CHAPTER 27

FIELD STUDY OF BREAKING WAVE CHARACTERISTICS

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ABSTRACT

This study focuses upon four elements of breaking wave behavior:

1) Relative breaking depth criteria

2) Breaking wave classification

3) Evaluation of the plunge distance

4) Breaking wave height prediction

The data set is 116 waves filmed at Virginia Beach, Va., on the Atlantic U.S. coast. The cine-photographic observation technique permitted the viewer to freeze the free surface profile at successive time steps as the waves passed an upright plane grid placed perpendicular to the beach. The results indicate that:

1) While the average value of $H_b/d_b \simeq 0.78$, there was a significant difference between plunging and non-plunging waves.

2) Neither the breaker classification of Galvin nor that of Battjes successfully discriminated between plunging and spilling breakers.

3) The distance travelled by the foreface of a plunging wave was found to be underestimated by the free fall trajectory model advanced by Galvin. The field observations show the weakness to be in the plunge time arising from neglect of the vertical velocity component.

4) The breaking wave height prediction formulation advanced by Komar and Gaughan adequately predicts the breaking wave height within the constraints of calculating deep water wave characteristics, neglecting wave refraction and frictional effects. The combined data set covers the breaker wave height between laboratory scale observations to greater than 3 m.

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INTRODUCTION

The purpose of the study was to compare field observations of breaking wave characteristics with existing formulations, which for the most part have been derived from wave tank observations. Four elements of breaking wave behavior are addressed:

1) Relative breaking depth criteria

2) Breaking wave classification

3) Evaluation of the plunge distance

4) Breaking wave height prediction

METHODS. The field measurements were obtained from cine-photography of waves as they passed through an upright plane grid installed perpendicular to the beach. The field site was at Virginia Beach, Va. (Fig. 1), an ocean beach receiving waves generated in the North Atlantic Ocean. The observations were made in the summer when the wave climate is dominated by swell with characteristic average wave period of about 8 seconds generated by the Bermuda "high". Levelled rectangular grid sections, 3.3 m in length, were fixed to pipes jetted into the substrate. The square unit cell of the pipe grid was 61 cm (2 ft) on a side with each side further indexed to 30.5 cm (1 ft), (Fig. 2). The grids extended from the top of the foreshore to about 40 m offshore (Fig. 3).

Cine photography was achieved by mounting a motor-driven 16 mm Bolex camera with wide angle lens on the existing pier some 50 m from the grid (Fig. 4). Film advance rate was 12.53 frames/sec. Three film runs, obtained over a one hour period on 26 September 1968 constitute the data base for this report. The semi-diurnal tide has a mean range of 1.04 m. The observations were made about 2 hours after high water.

After each run the sediment surface was profiled relative to the grid. As no significant changes were observed between runs, a single profile was used in the analysis (Fig. 3). Mean water level relative to the grid was determined from a series of graduated transparent pipes (5 cm diameter) with constricted oriface (0.32 cm) which acted to filter high frequency fluctuations. The internal water level within the tubes was visually monitored.

The observation method allowed the viewer to freeze the free surface profile at successive time steps so the complete transformation of individual waves could be traced. Basic film data reduction was achieved using a Lafayette Analyst Time-Motion Projector with the image projected on a wide rear surface screen. Water surface elevation was estimated to 3 cm (0.1 ft) on the grid cell.

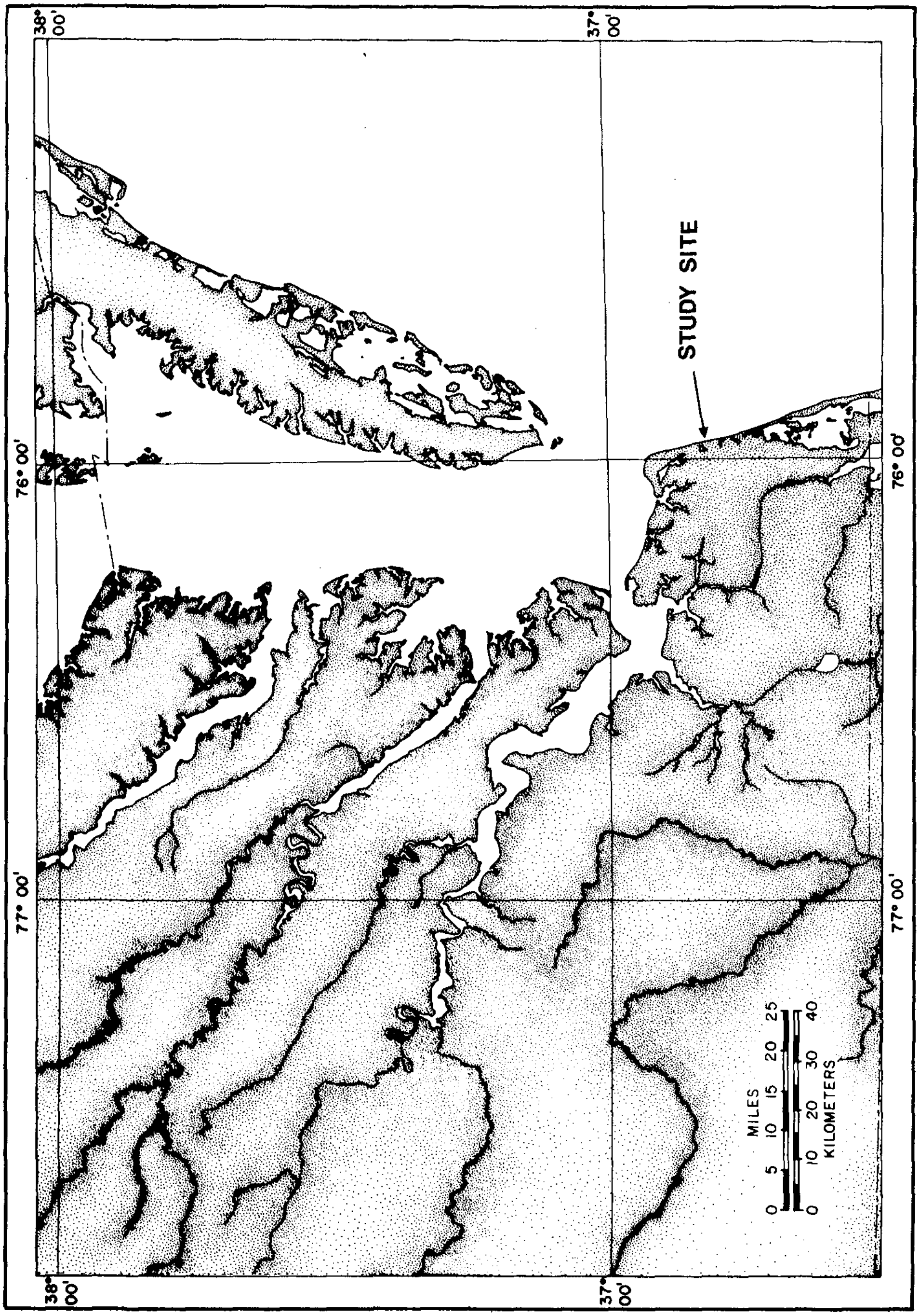

FIGURE 1. LOCATION MAP.

FIGURE 2. GRID IN PLACE.

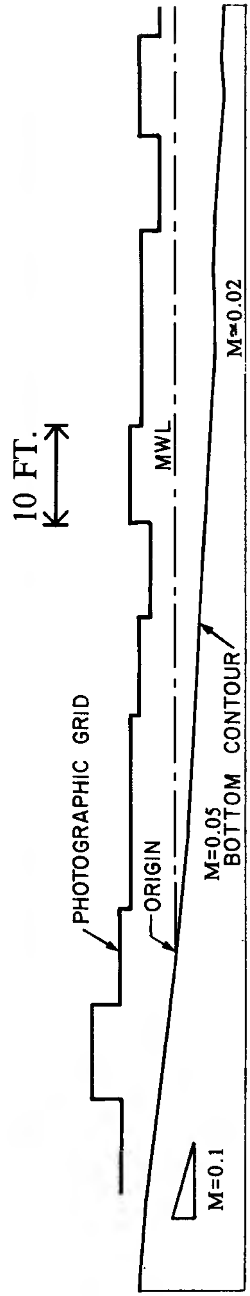

FIGURE 3. GRID LAYOUT DURING EXPERIMENT.

FIGURE 4. AERIAL VIEW OF PIER USED AS CAMERA PLATFORM.

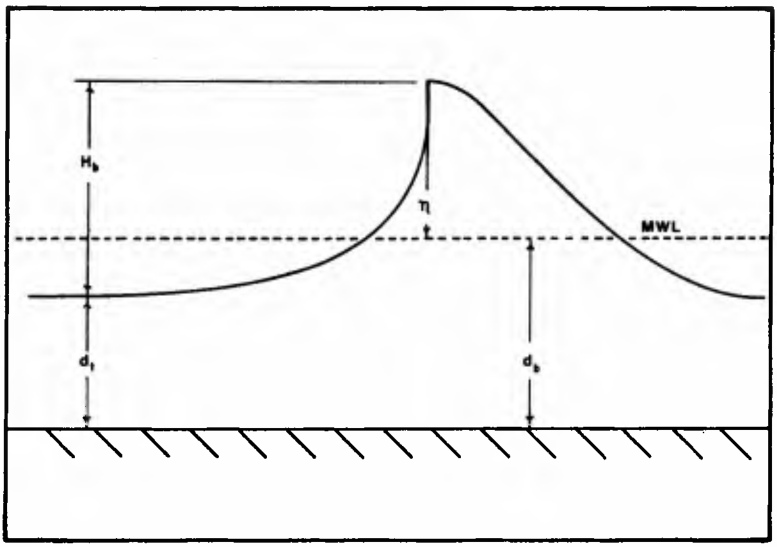

FIGURE 5. DEFINITION SKETCH FOR RELATIVE BREAKING DEPTH.

The observed waves were categorized according to breaker type. The criterion for breaking of "spilling" waves was the first appearance of foam cascading down the foreface. In the case of "plunging" waves, the criterion used was the position when initial overturning appeared with the formulation of a protruding lip or jet at the top of the near vertical foreface. While well-developed plunging waves and well-developed continuously spilling waves were distinct and easily separable, some breaker events were less clear. In some cases there was obvious interference between an incoming plunging wave and a strong backwash or cases when a wave was overtaking another while breaking. In a few cases the spilling crest would transform to a plunging crest. The 116 waves were distributed as follows: 70 well-developed plunging; 18 interference plunging; 21 well-developed spilling; and 7 spilling transformed to plunge.

For calculations involving breaker wave period, T_b , the observed period at a distance of 21 m from the mean water line was used. The period was considered to be the elapsed time between the passing of the wave in question and the passing of the prior wave. Wave celerity was considered to be the speed of travel over the 2 m distance prior to the inception of breaking. A data listing is given in Weishar (1976).

RELATIVE BREAKING DEPTH

The limiting wave height to depth ratio has been the subject of considerable study since the work of McCowan (1894). Galvin (1972) offers a complete review. McCowan derived a limiting value of η/d_b = 0.78 (see Fig. 5 for definition sketch) from solitary wave theory. The numerical value of 0.78 remains in common use although it is generally associated with the ratio, $\alpha = H_b/d_b$. Laboratory studies have demonstrated a dependency of α upon beach slope, m :

$$\frac{1}{\alpha} = 0.92 \quad \text{for } m \geq 0.07, \qquad = 1.40 - 6.85\,m \quad \text{for } m \leq 0.07 \qquad \text{Galvin (1969)} \tag{1}$$

$$\alpha = 0.724 + 5.6\,m\,, \qquad \text{Weggel (1972)} \tag{2}$$

$$\alpha = 0.75 + 25\,m - 112\,m^2 + 3870\,m^3\,, \quad \text{Camfield and Street (1969)} \tag{3}$$

RESULTS. Three ratios, η/d_b , H_b/d_b , and H_b/d_t (Fig. 5) were examined with the data set of 116 waves, which were segregated into plunging and non-plunging. The results are shown in Table 1.

TABLE 1

SUMMARY OF WAVE HEIGHT TO WATER DEPTH RATIOS

	Total Sample (116)		Plunging Waves (70)		Non-plunging Waves (46)	
Ratio	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. 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

Application of the Student "t" test indicates that the difference between the means of the plunging versus non-plunging waves is significant ($P < 0.01$) for the ratios H_b/d_b and H_b/d_t. In all cases the standard deviations are relatively large. It is of interest to note that the average value of H_b/d_b for the entire sample is very close to the value commonly used in engineering practice. The histograms for H_b/d_b are shown in Figure 6.

The observed values of H_b/d_b were also compared with the three relationships denoting slope dependency (Eqs. 1-3) using the local slope in the region of breaking. All the plots exhibited wide scatter as a result of the wide variation in H_b/d_b for a very small slope range. This data set is inappropriate to independently test for the effect of slope on relative breaking depth.

CLASSIFICATION OF BREAKER TYPE

Visual inspection of waves breaking in the nearshore or on the foreshore leaves even the casual observer with the impression that there are characteristic differences between the modes of breaking which range between a condition where the waves cascade foam down the foreface of the crest as they approach the foreshore to conditions where, on steeper slopes, the waves simply surge up the foreshore without "breaking". The qualitative dependence of wave breaker type on beach slope and wave steepness has been recognized for several decades. Perhaps the most important difference between breaker characteristics is the varying rates of wave energy dissipation as the wave interacts with the nearshore and beach morphology. Galvin (1968) formulated a quantitative classification of breaker type based upon laboratory studies which demonstrated a dependence upon beach slope, wave period, and deep water or breaking wave height. Battjes (1974), in the development of a general similarity parameter for breaking wave behavior, also presented a basis for breaker classification. Both authors formulated offshore and inshore parameters.

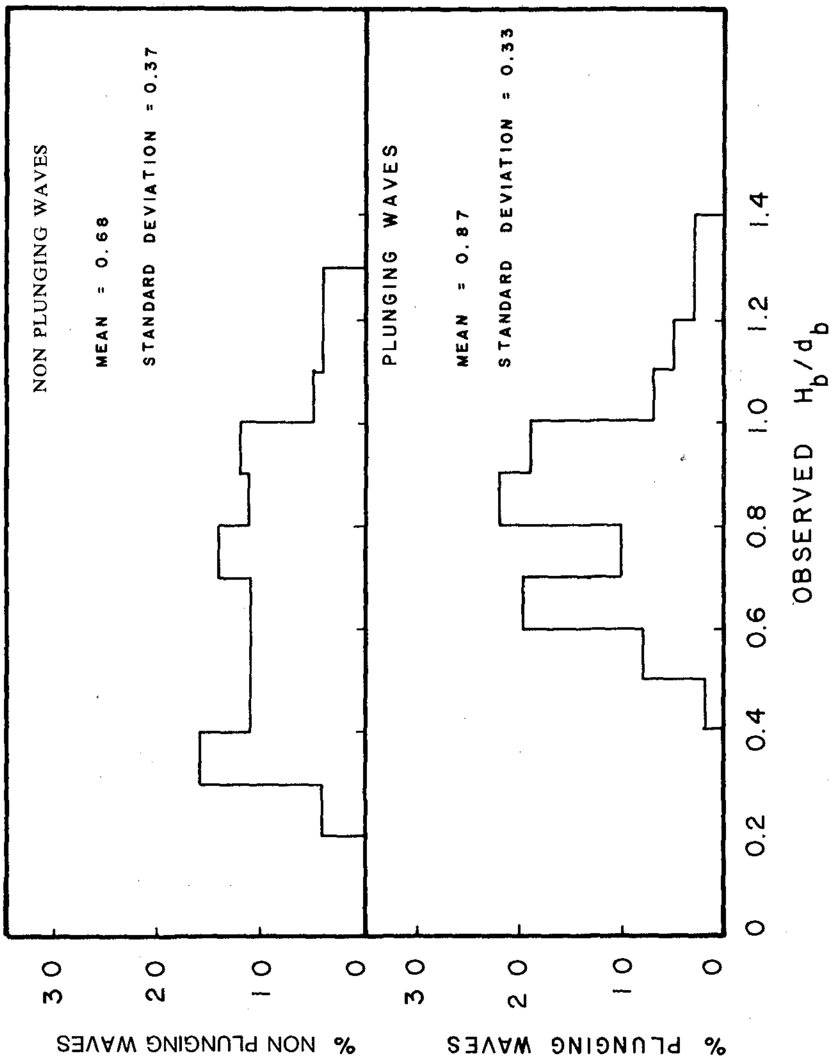

FIGURE 6. HISTOGRAMS OF H_b/d_b.

Offshore

$$\frac{H_o}{L_o\, m^2} \quad , \qquad \text{Galvin (1968)} \qquad (4)$$

$$\xi_o = \frac{m}{(H_o/L_o)^{1/2}} \quad , \qquad \text{Battjes (1974)} \qquad (5)$$

As noted by Battjes $\frac{H_o}{L_o\, m^2} = \frac{1}{\xi_o{}^2}$

Inshore

$$\frac{H_b}{gmT^2} \quad , \qquad \text{Galvin (1968)} \qquad (6)$$

$$\xi_b = \frac{m}{(H_b/L_o)^{1/2}} \quad , \qquad \text{Battjes (1974)} \qquad (7)$$

Galvin's results are shown in Figure 7. Battjes reexamined Galvin's data in terms of his classification parameters and suggested the following limits:

Offshore

surging	$\xi_o > 3.3$
plunging	$0.5 < \xi < 3.3$
spilling	$\xi_o < 0.5$

Inshore

surging	$\xi_b > 2.0$
plunging	$4.0 < \xi_b < 2.0$
spilling	$\xi_b < 0.4$

The purpose of this section is to test the criteria of Galvin and Battjes with the Virginia Beach data set.

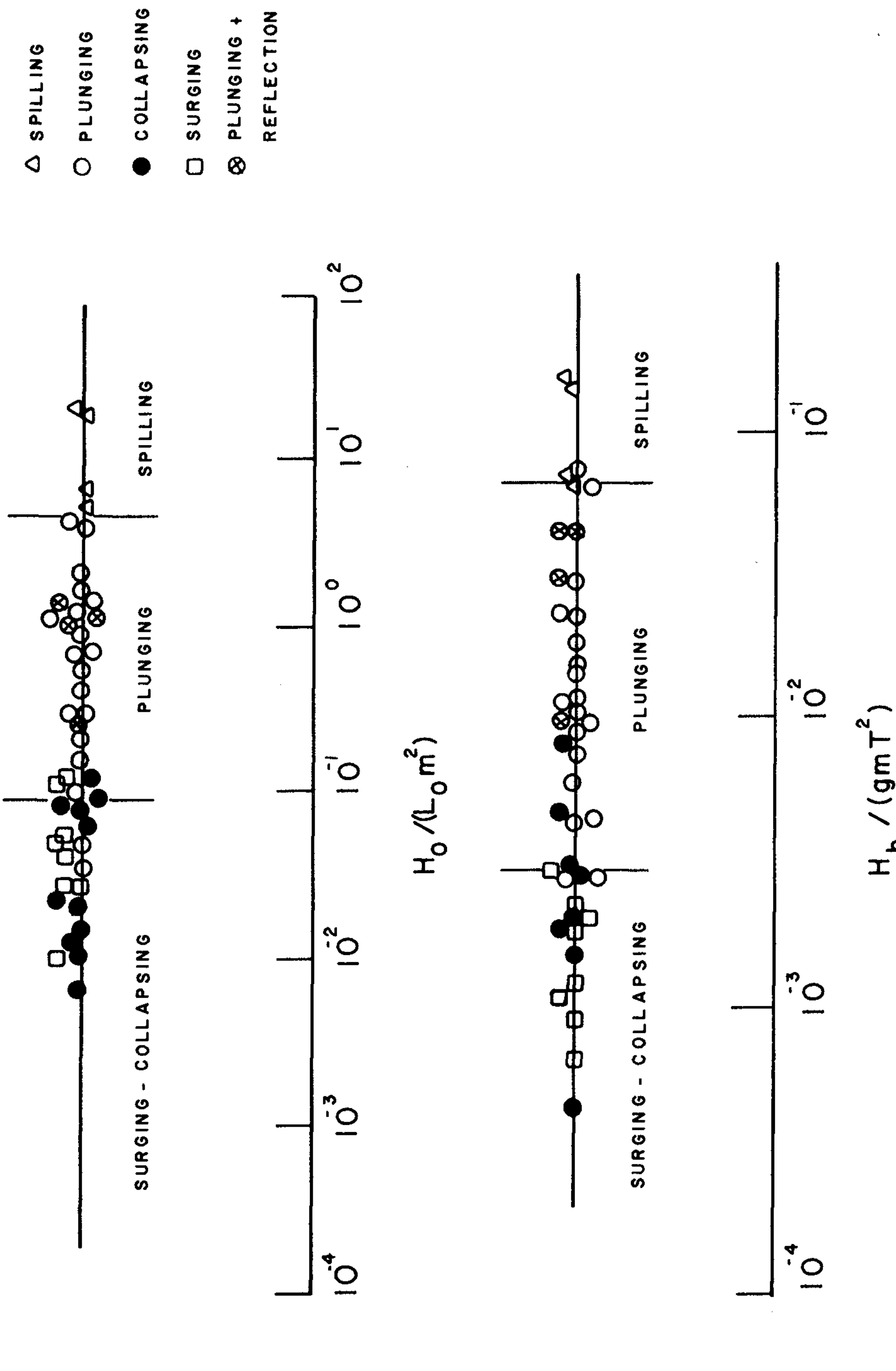

FIGURE 7. BREAKER TYPE AS A FUNCTION OF OFFSHORE AND INSHORE PARAMETER AFTER GALVIN,1968.

RESULTS. It is important to note that the criteria being tested were formulated under laboratory conditions with uniform slopes whereas the Virginia Beach data set represents conditions of small, but continuously variable slope. In the principal breaking region the average bottom slope was about 0.02 . The slope of the upper foreshore was 0.10 . The slope values entered in the parameter calculations were those of the local bottom slope at the breaker position. This procedure does not take into account the fact that the wave shape is conditioned by the slope conditions prior to the zone of breaking.

The results for the breaker type classification are shown in Figure 8. The most striking result is that the criteria do not separate plunging waves from spilling waves. Although the sample size for spilling waves is much smaller than that for plunging waves, the distribution of spilling waves does not favor the ranges of the parameters suggested by Galvin (1968) or Battjes (1974). On the other hand, most of the plunging waves do fall, or cluster, within the ranges found under laboratory conditions. Furthermore, it is to be noted that the data for plunging waves exhibit closer clustering on the basis of Battjes' criteria relative to those of Galvin. Finally, it may be noted that the "interference" plungers cluster with the "well-defined" plungers.

EVALUATION OF PLUNGE DISTANCE

As an aid in the design of coastal structures, Galvin (1969) presented results of laboratory experiments on the distance a plunging wave travels from the point of breaking to and including the splash region excited by the falling lip of the crest. Part of this distance is the plunge distance which is defined as the distance covered from the inception of breaking to the point where the falling, forward lip touches down in the preceeding wave trough (Fig. 9). The results presented in this section offer a comparison of field observations with Galvin's laboratory study.

Galvin's analysis is based on the assumptions that the internal particle velocities are given by the solitary wave phase speed, and that the falling forward face of the plunging wave can be approximated by a free fall trajectory (ballistics) model.

Thus, P , the plunge distance is given by the product of the phase speed, C_b , at breaking, and, t_p , the plunge time. For the free fall parabolic path condition, the time of fall is given by

$$t_p = 1/4 \, (H_b)^{1/2} \tag{8}$$

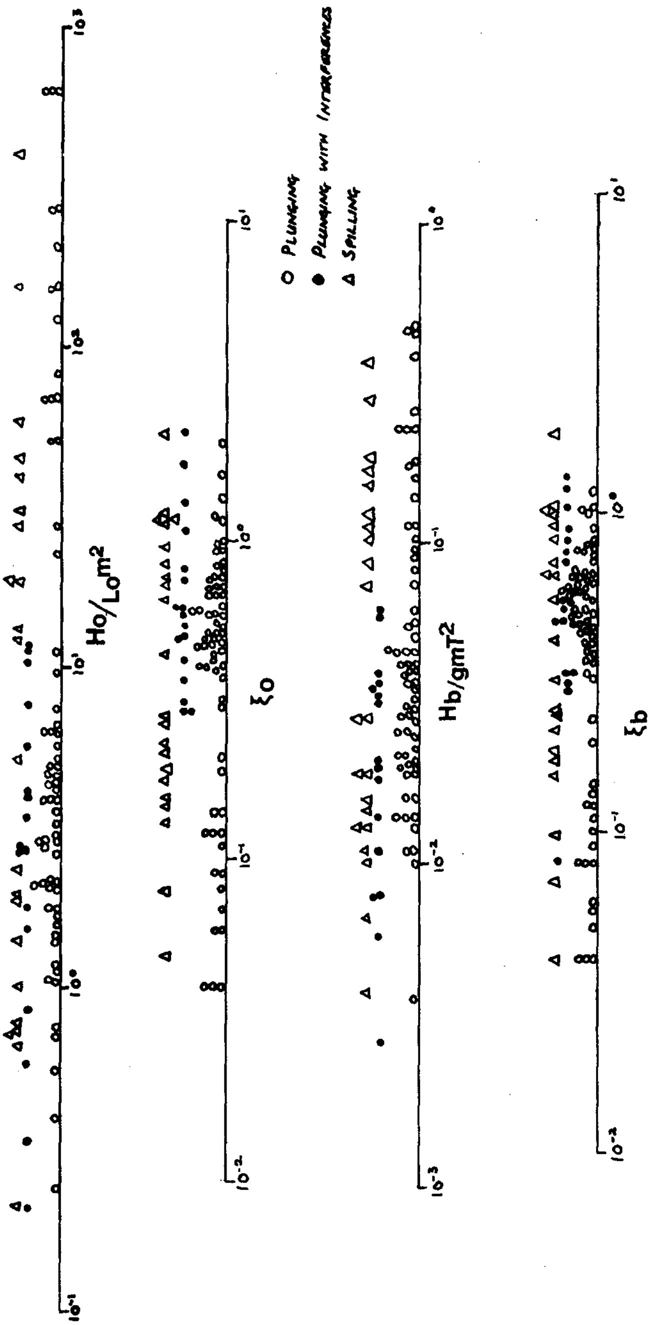

FIGURE 8. VIRGINIA BEACH BREAKER TYPES COMPARED WITH CRITERION OF GALVIN (1968) AND BATTJES (1974).

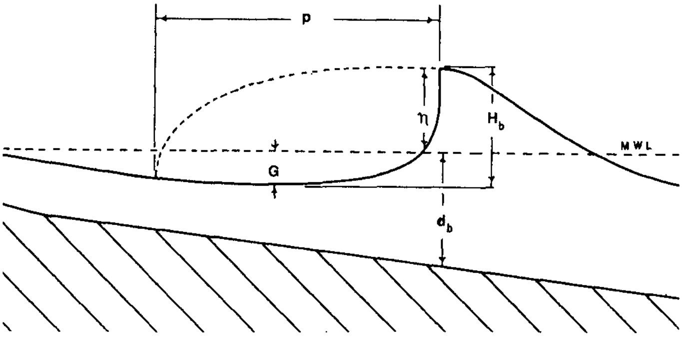

FIGURE 9. DEFINITION SKETCH FOR PLUNGE DISTANCE.

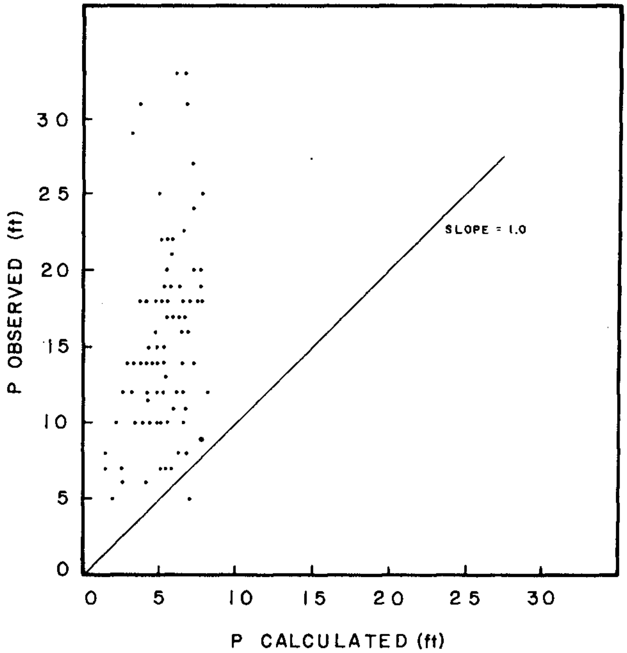

FIGURE 10. COMPARISON BETWEEN CALCULATED AND OBSERVED PLUNGE DISTANCE AT VIRGINIA BEACH.

The wave speed at breaking, C_b , is assumed to be (Fig. 9)

$$C_b = \{g\ (H_b - G + d_b)\}^{1/2} \tag{9}$$

Taking as approximations, $G \simeq 0.25\ H_b$ and $d/H_b \simeq 1.25$ substitution yields in units of feet per second

$$C_b \simeq 8\,(H_b)^{1/2} \tag{10}$$

Finally, combining Equations 8 and 10 yields

$$P/H_b \simeq 2 \tag{11}$$

Galvin (1969) recognized that the crest foreface has an upward velocity component at breaking and he expected Equation 11 to underestimate the plunge distance. His laboratory measurements showed P to range up to 4.5 with $\overline{P} \simeq 3$. Moreover, he found a dependency on bottom slope, P/H_b decreasing as slope increases. This dependency was attributed to the fact that on steeper slopes breaking occurs closer to the shore and the waves may have increasing interaction with the swash-backwash zone. From his averaged values on slopes of 0.05, 0.10 and 0.20, he found

$$P/H_b = 4.0 - 9.25\ m \tag{12}$$

Thus, for a horizontal bottom the expected $P/H_b = 4$.

RESULTS. The comparison of expected versus observed values of P is shown in Figure 10. The average value of P/H_b was 5.9 with the range extending from 1 to 10. The poor correspondence between the observed and "expected" values is obvious. In order to investigate the source of error, the observed breaker celerity, C_b , was compared with the calculated celerity using Equation 9. The average error between observed and expected was 12% , with the calculated celerity underestimating the observed at higher values. However, comparison of the calculated t_p (Eq. 8) with the observed t_p indicated an average error of 64% (Fig. 11). The principal source of error is thus in the calculation of plunge time. In order to determine the validity of the equation for plunge distance, $P = C_b t_p$, the observed plunge distance was compared with product of the observed values of breaker celerity and plunge time. These results are shown in Figure 12, wherein a close correspondence is observed. The remaining scatter is attributed to errors in determining the exact breaker position and the observed parameters, breaker celerity and plunge time.

The observations and analysis indicate that the free fall trajectory assumption is an incomplete model as an estimator for the trajectory of the plunging wave. A more complete model would have to include the vertical velocity components at breaking. The results presented for conditions of small beach slope do approximate, on the

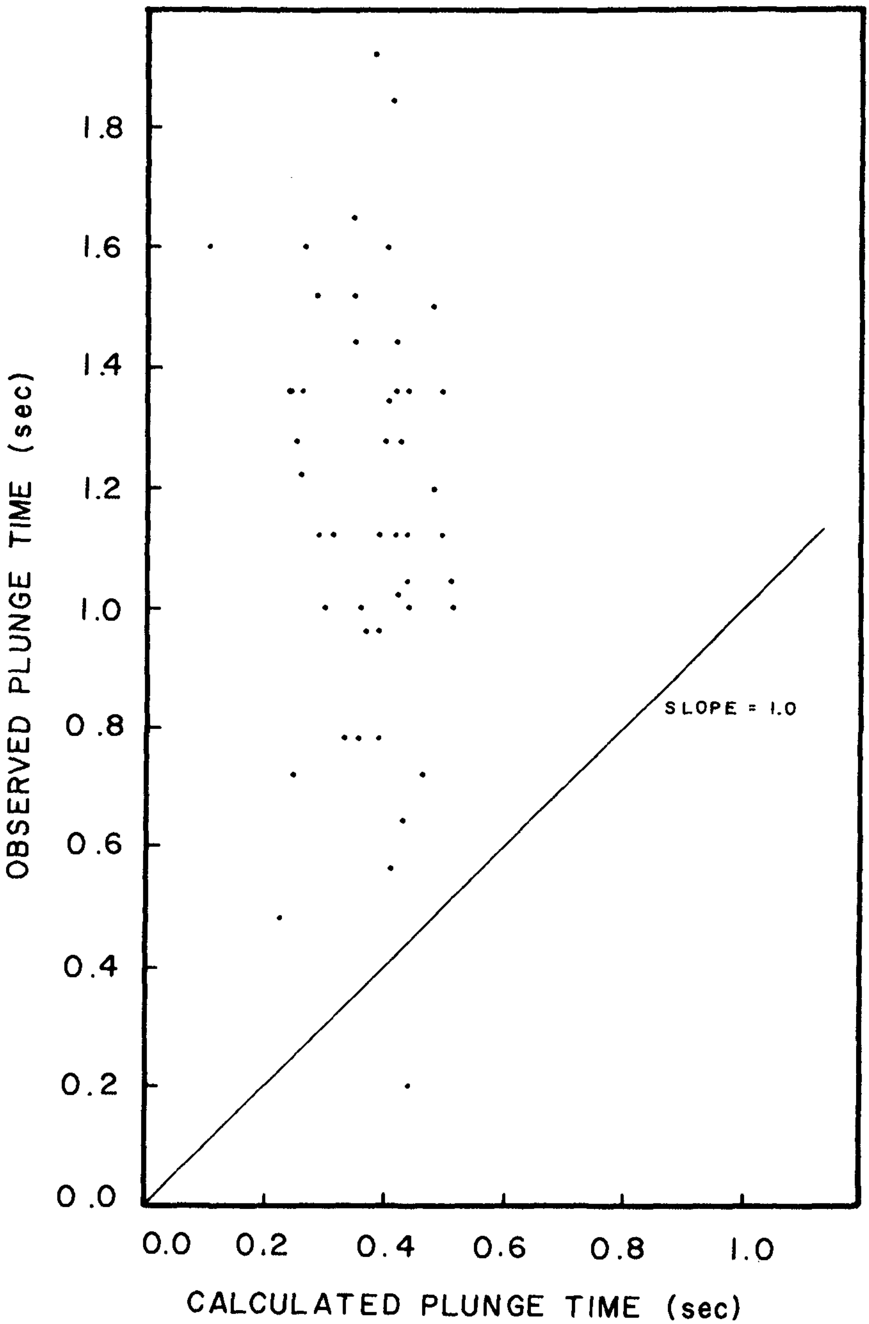

FIGURE 11. COMPARISON OF OBSERVED PLUNGE TIME WITH THAT EXPECTED FROM EQ.8.

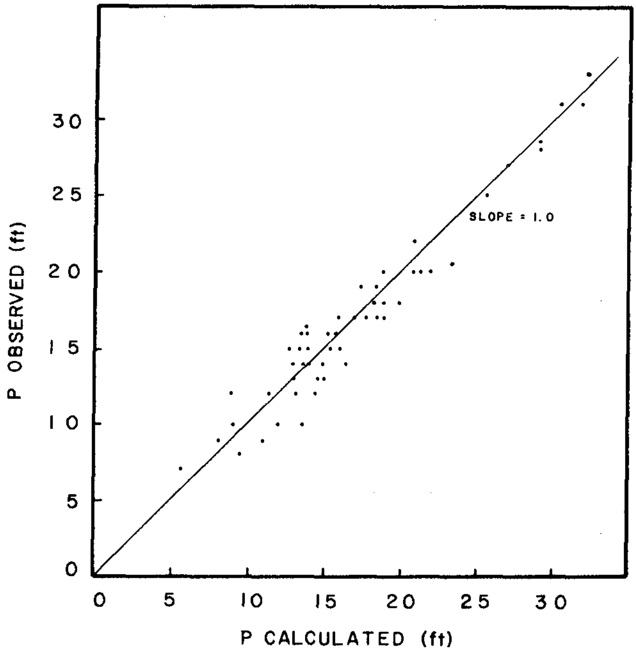

FIGURE 12. COMPARISON OF OBSERVED PLUNGE DISTANCE WITH THAT CALCULATED FROM THE CHARACTERISTIC EQUATION, $P = C_b t_b$.

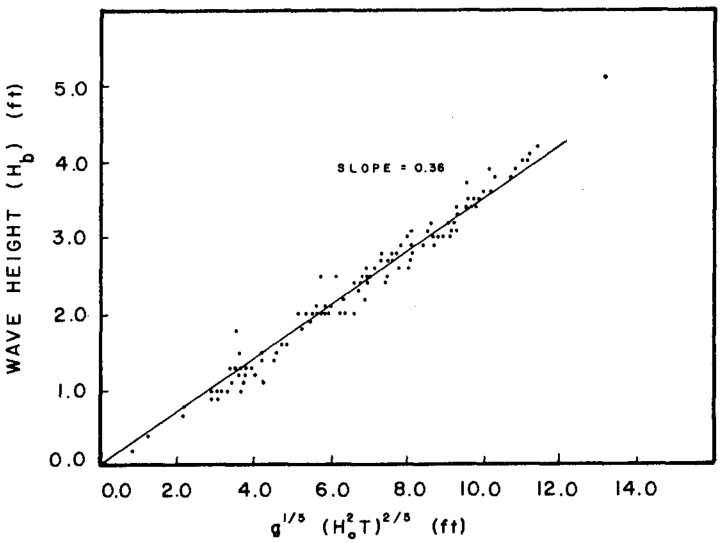

FIGURE 13. OBSERVED BREAKER HEIGHT AS A FUNCTION OF EQ. 17. REGRESSION GIVES K=0.36.

average, the experimental results of Galvin (1969) for small slope. Further field research on steeper beaches should clarify whether the level of approximation is consistent with experimental results.

PREDICTION OF BREAKING WAVE HEIGHT

Many engineering applications entail the estimation of wave breaker height given the deep water wave characteristics. Komar and Gaughan (1972) tested three sets of laboratory data and one set of field data from the California coast (Scripps pier) against a formulation for breaker height prediction using linear wave theory combined with a similarity criterion for relative breaking depth, H_b/d_b . The resulting relationship, empirically fitted to the data, was found to be an adequate predictor for breaker height over the range from small laboratory waves to the field data set consisting of breaker heights ranging from 1.2 to 3.5 m. Since the wave heights observed at Virginia Beach fall in the range intermediate to those tested, an independent evaluation was considered worthwhile.

From Komar and Gaughan (1972), the conservation of energy flux is,

$$(E\,C_\eta)_b = (E\,C_\eta)_o \tag{13}$$

From linear wave theory,

$$E_b = 1/8\;\rho g\;H_b^{\,2} \tag{14}$$

$$\text{and } C_b = \sqrt{g d_b} \tag{15}$$

Substitution of deep water characteristics and Equations 14 and 15 in Equation 13 yields, with reduction,

$$H_b^{\,2}\;(g\;d_b)^{1/2} = \frac{g}{4\pi}\;(T H_o^{\,2}) \tag{16}$$

Using $\alpha = H_b/d_b$ gives the results of Komar and Gaughan

$$H_b = K\;g^{1/5}\;(H_o^{\,2} T)^{2/5} \tag{17}$$

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

A best fit comparison with the experimental data yielded K = 0.39 . In passing, the predicted value of α for the combined data set of Komar and Gaughan is then 0.71 .

Also, if

$$C_b = \{g\ (\eta + d_b)\}^{1/2}$$

from this study $\eta = 0.92\ H_b$; $d_b = 1.26\ H_b$

then the expected K = 0.31 . Regression analysis of the Virginia Beach data yields K = 0.36 (Fig. 13).

The combined results of Komar and Gaughan, which compares the laboratory results of Komar and Simmons and Munk (1949), with the Virginia Beach data is shown in Figure 14. It appears that the relationship

$$H_b = 0.39\ g^{1/5}\ (TH_o{}^2)^{2/5}$$

provides a reasonable estimation of breaker height as a function of wave period and deep water wave height. However, it is to be noted that the effects of wave refraction or frictional dissipation are not incorporated. In the laboratory results the deep water wave height is calculated from linear wave theory. In the case of the data from Scripps pier H_o is an approximation as the shoaling coefficient varied between 0.99 and 1.44 . A more severe approximation applies for the Virginia Beach data set as the nearshore slope is slight and the effects of wave refraction and dissipation may be expected to be significant. Consequently, the fit to the deduced relationship should be viewed as a relationship between breaker height and the "apparent" deep water wave height with refraction and frictional effects ignored. The degree to which the relationship is an artifact borne of circular reasoning will await further testing which includes refraction and frictional effects. It should be further noted that calculation of the deep water parameters involved the use of the near breaking wave period which for individual waves bears an unknown, and perhaps indeterminate, relationship to deep water period for even "simple" wave trains.

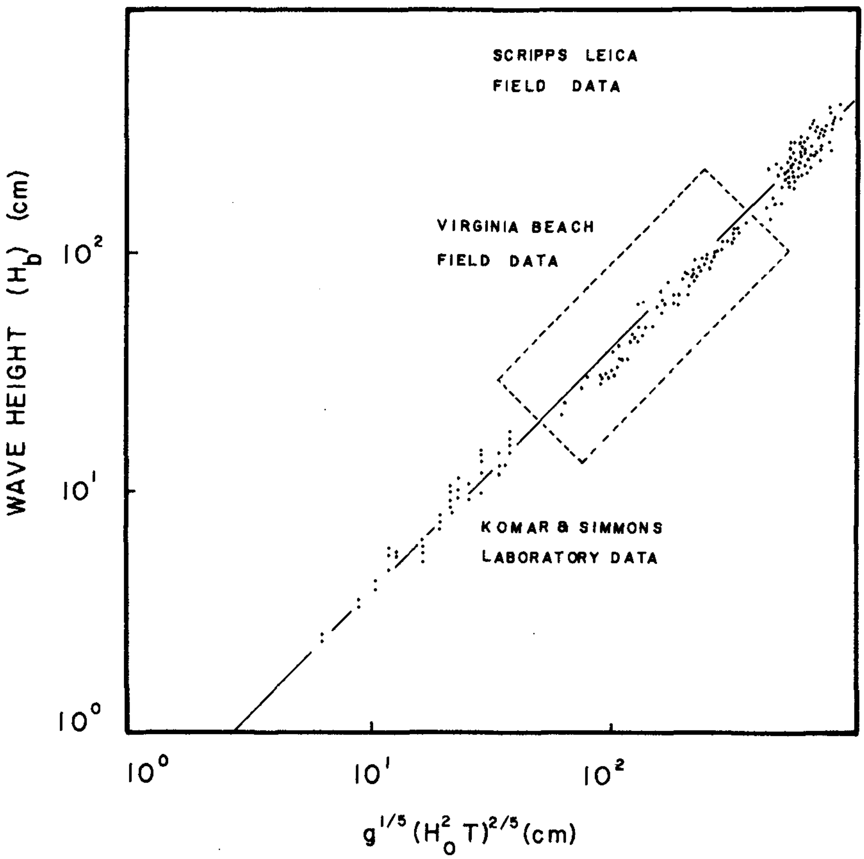

FIGURE 14. COMPOSITE DATA FOR BREAKING WAVE HEIGHT.

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