Physical and Numerical Modeling of Shoreline Evaluation of the Kızılrmak River Mouth, Turkey

Mehmet Ali Kökpinar', Yakup Darama', and İskhan Güler'i

'Stae Hydraulic Works Technical Research and Quality Control Department Hydraulics Laboratory 06100 Ankara, Turkey mehmetal@stai.gov.tr

Yuksel Project Co. Bizilik mah. S. Cad. No:41, Çankaya, 06510 Ankara, Turkey gulerm@yukselproje.com.tr

ABSTRACT

KOÇPINAR, M.A.; DARAMA, Y., and GÜLER, İ., 2006. Physical and numerical modeling of shoreline evaluation of the Kızılrmak River mouth, Turkey. Journal of Coastal Research, 21(2), 000-000. West Palm Beach (Florida), ISSN 0749-0208.

The sediment budget of the Kızılrmak River has been disturbed during the last decade because of the flow regulation structures constructed on the river. This disruption has led to coastal erosion at the river mouth and its environs. With the effect of erosion within this period, the Black Sea shoreline has eroded approximately 1.0 km toward the Bafra Plain. In this study, the coastal erosion in the area was examined using physical and mathematical models. A shore protection structure system based on the results of the physical model tests was developed and implemented at the site. The one-line model was also applied for this part of the shoreline to study the problem mathematically. Analysis of the numerical simulation showed similar trends to the results of the physical model and field observation. The one-line model successfully represented the behavior of protective structures on the evaluation of the Kızılrmak River Mouth. One year after the completion of the protection structures, observations in the field showed that the erosion was completely controlled, and certain parts of the shoreline at this reach had advanced approximately 50 m toward the Black Sea. These protection structures have altered the longshore sediment transport along the shoreline of the Bafra Plain nearby the Kızılrmak River mouth. Therefore, the erosion at the adjacent shoreline of this reach has been accelerated at the expected level.

ADDITIONAL INDEX WORDS: Coastal erosion, one-line model.

INTRODUCTION

Coastal erosion has long been a topic of concern among coastal residents and the engineers who are responsible for maintaining the beaches, inlets, and harbors. There is a plethora of research on shoreline erosion, covering almost every coast of the world. Some reports have dealt with long-term erosion trends, whereas others have examined rapid changes resulting from storms (Kana, 1977). Various flow regulation structures constructed on the rivers have an adverse impact on the sediment carried by the respective rivers if no measures are taken. If the rivers discharging to the sea are controlled by flow regulation structures, deltas are under the risk of significant adverse impact. One of the most important problems is coastal erosion. This problem has emerged at the Bafra Plain delta, which is located in northern Turkey where the Kızılrmak River discharges into the Black Sea (Figure 1).

The Kızılrmak River, which rises in the Eastern Anatolian Mountains, flows in a northwestern direction and discharges into the Black Sea by forming a conic alluvial delta (Figure 1). It is the longest river in Turkey, at 1,955 km in length, and drains a basin of 74,815 km². Annually, average volume of the river flow is 6.48 km³; this figure represents 3.5% of the total water resources of Turkey. Minimal flows usually predominate between May and December. Annually, average precipitation and temperature in the Kızılrmak River watershed are 600–700 mm and 14°C, respectively. Total sediment transport through the river was determined from the data collected between years 1962 and 1973 to be 3.86 × 10⁶ m³/y. Suspended sediment consisted of 25% sand and 74% silt and clay in suspension, and its yearly average concentration was determined as 6600 ppm (Altımkaya Dam Feasibility Report, 1975).

The Bafra alluvial plain, formed by the deposition of sediment transported by the Kızılrmak River for hundreds of years, has a conical shape toward the Black Sea at the northern part of Turkey. Because of its topographical shape, the shoreline of this plain has been under the effects of Black Sea waves. The northern tip of the plain especially has been under the severe influence of complex hydrodynamic currents produced by the waves. Even though natural lakes and large swampy areas in the eastern portion of the plain are inhabited by wildlife, the general geological structure of the plain is comprised of alluvial sand. The regulation of the Kızılrmak River flow for hydroelectric power production, irrigation, and water supply started more than a decade ago, and effective measures were taken to prevent surface erosion.

DOI:10.2112/04-0178.1 received 2 March 2004; accepted in revision 23 November 2004.
of the watershed of the Kızılırmak River. However, the sediment budget of the river has been altered, and both the western and the eastern parts of the plain have been eroding because of the effects of Black Sea waves. The sediment transported by the Kızılırmak River was 23.1 million tonnes before the construction of the first regulatory structure in 1960 (Hay, 1994; Savran and Oray, 2002). The sediment transported by the river dropped to 18 million tonnes until 1988, when the full regulation of river flow started. Thereafter, the amount of sediment transported by the river dropped dramatically, to 0.46 million tonnes. Other striking information, acquired from the Regional Directorate of State Hydraulic Works (DSI) and from local residents, indicated that an approximately 1-km-wide band of shoreline has eroded since 1988.

The objective of this study is to investigate and prevent the coastal erosion at the Kızılırmak River Mouth. The solution developed and implemented in 2000 to prevent the erosion is based on the findings of the physical and mathematical model studies conducted at the Hydraulic Model Laboratory of DSI.
in Ankara, Turkey. The change in the shoreline has been observed periodically since the completion of the construction in order to determine the effectiveness of the shore-protection structures. Furthermore, the results obtained from the physical and numerical models were compared with the numerical model study of Savran and Otay (2002) performed for the Kızılarık River mouth. The method of Haq and Evans (1989) developed for parabolic bay shapes was also applied to the problem studied, and the results were compared with the field observations. The data related to wave climate, bathymetry, and wind characteristics of the region that were necessary for the calibration and the development of physical and mathematical models were obtained from the Regional Directorate of DSI, Hat (1994), and Ergin, Buyukgunok, and Orhan (1995).

**WAVE CLIMATE AND MARINE DATA**

Because there was no regular basis of measurements for wind producing waves in this area, the Sinop Meteorological Station (see Figure 1) long-term wind data, covering the period between 1966 and 1985, were used in the analysis of wind data. Analysis of the data in terms of wind velocity and direction showed that the prevailing wind in this region occurred in the WNW-NW-NNW and ESE directions. Figure 2 shows the polar histogram of the wind directions in the region. Analysis of the 27-year period of wind records obtained from the Sinop Meteorological Station indicated that the average hourly wind speed was 6.14 m/s, and that the most prevailing direction was WNW. Wind occurring almost parallel to the shoreline, as shown in Figure 2, produced the most damaging process of erosion.

Wave parameters of the region, such as wave heights, periods, lengths, and prevailing directions, were obtained and analyzed before the construction of the physical model. The Sinop Meteorological Station data between 1966 and 1985 for hourly wind speed and directions were obtained, and these data were used to estimate the storms creating the waves. Long-term wave statistics for the 20-year period were estimated by using these storms in the method described by the Shore Protection Manual (SPM, 1984), and the probability distribution of deep-water significant wave heights shown in Figure 3. Statistical values of these long-term data for the Black Sea waves covered all directions. Even though waves approaching from all directions and magnitudes are responsible for the erosion and accretion of sand along the shoreline, it was more convenient to consider average significant deepwater wave heights approaching the coast from the most frequent and effective direction. Thus, these wave heights were computed by using long-term records including the average duration of occurrence within one year (Table 1). This statistical analysis showed that waves coming from the WNW-NW-NNW directions are responsible for the erosion of the Bafra shoreline at the eastern part of the Kızılarık River mouth.

**COASTAL MODELS OF THE KIZILRMAK RIVER MOUTH**

The analysis of data obtained from the measurements and observations conducted to determine the volume of erosion at the Kızılarık–Black Sea junction of the Bafra Plain constituted a basis for the development of the physical and mathematical models. The physical model was developed at the Hydraulic Model Laboratory of DSI, and experimental studies were conducted for the development of the necessary projects to prevent erosion in the region. In addition, a numerical

---

**Table 1. Wave direction, average wave heights, periods and duration of occurrence within one year.**

<table>
<thead>
<tr>
<th>Wave Direction</th>
<th>Wave Height $H$ (m)</th>
<th>Wave Period $T$ (s)</th>
<th>Average Duration of Occurrence in 1 yr (h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>1.34</td>
<td>4.5</td>
<td>173</td>
</tr>
<tr>
<td>NNE</td>
<td>1.86</td>
<td>5.5</td>
<td>49</td>
</tr>
<tr>
<td>NE</td>
<td>1.29</td>
<td>4.6</td>
<td>51</td>
</tr>
<tr>
<td>NE</td>
<td>1.43</td>
<td>4.7</td>
<td>31</td>
</tr>
<tr>
<td>E</td>
<td>0.82</td>
<td>3.5</td>
<td>236</td>
</tr>
<tr>
<td>ESE</td>
<td>0.87</td>
<td>3.6</td>
<td>258</td>
</tr>
<tr>
<td>W</td>
<td>2.37</td>
<td>5.0</td>
<td>8</td>
</tr>
<tr>
<td>WNW</td>
<td>1.98</td>
<td>5.5</td>
<td>909</td>
</tr>
<tr>
<td>NW</td>
<td>1.93</td>
<td>5.4</td>
<td>477</td>
</tr>
<tr>
<td>NNW</td>
<td>2.20</td>
<td>5.8</td>
<td>213</td>
</tr>
</tbody>
</table>

---

**Table 2. Effective physical and wave parameters and their scales in the model.**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Dimension</th>
<th>Scale Relationship</th>
<th>Model Scale</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical length</td>
<td>$L$</td>
<td>$L$</td>
<td>1:50</td>
</tr>
<tr>
<td>Horizontal length</td>
<td>$L$</td>
<td>$L$</td>
<td>1:100</td>
</tr>
<tr>
<td>Wave period</td>
<td>$T$</td>
<td>$T$</td>
<td>1:7</td>
</tr>
<tr>
<td>Wave length</td>
<td>$L$</td>
<td>$L$</td>
<td>1:50</td>
</tr>
<tr>
<td>Diameter of sand particle</td>
<td>$L$</td>
<td>$L_{ap} = (D_{ap}L_{ap})^{1/3}$</td>
<td>1</td>
</tr>
<tr>
<td>Density of sand</td>
<td>$M/L^2$</td>
<td>$L$</td>
<td>1</td>
</tr>
</tbody>
</table>
study was done by using the one-line model (HANSON and KRAUS, 1986) to simulate the long-term effects of the shore protection system.

The Physical Model

The bathymetric map of the sea along the cross sections at the outlet of the Kizilirmak River mouth and the topographic map of the region were necessary in order to achieve a complete geometric similarity between the physical model and the prototype. The topography of 1.5 km covering 300 m along the west and 1200 m along the east shores of the river mouth and the bathymetry of the region up to a depth of ~5 m were used for the physical model.

Similarity Law and Selection of the Model Scale

Previous studies showed that distorted physical modeling technique should be used for qualitatively modeling of estuaries with mobile beds, in which littoral drift is the dominating feature (DALRYMPLE, 1955; YALIN, 1971). In general, there are four basic parameters used to determine the model scale of short (wind) waves over a movable bed. These parameters are horizontal length ($L_w$), vertical length ($L_v$), median diameter of sand particle ($D_{50}$), and the specific weight of the sediment ($\gamma$) (YALIN, 1971). The scales of the physical model are the ratios between those parameters obtained from the model and the prototype and can be given by the following equation:

$$\lambda_{\text{parameter}} = \frac{\text{(parameter)$_{\text{model}}$}}{\text{(parameter)$_{\text{prototype}}$}}$$

Many investigators discussed the uncertainties concerned about using bed material lighter than sand for representing bottom sediment in the model experiments (GÜLER, 1985). In order to eliminate the uncertainties caused by using lighter material, sand was used for bottom material in the model experiments. The sand used in the experiments was brought from the Bafra coast, where the erosion was occurring. The physical analysis of the sediment revealed that the median diameter of the sand particle $D_{50} = 0.33$ mm, and the unit weight of the sand $\gamma_s = 2690$ kg/m$^3$. Because the bed material used in the model is the same as the bed material of the modeled coastal region, scales of the sediment particle size $\lambda_s$ and the specific weight of the sediment $\lambda_{\gamma_s}$ are equal to unity. The horizontal length scale ($\lambda_L$) can be found by taking the ratio of the model area size in terms of its horizontal dimensions with that of the size of the modeled region.
in the field. After the determination of the model horizontal length scale, vertical length scale ($\lambda_z$) can be determined by the relationship defined by Meraut (1970) as

$$\lambda_z = \lambda_{hr} \lambda_{uv}$$  \hspace{1cm} (2)

in which $\lambda$ is the scale of the fall velocity of the sediment particle in liquid. If the Reynolds number of the particle fall velocity is less than 0.1, and assuming that the kinematic viscosity of the fluid in the model and in the prototype are the same, then the scale of the fall velocity described by Stokes relationship can be written as

$$\lambda_{uv} = \lambda_{hr} \lambda_{uv,10}$$  \hspace{1cm} (3)

For high Reynolds numbers, the scale of particle fall velocity can be derived from Newton's relationship,

$$\lambda_{uv} = \lambda_{hr} \lambda_{uv,10}^{1/2}$$  \hspace{1cm} (4)

This equation is not explicit as is Equation (3), because $\lambda_{uv}$ must be determined as a function of $\lambda_{hr}$ from a Reynolds chart by trial and error. Because the physical features of the sand used in the model studies are the same as the physical features of the sand in the shoreline of Bafra, $\lambda_{hr}$ can be accepted as 1.0. Thus Equation (4) for high values of Reynolds numbers simplifies,

$$\lambda_{uv} \approx \lambda_{hr} \lambda_{uv,10}^{1/2}$$  \hspace{1cm} (5)

The vertical scale coefficient of the model for low values of Reynolds numbers can be derived by using Equation (2) as

$$\lambda_z = \lambda_{hr} \lambda_{uv,10} \lambda_{uv}$$  \hspace{1cm} (6)

and for high values of Reynolds numbers,

$$\lambda_z = \lambda_{hr}^{1/2} \lambda_{uv,10} \lambda_{uv}$$  \hspace{1cm} (7)

Because $\lambda = \lambda_{hr,10} = 1$ in the physical model, by choosing the horizontal length scale coefficient $\lambda_{hr} = 1/100$, the vertical scale coefficient could be determined using Equations (6) or (7) as $\lambda = 1/121.5$. Choosing these values of $\lambda_{hr}$, the distortion coefficient $\Omega$ was computed as 4.65. For practical purposes, if the distortion coefficient was taken as $\Omega = 5$, then the vertical scale coefficient could be calculated as 1/20.

The theoretical analysis given above clearly indicated that the physical model of the Bafra Plain should be distorted. Because of the limitation of the size of the model area, and the criteria for the distortion limit obtained from the theoretical analysis, only the 1500-m-long shoreline of the Bafra Plain where the erosion was occurring severely was considered.

Preliminary experimental studies of the physical model when $\lambda_{hr} = 1/100$ and $\lambda_{uv} = 1/20$ showed that physical similarity between the model and the prototype was not adequate for the reasons given below:

1. A high distortion coefficient produced a steeper slope for the sea bottom in the model than in the prototype case, and this affected the characteristics of waves.
2. Because the water depth in this section of the coastal zone of the Bafra Plain is so shallow, it was not easy to obtain physical similarity.
3. Sediment transport in the model was highly influenced by the distortion coefficient.

The preliminary experiments confirmed that the scientific approaches alone were not adequate for achieving a complete similarity between the physical model and the prototype for distorted coastal models with mobile beds (Guler, 1985; Hughes, 1983; Noda, 1978; Satou, 1984; Yalin, 1971). Therefore, the distorted coastal modeling with mobile bed appears still to be an "art" rather than a "science."

In light of this information, the value of the distortion coefficient $\Omega$ was reduced to 2. In case, keeping the horizontal scale $\lambda_{hr} = 1/100$, the vertical scale coefficient was determined as $\lambda = 1/60$. With these scale coefficients, the bathymetry in the model approached the bathymetry in the prototype, and thus sediment movement in the model became comparable with the sediment movement in the prototype.

Table 2 gives the scales of physical and wave parameters in the model as a function of horizontal and vertical scales.

Although waves in all directions within a one-year period influence the coastal erosion in this region, considering the directional distribution of waves in the physical model would be the best approach to study this phenomenon. However, the existing wave generator in the laboratory works unidirectionally, and therefore the effects of the distribution of waves in terms of their direction could not be investigated.
Figure 5. Photograph of the shoreline changes after the physical model studies.

Figure 6. Map of the shoreline changes after the physical model studies, showing the positions of the shoreline before groins (solid line) and after groins (dashed line).

Journal of Coastal Research, Vol. 21, No. 0, 0000
The Physical Setup

The physical setup shown in Figure 4 has a wave channel 8 m wide and 25 m long that contains a regular wave generator that has a steel pallet with an adjustable arm. Before each experiment, the deep-water wave height that was measured in front of the wave pallet in the model, \( H_{dm} \), was adjusted with the adjustable arm so that the tilting angle of the pallet could produce the desired wave height. The wave period, \( T_w \), was determined by adjusting the number of revolutions of the rotary motor. A screen was installed in front of the steel pallet in order to eliminate the disturbances caused by the vibration of the pallet in water. Furthermore, pebbles and stones were placed at both ends of the model basin to minimize the effect of the wave reflection. The wave heights in the model were measured by using a wave gauge probe functioning according to the electrical voltage principle and an amplifier connected thereto. The wave period was determined from the sinusoidal curve with a time axis generated at the printer with the signal input from the probe. Change in the shoreline was determined by placing different colored lines along the shoreline before and after the experiment; photos were taken to show the difference in the shoreline. The light-colored line in the photos represents the original condition of the shoreline, and the dark-colored line represents shoreline after the experiment. As mentioned before, the most effective wave directions were in NW-NNW and WNW; thus, constructing the physical model with a W wave direction was adequate to achieve similarity in direction. Several alternative layouts of groin shapes Y, T, and I were tested in the model to stabilize the east bank of the Kumurmak River mouth. The final layout, as shown in Figure 4, is constructed according to the model with two Y shapes and one I.

Figure 7. Shoreline changes obtained from the numerical model after 3546 h by using average wave heights (\( \Delta t = 3 \) h and \( \Delta t = 6 \) h).
shape, which was found to be the most effective to stabilize the shoreline.

**Experimental Study**

Regular waves of \( H_w = 5 \text{ cm} \) and \( T_w = 1.5 \text{ seconds} \) were produced in the model for a continuous test duration of six hours. The corresponding prototype values of these wave parameters are \( H_{w, p} = 2.5 \text{ m} \) and \( T_{w, p} = 10.5 \text{ seconds} \), respectively. Three different types of groin combinations (I, T and Y-shapes) were tested under similar experimental conditions and their influence on the shoreline was observed. The combination of two Y-shaped groins and one I-shaped groin was the most effective and optimum alternative to protect the approximately 1500-m long shoreline at the east of the Kuzhronak River mouth (Figure 5). Observations on the model experiment also showed that the erosion between Groin 3 (Y-shaped), Groin 4 (Y-shaped), and Groin 5 (I-shaped) was minimized, and that deposition occurred at the connection points of these groins. A light-colored line representing the original shoreline and a dark-colored line representing the shoreline after the experiments, as shown in Figure 5, demonstrate this deposition.

After the completion of the experimental studies and final design, the change in shoreline was drawn in Figure 6. The positions of the shoreline before and after groins were also plotted for comparison. It can be seen from this figure that the groin combination designed by using the results of the physical model studies stabilized the shoreline.

*Journal of Coastal Research, Vol. 21, No. 3, 2005*
Figure 9. Shoreline changes obtained from the one-line numerical model studies done by Sayvan and Otay (2002).

Numerical Model

Preliminary studies showed that the topography of the shoreline covering 300 m length along the west and 1200 m length along the east of the Kızılınar River mouth and the bathymetry of the sea extending to -5 m depth in this region were needed for the numerical model. These documents were acquired from the Regional Directorate of DBI, and are presented in Figure 6 together with the grain combination designed by means of physical model studies. The one-line numerical model (HANSON and KRAUS, 1986) was applied to the portion of shoreline and grain combination shown in Figure 6 to simulate the dynamic effects of the groins on the shoreline in a long period.

The governing equation of the one-line model for shoreline position is

$$\frac{dy}{dt} + \frac{1}{2} \frac{DQ}{Dx} = 0$$

in which $y$ is the position of the shoreline, $t$ is the time, $D$ is the depth of closure, $Q$ is the volumetric rate of sand transport along the shoreline, and $x$ is the distance along shoreline. The shoreline transport rate, $Q$, is computed by the "CERC" relationship (SPM, 1984).

$$Q = K' (H^2 C_s) \sin 2\theta_a$$

$$K' = \frac{K}{16s(s-1)\gamma}$$

where $K$ is the dimensionless empirical coefficient, which is equal to 0.4, $s$ is the specific gravity of sand, $\alpha$ is the ratio of the volume of solid to total volume, $\gamma$ is the conversion factor, $H$ is significant wave height (m), $C_s$ is the velocity of group waves (m/s), and $\theta_a$ is the angle of breaking waves to the shoreline, which is equal to the difference between the angle the breaking waves make with the x-axis and the angle the shoreline makes with the x-axis, and is defined by the following relationship:

$$\theta_a = \theta - \tan^{-1}\left(\frac{dy}{dx}\right)$$

where $\theta_a$ is the angle of breaking waves to the x-axis (°). The subscript b in Equation (9) indicates quantity at wave breaking, and the velocity of the group waves during diffraction is defined by the following relationship (SPM, 1984):

$$(C_{\text{bg}}) = \left(\frac{gH}{\gamma}\right)^{1/2}$$

where $g$ is the gravitational acceleration (m/s²) and $\gamma$ is the ratio of the wave height to the water depth at wave breaking, which is approximately equal to 0.78.

Equation (8) can be expressed in finite difference method as

$$Y_i = 2B(Q_i, Q_{i+1}) + y_i$$

in which $Y_i$ is the computed shoreline and $B$ is defined as $\Delta t / (2D\Delta x)$ where $\Delta t$ is time step (s) and $\Delta x$ is grid distance (m). The accuracy and stability properties of numerical schemes for solving Equations (8) and (9) are well known. The accuracy of numerical simulation can be improved somewhat by taking a smaller time step for a given space step, assuming that the truncation error in the numerical computation is negligible. Numerical accuracy is a measure of how well a finite difference scheme reproduces the solution of a differential equation. For an explicit scheme, there is a stringent limitation on the size of the largest possible time step, other variables being held constant. For small breaking wave angles,

$$R_s = \frac{1}{2}$$

and

$$R_s = \frac{2K}\Delta t (H^2 C_s) \frac{1}{\Delta x^2}$$

The quantity $R_s$ was called the "stability parameter" by KRAUS and HARKAI (1983). Equation (14) is an adequate indicator of stability in most applications, because breaking wave angles are usually small. The stability parameter gives an estimate of the numerical accuracy of the solution, with accuracy typically increasing for decreasing values of $R_s$.

Because all waves occurring in this region have a prominent effect on the movement of sand along the shoreline, the heights of the waves used in the one-line model are very important. Thus, using the average wave height in the model yields a better simulation of the shoreline change than using the maximum wave heights approaching from all directions (GÖLER, ERGIN; and YALCINER, 1998). The average wave height can be computed by the expression given below:

$$H = \frac{\sum H_i P_i}{\sum P_i}$$

In Equation (15), $P_i$ is the average deep-water wave height, $H_i$ is the significant deep-water wave height, and $P$ is the probability of occurrence of a wave whose height is $H_i$. 

Journal of Coastal Research, Vol. 21, No. 0, 0000
Numerical Simulation

The values of the wave heights given in Table 1 are the average wave heights approaching from all directions, computed by using the long-term deep-water wave statistics. Using those wave heights in the numerical model, wave breaking heights and breaking angles were computed. These computed values of wave parameters were used in the one-line model, and the change in the shoreline, including the effects of groins designed by the physical model studies, was simulated for $\Delta t = 3$- and 6-hour time intervals. The results of the numerical simulation are presented in Figure 7. The results of the numerical simulation for this scenario compared well with results obtained from the physical model studies.

Hanson and Kraus (1986) suggested that besides using the average wave height as an input to coastal sand transport model, significant deep-water wave heights in excess of 12 ft y could also be used as input. Thus, significant deep-water wave heights exceeding 17 ft y were computed and presented in Table 2. These waves were used as an input to one-line model for $\Delta t = 1$, 2- and 3-hour time increments, and analysis of the simulation results revealed that the result of the simulations were unstable for $\Delta t > 1$ hour, and for $\Delta t = 1$ hour the shoreline simulated by the model approached the prototype condition. Thus, the time increment criteria for simulation of the shoreline for different scenarios were determined in this study. After the determination of stable time increment for numerical simulation, the average wave heights and significant deep-water wave heights exceeding 12 ft y were used as an input to the one-line model. The results of the numerical simulation of shoreline, including the effects of the groin combination implemented in this region, were generated and presented in Figure 8 (Hanson and Kraus, 2001). Analysis of this figure showed that the groins designed and implemented by the physical model studies not only stopped erosion along the 1200-m-long coastal reach but also caused accumulation of sand between groins. This situation clearly documented that the physical model and numerical model results conducted for this region compared quite well.
Figure 11. Shoreline near by the Kızılırmak River mouth before the groins.

Figure 12. Shoreline of the Eafra Plain at the Kızılırmak River Mouth.
Coastal Erosion at the Kızılırmak River Mouth, Turkey

Table 3. Probability of 12-hour occurrence of deep water wave heights and periods in one year.

<table>
<thead>
<tr>
<th>Wave Direction</th>
<th>Wave Height $H$ (m)</th>
<th>Wave Period $T$ (s)</th>
<th>Duration of Occurrence in 1 y (h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>E</td>
<td>1.84</td>
<td>4.5</td>
<td>12</td>
</tr>
<tr>
<td>ENE</td>
<td>1.90</td>
<td>5.5</td>
<td>12</td>
</tr>
<tr>
<td>NE</td>
<td>2.14</td>
<td>4.6</td>
<td>12</td>
</tr>
<tr>
<td>NNE</td>
<td>2.95</td>
<td>5.7</td>
<td>12</td>
</tr>
<tr>
<td>N</td>
<td>3.84</td>
<td>3.5</td>
<td>12</td>
</tr>
<tr>
<td>NNW</td>
<td>4.54</td>
<td>3.8</td>
<td>12</td>
</tr>
<tr>
<td>NW</td>
<td>5.31</td>
<td>3.9</td>
<td>12</td>
</tr>
<tr>
<td>WNW</td>
<td>6.10</td>
<td>5.5</td>
<td>12</td>
</tr>
</tbody>
</table>

The numerical solution for the groin combination given in Figure 8 that was designed by the physical model studies was compared with the numerical solution of SAVRAN and OYAT (2002), who studied the same region numerically using the one-line model. They used an L-shaped groin combination (three L-shaped groins) that was one of the alternative setups tested in the physical modeling portion of the present study. The locations of those three L-shaped groins were the same as those groins given in Figure 8, and the results of their numerical solution are given in Figure 9. Comparison of the numerical solution given in Figure 8 with that of the numerical solution given by SAVRAN and OYAT (2002) (Figure 9) shows that the groin combination designed by means of the physical modeling studies done in this study performed better. As seen in Figure 9, the east sides of L-shaped groins eroded severely even though accretion occurred on the west sides; this situation is not seen in Figure 8. This situation was also observed in the physical modeling portion of this study in the L-shaped groin layout alternative. Furthermore, the comparison of the results of those alternatives tested during the physical modeling studies also showed that the Y-shaped groin combination given in Figures 6, 7 and 8 performed better than did those other alternatives. Therefore, this combination was accepted as the final solution.

COMPARISON OF PHYSICAL AND NUMERICAL MODEL RESULTS WITH PROTOTYPE OBSERVATIONS

The groin combination given in Figure 6 was constructed in prototype during summer 2000, and field measurements were performed after the construction. Field measurements were performed by the Regional Directorate of DSI at six months, 1 year, 2 years, 3 years, and finally 3 years after the completion of the construction to determine the effectiveness of the groins designed by the physical model studies (DARMA, KOYKINAR, and GÜLER, 2003; GÜLER, DARMA, and KOYKINAR, 2002; KÖYKINAR, GÜLER, and DARMA, 2000). These field measurements indicated that the erosion of the shoreline near the Kızılırmak River mouth was completely controlled by these protection structures built to stabilize the shoreline. Furthermore, the shoreline progressed toward the sea at some locations.

Comparison of the field measurements done in January 2003 with those obtained from the physical model and numerical model studies showed a similar trend in the deposition of sediment along the shoreline (Figure 10). Even though the physical and the numerical model results and field measure-
Figure 14. A view of the drainage channel and shoreline of the Bofra Plain near the Kuklimuk River mouth one year after the construction of the two Y-shaped and one I-shaped groins.
measurements showed similar trend in terms of accretion and erosion at certain locations of the reach, the magnitude of accretion determined from the field measurement was higher than what was produced by the physical and the numerical model studies. The distortion and the scale effect of the physical model, which might have influenced the hydrodynamic parameters of waves responsible for the accretion and erosion of sediment, might have caused this difference. In order to verify the effectiveness of the groin combination designed by the physical model study, two photos depicting the shoreline in the vicinity of the Kızırmak River mouth before the construction of the groins (Figures 11 and 12) and two photos depicting the situation one year after the construction of the two Y-shaped and one L-shaped groins (Figures 13 and 14) are given. These photos and Figure 10 clearly document that the groin combination designed by means of physical model studies to prevent the shoreline from erosion was quite successful. Because the continuity of sediment transport along the shoreline located at the east side of the river was altered by the construction of those groins, erosion was accelerated at the shore adjacent to the coastal reach where groins were constructed.

APPLICATION OF THE PARABOLIC BAY SHAPE METHOD TO THE PRESENT PREDICTIONS

Results of physical and numerical models and prototype observations were compared with the method of Hsu and Evans (Hanson and Kraus, 2001; Silverstein and Hsu, 1997) developed for estimation of bay shapes in static equilibrium. In accordance with Hsu and Evans, a parabolic bay shape for bays in static equilibrium can be defined by

\[ \frac{R}{R_0} = C_0 + C_1 \left( \frac{B}{B_0} \right)^{1/2} + C_2 \left( \frac{B}{B_0} \right)^{3/2} \]  

where \( R \) is the length of radii drawn from the point of deflection to the beach at angle \( \theta \) to the wave-crest line, \( B \) is
the wave obliquity, $R_4$ is the length of control line, and $C_1$, $C_2$, and $C_3$ are the coefficients depending on $R$.

Figure 15 shows the application of the parabolic bay shape equation (Equation (16)) to the shore protection system designed for the Kizilirmak River mouth. Equation (16) was sufficient to obtain bay outlines, which matched actual bays extremely well for the complete periphery. The bay shape predicted by the equation has almost the same bay shape measured in the field, especially for the shape of the bay between Grain 4 and Grain 5. However, for a bay in dynamic equilibrium such as the bay between Grain 3 and Grain 4, some deviations in bay outlines can be seen because of the movement of sediment by waves in this region. In addition, for a bay still in dynamic equilibrium, prediction of $\beta$ may not be oriented to that for static equilibrium. It is also important to note that some variables associated with natural bays, such as their beach profiles and the wave characteristics other than wave obliquity, are not included in the present method of prediction.

CONCLUSIONS

The shoreline of the Bafra Plain near the Black Sea and Kizilirmak junction has been eroding since the sediment budget of the Kizilirmak River was altered by flow regulation structures. A coastal band of the Bafra Plain approximately 35 m wide has been lost annually because of the effects of Black Sea waves. A groin combination was designed by physical and numerical model studies to prevent this erosion. The results of the physical and numerical model were compared with the field measurements conducted after the construction of the groin combination system in order to evaluate the effectiveness of physical and numerical modeling. Implementation of the groin combination stopped the erosion in the coastal reach near the Kizilirmak River mouth. Comparison of the field measurements and studies of the shoreline done in January 2005 with those results obtained from the physical and numerical model studies showed a similar trend in the accretion of sediment at certain sections of the shoreline where groins are constructed. This study also confirmed that the one-line numerical model is a very effective tool in the design of shore protection structures. Application of the parabolic bay shape method to the shore protection system designed for the Kizilirmak River Mouth showed quite reasonable predictions when compared with prototype measurements. The parabolic bay shape method, therefore, appears to be suitable for field applications.

LITERATURE CITED


