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WAVE BREAKING

A review of techniques for calculating
energy losses in breaking waves.

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ABSTRACT

Wave breaking is an important consideration in many maritime engineering design calculations. Over the past twenty years a wide variety of methods have been put forward for determining various features of breaking processes, ranging from simple empirically-determined expressions for the breaker wave height to computational models of the detailed structure of breaking waves. This report contains a literature review, concentrating on techniques for determining the wave height at breaking, and the subsequent energy losses as broken waves continue forwards. The emphasis is on methods suitable for hand calculations or inclusion in computational models of wave transformation in shallow water. Assessments of the techniques are made and recommendations given.

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1 INTRODUCTION

Wave breaking is the most obvious and spectacular of all the physical processes that affect water waves, and is an important consideration in engineering design both in deep water and close to the shoreline.

The main purpose of this report is to review critically the methods by which wave energy dissipation due to breaking can be estimated by hand calculations or incorporated into computational models of wave transformation from deep to shallow water. Chapter 2 contains a brief qualitative description of the phenomenon of wave breaking and the various processes that result from it, particularly within the surf zone. In Chapter 3, the approaches to predicting the "breaker height" criterion (i.e, the maximum wave height allowed by the breaking process at a given water depth and seabed slope) are outlined. In Chapter 4 the methods of modelling the rate of dissipation of wave energy in a broken wave are discussed. Finally in Chapter 5 a summary of the main conclusions and recommendations is given.

2 PHENOMENOLOGICAL DESCRIPTION OF WAVE BREAKING

Before breaking, waves have a relatively smooth water surface. After breaking the wave fronts are usually white and foamy often with a lot of spray and bubbles. Complex process takes place during breaking, involving a rapid change of wave shape and the conversion of wave energy to turbulence and subsequently heat. After a short distance, roughly several times the depth at breaking, the wave adopts a steady, well-organised profile which is more-or-less independent of the initial breaking behaviour, but still often with white water at the crest face. The term 'broken wave' will be used to describe this

steady phase, and 'breaking wave' will be used to describe the initial breaking process. Svendsen et al (1978) use the terms 'inner region' and 'outer region' respectively for these two phases.

The most visually apparent phase occurs when waves initially break and the wave crest overturns, or comes as close as possible to overturning, and generates white water. The different visual characteristics of breaking waves provides a classification of breaker types (Galvin (1968)), described below and shown in Fig 1:

(a) Spilling Breakers. Spilling breakers occur when white water initially appears at the wave crest and spreads down the front face. An overturning jet of water is either very small or absent all together. Spilling breakers usually occur on flat or gently sloping beaches.

(b) Plunging Breakers. Plunging breakers occur when the top of the wave crest forms a jet which overturns and plunges into the water in front of the face of the wave. A body of air is initially enclosed between the jet and the wave face, causing a lot of spray and white water. Plunging breakers are most common on moderately steep beaches.

(c) Surging Breakers. Surging breakers occur when a relatively small wave close to the shoreline builds up a crest as in a plunging breaker, but before the jet can form, the bottom of the wave surges forward up to the water line. Surging breakers occur on very steep beaches close to the water line.

These are the three main breaker types, but since there are smooth transitions between them, various sub-classifications have been proposed. The term "collapsing breaker" is sometimes used for breakers between plunging and surging, but further sub-classifications are not standardly used.

Attempts have been made to relate breaker type to some quantifiable property of the wave. The earliest criterion was that of Irribarren and Nogales (1949), who used the beach slope and deep-water wave steepness,

$$I_{\text{off}} = \frac{m}{(H_o/L_o)^{\frac{1}{2}}} \quad (2.1)$$

where m is the beach slope, H_o is the wave height at an offshore point and L_o is the wavelength at that point. The subscript 'off' indicates offshore conditions. I is generally known as the Irribarren number or the Surf Similarity parameter. Galvin (1968) carried out a series of experiments of breaking waves on various bottom slopes and proposed a new parameter,

$$I' = \frac{m}{(H_b/L_o)^{\frac{1}{2}}} \quad (2.2)$$

in which H_b is the wave height at breaking. It is probable that Galvin still used the offshore wavelength, L_o , because of the difficulty in measuring or estimating the wavelength in the surf zone. Nowadays, however, numerical models can calculate local values of H and L even for irregular beaches, and it is logical to define I completely in terms of local variables,

$$I = \frac{m}{(H/L)^{\frac{1}{2}}} \quad (2.3)$$

Yoo (1986) compared both I and I' against Galvin's experimental data and found, rather paradoxically, that I' gave a clearer classification of breaker type than I, despite the obvious weakness of using an offshore parameter, L_0 , to define an inshore process. This prompted Yoo to devise a new surf zone parameter, entirely in terms of local variables but which would give a more reliable breaker classification than I. Yoo's new surf zone parameter, β , is based on the fact that steeper bed slopes retard the speed at which the wave energy travels more strongly than shallower slopes. Using this idea Yoo derived the following surf zone parameter,

$$\beta = \frac{2m^2}{k^2hH} \quad (2.4)$$

in which k is the local wavenumber and h the local depth. β is related to I by

$$\beta = \frac{I^2}{\pi kh} \quad (2.5)$$

Comparisons with Galvin's experimental data indicate that β gives a clearer classification of breaker type than I, although further experimental measurements are desirable. On the basis of present measurements the breaker types can be classified according to

Surging $\beta > 2.1$

Plunging $0.2 < \beta < 2.1$ (2.6)

Spilling $\beta < 0.2$

The rapid dissipation of wave energy during the breaking process provides the dominant mechanism for other hydrodynamic phenomena in the surf zone. Two of the most important are a rising of the still water

level known as 'set-up' and the generation of longshore currents. Both phenomena are caused by the non-linear nature of the waves as they are about to break. An effect of this non-linearity is that the particle orbits are not closed, and there is an excess of momentum flux (known as the radiation stress) in the direction of wave propagation, after averaging over a wave cycle. When the waves break, this excess momentum flux is released from the waves. The longshore component of this excess momentum flux drives a current in the longshore direction, while the onshore component causes a set-up of the water level. The forward momentum of water during breaking and within the surf zone takes place largely between trough and crest level. This is matched by a return flow (or 'undertow') between the trough level and the top of the wave boundary layer, typically a few centimetres above the seabed.

It is not proposed to consider these surf zone processes in detail in this report, but they do serve to illustrate the dominant role of wave breaking in determining surf zone dynamics. It is particularly important therefore that computational models of wave transformation can accurately predict wave energy dissipation during breaking. This report will concentrate on reviewing the methods for achieving this aim. For readers more interested in the computational modelling of the detailed structure and evolution of breaking waves, the review paper of Peregrine (1983) is a good starting point.

3 THE BREAKER HEIGHT CRITERION

3.1 Introduction

An essential initial step in determining the rate of energy dissipation in breaking waves is the

calculation of the maximum wave height allowed by the breaking process for a given water depth, seabed slope and wave characteristics. The various methods of calculating the breaker height are considered in this chapter. Different techniques are used for monochromatic waves and random waves, and it is therefore advantageous to consider the two cases separately, starting with monochromatic waves.

3.2 Monochromatic Waves

The earliest breaker height criterion was based on an analysis of deep-water Stokesian waves. Michell (1893) found that breaking started to occur when the angle of the wave crest reached 120° . This criterion can be expressed alternatively as a limiting wave steepness (ie, the ratio of wave height, H , to wave length, L) of

$$\left(\frac{H}{L}\right)_b = 0.142 \quad (3.1)$$

where the subscript b denotes wave conditions at the onset of breaking. This limiting steepness occurs when the water particle velocity at the crest is just equal to the phase velocity of the wave. If the water particle velocity increases, the wave starts to break. In Equation 3.1, L should be interpreted as the Stokesian limiting wavelength which for deep water is about 20 per cent greater than that of sinusoidal waves of the same frequency.

Equation 3.1 is valid for deep water. As a wave moves into shallow water, the limiting steepness decreases from its value of 0.142, and is affected by the relative depth h/L , and the seabed slope, m . Analysis by McCowan (1891) of the solitary wave in

shallow water showed that breaking starts to occur when

$$\left(\frac{H}{h}\right)_b = 0.78 \quad (3.2)$$

This '0.78' criterion is still the most widely used breaker criterion for shallow water in present-day coastal engineering practice. Laitone (1960) and Wiegel and Mash (1961) have used cnoidal wave theory in shallow water to determine a similar criterion. Their expressions are in terms of elliptic integrals but are well approximated by the formula,

$$\left(\frac{H}{h}\right)_b = 0.727 - 1.12 \left(\frac{H}{gT^2}\right)^{\frac{1}{2}} \quad (3.3)$$

in which g is the acceleration due to gravity and T is the wave period. The additional term on the right-hand side is related to the wave steepness in deep water. Miche (1944) has used Stokes theory to derive a formula valid in all depths of water,

$$\left(\frac{H}{L}\right)_b = \frac{1}{7} \tanh\left(\frac{2\pi h}{L}\right) \quad (3.4)$$

It can be seen that Eq 3.4 reduces to Eq 3.1 in deep water and to a value (0.90) somewhat in excess of Eq 3.2 in shallow water.

These criteria have the limitation of not taking account of the seabed slope. The effect of the seabed slope can be important, allowing wave heights of up to $1.4h$ before breaking takes place. To remedy this, a number of researchers in the late sixties and early seventies attempted to determine empirical relationships between $(H/h)_b$ and the seabed slope, m , using experimental data. The following are three such attempts:

(a) Galvin (1969)

$$\left(\frac{H}{h}\right)_b = \begin{matrix} 1.086 & m \geq 0.07 \\ (1.4 - 6.85m)^{-1} & m < 0.07 \end{matrix} \quad (3.5)$$

(b) Collins and Weir (1969)

$$\left(\frac{H}{h}\right)_b = \begin{matrix} 1.28 & m \geq 0.1 \\ 0.72 + 5.6m & m < 0.1 \end{matrix} \quad (3.6)$$

(c) Madsen (1976)

$$\left(\frac{H}{h}\right)_b = \begin{matrix} 1.18 & m \geq 0.1 \\ 0.72 + 4.6m & m < 0.1 \end{matrix} \quad (3.7)$$

There are some differences between these results but they all show that $(H/h)_b$ increases with m up to a maximum value reached when m is about 0.1. For a flat seabed ($m = 0$), the experimental values are close to the theoretical value, Eq 3.2.

The most widely used of these empirical formulas is the one derived by Weggel (1972) using data from a large number of experiments,

$$\left(\frac{H}{h}\right)_b = \frac{b}{1 + \frac{ah}{gT^2}} \quad (3.8)$$

in which a and b are functions of seabed slope, m , given by,

$$a = 43.75 (1 - \exp(-19m)) \quad (3.9)$$

$$b = \frac{1.56}{1 + \exp(-19.5m)} \quad (3.10)$$

This is the breaker criterion recommended in the American Shore Protection Manual.

Other authors have attempted to refine Weggel's formula using additional experimental data. Scarsi and Stura (1980) suggested the use of Eq 3.8 for values of m greater than 0.05. For smaller values they proposed the formula,

$$\left(\frac{H}{h}\right)_b = (0.727 + (13m)^2) - (1.12 + (30m)^2) \left(\frac{H}{gT^2}\right)^{\frac{1}{2}} \quad (3.11)$$

This gives a better fit to the experimental data in this range of m , although Eq 3.8 and Eq 3.11 do not match at $m = 0.05$. Eq 3.11 does reduce to the formulas of Collins and Weir (Eq 3.6) and Madsen (Eq 3.7) for $m = 0$, and to the formula of Laitone (Eq 3.3) for solitary waves ($T \rightarrow \infty$). More recent formulas have been put forward by Singamsetti and Wind (1981),

$$\left(\frac{H}{h}\right)_b = 1.16 \left[m \left(H_o / L_o \right)^{-\frac{1}{2}} \right]^{0.22} \quad (3.12)$$

and Sunamura (1981),

$$\left(\frac{H}{h}\right)_b = 1.1 m^{1/6} \left(H_o / L_o \right)^{-1/12} \quad (3.13)$$

The formulas of these authors are based on experimental data in shallow water and do not apply to deep water. To obtain a formula that is uniformly valid, researchers such as Ostendorf and Madsen (1979) and Battjes and Janssen (1978) have returned to the Miche formula (Eq 3.4) and reworked it to include the effects of the seabed slope. These authors have suggested the equation,

$$\left(\frac{H}{h}\right)_b = \frac{1}{7} \tanh \left[\frac{7h}{L} \left(\frac{H}{h}\right)_{b, \text{shal}} \right] \quad (3.14)$$

in which $(H/h)_{b, \text{shal}}$ is the breaker height to depth ratio in shallow water.

Ostendorf and Madsen recommend Eq 3.7 for $(H/h)_b, \text{shal}$ but Eq 3.8 is probably a better choice in view of the inclusion of wave steepness and its greater experimental justification. It can be seen that Eq 3.14 reduces to Eq 3.8 in shallow water ($\tanh x = x$) and to Eq 3.1 in deep water ($\tanh x = 1$).

Yoo (1986) has re-analysed the experimental data used by Weggel and included more recent results from Van Dorn (1978) and Iwagaki et al (1974). He attempted to relate $(H/h)_b$ in shallow water to a suitable surf zone parameter. A regression analysis to a hyperbolic tangent function yielded the following best-fit curves,

$$\left(\frac{H}{h}\right)_b = \frac{2\pi}{7} [0.8 + \tanh (1.06 I)] \quad (3.15)$$

where I is the local Irribarren number, Eq 2.3, and

$$\left(\frac{H}{h}\right)_b = \frac{2\pi}{7} [0.8 + \tanh (3.0\beta)] \quad (3.16)$$

where β is Yoo's surf zone parameter, Eq 2.4. There is little to choose between Eq 3.15 and Eq 3.16 in terms of agreement with experimental data.

It has been found by a number of researchers that some surf zone phenomena, driven by the wave breaking process, are initiated not at the breaker point but at the plunge point (for plunging breakers). The phenomena affected in this way are the ones related to the changes in the wave radiation stresses as the waves break. Such phenomena include the set-up of the still water level and the generation of longshore currents. The reason is that the excess momentum, averaged over a wave cycle, is not released from the wave by the breaking process until the plunger strikes

the still water in front of the wave. It was found in Southgate (1988) that this effect could be simulated in the Weggel breaker criterion by replacing the 1.56 factor in Eq 3.10 by a variable $2a'$, so that the expression reads,

$$b = \frac{2a'}{1 + \exp(-19.5 m)} \quad (3.17)$$

In the original expression a' would therefore take the value 0.78. By increasing this value the onset of breaking will appear to occur further shorewards. A value of $a' = 1.18$ was found to give good agreement with experimental measurements of wave set-up and longshore current velocities. This 'tuning' of the breaker criterion depends on whether wave heights or radiation-stress-related quantities are of principal interest in a particular application.

In view of the scatter of experimental data on which the various breaker height criteria have been based, it is difficult to make a firm recommendation for any particular one. However, it is clear that the seabed slope, m , has an important effect on the breaker height, and therefore those criteria which do not take the seabed slope into account (including the '0.78' criterion) are not recommended except on flat or very nearly flat seabeds. The Weggel formula, Eq 3.8, includes the effect of offshore wave steepness (related to the gT^2 term) as well as seabed slope and has been derived from a large quantity of experimental data. This formula has been widely used in coastal engineering practice. For these reasons it is probably the most reliable existing formula for the shallow water breaker height, and can be extended to be valid for all depths using Eq 3.14. It can, furthermore, be tuned to the plunge point for the calculation of radiation-stress-related quantities, rather than wave heights, using Eq 3.17.

3.3 Random waves

In most real sea states, the wave energy is spread over a spectrum of periods and directions, rather than existing at a single period and direction. Waves in such a sea state are known as random waves. Modern computational models can describe the transformation of random waves as they approach the shoreline. A feature of random wave activity in shallow water is that there is no single well defined breaker line. Instead, higher waves will break further offshore and smaller waves nearer the coast, so creating a zone, rather than a line, where breaking occurs.

In order to analyse the breaking of random waves, within the context of a computational model of wave transformation, it is necessary to know the probability distribution of wave heights. The Rayleigh distribution, Eq 3.18, has been found to represent well most offshore random sea states,

$$P(H) dH = \frac{2H}{H_{rms}^2} \exp(-H^2/H_{rms}^2) dH \quad (3.18)$$

where H_{rms} is the root-mean-square wave height, and $P(H) dH$ is the probability of finding a wave height in the range dH centred on H . In shallow water the wave height probability distribution will be truncated at the breaker height (see Fig 2). The distribution will furthermore be distorted from the Rayleigh form by the shallow water wave processes (refraction, shoaling, etc), and some attempts have been made to modify Eq 3.16 for shallow water (Hughes and Borgman (1987)). However, a comparison with field data carried out by Thornton and Guza (1986) indicated that the Rayleigh distribution, suitably truncated, is still a good approximation in shallow water. The methods of determining the breaker height are the same as for monochromatic waves; the effect of a probability

distribution of wave heights on the decay of wave energy after breaking will be considered in the following chapter.

An alternative approach to modelling random waves is by representation as a spectrum in period and direction. Bouws et al (1984) have proposed an extension of the deep-water JONSWAP spectrum to be valid for all depths. Their new spectrum, known as the TMA spectrum, was used by Vincent (1984) to determine the significant wave height, H_s , under random waves for given values of depth, peak frequency and wind speed. The significant wave height is defined in Vincent's method as 4 times the square root of the zero spectral moment. Vincent's analysis gives for H_s ,

$$H_s = \frac{2\alpha^{\frac{1}{2}}}{k_m} \quad (3.19)$$

where k_m is the wavenumber corresponding to the peak frequency (f_m), and α is an extension of the Phillip's constant in the JONSWAP equation. Vincent proposed for α ,

$$\alpha = 0.0078 \left(\frac{U^2 k_m}{g} \right)^{0.49} \text{ power} \quad (3.20)$$

where U is the wind speed at 10m elevation. In the surf zone α can rise to a maximum value, representing the fact that wave heights are limited by the water depth. This maximum value is given by

$$\alpha_{\max} = 0.09 k_m^2 h^2 \quad (3.21)$$

* corresponding to an H_s/h ratio of 0.6

Being based on an analytical expression for the wave spectrum at all depths, Vincent's method is not as

well suited for inclusion in a wave transformation model as the approach using a wave height probability distribution. However, Vincent's method is valuable when used without a wave transformation model to determine H_g and its limiting breaking value, at inshore points of interest. For these applications, Vincent's method is in principle to be preferred to the monochromatic wave methods where the sea state is predominantly random, although greater experience of its use in practical problems is required.

Research has been carried out to extend the spectral representation of deep-water waves to include breaking. Experiments carried out by Ochi and Tsai (1983) indicated the following criterion for the breaking of deep-water random waves,

$$\left(\frac{H}{L}\right)_b = 0.126 \quad (3.22)$$

in which H is the wave height between a minimum surface elevation and the following maximum, and L is the distance between successive maxima. Using this criterion, Ochi and Tsai carried out a lengthy statistical analysis to determine the relationship between the significant wave height and the probability of breaking. Their analysis is dependent on the type of wave spectrum and requires a computational routine. Tung and Huang (1986) have carried out a similar type of statistical analysis based on a criterion that breaking occurs when the downward acceleration of the wave crest exceeds a certain fraction of the gravitational acceleration. Again, their analysis is very lengthy and requires a computer model. Readers are referred to the original papers for details.

4 ENERGY DISSIPATION IN BROKEN WAVES

4.1 Introduction

The previous chapter considered the methods of predicting the wave height at which waves start to break at a given location. Once waves have broken they will continue to travel forwards, but in a manner quite different from unbroken waves. This chapter is therefore concerned with the prediction of the (spatial) rate at which wave energy is dissipated from a broken wave as it travels shorewards. As in the previous chapter the methods of treating monochromatic broken waves and random broken waves are considered separately.

4.2 Monochromatic Waves

There are two basic approaches to the modelling of energy decay of monochromatic waves after breaking. The first approach is applicable only to wave breaking on beaches and is based on empirically-derived relationships between the broken wave height, H , and the water depth, h . The simplest, and most commonly used, relationship is the linear one,

$$H = \gamma h \quad (4.1)$$

where γ is defined as the ratio of H to h at breaking. Essentially it is assumed here that, as a broken wave travels inshore, it has a wave height equal to the highest allowed unbroken wave height. This implies that the various formulas described in Chapter 3 for $(H/h)_b$ can be used for γ in Eq 4.1. However, several investigators have established that the energy decay of broken waves in the surf zone

deviates significantly from this linear relationship. Experimental measurements typically show a concave curve in graphs of H versus h, indicating a relatively large dissipation of energy immediately after breaking, and a progressively smaller rate of dissipation further shorewards.

To better represent this concave profile, Smith and Kraus (1987) have proposed a power law for the wave height decay,

$$H = \gamma h_b \left(\frac{h}{h_b}\right)^n \quad (4.2)$$

where the subscript b denotes the breaking condition, and the exponent n, to be empirically determined, is dependent on the beach slope and breaking wave conditions. A comparison of predictions using Eq 4.2 with experimental data, principally from Horikawa and Kuo (1966), showed that n depended on γ and the bed slope, m. A multiple regression against the experimental data gave the following formula for n,

$$n = 0.657\gamma + \frac{0.0438\gamma}{m} - 0.0096m + 0.032 \quad (4.3)$$

γ was obtained from a suitable formula for $(H/h)_b$ such as those outlined in Section 3.2. Smith and Kraus were able to obtain a better fit to the experimental data using this method, although an additional empirical factor has had to be introduced. Miller (1987) has presented a similar type of analysis based on an exponential decay law.

These empirically-based methods have the advantage of simplicity but ideally require site-specific calibration. They are limited to the surf zone in front of a beach, and cannot be used seawards of the

breaker line or for depth profiles which do not vary monotonically, such as a bar-trough formation.

The second type of approach to modelling the energy decay of broken waves is based on solving the equation for wave energy balance,

$$\nabla \cdot (E c_g) = - D_b \quad (4.4)$$

where E is the mean wave energy density, c_g is the group velocity of the waves, D_b is the spatial rate of dissipation of wave energy flux by breaking, and ∇ is the 2-D horizontal gradient operator. For small amplitude linear waves E is given by,

$$E = \frac{1}{8} \rho g H^2 \quad (4.5)$$

where ρ is the water density.

In principle this method is to be preferred since it has a well-founded physical basis rather than relying on an ad hoc relationship between H and h in the surf zone. The method has other advantages. Additional dissipative or generative processes such as bottom friction and wind growth can be added to the right-hand side of Eq 4.4, thereby giving a straightforward means of combining such processes. There are, furthermore, no restrictions as to where Eq 4.4 can be used. It can, for instance, be used to transform waves all the way from deep water to the waterline on a beach, and can be used for any type of depth profile, including the bar-trough form. The main drawback to the use of Eq 4.4 has been the lack of knowledge of the physical mechanisms underlying the breaking process and hence of a reliable expression for D_b . It is probably for this reason that the

simpler empirically-based approach has been more popular. Nevertheless, with a greater understanding of energy losses in breaking waves, and the use of sophisticated computer models, methods based on Eq 4.4 are being increasingly used.

The most commonly used expression for D_b is based on the analogy with a tidal bore, a phenomenon similar in appearance to a broken wave,

$$D_b = \frac{\lambda \rho g^{3/2} k H^3}{8\pi h^{1/2}} \quad (4.6)$$

in which λ is an empirical constant, of the order one, to account for the differences between the breaking wave and tidal bore processes. Le Mehaute (1963) was the first to use this form for the dissipation of broken wave energy, and was followed by Divoky et al (1970), Hwang and Divoky (1979) and Stive (1984), all of whom compared breaking wave models based on the tidal bore analogy with experimental data using monochromatic waves. Svendsen (1984) also used Eq 4.6 but attempted also to take account (partially) of the wave non-linearity at breaking by altering the "1/8" factor in Eq 4.5 to a value more appropriate to the non-linear wave form. He also introduced an explicit expression for λ in terms of the wave crest elevation and the wave height-to-water depth ratio, H/h .

There exist other approaches to the problem of the decay of broken wave energy. Mizuguchi (1981) used the formula for energy dissipation due to internal viscosity, but replaced the kinematic viscosity by an eddy viscosity term,

$$D_b = 0.5 \rho g v_e (k H)^2 \quad (4.7)$$

in which v_e is the eddy viscosity coefficient. This

method has the practical difficulty of obtaining a reliable prediction of v_e . Dally et al (1985) assumed that the rate of dissipation of broken wave energy is proportional to the difference between the actual energy flux and a lower stable flux level,

$$D_b = \frac{K}{h} [E c_g - (E c_g)_s] \quad (4.9)$$

where K is a factor to be empirically determined, and the subscript s denotes the stable energy flux. In their analysis Dally et al used Eq 4.1 in their expression for $(E c_g)_s$.

$$(E c_g)_s = \frac{\rho g c_g \gamma^2 h^2}{8} \quad (4.10)$$

giving the model a hybrid character involving both types of basic approach. There are two empirical parameters to be determined, K and γ . Based on laboratory data from Horikawa and Kuo (1966), Dally et al recommended values of $K = 0.15$ and $\gamma = 0.4$. Ebersole (1987) has compared the models of Dally et al (1985) and Svendsen (1984) against field data, both models giving an rms error of about 10%. The scatter of field data tended to mask any genuine comparison of the models.

4.3 Random waves

The same two basic approaches to modelling the dissipation of broken wave energy apply equally to random waves as they do to monochromatic waves. The earlier random wave models were based on Eq 4.1 to provide a cut-off value to the local wave height distribution (Collins (1970), Battjes (1972) and Goda (1975)). Battjes and Janssen (1978) were the first to apply the energy balance equation, Eq 4.4, to the energy dissipation of random broken waves. In their method the shallow-water height distribution was taken

to be a Rayleigh distribution truncated at the breaker height. They used a modified Miche criterion (Eq 3.14) to determine the breaker height, but in principle any of the methods in Section 3.2 could be used. The truncated Rayleigh distribution therefore represented the probability distribution of unbroken wave heights. For the broken waves, Battjes and Janssen assumed that the wave height would initially be equal to the breaker height, resulting in a spike representing broken wave energy in the modified Rayleigh distribution (Fig 2). The "tidal bore" equation, Eq 4.6, was then used in conjunction with the energy balance equation, Eq 4.4, to determine the rate of energy decay of the broken wave as it travels forward.

Battjes and Stive (1985) included a comparison of this type of model with laboratory and field data for plane and barred beaches and a shoal. Southgate (1988) compared wave height, set-up and longshore current predictions using Battjes and Janssen's method with experimental data for a plane slope. Both investigations revealed good agreement between theory and measurements. Thornton and Guza (1983) have modified the Battjes and Janssen method, taking into account the detailed distribution of broken wave heights in the calculation of D_b rather than simply using an rms value. However, in their comparison with field data, this modification does not appear to have made a significant improvement to the prediction of broken-wave energy dissipation.

5 CONCLUSIONS AND RECOMMENDATIONS

Wave breaking is probably the most physically complex process that surface water waves undergo. In order to render any theory of breaking amenable to hand calculations or computational models of wave transformation, the representation of breaking has to

be considerably simplified. Furthermore, there is an inherent variability in the process, demonstrated by the scatter of experimental data. In view of this, predictions of breaking wave heights and energy losses cannot be considered reliable for individual waves, although they can represent average values over a series of waves which are sufficiently accurate for engineering applications. Bearing these points in mind, the following conclusions and recommendations can be made.

- a) Breaker Height. Experimental measurements show a definite dependence of breaker height on bed slope, and consequently formulas which do not include the effect of bed slope are not recommended. Of those that do, no one formula gives significantly better agreement with experimental data. For monochromatic waves, the Weggel formula (Eqs 3.8-10) has been compared with a large quantity of experimental data, and has been widely used in coastal engineering practice. It can also be tuned to the plunge line, rather than the breaker line, for the prediction of surf zone processes (Eq 3.17). For computational models of the propagation of random waves, good comparisons with laboratory and field measurements have been obtained using a Rayleigh probability distribution and a monochromatic breaker height formula. For hand calculations of random waves in shallow water, the method of Vincent (Eq 3.19-21) is appropriate. Although Vincent's method compares well with laboratory and field data, further experience of its use in practical engineering problems is desirable.

- b) Wave Energy Dissipation. For monochromatic waves in the surf zone a relationship between broken wave height, H_b , and water depth, h , such as that of Smith and Kraus (Eqs 4.2-3) is preferable to

the commonly-used linear relationship. The method is limited to monotonically varying depth profiles. An alternative approach, based on the wave energy balance equation, Eq 4.4, is physically better-founded and can be used for any type of depth profile and be combined readily with other dissipative or generative wave processes. This approach is, in principle, preferable to an ad hoc relation between H_b and h , but requires a computational wave transformation model. The recommended formula for the rate of dissipation of broken wave energy is based on the analogy with a tidal bore (Eq 4.6). This method can be extended to random waves using the method of Battjes and Janssen (Section 4.3).

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Figures

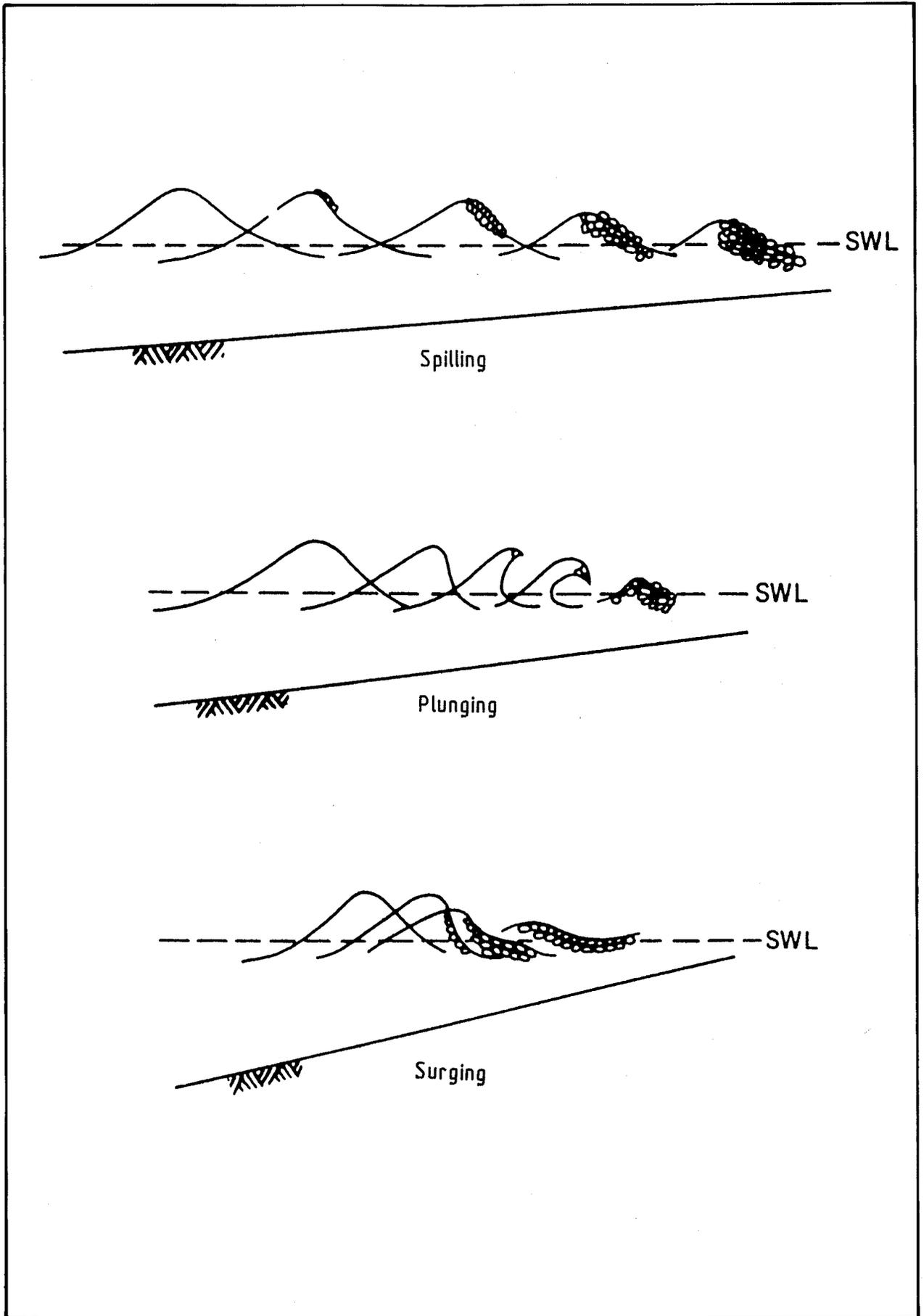


Fig 1 Classification of Wave Breaking

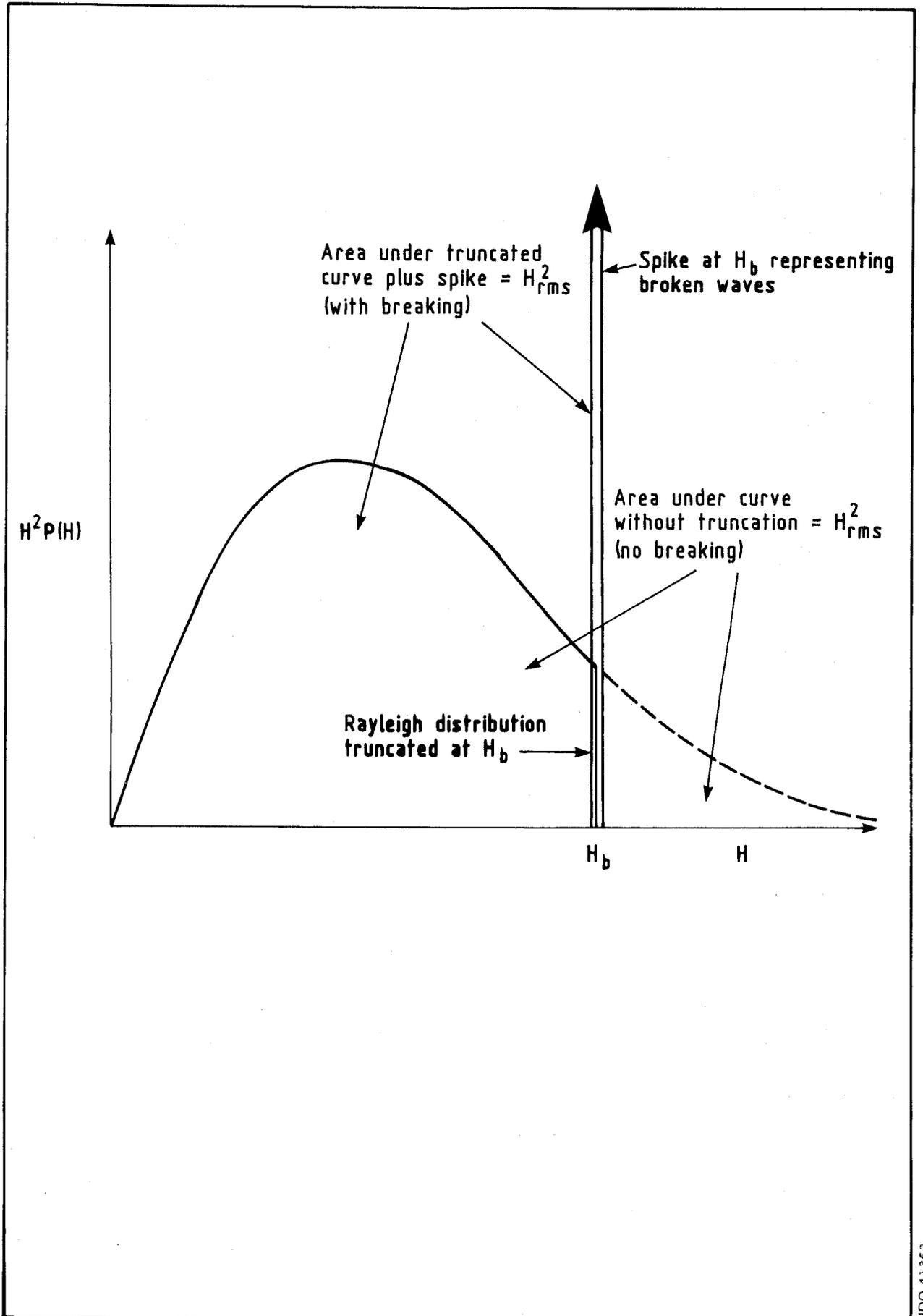


Fig 2 Truncated Rayleigh distribution of wave heights for broken waves