An investigation of shoreline change at Sandy Hook, New Jersey utilizing the Shoreline Modeling System

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An Investigation of Shoreline Change at
Sandy Hook, New Jersey
Utilizing the Shoreline Modeling System

by

Mary K. Tibbetts

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ABSTRACT

The coast of Sandy Hook, New Jersey has been subject to chronic erosion during the past few decades, particularly the segment of beach immediately north of the seawall terminus, the critical zone. Various components of the Shoreline Modeling System, SMS, (Gravens, 1992) are used to investigate rates of sediment transport and the resulting shoreline evolution at the critical zone and at the region just south of the critical zone, the feeder beach, for the time period of 1987 through 1994. A substantial portion of the investigation is focused on the calibration and verification of one model within the SMS, the Generalized Model for Simulating Shoreline Change, GENESIS, (Hanson, 1987, Hanson and Kraus, 1989, Gravens et al., 1991). The predicted sediment transport rates occurring at the feeder beach area were substantially smaller than the predicted sediment transport rates associated with the northern end of the critical zone, thus more sediment was leaving the site than was being introduced to it. This sediment budget problem largely explains the chronic erosion which is present at Sandy Hook.
1.0 INTRODUCTION

1.1 Background

Sandy Hook, a 16 km long spit, is located at the northern tip of the New Jersey barrier island system. As seen in Figure 1.1, the spit is bounded to the west by Sandy Hook Bay and to the east by the Atlantic Ocean. This study deals solely with the eastern side of the barrier island.

A review of the wave climate at the study area reveals that the direction of wave approach is primarily from the southeast to east. This is explained by the presence of Long Island, NY which shelters waves approaching from northern and northeastern directions. As expected, the dominant direction of longshore sediment transport is to the north (Caldwell, 1966).

The New Jersey barrier island coast to the south of Sandy Hook is heavily structured with both seawalls and groins. A nearly continuous 11 km long rubble mound seawall runs along the ocean side of the barrier island stopping at the southern end of South Beach at Sandy Hook. Numerous groin fields supplement the seawall, with the northern most groin located approximately 300 meters south of the end of the seawall. The presence of these structures and the deepening of the foreshore profile in front of the structures have caused a diminishing longshore transport rate during the past few decades. Consequently, the unprotected shoreline just north of the seawall terminus has experienced serious recession.
Figure 1.1 Location of Sandy Hook, New Jersey
Various studies have been conducted in the past which focused on the New Jersey barrier island coast and the potential longshore sediment transport rates along the coast (Caldwell, 1966, Kraus et al. 1988, USAE, 1954). This study deals predominately with the area of Sandy Hook which is typically referred to as the critical zone, the area just north of the end of the seawall, and the feeder beach, the groin compartment immediately south of the terminus of the seawall (see Figure 1.2). In the recent past this critical region has eroded so extensively that the roadway and the parking lots servicing the area have been damaged (Psuty & Namikas, 1991).

1.2 Objectives

The Shoreline Modeling System, SMS, (Gravens, 1992) is used throughout this study to investigate shoreline evolution along the critical section of beach at Sandy Hook, New Jersey. The SMS is a collection of computer programs which, among other things, allows for the manipulation of data sets, transformation of deep water waves, and prediction of resulting shoreline evolution. Although various components of the SMS are utilized, the work presented herein is focused primarily on the use of the Generalized Model for Simulating Shoreline Change, GENESIS, (Hanson, 1987, Hanson and Kraus, 1989, Gravens et al., 1991) to evaluate shoreline evolution and secondarily on the use of the Regional Coastal Processes Wave Propagation Model, RCPWAVE, (Ebersole, 1985, Ebersole et al., 1986, Cialone et al., 1992) to evaluate wave transformation over an arbitrary bathymetry.
Figure 1.2 Location of the critical zone and feeder beach at Sandy Hook, NJ (from Namikas, 1992).
The objectives of this project are to:

a.) Evaluate the effectiveness of GENESIS in predicting shoreline evolution using readily available hindcast wave data and aerial photographs that delineate successive historic shoreline positions.

b.) Investigate the sensitivity of GENESIS to empirical transport parameters and depth of closure.

c.) Investigate the effects of RCPWAVE on the prediction of shoreline evolution at Sandy Hook.

d.) Determine a range of contemporary longshore sediment transport rates at Sandy Hook.
2.0 GENESIS

GENESIS, an acronym for the GENERalized model for Simulating Shoreline change, is a one-line numerical model which has the capability to simulate long term shoreline change along a section of coast that possesses a fairly evident trend in shoreline orientation. The major sediment transport mechanism within the model is incident wave alongshore momentum flux, although GENESIS also accounts for transport due to a gradient in breaking wave height in the alongshore direction. The model can accommodate various numbers and combinations of groins, jetties, detached breakwaters, and seawalls, as well as, sediment transmission at groins and jetties, wave transmission at detached breakwaters, diffraction at detached breakwaters, jetties, and groins, beach fills and beach mining (Hanson and Kraus, 1989).

The model is operated through a data file interface with the following data required, as a minimum, to execute the program:

a.) Initial shoreline position data and at least two other sets of shoreline position data if the model is to be calibrated and verified

b.) Time series of offshore wave data
c.) Beach profiles and native sediment grain size to predict the depth of closure, which is the depth beyond which there is not a noticeable
change in depth, and the equilibrium slope

d.) Location and pertinent dimensions of coastal structures

e.) Date, volume, and location of beach fills

f.) Conditions at the project boundaries, known in GENESIS as the lateral boundary conditions

The required and optional input data files needed to run the model are seen in Figure 2.1, along with the GENESIS generated output files. Although seven input data files are seen in Figure 2.1, only the marked (*) files are required. The exact number of input files generated is case dependent.

The data file START.ext contains information about the project site, such as length of beach, time of simulation, native sediment grain size, location of structures, depth of closure, active berm height, etc. The START.ext file is also where the modeler specifies the empirical transport parameters.

The files SHORL.ext and SHORM.ext hold the shoreline position data used in calibrating and verifying the model. GENESIS requires shoreline position data at equal alongshore increments, therefore these file only contain “y values”, the distance to the shoreline from a baseline. SHORL.ext contains the measured shoreline positions at the start of the simulation, while SHORM.ext contains the measured shoreline positions at the end of the simulation. The SHORM.ext values
Figure 2.1  Schematic of input and output files used in the operation of GENESIS (from Hanson and Kraus; 1989)
are compared to the GENESIS calculated shoreline position values, held in the file SHORC.ext, in an effort to evaluate the accuracy of the shoreline prediction.

The time series of offshore wave data is held in the WAVES.ext file. GENESIS allows for the inclusion of two wave events. Thus for one time step, a sea event and a swell event may be specified. As is the case with all input files, the WAVES.ext file begins with a four line header which allows the modeler to specify information about the file, for example, the location and depth of the wave gage. This header is not read by GENESIS.

The SEAWL.ext file is not required to run GENESIS, unless there is a seawall or a revetment at the project site. The format of the SEAWL.ext file is similar to that of the SHORL.ext and SHORM.ext files. Instead of holding the position of the shoreline, it holds the position of the seawall or revetment measured from a baseline. GENESIS does not allow the shoreline to retreat landward of a seawall or revetment.

The remaining two input files, NSWAV.ext and DEPTH.ext, are optional files used to supplement the WAVES.ext file. NSWAV.ext and DEPTH.ext are used only in the case that an external wave transformation model, such as RCPWAVE, is being used. If the modeler specifies in the START.ext file that an external wave model is being used, GENESIS will look for and read the two files. If the modeler does not
specify the use of an external wave model, GENESIS will not read the two data files and only the WAVES.ext file will be read. NSWAV.ext holds the wave data which has been transformed from an offshore depth to some nearshore reference line and DEPTH.ext holds data specifying to what depth the waves have been transformed.

GENESIS produces three output files after the completion of each run. SETUP.ext holds much of the information that is specified in the START.ext file and also any warnings that are issued during the run. OUTPT.ext contains the major results, such as final shoreline position data, net sediment transport rates, and gross transport rates. The file SHORC.ext holds the calculated shoreline position data in the same format as SHORL.ext and SHORM.ext.

Further details on the data files can be found in Chapter 4.0, where the data specific to Sandy Hook is introduced. The following sections of this chapter delve into the assumptions and limitations of GENESIS and some of the theory behind the model and the model's governing equations.

2.1 Assumptions and Limitations

Typically, over time, beach profiles tend to maintain an average slope. Although a given beach may move seaward or landward depending on the wave conditions, the change in shape and slope of the profile may be relatively small. GENESIS was
designed around the assumption that although the profile may recede landward or accrete in a seaward direction, the shape of the profile remains the same. Thus, if the profile shape does not change only one point is needed, with respect to some baseline, to define the position of the beach. The complete beach section is defined by a single contour line, taken as the mean sea level shoreline. Because of the one contour line assumption, GENESIS is typically referred to as a "one-contour line" model or a "one-line" model.

GENESIS accounts for changes in shoreline position due to longshore sediment transport, however, it does not take cross-shore sediment transport into consideration. This may cause a problem if the time interval of the study is small or if the study is focused on storm induced erosion. However, with a long time interval, the effects of cross-shore sediment transport will often average out. Furthermore, if the model is verified using known shoreline position data, then implicitly the effects of cross-shore transport are included in the model.

A third assumption within the model is that sediment is transported alongshore between two well defined elevations on the beach profile. The landward limit for transport is the top of the active berm, while the seaward limit for transport is the depth of closure. These values are held constant in GENESIS and are required to define the active volume of sediment in the alongshore transport regime.
Finally, GENESIS operates on the assumption that there is a clear, long term trend in the shoreline.

With these assumptions in mind it is also important to investigate the limitations of the model. GENESIS is not capable of handling the effects of wave reflection from structures, tombolo development, nor changing tide levels. It also should be noted that there are a few restrictions on the placement, shape, and orientation of structures. The reader is referred to Hanson and Kraus (1989) for further details on the restrictions because these restrictions do not impact the analysis done here in.

2.2 Shoreline Change Prediction

One of the two major submodels within the GENESIS model calculates sediment transport rates and the resulting shoreline change. The theory of conservation of sand volume is the foundation for the shoreline change equation. Consider a segment of beach with a coordinate system such as the one in Figure 2.2, with the y-axis pointing offshore and the x-axis running parallel to the trend of the shoreline. Within this right-handed Cartesian coordinate system, y represents the shoreline position and x the distance alongshore. Based on the previously discussed assumptions that the beach maintains a constant slope over a time interval, Δt, and the transport is vertically confined between the depth of closure, $D_c$, and the active berm height, $D_b$, the change of volume of the section is $ΔV = ΔxΔy \left( D_c + D_b \right)$. 
Figure 2.2   Definition sketch for shoreline change equation
(from Hanson and Kraus, 1989)
The change in volume, $\Delta V$, is determined by the net amount of sand entering or exiting the beach segment from all four sides. Volume change can result from a change in the longshore transport rate, $Q$, at the lateral boundaries of the segment. The resulting net volume change is $\Delta Q \Delta t = (\partial Q/\partial x) \Delta x \Delta t$. Another volume change may result from a line source or sink of sand, $q = q_s + q_o$, adding or removing a volume of sand per unit width at a rate of $q_s$ at the shoreward side of the segment or at a rate of $q_o$ at the offshore side of the segment. This yields a volume change of $q \Delta x \Delta t$.

Equating the two aforementioned contributions with the volume change gives $\Delta V = \Delta x \Delta y (D_c + D_b) = (\partial Q/\partial x) \Delta x \Delta t + q \Delta x \Delta t$. Taking the limit as $\Delta t$ approaches zero and rearranging the equation gives the governing equation for shoreline change:

$$\frac{\partial y}{\partial t} + \frac{1}{(D_c + D_b)} (\frac{\partial Q}{\partial x} - q) = 0$$  \hspace{1cm} (2.1)

In order to solve equation 2.1 for a new shoreline position at the end of a time interval, the initial shoreline position, boundary conditions at both ends of the beach, $q$, $D_c$, $D_b$, and $Q$ must all be known. This study was conducted using Version 1 the SMS which does not have the capability to represent sources or sinks.

The depth of closure is specified by the modeler. Typically, a range of $D_c$ values
are considered, with the final choice of the $D_c$ being a calibration factor used by the modeler. If sequential beach profiles are available, they may be used in order to determine a range of $D_c$ values, however, if profiles are not available, the depth of closure may be estimated. Hallermeier (1983) stated that the maximum seaward limit of transport could be estimated by:

$$D_c = 2.28 \, H_{so} - 10.9 \frac{H_{so}^2}{L_{so}}$$

where

$H_{so}$ = deep water significant wave height

$L_{so}$ = deep water wave length

using a maximum seasonal or annual wave height. If possible, both profiles and Equation 2.2 should be used to determine the depth of closure. GENESIS uses a constant value of $D_c$ for an entire project reach.

The height of the active berm, $D_B$, is another quantity which is specified by the modeler. This value can be determined through site surveys or from topographic maps.

Determination of the wave induced sediment transport rate, $Q$, is discussed in the following section.
2.3 Sediment Transport Rate Prediction

The rate of longshore sediment transport is predicted within GENESIS through the use of an empirical equation,

\[ Q = (H_s^2 C_g) \left( a_1 \sin 2\theta_{be} - a_2 \cos \theta_{be} \frac{\partial H_s}{\partial x} \right)_b \]  

(2.3)

where

- \( H_s \) = significant wave height
- \( C_g \) = wave group celerity given by linear wave theory
- \( b \) = subscript referring to breaking wave conditions
- \( \theta_{be} \) = breaking wave angle

The parameters \( a_1 \) and \( a_2 \) are defined as:

\[ a_1 = \frac{K_1}{16(\rho_s/\rho - 1)(1 - \rho)(1.416)^{3/2}} \]  

(2.4)

and

\[ a_2 = \frac{K_2}{8(\rho_s/\rho - 1)(1 - \rho)\tan \beta(1.416)^{3/2}} \]  

(2.5)

where

- \( K_1, K_2 \) = empirical coefficients
- \( \rho_s \) = density of sand
- \( \rho \) = density of seawater
- \( \rho \) = density of seawater
- \( p \) = porosity of sand on the bed
- \( \tan \beta \) = average bottom slope from the shoreline to the depth of active
transport solved by Equation 2.2

The factor of 1.416 is present to convert the significant wave height to the root-mean-square wave height.

Equation 2.3 is a variation on the "CERC formula" presented in the Shore Protection Manual (USAE, 1984). The CERC formula calculates transport rates produced by obliquely incident breaking waves using the concept of "wave energy flux". Knowing wave conditions at a site, the longshore component of wave energy flux can be computed and related to the longshore transport rate through the use of an empirical curve (Galvin and Schwerppe, 1980). The CERC formula is seen in the first term of Equation 2.3.

As mentioned previously, the GENESIS transport formula is a variation on the CERC formula. This variation is seen in the second term of Equation 2.3. This term, not present in the CERC formula, describes the effect of the longshore gradient in breaking wave height, $\partial H_b/\partial x$, on the longshore transport rate. Generally the value of the second term is much smaller than that of the first, except in the vicinity of structures where diffraction causes a significant change in the breaking wave height over a considerable length of beach.

With regards to the empirical coefficient $K_1$, Komar and Inman (1970) first determined a value of $K_1 = 0.77$ from sand tracer experiments. Kraus et al. (1982)
recommended a slightly lower value of $K_1 = 0.58$ as a result of their tracer experiments. As $K_1$ is treated as a calibration parameter controlling the time scale of the simulated shoreline change and the magnitude of the longshore transport rate within GENESIS, it is recommended that a value between 0.1 and 1.0 be used (Hanson and Kraus, 1989). Optimally the value of $K_1$ falls between 0.58 and 0.77.

Hanson and Kraus (1989) recommended using values of $K_2$ between 0.5 to 1.0 times that of $K_1$. Choosing $K_2$ larger than $K_1$ tends to cause an exaggerated prediction of shoreline change at structures (Hanson and Kraus, 1989).

Although recommended values for the transport parameters, $K_1$ and $K_2$, have been presented, the actual values are case dependent; $K_1$ and $K_2$ are determined in the calibration and verification process by reproducing shoreline change and sediment transport rates and directions.

As the longshore transport equation requires the average nearshore bottom slope, the profile shape must be supplied by the modeler. This shape is also used to determine the location of breaking waves alongshore. GENESIS uses the equilibrium slope presented in Bruun (1954) and Dean (1977). They found that the average profile shape for a beach can generally be represented by:

$$D = Ay^{2/3}$$

(2.6)
where

\[ D = \text{water depth} \]

\[ A = \text{empirical scale parameter} \]

\[ y = \text{distance offshore} \]

Moore (1982) found that the parameter A depends on the beach grain size.

2.4 Internal Wave Transformation Model

In order to simulate shoreline change, GENESIS requires a time series of offshore wave data, this offshore wave data must then be transformed to obtain the nearshore breaking wave height and direction. As mentioned in the beginning of Section 2.2 there are two major submodels within GENESIS. The first submodel calculates sediment transport rates and change in shoreline position, and the second submodel is internal transformation model. This second submodel should not be confused with the external wave model, RCPWAVE, which is part of SMS package but not part of GENESIS.

The internal wave model transforms waves from an offshore location based on the assumption that the bottom contours are straight and parallel. It solves for breaking wave height and direction at alongshore grid points; the wave period remains constant as this model is based on monochromatic wave theory.

The wave transformation first takes place assuming there are no structures within
the model reach, thus neglecting the effects of diffraction. Then if structures are present, the results are modified to account for the diffraction.

Initially, neglecting diffraction, there are three unknowns, the breaking wave height, the breaking wave direction, and the breaking wave depth. The breaking wave height is found by taking shoaling and refraction into account:

\[ H_2 = K_R K_S H_{ref} \] (2.7)

where:

- \( H_2 \) = breaking wave height at a point alongshore
- \( K_R \) = refraction coefficient into the breaker point
- \( K_S \) = shoaling coefficient into the breaker point
- \( H_{ref} \) = offshore wave height

\( K_R \) is a function of the offshore wave direction and the wave direction at breaking and \( K_S \) is a function of wave period, the offshore wave depth and the breaking wave depth. The wave height, \( H_2 \), is compared with the possible breaking wave height at the depth where \( H_2 \) is solved. If breaking conditions are not reached, the calculation moves to a point closer to shore and the iterative process continues.

The breaking wave direction is found using Snell's Law:

\[ \frac{\sin \theta_b}{L_b} = \frac{\sin \theta_{ref}}{L_{ref}} \] (2.8)

where:
$$L = \text{wavelength}$$

$$\theta = \text{wave angle or direction}$$

$$b = \text{subscript referring to breaking conditions}$$

$$\text{ref} = \text{subscript referring to offshore}$$

Finally, the breaking wave depth is solved using:

$$H_b = \gamma D_b \quad (2.9)$$

where:

$$H_b = \text{breaking wave height}$$

$$\gamma = \text{breaker index}$$

$$D_b = \text{breaking wave depth}$$

The breaker index is a function of the deepwater wave steepness, $H_d/L_o$, and the average beach slope (Smith and Kraus, 1991). Further detail on determining $H_b$, $\theta_b$, and $D_b$ is found in Hanson and Kraus (1989).

If there are no diffraction producing structures, the transformed wave characteristics are input to Equation 2.3, the sediment transport rate equation. However, if there are detached breakwaters, groins, or jetties which extend beyond the surf zone, waves may be affected by diffraction prior to breaking. Thus, the wave characteristics need to be recalculated as the diffraction producing structures affect the response on the shoreline in the lee of the structure. The tip of the structure will produce a near-circular wave crest pattern, thus distorting the wave field. In
an effort to accurately model shoreline change, the combined effects of diffraction, shoaling, and refraction must be considered.

In areas affected by diffraction, Equation 2.10 is used to calculate the height of breaking waves which have been transformed by diffraction, refraction, and shoaling:

\[ H_b = K_D H'_b \]  

(2.10)

where

\[ K_D = \text{diffraction coefficient} \]

\[ H'_b = \text{breaking wave height at same cell without diffraction} \]

The diffraction coefficient is a function of the breaking wave depth and the angle, \( \theta_D \). \( \theta_D \), as defined in Figure 2.3, is the angle between the incident wave ray at \( P_1 \) and the straight line between \( P_1 \) and \( P_2 \), where \( P_2 \) is the breaker point. The three needed quantities, \( H_b \), \( D_b \), and \( \theta_b \) are determined at intervals alongshore by solving Equations 2.8, 2.9, and 2.10 iteratively. GENESIS uses the method of Goda, Takayama, and Suzuki (1978) to determine the value of the diffraction coefficient as it is seen as more representative of "in field" conditions than the method presented in the Shore Protection Manual (USAE, 1984). Hanson and Kraus (1989) should be consulted for further details on this subject.

It should also be noted that a contour modification routine is located within the
Figure 2.3 Definition sketch for diffraction coefficient calculation (from Hanson and Kraus, 1989)
internal wave model and as such the offshore contour lines, which remain parallel to the shoreline, are updated as the shoreline evolves over time. The details on this calculation, which improves the accuracy of the internal wave model, are found in Hanson and Kraus (1989).

2.5 Lateral Boundary Conditions

In order to run GENESIS, the modeler must specify boundary conditions at both lateral ends of the project area. Boundary conditions are a necessary part of the model because a finite difference scheme is used in solving the shoreline positions and sediment transport rates. The selection of boundary conditions is of tremendous importance as the boundary conditions affect the predicted shoreline positions within the project area. GENESIS will accept either a "Pinned-Beach Boundary Condition" or a "Gated Boundary Condition".

The pinned-beach boundary condition is the default condition within the model. It should be chosen when a point on the beach exhibits a long-term stability. For instance, the shoreline position of the project area should be plotted over time. If there is a point which does not move appreciably over the time span, that point would represent a pinned-beach boundary condition. The pinned boundary condition therefore implies a constant transport rate at the boundary.

The gated boundary condition is used to specify a groin, a jetty, or a shore
connected breakwater at the project lateral end. With this boundary condition, the transport rate is determined by specifying the beach slope at the structure, the permeability of the structure, and the distances from the shoreline to the seaward end of the structure. GENESIS simulates sand movement at structures by considering sand bypassing and sand transmission. Bypassing is assumed to take place if the depth at the end of the structure is less than the depth of active transport. Sand transmission is determined based on a modeler specified permeability factor for each structure.

Details on the mathematical representation of the pinned-beach and gated boundary conditions are found in Hanson (1987).

2.6 Wave Energy Windows

An energy window is a stretch of beach which is open to incident waves. The stretch of beach is constrained by two boundaries that are responsible for limiting the attack of waves to the beach. Windows are separated by jetties and groins which cause wave diffraction, nontransmissive detached breakwaters, and the tips of transmissive detached breakwaters (Hanson and Kraus, 1989). Wave energy enters through a window and makes its way to the nearshore area.

Version 1 of the SMS does not allow for wave energy to propagate through a shore connected structure. With this in mind, the concept of "sand transport calculation
domains" is presented. The domains are segments of coast constrained on each lateral end by a diffracting shore connected structure or a model boundary.

GENESIS solves the shoreline change equation for each domain independently, except in the cases where sand bypassing and sand transmission are present. Bypassing and transmission, by definition, allows for the exchange of sand across the boundaries of sand transport calculation domains.

2.7 Solution Scheme

If all of the data required to solve the shoreline change equation, Equation 2.1, the sediment transport rate equation, Equation 2.3, and the breaking wave equation, Equation 2.9, are known, the response of the shoreline to the incident wave conditions can be calculated. In order to solve Equation 2.1 realistically, GENESIS uses a numerical solution procedure. In this procedure the distance alongshore is divided into cells of a certain width and the simulation interval is divided into small time steps.

GENESIS uses the concept of a computational grid, see Figure 2.4, to solve for shoreline positions and sediment transport rates. As seen in the figure, shoreline positions, $y_i$, are defined at the center of grid cells ($y$-points) and transport rates, $Q_i$, are defined at cell walls ($Q$-points). Also illustrated in the figure are the left and right boundary conditions which are given at cell walls 1 and $N + 1$, respectively.
Figure 2.4  GENESIS finite difference grid
(from Hanson and Kraus, 1989)
The values $Q_i$ and $y_i$ are determined through the use of an implicit finite difference scheme. The following equations are used within GENESIS to solve the two unknowns:

$$y'_{i} = B' \left( Q'_{i} - Q'_{i-1} \right) + yc_i$$ \hspace{1cm} (2.11)

and

$$Q'_{i} = E_i \left( y'_{i-1} - y'_{i} \right) + F_i$$ \hspace{1cm} (2.12)

where:

- '$'$ denotes a quantity at a new time step.
- $B' = \Delta t/[2(D_B + D_C)\Delta x]$
- $yc_i =$ function of $q'_i$, $q_b$, and other known quantities
- $E_i =$ function of wave height, wave direction, and other known quantities
- $F_i =$ function similar to $E_i$

The so-called double-sweep algorithm is used to solve Equations 2.11 and 2.12. Details regarding the solution procedure are found in Hanson (1987).
The wave model discussed in Section 2.4, which is internal to GENESIS, is based on the assumption that the bottom contours at the site are plane and parallel. However, as is the case in many offshore environments, the bottom contours in the vicinity of Sandy Hook are not perfectly straight nor parallel and as such an external wave model which accounts for an arbitrary bathymetry may be needed. As part of this project, the external wave transformation model RCPWAVE, an acronym for the Regional Coastal Processes WAVE propagation model was used. RCPWAVE was selected over other wave propagation models as it is included within the SMS package.

For an open coast situation, like that at Sandy Hook, RCPWAVE has advantages for use with GENESIS:

a.) It solves for wave height and angle values directly on a grid.
b.) It includes diffractive effects produced by irregular bathymetry.
c.) It has proven to be very stable.

3.1 Assumptions and Limitations

RCPWAVE, a linear, monochromatic short-wave transformation model, transforms an offshore wave into a nearshore reference depth while accounting for refraction, shoaling, and diffraction resulting from local bathymetry. As is the case with the
internal wave model, RCPWAVE uses linear wave theory because it has been
shown to produce relatively accurate first-order solutions to wave propagation
problems at a fairly low cost (Cialone et al. 1992). A few limitations within
RCPWAVE are that the model does not account for diffraction caused by structures
and it neglects wave reflection outside of the surf zone.

3.2 Wave Transformation Equations

The governing equations solved in the model are a modified form of the "mild
slope" equation for linear, monochromatic waves (Berkhoff, 1972 and 1976), and
the equation specifying irrotationality of the wave phase function gradient.

Berkhoff's mild slope equation is defined as:

\[
\frac{\delta}{\delta x}(cc \frac{\delta \Phi}{\delta x}) + \frac{\delta}{\delta y}(cc \frac{\delta \Phi}{\delta y}) + \sigma^2 \frac{c_g \delta \Phi}{c} = 0
\]

(3.1)

where:

\[x, y = \text{orthogonal horizontal coordinate directions}\]
\[c = \text{wave celerity}\]
\[c_g = \text{wave group celerity}\]
\[\Phi = \text{complex velocity potential}\]
\[\sigma = \text{angular wave frequency}\]

The equation specifying the irrotationality of the wave phase function gradient can
be written as:
\[ \frac{\partial (k \sin \theta)}{\partial x} - \frac{\partial (k \cos \theta)}{\partial y} = 0 \]  

(3.2)

where:

\[ k = \text{wave number} = \frac{2\pi}{L} \]

\[ \theta = \text{wave propagation direction} \]

Thus to solve the governing equations, the modeler must specify offshore wave characteristics, including wave height, period, and direction, and also information regarding the bathymetry at the project site.

### 3.3 Solution Scheme

If all the data required to solve the governing wave transformation equations are known, the nearshore wave characteristics can be calculated using a finite-difference method. The finite difference procedure employs the RCPWAVE computational grid as seen in Figure 3.1

RCPWAVE initially estimates the values of wave and wave group celerities, wave angle, and wave height for all grid points by implementing the following procedure (Cialone et al., 1992):

a.) The wave number, \( k \), is computed at every cell using the dispersion relationship.

b.) The wave and wave group celerities are calculated at every cell as they are functions of the wave period and wave number.
Figure 3.1 RCPWAVE calculation grid (from Hanson and Kraus, 1989)
c.) The wave angle is estimated using the above information and Snell's law.

d.) Wave heights at each cell are estimated taking shoaling and refraction into account.

After the initial estimates of the wave characteristics are made, a row-by-row marching scheme begins. Each wave characteristic is solved through an iterative finite difference process, with repetition of the iterative process until a convergence criterion is met yielding the final values of the wave height, direction and period.

3.4 Integrating RCPWAVE and GENESIS

Hanson and Kraus (1989), Gravens et al. (1991), and Gravens (1992) have recommended integrating RCPWAVE and GENESIS by using RCPWAVE to propagate an offshore wave data set into some nearshore reference line so that the wave conditions are pre-breaking. The wave model presented in Section 2.4, which is internal to GENESIS, is then used to further transform the waves until breaking conditions are solved as GENESIS requires breaking wave conditions to solve its governing equations. This combination of the two models is illustrated in Figure 3.2.

Typically, GENESIS is used to investigate shoreline evolution over a relatively long time interval. As the time step for a simulation is usually on the order of 3 to 12 hours, it is impractical to run a model such as RCPWAVE for each time step
Figure 3.2 Wave transformation models
(from Hanson and Kraus, 1989)
due to the tremendous amount of execution time which would be involved. Rather than running RCPWAVE at every time step, it is suggested that the wave data be analyzed so that the offshore wave conditions may be divided into period and direction bands (Gravens et al. 1991). The periods are broken into ranges of 2 second intervals and the directions are broken into 22.5 degree intervals. This use of period and direction bands reduces the number of transformation runs within RCPWAVE. The time of execution is further reduced by using a unit wave height throughout the runs, thus producing a transformation coefficient, rather than a transformed wave height, along the nearshore reference line. Per linear wave theory, the actual transformed wave height is determined by simply multiplying the offshore wave height by the transformation coefficient.

Various programs which are all part of the SMS package and assist in determining the nearshore conditions given the period-direction band transformation coefficients are presented in Gravens et al. (1991). These programs help manage the "bookkeeping" which is involved with running a wave model which is external to GENESIS.

It should also be noted that information on the stability of RCPWAVE and a more detailed look at the theory behind the model development is found in Cialone et al. (1992), Ebersole (1985), and Ebersole et al. (1986).
Recalling the governing equations of GENESIS and RCPWAVE that were presented in the previous chapters, it becomes apparent that a significant amount of data is required to run the models. This data requirement includes shoreline position, beach profile, incident wave, and offshore bathymetry data, as well as information on structures, beach fills, and boundary conditions. The empirical transport coefficients, which also are required, are treated as the primary calibration tool.

4.1 Shoreline Position

Vertical areal photographs of Sandy Hook were gathered in an effort to determine shoreline positions over time. The final assembly of photos included those which were taken on April 23, 1971, October 23, 1977, March 23 1982, April 14, 1987, March 13, 1992, March 19, 1993, and April 9, 1994. In order to create shoreline position data files, a coordinate system was positioned on the photos, grid cell spacing was determined, and the shoreline positions were digitized.

GENESIS requires that the x-axis be placed parallel to the general trend of the shoreline (Hanson and Kraus, 1989). In this study the orientation of the shoreline differed from the critical zone to the feeder beach, posing a problem in the orientation of the x-axis. Since the major area of interest in this study was the
critical zone, the x-axis was positioned so that it was parallel to the trend of the
critical zone, approximately 357 degrees clockwise from north, and the y axis was
placed normal to the x-axis, pointing offshore, as seen in Figure 4.1. The origin of
the coordinate system was located as far away from the critical zone as possible
with the given data.

As seen in Figure 4.1 the area of interest was nearly 1000 meters in length,
encompassing the critical zone and the feeder beach. Due to the relatively small
nature of this area, a grid cell spacing of 25 meters was used in order to maintain
as much accuracy in reproducing the critical zone as possible. Based on the
availability of data and the limitation that Version 1 of the SMS allows for only
100 cells, a total project length of 3275 meters was represented by 91 cells.

The scale of the areal photos was determined by comparing distances on the photos
to distances that were measured in the field, for example, parking lot lengths. In
order to eliminate an abundance of digitizing, three years of photos were
eliminated. Two sets were not used based on length of beach photographed (April
1971) and clarity of the photos (March 1993). The third set of photos that was
eliminated was done so to reduce seasonal effects on the beach planform. The
October 1977 photos were not used based on the fact that all of the remaining
photos were taken in the spring. In order to reduce calibration and verification
errors, an effort was made to select photos which were all taken during the same
Figure 4.1  Location of coordinate system at study area
season. The remaining four years were digitized so that shoreline position was documented every 25 meters. Calibration of GENESIS was conducted using the 1992 and 1994 data and verification was completed using the 1987 and 1992 data. Shoreline positions from 1982 were not used during calibration nor verification, however they were digitized in order to visualize the massive erosion which had once eroded the critical zone so severely that the roadway was damaged. Shoreline positions for the entire project reach for the years 1982, 1987, 1992, and 1994 are seen in Figure 4.2. A magnified view of the beach positions in the critical zone and the feeder beach area is seen in Figure 4.3.

4.2 Active Berm Height and Depth of Closure

Equation 2.1, the shoreline position equation, requires the modeler to supply values of the height of the active berm, \( D_B \), and the depth of closure, \( D_C \), with respect to mean sea level (MSL). GENESIS holds these values constant throughout each simulation. As shoreline position data was known for 1982, 1987, 1992, and 1994, profile data corresponding to those time periods was located to determine \( D_B \) and \( D_C \) for the same time periods, if possible.

The active berm height was determined using recent beach profiles which were measured as part of a study conducted by Rutgers University (Namikas, 1992). Portions of this data were plotted, see Figures 4.4 and 4.5, for use in this study. The active berm height in June 1987 and 1988 was approximately 3 meters along
Figure 4.2 Shoreline positions at the study area
Figure 4.3  Shoreline positions at the critical zone and feeder beach
Figure 4.4  Mid-critical zone beach profiles from 1987 and 1988 (from Namikas, 1992)
Figure 4.5    Mid-critical zone beach profiles from 1990 and 1991 (from Namikas, 1992)
the length of the critical zone, while in May 1990 and August 1991 the active berm height was approximately 3.5 meters at the critical zone. The increase in berm height was assumed to be due to a beach fill project which was conducted in 1989. The values of active berm height used for both the calibration and verification simulations are found in Table 4.1. It should be noted that because profiles were lacking for time period of 1992 through 1994, the height of the active berm was assumed. The decrease from the 1988 - 1992 value, 3.5 meters, to 3 meters was assumed to take place based on the information known about the 1987 berm height which represents a measured berm height three years after a fill project, the 1984 fill. Similarly, the 1992 - 1994 time period was nearly 3 years after the 1989 fill and as such it was assumed that the berm height had eroded back to an equilibrium value of 3 meters.

<table>
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<tr>
<th>Time of Simulation</th>
<th>Active Berm Height (meters)</th>
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</thead>
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<tr>
<td>1987 - 1988</td>
<td>3.0</td>
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</tr>
<tr>
<td>1992 - 1994</td>
<td>3.0</td>
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Table 4.1 Active berm height values

Determination of the depth of closure was somewhat complicated as the more recent profiles did not extend far enough in a seaward direction. However, profiles presented in Phillips et al. (1984) from May 1982 and August 1983 did extend far
enough seaward to determine the depth of closure, see Figure 4.6. Although these profiles were not from the simulation period (1987 - 1994) it was assumed that they are representative of the area. From the available profiles, not all of which are presented here, the depth of closure was determined to range from 4 meters to 9 meters. The values of the depth of closure were estimated based on visual inspection of the profiles. For example, it can be seen from the May 1982 profile that the depth beyond which there is not a noticeable change in depth, the depth of closure, was approximately 4 meters. An empirical determination of the depth of closure based on annual maximum wave heights was calculated using Equation 2.2. As actual wave heights were not known for the time period in question, hindcast wave data determined as part of the U.S. Army Corps of Engineers' Wave Information Studies of the U.S. Coastlines, WIS, (Hubertz et al., 1993) was used. Further information on the wave data is found in the following section. The WIS data produced an average annual maximum wave height of 4.2 meters, which yielded an empirically predicted depth of closure of 6.2 meters. Hanson (1987) suggested the depth of closure is equal to twice the maximum annual significant wave height. This empirical relationship gave a depth of closure equal to 8.4 meters. Phillips et al. (1984) stated that sediment transport is limited to a depth of 4 meters. The value of the depth of closure used in this study was determined through calibration and verification of the model. The value of 8 meters was selected based on the fact that the shoreline positions predicted using the depth of closure equal to 8 meters best replicated the positions of the actual shoreline. The
Figure 4.6  Mid-critical zone beach profiles from 1982 and 1983 (from Phillips et al., 1984)
results of testing conducted with different depth of closure values are found in Chapter 5.0.

4.3 Wave Climate

A time series of wave data was needed to calculate shoreline position and sediment transport rates within GENESIS. The most readily available time series of wave data was the hindcast wave data determined as part of the U.S. Army Engineers' Wave Information Studies of U.S. Coastlines. For this project, the Revised Atlantic Hindcast Level II Data was used from station 73 (see Figure 4.7). Wave characteristics were given every three hours for the twenty year period of 1956 - 1975 for both sea events and swell events. The WIS data represents a "numerical gage" located at a depth of 18 meters and accounts for the sheltering due to the site geography.

Since the years of the WIS study do not coincide with the years of known shoreline position, three representative or typical years were determined. It was assumed that if these three years adequately represented the twenty year time frame of 1956 - 1975, then they should also be suitable to represent the next twenty years, 1976 - 1997.

The three typical years were selected based on statistics that were compiled for all twenty years. The compilation, found in Table 4.2, consists of the average
Figure 4.7 Location of WIS station 73
(from Hurbertz et al., 1993)
<table>
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<th>Year</th>
<th>Hs</th>
<th>20yr avg</th>
<th>Tp</th>
<th>Pdir</th>
<th>Hs</th>
<th>Comments</th>
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Table 4.2 Summary of yearly statistics of WIS hindcast data for station 73
significant wave height ($H_s$), the relationship between the average $H_s$ for the year of interest and the average $H_s$ for the twenty year set, the average peak period ($T_p$), the average peak direction ($P_{dir}$), the maximum $H_s$, and comments, mostly pertaining to the amount of long period waves experienced each year. These comments were developed using Table 4.3 which is a summary of how many of each period wave occurred for each year of the hindcast. For example, in 1956: 348 waves with a peak period of 3 seconds were hindcast for station 73.

By inspecting Table 4.2, it is obvious that there are more than three years that were "typical". Three years were chosen such that one year was slightly stormier than typical (1958), one year was slightly calmer than typical (1968), and one year was typical (1971). Also it is important to note the three years together preserved the average $H_s$, $T_p$ and $P_{dir}$ of the twenty year hindcast.

A time series of sea and swell wave data, given in three hour intervals, was constructed by appending the wave data from 1958, 1968, and 1971 together. In an effort to reduce computational time, RCRIT (Gravens et al., 1991), another program presented in the SMS package was utilized. RCRIT analyzes each wave event and determines if the event is calm or propagating offshore and if the event has the potential to produce a longshore transport rate in excess of the critical transport rate, defined in GENESIS as 3.9 m$^3$/s. If the event is either calm or propagating offshore or if it lacks the potential to transport a significant amount of sand it is
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Table 4.3 Summary of wave period statistics for station 73
flagged and the next event is processed. An event is deemed calm if the wave period associated with the event is equal to zero. A modeler specified shoreline orientation angle is used to determine whether a wave is travelling offshore. The potential longshore transport rate of an event is determined using Snell’s Law and the concept of wave energy flux. Once RCRIT is used, the time series of offshore wave data contains “flagged events” which are not processed when executing GENESIS, thus reducing the execution time.

Before the data set containing “flagged events” was used to model shoreline evolution, the effect of eliminating events on the transport rate was investigated by using SEDTRAN (Gravens et al., 1991). SEDTRAN, another part of the SMS package, determines the potential longshore sediment transport rate at a site using the energy flux method and the following data: shoreline orientation; offshore wave time series of height, period, and direction; and the depth corresponding to the wave data. SEDTRAN was utilized with the “unflagged” and “flagged” time series producing a gross transport rate of $1.098 \times 10^6$ m³/year and $1.087 \times 10^6$ m³/year, respectively. It was concluded that the effect of eliminating calm, offshore propagating, and small transport producing events was minimal compared to the amount of time that was saved in running GENESIS, thus all modeling efforts were conducted with the “flagged” time series.

As mentioned previously, the wave model which is internal to GENESIS transforms
the offshore time series of wave data based on the assumption that the offshore contours are straight and parallel. As this is not the case at Sandy Hook, the effects of the arbitrary bathymetry were dealt with using RCPWAVE. As discussed in Chapter 3.0, it is rather impractical to transform three years of wave data given in three hour intervals using RCPWAVE due to the execution time of the model.

In order to eliminate this problem all wave conditions were transformed with a unit wave height of 1 meter. By doing this, the nearshore wave heights produced by RCPWAVE were not actual heights, but rather, transformation coefficients. An actual nearshore wave height was determined by multiplying the offshore wave height by the transformation coefficient. Execution time was further reduced by categorizing the offshore time series into period and direction bands using the program WHEREWAV, another part of the SMS package. WHEREWAV reports the number of events occurring within each period and direction band and this information is utilized in determining the number of RCPWAVE simulations which must be executed in order to adequately represent the offshore time series.

The results of running WHEREWAV are seen Tables 4.4 and 4.5, the wave statistics for the primary wave (sea) events are seen in Table 4.4 and the statistics for the secondary wave (swell) events are seen in Table 4.5. From inspecting the tables it can be seen that 37 simulations are required to describe the sea events in the offshore time series and 36 events are needed to represent the swell events. Although 73 events may not seem minimal at first inspection, the rewards of using
### Classification of Primary Wave Events by Angle Band

<table>
<thead>
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<th>Angle Band Number</th>
<th>Range with Respect to Shore-Normal (degrees)</th>
<th>Number of Events</th>
<th>Average Wave Angle (degrees)</th>
<th>Period Bands</th>
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### Classification of Primary Wave Events by Period Band

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Table 4.4 Classification of primary wave events by angle and period bands
### Classification of Secondary Wave Events by Angle Band

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<th>Average Wave Angle (degrees)</th>
<th>Period Bands</th>
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### Classification of Secondary Wave Events by Period Band

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Table 4.5 Classification of secondary wave events by angle and period bands
this statistical analysis become clear when comparing the 73 "banded" events with the total 5218 events which are present in the offshore time series. The simulations representing the sea and swell events are seen Tables 4.6 and 4.7, respectively.

4.4 Bathymetry

Detailed information on the bathymetry of the site was needed in order to execute RCPWAVE. The model requires bathymetry from the shoreline seaward until the depth of the offshore numerical wave gage, 18 meters. NOAA (1984 and 1989) and Waterway Guide (1980) charts were used to determine depths at every cell on the RCPWAVE computation grid. The grid was constructed using an x-axis cell spacing of 100 meters and a y-axis cell spacing of 200 meters. The coarse grid spacing was due to the limitations regarding the number of permissible cells in the PC version of RCPWAVE and also the RCPWAVE stability requirement which controls the ratio of y cell spacing to x cell spacing.

4.5 Structures

The locations and dimensions of both hard and soft structures are required to accurately predict shoreline evolution. For the reach and time period considered in this project, there were three to four hard structures, depending on the simulation period, and two beach fills.

The hard structures consist of the rubble mound seawall which terminates just south
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Table 4.6  RCPWAVE simulations representing the primary wave events
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Table 4.7 RCPWAVE simulations representing the secondary wave events
of the critical zone, two jetties which front the seawall south of the critical zone, and a steel revetment, installed during the summer of 1988, which is north and landward of the seawall terminus.

The permeability of the groins was estimated based on site inspection, aerial photos, and through the calibration and verification procedure. A permeability coefficient of 0 implies an impermeable groin, while a coefficient of 1.0 refers to an ineffective groin. Both groins were estimated to be completely functioning and impermeable during both the calibration and verification simulations.

The soft structures consist of the two beach fills which were conducted during the 1980's. The first fill occurred, not continuously, during the time period of November 1982 to May 1984. Specifically, from November 1982 until August 1983 approximately 1,662,000 m\(^3\) of sand (dredge measured) was pumped onto the critical zone. During the same time period about 153,000 m\(^3\) of sand (dredge measured) was added to the feeder beach. The following spring additional funds became available and from approximately April 1984 through May 1984 roughly 458,000 m\(^3\) of sand (dredge measured) was placed onto the critical zone.

The second fill was conducted from September 1989 through November 1989. During this fill approximately 1,784,000 m\(^3\) and 446,000 m\(^3\) of sand (dredge measured) was placed onto the critical zone and feeder beach, respectively. Losses
from winnowing of fines were estimated to be 9% and 13% of the dredge measured volume of sediment for the 1982 and 1989 fills, respectively (Phillips et al., 1984 and Gahagan and Bryant, 1990). A loss rate of 13% was used for the 1989 fill to calculate the actual volume of sand placed.

4.6 Boundary Conditions

Inspection of the site shows that a groin is located at the southern or right lateral end of the reach. This allowed the author to place a gated boundary condition at the right boundary. The left lateral boundary condition was dealt with using the pinned beach condition as no structures were located in the vicinity. In order to reduce the error caused by placing a pinned beach condition at a location where one does not exist, the boundary was placed as far from the critical zone as possible. As mentioned previously, the version of GENESIS used in this study limited the author to using only 100 cells. This factor, coupled with the preference to use a small grid cell spacing caused the pinned beach boundary condition to be placed closer to the critical zone than would be ideally desired. Sensitivity tests were conducted to determine the effect of the location of the pinned beach boundary condition on the predicted shoreline. In doing so, it was determined that moving the location of the boundary condition over an extent of 350 meters only slightly altered the amount of erosion at the critical zone, and had minimal or no effects on the shape of the shoreline at the critical zone. The only negative effect of the pinned beach boundary condition was seen in the cells in the immediate vicinity of
the boundary condition (cells 1 through 15). As these cells are not included in the critical zone, these negative effects were considered acceptable.

4.7 Transport Parameters

The majority of the calibration and verification efforts of this study were focused on the determination of proper transport parameters. The procedures and results of the calibration and verification efforts along with a discussion of how the selected transport coefficients compared with the recommended coefficients are found in the following chapter.
5.0 CALIBRATION AND VERIFICATION

Calibration efforts were conducted by varying three input parameters, the depth of closure and the two transport parameters. Calibration and verification was initially conducted using only the wave model which is internal to GENESIS. During this effort, calibration was achieved and the results were verified, however, this was only accomplished by using low values of the transport parameters. In an effort to improve the results, the external wave model, RCPWAVE, was utilized in conjunction with the internal wave model.

Calibration of the model was completed using the 1992 and 1994 shorelines. This time period proved to be an adequate calibration period because no beach fills occurred between 1992 and 1994 and the execution time was tolerable due to the relatively small time period.

Verification of the model was conducted using the 1987 and 1992 shorelines. Unlike the calibration, the verification was conducted in two steps. First a shoreline was predicted for 1988, using an initial shoreline of 1987. This 1988 shoreline was then used as an "initial" shoreline in order to predict the shoreline in 1992. The reason for splitting the time period was two fold, the steel revetment was installed in 1988 thus necessitating an altered seawall file after 1988; also the beach fill which took place in 1989 increased the height of the active berm by
approximately 0.5 meters.

5.1 Internal Wave Model

5.1.1 Depth of Closure

As mentioned in Section 4.2 the empirically predicted depth of closure varied from 6 meters to 8 meters and the profiles yielded a depth of closure between 4 meters and 9 meters. As the exact value was unknown, this input parameter was used as a calibration tool. Numerous runs were conducted with varying values of the depth of closure. As a summary, the results of four runs conducted with different depths of closure are seen in Figure 5.1. The results seen in Figure 5.1 were all produced using transport parameters $K_1 = 0.20$ and $K_2 = 0.17$. The depth of closure equal to 4 meters caused severe erosion, while the depth of closure equal to 10 meters caused excessive accretion. The final value of the depth of closure, 8 meters, was determined based on the fact that the predicted shoreline most accurately replicated the measured shoreline.

5.1.2 Transport Parameters

The transport parameters, $K_1$ and $K_2$, affect not only the amount of erosion and accretion at the site but also the shape of the predicted beach. Hanson and Kraus (1989) suggested that the initial run of GENESIS be conducted with “average” values of the transport parameters, $K_1 = 0.5$ and $K_2 = 0.25$. The shoreline predicted using these values had a shape similar to that of the measured shoreline, however
Figure 5.1  Shoreline positions produced during model calibration using various depths of closure (internal wave model)
the predicted shoreline was nearly 100 meters landward of the measured shoreline at the critical zone.

It was determined through subsequent runs that in order for the predicted shoreline to replicate the measured shoreline in both shape and location, the value of $K_1$ needed to be significantly lower than the recommended values. The final value of $K_1$ was chosen to be 0.20. Hanson and Kraus (1989) suggested that the parameter $K_2$ be equal to or less than the value of $K_1$. Through calibration efforts the final value of $K_2$ was chosen as 0.17.

The results of four calibration runs are found in Figure 5.2. This figure shows the calibrated shoreline ($K_1 = 0.2$ and $K_2 = 0.17$), the shoreline produced using the initial values of $K_1 = 0.5$ and $K_2 = 0.25$, the shoreline produced using $K_1 = 0.77$ and $K_2 = 0.38$ as suggested by Komar and Inman (1970), and the shoreline produced using $K_1 = 0.58$ and $K_2 = 0.29$ per Hanson and Kraus (1989). Again, this figure reiterates the fact that for this area, with the given data, the selected transport parameters were lower than the recommended values.

5.1.3 Shoreline Positions

The measured 1992 and 1994 shorelines and the calculated 1994 shoreline that was produced using the calibrated model are seen in Figure 5.3. As seen in the figure, the calculated 1994 shoreline at the feeder beach is slightly seaward of the
Figure 5.2  Shoreline positions produced during model calibration using the two transport parameters (internal wave model)
Figure 5.3  Measured 1992 and 1994 shorelines and calculated 1994 shoreline produced with the calibrated model (internal wave model)
measured 1994 shoreline, however the shapes of the shorelines are the same. The calculated and measured 1994 shorelines at the critical zone are nearly identical, illustrating the success of the calibration. The goodness of fit achieved during calibration is further depicted by the calibration - verification error, the average deviation of the predicted shoreline from the measured shoreline per cell. The calibration - verification error for this trial was 4.7 meters per 25 meter cell.

Verification was conducted using parameters identical to those used in calibration with the exception of the height of the active berm. The reason for changing the active berm height, as mentioned previously, was the 1989 beach fill. The measured 1992 and 1994 shorelines and the predicted 1994 shoreline calculated during verification are seen in Figure 5.4. Again, the sufficient match between shorelines is illustrated through the calibration - verification error which was 4.4 meters per 25 meter cell.

5.1.4 Sediment Transport Rates

The predicted sediment transport rates at Sandy Hook varied with location and time. Accordingly, the rates were analyzed based on two spatial segments, the feeder beach area and the critical zone, and three time segments, mid 1987 through late 1989, late 1989 through mid 1992, and mid 1992 through mid April 1994. The feeder beach area was characterized as having a small or nonexistent beach face and a steep beach profile while the critical zone maintained a relatively large
Figure 5.4  Measured 1987 and 1992 shorelines and calculated 1992 shoreline produced with the verified model (internal wave model)
beach face, compared to the feeder beach area, and possessed a more gentle profile. The time periods were characterized based on their proximity to a beach fill event. The 1987 to 1989 and 1992 to 1994 time periods were well after fills, while the 1989 to 1992 period was immediately after a fill.

GENESIS predicted that the feeder beach experienced an average net transport rate of approximately 14,000 m$^3$ per year to the north during the 1987-1989 and 1992-1994 time periods. The predicted rate was fairly constant both in time and space. The small transport rate was due, in part, to the lack of sediment at the feeder beach and the steep nearshore profile of the feeder beach. During the 1989-1992 period, the feeder beach was replenished with sediment and the average net transport increased to approximately 54,000 m$^3$ per year to the north. This rate is not as meaningful as the rate which was predicted during the other periods because the 1989-1992 value was not constant over time. During the year immediately following the fill (1989-1990) the average net transport rate along the feeder beach was approximately 80,000 m$^3$ per year, as sediment eroded from the feeder beach during the following year the rate decreased to approximately 46,000 m$^3$ per year, and the third year the rate dropped lower to approximately 37,000 m$^3$ per year. The predicted transport rate at the critical zone was more difficult to quantify because it varied significantly in space and time. The values of the transport rates at southern end of the critical zone were similar to those at the feeder beach, however, the magnitude of the transport rates increased substantially along the
The magnitude of the predicted transport rates helped to explain the chronic erosion problem at the critical zone. Simply stated, the wave climate had the potential to transport a large quantity of sand, however there was a deficit of sediment at the feeder beach. When the waves, typically approaching from southern directions, reached the critical zone and the large sand supply, the rate of sediment transport increased. Due to the increase in transport rates to the north, more sediment was leaving the site than was being introduced to site. This unbalanced sediment budget largely explains the erosion problem at the critical zone.

5.2 Internal and External Wave Models

The external wave model, RCPWAVE, was used in an effort to improve the results that were just presented by taking into account the bathymetry which is present offshore of Sandy Hook. The depth of closure, 8 meters, determined using the internal wave model was a reasonable value based on the available data, and as such the same value was used in the following tests. Thus the calibration and verification efforts conducted using both wave models were focused on the transport parameters.

5.2.1 Transport Parameters

The first trial conducted with both wave models was done using the transport
parameters presented in section 5.1.2, that is $K_1 = 0.20$ and $K_2 = 0.17$. The results of this effort are seen in Figure 5.5. As is seen in the figure, the predicted 1994 shoreline was well seaward of the actual 1994 shoreline, implying that the values of the parameters had to be increased. Again, various runs were conducted in an effort to calibrate the model and a summary of the results are seen in Figure 5.6. The "average" parameter values, $K_1 = 0.50$ and $K_2 = 0.25$, caused the predicted shoreline to accrete too much, the Hanson and Kraus values, $K_1 = 0.58$ and $K_2 = 0.29$, also caused too much accretion, while the Komar and Inman values, $K_1 = 0.77$ and $K_2 = 0.38$, caused the shoreline to erode too much. The final transport parameters values, selected through calibration and verification efforts, for use with both wave models were $K_1 = 0.65$ and $K_2 = 0.32$. These values are more meaningful than those produced in section 5.1 as they are within the recommended range of $0.58 < K_1 < 0.77$.

5.2.2 Shoreline Position

The measured 1992 and 1994 shorelines and the 1994 shoreline calculated using the calibrated model are seen in Figure 5.7. As was the case with the results of section 5.1, the calculated 1994 shoreline at the feeder beach is slightly seaward of the measured 1994 shoreline, but the shapes of the shorelines are the same. The predicted and measured 1994 shorelines at the critical zone are nearly identical illustrating the success of calibration. The calibration - verification error for this trial was 4.7 meters per 25 meter cell.
Figure 5.5  Measured 1992 and 1994 shorelines and calculated 1994 shoreline produced using $K_1 = 0.20$ and $K_2 = 0.17$ (internal and external wave models)
Figure 5.6  Shoreline positions produced during model calibration using the two transport parameters (internal and external wave models)
Figure 5.7 Measured 1992 and 1994 shorelines and calculated 1994 shoreline produced with the calibrated model (internal and external wave models)
Verification was conducted yielding the results seen in Figure 5.8. From visual inspection the predicted 1994 shoreline again replicated the measured 1994 shoreline. Quantitatively the results were good as seen by the calibration - verification error associated with this trial which was 6.9 meters per 25 meter cell.

5.2.3 Sediment Transport Rates

The sediment transport rates which were predicted using both wave models were slightly smaller in magnitude than those predicted using only the internal wave. Although the predicted rates were lower than those presented in Section 5.1.4, they maintained the same spacial and temporal relationships as the previously presented rates. For the 1987-1989 and 1992-1994 time periods the net transport rate at the feeder beach was nearly 10,000 m$^3$ per year to the north. Again, this rate was fairly constant in both time and space. During the 1989-1992 period, the average net transport rate was approximately 46,000 m$^3$ per year to the north. This average represents three annual rates which varied greatly. In the year immediately after the fill the net transport rate was approximately 78,000 m$^3$ per year, the following year the average rate decreased to 45,000 m$^3$ per year, and the third year the rate was predicted to be 15,000 m$^3$ per year. As was the case in Section 5.1.4, the transport rates along the critical zone increased substantially to the north varying in both time and space. The predicted transport rates at the feeder beach were significantly lower than the predicted transport rates at the northern end of the critical zone, illustrating again that the erosion problem at Sandy Hook is in part
Figure 5.8 Measured 1987 and 1992 shorelines and calculated 1992 shoreline produced with the verified model (internal and external wave models)
due to a deficit of sand to the south.
6.0 CONCLUSIONS

Sandy Hook has been experiencing a chronic erosion problem during the past few decades due mainly to a sediment deficit at the site. The Shoreline Modeling System was utilized in order to investigate sediment transport rates and shoreline evolution at the critical zone and feeder beach areas of Sandy Hook. Transport rates and shoreline positions were predicted using two techniques: GENESIS was used alone and GENESIS was used in conjunction with RCPWAVE. A large portion of the study was focused on the calibration of GENESIS using the depth of closure and the transport parameters.

6.1 Depth of Closure and Transport Parameters

The depth of closure and the two sediment transport parameters were the primary mechanisms used to calibrate GENESIS. An estimate of the depth of closure at Sandy Hook was made through the use of empirical equations and through inspection of site profiles. The empirically predicted depth of closure was approximately 6 to 8 meters and the depth of closure determined from the profiles varied from approximately 4 meters to 9 meters. During model calibration the shape and position of the predicted shoreline best replicated that of the measured shoreline using a depth of closure equal to 8 meters. This value of 8 meters was also used when GENESIS was utilized in conjunction with RCPWAVE.

The transport parameters, $K_1$ and $K_2$, were also used during model calibration.
When only GENESIS was used, calibration was achieved by using $K_1 = 0.20$ and $K_2 = 0.17$. These values were below the recommended values of $K_1$ equal to 0.58 to 0.77 and $K_2$ equal to 0.5 to 1 times that of $K_1$ (Hanson and Kraus, 1989). However, geometric fit of the shoreline was not achieved with higher transport parameters. Geometric fit was achieved with higher transport parameters, however, when both GENESIS and RCPWAVE were used together. The measured shoreline was best replicated using values of $K_1 = 0.65$ and $K_2 = 0.32$.

6.2 Sediment Transport Rates

Within GENESIS the driving mechanism for sediment transport is incident wave action, therefore the accuracy of the wave data which was used requires some discussion. The wave data used was hindcast wave data which represented the average wave climate at Sandy Hook. Therefore, the sediment transport rates produced herein do not account for any large storms or any extreme calm periods which may have actually occurred from 1987 through 1994.

The transport rates predicted using GENESIS alone were slightly larger than those predicted using GENESIS in conjunction with RCPWAVE, however both techniques predicted similar trends with regards to the spatial and temporal variations of the transport rates. Using GENESIS alone, an average net transport rate of approximately 14,000 m$^3$ per year to the north was predicted for the feeder beach area during the 1987 to 1989 and the 1992 to 1994 time periods. This
transport rate was nearly constant which respect to both time and space. From 1989 to 1990 the net transport rate increased to approximately 80,000 m³ per year to the north due to a beach fill project. The rate decreased at the feeder beach as the time from the fill increased. This same trend was predicted when GENESIS was used with RCPWAVE, however, the predicted net transport rates the feeder beach were 10,000 m³ per year to the north for the 1987 to 1989 and 1992 to 1994 periods and 78,000 m³ per year to the north for the 1989 to 1990 period. Both techniques predicted transport rates at the critical zone which varied with both location and time. The rates at the critical zone increased significantly along the length of the beach to the north. There was a notable difference in transport rates between the feeder beach area and the critical zone, thus more sediment was leaving the site than was being introduced to it. This unbalanced sediment budget is largely responsible for the chronic erosion problem which is present at Sandy Hook.
REFERENCES


Moore, B. 1982. "Beach Profile Evolution in Response to Changes in Water Level and Wave Height," M.S. Thesis, Department of Civil Engineering, University of Delaware, Newark, DE.


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