Seventh International Congress on Advances in Civil Engineering, October11-13, 2006
Yildiz TechnicalUniversity, Istanbul, Turkey

A One-Line Numerical Model for Wind Wave Induced Shoreline Changes

Ayşen Ergin (1) **Işıkhan Güler (2)** **Ahmet Cevdet Yalçıner (1)**

Cüneyt Baykal (1) **Salih Serkan Artagan (1)** **Ilgar Şafak (1)**

(1): Middle East Technical University, Civil Engineering Department
Ocean Engineering Research Center, 06531, Ankara, Turkey
(2) : Yüksel Project Co., Birlik Mah. 9.Cad. No:41, Çankaya, 06610, Ankara, Turkey

Abstract

A numerical model is developed to determine wind wave induced shoreline changes by solving *sand continuity equation* and taking *one line theory* as a base, in existence of I-groins and T-groins, whose dimensions and locations may be given arbitrarily. The model computes the transformation of deep water wave characteristics up to the surf zone and eventually gives the result of shoreline changes with user-friendly visual outputs. Herein, a modification to a readily accepted one-line model as sheltering effect of groins on wave breaking and diffraction is introduced together with representative wave input as annual average wave height. Compatibility of the currently developed tool is tested by a case study and it is shown that the results, obtained from the model, are in good agreement qualitatively with field measurements.

Keywords: Longshore sediment transport, Shoreline change, Groin, Numerical modeling, Wave hindcasting

Introduction

Nearshore processes include several dynamics such as waves, currents, tides and also movement of sediments, which affect use of coastal zones significantly. Coastal sedimentation is one of the main concerns of coastal engineering profession since wave induced sediment transport cause the movement of shoreline and change in the nearshore bathymetry of coastal zones.

Dean (1991) considers coastal sedimentation studies on shoreline and beach profile evolution to be a challenge since even only a partial listing of forces on sediments are complicated and difficult to express with simple examples. Thus, in order to be able to make comparison among several alternatives and to select the best and most practical solution for coastal sedimentation problems, a consistent and user-friendly numerical model, with relatively lower operating costs, is required as well as physical modelling studies and site investigations. Some recent studies, though, recommend re-examination of beach behavior models without making the models more complex by including more variables (Thieler et al., 2000) and criticize deterministic mathematical models to be considered as the one and only method available (Cooper and Pilkey, 2004) . Whereas, shoreline change models, based on one-line theory such as GENESIS(Hanson,1987), are quite promising; if qualitative predictions of wave induced longshore sediment transport and resulting shoreline changes are required particularly. Such a numerical model *has to* provide qualitatively accurate results in the highlight of current studies, *must* be user-friendly and applicable to various boundaries and constraints.

One Line Theory

One-line theory implies that all contour lines have similar shapes and move landward and seaward together up to a limiting offshore depth, depth of closure, as if there were only one contour line (Kamphuis, 2000). Beach profile changes are usually associated with monthly or seasonal periods and variations of rate of longshore sediment transport, caused only by waves and wave induced currents, are deemed as the major agents in assessment of long-term shoreline change (Hanson, 1987). Pelnard-Considere (1956) provides analytical solutions for shoreline changes under various cases, in one of which longshore sediment transport trap by a single impermeable groin on its updrift side under constant wave climate is examined and therefore constitutes the milestone of one-line theory. Major assumption of Pelnard-Considere's analytical solutions of shoreline changes with a given value of sediment transport is that, beach profile remains unchanged and in equilibrium but only moves in parallel to itself (either seaward or shoreward) upto depth of closure, beyond which sediment motion is negligible.

Based on the assumptions that i) beach profile moves parallel to it and is stable in long term scale and ii) sediment movement is "restricted" upto a limiting depth of closure, differential equation defining shoreline movement , called *sand continuity equation*, is derived as follows for the coordinate system given in conventional but illustrative figure, *Figure 1* :

$$\frac{\partial y}{\partial t} = -\frac{1}{D_c + B}\left(\frac{\partial Q}{\partial x} + q_y\right) \qquad (1)$$

where
- y : alongshore distance
- t : time
- D_c : depth of closure
- B : beach *berm* height above still water level
- Q : longshore sediment transport rate
- x : longshore distance
- q_y : sources/sinks along the coast

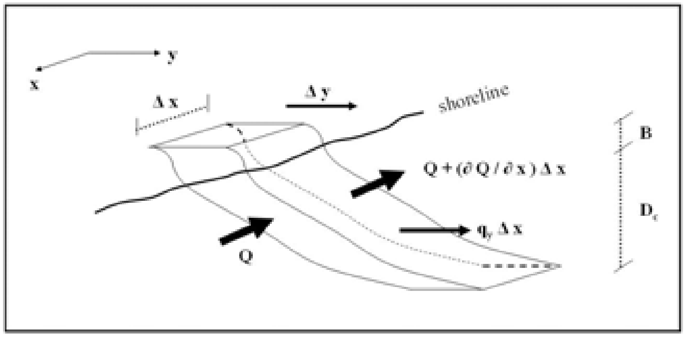


Figure 1 Sketch of sand continuity equation

Kamphuis (1991) conducts three-dimensional (3-D) physical model study with regular and irregular waves to obtain an expression for longshore sediment transport rate, which includes wide range of effective parameters. Consequently, non-dimensionalization of parameters are made together with the discussion of experimental results and the following formula is derived for sediment transport rate:

$$Q = 7.3\ H_{sb}^{2}\ T^{1.5}\ m_b^{0.75}\ D_{50}^{-0.25}\ \sin^{0.6}(2\alpha_{bs}) \qquad (m^3/hr) \qquad (2)$$

where H_{sb} : significant breaker wave height (in m.)
T : significant wave period (in sec.)
m_b : beach slope at breaker location
D_{50} : median grain size diameter (in m.)
α_{bs} : efficient wave breaking angle

In one-line model, interaction between incident waves and gradually changing shoreline is taken into account by *efficient* wave breaking angle, α_{bs} (Hanson and Kraus, 1993) .

Compared to previous approaches to compute longshore sediment transport rate such as CERC formula (Shore Protection Manual, 1984), Kamphuis (1991) formula produces more consistent predictions for both spilling and plunging breaking wave conditions due to inclusion of wave period in the expression, which has significant influence on the breaker type (Wang et al., 2002) and therefore is used in the numerical model.

Explicit Solution of Sand Continuity Equation

In the structure of one-line numerical model, instead of analytical solution, sand continuity equation, (1), is converted to an explicit finite difference scheme depending on longshore distance, y(x,t) and longshore sediment transport rate, Q(x,t). (Hanson and Kraus, 1986) where the expression given below is derived:

$$y_i' = y_i + \frac{\Delta t}{(D_c + B)\Delta x}(Q_i - Q_{i+1} - q_y.\Delta x) \qquad (3)$$

where Δt : time increment
Δx: longshore distance increment

Subscript (i+1) indicates the next increment in alongshore direction and prime (') indicates values at next time step. Dean and Yoo (1992) intelligibly describe explicit solution of shoreline problems using finite difference scheme as *“fixing the shoreline displacements to compute sediment transport rates at the same time step, and fixing sediment transport rates to compute shoreline displacements at the next time”*.

In the explicit scheme, since longshore distance of shoreline at every increment at $t=t_1+\Delta t$ depends on longshore distance of shoreline and longshore sediment transport rate at $t=t_1$, stability comes out to be an important parameter and is checked by the following expression:

$$\frac{Q}{\alpha_b(D_c+B)}\frac{\Delta t}{\Delta x^2}\leq\frac{1}{2} \tag{4}$$

Numerical Model Fundamentals

Wave Transformation

Longshore sediment transport and resulting shoreline changes under wave motion depend on wave breaking height (H_b) and wave breaking angle (α_b). Therefore, deep water significant wave parameters should be transformed into breaking conditions including the effects of refraction, shoaling and diffraction In the numerical model, the following method, demonstrated in Coastal Engineering Manual(2003), is used to compute the breaking parameters because of its applicability to get breaking parameters directly by inserting deep water wave characteristics:

$$H_b=(H_0)^{4/5}(C_{g0}\cos(\alpha_0))^{2/5}\left[\frac{g}{\gamma_b}-\frac{H_b g^2\sin^2(\alpha_0)}{\gamma_b^2 C_0^2}\right]^{-1/5} \tag{5}$$

where H_b: Wave breaking height
H_0: Deep water significant wave height
C_{g0}: Deep water group velocity
α_0:Deep water approach angle
g: gravitational acceleration
γ_b : breaker index (H_b/d_b)
C_{g0}: Deep water group velocity

Wave – Structure Interaction and Sediment Motion

One of the major causes of longshore sediment transport gradients, which result with shoreline changes, is diffracting effect of coastal stabilization structures in their sheltered zones. Breaking wave heights and resulting water level changes outside sheltered zone (H_{b1} & setup$_1$, H_{b3} & setup$_3$) are greater than those in sheltered zone (H_{b2} & setup$_2$) . These variations at water surface accelerate longshore currents and result with erosion both at updrift and downdrift sides *(Figure 2)* .

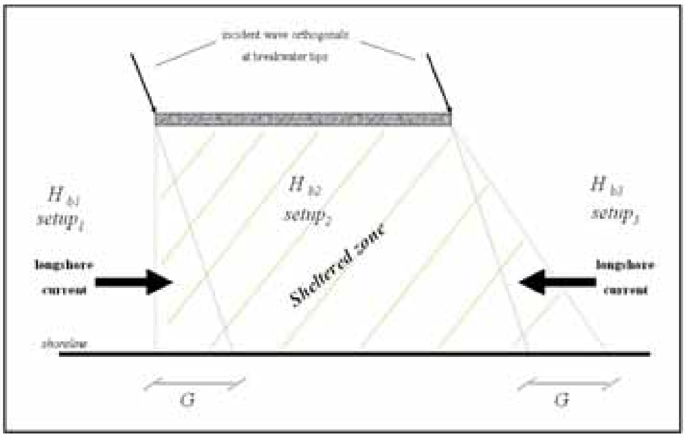


Figure 2 Sheltering effect of an offshore breakwater.

In the numerical model, wave breaking parameters such as wave breaking height (H_b), wave breaking depth (d_b) and wave breaking angle (α_b) are initially calculated within the modeled region. Afterwards, these parameters are modified to account for changes in wave patterns from each diffraction source at the breaking depth d_b, as presented in (Dabees, 2000):

$$H_{bd} = K_d H_b \tag{6}$$

where H_{bd} : modified wave breaking height
K_d : diffraction coefficient

The effects of structures on wave diffraction and computation of resulting wave breaking heights and wave breaking angles in sheltered zones of structures require significant attention. In order to comprehend shoreline changes under complex structure distributions, in the numerical model, diffraction coefficients (K_d) in the sheltered zones of the structures blocking out a part of incoming wave energy, are computed by relationships of Kamphuis (2000) based on the diffraction method for random seas (Goda et al., 1978). Kamphuis relates the diffraction coefficient to the angle θ between the principal wave direction and the point of interest in the shadow zone *(Figure 3)* as follows:

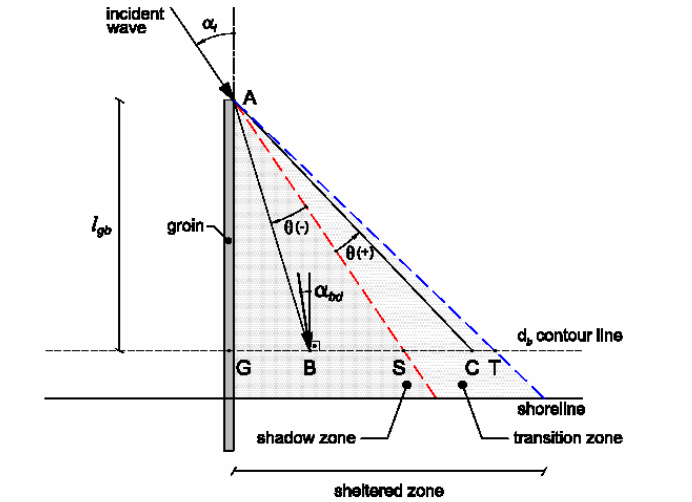


Figure 3 Definition sketch of wave diffraction near a groin.

$$K_d = 0.71-0.0093\,\theta+0.000025\,\theta^2 \quad \text{for } 0 \geq \theta > -90 \tag{7}$$

The trend of K_d coefficient in the sheltered zone is linearly extended to compute K_d coefficient in the transition zone, accepting the fact that diffraction coefficient should be equal to 1 beyond the sheltered zone. The modified breaking wave height H_{bd} is computed by (6). Wave breaking angle in the sheltered zone of groins is calculated from the equation given below:

$$\alpha_{bd} = \alpha_b K_d^{0.375} \tag{8}$$

where α_{bd} : diffracted wave angle
α_b : undiffracted breaking angle

Inside the shadow zone, however, a further decrease in the breaking angle is taken into account as:

$$\alpha_{bd} = \alpha_b K_d^{0.375}\left[\frac{2GB}{l_{g_b}\left(\tan\alpha_i + \tan(0.88\alpha_b)\right)}\right] \quad \text{if} \quad \frac{GB}{l_{gb}} < \frac{1}{2}\left\{\tan(\alpha_i) + \tan(0.88\alpha_b)\right\} \tag{9}$$

where l_{gb} : groin length from the seaward tip of groin to the breaking location
α_i : incident wave angle at the seaward tip of the groin
GB: distance away from the groin

The model developed specifically covers the sheltered region behind the coastal structures where smaller waves of higher occurrence probability break causing effective sediment transport.

A Case Study

Kızılırmak River discharges into Black Sea where it forms *Bafra* Delta. *(Figure 4)* Resulting from the construction of flow regulation structures on Kızılırmak River, coastal erosion, upto 30 m. per year, takes place in this region mostly due to the reduction of sediment budget. A shore protection system with 2 Y-shaped groins and 1 I-shaped groin is designed and constructed as being shown in *Figure 5* (Kökpınar et al., 2005). Measured field data before and after construction of these coastal structures between the years 1999-2003 are obtained from State Hydraulic Works (DSI) to be used in this case study.

Figure 4 Location and plan view of Bafra Delta.

In this study an application of one-line model is carried out to compare with the measured field data of the region with existing groin system.

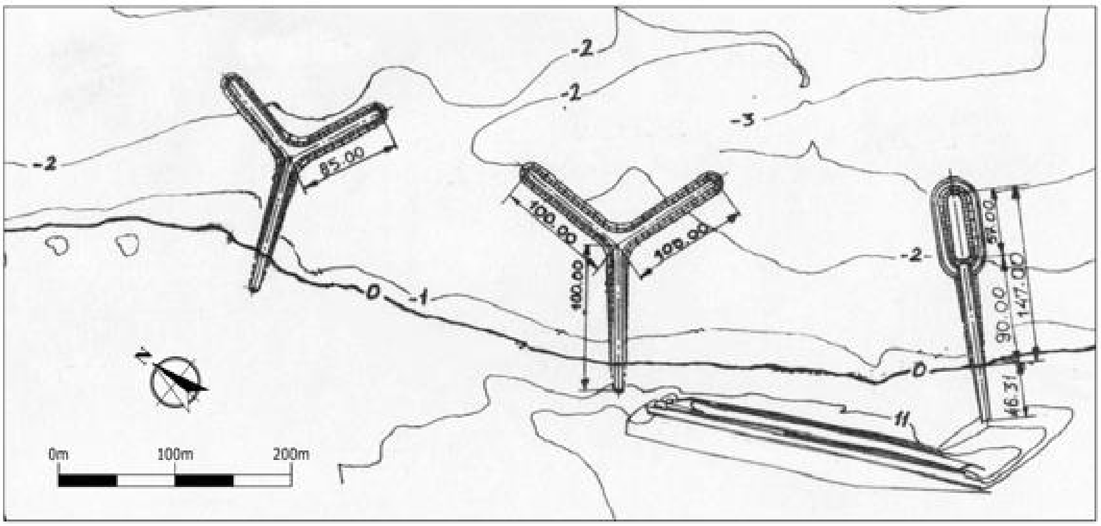


Figure 5 Final layout.

Model Wave Data

In one-line models, wave data input is the most important parameter which affects movement of sand along the shoreline. In this study, effects of smaller but more frequent waves are considered to be more appropriate to use rather than higher waves with less frequency. To check the validity of this assumption, in this respect, a concept of average wave height based on a probabilistic approach is developed. Thus, for each direction separately, average deep water significant wave height (H_{so}) is computed as (Güler ,1997 ; Güler et al.,1998) :

$$H_{so} = \frac{\sum (P_i . H_i)}{\sum P_i} \tag{10}$$

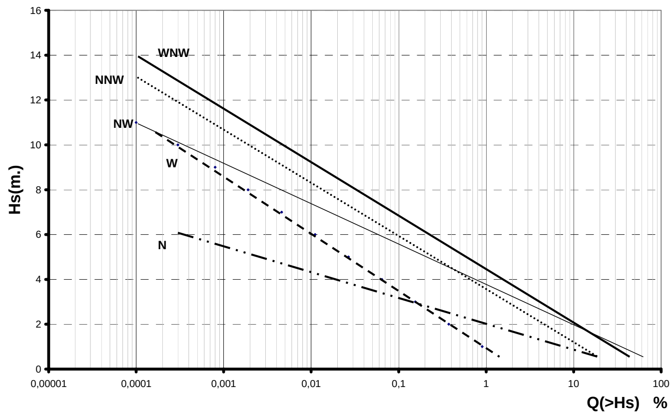
where H_i : wave height
P_i : occurrence probability of wave with height H_i

Occurrence probability (P_i) of wave with height H_i is computed by using the corresponding frequencies within the given range as follows:

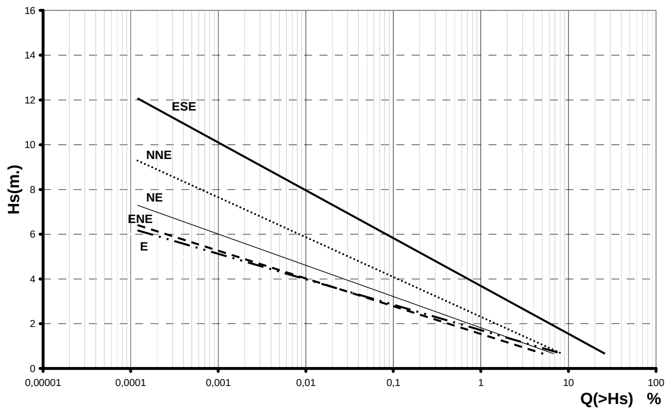
$$P_i = Q(H_i - k) - Q(H_i + k) \tag{11}$$

where Q : exceedence probability
k : an assigned range to compute occurrence probability

Wave prediction studies of Bafra region are carried out by using 20 year wind data, measured at the closest meteorological station at Sinop *(Figure 6).* As a result of long term wave statistics, probability distribution of deep water significant wave heights are given for wave directions WNW, NW,NNW, N in *Figure 6(a)* and for wave directions *NNE, NE,ENE, E, ESE in Figure 6(b)* .



(a)



(b)

Figure 6 Probability distribution of deep water significant wave heights.

Average deep water wave steepness in Bafra region is calculated as 0.042 from extreme wave statistics, which is consistent with the value given in Ergin and Özhan (1986). In *Table 1,* average wave heights, corresponding periods and annual frequencies from all directions are presented.

Table 1 Average wave heights and corresponding periods (annual) from all directions.

	H(m.)	T(sec.)	f(hrs.)
WNW	1.53	4.83	1365
NW	1.26	4.40	1798
NNW	1.53	4.83	507
N	0.99	3.89	562
NNE	1.24	4.35	185
NE	1.07	4.05	134
ENE	1.01	3.93	114
E	0.98	3.87	151
ESE	1.37	4.57	746

In the application of the model, Y shaped groins, which are site specific structures, are introduced as T-shaped groins and also it is assumed that no source or sink exists in the application of numerical modeling for the case in Bafra region.

The results of the numerical simulation are presented in *Figure 7* together with the initial (April 1999) and final (January 2003) field measurements.

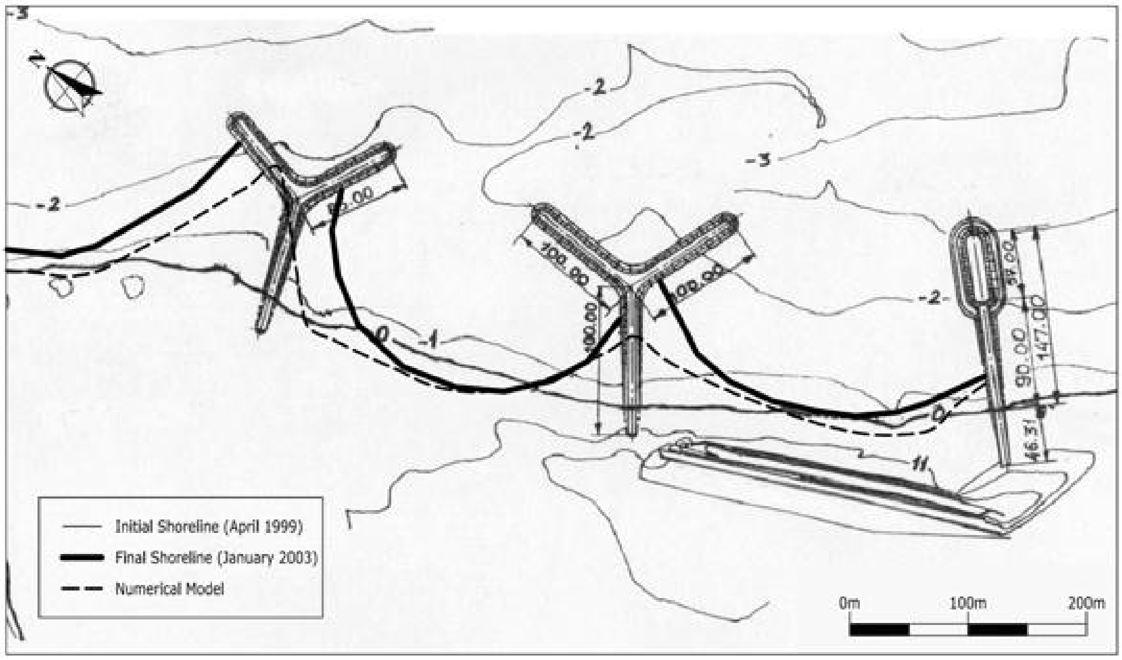


Figure 7 Comparison of site measurements and results of numerical simulations.

As it is seen from *Figure 7*, the model results reflect the trends of shoreline changes qualitatively, both at the updrift (western) and downdrift(eastern) sides of the groins.

Model results are also in good agreement quantitatively with the final field measurements, especially at western sides (updrift) of second and third groins. However, at eastern sides of first and second groins, model results are in agreement only qualitatively. The difference between model results and measurements can be attributed to the slight behaviour difference between Y-groins (field case) and T-groins (numerical model case) and measurement errors in the field together with numerical model assumptions.

Depending on these results, it can be concluded that, using annual average wave heights in the numerical model could be accepted to give qualitatively comparable results with the field measurements. Using annual average wave heights in model puts the emphasis on smaller wave heights with higher frequencies rather than larger wave heights with smaller frequencies. Such an approach pictures well the complex nature of sheltering effect of coastal structures on wave breaking and diffraction for modeling the actual phenomena.

Conclusion

Appending assumptions and limitations of one line theory to the uncertainties in coastal sedimentation studies coerce the evaluation of complex nature of wave breaking and diffraction especially behind the coastal structures. Therefore, in the development of the model, special emphasis is given on to wave breaking and diffraction within the sheltered zones of coastal structures e.g. I-groins and T-groins. A simplified approach for the computation of the modified wave breaking heights within the sheltered zone of these structures is demonstrated.

In the application of the model, concept of annual average wave height is presented instead of actual wave time series, which improve the applicability of the model. Application of the numerical model in a case study at Bafra Delta, Black Sea, with annual average wave data, proved that model results are qualitatively consistent with the field measurements. In general, it is well known that model results are dependent on wave data input. Thus, deriving a statistical panorama due to the input manner of wave data into the model may be an exciting and challenging discussion in further studies.

Acknowledgment

The authors acknowledge Dr. Mehmet Ali Kökpınar and Dr.Yakup Darama from State Hydraulic Works, for providing field data from Bafra Delta to be used in the case study.

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