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Project Title: Enhancements to a Beach Profile Change Model

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Contract No: DAJA45-91-C-0005

Final Report

July 1992

The research reported in this document has been made possible through the support and sponsorship of the U.S. Government through its European Research Office of the U.S. Army. ~~This report is intended only for the internal management use of the Contractor and the U.S. Government.~~

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PREFACE

This study was conducted jointly at the Department of Water Resources Engineering (DWRE), Institute of Technology, University of Lund, Lund, Sweden, and the Coastal Engineering Research Center (CERC), US Army Engineer Waterways Experiment Station, Vicksburg, MS, USA. The work described herein was performed over the period 4 January 1991 through 1 July 1992 by Dr. Magnus Larson, Assistant Professor, DWRE, in close cooperation with researchers at CERC.

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ENHANCEMENTS TO A BEACH PROFILE CHANGE MODEL

PART I: INTRODUCTION

Background and Problem Statement

1. The numerical storm erosion and beach profile change model SBEACH (acronym for Storm-induced BEAch CHange) was developed as a cooperational effort between the Department of Water Resources Engineering (DWRE), University of Lund (UL), Lund, Sweden, and the Coastal Engineering Research Center (CERC), U.S. Army Engineer Waterways Experiment Station (WES), Vicksburg, MS, (Larson 1988, Larson and Kraus 1989, Larson et al. 1990). This model is currently used at CERC in design of protective beach dunes and other applications for shore protection. SBEACH represents numerous physical processes including; varying wave conditions (height and period) and water level, irregular initial profile shape, bar formation and movement, setup and run up, and average grain size. Several field and laboratory comparisons have shown that the model gives reliable order of magnitude estimates of beach and dune erosion, with some capability to model beach accretion during the recovery stage after storms.

2. Although the model is considered to be the most advanced of its kind, the description of nearshore physical processes is still open for improvement. SBEACH was developed with a program structure consisting of separate calculation modules, thus, allowing parts of the model to be replaced as refined modeling techniques are derived for the different physical processes. Examples of areas where such modifications may enhance modeling results are:

* randomization of wave input to the model
* modeling of energy dissipation in the surf zone
* description of the landward boundary condition
* characterization of grain-size variation across a profile
* improvement of the numerical solution scheme including a variable grid
* description of beach profile accretion

This report primarily describes modifications introduced in SBEACH to improve the physical description with respect to the above areas.

3. In previous versions of SBEACH the wave input to the model consists of statistical measures characterizing the wave height, such as the significant or mean wave height, that are transformed across-shore as regular wave. Even though the effect of irregular waves on a beach should be correlated to a statistical wave height that is being transformed as a regular wave, the relationship is not clear and the choice of representative wave height is open to debate. The preferred solution would be to assume a certain wave height distribution in deep water, for example a Rayleigh distribution, transform individual waves randomly picked from this distribution across shore, and obtain an ensemble-averaged energy dissipation from which the transport rate could be calculated. Such a calculation technique is, however, extremely time consuming from a computational point of view, since a large number of waves must be transformed at every time step.

4. As pointed out in several papers on nearshore hydrodynamics (Roelvink and Stive 1989, Nairn et al. 1990), the effect of wave breaking on currents and water levels in the surf zone often displays a lag in time and space. For example, wave setup in the surf zone is typically shifted shoreward in comparison to commonly employed calculation models, and the same phenomenon occurs for the longshore current. The most likely explanation for this discrepancy between calculations and measurements is that the decay in organized wave motion is not instantaneously dissipated to heat, but some transport of turbulent momentum takes place in the upper part of the water column that introduces an onshore spatial shift in the forcing of nearshore currents and water levels (Svendsen 1984). An improved modeling of the energy dissipation in the surf zone, taking into account the transport of turbulent momentum, has considerable implications for sediment transport and beach change.

5. Several types of landward boundary conditions are encountered in practical applications for shore protection. These landward boundary conditions may be classified as (1) large, high beach and dune complex that is not penetrated by water, (2) small dune and beach complex that may be inundated and washed over, (3) large strong structures that resist erosion and inundation, and (4) small structures that may fail under storms of certain properties, such as large waves, scour at the structure, and combinations of such factors. Previous versions of SBEACH could only handle a beach where no overwash or inundation occurred or a simple seawall without structure failure.

6. In some previous model applications (Larson et al. 1990, Larson 1991), the variability in grain size along the profile has produced difficulties when trying to fit a traditional equilibrium profile with a 2/3-power (Dean 1977) to the measured profile data. The grain size close to the

shoreline is typically coarser than the material further offshore, implying a steeper profile in the nearshore and a flatter profile in the offshore. An equilibrium profile that allows a varying grain size across shore may give a more realistic profile response since the net transport rate in SBEACH to a large extent depends on how far from equilibrium the profile is locally.

7. In time-dependent numerical models such as SBEACH, where the wave transformation, transport rate, and beach change have to be solved for at every time step, any simplification in the numerical solution scheme could imply large amount of computational time saved. Thus, introducing a variable grid, where a coarse grid is used in regions of the profile where changes in wave and beach characteristics are slow and a fine grid where changes are rapid, could significantly reduce the calculation time.

8. In long-term simulations of beach profile response extensive periods occur when low-energy wave conditions prevail. During these periods, material is moving onshore to accrete on the foreshore and a berm is formed. Also, in the latter phase of a storm, when long-period waves of small height appear on the beach, strong accretion typically takes place and the beach recovers from the erosive damage caused by the storm. In both these cases, onshore sand transport has to be modeled, which is more difficult than determining profile response during erosion when the forcing conditions tend to be more uniform and a well-developed surf zone exists. In previous versions of SBEACH it was simply assumed that accretion is the reverse process of erosion, an assumption that could break down for certain combinations of waves and beach shapes.

Objectives

9. The main objectives of this report are to enhance and develop the numerical simulation model SBEACH to provide improved capabilities of:

(1) modeling irregular wave transformation in the surf zone
(2) modeling turbulent momentum transport and energy dissipation in the surf zone
(3) develop generalized landward boundary conditions that include inundation and overwash, and structures and structure failure
(4) incorporate grain-size dependence for the equilibrium profile shape
(5) efficient numerical solution schemes allowing for a variable grid
(6) modeling onshore sand transport and beach accretion

10. Much of the above enhancements were verified towards laboratory and/or field data. In the process of analyzing this data, a special program package was developed called BMAP (Beach Morphology Analysis Package) for determining geometric profile properties. This package will in the near future be distributed as a standalone software to be used for analysis of nearshore bottom topography data. Furthermore, in parallel with the enhancements of SBEACH a version of the code was developed in close cooperation with CERC that is adapted to the personal computer (PC) environment.

Procedure

11. The enhancements of the model were carefully tested against laboratory and field data to verify that the new methods produced improved description of the physical process in comparison with the original modeling approaches. The following paragraphs constitute a brief summary of the procedure used for developing and testing the various model improvements. All of the discussed work was done in close cooperation with researchers at CERC, and the programs developed at DWRE in connection with the model enhancements have been transferred to CERC.

Irregular wave transformation

12. A semi-analytic breaker decay model for irregular waves was developed from the model proposed by Dally et al. (1985). It was assumed that the properties of an irregular wave field in the nearshore zone could be obtained from studying the transformation of a set of individual waves. The wave model determines the variation in the root-mean-square (rms) wave height H_{rms} across shore from which the average energy dissipation may be derived (Larson 1992). Knowledge is required of the percentage of broken waves at each point along the profile and about the rms-value of the unbroken waves. For a profile where the depth increases monotonically with distance offshore, these two parameters are trivial to determine once the distribution function for the waves are known outside the surf zone (for example a Rayleigh distribution). In the case of a barred beach (non-monotonic depth increase with distance offshore), additional assumptions are required regarding wave reformation in order to determine H_{rms}. The applicability of the newly derived irregular wave model was verified through a Monte-Carlo simulation technique.

Modeling of energy dissipation

13. To improve the calculation of the energy dissipation in the surf zone, since the net transport rate largely depends on this parameter, a transport equation for turbulent kinetic energy

(tke) was introduced. By using this equation, a more realistic description of the wave energy dissipation is obtained, where it is no longer assumed that production and dissipation of tke occur at the same place. Thus, the net transport rate will be more accurately modeled and the distinction between the break and plunge point in SBEACH, used to heuristically take into account the spatial lag between production and dissipation of tke, is no longer needed. However, since adding a transport equation for tke considerably increases the execution times of the model, the relative merits of including this equation are still evaluated against the computational requirements. Improvements in model predictions when using a transport equation for tke was proved with field data on longshore current from Duck, NC, where the measured onshore spatial shift in the current distribution was satisfactorily reproduced (Smith et al. 1992).

Landward boundary conditions

14. Several generic types of landward boundary conditions were studied, and appropriate numerical descriptions of these boundary conditions were implemented in SBEACH. These boundary conditions included overwash and inundation, and seawalls with different modes of failure. To model overwash a simple geometric method was employed that produces qualitatively correct results. The investigated failure modes for a seawall involved specifying (1) a maximum allowable wave height in front of the structure, (2) the change in depth with respect to the initial profile, and (3) the absolute water depth immediately in front of the structure. In some applications it was necessary to implement a variable grid size on the foreshore to obtain a satisfactory spatial resolution in the description of the beach response in this region.

Grain-size dependence of the profile shape

15. An analytic solution for the equilibrium beach profile shape was developed that takes into account a varying grain size across shore (Larson 1991). The derivation was based on the concept presented by Dean (1977) assuming profile equilibrium when the energy dissipation per unit water volume is a constant, where the constant is a function of grain size that varies along the profile. Comparisons between the analytic model and profiles measured in the field with a marked variation in grain size showed that an improvement in the description of the profile shape was obtained when a variable grain size was included.

Modification of numerical solution scheme

16. The option of having a calculation grid with a variable length step was introduced in SBEACH. There are no restrictions on the choice of length step in the model, except with regard to numerical stability requirements, but an arbitrary grid cell spacing is allowed. A variable grid have two major advantages in comparison with a fixed grid, namely (1) a high

resolution in space is possible in areas that are of special interest such as the foreshore or where numerical calculations demand shorter length steps, and (2) in areas where calculation quantities are changing slowly a coarser grid may be used to cut down on execution times.

Modeling onshore sand transport

17. SBEACH was modified to describe onshore sand transport and beach accretion in a more realistic manner, avoiding the problem with excessive steepening of the profile at the foreshore for certain combinations of waves and beach properties. The local slope along the foreshore is allowed to approach a maximum foreshore slope, which is given as an input to the program. The modified description of beach accretion was qualitatively evaluated with field data.

Scope of Report

18. Part I of the report gives an introduction and summarizes the objectives of the study. A brief review of the profile response model SBEACH is presented in Part II including the wave decay model, transport rate calculations, and a discussion of previous model verification. In Part III is the improvement to the wave decay model discussed in detail regarding the randomization of the model and the addition of a transport equation for turbulent kinetic energy to enhance modeling of surf zone energy dissipation. Also, a small section is devoted to the response time of nearshore hydrodynamics to investigate the limits of a quasi-stationary approach for describing the nearshore wave dynamics in SBEACH. Part IV contains a description of the enhancements to SBEACH including a variable grid, grain-size dependent equilibrium profile, and modified description of onshore transport and beach accretion. Furthermore, several different types of landward boundary conditions have been studied such as inundation and overwash, and seawall failure. The two following chapters give an overview of the PC-version of SBEACH (Part V) and the analysis package BMAP (Part VI), which was developed as a tool for analyzing beach profiles in connection with the development of SBEACH. Finally, Part VII gives a summary of the report and possible future enhancements of SBEACH.

PART II: OVERVIEW OF THE BEACH PROFILE CHANGE MODEL SBEACH

19. This chapter gives a brief overview of the empirical foundation of the numerical model of beach profile change SBEACH, and much of the material is derived from Larson and Kraus (1989) and Larson et al. (1990). The wave decay model by Dally et al. (1985), which is used to calculate wave transformation in the surf zone, is first reviewed together with general characteristics of breaking waves that are of importance for beach profile response modeling. Cross-shore transport characteristics, derived from the large wave tank and field data, is discussed regarding direction and distribution. The different transport regions are introduced and motivated from a nearshore wave dynamics point of view, and the semi-empirical cross-shore transport rate equations are summarized. In conclusion, a brief review is given of the model verification of SBEACH against large wave tank and field data.

Wave Breaking and Wave Decay Modeling

20. As waves shoal while propagating onshore, the wave height and steepness increase until an incipient or initial breaking condition is reached (break point). At this point, the wave begins to dissipate energy through turbulence associated with the breaking process. In depth-limited breaking is the condition for incipient breaking determined by the ratio between wave height and water depth. Typically, this ratio is denoted by γ_b at the break point and referred to as the breaker depth index. Wave breaking is often the major cause for cross-shore sand transport and profile change, implying that knowledge of breaking wave characteristics are of primary importance for calculating profile response. Once the incipient breaking condition is reached waves may continue to break across the surf zone until they reform as unbroken waves, which could occur in the trough region on a barred profile leading to subsequent breaking closer to shore.

21. Regular waves tend to break at almost the same location and it is easy to estimate a representative break point. However, in the case of irregular waves in the field, the break point varies considerably for individual waves and a common fixed break point is impossible to identify. Thus, irregular waves will tend to produce a wider surf zone as compared to regular waves, assuming a similar onshore wave energy flux for the two wave conditions. In engineering models for predicting breaking wave properties to be used in cross-shore response calculations such as SBEACH it is common to treat statistical wave height measures, for example the

significant or root-mean-square wave height, as regular waves. This in spite of the fact that the effects on the profile predicted by the transformed statistical wave height will in general not be identical to what the irregular wave conditions would produce, even though some of the discrepancies could be corrected for in a calibration procedure against measured data.

22. The most simple criterion for incipient wave breaking may be expressed as,

$$H_b = \gamma_b d_b \tag{1}$$

where the subscript "b" denotes quantities taken at the break point and d is the total water depth. Galvin (1972) listed theoretically proposed values on the breaker depth index from various authors where γ_b ranged from 0.73 to 1.03, with an average value of 0.82. However, the most widely-accepted breaker depth index is $\gamma_b=0.78$ suggested by McCowan (1894), who obtained this value from solitary wave theory. Larson and Kraus (1989) obtained $\gamma_b=0.82$ as an average for data from seven laboratory and prototype experiments on plane-sloping beaches compiled by Smith and Kraus (1988). The beach slopes ranged between 1/90 and 1/10 in the experiments. In comparison, Larson and Kraus (1989) determined an average of $\gamma_b=1.0$ for prototype laboratory beaches where wave breaking occurred on the seaward side of a bar (seaward bar slopes between 1/50 and 1/5 with an average of 1/10). Thus, for milder slopes $\gamma_b=0.78$ seems to be an acceptable value for the breaker depth index, whereas for steeper slopes $\gamma_b=1.0$ is more appropriate.

23. Although there is in general a large scatter in measurements of the breaker depth index, a dependence on wave steepness and beach slope is typically noted (Galvin 1969, Weggel 1972, Singamsetti and Wind 1980). A steeper slope implies a larger γ_b and a higher wave steepness produces a smaller γ_b. These two variables may be merged together to form a surf similarity parameter (Battjes 1974), also known as the Irribarren number, expressed as,

$$\xi = \frac{\tan\beta}{\left(\frac{H}{L}\right)^{0.5}} \tag{2}$$

where β is a characteristic beach slope normally taken as the mean slope. Singamsetti and Wind (1980) reported an empirical expression obtained by Battjes for γ_b as a function of ξ_o (compare Sunamura 1980),

$$\gamma_b = 1.16\xi_o^{0.22} \tag{3}$$

where β is the mean beach slope and the subscript "o" refers to deepwater wave quantities. Larson and Kraus (1989) derived a very similar expression to Equation 3 using the aforementioned prototype laboratory data. However, in this case the beach slope was taken as an average for the seaward face of the bar. Predictive equations for γ_b similar to Equation 3 have been developed by Goda (1975), Ostendorf and Madsen (1979), and Izumiya and Isobe (1986). In SBEACH there is an option to set γ_b to a constant or to use a predictive relationship similar to Equation 3.

24. One important reason for the wide scatter in the breaker depth ratio is difficulties in defining the break point in an unambiguous manner, especially with respect to the different types of wave breaking that may occur. In general, four types of breaking waves are distinguished, namely spilling, plunging, collapsing, and surging (see Galvin 1972 for a detailed discussion), depending on beach and wave characteristics. The type of wave breaking could be significant for the cross-shore profile response, and many of the simplified engineering calculation methods are based on a spilling breaker assumption. Battjes (1974) used the surf similarity parameter to distinguish between the different types of breakers on a plane-sloping beach, whereas Smith and Kraus (1991) derived corresponding criteria for barred beaches. The following transition values were found for plane-sloping and barred beaches:

Breaker type	Plane beach	Barred beach
spilling	$\xi_o < 0.5$	$\xi_o < 0.4$
plunging	$0.5 < \xi_o < 3.3$	$0.4 < \xi_o < 1.2$
surging/collapsing	$\xi_o > 3.3$	$\xi_o > 1.2$

25. Most calculations in SBEACH regarding breaking waves are performed under the assumption of spilling breakers or plunging breakers, where the plunge distance is small and the breaking wave transforms rapidly to a bore. However, in some phases of beach evolution, especially during onshore sand transport and berm build-up, surging breakers may prevail. The insufficient description of nearshore wave dynamics for these wave conditions is one important reason why beach accretion is not well handled in previous versions of SBEACH.

26. The breaker height ratio Ω is defined as the ratio between the wave height at the break point and the deepwater wave height, that is, $\Omega = H_b/H_o$. A predictive equation may be

derived directly from a wave energy balance equation, assuming no energy losses going from deep water to the break point, where $H_b = \gamma_b d_b$, and neglecting refraction. However, due to non-linear shoaling close to the break point such an equation will typically produce results that deviate from laboratory and field measurements. Thus, the theoretically determined coefficient values in this equation have been modified empirically to comply with laboratory and field measurements. Komar and Gaughan (1972) derived the following equation for the breaker height ratio based on laboratory and field data:

$$\frac{H_b}{H_o} = 0.563\left(\frac{H_o}{L_o}\right)^{-1/5} \tag{4}$$

One weakness of relationships such as Equation 4 is that beach slope is not included and, as a result, they should be used with some caution. Singamsetti and Wind (1980) developed a predictive equation for Ω based on small-scale laboratory experiments that also involved beach slope:

$$\frac{H_b}{H_o} = 0.575(\tan\beta)^{0.031}\left(\frac{H_o}{L_o}\right)^{-0.254} \tag{5}$$

This relationship, however, displays a rather weak dependence on beach slope, and the power of the deepwater wave steepness is somewhat larger than what is obtained theoretically from linear wave theory. Smith and Kraus (1990) presented an overview of several predictive equations for the breaker height ratio and also developed a relationship based on small-scale laboratory data, where the power was expressed as a function of beach slope.

27. SBEACH relies on a predictive equation developed by Larson and Kraus (1989) that is similar to Equation 4 to estimate the breaking wave height in advance to the detailed wave transformation calculation. This estimate of the breaking wave height is used to determine the width of the moving average scheme that defines the filtered profile to be employed in the wave transformation calculation. However, the wave-steepness dependence is not on a power of -0.2, as in Equation 4, but instead a power of -0.25 is used.

28. After wave breaking, the broken waves translate through the surf zone and changes characteristics not only due to shoaling and refraction, but also because of energy dissipation. Outside the surf zone the approximation of no energy losses during onshore wave propagation is often valid, whereas inside the surf zone energy losses must be accounted for in any calculation method. Thus, the wave energy balance equation must include energy dissipation (D), and the

main difference between existing models for wave transformation in the surf zone is the formulation of D.

29. Since water depth is the primary factor controlling wave height in the surf zone, the most simple approach to describing wave decay is by assuming the local wave height to be proportional to the water depth. The wave height at any point in the surf zone is given by,

$$H = \gamma d \tag{6}$$

where γ is a constant typically assumed to be equal to γ_b for regular waves. In many cross-shore calculation methods, γ is assumed to be equal to 0.78 across the surf zone (compare calculation methods adapted by Federal Emergency Management Agency, FEMA). This is based on the assumption that broken waves are spilling breakers and could be characterized with the limiting solitary wave. As noted previously, however, numerous reasonable values of γ could be assumed depending on factors such as the breaker type and the local bottom slope.

30. A wave height decay according to Equation 6 implies the following energy dissipation in the wave energy balance equation:

$$D = \frac{5}{16}\rho g^{3/2}\gamma^2 d^{3/2}\frac{dd}{dx} \tag{7}$$

Since D must be positive, corresponding to an energy loss, Equation 6 can only be used on beaches where the depth increases monotonically with distance offshore in the surf zone ($dd/dx > 0$). In the case of a barred beach, regions where the beach slope is negative, such as in the trough, must be treated as regions where wave reformation occur and breaking ceases. Smith and Kraus (1988) presented a more general formulation of Equation 6 by assuming the local wave height to be proportional to an arbitrary power of the water depth. In spite of its limitations, Equation 6 is typically used for deriving the equilibrium profile of a beach exposed to breaking waves, leading to the well-known profile shape derived by Dean (1977). This type of simple wave analysis also allows for establishing a relationship between D for the equilibrium profile (D_{eq}) and the beach properties; SBEACH utilizes such a predictive relationship to estimate D_{eq} primarily from the median grain size.

31. It has been shown by several investigations, however, that broken waves tend to continue breaking until they reach some stable height given by,

$$H = \gamma_s d \tag{8}$$

where γ_s is a stable breaker depth index that is less than the incipient breaker depth index γ_b. This means that on a shelf beach (constant water depth shoreward of the break point) wave height decay will continue after breaking towards a height corresponding to Equation 8. Horikawa and Kuo (1966), for example, studied waves propagating on a shelf beach in the laboratory and found a stable wave height of 0.35 to 0.40 times the water depth. For plane-sloping beaches γ_s increased with the beach slope, although shoaling probably prevented fully stable wave conditions to develop.

32. More complex engineering models than Equation 6 for describing wave transformation in the surf zone involve physically or empirically based formulations for the energy dissipation including the possibility for the waves to reform. Some of these models permit analytical solution of the wave height decay over simple beach profile shapes, and numerical treatment for complex profile shapes is reasonably simple. When calculating wave transformation in the surf zone shallow-water linear wave theory is commonly employed, and a representative wave height is often used to characterize the surf zone waves across the profile. Probability-based wave transformation models have been developed that either assume a specific shape for the probability density function (pdf), or that empirically construct the pdf at points across shore from individual wave computations from a specified pdf in the offshore.

33. The earliest models for wave decay in the surf zone employed the analogy with a propagating turbulent bore to estimate the wave energy dissipation (Stoker 1957, LeMehaute 1962). Svendsen et al. (1978) used a bore model but found that it produced too low values on the energy dissipation in comparison with laboratory measurements. Ebersole (1987) compared the model by Svendsen et al. (1978) with high-quality field data from the US East Coast and found it to produce satisfactory agreement when applied to individual waves in the surf zone. Several authors have also applied a bore model in probability based wave decay models for the surf zone by using some ad hoc assumption about the pdf for breaking waves (Battjes and Janssen 1978, Thornton and Guza 1983).

34. The most commonly used engineering model for calculating wave height decay in the surf zone is the model proposed by Dally et al. (1985), which adapted the following expression for the energy dissipation,

where κ is an empirical coefficient characterizing the wave decay and F_s is stable wave energy flux associated with a stable wave height-to-depth ratio. Equation 9 states that once a wave starts

$$D = \frac{\kappa}{d}(F - F_s) \tag{9}$$

breaking it will continue to break until it reaches its stable height, and the energy dissipation depends on the difference between the actual and stable wave energy flux. The stable wave energy flux is written, using linear shallow-water wave theory,

$$F_s = \frac{1}{8}\rho g(\Gamma d)^2\sqrt{gd} \tag{10}$$

where the $H_s = \Gamma d$ has been used and Γ is an empirical coefficient. If complete linear wave theory is employed instead, $\sqrt{gd}$ in Equation 10 should be replaced by C_g. The original version of SBEACH employed shallow-water linear wave theory for determining wave decay in the surf zone, but in subsequent versions complete linear wave theory have been included.

35. The applicability of Equation 9 to describe wave height decay in the surf zone has been confirmed in numerous studies of wave transformation in the surf zone, where the model has been compared to both laboratory and field data. One particular strength of the model is that it includes wave reformation, implying that wave breaking ceases if $F < F_s$, allowing for a more realistic wave height description for complex bottom topographies. Furthermore, the two empirical coefficients κ and Γ display little variation in many different applications, although there is a tendency for a dependence on beach slope. The values recommended by Dally et al. (1985) were $\kappa=0.15$ and $\Gamma=0.4$ for typical beach slopes (1/80 to 1/20), whereas for steeper slopes κ and Γ could take on somewhat higher values. These values have been confirmed in several other studies involving both laboratory and field data, and could in general be employed with confidence in engineering studies without additional calibration (Ebersole 1987, Larson and Kraus 1989, Kraus and Larson 1991).

36. Using the expression for D proposed by Dally et al. (1985) and shallow-water linear wave theory neglecting refraction, the wave energy balance equation is written:

$$\frac{d}{dx}(H^2\sqrt{d}) = \frac{\kappa}{d}(H^2\sqrt{d} - \Gamma^2 d^{5/2}) \tag{11}$$

This equation can be solved numerically over any bottom topography, bearing in mind that wave reformation may occur for complex profile shapes, and analytically for special cases. For the most simple case of a planar beach of slope $m = \tan\beta$, the solution for the dimensionless wave height is (Dally et al. 1985),

$$\frac{H}{H_b} = \left[(1+\alpha)\left(\frac{d}{d_b}\right)^{\frac{\kappa}{m}-\frac{1}{2}} - \alpha\left(\frac{d}{d_b}\right)^2\right]^{\frac{1}{2}} \quad (12)$$

where:

$$\alpha = \frac{\kappa\Gamma^2}{\frac{5m}{2}-\kappa}\left(\frac{d_b}{H_b}\right)^2 \quad (13)$$

Equation 12 is not valid for slopes $m=2/5\kappa$ for which the denominator in Equation 13 approaches infinity, and for this special case reference is made to Dally et al. (1985). Dally et al. (1985) also presented analytical solutions for more complex beach profile shapes including an equilibrium profile of the form $h=Ax^{2/3}$.

37. For more complex bottom topographies, or if complete linear wave theory is employed, the wave energy balance equation has to be solved numerically to yield the wave height in the surf zone. The beach profile is divided into an arbitrary number of cells N with a fixed length step Δx characterized by the depth d_i in the middle of the cell. Larson et al. (1990) are referenced for a detailed definition of the grid and to what points different quantities in the calculation refer. Wave-related quantities, such as height and group speed, are computed at the boundaries between cells, and the depths needed for determining quantities between cells are interpolated from the depths in the middle of the cells. Writing the wave energy balance equation in finite-difference form, assuming normally incident waves, will produce the following relationship between the wave height at location i and i+1,

$$H_i = \left[\frac{H_{i+1}{}^2 C_{gi+1}(1 - A_i) + \Gamma^2 d_i^2 A_i (C_{gi} + C_{gi+1})}{C_{gi}(1 + A_i)}\right]^{\frac{1}{2}} \quad (14)$$

where:

$$A_i = \frac{1}{2}\frac{\kappa\Delta x}{d_i} \quad (15)$$

Thus, once the wave height at the offshore boundary of the calculation grid is known, assuming this point is located seaward of the breaker zone, the wave height computation may proceed onshore until the shoreward end of the grid is detected. The end of the grid is typically determined by dry land terminating the calculation, where setup may be included, or the presence of a seawall.

38. Since energy dissipation is accounted for only in the surf zone, κ is set to zero outside the break point, implying that $A_i = 0$ for all points seaward of the break point. The location of the break point is determined by calculating the breaker height ratio after each wave height computation and comparing it to the chosen γ_b. After a wave has started breaking it will continue until $F < F_s$, that is, wave reformation occurs and κ becomes zero again. This condition must be checked at each grid point after a wave has started breaking. Using a finite-difference approximation, the condition that a wave is reforming between grid point i and i+1 is expressed:

$$d_i > \left[\frac{H_{i+1}{}^2 C_{gi+1} + H_i^2 C_{gi}}{\Gamma^2 (C_{gi+1} + C_{gi})} \right]^{\frac{1}{2}} \tag{16}$$

If a wave is breaking and the depth d_i is greater than the right-hand side in Equation 16, wave reformation occurs and energy dissipation ceases. Wave reformation can only occur on beaches where the depth does not increase monotonically with distance offshore such as a barred profile.

39. As previously discussed, many engineering breaker decay models are employed to field conditions by calculating wave transformation for a representative wave height, such as the mean, significant, or root-mean-square wave height. However, models have also been developed that describe irregular wave transformation in the surf zone (Battjes 1972, Thornton and Guza 1983). Irregular breaker decay models typically rely on assumptions about the pdf and the percentage of breaking waves at points across shore. Dally (1990, 1992) employed his breaker decay model in a random fashion by transforming the pdf, assumed to be a Rayleigh distribution outside the surf zone, on an individual wave basis through the surf zone. Thus, the empirical pdf may be constructed at any point by adding the contributions from individual waves at that point, without any a priori assumption about the shape of the pdf. The wave-by-wave approach for developing the pdf for breaking waves was verified by Dally (1992) using data collected by Ebersole and Hughes (1987). In Part III of this report a newly developed semi-analytic model for describing irregular wave decay in the surf zone are presented that is based on the wave-by-wave approach.

40. As waves propagate over a sloping bottom, a variation in the flux of momentum arises because of shoaling, refraction, and breaking and alters the mean water elevation. The

excess momentum flux due to the presence of waves is called radiation stress and is in the general case a tensor with components in both coordinate directions. Using linear wave theory, the onshore-directed momentum flux that is transported towards the coast S_{xx} is expressed,

$$S_{xx} = \frac{1}{8}\rho g H^2 \left[n(\cos^2\theta + 1) - \frac{1}{2} \right] \tag{17}$$

where n is C_g/C and θ is the incident wave angle with respect to the bottom contour orientation. In shallow water and for waves normal to the coast, S_{xx} may be simplified to yield:

$$S_{xx} = \frac{3}{16}\rho g H^2 \tag{18}$$

Wave shoaling prior to breaking produces an increase in wave height and a corresponding increase in momentum flux. This flux increase is balanced by lowering of the mean water elevation, called setdown (Longuet-Higgins and Stewart 1962, 1963). Inside the surf zone, as waves break and decrease in height, the momentum flux decreases and an increase in mean water elevation occurs, known as setup.

41. The displacement η of the mean water surface (setup or setdown) produced by waves is determined from the cross-shore momentum equation,

$$\frac{dS_{xx}}{dx} = -\rho g d \frac{d\eta}{dx} \tag{19}$$

where $d = h+\eta$, and h is the still-water depth. Equation 19 may be solved analytically seaward of the break point to give a setdown (Longuet-Higgins and Stewart 1963):

$$\eta = -\frac{1}{4}\frac{\pi H^2}{L \sinh(\frac{4\pi h}{L})} \tag{20}$$

Thus, the maximum setdown η_b occurs at the break point, and using shallow-water wave theory Equation 20 reduces to,

$$\eta_b = -\frac{1}{16}\gamma_b H_b \tag{21}$$

where $\gamma_b = H_b/d_b$ and the subscript b refers to the break point.

42. Inside the surf zone, when waves dissipate energy such that the wave height is reduced, gradients in radiation stress produce an elevated water level (setup) that has its

maximum elevation at the shoreline intercept (where it must transition into the beach ground water table) and decreases offshore to the break point. The variation in setup across an arbitrary profile may be determined by solving Equation 19, which could be done analytically for simple cases. In SBEACH, Equation 19 is solved simultaneously with the wave energy balance equation, since the two equations are coupled through the wave height and water depth. However, using the most simple description of the wave height variation in the surf zone (Equation 6), where it is assumed that $H = \gamma d = \gamma(h+\eta)$, Equation 19 may be integrated for an arbitrary beach profile shape to yield,

$$\eta = \eta_b \frac{1}{1 + \frac{8}{3\gamma^2}} (h_b - h) \tag{22}$$

where shallow-water wave theory was used.

43. For more complex models of wave decay in the surf zone, Equation 19 must be solved numerically together with the wave energy balance equation. Formally, these two equations should be solved simultaneously and an iterative process is required at each grid point across shore. However, since the wave setup in general varies rather slowly with distance offshore, except for extremely steep beaches, it is often sufficient to calculate the setup from wave quantities based on depths that are slightly shifted in space (using setup from a preceding cell), avoiding iteration between the wave and setup calculations. This kind of simplification is used in SBEACH when calculating wave height variation and setup across the surf zone.

44. If Equation 19 is written in difference form, a quadratic equation in the setup is obtained that has only one physical root. Using the same notation and numerical grid as for the wave calculation, the solution is written,

$$\eta_i = -h_i + \left[h_i^2 + \frac{2\Delta S_{xxi}}{\rho g} + \eta_{i+1}^2 + 2\eta_{i+1} h_i \right]^{\frac{1}{2}} \tag{23}$$

where:

$$\Delta S_{xxi} = (S_{xx})_{i+1} - (S_{xx})_i \tag{24}$$

The setup (setdown) is computed at point i once all quantities at point i+1 is known, and $(S_{xx})_i$ has been determined from the wave calculation. The setup calculation is performed in parallel with the wave calculation, starting at the seaward end of the grid, where the wave conditions are specified, and continuing onshore until the water level intersects the beach profile. The setup is calculated on the boundaries between cells, in the same points as the wave heights, whereas the still-water depth is defined in the middle of a cell.

Transport Rate Calculations

45. The cross-shore movement of sediment in a surf zone is governed by the properties of the velocity field and the sediment concentration. Thus, a description of the velocity and concentration field is necessary not only across the surf zone but also through the water column at every point. At our present stage of knowledge, such a detailed prediction is not possible in engineering numerical models of beach profile change to be used on natural beaches. The sediment concentration in the surf zone is closely related to the generation of turbulent motion, which depends on the wave breaking. Accordingly, it is plausible that the sediment concentration, and thus the amount of material available for transport, is closely related to the wave energy dissipation due to wave breaking.

46. In the profile model SBEACH, no prediction of the cross-shore velocity field is made but a simple engineering approach is used to determine the transport rate distribution. The direction of the transport is predicted from an empirical criterion derived from the large wave tank data and verified with field data (Kraus et al. 1991), whereas the amount of transport is given by the wave energy dissipation per unit water volume (Larson and Kraus 1989). This simplifying approach implies a transport rate distribution directed either on- or offshore along the entire profile at a specific instant in time.

Cross-Shore Transport Direction

47. Criterion for predicting the overall profile evolution has been suggested by a number of authors (for a brief review, see Larson and Kraus 1989). Most of the criteria refer to the type of profile (winter/summer, storm/normal, bar/berm, dissipative/reflective etc) which evolves under certain wave and sediment conditions, whereas some authors have focused on predicting the transport direction. In general, there is no contradiction between the two types of criteria but they could be regarded as equivalent in a macroscopic sense. A profile where bars develop has a transport direction predominantly offshore, whereas a profile exhibiting berm build-up is mainly subject to onshore sand movement. If a criterion for the transport direction is expressed in local wave properties however, there could be a significant difference between transport direction and the overall profile type that evolves.

48. Criterion for predicting profile-type (overall transport direction) normally comprehends one parameter characterizing the incident wave conditions and one parameter involving some property of the sediments (grain size or fall speed). Wave steepness is the most commonly used descriptor of the wave conditions (Waters 1939, Rector 1954, Iwagaki and Noda 1962, Nayak 1970, Dean 1973, Sunamura and Horikawa 1974), whereas several parameters involving sediment properties have been proposed such as, D_{50}/L_o (Rector 1954), H_o/D_{50} (Iwagaki and Noda 1962), and $\pi w/gT$ (Dean 1973) (D_{50}=median grain size, L=wavelength, H=wave height, w=sand fall speed, g=acceleration of gravity, and T=wave period, where o denotes deepwater conditions). In SBEACH the two parameters H_o/L_o and H_o/wT are used to distinguish between on- and offshore transport according to the equation:

$$\frac{H_o}{L_o} = 0.0007 \left(\frac{H_o}{wT}\right)^3 \tag{25}$$

If the left side of Equation 25 is less (greater) than the right side, the profile is predicted to erode (accrete). The wave steepness describes the wave asymmetry, whereas the wave height and period appearing in the fall speed parameter account for the absolute magnitudes of those quantities. Saville (1957) showed that the magnitude of the wave height controls erosion an accretion in addition to the wave steepness.

Transport Regions and Transport Rates

49. From findings in nearshore wave dynamics regions with different hydrodynamic properties regarding wave shoaling and wave decay have been identified. Svendsen, Madsen, and Buhr Hansen (1978) defined an outer and inner region of the surf zone depending on the wave characteristics. The outer region, located immediately shoreward of the break point is distinguished by a rapid transition in wave height with large-scale flow patterns. At some point, the large-scale flow is broken up and turned into small-scale turbulent fluctuations, simultaneously the wave changes character and becomes bore-like, and the region shoreward from this point is known as the inner region extending to where the runup starts.

50. The concept of two different region in the surf zone was suggested already by Miller (1976) who distinguished a vortex and a bore region (see also Basco 1985 and Jansen 1986). He also noted that the main difference between plunging and spilling breakers was the generation of vortices in the vortex (outer) region. In the bore (inner) region was the flow conditions very similar independently of breaker type as was noted by Svendsen, Madsen, and Buhr Hansen (1978). Svendsen (1987) analyzed flow measurements to determine the turbulent kinetic energy

in the surf zone. He found that the variation over the depth of the time-mean of the turbulent kinetic energy was very small (Peregrine and Svendsen 1978), attributed to the strong mixing created by large-scale vortices. Skjelbreia (1987) studied breaking of solitary waves in the laboratory and defined four different zones of wave transformation namely, gradual shoaling, rapid shoaling, rapid decay, and gradual decay.

51. The identification of regions with different wave characteristics in the nearshore stimulated Larson and Kraus (1989) to perform a separate analysis of calculated net transport rates in these regions. An important assumption behind SBEACH is the significance of breaking and broken waves in determining the cross-shore transport rate. Quite different transport characteristics should be found in- and outside the surf zone making it reasonable to assume that the transport relationships are different in a macroscopic sense. Furthermore, the dynamics of the transport in the swash are governed by the properties of the uprush bore and the backwash, sediment characteristics, and beach slope, which makes it logical to define this as a special transport region.

52. Based on the division of the profile applied in nearshore wave dynamics and the physical characteristics of sediment transport under various flow conditions, four different zones of transport were introduced by Larson and Kraus (1989). These zones are

Zone I:	From the seaward depth of effective sand transport to the break point (pre-breaking zone)
Zone II:	From the break point to the plunge point (breaker transition zone)
Zone III:	From the plunge point to the point of wave reformation or to the swash zone (broken wave zone)
Zone IV:	From the shoreward boundary of the surf zone to the shoreward limit of runup (swash zone)

53. The zone between the break point and the plunge point is analogous to the outer region (vortex region) proposed by Svendsen, Madsen, and Buhr Hansen (1978). A certain distance is required after breaking before the turbulent conditions are approximately uniform throughout the water column. From a cross-shore transport point of view, assuming the transport rate being proportional to the energy dissipation per unit water volume, the recognition of this zone is important and physically motivated. A more detailed discussion of the different transport zones are given in Larson and Kraus (1989). In this report only the empirically-based transport

relationships for the zones derived from the large wave tank data will be presented. These formulas are used in the numerical model to determine the transport rate distribution across-shore.

54. The transport rate relationships derived based on physical considerations and analysis of the large wave tank data may be summarized for the four different zones

$$\textit{Zone I}: \quad q = q_b e^{-\lambda_1 (x-x_b)} \qquad x_b < x \tag{26}$$

$$\textit{Zone II}: \quad q = q_p e^{-\lambda_2 (x-x_p)} \qquad x_p < x \le x_b \tag{27}$$

$$\begin{aligned} \textit{Zone III}: \quad & q = K\left(D - D_{eq} + \frac{\epsilon}{K}\frac{dh}{dx}\right) \qquad q > 0 \\ & q = 0 \qquad q < 0 \\ & x_s \le x \le x_p \end{aligned} \tag{28}$$

$$\textit{Zone IV}: \quad q = q_s \frac{x - x_r}{x_s - x_r} \qquad x_r \le x < x_s \tag{29}$$

where

q = cross-shore net transport rate (m^3/m/sec)

x = cross-shore coordinate pointing offshore (m)

λ = spatial decay coefficient (1/m)

K = transport rate coefficient (m^4/N)

D = wave energy dissipation per unit water volume (Nm/m^3/sec)

D_{eq} = equilibrium wave energy dissipation per unit water volume (Nm/m^3/sec)

ϵ = slope-related transport rate coefficient (m^2/sec)

h = water depth (m)

The subscripts b, p, s, and r stands (in order) for quantities evaluated at the break point, plunge point, end of the surf zone, and runup limit. Two different spatial decay coefficients are used in Zone I and II (with the subscripts 1 and 2 respectively) to describe the decrease in transport rate with distance.

55. Empirical expression for the spatial decay coefficients were derived from the large wave tank data (Larson and Kraus 1989). The decay coefficient in Zone I was related to the median grain size and the breaking wave height:

$$\lambda_1 = 10.4\left(\frac{D_{50}}{H_b}\right)^{0.47} \quad \textbf{(30)}$$

In Zone II limited data suggested that:

$$\lambda_2 = 0.2\lambda_1 \quad \textbf{(31)}$$

When calculating the transport rate in Zone I and II the transport rate is first determined at the plunge point from Equation 28 and then the exponential decays are applied seaward in respective zone.

56. The cross-shore transport rate in zones of fully broken waves is related to the wave energy dissipation per unit water volume. This type of transport formula has previously been used by Moore (1982), Kriebel (1982, 1986), and Kriebel and Dean (1985). Analysis of the large wave tank data substantiated this type of transport relationship in zones of broken waves (Larson 1988, Larson and Kraus 1989). The transport relationships in the other zones are empirical and based on a large amount of data from the wave tank experiments. For example, in the swash zone the transport rate simply decreases linearly from the end of the surf zone to the runup limit. This property of the transport rate distribution appeared for many large wave tank cases, both with on- and offshore movement, and similar characteristics have been noted on natural beaches where the beach face receded uniformly during erosional and accretionary events (Seymour 1987).

Model Verification

57. The numerical model of beach profile change, developed using large wave tank data, has been verified against data on profile evolution obtained from natural beaches. Larson (1988) and Larson and Kraus (1989) evaluated the capabilities of the numerical model of reproducing bar formation and bar movement by using data from the U.S. Army Field Research Facility in Duck, North Carolina. In Larson et al. (1990) additional model verification was carried out using field data on berm and dune erosion both from the US East and West coast. The overall model performance of simulating the erosional part of a storm event was satisfactory, whereas the

recovery phase following a storm event, when the berm is rebuilt, was less well described and essentially only in qualitative agreement with the measured profile evolution.

58. In Larson et al. (1990), a high-quality data set obtained during a storm off the coast of New Jersey was used to investigate the ability of the model to simulate berm and dune erosion. Simultaneous measurements of wave height, wave period, and water level were made together with surveys of the beach profile before and after the storm event along several transects. The numerical model predicted eroded volumes and contour retreat in agreement with what was observed during the erosional part of the storm event. When the storm waned and the waves became accretionary, a large berm formed on the foreshore having a size that was not predicted by the model. However, the qualitative features of the recovery was reproduced in the model, especially regarding the development of profiles with different profile geometry such as offshore profile shape and dune height.

59. To evaluate the possibilities of the model to describe profile change on the US West coast, and thus a large variability in water level due to change in tide level, data from the beach at Torrey Pines in California were employed by Larson et al. (1990). The contour retreat on the foreshore and lower part of the dune was well reproduced with the model as well as the general profile shape. The amount of erosion was underpredicted as larger sand volumes were eroded close to the runup limit than what was predicted by the model.

60. The numerical model has also been used to simulate the short-term fate of beach fill, both with respect to the initial fill adjustment and the fill response to storm events (Larson and Kraus 1991). Hypothetical cases were studied with the main variables of the beach fill being fill geometry and grain size, whereas the response to different synthetic storm events were investigated. The synthetic storms were chosen as being typical for the US East coast corresponding to tropical- (hurricane) and extratropical-type (northeaster) storms. Two main fill geometries were evaluated, namely the artificial berm and the profile nourishment technique.

61. The simulations of the fill response to storm events showed the importance of both the peak surge level, surge duration, and the wave conditions. For example, a high surge with a low duration could produce as much erosion as low surge with a longer duration. The profile nourishment technique created a fill which responded better to the storms than the artificial berm. The reason was probably that the fill was more in accordance with the natural equilibrium profile minimizing the necessary profile adjustments as the wave and water level conditions changed. This assumption was further substantiated by the stronger recovery shown by the fill carried out with the profile nourishment technique. The model simulation also indicated that little fill stability

during a storm event was gained when the fill grain size exceeded 0.40 mm. In summary, the numerical model performed well for simulating the response of beach fills and could be used in a cost-benefit analysis of finding the optimal fill design.

62. The conditions under which the present version of SBEACH is expected to give reliable results may be summarized:

1. Breaking waves are the main cause of sand transport and thus profile change.
2. Longshore effects are negligible or small, that is, the incident waves are normal or the differentials in longshore transport are minor (influence of structures blocking the longshore transport is small and the shoreline is straight).
3. The median beach grain size is in the interval 0.1-2.0 mm and reasonably uniform across-shore.
4. Micro-scale morphologic features are not described such as ripples but only major features such as bars and berms are described.
5. Tracking of individual grains are not made in the model implying that a simplified approach is used to describe a beach fill with a different grain size than the native beach.

Under this restriction the numerical model should perform well and produce results that are reasonably accurate. However, it is important to use some kind of data from the site to be studied in order to calibrate the model and thus establish reliable values on the model parameters, primarily the transport rate coefficient.

PART III: ENHANCEMENTS TO THE WAVE DECAY MODEL

Randomization of Wave Decay Model

63. An essential requirement in all engineering models of nearshore processes is an accurate description of wave decay in the surf zone. Models requiring this information include prediction of nearshore currents and beach topography change due to longshore and cross-shore sediment transport. Wave breaking is closely associated with the generation of these currents and also with the mobilization and entrainment of sediment. At present, engineering models such as SBEACH rely on prediction of macrofeatures of the breaking wave for hydrodynamics and sediment transport calculations.

64. Several models have been proposed for describing wave decay in the surf zone, differing mainly in the formulation of the energy dissipation due to breaking and whether they were developed for regular or irregular waves (Battjes and Janssen 1978, Thornton and Guza 1983, Svendsen 1984, Dally et al. 1985). The models for irregular waves transform a representative statistical wave height across shore, typically the root-mean-square (rms) wave height, requiring some assumptions to be made about the probability distribution function (pdf) for the waves in the surf zone. The other class of models that were originally based on the study of regular waves have later been modified to simulate transformation of irregular waves by using a wave-by-wave approach (Dally 1990, 1992, Kraus and Larson 1991). The wave-by-wave approach has been verified with high-quality field data and has the inherent advantage of not requiring an assumption about the shape of the pdf in the surf zone. However, this approach is computationally intensive and therefore difficult to employ in time-dependent nearshore models.

65. In order to improve the description of wave transformation in SBEACH, a model for the decay of irregular waves in the surf zone was introduced that requires transformation of only one representative wave height. This semi-analytic model reproduces macrofeatures of wave height and flux transformation across the surf zone, including breaking and reformation, in agreement with a computation-intensive statistical wave-by-wave approach that involves transformation of many individual waves.

66. The basis for the semi-analytic model is a newly derived equation (Larson 1992) describing the transformation of irregular waves. The derivation is based on the decay model proposed by Dally et al. (1985) and a wave-by-wave approach to yield,

$$\frac{d}{dx}\left(H_{rms}^2 C_g\right) = \frac{\kappa}{d}\left\{H_{rms}^2 C_g - [(1-\alpha)H_{nb}^2 + \alpha\Gamma^2 d^2]C_g\right\} \tag{32}$$

where H_{rms} is the rms wave height for nonbreaking and broken waves (to be solved for), α the percentage of broken waves, H_{nb} the rms wave height for nonbreaking waves, and the other variables as in the original Dally model. In the general case, the nonbreaking waves consist both of waves that have never broken (unbroken) and reformed waves. For the case of a beach with monotonically increasing depth with distance offshore, both α and H_{nb} may be determined analytically, assuming a Rayleigh distribution in deep water. However, for a barred profile, wave reformation will occur and α is not uniquely related to the water depth; therefore, an additional assumption about the reformation process is needed. Introducing the assumption that the rate at which waves reform along negatively sloping sections of the beach is proportional to the local slope and the percentage of broken waves allows calculation of H_{rms} across shore by addition of only one parameter (λ) to the wave model. This modification permits calculation of rms wave height across shore on irregular beach profiles.

67. Assuming a Rayleigh distribution at some point outside the surf zone where the rms wave height is H_m, the rms wave height H_x at any point along the profile, neglecting wave breaking, will be,

$$H_x^2 = \frac{C_{gm}}{C_{gx}} H_m^2 \tag{33}$$

where index x refers to a specific point and m to the location outside the surf zone where the Rayleigh distribution is valid. If all waves break when $H > \gamma h$, the percentage of broken waves α may be obtained from the truncated Rayleigh distribution:

$$\alpha = e^{-\left(\frac{\gamma d}{H_x}\right)^2} = e^{-\left(\frac{\gamma d}{H_m}\right)^2 \frac{C_{gx}}{C_{gm}}} \tag{34}$$

Also, from the Rayleigh distribution it is possible to calculate:

$$(1 - \alpha)H_{nb}^2 = H_x^2 - e^{-\left(\frac{\gamma d}{H_x}\right)^2}\left[H_x^2 + (\gamma d)^2\right] \tag{35}$$

Combining Equations 32 and 35 yields,

$$\frac{d}{dx}(F_{rms}) = \frac{\kappa}{d}(F_{rms} - F_s^m) \tag{36}$$

where,

$$F_{rms} = \frac{1}{8}\rho g H_{rms}^2 C_{gx} \tag{37}$$

and:

$$F_s^m = \frac{1}{8}\rho g C_{gx}\left[H_x^2(1 - \alpha) - \alpha d^2(\gamma^2 - \Gamma^2)\right] \tag{38}$$

Thus, solving Equation 36 together with Equations 33 and 34 will give the variation in H_{rms} across shore.

68. Larson (1992) compared the above-described approach with the complete random calculation, where a large number of waves were transformed across shore (Monte-Carlo simulation) and H_{rms} calculated at each point from all wave components, for the beach profiles presented in Kraus and Sasaki (1979) and Kraus and Larson (1991). The Kraus-Sasaki profile had depth values that increased monotonically with distance offshore, whereas the profile from Kraus and Larson (1991) had a distinct bar. The agreement between the two approaches was almost perfect, both for the rms wave height and for the average energy dissipation. Thus, using the irregular wave decay model will save calculation time, while maintaining a probabilistic description of the effect of the waves on the beach.

Transport Equation for Turbulent Kinetic Energy

69. In order to model the spatial lag between the production and dissipation of turbulent kinetic energy (tke), a transport equation for tke was introduced (Smith et al. 1992). As waves break, the organized wave energy is not immediately dissipated, but the turbulence generated through breaking contributes to the momentum flux before it is eventually dissipated into heat (Svendsen 1984). This transport of turbulent momentum must be modeled to obtain a satisfactory description of currents and water level changes induced by wave breaking in the nearshore. By solving a transport equation for tke, including advection and diffusion of tke, the distribution of tke in the surf zone is obtained and the contribution from turbulent transport in the momentum equation may be estimated.

70. A layer model is used to characterize surf zone hydrodynamics, with an upper layer extending from trough level and above, and a lower layer below trough level. Also, it assumed that the production and decay of turbulence due to wave breaking mainly occur above trough

level, and only a small portion of the turbulence is transported vertically down below trough level (Svendsen 1984 and Nadaoka 1986). Thus, by modeling the tke in the upper layer, the momentum transport related to the turbulence may be derived, and no mixing should be included to take into account this type of Reynold's stresses. However, in the lower layer, mixing could still occur due to current shearing or the turbulence transported down from the upper layer or up from the bottom (to describe this mixing a diffusion-type term could be included in the alongshore momentum equation).

71. The basic expression for the time-averaged onshore momentum transport per unit width S_{xx} past a section x (extending from the bottom (-h) up to the water surface (η)) perpendicular to the direction of propagation due to waves and turbulence induced by breaking is,

$$S_{xx} = \overline{\int_{-h}^{\eta} (p+\rho u^2)\,dz} - \int_{-h}^{0} p_o\,dz \tag{39}$$

where $u=u_w+u_t$ (subscript w denotes waves and t turbulence), both fluctuations with a time mean equal to zero, the overline denotes a time-average operation, and p_o is the hydrostatic pressure. If u_w and u_t are uncorrelated (temporal mean of the product $u_w u_t$ is zero), the momentum transport due to the oscillatory wave and the turbulence may be evaluated separately to yield (neglecting any influence of the turbulent motion on the pressure distribution):

$$S_{xx}^{w} + S_{xx}^{t} = \overline{\int_{-h}^{\eta} (p+\rho u_w^2)\,dz} - \int_{-h}^{o} p_o\,dz + \overline{\int_{-h}^{\eta} \rho u_t^2\,dz} \tag{40}$$

72. The two first integrals in Equation 40 may be evaluated according to Longuet-Higgins and Stewart (1964) using linear wave theory to yield the onshore momentum transport due to an oscillatory wave $(S_{xx})^w$. To determine the third integral in Equation 40, the layer model will be introduced, where only the turbulence in the upper layer will be taken into account. Thus, the integration should only be performed over the upper layer, which is assumed to have a characteristic depth l, to obtain the momentum transport due to turbulence from wave breaking:

$$S^t_{xx} = \overline{\int_l \rho u_t^2 dz} \tag{41}$$

The layer depth approximately extends from the trough level to the free surface. Evaluating the integral in Equation 41 requires some assumptions about the turbulence in the upper layer, and assuming it to be fairly homogeneous vertically through the layer is probably a good approximation (characterized by the mean tke over the layer denoted k). Also, depending on the type of turbulent flow, the rms-value of the fluctuations in the different coordinate directions will be in fixed proportion to each other (see for example table by Svendsen 1987). The values of these proportions for a breaking wave are still uncertain, analogies with a mixing layer or a wake have employed by various authors, but in either case the rms-value of the fluctuations could be set proportional to k. Thus, Equation 41 could be expressed as:

$$\overline{\int_l \rho u_t^2 dz} \sim \rho k l \tag{42}$$

73. The layer depth l is assumed to vary slowly with x and should be related to wave height, which is related to water depth in the surf zone. In the same manner as Equation 42 was derived for the onshore momentum transport due to turbulence, the alongshore momentum transport due to the turbulence may be obtained:

$$S^t_{yy} = \overline{\int_l \rho v_t^2 dz} \sim \rho k l \tag{43}$$

74. However, for the case of waves that are not normally incident, cross-terms will be generated for the turbulent transport of momentum, just as for the case of momentum transport by waves, representing onshore transport of momentum that are directed alongshore; a momentum transport that influences, for example, the generation of a longshore current. These cross-terms may be obtained by a tensor transformation to yield,

$$S^t_{xy} = c_t \rho k l \sin\theta \cos\theta \tag{44}$$

where c_t is a coefficient with a value that could be estimated through analogy with other turbulent flows, and θ is the incident wave angle.

75. The horizontal distribution of k (depth-averaged over the layer as before) in the upper layer is determined from a transport equation that includes advection and diffusion of tke together with the production and dissipation terms. If the layer depth varies slowly across-shore, the transport equation could be written,

$$\frac{d}{dx}(kC\cos\theta) = \frac{d}{dx}(\epsilon\frac{dk}{dx}) + P_d - c_d\frac{k^{3/2}}{l} \tag{45}$$

where C is the wave speed and P_d is the production of turbulence per unit volume of the upper layer as given by the breaker decay model. The diffusion coefficient ϵ is taken as proportional to $k^{0.5}l$ (Rodi 1980), and the length scale for the most effective turbulent eddies in transporting and dissipating tke is assumed equal to the layer depth l. The advective speed in Equation 45 is set equal to the wave speed in the onshore direction, which should give a reasonable estimate in the upper layer (compare with measurements by Nadaoka (1986)).

76. The wave setup (longshore current) is obtained as before by solving the depth-integrated cross-shore (alongshore) momentum equation, where the transport of turbulent momentum is included (only for the upper layer). As previously discussed, a lateral mixing term may still be included in the governing equations, but this term has to model the Reynold's stresses due to the turbulent exchange in the lower layer since the turbulence in the upper layer has already been accounted for in the driving terms. The question arises, however, how to determine the lateral mixing coefficient; one method would be simple to assume that the tke in the lower layer, arising from turbulence transported down from the upper layer, is vertically uniform at a specific section (compare Svendsen (1987)) and proportional to the tke in the upper layer at that section.

Response Time of Nearshore Hydrodynamics

77. SBEACH is a time-dependent model that allows for variable input regarding waves and water levels. However, at each time step quasi-stationary conditions are assumed implying that the length of the time step is much greater than the time needed for the nearshore conditions to attain a new steady-state when the forcing changes. The validity of this assumption has been investigated by quantifying the governing time scales of nearshore hydrodynamics (Larson 1993). The study of analytic solutions for simple cases, such as a plane-sloping beach where a sudden change in the incident waves occur, have provided insight to how rapid the conditions in the surf

zone respond to external changes. Thus, a quantitative measure of the applicability of a quasi-stationary approach has been derived from these simplified analytic cases. However, the result of these analytic studies is not discussed here, but reference is made to an upcoming report that will specifically deal with this topic (Larson 1993).

PART IV: ENHANCEMENTS TO SBEACH

Variable Grid

78. The option of having a calculation grid with a variable length step was introduced in SBEACH. In the present version, there are no restrictions on the choice of length step in the model, except with regard to numerical stability requirements, but an arbitrary grid cell spacing is allowed. A variable grid have two major advantages in comparison with a fixed grid, namely (1) a high resolution in space is possible in areas that are of special interest such as the foreshore or where numerical calculations demand shorter length steps, and (2) in areas where calculation quantities are changing slowly a coarser grid may be used to cut down on execution times.

79. Major additions/modifications were necessary in the following routines in SBEACH to implement a grid with a varying length step:

a. a subroutine for generating or inputing a variable grid
b. the finite difference scheme for the wave calculations, especially the setup computations using the cross-shore momentum equation
c. the finite difference scheme for the cross-shore transport rate calculations
d. the avalanching routine preventing the development of unreasonably steep beach slopes

The modified version of SBEACH was tested using several different cases with variable grid spacing, both involving monochromatic and random wave input. Numerically induced oscillations were encountered in some cases when a short length step was employed on the foreshore with respect to the chosen time step. In such cases the time step has to be correspondingly reduced to achieve stability in the calculations. The variable grid option has also been implemented in the PC-version of SBEACH.

80. Three different options exist presently in SBEACH for specifying the grid: (1) a constant length step Δx, (2) an arbitrary number of profile sections with a constant Δx, (3) completely arbitrary Δx, which is read from a file. The same grid is used throughout a specific calculation; however, in the future it is possible to implement an adaptive grid, where the length step along the profile is related to the bottom topography (slope etc) and updated at each time step.

Grain-Size Dependent Equilibrium Profile

81. Even though the 2/3-power law from Bruun (1954) and Dean (1977) provides a good fit to many beach profiles in the field, significant deviations from this shape may be encountered if the grain size varies markedly along the profile. For example, on most beaches the sediment is sorted with coarser material located close to the shoreline and a median grain diameter that decreases with distance offshore. Significant sediment sorting may produce an equilibrium profile that is steep in the shoreward part of the profile and slopes off more gently in the seaward part. This profile shape is not well described by the classical equilibrium profile equation (Kraus et al. 1988, Larson et al. 1990), and some modification has to be introduced to account for the varying grain size (Larson 1991).

82. If the assumption is made in accordance with Dean (1977) that equilibrium conditions imply constant wave energy dissipation per unit water volume along the beach, and D_{eq} is primarily dependent on grain size, the following solution is obtained,

$$h = \left[\frac{3}{2K} \int_{o}^{x} D_{eq}\, dx' \right]^{\frac{2}{3}} \tag{46}$$

where small-amplitude wave theory for shallow water was used together with the assumption that the wave height is proportional to the water depth in the surf zone. The coefficient K in Equation 46 is written:

$$K = \frac{5}{16} \rho g \sqrt{g}\, \gamma^2 \tag{47}$$

In order to evaluate the integral in Equation 46, two pieces of information are needed, namely knowledge of the variation in median grain size d_{50} across the profile and the relationship between D_{eq} and the median grain size. An empirically derived curve by Moore (1982, modified by Dean (1987)) gives such a relationship, although the analysis of the field data was carried out on the basis of one representative grain size for a specific beach.

83. If the variation in D_{eq} is expressed in terms of some elementary function, the integral in Equation 46 may be solved analytically, thereby maintaining the usefulness of a simple mathematical expression yet providing a more rigorous description of the beach characteristics. The median grain size is expected to decrease with distance offshore, with a coarser grain size

located closer to the shoreline, and the decrease in the grain size is typically more steep closer to the shoreline as compared to in the offshore. Thus, the following simple description of the variation in D_{eq} with distance from the shoreline may be assumed,

$$D_{eq} = D_{\infty} + (D_o - D_{\infty})\, e^{(-\lambda x)} \tag{48}$$

where D_o and D_{∞} are the equilibrium energy dissipation for the material at the shoreline and in the offshore respectively ($D_o \geq D_{\infty}$), and λ is an empirical coefficient describing the rate at which D_{eq} approaches D_{∞}. Using Equation 48, the equilibrium profile shape given by Equation 46 is,

$$h = A \left\{ x + \frac{1}{\lambda} \left(\frac{D_o}{D_{\infty}} - 1 \right) \left(1 - e^{(-\lambda x)} \right) \right\}^{\frac{2}{3}} \tag{49}$$

in which $A = (3D_{\infty}/2K)^{2/3}$.

84. In order to investigate the possibility of improving the description of the equilibrium profile shape for a beach with varying grain size, field data from Point Pleasant Beach, New Jersey, were used to compare between theoretical predictions and recorded beach profiles. A thorough description of the profile data and how a representative profile was derived may be found in Larson et al. (1990). Initially a 2/3-power curve was least-square fitted to the profile data, which could explain a large portion of the variation in the data, but visual inspection showed deficiencies in the profile description. Directly seaward of the shoreline the representative profile was steeper than the fitted profile, whereas the opposite occurred in the offshore region. In the least-square fit, both the shape parameter A and the horizontal location of the still-water shoreline were estimated. The optimal estimate of the A parameter, minimizing the sum of squares of the difference between measured and predicted depths, was 0.20 $m^{1/3}$ which corresponds to a grain size of 0.47 mm according to Moore (1982).

85. To improve the description of the beach profile at Point Pleasant, the variation in the median grain size across-shore was taken into account. An equation such as Equation 48 reflects the main features of the grain size variation, with a large equilibrium energy dissipation close to the shoreline which decreases towards a lower constant value in the offshore. Thus, as a first attempt Equation 49 was least-square fitted to the profile data. Three parameters must be estimated, that is, the shape parameter A (based on D_{∞}), the ratio D_o/D_{∞}, and the spatial decay coefficient λ. The optimum fit occurred for $A = 0.10\ m^{1/3}$, $D_o/D_{\infty} = 5$, and $\lambda = 0.007\ m^{-1}$.

The agreement between the measured and calculated profile improved considerably if Equation 49 was used in comparison with the classical 2/3-power curve.

86. Larson (1991) presented a more extensive discussion of the modified equilibrium profile and gave additional examples of how well this profile shape fitted other field measurements where the grain size varied appreciably across shore. Measured profiles from Duck, NC, and Baie Rouge, St. Martin in the Caribbean, were employed, which further illustrated the improved agreement with the modified equilibrium profile in comparison with the 2/3-power curve. The disadvantages of using an expression such as Equation 49 are the increase in the number of parameters to be estimated and the added complexity of least-square fitting the equation to field data.

Landward Boundary Condition

87. The formulation of the landward boundary condition is of primary importance for determining the effect of the waves on the subaerial portion of the beach profile. Description of the profile response should encompass not only foreshore and dune erosion, but also overwash and inundation together with the effects of a seawall. Furthermore, successful modeling of onshore transport and berm build-up on the foreshore is another process that is intimately linked to the choice of landward boundary condition.

88. The possibility to verify complex landward boundary conditions is limited depending on the lack of data from laboratory experiments and field measurements. Thus, the formulation of the landward boundary conditions often has to be validated through qualitative comparisons with little amount of data. For example, to check the seawall failure algorithm simulations were carried out for an event during which it was known that the seawall failed, although no detailed information existed on exactly when the structure collapsed.

Inundation and overwash

89. In order to model inundation and overwash a simple geometric method was employed that qualitatively reproduces the features of dune erosion during overwash. The concept of a "wetted beach volume" was introduced, where a runup water volume is defined from which the portion of the dune affected by the overwash is determined. Overwash occurs when the predicted runup height exceeds the highest elevation of the dune crest. The runup height is calculated with

respect to the still-water level, including storm surge and setup due to wind, but excluding wave setup. A runup volume V_{ru} is calculated based on the predicted runup height,

$$V_{ru} = \frac{1}{2}\frac{R^2}{\beta} \tag{50}$$

where β is the foreshore slope.

90. If it is assumed that the runup volume represents a water volume that is going to be absorbed by the beach, the most shoreward point affected by the overwash may be determined from simple geometric considerations. The sand volume of the subaerial portion of the beach is determined by numerical integration starting from the shoreline and moving landward. When the beach sand volume coincides with the runup volume, the shoreward boundary for sand transport has been found. At this calculation point the transport rate is set to zero, and at the dune crest (point of maximum elevation) the transport rate is allowed to change sign so that the most landward portion of the transport rate distribution simulates the onshore transport occurring during overwash.

91. High-quality data on overwash in connection with severe storms, encompassing pre- and poststorm profiles together with wave and water level conditions during the storm, are rare. Available data sets typically only involve observations whether overwash took place or not and rough estimates of the sand volume transported beyond the former dune crest. To verify the proposed overwash routine, a qualitative comparison were made for a storm at Ocean City, MD, during which overwash occurred along some stretches of the coast. The overwash routine qualitatively reproduced the profile response recorded in the field.

Seawall failure

92. SBEACH allows the user to specify several different failure modes for a seawall. Since the model only describes macro-features of profile change, no detailed modeling of the local scour in front of the seawall is carried out, but calculated depth changes in the vicinity of the seawall should be regarded as caused by the overall cross-shore transport pattern. The three different failure modes that the model checks for are:

1. Depth change relative to initial depth
2. Absolute water depth

3. Local wave height

The model checks at every time step for all of these failure modes, and if a structure collapses the beach is allowed to erode shoreward without any restrictions. A seawall is described in SBEACH as a boundary where the cross-shore sand transport rate is zero.

93. Mode 1 implies that the seawall fails if erosion causes a certain depth change with respect to the initial profile depth in front of the seawall. However, it is also possible that absolute water depth at the seawall could make it collapse, so this condition is investigated too. The third mode of failure is related to the local wave height that might impact the structure and, thus, cause it to destruct. As in the case of overwash, very little data exist on seawall failure, but mainly qualitative comparisons were made to evaluate model performance.

Beach Profile Accretion

94. Waves of low steepness have a tendency of transporting material onshore, which builds up the foreshore and creates a berm. As sand is pushed onshore, the profile depth increases in the subaqueous portion of the profile and the break point moves onshore. The onshore movement of the break point implies a narrowing of the surf zone, and at the same time the foreshore will steepen because of the ongoing accumulation. Thus, for some combinations of waves and beach properties this development may lead to an unrealistically steep foreshore with slopes that are only controlled by the angle of repose.

95. Larson and Sunamura (1991) studied berm build-up and step formation in the laboratory, where waves were allowed to act on an initially plane-sloping beach until equilibrium was attained. One important feature of this development was the change in breaking wave characteristics, with initially plunging breakers transitioning to surging breakers at the end of an experiment when a distinct step-berm formation existed. For surging breakers, no distinct surf zone exists, but the wave breaks at the step and rushes up the foreshore. No net transport of sand occurs at equilibrium, and even though material is moved up and down during individual swash cycles the profile displays no changes in time.

96. The wave decay model by Dally et al. (1985), used in SBEACH to determine wave height variation across shore, assumes spilling breakers. Thus, during some phases of the berm build-up when surging breakers prevail, this model is not appropriate for describing nearshore

wave properties. Presently, no suitable wave decay model exists that describe the macro-features of surging breakers at a detail compatible with the other calculation modules in SBEACH.

97. In order to model beach accretion more realistically, the transport rate distribution is modified during onshore transport as opposed to trying to improve the wave height calculations. A maximum foreshore slope β_{max} is specified by the user that limits the steepening of the beach face. Thus, if onshore transport occurs, the calculated net transport rates are reduced according to:

$$q_{red} = q\left(1 - \frac{\frac{dh}{dx}}{\beta_{max}}\right) \tag{51}$$

Employed at every time step, Equation 51 will allow the foreshore to gradually approach the slope β_{max}, if the wave conditions are favorable, and the problem of excessive foreshore steepening is avoided.

98. To evaluate the capability of Equation 51 to describe foreshore evolution during beach accretion, field data collected in connection with a northeaster passing Point Pleasant, NJ, were used (Larson et al. 1990). During the decline of the storm, strong recovery prevailed as long-period waves with low height acted upon the beach. Before implementing Equation 51 some problems were encountered in the SBEACH simulation associated with the growth of the foreshore slope. However, after the above-discussed modifications of the model, a profile response during the accretionary phase was achieved more in agreement with the measured post-storm profile. The predicted sand volume accreted on the foreshore during post-storm recovery was considerable lower than what was measured, also after modification of SBEACH. Thus, even if the profile response during accretion simulated by SBEACH displays qualitative agreement with measurements in the field, quantitative predictions are still difficult.

PART V: PC-VERSION OF SBEACH

99. In order to facilitate wide-spread use of SBEACH within the U.S. Army Corps of Engineers a generalized version of the model was developed for use on personal computers (PC). A PC-version of the model was previously developed that employed a more simplified description of the nearshore wave conditions, only using shallow-water linear wave theory. The new PC-version utilizes complete linear wave theory including refraction, and several other new features have been added including the possibility of specifying a beach fill and optional choice of describing the sand transport rate in zones of unbroken waves. Furthermore, the numerical method for calculating wave setup on the foreshore was improved by using a more sophisticated approximation of wave-related quantities in this region. The development of the new PC-version of SBEACH is being carried out in close cooperation with researchers at CERC. The user's manual (Cerc 1992) is referred to for details about the PC-version of SBEACH.

PART VI: BEACH PROFILE ANALYSIS PACKAGE

100. A beach profile analysis package called BMAP (Beach Morphology Analysis Package) was developed to facilitate data analysis in connection with laboratory and field studies on beach topography change. Critical data analysis was essential for developing and carrying out the enhancements of SBEACH, especially with respect to modeling the net sand transport rate in different regions along the profile. BMAP encompasses routines for carrying out the following operations:

a. general manipulations of beach profiles such as interpolation, vertical and horizontal translation, in- and output
b. average and equilibrium profile calculations
c. contour movement and volume change
d. transport rate calculations
e. calculations of bar and berm properties such as volume, depth to crest, distance offshore, slopes, height, and speed of movement

BMAP was also used during the large wave tank data collection project "SuperTank" carried out by researchers at CERC for on-line evaluation of beach profile change (Kraus et al. 1992). This project has provided valuable data on beach profile change to be used in further enhancements of SBEACH.

101. BMAP is constructed as a user-friendly software to be executed interactively in a PC-environment, allowing a large number of profiles to be manipulated and analyzed simultaneously. The program routines for the analysis were developed at University of Lund, whereas the user-interface and adaption to the PC environment are done at CERC. A user's manual is under development by CERC, and in the near future BMAP will be distributed to Corps of Engineers District offices for use by engineers in the field.

PART VII: SUMMARY AND FUTURE WORK

102. The following improvements have been made with respect to different calculation modules in the profile response model SBEACH:

1. The wave decay model was generalized to handle transformation of irregular waves over an arbitrary bottom profile.

2. A more complex method for modeling energy dissipation in the surf zone was developed that includes a transport equation for turbulent kinetic energy.

3. Several new landward boundary conditions were implemented including overwash and inundation, and seawall failure.

4. A modified equilibrium beach profile was developed that takes into account a variable grain size across shore.

5. The numerical solution scheme in SBEACH was modified to handle a variable grid with the option to specify a completely arbitrary length step.

6. The description of onshore transport and beach profile accretion was modified to better model profile response during beach recovery.

Furthermore, a special program package called BMAP was developed, initially as a tool for analyzing laboratory and field data on profile change in connection with the development and enhancements of SBEACH, but in the near future it is going to be released as a standalone software for Corps of Engineer users. SBEACH has also been released in a user-friendly PC-version.

103. Although significant improvements have been made of the original version of SBEACH, a continuous update of the model is needed as research progress and better techniques are developed to model physical processes. As pointed out previously, SBEACH consists of calculation modules that are easily substituted when improved methods are derived. Specifically, future work regarding enhancements of SBEACH could involve:

1. Parameterizing the cross-shore flow and sediment concentration profiles in the surf zone, and determining the net cross-shore transport rate from these profiles.

2. Include the effects of long-period waves, especially with regard to foreshore profile response.

3. Develop a robust sediment transport relationship for unbroken waves to be employed outside the surf zone.

4. Develop a robust sediment transport relationship in the swash zone.

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