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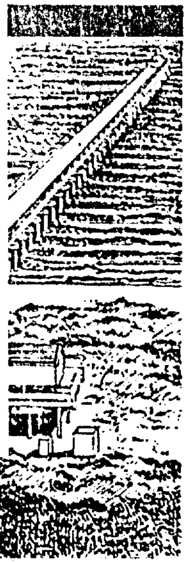
CERC

TECHNICAL REPORT REMR-CO-12

STABILITY OF TOE BERM ARMOR STONE AND TOE BUTTRESSING STONE ON RUBBLE-MOUND BREAKWATERS AND JETTIES

Physical Model Investigation



by

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

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	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 — One layer of 7.5-ton tribars used on 8- to 12-ton toe buttressing stone. Tribar and concrete ribcap rehabilitation of a portion of the Hilo Breakwater, Hilo Harbor, Hawaii.

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A series of two-dimensional (2-D) and three-dimensional (3-D) breakwater stability experiments was developed and conducted to address the sizing of toe berm and toe buttressing stone in breaking wave environments. The 2-D tests focused on sizing of toe stone on rubble-mound structure trunks exposed to 90-deg wave attack, i.e. wave orthogonals perpendicular to structure crest. Toe berm armor stone sizing for oblique wave attack on rubble-mound structure heads and trunks was examined in the 3-D model tests. Guidance for sizing toe berm armor stone was developed for a range of wave and still-water level conditions. Guidance for sizing of toe buttressing stone was addressed for a limited set of incident wave conditions on structure trunks.

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PREFACE

Authority to carry out this study was provided to the US Army Engineer Waterways Experiment Station (WES) Coastal Engineering Research Center (CERC) by the US Army Corps of Engineers (USACE), under the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program Civil Works Research Work Unit 32278, "Rehabilitation of Rubble-Mound Structure Toes."

Physical model tests of both toe berm and toe buttressing stone for rubble-mound coastal structures were conducted under the general direction of Mr. James E. Crews and Dr. Tony C. Liu, REMR Overview Committee, USACE; and Messrs. Jesse A Pfeiffer, Jr., Directorate of Research and Development, USACE; John H. Lockhart, Coastal Technical Monitor, USACE; William F. McCleese, REMR Program Manager, WES, and D. D. Davidson, REMR Coastal Problem Area Leader, WES.

The study was conducted by personnel of CERC under the general direction of Dr. James R. Houston, Chief, and Mr. Charles C. Calhoun, Jr., Assistant Chief, CERC; and under direct supervision of Mr. C. E. Chatham, Chief, Wave Dynamics Division and Mr. Davidson, Chief, Wave Research Branch. The study was designed and planned by Mr. Dennis G. Markle, Research Hydraulic Engineer, Wave Research Branch. Models were constructed and tests were carried out by Messrs. Willie G. Dubose, Marshall P. Thomas, and C. Ray Herrington, Engineering Technicians, assisted by Messrs. Cornelius Lewis, Engineering Technician, Raymond Reed, Contract Student, and Ernest Galloway, Student Aid, under the supervision of Mr. Markle. This report was prepared by Mr. Markle.

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

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
feet	0.3048	metres
pounds (force)	4.448222	newtons
pounds (force) per cubic foot	157.089467	newtons per cubic metre

STABILITY OF TOE BERM ARMOR STONE AND TOE BUTTRESSING STONE ON RUBBLE-MOUND BREAKWATERS AND JETTIES

Physical Model Investigation

PART I: INTRODUCTION

Background

1. Failure of rubble-mound breakwater and jetty toes is a problem that has plagued a majority of US Army Corps of Engineers (CE) divisions and districts responsible for designing, constructing, and maintaining these structures. Instability and partial failure of a rubble-mound structure's toe does not become evident until it has resulted in damage to the primary armor which has progressed up to or above the still-water level (swl).

2. Under the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program, the US Army Engineer Waterways Experiment Station's Coastal Engineering Research Center was authorized and funded to conduct a work unit under the Construction, Operation, and Maintenance Research Area entitled Rehabilitation of Rubble-Mound Structure Toes. The first objective of this work unit was to gain an understanding of the toe stability problems experienced by field designers and determine what research was needed to develop adequate guidance for design of stable rubble-mound toes. The results of a field experience survey, conducted within the CE division and district offices (Markle 1986), are summarized as follows:

> In general, there appear to be three major problem areas with rubble-mound coastal structure toes. One of these pertains to the proper sizing and placement of toe buttressing stone. The purpose of buttressing stone is to stabilize the slope armor by preventing downslope slippage of the armor layer. For the buttressing stones to function properly, they must be of sufficient weight and placed in such a way that they are stable in the wave and flow environment to which the structure is subjected. The second major problem area is with toe berms. A toe berm's primary function is to protect a structure placed on an erodible bottom from being undermined by wave- and/or flow-induced scour. Resisting downslope slippage of the primary slope armor is a secondary function of the toe berm. For a toe berm to function properly, it, like the toe buttressing stone, must be composed of materials and constructed in a manner that will be stable in the

> incident wave and flow environment. The third major problem area deals with failure induced by an eroding foundation. Toe buttressing stones and toe berms are susceptible to damage and failure when placed on erodible bottom material. The stones may be sized adequately for the expected level of energy to which they may be exposed, but the exposed bottom material at the outer perimeter of the structure may erode readily under these conditions. Also, an inadequately designed bedding, or filter, material may allow foundation material to leach through it and the toe berm or buttressing armor. Either one or a combination of both of these factors can result in the undermining and displacement of stones that were otherwise stable in the wave and flow environment.
>
> In summary, a toe failure may stem from any one or a combination of the above. Guidance exists for proper design of bedding (filter) layers based on soil types, but very little guidance is available for the sizing and geometries needed for the proper design of toe berms and buttressing stone for incident wave environments. Most design work done in this area by CE districts is based on field experience and engineering judgment. A scouring bottom is a problem in itself. No matter how well a toe is designed, if the local bottom materials (sands, silts, clays, etc.) are exposed to sufficient energy levels for scour to occur, the toe of the structure is likely to fail unless the toe berm is extended to a point where the energy levels are below that of scour initiation. In most cases this is not practical or feasible. In these instances, sufficient toe berm material, that in itself is stable for the wave and/or flow environment, must be placed so that as the structure toe undermines the berm and bedding material can slough off into the scour hole. This occurrence will provide some armoring to reduce the rate of scour and thus increase the usable, or functional, life of a structure.

3. Based on survey findings, it was concluded that design guidance is seriously needed on the proper sizing and placement configurations required to provide adequate buttressing stone and toe berms for rubble-mound coastal structures. Existing design guidance for toe berms is based on field experience and engineering judgment (weight of toe armor should be at least one-tenth the weight of the primary armor (Shore Protection Manual (SPM) 1984) or on research by Brebner and Donnelly (1962) and Tanimoto, Yagyu, and Goda (1982) on foundation and toe berm materials lying beneath and/or in front of vertical structures, i.e. caissons, timber cribs, etc.

Purpose and Approach

4. The purpose of this study was to develop suitable design guidance for the sizing of toe berm and toe buttressing stone using experimental results from laboratory physical models. A series of two-dimensional (2-D) and three-dimensional (3-D) stability tests was developed and conducted in physical models to address the sizing of toe berm and toe buttressing stone in breaking wave environments. The 2-D tests focussed on toe stone sizing on rubble-mound structure trunks exposed to 90-deg wave attack, i.e., wave orthogonals perpendicular to structure crest. Toe berm armor stone sizing for oblique wave attack on rubble-mound structure heads and trunks was examined in the 3-D model tests. Guidance for sizing toe berm armor stone was developed for a range of wave and swl conditions. Guidance for sizing of toe buttressing stone was developed for a limited set of incident wave conditions on structure trunks.

PART II: MODEL DESIGN AND SETUP

Test Facilities

2-D tests

5. All tests were conducted in concrete flumes equipped with vertical displacement, monochromatic wave generators. Toe buttressing stone sections were tested in a 5-ft-wide* flume (Figure 1), while four toe berm stone sections were tested simultaneously in 5- and 6.75-ft-wide flumes that share a common generator (Figure 2). Two toe berm test sections were constructed adjacent to one another in each of the flumes (Photo 1).

3-D tests

6. A majority of the tests were conducted in an L-shaped wave basin which has overall dimensions of 250 ft long and 50 and 80 ft wide at the top and bottom of the L, respectively, and 4.5 ft deep in the test area (Figure 3). The L-shaped basin was equipped with a flap wave generator, and monochromatic waves were generated for these tests.

7. Some of the tests were conducted in a T-shaped wave basin 164 ft long, 43 and 15 ft wide at top and bottom of the T, respectively, and 3.3 ft deep (Figure 4). The basin was equipped with a horizontal displacement wave generator capable of making both monochromatic and spectral waves.

Test Facility Calibration

8. Prior to construction of the test sections, each test facility was calibrated for a range of incident wave conditions and water depths. This is the preferred method of calibration because it eliminates the influence of reflected waves from the structure so that wave conditions match the design conditions (defined through prototype wave gaging and/or hindcasting done prior to prototype design and construction). Changes in model water surface elevations as a function of time, i.e., wave heights, were measured by electrical parallel resistance or capacitance wave gages. Wave gages, or gage arrays in the case of spectral wave conditions, were placed in each of the test facilities at the approximate location where the sea-side toe of the

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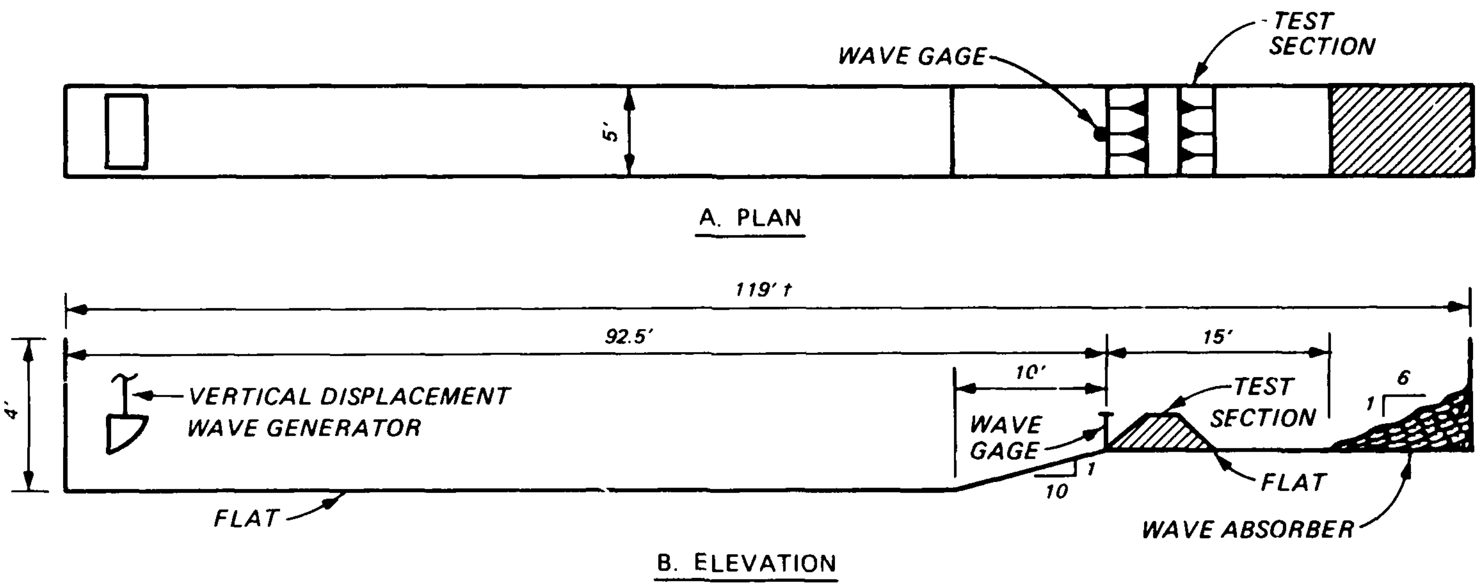
* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.



† ALL DIMENSIONS IN MODEL FEET

Figure 1. Test flume geometry and wave gage and test section locations for two-dimensional toe buttressing stone tests

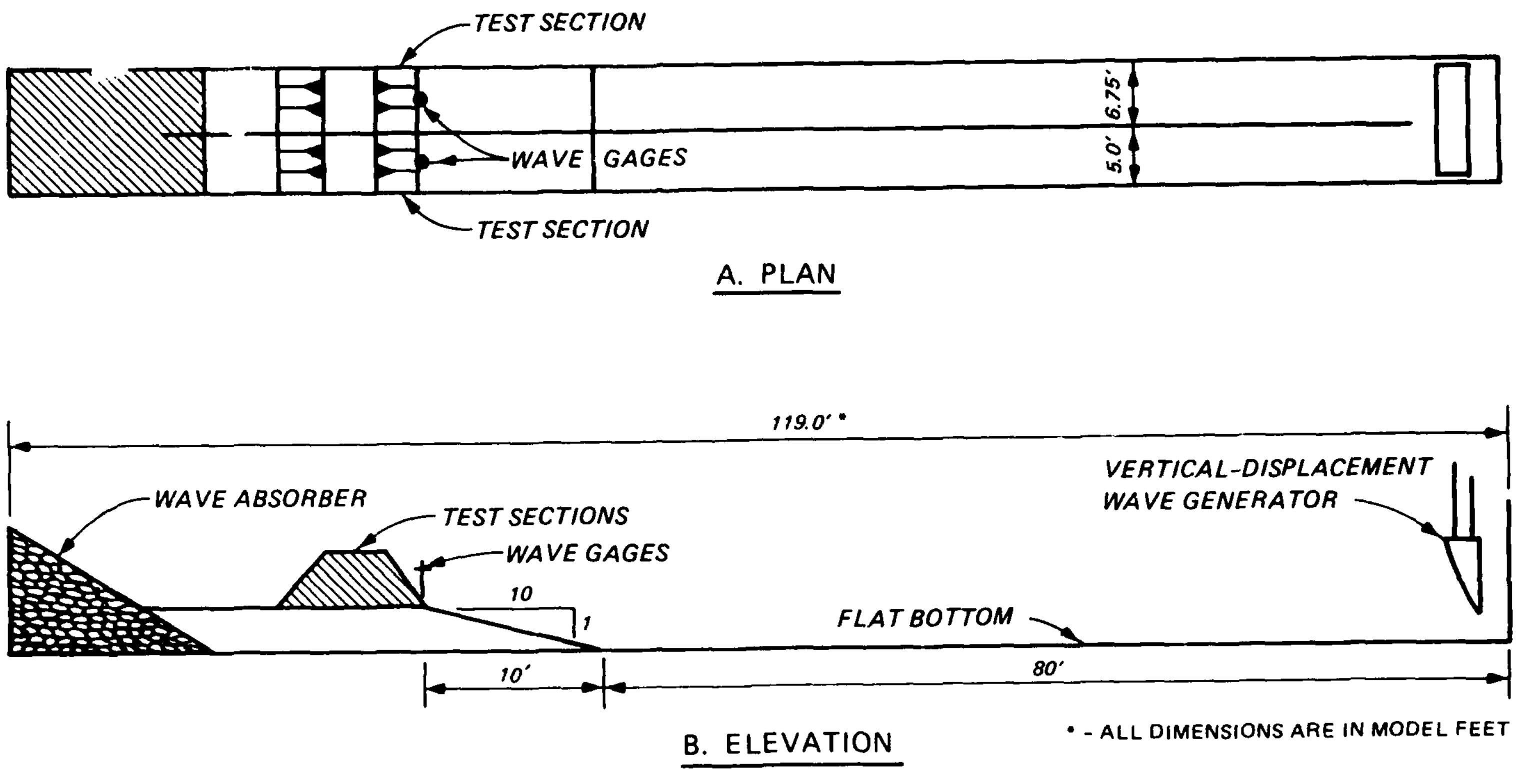


Figure 2. Test flume geometry and wave gage and test section locations for two-dimensional toe berm stone tests

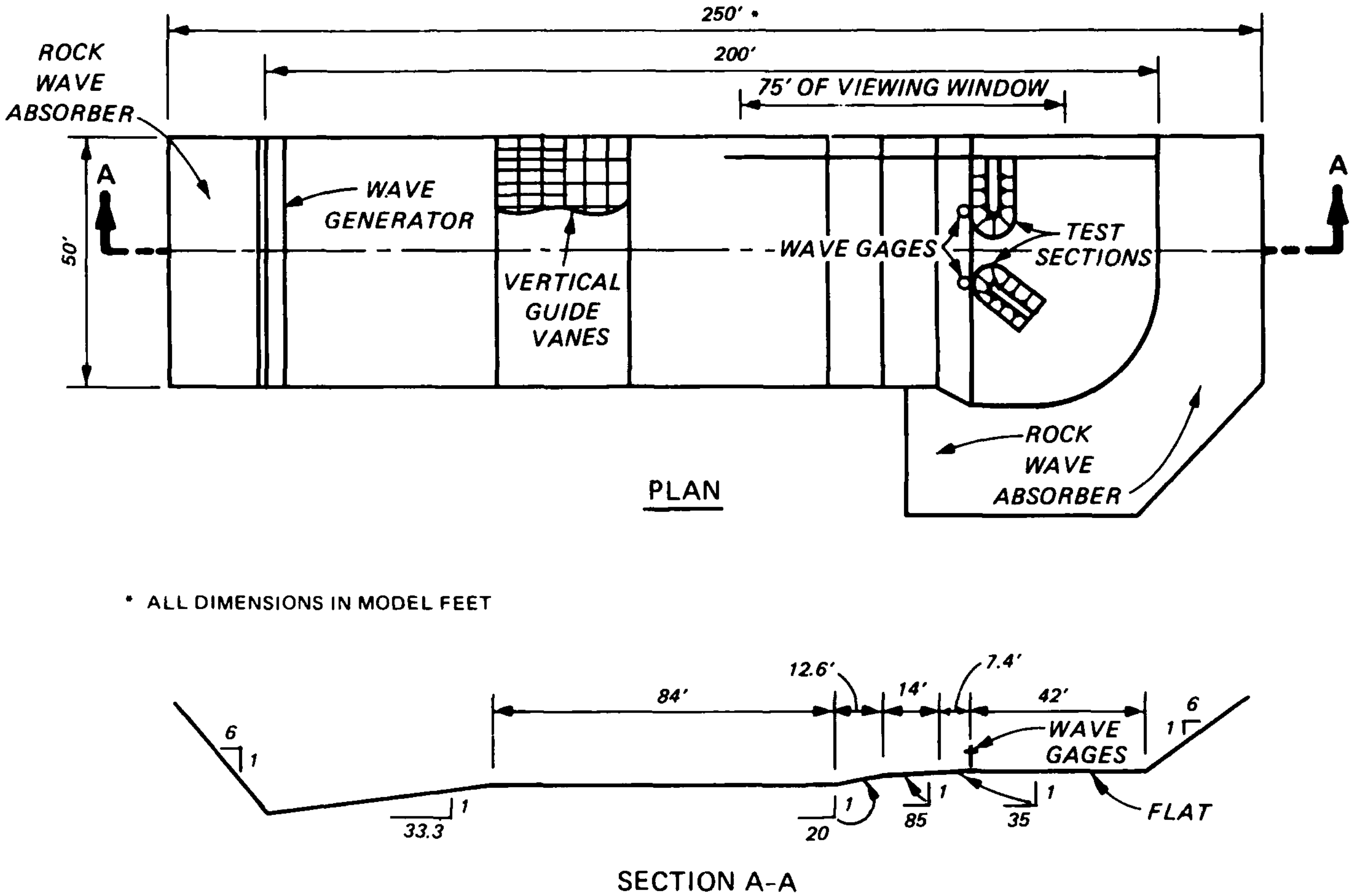


Figure 3. L-shaped wave basin geometry and wave gage and test section locations for three-dimensional toe berm stone tests

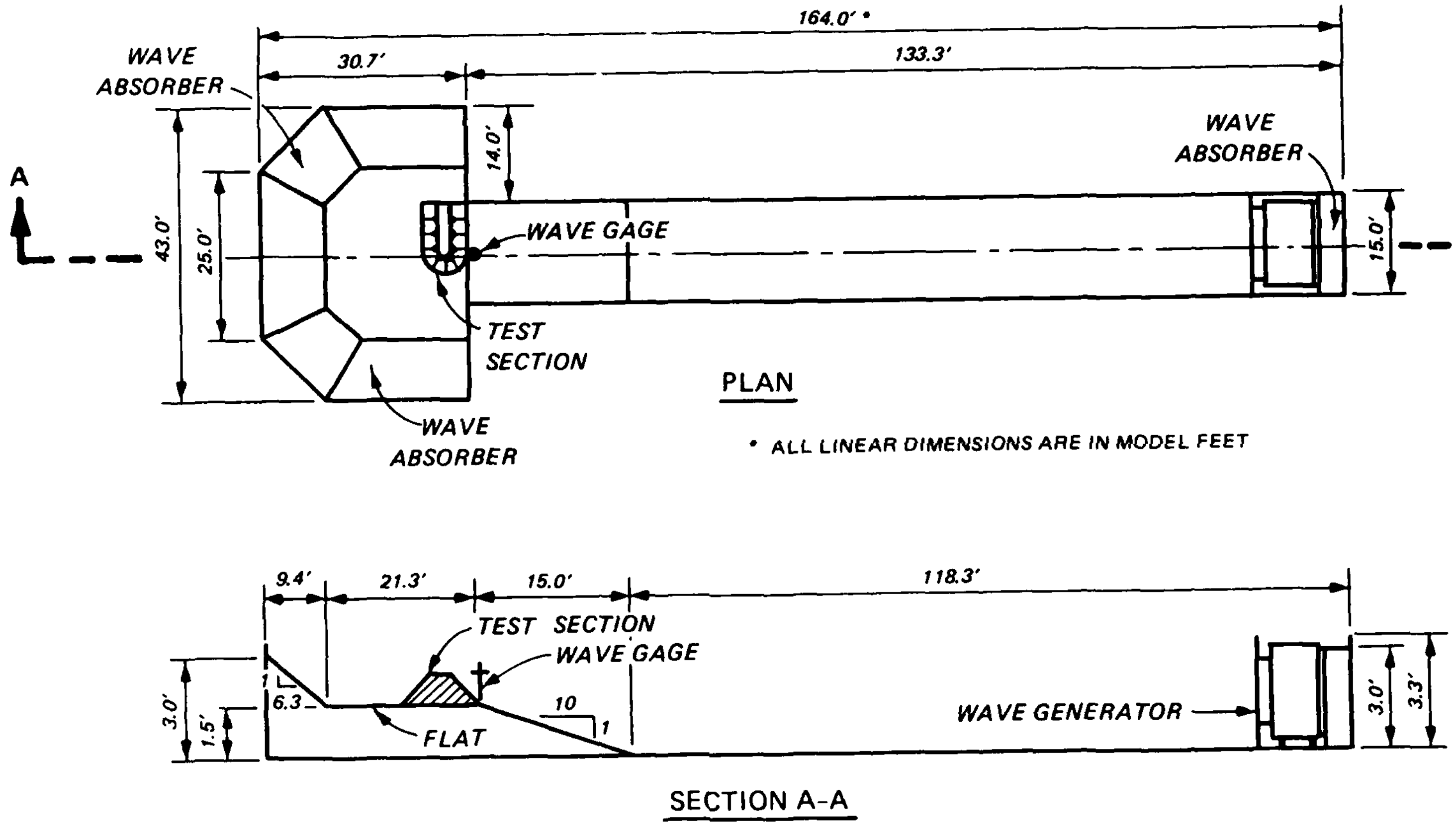


Figure 4. T-shaped wave basin geometry and wave gage and test section locations for three-dimensional toe berm stone tests

structure would be situated (Figures 1-4). Unless otherwise mentioned, all wave heights H and water depths d_s defined herein are those measured at the sea-side toe of the test sections.

Test Sections

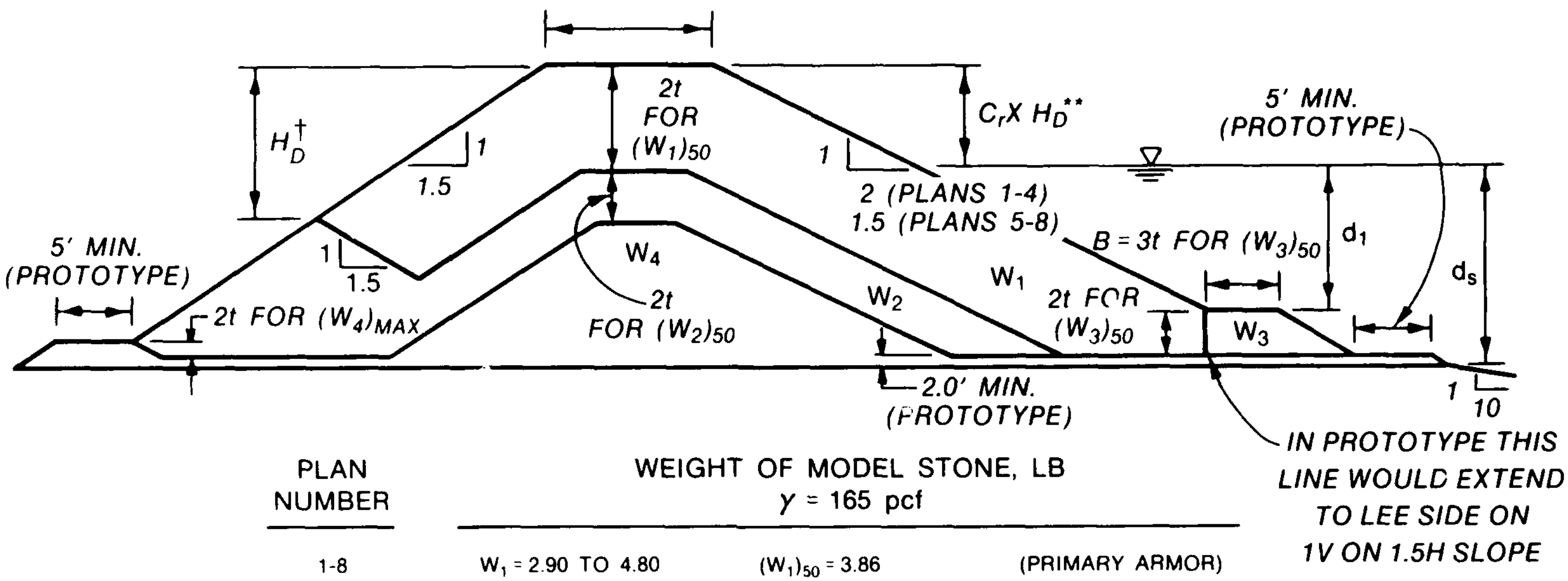
9. All rubble-mound test sections were constructed to reproduce as closely as possible typical results of full-scale construction. The core material was dumped and compacted to simulate natural consolidation that would occur during construction due to wave action. Underlayer stone was dumped and smoothed to grade. Primary armor units--in this case either stone, dolosse, or tribars--were added to the structure one at a time. The one-layer tribar armor was constructed using uniform placement. (All vertical tribar legs were placed normal to structure slope.) Except for the dolos toe where the first two rows were specially placed to accomplish maximum interlocking, the stone and dolos armor layers were constructed using random placement. (Random placement means that the units were randomly selected from a stock pile and were placed without any special orientation or interlocking with one another.) Berm armor stones, with weights equal to or greater than one-quarter the weight of the primary armor stone needed for stability, were added using random placement. Berm stones with smaller weights were dumped and smoothed to grade. Toe buttressing stones were placed after the tribar armor and with a conscious effort of maximizing contact between the buttressing stone and the bottom row of tribars.

2-D toe berm stone tests

10. Eight toe berm armor stone plans were tested. All cross sections tested were conventional three-layered stone structures designed for one-sided breaking wave attack with no solid water overtopping (SPM 1984). Plan details for model test sections are shown in Figure 5. Side and sea-side views of a typical test section are shown in Photos 2 and 3.

2-D buttressing stone tests

11. In conjunction with the REMR research work unit "Use of Dissimilar Armor for Repair and Rehabilitation of Rubble-Mound Coastal Structures," limited tests were conducted of toe buttressing stone fronting one-layer, uniformly placed tribar overlays on 1V on 1.5H slopes (Carver and Wright 1988) (Figure 6 and Photos 4 and 5). This type of structure exists on a section of



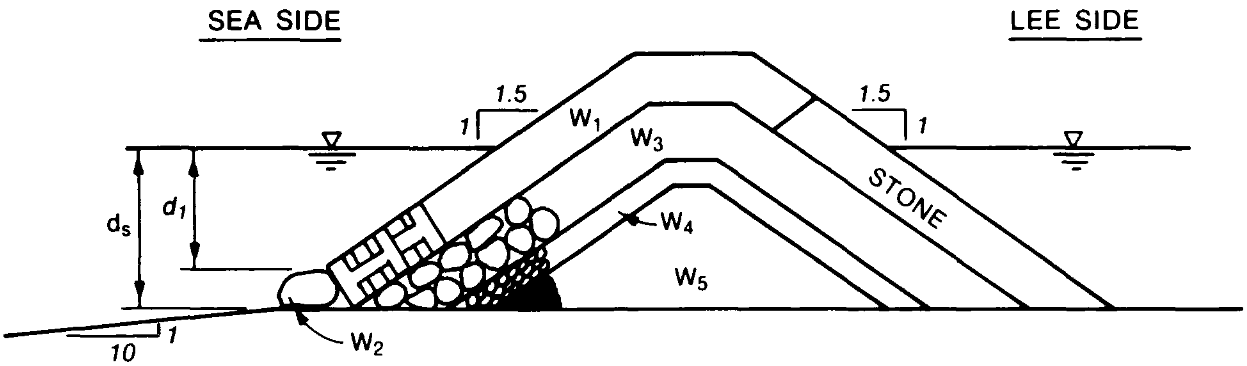
PLAN NUMBER	WEIGHT OF MODEL STONE, LB γ = 165 pcf		
1-8	W_1 = 2.90 TO 4.80	$(W_1)_{50}$ = 3.86	(PRIMARY ARMOR)
1-8	W_2 = 0.27 TO 0.50	$(W_2)_{50}$ = 0.39	(UNDERLAYER)
1&5	W_3 = 0.27 TO 0.50	$(W_3)_{50}$ = 0.38	(BERM ARMOR)
2&6	W_3 = 0.15 TO 0.27	$(W_3)_{50}$ = 0.21	(BERM ARMOR)
3&7	W_3 = 0.11 TO 0.20	$(W_3)_{50}$ = 0.15	(BERM ARMOR)
4&8	W_3 = 0.07 TO 0.13	$(W_3)_{50}$ = 0.10	(BERM ARMOR)
1-8	W_4 = 0.001 TO 0.019		(CORE & BEDDING)

* $t = k_\Delta (W/\gamma)^{1/3}$, WHERE k_Δ = 1.0 FOR ROUGH ANGULAR STONE AND $W = W_{50}$ UNLESS OTHERWISE NOTED.

** CREST HEIGHT SET AT $C_r \times H_D$ ABOVE MAXIMUM SWL, WHERE H_D IS DESIGN WAVE HEIGHT ASSOCIATED WITH MAXIMUM SWL AND C_r IS MAXIMUM RUNUP COEFFICIENT FOR MAXIMUM SWL DESIGN CONDITIONS. C_r = 0.85 FOR PLANS 1-4 AND C_r = 1.0 FOR PLANS 5-8.

† THIS DIMENSION ARBITRARILY SET EQUAL TO DESIGN WAVE HEIGHT (H_D) ASSOCIATED WITH MAXIMUM SWL.

Figure 5. Details of 2-D toe berm stone test sections



MODEL MATERIAL CHARACTERISTICS

W_1 = 0.627 LB TRIBARS @ 140.4 PCF (ONE LAYER, UNIFORM PLACEMENT)
W_2 = 0.860 LB STONE @ 165 PCF (SINGLE ROW OF BUTTRESSING STONE)
W_3 = 0.55 LB STONE @ 165 PCF (TWO LAYERS, RANDOM PLACEMENT)
W_4 = 0.055 LB STONE @ 165 PCF (TWO LAYERS, DUMPED)
W_5 = 0.003 LB STONE @ 165 PCF (CORE)

Figure 6. Details of 2-D toe buttressing stone test section

the Hilo Breakwater in the US Army Engineer Division, Pacific Ocean (Figure 7).

3-D toe berm stone tests

12. A two-layer toe berm design similar to the one used in the 2-D tests was incorporated into five different 3-D test sections (Figures 8 and 9). Unlike the 2-D design, where the berm crest was three stones long for

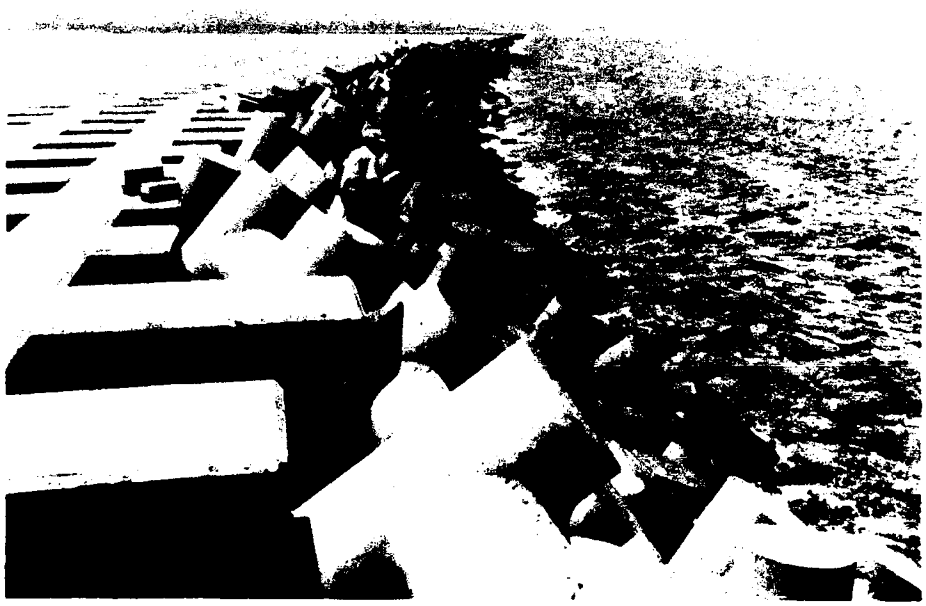
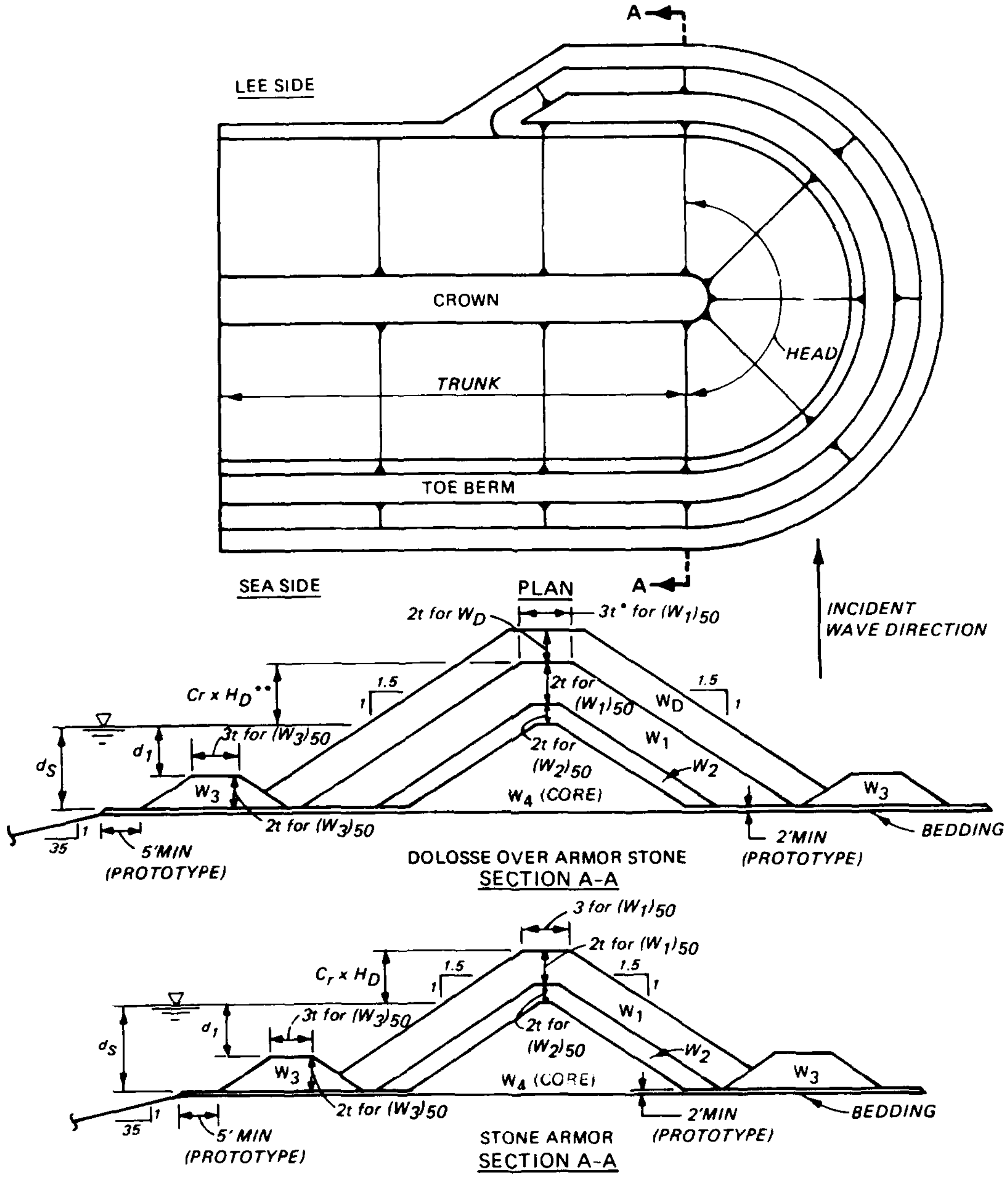


Figure 7. Tribar with toe buttressing stone and concrete rib cap repair section of Hilo Breakwater, Hilo Harbor, Hawaii

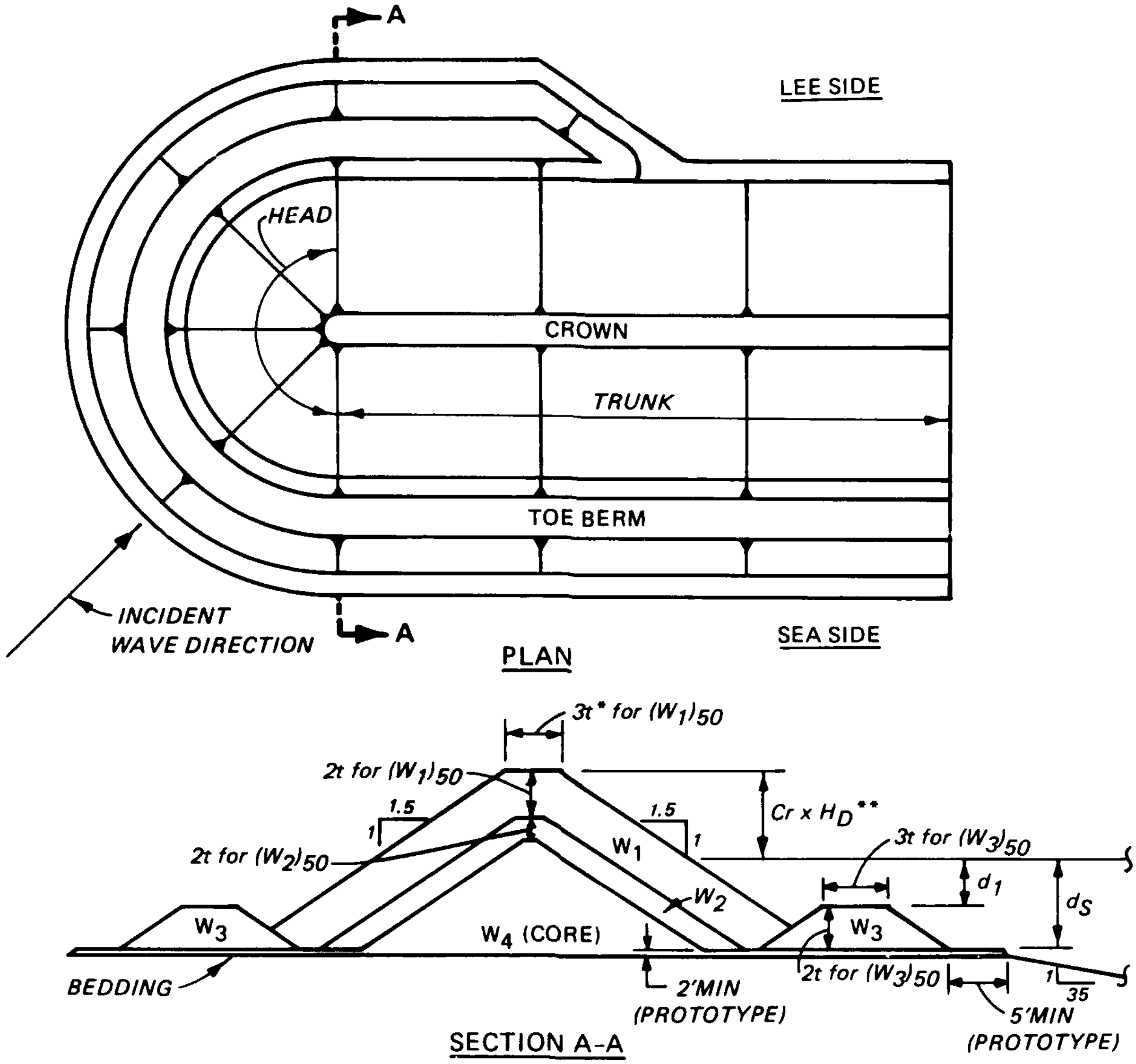


W_D = DOLOS OVERLAY (W_D = 0.276 LB)
W_1 = PRIMARY ARMOR ($(W_1)_{50}$ = 0.38 OR 0.55 LB STONE)
W_2 = UNDERLAYER ($(W_2)_{50}$ = 1/10 $(W_1)_{50}$
W_3 = TOE BERM STONE ($(W_3)_{50}$ RANGED FROM 0.55 TO 0.055 LB; WEIGHT VARIED WITH d_S AND H_D)
W_4 = CORE AND BEDDING STONE (W_4 = $W_1/200$ TO $W_1/4000$

* $t = K_\Delta (W/\gamma)^{1/3}$, WHERE K_Δ = 1.0 FOR ROUGH ANGULAR STONE AND 0.94 FOR DOLOSSE AND $W = W_{50}$

** CREST HEIGHT SET AT $C_r X H_D$ ABOVE MAXIMUM SWL, WHERE H_D IS DESIGN WAVE HEIGHT ASSOCIATED WITH MAXIMUM SWL AND C_r IS MAXIMUM RUNUP COEFFICIENT FOR MAXIMUM SWL DESIGN CONDITIONS. C_r = 0.80 FOR DOLOSSE AND C_r = 1.0 FOR STONE ARMOR

Figure 8. Details of 3-D toe berm stone test sections exposed to 90-deg wave attack



W_1 = PRIMARY ARMOR ($(W_1)_{50}$ = (0.38 OR 0.55 LB STONE OR 0.276 LB DOLOS,VARIED FROM PLAN TO PLAN)

W_2 = UNDERLAYER STONE ($(W_2)_{50}$ = 1/10$(W_1)_{50}$ FOR STONE ARMOR AND $(W_2)_{50}$ = 1/5$(W_1)_{50}$ FOR DOLOS ARMOR)

W_3 = TOE BERM STONE $(W_3)_{50}$ RANGED FROM 0.55 TO 0.055 LB; (WEIGHT VARIED WITH d_S AND H_D)

W_4 = CORE AND BEDDING STONE (W_4 = W_1/200 TO W_1/4000)

* $t = K_\Delta (W/\gamma)^{1/3}$, WHERE K_Δ = 1.0 FOR ROUGH ANGULAR STONE AND 0.94 FOR DOLOSSE AND $W = W_{50}$

** CREST HEIGHT SET AT $C_r X H_D$ ABOVE MAXIMUM SWL, WHERE H_D IS DESIGN WAVE HEIGHT ASSOCIATED WITH MAXIMUM SWL AND C_r IS MAXIMUM RUNUP COEFFICIENT FOR MAXIMUM SWL DESIGN CONDITIONS. C_r = 0.80 FOR DOLOSSE AND C_r = 1.0 FOR STONE ARMOR

Figure 9. Details of 3-D toe berm stone test sections exposed to oblique wave attack

each berm stone weight tested, the 3-D berm crest kept a fixed length of approximately 0.4 ft in the model. This difference was the result of testing several berm armor stone weights in various areas on a given test section. The remainder of the 3-D test sections consisted of a head and trunk section of a typical multilayered, nonovertopping design armored with stone or dolosse. In one instance, the test section represented an old stone structure that had been overlaid with dolos armor. Photographs 6 and 7 show two examples of the 3-D test sections.

Test Conditions

2-D and 3-D toe berm stone tests

13. Prototype toe berm armor stones are exposed to various combinations of wave conditions and water depths. The weight of individual stones required for wave stability will vary greatly with incident conditions. A wide range of toe berm stone weights (from the maximum weight capable of being moved in the test facility to the minimum weight that can be tested outside of stability scale effects (Hudson 1975)), were tested to determine the stability response for toe berm stones exposed to breaking wave conditions over a wide range of water depths for wave orthogonals approaching both normal (90-deg angle to structure crest) and oblique (45-deg angle to structure crest) to the toe berms. The foreslopes used in the 2-D tests in the 5- and 6.75-ft wave flumes (1V on 10H) and the 3-D tests in the T-shaped wave basin (1V on 10H) were steeper than the foreslope in the L-shaped wave basin (1V on 35H). This steepness resulted in some difference in severity of incident wave conditions, with the steeper foreslope producing more severe plunging waves than those produced by the milder slope. Thus, the data derived from these tests cover steep and intermediate foreslopes.

General design guidance

14. Specific model test conditions can be nondimensionalized for use in developing general design guidance. Relative water depth at the toe d_s/L_s* , relative water depth at top of berm d_1/L_1 , relative wave height at the toe H_D/d_s , wave steepness at the toe H_D/L_s , relative berm depth d_1/d_s , and relative berm length B/L_1 , were thought to be major parameters influencing

* Symbols and abbreviations are listed in the Notation (Appendix A).

toe berm armor stability. Some of these parameters are discussed in Brebner and Donnelly (1962) and Tanimoto, Yagyu, and Goda (1982). Table 1 lists the ranges for each of these parameters that was capable of being, but was not necessarily, addressed in each test facility used in this test series.

15. Reflective properties of a structure relative to incident wave conditions also should have a direct impact on toe berm stability. Unlike Brebner and Donnelly (1962) and Tanimoto, Yagyu and Goda (1982) who examined toe berms fronting highly reflective vertical structures, these tests addressed toe berms fronting less reflective rubble-mound structures. This decrease in reflectivity should lead to differences in berm stone stability from that determined by Brebner and Donnelly (1962) and Tanimoto, Yagyu, and Goda (1982).

16. Other parameters that can influence berm armor stability are stone shape k_Δ, unit weight γ_r, gradation and porosity p, stone placement techniques, and angle relative to horizontal (slope) on which the berm stone is placed. For the berm stone designs developed and recommended herein, all these parameters were held constant (i.e., stone with rough angular shape ($k_\Delta = 1.0$), unit weight of 165 pcf, berm stone weight gradation of ±30 percent of W_{50}, and in-place porosity of approximately 37 percent for randomly placed berm stone on a flat slope). If a proposed design deviates greatly from these, some difference in berm armor stability response should be expected.

Buttressing stone tests

17. Using the existing flume calibration, the tribar and toe buttressing stone test section was subjected to wave and water level conditions that were very close to the design conditions for the tribar rehabilitation work done on the Hilo Breakwater (Markle (1986) and Sargent, Markle, and Grace (1988)) as well as some additional conditions. A tabulation of tests is presented in Table 2.

PART III: TEST TYPES AND RESULTS

2-D Toe Berm Stone Tests

18. Twenty-one tests were conducted. A test consisted of exposing from two to four toe berm plans to a range of wave heights at one wave period and water depth combination. Thirteen tests were conducted using berm armor as specified in Figure 5 as Plan 1 and Plan 2, and eight tests were completed for Plans 5 and 6. Plans 3, 4, and 7 were subjected to five tests each, and Plan 8 was tested for four incident wave and water level conditions. Table 3 lists the test conditions, nondimensional parameters, design wave height and toe berm stone stability number N_s associated with all tests which showed acceptable toe berm stability (i.e., some stone movement occurred, which showed that the stone was not over designed but the amount of movement was minor and acceptable). Tests where the toe berm stone either did not move or exhibited excessive (i.e., unacceptable) movement could not be used to formulate design guidance; therefore, these tests are not listed. Stability number is defined as follows:

$$N_s = \left(\frac{\gamma_r}{W_{50}}\right)^{1/3} \frac{H_D}{\left(S_r - 1\right)} \tag{1}$$

where

γ_r = unit weight of berm stone, pcf

H_D = design wave height, ft

W_{50} = median weight of individual berm stone, lb

S_r = specific gravity of berm stone relative to the water in which the structure is situated, i.e., $S_r = \gamma_r/\gamma_w$

γ_w = unit weight of water in which structure is situated, pcf

By cubing both sides and rearranging, Equation 1 takes the following form which can be used to directly calculate median berm stone weight:

$$W_{50} = \frac{\gamma_r\left(H_D\right)^3}{N_s^3\left(S_r - 1\right)^3} \tag{2}$$

19. Plots of stability number N_s versus wave steepness at the toe

H_D/L_s , relative wave height at the toe H_D/d_s , relative berm length B/L_1 , relative water depth at the toe d_s/L_s , relative water depth at top of berm d_1/L_1 , and relative berm depth d_1/d_s are presented in Plates 1-3.

20. Stability number shows no significant trend with increasing values of H_D/L_s or H_D/d_s (Plate 1). For the range of conditions and berm armor stone weights presented herein, wave conditions representative of nonbreaking waves at the toe did not cause berm stone damage. Hence, the guidance developed from this test series is strictly limited to breaking wave design conditions. From Plate 1 it is seen that all but one data point represented N_s values for H_D/d_s greater than 0.7. A similar indication is given by the narrow band of high H_D/L_s values (Plate 1) typical of breaking waves. The lack of a strong trend in N_s with increasing values of wave steepness and relative wave height is possibly due to the fact that all reported test conditions are breaking waves. It is very likely that an increase in stability number would be realized for relative wave height and wave steepness values associated with nonbreaking waves.

21. The range of relative berm lengths tested was rather narrow. (For consistency with Tanimoto, Yagyu, and Goda (1982), what is commonly referred to by many individuals as berm width is being referred to herein as berm length and is defined as a horizontal distance measured across the berm crest and normal to the structure crest.) For all 2-D tests, the berm length B was equal to 3t , which defines the length of three armor stones set side by side (Figure 5). Tanimoto, Yagyu, and Goda (1982) showed that berm length relative to incident wave length (relative berm length B/L_1) to be an important parameter for stability of berm stone fronting impervious vertical walls. A fixed crest length of approximately 0.4 ft was used in the 3-D tests, while the 2-D tests used berm crests which were three stones long, resulting in a rather narrow range of tested B/L_1 values. For this range, no significant correlation between stability number and relative berm length was noted (Plate 2). This lack of trend, as compared to the one developed by Tanimoto, Yagyu, and Goda (1982), is possibly due to the lower reflectivity of rubble as compared to impermeable vertical structures and to the shortness of the toe berm lengths tested relative to incident wave length.

22. The stability number shows a general trend to increase with increasing values of d_s/L_s and d_1/L_1 (Plates 2 and 3, respectively). This phenomenon follows the logic that the longer the wave period the deeper the

effects of the wave are felt, resulting in decreasing berm stone stability.

23. The stability number exhibits a well defined trend of increasing with increasing values of d_1/d_s (Plate 3). This is an expected trend because as the water depth over the berm decreases the berm stone becomes more exposed to incident wave energy requiring larger stone weights to ensure wave stability. This is the same trend shown by Brebner and Donnelly (1962).

3-D Toe Berm Stone Tests

24. Twenty-seven tests were conducted using four different head and trunk designs exposed to 90-deg and oblique wave attack in the L-shaped wave basin. Ten tests were conducted on one head and trunk design constructed in the T-shaped wave basin. As mentioned earlier, the slope fronting the test sections was steeper in the T-shaped wave basin than in the L-shaped test facility. The toe berm on each test section was constructed using stones of various weights in several areas (Figure 10). The selection of stone weights for testing was based on both incident wave and water level conditions and placement location on the structure. For example, two to three larger stone sizes might be used on the trunk which is exposed to 90-deg wave attack, while berm areas on the head, which experienced less severe wave conditions, were constructed with several smaller stone sizes. When a test section built in this manner was exposed to one fixed wave and water level, it was probable that some stone sizes would be large enough that no movement or in-place rocking would be observed (oversized for the test condition), while areas with smaller stone would exhibit large amounts of displacement (undersized for the test condition); and an intermediate stone size would sustain little or no displacement but would show some minor movement (correct stone size for the test condition). By conducting tests in this manner, design data could be obtained from a larger percentage of tests than would have been possible if each test section were constructed with only one weight of berm stone. Tables 4-6 list test conditions, nondimensional parameters, design wave height and toe berm stone N_s associated with trunk tests in the L-shaped wave basin, head tests in the L-shaped wave basin, and trunk and head tests in the T-shaped wave basin, respectively, from which toe berm design data were gathered.

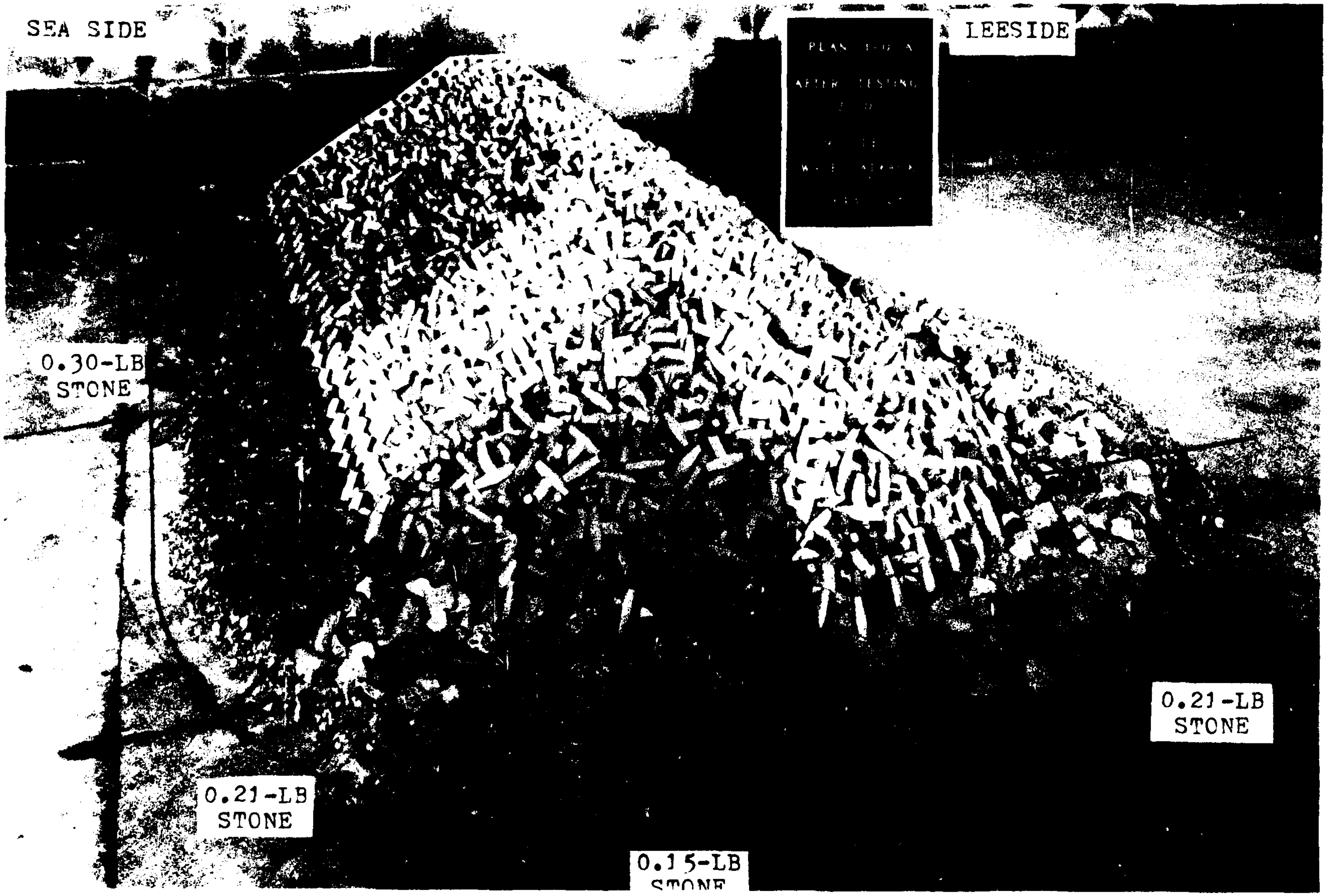


Figure 10. Example of three-dimensional toe berm stone test section constructed with various weights of toe berm stone

25. Plots of N_s , H_D/L_s , H_D/d_s , B/L_1 , d_s/L_s , d_1/L_1 , and d_1/d_s for all of the 3-D tests are presented in Plates 4-21. The data plots are separated based on whether they are for trunks or heads, 90-deg or oblique wave attack, and the type of facility the tests were conducted in. The last test was done to see if the steeper foreslope used in the T-shaped basin would affect the data trends differently from the milder slope in the L-shaped test facility.

26. The wealth of data points per plot is less for the 3-D tests than for the 2-D tests addressed earlier; but where sufficient data exist, the same trends or lack of data trends as exhibited by the 2-D data are seen in the 3-D plotted data. Stability number shows no significant trend with increasing values of wave steepness and relative wave height, but these data do show that all tests apply to breaking wave test conditions. Stability numbers versus relative berm length shows no obvious trend, while plots of N_s versus d_s/L_s and d_1/L_1 show a trend for stability to increase with increasing values of relative water depth. A strong trend is shown when N_s is plotted against relative berm depth d_1/d_s . Stability number shows a definite trend of increasing with increasing relative berm depth.

2-D and 3-D Toe Berm Stone Tests

27. Figures 11 and 12 present N_s plotted against relative water depth at the toe and relative berm depth, respectively, for all tests. The stability number shows a trend with both parameters, but the trend with relative berm depth appears to be stronger. Contours of equal stability number were incorporated into a plot of relative berm depth versus relative water depth at the toe (Figure 13). This plot reveals that within the range of conditions tested toe berm armor stability shows only a minor overall dependency on variations in wave length (i.e., wave period) and that the major parameter in selecting a breaking wave stability number is relative berm depth. A plot of stability number cubed versus d_1/d_s for all tests is presented in Figure 14. This plot allows the direct reading of N_s^3 for use in Equation 2 to calculate the required berm armor weight W_{50} . As explained in the plot legend, the data for various test categories are plotted using various symbols. For a given relative berm depth, there is no great difference in stability associated with differing angles of wave attack or location of the berm stone on

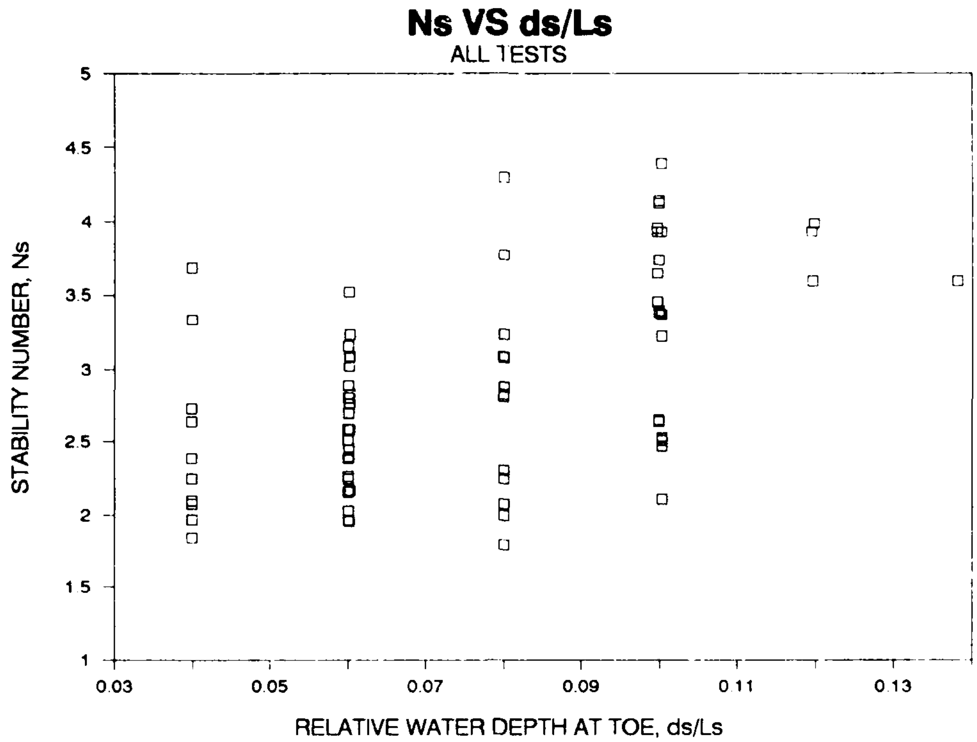


Figure 11. Stability number versus relative water depth at toe for toe berm stone tests which produced design data

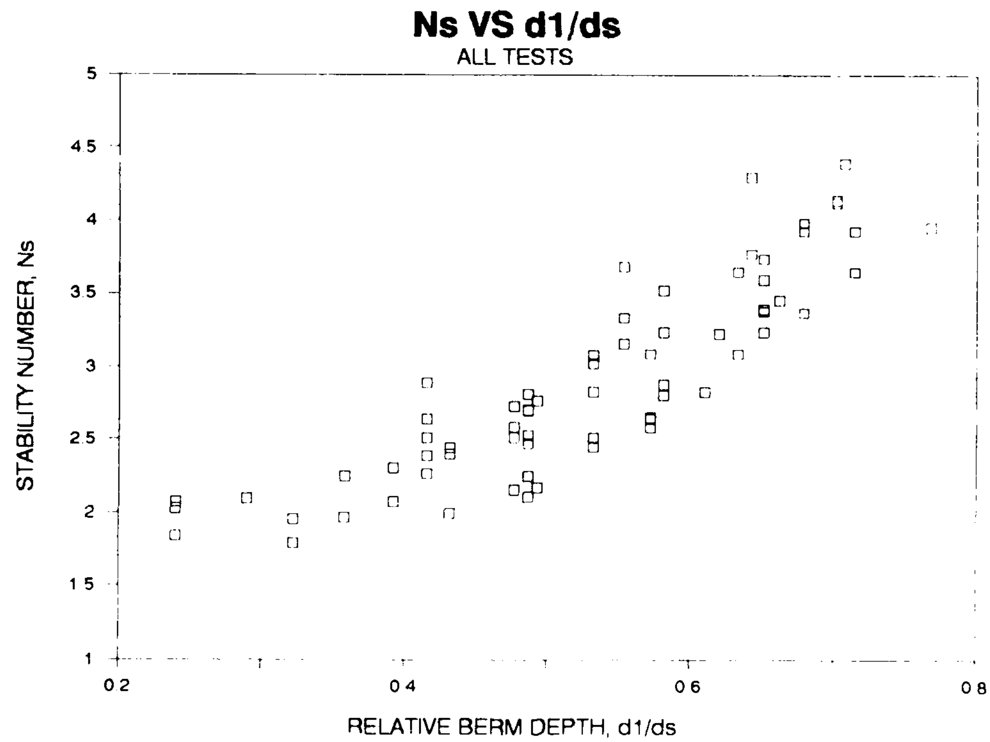


Figure 12. Stability number versus relative berm depth for all toe berm stone tests which produced design data

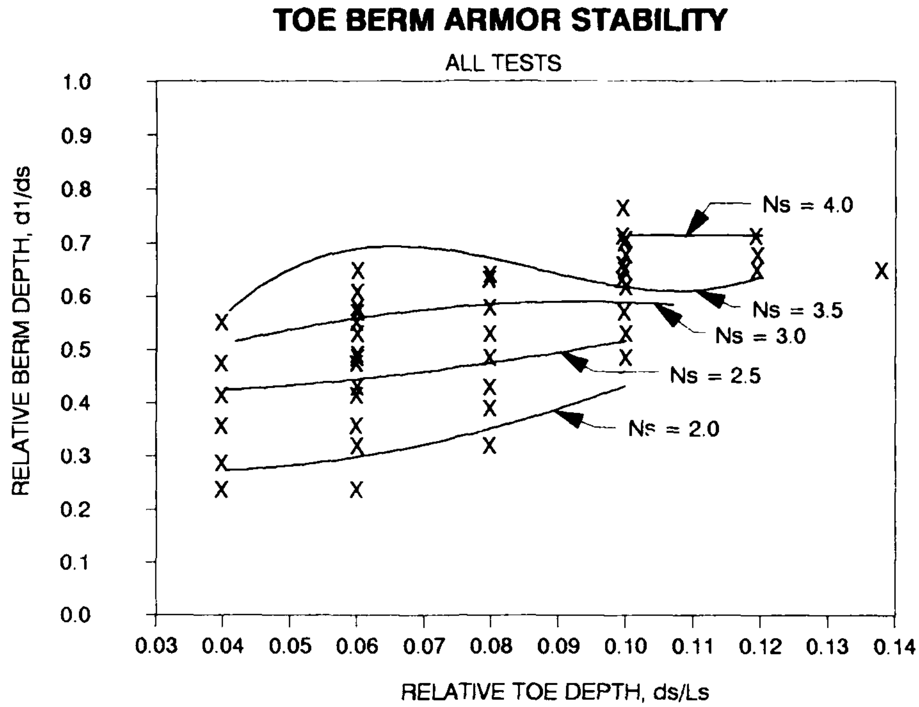


Figure 13. Contours of equal stability number for the ranges of relative berm depth and relative water depth at toe which produced design data for toe berms

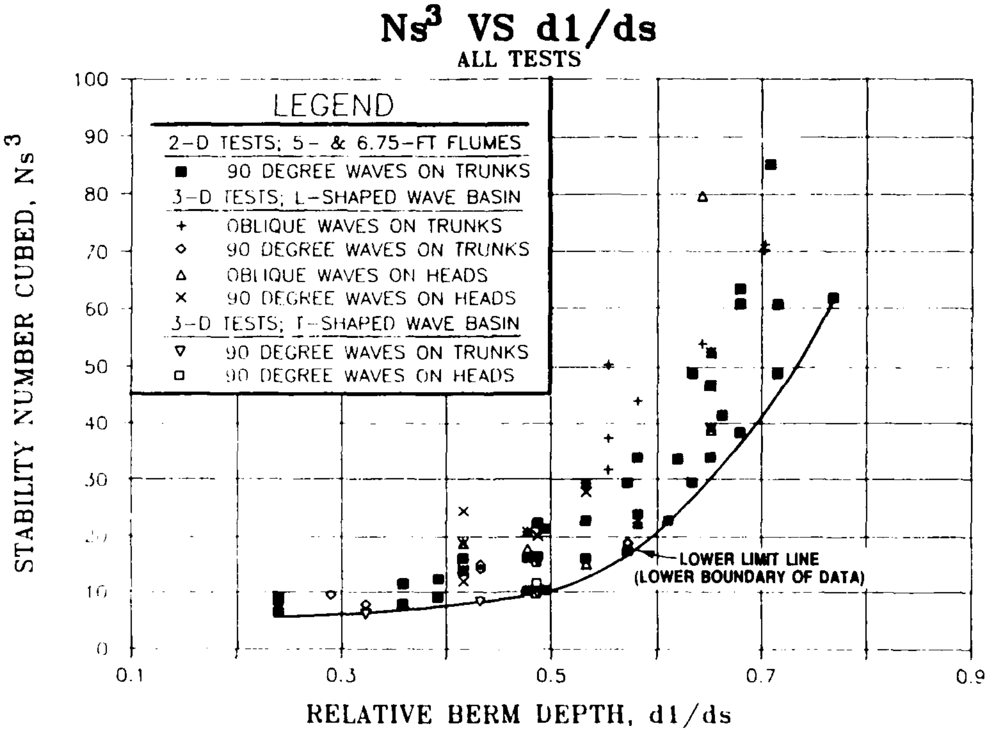


Figure 14. Stability number cubed versus relative berm depth for all toe berm stone tests which produced design data

the structure. Some general trends of higher stability on heads and for oblique wave attack can be seen, but the trends are not well defined. For this reason, a lower limit line has been incorporated into Figure 14. When designing for breaking waves and designs are not being verified and/or optimized with physical model tests, values of N_s^3 equal to or less than those defined by this line should be used for design. In addition, design for conditions outside the ranges of d_1/d_s and d_s/L_s tested in the model (as shown in Figures 11 and 12) should be carefully examined. This lower limit line also has been incorporated into a plot with Brebner's and Donnelly's (1962) data as presented in the SPM (1984) (Figure 15).

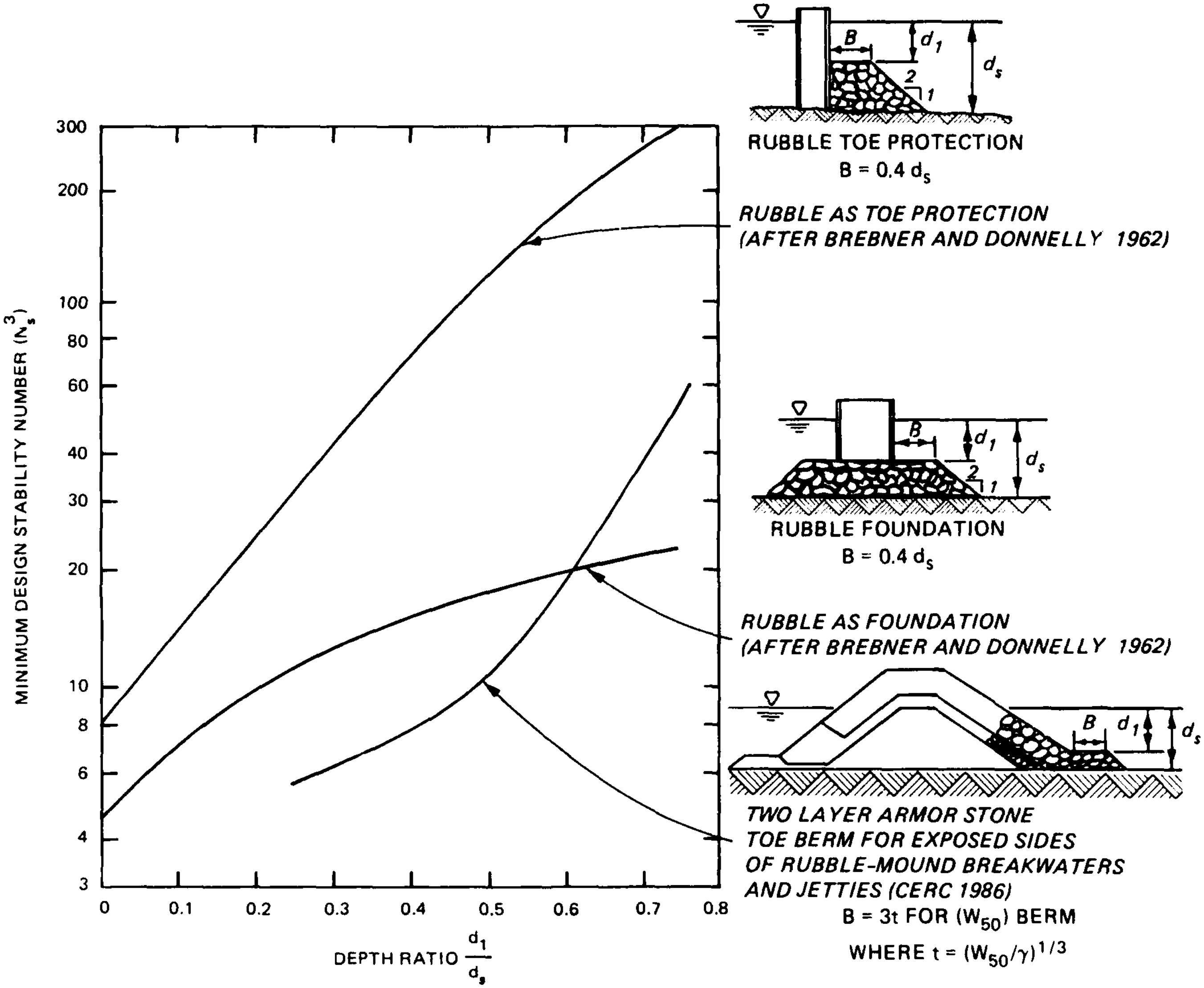
3-D Toe Berm Stone Tests Conducted With Spectral Waves

28. All test conditions and test results discussed and reported up to this point in the report have been relative to monochromatic test wave conditions. Near the completion of this study, a spectral wave generator was installed in the L-shaped wave basin; therefore, limited comparative spectral tests were conducted.

29. Two rubble-mound structure head and trunk plans were exposed to spectral wave conditions for both 90-deg and oblique wave attack. Joint North Sea Wave Project (JONSWAP) spectra with γ 3.3, slope parameters $\sigma_{low} = 0.07$ for f less than f_p (where f refers to frequency and f_p refers to peak spectral frequency), $\sigma_{high} = 0.09$ for f greater than f_p and the peak period and water depth combinations shown in Table 7 were selected for testing (see Figure 16 for a definition sketch). Goda and Suzuki's (1976) method was used to resolve incident and reflected spectra at the sea-side toe of the structures. The zeroth moment wave height H_{mo} is defined as follows:

$$H_{mo} = 4(E)^{1/2} \tag{3}$$

where E is a measure of total spectral energy and is equal to the area under the curve on a spectral energy density versus frequency plot. Both the measured H_{mo} and the theoretical maximum H_{mo} (Vincent 1984 and Hughes 1984) based on depth limitation, are presented in Table 7. These H_{mo} values gave similar toe berm stability to that observed during monochromatic wave tests conducted at the same wave period and water depth combinations. The H_{mo}



NOTE: N_S^3 VALUES FOR TOE BERMS FRONTING RUBBLE-MOUND STRUCTURES ARE FOR BREAKING WAVE DESIGN CONDITIONS.

Figure 15. Stability number cubed versus relative berm depth for toe berms fronting rubble-mound structures and rubble toes and foundations for impermeable vertical structures

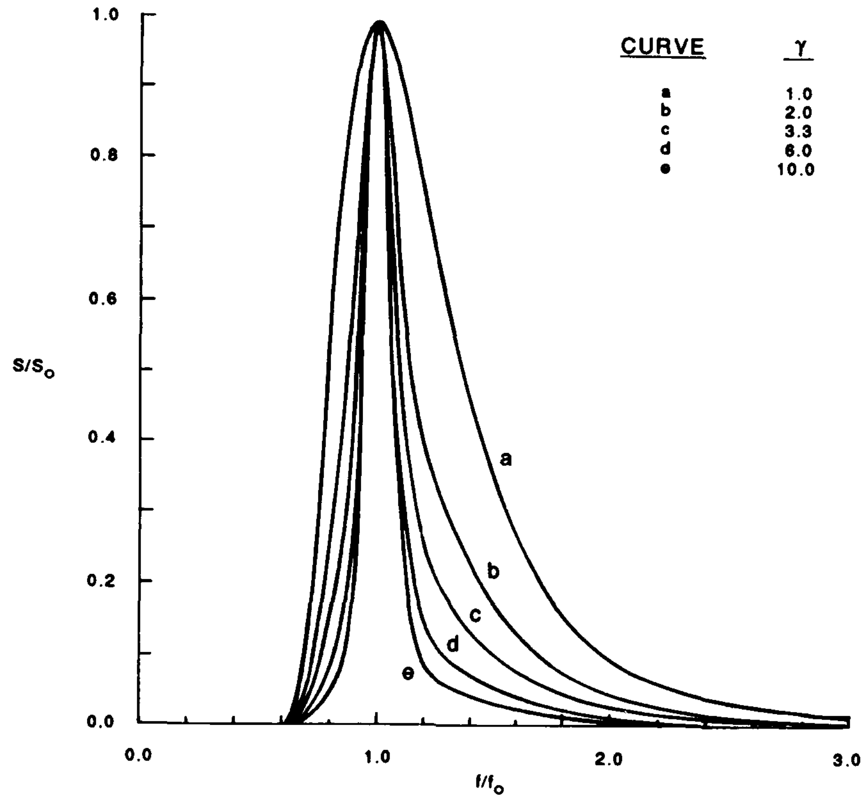


Figure 16. Five examples of JONSWAP spectra in dimensionless form (Case (a) is a Pierson-Moskowitz spectrum; Case (c) is the result of the JONSWAP experiment)

values were substituted for H_D in Equation 1, and respective stability numbers are presented in Table 7. These stability numbers were cubed and are plotted against relative berm depth (Figure 17). To make comparison easier, the stability data for all 2-D and 3-D monochromatic wave toe berm tests were added to the plot. The spectral data show the same trend, but the use of H_{mo} in Equation 1 results in lower stability numbers. The use of H_{mo} in Equation 1 does not show that the spectral wave conditions were more severe than the monochromatic breaking waves but instead points out that the magnitude of H_{mo} is smaller than the monochromatic breaking wave heights that were measured at the toe of the structures for the same incident wave period and water depth. Thus, when H_{mo} is substituted for H_D in Equation 1 the resulting stability number is smaller than those calculated using the breaking wave height.

30. The limited spectral toe berm stability tests described herein are by no means intended to represent extensive enough parameters from which

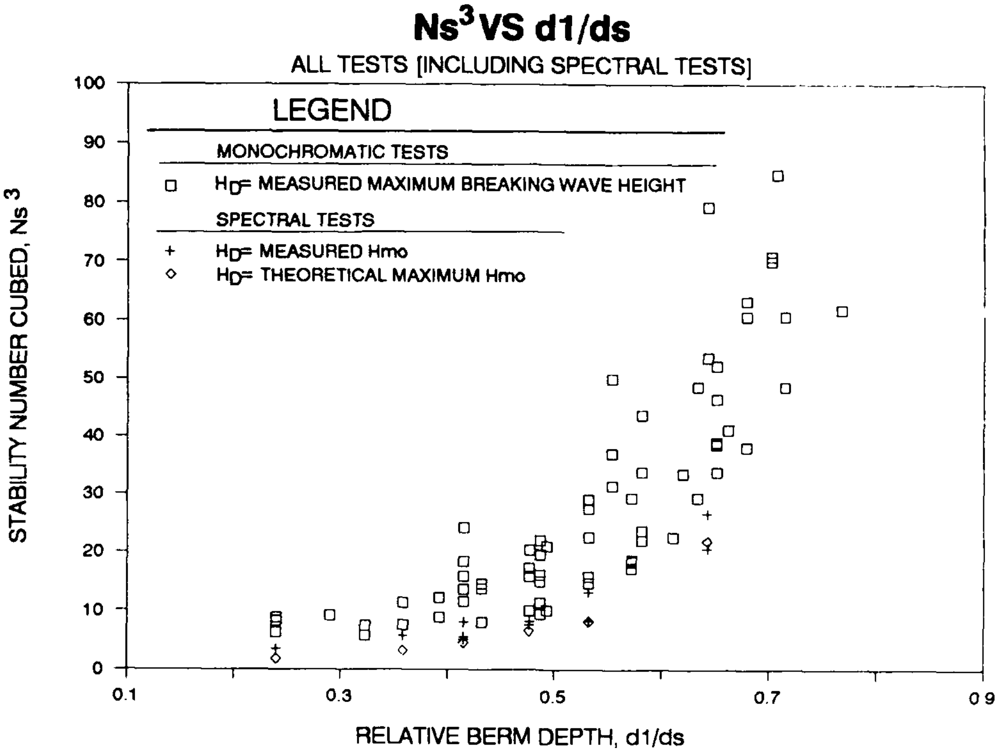


Figure 17. Stability number cubed versus relative berm depth for all toe berm stone tests including spectral wave tests

general spectral design guidance can be developed. The data show that if shallow-water spectral H_{mo} values were used in Equation 2 in conjunction with stability numbers associated with breaking wave height (Figure 14), the toe berm armor stone would likely be undersized. Thus, once spectral design conditions are known, an estimate of the maximum breaking wave height associated with the spectrum must be used in sizing the toe berm stone.

Buttressing Stone Tests

31. Results of the four stability tests of tribar overlays with toe buttressing stone are presented in Table 2. For the limited tests conducted, the stability of the toe buttressing stones seemed to be independent of d_s/L_s , d_1/d_s , and H_D/d_s . Average stability numbers for the tribars and toe buttressing stones were 2.2 and 1.6, respectively, confirming the US Army Engineer Division, Pacific Ocean, design decision that toe buttressing stones need to be approximately 1.3 times the weight of tribar needed for stability in a breaking wave environment.

PART IV: CONCLUSIONS

Toe Berm Stone Tests

32. Based on the 2-D and 3-D wave stability test conditions and test results reported herein for two-layer, randomly placed armor stone toe berms on breakwater and jetty trunks and heads with a length B equal to 5 ft or 3 widths of W_{50} stone (whichever is greater) and designed for breaking wave environments where wave crests are either parallel or oblique to the berm, it is concluded that:

a. The stability number N_s appears to be relatively insensitive to changes in wave steepness H_D/L_s and relative wave height H_D/d_s for the range of values tested.

b. For the narrow range of relative berm length B/L_1 tested, the stability number shows no well-defined trend.

c. The stability number shows a minor trend to increase with increasing values of d_s/L_s, which indicates a small dependency on wave length, i.e., wave period. The best defined trend is the one of increasing values of N_s with increasing values of relative berm depth d_1/d_s. The spread of the data which defines this trend appears to be a function of wave period, foreslope fronting structures, angle of wave attack, and whether the toe berm is on the head or the trunk. These secondary trends are minor relative to the trend with relative berm depth, and attempts to develop multiparameter functional relationships were less than satisfactory. Therefore, for general design purposes, unless site-specific model tests are conducted to justify higher values of N_s, stability number should be selected based on the lower limit curve presented in Figures 14 and 15, and the individual toe berm armor stone weights should range from a maximum $1.3\ W_{50}$ to a minimum of $0.7\ W_{50}$.

d. Insufficient spectral stability data are available to recommend general design guidance relative to spectral H_{mo} values. It is recommended that an estimate of the maximum breaking wave height associated with the selected design spectrum be used in Equation 2 when sizing toe berm armor stone.

Buttressing Stone Tests

33. Based on the limited 2-D wave stability test conditions and test results reported herein for toe buttressing stone fronting one-layer uniformly placed tribars, it is concluded that a stability number N_s equal to 1.5

should be used to design toe buttressing stone for a breaking wave environment.

PART V: DISCUSSION

34. Both the toe berm stone and buttressing stone addressed herein are required when designing for a high-energy wave environment. When either toe buttressing stone or toe berm stone is used on a structure being constructed on erodible bottom material, adequate thicknesses and gradations of filter or bedding layers need to be incorporated into the design to prevent the leaching of foundation material. Failure to prevent leaching could result in the ultimate failure of the entire structure.

35. During conduct of the 2-D toe berm stone tests, damage measurements were made by observation for a range of H/H_D . During some tests, the toe berm stone design wave would be reached prior to reaching the maximum wave that could be created in the test flume with the 1V on 10H foreslope and at the selected wave period and water depth. By extending the tests to conditions which exceeded the design level, general data on damage related to extreme wave heights (wave heights which exceed the design height) were obtained. These data are presented in Figures 18-20. Figure 18 presents percentage of berm armor stone showing in-place rocking as a function of H/H_D . The percentage of toe berm armor stone displaced from its original position is plotted against H/H_D in Figure 19. The percentages of berm armor stone rocking in place and displaced at a given value of H/H_D were summed for each test and are presented in Figure 20.

36. Although it is not recommended, it is understood that there are occasions when a designer is forced by economic constraints or other considerations to design for a lower wave environment and accept the damage and resulting maintenance costs that will occur due to damage accrued at larger wave conditions. Figures 18-20 have been included to provide some insight into what has become known as "designing for damage". The "upper limit damage line" in Figure 20 could be used for making rough predictions of possible damage that could occur for H/H_D range of 1.0 to 1.3. The upper limit damage line is essential due to the large scatter associated with data points over this range of H/H_D . An example of how to use this upper limit damage line is in the following paragraph

37. Maximum depth-limited breaking wave height that could occur at the structure toe equals 13 ft, but economics requires a design wave height of 10 ft be used. Thus, it is possible to get a wave condition at the site which

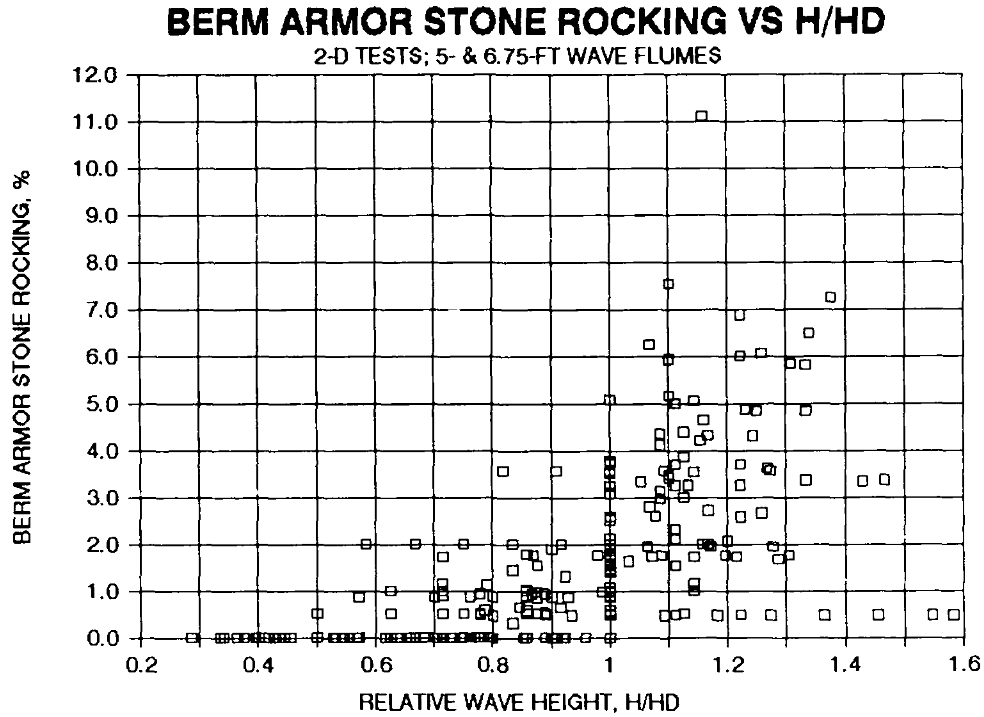


Figure 18. Percentage of toe berm armor stone which exhibited rocking versus relative wave height (2-D tests)

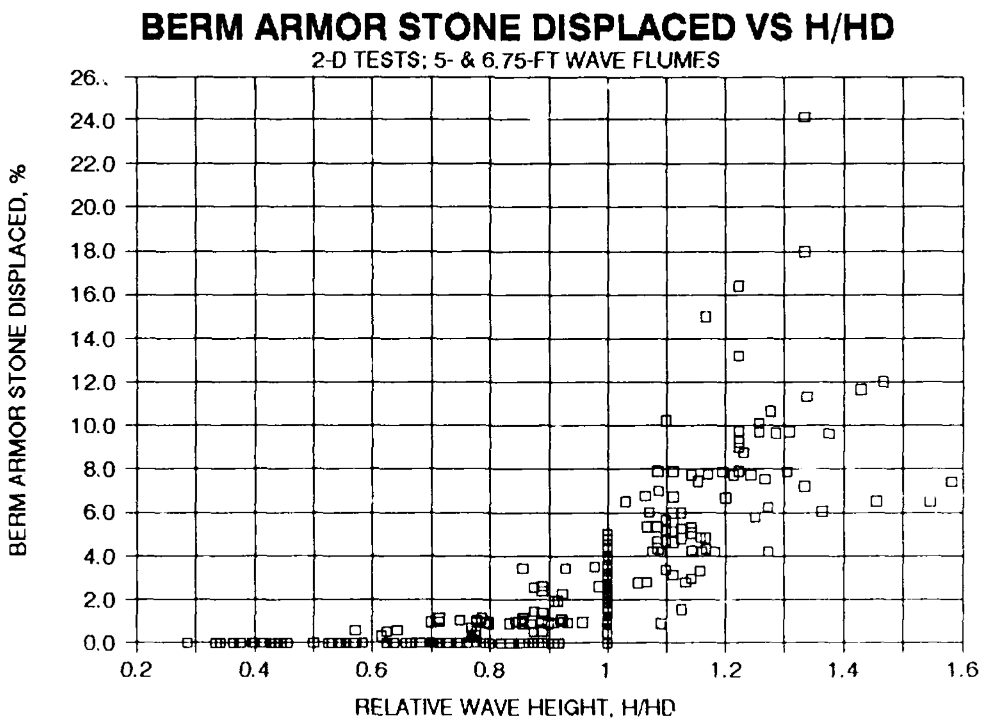


Figure 19. Percentage of toe berm armor stone displaced versus relative wave height (2-D tests)

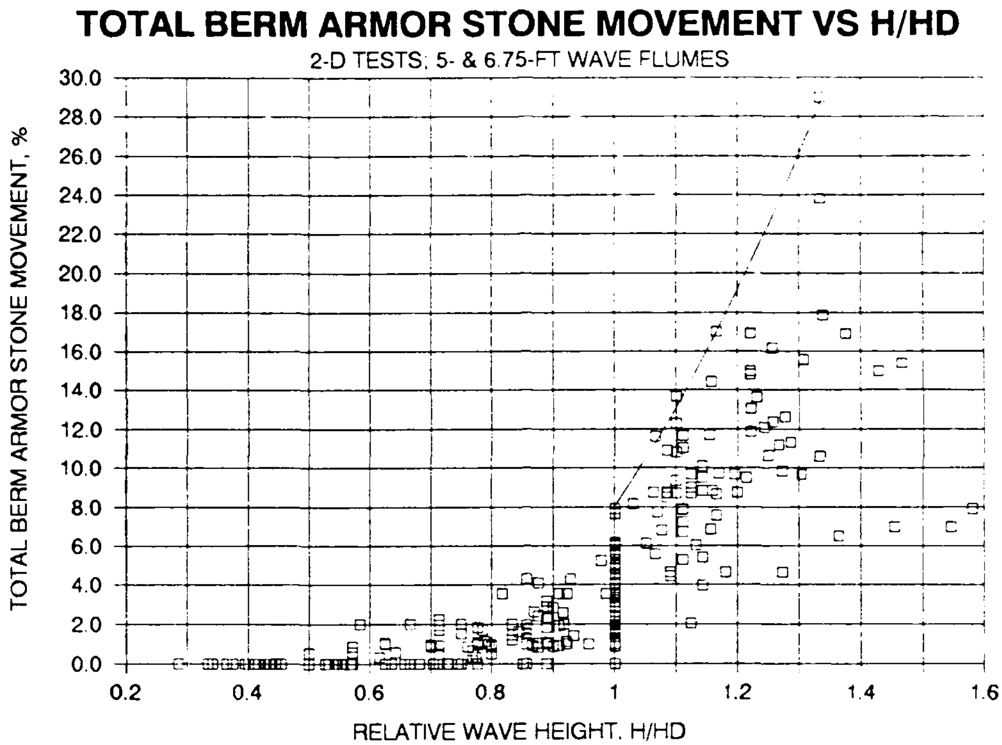


Figure 20. Total percentage of berm armor stone showing any type of movement (rocking or displacement) versus relative wave height (2-D tests)

exceeds the design wave height by 30 percent ($H/H_D = 1.3$). For $H/H_D = 1.3$, total berm armor movement (Figure 20) could be as high as 26 percent. This is an increase of 18 percent in possible damage over the maximum value of 8 percent associated with $H/H_D = 1.0$.

38. The stability numbers recommended herein for toe berms are for use when designing for breaking waves. For toe berms being designed for nonbreaking waves, the SPM (1984) recommends that the toe berm armor stone weight W_{50} be no less than one tenth the weight of the primary armor stone that would be needed for acceptable stability. It is recommended that this guidance continue to be followed; and for critical structures, the design adequacy should be checked through site-specific model tests.

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Table 1

Nondimensional Parameter Ranges Included in the Toe Berm Test Calibrations

Parameter Name	Parameter Notation	Range
Relative water depth at toe	d_s/L_s	0.04 - 0.14
Relative water depth at top of berm	d_1/L_1	0.019 - 0.12
Relative wave height at toe	H_D/d_s	0.22 - 1.15
Wave steepness at toe	H_D/L_s	0.02 - 0.12
Relative berm depth	d_1/d_s	0.24 - 0.77
Relative berm length	B/L_1	0.03 - 0.13

Table 2

Test Conditions and Test Results of Toe Buttressing Stone Tests

Test	Water Depth at Toe (d_s) ft	Wave Period sec	Design Wave Height (H_D) ft	Relative Water Depth at Toe (d_s/L_s)	Relative Wave Height at Toe (H_D/d_s)	Relative Buttressing Stone Depth (d_1/d_s)	Stability Number (N_s) Tribars	Stability Number (N_s) Buttressing Stone
1	0.45	2.02	0.46	0.06	1.02	0.62	2.23	1.61
2	0.55	1.39	0.45	0.10	0.82	0.68	2.19	1.58
3	0.60	1.24	0.45	0.12	0.75	0.71	2.19	1.58
4	0.65	1.13	0.46	0.14	0.71	0.73	2.23	1.61

Table 3

Test Conditions and Test Results for Toe Berm Stone Tests

TEST	PLAN	WATER DEPTH AT TOE [ds] FT	WAVE PERIOD [T] SEC	DESIGN WAVE HEIGHT [HD] FT	RELATIVE WATER DEPTH AT TOE [ds/Ls]	RELATIVE WATER DEPTH AT TOP OF BERM [d1/L1]	RELATIVE WAVE HEIGHT AT TOE [HD/ds]	WAVE STEEPNESS AT TOE [HD/Ls]	RELATIVE BERM DEPTH [d1/ds]	RELATIVE BERM LENGTH [B/L1]	STABILITY NUMBER [Ns]
						2-D Tests : 5- and 6.75-Ft Wave Flumes					
						90 Degree Wave Attack on Structure Trunks					
1	1	0.900	1.780	0.750	0.100	0.079	0.833	0.083	0.662	0.053	3.454
1	2	0.900	1.780	0.650	0.100	0.083	0.722	0.072	0.715	0.042	3.647
2	2	0.900	1.520	0.700	0.119	0.099	0.778	0.093	0.715	0.050	3.928
4	1	0.800	1.670	0.700	0.100	0.077	0.875	0.088	0.620	0.062	3.223
4	2	0.800	1.670	0.600	0.100	0.081	0.750	0.075	0.679	0.048	3.367
5	2	0.800	1.430	0.710	0.120	0.096	0.888	0.106	0.679	0.057	3.984
7	2	0.700	1.920	0.550	0.080	0.063	0.786	0.063	0.633	0.046	3.086
9	2	0.700	1.570	0.650	0.100	0.078	0.929	0.093	0.633	0.057	3.647
10	1	0.400	2.820	0.450	0.040	0.019	1.125	0.045	0.240	0.080	2.072
10	2	0.400	2.820	0.350	0.040	0.024	0.875	0.035	0.358	0.054	1.964
11	1	0.400	1.900	0.440	0.060	0.029	1.100	0.066	0.240	0.119	2.026
11	2	0.400	1.900	0.400	0.060	0.035	1.000	0.060	0.358	0.080	2.245
11	3	0.400	1.900	0.380	0.060	0.038	0.950	0.057	0.416	0.067	2.386
11	4	0.400	1.900	0.350	0.060	0.041	0.875	0.053	0.477	0.055	2.515
12	1	0.600	2.320	0.470	0.060	0.042	0.783	0.047	0.493	0.056	2.164
12	2	0.600	2.320	0.460	0.060	0.045	0.767	0.046	0.572	0.043	2.581
12	3	0.600	2.320	0.450	0.060	0.047	0.750	0.045	0.610	0.037	2.825
12	4	0.600	2.320	0.450	0.060	0.048	0.750	0.045	0.651	0.031	3.234
13	1	0.500	1.620	0.500	0.080	0.049	1.000	0.080	0.392	0.099	2.303
13	2	0.500	1.620	0.500	0.080	0.055	1.000	0.080	0.486	0.073	2.806
13	3	0.500	1.620	0.450	0.080	0.057	0.900	0.072	0.532	0.063	2.825
13	4	0.500	1.620	0.400	0.080	0.060	0.800	0.064	0.581	0.052	2.874
14	6	0.900	1.780	0.700	0.100	0.083	0.778	0.078	0.715	0.042	3.928
14	8	0.900	1.780	0.550	0.100	0.086	0.611	0.061	0.767	0.032	3.952
15	5	0.800	1.670	0.700	0.100	0.077	0.875	0.088	0.620	0.062	3.223
15	6	0.800	1.670	0.700	0.100	0.081	0.875	0.088	0.679	0.048	3.928
15	7	0.800	1.670	0.700	0.100	0.083	0.875	0.088	0.708	0.043	4.394
16	6	0.400	1.900	0.350	0.060	0.035	0.875	0.053	0.358	0.080	1.964
16	7	0.400	1.900	0.400	0.060	0.038	1.000	0.060	0.416	0.067	2.511
16	8	0.400	1.900	0.300	0.060	0.041	0.750	0.045	0.477	0.055	2.156
17	5	0.600	2.320	0.600	0.060	0.042	1.000	0.060	0.493	0.056	2.763
17	6	0.600	2.320	0.550	0.060	0.045	0.917	0.055	0.572	0.043	3.086
18	5	0.500	1.620	0.450	0.080	0.049	0.900	0.072	0.392	0.099	2.072
18	6	0.500	1.620	0.500	0.080	0.055	1.000	0.080	0.486	0.073	2.806
18	7	0.500	1.620	0.450	0.080	0.057	0.900	0.072	0.532	0.063	2.825
18	8	0.500	1.620	0.450	0.080	0.060	0.900	0.072	0.581	0.052	3.234
19	5	0.400	2.820	0.400	0.040	0.019	1.000	0.040	0.240	0.080	1.842
19	6	0.400	2.820	0.400	0.040	0.024	1.000	0.040	0.358	0.054	2.245
21	6	0.500	1.320	0.450	0.100	0.068	0.900	0.090	0.486	0.091	2.525
21	7	0.500	1.320	0.400	0.100	0.071	0.800	0.080	0.532	0.078	2.511

Table 4

Test Conditions and Test Results for Toe Berm Stone Tests

TEST	PLAN	WATER DEPTH AT TOE [ds] FT	WAVE PERIOD [T] SEC	DESIGN WAVE HEIGHT [HD] FT	RELATIVE WATER DEPTH AT TOE [ds/Ls]	RELATIVE WATER DEPTH AT TOP OF BERM [d1/L1]	RELATIVE WAVE HEIGHT AT TOE [HD/ds]	WAVE STEEPNESS AT TOE [HD/Ls]	RELATIVE BERM DEPTH [d1/ds]	RELATIVE BERM LENGTH [B/L1]	STABILITY NUMBER [Ns]
3-D Tests : L-Shaped Wave Basin											
Oblique Wave Attack on Structure Trunks											
22-N-45	2-N-45	0.400	2.820	0.380	0.040	0.030	0.950	0.038	0.553	0.049	3.333
12-N-45	2-N-45	0.500	2.120	0.490	0.060	0.045	0.980	0.059	0.581	0.057	3.521
25-N-45	2-N-45	0.500	1.620	0.430	0.080	0.063	0.860	0.069	0.643	0.072	3.771
14-N-45	2-N-45	0.400	2.820	0.420	0.040	0.030	1.050	0.042	0.553	0.049	3.684
21-N-45	2-N-45	0.400	2.820	0.380	0.040	0.027	0.950	0.038	0.477	0.053	2.731
18-N-45	2-N-45	0.500	2.120	0.390	0.060	0.045	0.780	0.047	0.581	0.057	2.803
15-N-45	2-N-45	0.600	1.450	0.470	0.100	0.082	0.783	0.078	0.702	0.071	4.122
23-N-45	2-N-45	0.400	1.900	0.360	0.060	0.044	0.900	0.054	0.553	0.073	3.157
19-N-45	2-N-45	0.500	2.120	0.390	0.060	0.045	0.780	0.047	0.581	0.057	2.803
16-N-45	2-N-45	0.600	1.450	0.472	0.100	0.082	0.787	0.079	0.702	0.071	4.140
90 Degree Wave Attack on Structure Trunks											
10-0	1-0-B	0.600	1.240	0.500	0.120	0.094	0.833	0.054	0.651	0.091	3.593
20-N-90	2-N-90	0.500	2.120	0.480	0.060	0.034	0.960	0.054	0.322	0.079	1.954
12-N-90	2-N-90	0.500	2.120	0.490	0.060	0.039	0.980	0.054	0.432	0.073	2.441
15-N-90	2-N-90	0.600	1.450	0.470	0.100	0.074	0.783	0.038	0.572	0.049	2.637
16-N-90	2-N-90	0.600	1.450	0.472	0.100	0.074	0.787	0.038	0.572	0.053	2.649
1-0	1-0	0.500	2.120	0.490	0.060	0.039	0.980	0.038	0.432	0.049	2.441
13-N-90	2-N-90	0.500	1.620	0.490	0.080	0.057	0.980	0.038	0.532	0.049	3.076
6-0	1-0-A	0.400	2.820	0.420	0.040	0.021	1.050	0.054	0.290	0.073	2.093
9-0	1-0-B	0.600	1.450	0.520	0.100	0.079	0.867	0.059	0.651	0.057	3.737
19-N-90	2-N-90	0.500	2.120	0.481	0.060	0.039	0.962	0.038	0.432	0.053	2.397

Table 5

Test Conditions and Test Results for Toe Berm Stone Tests

TEST	PLAN	WATER DEPTH AT TOE [ds] FT	WAVE PERIOD [T] SEC	DESIGN WAVE HEIGHT [HD] FT	RELATIVE WATER DEPTH AT TOE [ds/Ls]	RELATIVE WATER DEPTH AT TOP OF BERM [d1/L1]	RELATIVE WAVE HEIGHT AT TOE [HD/ds]	WAVE STEEPNESS AT TOE [HD/Ls]	RELATIVE BERM DEPTH [d1/ds]	RELATIVE BERM LENGTH [B/L1]	STABILITY NUMBER [Ns]
3-D Tests : L-Shaped Wave Basin											
Oblique Wave Attack on Structure Heads											
19-N-45	2-N-45	0.500	2.120	0.390	0.060	0.045	0.780	0.042	0.581	0.049	2.803
10-N	1-N-B	0.600	1.240	0.500	0.120	0.094	0.833	0.078	0.651	0.071	3.593
16-N-45	2-N-45	0.600	1.450	0.472	0.100	0.079	0.787	0.047	0.651	0.057	3.392
19-N-45	2-N-45	0.500	2.120	0.390	0.060	0.043	0.780	0.047	0.532	0.057	2.448
13-N-45	2-N-45	0.500	1.620	0.490	0.080	0.063	0.980	0.079	0.643	0.071	4.297
23-N-45	2-N-45	0.400	1.900	0.360	0.060	0.041	0.900	0.100	0.477	0.088	2.587
20-N-45	2-N-45	0.500	2.120	0.390	0.060	0.045	0.780	0.058	0.581	0.076	2.803
12-N-45	2-N-45	0.500	2.120	0.490	0.060	0.043	0.980	0.059	0.532	0.066	3.076
18-N-45	2-N-45	0.500	2.120	0.390	0.060	0.045	0.780	0.078	0.581	0.079	2.803
22-N-45	2-N-45	0.400	2.820	0.380	0.040	0.027	0.950	0.079	0.477	0.079	2.731
9-N	1-N-B	0.600	1.450	0.520	0.100	0.079	0.867	0.059	0.651	0.066	3.737
21-N-45	2-N-45	0.400	2.820	0.380	0.040	0.026	0.950	0.078	0.416	0.079	2.386
18-N-45	2-N-45	0.500	2.120	0.390	0.060	0.043	0.780	0.042	0.532	0.067	2.448
15-N-45	2-N-45	0.600	1.450	0.470	0.100	0.079	0.783	0.087	0.651	0.074	3.377
14-N-45	2-N-45	0.400	2.820	0.420	0.040	0.026	1.050	0.058	0.416	0.066	2.637
90 Degree Wave Attack on Structure Heads											
10-0	1-0-B	0.600	1.240	0.500	0.120	0.094	0.833	0.047	0.651	0.057	3.593
20-N-90	2-N-90	0.500	2.120	0.480	0.060	0.041	0.960	0.100	0.486	0.088	2.694
13-N-90	2-N-90	0.500	1.620	0.490	0.080	0.057	0.980	0.079	0.532	0.074	3.076
21-N-90	2-N-90	0.400	2.820	0.380	0.040	0.026	0.950	0.047	0.416	0.060	2.386
12-N-90	2-N-90	0.500	2.120	0.490	0.060	0.043	0.980	0.078	0.532	0.072	3.076
9-0	1-0-B	0.600	1.450	0.520	0.100	0.079	0.867	0.054	0.651	0.079	3.737
23-N-90	2-N-90	0.400	1.900	0.360	0.060	0.038	0.900	0.047	0.416	0.057	2.260
18-N-90	2-N-90	0.500	2.120	0.481	0.060	0.043	0.962	0.059	0.532	0.060	3.019
19-N-90	2-N-90	0.500	2.120	0.481	0.060	0.043	0.962	0.047	0.532	0.057	3.019
22-N-90	2-N-90	0.400	2.820	0.380	0.040	0.027	0.950	0.038	0.477	0.053	2.731
7-0	1-0-A	0.400	1.900	0.460	0.060	0.038	1.150	0.087	0.416	0.074	2.888
18-N-90	2-N-90	0.500	2.120	0.481	0.060	0.041	0.962	0.038	0.486	0.056	2.699
14-N-90	2-N-90	0.400	2.820	0.420	0.040	0.026	1.050	0.047	0.416	0.060	2.637
19-N-90	2-N-90	0.500	2.120	0.481	0.060	0.041	0.962	0.078	0.486	0.074	2.699
16-N-90	2-N-90	0.600	1.450	0.472	0.100	0.079	0.787	0.042	0.651	0.056	3.392
15-N-90	2-N-90	0.600	1.450	0.470	0.100	0.079	0.783	0.100	0.651	0.088	3.377

Table 6

Test Conditions and Test Results for Toe Berm Stone Tests

TEST	PLAN	WATER DEPTH AT TOE [ds] FT	WAVE PERIOD [T] SEC	DESIGN WAVE HEIGHT [HD] FT	RELATIVE WATER DEPTH AT TOE [ds/Ls]	RELATIVE WATER DEPTH AT TOP OF BERM [d1/L1]	RELATIVE WAVE HEIGHT AT TOE [HD/ds]	WAVE STEEPNESS AT TOE [HD/Ls]	RELATIVE BERM DEPTH [d1/ds]	RELATIVE BERM LENGTH [B/L1]	STABILITY NUMBER [Ns]
3-D Tests : T-Shaped Wave Basin											
90 Degree Wave Attack on Structure Trunks											
4	1	0.500	1.620	0.440	0.080	0.044	0.880	0.070	0.322	0.101	1.791
5	1	0.500	1.620	0.400	0.080	0.051	0.800	0.064	0.432	0.087	1.993
7	1	0.500	1.320	0.375	0.100	0.068	0.750	0.075	0.486	0.102	2.104
8	1	0.500	1.320	0.440	0.100	0.068	0.880	0.088	0.486	0.102	2.469
2	1	0.600	1.100	0.500	0.138	0.107	0.833	0.115	0.651	0.100	3.593
90 Degree Wave Attack on Structure Heads											
5	1	0.500	1.620	0.400	0.080	0.055	0.800	0.064	0.486	0.082	2.245
8	1	0.500	1.320	0.440	0.100	0.068	0.880	0.088	0.486	0.102	2.469
7	1	0.500	1.320	0.375	0.100	0.068	0.750	0.075	0.486	0.102	2.104
2	1	0.600	1.100	0.500	0.138	0.107	0.833	0.115	0.651	0.100	3.593

Table 7

Test Conditions and Test Results for Toe Berm Stone Tests Conducted With Spectral Waves

TEST	PLAN	WATER DEPTH AT TOE [ds] FT	WAVE PERIOD [T] SEC	MEASURED Hmo [HD] FT	THEORETICAL MAXIMUM HMO [HD] FT	RELATIVE BERM DEPTH [d1/ds]	STABILITY NUMBER [Ns] FOR HD= MEASURED Hmo	STABILITY NUMBER [Ns] FOR HD= THEORETICAL MAXIMUM Hmo
3-D Tests : L-Shaped Wave Basin : Spectral Waves								
90 Degree and Oblique Wave Attack on Structure Heads and Trunks								
29-N-90	2-N-90	0.400	2.820	0.316	0.258	0.358	1.773	1.448
29-N-90	2-N-90	0.400	2.820	0.316	0.258	0.416	1.984	1.620
29-N-45	2-N-45	0.400	2.820	0.278	0.258	0.477	1.998	1.854
29-N-45	2-N-45	0.400	2.820	0.278	0.258	0.416	1.745	1.620
31-N-90	2-N-90	0.400	1.900	0.322	0.258	0.240	1.483	1.188
31-N-45	2-N-45	0.400	1.900	0.271	0.258	0.477	1.947	1.854
31-N-45	2-N-45	0.400	1.900	0.271	0.258	0.416	1.701	1.620
32-N-90	2-N-90	0.500	1.620	0.375	0.318	0.532	2.354	1.996
32-N-45	2-N-45	0.500	1.620	0.340	0.318	0.643	2.982	2.789
32-N-45	2-N-45	0.500	1.620	0.340	0.318	0.643	2.982	2.789
33-N-90	2-N-90	0.500	1.320	0.320	0.318	0.532	2.009	1.996
33-N-90	2-N-90	0.500	1.320	0.320	0.318	0.532	2.009	1.996
33-N-45	2-N-45	0.500	1.320	0.312	0.318	0.643	2.736	2.789
33-N-45	2-N-45	0.500	1.320	0.312	0.318	0.643	2.736	2.789

Photo 1. Sea-side view of Plans 5-8 after typical 2-D toe berm stone test

Photo 2. Side view of Plans 1 and 2 before testing (2-D toe berm stone tests)

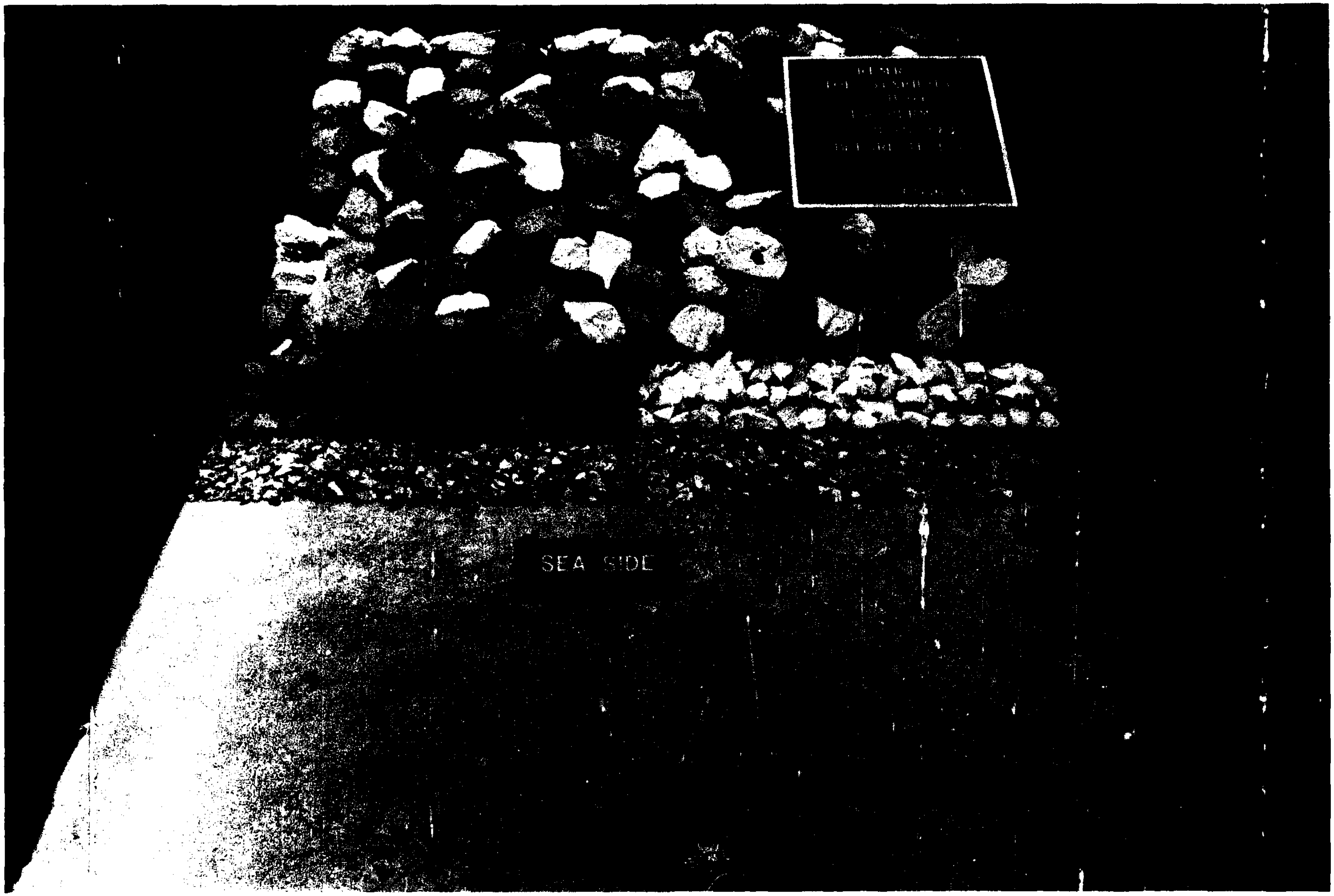


Photo 3. Sea-side view of Plans 1 and 2 before testing (2-D toe berm stone tests)

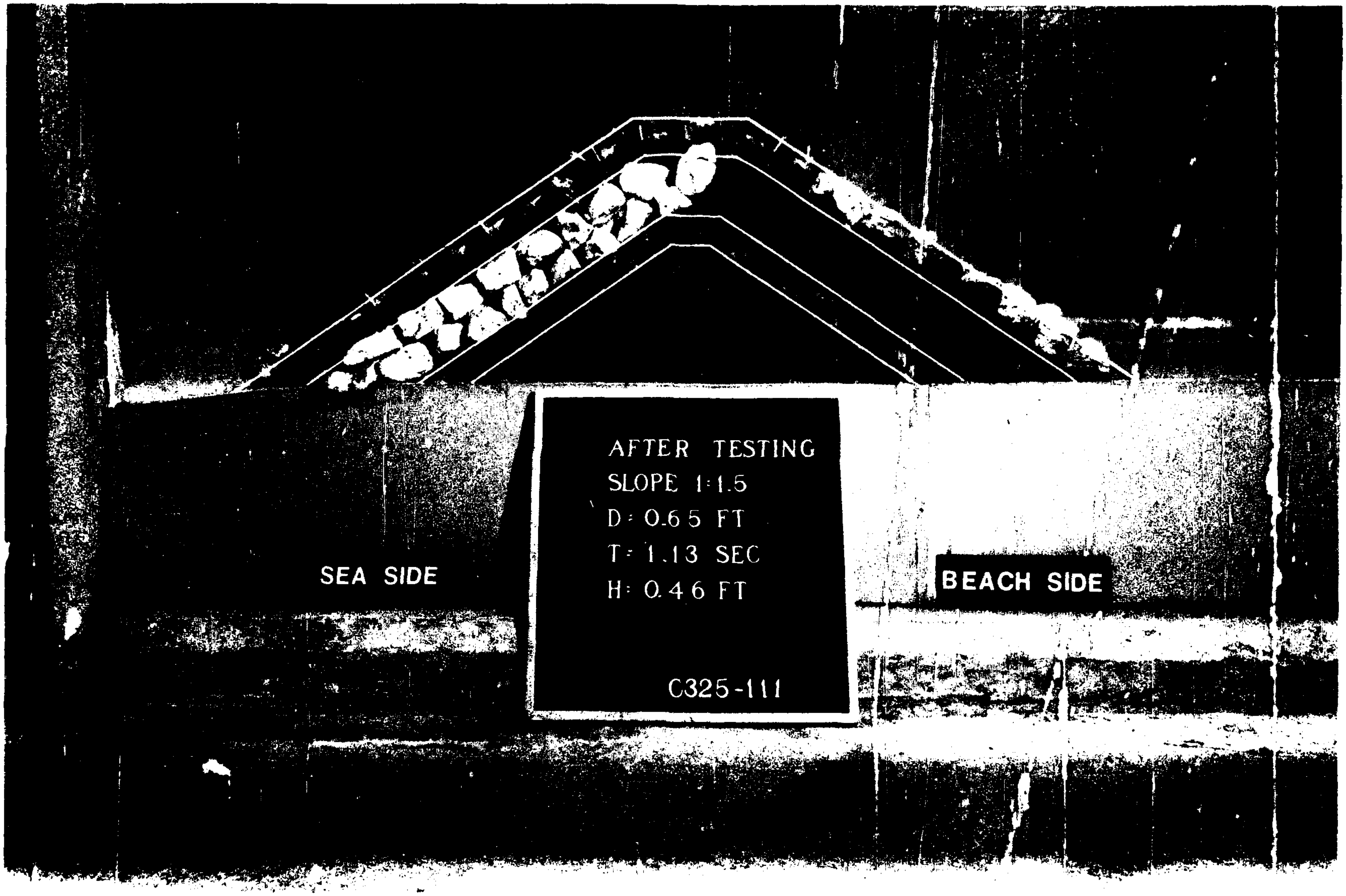


Photo 4. Side view of toe buttressing stone test section after testing

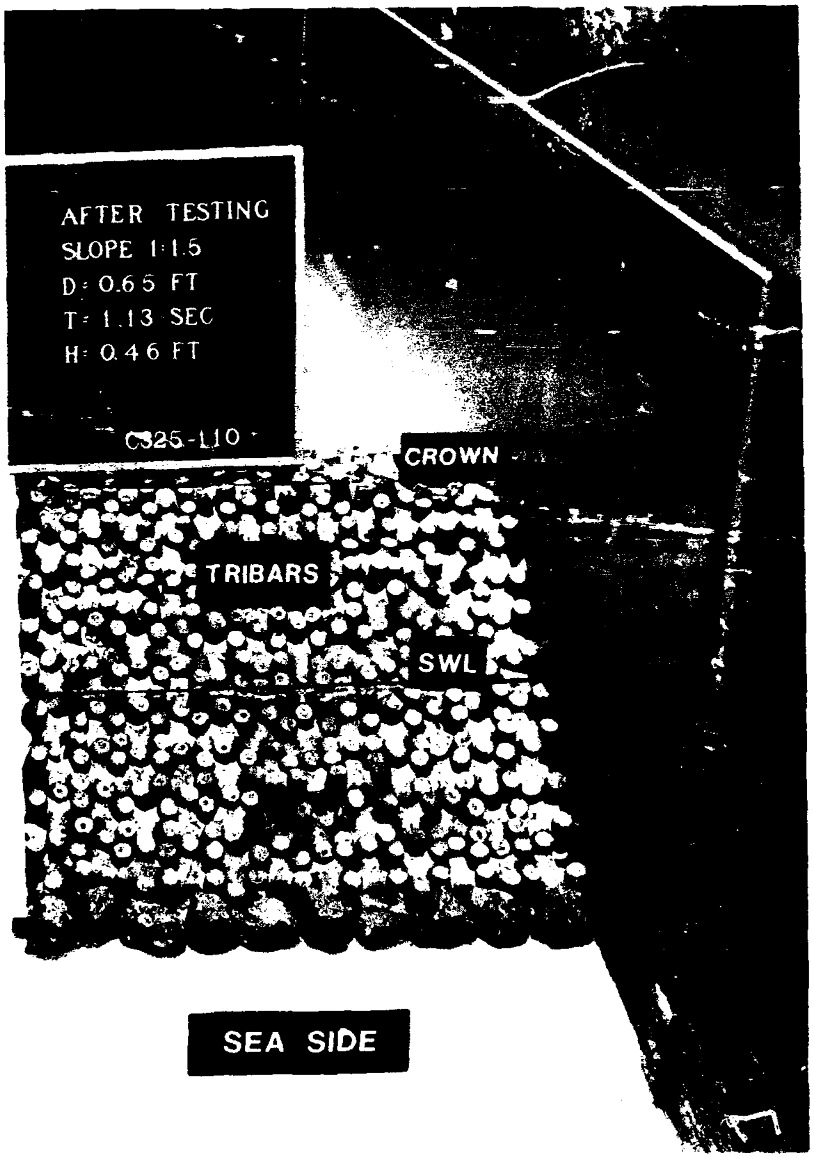


Photo 5. Sea-side view of toe buttressing stone test section after testing

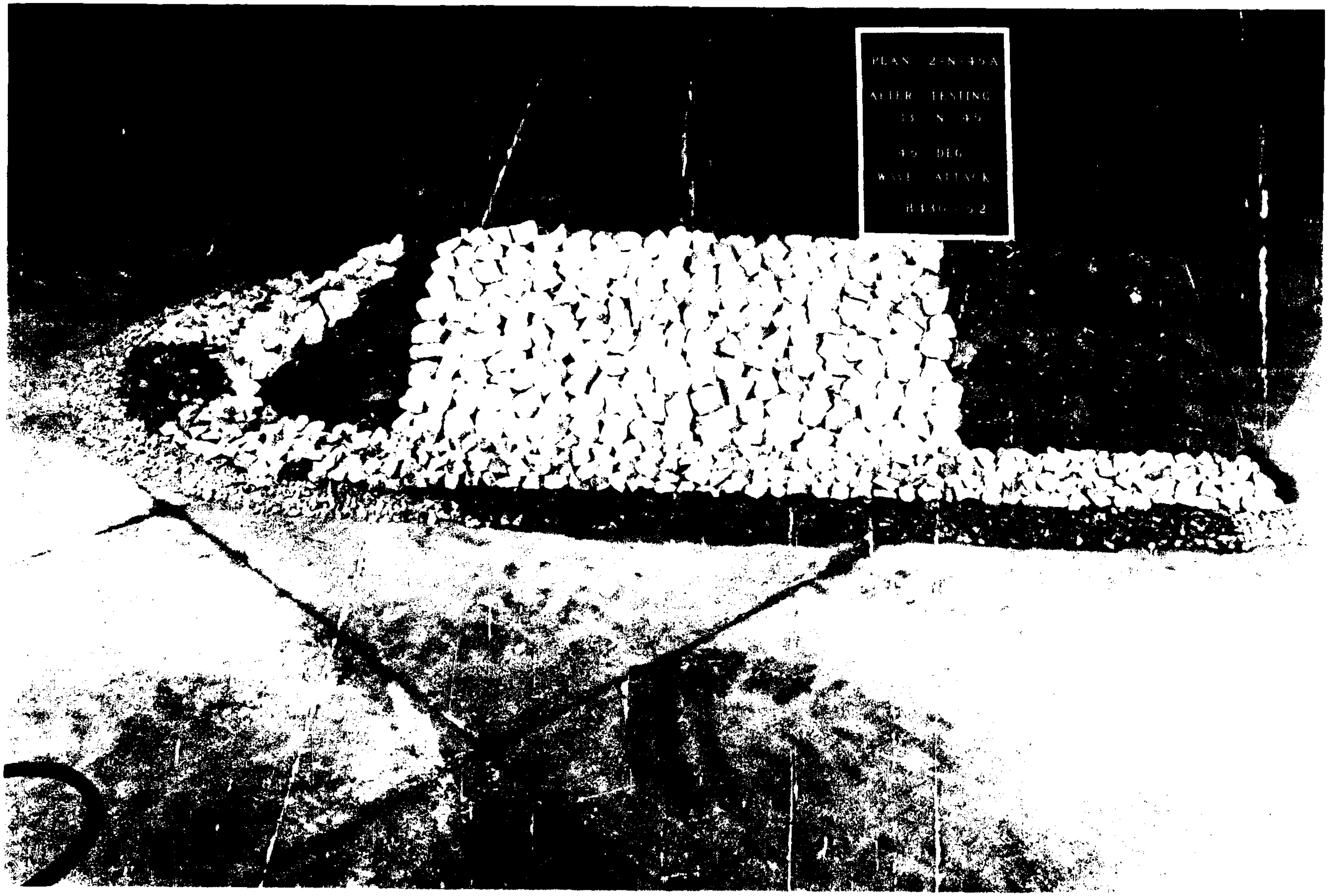


Photo 6. Sea-side view of stone armored, rubble-mound test section after exposure to oblique wave attack (3-D toe berm stone tests)

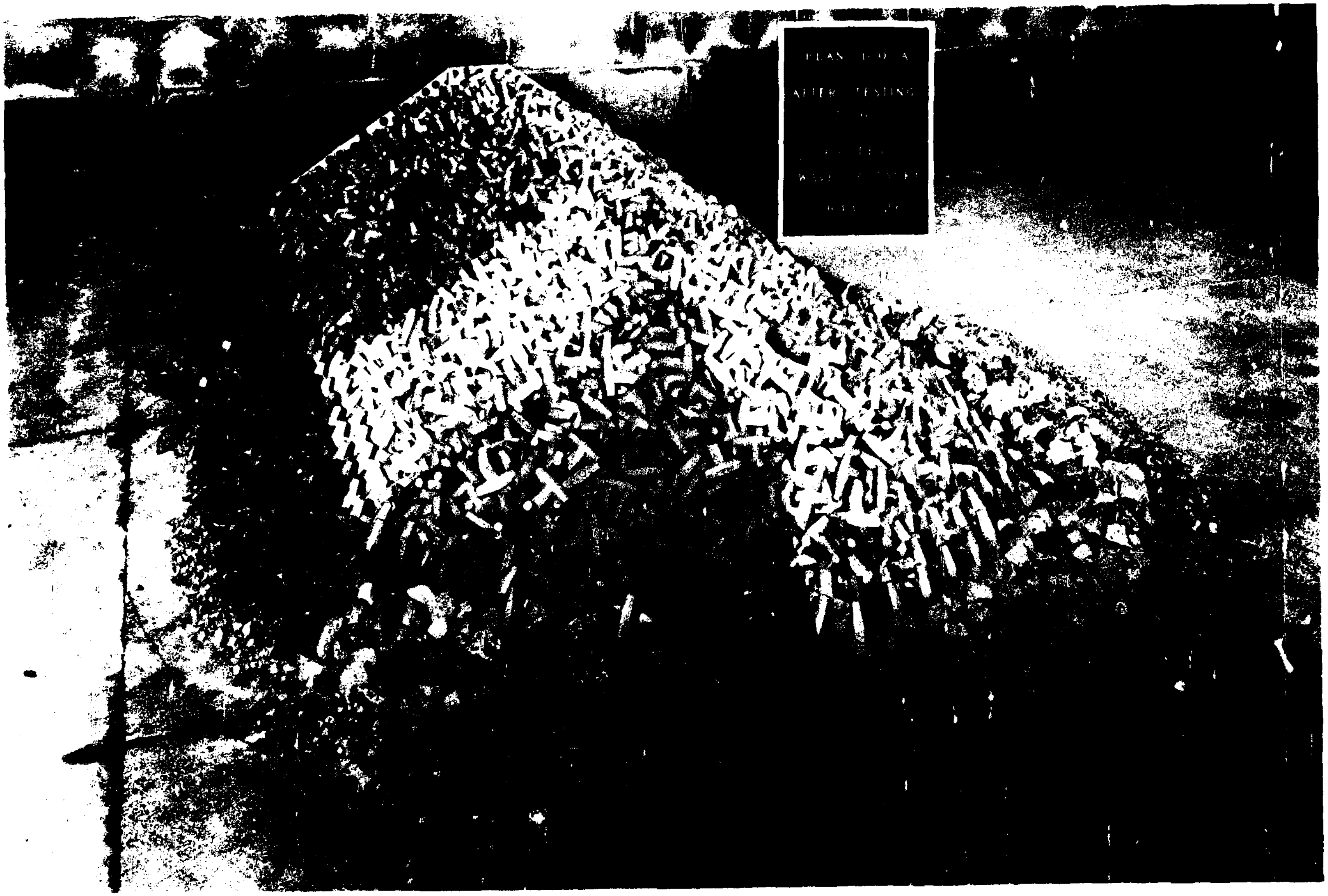
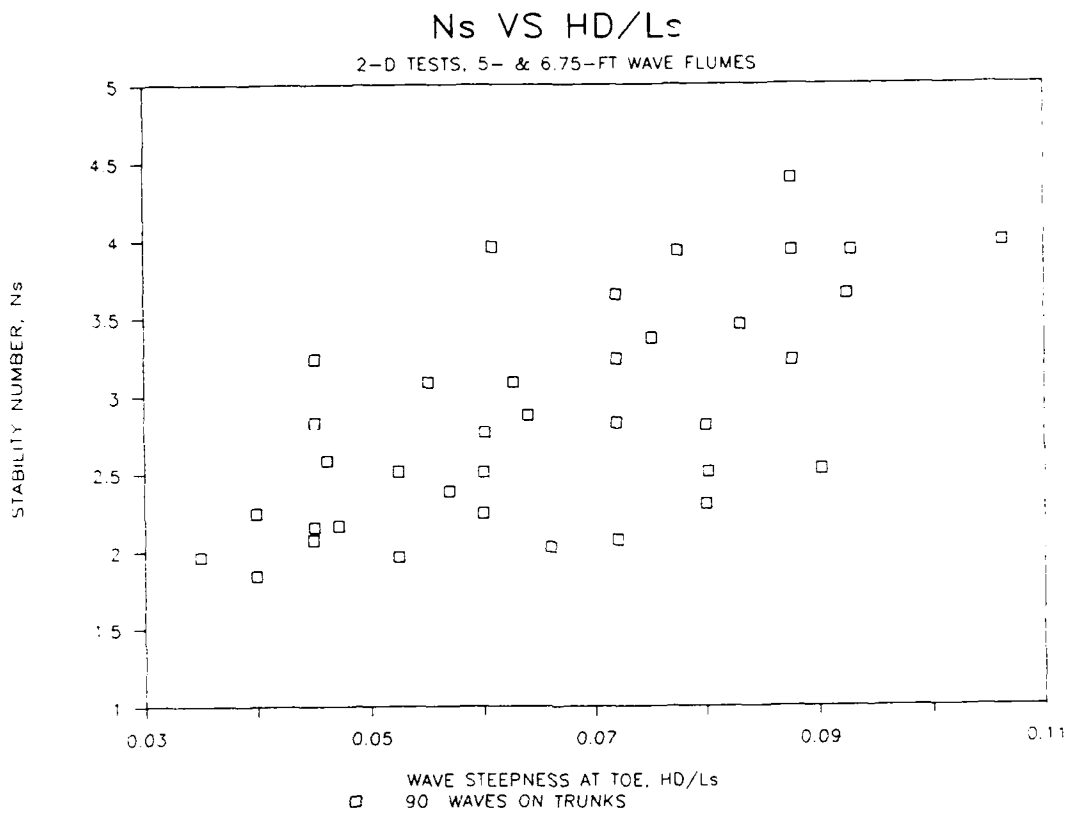
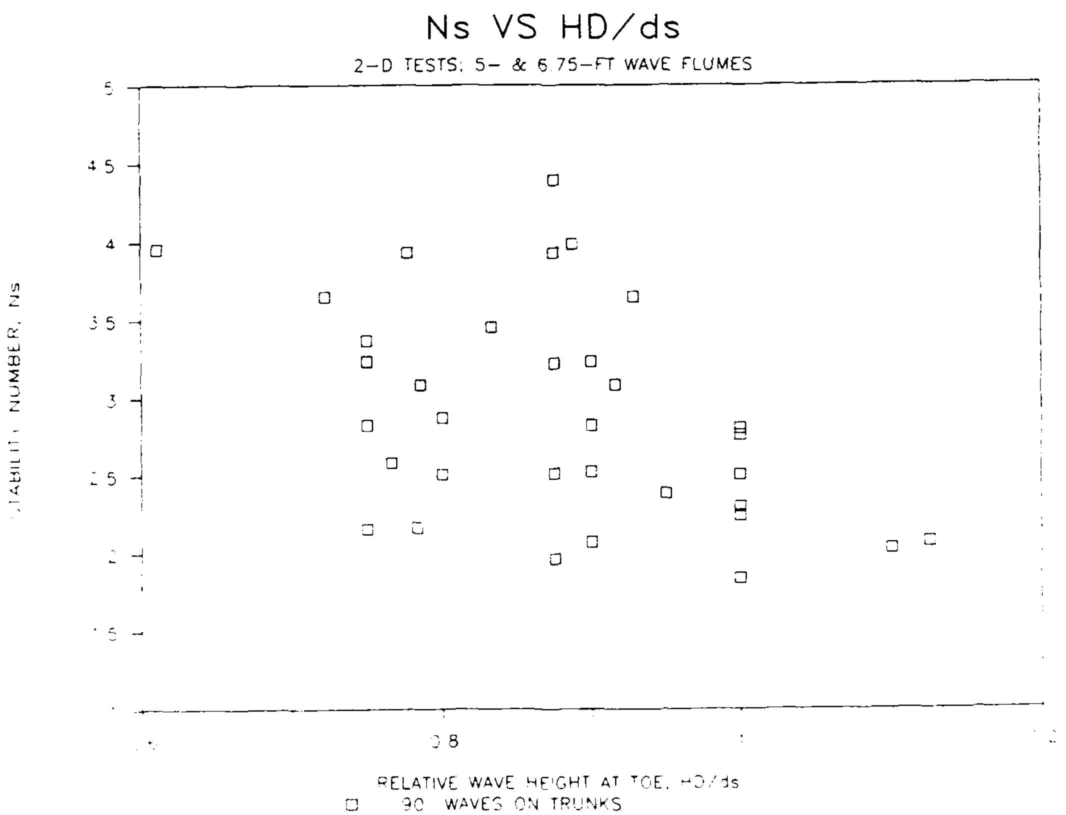
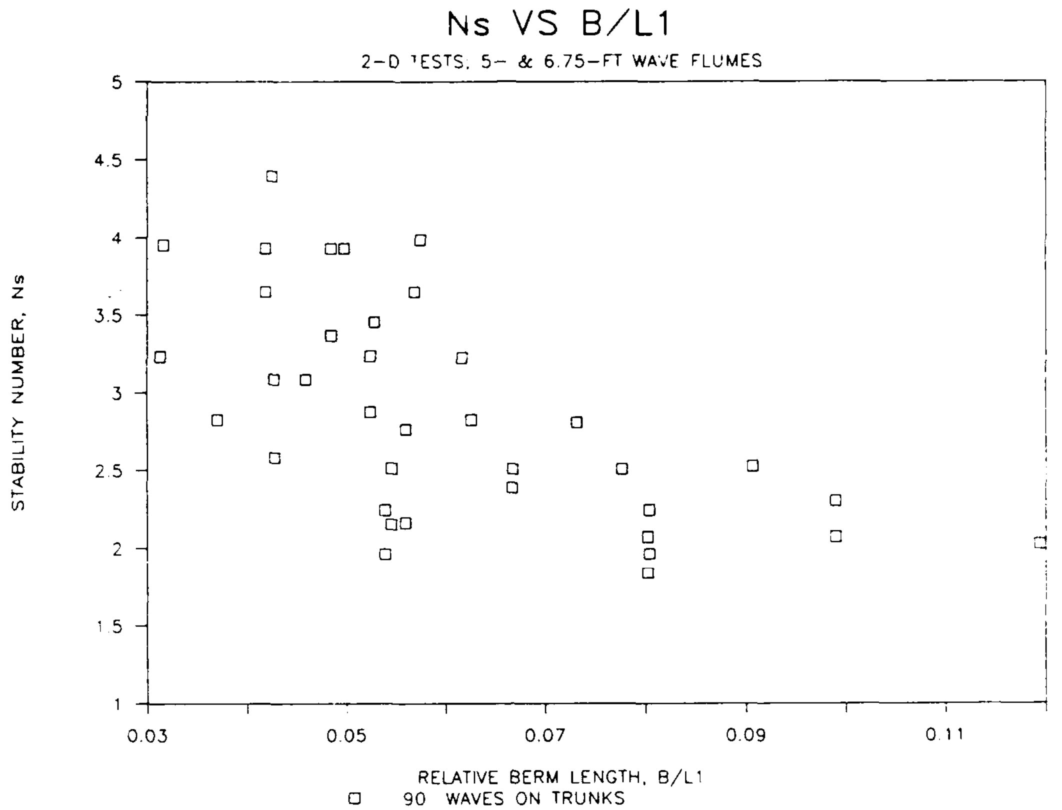


Photo 7. Sea-side view of stone armored rubble-mound test section after exposure to 90-deg wave attack (3-D toe berm stone tests)


Ns VS HD/Ls
2-D TESTS, 5- & 6.75-FT WAVE FLUMES
STABILITY NUMBER, Ns
5
4.5
4
3.5
3
2.5
2
1.5
1
0.03
0.05
0.07
0.09
0.11
WAVE STEEPNESS AT TOE, HD/Ls
90 WAVES ON TRUNKS


Ns VS HD/ds
2-D TESTS; 5- & 6.75-FT WAVE FLUMES
NUMBER, Ns
5
4.5
4
3.5
3
0.8
RELATIVE WAVE HEIGHT AT TOE, HD/ds
90 WAVES ON TRUNKS


Ns VS B/L1
2-D TESTS, 5- & 6.75-FT WAVE FLUMES
STABILITY NUMBER, Ns
RELATIVE BERM LENGTH, B/L1
90 WAVES ON TRUNKS

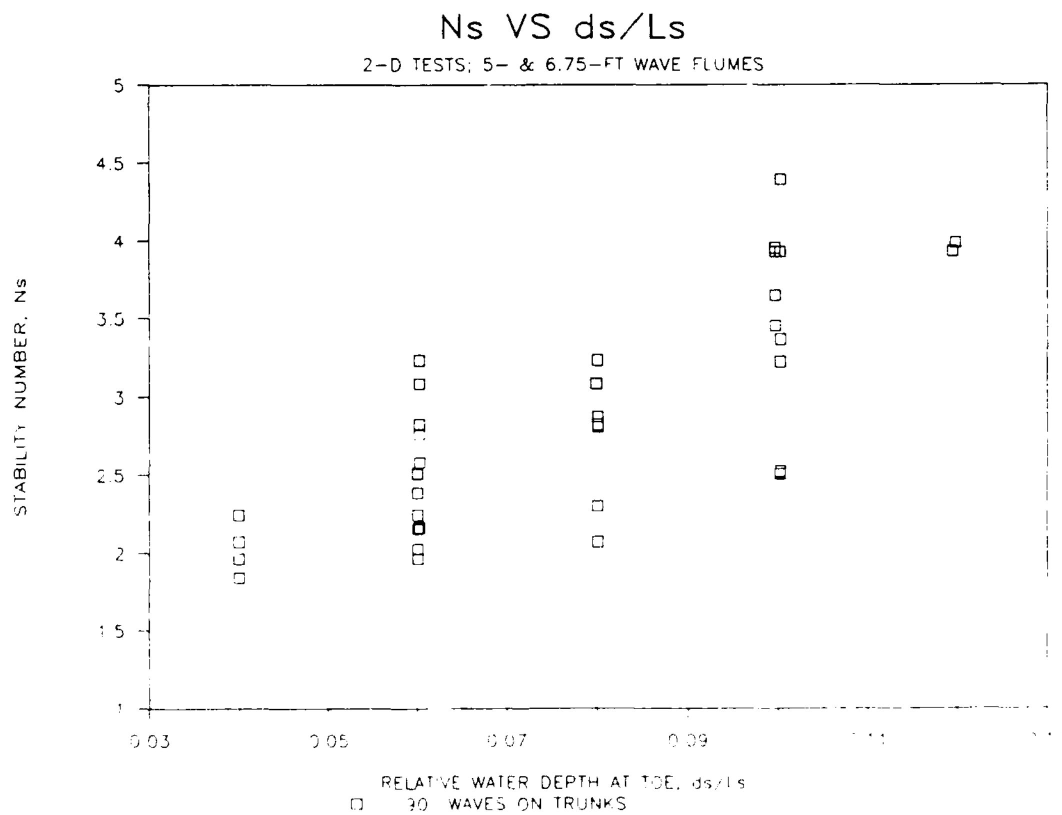
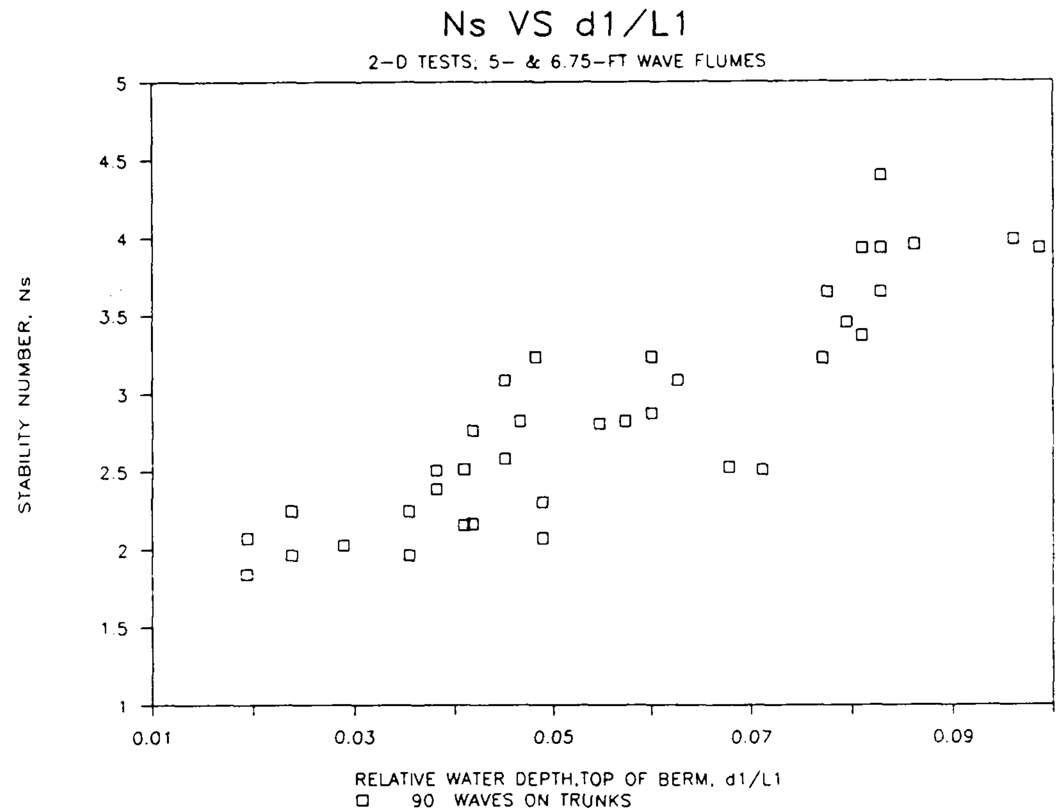
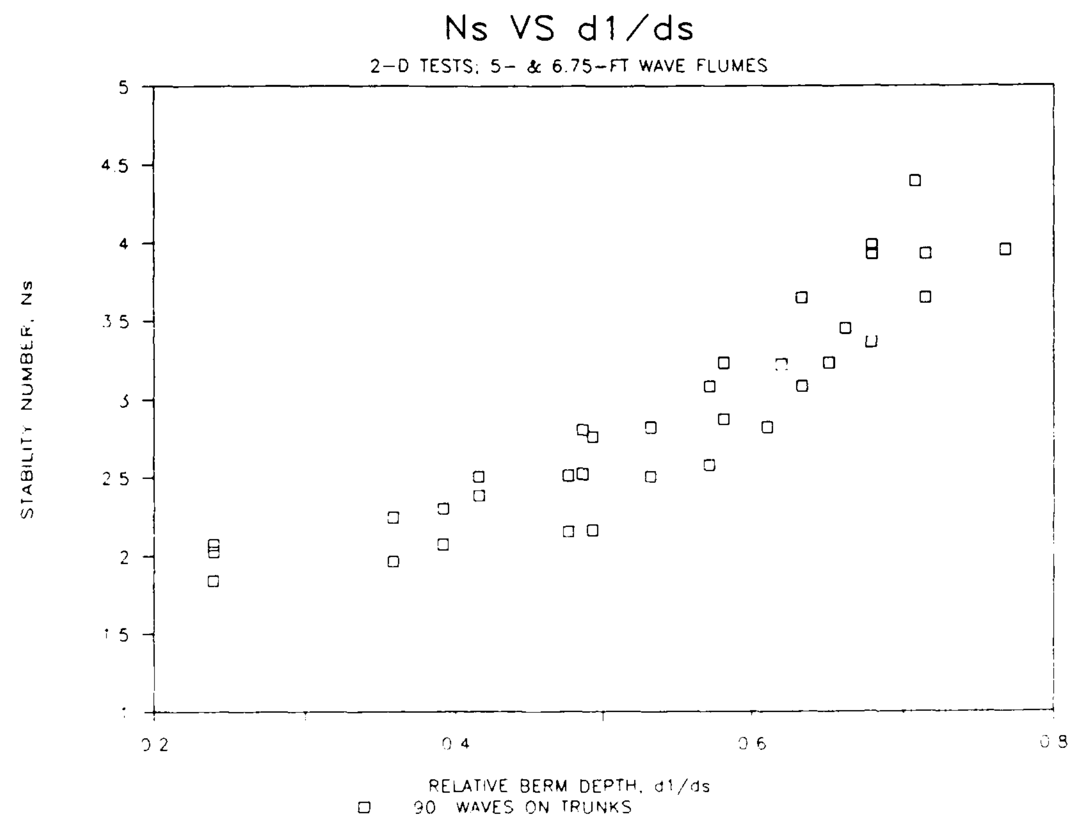
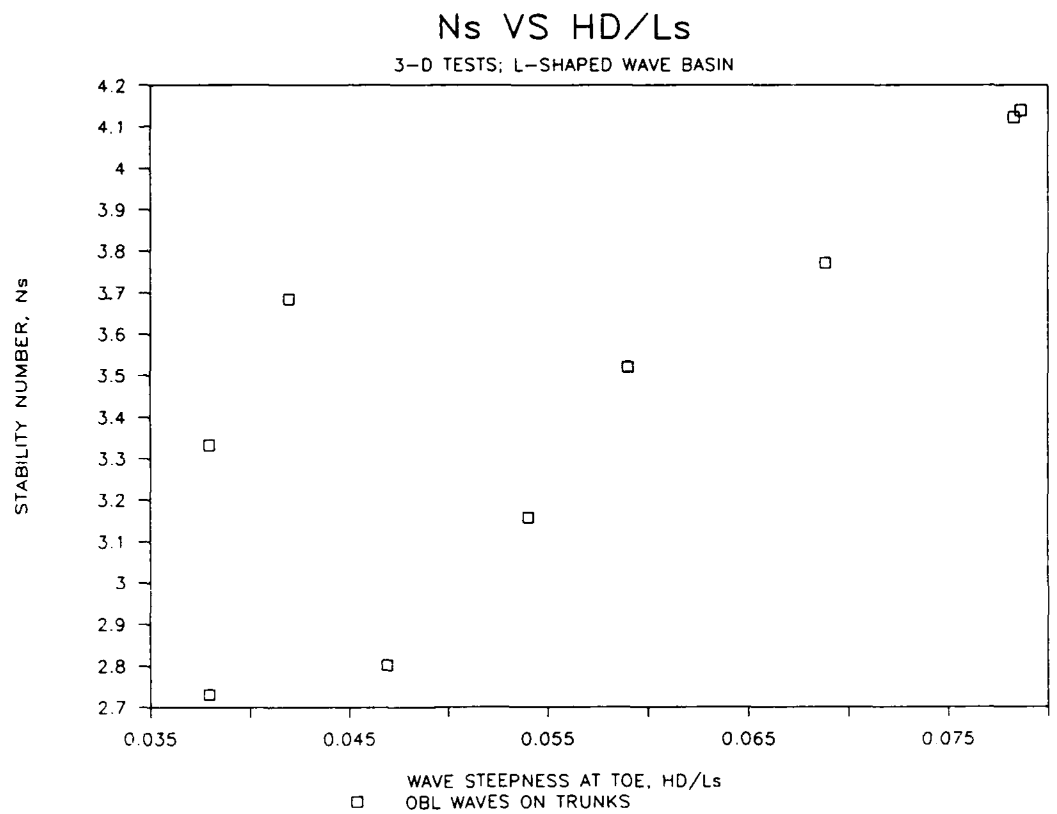

Ns VS ds/Ls
2-D TESTS; 5- & 6.75-FT WAVE FLUMES
STABILITY NUMBER, Ns
RELATIVE WATER DEPTH AT TOE, ds/Ls
90 WAVES ON TRUNKS

PLATE 2


Ns VS d1/L1
2-D TESTS; 5- & 6.75-FT WAVE FLUMES
STABILITY NUMBER, Ns
RELATIVE WATER DEPTH,TOP OF BERM, d1/L1
90 WAVES ON TRUNKS


Ns VS d1/ds
2-D TESTS; 5- & 6.75-FT WAVE FLUMES
STABILITY NUMBER, Ns
RELATIVE BERM DEPTH, d1/ds
90 WAVES ON TRUNKS


Ns VS HD/Ls
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
4.2
4.1
4
3.9
3.8
3.7
3.6
3.5
3.4
3.3
3.2
3.1
3
2.9
2.8
2.7
0.035
0.045
0.055
0.065
0.075
WAVE STEEPNESS AT TOE, HD/Ls
OBL WAVES ON TRUNKS

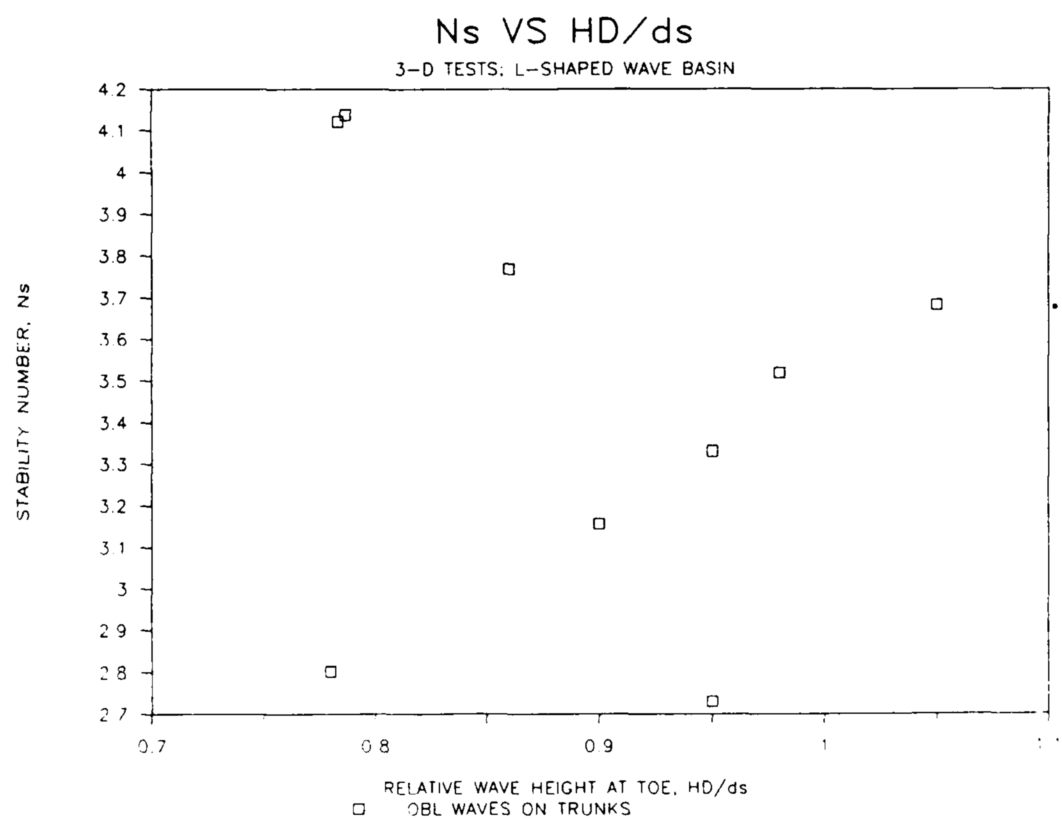
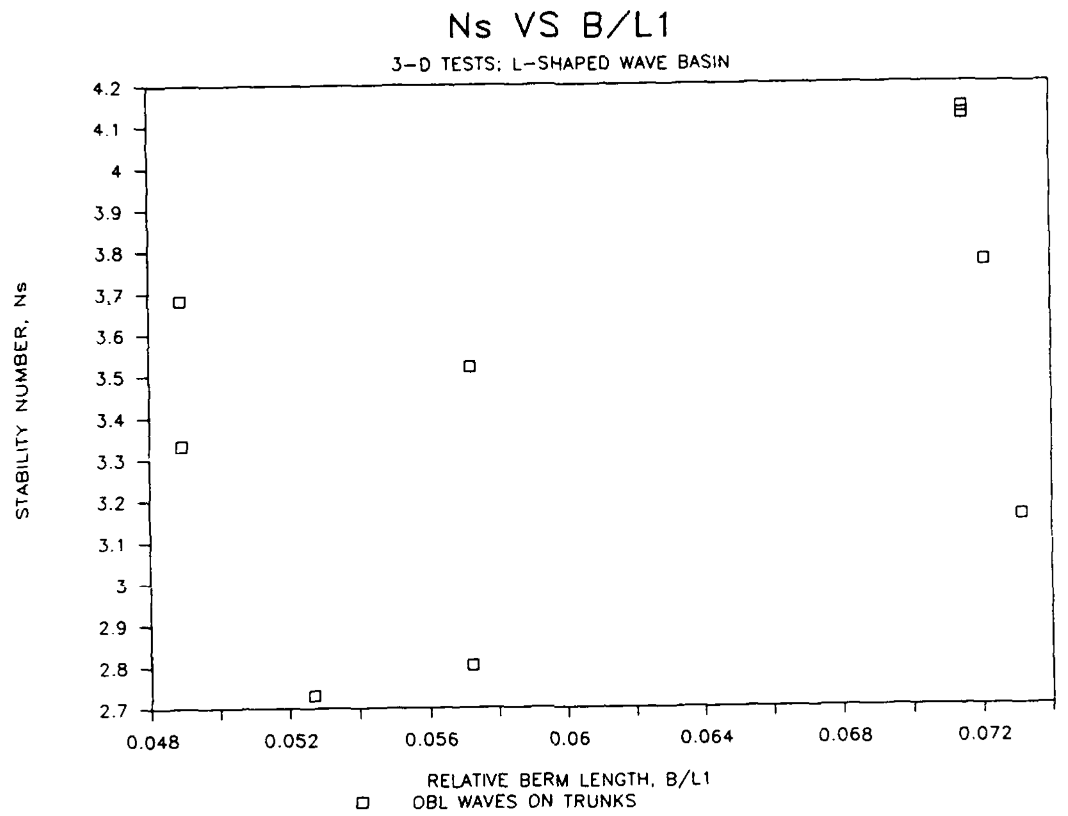
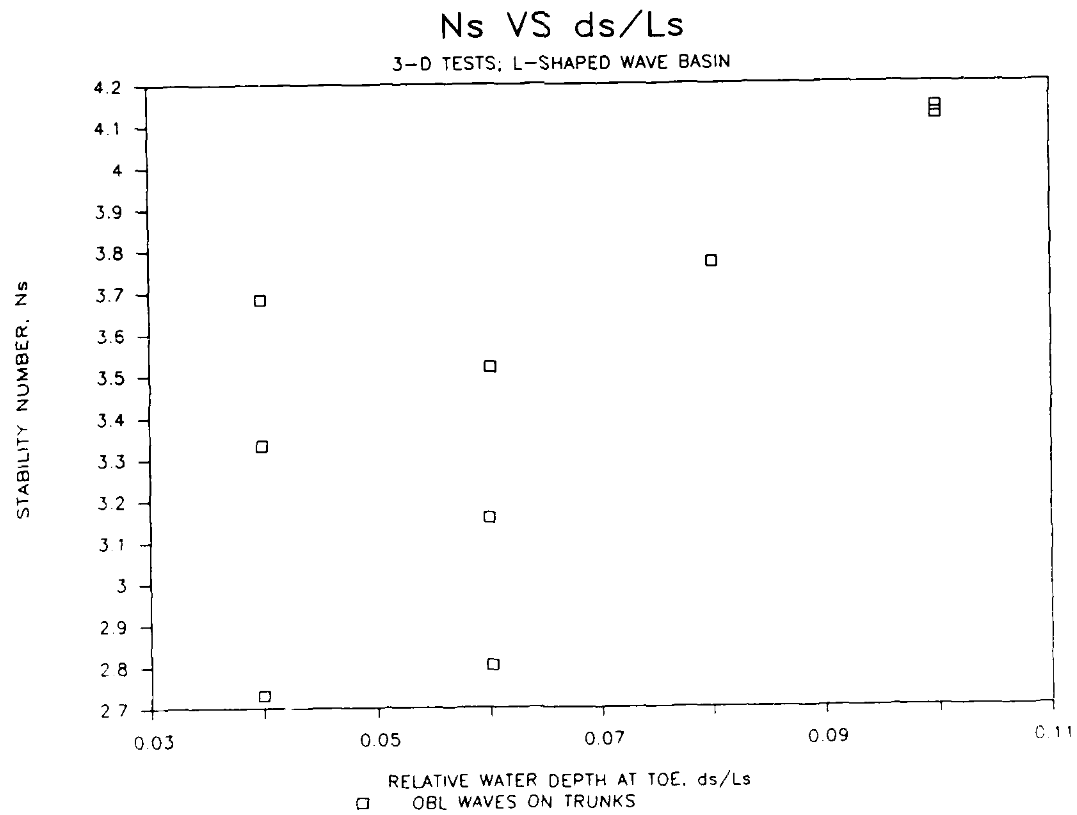

Ns VS HD/ds
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
4.2
4.1
4
3.9
3.8
3.7
3.6
3.5
3.4
3.3
3.2
3.1
3
2.9
2.8
2.7
0.7
0.8
0.9
1
RELATIVE WAVE HEIGHT AT TOE, HD/ds
OBL WAVES ON TRUNKS

PLATE 4

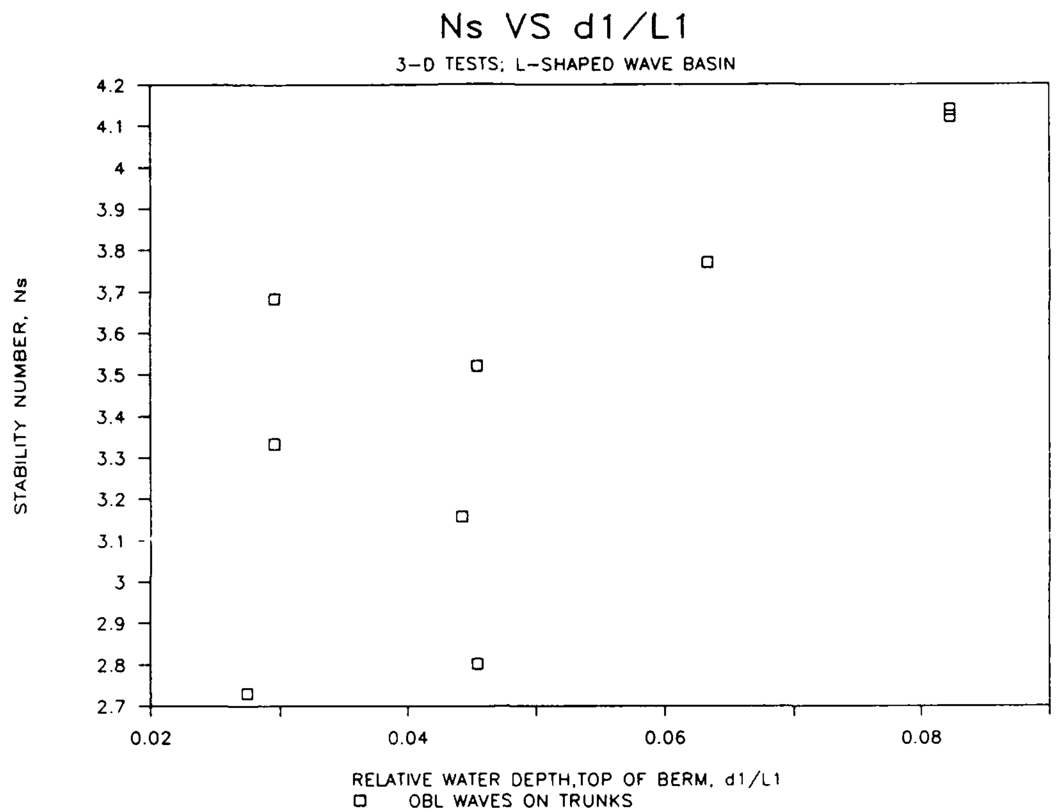


Ns VS B/L1
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
4.2
4.1
4
3.9
3.8
3.7
3.6
3.5
3.4
3.3
3.2
3.1
3
2.9
2.8
2.7
0.048
0.052
0.056
0.06
0.064
0.068
0.072
RELATIVE BERM LENGTH, B/L1
OBL WAVES ON TRUNKS



Ns VS ds/Ls
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
4.2
4.1
4
3.9
3.8
3.7
3.6
3.5
3.4
3.3
3.2
3.1
3
2.9
2.8
2.7
0.03
0.05
0.07
0.09
0.11
RELATIVE WATER DEPTH AT TOE, ds/Ls
OBL WAVES ON TRUNKS

PLATE 5


Ns VS d1/L1
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
RELATIVE WATER DEPTH,TOP OF BERM, d1/L1
OBL WAVES ON TRUNKS

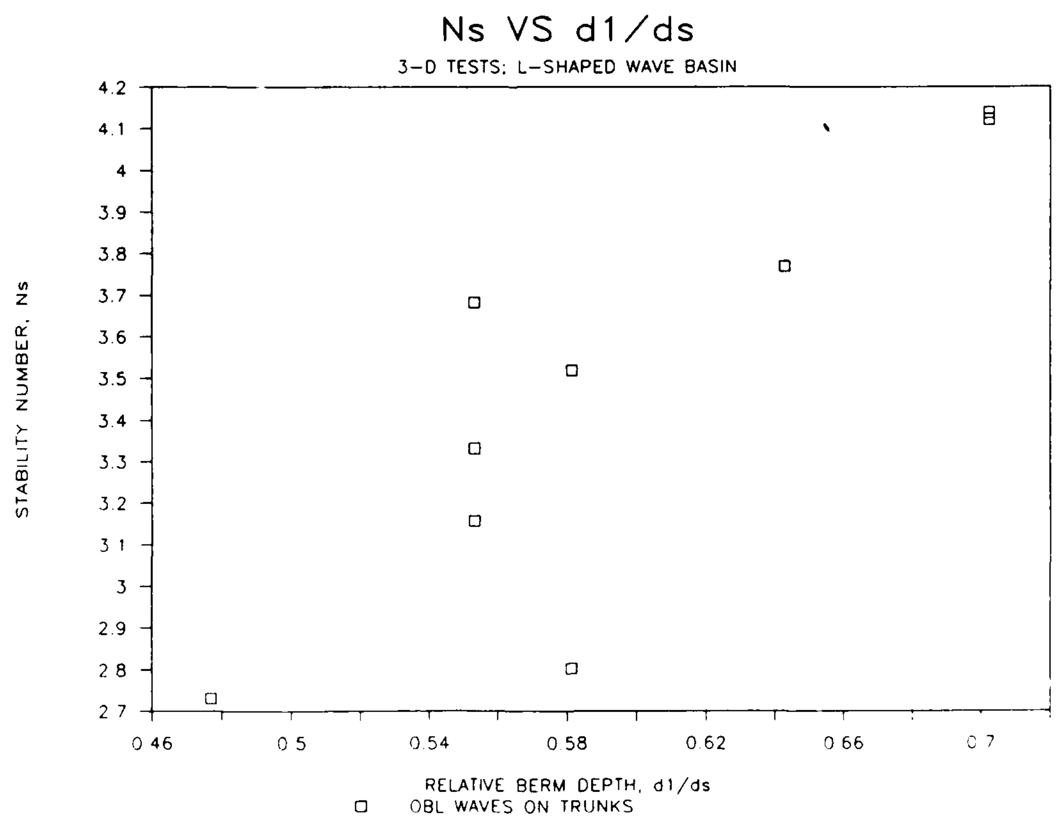
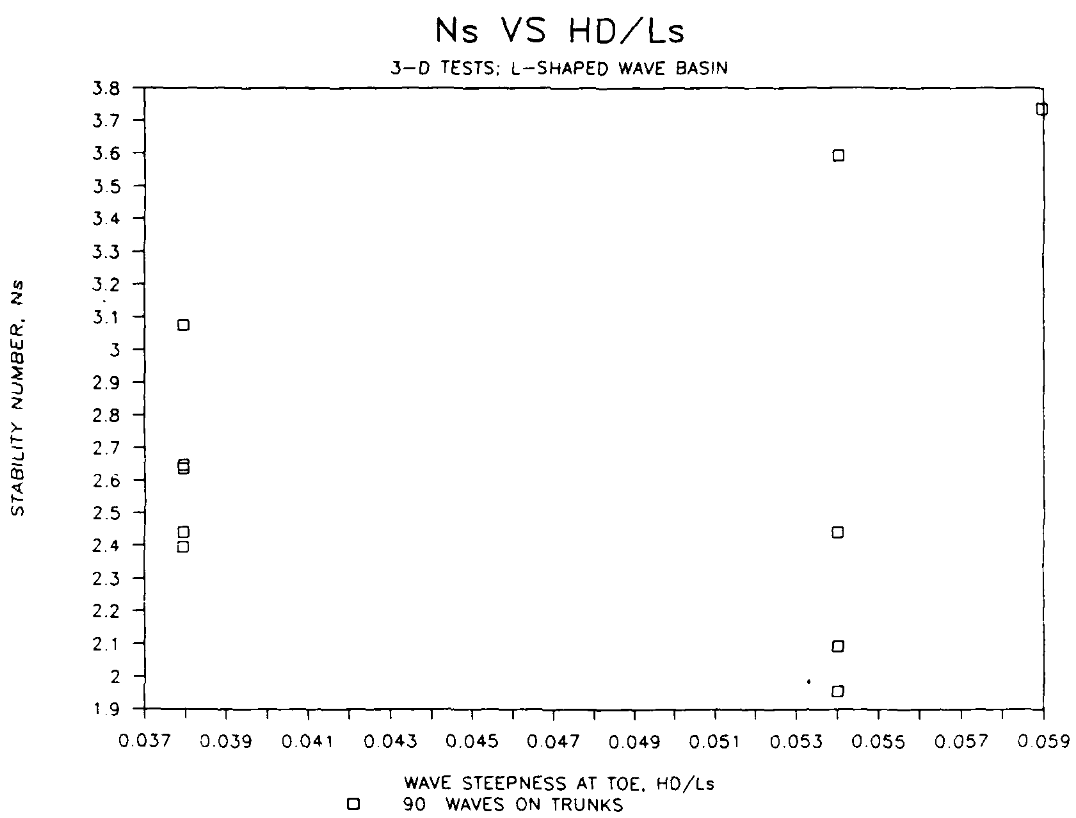
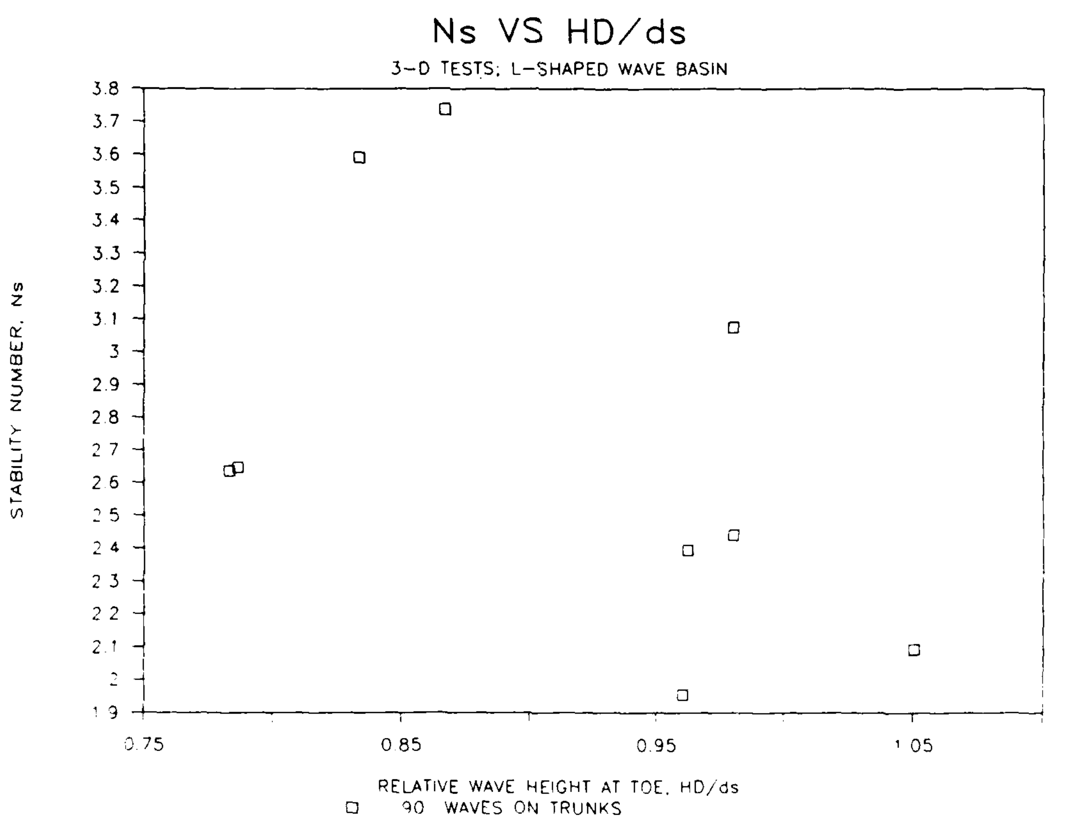

Ns VS d1/ds
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
RELATIVE BERM DEPTH, d1/ds
OBL WAVES ON TRUNKS

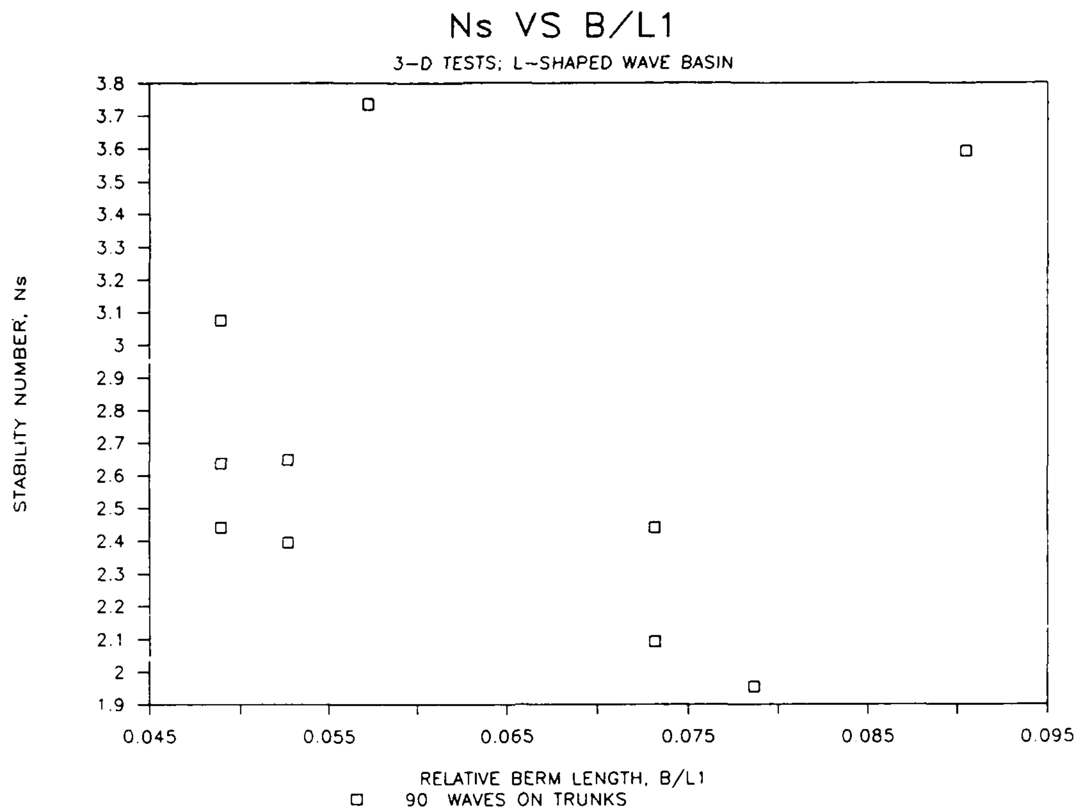
PLATE 6



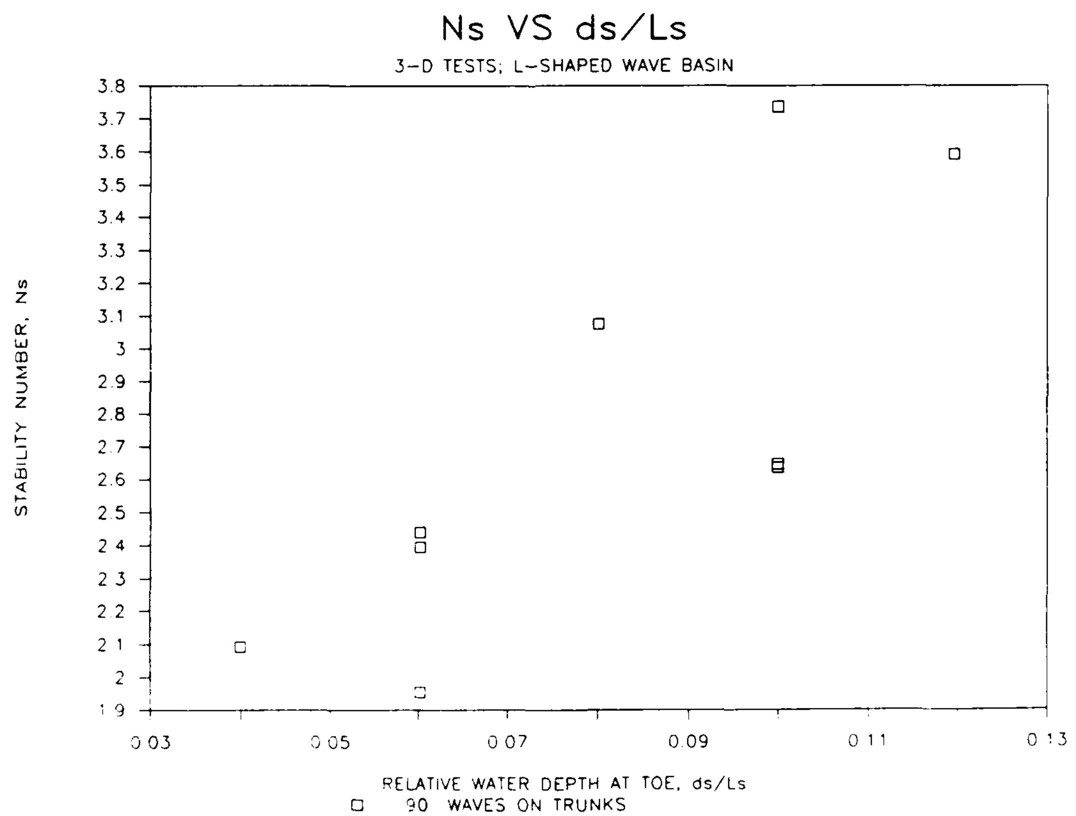
Ns VS HD/Ls
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
3.8
3.7
3.6
3.5
3.4
3.3
3.2
3.1
3
2.9
2.8
2.7
2.6
2.5
2.4
2.3
2.2
2.1
2
1.9
0.037
0.039
0.041
0.043
0.045
0.047
0.049
0.051
0.053
0.055
0.057
0.059
WAVE STEEPNESS AT TOE, HD/Ls
90 WAVES ON TRUNKS



Ns VS HD/ds
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
3.8
3.7
3.6
3.5
3.4
3.3
3.2
3.1
3
2.9
2.8
2.7
2.6
2.5
2.4
2.3
2.2
2.1
2
1.9
0.75
0.85
0.95
1.05
RELATIVE WAVE HEIGHT AT TOE, HD/ds
90 WAVES ON TRUNKS

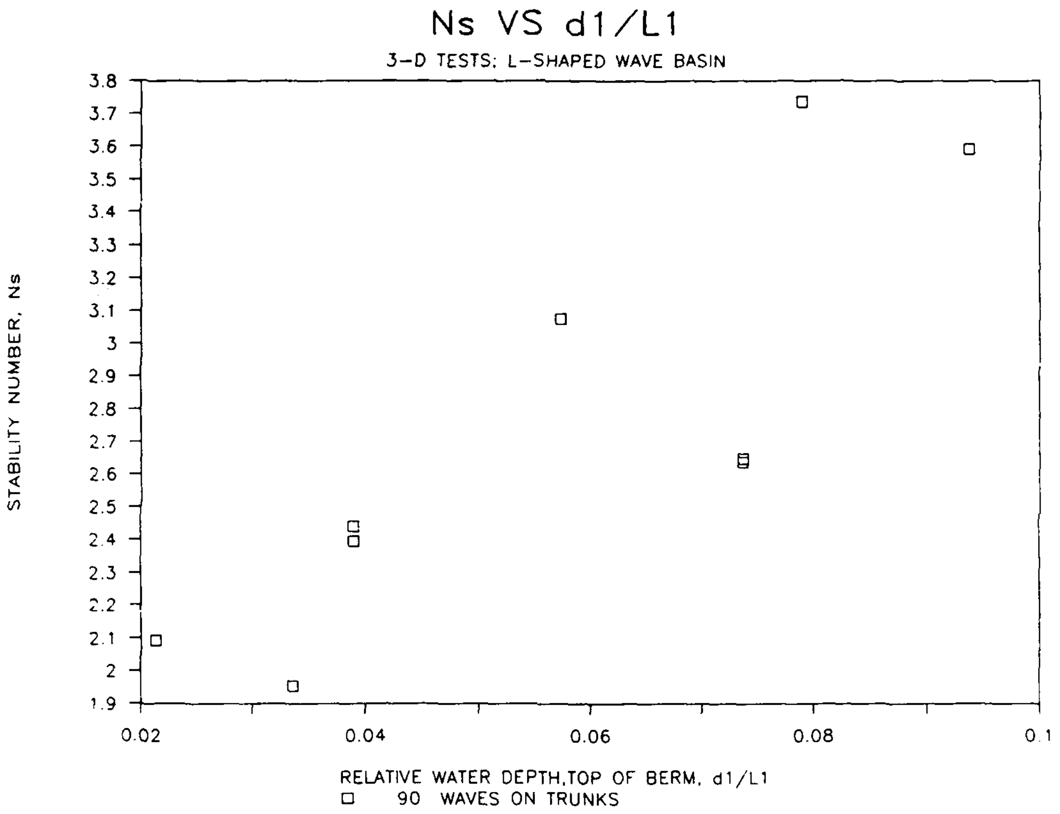


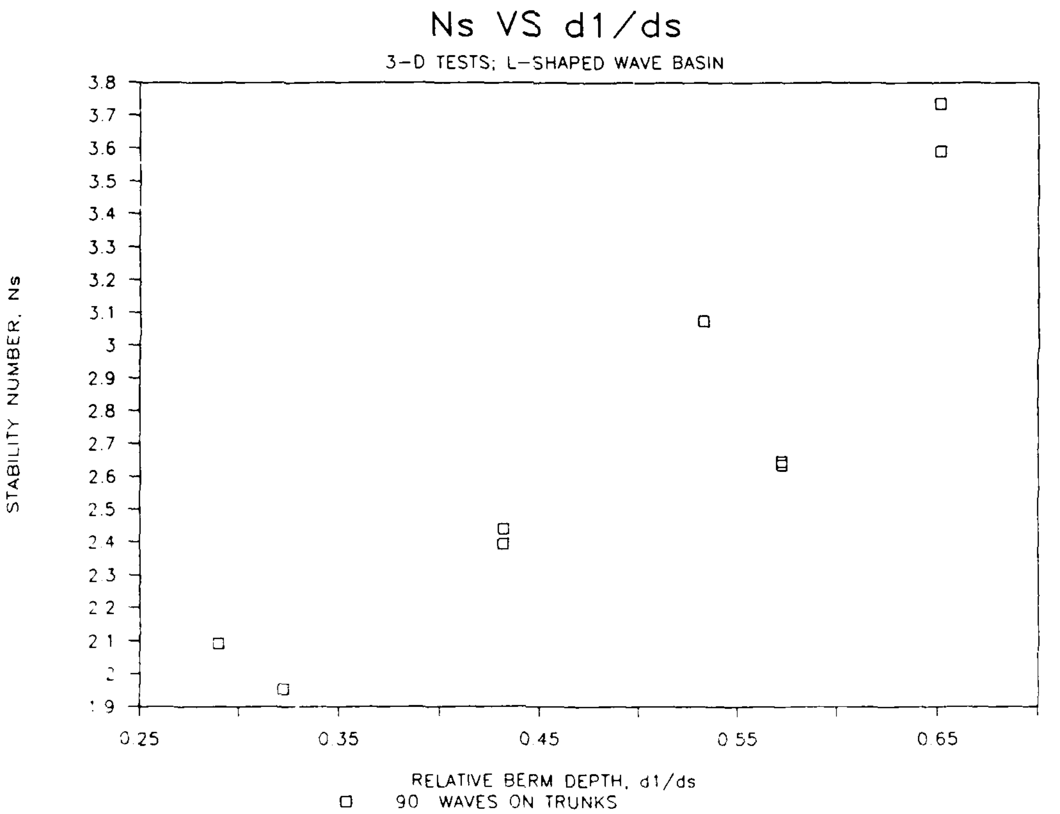
Ns VS B/L1
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
3.8
3.7
3.6
3.5
3.4
3.3
3.2
3.1
3
2.9
2.8
2.7
2.6
2.5
2.4
2.3
2.2
2.1
2
1.9
0.045
0.055
0.065
0.075
0.085
0.095
RELATIVE BERM LENGTH, B/L1
90 WAVES ON TRUNKS

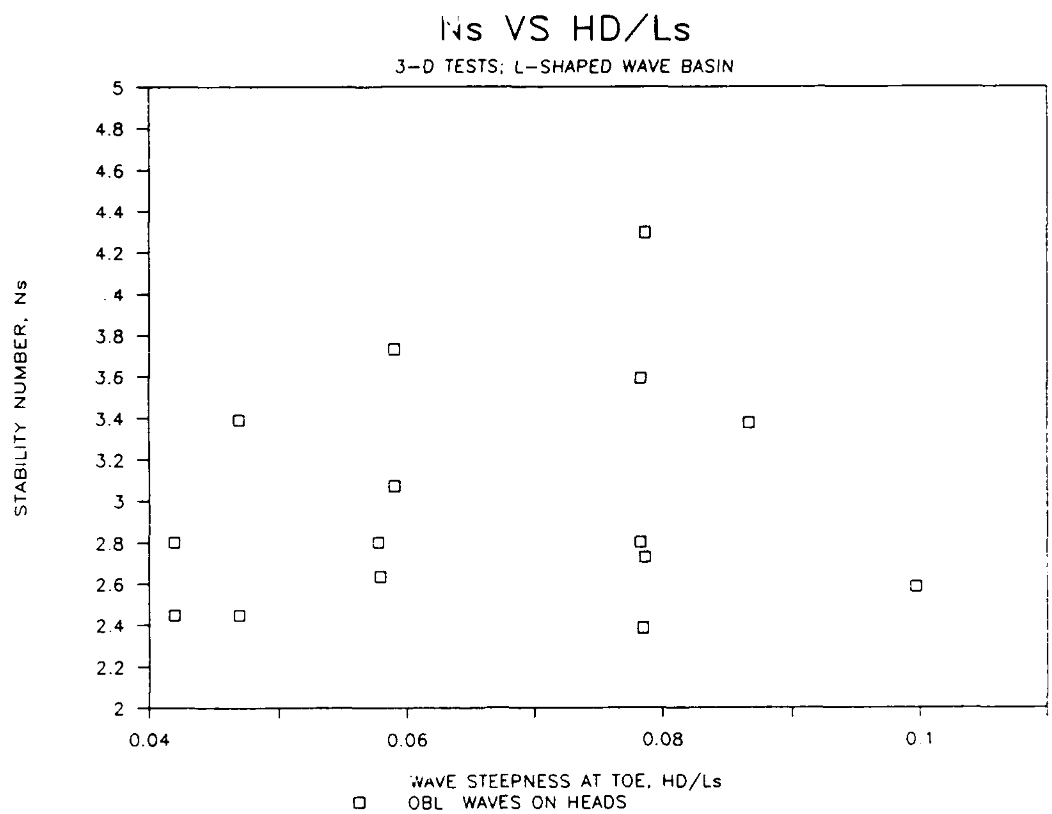


Ns VS ds/Ls
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
3.8
3.7
3.6
3.5
3.4
3.3
3.2
3.1
3
2.9
2.8
2.7
2.6
2.5
2.4
2.3
2.2
2.1
2
1.9
0.03
0.05
0.07
0.09
0.11
0.13
RELATIVE WATER DEPTH AT TOE, ds/Ls
90 WAVES ON TRUNKS

PLATE 8


Ns VS d1/L1
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
RELATIVE WATER DEPTH,TOP OF BERM, d1/L1
90 WAVES ON TRUNKS


Ns VS d1/ds
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
RELATIVE BERM DEPTH, d1/ds
90 WAVES ON TRUNKS


Ns VS HD/Ls
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
5
4.8
4.6
4.4
4.2
4
3.8
3.6
3.4
3.2
3
2.8
2.6
2.4
2.2
2
0.04
0.06
0.08
0.1
WAVE STEEPNESS AT TOE, HD/Ls
OBL WAVES ON HEADS

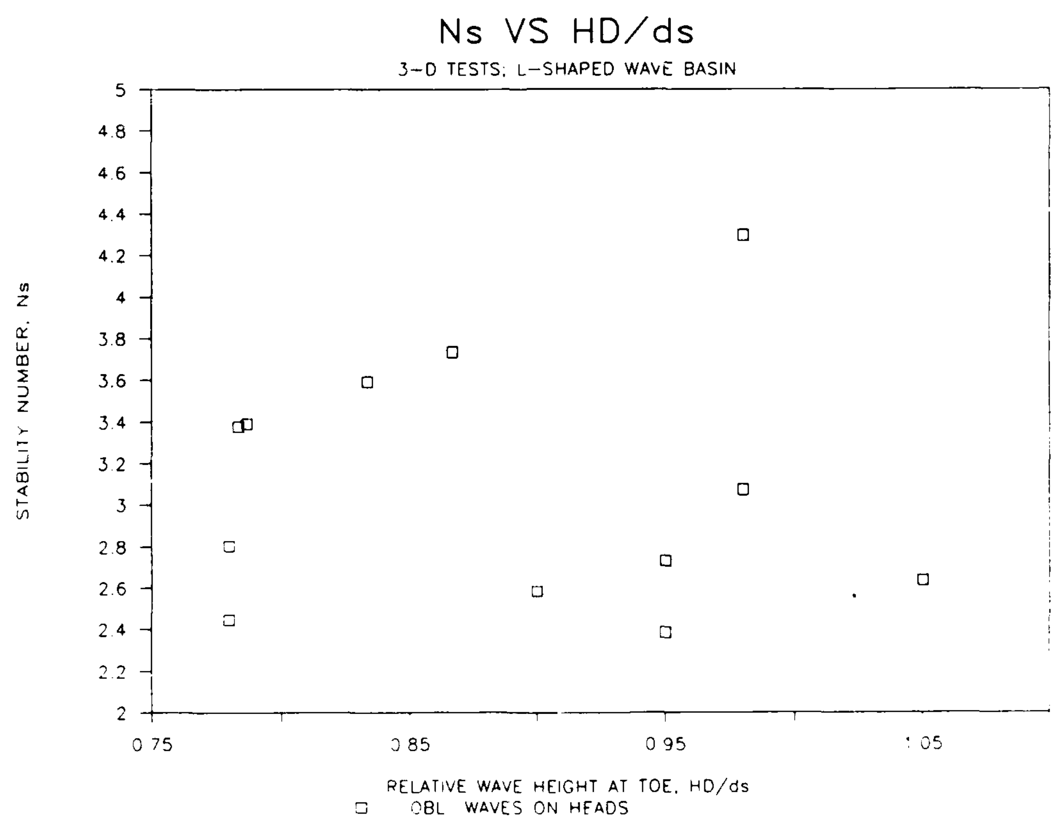
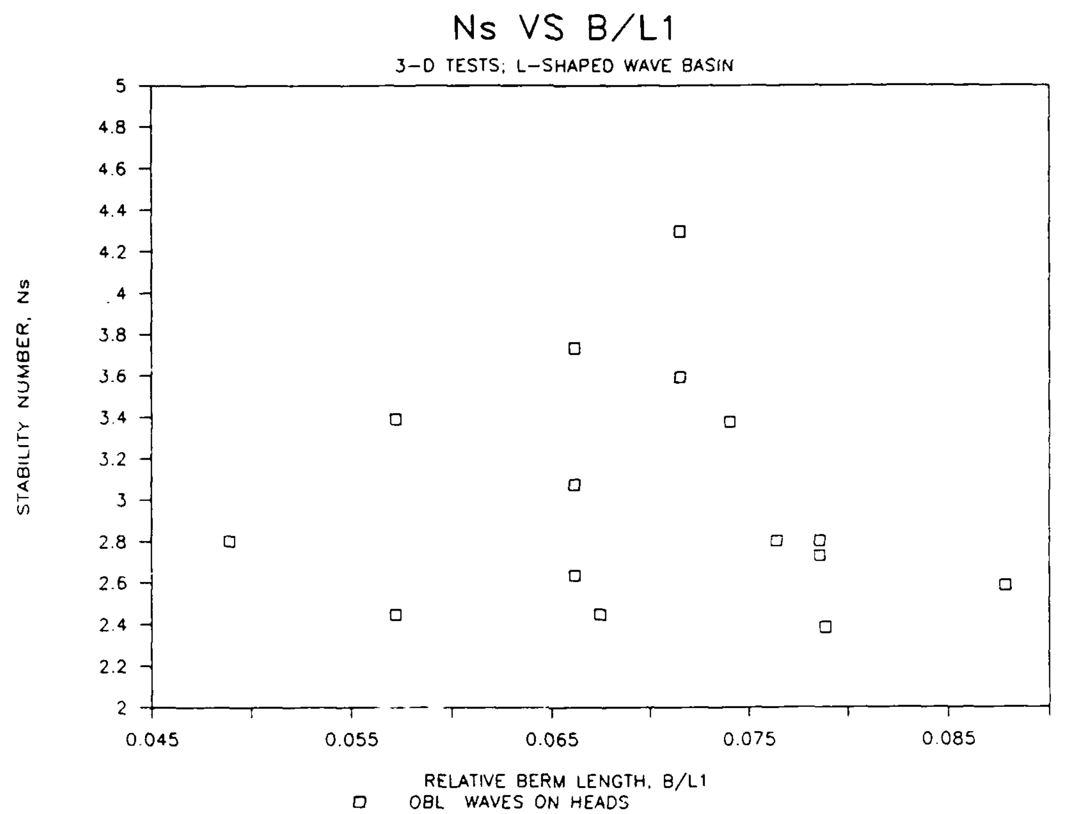
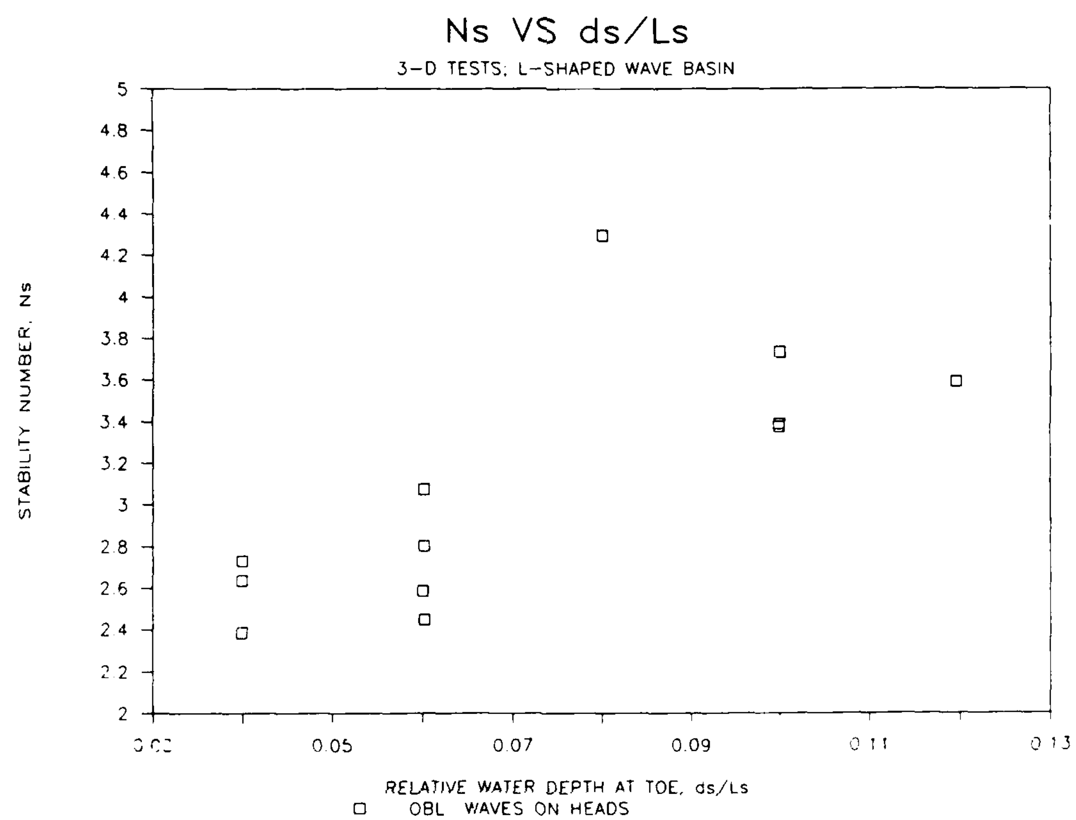
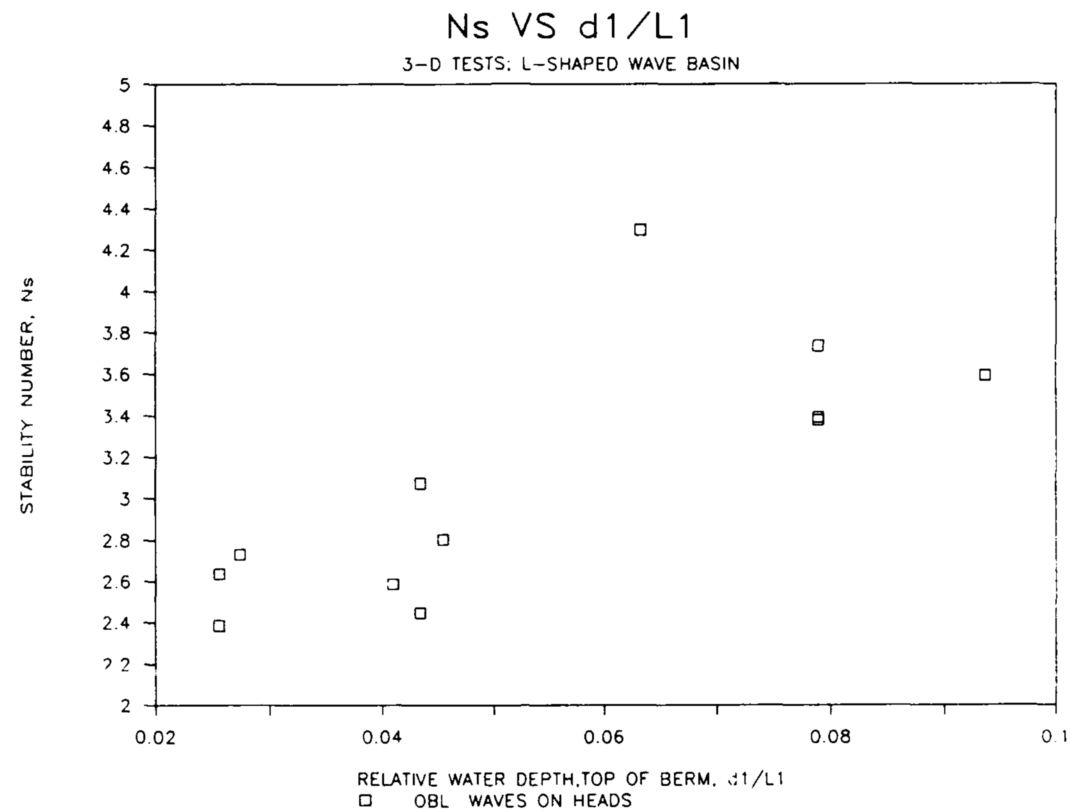

Ns VS HD/ds
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
5
4.8
4.6
4.4
4.2
4
3.8
3.6
3.4
3.2
3
2.8
2.6
2.4
2.2
2
0.75
0.85
0.95
1.05
RELATIVE WAVE HEIGHT AT TOE, HD/ds
OBL WAVES ON HEADS

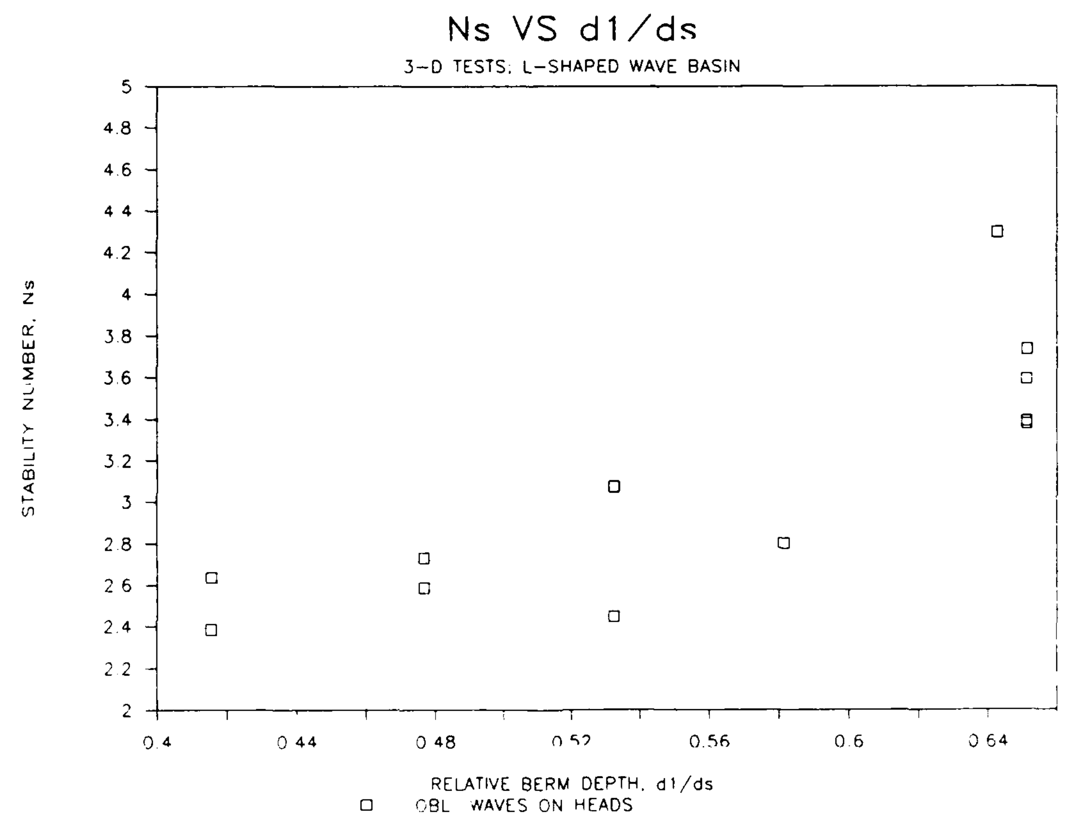
PLATE 10


Ns VS B/L1
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
RELATIVE BERM LENGTH, B/L1
OBL WAVES ON HEADS


Ns VS ds/Ls
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
RELATIVE WATER DEPTH AT TOE, ds/Ls
OBL WAVES ON HEADS

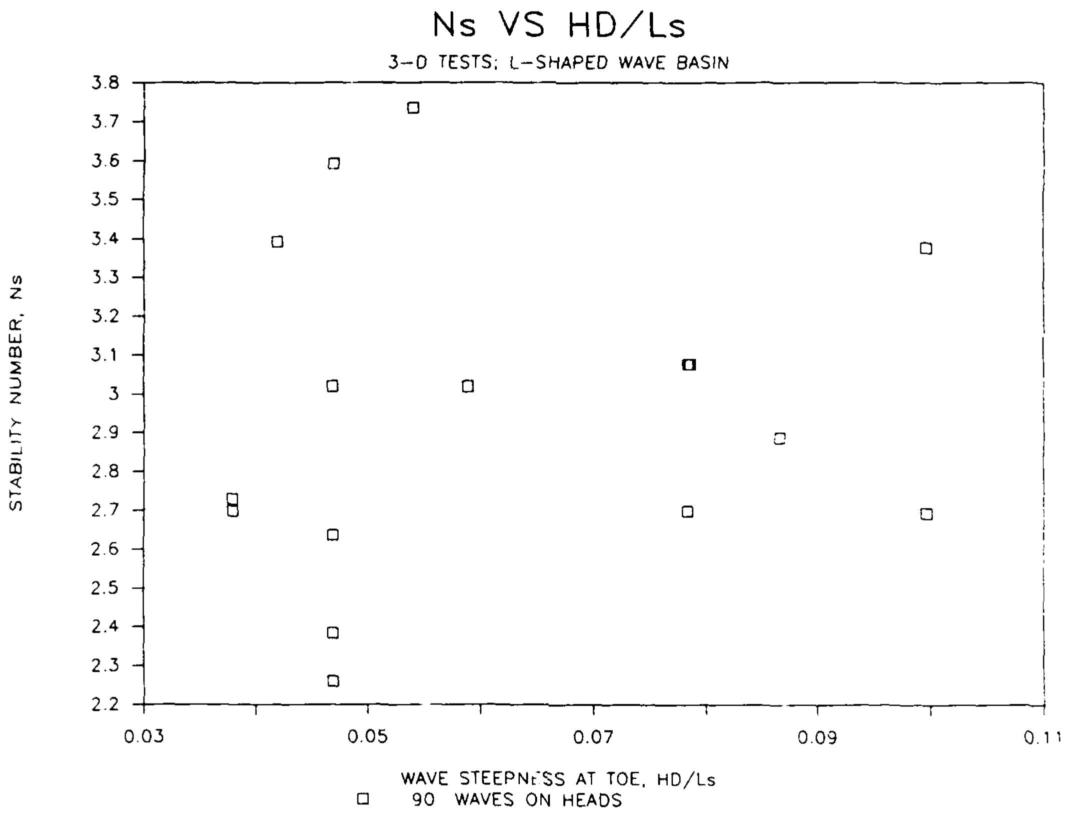


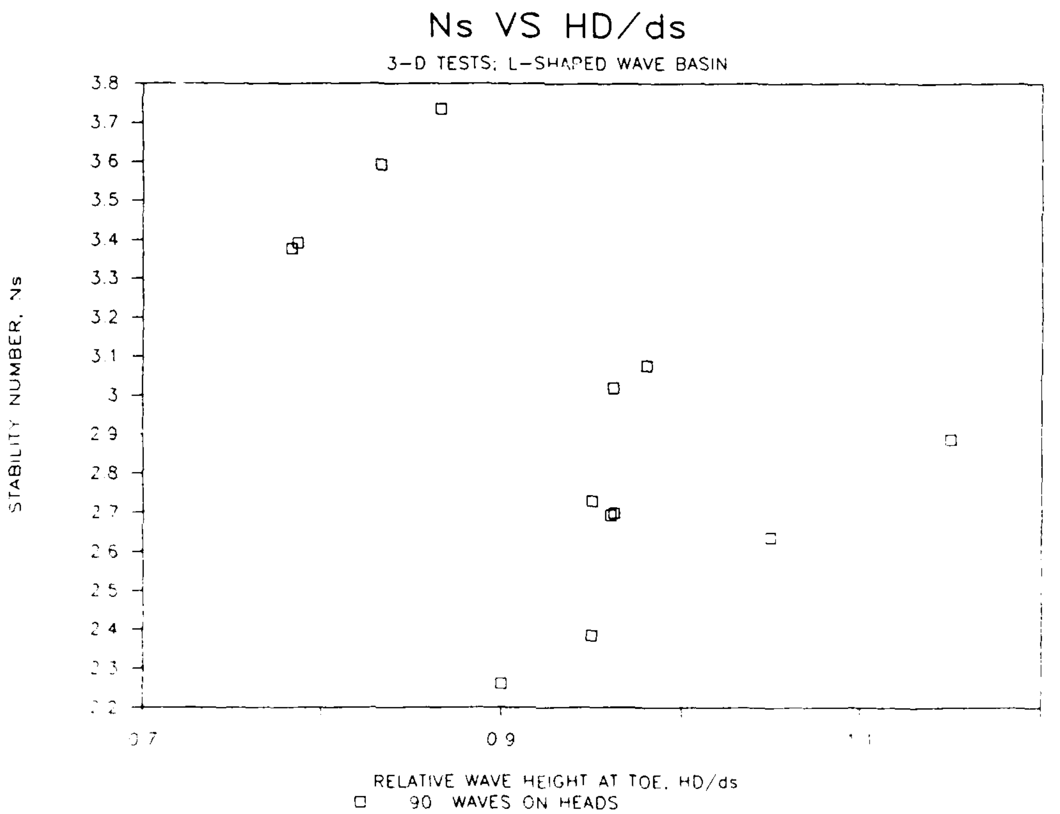
Ns VS d1/L1
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
5
4.8
4.6
4.4
4.2
4
3.8
3.6
3.4
3.2
3
2.8
2.6
2.4
2.2
2
0.02
0.04
0.06
0.08
0.1
RELATIVE WATER DEPTH,TOP OF BERM, d1/L1
OBL WAVES ON HEADS

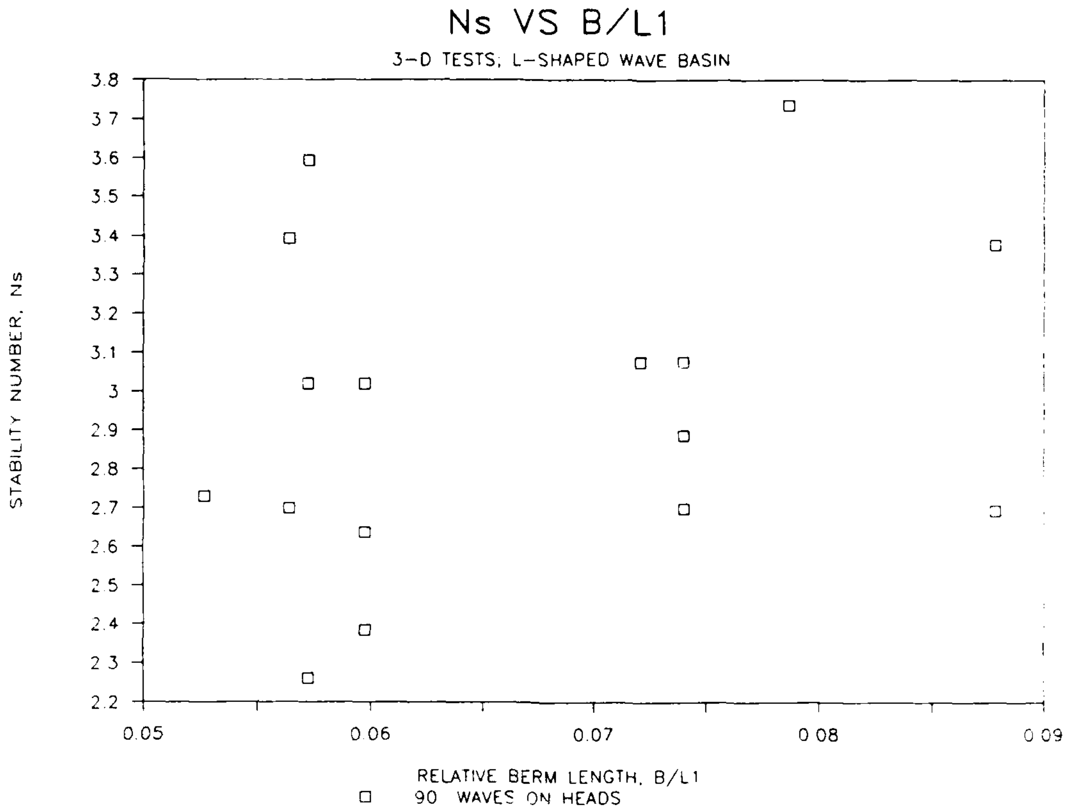


Ns VS d1/ds
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
5
4.8
4.6
4.4
4.2
4
3.8
3.6
3.4
3.2
3
2.8
2.6
2.4
2.2
2
0.4
0.44
0.48
0.52
0.56
0.6
0.64
RELATIVE BERM DEPTH, d1/ds
OBL WAVES ON HEADS

PLATE 12


Ns VS HD/Ls
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
3.8
3.7
3.6
3.5
3.4
3.3
3.2
3.1
3
2.9
2.8
2.7
2.6
2.5
2.4
2.3
2.2
0.03
0.05
0.07
0.09
0.11
WAVE STEEPNESS AT TOE, HD/Ls
90 WAVES ON HEADS


Ns VS HD/ds
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
3.8
3.7
3.6
3.5
3.4
3.3
3.2
3.1
3
2.9
2.8
2.7
2.6
2.5
2.4
0.9
RELATIVE WAVE HEIGHT AT TOE, HD/ds
90 WAVES ON HEADS


Ns VS B/L1
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
RELATIVE BERM LENGTH, B/L1
90 WAVES ON HEADS

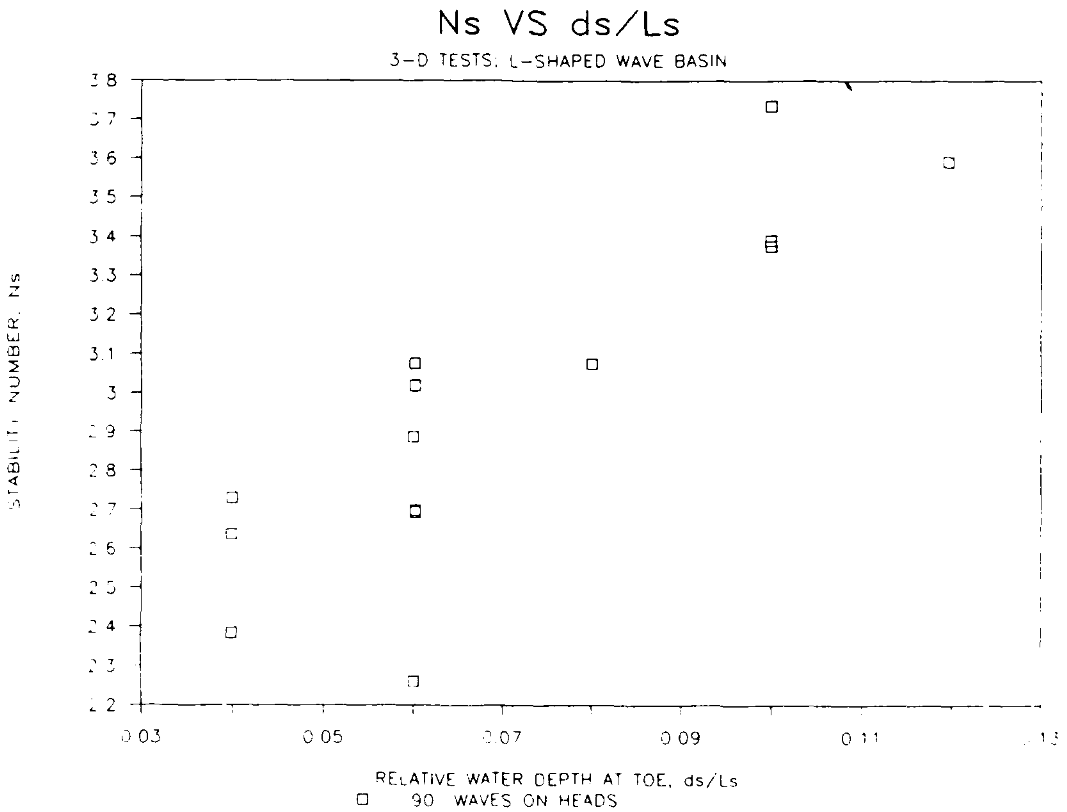
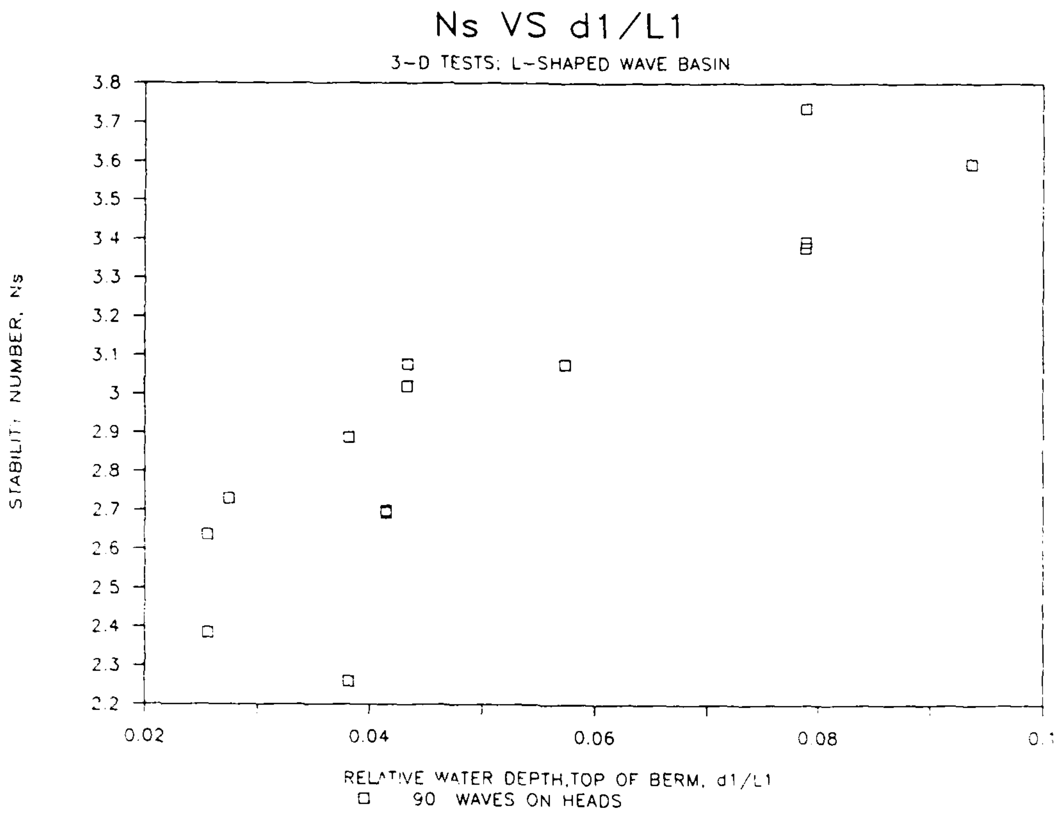
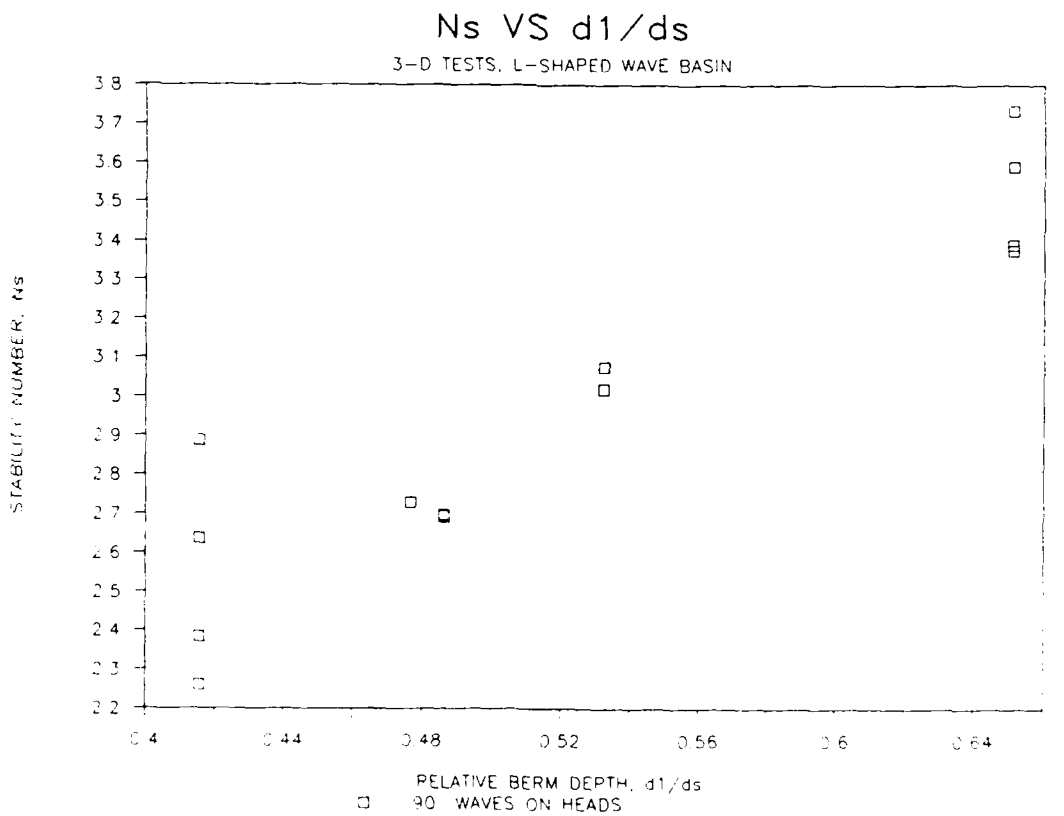

Ns VS ds/Ls
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
RELATIVE WATER DEPTH AT TOE, ds/Ls
90 WAVES ON HEADS

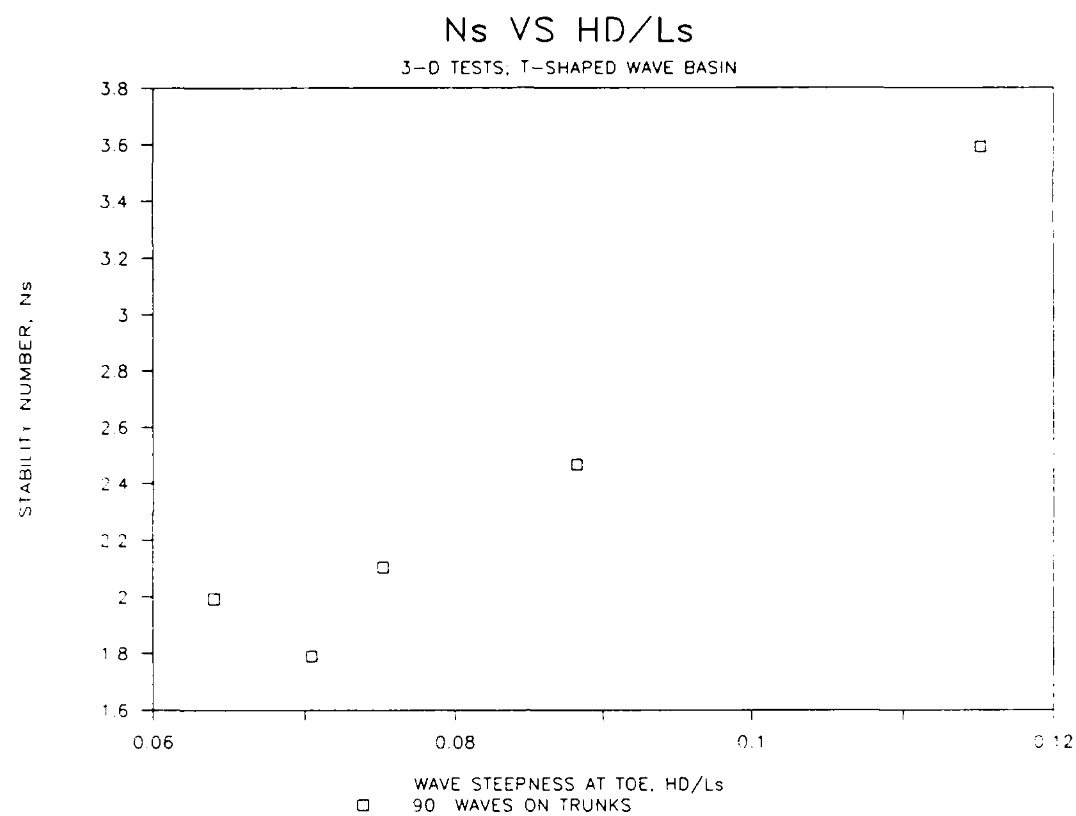
PLATE 14



Ns VS d1/L1
3-D TESTS; L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
3.8
3.7
3.6
3.5
3.4
3.3
3.2
3.1
3
2.9
2.8
2.7
2.6
2.5
2.4
2.3
2.2
0.02
0.04
0.06
0.08
0.1
RELATIVE WATER DEPTH,TOP OF BERM, d1/L1
90 WAVES ON HEADS



Ns VS d1/ds
3-D TESTS, L-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
3.8
3.7
3.6
3.5
3.4
3.3
3.2
3.1
3
2.9
2.8
2.7
2.6
2.5
2.4
2.3
2.2
0.4
0.44
0.48
0.52
0.56
0.6
0.64
RELATIVE BERM DEPTH, d1/ds
90 WAVES ON HEADS


Ns VS HD/Ls
3-D TESTS; T-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
3.8
3.6
3.4
3.2
3
2.8
2.6
2.4
2.2
2
1.8
1.6
0.06
0.08
0.1
0.12
WAVE STEEPNESS AT TOE, HD/Ls
90 WAVES ON TRUNKS

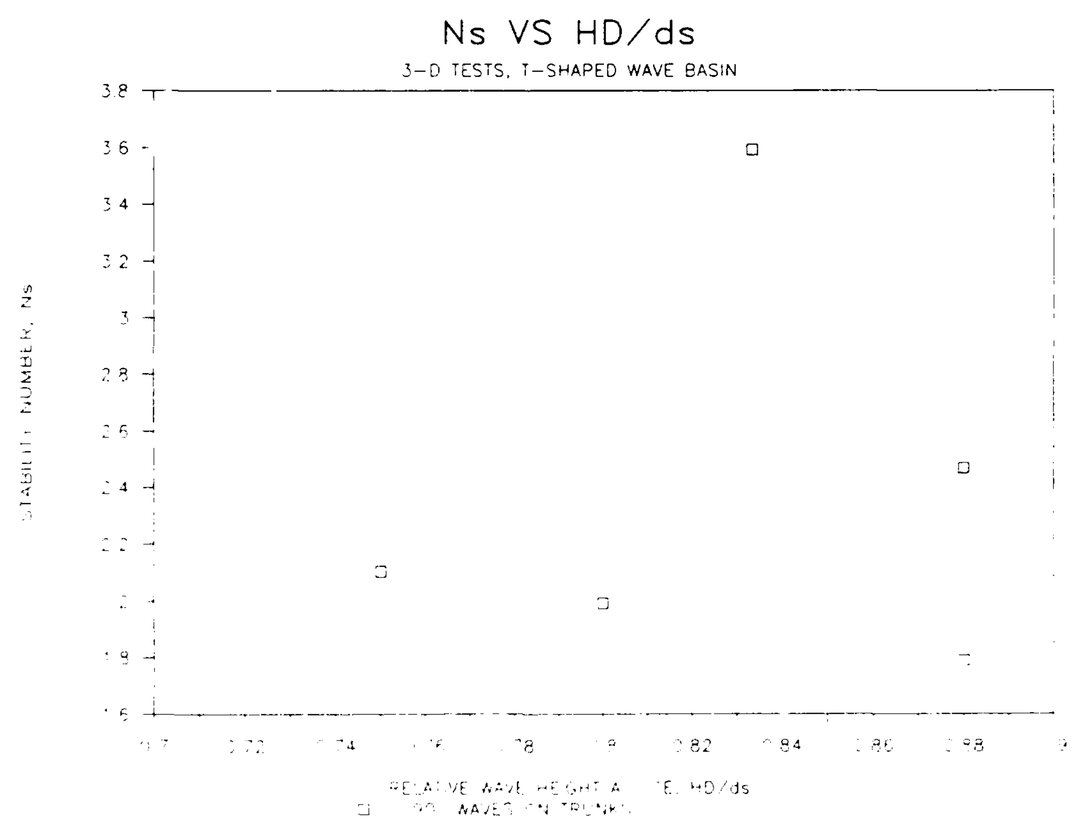
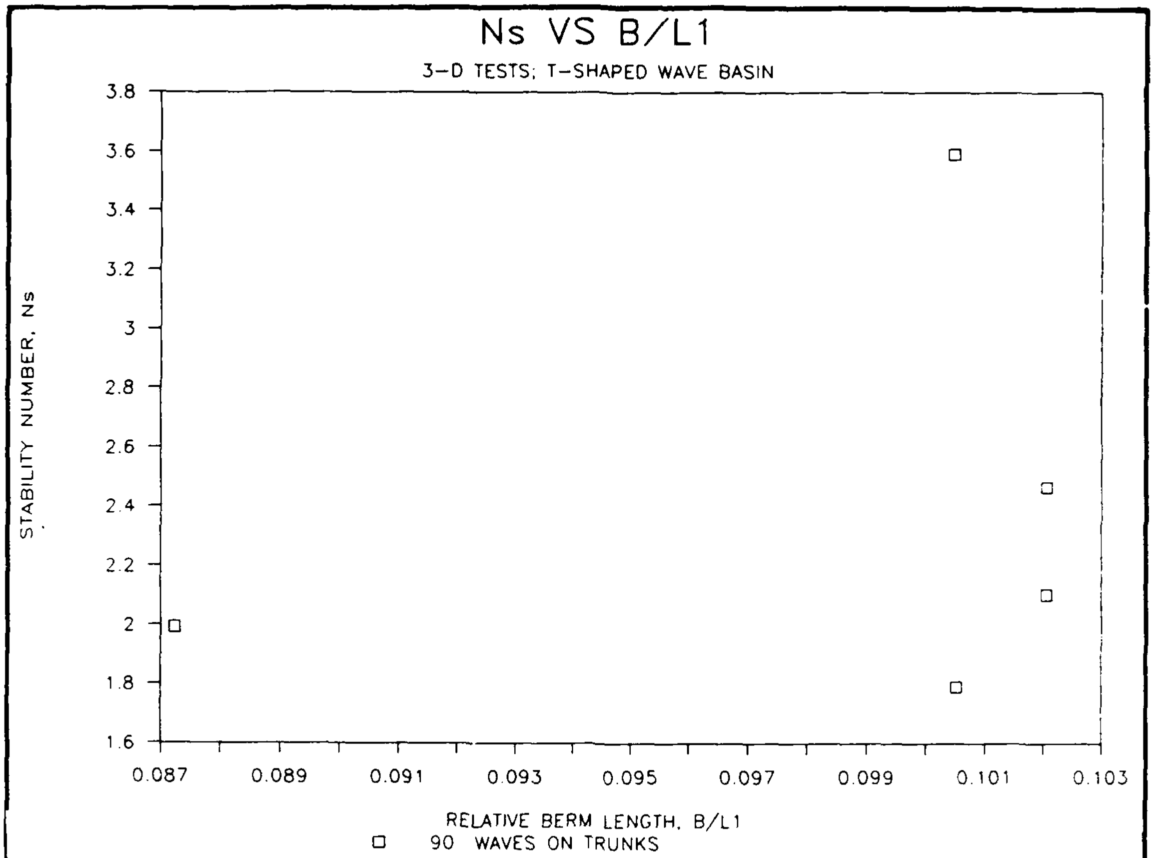
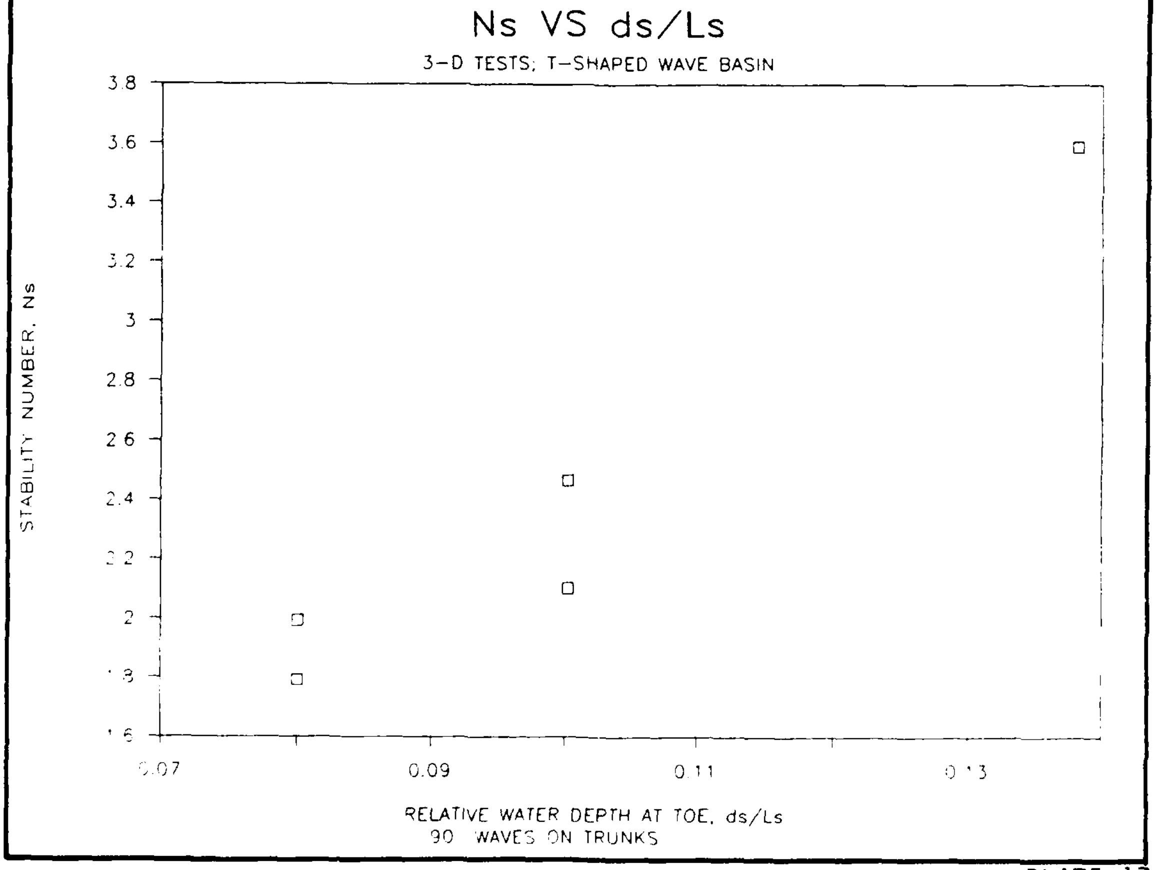

Ns VS HD/ds
3-D TESTS, T-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
3.8
3.6
3.4
3.2
3
2.8
2.6
2.4
2.2
2
1.8
1.6
0.82
0.84
0.88
0.9

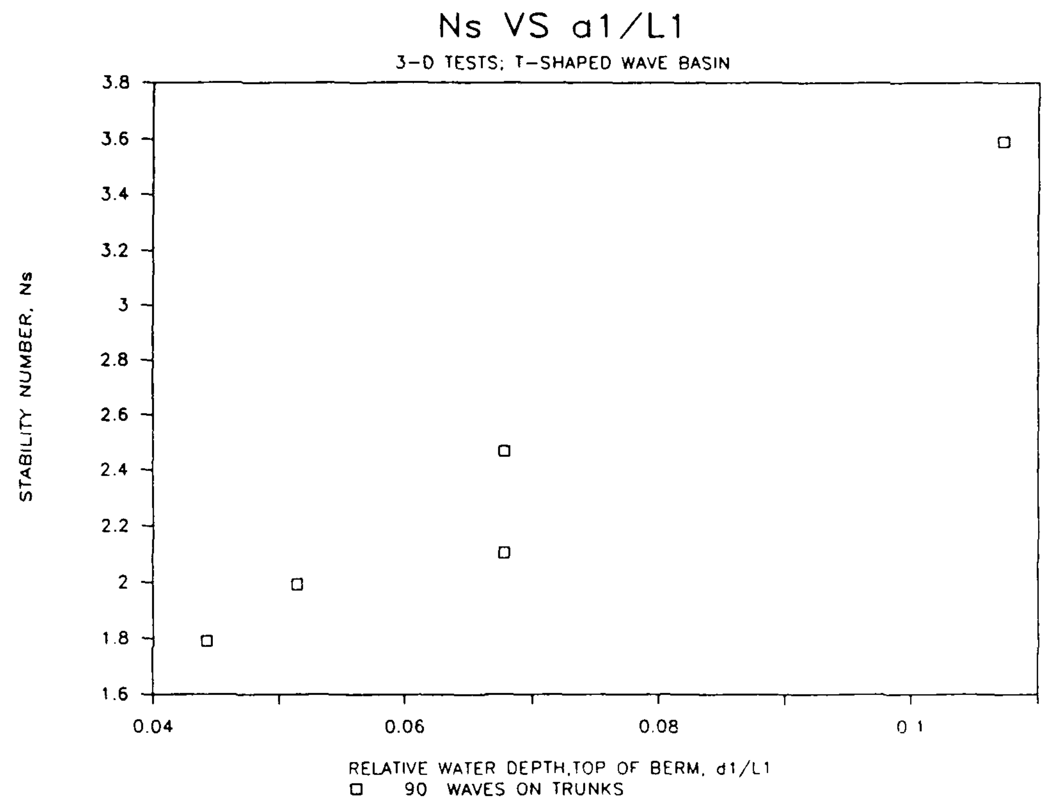
PLATE 16



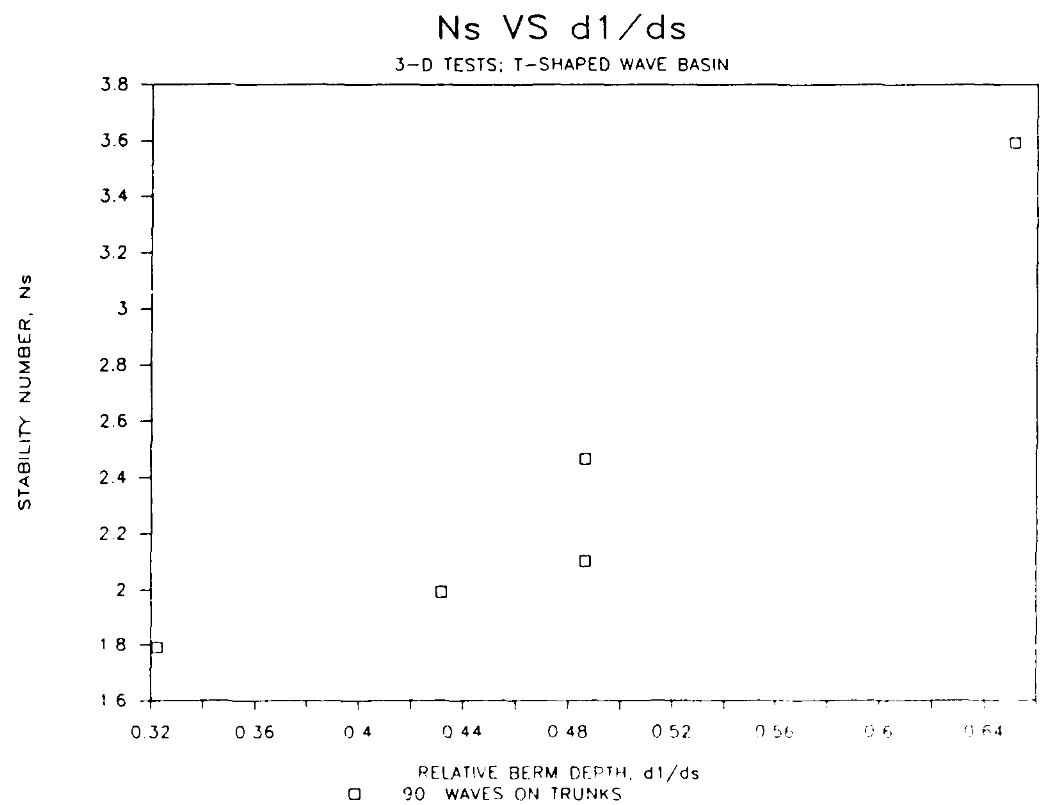
Ns VS B/L1
3-D TESTS; T-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
3.8
3.6
3.4
3.2
3
2.8
2.6
2.4
2.2
2
1.8
1.6
0.087
0.089
0.091
0.093
0.095
0.097
0.099
0.101
0.103
RELATIVE BERM LENGTH, B/L1
90 WAVES ON TRUNKS



Ns VS ds/Ls
3-D TESTS; T-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
3.8
3.6
3.4
3.2
3
2.8
2.6
2.4
2
0.09
RELATIVE WATER DEPTH AT TOE, ds/Ls
90 WAVES ON TRUNKS

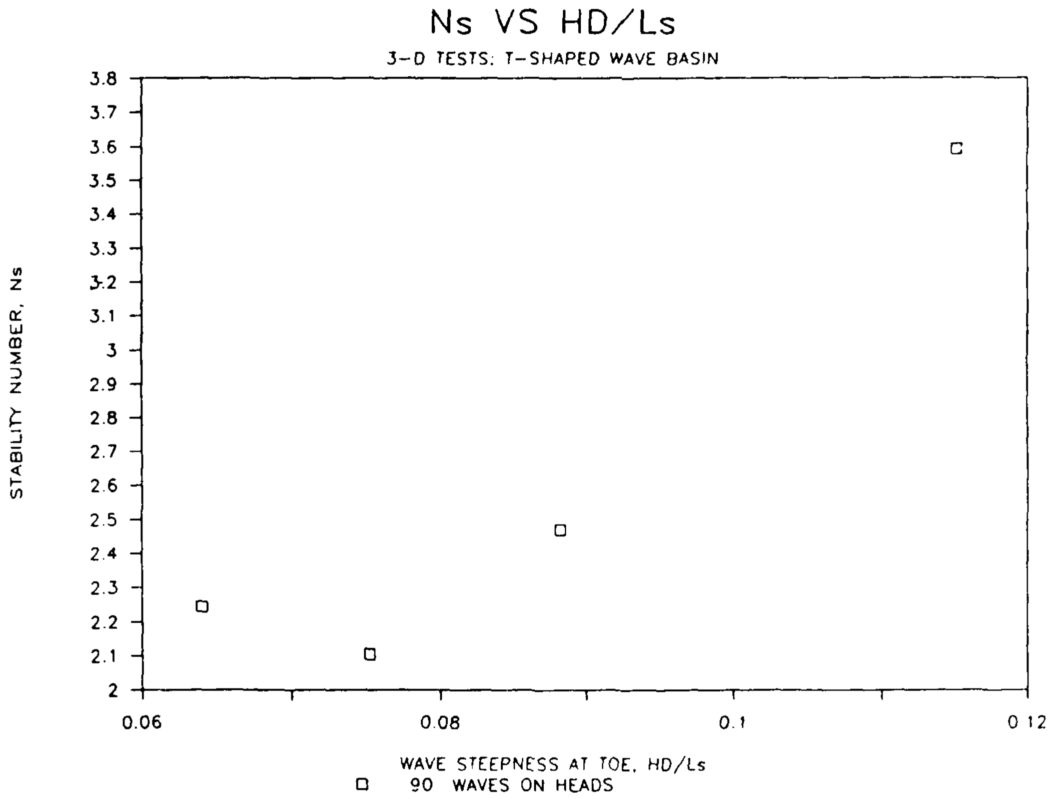


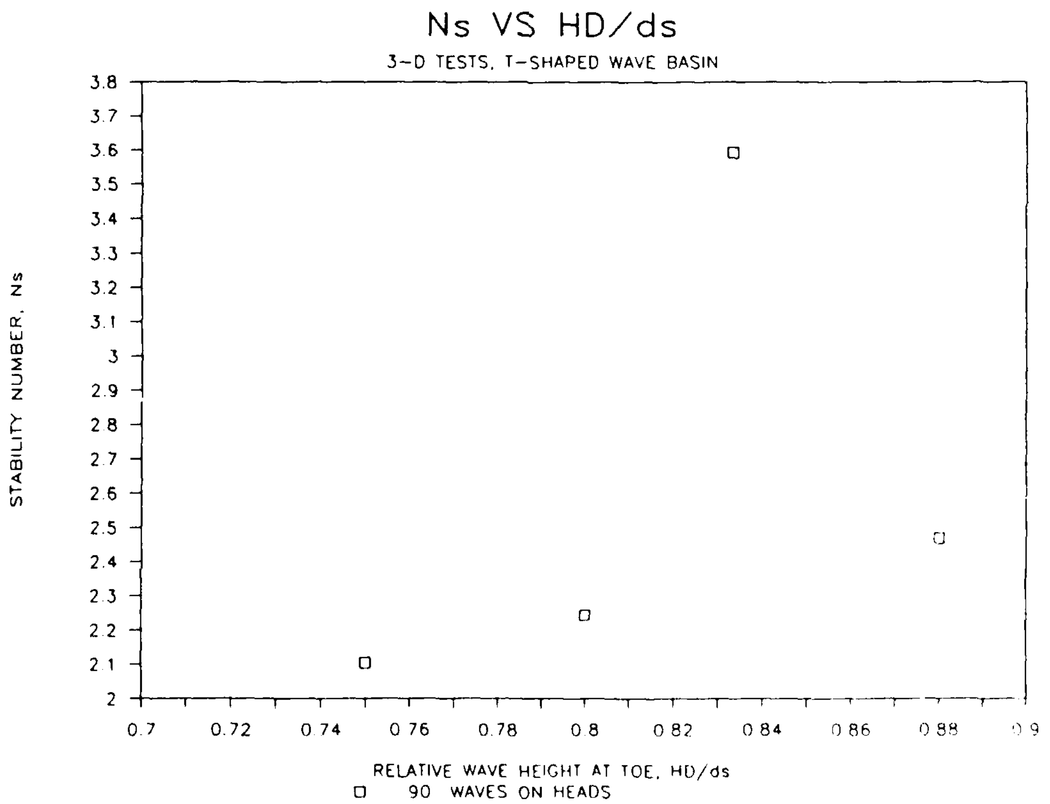
Ns VS a1/L1
3-D TESTS; T-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
3.8
3.6
3.4
3.2
3
2.8
2.6
2.4
2.2
2
1.8
1.6
0.04
0.06
0.08
0.1
RELATIVE WATER DEPTH,TOP OF BERM, d1/L1
90 WAVES ON TRUNKS

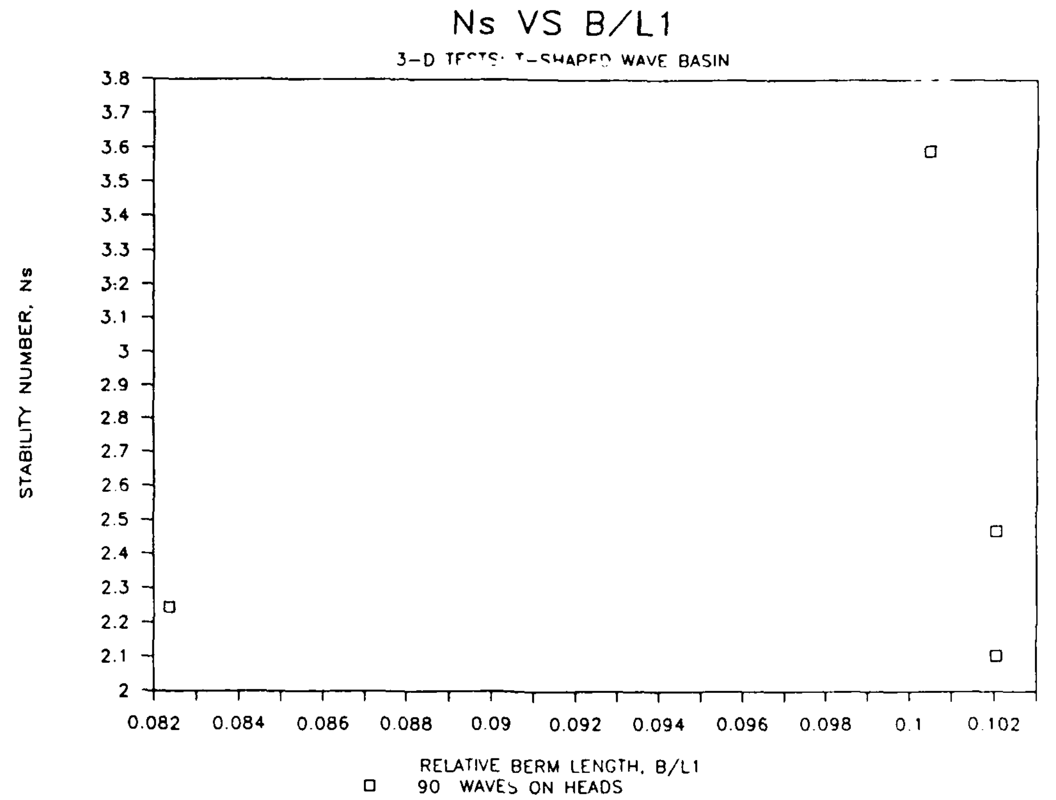


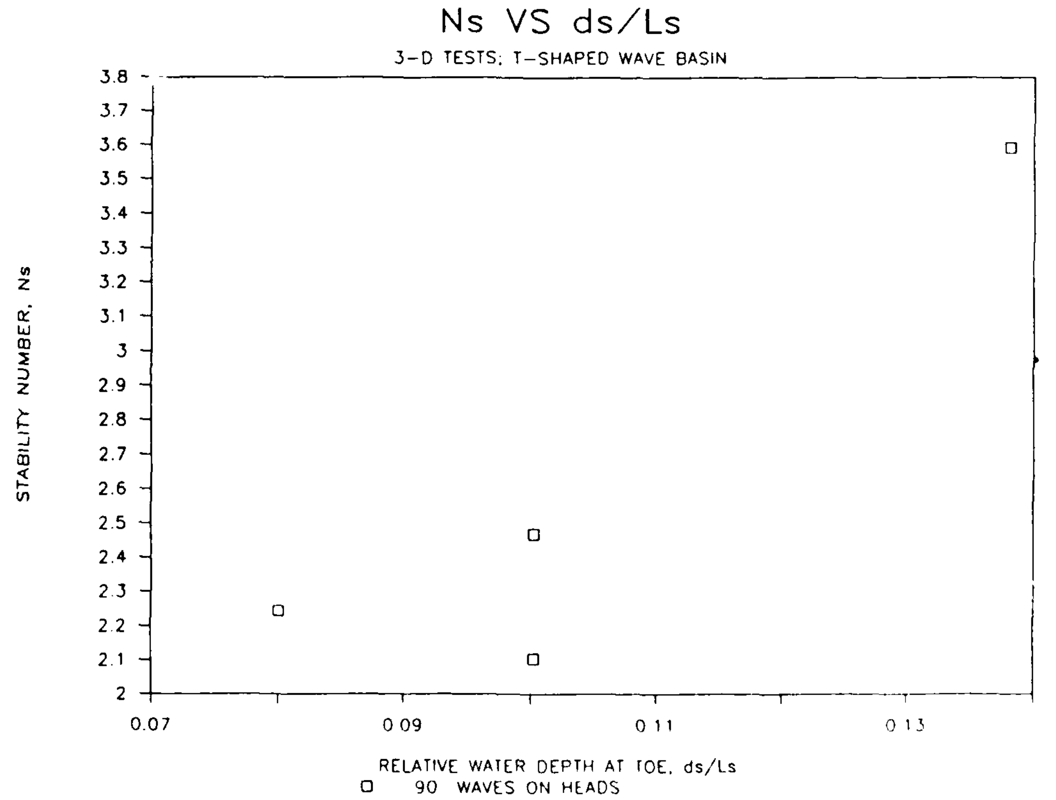
Ns VS d1/ds
3-D TESTS; T-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
3.8
3.6
3.4
3.2
3
2.8
2.6
2.4
2.2
2
1.8
1.6
0.32
0.36
0.4
0.44
0.48
0.52
0.56
0.6
0.64
RELATIVE BERM DEPTH, d1/ds
90 WAVES ON TRUNKS

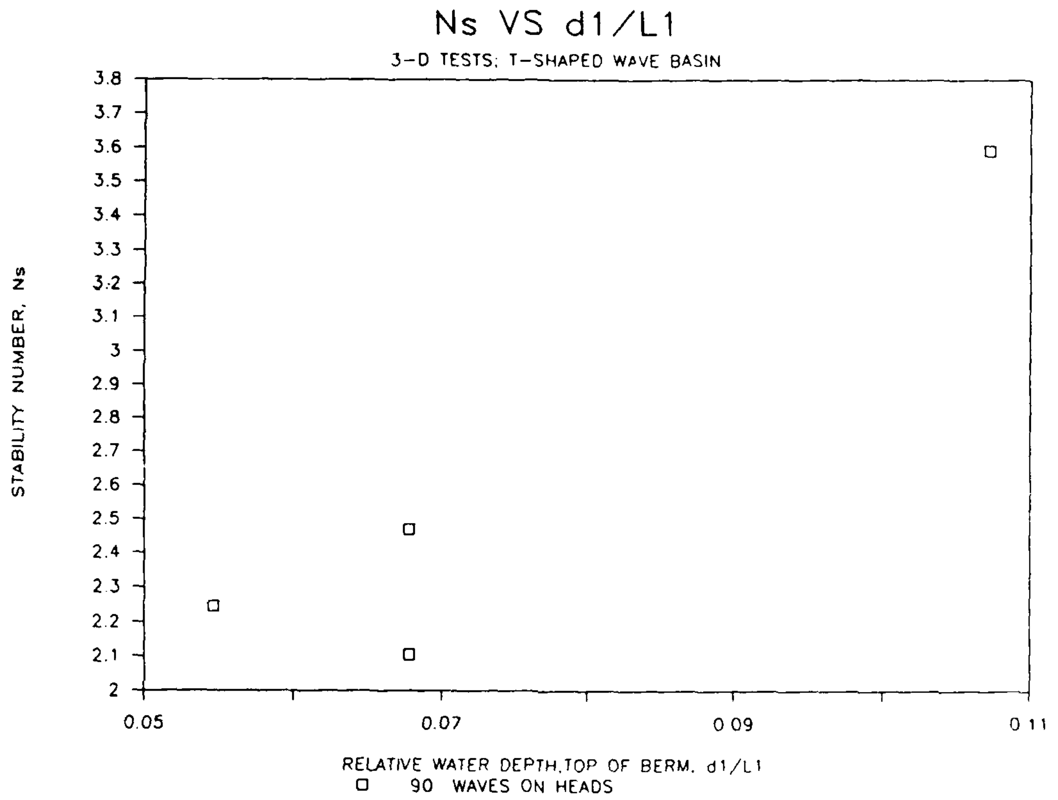
PLATE 18

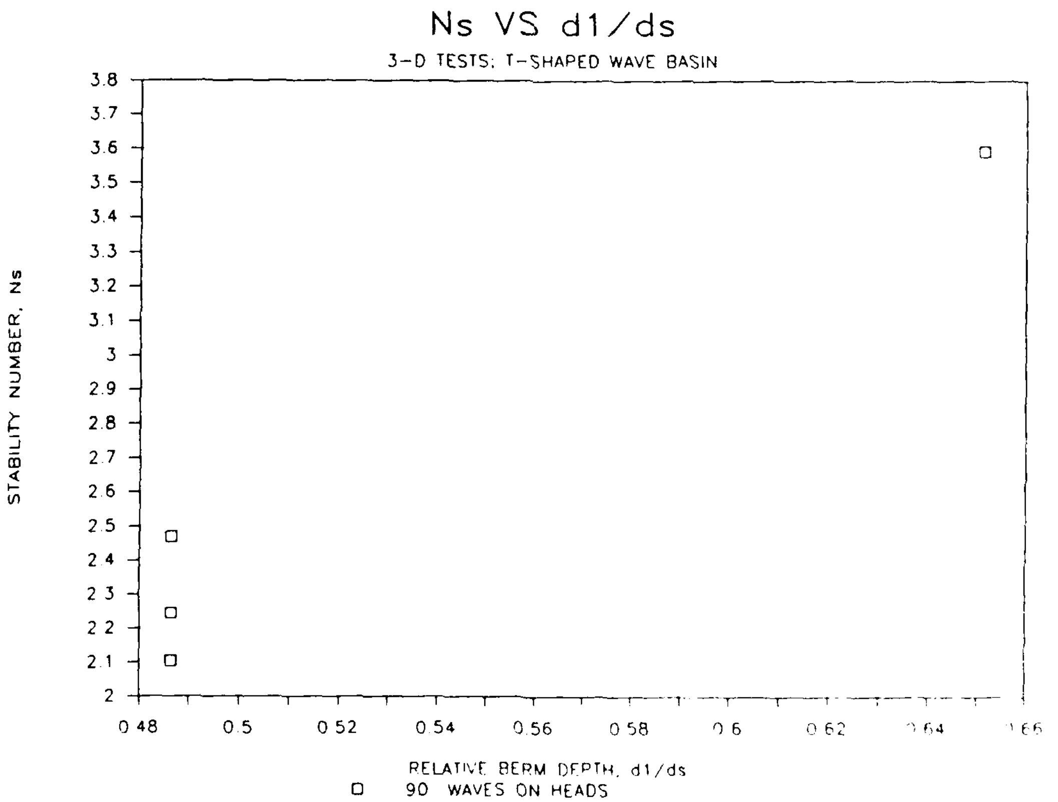

Ns VS HD/Ls
3-D TESTS: T-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
3.8
3.7
3.6
3.5
3.4
3.3
3.2
3.1
3
2.9
2.8
2.7
2.6
2.5
2.4
2.3
2.2
2.1
2
0.06
0.08
0.1
0.12
WAVE STEEPNESS AT TOE, HD/Ls
90 WAVES ON HEADS


Ns VS HD/ds
3-D TESTS, T-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
3.8
3.7
3.6
3.5
3.4
3.3
3.2
3.1
3
2.9
2.8
2.7
2.6
2.5
2.4
2.3
2.2
2.1
2
0.7
0.72
0.74
0.76
0.78
0.8
0.82
0.84
0.86
0.88
0.9
RELATIVE WAVE HEIGHT AT TOE, HD/ds
90 WAVES ON HEADS


Ns VS B/L1
3-D TESTS: T-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
3.8
3.7
3.6
3.5
3.4
3.3
3.2
3.1
3
2.9
2.8
2.7
2.6
2.5
2.4
2.3
2.2
2.1
2
0.082
0.084
0.086
0.088
0.09
0.092
0.094
0.096
0.098
0.1
0.102
RELATIVE BERM LENGTH, B/L1
90 WAVES ON HEADS


Ns VS ds/Ls
3-D TESTS: T-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
3.8
3.7
3.6
3.5
3.4
3.3
3.2
3.1
3
2.9
2.8
2.7
2.6
2.5
2.4
2.3
2.2
2.1
2
0.07
0.09
0.11
0.13
RELATIVE WATER DEPTH AT TOE, ds/Ls
90 WAVES ON HEADS


Ns VS d1/L1
3-D TESTS; T-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
RELATIVE WATER DEPTH,TOP OF BERM, d1/L1
90 WAVES ON HEADS


Ns VS d1/ds
3-D TESTS; T-SHAPED WAVE BASIN
STABILITY NUMBER, Ns
RELATIVE BERM DEPTH, d1/ds
90 WAVES ON HEADS

APPENDIX A: NOTATION

γ	Spectral shape parameter
γ_r	Unit weight (saturated surface dry) of individual stone, pcf
γ_w	Unit weight of water in which stone are placed, pcf
θ	Angle measured relative to horizontal
σ_{low}	Shape parameter for face of spectral energy density curve for f less than f_p
σ_{high}	Shape parameter for face of spectral energy density curve for f greater than f_p
B	Berm length
C_r	Wave reflection coefficient equal to H_i/H_r
d_s	Water depth at structure toe, ft
f	Frequency, Hz
f_p	Peak frequency, Hz
d_1	Water depth at top of berm stone or buttressing stone, ft
H	Test wave height, ft
H_D	Design wave height, ft
H_i	Incident wave height, ft
H_r	Reflected wave height, ft
k_Δ	Armor stone shape coefficient (equal to 1.0 for rough angular stone)
L_s	Wave length in water depth d_s , ft
L_1	Wave length in water depth d_1 , ft
N_s	Toe berm stone and toe buttressing stone stability number
S_r	Specific gravity of armor stone relative to water in which it is placed
t	Thickness of armor, ft
T	Wave period, sec
W	Weight of individual armor stone, lb
W_{50}	Weight of median size armor stone, lb
B/L_1	Relative berm length
d_1/d_s	Relative berm stone or buttressing stone depth
d_1/L_1	Relative water depth at top of berm stone or buttressing stone
d_s/L_s	Relative water depth at toe
H_D/L_s	Wave steepness at structure toe
H_D/d_s	Relative wave height at structure toe