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Estimating Nearshore Conditions for Irregular Waves

by

William N. Seelig and John P. Ahrens

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Methods for estimating nearshore irregular wave conditions for the case of continuously shallowing bottom contours, given the bottom slope and offshore wave characteristics, are presented. The methods are based on the work of ~~Goda~~ (1975a, ~~1975b) and~~ Goda, Takayama, and Suzuki ~~(1978)~~. A sensitivity analysis is performed to show the influence of various input parameters on predicted nearshore significant wave height. The methods are applied to the nearshore region at the CERC Field Research Facility at Duck, North Carolina, and results are compared to observed nearshore wave height changes measured at the facility. Design curves are included.

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PREFACE

This report presents state-of-the-art irregular wave prediction techniques developed by Goda (1975a, 1975b) and suggested by Goda, Takayama, and Suzuki (1978). When this report was prepared, little field data were available for checking the techniques presented. New data are being gathered at the U.S. Army Coastal Engineering Research Center's (CERC) Field Research Facility (FRF) at Duck, North Carolina, and results of comparing the new prediction techniques with this data are expected in early 1981.

Much of the design information in the Shore Protection Manual (SPM) assumes monochromatic waves; however, input wave conditions for many design analyses are often irregular. This report, which supplements SPM Sections 2.3, 3.85, and 7.12, presents methods developed for open sections of the coast with continuously shallowing bottom contours. The major emphasis of the report is on predicting nearshore wave height distributions. The work was carried out under the offshore breakwater for shore stabilization and wave runup and overtopping programs of CERC.

Goda's methods represent an important increase in the engineering community's ability to predict waves propagating into shallow water. The method is based on a number of simplifications and empirical adjustments but appears to represent laboratory and limited field data reasonably well. Additional comparisons of predictions with field data are underway. Meanwhile, prediction results should be carefully evaluated to confirm that the method is not used outside of its range of applicability.

The report was prepared by William N. Seelig and John P. Ahrens, under the general supervision of Dr. R.M. Sorensen, Chief, Coastal Processes and Structures Branch.

Comments on this publication are invited.

Approved for publication in accordance with Public Law 166, 79th Congress, approved 31 July 1945, as supplemented by Public Law 172, 88th Congress, approved 7 November 1963.

Ted E. Bishop

TED E. BISHOP
Colonel, Corps of Engineers
Commander and Director

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CONVERSION FACTORS, U.S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U.S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	by	To obtain
inches	25.4	millimeters
	2.54	centimeters
square inches	6.452	square centimeters
cubic inches	16.39	cubic centimeters
feet	30.48	centimeters
	0.3048	meters
square feet	0.0929	square meters
cubic feet	0.0283	cubic meters
yards	0.9144	meters
square yards	0.836	square meters
cubic yards	0.7646	cubic meters
miles	1.6093	kilometers
square miles	259.0	hectares
knots	1.852	kilometers per hour
acres	0.4047	hectares
foot-pounds	1.3558	newton meters
millibars	1.0197×10^{-3}	kilograms per square centimeter
ounces	28.35	grams
pounds	453.6	grams
	0.4536	kilograms
ton, long	1.0160	metric tons
ton, short	0.9072	metric tons
degrees (angle)	0.01745	radians
Fahrenheit degrees	5/9	Celsius degrees or Kelvins[1]

[1]To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use formula: $C = (5/9)(F - 32)$.

To obtain Kelvin (K) readings, use formula: $K = (5/9)(F - 32) + 273.15$.

SYMBOLS AND DEFINITIONS

d or d_{swl}	stillwater depth
E_θ	wave energy moving in direction θ
E_i	wave energy of component "i"
g	acceleration due to gravity
H_o'	deepwater equivalent significant wave height
H_o	deepwater significant wave height
H_s	significant wave height or average of the highest one-third waves
H_{rms}	root-mean-square wave height
H_1	average of the highest 1-percent waves
$\bar{H}$	mean wave height
K	constant with energy units
K_R	wave refraction coefficient
K_{Ri}	wave refraction coefficient of direction "i"
L_o	deepwater wavelength
m	beach slope
N	number of direction components used in refraction calculations
S_w	wave setup
S*	a parameter to quantify the directional spreading of wave energy
T_s	average wave period of the highest one-third waves
α	angle of nearshore wave energy vector
α_o	deepwater dominant wave direction
θ	wave angle

ESTIMATING NEARSHORE CONDITIONS FOR IRREGULAR WAVES

by
William N. Seelig and John P. Ahrens

I. INTRODUCTION

Waves are often irregular in height, period, and direction with some or most of the waves breaking or near the point of breaking during extreme wave conditions. Procedures for predicting design wave conditions for irregular waves are not discussed in the Shore Protection Manual (SPM) (U.S. Army, Corps of Engineers, Coastal Engineering Research Center, 1977). Much of the information in the SPM is for uniform unidirectional waves. This report supplements SPM Sections 2.3, 3.85, and 7.12 with state-of-the-art irregular wave prediction techniques developed by Goda (1975a, 1975b) and suggested by Goda, Takayama, and Suzuki (1978). Easy-to-use methods for estimating nearshore wave height, angle of approach, and resulting water level setup for irregular waves are presented. The methods are intended for open sections of the coast with continuously shallowing depth contours. Wave setup due to narrow band frequency spectra and surf beat is considered but other forms of wave-wave and wave-current interaction and spectral shape factors have been neglected. Design curves and examples of estimating the nearshore significant wave height are also available in Seelig (1979).

The transformation and attenuation of waves propagating from deep water to a beach is a problem of considerable difficulty because of a lack of field data and a poor theoretical understanding of the complex wave deformation process. The methods presented in this report are empirical in nature with the physics of the actual problem only partially understood. The results, based on laboratory and limited field data, are considered promising enough to recommend their application in selected field calculations. The results of calculations should be carefully examined to assure that the basic assumptions of the method have not been violated and that reasonable answers result.

In Section II the directional spreading of a wave energy model suggested by Goda, Takayama, and Suzuki (1978) is used to predict the refraction coefficient and refracted wave height for the nearshore point of interest. The height is then used, in Section III, as input to the surf zone wave height distribution model developed by Goda (1975a, 1975b) to estimate the nearshore wave conditions and setup.

The input information necessary for the application of these prediction techniques includes the deepwater significant wave height, wave period of peak energy density and dominant wave direction, the directional spreading of wave energy parameter, S^*, beach slope, and water depth at the point of interest. In any given design situation, some of these parameters may be more reliable than others. Analyses in Section IV show the sensitivity of predicted nearshore wave heights to these input parameters.

Section V applies the techniques presented in Sections II and III to predict wave transformation along the 580-meter-long pier at the Coastal Engineering Research Center's (CERC) Field Research Facility (FRF) at Duck, North Carolina, and compares calculated and measured results. An example is given in Section VI that applies the prediction methods to estimate nearshore significant wave height.

II. WAVE REFRACTION - DIRECTIONAL SPREADING OF WAVE ENERGY

The confused sea state in deep water may be described as the sum of wave trains simultaneously moving in various directions. As these wave trains move toward the coast, the waves with the largest angles between their crest and the bottom contours refract the most, so that nearshore waves appear to be less confused. The reason for the directional spreading of wave energy in refraction calculations for the case of *straight parallel bottom contours* is discussed in this section.

1. Directional Spreading of Wave Energy.

Wave direction in deep water is difficult to measure, especially when many wave trains of various energy levels are simultaneously moving in different directions. However, a few basic measurements will help to quantify this directional spreading. Longuet-Higgins, Cartwright, and Smith (1963) suggest the following density function for wave energy:

$$E_\theta = K\left(\cos\left(\frac{\theta}{2}\right)\right)^{2S^*} \qquad (1)$$

where

θ = wave direction angle with respect to the dominant deepwater direction, α_0 (Fig. 1; note that α_0 is measured from a line perpendicular to the shoreline)

K = a constant used to define the total wave energy

E_θ = the density of wave energy in a given direction, θ

S^* = a parameter that defines the variation of energy level with wave direction.

(Since the frequency dependance of directional spreading of wave energy is of secondary importance for diffraction (Goda, Takayama, and Suzuki 1978) and refraction calculations (Y. Goda, Director, Marine Hydrodynamics Division, Port and Harbor Research Institute, Japan, personal communication, 1979) it is neglected in this report.) Smaller values of S^* yield higher amounts of directional spreading of wave energy. Goda, Takayama, and Suzuki recommend the values of S^* in Table 1 for design purposes.

2. Refraction Calculations.

Refraction calculations are based on the energy-weighted superposition of refraction coefficients obtained from linear theory. If E_i and K_{Ri} are

Sample values of E_θ/K for various combinations of θ and S^*

S^*	$\theta = 0°$	15°	30°	60°	90°
4	1.000	0.934	0.758	0.316	0.063
12	1.000	0.814	0.435	0.032	0.0003
37	1.000	0.530	0.077	0.000	0.000

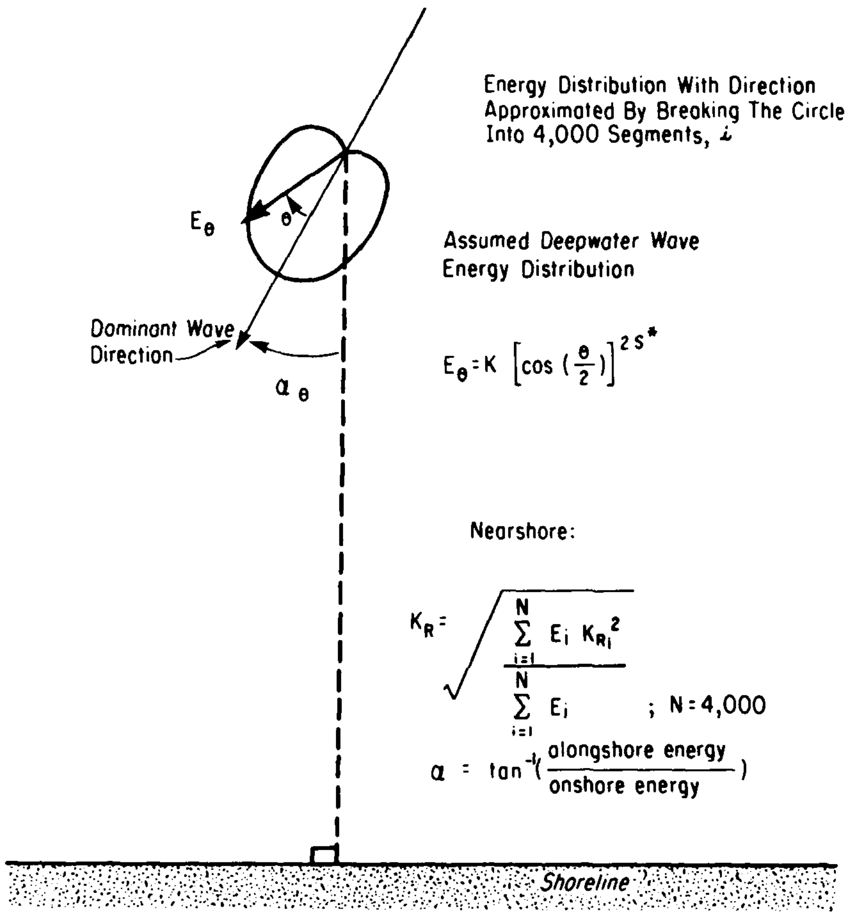


Figure 1. Wave refraction including directional spreading of wave energy.

Table 1. Recommended values of S*.

S*	Wave condition
4	Wind waves
12	Swell (short-to-moderate decay distances)
37	Swell (moderate-to-long decay distances)

the wave energy and refraction coefficient, respectively, for a wave direction, i, then the composite refraction coefficient for waves from several simultaneous directions is taken as

$$K_R = \sqrt{\frac{\sum_{i=1}^{N} E_i \, K_{Ri}^2}{\sum_{i=1}^{N} E_i}} \tag{2}$$

where N is the number of wave directions. The computations in Appendix A were performed by dividing the 360° circle into 4,000 equal angle segments (N = 4,000) and using the spreading function in equation (1). Individual refraction coefficients for each segment, K_{Ri}, are computed from the standard methods described in Section 2.32 of the SPM or McClenan (1975). The use of equation (2) in the calculation is to obtain a better estimate of refraction coefficients for irregular waves than would be obtained if a single value of the refraction coefficient obtained from linear theory were used. It is recognized that the design curves in Appendix A do not account for changes in spectral shape. Note that as S* approaches infinity the monochromatic wave refraction solution is obtained (agrees with Fig. 2-19 in the SPM).

3. Wave Refraction Analysis.

Calculation of refraction coefficients and nearshore wave direction angles using the design curves in Appendix A requires the dominant deepwater wave direction angle, α_o, an estimate of S*, and the representative wave period, T_s. For irregular waves the period of peak energy density, which is approximately equal to the mean period of the highest one-third waves, should be used as the representative wave period. Each figure in Appendix A is for a different value of S*. The refraction coefficient, K_R, is determined by finding the intersection of the deepwater wave angle, α_o, on the abscissa with the value of $d/(gT_s^2)$ on the ordinate where d is the nearshore water depth of interest and g is the acceleration due to gravity. K_R is estimated by interpolation between curves of constant K_R. The nearshore angle of the wave energy vector, α (also shown in terms of contours of constant values) is the angle that should be used, for example, in determining the component of wave energy flux in the longshore direction.

Table 2 illustrates sample values of K_R and α for various values of S* and α_o where $d/(gT_s^2) = 0.01$. Note that for small, deepwater wave angles the wave refraction coefficient is smaller for lower values of S*; for large deepwater wave angles the refraction coefficient is larger for smaller values of S*.

Table 2. Comparison of refraction coefficients for selected S^* and α_o values at $d/(gT_s^2) = 0.01$.

S^*	2	4	10	25	75	Infinity
			K_R			
α_o						
0°	0.85	0.92	0.97	0.98	0.99	1.00
45°	0.77	0.82	0.85	0.87	0 88	0.88
90°	0.56	0.52	0.44	0.36	0.29	0.00
			α			
α_o						
0°	0	0	0	0	0	0
45°	11	16	22	25	25	25
90°	22	30	34	39	40	-

The equivalent deepwater wave height, H_o', is determined from

$$H_o' = K_R H_o \tag{3}$$

where H_o is defined as the deepwater significant wave height. H_o' should also include diffraction or any other loss coefficients if they are significant.

4. Example Problem in Wave Refraction.

GIVEN: The wave period, $T_s = 10$ seconds, a dominant deepwater wave angle, $\alpha_o = 40^o$, and a significant wave height, $H_o = 2.0$ meters.

FIND: The equivalent deepwater significant wave height, H_o', and the nearshore angle, α, for wind waves at a water depth of 1.0 meter.

SOLUTION: From Table 1 the value of $S^* = 4$ is selected for wind waves. The term $d/(gT_s^2)$ has the value

$$\frac{1.0}{(9.8 \times 10^2)\ 0.001}$$

From Appendix A, $K_R = 0.80$ and $\alpha = 4.5^o$, so from equation (3) the deepwater equivalent wave height is

$$H_o' = K_R H_o = 0.80(2.0) = 1.60 \text{ meters}$$

III. NEARSHORE WAVE BREAKING MODEL

Goda (1975a, 1975b) developed a nearshore wave height prediction model for irregular waves that accounts for wave breaking, nonlinear wave shoaling, irregular wave setup, and surf beat. A brief description of Goda's model is given below.

1. Characteristics of Goda's Model.

Goda assumed that the deepwater significant wave height, H_o, and the average period of the significant waves, T_s, are known or can be estimated. The wave heights are assumed to have a Rayleigh distribution in deep water and this distribution is used to characterize wave heights as the waves move into shallower water until a depth is reached where the waves begin breaking. Goda's approach allows the broken waves to reform at a lower height, so that the wave height distribution is no longer described by the Rayleigh distribution. As the waves move into shallower water the nonlinear method developed by Shuto (1974) is used to estimate wave shoaling coefficients. Shuto's method of calculating shoaling coefficients usually gives somewhat higher waves than would be predicted using the conventional linear shoaling method. Nonlinear shoaling is consistent with the observed behavior of waves in shallow water and is conservative when compared to linear shoaling.

The offshore beach profile is assumed to be represented by a straight plane surface. Dissipation of wave energy by bottom friction is usually very small for typical sand beaches; therefore, bottom dissipation is neglected.

Wave setup or setdown and surf beat are related to the wave breaking process and are accounted for in the model. The radiation stress (i.e., wave-induced transport or momentum) of the waves progressing toward the shore causes wave setup which can either increase or decrease the local water depth; a decrease is often referred to as setdown. Setdown occurs *seaward* of the breaker zone and setup occurs *shoreward* from the point where a significant number of the waves break. Surf beat is the longer period component of water level oscillation (periods from 20 seconds to several minutes) due to longer period irregularities in wave action. The magnitude of the surf beat is amplified in shallow water. Figure 2 shows the conditions used in this model.

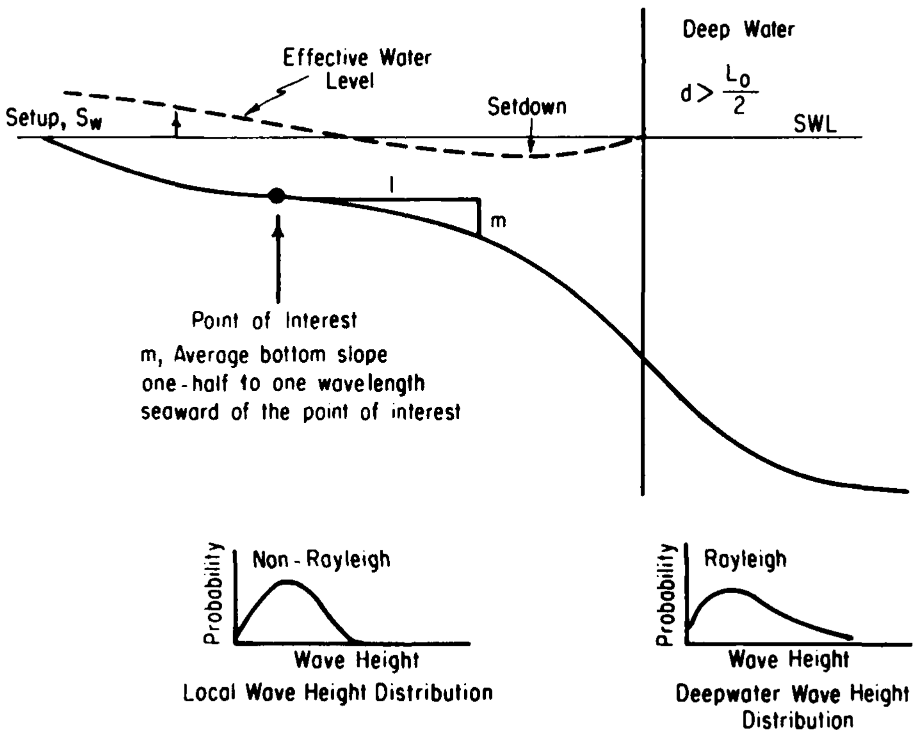


Figure 2. Definition sketch.

2. Design Curves.

Appendixes B and C present formats of design curves obtained from Goda's analytical model (Seelig, 1978). The first format (App. B) includes plots of five variables in the nearshore zone: (a) The maximum wave height, H_1, defined as the mean of the highest 1 percent of the waves; (b) the significant wave height, H_s, defined as the mean of the highest one-third waves (the significant wave height is approximately equal to four times the root-mean-square (rms) surface elevation of a water level record); (c) the rms wave height, H_{rms}; (d) the mean wave height, $\overline{H}$; and (e) the wave setup, S_w. All of these variables are divided by the deepwater significant wave height. The five variables are plotted on the ordinate versus the ratio of the local stillwater depth, d, to the deepwater significant wave height, H_o', on the abscissa. One plot is available for each wave steepness and for each beach slope.

The second format (App. C) gives the ratio of the local significant wave height divided by the stillwater depth on the ordinate versus $d/(gT^2)$ on the abscissa.

Both formats give the same information about the significant wave height, but tne first format is often more useful when starting with the deepwater parameters H_o' and T_s. The second format is useful for taking highcast significant wave heights measured at one water depth and using this information to estimate the significant height at another shallower depth.

3. Calculation of Nearshore Wave Heights and Water Level Parameters.

a. Parameters. Parameters needed to use the curves in Appendixes B and C are the equivalent deepwater significant wave height, H_o', the representative deepwater wave period, T_s, and the offshore bottom slope, m. For an irregular profile, take the average profile slope one-half to one wavelength seaward of the point of interest.

b. Procedure for Computation of the Offshore Wave Steepness, H_o'/L, and Depth-to-Height Ratio, d/H_o'. If H_o' is in feet and T_s is in seconds, then

$$\frac{H_o'}{L_o} = \frac{H_o'}{(5.12\ T_s^2)} \tag{4}$$

L_o is taken as defined by monochromatic linear theory, so that L_o can be considered a reference wavelength. The actual wavelengths during irregular wave conditions may be variable and influenced by factors such as spectral shape and wave interaction. The advantage of using linear theory to describe L_o is that the same wave and setup information can be provided in a more convenient form than if some nonlinear theory were used to characterize wavelength. If H_o' is in meters and T_s is in seconds, then

$$\frac{H_o'}{L_o} = \frac{H_o'}{(1.56\ T_s^2)} \tag{5}$$

The ratio of the water depth at the point of interest to H_o' is determined by using equation (3)

$$\frac{d}{H_o'}$$

c. Evaluation of Parameters of Interest. To evaluate the nearshore parameters, use the figure in Appendix B having the offshore slope, m, and offshore wave steepness (H_o'/L_o) closest to the values of interest. Enter the value of d/H_o' on the ordinate of the graph, and select the curve of the parameter of interest. Five dimensionless parameters are presented:

Curve	Description
1	S_w/H_o'
2	H_{rms}/H_o'
3	$\overline{H}/H_o'$
4	H_s/H_o'
5	H_1/H_o'

Use the abscissa on the right for curve 1 (setup) and the left abscissa for curves 2 to 5. Setup has an enlarged scale because it is small compared to wave height.

4. Example Problems to Determine Nearshore Conditions (Neglecting Refraction).

* * * * * * * * * * * * * * EXAMPLE PROBLEM 1 * * * * * * * * * * * * * * * *

GIVEN: The offshore slope is 1/100, $H_o' = 3.0$ meters, and $T_s = 14$ seconds.

FIND: The rms, mean, significant, and maximum wave heights and mean water level at a stillwater depth of 6.0 meters.

SOLUTION: The refracted wave steepness from equation (6) is

$$\frac{H_o'}{L_o} = \frac{3.0}{(156(14)^2)} = 0.01$$

and the depth to deepwater wave height ratio is

$$\frac{d}{H_o'} = \frac{6.00}{3.00} = 2.0$$

From Figure B-4 in Appendix B,

Curve	Variable	Ratio value	Value (m)
1	S_w/H_o'	-0.02	S_w = -0.06
2	H_{rms}/H_o'	0.87	H_{rms} = 2.6
3	$\overline{H}/H_o'$	0.81	$\overline{H}$ = 2.4
4	H_s/H_o'	1.23	H_s = 3.7
5	H_1/H_o'	1.57	H_1 = 4.7

Mean water level = $d + S_w$ = 6.00 - 0.06 = 5.94 meters.

* * * * * * * * * * * * * * * * EXAMPLE PROBLEM 2* * * * * * * * * * * * * * * * *

GIVEN: The average offshore slope is 1/100 and T_s = 8.0 seconds.

FIND: The variation of significant wave heights at d = 1.50 meters for various offshore wave steepnesses assuming no refraction effects.

SOLUTION: $L_o = 1.56\ T_s^2 = 99.8$ meters and from Figure C-1 (App. C) for a slope of 1 on 100 and $d/L_o = 1.5/(1.56(8)^2) = 0.015$, the following values of H_s/d and H_s are obtained for selected values of H_o'/L_o:

$\frac{H_o'}{L_o}$	H_o' (m)	$\frac{H_s}{d}$	H_s at d_{swl} = 1.5 m (m)
0.005	0.5	0.5	0.8
0.01	1.0	0.66	1.0
0.02	2.0	0.73	1.1
0.04	4.0	0.77	1.2
0.08	8.0	0.85	1.3

Depth-limited breaking is important in this example for the larger waves, so that as deepwater height increases from 0.5 to 8.0 meters, the nearshore height only increases from 0.8 to 1.3 meters in a 1.5-meter water depth.

* * * * * * * * * * * * * * * EXAMPLE PROBLEM 3 * * * * * * * * * * * * * * * *

GIVEN: A wave gage in 4.0-meter stillwater depth measured a significant wave height of 2.0 meters with T_s = 7.6 seconds. The nearshore bottom slope was m = 1/50.

FIND: The significant wave height at a second location with a stillwater depth of 1.0 meter.

SOLUTION: For these conditions at location 1,

$$\left(\frac{H_s}{d}\right)_1 = \frac{2.0}{4.0} = 0.50$$

$$\left(\frac{d}{gT^2}\right)_1 = \frac{4.0}{(9.8 \times 7.6^2)} = 0.007$$

At location 2,

$$\left(\frac{d}{gT^2}\right)_2 = \frac{1.0}{(9.8 \times 7.6^2)} = 0.0018$$

To find the significant wave height at location 2, enter Figure C-2 (m = 1/50) for $(d/gT^2)_1 = 0.007$ and find the H_o'/gT^2 where $(H_s/d)_1 = 0.50$. In this case

$H_o'/gT^2 = 0.0032$. Follow the curve for H_o'/gT^2 to the point where $(d/gT^2)_2 = 0.0018$ and read off the ordinate $(H_s/d)_2 = 0.81$. The significant wave height at location 2 is

$$H_s = \left(\frac{H_s}{d}\right)_2 \times d_2 = 0.81 \times 1.0 = 0.81$$

Note that the ratio of wave height to stillwater depth is large because wave setup increases the effective water depth in this case for a steep wave and a steep beach.

IV. SENSITIVITY ANALYSIS

In any given design situation the quality of the input information may vary. An important question to ask when predicting nearshore wave conditions is: How sensitive are the predicted conditions to errors of uncertainty in the input parameters? The sensitivity of the predicted nearshore significant wave height to input parameters is illustrated below with an example. A reference condition is chosen and nearshore significant wave heights are estimated. Each of the input parameters is then systematically varied and the results compared to the reference condition.

The reference condition was selected to have the following input parameters:

H_o' = deepwater significant wave height (5 meters)

T_s = wave period (10 seconds)

d = nearshore water depths (0.5 to 30 meters)

m = beach slope (1/100)

For the reference condition the predicted nearshore significant wave height is slightly less than the deepwater wave height for water depth of 10 to 30 meters due to wave setdown (Table 3). The predicted height then decreases in shallow water to a value of 0.96 meter in a 1-meter water depth (stillwater level). Systematic variations are made from the reference condition and from the resulting predicted nearshore wave heights given in Table 3. Comparisons with the reference condition are shown in Figure 3.

Table 3. Sensitivity analysis for nearshore wave height prediction for a selected deepwater condition.

Stillwater depth (m)	0.5	1.0	3.0	5.0	10.0	20.0	30.0
Reference d/H_o'	0.1	0.2	0.6	1.0	2.0	4.0	6.0
Condition	Predicted nearshore significant wave height (m)						
Reference[1]	0.7	0.96	2.1	3.2	4.9	4.6	4.6
Variation							
Water 1 m deeper	1.3	1.6	2.7	3.8	5.0	4.7	4.7
1/20 slope	0.9	1.3	2.6	3.9	5.2	4.8	4.8
H_o' = 6 m	0.7	0.9	2.2	3.3	5.4	5.6	5.8
H_o' = 4 m	0.6	0.9	2.1	3.1	4.0	3.8	3.7
α_o = 45°, S^* = 4	0.7	0.9	2.0	3.1	4.1	3.9	4.0
T_s = 6 s	0.5	0.8	1.9	2.8	4.1	4.8	5.0
T_s = 18 s	0.8	1.1	2.3	3.5	6.0	5.2	5.1

[1]Parameters: H_o' = 5.0 meters; T_s = 10.0 seconds; beach slope = 1/100; no refraction

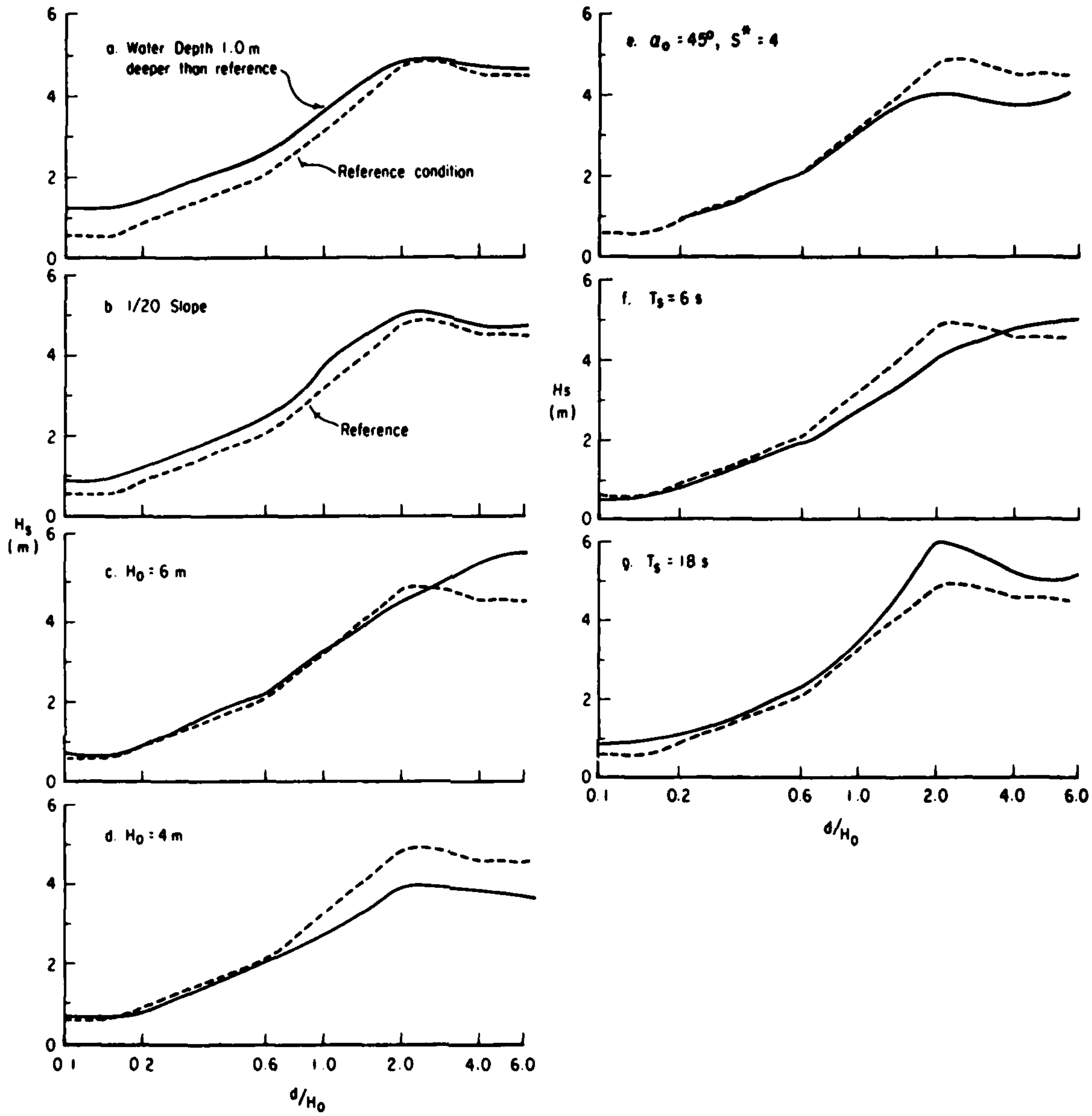


Figure 3. Sensitivity analysis of nearshore predicted significant wave heights.

The example in Table 3 and an inspection of Appendixes A, B, and C suggest the following general rules when predicting nearshore wave height from deepwater conditions:

(a) Water depth becomes an increasingly important factor for predictions of wave height in shallower water where $(d/H_o' < 1)$.

(b) Deepwater wave height is not important to predictions of nearshore wave height where $d/H_o' < 1$, but it is more important in deeper water where $d/H_o' > 1$.

(c) Substantial errors in wave period produce underestimates or overestimates of wave height where $d/H_o' > 1$, but are less important when $d/H_o' < 1$.

(d) Refraction and directional spreading of wave energy are not important variables in shallow areas where $d/H_o' < 1$ because refraction and breaking effects tend to cancel one another; i.e., at a given depth, less refraction is associated with more wave breaking and more refraction means less breaking. Large errors in deepwater wave angle will have a pronounced effect where $d/H_o' > 1$ or for large wave angles.

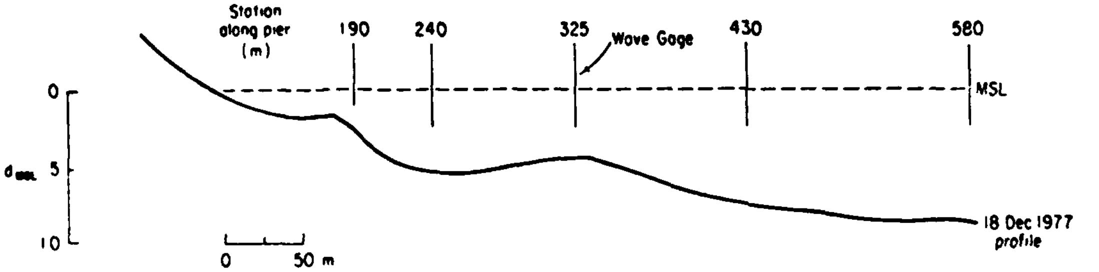
(e) In shallow water where $d/H_o' < 1$, much of the information about deepwater conditions is lost due to wave breaking and refraction effects.

V. ANALYSIS OF NEARSHORE WAVE HEIGHT CHANGES AT THE CERC FIELD RESEARCH FACILITY

The usefulness of the nearshore wave prediction methods presented in Sections II and III is illustrated by comparing observed and predicted nearshore wave heights measured at points along the 580-meter-long pier at the CERC Field Research Facility (FRF). Note that this is by no means a definitive test of the prediction methods for several reasons. First (as discussed in Section (IV), some input parameters are more important than others and in the case of the FRF data the accuracy of the input information is unknown. Since water depth was shown to be especially important in Section IV, profiles were taken on either side of the pier and tides were estimated hourly. The presence of bars or scour holes under the pier near the gages is not known. Second, the effects of reflection, wave direction, runup, and spray from the pier piles on the observed wave records at each of the gages are unknown. However, a comparison of observed and predicted wave heights measured at the pier is considered useful.

Since extreme conditions are generally of the most interest to designers, waves produced by two storms are examined. The first storm (18 to 23 December 1977) had a relatively long duration, produced a storm surge of approximately 0.3 meter, and had maximum significant wave heights of 3.5 meters at the end of pier. A second storm of shorter duration and less wave activity occurred on 13 September 1978 with maximum significant wave heights of approximately 2 meters off the end of the pier.

Methods developed in this report are illustrated by using the observed conditions from one gage to predict wave heights at other locations along the pier. For the December 1977 storm, the gage on the end of the pier at station 580 meters was used to predict significant wave heights at station 190 meters (Fig. 4), and observed and predicted waves were compared throughout the storm (other gages were not in operation). For the September 1978 storm, a wave gage 2,250 meters beyond the end of the pier was used to predict the significant wave height at points along the entire pier length. Observed and predicted wave conditions were then compared.



Survey Date	Water Depth, d_{MSL} (m) Station Along Pier (m)	190	240	325	430	580	Offshore Gage 2,250 meters off end of pier →
18 Dec 1977		1.8	–	–	–	7.9	
8 Sept 1978		0.9	4.6	5.1	6.6	8.8	

Figure 4. Profile at the Field Research Facility.

The beach slope is approximately 1/80 at the FRF, so a diagram of the nearshore significant wave height was prepared, similar to the curves in Appendix C (Fig. 5).

The observed tide, wave period of the highest one-third waves, T_s, and the significant wave height, H_s (defined as four times the standard deviation of the wave record) are given in Figure 6 for the December 1977 storm. Visual observations suggested that the wave angle at the end of the pier was small, so α_0 was taken as 0.0°. Since the waves were short-crested (Fig. 7), $S^* = 4$ was selected. A prestorm survey indicated that the water depth below mean sea level was 7.9 meters at the end of the pier and 1.8 meters at station 190 where predictions of significant wave height were made using Figures 5 and A-3 with the observed conditions at the end of the pier. Observed (solid line) and predicted significant wave heights (symbols) show agreement, especially during the highest wave conditions (Fig. 6). Tidal variation in water levels is associated with wave height changes and this is clearly shown in observed and predicted wave heights (not shown in observations at station 190 during the end of the storm due to sparse observations).

For the 13 September 1978 storm, a wave gage 2,250 meters beyond the end of the pier was used to record offshore wave height and period. Radar images were used to estimate dominant wave direction. Predicted tides and a profile survey of 8 September 1978 were used to determine water depth along the pier. Figure 8 shows nearshore wave conditions.

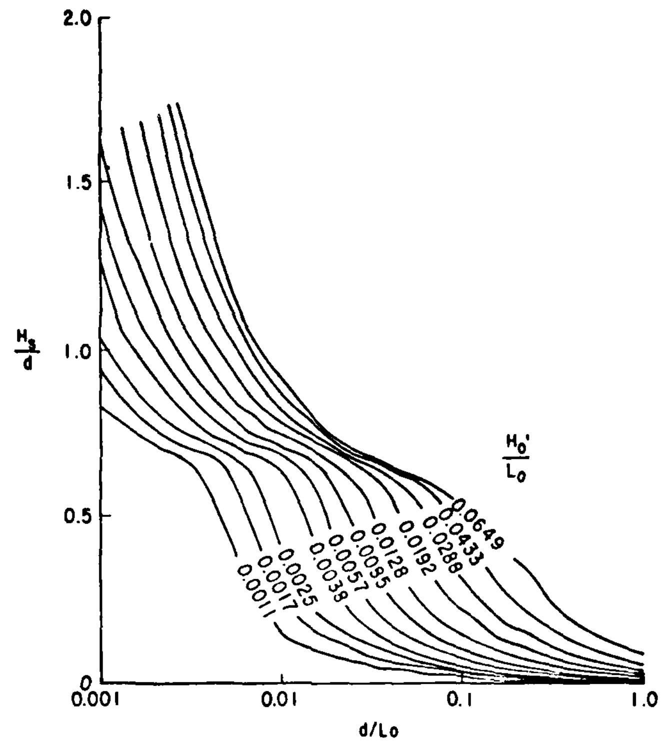


Figure 5. Nearshore significant wave height diagram for the FRF.

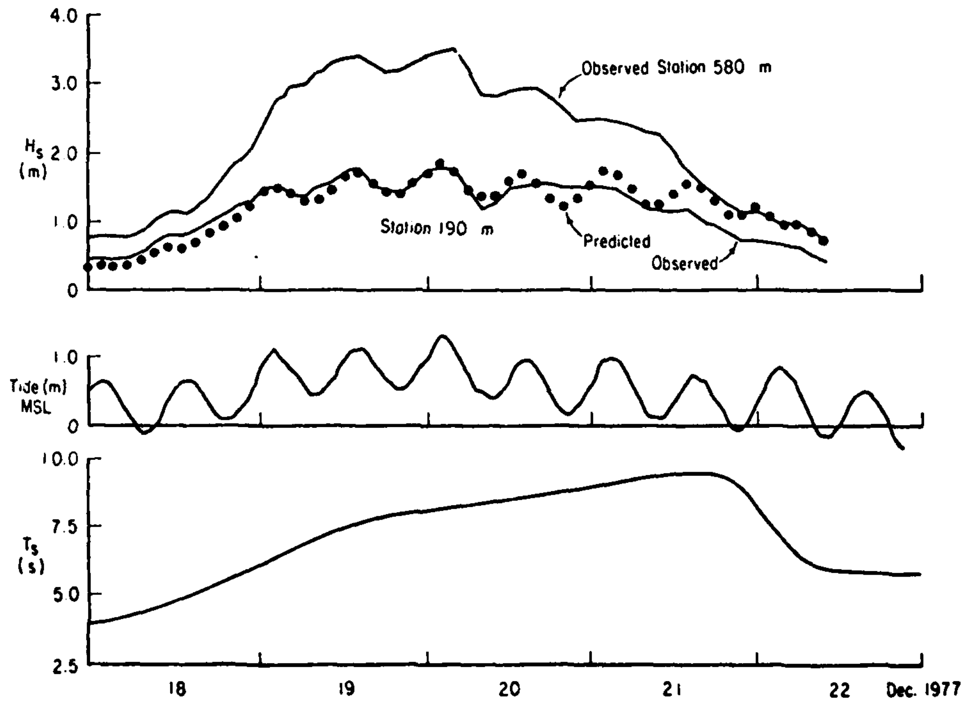


Figure 6. Observed and predicted wave conditions at the FRF 18 to 22 December 1977.

Figure 7a. Waves off the end of the FRF pier (19 December 1977).

Figure 7b. Waves along the FRF pier (19 December 1977).

Figure 7c. Waves near the shoreline at the FRF (19 December 1977).

Figure 8. Waves at the FRF on 13 September 1978.
Note waves breaking on the piles.

The recommended value of $S^* = 4$ and Figures 5 and A-3 were used to predict nearshore wave heights. Predicted and observed wave heights are similar for most of the profile (Fig. 9), with the observed wave height in the shallowest water higher than predicted.

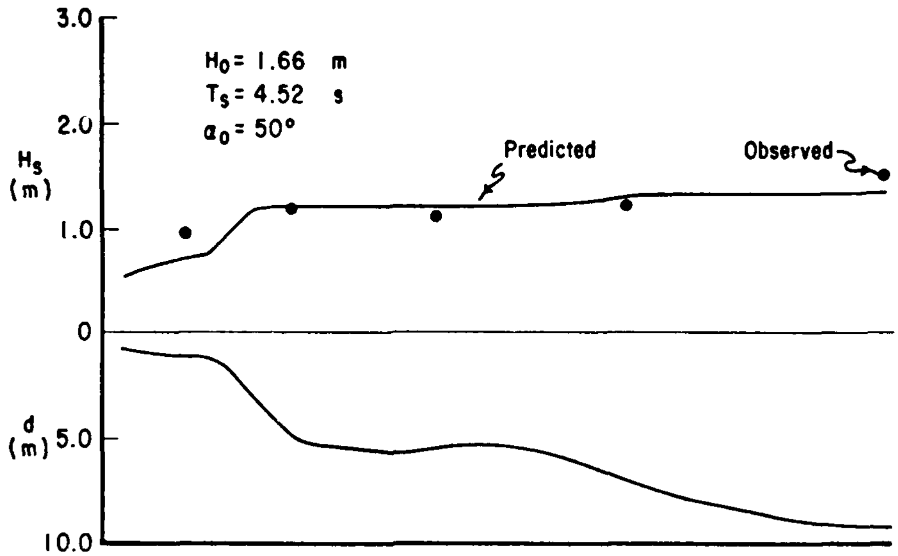


Figure 9. Observed and predicted wave height at the FRF, 13 September 1978.

VI. EXAMPLE PROBLEM (USING NEARSHORE WAVE HEIGHT PREDICTION TECHNIQUES)

GIVEN: A plane bottom slope of 1 on 100 and the input deepwater wave conditions in Figure 10.

FIND: The nearshore significant wave height for water depths between 0.5 and 6.0 meters for waves coming from each of the three deepwater wave directions.

SOLUTION: Nearshore conditions are estimated assuming $S^* = 4$ and the deepwater wave directions (directions 1, 2, and 3) have absolute wave angles of 60°, 0°, and 60° with respect to the shoreline. Table 4 summarizes nearshore wave predictions for waves from deepwater direction 3; Figure 11 gives wave predictions from all directions.

CONCLUSIONS: Results show that the local significant wave height is primarily controlled by depth-limited breaking in 3.0 meters of water or less. In this example, wave direction, refraction effects, and wave period are relatively unimportant input parameters. The lack of influence of these parameters can be seen by examining the predicted nearshore wave heights for directions 1 and 2. The deepwater wave directions differ by 60°, but the deepwater angle has almost no effect on the predicted nearshore wave height in water depths of 6.0 meters or less.

The above example suggests that knowing the total water depth at the point of interest is crucial when designing for extreme wave conditions.

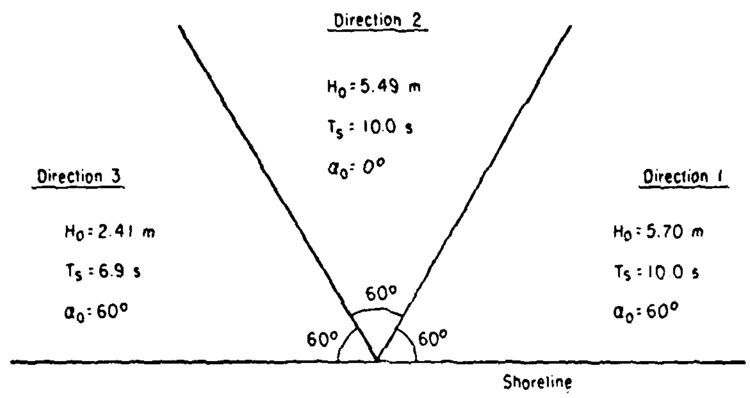


Figure 10. Deepwater wave conditions.

Table 4. Predicted nearshore significant wave height (direction 3).[1]

d_{swl} (m)	$\frac{d}{gT_s^2}$	K_R	H_o'	$\frac{H_o'}{L_o}$	$\frac{d_{swl}}{H_o'}$	H_s (m)
6.0	0.013	0.72	1.74	0.023	3.46	1.72
4.0	0.0086	0.71	1.70	0.023	2.33	1.75
3.0	0.0064	0.70	1.69	0.023	1.76	1.69
2.0	0.0043	0.70	1.68	0.023	1.17	1.28
1.0	0.0021	0.69	1.67	0.022	0.58	0.20
0.5	0.0011	0.69	1.66	0.022	0.30	0.43

[1] H_o = 2.41 meters, T_s = 6.90 seconds, α_o = 60°, S^* = 4.

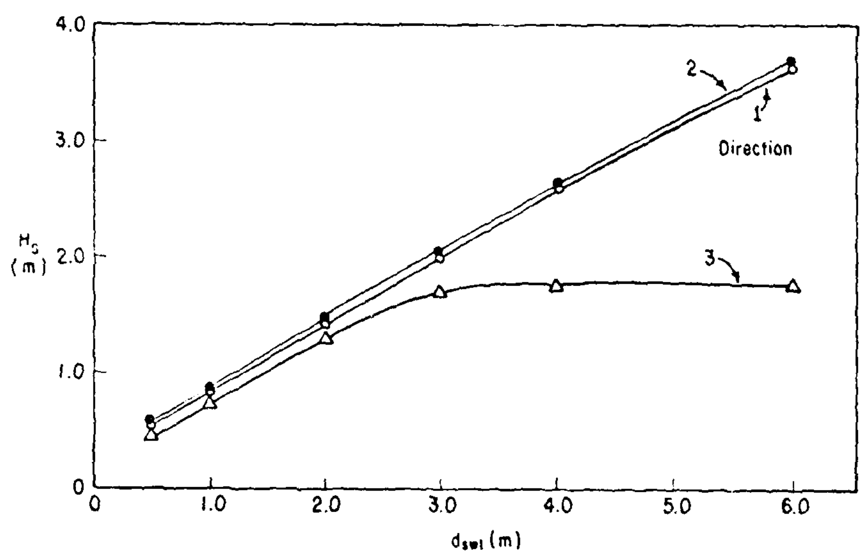


Figure 11. Predicted nearshore wave height.

VII. SUMMARY AND CONCLUSIONS

Methods developed by Goda (1975a, 1975b) and suggested by Goda, Takayama, and Suzuki (1978) for predicting nearshore irregular wave conditions for the case of continuously shallowing bottom contours have been presented in forms convenient for designers. A sensitivity analysis of the methods shows the relative importance of the input parameters on the predicted nearshore significant wave height. Comparison with observed wave height changes at the CERC FRF pier indicates that the methods may be suitable for preliminary design, although the influence of the pier piles and variable hydrography caused by the pier on wave activity has not been determined. Examples are presented and design curves are given in Appendixes A, B, and C.

The design curves in Appendixes B and C were produced by assuming that the deepwater wave heights have a Raleigh distribution. In a design situation where the deepwater waves are known to be non-Raleigh, which may occur with multipeaked spectra, methods described in this report should not be used.

The analytical model also assumes that the beach has continuously shallowing bottom contours; the effect of offshore bars on nearshore wave height is unknown. As a first approximation for barred coasts the wave height shoreward of the bar should be taken as equal to the predicted height at the bar crest location (Y. Goda, personal communication, 1979). Where the water depth becomes less than the depth at the bar crest, the curves can once again be used to predict wave heights shoreward of the bar.

The wave predictions for structural design in very shallow water (approximately $d/H_o' < 0.1$) should be used with caution. In very shallow water the wave begins to behave like a runup bore with a large forward momentum, and for this case the wave height alone may not provide enough information for design of a structure.

When this report was prepared, little wave data were available; therefore, only a few observations are presented in Section V. However, large amounts of wave data are now becoming available from CERC's Field Research Facility at Duck, North Carolina, and CERC is in the process of making a more comprehensive evaluation of methods described in this report. Preliminary results suggest that on the average, predictions are conservative for $d/H_s > 1.5$ while the predicted significant wave height may be underestimated for $d/H_s < 1.5$. Results of CERC's analysis will be published in a later report.

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APPENDIX A

WAVE REFRACTION DIAGRAMS

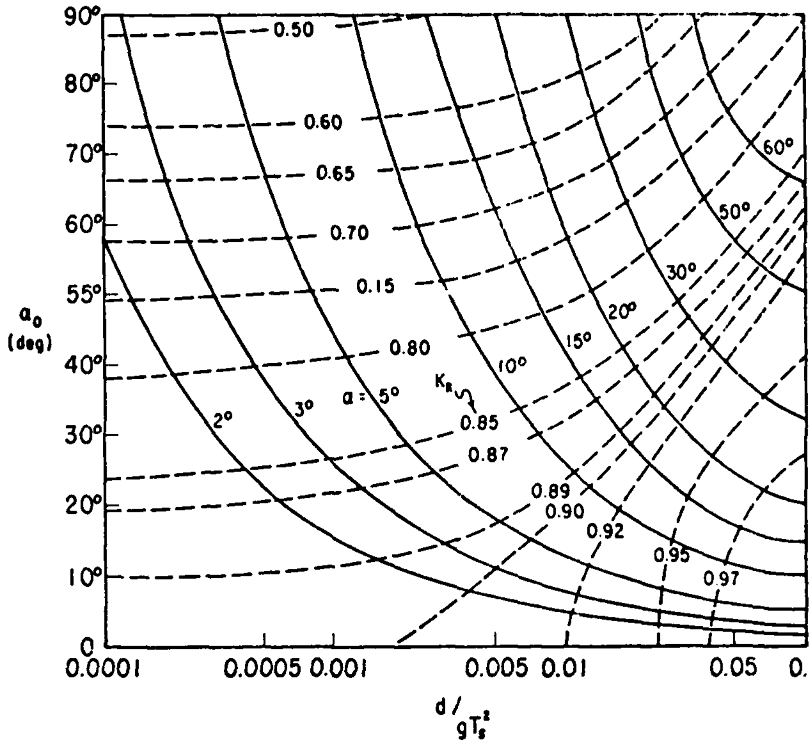


Figure A-1. Wave refraction for S* = 4.

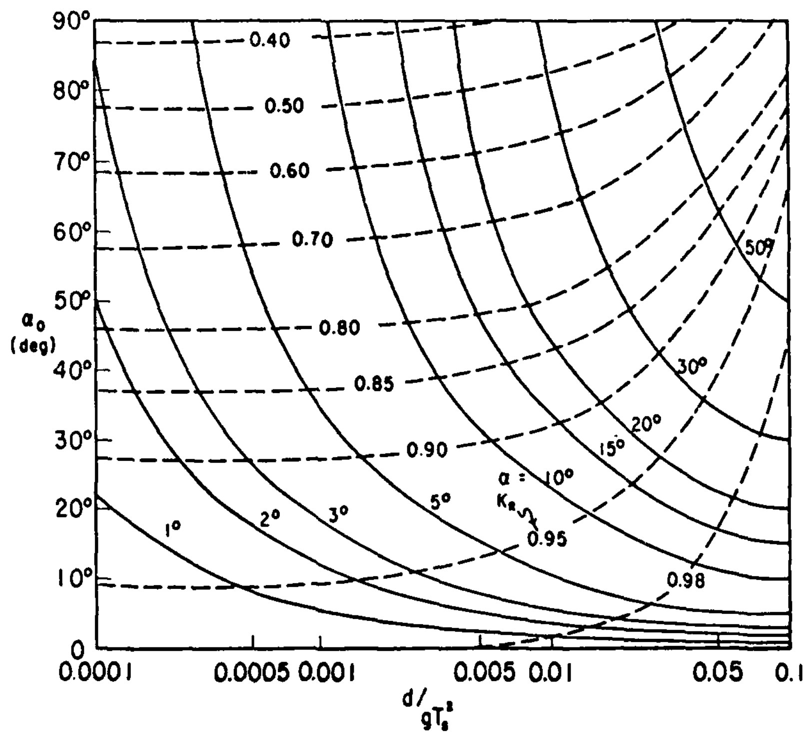


Figure A-2. Wave refraction for S* = 12.

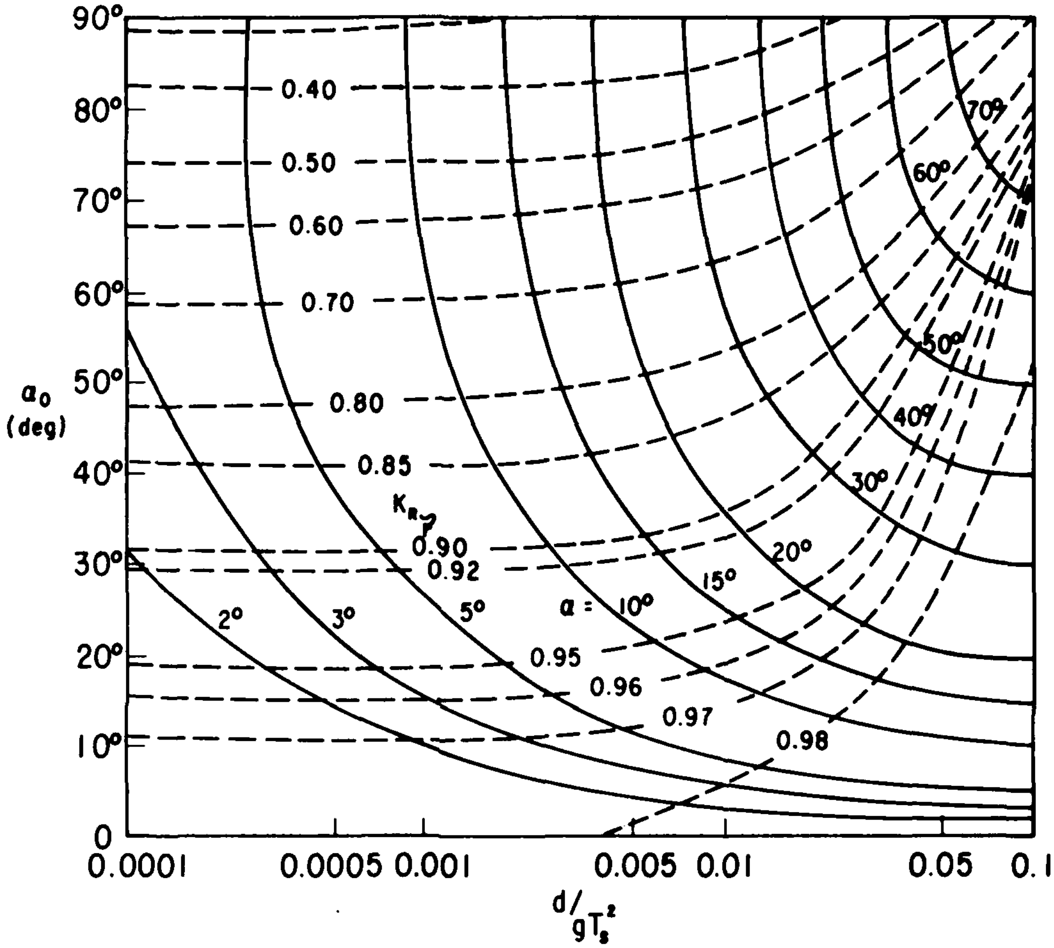
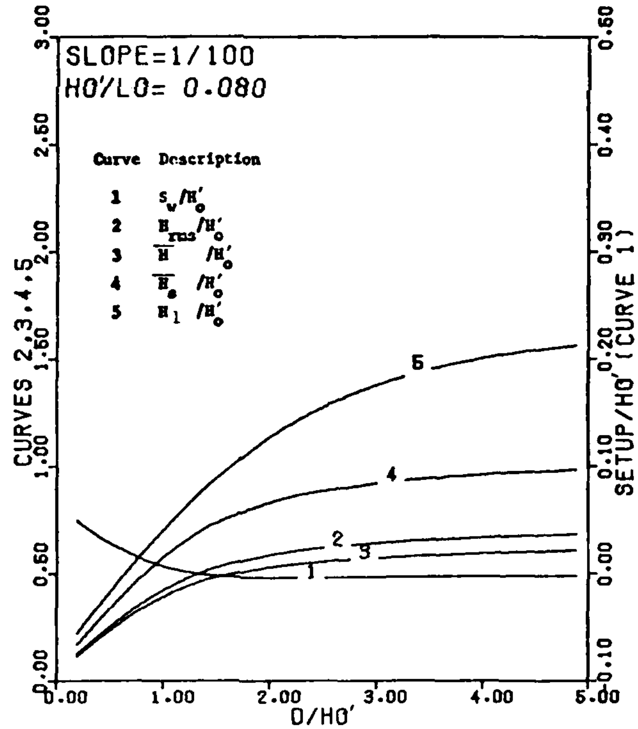


Figure A-3. Wave refraction for S* = 37.

APPENDIX B

NEARSHORE WAVE CONDITIONS AND SETUP DIAGRAMS



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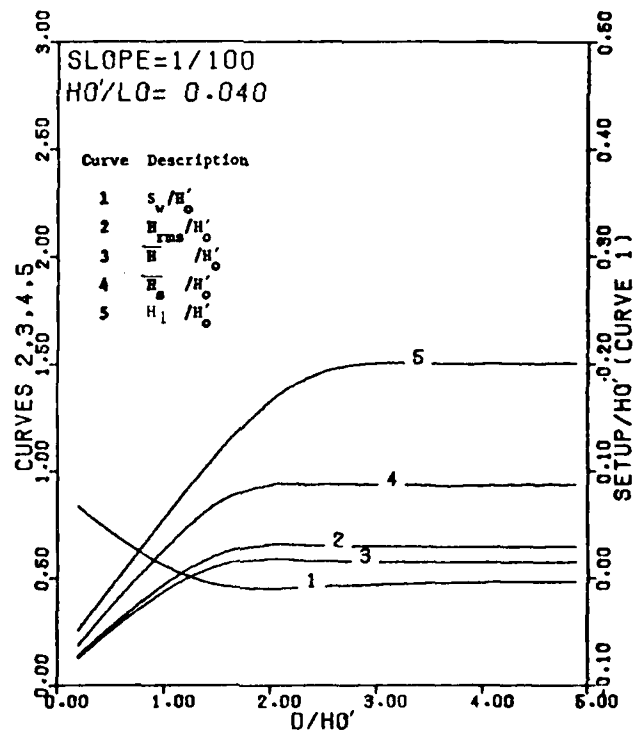

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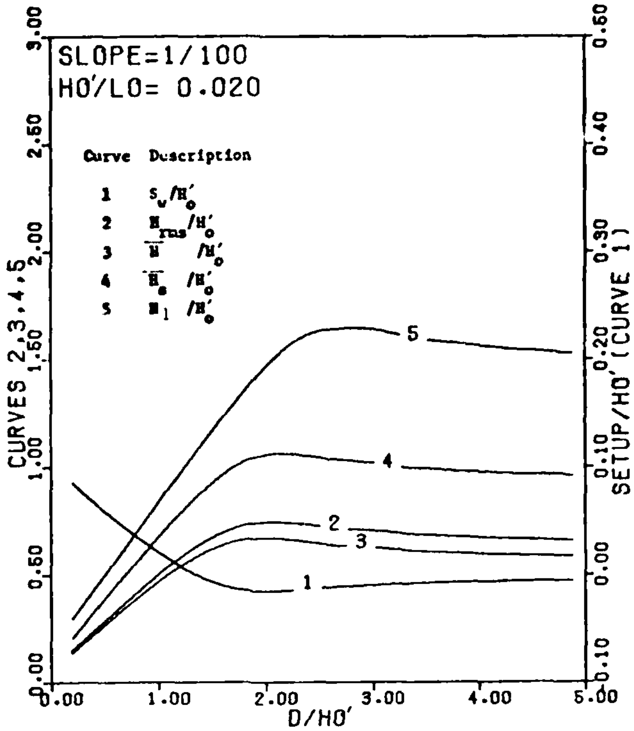

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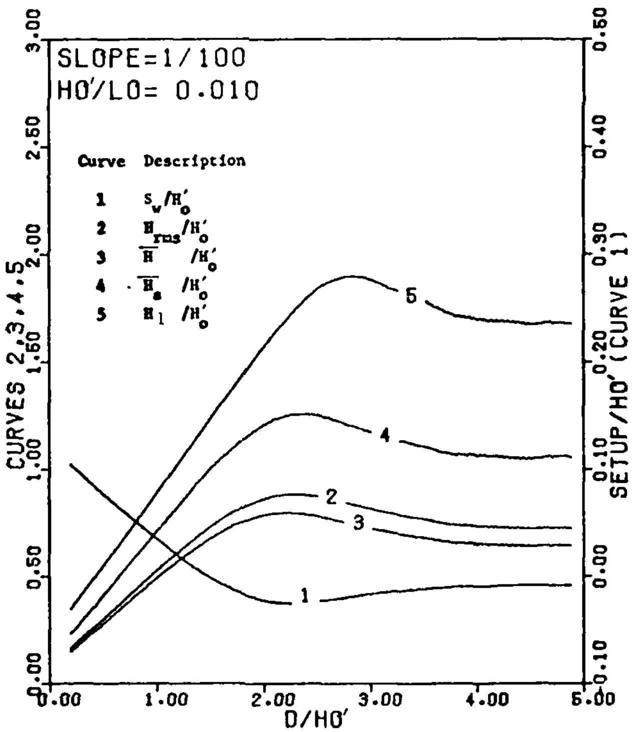

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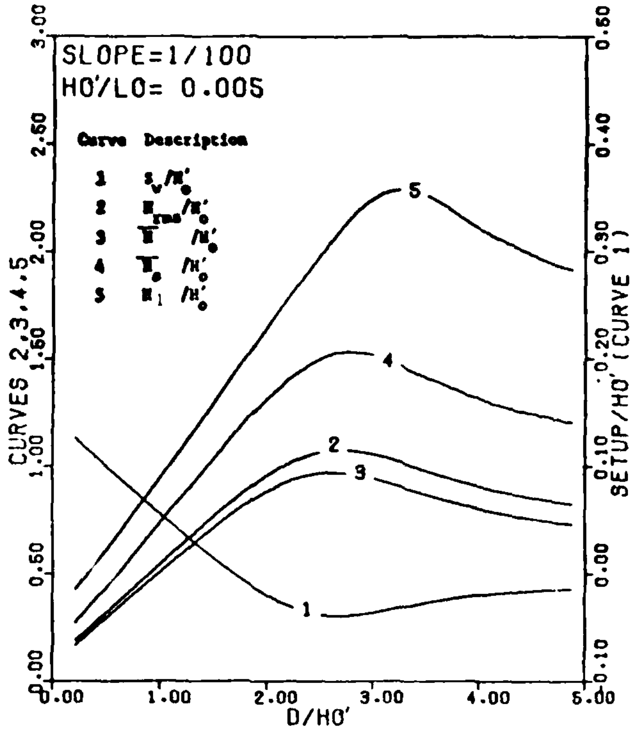

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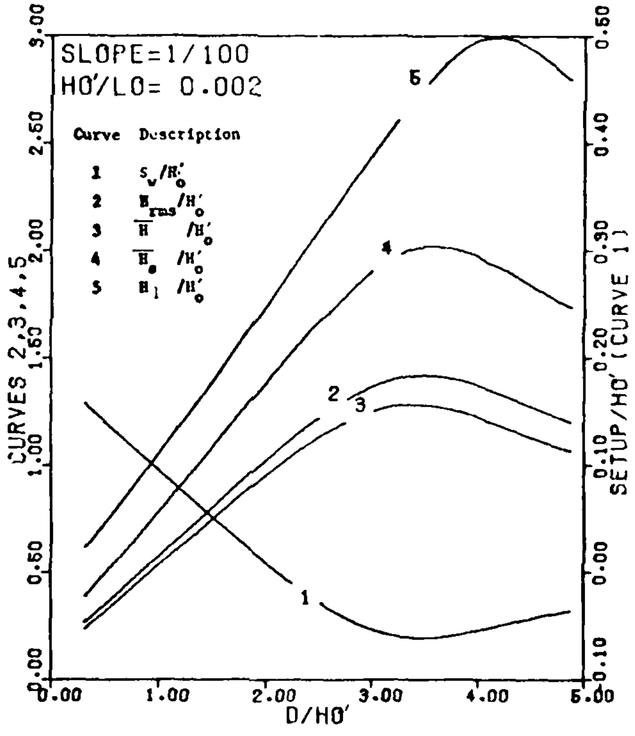

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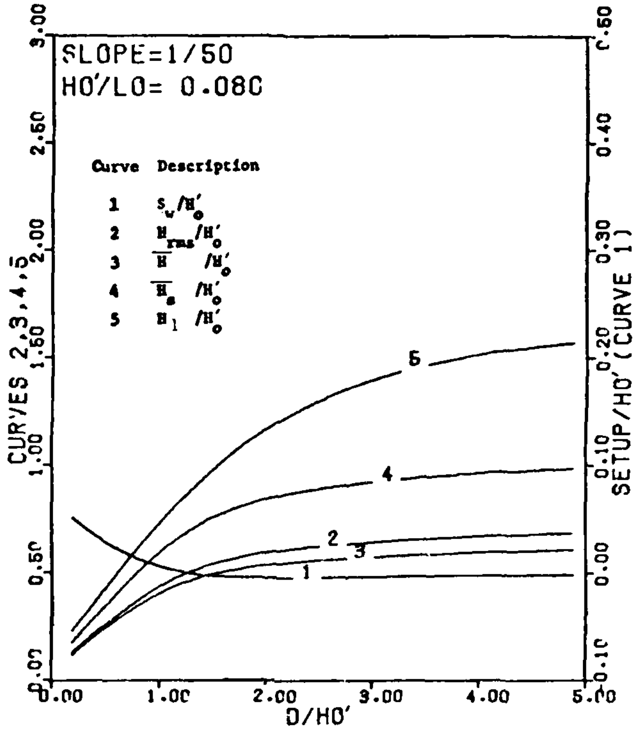

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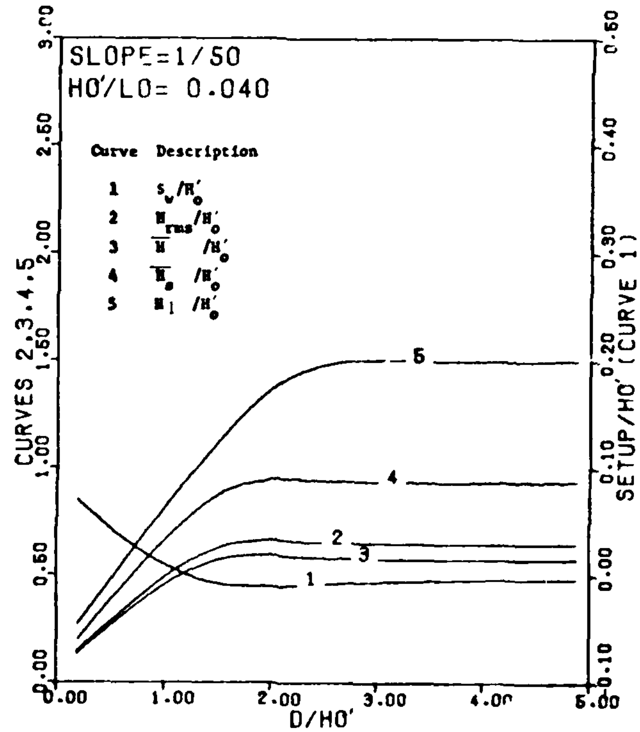

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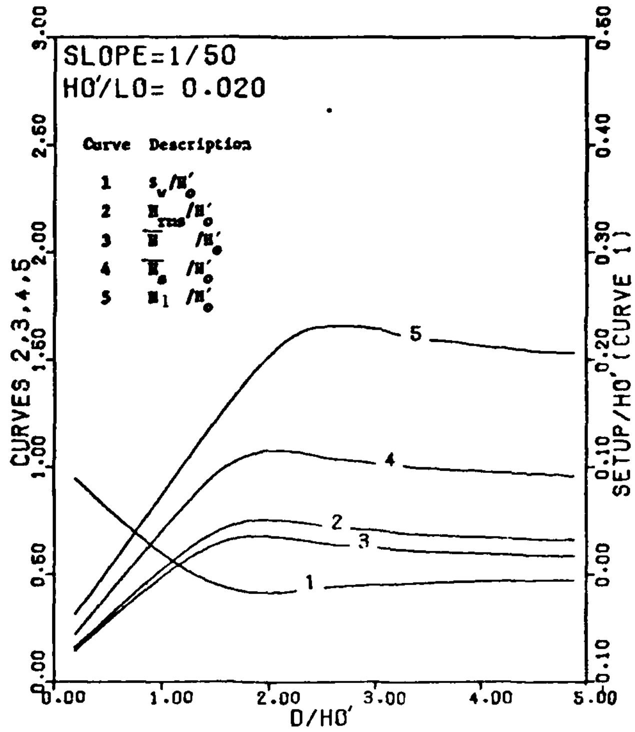

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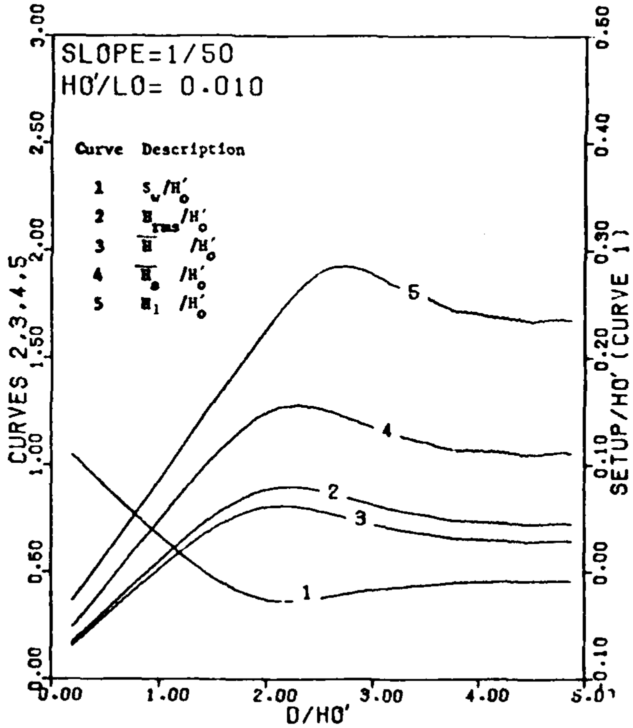

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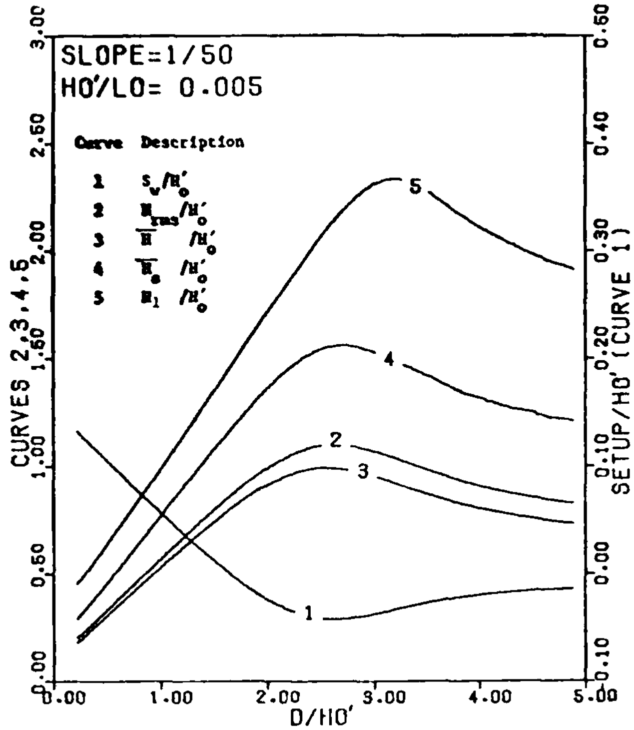

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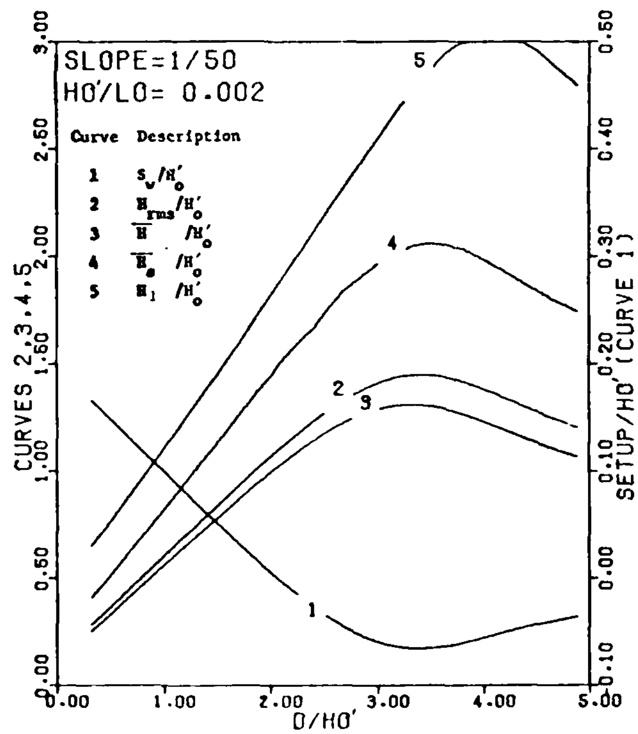

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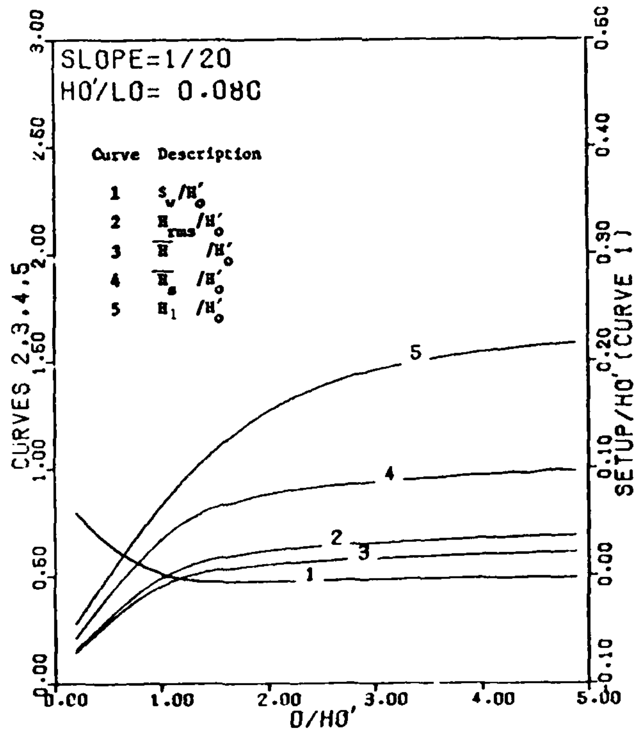

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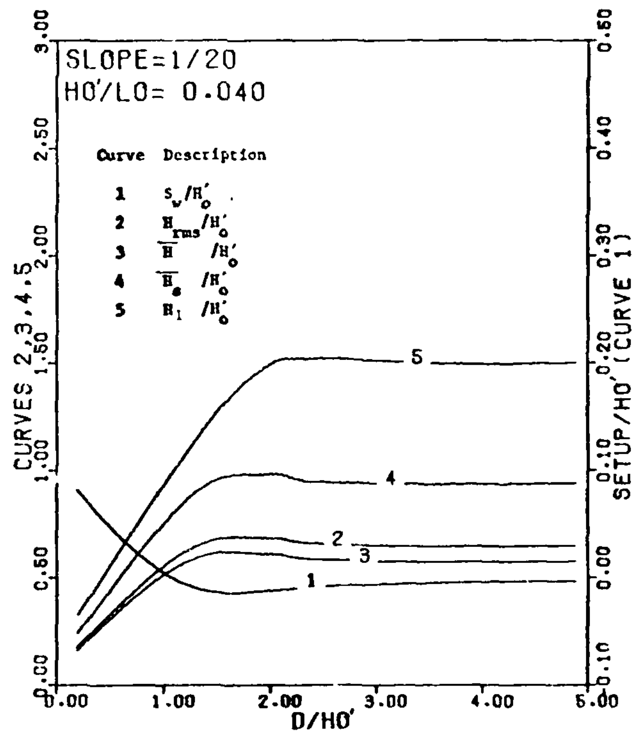

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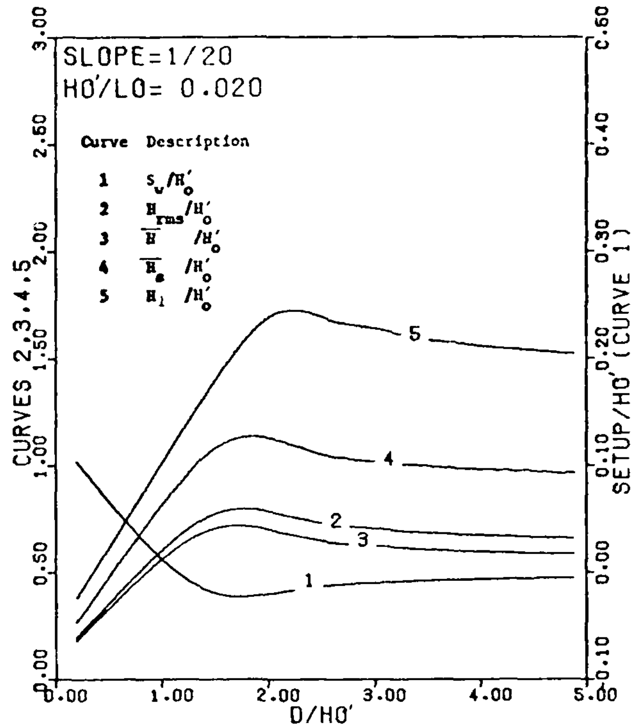

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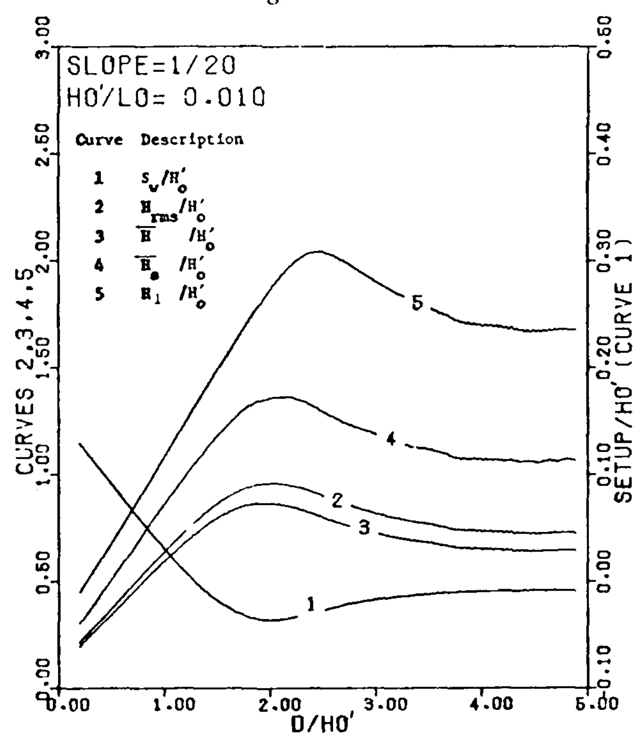

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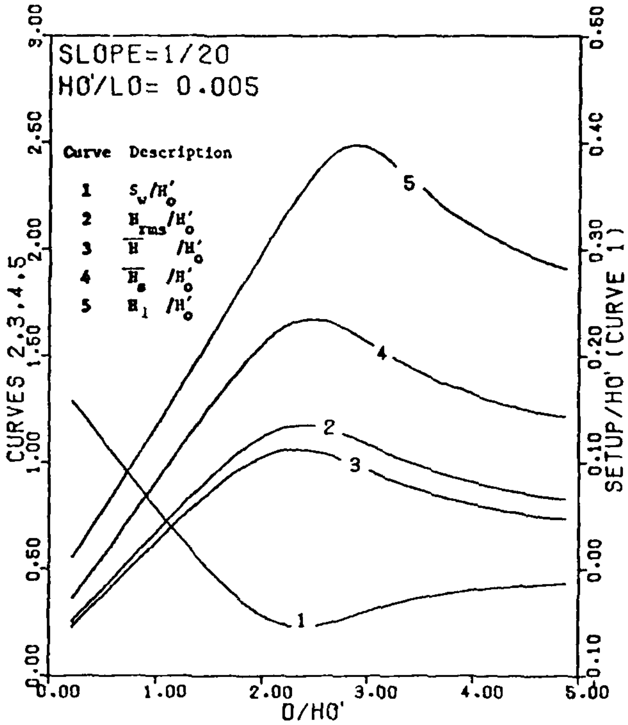

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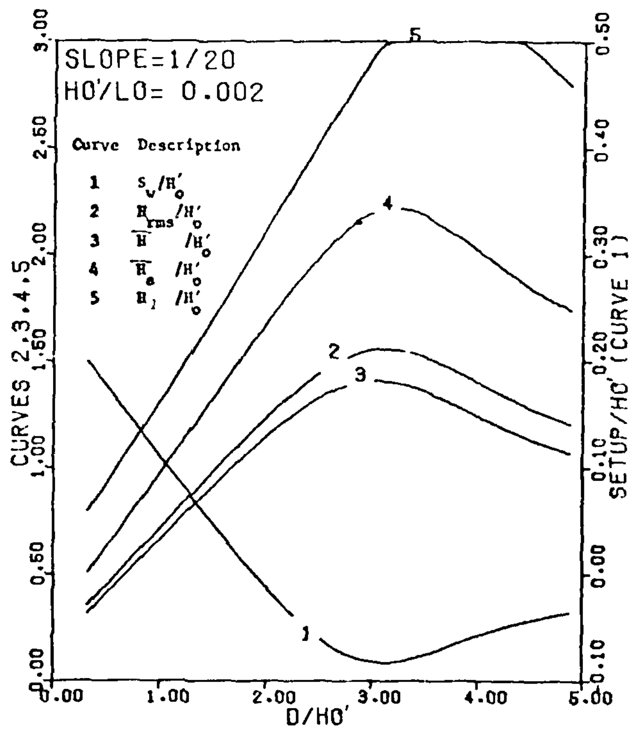

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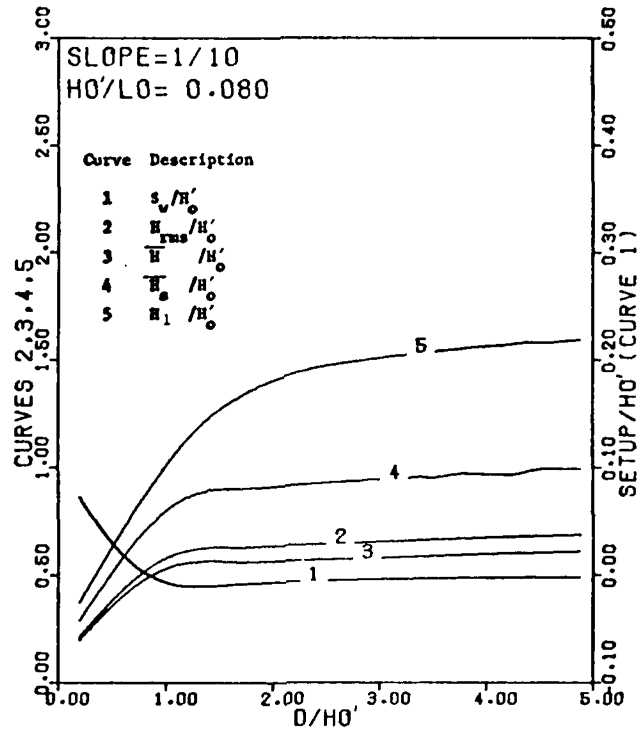

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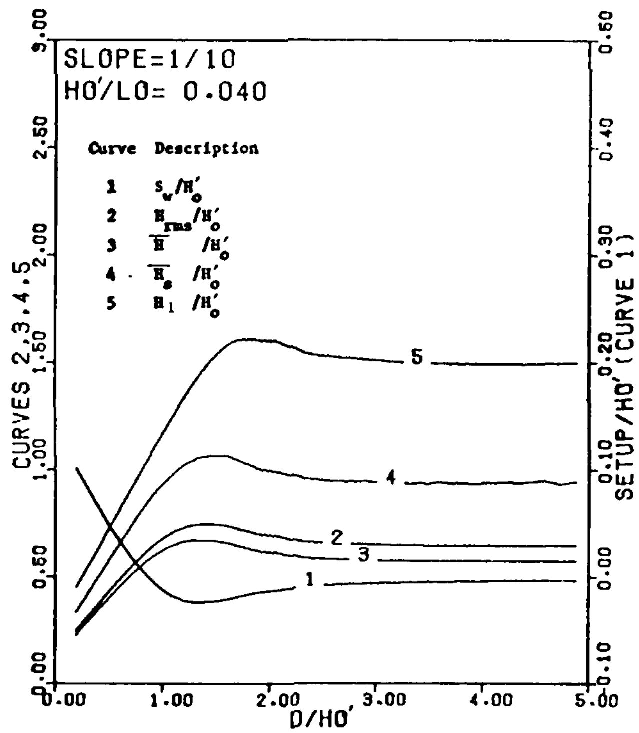

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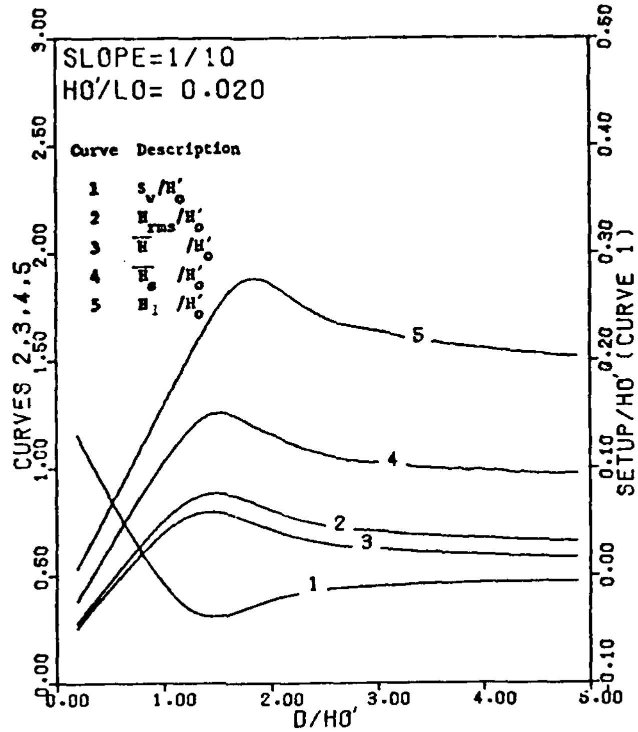

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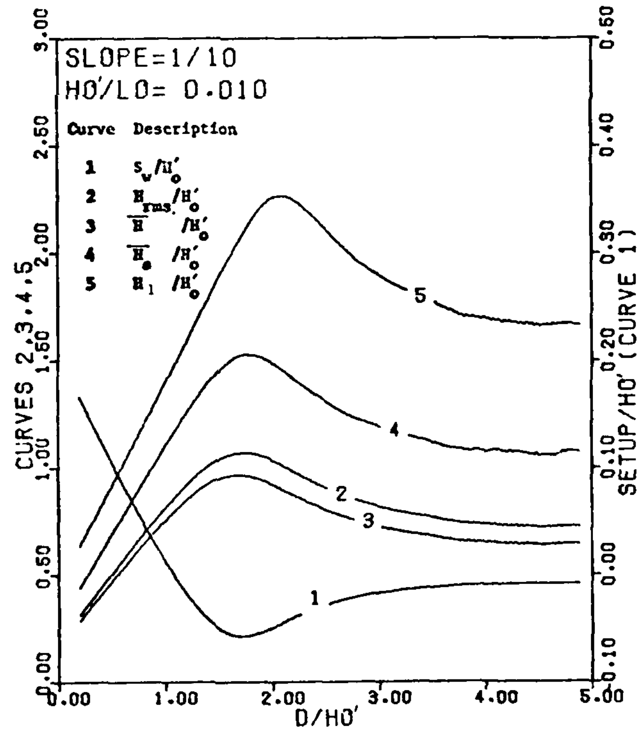

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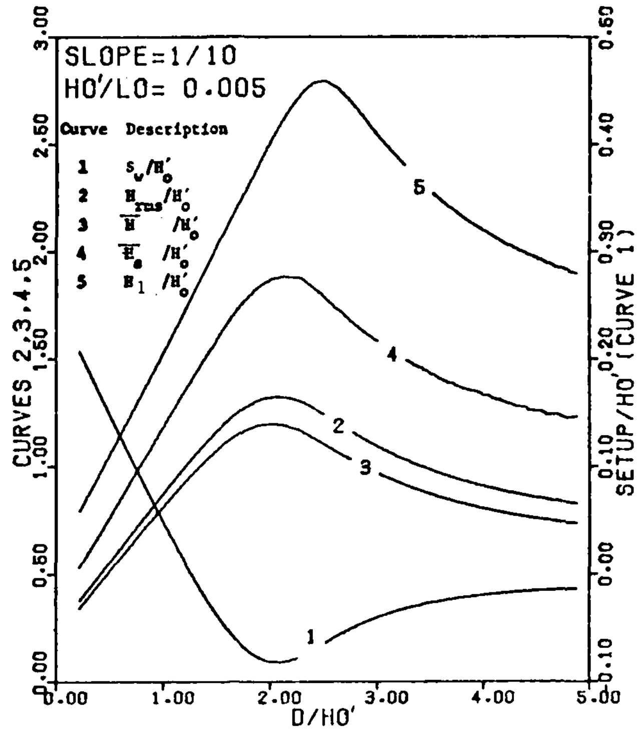

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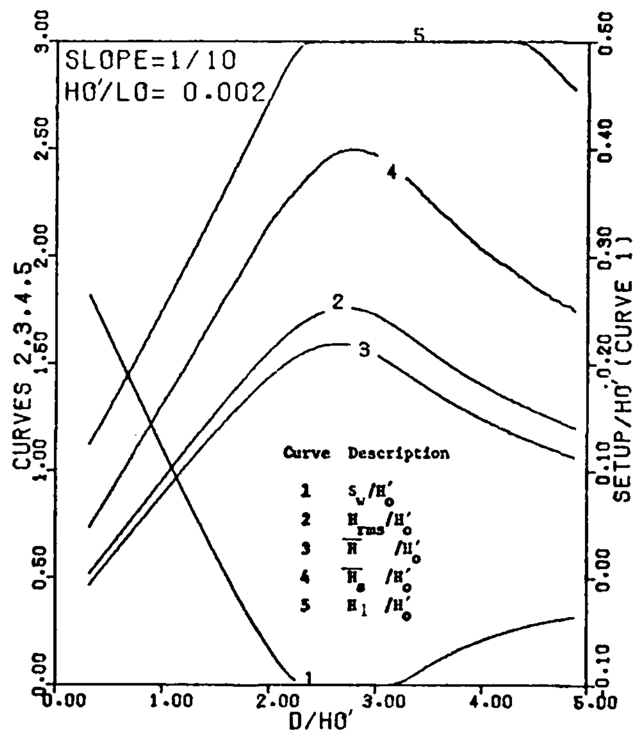

Figure B-24.

APPENDIX C

NEARSHORE SIGNIFICANT WAVE HEIGHT DIAGRAMS

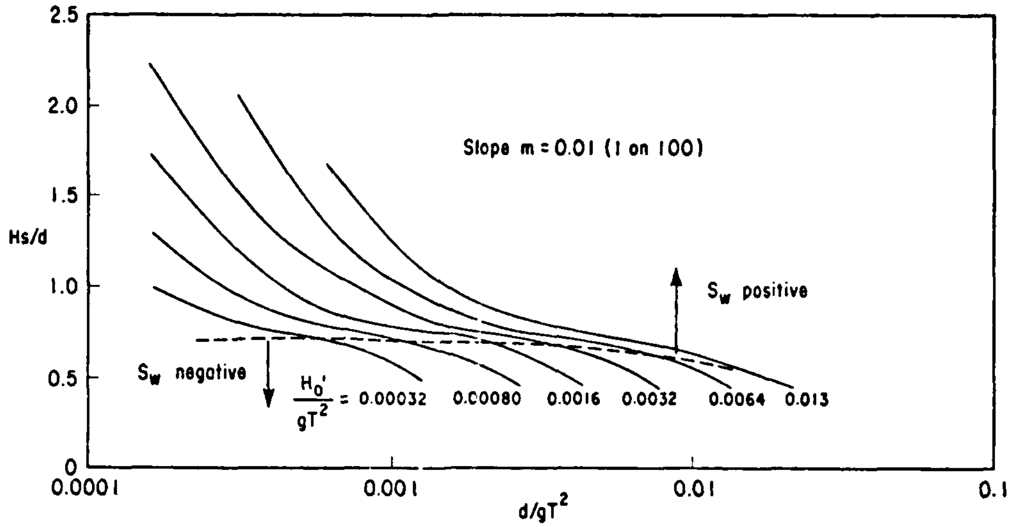

Figure C-1. Nearshore significant wave height diagram for a 1/100 beach slope.

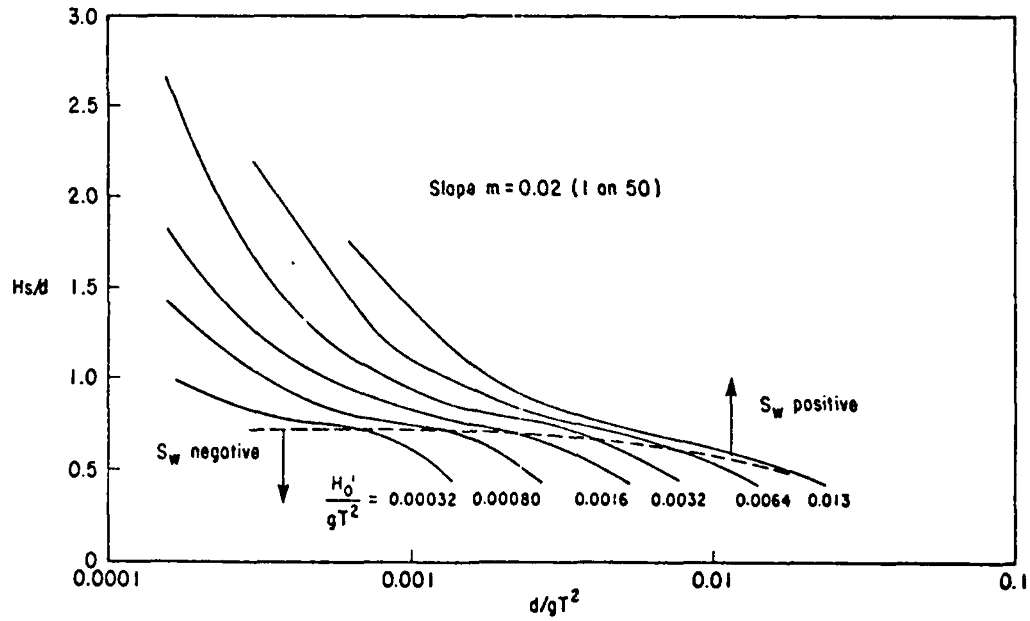

Figure C-2. Nearshore significant wave height diagram for a 1/50 beach slope.

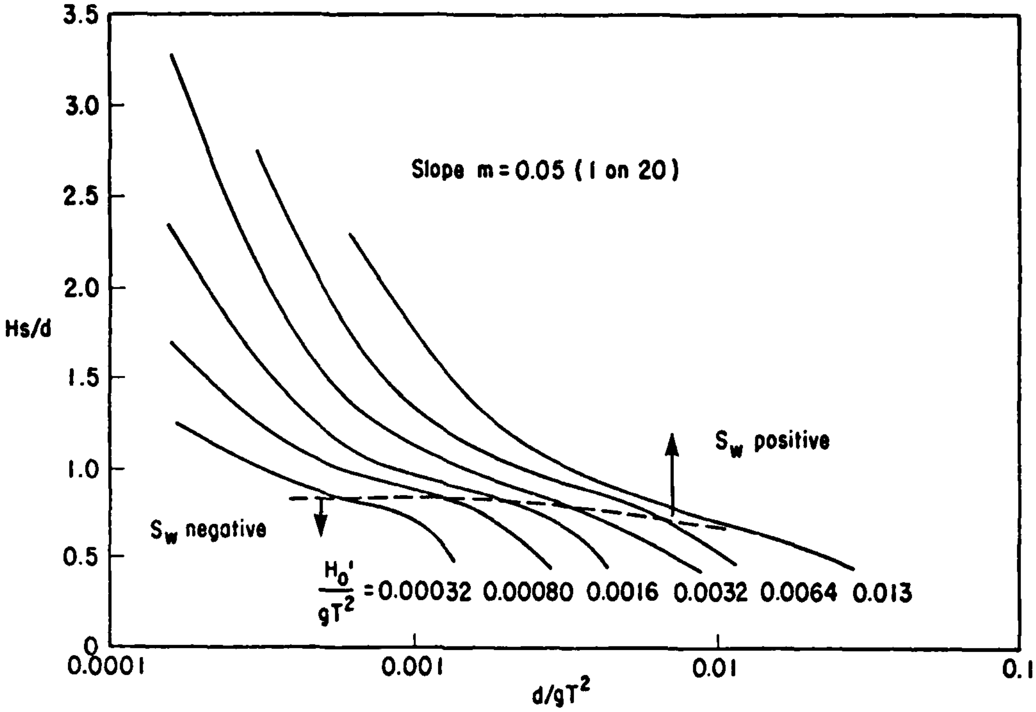

Figure C-3. Nearshore significant wave height diagram for a 1/20 beach slope.

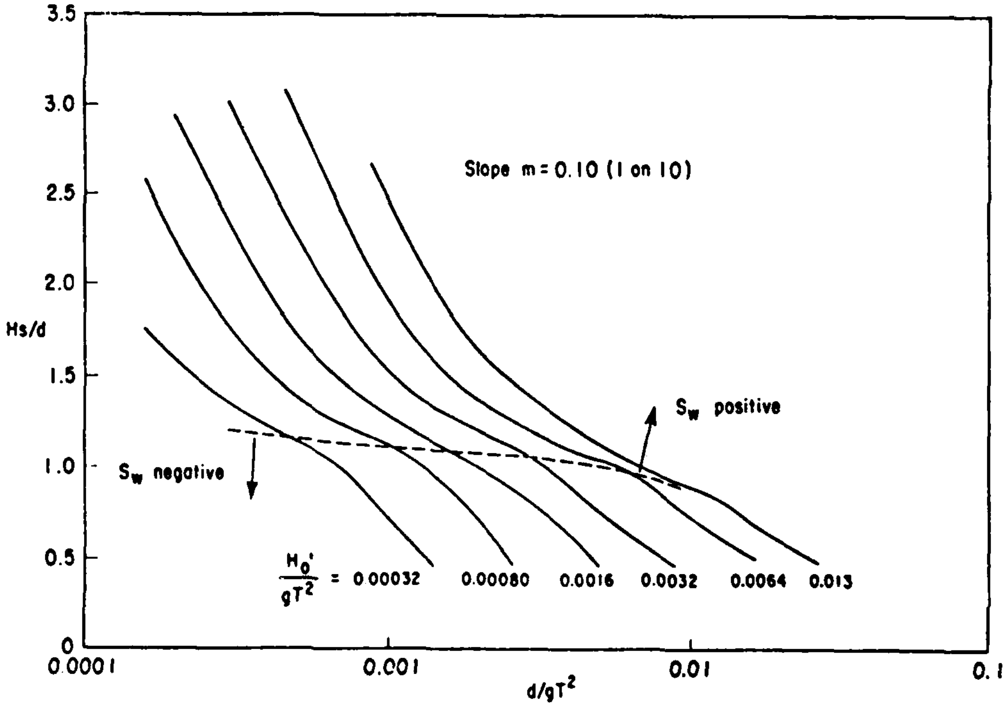

Figure C-4. Nearshore significant wave height diagram for a 1/10 beach slope.

Seelig, William N.
Estimating nearshore conditions for irregular waves / by William N. Seelig and John P. Ahrens – Fort Belvoir, Va. : U.S. Coastal Engineering Research Center ; Springfield, Va. : available from National Technical Information Service, 1980.
[47] p. ill. ; 27 cm. – (Technical paper – U.S. Coastal Engineering Research Center ; no. 80-3)
Methods for estimating nearshore irregular wave conditions for continuously shallowing bottom contours, given the bottom slope and offshore wave characteristics, are presented. A sensitivity analysis is performed to show the influence of various input parameters on predicted nearshore significant wave height. The methods are applied to the nearshore region at CERC's FRF at Duck, N.C., and results are compared to observed nearshore wave height changes measured at the facility.
1. Wave refraction. 2. Wave height. 3. Waves. I. Title. II. Ahrens, John P. III. U.S. Coastal Engineering Research Center. Field Research Facility. IV. Series: U.S. Coastal Engineering Research Research Center. Technical paper no. 80-3.
TC203 .U581tp no. 80-3 627

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Estimating nearshore conditions for irregular waves / by William N. Seelig and John P. Ahrens – Fort Belvoir, Va. : U.S. Coastal Engineering Research Center ; Springfield, Va. : available from National Technical Information Service, 1980.
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