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BREAKING WAVES

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SUMMARY

Literature on breaking waves is reviewed in the context of offshore structure design. Particular attention is paid to deep and intermediate water. The review covers publications up to 1993 on the probablilty of breaking, wave elevations, particle kinematics and resultant forces on typical members. The effects of three-dimensionality and randomness on the water particle kinematics and slam and run-up on vertical members are considered.

Recommendations are made for alteration of the design wave in respect of elevation/height ratio; deep water steepness limits; and intermediate water breaking limits. Further studies are recommended in the areas of probability; the variation of force coefficients with elevation and phase; and the effects of three-dimensionality and randomness on kinematics and forces.

CONTENTS

NOTATION

a	downward acceleration at the wave crest
c_p	phase celerity of wave crest
C_d	Morison's drag coefficient
C_m	Morison's inertia coefficient
C_s	coefficient of slam
F_i	component of force on a pile from Morison's equation
F_s	slam force
g	gravitational acceleration
H	wave height
I	surf-zone parameter
k	wave number
m_4	fourth spectral moment
Q	fraction of waves which are breaking
s	wave steepness
T	wave period
u	horizontal water velocity
α	coefficient of acceleration (equ 6)
β	surf-zone parameter
ε	wave slope ratio
γ	wave height/depth ratio
η	elevation of crest above swl
λ	wave length
μ	vertical asymmetry

1. INTRODUCTION

1.1 DEEP WATER

There are two commonly perceived causes of wave breaking: limiting steepness and limiting depth. This may be easily mapped onto deep water and shallow water breaking mechanisms. This section will deal with deep water breaking mechanisms. Waves in which the bottom topography plays an important role are dealt with in Section 1.2.

This chapter will concentrate on breaking wave probability and breaking wave limits in the context of offshore wave loading. Banner and Peregrine (1993) provide a more general introduction to deep water breaking waves, covering experimental and theoretical developments for probability and kinematics.

1.1.1 Regular wave limits

In deep water, regular, 2-D waves the maximum possible wave height, H, of the Stokes wave is about 1/7 of the wavelength, L, and the limit may be expressed in terms of the wave period, T, as:

$$H_{max} = sgT^2 \qquad 1$$

where s=0.027. This wave also has a limiting downward acceleration in the crest of a=0.5g and a horizontal water particle velocity at the crest of $u=c_p$, the wave celerity (Longuet-Higgins, 1969).

This theoretical wave does not show some of the key attributes of an extreme water wave. The main difference is that it does not show the observed asymmetry between the forward and rear crests. However, it does demonstrate the asymmetry between the amplitude of the crest, η, and the depth of the trough which is observed for large waves. The values for the limits of the Stokes regular wave have been taken to be indicators above which breaking occurs. Therefore we may state that, for a regular wave, breaking will occur if one of the following conditions is satisfied:

$$s > 0.027;\ a > 0.5g;\ u > c_p \qquad 2$$

As well as giving us the specific conditions of Equation 2, this approach has also given the main methods by which we analyse wave records to determine predictors of breaking. In other words, even in a fully 3-dimensional random sea, concepts such as the wave steepness, particle velocity and acceleration, and other geometrical parameters associated with individual crests are used as predictors of breaking.

Kjeldsen (1981) has summarised the significant geometrical parameters for a wave as shown in figure 1. The most significant of these is the horizontal asymmetry factor, μ, representing the degree to which the height of the wave is due to the elevation of the crest, η,

$$\mu = \eta / H \qquad 3$$

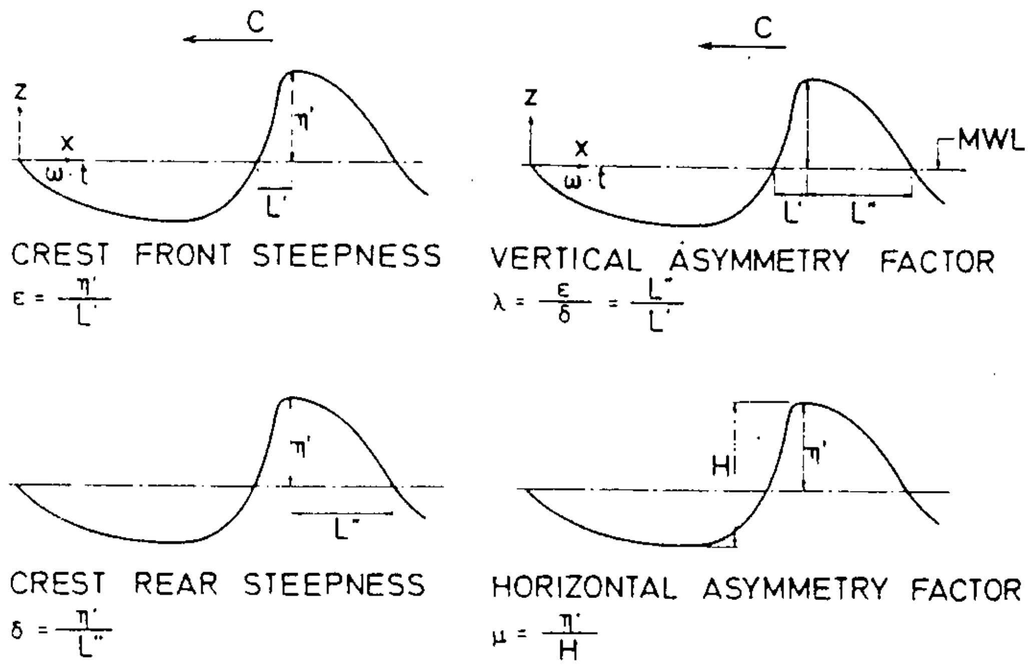

Figure 1
Definition of geometric parameters for large waves (Kjeldsen, 1981)

For a small amplitude wave μ=0.5; for a Stokes limiting wave μ=0.67. This gives a further condition to add to those of Equation 2 for the regular breaking wave, namely that a wave will be breaking if

$$\mu>0.67 \qquad 4$$

Unfortunately, it is not possible to generate a steady breaking wave to test these limits. Breaking results in a loss of energy and is therefore a transient event for which there must be some energy source. However, they should appear as limiting cases for random waves with very narrow band spectra, shoaling waves where the water is fairly deep and the bed slope is small, and 3-D waves where the wave spreading is negligible.

She (1994) shows that, for a regular, focussed wave in a 3-D tank the crest asymmetry, μ, is not sensitive to angle and has a mean value of 0.65. However, the wave steepness was found to be sensitive to angle, but was close to the Stokes limit for the smallest spreading angle investigated (30 degrees), with s=0.026. Similarly, although asymmetry is a function of depth, for the deepest shoaling wave experiments, Griffiths (1992) finds μ =0.68. However, his greatest wave steepness is s~0.020, significantly less than the Stokes value.

1.1.2 Random waves and groupiness

The most commonly accepted limit for wave breaking in random seas is that proposed by Ochi and Tsai (1983), based on the wave steepness,

$$H=0.020gT^2 \qquad 5$$

This is significantly lower than the regular wave condition (equation 2) and was derived from wave flume tests with representative sea spectra. Ochi (1990) also reports values of s=0.019 and s=0.021 obtained by independent researchers. Ochi demonstrates how this relationship can be used to predict the probability of breaking based on the joint probability of period and waveheight with the cut-off relationship above (figure 2).

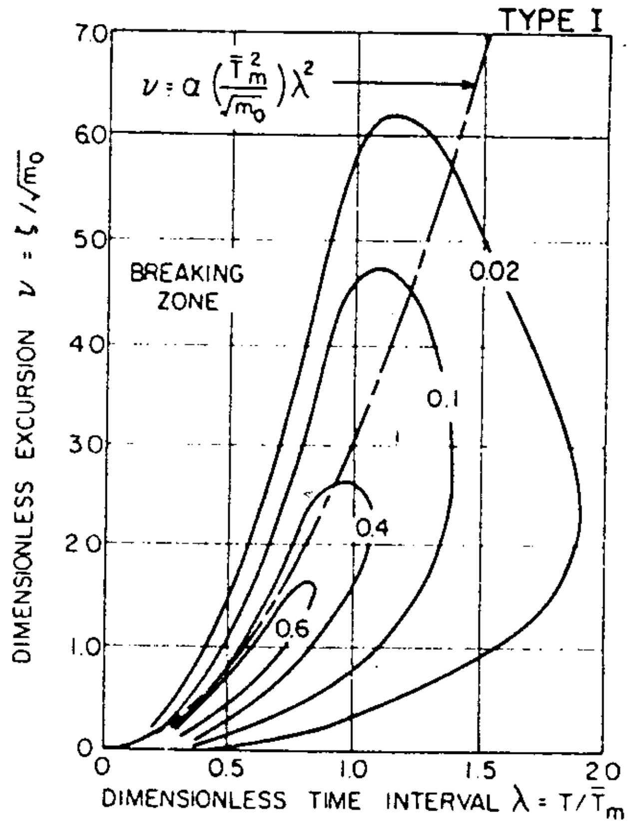

Figure 2
Non-dimensionalised breaking wave height and period with cut-off value predicted by Ochi (1983)

In his thesis on the kinematics of wave groups, Sutherland (1992) found that waves in a shorter group broke at lower height. The study was on the particle velocities in two component waves with the number of waves in the group varying from 3 to 12, with comparisons being made with various theories and with measured regular wave kinematics. It was found that, in the shortest groups, the maximum steepness which could be obtained was 65% of the regular limit.

Srokosz (1986), has related the acceleration condition for breaking with the steepness relationship and found that s=0.020 is equivalent to a=0.4g, which is smaller than the result for regular waves. Srokosz also pointed out that the theory is based on Gaussian statistics and would break down if the sea surface was non-Gaussian. Nevertheless, he produced an equivalent formulation for breaking probability to that of Ochi, in terms of acceleration, with

$$B = \exp\left(-\frac{\alpha^2 g^2}{2m_4} \right) \qquad 6$$

where m_4 is the fourth spectral moment, and α is the coefficient of acceleration in $a=\alpha g$.

Snyder and Kennedy (1983) use a different approach, still based on the acceleration criterion, to find the proportion of the sea surface covered by breakers,

$$\beta = 1 - \Phi\left(\alpha\, g m_4^{-1/2}\right) \qquad 7$$

Both of the above methods rely on the fourth moment of the spectrum and the cut-off frequency of the calculation or the measuring system, if applied to experimental or field measurements.

The asymmetry value (Equation 4) is not so well researched for random waves. Bonmarin (1989) watched regular waves in a long tank modulate due to their instability and measured the characteristics of breaking. All waves had $s< 0.027$, and he divided the results for asymmetry into spilling and plunging breakers,

Plunging	$0.62 < \mu < 0.93$	$\bar{\mu} = 0.77$	8
Spilling	$0.59 < \mu < 0.8$	$\bar{\mu} = 0.69$	9

This indicates that plunging waves have a higher crest-to-trough ratio than spillers, and that a breaking criterion of $\mu=0.6$ is appropriate for random waves.

1.1.3 Three-dimensional waves

Very little work has been done on the effects of 3-dimensionality on wave breaking. However, the results which do exist are particularly significant, in that they question the whole basis of the above formalism.

Mather (1988) carried out studies on the joint probability of period and wave height for waves generated in the 3-D facility at the University of Edinburgh. The purpose of the work was to validate Ochi and Tsai's steepness limit. However, the results were completely unexpected, showing that, although $H=0.020gT^2$ did indeed form an upper limit for the probability density function (figure 3), it did not provide a breaking criterion. In fact, the breaking waves were evenly scattered throughout the H,T plane, indicating that, in a random, 3-dimensional sea, waves of any steepness were likely to break.

Furthermore, in a similar, offshore study (Holthuijsen, 1986), the breakers were again marked on the wave record and separate probability density functions plotted (figure 4). The histograms of the joint probability density functions were remarkably similar again, although the mean height of breakers was about 30% greater than for all waves. They also plotted the likelihood that any wave group would contain a breaking wave, and, in confirmation of Sutherland's result above, found that a wave in a shorter group was more likely to break. Further findings were that the average position of a breaker was forward in the group, and that the elevation of breaking waves was 60% greater than the elevation of the non-breakers. However, they also state (but, unfortunately do not show) that other geometric criteria such as the asymmetry, μ, are not sufficient to discriminate breaking waves from non-breaking waves.

In a study to investigate purely the 3-D effects, without the random nature of the wave spectrum, She (1994) looked at the focussing of a regular wave in a 3-D tank. The spread of the waves was increased from 30 to 80 degrees and it was found that the steepness at which the wave broke increased from $s=0.026$ to $s=0.040$, for waves with components focussing from almost a full half-circle. The asymmetry remained at around μ=0.65 for all angles, in agreement with the value for a regular Stokes wave.

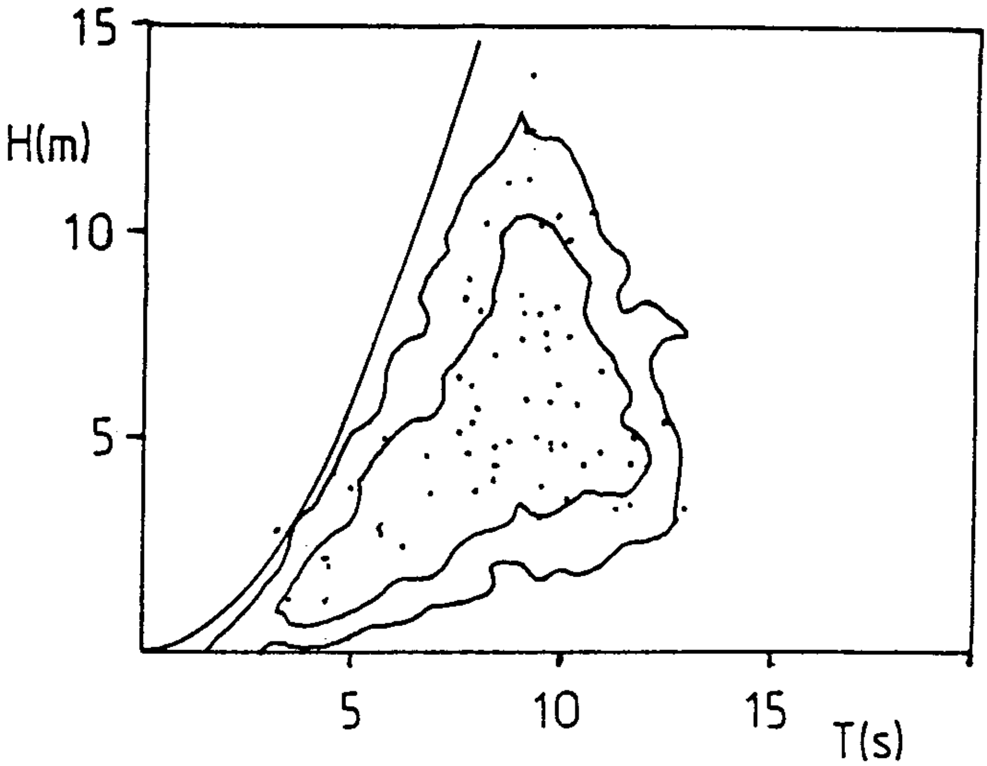

Figure 3
Joint probability of wave height and period for 3-D basin tests. The data has been scaled to typical offshore values. The contours are the probability density function for all waves. The points are breaking events. The theoretical limit s=0.020 is also shown.

In tests performed in a flume, but with a circular paddle, short-crested regular waves were generated (Kolain, 1993). These were found to modulate into groups. Values of $0.015<s<0.034$ were observed, in common with the Sutherland group results ($s>0.014$) and the She 3-D results ($s<0.040$). Kolain proposes an explanation for the high upper limit based on the theory that, as the wave is composed of components from different angles, the forward fluid particle velocity (added vectorially) will be smaller than its equivalent long crested wave of the same height. A shorthand calculation based on the down-wave (η_x) and cross-wave (η_y) slope is proposed,

$$s=0.027(1+\varepsilon^2) \quad \text{where} \quad \varepsilon = \frac{\sqrt{\eta_y^2}}{\sqrt{\eta_x^2}} \qquad 10$$

This reduces to the standard case for a long crested wave (ε=0), and for a wave which is circular, gives ε=1, and an upper limit of steepness s=0.054. It is worth noting here that the largest wave ever produced in the 3-D basin at Edinburgh was a bulls-eye with 180 degree components and frequency focussing which had an estimated steepness (H/L) of 1 in 4, equivalent to s=0.050.

However, Kaolin also concluded that wave steepness was not a good predictor for breaking.

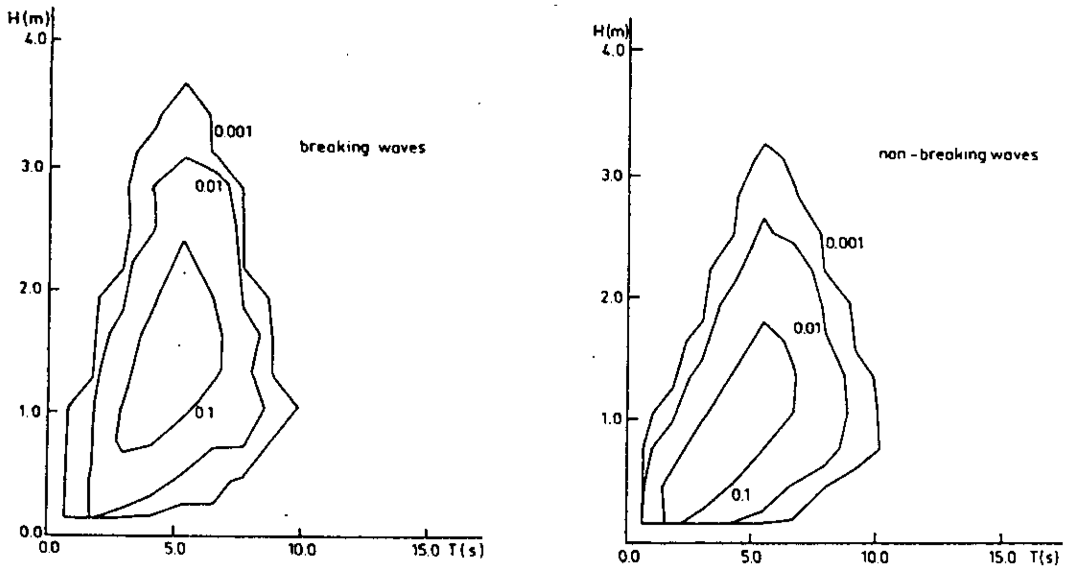

Figure 4
Joint pdf from measured offshore data (Holthuijsen, 1986). Note the overlap between the two data sets.

1.1.4 Summary

It would appear that the simple, geometric formulation of the breaking criteria break down when applied to very wide band seas and fully 3-dimensional seas. In his paper on velocity measurements at the free surface Melville (1988) anticipated that "simple 'breaking criteria' based on local wave properties may prove to be incorrect or at least ambiguous." Studies in which the proportion of waves observed to break and the proportion predicted to break are in agreement may well be fortuitous, as randomness and 3-dimensionality act in opposite senses on the steepness criterion.

1.2 SHALLOW WATER

In addition to the above complications of randomness and 3-dimensionality, shallow water waves are influenced by depth and bed topography. The joint effect of these has not been the subject of significant study, partly because, in the presence of shallow water, waves tend to become longer crested and to align themselves perpendicular to the bed slope. Only 2-dimensional waves will be looked at here.

1.2.1 Regular waves

The well-known breaking criterion for shallow water wave design is

$$\frac{H}{h} = 0.78 \qquad 11$$

This is still used by inshore designers as the upper limit on wave height due to breaking. However, it was derived from the flat bed solution to the shallow water solitary wave problem. This is not directly applicable to most design situations, and there has been considerable effort devoted to modifying this simple equation to take account of the bed slope, the offshore wave steepness and the random nature of the wave field. Southgate (1993) gives an excellent review of the literature.

There are four main breaking criteria (known as surf zone parameters) which have been studied. All are similar in form and are given below (notation after Southgate):

$$I_0 = \frac{m}{\left(H_0 / L_0\right)^{1/2}}$$ Irribaren (1949) 12

$$I' = \frac{m}{\left(H_b / L_0\right)^{1/2}}$$ Galvin (1968) 13

$$I = \frac{m}{\left(H_b / L_b\right)^{1/2}}$$ Yoo (1986) 14

$$\beta = \frac{2m^2}{k_b^2 h H_b} \qquad \left(= \frac{I^2}{\pi k_b h} \right)$$ Yoo (1986) 15

where m is the local bed slope, k is the wave number, and the subscripts b and o indicate parameters at the breaking point and offshore respectively. The values of these numbers delimit the type of breaker which can be expected, but also feed into the equation for the maximum wave height before breaking as,

$$\gamma_s = \left(\frac{H}{h}\right)_b = \frac{2\pi}{7}\left(0.8 + \tanh(1.06I)\right)$$ Yoo (1986) 16

which gives the best fit to date, according to Southgate.

Miche (1944) proposed that this be extended to intermediate water depths by,

$$\left(\frac{H}{L}\right)_b = \gamma_d \tanh\left[\left(\frac{h}{L}\right)_b \frac{\gamma_s}{\gamma_d}\right]$$ 17

where γ_d is the offshore limiting wave steepness for regular waves (nominally 0.142).

Griffiths (1987) and Easson (1988) performed experiments with breaking waves on various bed slopes and compared the results with those of other investigators and the empirical Equation of Weggel (1972), currently used in the American Shore Protection Manual (figure 5). They found general agreement with the other investigators and were able to extend the results towards deeper water, where wave steepness and height/depth ratio both have an influence. For flat beds γ was seen to tend towards the shallow water limit of 0.78, but this rapidly reduced to approximately 0.6 at intermediate depths.

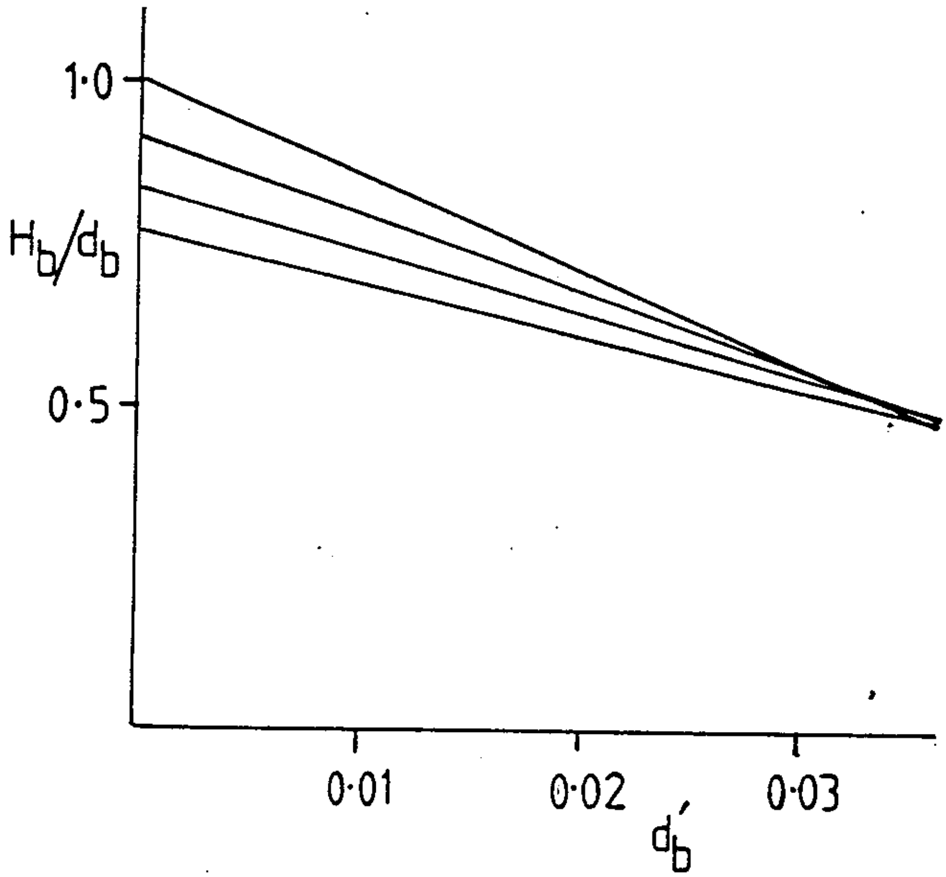

Figure 5
Breaking wave limits for various bed slopes. From the top down the slopes are 1:15, 1:30, 1:50, flat. Adapted from Griffiths (1989).

1.2.2 Random Waves

The breaking of random waves in shallow water has not been the subject of such detailed study as any of the other topics discussed. Most investigators still use a regular wave breaking criterion, partly because it is assumed that the waves will behave in a regular manner, being fairly non-dispersive. However, work which has been carried out (Battjes and Stive, 1978) indicates a different relationship for irregular waves,

$$\gamma_s = 0.5 + 0.4\tanh(33 s_0) \qquad 18$$

where s_0 is the offshore wave steepness (H/L). They found no slope dependant behaviour in their tests. The waves from which Battjes derives Equation 19 have very small offshore wave steepnesses (0.01-0.04), around 7%-28% of the deepwater limiting wave height, and the variation in γ is not significant over that range, going from 0.6-0.8 with an error of +/- 0.1. Indeed, a straight line at the regular wave value of 0.78 would provide a perfectly good fit to the higher offshore steepness data. The main thrust of the paper is an energy dissipation model, and the wave-by-wave study is limited.

The energy dissipation relies on the prediction of the fraction of waves which are breaking, Q, given by,

$$\frac{1-Q}{-\ln Q}=\left(\frac{H_{rms}}{H_m}\right)^2 \qquad 19$$

where H_{rms} is the rms height of all waves and Hm is the nominal depth limited height given by,

$$H_m = 0.88\,k_p^{-1}\tanh(\gamma_s\,k_p h/0.88) \qquad 20$$

Ewing (1993) has proposed a model to predict the proportion of breakers using the surf similarity parameter, and the equations from Battjes model. The implementation is as follows:

1. Find Hrms from the significant wave height:

$$H_{rms}=H_s/\sqrt{2} \qquad 21$$

2. Find L_0 from the zero crossing wave period:

$$T_0 = 1.28T_z; L_0 = gT_0^2/2\pi \qquad 22$$

3. Calculate. γ using H_{rms} and L_0 for s_0 in Equation 19 and use this value to calculate the breaking limited wave height :

$$H_b = \gamma\,h \qquad 23$$

4. The values for H_b and H_{rms} may now be used in the recursive Equation 20 to calculate the proportion of waves which are breaking.

There are a number of concerns with this approach:

1. The formulation is for shoaling waves and these are not shoaling;
2. Offshore values have been replaced by local values without modification;
3. Battjes reports values for his own results, and those of other investigators of $0.1 < Q < 0.3$. Ewing predicts typical values of $Q < 0.01$.

1.2.3 Summary

The study of random wave breaking in shallow water has not been tailored to the needs of the offshore industry. It is recommended that designers continue to rely on the regular wave theory and that the design guidance notes remain unchanged.

1.3 EFFECTS OF WIND AND CURRENT

There is a shortage of literature on these two subjects in the particular area of large wave kinematics and loading. Thus, whilst it is to be expected that a following wind will increase wave energy, and hence kinematics and loading, no publications of direct use to the offshore engineer were found. The loading due to wind-driven waves is, in some sense, already accounted for in the wave statistics. However, there is also the possibility of a local wind-wave interaction, which would modify the wave kinematics. The combined effect of wind loading and wave loading is a separate problem and is beyond the scope of this document.

Waves riding on an even current will be Doppler shifted, when viewed from the frame of reference of a platform. That is to say, that the wave kinematics should be calculated in the frame of reference of the current, then added to the current velocity itself to get the total wave-current kinematics. The wave breaking limits are those which pertain in the frame of reference of the current. For further details on the Doppler technique see Skyner et al (1992). In the same study it was found that for waves on moderately sheared currents the appropriate reference frame was the velocity of the undisturbed surface current. The velocity field in the crest of a wave was then well predicted by adding the stretched current profile to the wave-only kinematics.

The Doppler procedure is not presently recommended in design guidance notes. Application of such a procedure is complicated because the metocean data already contains wave statistics which have been gathered in the presence of currents and are not easy to separate. It may be necessary to carry out further work on understanding the statistical data before implementing a Doppler shifting technique.

Contents

2. WAVE KINEMATICS

The kinematics of water particles in a breaking wave of given form are now fairly well understood. New programming techniques have enabled detailed solutions to be calculated. New optical measurement techniques such as laser anemometry (LDA), and more recently particle image velocimetry (PIV) have enabled kinematics to be measured in the crest to trough region of the wave. This is the area in which the wave theories tend to diverge rapidly.

However, the application of the most advanced methods is not trivial and the form which the breaker may take is not always clear. It is therefore worth reviewing some of the key concepts before examining the detailed application of available theories to the design code.

2.1 REGULAR WAVES

Regular waves do not break (by definition). However, it has been shown that even in deep water their form is unstable. Large wave trains tend to modulate into groups. Mathematically, this implies that the regular wave is not an exact solution of the dynamic equations for flow with a free surface. Nevertheless, the kinematics of the wave field can be deduced given the shape of the free surface and assuming that the wave is slowly varying. The shape of the free surface can be deduced by making simple approximations to the wave solution, linear theory being the first order approximation.

There seems to be general agreement that, for regular waves, the traditional solutions provide a good fit. Swan (1990) measured the kinematics in the crest of regular waves up to s=0.010 and found good agreement with Stokes V. Sutherland (1992) compared measurements of regular waves with s=0.010 to s=0.017 with a variety of theories and found that the fourier method of Reinecker and Fenton (1981) and the Dean's stream function method (Dean, 1965) gave the best fit to the measured crest kinematics. The other comparisons - linear theory, Wheeler & Chakrabarti stretching, delta stretching and extrapolation all performed badly, unless high order wave components were added in to assist with the non-linearity. For the Stokes wave, the stream function theory and the fourier approximation, the crest velocity in a maximum steepness wave always tends to the phase velocity.

For a regular wave, the method for calculating the water particle kinematics will have an order dependant on the steepness of the wave, as implemented in the 4th edition of Offshore Installations: Guidance on design, construction and certification (1990).

2.2 RANDOM WAVES

Random, non-breaking waves may be described by the application of high order theories (Deans stream function may be applied to an asymmetric wave profile), or by stretching techniques. However, Sutherland (1992) found that for two component wave groups, stretching theories could out-perform fourier theory, which could not model the rapid change in horizontal velocity at the wave crest. Similarly, Gutmestad (1993) proposes that Wheeler stretching provides a good engineering approximation to the crest velocities, based on measurements made at Trondheim. Thus it would appear that random waves may be described by some simple superposition theory, provided that the waves do not break.

2.3 BREAKING WAVES

2.3.1 Maximum velocity

Using the regular wave breaking limit, it seems natural to assume that the maximum velocity in a breaking wave will be slightly greater than the wave celerity. This must be allied to the observation that the wave celerity is dependent on the wave steepness, and for shallow water waves, on the water depth. This was the finding of Easson (1984), who measured crest water particle velocity at the spout of a deep water breaking wave using laser anemometry, and the phase velocity by measuring the progress of the zero down-crossing of the wave crest.

Griffiths (1992) tabulated the findings of a number of investigators and found that with one exception, maximum velocities for deep and shallow plunging breakers were around the phase velocity. For the anomalous result (Kjeldsen, 1980), where u=2.8c it was proposed that the anomaly was due to the method of calculation of the phase velocity, which is non-trivial for a summation of component frequencies and could result in an under-estimate of c by a factor of 3.

Interestingly, more recent work by Kjeldsen (1989) indicates maximum velocities in the region of $1.3c_0$-$1.5c_0$ which, given that the phase velocity increases by around 20% for large waves, suggests that the crest kinematics are slightly greater than the local phase velocity as expected.

She (1994) gives velocity measurements in 3-D breaking waves. These are symmetric single frequency breakers produced at the glass wall of a 3-D wave tank. Measurements were made in the plane of symmetry of the wave which had spreading angles of $+/-30^o$ to $+/- 60^o$. The severity of breaking was dependent on the spreading angle, with 30^o spreading only resulting in a spilling breaker. The plungers all had maxima between $u=c_0$ and $u=1.5c_0$; the spiller had a maximum at $u=c_0$. She showed that, for his measurements, the under crest velocities in the plunging breakers did, in fact, tend to the celerity. But the maxima were ahead of the crest in the area of the plunging spout.

Chaplin (1992), measured breaking wave kinematics into the crest at two scales, in the City University flume, with H~0.5m, and in the Delta flume at de Voorst, in the Netherlands, with H~3m. He concluded that the maximum velocity always lay in the range $u=c_0$ to $u=1.3c_0$.

2.3.2 Velocity profiles

Skyner (1992,1996) has provided the first exact comparison of a numerical breaking wave and an experimental wave in a flume. Using the code of Dold & Peregrine(1986) he produced a numerical breaking wave by frequency superposition. Modelling the exact spectrum in the wave flume he was able to show that the breaking position, time of breaking, height, wave profile and internal kinematics were in agreement to within a few percent. It can thus be said to be possible to predict the kinematics of a breaking wave exactly if enough information about the wave is present.

New (1985), using the Dold & Peregrine code, demonstrated that breaking waves had velocities slightly in excess of the celerity at the crest, and that unusually large accelerations (~5g) were present in the crest front region, below the plunging spout. It was also found that the acceleration/velocity relationship was no longer orthogonal.

Chaplin (1992) found that a 17th order stream function solution gave a reasonable fit to his velocity profiles, and that Stokes V was inadequate. Easson (1988), in breaking waves on beaches work, found that the measured kinematics were in agreement with the predictions of Peregrine, and that high order stream function theory also gave an adequate approximation. Again, low order stream function and Stokes V underpredicted the crest velocities. The cnoidal theory, commonly used for shallow water waves was found to be insufficiently accurate for breaking waves in shallow water tests.

2.4 SUMMARY

Over the past decade, the position regarding wave kinematics has become much clearer. It is now widely accepted that high order theories are required for steep waves. Although Peregrine has proposed the use of the non-linear programs for the prediction of breaking waves it is not yet clear that they exist in a suitable form, or that the method by which they would be applied is entirely clear.

3. WAVE FORCES

3.1 OVERVIEW

There have been many advances recently in the study of forces due to large and breaking waves. Most investigators are content to conclude with modifications to Morison's equation, particularly with respect to the force coefficients. However, some have suggested that a radical overhaul of the way we write the force equations is required. As the volume of early work is so great, no attempt will be made here to provide a summary of the conclusions. It is worth stating, however, that at the time of writing Morison's equation is still holding its own, with the force on a rigid pile in the direction i (=x ,y,z) being expressed as

$$F_i = \frac{1}{2} C_d \rho A u_i^2 + 2 C_m \rho V \dot{u}_i \qquad 24$$

where the theoretical values for the coefficients are $C_m = 2$ and $C_d = 1$.

Sarpkaya & Issacson (1979) give a detailed account of the standard application of Morison's equation.

3.2 SLAM FORCES

Breaking waves differ from other types of wave in that, as the wave overturns, some part of the crest becomes vertical, and may therefore impact on a vertical structure in a very severe manner. Miller (1980) and Cambpell (1980) investigated the force on a parallel cylinder entering a flat free surface and deduced that the equation describing the initial impact force could be written in the form,

$$F_s = \frac{1}{2} C_s \rho A u^2 \qquad 25$$

which is similar to the drag term in Morison's equation, except that the theoretical value for the coefficient of slam is $C_s = 2\pi$. Note that u here is the velocity of impact, ie the relative velocity of the cylinder and the water surface perpendicular to the water surface at the point of impact.

This has been studied experimentally, by Campbell and Weynberg (1980) , who found that for parallel entry large coefficients could be achieved, and that the full value of C_s was achieved when 10% of the cylinder diameter became immersed. Easson (1984, 1985) performed tests on a horizontal cylinder in the crest/trough zone, and discovered that the coefficient was reduced dramatically by the effects of surface roughness and incident wave angle.

Chan(1991a) measured forces on a vertical wall and found that, due to the effects of air entrappment between the plunging crest and the wall, it was possible for values of $C_s = 10$ to be achieved. This is a major problem for the design of breakwaters, where long-crested waves can indeed impinge on the structure in such a way that the air becomes trapped.

However, in the offshore industry, horizontal members are not designed in the splash zone. Hence concern arises only for that small part of a vertical member which is impacted by the vertical front of a breaking wave. In experiments designed to maximise the breaking wave impact and assess the spread in forces due to slight phase changes in the impact of a breaking wave on a vertical Cylinder Chan (1991b) found that the highest pressures were encountered on the cylinder just under the wave height. However, very small changes in the phase of the wave at impact (of order 10%) led to reductions in the impact pressures of a factor of 8. He concluded that a design model which involved breaking waves would have to include the pressure probability distribution for the local phase of the wave and the variability of breaking wave kinematics.

At a larger scale Chaplin(1992) carried out experiments on a D=0.5m cylinder in the Delta Flume at de Voorst. He did not directly observe the impact velocity of the front face of the wave on the cylinder. However, he did compare the maximum horizontal velocity at each height with the maximium impulsive force at that height, deriving a coefficient of breaking, C_b,

$$F_b = \tfrac{1}{2} C_b \rho A u^2 \qquad (26)$$

His results range from C_b = 1, for large, non-breaking waves, to C_b = 2.5 for plunging breakers. Even spilling breakers generated values up to C_b = 2. Again, a significant variation was found in the peak impact pressures, with mean values, measured over 20 waves, being $1.5<C_b<2$. In repeats of these tests in the City flume (D=0.16m), Chaplin confirmed these results, and was able to produce even more severe breakers, which generated even larger values of C_b = 4.

Kjeldsen and Akre (1985) measure forces in large and breaking waves in a flume and a 3-D facility. Although they do not directly consider slam forces, it is clear that the breaking wave forces are typically 4-5 times those of the large non-breakers (figure 6). It seems likely that some combination of increased crest velocity and slam coefficient would be required to produce such forces. They conclude that more work is required on wave slamming, and that breaking waves should also be investigated in shallow water where they tend to be more severe.

3.3 FORCE COEFFICIENTS

3.3.1 Breaking waves

The 4th edition of Offshore Installations: Guidance on design, construction and certification (1990) provides details of valid force coefficients for use in offshore situations. Research is continuing on the values of these, particularly with reference to the high Reynolds number range, and breaking waves. Analysing the force records for breaking waves after initial impact Chaplin (1992) finds that the values for C_d are relatively lower in the crest, where the water particle velocities are significantly higher. This is ascribed to the changing Reynolds number in the wave with Re ~ 2 x 10^6 at the crests of the largest breakers. He proposes the values given in table 1, whilst noting that the standard Morison's equation does not provide a good fit.

	mwl	crest
C_d	0.5	0.3
C_m	1.5	0.2

Table 1
Force coefficients at mean water level and at the crest of a breaking wave

Contents

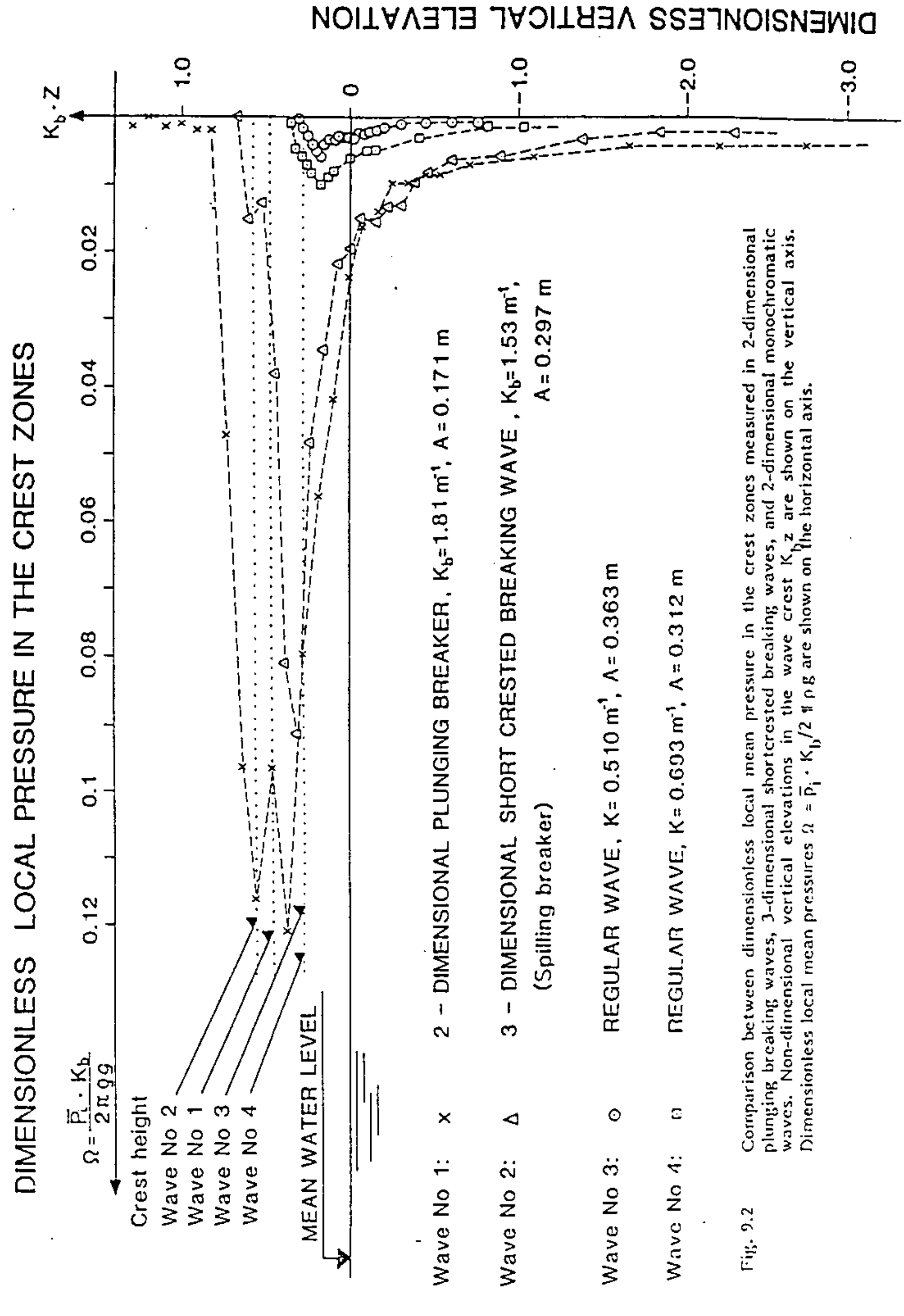

Figure 6
Pressure measurements of Kjeldsen and Akre (1985) showing the difference between breaking and non-breaking waves of similar heights

Kjeldsen(1985) analysed his results in a similar manner, but ignored the inertia term, and used a measured value for the horizontal velocity. He derived a value of $C_d' = 0.25$, the prime being used here to denote that Morison's equation is being used without the inertia

term. The Reynolds number was at the critical level of 2×10^5. Chaplin performed a fit of his data to a drag only equation and found $C_d' = 0.3$.
These values are significantly lower than the recommended design coefficients and may be representative of the full-scale expectations. However, they are both obtained using actual velocity measurements, which are significantly higher than normal theory would predict. Any attempt to use such low coefficients in design would have to be matched by an acceptable wave theory.

3.3.2 Time-dependent coefficients

Torum (1985), in force measurements in the surf zone of regular waves, observes that Morison's equation does not provide a good fit to the forces for fixed values of C_m, C_d. He therefore proposes that time-dependent values be used. This is a reasonable proposition, since the origin of the drag term in Morison's equation is the constant flow past a cylinder. However, the study uses sub-critical velocities, and 2nd order Stokes theory is fitted to the wave crests, which are around 40% of maximum steepness. Furthermore, to account for the asymmetry of the wave, a forward and backward wave period have been defined, giving different results for the crest and trough phases. This leads to some unusual features, and any conclusions based on this work must be treated with extreme caution.

3.4 RUN-UP

Kjeldsen (1985), Torum (1985), and Chan (1991) all report maxima in the vertical force profile beneath the maximum crest elevation. Torum discusses this in the context of the run-up studies of Hallermeier (1976) and Dean (1981).

Hallermeier observed waves passing a thin vertical cylinder and noted that in the low pressure wake, the free surface was depressed, and at the front of the cylinder, the free surface ran up the cylinder. From a series of measurements he deduced that the extent of the run-up was equal to the kinematic pressure of the water at the crest,

$$\Delta H = u^2/2g \qquad 27$$

Dean proposed that the run-up would result in a force above the maximum elevation of the wave crest, but that the drag effect to an equivalent distance below the undisturbed crest level would be reduced. This was based on measured force histories from offshore data, where it was observed that the maximum force was below the crest, and this dropped of gradually to zero at some point above the crest.

Torum carried out a detailed analysis of this effect and found his maximum force at a distance below the surface of approximately 1-2ΔH. He proposed that the force profile, calculated from the drag term should be plotted to reach a maximum at H-ΔH, dropping off linearly to zero at H+ΔH (figure 7).

These analyses are limited to small regular waves (in the case of Torum and Hallermeier) and unspecified waves in Dean's case. Dean also used his stream function solution, suggesting that the waves were non-breaking. Whilst this is clearly a very real effect, which is also evident (but not discussed) in the work of other researchers, it should not be incorporated into design code until it has been thoroughly tested for very large and breaking waves.

Contents

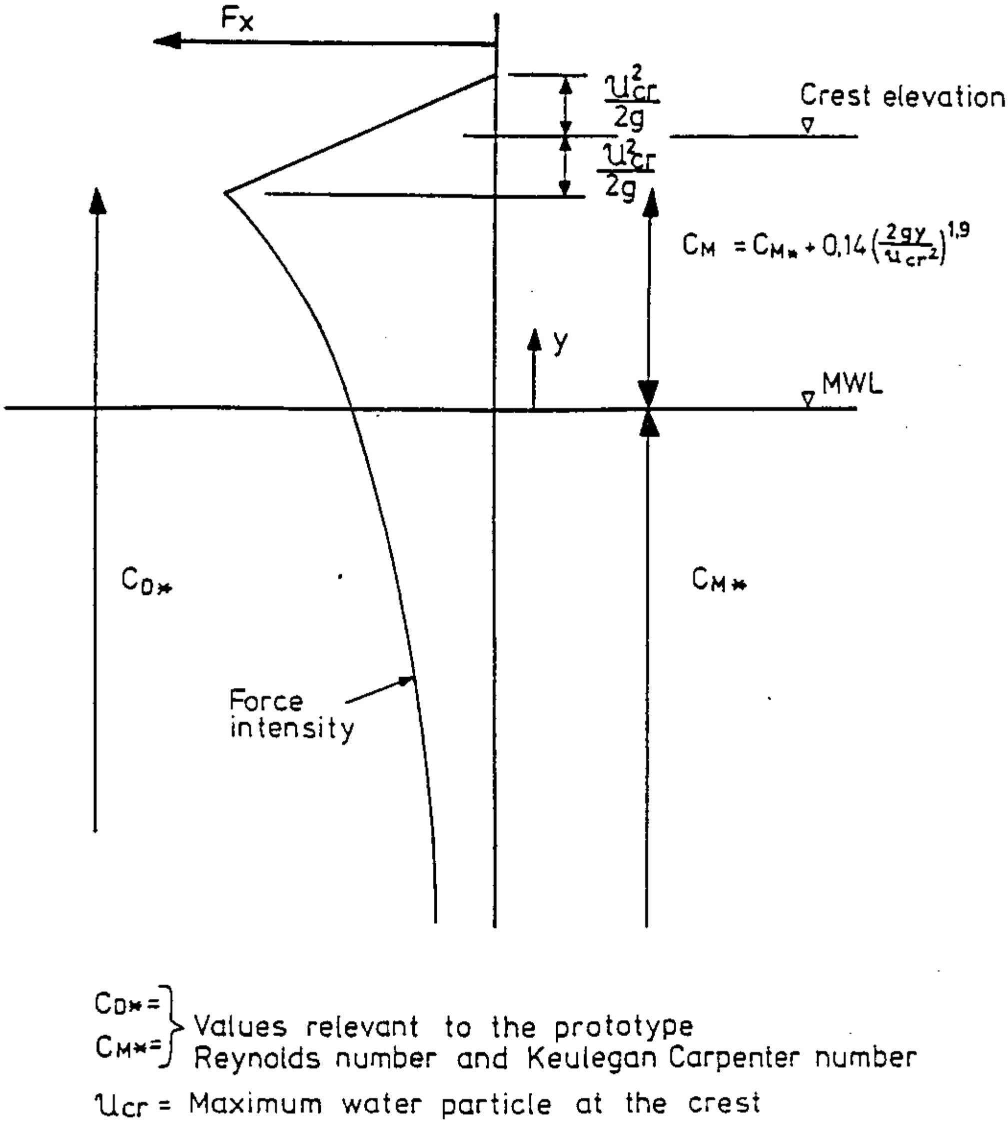

Figure 7
Force profile taking account of free surface effects (Torum 1985)

3.5 PHASE CONSIDERATIONS

It has already been noted (section 2.3.2) that the orthogonal velocity/acceleration relationship from simple wave theory does not hold for breaking waves. Implementation of a full non-linear model would ensure that the correct phase relationship was taken into account when evaluating the wave kinematics. However, Morison's equation was derived and tested on the assumption that these are orthogonal, and may not be expected to hold when part of the acceleration is in-line with the velocity. At the free surface, the situation is further complicated by the suppressed wake effect measured by Hallermeier. When there is no fluid behind the cylinder the traditional drag force does not exist, but is replaced by a pressure difference, which Hallermeier equates to an inertia force on the cylinder, in phase with the velocity.

It has already been seen that, for slender structures, the force in a breaking wave may be adequately described by use of a single coefficient. This may be due to the coincidence of a large part of the inertia force, in phase with the drag force, combined with the fact that for small cylinders the drag force is dominant anyway.

Rainey (1991), argues that the inclusion of the convective acceleration provides a better fit for Morison's equation, and that for a full formulation, the exact potential flow loading should be used in place of Morison's inertia force. However, this must be set against the original paper (Rainey, 1989) in which he states that his second order results are limited to one of,

$$10D>L;\quad 5D>H;\quad 30D>\lambda \qquad 28$$

where L is the cylinder length. For a thin lattice structure in an extreme wave, none of these is likely to apply. The example given in Rainey (1991) of the force on a 1m cylinder shows that Morison's equation is conservative when compared to the potential flow method for a vertical cylinder. The method has not been compared to experimental results.

3.6 3-DIMENSIONAL EFFECTS

Kjeldsen (1985) concludes from his measurements that the forces in short-crested waves are smaller than those in 2D waves. This is in agreement with the results of Chaplin (1993) who confirmed, experimentally, the predictions of Aage (1990) that a reduction of 15% could be expected in wave spreads typical of the North Sea. Chaplin measured the force distribution under the trough level for JONSWAP spectra with spreads of s=2,8,∞. He found a reduction of about 15% between each case.

This backs up the theory of Kolain (see Section 1.1.3 above), that steeper waves can be produced in 3D due to the reduced forward component of velocity, for this also implies that, in waves with the same significant wave height, the forward component of velocity will be smaller in the short-crested waves, leading to lower drag forces.

4. DESIGN IMPLICATIONS

4.1 DESIGN WAVE

The 4th edition of Offshore Installations: Guidance on design, construction and certification (1990), section 11, proposes that the extreme wave be modelled from the wave height statistics. The period of the wave is given as a function of the height, so that the wave shape, in deep water, is fully specified by the wave height.

4.1.1 Deep water

The extreme wave height is derived directly from the significant wave height

$$H_{50} = 1.86H_{S50} \qquad 29$$

and the extreme wave elevation (the maximum extent of the wave above mwl) is

$$C_{50} = 1.03H_{S50} \qquad 30$$

The ratio of these is

$$\mu_{50} = 0.55 \qquad 31$$

significantly less than that which has been found for breaking and near breaking waves.

The zero-crossing and wave period associated with the 50 year design wave are derived as follows:

1. $$3.2(H_{S50})^{1/2} < T_z < 3.6(H_{S50})^{1/2} \qquad 32$$

2. $$1.05T_z < T_{ass} < 1.4T_z \qquad 33$$

Substituting the range of associated wave periods (Equation 33) and the design wave height (Equation 29) into Equation 32 gives

$$0.007 < \frac{H_{50}}{gT_{ass}} < 0.017 \qquad 34$$

The design guidance thereby gives a maximum steepness for the design wave which is well below the breaking limit for a regular wave (Equation 1). However, the upper limit is within the range of breaking wave steepness found in Section 1.1.2 for waves which belong to groups of around 3, which had a frequent occurrence according to Holthuijsen.

A number of studies have shown that breaking waves have been found not to be limited to maximum steepness in 3D sea states. This would suggest that a more rational approach may be to include breaking waves, but within the present extreme wave height. Furthermore the definition of maximum crest elevation would be extended, in accordance with Section 1.1, so that

$$C_{50} = 0.65H_{50} \qquad 35$$

This will have implications for the design airgap of the structure, as well as increasing the local kinematics of the wave field.

4.1.2 Shallow water

The shallow water design wave is presently limited by the shallow water criterion derived from extreme shallow water wave theory. The limit only applies to long solitary waves, and is not adequate as a breaking criterion for intermediate waves. Limits such as those of Yòo and Weggel are specifically aimed at shallow water breaking and do not extend into intermediate water.

In particular, at the upper end of the specified shallow water limit, h/L<0.25, investigators have consistently found that H/h ~ 0.6. This would imply that there is a far wider range of heights over which design waves in shallow water can be breaking than the guidance presently allows. The value of H/h =0.78 should only be used for very shallow wave conditions, where it must be modified by local slope effects. For intermediate water it is recommended that a joint criterion of the form given in figure 8 be adopted.

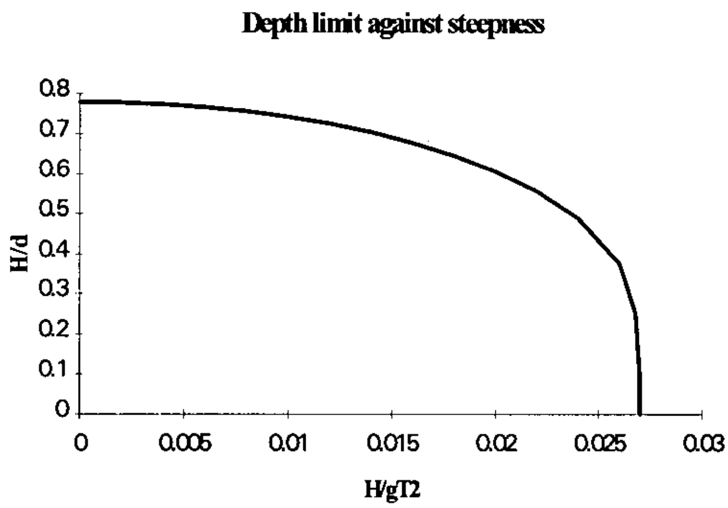

Figure 8
tanh fit to deep and shallow water breaking limits

One effect of shallow water on breakers is to increase the horizontal asymmetry, which must be allowed to vary between

$$0.65<\mu<0.8 \quad (36)$$

from deep to very shallow water. Again, this has significant implications for the airgap.

4.2 WAVE KINEMATICS

Large waves should be modelled, where possible, using a fully non-linear solution of the wave equations. Alternatively, and for the case of deep water breakers, where no extraneous influences are present (eg, crossing seas, current changes), the wave should be assumed to be spilling and the highest order solution, tending to u=c at the crest should be employed.

Contents

4.3 WAVE FORCES

Slam and slap forces must be taken into account on individual members where appropriate.

If breaking waves are being used, calculations must take appropriate account of the large inertia forces in the forward area of the wave, under the crest.

It is recommended that, if the above measures are employed, the drag coefficient be reduced in accordance with high Reynolds number measurements.

It must be assumed that there will be a run-up on the structure, of at least $u^2/2g$. This should be taken into account in the design for airgap and force calculations.

Where the angular spread of the 50-year storm is known, the design wave may have the forward component of its velocity field reduced by an appropriate percentage.

4.4 IMPLEMENTATION

The above recommendations (Sections 4.2 and 4.3) are wide-ranging, and inter-dependent. It is clear that they may only be implemented as a package. This kind of change in the way that the design wave is approached has been proposed before (Kjeldsen, 1985). Others have suggested the implementation of more representative non-breaking design waves (Swan, 1990).

The combined effect of the changes would be to increase wave heights and wave kinematics, but to decrease the loading from the kinematics due to reduced coefficients and the allowance for 3-dimensionality.

5. FURTHER WORK

5.1 ENERGY FLUX

A breaking wave is simply the local manifestation of a much wider wave field, as shown by Sutherland (1990) and by Holthuijsen (1986) in their group work. The wave crest is travelling at around twice the group velocity, the rate at which energy is being transported. A wave which breaks near the maximum of a group is breaking due to the energy concentration, as is a wave which breaks due to the focussing of angular components or frequency components which have different celerities. It is therefore proposed that a new approach be taken to the study of breaking, based on the rate at which energy is being concentrated in a crest.

One example of such an approach is that of Kjeldsen (1990), who looks at the energy flux at a point. The theory requires three input parameters, η, and its spatial derivatives in both horizontal axes. He gives a convincing demonstration of this method by using it on a raw time series containing a two-wave group, to predict (correctly) that the forward, smaller wave, breaks because its predicted crest velocity exceeds the phase speed of the wave (figure 9).

5.2 BREAKING PROBABILITY

A study of the kinematics of 2-D random seas and a study of the probability of breaking on a flat bed and shallow slopes for 3-D random seas should be undertaken in appropriate facilities. The purpose of the 3-D experiments would be to:

1. Determine slope and offshore steepness dependence for a wide range of parameters.

2. Relate the offshore steepness to the local steepness.

3. Measure the proportion of breakers as a function of the spectral shape.

4. Find the proportion of large waves which are breakers.

5. To test and modify accordingly the methods proposed by Ewing and Southgate.

An experimental study of the kinematics of breaking of wave groups on sloping beds is underway at the University of Edinburgh.

5.3 WAVE KINEMATICS

It is clearly desirable to introduce predictive methods which will give the likely form of the design wave (see chapter 1). It is similarly desirable that the code be made available which can correctly model these highly non-linear waves. It is therefore proposed that studies be set up in the following areas:

1. Further measurements of wave kinematics in irregular seas and 3-D seas.

2. Extension of codes to model bed topography and 3-dimensionality.

3. Generalisation of codes to allow their use with reduced input information.

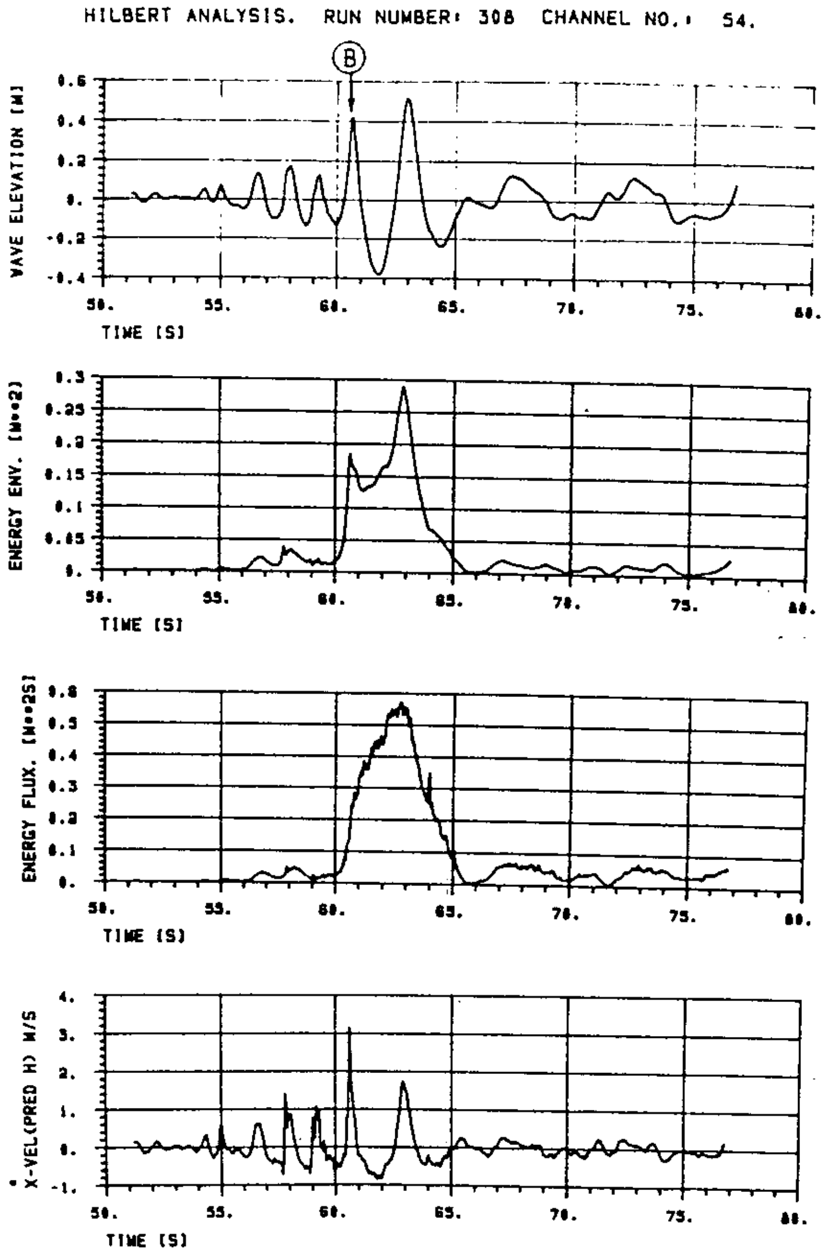

Figure 9
Kjeldsen's 'energy method'. Although the non-breaking wave is larger (top), the breaking wave has higher kinematics (bottom)

5.4 WAVE FORCES

The available literature indicates that the wave forcing mechanisms in breaking waves cannot be properly described by Morisons equation. It is proposed that studies proceed on two fronts:

1. Large scale wave by wave analysis of the velocity field and forces on a vertical cylinder. The data procured by Chaplin is ideal for this.

2. An investigation of new ways in which the force may be calculated from the wave kinematics.

Contents

6. CONCLUSIONS

1. The simple, geometric formulations, such as steepness and crest asymmetry, are of limited value for predicting breaking when applied to very wide-band seas and fully 3-dimensional seas.

2. As breaking waves have been found not to be limited to maximum steepness in 3D sea states there is evidence to suggest that the design conditions should be extended to allow breaking waves, but within the present extreme wave height.

3. The study of random waves breaking in shallow water has not been tailored to the needs of the offshore industry. As there is insufficient evidence to suggest changes, it is recommended that designers continue to rely on the regular wave theory.

4. The breaking criterion $H/h > 0.78$ should only be used for very shallow wave conditions, where it must be modified by local slope effects. For intermediate water it is recommended that a joint criterion of the form given in figure 9 be adopted, giving values as low as $H/h > 0.6$.

5. Over the past decade, the position regarding wave kinematics has become much clearer. It is now widely accepted that high order theories are required for steep waves, giving values of $u=c$ at the crest of the steepest waves.

6. There is evidence to suggest that the definition of maximum crest elevation should be extended such that

   $$C_{50} = 0.65H_{50} \qquad 35$$

   This will increase the design airgap, as well as increasing the local kinematics of the wave field.

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