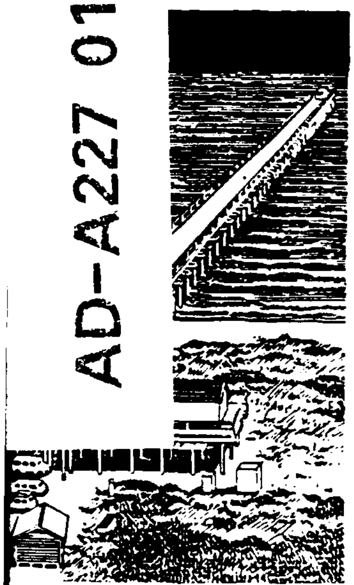
US Army Corps of Engineers



REMR RESEARCH PROGRAM

REPAIR, EVALUATION, MAINTENANCE, AND REHABILITATION RESEARCH PROGRAM

TECHNICAL REPORT REMR-CO-14

REPAIR OF LOCALIZED ARMOR STONE DAMAGE ON RUBBLE-MOUND STRUCTURES

Coastal Model Investigation

by

Donald L. Ward, Dennis G. Markle

Coastal Engineering Research Center

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

US ARMY ENGINEER WATERWAYS EXPERIMENT STATION

September 1990

Final Report

Approved For Public Release; Distribution Unlimited

Prepared for DEPARTMENT OF THE ARMY
US Army Corps of Engineers
Washington, DC 20314-1000

Under Work Unit 32416

The following two letters used as part of the number designating technical reports of research published under the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program identify the problem area under which the report was prepared:

	Problem Area		Problem Area
CS	Concrete and Steel Structures	EM	Electrical and Mechanical
GT	Geotechnical	EI	Environmental Impacts
HY	Hydraulics	OM	Operations Management
CO	Coastal		

COVER PHOTOS:

TOP — Field Research Facility, Duck, North Carolina.

BOTTOM — Wave action on structure in 5.00-ft flume during Phase I.

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE

REPORT DOCUMENTATION PAGE

Form Approved OMB No. 0704-0188

1a. REPORT SECURITY CLASSIFICATION Unclassified	1b. RESTRICTIVE MARKINGS
2a. SECURITY CLASSIFICATION AUTHORITY	3. DISTRIBUTION/AVAILABILITY OF REPORT Approved for public release; distribution unlimited
2b. DECLASSIFICATION/DOWNGRADING SCHEDULE	
4. PERFORMING ORGANIZATION REPORT NUMBER(S) Technical Report REMR-CO-14	5. MONITORING ORGANIZATION REPORT NUMBER(S)
6a. NAME OF PERFORMING ORGANIZATION USAEWES, Coastal Engineering Research Center 6b. OFFICE SYMBOL (If applicable)	7a. NAME OF MONITORING ORGANIZATION
6c. ADDRESS (City, State, and ZIP Code) 3909 Halls Ferry Road Vicksburg, MS 39180-6199	7b. ADDRESS (City, State, and ZIP Code)
8a. NAME OF FUNDING/SPONSORING ORGANIZATION US Army Corps of Engineers 8b. OFFICE SYMBOL (If applicable)	9. PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER
8c. ADDRESS (City, State, and ZIP Code) Washington, DC 20314-1000	10. SOURCE OF FUNDING NUMBERS PROGRAM ELEMENT NO. / PROJECT NO. / TASK NO. / WORK UNIT ACCESSION NO. 32416

11. TITLE (Include Security Classification)
Repair of Localized Armor Stone Damage on Rubble-Mound Structures; Coastal Model Investigation

12. PERSONAL AUTHOR(S)
Ward, Donald L.; Markle, Dennis G.

13a. TYPE OF REPORT	13b. TIME COVERED	14. DATE OF REPORT (Year, Month, Day)	15. PAGE COUNT
Final report	FROM ____ TO ____	September 1990	34

16. SUPPLEMENTARY NOTATION A report of the Coastal Problem Area of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program. Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

17. COSATI CODES		
FIELD	GROUP	SUB-GROUP

18. SUBJECT TERMS (Continue on reverse if necessary and identify by block number)
Breakwaters
Jetties
Localized damage
Repair
Rubble mound

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

A series of physical model tests were conducted on small-scale rubble mounds to determine the effectiveness of different repair methods. The structures were subjected to wave conditions that caused localized damage to the armor stone. After repairs were made by one of six repair options tested, the structure was again subjected to the wave conditions. Parameters that were varied during the tests included sea-side slope, water depth, and wave period, with wave height chosen to cause moderate damage for the current set of parameters. Scatter in the results rendered the tests inconclusive regarding the relative effectiveness of the various repair methods, but key factors in the repair of localized armor stone damage were demonstrated.

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

DD Form 1473, JUN 86 — Previous editions are obsolete.

SECURITY CLASSIFICATION OF THIS PAGE
Unclassified

Preface

The work described in this report was authorized by Headquarters, US Army Corps of Engineers (HQUSACE), as part of the Coastal Problem Area of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program. The work was performed under Work Unit 32416, "Experimental Testing of Methods and Materials for Repair of Localized Damage to Rubble-Mound Structures," for which Mr. Donald L. Ward was Principal Investigator. Mr. John H. Lockhart (CECW-EH) was the REMR Technical Monitor for this work.

Mr. Jesse A. Pfeiffer, Jr. (CERD-C) was the REMR Coordinator at the Directorate of Research and Development, HQUSACE; Mr. James E. Crews (CECW-OM) and Dr. Tony C. Liu (CECW-ED) served as the REMR Overview Committee; Mr. William F. McCleese (CEWES-SC-A), US Army Engineer Waterways Experiment Station (WES), was the REMR Program Manager. Mr. D. D. Davidson (CEWES-CW-R) was the Problem Area Leader.

The work was performed at WES, and this report was prepared by Messrs. Donald L. Ward and Dennis G. Markle of the Coastal Engineering Research Center (CERC), WES, under the general supervision of Dr. James R. Houston, Chief, CERC, and Mr. Charles C. Calhoun, Jr., Assistant Chief, CERC, and under the direct supervision of Mr. C. Eugene Chatham, Chief, Wave Dynamics Division (WDD), and Mr. Davidson, Chief, Wave Research Branch (WRB), WDD. The models were operated by Messrs. Willie G. Dubose; Cornelius Lewis, Sr.; and Marshall P. Thomas, Laboratory Technicians, WRB.

Commander and Director of WES during publication of this report was COL Larry B. Fulton, EN. Technical Director was Dr. Robert W. Whalin.

<u>Contents</u>

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
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre

REPAIR OF LOCALIZED ARMOR STONE DAMAGE ON RUBBLE-MOUND STRUCTURES

Coastal Model Investigation

Introduction

Background

1. A survey of breakwaters and jetties maintained by the US Army Corps of Engineers revealed that localized damage to the trunks and heads of rubble-mound structures was a major problem. Failure to repair the damage could lead to unraveling, sloughing, and loss of core material, requiring major repairs or rehabilitation of the structure. However, no standardized design guidance on methods of optimizing repairs of rubble-mound structures is available. Repair options range from randomly placing stone in the area of damage to rehabilitation of that portion of the structure, but the relative merits of the different methods have not been determined. Design guidance is necessary to ensure that repairs are made in the most cost-efficient manner that provides the required stability and protection.

Purpose

2. The purpose of this study was to use small-scale physical model tests for determining and comparing the stability of various methods for repair of localized damage to randomly placed armor stone on rubble-mound structures.

The Model

Test facilities and equipment

3. Model testing was conducted in three phases. Phase I tests were conducted in a flume divided into two compartments, 5.00 and 6.75 ft* wide by 4.00 ft deep and 119 ft long, sharing a plunger-type electro-mechanical wave generator (Figure 1). The wave generator is capable of producing monochromatic waves of various periods and amplitudes by varying the frequency

---

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

and stroke of the wave plunger.

4. Test sections in both compartments were located 85.5 ft from the wave plunger and were preceded by a 1:10 (V:H) slope providing a vertical rise of 1 ft above the flat bottom of the flume. The purpose of the 1:10 slope was to cause shoaling of the incident wave train and to provide the most critical depth-limited breaking wave design conditions as determined by previous laboratory studies and field surveys. Repair methods determined to be stable under these conditions should also be stable under less severe conditions; however, additional testing will be required to determine the relative economic merit of the repair options under conditions other than those tested.

5. Phase II tests were conducted in the same facility with the same test section design, but utilized only the 6.75-ft-wide flume.

6. Phase III tests were conducted in a 3.0-ft-deep T-shaped flume with a 133.0-ft-long by 15.0-ft-wide trunk area, and a 30.7-ft-long by 43.0-ft-wide head area (Figure 2). The head of the flume contained a flat test area, 21.3 ft long by 25.0 ft wide, at a height of 1.5 ft above the flat bottom of the flume, surrounded by a slope of 1:6.3 covered by wave absorber. A 1:10 slope in the trunk of the flume connected the test area in the head to the flume floor.

Test flume calibration

7. Prior to construction of the test sections, the test flumes were calibrated for selected water levels and wave conditions. Changes in water-surface elevation (wave heights) as a function of time at the top of the 1:10 slopes were measured by electrical resistance gages and recorded on chart paper by an electrically operated oscillograph. Measurements taken in this way (i.e., without structure in place) avoid waves reflected from the structure and allow accurate reproduction of incident wave conditions.

Test sections

8. Rubble-mound trunk sections with sea-side slopes of 1:1.5 (Figure 3) and 1:2.0 (Figure 4) were constructed in the 5.00- and 6.75-ft-wide flumes, respectively, for Phase I, with the structure in the 6.75-ft flume also being used in Phase II. Both structures spanned the full width of the flumes. The randomly placed armor stone test sections followed the design guidance in the

Shore Protection Manual* (SPM) (1984) for trunk sections exposed to breaking waves with little or no overtopping. The structure included a core overlain by a secondary armor stone underlayer and a primary armor layer. The primary armor and toe berm armor consisted of two layers of randomly placed rough angular armor stone. Primary armor stone and berm armor stone sizing relative to test conditions was carried out using guidance provided by Carver** and Markle,+ respectively.

9. Phase III used a structure with a 6.0-ft-long trunk section with a sea-side slope of 1:1.5 similar to that used in the 5.00-ft flume in Phase I, and a 2.7-ft-long head section constructed in a similar manner following design guidance in the SPM++ (Figure 5). The structure was placed perpendicular to the incident waves.

10. The models were constructed in a manner to simulate as closely as possible prototype construction. The 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 that would result from wave action during construction of the prototype breakwater. During the initial construction of a model, the primary armor layer was placed by hand by randomly selecting a stone 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. Initial armor stone placement thus was conducted in a manner to reproduce the results expected on the prototype from an experienced crane operator using random armor stone placement techniques.

---

* Shore Protection Manual, 1984, 4th Ed., 2 Vols, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

** R. D. Carver, 1983, "Stability of Stone- and Dolos-Armored, Rubble-Mound Breakwater Trunks Subjected to Breaking Waves with No Overtopping," Technical Report CERC-83-5, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

\+ D. G. Markle, 1988, "Stability of Toe Berm Armor Stone and Toe Buttressing Stone on Rubble-Mound Breakwater and Jetty Trunks," Engineering Technical Letter 1110-2-308, Department of the Army, Office of the Chief of Engineers, Washington, DC.

++ SPM, op. cit.

Tests and Test Results

Descriptions of repair options tested

11. Six repair options were tested during Phase I, and four options during Phases II and III (Figures 6 and 7). All repair methods tested used the same size and shape of armor stone as were used during original construction of the test section. The repair methods are described as follows:

a. Spot repair above the still water level (swl). All voids at or above the swl were filled with new armor stone if the void was large enough to accept a stone of the proper size. No stones around the void area were moved in order to improve the seating of the new stones. The voids being filled were created by either direct displacement of an armor stone or the downslope slippage of several stones during wave attack. This repair option is abbreviated in Table 1 and Figures 8-15 as FILL, SWL.

b. Spot repair with reorientation above the swl. Stones were placed in voids as in option a, above, but stones surrounding the voids were reoriented to improve the seating of the new stones. Stones within one or two armor stone diameters of a void were shifted to ensure that the voids were completely filled and that all armor stones were keyed together. This repair option is abbreviated in Table 1 and Figures 8-15 as SEAT, SWL.

c. Localized rehabilitation above the swl. This repair consisted of adding new stone and reseating existing stone as necessary to produce a structure that looked like new construction from the swl to the crown in and above the area of localized damage. Stones located directly above the void were removed and repositioned in the void; thus the location of the void was gradually moved toward the crown while the area below the void had been rebuilt. Additional stones then were added to the structure at the crown. This method eliminated the need for multiple handling and stockpiling of existing stone. This repair option is abbreviated in Table 1 and Figures 8-15 as REHAB, SWL.

d. Spot repair on entire sea-side slope. This repair was the same as in option a, above, except that repair was carried out from sea-side primary armor toe to structure crown. This repair option is abbreviated in Table 1 and Figures 8-15 as FILL, TOE.

e. Spot repair with reorientation on entire sea-side slope. This repair was the same as in option b, above, except that repair was carried out from sea-side primary armor toe to structure crown. This repair option is abbreviated in Table 1 and Figures 8-15 as SEAT, TOE.

f. <u>Localized rehabilitation on entire sea-side slope.</u> This repair was the same as in option c, except that repair was carried out from sea-side primary armor toe to structure crown. This repair option is abbreviated in Table 1 and Figures 8-15 as REHAB, TOE.

12. In simulating typical prototype construction, armor stones that had been dislodged and fallen off the structure were left where they lay, and new stone was used for the repairs.

13. The most typical inspection for a rubble-mound structure is a visual reconnaissance of the armor conditions above the swl. In most instances, below-water conditions of the armor are inferred from observed conditions abovewater. Observation and repair below swl are usually difficult or not feasible, but were tested here to determine the significance of repairing the damage below the swl.

14. Due to the significantly higher costs of localized rehabilitation repairs (options c and f), these options are not in common practice by the Corps. These testing options were therefore discontinued after Phase I, and repair options a, b, d, and e were used in Phases II and III.

<u>Test conditions</u>

15. The effects of different repair methods were tested under varying wave periods, water depths, and structure side slopes, as well as comparing the effects of the repair on both a rubble-mound trunk and head. Wave periods tested were 1.24 sec (Phase I) or 1.26 sec (Phases II and III) (both periods hereafter referred to as 1.25 sec), 1.43 sec (Phases II and III), and 1.67 sec (Phases II and III). Water depths at the structure toe included 0.6 ft (Phases I and III) and 0.8 ft (Phase II). Rubble-mound section side slopes tested were 1:1.5 (Phases I and III) and 1:2.0 (Phases I and II). Phase III included tests on both head and trunk.

<u>Test procedures</u>

16. Once flume calibration was completed and test sections were in place, the structures were exposed to wave attack to verify their design adequacy and to see if the wave and water level conditions would produce localized damage on which a repair method could be tested. Detailed observations of the structure's response to incident wave attack were documented during each test. This provided documentation of where damage was initiated and a chronological response of the structure to selected test conditions.

After each test, the flume was drained and the structure's condition was documented with photographs and written summaries. Photo 1 shows a typical wave attack on the structure in the 5.00-ft flume during Phase I tests.

Phase I

17. During the first phase of the test program, each of the six repair options described above was tested using the same water level and wave conditions to compare the adequacy of the various repair methods.

18. The Phase I test procedure was performed as follows. After the structure was completed in the facility, photographs of the structure were taken to record the initial condition of the structure. The flume then was filled to the selected water depth, and a set of waves of moderate height was run to settle the structure and simulate exposure of the structure to natural wave conditions prior to a design storm. Design waves then were generated for 30 min for each test. To minimize contamination of the incident waves by reflected waves, test time was accumulated on the structure in cycles of 30-sec generation time followed by a 5- to 7-min stilling time. The structure then was drained and the damage documented by photographs and written summaries. Damage to the structure was repaired using a selected repair method, and the structure was again documented with photographs. The structure then was exposed to the same test conditions that caused the damage to determine the repair option's response to wave attack. After testing a repair method, the flume was drained and the after-test conditions of the test sections were documented with photographs and written summaries. The test section armor layers and berm stone then were rebuilt, documented with photographs, and exposed to test conditions to produce damage for testing another repair method. The Phase I test procedure is shown in Figure 6; typical wave damage to the structures is shown in Photos 2 (5.00-ft flume) and 3 (6.75-ft flume).

19. Previous studies had found that the plunger-type wave generator used in these flumes would sometimes create a single, larger wave when the generator was turned off. This wave would follow the set of waves intended for the test and could cause substantial damage to the structure. A board therefore was lowered into the flume at the end of each cycle of waves to cut off the last wave before it could impact the structure.

Phase II

20. Design of the test structure for Phase II was the same as in

Phase I, but the swl was raised to concentrate the localized damage on the sea-side slope near the crown. Various wave heights were tried to determine a height that would provide a moderate amount of damage to the structure at each of the wave periods selected. Repair options a, b, d, and e were tested in Phase II.

21. During the Phase I testing, it was noted that the structures stabilized during the first half of the test and that damage seldom occurred during the remainder of the test sequence. When damage was repaired, those portions of the structure unaffected by the repair remained stable when the wave attack was repeated on the repaired structure. For this reason, it was decided not to repeat the initial set of waves for each repair option. Instead, damage sustained by the structure during testing of repair option a was then repaired by option b and the test continued. Similarly, damage sustained during testing of repair option d was then repaired by option e and the test continued. This reduced the length of time required for a test series and was deemed adequate based on observations of Phase I tests.

22. The Phase II test procedure was performed as follows. After the structure was placed in the flume and documented, the tank was filled to the selected water level, and a set of waves of moderate height was run to settle the structure as in Phase I. Design waves then were generated for 30 min, the tank was drained, and the damage was documented with photographs and written summaries. The structure was repaired with repair option a, documented with photographs, and the flume filled to the selected water level. Design waves were generated for another 30 min; the tank was drained; and damage was documented with photographs and written summaries, repaired with repair option b, and documented again with photographs. The flume then was refilled to the selected water level, design waves were generated for 30 min, the tank was drained, and the damage documented with photographs and written summaries.

23. At this point, the armor layer was removed, and the structure was rebuilt. Photographs were taken to document the initial condition of the structure, and the procedure described in paragraph 20 was repeated except that repair options d and e replaced options a and b, respectively. Figure 7 illustrates the procedure used in Phase II; typical damage to the structure in Phase II is shown in Photo 4.

24. As in Phase I, test time with design waves was accumulated with

cycles of 30-sec of wave generation followed by 5 to 7 min of stilling time, and a board was lowered into the flume to prevent the final wave in each series from reaching the structure.

Phase III

25. In Phase III, three-dimensional tests were conducted using a structure with a trunk section similar to that used in Phase I and a head section designed according to the same design guidance and for the same design conditions. Because possible reactions of the head section to the design waves were unknown, the shortened test procedure used in Phase II was deemed inappropriate. Instead, the structure was torn down and rebuilt after each repair option tested as in Phase I. Repair options a, b, d, and e were tested in Phase III.

26. Phase III followed the general testing procedure used in Phase I, but with two differences related to differences in the testing facilities. First, because the flume used in Phase III had a piston-type wave generator rather than the plunger-type used in Phases I and II, the single larger wave noted above at the end of each test series was not generated. The last wave in each test series was therefore not isolated in Phase III. Second, due to the greater length of the flume used in Phase III, waves could be generated for a longer period of time before being contaminated by reflected waves. Design waves were therefore generated for 30 min for each test with the time being accumulated in cycles of 45 sec of test time followed by 5 to 7 min of stilling time. Typical damage to the structure during Phase III testing is shown in Photo 5.

Test results

27. Results of the tests are given in Table 1, which shows the number of stones displaced after each repair in each test, and displayed graphically in Figures 8, 9, and 10. Because the tests were conducted on three different lengths of breakwater section, results were standardized for the figures by dividing stones displaced by length of breakwater section. The column heights in Figures 8, 9, and 10 indicate the average number of stones displaced per foot of breakwater trunk section per test for each of the repair options for Phases I, II, and III, respectively. Each column in the figures is also divided to show the relative portion of each total contributed by each of the wave periods tested. Note that these portions have been weighted by the

number of tests conducted at that period.

28. Figure 11 shows the number of stones displaced on the structure head during Phase III testing. Because only one breakwater head design was tested, this figure shows the actual number of stones displaced, rather than average stones per foot as used in Figures 8, 9, and 10.

Observations

Data variability

29. A large amount of variability is evident in the results. This is due to several factors, including the irregular shape of the stones and random stone placement on the structures. A major additional factor in the scatter was the decision to allow damage to occur naturally on the structure by wave action, rather than artificially damaging the structure by methods such as manually removing stones to create a void. It should also be emphasized that damage being considered was "localized" and that a damage area was frequently only one or two stone diameters across. Local instabilities and irregularities could therefore significantly affect an individual repair.

30. Nearly all the damage to the structures occurred during the first half of the tests, after which the structure would usually stabilize. While the structure remained stable against the regular monochromatic waves impinging against it, the structure frequently had large holes in the armor layer that would probably fail if the wave conditions were to change. In these cases, repairs made to the structure could destabilize the area and cause significant damage, producing much scatter in the results.

General observations

31. Although no two tests produced the same results, several general trends were evident in the tests and are reported here. These observations may not have occurred in every test, but were typical of the results.

32. The shakedown cycle produced no noticable movement of the armor stones. No armor stones were displaced, and rocking was slight if detected at all. During the test cycles, rocking was observed during both the uprush and downrush periods of wave action, while displacement usually occurred only during the downrush portion.

33. A slight shifting of stones below the swl during the test cycles

tended to tighten the outer armor layer. This movement was not directly observed, but was indicated by gaps that developed between the stones in the outer armor layer above the swl.

34. Damage to the structures originated near the swl, with damage concentrated at and slightly below the swl during the 1.25- and 1.43-sec tests, and at and slightly above the swl during the 1.67-sec tests.

35. All damage to the trunk of the structure occurred on the outer armor layer. A typical damage sequence on the trunk would start with stones being packed closer together below the swl, which loosened stones higher up the structure. Stones thus loosened or improperly keyed during initial construction would then be displaced, leaving a hole in the outer armor layer. This hole would either be filled by sloughing of stones from the outer armor layer farther up the structure, or the surrounding stones would interlock forming a tight ring around the hole (which might be several stone widths in diameter). Eventually, all holes were surrounded by an interlocked ring of stones, and the structure stabilized. This condition was usually reached during the first half of the test cycles.

36. Damage on the head of the structure tended to penetrate deeper into the structure, with the filter layer being exposed during the 1.67-sec tests. Less sloughing was observed, while movement of stones in the underlayer was noticeable.

37. It should be emphasized that the structure at this point was stable for the regular, monochromatic wave conditions under which the testing was conducted. Should the severity of the wave attack increase, it is likely that the stability of the ring of stones surrounding the holes would be lost and extensive damage would occur.

38. Results from the various phases of testing tended to contradict each other in several ways. In Phase I, both in the repairs above the swl and the repairs on the entire sea-side face, repairs performed better when voids were simply filled with new stone than when the stones were reoriented to improve the seating during the repairs. This is the opposite of the results shown in Phase II, whereas in Phase III the seated stones performed better on the repairs from structure toe to crown, but the simple fill performed better on the repairs from swl to crown. For a given repair type, repairing the structure on the entire sea-side face was more effective in Phase I than

repairing only above the swl, while the reverse is shown in Phases II and III.

39. It was surprising to find that in some tests the repairs performed better when the voids were simply filled rather than filled with reorientation of the surrounding stone, and that some tests showed superior performance if the structure was only repaired above the swl rather than on the entire sea-side face. These findings may be explained as follows. As noted earlier, the structure tended to stabilize midway through the test cycles with voids surrounded by rings of stable, interlocked armor units. Reorienting the stones to improve the seating of the new stones may have disturbed the stability of the ring surrounding the void, thus increasing the damage. Further, in the area below the swl, wave action appeared to shift and tighten the armor stones. This too could be disturbed by reorientation during repair.

40. Figure 12 shows the combined results of all phases of the tests on the structure trunks. Although the trends varied during each of the tests, the figure shows that overall there was very little difference in the various repair methods above the swl. Repair of the entire sea-side face by just filling the voids showed the poorest performance overall, while local rehabilitation from the toe to the crown showed the best performance.

41. A more noticeable difference is seen in Figure 11 for the head tests, where the improvement in structure stability by repairing from the toe to the crown rather than swl to crown is obvious. However, the limited number of head tests conducted and the scatter seen in the results render this trend suspect.

42. Because wave conditions were selected to yield a similar amount of damage during each test, variations in wave period, water depth, or sea-side slope should have little effect on the performance of the repair. Figures 13, 14, and 15 plot the results based on wave period, water depth at the toe of the structure, and sea-side slope, respectively, to demonstrate that there are no evident trends caused by these factors.

Conclusions

43. The tests were inconclusive regarding the relative effectiveness of the various repair methods, but demonstrated some important factors to be considered in the repair of rubble-mound structures. Much of the stability of

these structures is due to the interlocking of armor stones. Shifting or reseating these stones during a repair can weaken the interlocking and decrease the stability of the structure. Great care must therefore be taken in the handling of stones on the structure. Similarly, new stone placed on the structure must be properly seated to maximize the interlocking of the new stone with the existing stone to ensure a stable repair.

44. Design guidance for repair of rubble-mound structures is necessary to ensure stable and cost-effective repairs. Additional testing will be needed to provide this guidance, particularly with irregular wave conditions. With a larger data set, trends in the results should be identifiable, and design guidance may then be prepared.

Table 1

Number of Stones Displaced After Repairs

Flume Repair Type	Phase I 6.75 ft	Phase I 5.00 ft	Phase II 6.75 ft	Phase III Trunk 15.00 ft	Phase III Head 15.00 ft
			Period = 1.25 sec		
FILL, SWL	6	9	8	5	0
SEAT, SWL	8	9	9	1	0
REHAB, SWL	11	1			
FILL, TOE	4	4	8	15	1
SEAT, TOE	7	0	13	15	0
REHAB, TOE	3	5			
			Period = 1.43 sec		
FILL, SWL			2	9	3
SEAT, SWL			1	7	1
FILL, TOE			11	8	1
SEAT, TOE			1	4	3
			Period = 1.67 sec		
FILL, SWL			14	2	9
SEAT, SWL			1	10	6
FILL, TOE			12	3	1
SEAT, TOE			7	3	0

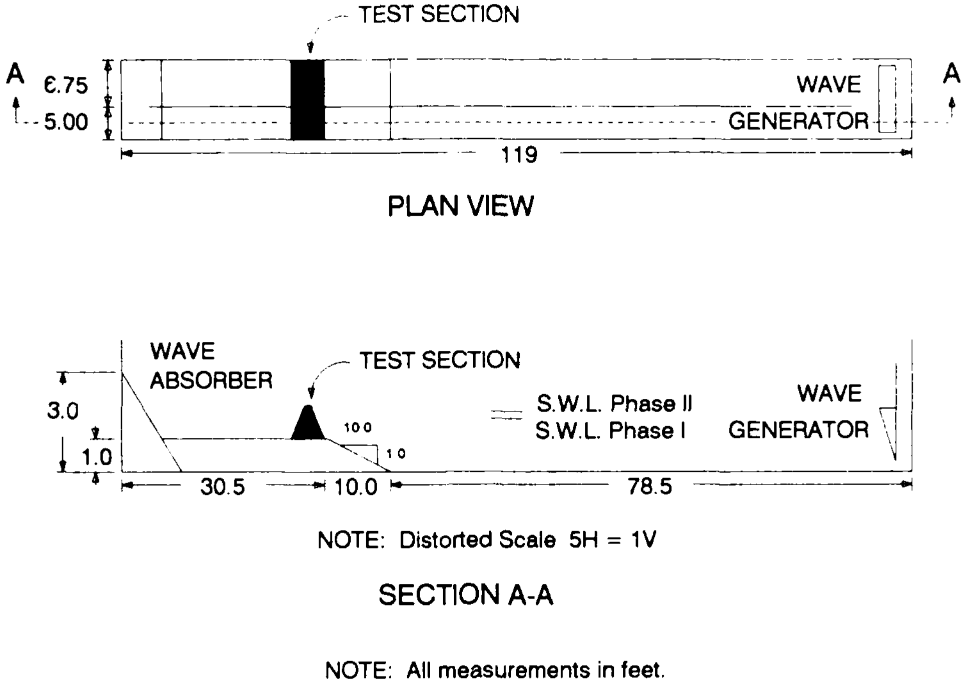


Figure 1. Wave flume used during Phases I and II

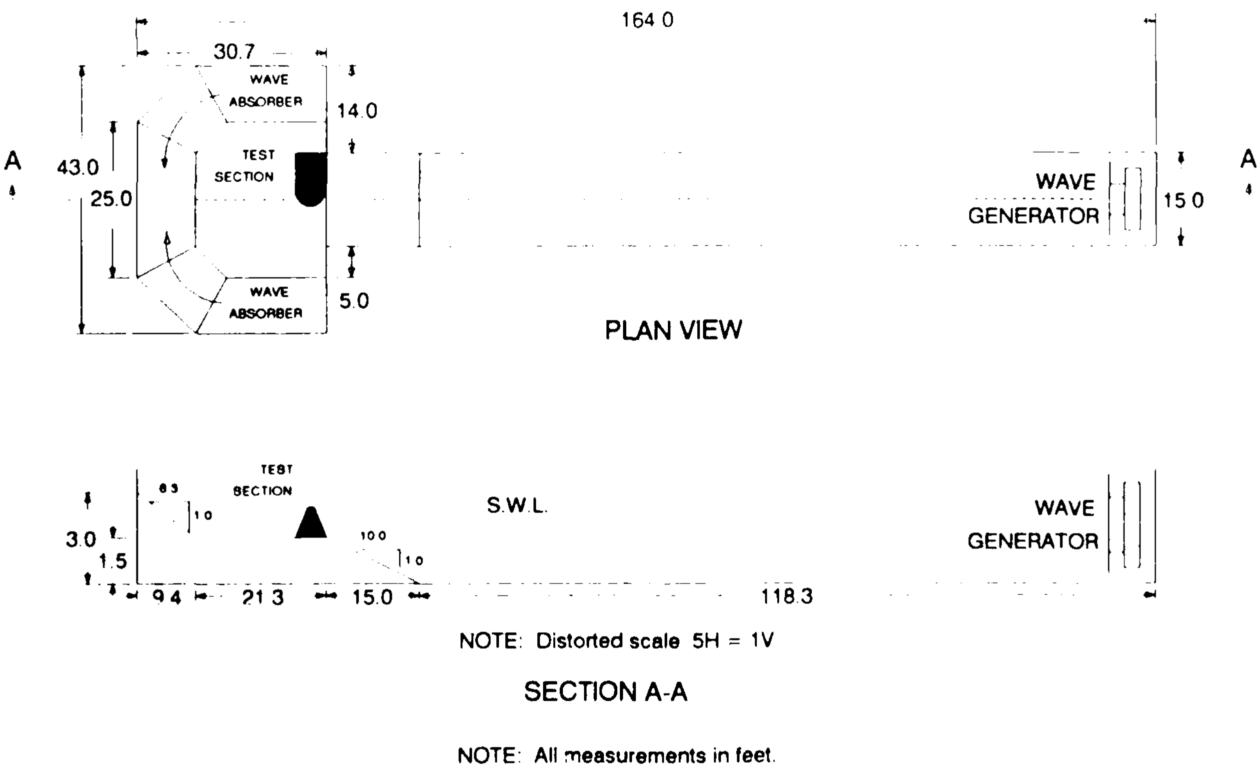
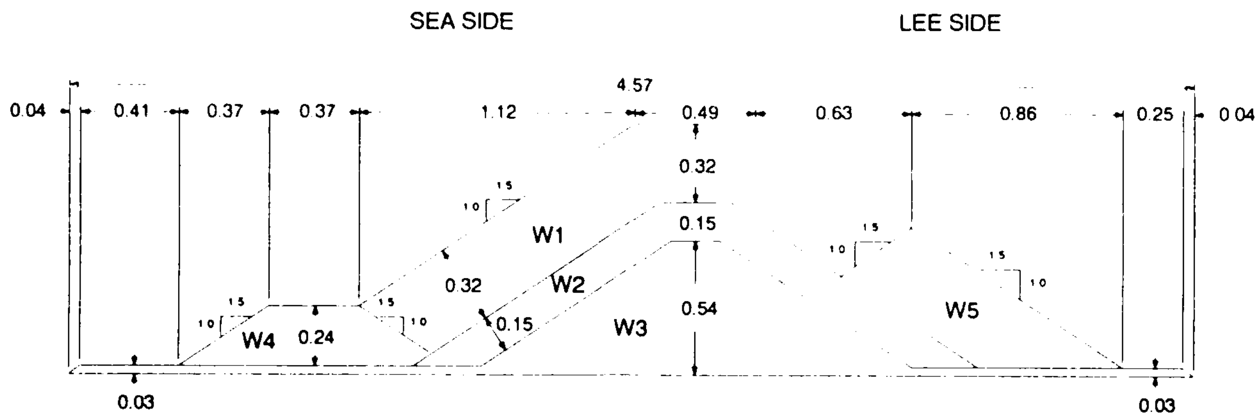


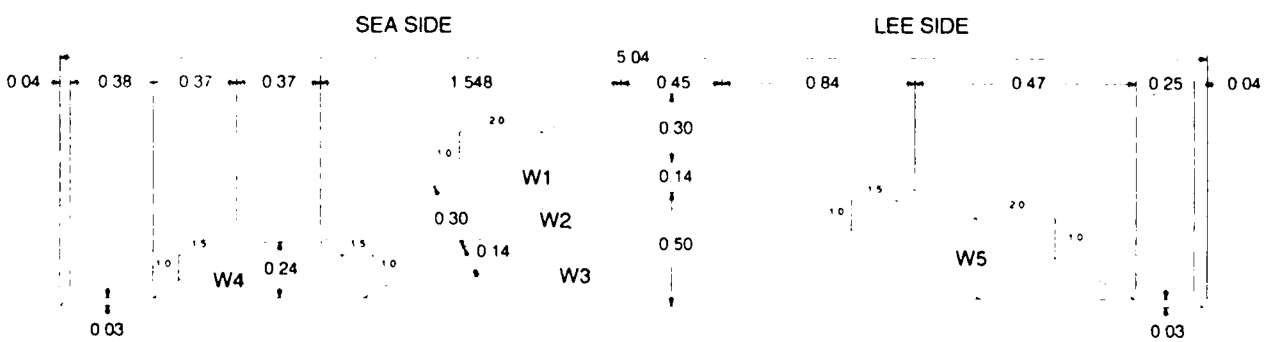
Figure 2. Wave flume used during Phase III



NOTE: ALL MEASUREMENTS IN FEET.

W1 0.53 lb - 0.89 lb	at 165 pcf
W2 0.05 lb - 0.09 lb	at 165 pcf
W3 0.0002 lb - 0.0035 lb	at 165 pcf
W4 0.07 lb - 0.13 lb	at 165 pcf
W5 > 0.38 lb	at 165 pcf

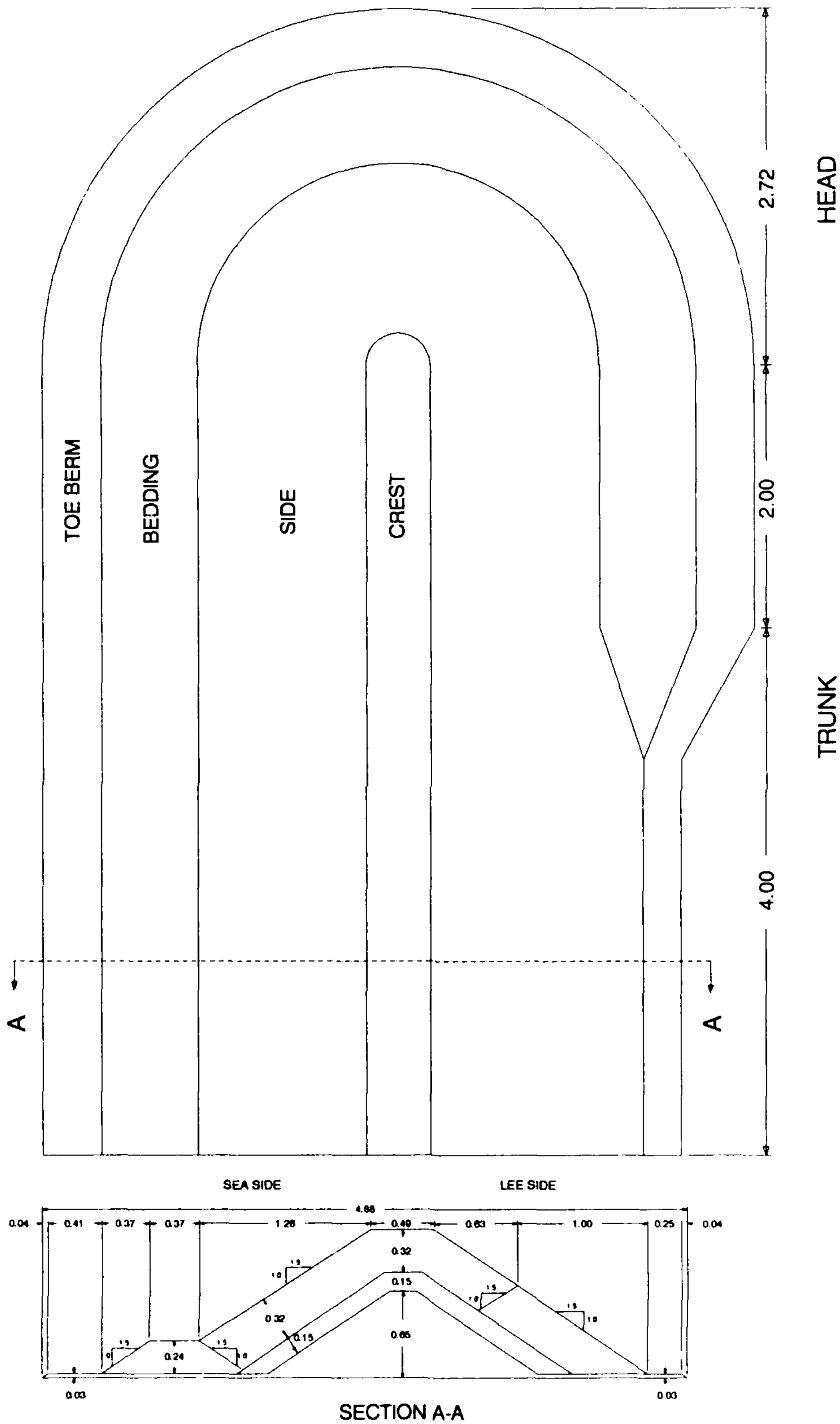
Figure 3. Cross section of structure used in 5.00-ft flume during Phase I



NOTE ALL MEASUREMENTS IN FEET

W1 0 41 lb - 0 69 lb	at 165 pcf
W2 0 04 lb - 0 07 lb	at 165 pcf
W3 0 0001 lb - 0 0028 lb	at 165 pcf
W4 0 07 lb - 0 13 lb	at 165 pcf
W5 > 0 38 lb	at 165 pcf

Figure 4. Cross section of structure used in 6.75-ft flume in Phases I and II



NOTE: All measurements in feet.

Figure 5. Test section used in Phase III. The head section was symmetrical about the crest

INITIALIZE DAMAGE

1. Construct new test section.

2. Expose test section to wave action for 30 min to produce damage.

3. Document damage.

REPAIR OPTION

4. Repair test section.

5. Document repair.

6. Expose repaired test section to wave action for 30 min using same wave conditions as above.

7. Document damage.

NEXT TEST

8. Disassemble test structure.

9. Repeat steps 1 through 8 for each of the 6 repair options.

Figure 6. Phases I and III test procedure

INITIALIZE DAMAGE

1. Construct new test section.
2. Expose test section to wave action for 30 min to produce damage.
3. Document damage.

FIRST REPAIR OPTION

4. Repair test section using repair option (a).
5. Document repair.
6. Expose repaired test section to wave action for 30 min using same wave conditions as above.
7. Document damage.

SECOND REPAIR OPTION

8. Repair test section with repair option (b).
9. Document repair.
10. Expose repaired test section to wave action for 30 min using same wave conditions as above.
11. Document damage.

NEXT TEST

12. Disassemble structure.
13. Repeat steps 1 through 12 for each of the wave conditions.
14. Repeat steps 1 through 13 using repair options (d) and (e).

Figure 7. Phase II test procedure

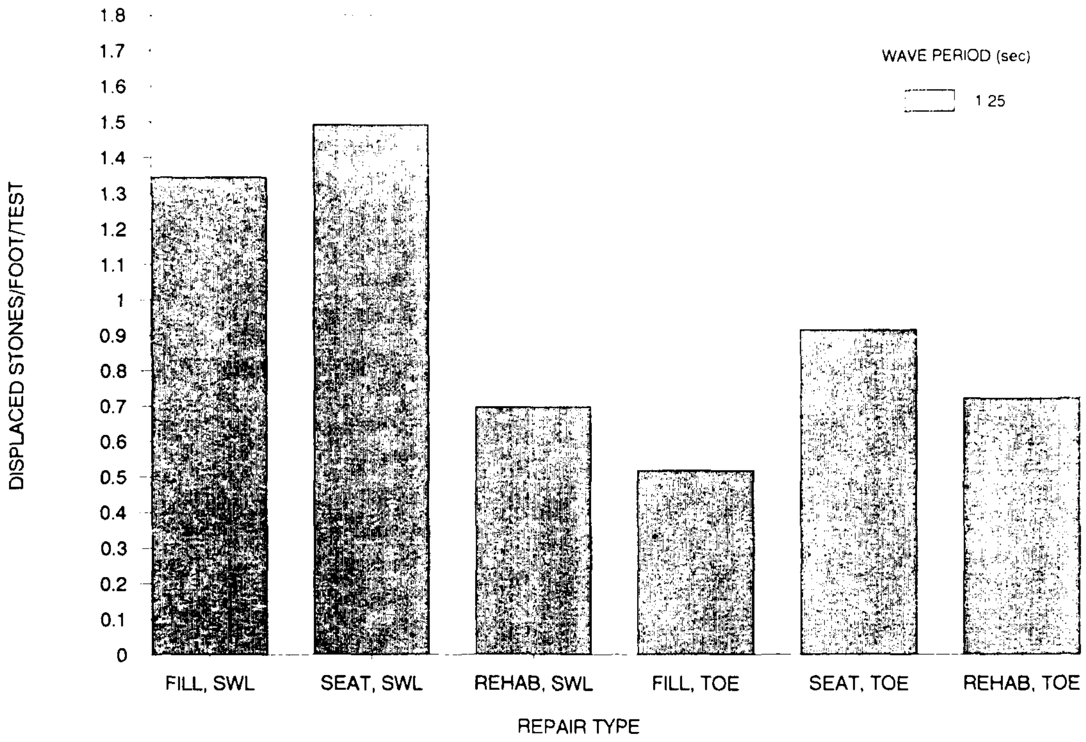


Figure 8. Displaced stones after repairs for the repair tests in Phase I

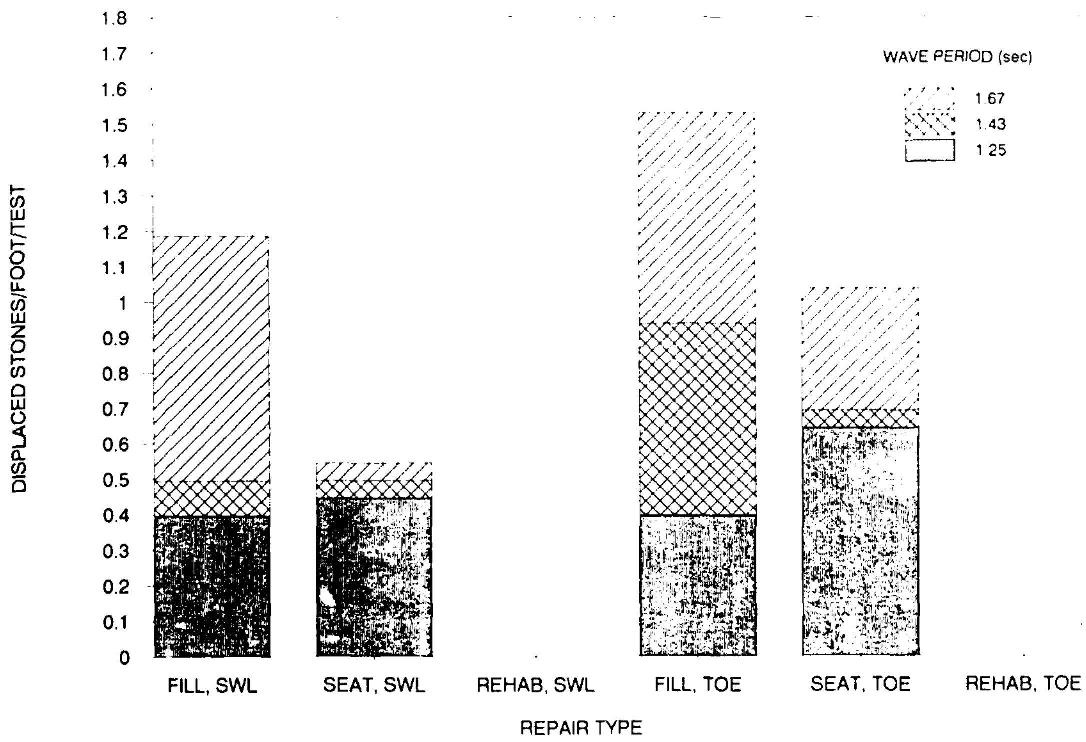


Figure 9. Displaced stones after repairs for repair tests in Phase II

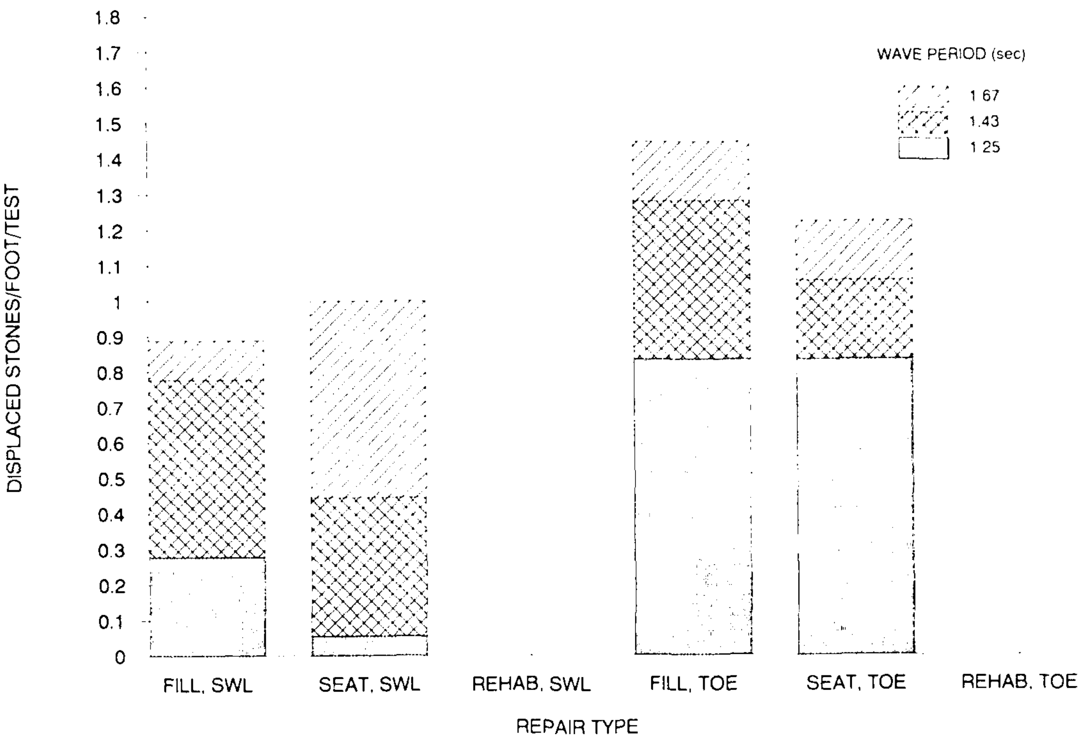


Figure 10. Displaced stones on trunk after repairs for repair tests in Phase III

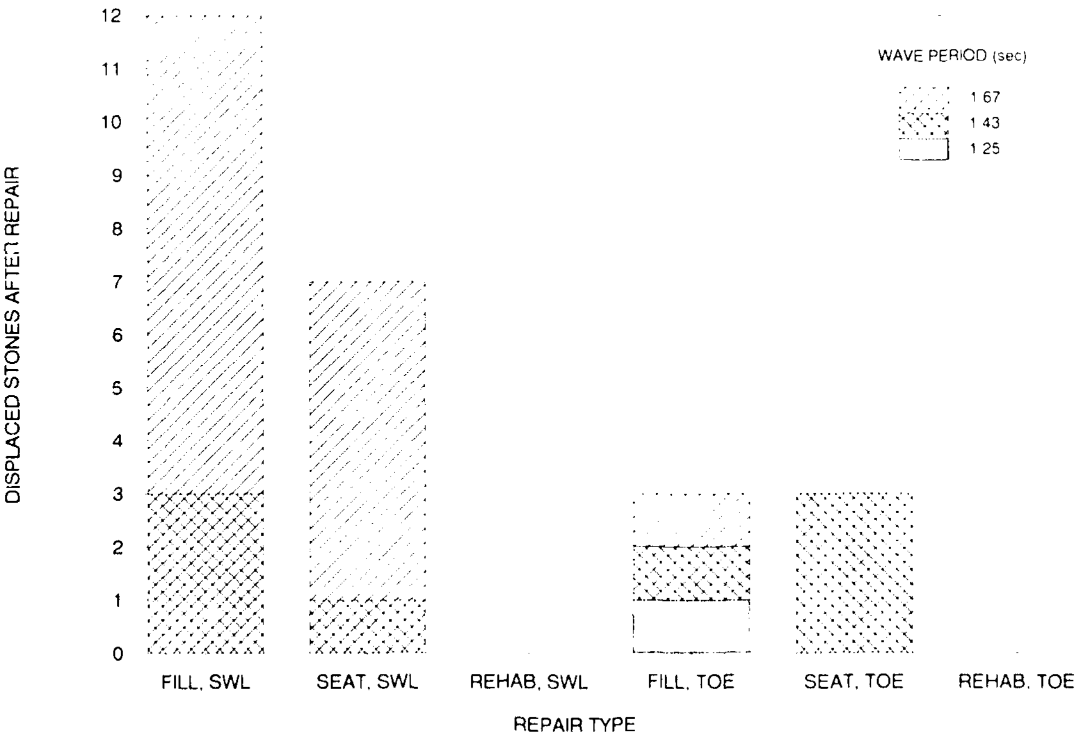


Figure 11. Displaced stones on structure head after repair tests in Phase III

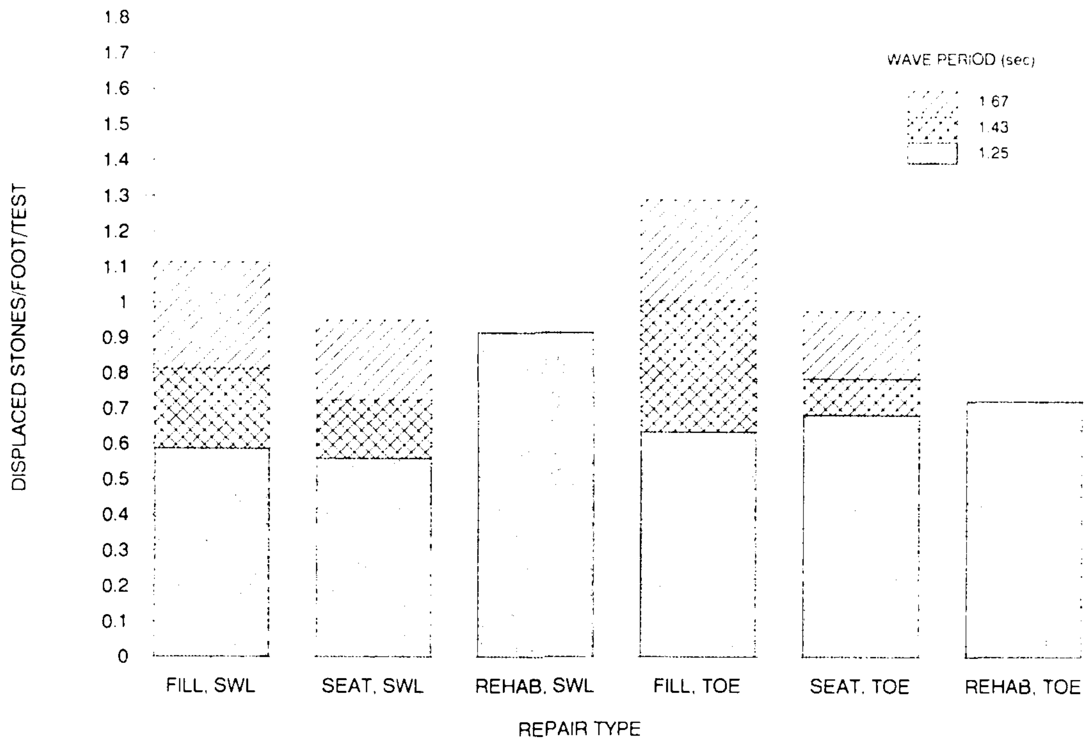


Figure 12. Averages of stones displaced after repairs in Phases I, II, and III (trunk)

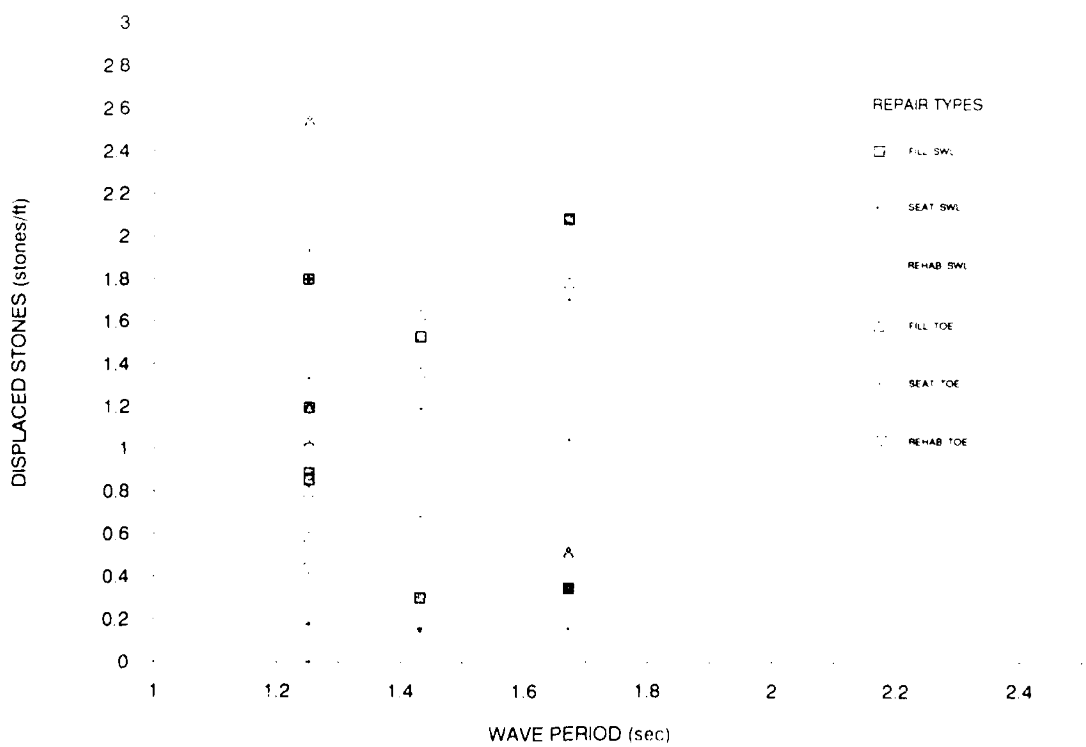


Figure 13. Effect of wave period on number of stones displaced after repairs

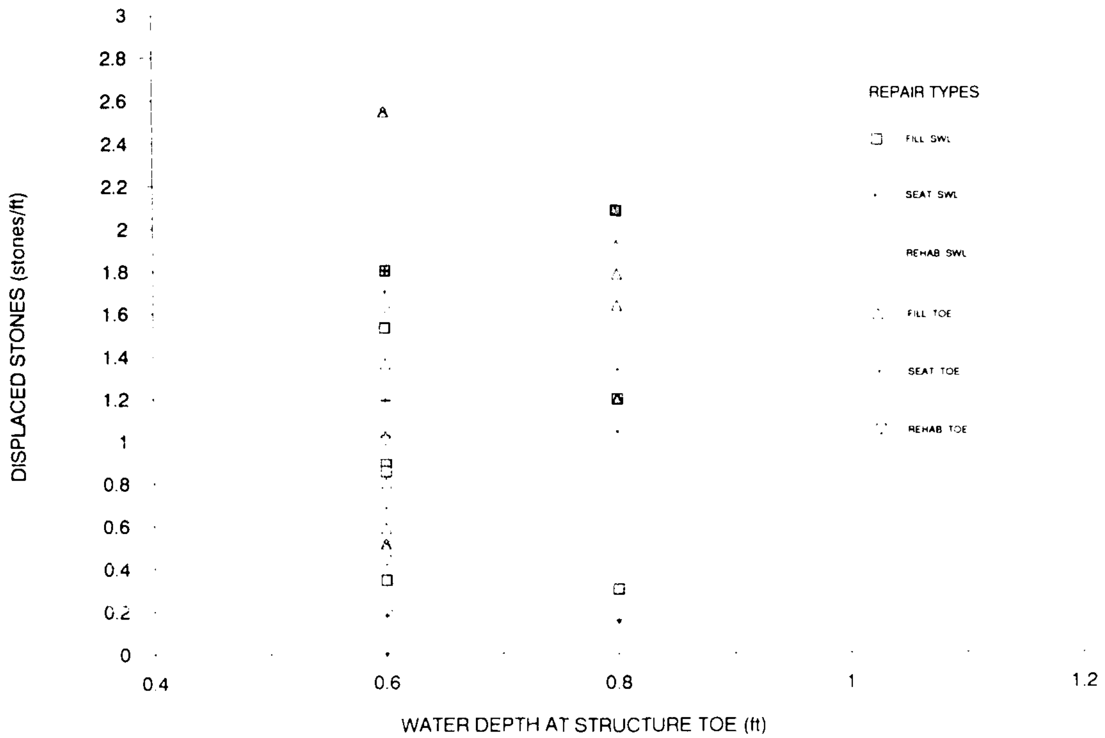


Figure 14. Effect of water depth on number of stones displaced after repairs

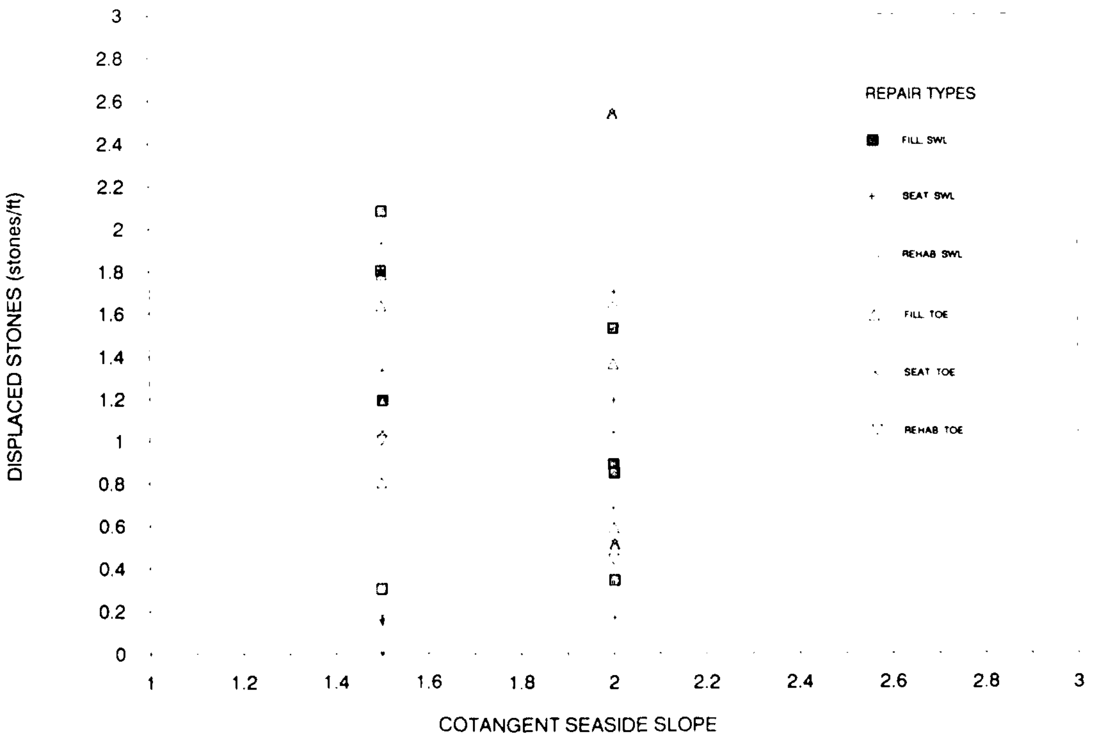


Figure 15. Effect of sea-side slope on number of stones displaced after repairs

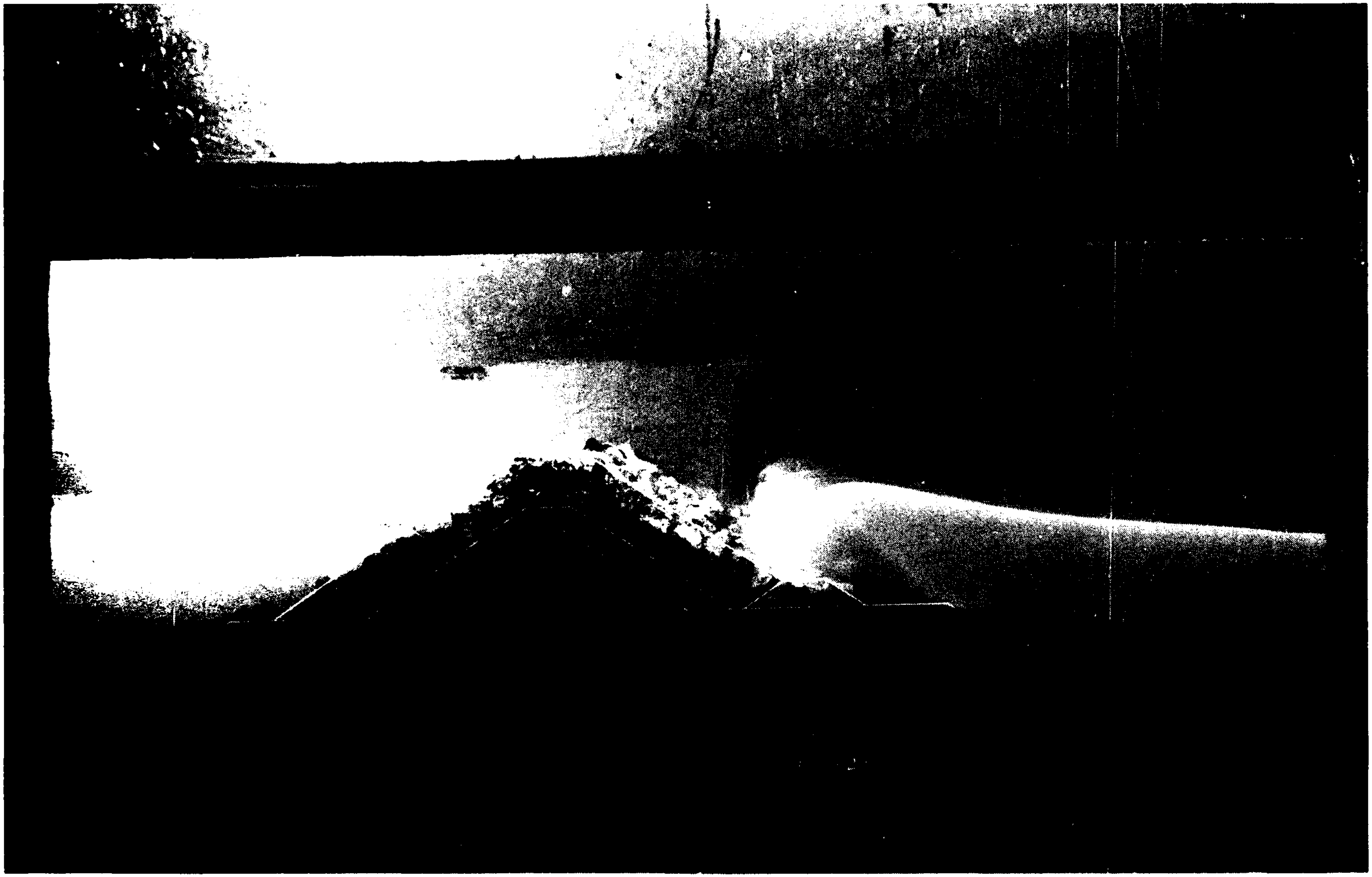


Photo 1. Wave action on structure in 5.00-ft flume during Phase I

Photo 2. Typical damage to structure in 5.00-ft flume during Phase I

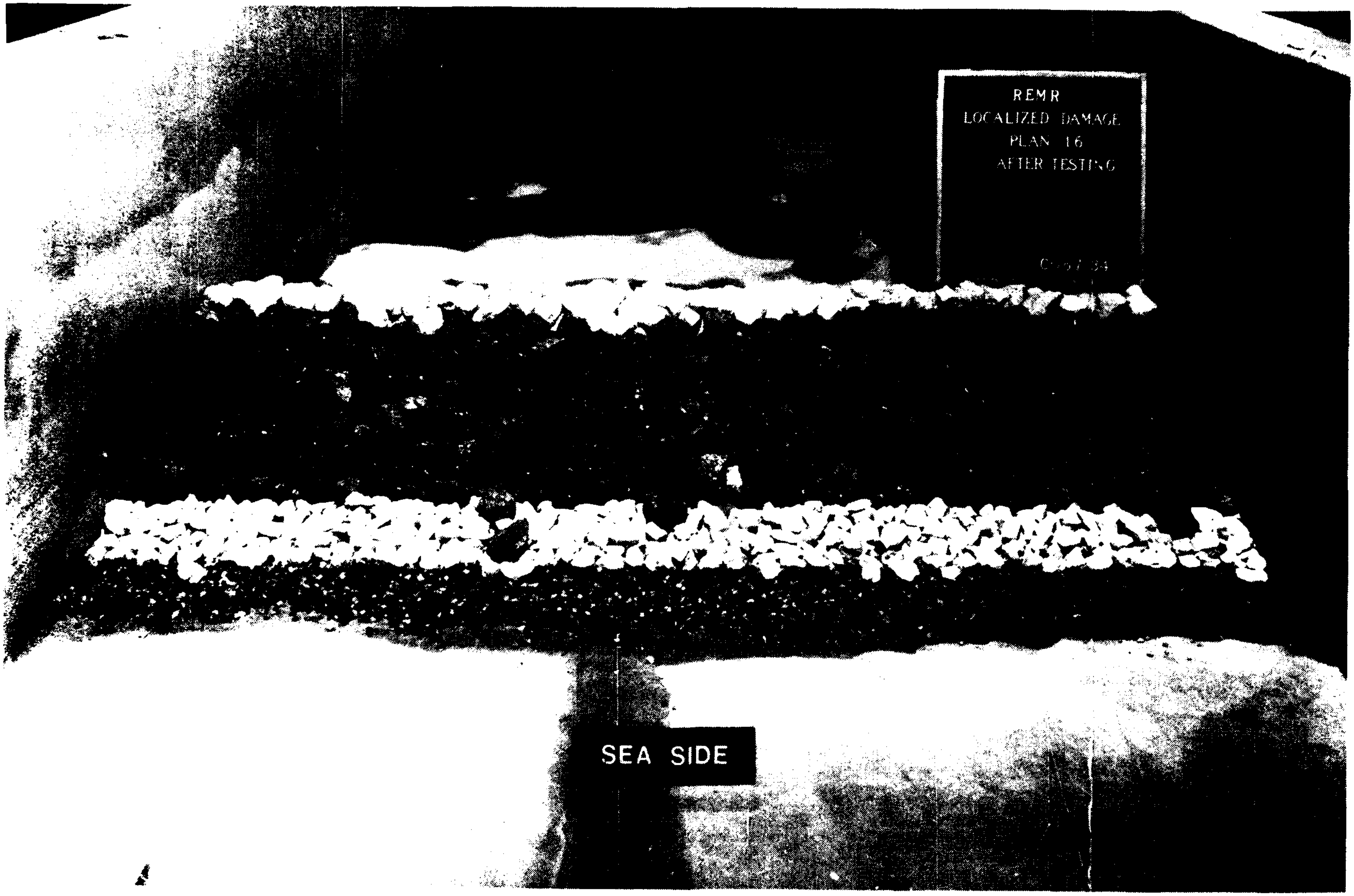


Photo 3. Typical damage to structure in 6.75-ft flume during Phase I

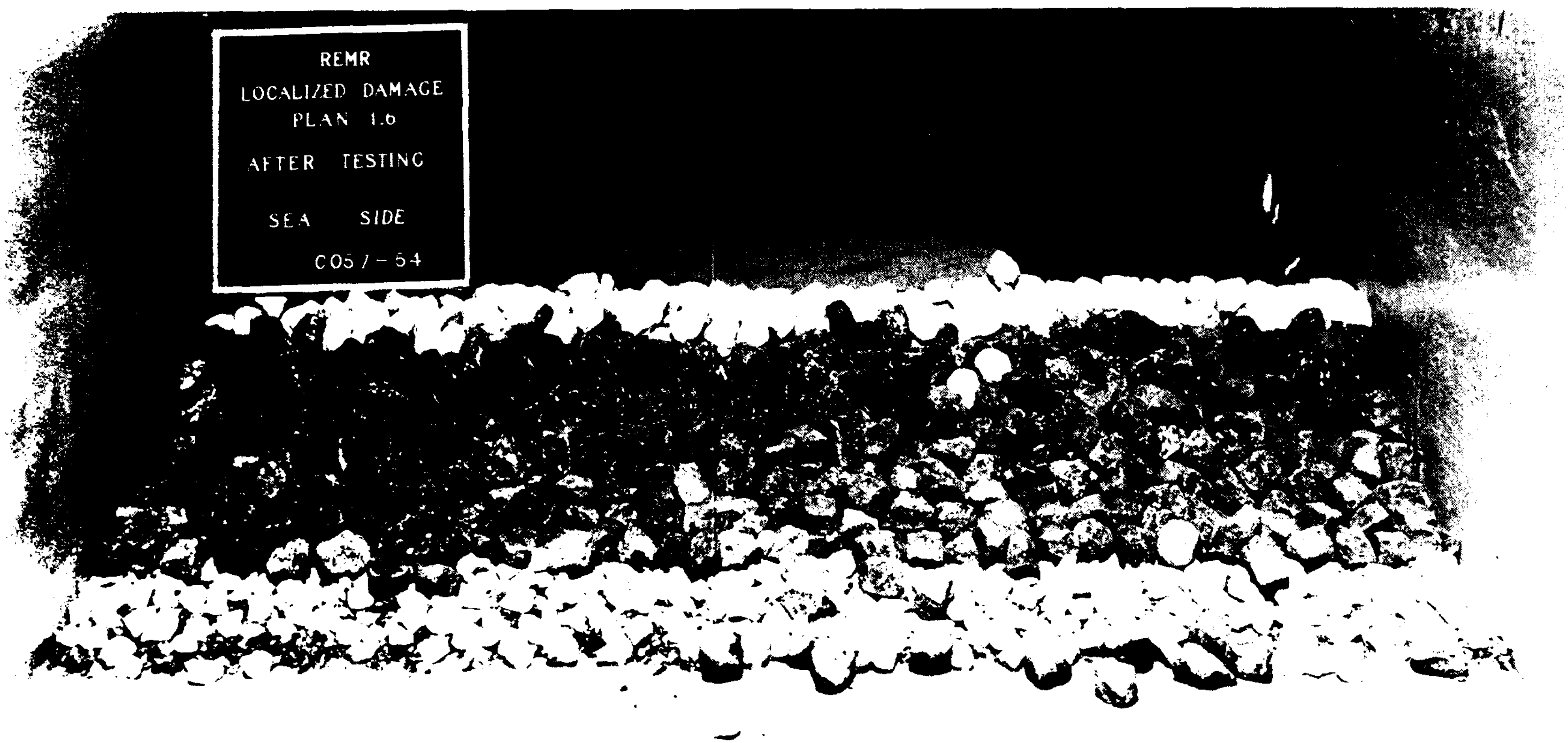


Photo 4. Typical damage to structure in 6.75-ft flume during Phase II

Photo 5. Typical damage to structure in T-shaped flume during Phase III