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An Examination of Shoaling Wave Parameters

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AN EXAMINATION OF SHOALING WAVE PARAMETERS

A Thesis

Presented to

The Faculty of the School of Marine Science
The College of William and Mary in Virginia

In Partial Fulfillment

Of the Requirements for the Degree of
Master of Arts

by

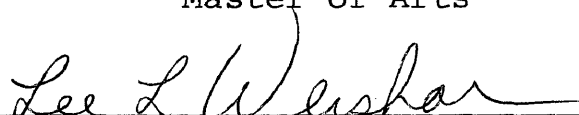
Lee L. Weishar

//

1976

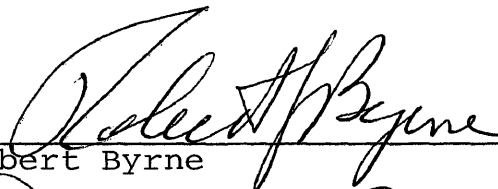
APPROVAL SHEET

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the requirements for the degree of
Master of Arts

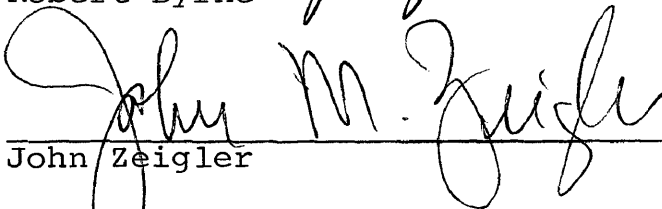


Author

Approved



Robert Byrne



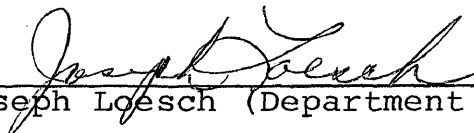
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ABSTRACT

The purpose of this investigation is to determine the applicability of five empirically determined equations treating the behavior of waves in the shoaling breaker zone. They are (1.) the relative wave height H_b/d_b , (2.) the proportion of the wave height above mean water level, (3.) the classification of the plunging wave, (4.) the prediction of the breaking wave height, and (5.) the prediction of the distance traveled by a plunging wave while it is breaking.

These laboratory developed formulas are compared with data that was obtained photogrametrically in the field at Virginia Beach Virginia. The formulas are compared showing their predictive capabilities as is the amount of variation that occurs in the results between the laboratory and the field data.

With the exception of the prediction of the breaking wave height, the empirically derived formulas fail to account for the non-uniform field condition. Variations in the field results occur when one or more of the parameters are oversimplified to a point where it no longer represents the phenomenon that it is meant to describe. This investigation yields the following conclusions:

- 1.) The breaking wave ratio, H_b/d_b , fails to accurately predict the relative breaking height. Although the mean of this ratio was 0.79, for the field data, the standard deviation was extremely large, 0.36. The best ratio tested was η/d_b which has a mean of 0.67 and standard deviation of 0.27.
- 2.) There is remarkable agreement between the laboratory and field data when predicting the breaking wave height by $H_b = g^{1/5} (H_o^2 T)^{2/5}$ as formulated for tank conditions by Komar.
- 3.) The onshore and offshore breaking wave classification failed to provide conclusive results when applied to the field data.
- 4.) The laboratory empirical formula did not predict the plunge distance whereas the empirical relationship $P = 5.6 H_b$ had a very high correlation when applied to the field data.

- 5.) The plunging at the break point was found to travel with 80% of its wave height above M.W.L., while the spilling waves did not demonstrate any conclusive grouping.

AN EXAMINATION OF SHOALING WAVE PARAMETERS

CHAPTER I
INTRODUCTION

Water gravity waves are generated by the wind blowing across the water's surface. The wave will begin as a ripple, then grow into a wave whose speed is dependent on the wave period. As the wave travels shoreward through progressively shallower water, it transforms from a deep to a shallow water wave. During the transformation, several distinct changes occur in the wave characteristics. The wave height increases and the wave length decreases. When the wave reaches a critical point, called the break point, it breaks. After which it becomes a translational wave and runs up the beach.

The purpose of this thesis is to determine the reliability of existing predictive equations in five specific areas of the natural shoaling wave zone.

These five areas are:

- 1.) The relative wave height breaking criteria (H_b/d_b), originally based on solitary wave theory.
- 2.) The examination of the proportion of the total wave height which is above mean water level (M.W.L.).
- 3.) The prediction of the wave height at the break point.
- 4.) The classification of breaking wave types by a quantifying parameter.
- 5.) The prediction of the distance traveled by a plunging wave during breaking.

The existing predictive equations were developed by other investigators from wave tank studies. For the most part, these relationships have not been compared with field observations. The goal of this work is to make this important comparison.

CHAPTER II

METHODS

The data utilized in this report was obtained by Robert J. Byrne (V.I.M.S.) at Virginia Beach, Virginia, during August and September of 1968. The observations were made immediately adjacent to and on the north side of the 15th street fishing pier (Figure 1). The intent of the field measurement program was to trace the behavior of waves from the time just prior to breaking through the run-up process. To achieve this, the wave phenomenon were filmed as they passed through a rectangular, vertical plane, grid which was placed perpendicular to the beach, extending from the top of the foreshore to about 100 ft offshore. A series of steel support pipes were jettied into the sand bottom at approximately 10 ft intervals. The rectangular grid sections were then hung plumb from these support pipes. The basic grid element was 2 ft by 2 ft (Figure 2) with each cell painted in contrasting color every foot. As the wave profile passed the grid, the elevation of the free surface could then be recorded from the series of points formed by the intersection of the wave profile and the grid. At the conclusion of each test run the sand bottom profile was obtained by levelling along the grid. The combined information permitted graphic construction of the beach profile, the position of still water

level and if desired, the instantaneous profile of the wave.

The recording camera, a 16 millimeter motor-driven Bolex with variable focus lens, was mounted on the pier with the camera view axis perpendicular to the grid. Film advance rate was 12.53 frames/sec (one frame each 0.0798 sec).

Basic film data reduction was achieved using a Lafayette Analyst Time Motion Projector with the image projected on a rear surface screen. Water surface elevation was estimated to 0.1 ft on the grid.

Three runs, obtained over a one hour period on 26 September 1968, constitute the data base for this report. Each run contains approximately 40 well defined breaking waves yielding a total sample of 120 waves. On the date of these measurements a relatively well defined swell was incident, with the wave fronts almost parallel with the shoreline. There was very little local wind wave activity.

The data obtained are listed in appendix B. The variables measured directly from the film are the following (see Figure 3)

B_x = The horizontal distance between the bore collapse point and the intersection of mean water level (M.W.L.) and the beach; (ft).

C_b = Celerity of the wave crest just prior to the break point (ft/sec) measured by counting the film frames required for the wave to travel 6 ft prior to breaking.

F = The horizontal distance between the position of the preceding trough X_t and the break point X_b ; (ft).

G = The vertical distance between the position of the water surface at X_t and M.W.L.; (ft).

H_b = The total wave height after breaking. The vertical distance between the wave crest and preceding trough at breaking; (ft).

η = The vertical distance between the wave crest and M.W.L.; (ft).

Figure 1. Location of the field site at Virginia Beach and the surrounding area.

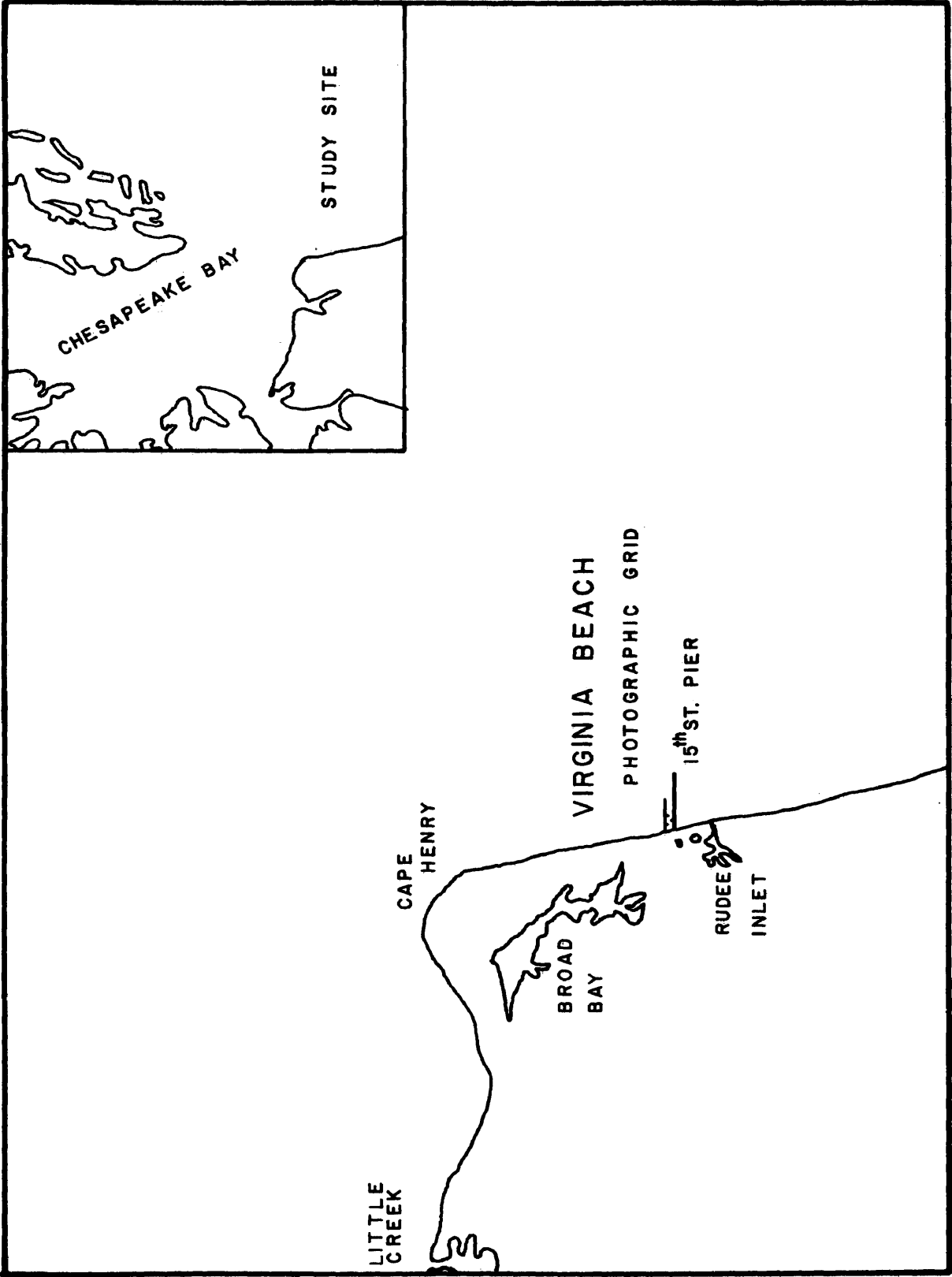


Figure 2. Schematic representation of the photographic grid that was used for data reduction.

PHOTOGRAPHIC

GRID

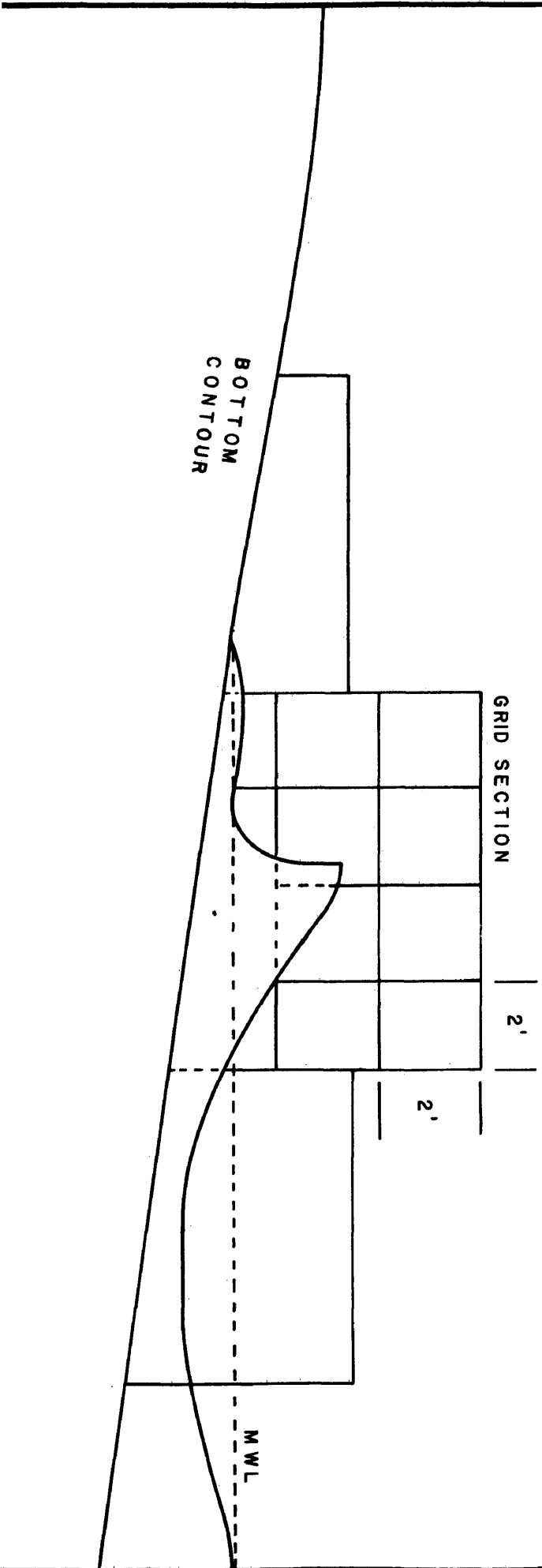
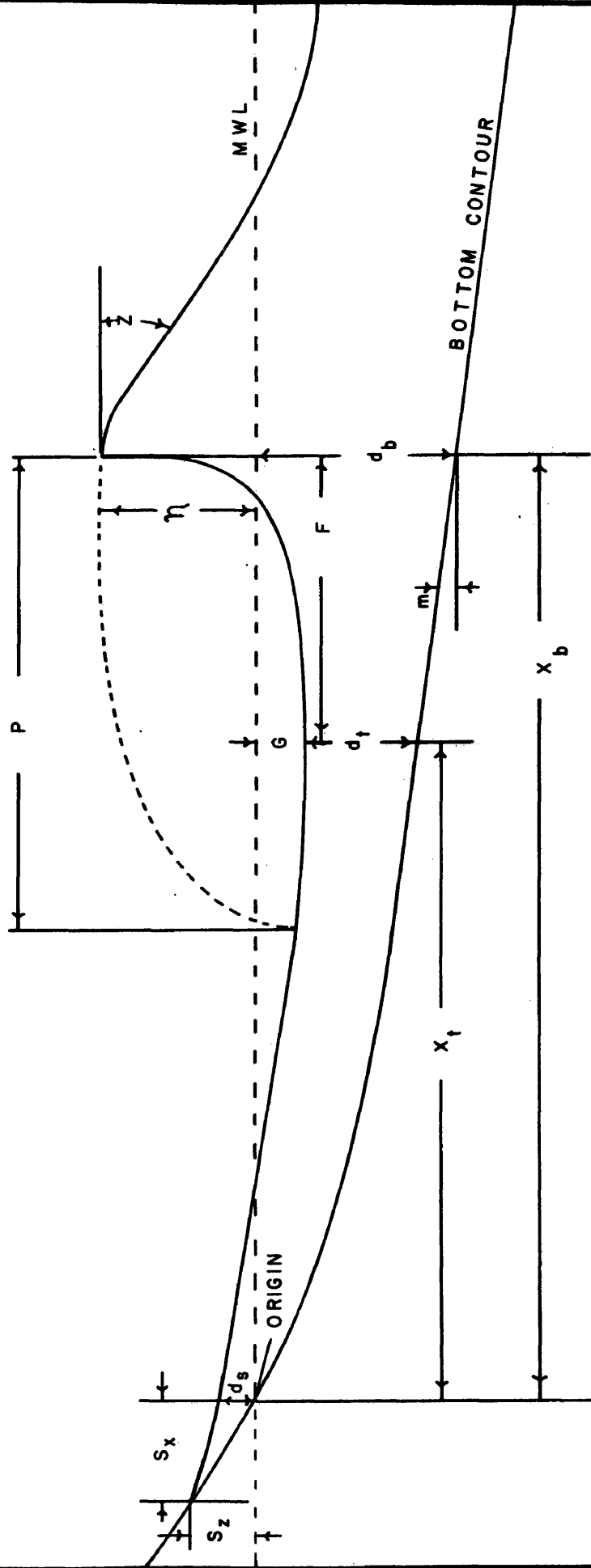


Figure 3. Parameters obtained during the data reduction process of the Virginia Beach field data are shown graphically (see text).

OBSERVED
WAVE PARAMETERS



- P = Horizontal distance traveled by the plunging wave during breaking; (ft).
- R_x = The horizontal run up distance relative to the origin placed at the intersection of M.W.L. and shore; (ft).
- R_z = The vertical run up distance relative to the intersection of M.W.L. and the shoreline; (ft).
- S^1 = Rate of swash advance (ft/sec) measured by counting frames required for advance.
- S_x = Horizontal distance to swash edge from the intersection of M.W.L. and shoreline when the following wave is at its break point; (ft).
- S_z = Vertical distance to swash edge from the intersection of M.W.L. and shoreline when the following wave is at its break point; (ft).
- T_b = The elapsed time between two consecutive breakers reaching their break points; (sec).
- T_{64} = The period of the wave at the 64 ft mark, measured from the origin which was the intersection of M.W.L. and the beach; (sec).
- X_b = The distance from the wave crest at its break point to the intersection of M.W.L. and the shore line; (ft).
- X_t = The distance from the preceding trough to the intersection of M.W.L. and the shore line when the wave crest is at its break point; (ft).
- Z = The tangent of the (breaker backface) angle formed between the wave crest and the position of the water surface on the backface, measured six horizontal feet from the crest.
- d_b = The depth from M.W.L. to the sand bottom beneath the wave at its break point; (ft).
- d_s = The vertical height of the water's surface relative to the intersection of M.W.L. and the shore line when the wave is at the break point; (ft).
- d_t = The depth beneath the preceding trough when the wave is at the break point; (ft).
- m = The beach slope beneath the break point of the wave.
- t_c = The time elapsed from the bore formation until the bore encounters the shore line; (sec).

t_{cs} = Swash time $t_c - t_s$ (sec).

t_s = The time measured when the swash reaches its furthest horizontal extent on the beach; (sec).

CHAPTER III

THE RELATIVE DEPTH BREAKING CRITERIA

The most widely used criterion to calculate the depth of breaking in the shoaling zone is the relative height ratio H_b/d_b . The origin of the relative wave height criterion for breaking waves is found in solitary wave theory. Russell (1844) first observed the solitary wave but it was McCowan (1891) who advanced this theory. Munk (1949) completed the first exhaustive study of applications of solitary wave theory to wave behavior in the shoaling zone.

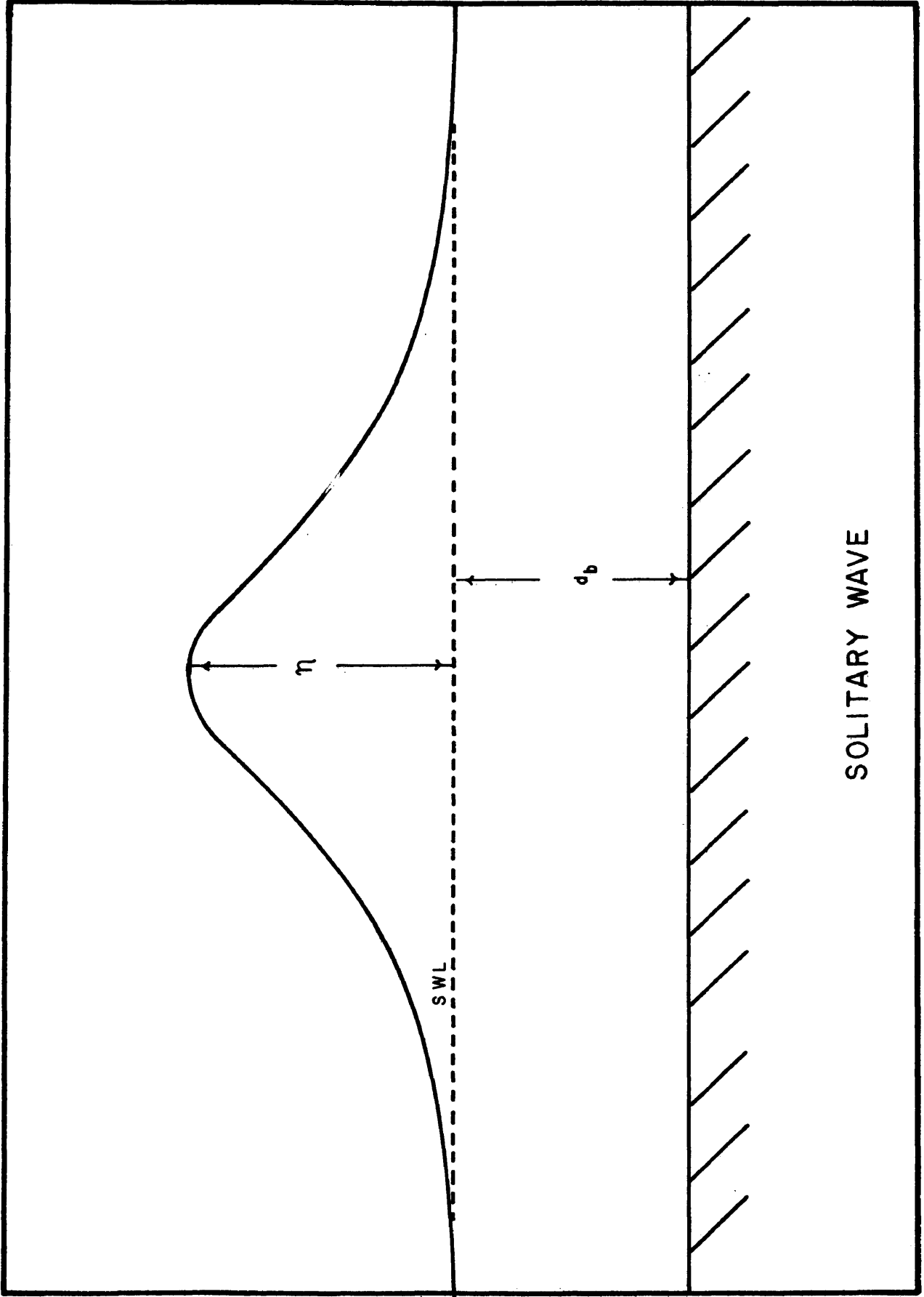
The basic conceptual elements of the theoretical solitary wave are:

- 1) The wave crest must be totally above still water level.
- 2) The wave must be a single discrete disturbance, therefore the wave length is infinite.
- 3) The wave must be symmetrical and must not change shape.
- 4) There must be no boundry in the direction of propogation.

Figure 4 depicts some of the characteristics of the solitary wave.

McCowan (1891) was forced to approximate the conditions for the solitary wave because a breaking wave violates the boundary conditions of the theory. He assumed that the wave would break when the horizontal particle velocity beneath the crest equaled the wave celerity. He simplified the equation in terms of surface elevation above still water

Figure 4. The theoretical Solitary wave profile and its relation to still water level.



SOLITARY WAVE

level by calculating the following parameters for the breaking wave.

$$Md_b \approx 1.10 \quad (1)$$

and

$$M \approx 0.90 \quad (2)$$

where M is a constant of integration in terms of η , the wave height, obtained from solitary wave theory, and d , the water depth below still water level. Equations 1 and 2 were combined to obtain the first approximation of the relative wave height at breaking:

$$\eta / d_b = 0.81 \quad (3)$$

McCowan observed that just prior to breaking, η was $0.75 d_b$. Later refinements of his solution resulted in the now classical value of 0.78 for the relative wave breaking criteria.

The solitary wave (Figure 4) is a single disturbance totally above the still water level and with an infinite wave length; however, the natural wave (Figure 5) occurs as part of a wave train and has a definite trough that is below the still water level. This discrepancy between the solitary and the natural wave poses a very real problem to the investigator who tries to apply solitary wave theory to surf conditions. What does he choose for values of wave height and water depth? Solitary wave theory uses the wave height, η , which is the wave height above the still water level and d_b , the water depth below the still water level. The natural wave has a height that is both above and below the mean water

level. Wave height may be interpreted as the height above mean water level, η , or the total wave height, H_b (Figure 4). A similar predicament arises when choosing a value for the water depth. Does the investigator choose the depth below mean water level, d_b or the depth below the trough preceding the wave, d_t ? It often is difficult to reproduce or compare data sets because many investigators fail to state which values they have used. Various wave height and depth values used by several authors whose methods were used in this investigation are presented in Table 1.

The critical η/d_b value for a solitary breaking wave, as noted above is 0.78. Galvin (1972) in an extensive review of the literature compiled a chronological table of the critical values for the breaking solitary wave (Table 2). The average of these tabulated values is 0.82.

Analysis of the photographic data of the waves at Virginia Beach yields a mean η/d_b of 0.69 with a standard deviation of 0.31 (Figure 6). This ratio was reexamined after separating the waves by breaker type, plunging and spilling, to determine whether or not the means of the segregated data were significantly different. While the mean and standard deviation of the plunging waves fell at 0.73 and 0.27, the respective values for the spilling waves were 0.67 and 0.32 (Figure 7). The difference between the means of the segregated data compared by the Student "t" test was nonsignificant ($P > 0.10$, i.e., the probability (P) of a deviation \geq that observed was due to chance was $> 10\%$).

Figure 5. The wave shape observed in the field and various descriptive parameters for wave height and depth.

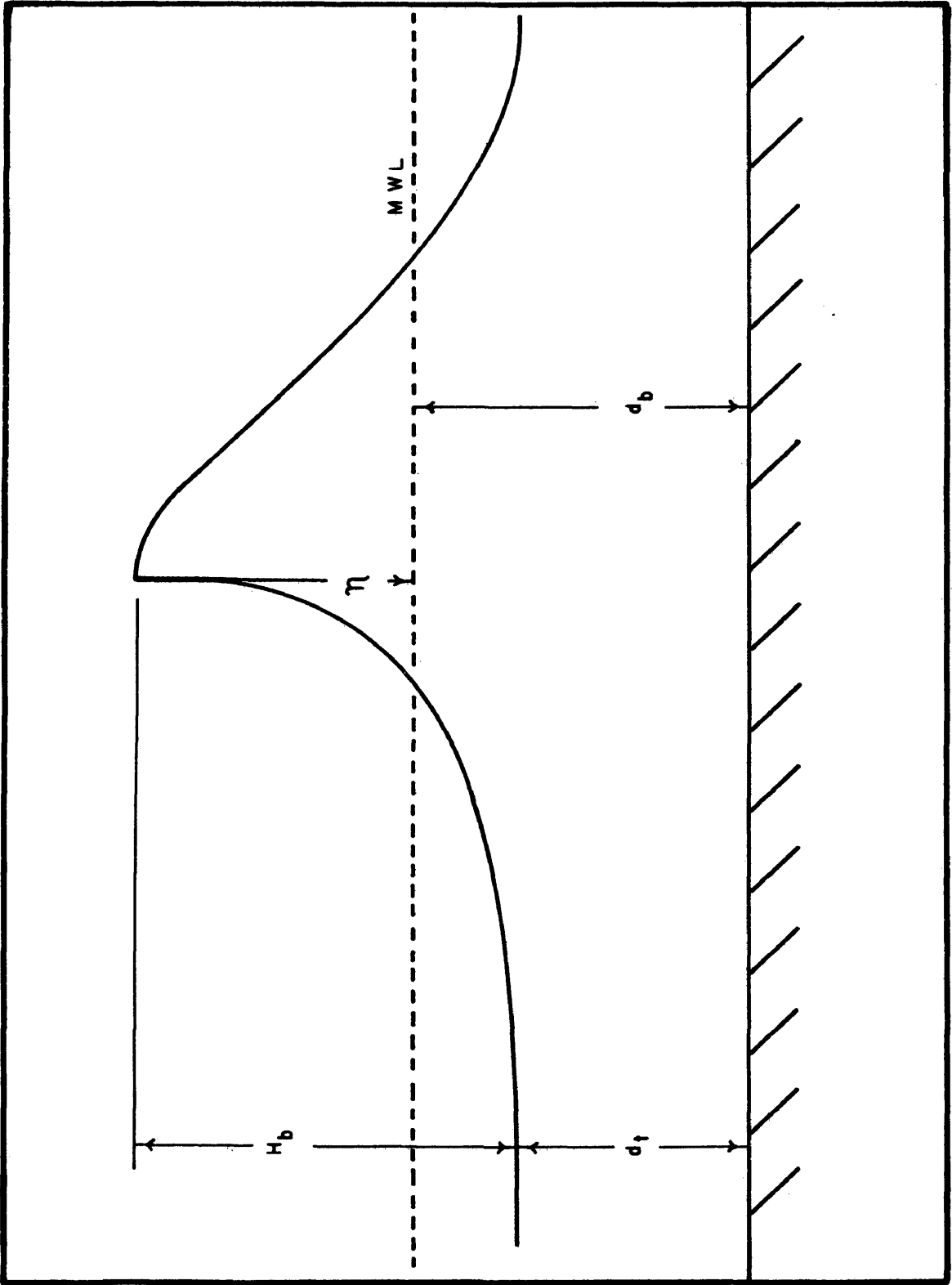


Table 1

Wave Height and Water Depth Parameters Used by Various Authors

| AUTHOR | H_b | η | d_b | d_t |
|--------------------------------|-----------|--------|-------|-------|
| McCowan (1891) | | X | X | |
| Munk (1949) | | X | X | |
| Iverson (1951) | X | | X | |
| Ippen (1955) | | X | X | |
| Weiggle & Beebe (1956) | undefined | | | |
| Hunt (1959) | X | | X | |
| Reid & Bretschneider (1960) | X | X | X | |
| Galvin (1968) | X | | X | |
| Weggel (1972) | undefined | | | |
| Komar & Gaughan (1972) | X | | X | |

Table 2*

| Summary of the Years of Publication and the Ratio a/d_s | | |
|---|------|------------|
| Investigator | Year | a/d_s ** |
| Boussinesq | 1871 | 0.73*** |
| McCowan | 1891 | 0.78 |
| McCowan | 1894 | 0.78 |
| Gwyther | 1900 | 0.83 |
| Packham | 1952 | 1.03 |
| Chappelear | 1959 | 0.87 |
| Laitone | 1966 | 0.73 |
| Lenau | 1966 | 0.83 |

* From Galvin (1972)

** Quoted by Ippen and Kulin (1955)

*** Note that for solitary wave the amplitude (a) equals the wave height (H_s) and (η) because the solitary wave travels^b completely above still water level.

Given the wide standard deviation of η/d_b the ratio of the total wave height, H_b , to the water depth below mean water level, d_b , was evaluated. Figure 8 is a histogram of critical H_b/d_b ratios. The mean value for the combined sample was 0.79. Although there is a small cluster about the mean, the standard deviation was large, 0.36. To determine if the scatter could be reduced, the ratio was reexamined after the sample was segregated by breaker type. The mean and standard deviation of the spilling waves was 0.68 and 0.37, while the mean and standard deviation for the plunging waves was 0.87 and 0.33 (Figure 9). A Student "t" test indicated a significant difference in the means ($P < 0.01$).

As solitary wave theory requires that the entire disturbance of the surface be above still water level, the ratio H/d was recalculated using the water depth beneath the preceding trough, d_t (Figure 5). The results of this calculation are shown in Figure 10. The mean and standard deviation was 0.89 and 0.46. This ratio was reexamined in conjunction with the data segregated by breaker type. The mean and standard deviation of the spilling waves was 0.69 and 0.31, while the mean and standard deviation for the plunging waves was 1.03 and 0.50 (Figure 11). A Student "t" test indicated a significant difference in the means ($P < 0.01$).

Many investigators noticed the poor agreement of the theoretical and observed values of the wave height to water depth ratio for breaking waves. They attributed this poor agreement to the lack of inclusion of terms that, either

directly or indirectly accounted for either the beach slope or bottom friction. Galvin (1969) while studying various ratios for predicting the relative breaking depth including the ratio η/d_b , found after examining his own laboratory data and that of Iverson (1952) that the ratio H_b/d_b was the best criterion. He derived an empirical formula for H_b/d_b which includes the beach slope.

$$H_b/d_b = 1.0/\beta \quad (4)$$

$$\text{where } \beta = 0.92 \text{ for } m \geq 0.07 \text{ and} \\ \beta = 1.40 - 6.85m \text{ for } m \leq 0.07$$

where m is the tangent of the beach slope. When equation 4 was applied to wave tank studies very reliable results were obtained. The tangent of the beach slope accounts indirectly for the friction factor and directly in the beach slope. Figure 12 is a scatter plot of observed H_b/d_b values and the theoretical H_b/d_b values, calculated by Galvin's method (equation 4). Appendix A is a tabulation of the calculations.

Weggel (1972) also developed an empirical breaker criteria (equation 5)

$$H_b/d_b = 0.724 + 5.6m \quad 95 \ 0$$

where m is the tangent of the beach slope

Figure 13 is a scatter plot of the observed H_b/d_b ratios and the values calculated from this equation. As is demonstrated by the great scatter of the data points, this equation did not prove to be accurate when applied to the field data. Appendix A contains the calculations.

Camfield and Street (1969) developed another equation for breaking waves using a power series of the beach slope:

$$H_b/d_b = 0.75 + 25m - 112m^2 + 3870m^3 \quad (6)$$

Figure 6. Histogram depicting frequency of occurrence of g/d_b
for breaking wave data from Virginia Beach.

Sample size for combined sample was equal to 120 waves.

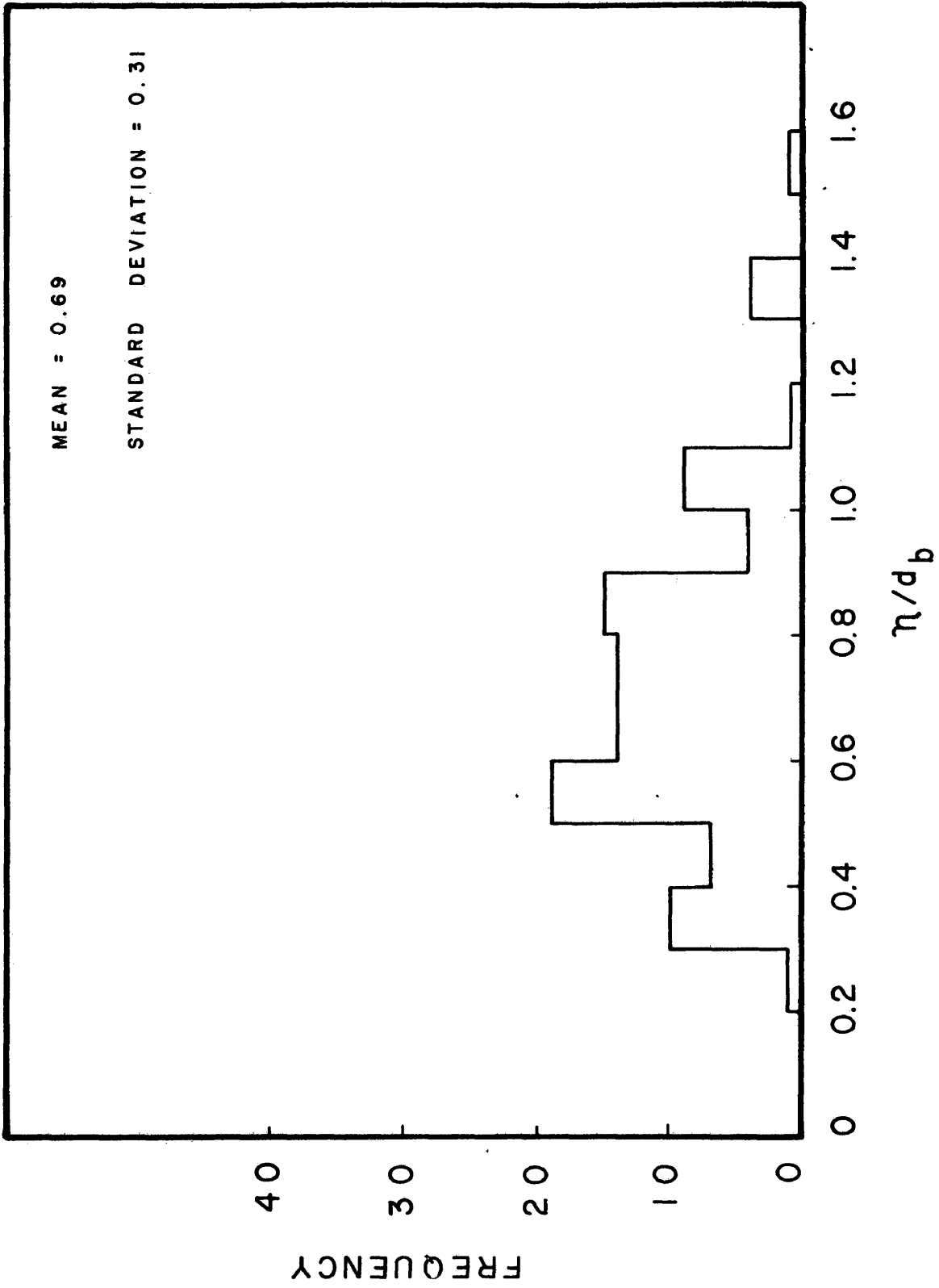


Figure 7. η/d_b vs ξ of breaking waves segregated by breaker type, plunging and spilling. There were 51 spilling waves and 69 plunging waves in the respective samples.

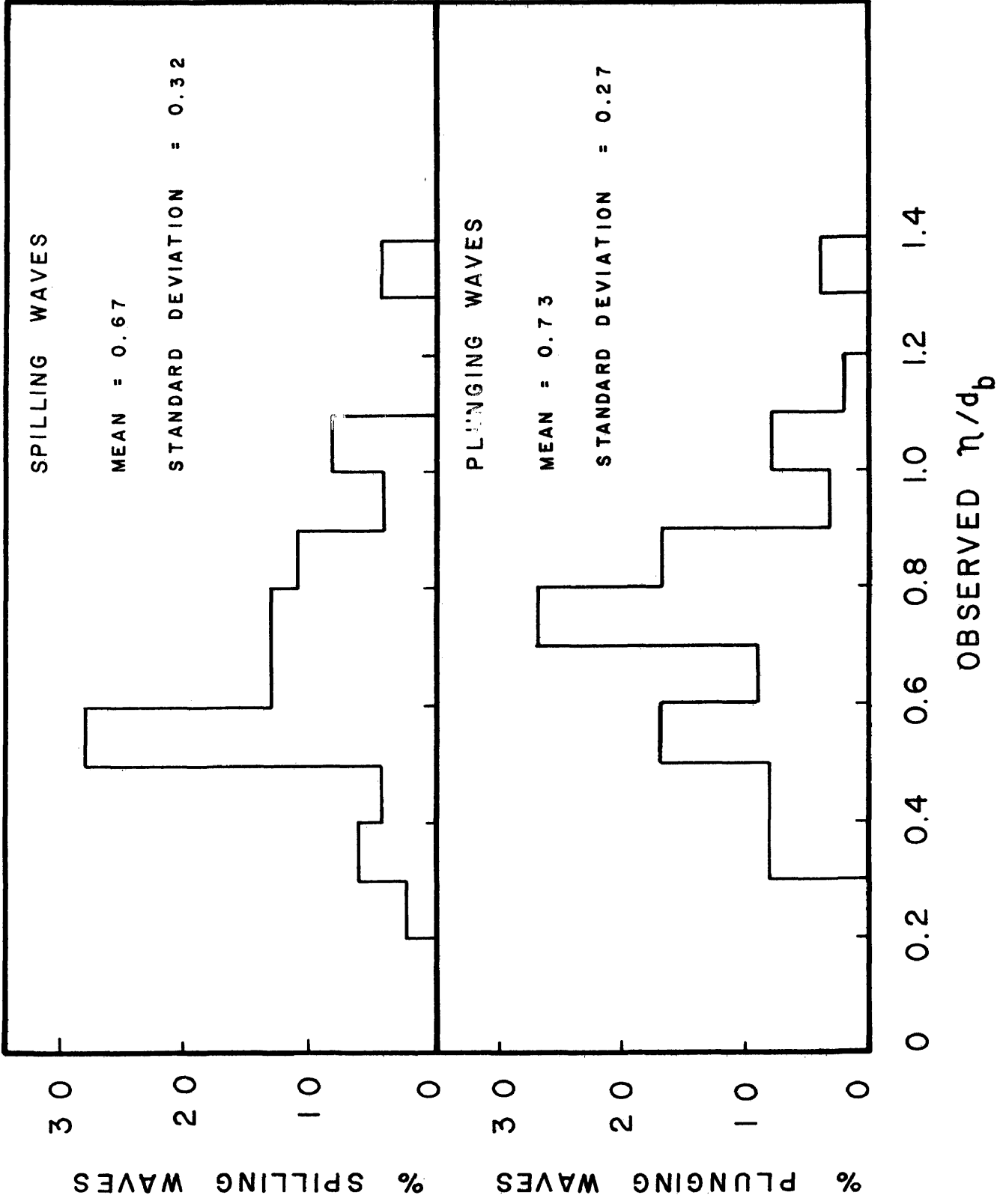


Figure 8. Histogram depicting frequency of occurrence of

H_b/d_b for breaking wave data from Virginia Beach.

The sample size for the combined sample was 120 waves.

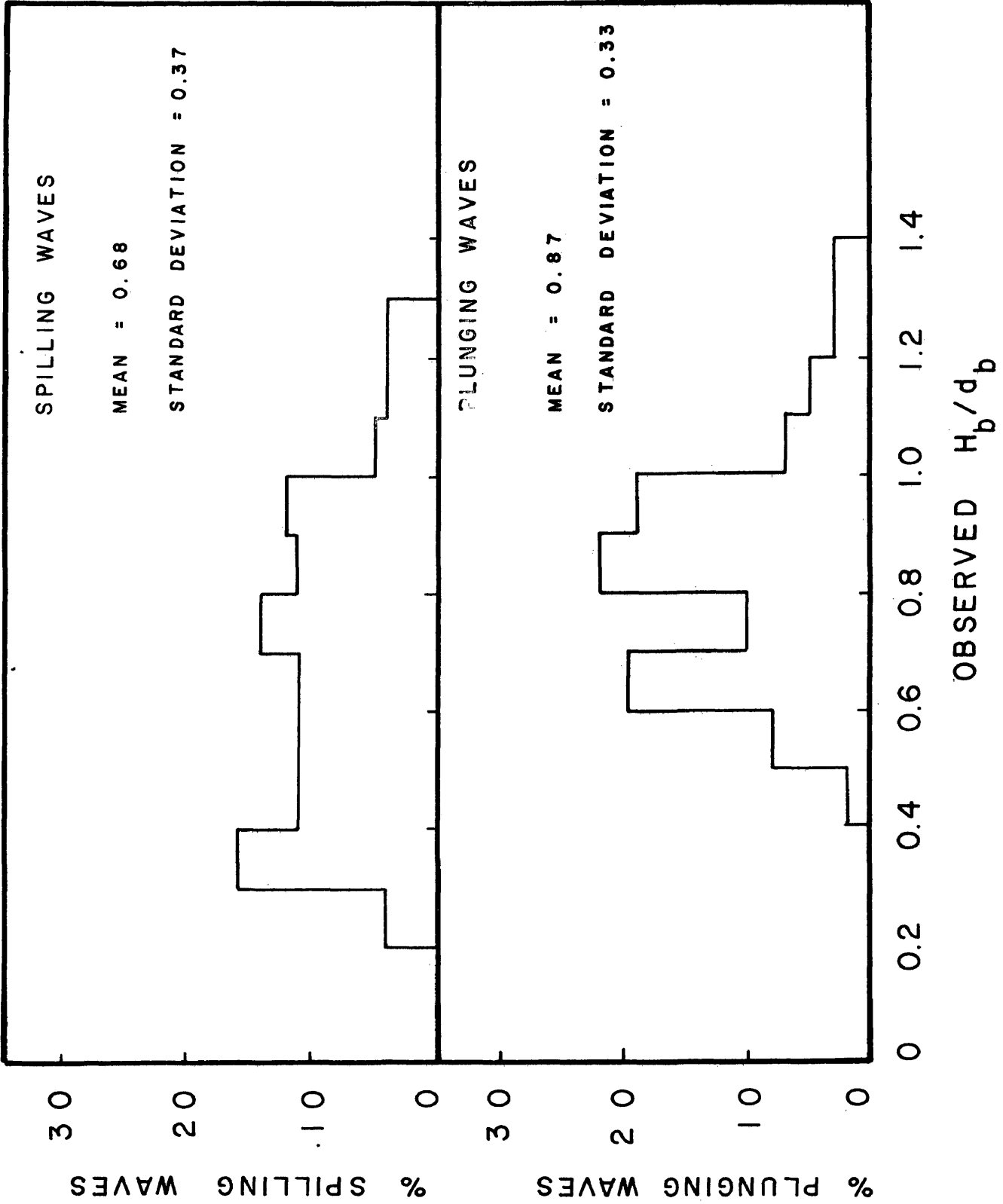


Figure 9. H_b/d_b vs % of breaking waves segregated by breaker type, plunging and spilling. There were 51 spilling waves and 69 plunging waves in the respective samples.

MEAN = 0.79

STANDARD DEVIATION = 0.36

FREQUENCY

40

30

20

10

0

0

0.2

0.4

0.6

0.8

1.0

1.2

1.4

1.6

H_b/d_b

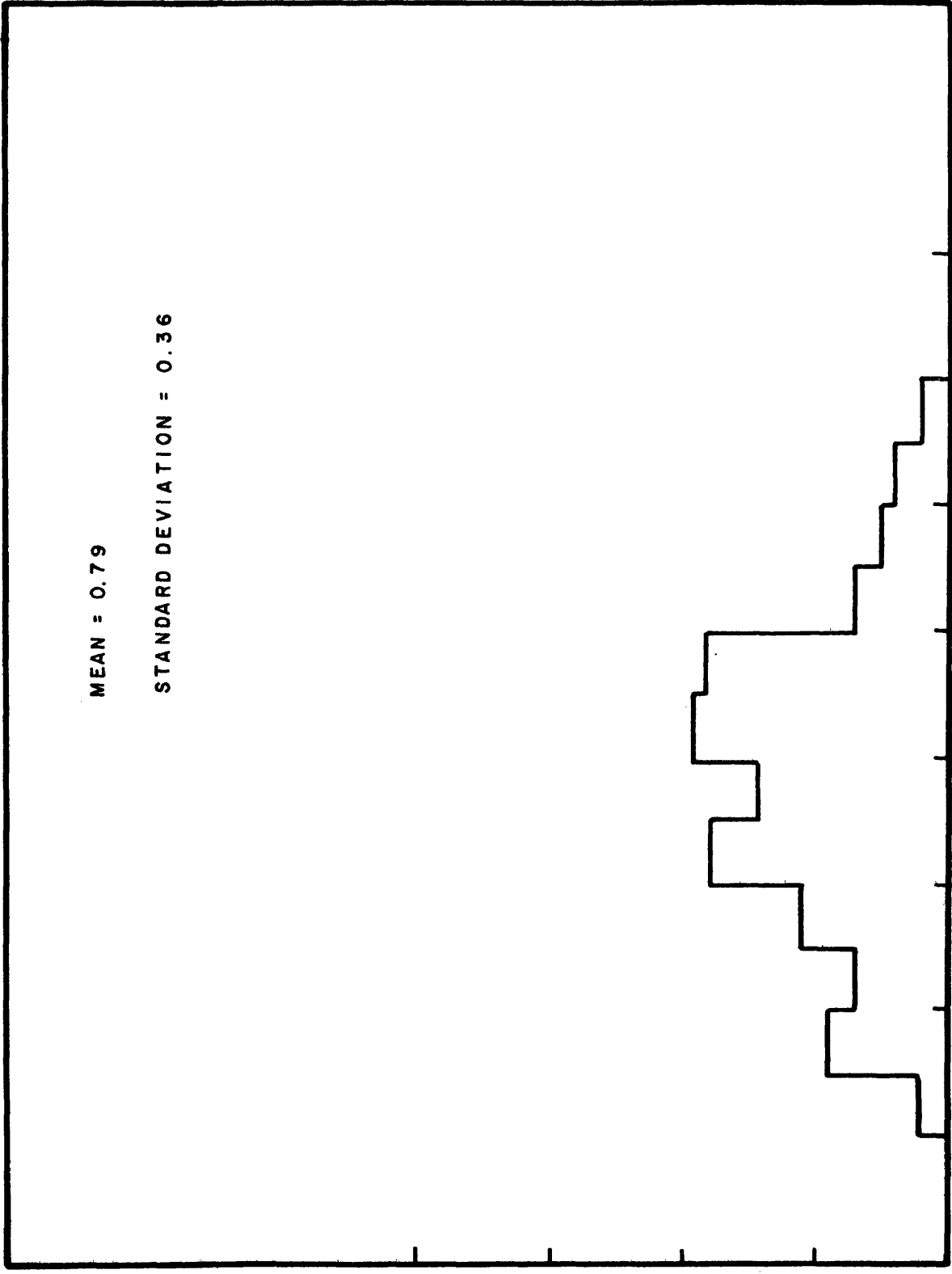


Figure 10. Histogram depicting frequency of occurrence of H_b/d_t for breaking wave data for Virginia Beach. The sample size for the combined sample was 120 waves.

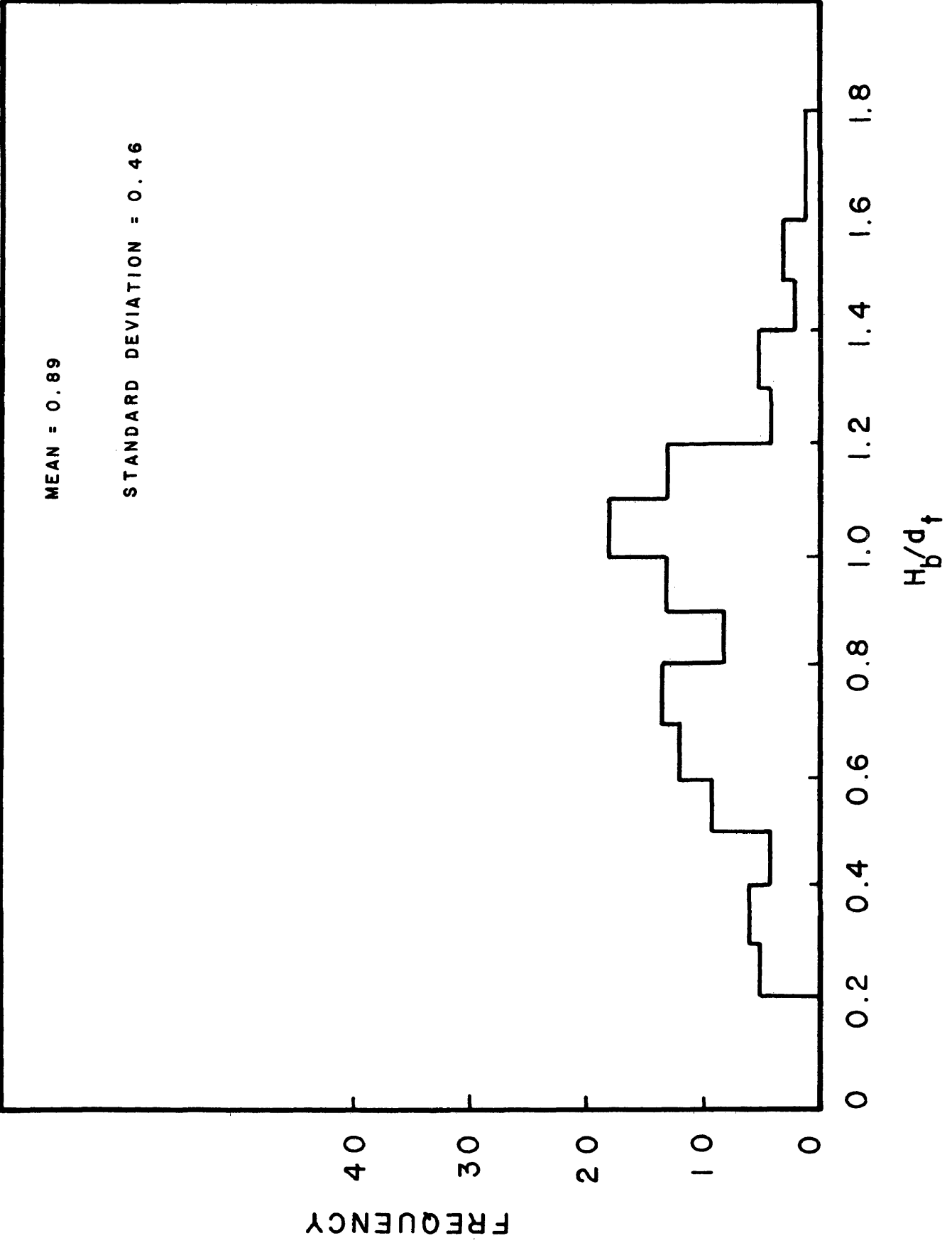
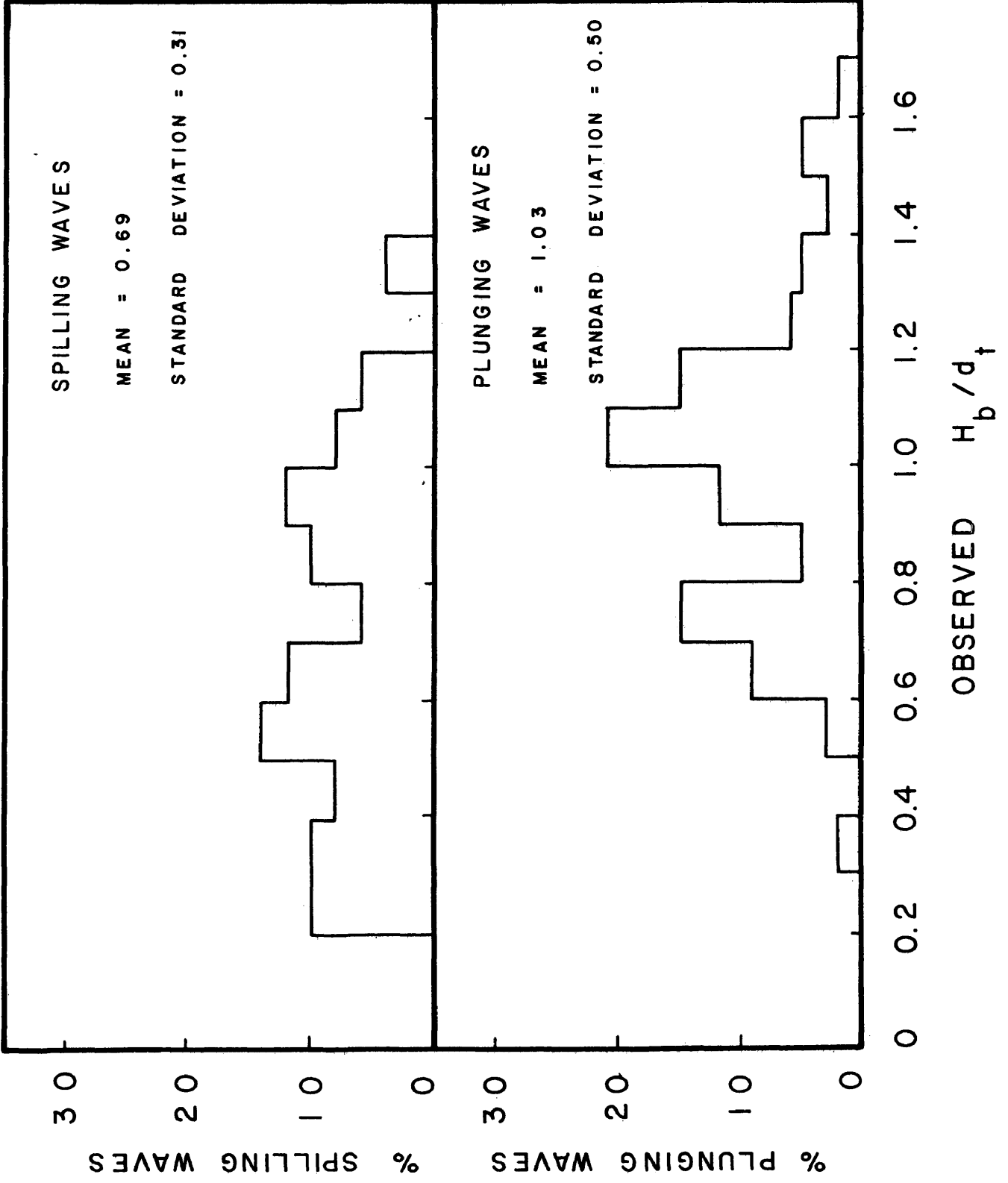


Figure 11. H_b/d_t vs % of breaking waves segregated by breaker type, plunging and spilling. There were 51 spilling waves and 69 plunging waves in the resective samples.



There is not a plot of values presented for this formula since the calculated, or predicted, values are an order of magnitude greater than the observed. The calculations, however, are included in Appendix A.

The results of these several comparisons of observed and predicted values for a breaking criteria indicate that approximations from Solitary wave theory does not provide an adequate basis for predicting the behavior of real breaking waves. The boundary conditions for Solitary wave theory do not allow the inclusion of beach slope, bottom friction, variations in wave geometry, or the interaction of the wave with backwash.

This investigation was conducted to determine the validity of several methods of predicting where a wave would break, a summary of these ratios are contained in Table 3. While the ratio η/d_b was the most accurate mathematically, this ratio is extremely difficult to obtain in the field unless a comprehensive study is conducted. On the other hand, the mean of the ratio H_b/d_b fell very close to the theoretically value, 0.78, as derived from solitary wave theory. The large standard deviation suggests that this ratio may not be used to calculate the exact point of breaking. Although this ratio does not define the exact point where breaking take place, it does define the breaking zone. This will be of interest to the coastal engineer when considering the placement of structures in the shoaling zone. It will enable the engineer to accurately predict the limits of the breaking zone, thus

Figure 12. The observed breaking parameter, H_b/d_b , compared to the calculated value, calculated by Galvin's (1968) formula:

$$H_b/d_b = 1/\beta$$

were = 0.92 for $m \geq 0.07$ &
= 1.40 - 6.85m for $m \leq 0.07$

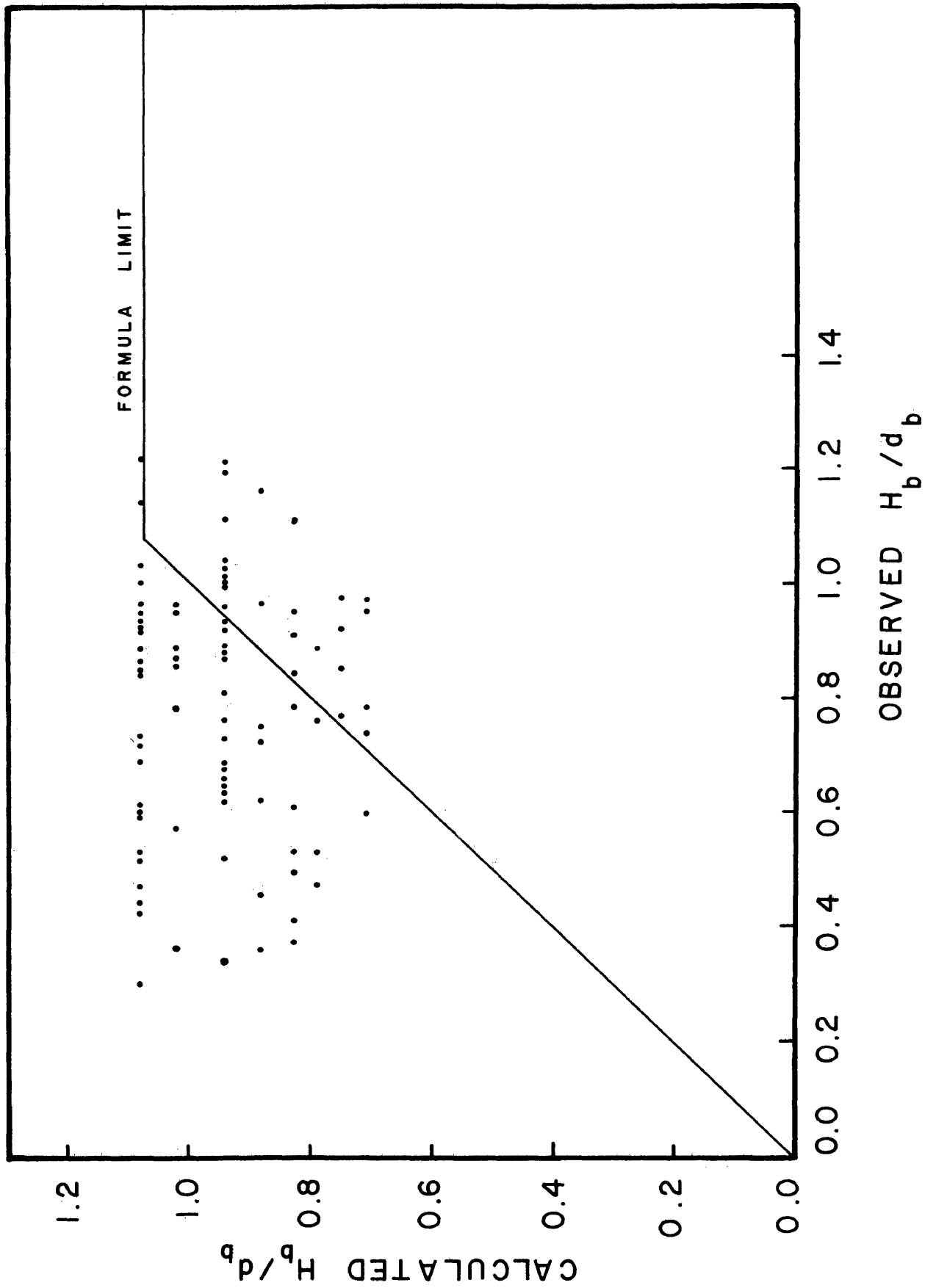
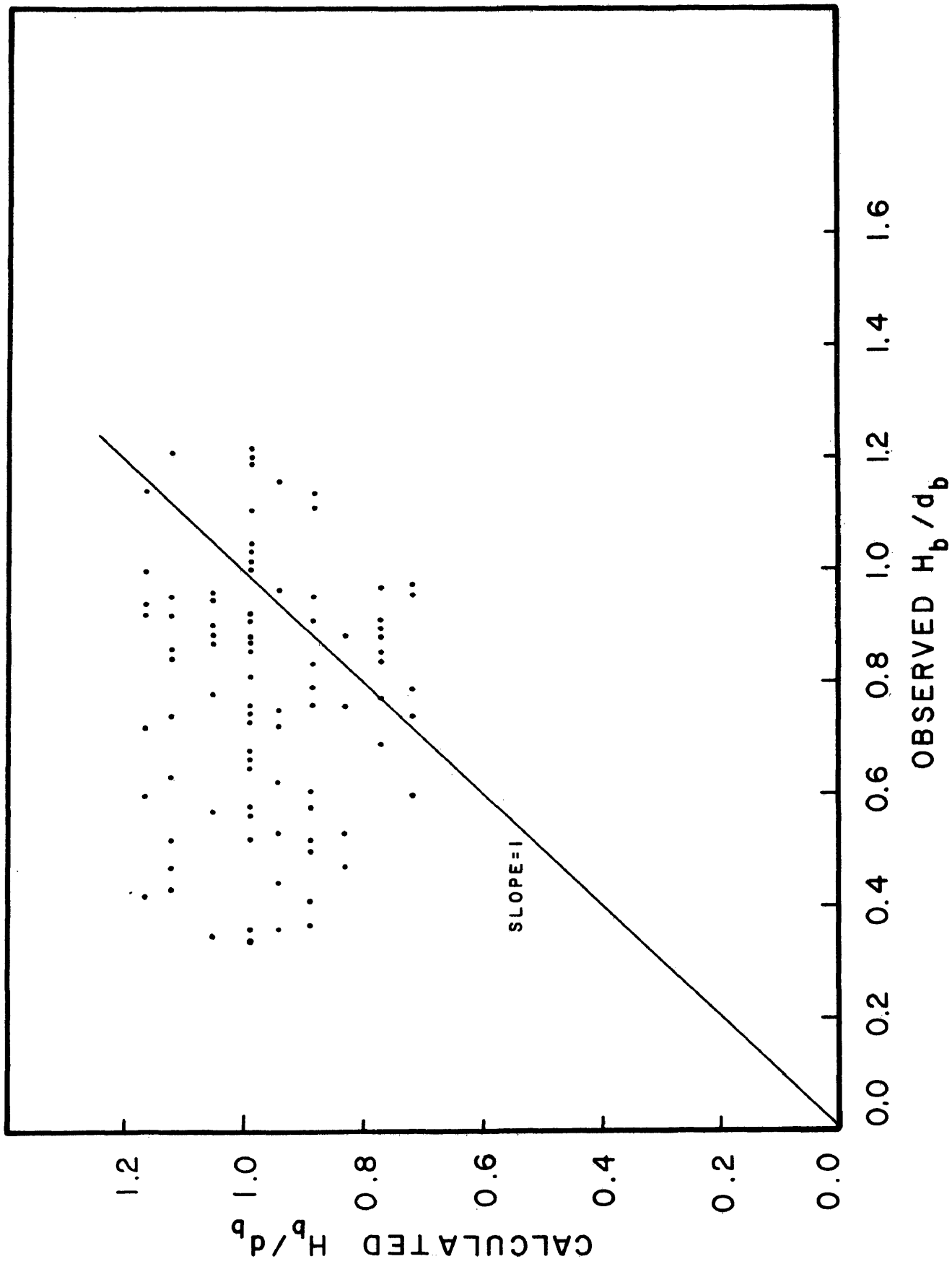


Figure 13. The observed breaking parameter H_b/d_b , compared to Weggel's formula. Values calculated by Weggel's (1972) formula $H_b/d_b = 0.72 + 5.6m$.



enabling him to define spacially the zone where breaking will occur.

Table 3

Summary of Wave Height to Water Depth Ratios

| Ratio | Total Sample | | Plunging Waves | | Spilling Waves | |
|------------|--------------|---------|----------------|---------|----------------|---------|
| | Mean | St.Dev. | Mean | St.Dev. | Mean | St.Dev. |
| η/d_b | 0.69 | 0.31 | 0.73 | 0.27 | 0.67 | 0.32 |
| H_b/d_b | 0.79 | 0.36 | 0.87 | 0.33 | 0.68 | 0.37 |
| H_b/d_t | 0.89 | 0.46 | 1.03 | 0.53 | 0.69 | 0.31 |

CHAPTER IV

THE POSITION OF THE WAVE CREST RELATIVE TO MEAN WATER LEVEL (M.W.L.)

Bascom (1964) indicated without presentation of data, that seven tenths of the total wave height was positioned above the mean water level. Bretschneider (1960) conducted a separate field investigation of this ratio as does this study. The distribution shown in Figure 14 gives Bretschneider's results for shallow water breaking waves in Lake Okeechobee Florida. The mean of the distribution is 0.707 and the standard deviation is 0.086.

As background to the question of the position of the wave crest relative to M.W.L., it is of interest to examine the results expected from various wave theories. The six wave theories that were used are (1) Solitary wave theory (2) Airy wave theory (3) Cnoidal wave theory (4) Second order Stokes wave theory (5) Third order Stokes wave theory and (6) Fifth order Stokes wave theory (LeMehaute, Divoky, and Lin, 1968). Figures 15 and 16 are the wave profiles and associated η/H_b values for the six different theoretical waves.

All except Airy and Solitary wave theories predict the values of 0.70 to 0.76 for η/H_b in breaking waves. All the profiles depict a wave in the shoaling zone. They are neither true deep water profiles nor breaking wave profiles.

Figure 14. Histogram depicting frequency of occurrence of η/H_b value for near breaking wave data in shallow water (Bretschneider 1960). There were 168 waves in the sample.

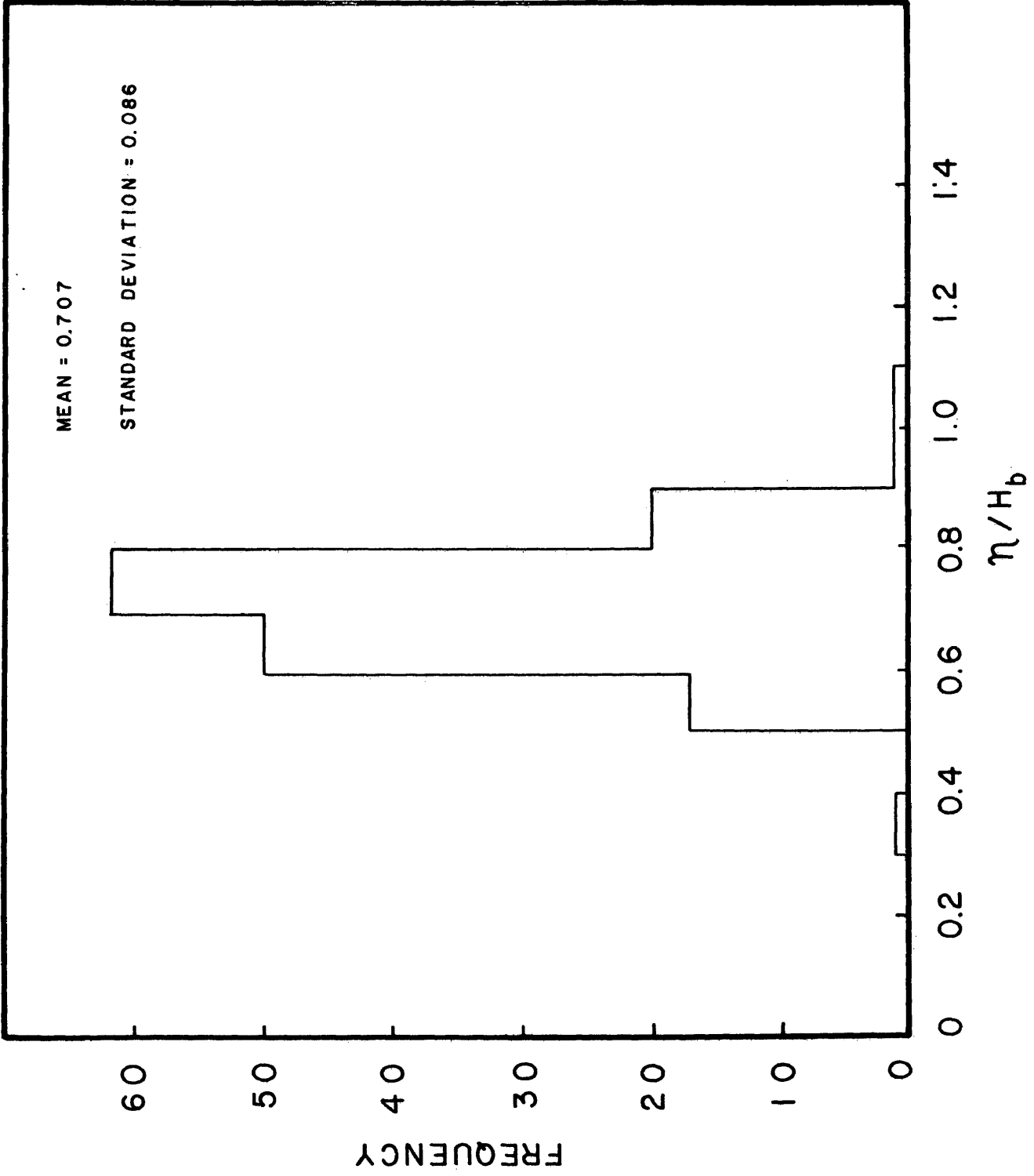


Figure 15. Depicts theoretical wave profile for

- 1) Solitary wave theory
- 2) Third order Stokes wave theory
- 3) Airy wave theory

| | | |
|----------------|--------------|--------------------|
| $\eta/H = 1.0$ | $\eta = 4.6$ | } SOLITARY |
| | $H = 4.6$ | |
| $\eta/H = .76$ | $\eta = 3.5$ | } 3rd ORDER STOKES |
| | $H = 4.6$ | |
| $\eta/H = .50$ | $\eta = 2.5$ | } AIRY WAVE |
| | $H = 5.0$ | |

SOLITARY

3rd ORDER STOKES

AIRY WAVE

MWL

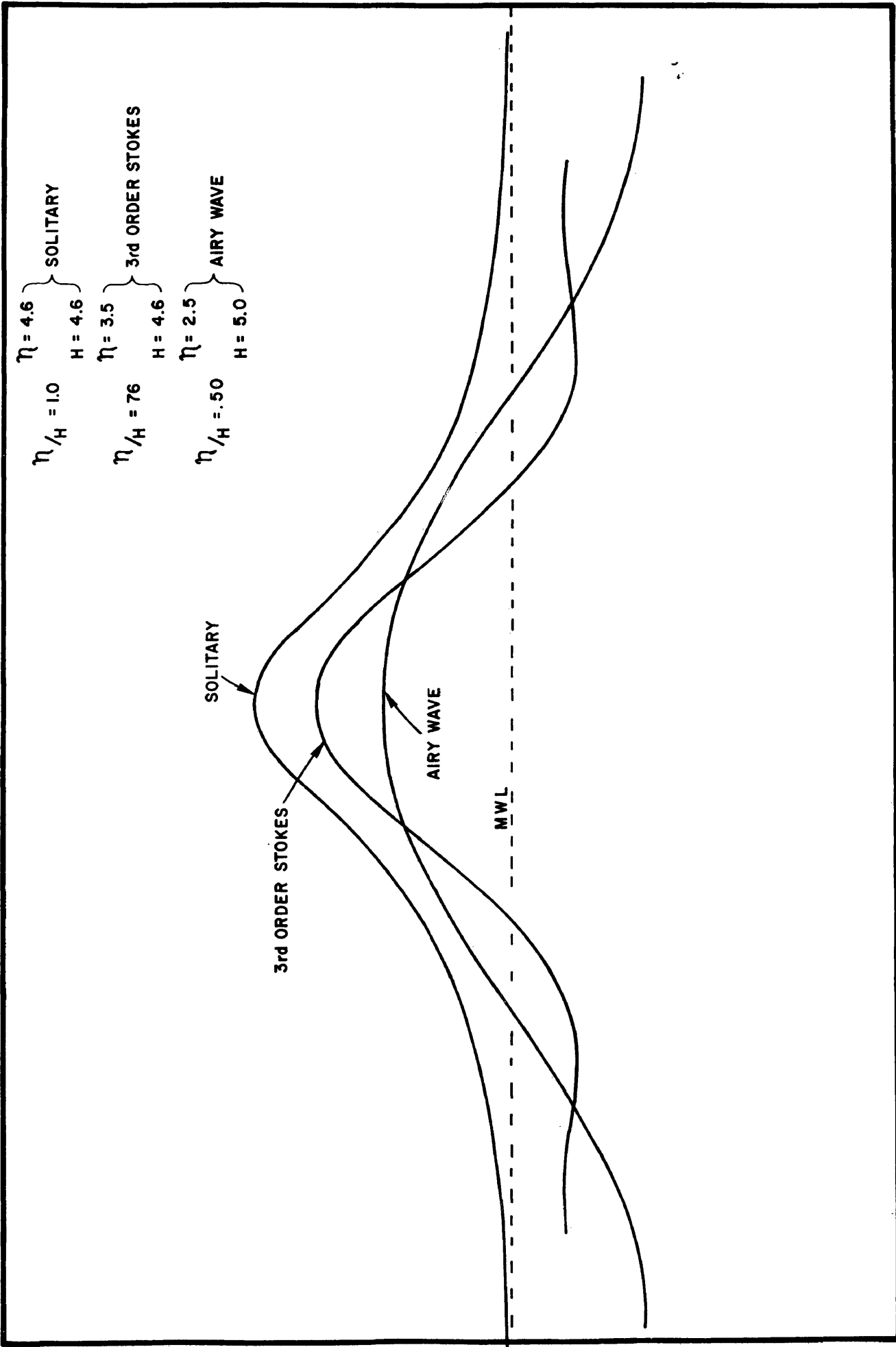


Figure 16. Depicts the theoretical wave profile for

- 1) Second order Stokes wave theory
- 2) Cnoidal wave theory
- 3) Fifth order Stokes wave theory

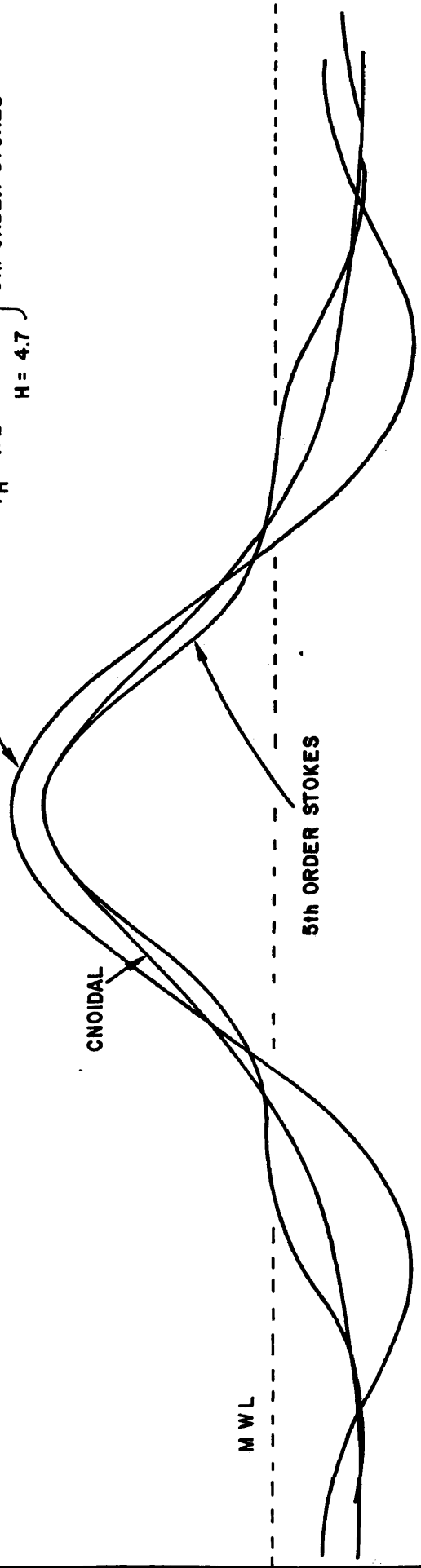
| | | |
|----------------|--------------|--------------------|
| $\eta/H = 6.5$ | $\eta = 3.9$ | } 2nd ORDER STOKES |
| | $H = 6.6$ | |
| $\eta/H = .72$ | $\eta = 3.5$ | } CNOIDAL |
| | $H = 4.7$ | |
| $\eta/H = .72$ | $\eta = 3.4$ | } 5th ORDER STOKES |
| | $H = 4.7$ | |

2nd ORDER STOKES

CNOIDAL

5th ORDER STOKES

MWL



It should be noted that the ratio η/H_b may obtain a value greater than one when η has a larger value than H_b . This may occur when the preceding trough, due to set up, or a small wave overtaking a larger wave, rides above mean water level. When this occurs H_b , the distance from the wave crest to the preceding trough, is smaller than η .

A search of the literature did not disclose previous field studies of deep water waves wherein the relative crest position was determined. However, Bretschneider (1960) reports on wave tank measurements for deep water η and H values. Figure 17 is a plot of these results in terms of η/H vs H/T^2 . It is noteworthy that the ratio η/H increases as the steepness parameter H/T^2 increases. The maximum reported η/H value is 0.58, which is considerably less than the averaged value of 0.68 for the same waves breaking at the beach.

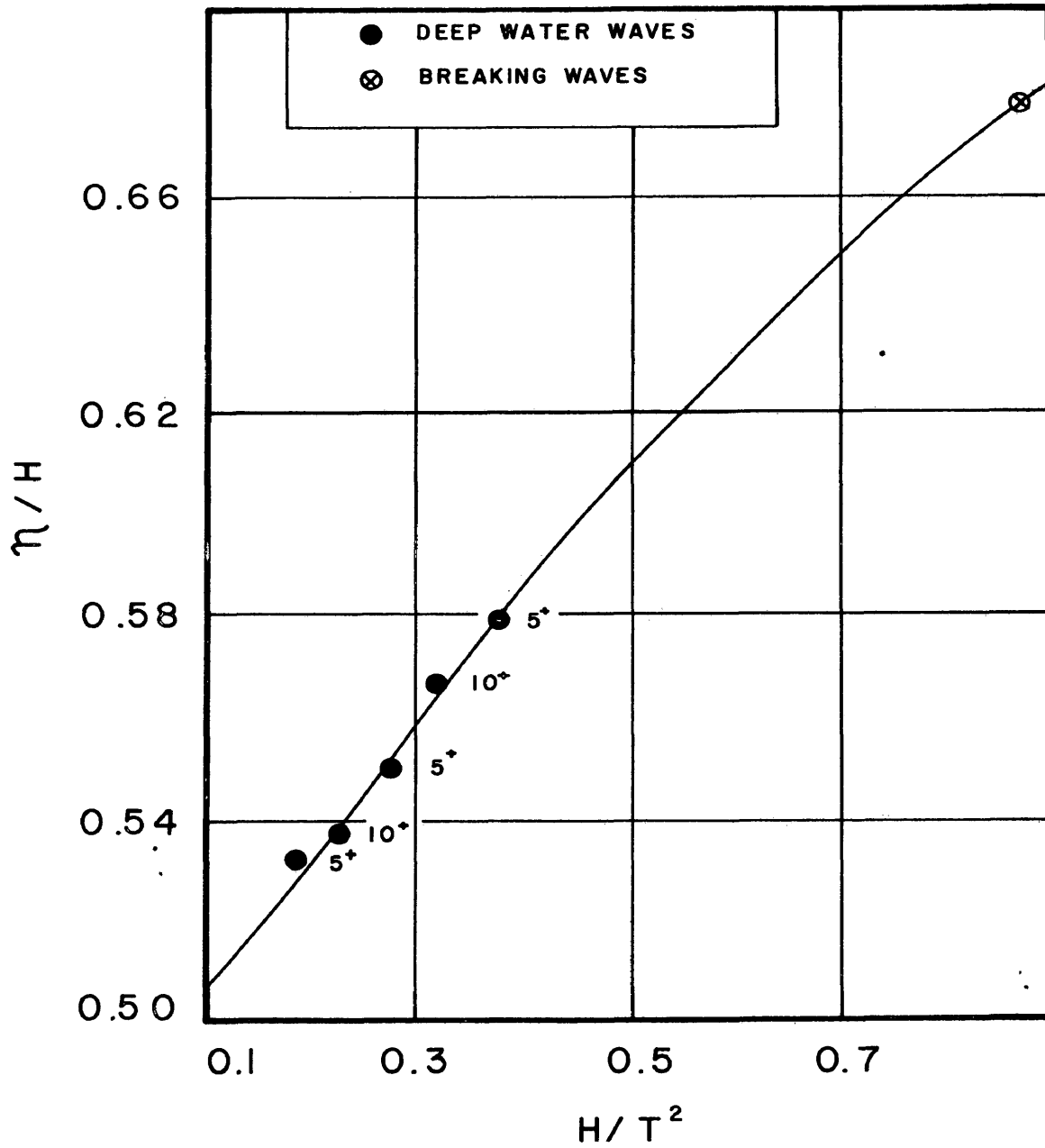
The distribution of η/H_b values for plunging and spilling waves from the Virginia Beach observation is shown in Figure 18A and 18B and the combined distribution is shown in Figure 19. It is apparent that the field data fall into separate classes for plunging and spilling waves.

Ninety-four percent of the plunging waves fall within the range of 0.7 to 0.9 η/H_b whereas none of the spilling waves fell within that range. Moreover the spilling waves do not appear to form a distinct mode.

It is apparent, from an examination of Figure 18, that the troughs of the spilling waves are above mean water level. This is reflected by the ratio of η/H_b having a value greater than

one. This occurs only when the value of η is larger than H_b . It is also of interest to note that from Figure 18 the plunging waves have a breaking ratio η/H_b of 0.81. This value is very close to the theoretical value of this obtained from 3rd order Stokes wave theory. Of the six wave theories examined, 3rd order Stokes wave theory gave the best geometrical description of the wave prior to breaking. Of the two types of waves examined, plunging and spilling, it may be inferred from Figure 18 that the plunging wave will break when approximately 80% of the crest is above mean water level.

Figure 17. η/H vs H/T^2 for wave tank observations.
The solid circles indicate "deep water" nonbreaking waves and the open circle with the X inside indicates the same waves breaking on the beach slope in the wave tank.



† THE NUMERAL SIGNIFIES THE NUMBER OF OBSERVATIONS
 AT THIS POINT

Figure 18. Histogram depicts the percent of spilling and plunging waves for the values of η/H_b . The mean and standard deviation for the spilling waves is 1.079 and 0.43 while, the mean and standard deviation for the plunging waves is 0.81 and 0.16. There were 51 and 69 waves in the spilling and plunging wave samples.

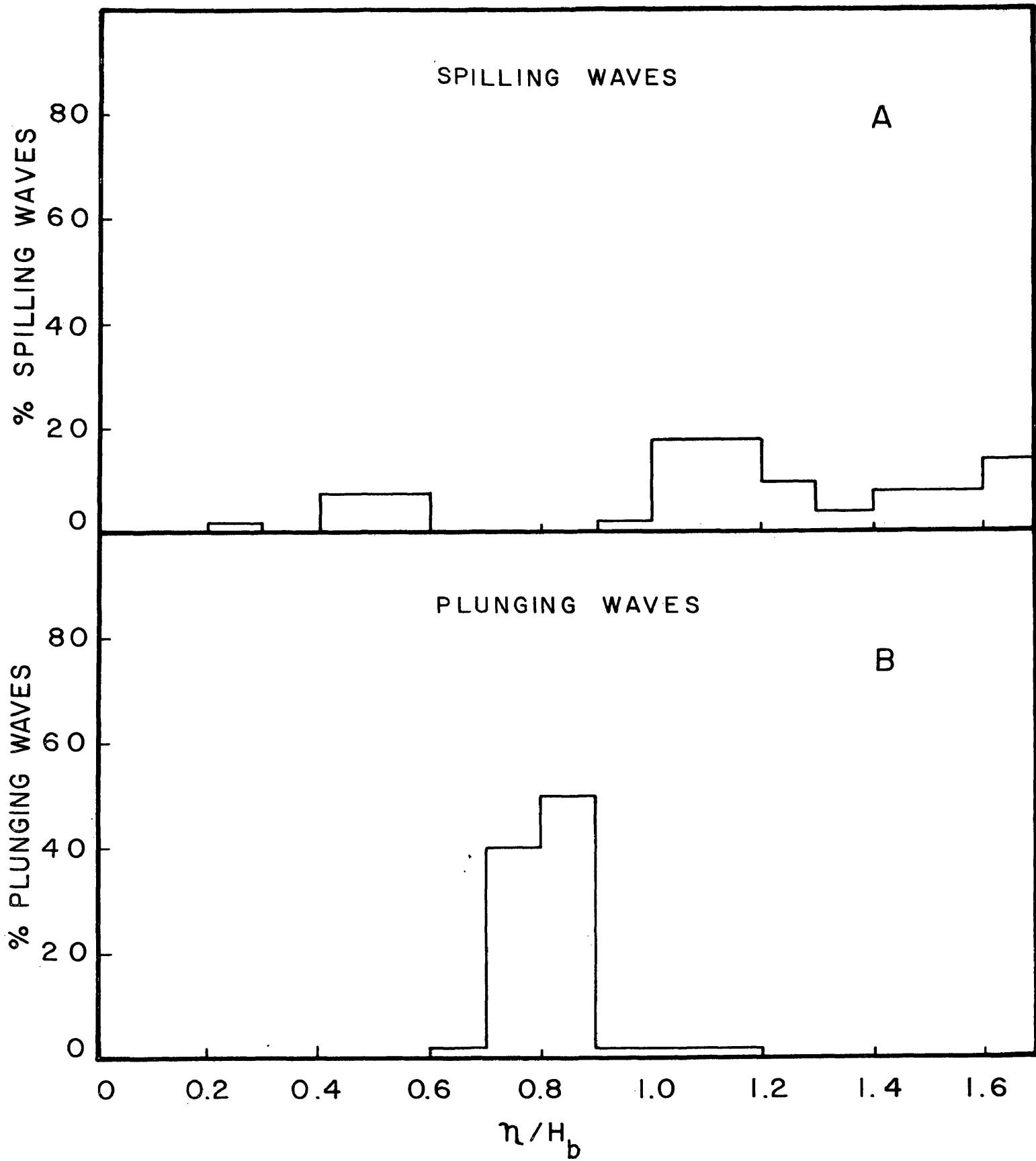
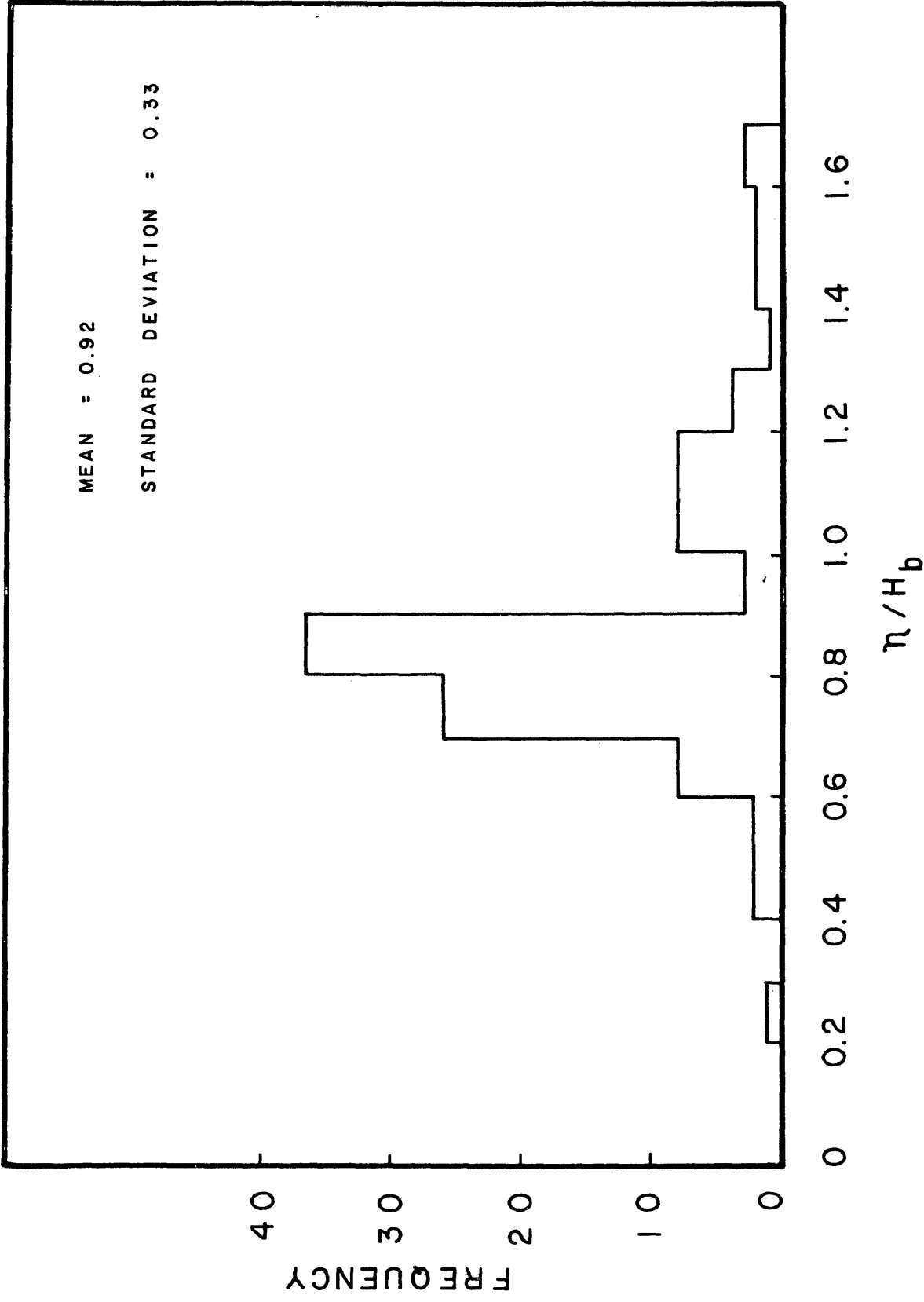


Figure 19. Histogram depicting frequency of occurrence of η/H_b value for breaking wave data from Virginia Beach. The mean and standard deviation for the combined sample is 0.92 and 0.33.



CHAPTER V

BREAKING WAVE HEIGHT PREDICTION

Analytic wave theories do not explicitly consider the condition of breaking waves in shallow water. Consequently the attempts at predicting the height of breaking waves have depended upon semi-empirical approaches. Weggles (1972) for example, presented a monograph procedure which utilized linear wave theory transformations and the relative breaker depth H_b/d_b .

Using linear wave theory, Komar and Gaughan (1972) devised an equation for predicting breaker height. The following derivation (Komar (1976), personal communication) follows from the conservations of energy flux.

$$E_o C_o n_o = E_b C_b n_b \quad (7)$$

Where E is wave energy density per unit of wave length, C is the phase velocity, and n the transmission coefficient. This equation makes the following assumptions that

- 1) $C_o = gT/2\pi$ (deep water)
- 2) $C_b = (gd_b)^{1/2}$ (shallow water)
where d_b equals the depth below M.W.L.
- 3) $n = (1/2) \left(\frac{1 + (4\pi d/L)}{\sinh(4\pi d/L)} \right)$
where the deep and shallow water limits of the sinh term yield n_o equal to $1/2$ and n_b equal to 1.0.
- 4) E (per wave length) and shallow water values are obtained by substituting the appropriate wave height.

ρ is the water density.

Now the energy equation may be rewritten as

$$\left(\frac{1}{8}\right) \rho g H_o^2 \left(\frac{g}{2\pi} T\right)^{\frac{1}{2}} = \left(\frac{1}{8}\right) (\rho g H_b^2) (g d_b)^{\frac{1}{2}} \quad (8)$$

which after reduction is equation 9

$$\left(\frac{g}{4\pi}\right) T H_o^2 = H_b^2 (g d_b)^{\frac{1}{2}} \quad (9)$$

The substitution of $\alpha = H_b/d_b$, the relative breaker depth allows the further reduction of the right side of the equation to the single shallow water variable H_b . Performing the substitution

$$\left(\frac{g}{4\pi}\right) H_o^2 T = H_b^2 (g H_b / \alpha)^{\frac{1}{2}} \quad (10)$$

solving for H_b ,

$$H_b = \left(\frac{g \alpha}{4\pi}\right)^{\frac{1}{2}} \left(\frac{H_o^2 T}{g}\right)^{\frac{2}{5}} \quad (11)$$

or approximately

$$H_b = K g^{1/5} (H_o^2 T)^{2/5} \quad (12)$$

where $K = \alpha^{1/5} (4\pi)^{2/5}$

Komar determined the proportionality constant $K = 0.39$, for wave tank experiments in which the beach slope ranged from 1:96 to 1:50. The deep water wave height, as determined by application of linear wave transformations to the tank data, ranged from 4.9 cm (.16ft) to 14.3 cm (.47 ft) and the wave period ranged from 0.80 to 0.50 seconds. Figure 20 is Komar's visual best fit line used in the determination of K.

In addition to the laboratory data Komar also used Munk's (1949) field observations from "deep" water wave gage on Scripps pier. Munk's data clusters around an extension of the line from the wave tank data where the waves were an order of magnitude smaller.

Although Komar used $C = (gd_b)^{1/2}$ as the expression for celerity, it is known that the total water depth $(\eta + d_b)$ is a more appropriate parameter, which results in $C = g(\eta + d_b)^{1/2}$ for celerity,

Equation 9 then becomes

$$(g/4\pi) T H_o^2 = H_b^2 g(\eta + d_b)^{1/2} \quad (13)$$

Using the results from Chapter III where $\bar{\eta} = 0.92H_b$ and $\bar{d}_b = 1.26H_b$ we have:

$$(g/4\pi) T H_o^2 = H_o^2 (g(2.18H_b))^{1/2} \quad (14)$$

Reducing further we have

$$H_b = K' (g)^{1/5} (T H_o^2)^{2/5} \quad (15)$$

$$\text{where } K' = (1/(1.48) (4\pi))^{2/5} = 0.31$$

In as much as the true deep water height was not available for the Virginia Beach study, linear wave transformations were used to calculate H_o viz:

$$H_o = H_b / ((1/2) (1/N) (C_o/C_b))^{1/2} \quad (16)$$

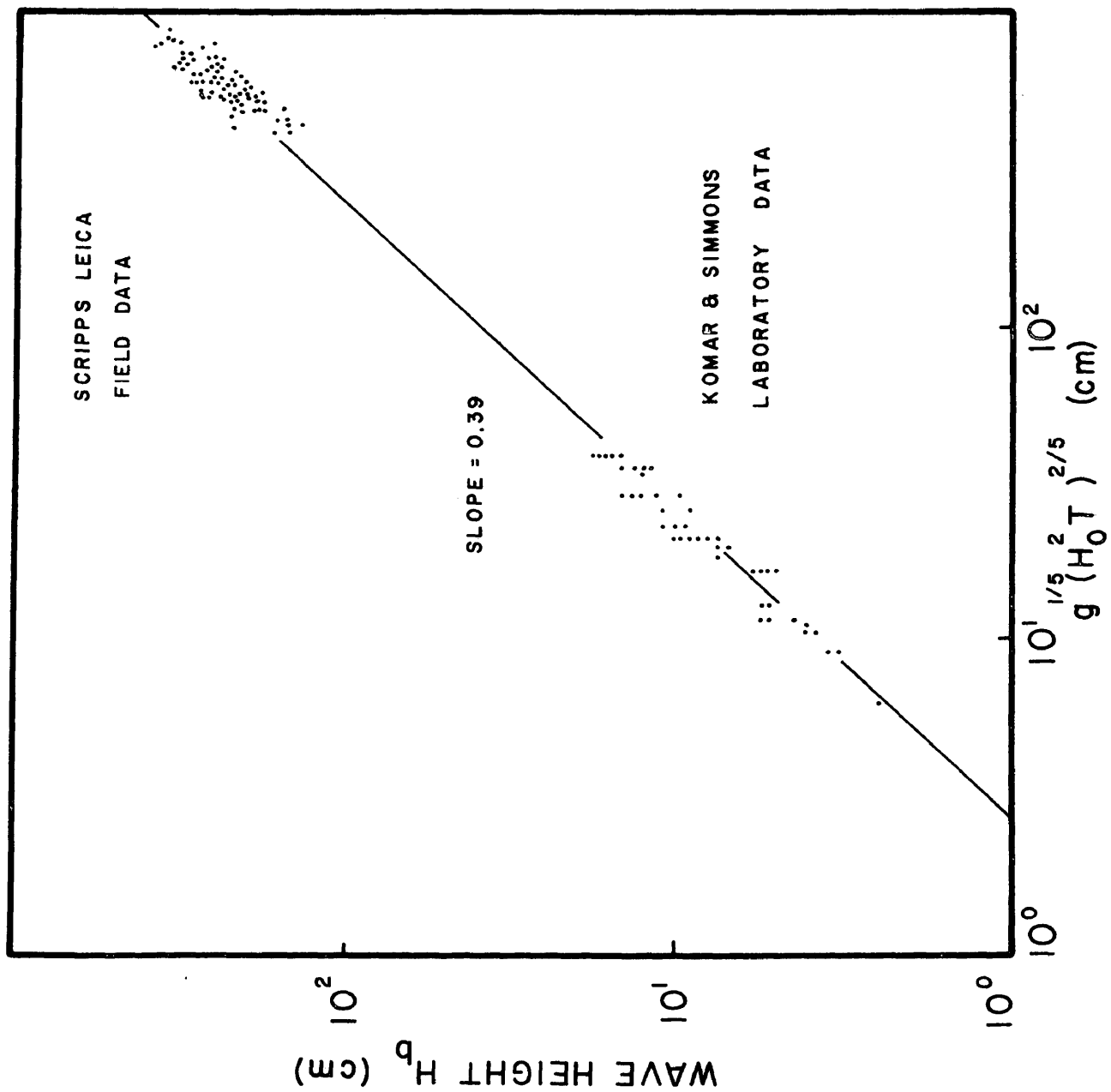
$$\text{where } C_o = (g/2\pi) T$$

and C_b , H_b and T were observed values.

The results from the Virginia Beach data are shown in Figure 21 wherein the observed celerity just prior to breaking is used in Equation 13. The least squares best fit line has a slope of 0.33. This value is very close to that expected for K' from the use of independent averages for η and d_b (Equation 15).

A more appropriate comparison between the empirically determined slope of K' would be that based upon calculated celerities (Equation 13 and 16). These results are shown

Figure 20. After Komar and Gaughan (1972). The observed breaking wave height (cm) plotted against the calculated breaking wave height (cm).



in Figure 22 where the least squares fit yields a slope of 0.36. The difference between this value (0.36) and that of K' (0.31) may be attributable to the fact that K' was determined by the independent averages, $\bar{\eta}$ and \bar{d}_b (Chapter III) whereas the least squares fit weights the average of the individual celerities based on the term, $(\eta + d_b)$.

Figure 23 is a scatter plot of the data from Komar (1972), Munk (1949), and the Virginia Beach field study. Equation 12 appears to fit the data well for breaking wave height ranging from 3 to 400 cm.

Although it appears that Komar has formulated a useful tool for predicting breaker height, it is important to remember that, with the exception of Munk's data, Komar used fictitious deep water wave height calculated from linear wave theory. Wave height changes due to refraction, diffraction, and bottom friction which did not enter the calculation, as Komar used wave period and local d_b/L to calculate H_o . The Virginia Beach data is similar except for the substitution of an observed breaker celerity. The H_o from Scripps pier is an approximation, not a true deep water wave height. The shoaling coefficient at Scripps pier varied from 0.99 to 1.44.

The next logical step would be to incorporate the effects of refraction and bottom friction with Equation 12 and then test the validity of the relationship. It is important to note that there is no explicit consideration of the beach slope in the preceding equations. The relative breaker depth, α , as discussed in Chapter III, may implicitly account for the effects of the beach slope.

Figure 21. The observed field breaking wave height (ft) plotted against the calculated breaking wave height (ft). The formula $g^{1/5} (H_0^2 T)^{2/5}$ was calculated using observed values of celerity and period. The equation of the regression line is:
 $y = 0.33x + b.$

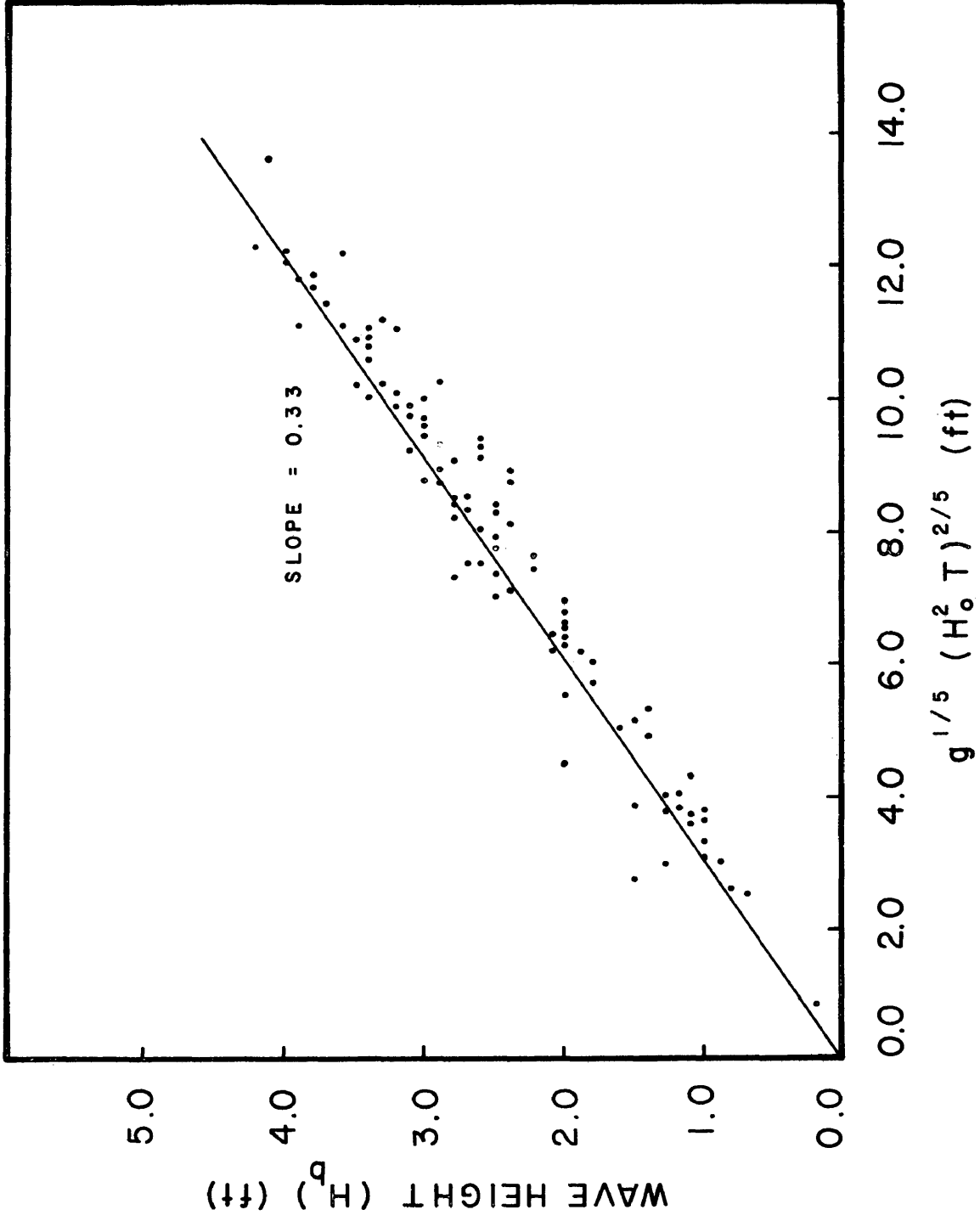


Figure 22. The observed breaking wave height (ft) compared to the calculated breaking wave height (ft). The formula $g^{1/5} (H_0^2 T)^{2/5}$ was calculated using the observed period and the celerity calculated from Solitary wave theory. The regression line was determined from the equation:
 $y = 0.36x + b$

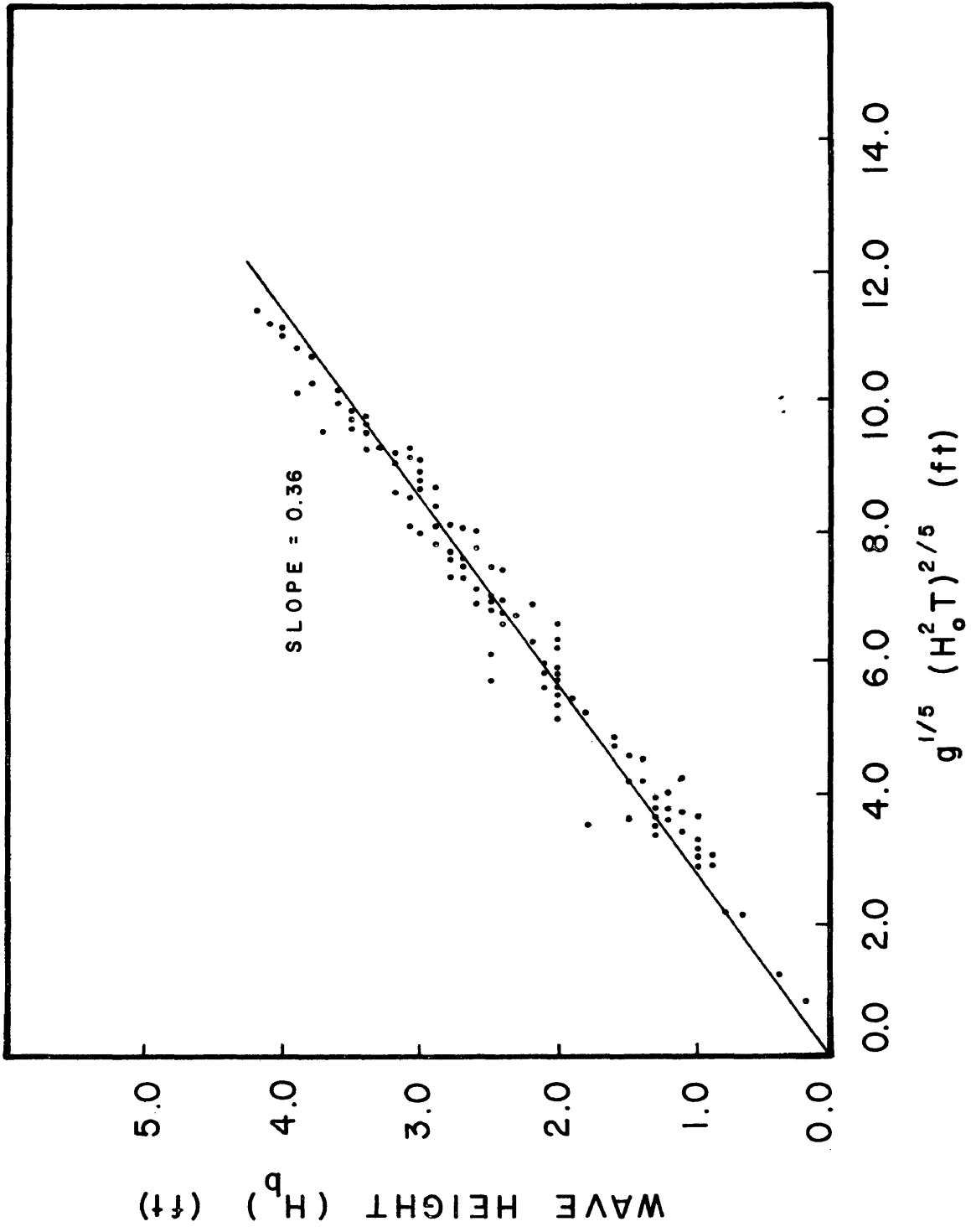
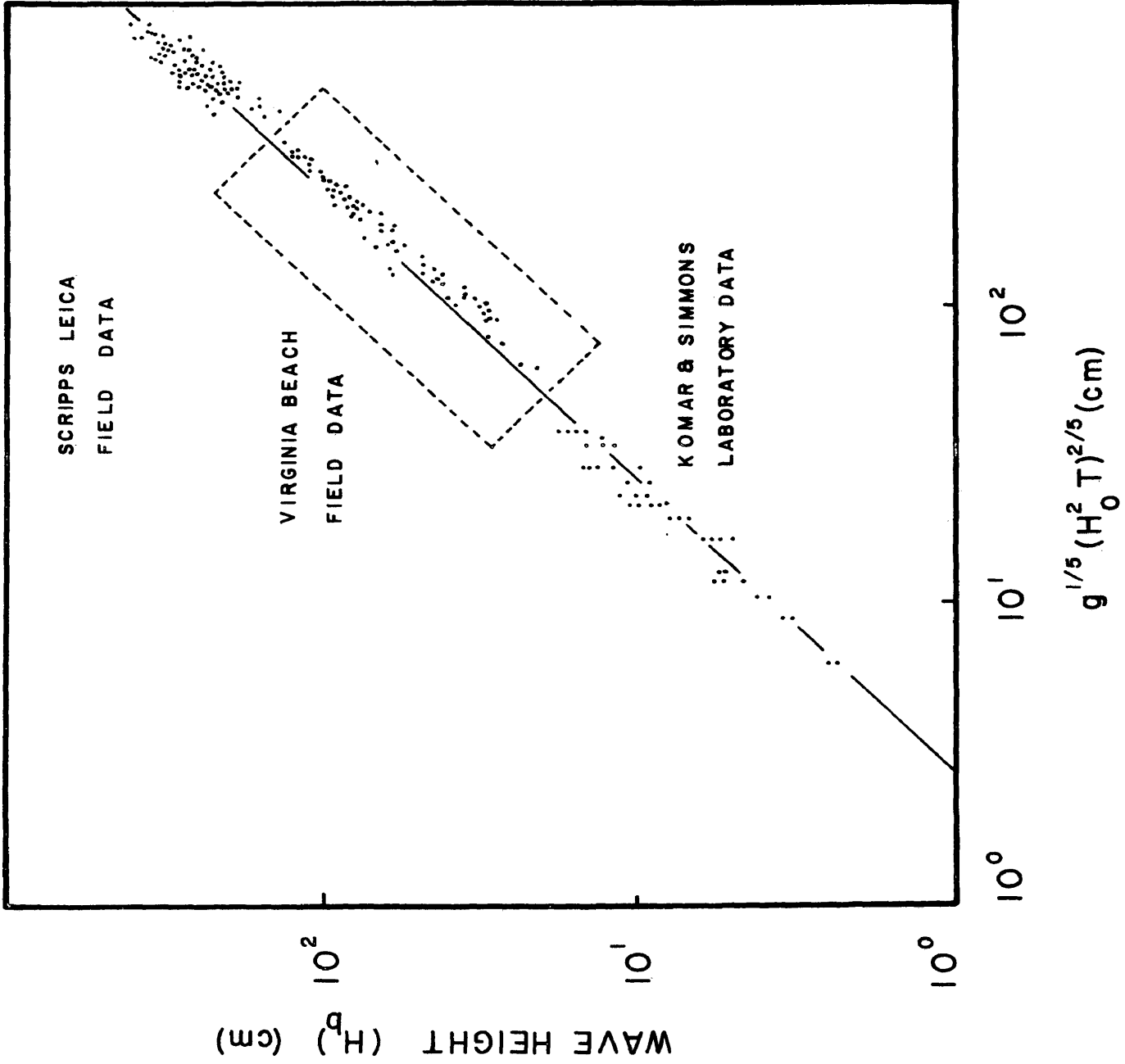


Figure 23. Compares the Scripps Leica field data
(Munk 1949) to the Virginia Beach field
data (1968) and to Komar's laboratory
data (1972).



CHAPTER VI

QUANTITATIVE BREAKING WAVE CLASSIFICATION

The ability to predict the type of breaking wave that will impinge on the shoreline is an important consideration to a coastal engineer designing a shore structure. Wave types differ in the rate of energy dissipation and the amount of force expended per unit cross sectional area of the wave. Moreover, breaker classification is important when making comparisons between laboratory and field data. As seen in Chapter III, the results obtained from various predictive ratios and formulas are significantly affected by the type of wave examined. Thus when comparing data obtained from different environments, it is useful to have a quantitative classification system that allows comparisons of wave similarity.

Across the spectrum of breaking waves there is a continuous gradation of breaker types. Galvin (1967) has divided breaker types into three classes, spilling, plunging, and surging.

A spilling wave is one in which the wave crest becomes unstable at the top and flows down the front face of the wave producing an irregular foaming surface. A plunging wave is one in which the wave crest curls over the front face of the wave and falls into the base of the wave producing an

enclosed vortex. Last, a surging wave is a wave in which the crest does not break while the base of the foreface of the wave advances up the beach.

Galvin (1968A) used deep water wave steepness, H_o/L_o , where H_o is deep water wave height and L_o is deep water wave length, as a predictor of breaker types. His calculations were based upon linear wave theory transformations applied to monochromatic waves generated in a wave tank.

In order to compare data from different beaches, it was necessary to group the beaches according to beach slope. Galvin (1968A) noticed that if all other conditions remained constant, breaker type would vary with the beach slope. He found that by adding a reciprocal slope squared term to his equation that he could predict breaker type in the wave tank, and he no longer needed to separate the data according to beach slope.

$$\text{Offshore Parameter} = (H_o/L_o m^2) \quad (17)$$

Using the offshore parameter Galvin was able to classify breakers according to the calculated value. Figure 24A is a plot of his data. Spilling waves have been assigned values below 0.09, plunging waves 0.09 to 4.80, and surging waves above 4.80. Using this equation, only plunging waves form a distinct group (Figure 24A).

Galvin, beginning with wave steepness at breaking, H_b/L_b , separated the wave length into its component parts, wave celerity, C , and wave period, T . By making the substitution $L = CT$ and squaring, the breaking wave steepness

factor becomes $H_b^2/(gd_b T^2)$. By factoring and omitting the constant H_b/d_b (refer to Chapter III) the augmented wave steepness factor becomes (H_b/gT^2) . Galvin then discovered that his factor, divided by the beach slope resulted in an accurated inshore parameter, Equation 18, for classifying breaker types in the wave tank.

$$\text{Inshore Parameter} = H_b/(gmT^2) \quad (18)$$

Figure 24B is Galvin's plot of the wave tank inshore parameter and breaker type. Breaker types have been empirically assigned inshore parameter values of 0.003 and less for surging waves, 0.003 to 0.068 for plunging waves and 0.068 and larger for spilling waves.

The first comparison of Galvin's wave tank data and the Virginia Beach field data utilized the offshore parameter. Equation 19, derived from Airy wave theory was used to calculate deep water wave height for the Virginia Beach field data.

$$H_o = H_b / ((\frac{1}{2}) (1/n) (C_o/C))^{1/2} \quad (19)$$

Where n is the transformation coefficient; breaker height, H_b , and celerity, C , are observed values whereas deep water celerity, C_o , is calculated using Equation 20

$$C_o = (g/K)^{1/2} \quad (20)$$

$$K = 2\pi/L_o \quad (21)$$

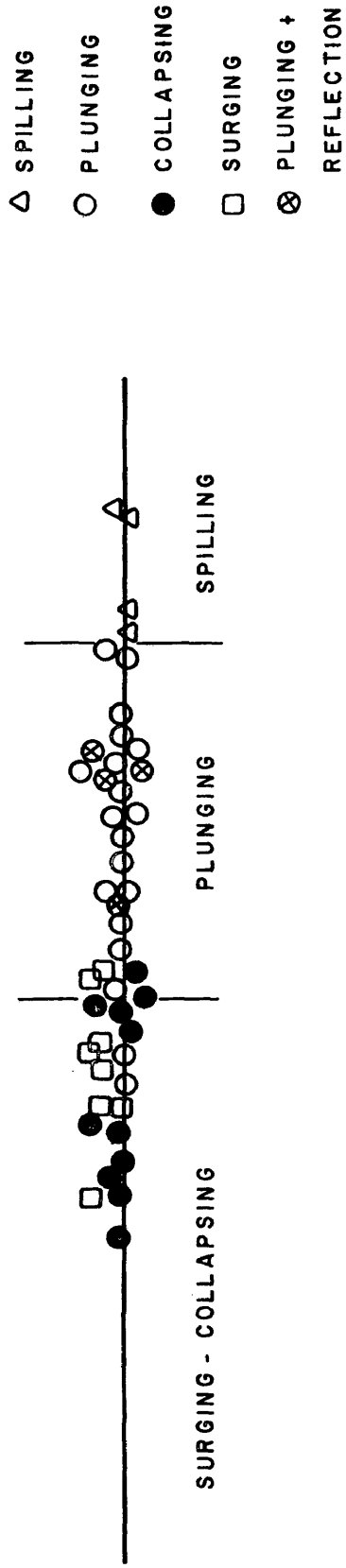
and deep water length, L_o , is calculated from equation 22

$$L_o = gT^2/2\pi \quad (22)$$

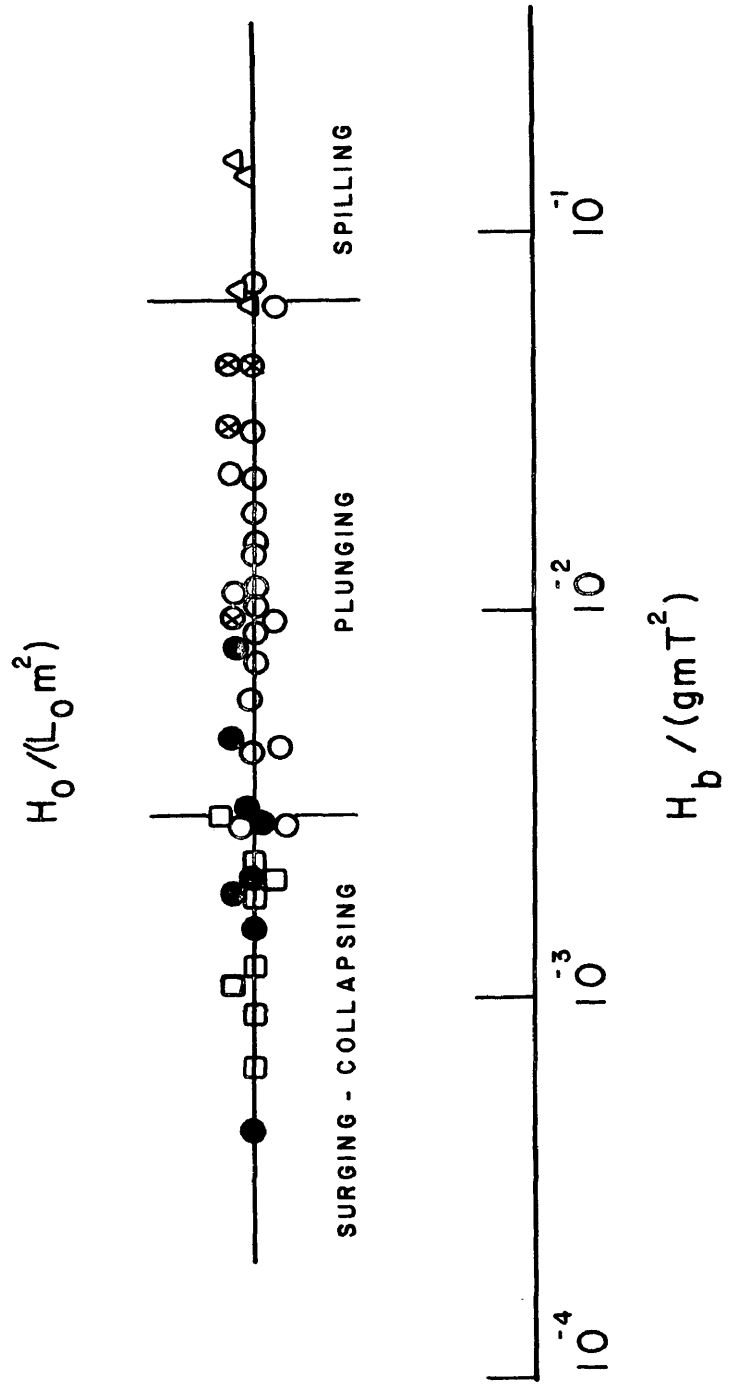
Figure 25A shows the results of the calculations of the offshore parameter that were made using equation 17 as seen

Figure 24. The grouping of spilling, plunging, collapsing, and surging waves observed in the laboratory by Galvin (1968A). Part A shows the offshore parameter, part B shows the onshore parameter.

A



B



△ SPILLING

○ PLUNGING

● COLLAPSING

□ SURGING

⊗ PLUNGING + REFLECTION

in this figure, the values of the offshore parameter calculated for the observed waves did not fall within Galvin's range of values. Moreover, this parameter does not exhibit any clearly defined breaker classes. The best group was for plunging waves which had a range of values from 0.9 to 15.0.

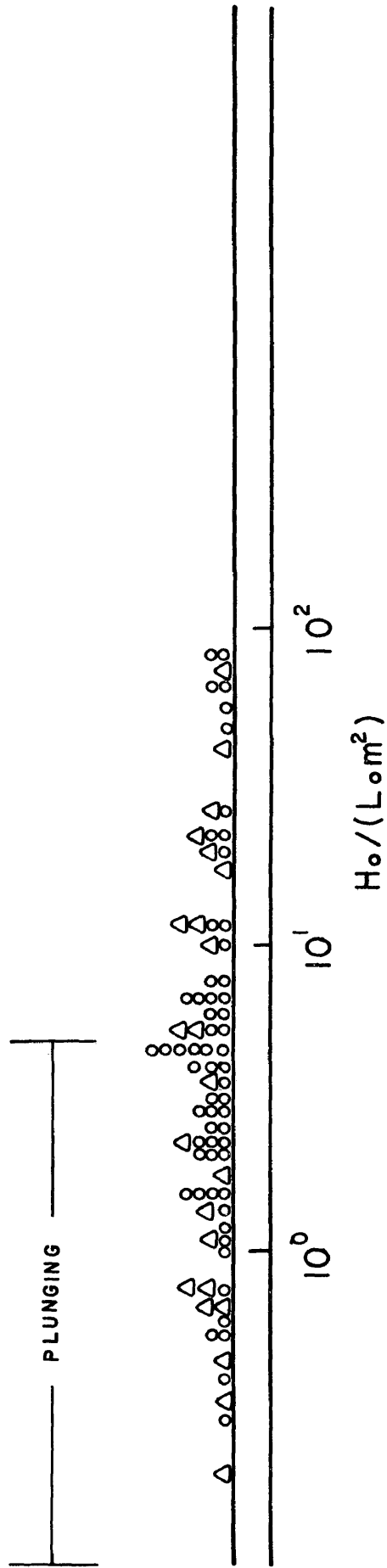
Figure 25B is a plot of the onshore breaker parameter. Again, there are no clearly defined groupings of data that may be used for breaker classification.

The offshore and onshore breaker classification predictors, equations 17 and 18, utilize equations that are similar to the wave steepness equations. Whereas the criteria worked very well for wave tank data, they did not provide a reliable basis for differentiating the waves observed at Virginia Beach. The best grouping of the offshore parameter for the observed waves was for plunging waves which had a value of 0.9 to 15.0. Forty-five percent of the spilling waves fell within this range, whereas 25% of the plunging waves did not. The onshore parameter did not suggest any classes as the spilling and plunging waves were intermixed. Although the quantitative breaker type predictors have proved useful in the laboratory, in its present form it is not directly applicable to the oceanic breaker zone.

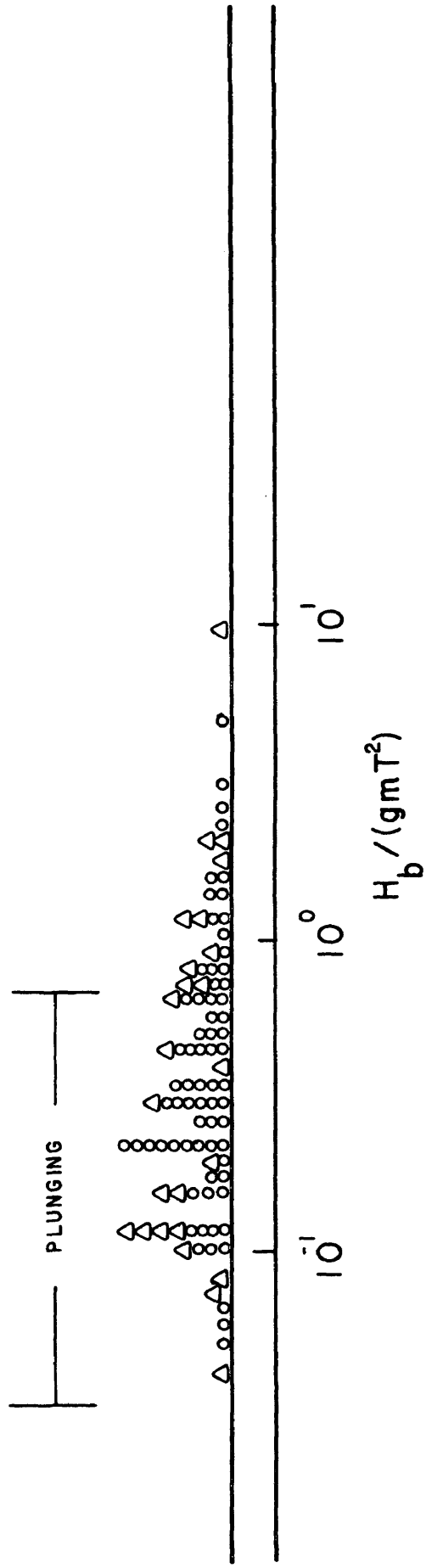
Figure 25. Part A shows the offshore parameter, part B shows the onshore parameter. The parameters are calculated with observed values of the wave height, beach slope, and wave period obtained from the Virginia Beach field data. The ranges marked for plunging in the drawing show the highest concentrations of plunging waves determined by visual best fit.

○ PLUNGING
 △ SPILLING

A



B



CHAPTER VII

EVALUATION OF PLUNGE DISTANCE

The distance the face of a plunging wave travels during breaking is known as the plunge distance (Galvin 1968B). A more formal definition of the plunge distance is the horizontal distance the wave crest travels from the defined break point until the crest again come in contact with the water surface (Figure 26). Because of differing wave geometry it is extremely difficult to define a single break point. To minimize this difficulty, the break point is defined as the point at which any part of the wave foreface becomes vertical. Galvin obtained his data by photogrametric observations in the wave tank.

Plunge distance (X) may be calculated from the characteristic velocity (U) and time (t) as

$$X = Ut \quad (23)$$

by analogy

$$P = C_b t_b \quad (24)$$

Where P is plunge distance, C_b is celerity at breaking and t_b , is the time elapsed between the break point and the contact of the crest with the water surface.

The time required for a particle to fall from the crest of the wave to the trough may be calculated as in elementary physics for a particle subject to gravitational acceleration,

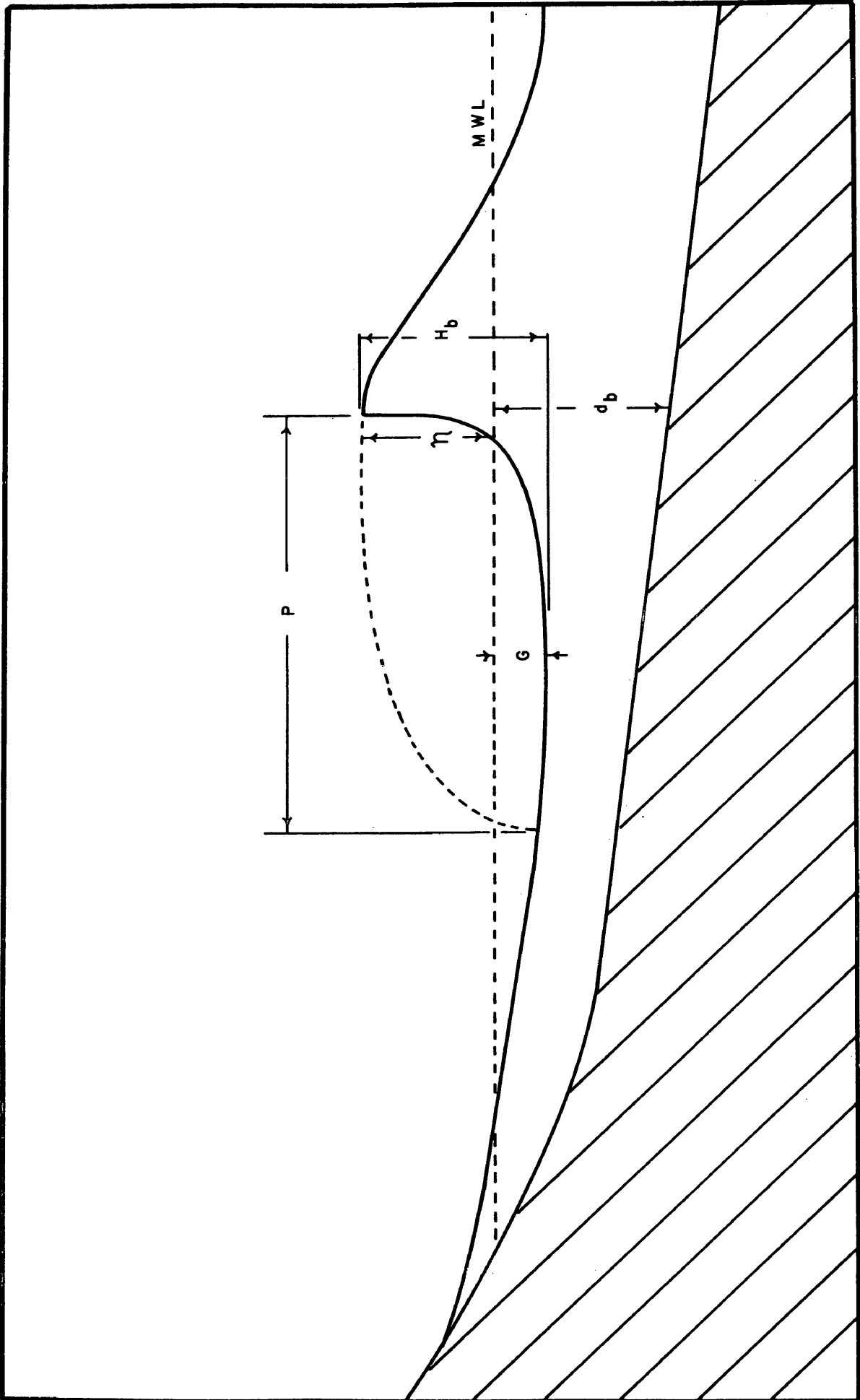
Figure 26 Depicts the breaking wave and the parameters used in the evaluation of the plunge distance.

H_b = breaking wave height from crest to trough,

G = the trough depression below M.W.L.,

P = horizontal plunge distance

η = wave height from top of the crest to M.W.L.



where in

$$y = y_o + V_{yo} + \frac{1}{2}a_y^2 \quad (25)$$

using the following 4 assumptions, equation 22 may be simplified to equal equation 23:

- 1.) The break point of the wave is considered to be the origin, $y_o = 0$.
- 2.) There is no vertical motion at the break point, $V_{yo} = 0$.
- 3.) Gravity accounts for all the acceleration in the vertical plane, $a_y = g$.
- 4.) The vertical distance traveled by the water partical is the height of the breaking wave, $y = H_b$.

so that

$$H_b = (\frac{1}{2})gt_p^2 \quad (26)$$

Solving for the Plunge time, t_p ,

$$t_p = (\frac{1}{g}) (H_b)^{\frac{1}{2}} \quad (27)$$

Plunge time is now defined in terms of wave height and a constant.

At the breaker position the internal partical velocity is assumed to be equal to the wave celerity (Galvin 1968B).

The celerity is given by:

$$C_b = ((g) (H_b - G + d_b))^{\frac{1}{2}} \quad (28)$$

Equation 28 is the solitary wave expression for wave celerity, the terms are defined in Figure 26. The depth of the trough below mean water level, G , may be approximated in terms of wave height, Bascom (1964) (Also see chapter IV).

$$G = 0.25H_b$$

Substitution in Equation 28 and simplifying and factoring H_b , yields the breaker celerity.

$$C_b = ((g H_b) (0.75 + \beta))^{1/2} \quad (30)$$

where β is the reciprocal of the breaking height ratio

$$d_b/H_b = 1.28.$$

Equation 30 may then be further reduced

$$C_b = 8(H_b)^{1/2} \quad (31)$$

Now both the plunge time and breaker celerity are defined in terms of breaker height. Substituting into equation 24 yields the plunge distance in terms of wave height.

$$P = 2H_b \quad (32)$$

Galvin determined for his wave tank data that the observed range of P/H_b was 1.0 to 4.5 with an average value of 3.0. Thus experimentally he found

$$P = 3H_b \quad (33)$$

He attributed the variation to an error in assuming that the water particles were in free fall at the break point.

Figure 27 is a comparison of Equation 32 and the observed values at Virginia Beach. The equation clearly does not predict the plunge distance.

The average ratio of plunge distance to wave height (P/H_b) for the Virginia Beach data was 5.9.

Figure 28 depicts the results of a standard regression analysis correlation of plunge distance and wave height. The slope of the least squares fit line is 5.6. The r^2 term, which defines the amount of variation explained by regression using only the two variables H_b , P , was 13.9%. The residual, $(1-r^2)$, is approximately 86%. This is the amount of error unaccounted for by regression.

Figure 27. Comparison of the observed plunge distance to the plunge distance predicted by Equation 32. The slope of the line through the data points was determined by the method of least squares. There were 120 waves used in this analysis.

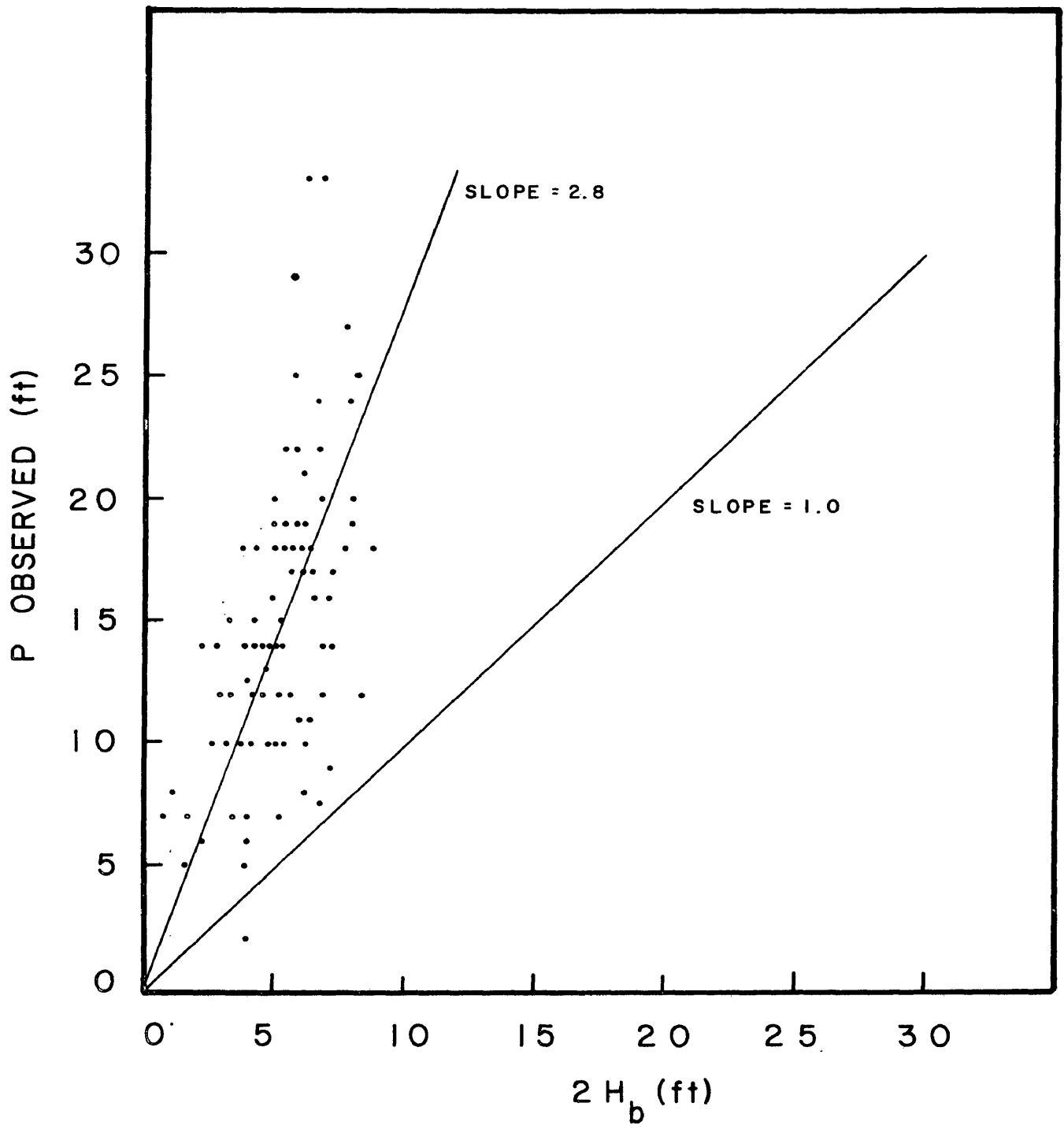
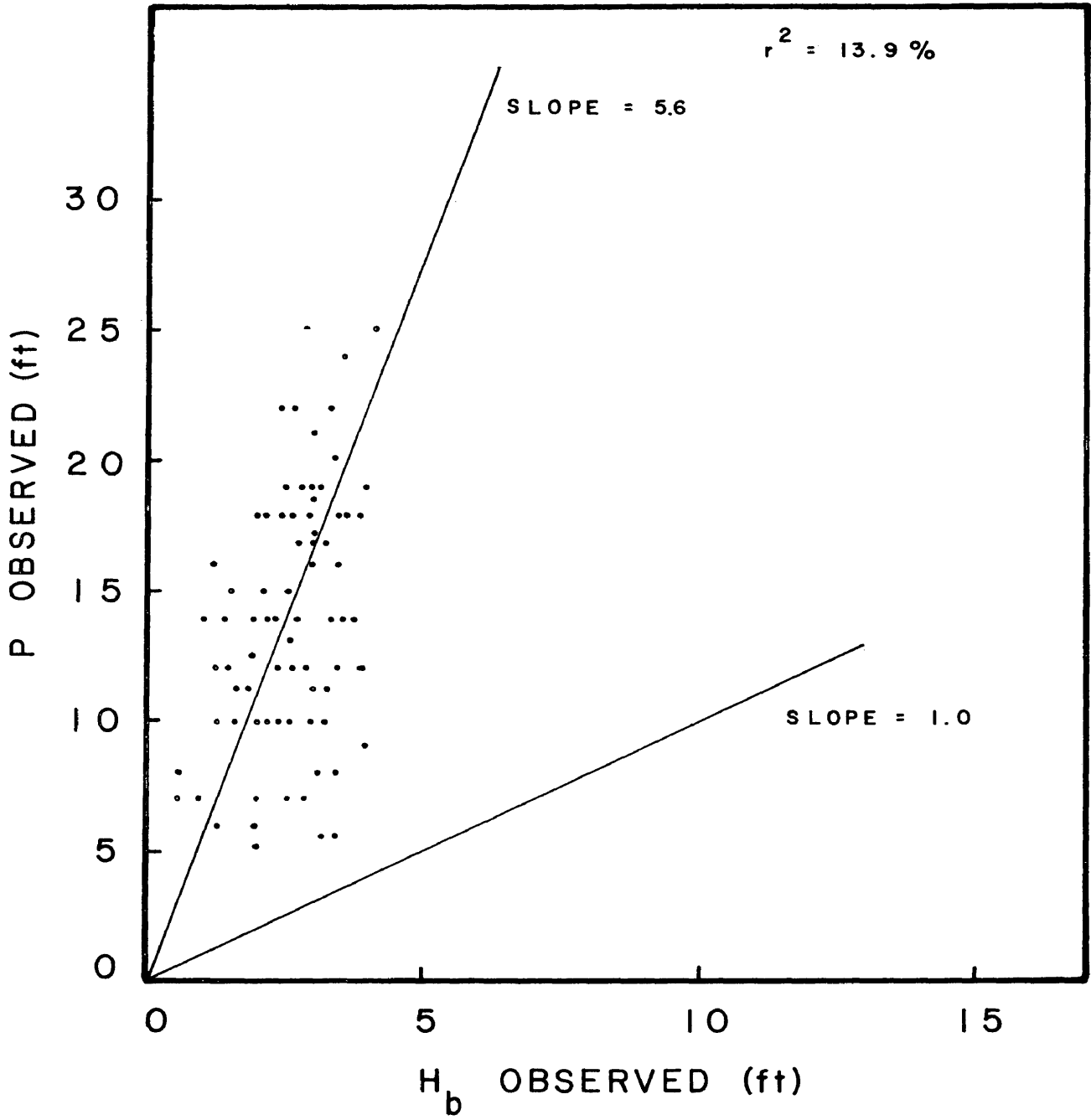


Figure 28. A comparison of the observed plunge distance, P , to the observed wave height, H_b . The slope of the regression line was determined from the equation $y = 5.6x + b$.

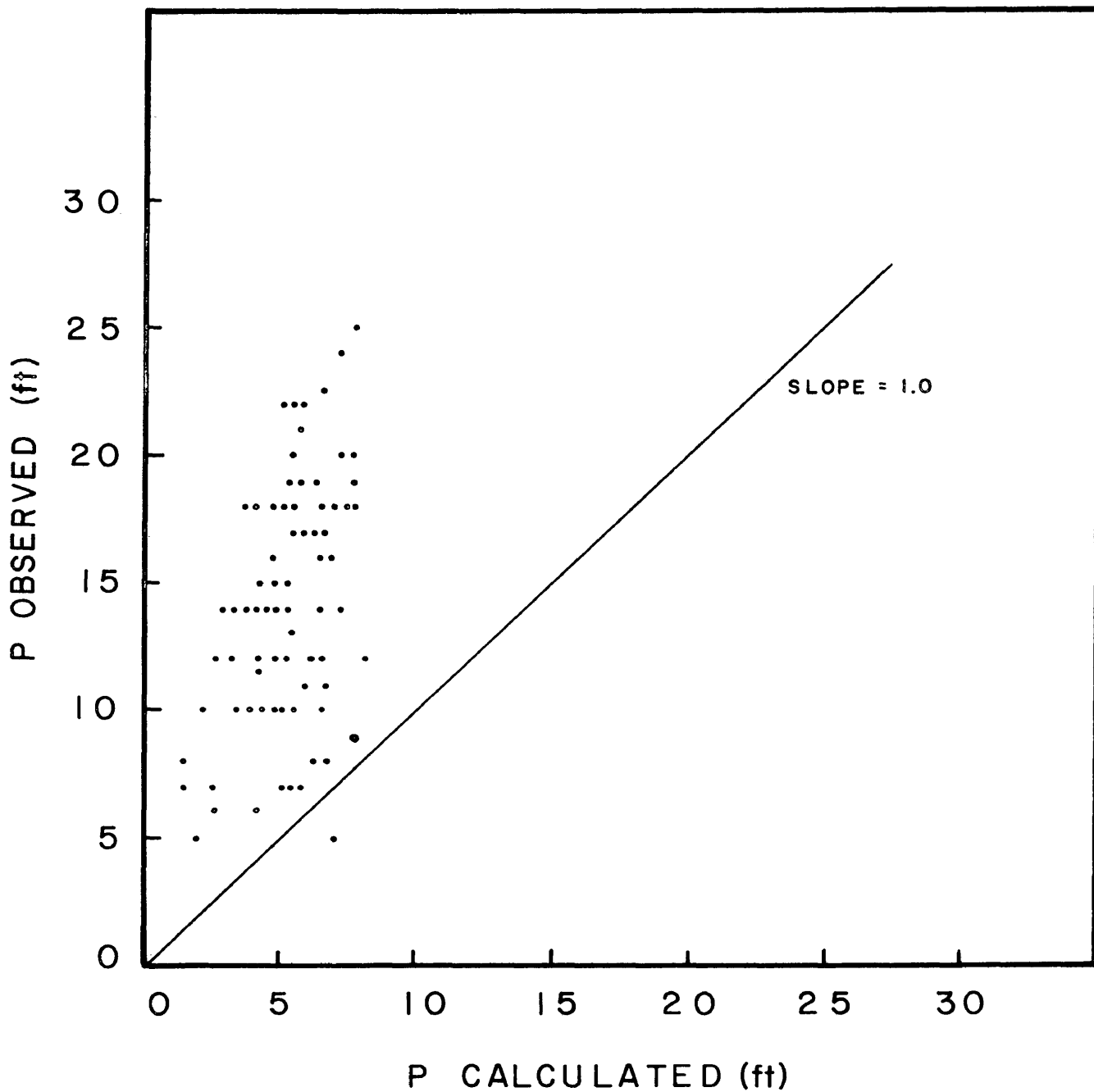


The variation between the predicted and the observed plunge distance in part is explained by the derivation of Equation 21 and by the oversimplification of the many variables. Figure 29 is a comparison of the observed plunge distance and the plunge distance calculated through Equation 24. The celerity and the plunge time were calculated by Equation 27 and 28, where H_b , G , and d_b are observed values. As seen in Figure 29 the use of observed H_b , G , and d_b in these calculations does not reduce the scatter of the data.

A closer examination of the simplifying assumptions of Equation 21 may reduce some of the discrepancies. The internal particle velocity is assumed to be totally in the horizontal plane with no accelerations at the break point. Actually, the water particles do not move totally in the horizontal plane as, by definition, the break point is reached when any portion of the wave face becomes vertical. The water particles must have a vertical velocity component in order to enable the wave foreface to curl over to form a plunging wave (Morrison & Crooke 1953).

The predictive equations for the plunge distance utilize a basic physics formula involving the celerity at breaking and plunge time. The celerity was calculated from solitary wave theory, Equation 28. In order to determine the accuracy of the equation, the calculated celerity and the observed celerity were compared (Figure 30). An average of 12% of the variation in Equation 24 may be attributed to the calculation of celerity.

Figure 29. A comparison of the observed plunge distance, P , to the plunge distance calculated by Galvin's (1968B) formula $(\frac{1}{4}) (H_b)^{\frac{1}{2}} (g(H_b - G + d_b))^{\frac{1}{2}}$. The formula is calculated using observed values of H_b , G , d_b . The first term represents the plunge time and the second term represents the wave celerity.



$$P = (1/4)(H_b)^{1/2}(g(H_b - G + d_b))^{1/2}$$

Figure 30. A comparison of the observed celerity to the celerity calculated by solitary wave theory.

The plunge time is the time that is required for the wave crest to travel from the break point to the point where the crest contacts the water surface. Figure 31 is a comparison of the observed and calculated plunge times at Virginia Beach. Sixty-four percent of the variations observed for Equation 24 may be attributed to calculation of the plunge time.

The error attributed to the plunge time and the wave celerity was calculated by comparing the observed values to the calculated values of the celerity of each individual wave. This difference was then transformed to an averaged error by the following equation:

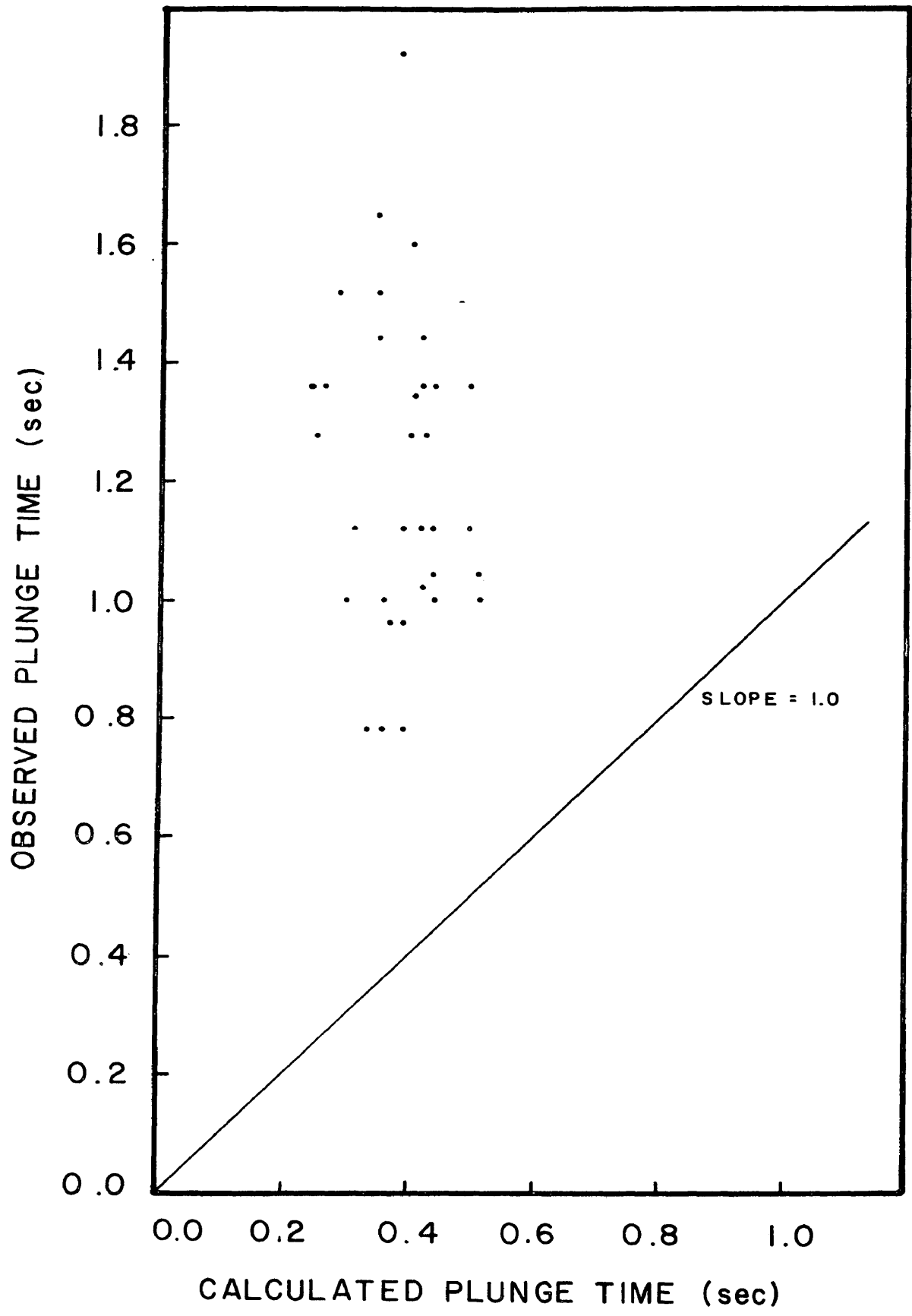
$$\frac{\sum \frac{C_{OB} - C_{cal}}{C_{ob}}}{N} \times 100 = \% \text{ ERROR} \quad (34)$$

The error attributed to the calculated plunge time was also calculated with the use of Equation 34.

Thus it is clear that the weak link in the analysis is associated with the assumptions used in the calculation of the plunge time. It is apparent the simple free fall trajectory model is insufficient to describe the kinematics of the post breaking wave. This insufficiency may be attributed to the fact that the water particles have acceleration components in both the vertical and the horizontal direction after the wave has passed the defined break point.

To determine if the observed variations in Figures 29, 30, and 31 were due to the oversimplifications of the celerity and plunge time which neglect local terms such as particle accelerations,

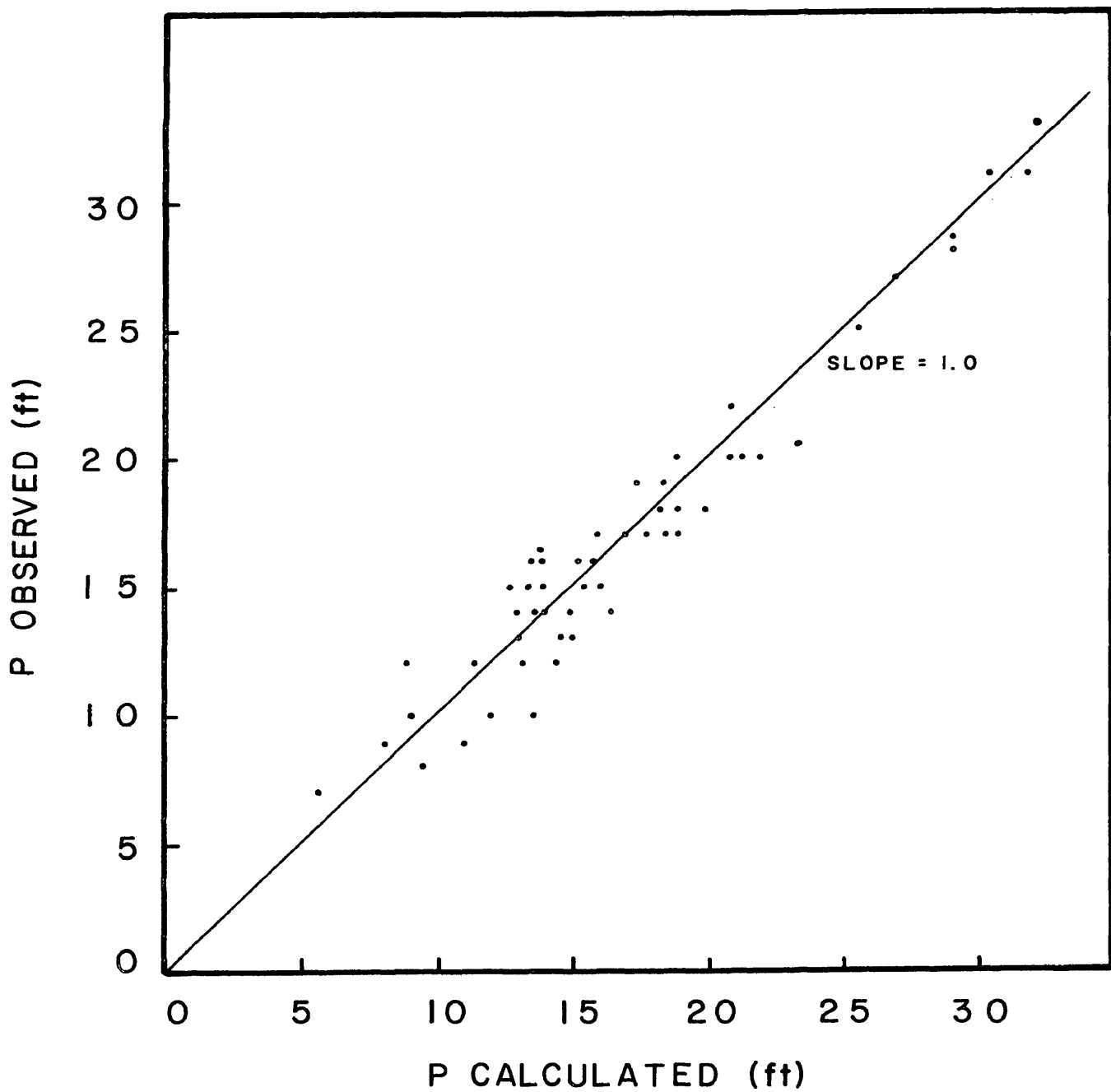
Figure 31. A comparison of the observed plunge time
to the calculated plunge time.



local accelerations, and definition of the break point, the observed plunge distances were compared to the plunge distance calculated with the observed wave celerity and the observed plunge time (Figure 32). The calculated plunge time and wave celerity that were both taken directly from the filmed data. An examination of Figure 32 shows there is a small amount of variation. This variation can be directly attributed to the difficulty in determining the exact break point and the exact touch down point.

The results of this study may be compared to Galvin's work to see if there are scale effects. Galvin's averaged constant of 3 (Equation 33) was obtained in the laboratory with a maximum wave height of 0.58 feet. In a later study Galvin (1968B) again performed a laboratory experiment on composite beach slopes where the maximum wave height was 2.50 feet and obtained an average constant of 4.4. The Virginia Beach study's maximum wave height was 5. feet, the averaged constant was 5.9. The increase of both scale and constant in the three studies suggests that as the wave height increased so will the constant.

Figure 32. A comparison of the observed plunge distance to the plunge distance calculated with the observed plunge time and the observed wave celerity. The observed values for plunge distance and wave celerity were obtained directly from the filmed data.



CHAPTER VIII

CONCLUSIONS

The validity of five laboratory developed formulas involving wave behavior in the shoaling zone prior to and during breaking have been tested using field data. The conclusions derived from this investigation are the following:

1. The breaking wave ratio H_b/d_b was found to be a poor criteria when tested by the field data. The mean (0.79) of the ratio falls close to the expected value of 0.78 but the wide standard deviation of 0.36 restricts the use of H_b/d_b as a breaking indicator. Of the three ratios investigated η/d_b , H_b/d_b , H_b/d_t , the most mathematically accurate indicator was η/d_b . Although η/d_b was the most mathematically accurate, when used to predict the exact break point, the classical ratio H_b/d_b may be used to define the breaking zone rather than the exact break point.
2. The prediction of the breaking wave height by the use of Komar's formula $H_b = g^{1/5} (H_o^2 T)^{2/5}$ produces remarkable agreement with laboratory and field measurements of others. The empirical relationship appears to hold for breakers ranging from 0.16 ft to 11.4 ft. However, since H_o is calculated from

shallow water parameters additional investigation is required to modify the relationship to account for refraction effects and frictional losses.

3. The onshore and offshore classification developed by Galvin in the laboratory did not provide conclusive results when applied to field conditions. Only the plunging wave proved to have a definable range.
4. The plunge distance was elevated unsuccessfully using a simplified physics equation for the trajectory of a particle acting solely under gravitational force. The failure of the predictive equations was determined to be due to the oversimplification of the plunge time. This oversimplification failed to account for the acceleration of the water particles beneath the wave crest, which decreased the predicted plunge time and, therefore, lead to the under prediction of the plunge distance. The best fit line through the field data was $P = 5.6H_b$, determined by the method of least squares, with a determined error $r = 13.9\%$.
5. The plunging wave at the break point was found to travel with 80% of its' wave height above mean water level, while the spilling waves did not show any conclusive grouping. It was not possible to determine conclusively that only a shallow water plunging waves at the break point demonstrated this

wave height to M.W.L. relationship. Further investigation should show that this will occur only in the shoaling wave approaching the break point.

TABLE IV
DEFINITION OF VARIABLES

- B_x = The horizontal distance between the bore collapse point and the intersection of still water level and the beach.
- C = The wave celerity.
- C_b = Celerity of the wave crest just prior to the break point (ft/sec) measured by counting the film frames required for the wave to travel 6 ft before breaking.
- C_o = The deep water wave celerity.
- E = Energy
- E_b = The energy at the break point.
- E_o = Wave energy in deep water; wave energy at breaking.
- F = The horizontal distance between the position of the preceding trough X_t and the break point X_b ; (ft).
- G = The vertical distance between the position of the water surface at X_t and M.W.L.; (ft).
- H_b = The total wave height. The vertical distance between the wave crest and preceding trough at breaking; (ft).
- K = Constant
- L = Wave Length
- L_o = Deep Water
- P = The horizontal distance traveled by the plunging wave during breaking; (ft).
- R_x = The horizontal run up distance relative to the origin placed at the intersection of M.W.L. and shore; (ft).

- R_z = The vertical run up distance relative to the intersection of M.W.L. and the shoreline; (ft).
- S^1 = The rate of swash advance (ft/sec) measured by counting frames required for the swash advance.
- S_x = Horizontal distance to swash edge from the intersection of M.W.L. and shoreline, when the following wave is at its break point; (ft).
- S_z = The vertical distance to swash edge from the intersection of M.W.L. and shoreline, when the following wave is at its break point; (ft).
- T_b = The elapsed time between two consecutive breakers reaching their break points; (sec).
- T_{64} = The period of the wave at the 64 ft mark, measured from the first pipe anchored in the beach; (sec).
- V = Velocity
- $V_{y\theta}$ = The velocity along the Y axis at the origin.
- X_b = The position of the wave crest at its break point relative to the intersection of M.W.L. and the shoreline; (ft).
- X_t = The position of the preceding trough relative to the intersection of M.W.L. and the shoreline when the wave crest is at its break point; (ft).
- Z = The tangent of the breaker backface angle formed by the wave crest to the position where the backface measured six horizontal feet from the crest.
- a_y = The acceleration along the Y axis.
- d_b = The depth from M.W.L. to the sand bottom beneath the wave at its break point; (ft).
- d_s = The vertical distance of the water's surface at the intersection of M.W.L. and the shoreline when the wave is at the break point; (ft).
- d_t = The depth beneath the preceding trough when the wave is at the break point; (ft).
- g = The gravitational acceleration.
- h_b = The depth from M.W.L. to the bottom beneath the breaking wave crest; (ft).

m = The beach slope beneath the break point of the wave.

n_b = Transmission coefficient for the breaking wave equal to 1.0.

n_o = Transmission coefficient for deep water wave equal to $\frac{1}{2}$.

t_c = The time elapsed from the bore formation until the bore encounters the shoreline; (sec).

t_{cs} = The swash time $t_c - t_s$; (sec).

t_p = The time between when the wave passes the break point until the crest contacts the water surface; (sec).

t_s = The time measured when the swash reaches its furthest horizontal extent on the beach; (sec).

β = d/H

b = d_b/H_b

η = The vertical distance between the wave crest and M.W.L.; (ft).

α = Relative wave height H_b/d_b .

APPENDIX A
 TABULATED VALUES FOR THE RATIO H_b/d_b

| Wave Number | Measured | Galvin | Weggle | Camfield & Street |
|-------------|----------|--------|--------|-------------------|
| 1 | 0.0 | 1.09 | 0.72 | 0.75 |
| 2 | 1.03 | 0.94 | 1.17 | 4.01 |
| 3 | 1.11 | 0.84 | .99 | 2.20 |
| 4 | 0.83 | 1.09 | .89 | 1.50 |
| 5 | 1.21 | 1.09 | 1.11 | 3.27 |
| 6 | 2.5 | 0.89 | 1.22 | 4.91 |
| 7 | 1.16 | 1.09 | .94 | 1.81 |
| 8 | 0.92 | 0.94 | 1.11 | 3.27 |
| 9 | 0.63 | 1.09 | .99 | 2.20 |
| 10 | 0.42 | 0.94 | 1.17 | 4.01 |
| 11 | 0.56 | 0.84 | .99 | 2.20 |
| 12 | 0.79 | 0.94 | .88 | 1.50 |
| 13 | 0.63 | 0.94 | .99 | 2.20 |
| 14 | 0.87 | 0.89 | .99 | 2.20 |
| 15 | 0.72 | 0.79 | .94 | 1.81 |
| 16 | 0.37 | 1.01 | .88 | 1.50 |
| 17 | 0.78 | 0.79 | 1.05 | 2.68 |
| 18 | 0.88 | 0.94 | 0.83 | 1.23 |
| 19 | 0.66 | 1.01 | .99 | 2.20 |
| 20 | 0.96 | 0.89 | 1.05 | 2.68 |
| 21 | 0.96 | 0.84 | 0.94 | 1.81 |

| Wave Number | Measured | Galvin | Weggle | Camfield & Street |
|-------------|----------|--------|--------|-------------------|
| 22 | 1.13 | 1.09 | 0.88 | 1.50 |
| 23 | 0.52 | 0.94 | 1.11 | 3.27 |
| 24 | 1.06 | 1.01 | 0.99 | 2.20 |
| 25 | 0.87 | 0.75 | 1.05 | 2.68 |
| 26 | 0.88 | 0.94 | 0.77 | 0.99 |
| 27 | 0.81 | 0.94 | 0.99 | 2.20 |
| 28 | 0.76 | 1.01 | 0.99 | 2.20 |
| 29 | 0.89 | 0.75 | 1.05 | 2.68 |
| 30 | 0.69 | 1.09 | 0.77 | 0.99 |
| 31 | 0.95 | 0.75 | 1.11 | 3.27 |
| 32 | 0.85 | 0.95 | 0.77 | 0.99 |
| 33 | 0.65 | 1.09 | 0.99 | 2.20 |
| 34 | 0.86 | 0.84 | 1.11 | 3.27 |
| 35 | 0.58 | 1.01 | 0.88 | 1.50 |
| 36 | 0.57 | 1.09 | 1.05 | 2.68 |
| 37 | 0.92 | 0.94 | 1.16 | 4.01 |
| 38 | 1.0 | 0.94 | 0.99 | 2.20 |
| 39 | 1.0 | 0.94 | 0.99 | 2.20 |
| 40 | 0.92 | 0.71 | 0.99 | 2.20 |
| 41 | 0.60 | 0.79 | 0.72 | 0.75 |
| 42 | 0.53 | 1.09 | 0.83 | 1.23 |
| 43 | 0.86 | 1.01 | 1.11 | 3.27 |
| 44 | 0.36 | 0.94 | 1.05 | 2.68 |
| 45 | 0.68 | 0.94 | 0.99 | 2.20 |
| 46 | 0.96 | 1.01 | 0.99 | 2.20 |
| 47 | 0.87 | 0.71 | .05 | 2.68 |

| Wave Number | Measured | Galvin | Weggle | Camfield & Street |
|-------------|----------|--------|--------|-------------------|
| 48 | 0.97 | 1.09 | 0.72 | 0.75 |
| 49 | 1.14 | 0.94 | 1.16 | 4.01 |
| 50 | 1.03 | 0.75 | 0.99 | 2.20 |
| 51 | 1.24 | 0.83 | 0.77 | 0.99 |
| 52 | 0.60 | 1.01 | 0.88 | 1.50 |
| 53 | 0.94 | 0.75 | 1.05 | 2.68 |
| 54 | 0.90 | 0.89 | 0.77 | 0.99 |
| 55 | 0.36 | 0.94 | 0.94 | 1.81 |
| 56 | 0.92 | 0.75 | 0.99 | 2.20 |
| 57 | 0.90 | 0.75 | 0.77 | 0.99 |
| 58 | 0.77 | 0.71 | 0.77 | 0.99 |
| 59 | 0.97 | 0.94 | 0.72 | 0.75 |
| 60 | 0.52 | 0.88 | 0.99 | 2.20 |
| 61 | 0.46 | 1.08 | 0.94 | 1.81 |
| 62 | 0.72 | 0.94 | 1.16 | 4.01 |
| 63 | 1.19 | 0.75 | 0.99 | 2.20 |
| 64 | 0.97 | 0.84 | 0.77 | 0.99 |
| 65 | 0.41 | 0.94 | 0.88 | 1.50 |
| 66 | 0.73 | 1.09 | 0.99 | 2.20 |
| 67 | 0.93 | 0.79 | 1.16 | 4.01 |
| 68 | 0.76 | 0.89 | 0.83 | 1.23 |
| 69 | 0.75 | 0.84 | 0.94 | 1.81 |
| 70 | 0.95 | 0.88 | 0.88 | 1.50 |
| 71 | 0.62 | 0.71 | 0.94 | 1.81 |
| 72 | 0.79 | 0.94 | 0.72 | 0.75 |
| 73 | 0.87 | 1.01 | 0.99 | 2.20 |
| 74 | 1.32 | 0.94 | 1.05 | 2.68 |

| Wave Number | Measured | Galvin | Weggle | Camfield & Street |
|-------------|----------|--------|--------|-------------------|
| 75 | 1.21 | 1.09 | 0.99 | 2.20 |
| 76 | 0.43 | 1.09 | 1.11 | 3.27 |
| 77 | 0.30 | 0.94 | 1.16 | 4.01 |
| 78 | 1.20 | 1.09 | 0.99 | 2.20 |
| 79 | 0.47 | 1.08 | 1.11 | 3.27 |
| 80 | 0.62 | 1.08 | 1.11 | 3.27 |
| 81 | 0.74 | 0.94 | 1.11 | 3.27 |
| 82 | 0.36 | 0.75 | 0.99 | 2.20 |
| 83 | 0.85 | 1.01 | 0.77 | 0.99 |
| 84 | 2.50 | 0.83 | 1.05 | 2.68 |
| 85 | 0.91 | 0.84 | 0.88 | 1.50 |
| 86 | 0.86 | 1.09 | 0.88 | 1.50 |
| 87 | 0.60 | 0.94 | 1.16 | 4.01 |
| 88 | 0.0 | 1.09 | 0.99 | 2.20 |
| 89 | 1.0 | 1.09 | 1.16 | 4.01 |
| 90 | 0.60 | 1.01 | 1.16 | 4.01 |
| 91 | 0.07 | 0.71 | 1.05 | 2.68 |
| 92 | 0.74 | 0.79 | 0.72 | 0.75 |
| 93 | 0.47 | 0.94 | 0.83 | 1.23 |
| 94 | 0.88 | 1.09 | 0.99 | 2.20 |
| 95 | 0.53 | 0.71 | 1.16 | 4.01 |
| 96 | 1.92 | 0.94 | 0.72 | 0.75 |
| 97 | 0.62 | 0.94 | 0.99 | 2.20 |
| 98 | 0.34 | 0.83 | 0.99 | 2.20 |
| 99 | 0.50 | 0.83 | 0.88 | 1.50 |
| 100 | 0.50 | 0.94 | 0.88 | 1.50 |

| Wave Number | Measured | Galvin | Weggle | Camfield & Street |
|-------------|----------|--------|--------|-------------------|
| 101 | 0.57 | 0.75 | 0.99 | 2.20 |
| 102 | 0.22 | 1.01 | 0.77 | 0.99 |
| 103 | 0.43 | 1.01 | 1.05 | 2.68 |
| 104 | 0.75 | 0.94 | 1.05 | 2.68 |
| 105 | 0.80 | 0.84 | 0.99 | 2.20 |
| 106 | 0.39 | 0.94 | 0.88 | 1.50 |
| 107 | 0.68 | 0.71 | 0.99 | 2.20 |
| 108 | 0.88 | 0.79 | 0.72 | 0.75 |
| 109 | 0.61 | 0.79 | 0.83 | 1.23 |
| 110 | 0.78 | 0.75 | 0.83 | 1.23 |
| 111 | 0.81 | 0.79 | 0.77 | 0.99 |
| 112 | 0.32 | 1.08 | 0.83 | 1.23 |
| 113 | 0.33 | 0.94 | 1.16 | 4.01 |
| 114 | 0.96 | 1.01 | 0.99 | 2.20 |
| 115 | 0.69 | 0.75 | 1.05 | 2.68 |
| 116 | 0.68 | 1.54 | 0.77 | 0.99 |
| 117 | 1.00 | 1.01 | 1.33 | 7.29 |
| 118 | 0.89 | 0.75 | 1.05 | 2.68 |
| 119 | 0.55 | 1.08 | 0.77 | 0.99 |
| 120 | 0.26 | 1.08 | 1.11 | 3.27 |

APPENDIX B

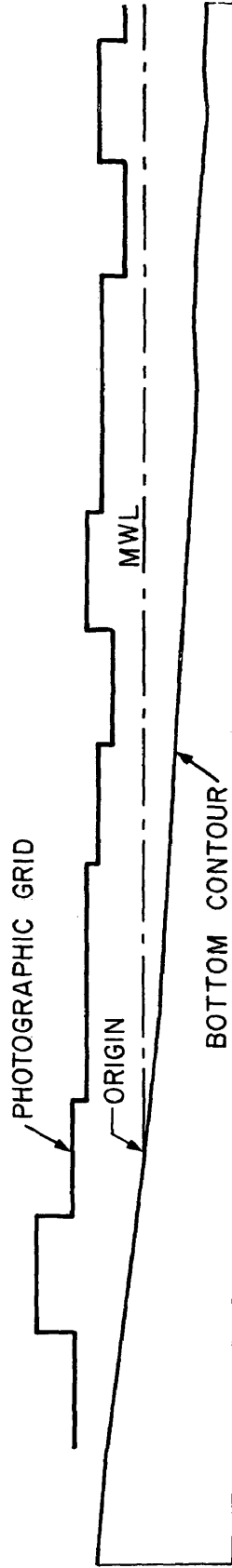
Tabulated data of the wave parameters that were obtained directly from the film taken at Virginia Beach. A complete definition of characters and symbols is contained in Chapter II, and in Table IV.

| Wave # | T _b | T ₆₄ | X ₀ | X _c | F | G | d _b | m | Z | d _c | η | H _b | C _b | d _s | S _x | S _z | S ⁱ | t _s | R _x | R _z | t _c | t _{cs} | B _x | P |
|--------|----------------|-----------------|----------------|----------------|------|------|----------------|-------|-------|----------------|-----|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|-----------------|----------------|------|
| 1 | ---- | 8.78 | 68.3 | 32.3 | 36.0 | -0.3 | -4.3 | 0.00 | 0.250 | 2.1 | 2.3 | 2.6 | 10.74 | 1.4 | -3.7 | 1.4 | 6.27 | 7.26 | -20.7 | 2.6 | 3.51 | 3.75 | -1.7 | 6 |
| 2 | 5.03 | 7.42 | 88.3 | 64.3 | 24.0 | -0.1 | -4.9 | -0.02 | 0.083 | 4.2 | 2.5 | 2.6 | 12.53 | 1.3 | -12.7 | 2.1 | 7.16 | 10.37 | -13.7 | 3.1 | 7.18 | 3.19 | +2.3 | |
| 3 | 11.03 | 0.72 | 26.3 | 22.3 | 14.0 | -0.3 | -2.3 | 0.07 | 0.250 | 1.8 | 1.7 | 2.0 | 9.40 | 0.9 | +0.3 | 1.2 | -6.27 | 2.79 | -21.7 | 2.2 | 3.83 | 3.11 | -5.7 | 28 |
| 4 | 0.24 | 9.82 | 46.3 | 37.3 | 9.0 | +1.0 | -3.3 | 0.06 | 0.167 | 3.6 | 2.2 | 1.2 | 18.80 | 1.3 | -16.7 | 2.3 | -9.40 | 9.42 | -14.7 | 2.6 | 3.99 | 3.67 | +4.3 | 19 |
| 5 | 9.18 | 4.71 | 54.3 | 39.3 | 15.0 | -0.3 | -3.8 | 0.05 | 0.083 | 2.5 | 2.3 | 2.6 | 18.80 | 1.1 | +5.3 | 0.5 | -5.01 | 7.66 | -13.7 | 2.5 | 3.75 | 2.71 | -9.7 | 29 |
| 6 | 6.62 | 11.09 | 37.3 | 32.3 | 5.0 | -0.7 | -2.6 | 0.05 | 0.167 | 1.7 | 1.8 | 2.5 | 12.53 | 1.3 | -14.7 | 2.5 | 3.13 | 5.99 | -17.7 | 2.5 | 3.75 | 2.71 | -4.7 | 10 |
| 7 | 11.17 | 8.78 | 46.3 | 20.3 | 26.0 | -0.4 | -3.3 | 0.06 | 0.200 | 1.4 | 2.5 | 2.9 | 12.53 | 0.4 | +12.3 | 0.3 | 8.35 | 6.06 | -28.7 | 3.6 | 3.99 | 3.67 | +4.3 | 19 |
| 8 | 7.34 | 3.75 | 68.3 | 34.3 | 34.0 | -0.7 | -4.3 | 0.00 | 0.250 | 1.7 | 1.9 | 1.6 | 7.52 | 1.1 | -17.7 | 2.4 | -5.01 | 8.94 | -17.7 | 2.5 | 4.79 | 4.15 | +4.3 | 12 |
| 9 | 8.14 | 7.82 | 14.3 | 10.3 | 4.0 | +0.3 | -1.4 | 0.08 | 0.083 | 2.7 | 1.9 | 1.6 | 7.52 | 1.1 | -17.7 | 2.4 | -5.01 | 3.83 | -3.7 | 1.4 | 3.03 | 0.80 | -1.7 | 8 |
| 10 | 8.75 | 11.09 | 36.3 | 30.3 | 6.0 | -0.5 | -2.6 | 0.05 | 0.083 | 1.8 | 2.2 | 2.1 | 12.53 | 1.4 | -7.7 | 1.3 | -6.27 | 5.83 | -19.7 | 2.7 | 2.47 | 2.66 | +0.3 | 22 |
| 11 | 8.94 | 5.67 | 70.3 | 50.3 | 20.0 | -0.7 | -4.1 | 0.01 | 0.217 | 2.6 | 4.4 | 5.1 | 12.53 | 1.4 | -5.7 | 1.2 | -6.27 | 9.18 | -20.7 | 2.8 | 5.51 | 3.67 | -1.7 | 14 |
| 12 | 8.77 | 7.02 | 28.3 | 20.3 | 8.0 | +0.5 | -2.3 | 0.03 | 0.033 | 2.4 | 1.9 | 1.4 | 12.53 | 1.4 | -19.7 | 2.7 | -5.01 | 5.03 | -9.7 | 1.6 | 2.87 | 2.16 | +4.7 | 14 |
| 13 | 5.91 | 5.51 | 42.3 | 35.3 | 7.0 | -0.6 | -2.8 | 0.06 | 0.083 | 1.9 | 2.1 | 2.7 | 12.53 | 0.9 | -9.7 | 1.3 | -8.35 | 6.14 | -17.7 | 2.5 | 3.24 | 2.87 | +2.3 | 15 |
| 14 | 3.75 | 9.42 | 67.3 | 46.3 | 21.0 | -0.3 | -4.4 | 0.01 | 0.250 | 2.2 | 3.7 | 4.0 | 15.04 | 1.3 | -1.7 | 0.7 | 8.35 | 8.14 | -23.7 | 3.0 | 5.19 | 2.95 | +9.7 | 14 |
| 15 | 8.46 | 2.87 | 76.3 | 58.3 | 18.0 | +1.2 | -4.4 | 0.04 | 0.283 | 3.7 | 2.8 | 1.6 | 12.53 | 1.4 | -29.7 | 3.7 | -6.27 | 8.54 | +0.3 | 1.1 | 5.99 | 2.55 | +24.6 | 15 |
| 16 | 5.90 | 5.83 | 18.3 | 30.3 | 8.0 | -0.5 | -2.7 | 0.05 | 0.083 | 1.8 | 2.0 | 2.5 | 15.04 | 0.3 | +6.3 | -0.6 | 3.58 | 5.99 | -21.7 | 2.7 | 3.51 | 2.48 | +3.3 | 16 |
| 17 | 3.91 | 7.34 | 64.3 | 54.3 | 10.0 | -0.8 | -4.4 | 0.01 | 0.167 | 1.7 | 3.2 | 4.0 | 15.04 | 0.4 | -9.7 | 1.8 | 6.27 | 7.42 | -9.7 | 2.0 | 5.91 | 1.51 | -3.7 | 19 |
| 18 | 7.43 | 6.06 | 64.3 | 54.3 | 10.0 | 0.0 | -4.4 | 0.01 | 0.300 | 2.5 | 3.4 | 3.4 | 15.04 | 1.3 | -11.7 | 1.9 | 3.13 | 8.22 | -17.7 | 2.5 | 5.11 | 3.11 | -5.7 | 15 |
| 19 | 5.74 | 8.30 | 69.3 | 46.3 | 23.0 | -0.6 | -4.2 | 0.00 | 0.183 | 2.6 | 3.5 | 4.1 | 18.80 | 1.4 | -10.7 | 2.0 | 3.13 | 8.38 | -21.7 | 2.4 | 5.35 | 3.03 | -7.7 | 25 |
| 20 | 6.78 | 7.50 | 81.3 | 45.3 | 36.0 | -0.7 | -4.6 | 0.05 | 0.133 | 2.4 | 1.7 | 2.4 | 15.04 | 1.4 | -19.7 | 2.6 | 2.78 | 8.86 | -13.7 | 2.1 | 6.54 | 2.32 | +0.3 | 13 |
| 21 | 7.66 | 9.98 | 70.3 | 50.3 | 16.0 | +0.1 | -4.3 | 0.04 | 0.133 | 2.2 | 2.4 | 2.0 | 15.04 | 1.4 | -7.7 | 1.5 | 6.27 | 10.85 | -18.7 | 2.6 | 5.44 | 4.31 | +2.7 | 7 |
| 22 | 11.97 | 6.46 | 50.3 | 30.3 | 20.0 | -0.7 | -3.3 | 0.08 | 0.150 | 1.6 | 1.7 | 2.4 | 10.74 | 1.2 | +8.3 | 0.4 | -6.27 | 8.62 | -22.7 | 2.9 | 3.83 | 4.79 | +2.3 | 14 |
| 23 | 7.83 | 5.83 | 36.3 | 28.3 | 8.0 | -0.7 | -2.6 | 0.05 | 0.083 | 1.6 | 2.4 | 3.1 | 12.53 | 1.3 | -21.7 | 2.8 | 3.58 | 5.99 | -9.7 | 1.7 | 4.07 | 1.92 | +5.7 | 21 |
| 24 | 3.67 | 6.78 | 70.3 | 54.3 | 16.0 | -0.6 | -4.1 | 0.01 | 0.167 | 3.2 | 3.4 | 4.0 | 15.04 | 1.4 | -5.7 | 1.7 | 2.09 | 7.82 | -11.7 | 2.0 | 5.43 | 2.39 | +0.3 | 20 |
| 25 | 4.78 | 6.06 | 87.3 | 64.3 | 23.0 | +0.3 | -4.8 | 0.03 | 0.20 | 2.8 | 2.3 | 2.0 | 15.04 | 1.2 | +6.3 | 0.9 | 6.27 | 9.82 | -11.7 | 2.0 | 7.26 | 2.56 | +2.3 | 13 |
| 26 | 8.18 | 7.50 | 72.3 | 56.3 | 19.0 | +0.1 | -4.3 | 0.00 | 0.133 | 1.9 | 1.3 | 2.8 | 15.04 | 1.3 | -10.7 | 1.9 | 3.58 | 6.87 | -17.7 | 2.1 | 5.83 | 2.49 | -7.7 | 17 |
| 27 | 7.82 | 6.46 | 48.3 | 32.3 | 16.0 | -0.4 | -3.3 | 0.08 | 0.167 | 2.1 | 2.7 | 3.1 | 15.04 | 0.9 | -17.7 | 2.5 | -5.01 | 6.14 | -25.7 | 3.2 | 3.35 | 2.79 | -9.7 | 33 |
| 28 | 5.43 | 7.42 | 62.3 | 35.3 | 27.0 | -0.8 | -4.2 | 0.02 | 0.167 | 1.7 | 2.4 | 3.2 | 18.80 | 1.3 | -23.7 | 3.2 | 2.28 | 6.86 | -12.7 | 2.1 | 4.63 | 2.23 | -2.7 | 29 |
| 29 | 6.06 | 4.71 | 80.3 | 45.3 | 35.0 | -1.0 | -4.4 | 0.04 | 0.183 | 2.1 | 2.3 | 3.3 | 15.04 | 1.4 | -11.7 | 1.8 | 4.18 | 8.94 | -19.7 | 2.6 | 6.06 | 2.88 | +0.3 | 14 |
| 30 | 9.02 | 6.54 | 75.3 | 55.3 | 20.0 | -0.8 | -4.4 | 0.03 | 0.167 | 3.2 | 3.4 | 4.2 | 18.80 | 1.4 | -19.7 | 2.2 | 1.67 | 8.38 | -19.7 | 2.6 | 5.11 | 3.27 | +3.7 | 24 |
| 31 | 1.20 | 2.55 | 19.3 | 16.3 | 3.0 | -0.0 | -2.4 | 0.04 | 0.217 | 1.6 | 1.5 | 1.5 | 7.52 | 1.3 | -19.7 | 2.2 | -8.35 | 2.87 | +5.3 | 1.0 | 2.31 | 0.56 | +6.3 | 3 |
| 32 | 8.18 | 7.50 | 72.3 | 56.3 | 19.0 | +0.1 | -4.3 | 0.00 | 0.133 | 1.9 | 1.3 | 2.8 | 15.04 | 1.3 | -10.7 | 1.9 | 3.58 | 7.90 | -17.7 | 2.1 | 5.83 | 2.49 | -7.7 | 20 |
| 33 | 9.02 | 7.98 | 44.3 | 38.3 | 16.0 | -0.4 | -3.1 | 0.05 | 0.317 | 1.9 | 2.3 | 2.7 | 15.04 | 1.0 | -1.7 | 0.7 | -8.35 | 6.62 | -22.7 | 2.5 | 3.27 | 3.35 | -1.7 | 14 |
| 34 | 7.98 | 6.38 | 42.3 | 33.3 | 9.0 | -0.7 | -2.8 | 0.06 | 0.183 | 1.7 | 3.0 | 3.7 | 15.04 | 1.3 | -6.3 | 0.5 | -8.35 | 6.46 | -10.7 | 1.8 | 3.91 | 2.55 | -1.7 | 14 |
| 35 | 6.31 | 5.43 | 45.3 | 33.3 | 12.0 | -0.4 | -3.2 | 0.05 | 0.333 | 2.0 | 3.5 | 3.9 | 12.53 | 1.2 | -11.7 | 1.8 | -6.27 | 7.22 | -18.7 | 1.8 | 3.83 | 2.39 | -7.7 | 27 |
| 36 | 7.02 | 1.92 | 26.3 | 22.3 | 4.0 | +0.3 | -2.3 | 0.07 | 0.10 | 2.4 | 1.3 | 1.0 | 9.40 | 1.3 | -18.7 | 2.2 | -6.27 | 4.31 | +0.7 | 0.4 | 4.03 | 0.88 | +0.3 | 8 |
| 37 | 7.27 | 3.75 | 12.3 | 28.3 | 19.0 | +0.3 | -1.3 | 0.08 | 0.033 | 1.0 | 0.7 | 0.4 | 9.40 | 0.9 | +1.3 | 1.5 | 5.01 | 3.03 | -11.7 | 2.0 | 1.66 | 1.67 | +0.3 | 7 |
| 38 | 2.15 | 9.18 | 36.3 | 26.3 | 8.0 | -0.6 | -2.5 | 0.05 | 0.167 | 1.7 | 2.6 | 3.2 | 15.04 | 1.3 | -7.7 | 1.6 | 8.35 | 3.51 | -24.7 | 3.3 | 3.27 | 2.24 | -11.7 | 18 |
| 39 | 9.98 | 3.19 | 22.3 | 16.3 | 6.0 | -0.6 | -2.1 | 0.07 | 0.083 | 1.0 | 0.4 | 1.0 | 7.52 | 0.4 | +16.3 | -0.6 | -8.35 | 4.15 | -15.7 | 1.9 | 1.76 | 2.39 | +9.3 | 5 |
| 40 | 4.95 | 6.38 | 16.3 | 10.3 | 6.0 | +0.3 | -1.6 | 0.07 | 0.167 | 2.4 | 1.3 | 1.0 | 10.74 | 1.4 | -15.7 | 2.4 | -5.01 | 4.23 | -20.7 | 2.6 | 2.29 | 1.84 | -9.7 | 13 |
| 41 | 3.43 | 10.93 | 56.3 | 38.3 | 18.0 | -0.8 | -3.9 | 0.07 | 0.317 | 1.9 | 2.1 | 2.9 | 15.04 | 1.4 | -17.7 | 2.6 | 2.78 | 7.26 | -6.7 | 1.2 | 4.71 | 2.55 | +4.3 | 19 |
| 42 | 12.85 | 8.78 | 34.3 | 36.3 | 19.0 | +0.1 | -4.3 | 0.05 | 0.083 | 1.9 | 2.1 | 2.9 | 15.04 | 1.4 | -17.7 | 2.6 | 2.78 | 7.26 | -6.7 | 1.2 | 4.71 | 2.55 | +4.3 | 19 |
| 43 | 8.63 | 7.32 | 32.3 | 34.3 | 18.0 | -0.3 | -3.8 | 0.05 | 0.267 | 1.9 | 3.7 | 4.6 | 15.04 | 1.3 | -12.7 | 2.5 | 3.13 | 5.99 | -8.7 | 1.4 | 3.47 | 2.40 | -7.7 | 16 |
| 44 | 8.38 | 4.31 | 40.3 | 30.3 | 10.0 | +0.5 | -2.8 | 0.06 | 0.033 | 2.8 | 1.5 | 2.0 | 10.74 | 1.4 | -25.7 | 3.5 | 8.35 | 7.72 | +0.7 | 0.4 | 4.49 | 2.95 | +12.3 | 14 |
| 45 | 6.14 | ---- | 30.3 | 23.3 | 7.0 | -0.7 | -2.3 | 0.03 | 0.050 | 1.4 | 1.4 | 2.1 | 15.04 | 0.4 | +3.3 | 0.4 | 5.61 | 4.71 | +29.7 | 3.8 | 2.42 | 2.24 | -7.7 | 18 |
| 46 | 5.51 | 5.84 | 26.8 | 15.3 | 11.5 | 0.0 | -2.3 | 0.03 | 0.216 | 1.5 | 2.0 | 2.0 | 12.53 | 1.0 | -9.2 | 1.5 | 1.57 | 7.02 | -18.7 | 2.5 | 2.95 | 3.11 | -1.7 | 11.5 |
| 47 | 10.30 | 4.72 | 48.3 | 31.3 | 17.0 | -0.5 | -3.3 | 0.08 | 0.083 | 0.9 | 1.5 | 2.0 | 12.53 | 0.8 | -24.7 | 2.5 | 11.39 | 6.86 | -6.7 | 1.3 | 3.35 | 3.51 | -8.3 | 2.0 |
| 48 | 3.51 | 5.28 | 59.3 | 49.3 | 10.0 | +1.5 | -4.1 | -0.85 | 0.267 | 4.6 | 3.5 | 4.6 | 17.53 | 0.5 | -6.7 | 0.4 | -6.27 | 7.90 | -8.7 | 2.9 | 5.83 | 2.49 | -7.7 | 15 |
| 49 | 8.63 | 7.32 | 32.3 | 34.3 | 18.0 | -0.3 | -3.8 | 0.05 | 0.083 | 1.9 | 2.1 | 1.3 | 7.52 | 0.7 | -7.7 | 1.0 | 4.5f | +15 | -13.3 | 3.0 | 3.07 | 0.06 | -6.7 | 10 |
| 50 | 3.60 | 3.60 | 48.3 | 36.3 | 12.0 | +1.0 | -3.3 | 0.08 | 0.166 | 2.7 | 3.0 | 2.0 | 15.04 | 1.5 | -12.7 | 1.6 | 5.01 | 7.34 | -20.7 | 3.0 | 3.75 | 3.59 | -1.7 | 25 |
| 51 | 4.31 | 8.08 | 40.3 | 22.3 | 18.0 | +0.3 | -2.8 | 0.06 | 0.150 | 2.2 | 0.5 | 0.2 | 10.74 | 0.7 | -8.7 | 1.5 | 12.53 | 4.79 | -10.7 | 2.2 | 3.67 | 1.12 | -7.7 | 14 |
| 52 | 5.50 | 8.16 | 72.3 | 42.3 | 30.0 | -0.7 | -4.3 | 0.00 | 0.333 | 0.1 | 2.5 | 3.2 | 15.04 | 1.3 | -13.7 | 2.2 | 4.18 | 8.94 | -11.7 | 2.2 | 6.22 | 2.72 | -3.7 | 11 |
| 53 | 8.06 | 7.20 | 26.3 | 16.3 | 10.0 | +0.4 | -2.3 | 0.02 | 0.083 | 1.8 | 1.5 | 1.1 | 10.74 | 1.2 | -9.7 | 1.0 | 1.25 | 4.73 | -23.7 | 2.7 | 2.62 | 2.00 | -5.7 | 14 |
| 54 | 6.22 | 6.34 | 38.3 | 41.3 | 3.0 | -0.7 | -2.7 | 0.05 | 0.166 | 0.7 | 3.7 | 2.4 | 15.04 | 1.3 | -12.7 | 2.5 | 3.13 | 5.99 | -8.7 | 1.4 | 3.75 | 3.59 | +2.3 | 12 |
| 55 | 8.28 | 12.3 | 12.3 | 5.0 | +0.5 | -1.3 | 0.08 | 0.083 | 1.3 | 1.2 | 0.7 | 9.40 | 1.2 | -7.7 | 1.3 | 1.57 | 4.15 | -15.7 | 2.5 | 1.68 | 2.47 | -7.7 | 8 | |
| 56 | 2.72 | 10.48 | 72.3 | 38.3 | 34.0 | -0.7 | -4.3 | 0.00 | 0.250 | 0.1 | 1.8 | 2.5 | 15.04 | | | | | | | | | | | |

Figure 33. The photographic grid used during the field study at Virginia Beach. The drawing is to scale and shows the mean water level (M.W.L.), bottom contour, and the time of each photographic run.

SCALE
1/16" = 1'

TIME
10:48 EDT RUN 31
11:15 EDT RUN 32
11:45 EDT RUN 33



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