TECHNICAL REPORT NO. 2-697

# STABILITY OF RUBBLE-MOUND BREAKWATERS, NASSAU HARBOR NASSAU, NEW PROVIDENCE, BAHAMAS

Hydraulic Model Investigation

by

R. A. Jackson

October 1965

Sponsored by

Ministry for Maritime Affairs
Nassau, Bahamas

Conducted by

U. S. Army Engineer Waterways Experiment Station
CORPS OF ENGINEERS
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ARMY-MRC VICKSBURG, MISS.

## FOREWORD

The model investigation described herein was requested by Frederick Snare Corporation, Engineers and Contractors, New York, N. Y., consultants for the Ministry of Maritime Affairs, Nassau, New Providence, Bahamas, in a letter dated 26 April 1963. Authority to perform the investigation was granted by the Chief of Engineers in a teletype to the U. S. Army Engineer Waterways Experiment Station (WES) dated 14 November 1963. The tests were conducted at the WES during the period September-December 1964.

Mr. A. D. Quinn, Vice-President and Chief Engineer, Frederick Snare Corporation, attended conferences at WES in connection with the model study.

The model study was conducted in the Wave Dynamics Section, Water Waves Branch, Hydraulics Division, WES, under the direction of Mr. E. P. Fortson, Jr., Chief of the Hydraulics Division, and Mr. R. Y. Hudson, Chief of the Water Waves Branch. Tests were conducted by Mr. R. A. Jackson, Project Engineer, assisted by Mr. G. G. Stout, Engineering Technician. This report was prepared by Mr. Jackson.

Directors of the WES during the conduct of this investigation and publication of this report were Col. Alex G. Sutton, Jr., CE, and Col. John R. Oswalt, Jr., CE. Technical Director was Mr. J. B. Tiffany.

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## GLOSSARY

| | |
|---|---|
| $d$ | Water depth referred to stillwater level, ft |
| $H$ | Wave height, ft |
| $H_D$ | Selected design wave height, ft |
| $H_{D=0}$ | Selected design wave height based on no-damage criterion, ft |
| $k$ | Coefficient |
| $K$ | Experimental coefficient |
| $L$ | Length, wave length, ft |
| $n$ | Number of layers of armor units |
| $r$ | Thickness of cover layer, measured perpendicular to slope face, ft |
| $S$ | Specific gravity ($S_r = \gamma_r/\gamma_w$) |
| Ssd | Shattuck scale datum (equals -2.991 ft mean sea level) |
| swl | Stillwater level |
| $T$ | Wave period, sec |
| $W$ | Weight, lb or tons |
| $\alpha$ | Angle of breakwater slope, measured from horizontal, deg |
| $\gamma$ | Specific weight, $lb/ft^3$ |
| Subscript m | refers to model |
| Subscript p | refers to prototype |
| Subscript r | refers to armor stone |
| Subscript w | refers to water |
| Subscript $\Delta$ | refers to shape factor of armor stone |

## SUMMARY

Four series of tests were conducted on 1:36-, 1:45-, and 1:48-scale models of breakwater trunks and heads constructed of quadripods, tetrapods, and tribar armor units to obtain data from which alternate designs could be developed for construction of breakwaters at Nassau Harbor, Nassau, New Providence, Bahamas. Design criteria for both head and trunk were desired for conditions of (a) no damage, (b) overtopping, and (c) waves breaking directly on the structure. Although proposed designs involved protective cover layers of tetrapods and tribars, the available range of size of model tetrapods was not sufficient for testing, and quadripods, which have nearly the same stability as tetrapods, were used. Another series of tests was conducted on a 1:56-scale model to determine the cross-sectional area of the rubble mound on the seaside of the vertical-wall structure that would reduce the maximum shock pressures resulting from breaking waves to pressures equivalent to those obtained from the Sainflou theory. Also, a method of placing tetrapods and tribars in a rectangular trench at the toe of the slope of the cover layer was developed.

Results of this investigation, and conclusions based thereon and on similar tests conducted at WES, are as follows:

a. Because of the very large cross section required, it is not practical to construct a mound of rubble on the seaside of the impermeable vertical wall capable of reducing the maximum wave-generated shock pressures to values obtained from the Sainflou theory.

b. The stability of the east breakwater proposed for Nassau Harbor was not investigated; however, the design coefficients obtained from tests on the west breakwater are applicable for design of similar armor layers on the east breakwater provided the geometry of the east breakwater is similar to that of the west breakwater.

c. A $K_\Delta$ of 8 in the WES breakwater stability equation is adequate for design of a tetrapod armor layer for the trunk of the west breakwater when the crown elevation of the armor layer is +10 ft referred to the Shattuck scale datum (Ssd), and the crown elevation of the vertical-wall bulkhead is +15 ft Ssd.

d. The results of other stability tests conducted at WES using cover layers composed of tribar and tetrapod armor units have shown that tribar armor layers designed using a $K_\Delta$ of 8 will be somewhat more stable than a tetrapod layer designed on the same basis.

e. For design of a simple conical wraparound breakwater head in which the crown elevations of both armor layer and vertical-wall bulkhead are +15 ft Ssd, a $K_\Delta$ of 5 is adequate for determining the required weight of tetrapod armor units.

f. For design of a simple conical wraparound breakwater head in which the crown elevation of the armor layer is +10 ft Ssd and the crown elevation of the vertical-wall bulkhead is +15 ft Ssd, $K_\Delta$ values of 4 and 6 are recommended for armor layers of tetrapods and tribars, respectively.

g. Stability tests showed that anchoring the armor units in a V-shaped trench at the toe of the slope would prevent en masse sliding of the armor layer.

h. Stability tests were not conducted on the breakwaters with the toe constructed in a rectangular trench, but it is believed that the rectangular trench will be as effective as the V-shaped trench tested for anchoring of the cover layer, and the dredging of the rectangular trench should be less expensive.

# STABILITY OF RUBBLE-MOUND BREAKWATERS
# NASSAU HARBOR, NASSAU, NEW PROVIDENCE, BAHAMAS

## Hydraulic Model Investigation

## PART I: INTRODUCTION

### Description of Prototype

1. Nassau Harbor (plate 1) is located on the northern coast of New Providence Island, Bahamas, about 190 miles east-southeast of Miami, Florida. The harbor area is a wide shallow channel between New Providence Island and Paradise Island. The 250-ft-wide, 25-ft-deep entrance channel is in the deeper water area between Silver Cay and Paradise Island. The turning basin, adjacent to the Prince George Wharf, is 25 ft deep with an effective width of about 800 ft. The dimensions of the entrance channel and turning basin are inadequate to provide safe navigation for the modern cruise ships serving Nassau, and these ships are forced to anchor offshore from Paradise Island. To provide better navigation and mooring facilities, a harbor development program is proposed. The principal elements of the proposed development are: (a) dredging of a 500-ft-wide entrance channel and a 1500-ft-wide turning basin to a depth of 36 ft; (b) construction of an additional pier opposite the existing Prince George Wharf; (c) construction, with the dredged material, of an artificial island in a shallow area west of the entrance channel and between Silver Cay and New Providence Island; and (d) construction of a breakwater system between Silver Cay and Paradise Island to protect the entrance channel, the inner harbor, and the artificial island from wave action.

### The Problem

2. This investigation was concerned with the design of the cover-layer units for the rubble mounds on the seaside faces and terminal ends, or heads, of the proposed vertical-wall breakwaters. Nassau Harbor is exposed to storm waves originating from directions between northwest

and northeast, and storm waves of sufficient height relative to water depths at the breakwater sites to break directly on the structures are expected to occur. The bed material at the breakwater site is relatively soft limestone with a comparatively smooth surface, and it was considered probable that the toe of the cover layer would require an anchoring trench to prevent en masse sliding of the cover-layer units. Very little information is available concerning the magnitude of wave forces and wave overtopping of rubble mounds forming the seaside face of vertical-wall breakwaters exposed to breaking waves. Previous model tests at the Waterways Experiment Station (WES) in connection with Engineering Studies (ES) item 815, "Stability of Rubble-Mound Breakwaters," [1,2*] have provided comparative design data for cover layers consisting of smooth quarrystones, tetrapods, and tribars for the no-damage and no-overtopping criteria. However, in a majority of the breakwater tests completed to date in connection with ES 815, water depths were sufficient to prevent waves from breaking directly on the structure. Also, the ES 815 studies were made on permeable rubble-mound breakwaters. Since it was desired that the Nassau breakwaters be vertical-wall structures (sheet steel piling filled with dredged limerock and capped with concrete) with rubble on the seaside and that they be designed to withstand overtopping and breaking waves, it was concluded that a model study of the Nassau Harbor breakwaters was necessary to obtain adequate design data.

## Purpose and Scope of Model Study

3. The purpose of the study reported herein was to obtain data from which alternate designs could be developed for construction of the breakwater trunks and heads. Design criteria for both head and trunk were desired for conditions of (a) no damage, (b) overtopping, and (c) waves breaking directly on the structure. Also, it was desired to determine the cross-sectional area of the seaside rubble mound that would reduce the maximum shock pressures resulting from breaking waves to pressures

* Raised numerals refer to similarly numbered items in the Literature Cited at end of text.

equivalent to those obtained from the Sainflou theory.[3] It was also desired to investigate the need for and possible designs of a toe trench to prevent en masse sliding of the cover layer.

4. It was decided that only the west breakwater would be tested in the model, since it was assumed that results obtained thereon would be equally applicable to the east breakwater, provided the two breakwaters are geometrically similar in cross section.

## PART II: THE MODEL, TEST PROCEDURES, AND TEST CONDITIONS

### Design of Model

#### Selection of model scales

5. Stability tests, wave pressure tests, and toe trench tests were conducted on section models of the head and trunk of the proposed west breakwater for Nassau Harbor. These tests were conducted in a concrete flume 119 ft long, 5 ft wide, and 4 ft deep. Sections of the breakwater were reproduced using linear scales of 1:36, 1:45, 1:48, and 1:56. Selection of these model scales was based on the size of the model armor units available compared with the estimated size of the prototype units required for stability and on the depths of water at the toe of the slope of the proposed prototype breakwater. Based on Froude's model law, linear scales of 1:36, 1:45, 1:48, and 1:56, and a specific weight scale of 1:1, the following model-to-prototype relations were derived:

| | Model-to-Prototype Scales for Linear Scales of | | | |
|---|---|---|---|---|
| Characteristic | 1:36 | 1:45 | 1:48 | 1:56 |
| Area | 1:1,296 | 1:2,025 | 1:2,304 | 1:3,136 |
| Volume | 1:46,656 | 1:91,125 | 1:110,592 | 1:175,616 |
| Time | 1:6 | 1:6.71 | 1:6.93 | 1:7.48 |
| Velocity | 1:6 | 1:6.71 | 1:6.93 | 1:7.48 |
| Unit pressure | 1:36 | 1:45 | 1:48 | 1:56 |
| Force | 1:46,656 | 1:91,125 | 1:110,592 | 1:175,616 |

6. The specific weight of the water used in the model was 62.4 $lb/ft^3$, and that of sea water is approximately 64 $lb/ft^3$. Also, the specific weights of the model armor units were not the same as those of the armor units that will be used for constructing the prototype breakwater. The relations between these variables, model to prototype, were determined from the following transference equation:

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

where subscripts m and p refer to model and prototype quantities, respectively; $W_r$ is the weight of an armor unit in lb; $\gamma_r$ is the specific weight of the armor unit in lb/ft$^3$; $L_m/L_p$ is the linear scale of the model; and $S_r$ is the specific gravity of an armor unit relative to the water in which the breakwater is constructed; i.e. $S_r = \gamma_r/\gamma_w$, where $\gamma_w$ is the specific weight of water in lb/ft$^3$.

Selection of armor units

7. Because of the need for various sizes of model armor units, and because of the possibility that either tribars or tetrapods would be used for actual construction, depending on the comparative costs of these two types of units, model tests were conducted using quadripod, tetrapod, and tribar armor units. Some desired sizes of tetrapods were not available, but tests previously conducted at WES have shown that the stability characteristics of tetrapods and quadripods are essentially the same. These tests also indicate that tribars are somewhat superior to quadripods and tetrapods.

8. The specific weight of the model armor unit was 140.4 lb/ft$^3$, and that of the prototype armor unit was 150 lb/ft$^3$. The weights of the quadripods, tetrapods, and tribars used in the model tests and their corresponding prototype equivalents based on the scale relations presented in paragraph 5 and the transference equation in paragraph 6 are as follows:

| Type of Armor Unit | Model $W_r$ lb | Prototype $W_r$ in Tons for Linear Scales of 1:36 | 1:45 | 1:48 | 1:56 |
|---|---|---|---|---|---|
| Tetrapods | 0.22 | 4.3 | -- | 10.5 | 17.0 |
| Tetrapods | 0.36 | -- | 14.0 | 17.0 | 27.0 |
| Tribars | 0.30 | -- | -- | 14.3 | -- |
| Quadripods | 0.18 | -- | 7.0 | -- | -- |

Size of underlayer units

9. The prototype weight of the A stone in the first underlayer was 1 to 2 tons and that of the B stone in the second underlayer was 50 lb to 1 ton. In the model the size of the A stone was 1/2 to 3/4 in. and that of the B stone was 1/4 to 1/2 in.

### Placement of armor units

10. The stability of rubble-mound breakwaters, whether constructed of natural quarrystones or of molded armor units, is dependent to some extent on the method used for placing the armor units. In this investigation the armor units were placed by hand, one at a time, without any attempt to interlock the units; thus, the armor units were essentially random-placed in a manner that corresponded as closely as possible to prototype placement by crane.

## Test Procedures

### Test setup

11. The stability of the various breakwater head and trunk test sections was determined by installing the breakwater sections in one end of the wave flume and subjecting them to waves generated by a plunger-type wave machine located at the other end of the flume. A concrete bottom reproduced the natural beach slope of 1:50 from the toe of the breakwater test section seaward for a distance of 38 ft (model dimension). A bottom slope of 1:30 was used to complete the transition from the 1:50 beach slope to the relatively deep water at the wave machine.

### Model data obtained

12. A test consisted of subjecting a given breakwater section to attack by test waves of specified height and period for durations varying from a few minutes to as much as 2 hr prototype time. The behavior of the test sections during wave attack, including the extent of damage, was determined by visual observations. Design waves for the no-damage criterion, i.e. the largest waves that did not remove armor units from the test sections, were also selected by means of visual observations. Waves slightly smaller than the selected design waves did, at times, remove a few loose armor units without causing significant damage.

13. Wave heights were measured with electrical wave-height gages in conjunction with a recording oscillograph. Photographs were made of nearly all wave conditions and breakwater sections tested. Selected photographs are presented in this report as photographs 1-10.

14. Wave pressures were measured using four pressure transducers

(cells) installed at different levels, one above and three below stillwater level (swl). The locations of the cells are shown in plate 2. Base-test pressures (table 1) were measured without a rock-tetrapod wave absorber on the seaside of the test structure. To show the spread in the pressure measurements for identical test conditions, the maximum, minimum, and four intermediate pressures were recorded and plotted. Since the structure will be subjected to hydrostatic pressure corresponding to swl on the harborside, it will be necessary to subtract swl hydrostatic pressure from the model-measured pressures to obtain the net overturning pressures.

## Selected Test Conditions

### Design wave dimensions and still-water level

15. Available prototype data indicate that deep-water waves with heights of about 15 ft will occur with sufficient frequency to warrant selection of this size wave as the prototype design wave height ($H_D$). The water depth at the toe of the head of the west breakwater is about 20 ft referred to the Shattuck scale datum (Ssd).* An swl of +5 ft Ssd was selected for all stability tests. Thus, the water depths at the toe of both the head and trunk of the west breakwater near its seaward end were 25 ft referred to swl. Based on an 11-sec wave, a water depth of 25 ft, and the corresponding shoaling coefficient, a shallow-water design wave height of 16 ft was selected for stability tests on the head and the seaward end of the trunk of the west breakwater. Stability tests were also conducted on the west breakwater trunk in a water depth of 17 ft referred to swl. In these tests the largest wave that could be generated without breaking before reaching the site of the breakwater was 13.5 ft in height.

16. In the pressure tests the largest wave that could be made to attack the structure without breaking seaward of the structure was used. This test wave was 11 sec in period (T) and 18 ft in height (H),

* The Shattuck scale datum is used on local maps and is equal to -2.991 ft mean sea level.

measured at the breaking point on the seaward side of the structure with the structure installed in the model. The deep-water wave height corresponding to a shallow-water wave height of 18 ft is 17 ft. Preliminary tests showed that the 11-sec, 18-ft wave resulted in the largest pressures on the wall of any size wave that could reach the breakwater.

Test waves

17. Hindcast data obtained from available records of severe storms indicated that the periods of waves attacking the Nassau Harbor breakwater site range from about 7 to 11 sec. Stability tests were conducted using wave periods of 7, 9, and 11 sec. Test waves ranged in height from the largest waves that could be generated in the selected water depth to slightly smaller than the waves that would not damage the test sections. The heights (H) of the test waves (in prototype values) used, the three wave periods (T), and the various depths of water (d) selected for the tests were as follows:

| Model Scale 1:36 T = 11 sec | | Model Scale 1:45 T = 11 sec | | Model Scale 1:56 T = 11 sec | |
|---|---|---|---|---|---|
| H at Wave Generator, ft (d = 65 ft swl) | H at Toe of Breakwater, ft (d = 17 ft swl) | H at Wave Generator, ft (d = 84 ft swl) | H at Toe of Breakwater, ft (d = 25 ft swl) | H at Wave Generator, ft (d = 99 ft swl) | H at Toe of Breakwater, ft (d = 25 ft swl) |
| 6.0 | 9.0 | 8.0 | 10.5 | 11.0 | 12.0 |
| 6.5 | 10.0 | 8.5 | 11.0 | 12.0 | 15.0 |
| 7.0 | 11.0 | 9.0 | 12.0 | 13.0 | 17.0 |
| 7.5 | 12.0 | 9.5 | 12.5 | 14.0 | 18.0 |
| 8.0 | 13.0 | 10.0 | 13.0 | 15.0 | 18.0 |
| 8.5 | 14.0 | 10.5 | 13.5 | 16.0 | 18.0 |
| | | 11.5 | 14.0 | | |
| | | 12.0 | 15.0 | | |
| | | 13.0 | 15.5 | | |
| | | 13.5 | 16.5 | | |

| Model Scale 1:48 T = 7 sec | | T = 9 sec | | T = 11 sec | |
|---|---|---|---|---|---|
| H at Wave Generator, ft (d = 88 ft swl) | H at Toe of Breakwater, ft (d = 25 ft swl) | H at Wave Generator, ft (d = 88 ft swl) | H at Toe of Breakwater, ft (d = 25 ft swl) | H at Wave Generator, ft (d = 88 ft swl) | H at Toe of Breakwater, ft (d = 25 ft swl) |
| 10.0 | 12.0 | 8.5 | 11.0 | 7.5 | 12.0 |
| 12.0 | 13.0 | 9.5 | 12.0 | 8.5 | 13.0 |
| 13.0 | 14.5 | 10.0 | 13.0 | 8.5 | 13.5 |
| 14.0 | 16.0 | 10.5 | 14.0 | 9.0 | 14.0 |
| | | 11.0 | 15.0 | 9.5 | 15.0 |
| | | 12.0 | 16.0 | 10.0 | 16.0 |
| | | 13.0 | 17.0 | 10.5 | 16.5 |
| | | | | 11.0 | 17.0 |
| | | | | 12.0 | 17.5 |
| | | | | 12.5 | 18.0 |

## PART III: WEST BREAKWATER PLANS, TESTS, AND RESULTS

### Development of Plans

#### Breakwater trunk

18. In the development of plans for construction of the rock-tetrapod or rock-tribar wave absorber on the seaside of the vertical-wall structure, it was desired to determine the most economical design. The purpose of the wave absorber was to (a) reduce the wave energy that would be reflected across the navigation channel from the vertical-wall breakwater; (b) reduce wave-induced shock pressures acting on the vertical-wall structure; (c) reduce shock pressures acting on the concrete encasement of the sheet steel piling at the top of the vertical wall; and (d) prevent erosion of the soft limestone at the base of the wall. Three series of tests were conducted on the breakwater trunk. Test series 1 was concerned with determining the cross-sectional area of the seaside rubble mound that would reduce breaking-wave shock pressures to pressures equivalent to those obtained from the Sainflou theory. Test series 2 was conducted to obtain design criteria for the west breakwater trunk in a water depth of 25 ft referred to swl. Test series 3 was conducted to obtain design criteria for that portion of the west breakwater trunk in a water depth of less than 20 ft referred to swl. In addition, it was desired to develop a method of placing tetrapods and tribars in a rectangular trench at the toe of the slope of the cover layer; no stability tests were conducted with the rectangular trench installed in the model.

#### Breakwater head

19. The purpose of the wave absorber constructed on the wrap-around at the head or seaward end of the west breakwater was the same as that described for the breakwater trunk. Two series of tests were conducted to determine design criteria for the breakwater head. In test series 4 the crown elevation of the rubble wave absorber was the same as that of the vertical-wall structure. In test series 5 the crown elevation of the rubble was 5 ft lower than that of the vertical wall.

## Test Series 1

### Purpose of tests

20. Test series 1 was conducted on a 1:56-scale model to determine the cross-sectional area of the rubble wave absorber required to reduce the intensity of breaking-wave shock pressures to that obtained from the Sainflou theory.

### Plans tested

21. Test series 1 comprised plans 1, 2, 2A, 2B, 2C, and 2D. Sketches of these plans are shown in plate 2. These plans were similar except for variations in the weight of cover-layer units, crown elevation of cover layer, and width of crown of cover layer. All of these plans were constructed using two layers of tetrapods random-placed on a slope of 1:1.5 with the breakwater constructed in a water depth of 25 ft referred to swl. Plan 1 was constructed of two layers of 27-ton tetrapods random-placed. The crown elevation of the tetrapod section was +5 ft Ssd, and the horizontal width of the base of the crown of the cover layer was two armor units. Plan 2 was the same as plan 1 except that the weight of the tetrapod armor units was reduced from 27 to 17 tons. Plan 2A was the same as plan 2 except that the crown elevation of the tetrapod armor layer was raised from +5 to +10 ft Ssd. Plan 2B was the same as plan 2A except that the base of the crown of the cover layer was increased from two armor units wide to three units wide. In plan 2C, the crown elevation of the tetrapod armor layer was raised from +10 to +15 ft Ssd. Plan 2D was constructed with the base of the crown of the tetrapod armor layer four units wide, and with the crown elevation of the armor layer +15 ft Ssd.

### Tests and results

22. All breakwater sections were tested with an 11-sec, 18-ft wave. Results (table 1 and plate 2) indicate that increasing the cross-sectional area of the rubble mound on the seaside face of the vertical wall reduced the intensity of the total pressure acting on the wall. However, reduction of the maximum pressures did not exceed about 60 percent. Even with a 60 percent reduction, the pressures on the upper part of the wall were greatly in excess of those obtained from the Sainflou theory. It is concluded that reduction of the maximum shock pressures, which have

durations measured in thousandths of a second, to pressures equivalent to those obtained from the Sainflou theory by construction of a mound of rubble on the seaside of the vertical wall is not economically practical. The inertia of the structure is believed to be sufficient to prevent appreciable response to pressures of very short duration; however, to ensure that the concrete used to encase the upper part of the steel piling will not be damaged, it may be necessary to increase the thickness of the concrete encasement.

## Test Series 2

### Purpose of tests

23. Test series 2 was conducted to determine design criteria for cover-layer units for the west breakwater trunk situated in a water depth of 25 ft referred to swl. The prototype design wave selected for this water depth was 11 sec in period and 16 ft in height.

### Description of plans

24. Test series 2 comprised plans 3, 4, and 4A. In these plans the crown elevation of the vertical-wall bulkhead was +15 ft Ssd, and the crown elevation of the cover layer was +10 ft Ssd. The cover layer consisted of two layers of armor units random-placed on a 1:1.5 slope. The base of the crown was two armor units wide. Plan 3 was tested using a linear scale of 1:45, model to prototype, and the armor units were quadripods weighing 7 tons. Plan 4 was similar to plan 3 except that the armor layers comprised 10.5-ton tetrapods. Plan 4A (plate 3) was the same as plan 4 except that the armor units at the toe of the slope were anchored by placing them in a V-shaped trench. Plans 4 and 4A were tested using a linear scale of 1:48, model to prototype.

### Tests and results

25. Plan 3. Stability tests were conducted using 11-sec waves ranging in height from 12 to 16.5 ft. The breakwater was stable for 14-ft waves, and was damaged slightly by the breaking action of 15.5- and 16.5-ft waves.

26. Plan 4. Plan 4 was tested using 11-sec waves ranging in height from 12 to 18 ft. The 10.5-ton tetrapods were stable for waves as high as

15 ft. For waves larger than 15 ft, damage resulted from en masse sliding of the cover layer. Photograph 1 shows the action of 11-sec, 18-ft waves on the breakwater, and photograph 2 shows damage caused by waves greater than 15 ft in height. To ensure that the en masse sliding of the cover layer was not accidental, a check test was conducted. The results of the check test agreed with those of the initial tests.

27. Plan 4A. As mentioned previously, plan 4A (plate 3) was the same as plan 4 except that a V-shaped trench was installed in the model at the toe of the test section. Stability tests were conducted using 11-sec waves ranging in height from 12 to 18 ft. The breakwater was stable for waves up to 16 ft in height. The crown of the breakwater was damaged by waves 17 to 18 ft in height. Armor units on the seaward shoulder of the crown were displaced by these waves; most came to rest against the face of the vertical wall, but one unit rolled to the toe of the slope. Except for damage to the crown at its seaward shoulder, the breakwater was not damaged. Photograph 3 shows plan 4A after attack by 11-sec, 16- and 18-ft waves.

Discussion of results

28. The stability data obtained from test series 2 are summarized in the following tabulation. Values of $K_\Delta$ (an experimental coefficient) were calculated from the experimentally determined design wave heights for the no-damage condition ($H_{D=0}$), the WES stability equation

$$W_r = \frac{\gamma_r \, (H_{D=0})^3}{K_\Delta \, (S_r - 1)^3 \cot \alpha} \tag{1}$$

where $\alpha$ is the angle of breakwater slope in degrees, and the test conditions listed.

| Plan | Model Scale | Type of Armor Unit | Weight of Armor Unit tons | $H_{D=0}$ ft | $K_\Delta$ |
|---|---|---|---|---|---|
| | Wave Period = 11 sec; cot $\alpha$ = 1.5 | | | | |
| 3 | 1:45 | Quadripods | 7.0 | 14 | 8.2 |
| 4 | 1:48 | Tetrapods | 10.5 | 15 | 6.8 |
| 4 | 1:48 | Tetrapods | 10.5 | 15* | 6.8* |
| 4A | 1:48 | Tetrapods | 10.5 | 16** | 8.2** |

* Repeat tests.

** Armor units placed in V-shaped trench at toe of cover layer.

The data show that plan 4A would be stable for the 16-ft prototype design wave. The V-shaped trench at the toe of the test section prevented sliding of the entire cover layer. These data also indicate that for the type of breakwater tested a design coefficient ($K_\Delta$) of 8 and equation 1 would be satisfactory for determining the size tetrapod armor unit required to resist wave action on the trunk of the west breakwater when the cover layer is anchored to prevent en masse sliding.

## Test Series 3

### Purpose of tests

29. The water depth along the site of the west breakwater varies from about 3 ft referred to Ssd near the shore to about 20 ft referred to Ssd at the seaward end of the structure. As the water depth decreases toward the shore, the height of the waves that can attack the breakwater also decreases. Data obtained from test series 2 provided design criteria for sections of the breakwater situated in water depths of 20 ft referred to Ssd. To ensure that reliable design criteria for those portions of the west breakwater situated in depths less than 20 ft referred to Ssd were available, test series 3 (plans 5 and 5A) was conducted. The depth of water at the toe of the test section was 17 ft referred to swl. For the conditions tested, an 11-sec, 14-ft wave was the largest wave that would not break before reaching the site of the test structure.

### Description of plans

30. The crown elevation of the vertical-wall bulkhead for plans 5 and 5A was +15 ft Ssd, and the crown elevation of the tetrapod-rock wave absorber on the seaside of the structure was +10 ft Ssd. The cover layers consisted of two layers of tetrapod armor units weighing 4.3 tons random-placed on a slope of 1:1.5. Both plans 5 and 5A were tested using a linear scale of 1:36, model to prototype. The base of the crown of the cover layer was two armor units wide. Plans 5 and 5A were identical except that in plan 5A a V-shaped trench was installed to prevent en masse sliding of the cover layer. The elements of plan 5A are shown in plate 4.

### Tests and results

31. Plan 5. Stability tests on plan 5 were conducted using 11-sec

waves ranging from 10 to 14 ft in height. The breakwater was stable for 12-ft waves, and was damaged slightly by 14-ft waves. Most of the damage occurred on the crown of the structure. Three or four armor units were displaced from the seaward shoulder and came to rest against the vertical wall. One armor unit situated just below swl washed downslope. The V-shaped trench was not used in plan 5, but there was no evidence of en masse downslope sliding of the cover layer. However, slight settlement of the crown adjacent to the vertical wall was observed. Photograph 4 shows plan 5 after attack by 11-sec, 12- and 14-ft waves. In test conducted on plan 4 with a water depth of 25 ft referred to swl and without the V-shaped trench at the toe, en masse downslope sliding was observed. Because of the variation between results of plans 4 and 5, a check test of the plan 5 breakwater was conducted. The results of the repeat test were similar to those of the initial tests except that in the repeat tests no settlement of the crown adjacent to the vertical wall was observed.

32. Plan 5A. Plan 5A was tested to determine the effect of anchoring the toe of the cover layer in a V-shaped trench at its toe; 11-sec waves varying from 10 to 14 ft in height were used. The breakwater was stable for 12-ft waves, and the crown was damaged slightly by 14-ft waves. Two or three units were washed from the seaside shoulder and came to rest against the vertical wall, and another unit was washed downslope from the shoulder of the breakwater. Armor units below swl were not displaced from their nested positions. Photograph 5 shows plan 5A after attack by 11-sec, 12- and 14-ft waves.

Discussion of results

33. Stability data obtained from test series 3 are as follows:

| Plan | Type of Armor Unit | Weight of Armor Unit tons | $H_{D=0}$ ft | $K_\Delta$ |
|---|---|---|---|---|
| | Wave Period = 11 sec; cot $\alpha$ = 1.5 | | | |
| 5 | Tetrapods | 4.3 | 12 | 8.3 |
| 5 | Tetrapods | 4.3 | 12 | 8.3* |
| 5A | Tetrapods | 4.3 | 12 | 8.3** |

* Repeat tests.

** Armor units placed in V-shaped trench at toe of cover layer.

These data indicate that the V-shaped trench at the toe of the armor layer did not affect the stability of the test section. However, as mentioned in paragraph 31, there was some slight settlement of the crown of the armor layer adjacent to the vertical wall in plan 5, whereas in plan 5A in which the V-shaped trench was used no settlement of the crown was observed. Although stability tests showed very little difference between plans 5 and 5A, it is believed that use of some form of trench to anchor the toe of the cover layer will ensure a more stable structure. Comparison of $K_\Delta$ values obtained from breakwater stability tests on the west breakwater situated in a water depth of 25 ft referred to swl (see paragraph 28) with those tabulated above shows that, for similar test conditions, the difference in water depths did not influence the values of $K_\Delta$ obtained from the stability tests. The results of test series 2 and 3 show that a $K_\Delta$ value of 8 should be satisfactory for design of the tetrapod-rock wave absorber on the seaside of the west breakwater.

## Test Series 4

### Purpose of tests

34. Test series 4 was conducted to obtain design criteria for armor layers on the conical wraparound at the head of the breakwater with the crown elevation of the rubble wave absorber the same as that of the vertical-wall structure. These tests involved both quadripods and tetrapods.

### Description of plans

35. Test series 4 comprised plans 6 through 8. In these plans the crown elevations of both the rubble wave absorber and the vertical-wall structure were +15 ft Ssd; water depth was 25 ft referred to swl. Elements of plan 8 are shown in plate 5. In plans 6 through 8 the cover layers were composed of two layers of molded armor units random-placed on a slope of 1:1.5. The base of the crown of the cover layer was two armor units wide, and the V-shaped trench at the toe of the cover layer was not used. The linear scales, model to prototype, and the type and weight of armor units used for construction of plans 6 through 8 were as follows:

| Plan | Model Scale | Type of Armor Unit | Weight of Armor Unit tons |
|---|---|---|---|
| 6 | 1:45 | Quadripods | 7 |
| 7 | 1:45 | Tetrapods | 14 |
| 8 | 1:48 | Tetrapods | 17 |

Tests and results

36. Plan 6. Stability tests of plan 6 were conducted using 11-sec waves ranging in height from 10.5 to 13.5 ft. The structure was stable for 12-ft waves. However, the harborside quadrant was damaged by the 13.5-ft waves; almost all of the damage was at the swl and about 45 deg harborward from the breakwater center line.

37. Plan 7. This plan was tested using 11-sec waves ranging from 12 to 16.5 ft in height. Waves 12 to 15 ft in height dislodged one unit from the harborside quadrant of the head, and two or three units were observed to rock in their nested positions. However, the plan 7 head section was considered stable for up to 15-ft waves. Three armor units were rolled downslope during the attack of waves 12 to 16.5 ft in height.

38. Plan 8. Plan 8 was tested using a linear scale of 1:48, model to prototype. Changing the linear scale from 1:45 to 1:48 increased the weight of the prototype unit simulated, which in turn increased the height of the design wave for the no-damage criterion. This model technique permits determining directly the weight of the model unit required to resist the erosive action of the selected prototype design wave ($H_D$ = 16 ft). The stability of plan 8 was investigated using 11-sec waves ranging from 12 to 18 ft in height. No movement of armor units was observed during attack by waves less than 16.5 ft in height. Waves 16.5 to 18 ft in height caused two armor units to rock in their nested positions. Photograph 6 shows plan 8 before wave attack and after attack by 11-sec, 16- and 18-ft waves. The test section was considered stable for an 11-sec, 16-ft wave.

Discussion of results

39. Stability data obtained from test series 4 are as follows:

| Plan | Type of Armor Unit | Weight of Armor Unit tons | $H_{D=0}$ ft | $K_\Delta$ |
|---|---|---|---|---|
| | Wave Period = 11 sec; cot $\alpha$ = 1.5 | | | |
| 6 | Quadripods | 7 | 12 | 5.1 |
| 7 | Tetrapods | 14 | 15 | 4.9 |
| 8 | Tetrapods | 17 | 16 | 4.9 |

These data show that a $K_\Delta$ value of 5 could be used for design of a tetrapod or quadripod cover layer at the head of the west breakwater at Nassau Harbor when the crown elevation of the armor layer is the same as that of the vertical wall. These data also show that a cover layer of 17-ton tetrapods random-placed on a slope of 1:1.5 will be stable under the attack of the 16-ft prototype design wave. The test section was overtopped by waves greater than about 14 ft in height, but this overtopping action did not damage the harborside of the test section.

## Test Series 5

### Purpose of tests

40. Test series 5 was conducted to obtain design criteria for the conical wraparound with the crown of the rubble wave absorber at elevation +10 ft Ssd and the crown of the vertical wall at elevation +15 ft Ssd. These tests involved both tetrapods and tribars. To determine the effect of wave period on the stability of the head section, tests using 7-, 9-, and 11-sec waves were conducted on wraparound sections constructed of 17-ton tetrapods.

### Description of plans

41. Test series 5 comprised plans 9 and 10. In plan 9 the armor layer was two layers of 17-ton tetrapods random-placed on a slope of 1:1.5. Plan 10 was the same as plan 9 except that the armor units comprised two layers of 14.3-ton tribars random-placed on a slope of 1:1.5.

### Tests and results

42. Plan 9. The stability of plan 9 was investigated using 11-sec waves ranging in height from 12 to 18 ft. The head section was stable for 15-ft waves. It was damaged slightly by 16.5-ft waves, and 18-ft waves

dislodged eight armor units near the swl in an area extending from about 45 deg seaward of the breakwater center line to about 20 deg harborward from the center line. About half of the dislodged units rolled to the toe of the slope, and the others lodged on the slope about halfway between swl and the bottom. Photograph 7 shows plan 9 before wave attack and after attack by 11-sec, 15- and 18-ft waves.

43. Plan 9 was also tested using 7- and 9-sec waves ranging from 12 to 17 ft in height. The test section was not damaged by any of these test waves. Design wave heights for the no-damage criterion for 7- and 9-sec waves could not be accurately selected because waves large enough to damage the test section could not be generated for the selected test conditions. However, these tests showed that 7- and 9-sec waves are no more damaging than 11-sec waves.

44. Plan 10. The plan 10 tribar armor layer was tested using 11-sec waves ranging from 12 to 18 ft in height. Photograph 8 shows plan 10 before wave attack and after attack by 11-sec, 16- and 18-ft waves. Waves ranging from 12 to 16 ft in height caused no motion of armor units. Waves ranging in height from 16.5 to 18 ft dislodged three armor units from the seaside quadrant of the head section at about the swl. These units became lodged on the wraparound slope a few feet below swl. The head section was considered stable for a 16-ft wave.

Discussion of results

45. Stability data obtained from test series 5 are as follows:

| Plan | Type of Armor Unit | Weight of Armor Unit tons | $H_{D=0}$ ft | $K_\Delta$ |
|---|---|---|---|---|
| Wave Period = 11 sec; cot $\alpha$ = 1.5 | | | | |
| 9 | Tetrapods | 17 | 15 | 4.1 |
| 10 | Tribars | 14.3 | 16 | 6.1 |

Using a $K_\Delta$ of 4, a design wave height of 16 ft, and the WES stability equation (equation 1), it is found that a 21-ton tetrapod will resist the erosive action of waves 16 ft in height. Similar calculations show that, using a $K_\Delta$ value of 6, a 14-ton tribar will be stable for a 16-ft design wave. Also, the results of breakwater stability tests conducted previously

at WES in which two layers of tribar armor units were random-placed on the breakwater trunk showed that tribars were slightly more stable than tetrapod armor units.[2]

## Placement of Armor Units in Rectangular Trench

46. The V-shaped trench used in the model may be impractical for use in the prototype because the appreciable depth of the V-shaped trench could require special dredging equipment that would increase the cost of excavation prohibitively. A rectangular trench was believed to be more practical. Therefore, a method of placing tetrapods and tribars in a rectangular trench at the toe of the slope of the cover layer was developed. The trench was designed and the cover layer and underlayer arranged so that the slope of the face of the breakwater near its toe would be continuous and the units at the toe would be restrained from sliding. Plate 6 shows the details of the trench used at the toe of both the tetrapod and tribar cover layers. Photograph 9 shows the arrangement of the first and second rows of tetrapods placed in the toe trench, the A stone in the trench to provide a 1:1.5-slope underlayer for the armor units, and the seaside face of the completed armor layer. Similar views of the installation of the tribar armor layer are shown in photograph 10.

47. Stability tests were not conducted on the tetrapod and tribar armor layers anchored by the rectangular trench at the toe of the slope. However, it is believed that breakwater designs in which rectangular trenches are used will be as stable as those tested using V-shaped trenches at the toe of the structures.

## PART IV: SUMMARY OF RESULTS AND CONCLUSIONS

48. Results of this investigation, and conclusions based thereon and on similar tests conducted previously at WES, are as follows:

a. Because of the very large cross section required, it is not practical to construct on the seaside of an impermeable vertical wall a mound of rubble capable of reducing the maximum wave-generated shock pressures to values that are obtained from the Sainflou theory.

b. Stability tests were not conducted on the proposed east breakwater for Nassau Harbor; however, the design coefficients obtained from tests on the west breakwater are applicable for design of similar armor layers on the east breakwater provided the geometry of the east breakwater is similar to that of the west breakwater.

c. A $K_\Delta$ value of 8 in the WES breakwater stability equation (equation 1) is adequate for design of a tetrapod armor layer for the trunk of the west breakwater when the crown elevation of the armor layer is +10 ft Ssd, and the crown elevation of the vertical-wall bulkhead is +15 ft Ssd.

d. The results of other stability tests conducted at WES using cover layers composed of tribar and tetrapod armor units have shown that tribar armor layers designed using a $K_\Delta$ of 8 will be somewhat more stable than a tetrapod armor layer designed on the same basis.

e. For design of a simple conical wraparound breakwater head in which the crown elevations of both armor layer and vertical-wall bulkhead are +15 ft Ssd, a $K_\Delta$ value of 5 is adequate for determining the required weight of tetrapod armor units.

f. For design of a simple conical wraparound breakwater head in which the crown elevation of the armor layer is +10 ft Ssd and the crown elevation of the vertical-wall bulkhead is +15 Ssd, $K_\Delta$ values of 4 and 6 are recommended for armor layers of tetrapods and tribars, respectively.

g. Stability tests showed that anchoring the armor units in a V-shaped trench at the toe of the slope would prevent en masse sliding of the armor layer.

h. Stability tests were not conducted on the breakwaters with the toe constructed in a rectangular trench, but it is believed that the rectangular trench will be as effective as the V-shaped trench tested for anchoring of the cover layer, and the dredging of the rectangular trench should be less expensive.

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2. _____, Design of Tribar and Tetrapod Cover Layers for Rubble-Mound Breakwaters, by R. Y. Hudson and R. A. Jackson. Miscellaneous Paper No. 2-296, Vicksburg, Miss., January 1959.

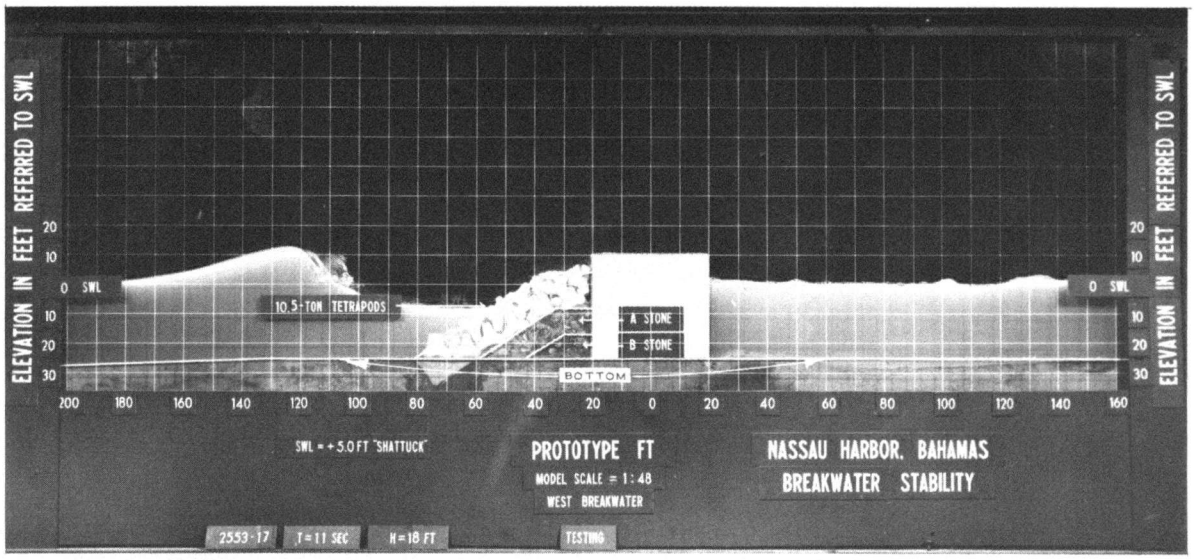
3. Sainflou, M., "Essai sur le diques maritime verticales (Essay on vertical breakwaters)." Annales des Ponts et Chaussées, vol 2, No. 4 (July-August 1928), pp 5-48. Translation by C. R. Hatch, for U. S. Army Engineer Division, Great Lakes, CE.

Table 1

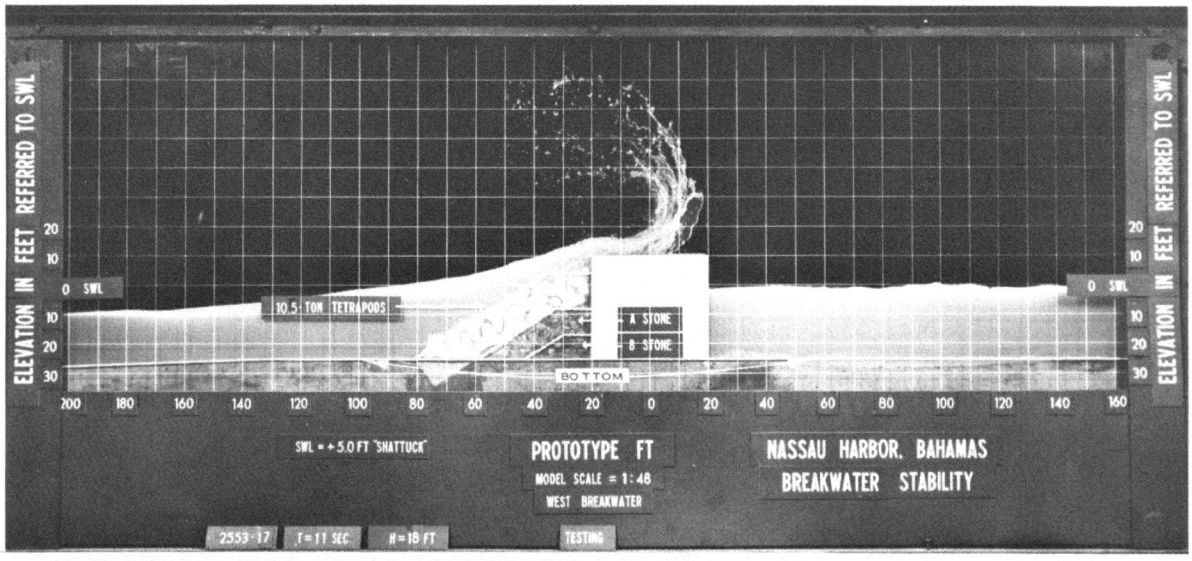
Wave Pressures on Vertical Wall

Water Depth (d) = 25 ft; Wave Height (H) = 18 ft; Wave Period (T) = 11 sec

| Pressure Cell No. | Elevation of Pressure Cell ft swl | Hydrostatic Pressure lb/ft$^2$ | Total Wave Pressure Sainflou lb/ft$^2$ | Total Wave Pressure, Measured, lb/ft$^2$ Max | Min | Intermediate Values | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | Base Test | | | | | | |
| 1 | +4.17 | 0 | 1125 | 10,100 | 1620 | 8880 | 7500 | 6850 | 5890 |
| 2 | -1.67 | 107 | 1420 | 8,990 | 2370 | 5960 | 5030 | 4790 | 4530 |
| 3 | -8.67 | 555 | 1840 | 4,440 | 2170 | 4350 | 4195 | 3380 | 2820 |
| 4 | -15.67 | 1000 | 2200 | 2,940 | 2050 | 2780 | 2620 | 2530 | 2330 |
| | | | Plan 1 | | | | | | |
| 1 | +4.17 | 0 | 1125 | 4,430 | 1210 | 4040 | 3630 | 3380 | 3230 |
| 2 | -1.67 | 107 | 1420 | 6,160 | 2130 | 4860 | 4150 | 3860 | 3750 |
| 3 | -8.67 | 555 | 1840 | 3,790 | 2180 | 3630 | 3380 | 3300 | 2575 |
| 4 | -15.67 | 1000 | 2200 | 3,100 | 1807 | 3020 | 2860 | 2620 | 2450 |
| | | | Plan 2 | | | | | | |
| 1 | +4.17 | 0 | 1125 | 3,710 | 1450 | 3230 | 2660 | 2420 | 2000 |
| 2 | -1.67 | 107 | 1420 | 6,560 | 1800 | 5110 | 4630 | 3580 | 2610 |
| 3 | -8.67 | 555 | 1840 | 3,540 | 2000 | 3220 | 3060 | 2490 | 2170 |
| 4 | -15.67 | 1000 | 2200 | 2,820 | 2010 | 2610 | 2530 | 2330 | 2130 |
| | | | Plan 2A | | | | | | |
| 1 | +4.17 | 0 | 1125 | 5,450 | 1140 | 3330 | 3000 | 2270 | 1780 |
| 2 | -1.67 | 107 | 1420 | 5,230 | 2060 | 4890 | 4000 | 3750 | 3270 |
| 3 | -8.67 | 555 | 1840 | 3,460 | 1440 | 2600 | 2300 | 2000 | 1770 |
| 4 | -15.67 | 1000 | 2200 | 3,000 | 1970 | 2370 | 2300 | 2140 | 2050 |
| | | | Plan 2B | | | | | | |
| 1 | +4.17 | 0 | 1125 | 8,240 | 2990 | 6860 | 5970 | 4280 | 3310 |
| 2 | -1.67 | 107 | 1420 | 8,500 | 2210 | 7860 | 6000 | 5030 | 4060 |
| 3 | -8.67 | 555 | 1840 | 3,710 | 2170 | 3390 | 3140 | 2740 | 2560 |
| 4 | -15.67 | 1000 | 2200 | 3,500 | 2290 | 3340 | 3180 | 2850 | 2450 |
| | | | Plan 2C | | | | | | |
| 1 | +4.17 | 0 | 1125 | 6,450 | 1690 | 4600 | 4000 | 3230 | 2100 |
| 2 | -1.67 | 107 | 1420 | 3,330 | 1800 | 3010 | 2690 | 2370 | 2130 |
| 3 | -8.67 | 555 | 1840 | 2,410 | 1360 | 2010 | 1770 | 1690 | 1520 |
| 4 | -15.67 | 1000 | 2200 | 2,610 | 1810 | 2290 | 2130 | 2000 | 1810 |
| | | | Plan 2D | | | | | | |
| 1 | +4.17 | 0 | 1125 | 4,200 | 810 | 2180 | 1610 | 1210 | 970 |
| 2 | -1.67 | 107 | 1420 | 3,210 | 1240 | 2130 | 1960 | 1800 | 1640 |
| 3 | -8.67 | 555 | 1840 | 2,190 | 1610 | 2010 | 1930 | 1850 | 1770 |
| 4 | -15.67 | 1000 | 2200 | 2,370 | 1810 | 2230 | 2210 | 1970 | 1890 |

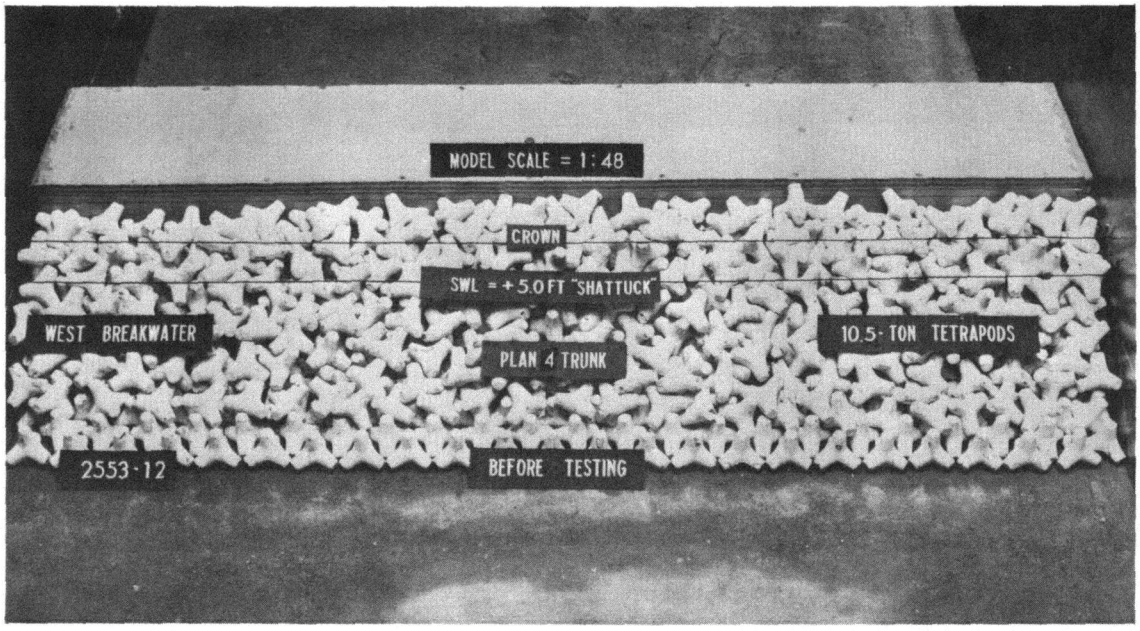



a. Before waves break

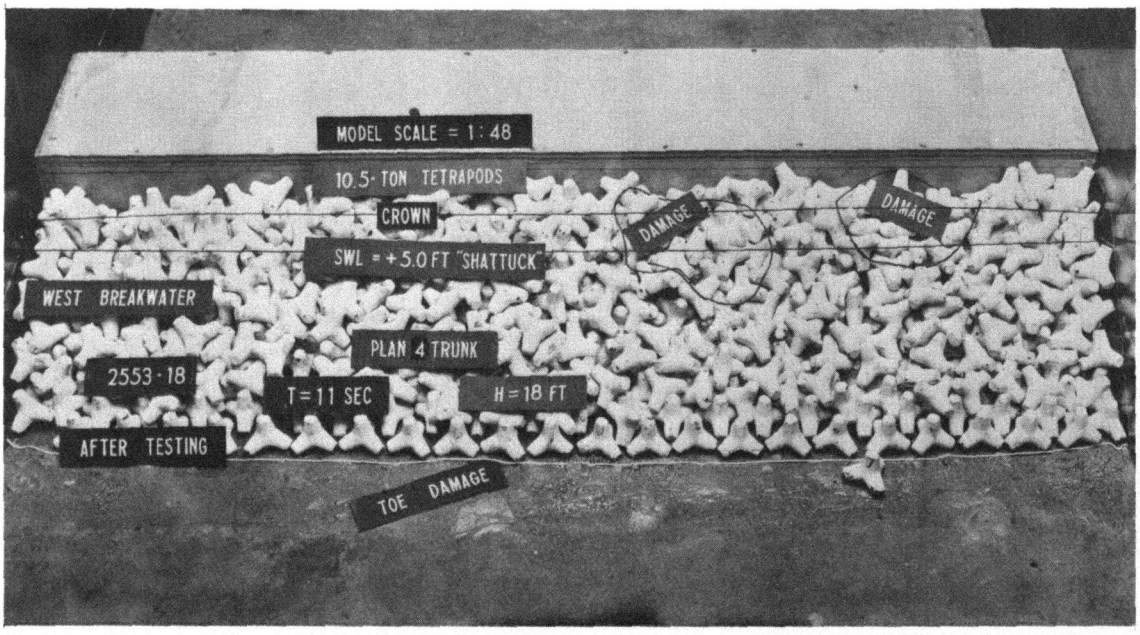



b. Waves breaking on structure

Photograph 1. Plan 4 trunk (10.5-ton tetrapods random-placed) during attack by 11-sec, 18-ft waves (model scale 1:48); d = 25 ft

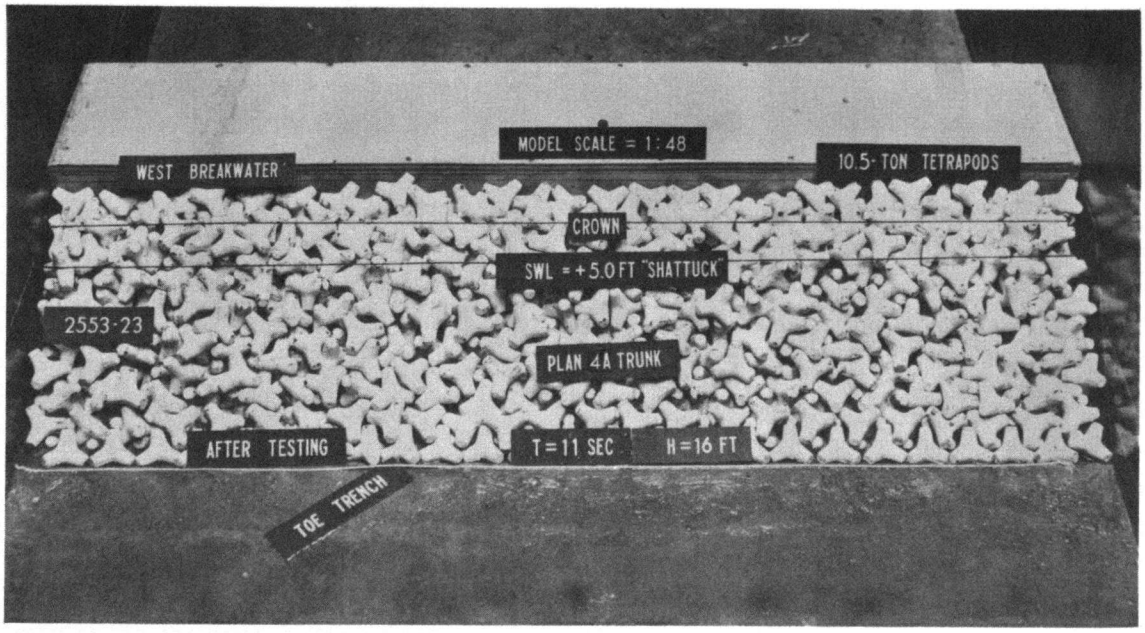



a. Before attack

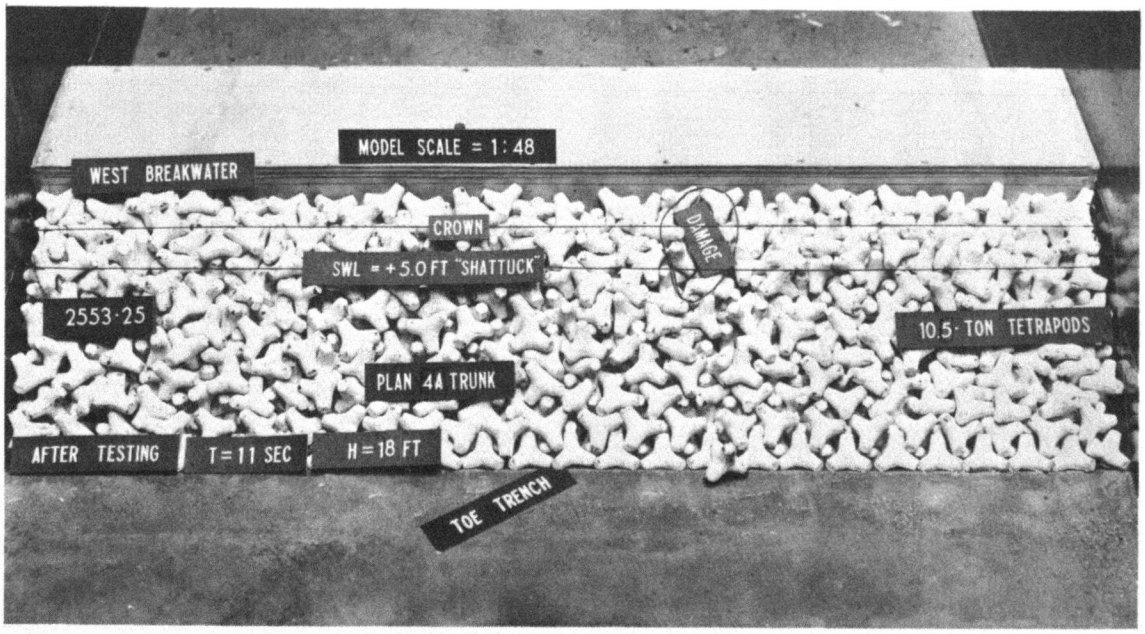



b. After attack

Photograph 2. Plan 4 trunk (10.5-ton tetrapods random-placed) before and after attack by 11-sec, 18-ft waves (model scale 1:48); d = 25 ft

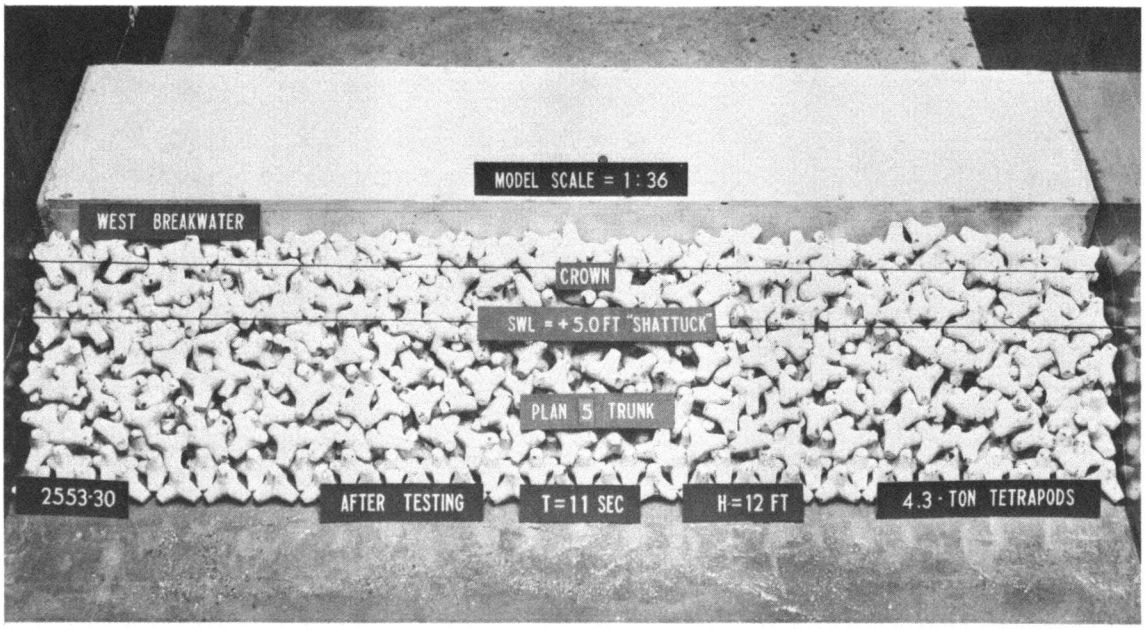



a. After attack by 11-sec, 16-ft waves

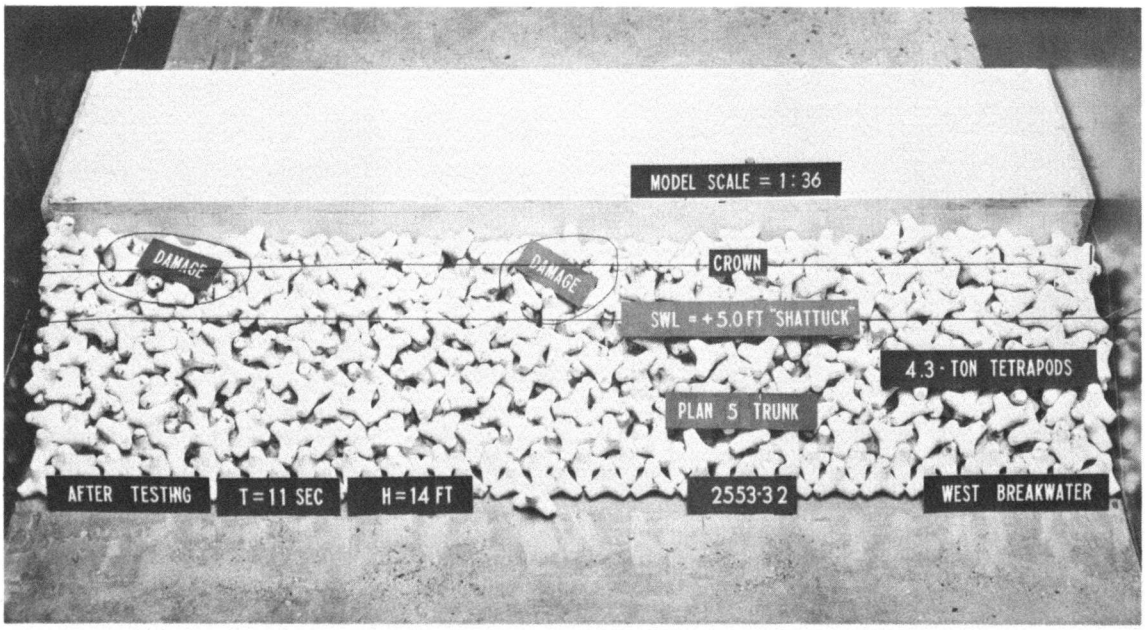



b. After attack by 11-sec, 18-ft waves

Photograph 3. Plan 4A trunk (10.5-ton tetrapods random-placed) (model scale 1:48); d = 25 ft

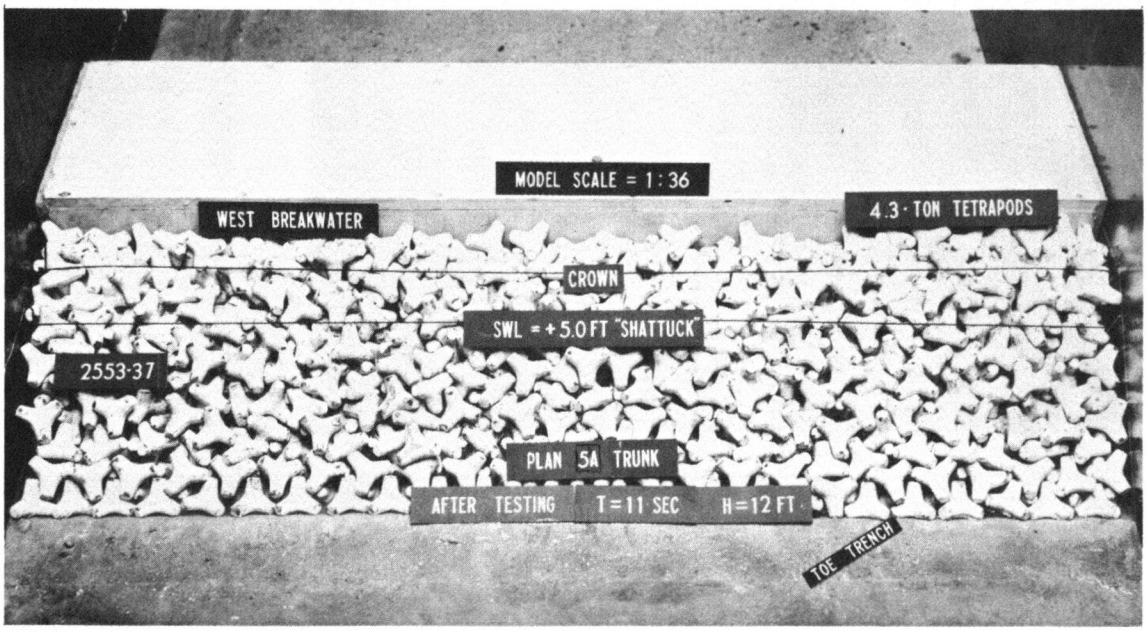



a. After attack by 11-sec, 12-ft waves

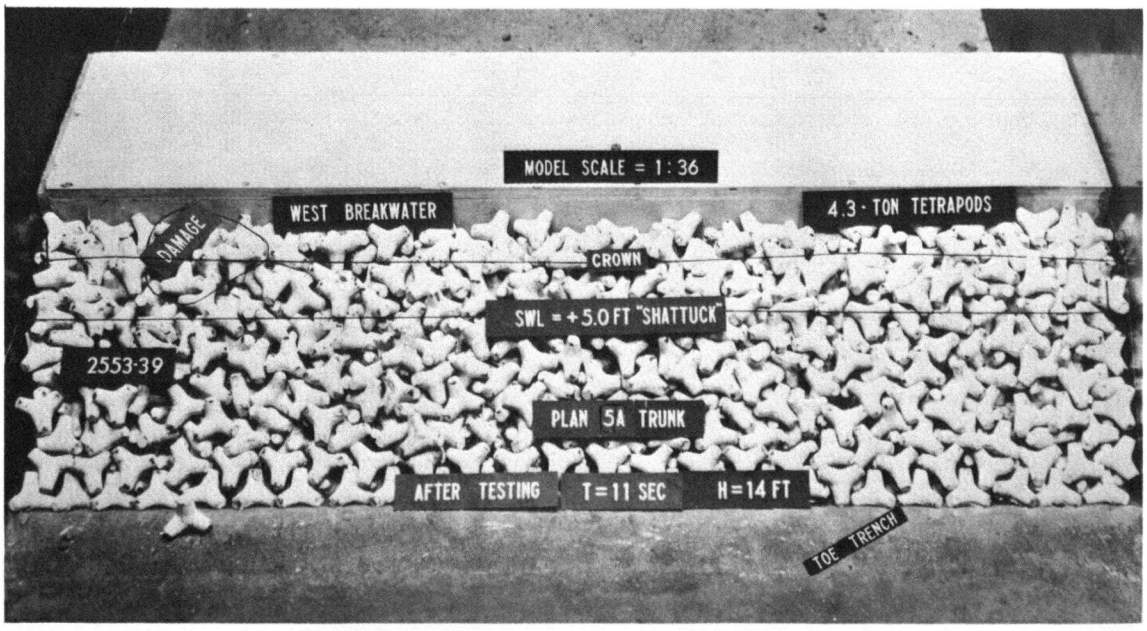



b. After attack by 11-sec, 14-ft waves

Photograph 4. Plan 5 trunk (4.3-ton tetrapods random-placed) (model scale 1:36); d = 17 ft

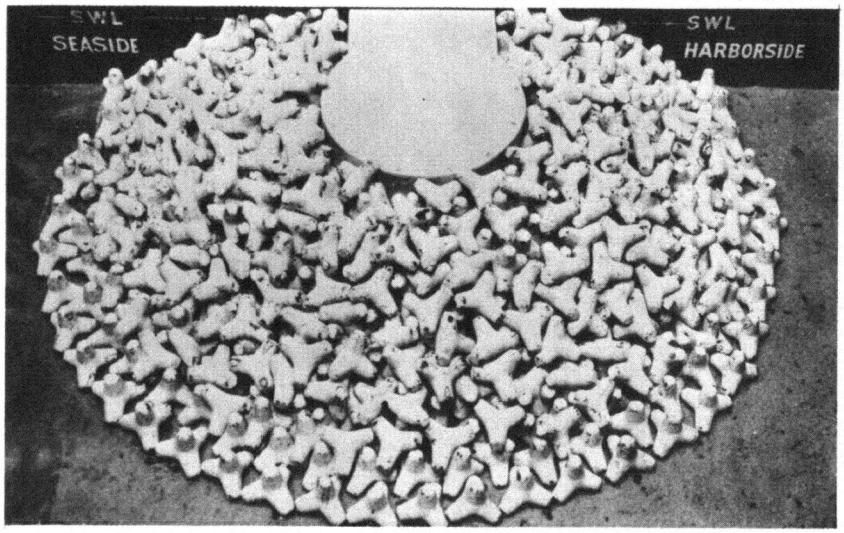



a. After attack by 11-sec, 12-ft waves

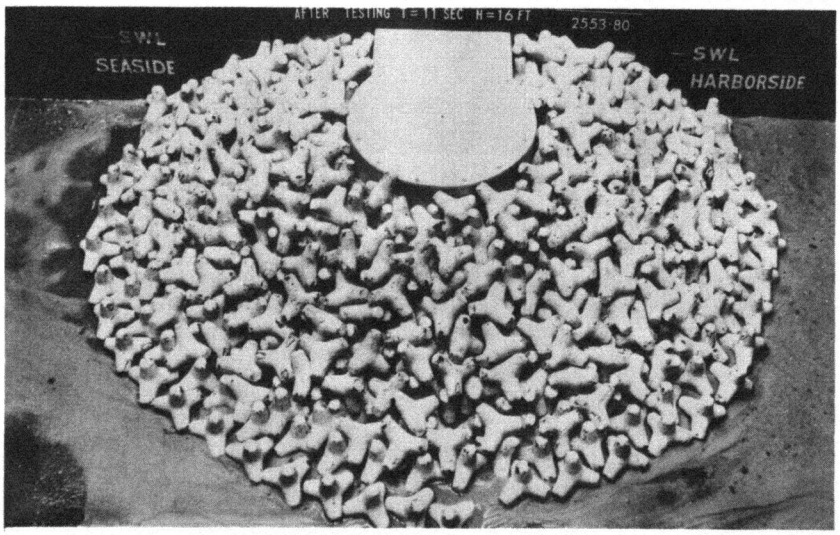



b. After attack by 11-sec, 14-ft waves

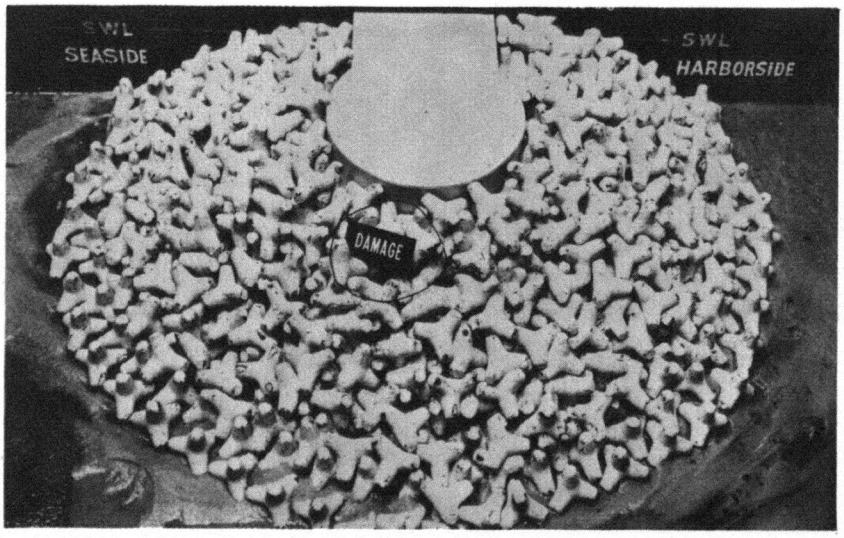
Photograph 5. Plan 5A trunk (4.3-ton tetrapods random-placed) (model scale 1:36); d = 17 ft




a. Before attack



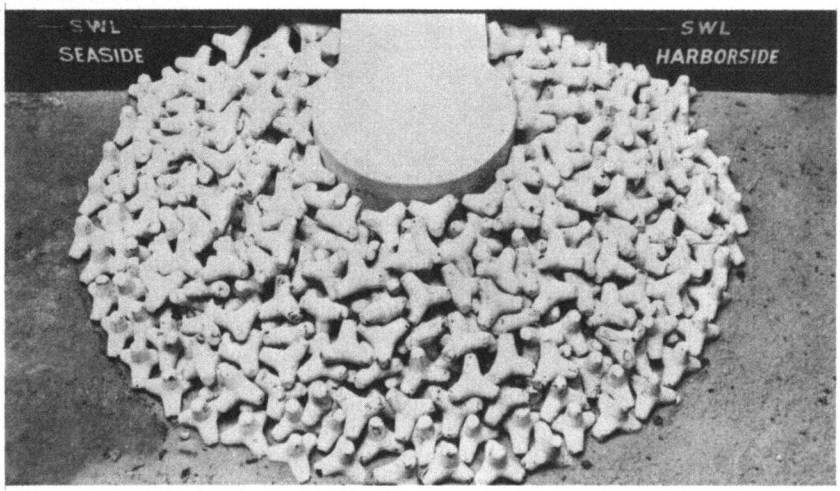

b. After attack by 11-sec, 16-ft waves

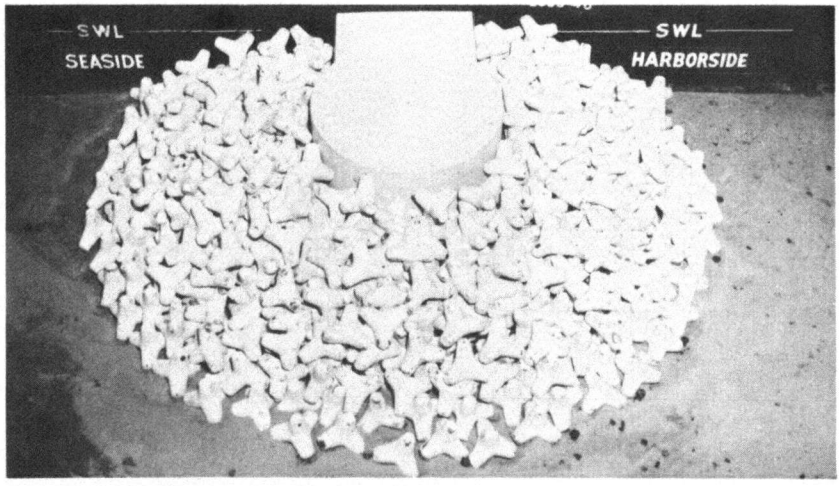



c. After attack by 11-sec, 18-ft waves

Photograph 6. Plan 8 head (17-ton tetrapods random-placed) before and after wave attack (model scale 1:48); d = 25 ft

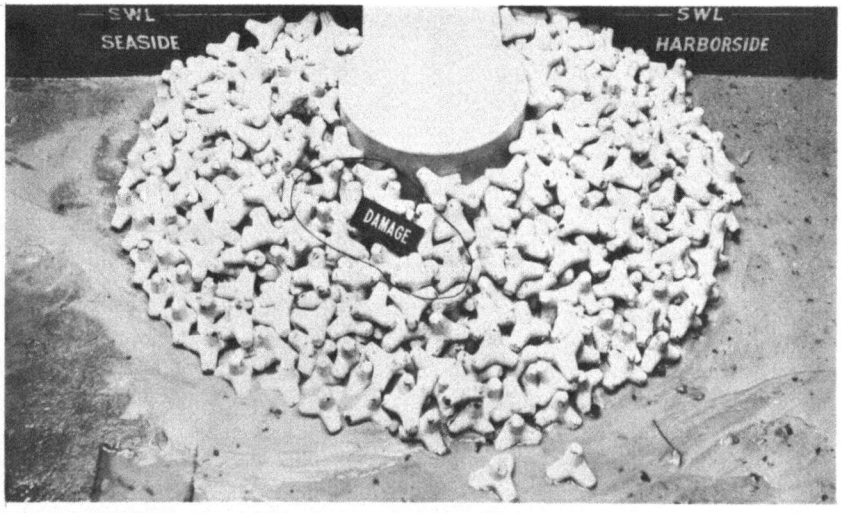
a. Before attack



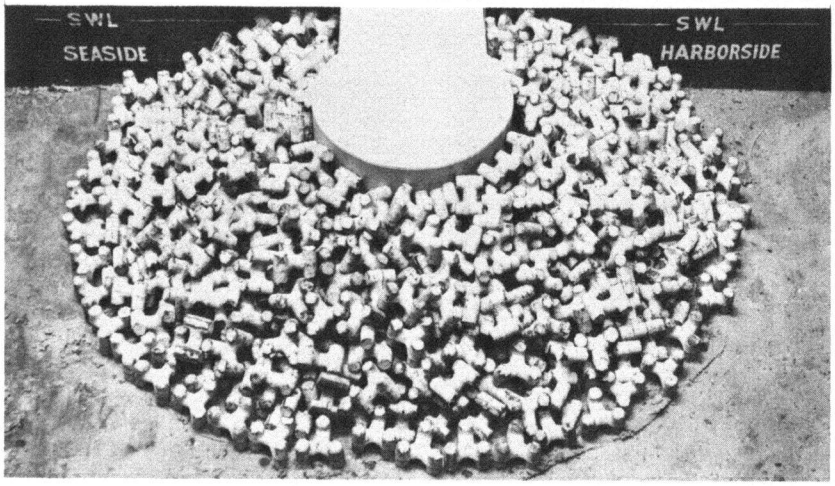




b. After attack by 11-sec, 15-ft waves

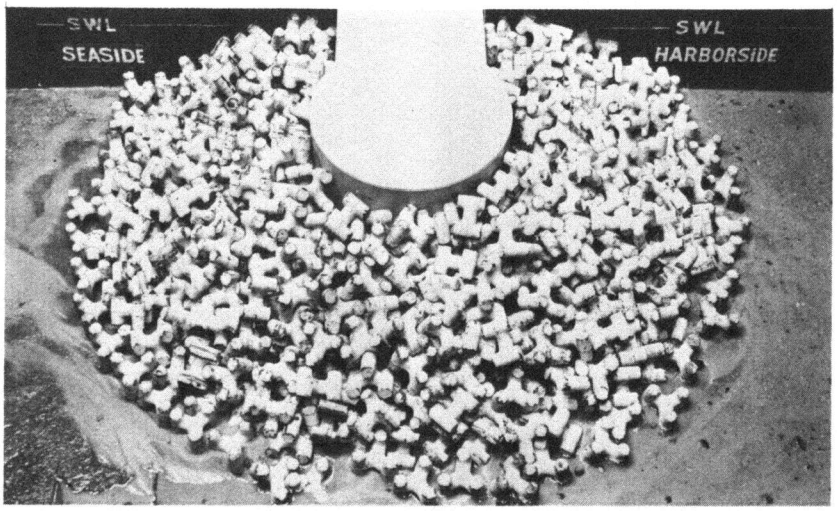
c. After attack by 11-sec, 18-ft waves



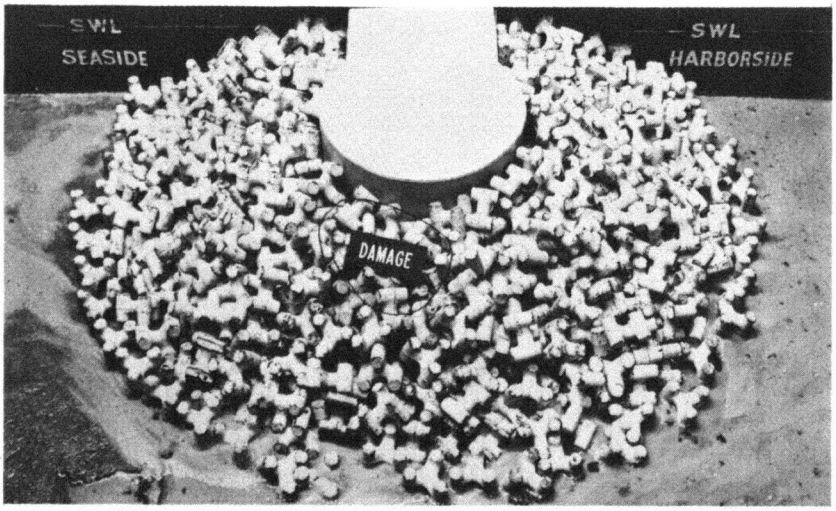

Photograph 7. Plan 9 head (17-ton tetrapods random-placed) before and after wave attack (model scale 1:48); d = 25 ft




a. Before attack

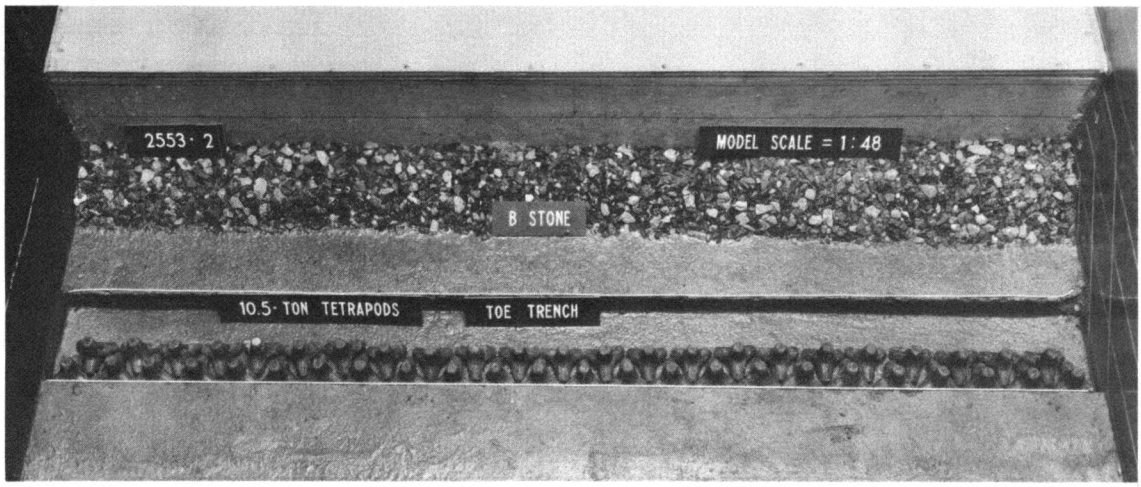



b. After attack by 11-sec, 16-ft waves

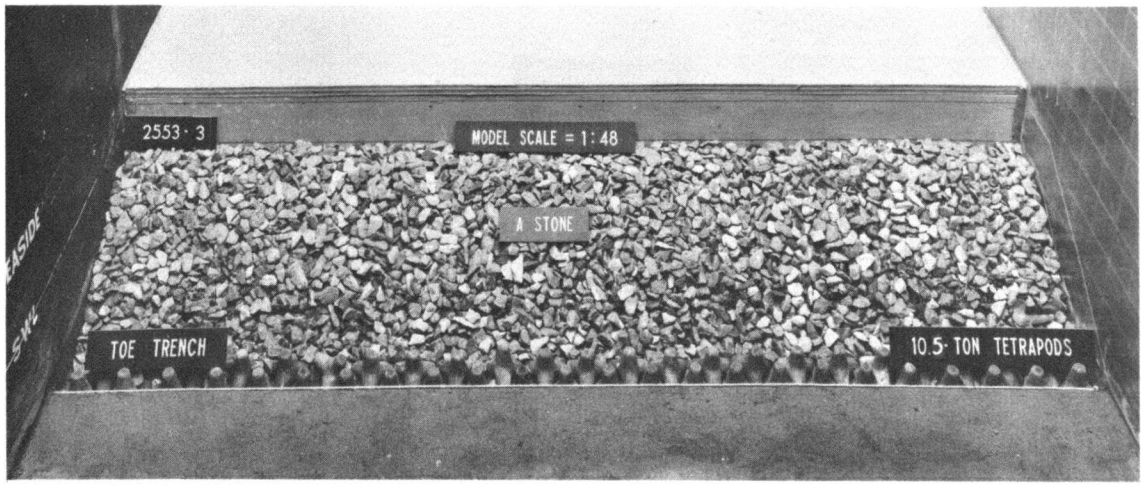



c. After attack by 11-sec, 18-ft waves

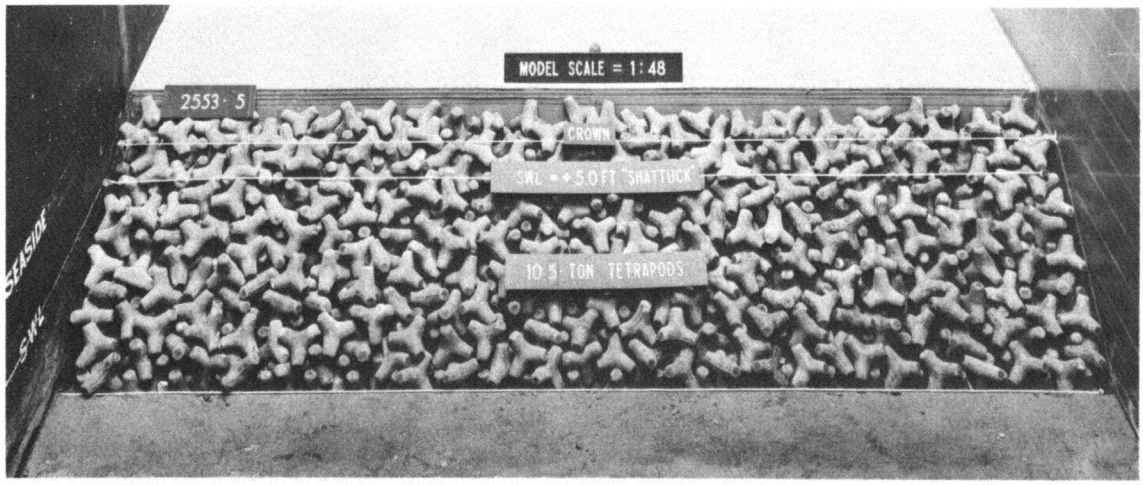
Photograph 8. Plan 10 head (14.3-ton tribars random-placed) before and after wave attack (model scale 1:48); d = 25 ft




a. Arrangement of first and second rows of tetrapods in toe trench

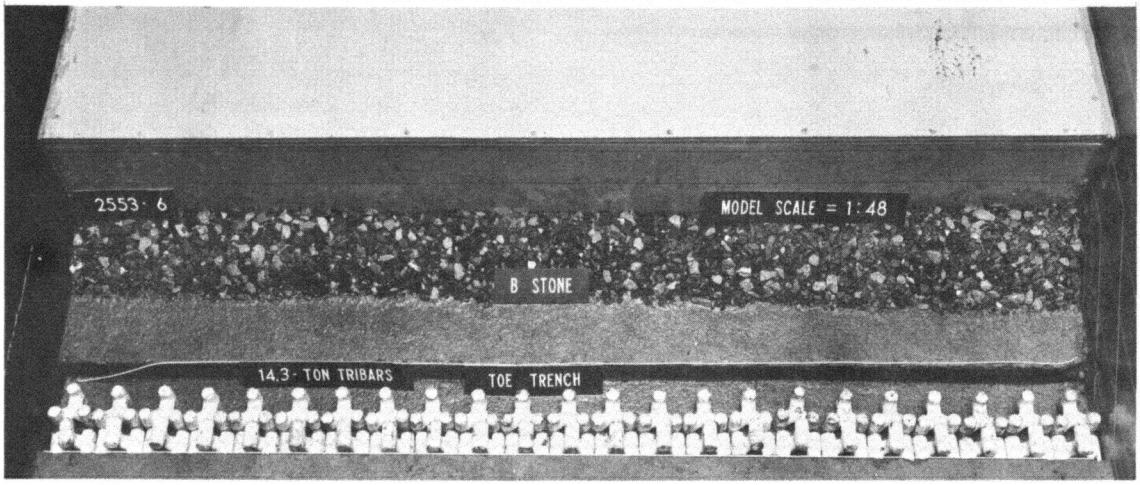



b. Two rows of tetrapods in toe trench with A stone in place

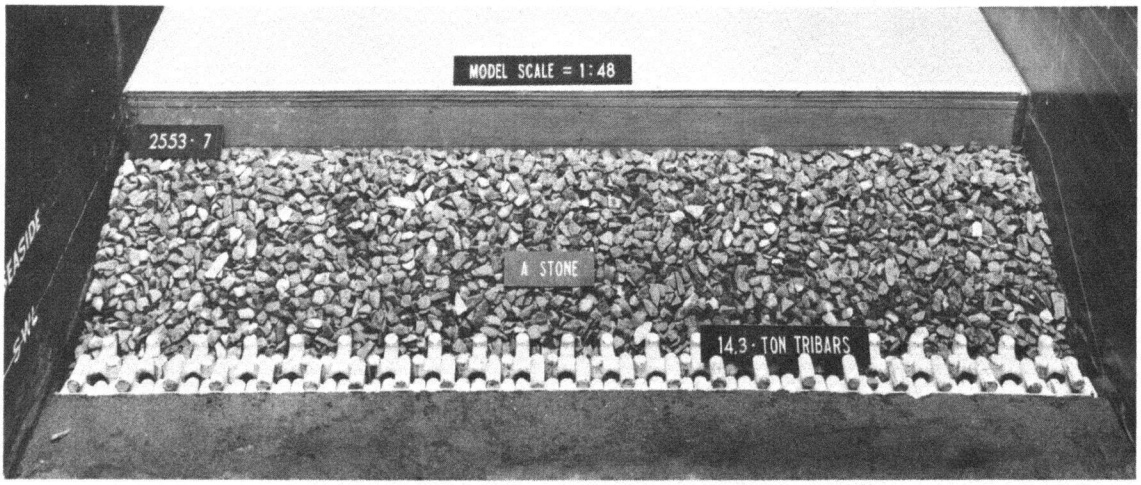



c. Seaside face with armor layer in place

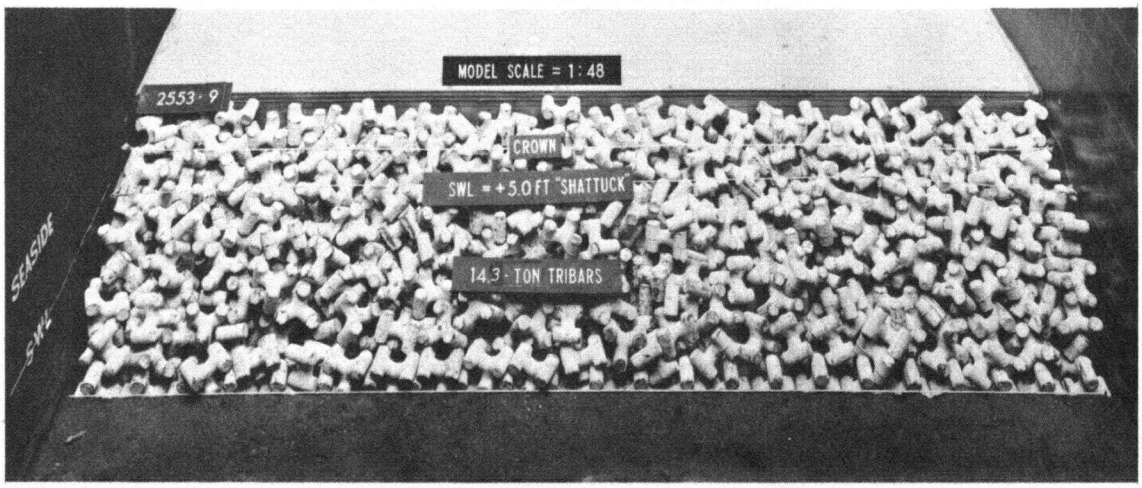
Photograph 9. Placement of 10.5-ton tetrapod armor layer in rectangular toe trench




a. Arrangement of first and second rows of tribars in toe trench

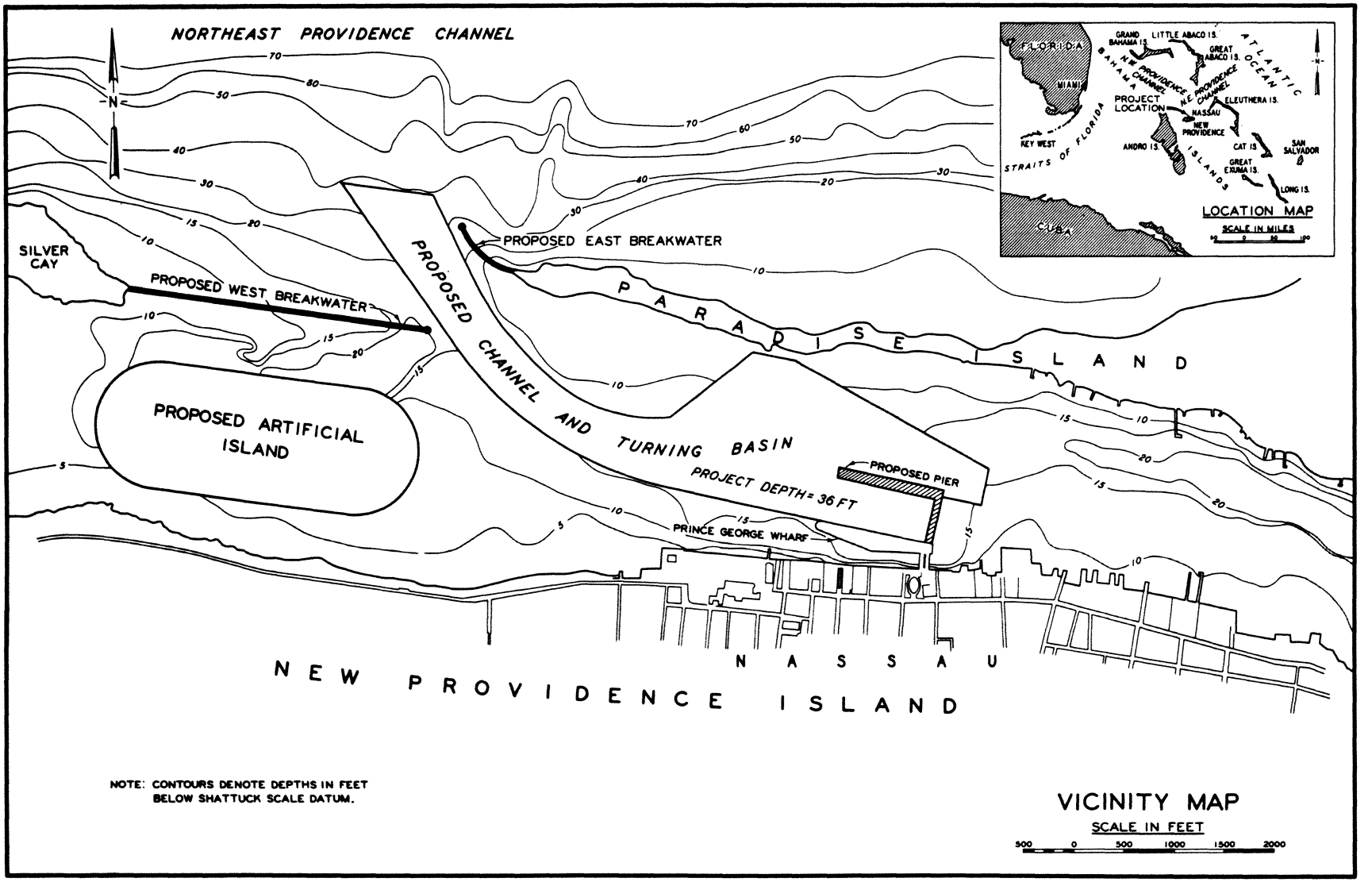



b. Two rows of tribars in toe trench with A stone in place




c. Seaside face with armor layer in place

Photograph 10. Placement of 14.3-ton tribar armor layer in rectangular toe trench



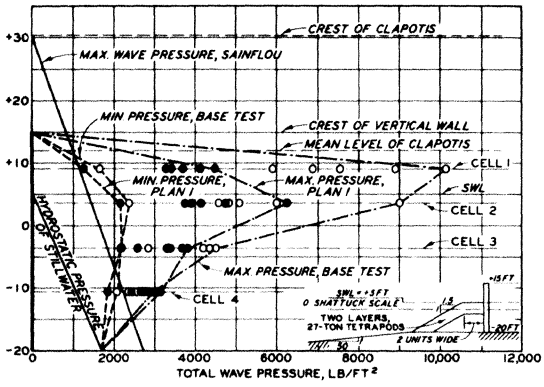

NOTE: CONTOURS DENOTE DEPTHS IN FEET BELOW SHATTUCK SCALE DATUM.

VICINITY MAP

SCALE IN FEET

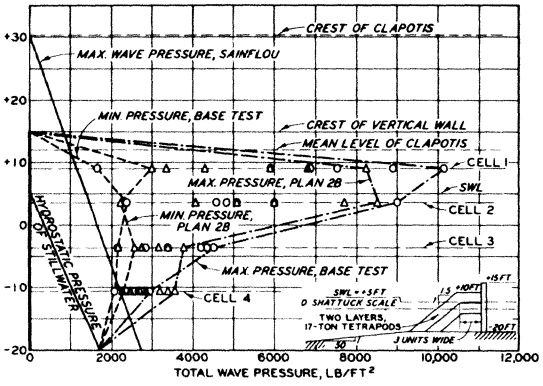
500 0 500 1000 1500 2000

PLATE I



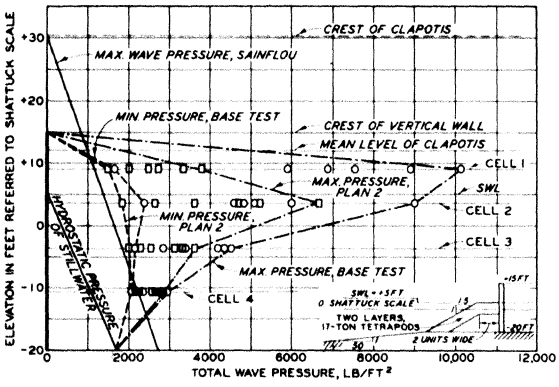

PLAN 1

27-TON TETRAPODS, TWO LAYERS
CROWN ELEVATION OF ARMOR LAYER = +5 FT SHATTUCK SCALE
COVER LAYER TWO UNITS WIDE AT BASE OF CROWN



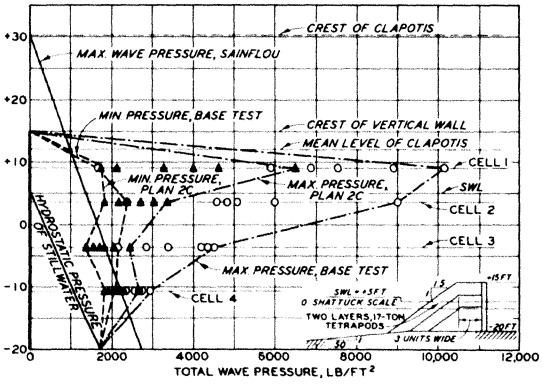

PLAN 2B

17-TON TETRAPODS, TWO LAYERS
CROWN ELEVATION OF ARMOR LAYER = +10 FT SHATTUCK SCALE
COVER LAYER THREE UNITS WIDE AT BASE OF CROWN



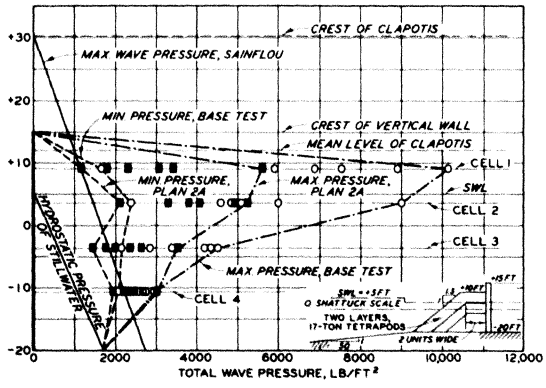

PLAN 2

17-TON TETRAPODS, TWO LAYERS
CROWN ELEVATION OF ARMOR LAYER = +5 FT SHATTUCK SCALE
COVER LAYER TWO UNITS WIDE AT BASE OF CROWN



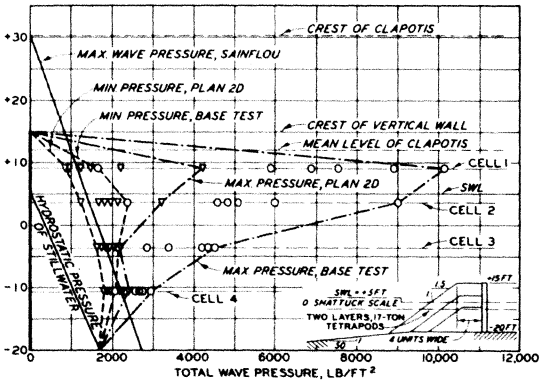

PLAN 2C

17-TON TETRAPODS, TWO LAYERS
CROWN ELEVATION OF ARMOR LAYER = +15 FT SHATTUCK SCALE
COVER LAYER THREE UNITS WIDE AT BASE OF CROWN




PLAN 2A

17-TON TETRAPODS, TWO LAYERS
CROWN ELEVATION OF ARMOR LAYER = +10 FT SHATTUCK SCALE
COVER LAYER TWO UNITS WIDE AT BASE OF CROWN




PLAN 2D

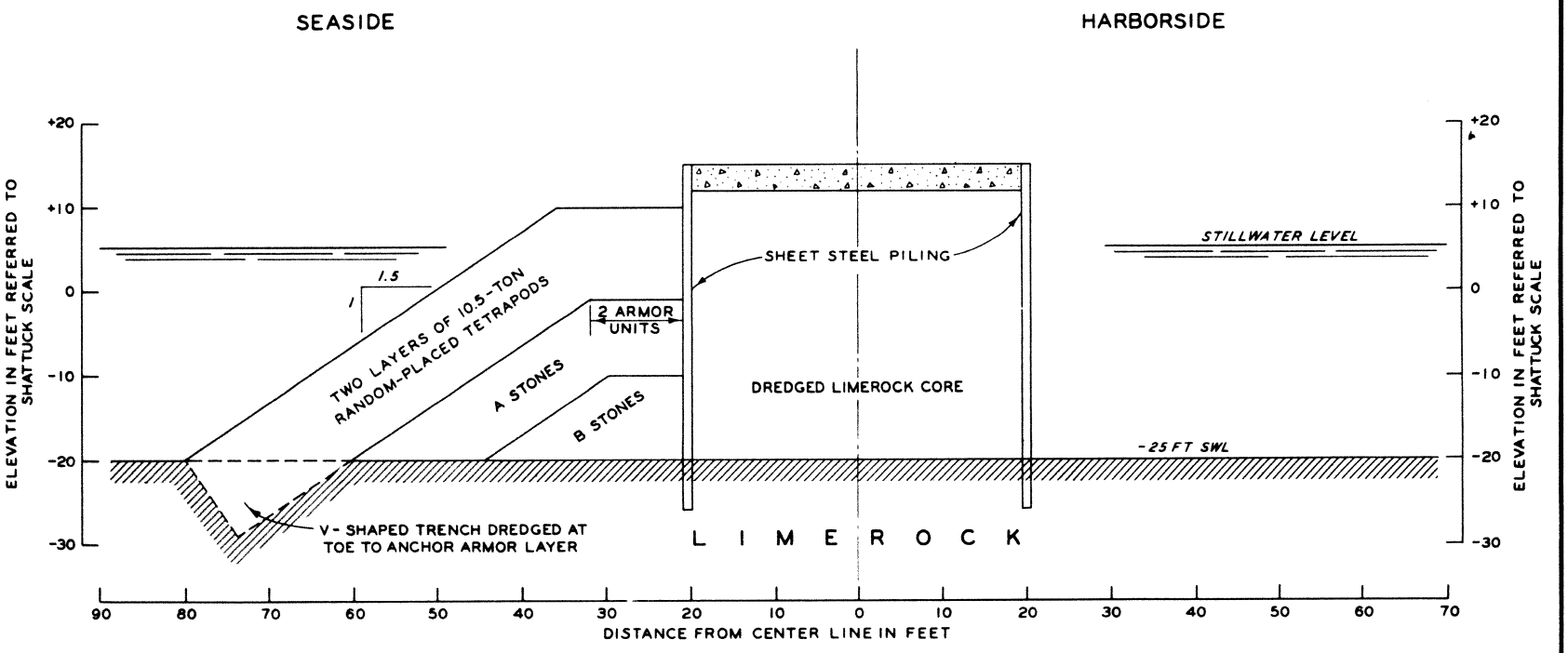
17-TON TETRAPODS, TWO LAYERS
CROWN ELEVATION OF ARMOR LAYER = +15 FT SHATTUCK SCALE
COVER LAYER FOUR UNITS WIDE AT BASE OF CROWN

**LEGEND**

- ○ BASE TEST (NO WAVE ABSORBER ON SEASIDE)
- ● PLAN 1
- □ PLAN 2
- ■ PLAN 2A
- △ PLAN 2B
- ▲ PLAN 2C
- ▽ PLAN 2D

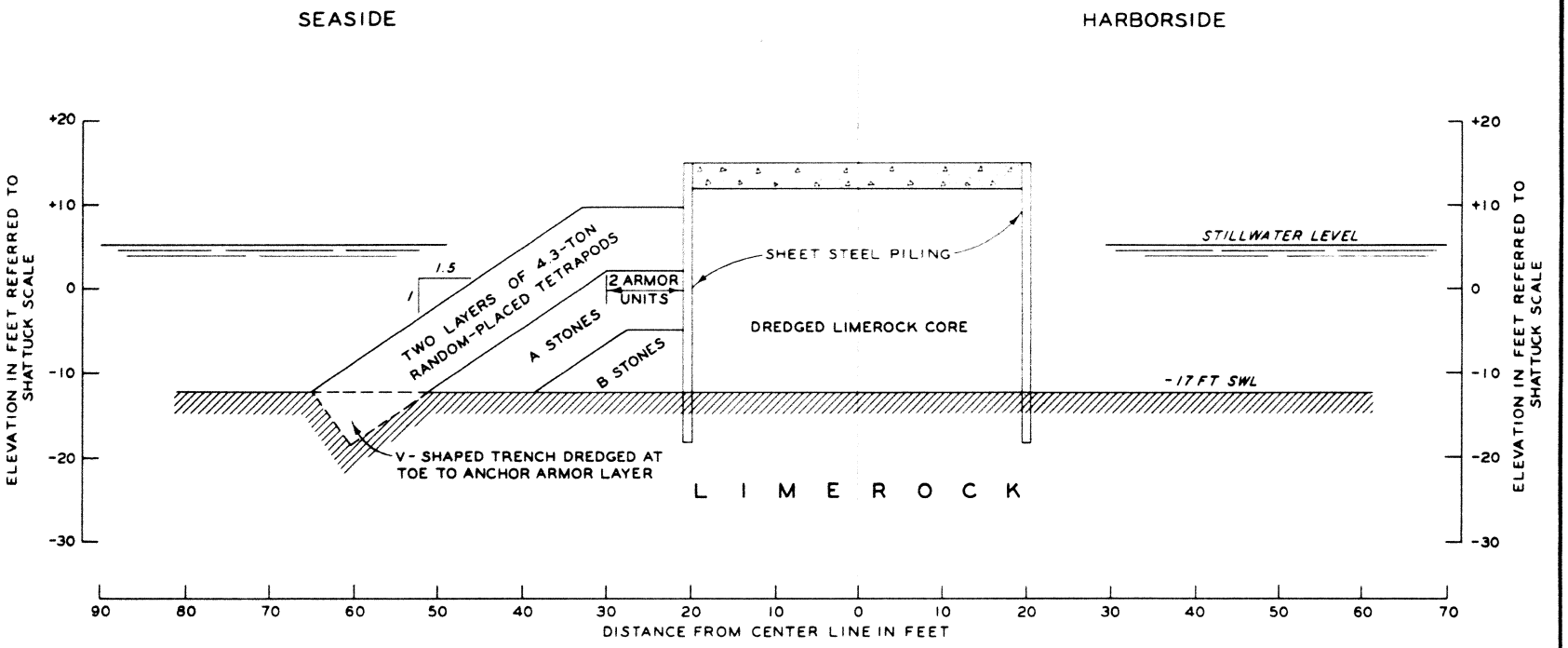
# WAVE PRESSURES ON VERTICAL WALL

TETRAPOD-ROCK WAVE ABSORBER ON SEASIDE
WAVE PERIOD = 11 SEC, BREAKING WAVE HEIGHT = 18 FT
DEEP-WATER WAVE HEIGHT = 17 FT




NOTE: WATER DEPTH (d) AT TOE OF BREAKWATER SLOPE = 25 FT REFERRED TO SWL
STILLWATER LEVEL = +5 FT SHATTUCK SCALE

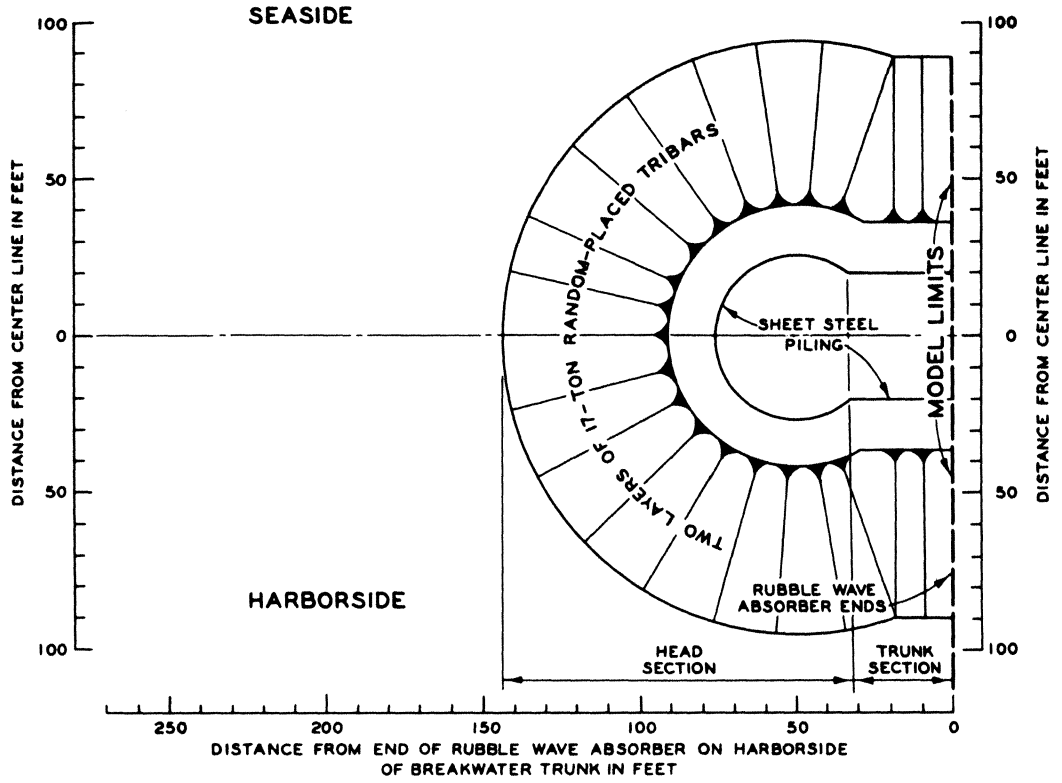
# ELEMENTS OF TEST SECTION
# WEST BREAKWATER TRUNK
## PLAN 4A



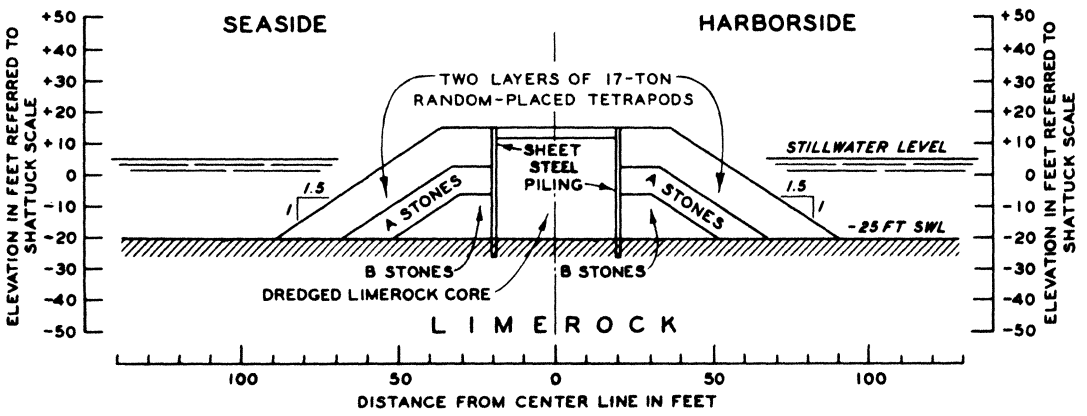

NOTE: WATER DEPTH (d) AT TOE OF BREAKWATER SLOPE = 17 FT REFERRED TO SWL
STILLWATER LEVEL = +5 FT SHATTUCK SCALE

# ELEMENTS OF TEST SECTION WEST BREAKWATER TRUNK

PLAN 5A




PLAN OF WEST BREAKWATER HEAD AND TRUNK

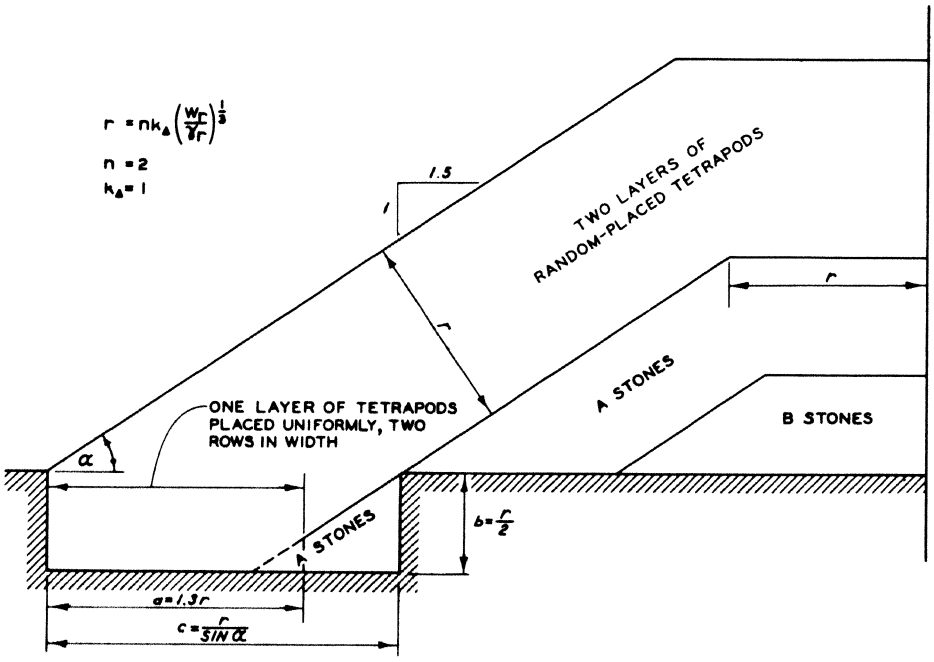



SECTION AT END OF RUBBLE WAVE ABSORBER ON HARBORSIDE OF BREAKWATER TRUNK

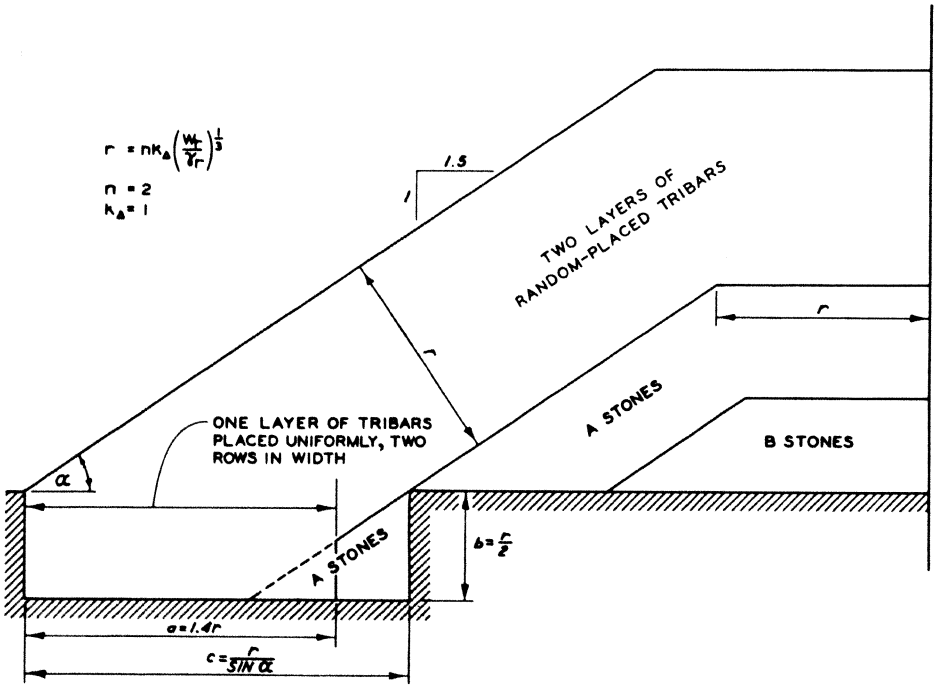
NOTE: WATER DEPTH (d) AT TOE OF BREAKWATER SLOPE = 25 FT REFERRED TO SWL
STILLWATER LEVEL = +5 FT SHATTUCK SCALE

## ELEMENTS OF TEST SECTION WEST BREAKWATER HEAD AND TRUNK

PLAN 8




TETRAPOD ARMOR LAYER

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13. ABSTRACT

Tests were conducted on 1:36-, 1:45-, 1:48-, and 1:56-scale models of breakwater trunks and heads constructed of quadripods, tetrapods, and tribar armor layers to obtain data from which alternate designs could be developed for construction of breakwaters at Nassau Harbor, Nassau, New Providence, Bahamas. It was determined that because of the very large cross section required it would be uneconomical to construct a mound of rubble on the seaside of an impermeable vertical wall capable of reducing the maximum wave-generated shock pressures to values obtained from the Sainflou theory. Values of $K_{\Delta}$ in the WES breakwater stability equation were determined for design of tetrapod and tribar armor layers for both the trunk and head of the west breakwater. Stability of the east breakwater proposed for Nassau Harbor was not investigated; design coefficients for the west breakwater are applicable to geometrically similar structures for the east breakwater. A method was developed for placing armor units in a rectangular trench at the toe of the slope of the cover layer to prevent en masse sliding of the armor units.

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| 14. KEY WORDS | LINK A ROLE | LINK A WT | LINK B ROLE | LINK B WT | LINK C ROLE | LINK C WT |
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| Hydraulic models | | | | | | |
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