.00
	8	-158	13.01	13.81	12.16	11.0	12.95	11	1.00	1.07	0.94	1.00
	9	-270	11.01	11.64	9.74	11.0	8.97	11	1.23	1.30	1.09	1.00
	10	-396	9.79	11.01	8.40	11.0	9.18	11	1.07	1.20	0.92	1.00
14	1	2102	12.41	12.60	12.13	14.0	12.94	14	0.96	0.97	0.94	1.00
	2	1652	11.51	12.02	11.07	14.0	13.62	14	0.85	0.88	0.81	1.00
	3	418	12.58	12.90	12.28	14.0	12.35	14	1.02	1.04	0.99	1.00
	4	238	10.89	11.19	10.57	14.0	13.93	14	0.78	0.80	0.76	1.00
	5	148	13.81	14.01	13.30	14.0	14.16	14	0.98	0.99	0.94	1.00
	6	58	14.65	14.99	13.96	14.0	14.28	14	1.03	1.05	0.98	1.00
	7	-58	20.48	21.79	19.44	14.0	13.26	14	1.54	1.64	1.47	1.00
	8	-158	16.33	17.62	12.00	14.2	12.75	14	1.28	1.38	0.94	1.01
	9	-270	11.32	11.99	10.57	14.0	8.61	14	1.31	1.39	1.23	1.00
	10	-396	10.29	11.46	9.58	14.0	8.72	14	1.18	1.31	1.10	1.00
15	1	2102	8.86	9.06	8.65	15.9	10.59	16	0.84	0.86	0.82	0.99
	2	1652	12.16	12.44	11.97	15.7	11.14	16	1.09	1.12	1.07	0.98
	3	418	10.85	11.09	10.42	15.9	10.05	16	1.08	1.10	1.04	0.99
	4	238	12.30	12.77	11.92	15.9	11.37	16	1.08	1.12	1.05	0.99
	5	148	11.55	11.69	11.33	15.9	11.55	16	1.00	1.01	0.98	0.99
	6	58	14.44	14.82	14.05	15.9	11.65	16	1.24	1.27	1.21	0.99
	7	-58	18.03	18.86	17.09	15.9	10.72	16	1.68	1.76	1.59	0.99
	8	-158	11.97	13.52	11.22	15.9	10.27	16	1.17	1.32	1.09	0.99
	9	-270	10.92	11.85	9.78	15.9	6.85	16	1.59	1.73	1.43	0.99
	10	-396	10.74	11.78	9.79	15.9	6.90	16	1.56	1.71	1.42	0.99
16	1	2102	8.31	8.58	7.96	11.0	10.78	11	0.77	0.80	0.74	1.00
	2	1652	10.44	11.25	9.79	11.0	10.81	11	0.97	1.04	0.91	1.00
	3	418	11.45	11.57	11.20	11.0	12.22	11	0.94	0.95	0.92	1.00
	4	238	10.86	11.21	10.53	11.0	13.78	11	0.79	0.81	0.76	1.00
	5	148	15.02	15.43	14.54	11.0	14.03	11	1.07	1.10	1.04	1.00
	6	58	13.58	13.83	13.18	11.0	14.23	11	0.95	0.97	0.93	1.00
	7	-58	16.15	16.92	15.30	11.0	14.44	11	1.12	1.17	1.06	1.00
	8	-158	14.14	15.38	13.21	11.0	14.40	11	0.98	1.07	0.92	1.00
	9	-270	10.12	10.55	9.48	11.0	11.76	11	0.86	0.90	0.81	1.00
	10	-396	10.82	11.73	10.04	11.0	12.39	11	0.87	0.95	0.81	1.00

(Continued)

(Sheet 4 of 5)

Table B1 (Concluded)

CASE NO.	GAGE NO.	DIST. FROM JETTY HEAD (ft)	PHYSICAL MODEL DATA HSIGp (ft)	HMAXp (ft)	HAVGp (sec)	Tp (sec)	NUMERICAL MODEL DATA Hn (ft)	Tn (sec)	HSIGp/Hn	HMAXp/Hn	HAVGp/Hn	Tp/Tn
17	1	2102	12.66	13.08	12.32	14.0	13.74	14	0.92	0.95	0.90	1.00
	2	1652	11.68	12.03	11.27	14.0	13.91	14	0.84	0.86	0.81	1.00
	3	418	17.27	17.55	16.68	14.0	15.15	14	1.14	1.16	1.10	1.00
	4	238	12.46	13.13	11.63	14.0	16.98	14	0.73	0.77	0.68	1.00
	5	148	15.91	16.40	15.11	14.0	14.83	14	1.07	1.11	1.02	1.00
	6	58	15.85	16.18	15.08	14.0	13.72	14	1.16	1.18	1.10	1.00
	7	-58	21.19	22.27	20.16	14.0	13.30	14	1.59	1.67	1.52	1.00
	8	-158	18.37	20.61	14.96	14.0	13.17	14	1.39	1.56	1.14	1.00
	9	-270	12.52	13.37	11.68	14.0	13.22	14	0.95	1.01	0.88	1.00
	10	-396	11.17	11.68	10.51	14.0	13.45	14	0.83	0.87	0.78	1.00
18	1	2102	10.18	10.46	9.98	15.9	13.14	16	0.77	0.80	0.76	0.99
	2	1652	12.90	13.25	12.31	15.9	13.46	16	0.96	0.98	0.91	0.99
	3	418	12.79	13.00	12.30	15.9	14.16	16	0.90	0.92	0.87	0.99
	4	238	14.50	15.09	14.07	15.9	15.87	16	0.91	0.95	0.89	0.99
	5	148	15.29	15.46	14.84	15.9	16.12	16	0.95	0.96	0.92	0.99
	6	58	15.91	17.73	11.61	15.9	16.25	16	0.98	1.09	0.71	0.99
	7	-58	19.59	21.53	18.40	15.9	15.92	16	1.23	1.35	1.16	0.99
	8	-158	15.42	17.11	13.97	15.9	15.69	16	0.98	1.09	0.89	0.99
	9	-270	12.24	14.10	11.51	15.9	15.66	16	0.78	0.90	0.73	0.99
	10	-396	11.60	12.53	10.71	15.9	15.88	16	0.73	0.79	0.67	0.99

(Sheet 5 of 5)

Table B2

Results of Regression Analysis

CASE NO.		DATA REGRESSION FOR HSIGp	DATA REGRESSION FOR HMAXp	DATA REGRESSION FOR HAVGp
		HSIGp	HMAXp	HAVGp
1	Regression Output:			
	Constant	-3.60798	-2.21626	-3.81705
	Std Err of Y Est	1.719109	1.610062	1.727117
	R Squared	0.865686	0.873630	0.859666
	No. of Observations	10	10	10
	Degrees of Freedom	8	8	8
	X Coefficient(s)	1.368757	1.327660	1.340629
	Std Err of Coef.	0.190616	0.178525	0.191504
2	Regression Output:			
	Constant	-0.08651	1.870475	-3.83389
	Std Err of Y Est	3.477695	3.455330	3.809986
	R Squared	0.665208	0.637528	0.679567
	No. of Observations	10	10	10
	Degrees of Freedom	8	8	8
	X Coefficient(s)	1.295095	1.210657	1.465853
	Std Err of Coef.	0.324836	0.322747	0.355874
3	Regression Output:			
	Constant	-1.41677	1.106554	-5.34968
	Std Err of Y Est	4.029533	4.629760	3.649454
	R Squared	0.635011	0.538092	0.732862
	No. of Observations	10	10	10
	Degrees of Freedom	8	8	8
	X Coefficient(s)	1.377219	1.294811	1.566276
	Std Err of Coef.	0.369153	0.424141	0.334333
4	Regression Output:			
	Constant	10.42388	12.12660	8.71645[illegible]
	Std Err of Y Est	2.444087	2.857663	2.132247
	R Squared	0.143861	0.075310	0.236558
	No. of Observations	10	10	10
	Degrees of Freedom	8	8	8
	X Coefficient(s)	0.241610	0.1966[illegible]	0.286232
	Std Err of Coef.	0.208387	0.243649	0.181799

(Continued)

(Sheet 1 of 5)

Table B2 (Continued)

CASE NO.	DATA REGRESSION FOR HSIGp HSIGp		DATA REGRESSION FOR HMAXp HMAXp		DATA REGRESSION FOR HAVGp HAVGp	
5	Regression Output:		Regression Output:		Regression Output:	
	Constant	14.23611	Constant	15.93439	Constant	12.32785
	Std Err of Y Est	3.354740	Std Err of Y Est	3.656948	Std Err of Y Est	3.096295
	R Squared	0.048373	R Squared	0.024111	R Squared	0.100555
	No. of Observations	10	No. of Observations	10	No. of Observations	10
	Degrees of Freedom	8	Degrees of Freedom	8	Degrees of Freedom	8
	X Coefficient(s)	0.168746	X Coefficient(s)	0.128242	X Coefficient(s)	0.230973
	Std Err of Coef.	0.264617	Std Err of Coef.	0.288455	Std Err of Coef.	0.244231
6	Regression Output:		Regression Output:		Regression Output:	
	Constant	2.433530	Constant	3.985099	Constant	-0.94163
	Std Err of Y Est	1.875577	Std Err of Y Est	2.248521	Std Err of Y Est	1.787394
	R Squared	0.591100	R Squared	0.469185	R Squared	0.692386
	No. of Observations	10	No. of Observations	10	No. of Observations	10
	Degrees of Freedom	8	Degrees of Freedom	8	Degrees of Freedom	8
	X Coefficient(s)	0.852464	X Coefficient(s)	0.799129	X Coefficient(s)	1.013704
	Std Err of Coef.	0.250673	Std Err of Coef.	0.300518	Std Err of Coef.	0.238888
7	Regression Output:		Regression Output:		Regression Output:	
	Constant	-1.50416	Constant	0.725603	Constant	-3.59340
	Std Err of Y Est	3.220874	Std Err of Y Est	3.433808	Std Err of Y Est	3.001156
	R Squared	0.251401	R Squared	0.202552	R Squared	0.305795
	No. of Observations	10	No. of Observations	10	No. of Observations	10
	Degrees of Freedom	8	Degrees of Freedom	8	Degrees of Freedom	8
	X Coefficient(s)	1.097651	X Coefficient(s)	1.017711	X Coefficient(s)	1.171362
	Std Err of Coef.	0.669668	Std Err of Coef.	0.713940	Std Err of Coef.	0.623985
8	Regression Output:		Regression Output:		Regression Output:	
	Constant	15.84469	Constant	18.80092	Constant	12.27928
	Std Err of Y Est	4.703305	Std Err of Y Est	5.005521	Std Err of Y Est	4.269756
	R Squared	0.035662	R Squared	0.007741	R Squared	0.109060
	No. of Observations	10	No. of Observations	10	No. of Observations	10
	Degrees of Freedom	8	Degrees of Freedom	8	Degrees of Freedom	8
	X Coefficient(s)	0.210447	X Coefficient(s)	0.102859	X Coefficient(s)	0.347583
	Std Err of Coef.	0.386905	Std Err of Coef.	0.411766	Std Err of Coef.	0.351240

(Continued)

(Sheet 2 of 5)

Table B2 (Continued)

CASE NO.	DATA REGRESSION FOR HSIGp	HSIGp	DATA REGRESSION FOR HMAXp	HMAXp	DATA REGRESSION FOR HAV6p	HAV6p
9	Regression Output:		Regression Output:		Regression Output:	
	Constant	13.78708	Constant	16.45875	Constant	8.417713
	Std Err of Y Est	3.730075	Std Err of Y Est	3.759185	Std Err of Y Est	2.929116
	R Squared	0.073381	R Squared	0.024347	R Squared	0.317557
	No. of Observations	10	No. of Observations	10	No. of Observations	10
	Degrees of Freedom	8	Degrees of Freedom	8	Degrees of Freedom	8
	X Coefficient(s)	0.252474	X Coefficient(s)	0.142834	X Coefficient(s)	0.480587
	Std Err of Coef.	0.317197	Std Err of Coef.	0.319673	Std Err of Coef.	0.249085
10	Regression Output:		Regression Output:		Regression Output:	
	Constant	0.868690	Constant	1.419743	Constant	-1.43355
	Std Err of Y Est	1.963109	Std Err of Y Est	2.352720	Std Err of Y Est	1.801166
	R Squared	0.521429	R Squared	0.426616	R Squared	0.648233
	No. of Observations	10	No. of Observations	10	No. of Observations	10
	Degrees of Freedom	8	Degrees of Freedom	8	Degrees of Freedom	8
	X Coefficient(s)	0.933337	X Coefficient(s)	0.924347	X Coefficient(s)	1.113683
	Std Err of Coef.	0.316132	Std Err of Coef.	0.378873	Std Err of Coef.	0.290053
11	Regression Output:		Regression Output:		Regression Output:	
	Constant	0.879684	Constant	2.489846	Constant	-2.33593
	Std Err of Y Est	1.991064	Std Err of Y Est	2.199162	Std Err of Y Est	1.936240
	R Squared	0.615564	R Squared	0.520620	R Squared	0.716272
	No. of Observations	10	No. of Observations	10	No. of Observations	10
	Degrees of Freedom	8	Degrees of Freedom	8	Degrees of Freedom	8
	X Coefficient(s)	1.097073	X Coefficient(s)	0.997937	X Coefficient(s)	1.339592
	Std Err of Coef.	0.306524	Std Err of Coef.	0.338561	Std Err of Coef.	0.298084
12	Regression Output:		Regression Output:		Regression Output:	
	Constant	-3.03698	Constant	-2.38614	Constant	-6.07944
	Std Err of Y Est	1.970428	Std Err of Y Est	2.134919	Std Err of Y Est	1.596385
	R Squared	0.737405	R Squared	0.712597	R Squared	0.841138
	No. of Observations	10	No. of Observations	10	No. of Observations	10
	Degrees of Freedom	8	Degrees of Freedom	8	Degrees of Freedom	8
	X Coefficient(s)	1.335762	X Coefficient(s)	1.359928	X Coefficient(s)	1.486005
	Std Err of Coef.	0.281821	Std Err of Coef.	0.305347	Std Err of Coef.	0.228323

(Continued)

(Sheet 3 of 5)

Table B2 (Continued)

CASE NO.	DATA REGRESSION FOR HSI6p HSI6p		DATA REGRESSION FOR HMAXp HMAXp		DATA REGRESSION FOR HAV6p HAV6p	
13	Regression Output:		Regression Output:		Regression Output:	
	Constant	9.771659	Constant	11.61906	Constant	6.824665
	Std Err of Y Est	2.166919	Std Err of Y Est	2.296731	Std Err of Y Est	2.027670
	R Squared	0.009765	R Squared	0.000000	R Squared	0.077132
	No. of Observations	10	No. of Observations	10	No. of Observations	10
	Degrees of Freedom	8	Degrees of Freedom	8	Degrees of Freedom	8
	X Coefficient(s)	0.107571	X Coefficient(s)	-0.00016	X Coefficient(s)	0.293044
	Std Err of Coef.	0.382987	Std Err of Coef.	0.405931	Std Err of Coef.	0.358376
14	Regression Output:		Regression Output:		Regression Output:	
	Constant	6.212978	Constant	7.947105	Constant	5.191098
	Std Err of Y Est	3.015088	Std Err of Y Est	3.367547	Std Err of Y Est	2.642102
	R Squared	0.153933	R Squared	0.094710	R Squared	0.195198
	No. of Observations	10	No. of Observations	10	No. of Observations	10
	Degrees of Freedom	8	Degrees of Freedom	8	Degrees of Freedom	8
	X Coefficient(s)	0.578881	X Coefficient(s)	0.490282	X Coefficient(s)	0.585692
	Std Err of Coef.	0.479822	Std Err of Coef.	0.535913	Std Err of Coef.	0.420465
15	Regression Output:		Regression Output:		Regression Output:	
	Constant	7.435677	Constant	9.403617	Constant	5.323545
	Std Err of Y Est	2.498026	Std Err of Y Est	2.700142	Std Err of Y Est	2.295462
	R Squared	0.112330	R Squared	0.052195	R Squared	0.208803
	No. of Observations	10	No. of Observations	10	No. of Observations	10
	Degrees of Freedom	8	Degrees of Freedom	8	Degrees of Freedom	8
	X Coefficient(s)	0.469514	X Coefficient(s)	0.334789	X Coefficient(s)	0.623054
	Std Err of Coef.	0.466638	Std Err of Coef.	0.504394	Std Err of Coef.	0.428799
16	Regression Output:		Regression Output:		Regression Output:	
	Constant	-6.79860	Constant	-6.86758	Constant	-6.85690
	Std Err of Y Est	1.341167	Std Err of Y Est	1.479249	Std Err of Y Est	1.264689
	R Squared	0.742663	R Squared	0.716867	R Squared	0.754505
	No. of Observations	10	No. of Observations	10	No. of Observations	10
	Degrees of Freedom	8	Degrees of Freedom	8	Degrees of Freedom	8
	X Coefficient(s)	1.465973	X Coefficient(s)	1.514481	X Coefficient(s)	1.426568
	Std Err of Coef.	0.305095	Std Err of Coef.	0.336507	Std Err of Coef.	0.287698

(Continued)

(Sheet 4 of 5)

Table B2 (Concluded)

CASE NO.	DATA REGRESSION FOR HSIGp	HSIGp	DATA REGRESSION FOR HMAXp	HMAXp	DATA REGRESSION FOR HAV6p	HAV6p
17	Regression Output:		Regression Output:		Regression Output:	
	Constant	22.01254	Constant	25.06453	Constant	18.28792
	Std Err of Y Est	3.478549	Std Err of Y Est	3.777374	Std Err of Y Est	3.175028
	R Squared	0.032563	R Squared	0.047925	R Squared	0.014906
	No. of Observations	10	No. of Observations	10	No. of Observations	10
	Degrees of Freedom	8	Degrees of Freedom	8	Degrees of Freedom	8
	X Coefficient(s)	-0.50219	X Coefficient(s)	-0.66689	X Coefficient(s)	-0.30733
	Std Err of Coef.	0.967772	Std Err of Coef.	1.050909	Std Err of Coef.	0.883329
18	Regression Output:		Regression Output:		Regression Output:	
	Constant	-6.99470	Constant	-11.7862	Constant	-1.05130
	Std Err of Y Est	2.281651	Std Err of Y Est	2.537640	Std Err of Y Est	2.338821
	R Squared	0.358610	R Squared	0.423385	R Squared	0.191192
	No. of Observations	10	No. of Observations	10	No. of Observations	10
	Degrees of Freedom	8	Degrees of Freedom	8	Degrees of Freedom	8
	X Coefficient(s)	1.382629	X Coefficient(s)	1.762225	X Coefficient(s)	0.921544
	Std Err of Coef.	0.653748	Std Err of Coef.	0.727095	Std Err of Coef.	-0.670128

(Sheet 5 of 5)

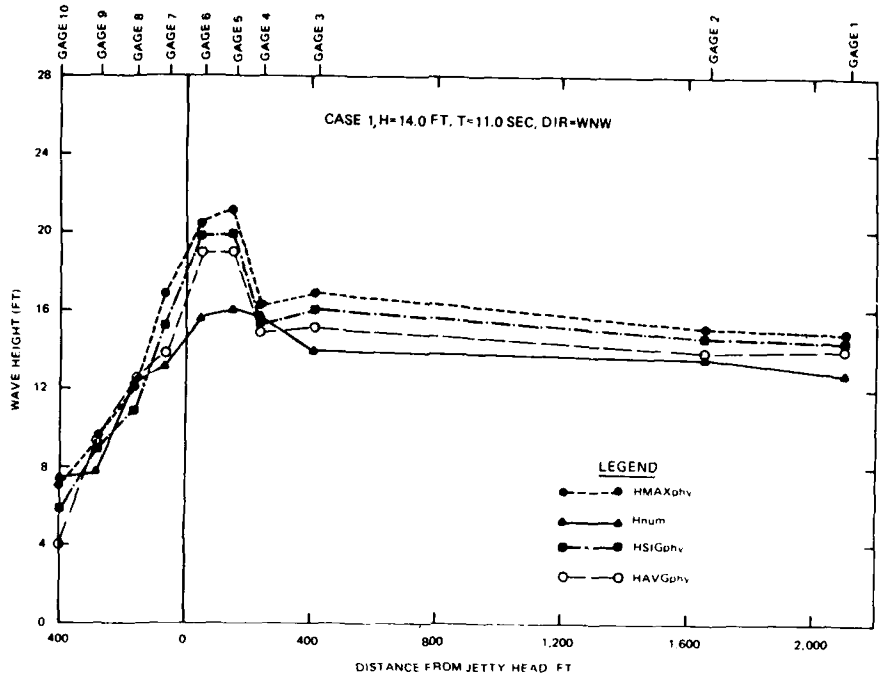


Figure B1. Wave height versus distance from the jetty head, Case 1

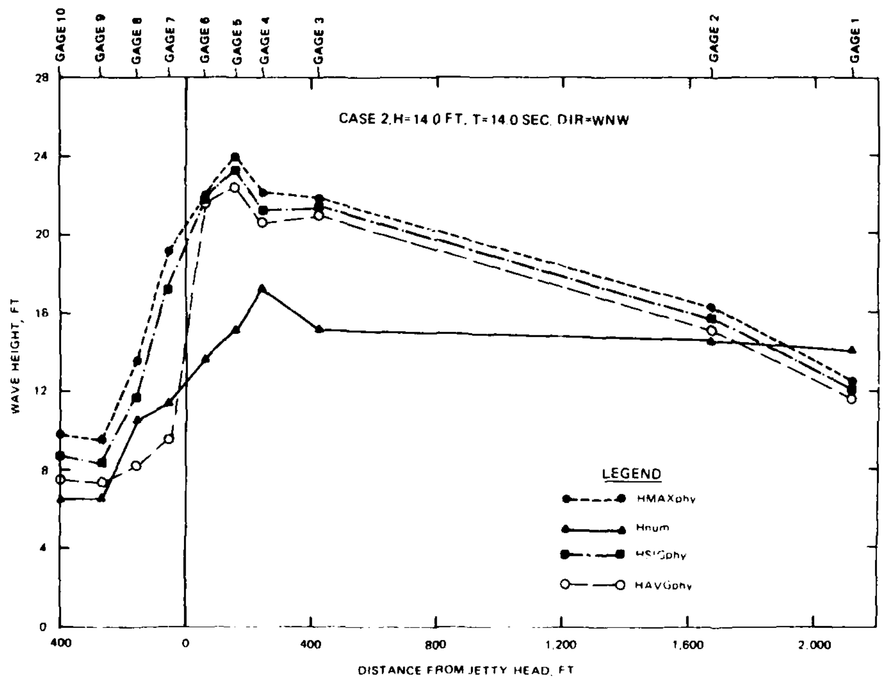


Figure B2. Wave height versus distance from the jetty head, Case 2

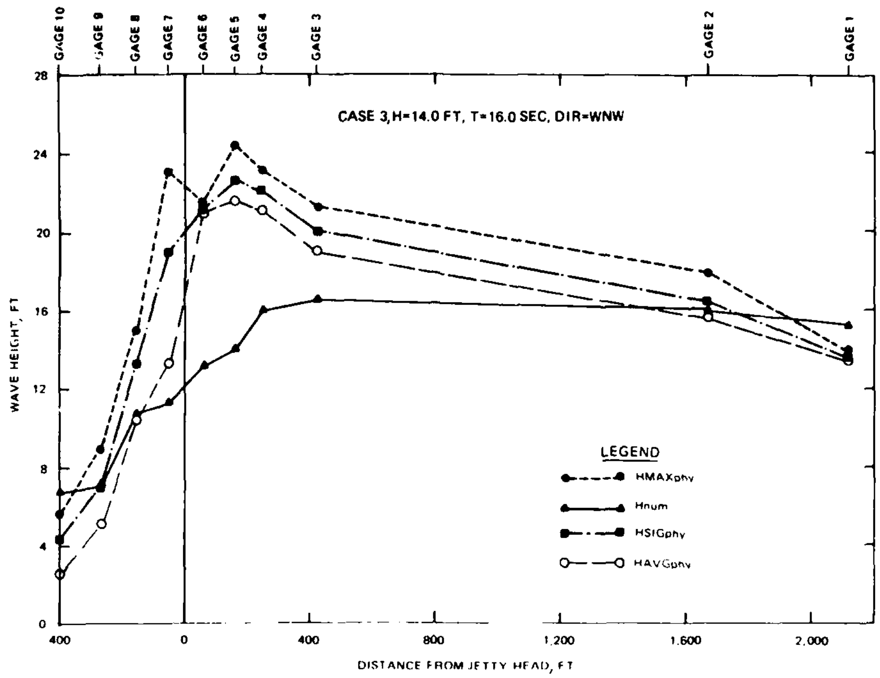


Figure B3. Wave height versus distance from the jetty head, Case 3

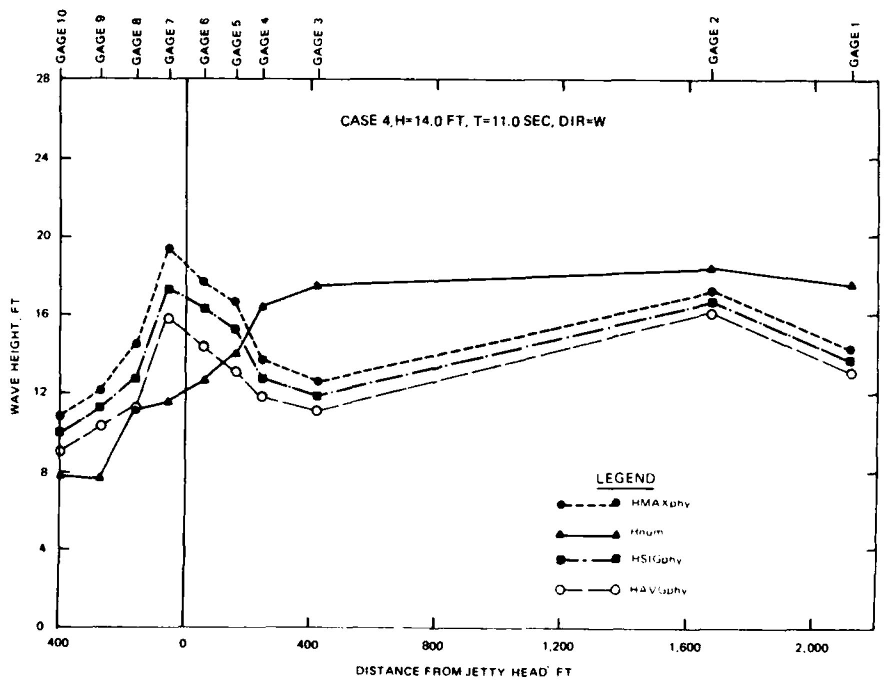


Figure B4. Wave height versus distance from the jetty head, Case 4

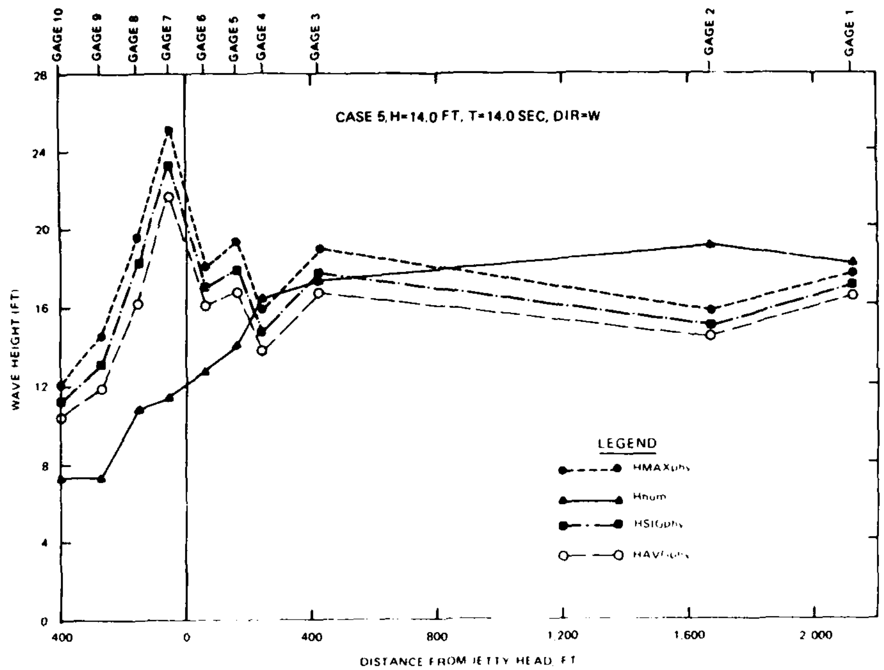


Figure B5. Wave height versus distance from the jetty head, Case 5

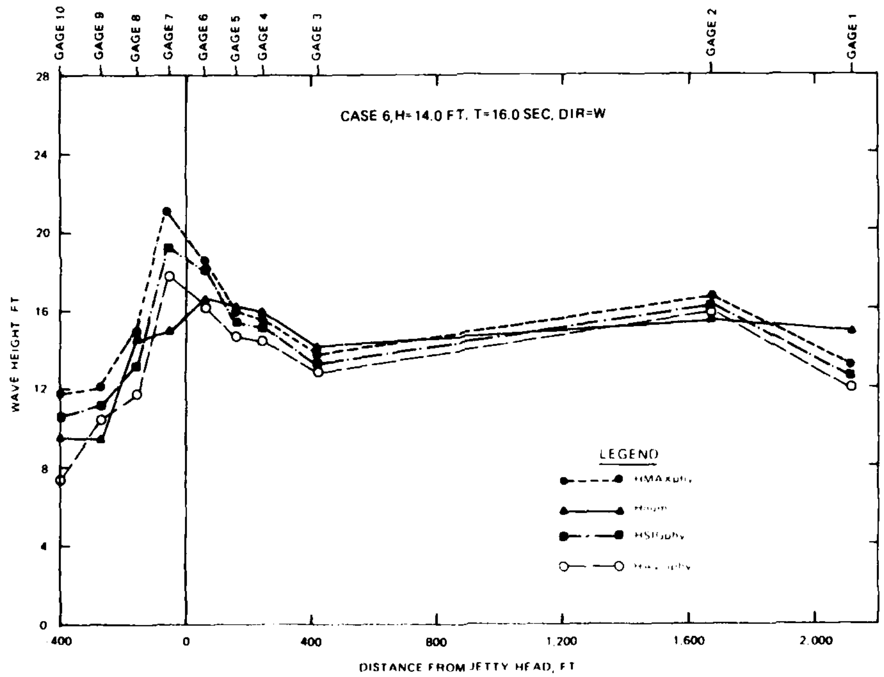


Figure B6. Wave height versus distance from the jetty head, Case 6

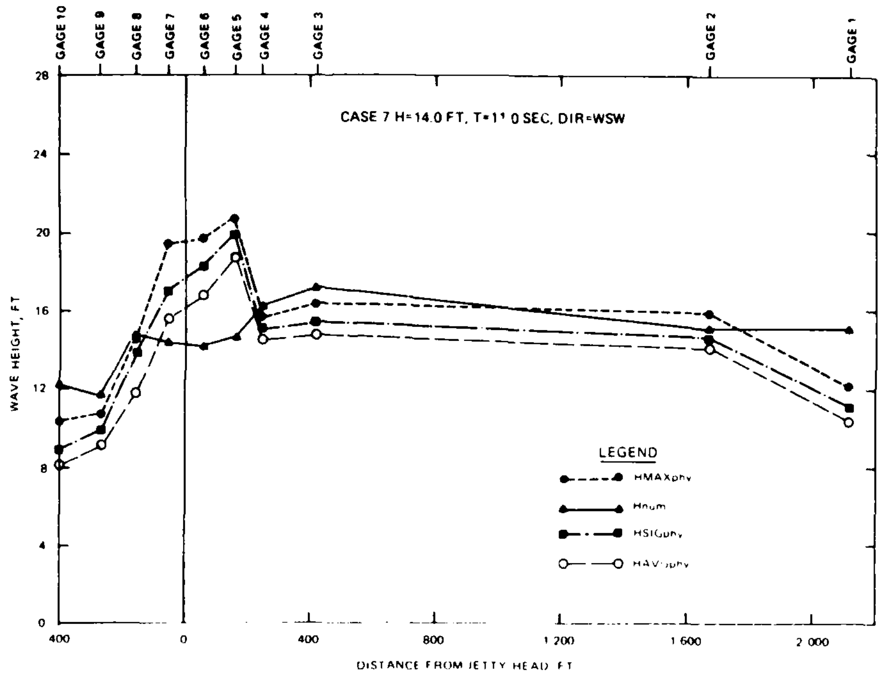


Figure B7. Wave height versus distance from the jetty head, Case 7

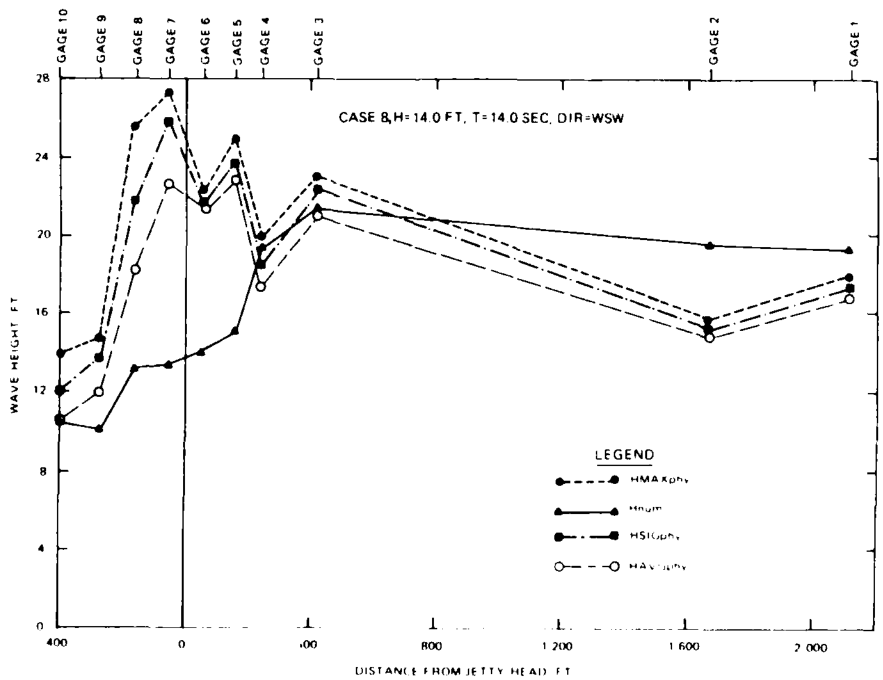


Figure B8. Wave height versus distance from the jetty head, Case 8

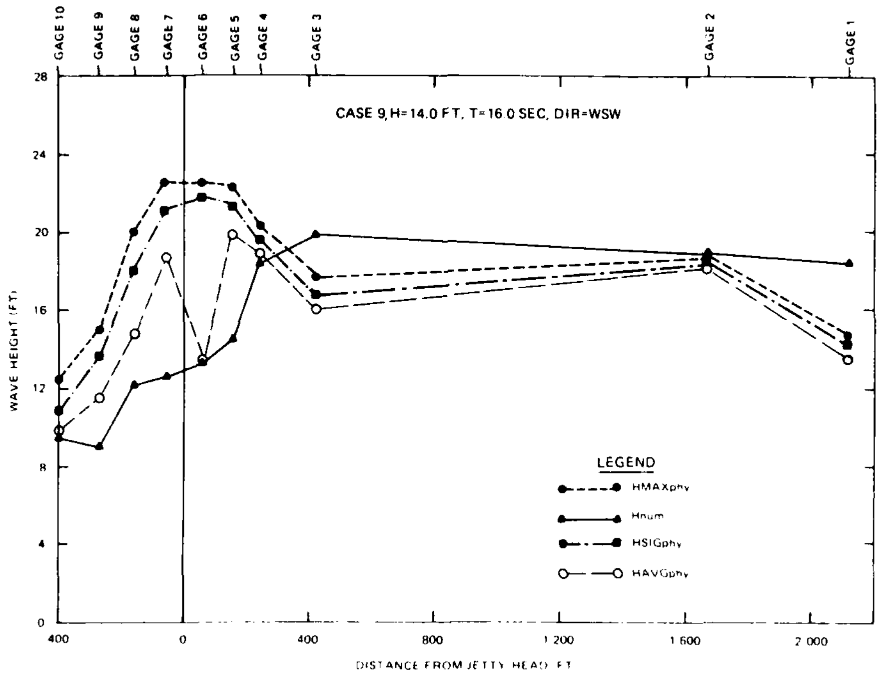


Figure B9. Wave height versus distance from the jetty head, Case 9

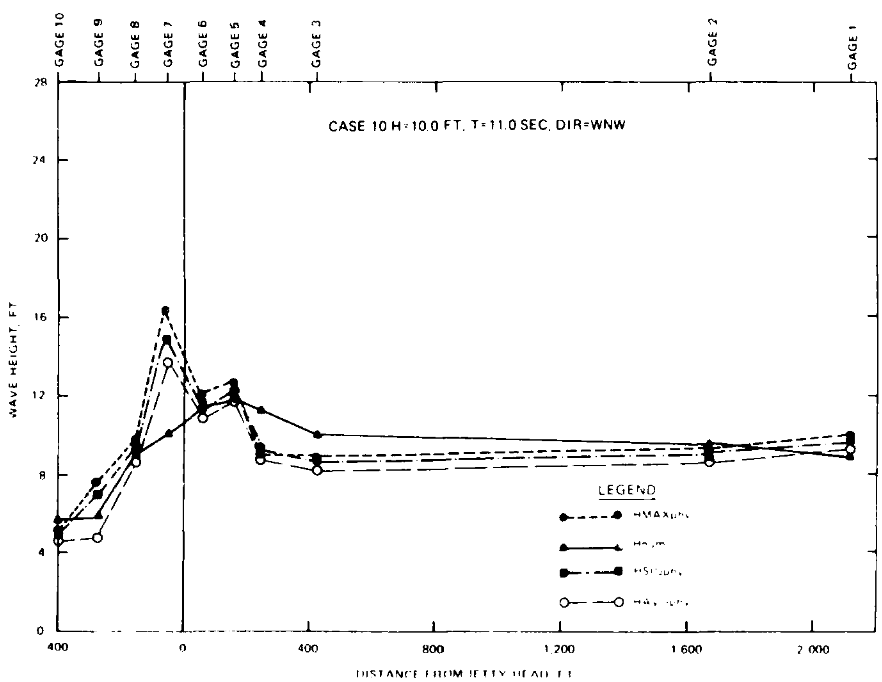


Figure B10. Wave height versus distance from the jetty head, Case 10

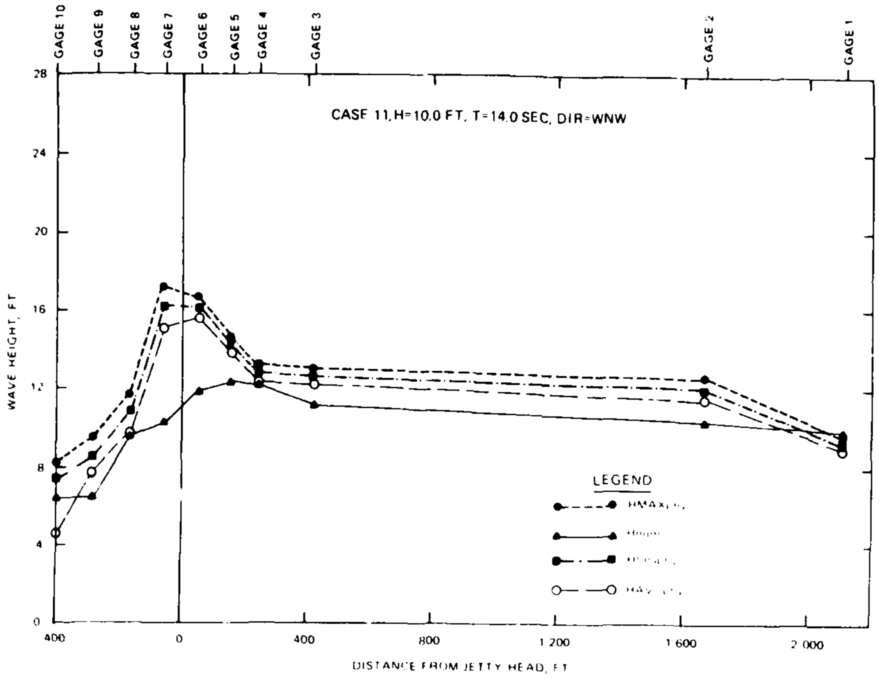


Figure B11. Wave height versus distance from the jetty head, Case 11

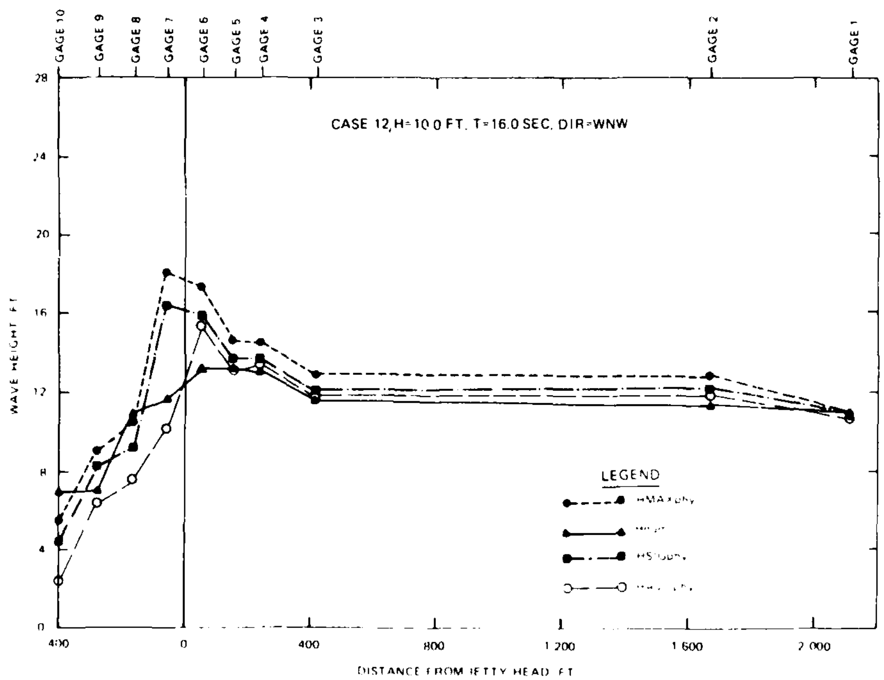


Figure B12. Wave height versus distance from the jetty head, Case 12

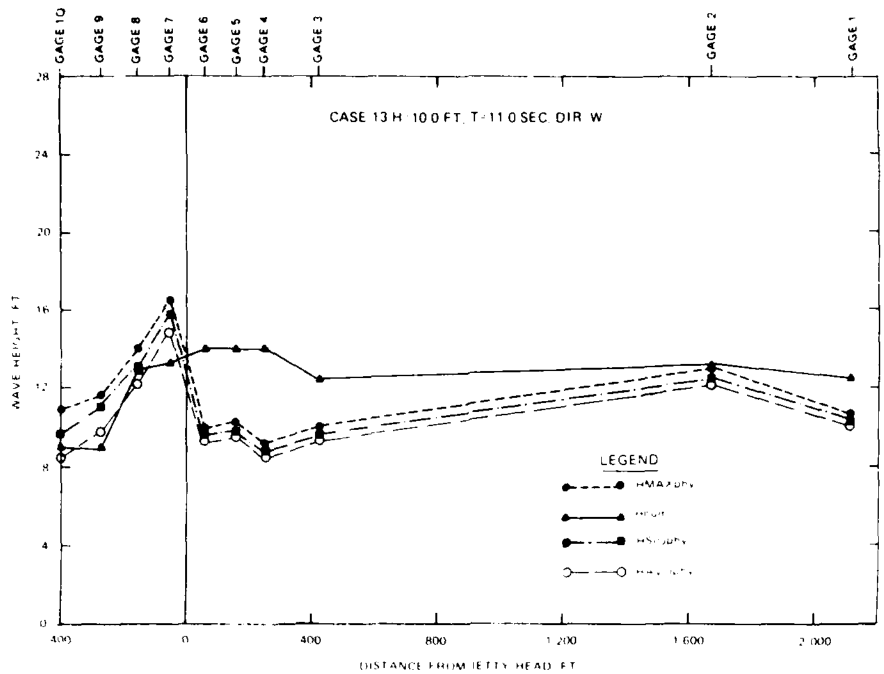


Figure B13. Wave height versus distance from the jetty head, Case 13

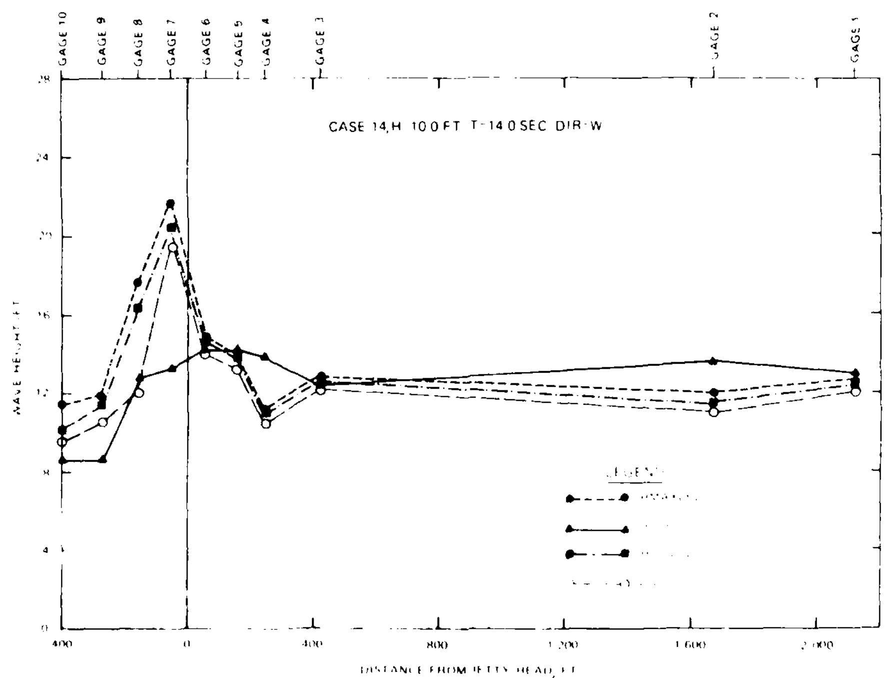


Figure B14. Wave height versus distance from the jetty head, Case 14

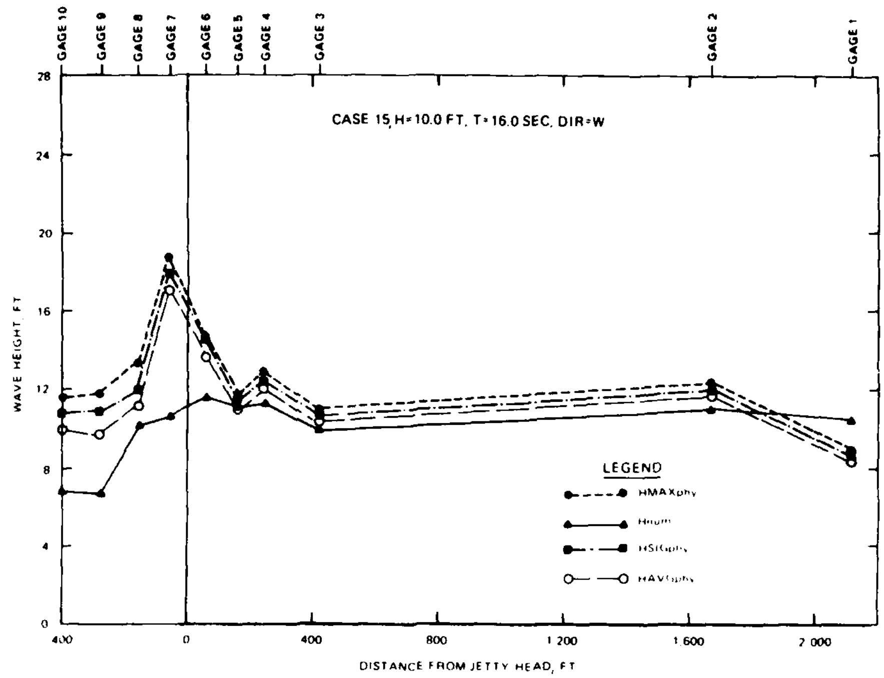


Figure B15. Wave height versus distance from the jetty head, Case 15

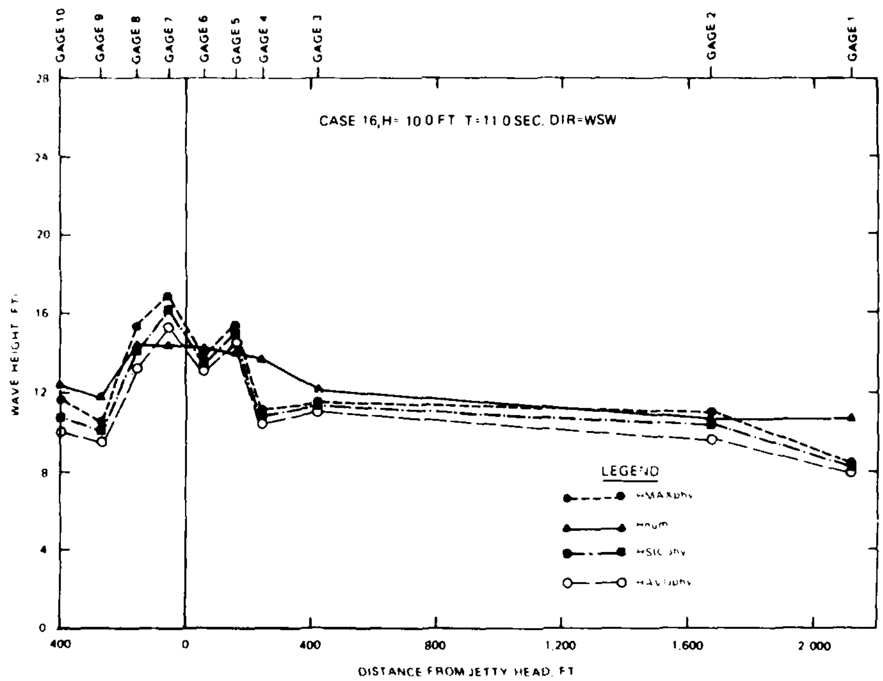


Figure B16. Wave height versus distance from the jetty head, Case 16

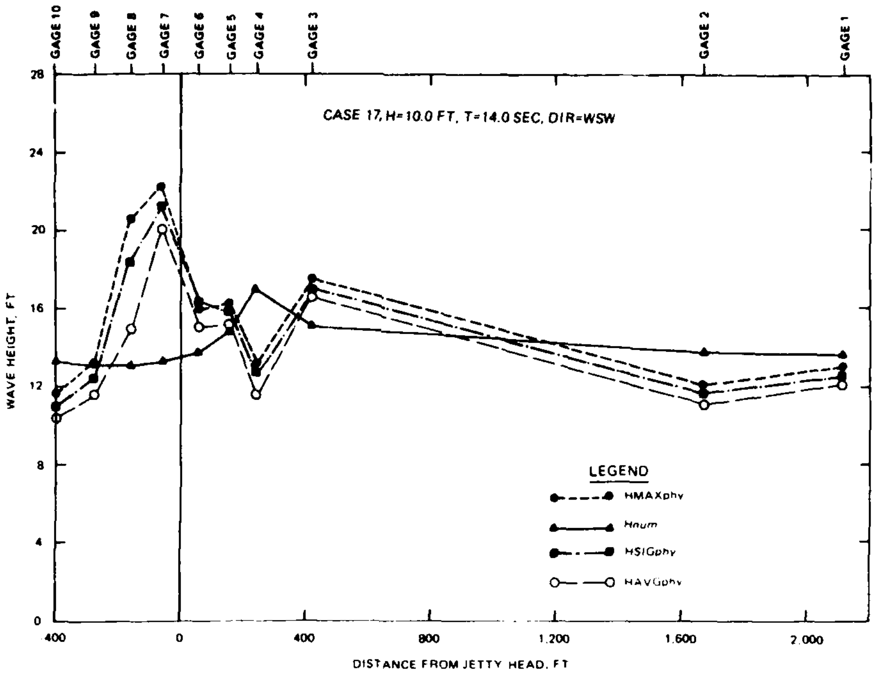


Figure B17. Wave height versus distance from the jetty head, Case 17

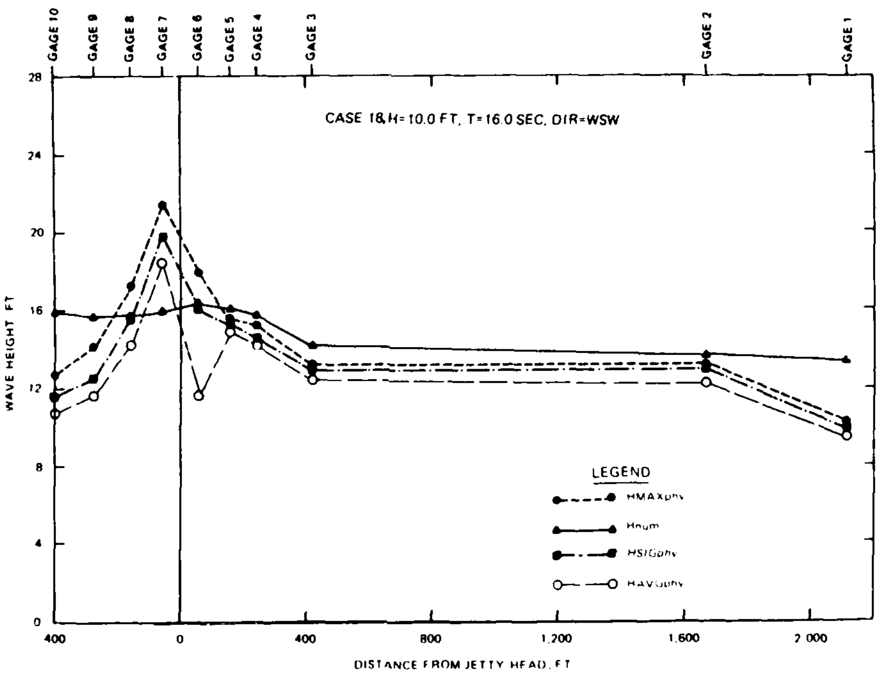


Figure B18. Wave height versus distance from the jetty head, Case 18

CASE 1, H = 14.0 FT, T=11.0 SEC, DIR=WNW

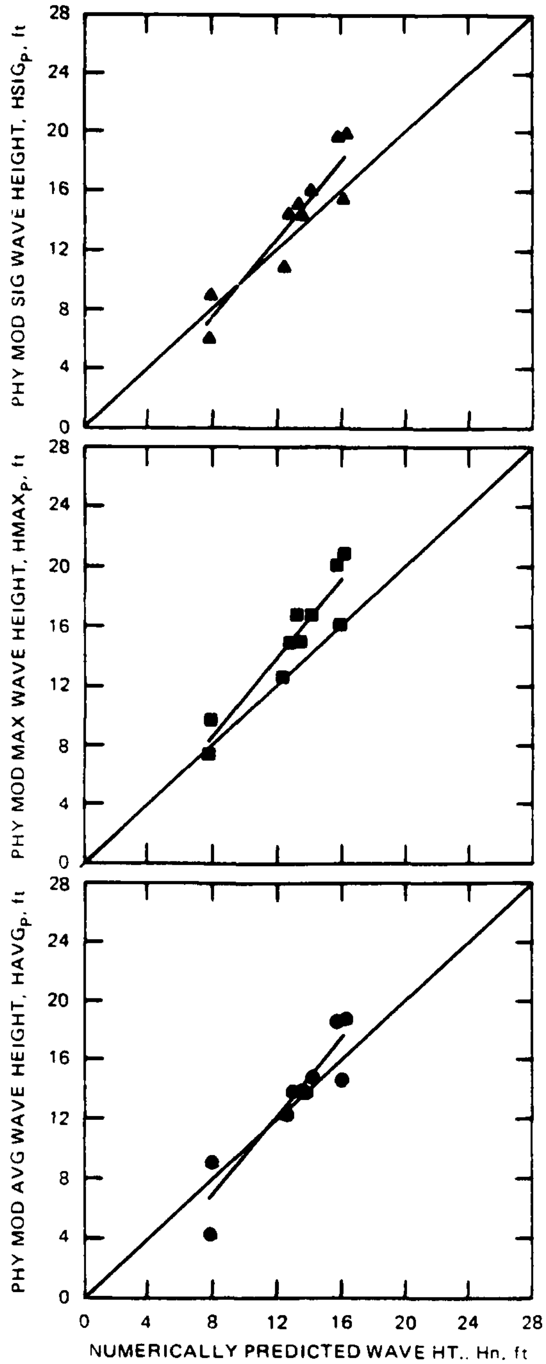


Figure B19. Physically modeled wave heights versus numerically predicted wave heights, Case 1

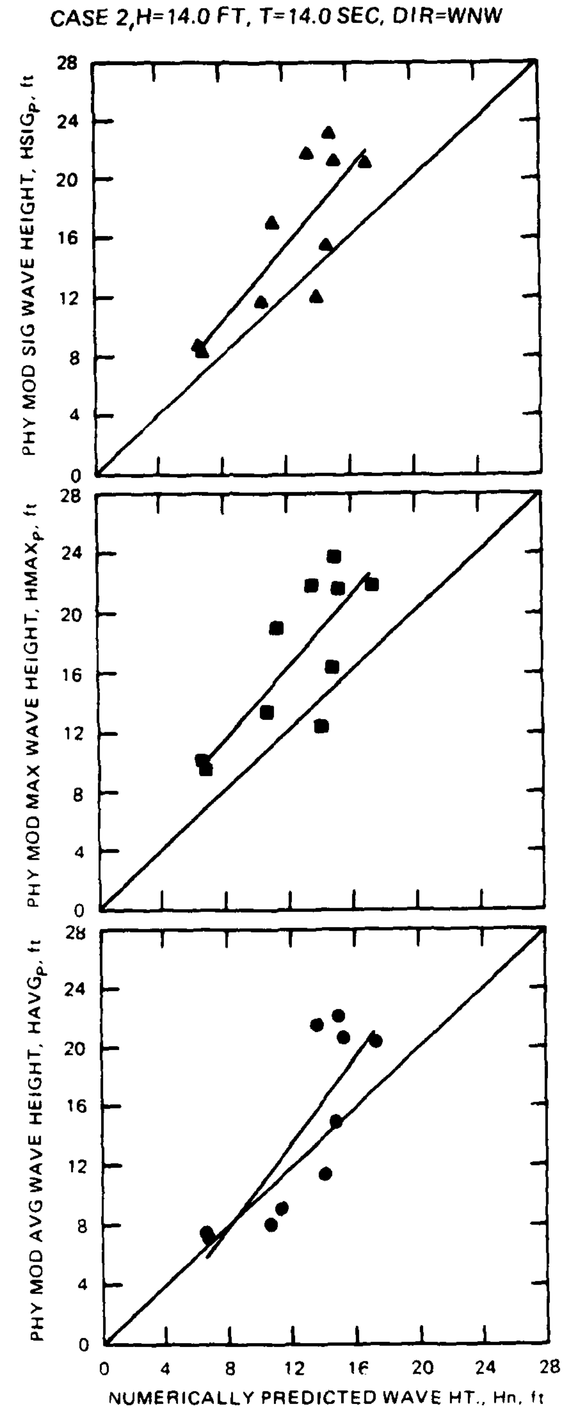


Figure B20. Physically modeled wave heights versus numerically predicted wave heights, Case 2

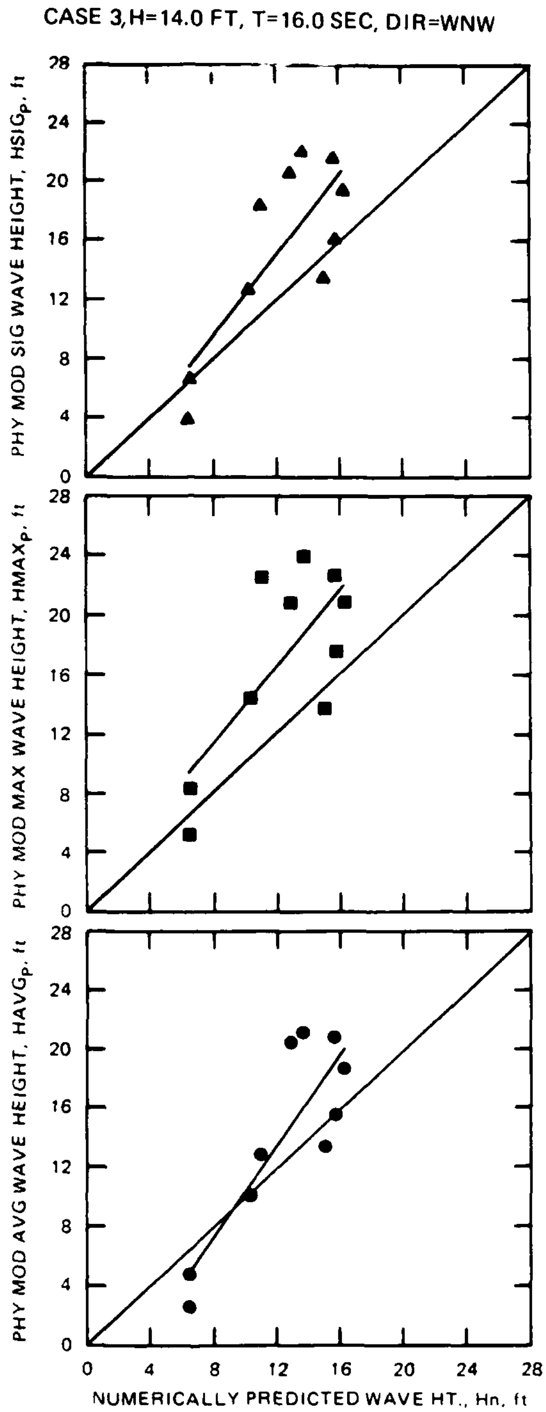


Figure B21. Physically modeled wave heights versus numerically predicted wave heights, Case 3

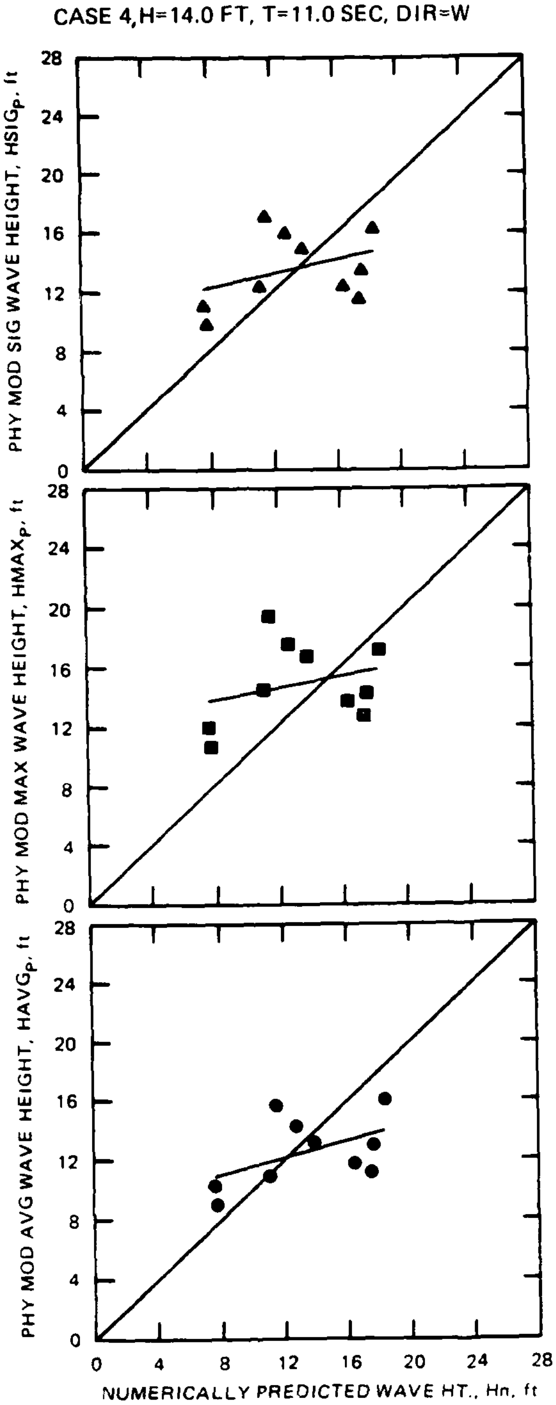


Figure B22. Physically modeled wave heights versus numerically predicted wave heights, Case 4

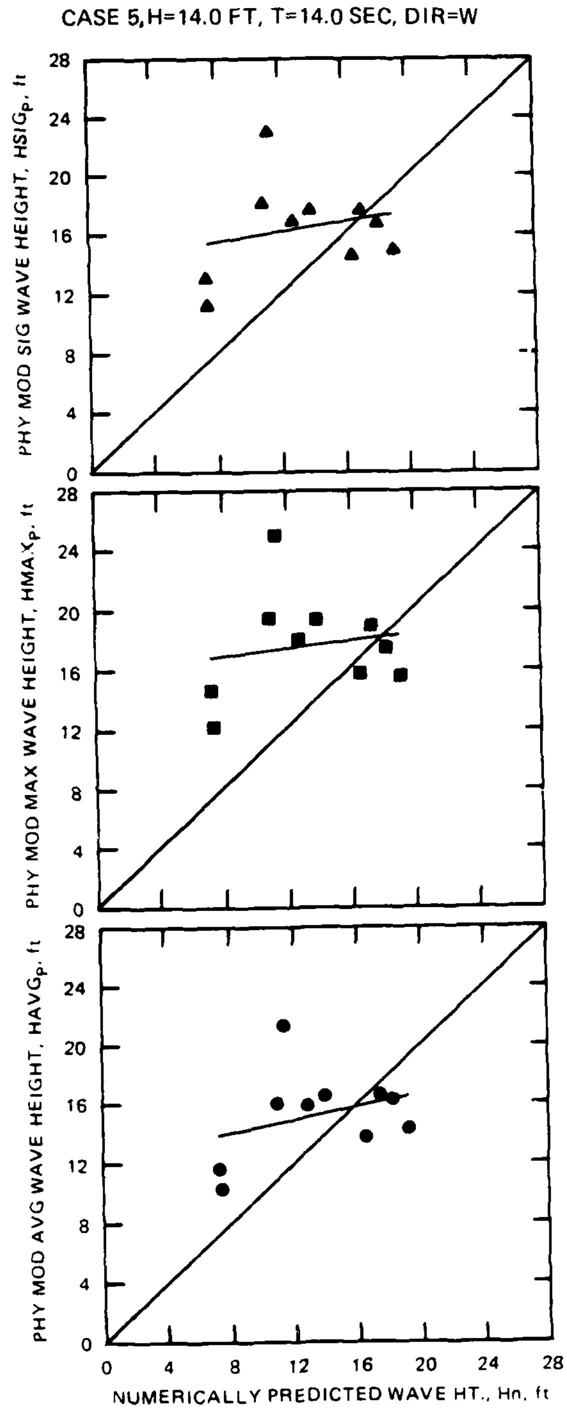


Figure B23. Physically modeled wave heights versus numerically predicted wave heights, Case 5

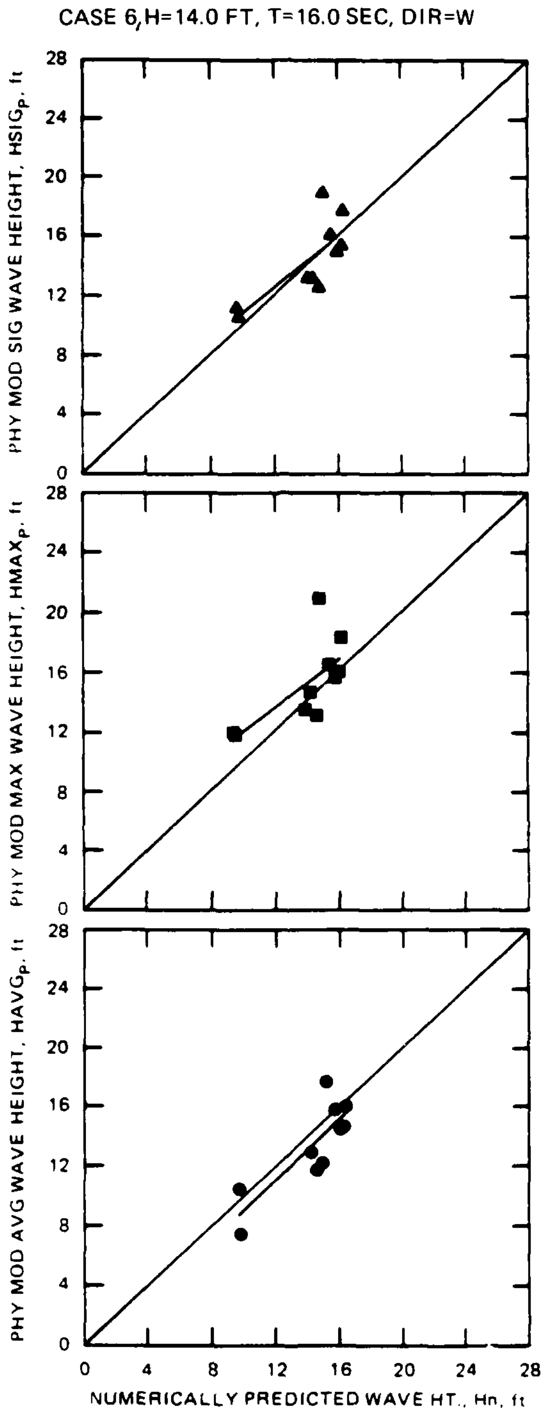


Figure B24. Physically modeled wave heights versus numerically predicted wave heights, Case 6

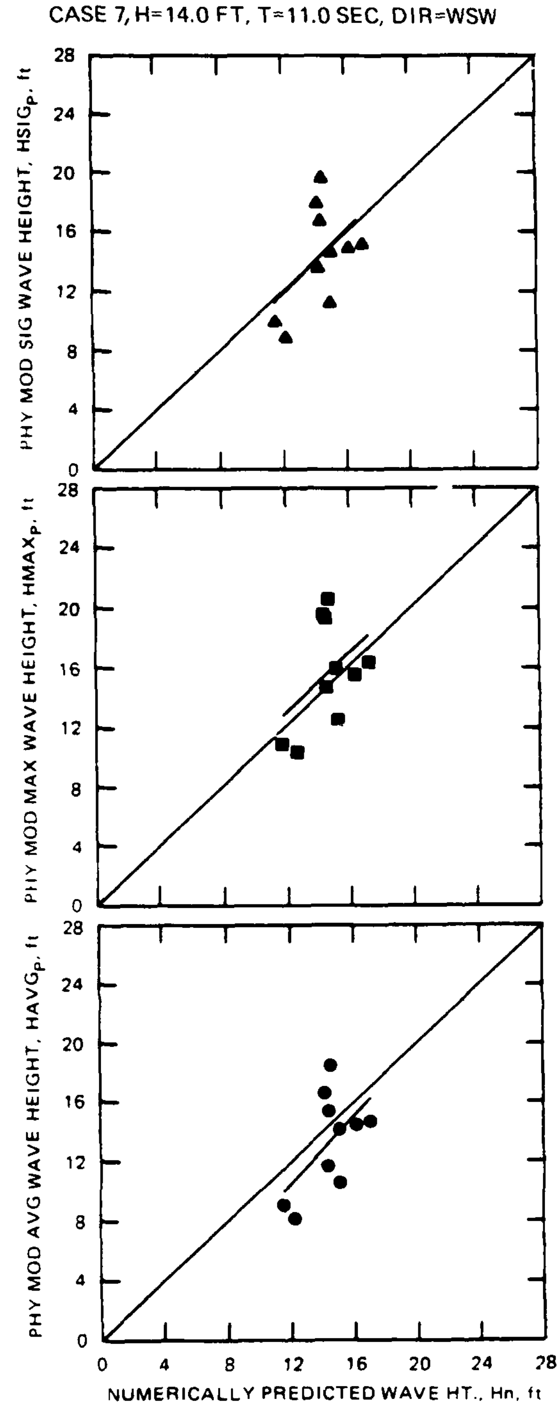


Figure B25. Physically modeled wave heights versus numerically predicted wave heights, Case 7

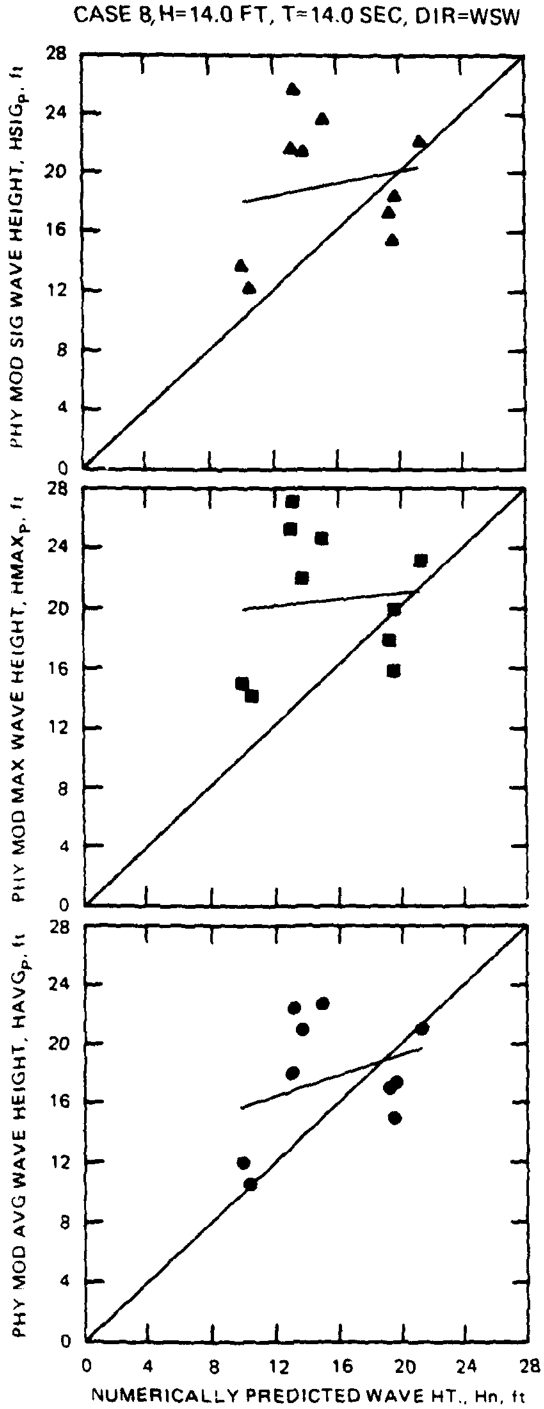


Figure B26. Physically modeled wave heights versus numerically predicted wave heights, Case 8

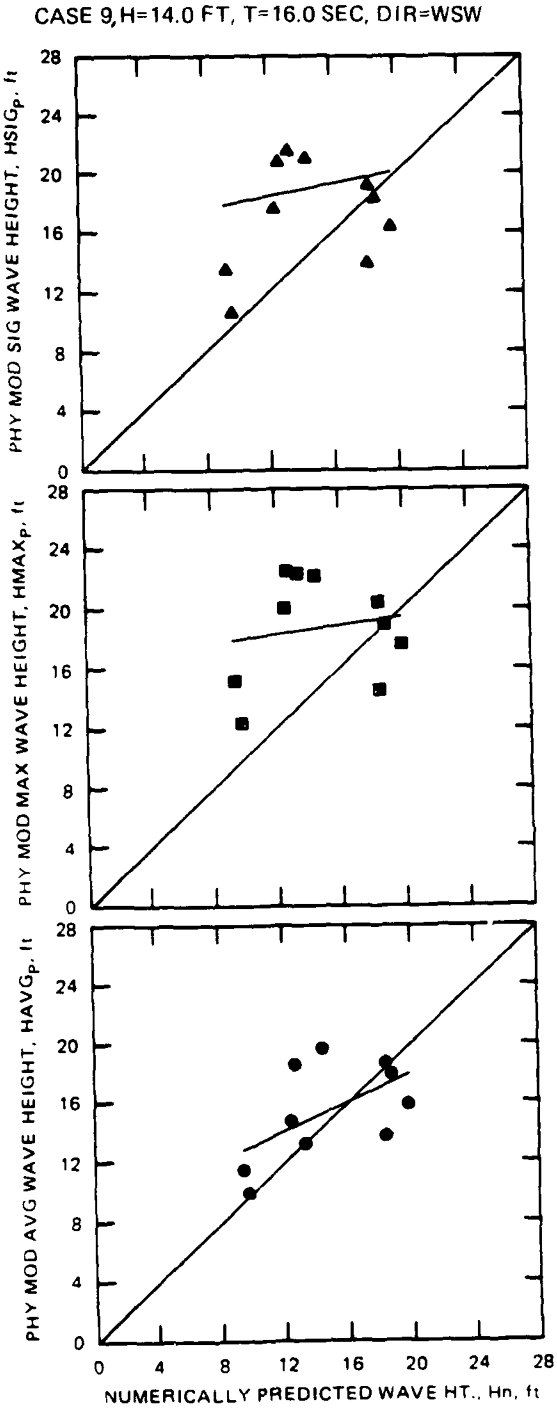


Figure B27. Physically modeled wave heights versus numerically predicted wave heights, Case 9

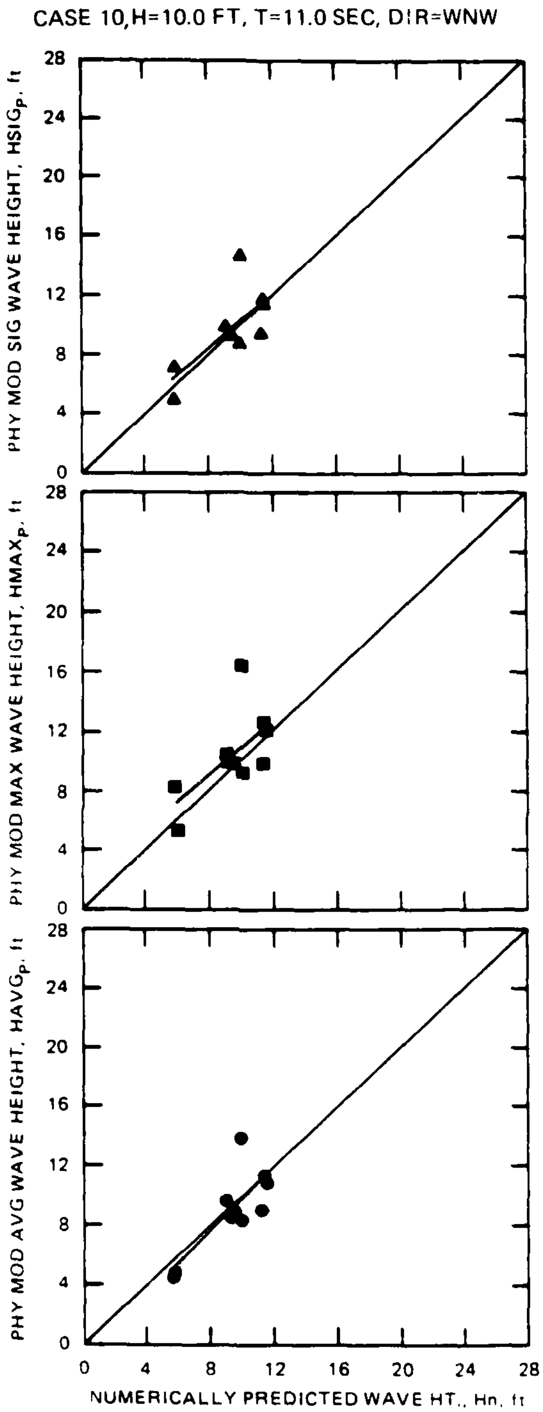


Figure B28. Physically modeled wave heights versus numerically predicted wave heights, Case 10

CASE 11, H=10.0 FT, T=14.0 SEC, DIR=WNW

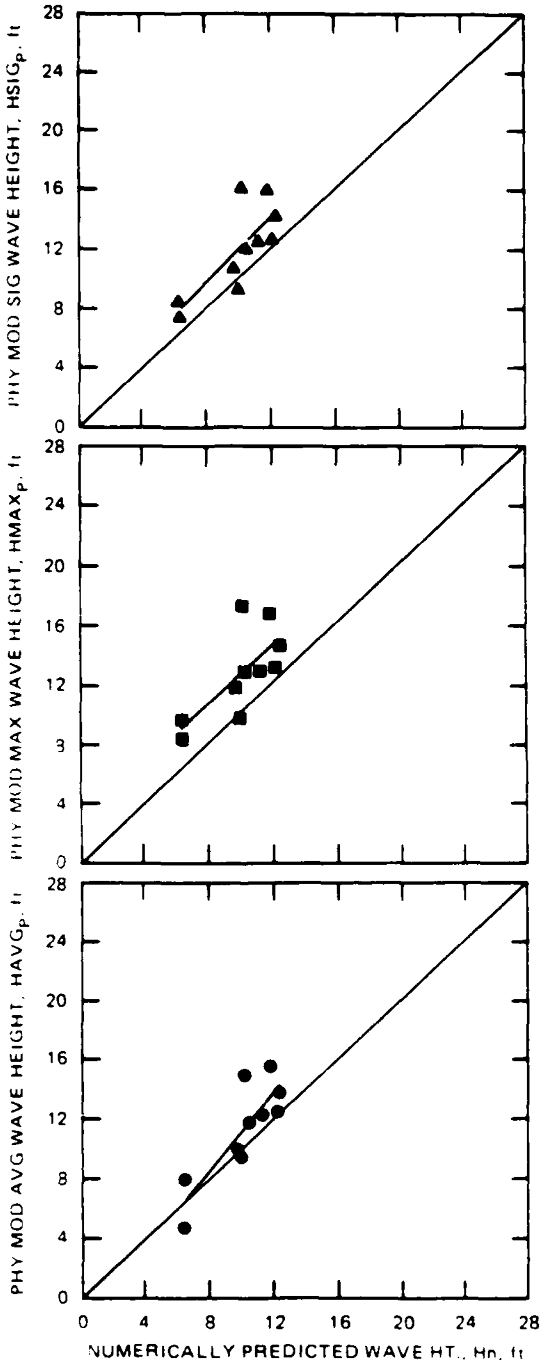


Figure B29. Physically modeled wave heights versus numerically predicted wave heights, Case 11

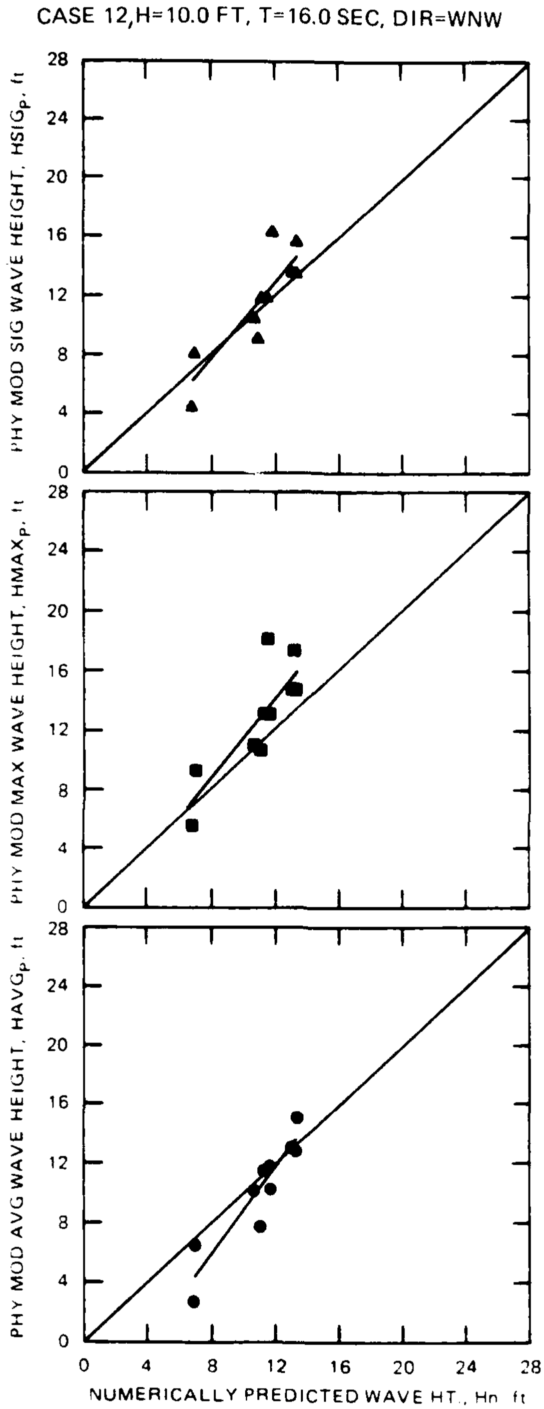


Figure B30. Physically modeled wave heights versus numerically predicted wave heights, Case 12

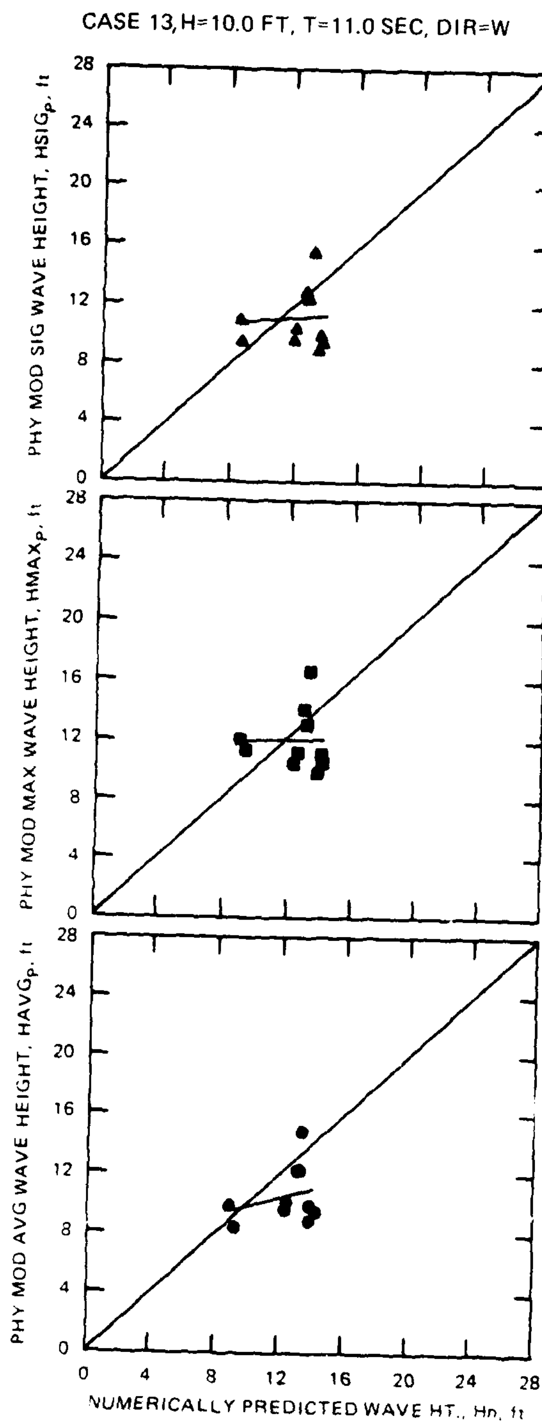


Figure B31. Physically modeled wave heights versus numerically predicted wave heights, Case 13

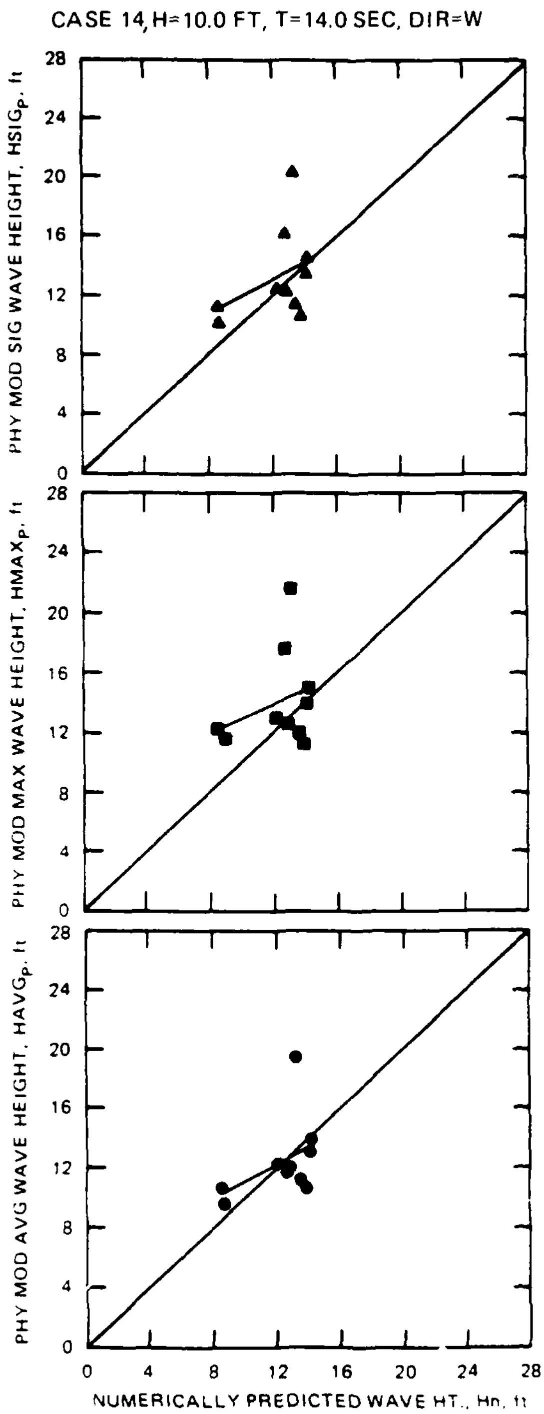


Figure B32. Physically modeled wave heights versus numerically predicted wave heights, Case 14

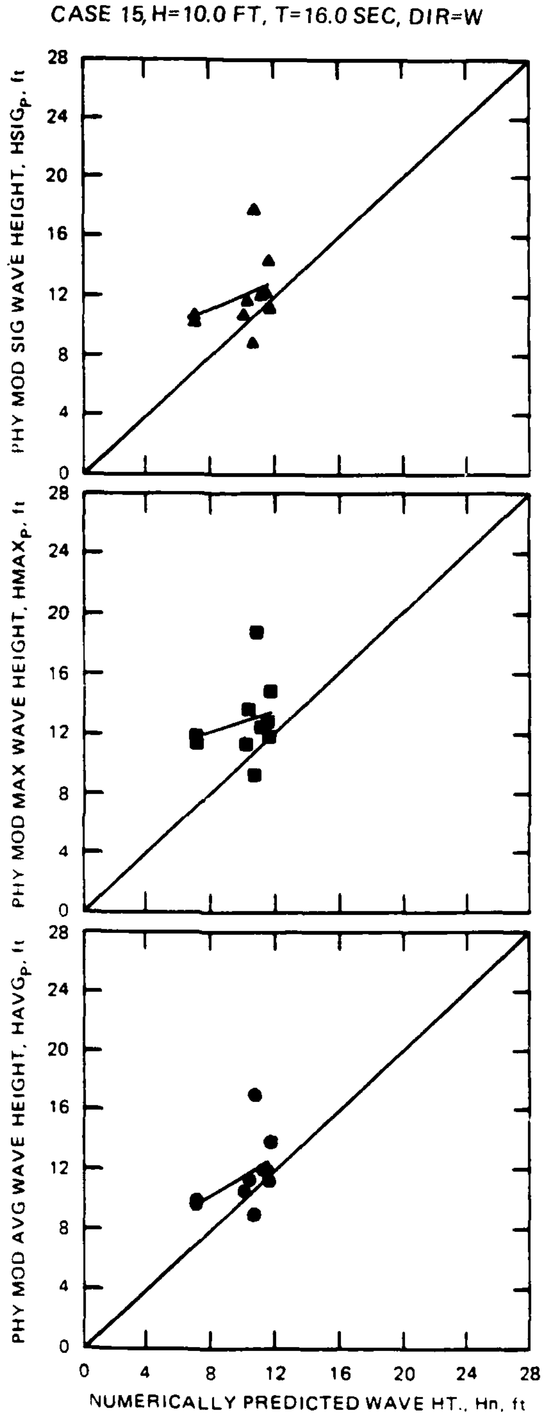


Figure B33. Physically modeled wave heights versus numerically predicted wave heights, Case 15

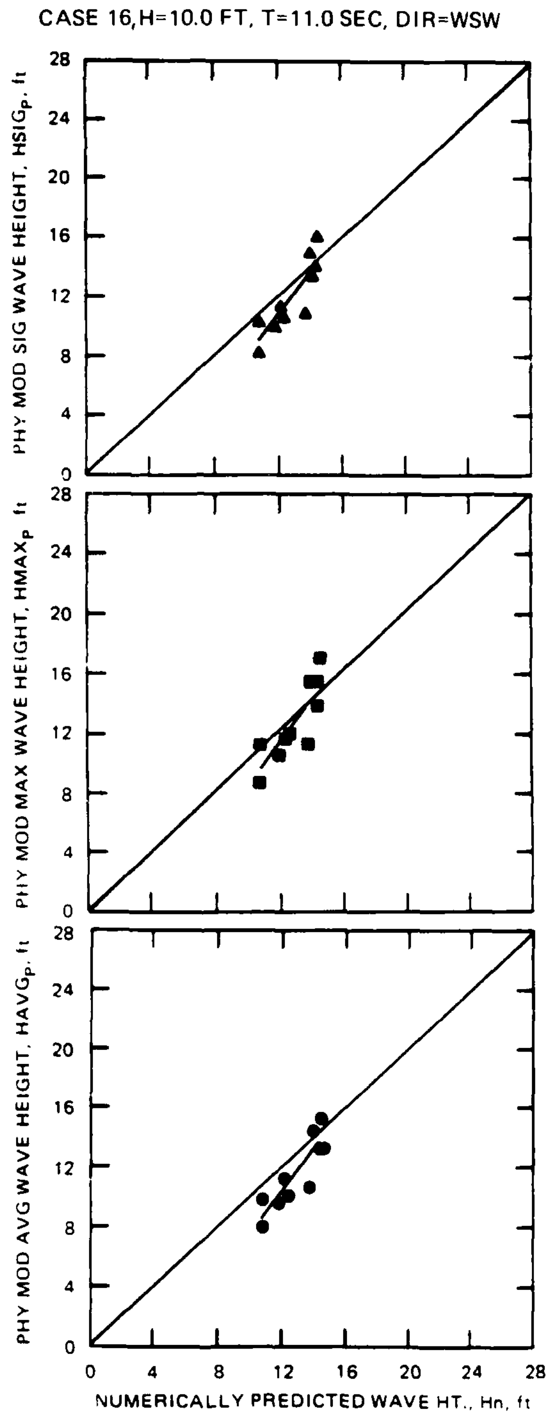


Figure B34. Physically modeled wave heights versus numerically predicted wave heights, Case 16

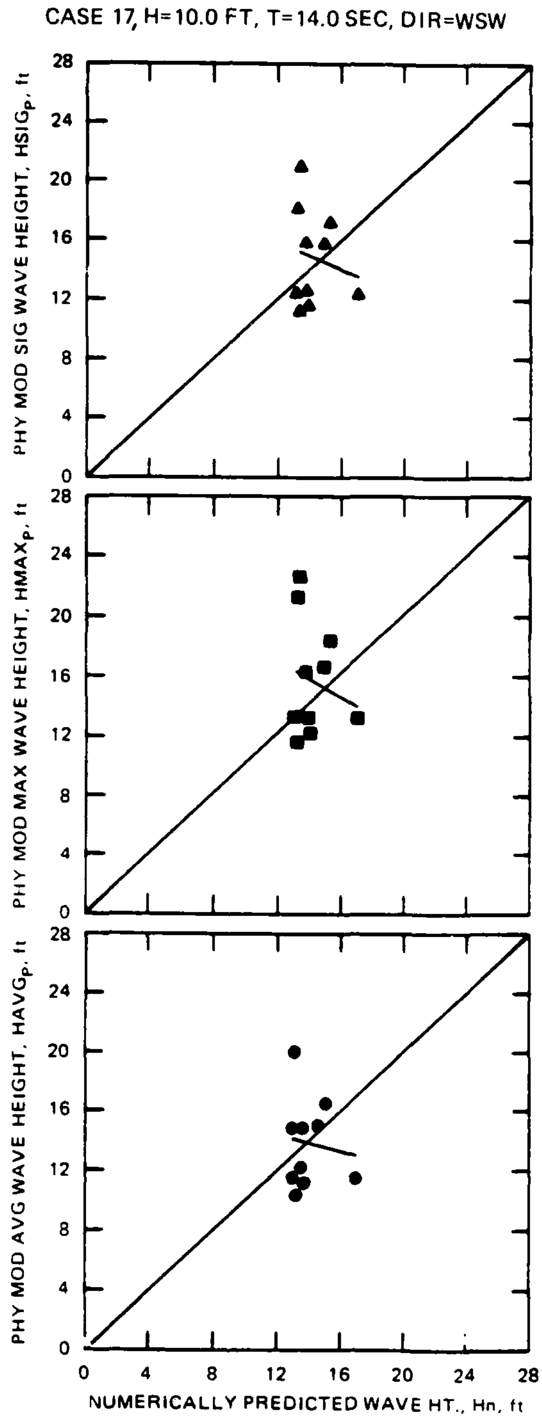


Figure B35. Physically modeled wave heights versus numerically predicted wave heights, Case 17

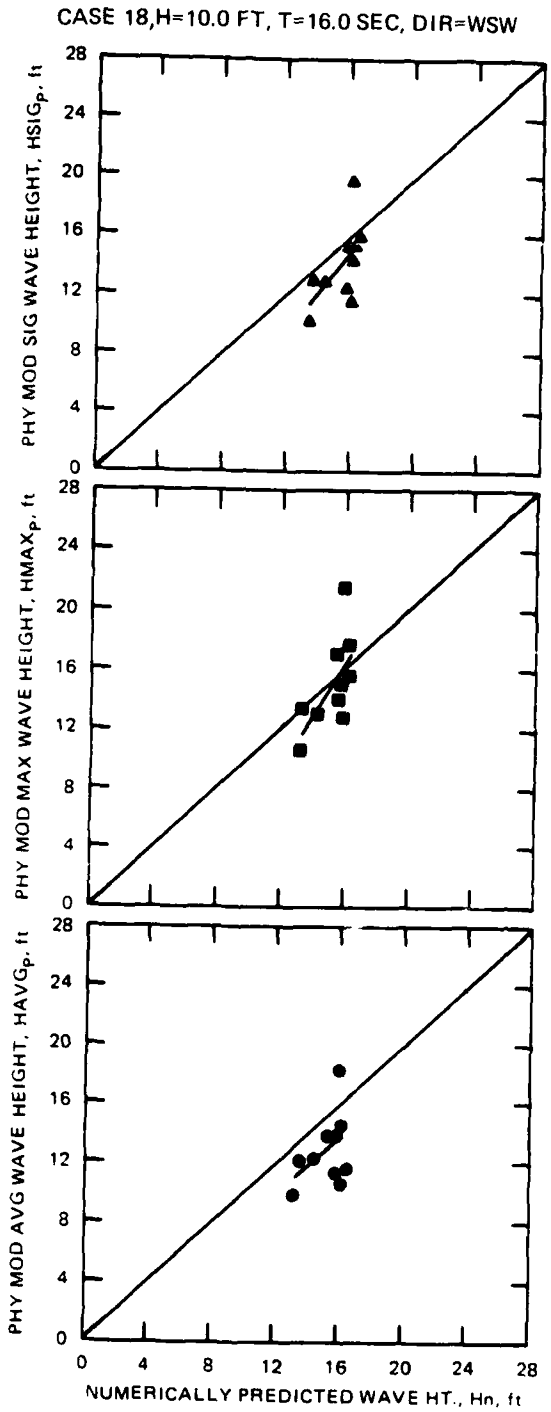


Figure B36. Physically modeled wave heights versus numerically predicted wave heights, Case 18