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Effect of Uncertainties in Wave Characteristics on Shoreline Evolution

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ABSTRACT

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Shoreline prediction due to a shore-connected structure is numerically modeled based on the approach proposed by KRAUS and HARIKAI (1983). The influence of the different coastal variables on the shoreline evolution is brought out by carrying out a sensitivity analysis. Since the shoreline response is influenced by a number of variables, which are random in nature, a probabilistic approach similar to that of Vrijling has been considered. The developed model is applied to predict the shoreline advance to the south of Madras Port, India and the predictions are validated with field measurements. Further, the model is used to predict the seasonal variations in shoreline response.

ADDITIONAL INDEX WORDS: *Shoreline change, erosion, accretion, coastal sediments.*

INTRODUCTION

Prediction of shoreline response to the presence of a structure is important because of the high cost involved with the protection of the coast from the erosion. India has a vast coastline of about 7,000 km. A considerable portion of it, especially the stretches along the east coast, experiences severe erosion due to the presence of natural headlands or man made structures.

Numerical models offer the capability to incorporate wave characteristics and sediment transport and have the potential of providing a reasonable estimate of the shoreline response. The increased capacities of computational facilities with improved numerical algorithms have resulted in an extremely promising potential for numerical modeling of the near shore phenomena.

As the wave characteristics are random in nature, a probabilistic approach is more realistic and is adopted in the present study. The probabilistic approach of the shoreline computations is carried out using the methodology similar to that of VRIJLING and MEIJER (1992). The shoreline evolution in this case is represented with a mean shoreline obtained in response to the average wave data along with the maximum possible deviation of the shoreline due to the uncertainties in the wave characteristics.

DEVELOPMENT OF NUMERICAL MODEL

General

The mathematical modeling of shoreline evolution essentially relates the change of the beach volume to the rate of material transported from the beach. When the changes in the mean sea level and change in beach slope are not significant, the governing differential equation of a oneline model will take the form.

$$(b + D_c)\frac{\partial y}{\partial t} = -\frac{\partial Q}{\partial x} + q(x) \qquad (1)$$

where y is the shoreline position, positive towards the offshore is the function of x measured along the shore and time t, b: height of berm, D_c: limit of active sand transport beyond which sand transport changes can be assumed to be negligible (Depth of closure), Q: wave induced longshore sediment transport and q(x): quantity of sediment added per unit length of shoreline by various agencies like sand deposited or dredged during the beach nourishment, loss or gain of sand by wind *etc.* HALLERMEIER (1981) has expressed D_c as a function of significant wave height and period, which underestimates D_c leading to overestimates of shoreline changes. In the present study, D_c which depends on the grain size of the sediments, D_{50}, is taken as a depth at which the sediments start lifting from the sea bed and is computed based on the formula proposed by U.S. Army (1984):

$$U_{max(-D_c)} = \left(8\left(\frac{\gamma_s}{\gamma} - 1\right)gD_{50}\right)^{1/2} \qquad (2)$$

in which $U_{max(-D_c)}$ = maximum horizontal particle velocity at sea bed defined by the linear wave theory, γ_s = specific weight of sediments, γ = specific weight of water and g = gravitational constant.

The wave induced longshore sediment transport based on the longitudinal energy flux according to U.S. Army (1984) is:

$$Q = 1{,}290 \times P_l (m^3/year) \qquad (3)$$

where

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P_l = longitudinal energy flux

$$= \frac{\rho g^2}{32\Pi} H_0^2 T K_R^2 \sin(\alpha_b)\cos(\alpha_b)$$

ρ = mass density of sea water in $Nsec^2/m^4$

H_0 = deep water wave height in meters

T = wave period in seconds

K_R = refraction coefficient = $\sqrt{\frac{\cos(\alpha_0)}{\cos(\alpha_b)}}$

in which α_b and α_0 are the wave angles at the breaker depth and deep water respectively. LE-MEHAUTE and KOH (1981) approximated breaker angle α_b as

$$\alpha_b = \beta\alpha_0 \quad (4)$$

where $\beta = 0.25 + 5.5(H_0/L_0)$, in which L_0 is deep water wave length. The effects of diffraction are accounted by replacing the refraction coefficient K_R by $K_R K_D$.

$$Q = AK_D^2 \cos(\alpha_0)\sin(\alpha_b) \quad (5)$$

in which $A = (1{,}290/32\Pi)\rho g^2 H_0^2 T$ and K_D is diffraction coefficient. The diffraction coefficient K_D is calculated by the method proposed by DEAN and DALRYMPLE (1984). Setting the scale factors for y, x, t, and Q Eq. 1 may be written in a nondimensional form as

$$\frac{\partial y^*}{\partial t^*} = -\frac{\partial Q^*}{\partial x^*} + q^*(x) \quad (6)$$

where

$$y^* = \frac{y}{(b + D_c)},$$

$$x^* = \frac{x}{(b + D_c)},$$

$$t^* = t(A/(b + D_c)^3),$$

$$Q^* = Q/A \quad \text{and}$$

$$q^*(x) = q(x) \times (b + D_c)/A$$

Present Numerical Model

The nondimensional equation of shoreline evolution is expressed in the finite difference scheme as

$$y^*_{n,t^*+1} = B\{Q^*_{n,t^*+1} - Q^*_{n+1,t^*+1}\} + C_n \quad (7)$$

where

$$B = \frac{\delta t^*}{2 \times \delta x^*} \quad \text{and}$$

$$C_n = B\{Q^*_{n,t^*} - Q^*_{n+1,t^*} + 2\delta x^* q^*_{n,t^*}\} + y^*_{n,t^*}$$

The nondimensional shoreline is divided into N grid points at equal nondimensional interval δx^*. Then shoreline changes over a nondimensional time δt^* is calculated using Crank-Nicholson finite difference scheme.

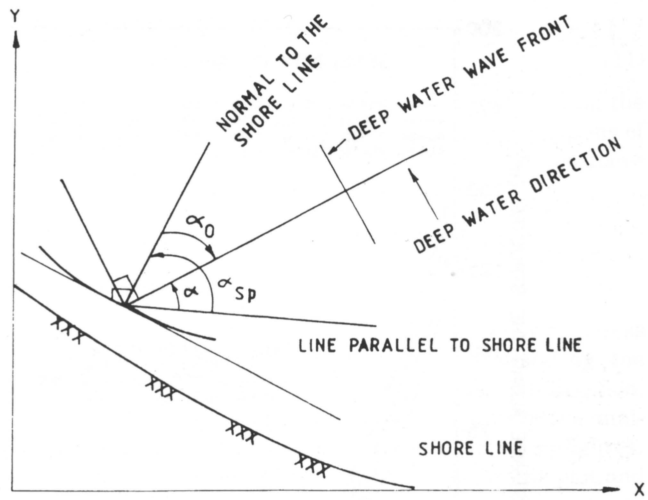

Figure 1. Sketch defining angles α_0, α and α_{sp}.

In this method Q^* at the time interval $(t^* + 1)$ is expressed in terms of the shoreline co-ordinate of y^*, first isolating the term involving α_{sp} (angle of shoreline normal to x-axis) using trigonometric identities. One of the terms involving α_{sp} is then expressed as first order quantities in y^* at the time step $(t^* + 1)$.

$$Q^* = K_D^2 \cos(\alpha_0)\sin(\alpha_b) \quad (8)$$

where $\alpha_0 = \alpha - \alpha_{sp}$ and α = wave direction with respect to x-axis.

The angles α_0, α and α_{sp} are defined in Figure 1.

$$Q^* = K_D^2 \cos(\alpha - \alpha_{sp})\sin(\alpha_b)$$

$$Q^* = K_D^2 \sin(\alpha_b)\cdot\{\cos(\alpha)\sin(\alpha_{sp})\cot(\alpha_{sp}) + \sin(\alpha)\sin(\alpha_{sp})\}$$

$$Q^* = E_n\{y^*_{n-1,t^*+1} - y^*_{n,t^*+1}\} + F_n \quad (9)$$

where

$$E_n = K_D^2\{\cos(\alpha)\sin(\alpha_{sp,t^*})\sin(\alpha_{b,t^*})\}/\delta x^*$$

and

$$F_n = K_D^2\{\sin(\alpha)\sin(\alpha_{sp,t^*})\sin(\alpha_{b,t^*})\}$$

Substituting eq. 9 in eq. 7 will result in an expression of the form

$$BE_nQ^*_{n-1,t^*+1} - (1 + 2BE_n)Q^*_{n,t^*+1} + BE_nQ^*_{n+1,t^*+1}$$
$$= E_n[C_n - C_{n-1}] - F_n \quad (10)$$

The above equation represents a set of $(N - 1)$ linear equations for $(N - 1)$ unknowns. The end values are specified as boundary conditions *i.e.,* $Q^*_1 = 0$ and $Q^*_{N+1} = Q^*_N$. The above equation results into a tridiagonal form which is solved for Q^*. The y^* is then calculated using Eq. 7. This process is repeated for the entire duration and nondimensional quantity is converted into real quantities using the corresponding scale factors.

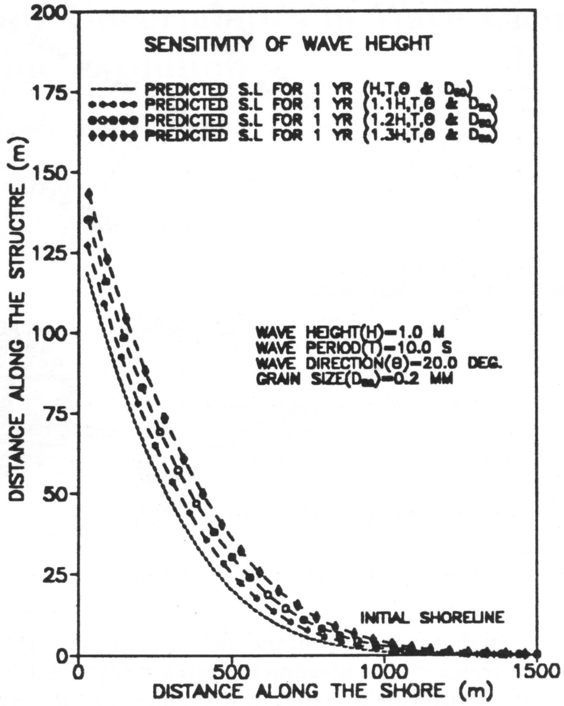

Figure 2a

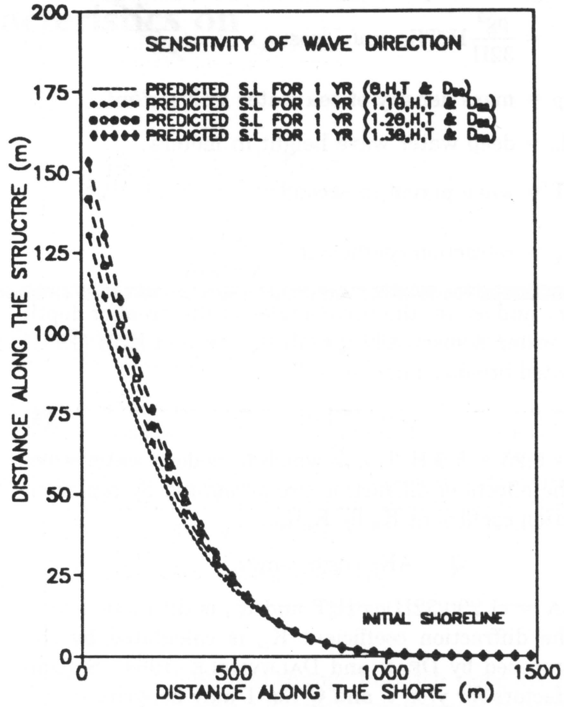

Figure 2b

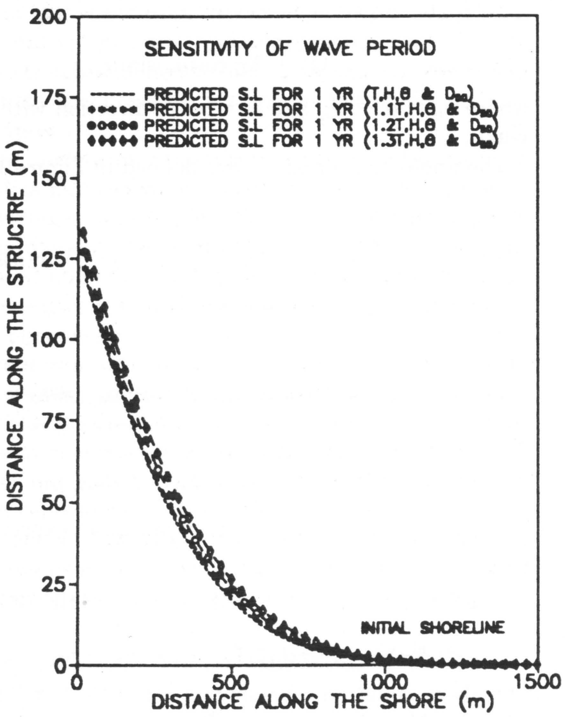

Figure 2c

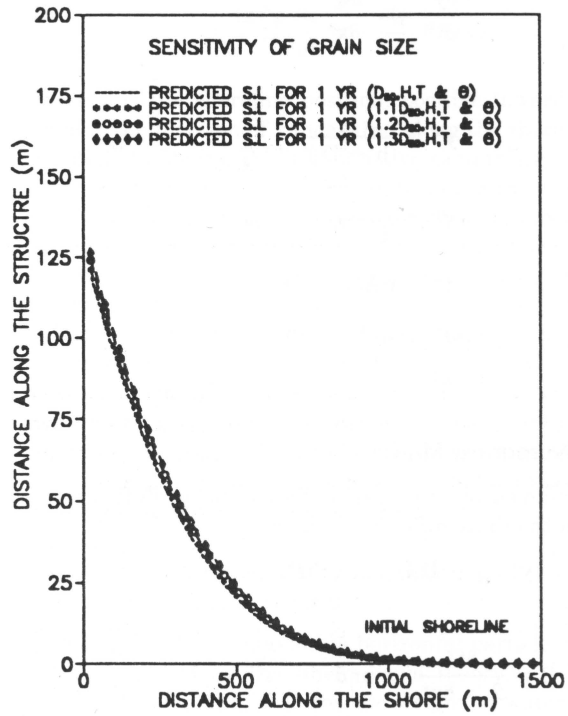

Figure 2d

Figure 2. Sensitivity of wave characteristics and grain size on shoreline evolution.

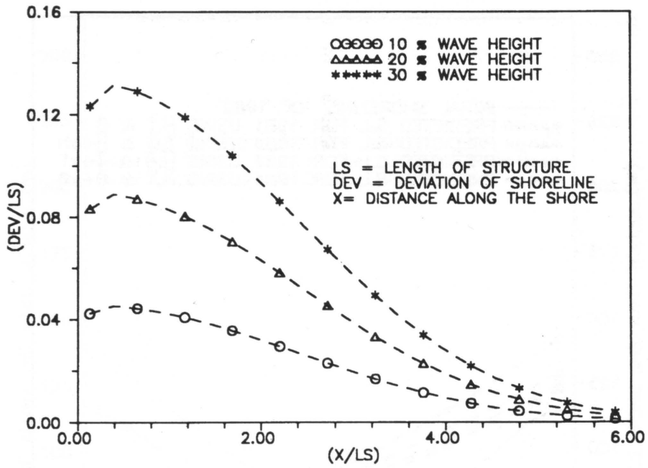

Figure 3a

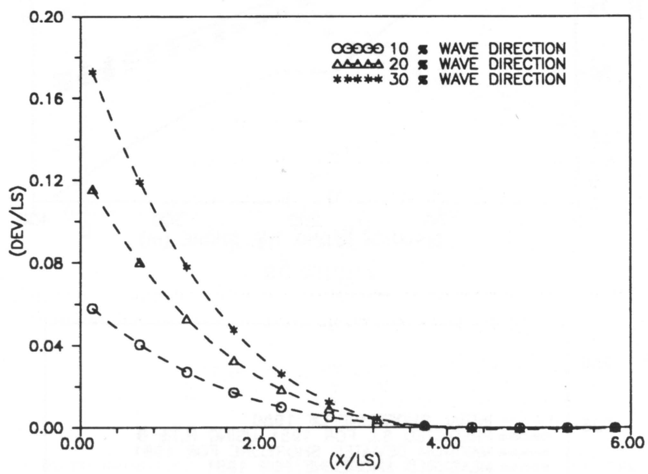

Figure 3. Effect of wave characteristics on shoreline evolution.

Probabilistic Approach

The probabilistic analysis of shoreline is based on the assumption that the input variables are uncorrelated and are represented by Gaussian distribution. In this paper, only the wave characteristics are taken as the variables having both mean and standard deviation and active depth of sediment transport as constant; because, it is calculated based on D_{50} of the grain size distribution. In the present study only three parameters, viz, wave height, wave period and wave direction are considered and hence shoreline simulation has to be repeated four times. The first simulation is done with mean values of the wave height, period and direction. The second simulation is done with the mean wave height and its corresponding standard deviation keeping the values of wave period and its direction at its mean values. The third and fourth simulations repeat the above said procedure with the respective standard deviation of wave period and wave direction with wave height at its mean value. The mean shoreline is taken as the shoreline simulated with mean of wave characteristics. The deviation of shoreline at a point is calculated by the equation given below.

$$\sigma_Y^2 = \sigma_1^2 + \sigma_2^2 + \sigma_3^2 \tag{11}$$

where σ_Y: standard deviation of shoreline resulting from the deviation of H, T and θ. σ_1, σ_2, and σ_3 are std. deviations of shoreline due to 2nd, 3rd and 4th simulations as stated above from that due to first simulation respectively.

RESULTS AND DISCUSSIONS

Sensitivity Analysis

A sensitivity analysis was carried out in order to assess the importance of the different variables dictating the shoreline evolution, viz, wave height, period and direction, and grain size of the sediments D_{50}. The results of the analysis are depicted in Figure 2a, b, c and d. In these figures, the initial shoreline is taken as the abscissa of the plot and shoreline evolution is carried out for one year. In order to carry out the sensitivity analysis, a structure of length 200 m was subjected to waves of height 1.0 m, period of 10 sec approaching the structure at an angle 20° with respect to shore normal. A grain size of 0.2 mm is adopted for the computation, with the depth at the tip of structure and the berm height taken as 5 m and 3 m, respectively. The shoreline advance on the updrift side of the structure is computed. The variation of the shoreline with respect to wave height is then carried out by increasing of wave height to the extent of 10%, 20% and 30% of the initial wave height of 1.0 m and retaining all other variables; corresponding computed shorelines are reported in Figure 2a. The above procedure was repeated by changing wave direction, wave period and grain size in order to assess their significance in the shoreline oscillations and the results are presented in Figure 2b, c and d, respectively. From the results obtained, it is clear that any change in the wave height results in a significant effect on the shoreline evolution; whereas, the change in wave direction has a significant effect on the shoreline evolution nearer to the structure only. The changes in the wave period and grain size are the least influencing parameters on the shoreline response. In order to examine closely the sensitivity of the wave height and wave direction, the deviation of the shoreline, DEV, is normalized with the length of the structure, LS, and has been presented as a function of the distance from the structure, X, normalized with the structure length. These plots are shown in Figure 3a and b. The results indicate that a change in wave height of 10 percent leads to the maximum change in the shoreline to the extent of 50 percent, gradually reduced away from the structure. Similar trends are seen in the case of wave direction; however, its effect is felt for a shorter distance from the structure. The foregoing discussion reveals how important the assigning of wave height and direction is when studying the simulation of shoreline response.

Validation of the Numerical Model with Field Measurements

The numerical model based on the probabilistic approach has been applied to the updrift side of the sand screen of

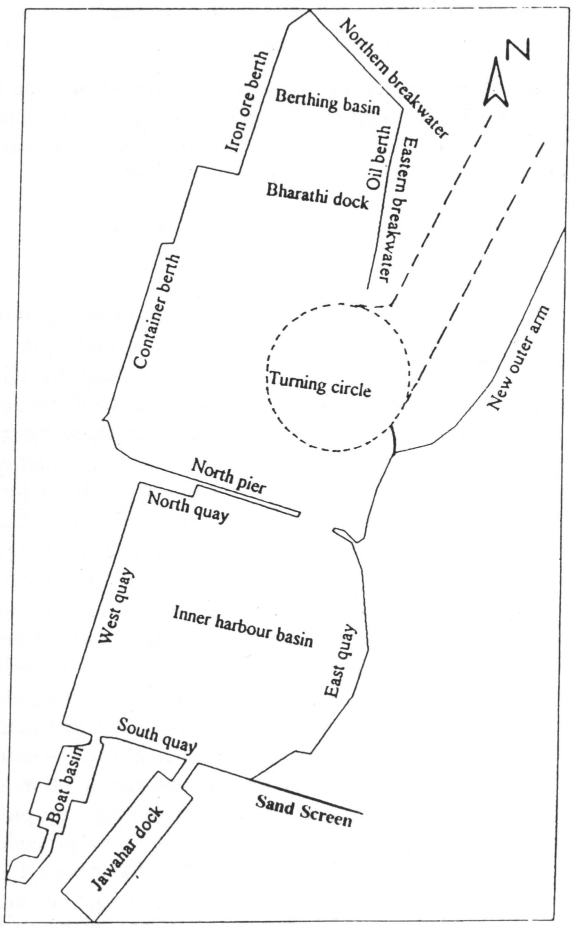

Figure 4. Layout map of Madras Port.

Madras Port, India for its validation with field measurements. The layout map of Madras Port is shown in Figure 4. For this purpose, measured shoreline of 1980 was considered as an initial shoreline and the shoreline response is simulated for one year using the wave characteristics of 1981. The initial shoreline length of about 400.0 m is divided into a grid size of 20.0 m, the mid-value of each grid forms the input to the numerical model. The annual and seasonal mean wave characteristics and corresponding deviation for the study area for the year 1981 are tabulated in Table 1. The statistical values of the wave characteristics have been applied to study their effect on the shoreline simulations. The shoreline evolution has been carried out for (i) average wave characteristics, $\overline{H}$, $\overline{T}$ and $\overline{\theta}$, (ii) $\overline{T}$, $\overline{\theta}$ and $\overline{H} + \sigma H$, (iii) $\overline{H}$, $\overline{\theta}$ and $\overline{T} + \sigma T$ and (iv) $\overline{H}$, $\overline{T}$ and $\overline{\theta} + \sigma\theta$. The resulting shorelines are plotted in Figure 5a. From the figure, it is clear that the shoreline pattern corresponding to the wave direction uncertainty is deviating more from the average shoreline, while that due to the uncertainty of wave period is small. This is due to the fact that the standard deviation of wave direction is larger and the wave direction

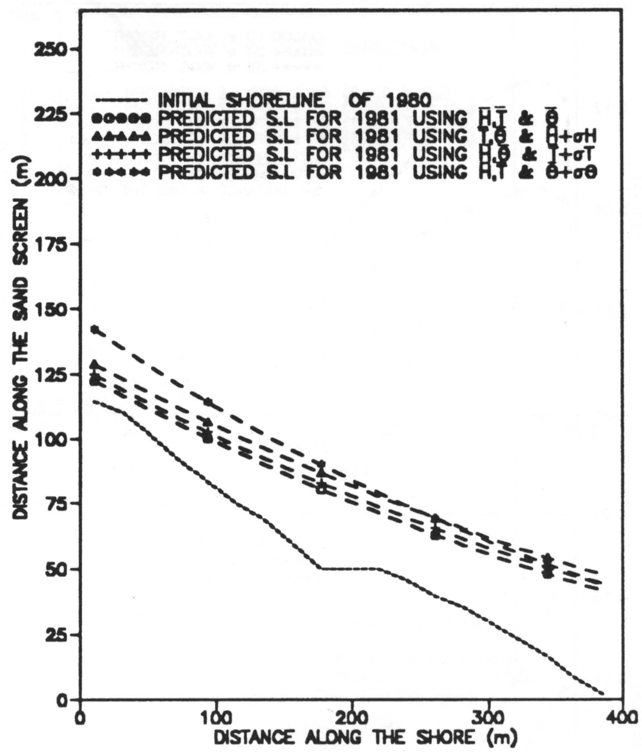

Figure 5a

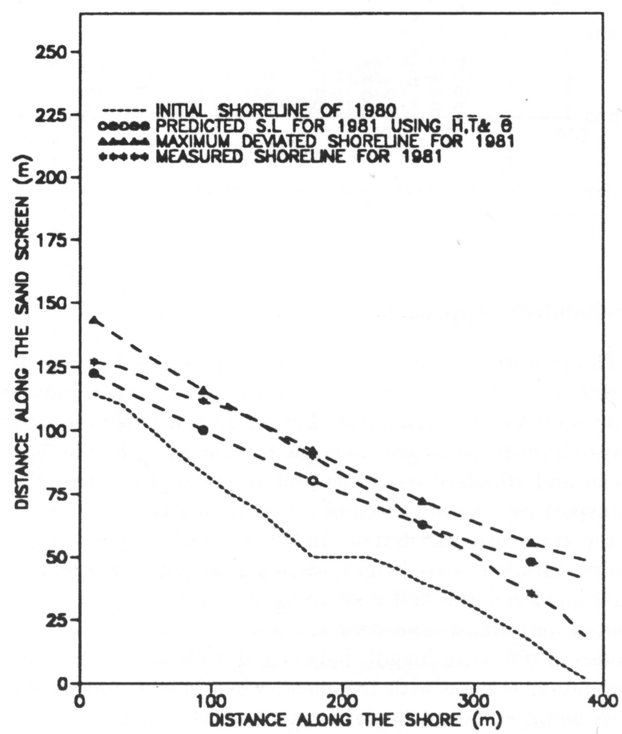

Figure 5b

Figure 5. Simulated shoreline on the updrift of the Madras Port.

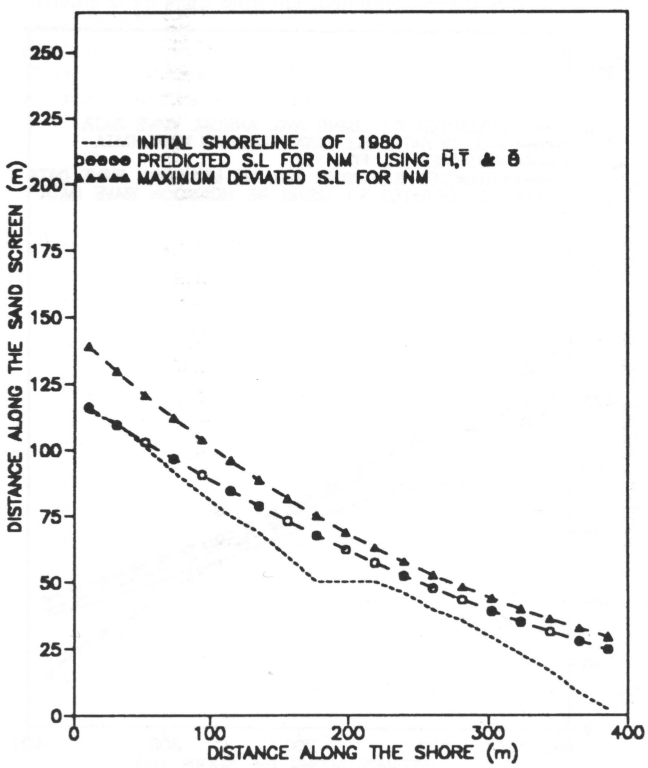

Figure 6. Shoreline predictions for non-monsoon season.

Table 1. *Seasonal variations of wave characteristics.*

Season	$\bar{H}$ (m)	σH (m)	$\bar{T}$ (sec)	σT (sec)	$\bar{\Theta}$ (deg)	$\sigma\theta$ (deg)
Non-monsoon	0.785	0.15	8.75	1.25	70.0	10.0
Southwest Monsoon	0.82	0.26	9.4	1.23	65.5	6.1
Northeast Monsoon	0.89	0.27	8.2	1.5	95.6	8.5
Annual	0.84	0.20	9.3	1.3	74.0	8.2

Note: Directions are measured with respect to shoreline axis

is more sensitive, especially near the structure as observed earlier. Further, the maximum possible deviation of shoreline due to uncertainty in the wave characteristics has been computed using Eq. 11. The maximum deviated shoreline for 1981 along the field measurements are shown in Figure 5b. The above figure clearly indicates that the measured shoreline lies in between the maximum deviated shoreline and the mean shoreline evaluated based on the average wave characteristics, especially nearer to the sand screen. However, it is closer to the maximum deviated shoreline for a distance of up to about 300.0 m from the sand screen.

Application of the Model

The developed numerical model after its validation has been used to predict the seasonal shoreline changes on the updrift of the sand screen of the Madras Port. For this purpose, the shoreline measured in 1980 again forms the initial input and wave characteristics for 1981 and are utilized to derive the shoreline oscillation due to seasonal variations in wave climate.

The east coast of India experiences two monsoons, namely the southwest monsoon (June–September) and the northeast monsoon (October–December). Wave characteristics are affected by these monsoons. During the non-monsoon season (January–May) the wave characteristics are in transition from the northeast monsoon to the southwest monsoon. In regard to the sediment transport, the littoral drift during the non-monsoon and southwest monsoon is towards the north; whereas during the northeast monsoon, littoral drift is towards the south, with the annual net drift moving towards the north. Due to the above phenomena, all the ports on the east coast of India experience deposition or advancement of shoreline on their southern sides, resulting in erosion on the north side of the harbours. The seasonal effects on the simulated shoreline, with a mean shoreline and maximum deviated shoreline, are presented in Figures 6, 7 and 8 for the non-monsoon season, SW monsoon and NE monsoon, respectively. The simulation for the non-monsoon period is carried out using 1980 statistics to represent the initial shoreline, while the average shoreline resulting after non-monsoon and SW monsoon have been used for shoreline

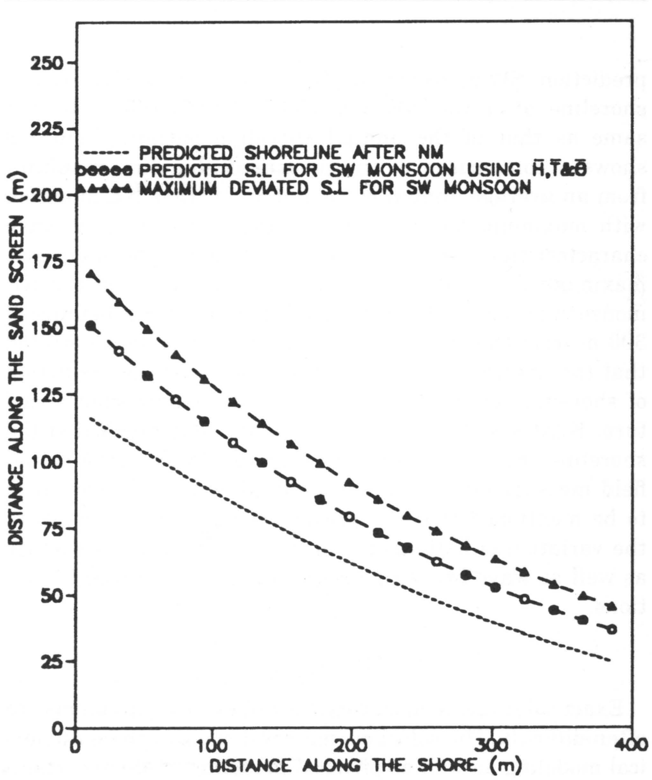

Figure 7. Shoreline prediction for SW monsoon season.

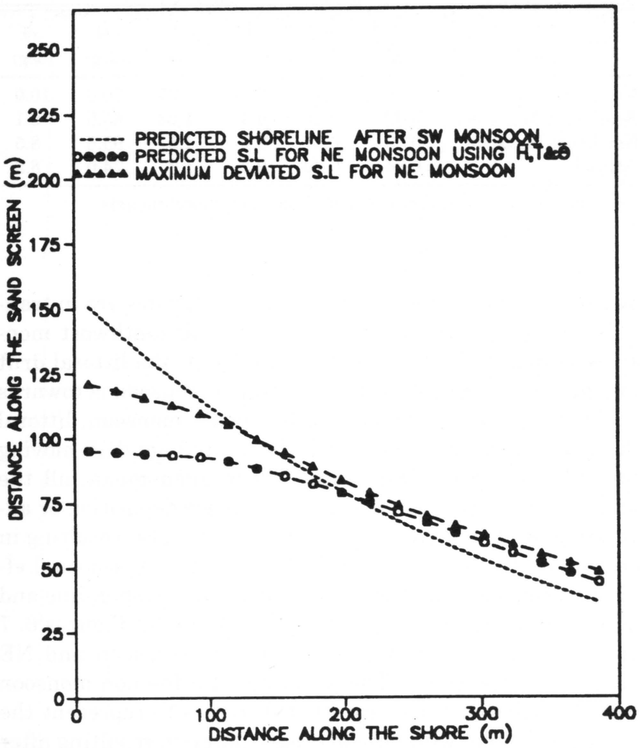

Figure 8. Shoreline predicted for NE monnsoon season.

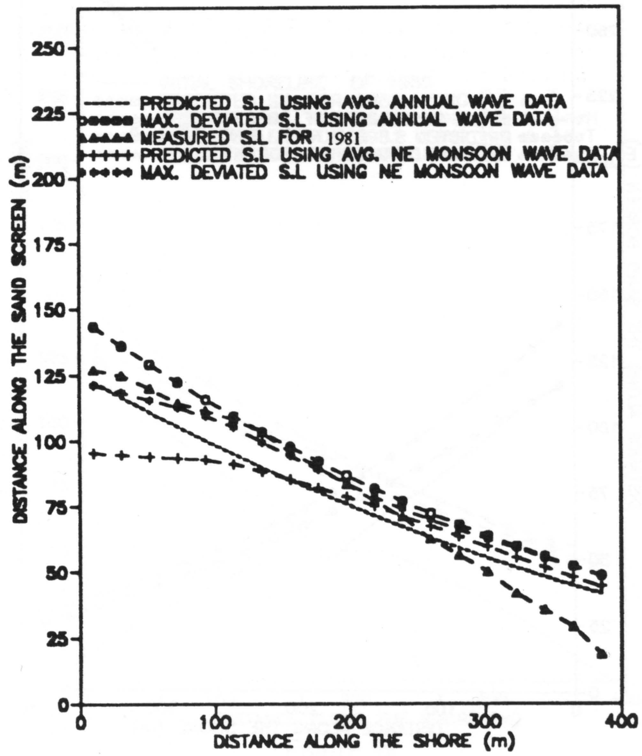

Figure 9. Measured and predicted shorelines versus maximum deviated shorelines.

prediction, SW monsoon and NE monsoon respectively. The shoreline after the NE monsoon must, therefore, be the same as that of the annual shoreline pattern. Figure 9 shows a comparison of the measured shoreline resultant from an average annual wave and an average NE monsoon with maximum deviation due to the uncertainty in wave characteristics. From the above plot, it can be seen that maximum deviated shoreline for both annual and the NE monsoon is close to the measured shoreline for a distance of 300 m from the sand screen. This study has demonstrated that the application of an oneline model, for the prediction of shoreline, changes adjacent to a shore-connected structure. KRAUS and HARIKAI (1983) have also predicated the shoreline changes adjacent to a groin, closer to the actual field measurements of a non-uniform coast. However, it is to be mentioned that the present model does not consider the variation of bathymetry in the vicinity of the structure as well as wave breaking process which are serious limitations.

CONCLUSIONS

Exact solutions to most Coastal Engineering problems are often difficult. The solutions mainly depend both on numerical modeling as well as physical modeling while uncertainties still exist in modeling exactly the nearshore phenomena. The deviations in the shoreline prediction resulting from the uncertainties in the variables describing the wave climate have been brought out clearly in this paper.

In this paper, efforts have been made to develop a numerical model for shoreline oscillation by a probabilistic approach. The developed model is applied to Madras Port. It is hypothesized that the numerical model developed and reported in this paper will be useful considering wave induced sediment transport in the planning of construction of waterfront structures.

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