Technical Report CERC-94-13
September 1994

US Army Corps
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Investigation of Wave Grouping Effects on the Stability of Stone-Armored, Rubble-Mound Breakwaters

by Robert D. Carver, Brenda J. Wright

WES

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Prepared for Headquarters, U.S. Army Corps of Engineers

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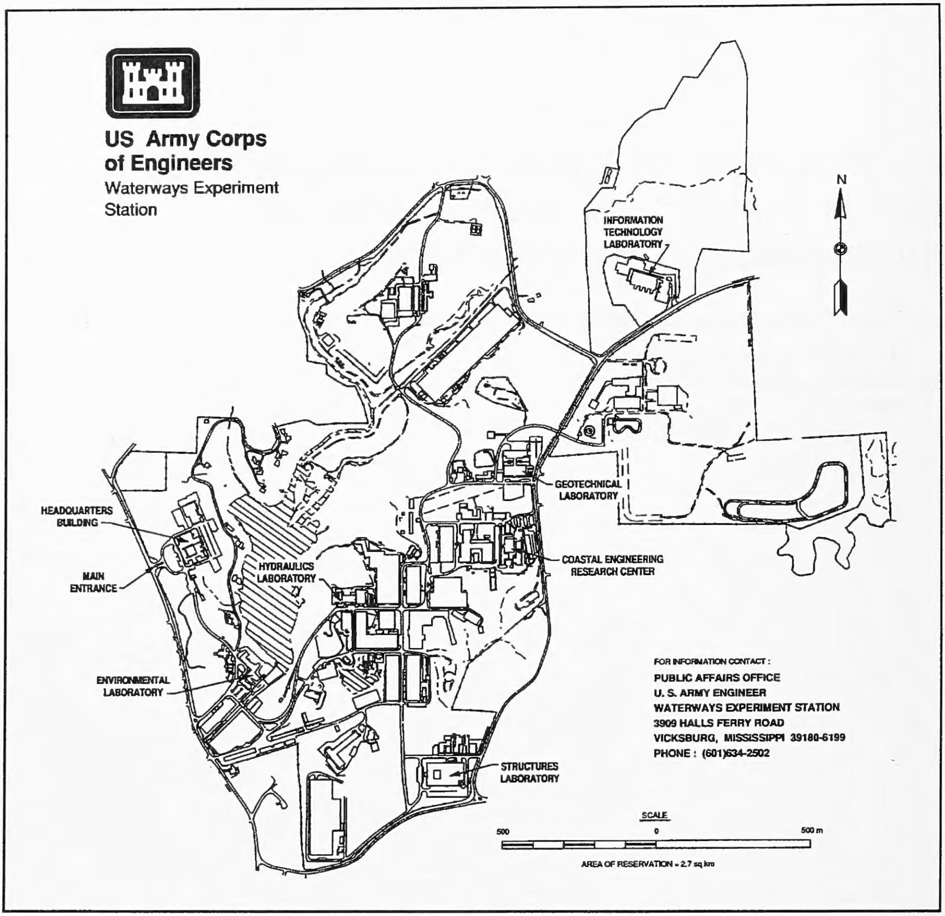

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Contents

Preface

Authority for the U.S. Army Engineer Waterways Experiment Station (WES), Coastal Engineering Research Center (CERC), to conduct this study was granted by Headquarters, U.S. Army Corps of Engineers (HQUSACE), under Work Unit 32534, "Breakwater Stability - A New Design Approach," Coastal Structure Evaluation and Design Program, Coastal Engineering Area of Civil Works Research and Development. HQUSACE Technical Monitors for this research were Messrs. John H. Lockhart, Jr., Barry W. Holliday, John F. C. Sanda, and John G. Housley. CERC Program Manager is Ms. Carolyn Holmes.

The study was conducted by personnel of CERC under the general direction of Dr. James R. Houston, Director, CERC, and Mr. Charles C. Calhoun, Jr., Assistant Director, CERC. Direct supervision was provided by Messrs. C. E. Chatham, Chief, Wave Dynamics Division (WDD), and D. Donald Davidson, Chief, Wave Research Branch (WRB). This report was prepared by Mr. Robert D. Carver, Principal Investigator and Ms. Brenda J. Wright, Engineering Technician, WRB, WDD. The model was operated by Ms. Wright.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

Conversion Factors, Non-SI to SI Units of Measurement

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

Multiply	By	To Obtain
cubic feet	0.02831685	cubic meters
degrees (angle)	0.01745329	radians
feet	0.3048	meters
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter
square feet	0.09290304	square meters

1 Introduction

Background

High sea waves tend to appear in groups rather than individually. Because of the nature of wave grouping, it appears that it may be an important influence on the stability of rubble-mound structures.

A succession of high waves that exceeds some arbitrary threshold value (typically mean or significant wave height) is called a run of high waves, and the number of waves in this run is the run length. The total or complete run is the combination of the run of high waves followed by the run of low waves. Reference to a wave group assumes that a run of high waves is intended. In the present investigation, a group of waves is defined as three or more successive waves that have heights equal to or exceeding the significant wave height of the entire test run. Also, the grouping intensity (GI) is defined as the number of these groups per hour of test waves.

Purpose of Study

The purpose of the present investigation is to obtain a better understanding of the effects of wave grouping on the stability of stone armor when used on breakwater trunks.

Approach

Previous breakwater stability investigations conducted by Carver (1983) and Carver and Wright (1991) have shown that relative depth (d/L) and relative wave height (H/d) are two of the most important dimensionless variables influencing breakwater stability with minimum stability occurring at the lower values of d/L and higher values of H/d, i.e., longer wave periods in shallower water. Therefore, initial tests were conducted with period depth combinations that are in the minimum stability range.

The amount of groupiness in a series of waves is influenced by the spectral width parameter (γ). Previous work has shown that groupiness increases as gamma increases and the spectra become narrower or more sharply peaked. Therefore, tests were initiated using gamma values of 1, 10, and 20.

2 Tests and Results

Stability Scale Effects

If the absolute sizes of experimental breakwater materials and wave dimensions become too small, flow around the armor units enters the laminar regime; and the induced drag forces become a direct function of the Reynolds number. Under these circumstances prototype phenomena are not properly simulated, and stability scale effects are induced. Hudson (1975) presents a detailed discussion of the design requirements necessary to ensure the preclusion of stability scale effects in small-scale breakwater tests and concludes that scale effects will be negligible if the Reynolds stability number (R_N) expressed in the equation below is equal to or greater than 3×10^4.

$$R_N = \frac{g^{1/2} H^{1/2} 1_a}{\nu}$$

where

g = acceleration due to gravity, ft/sec^2

H = wave height, ft

1_a = characteristic length of armor unit, ft

ν = kinematic viscosity

For all tests reported herein, the sizes of experimental armor and wave dimensions were selected such that scale effects were insignificant (i.e., R_N was greater than 3×10^4).

Method of Constructing Test Sections

All experimental breakwater sections were constructed to reproduce as closely as possible results of the usual methods of constructing full-scale breakwaters. The core material was dampened as it was dumped by bucket or shovel into the flume and was compacted with hand trowels to simulate natural consolidation resulting from wave action during construction of the prototype structure. Once the core material was in place, it was sprayed with a low-velocity water hose to ensure adequate compaction of the material. The underlayer stone then was added by shovel and smoothed to grade by hand or with trowels. Armor units used in the cover layers were placed in a random manner corresponding to work performed by a general coastal contractor; i.e., they were individually placed but were laid down without special orientation or fitting. After each test the armor units were removed from the breakwater, all of the underlayer stones were replaced to the grade of the original test section, and the armor was replaced.

Test Equipment and Materials

Equipment used

Tests were conducted in a concrete wave flume, 11 ft wide, 6 ft deep, and 245 ft long.[1] The cross section of the tank in the vicinity of the structures was partitioned into two 3-ft-wide channels and two 2.5-ft-wide channels (Figure 1). Identical test sections were constructed in the 3-ft channels while wave absorption was achieved in the 2.5-ft channels, which were left empty. The flume is equipped with an electro-hydraulic, horizontal-displacement wave generator capable of producing monochromatic and irregular waves of various periods and heights. Changes in water surface elevation as a function of time (wave heights) were measured by electrical capacitance-type gauges at selected locations. The wave machine was controlled by and data were collected with an on-line Dec MicroVax I computer. Data then were transferred to a Vax 3600 for analysis.

Materials used

Rough hand-shaped granitic stone (W_a) with an average length of about two times its width, average weight of 0.38 lb, and a specific weight of 167 pcf was used. Sieve-sized angular-shaped limestone (unit weight = 165 pcf) was used for the underlayers and core.

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[1] A table of factors for converting non-SI units of measurement to SI units is presented on page v.

Selection of Test Conditions

All tests were conducted with a Texel, Marsen, Arsloe (TMA) spectrum. For tests described herein, the wave flume was calibrated for periods of 1.5, 2.25, 3.0, and 4.0 sec in water depths of 0.80 and 1.60 ft, thus assuring a range of relative depths (d/L's) that is consistent with the majority of conditions to which prototype structures are exposed. Goda and Suzuki's (1976) method was used to resolve the incident and reflected spectra.

All tests were conducted on stone sections of the type shown in Figures 2 and 3 and Photos 1-4. Both sea-side and beachside slopes were held constant at 1V on 1.5H.

Design wave heights for the no-damage criterion were determined by subjecting the test sections to irregular waves successively larger in height in 0.01- to 0.02-ft increments until the maximum heights for which the armor was stable were reached. Each was allowed to attack the breakwater for a time equivalent to at least 1,000 peak wave periods, then the test sections were rebuilt prior to attack by the next added increment wave. This 1,000-wave duration allowed sufficient time for a statistically stable irregular wave condition to develop in the wave tank and also was sufficient for the test sections to stabilize.

Shallow-Water Test Results (d = 0.80 ft)

Shallow-water stability test results are summarized in Table 1. Presented therein are experimentally determined design wave heights and corresponding stability coefficients as functions of wave period, spectral width parameter (gamma), GI, and relative depth (d/L). Photos 5-8 show typical after-testing views of the structures at the 0.80-ft depth. As evidenced in these photos, the design wave conditions allowed occasional displacement of a few random armor units, but the damage never exceeded the acceptable design criteria of more than 2 percent of the total number of armor units in the primary cover layer. Results of a few tests did exceed the acceptable design criteria, however, the test conditions were never allowed to totally destroy the test section.

Figure 4 presents K_D, the Hudson stability coefficient, as a function of gamma for all wave periods investigated and Figures 5-8 present results for constant wave period. These data show stability to be influenced by wave period with the lower stabilities being observed at the longer wave periods. Also, the lower stabilities generally occur at the higher values of gamma. Figure 9 depicts stability as a function of grouping intensity, i.e., number of wave groups per hour of test waves. As would be expected, the lower stabilities are generally associated with the higher grouping intensities.

Deeper Water Test Results (d = 1.60 ft)

Test results for the 0.80-ft depth showed the lower stabilities consistently occurred at the higher values of gamma; therefore, tests at the 1.60-ft depth (Table 2) were conducted using gamma values of 10 and 20 only. Figure 10 presents K_D as a function of gamma for all wave periods and Figures 11-14 present results for constant wave period. Figure 15 presents stability as a function of grouping intensity. As with the 0.80-ft depth, the lower stabilities are again observed for the longer wave periods and the higher values of gamma and grouping intensity.

Summary and Nondimensionalization

Stability is presented as a function of grouping intensity for both water depths in Figure 16. These data show a decrease in stability with increasing T and GI; however, no strong depth-dependent trend is evident. Test results are nondimensionalized in Figures 17-19. Presented therein are the stability coefficients as a function of relative depth (d/L) for the two depths individually and collectively. These data show the influence of wave period with the lower stabilities occurring at the lower values of d/L, i.e., longer wave periods in shallower water. As discussed previously, a group of waves is defined as three or more successive waves which have heights equal to or exceeding the significant wave height of the entire test run. The maximum number of waves observed in a group was six.

Discussion

Results of this study show stability to be influenced by wave period, spectral width, and wave grouping intensity. As would be expected, the lowest stabilities are observed for the longest wave periods and the most highly grouped waves. Minimum stability coefficients observed herein (values of 0.8, 1.1, 1.6, and 1.8) are especially significant in that they are less than the minimums presently recommended for design (*Shore Protection Manual* 1984). The levels of wave grouping tested herein are achievable at some, but not all, prototype locations; therefore, these results should be applied on a case-by-case basis.

3 Conclusions

Based on tests and results described herein, in which stone armor is used on breakwater trunks and subjected to spectral wave attack, it is concluded that:

a. Armor stability is influenced by wave period with the lower stabilities being observed at the longer wave periods in shallower water.

b. The lower stabilities generally occur for the more highly grouped waves.

c. Minimum stability coefficients observed herein are especially significant in that they are less than the minimums presently recommended for design.

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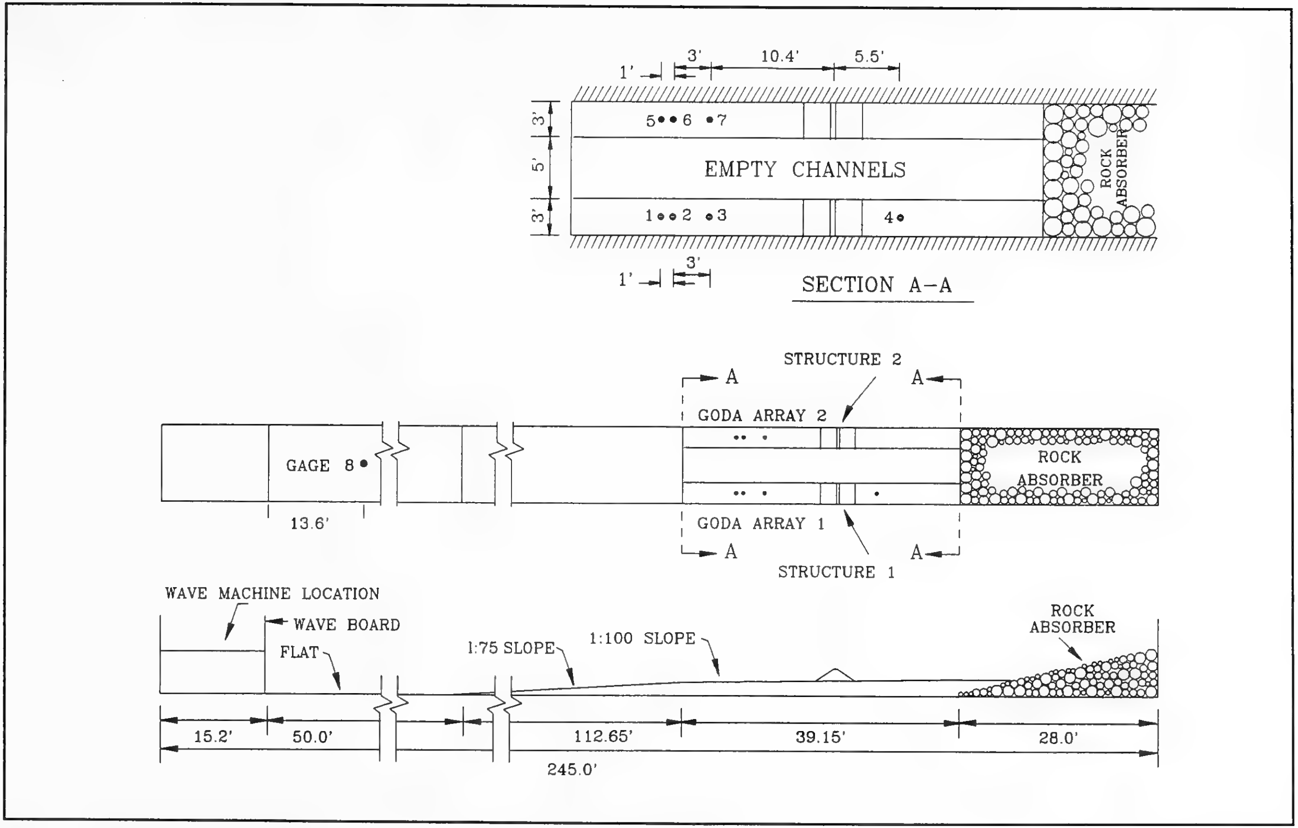
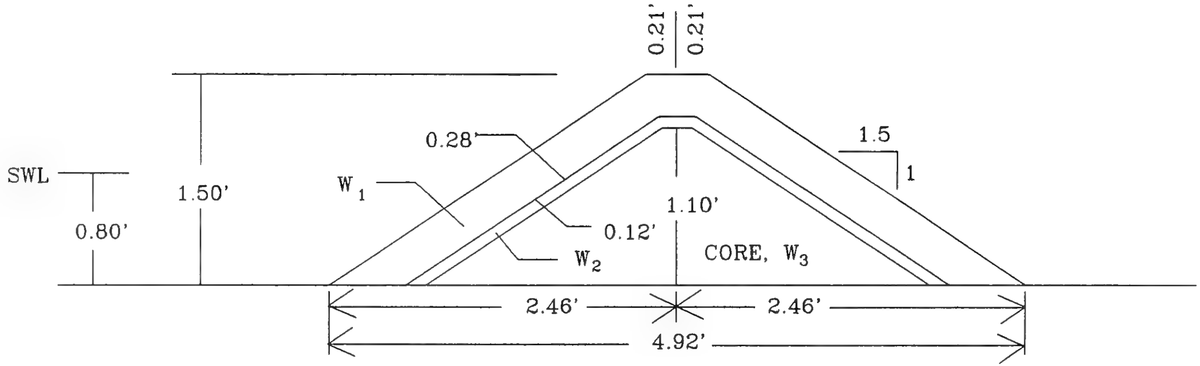


Figure 1. Wave tank cross section



MATERIAL CHARACTERISTICS

W_1 = 0.38-LB STONE
W_2 = 0.038-LB STONE
W_3 = 0.0022-LB STONE

Figure 2. Typical breakwater cross section; d = 0.8 ft

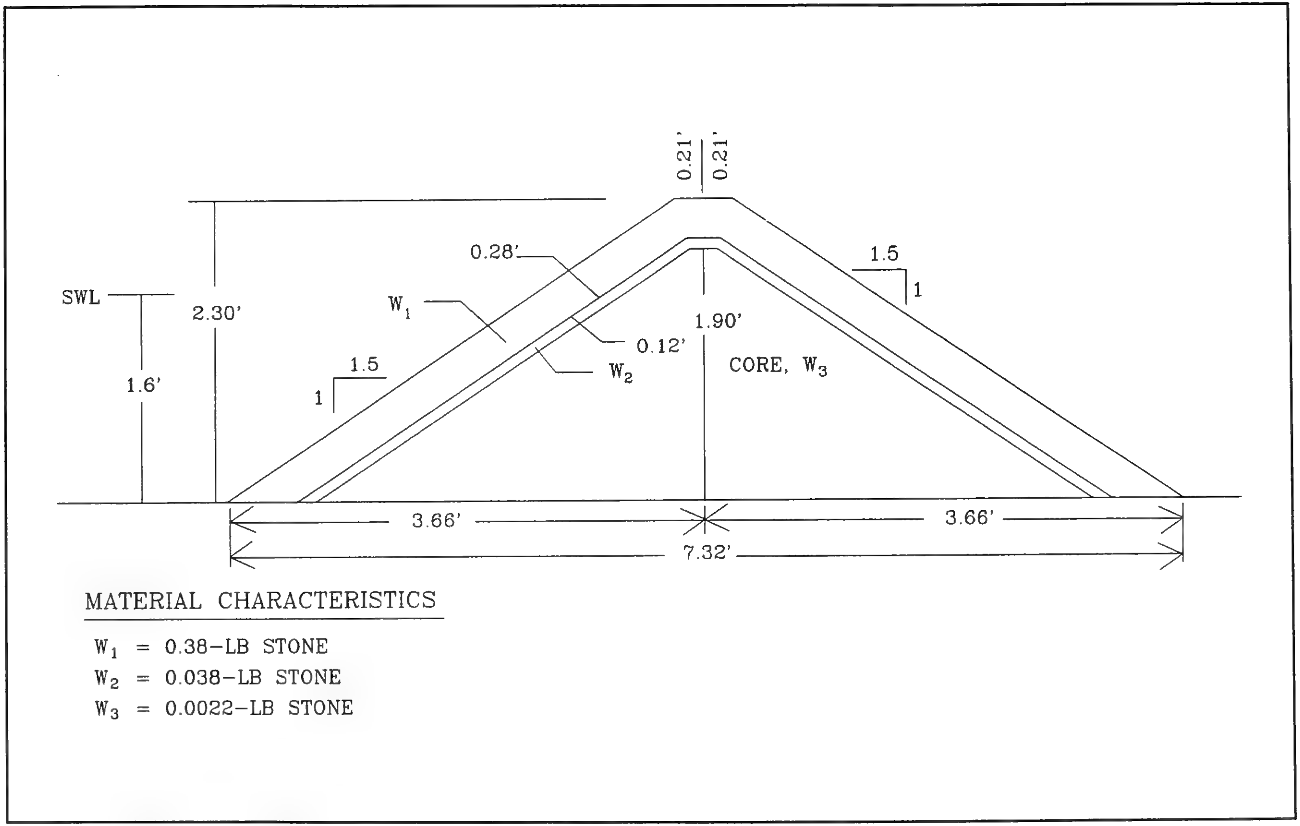


Figure 3. Typical breakwater cross section; d = 1.6 ft

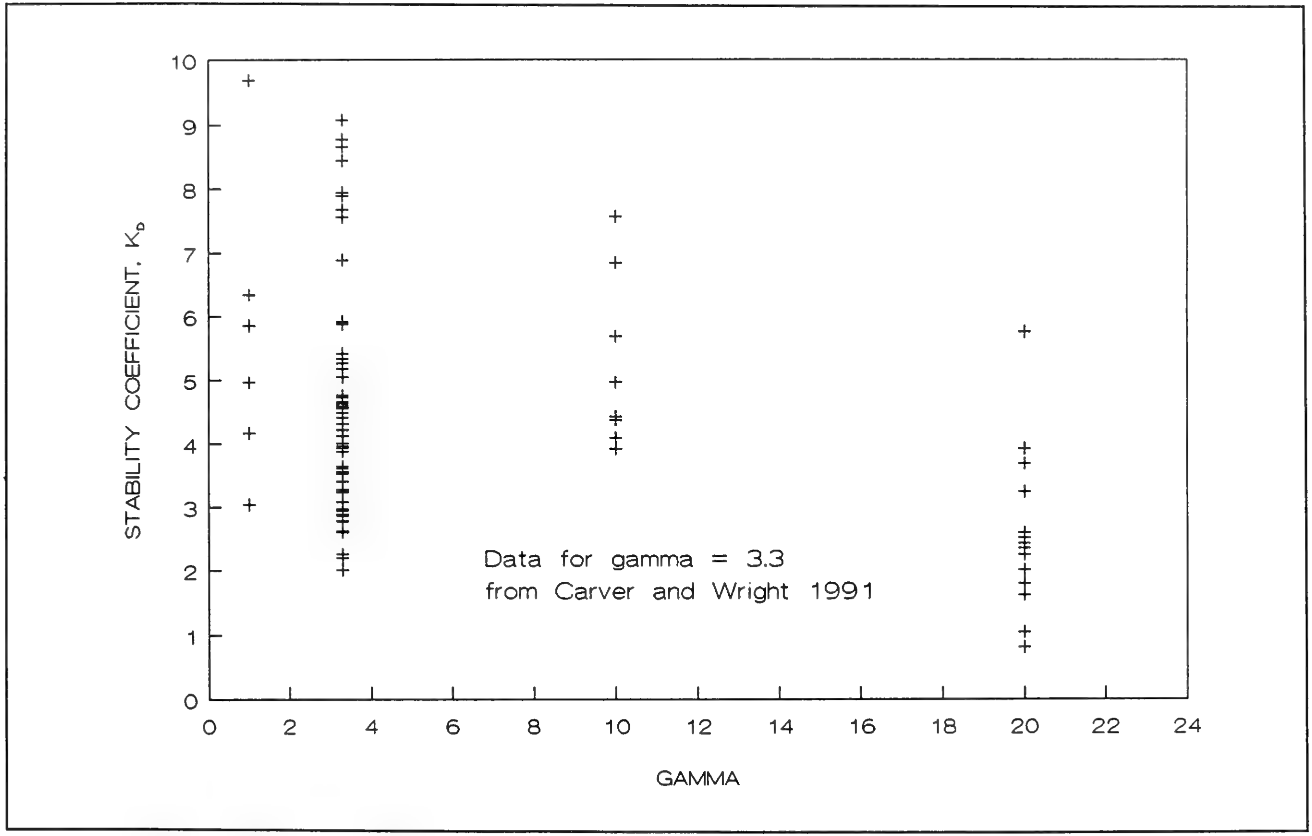


Figure 4. Stability coefficient versus gamma; d = 0.8 ft

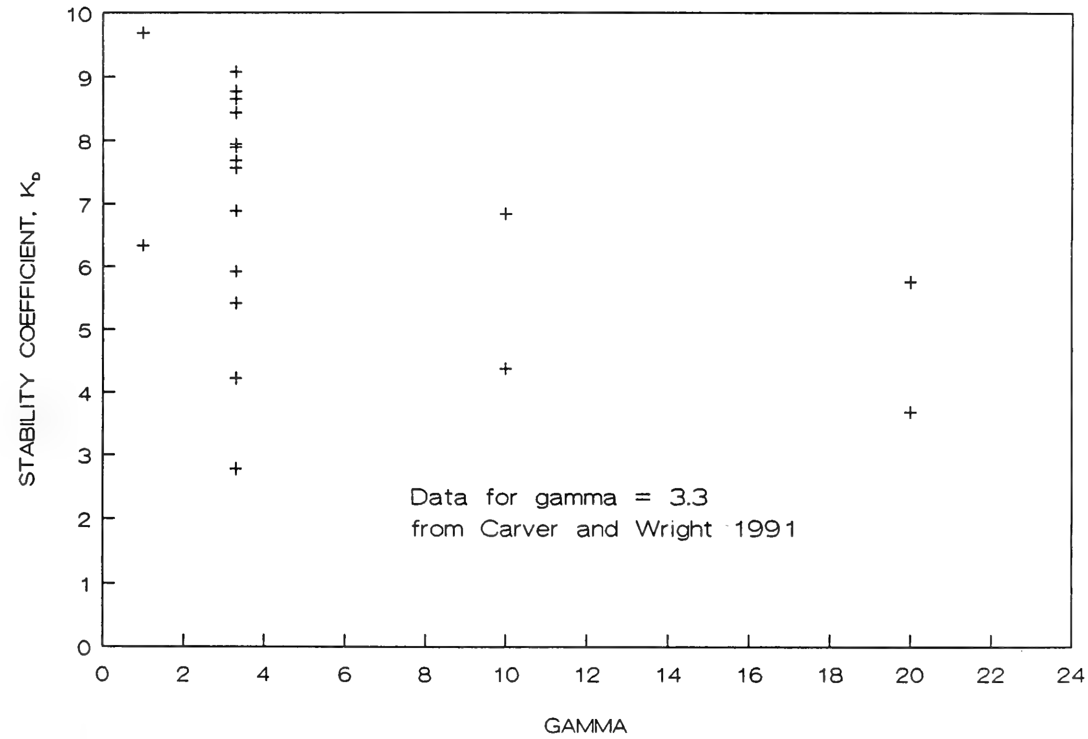


Figure 5. Stability coefficient versus gamma, d = 0.8 ft, T = 1.5 sec

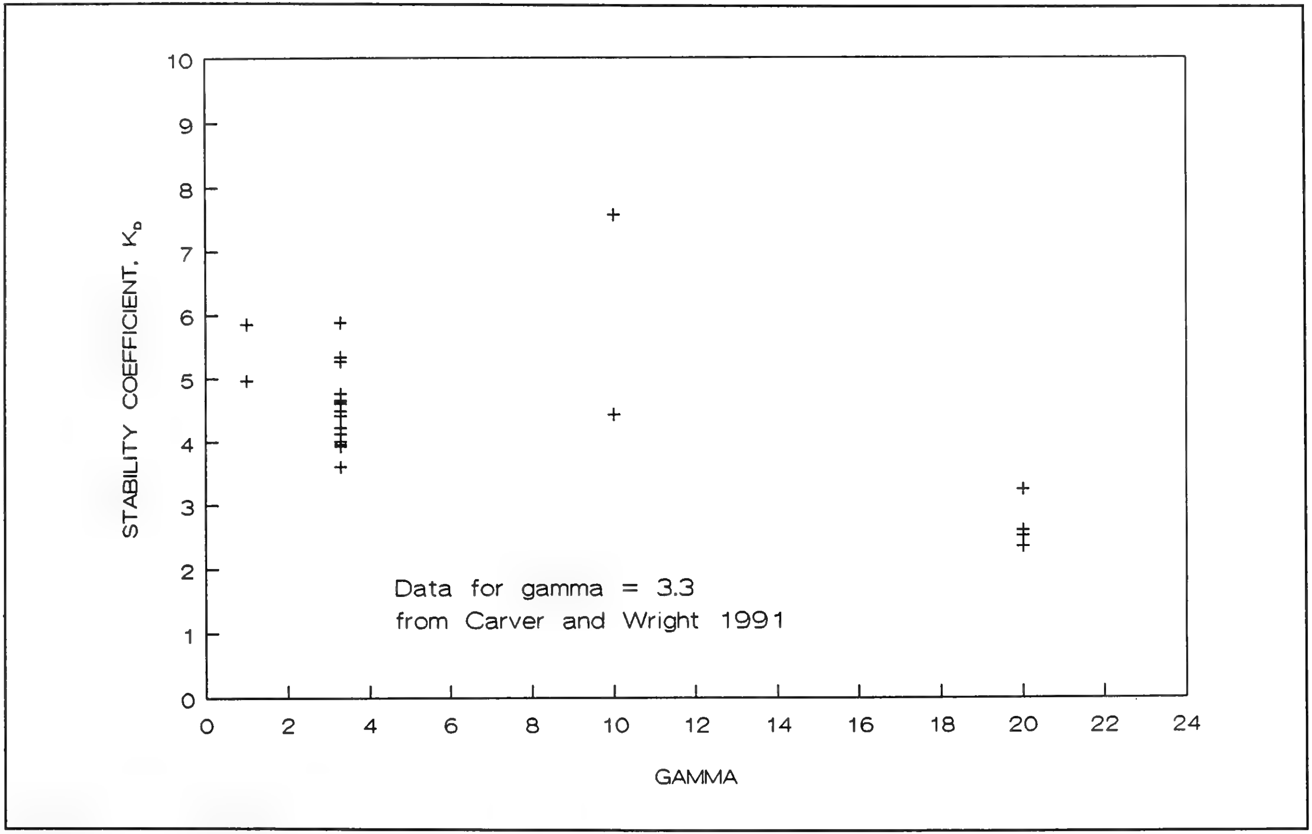


Figure 6. Stability coefficient versus gamma, d = 0.8 ft, T = 2.25 sec

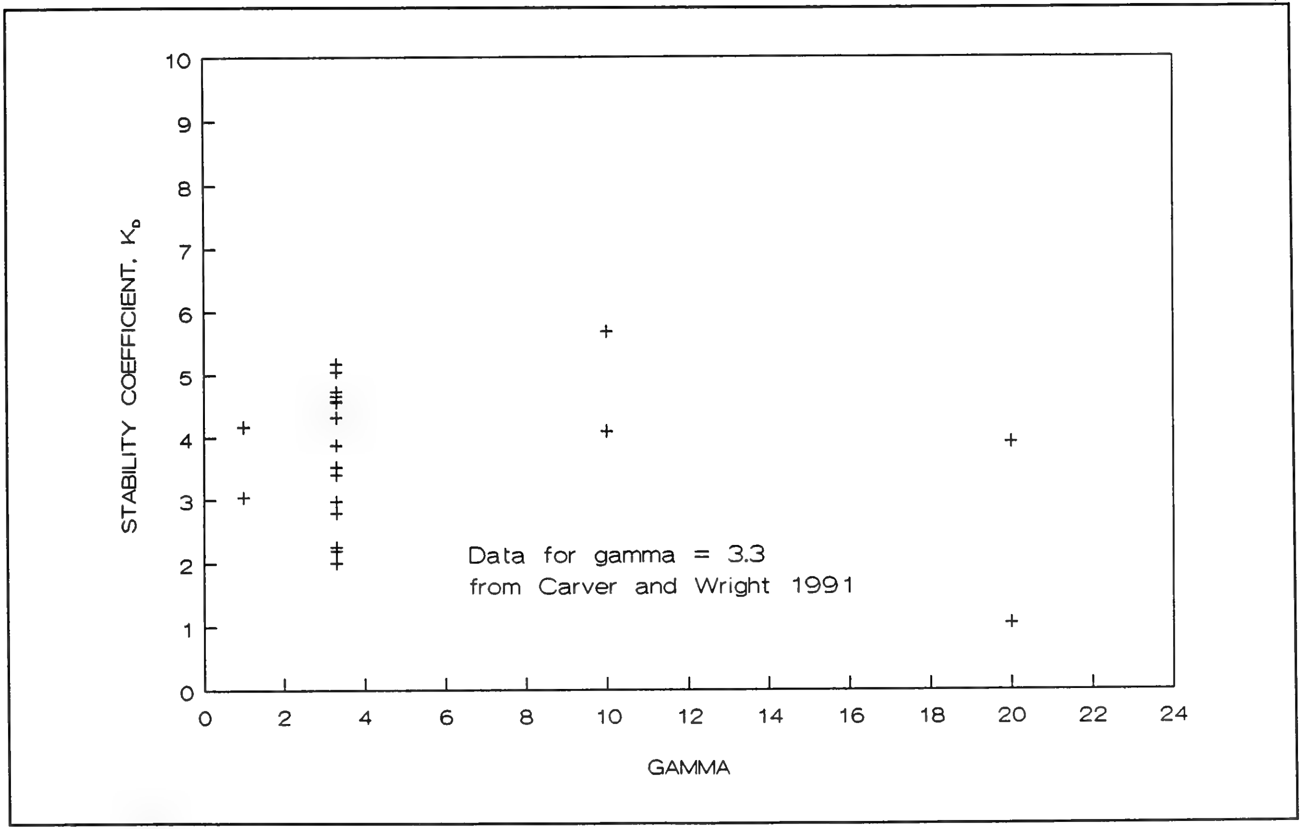


Figure 7. Stability coefficient versus gamma; d = 0.8 ft; T = 3 sec

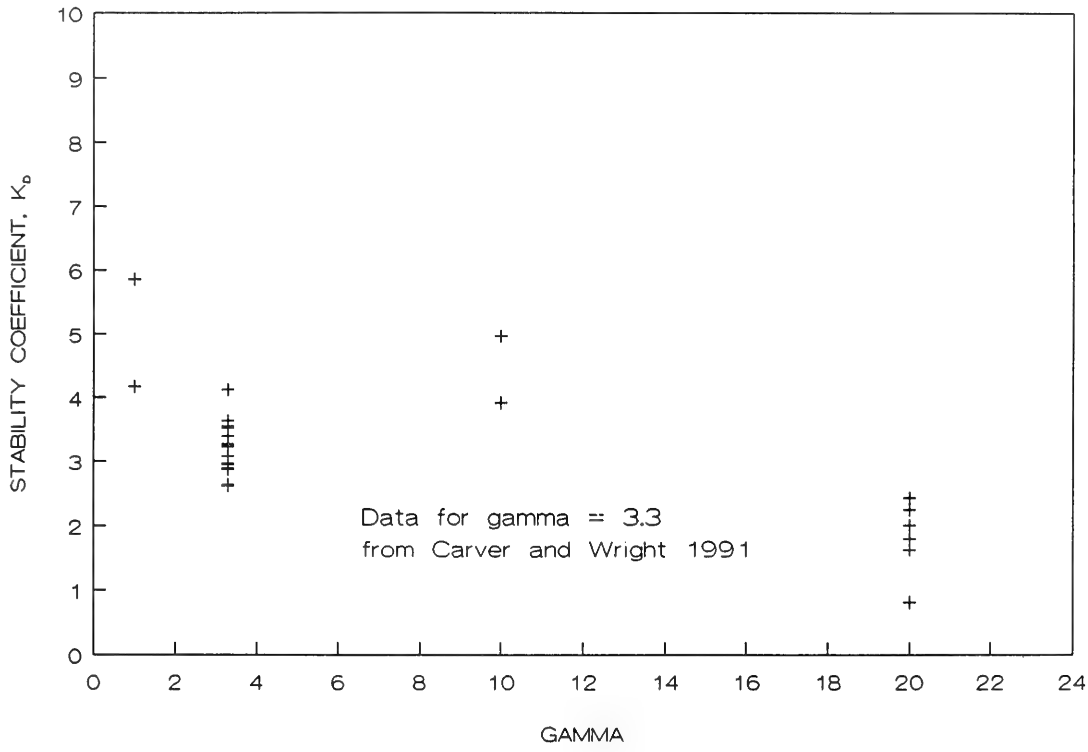


Figure 8. Stability coefficient versus gamma; d = 0.8 ft, T = 4 sec

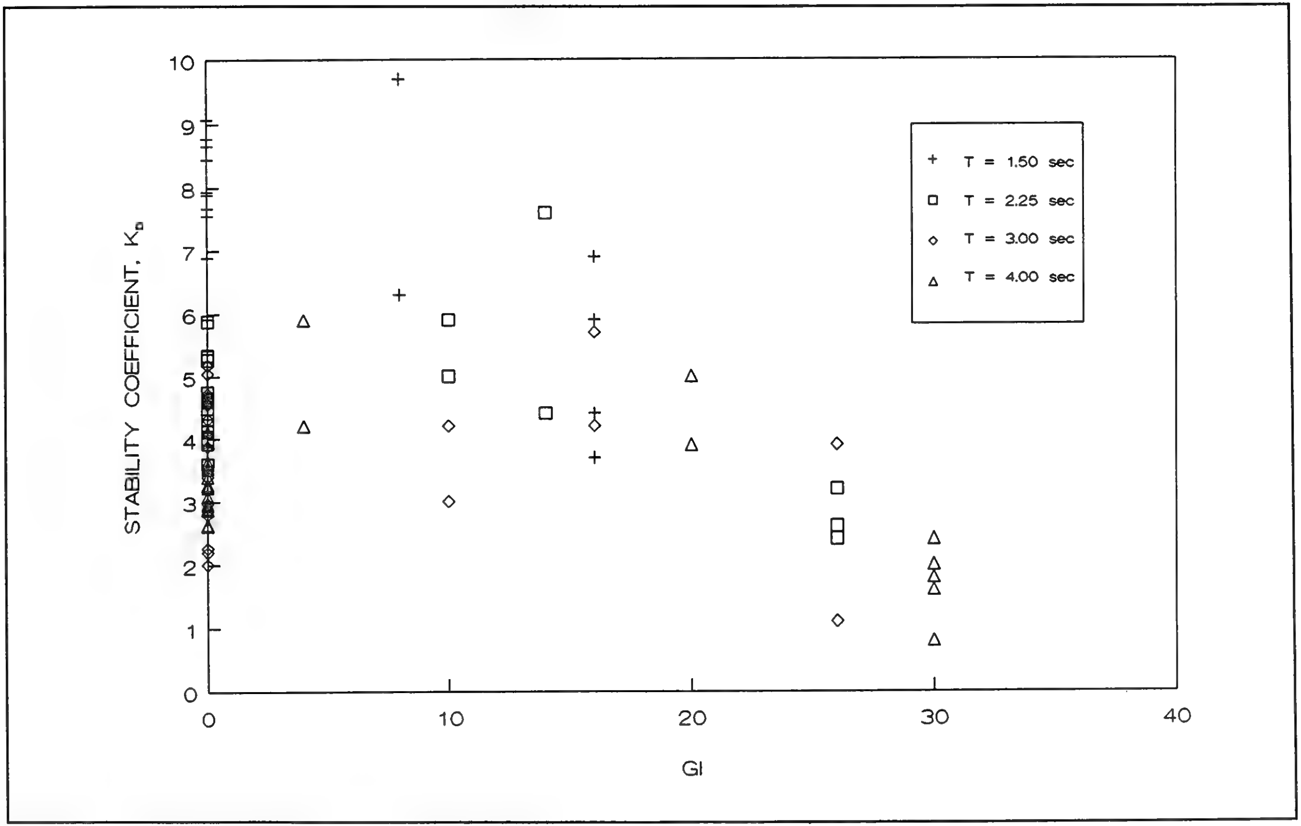


Figure 9. Stability coefficient versus grouping intensity (GI); d = 0.8 ft

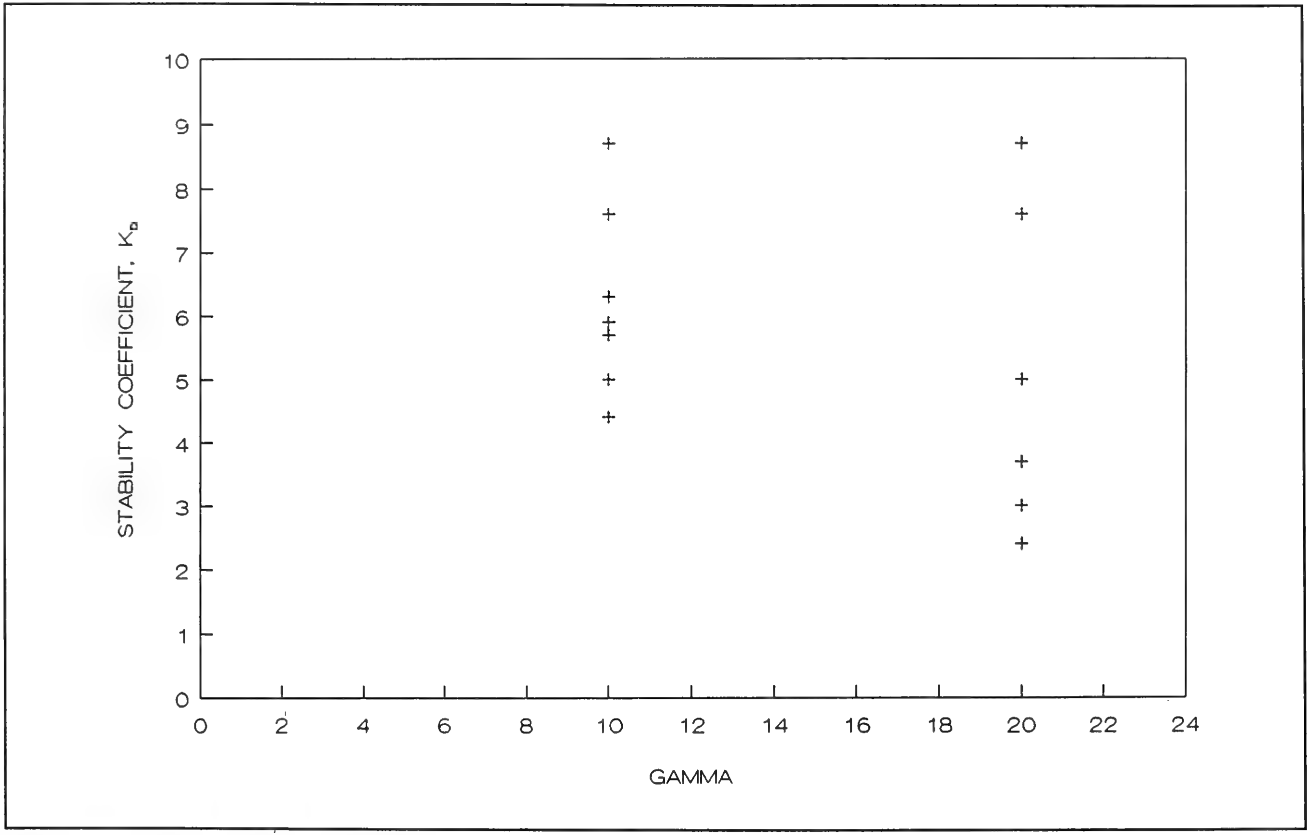


Figure 10. Stability coefficient versus gamma; d = 1.6 ft

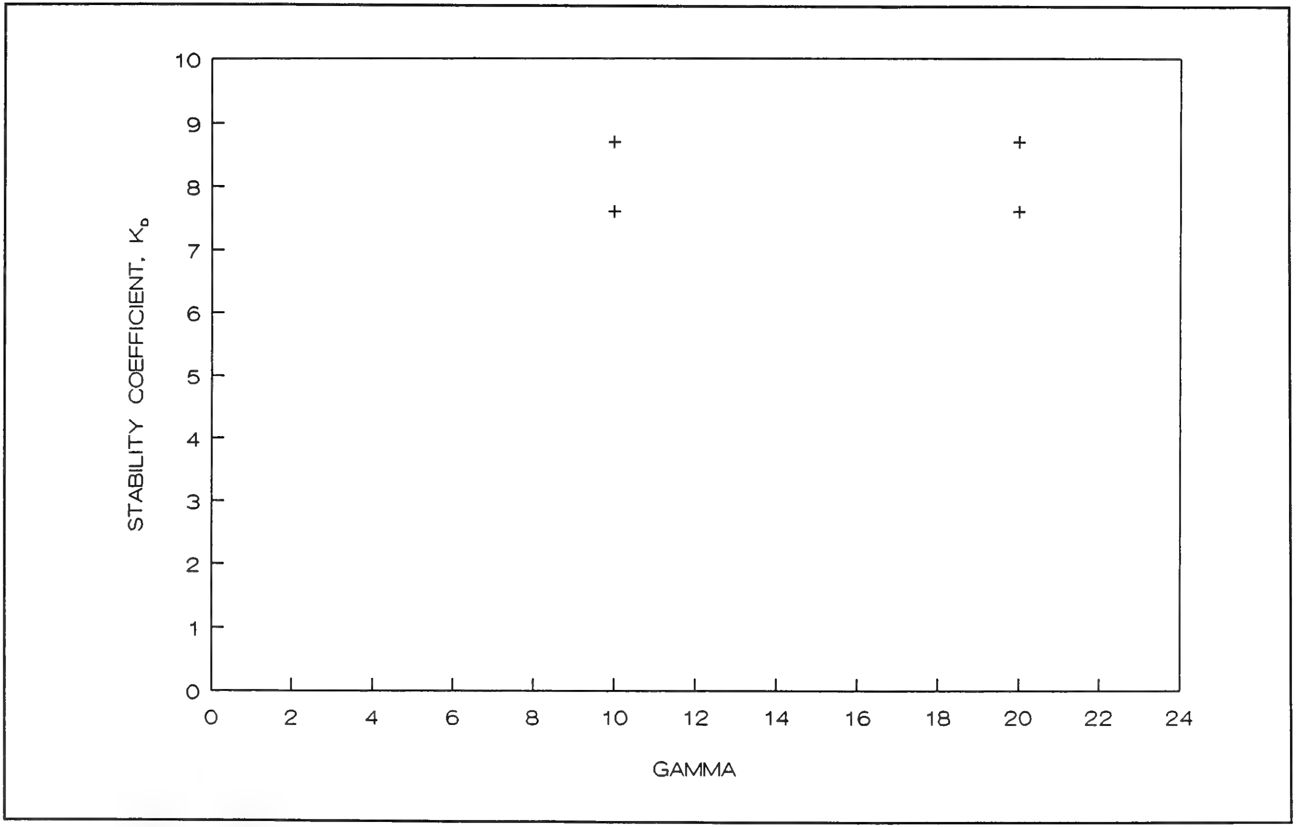


Figure 11. Stability coefficient versus gamma; d = 1.6 ft, T = 1.5 sec

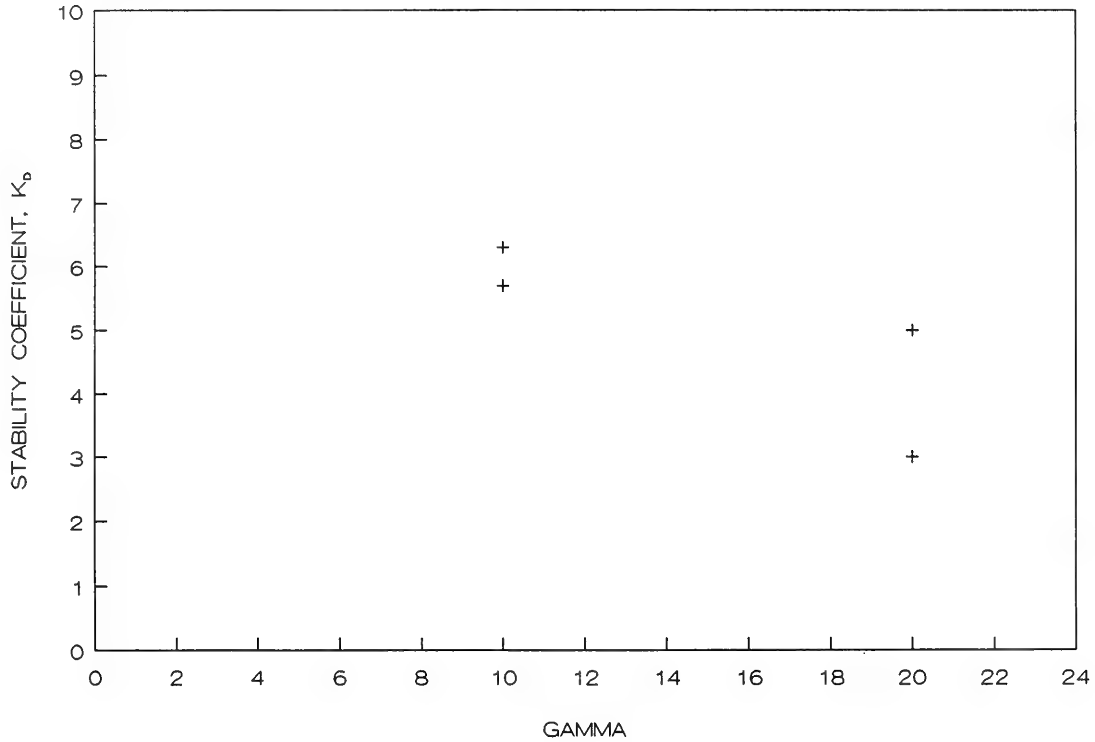


Figure 12. Stability coefficient versus gamma; d = 1.6 ft, T = 2.25 sec

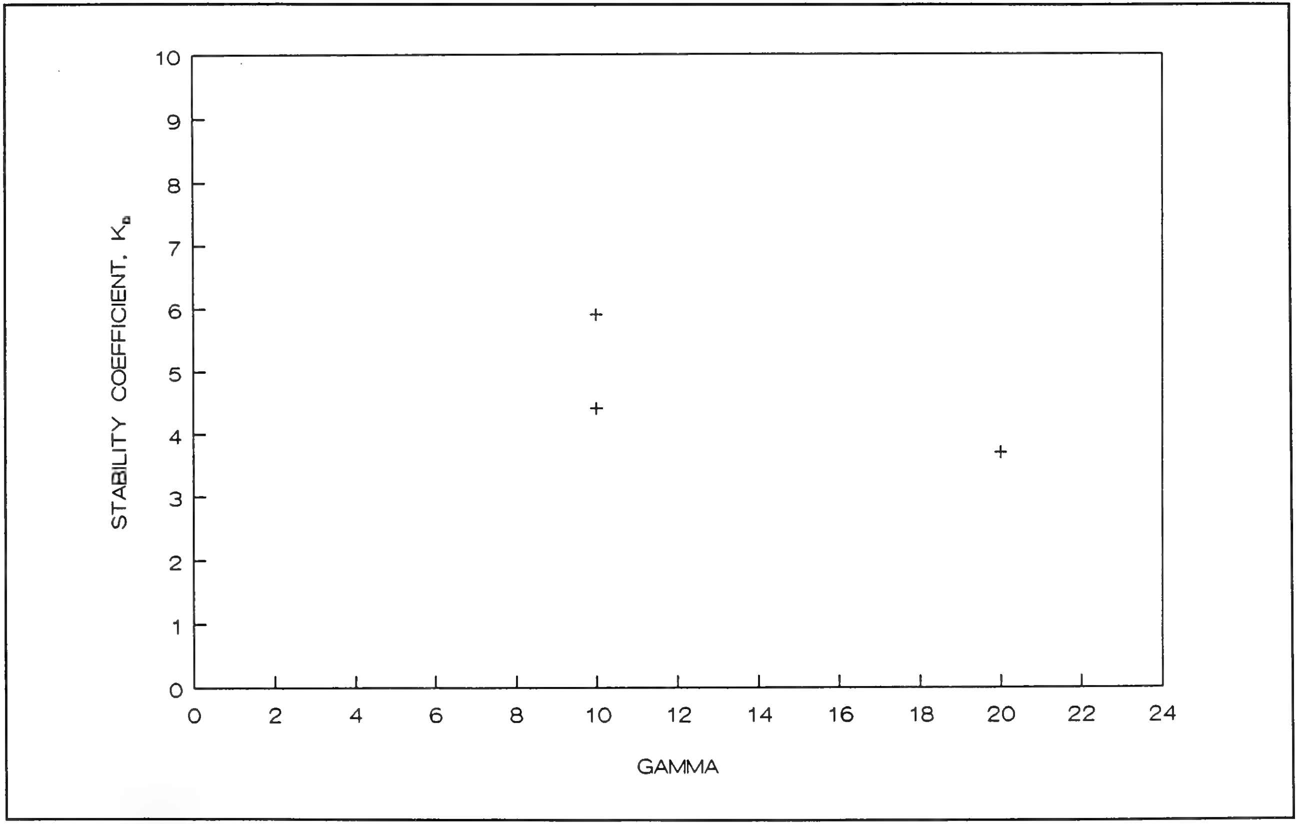


Figure 13. Stability coefficient versus gamma; d = 1.6 ft, T = 3.0 sec

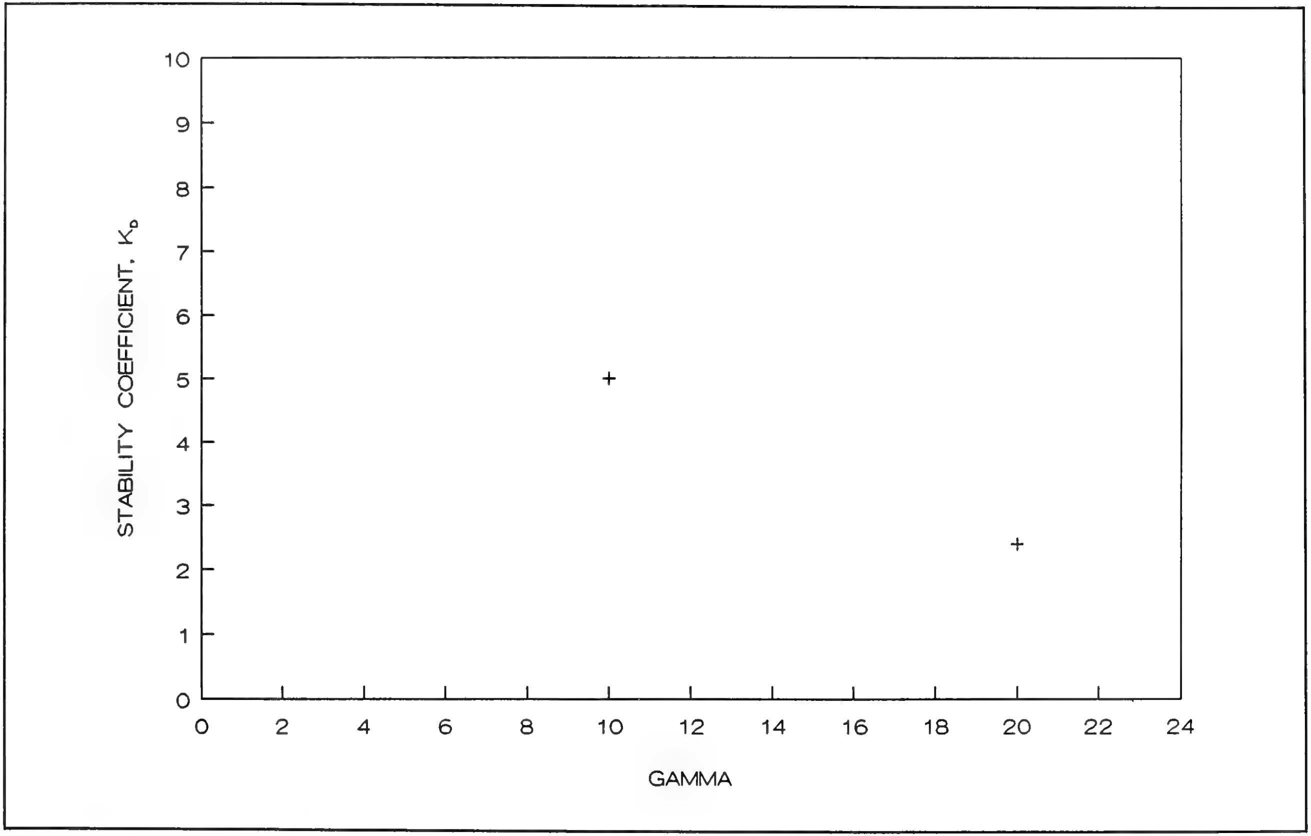


Figure 14. Stability coefficient versus gamma; d = 1.6 ft, T = 4.0 sec

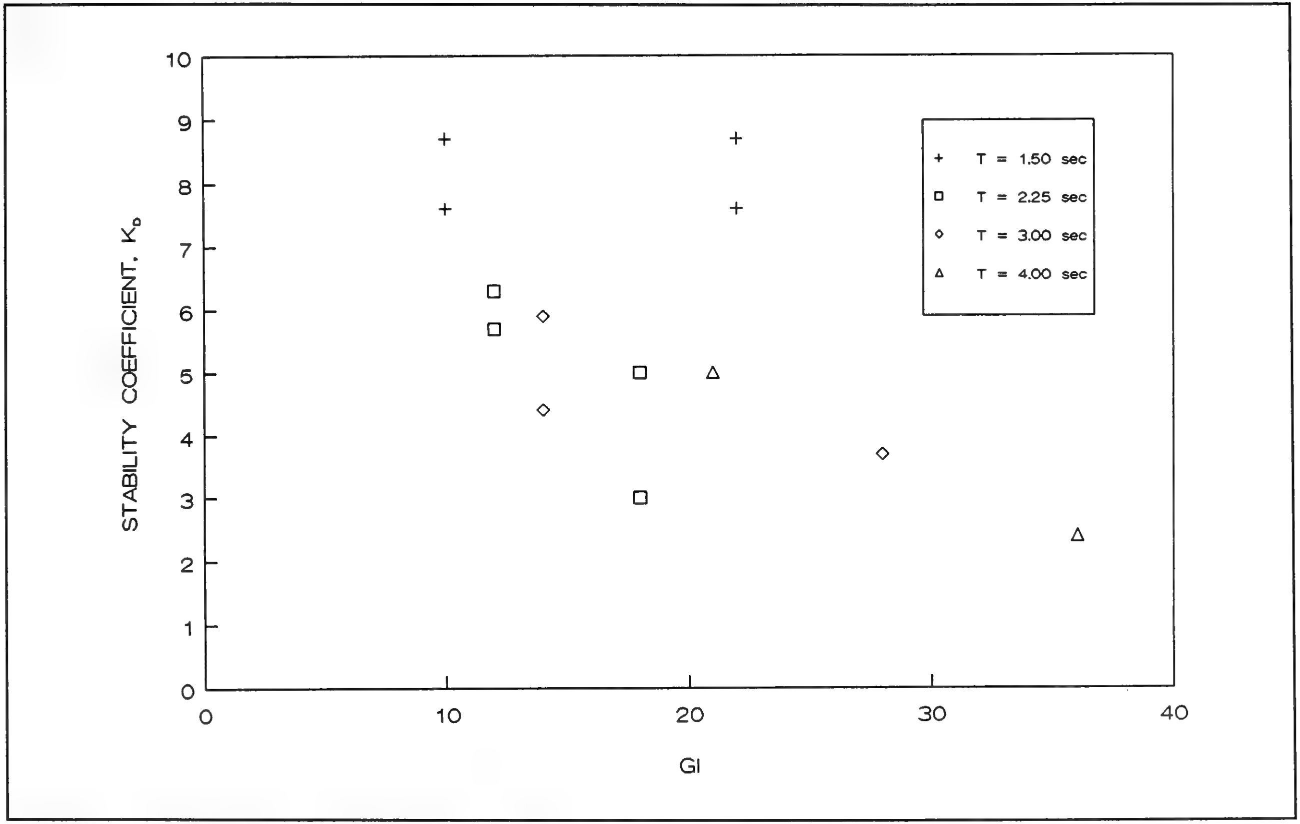


Figure 15. Stability coefficient versus grouping intensity (GI); d = 1.6 ft

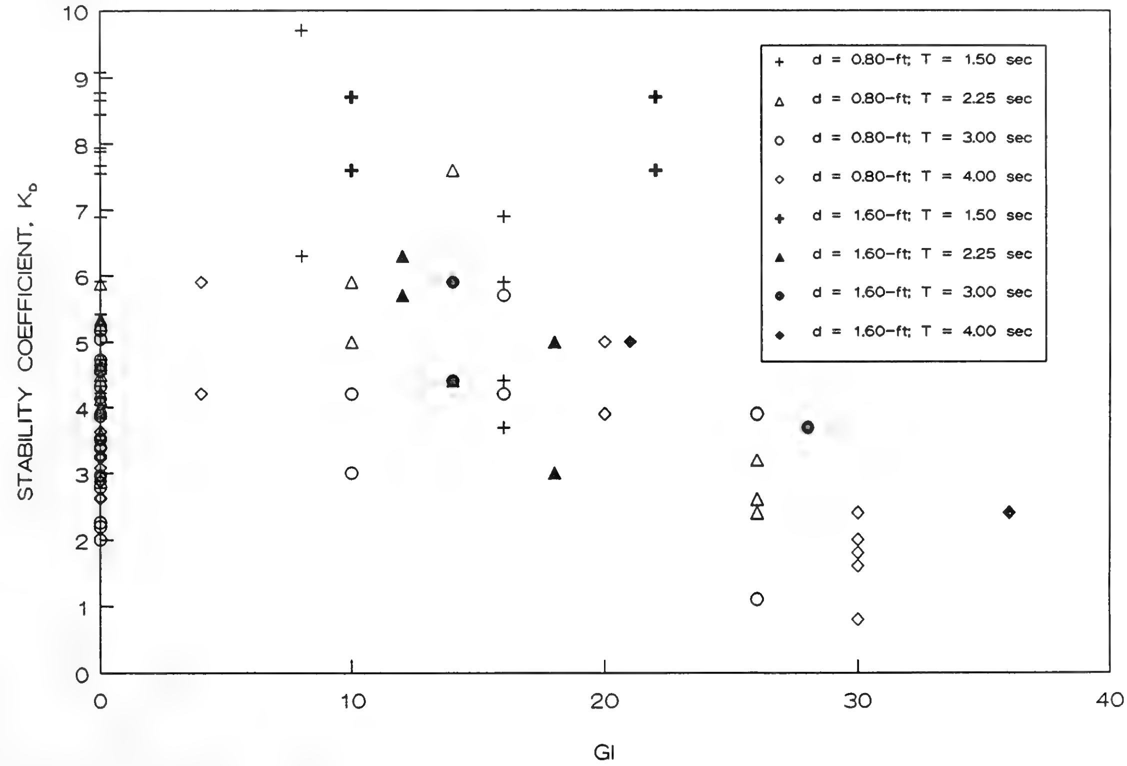


Figure 16. Stability coefficient versus grouping intensity (GI); 0.80-ft and 1.6-ft depths

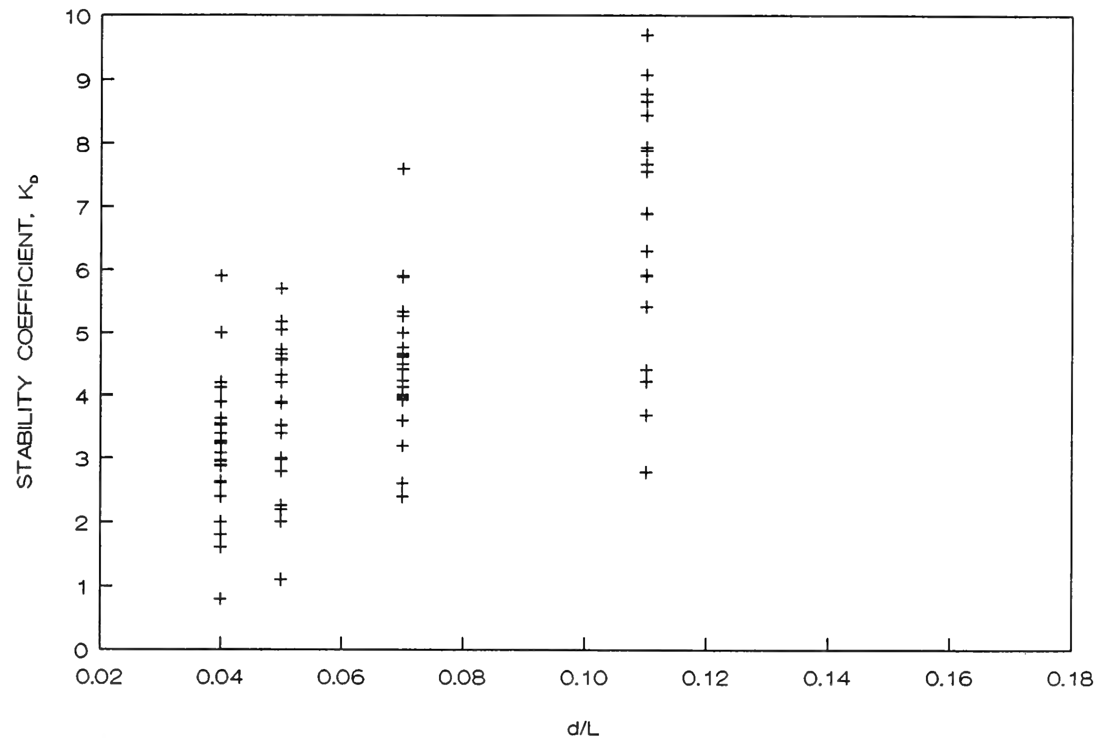


Figure 17. Stability coefficient versus d/L; d = 0.8 ft

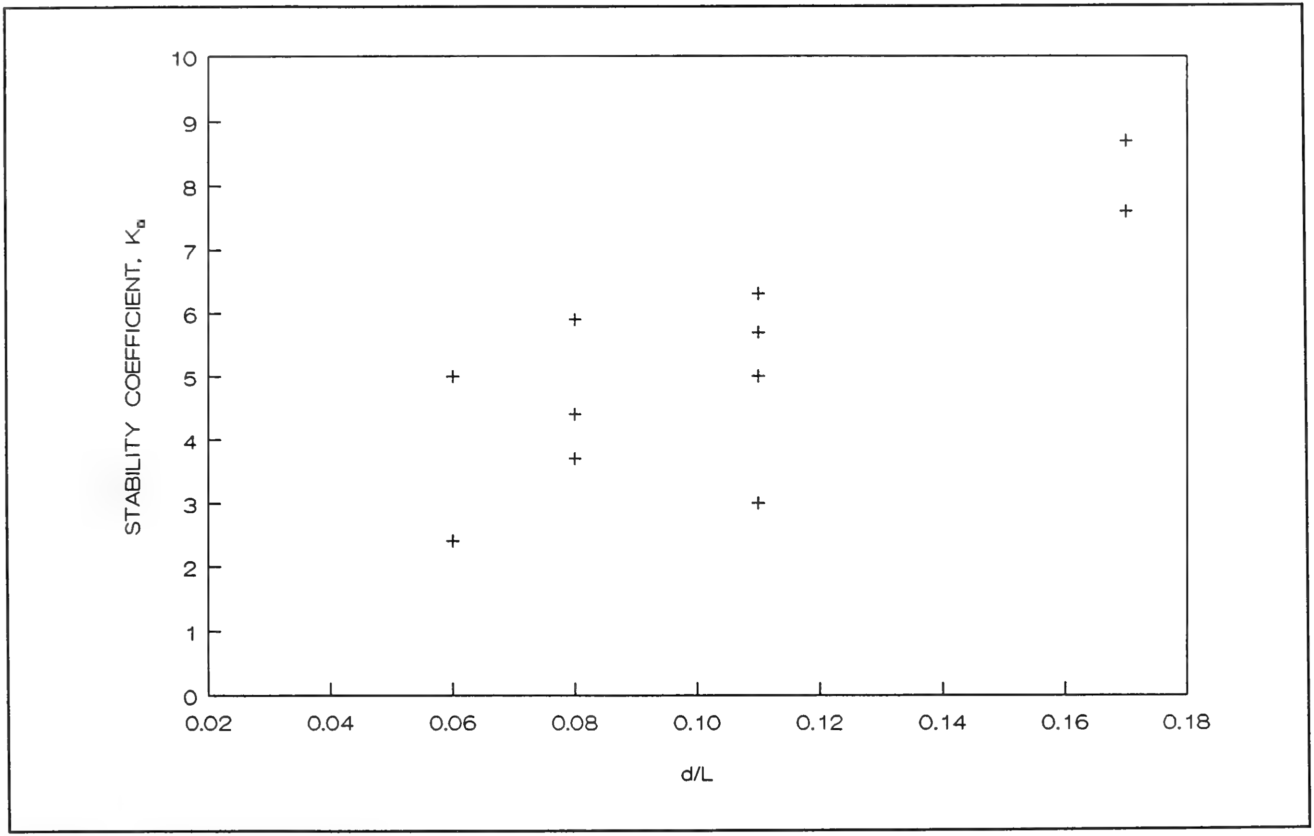


Figure 18. Stability coefficient versus d/L; d = 1.6 ft

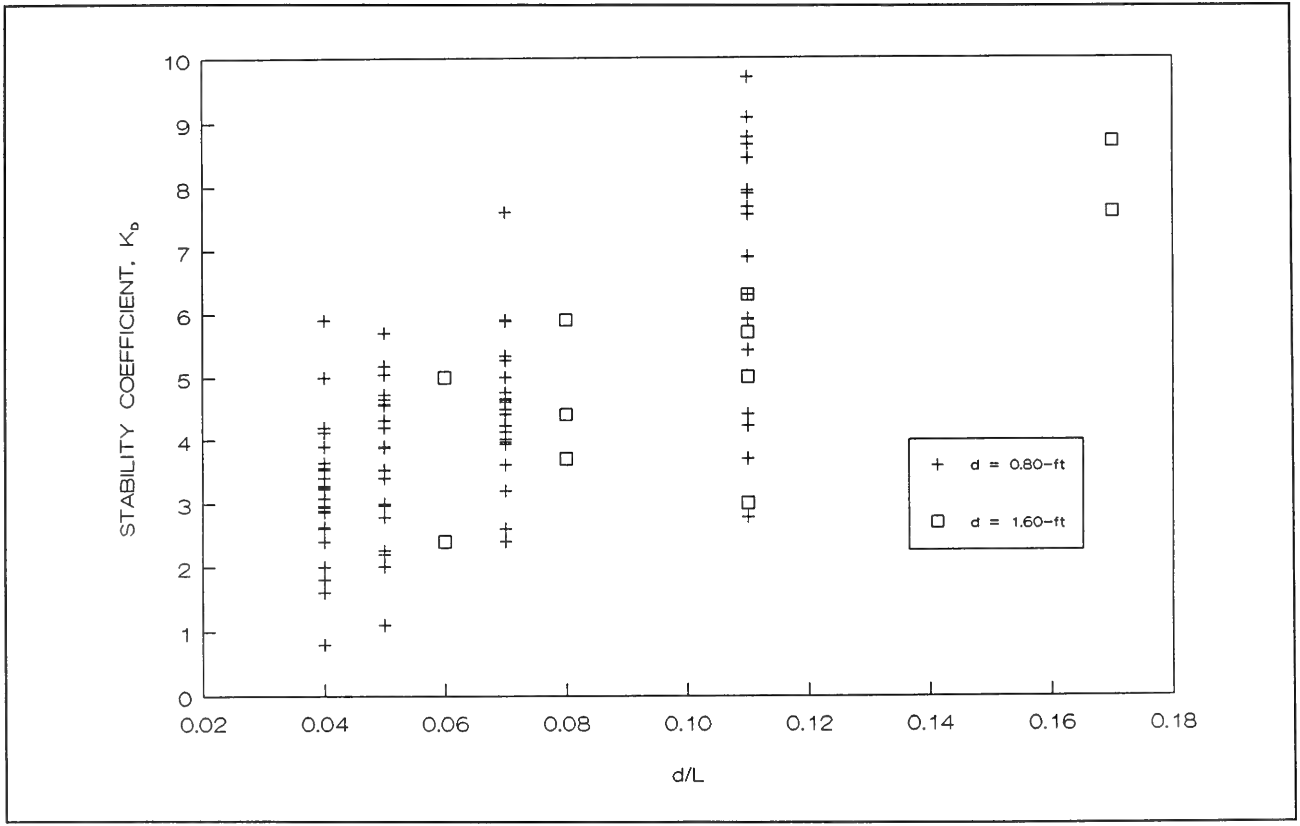


Figure 19. Stability coefficient versus d/L

Table 1
Shallow-Water Stability Test Results (0.80-ft depth)

Gamma	T_p, sec	d/L	H_{mo}, ft	GI	K_D
1.0	1.50	0.11	0.46	8	6.3
1.0	1.50	0.11	0.53	8	9.7
1.0	2.25	0.07	0.42	10	5.0
1.0	2.25	0.07	0.45	10	5.9
1.0	3.00	0.05	0.36	10	3.0
1.0	3.00	0.05	0.40	10	4.2
1.0	4.00	0.04	0.40	4	4.2
1.0	4.00	0.04	0.45	4	5.9
10.0	1.50	0.11	0.41	16	4.4
10.0	1.50	0.11	0.47	16	6.9
10.0	2.25	0.07	0.41	14	4.4
10.0	2.25	0.07	0.49	14	7.6
10.0	3.00	0.05	0.40	16	4.2
10.0	3.00	0.05	0.44	16	5.7
10.0	4.00	0.04	0.39	20	3.9
10.0	4.00	0.04	0.42	20	5.0
20.0	1.50	0.11	0.38	16	3.7
20.0	1.50	0.11	0.45	16	5.9
20.0	2.25	0.07	0.33	26	2.4
20.0	2.25	0.07	0.34	26	2.6
20.0	2.25	0.07	0.34	26	2.6
20.0	2.25	0.07	0.37	26	3.2
20.0	3.00	0.05	0.25	26	1.1
20.0	3.00	0.05	0.39	26	3.9
20.0	4.00	0.04	0.23	30	0.8
20.0	4.00	0.04	0.29	30	1.6
20.0	4.00	0.04	0.30	30	1.8
20.0	4.00	0.04	0.31	30	2.0
20.0	4.00	0.04	0.33	30	2.4
20.0	4.00	0.04	0.33	30	2.4

Table 2
Test Results, 1.6-ft depth

Gamma	T_p, sec	d/L	H_{mo}, ft	GI	K_D
10.0	1.50	0.17	0.49	10	7.6
10.0	1.50	0.17	0.51	10	8.7
10.0	2.25	0.11	0.44	12	5.7
10.0	2.25	0.11	0.46	12	6.3
10.0	3.00	0.08	0.41	14	4.4
10.0	3.00	0.08	0.45	14	5.9
10.0	4.00	0.06	0.42	21	5.0
10.0	4.00	0.06	0.42	21	5.0
20.0	1.50	0.17	0.49	22	7.6
20.0	1.50	0.17	0.51	22	8.7
20.0	2.25	0.11	0.42	18	5.0
20.0	2.25	0.11	0.36	18	3.0
20.0	3.00	0.08	0.38	28	3.7
20.0	3.00	0.08	0.38	28	3.7
20.0	4.00	0.06	0.33	36	2.4
20.0	4.00	0.06	0.33	36	2.4

Photo 1. End view before wave attack at the 0.80-ft depth

Photo 2. Sea-side view before wave attack at the 0.80-ft depth. Change in stone color denotes still-water level

Photo 3. End view before wave attack at the 1.60-ft depth

Photo 4. Sea-side view before wave attack at the 1.60-ft depth. Change in stone color denotes still-water level

Photo 5. End view after wave attack of 4.0-sec, 0.39-ft waves at the 0.80-ft depth; gamma = 10

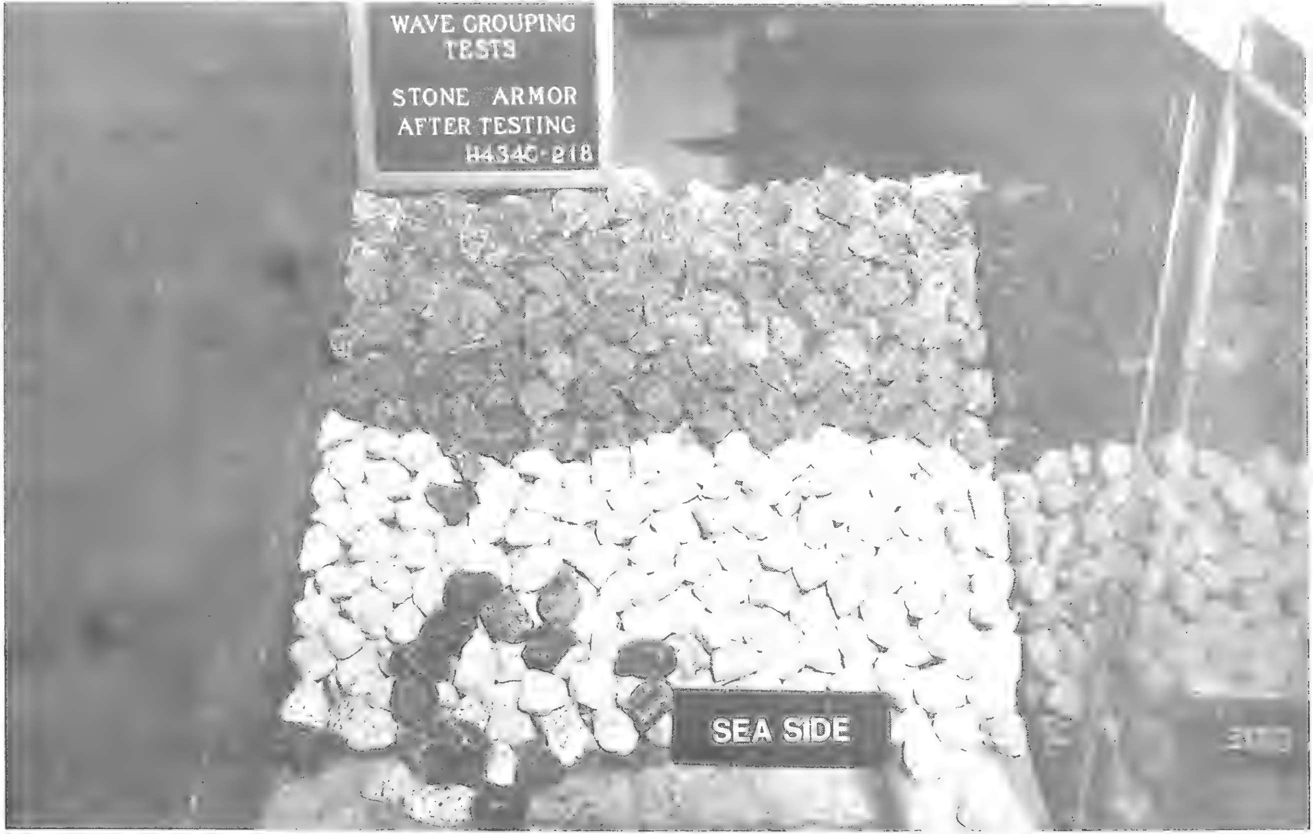


Photo 6. Sea-side view after wave attack of 4.0-sec, 0.39-ft waves at the 0.80-ft depth; gamma = 10

Photo 7. End view after wave attack of 4.0-sec, 0.30-ft waves at the 0.80-ft depth; gamma = 10

Photo 8. Sea-side view after wave attack of 4.0-sec, 0.30-ft waves at the 0.80-ft depth; gamma = 20

Photo 9. Sea-side view after wave attack of 2.25-sec, 0.44-ft waves at the 1.60-ft depth; gamma = 10

Photo 10. Sea-side view after wave attack of 4.0-sec, 0.42-ft waves at the 1.60-ft depth; gamma = 10

Photo 11. Sea-side view after wave attack of 2.25-sec, 0.36-ft waves at the 1.60-ft depth; gamma = 20

Photo 12. Sea-side view after wave attack of 4.0-sec, 0.33-ft waves at the 1.60-ft depth; gamma = 20

Appendix A
Notation

d/L	Relative depth, dimensionless
g	Acceleration due to gravity, ft/sec^2
H	Wave height, ft
H/d	Relative wave height
H_{mo}	Zero-moment wave height, ft
K_D	Hudson stability coefficient, dimensionless
l_a	Characteristic length of armor unit, ft
R_N	Reynolds stability number
T_p	Wave period of peak energy density of spectrum, sec
W_a	Granitic stone weight
γ	Spectral width parameter
ν	Kinematic viscosity of experimental fluid medium, ft^2/sec

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12b. DISTRIBUTION CODE

13. ABSTRACT *(Maximum 200 words)*

High sea waves tend to appear in groups rather than individually. Because of the nature of wave grouping, it appears that it may be an important influence on the stability of rubble-mound structures. The research documented in this report was conducted to obtain a better understanding of the effects of wave grouping on the stability of stone armor when used on breakwater trunks. Results of this study show stability to be influenced by wave period, spectral width, and wave grouping intensity. Levels of wave grouping tested herein are achievable at some, but not all, prototype locations; therefore, results should be applied on a case-by-case basis.

14. SUBJECT TERMS
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