ATTENUATION OF WAVE CHARACTERISTICS FOLLOWING SHORE-BREAKING ON LONGSHORE SAND BARS

by

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FOREWORD

The work presents a numerical description of littoral wave mechanics due to shore-breaking induced by the presence of submerged longshore sand bars, and the expected characteristics of reformed waves (i.e., height, period and length) following bar-breaking, should wave reformation be possible. It provides the basic support methodology required in the development of a multiple shore-breaking wave transformation model described in subsequent work.

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 contractual 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

The geometry of littoral profiles may be simply described by a transcendental power curve function (Bruun, 1954; Dean, 1977; Hughes and Chiu, 1978), or by apparently ultimate complexity due to occurrence of longshore bars which cannot be simply described. While longshore bars are produced by waves (water level rise being important), they also are instrumental in influencing post-incident wave characteristics. Available field and laboratory data are used to develop a mathematical description of wave transmission following longshore bar incidence. The description assumes the design case where the longshore bar bedform has been produced or is being maintained by the incident waves which, by definition, must be shore-breaking and in the plunging-type category. Wave height reformation following shore-breaking over longshore bars is given by:

$$\frac{H_r}{H_b} = 0.26\, e^{\tanh 175 \frac{H_b}{g\,T^2}}$$

where H_r is the average reformed wave height, H_b is the shore-breaking wave height over the bar crest, and T is the bar-incident wave period.

The reformed wave period T_r, is given by:

$$\frac{T_r}{T} = \left(\tanh 5 \frac{H_b}{g\,T^2}\right)^{0.18}$$

INTRODUCTION

Longshore bars are found as characteristically permanent or seasonal features located in littoral zones of lakes and oceans. They may occur singly or in a series and are associated with sandy beaches and nearshore areas. A single bar consists of a ridge of sand separated from the shoreline by a trough. A series of longshore bars consists of a number of associated crests and troughs. They are defined (U. S. Army, 1977) to be "... located at the breaker position and, at least in part, are eroded out of the bottom by the falling breaker ...", and they seem to be related to the height of the larger breakers, though not necessarily the highest. Evans (1940) and Shepard (1950) report that plunging shore-breakers are essential for longshore bar formation. Shepard (1950) and Miller (1976) report that longshore bars can be destroyed by spilling shore breakers, although in an extensive field study Hands (1976) observed that practically all incident waves during the study were of the spilling variety and that even under such conditions a well-developed system of longshore bars persisted.

The generation, physiography, and movement of longshore bars have been studied by geographers, geologists and coastal engineers for over a century, resulting in a list of publications far too long to refer to here. Most of the existing work is concerned with the formation and behavior of bars relative to wave and water level changes ... i.e., the effect of waves on the structure. However, in this work attention is focused on the effect that the structure has on the waves, which by comparison has received less attention in the literature.

With the exception of seismic sea waves, wind-generated waves produce the most critical forces to which coastal structures, natural or manmade, may be subjected (U. S. Army 1977). When extreme events such as storms and hurricanes impact the coast, the destructive potential of waves is significantly increased because, propagating upon the super-elevated water surface (i.e., storm surge) accompaning extreme events, the waves impact coastal elevations not normally attained and areas of the coast not in equilibrium with wave forces.

It is here suggested that natural littoral profile shape may be described in terms of the complexity of the geometry. The simplest geometry may be described by a transcendental power curve function (Bruun, 1954; Dean, 1977; Hughes and Chiu, 1978). Dean (1977) demonstrated both theoretically and physically that the smooth, concave upward, equilibrium nearshore profile may be given by:

$$d = a_s \; x^{2/3} \qquad (1)$$

where d is the water depth, x is the distance offshore, and a_s is the shape coefficient for a smooth profile given by:

$$a_s = \left(\frac{24 \; E_{du}}{5 \; \rho_f \; g^{1.5}}\right)^{2/3} \left(\frac{d_b}{H_b}\right)^{4/3} \qquad (2)$$

in which ρ_f is the fluid mass density, g is the acceleration of gravity, E_{du} is the uniform energy dissipation factor, d_b is the water depth at which shore-breaking occurs, and H_b is the shore-breaking wave height. Of the principal design shore-breaker types (i.e., spilling, plunging, and surging waves), the only type which can satisfy the uniform energy dissipation constraint, is the spilling shore-breaker. Hence, where at shore-breaking $d_b/H_b = 1.28$ (McCowan, 1894; Munk, 1949; Balsillie, 1983a), equation (2) becomes:

$$a_s = \frac{20}{3}\left(\frac{E_{du}}{\rho_f \quad g^{1.5}}\right)^{2/3} \qquad (3)$$

Such a profile shape appears to be most representative of long-term littoral profile geometries.

The most complex natural littoral profile geometry is characterized by longshore bar bed form perturbations (profile geometrics such as step profiles are intermediate between the extremes). No tractable mathematical description has been developed for such profiles, although Figure 1 illustrates that where bar crest and bar trough depths are considered, the power curve function appears to provide successful description. In terms of wave activity, barred nearshore profiles differ significantly from smooth power curve profiles since they are: 1. associated with plunging shore-breakers, and 2. characterized by wave reformation.

In terms of the destructive potential of waves, studies have shown that not only do breaking and broken waves result in larger impact pressures than less deformed waves in deeper water, but differ in magnitude depending on the shore-breaker type, increasing from surging to spilling to plunging (Miller et al. 1974a, 1974b; Miller, 1976).

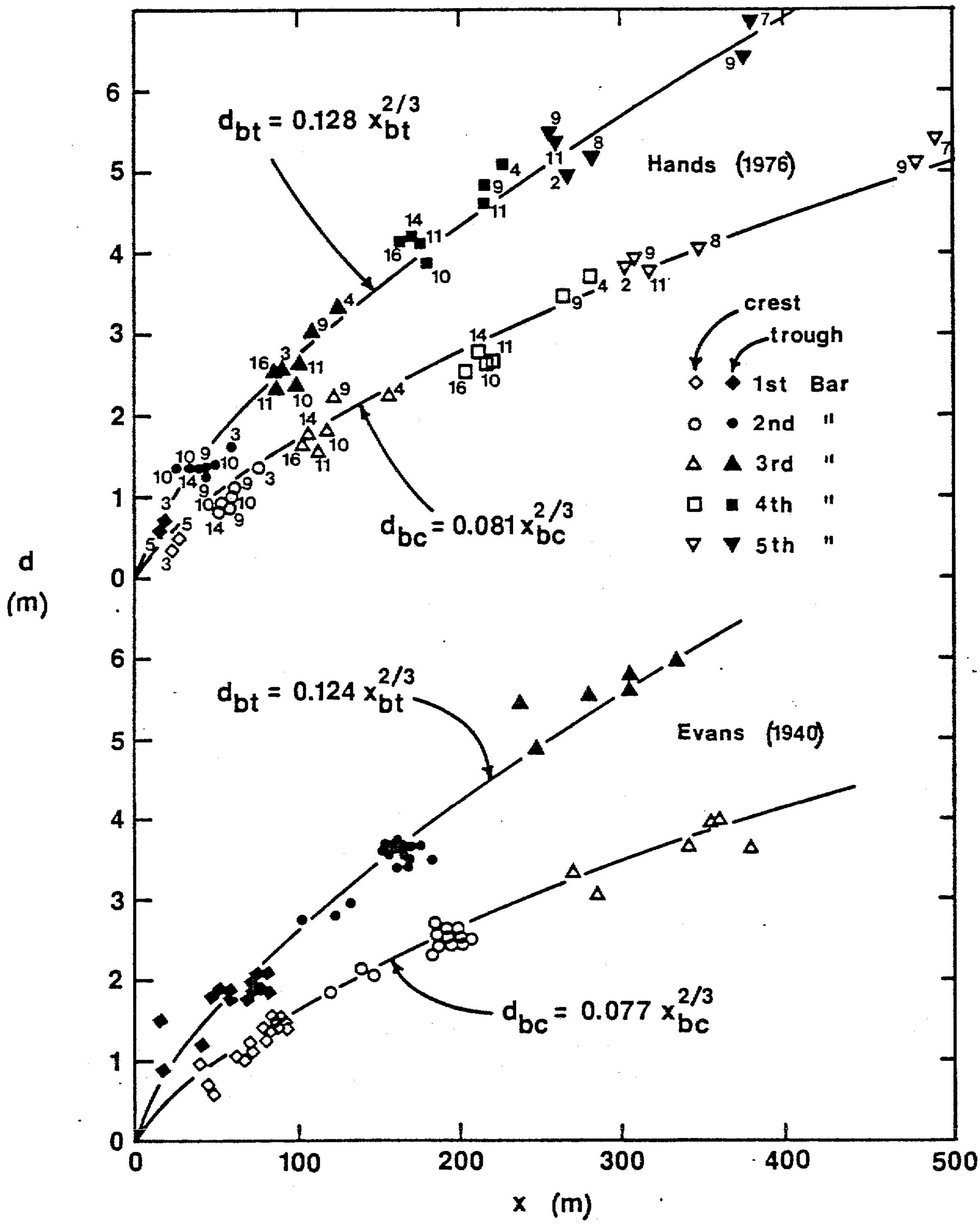

Figure 1. Illustration of power curve fits to longshore bar crests and troughs for multi-barred profiles (d_{bc} is the water depth over the bar crest, d_{bt} is the water depth over the bar trough). Numbers for the data of Hands (1976) refer to averages for each survey.

Wave reformation following initial shore-breaking may occur under certain littoral bathymetric conditions. In an extensive laboratory investigation of wave behavior on fixed linear beds, Nakamura, Shiriashi, and Sasaki (1966a) report that where the bed slope is less than about 0.02 to 0.03, wave reformation shoreward of shore-breaking is likely. If in fact, the slope of the linear segment connecting the longshore bar crest and the shoreline is used as a measure, the criterion appears to be appropriate (Figure 2). Considering, in addition, a similar line to the bar trough to provide further guidance, a limiting slope $\tan \alpha^*$, may be empirically proposed (in S. I. units) according to:

$$\tan\alpha^* \begin{cases} < 0.027\, e^{-0.137\, d_{bc}}, & \text{wave reformation may occur,} \\ > 0.027\, e^{-0.137\, d_{bc}}, & \text{no wave reformation.} \end{cases} \quad (4)$$

where d_{bc} is the water depth over the bar crest.

In view of the differences in behavior of wave activity for the extremes of profile types, it appears that the effect barred littoral profiles on wave activity requires quantification if such activity is to be responsibly accounted for in coastal engineering assessments. Following extensive review of the available literature, a set of mathematical descriptions are herein developed to describe wave transmission following shore-breaking over longshore bars. Parameters include the wave height, period and length.

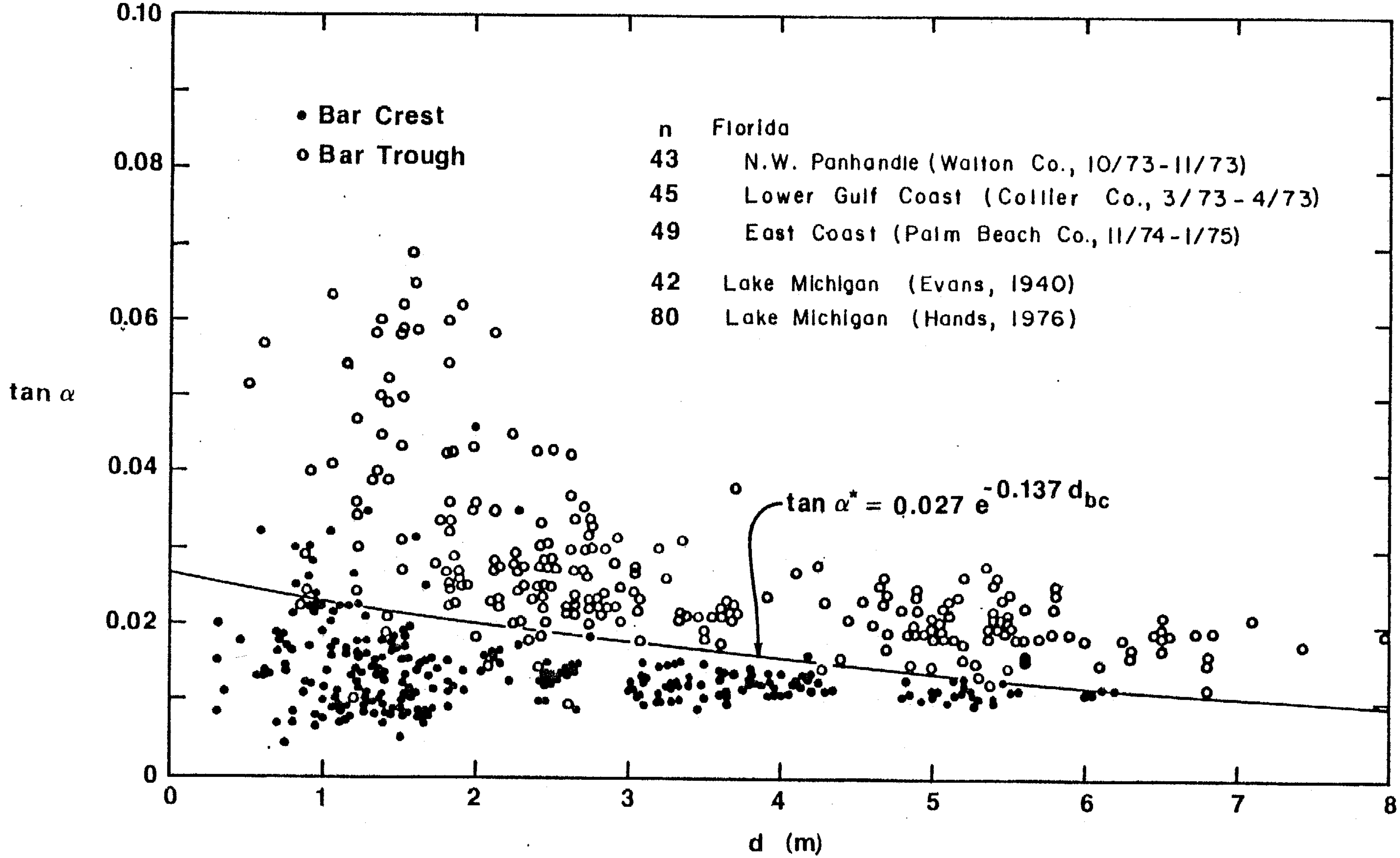

Figure 2. Illustration of the likely slope on which wave reformation will occur following initial shore-breaking, using longshore bar crest and trough data as a guide (d is the water depth over the bar crest or trough, $\tan \alpha$ is the slope of the line connecting the measured depth and the shoreline, and the plotted equation is evaluated in S. I. units).

PREVIOUS WORK

The effect that longshore bars have on attenuating wave activity has received considerable attention in the published literature. It is not suprising, therefore, that following nature's lead, consideration has also been focused on less mobile submerged barriers that can be designed to reduce the destructive potential of anticipated nearshore wave activity.

Prior to any definitive information on the effect of submerged barriers to reduce wave activity, it is understandable that prognostications were made using available theory. Early theoretical studies have been published by Jeffreys (1944) for submerged rectangular barriers; Dean (1945), Ursell (1947) and John (1948) for submerged plane vertical barriers; Lamb (1945) for a step-type profile; and Heins (1950) for submerged plane horizontal barriers (see Johnson, Fuchs, and Morison, 1951, for a more detailed discussion).

In the interim, however, somewhat more definitive information has become available. Nevertheless, because of the difficulty in collecting field data at the time of longshore bar formation, most field accounts are descriptive. Laboratory investigations have been conducted which attempt to replicate the effect of naturally barred nearshores. Other laboratory studies are concerned with fixed, immobile submerged barriers as a design alternative to protect the coast. The review of previous work is, therefore, considered separately in terms of field investigations and laboratory studies.

Field Studies

Byrne (1969) reports two sets of field observations of waves passing over a barred nearshore on Cape Cod. While the waves did not shore-break on the bar, they were affected by the shoaling bathymetry of the bar (i.e., they peak-up as if to shore-break), and the number of waves shoreward of the bar was double the number incident to the bar. He reports that after the wave crest passes the bar crest, the first part of the following trough becomes depressed over the shoreward flank of the bar resulting in secondary gravity wave crest formation.

Wood (1970, 1971) conducted field monitoring studies on Lake Michigan in which nine wave gages were placed across the littoral zone. For the longshore bar investigation, four wave gages were located on the stoss slope of the bar, and five gages between the bar crest and shoreline. Over 400 consecutive waves were measured, with the dominant breaker type being spilling. The bar crest was located 0.5 m below the still water level (*SWL*), and the bar trough 1.2 m below the *SWL*.

Suhayda and Roberts (1977) present a relationship between reformed and incident waves breaking over fringing coral reefs:

$$\frac{H_r}{H_i} = 1.0 - 0.8\, e^{-0.6\, d_c/H_i} \qquad (5)$$

where H_r is the reformed wave height, H_i is the incident wave height near breaking, and d_c is the water depth over the barrier crest. They also report that the average wave period landward of the reefs "... is smaller by about 50% to 75% than the wave period offshore ... " of the reefs.

Keady and Coleman (1980) inspected vertical aerial photography from North Carolina and northern California which depict shore-breaking over nearshore sand bars. They report the ratio of the reformed wave length to the wave length near bar-breaking, L_r/L_i, to range from 0.12 to 0.5 with estimated values of the ratio of the reformed to incident wave height, H_r/H_i, ranging from 0.25 to 0.5.

Dette (1980) records bathymetric, wave height and period conditions during a 1978 storm which struck the Kiel Bight coast on the Baltic Sea. A multi-barred profile and wave height attenuation following bar-breaking are reported.

Carter and Balsillie (1983) report on wave energy attenuation over nearshore sand bars at localities in Florida and Northern Ireland. The data are reported as shore-breaker heights, from which it is suggested (assuming wave energy to be directly proportional to the square of the wave height):

$$\frac{H_{br}}{H_{br}} = 0.05 + 0.18 \log \left(1.0 + \frac{c_{bi}}{w_{bi}}\right) \qquad (6)$$

where H_{bi} is the incident shore-breaker height over the bar, H_{br} is the reformed height at secondary shore-breaking, w_{bi} is the width of the incident breaker zone, and c_{bi} is the wave speed at incident bar-breaking.

Laboratory Studies

The first laboratory study known to the author was conducted by Stucky and Bonnard (1937, 1938) to test the anticipated construction of a submerged breakwater at Oporto, Portugal. Results of their work (10 experimental values) are discussed by Hall (1939, appendix II).

The first detailed study on the subject was conducted by Hall (1939) for three submerged barrier configurations (Figure 3). Results for 64 experimental runs are reported, providing height, period and length of the undeformed incident waves, and the height and length of the reformed waves. He discusses the effect that submerged barriers have on attenuating wave height, and reports the wave length to remain conserved.

Mason and Keulegan (1944) investigated the effect of a step-type profile on wave transmission for military landing craft operations. They found that:

$$\left(\frac{H_o \quad L_o}{4 \quad d_c^2}\right)^{0.5} \begin{cases} < 1.0, & \text{multiple wave crests will not form shoreward of the step} \\ > 1.0, & \text{multiple wave crests will form shoreward of the step.} \end{cases} \qquad (7)$$

where H_o and L_o are the deep water wave height and length, respectively. Their data show that shore-breaking need not necessarily occur for multiple wave crest formation. Equation (7) was further verified by Horikawa and Wiegel (1959). The results of Mason and Keulegan primarily deal with wave length attenuation. The author has fitted an equation to their data (80 experimental results) to yield:

$$\frac{L_r}{L_o} = \left(\tanh 4.3 \quad \frac{d_c}{L_o}\right)^{0.38} \qquad (8)$$

Putnam (1945) reports results for three incident wave

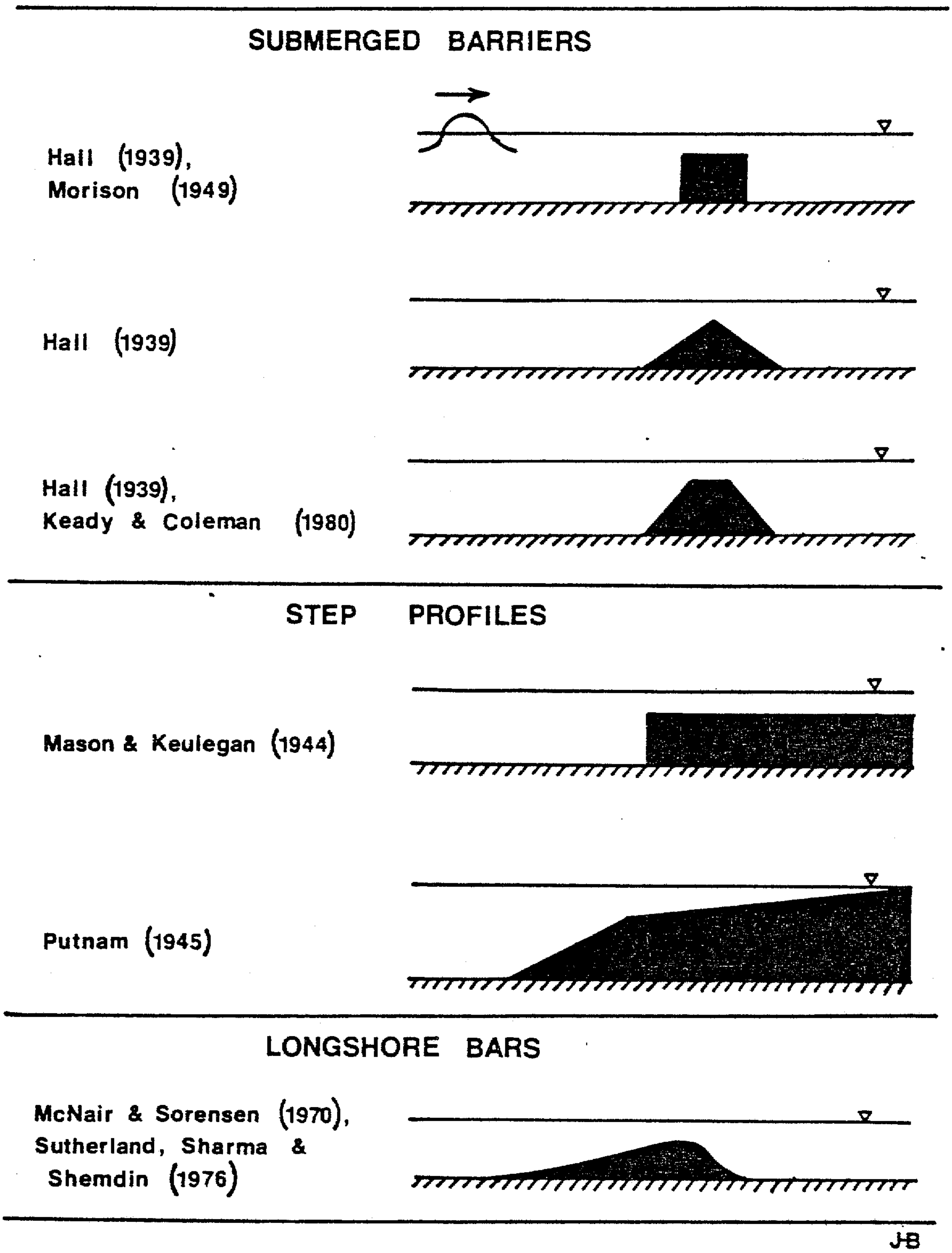

Figure 3. General types of fixed laboratory profile configurations considered in the present study.

conditions over a step profile. These data are particularly valuable since the continuous measurement of the transforming waves not only provides undeformed incident and reformed wave characteristics, but also wave characteristics at the shore-breaking position.

Morison (1949) conducted a series of experiments for rectangular submerged barriers on horizontal and sloping beds. He reports that wave height transmission is a function of barrier height and width, and incident wave steepness. The study is particularly important since photographs are presented which clearly identify the breaker type where breaking occurred over the barriers.

The first attempt at comprehensive review of submerged barriers and effect on wave height transmission is presented by Johnson, Fuchs, and Morison (1951), formally presenting the work of Morison (1949). Wave height transmission is reported to be dependent on the structure height and barrier width relative to incident wave conditions. For shallow water they suggest:

$$\frac{H_r}{H_i} = \left(1.0 - \frac{h_{bc}}{d}\right)^{0.5} \qquad (9)$$

where h_{bc} is the barrier height and d is the water depth measured at the seaward toe of the structures. Equation (9) is applicable where $d_{bc}/(g\ T^2) < 0.00155$ on a step profile. Equation (9) fails, however, where $h_{bc}/d = 1.0$ (i.e., the barrier crest is coincident with the water surface) for which $H_r/H_i = 0$, a condition known to be untrue. Unfortunately, Johnson, Fuchs, and Morison did not present any of their data in other than dimensionless terms.

Diephuis (1957) investigated wave height and period attenuation following incidence on a fixed bar-type feature, including several cases where the shore-breaker type can be determined. Unfortunately, the magnitude of his wave characteristics were in the range where surface tension and viscous effects further accentuated attenuation of the reformed height and period.

In an investigation on shore-breaking on fixed, linear slopes, Nakamura, Shiriashi and Sasaki (1966a) found that where the slope, tan α_b, was less than about 0.02 to 0.03, wave reformation could occur following initial shore-breaking. The author has fitted equations to their data to yield the following relationships:

$$x_r = 26.1 \; H_o \qquad (10)$$

where x_r is the distance to wave reformation following initial shore-breaking,

$$\frac{T_r}{T} = \left(\tanh 100 \; \frac{d_b}{L_o}\right)^{0.44} \qquad (11)$$

where T is the incident wave period, T_r is the reformed wave period and d_b is the water depth at shore-breaking, and:

$$\frac{L_r}{L_b} = \left(\tanh 100 \; \frac{d_b}{L_o}\right)^{0.21} \qquad (12)$$

where L_b is the wave length at shore-breaking. Equations (11) and (12) are for a linear bed slope of tan α_b = 0.010.

Nakamura, Shiriashi and Sasaki (1966b) also investigated wave transmission over fixed rectangular barriers on a horizontal bed. They found that the ratio of the reformed to incident undeformed wave height, H_r/H_i, depends on the ratio of the barrier width to the incident wave length, l_c/L_i, and to the ratio of water depth over the barrier crest to the incident wave height, d_c/H_i. The author has fitted equations to their data to yield the following:

$$\frac{H_r}{H_i} = \Phi_1 \, e^{\Phi_2 \tanh \Phi_3 \; d_c/H_i} \tag{13}$$

where:

$$\Phi_1 = 0.312 \, e^{-1.082 \; l_c/L_i} \tag{14}$$

and

$$\Phi_2 = 1.18 + 1.087 \; l_c/L_i \tag{15}$$

and

$$\Phi_3 = 0.677 - \left[0.183 \tanh (2.4 \; l_c/L_i) \right] \tag{16}$$

and

$$\frac{T_r}{T} = \left(\tanh 0.5 \; \frac{d_{bt}}{L_o} \right)^{0.18} \tag{17}$$

and

$$\frac{L_r}{L_o} = \left(\tanh 0.75 \; \frac{d_{bt}}{L_o} \right)^{0.45} \tag{18}$$

where d_{bt} is the water depth just landward of the barrier crest.

Dick (1968) found that where waves break on a submerged breakwater, the period of the reformed wave was the same as the incident period, although other reformed crests of lesser height and of higher frequencies were superimposed on the fundamental reformed wave.

McNair and Sorenson (1970) investigated wave transformation of regular waves incident to a typical fixed bar geometry, which they describe as characteristically having a gently sloping stoss flank, rounded crest and steep lee slope. Results of 35 experiments are reported where the incident waves are relatively undeformed. Upon reformation following bar-incidence, the waves are reported to be irregular, introducing complications that required special considerations to determine the reformed wave heights. It is reported that, in general, the wave period did not change following bar-breaking, although in all reformed waves a secondary energy peak occurred that was consistently twice the frequency of that associated with the incident wave. They conclude that H_r/H_i is dependent primarly on the incident wave height and the water depth over the bar crest.

Sawaragi and Iwata (1974) investigated shore-breaking on a step profile, including wave height attenuation and various post-shore breaking milestone measures. Equations fitted to their data by the author are:

$$x_{td} = 4 L \frac{H_o}{L_o} \qquad (19)$$

$$x_v = 5\ L\ \frac{H_o}{L_o} \qquad (20)$$

$$x_s = 7.5\ L\ \frac{H_o}{L_o} \qquad (21)$$

and

$$x_a = 26\ L\ \frac{H_o}{L_o} \qquad (22)$$

where x_{td} is the distance from shore-breaking to the point where the plunging vortex (i.e., curl of the plunging shore-breaker) touches down onto the water surface fronting the breaking crest, x_v is the distance from shore-breaking to the point where the plunging vortex reaches the bed, x_s is the splash distance and x_a is the distance where air bubbles disappear from the water column.

Sutherland, Sharma, and Shemdin (1976) modeled a typical Florida east coast profile to investigate runup occurring where waves first encounter a longshore bar. Wave height reduction following bar incidence is reported.

Battjes and Janssen (1978) illustrate two examples of waves passing over a bar-type profile, and the resultant wave height reduction.

Keady and Coleman (1980) investigated the effect of a trapezoidal submerged barrier (on a horizontal bed) on attenuating the wave height. Their study was limited to waves which bar-broke as plunging shore-breakers, and suggest:

$$\frac{H_r}{H_i} = 0.58 \left(\frac{H_i}{d_c}\right)^{-0.8} \qquad (23)$$

Equation (23) suffers, however, from a similar problem facing equation (9) ... when $d_c = 0$, the value of H_r/H_i represents a non-real value. They also report that a greater number of reformed wave crests are produced as the value of d_c is decreased, and as d_c increases the reformed wave appears as two crests. For the latter case, one of the crests becomes higher than the other, and changes position on the fundamental wave form as the wave progresses.

SOLITONS AND SHOALING-INDUCED WAVE DISPERSION

Under certain conditions a shoaling wave can decompose into a train of waves (i.e., multiple crest production) without the wave first breaking. Galvin (1970) studied waves generated in a uniform depth of water where the waves were shoaling (i.e., $d/H \simeq 1.15$ and $d/L \simeq 0.0234$). He found that the wave broke down into several waves called solitons. The same result can apparently be produced over a step profile. The train of waves produced are characterized by a highest first formed crest followed by crests of decreasing height, and if solitons can propagate a sufficient distance the crests can recombine and again separate into solitons. Other studies on the phenomenon (Camfield, 1980) are reported by Horikawa and Wiegel (1959); Benjamin and Feir (1967); Street, Burgess and Whitford (1968); Madsen and Mei (1969); Zabusky and Galvin (1971); Chandler and Sorensen (1972); and Hammack and Segur (1974).

Byrne (1969) noted secondary wave crest formation following longshore bar incidence in the field, in which breaking did not first occur. Some studies have suggested that the observation exemplifies soliton formation. Equation (7), developed by Mason and Keulegan (1944), is one measure that can be used (see discussion of Camfield, 1980) to determine when multiple crest formation can occur, and does not necessarily represent conditions where shore-breaking occurs. Hence, it is apparent that three classes of conditions occur when waves encounter a longshore bar: 1. shoaling water depths are great enough that the waves are not affected, 2. water depths are in the range where the waves do not shore-break, but do disperse, and 3. water depths are limiting and shore-breaking results. It is this latter case, viewed as a design condition in littoral processes, that defines the focus of attention in this paper.

THE LONGSHORE SAND BAR

It is apparent from the preceding sections that transformation of wave characteristics following submerged barrier incidence is dependent on many factors. Principal factors are the water depth over the barrier crest, d_{bc}, barrier height measured at the seaward toe of the barrier, h_{bc}, barrier crest length, l_{bc}, barrier stoss slope, tan α_{bs}, water depth landward of the barrier crest, d_{bt}, and incident wave characteristics H, L, T, with combinations thereof, such as the wave steepness. However, in this paper only natural longshore bars composed of sand-sized sediment are considered, and conditions may be simplified.

Natural longshore sand bars are produced by shore-breaking waves of the plunging type, and water depth limitations can be identified. The ratio of water depth to shore-breaker height d_b/H_b, has been found (Balsillie, 1983a) to be accurately represented by:

$$\frac{d_b}{H_b} = 1.28 \tag{24}$$

illustrated in Figure 4. Equation (24) does not, in all probability, represent the water depth over the bar crest apex. Rather, the average breaking position undoubtably lies some distance seaward of the crest apex, over the stoss slope. Field measurements of shore-breaking wave activity for single wave trains and associated surf zone widths were obtained by Balsillie and Carter (1980) and suggest (Figure 5) that the surf zone width, w_b, may be given by:

$$w_b = \frac{H_b}{\tan \alpha_b} \tag{25}$$

Additionally, Balsillie and Carter (1984) found that for a shore-breaking wave train the average shore-breaker height and the standard deviation, s_b, associated with the average can be related according to:

$$\frac{s_b}{H_b} = 0.21 \tag{26}$$

illustrated in Figure 6. Assuming that $\pm 2\ s_b$ represents the distribution of shore-breaker heights for a wave train, then from equations (24) and (26):

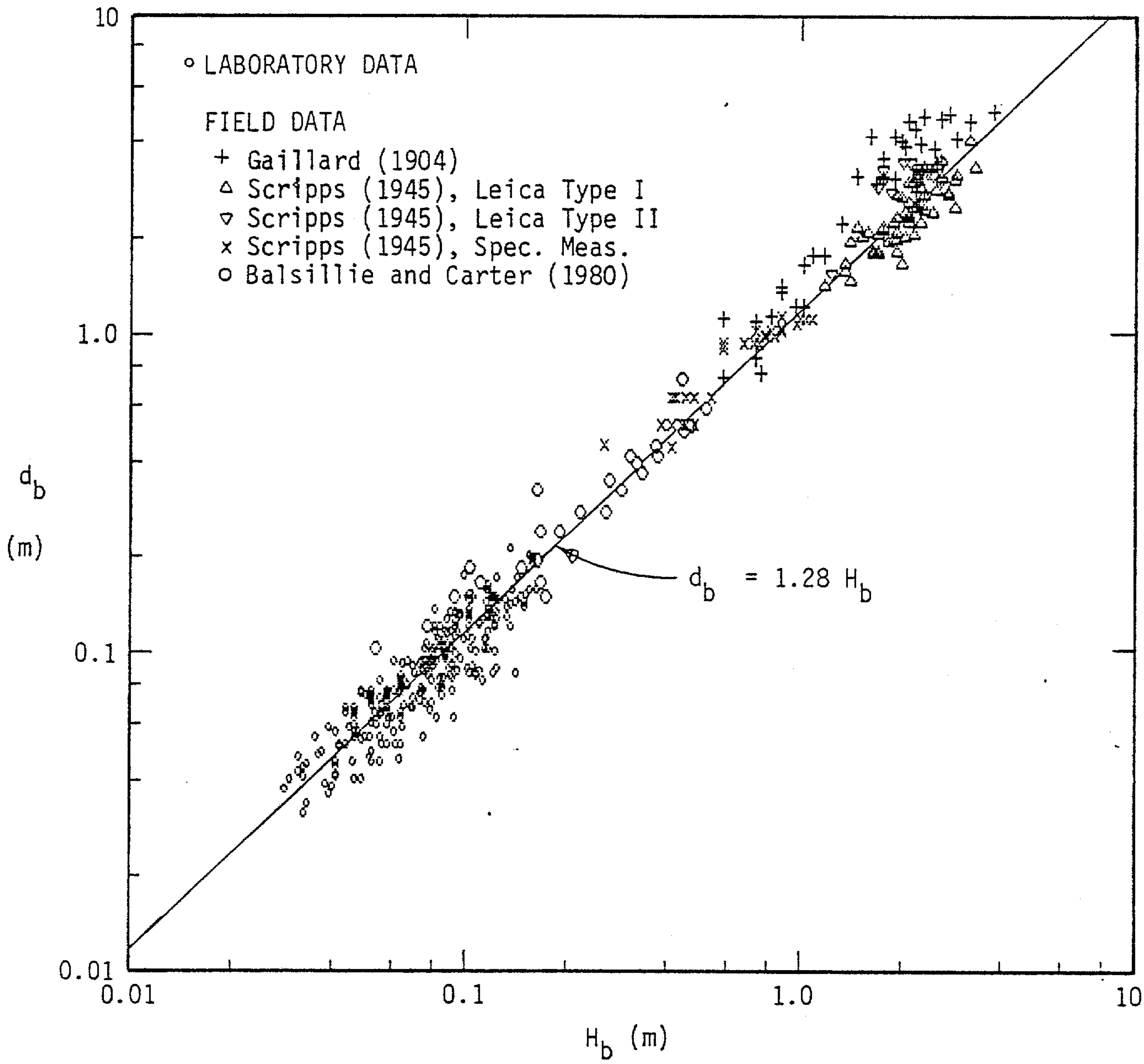

Figure 4. Relationship between the water depth at shore-breaking, d_b, and the average shore-breaking wave height, H_b (from Balsillie, in manuscript; Scripps data reported by Munk, 1949).

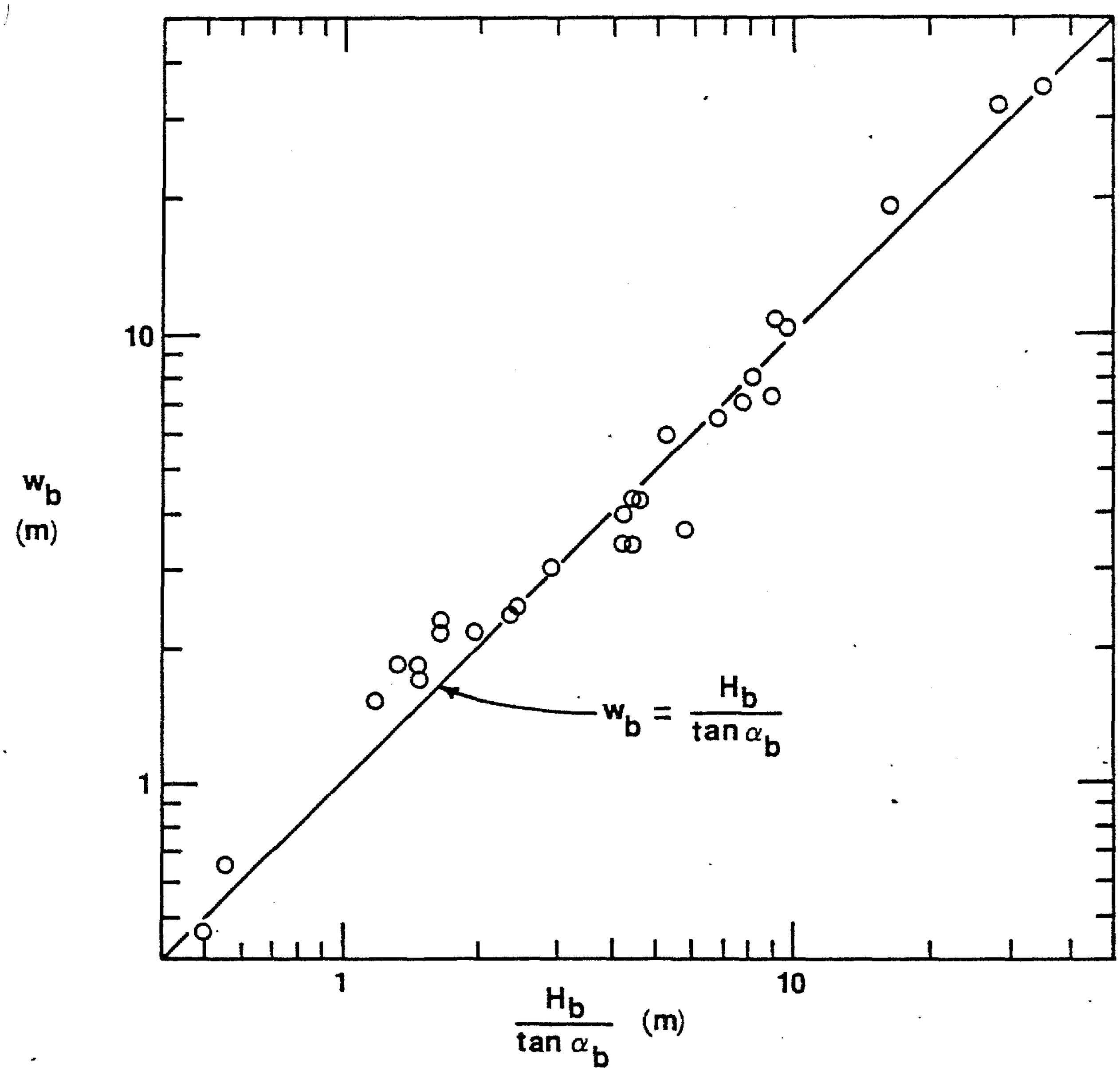

Figure 5. Relationship between the width of the shore-breaking zone, w_b, and the average shore-breaking wave height, H_b, and bed slope tan α_b.

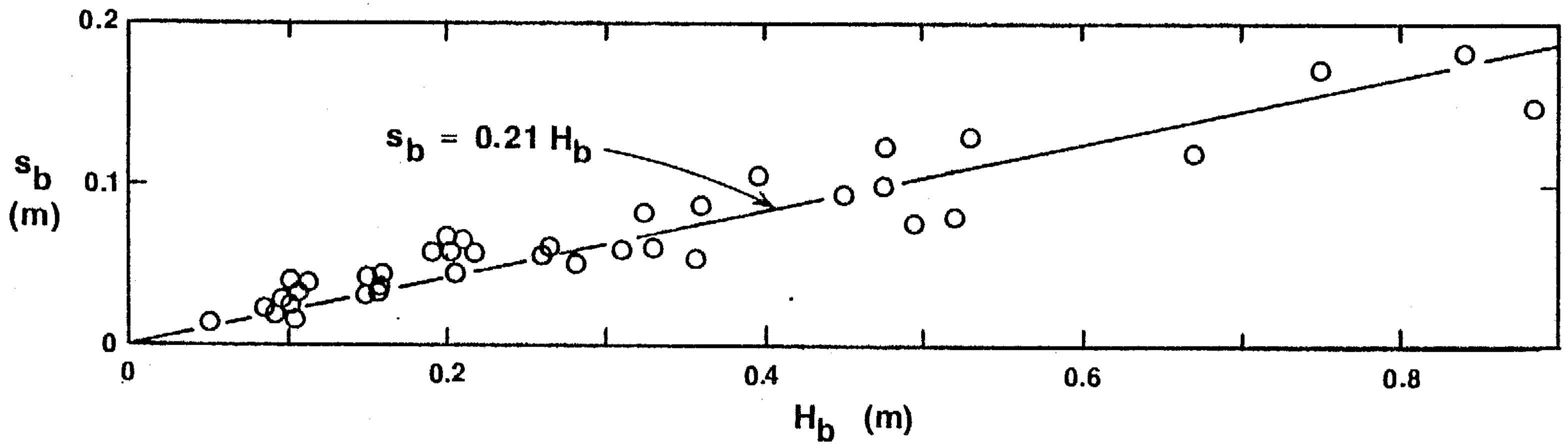

Figure 6. Relationship between the average shore-breaking wave height, H_b, and the standard deviation of the average height, s_b, for single shore-breaking wave trains (from Balsillie and Carter, in manuscript).

$$w_b = \frac{1.28 \left[(H_b + 0.42\ H_b) - (H_b - 0.42\ H_b) \right]}{\tan \alpha_b}$$

$$w_b = 1.08 \frac{H_b}{\tan \alpha_b} \qquad (27)$$

which is in agreement with equation (25).

Bruun (1963) reports that many waves "... seem to slip over the bar unbroken or without making a complete break ...", and that "during the peak of storm wave activity, it appears that 50 per cent of all waves break." He assumed that the depth over the bar crest is approximately equal to 0.8 H_{bs}, where H_{bs} is the significant shore-breaker height. Where at shore-breaking $H_{bs} = 1.23\ H_b$, illustrated in Figure 7 (from Balsillie and Carter, 1984), then:

$$\frac{d_{bc}}{H_b} = 0.98 \qquad (28)$$

Assuming that 50% of the waves from a single wave train generating a longshore bar shore-break over the crest as plungers, the corresponding breaker zone width over the bar crest, w_{bc}, becomes from equation (25) 0.5 $H_b \tan^{-1} \alpha_{bs}$. The average point of shore-breaking is therefore located a distance of 0.25 w_{bc} seaward of the crest apex. Hence, using equation (24):

$$\frac{d_{bc}}{H_b} = 1.03 \qquad (29)$$

which is close to equation (28). These several lines of reasoning suggest, then, that the depth of water over the long-

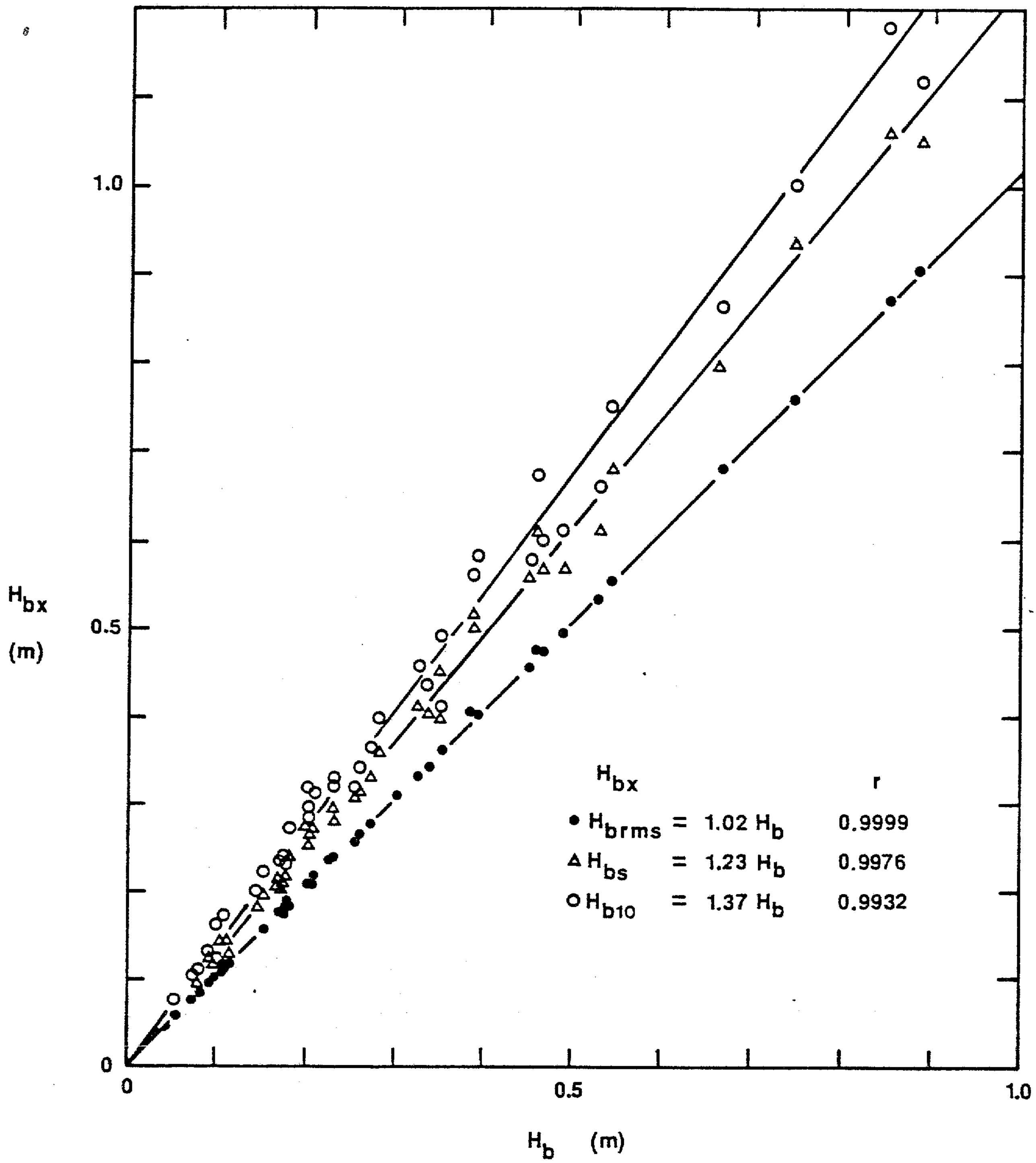

Figure 7. Relationship between statistical wave height measures, H_{bx}, and the average wave height, H_b, for single shore-breaking wave trains; H_{brms} is the root-mean-square shore-breaker height, H_{bs} is the significant shore-breaker height, and H_{b10} is the average of the highest 10% of shore-breaker heights (from Balsillie and Carter, in manuscript).

shore bar crest apex is equal to the average shore-breaker height occurring over the bar slope (i.e., $d_{bc} = H_b$).

Based on the work of Battjes (1974a , 1974b) and Bruun and Gunbak (1977), it is also possible to specify the stoss slope required to produce plunging shore-breakers. Based on recent work (Balsillie, in press), modified their results to suggest:

$$\xi_b = \left(\frac{g\ T^2\ \tan^2\alpha_{bs}}{H_b}\right)^{0.5} \begin{cases} < 0.64, \text{ Plunging,} \\ 0.64 \text{ to } 5.0, \text{ Spilling,} \\ > \ 5.0, \text{ Surging} \end{cases} \qquad (30)$$

Where ξ_b is the modified surf similarity parameter. Further, Balsillie (in press) found an optimum plunging value occurs where $\xi_b = 1.0$ (corresponding to about 76% of the top of the breaker crest comprising the plunging curl). Hence, a representative stoss slope, tan , required to produce a well formed plunger is given by:

$$(\tan\ \alpha_{bs})_{req} \quad = \quad \left(\frac{H_b}{g\ T^2}\right)^{0.5} \qquad (31)$$

The depth of the bar trough, d_{bt}, just landward of the bar crest can also be identified. Based on over 1000 field, and laboratory measurements (Table 1), and giving equal weight to field and laboratory averages (since laboratory data represent conditions at the time of longshore bar formation, although scaling errors may be included), and where from above $d_{bc} = H_b$,

Table 1. Relationship between the water depth over the longshore bar crest and the bar trough just shoreward of the crest.

I. D.	Locations/Conditions	n	$\frac{d_{bt}}{d_{bc}}$
FIELD DATA			
Lehman (1884)*	Pomeranian Coast	4	1.61
Otto (1911)*	Pomeranian Coast	5	1.76
Hartnack (1924)*	Pomeranian Coast	2	1.82
Evans (1940)	Lake Michigan	43	1.45
Shepard (1950)	Scripps Pier	276	1.23
	California	66	1.63
	Oregon and Washington	38	1.93
	West Coast U. S. (other)	116	1.63
	Cape Cod, Mass.	62	1.47
Hands (1977)	Lake Michigan	162	1.50
Herbich (1978)	Texas	27	1.37
Present Study	Florida		
	Lower East Coast (1975)	49	1.64
	Panhandle Coast (1973)	43	1.93
	Panhandle Coast (1981)	85	1.66
	Lower Gulf Coast (1973)	45	1.53
Field Total and Weighted Average		1023	1.50
LABORATORY DATA			
Keulegan (1948)	Initial Slope: 0.0667	16	1.17
	0.0333	15	1.72
	0.0200	5	1.60
	0.0143	9	1.69
Scott (1954)		4	1.89
Laboratory Total and Weighted Average		49	1.73

* Reported by Keulegan (1948).

then:

$$\frac{d_{bt}}{d_b} = \frac{d_{bt}}{H_b} = 1.6 \tag{32}$$

Two additional longshore bar measures are of special concern. These are the length of the bar lee slope, x'_{bt}, measured from the bar crest landward to its companion bar trough, and where a multi-barred nearshore occurs the longshore bar spacing, x_{bcs}, measured from bar crest to bar crest. Significantly large samples of field measurements reported by Evans (1940) and Hands (1976) for multi-barred nearshores along the eastern shore of Lake Michigan are plotted in Figures 8 and 9. While the longshore bars were apparently not formed at the time that field measurements were made, sufficient data are reported to provide averages thereby reducing the effects of secondary sedimant redistribution due to other modifying influences. The equations are:

$$x_{bcs} = 42\, d'_{bc} = 70\, d''_{bc} \tag{33}$$

where d'_{bc} and d''_{bc} are defined in Figure 9, and

$$x'_{bt} = 14.4\, d_{bc} \tag{34}$$

Therefore, it is possible to predict the principal physiographic features of longshore sand bars in terms of incident shore-breaking wave characteristics.

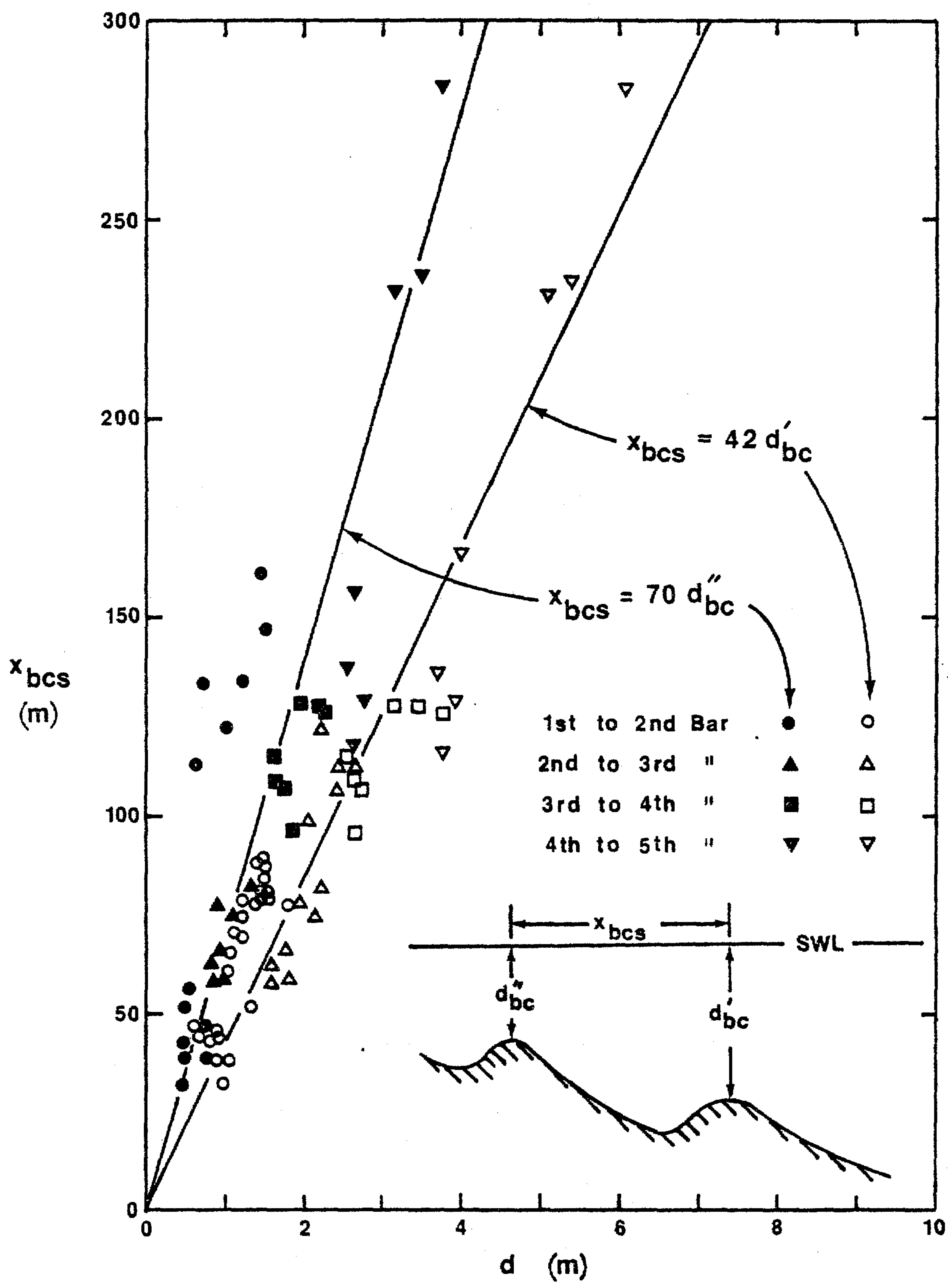

Figure 8. Relationship between the water depth over the bar crest, d_{bc}, and the bar crest spacing, x_{bcs}, for multi-barred profiles (data from Evans, 1940; and Hands, 1976).

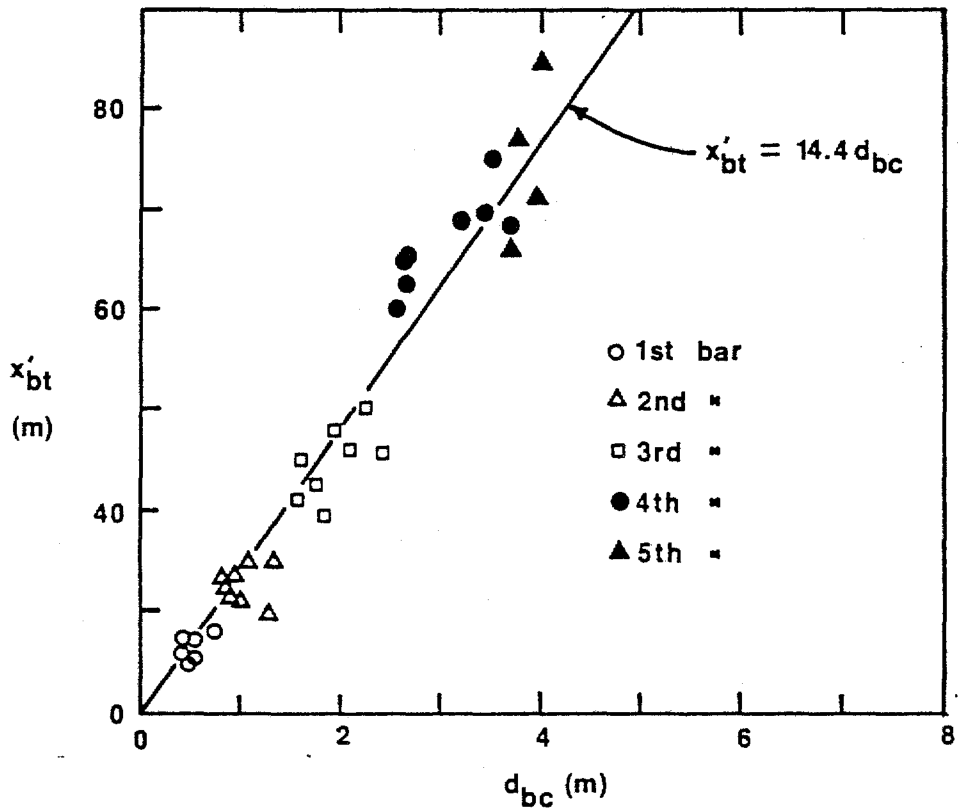

Figure 9. Relationship between the water depth over the bar crest, d_{bc}, and the distance to the bar trough landward of the crest, x'_{bt} (data from Hands, 1976, points represent averages as indicated in Figure 8).

WAVE HEIGHT ATTENUATION

Existing accounts suggest that the relative height of the reformed wave following initial shore-breaking primarily depends on the ratio of the water depth over the submerged structure to a measure of the incident wave height d_c/H_i (e.g., Nakamura, Shiriashi and Sasaki, 1966b; McNair and Sorensen, 1970; Suhayda and Roberts, 1977; and Keady and Coleman, 1980). However, in the preceding section it has been suggested that for natural longshore bars the ratio d_{bc}/H_b has an average value of unity, where H_b is the shore-breaking wave height producing or in near equilibrium with the bed form. When applied to existing relationships, consistency results. For example, the relationship of Suhayda and Roberts, given by equation (5) results in H_r/H_b = 0.56. The relationship developed from the data of Nakamura, Shiriashi and Sasaki, given by equation (13), results in H_r/H_b = 0.54 (assuming $l_c/L_i \simeq l_c/L_b$= 0.173, where from the preceding section and incorporating equation (32), $l_c = 0.5\ w_b = 0.22\ T\sqrt{g\ H_b}$, and from Balsillie (1984) $L_b = T\sqrt{1.6\ g\ H}$). The relationship of Keady and Coleman, given by equation (23) results in H_r/H_i = 0.58; and the relationship of Johnson, Fuchs and Morrison yields H_r/H_i = 0.59 (assuming from the averaged field data of Hands, 1976, figure 29, that $h_{bc} \simeq 1.37$ R where R is the bar relief with R = 0.5 d_{bc}, and $d = h_{bc} + H_b$). These relationships result in an average value of $H_r/H_i \simeq H_r/H_b \simeq 0.59$. Further study shows, however, that additional factors affect wave height transmission, which can be identified to further refine the predictive relationship as it applies to natural longshore bars.

If one discounts h_{bc}, introduced earlier, since its evaluation is not applicable for natural littoral conditions and because it can easily be expressed in other terms, then T, H_b, L_b, d_{bc}, tan α_{bs}, d_{bt} and M_ϕ are the principal factors on which H_r/H_b should be dependent. Furthermore, the average grain size of sediment, M_ϕ, may be eliminated since we are dealing with sand, the value of d_{bc}/H_b is close to unity, the value of d_{bc}/d_{bt} is approximately 1.6, and the magnitude of tan α_{bs} must be sufficient to produce the wave type necessary for longshore bar formation and maintenance. The remaining factors, then, are T, H_b and L_b. The most basic remaining parameter is, therefore, the wave steepness H_b/L_b, or equivalent wave steepness $H_b/(g\ T^2)$.

As noted earlier, for various reasons, few field data are available to quantify the effect that longshore bars have on transforming incident waves. Therefore, while all available field data are used in this investigation, the data are augmented by available laboratory results which appear to approximate conditions exhibited by natural longshore bars. Based on considerations set forth earlier, laboratory data selected were required to meet the following conditions: 1. incident waves shore-break on the barrier crest or barrier stross slope, 2. $0.6 \leq d_{bc}/H_b \leq 1.4$ (except where plunging-type breakers were known to occur over the barrier crest), and 3. assuming the waves shore-break as plungers, the touch-down point was always landward of the barrier crest apex. All conditions were tested numerically where photographic evidence was

Table 2. Field and laboratory data for wave height transmission over bar-type submerged barriers.

I. D.	$\frac{d_{bc}}{H_b}$	$\frac{H_r}{H_b}$	$\frac{H_b}{g\,T^2}$	tan α_{bs}	Breaker Type	$\frac{d_{bt}}{d_{bc}}$
FIELD DATA -- all natural, sandy beaches.						
Wood (1970,1971)	1.250	0.45	0.00349	0.0556	Sp	2.400
Dette (1980) storm waves	---	0.55	0.00567	0.012	Sp**	---
Balsillie and Carter (1980)	1.505	0.451	0.00108	0.0011	Sp/Pl	6.471
	1.072	0.288	0.00170	0.0017	Sp	1.979
	1.026	0.360	0.00222	0.0509	Sp	1.233
	1.101	0.229	0.00193	0.0968	Sp/Pl	1.491
	1.344	0.284	0.00055	0.0375	Pl	2.042
	1.143	0.650	0.00990	---	Sp	---
LABORATORY DATA -- all fixed beds.						
Hall (1939)	0.689*	0.390	0.00110	0.6667	Su**	11.000
	0.981*	0.356	0.00183	"	Su**	6.000
	0.771*	0.470	0.00399	"	Pl**	6.000
	0.801*	0.740	0.00758	"	Pl**	6.000
	1.197*	0.245	0.00190	"	Su**	4.333
	1.252*	0.537	0.00396	"	Pl**	4.333
	1.117*	0.816	0.00815	"	Pl**	4.333
	1.275*	0.433	0.00648	"	Pl**	3.000
	0.926*	0.423	0.00208	"	Su**	6.000
	0.825*	0.437	0.00382	"	Pl**	6.000
	0.693*	0.598	0.00796	"	Pl**	6.000
	1.069*	0.392	0.00453	"	Pl**	4.333
	1.044*	0.641	0.00844	"	Pl**	4.333
	1.128*	0.361	0.00165	"	Su**	6.000
	0.665*	0.308	0.00497	"	Pl**	6.000
	0.748*	0.551	0.00481	"	Pl**	6.000
	1.236*	0.445	0.00465	"	Pl**	4.333
	1.364*	0.770	0.00646	"	Pl**	4.333
	1.389*	0.424	0.00554	"	Pl**	3.000
	0.694*	0.504	0.00465	"	Pl**	6.000
	1.012*	0.414	0.00467	"	Su**	4.333
	1.126*	0.780	0.00783	"	Pl**	4.333
Mason and Keulegan (1944)	1.579	0.223	0.00023	Step	----	1.000
Putnam (1945)	1.153	0.558	0.00551	0.425	Pl	1.000
	1.177	0.502	0.00426	"	Pl	1.000
	1.119	0.385	0.00227	"	Pl	1.000

Table 2. (cont.)

I. D.	$\frac{d_{bc}}{H_b}$	$\frac{H_r}{H_b}$	$\frac{H_b}{g T^2}$	tan α_{bs}	Breaker Type	$\frac{d_{bt}}{d_{bc}}$
Morison (1949)	1.459*	0.750	0.01560	Step	Pl	2.665
	1.490*	0.663	0.01560	"	Pl	1.598
	0.726*	0.548	0.01560	"	Pl	5.344
	0.752*	0.471	0.01560	"	Pl	5.166
	1.130*	0.725	0.00597	"	Pl	2.754
	1.130*	0.629	0.00597	"	Pl	2.754
	1.155*	0.643	0.00597	"	Pl	2.294
	0.600*	0.500	0.00597	"	Pl	5.294
	0.602*	0.488	0.00597	"	Pl	5.166
	0.905*	0.692	0.01153	"	Pl	5.347
	0.927*	0.619	0.01153	"	Pl	5.166
	0.817*	0.562	0.00756	"	Pl	2.431
McNair and Sorensen (1970)	0.875*	0.297	0.00294	0.1000	Pl**	7.000
	0.784*	0.389	0.00206	"	Su**	4.000
	0.870*	0.458	0.00214	"	Su**	4.000
	0.812*	0.379	0.00387	"	Pl**	4.000
	0.942*	0.468	0.00451	"	Pl**	4.000
	1.093*	0.510	0.00585	"	Pl**	4.000
	0.941*	0.483	0.00633	"	Pl**	4.000
	0.947*	0.518	0.01009	"	Pl**	4.000
	1.142*	0.605	0.00711	"	Pl**	2.800
	1.088*	0.589	0.00500	"	Pl**	2.800
	0.775*	0.375	0.00452	"	Pl**	2.800
Sutherland, Sharma and Shemdin (1976)	1.171*	0.389	0.00427	0.1038	Pl**	2.000
	0.801*	0.571	0.00624	"	Pl**	2.000
	1.096*	0.535	0.00532	"	Pl**	1.667
Battjes and Janssen (1978)	0.84	0.43	0.00410	0.0500	Pl**	1.917
Keady and Coleman (1980)	0.952*	0.489	0.00627	0.1493	Pl	3.000
	0.783*	0.475	0.00529	"	Pl	3.000
	1.169*	0.649	0.01197	"	Pl	2.333
	1.051*	0.624	0.00851	"	Pl	2.333
	1.082*	0.642	0.00575	"	Pl	2.333
	1.027*	0.575	0.00403	"	Pl	3.000
	1.354*	0.758	0.00813	"	Pl	2.000

*Breaker heights predicted (see text); ** Shore-breaker type predicted using method of Battjes (see text).

not available. The acceptable data are listed in Table 2, and represent about 40% of the available laboratory results for submerged barriers.

Expression of wave height transmission is limited to the shore-breaking wave height, H_b , occuring over the barrier, and the reformed wave height, H_r. The H_b measure is used rather than H_i the incident wave height representing the stable coasting wave traveling over the constant depth portion of the laboratory channel near the structure toe. It is known that waves peak during the shore-breaking process on shoaling slopes and that H_b is often significantly greater in value than H_i (Balsillie 1980, 1983b). The peaking mechanism is illustrated in Figure 10, where the wave undergoes height attenuation across the shoaling slope until, just before shore-breaking, the wave suddenly peaks up (termed alpha wave peaking, denoted as α_p). The process is non-linear. Further, it is the shore-breaking wave type that is ultimately responsible for producing longshore bars. Therefore, it is submitted that H_i is a poor wave height measure to use in assessing wave transmission over submerged barriers. For this reason, where H_b values are not reported, H_b is determined using the following relationship:

$$\frac{H_b}{H_i} = 1.0 - 0.4 \ln \left[\tanh \left(100 \frac{H_i}{g T^2} \right) \right] \qquad (35)$$

developed by Balsillie (1983b) and illustrated in Figure 11.

The data of Table 2 are plotted in Figure 12. However, before the form of the equation representing the data can be

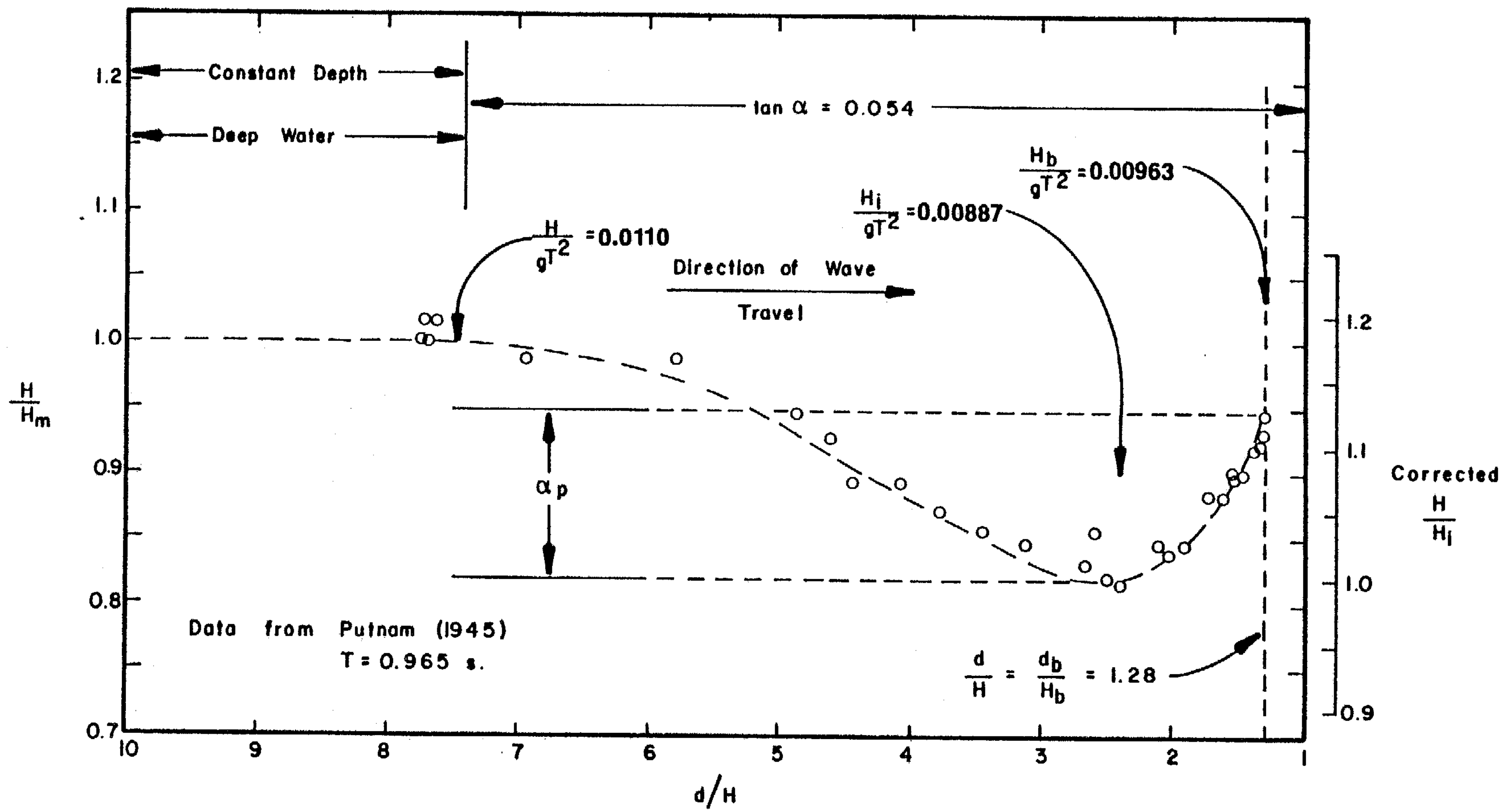

Figure 10. Illustration of the alpha wave peaking process during shore-breaking, denoted as α_p, where H_m is the wave height in the constant depth portion of the wave channel, and H_i is the wave height at the initiation of alpha wave peaking (from Balsillie, in manuscript).

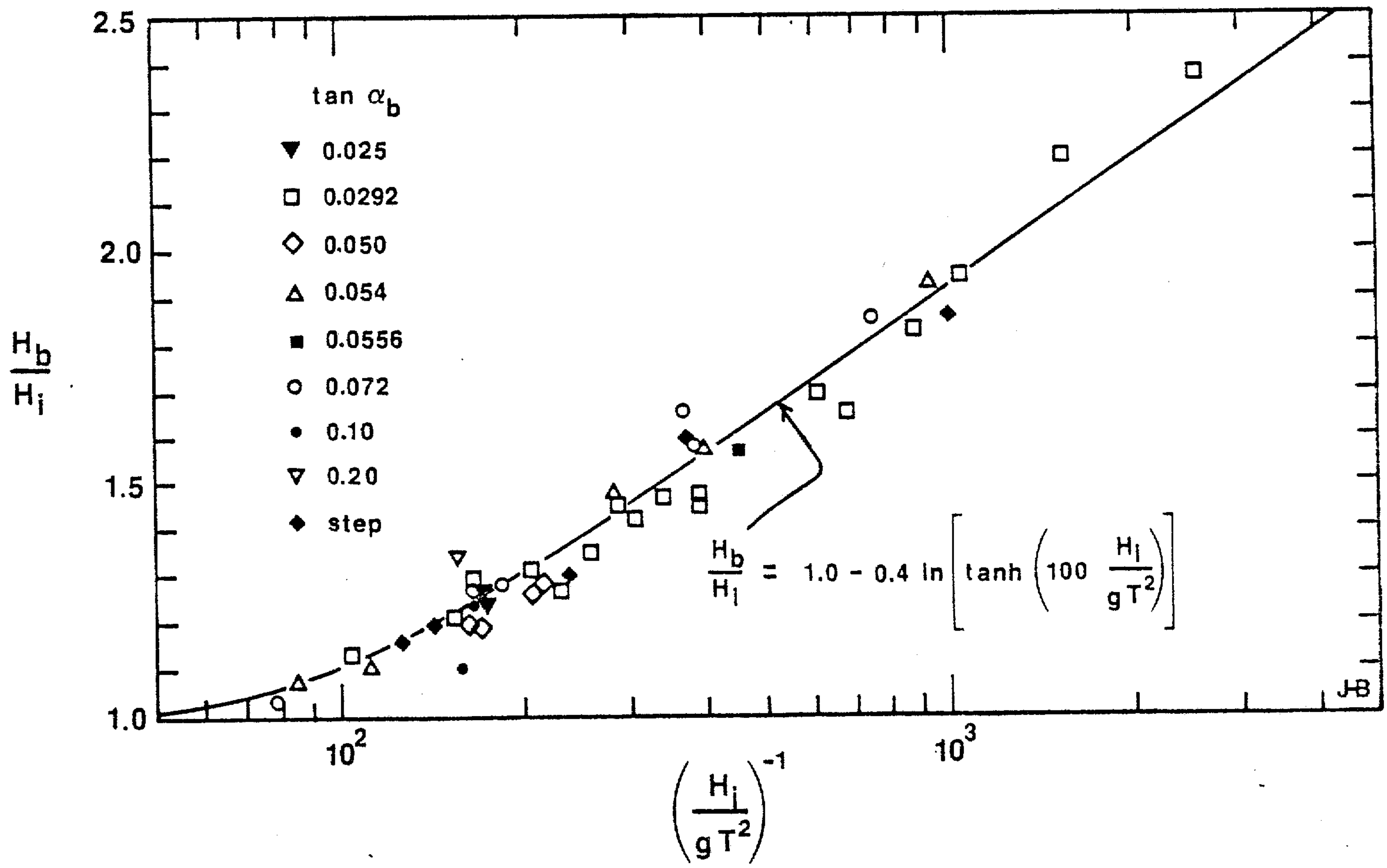

Figure 11. Relationship for prediction of the shore-breaker height from the initial equivalent wave steepness parameter illustrated in Figure 10 (from Balsillie, in manuscript).

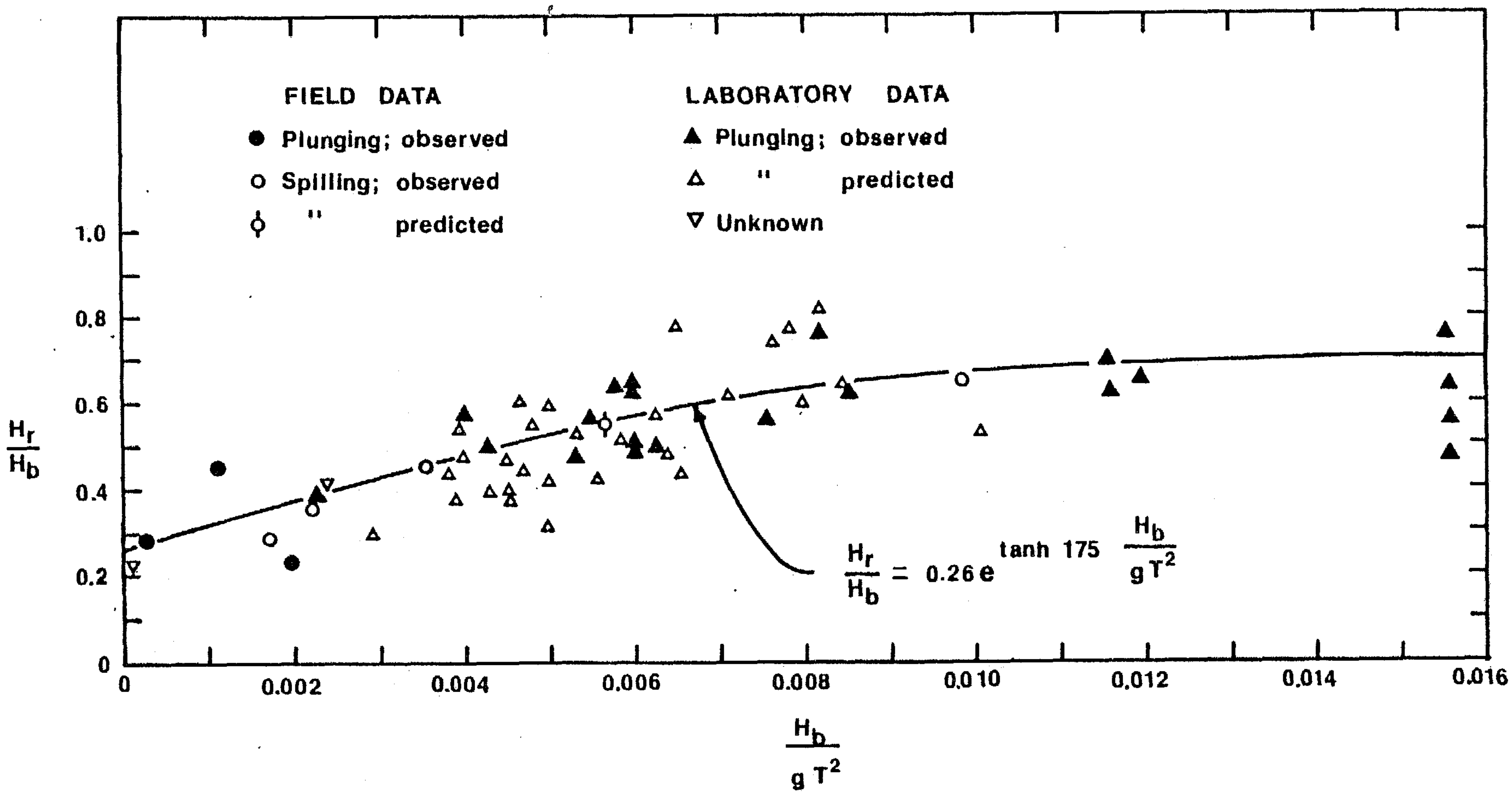

Figure 12. Relative reformed wave height, H_r/H_b, versus the equivalent shore-breaking wave steepness parameter, $H_b/(g\ T^2)$, where H_r is the reformed wave height, H_b is the shore-breaking wave height occurring over the barrier crest and T is the barrier-incident wave period.

determined, some guidance concerning lower and upper limits is required. Concerning the lower limit, the question arises if the equation passes through the origin of the plot or intersects the ordinate at some positive value? First, it is to be reconciled that H_r/H_b represents a real solution, and that H_b cannot go to zero since the value of H_r/H_b would be non-sensical, even if H_r goes to zero (which it would). Second, it is possible for T to be large, thereby producing a small value of $H_b/(g\ T^2)$. In fact, if one is dealing with Solitary waves, $H_b/(g\ T^2)$ would essentially be zero and a positive y-intercept value would result. Visual inspection of the data of Figure 12 suggests that where $H_b/(g\ T^2) = 0$, H_r/H_b would have a value somewhere in the range of from 0.2 to 0.3. Hence, a transcendental power curve function is eliminated as a form of the representing equation.

An upper limit absissca value may be estimated for forced wave conditions. Michell (1893) found that the limiting criterion for the stability of waves in deep water is given by $(H_o/L_o)_{max} = 1/7$. Where from Airy wave theory $L_o = g\ T^2/(2\ \pi)$, then $H_o/(g\ T^2) = 0.0227$. Noting that H_o and H_b are seldom equivalent and that for plunging shore-breakers H_b could very well be greater than H_o, then where on the average $H_b = 1.3\ H_o$ (Balsillie, 1980, assuming $H_o = H_i$), the ratio may be given as $H_b/(g\ T^2) = 0.0175$. Applying these values as guidelines to the data of Figure 12, appears to identify the upper limit of values for the absissca. Visual inspection of the data suggests that the value of H_r/H_b may asymptote at about 0.7 for about $0.014 < H_b/(g\ T^2) < 0.016$.

The equation fitted to the data is of the transcendental exponential form yielding:

$$\frac{H_r}{H_b} = 0.26\, e^{\tanh 175 \frac{H_b}{g\, T^2}} \qquad (36)$$

Equation (36) is fitted primarily to those data where the plunging shore-breaker type was known to occur.

To some extent, the laboratory results of Morison (1949) determine where the design curve asymptotes. His results for high wave steepness values, affecting the equation, occur for two values, at $H_b/(g\ T^2) = 0.01153$, and for four values at $H_b/(g\ T^2) = 0.0156$. In large part, Morison's results for high wave steepness values reflect the influence of barrier width, but do not represent the effects of a bar stoss slope. It is possible that the asympotic value is somewhat different than shown and further research appears warrented. Even so, equation (36) appears to provide a substantial refinement in the design solution for longshore bars.

WAVE PERIOD ATTENUATION

It is generally viewed that as long as shoaling waves are undergoing refraction, the wave period is conserved. However, from much of the existing laboratory research it is reported that even where shore-breaking waves reform, the wave period remains conserved, predicated on the observation that the "fundemental" reformed wave crest has a period equivalent to the incident shore-breaking wave. In some cases, this appears to be a function of problems in measuring the reformed wave characteristics (e.g., McNair and Sorensen, 1970).

More definitive results are observed in the field. Byrne (1966) reports a 50% reduction in the reformed wave period following bar incidence for non-breaking waves. Carter and Balsillie (1983) report that significant wave period reduction can occur following shore-breaking over longshore bars. Suhayda and Roberts (1977) found that the average wave period shoreward of reefs over which breaking occurred were smaller by about 50% to 75% than the offshore wave periods. However, the paucity of field data does not allow for the development of a prediction method.

In the extensive laboratory study of Nakamura, Shiriashi and Sasaki (1966a, 1966b) wave period attenuation due to breaking was reported. Relationships determined by the author for the data ... equations (11) and (17)... can be transformed to represent conditions at shore-breaking. Where from Airy wave theory $L_o = g\,T^2/(2\,\pi)$, and $d_b = 1.28\,H_b$, equation (11) becomes:

$$\frac{T_r}{T} = \tanh 256\,\pi \left(\frac{H_b}{g\,T^2}\right)^{0.44} \qquad (37)$$

for a smooth linear slope of tan = 0.010, and:

$$\frac{T_r}{T} = \tanh \pi \left(\frac{H_b}{g\,T^2}\right)^{0.18} \qquad (38)$$

for submerged rectangular barriers. Where $d_{bt} = 1.6\,H_b$, equation (38) becomes:

$$\frac{T_r}{T} = \left(\tanh 5 \quad \frac{H_b}{g\ T^2}\right)^{0.18} \qquad (39)$$

for longshore bars. Some field data from Byrne (1966), Wood (1970, 1971), Carter and Balsillie (1983) and an additional point collected by the author, and some laboratory data from Mason and Keulegan (1944) are plotted in Figure 13 along with equations (37) and (39). It is apparent from the figure that scatter in the data is considerable, although the data fall within the envelope described by equations (37) and (39). In view of the problems encountered in other laboratory studies, it is unfortunate that Nakamura, Shiriashi and Sasaki did not elaborate on their method(s) of measurement of reformed wave characteristics. While additional study on wave period transformation over bar-type features is warrented, equation (34) provides the only existing formalization presently available. It is to be noted, however, that equation (39) does conform to reported field results.

WAVE LENGTH ATTENUATION

Available field data (Wood, 1970, 1971; Keady and Coleman, 1980) verifies the significant reduction in wave length following shore-breaking over longshore bars. As with the wave period, there is insufficient field data on which to base a predictive relationship. However, wave length attenuation has received considerable attention in laboratory studies (Mason and Keulegan, 1944; Nakamura, Shiriashi, and Sasaki, 1966a, 1966b). Equations (8), (12) and (18) have been fitted

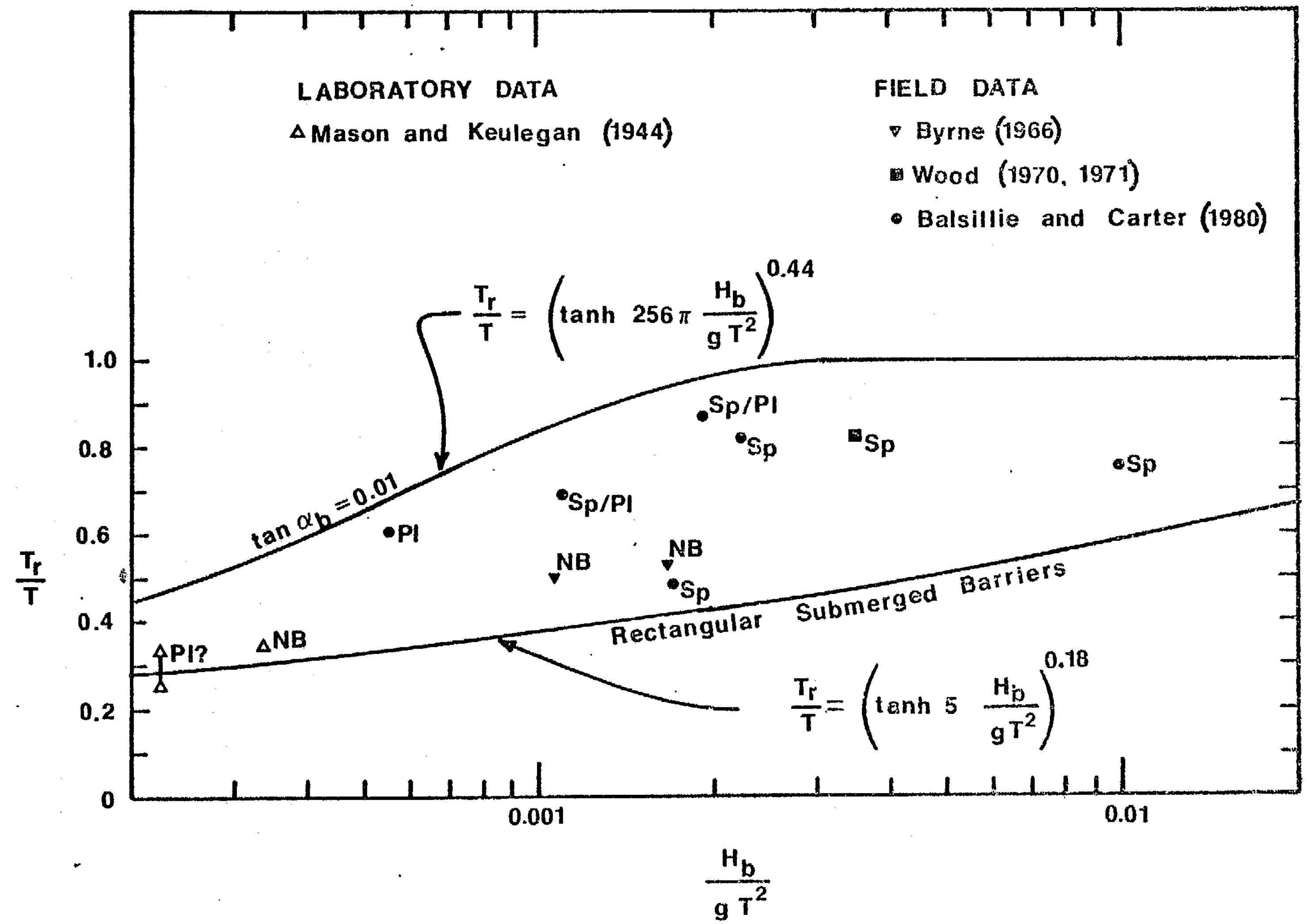

Figure 13. Relationships for prediction of the reformed wave period, T_r.

by the author to experimental data which represent a linear slope, step slope, and submerged barriers. Using developments applied in the previous section, equation (12), (8) and (18) are transformed to conditions at shore-breaking, to yield:

$$\frac{L_r}{L_b} = \left(\tanh 805 \frac{H_b}{g\ T^2}\right)^{0.214} \tag{40}$$

for a linear slope of tan α_b = 0.010,

$$\frac{L_r}{L_b} = \left[\tanh 3.39 \left(\frac{H_b}{g\ T^2}\right)^{0.5}\right]^{0.38} \tag{41}$$

for a step profile, and:

$$\frac{L_r}{L_b} = \left[\tanh 0.95 \left(\frac{H_b}{g\ T^2}\right)^{0.5}\right]^{0.45} \tag{42}$$

for rectangular submerged barriers.

Equations (40), (41) and (42) are plotted in Figure 14. As with the wave period, field data need to be obtained to test the laboratory derived relationships. In the interim, equation (42) is the only available predictive relationship.

POST-BAR BREAKING DISTANCE MEASURES

Details of wave height attenuation following shore-breaking have been investigated for shore-breaking waves on linear profiles (Horikawa and Kuo, 1966; Kishi and Saski, 1966; Bowen, Inman and Simmons, 1968; Nakamura, Shiriashi and Saski,

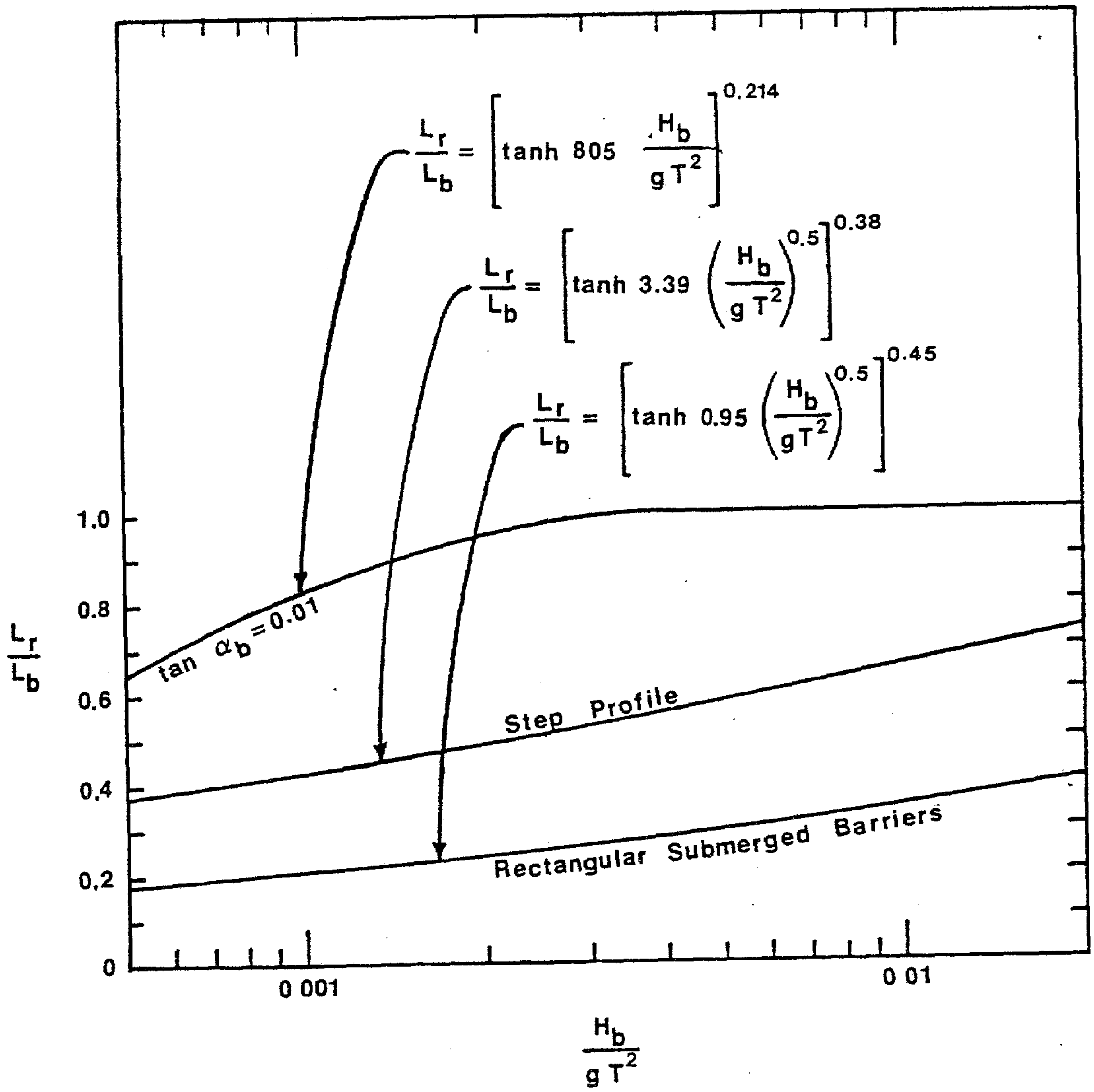

Figure 14. Relationships for prediction of the reformed wave length, L_r.

1966a and Singamsetti and Wind, 1980), and on step-type profiles (Putnam, 1945; Horikawa and Kuo, 1966; and Sawaragi and Iwata, 1974). However, no such details are available for waves which shore-break on longshore bar-type features. Nevertheless, in the absence of detailed results, there is a need to have knowledge of certain post bar breaking wave transformation milestones. Critical milestones are the distance to the bar trough, and the point of wave reformation.

It is likely that the distance to wave reformation does not coincide precisely with the distance to the maximum depth of the bar trough, x_{bt}, measured from the bar-breaking point. In fact, there is no presently available predictive measure of x_{bt} suitable for phototype conditions. A number of related measures may, however, be employed to attempt to identfy a least equivocal distance.

A minimum measure of x_{bt} might be given by x_{td}, the distance from shore-breaking over the bar crest to the point where the curl of the plunging wave touches down onto the water surface fronting the wave crest. On a horizontal bed, which is approximately applicable to bar-breaking, Galvin (1969) suggests:

$$x_{td} = 4 H_b \tag{43}$$

and

$$x_s = 8 H_b \tag{44}$$

where x_s is the horizontal distance from shore-breaking to the end of the splash point. Weishar and Byrne (1978) suggest that the value of x_{td} from field data may be more nearly given by:

$$x_{td} = 5.6\ H \qquad (45)$$

An average of equations (43) and (45) yields a "working" form of the equation as:

$$x_{td} = 5\ H_b \qquad (46)$$

Sawaragi and *Iwata* (1974) investigated *shore-breaking* wave activity on a *step-type* profile, given by equations (19) through (22). These equations may be assessed at shore-breaking by equating equations (19) and (46), setting up a proportionality and readjusting equations (20) through (22) to yield:

$$x_v = 6.1\ H_b \qquad (47)$$

$$x_s = 9.2\ H_b \qquad (48)$$

and

$$x_a = 33\ H_b \qquad (49)$$

where x_a is the distance measured from shore-breaking to the point where air bubbles disappear from the water column.

Nakamura, *Shiriashi* and *Sasaki* (1966a) found that for uniform nearshore slopes less than about 0.02 to 0.03, the re-formation distance measured from the shore-breaking position may be given by equation (10). Using Komar and Gaughan's (1972 development which relates the shore-breaking wave height to the unrefracted deep water wave height, given by:

$$\frac{H_b}{H_o} = \frac{14}{25} \left(\frac{H_o}{L_o}\right)^{-0.2} \tag{50}$$

and assuming forced storm wave conditions where according to Michell (1893) $(H_o/L_o)_{max}$ = 1/7, then H_o = 1.2 H_b, and equation (10) becomes:

$$x_r = 32\ H_b \tag{51}$$

which is close to equation (49).

Where equation (50) may be restated as:

$$H_o = \pi H_b \left(\frac{H_b}{g\ T^2}\right)^{0.25} \tag{52}$$

a form of equation (10) accounting for all wave steepness, may be given by:

$$\frac{x_r}{H_b} = 26.1\pi \left(\frac{H_b}{g\ T^2}\right)^{0.25} \tag{53}$$

Equation (50) or similar forms developed by Munk (1949) or Van Dorn (1978) can be used to transform equations (19) through (22), although several problems occur: 1. it is difficult to assess the value of the local wave length, L, in equations (19) through (22), 2. where it is assumed L_r = L, the transformed equations yield results less than for other equations developed in this section, and 3. it is not known if Komar and Gaughan's equation is applicable to step-type profiles. That the value of x_r decreases with a decrease in $H_b/(g\ T^2)$ does not, at this time, appear to be problematic. However, because such transformations introduce additional complexities and pending further investigation, reliance is placed in the more simple and straightforward developments.

A set of equations are now available from which to "bracket" the location of the bar trough. We know that the bar trough position should be less than the reformation distance, but greater than the distance where the plunging vortex reaches the bed, i.e., $x_r > x_{bt} > x_v$. In fact, the splash distance may be a good measure of the location of the deepest part of the bar trough, since it approximates the location where the turbulence surfaces. While equations (44) and (48) are close, there is a need to account for the additional water depth characterizing the longshore bar trough morphology. A proportionality of existing equations yields:

$$x_{bt} = 14.7\ H_b \qquad (54)$$

which is close to physiographically based equation (34).

Equation (51) for the distance to wave reformation is 28% less than physiographically based equation (33) for longshore bar crest spacing. This appears to provide corroborating evidence, sinc ethe wave reformation distance for multi-barred nearshores would be required to be less than the bar crest spacing. If reformed waves are primarily responsible for secondary longshore bar formation.

CLOSURE

Natural littoral profiles can be defined in terms of complexity of shape. Simple geometry may be defined by a power curve function representing a smooth profile shape. Based on the work of Dean (1977), these profiles are considered to be associated with spilling-type shore-breakers and represent long-term, average nearshore profile geometry. The most complex geometry is encountered where the profile is characterized by longshore bars, which are generated by plunging-type shore-breakers.

The generation of barred profiles is, as one can imagine, highly complex. In this work, however, the longshore bar is present and it's effect on wave reformation, once the waves have shore-broken on the bar crest, has been investigated. The height of the reformed wave is given by equation (36), reformed period by equation (39), and reformed wave length by equation (42). In addition, certain post-bar-breaking critical distance measures, such as distance to the bar trough and the point of wave reformation, are investigated. Much additional work is required to quantify the effect of the waves on the structure ...i.e., longshore bar generation. However, by investigating the effect of the structure on the waves, the "end-member" aspect of longshore bar formation, has been addressed.

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