ON THE DETERMINATION OF WHEN WAVES BREAK IN SHALLOW WATER

by

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FOREWORD

Various theoretically and empirically derived methodologies have been proposed for predicting when waves become unstable and shore-break in the nearshore or littoral zone. This work reviews and discusses such methodology, and incorporating a significant amount of newly acquired field data, assesses the most reliable design relationship using statistical techniques.

The work described herein constitutes partial fulfillment of contractual obligations with the Federal Coastal Zone Management Program (Coastal Zone Management Act of 1972, as amended) through the Florida Office of Coastal Management subject to provisions of contract CM-37 entitled "Engineering Support Enhancement Program". Under provisions of DNR contract C0037, this work was reviewed by the Beaches and Shores Resource Center, Institute of Science and Public Affairs, Florida State University. The document has been adopted as a Beaches and Shores Technical and Design Memorandum in accordance with provisions of Chapter 16B-33, Florida Administrative Code.

At the time of submission for contractural compliance, James H. Balsillie was the contract manager and Administrator of the Analysis/Research Section, Hal N. Bean was Chief of the Bureau of Coastal Data Acquisition, Deborah E. Flack Director of the Division of Beaches and Shores, and Dr. Elton J. Gissendanner the Executive Director of the Florida Department of Natural Resources.

Deborah E. Flack

Deborah E. Flack, Director
Division of Beaches and Shores

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ABSTRACT

Prediction of when shore-propagating waves become unstable and break is generally considered to be a function of three factors: 1. wave height, 2. bed slope, and 3. wave steepness. Various relationships have been proposed for the prediction of shore-breaking occurrence. These relationships have either been founded on relatively small data bases or have been designed to predict for specialized conditions. In this work a significantly large sample of wave information, including both field and laboratory data, is used to evaluate popularly used predictive relationships. Graphical and statistical results support the originally proposed relationship of McCowan (1894), where $d_b = 1.28\ H_b$, as the best prediction of shore-breaking occurrence.

INTRODUCTION

Not only is the determination of the point at which shore-propagating waves become unstable and break a basic task in many coastal engineering problems, but breaking also identifies the point at which a major change in wave behavior occurs. It is fundamental that the "... ultimate limitation of any wave theory based on potential wave theory is given by the condition at which the wave breaks" (Madsen, 1976, p. 79). In terms of the destructive potential of waves, studies have shown (Miller et al. 1974a, 1974b; Miller, 1976) that breaking and broken waves result in greater impact pressures than the more symmetrical, less deformed waves in relatively deeper water. Dune and bluff erosion accompanying shore-incident storm and hurricane impact is a function not only of the storm surge but also of runup and setup produced by final shore-breaking activity. It becomes clear, therefore, that certain conditions at and following breaking necessary for successful coastal engineering design solutions require specialized types of predictive procedures. One of these is the prediction of the point at which shore-breaking occurs.

Results from available research indicate that the water depth, bed slope and wave steepness constitute the major variables influencing wave stability. Considering these variables, ocean waves are generally thought to become unstable and break as follows.

Deep water conditions represent one extreme where the water depth and bed slope do not influence waves passing above, and the waves become unstable and break because they become critically steep. Critical steepness occurs under forced wave conditions (Mooers, 1976; Balsillie et al. 1976) wherein significantly high wind stresses produce instability and breaking. Such waves commonly appear as spilling type breakers, often called white caps.

The other extreme occurs in nearshore shallow water depths. The water depth is the most critical factor influencing wave stability. The bed slope and wave steepness, although they have been considered to be influential, are apparently of secondary importance. Breaking wave types, while they may include spilling breakers, also may include other generally recognized types such as plunging, surging and collapsing breakers.

Between the two extremes, the stability of waves is apparently dependent on all three parameters, each of which may play a significant role. Breaker type is commonly of the spilling type.

Where breaking occurs in nearshore shallow water depths, resulting in littoral zone activity, the author has adopted the terminology, shore-breaking waves. In deeper water, wave instability is simply termed as breaking.

The work presented herein is concerned with conditions at the shore-breaking position, and includes newly acquired field shore-breaker data (Balsillie and Carter, 1980) in addition to the re-evaluation of existing field and laboratory data. The goal of the work is to identify and evaluate criteria useful for least equivocal coastal engineering design solutions which require determination of conditions that induce shore-breaking.

PREVIOUS WORK

In deep water Michell (1893) found that the maximum limiting wave steepness above which breaking occurs may be given by:

$$\left(\frac{H_o}{L_o}\right)_{max} = \frac{1}{7} \tag{1}$$

where H_o and L_o are the deep water wave height and length, respectively, or where from small amplitude wave theory $L_o = g\,T^2/(2\pi)$, by:

$$\left(\frac{H_o}{g\,T^2}\right)_{max} = \frac{1}{14\,\pi} \tag{2}$$

Using the data of the Beach Erosion Board (1941) and the fifth order Stokes-Levi-Civita solution (Levi-Civita, 1924), for forced wave conditions given by equations (1) and (2), then:

$$\left(\frac{H_o'}{H_o}\right)_{max} = 0.64 \tag{3}$$

as discussed by Balsillie (in manuscript) where H_o' is the height of the deep water wave crest lying above the still water level (SWL).

The maximum steepness for progressive waves in any depth of water is given by Miche (1944) as:

$$\left(\frac{H}{L}\right)_{max} = \frac{1}{7} \tanh \frac{2 \pi d}{L} \tag{4}$$

McCowan (1891) found that at the shore-breaking position the stability of the wave profile is primarily dependent on the water depth to wave height ratio, according to:

$$\frac{d_b}{H_b} = 1.28 \tag{5}$$

subsequently supported Munk (1949), where the subscript 'b' refers to conditions of the water depth and wave height at the shore-breaking position. Equation (5) was developed using solitary wave theory where the entire wave lies above the SWL. However, recent work (Weishar, 1976; Weishar and Byrne, 1978; Hansen, 1976; Balsillie, in manuscript) suggests that:

$$\frac{H_b'}{H_b} = 0.84 \tag{6}$$

where H_b' is that portion of the shore-breaker crest lying above the SWL (see definition sketch of Figure 1).

Investigation subsequent to development of Equation (5) (Iverson, 1952a, 1952b; Galvin, 1968, 1969; Collins and Wier, 1969; Weggel and Maxwell, 1970;

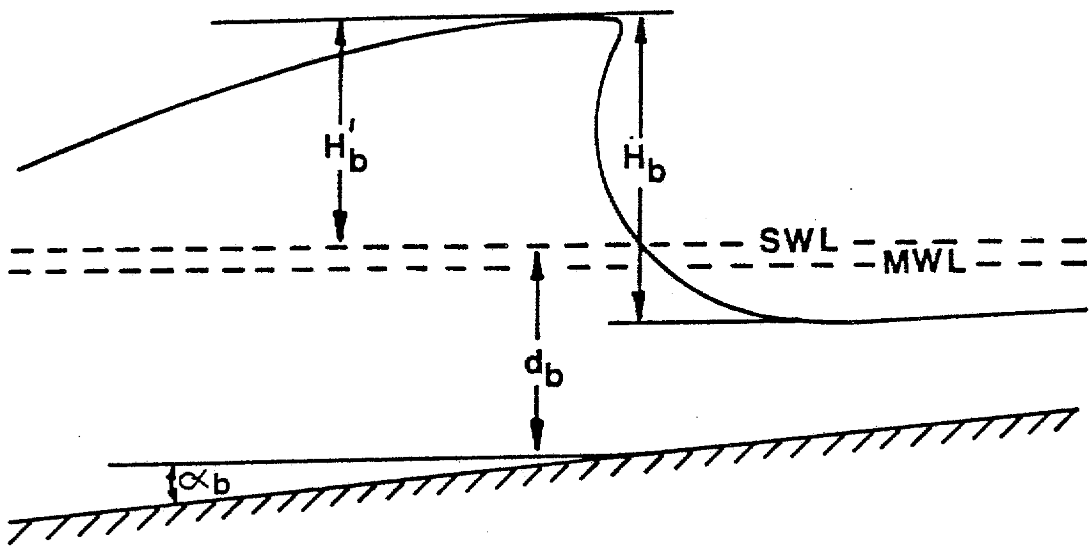

Figure 1. Definition sketch of wave parameters at the shore-breaking position (plunging shore-breaker).

Weggel, 1972a, 1972b; and Mallard, 1978) indicate, at the shore-breaking position, that in addition to the water depth, there is a residual dependence on the bottom slope. Galvin (1969) found that:

$$\frac{d_b}{H_b} = 0.92 \quad \Bigg\| \quad \tan \alpha_b > 0.07 \tag{7}$$

where α_b is the bottom slope leading to shore-breaking, and:

$$\frac{d_b}{H_b} = 1.4 - 6.85 \tan \alpha_b \quad \Bigg\| \quad \tan \alpha_b < 0.07 \tag{8}$$

which are both referenced to the mean water level (MWL) rather than to the SWL used in this work. Galvin (1969) suggests that for tan α_b on the order of from 0.05 to 0.1 the SWL is higher than the MWL (Figure 1) by a factor of 0.04 H_b , and where tan α_b is about 0.2 by a factor of 0.08 H_b.

Collins and Wier (1969) suggest that:

$$\frac{d_b}{H_b} = (0.72 + 5.6 \tan \alpha_b)^{-1} \tag{9}$$

and Mallard (1978) concludes:

$$\frac{d_b}{H_b} = \left[0.73 + 2.87 (\tan \alpha_b)^{0.997}\right]^{-1} \tag{10}$$

Equations (7) through (10) are plotted in Figure 2. For equations (8), (9) and (10), values of d_b/H_b are close for tan α_b less than about 0.01 and may

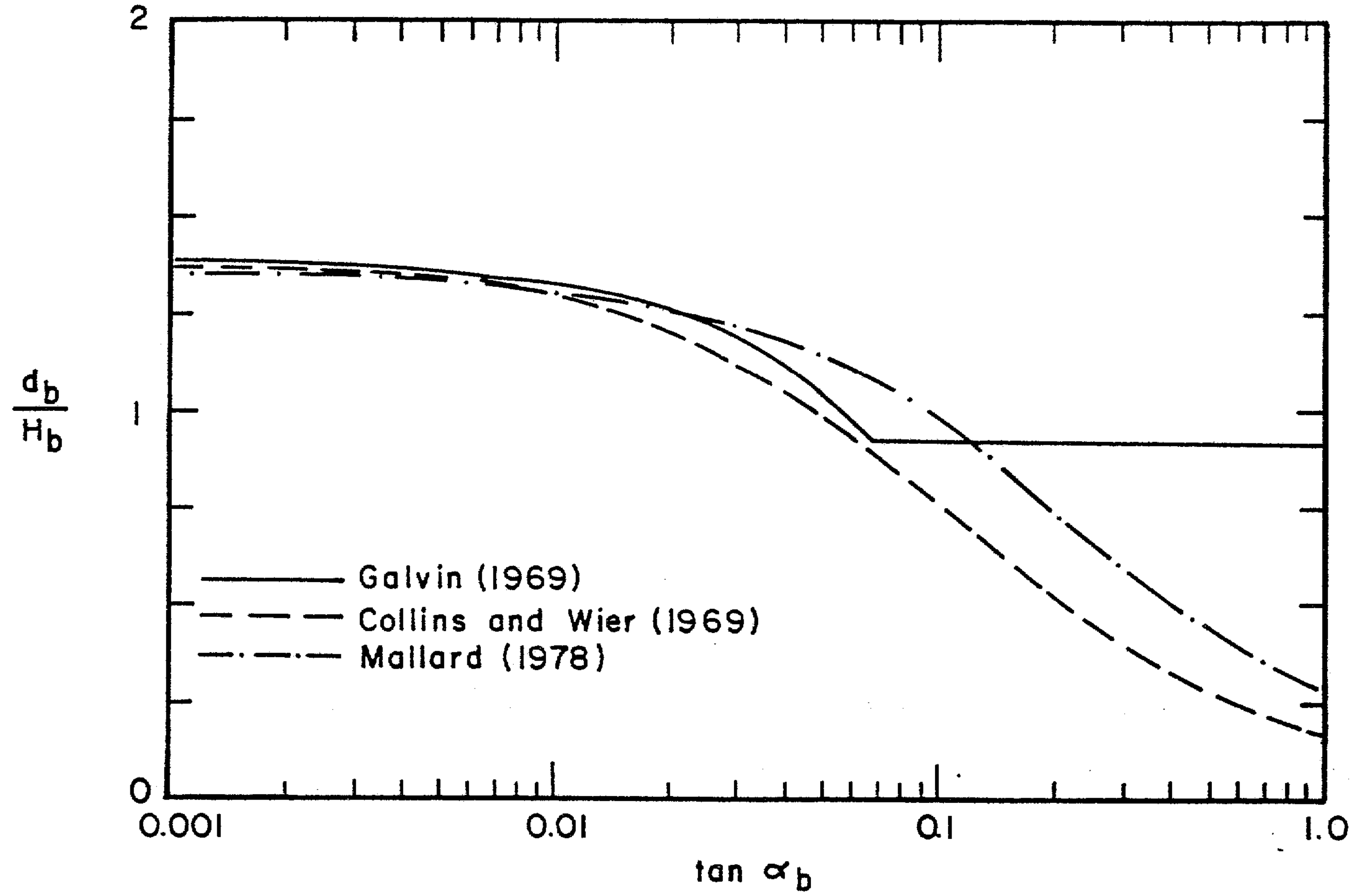

Figure 2. Comparison of results of predicted d_b/H_b using the relationships of Galvin (1969), Collins and Wier (1969), and Mallard (1978) given by equations (7) through (10) in text.

satisfactorily predict d_b/H_b where $\tan \alpha_b < 0.1$. However, where $\tan \alpha_b$ is greater than about 0.1 (i.e., $\alpha_b > 6^o$), the use of equations (7), (9) and (10) is not recommended.

An additional parameter which may significantly influence shore-breaking was investigated by Weggel (1972a, 1972b). The parameter, $H_b/(g\ T^2)$, termed the <u>equivalent breaker steepness parameter</u>, whose derivation is given by Battjes (1974, p. 469) can be introduced into Weggel's empirical result, to yield:

$$\frac{d_b}{H_{b\ max}} = \left[c_3 - c_1 \frac{H_b}{g\ T^2} \right]^{-1} \tag{11}$$

where

$$c_1 = c_2\ g \left(1.0 - e^{-19 \tan \alpha_b} \right) \tag{12}$$

in which c_2 = 4.462 m^2/s = 1.36 ft^2/sec in unit-consistent terms, and:

$$c_3 = 1.56 \left(1.0 + e^{-19.5 \tan \alpha_b} \right)^{-1} \tag{13}$$

As noted, Weggel's relationship is concerned with predicting a maximum design shore-breaker height. Even so, based on physical reasoning he has incorporated limiting constraints for extreme values of the bed slope and equivalent shore-breaker steepness parameter. He suggests that where the bed slope approaches infinity (i.e., a vertical wall), the minimum value of d_b/H_b will be one-half the theoretical value (based on the sum of the incident and perfectly reflected wave components) wherein c_3 approaches 1.56

and the value of $(d_b/H_b)_{min}$ approaches 0.64 when the equivalent shore-breaker steepness parameter approaches zero. It is interesting to note that for a vertical slope, the maximum value of $H_b/(g\ T^2)$ will be 0.0356. For deep water conditions where according to the Michell (1893) condition $(H_o/L_o)_{max} = 1/7$, then $H_b/(g\ T^2)_{max} = 1/(14\pi) = 0.0227$ which is 36% less than Weggel's value. As the bed slope approaches a value of zero (i.e., a flat bed), Weggel assumes that the effect of the slope should diminish and the theoretical value of McCowan shall be more nearly valid, hence the value of c_1 approaches zero while c_3 approaches 0.78 and the value of $(d_b/H_b)_{max}$ becomes 1.28.

DISCUSSION

In this section several commonly recognized relationships for predicting where shore-breaking will occur are discussed and evaluated, progressing from simplest to most complex. A major problem encountered when dealing with this subject is that one deals with quite small differences between input variables and parameters when, in fact, the errors and variability encountered when measuring hydraulic conditions in the littoral zone are often comparable.

The measurement of wave heights and water depths at shore-breaking, whether in the laboratory or field, is invariably difficult, if only because one is dealing with a moving wave form. Laboratory measurements involve highly sophisticated types of sensors for measuring surface elevations. However, no sensors are capable of determining the point at which a wave shore-breaks ..... that is, for spilling, plunging, surging, collapsing, etc., ... shore-breaker types, which is, ultimately, dependent on visual recognition. Hence, it appears realistic to accept that errors creep into laboratory results. In addition, shore-breaking seldom occurs in precisely the same water depth because of complicating factors such as wave reflection and wave interference, and where the bed is mobile by changes in the subaqueous morphology caused by sediment transport processes. Field measurements encounter similar problems. In the field, however, more than one wave train is usually present which can introduce additional wave interference problems. Because of the variability of breaker depth and the magnitude of impact forces associated with shore-breaking waves in the field, the use of laboratory-equivalent sophisticated sensor equipment is not possible for the higher waves. Hence, less sophisticated measurement techniques can potentially allow additional error to be associated with the results. However, by dealing with the higher field waves, relative to the smaller laboratory waves, field error may in cases be offset or minimized.

From the above discussion it becomes evident, given state-of-the-art measurement techniques, that error will invariably be associated with shore-breaking data. While it is recognized that scientific progress requires the development of measurement techniques which reduce the amount of error, existing data already evaluated plus that developed in the interim periodically deserves re-evaluation. It becomes not only important that as much data as possible is available but that the range in data is as large as possible. General characteristics of the field and laboratory data used in this study are listed in Table 1. To date no study known to the author has included such a large data base with such a large range in values.

Effect of Shore-Breaker Height

The most straightforward and first known successful relationship predicting where shore-breaking will occur was suggested by McCowan (1894), given by equation (5). The relationship is evaluated in Figure 3 using 418 simultaneous measurements of d_b and H_b (i.e., 167 field and 251 laboratory data pairs). Equation (5) is superimposed upon the data of Figure 3 and

Table 1. General characteristics of the field and laboratory data used in analyses.

I. D.	n	$\tan\alpha_b$	H_b (m)	d_b (m)	T (s)	$\frac{H_b}{g\,T^2}$
			FIELD DATA			
Gaillard (1904)	26-38	0.0200-0.0154	0.610-3.962	0.792-5.456	3.85-10.98	0.00201-0.01103
Scripts (1945) *Spec.* Meas.[1]	29	———	0.27 -1.13	0.49 -1.26	———	———
Scripts (1945) Leica Type I[1]	56	0.0159	1.129-3.475	1.554-4.450	7.2 -13.7	0.00103-0.00612
Scripts (1945) Leica Type II[1]	18	0.049	2.740-1.28	1.680-3.720	7.0 -13.0	0.00107-0.00572
Balsillie and Carter (1980)	26	0.005 -0.25	0.056-0.541	0.11 -0.65	1.38- 8.57	0.00024-0.00560
			LABORATORY DATA			
Putnam, Munk and Traylor (1949)						
Natural *Sand* Beach	7	0.066	0.088-0.143	0.112-0.229	1.0 - 1.32	0.00458-0.01459
" " "	3	0.144	0.046-0.049	0.070-0.073	1.9 - 2.22	0.00095-0.00139
" " "	4	0.241	0.067-0.107	0.082-0.158	0.72- 1.22	0.00459-0.01673
Smooth Metal and Concrete	4	0.100	0.049-0.073	0.067-0.098	0.99- 1.98	0.00128-0.00760
" " " "	5	0.139	0.061-0.085	0.073-0.131	0.83- 1.35	0.00342-0.01259
" " " "	5	0.260	0.061-0.104	0.07 -0.189	0.8 - 1.27	0.00411-0.01658
≲1/4-inch Pea Gravel	4	0.098	0.037-0.079	0.058-0.11	0.95- 1.99	0.00095-0.00893
" " "	5	0.143	0.061-0.101	0.079-0.143	1.08- 2.32	0.00127-0.00884
Munk (1949), Berkeley Exps.[3]	5	0.009	0.087-0.100	0.118-0.145	1.05- 1.98	0.00226-0.00926
	5	0.054	0.068-0.092	0.063-0.138	0.86- 1.97	0.00179-0.01269
	6	0.072	0.082-0.099	0.071-0.126	0.90- 1.97	0.00216-0.01247
Munk (1949), B.E.B. Exps.[3]	9	0.030	0.031-0.054	0.043-0.081	0.75- 1.03	0.00327-0.00599
	15	0.049	0.043-0.130	0.043-0.187	0.73- 1.08	0.00446-0.01137
	13	0.159	0.034-0.121	0.044-0.170	0.74- 1.09	0.00376-0.01039
Iverson (1952)[3]	13	0.020	0.055-0.121	0.065-0.156	0.90- 2.65	0.00142-0.00939
	15	0.033	0.053-0.126	0.070-0.155	1.05- 2.65	0.00080-0.00990
	19	0.050	0.043-0.128	0.049-0.165	0.74- 2.24	0.00195-0.01081
	16	0.100	0.049-0.122	0.043-0.137	0.80- 2.50	0.00119-0.01092
Morison and Crooke (1953)[3]	3	0.020	0.056-0.084	0.070-0.101	0.78- 2.62	0.00120-0.00939
	3	0.100	0.073-0.113	0.077-0.129	1.0 - 2.5	0.00119-0.01092
Bowen, Inman and *Simmons* (1968)[3]	11	0.082	0.04 -0.127	0.042-0.097	0.82- 2.37	0.00214-0.00895
Komar and *Simmons* (1968)[2,3]	10	0.036	0.074-0.166	0.093-0.212	1.14- 2.37	0.00193-0.00958
	14	0.070	0.030-0.159	0.040-0.164	0.81- 2.37	0.00173-0.01036
	11	0.086	0.042-0.147	0.041-0.162	0.81- 2.37	0.00174-0.00989
	9	0.105	0.035-0.170	0.036-0.170	0.81- 2.37	0.00098-0.01123
Weggel and Maxwell (1970)[3]	9	0.057	0.089-0.162	0.087-0.169	1.26- 2.05	0.00216-0.00803
Van Dorn (1978)[3]	4	0.022	0.13 -0.166	0.138-0.208	1.65- 4.8	0.00062-0.00622
	4	0.040	0.119-0.162	0.111-0.217	1.65- 4.8	0.00053-0.00607
	4	0.083	0.108-0.156	0.093-0.154	1.65- 4.8	0.00048-0.00585
Buhr Hansen and *Svendsen* (1979)[3]	16	0.0292	0.043-0.129	0.047-0.149	0.83- 3.33	0.00087-0.01092

[1]Reported by Munk (1949); [2]reported by Gaughan, Komar and Nath (1973); [3]smooth, fixed beds.

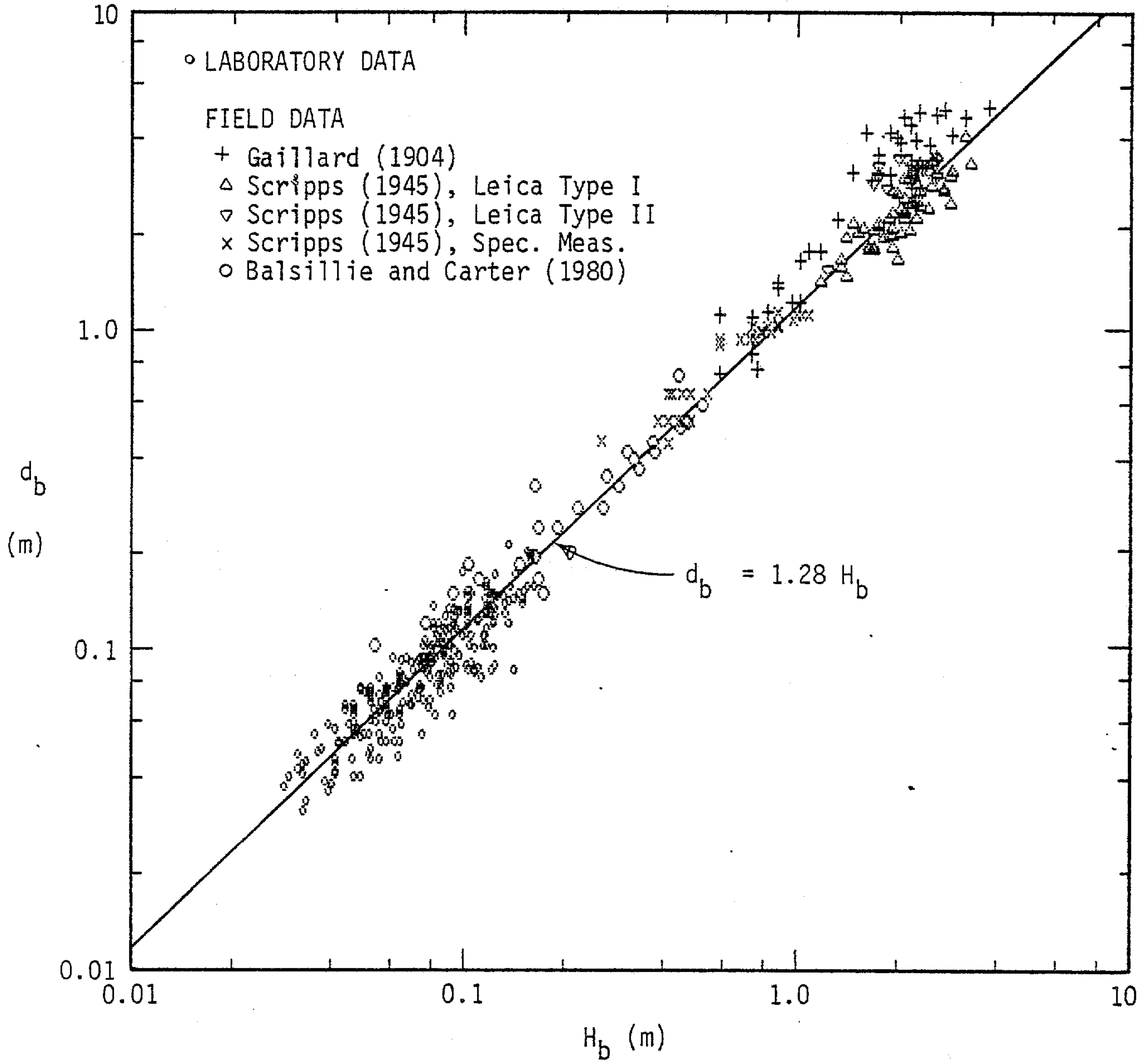

Figure 3. The McCowan equation superimposed on the data of Table 1. Data include 167 field and 251 laboratory points.

appears to successfully represent the central trend of the data. It is to be noted that the relative magnitude of scatter about the line appears to be approximately equivalent for both laboratory and field data.

Statistical methods provide better assurance of the goodness of fit. Commonly used regression techniques employ predictive regression which upon regressing x on y minimizes only the sum of the squares of the horizontal distances from the points to the fitted line. However, enhanced assessment of the goodness of fit can be determined using functional regression procedures discussed by Ricker (1973). He suggests, based on the work of Teissier (1948), that the central tendency of the data might be more adequately determined by finding the line which minimizes the sum of the products of both the vertical and horizontal distances of each point from the line. The slope of this line forced through the origin (i.e., $x = 0$, $y = 0$) is given by:

$$m = \sqrt{\frac{\Sigma y^2}{\Sigma x^2}} \tag{14}$$

Plus and minus limits, s', of the central line fitted by equation (14) to the 95% confidence interval limit, are given by Ricker (1973), according to:

$$s' = \pm \left[t_{\alpha/2}\ (n - 2)\right]\sqrt{\frac{1 - r^2}{n - 2}\ m^2} \tag{15}$$

where n is the sample size, r is the Pierson product-moment correlation coefficient, and $t_{\alpha/2}(n - 2)$ is the Student's t value for $n - 2$ degrees of freedom.

However, Equation (14) as it applies to the data of Figure 3 is influenced more by the larger data values. For instance, for the field data, m = 1.426 and for the laboratory data, m = 1.168, but for the data combined m = 1.425. Hence, even though there are more laboratory data than field data, the laboratory data influences the overall slope by only 0.07%. This condition introduces a significant problem since it is the smaller magnitude laboratory data which probably represent the more precise information due to the more sophisticated measurement techniques used. For this reason, functional regression techniques are applied separately to the laboratory and field data and the fitted slope, m, determined as a weighted average. Resulting statistics are listed in Table 2, where for the data of Figure 3 the weighted average fitted slope is 1.271. In fact, if the listed values of m for the laboratory and field data and the sample size of the field data are held constant, then only 3 more laboratory measurements would be required to result in the McCowan coefficient of 1.28 for the laboratory and field data combined (i.e., weighted average value). Plus and minus limits of the McCowan equation (i.e., where m = 1.28) are, from equation (15), 1.251 and 1.309.

While it could be viewed that the statistical results listed in Table 2 for equation (5) are fortuitous, two considerations should be noted:

1. both the laboratory and field data samples are significantly large, and
2. recognizing that statistical methods cannot always provide definitive answers to specific numerical problems, the graphical approach can be used to allow the reader to observe and render his or her own judgements.

Effect of Shore-Breaker Height and Bed Slope

Of the previously introduced equations which consider the bed slope in addition to the shore-breaker height, the equation of Mallard (1978) ....., i.e., equation (10)..... has been selected for evaluation. Reasons for

Table 2. Statistical results -- goodness of fit of functional regressions.

I. D.	Field Data				Laboratory Data				All Data			
	n	m	r	s'	n	m	r	s'	n	m	r	s'
McCOWAN RELATIONSHIP (Figure 3)												
Independent Fit	167	1.426	0.9125	0.090	251	1.168	0.8776	0.070	418	1.271*	0.9636	0.037
McCowan Equation	167	1.28	0.9125	0.070	251	1.28	0.8776	0.055	418	1.28	0.9636	0.029
MALLARD RELATIONSHIP (Figure 4)												
Independent Fit	138	1.175	0.8403	0.108	251	1.055	0.8413	0.072	389	1.098*	0.9447	0.039
Correction	138	1.098	0.8403	0.099	251	1.098	0.8413	0.065	389	1.098	0.9447	0.035
WEGGEL RELATIONSHIP (Figure 5)												
Independent Fit	126	1.254	0.8713	0.109	251	1.159	0.9258	0.055	377	1.191*	0.9552	0.038
Correction	126	1.191	0.8713	0.092	251	1.191	0.9258	0.046	377	1.191	0.9552	0.033

* Weighted average upon which assessment of the McCowan equation (assuming 1.271 and 1.28 are essentially equivalent) and the corrected equations, are based.

selection are: 1. the wave heights and water depths are referenced to the SWL, and 2. Mallard's analysis used a significantly large sample (i.e. n = 213) including seven laboratory studies and one field study.

Application of equation (14) to the available data indicates that Mallard's equation underestimates the central tendency of the data by 10% (i.e., from Table 2, $100[1-(s_c'/s_o')] = 100[1-(0.035/0.039)] = 10.2\%$, where s_c' is associated with the corrected equation and s_o' is associated with the original equation). The corrected equation is given by:

$$\frac{d_b}{H_b} = 1.098 \left[0.73 - 2.87 (\tan \alpha_b)^{0.997} \right]^{-1} \qquad (16)$$

illustrated in Figure 4. Plus and minus limits of the coefficient correcting the Mallard equation (m = 1.098), from equation (15), are 1.063 and 1.132 (Table 2).

Visual inspection of Figure 4 illustrates that scatter associated with equation (16) is somewhat greater than for the McCowan equation illustrated in Figure 3. Statistics listed in Table 2 support the visual comparison, wherein the correlation coefficient associated with the McCowan equation has a larger value and the degree of relative scatter about the fitted regression lines, given by equation (15), is 17% less than that associated with Mallard's corrected equation.

Effect of Shore-Breaker Height, Bed Slope, and Equivalent Shore-Breaking Wave Steepness Parameter

While (Weggel, 1972a, 1972b) introduced physical constraints in order to yield more reasonable results for extreme conditions (equations (11) through

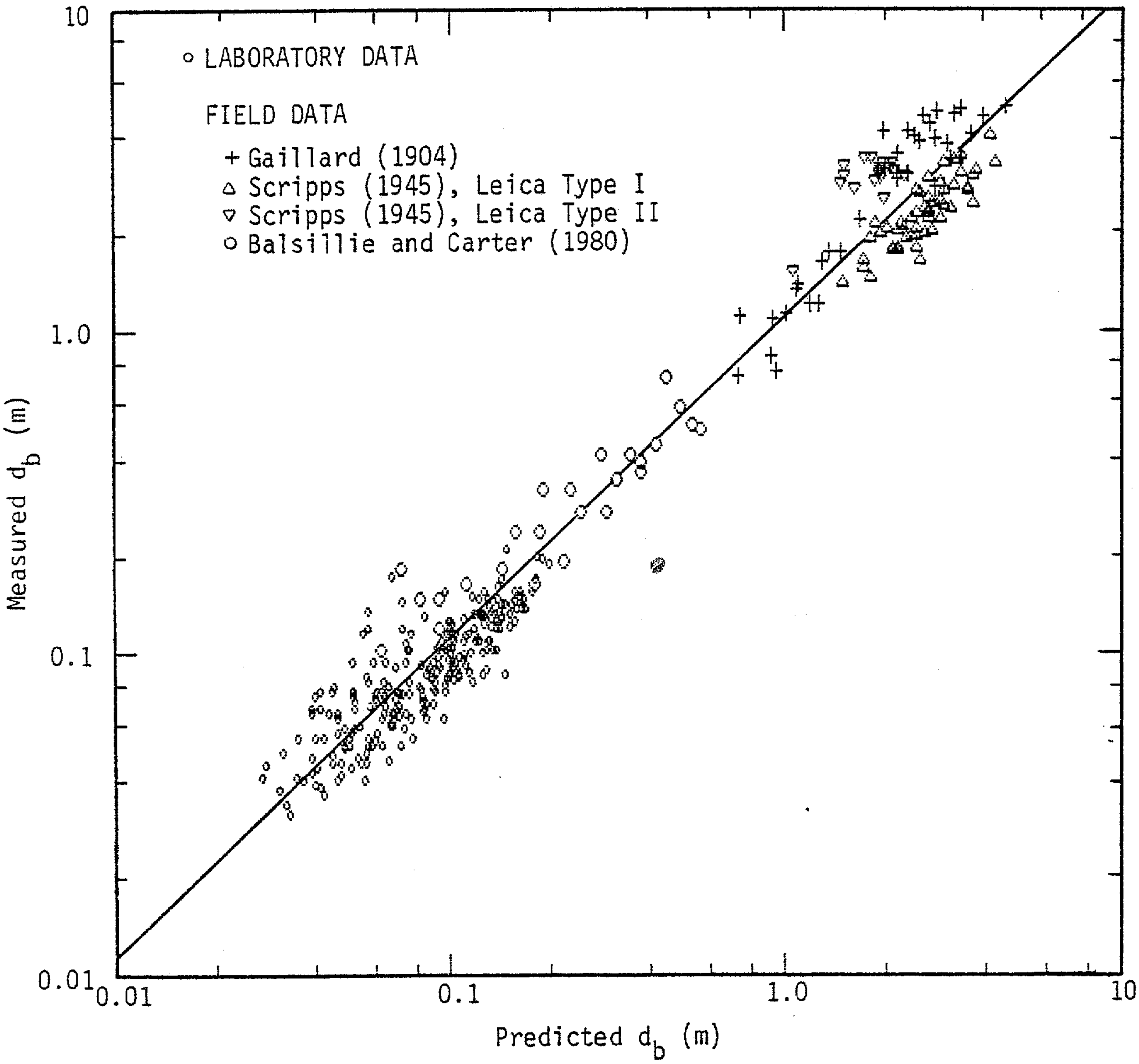

Figure 4. Measured water depth at shore-breaking versus the depth predicted from the corrected Mallard relationship given by equation (16). Data include 138 field and 251 laboratory points.

(13)), he was primarily concerned with predicting the maximum shore-breaker height. In this work, however, the intent is to determine the water depth at shore-breaking for a shore-breaker more closely representing average height conditions.

Application of equation (14) to the data yields a corrected equation given by

$$\frac{d_b}{H_b} = 1.191 \left[c_3 - c_1 \frac{H_b}{g\,T^2} \right]^{-1} \tag{17}$$

illustrated in Figure 5, where c_1 and c_3 are given by equations (12) and (13) and c_2 remains as specified earlier. Plus and minus limits of the corrected equation, from equation (15), are 1.158 and 1.224.

Equation (17) results in better predictive precision than equation (11) for average conditions by 13% (i.e., from Table 2 $100[1-(s_c'/s_o')] = 100[1-(0.033/0.038)] = 13.2\%$).

Visual comparison of Figures 3 and 5 illustrate that there is less scatter for the McCowan equation than for equation (17). The correlation coefficient for the McCowan equation is higher than for equation (17), and the relative scatter about the fitted line is 12% less for the McCowan equation.

Similar comparisons also show that Weggel's modified equation results in somewhat better predictive precision than the corrected Mallard equation

A NOTE ON THE SHORE-BREAKER TYPE

Weishar and Byrne (1979) report a statistically significant difference in the average value of d_b/H_b for plunging and non-plunging shore-breaker types (note that Weishar (1976) originally defined the non-plunging waves

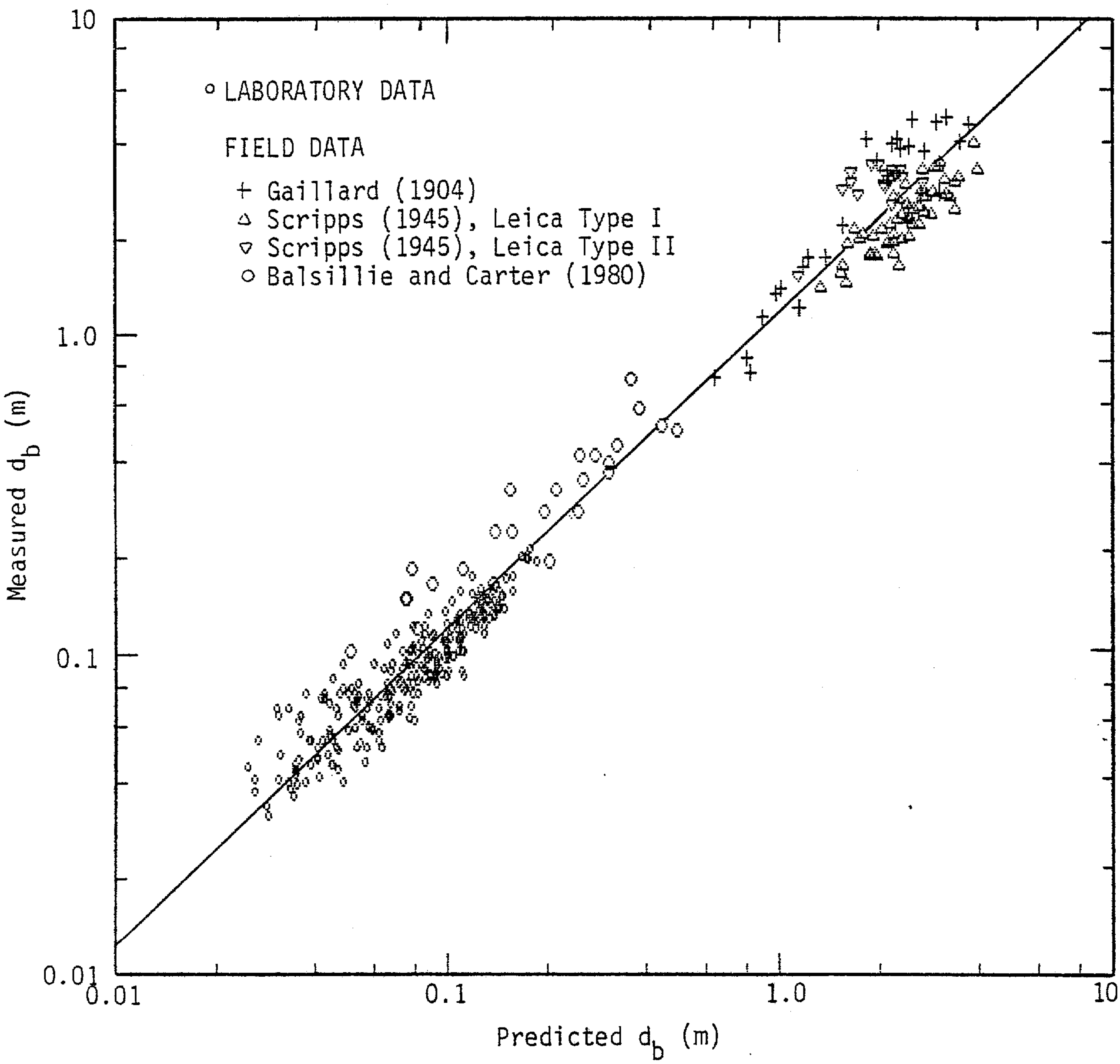

Figure 5. Measured water depth at shore-breaking versus the depth predicted from the modified Weggel relationship given by equation (17). Data include 126 field and 251 laboratory points.

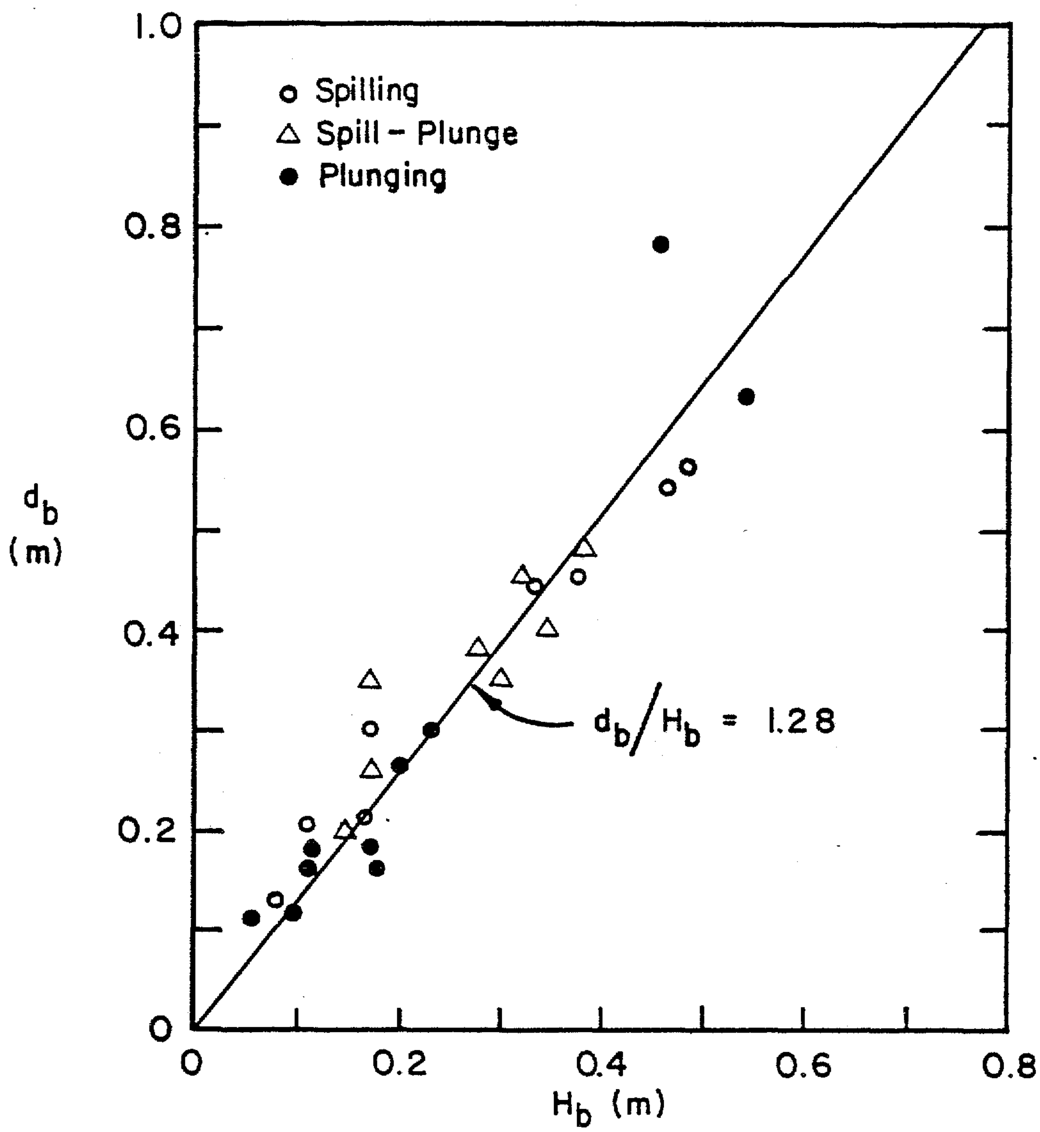

Figure 6. Illustration of the lack of dependence of shore-breaker type on the depth of water at shore-breaking. Data from Balsillie and Carter (1980).

CLOSURE

With the recognition that conditions at shore-breaking are complex and that the difference between d_b and H_b is relatively small, then one may expect a certain amount of inherent variability (which can commonly be significant relative to the small difference between d_b and H_b) and, hence, scatter in the measured data. For this reason, a significantly large sample of data, characterized by a wide range in values, has been used to reassess commonly used relationships for prediction of shore-breaking occurrence. Relationships evaluated proceed from simple to complex incorporating, progressively, the wave height, bed slope, and wave steepness.

The more complex relationships such as that in the form of the corrected Mallard equation which incorporates the wave height and bed slope (valid only where the bed slope is less than about 0.1), and that of the modified Weggel equation which incorporates wave height, bed slope and wave steepness, result in greater scatter, both statistically and graphically, than does the McCowan equation which incorporates the wave height only. This result does not absolutely discount the applicability of the more complex equations (bearing in mind any noted domain limitations). It does, however, on the basis of existing data and its associated variability which may be expected from existing measurement techniques, suggest that until refined measurement methods are found which may be used in both the field and laboratory, the McCowan equation provides the best predictor of d_b/H_b.

as spilling shore-breakers). For plungers the average value of d_b/H_b was 1.15 (n = 70), for non-plungers 1.47 (n = 46), and for the data combined 1.27 (n = 116). Weishar obtained his data from a photographic study in the field. The data base was obtained from three film runs over a one hour period. Assuming that wave conditions may change significantly within a 20-minute period (Balsillie and Carter, in manuscript), then at a minimum the data represent three wave trains. Judging from shore-breaker type frequency plots (Weishar and Byrne, 1979, Figure 6, p. 494) three to four wave trains may have been shore-incident during the experiments. Hence, assuming a maximum of four wave trains, their data may actually represent a maximum of 12 points. In other words, it may have been more reasonable to calculate the average value of d_b/H_b for each wave train, rather than averaging all the data.

The data of Balsillie and Carter (1980) were manually measured by two individuals using a staff at the shore-breaking location (i.e., where the front face of the wave crest was vertical for plunging shore-breakers, and where the top of the wave crest began to break and "foam" for spilling shore-breakers). Crest and trough heights were measured for thirty shore-breakers taking care that the measurements represented a single wave train, from which average values of d_b and H_b were determined. Results for 26 sets of data, representing 780 individual crest-height measurements, are plotted in Figure 6. Spill-plunge shore-breakers occurred where, in the longshore direction, a combination of shore-breaking characteristics were noted to consistently occur along the wave crests. Such mixed shore-breaker type occurrence per wave crest probably corresponded to local alongshore differences in bed slope, and diffraction effects, etc. It is suggested from Figure 6 that d_b/H_b applies equally to shore-breakers regardless of the shore-breaker type.

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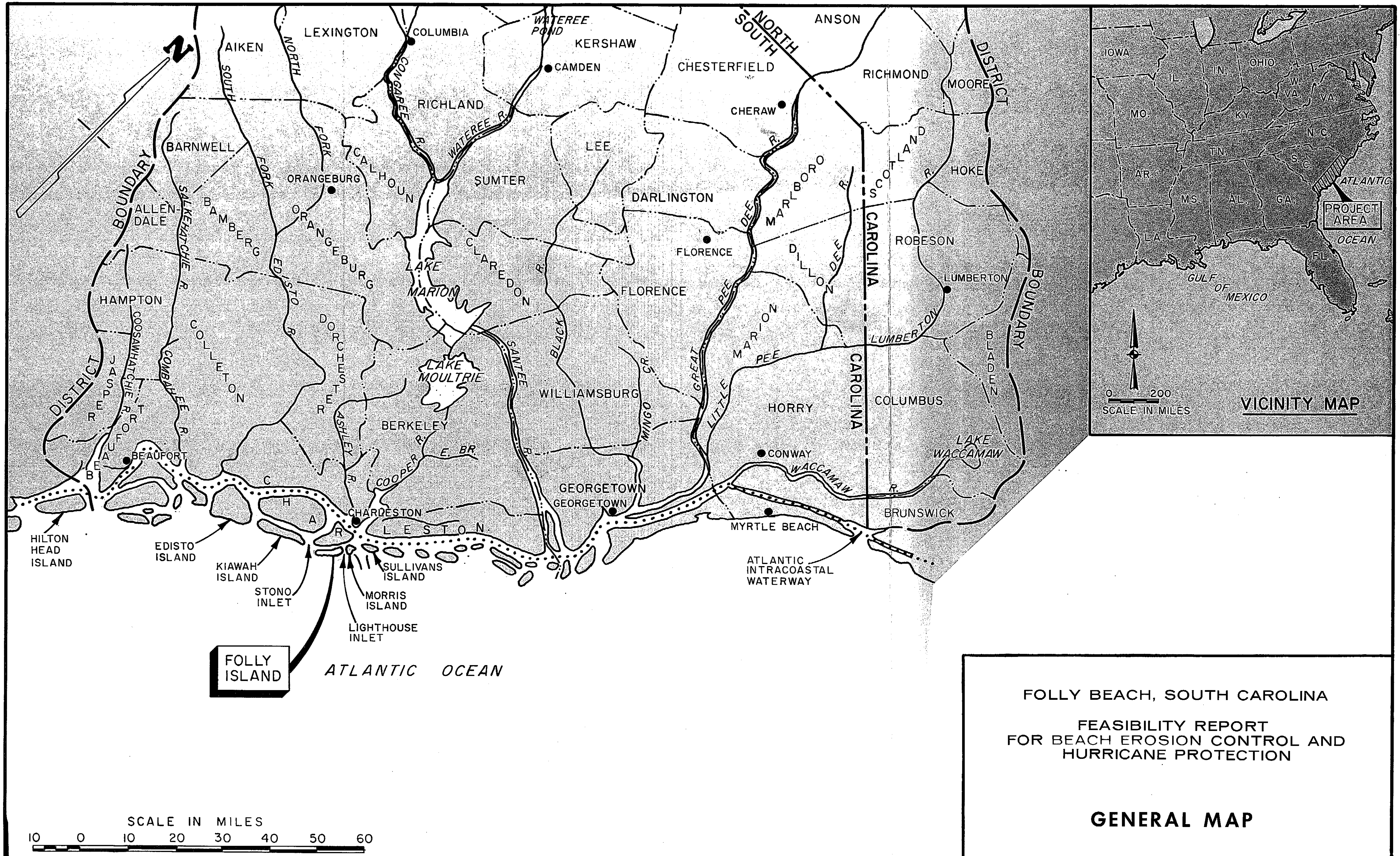

FOLLY BEACH, SOUTH CAROLINA

FEASIBILITY REPORT
FOR BEACH EROSION CONTROL AND
HURRICANE PROTECTION

GENERAL MAP

PLATE 1