CHAPTER 110

PREDICTION OF BEACH PLANFORMS WITH LITTORAL CONTROLS

by

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ABSTRACT

Three numerical models representing shoreline response are described and applied to a number of problems of coastal engineering interest. Included are: (1) a one-line explict model, (2) a one-line implicit model, and (3) a two-line explicit model. Simplified refraction and diffraction schemes are incorporated into the models. The models allow grid elements to be activated and deactivated as sand is initially deposited in or the last sand removed from a grid at a jetty inclined to the shoreline. The one-line explicit model and two-line explicit model are similar to those described by Bakker, Breteler and Roos[(3)]. The one-line implicit model offers the advantage of stable computations for much longer time steps.

Example applications of shoreline response are presented to illustrate the utility of the model including: permeable versus impermeable jetties, shoreline perturbations caused by jetties which are aligned with the incoming waves versus jetties perpendicular to shore, shoreline response inside a groin compartment and the effect of a littoral barrier. In addition, the one-line implicit model is applied to predict the shoreline response in the vicinity of an offshore breakwater at Channel Islands Harbor, California where sediment is accumulated and dredged periodically. In this case, excellent shoreline response data are available from the prototype; however, only visual wave observations were available for this comparison and it was found necessary to increase the sediment transport relationship fourfold to achieve even approximate correspondence.

Numerical models offer a powerful means which, when combined with good judgement, should strengthen the coastal engineer's ability to predict the effects of a coastal engineering design. The research needs to improve numerical modeling capabilities are presented.

INTRODUCTION

In considering the installation of a coastal structure it is important to know the impact of the structure on shoreline fluctuations. Theoretical solutions are available for idealized cases of simple littoral barriers, unidirectional waves and linearized transport. Most actual conditions are considerably more complex due to reversals

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of longshore transport, a time-varying wave height and possibly more than one structure present. Numerical modeling has been developed to a fairly high degree of reliability for some applications such as tidal wave propagation in estuaries. Conversely, the numerical modeling of shoreline evolution must be considered in a relative state of infancy, however, it does appear to offer the potential of providing a substantial improvement to existing design and assessment capabilities. Moreover, it is anticipated that the use of numerical models will identify those critical sediment transport mechanisms requiring improved understanding and will provide the framework which, when applied in conjunction with well-designed and instrumented laboratory and/or field experiments, will allow these mechanisms to be better defined.

In this paper, three numerical models to represent shoreline evolution will be described and examples illustrating their application presented. Two of these models are quite similar to those developed and described by Bakker, Breteler and Roos(3) and by Hulsbergen, Bakker and van Bochove(9). The third utilizes an implicit approach and allows much longer time steps to be utilized. Additionally, the latter model is applied to several interesting coastal engineering problems not previously treated by this approach.

REVIEW OF PREVIOUS WORK

Analytical Approaches

Previous efforts to predict shoreline evolution include both analytical and numerical approaches. The earliest approach was that of Pelnard Considére(13) in which the linearized longshore sediment transport equation and conservation of mass equation were combined to yield the diffusion equation in terms of a shoreline coordinate, y

$$\frac{\partial y}{\partial t} = A \frac{\partial^2 y}{\partial x^2} \tag{1}$$

and where A incorporates the wave and beach characteristics. Eq. (1) has been solved for a number of conditions including that of a long littoral barrier, the spreading out of a deposit of sand on an otherwise straight shoreline, etc.

Bakker, Breteler and Roos(3) have extended the theory of Pelnard Considére by representing the beach profile by two contours, say y_1 and y_2. In this case, there are two governing equations which incorporate the effect of the onshore or offshore motion between the two contours due to a non-equilibrium beach slope. Bakker has solved these equations for a number of interesting cases including those of single and multiple groins along a shoreline. Accumulation occurs at a groin until the accretion of the shoreline results in a profile that is so steep that the offshore gradient causes the sand to be diverted to the offshore contour where it is transported around the tip of the groin.

There have been a number of laboratory experiments(3)(4)(9)(16) of shoreline evolution; the results of these experiments are generally in good agreement with the analytical solutions.

Numerical Approaches

In order to avoid the assumptions and limitations associated with the analytical approaches, a number of studies(2) (3) (9) (15) (16) (18) have been carried out to investigate the application of numerical models to shoreline evolution problems. The advantages of numerical modeling include the capability to readily incorporate many features, including: changes in wave conditions, the full (nonlinear) equations, complicated structure geometry, wave diffraction, structure permeability, crest elevation, etc. Almost any feature or mechanism for which a relationship is known (or is suggested) can be incorporated into the numerical model.

METHODOLOGY

Three numerical models will be described and illustrated by examples. These include: (1) a one-line explicit model, (2) a one-line implicit model, and (3) a two-line explicit model.

One-Line Explicit Model

This type of numerical model has been applied by a number of investigators to the problem of shoreline response. The governing equations include the sand transport equation and the conservation of mass equation.

Sand Transport Equation. The relationship governing the transport of sand along a straight shoreline as expressed by Inman and Bagnold[10] is

$$I = K\ P_{\ell s} \tag{2}$$

in which I is the immersed weight transport rate, K is a dimensionless constant (≈ 0.8) and $P_{\ell s}$ is the longshore component of wave energy flux at the breaker line, given by

$$P_{\ell s} = \rho g \frac{H_b^2}{8} C_{G_b} \sin \delta_b \cos \delta_b \tag{3}$$

in which ρ is the mass density of water, g is the gravitational constant, H_b is the breaking wave height, C_{G_b} is the group velocity at the breaker line and δ_b is the breaking wave crest angle relative to the shoreline,

$$\delta_b = \beta_B - \beta' - \alpha_b$$

and α_b is the azimuth from which the waves propagate, β_B is the azimuth of the outward normal of the baseline and β' is the inclination of the shoreline relative to the baseline. One advantage of Eq. (2) over other forms is that I and $P_{\ell s}$ have the same dimensions. The immersed weight sand transport rate, I, and the volumetric sand transport rate, Q, are related by

$$Q = \frac{I}{\rho g (S_s - 1) a} \tag{4}$$

in which S_s is the specific gravity of the sediment relative to the fluid and a is the complement of the porosity (a ≈ 0.6 to 0.7).

Conservation of Mass Equation. The conservation of mass equation, considering only longshore transport is

$$\frac{\partial \forall}{\partial t} + \frac{\partial Q}{\partial x} = 0 \tag{5}$$

in which x is the longshore coordinate and ∀ represents the total volume of the beach profile per unit length.

Solution. The one-line explicit model utilizes a time-marching space and time-staggered procedure in which the shoreline orientation is held fixed for one time step (from $n\Delta t$ to $(n+1)\Delta t$) and the sand transport calculated, and the transport held fixed over a time step (from $(n+\frac{1}{2})\Delta t$ to $(n+\frac{3}{2})\Delta t$) and the changes in shoreline position are determined. The finite difference equations are

$$Q_i^{n+1} = \frac{K\left(H_b^2\right)^n}{8} \frac{C_{G_b}^n}{(S_s-1)a} \sin \delta_b^{n+\frac{1}{2}} \cos \delta_b^{n+\frac{1}{2}} \tag{6}$$

$$y_i^{n+\frac{3}{2}} = y_i^{n+\frac{1}{2}} - \frac{\Delta t}{D\Delta x}\left[Q_{i+1}^{n+1} - Q_i^{n+1}\right] \tag{7}$$

where the superscripts denote the time level at which the variable is evaluated and D represents the total depth of the "active" beach profile which is assumed to be displaced landward or seaward without change of form. The shoreline orientation β' relative to the baseline is determined by

$$\left(\beta_i'\right)^{n+\frac{1}{2}} = \tan^{-1}\left(\frac{y_{i+1}^{n+\frac{1}{2}} - y_i^{n+\frac{1}{2}}}{\Delta x}\right) \tag{8}$$

Figure 1 presents the grid system for the one-line model.

Wave Refraction. The orientation of the deep water waves is considered known. The waves are then refracted to the breaking depth in accordance with Snell's Law, where the orientation of the contour at breaking is assumed to be the same as that of the shoreline for the same grid.

Wave Diffraction. An approximate diffraction procedure is incorporated in the model, based on the results of Penny and Price(14). The reader is referred to Perlin(15) for greater detail on the manner of representing diffraction in the model.

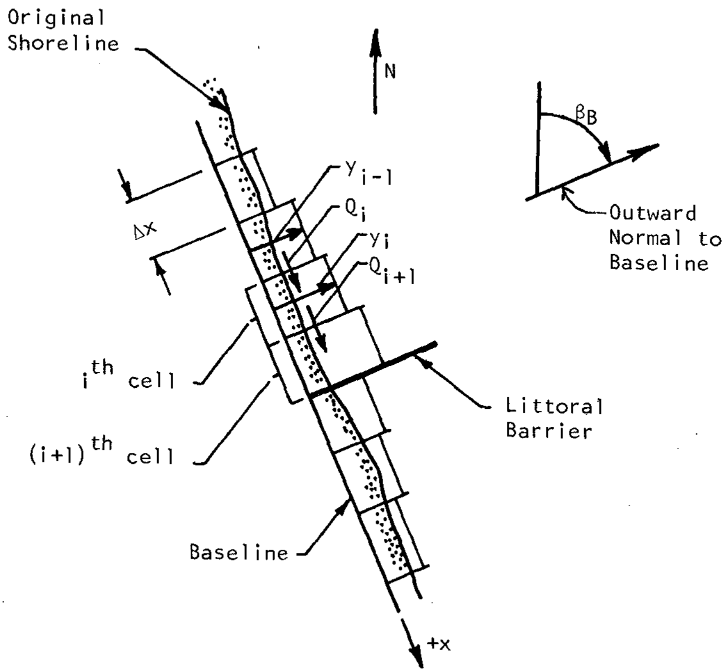

Figure 1. Shoreline Representation for One-Line Numerical Model.

One-Line Implicit Model

The one-line implicit model is based on the same equations (Eqs. (5) and (6)) as the one-line explicit model; however, for the implicit model they are solved simultaneously and thus greater numerical stability results. The main features of the implicit model will be described in detail, since this type of model does not appear to have been applied before for shoreline evolution.

Sand Transport Equation. The sand transport equation is the same as introduced earlier and is written below in abbreviated form as

$$Q = \Gamma H_b^{5/2} \sin 2\delta_b \tag{9}$$

where

$$\Gamma = \frac{K\sqrt{g}}{16(S_s-1)\sqrt{\kappa}\, a}$$

in which the shallow water approximation $C_{G_b} = \sqrt{gh_b}$ and the usual spilling breaking assumption $H_b = \kappa h_b$ have been introduced. The quantity h_b

represents the breaking depth. Considering that the appropriate sand transport value for establishing changes between the n^{th} and $(n+1)^{th}$ time step as that at $(n+\frac{1}{2})\Delta t$, expanding Eq. (9) and accounting for first order effects of changes in y

$$Q_i^{n+\frac{1}{2}} = \Gamma(H^{5/2})^n \left[\sin 2(\beta_B - \alpha_{b_i})^n \cos(2\beta_i')^n - \cos 2(\beta_B - \alpha_{b_i})^n \frac{y_i^{n+1} + y_i^n - y_{i-1}^{n+1} - y_{i-1}^n}{\sqrt{(\Delta x)^2 + (y_i^n - y_{i-1}^n)^2}} \right] \tag{10}$$

Other forms for $Q_i^{n+\frac{1}{2}}$ could be equally valid or more valid.

Conservation of Sand Equation. The equation for the conservation of sand is

$$y_i^{n+1} = y_i^n - \frac{\Delta t}{D\Delta x}\left[Q_{i+1}^{n+\frac{1}{2}} - Q_i^{n+\frac{1}{2}}\right] \tag{11}$$

Solution to Equations. Prior to describing the equations to be solved, they will be rewritten in the following forms

$$A_i y_i^{n+1} + Q_i^{n+\frac{1}{2}} - A_i y_{i-1}^{n+1} = D_i \tag{12}$$

$$B_i Q_{i+1}^{n+\frac{1}{2}} + y_i^{n+1} - B_i Q_i^{n+\frac{1}{2}} = E_i \tag{13}$$

where

$$A_i = \Gamma(H^{5/2})^n \frac{\cos 2(\beta_B - \alpha_{b_i})^n}{\sqrt{\Delta x^2 + (y_i^n - y_{i-1}^n)^2}} \tag{14}$$

$$D_i = \Gamma(H^{5/2})^n \left[\sin 2(\beta_B - \alpha_{b_i})^n \cos 2(\beta_i')^n - \cos 2(\beta_B - \alpha_{b_i})^n \frac{y_i^n - y_{i-1}^n}{\sqrt{\Delta x^2 + (y_i^n - y_{i-1}^n)^2}} \right] \tag{15}$$

$$B_i = \frac{\Delta t}{D\Delta x} \tag{16}$$

$$E_i = y_i^n \tag{17}$$

and Eqs. (12) and (13) represent the sand transport and conservation of sand equations respectively.

Eqs. (12) and (13) are solved by the so-called double-sweep method[1][16] in which it is assumed that the adjacent Q and y values are related linearly, i.e.

$$Q_{i+1}^{n+\frac{1}{2}} = G_i y_i^{n+1} + H_i \tag{18}$$

$$y_i^{n+1} = G_i^* Q_i^{n+\frac{1}{2}} + H_i^* \tag{19}$$

It is clear that if the G, G*, H and H* values were known for all i and if either y or Q were also known at one boundary, it would be possible to calculate all Q and y values from Eqs. (18) and (19). The double-sweep method proceeds by substituting Eq. (18) into Eq. (13) and solving for y_i^{n+1}, which results in

$$y_i^{n+1} = \frac{B_i}{B_i G_i + 1} Q_i^{n+\frac{1}{2}} + \frac{E_i - B_i H_i}{B_i G_i + 1} \tag{20}$$

and substituting Eq. (19) into Eq. (12) yields

$$Q_i^{n+\frac{1}{2}} = \frac{A_i}{A_i G_i^* + 1} y_{i-1}^{n+1} + \frac{D_i - A_i H_i^*}{A_i G_i^* + 1} \tag{21}$$

Comparison of Eqs. (20) and (21) with (18) and (19) establish a relationship between the unknowns, G_i, H_i, G_i^* and H_i^* and the knowns A_i, B_i, D_i and E_i as

$$G_i = \frac{A_{i+1}}{A_{i+1}\, G_{i+1}^* + 1}, \quad H_i = \frac{D_{i+1} - A_{i+1}\, H_i^*}{A_{i+1}\, G_{i+1}^* + 1} \qquad (22),\ (23)$$

$$G_i^* = \frac{B_i}{B_i G_i + 1}, \quad H_i^* = \frac{E_i - B_i H_i}{B_i G_i + 1} \qquad (24),\ (25)$$

As an example, the solution proceeds from the right-hand boundary (i = I) where either Q_I or y_I is specified. If Q_I is known, then $H_{I-1} = Q_I$ and $G_{I-1} = 0$. If y_I is specified, then $H_I^* = y_I$ and $G_I^* = 0$. To illustrate the process further, suppose Q_I is specified, then H_{I-1} and G_{I-1} can be determined, and from Eqs. (24) and (25) it is possible to establish G_{I-1}^* and H_{I-1}^*. Continuing in a stepwise fashion, from Eqs. (22) and (23), G_{I-2} and H_{I-2} are calculated, etc. Considering that Q_1 is the value stated at the left-hand end of the grid system the determination of G_1^* and H_1^* complete the first "sweep" from right to left. The second "sweep" is from left to right and comprises the establishment of the sand transport values $Q_i^{n+\frac{1}{2}}$ and y_i^{n+1} for all i using Eqs. (18) and (19). This completes the procedure for one time step and the process is repeated for subsequent time steps.

Unless otherwise stated all examples presented subsequently will be based on the implicit model.

A M-Line Model

A M-line model is an extension of the similar two-line explicit model presented by Bakker, Breteler and Roos[3]. The displacements of M contours are calculated at each time step including onshore-offshore transport due to the beach slope being milder or steeper than the equilibrium, respectively, see Figure 2. A multi-line model provides a much better representation of shoreline response in the vicinity of structures where, for example, the steepening of the profile on the updrift side causes offshore transport and bypassing of the structure before the mean water level contour has advanced to the tip of the structure.

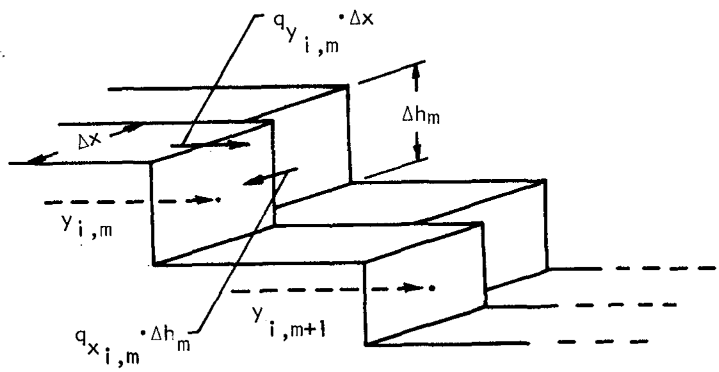

Figure 2. Beach Profile and Transport Representation For Multiple-Line Numerical Model.

Sand Transport Equations. The longshore sand transport equation for the m^{th} line is

$$Q_{x_{i,m}} = \mu_m Q$$

where Q is the value given by Eqs. (2) and (4) and μ_m is the proportion of the transport associated with the m^{th} line and it follows that

$$\Sigma \mu_m = 1 \tag{26}$$

The equation for offshore transport per unit beach length, q_{y_m} from the $(m-1)^{th}$ to the m^{th} contour is

$$q_{y_m} = K_{y_m}(y_{m-1} - y_m + W_m) \tag{27}$$

in which K_{y_m} is a transport coefficient for the m^{th} contour line and W_m is the equilibrium separation distance for the (m-1) and m contours. It follows that if the profile is steeper or milder than the equilibrium the transport will be offshore or onshore respectively. In the two-line model utilized here, the equilibrium distance, W, was based on an analysis of equilibrium profiles by Dean[8] and the sand transport coefficient, K_y, was based on wave tank tests by Saville[17].

Conservation of Sand Equation. The sand conservation equation, for the m^{th} contour, including the effects of onshore-offshore transport, is

$$\Delta h_m \frac{\partial y_m}{\partial t} + \frac{\partial Q_{x_m}}{\partial x} - q_{y_m} + q_{y_{m+1}} = 0 \tag{28}$$

Solution to Equations. In the present study, the equations for the two-line model were solved explicitly; however, there should be no difficulty in utilizing a mixed (explicit-implicit) procedure.

APPLICATIONS

As with any model, a complete evaluation requires comparison with as many other analytical and physical model results as possible. First a comparison between explicit and implicit one-line models will be demonstrated to provide confidence in the implicit solution followed by a comparison between the numerical model and the theory of Pelnard Considére, using two different transport equations in the numerical model. This is followed by specific applications which demonstrate the utility of the model.

In addition to intercomparison of numerical and analytical models, the model developed here is compared with the results from two physical model tests. Finally, the model predictions are compared with the results of a field measurement program.

Explicit Versus Implicit Schemes

To evaluate the implicit scheme, comparisons were carried out with results obtained from the explicit model. The wave conditions used in both models were a breaking wave height of 5 ft., an angle of wave approach of 45° from the north, and a duration of 1.39 days. The jetties were 1500 ft. long, and both the north jetty and the south jetty were oriented at angles of 20° to the shoreline. The time step in the explicit and implicit models are 600 and 6000 seconds, respectively. Because the differences between predicted shoreline changes by the two methods are small, rather than presenting plots, the results are tabulated in Table 1. The accretion is represented as a positive change and the erosion as negative. Both the x-distances (distance from shore end of the jetty at the baseline) and the y-coordinates have been rounded to the nearest tenth of a foot. Also, y-coordinates which did not change due to being outside the region of jetty influence are not presented in Table 1.

The distances proceed outwards in both directions from the jetty. Note that grid point 50 has a value of 176.2 ft. which is less than the value at grid 48. The explanation is simply that 176.2 ft. is the distance from the baseline to the jetty at grid 50 (i.e., the jetty is impounded with sand at this point). It is also worth noting that the south beach is affected for a larger distance from the jetty because the diffraction changes the wave heights along this stretch of beach and, therefore, the transport rate is not uniform even for the straight beach condition until the beach is out of the diffraction shadow zone.

TABLE 1
COMPARISON OF EXPLICIT AND IMPLICIT ONE-LINE MODELS
SHORELINE CHANGES ADJACENT TO JETTIES AT AN INLET

Grid Point	Distance From Base of Respective Jetty (ft.)	Shoreline Changes		Percent Difference (Explicit vs. Implicit)
		Explicit (ft)	Implicit (ft)	
North Jetty				
50	64.1	176.2	176.2	0.0
48	192.4	276.4	265.6	3.9
46	320.6	182.2	171.8	5.7
44	448.9	114.2	106.5	6.7
42	632.0	47.0	42.4	9.8
40	870.1	12.0	10.9	9.2
38	1108.0	2.4	2.3	4.2
36	1346.3	0.4	0.4	0.0
34	1158.4	0.1	0.1	0.0
South Jetty				
62	119.0	-16.8	-16.4	2.4
64	357.1	-13.7	-13.4	2.4
66	595.2	-29.0	-28.6	1.4
68	833.3	-55.6	-55.3	0.5
70	1071.4	-91.2	-90.9	0.3
72	1309.4	-126.9	-124.6	1.8
74	1547.5	-117.7	-114.5	2.7
76	1785.6	-55.9	-53.1	5.0
78	2023.7	21.1	22.9	-8.5
80	2261.8	13.1	12.9	1.5
82	2499.8	5.1	5.1	0.0
84	2737.9	16.0	1.6	0.0
86	2976.0	0.4	0.4	0.0
88	3214.1	0.1	0.1	0.0

The last column on the table, which gives percent differences demonstrates that the two methods are reasonably close. The error in the explicit procedure is due to the fact that the y values are computed using Q values determined at previous time steps whereas in the implicit scheme, they are solved simultaneously. As a summary statement of this comparison, the two approaches yield results which are in generally good agreement.

Theory of Pelnard Considére, Numerical Model With Pelnard Considére Transport Equation, and Comparison to Both With One-Line Implicit Scheme

The solution of Pelnard Considére has been plotted on Figure 3 for the following conditions: $\alpha_o = 22^o$, $A = 1.045\ ft^2/sec$, $t = 86,400$ sec. = 1 day.

As a check of the numerical model, the linearized transport equation of Pelnard Considére was introduced into the numerical model and the solution obtained as indicated by the dash-dot line in Figure 3. Also presented on Figure 3 is the solution resulting from the one-line implicit model. As expected, the shoreline has not accreted as far as with the other two models because the angle of wave attack is modified along the beach such that the sine of twice the difference between the wave angle and the shoreline decreases, thereby reducing the sediment transport. This exercise demonstrates that the finite-difference equations approach the analytic solution.

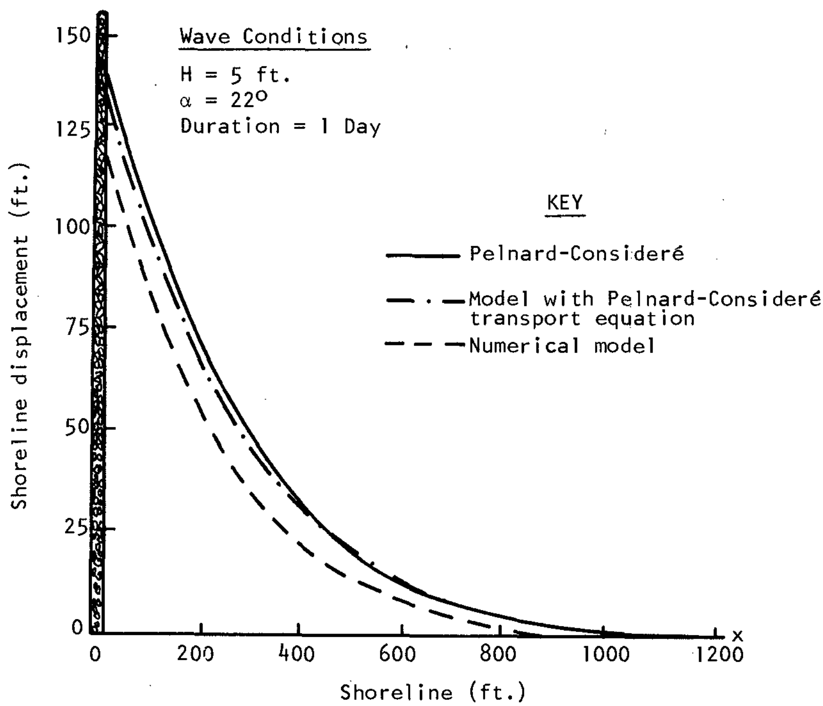

Figure 3. Comparison of the Three Predictions for Sand Accumulation Against a Littoral Barrier

Jetties Oriented Directly Into the Dominant Waves Versus Jetties Perpendicular to the Original Shoreline, With and Without Permeable Jetties

It appears that shoreline fluctuations adjacent to jetties could be minimized by orienting the jetties into the dominant wave approach, rather than perpendicular to the shoreline. In order to examine this problem, particular waves and jetty orientations were selected for modeling. The breaking wave height was 5 ft. incident on the beach at a +20° angle, with a period of 8 seconds and a duration of 1 day. Then, the wave angle was shifted to -20° for another day, +20° for a third day and finally back to -20° for the fourth and final day. First, these conditions were run with the 1500 ft. jetties perpendicular to the original baseline, and then the 1500 ft. jetties were oriented inward, each at an angle of 20°, i.e. into the incoming waves.

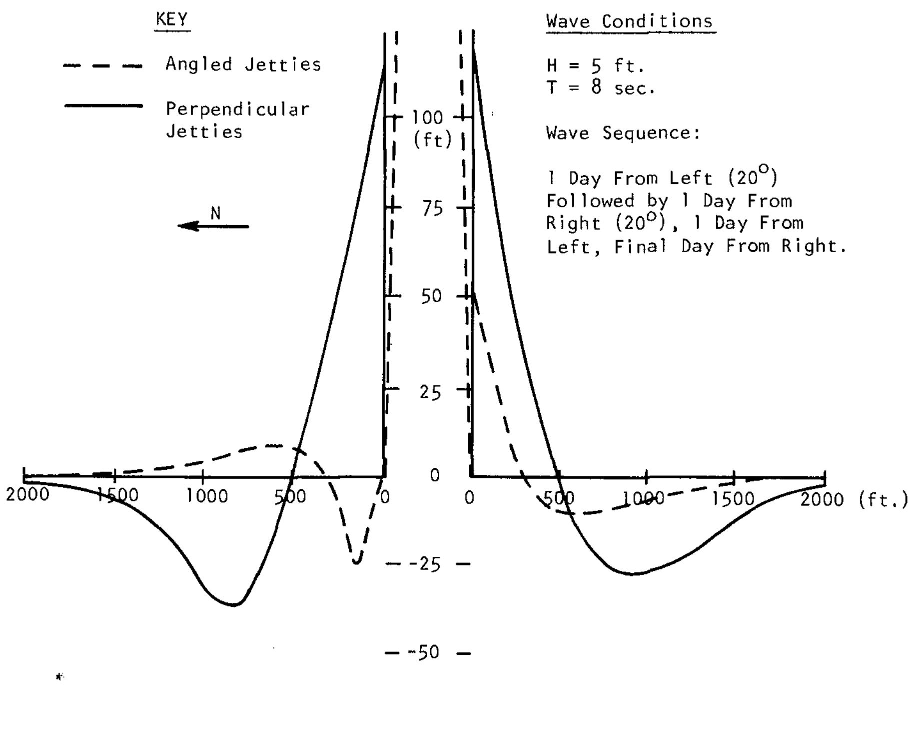

Figure 4. Comparison of Shoreline Response Due to Jetties Perpendicular to the Shoreline Versus Jetties Oriented Into Oncoming Waves.

The results of the simulation are presented graphically in Figure 4. Note the significant differences in magnitude in the shoreline changes for the two cases. With the angled jetties (the dashed lines in the diagram), the disturbance is considerably less, i.e., the shoreline changes for the perpendicular jetty case is modified more by the presence of the jetty. This is due to the redistribution of wave energy caused by the diffraction, or lack of diffraction in the perpendicular and angled jetty cases, respectively. Because each of the two wave directions causes shadow zones on the instantaneous downdrift beach, the sand is displaced on both sides and once the fillets have developed the waves are not able to transport the sand readily; therefore, the fillets remain.

In nature, waves are not monochromatic, nor do they originate from two directions, however, this simulation suggests that a structure should be designed considering the perturbations that different jetty orientations would cause to adjacent beaches.

Another application of the numerical model is to represent jetties which are not sand tight (i.e., permeable). The permeability characteristics of a particular barrier are, by far, not easy to determine. However, for the purpose of the model, a jetty with a permeability of 20% was selected. The definition of permeability, as used here, is that 20% of the sand which arrives at the grid adjacent to the barrier is carried through and lost to the system. In the case of two jetties, the sand is only carried through the respective jetty during periods when the wave conditions are such as to transport sediment toward that jetty.

Using the same wave conditions as were used for the comparison of perpendicular jetties and angled jetties, and a permeability for both jetties of 20%, the resulting beach in planform is shown in Figure 5.

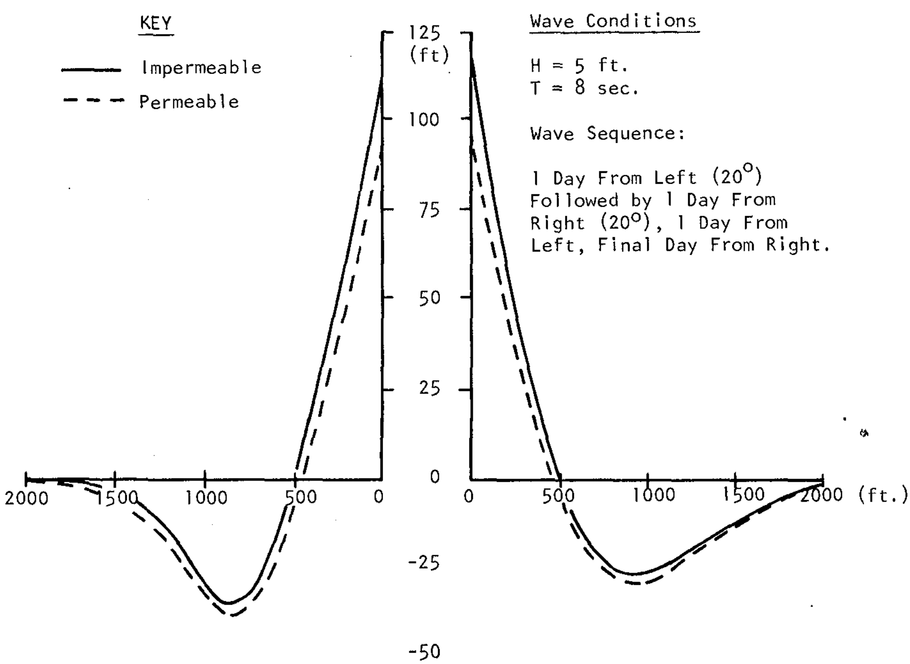

Figure 5. Effect of Permeable Versus Impermeable Jetties Oriented Perpendicular to Shore.

The difference between the total sand accumulation in the impermeable and permeable simulations for 1 day is 16%. The explanation of the difference between the permeability (20%) and the 16% difference is that only the grid adjacent to each jetty allows sand through it. As the beach accretes in the fillets, the transport rates in the adjacent grids to the jetty decrease due to the angle changes which occur. Therefore, a difference of 4% seems reasonable.

The effect of permeability on accretion of the fillets is of interest. Examining the south jetty, it is seen that the impermeable barrier impounded sand extending out from the original shoreline a distance of 112.5 ft., while the permeable barrier's beach extended out to approximately 92.5 ft. This is a change of slightly less than 18% which again seems reasonable.

The Physical Model of Barceló as Compared With the Numerical Model Prediction

One of the few physical models presented in the literature which contains sufficient information to model is that of Barceló[4]. Numerical model predictions for this hydraulic model test have been presented by Hulsbergen, Bakker and van Bochove[9]. In Barceló's model, two groins were present on each side of a beach which measured slightly less than 45 ft. in length. The updrift groin was approximately 18 ft. from his original shoreline with the downdrift groin approximately 7 ft. The beach was composed of pumice stones.

In order to simulate his model conditions, several of the constants had to be calculated and many other parameters had to be scaled from the figures in his article. The beach slope used was 8%, the length of the first jetty which caused the diffraction and was perpendicular to the original beach was 18.27 ft., the total beach length (including groin compartment) was 88.58 ft., and the length of time the model was run was 35 hours.

The depth, D, of the active profile was not given explicitly in the paper, however, examining the offshore profiles suggested an approximate value of 0.33 ft. with very small changes occurring for depths greater than 0.20 ft. For reasons to be discussed later, runs were carried out with $D = 0.33$ ft. and $D = 0.72$ ft.

The constant used in the sediment transport equation was changed because the mass density of pumice stone is different than quartz.

The results of the runs are shown in Figure 6. The most obvious differences are that for $D = 0.33$ ft. the numerical model predicts more rapid changes than measured in the physical model and it is noted that the beach in the immediate vicinity of the jetty adjacent to the eroded portion of the beach has a slightly different shape than the numerical model. The first difference is expected because different D values can be interpreted as different time scales (or transport proportionality factors) and laboratory data fall significantly below the field data on the curves used to establish the transport constant, see the Shore Protection Manual, 1973. Since the value of K used in the model ($K = 0.77$), was determined from field data, it is reasonable that the numerical model would predict higher degrees of erosion and accretion.

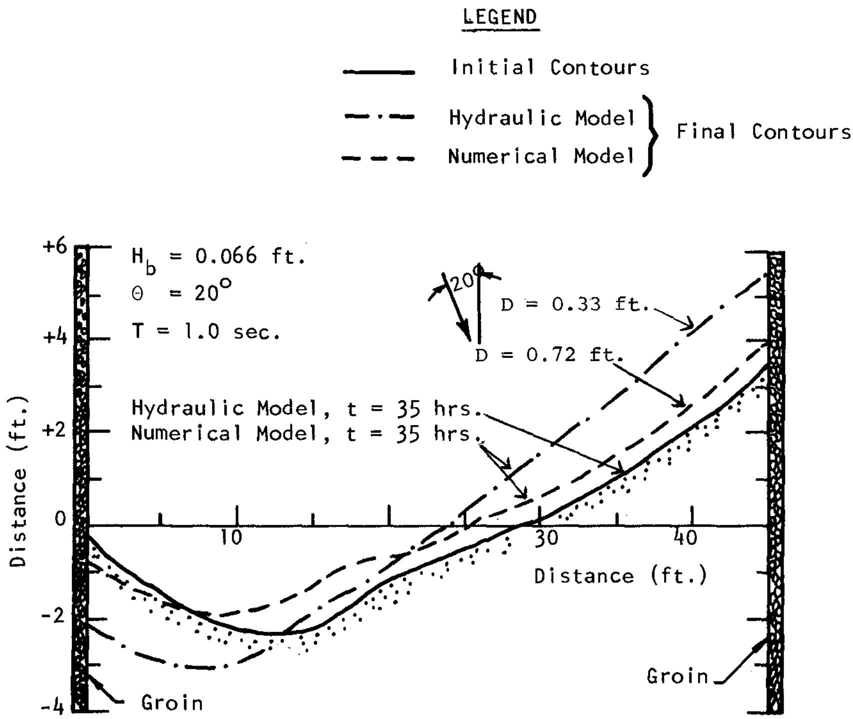

Figure 6. Comparison of One-Line Numerical Model Predictions with Hydraulic Model Tests by Barceló. Groin Compartment.

Comparison of a Two-Line Physical Model and the Two-Line Numerical Model

In order to evaluate the ability of the two-line numerical model to predict shoreline changes, the two-line physical model of Hulsbergen, Bakker and van Bochove[9] was simulated. Test "T22" was the run chosen to model because it was stated in the article as one of their best runs. The length of the beach was approximately 106.6 ft. and the duration of the test was 50 hrs. The beach material was comprised of dune sand and was fed into the model at the updrift boundary.

A list of the constants used is presented in Table 2. The sediment transport constant, Γ, was taken as 0.326 and the other required input data were taken or scaled from the article. It should be noted that a plane beach was not used in this model, but rather the idealized beach profile (Dean[8]) discussed earlier. Also, the value of "W" used in the numerical model was obtained by scaling from Figure 14 of the paper by Hulsbergen, Bakker and van Bochove. "W" represents the equilibrium distance between the two lines and the lines in this figure at t = 0 are supposed to be at equilibrium (the model was first run without the groins to reach equilibrium). The value scaled from the diagram was an average value of 5.97 ft., this being the value used in the numerical model.

TABLE 2

Constants	Value Used in Two-Line Model
Length of Groin	13.78 ft.
Length of Beach	106.63 ft.
Angle of the Groin	0^{o}
Shape Factor for Equilibrium Profile Dependent on Bed Material, see Reference 8	0.0778 $ft^{1/3}$
Reciprocal Depth of Closure	0.802 ft^{-1}
Constant of Proportionality, Γ, in Sediment Transport Equation	0.326
Equilibrium Distance Between Two Lines	5.97 ft.
Height of Breaking Wave	0.246 ft.
Wave Period	1.55 sec.
Direction of Wave Attack	5^{o}
Time Step	0.5 min.
Duration	6000 iterations (= 50 hours simulated)

The values input as "y_1" and "y_2" initial conditions were scaled from an enlarged version of Figure 14 of their paper. The only other change made in the numerical model was the boundary condition at the groin. The only way in which sand would be moved in the model with a wave height of only 0.246 ft. was by onshore/offshore sediment motion because breaking waves only occurred inshore of Line 2. This seemed to be realistic except at the boundary (groin) between the two beaches. Here, the amount of transport across the boundary was computed in the following manner. The distance from the tip of the groin to the y_2 grid point was divided by the total distance between the two lines, y_1 and y_2. This portion of the total offshore transport at the groin became the value of sediment transported around the barrier (i.e., at Line 2) and the remaining portion resulted in a seaward advancement of the first line, y_1. The sediment transported around the end of the barrier is used as the transport at the second line. This sand is then moved onshore accordingly.

Results of the comparison between the physical model tests and the numerical model tests are shown for the test duration of 50 hours in Figure 7. The prediction seems quite good with the exception of the smoothing that took place as expected. Certainly, the magnitude of the changes are approximately the same along with the general shapes. As noted in Reference 9, some of the perturbations apparent in the physical model could be due to rip currents which are not included in the numerical model.

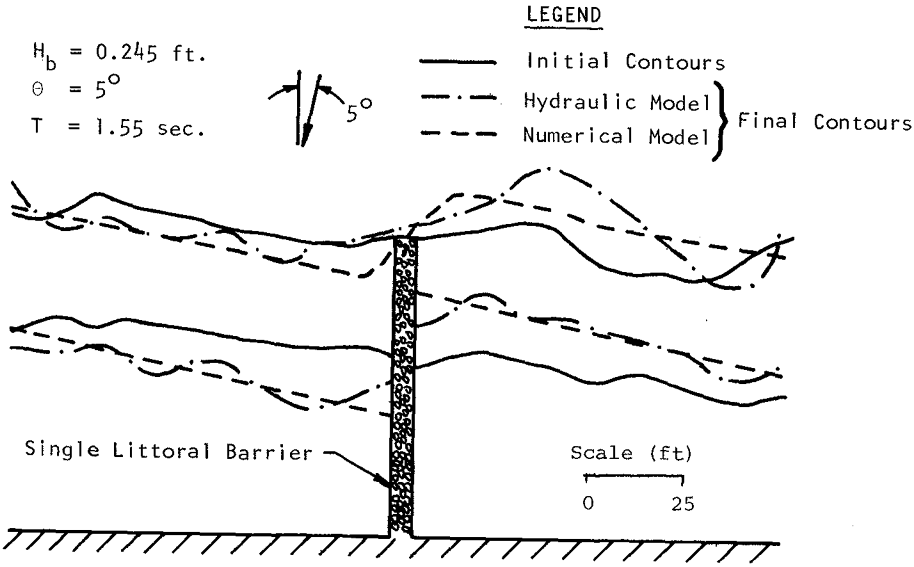

Figure 7. Comparison of Two-Line Numerical Model Predictions With Hydraulic Model Tests of Hulsbergen, Bakker and van Bochove.

Channel Islands Harbor Simulation

As a final application/evaluation of the model, it is desirable to compare model predictions with field data. Channel Islands Harbor was selected because concurrent wave data and planform change data exist. The wave data available at the time of writing this paper are LEO (Littoral Environmental Observational) visual data collected by the Coastal Engineering Research Center (CERC). The Channel Islands Harbor area of interest consists of two entrance jetties and an overlapping offshore breakwater. This structure system was idealized as shown in Figure 8. Two modeling efforts were carried out. The first utilized the recommended K value (c.f. Eq. (2)) of 0.77 and it was found that the amount of sediment accumulation behind the jetty was much too small. The second modeling was with a fourfold increase in K(= 3.08) and the results shown in Figure 8 were determined, which still indicates that not enough sediment is being transported and deposited behind the breakwater.

The Channel Islands Harbor study is being conducted by the Coastal Engineering Research Center as a full-scale sediment trap to attempt to determine the value of the constant, K (c.f., Eq. (2) of this paper). To date, it has been the general finding of that study that the constant (K = 0.77) is too small and perhaps should be increased by as much as twofold, Bruno and Gable[5]. There are various possibilities for the differences noted. The constant in the sediment transport equation could be too small for the transport at Channel Islands Harbor. Also, it is possible that on ebb tide a cell circulation is set up such that sand is transported toward the jetty by forces other than just those of the waves. The wave data were observed visually and a consistent underestimate in wave height by 25% would result in a sand transport which was too low by 51%. The effect of the dredged area would be to cause sand transport behind the breakwater, a three-dimensional effect not accounted for in the present sand transport relationship.

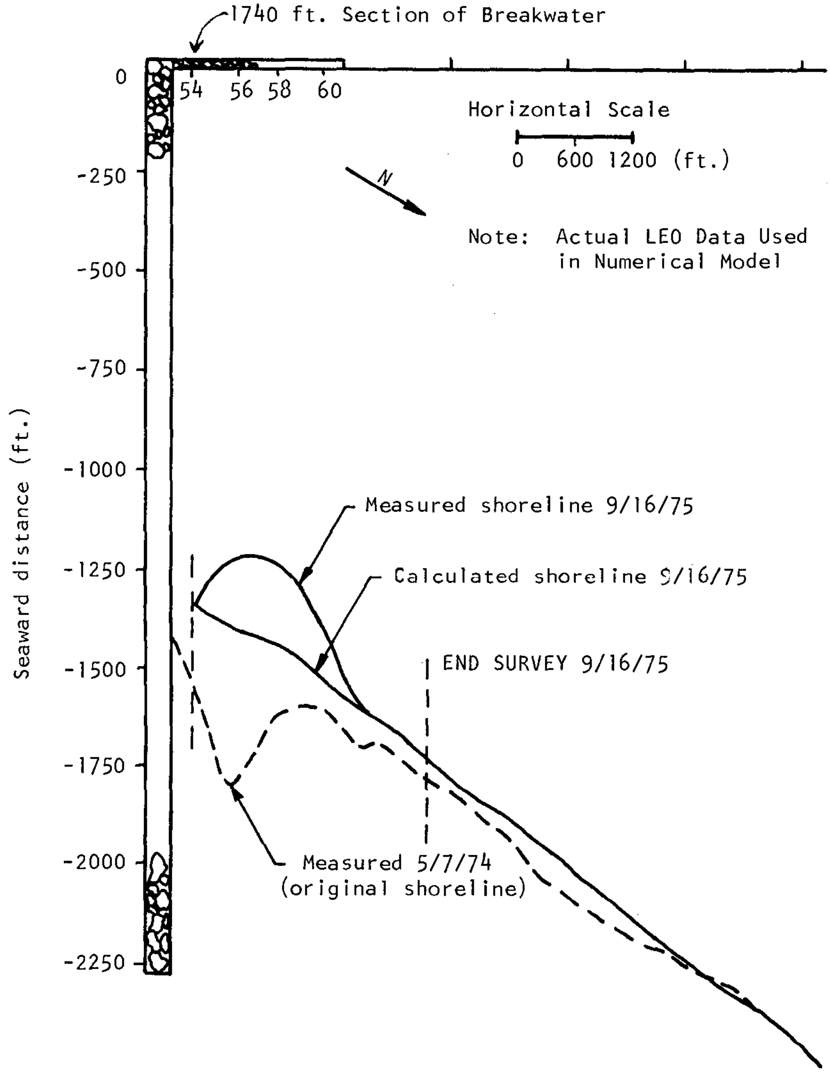

Figure 8. Comparison of Calculations With Field Surveys at Channel Islands Harbor, California. Longshore Transport Constant, K = 3.08.

SUMMARY

The governing equations and solution algorithms are presented for three types of numerical models developed to represent shoreline response to coastal structures. The models include: (1) a one-line explicit model, (2) a one-line implicit model, and (3) a two-line explicit model. This appears to be the first application of the implicit model for shoreline representation. In addition, the one-line models allow for activation and deactivation of grids adjacent to a structure oriented at an angle to the shoreline.

A number of problems of relevance to coastal engineering are investigated by the models including: permeable jetties, shoreline effects due to jetties aligned with the incoming waves compared with jetties perpendicular to the beach, shoreline response inside a groin compartment, and the effect of a littoral barrier. In addition, the one-line implicit model was applied to predict the shoreline response at Channel Islands Harbor, California where sediment is accumulated and dredged periodically from behind an offshore breakwater. Although in this case, only visual wave observations were available, it was necessary to increase the proportionality factor in the sediment transport relationship by a factor of four to obtain even approximate agreement between measurements and predictions.

The results of this paper are in accordance with those of other investigators of numerical models[3][9][12][16][18], namely that the potential is good for predicting shoreline response. Particular research needs to improve numerical modeling are:

(1) An improved understanding of the distribution of longshore sediment transport across the surf zone,

(2) An improved understanding of the mechanics of onshore-offshore sediment transport as affected by beach slopes milder and steeper than the equilibrium slopes, respectively,

(3) The effects of longshore bottom slopes such as would exist in the vicinity of a dredged hole, and

(4) Quantification of the sand transport processes in close proximity to structures. Of particular interest is the transport over a sill which is lower than the natural equilibrium profile elevation, and the transport through a permeable structure.

There appear to be no major difficulties in developing a "m-line" model and this should allow a more detailed description of shoreline response in the vicinity of structures.

REFERENCES

1. Abbott, M. B., "Numerical Methods", International Course in Coastal Engineering, Delft University, 1971, pp. 69-71.

2. Bakker, W. T., "The Dynamics of a Coast With a Groin System", Proceedings of the Eleventh Conference on Coastal Engineering, ASCE, 1968, pp. 492-517.

3. Bakker, W. T., E. H. J. Klein Breteler and A. Roos, "The Dynamics of a Coast With a Groyne System", Proceedings of the Twelfth Conference on Coastal Engineering, ASCE, 1970, pp. 1001-1020.

4. Barcelo, J., "Experimental Study of the Hydraulic Behavior of Groin Systems", Proceedings of the Eleventh Conference on Coastal Engineering, ASCE, 1968, pp. 526-548.

5. Bruno, R. O. and C. G. Gable, "Longshore Transport at a Total Littoral Barrier", Proceedings of the Fifteenth Conference on Coastal Engineering, ASCE, 1976, pp. 1203-1222.

6. Carnahan, B., H. A. Luther and J. O. Wilkes, "Applied Numerical Methods", John Wiley and Sons, 1969, pp. 429-450.

7. Dean, R. G., "Beach Erosion: Causes, Processes and Remedial Measures", CRC Critical Reviews in Environmental Control, Vol. 6, Issue 3, 1976, pp. 259-296.

8. Dean, R. G., "Equilibrium Beach Profiles: U. S. Atlantic and Gulf Coasts", Ocean Engineering Report No. 12, University of Delaware, 1977, pp. 1-45.

9. Hulsbergen, C. H., W. T. Bakker and G. van Bochove, "Experimental Verification of Groyne Theory", Proceedings of the Fifteenth Conference on Coastal Engineering, ASCE, 1976, pp. 1439-1458.

10. Inman, D. L. and R. A. Bagnold, "Littoral Processes", in The Sea; Ideas and Observations, Vol. 3, The Earth Beneath the Sea (Edited by M. N. Hill) Interscience Publications, pp. 529-553.

11. Komar, P. D. and D. L. Inman, "Longshore Sand Transport on Beaches", Journal of Geophysical Research, Vol. 75, 1970, pp. 5914-5927.

12. Le Mehaute, B. and M. Soldate, "Mathematical Modeling of Shoreline Evolution", A Literature Survey, Tetra Tech Report No. TC-831, January, 1977.

13. Pelnard-Considére, R., "Essai de Theorie de l'Evolution des Formes de Rivages en Plages de Sable et de Galets", U. S. Army Corps of Engineering Map Service Translation, 1954.

14. Penny, W. G. and A. T. Price, "The Diffraction Theory of Sea Waves and the Shelter Afforded by Breakwaters", Philosophical Transactions of the Royal Society of London, Ser. A, 244, March, 1952, pp. 236-253.

15. Perlin, M., "A Numerical Model to Predict Beach Planforms in the Vicinity of Littoral Barriers", M. S. Thesis, Department of Civil Engineering, University of Delaware, June, 1978.

16. Price, W. A., K. W. Tomlinson and D. H. Willis, "Predicting Changes in the Plan Shapes of Beaches", Proceedings of the Thirteenth Conference on Coastal Engineering, ASCE, 1972, pp. 1321-1329.

17. Saville, T., "Beach Profile Laboratory Tests, Large Wave Tank", Coastal Engineering Research Center, unpublished.

18. Willis, D. H., "An Alongshore Current Evolution Model", National Research Council of Canada, Hydraulics Laboratory, Report No. H4-92, March, 1978.