

Genesis—A Generalized Shoreline Change Numerical Model

Hans Hanson

Department of Water Resources Engineering
Lund Institute of Technology
University of Lund
S-221 00 LUND, Sweden



ABSTRACT

HANSON, H., 1988. GENESIS—A generalized shoreline change numerical model. *Journal of Coastal Research*, 5(1), 1-27, Charlottesville (Virginia). ISSN 0749-0208.

This report describes a numerical model called GENESIS (*GEN*eralized model for *SI*mulating *SH*oreline change), developed for calculating shoreline change as caused primarily by wave action. The model is based on the one-line theory, for which it is assumed that the beach profile remains unchanged, thereby allowing beach change to be described uniquely in terms of the shoreline position. As opposed to previous models based on the same concept, GENESIS is generalized in the sense that a simple user interface allows the system to be applied to a diverse variety of situations involving almost arbitrary numbers, locations, and combinations of groins, jetties, detached breakwaters, seawalls, and beach fills. Other features included in the system are wave shoaling, refraction, and diffraction; sand passing through and around groins, and sources and sinks of sand. An overview of the modeling system is presented, and comparisons to analytic solutions as well as prototype situations are presented to demonstrate the capabilities of the system.

ADDITIONAL KEY WORDS: Numerical model, simulation, shoreline evolution, beach change, coastal structures, diffraction, refraction, sand transport.

INTRODUCTION

The interaction between waves, bottom sediment (sand), and coastal structures is extremely complex, ranging from microscale physical phenomena, such as the movement of a particular sand grain, to macroscale phenomena such as the influence of the global mean sea level rise on beach change. Being intended mainly for engineering use, the analysis of GENESIS is restricted to the modeling of important physical processes to a predetermined level of sophistication. This level is mainly constrained by our limited knowledge of relevant physical processes involved and by the amount and quality of input data available in a typical prototype application.

GENESIS allows simulation of shoreline change occurring over a period of months to years, as caused primarily by wave action. The horizontal length scale varies from one up to tens of kilometers. The system is generalized in the sense that the model 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 from an arbitrary depth as a single value specified by the user involving simplified wave refraction calculation, assuming parallel bottom contours, or through interaction with a more rigorous wave refraction model allowing specification of an irregular bottom bathymetry.

Before the development of GENESIS, each application of a numerical shoreline model required extensive modification of an existing model and special refinements, as necessary, for the particular study. Considerable time was spent in altering the internal structure of the model, as well as on the data entry to arrive at a configuration which allowed easy modification in order to investigate design alternatives. With the experience gained in a variety of applications, the possibility became apparent of combining, in a general way, all major features of previous models into one shoreline modeling system. The remaining task would be to structure the system in such way that a general interface would allow the user to operate the model with minimum effort. In essence, the

user would interact only with the interface and not with the model system itself.

At present the model can be applied to simulate shoreline change including the effects 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.

PREVIOUS WORK

The observation that the profile of a particular beach oscillates about an apparent constant shape over the long term led PELNARD-CONSIDERE (1956) to develop a mathematical model, now called the one-line model. GRIJM (1961, 1965) used the one-line theory to derive analytic solutions for delta formations from rivers discharging sand. BAKKER and EDELMAN (1965) further investigated the possibilities of closed form solutions of river delta evolution, assuming a somewhat different sand transport equation to allow for an analytical approach. In BAKKER (1969), the one-line theory is extended to include two lines for describing the shoreline change: one line representing the shoreline and one representing an offshore contour. LE MEHAUTE and SOLDATE (1977) present several analytic solutions and discuss the underlying principles of the one-line and two-line theories. WALTON and CHIU (1979) give a brief review of analytic solutions, mainly concerning the dispersion of different beach fill configurations. LARSON *et al.* (1987) present a large number of analytic solutions, concerning shoreline evolution on natural beaches as well as on beaches protected by various kinds of coastal structures. HANSON and LARSON (1987) compares, through the analysis of simple examples, analytic and numerical solutions of shoreline evolution.

The one-line theory was first numerically implemented by PRICE *et al.* (1973), followed by many others. Only the most important studies will be mentioned here. REA and KOMAR (1975) present a technique for studying shoreline evolution for hooked beaches using a two-dimensional grid. HORIKAWA *et al.* (1977) discuss the effect of dredged holes on shoreline

evolution. WILLIS (1977) applies the one-line model to prototype conditions, comparing the traditional CERC formula for calculating long-shore sand transport with a new expression. Wave refraction over an irregular bottom is included. SASAKI and SAKURAMOTO (1978) report the verification of a one-line model using very precise field data. PERLIN (1979) simulates hypothetical case studies involving detached breakwaters. LE MEHAUTE and SOLDATE (1980) present an implicit numerical model and test it against field data. MIMURA *et al.* (1983) compare their computer simulations against high quality laboratory data.

As reflected in the previous paragraphs, numerous studies have been made with the one-line model to examine shoreline change in laboratory (physical) models as well as under prototype conditions. However, only KRAUS *et al.* (1985), KRAUS *et al.* (1988), and HANSON and KRAUS (1986b) present an attempt to use the model 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.

The model presented by KRAUS and HARIKAI (1983), KRAUS *et al.* (1985), and HANSON and KRAUS (1986a, 1986b), developed specifically to simulate conditions at Oarai Beach, Japan, was reformulated in a generalized form, leading to the modeling system GENESIS, making the model applicable to an arbitrary open-coast beach.

Moreover, the equilibrium beach profile concept led to the development of the so-called "n-line" or "multi-line" model, in which cross-shore sand transport, and associated changes in the bottom profile, can be characterized to some extent, as well as the cross-shore distribution of the long-shore transport rate. This was first accomplished by treating two contour lines (BAKKER, 1969; BAKKER *et al.*, 1971) in terms of analytic solutions and by HORIKAWA *et al.* (1979) by using a numerical model. A numerical model representing the bathymetry by an arbitrary number of lines was presented by PERLIN and DEAN (1979, 1983). However, multi-line models are much more costly to run, in terms of both required computer memory and execution time, as compared to a one line model. In addition, due to a lack of understanding of the physical phenomena involved, in particular of the cross-shore transport rate, for

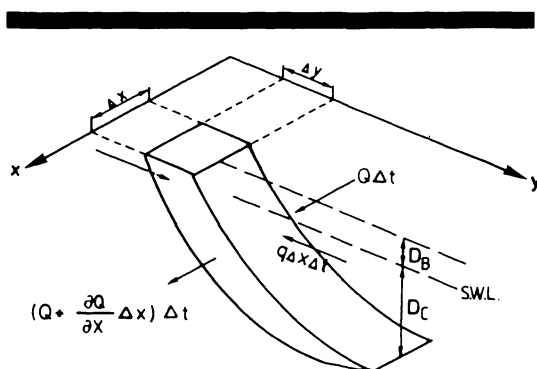


Figure 1. Continuity equation interrelations.

which no reliable quantitative relation has yet been established, these multi-line models have not found much engineering use.

LIMITATIONS AND ASSUMPTIONS

The fundamental assumption of the one-line model is, that the bottom profile does not change in time. Important implications of this assumption are that only longshore sand transport can be taken into account and that the profile is always in equilibrium. The second major assumption of the model is that sand actively moves over the profile to a certain limiting depth, beyond which the bottom does not move. This depth is called the depth of closure, D_C .

As explained in more detail below, for the wave and sand transport calculations in GENESIS, the bottom profile is assumed to follow the shape of the equilibrium beach profile (BRUUN, 1954; DEAN, 1977). One implication of this is that the depth increases monotonically. Thus, a particular point on the beach profile can be determined uniquely from the water depth, and a location at a greater water depth is always seaward of one at a lesser depth.

FUNDAMENTAL RELATIONS

Mass Conservation

Following the assumption that the bottom profile moves in parallel to itself to the depth of closure, continuity of sand for an infinitely small length, dx , of shoreline can be formulated as (Figure 1):

$$\frac{\partial y}{\partial t} + \left(\frac{\partial Q}{\partial x} + q \right) / (D_B + D_C) = 0 \quad (1)$$

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), Q is the longshore sand transport rate (m^3/s), and q represents line sources and/or sinks along the coast ($m^3/s/m$ shoreline) – positive for sources. In order to solve Equation (1), expressions for the three quantities D_C , Q , and q must be formulated. The berm height, D_B , is taken from the measured or assumed profile.

Depths of Sand Transport and Profile Change

For applications involving bypassing of sand at structures, knowledge of the depth to which sand is actively transported alongshore is required. This depth, assumed to be related to the incident wave conditions which vary with time, is here called the depth of longshore transport, D_{LT} . Without cross-shore sand movement, the beach profile would change between this depth and the shoreline only, whereas other parts of the profile would not move. However, on real beaches cross-shore sand transport acts to smooth out the profile.

As previously mentioned, studies of beach change taking place over a long period of time (years) indicate that the profile varies out 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., KRAUS and HARIKAI, 1983; WILLIS and PRICE, 1975; SUNAMURA and HIRAKAWA, 1977; WALTON and CHIU, 1979; HANDS, 1984). 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 for calculating the depth of closure as:

$$D_C = 2 H_{mas} \quad (2)$$

where H_{mas} is the maximum annual significant wave height (m) for the existing shore site. The value of H_{mas} for a given site must be specified

by the user when operating GENESIS. 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. The berm crest height, D_B , is specified by the user in the input file.

To the author's knowledge, no reliable quantitative relation between the instantaneous wave climate and the depth of longshore transport has been reported in the literature. However, as the relation presented by HALLER-MEIER (1983) appears to be very well justified by data, this relation is assumed to be valid also on a short term basis (hours). This makes possible the formulation of such a quantitative expression for the depth of longshore sand movement according to:

$$D_{LT} = 2.3 H_s - 10.9 \frac{H_s^2}{L} \quad (3)$$

where H_s is the significant wave height (m) and L is the wave length (m), both calculated in deep water. The second term in this equation is typically one order of magnitude smaller than the first; the depth of longshore transport is thus approximately twice the significant wave height in deep water. Thus, we can conclude that the longshore transport takes place well beyond the breaker line.

As a conclusion, although the longshore transport is assumed to be confined to a limited portion of the active beach profile, from the shoreline to the depth of longshore transport, the on-offshore water particle velocity under waves, and mean wave-induced and tidal cross-shore currents as well as eolian transport on the dry beach cause the beach profile to move, while its shape remains relatively unchanged from the berm crest to the depth of closure (*c.f.* Figure 1).

From the above discussion, it is seen that whereas the depth of longshore sand transport, D_{LT} , associated with the instantaneous longshore sand movement, determines the amount of sand bypassing groins, the calculated shoreline change is related to the depth of closure, D_C , which includes a variety of physical phenomena and is evaluated over a longer period of time.

Longshore Sand Transport

As emphasized by KRAUS *et al.* (1981) and KRAUS and HARIKAI (1983) a realistic sim-

ulation 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

$$Q = (H^2 C_g)_b (a_1 \sin 2\alpha_{bs} - a_2 \cos \alpha_{bs} \frac{\partial H}{\partial x})_b \quad (4)$$

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:

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

$$a_2 = \frac{K_2}{8 (\rho_s/\rho - 1) (1 - p) \tan \beta 1.416^{5/2}} \quad (6)$$

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 from the shoreline to the depth of longshore transport, D_{LT} . The factor 1.416 is used to convert from significant to RMS wave height.

The first term in Equation (4) 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. The calibration parameters K_1 and K_2 determine not only the relative strength between the two terms, but also the time scale in the model.

Bypassing at Groins

A thorough analysis of sand being transported by waves around groins would have to include the cross-shore distribution of the longshore sand transport rate, as well as the two-dimensional horizontal pattern of sand transport. However, up to date, there is no such reliable predictive expression that has been verified for prototype conditions. Under these circumstances, the simplest assumption producing reasonable results was adopted.

Thus, in GENESIS, the longshore sand transport rate is assumed to have a uniform cross-

shore distribution. Although simple, it has been shown to work well. Moreover, recent field measurements of the sand transport, made by US Army, Coastal Engineering Research Center, supports the chosen approach (KRAUS and DEAN, 1987).

Bypassing around a groin is assumed to take place when the depth at the groin tip, D_G , is smaller than the depth of longshore transport, D_{LT} . Having determined the shape of the bottom profile the depth D_G is uniquely determined from the distance between the groin tip and the shoreline location. However, as groins are always located on the boundary between two calculation cells, this depth is not unique. In GENESIS, the conditions on the up-drift side are used.

The bypassed amount of sand is determined in terms of a bypassing factor BYP ($0 \leq BYP \leq 1$), denoting the fraction passing by the groin out of the up-drift transport rate, and calculated as:

$$BYP = 1 - \frac{D_G}{D_{LT}} \quad (7)$$

When the water depth at the tip of the groin is greater than the depth of longshore transport, BYP is set to zero. As BYP depends on the distance to the shoreline, its value will change each time step.

It is recognized that the procedure of calculating groin bypassing is arbitrary and difficult to verify. Our understanding of the highly complex interaction between groins and sandy beaches is very poor. Still, through simulations of hypothetical as well as real beaches, it has been found that the groin bypassing algorithm produces reasonable results.

Angled Groin

Groins built normal to the shoreline often suffer from significant shoreline fluctuations adjacent to the groin. In order to diminish this effect, groins are often oriented in the direction of the predominant incident waves. As breakwaters and jetties are designed on the basis of other criteria, they also appear with an angle to the shore-normal direction. GENESIS was originally developed for simulating conditions at Oarai Beach, Japan. Here, the shoreline evolution is, to a large degree, controlled by a large angled breakwater. For this reason, the capa-

bility of simulating an angled groin/breakwater was included in GENESIS following the technique presented in PERLIN (1978).

Cross-Shore Transport

GENESIS, as a one-line model, cannot describe shoreline change produced by cross-shore transport as caused, *e.g.*, by a change in wave steepness. However, cross-shore transport can be simulated in a schematic way, in terms of non-wave induced sources and/or sinks along the coast (*e.g.*, discharge from rivers, shoaling of harbors, removal of sediment by mining, *etc.*).

Bottom Profile

The continuity equation (Equation 1) does not require specification of the shape of the bottom profile, since it was derived under the assumption that the profile moves in parallel to itself. However, in order to calculate the average nearshore bottom slope, $\tan \beta$, to be used in the transport equation (Equation 4), as well as for determining the location of the breaking waves, a shape of the profile is needed.

According to BRUUN (1954) and DEAN (1977), the shape of the bottom profile can be expressed as:

$$D = A y^{2/3} \quad (8)$$

where D is the water depth (m), A is a scaling parameter ($m^{1/3}$), and y is the distance from the shoreline (m). MOORE (1982) has given an empirically determined curve for A as a function of grain size. Defining $\tan \beta$, appearing in the second term in Equation 4, as the average bottom slope from the shoreline out to the depth of longshore sand movement, D_{LT} , leads to:

$$\tan \beta = \left[\frac{A^3}{D_{LT}} \right]^{1/2} \quad (9)$$

Wave Refraction

Local wave refraction from the input (reference) water depth to the breaker line is simulated using Snell's Law for refraction of light. One of the basic assumptions in the original one-line formulation, is that the offshore contours move in parallel to the shoreline (Figure 2). If applied to the wave model in GENESIS,

this assumption may produce either numerical instability or unrealistic wave refraction. Therefore, GENESIS is calculating an offshore contour, representing the trend of all bottom contours. The curvature of this line is obtained by smoothing the shoreline to reflect the major features in the shoreline, but to filter out possible abrupt variations. Local wave refraction in GENESIS is performed using the orientation of this line.

Wave Diffraction Behind Structures

For monochromatic wave analyses, such as theoretical or laboratory studies, the diffraction coefficient is mostly calculated using the Sommerfeldt solution for the diffraction of light (WIEGEL, 1964, Chapter 8). However, as demonstrated by GODA *et al.* (1979), the directional spreading of real wind-generated waves results in a higher diffraction coefficient than predicted by monochromatic wave diffraction theory. Being developed primarily to reproduce prototype conditions, GENESIS therefore uses the simplified diffraction calculation procedure for waves with directional spread presented by GODA *et al.* (1979) and GODA (1984) to represent diffraction at structures such as detached breakwaters and jetties.

The degree of directional spread in a wave

field is represented by a wave concentration parameter denoted by S_{max} . With S_{max} determined at the tip of the diffracting structure (for details see GODA *et al.*, 1979 or HANSON, 1987), the diffraction coefficient, K_D , along a line making the angle θ to the incident wave direction at the tip can be obtained from Figure 3. These are produced using the closed form approximation given in KRAUS (1981) for the original energy cumulative curves in GODA (1984). The area where the angle is negative is called the shadow region, taking the diffraction of light as an analogy. Consequently, the area where the angle is positive is called the illuminated region.

In field applications, the method described in this section, does not only provide a better description of prototype wave diffraction as compared with monochromatic diffraction theory. It is also computationally much faster. The quantity S_{max} is only calculated once for each diffraction source and time step. Then K_D is given at each location alongshore using closed form approximations for the curves in Figure 3. In monochromatic wave theory, the Fresnel integrals have to be evaluated at each calculation point.

In applications involving harbor design, the more rigorous procedure also presented in GODA *et al.* (1979) and GODA (1984) is rec-

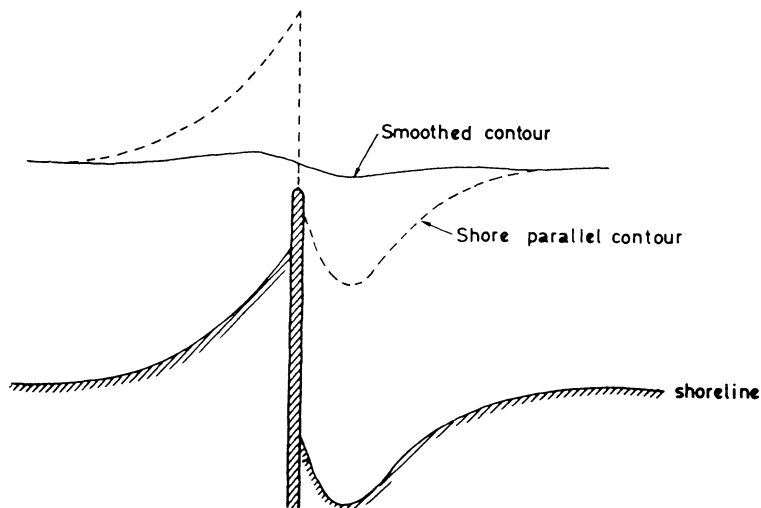


Figure 2. Shore parallel versus smoothed bottom contours.

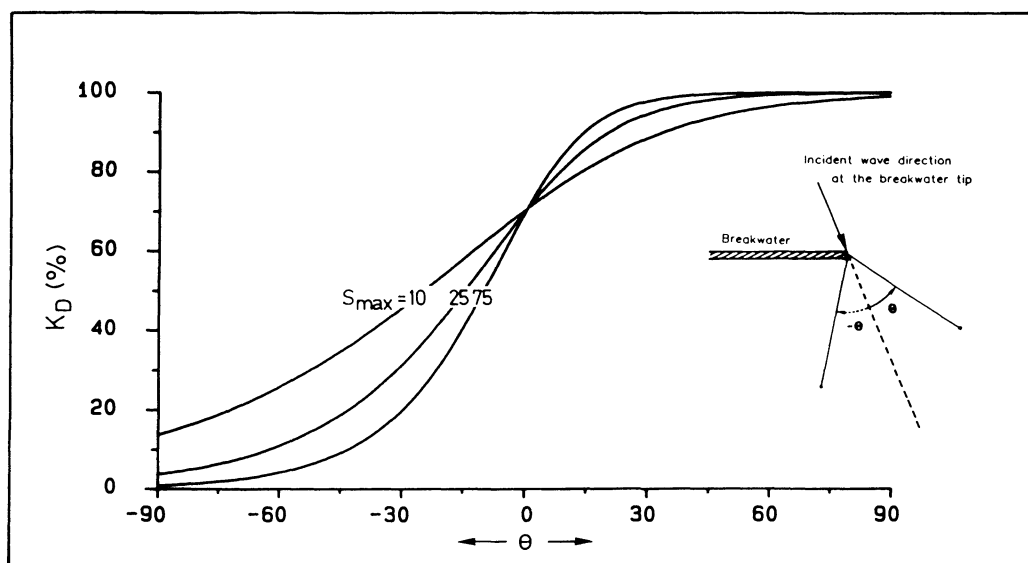


Figure 3. Wave diffraction curves.

ommended, but not reviewed here. For numerical modeling of monochromatic laboratory waves, the Sommerfeldt solution is more appropriate.

Wave Breaking

According to Equation (4), the longshore sand transport rate is controlled by the wave characteristics at the breaking point. In GENESIS, the standard depth-controlled spilling breaker criterion is used:

$$H_b = 0.78 D_b \quad (10)$$

Combined Refraction/Diffraction

The success of a rigorous calculation of the combined refraction/diffraction of waves behind structures is largely limited by the lack of precise knowledge of the wave breaking phenomena and the two-dimensionality of the problem. Also, from a computer run time point of view, the procedure for calculating the wave height and angle behind a structure should be as simple as possible. Still, from an engineering standpoint, the calculated results must show reasonable agreement with observed prototype data.

In GENESIS, although regarded as being simultaneous, the two physical phenomena, wave refraction and diffraction, are treated separately. Outside a region influenced by diffraction, the wave height is given by:

$$H = H_{rf} K_R(\alpha_{rf}, D_{rf}, D) K_S(D) / K_S(D_{rf}) \quad (11)$$

where H_{rf} is the wave height at the input (reference) depth D_{rf} , K_R is the refraction coefficient, α_{rf} is the wave angle at the reference depth, K_S is the shoaling coefficient, D is the local water depth given by Equation (6). The ratio $H_{rf} / K_S(D_{rf})$ represents the wave height in deep water if the wave is not refracted (usually denoted by H_0). The wave height, H , is then compared with the possible breaking wave height at the particular depth. If breaking conditions are not reached, the calculation moves to a point closer to shore, until the breaking criterion is satisfied.

In a shadow region, the wave height is given by:

$$H = H_{tp} K_D(\theta) K_R(\alpha_{tp}, D_{tp}, D) K_S(D) / K_S(D_{tp}) \quad (12)$$

where K_D is the diffraction coefficient, θ is the geometric angle for a line from the diffracting tip to the point considered, measured relative to the wave direction at the diffracting tip (Figure

3). The subscript, tp, refers to conditions at the diffraction tip.

Wave Energy Windows

A central concept used in GENESIS, and one which determines the algorithmic structure of the model, is that of wave energy windows. This concept is simple, but provides a powerful and general means to describe a variety of structural configurations and physical conditions. An energy window is defined as an area open to the incident waves. Windows are separated by structures, such as a detached breakwater or a diffracting groin. Incident wave energy must enter through one of these windows to reach a location in the nearshore area (Figure 4).

Multiple Diffraction

For each energy window, the diffraction coefficients for the two tips are calculated and denoted K_{D1} and K_{D2} , respectively. If a window is open to one side, the corresponding diffraction coefficient is set to 1.0. The effective diffraction coefficient, taking both diffracting tips into account, is calculated as (Figure 5):

$$K_{D12} = K_{D1} K_{D2} \quad (13)$$

SENSITIVITY ANALYSIS

Effect of Input Errors

The measurement of prototype wave characteristics (height, period, and direction) is a dif-

ficult task. When using such data as input to a numerical (or any other type of) model, it is therefore important to be aware of the potential uncertainties involved in the determination of these wave data, as well as the effects any errors might have on the model predictions. In this section, a simple sensitivity analysis is made, as an attempt to obtain a quantitative measure of the effects of small errors in the breaking wave height and angle. The change in the calculated value of the longshore sand transport rate, Q , is used as the sensitivity criteria, as this is the primary variable of importance for the shoreline change. The analysis is carried out to the first order, which is accurate within 1 to 2 percent. Assuming shallow water at the location of wave breaking, the wave group velocity, C_{gb} , can be approximated:

$$C_{gb} \approx C_b \approx \sqrt{g D_b} = \sqrt{g H_b / \gamma} \quad (14)$$

where C_b is the breaking wave celerity (m/s), γ is the breaker index ($= H_b / D_b$, c.f. Equation 10, $\gamma = 0.78$), and g is the acceleration of gravity (m/s^2). This relation inserted into Equation (4) with $a_2 = 0$ and using α as short for α_b , yields:

$$Q = Q(H, \alpha) = (H^{5/2} \sin 2\alpha)_b a_1 \sqrt{g/\gamma} \quad (15)$$

The relative error in Q due to an error dH in the breaking wave height can be determined as, approximated to the first order in a Taylor series (omitting the subscript b for breaking):

$$\begin{aligned} \frac{Q(H \pm dH, \alpha)}{Q(H, \alpha)} &= \frac{(H \pm dH)^{5/2}}{H^{5/2}} \approx 1 \pm \frac{5}{2} \frac{dH}{H} \quad (16) \end{aligned}$$

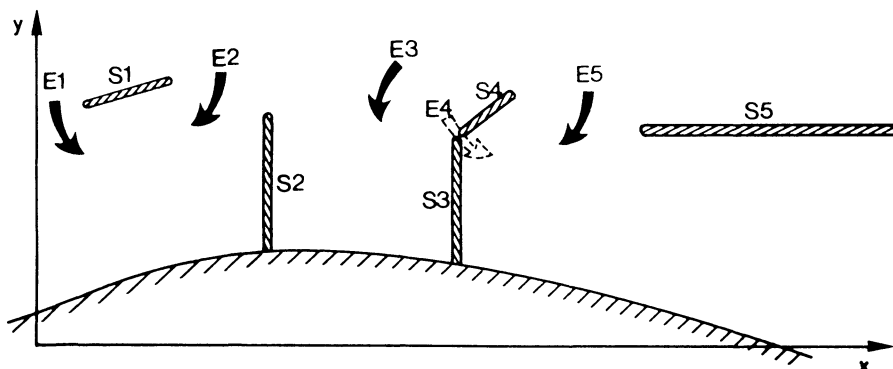


Figure 4. Wave energy windows E1 - E5 and structural elements S1 - S5.

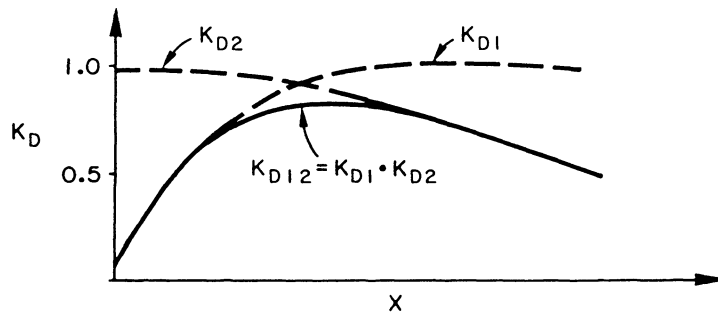


Figure 5. Schematic effective diffraction coefficient. (From HANSON and KRAUS, 1986b)

A similar analysis for a wave angle error $d\alpha$ gives:

$$\frac{Q(H, \alpha \pm d\alpha)}{Q(H, \alpha)} = \frac{\sin(2\alpha \pm 2d\alpha)}{\sin(2\alpha)} \\ \approx \frac{\sin(2\alpha) \pm 2d\alpha \cdot 2\cos(2\alpha)}{\sin(2\alpha)} \approx 1 \pm 2 \frac{d\alpha}{\alpha} \quad (17)$$

Consequently, if the two errors appear simultaneously, the relative error in Q would be:

$$\frac{Q(H \pm dH, \alpha \pm d\alpha)}{Q(H, \alpha)} \approx \left(1 \pm \frac{5}{2} \frac{dH}{H}\right) \left(1 \pm 2 \frac{d\alpha}{\alpha}\right) \\ = 1 \pm \frac{5}{2} \frac{dH}{H} \pm 2 \frac{d\alpha}{\alpha} \pm 5 \frac{dH}{H} \frac{d\alpha}{\alpha} \quad (18)$$

Assuming the errors dH and $d\alpha$ to be 10% each, which is considered to be a low number, the relative errors in Q would be 25%, 20%, and 50% (!) for the three respective cases. Thus, it is seen that even with small errors in the determination of breaking wave heights and angles result in significant errors in the longshore sand transport rate. Viewing Equation (1), it is seen that deviations of the same order will appear in the shoreline change calculation. With this in mind, it is reasonable to obtain variations in the calibration parameters by a factor of 2 or more from one site application to another.

Accuracy and Efficiency of Solution Schemes

The numerical scheme in GENESIS can be solved either with an explicit or an implicit solution technique. In the explicit scheme the new shoreline position depends only on values calculated at a previous time level, while in the

implicit scheme values on the old as well as on the new time level is used. The main advantages of the explicit scheme are easy programming and simple expression of boundary conditions. A major disadvantage is, however, the stability of the solution, expressed by a ratio between the time step and the finite grid length alongshore. This stability ratio, expressed by a parameter R_s , imposes a severe constraint on the longest possible time step for given values on model constants and parameters. The main advantage of the implicit scheme is that it is stable for almost any value of the stability parameter, although the computed results become increasingly inaccurate with larger stability ratios.

In order to compare the relative accuracy and efficiency of the two solution schemes used in GENESIS, the shoreline evolution in a hypothetical pocket beach was simulated. The accuracy is determined in terms of the calculated shoreline position and the efficiency in terms of the execution time. Although the obtained results are necessarily site-specific, previous similar analyses (KRAUS and HARIKAI, 1983) support the results quantitatively.

The curved pocket beach is 2 km long and bounded by two headlands which contain the sand transport (Figure 6). A seawall is located 4 meters behind the initial shoreline. The waves arrive from the right side of the Figure for 126 hours, resulting in the shoreline planform shown in Figure 6b. As expected, the sand is moved towards the left headland. The shoreline change, that would have occurred without the presence of the seawall, is represented by a dashed line. The wave height increased linearly

from 2.5 meters at the right headland to 3 meters at the left headland. The wave period was 8 seconds.

In the comparisons presented in Table 1, the time step, DT , was varied. The calculation run made the explicit solution scheme with $DT = 6$ hours ($R_s = 0.51$), was used as reference, Δy_r . In applications using the explicit solution scheme it is found that for R_s values greater than about 0.5 the scheme becomes unstable. The calculated shoreline changes adjacent to and 500 m from the right headland were used for comparison. As seen in Table 1, the explicit scheme is computationally faster than the implicit scheme using the same time increment, DT . On the other hand, the implicit scheme can take larger time steps while preserving reasonable accuracy.

Viewing the values for the accuracy of the implicit scheme in Table 1, we can conclude that running the implicit calculation scheme with a time step four times longer than the longest possible time step for the explicit scheme, the total execution time was reduced to about 2/3 of that of the explicit scheme. Still, the difference between the results for the explicit and implicit schemes, $(\Delta y - \Delta y_r)/\Delta y_r$, was less than 1 per cent. Using values on R_s of about 0.5 and 5.0 in the explicit and implicit schemes respectively, the implicit scheme is up to about three times faster and therefore the method normally used in the calculations. For a simulation not involving a seawall, the run-time comparison above would be even more to the advantage of the implicit scheme.

Effect of Discretization in Space and Time

The size, DX , of the calculation cells is determined on the basis of a compromise between computer run-time/memory/cost and space resolution. The time step, DT , is determined in a similar way. In addition, a desire to update the waves with a certain periodicity, as well as limited information about the waves will affect the choice of DT .

Besides these considerations, for any type of numerical model, we have to make sure that the calculated results are grid and time step independent. In order to investigate the sensitivity of the model output to the size of the discrete steps in space and time, a series of calculations were performed. In all cases, the stability parameter R_s was held constant and low ($= 0.26$) in order to avoid the effect of stability errors. The calculation time in each simulation was 480 hr. Other parameters were varied according to Figure 7.

In all runs, the breaking wave height and angle were held constant, at 0.7 m and -15 degrees respectively. The result of the calculations is shown in Figure 7. As a comparison, the analytic solution for the same case is presented in the Figure. Case No. 3 represent typical values of DX and DT for field applications.

As seen the differences are very small even for extremely large steps, indicating a negligible grid and time step dependence. The difference between case 1 and the analytic solution (A) is due to the violation of the small angle assumption in the analytic solution.

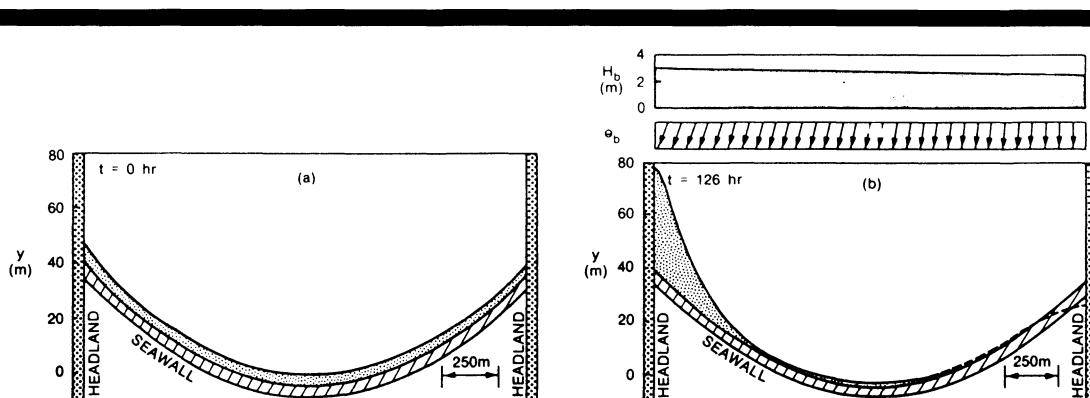


Figure 6. Hypothetical example of shoreline change along a curved pocket beach backed by a seawall. (From HANSON and KRAUS, 1986a)

Table 1. Stability and accuracy of explicit and implicit solution schemes. (From Hanson and Kraus 1986a)

Δt (hr)	Stability parameter R_s	Relative execution time	$\frac{\Delta y - \Delta y_r}{\Delta y_r}$	%
<i>Explicit scheme</i>				
			$i=1$	$i=10$
1	0.08	5.30	-0.6	-3.1
2	0.17	2.72	-0.5	-2.8
4	0.34	1.43	-0.2	-1.2
6	0.51	1.00	0.0	0.0
8	0.67	unstable		
<i>Implicit scheme</i>				
			$i=1$	$i=10$
6	0.51	2.24	-0.5	-3.1
12	1.01	1.19	-0.2	-2.0
24	2.02	0.67	-0.7	0.0
60	5.05	0.35	13.4	2.3
120	10.11	0.24	22.1	-7.9

Effect of Wave Sequence

Even if the statistical properties of the future wave climate are determined (which is a difficult task in itself), the exact sequence of future events can not be known. Still, as shown by LE MEHAUTE *et al.* (1983), the calculated shoreline position is sensitive to the order of wave angle sequence, especially for open beaches not affected by diffraction structures. Therefore

when forecasting shoreline evolution for real beaches, the future shoreline configurations cannot be presented individually. Instead, it is more appropriate to generate a band of shorelines, using waves with different sequences, within which the "true" shoreline can be expected to lie.

In order to investigate the influence of wave angle sequence on GENESIS, the shoreline evolution near a groin was analyzed. For this reason, a set of 320 wave triplets (wave height, period, and direction) was produced. The same set was used to produce all shorelines shown in Figure 8. Only the relative order of the triplets was varied. Thus, the total wave energy flux was held constant. In all runs, the breaking wave height was held constant (1.4 m), while the breaker angle was varied. The total simulation time was 480 hr.

As an attempt to examine the maximum impact of resequencing, two unrealistic, ordered wave sequences were examined. In the first set, the wave angle increased linearly from -15 to 15 degrees, and in the second the angle decreased linearly between the two limits. The calculated shorelines are displayed as line 1 and 2 in Figure 8.

As seen, the two sets of waves result in fun-

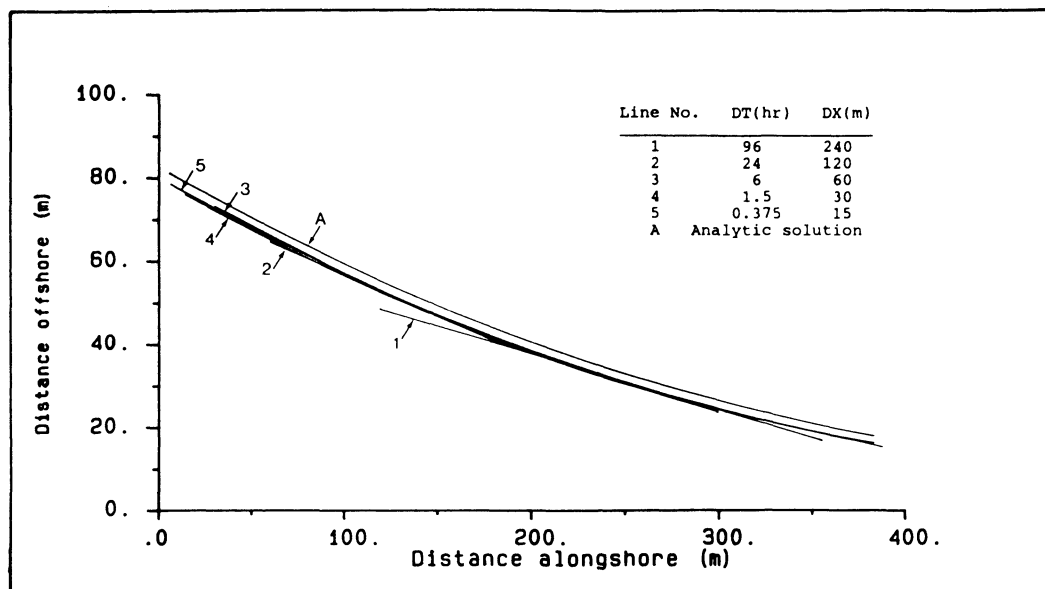


Figure 7. Influence of grid size and time step on the calculated accumulation up-drift of a groin.

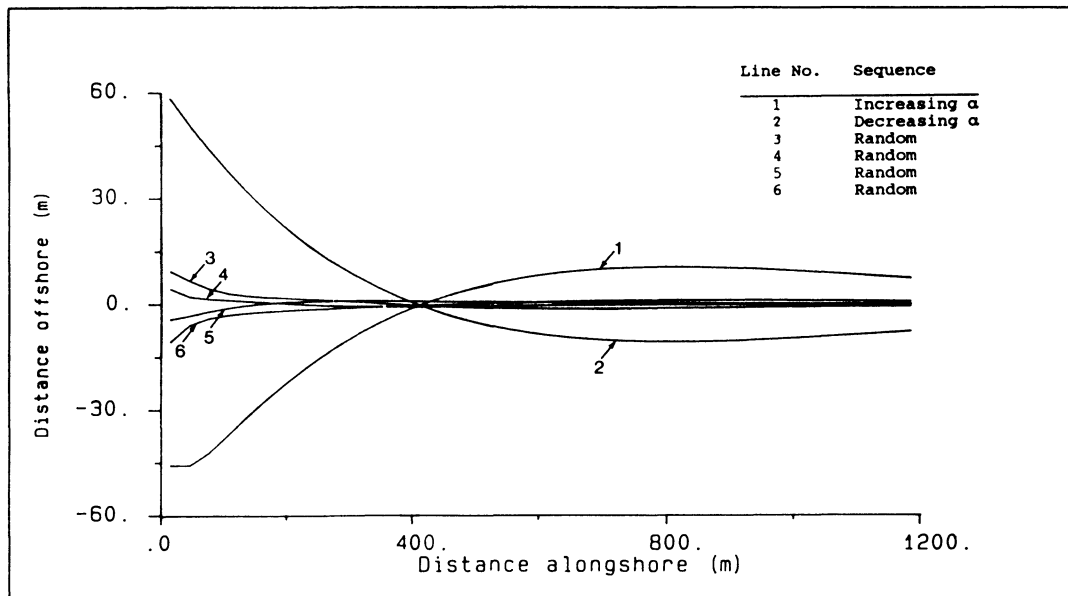


Figure 8. Influence of wave angle sequence on shoreline change near a groin.

damentally different shorelines. In addition, a large number of shorelines were simulated, using wave sets obtained using a Monte Carlo simulation technique to resequence the original data set. Four of these are shown as lines 3 to 6 in Figure 8. The shoreline changes for these simulations were rather small as expected, as the angle varies randomly around its mean value ($\alpha = 0$).

The analysis can be extended to include also variations in wave height. In the simulations shown in Figure 9, the breaking wave angle was held constant ($\alpha = -15$ degrees). The breaking wave height varied between 0 and 1.4 meters, thus with the same average height as in the previous case. Figure 9 shows small differences between the two extreme sequences with the wave height increasing (line 1) and decreasing (line 2) linearly between the two limits. This is consistent with the observation made by LE MEHAUTE *et al.* (1983). As clearly demonstrated by line 3, the average situation cannot be represented by the average wave height. In this simulation the total energy flux is less than for the other curves, explaining the position of the beach well behind the others. If, instead, a constant wave height corresponding to the mean wave energy flux is used, a shore-

line (line 4) falling between the two extremes is produced. A number of Monte Carlo simulations were also made, but since they all fell on top of lines 1, 2 and 4, they were not included in Figure 9.

As a conclusion, shoreline evolution is extremely sensitive to wave angle sequence, whereas for the wave height, an energy flux weighted mean can be used, provided that the wave height is fairly independent of wave direction.

Groin Permeability

Real groins often, intentionally or not, allow sand to pass through. In GENESIS, the groin sand permeability, defined as the fraction passing through the groin out of the upstream sand transport rate is specified for each groin. However, it is recognized that the permeability is difficult to quantify. Therefore, it is important to investigate the sensitivity of GENESIS to variations in the value of this parameter. For this reason, a series of simulations were made, illustrating the influence of groin permeability on the sand accumulation up-drift of a groin exposed to 0.7 m breaking waves with an angle

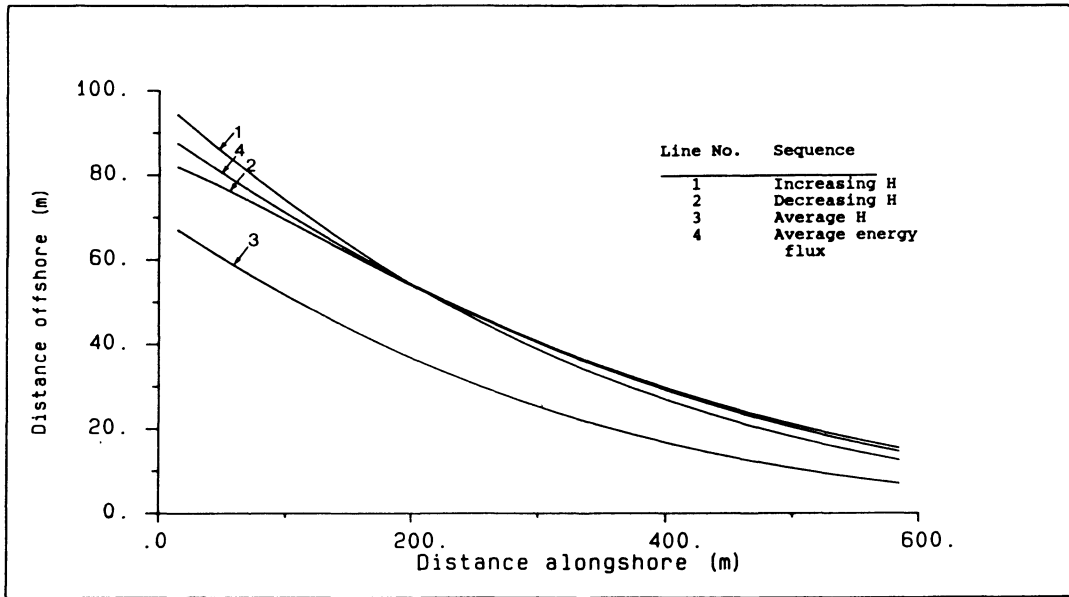


Figure 9. Influence of wave height sequence on shoreline change near a groin.

of -15 degrees to the x-axis for 480 hours (see Figure 10).

If the longshore sand transport would be inde-

pendent of the shoreline orientation, $\partial y/\partial x$, the difference in shoreline location, close to the groin, would be proportional to the difference in

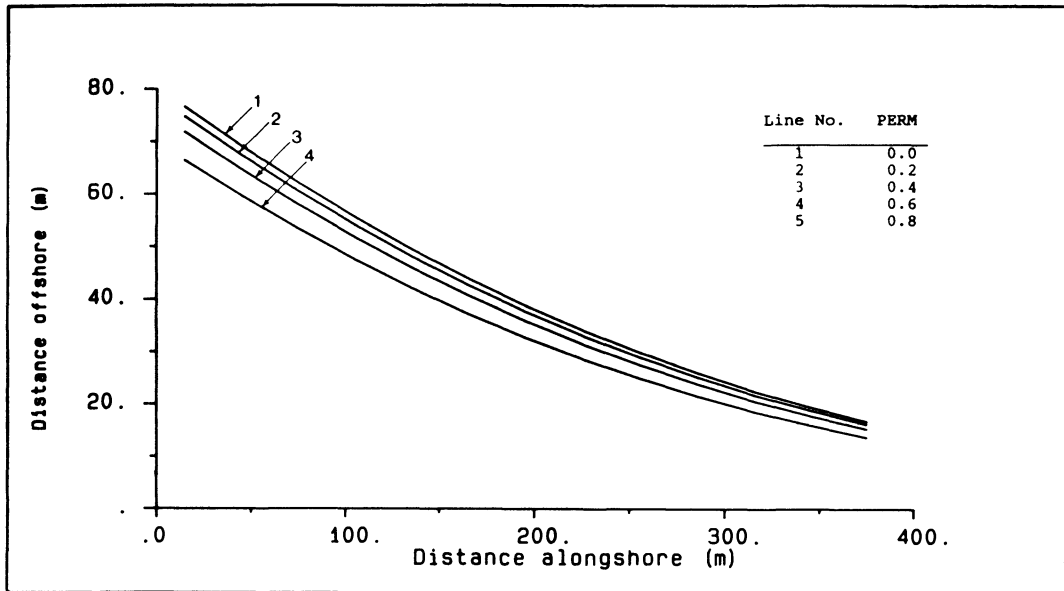


Figure 10. Influence of groin permeability on shoreline change near a groin.

permeability. As, in one-line models, such a dependence exists, the decrease in sand accumulation caused by the permeability is partly compensated by the reduced speed at which the sand transport rate decreases up-drift of the groin. This is confirmed in Figure 10 where the differences between the runs are very small. If diffraction was not taken into account, the eroded shoreline down-drift of the groin would be antisymmetric to that on the up-drift side.

A precise determination of the groin permeability is not possible to make. Fortunately, we can conclude, that GENESIS is rather insensitive to changes in this value.

Sand Grain Size

The sand grain size enters GENESIS through the equilibrium beach profile. A finer sand material results in a more gentle beach profile slope, causing the waves to break further offshore. However, in GENESIS the breaking wave height is not affected, in areas not influenced by diffraction. The average beach slope, $\tan\beta$, appears in the second term in the transport equation. Thus, a steeper beach decreases the influence of this term.

Inside a diffraction zone, the breaking wave height and angle are strongly affected, since these parameters depend on the breaking location. The general implication of this is that a coarser bed material results in smaller shoreline changes. On the down-drift side of a groin, a coarser sand material will cause the waves to break closer to shore, deeper into the shadow region. The breaking wave heights and angles will be smaller, resulting in less erosion close to the groin. In addition, the wave height gradient, $\partial H/\partial x$, will increase close to the groin, again resulting in less erosion.

A fundamentally different situation is displayed in Figure 11, showing the shoreline change behind a detached breakwater exposed to 1.4 m breaking waves, with a period of 5 sec and perpendicular to the initial straight shoreline. The simulation time was 50 hr. The breakwater is 200 m long and placed 200 m from the initial shoreline. The influence of grain size is similar to that in the groin case. A finer bed material causes the waves to break further offshore. As a result, the breaking wave heights will be smaller and the direction will be pointing more into the shadow zone. Thus, both

terms in Equation (4), through α_{bs} and $\partial H/\partial x$, will attract more sand into the shadow zone behind the breakwater, resulting in larger salients.

Although the impact of sand grain size can be determined qualitatively, a quantitative measure is not possible to give. In the case of a detached breakwater, as discussed above, the effect depends on physical parameters such as wave period, length of the breakwater and its distance from the shore, but also on the values of the model calibration parameters, K_1 and K_2 , for the case above set arbitrarily to 0.5 and 0.3 respectively.

For real beaches, the choice of a representative sand grain size has to rest on engineering judgement. For many beaches, significant variations appear both in the alongshore and in the cross-shore distribution, the latter usually being the greater. BASCOM (1951) shows, on the basis of data on the US Pacific Ocean coasts, that the cross-shore sand grain size varies with a factor of about 2 in the nearshore area.

Berm Height and Depth of Closure

As seen from Equation 1, the shoreline change is inversely proportional to the sum of D_B and D_C . Whereas the depth of closure is determined by the maximum significant wave height, the berm height has to be specified by the user. For many real beaches it can be difficult to give a representative value of the berm height. As a result, the user specified average berm height value, will exceed the real height on some parts of the beach and be below on others. A user of the model therefore needs to know the sensitivity of the model to variations in these two parameters, and qualitatively how an over- and under-estimation, respectively, may change the simulated shoreline change along a beach.

Four runs of evenly spaced values of $1/(D_B + D_C)$ were made. The beach was exposed to 0.7 m waves with an angle of -15 degrees to the x-axis. The simulation time was 480 hrs. As demonstrated in Figure 12, the simulations show the same qualitative features as the groin permeability simulations above. Again, the correlation between shoreline orientation and sand transport rate explains the relative small sensitivity of GENESIS to changes in the input parameters. Whereas the depth of closure,

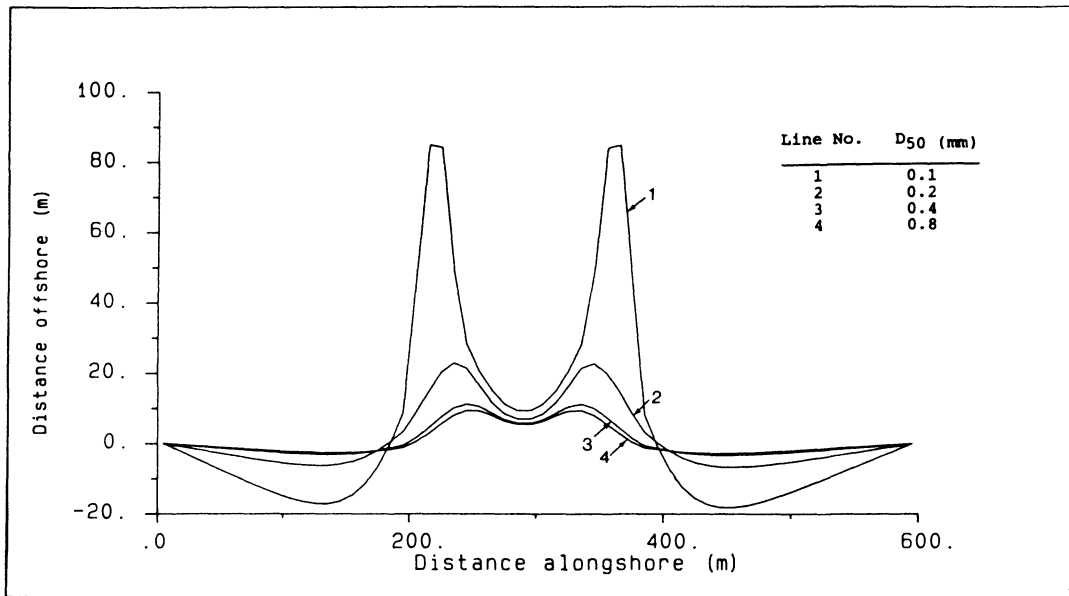


Figure 11. Influence of sand grain size on shoreline change behind a detached breakwater.

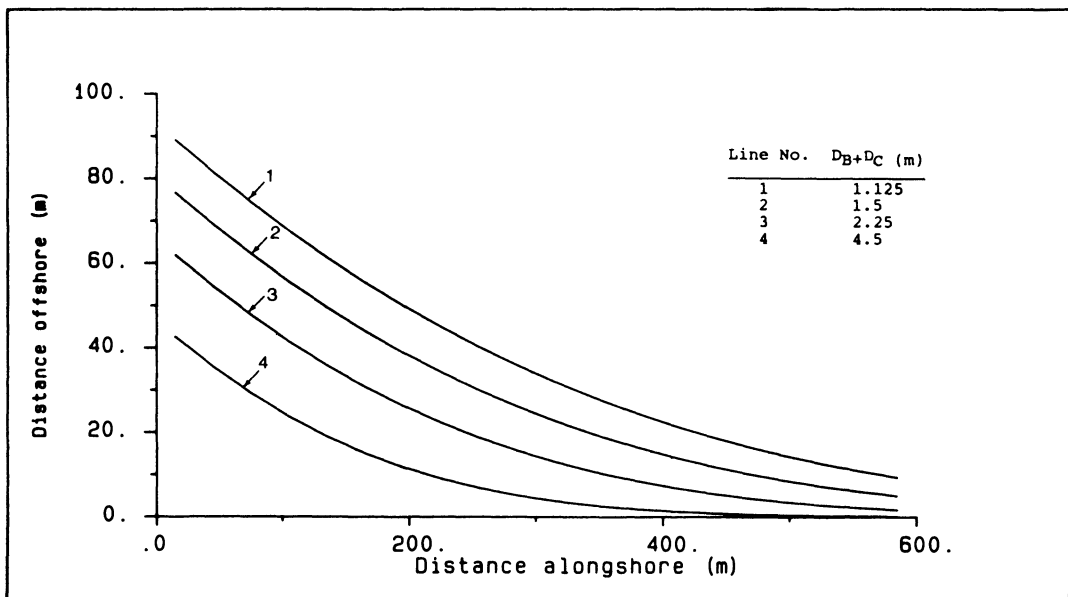


Figure 12. Influence of berm height and depth of closure on shoreline change near groin.

between the first and the last runs was increased by a factor of 4, the calculated shoreline change only decreased to about 50 per cent.

MODEL STRUCTURE

GENESIS can be thought of as consisting of two models—a wave model calculating the breaking wave characteristics alongshore and a transport model calculating the longshore sand transport and the associated shoreline change. The wave model in GENESIS was developed to describe a wave field dominated by diffraction by structures. As such it is based on the assumption of plane and parallel bottom contours. However, for open-coast calculations without diffracting structures it may be desirable to use a more sophisticated wave model for bringing the waves from deep to shallow water over an irregular bottom topography. At present, GENESIS is set up to communicate with a regional linear wave transformation model, RCPWAVE (EBERSOLE, 1985; EBERSOLE *et al.*, 1985), giving the pre-breaking wave conditions. Subroutines in the wave model part of GENESIS then bring the nearshore waves to the breaking point (for details, see HANSON, 1987).

A numerical model of shoreline change can be a very powerful tool for predicting shoreline change under complex design and wave conditions. However, it is of great importance for the user to correctly operate the model and to interpret the results appropriately. The user must be aware of all the underlying assumptions and simplifications, as well as the general characteristics of the model. It is therefore strongly recommended that the user of the model should operate it for various simple conditions, to see how the model performs, before applying it to a prototype case. Accordingly, considerable effort was devoted to simplify the input interface of GENESIS and to structure the program logically and lucid. The main structure of GENESIS is shown in Figure 13 in which names of subroutines are enclosed by solid lines and names of data files by dashed lines. GENESIS is operated through interaction with data files, developed to allow representation of a large number and variety of coastal structure and shoreline configurations.

In order to give a picture of the overall structure of GENESIS, a brief presentation of the

subroutines is presented below. The subroutines are listed in order of their use. The principal functions of the individual subroutines are described in more detail in HANSON (1987).

SHOIN:	Reads the initial shoreline position.
SWLIN:	Reads the seawall position.
DEPIN:	Reads the depths along the shoreline.
WAVIN:	Reads the wave climate.
OFFIN:	Calculates an offshore contour used for wave refraction.
BFILL:	Keeps track of and performs beach fill operations as necessary.
SNELL:	Refracts the waves.
FINDBR:	Calculates breaking wave characteristics.
WAVSTA:	Calculates wave and sand transport statistics.
SPARAM:	Determines conditions controlling wave diffraction at tips of structures.
KDGODA:	Calculates wave diffraction.
ZBREAK:	Refracts waves inside the diffraction zones.
STABIL:	Computes the value of the stability parameter R_s .
TRANSE:	Explicit calculation of the longshore sand transport rate.
TRANSI:	Implicit calculation of the longshore sand transport rate.
BYPASS:	Computes sand bypassing at groins.
YSEXP:	Explicit formulation of the constraint imposed on the shoreline by the presence of seawalls.
YSIMP:	Implicit formulation of the constraint imposed on the shoreline by the presence of seawalls.

First, the breaking wave heights and directions are calculated along the coast, omitting diffraction as if the structures were not there. Then, the structures are introduced, treating each wave energy window separately. When the diffracted breaking wave heights and directions are determined, the associated longshore sand transport rates are calculated. This procedure is repeated for each of the energy windows.

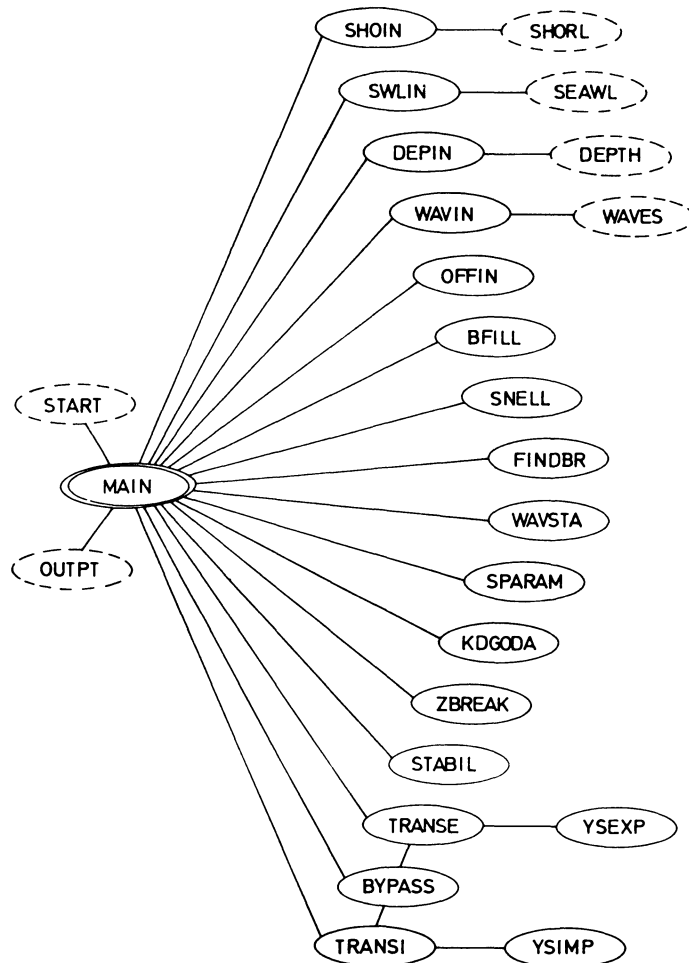


Figure 13. Main program structure of GENESIS.

Then, the transport rates are added to obtain the total rate as produced by all energy windows for each calculation element along the beach. Finally, the resulting shoreline changes are determined and, if necessary, corrected according to the seawall constraint. Details on the effects of seawalls are given in HANSON and KRAUS (1985; 1986a).

COMPARISON WITH ANALYTIC SOLUTIONS

Analytic solutions originating from mathematical models which describe the basic physics involved to a satisfactory level of accuracy, are

often valuable for investigating the properties of physical phenomena. Essential features of shoreline change in response to coastal structures or coastal engineering activities, such as beach nourishment, can easier be isolated to give qualitative insights in analytic models than in complex numerical or laboratory models. Another useful property of analytic solutions is their capability to determine equilibrium conditions from asymptotic solutions. However, it is important to be aware of the limitations of analytic solutions and the errors introduced by violating these limitations. Closed-form mathematical models cannot be expected to provide quantitatively accurate

solutions to problems involving complex boundary conditions and wave inputs. Thus, engineers are encouraged to study analytic solutions for educational purposes, while in a real design situation, the use of a numerical model of shoreline change would be more appropriate.

Although several authors have presented analytic solutions for certain simplified conditions as discussed above, very little attention has been directed towards the comparison between analytic and numerical solutions and the limits within which they are valid. This section examines shoreline evolution for simple configurations as predicted using one analytical solution (A) and two different numerical formulations (N1 and N2).

In order to be able to solve the basic equations analytically it is necessary to set $a_2 = 0$ in Equation (4). Then, the longshore transport rate can be expressed in the following general way:

$$Q = Q_o \sin(2\alpha_b) \quad (19)$$

where Q_o is the amplitude of the longshore sand transport rate, in the numerical model thus calculated as (for explanation of notation, see Eq. 6)

$$Q_o = (H^2 C_g)_b K_1 / (16(\rho_s/\rho - 1)(1 - p) 1.416^{5/2}) \quad (20)$$

Analytic Solution Technique (A)

For beaches with mild slopes, it can safely be assumed that the breaking wave angle to the shoreline is small. If also the shoreline angle to the x-axis, being oriented along the main trend of the shoreline, is assumed to be small, Equations (19) and (4) can be approximated to the first order in a Taylor series:

$$Q = Q_o \left(2\alpha_b - 2\frac{\partial y}{\partial x} \right) \quad (21)$$

If, in addition, the amplitude of the longshore sand transport rate, as well as the breaking wave angle, are assumed to be independent of x and t, and that any contributions from sources and sinks are assumed to be negligible ($q = 0$), Equations (1) and (21) can, as previously shown, be formulated as:

$$\frac{\partial y}{\partial t} = \epsilon \frac{\partial^2 y}{\partial x^2} \quad (22)$$

$$\epsilon = 2 \frac{Q_o}{D_b + D_c} \quad (23)$$

As Equation (22) is analogous to the one-dimensional heat diffusion equation, it can be solved analytically for many initial and boundary conditions. The coefficient, ϵ , can be interpreted as a diffusion coefficient expressing the time scale of shoreline change following a disturbance (wave action). A high value of the amplitude of the sand transport rate results in a rapid shoreline response, whereas a larger depth of closure, meaning that the longshore transport will be distributed over a larger portion of the beach profile, leads to a slower shoreline response.

Numerical Solution Technique (N1)

Solving Equations (1), (19), and (20) numerically, we are no longer constrained by small angle assumptions, making possible the solution of a wider variety of shoreline/structure configurations and a more realistic wave climate. However, in order to obtain a solution as close to the analytic solution as possible, in order to isolate the error introduced into the analytic solution by linearizing Equations (1) and (19), H_b and α_b were specified on the breaker line and held constant alongshore in time, as in the analytic case.

Numerical Solution Technique (N2)

Variations in space and time in the shoreline orientation, $\partial y/\partial x$, are reflected on the nearshore bathymetry. Thus, due to depth refraction, even when the offshore wave climate is constant alongshore, the breaking conditions will vary along the coast. Therefore, a more realistic description of a real beach requires the incorporation of this wave/bottom interaction. The feedback mechanism was accounted for by specifying the wave characteristics on the 5 meter depth contour line. The wave height and angle at this depth were chosen to give the same breaking conditions as in type A and N1 solutions above, far away from any groin, fill, or

river where the shoreline is constantly straight and unaffected by the presence of the groin, fill, or river. Closer to these, the breaking wave height and angle will vary, according to the local bottom contour curvature.

Simulations

In order to demonstrate the effects of various assumptions on the three types of solutions, their limitations and possibilities, two idealized shoreline and structure configurations will be discussed. During all these simulations, the breaking wave height, H_b , is held constant = 0.7 m. As mentioned above, the expression "held constant" is only true far away from structures/activities in the type N2 solutions. The wave period is 5 seconds.

Groin Exposed to Waves with Varying Angle

In general, the analytic solution for the accumulation up-drift of a groin, as first formulated by PELNARD-CONSIDERE (1956) for waves with a constant angle, works very well whereas it gives a poor description immediately down-drift of the groin, since diffraction is not accounted for. In a model, the groin is represented by the boundary condition $Q = 0$. Mathematically, this boundary condition can be expressed as:

$$\frac{\partial y}{\partial x} = \tan \alpha_0 \quad \text{for } x = 0 \quad (24)$$

This equation states that the shoreline at the groin is at every instant parallel to the breaking wave crests. In the numerical solutions, the groin boundary condition was expressed as ($i = 1$ corresponds to the calculation element closest to the groin):

$$Q_i = 0 \quad \text{for } i = 1 \quad (25)$$

If the incident breaking wave angle is varying sinusoidally with time, some interesting features of shoreline evolution may be noted up-drift the groin. The breaking wave angle is assumed to vary around a mean value α_0 according to the following expression:

$$\alpha_b(t) = \alpha_0 (1 + \sin \omega t) \quad (26)$$

where ω is the angular frequency of the wave direction. The analytic solution may be derived with the help of Laplace transform technique (CARSLAW and JAEGER, 1959) to yield:

$$y(x,t) = \alpha_0 \left[2\sqrt{\epsilon t} \operatorname{ierfc} \left(\frac{x}{2\sqrt{\epsilon t}} \right) + \frac{e^{-\sqrt{\omega/2\epsilon} x}}{\sqrt{\omega/\epsilon}} \sin \left(\omega t - \sqrt{\frac{\omega}{2\epsilon}} x - \frac{\pi}{4} \right) + \frac{1}{\pi} \int_0^\infty \right. \quad (27)$$

$$\left. \frac{\omega \cos \left(\sqrt{\frac{\xi}{\epsilon}} x \right) e^{-\xi t}}{\sqrt{\frac{\xi}{\epsilon}} (\xi^2 + \omega^2)} d\xi \right]$$

for $t > 0$ and $x \geq 0$.

where ierfc is the integrated complementary error function. The integral part of the equation is a transient, which will disappear with time. Accordingly, the solution consists mainly of two parts, one identical to the shoreline evolution up-drift a groin exposed to waves with a constant breaking wave angle, α_0 , and one part expressing a damped sinusoidal variation, with the attenuation proportional to the distance from the groin by a factor $\sqrt{\omega/2\epsilon}$ (Figure 14). From Equation (27) the "crests" of the wave-shaped shoreline can be shown to travel with the speed $\sqrt{2\epsilon}$ up-drift from the groin, and the phase lag between the variation in shoreline position at the groin and at a specific location x is $\sqrt{\omega/2\epsilon} x + \pi/4$. In Figure 14, the shoreline evolution at two different locations alongshore are plotted as a function of time. The period of the variation in breaking wave angle is 14 days (336 hours). As indicated, the shoreline position varies rhythmically in time, with the fluctuations decreasing with the distance from the groin (lines 1 and 2). However, the long term trend is accretion on all locations along the beach, since the breaking wave angle always produces a longshore sand transport towards the groin. As expected, an angle too large will impair the analytic solution, overestimating the speed of shoreline response. In the calculated example, α_0 was set to 10 degrees.

A comparison between the type A solutions (lines 1 and 2) and the type N1 solutions (lines 3 and 4), shows that the linearization procedure

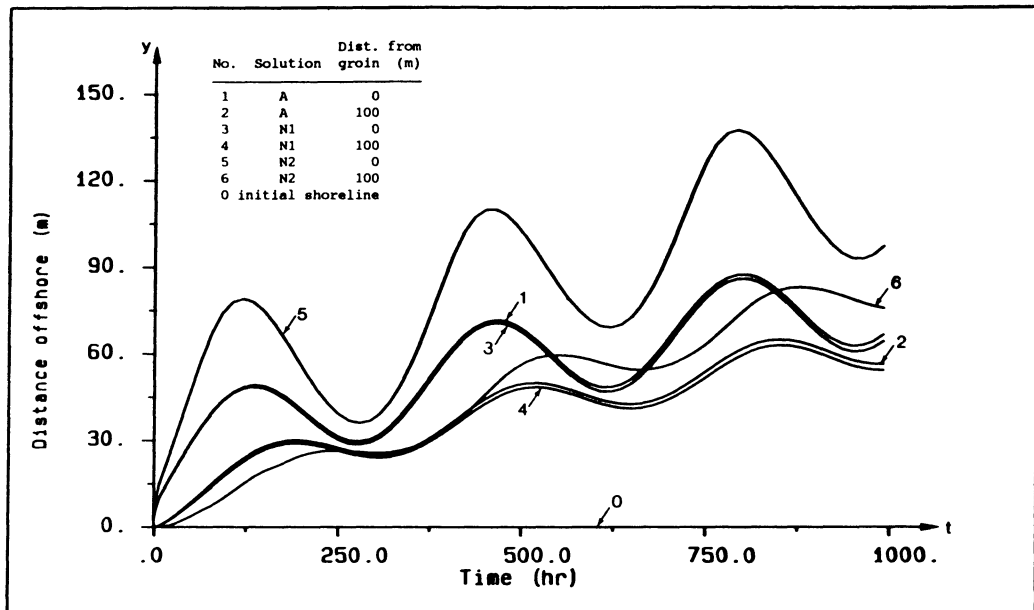


Figure 14. Accumulation up-drift of a groin exposed to waves with a sinusoidally varying angle. (From HANSON and LARSON, 1987)

in the analytic solutions causes only small errors on the calculated shoreline evolution. By including wave refraction, we see that close to the groin the wave refraction promotes a more rapid accumulation (line 5) while further away from the groin, it slows down the rate of shoreline response (line 6). As accumulation close to the groin continues, the area where refraction has a positive influence on sand accretion grows to include parts of the beach further up-drift. In the calculated example, the up-drift location is experiencing increased accretion after about 400 hours, when the type N2 solution (line 6) exceeds the other two solutions (lines 2 and 4). The phase lag between the former and the two latter solutions is explained by the initial setback of line 6.

Half Circle-Shaped Beach Fill

A beach fill (or a natural cape) shaped like a half circle exposed to incident breaking waves, parallel to the x-axis running alongshore, may be treated analytically by approximating its shape with a polygon having a large (in theory an infinite) number of corners. The half circle

fill has initially a radius a and the approximating polygon has N corners. The solution may be written (for details, see LARSON *et al.*, 1987):

$$y(x,t) = \frac{1}{2} \sum_{i=1}^{N-1} \left[(k_i x_i^L + y_i^L - k_i x) \left[\operatorname{erf} \left[\frac{x_i^R - x}{2\sqrt{\epsilon t}} \right] - \operatorname{erf} \left[\frac{x_i^L - x}{2\sqrt{\epsilon t}} \right] \right] + 2k_i \sqrt{\frac{\epsilon t}{\pi}} \exp \left[-\frac{(x_i^R - x)^2}{4\epsilon t} \right] - \exp \left[-\frac{(x_i^L - x)^2}{4\epsilon t} \right] \right] \quad (28)$$

for $t > 0$ and $-1 < x < 1$

using the quantities:

$$x_i^R = a \cos \left[\frac{(i-1)\pi}{N-1} \right] \quad (29a)$$

$$x_i^L = a \cos \left[\frac{i\pi}{N-1} \right] \quad (29b)$$

$$y_i^L = a \sin \left[\frac{i\pi}{N-1} \right] \quad (29c)$$

$$k_i = \frac{1}{\tan \left[i \left(\pi - \frac{1}{2} \right) \right]} \quad (29d)$$

where erf is the error function. As can be seen in Figure 15, displaying shoreline positions after 60 hours, the initial half circle (line 0, $N = 101$) is dispersed with time as material is transported away from the fill (line 1). As the problem is symmetrical around the center of the fill, only the right hand side is shown. Initially, in the analytic solution, the transport rate is infinitely large where the fill meets the straight shoreline ($x = a$) but decreases as the shoreline gradient decreases. The large transport rate is due to the linearization (c.f. Equation 21) which implies a transport rate proportional to the shoreline gradient assuming small angles. This assumption is strongly violated at the ends of the fill. In fact, the original relation for the longshore transport rate, Equation (19), gives a zero transport rate at these locations. Consequently, the linearization procedure artificially increases the erosion of the fill, thus making

any time estimates of loss percentages based on analytic solutions on the conservative side.

The numerical solutions use Equations (19) and (4) to calculate the sand transport rates. These equations give a maximum transport rate at a breaking wave angle of 45 degrees to the shoreline. A very interesting fill reformation occurs when applying the solution type N1, to a half circle beach fill as shown by line 2 in Figure 15. As the transport rate on the upper part of the fill ($\alpha_s < 45$ degrees) is increasing with the distance from the y-axis, this portion of the fill will consequently erode. The reversed condition prevail on the lower part of the fill ($\alpha_s > 45$ degrees). As α_s decreases, the transport rate will also decrease, causing this part of the fill to accrete. Thus, the equilibrium configuration for this case seems to resemble a rectangular-shaped fill, anticipated to be a poor description of the prototype case. Allowing the waves to refract towards the fill, the shoreline evolution (line 3) is believed to look more realistic.

General

The analysis concludes that the linearization of the sand transport equation, upon which the

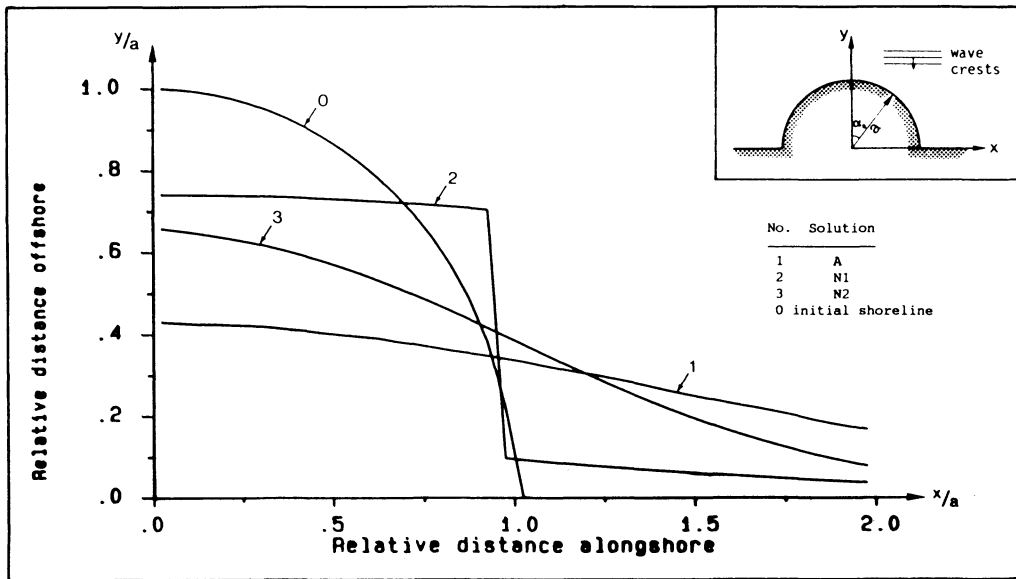


Figure 15. Spreading of an initially half circle beach fill. (From Hanson and Larson, 1987)

analytic solution is based, produces only small errors if the angle is kept within the order of 30 degrees. However, it is indicated that potentially much greater errors, both in the analytic and the simpler numerical solution, is caused by their inability to account for wave refraction. As the shoreline change is reflected in the bottom contours the breaking wave height and angle will vary continuously in space and time, even when offshore wave conditions are constant.

The calculated examples show that the exclusion of wave refraction tends to flatten out shoreline irregularities. The implications of this varies with the type of application. Starting the simulation with an irregularity (*e.g.*, a beach fill), the speed of erosion will be overestimated, whereas in simulations of beach change caused by disturbances (*e.g.*, groins, rivers) along the coast, the shoreline response rate close to the disturbance will be underestimated. The general error caused by linearizing the transport equation is an over-estimation of the speed of shoreline response. Again, this may or may not give solutions on the conservative side, depending on the application.

LAKEVIEW PARK SIMULATIONS

The model has mainly been tested on Oarai Beach, Japan and Sea Bright, New Jersey. However these studies are already extensively described in numerous publications (*e.g.*, KRAUS *et al.*, 1985; HANSON and KRAUS, 1986b; HANSON, 1987; KRAUS *et al.*, 1988). Instead, preliminary modeling results for Lorain, Ohio will be presented here.

Background

In 1977, three rubble-mound detached breakwaters were constructed at Lakeview Park, Lorain, Ohio. These were the first breakwaters in the United States, intended specifically to protect and stabilize a bathing beach (POPE, 1983), in this case an artificial beach fill (Figure 16). 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 through the use of one groin on each side.

The shoreline and bottom contours were carefully monitored by the Corps of Engineers, both

before and after the fill, providing excellent data for a numerical model simulation. Also, limited data are available on the statistical frequency of long term directional distribution of wave heights and periods (SAVILLE, 1953; RESIO and VINCENT, 1976). However, little information exists on the actual wave climate (height, period, and direction) between shoreline surveys. Thus, the wave series used in the model calibration/verification procedure had to be established for application of GENESIS.

As a test on the capability of GENESIS to reproduce prototype shoreline change, an attempt was made to simulate the shoreline change taking place during the first 24 days after the fill was completed. All necessary shoreline and structure configuration data were taken from survey charts (*c.f.* Figure 16). The wave data immediately available was limited, only giving representative wave heights and periods from five different directions and their percentage distribution in time. It is therefore likely, that for a short term simulation as made here, the actual mean wave climate could deviate significantly from the representative values.

Calibration

Starting with the initial fill shoreline of 1 October 1977, a series of simulations were carried out in order to reproduce the true shoreline of 24 October 1977 (see Figure 17). 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 deep water wave direction deviated 20 degrees to the east from the representative values given by the input wave data. This calibration procedure suggested values of the two calibration parameters to be $K_1 = 0.3$ and $K_2 = 0.3$.

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

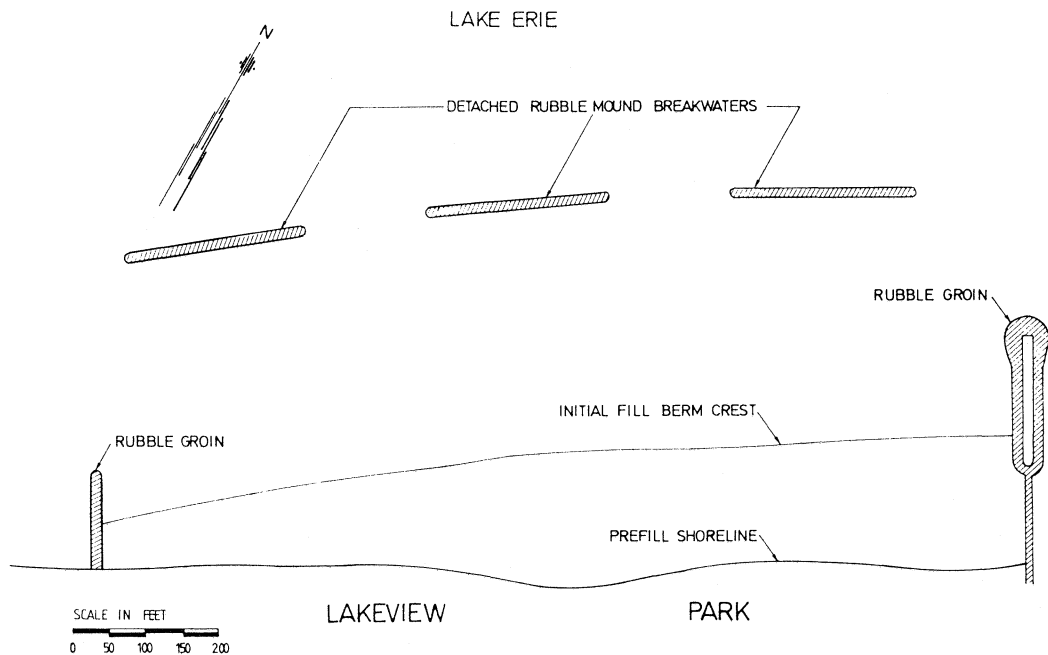


Figure 16. Shoreline and structure configuration at Lakeview Park, Lorain, Ohio.

Instead, all wave calculations were made within GENESIS, assuming bottom contours were parallel to the calculated representative offshore contour line.

In a beach fill project of this type, the volumetric changes can be as informative as the shape of the shoreline. In terms of this volumetric change, the computational results were very encouraging: the measured gain was 59,000 ft³ and the calculated gain was 53,000 ft³. Thus, the model accounted for 90 percent of the volumetric change.

Evaluation

Being a small and well documented area, Lakeview Park, serves well as a test case for a simulation model. Unfortunately, the extensive monitoring of the bathymetry was not balanced with a similar wave documentation. Therefore, 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 to most beaches. It was therefore considered as an interesting

test on GENESIS. Under the circumstances, and the limited effort spent on calibrating the model, the results were very encouraging.

In order to make more accurate predictions of the shoreline evolution, 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). Wave transmission through the detached breakwaters is believed to have a significant influence on shoreline change and should therefore have been accounted for. Also, a variation of the sand transport over the model boundaries is believed to have improved the simulation results.

SUMMARY AND CONCLUSIONS

This paper describes a numerical model called GENESIS, developed to simulate the interaction between waves, longshore sediment transport, coastal structures, and other human activities in the nearshore area. The purpose of the model is to simulate shoreline change in a long term perspective, ranging from a few months to several years.

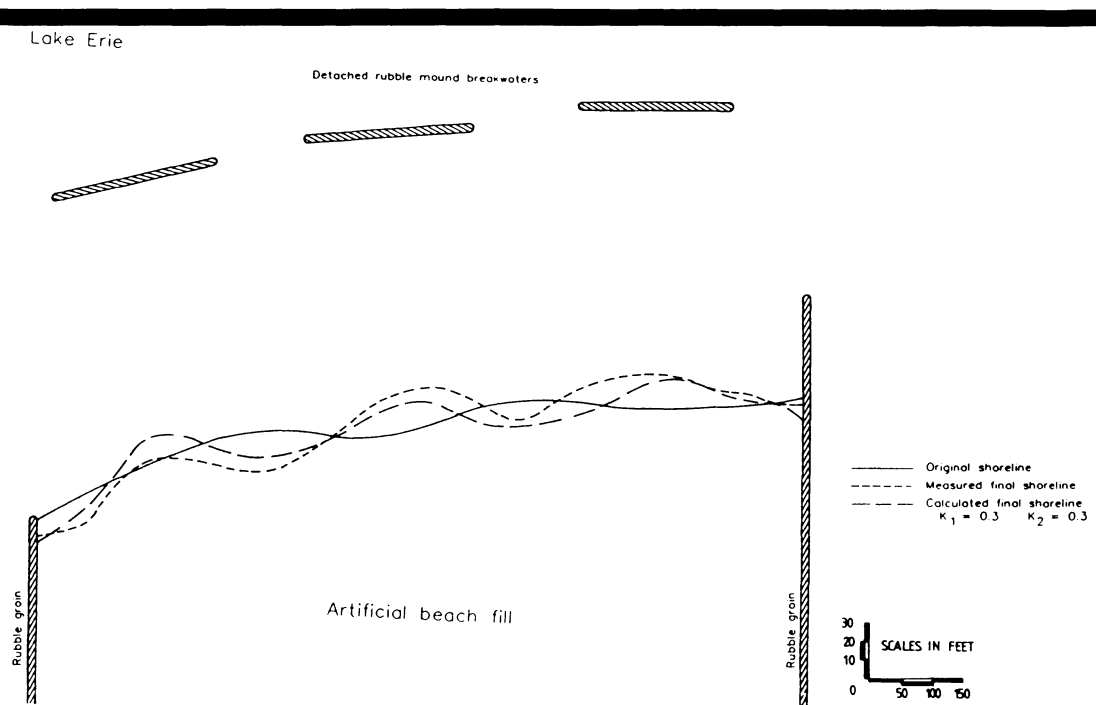


Figure 17. Measured and calculated shorelines at Lakeview Park.

As opposed to previous shoreline models GENESIS is generalized in the sense that it can be easily applied to almost any open sandy coast. Among other novelties in GENESIS, its capability to simulate wave diffraction from multiple coastal structures and the representation of seawalls, is of major importance.

The easy modification of the model to represent real or hypothetical applications have many implications. GENESIS can be used as an educational tool, illustrating the effects of various physical properties and human activities. It also makes GENESIS economical to run and, therefore, simulations can be made for small low-budget projects. In larger projects, simulations can be performed for wide spatial extents, long time intervals, and a large number of design alternative.

Calculated examples show that GENESIS reasonably describes a variety of hypothetical situations, and that it is possible to use the modeling system for preliminary prototype design. Still, much model development remains to be done, of which main improvements, already in progress, are representation of wave transmission through detached breakwaters

and capability to more realistically describe sources and sinks of sand along the shore.

GENESIS was developed to serve as an engineering tool, but also to provide insights in the dynamics controlling shoreline change, with emphasis on the interaction with structures. In this report, much effort was spent on sensitivity analysis and the description of prototype applications. This was done with the ambition to provide guidance for engineers on how to apply this or their own model to their particular beach problem.

In order to run GENESIS for a real beach much information must be collected. The most fundamental requirement for running the model is to have at least two measured shorelines covering the area in question for the model calibration. In order to verify the model, one or two more shorelines are needed. The model cannot be run without calibration. In addition, positions and characteristics of structures and other human activities must be known for the simulation time interval. Moreover, relevant boundaries and associated boundary conditions must be identified.

The quality of the predictions are to a large

degree dependent on the wave input. However, in many cases relevant wave data is lacking. In such cases, information about the wind climate can be used to hindcast a wave climate, keeping the uncertainties in mind. As mentioned above, some of the input parameters are hard to quantify. Effort should be spent on determining an interval within which the true value of the respective parameter is expected to lie, rather than specifying a single value. By running the model for the mean and extreme values on the parameters in question, the uncertainties are translated into possible shoreline positions. As shoreline change normally is a slow process, it is recommended to start data collection on shorelines and waves several years ahead. With appropriate data, GENESIS has the potential of serving as an aid in the planning and engineering design of the coastal environment.

ACKNOWLEDGEMENT

The author wishes to thank Dr. Nicholas C. Kraus, US Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, USA, (CERC) for major contributions in the development of GENESIS. The study was made possible through financial support from CERC and the Swedish Council for Building Research.

LITERATURE CITED

- BAKKER, W.T., 1969. The dynamics of a coast with a groyne system. *Proceedings of 11th Coastal Engineering Conference*, American Society of Civil Engineers, pp. 492-517.
- BAKKER, W.T. and EDELMAN, T. 1965. The Coastline of river deltas. *Proceedings of 9th Coastal Engineering Conference*, American Society of Civil Engineers, pp. 199-218.
- BAKKER, W.T.; KLEIN-BRETELER, E.H.J., and ROOS, A., 1971. The Dynamics of a coast with a groyne system. *Proceedings of 12th Coastal Engineering Conference*, American Society of Civil Engineers, pp. 1001-1020.
- BASCOM, W.N., 1951. The relationship between sand size and beach-face slope. *Transactions of the American Geophysical Union*, 35(6), pp. 866-874.
- BRUUN, P., 1954. Coast erosion and the development of beach profiles. *TM-44*, US Army Corps of Engineers, Beach Erosion Board.
- CARSLAW, H. and JAEGER, J., 1959. *Conduction of Heat in Solids*. Oxford: Clarendon Press, 496p.
- DEAN, R.G., 1977. Equilibrium beach profiles: US Atlantic and Gulf coasts. *Ocean Engineering Report No. 12*, Department of Civil Engineering, University of Delaware, Newark, Del., 45p.
- EBERSOLE, B., 1985. Refraction-diffraction model for linear water waves. *Journal of Waterway, Port, Coastal and Ocean Engineering*, Vol. 111, No. WW6 pp. 939-953.
- EBERSOLE, B.; CIALONE, M.A., and PRATER, M.D., 1985. RCPWAVE a linear wave propagation model for engineering use. *Technical Report CERC-86-4*, US Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, 160p.
- GODA, Y., 1984. *Random Seas and Design of Maritime Structures*. Tokyo: University of Tokyo Press, 232p.
- GODA, Y.; TAKAYAMA, T., and SUZUKI, Y., 1979. Diffraction diagrams for directional random waves. *Proceedings of 16th Coastal Engineering Conference*, American Society of Civil Engineers, pp. 628-650.
- GRIJM, W., 1961. Theoretical forms of shorelines. *Proceedings of 7th Coastal Engineering Conference*, American Society of Civil Engineers, pp. 197-202.
- GRIJM, W., 1965. Theoretical forms of shorelines. *Proceedings of 9th Coastal Engineering Conference*, American Society of Civil Engineers, pp. 219-235.
- HALLERMEIER, R.J., 1983. Sand transport limits in coastal structure design. *Proceedings of Coastal Structures '83*, American Society of Civil Engineers, pp. 703-716.
- HANDS, E.B., 1983. Erosion of the Great Lakes due to changes in the water level. In: Komar, P.D., (ed.), *CRC Handbook of Coastal Processes and Erosion*, pp. 167-190.
- HANSON, H., 1987. GENESIS—A Generalized Shoreline Change Numerical Model for Engineering Use. PhD Thesis, Dept. of Water Resources Engineering, University of Lund, Sweden.
- HANSON, H. and KRAUS, N.C., 1985. Seawall constraint in shoreline numerical model. *Journal of Waterway, Port, Coastal and Ocean Engineering*, 111 (WW6), pp. 1079-1083.
- HANSON, H. and KRAUS, N.C., 1986a. Seawall boundary condition in numerical models of shoreline change. *Technical Report CERC-86-3*, US Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, 59p.
- HANSON, H. and KRAUS, N.C., 1986b. Forecast of shoreline change behind multiple coastal structures. *Coastal Engineering in Japan*, Vol. 29, pp. 195-213.
- HANSON, H. and LARSON, M., 1987. Comparison of analytic and numerical solutions of the one-line model of shoreline change. *Proceedings of Coastal Sediments '87*, American Society of Civil Engineers, pp. 500-514.
- HORIKAWA, K.; HARIKAI, S., and KRAUS, N.C., 1979. A physical and numerical modeling of waves, current and sediment transport near a breakwater. *Annual Report of the Engineering Research Institute*, University of Tokyo, No. 38, pp. 41-48.
- HORIKAWA, K.; SASAKI, T., and SAKURAMOTO, H., 1977. Mathematical and laboratory models of shoreline change due to dredged holes. *Journal of the Faculty of Engineering*, University of Tokyo, XXXIV(1), pp. 49-57.

- KRAUS, N.C., 1981. One-line development and simulation for Oarai beach. *NERC Report No. 13*, Cooperative Research of Surf Zone Dynamics, Part 3, Beaches Near Breakwaters and Rocky Coasts, Near-shore Environment Research Center, pp. 155-192.
- KRAUS, N.C. and DEAN, J.L., 1987. Longshore sediment transport rate distribution measured by trap. *Proceedings of Coastal Sediments '87*, American Society of Civil Engineers, pp. 881-896.
- KRAUS, N.C.; HANSON, H., and HARIKAI, S., 1985. Shoreline change at Oarai beach past, present and future. *Proceedings of 19th Coastal Engineering Conference*, American Society of Civil Engineers, pp. 2107-2123.
- KRAUS, N.C.; HARIKAI, S., and KUBOTA, S., 1981. A numerical simulation of shoreline evolution at Oarai beach. *Proceedings of 28th Japanese Conference on Coastal Engineering*, Japanese Society of Civil Engineers, pp. 632-645.
- KRAUS, N.C. and HARIKAI, S., 1983. Numerical model of the shoreline change at Oarai beach. *Coastal Engineering*, 7(1), pp. 1-28.
- KRAUS, N.C.; HANSON, H.; SCHEFFNER, N.W.; CHOU, L.W.; CIALONE, M.A.; SMITH, J.M., and HARDY, T.A., 1988. Coastal processes at Sea Bright to Ocean Township, New Jersey. *Miscel. Paper CERC-00*, US Army Engineer, Waterways Experiment Station, Coastal Engineering Research Center.
- LARSON, M.; HANSON, H.; and KRAUS, N.C., 1987. Analytic solutions of the one-line model of shoreline change. *Technical Report CERC-87-15*, US Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, 100p.
- LE MEHAUTE, B. and SOLDATE, M., 1977. Mathematical modeling of shoreline evolution. *Miscellaneous Report 77-10*, US Army Corps of Engineers, Fort Belvoir, Coastal Engineering Research Center, 56p.
- LE MEHAUTE, B., and SOLDATE, M., 1980. A numerical model for predicting shoreline changes. *Miscellaneous Report 80-6*, US Army Corps of Engineers, Fort Belvoir, Coastal Engineering Research Center, 72p.
- LE MEHAUTE, B.; WANG, J.D., and LU, C.C., 1983. Wave discretization for shore line processes. *Journal of Waterway, Port, Coastal and Ocean Engineering*, 109 (WW1), pp. 63-78.
- MIMURA, N.; SHIMIZU, T., and HORIKAWA, K., 1983. Laboratory study on the influence of detached breakwater on coastal change. *Proceedings of Coastal Structures '83*, American Society of Civil Engineers, pp. 740-752.
- MOORE, B., 1982. Beach Profile Evolution in Response to Changes in Water Level and Wave Height, Unpublished M.S. Thesis, Department of Civil Engineering, University of Delaware, Newark, DL.
- OZASA, H. and BRAMPTON, A.H., 1980. Mathematical modeling of beaches backed by seawalls. *Coastal Engineering*, 4(1), pp. 47-64.
- PELNARD-CONSIDERE, R., 1956. Essai de Theorie de l'Evolution des Forms de Rivage en Plage de Sable et de Galets. *4th Journees de l'Hydraulique, Les Energies de la mer*, Question III, Rapport No. 1, pp. 289-298.
- PERLIN, M., 1978. A Numerical Model to Predict Beach Planforms in the Vicinity of Structures, Master's thesis, University of Delaware, Newark, DL., 172p.
- PERLIN, M., 1979. Prediction beach planforms in the lee of a breakwater. *Proceedings of Coastal Structures '79*, American society of Civil Engineers, pp. 792-808.
- PERLIN, M. and DEAN, R.G., 1979. Prediction of beach planforms with littoral controls. *Proceedings of 16th Coastal Engineering Conference*, American Society of Civil Engineers, pp. 1818-1838.
- PERLIN, M. and DEAN, R.G., 1983. A numerical model to simulate sediment transport in the vicinity of coastal structures. *Miscellaneous Report No. 83-10*, US Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, 119p.
- POPE, J., 1983. Breakwaters for beach protection at Lorain, OH. *Proceedings of Coastal Structures '83*, American Society of Civil Engineers, pp. 753-768.
- PRICE, W.A.; TOMLINSON, D.W., and WILLIS, D.H., 1973. Prediction changes in the plan shape of beaches. *Proceedings of 13th Coastal Engineering Conference*, American Society of Civil Engineers, pp. 1321-1329.
- REA, C.C. and KOMAR, P.D., 1975. Computer simulation models of a hooked beach shoreline configuration. *Journal of Sedimentary Petrology*, 45(4), pp. 866-872.
- RESIO, D.T. and VINCENT, C.L., 1976. Design wave information for the Great Lakes. *Technical Report H-76-1*, US Army Engineer Waterways Experiment Station, Hydraulics Laboratory.
- SASAKI, T.O. and SAKURAMOTO, H., 1978. Field verification of a shoreline simulation model. *International conference on Water Resources Engineering*, International Association for Hydraulic Research, pp. 501-518.
- SAVILLE, T., Jr., 1953. Wave and lake level statistics for Lake Erie. *TM-37*, US Army Corps of Engineers, Beach Erosion Board.
- SHORE PROTECTION MANUAL, 1984. 4th ed., 2 vols. US Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, US Government Printing Office, Washington, DC.
- SUNAMURA, T. and HORIKAWA, K., 1977. Sediment budget in Kujukuri coastal area. *Japan, Proceedings of Coastal Sediments '77*, American Society of Civil Engineers, pp. 475-487.
- WALTON, T.L. and CHIU, T.Y., 1979. A Review of analytical techniques to solve the sand transport equation and some simplified solutions. *Proceedings of Coastal Structures '79*, American Society of Civil Engineers, pp. 809-837.
- WIEGEL, R.L., 1964. *Oceanographical Engineering*. Englewood Cliffs: Prentice Hall, 532p.
- WILLIS, D.H., 1977. Evaluation of alongshore

transport models. *Proceedings of Coastal Sediments '77*, American Society of Civil Engineers, pp. 350-365.

WILLIS, D.H. and PRICE, W.A., 1975. Trends in

the application of research to solve coastal engineering problems. In: Hails, J. and Carr, A. (Eds.), *Nearshore Sediment Dynamics and Sedimentation*, New York: Wiley, pp. 111-121.

□ RESUMEN □

Este artículo describe un modelo numérico, llamado GENESIS para el cálculo de la variación de la línea de costa bajo la acción del oleaje. El modelo es "unilinear" en el sentido que asume que el perfil permanece constante, es decir, no se modifica. A diferencia de otros modelos, el modelo GENESIS permite con el simple uso de una interfase estudiar condiciones diversas, tales como número arbitrario, localización y combinación de espigones, diques de encausamiento, muralla de mar y rellenos de playas. El modelo tiene en cuenta: la aproximación, refracción y difracción del oleaje; el rebase de arena a través y alrededor de espigones, y fuentes y sumideros de material. La eficacia del modelo se comprueba comparando sus resultados tanto con soluciones analíticas como con prototipos.—*Department of Water Sciences, University of Cantabria, Santander, Spain.*

□ RÉSUMÉ □

Décrit le modèle numérique GENESIS qui permet de calculer les changements du littoral ayant pour origine l'action des vagues. Le modèle repose sur la théorie "unilinéaire" qui suppose que les profils de plage demeurent inchangés, ce qui permet de décrire les variations de la plage en seuls termes de position du rivage. A l'inverse des modèles antérieurs du même type, GENESIS est généralisé: un simple interface d'utilisation permet d'appliquer le système à diverses situations, y compris la localisation, les ouvrages de défense et le remplissage de plage. D'autres éléments inclus sont l'approximation, la réfraction et la diffraction des vagues, le transit sédimentaire au travers des épis, ou les sources d'approvisionnement. Une revue des modèles est présentée. Les modèles sont comparés aussi bien à des solutions analytiques, que à des situations prototype, en vue d'en montrer les possibilités.—*Catherine Bressolier, U.A. 910 du CNRS, EPHE, Montrouge, France.*

□ ZUSAMMENFASSUNG □

Dieser Bericht beschreibt ein numerisches Modell, das sich GENESIS nennt (*Generelles Modell zur Simulation von Strandlinienveränderungen*). Es wurde entwickelt, um in erster Linie durch Wellenenergie hervorgerufene Strandveränderungen berechnen zu können. Bei diesem Modell wird davon ausgegangen, daß das Küstenprofil unverändert bleibt. Dadurch ist es möglich, die Strandveränderung allein in Abhängigkeit von der Lage der Küstenlinie zu beschreiben. Im Gegensatz zu früheren Modellen, die auf dem gleichen Konzept basieren, ist GENESIS in der Weise allgemeiner, daß es für den Benutzer möglich ist, das System auf eine Vielfalt von Situationen zu beziehen. Dies schließt fast zufällige Zahlen, Standorte und ein Kombination von Buhnen, Molen, freistehenden Wellenbrechern, Kaimauern und Strandbefestigungen ein. Andere Parameter, die das System einbezieht, sind Wellenscharung, -brechung und -diffraktion, Sandtransport durch und um Buhnen, sowie Liefer- und Sedimentationsgebiete von Sand. Schließlich wird ein Überblick über das Modell vorgesteallt, und Vergleiche zu analytischen Lösungen sowie prototyp Situationen werden dargestellt, um die Leistungsstärke des Systems zu demonstrieren.—*Helmut Brückner, Geographisches Institut, Universität Düsseldorf, F.R.G.*