

Redefinition of Shore-Breaker Classification as a Numerical Continuum and a Design Shore-Breaker¹

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ABSTRACT

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Based on an initial appraisal of data, new considerations are used on which to base a least equivocal numerical definition of shore-breaking waves. Redefinition is based on the amount of the wave crest, H_b'' , measured from the wave crest top down, involved in shore-breaking, and is given by: $H_b''/H_b = \tanh 0.4 \xi_b$ where H_b is the shore-breaking wave height, and ξ_b is a slightly modified version of the surf similarity parameter. The above equation also appears to represent the position at which the maximum impact pressure in a shore-breaking wave occurs. Relating equations are: $z_{\max}/H_b = 2.12 - \tanh 0.4 \xi_b$ or $z_{\max SWL}/H_b = 0.84 - \tanh 0.4 \xi_b$ in which z_{\max} and $z_{\max SWL}$ are the vertical distance above the bed and still water level (SWL) at shore-breaking, respectively, where the maximum horizontal impact pressure occurs. Available impact pressure data indicates that the shore-breaker with the most destructive potential has a value of $\xi_b = 1.0$. This results in a design shore-breaking wave which imparts the greatest horizontal impact at a distance of $0.62 H_b$ above the wave trough, or $0.46 H_b$ above the SWL. Recalibration of the numerical results suggests the following modification in the existing descriptive shore-breaker type scale: $\xi_b < 0.64$, spilling shore-breakers; $\cong 0.64$ to 5.0 , plunging shore-breakers; > 5.0 , surging shore-breakers.

ADDITIONAL INDEX WORDS: Collapsing breaker, design shore-breaker, impact pressure, plunging wave, shore-breaking wave, spilling wave, surf similarity parameter, surging wave, wave crest.



INTRODUCTION

Shore-breaking waves, because of the highly complex nature of the shore-breaking process, have been defined in terms of the visually observed crest geometry in profile view. Three general categories of shore-breakers are commonly recognized (WEGGEL, 1964) as spilling, plunging and surging, although GALVIN (1968) has described additional sub-types.

Spilling is defined to occur at the point where the top of the wave crest becomes unstable and aeriated and turbulent water slip down and across the front face of the wave crest (Figure 1). Up to 25% of the

top of the wave crest is defined to be involved in spilling-type shore-breakers, and the spilling can occur over quite a distance (U.S. ARMY, 1977).

Plunging occurs where the upper portion of the wave crest curls over, forms an air pocket and the curling crest eventually falls onto the trough fronting the crest. The plunge point is defined to occur when the upper portion of the wave crest front, which will form the curl, becomes vertical (Figure 1). The laboratory data of WEGGEL (1968) suggest that greater than 25% of the wave crest height, measured from crest top down, is involved in forming the curl.

Surging is defined to occur where the bottom portion of the wave rushes forward from under the wave crest, sliding up the beach face with a minimum of bubble production (Figure 1).

A fourth general shore-breaker type has gained recognition. . . the collapsing breaker. . . identified by GALVIN (1968). Within state-of-the-art numeri-

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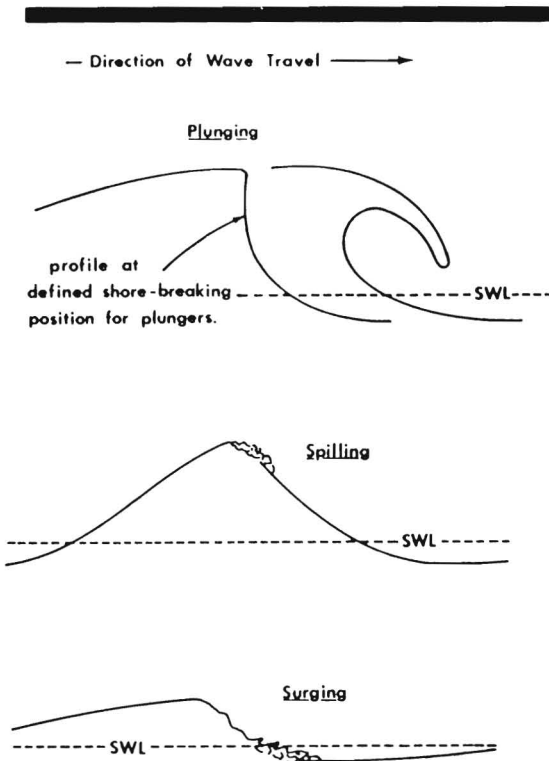


Figure 1. Three basic categories of the shore-breaking wave types.

cal developments for predicting shore-breaker type, this breaker falls in the surging category. Based on observations by the author, these breakers result from special conditions which may largely depend on bed roughness and porosity which in turn affects the phase and particle velocities; for instance, WEGGEL (1972) noted the possible influential effect of such breakers on rubble-mound structures.

A continuum of the general shore-breaker type, grading from spilling to plunging to surging, has been identified (GALVIN, 1968; BATTJES, 1974; KOMAR, 1976). Spilling shore-breakers tend to occur on shallow nearshore slopes where waves have high wave steepness values; plungers occur on beaches with both intermediate bed slope and wave steepness values; and surging shore-breakers on high-gradient beaches where wave steepness values are small.

While verbal description is necessary towards establishing an understanding of a natural process, a numerical method which describes the continuous nature of the process is ultimately desirable. For instance, GALVIN (1968) and BATTJES (1974)

have described numerical parameters which indicate the point of transition between spilling, plunging and surging breaker types. While their work is important, the net result is that the verbal classification remains. What is ultimately needed, however, is a method for quantifying shore-breaker wave behavior across the entire spectrum of conditions that is independent of the verbal description. For scientific and engineering purposes, such a method needs to be expressed mathematically, and defines the purpose of this paper.

PREVIOUS WORK

Delineation of conditions producing the basic, visually recognized categories of shore-breaking wave types is, by no means, a new endeavor. By 1946, Dean M.P. O'Brien had conceptually detailed the wave steepness and bed slope conditions required to produce spilling and plunging shore-breakers (BEACH EROSION BOARD, 1949). At about the same time, IRIBARREN and NOGALES (1949) suggested a parameter, now termed the "surf similarity parameter," for determining where shore-breaking occurs. PATRICK and WIEGEL (1955) are generally accredited with formalizing the continuum and adding the surging shore-breaker type. Some years later, using what was an independently derived variation of the surf similarity parameter, GALVIN (1968) suggested that:

$$\frac{H_b}{g T^2 \tan \alpha_b} \begin{cases} > 0.068, \text{ spilling shore-breakers} \\ 0.003 \text{ to } 0.068, \text{ plunging shore-breakers} \\ < 0.003, \text{ surging shore-breakers} \end{cases} \quad (1)$$

where H_b is the shore-breaking wave height, g is the acceleration of gravity, T is the wave period and $\tan \alpha_b$ is the bed slope. BATTJES (1974), using the surf similarity parameter, suggested that:

$$\frac{\tan \alpha_b}{\sqrt{\frac{H_b}{g T^2}} / 2 \pi} \begin{cases} < 0.4, \text{ spilling shore-breakers} \\ 0.4 \text{ to } 2.0, \text{ plunging shore-breakers} \\ > 2.0, \text{ surging shore-breakers} \end{cases} \quad (2)$$

Investigations have also been conducted to quantify distortion of waves as they shoal and shore-break. ADEYEMO (1968) measured the horizontal and vertical asymmetry of wave profiles about the still water level. More recently, similar information has been obtained by SINGAMSETTI and WIND (1980). Others have investigated the relationship between the degree of wave profile distortion and the vertical distribution of horizontal water particle

velocities (MORISON and CROOKE, 1953; MILLER and ZIEGLER, 1964; ADEYEMO, 1970; WOOD, 1970; IWAGAKI *et al.*, 1974; KEMP, 1975; SAKAI and IWAGAKI, 1978). However, no definitive relationship has surfaced from such studies for prediction of shore-breaker type.

SHORE-BREAKER REDEFINITION

Redefinition of the manner in which shore-breaking waves are viewed in coastal engineering applications, is based on two considerations: (1) the amount of the wave involved in shore-breaking, and (2) the destructive potential of shore-breaking waves.

The first consideration is, in fact, already evident from existing observation-based definitions of shore-breaker types; namely that progressively more of the wave crest is involved (measured from the crest top down) in breaking when proceeding from spilling to plunging to surging. This consideration, to the author's knowledge, has not been pursued in developing methods for breaker-type prediction. Such an approach, however, provides a fortuitous analytical methodology since if the domains of parameters are carefully selected, a mathematical description of the solution, both intuitively and practicably, should vary only within narrow limits.

From the results of a field investigation, WEISHAR (1976) and WEISHAR and BYRNE (1978) found that BATTJES' (1974) parameter, given by equation (2), results in more accurate shore-breaker type prediction than the parameter proposed by GALVIN (1968), given by equation (1). This finding along with other numerical problems encountered during analysis using GALVIN's parameter, have led to singular consideration of the surf similarity parameter and BATTJE's evaluation given by equation (2).

We shall select as the dependent variable the dimensionless quantity H_b'' / H_b , where H_b is the shore-breaking wave height, and H_b'' is the amount of H_b (measured from the breaker crest top down) that is involved in shore-breaking. The independent variable becomes a modification of the surf similarity parameter (*i.e.*, 2π removed) to yield:

$$\xi_b = \frac{\tan \alpha_b}{\sqrt{H_b / (g T^2)}} \quad (3)$$

from which it may now be stated:

$$\xi_b \begin{cases} < 1.0, \text{ spilling shore-breakers} \\ 1.0 \text{ to } 5.0, \text{ plunging shore-breakers} \\ > 5.0, \text{ surging shore-breakers} \end{cases} \quad (4)$$

A visually fitted equation for consideration is given by:

$$H_b'' / H_b = \tanh 0.4 \xi_b \quad (5)$$

which is plotted in Figure 2. Note that from equation (2), $1 / 2\pi = 0.4$ which appears in equation (5).

There is little published data available representing the value of H_b'' . An example for a plunging shore-breaker (traced from photographs published by MILLER (1976, Figures 1 and 2) is illustrated in Figure 3 in which H_b'' / H_b has a value of about 0.3 and appears to remain constant throughout the plunging process to curl touchdown.

In a laboratory investigation on impact pressures accompanying shore-breaking waves, WEGGEL (1968) measured from film frames the distance, s , above the bed where a vertical line became tangent to the vertical portion of the front face of plunging wave crests. From WEGGEL (1968) and WEGGEL and MAXWELL (1970), s appears to represent the location of the center of the curl of the plunger. The average value from 16 experimental results of WEGGEL is plotted in Figure 2; an average value is used since ξ_b had a small range of from 0.619 to 1.098, while the range of H_b'' / H_b was from 0.232 to 0.507. It is noted that WEGGEL's data fall within the spilling category, even though they were observed to be plunging (verified from personal communications with J.R. Weggel). However, exceptions to the delineating value separating spilling and plunging shore-breakers are not, from existing studies, uncommon.

WEGGEL's directly measured data is the only systematically collected information known to the author. Two other sources, each providing a single breaking wave example, have been found. IPPEN and KULIN (1954) published profile traces of waves at the shore-breaking position. Only one of the traces appeared to represent a shore-breaker (*i.e.*, plunging) from which s could be identified with certainty. Since the waves that IPPEN and KULIN investigated were solitary, the wave period was calculated as $T = (1.6 g H_b)^{0.5} L_b^{-1}$ (developed by BALSILLIE, 1984) in which L_b was determined by extending the stoss breaker surface to the still water level plane and doubling the value. The other source is from WIEGEL and SKJEI (1958) for a periodic wave (a plunger from run 26), where s was identifiable.

It is unfortunate that more data are not available to determine s for the various types of shore-breakers, principally because such information should not be difficult to obtain (*e.g.*, from filmed wave activity).

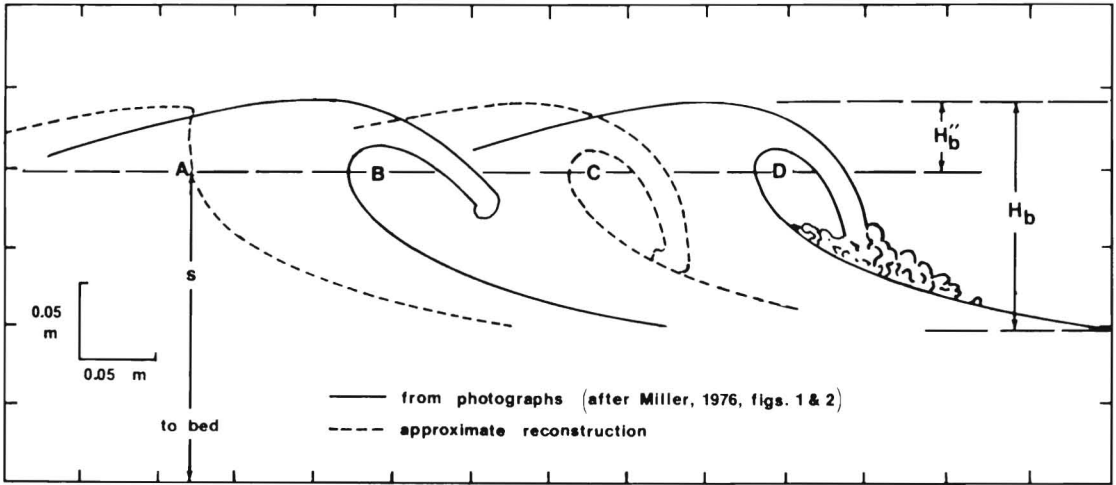


Figure 3. History of a plunging wave during shore-breaking. Position A describes the defined shore-breaking point of a plunging breaker, position C the touchdown point.

Figure 2, from which H_b'' is determined as $z_{max} - (d_b + H_b')$ where H_b' is the amount of the wave crest lying above the still water level (see definition sketch of Figure 4).

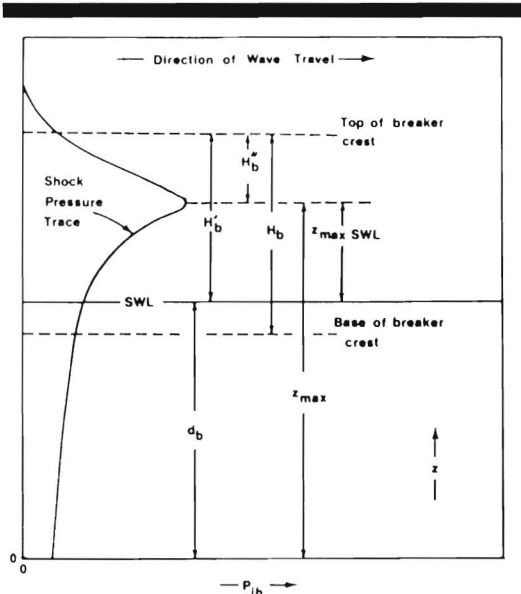


Figure 4. Vertical distribution of horizontal impact pressure, P_{ih} , beneath a shore-breaking wave crest; the assumption of similitude in location of z_{max} and the center of the curl of a plunging wave above the bed, s (see text), is based on the results of WEGGEL (1968).

There is considerable scatter in the impact pressure data plotted in Figure 2, which may be due in part to differences in experimental setup and approach from investigation to investigation (see KIRKGOZ, 1982, p. 81-82, 88-89). The data do, however, appear to support the trend suggested by equation (5).

Restatement of equation (3) yields alternative design relationships. These relationships depend upon assumptions that $d_b / H_b = 1.28$ (McCOWAN, 1894; MUNK, 1949; BALSILLIE, 1983a) and $H_b' / H_b = 0.84$ (BALSILLIE, 1983b) from which $H_b'' = (d_b + 0.84 H_b) - z_{max} = 2.12 H_b - z_{max}$. Substitution into equation (3) yields:

$$z_{max} / H_b = 2.12 - \tanh 0.4 \xi_b \tag{6}$$

or

$$z_{max SWL} / H_b = 0.84 - \tanh 0.4 \xi_b \tag{7}$$

in which z_{max} and $z_{max SWL}$ are the vertical distances above the bed and still water level, respectively, at the shore-breaking position where the maximum horizontal impact pressure occurs.

It is to be noted from Figure 4 that the vertical distribution of the horizontal impact pressure can occur above the shore-breaker crest. This occurs because the pressure sensors are often mounted on vertical walls of significant width relative to inci-

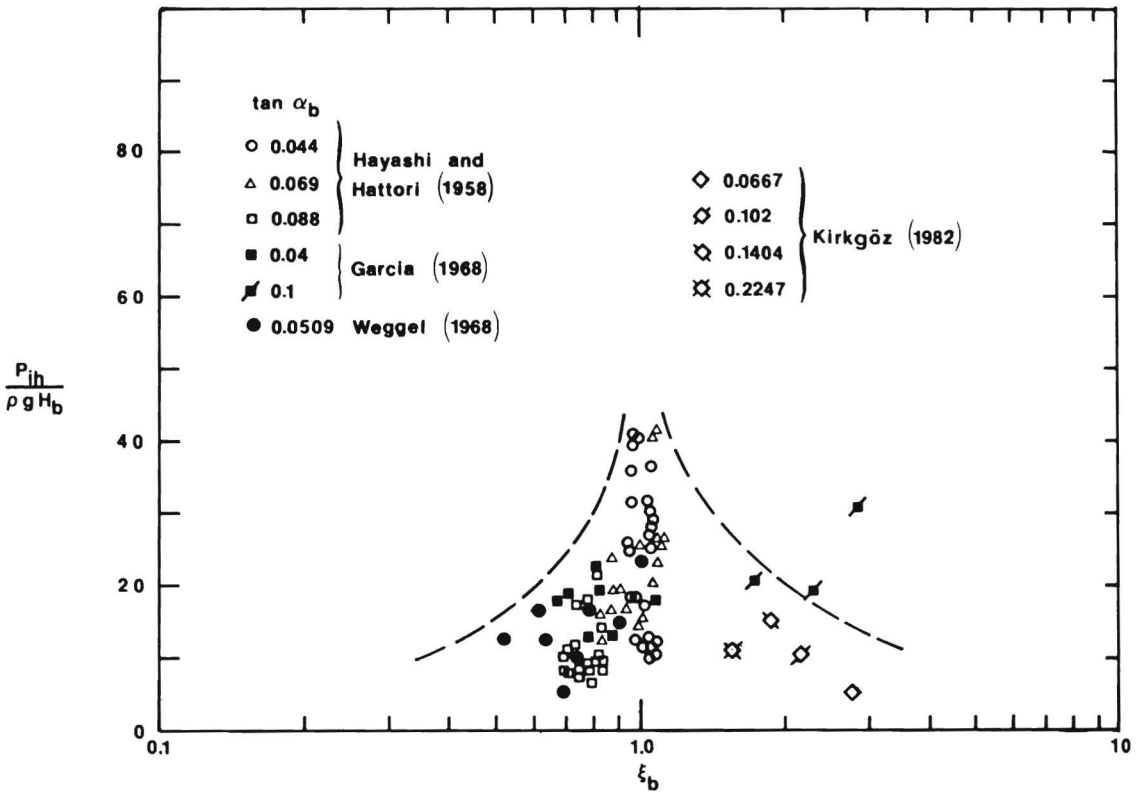


Figure 5. Plot of the relative maximum horizontal impact pressure occurring from a shore-breaking wave (i.e., within the vertical distribution beneath the wave crest) and the modified surf similarity parameter.

dent wave characteristics, and wave crest setup occurs. This should not be viewed as an experimental artifact, however, since the same can occur under prototypical design conditions. Hence, equations (6) and (7) do not mean to imply that the portion of the wave, including breaker crest setup, above z_{max} should be ignored in design solutions.

THE DESIGN SHORE-BREAKER

Results from field studies (MILLER *et al.*, 1974a, 1974b; MILLER, 1976) indicate that impact forces from shore-breaking and broken waves significantly exceed those from non-breaking waves. Highest impact pressures occur in post-breaking bores, with greater pressures occurring from plunger-generated than spilling-generated bores. Shore-breaking waves produced next highest impact pressures, with greater pressures occurring for plunging than spilling shore-breakers. The difference between breaking and post-breaking pressures is the elevation,

z_{max} , at which the maximum horizontal impact pressure occurs. For post-breaking bores z_{max} is low, occurring near the mean water level surface. For waves at the shore-breaking position, z_{max} occurs in the upper portion of the wave crest, well above either the mean water level or the still water level. Because of the higher elevations associated with shore-breakers, they define the condition of interest.

Existing horizontal impact pressure data from studies by HAYASHI and HATTORI (1958), GARCIA (1968), WEGGEL (1968) and KIRKGOZ (1982) are plotted in Figure 5 as the dimensionless quantity $P_{ih} (\rho g H_b)^{-1}$ in which P_{ih} is a measure of the maximum horizontal impact pressure and ρ is the fluid mass density. It is apparent from Figure 5, that $P_{ih} (\rho g H_b)^{-1}$ reaches a peak at $\xi_b = 1.0$. Substitution into previously developed equations yields the following relationships:

$$(H_b^* / H_b)_{des} = 0.38 \tag{8}$$

$$(z_{\max} / H_b)_{\text{des}} = 1.7 \quad (9)$$

and

$$(z_{\max \text{ SWL}} / H_b)_{\text{des}} = 0.46 \quad (10)$$

which provide an indication of the design shore-breaking wave conditions. Where $H'_y / H_b = 0.84$, then z_{\max} above the wave trough fronting the shore-breaking wave is 62% of H_b , or 38% of the wave crest top is involved in shore-breaking.

RECALIBRATION OF THE VERBAL SHORE-BREAKER CLASSIFICATION

As noted earlier, there are well known exceptions to the value of $\xi_b = 1.0$, which is suggested by equation (4) as the value separating spilling and plunging shore-breaker types. There appears, however, some basis on which to redefine the value. First, all waves considered are plunging. Second, the original definition states "up to 25% of the top of the wave crest" is involved in spilling. Hence, substitution of this value into equation (3) yields a value of $\xi_b = 0.64$, and:

$$\xi_b \begin{cases} < 0.64, \text{ spilling shore-breakers} \\ 0.64 \text{ to } 5.0, \text{ plunging shore-breakers} \\ > 5.0, \text{ surging shore-breakers} \end{cases} \quad (11)$$

As illustrated in Figure 2, equation (11) appears to satisfy both considerations.

DISCUSSION AND CLOSURE

It is apparent that different types of shore-breaking waves can produce quite different results. It is not surprising, therefore, that research has been conducted to delineate methods for the prediction of shore-breaker type. Existing methodology persists, however, in defining a numerical scale which has meaning only when transliterated to the traditional verbal description of the shore-breaker type. Even then, the predicted shore-breaker leaves little insight as to meaning and application. It becomes even more apparent, therefore, that additional consideration is necessary to bring such a scale into perspective.

Such consideration depends on the purpose(s) to which the shore-breaking waves are to be applied.

In this paper, the emphasis is on the combined consideration of the amount of the wave crest involved in breaking, and the destructive potential of shore-breaking waves assessed in terms of the horizontal impact (or shock) pressure, based on an initial appraisal of available data. For the data investigated, it became apparent that the amount of the wave crest involved in breaking and the point of occurrence of maximum impact pressure, appeared to be directly related. Results indicate that a shore-breaker which has a value of $\xi_b = 1.0$ produces the greatest horizontal impact pressure, occurring at an optimum distance of $0.46 H_b$ above the still water level.

To one extent or another, it is apparent that the results of this work are based on simplifying assumptions. For instance, that the amount of the wave crest involved in shore-breaking and the point of application of the maximum impact pressure during shore-breaking are coincident, may be fortuitous. Maximum impact pressures delineating a value of $\xi_b = 1.0$ may be a function of experimental conditions; the relationship between model and prototype impact pressures have been subject to considerable discussion (BIJKER, 1969). Additionally, while many aspects of shore-breaking waves are capable of measurement by instrumentation, identification of shore-breaker type can be made by visual observation only (BALSILLIE, 1983a). The concept presented, however, is not without basis. While the purpose of this paper is, in large part, to introduce a new, potentially useful way of viewing shore-breaking waves, it is also a plea for more data on which to test and, if corroborative to the findings of this work, refine the concept presented.

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