CHAPTER 94

PROTOTYPE APPLICATIONS OF A GENERALIZED SHORELINE CHANGE NUMERICAL MODEL

by

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ABSTRACT

Three case studies are described in which long-term shoreline response was simulated using a numerical model. One project was along Kachemak Bay, Alaska and involved evaluation of design alternatives that included a detached breakwater, beach fill, and a revetment. The second project was on the north New Jersey coast and characterized by a long seawall and numerous groins. The third project was a model test of shoreline change behind three detached breakwaters Lake Erie. The shoreline response model used, called GENESIS, is demonstrated to have applicability to a wide range of commonly encountered shore protection situations.

INTRODUCTION

Numerical simulation models provide a powerful and unique capability for engineering studies of complex shoreline change occurring under realistic field conditions. Before the development of the modeling system GENESIS, each application of a numerical shoreline response model required extensive modification of an existing model and special refinements for the particular study. To allow application to an arbitrary prototype situation, considerable time was spent on devising a flexible and general internal structure of GENESIS. Through a simple interface, the user can simulate the effects on the shoreline of groins, jetties, detached breakwaters, seawalls, and beach fills. Almost arbitrary numbers, locations, and combinations of such structures can be represented, and user-specified operations can be introduced almost arbitrarily in space and time. The model is economical to run and, therefore, simulations can be performed for wide spatial extents and long time intervals. This paper presents results of applications of GENESIS to three prototype situations. The case studies demonstrate that GENESIS is capable of simulating long-term shoreline change in the field and assisting in the refinement of engineering analysis of shore protection alternatives involving beach fill and various types of structures.

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GENESIS

GENESIS allows simulation of shoreline change occurring over a period of months to years as caused primarily by the action of breaking waves. The project horizontal length scale typically varies from one up to tens of kilometers. The model is generalized in the sense that the it can be used to simulate shoreline change under a wide variety of user specified beach and coastal structure configurations. In addition, the input wave conditions can be entered either (a) from an arbitrary depth as a single value specified by the user and involving a simplified wave refraction calculation, assuming parallel bottom contours, or (b) through interaction with a more rigorous wave refraction model RCPWAVE (Ebersole et al. 1985) allowing specification of an irregular bottom bathymetry (Figure 1).

GENESIS is based on the one-line (shoreline contour) theory of beach change (Pelnard-Considère, 1956). The same concept has been used in a number of previous studies (e.g., Price et al. 1973, Perlin 1979, LeMéhauté and Soldate 1980). In particular, Kraus et al. (1985), Kraus et al. (1988), and Hanson and Kraus (1986a) present applications of the model employed as an engineering tool for making shoreline change forecasts for a real beach. Based upon the results of these studies, recommendations for remedial measures were given.

BASIC EQUATIONS

As an extensive description of GENESIS, discussing basic assumptions, limitations, and governing equations is given in Hanson (1987), only a short review will be presented here.

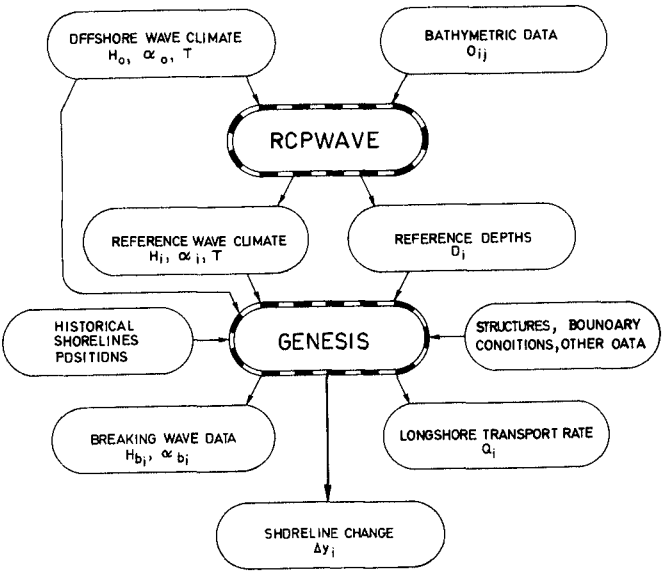

Figure 1. Relation between the wave model RCPWAVE and the shoreline change model GENESIS.

Mass conservation

Following the assumption that the bottom profile moves in parallel to itself to the depth of closure, conservation of sand for an infinitely small length, dx, of shoreline can be formulated as:

$$\frac{\partial y}{\partial t} + \frac{1}{D_B + D_C} \frac{\partial Q}{\partial x} = 0 \tag{1}$$

where y is the shoreline position (m), x is the longshore coordinate (m), t is the time (s), D_B is the average berm height above the mean water level (m), D_C is the depth of closure (m), and Q is the longshore sand transport rate (m^3/s). In order to solve Eq. (1), expressions for the two quantities D_C and Q must be formulated. The berm height, D_B, is taken from the measured or assumed profile.

Depth of closure

For applications involving bypassing of sand at structures, knowledge of the depth to which sand is actively transported alongshore is required. Studies of beach change taking place over a long period of time (years) indicate that the profile varies to the depth of closure, D_C, associated with the wave climate over this long time period. Various values have been suggested for this depth (e.g. Willis and Price, 1975; Kraus and Harikai, 1983; Hands, 1983). These are all of the same order as the formulation of Hallermeier (1983), giving the annual depth of closure as slightly more than twice the extreme annual significant wave height. In the light of these formulations, and keeping the potential errors involved in determining these relations in mind, GENESIS uses a simple relation:

$$D_C = 2\, H_{mas} \tag{2}$$

where H_{mas} is the maximum annual significant wave height (m) for the existing site. The value of H_{mas} for a given site must be specified by the user operating GENESIS. Alternatively, measured profiles can be compared and a closure depth estimated. H_{mas} would then be back-calculated by Eq. (2). It is also assumed that the dry portion of the beach profile, from the shoreline to the berm crest, moves with the wet part of the profile while maintaining its shape.

Longshore sand transport

Kraus et al. (1981) and Kraus and Harikai (1983) showed that the simulation of shoreline evolution, especially in the diffraction shadow zone near structures, is greatly promoted by taking the longshore gradient of breaking wave heights into account. For this reason, in GENESIS, the longshore sand transport volume rate, Q, is calculated as:

$$Q = (H^2 C_g)_b (a_1 \sin 2\alpha_{bs} - a_2 \cos\alpha_{bs} \frac{\partial H}{\partial x})_b \tag{3}$$

where C_g is the wave group velocity (m/s), α_{bs} is the angle of wave crests to the shoreline, the subscript b denotes the breaking condition, and the non-dimensional parameters a_1 and a_2 are given by:

$$a_1 = \frac{K_1}{16\,(\rho_s/\rho - 1)\,(1 - p)\,1.416^{5/2}} \tag{4}$$

$$a_2 = \frac{K_2}{8\,(\rho_s/\rho - 1)\,(1 - p)\,\tan\beta\,1.416^{5/2}} \qquad (5)$$

where K_1 and K_2 are calibration parameters, ρ_s and ρ are the densities of the sediment (quartz sand) and water (kg/m^3), p is the sediment porosity, and $\tan\beta$ is the average bottom slope. The factor 1.416 is used to convert from significant to RMS wave height. The first term in Eq. (3) expresses the longshore transport rate due to obliquely incident waves and is commonly known as the CERC-formula (SPM 1984). The second term, introduced by Ozasa and Brampton (1980), accounts for the longshore sand transport rate caused by the longshore variation in breaking wave height and is especially important in the diffraction zones near structures where the longshore wave height gradient often is strong (Kraus 1983).

HOMER SPIT SIMULATION

Homer Spit, a narrow peninsula southeast of the City of Homer, is between 90 and 450 m wide and extends approximately 7 km into Kachemak Bay in southcentral Alaska (Figure 2). At the tip of the spit are a small-boat harbor and a city dock that is used for year-round shipping. Apart from its commercial importance Homer Spit is also a cultural and social asset, being the only recreational beach in this part of Alaska.

A two-lane road leads from Homer to these developments following the southwestern shores of the peninsula (Figure 3). Since its construction in 1927, the inshore half of the roadway has experienced maintenance problems. Severe storms causing high water levels in combination with intense wave action have overtopped and washed out stretches of the roadway. Various means, including the installation of groins, revetments, and bulkheads have been attempted to control the erosion at the

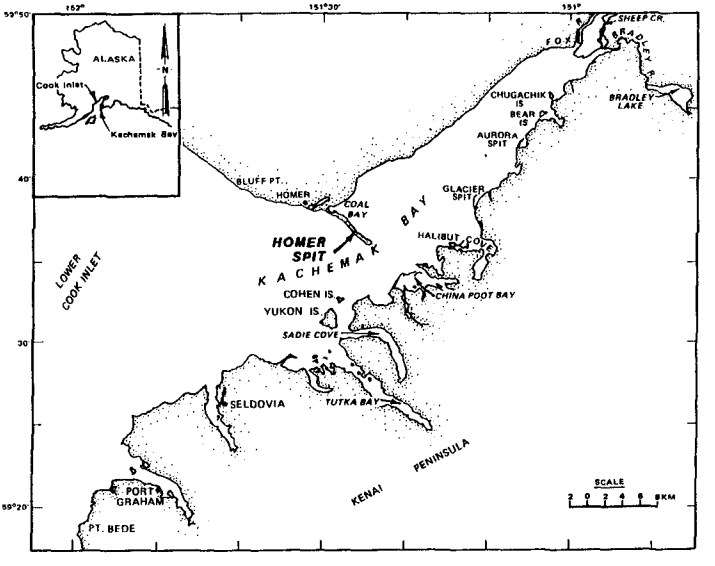

Figure 2. Location map for the study area at Homer Spit.

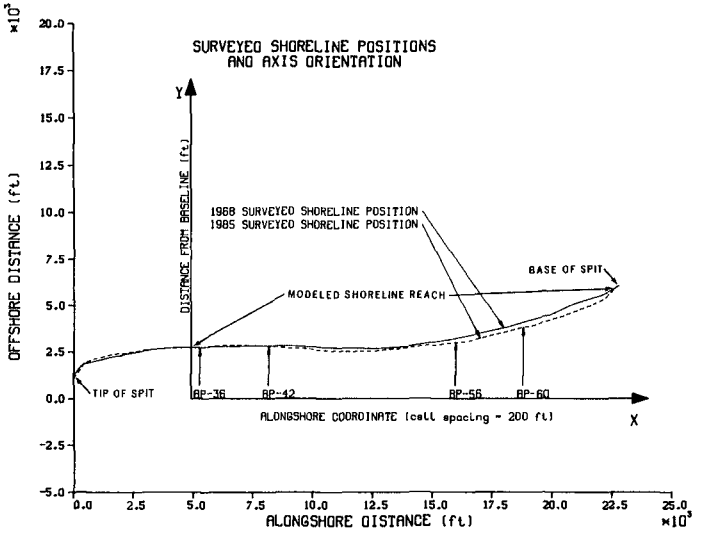

Figure 3. Map of Homer Spit, Alaska showing the extent of the calculation area.

southwestern beach facing outer Kachemak Bay and to mitigate damages to the roadway during extreme storm events. Results of these efforts have not been satisfactory.

The analyses performed by the Coastal Engineering Research Center (CERC) for the Alaska District of the Corps of Engineers (Smith et al. 1985, Chu et al. 1987) included: wind data analysis, deep-water wave hindcast, wave transformation modeling from deep to shallow water , tidal circulation analysis, geological history and sediment sample analysis, beach profile analysis, and longshore sediment transport estimate. By means of wave transformation modeling, a representative 1-year time-varying series of nearshore wave height, period, and direction specified at a 6-hr interval for each of the 79 model calculation grid cells was established.

Modeling Conditions

The input wave data set was prepared at 6-hr intervals, and the longshore sediment transport rates and associated shoreline change were also calculated with a simulated 6-hr time-step. Due to the large tidal range and the significant differences in the average sediment grain size at low, mean, and high tidal levels, the standard version of GENESIS was modified. For compatibility with the shoreline change model, the tide level was simulated at successive time-steps in a cyclical fashion through four representative stages: mean tide, mean high, mean, and mean low. This procedure, in effect, results in a semi-diurnal representation of the tidal cycle. The tides at Homer Spit are indeed semi-diurnal but do have a pronounced diurnal inequality. However, simulation of the tides in the stated manner is consistent with the accuracy of the shoreline change model. Representative grain sizes associated with the respective tidal stage were as determined from the sediment sample analyses: mean low tide, 0.25 mm; mean tide level, 10.23 mm; and mean high tide, 8.13 mm.

The inclusion of tidal changes necessitated input of the average water depth (controlling wave refraction and diffraction) and berm height (affecting the continuity relation) at the various tidal levels. Consistent with the one-line theory of shoreline change, only one contour line was modeled (shoreline at mean tide elevation), but the longshore sediment transport rate (Eq. 3) and corresponding shoreline change (Eq. 1) were calculated at three tide levels based on the particular physical properties (transport parameter K_1 and berm height) estimated for each elevation of the profile.

The longshore grid spacing in GENESIS was set to 200 ft (61 m). This spacing was considered sufficient for evaluating alternative plans but still allowed economical computer execution times. The grid was extended beyond the project area on both sides to obtain termination points that would provide appropriate boundary conditions. Two shoreline surveys of Homer Spit were judged to be adequate for use in the shoreline modeling calibration: October 1968 and August 1985. From inspection of the survey data, two nearly stationary sections of the shoreline were identified. Therefore, a fixed-beach boundary condition, in which the boundary shoreline position is constrained not to move, was implemented at each location. This condition allows sediment to move across the boundary from each side.

Model Calibration

The calibration procedure for GENESIS is to determine the transport parameters K_1 and K_2 of Eqs. (4) and (5) by reproducing measured shoreline change that occurred at the target site between two surveys (here, the 1968 and 1985 surveys). The simulation of shoreline change for this 17-year period was accomplished with the initial shoreline position given by the 1968 measured shoreline. The calibration constants K_1 and K_2 were systematically varied in successive runs of the model, and comparisons were made between the calculated and the measured shorelines of 1985. In addition to visual comparisons of plots, a measure of the calibration error, denoted as Yerr, was calculated to obtain an objective fitting criterion:

$$Yerr = \sum_{i=1}^{N} \frac{|Ycalc85 - Ymeas85|}{|Ymeas68 - Ymeas85|}_i \qquad (6)$$

where Ycalc85 = calculated shoreline position 1985; Ymeas85 = measured shoreline position 1985; Ymeas68 = measured shoreline position 1968; N = total number of calculation cells; and i = cell number.

As Yerr approaches zero, the accuracy of the calibration increases. Although Yerr was used as a numerical indicator of the relative accuracy of the calibration runs (on the order of 100 runs), the final judgement was based on inspection of plots of the full two-dimensional features of the shoreline planform. The model run chosen as the best fit is shown in Figure 4, giving the values of the calibration constants K_1 (mean low tide, 0.77; mean tide, 0.50; mean high tide, 0.55) and K_2 (0.15). The measured and calculated shorelines agree well.

If sufficient shoreline survey and wave data are available, the calibrated model is run to simulate observed shoreline change in a time interval not spanned by the calibration to verify that the calibration constants are independent of the time interval. Because of a severe earthquake in 1964, only the two previously mentioned shoreline surveys

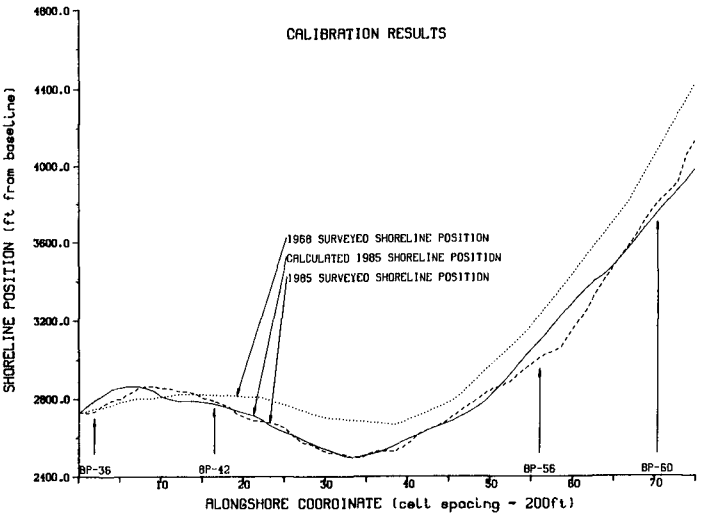

Figure 4. Results of shoreline change model calibration for Homer Spit.

were found suitable. Hence, verification could not be performed, thereby necessitating scrutiny of the sensitivity of the calibrated model to quantify variations in predicted results. A full description of this procedure is given in Chu et al. (1987).

Evaluation of Alternative Plans

Five design alternatives for erosion control (some alternatives including several variations) were analyzed using GENESIS. The alternatives simulated were: a. without-project, b. revetment extension, c. revetment extension with beach fill, d. beach fill, and e. offshore breakwater. A total of 19 alternative erosion control measures were modeled and studied. One best alternative from each of the general design options was selected for a comparative study (see Figure 5). Three of the alternatives shown (1A, 2A, and 5F) indicated considerable erosion and were not recommended for implementation. The remaining two alternatives were (3C) the extension of the existing revetment 610 m towards the tip of the spit combined with a beach fill (30 m added berm width and 610 m alongshore) and (4C) a major beach fill (23 m added berm width and 2,320 m alongshore).

It was difficult to determine which of these two alternatives would best solve the erosion problems at Homer Spit in that the extent of the specified beach fill will ultimately determine the shoreline position. However, model results clearly indicated that nourishment of the existing beach could control coastal erosion problems at Homer Spit. Revetment extension is required to protect the roadway during periods of high tide and storms. A structural approach without beach nourishment may resolve a local problem, but the area of erosion will migrate further down-drift.

Thus, as a conclusion, beach fill, along with extended revetment was considered as the most effective means for erosion control and storm damage reduction at the project area.

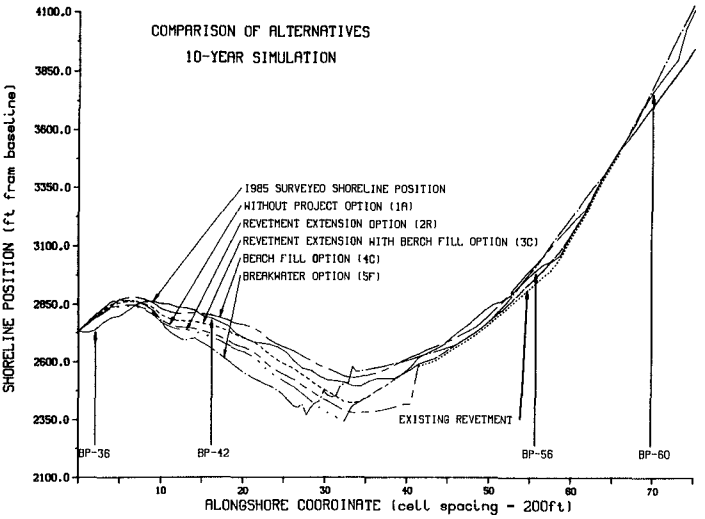

Figure 5. Ten-year simulations of alternative plans for Homer Spit.

SANDY HOOK SIMULATION

This study area constitutes 30 km of the New Jersey barrier island chain from Sandy Hook to Shark River (Figure 6). The coast has been suffering from severe erosion for a century. As an attempt to restore the beach and to secure the integrity of the shore protection structures, a shore protection plan was prepared by the New York District of the Corps of Engineers. The scope of the CERC study was to interpret data and to provide a predictive shoreline change model to assist in the implementation of this plan.

The northern part of the study area consists of Sandy Hook, one of the most famous and most investigated spits in the world. The spit, extending about 16 km into the New York Bight, was originally formed by sand eroding from the adjacent 30 km of the barrier island shores. Ever since the erosion became an increasing problem in the late 19th century, especially from Sea Bright to Monmouth Beach, an increasing portion of the beach has been protected with seawalls, revetments, and groins. Partially as a result of blocking off this sediment supply, sediment transport to the north decreased, causing erosion at the southern part of Sandy Hook, called the "critical zone". This critical zone was of particular interest for the present study as GENESIS was calibrated for this area.

Modeling Conditions

A large portion of the beach south of Sandy Hook is protected by an almost continuous rubblemound seawall and numerous groins. The seawall was modeled according to the principles discussed in Hanson and Kraus (1986b). This seawall constraint is imposed at the same level of approximation as the assumptions used to derive the shoreline change model. Thus, wave reflection, scouring, and flanking are not simulated. The lengths and locations of 91 operating groins were taken from aerial photographs and maps. Very short groins and non-functioning groin remnants were not included. The longshore transport of sand around the

end of a groin is called bypassing, and the transport of sand over and through a groin is called transmission. In this study, there were no data sets available to estimate groin bypassing or transmission. Since the groins in the modeled area are mainly built of heavy, grouted stone, the transmission was set to zero. It is recognized that a limited amount of overtopping does occur during high tide and high wave conditions, but specification of that effect is not possible at the present time. Sand

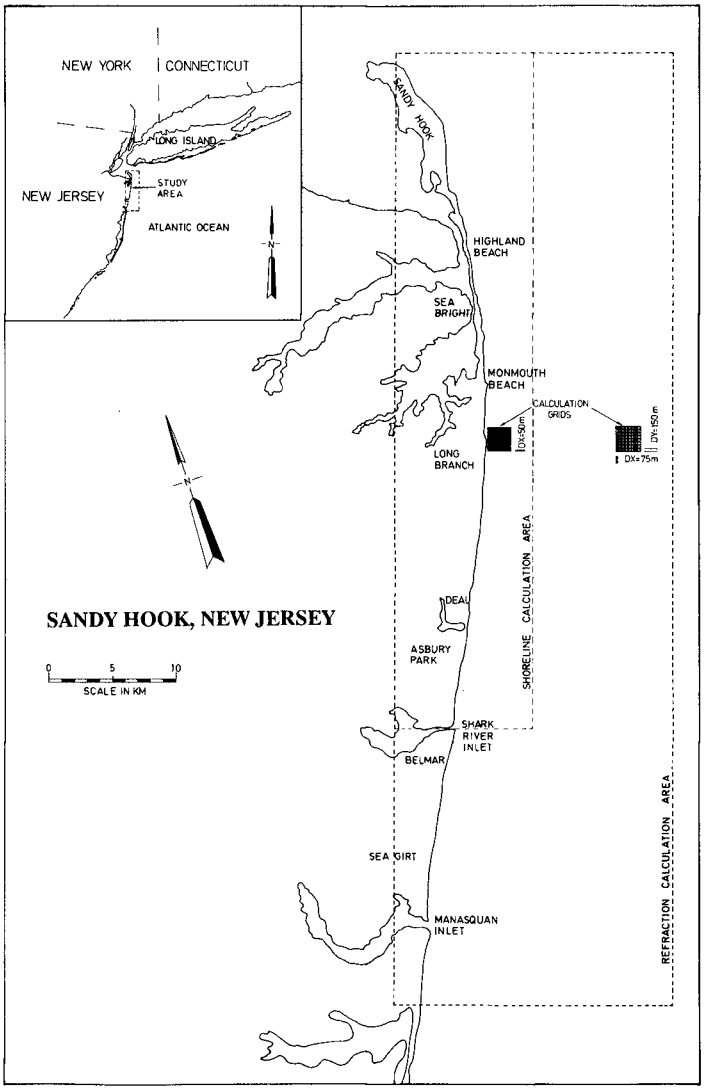

Figure 6. Location map for the study area, showing the extent of the calculation areas and grids for north New Jersey. (Modified from Kraus et al. 1988).

bypassing was calculated according to algorithms incorporated in CENESIS (Hanson 1987).

As no long-term wave measurements were available for the study area, the Wave Information Study (WIS) technique (Jensen, 1983) was used to generate a 20-year hindcast time series of wave height, period and direction at 18 m depth. To keep the data files to a reasonable size, three representative years were selected from the 20-year time series. Wave transformation along the shoreline from the nominal 18 m depth up to breaking was calculated using the wave model RCPWAVE.

Model Calibration

Due to extensive and unrecorded shore protection activities, it was not possible to calibrate and verify the model for the project coast south of Sandy Hook. Instead, the southern part of Sandy Hook, exhibiting beach erosion and with very little human intervention, was used. The dates of major beach fills were known and could be avoided. The calibration was made for an 11-year period, using shoreline surveys taken in 1971 and 1982. An objective fitting criterion, analogous to Eq. (6), was used to obtain $K_1 = 0.4$ and $K_2 = 0.1$. Using these values for K_1 and K_2, the model was verified from 1932 to 1953. As shown in Figure 7, agreement between the measured and calculated shorelines was very good.

As seen from the basic Eq. (1), the shoreline change in time, $\partial y/\partial t$, is proportional to the longshore gradient of the longshore sand transport rate, $\partial Q/\partial x$, and not to Q itself. Thus, in principle, it would be possible to calibrate and verify the model and still have a significant error in the magnitude of the longshore transport rate. Therefore, it was decided to calculate the annual longshore sand transport from Shark River to Sandy Hook and to compare these figures with previous studies. The result from the calculation is shown in Figure 8. The numbers along the shore give the average annual longshore transport rates in thousands of cubic meters for the three years of wave data. The results agree qualitatively with previous studies: the transport being to the north and somewhat increasing with distance moved northward. Quantitatively, however, the rates are less than those reported in previous

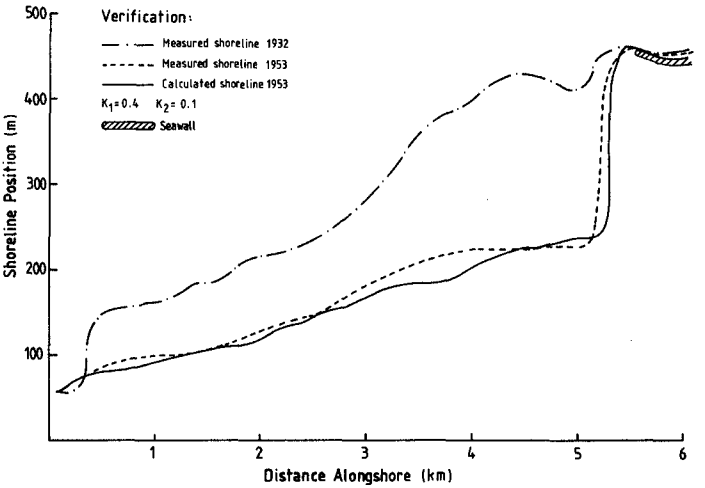

Figure 7. Result of model verification for Sandy Hook, New Jersey.

studies. Therefore, a reanalysis of the wave data, explicitly taking wave sheltering from Long Island into account, is presently being performed.

LAKEVIEW PARK SIMULATIONS

In 1977, three rubble-mound detached breakwaters were constructed at Lakeview Park, Lorain, Ohio, located on Lake Erie. These were the first breakwaters in the United States intended specifically to protect and stabilize a bathing beach (Pope and Rowen, 1983), in this case created by a beach fill (Figure 9). The purpose of the fill was to protect the park and serve as a recreational beach at the same time. In addition to the breakwaters, the beach fill was held in place by one groin on each side.

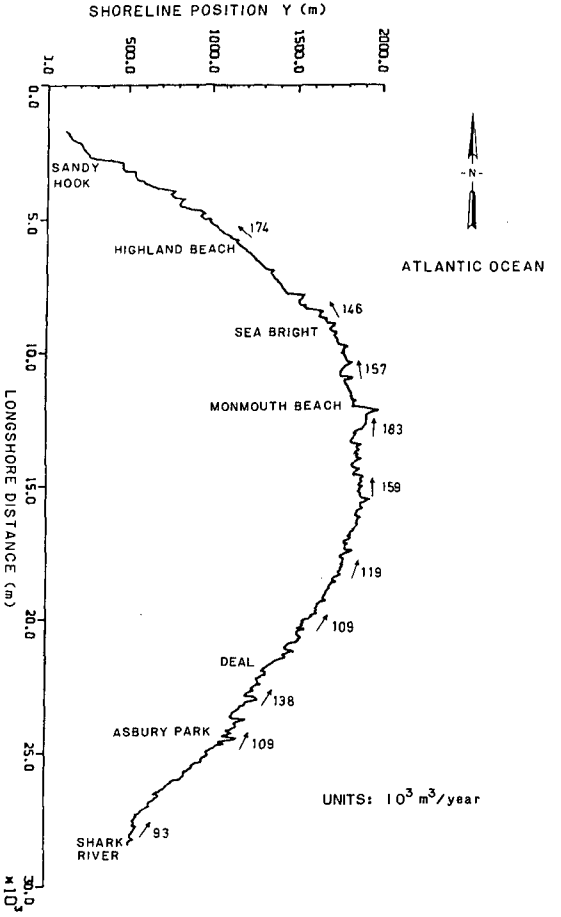

Figure 8. Calculated sand transport rates for northern New Jersey coast. (Modified from Kraus et al. 1988)

Modeling Conditions

The shoreline position and bottom contours were monitored by the Corps of Engineers, both before and after the fill, providing excellent data for a numerical model simulation. The wave data immediately available was limited, however, including only representative wave heights and periods from five different directions and their percentage distribution in time. Little information existed on the actual wave climate (height, period, and direction) between shoreline surveys. A wave time series, prepared at 6-hr intervals to be used in the model calibration/verification procedure, was synthesized for application of GENESIS. All necessary shoreline and structure configuration data were taken from survey charts on a 25-ft (7.6 m) interval. The total distance between the two groins was 1,200 ft (366 m).

Using this limited amount of data, the shoreline change was simulated for the first 24 days after the fill was completed. As the wave input was established on limited information it is likely that, for a short-term simulation as the one made here, the actual mean wave climate could deviate significantly from the representative values.

Model Calibration

Starting with the initial fill shoreline of 1 October 1977, a series of simulations were carried out to reproduce the measured shoreline position of 24 October 1977 (Figure 10). In addition to varying the calibration parameters K_1 and K_2 between the respective simulations, it was found necessary to assume that the average deepwater wave direction deviated 20 deg to the east from the representative values given by the input wave data. This calibration procedure gave values of $K_1 = 0.3$ and $K_2 = 0.3$.

A comparison between the measured and calculated shorelines of 24 October shows that the agreement, from a qualitative standpoint, is good. The model produces three well-developed salients (emerging tombolos) at

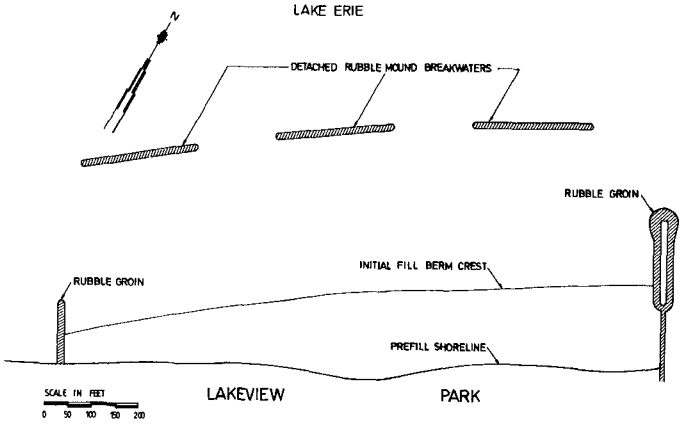

Figure 9. Shoreline and structure configuration at Lakeview Park, Lorain, Ohio.

the proper locations. However, the left-most calculated salient is somewhat too large whereas the other two are too small. An explanation for these discrepancies could be the simplified description of the bathymetry in the area. Due to the limited available wave data, it was decided not to use the wave model RCPWAVE. lnstead, all wave calculations were made within GENESIS, assuming bottom contours were parallel to the calculated representative offshore contour line.

ln a beach fill project of this type, the volumetric changes can be as informative as the shape of the shoreline. ln terms of this volumetric change, the computational results were successful: the measured gain was 59,000 ft^3 (1,670 m^3) and the calculated gain was 53,000 ft^3 (1,500 m^3). Thus, the model accounted for 90 per cent of the volumetric change.

Evaluation

Being a small and well documented area, Lakeview Park serves well as a test case for a simulation model. Extensive monitoring of the bathymetry was not balanced with a similar wave documentation. The success of a model application is, to a large extent, limited by the degree to which the true wave climate can be reproduced. However, the lack of reliable wave data at the same time makes the area representative of most coastal projects. The site was therefore considered as an interesting and realistic test of GENESlS. Under the circumstances, and considering the limited effort spent on calibrating the model, the results were very encouraging.

ln order to make more accurate predictions of shoreline evolution at the site, the following improvements would have to be made. The wave refraction pattern should be analyzed using the wave model RCPWAVE and the true bottom topography (this was not possible at the time). The breakwaters may also be overtopped, implying that wave transmission will contribute to mould the beach plan form. ln addition, wave transmission through the detached breakwaters is believed to have a significant influence on shoreline change and should be represented.

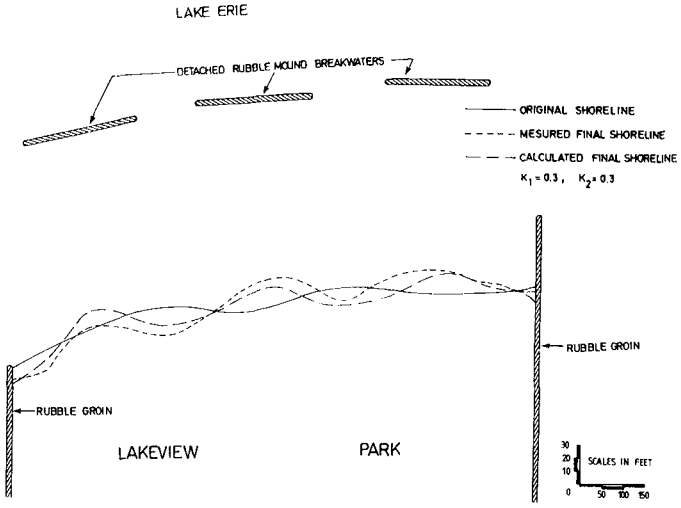

Figure 10. Measured and calculated shorelines at Lakeview Park.

SUMMARY

A numerical modeling system called CENESIS was developed to simulate the interaction between waves, longshore sediment transport, coastal structures, and other engineering activities in the nearshore area. The purpose of the model is to simulate shoreline change on a regional scale and in a long-term perspective ranging from a few months to several years. As opposed to previous models, CENESIS is generalized in the sense that it can be easily applied to almost any open coast and simulate the effects of almost arbitrary numbers, locations, and combinations of groins, jetties, breakwaters, seawalls, and beachfills. The capabilities of GENESIS were demonstrated through three prototype applications: Homer Spit, Alaska; Sea Bright, New Jersey; and Lorain, Ohio.

The calculated examples show that GENESIS is easily applied to quite complex prototype conditions and that the model is capable of simulating long-term shoreline change along coasts controlled by structures as well as open-coast natural beaches.

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