

INFLUENCE OF STEEP SEABED SLOPES ON BREAKING WAVES FOR STRUCTURE DESIGN

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ABSTRACT

Prediction of limiting wave heights in conditions of depth-induced breaking is subject to considerable uncertainties, yet the (local) wave height is probably the most important input variable in design of coastal, harbour or shoreline structures subject to wave action.

This paper presents selected results from laboratory experiments to measure depth-limited wave breaking over steep bed slopes (1:50, 30, 20, and 1:10) in fully random wave conditions. Experimental measurements are compared with predictions for H_{rms} , H_s and H_{max} under shoaling and breaking. The effects of shoaling and breaking on the wave height distributions are explored. An alternative empirical method to predict $H_{1/3}$ and H_{max} is suggested.

1. INTRODUCTION

1.1 Is there a problem?

Many coastal structures and some harbour breakwaters are constructed in relatively shallow water depths where the larger wave heights that constitute the primary input parameters in structure design are significantly influenced by depth-limited breaking. Prediction methods to calculate hydraulic or stability responses of these structures generally use the incident significant wave height (H_s) as primary input variable, often defined in the water depth at the seaward toe of the structure, h_s . Where wave breaking has significant influence on design wave heights, this approach therefore requires that prediction methods for depth-limiting must be robust and reliable.

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Design methods for wave overtopping, armour movement and related responses require values of the incident significant wave height, $H_{1/3}$. In contrast, calculations of wave forces using Goda's method often use an upper limit estimate of H_{\max} such as $H_{1/250}$. Neither of these values are reliably derived from wave breaking prediction methods which use spectral measures.

A few design methods use offshore wave heights in deeper water, say H_{s0} , and use empirical methods to predict the response directly, calibrated for a range of simple bed slopes. The best-known examples are methods developed by Goda (1975, 1985) to predict overtopping or forces on vertical walls which use a single equivalent sea bed slope. Such methods assume that each approach bathymetry may be represented by a simple bed slope, and that the empirical prediction methods fully represent the effects of different wave transformations on the response of interest.

Most experimental studies on wave breaking have been on bed slopes shallower than 1:30, typically 1:50 or 1:100. On these slopes, wave shoaling is relatively mild, and wave breaking reasonably well understood, but there is growing evidence that steep bed slopes transform waves differently and give more severe hydraulic and structure responses. Jones & Allsop (1994), Southgate & Stripling (1996), Hamm & Peronnard (1997), Nelson (1997) and McConnell & Allsop (1998) demonstrate not only that many methods to predict wave conditions under breaking suffer from significant limitations, but that some responses seem to be particularly influenced by local sea bed slopes, in some instances to an extent not covered by using local wave heights in the calculations. These seem to be especially noticeable for slopes steeper than 1:50.

1.2 Research studies

New or improved methods to predict wave behaviour and breaker heights are needed to improve safety of structures constructed in the surf zone. In the first instance, a series of hydraulic model studies were completed by HR Wallingford for the Flood & Coast Defence division of the Ministry of Agriculture, Fisheries and Food (MAFF) to provide more information on wave breaking behaviour. Co-operation with Alkyon Consultancy & Research and TUDelft in the Netherlands to share laboratory and field data on wave breaking is intended to ensure as large and reliable a dataset as possible. It should however be noted that the results presented in this particular paper represent only part of the data, and relatively simple levels of analysis.

Other studies on wave breaking at vertical or composite breakwaters have been conducted as part of the EU PROVERBS project, see particularly Calabrese & Allsop (1998). Those studies have concentrated on whether wave conditions, depth and geometry will cause wave impacts on the wall, and are not intended to predict breaking / broken wave heights, so will not be addressed further here.

2. PREDICTION METHODS FOR WAVE BREAKING

This paper does not attempt to give an overall review of prediction methods for wave breaking, but draws on the review by Southgate (1995). Additional papers or reports are cited where they amplify or up-date that review. It may be noted that many of the methods cited by Southgate give information only at a single point, but in design of realistic structures, it is important that predictions be valid over a wide range of

relative depths / breaking regions. Predictions of wave breaking are tested here against measurements from the onset of breaking onwards.

The primary cause for wave breaking in deep water is that the wave steepness exceeds the fundamental limit given for individual waves by:

$$(H / L)_{\max} = 0.142 \quad (1)$$

In shallow water, the main processes of interest in wave breaking may be divided into two. The first processes are those of wave transformations up to, but not beyond, the point of breaking. These include refraction and diffraction (neither discussed further here), and shoaling (here of considerable interest). These processes involve no significant loss of energy and are essentially reversible. The second set of processes are those which occur from breaking onwards. These processes involve significant loss of energy and are not reversible. It is noted that in some analysis, effects of shoaling and then breaking have been somewhat confused. Confusions may also have arisen from differences between regular waves (where all waves behave the same) and random waves (in which breaking positions and other features vary with period and height of each wave). These differences are particularly evident where methods to predict wave breaking have been developed using regular waves only, but are then applied to "real sea" cases where waves are random.

A final source of confusion is the use of significant wave height H_s in design methods, and numerical models to predict wave conditions, without more careful definition of its derivation, be it spectral ($H_s = H_{m0}$) or statistical ($H_s = H_{1/3}$). Differences between $H_{1/3}$ and H_{m0} were first highlighted by Thompson & Vincent (1984) and described more recently by Hamm & Peronnard (1997). The main problem arises where one method has been used to define wave conditions in model testing used to derive empirical design methods, and then a different method is used to derive design wave conditions. Comparisons in this study have shown that differences between $H_{1/3}$ and H_{m0} are greatest for low wave steepnesses when non-linear shoaling is pronounced.

2.1 Flat bed slopes

For very shallow bed slopes, usually taken as flatter than 1:100, it is often assumed that a simple limit to the individual wave height relative to local water depth may be given by:

$$H_{\max} / h = 0.78 \quad (2a)$$

Later researchers showed that this limit, suggested by McCowan based on solitary wave theory (see Southgate, 1995), might be increased to $H_{\max} / h = 0.83$. Perversely, Le Mehaute appears to give a much lower limit of individual wave height relative to local water depth:

$$H_{\max} / h = 0.55 \quad (2b)$$

2.2 Sloping seabeds

The methods most frequently used in practical design calculations for structures are those by Weggel (1972), Goda (1975) and Owen (1980). Weggel used regular wave test data to derive simple empirical expressions to predict maximum wave heights in depth h_s :

$$H_{\max} / h_s = b / (1 + a h_s / (gT^2)) \quad (3a)$$

where coefficients a and b are defined in terms of the seabed slope m:

$$a = 43.75 (1 - \exp(-19m)) \tag{3b}$$

$$b = 1.56 / (1 + \exp(-19.5m)) \tag{3c}$$

Goda (1975) developed a prediction method for irregular wave breaking, even suggesting a method to transform the wave height distribution under breaking conditions. For shoaling, Goda used Shuto's method instead of simple linear wave methods to give the shoaling coefficient K_s .

For wave breaking where $h/L_{po} \geq 0.2$:

$$H_{1/250} = 1.8 K_s H_{s0} \tag{4a}$$

For $h/L_{po} < 0.2$:

$$H_{1/250} = \min \{ (\beta_0^* H_{s0} + \beta_1^* h), \beta_{max}^* H_{s0}, 1.8 K_s H_{s0} \} \tag{4b}$$

where:

$$\beta_0^* = 0.052 (H_{s0}/L_{po})^{-0.38} \exp(20m^{1.5}) \tag{4c}$$

$$\beta_1^* = 0.63 \exp(3.8m) \tag{4d}$$

$$\beta_{max}^* = \max \{ 1.65, 0.53(H_{s0}/L_{po})^{-0.29} \exp(2.4m) \} \tag{4e}$$

Where $H_{1/3}$ is needed in design, Goda suggested a similar method for $H_{1/3}$.

For $h/L_{po} \geq 0.2$:

$$H_{1/3} = K_s H_{s0} \tag{4f}$$

For $h/L_{po} < 0.2$:

$$H_{1/3} = \min \{ (\beta_0 H_{s0} + \beta_1 h), \beta_{max} H_{s0}, K_s H_{s0} \} \tag{4g}$$

where:

$$\beta_0 = 0.028 (H_{s0}/L_{po})^{-0.38} \exp(20m^{1.5}) \tag{4h}$$

$$\beta_1 = 0.52 \exp(4.2m) \tag{4i}$$

$$\beta_{max} = \max \{ 0.92, 0.32(H_{s0}/L_{po})^{-0.29} \exp(2.4m) \} \tag{4j}$$

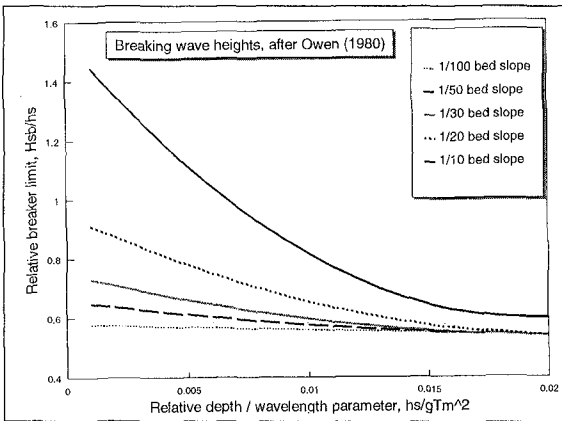


Figure 1 Simple breaking curves, after Owen

Noting that for steep bed slopes, waves may shoal substantially before breaking starts, Owen (1980) developed a simple method to provide first-estimates of the upper limit to the (significant) wave height H_{sb} in any water depth h_s for each of five bed slope. The method was derived as a part-way point in predicting wave overtopping of seawalls, and was not itself validated against any data on breaking wave heights. Owen's simple curves were derived graphically, see Figure 1, but were later described by empirical equations relating breaker index H_{sb}/h_s to relative depth h_s/gT_m^2 :

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Slope	Breaking limit, H_{sb}/h_s	
1/100	$H_{sb}/h_s = 0.58 - 2 (h_s/gT_m^2)$	(5a)
1/50	$H_{sb}/h_s = 0.66 - 10.583 (h_s/gT_m^2) + 229.17 (h_s/gT_m^2)^2$	(5b)
1/30	$H_{sb}/h_s = 0.75 - 20.083 (h_s/gT_m^2) + 479.17 (h_s/gT_m^2)^2$	(5c)
1/20	$H_{sb}/h_s = 0.95 - 38.417 (h_s/gT_m^2) + 895.83 (h_s/gT_m^2)^2$	(5d)
1/10	$H_{sb}/h_s = 1.54 - 97.83 (h_s/gT_m^2) + 2541.67 (h_s/gT_m^2)^2$	(5e)

In analysing laboratory and field data for slopes up to 1:20, Battjes & Stive (1984) did not detect any systematic dependence of wave conditions on slope, but did find an influence of wave steepness. They developed an expression for a breaking coefficient, taken by Southgate (1995) to give the limiting r.m.s. wave height:

$$H_{rms} / h = 0.5 + 0.4 \tanh (33 H_{rms,o} / L_{po}) \quad (6)$$

The literature gives relatively little advice on changes to wave height distributions with breaking, but Simm (1991) cites equations for extreme wave heights $H_{0.1\%}$ and $H_{1\%}$ probably originating from Klopman & Stive (1989):

$$H_{1\%} = 1.517 H_s / (1 + (H_s/h_s))^{1/3} \quad (7a)$$

$$H_{0.1\%} = 1.859 H_s / (1 + (H_s/h_s))^{1/2} \quad (7b)$$

2.3 Numerical models

During this study, many of the wave measurements were also compared with two numerical models: WENDIS and COSMOS2D. **WENDIS** is a Wave ENergy DISSIPation model, designed to estimate near-shore wave conditions at coastal structures where *shoaling* (linear shoaling theory), *bed friction* (Hunt and Bretschneider & Reid), and *wave breaking* (Weggel) may be significant. **COSMOS2D** is a numerical model of near-shore hydrodynamics, sediment transport and morphological changes which includes wave transformation by *refraction*, *shoaling* (linear wave theory), *bed friction* and *wave breaking* (Battjes & Stive and Weggel). Some of these comparisons were discussed by Durand & Allsop (1997).

3. EXPERIMENTAL STUDY

Wave conditions were measured using statistical (and later spectral) methods for 2 or 3 water levels over five different bed slopes:

1:50, indicative of shallow sand beach slopes;

1:30 and 20, indicative of steeper sand beaches;

1:10 and 1:7, indicative of rock coasts and shingle beaches.

The results discussed in this paper were mostly derived from supplementary tests conducted by the visiting researcher on a 1:30 slope, with some comparisons with data from the main series of tests on bed slopes of 1:50, 1:20 and 1:10. Additional data for a 1:10 slope by Allsop (1990) and on a 1:20 slope by Southgate & Stripling (1996) has not yet been included in these analyses.

3.1 Outline of experiments

The main tests used uni-modal or bi-modal seas, 30 wave conditions divided into six sequences of five tests, described by Hawkes et al (1998) and Coates et al (1998):

- Wind-sea only

- b) More wind-sea (80%) than swell (20%)
- c) Equal wind-sea and swell
- d. More swell (80%) than wind-sea (20%)
- e. Swell only

Swell and wind sea conditions used standard JONSWAP spectra ($\gamma=3.3$) defined by H_s and T_p . Bi-modal conditions combined two JONSWAP spectra, each defined by H_s and T_p . Supplementary tests of wave breaking included a few regular wave conditions; some modified spectra of different spectral peakedness ($\gamma=1$ to 7), and a few rectangular spectra. Other measurements in tests reported by Hawkes et al (1998) and McConnell & Allsop (1998) included wave overtopping discharges, rock armour stability, and wave pressures / forces on vertical walls.

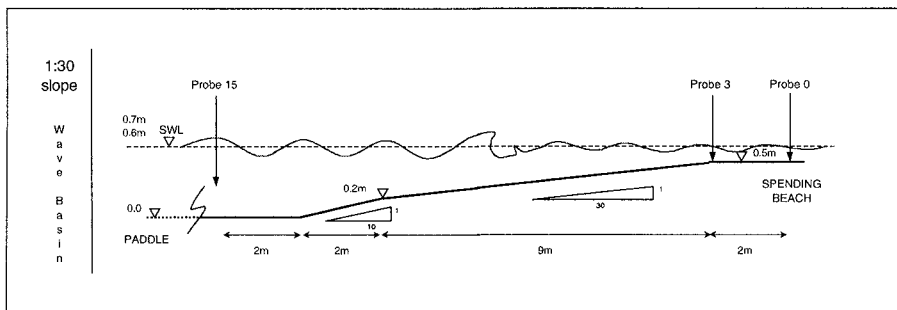


Figure 2 Configuration for supplementary tests on 1:30 bed slope

Most of the tests on wave breaking used the Absorbing Flume at Wallingford with a working length of 36m, and equipped with a random wave generator with computer-controlled absorption system. In each instance, the sea bed slope was terminated in a horizontal section below water, behind which was a gravel beach to absorb remaining wave energy. The supplementary tests on 1:30 slope used a 6m wide flume within a wave basin with two mobile piston paddles, and the 1:30 slope itself was slightly unusual as it featured a short steep (1:10) approach ramp at the toe, Figure 2.

Up to 16 wave probes were placed along the test sections, usually 1 or 2 probes in deep water near the wave paddle, 3 or 4 being placed along the horizontal section at the top of the slope, and the remaining probes up the slope. Each test was run for 500 waves or longer, sampled at 20 Hz (model) giving an average of 20-60 points per wave. In general, wave measurements were analysed statistically using a zero down-crossing definition for each wave. Selected data files were later also analysed spectrally, and some of the results of this analysis are discussed by Hurdle et al (1998) in an accompanying paper.

4. RESULTS

4.1 Parameters derived

Most wave measurements discussed here are presented as significant wave heights, $H_{1/3}$. Both spectral and statistical methods have been used to derive wave heights during these studies, but most weight in this paper will be given to statistical measures of wave height, particularly $H_s = H_{1/3}$. This should avoid problems of confusion

between H_{m0} and $H_{1/3}$ highlighted originally by Thompson & Vincent (1984), and most recently by Hamm & Peronnard (1997).

Two other measures of wave heights may be used. The maximum wave height H_{max} will depend on sample size, and is itself relatively unstable. In this study, H_{max} is generally given by $H_{99.8\%}$ or $H_{99.9\%}$. A more stable measure of wave height favoured by morpho-dynamic researchers is root mean square wave height, H_{rms} , defined spectrally as $H_{rms} = H_{m0}/\sqrt{2}$.

The main measures of wave period used here are the spectral peak period, T_p or mean period T_m . The peak period is more stable than the mean period measured either spectrally or statistically, and is less susceptible to distortion by measurement or calculation errors. Sea states have been categorised by the (fictional) steepness, $s_{po} = H_{so}/L_{po}$, where $L_{po} = gT_p^2/2\pi$, or the equivalent using the mean period, $s_{mo} = H_{so}/L_{mo}$.

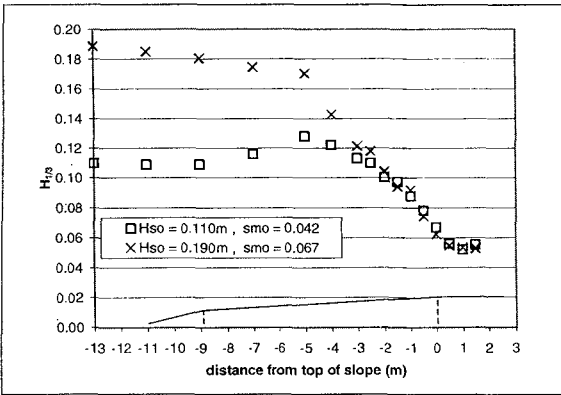


Figure 3 Effect of wave height on breaking for constant wave period, 1:30 slope

4.2 General shoaling and breaking

The complex nature of breaking under random waves precludes any presentation of all features in a single graph. The initial form of presentation by Durand & Allsop (1997) is used here for the first few examples with measurements of local significant wave height against distance along the test flume, see Figure 3.

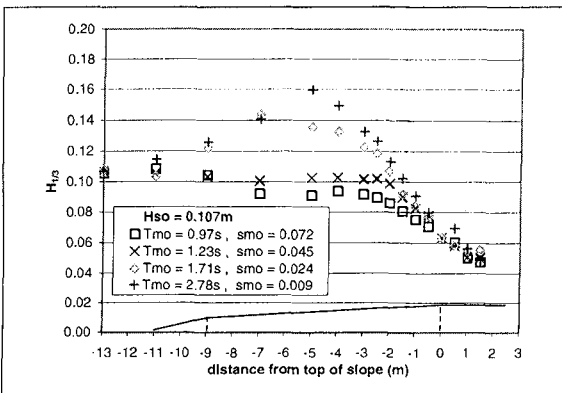


Figure 4 Effect of wave period on breaking for constant offshore wave height, 1:30 slope

This presentation allows the effect of shoaling to be identified as waves react to the rising seabed. The onset of breaking occurs at the peak of the wave height, although in some tests this onset was quite difficult to assess. Breaking continues as the waves move into shallower water or, for some tests, over the

horizontal bed at the top of the slope, even if at a slower rate. For most of these tests, rates of breaking appear to be relatively constant for a particular bed slope, but breaking starts at different points along the slope and at different depths, depending upon offshore conditions.

The lowest steepness waves in Figure 4, $s_{mo} = 0.009$, shoal more than low steepness waves, $s_{mo} = 0.024$, or than the moderate steepness waves, $s_{mo} = 0.045$. The conditions shown in Figure 4 show little increase in wave height due to shoaling for wave steepnesses of $s_{mo} \geq 0.045$, but breaking starts quite far up the approach slope. Lower wave steepnesses show more shoaling, reach a greater wave height and breaking starts earlier.

Durand & Allsop (1997) compared transformations of the same (offshore) wave condition for bed slopes of 1:10, 1:20, and 1:30. Wave breaking on the 1:10 and 1:20 slopes was delayed compared with the 1:30 slope, as might be expected, but the breaking appeared to occur in similar water depths. Over the steeper slopes, the process of breaking and energy dissipation was compressed into rather shorter distances.

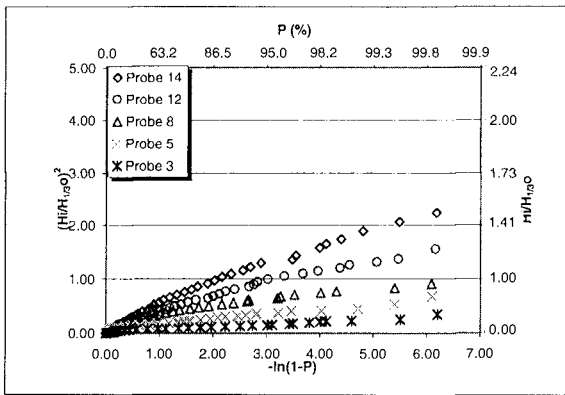


Figure 5 Individual wave heights, $s_{p0} = 0.054$, 1:30 slope

4.3 Wave height distributions

Distributions of individual wave heights are not necessarily of great interest to sediment and wave transformation modellers, but for designers of coastal / harbour or shoreline structures, changes to the distribution of wave heights, particularly to the probabilities of larger individual heights, are of considerable interest.

In deep water, individual wave heights, H_i , generally conform to a Rayleigh distribution. Such distributions plot as straight lines in the format used in Figures 5-8, where individual wave heights are presented as $(H_i/H_{s0})^2$, and non-exceedance probabilities as $-\ln(1-P)$. On these scales, $-\ln(1-P) = 2.0$ corresponds to $H_{1/3}$ for a true Rayleigh distribution, $H_{98\%}$ is given by $-\ln(1-P) = 3.91$, $H_{99\%}$ is given by $-\ln(1-P) = 4.61$, and $H_{99.6\%}$ by $-\ln(1-P) = 5.52$.

In Figures 5-8, wave probe 14 is in deepest water, probe 12 is about 2m (model) up the 1:30 slope, probes 8 and 5 are over the slope, and probe 3 is at the top of the slope.

Under breaking, the largest waves in the distribution break first, reducing towards the breaking limit. After some breaking, it is likely that a proportion of the energy will be re-distributed, perhaps combining with waves that have shoaled further. In still

shallower depths, further breaking will then apply over a greater part of the wave height distribution.

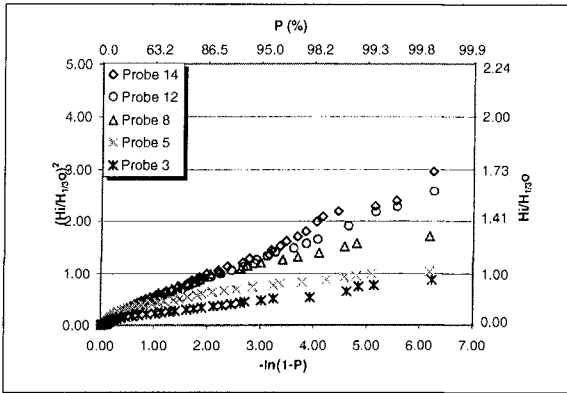


Figure 6 Individual wave heights, $s_{po} = 0.034$, 1:30 slope

In these test facilities, wave heights at the gauge in deepest water (probe 14) give only a slight curve away from the theoretical Rayleigh distribution. The effect of breaking for this sea state is relatively uniform, shown by the steady decrease of wave heights at each successive probe from that in deepest water (probe 14) to the shallowest (probe 3).

Some of these processes are illustrated for a typically steep storm sea state ($s_{po} = 0.054$) in Figure 5. These results do not support the suggestion that individual waves above the breaking limit ($H_i > H_b$) will fall to that limit whilst waves smaller than the breaking limit are unaffected.

Even in the relatively shallow water in these

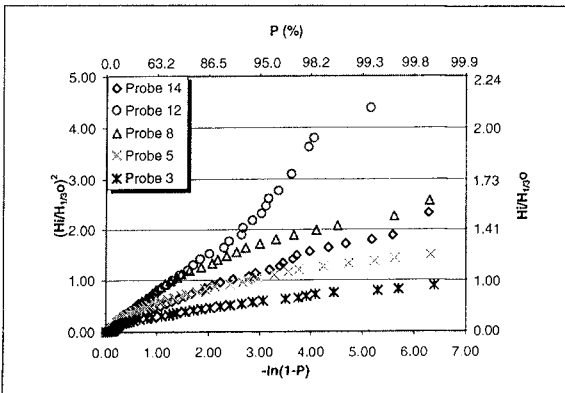


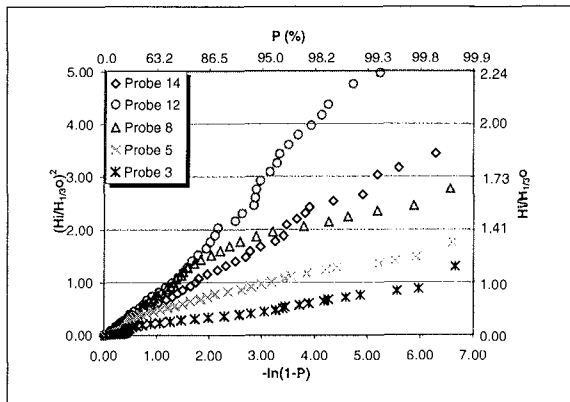
Figure 7 Individual wave heights, $s_{po} = 0.018$, 1:30 slope

behaviour will be important for any response that is more strongly influenced by the largest waves in the distribution. Further inshore at probes 5 and 3, the effect of wave breaking again gives more uniform reductions of wave height across the full range of exceedance values.

Results for a smaller wave height and lower wave steepness ($s_{po} = 0.034$) shown in Figure 6 more closely match normal expectations. Individual heights up to H_s (given here by $H_{1/3}$ at $-\ln(1-P)=2.0$) are hardly affected by wave breaking up to probe 8. There are however noticeable reductions in wave heights above 98%, reducing $H_{99.4\%}$ by perhaps 20%. This

Retaining the same offshore significant wave height, but further reducing the wave steepness to $s_{po} = 0.018$ in Figure 7, and $s_{po} = 0.007$ in Figure 8 gives more surprising

results, although some of these effects might have been anticipated from the shoaling behaviour shown by some of the results examined by Durand & Allsop (1997).



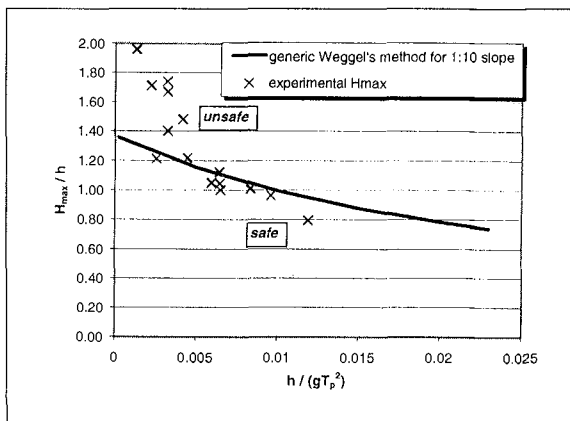
In Figure 7, the largest waves occur over the start of the approach slope (probes 12 and 8), particularly noticeable for wave heights above $H_{1/3}$. For very low steepnesses in Figure 8, wave heights at $H_{1/3}$ and higher non-exceedance levels increase markedly from the seaward point (probe 14) up the early part of the approach slope (probes 12 and 8). At this last position (probe 8), breaking only reduces individual wave

Figure 8 Individual wave heights, $s_{p0} = 0.007$, 1:30 slope

heights below their offshore value above about 98% non-exceedance. Breaking only starts to reduce $H_{1/3}$ by probe 5, well up the approach slope.

4.4 Comparison with prediction methods

As discussed above, random waves do not give a single breaking point, so the identification of onset of breaking under random waves is difficult and may give inaccurate results. For simple prediction methods, more useful comparisons are with wave heights measured after the onset of breaking. These comparisons are however complicated by different definitions of wave height given by the different prediction methods. Weggel's and Goda's methods give estimates of wave heights close to the maximum, H_{max} and $H_{1/250}$, Owen's simple method gives estimates of H_{sb} .



Considering first maximum wave heights, measurements of H_{max} shown relative to the local water depth as H_{max}/h_s are compared with predictions using Weggel's method for a bed slope of 1:10 in Figure 9, for a slope of 1:30 in Figure 10, and for a slope of 1:50 in Figure 11. The results have been taken from the point of breaking (indicated by the first

Figure 9 Weggel's H_{max} prediction, 1:10 slope

reduction of H_s , see for example Figures 3 & 4). Weggel's method does not predict shoaling, so no comparison is made for positions seaward of the breaking point.

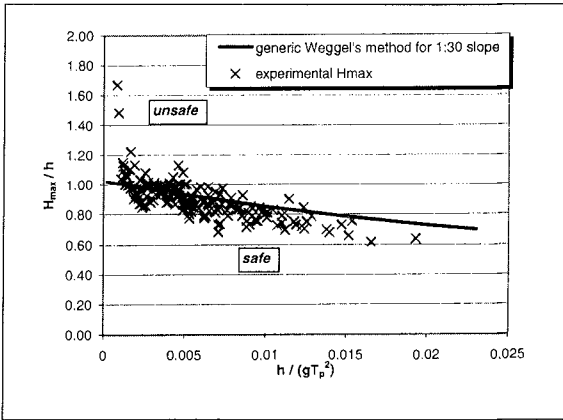


Figure 10 Weggel's H_{max} prediction, 1:30 slope

For the 1:10 slope in Figure 9, Weggel's prediction gives relative breaking heights H_{max}/h_s up to 1.4 at the lowest values of relative depth. Measured maximum wave heights significantly exceed the predicted values for $h/gT_p^2 < 0.005$, suggesting that the prediction method may give unsafe results in this region. Safer predictions for the 1:10 slope are given for $0.005 < h/gT_p^2 < 0.015$.

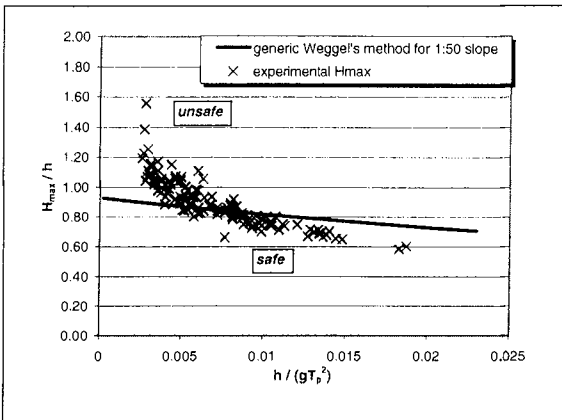


Figure 11 Weggel's H_{max} prediction, 1:50 slope

For the 1:30 slope in Figure 10, Weggel's prediction gives relative breaking heights H_{max}/h_s up to about 1.0 at the lowest values of relative depth. Measured maximum wave heights significantly exceed predicted values for $h/gT_p^2 < 0.002$, suggesting that the prediction method may give unsafe results in this region. Safer predictions for the 1:30 slope are given for $0.002 < h/gT_p^2 < 0.02$.

For the 1:50 slope in Figure 11, Weggel's prediction gives relative breaking heights H_{max}/h_s up to about 0.9 at the lowest values of relative depth measured here. Maximum wave heights however significantly exceed predicted values for $h/gT_p^2 < 0.007$, suggesting that Weggel's method may give unsafe results in this region. Safer predictions for the 1:50 slope are given for $0.007 < h/gT_p^2 < 0.02$, but the prediction method does still not seem to reproduce well the general effect of increasing breaker index with decreasing relative depth h/gT_p^2 .

Measurements of H_{max} from tests on the 1:30 slope were compared with predictions by Weggel's and Goda's methods by Durand & Allsop (1997). It was noted that only

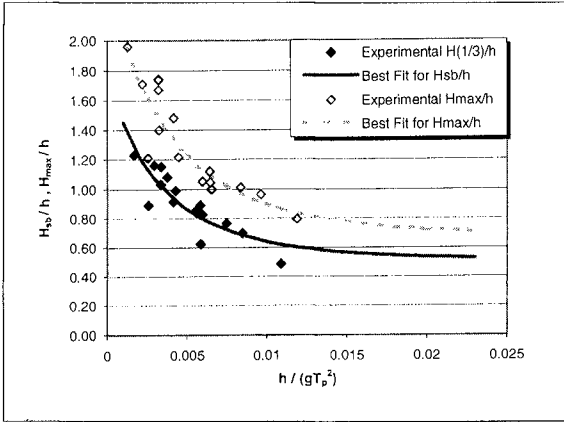


Figure 12 New prediction, 1:10 slope

a few design methods for coastal structures / breakwaters use the maximum wave height, or $H_{1/250}$ as used by Goda for wave forces on caissons. Most such methods require predictions of $H_{1/3}$. Goda's predictions for $H_{1/3}$ would therefore be potentially more useful, but these predictions agreed less well with measurements of $H_{1/3}$ than the agreement with H_{max} .

An alternative approach was therefore sought in which a revised empirical method was fitted to results for bed slopes of 1:10, 1:30 and 1:50, see figures 12-14. In deriving the new empirical method, it was important that the new method will:

- a) reproduce the general form of breaking with respect to h/gT_p^2 ;
- b) reproduce reliably the asymptote at high relative depths;
- c) give better description of the breaking limits at low values of h/gT_p^2 ;
- d) describe limits for H_s and H_{max} using equations of the same form.

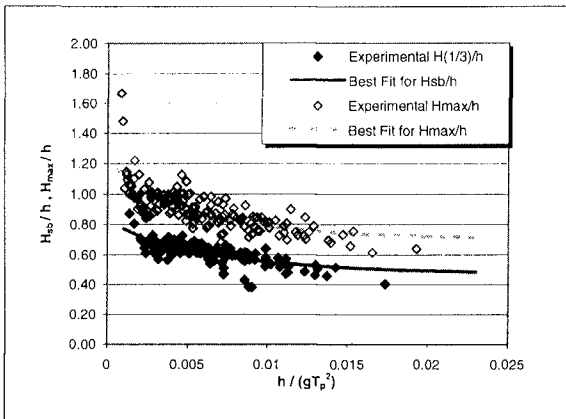


Figure 13 New prediction, 1:30 slope

It was also intended that the new method would be simple to apply in desk study calculations.

A form of equation relating both H_s / h_s and H_{max} / h_s to relative depth h/gT_p^2 was sought. Each potential method was tested against data from these studies for H_s and H_{max} , and for bed slopes of 1:10, 1:30 and 1:50. The equation was:

$$y = y_{\infty} + (y_0 - y_{\infty}) e^c \tag{8}$$

where $y = H_{max} / h_s$ or $H_{1/3} / h_s$ $x = h_s / gT_p^2$
 and $c = -bx^{0.78}$

Values of the coefficients were derived by minimising total errors in the fit to measured wave heights from these studies, and are summarised in Table 1:

Table 1: Coefficients for eqn 8

	Bed slope		
	1:50	1:30	1:10
Coefficients for $H_{1/3}$			
$(y_0 - y_\infty)$	0.831	0.394	1.394
b	69.33	44.78	69.88
y_∞	0.419	0.464	0.513
Coefficients for H_{max}			
$(y_0 - y_\infty)$	1.22	0.587	1.937
b	79.72	69.83	79.94
y_∞	0.621	0.700	0.679

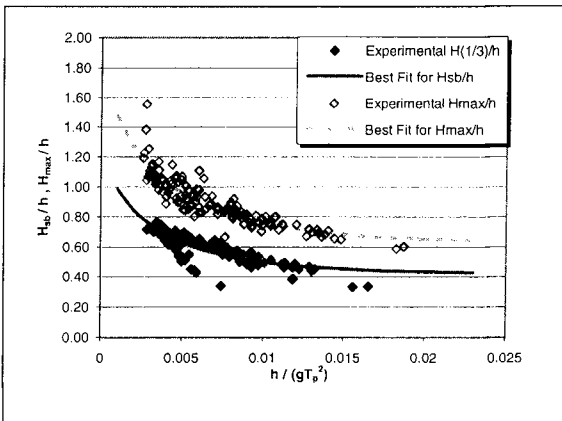


Figure 14 New prediction, 1:50 slope

5. CONCLUSIONS

These studies have demonstrated that present methods to predict inshore wave conditions on steep bed slopes for structure design suffer some important limitations.

Wave shoaling is particularly important in increasing wave heights on steep bed slopes, but requires the use of non-linear methods such as

Shuto's to give the full increase of wave height over steep bed slopes.

Weggel's method is widely used to represent wave breaking in many numerical methods, but under-predicts breaking for low relative depths. A new set of equations / coefficients have been developed and are suggested here. Whilst still requiring further checking, the new methods seem relatively robust.

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