

CHAPTER 193

A TIME—DEPENDENT NEARSHORE MORPHOLOGICAL RESPONSE MODEL

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Abstract

This paper presents a numerical model for computing time-dependent near-shore hydrographic changes including beach profile responses. The time scale of the model is suitable for storm events to seasonal changes. The model is very stable and is capable of handling complicated topographies including inlets and irregularly-shaped structures such as curved jetties and breakwaters.

The basic approach is similar to many previous investigations utilizing a hydrodynamic model to drive a sediment transport model. The hydrodynamic model computes fully interacted current and wave fields based on coupled mild slope wave equation and depth-averaged circulation equations. The sediment transport model is of energetic type treating the rate of sediment as the summation of two energetic mechanisms one due to the mean current and the other due to the wave induced turbulence.

The model has been successful comparing with the evolution of beach profiles in large wave tank tests as well as other 2-dimensional numerical models of profile evolution. The model is able to predict the bar formation realistically without introducing constraints such as the bar genesis. At present the model consists of four modulars that apply separately for the situation of beach with no subaerial structure but could include non-reflective bottom structure, beach with shore-connected structures, beach with shore-detached structures and beach-inlet system. Test applications are presented here with some comparisons with field data and 3-D movable bed experiments.

INTRODUCTION

The nearshore zone is the area where the action of waves and currents on the sea bed is most intense, and where the bed material is almost always in motion. Erosion/accretion of the beach and change in offshore bottom topography occur naturally through the transport of sediment by waves and currents. Perturbations introduced by coastal structures, beach fills and other engineering activities may result in unexpected deformation of beach shapes and nearshore topographies. In the past, the prediction of beach evolution was mainly conducted by relying on experience of similar cases and on the results of hydraulic model tests. In

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recent years numerical models have gradually been developed and applied for this purpose.

The purpose of three-dimensional models is to predict the change of bottom topography from the spatial distribution of the sediment transport rates, which are evaluated from the nearshore wave and current fields computed point by point in small areas defined by a horizontal grid placed over the region of interest. Models of 3-D beach topography change require much fewer idealizations than do the line models. In this paper, a time-dependent nearshore morphological response model was developed. Test applications are presented together with comparisons of 3-D movable model test results and some field measurements.

OUTLINE OF THE MODEL

The model consists of three submodels for calculating (1) waves, (2) nearshore currents, and (3) sediment transport and bottom changes. The first two models are fully coupled to provide nearshore hydrodynamic condition. Nearshore waves, through radiation stresses, provide the driving force for the currents which, in turn, modifies the wave field. A compatible current and wave field is obtained through iterative process. This combined sediment transport model is then used as the driving force for the sediment transport model which also calculates bottom changes through sediment conservation equation. The change in topography again modifies the hydrodynamics. therefore, yet another level of current-wave-topographic interaction is required through frequent updating. At the first step, the initial beach topography and the geometry of the structures for the study area are given as input. Next, the wave model determines the spatial distributions of radiation stresses and near-bottom orbital velocities. The circulation model, then, computes the mean water surface level and the depth averaged mean currents using the radiation stresses from the wave model as the forcing terms and includes bottom friction, advective acceleration and the lateral diffusion terms. Finally, the sediment transport rates are computed at the local points from the wave-current conditions calculated in advance, and then the three dimensional bottom topography change is computed by solving the equation of sediment mass conservation. The wave and current fields are updated hourly to incorporate topographic changes and tidal variations.

Wave Model

Five contemporary numerical wave models were evaluated for their suitability (Lee and Wang, 1992). Two of them originally developed by Winer (1990) and Lee and Wang (1992), respectively were selected and modified for the present purpose. Both models account for current-wave-topography induced shoaling, refraction and diffraction effects. Empirical surf zone mechanics are also incorporated.

Both models are based upon the mild-slope equation given by Kirby (1984). Winer's model is the parabolic approximation version and theoretically is valid only for small incident wave angles. It employs a Crank-Nicholson finite difference scheme to solve the complex wave amplitude, A , of the following set of equations:

$$(C_g \cos \theta + U)A_x + \frac{\sigma}{2} \left(\frac{C_g \cos \theta + U}{\sigma} \right)_x A + V A_y + \frac{\sigma}{2} \left(\frac{V}{\sigma} \right)_y A - \frac{i}{2} k C_g (1 - \cos^2 \theta) A - \frac{i}{2} \left[C C_g \left(\frac{A}{\sigma} \right)_y \right]_y + \frac{\kappa}{2} A = 0 \quad (1)$$

$$\omega = \sigma + \vec{k} \cdot \vec{U} \quad (2)$$

$$\sigma^2 = gk \tanh kh \quad (3)$$

where t is the time, ∇_h is the horizontal gradient operator, $\vec{U}$ is the depth averaged horizontal velocity vector, C is the phase velocity, C_g is the group velocity and σ is the angular frequency. The last term in Eq. 1 is the energy dissipation term, where κ is the energy dissipation coefficient. This term has been added in order to deal with the wave decay and recovery after breaking. Eventually coefficient κ will be related to the energy dissipation due to wave breaking following the work of Dally et al. (1984). This model is exceptionally stable and efficient. It usually only requires very minor adjustment for different applications.

The other model is based upon elliptic wave equation using the Gragg's method to solve for $\tilde{\phi}$ which is the wave part of the velocity potential at the mean water level:

$$\begin{aligned} -i\omega[2U \cdot \nabla \tilde{\phi} + (\nabla \cdot U)\tilde{\phi}] + (U \cdot \nabla)(U \cdot \nabla \tilde{\phi}) + (\nabla \cdot U)(U \cdot \nabla \tilde{\phi}) \\ - \nabla \cdot (CC_g \nabla \tilde{\phi}) + (\sigma^2 - \omega^2 - k^2 CC_g - i\sigma\kappa)\tilde{\phi} = 0 \end{aligned} \quad (4)$$

This model has no small wave angle restriction and can accommodate reflective boundary conditions. The model is less stable and sometimes does not converge.

Circulation Model

The governing equations for the circulation model are the depth integrated time averaged horizontal equations of momentum (Ebersole and Dalrymple, 1979):

$$\frac{\partial U}{\partial t} + U \frac{\partial U}{\partial x} + V \frac{\partial U}{\partial y} + g \frac{\partial \bar{\eta}}{\partial x} + \frac{1}{\rho D} \bar{\tau}_x - \frac{1}{\rho D} \tau_{sx} + \frac{1}{\rho D} \left(\frac{\partial S_{xx}}{\partial x} + \frac{\partial S_{xy}}{\partial y} \right) + \frac{1}{\rho} \frac{\partial \tau_l}{\partial y} = 0 \quad (5)$$

$$\frac{\partial V}{\partial t} + U \frac{\partial V}{\partial x} + V \frac{\partial V}{\partial y} + g \frac{\partial \bar{\eta}}{\partial y} + \frac{1}{\rho D} \bar{\tau}_y - \frac{1}{\rho D} \tau_{sy} + \frac{1}{\rho D} \left(\frac{\partial S_{xy}}{\partial x} + \frac{\partial S_{yy}}{\partial y} \right) + \frac{1}{\rho} \frac{\partial \tau_l}{\partial x} = 0 \quad (6)$$

and the continuity equation

$$\frac{\partial \bar{\eta}}{\partial t} + \frac{\partial}{\partial x}(UD) + \frac{\partial}{\partial y}(VD) = 0 \quad (7)$$

where (U, V) are the depth-averaged velocity components in the x and y directions, respectively, $D = h + \bar{\eta}$, h is the still water depth, and $\bar{\eta}$ is the elevation of the mean water level due to wave setup/setdown, τ_l is the lateral shear stress due to turbulent mixing, $(\bar{\tau}_x, \bar{\tau}_y)$ are the bottom shear stresses, (τ_{sx}, τ_{sy}) are the surface shear stresses, S_{xx} , S_{xy} and S_{yy} are the radiation stress components which arise from the excess momentum flux due to waves. The radiation stress terms are forcing terms, whereas the bottom friction terms and the lateral mixing terms represent flow impedances. These equations are obtained by integrating the local x and y momentum equations and the continuity equation over the depth of the water column and then time averaging the results. The governing equations in the circulation model are solved by alternating direction implicit (ADI) scheme. In order to treat the wave-current interaction, alternate computations of waves and of currents are necessary.

Sediment Transport Model

Like the selection of wave model, a number of proven sediment transport models, mainly for 2-D profile modeling, were evaluated. They were compared on their basic transport mechanism. This was accomplished by comparing the transport forcing function expressed in equivalent shear stress term. Table 1 shows the equivalent shear stress terms from various models. The comparison under one specific wave condition is illustrated in Figure 1. It can be seen that they all assume similar shape. The transport model following that developed by Ohnaka and Watanabe (1990) was selected because the formulation is most suitable for 3-D application. The rate of sediment transport is treated as the summation of two energetic mechanisms one due to the mean current and the other due to the wave induced turbulence. The transport due to the mean current is

Table 1: Equivalent Shear Stress from Different Sediment Transport Models

Energy Loss (Dally, Kriebel, Larson and Kraus)
$\tau_e = -\frac{2}{H} \sqrt{\frac{h}{g}} \frac{\partial(EC_d)}{\partial x}$
Time-Averaged Bottom Velocity (Dally)
$\bar{\tau} = \frac{1}{T} \int_0^T \rho \frac{t}{2} u_b^2 dt \quad u_b = \frac{H}{2} \sqrt{\frac{g}{h}} \cos \omega t$
Deficit of Moment of Momentum (Houston and Dean)
$\bar{\tau}_b = -\frac{2}{h} \frac{\partial}{\partial y} \left[\frac{Eh}{4} \left(1 + \frac{H^2}{2h^2} \right) \right]$
Combined Bottom Velocity (Watanabe)
$\tau_c = U^2 + V^2 + \hat{u}^2 + 2U\hat{u} \cos \theta + 2V\hat{u} \sin \theta$
$\hat{u} = \frac{\pi H}{T \sinh kh}$

$$\vec{q}_c = \frac{A_c(\tau - \tau_{cr})\vec{U}}{\rho g} \quad (8)$$

where A_c is a nondimensional coefficient (of the order 0.1 to 1.0), values of which should be empirically determined. τ is the maximum value of the bottom shear stress in a wave-current coexistent field. τ_{cr} is the critical shear stress for the onset of sediment movement, ρ is the density of water, and g is the gravitational acceleration. If $\tau \leq \tau_{cr}$, $\vec{q}_c$ is zero. The transport due to waves is

$$\Phi = B_w(\Psi_m - \Psi_c)\Psi_m^{1/2} \quad (9)$$

where $\Phi = (1 - \lambda_v)q_w/w_0d$ is a dimensionless net transport rate, d , w_0 and λ_v are the diameter, settling velocity, and void ratio of the sediment, respectively, B_w

is a nondimensional coefficient (of order 2 to 10), Ψ_m is the Shields parameter and Ψ_c is the critical value of Ψ_m for the onset of general movement of sediment. Since q_w is the absolute value of the net transport rate, a method is needed to determine the direction of the net transport. Criteria for predicting whether a beach will erode or accrete through cross-shore sand transport processes have been suggested by a number of authors. The two most commonly used parameters include one for characterizing the deepwater wave steepness, H_0/L_0 , and the other one related to the relative sediment fall velocity, H_0/w_0T , also known as the Dean's parameter. A version proposed by Larson et al. (1989) in their SBEACH model was adopted here. The criterion for distinguishing beach erosion and accretion can be expressed by the following equations:

$$\frac{H_0}{L_0} \leq C \left(\frac{H_0}{w_0T} \right)^3, \quad \text{erosion}$$

$$\frac{H_0}{L_0} > C \left(\frac{H_0}{w_0T} \right)^3, \quad \text{accretion}$$

in which $C = 0.00070$ is an empirical coefficient.

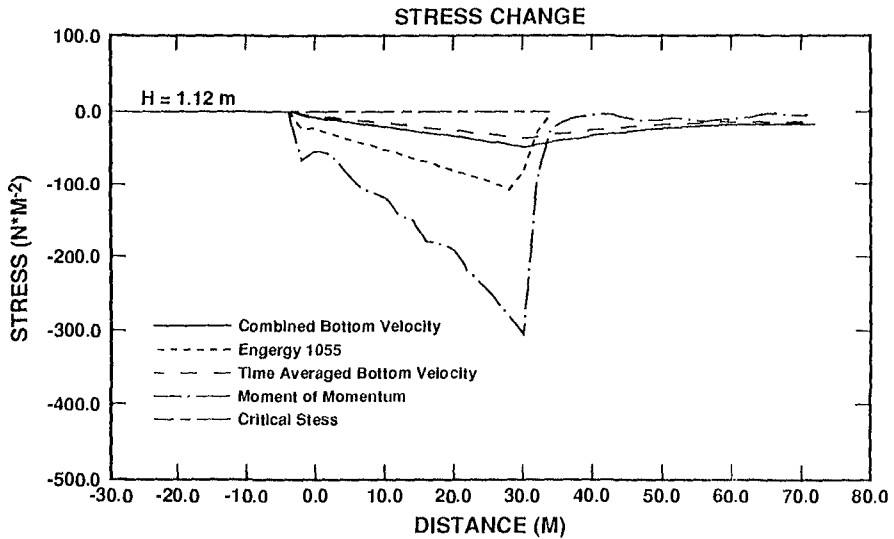


Figure 1: Comparisons of equivalent shear stresses for different transport models.

The change in local bottom elevation, z_b , or water depth, h , can readily be computed from the spatial distribution of the sediment transport rates by solving the following equation for the conservation of sediment volume

$$\frac{\partial h}{\partial t} = \frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} \quad (10)$$

$$\vec{q} = \vec{q}_w + \vec{q}_c = (q_x, q_y) \quad (11)$$

MODEL VALIDATION AND APPLICATIONS

In order to validate the 3-D morphological response model, several large wave tank results of dune erosion were simulated. These included Saville's (1957) large wave tank tests and the case of a sand beach backed by a sloping dike tested in the BIG WAVE FLUME, Germany (Dette and Uliczka, 1986).

Figure 2 shows a comparison of measured and calculated profiles for CE Case 400, from Saville, for which: initial slope = $1/15$; grain size = 0.22 mm ; wave height and period of 1.62 m and 5.6 sec in the horizontal section of the tank (depth = 4.42 m); and constant water level. The numerical and test results are shown for simulation times of 1, 3, 5, 10, 15, and 20 hours. The numerical model satisfactorily reproduced the observed forshore erosion and main breakpoint bar development. Simulated shoreline retreat and bar growth was initially rapid and gradually slowed as the bar moved offshore to reach a location close to that of the observed bar at the end of the run (20 hr).

A similar comparison is shown in Figure 3 for the case of 'dune with fore-shore': $+2\text{ m}$ above SWL and 10 m wide dune with 1 to 4 seaward slope down to 1 m below SWL and following 1 to 20 slope down to channel floor. The BIG WAVE FLUME is 324 m long, 7 m deep and 5 m wide. The test profile was subjected to regular waves ($H = 1.5\text{ m}$, $T = 6\text{ s}$, $h = 5.0\text{ m}$). The predicted profiles are shown at times of 62, 111, 190 and 273 minutes and are compared to the measured profiles. The computed wave height distribution across shore is also shown. The waves cut back the foreshore to produce a vertical scarp, and a bar formed near the break point which grew and moved offshore with continued wave action. The volume of the main breakpoint bar and the amount of erosion on the foreshore are rather well predicted by the numerical model. However, the bar trough is less well reproduced. The model is incapable of simulating micro features such as the bottom undulations inside the breaking zone.

The model is also compared with other 2-D dune erosion models including Kriebel and Dean (1984) and Larson et al. (1989). The comparisons of profile changes in the nearshore zone are all very close; they deviate in bar formations. Figure 4 shows the comparison between the present model and the SBEACH applied to a prototype profile and wave condition. The Kriebel's model (not shown here) almost duplicates the SBEACH in the nearshore zone but produces no bar.

The present numerical model has been applied to a variety of cases including beach nourishment, beach with submerged rocks, offshore detached breakwater, shore-perpendicular structures, inlet-beach system, etc. The model results of waves, currents and topographic changes for a region around Sebastian Inlet are given here as an illustration. Sebastian Inlet is located at the east coast of Florida between the Brevard and Indian River County line approximately 45 miles south of Port Canaveral entrance. The bathymetric map of 1989 is given in Figure 5. It is located in a littoral drift zone predominately from north to south. The inlet has been kept open by means of maintenance dredging and jetty improvements. It suffers from channel shoaling and downdrift beach erosion. Movable bed physical model and field studies were carried out (Wang et al., 1991, 1992) to seek improvement measures.

A few examples are given here to show the comparison between numerical simulation and field or laboratory results. First of all, the wave patterns under

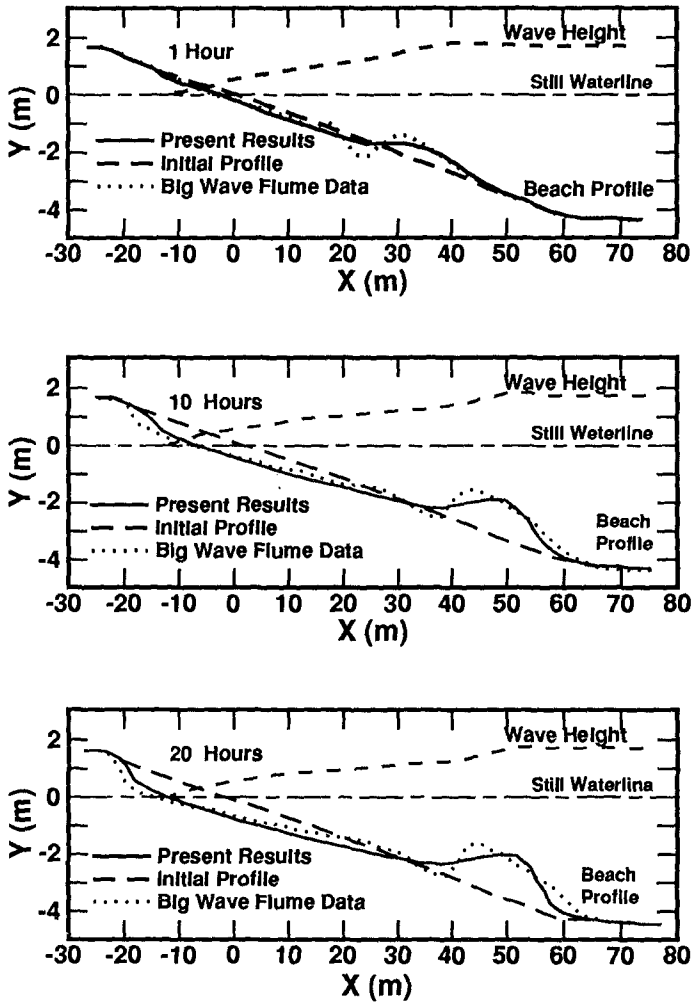


Figure 2: CE case 400, Comparison between simulated and measured beach profiles. $t = 1, 3, 5, 10, 15, 20$ hours. Data from Saville (1957).

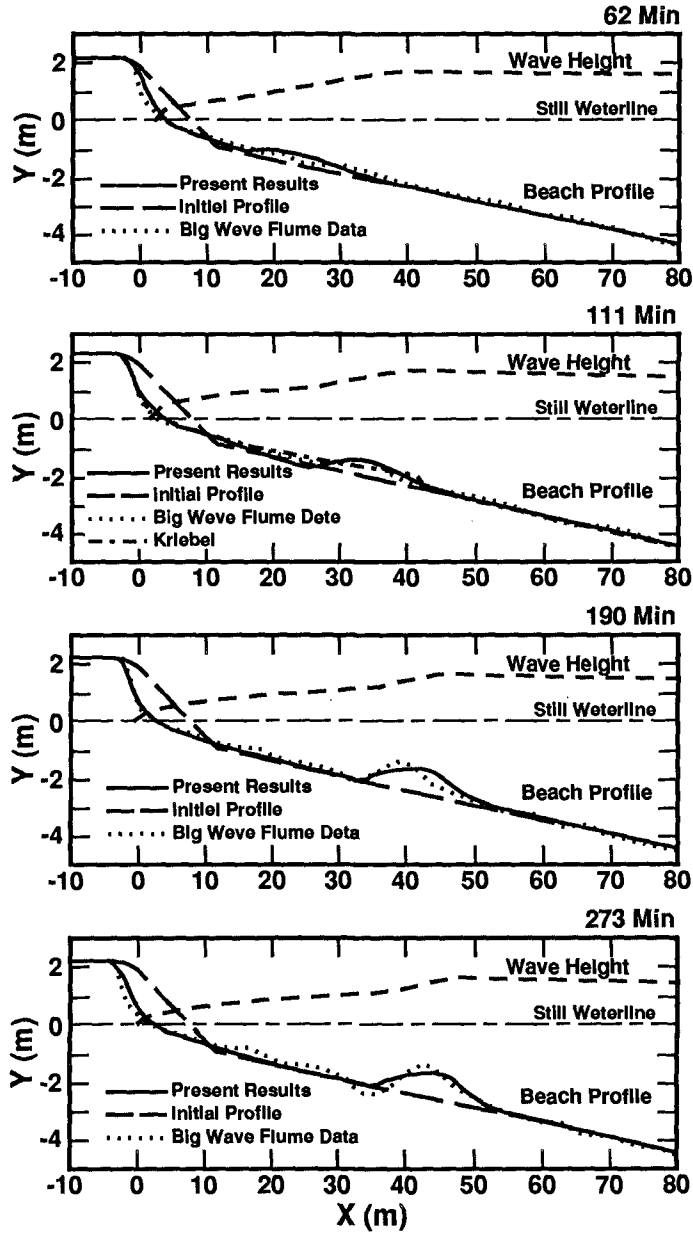


Figure 3: Comparison between simulated and measured beach profiles, measured data from Dr. Dette. $t = 62, 111, 190, 273 \text{ min}$

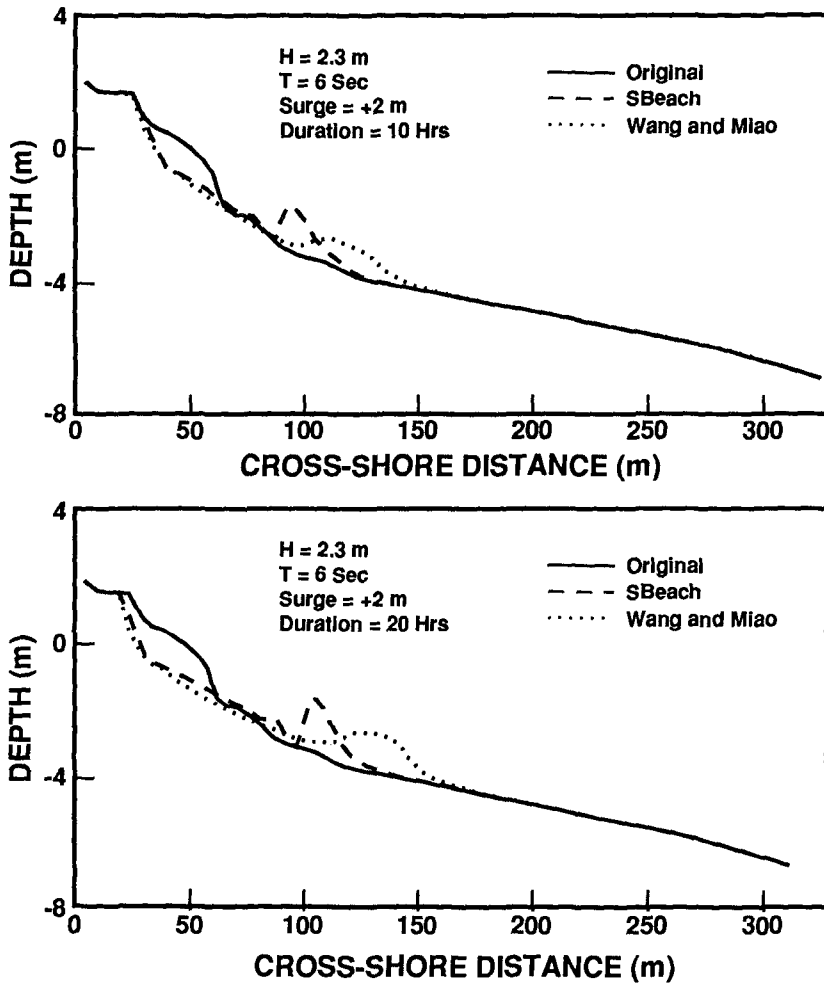


Figure 4: Comparison between present model and SBEACH model

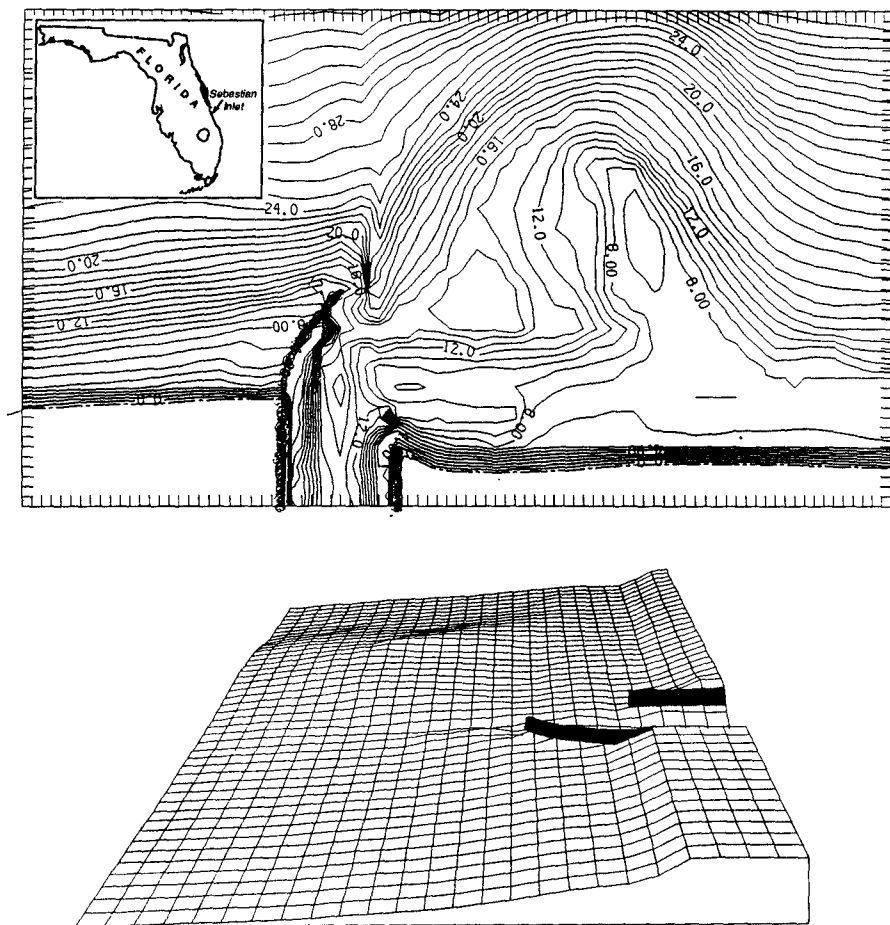


Figure 5: Sebastian Inlet bathymetric map for year 1989.

long crested conditions can be closely simulated by the numerical model. This can be seen in Figure 6 which compares wave pattern recorded by aerial photo with simulation.

The current in the inlet is tidal driven, often reaching to 2.5 m/s during ebb and flood. In the vicinity and outside the inlet, the current field is very complicated. During ebb cycle, the current behaves like a jet carrying with it a rather concentrated seaward momentum. This jet when deflected by the curved north jetty directs itself towards the southeast, which gradually shaped the main flood-ebb channel. A clockwise vortex is formed on the south side behind the jet stream. During flood cycle, the current converges to the inlet like flow into a funnel. The flow pattern is influenced by both the ebb shoal and the jetties. A very strong current developed near the tip of the south jetty, causing a major scouring hole at its tip. The current patterns simulated by the numerical model are compared with those measured in the field in Figure 7. Again, like wave field simulation, the current pattern appears to be quite reasonably reproduced.

For morphological changes the numerical model is compared with movable bed physical model tests. The model has a horizontal scale of 60 using finer sand than the prototype as bed material (detail see Wang et al., 1992). The actual physical dimensions of the model are approximately alongshore and offshore. Figure 8 compares the numerical and physical model results for a specific case with the following input conditions: the offshore boundary is $H = 2\text{ m}$, $T = 8\text{ s}$, and the incident wave angle at the offshore boundary is 10° (from the north-east). The tide is semi-diurnal with a range of 1.0 m . The plot gives the cumulative bottom changes after a time period equivalent to 6-days in prototype. The solid lines represent accretion and dash lines represent erosion with contour interval of 0.1 m . The most visible bathymetric changes occur in the nearshore region behind the ebb shoal including the area near the tip of the south jetty. Beach face is eroded by wave action and sand is carried out towards offshore forming offshore bars. Meanwhile, the ebb shoal experienced vigorous sediment motion due to wave breaking and the sediment is also carried into the nearshore region. Although the details are somewhat different the agreement can be considered good macroscopically as most of the prominent topographical changes are correctly predicted.

CONCLUSIONS

A numerical model has been developed for computing time-dependent near-shore hydrographic changes including beach profile responses. The time scale of the model is suitable for storm events to seasonal changes. The model is very stable and is capable of handling complicated topographies including inlets and irregularly-shaped structures such as curved jetties and breakwaters.

The model has been fairly rigorously calibrated against 2-D large scale beach profile evolution tests carried out in the GWK tank and in the CERC's large tank. The model is also verified with other 2-D numerical models. The model is capable of describing the growth and movement of main breakpoint bars and corresponding berm processes with reasonable reliability. Calibration and verification for 3-D cases are limited. So far, a set of field and laboratory movable bed data concerning one specific inlet configuration has been utilized. The numerical model seems to perform well to predict wave patterns of long crested waves; gives reasonable current patterns. Consequently, the patterns of topographic changes can also be reasonably predicted. Of particular importance is the ability of the model to depict correctly the sediment transport patterns that can add to the

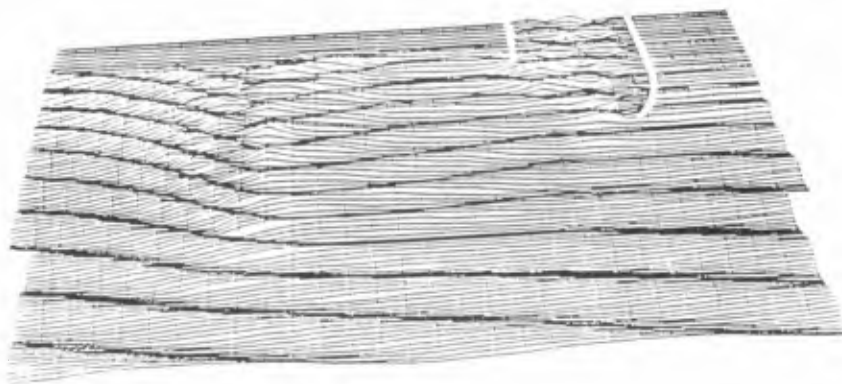


Figure 6: Comparisons of wave patterns from aerial photography and model at Sebastian Inlet.

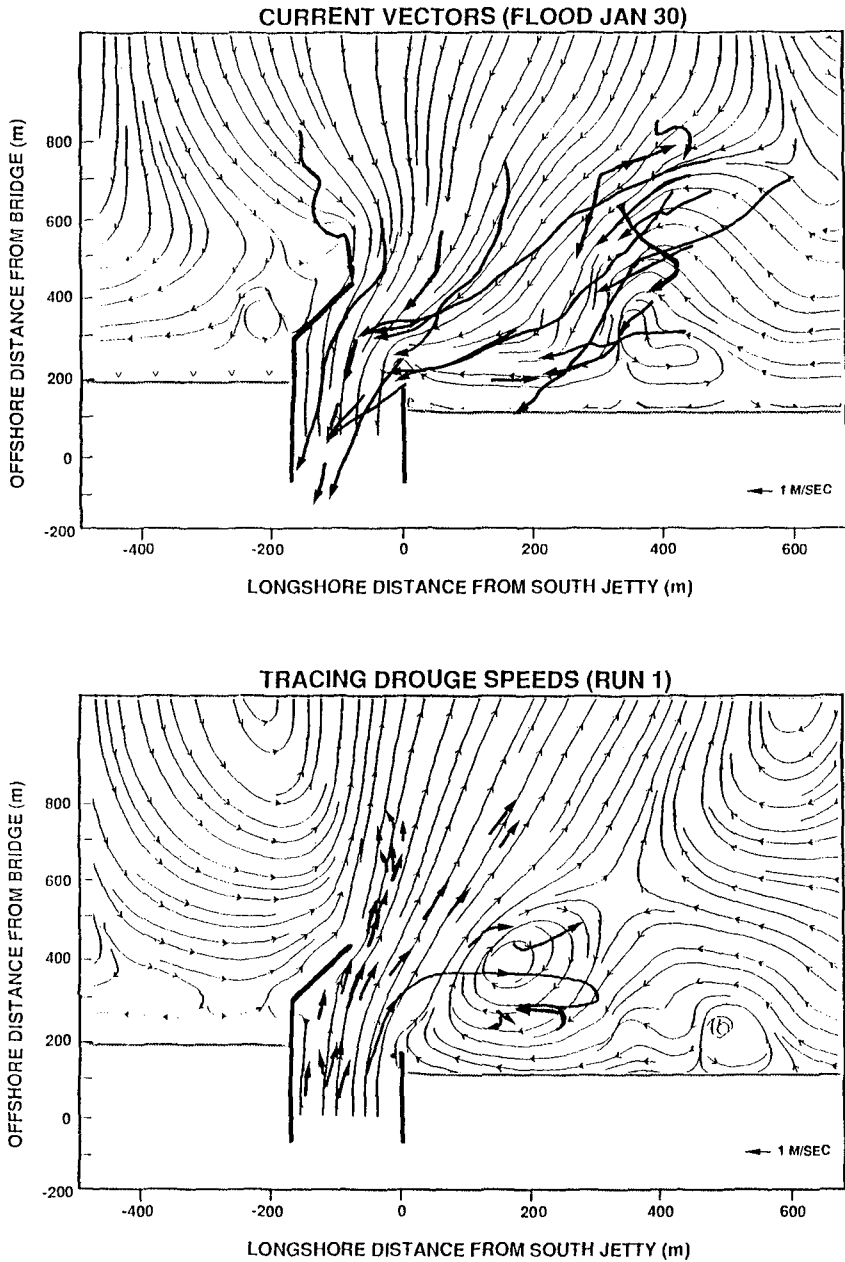


Figure 7: Comparisons of current patterns from the numerical model and field measurements.

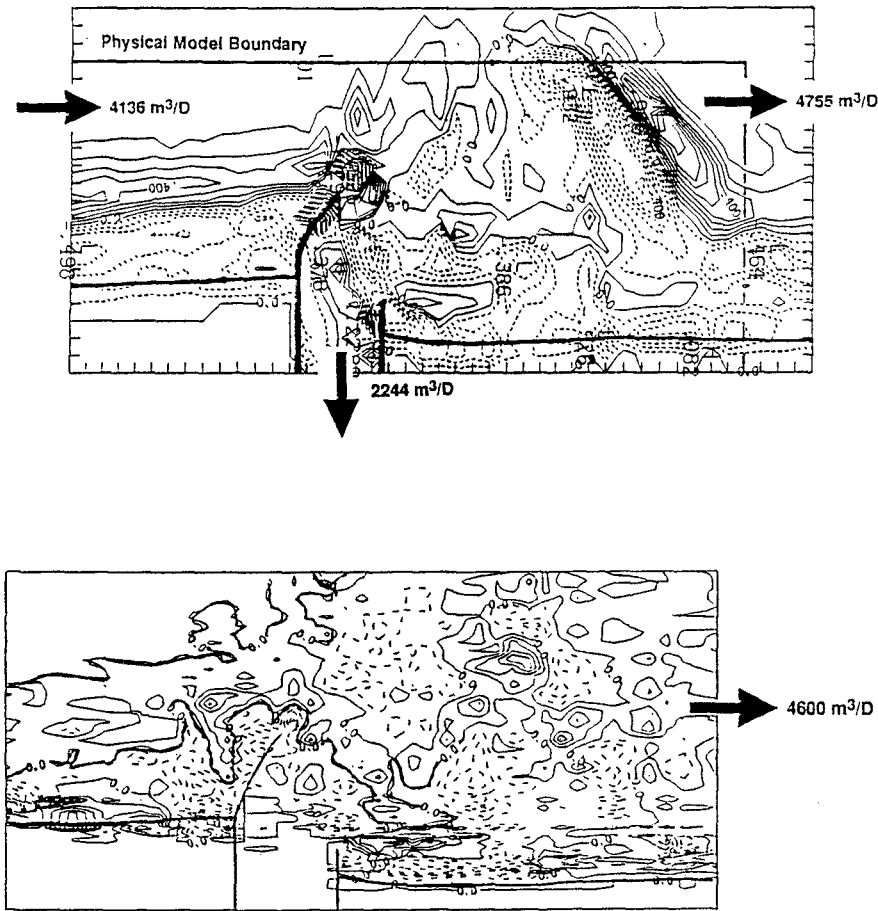


Figure 8: Comparison of bathymetric changes after 6-day NE storm.

insight to the sediment transport process. Of less confidence is the quantity of transport as well as detailed topographic changes. Work is still in progress to improve the reliability of the model as well as expanding the scope of the model.

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