

FDEP Dune Erosion Modeling Historical Review with discussion of Models Under Review

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Historical Overview of Dune Erosion Models utilized for CCCL Evaluation

The first model to be used for dune erosion purposes in the Setback Line studies (prior to the Coastal Construction Control Line CCCL) was an analytical model as noted by Chiu (1972). This undocumented model was a modified version of a Dutch dune erosion model due to Edelman (1968). Although this model may have been used somewhat in the late 1970's or early 1980's, as best can be determined, this model was not used in the later CCCL studies initiated in the early 1980's. As cross shore transport process research progressed, the first attempts at numerical dune erosion modeling began. Although longshore sediment transport models had been studied for the past five decades, cross-shore sediment transport modeling was relatively recent (about 20 years) and uncertainty in prediction was (and still is) much greater. Cross-shore sediment transport models can be broadly classified into two groups: "open loop" and "closed loop" models. An "open loop" model is not constrained a priori to the final profile and the sediment transport is determined by sediment concentration and fluid velocities. "Closed loop" sediment transport models are based on equilibrium beach profile concepts of Dean (1977), and assume that a profile will eventually achieve equilibrium if exposed to the same conditions for a long time. Cross-shore transport in these models is primarily assumed to be caused by an out of equilibrium beach profile (due to storm tide and waves) attempting to reach a new equilibrium imposed by higher energy storm conditions.

Several "closed loop" numerical models have been under investigation and partially supported by the FDEP during the past twenty years. These models include the original "Kriebel Model" (also known as EBEACH as well as a later "enhanced" model EDUNE (Kriebel, 1982; Kriebel and Dean, 1985; Kriebel, 1986) and the model used in establishing a coastal hazard zone in Florida, termed the Coastal Construction Control Line (CCCL) (Chiu and Dean, 1984, 1986). Another model "SBEACH" (Larson et al., 1989; Larson and Kraus, 1989) developed by the U.S. Army Corps of Engineers has also been reviewed by FDEP for possible use. These models have all been developed utilizing the equilibrium beach profile concepts noted above and typically require a continuity equation and a transport equation that govern the rate at which the processes occur. The conservation equation balances the differences between inflows and outflows from a region as predicted by the transport equation. In addition, boundary conditions must be employed at the landward and seaward ends of the region modeled. These boundary conditions can usually be expressed in terms of a maximum limiting slope such that if the slope is exceeded, adjustment of the profile will occur, a condition sometimes referred to as avalanching. The locations of the seaward and landward boundaries are typically at the limits of wave breaking and wave run-up. Numerical models for which computer codes were available (and considered by the FDEP) at the time of Coastal Construction Control Line (CCCL) reestablishment in various Florida counties included the so-called "Kriebel Model" (Kriebel and Dean (1985), Kriebel (1982, 1986)), and the SBEACH model (Larson (1988), and Larson and Kraus (1989, 1990)). The CCCL Dune Erosion Model developed by Dean (circa 1984) under subcontract to BSRC and approved utilizing funding provided by FDEP was an outgrowth of research during the "Kriebel Model" development and of a numerical implementation of an analytical model developed by Kriebel and Dean (1993).

In numerical modeling, two representations of the physical domain are considered by the various models developed and under review by the FDEP. In the first type, the cells are finite increments

of the offshore distance variable x . Thus the distance is the independent variable and the dependent variable is depth, which varies with time. The SBEACH model as discussed in more depth later is of this type. An advantage of this type of grid is that the presence of bars can be represented with no difficulties. In the second type of grid approach, the computational cells are formed by finite increments of the depth h . In this case, the independent variable is h and x varies with time for each h value and the profile is required to be monotonic (decreasing offshore) for the model to run. A smoothing approach is utilized as a preprocessing step in this type of model to provide the required monotonic profile for the model. The "Kriebel" Model, the "CCCL Dune Erosion" Model, and the "Cross" Model to be discussed below are presently of this type grid system. Models require a continuity equation and a transport equation to drive the numerical solution regardless of the grid scheme employed as will be discussed.

Discussion and Technical Overview of Utilized Cross Shore Models

Kriebel Model

The numerical model described by Kriebel and Dean (1985) and Kriebel (1982, 1986, 1990), was the first widely used numerical model developed with some FDEP funding support to simulate storm-induced erosion based on Dean's(1977) equilibrium beach profile (EBP) concept. Much of the following discussion is derived from these references. This EBP concept states that most equilibrium profiles correspond to uniform energy dissipation per unit volume with the scale of the profile represented by the parameter "A" which depends primarily on sediment characteristics and secondarily on wave characteristics, i.e.

$$h(x) = Ax^{2/3}$$

The parameter, "A", and the uniform wave energy dissipation per unit volume, "D*", are related for linear spilling breaking waves. The "Kriebel Model" assumes that a beach profile will evolve toward an equilibrium form in response to changing water levels and wave conditions and that the sediment transport rate is proportional to the disequilibrium existing between the time evolving actual profile and the equilibrium profile form at the elevated surge water level at that instant in time. This is quantified in terms of the excess energy dissipation per unit volume in the surf zone. The energy dissipation per unit volume at any location in the surf zone is based on the assumption of shallow-water breaking waves. The "A" parameter in the Dean EBP theory is given by the equation

$$h(x) = Ax^{2/3}$$

and is determined either from the sediment grain size, or from a best-fit of the equilibrium profile equation to the measured beach profile.

In the EBEACH program the user must initially input the schematic beach-dune profile characteristics, from which the discrete form of the initial profile is established. The main program consists of a DO loop in which each iteration represents one time step, simulating one-half hour of real time. At the beginning of each iteration, the water level and wave height for the time step are entered into the program from a data file. From the updated water level and wave height, the surf zone and active onshore profile limits are established according to the wave breaking depth and the location of the new water level relative to the onshore portion of the profile. With the boundary

conditions defined by the dune height and the breaking depth (determined from wave height), the active profile is divided into two regions:

- (1) A dynamic region below the water line, in which profile changes are governed by the calculated energy dissipation and continuity, and
- (2) The geometric region, above the water line, in which profile changes are governed by continuity only.

First, the energy dissipation per unit volume and the sediment transport function are determined to calculate profile changes in the dynamic region. Starting from an onshore boundary condition for sediment transport, an offshore sweep is used to determine coefficients in the governing equation; then, starting from an offshore boundary condition, defining the limiting depth of profile change, an onshore sweep is used to determine the horizontal change for each point in the profile up to near the water level (existing in the model at that time). Landward of that point, the profile is governed by continuity according to changes in the offshore profile and the limits of onshore sediment motion. Similar to storm surge-flooding models that “flood” land by opening and closing discrete cells, the EBEACH model activates erosion in portions of the onshore profile by opening and closing sections of the composite beach-dune face. The procedure is based on the elevation of the water line and the profile shape, principally depending on the berm width. Offshore, at the breaking depth, a loop is included to geometrically maintain a smooth transition between the active surf zone and the adjacent offshore profile. The final profile changes over the time step resulting from the dynamic and geometric solutions, are recorded as the incremental retreat or advance of each contour line. Updated profile information is then used as the initial condition for the next time step and the process is repeated for the duration of the storm surge. Two types of storm surge input may be used:

- 1) The actual measured storm surge hydrograph.
- 2) The calculated storm surge hydrograph resulting from a numerical storm surge model

For standard model application, numerical storm surge predictions can be input directly into the model. It should be noted, however, that model does not allow dune overtopping, therefore, the model should only be used in cases where dune crest is higher than peak storm surge elevation.

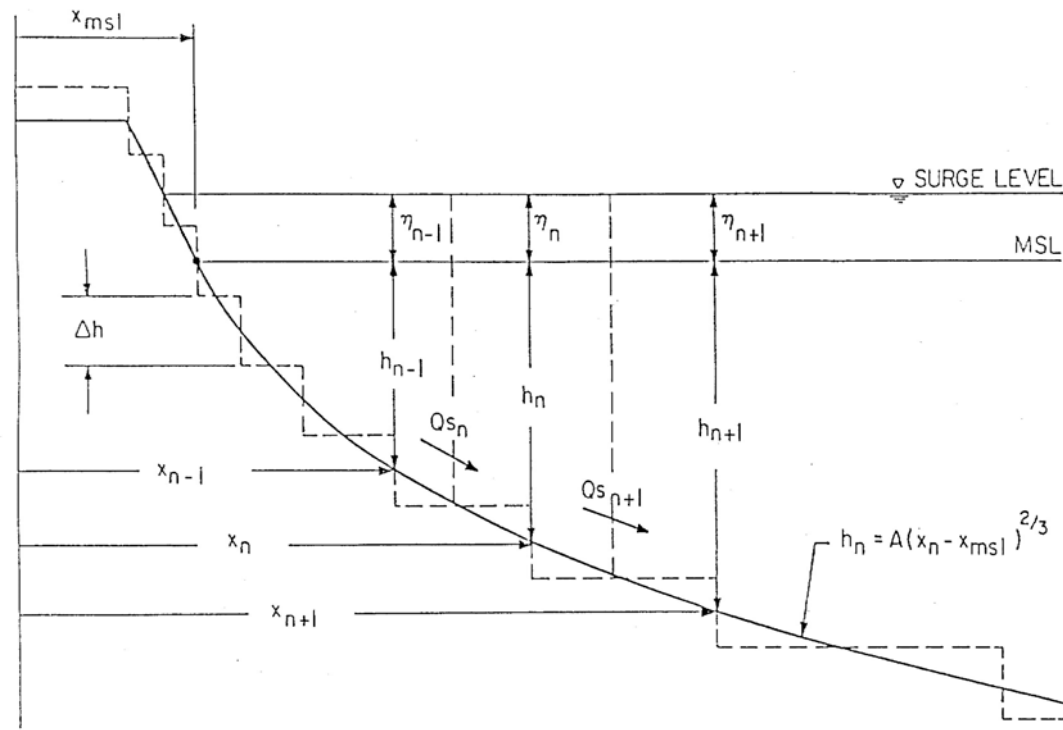
Two methods may be used to input the wave height:

- 1) The observed, hindcast, or forecast wave height can be read at each time step from a data file.
- 2) A constant representative wave height can be used.

The offshore profile shape is governed by two parameters, the shape factor, A , and the equilibrium energy dissipation per unit volume, $DISSE$. For a given sand grain size, these values may be determined by fitting the EBP curve to the actual profile. Onshore, the composite beach-dune profile is defined by the height and horizontal location of each break in slope. Generally, a representative beach face slope, and dune face slope, are determined for the profile of interest. The dune height, the berm height, and the depth of intersection, must be specified. To establish initial horizontal locations of various points, the profile must be classified into one of two types, a wide berm profile or a no berm profile.

To establish the initial profile shape, the elevation of each grid point is first established. From the initial elevation, the initial location of each grid point, is also determined according to the profile specifications above. The numerical solution for profile response is based on a finite difference solution to the sediment conservation equation and transport equations. In this case, the profile is

gridded in a stair-step form as shown below so that erosion or accretion (retreat or advance) of each elevation contour is determined by vertical gradients in the sediment transport rate.



Model Representation of Beach Profile, Showing Depth and Transport Relation to Grid
(from Kriebel (1982))

The profile was schematized as per above in a series of depth contours, h_n , the locations of which are specified by coordinates, x_n , measured from an arbitrary baseline. The profile is thus inherently monotonic and at each time step, the x_n values of each of the active contours are updated.

As noted previously, in most transport problems there are two governing equations. One is an equation describing the transport in terms of a gradient or some other feature. The second is a continuity or conservation equation (continuity in the cross shore direction) which accounts for the net fluxes into and out of a grid cell. As discussed previously, the cross-shore transport is primarily a function of the excess energy dissipation per unit volume in the "closed loop" type models.

A number of methods could be employed for solving the transport and continuity equations. For example, explicit methods would be direct to program, however, the maximum allowable time increment would be relatively small resulting in a program expensive to run. Implicit methods are more difficult to program, but have the desirable feature of remaining stable with a much longer time step. Because of the original planned application to long-term simulation for a 500 year time

period with approximately three hundred storms to be modeled, each with an erosional phase of six to twelve hours, an implicit method was adopted. The method utilized was as noted previously, a double sweep approach in which the Q (transport value) and the x_n values are updated simultaneously at each time step. For h values of 1 ft, and a time step of thirty minutes, the system of equations was found to be stable. The boundary conditions used were somewhat intuitive. At the shoreward end of the system, erosion proceeded with a specified slope above a particular depth, h^* . The depth, h^* , is the depth that the equilibrium slope and the slope corresponding to the beach face are the same. Thus a unit of recession of the uppermost active contour causes an erosion of the profile above the active contour that is "swept" by this specified slope. This material is then placed as a source into the uppermost active contour. The offshore boundary condition is that the active contours are those within which wave breaking occurs. If an active contour extends seaward, thereby encroaching over the contour below to an extent that the angle of repose is reached, the lower contour (and additional lower contours if necessary) are displaced seaward to limit the slope to that of the angle of repose. At each time-step, the local depth h is the depth to the sand bed below the time-varying storm surge. As a result, beach profile change is driven primarily by changes in water level associated with storm surge. Breaking waves are treated by assuming shallow-water, spilling-breakers and, have a secondary effect on profile response. The breakpoint serves to separate the two main computational domains in the model: the offshore region, where the sediment transport rate is assumed equal to zero, and the surf zone, where the transport rate has been discussed previously. In this model, an increase in water level due to storm surge allows waves to break closer to shore (due to the nature of the profile form), thus temporarily decreasing the width of the surf zone and increasing the energy dissipation per unit volume above the equilibrium level. This leads to offshore directed sediment transport, the gradients of which cause erosion of the foreshore, deposition near the breakpoint, and an overall widening of the profile toward a new equilibrium form. At each time-step in the finite-difference solution, the profile moves toward an equilibrium, but this is rarely achieved due to the limited storm durations.

The transport relationship used in the "Kriebel Model" requires calibration of a single empirical sediment transport parameter K based on the cross shore sediment transport equation

$$Q = K(D - D_{eq})$$

This transport relationship proposed by Kriebel and Dean(1985) uses an equilibrium beach profile (EBP) formulation consistent with uniform wave energy dissipation per unit water volume " D_{eq} " in the surf zone as noted previously. This equation is not rigorously based on the mechanics of sediment mobilization or transport; however, it does provide a simplified mechanism for estimating sediment transport rates and it ensures that the profile will respond toward equilibrium. In addition, the equation is similar to transport relationships used by Bakker (1968), Swart (1974), and Perlin and Dean (1983), all of which estimated the rate of offshore sediment transport in terms of the difference between actual and equilibrium profile geometries. Kriebel and Dean adopted this transport relationship in the form such that at equilibrium $D = D_{eq}$, the previously discussed power law equilibrium beach profile would result given sufficient time and a constant surge level. From dimensional analysis, the transport rate parameter, K , must vary according to some length scale. In the equation above the length scale has been assumed consistent with a linear relationship to the profile disequilibrium. More will be noted later concerning this particular linear relationship (which is also assumed in the SBEACH model).

Because of this simplified form, the driving “forces” in the model are provided by the increased water levels in the surf zone. The breaking wave height must be specified to provide an offshore boundary condition limiting the width of the active surf zone. As a result, the EDUNE model is applicable only to severe storms where the nearshore water levels are significantly elevated relative to mean sea level. In the EDUNE model, as mentioned previously, the continuity equation is cast into an implicit, space-centered, finite difference form, e.g. Kriebel and Dean (1985a). By numerically integrating this equation, together with the expression for sediment transport rate, the change in position of elevation contours across the beach profile may be estimated over a given time step. In this case, the actual energy dissipation per unit volume, D , and the resulting sediment flux, Q , vary for each elevation contour depending on the water level, wave height, and profile form at the beginning of each time step. The time-dependent profile response for an arbitrary storm surge may then be simulated by simply varying water levels and wave heights at each time step for the duration of the storm event. Detailed descriptions of the original EBEACH model development are given by Kriebel (1982) and by Kriebel (1984a, 1984b), as well as by Kriebel and Dean (1984, 1985a). In these publications, numerical results from the EBEACH model are shown to agree qualitatively with erosion characteristics of natural and laboratory beach profiles for a variety of water level, wave height, beach slope, and sediment characteristics.

Moore (1982) had previously developed a numerical model based on the same governing equations and found a value of $K = 0.001144 \text{ ft}^4/\text{lb}$. This value was originally adopted for use in the EBEACH model. A preliminary evaluation of the EBEACH model was made with prototype data from beach and dune erosion associated with Hurricane Eloise for Bay County, Florida. Although no tide gage records existed for the open coast of Bay County where the maximum storm surge occurred, it appears that the peak surge height was 2.4-3.7 m (Burdin, 1977; Chiu, 1977; Dean and Chiu, 1982). For the erosion simulation, the storm surge hydrograph was reconstructed with a numerical storm surge model based on the Bathystrophic Storm Tide Theory of Freeman et al. (1957) using the offshore bathymetry of the Bay County area. A storm surge hydrograph was produced for the west end of Bay County using the 1 D numerical storm surge model. The produced storm surge was characterized by a rapid rise and fall of the water level as is typical of a fast moving hurricane and the peak surge height of 3.2 m agreed well with other numerical hindcasts (Kriebel 1984). Beach profiles for Bay County show natural variation in terms of dune height, beach slopes, and berm-dune configurations, however, Chiu (1977) and Hughes and Chiu (1982) gave several schematic profile forms, from which a representative profile form was defined for the western portion of Bay County where high dunes are present. The initial pre-storm profile was classified as a concave profile, with a high dune and no well defined berm. The representative profile had an average dune height of 5.2 m and a break-in-slope at a vegetation line at approximately 2.1 m. Average dune slopes were 1:2; average beach slopes steep and typically on order of 1:10 to 1:13. The offshore profile was characterized by the Dean (1977) EBP relationship in which h is the water depth at a distance x from the intersection of the still water line with the beach profile. The equilibrium profile in Bay County was characterized by an "A" value consistent with 0.26 mm sand. For the profile chosen, 20 modified profile test cases were simulated:

- (1) to evaluate the validity of the method relative to observed erosion; and
- (2) to display the sensitivity of the solution to changes in basic parameters.

Since exact values of input parameters were not known, various combinations of two peak surge levels (3.2 and 3.5 m), three beach slopes (1:10, 1:13 and 1:15), two dune slopes (1:2 and 1:4) and

two wave heights (2.3 and 4.6 m) were included. The maximum erosion was shown in terms of volumetric erosion and in terms of contour change. Upon inspection, the range of calculated erosion magnitudes agreed with observed values fairly well. In the 20 test profile cases, volumetric erosion varied from 20.8 to 38.4 m^3/m , compared to average observed values of 18.3 to 20.4 m^3/m for Bay and Walton counties respectively and an average of 25.1 m^3/m near the area of the peak surge. While the predicted values were somewhat larger than observed, actual post-storm profiles reflect partial recovery of the beach face. Chiu (1977) noted the presence of a ridge of sand on the beach face and suggested that 5 m^3/m had been returned to the beach face at the time of the post-storm survey. Accounting for this additional sand volume, observed average erosion values would vary from 23.3 to 30.1 m^3/m . It appeared that the numerical solution accounted for time-dependent erosion in such a way that the magnitudes of the eroded volumes were in general agreement with field data of average volumetric erosion. The beach profile response as measured by contour advance or retreat was also considered in the study. An eroding berm or dune face steepens considerably approaching a vertical slope due to undermining and slumping as evident in observed profiles from Bay and Walton counties, presented by Chiu (1977). In the numerical model, the dune face was approximated by a linear slope which was maintained as the dune eroded. The model solutions showed a nearly equal recession of the 3.0 and 4.6 m contours; although there was a small difference since the peak surge level was slightly greater than 3 m. Since no attempt was made to simulate dune steepening, numerical results did not strictly match observed dune retreat profiles but because the volume of material eroded was essentially conserved, the uniform dune retreat represented an average retreat of all dune contours that compared favorably with the average recession of the 3.0 and 4.6 m contours suggesting that the model results were of the correct order of magnitude.

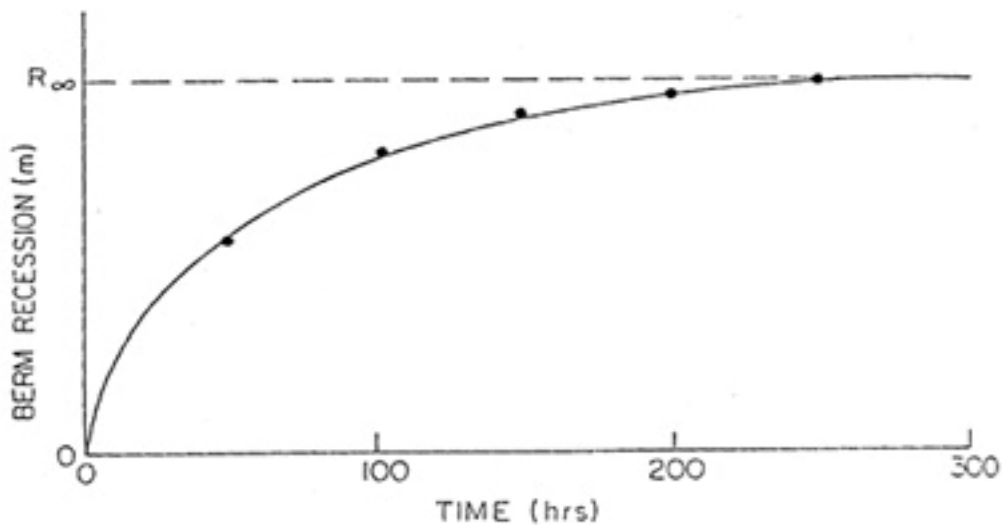
This study was the only "calibration" accomplished for the EBEACH model for FDEP purposes and was considered preliminary for several reasons. The first reason for considering the study to be preliminary was that no effort was made to simulate the detailed response of individual pre-storm profiles or to compare individual predicted and observed post-storm profiles. Instead, erosion was simulated and compared with 20 representative profile scenarios having profile characteristics from the Bay-Walton area as reported by Chiu (1977). A second reason for considering the study to be preliminary was that the rate parameter, K , was adopted as the value determined by Moore (1982) as opposed to fitting to the data. As a result of the stated two reasons, the EBEACH model was not considered to have undergone rigorous calibration. Despite the lack of rigorous calibration or verification, the EBEACH model was subsequently adopted by the Florida Department of Natural Resources (see Kriebel (1984a, 1984b)) for use in reestablishing the CCCL in coastal counties. It is not known as to which model (CCCL Dune Erosion Model or EBEACH) was used in early CCCL reestablishment county studies although sometime in the mid 1980's it appears that the CCCL Dune Erosion Model was exclusively utilized for dune erosion work and CCCL line reestablishment. Reasons for the possible preference of the later developed CCCL Dune Erosion Model over the EBEACH model may have been "ease of use", "robustness", "less subjectivity in profile input", and "better calibration".

The original "Kriebel Model" does not simulate bar and trough features as has been noted previously. An early version of the "Kriebel Model" was provided in the original U.S. Army Corps of Engineers ACES software package (Leenknecht, Szuwalski, and Sherlock (1992)). A more

detailed discussion of the Kriebel Model development is provided in Chiu and Dean (2002) along with references provided in the "Kriebel" Model section of the listed references. Source code was available for the model at the time of the original CCCL related work on the model and User Manuals for the software are as provided in Kriebel (1984a, 1984b). A later version of the model "EDUNE" which was an "enhanced" version of the original "Kriebel" Model incorporating limited wave runup and overtopping as well as a number of other improvements. The EDUNE model was never utilized in the CCCL studies although it was reviewed in a report by Wang and Chiu (1993) in connection with the Palm Beach County Peer Review Group suggestion that the model be considered as it provided less dune erosion than the CCCL Dune Erosion Model. Findings from the Wang and Chiu (1993) review of the EDUNE model suggested that a larger K value was needed in the EDUNE Model to make model predictions in better agreement with measurements for Palm Beach County storm eroded profiles.

Analytical Model

An analytical model (supported by the FDEP) for predicting dynamic profile response during storms was used as a basis for the CCCL Dune Erosion Model that evolved to become the primary erosion model used by the FDEP. The analytical model given in Kriebel and Dean (1993) provided the basis for the CCCL model approach but has never been utilized for CCCL establishment. Much of the following discussion is derived from the above reference. The analytical method approach is based on the observation that beaches tend to respond toward a new equilibrium in an exponential fashion over time. This exponential response is supported by laboratory data. An example of an exponential response to recession is provided in the figure below.



Comparison of Asymptotic Berm Recession from Model (—) and as Calculated

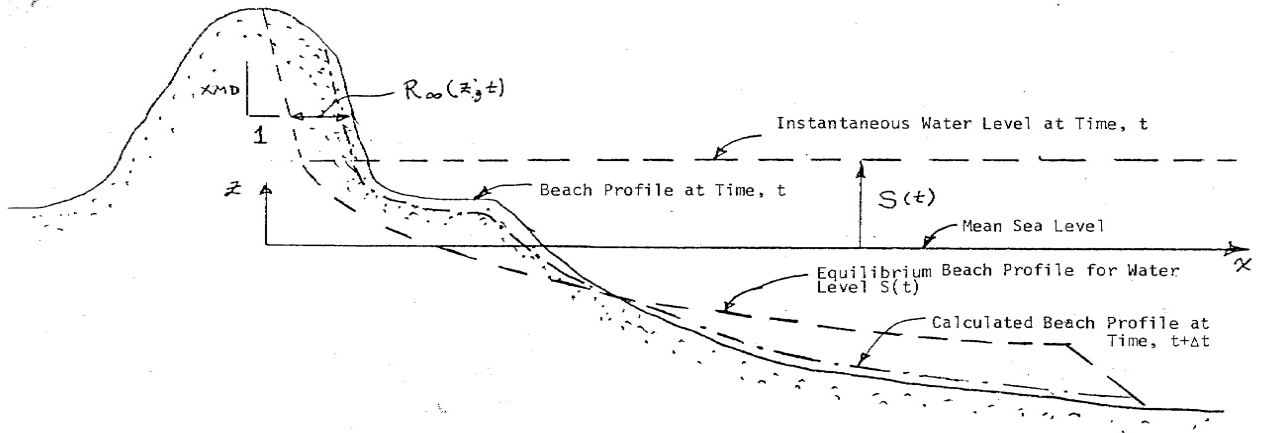
For laboratory conditions, where a beach is suddenly subjected to steady wave action, a time-dependent shoreline (contour) response $R(t)$ may be approximated by a functional form dependent on R_{∞} where R_{∞} is the equilibrium beach response to the maximum storm level at

$t \rightarrow \infty$. This functional form is dependent on "TS", the characteristic time-scale of the system which is dependent on wave and beach characteristics. An exponential response of $R(t)$ toward R_∞ has been observed in numerous wave tank experiments as noted previously. The general solution to recession from this approach to modeling may be expressed as a solution to a convolution integral that is storm hydrograph dependent along with knowledge of the response time determined from "TS" and the equilibrium beach recession value. Because of the exponential response characteristics of the beach system, the beach response will be damped so that the actual maximum response will be less than the maximum erosion potential of the storm if a step function storm surge existed. The morphological time scale relationship for "TS" in this approach was estimated from numerical erosion calculations utilizing the "Kriebel" Model. If the maximum erosion at a given level can be determined from knowledge of the beach system and a step function storm surge, then values of $R(t)$ at various levels can be determined using this technique. As noted previously, the CCCL Dune Erosion Model is a numerical approximation to this type of approach utilizing the actual initial beach profile along with an actual storm hydrograph.

CCCL Model

The CCCL model was developed in the 1981-1982 period, and for purposes of consistency, has not undergone major modifications. This model has been used in establishing the recommended location of the Coastal Construction Control Line (CCCL) in 22 of the 24 coastal counties in the FDEP CCCL program. At the time that the CCCL model was developed, no other models were available that could be applied readily without requiring considerable judgment in the preparation of the input data and/or without the possibilities of instabilities which would raise questions regarding the output. Thus, in its development, priority was given to a model which was robust and simple in concept and that could be run with limited exercise of judgment in input preparation. It is felt that these requirements are still of importance in model choice by the FDEP. It has always been recognized that at some time in the future, the model would need updating either through modification or replacement to reflect advances in the state of cross shore model art. For computational ease and economy in establishment of the CCCL line, a simple numerical erosion model was developed utilizing results from both the Kriebel Model and the Analytical Model noted above. This numerical model has evolved to the existing CCCL model utilized today by the FDEP. This model is not physics-based in its hydrodynamics or sediment transport, yet it retains the overall characteristic response described previously in the Kriebel model. The model is based on the characteristic response exhibited in the figure below and the exponential response noted previously in the analytical approach which is

$$\frac{R(t)}{R_\infty} = (1 - e^{-Kt})$$



Features of Simplified Beach Erosion Model

Because the CCCL model tracks the displacement of various elevation grids similar to the methodology in the "Kriebel Model" discussed previously, it requires a monotonic profile as input. Bars cannot be accommodated in the input profile and therefore a "smoothing" subroutine has been developed that maintains all of the sand volume at the same elevation as occurs at its original elevation. This was accomplished by displacing landward those elements that are seaward of and at the same elevation of more landward elements. The smoothed input profile is part of the CCCL model program output. The basis for the CCCL model is that, as in many other physical processes, any contour responds to disequilibrium by approaching the equilibrium position in a manner that is rapid at first and much slower as its equilibrium position is approached. Based on numerical modeling with the "Kriebel Model" (a forerunner of the EDUNE model), it had been established that this response could be expressed approximately as

$$x_i^{n+1} = x_i^n + \Delta x_i^n (1 - e^{-K \Delta t})$$

in which

$$\Delta x_i^n = \Delta x_{eq_i}^n - x_i^n$$

where i denotes the contour level, K is the rate constant, and Δt is the time increment between the n and the $(n+1)$ time levels.

All hydraulic and sand flow models require a "dynamic" transport equation and a "continuity" equation. The dynamic equation controls the rate and direction of the local flows and the continuity equation accounts for the differences between the flows into and out of a grid and changes the volume within that grid accordingly. In the CCCL model, the first equation shown above fulfills the role of a dynamic transport equation. The continuity requirement ensures that the volumes are conserved from time step to time step and is accomplished at each time by first locating the vertical origin of the "target" profile at the current water level and then shifting the target profile a horizontal distance such that the net sand volume change between the two profiles is zero. This step

establishes Δx_i^n as per the second equation noted above. For each level, the differences in the x values of the target (equilibrium) and actual profiles is the quantity Δx_i^n . The first equation is then applied for all active contour levels to update the entire profile. This procedure is continued from the initial profile until the final computational time. At the end of this stage, there are no more adjustments to the calculated profile.

At each time step, for the instantaneous value of the time-varying water level $S(t)$ and the selected breaking wave height, H_b , the equilibrium profile and the associated equilibrium recession, $R(t)$, from the existing profile is established. The characteristics of the equilibrium profile are that:
 (1) the eroded volume above the "hinge point" equals the deposited volume below the hinge point;
 (2) the equilibrium profile is in accordance with the scale parameter "A" either specified by the grain size, or determined as a best-least squares fit to the measured profile; and,
 (3) the equilibrium profile above the instantaneous water level is characterized by a uniform slope, XMD, on the order of 2 or 3 as determined by post-storm profile measurements following Hurricane Eloise and other storms. With the equilibrium profile established, the erosion occurring from time t to time $t + \Delta t$, Δx is given by

$$\Delta x = x(t + \Delta t) - x(t) = -R_\infty (1 - e^{-K\Delta t})$$

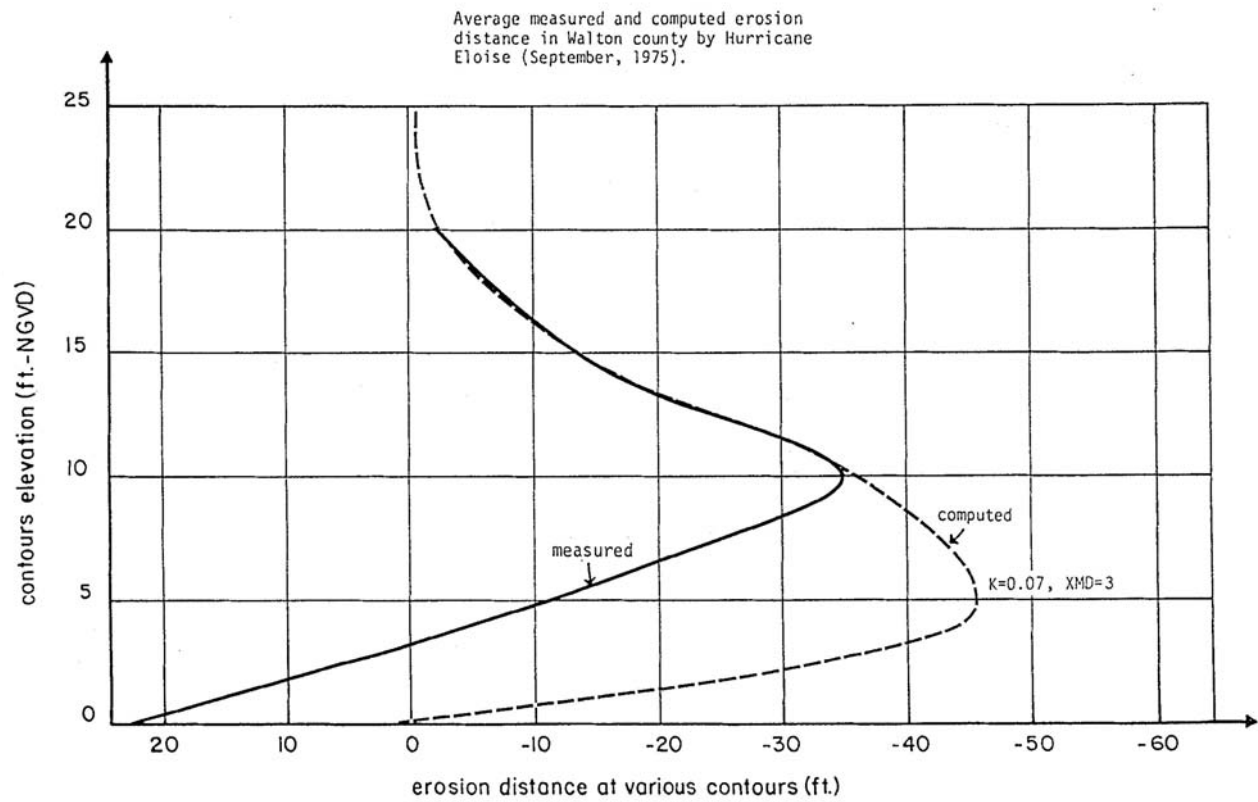
The CCCL Dune Erosion Model can accommodate time varying forcing, i.e. variations of wave height and storm tide with time. As noted previously, the erosion model accounts only for cross shore sediment transport. Through comparison with the average erosion occurring in Hurricane Eloise as discussed by Chiu and Dean(1984) in a CCCL methodology report (see figure below), a $K = 0.075 \text{ sec}^{-1}$ value was found based on the mean profile erosion (see figure below) found in a representative high dune area of Bay and Walton Counties (Chiu 1977).

There are a number of factors which result in considerable variability about the mean recession value of any given elevation contour. Probably the greatest causes of this variability are the gradients in longshore sediment transport as a result of the relatively small scale of the hurricane system, and natural variability due to inhomogeneities in the system (sand size, consolidation, offshore bay system, etc.). Regardless, the variability of erosion was recognized and documented by Chiu (1977) in Hurricane Eloise where it was found that the maximum erosion at the 10 and 15 foot contours was approximately 2.5 times the average erosion at those levels. This 2.5 factor (which might be considered as a safety factor) was incorporated in the erosion considerations employed in the recommended CCCL positions by modifying the previous equation as follows

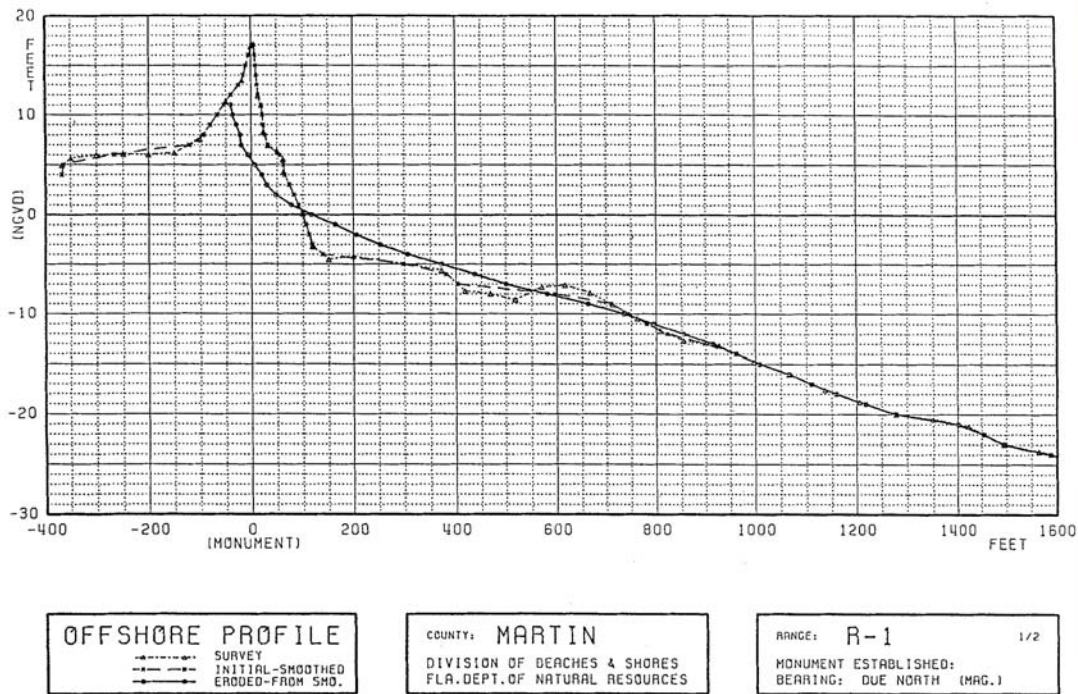
$$\Delta x = -(2.5) R_\infty (1 - e^{-k\Delta t})$$

The 2.5 safety factor became a general aspect of the CCCL model to account for profile longshore variability. Further checking of the model with limited after storm profiles from two other storms is reported on in Chiu and Dean (1986). After Hurricane Opal hit the Florida Panhandle, a further (undocumented) calibration was made of the CCCL model which adjusted the erosion safety factor of the lower contours for considerations of vertical erosion and establishment of design elevations

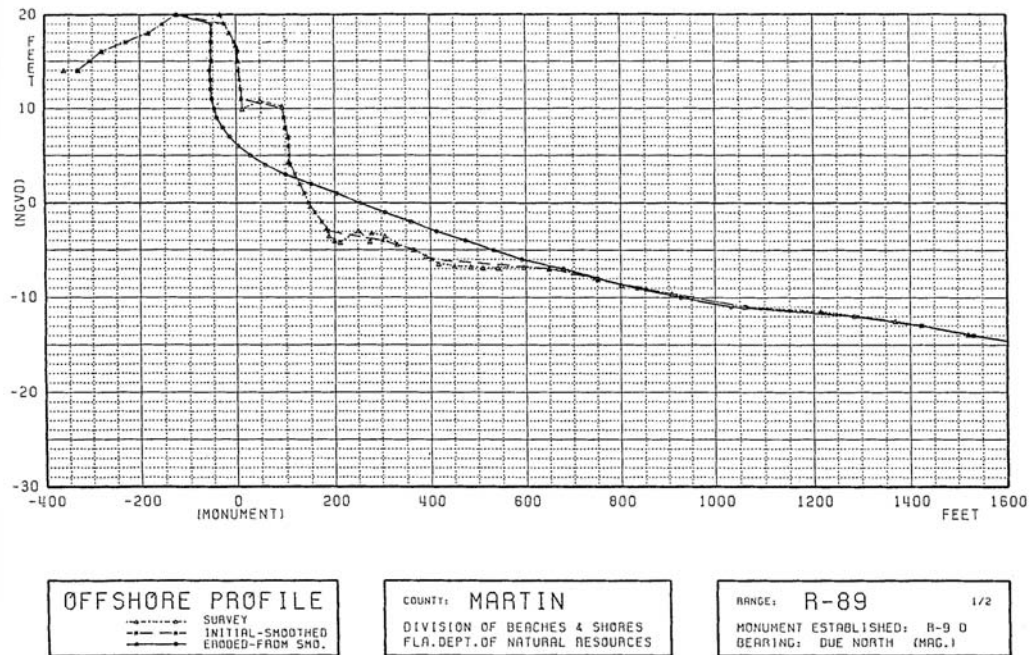
for the lowest horizontal structural member (LHSM) of habitable structures located Seaward of the Coastal Construction Control Line (S.Y.Wang-personal communication). This erosion safety factor work changes the model output in the lower elevation contours for use in setting design elevation for lowest horizontal structural member building requirements and was used in establishment of the existing building requirements documented in the following report: Guidelines for Design Elevations Seaward of the Coastal Construction Control Line, Florida Department of Environmental Protection, Office of Beaches and Coastal Systems, (undated report). This variable elevation safety factor modification to program was never utilized for CCCL reestablishment work but only as noted previously for development of building requirements for structures located seaward of the CCCL. Since the initiation of the first CCCL model (originally termed the DNR model as per Chiu and Dean(1984) original CCCL Methodology report), the model has undergone only minor (undocumented) revisions. A separate High Frequency version of the model (which incorporates predefined storm surge information) has been developed for use where erosion return periods are of 25 years or less. Documentation for the High Frequency version of the CCCL model is provided in Dean and Malakar (1995).



Average computed (with 2.5 factor) and measured erosion at various contours in Walton County from Hurricane Eloise



Results of Applying Erosion Model to Range R-1, Martin County (Hutchinson Island), 100 Year Storm Tide, Average Erosion



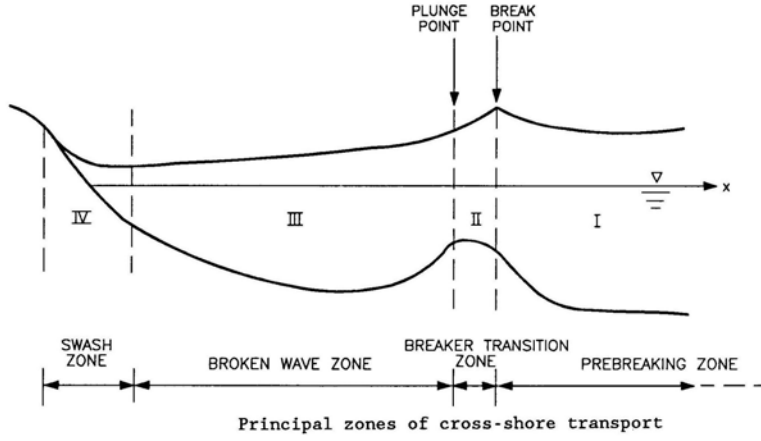
Results of Applying Erosion Model to Range R-89, Martin County (Jupiter Island),
 100 Year Storm Tide, Average Erosion

SBEACH Model

The SBEACH numerical model (Larson (1988) and Larson and Kraus(1989, 1990)), is similar in some ways to the model of Kriebel and Dean(1985) but contains a more detailed description of breaking wave transformation and sediment transport across the beach profile, especially near the breakpoint as well as empirical relationships for sediment transport based on bar formation.

Much of the following discussion is derived from the above noted references. As a result of the more empirical nature of the SBEACH model and it's capability to represent a bar feature, additional parameters are needed in the model beyond just the K transport factor as per the "Kriebel model". SBEACH approximates the equation for conservation of sand in finite difference form based on the profile grid of the type where vertical changes in water depth are determined by horizontal gradients in sediment transport rate. In contrast to the "Kriebel model", this allows simulation of breakpoint bar formation and evolution. Sediment transport rates in the surf zone are generally determined in terms of excess energy dissipation, but with an additional effect of the local bottom slope that comes into play through an additional parameter " ε ". Because of this additional term, early versions of the model require calibration through adjustment of two parameters, K and ε making for more difficult calibration. Additional limiting slope values are also required as for the "Kriebel" Model. The breaking wave model employed in the Larson and Kraus model has more sophisticated wave breaking than that used by Kriebel and Dean (1985), and is based on the breaking wave model of Dally, Dean, and Dalrymple (1985). This breaking wave model introduces gradients in the breaking wave height and energy dissipation that, in turn, lead naturally to gradients in sediment transport that produce bar/trough formations. Because of this improved breaking wave model, beach profile changes can be driven by changes in wave conditions, in addition to changes in water level.

The computational domain used in the Larson and Kraus model is divided into four regions across the beach profile and the exact sediment transport relationship is adjusted somewhat for each of the four regions. In the surf zone, which is the major region for cross-shore sediment transport in the model, transport directions are first determined from the following critical value of wave steepness. At each time-step in the solution, if the actual value of wave steepness exceeds an empirically determined critical value, then transport is directed offshore over the entire active profile. Transport is onshore if wave steepness is smaller than this critical value. Transport magnitudes in the surf zone are turned off if the wave energy dissipation is below a critical value since this might cause a reversal in transport direction in conflict with the transport direction determined from wave steepness. It is noted, however, (at least in early versions of the model) that the relationship determining transport direction does not include any of the profile characteristics. In some cases where the profile is very steep, as may occur after beach nourishment, the transport direction utilized in the model may be onshore based on wave and sediment characteristics whereas the actual transport would likely be offshore, even under mild wave conditions, due to the artificially high profile slope. The figure below shows the SBEACH transport zones where it can be seen that Zone II is very similar to the transport in the "Kriebel Model" except with the addition of an extra bottom slope term requiring an additional parameter.



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

$$\text{Zone II: } q = q_p e^{-\lambda_2(x-x_p)} \quad x_p < x \leq x_b$$

$$\text{Zone III: } q = \begin{cases} K \left[D - D_{eq} + \frac{\varepsilon}{K} \frac{dh}{dx} \right] & D > \left[D_{eq} - \frac{\varepsilon}{K} \frac{dh}{dx} \right] \\ 0 & D \leq \left[D_{eq} - \frac{\varepsilon}{K} \frac{dh}{dx} \right] \end{cases}$$

$$\text{for } x_z \leq x \leq x_p$$

$$\text{Zone IV: } q = q_z \left[\frac{x - x_r}{x_z - x_r} \right] \quad x_r < x < x_z$$

where

- q = net cross-shore sand transport rate, $m^3/m\text{-sec}$
- $\lambda_{1,2}$ = spatial decay coefficients in Zones I and II, $1/m$
- x = cross-shore coordinate directed positive offshore, m
- K = sand transport rate coefficient, m^4/N
- D = wave energy dissipation per unit water volume, $N\text{-m}/m^3\text{-sec}$
- D_{eq} = equilibrium wave energy dissipation per unit water volume, $N\text{-m}/m^3\text{-sec}$
- ε = slope-related sand transport rate coefficient, m^2/sec
- h = still-water depth, m

SBEACH transport zone expressions.

In SBEACH, the transport rate coefficient K , and the coefficient for slope-dependent term, E_{ps} are calibration parameters which can be changed in the SBEACH model GUI. Detailed technical information relating to these variables is given in Larson and Kraus (1989) and Larson et al.(1990). Larger K values produce greater sand transport and more prominent bar features, and larger values of E_{ps} produce a more subdued bar. The transport rate seaward of the breaker line and from the wave reformation point to the next break point (if wave reformation occurs) decays according to the transport rate decay coefficient multiplier which can also be adjusted in the SBEACH model. Larger values of the parameter result in more rapid transport rate decay as noted in Larson and Kraus (1989) and Larson et al.(1990). An overwash transport parameter has also been included in the recent versions of SBEACH to account for overwash caused by wave run-up (not inundation). Overwash is discussed in a CETN on Coastal Overwash noted in the SBEACH reference section.

SBEACH has been compared with laboratory data from two sets of large wave tank tests in the United States as well as wave tank tests in Japan and Germany. The regular waves used in one of the United States wave tank tests (Saville(1957)) were monochromatic, which tend to favor well-developed and concentrated bars that are simulated quite well by the numerical model. The figure below shows results for comparisons of specific lab test calibrated runs of SBEACH to the Saville (1957) data [upper panel] and Japanese wave tank tests [lower panel]. Another SBEACH comparison to Duck, N.C. field data which has a bar is provided in the figure below. In this comparison, the evolution of a profile over a 3 day period was compared with model runs of SBEACH calibrated for this specific data set. The simulations provide a reasonable qualitative representation of the evolution but also show the inability of the model to completely simulate the actual event with high accuracy. On the other hand, SBEACH can simulate a bar feature which other models cannot do. As the FDEP does not require accuracy of prediction of subaerial features, the ability to deal with bar features is not as important to FDEP concerns as is the ability to gage storm erosion in the above water portion of the profile.

SBEACH has been tested for a number of field data sets. The Duck, N.C. data set mentioned above is probably the most extensive and well tested data set due to the high survey accuracy of the four cross shore transects considered, but was noted by Larson et al.(1990) not to be useful for testing of dune erosion. Other field data sets tested and reported on in various SBEACH reports include:

Torrey Pines, CA. (3 cross shore transects for 2 storm events);

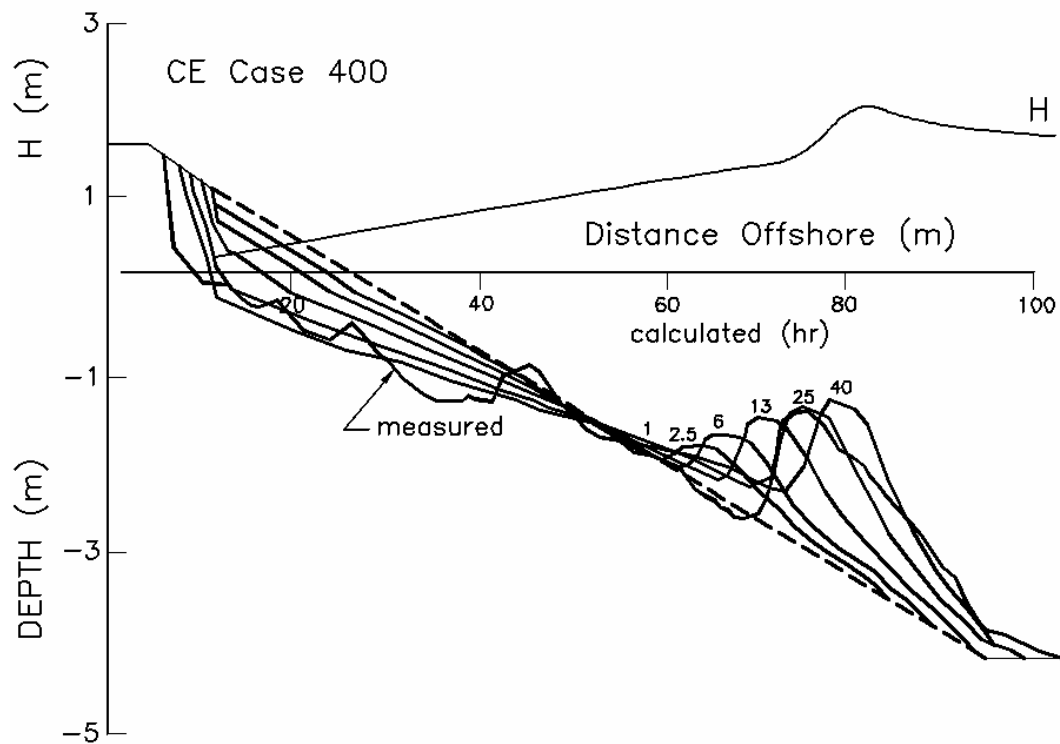
Manasquan, N.J. and Point Pleasant, N.J. (17 cross shore transects for a storm)

Ocean City, MD. (various profiles for a storm event).

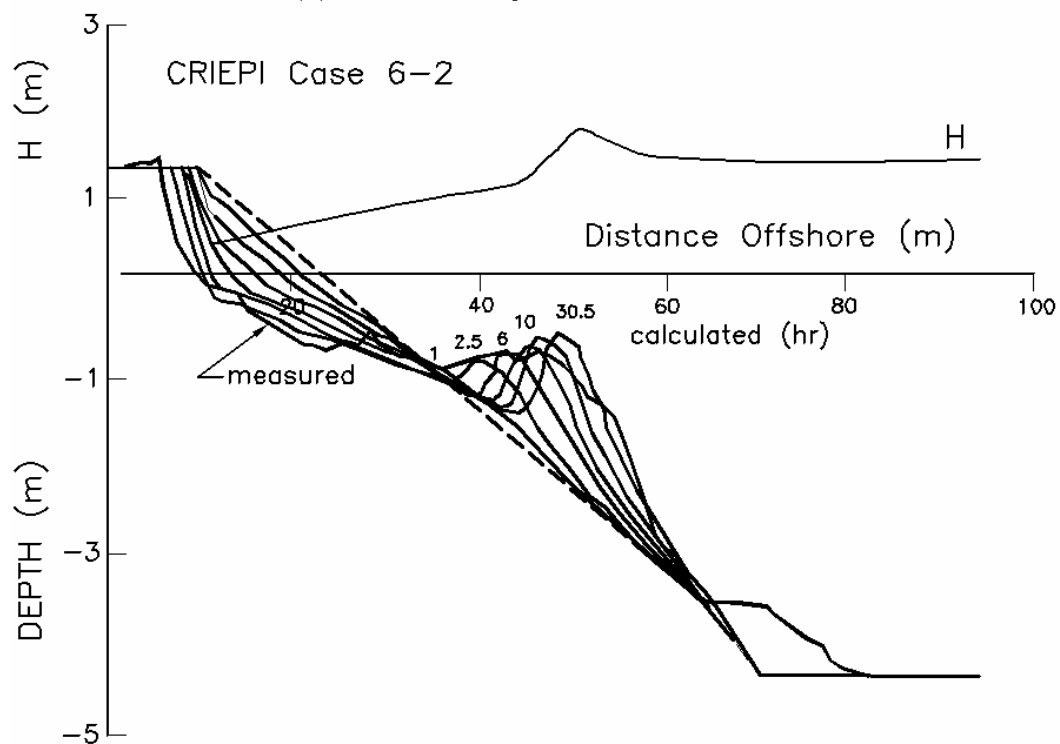
which show reasonable results with the calibration factors utilized for the individual studies.

SBEACH provides overwash capabilities as noted previously, but only to the extent that the overwash is caused by wave run-up and not by actual water flowing over a barrier island (i.e. barrier island inundation). When water elevation reaches the top of the high point on profile, the limited physics utilized in SBEACH is no longer valid. As overwash scenarios that are of most importance to the FDEP involve barrier inundation, overwash capability as provided by SBEACH is not thought to be a vital factor in determination of a model for FDEP usage. SBEACH does provide a boundary condition in the model that allows for a hard bottom at which no erosion can occur, a feature not available in other models. Although such a boundary condition is not required

when the main interest of the model is for dune and beach erosion, the physical constraint imposed by such a boundary condition where hard bottom exists should provide for a more realistic model where model physics are correct. Documentation on both the overwash approach and the hard bottom boundary condition are provided in documents listed in the SBEACH references included in the references section. In previous work done for the Palm Beach Peer Review Group in connection with the reestablishment of the CCCL in Palm Beach County, the Peer Review Group (PRG) noted that the SBEACH Model could be dismissed for use in setting the CCCL based on what were considered unrealistic profile responses to erosion in some cases as well as to the fact the model consistently underestimated erosion. Results of the PRG overview are provided in a separate section on Specific Task Force and CCCL Challenges to Dune Erosion Modeling.

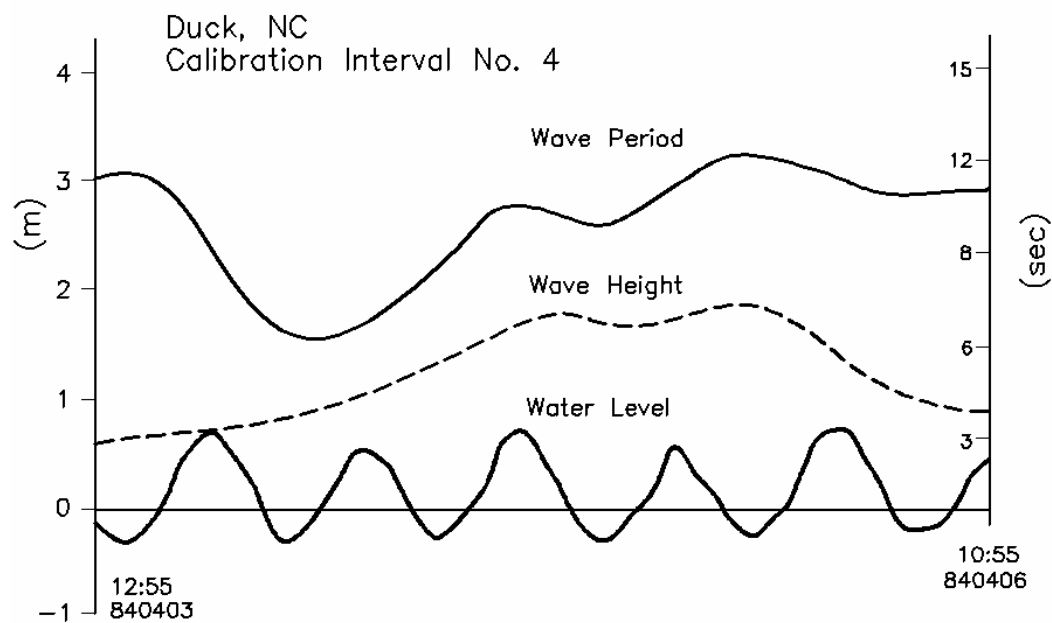


(a) Test from large Beach Erosion Board tank.

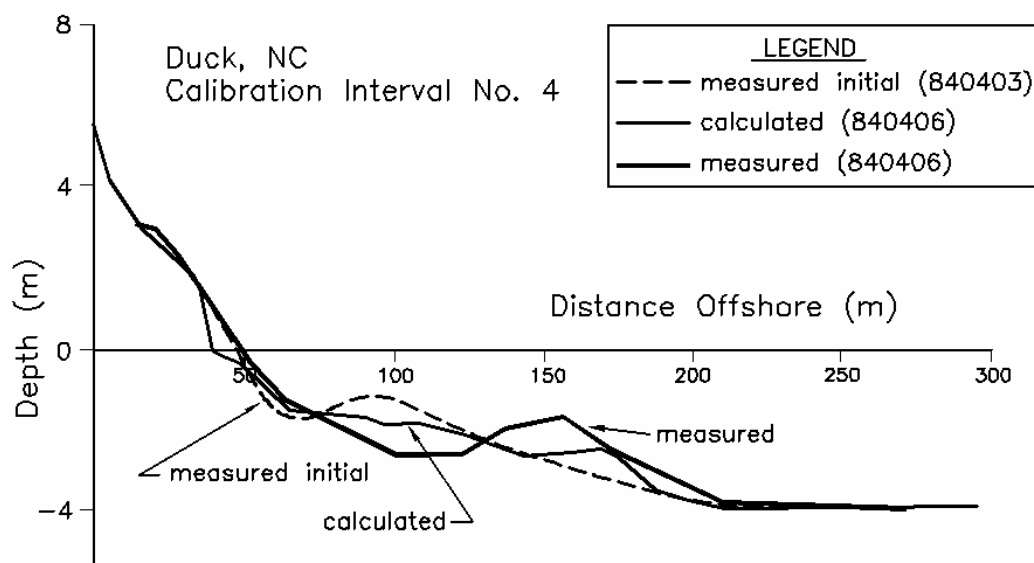


(b) Test from large Japanese tank.

SBEACH comparisons to wave tank tests.



a) Input measured wave height, wave period and water level



b) Result of simulation

SBEACH field data calibration of profile evolution at Duck, N.C.

Model documentation for the SBEACH model consists of a number of technical reports that document various aspects of various versions of the model but there is no manual that documents the existing version of the model, only online help limited documentation in the GUI (graphical user interface) provided by Veri-Tech Inc., the exclusive distributor of the model.

Known documents concerning the SBEACH model and its theory/empiricism are provided in the reference section below. Source code and executable code (outside the GUI) are not available to the public in general. The model must be purchased in a GUI interface entitled "CEDAS" and distributed by Veri-Tech, Inc under a government executed CRADA (Cooperative Research and Development Agreement). User assistance must be purchased at cost although training courses (which must be paid for) are available. User assistance is apparently available to Corps of Engineers Offices but is not available to the public in general except as noted previously via payment to Veri-Tech or the Coastal Hydraulics Laboratory of the Corps of Engineers (very expensive).

CROSS Model

Moore (1982), Kriebel (1982) and Kriebel and Dean (1985) proposed a proportionality between the seaward sediment transport rate per unit beach width, Q , and the deviation of local wave energy dissipation per unit volume from the equilibrium value at each location across the surf zone:

$$Q = K(D - D_{eq})^n$$

where $n=1$ was assumed (linear), and where D represents the local wave energy dissipation per unit water volume, and the transport coefficient, K is assumed to be a dimensional constant. The Kriebel Model utilizes this linear transport relationship as noted previously. From scaling relationships Zheng (1996) noted that proper scaling should occur when $n=3$ rather than $n=1$. If K is independent of the length scale and equals unity, $n = 3$ is determined such that both the scaling relationship and convergence to the equilibrium profile are satisfied. The Cross model utilizes a sediment transport relationship similar to that of the "Kriebel Model" but where transport is a function of the non-linear measure of disequilibrium and is based on the non-linear transport equation above with $n=3$. This nonlinear transport equation provides an explanation for the range of time scales observed in laboratory experiments of beach profile evolution. The time dependent profile response in CROSS is determined by the numerical solution (Zheng, 1996) of the transport and continuity equations where x is the offshore position of a particular contour, h , from a reference baseline, similar to the grid system in the "Kriebel Model" and to the CCCL Dune Erosion Model. A problem with this type of grid system as noted previously is the inability to represent bar type features (i.e. the profile is required to be monotonic).

The CROSS Model allows the use of irregular waves as opposed to monochromatic waves similar to SBEACH. As the present methodology in the CCCL line establishment utilizes depth limited breaking based on monochromatic wave height, this advantage is not deemed a requirement for the FDEP model at present, although the future use of a cross shore model with known [measured] irregular wave input may make such an advantage worth utilizing.

Three sets of large wave tank data were utilized in calibration of the CROSS model, including three cases in the large German wave flume in Hannover (Dette and Uliczka, 1987) three cases of

Saville's large wave tank experiments (Saville 1957) and one case in the SUPERTANK experiments at Oregon State University (Kraus and Smith, 1994). The CROSS Model non-linear transport relationship with $n = 3$ was compared with the linear transport relationship, $n = 1$, for seven large wave tank experiments from three different facilities (Saville (1957), Dette and Uliczka (1987), and Kraus and Smith(1994)) . Among these experiments, five were carried out with monochromatic wave conditions, and the other two were conducted with random waves. Calibrations of the transport relationships were accomplished by a series of simulations in which trial K values were used to simulate profile evolution for each experiment. The best-fit K values were determined as the value yielding the overall least squares error. The non-linear relationship between transport and energy dissipation disequilibrium was found to provide more realistic results than the linear relationship as used in the "Kriebel Model" and in SBEACH. The active profile considered in the numerical model was from the wave run-up limit to the wave breaking point, where the breaking water depth is 1.28 times the breaking wave height. Outside this active region, the net sediment transport is set to zero in the model. At each time step, the water level is determined by the sum of storm surge, tide and wave set-up. The wave set-up in the surf zone is calculated according to the balance of pressure and radiation stress gradients as per (Bowen et al., 1968) and the formula of Hunt (1958) is applied to calculate the wave run-up. Three characteristic profile slopes (dune, shoreline and offshore slopes) must be specified in the numerical model. The dune slope, which is defined as the averaged dune scarp slope after erosion, is the maximum slope that the profile is allowed to achieve. The shoreline slope is the anticipated profile slope between the shoreline and the run-up limit. After each time step, if the profile is steeper than the given shoreline slope at a point, the profile is smoothed toward the shoreline slope with an exponential "folding time" of 0.1 hours. Sand conservation across the profile is ensured by the continuity equation, so that the volume eroded from the beach face must be deposited offshore. The offshore slope is introduced to control the slope at the seaward end of deposited volume. Summarizing the calibration work of the CROSS Model, a total of seven experiments were employed and comparison of the proposed non-linear versus the linear transport relationships was made. Different sediment sizes, initial profiles, and wave conditions characterized the experiments and it was found that the fine sediment generally has relative mild dune, shoreline and offshore slopes and the coarse sand corresponds to somewhat steeper slopes, consistent with early findings by Bascom (1951). The best-fit transport coefficient, K in the non-linear ($n = 3$) relationship was found to have a much narrower range than that for the linear relationship (which is a positive attribute for a model). Based on the testing results, the non-linear transport relationship with $n = 3$ was considered more appropriate for a cross-shore sediment transport model. Under field conditions, it was recommended that the measured pre-storm shoreline slope be applied as the shoreline slope input.

In a follow-up comparison of various FDEP funded models along with the SBEACH model, the CROSS model was compared with three existing models: EDUNE [a derivative model of the "Kriebel Model that includes a simplified version of overwash]; SBEACH; and the CCCL Dune Erosion Model; using beach erosion measurements after two storms at Ocean City, Maryland from the 1991 and 1992 winter storm season. The predictions of the proposed CROSS model and three other models: the EDUNE Model (Kriebel, 1982, 1986; Kriebel and Dean, 1985), two versions of the SBEACH Model(Larson et al., 1989, Larson and Kraus, 1989) and the CCCL Dune Erosion Model (Chiu and Dean, 1984, 1986), were compared with the storm erosion measured at Ocean City, Maryland (a data set also utilized in calibration work of SBEACH). The input parameters for

each model in this study were selected to represent the conditions for which each model was originally calibrated. The run conditions for each model are described briefly as follows:

CROSS Model calibration factors

The dune and offshore slopes were set equal to 1 and 0.15, respectively, as default conditions, and the shoreline slope was set to the average shoreline slope value of the measured pre-storm profiles (0.05). The variable profile parameter, A , was determined according to the variable sediment size as per Dean (1977). Additionally, as a basis for comparison with the other three models, a constant grain size of 0.35 mm was used. During each storm, a random wave series was generated according to the time history of the measured significant wave height and peak spectral wave period. Both wave set-up and run-up are included. The active water depth is 1.28 times the instantaneous incoming individual wave height.

CCCL Model calibration factors

The default input dune slope was set to 1. Since this model cannot handle variable sediment size along a profile, a uniform grain size of 0.35 mm was applied. The set-up at the shoreline was calculated as noted previously according to the significant wave heights. No wave run-up is included in the model. The input water level during a storm is given by adding the set-up at the shoreline to the measured storm surge. After calculating the model estimated recession, a factor of 2.5 was applied to contours which receded.

EDUNE Model calibration factors

The default input dune slope was set to 1 and the input shoreline was taken as 0.05 which was the average shoreline slope of the measured pre-storm profiles. For the same reason as CCCL, a sediment size of 0.35 mm was used. The input significant wave height was applied as a regular wave height for each time step. In EDUNE, the wave run-up is a constant value. According to the EDUNE Users Manual (Kriebel, 1989), the value of run-up should be based on field data or experience from a particular location and does not necessarily simulate a realistic wave run-up limit at each time step. Calibrated to match the dune erosion level of post-storm profiles at Ocean City, the run-up calculated by Hunt's formula for the maximum significant wave height was used in the entire simulation time for each storm. No set-up was included in the simulations.

SBEACH model calibration factors

Although source codes for this model were not available, both versions (2.0 and 3.0) of SBEACH model are believed to incorporate wave run-up and set-up. The maximum slope that a predicted profile is allowed to achieve is required and was set to 17.5 degrees as a default condition; this corresponds to a slope of 0.32. A uniform sediment size of 0.35 mm was used. The wave conditions used in the SBEACH model simulations were as measured from wave gages at the site. SBEACH model provides the choice of wave type (monochromatic or irregular). The option of irregular waves was chosen for this study.

Summarizing the model comparison, it was found using eroded volume and beach retreat at the 3 meter contour as metrics, that CROSS with variable sand size, along with EDUNE and SBEACH (version 3.0) presented comparable predictions. As the numerical simulations of CROSS and CCCL do not include the effect of over-wash, profiles that had overwash (caused by run-up) may have been modeled better by EDUNE and SBEACH. It is expected that the under-predictions of

CROSS could be improved by incorporating dune over-wash processes in the model. Typically CCCL over-predicted dune erosion in individual profiles, whereas the other three models typically under-predicted the erosion (i.e. un-conservative estimation of erosion). Among the four models, CCCL was the only model over-predicting average dune erosion, with the other three models under-predicting. The over-prediction by the CCCL model was considered consistent with its calibration (and 2.5 safety factor application) which incorporates natural longshore variability of dune erosion. CROSS with fixed sand size provided the smallest differences between the measured and computed profiles.

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