SCOUR OF SIMULATED GULF COAST SAND BEACHES DUE TO WAVE ACTION IN FRONT OF SEA WALLS AND DUNE BARRIERS

By

CHARLES B. CHESNUTT and ROBERT E. SCHILLER, JR.
Coastal and Ocean Engineering Division
Texas A&M University

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ABSTRACT

This study was the first attempt to investigate scour in front of sea walls and dune barriers for conditions simulating Texas Gulf Coast beaches. Texas beach sand, which was found to be uniform in grain size along the coast from Sabine Pass to mid-way of Padre Island, was used in conducting the experiments. Three beach slopes, 1:40, 1:70 and 1:100, typical of Texas beaches, were studied. Sea walls with inclinations of 15°, 30°, and 90° from the horizontal were inserted in each of the three slopes for one wave condition and in the 1:40 slope for three additional wave conditions. The position of the sea wall (15°, 30° and 90°) with respect to the point where the wave breaks was studied for one wave condition.

Studies were also conducted on beach formations without a sea wall. Waves acting on a laboratory beach with a slope of 1:40 built a berm or dune barrier with a slope of 10° to 15°. Tests of scour in front of these dune barriers have shown that the ultimate scour depth is approximately equal to the deep water wave height.

The sea wall, when placed in the position of X/X_b = 0.5 to 0.67 (where X is the distance from the sea wall to the natural shoreline and X_b is the distance from the point where waves break to the shoreline) was subject to greater scour than at any other position along the profile.

The ultimate scour depth just in front of the wall decreased as the beach slope flattened and as the angle of inclination of the sea wall decreased. As wave height increased, the scour depth increased.

PREFACE

In September, 1969 a study of scour of Gulf Coast sand beaches due to wave action in front of sea walls and dune barriers was begun as part of the research program of the Coastal and Ocean Engineering Division of Texas A&M University. The three year project was partially funded by the National Science Foundation Sea Grant Program institutional grants GH-59 and GH-101 made to Texas A&M University. This report describes the work conducted on this project from January, 1970 through January, 1971.

The report was written by the senior author in partial fulfillment of the Master of Science requirement under the supervision of the junior author who was his major advisor and principal investigator on the entire project.

The authors would like to thank Dr. Arnold H. Bouma and Dr. John B. Herbich for their review of this paper. The work of "Butch" Buck, Jack Dysart, Larry Finley, Mickey Jones, Stuart Schertz, and Bill Scott in preparing each test was sincerely appreciated.

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INTRODUCTION

The problem of scour in front of sea walls has plagued coastal engineers since man first became concerned with protection of coastal areas. The design engineer in predicting the scour depth is hampered by the multiplicity of variables involved in the scour phenomenon. The most significant variable is the wave height. In addition, the wave length, the angle of inclination of the sea wall, the coefficient of reflection, the position of the sea wall with respect to where the wave breaks, the sediment size, and the slope of the beach affect the depth of scour.

The U. S. Army Corps of Engineers Coastal Engineering Research Center has developed a rule of thumb for predicting scour in front of sea walls. The maximum scour depth is said to be "equal to the height of the maximum unbroken wave that can be supported by the original depth of water at the toe of the structure." (8) Studies have been conducted simulating beach conditions along the typical United States coastlines and along the coasts of Japan. The fine sand and extremely flat beaches found along the beaches of the Gulf of Mexico are unique enough to warrant a special investigation. This was the first attempt, to the authors'

The citations on the following pages follow the style of the Journal of the Waterways, Harbors and Coastal Engineering Division of the American Society of Civil Engineers.

knowledge, to investigate this phenomenon under conditions simulating Texas Gulf Coast beaches.

This report is part of a project funded by the National Sea Grant Program under Institutional Grant No. 101 to Texas A&M University to study this problem. An initial study of sand samples taken at various locations along the Texas coast from Sabine Pass to mid-way down Padre Island indicated that the beach sand did not vary greatly and had a d_{50} of about 0.17mm. Beach profile data from the Galveston office of the U. S. Army Corps of Engineers and other studies conducted by the Coastal and Ocean Engineering Division of the Civil Engineering Department of Texas A&M revealed that the slope of Texas beaches ranged from 1:40 to 1:100.

LITERATURE SURVEY

In 1953, Russell and Inglis (5) studied scour in front of a vertical wall. A 6-hour tidal cycle, with an 8 in. amplitude, was used. A constant wave height of 3 in. was run, which required that the wave period, and therefore the length, vary as the depth changed. Mean water level in the 2 ft by 46 ft channel was 9 in. at the offshore end and the sand was initially spread to a 1:60 slope in the offshore region and steeper near the shoreline. After seven tidal cycles, when the beach had reached an equilibrium profile, a vertical sea wall was inserted in the beach at a point where the elevation was about 3 in. above mean high water. The experiment continued under the same conditions until equilibrium was again reached. The ultimate scour depth was found to be about one wave height below mean low water.

Herbich, et al, (3) used a white silica sand, d_{50} = 0.48mm, smoothed to a uniform bed thickness of 5 in. in front of an adjustable slope sea wall. Two series of experiments were conducted. The first series consisted of running three wave conditions at each of three sea walls: 45°, 67-1/2° and 90°.[1] The ultimate scour depth was taken to be the significant ultimate scour depth (average of the one-third greatest scour depths along

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[1] The angle of inclination is measured from the horizontal.

the entire bed). The ultimate scour depth was found to be the same for all three walls for each wave condition. The distance between sand crests was found to be a function of the water wave length and was half the wave length. The second series of experiments were conducted using 15° and 30° walls. It was found that the relative scour depth, S/H, and the coefficient of reflection were greater for the 30° wall than for the 15° wall. For a constant value of L/d, there was a minimal value of S/H at H/d = 0.41. For a constant value of H/d, there was a minimal value of S/H between L/d = 9 and L/d = 10. The ultimate depth of scour reached a constant value for reflection coefficients greater than 54%, for the 30° wall.

Sawaragi (7) studied scour at the toe of permeable structures. He determined the relationship between the void ratio of the structure and the coefficient of reflection, and the relationship between the coefficient of reflection and the relative scour depth. The scour depth was found to become proportionately larger with increase in reflection, for reflections greater than 25%. For reflections less than 25%, there was little scour.

Herbich and Ko (2) developed a mathematical model for determining the ultimate depth of scour:

$$S = (d - \frac{1}{2}A)\ [(1 - C_r)\ U_*(\frac{3}{4}\ C_D\ \rho\ \frac{\cot\ \phi}{d_{50}\ (\gamma_s - \gamma)})^{1/2} - 1]\ . \quad (1)$$

In addition, laboratory tests were conducted to verify this equation. The procedure used was much the same as that of Herbich, et al, (3). The median grain size of the sand was 0.32 mm. Sea walls of 15° and 45° were studied, with wave height, wave length, and water depth as the other variables. Reasonable agreement was found between calculated values and experimental values. However, no relationship was found between the relative ultimate scour depth, S/K, (where K is the distance from the wave trough to the bottom) and the coefficient of reflection.

Sato, et al, (6) in 1968 studied scour in front of 30°, 60° and 90° sea walls for both normal and storm beach profiles, using several grain sizes (d_{50} = 0.21, 0.38 and 0.69mm). A sea wall was installed after waves had acted upon a composite beach of 1:100 and 1:10 slopes and the beach had reached an equilibrium profile. Sato, et al, observed five types of scour and defined the mechanism producing each type of scour. They found that the relative scour depth, S/H_o, was greater for flatter waves, but that for waves of steepness 0.02 to 0.04 (normal storm waves) the relative scour depth was equal to 1.0. This indicated that the largest scour depth would be no greater than the deep water wave height that produced it. It was also found that the relative scour depth decreased with decreased relative median grain size (d_{50}/H_o). The placement of the sea wall along the beach profile, with respect to where the wave breaks, was studied. In the region seaward of the longshore bar and near the shoreline, the relative scour decreased as the angle of inclination decreased. Attempts were made to trace the movement of the sand during the tests using the radio-isotope of A_u 198 adhered to the sand.

Herbich (1) analyzed beach profile data collected at various locations along the Texas Gulf Coast. Scour depth was plotted against sand wave length, and the relationship was approximated by a straight line having a slope of 0.004. He also attempted to determine the relationship between the water depth to the trough of the sand wave and the depth to the crest of the sand bar. This ratio was determined for each profile taken and averaged for a particular location. The ratio was found to be 1.25 at Matagorda, 1.32 at Galveston, and 1.35 at Yarborough Pass on Padre Island. Keulegan (4) had found a value of 1.69 for his laboratory experiment.

TEST PROCEDURES

Facilities

Experiments were conducted in two wave channels. A glass-walled channel (2 ft wide x 3 ft deep x 120 ft long) was used for 20 tests. The wave generator was an oscillating-pendulum type with variable stroke and speed, which provided variable wave height and period. A dual channel Ultrasonic Distance Meter manufactured by Automation Industries was used to determine wave characteristics and the bottom profile and a Sanborn 4-channel recorder was used to record the data, see Figs. 1 and 2.[1]

The remaining experiments were conducted in a plexiglas flume (8 in. wide x 18 in. deep x 40 ft long). The wave generator, which also had variable stroke and speed, rolls on tracks, see Fig. 3. Wave data was collected using a capacitance-type wave gauge and recorded on a Hewlett-Packard two-channel recorder. Beach profile data was taken visually using a 1 in. grid system taped on one side of the flume, see Fig. 4. The sea walls were constructed of marine plywood.

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[1]All figures are collected in Appendix D at the end of the paper.

Procedure

Approximately 15 cu. yd. of (dry, uncompacted) sand was collected on the beach of Matagorda Peninsula about 1 mile northeast of the mouth of the Colorado (Texas) River. One cu. yd. of the sand was washed, cleaned and placed in the plexiglas flume. The washing removed most of the fine particles that become suspended in the water, which provided a clear view of discrete particle movement. Ten cu. yd. of sand was required to fill the glass-walled channel to the proper level. The sand was passed through a wire screen to remove shells, grass and broken glass. Due to the great volume of sand involved, it was impractical to wash the sand. As a result, visibility was almost completely obscured. The suspended sediment produced some scatter on the wave data taken by the Ultrasonic Distance Meter because the suspended sediment also returned signals to the probes, as did the air-water surface.

At the beginning of each test, the sea wall was installed at the proper location and the sand in front of it smoothed to the proper slope. Permagum sealed the space between the wall and the side of the channel.

For the sand to reach its ultimate scour depth generally required 24 hours. Some tests required as much as 36 to 48 hours. The data was taken at the end of each test.

TEST PARAMETERS

The significant variables in this study were (1) wave height, H; (2) wave length, L; (3) ultimate scour depth at the sea wall, S; (4) original water depth at the sea wall, d_s; (5) distance from the shoreline, X; (6) distance from the shoreline to point where waves break on a natural beach, X_b; (7) angle of inclination of the sea wall, measured from the horizontal, θ; (8) slope of the beach, α; and (9) coefficient of reflection, C_r. An expression combining these variables is

$$f(H, L, S, d_s, X, X_b, \theta, \alpha, C_r) = 0 \quad \text{. . . . . (2)}$$

From this expression eight dimensionless π-terms can be extracted:

$$\pi_1 = \frac{S}{H} \quad \text{. . . . . (3)}$$

$$\pi_2 = \frac{S}{L} \quad \text{. . . . . (4)}$$

$$\pi_3 = \frac{S}{d_S} \quad \text{. . . . . (5)}$$

$$\pi_4 = \frac{S}{X} \quad \text{. . . . . (6)}$$

$$\pi_5 = \frac{S}{X_b} \quad \text{. . . . . (7)}$$

$$\pi_6 = \theta \quad \text{. . . . . (8)}$$

$$\pi_7 = \alpha \quad \text{. . . . . (9)}$$

$$\pi_8 = C_r \quad \text{. . . . . (10)}$$

Other useful expressions can be formed by combining two or more of these π-terms:

$$\pi_9 = \frac{\pi_5}{\pi_4} = \frac{X}{X_b} \quad (11)$$

$$\pi_{10} = \frac{\pi_2}{\pi_1} = \frac{H}{L} \quad (12)$$

$$\pi_{11} = \frac{\pi_2}{\pi_3} = \frac{d_s}{L} \quad (13)$$

These π-terms can now be reformed into either of two expressions:

$$\frac{S}{H} = f' \left(\frac{d_s}{L}, \frac{X}{X_b}, \theta, \alpha, C_r\right) \quad (14)$$

or

$$\frac{S}{d_s} = f'' \left(\frac{H}{L}, \frac{X}{X_b}, \theta, \alpha, C_r\right) \quad (15)$$

In the 2 ft wide x 3 ft deep x 120 ft long wave channel, 20 tests were conducted. Studies conducted by Herbich, et al (3) had indicated that there was little difference in reflection from 45°, 67-1/2° and 90° sea walls, but there was significant variation between the 15°, 30° and greater than 45° walls. As a result, the 15°, 30° and 90° walls were selected for this study. The beach slopes selected represent those typical of Texas beaches: 1:40, 1:70 and 1:100. Four wave conditions were run against each of the three sea walls on a 1:40 slope. The steepest of these wave conditions was run against each wall on the 1:70 and 1:100 slopes.

The other two tests run were on 1:70 and 1:100 slopes without a sea wall.

In order to maintain a uniform condition for all tests, the sea wall was placed at the point where the wave broke on a natural beach ($X/X_b = 1.0$). For the 15° and 30° walls, the distance X was measured to the point where the sea wall intersected the beach slope. The wave periods selected for this project were 2 and 3 seconds, which determined the wave lengths. The deep water wave lengths were 20.48 ft and 46.08 ft. The wave heights were then selected by running several heights for each wave period on the beach without a sea wall and choosing waves that broke at a point 20 to 25 feet from the end of the channel. If the sea wall were placed much closer to the wave generator, the wave would not have a chance to be perfectly formed. The deep water wave heights were 0.15 ft and 0.21 ft for the 3 second waves, and .20 ft and .30 ft for the 2 second waves. Small amplitude wave theory was used to determine wave heights and lengths.

The 8 in. wide x 18 in. deep x 40 ft long wave channel was used to determine scour in front of dune barriers and the effect of placement of the sea wall on scour. A slope of 1:36 was selected in order to make the most effective use of the channel. Preliminary tests were run without a sea wall to determine the amount of scour that occurred naturally. In every case tested, using varying wave steepnesses (H_o/L_o), a dune barrier was formed at a position of $X/X_b = 0.2$ to 0.3 with a slope of 10° to 15°. Nine tests were run with the deep water wave steepness ranging from 0.001 to 0.050 to determine scour in front of dune barriers.

The next phase of testing in the small flume attempted to determine the effect of placement of the sea wall, with respect to where the wave breaks, on scour. One wave condition ($H_o = 0.151$ ft, $L_o = 20.48$ ft, $H_o/L_o = 0.007$) was run against 15°, 30° and 90° walls. The position of the 90° sea wall was varied with X/X_b = 0.19, 0.29, 0.33, 0.38, 0.50, 0.67, 1.00 and 1.20. For the 15° and 30° walls, X/X_b = 0.33, 0.50, 0.67, 1.00 and 1.20. Another wave condition ($H_o = 0.182$ ft, $L_o = 5.12$ ft, and $H_o/L_o = 0.036$) was run against the 90° wall at X/X_b = 0.33, 0.50, 0.67, 1.00 and 1.20.

The water depth was held constant in both tanks, which insured that wave conditions were identical from test to test. The depth of scour was measured from the original sand level.

THEORETICAL CONSIDERATIONS

Wave Theory

Small amplitude (Airy) wave theory assumes the wave shape to be sinusoidal and defines wave length as:

$$L = \frac{gT^2}{2\pi} \tanh \frac{2\pi d}{L} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (16)$$

which for deep water waves can be expressed as:

$$L_o = \frac{gT^2}{2\pi} = 5.12\ T^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (17)$$

The horizontal particle velocity can be written as:

$$u = \frac{gHT}{L} \frac{\cosh[\frac{2\pi}{L}(d+z)]}{\cosh(\frac{2\pi d}{L})} \sin(\frac{2\pi x}{L} - \frac{2\pi t}{T}) \quad \ldots\ldots \quad (18)$$

and the vertical particle velocity can be expressed as:

$$v = \frac{gHT}{L} \frac{\sinh[\frac{2\pi}{L}(d+z)]}{\cosh(\frac{2\pi d}{L})} \cos(\frac{2\pi x}{L} - \frac{2\pi t}{T}) \quad \ldots\ldots \quad (19)$$

The horizontal particle velocity is a function of wave height, wave length, wave period and water depth. For sinusoidal waves, the maximum shoreward horizontal velocity occurs under the crest of the wave. The maximum seaward horizontal velocity occurs under the trough of the wave and is equal to the shoreward maximum. As a wave moves into shallow water the wave form changes to a longer, shallower trough and a shorter, higher crest. As a result of this change in form, the particle velocities change so that the shoreward velocity is greater than the seaward velocity. This results in a net movement shoreward. Since water particle velocities are a function of wave height, wave length, wave period and water depth, they are dependent variables and, therefore, it was not necessary to include them as variables in this study.

The reflection of waves from a sea wall confuses the above simple situation. Particle velocities under the incident and reflected waves must be added vectorially at each point along the profile.

Coefficient of Reflection

For sinusoidal waves, over a constant depth, the reflection of waves results in a wave envelope with points of maximum and minimum wave heights. Using a wave gauge and recorder, the wave envelope can be recorded and the maximum and minimum wave heights measured. From this the incident wave height, reflected wave height and coefficient of reflection can be calculated using these equations:

$$H = \frac{H_{MAX} + H_{MIN}}{2} \quad (20)$$

$$H_R = \frac{H_{MAX} - H_{MIN}}{2} \quad (21)$$

$$C_r = \frac{H_R}{H} = \frac{H_{MAX} - H_{MIN}}{H_{MAX} + H_{MIN}} \quad (22)$$

Mechanics of Sediment Motion

The forces acting on a particle of sand on a beach under wave action are the force of gravity, the hydrodynamic forces, the reaction forces and the force of rolling friction. The forces acting parallel to the bottom and, therefore, those affecting motion along the bottom are the component of gravity in that direction, the hydrodynamic forces, which are a function of the horizontal particle velocity under the wave, and the force of rolling friction.

As the slope of the beach flattens, the force of gravity in the direction of motion becomes negligible. Under shoaling waves the maximum hydrodynamic forces occur under the crest of the wave and act in the shoreward direction. The reflection of waves complicates the situation and results in more exaggerated patterns of sand bars and troughs.

To determine sand movement along the profile would require the calculation of water particle velocities at each point under the incident and reflected waves for each depth.

The problem of scale effects is inherent in any movable-bed model. No attempt was made to model the tide, so there was no scale applied to this undistorted model. From a study such as this, it would be difficult to predict any definite relationship between the model study and prototype conditions. The extremely fine sand involved particularly complicates such efforts.

RESULTS

Tests Without a Sea Wall

Experiments were conducted to determine the amount of scour that occurs naturally (without a sea wall) on a smooth slope as a result of a particular wave condition. Preliminary tests were run in the plexiglas flume on a 1:36 beach slope using a variety of wave conditions. At the point where the wave broke a bar was formed and eventually the entire profile became a pattern of sand waves. The distance between sand crests, as other investigations have found, was approximately equal to half the water wave length at that point. In the region of $X/X_b = 0.2$ to 0.4 a dune barrier or berm was formed for every wave condition tested. The berm was built at the point where the incident wave, carrying sediment forward with it, met the backwash from the previous wave, causing the sand to settle. The slope of this berm was approximately 1:5 and according to Wiegel (8) is a function of the median grain size for a given degree of beach protection. The data from the study was well below the average slope (1:30) for protected beaches found by Wiegel. The still water level was kept constant, thus eliminating tidal effects. Because of the small tidal range along the Gulf Coast, this was not unrealistic.

A preliminary test was conducted in the glass-walled channel on the 1:40 slope without a sea wall. No data was taken, but it was observed that a dune barrier was also formed in approximately the same position and had the same slope. When tests were run on the 1:70 and 1:100 slopes, a bar was formed in the region of $X/X_b = 0.2$, but a berm was not. The wave broke at approximately the same depth, but the distance X_b was greater due to the flatter slope. As a result of the greater distance, more of the wave energy was expended in bottom friction and less sand was carried. The flatter slopes resulted in less wave energy reaching the shoreline and even less energy being reflected. Wave records for these tests indicated that the naturally occurring phenomenon of surf beat occurred, which further emphasized the authenticity of the experiments. Results showed that scour depth decreased as the slope flattened. Fig. 5 is a graph of scour depth versus relative distance from the shoreline for the three beach slopes.

Scour in front of dune barriers was studied for nine wave conditions on the 1:36 slope in the small channel. The only variables were wave height and wave length. Figs. 6-8 are plots of scour depth versus relative distance from the shoreline for each wave height. Figs. 6 and 7, for L_o = 5.12 ft and 20.48 ft respectively, indicate an increase in scour with increase in wave height. The three graphs also reveal a point of maximum scour at $X/X_b = 0.5$ to 0.9, or just inshore of the bar where the wave breaks. The region just inshore of the bar was the area of greatest turbulence and should be expected to be the area of maximum scour. In several cases tested, the beach profile offshore from the berm flattened to approximately 1:100. Fig. 9 is a plot of the scour depth to original water depth ratio versus relative distance from the shoreline for each wave steepness, and indicates that scour is not a function of wave steepness. Fig. 10 is a graph of relative scour (S/H_o) versus relative distance from the shoreline for each wave length and gives little indication of the effect of wave length on scour depths.

Fig. 11 is a graph of maximum scour depth for a particular wave condition versus the deep water wave height for each wave length. From this plot it can be approximated that the maximum scour depth is equal to the deep water wave height.

Beach profiles have been taken on other research projects along Matagorda Peninsula near the mouth of the Colorado (Texas) River and near Brown Cedar Cut. The beach near Brown Cedar Cut has a slope of approximately 1:46 and a steep berm, slope of 1:12, exists at the water line. The laboratory study with 1:36 and 1:40 slopes produced a berm with a slope of 1:6 to 1:10. Surveys taken near the mouth of the Colorado River show a slope of 1:60 with a slightly steeper slope near the shoreline. Laboratory tests with 1:70 and 1:100 resulted in profiles similar to that near the Colorado River. It would be unwise to attempt to set a linear scale between the model and prototype or to predict exact quantities in the prototype from results of this study.

Sea Wall Tests

Effect of Relative Distance

Experiments were conducted in the small flume to determine the effect of relative distance. In all four sets of experiments, there existed a point of maximum scour when the sea wall was placed in the region of $X/X_b = 0.5$ to 0.67. When the sea wall was placed seaward of this region, less scour occurred at the toe of the wall, even at the point of wave breaking, $X/X_b = 1.0$. As the sea wall was moved shoreward of the point of maximum scour, the scour decreased until the wall was placed at a "critical point." At this critical point the sand built up to a certain level and then the wave action scoured out the sand build-up. When the wall was inserted shoreward of this critical point, accretion occurred from the beginning of the test. Fig. 12 is a graph of scour depth in front of a 90° wall versus relative distance for two wave conditions ($H_o/L_o = 0.007$ and 0.036). Fig. 13 is a plot of scour depth versus relative distance of the sea wall from the shoreline for one wave condition ($H_o/L_o = 0.007$) against 15°, 30° and 90° sea walls. Points where zero scour is indicated represent accretion. The regions of scour and accretion for the sea wall tests were similar to regions of scour and accretion for natural beach tests. The maximum scour occurred when the sea wall was placed in the area of maximum turbulence, $X/X_b = 0.5$ to 1.0.

Beyond the critical point on the profile where accretion occurred, the reformed waves lost energy due to bottom friction. The reflected wave energy decreased sufficiently to cause the shoreward movement of sand to meet less resistance and resulted in a net movement of sand shoreward.

Effect of Wave Height

The attempt to determine the effect of wave height was limited to four wave conditions (two wave heights for each of two wave lengths) because of insufficient time available for use of the test facilities. Fig. 14 is a graph of scour depth just in front of the sea wall versus the deep water wave height for the two wave lengths and for the 15° and 90° walls. For a particular wave length scour increased with increase in wave height, because of the greater energies and velocities associated with higher waves. With the limited amount of data involved, definite relationships between scour depth and wave height were impossible to obtain.

Effect of Wave Length

As in the case of wave height, attempts to determine the effects of wave length were hampered by an abbreviated test schedule. The results plotted on Fig. 14 were taken from tests conducted in the glass-walled channel. It would be unreasonable to expect to determine an exact relationship from this sparse amount of data, but it appeared that for a particular wave height scour increased with increased wave length.

Fig. 12 results were taken from two sets of experiments conducted in the plexiglas channel. The wave heights in both cases were approximately the same; that is, the generated wave heights were identical. Because the wave lengths differed, the deep water wave heights differed. For longer waves the scour increased at all points where the wall was installed. The greater wave length resulted in greater energy and particle velocities and, therefore, greater sand movement.

Effect of Beach Slope

To determine the effect of beach slope, one wave condition (H_o = 0.30 ft, L_o = 20.48 ft, H_o/L_o = 0.015) was run against each of the three sea walls for 1:40, 1:70 and 1:100 beach slopes. Fig. 15 is a plot of scour versus beach slope for the three sea walls. With the exception of one data point, the general tendency apparent in the graph was a decrease in scour as the beach slope flattened. On the 1:100 slope accretion occurred at the toe of the 15° and 30° walls. This happened as a result of a change in the relative distance X/X_b. A test was conducted without a sea wall to determine the point where waves broke on a natural beach. The sea walls were then placed at the point where waves broke, making X/X_b = 1.0 and a constant. For the 1:100 slope the point at which the waves broke was 55 ft from the wave generator, which would have left 45 ft of the tank unused. Rather than change the water level, which would have necessitated changing wave height, some of the sand was removed. This moved the position of the wall back 25 ft and made more effective use of the channel. When the 15° and 30° walls were installed, the wave action moved the sand into the normal sand wave profile. However, the bar just seaward of the wall built up to the point where waves broke over it and changed the relative distance of the sea wall from $X/X_b = 1.0$ to 0.74. This change, to some degree, invalidated the test for its original purpose. The results did, however, manifest a problem not considered in this study and magnified the importance of future consideration.

The occurrence of this phenomenon illustrated that the location of the "critical point" found in the study of the effect of relative distance might depend, to some degree, on beach slope. As was brought out in the natural beach tests on the three slopes, the loss of energy due to bottom friction decreased the sand movement and changed scour patterns.

Effect of Angle of Inclination of Sea Wall

The experiments in the glass-walled channel were conducted with angle of sea wall as the primary variable of importance. The results obtained did not indicate any relationship between scour and sea wall angle. Fig. 14 illustrates this variation in data. For the 20 ft wave the 15° wall produced greater scour, while the 90° wall produced greater scour for the 46 ft wave. One explanation for this discrepancy could be the placement of the inclined sea walls. In every case the 15° and 30° wall was placed so that the sea wall intersected the beach slope at the point where waves broke. The other alternative would have been to place the sea wall where it intersected the water level at the point of wave breaking.

Tests conducted in the plexiglas channel provided more conclusive evidence of the effect of the angle of the sea wall. Fig. 13 illustrates the effect of variation of sea wall slope for one wave condition ($H_o/L_o = 0.007$). The 30° wall produced 50% less maximum scour than the 90° wall, and at only one location did the 15° wall produce scour, that being at $X/X_b = 1.2$.

Effect of Coefficient of Reflection

The most disappointing part of the research was the attempt to determine the reflection. Undoubtedly, reflection of wave energy produced significant changes in water particle velocities and sediment carrying potential. The problem arose in measuring this coefficient of reflection. It was thought that the previously described theory for determining reflection coefficient would suffice for this case. The results indicated the contrary, however. In order to determine this coefficient, a section of constant depth was necessary. This was not possible for tests conducted in the plexiglas channel or for tests conducted in the glass-walled channel for the 1:70 and 1:100 slopes. The constant depth section was necessary in order to eliminate effects of wave height changes due to shoaling. Even when a constant depth was available for these measurements (for the 1:40 slope tests in the large channel) useful results were not obtained. The waves produced were in the range of cnoidal wave theory and were not sinusoidal, which negated the use of the sinusoidal theory for reflection coefficient. A wire mesh filter installed in front of the wave generator did not adequately filter out the longer, flatter waves, which resulted in re-reflected waves and affected the wave profiles. The sand wave profile produced, in some cases, rather deep troughs. Waves traveling over the bar generated reflected waves when passing suddenly into a deeper section. When waves reflected from the wall and hit incident waves, the sum of the two wave heights was often great enough to cause the wave to break which reduced both the incident and reflected wave heights an undetermined amount. For cnoidal waves the reflection should have been measured at the sea wall and should have been the ratio of the reflected and incident wave amplitudes measured from still water level.

Problems Encountered

The author encountered several problems in conducting these experiments. Surf beat and long waves produced in the tank caused problems in determining still water level in the wave envelope when taking measurements continuously along the channel. This also produced a problem in trying to calibrate the recorder, which was done by measuring the difference between the crest and trough of a wave at a particular point and comparing this difference with the recorded heights.

Waves generated in a wave channel have produced transverse oscillations when the wave length was an exact integer multiple of the channel width. Researchers studying a constant depth have avoided this problem by avoiding those critical wave lengths. This problem can not be avoided when using shoaling waves. During several of the tests conducted in the glass-walled channel, the generated waves were complicated by the presence of these transverse oscillations. It would be difficult to determine the amount of energy lost to these oscillations from the generated wave. The effect on scour formations was likewise impossible to determine.

The wave generator did not produce constant wave heights due to an inefficient wave absorber behind the generator. This problem was overcome by averaging the wave heights.

Scatter on the wave record produced on the Ultrasonic Distance Meter by suspended sediment caused some problems in determining the crest of the waves. No problem was encountered in determining the beach profile.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

Natural Beach Tests

Sufficient data has been taken on the 1:40 slope to conclude that the existence of the berm formed in the region of $X/X_b = 0.2$ is not a function of wave conditions, but of the beach slope. The slope of this berm was a constant and according to Wiegel (8) is a function of the sediment size. Maximum scour on the 1:40 slope is approximately equal to the deep water wave height. Further study would be necessary to determine what effect various wave conditions have on 1:70 and 1:100 slopes. The difference in results from the same wave condition on the three slopes magnifies the need for further study.

Sea Wall Tests

The most significant result of this research was the effect of the position of the sea wall. Obviously once a sea wall is constructed its position does not change, but its relative position changes with each change in wave length. This relative position is perhaps as critical as the wave height to the amount of scour that occurs.

As other investigators have found, wave height is the most significant variable to the depth of scour. The exact effect of wave length is not apparent as yet, and should be determined in further studies.

As the beach slope flattens, scour decreases. Scour decreases as the angle of inclination of the wall decreases, which indicates that scour decreases with decrease of coefficient of reflection.

When the recorded wave envelope does not indicate as great a variation in the wave trough as in the wave crest, the sinusoidal theory for wave reflection does not hold.

Recommendations

Future projects should study the effect of beach slope and the relative position of the wall on more slopes and under a greater variety of wave conditions. Wave conditions that are irregular to some degree should also be tried. The speed of the generator could be varied 10% either side of the desired position for equal lengths of time or wind generated waves could be used. Dyed sand could be used to trace the discrete particle movement at various points along the profile. Further study should be devoted to adequately determine reflection of cnoidal waves. A study could also be conducted to determine the degree of sediment sorting that occurs.

APPENDICES

APPENDIX A

REFERENCES

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APPENDIX B

NOMENCLATURE

A = $H + H_R$

C_D = drag coefficient

C_r = reflection coefficient

d = mean water depth

d_c = depth to crest of sand bar

d_s = initial water depth at sea wall

d_T = depth to trough of sand wave

d_{50} = median grain size

g = acceleration of gravity

H = wave height

H_{MAX} = maximum wave height

H_{MIN} = minimum wave height

H_o = deep water wave height

H_R = reflected wave height

L = wave length

L_o = deep water wave length

S = ultimate scour depth at the sea wall or ultimate scour depth of a particular point along the profile without a sea wall

T = wave period

t = time

U_* = horizontal velocity within boundary

u = horizontal particle velocity

v = vertical particle velocity

X = distance from the sea wall or a particular point along the profile without a sea wall to the original shoreline

X_b = distance from the point where the wave breaks to the original shoreline

z = distance from still water level to wave surface

ρ = water density

ϕ = angle of repose of sand

θ = angle of inclination of the sea wall, measured from the horizontal

γ = unit weight of water

γ_s = unit weight of sand

α = beach slope

APPENDIX C

TEST DATA

Table No. 1 - Tests Conducted Without Sea Wall

Test No.	Test Facility	α	L_o	H_o	H_o/L_o	X_b	X	X/X_b	S
11	A	1:36	5.12	0.091	0.018	6.17	2.7	0.44	0.058
							3.9	0.63	0.067
							5.3	0.89	0.063
12	A	1:36	5.12	0.118	0.023	7.50	2.7	0.36	0.100
							3.9	0.52	0.100
							5.3	0.71	0.100
							6.6	0.88	0.083
							8.3	1.11	0.083
							9.8	1.31	0.067
							11.4	1.52	0.042
							13.0	1.73	0.033
							14.8	1.97	0.025
							16.7	2.23	0.033
							18.8	2.51	0.054
14	A	1:36	5.12	0.182	0.036	6.50	2.8	0.43	0.125
							4.1	0.63	0.146
							5.4	0.83	0.133
							7.0	1.08	0.108
							8.5	1.31	0.083
							10.2	1.57	0.083
							11.8	1.82	0.042
							13.5	2.08	0.000
							16.8	2.58	+0.050

Table 1 - Continued

Test No.	Test Facility	α	L_o	H_o	H_o/L_o	X_b	X	X/X_b	S
13	A	1:36	5.12	0.254	0.050	13.50	3.2	0.24	0.158
							4.5	0.33	0.142
							6.3	0.47	0.117
							11.4	0.84	0.300
							15.0	1.11	0.067
							16.8	1.24	0.000
15	A	1:36	20.48	0.075	0.004	10.42	2.9	0.28	0.042
							4.9	0.47	0.058
							8.0	0.77	0.017
16	A	1:36	20.48	0.151	0.007	10.50	3.0	0.29	0.092
							4.2	0.40	0.142
							6.7	0.64	0.175
							10.0	0.95	0.158
							13.0	1.24	0.067
							16.8	1.60	0.033
17	A	1:36	20.48	0.211	0.010	15.50	3.4	0.22	0.133
							4.6	0.30	0.200
							7.9	0.51	0.192
							11.1	0.72	0.208
							14.8	0.95	0.167
							18.5	1.19	0.017
18	A	1:36	46.08	0.063	0.001	6.67	6.6	0.99	0.192
							11.4	1.71	0.067
							15.4	2.31	0.025

Table 1 - Continued

Test No.	Test Facility	α	L_o	H_o	H_o/L_o	X_b	X	X/X_b	S
19	A	1:36	46.08	0.126	0.003	12.67	6.7	0.53	0.058
							7.0	0.55	0.067
							8.5	0.67	0.192
							9.8	0.77	0.017
							14.6	1.15	0.133
							18.8	1.48	0.092
119	B	1:70	20.48	0.30	0.015	30.0	1.2	0.04	0.038
							15.0	0.50	0.093
							18.7	0.62	0.116
							21.75	0.73	0.130
							25.9	0.86	0.137
							39.0	1.30	0.049
							55.0	1.83	0.120
120	B	1:100	20.48	0.30	0.015	52.5	16.2	0.31	0.017
							41.8	0.80	0.085

Test Facility A - 8 in. wide x 18 in. deep x 40 ft long plexiglas flume.

Test Facility B - 2 ft wide x 3 ft deep x 120 ft long glass-walled channel.

Table No. 2 - Tests Conducted with a Sea Wall

Test No.	Test Facility	α	θ	L_o	H_o	H_o/L_o	X_b	X/X_b	S
53	A	1:36	90°	20.48	0.151	0.007	10.50	0.19	+0.142
52	"	"	"	"	"	"	"	0.29	+0.108
56	"	"	"	"	"	"	"	0.33	0.083
55	"	"	"	"	"	"	"	0.38	0.192
54	"	"	"	"	"	"	"	0.50	0.250
51	"	"	"	"	"	"	"	0.67	0.242
57	"	"	"	"	"	"	"	1.00	0.167
58	"	"	"	"	"	"	"	1.20	0.175
64	"	"	30°	"	"	"	"	0.29	0.083
63	"	"	"	"	"	"	"	0.38	0.092
62	"	"	"	"	"	"	"	0.50	0.083
61	"	"	"	"	"	"	"	0.67	0.125
60	"	"	"	"	"	"	"	1.00	0.083
59	"	"	"	"	"	"	"	1.20	0.033
66	"	"	15°	"	"	"	"	0.33	0.000
67	"	"	"	"	"	"	"	0.50	+0.058
68	"	"	"	"	"	"	"	0.67	0.000
69	"	"	"	"	"	"	"	1.00	0.000
70	"	"	"	"	"	"	"	1.20	0.117
71	"	"	90°	5.12	0.182	0.036	6.50	0.33	+0.083
72	"	"	"	"	"	"	"	0.50	0.100
73	"	"	"	"	"	"	"	0.67	0.108
74	"	"	"	"	"	"	"	1.00	0.108
75	"	"	"	"	"	"	"	1.20	+0.125

Table 2 - Continued

Test No.	Test Facility	α	θ	L_o	H_o	H_o/L_o	X_b	X/X_b	S
105	B	1:40	15°	46.08	0.15	0.003	15.00	1.00	0.152
103	"	"	30°	"	"	"	"	"	0.249
106	"	"	90°	"	"	"	"	"	0.277
112	"	"	15°	"	0.28	0.006	20.00	"	0.416
109	"	"	30°	"	"	"	"	"	0.273
108	"	"	90°	"	"	"	"	"	0.433
104	"	"	15°	20.48	0.20	0.010	15.00	"	0.208
102	"	"	30°	"	"	"	"	"	0.512
101	"	"	90°	"	"	"	"	"	0.136
111	"	"	15°	"	0.30	0.015	20.00	"	0.328
110	"	"	30°	"	"	"	"	"	0.375
107	"	1:70	90°	"	"	"	"	"	0.275
113	"	"	15°	"	"	"	30.00	"	0.077
114	"	"	30°	"	"	"	"	"	0.384
115	"	1:100	90°	"	"	"	"	"	0.234
118	"	"	15°	"	"	"	52.2	"	+0.229
117	"	"	30°	"	"	"	"	"	+0.565
116	"	"	90°	"	"	"	"	"	0.061

Test Facility A - 8 in. wide x 18 in. deep x 40 ft long plexiglas flume.

Test Facility B - 2 ft wide x 3 ft deep x 120 ft long glass-walled channel.

Table No. 3 - Data for Figs. 9 and 10

Test No.	L_o	H_o	H_o/L_o	X	d	X/X_b	S	S/d	S/H_o
11	5.12	0.091	0.018	2.7	0.075	0.44	0.058	0.773	0.637
				3.9	0.108	0.63	0.067	0.670	0.736
				5.3	0.147	0.89	0.063	0.429	0.692
12	5.12	0.118	0.023	2.7	0.075	0.36	0.100	1.333	0.874
				3.9	0.108	0.52	0.100	0.926	0.874
				5.3	0.147	0.71	0.100	0.680	0.874
				6.6	0.183	0.88	0.083	0.454	0.703
				8.3	0.231	1.11	0.083	0.359	0.703
				9.8	0.272	1.31	0.067	0.246	0.568
				11.4	0.317	1.52	0.042	0.132	0.356
				13.0	0.361	1.73	0.033	0.091	0.280
				14.8	0.411	1.97	0.025	0.061	0.212
				16.7	0.464	2.23	0.033	0.071	0.280
				18.8	0.522	2.51	0.054	0.103	0.458
14	5.12	0.182	0.036	2.8	0.078	0.43	0.125	1.603	0.687
				4.1	0.114	0.63	0.146	1.281	0.802
				5.4	0.150	0.83	0.133	0.887	0.731
				7.0	0.194	1.08	0.108	0.557	0.593
				8.5	0.236	1.31	0.083	0.352	0.456
				10.2	0.283	1.57	0.083	0.293	0.456
				11.8	0.328	1.82	0.042	0.128	0.231
				13.5	0.375	2.08	0.000	0.000	0.000
				16.8	0.467	2.58	+0.050	0.000	0.000

Table 3 - Continued

Test No.	L_o	H_o	H_o/L_o	X	d	X/X_b	S	S/d	S/H_o
13	5.12	0.254	0.050	3.2	0.089	0.24	0.158	1.775	0.622
				4.5	0.125	0.33	0.142	1.136	0.559
				6.3	0.175	0.47	0.117	0.669	0.461
				11.4	0.317	0.84	0.300	0.946	1.181
				15.0	0.417	1.11	0.067	0.161	0.264
				16.8	0.467	1.24	0.000	0.000	0.000
15	20.48	0.075	0.004	2.9	0.081	0.28	0.042	0.519	0.560
				4.9	0.136	0.47	0.058	0.426	0.773
				8.0	0.222	0.77	0.017	0.077	0.227
16	20.48	0.151	0.007	3.0	0.083	0.29	0.092	1.108	0.609
				4.2	0.117	0.40	0.142	1.214	0.940
				6.7	0.186	0.64	0.175	0.941	1.159
				10.0	0.278	0.95	0.158	0.568	1.046
				13.0	0.361	1.24	0.067	0.186	0.444
				16.8	0.467	1.60	0.033	0.071	0.219
17	20.48	0.211	0.010	3.4	0.094	0.22	0.133	1.415	0.630
				4.6	0.128	0.30	0.200	1.563	0.948
				7.9	0.219	0.51	0.192	0.877	0.910
				11.1	0.308	0.72	0.208	0.675	0.986
				14.8	0.411	0.95	0.167	0.406	0.791
				18.5	0.514	1.19	0.017	0.033	0.081
18	46.08	0.063	0.001	6.6	0.183	0.99	0.192	1.049	3.048
				11.4	0.317	1.71	0.067	0.211	1.063
				15.4	0.428	2.31	0.025	0.058	0.397

Table 3 - Continued

Test No.	L_o	H_o	H_o/L_o	X	d	X/X_b	S	S/d	S/H_o
19	46.08	0.126	0.003	6.7	0.185	0.53	0.058	0.312	0.460
				7.0	0.194	0.55	0.067	0.345	0.532
				8.5	0.236	0.67	0.192	0.814	1.524
				9.8	0.272	0.77	0.017	0.063	0.135
				14.6	0.406	1.15	0.133	0.328	1.056
				18.8	0.522	1.48	0.092	0.176	0.730

APPENDIX D

FIGURES

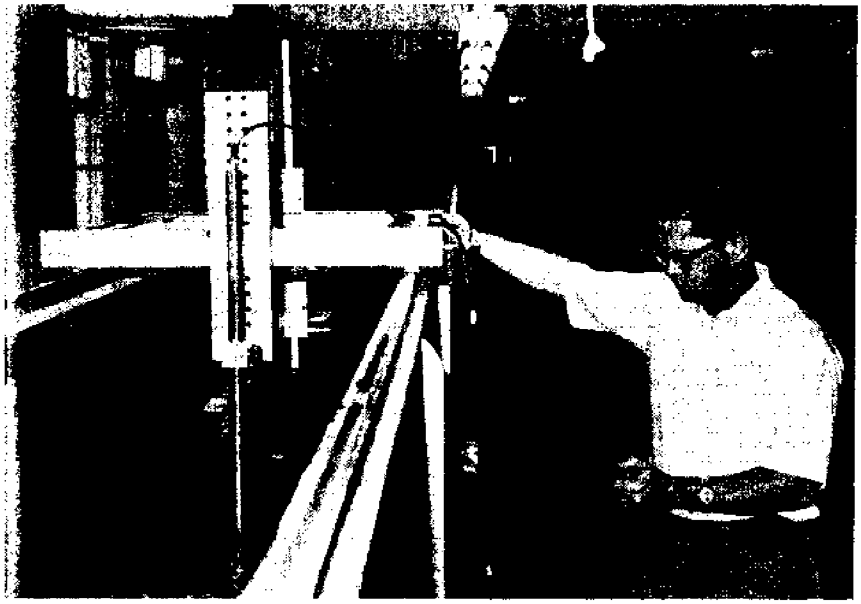


Fig. 1 - Ultrasonic Distance Meter probes in glass-walled channel

Fig. 2 - UDM Transmitter/Receiver and Sanborn Recorder

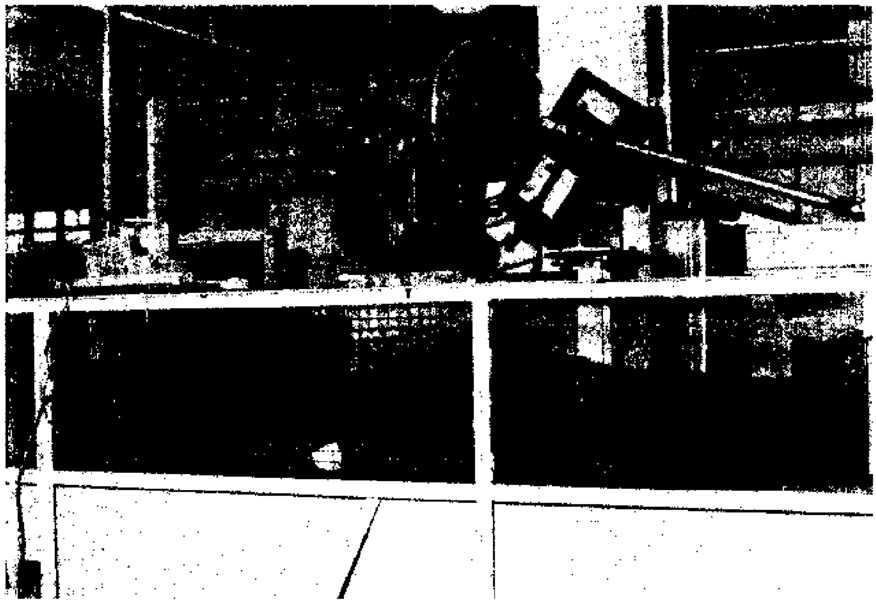


Fig. 3 - Wave Generator in Plexiglas Flume

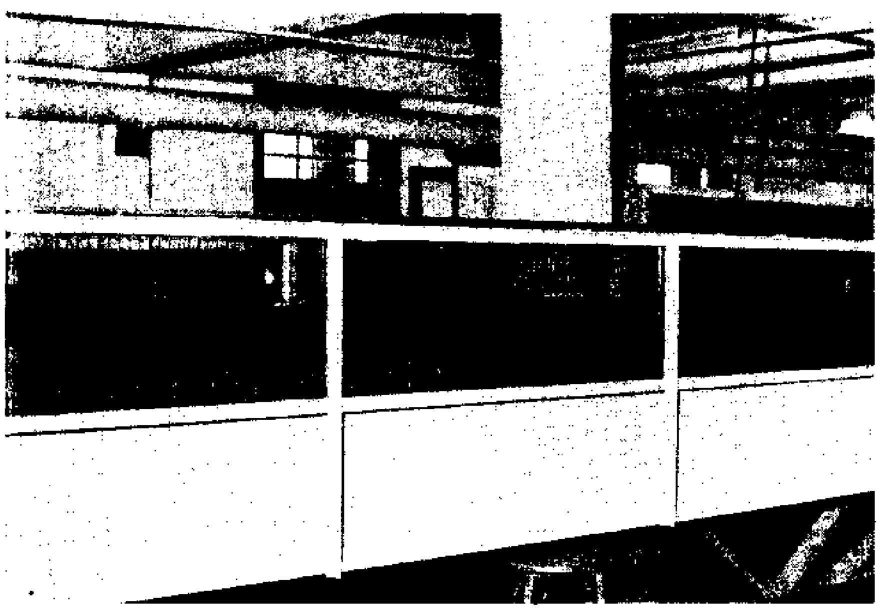


Fig. 4 - Plexiglas Flume

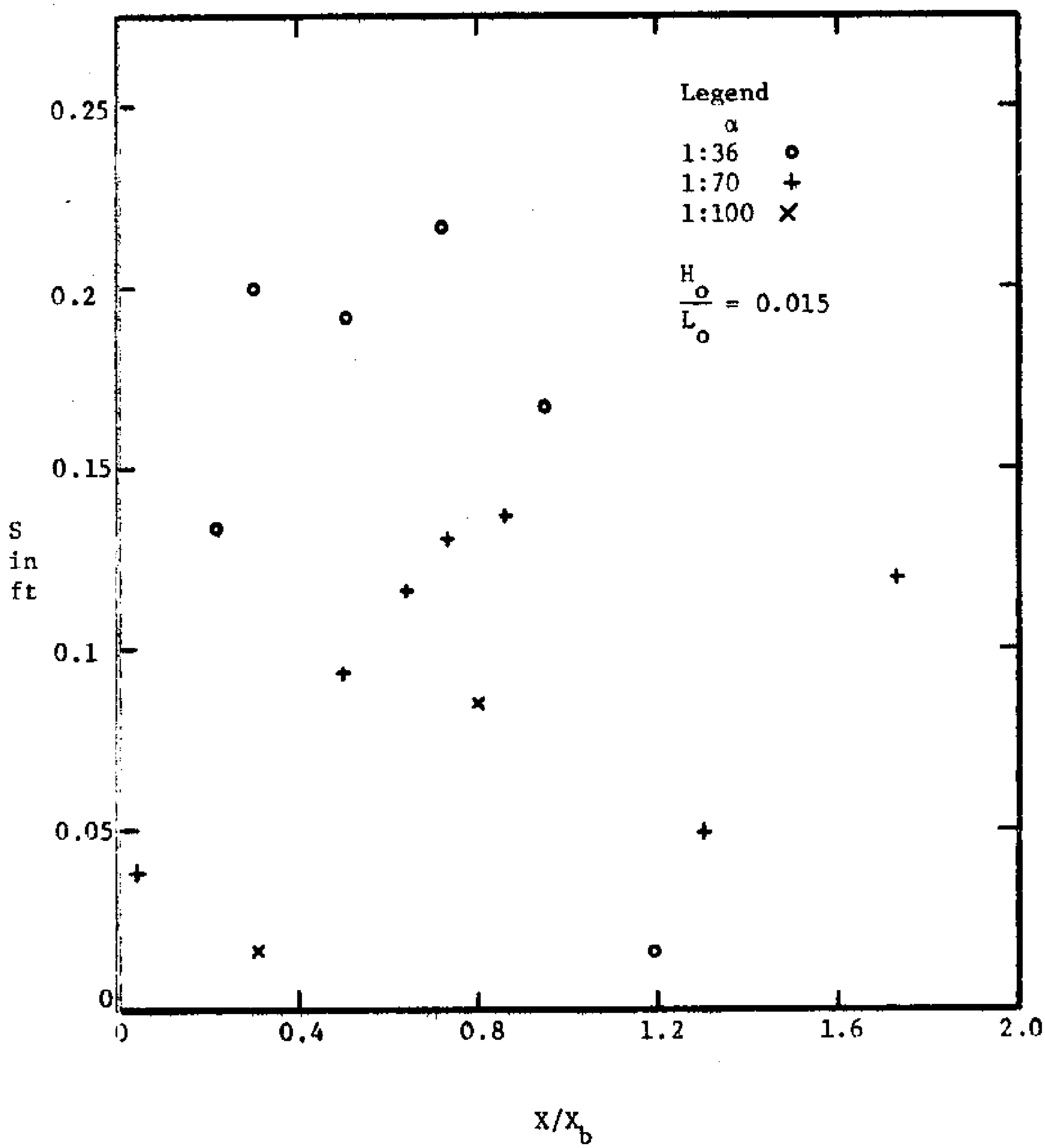


Fig. 5 - Scour depth versus relative distance from the shoreline for three beach slopes without a sea wall

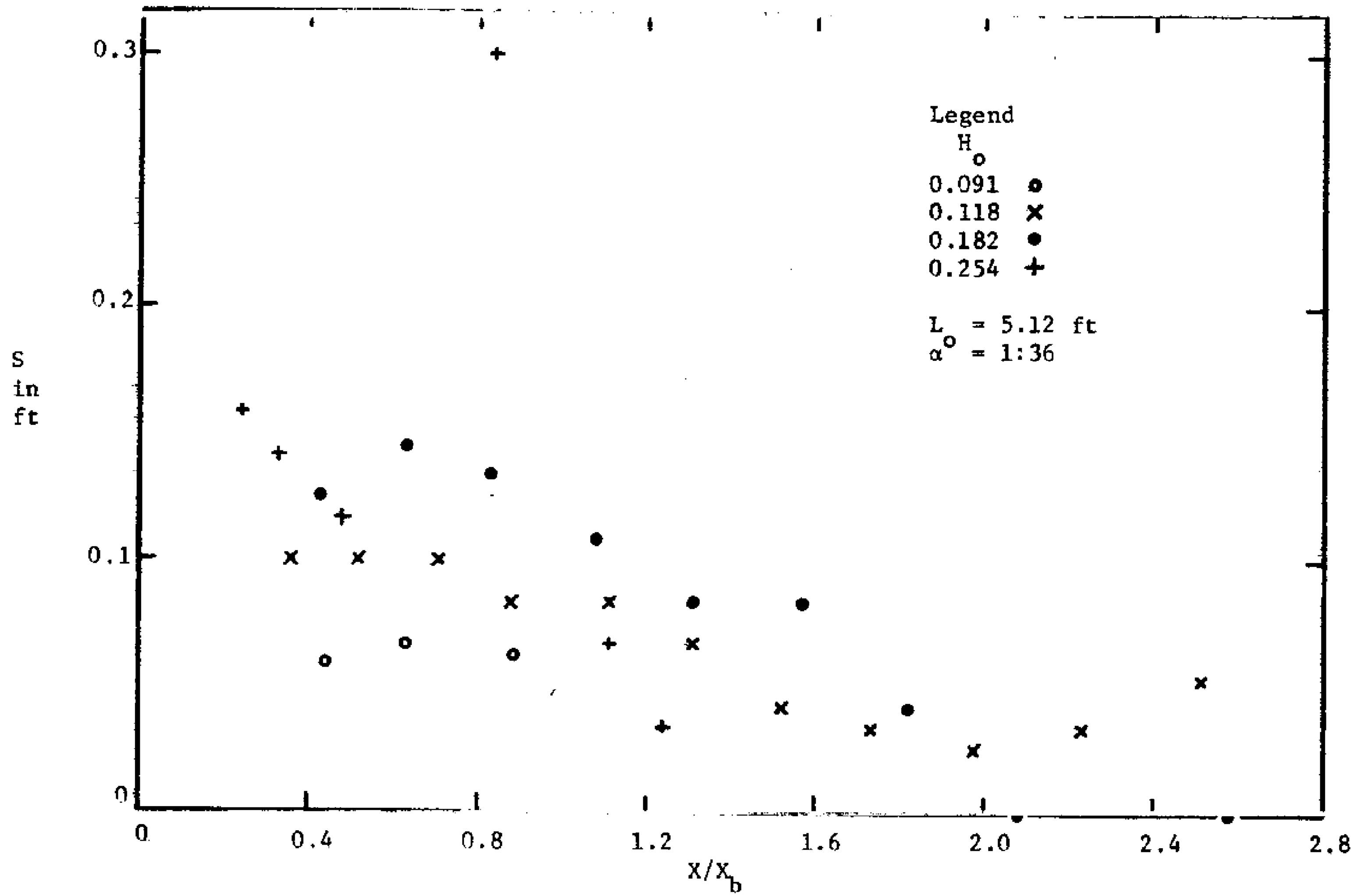


Fig. 6 - Scour depth versus relative distance from the shoreline for four wave heights and L_o = 5.12 ft

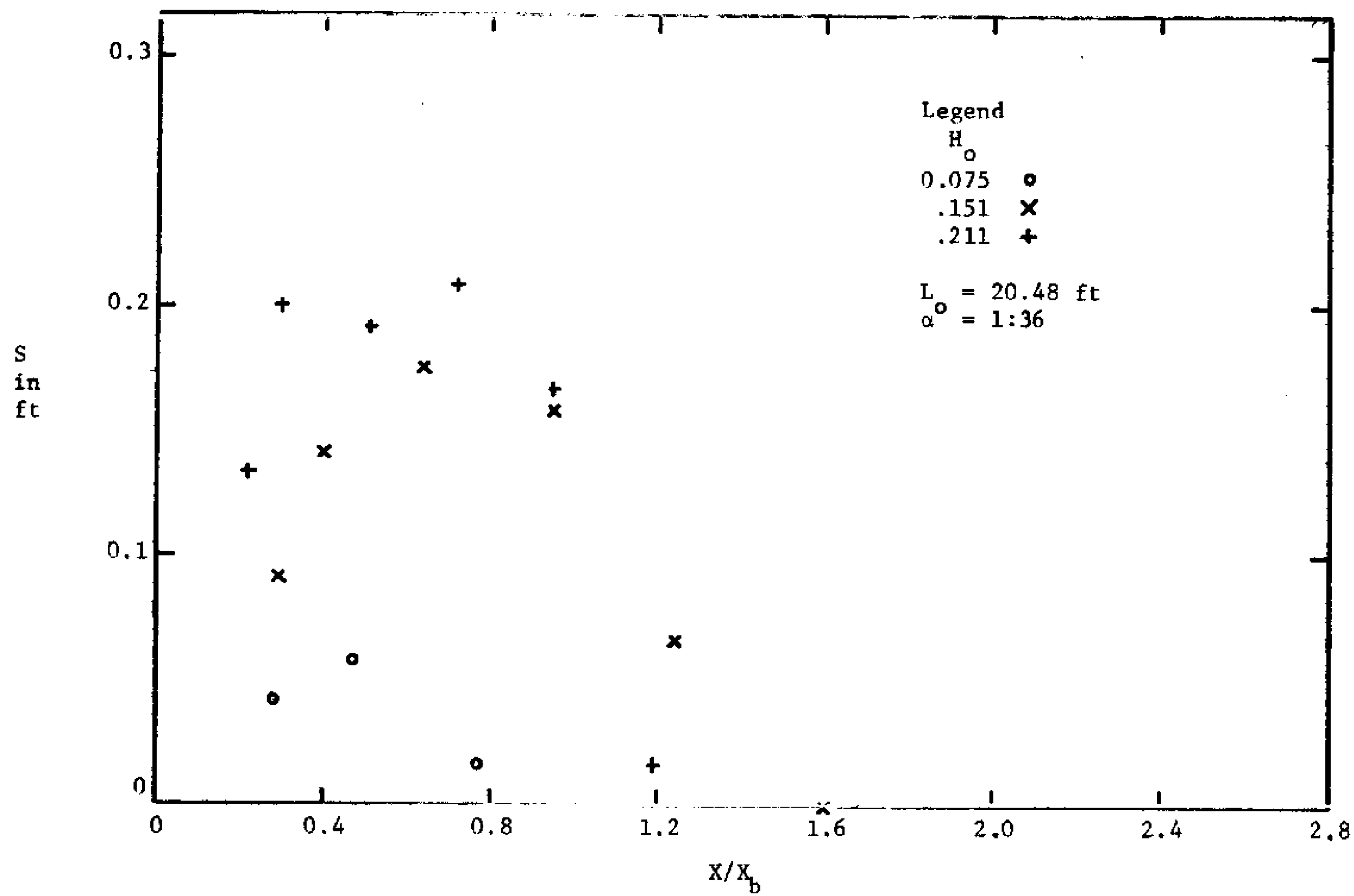
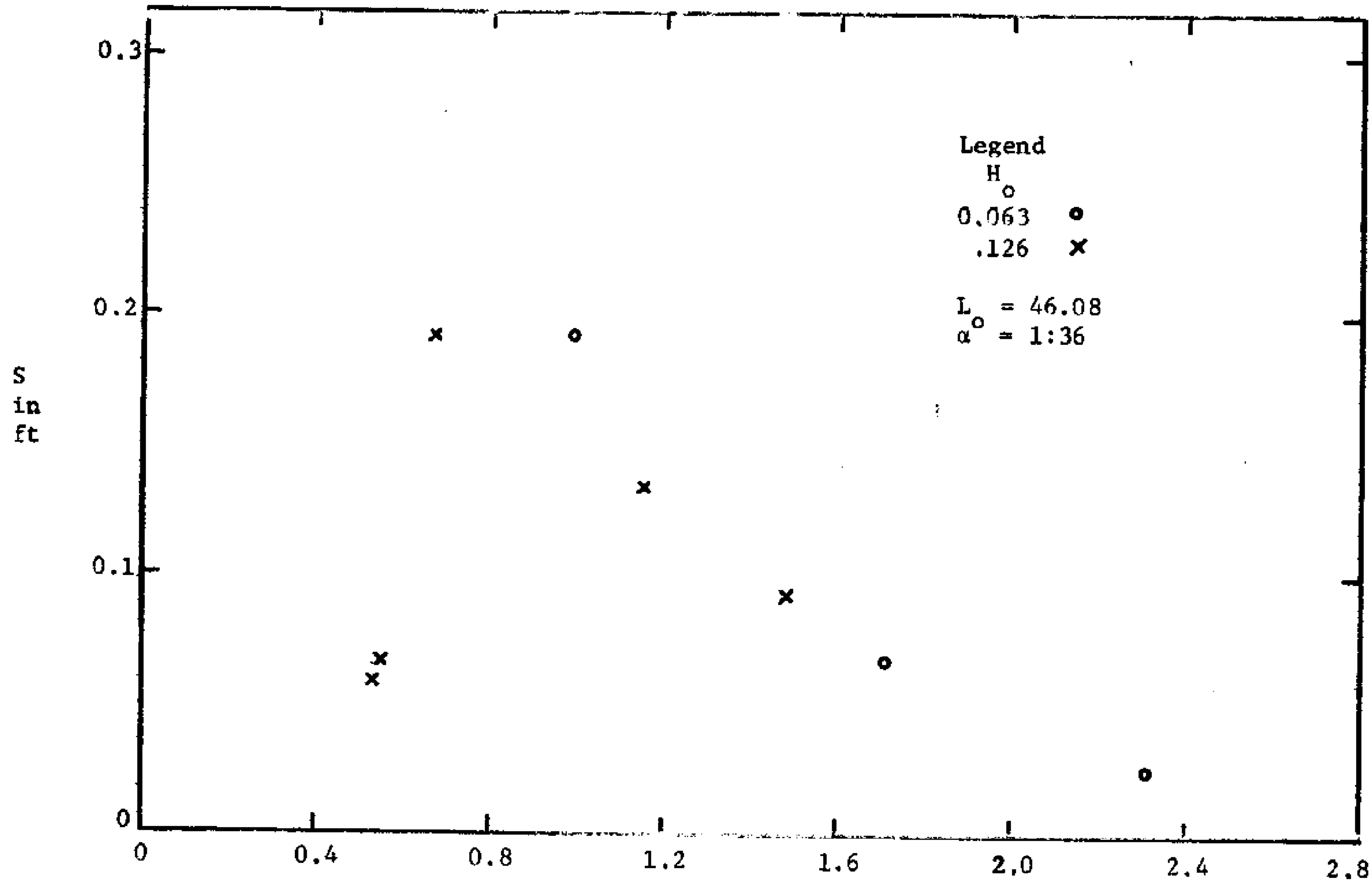


Fig. 7 - Scour depth versus relative distance from the shoreline for three wave heights and L_o = 20.48 ft



X/X_b

Fig. 8 - Scour depth versus relative distance from the shoreline for two wave heights and L_o = 46.08 ft

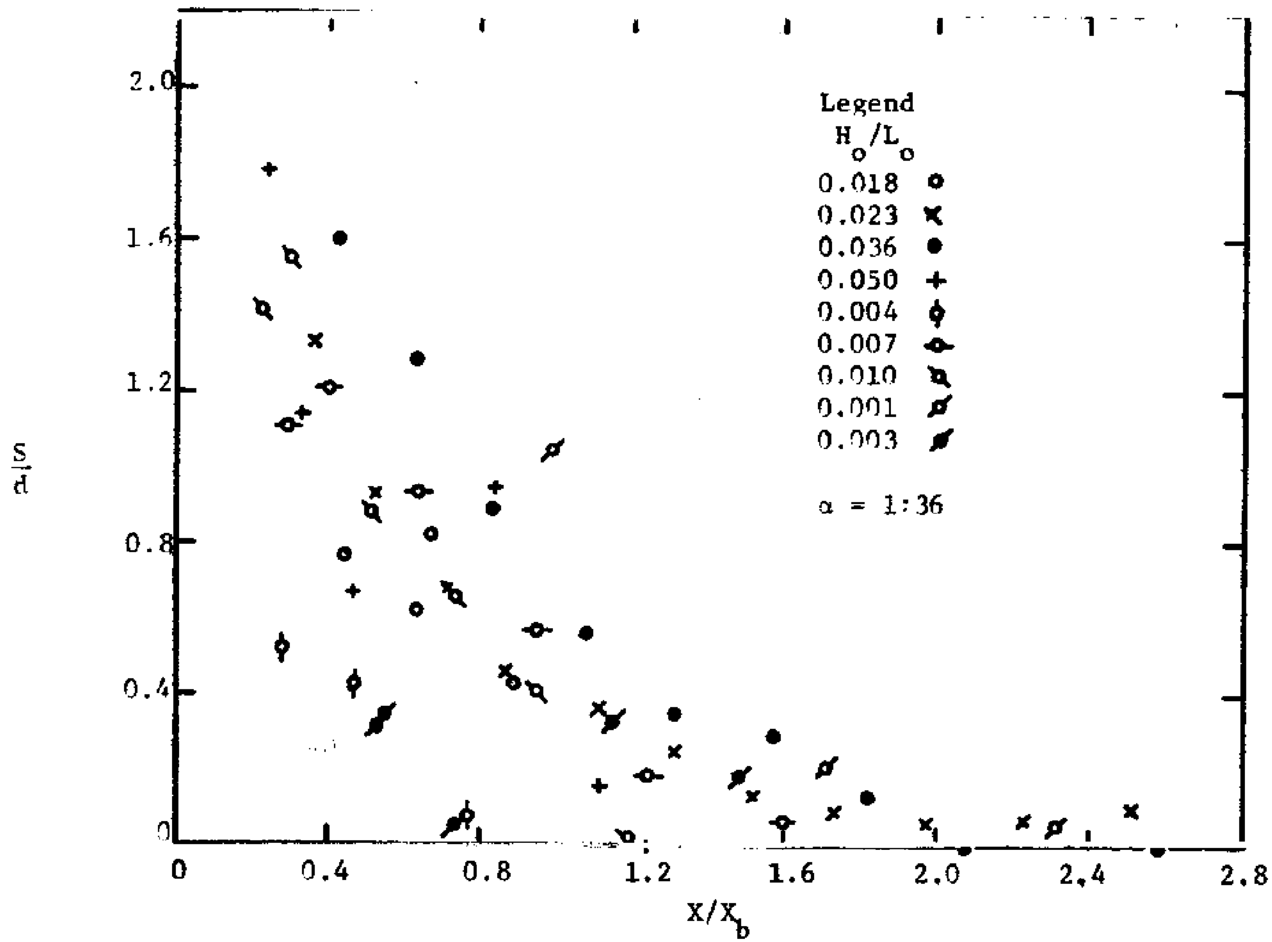


Fig. 9 - Scour depth to original water depth ratio versus relative distance from the shoreline for varying wave steepness

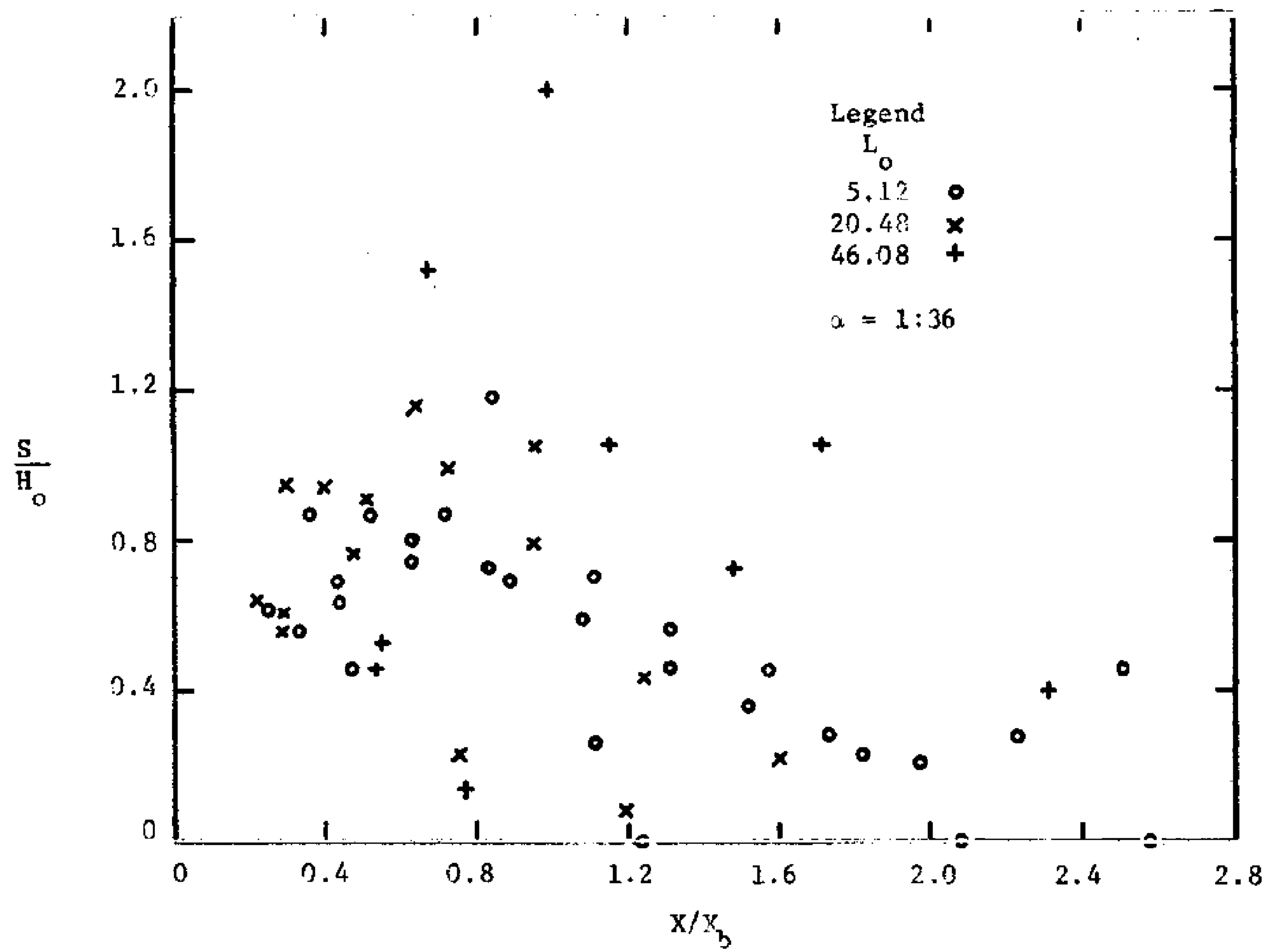


Fig. 10 - Scour depth to deep water wave height ratio versus relative distance from the shoreline for varying wave length

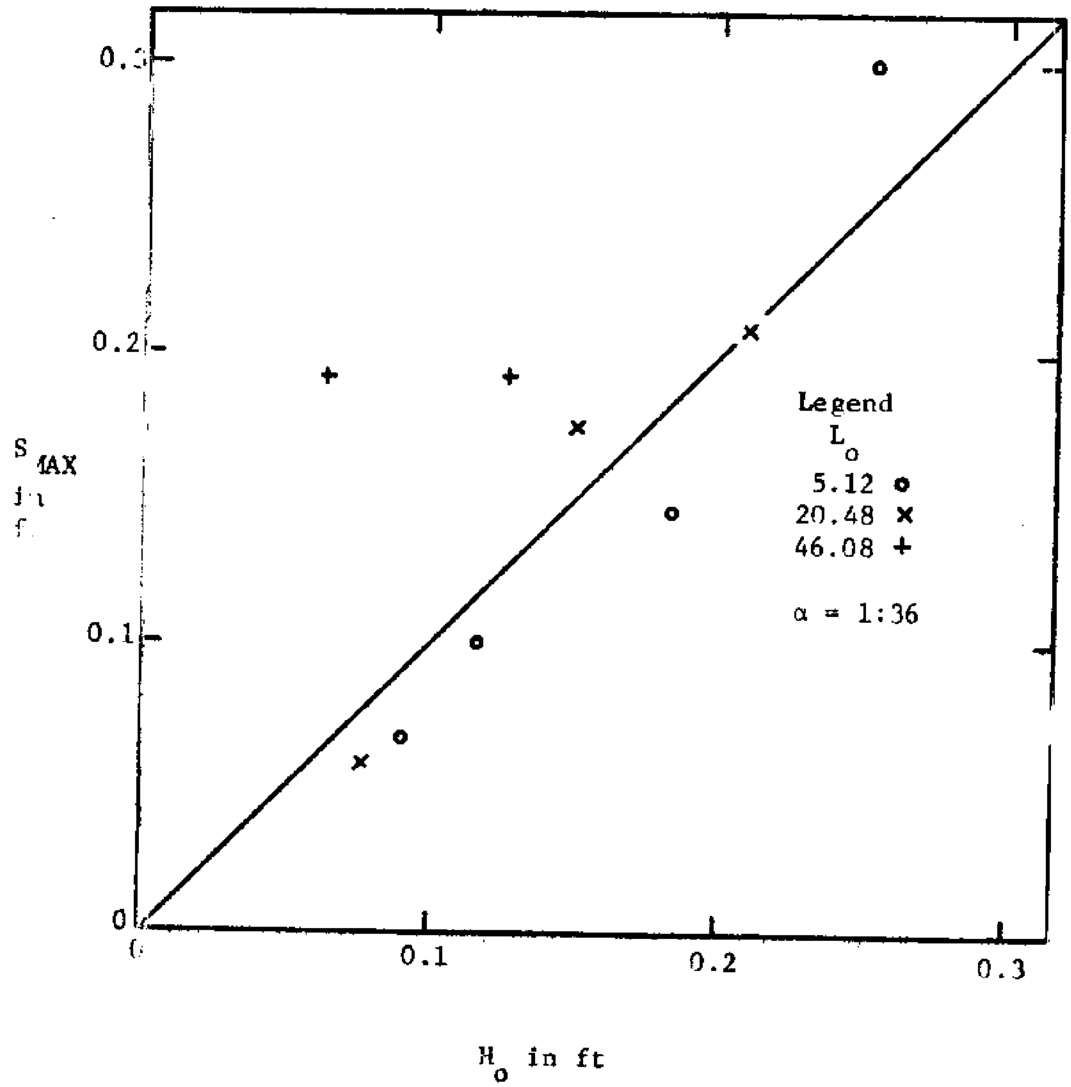


Fig. 11 - Maximum scour depth versus deep water wave height for varying wave length

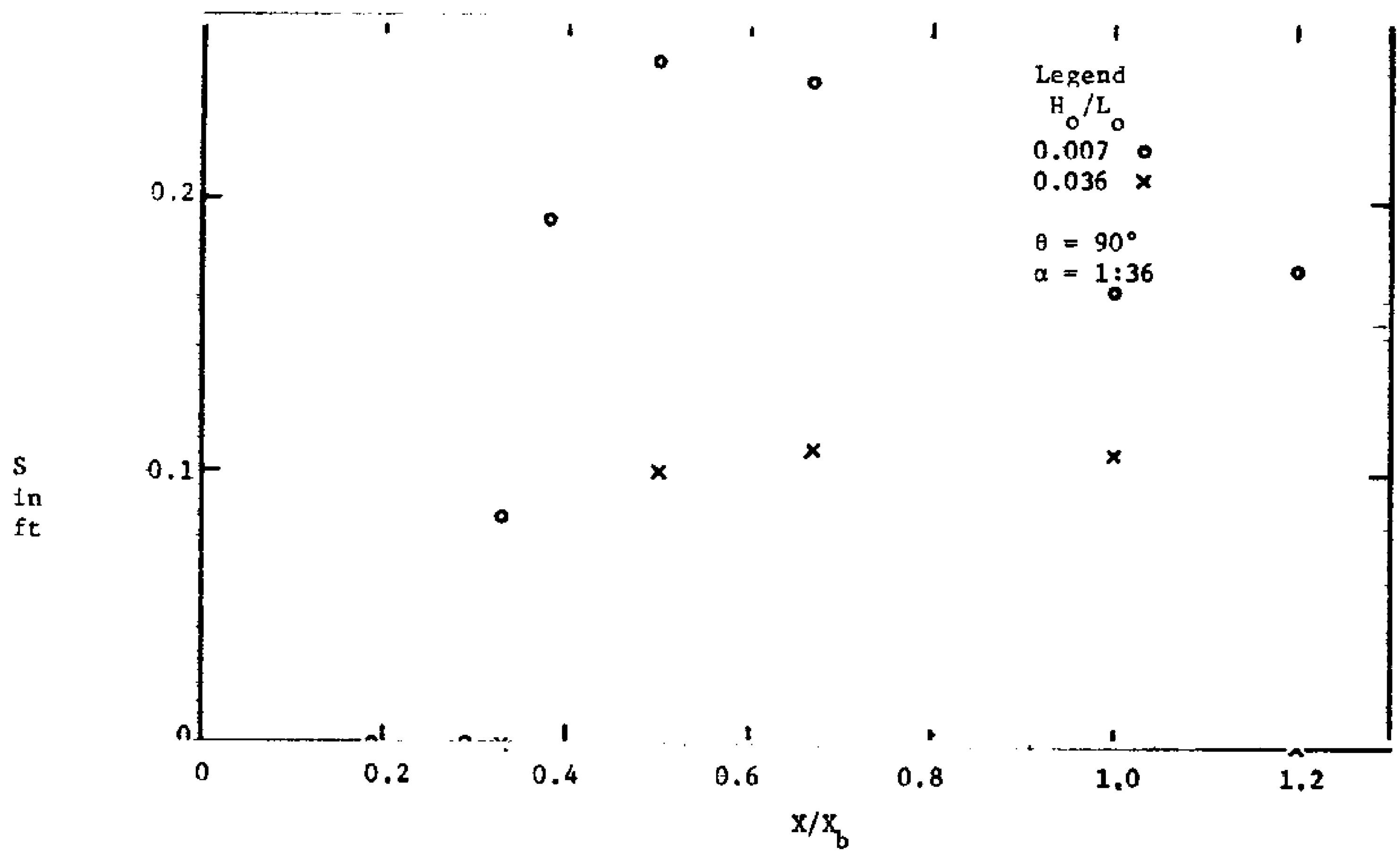


Fig. 12 - Scour depth at the sea wall versus relative distance of the wall from the shoreline for 90° wall and varying wave steepness

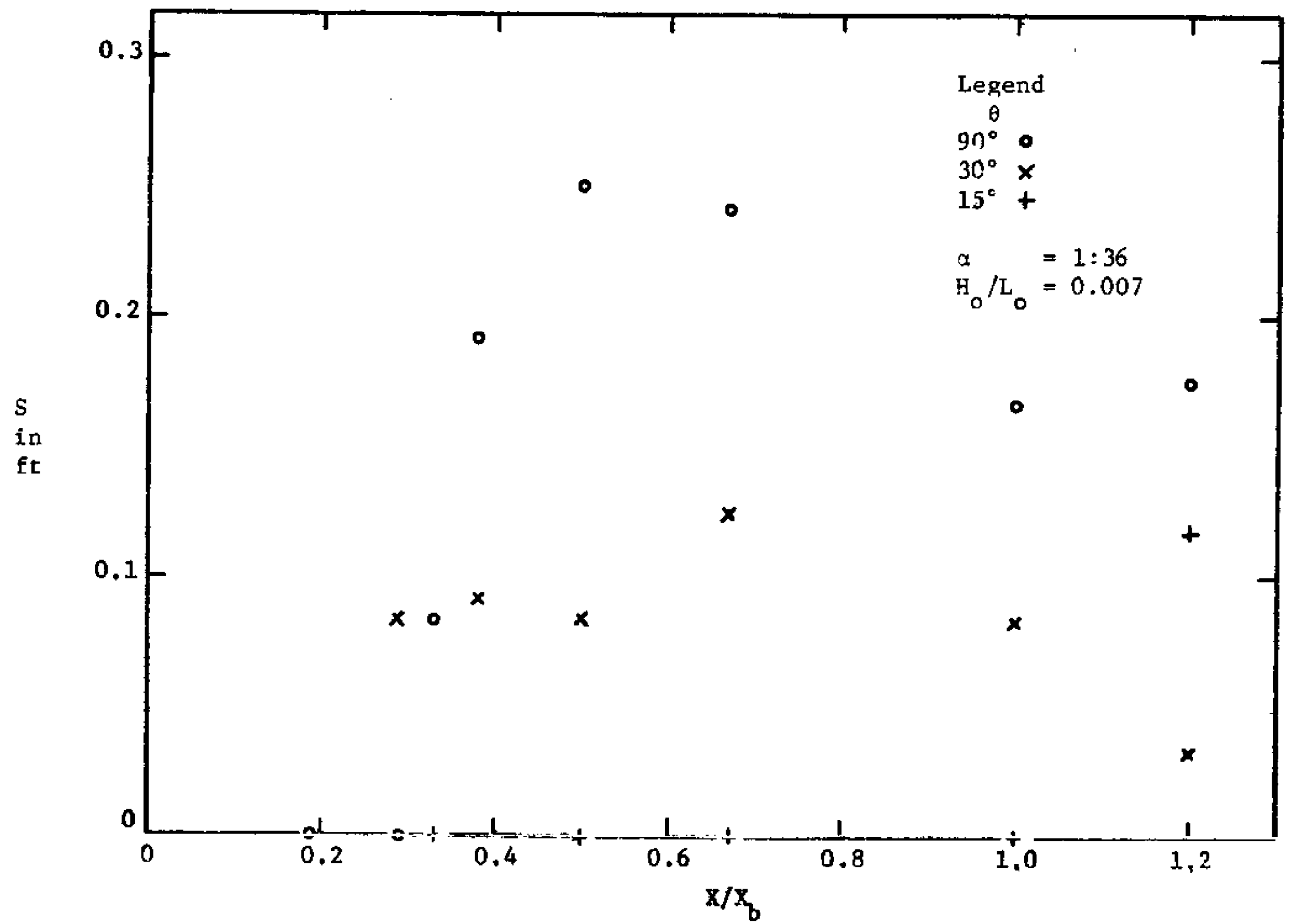


Fig. 13 - Scour depth at the sea wall versus relative distance of the wall from the shoreline for constant wave steepness and varying sea wall angle

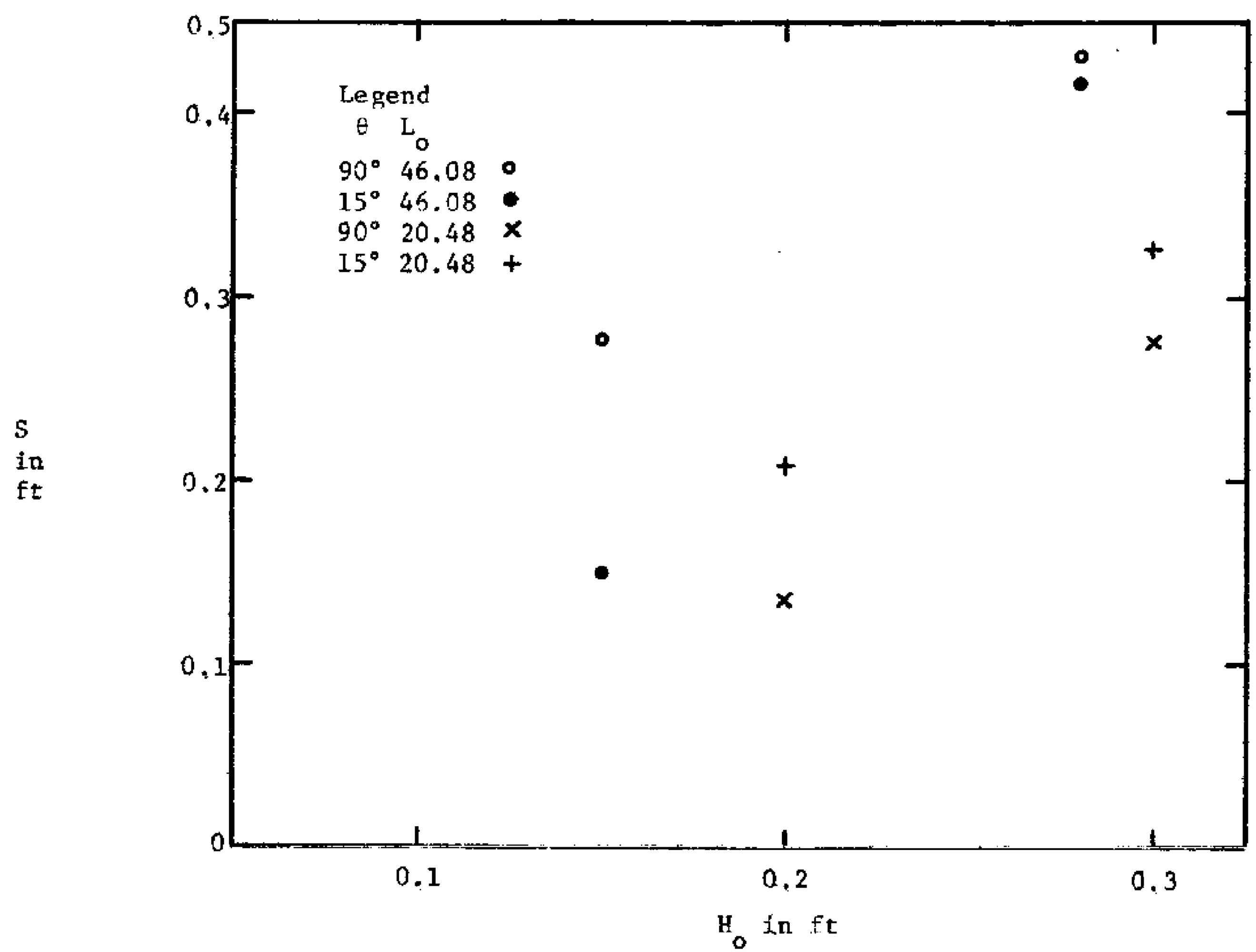


Fig. 14 - Scour depth at the sea wall versus deep water wave height for varying wave length and sea wall angle

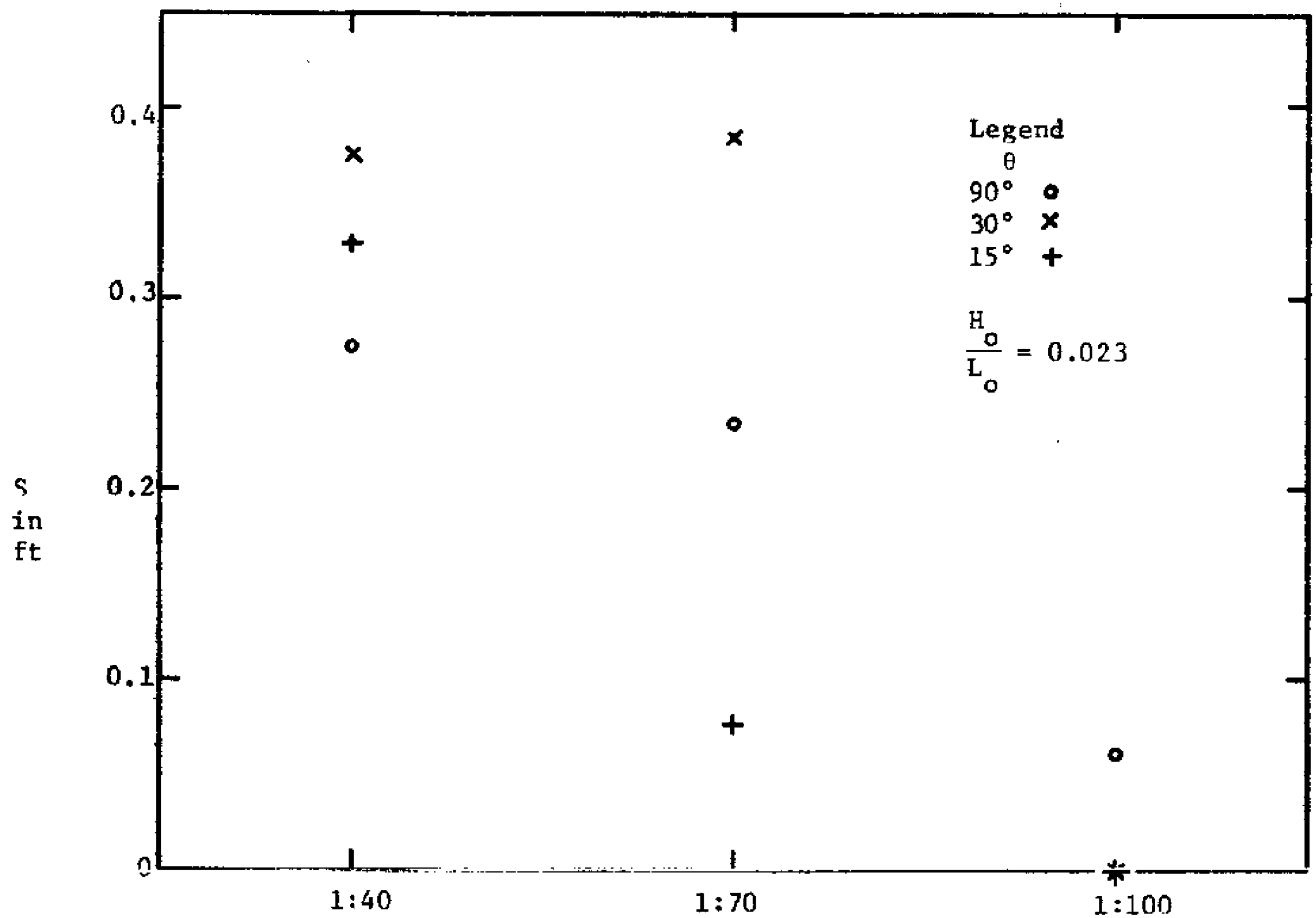


Fig. 15 - Scour depth at the sea wall versus beach slope for constant wave steepness and varying sea wall angle