CHAPTER TWO

BREAKING WAVE DESIGN CRITERIA

E.B. Thornton (M. ASCE)[1], C.S. Wu (M. ASCE)[1], R.T. Guza[2]

ABSTRACT

Breaking wave heights measured in both field and random wave laboratory experiments are examined. The dependence of breaker height and breaker depth on beach slope and deep water steepness is presented. The results are compared with the design curves of the Shore Protection Manual (SPM) and the predictions of the random wave model by Goda (1975). The comparisons indicate that the significant breaker height, based on Goda's model, is slightly conservative for the experimental cases; but the maximum breaker heights are reasonably predicted by the model. The design procedures in the SPM are based on a monochromatic wave breaking, and appear overly conservative, particularly for low wave steepness (less than 0.01) which occur frequently on the West Coast of the United States. The use of the Rayleigh distribution to predict wave height statistics is tested with random wave data for both deep and shallow water regions.

INTRODUCTION

The selection of breaking design waves is essential for the design of a coastal structure or for the coastal sediment problem. The present design practice is to specify maximum breaking waves based on empirical curves derived primarily from laboratory experiments of monochromatic waves (constant period and wave height). Several concerns arise from using monochromatic laboratory wave data as a basis for prototype design. Uncertainties exist in the scaling of laboratory waves to the prototype. More importantly, waves in nature are not monochromatic but random, having variable period, height and direction. The observed mean breaker height for random waves is generally 30-40% below the breaker inception height for periodic waves. Hence, uncertainty exists when applying criterion based on monochromatic waves to actual conditions in nature.

The objectives of this paper are to synthesize available random wave experiments, both in the field and laboratory, and to compare the

[1]Department of Oceanography, Naval Postgraduate School, Monterey, CA 93943

[2]Shore Processes Laboratory, Scripps Institution of Oceanography-A009, University of California, La Jolla, Califronia 92093

results with the random wave model of Goda (1975) and the breaking wave design curves in the Shore Protection Manual (US Army Corps of Engineers, 1977).

RANDOM WAVE DATA

During the past decade, there has been a growing recognition that significant differences exist between the results of monochromatic and random wave experiments. At the same time, primarily due to better instrumentation, a large number of comprehensive nearshore field experiments have been conducted. A difficulty in synthesizing various experiments, particularly the field data, is that the data were collected in different manners. The requirements for inclusion in the data base here are: 1) the waves are random, either measured in the field or simulated in the laboratory; 2) the data are for dissipative, progressive waves on relatively plane sloping, unbarred beaches; 3) the wave measurement locations be close enough to accurately define the position of the mean breaking wave height; 4) the data be given in terms of either significant height, $H_{1/3}$, or maximum height, H_{max}. Based on the above requirements, two sets of field data collected under the Nearshore Sediment Transport Studies (NSTS) and two sets of laboratory experiments on wave shoaling are included in the present paper.

Torrey Pines Beach, San Diego, California. The beach and nearshore at Torrey Pines Beach is gently sloping with nearly parallel and plane contours. During the experiments, significant offshore wave heights varied between 60 and 160 cm. The average peak frequency of the incident wave spectra varied little during the experiments and was about 0.07 Hz. Shadowing by offshore islands and offshore refraction limit the angles of wave incidence in 10-m depth to less than 15°. It was shown by Guza and Thornton (1980) that because of the small incident angles, refractive effects can be neglected in calculating shoaling processes. The condition of nearly normally incident spilling (or mixed plunging-spilling) waves, breaking in a continuous way across the surf zone, prevailed during most of the experiments. Winds during the experiments were generally light and variable in direction. Surface elevation and horizontal, orthogonal velocity components were measured by using a closely spaced array of up to 17 instruments in a shore-normal transect from offshore at the 10-m depth contour to across the surf zone (see Figure 2 in Thornton and Guza [1983]).

Leadbetter Beach, Santa Barbara, California. The mean nearshore slope at Leadbetter Beach varied between 0.017 and 0.05 during the experiment, depending on the wave climate. No offshore bar was apparent. The shoreline has the unusual east-west orientation along a predominantly north-south coast. The open ocean waves are limited to a narrow window of approach (±9° centered on 270°) because of the protection from Point Conception to the north and the Channel Islands to the south. The generally highly filtered ocean swell type waves from almost due west must make a right angle turn to approach the beach normally. As a result, waves approach at large oblique angles to the bottom contours in the surf zone and drive a strong longshore current.

Because of the relatively large incident wave angles, refractive effects must be accounted for in the shoaling calculations. A similar array to that at Torrey Pines was used to measure the wave height transformation from 9-m depth to the shoreline (See Thornton and Guza, 1984).

Laboratory Experiments by Goda (1975) and CERC. Goda (1975) conducted a series of experiments in a 30 m long laboratory wave flume using random waves. Two beach slopes of 0.02 and 0.1 were used. Different wave spectra were employed simulating single peaked wind waves, douple peaked sea and swell superposed, narrow swell waves, and relatively broad banded waves. Wave heights were calculated at six locations spanning the surf zone. Random wave laboratory experiments were also performed at CERC and have been variously described by Seelig et al (1983), Thompson and Vincent (1984), and Vincent (1984). The plane bottom slope was 1:30 in a 45.7 m long tank. Measurements were made at nine locations. Various theoretical wave spectra were simulated, including the Pierson-Moskowitz, JONSWAP and Ochi-Hubble Spectra.

The wave height statistics of H_{rms}, $H_{1/3}$ or H_{max} were calculated using the zero-up-cross technique. The surface elevations for the field data were first band-pass filtered (0.05-0.5 H_z). Goda (1975) uses $H_{1/260}$ for H_{max}, which is essentially the same statistic. All statistics are compared (nondimensionalized) using the deep water significant wave height H_o and deep water water wave length defined as $L_o = (g/2\pi)T_p^2$, where T_p corresponds to the wave period at the peak of the spectrum. Deep water wave heights were calculated by translating the measured nearshore wave heights to offshore accounting for shoaling and refraction using linear wave theory. For the data considered, refractive effects are needed to be accounted for only in the Santa Barbara data.

Monochromatic waves break on a plane beach at essentially a single location with a constant breaker height. Hence, a breaker height and depth are unambiguously defined. In contrast to monochromatic waves, there is no well-defined breakpoint for random waves; the largest waves tend to break farthest offshore and the smaller waves closer to shore. The result is a spatial distribution of breaking and unbroken waves. However, it is found that the use of a simple terminology for describing breaking wave parameters is informative and simplifies the analysis. For this reason, we introduce a mean breaker line for random waves. A "mean breaker line" is defined as the mean location where the averaged wave height reaches its maximum as the waves shoal from deep water and then dissipate due to breaking. As an example, the rms wave heights measured at Torrey Pines are shown in Figure 1. The mean rms breaker height H_B and surf zone width X_B are defined where H_{rms} reaches a maximum. Similar statistics are defined for $H_{1/3}$ and H_{max}, and an example is shown in Figure 2. This definition of mean breaker height means that the H_{rms} and $H_{1/3}$ statistic are made up of broken and unbroken waves. The H_{max} statistic

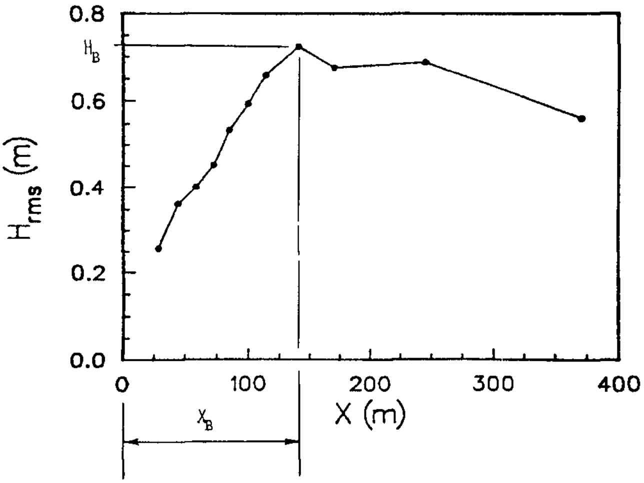

Figure 1. Definition of mean breaking wave height H_B, and corresponding surf zone width X_B.

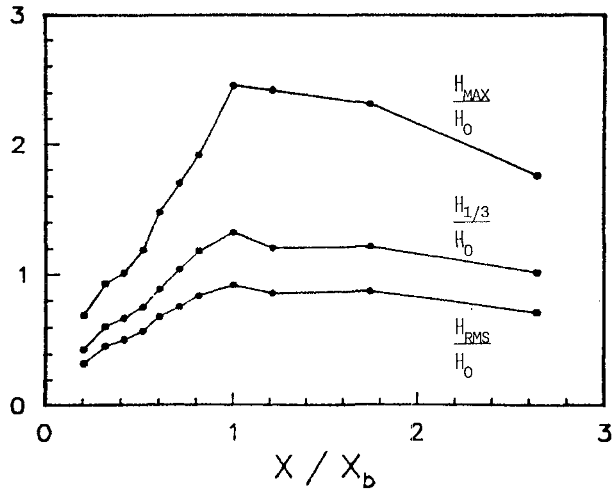

Figure 2. Wave height statistics H_{max}, $H_{1/3}$, and H_{rms} normalized by deep water significant wave height H_o, as functions of distance offshore.

corresponds to the single largest wave measured during the experimental interval and presumably corresponds to a breaking, or incipient breaking wave.

The difficulty of using this definition for mean breaker height is that measurement locations need to be closely spaced to accurately locate the point of the maximum wave height. Also, in some of the data there appeared to be no maximum; this result occurred for both the field and laboratory data (where the obvious blame on refractive effects are not present).

RESULTS

The breaking wave height data are compared with breaking wave design curves calculated by Seelig (1979), who employs the random wave height transformation theory by Goda (1975). Goda's theory describes the wave heights using a modified Rayleigh distribution in which the tail of the distribution is shortened, supposedly to represent the decrease in wave height due to breaking. Wave transformation is described using the nonlinear theory by Shuto (1974). Breaker height is expressed by

$$\frac{H_b}{H_o} = A\frac{L_o}{H_o}[1- \exp(-1.5\cdot\frac{h}{L_o}[1 + K\tan^s\beta])] \qquad (1)$$

where H_b is the breaking wave height, h is the local depth and $\tan\beta$ is the beach slope. The breaking wave heights are described as varying linearly over a range of values from most frequent breaker height to maximum breaker height dependent on the coefficient $A = (0.12, 0.18)$, and other coefficients $K = 15$ and $s = 4/3$. The coefficient values were suggested by Goda (1975). Goda's theory predicts the shoreward transformation of the distribution (non-Rayleigh) of wave heights, including both broken and unbroken waves, accounting for wave growth due to shoaling and attenuation due to breaking.

Seelig (1979) used Goda's theory to calculate the "mean breaker height" and breaker depth at that location. Seelig defined the mean random breaking wave height in the same manner as used to define the breaker line for the data, i.e. the location of the maximum wave height in the shoaling transformation of the waves from offshore to the beach. Seelig calculated a series of random wave breaking design curves for various beach slopes and initial deep water wave steepnesses.

The significant breaking wave heights, $H_{1/3}$, are compared for various beach slopes in Figure 3. Laboratory data are indicated by open symbols and field data by closed symbols. The field data have lower wave steepness due to the predominantly low frequency Pacific swell (0.07 Hz) that prevailed during the field experiments.

The curves by Seelig, corresponding to beach slopes 0.1, 0.05 and 0.01, are shown as solid lines. The SPM breaking wave design curve based on monochromatic wave data for beach slope 0.02 is presented for

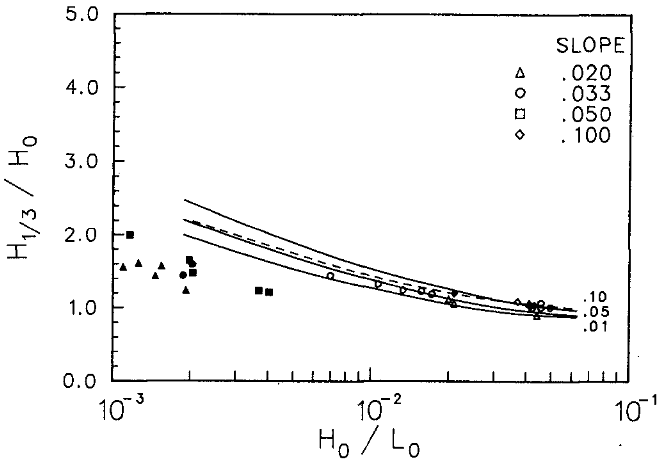

Figure 3. Significant breaking wave height, $H_{1/3}$, as a function of deep water wave steepness and beach slope. Shown are Seelig (1980) design curves for beach slopes 0.1, 0.05, and 0.01 based on Goda (1975) theory (solid lines) and SPM curve for beach slope 0.02 (dashed line).

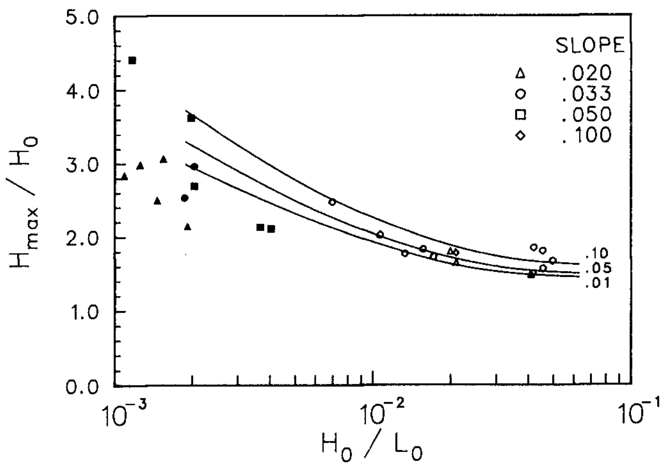

Figure 4. Maximum breaking wave height, H_{max}, as a function of deep water steepness and beach slope.

comparison (dashed line). The SPM curve is flatter, but generally falls within the range of the Seelig curves for random waves.

The Seelig curves reasonably predict the steeper wave slope laboratory data. This is expected since the coefficients used in (1) specifying Goda's theory are based on the same laboratory data collected by Goda. But the curves overpredict the significant breaker heights for initially low slope waves. Therefore, the Seelig curves appear to reasonably predict significant breaker heights for initially steeper waves ($H_0/L_0 > 0.7X10^{-3}$), but appear overly conservative for predicting significant breaker heights for initially low steepness waves.

The maximum breaking wave heights are compared with the Seelig design curves in Figure 4. The curves give reasonable predictions of maximum breaking waves, although the data do not align well with the beach slope dependence of the curves. It is pointed out that for the low wave steepness data, the maximum wave heights compare well with that by Goda's model, whereas the significant wave heights are over-predicted. This is fortuitous. The reason is that the actual wave heights conform more closely to a Rayleigh distribution than a Rayleigh distribution with a shortened tail. The Goda model, employing a modified Rayleigh distribution with a shortened tail, predicts a smaller increase in wave height from $H_{1/3}$ to H_{max} than the data, so that the H_{max} curves do not overpredict the measured values as much.

Thornton and Guza (1983) showed that for the Torrey Pines data the Rayleigh distribution could be used to calculate the H_{max} with an average error of -7% (under-prediction). Comparisons of the Santa Barbara data with the Rayleigh distribution are shown for $H_{1/3}$ in Figure 5 and for H_{max} in Figure 6. The Rayleigh distribution predicts

$$H_{1/3} = 1.41\ H_{rms} \tag{2}$$

Figure 5 shows that most of the $H_{1/3}$ wave heights plotted as a function of depth fall within $\pm5\%$ (dashed line) of (2). The wave heights in deeper water (depth $\geq$ 4 m) appear to agree better with the Rayleigh distribution than wave height in shallower water within the surf zone. The H_{max} data and values predicted from the Rayleigh distribution are compared in Figure 6. The average error of the regression curve (dashed line) from the 45° line is -9%, i.e., the Rayleigh distribution under-predicts the data by 9% on the average, although the scatter is considerably greater. This implies that the use of a modified Rayleigh distribution with a shortened tail as described by Goda (1975) to predict breaking wave height design conditions is nonconservative; it is found from field measurements that the use of Rayleigh distribution is also nonconservative.

The depth at the significant breaking wave height, d_b, is plotted as a function of wave steepness and beach slope in Figure 7. A beach slope dependence is evident. The data are reasonably represented by the Seelig curves and are only underestimated at the very lowest wave steepnesses.

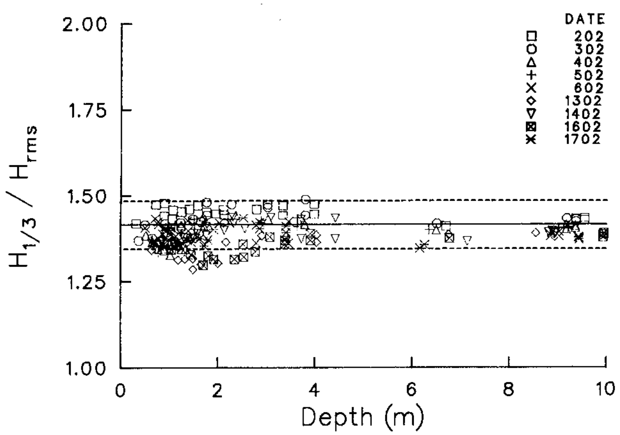

Figure 5. Significant wave heights, $H_{1/3}$, measured at Santa Barbara, California compared with wave height predicted by Rayleigh distribution (solid line) as a function of depth. Dashed lines indicate ±5% error.

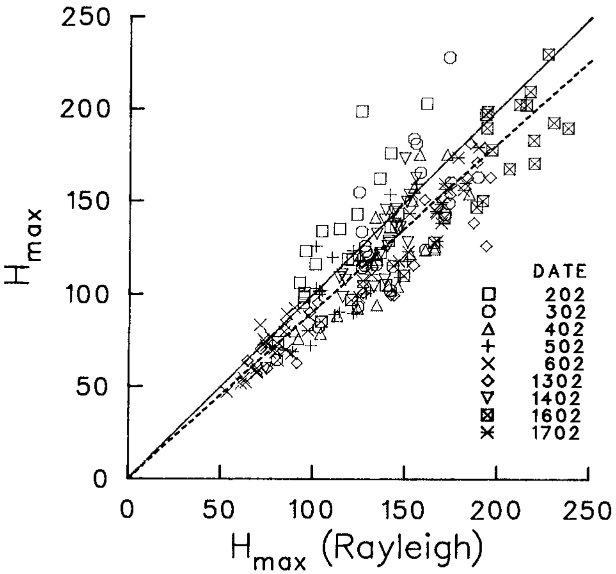

Figure 6. Maximum wave heights, H_{max}, measured at Santa Barbara, California compared with wave height predicted by Rayleigh distribution. Mean regression line is indicated by dashed line.

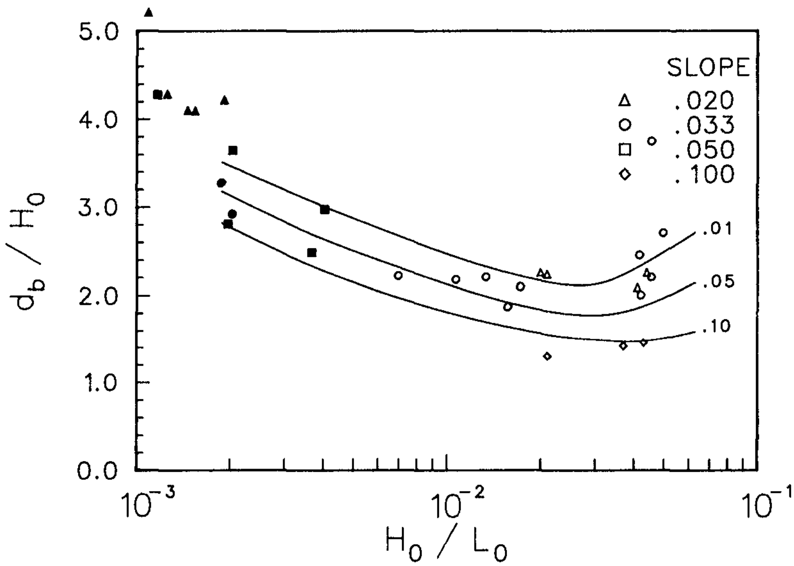

Figure 7. Depth at significant breaking wave height, d_b, as a function of wave steepness and beach slope.

SUMMARY AND CONCLUSIONS

Breaking wave heights measured in the field and in random wave experiments in the laboratory are compared with the random wave model of Goda (1975) as calculated by Seelig (1980) and with the Shore Protection Manual (1977). The random wave model suggests that wave breaking is dependent on beach slope and wave steepness. The data spans a range of beach slopes (0.02, 0.033, 0.05 and 0.10) and deep water wave steepness. The dependence on beach slope is, however, not obvious from the data. The laboratory data are of higher wave steepness ($H_0/L_0 > 0.7X10^{-3}$). The field data corresponds to low wave steepness as the result of low frequency (0.07 Hz) Pacific Ocean swell waves.

The Goda's model reasonably predicts $H_{1/3}$ and H_{max} for the higher wave steepness laboratory data; a reason being that much of the laboratory data is taken from Goda (1975), which is the same data used to calibrate the random wave model in the first place. For initially low steepness waves, the Goda model overpredicts $H_{1/3}$, but more reasonably predicts H_{max}. The H_{max} predictions are based on using a modified, shortened tail, Rayleigh distribution for which the H_{max} statistics are compensated for by the overprediction of $H_{1/3}$. Actual shallow water wave height data compare better, or are even underestimated in the tail, with a Rayleigh distribution as is demonstrated with the field data. Breaking wave heights do not exhibit a shortened or truncated, tail in their distributions.

The depth at breaking corresponding to the breaking wave height compared favorably with the Goda model for all wave steepness values. Depth at breaking exhibited a definite dependence on beach slope as suggested by the Goda model and the data.

ACKNOWLEDGEMENTS

This research was supported by the Office of Naval Research, Coastal Sciences Branch, under contract numbers NR 388-114 (E.B. Thornton), Naval Postgraduate School Foundation (C.S. Wu) and N00014-75-C-0300 (R.T. Guza). Much of the analysis was performed by D.O. Burych.

REFERENCES

Goda, Y., "Irregular wave deformation in the surf zone", Coastal Engineering in Japan 18, 13-26, 1975.

Guza, R.T. and E.B. Thornton, "Local and Shoaled Comparisons of Sea Surface Elevations, Pressures and Velocities", J. of Geophysical Research, 85, 1524-1530, 1980.

Seelig, W.N., "Maximum Wave Heights and Critical Water Depths for Irregular waves in the Surf Zone", Report No. 80-1, U.S. Army Corps of Engineers Coastal Engineering Research Center, 11 p., 1980.

Seelig, W.N., J.P. Ahrens and W.G. Grosskopf, "The Elevation and Duration of Wave Crests", Report No. 83-1, U.S. Army Corps of Engineers Coastal Engineering Research Center, 73 p., 1983.

Shuto, N., "Nonlinear long waves in a channel of variable section", Coastal Engineering. in Japan, 17, 1-12, 1984.

Thompson, E.F. and C.L. Vincent, Shallow Water Wave Height Parameters", J. Waterway, Port, Coastal and Ocean Engineering Division, ASCE, 110, 293-298, May, 1984.

Thornton, E.B. and R.T. Guza, "Transformation of Wave Height Distribution", J. of Geophysical Research, 8, 5925-5938, 1983.

Thornton, E.B. and R.T. Guza, "Longshore Current due to Random Waves", (Submitted to the J. of Geophysical Research) 1984.

U.S. Army Corps of Engineers, Shore Protection Manual, Coastal Engineering Research Center, 1977.

Vincent, C.L., "Energy Saturation of Irregular Waves During Shoaling", (Submitted to J. Waterway, Port, Coastal and Ocean Engineering, Division, ASCE) 1984.