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ST. PAUL HARBOR BREAKWATER STABILITY STUDY ST. PAUL, ALASKA

Hydraulic Model Investigation

by

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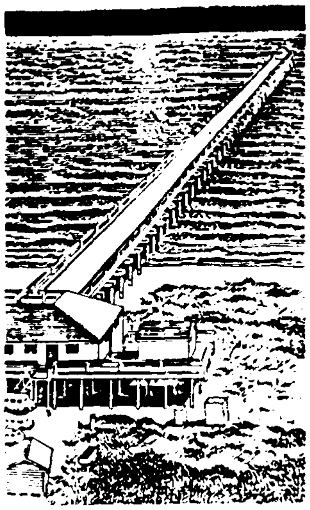
US Army Corps of Engineers



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

A hydraulic model investigation of a proposed breakwater extension for St. Paul Harbor, St. Paul, Alaska, was conducted at a geometrically undistorted scale of 1:38.5, model to prototype. Tests were conducted for a typical cross section at the -25 ft mllw contour. Data collected included wave runup, rundown, overtopping rate, and displacement of armor stones. Maximum breaking wave conditions were used for prototype wave periods of 11, 14, and 16 sec.

Breakwater cross sections tested were (a) the proposed design with a prototype crest elevation of +37.0 ft mllw and a sea-side slope of 1V:2.5H; and (b) revisions to the proposed design with crest elevations of +30.0 ft mllw and +32.0 ft mllw and sea-side slopes of 1V:2H. The plan with a crest elevation of +32.0 ft mllw was determined to be the most economical structure that provided satisfactory stability and overtopping rates.

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PREFACE

The model investigation reported herein was initially requested by US Army Engineer District, Alaska (NPA), to US Army Engineer Waterways Experiment Station (WES) at a conference held at WES on 8 May 1987. Funding authorization by NPA was granted in NPA Intra-Army Order E86874040 dated 14 May 1987, and Change Orders No. 1 dated 8 Jul 1987, No. 3 dated 27 Aug 1987, No. 5 dated 2 Oct 1987, and No. 6 dated 30 Nov 1987.

Model tests of the breakwater stability were conducted at WES during the period May 1987 to Dec 1987 under the general direction of Dr. J. R. Houston, Chief, Coastal Engineering Research Center (CERC), Mr. C. E. Chatham, Chief, Wave Dynamics Division, and Mr. D. D. Davidson, Chief, Wave Research Branch (WRB). Tests were conducted by Mr. D. L. Ward, Hydraulic Engineer, WRB, assisted by Mr. M. P. Thomas, Engineering Technician, WRB. This report was prepared by Mr. Ward and edited by Mrs. N. Johnson, Information Technology Laboratory, under the Inter-Governmental Personnel Act.

Liaison was maintained during the course of the investigation with NPA and US Army Engineer Division, North Pacific (NPD), by means of conferences, progress reports, and telephone conversations. Point of contact with NPA was Mr. Kenneth Eisses; point of contact with NPD was Mr. John Oliver.

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
cubic feet	0.02831685	cubic metres
feet	0.3048	metres
inches	2.54	centimetres
miles (US statute)	1.609347	kilometres
pounds (force)	4.448222	newtons
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
tons (2,000 pounds, mass)	907.1847	kilograms

ST. PAUL HARBOR BREAKWATER STABILITY STUDY

ST. PAUL, ALASKA

Hydraulic Model Investigatior

PART I: INTRODUCTION

The Prototype

1. St. Paul Island, Alaska, is the northernmost and largest of the Pribilof Islands, located in the central-southeast Bering Sea about 200 miles* north of the Aleutian Islands (Figure 1). Two-thirds of the world's population of northern fur seals migrate to the islands annually for breeding, and the islands' economy has been based on the fur seal industry since the Russians settled native Aleuts on the islands in the 1800's to harvest the seals.

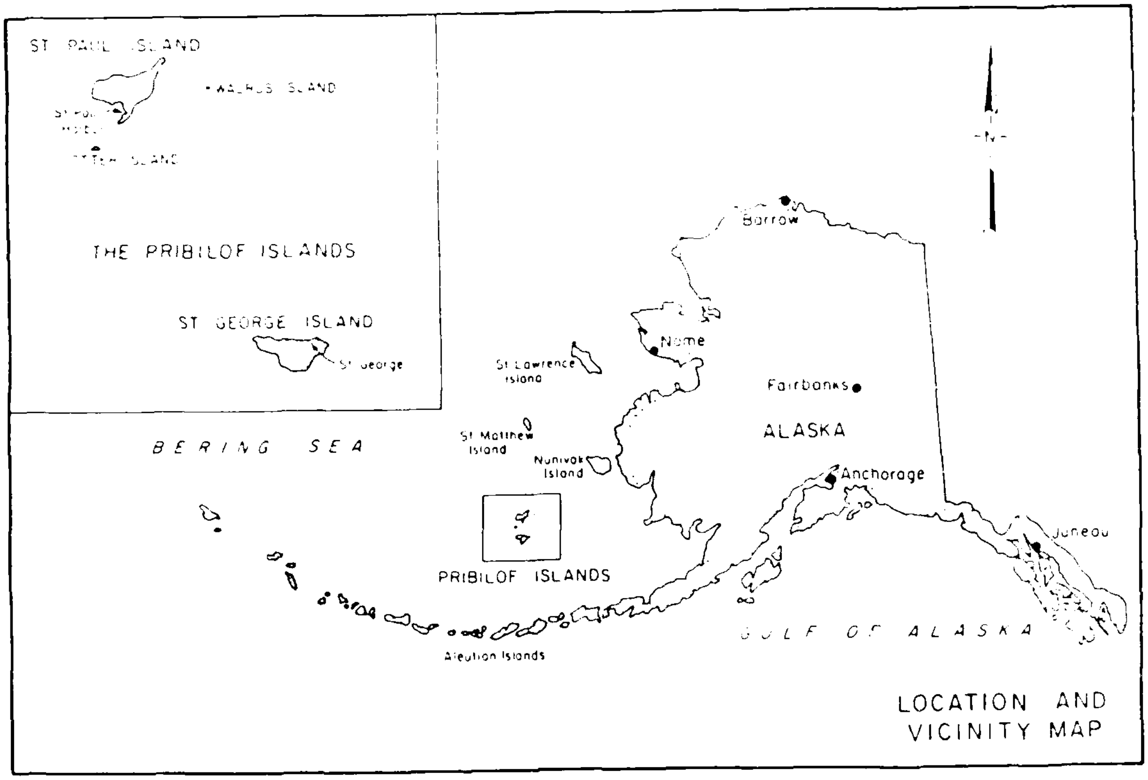

Figure 1. Project location map

* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

2. The recent ban on the harvest of marine mammals in the United States has forced the inhabitants to seek a different basis for their economy. The Bering Sea surrounding St. Paul Island has tremendous potential for commercial fisheries of shrimp, crab, and bottom fish, but lacks a port to provide the services necessary to support a fishing fleet in the region. The inhabitants of St. Paul are therefore developing a harbor on the southeast side of the island to meet the needs of the fishing industry.

3. The island is subjected to windy periods throughout the year, with frequent storms from October to April accompanied by gale-force winds producing blizzard conditions. Under prolonged north or northeast winds between January and April, the ice pack may move south to completely surround the island. Wave heights in excess of 25 ft are expected offshore of St. Paul Harbor on at least an annual basis. The project site at Village Cove is directly exposed to deepwater waves approaching from the west and southwest, and a breakwater is required to protect the harbor against waves from these directions. St. George Island and Otter Island provide some protection from the south and southeast, and the site is otherwise in the lee of St. Paul Island.

4. The original breakwater at the harbor was constructed using the "berm breakwater" concept, but failed during storms in 1984. A rubble-mound breakwater was completed to a 750-ft length in 1985, and an extension to 1,800 ft has been proposed. The extension would be of rubble-mound construction, using stone from a local quarry except for the primary armor layer. With a proposed design weight of 14 to 22 tons each, the primary armor stones exceed the capabilities of the local quarry and would be barged 500 miles from a quarry near Nome, Alaska.

The Problem

5. The existing breakwater has survived two winter seasons and appears to be functioning well, but the proposed addition would extend the breakwater into deeper water with a more severe wave climate. A proposed design was prepared by Tetra Tech, Inc.,* to provide the necessary structural stability. Developed from design curves, equations, and experience, the plan had not yet

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* Tetra Tech, Inc. 1987. "St. Paul Harbor and Breakwater Technical Design Report," Pasadena, CA.

been tested in a model. Since mobilization and demobilization costs are extremely high due to the remoteness of the site, it was particularly important that the structure be tested to check the adequacy of the design.

Purpose of Model Study

6. The purpose of the model study was twofold. First, evaluate overall stability of the proposed breakwater and determine its wave runup and overtopping characteristics when exposed to a range of design wave and water level conditions. Second, based on results of initial tests, make revisions to the proposed breakwater design (increase or decrease armor stone size and/or modify structure geometry) and test adequacy of the revised design when exposed to the same design wave and water level conditions.

PART II: THE MODEL

Design of Model

7. Tests were conducted at a geometrically undistorted scale of 1:38.5, model to prototype. Scale selection was based on the absolute size of the model breakwater section necessary to preclude stability scale effects,* available weights of model armor stone, capabilities of the available wave generator, and depths of water at the toe of the breakwater sections to be modeled. Based on Froude's model law** and a linear scale of 1:38.5, the following model to prototype relationships were derived. Dimensions are in terms of length, L , and time, T .

Characteristic	Dimension	Scale Relations Model:Prototype
Length	L	$L_r = 1:38.5$
Area	L^2	$A_r = (L_r)^2 = 1:1,480$
Volume	L^3	$V_r = (L_r)^3 = 1:57,100$
Time	T	$T_r = (L_r)^{1/2} = 1:6.20$

8. With the exception of the primary armor layer, the prototype extension will be built with stone from the Kaminista Quarry on St. Paul Island, having a specific weight of 176 pcf. In the model, rough angular stone with a specific weight of 165 pcf was used for these portions of the breakwater. The primary armor layer in the prototype will be constructed with stone from Cape Nome Quarry near Nome, Alaska, having a specific weight of 166 pcf. A site visit to the Cape Nome Quarry by pe sonnel from the US Army Engineer Watcrways Experiment Station's (WES) Coastal Engineering Research Center (CERC) and the US Army Engineer District, Alaska (NPA), in June 1987 determined that 50 percent of the armor stones have a rough angular shape, while the remaining 50 percent have a more regular parallelepiped shape. An example of parallelepiped-shaped stones at Cape Nome Quarry is shown in Figure 2. In

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* R. Y. Hudson. 1975. "Reliability of Rubble-Mound Breakwater Stability Models," Miscellaneous Paper H-75-5, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

** J. C. Stevens, C. E. Bardsley, E. W. Lane, and L. G. Straub. 1942. "Hydraulic Models," Manuals on Engineering Practice No. 25, American Society of Civil Engineers, New York.

Figure 2. Parallelepiped-shaped stones from Cape Nome Quarry, Nome, Alaska

the model, the parallelepiped-shaped stones were handmade and shaped to meet general prototype dimensions from granite blocks having a specific weight of 167 pcf, whereas the angular primary armor stones had a specific weight of 165 pcf. Assuming a specific weight of 64.0 pcf for seawater and 62.4 pcf for fresh water, the weights of individual stones used in the model were determined by the transference equation

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

where

W_r = weight of an individual stone, lb

subscripts m,p = model and prototype values, respectively

γ_r = specific weight of an individual stone, pcf

L_m = linear scale of the model

L_p = linear scale of the prototype

S_r = specific gravity of an individual stone relative to the water in which the breakwater is constructed, i.e., $S_r = \gamma_r/\gamma_w$

γ_w = specific weight of water, pcf

Test Facilities and Equipment

9. All stability tests were conducted in a 5-ft-wide, 4-ft-deep, 119-ft-long wave flume equipped with a vertical-displacement wave generator capable of producing monochromatic waves of various periods and heights (Figure 3). The sea-side toe of the breakwater was located 87 ft from the wave generator, and was preceded by 50 ft of 1:100 slope, 10 ft of 1:20 slope, and 27 ft of flat bottom. Thus, the structure toe was 1 ft above the flat bottom of the wave flume.

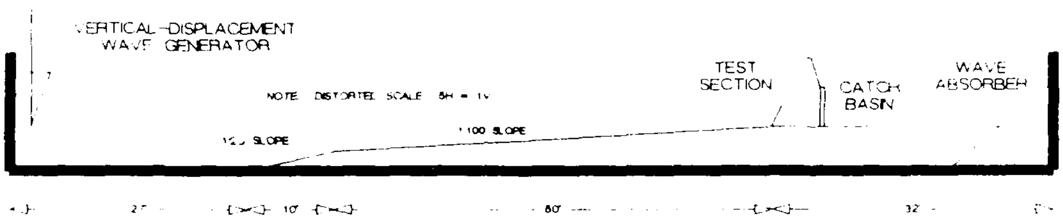

Figure 3. Wave flume cross section

10. Following construction of the local bathymetry and prior to installation of the first test section, the test flume was calibrated for the wave periods and water depths chosen for this study. Test waves of the required characteristics were generated by varying the frequency and amplitude of the wave generator plunger. Changes in water-surface elevation (wave heights) as a function of time at the top of the 1:100 slope were measured by electrical resistance gages and recorded on chart paper by an electrically operated oscillograph. Measurements taken in this way avoid waves reflected from the structure and are analogous to hindcast wave conditions.

Description of Test Section

11. The breakwater design proposed by Tetra Tech, Inc.,* is shown in Figure 4. The test section modeled a cross section of the proposed design from the seaward toe to the roadway on the harbor side of the structure, including the core, bedding layer, underlayer, and primary armor layer.

12. The proposed design included a 700-ft caisson-style dock on the

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* Op. cit., page 5.

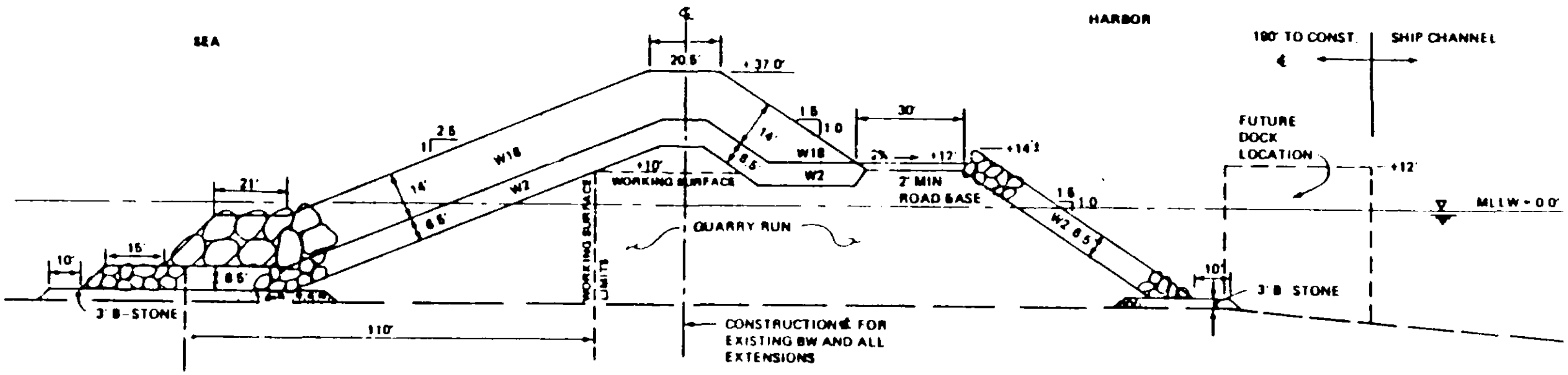

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Figure 4. Proposed design of St. Paul Harbor breakwater extension

harbor side of the breakwater in addition to the 200 ft of dock along the existing breakwater. As the caissons would block flow through the prototype breakwater, the harbor side of the model was made impervious with a plywood barrier. In order to measure the amount of overtopping that would be reaching the roadway in the prototype, the plywood barrier was positioned at the sea-side edge of the roadway, and a catch basin was placed at the location of the roadway. The barrier and the catch basin extended the width of the flume and vertically from the floor of the flume to +10.0 ft mllw, the top elevation of the bedding layer of the proposed roadway. The catch basin was made of sheet metal and measured 6 in. from front to back. A vertical board was placed behind the catch basin when necessary to prevent wave splash from extending past the catch basin.

13. Tests conducted on Plans 3.1 and 3.2 (paragraphs 55 and 56) required that the breakwater be porous. To achieve this, a series of holes were drilled in the plywood barrier and the catch basin was removed from the back side of the barrier. A new catch basin was constructed and located 6 in. behind the barrier with a ramp to carry the overtopping to the basin. The basin was built on legs to leave a 4-in. gap between the bottom of the basin and the floor of the flume so water flow through the breakwater would not be impeded.

Method of Constructing Test Section

14. The model was constructed in a manner to simulate as closely as possible prototype construction. The bedding, core, and secondary armor layers were each placed by dumping from a shovel to predetermined grade lines. Hand trowels were used to compact the core material in an effort to simulate natural consolidation which would result from wave action during construction of the prototype breakwater. The primary armor layer, two stones thick, consisted of a 50:50 mixture of parallelepiped-shaped stones and rough angular stones. Above the toe (approximately 0.0 ft mllw), individual stones in the primary armor layer were placed by hand to simulate placed-stone construction methods described in paragraph 16. The toe of the prototype will be below the still-water level (swl), where special stone placement would not be practical. Therefore, random placement was used below 0.0 ft mllw.

15. Random placement was achieved by selecting a stone at random from a stockpile and placing it in contact with adjacent stones on the structure. No

attempt was made to orient the axes of the stone or key the stone to the structure. Placement was made based on the assumption that prototype placement would be underwater and the crane operator would be unable to see the placement of the stone, but would know the location in which to place it and could tell when the stone was butted against adjacent stones.

16. For the placed-stone construction, a small group of stones were randomly selected and placed in a stockpile. The structure was then built by choosing from this stockpile the stone that would best fit the next position in the armor layer. The stone was keyed into the structure with the long axis perpendicular to the slope or crest. No attempt was made to key the stone into the structure to any greater extent than would be feasible when using prototype construction equipment.

17. The main differences between random placement and placed-stone construction are that in placed-stone construction a stone is selected from a small stockpile to best fit the next position in the armor layer, and the stone is oriented to key into the structure with the long axis perpendicular to the slope or crest. Construction of the model was based on the assumptions that prototype construction would be conducted with a crane operator experienced in placed-stone construction techniques, and that a comprehensive inspection program would be implemented to ensure quality construction.

Method of Measuring Runup, Rundown, and Overtopping

18. Runup and rundown were measured by placing a measuring stick along the sea-side slope of the structure on the outside of a glass viewing plate in the flume wall. Runup and rundown along the sea-side slope of the structure were then read directly off the measuring stick. Measurements along the slope were converted to elevations above or below mllw by trigonometry.

19. Overtopping rates were determined by measuring the water accumulated in a catch basin during a specific test time and converting the measured accumulation to an average flow rate.

Selection of Test Conditions

20. Selection of test conditions was discussed at a meeting among CERC, NPA, US Army Engineer Division, North Pacific (NPD), and Headquarters, US Army Corps of Engineers (HQUSACE), on 6 May 1987.

21. The breakwater was tested for stability and survivability at swl's of 0.0 and +5.0 ft mllw. These swl's were selected by NPA to represent tide range plus storm setup. Mean higher high water at St. Paul Harbor is +3.2 ft mllw, with an estimated extreme storm high tide of +6.0 ft mllw. Toe stability tests were conducted at a swl of -5.0 ft mllw to concentrate wave action on the structure toe, and at +5.0 ft mllw to allow the maximum wave energy to reach the structure. Overtopping tests were conducted at +5.0 ft mllw to maximize the rate of overtopping.

22. Twenty-four storm events selected from wind records taken between 1962 and 1981 were hindcasted to a location offshore of Village Cove in a depth of 66 ft by Delft Hydraulics Institute* to determine the recurrence intervals of extreme offshore wave climates. The results of the hindcast were summarized by Tetra Tech, Inc.,** and are presented in Table 1.

Table 1

Recurrence Intervals for Extreme Wave Climates Offshore of Village Cove, St. Paul Island, Alaska

Recurrence Interval years	Significant Wave Height ft	Wave Period sec
1	25.3	11
10	28.5	12
50	30.5	13
100	31.2	14

Note: Recurrence interval stated in terms of 3-hr duration per 1-, 10-, 50-, and 100-year period.

23. In addition, wave data measured by the offshore drill rig Ocean Odyssey in December, 1984, and analyzed by Tetra Tech, Inc.,** showed that "with $H_s \geqq 16$ ft and $H_s \geqq 24$ ft, the modal wave periods were approximately 11.0 and 13.0 sec, respectively. Significant wave periods as long as 14.6 sec were observed for sea states where $H_s \geqq 24$ ft."

* Delft Hydraulics Institute. 1982. "Pribilof Islands, Wave Study, Alaska, U.S.A.," Final Report, Vol 1, The Netherlands.
** Op. cit., page 5.

24. Based on these results, it was decided to use wave periods of 11 and 14 sec for the structural stability tests.

25. Hindcasts of the 13 November 1984 and 7 December 1984 storms that damaged the initial breakwater construction showed that peak periods associated with these storms were 16 and 13 sec, respectively, and offshore significant wave heights were 30 and 22 ft, respectively. Therefore, wave period of 16 sec was chosen for the survivability tests. A 10-year (1966 through 1975) wave hindcast study of the wave climate offshore of Village Cove conducted by CERC indicated that 16-sec significant wave periods could be expected about once a year in that region.

26. The proposed design placed the sea-side toe of the breakwater at approximately -25.0 ft mllw. With the design high-water level of +5.0 ft mllw, the depth at the toe would be 30 ft. Since measured and hindcast offshore wave heights exceed the heights that are obtainable at the structure toe, depth-limited breaking waves were chosen for the tests. In order to find the most severe breaking wave (i.e., the most damaging wave) for each wave period/water depth combination, the initial breakwater section was installed in the flume, and the stroke adjustment on the wave generator was increased in small increments until the wave condition which produced the most detrimental action on the structure was observed. Wave heights of higher amplitude would break seaward of the structure and dissipate their energy so that they were less damaging than the critically tuned wave, whereas waves of lower amplitude did not form the critical breaking wave. In this manner, wave heights that imparted maximum breaking energy on the sea-side slope of the structure were obtained. Wave heights thus selected are listed in Table 2.

Table 2

Depth-Limited Breaking Wave Heights Selected to Maximize Breaking Wave Energy on the Sea-Side Slope of the Proposed Breakwater at St. Paul Island, Alaska

Wave Period sec	Still-Water Level, ft, mllw 0.0	+5.0
11	15.6	20.1
14	16.7	21.2
16	22.3	24.1

27. After the first four tests on Plan 1 (paragraph 36) demonstrated the stability of the proposed structure, a meeting was held on 8 October 1987 among CERC, NPA, NPD, HQUSACE, City of St. Paul, and Tetra Tech, Inc., to discuss revisions of the proposed design. Concern was expressed at that time over the stability of the sea-side toe, and additional tests were requested to verify its stability. Accordingly, an attempt was made to maximize wave forces on the toe by "tripping" the incident wave train with a submerged offshore reef to increase the breaking action on the toe. The reef was constructed of a double row of large stones and represented a prototype reef 10 ft high by 25 ft wide, centered 75 ft seaward of the toe for tests at a swl of -5.0 ft mllw and 115 ft seaward of the toe for tests at a swl of +5.0 ft mllw.

28. With the offshore reef in place for toe stability tests, wave heights were adjusted to impart maximum breaking wave energy on the sea-side toe. Wave heights thus selected are given in Table 3.

Table 3

Depth-Limited Breaking Wave Heights Selected to Maximize Breaking Wave Energy on the Sea-Side Toe of the Proposed Breakwater at St. Paul Island, Alaska, With an Offshore Reef Placed to Trip the Incident Waves

Wave Period sec	Still-Water Level, ft, mllw -5.0	+5.0
14	16.1	*
16	17.7	23.5

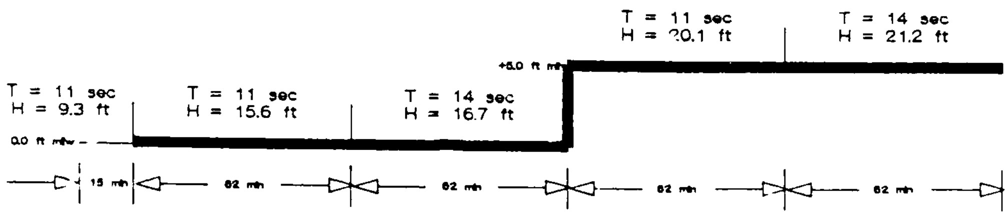
* No tests were conducted using a reef with 14-sec waves at a swl of +5.0 ft mllw.

29. Test time was accumulated on the structure in 30-sec cycles followed by a 5- to 7-min stilling time. This was done to minimize contamination of the incident waves by reflected waves. The cycles were then combined into simple storm hydrographs for the stability and survivability tests to include the different water levels, wave heights, and wave periods tested.

Stability Test

30. For the stability tests, 15 min of 11-sec waves were run at a swl of 0.0 ft mllw with a moderate wave height to shakedown the structure. This

represented typical prototype consolidation caused by wave action during construction. Maximum breaking wave conditions were then used for the remainder of the hydrograph, which consisted of 62 min of 11-sec waves followed by 62 min of 14-sec waves, both at a swl of 0.0 ft mllw, then 62 min of 11-sec waves and 62 min of 14-sec waves at a swl of +5.0 ft mllw. The stability hydrograph represented a storm of 4-hr and 8-min duration, after shakedown. The stability hydrograph is shown in Figure 5.



Step	Still-Water Level ft, mllw	Wave Period sec	Wave Height ft	Test Duration min	Wave Type
	0.0	11.0	9.3	15.5	Shakedown
1	0.0	11.0	15.6	62.0	Breaking
2	0.0	14.0	16.7	62.0	Breaking
3	5.0	11.0	20.1	62.0	Breaking
4	5.0	14.0	21.2	62.0	Breaking

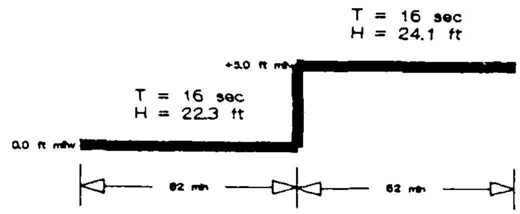
Figure 5. Hydrograph for stability tests

Survivability test

31. The structure was not re'uilt after the stability test; therefore, a shakedown was not necessary for the survivability test. Using maximum breaking wave conditions with a 16-sec period, the survivability hydrograph consisted of 62 min at a swl of 0.0 ft mllw followed by 62 min at a swl of +5.0 ft mllw. The survivability hydrograph is shown in Figure 6.

Toe stability test

32. Toe stability tests were conducted using wave periods of 14 and 16 sec at a swl of -5.0 ft mllw, and 16 sec at +5.0 ft mllw. Each of the tests consisted of 62 min of waves at each wave period/water depth combination. A submerged offshore reef (paragraph 27) was used in each of the toe stability tests.



Step	Still-Water Level ft, mllw	Wave Period sec	Wave Height ft	Test Duration min	Wave Type
1	0.0	16.0	22.3	62.0	Breaking
2	5.0	16.0	24.1	62.0	Breaking

Figure 6. Hydrograph for survivability test

Overtopping test

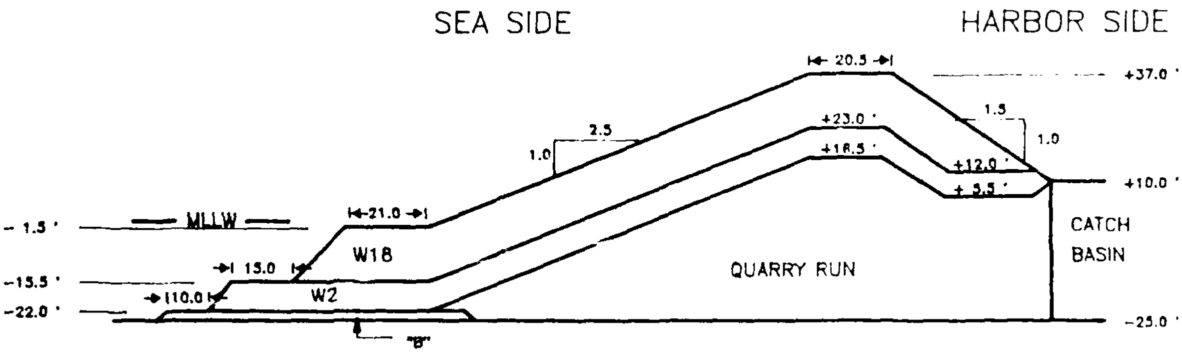
33. Runup and overtopping were measured during the stability and survivability tests on each plan, and a special overtopping test series was conducted on Plans 3.1 and 3.2. This latter test consisted of 62 min of 16-sec, 24.1-ft waves at a swl of +5.0 ft mllw.

PART III: TESTS AND TEST RESULTS

Plan 1

Description

34. Plan 1 (Figure 7, Photos 1-6) modeled a section of the trunk of the proposed breakwater extension as designed by Tetra Tech, Inc.* Prototype dimensions reflected in the model included a crown elevation of +37.0 ft mllw, crown width of 20.5 ft, depth at toe of -25.0 ft mllw, and side slopes of 1:2.5 on the sea-side from the crown to the top of the toe, and 1:1.5 on the harbor side from the crown to a roadway running along the breakwater at an elevation of +12.0 ft mllw.



Note: All elevations in feet referred to mllw.

Material Characteristics

Size	Model	Prototype
W18	0.421-0.662 lb @ 165 pcf	14-22 tons @ 166 pcf
W2	0.057-0.106 lb @ 165 pcf	2,800-5,200 lb @ 176 pcf
Quarry Run	<0.057 lb @ 165 pcf	<2,800 lb @ 176 pcf
B	<0.008 lb @ 165 pcf	<400 lb @ 176 pcf

Figure 7. Plan 1, St. Paul Harbor breakwater stability study

* Op. cit., page 5.

35. The proposed design included a core of quarry run material weighing less than 2,800 lb overlain by two layers (6.5 ft) of W2 stone weighing 2,800 to 5,200 lb and covered with two layers of W18 armor stone (about 14 ft) weighing 14 to 22 tons. A 3-ft-thick blanket was placed under the seaward toe of the structure using B stone weighing less than 400 lb.

Results

36. Five tests as outlined in Table 4 were conducted with Plan 1. A stability test (Test 1) was followed by a survivability test (Test 2); the structure was torn down and rebuilt and the tests repeated (Tests 3 and 4); and the structure was then tested for toe stability (Test 5). Wave runup, rundown, and overtopping results of the tests are given in Table 4. Plan 1 is shown before Test 3 in Photos 1 and 2, after Test 4 in Photos 3 and 4, and after Test 5 in Photos 5 and 6.

37. No stones were displaced during any of the tests. Four stones were observed rocking for a maximum of six cycles during Test 1; otherwise, no stone movement was seen.

38. Runup, rundown, and overtopping were measured during the first four tests. During the stability tests, maximum runup of +26.4 ft mllw occurred during the 14-sec, 21.2-ft waves at a swl of +5.0 ft mllw. During the survivability test with 16-sec waves at a swl of +5.0 ft mllw, a maximum runup of +30.7 ft mllw was recorded.

39. Wave overtopping consisted mainly of flow through the structure, along with some splashover. At the low-water level (0.0 ft mllw), the overtopping was insignificant, with a rate (trace) of no more than 0.0002 cfs/ft of breakwater. With the swl at +5.0 ft mllw, the maximum overtopping rates were 0.011 cfs/ft for the 11-sec waves, 0.003 cfs/ft for the 14-sec waves, and 0.132 cfs/ft for the 16-sec waves.

40. The toe stability test (Test 5) demonstrated adequate stability of the toe. No stones were displaced and very little movement was observed.

Plan 2

Description

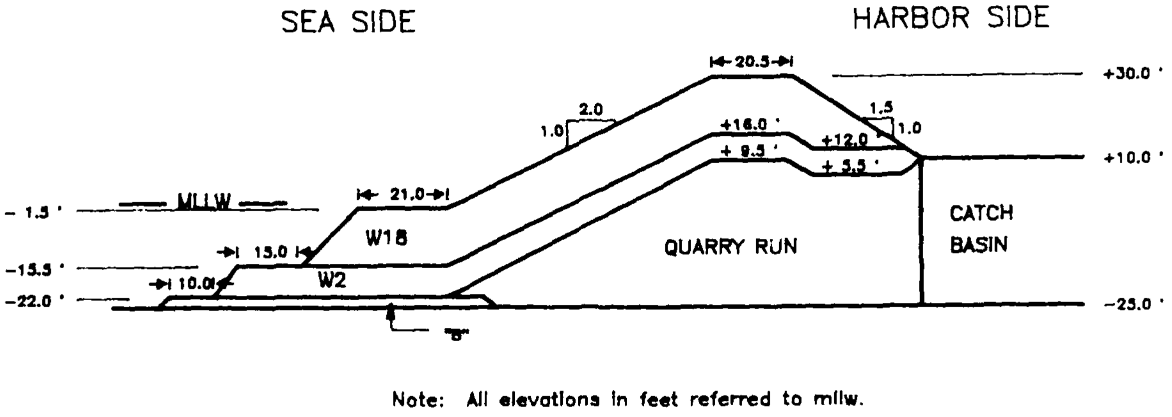
41. Plan 2 (Figure 8, Photos 7-12) was developed based on test results from Plan 1. The crown elevation was lowered to +30.0 ft mllw and the seaside slope was steepened to 1:2 from the crown to the top of the toe. Stone

Table 4

Test Results from Plan 1, St. Paul Harbor Breakwater Study

Test No.	Type	Still-Water Level ft, mllw	Wave Period sec	Wave Height ft	Test Duration min	El of Max Runup ft, mllw	El of Max Rundown ft, mllw	Overtopping Rate cfs/ft	Stones Displaced From Toe
	Shakedown	0.0	11.0	9.3	15.0	*	*	*	0
1	Stability	0.0	11.0	15.6	62.0	10.0	-6.1	Trace	0
		0.0	14.0	16.7	62.0	11.4	-6.1	Trace	0
		5.0	11.0	20.1	62.0	21.4	-9.3	0.011	0
		5.0	14.0	21.2	62.0	26.4	-7.2	0.003	0
2	Survivability	0.0	16.0	22.3	62.0	19.3	-10.0	Trace	0
		5.0	16.0	24.1	62.0	30.7	-9.3	0.132	0
	Shakedown	0.0	11.0	9.3	15.0	*	*	*	0
3	Stability	0.0	11.0	15.6	62.0	10.9	-10.7	Trace	0
		0.0	14.0	16.7	62.0	14.3	-10.9	Trace	0
		5.0	11.0	20.1	62.0	24.3	-8.6	0.002	0
		5.0	14.0	21.2	62.0	26.4	-8.6	0.001	0
4	Survivability	0.0	16.0	22.3	62.0	19.4	-7.9	Trace	0
		5.0	16.0	24.1	62.0	30.7	-3.6	0.124	0
5	Toe stability	-5.0	14.0	16.1	62.0	7.2	-12.1	Trace	0

Note: The structure was disassembled and rebuit after Test 2.
* Data not taken.



Material Characteristics

Size	Model	Prototype
W18	0.421-0.662 lb @ 165 pcf	14-22 tons @ 166 pcf
W2	0.057-0.106 lb @ 165 pcf	2,800-5,200 lb @ 176 pcf
Quarry Run	<0.057 lb @ 165 pcf	<2,800 lb @ 176 pcf
B	<0.008 lb @ 165 pcf	<400 lb @ 176 pcf

Note: W18 on structure toe of Plan 2 selected from smallest 25 percent of W18 range.

Figure 8. Plan 2, St. Paul Harbor breakwater stability study

weights were unchanged, except the toe was constructed of stones selected from the lowest 25-percent weight range of the W18 armor stones.

Results

42. Table 5 summarizes the test conditions and wave runup, rundown, and overtopping results of the six tests conducted on Plan 2. A stability test (Test 6) was followed by a survivability test (Test 7), the structure was disassembled and rebuilt and the tests repeated (Tests 8 and 9), then the structure was tested for toe stability (Tests 10 and 11). Plan 2 is shown before Test 8 in Photos 7 and 8, after Test 9 in Photos 9 and 10, and after Test 11 in Photos 11 and 12.

Table 5

Test Results from Plan 2, St. Paul Harbor Breakwater Study

Test No.	Type	Still-Water Level ft, mllw	Wave Period sec	Wave Height ft	Test Duration min	El of Max Runup ft, mllw	El of Max Rundown ft, mllw	Overtopping Rate cfs/ft	Stones Displaced From Toe
	Shakedown	0.0	11.0	9.3	15.0	*	*	*	0
6	Stability	0.0	11.0	15.6	62.0	20.7	-12.6	Trace	2
		0.0	14.0	16.7	62.0	21.9	-12.6	Trace	0
		5.0	11.0	20.1	62.0	29.1	-8.3	0.023	0
		5.0	14.0	21.2	62.0	28.8	-8.3	0.028	0
7	Survivability	0.0	16.0	22.3	62.0	23.8	-12.9	0.027	0
		5.0	16.0	24.1	62.0	30.0	-7.9	0.733	0
	Shakedown	0.0	11.0	9.3	15.0	*	*	*	0
8	Stability	0.0	11.0	15.6	62.0	18.1	-14.6	Trace	0
		0.0	14.0	16.7	62.0	18.9	-14.6	Trace	0
		5.0	11.0	20.1	62.0	28.2	-12.2	0.042	0
		5.0	14.0	21.2	62.0	29.1	-8.1	0.016	0
9	Survivability	0.0	16.0	22.3	62.0	23.2	-12.7	0.027	0
		5.0	16.0	24.1	62.0	30.0	-8.1	0.733	2
10	Toe stability	-5.0	14.0	16.1	62.0	9.8	-13.8	Trace	0
11	Toe stability	-5.0	16.0	17.7	62.0	*	*	Trace	0

Note: The structure was disassembled and rebuit after Test 2.
* Data not taken.

43. The first cycle of Test 6 displaced two stones from the toe and moved them seaward of the structure. No other stones were displaced during Test 6, nor were any stones dislodged during the repeat of the stability test (Test 8). Stone movement was observed on the structure toe during both survivability tests (Tests 7 and 9), with several stones rocking in the upper layer of the sea-side toe. After the tests, it was observed that several stones had been reoriented, but no actual displacement had occurred.

44. A limited amount of rocking was observed during the initial toe stability test (Test 10) using 14-sec waves, but no stones were displaced. Due to the large amount of movement observed during the 16-sec waves of the survivability test, it was decided to repeat the toe stability test with 16-sec waves (Test 11). Again, there was very little movement. As the toe stability tests were conducted at a shallower depth than the stability and survivability tests, the depth-limited breaking waves were smaller, accounting for the reduction in observed movement.

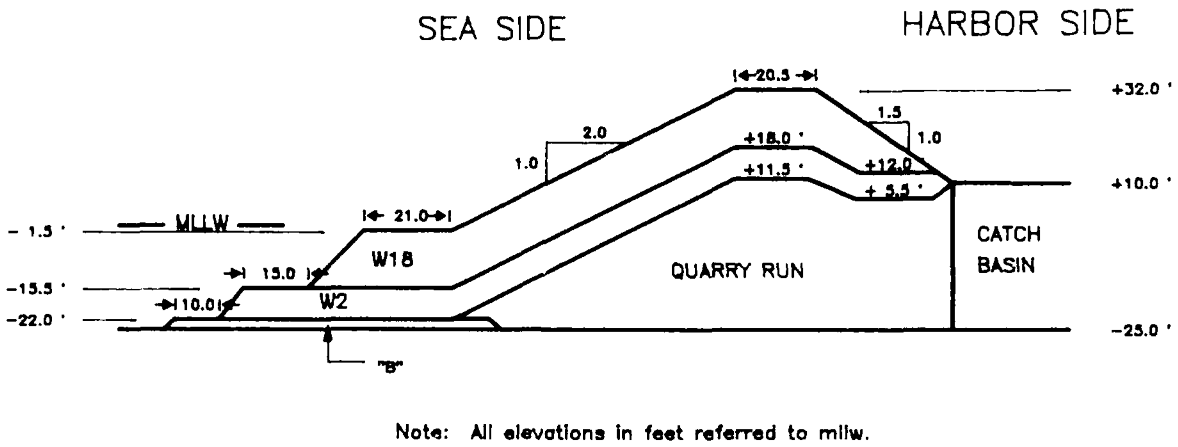
45. Overtopping on Plan 2 for stability tests at both 0.0 and +5.0 ft mllw swl's and survivability tests at 0.0 ft mllw swl consisted mainly of flow through the structure with some splashover, and overtopping rates ranging from traces to 0.042 cfs/ft. Maximum elevations of runup on these tests were about +28 to +29 ft mllw for the stability tests at +5.0 ft mllw swl.

46. During survivability tests at a swl of +5.0 ft mllw, waves overtopped the structure with solid water, as opposed to splash overtopping that had occurred during previous tests. An overtopping rate of 0.733 cfs/ft was recorded during both survivability tests. This is approximately 17 times greater than the maximum overtopping measured for any other test conditions used on Plan 2.

Plan 3

Description

47. The overtopping rate in Plan 2 for 16-sec waves at a swl of +5.0 ft mllw was considered unacceptable; therefore, the crest elevation was raised from +30.0 ft mllw in Plan 2 to +32.0 ft mllw for Plan 3 (Figure 9, Photos 13-18). In addition, the toe of Plan 3 was constructed from the full range of W18 stones, rather than using the lowest 25 percent of the range as in Plan 2, to reduce the amount of toe stone movement.



Material Characteristics

Size	Model	Prototype
W18	0.421-0.662 lb @ 165 pcf	14-22 tons @ 166 pcf
W2	0.057-0.106 lb @ 165 pcf	2,800-5,200 lb @ 176 pcf
Quarry Run	<0.057 lb @ 165 pcf	<2,800 lb @ 176 pcf
B	<0.008 lb @ 165 pcf	<400 lb @ 176 pcf

Figure 9. Plan 3, St. Paul Harbor breakwater stability study

Results

48. Table 6 summarizes the test conditions and wave runup, rundown, and overtopping results of the seven tests conducted on Plan 3. A stability test (Test 12) was followed by a survivability test (Test 13) and a high-water toe stability test (Test 14). The structure was then disassembled, rebuilt, and tested with a stability test (Test 15), survivability test (Test 16), low-water toe stability test (Test 17), and high-water toe stability test (Test 18). Plan 3 is shown before Test 15 in Photos 13 and 14, after Test 16 in Photos 15 and 16, and after Test 18 in Photos 17 and 18.

49. During the first stability test, no stones were displaced with the swl at 0.0 ft mllw. When the depth was increased to +5.0 ft mllw, two stones were displaced during the 11-sec waves and one stone during the 14-sec waves.

Table 6

Test Results from Plan 3 (Tests 12-18), Plan 3.1 (Test 19), and Plan 3.2 (Test 20), St. Paul Harbor Breakwater Study

Test No.	Type	Still-Water Level ft, mllw	Wave Period sec	Wave Height ft	Test Duration min	El of Max Runup ft, mllw	El of Max Rundown ft, mllw	Overtopping Rate cfs/ft	Stones Displaced From Toe
	Shakedown	0.0	11.0	9.3	15.0	*	*	*	0
12	Stability	0.0	11.0	15.6	62.0	17.2	-15.5	Trace	0
		0.0	14.0	16.7	62.0	21.5	-15.5	Trace	0
		5.0	11.0	20.1	62.0	29.6	-13.9	*	2
		5.0	14.0	21.2	62.0	29.1	-13.9	0.022	1
13	Survivability	0.0	16.0	22.3	62.0	18.9	-13.8	0.040	0
		5.0	16.0	24.1	62.0	29.1	-3.4	0.653	0
14	Toe Stability	5.0	16.0	23.5	62.0	31.7	-2.7	*	1
	Shakedown	0.0	11.0	9.3	15.0	*	*	*	0
15	Stability	0.0	11.0	15.6	62.0	17.2	-15.5	Trace	2
		0.0	14.0	16.7	62.0	21.5	-15.5	Trace	0
		5.0	11.0	20.1	62.0	25.7	-12.2	0.014	1
		5.0	14.0	21.2	62.0	29.4	-13.9	0.020	0
16	Survivability	0.0	16.0	22.3	62.0	18.9	-13.8	0.055	3
		5.0	16.0	24.1	62.0	28.9	-3.6	0.450	0
17	Toe stability	-5.0	14.0	16.1	62.0	9.6	-11.9	Trace	0
18	Toe stability	5.0	16.0	23.5	62.0	31.5	-2.7	*	0
19	Overtopping	5.0	16.0	24.1	62.0	*	*	0.123	0
20	Overtopping	5.0	16.0	24.1	62.0	*	*	0.040	0

Note: The structure was disassembled and rebuit after Test 2.
* Data not taken.

No additional stones were displaced by the 16-sec waves (survivability test) until the offshore reef was added for the high-water toe stability test, at which time one additional stone was displaced.

50. The four displaced stones had been located at the front edge of the toe where the stones were exposed to wave action on their front and top sides, and thus were not keyed as tightly into the structure as other stones on the toe. With the stones displaced, there was no additional unraveling and no indication that damage would progress to the point where the stability of the slope was threatened. Little movement was observed on stones other than those along the front edge of the toe.

51. After Plan 3 was rebuilt, it was noticed that the stone placement was not as tight as in previous tests, particularly with the random stone placement on the toe. Construction was a valid representation of prototype construction techniques, and it is difficult to ascertain the tightness of the stones below the water level in the prototype, but additional stone displacement was expected in these tests due to the looser toe construction.

52. Two stones were displaced during 11-sec waves at +0.0 ft mllw swl, an additional toe stone was displaced during 11-sec waves at +5.0 ft mllw swl, and three more toe stones were lost during the survivability test at +0.0 ft mllw swl.

53. In general, Plan 3 showed less movement than Plan 2, although more stones were displaced from Plan 3. It appeared there were fewer stones rocking and less reorientation of stones on the toe of Plan 3. The displaced stones were almost exclusively of the rough angular type. The parallelepiped stones have greater surface contact between them, and therefore exhibited greater stability.

54. Overtopping rates for 16-sec waves at a swl of +5.0 ft mllw (survivability test) were reduced from 0.733 cfs/ft on Plan 2 to 0.653 cfs/ft and 0.450 cfs/ft for the two series of tests on Plan 3.

Plans 3.1 and 3.2

Description

55. Tests 19 and 20, Table 6, were conducted with 16-sec waves at a swl of +5.0 ft mllw on two modified versions of Plan 3 to determine what effects proposed design changes would have on the overtopping rates. The first

modification (Plan 3.1) was to move the catch basin away from the structure and drill holes in the plywood supporting the backside of the test section to make the breakwater porous and allow flow through the structure. The second modification (Plan 3.2) maintained the porous structure and added a row of W18 stones along the crest, one layer high and two layers wide, to block some of the solid water overtopping. Plan 3.2 is shown in Photos 19 and 20.

Results

56. The porous back to the structure (Test 19) reduced the measured overtopping rate from 0.653 and 0.450 cfs/ft (Tests 13 and 16) to 0.123 cfs/ft. There was no noticeable difference in the quantity of water passing over the structure, but flow through the structure was passing through the harbor side of the breakwater below +10.0 ft mllw and no longer entering the catch basin. With the row of W18 stones added to the crest of the structure (Test 20), the overtopping rate was further reduced to 0.040 cfs/ft. Results of the tests on Plans 3.1 and 3.2 are included in Table 6.

Height of Breakwater

57. The design submitted by Tetra Tech, Inc.,* indicated the thickness of two layers of W18 stones would be about 14 ft. As previously agreed with NPA, the test structures were constructed to the indicated elevations for the W2 underlayer, then covered with a double layer of specially placed W18 stones without regard for the final crown elevation achieved.

58. After testing was completed on Plans 3.1 and 3.2, the crest of the breakwater was surveyed by placing a Philadelphia rod vertically on the crest and reading the rod with an engineer's level. Measurements were taken at 19.25-ft (prototype) intervals along the crest, for a total of nine measurements. The survey was taken first across the top of the extra layer of W18 stones used on Plan 3.2, the extra layer was removed and the breakwater crest was surveyed, then the primary armor layer of the test section was removed and the W2 layer surveyed. Results of the survey are shown in Table 7.

59. The survey showed the average elevation of the breakwater crest to be +34.64 ft mllw, indicating the primary armor layer was 16.71 ft thick, or 8.35 ft per layer of W18 stones placed with the long axis perpendicular to the

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* Op. cit., page 5.

Table 7

Elevations from Surveys Taken of Breakwater

Test Structure after Test 20

Layer	Elevation, Prototype Ft Above mllw		
	Average	Minimum	Maximum
W2 underlayer	17.93	16.89	18.70
W18 armor	34.64	33.44	35.83
Extra layer of W18 on crest	38.99	37.29	39.83

crest or slope. The extra layer of W18 stones along the crest of the breakwater on Plan 3.2 was placed with the long axis of the stones parallel to the longitudinal axis of the crest. The average elevation of the tops of these stones was +38.99 ft mllw, indicating a layer of W18 stones placed in this manner had a height of approximately 4.35 ft.

PART IV: DISCUSSION

Stability

60. Plans 1, 2, and 3 are shown together for comparison in Figure 10. All plans tested demonstrated excellent stability on the structure slope and crown, with no stones displaced from either of these areas in any of the tests and only minimal rocking observed. This stability was achieved by the use of special placement techniques, and the assumption that one-half of the stones in the primary armor layer would be parallelepiped-shaped, with the rest of the stones being rough angular. Results given in this report are therefore valid for the prototype only if the following conditions are met:

- a. The placed-stone construction techniques used in the model are reproduced during prototype construction.
- b. At least 50 percent of the stone used in the primary armor layers are parallelepiped in shape and the remaining stones are rough angular in shape. Also, the two stone shapes should be kept well mixed on the structure.

61. The only portion of the structure that exhibited some measure of armor instability was the W18 layer on the sea-side toe. Toe stones exhibited either rocking and in place reorientation or minor displacement during most of the tests on Plans 2 and 3.

62. Plan 2 showed the greatest total amount of toe stone movement, including rocking, reorientation, and displacement. This was expected, as Plan 2 used stones in the lowest 25-percent weight range of the W18 class stone for the toe, while Plans 1 and 3 used the entire W18 weight range. In general, rocking was observed during the uprush portion of the wave action, while displacement occurred during the downrush. Displaced stones generally came from the exposed outer, upper edge of the seaward toe. There was no tendency for additional unraveling, nor any indication that damage would progress to the point where the slope stability was threatened. Displaced stones were almost exclusively of the rough angular shapes. Though not confirmed by model tests, it is felt that increased stability should be obtained by using parallelepiped-shaped stones on the seaward toe.

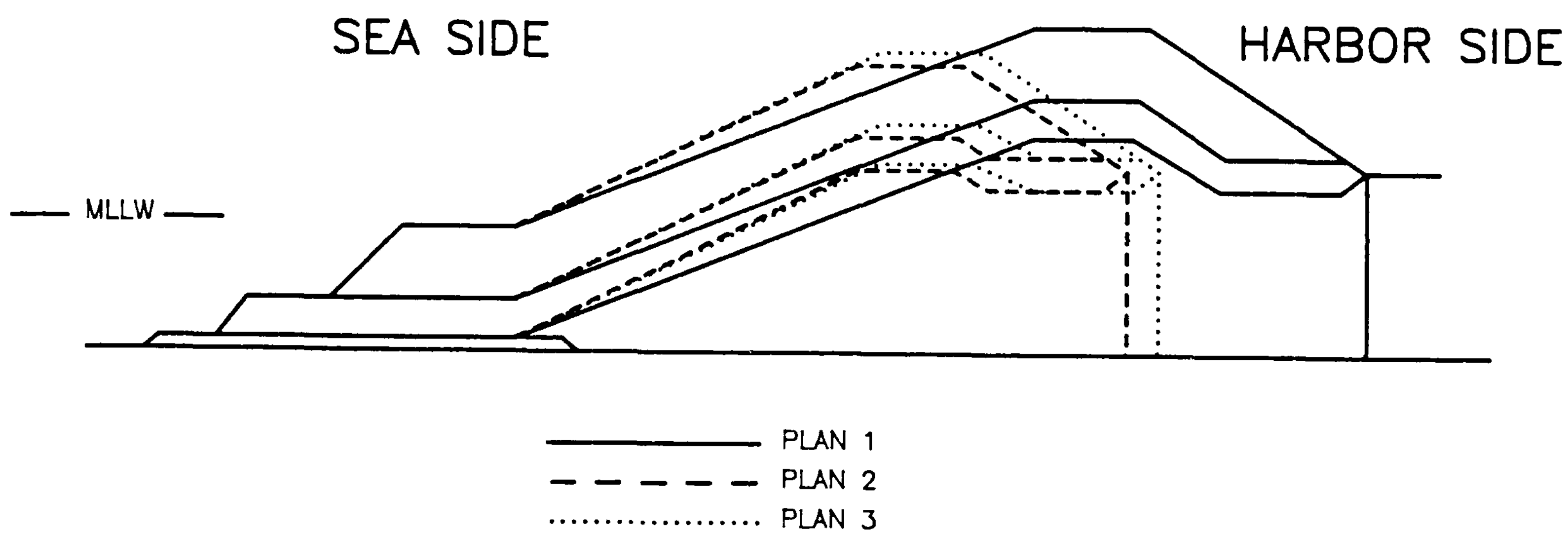

Figure 10. Comparison of Plans 1, 2, and 3, St. Paul Harbor breakwater stability study

Overtopping

63. Although some splashing over the crown contributed to the measured overtopping rates, most of the overtopping was either from flow through the structure (Plans 1, 2, and 3) or solid water overtopping the structure (16-sec waves with the swl at +5.0 ft mllw on Plans 2, 3, 3.1, and 3.2). With the exception of tests conducted with 16-sec waves at a swl of +5.0 ft mllw, overtopping rates did not exceed 0.055 cfs/ft in any of the test plans for any wave period/wave height/water depth combination tested.

64. Long period waves at the high-water level caused solid water to overtop the structures on Plans 2 and 3 to such an extent that working on the dock on the harbor side of the breakwater is not expected to be feasible under this storm condition. Modifications to Plan 3 were directed at reducing the overtopping. A significant reduction in the overtopping rate was achieved by allowing the flow to pass through the structure by making the harbor side porous (Plan 3.1). In the prototype, this would be achieved by replacing the caisson-style dock structure with a pile structure or some other type that would not restrict flow through the breakwater. Water flowing over the top of the breakwater was reduced by placing a layer of W18 stones, one layer high and two layers wide, along the crest of the breakwater (Plan 3.2).

Height of Breakwater

65. The W2 underlayer of Plan 3 was constructed to an elevation of +18.0 ft mllw prior to testing. The survey conducted after testing showed an average elevation of the underlayer of +17.93 ft mllw, indicating that very little settlement occurred during the testing. Some of the observed settlement probably occurred during the modification to make the harbor side of the structure porous.

66. Surveys indicated the thickness of the primary armor layers was 16.71 ft, rather than 14 ft as indicated in the plans. Assuming a similar construction method in the prototype, that is, construction to grade with the W2 layer, then special placement of two layers of W18 stones, the crest of the finished breakwater should be 2 to 3 ft higher than originally predicted. However, if the crest is constructed to the heights specified in the plans, it will be 2 to 3 ft lower than the sections used in the tests, which could

significantly increase overtopping values over those given in this report. All crest elevations in this report, with the exception of the survey results (Table 7 and paragraph 59), are the crest elevations specified in the plans and not crest elevations based on the thickness of the armor layer determined by the survey.

Application of Model Results to Prototype

67. Conservatism was built into the tests in two ways. First, the typical bottom slope in front of the prototype was determined to be 1:200 based on navigation charts of the area, but was modeled at 1:100. The steeper slope used in the model would be expected to produce a slightly higher depth-limited breaking wave, and the action of a breaking wave tends to be more severe with a steeper slope. Therefore, waves of a given period imparted more energy to the structure than if the model had duplicated the 1:200 bottom slope.

68. Second, design wave conditions were run continuously in the tests, while the prototype will likely be subjected to design waves intermittently for the duration of a storm event. Therefore, the number of waves of design magnitude reaching the structure during a test is likely to be considerably higher than the number reaching the prototype during a storm.

PART V: CONCLUSIONS AND RECOMMENDATIONS

69. For the test conditions and test results reported herein, it is concluded that:

a. Each of the breakwater plans tested demonstrated acceptable stability on the crown and sea-side slope. The sea-side toe should be constructed with the full weight range of W18 stones, and care should be taken to ensure good construction of the randomly placed toe stones.

b. Of the plans tested, Plan 3.2 showed the most desirable combination of armor stability and wave overtopping rates.

70. In addition, the following recommendations are based on observations of the test series.

a. The seaward toe should be constructed with parallelepiped-shaped stone. The increased stability of the parallelepiped-shaped stones over the rough angular stones makes their use particularly important due to the random placement of the toe stones.

b. The two-stone-wide row of W18 stones placed along the top of the breakwater on Plan 3.2 may be placed with the long axis of the stones parallel to the longitudinal axis of the breakwater. Observed wave forces at this elevation were not great enough to require the increased stability that could be obtained by placing the long axis of the stones perpendicular to the longitudinal axis of the breakwater.

c. A pile supported dock facility should be used rather than the proposed caisson-style dock facility to increase the porosity of the breakwater/dock structure and reduce the overtopping rates.

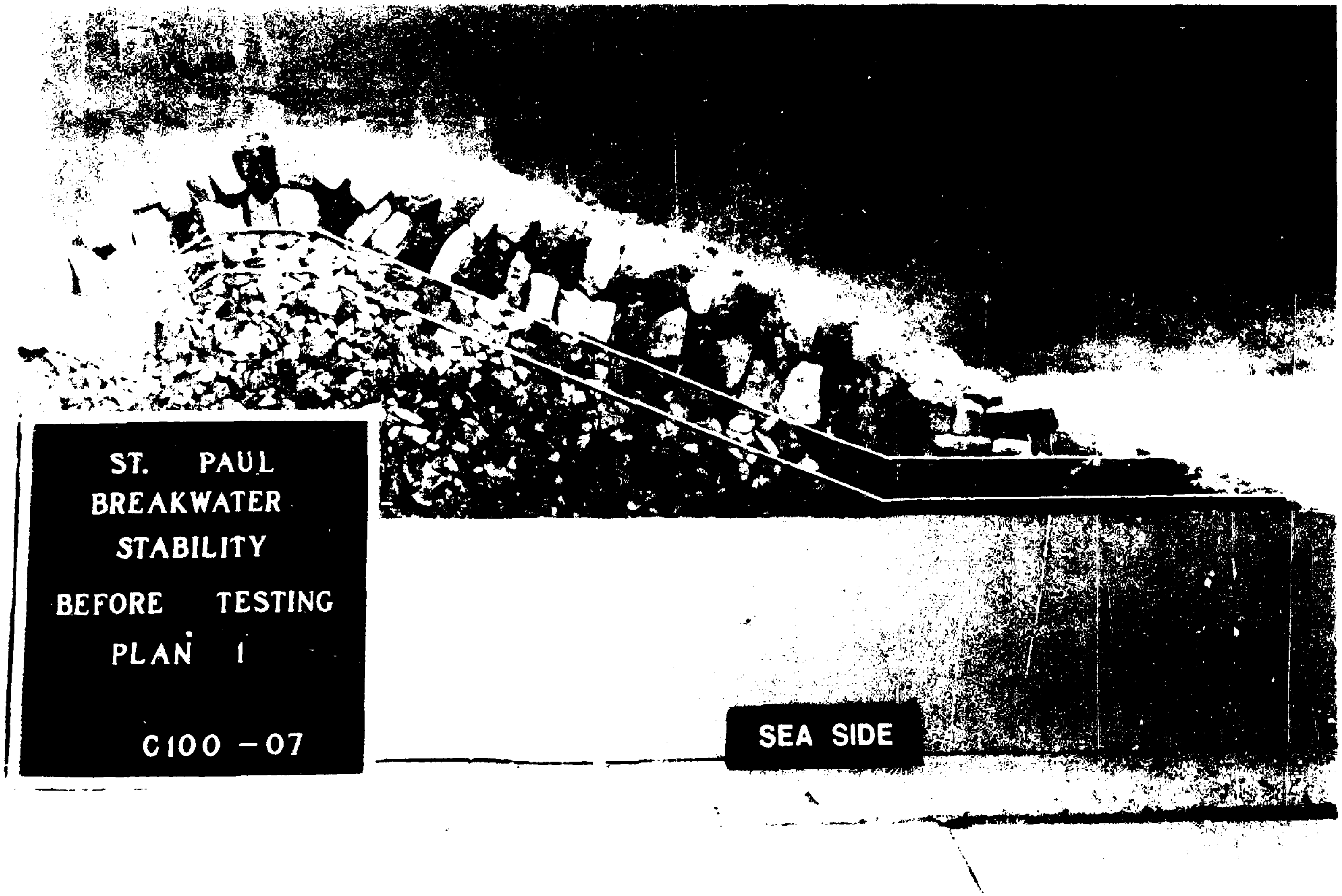

Photo 1. Plan 1, before Test 3, side view; St. Paul Harbor breakwater stability study

Photo 2. Plan 1, before Test 3, sea-side view; St. Paul Harbor breakwater stability study

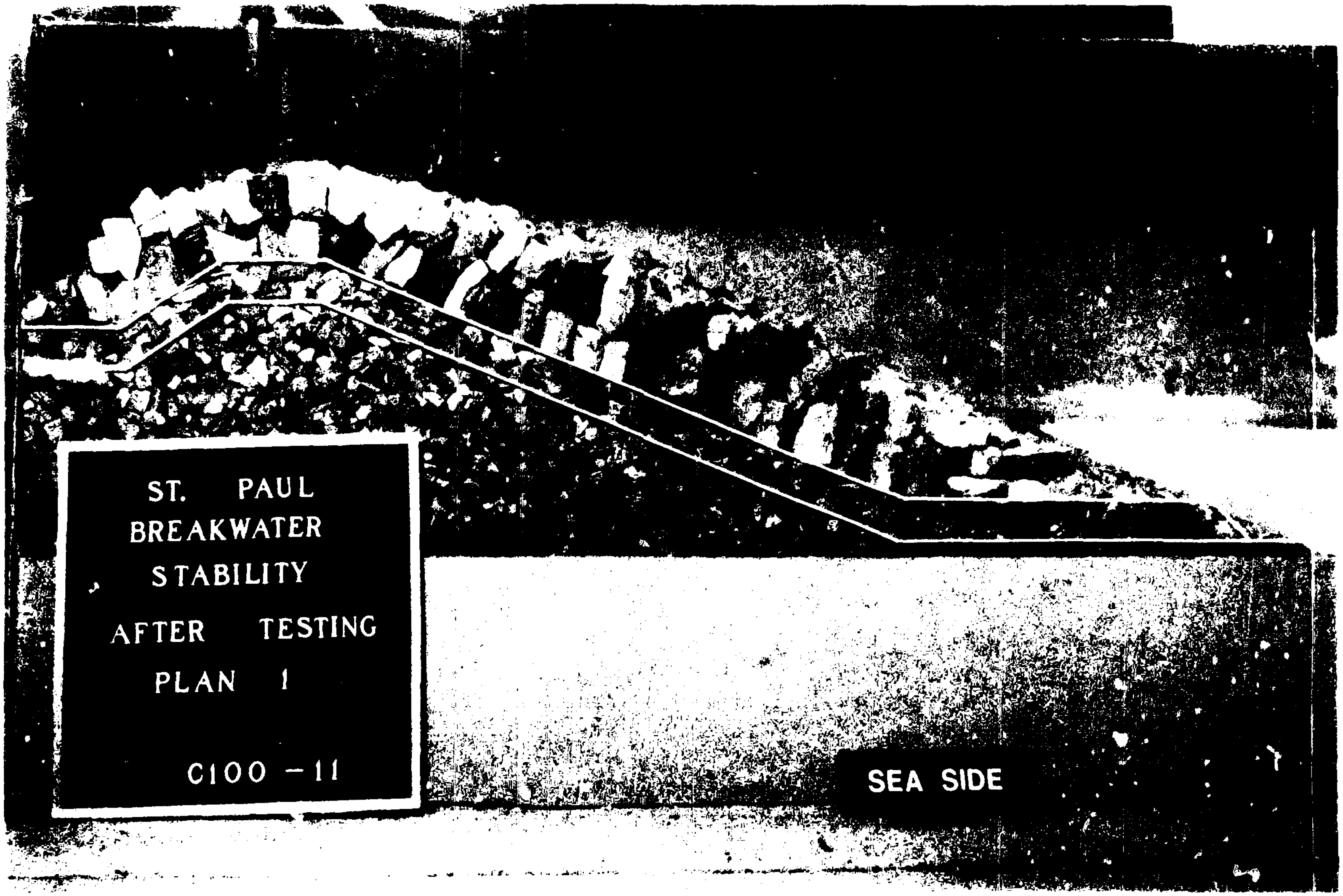

Photo 3. Plan 1, after Test 4, side view; St. Paul Harbor breakwater stability study

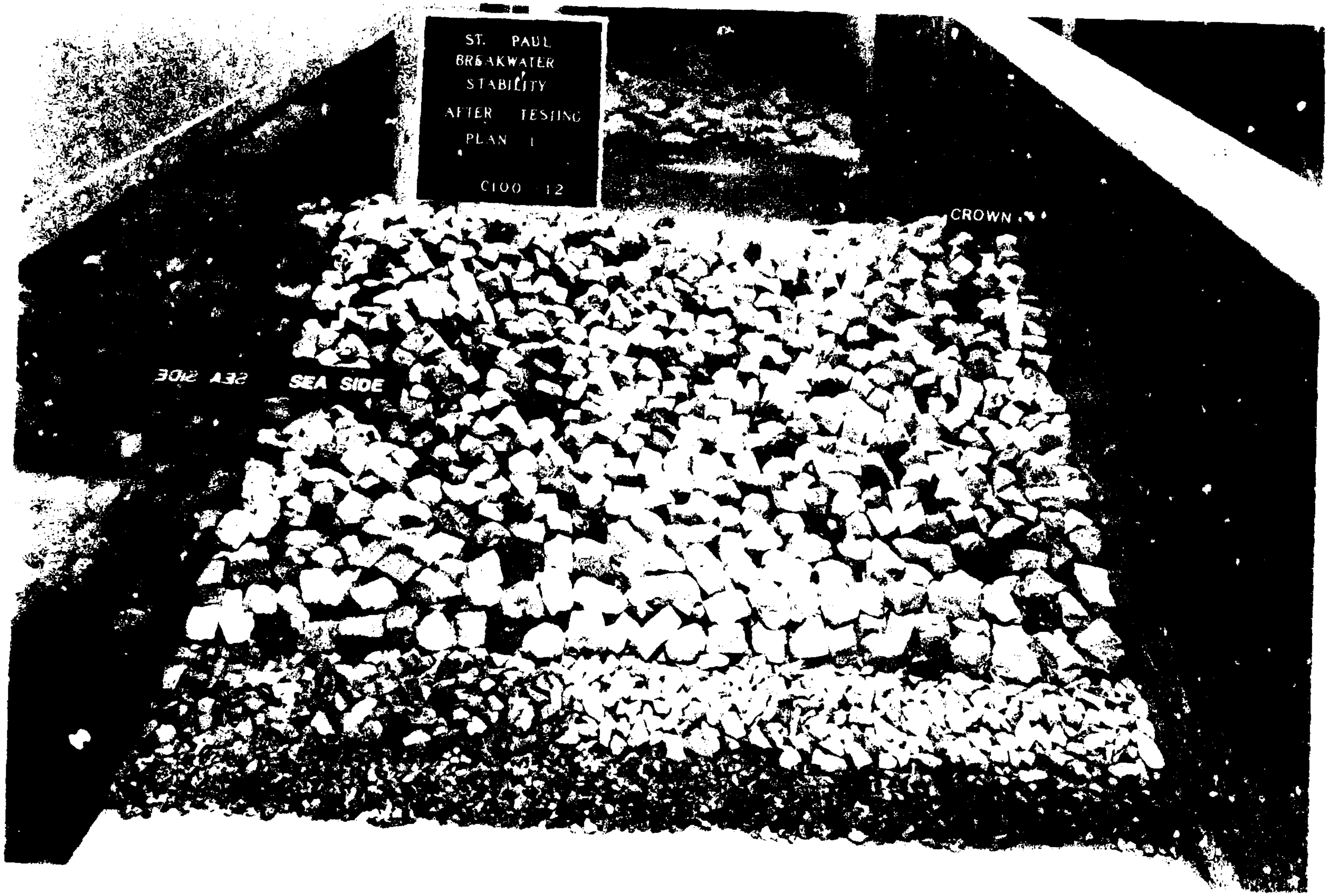

Photo 7. Plan 1, after Test 4, sea-side view; St. Paul Harbor breakwater stability study

Photo 5. Plan 1, after Test 5, side view; St. Paul Harbor breakwater stability study

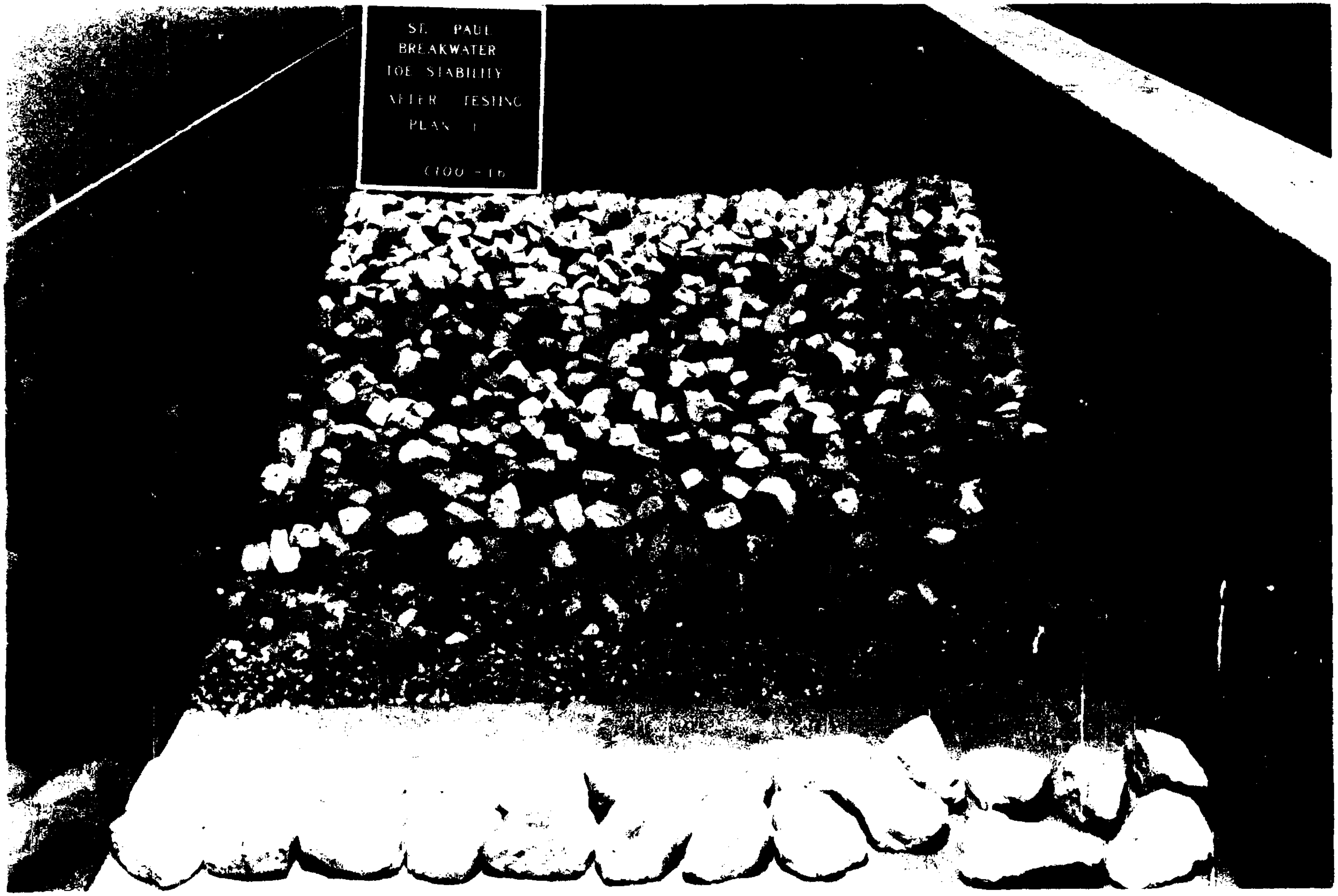

Photo 6. Plan 1, after Test 5, sea-side view; St. Paul Harbor breakwater stability study

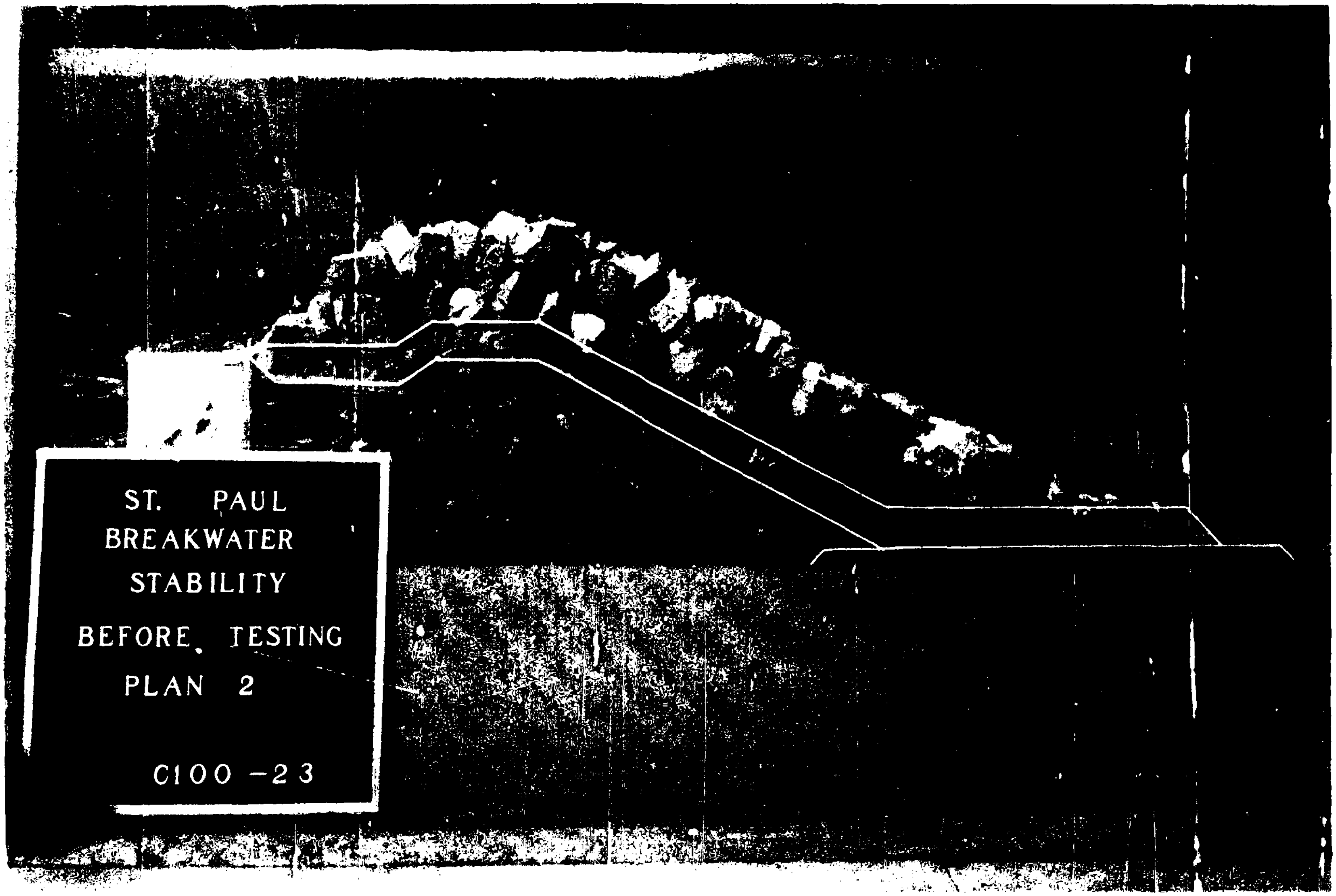

Photo 7. Plan 2, before Test 8, side view; St. Paul Harbor breakwater stability study

Photo 8. Plan 2, before Test 8, sea-side view; St. Paul Harbor breakwater stability study

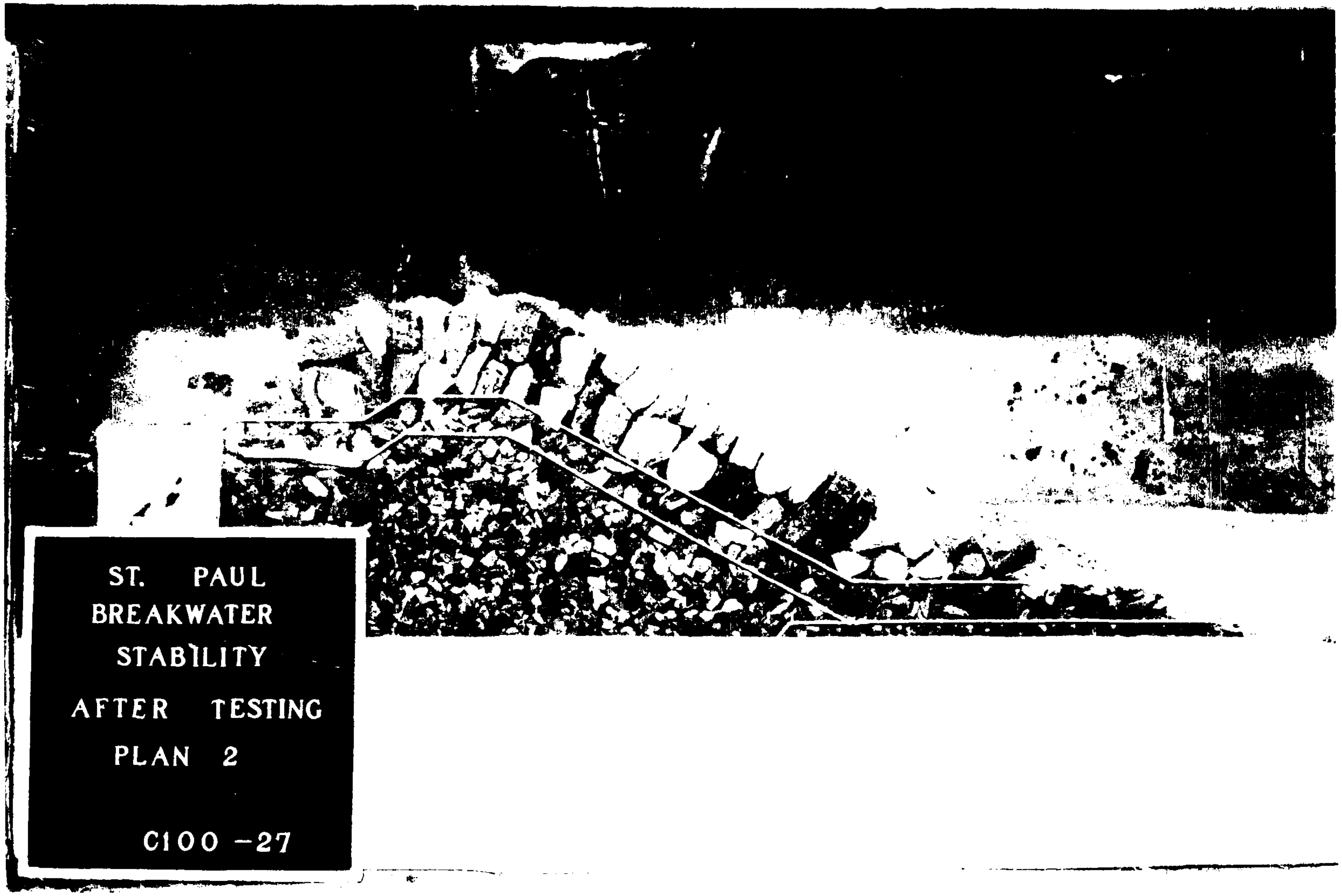

Photo 9. Plan 2, after Test 9, side view; St. Paul Harbor breakwater stability study

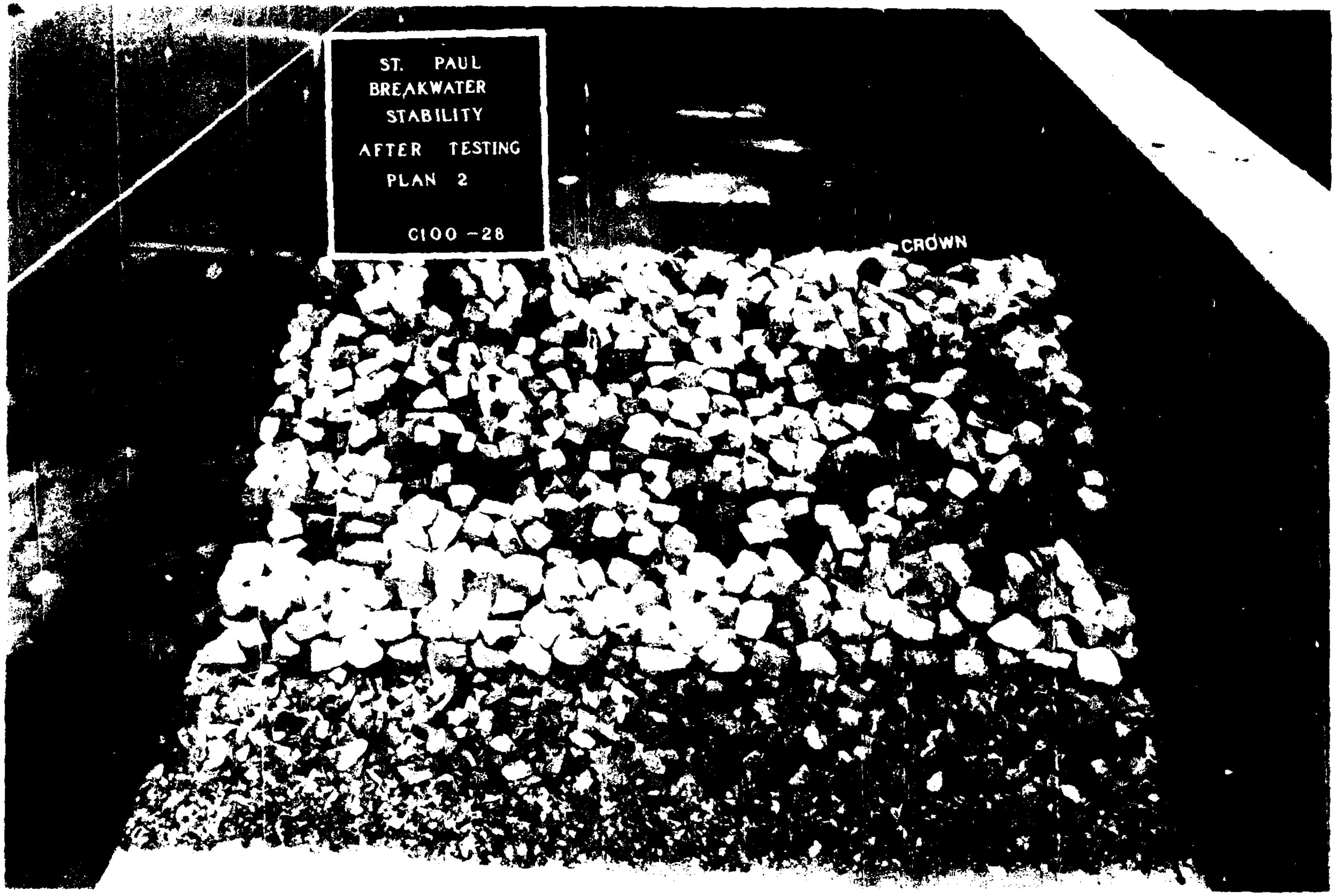

Photo 10. Plan 2, after Test 9, sea-side view; St. Paul Harbor breakwater stability study

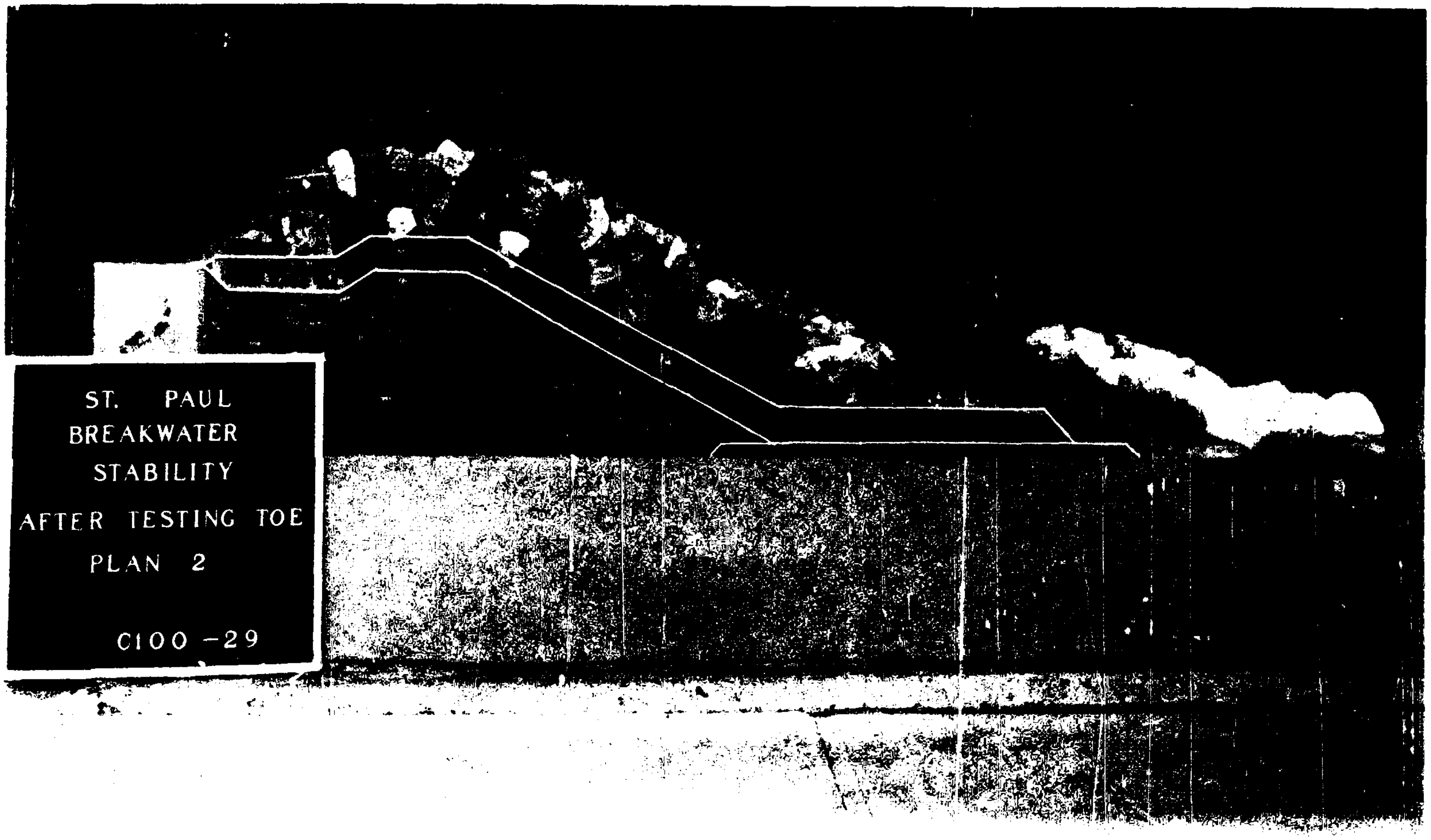

Photo 11. Plan 2, after Test 11, side view; St. Paul Harbor breakwater stability study

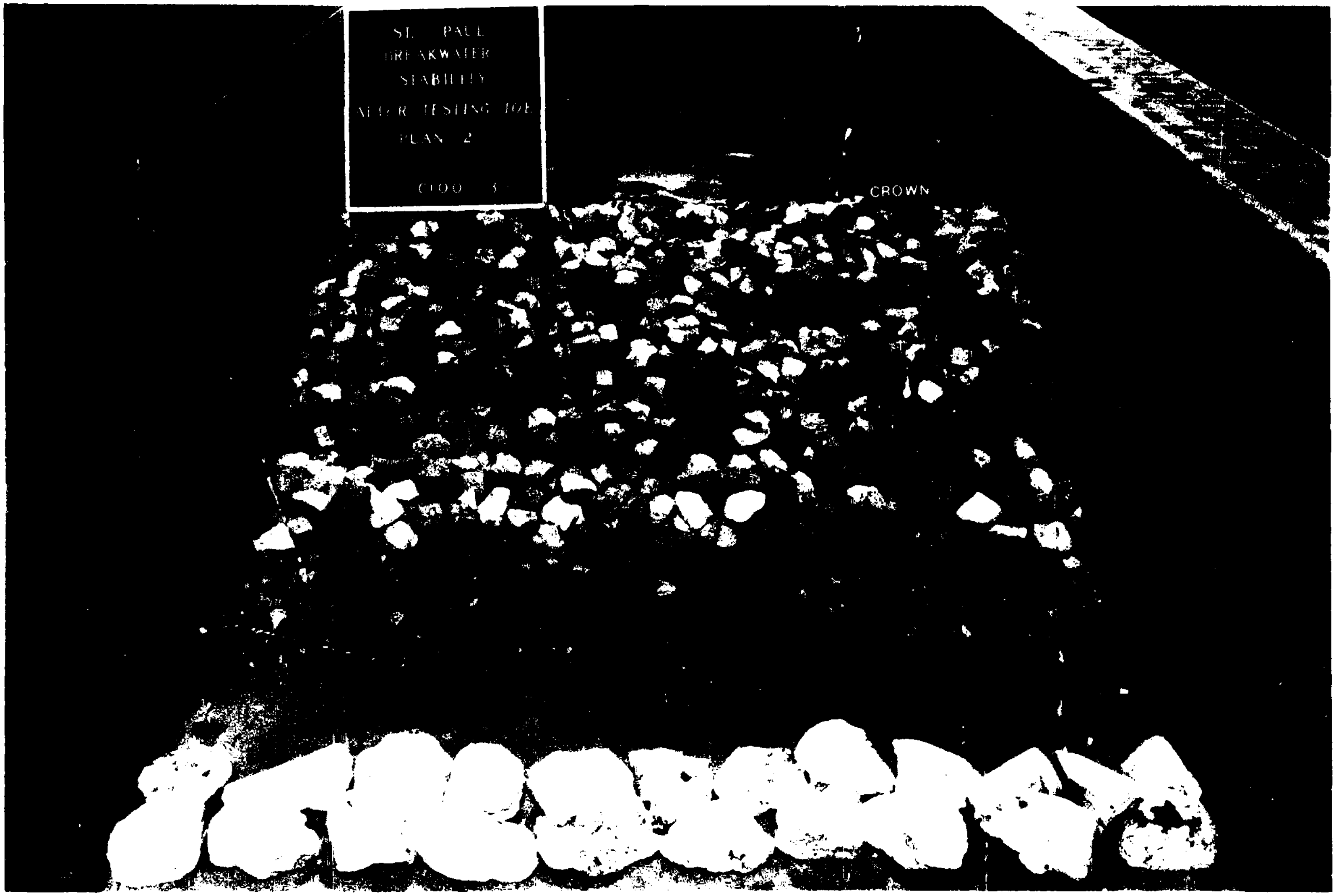

Photo 12. Plan 2, after Test 11, sea-side view; St. Paul Harbor breakwater stability study

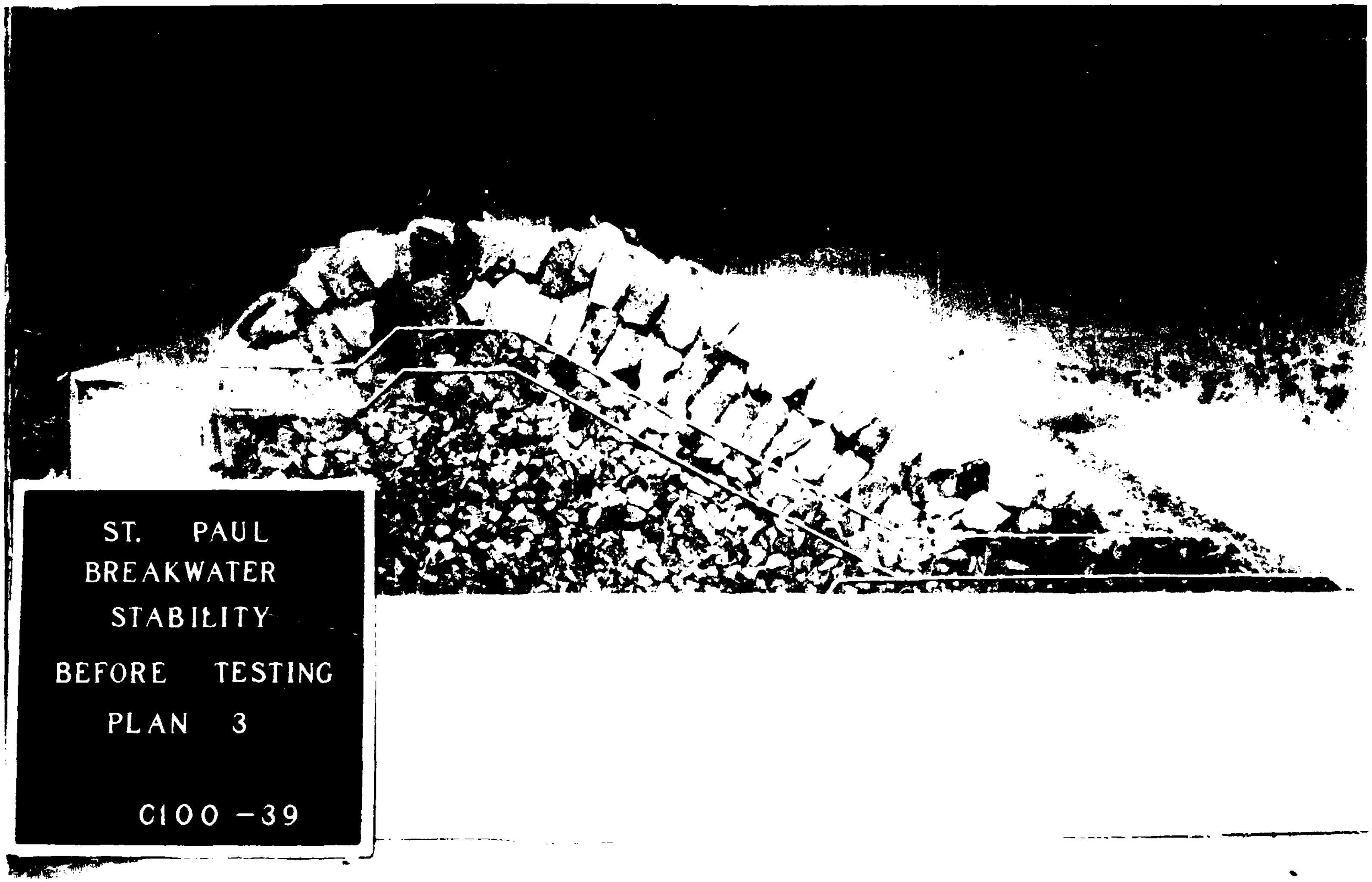

Photo 13. Plan 3, before Test 15, side view; St. Paul Harbor breakwater stability study

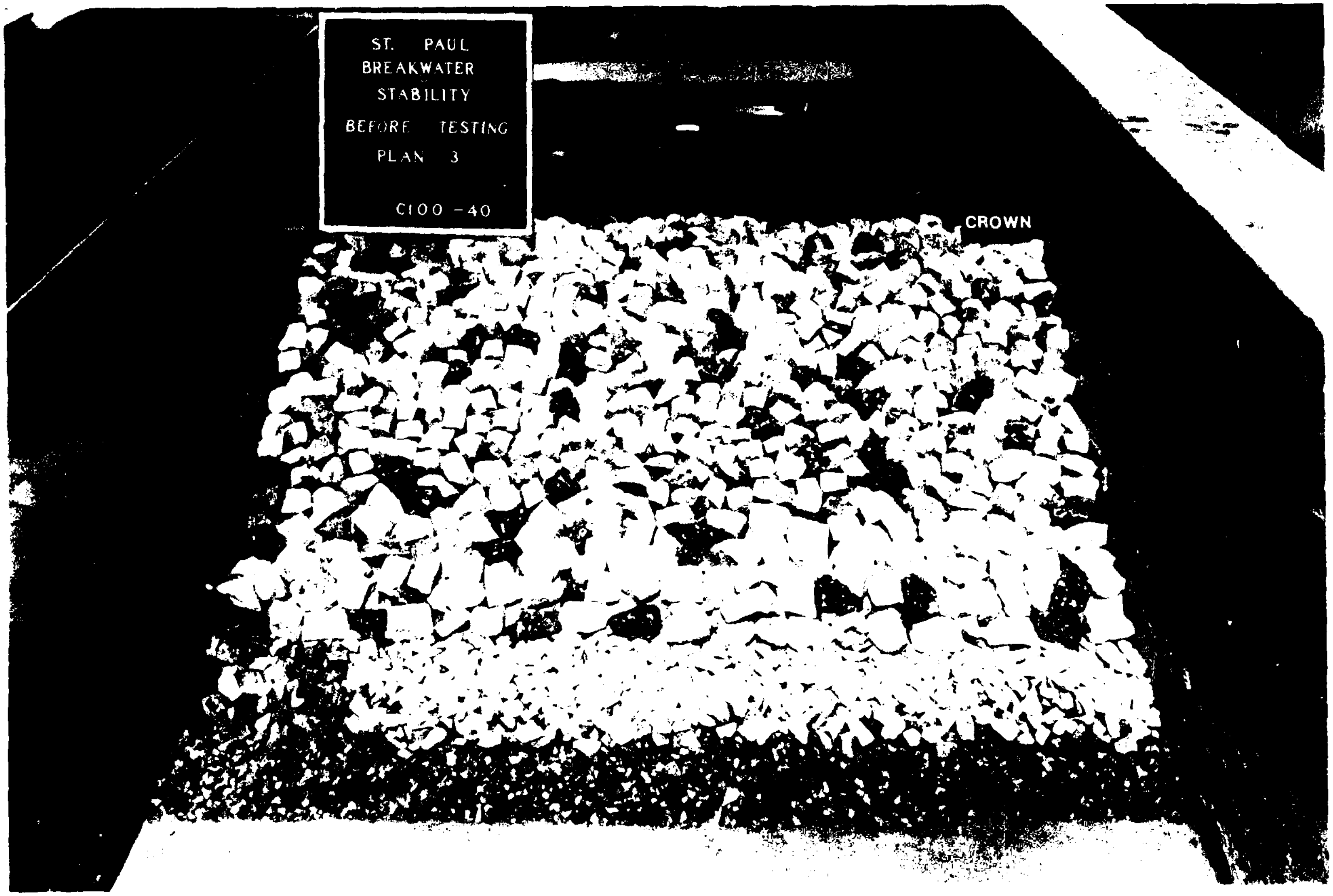

Photo 14. Plan 3, before Test 15, sea-side view; St. Paul Harbor breakwater stability study

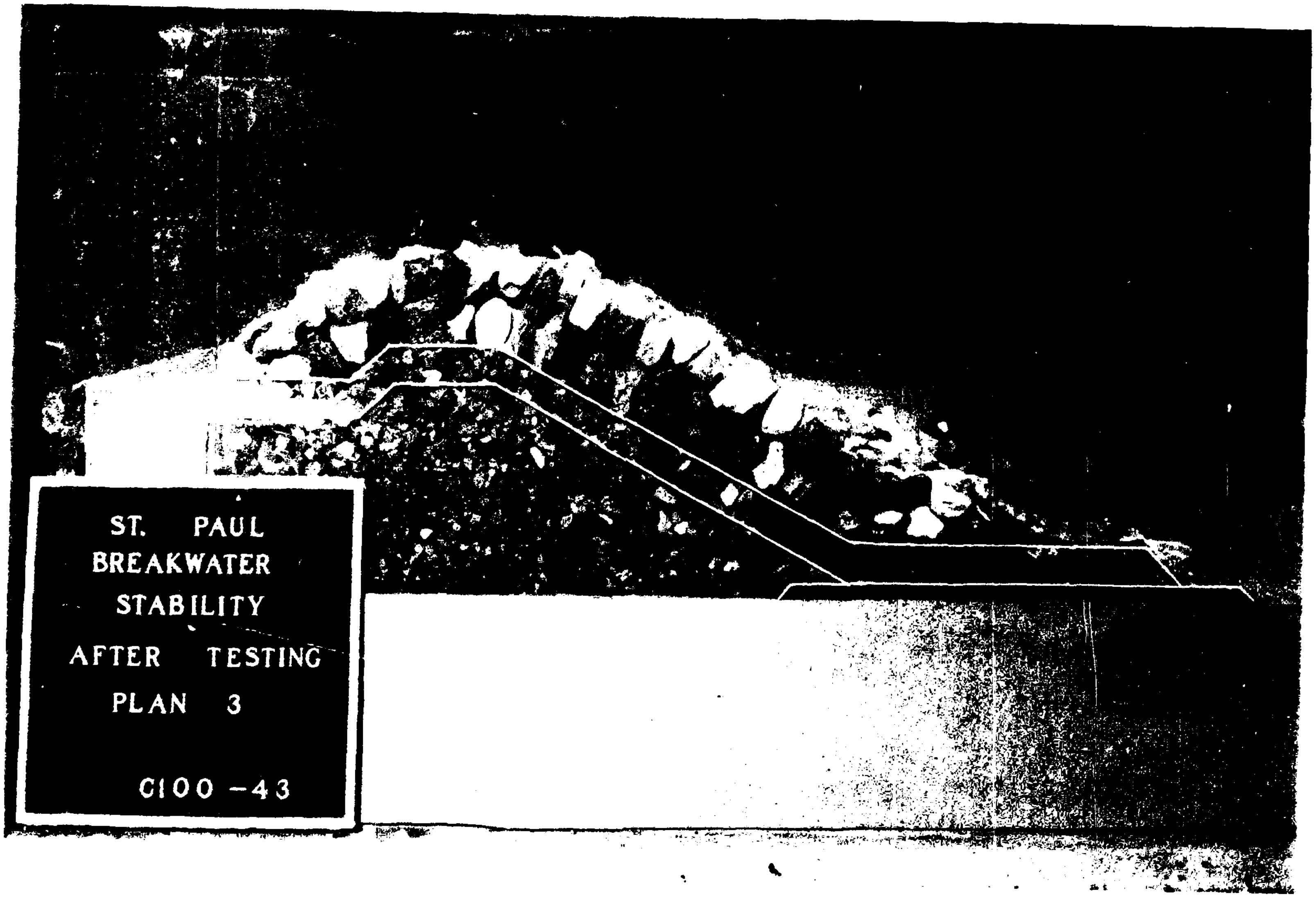

Photo 15. Plan 3, after Test 16, side view; St. Paul Harbor breakwater stability study

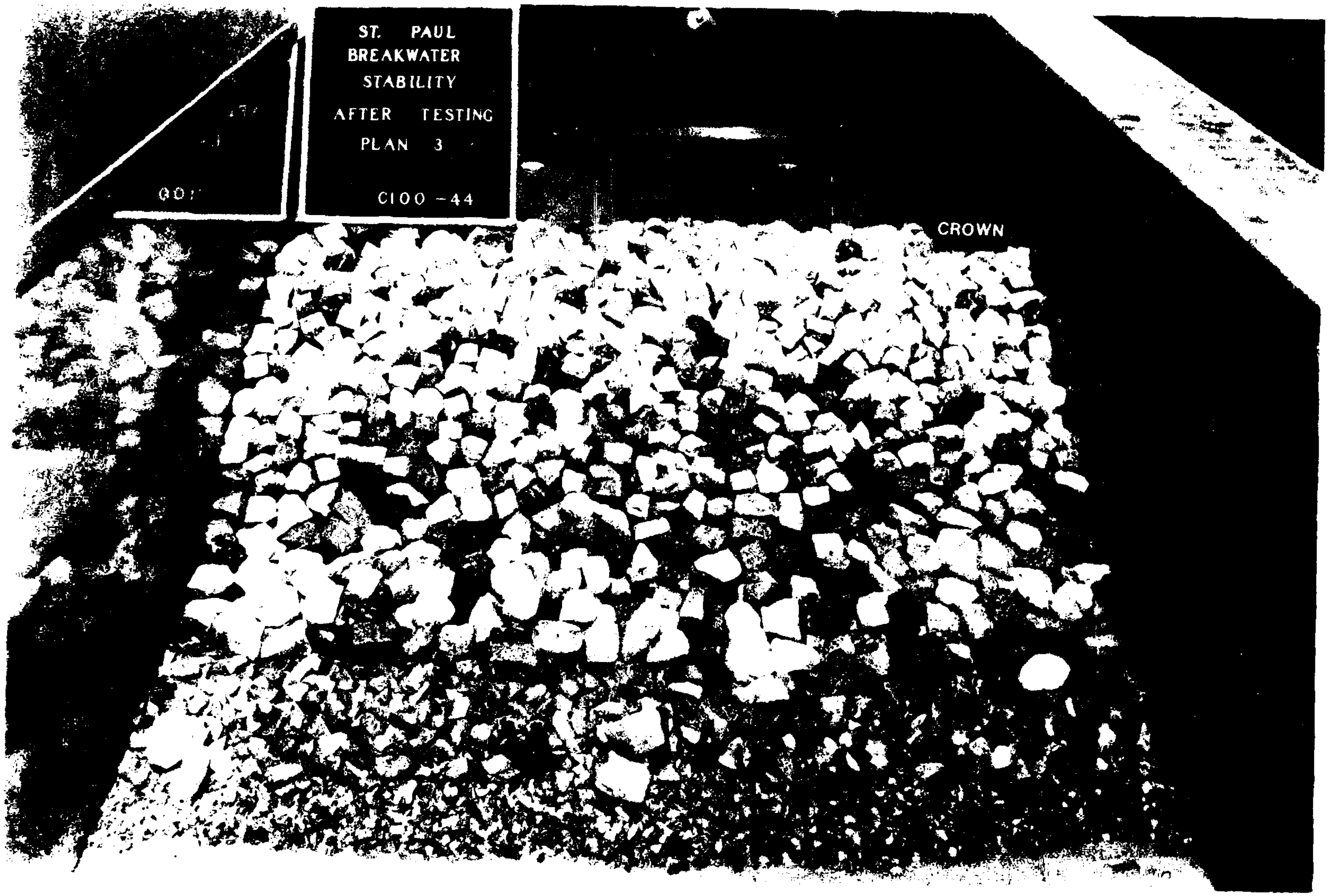

Photo 16. Plan 3, after Test 16, sea-side view; St. Paul Harbor breakwater stability study

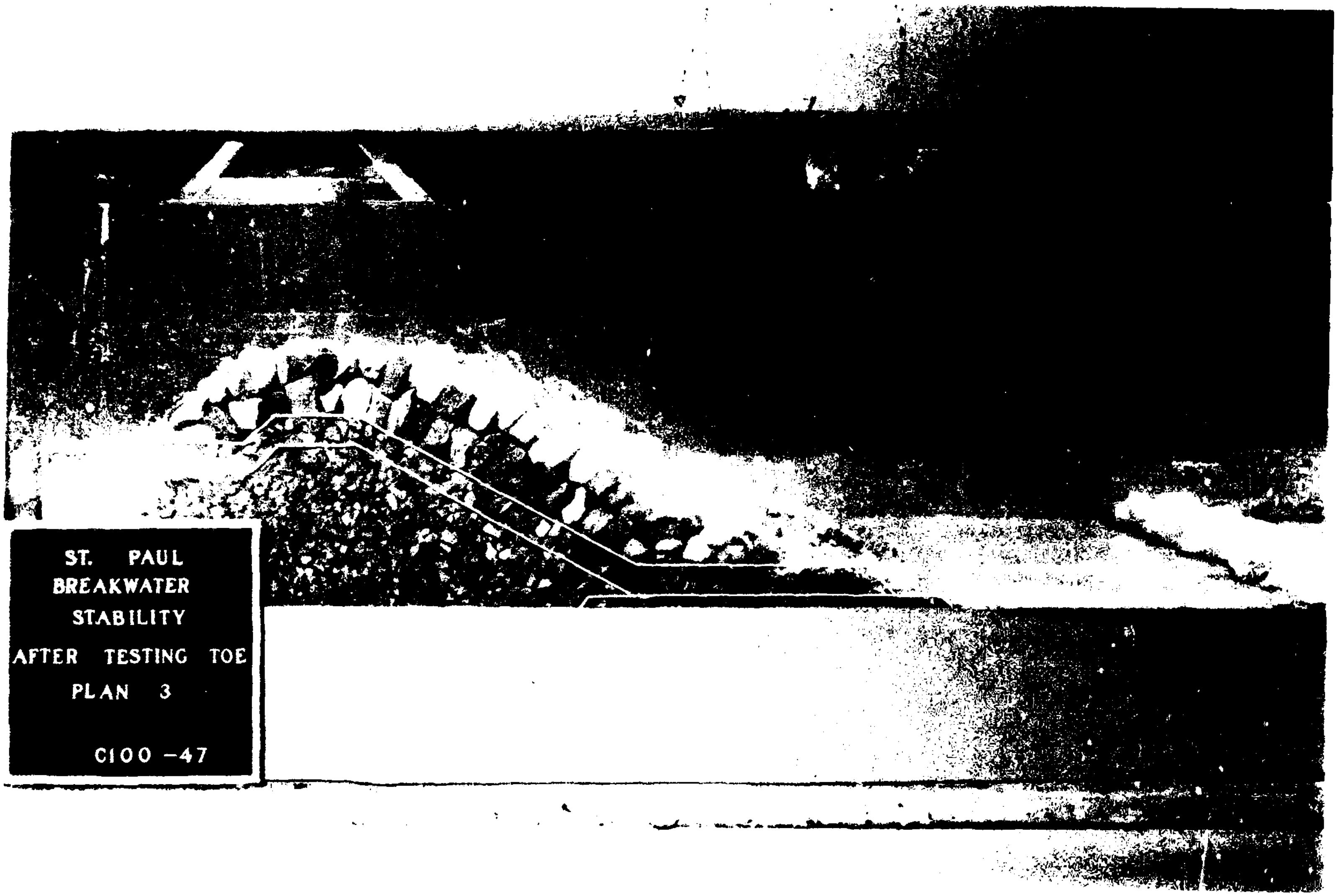

Photo 17. Plan 3, after Test 18, side view; St. Paul Harbor breakwater stability study

Photo 18. Plan 3, after Test 18, sea-side view; St. Paul Harbor breakwater stability study

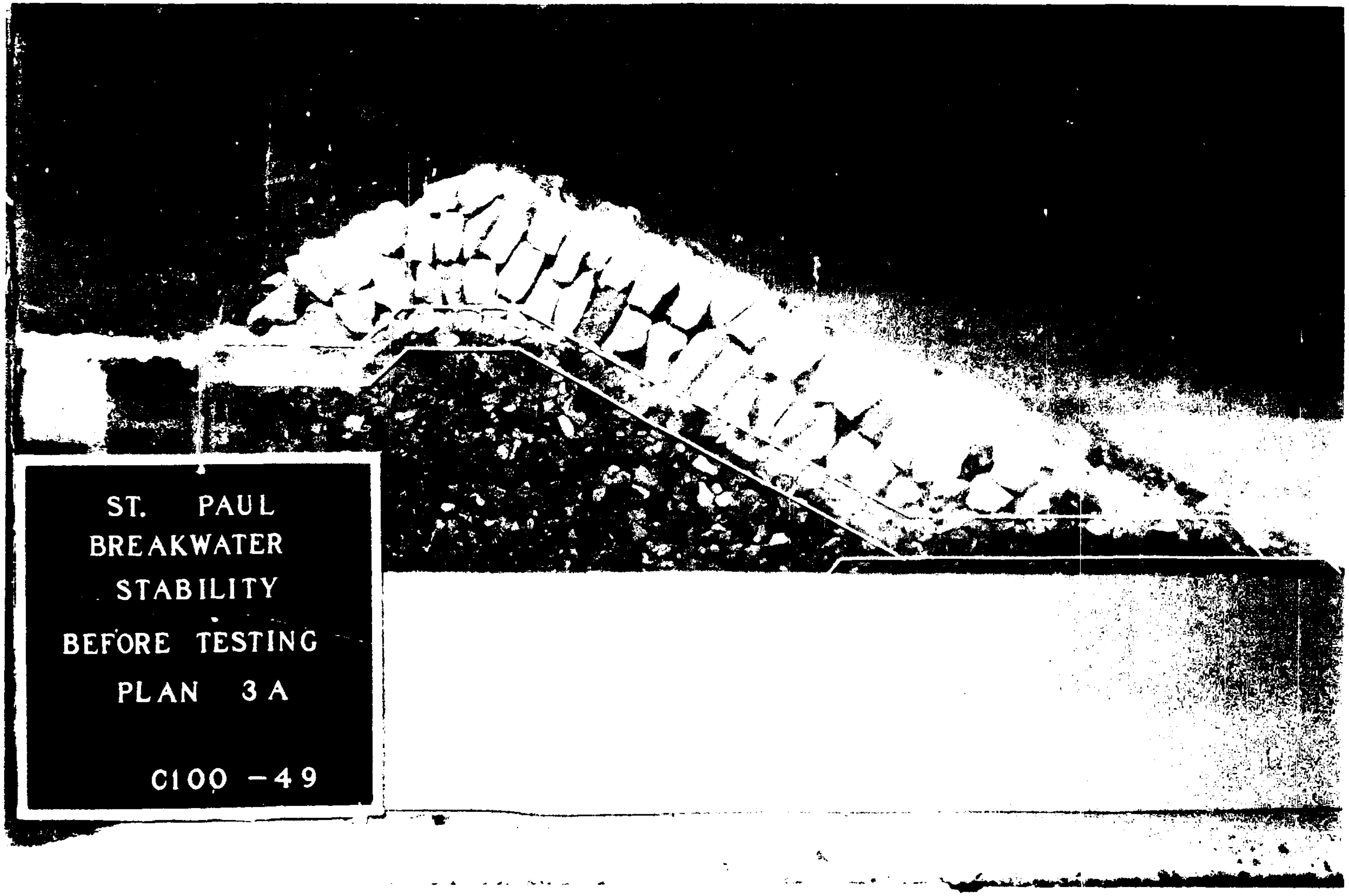

Photo 19. Plan 3.2, before Test 20, side view; St. Paul Harbor breakwater stability study

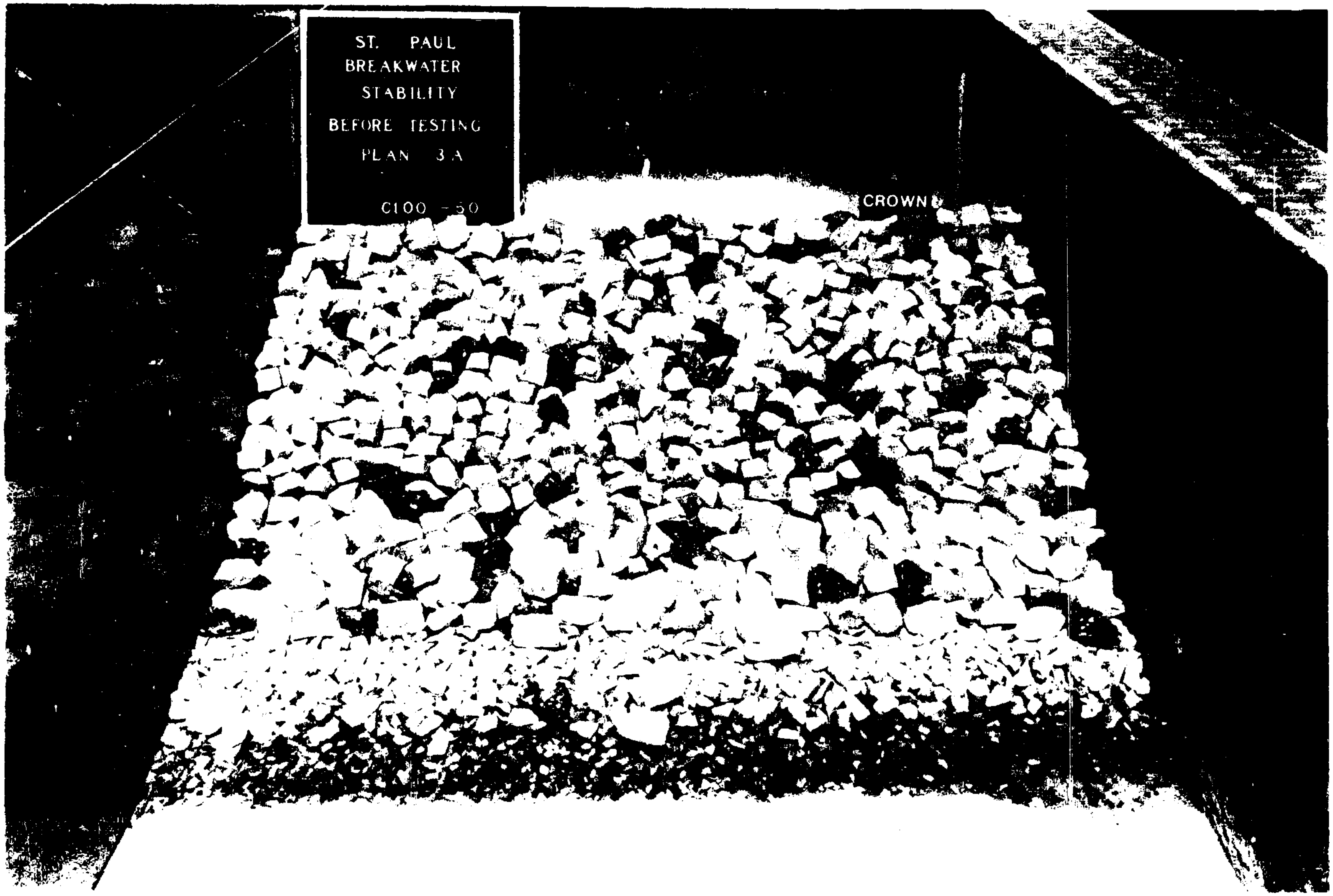

Photo 20. Plan 3.2, before Test 20, side view; St. Paul Harbor breakwater stability study