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Intervention of Human Activities on Geomorphological Evolution of Coastal Areas: Cases from Turkey

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1. Introduction

Coastal engineers, geomorphologists and scientists have been trying to understand and find the answer to one simple and a major question among a wide variety of challenging coastal problems for decades. Where will the shoreline be tomorrow? Or after a severe storm? Or next year? Or in a decade? In other words, how will the coasts of our earth evolve in time? And where do we stand as human being in this highly complex, yet fragile evolution of coasts?

Coastal areas are often highly scenic and offer plenty of natural resources. Over 50 percent of the world's population lives within 200 kilometers of coastline (Hinrichsen, 1994; Deichmann, 1996). According to the projections of future population, 75 percent of the world's population, or 6.3 billion people, could reside in coastal areas by 2025 (Hinrichsen, 1996). Increasing population and urbanization at coastal areas, unconscious exploitation of natural resources, incompetent management and education strategies and lack of control mechanisms increase the complexity of problems and severity of measures at coastal areas. One major coastal problem that almost every country with some kilometers of coastline faces and spends millions of dollars to solve the imbalance in coastal sediment budget at coastlines, resulting in severe erosion or accretion problems and loss of income from tourism and other coastal opportunities. Miami Beach, in Florida is a typical example for the above given problem. In the early 1970s, there was severe erosion at Miami Beach, where it was lined with seawalls, and compartmented by long steel groins. The numbers of visitors to Miami Beach and hotel occupancy rates were in decline. As a remedial measure, from 1976 to 1981, the Miami Beach Nourishment Project was undertaken, widening the beach by 100 m over a length of 16 km at a cost of $64 million USD. The project required in excess of 10 million cubic meters of sand obtained from offshore borrow areas by large hydraulic dredges. (Dean & Dalrymple, 2002). As a result, number of visitors at the beach increased from 8 million in 1978 to 21 million in 1983 (Houston, 1995a).

To control sediment budget at coastal areas, several types of measures are applied. These measures are categorized in two groups as hard measures (jetties, groins, detached

breakwaters, seawalls, dykes etc.) and soft measures (beach nourishments, sediment traps, etc.). The hard measures involve the construction of solid structures within the intertidal zone to reduce wave energy and stop the sea from interacting with the hinterland. In contrast, soft measures generally avoid the solid structure approach, but use natural environments and sediments to reduce wave action (French, 2001; Dean, 2002). However, application of these measures usually fails or does not work as planned if applied without a clear understanding of the picture in the problematic coastal area and a well-designed and strictly implemented integrated coastal zone and river basin management structure.

This chapter gives brief information on the theoretical background of sources of sediment transport mechanisms and physical and numerical modeling attempts to understand these mechanisms. The governing parameters of these mechanisms and their temporal and spatial effects on the geomorphology of coastal areas are briefly discussed. Special emphasis is given to the numerical modeling of shoreline changes due to longshore sediment transport which becomes one of the governing mechanisms in the long term at coastlines where human induced activities put a pressure on the sediment supply resources e.g. dunes, cliffs and rivers. Two case studies of coastal erosion problems from Turkey are covered in this chapter, focusing on the numerical modeling of shoreline changes around coastal defense structures. The first case study is from northern coasts of Turkey, Bafra alluvial plain where the Kızılırmak River discharges into the Black Sea. It is one of the RAMSAR protection sites in Turkey. It has a rich biodiversity and is an important habitat for globally endangered bird species. The amount of sediment carried by the river has reduced drastically after the construction of flow regulation structures on the river, which led to serious coastal land erosion at the river mouth. The second case study, as another example to human induced type of coastal erosion problems, is from one of the touristic areas of Turkey, Side beaches in Antalya at southern coasts of Turkey. Alteration of the characteristics of the sea bottom topography due to touristic considerations led a serious erosion problem in front of the Perissia Hotel, in Side. For these two cases, the implemented remedial coastal defense measures, the shoreline changes after the implementation of these measures and applications of numerical modeling to predict these shoreline changes are covered within the case studies.

2. Sediment transport at coastal areas

Coastal areas are the environments with a very sensitive and dynamic balance governed by various complex physical processes. The shape of coastal landforms is a response of the materials that are available to these processes acting on them (Woodroffe, 2003). Among these processes, sediment transport plays an important role in continuous geomorphological evolution of coastal areas. This evolution mainly depends on the sources and sinks (losses) of the sediment budget of the coastal area. Major sources of sediment in coastal areas are weathering of cliffs by marine action (waves, tides and currents), wind and rain, freeze-thaw process and groundwater flow, sediment carried by rivers, dunes, wind-blown inland material, biogenic materials such as shells and coral fragments especially in some tropical areas and human induced sources like artificial beach nourishment, nearshore disposal of dredged soil and industrial waste tipping. The losses in the sediment budget can be listed as erosion by waves, tides and currents, submarine canyons, human extraction along rivers (sand mining) and at nearshore for commercial purposes or improve navigation, damming

of rivers and streams, fishing by the use of explosives and trapping of sand on the upstream side of the coastal structures (CIRIA, 1996).

Sediment in coastal areas are transported in longshore and cross-shore directions by waves, by tidal and other steady or quasi steady currents like wind induced circulation, by winds from land (cliffs, dunes and inland), by rivers, small streams and gullies and also by human agencies. In temporal scale, effects of longshore sediment transport become more distinct in long term (years or decades) like recession of shoreline or accretion of sediments. As for the short term event like seasonal changes or significant storm events, cross-shore movement of sediments becomes the main actor in geomorphological changes in coastal areas such as bar formations in the surf zone (Kamphuis, 2000; Masselink & Hughes, 2003).

2.1 Numerical modeling of sediment transport

Prediction of morphological changes in coastal areas mostly depends on determination of long and short term effects of these transport mechanisms and changes in source-sink parameters of sediment budgets of coastal areas. Depending on the effective coastal processes, sediment budget knowledge and scope of the study, coastal sediment transport problems can be solved using physical modeling techniques (Hamilton & Ebersole, 2001; Wang et al., 2002; Gravens & Wang, 2007; Kökpınar et al., 2007), or with numerical morphological models expanding from one-dimensional (Hanson, 1987; Dabees & Kamphuis, 1998) to sophisticated 2D or 3D models (Roelvink et al., 2009), or with analytical and empirical approaches. Both empirical and analytical solutions give quick results to the scientists and engineers and are quite practical in early stages of understanding the coastal problems, computation of beach evolution and preliminary design studies, yet, for a detailed investigation of natural processes in coastal areas, physical or numerical models are commonly implemented. Compared to physical modeling, numerical models possess certain advantages in allowing the investigation of a wide range of parameters, adaptability to a variety of sites, economical operation, and the absence of scale effects (Hanson, 1987). These advantages sometimes may lead misuse of these models or misinterpretation of the results of these models due to lack of knowledge related to the actual site conditions. The uses of numerical models change for different spatial and temporal scales. For example, one-dimensional numerical shoreline change models are preferable for most of the cases due to their applicability in wide ranged temporal (from 1 year up to 20 years) and spatial scales (from 0.5 kilometer up to 10 kilometers), whereas more sophisticated 2D and 3D models are generally utilized for medium-term (up to 10 years) and short (a single storm up to 1 year) shoreline changes, respectively, due to their complexity and intensive computations (Hanson et al., 2003).

2.1.1 One-line theory

The fundamentals of shoreline change models were first established by Pelnard-Considere (1956), who set down the basic assumptions of the "one-line" theory, derived a mathematical model, and compared the solution of shoreline change at a groin with laboratory experiments. The basic assumption of "one-line" theory is the concept of "equilibrium beach profile", that cross-shore transport effects such as storm-induced erosion and cyclical movement of shoreline position associated with seasonal changes in wave climate are assumed to cancel over a long simulation period. In this concept, the change of

shoreline position (in cross-shore direction) in time is due to longshore sediment transport and local sources or sinks along the coast (such as river discharges, beach nourishment or net cross-shore sand loss). The boundaries of "equilibrium beach profile" are the berm height (B) on land, maximum elevation of sediment transport during the uprush (Masselink & Hughes, 2003), and closure depth (D_C) at sea, which is limiting depth beyond which no significant longshore and cross-shore transports take place due to littoral transport processes (Mangor, 2004), (Fig. 1).

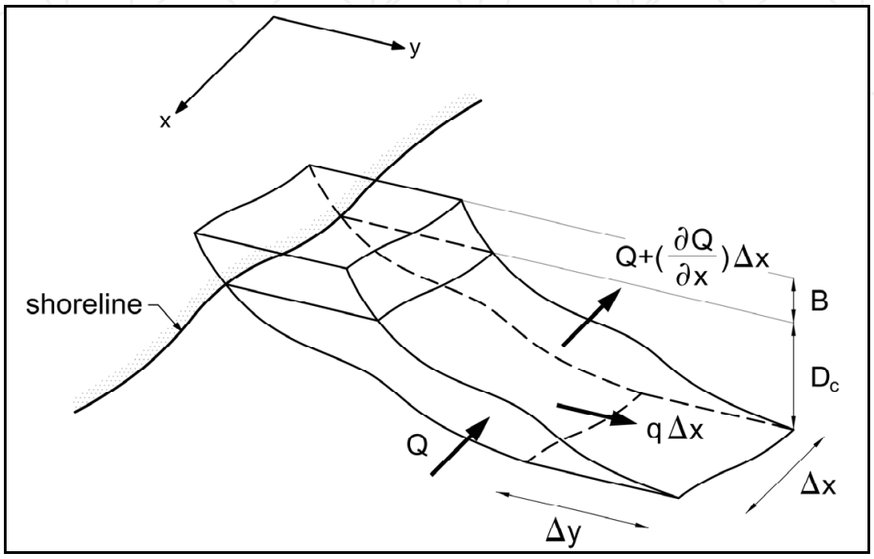


Fig. 1. Definition sketch for conservation of mass for a sandy beach system (Baykal, 2006)

Based on the above given assumption, "one-line" theory is expressed with the following differential equation which is also called as sand continuity equation,

$$\frac{\partial y}{\partial t} = -\frac{1}{(D_C + B)} \cdot \left(\frac{\partial Q_{t,lst}}{\partial x} + q \right) \tag{1}$$

where y is the shoreline position, x is the longshore coordinate, t is the time, $Q_{t,lst}$ is the total longshore sand transport rate, q represents local sand sources or sinks. Hallermeier (1978) relates the closure depth to $H_{s,12}$ and T_s which are the significant wave height exceeded 12 hours per year and corresponding significant period respectively (Eq.2).

$$D_C = 2.28 \cdot H_{s,12} - 68.5 \cdot \left(\frac{H_{s,12}^2}{g T_s^2} \right) \tag{2}$$

For the cases where the governing transport mechanism is assumed to be longshore sediment transport only, Hanson (1987) suggests to use the limiting depth of longshore sediment transport computed with the same equation (Eq.2) using significant breaking wave height ($H_{s,b}$) instead of $H_{s,12}$.

For the total longshore sediment transport rates including both bed and suspended sediment loads, the most widely used equations are Coastal Engineering Research Center of

The United States Army Corps of Engineers [thereafter CERC or USACE] (1984) and Kamphuis (1991) formulas. These formulas consider only wave generated currents and disregard other mechanisms like tidal or wind-induced currents. In CERC formula (Eq.3), the total volume of sand transported alongshore is related to the longshore component of wave power per unit length of beach,

$$Q_{t,lst} = \frac{K}{16 \cdot (\rho_s/\rho - 1) \cdot (1-p)} \cdot \sqrt{\frac{g}{\gamma_b}} \cdot H_{s,b}^{5/2} \cdot \sin(2\alpha_b) \quad (3)$$

where $Q_{t,lst}$ is the total volume of sediment moving alongshore per unit time (m^3/sec), K is the dimensionless empirical proportionality coefficient given by Komar & Inman (1970) as $K_{SPM\ sig}$ = 0.39 based on field studies utilizing the significant wave height, ρ_s is sediment density taken as 2,650 kg/m^3 for quartz-density sand, ρ is the water density (1,025 kg/m^3 for 33 parts per thousand (ppt) salt water and 1,000 kg/m^3 for fresh water), g is the gravitational acceleration (9.81 m/sec^2) and p is the in-place sediment porosity that may be taken as 0.4. The breaker index (γ_b: ratio of breaking wave height to breaking depth) is taken as 0.78 for flat beaches (Weggel, 1972). $H_{s,b}$ and α_b are the significant breaking wave height and breaking wave angle respectively.

In the Kamphuis's (1991) formula (Eq.4), the effect of significant wave period (T_s), median particle size in surf zone (D_{50}) and the beach slope (m_b) at the depth of breaking, the slope over one or two wavelengths seaward of the breaker line are taken into consideration in addition to the significant breaking wave height and breaking wave angle.

$$Q_{t,lst} = 2.27 \cdot H_{s,b}^2 \cdot T_s^{3/2} \cdot m_b^{3/4} \cdot D_{50}^{-1/4} \cdot \sin^{3/5}(2\alpha_b) \quad (4)$$

Both CERC (1984) and Kamphuis (1991) formulas are utilized in shoreline evolution models extensively. Although these expressions have been verified in several researches for years, their predictive capabilities are limited mostly by site specific conditions (i.e. bathymetrical conditions, wave climate characteristics, sediment size and grading). Wang et al. (2002) state that Kamphuis (1991) formula produces more consistent predictions than the CERC formula 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. However, it seems most appropriate to use the CERC formula for high energy wave events and the Kamphuis (1991) formula for low-energy events (less than 1 m in wave height).

2.1.2 An example to one-line numerical modeling

For the purpose of understanding and predicting long term shoreline changes under the actions of wind waves, a numerical shoreline change model, CSIM (acronym for Numerical Model for Coastline-Structure Interaction), based on "one-line" theory is written in Ocean Engineering Research Center of Civil Engineering Department in Middle East Technical University (Şafak, 2006; Artagan, 2006; Baykal, 2006 and Esen 2007).

The numerical model is based on "one-line" theory with a built-in wave transformation module capable of simulating combined wave refraction and diffraction mechanisms in the vicinity of coastal structures such as groins and offshore breakwaters. It solves the sand continuity equation with both explicit and implicit finite difference schemes. The model can

also simulate shoreline changes for some other coastal defense measure applications such as seawalls and artificial beach nourishment. The significant breaking wave heights in the surf zone are computed with formula given in CEM (2003);

$$H_{s,b} = (H_{s,0})^{4/5} \cdot [C_{g0} \cdot \cos(\alpha_0)]^{2/5} \cdot \left[\frac{g}{\gamma_b} - \frac{H_b \cdot g^2 \cdot \sin^2(\alpha_0)}{\gamma_b^2 \cdot C_0^2} \right]^{-1/5} \quad (5)$$

where $H_{s,b}$ is the significant breaking wave height, $H_{s,0}$ is the deep water significant wave height, α_0 is the deep water wave angle between wave crests and bottom contours, C_0 and C_{g0} are the deep water wave celerity and group celerity, respectively.

For combined effects of wave diffraction and refraction in the shadow zone of the coastal defense structures, Kamphuis's methodology (2000), which is based on Goda's diffraction diagrams (Goda et al., 1978), and refraction of regular waves are utilized. Combined effects of wave diffraction from multiple sources like consecutive offshore breakwaters are considered based on Vafaei's (1992) approach which is basically vectorial summation of wave rays coming from different sources of diffraction in proportion to the square of diffracted wave heights of these rays. More details and applications of the numerical model are given in Ergin et al. (2006) and Güler et al. (2008).

3. Case study-1: Coastal erosion problem at the Kızılırmak River mouth

In most of the developing countries, denser the population in coastal areas, the more vulnerable they become to severe environmental problems such as coastal erosion, exploitation and depletion of natural resources and extinction of endangered species. Wetlands at coastal areas are one of the most adversely affected areas due to their diverse floras and faunas. In Turkey, there are 13 sites designated as "Wetlands of International Importance" with a total surface area of 179,898 ha and 5 of these sites are located at coastal areas. One of these sites is Bafra alluvial plain (Kızılırmak Delta) where the Kızılırmak River discharges into the Black Sea. The site was designated as RAMSAR Area in 15.04.1998. It has a surface area of 21,700 ha including dunes, beaches, shallow lakes, seasonal marshes and wooded areas (URL-1). Numerous species of water birds, several of which are globally threatened, breed at this site. Over 92,000 water birds of various species winter at the site. In recent years, eutrophication, deforestation, illegal constructions and coastal erosion have become increasingly problematic in Kızılırmak coastal wetland (Kuleli et al., 2011). The location of Bafra alluvial plain is shown in Fig.2 and Fig.3.

The Kızılırmak 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 (Fig. 2). It is the longest river in Turkey, with a length of 1,355 km, draining a basin of 74,515 km² (Kökpınar et al., 2007). The amount of sediment carried by the Kızılırmak River was 23.1 million tons/year till 1960's prior to any flow regulatory structures and decreased to 18 million tons/year following the construction of Hirfanlı Dam in 1960, and almost came to a cease with the total amount of 0.46 million tons/year after the constructions of Altınkaya Dam in 1988 and Derbent Dam in 1991 (Hay, 1994). This drastic decrease in the amount of sediment carried by the Kızılırmak River resulted in severe erosion with a maximum 1 km wide band of shoreline since 1988 according to the Regional Directorate of State Hydraulic Works and from local residents (Kökpınar et al., 2007).



Fig. 2. Location of Bafra alluvial plain

Regarding the coastal erosion problem at Bafra alluvial plain, Kuleli et al. (2011) focused on the shoreline change rate analysis by automatic image analysis techniques using multi-temporal Landsat images and Digital Shoreline Analysis System (DSAS) along five Ramsar wetlands of Turkey. For Kızılırmak Delta, they have used three satellite images for the years 1989, 1999 and 2009 and found 16.1 m/year erosion rate for the Kızılırmak Delta.



Fig. 3. Bafra alluvial plain and plan view of the existing shore protection system at the Kızılırmak River mouth (Google Earth, 2011)

The first remedial measure against this severe coastal erosion problem at the river mouth was held in 2000 by State Hydraulic Works (DSİ) based on the findings of the physical and mathematical model studies conducted at the Hydraulic Model Laboratory of DSİ in Ankara, Turkey (Kökpınar et al., 2007). It was composed of two Y-type and one I-type groins constructed at the eastern shoreline of the river mouth (Fig. 3). Ergin et al. (2006) studied the shoreline changes around these structures with the numerical shoreline change model, CSIM, using the shoreline measurements taken in 1999 (April) and 2003 (January) by DSİ. Despite utilizing T-groins instead of Y-groins and the numerical model's lack of capability of modeling tombolo formation, the model results was in good agreement quantitatively with the field measurements, especially at western sides (updrift) of second and third groins.

After the construction of first remedial system (two Y-type and one I-type groins), the shoreline retreat slowed down between the groins and trapping of sediment initiated. However, recession at the shoreline due to wave action continued to the east from the third groin (I-groin) as almost no sediment is carried by the Kızılırmak River. Later, two jetties were constructed at the west and east sides of river mouth between the years 2001-2004 to prevent seasonal closure of the river mouth. Between the years 2004-2005, the coastal defense system was extended with the construction five more I-type groins to prevent the collapse of drainage channel. Although, the drainage channel has been saved against wave action constructing the new series of five I-type groins, shoreline retreat at the east side of the defense system could not been prevented and continued to further east.

In this study, the performance of the second series of groins (5 I-type groins) and the shoreline changes around these groins between the years 2003-2007 are studied using the one-dimensional numerical shoreline change model, CSIM. The corresponding shoreline GPS measurements were provided by Regional Directorate of DSİ (Fig. 4).

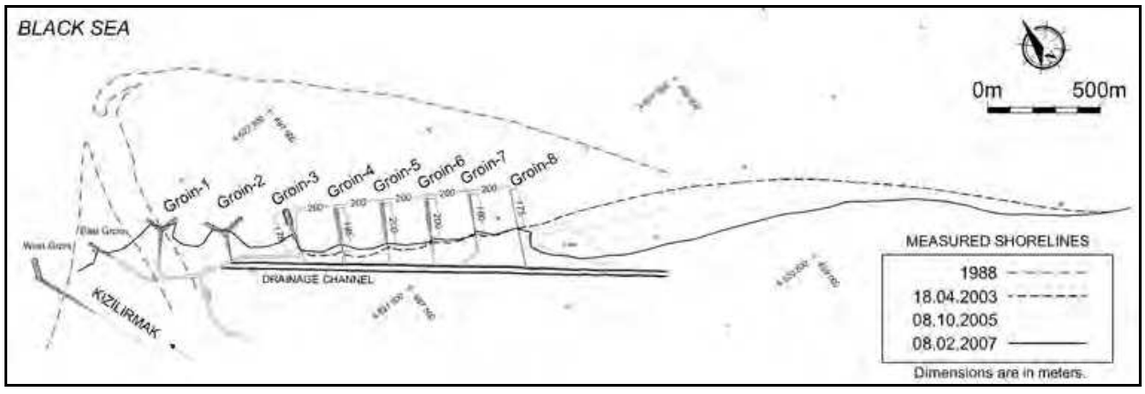


Fig. 4. Kızılırmak River mouth shore protection system plan view

3.1 Wave climate study: Kızılırmak River mouth

Long term geomorphological evolution of coastal areas under wave action results from series of short-term wave events occurring randomly. When there exist no time histories of these wave events or no continuous wave measurements, yet, wind measurements exist for such coastal areas, wave hindcasting studies are performed. For each wave direction, the wind velocities and effective fetch distances are used to hindcast the wave climate history of

the site and a long-term wave statistics study is carried out to determine yearly or seasonal deep water wave characteristics.

There exist no continuous wave measurements for the shores close to Bafra alluvial plain. Therefore, a wave hindcasting study has been performed using hourly average wind data measured at 10 m above ground level by Sinop Meteorological Station between the years 2000-2009, obtained from General Directorate of Meteorological Affairs (DMİGM). The location of the river mouth is open to waves approaching from a wide directional sector from West to East-South-East. The effective fetch distances for the directions in this directional sector are determined from the navigation maps of Navigation, Hydrography and Oceanography Department of Turkish Naval Forces (SHODB). In the computation of effective fetch distances, for each direction, the effective generation area is considered as a sector from -22.5^0 to $+22.5^0$ totally covering an area of 45^0 with 7.5^0 intervals (USACE, 1984). The effective fetch directions and distances are shown in Fig. 5.

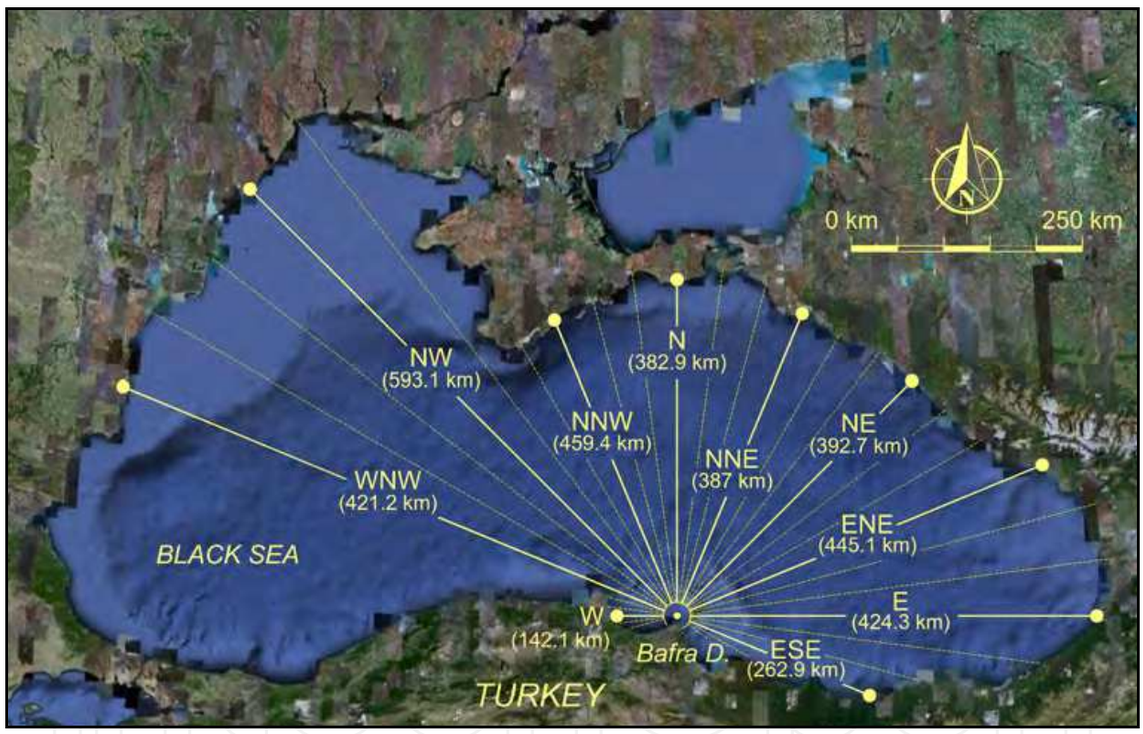


Fig. 5. Wave directions for Bafra region (Google Earth, 2011)

Using the effective fetch distances and the wind data obtained from Sinop Coastal Meteorological Station, deep water wave parameters (H_{s0} : deep water significant wave height, T_s : significant wave period) are obtained for the storms occurred during 10 years (2000-2009) by using the numerical model, W61, developed at Middle East Technical University, Department of Civil Engineering, Ocean Engineering Research Center (Ergin & Özhan, 1986; Ergin et al., 2008; Ergin et al., 2009).

The characteristic deep water wave steepness value (H_{s0}/L_0), the ratio of deep water significant wave height (H_{s0}) to the corresponding deep water wave length (L_0), for the

project area is obtained as 0.040 from deep water significant wave heights and deep water wave lengths computed from corresponding significant wave periods (T_s) of each individual storm ($L_0 = gT_s^2/2\pi$ where g is gravitational acceleration in m/s^2).

To represent the actual wave conditions better and minimize the effect of wave data sequence in one-line numerical shoreline change modeling as discussed in Şafak (2006), a seasonal based wave data input is prepared. For this purpose, long term wave statistics is carried out for seasonal wave data. The hindcasted wave data is classified in 0.4m ranges for each season (winter season is assumed to include January, February and March, the other seasons followed accordingly) and the cumulative number of occurrences of each wave height class is plotted on to a semi-log graphical paper. The cumulative exceedance probability of deep water significant wave height, H_{s0}, is given as;

$$Q(>H_{s0}) = \exp\left[(H_{s0} - B)/A\right] \tag{6}$$

where $Q(>H_{s0})$ is the cumulative exceedance probability of a deep water significant wave height (H_{s0}). This equation indicates that if data points corresponding to H_{s0} and $Q(>H_{s0})$ are plotted on a semi-log graphical paper (H_{s0} on normal, and $Q(>H_{s0})$ on logarithmic scales), they should lie on a straight line with a slope of A and intercept of B when $Q(>H_{s0})$ is the horizontal axis.

Another major assumption in the preparation of wave data input is such that the effects of smaller but more frequent waves are considered to be more appropriate for a better representation of long term wave climate rather than higher waves with less frequency. By using the long-term statistics in seasonal time scale, the representative deep water significant heights ($H_{rs,0}$) of waves coming from every direction, their periods and seasonal frequencies in hours for each season are calculated using (Güler, 1997; Güler et al., 1998 and Şafak, 2006);

$$H_{rs,0} = \frac{\sum(P_i \cdot H_i)}{\sum P_i} \tag{7}$$

where H_i is the wave height and P_i is the occurrence probability of wave height H_i. Occurrence probability (P_i) of wave height (H_i) is computed by using the corresponding occurrence durations within the given range as follows;

$$P_i = Q(H_i - k) - Q(H_i + k) \tag{8}$$

where Q is the exceedance probability and k is an assigned range to compute occurrence probability. In Table 1, the seasonal wave data input for the model consisting of representative wave heights ($H_{rs,0}$), corresponding periods (T_s) and seasonal occurrence durations (Δt in hours) from all directions is presented.

As seen from Table 1, for all seasons the highest occurrences of waves within the range of 1 meter are from West-North-West and North-West.

3.2 Numerical modeling study: Kızılırmak River mouth

The shoreline changes between the second series of groins (5 I-type groins) constructed in addition to the first part (between years 2004-2005) starting from Groin-3 to Groin-8 with

200 meter spacing between adjacent groins (Fig. 4) are studied using the one-dimensional numerical shoreline change model, CSIM. The shoreline measurements for the years 2003 and 2007 are discretized at 10 m intervals. Starting from Groin-3, the apparent lengths of the groins are given as 175, 195, 200, 200, 180 and 175 meters respectively. From the sieve analyses of the sediment samples taken from the site, the median grain size diameter (D_{50}) is determined as 0.23 millimeters (Kökpınar et al., 2007). The berm height (B), the landward end of the active profile, is assumed as 2 meters. In the application of numerical modeling for the case in the Kızılırmak River mouth, it is assumed that no source or sink exists. The sequence of wave data input (Table 1) in the simulation starts with spring waves from W to ESE directions and continues with summer, autumn and winter waves.

Directions	WINTER			SPRING			SUMMER			AUTUMN		
	$H_{rs,0}$ (m)	T_s (sec)	Δt (hrs)	$H_{rs,0}$ (m)	T_s (sec)	Δt (hrs)	$H_{rs,0}$ (m)	T_s (sec)	Δt (hrs)	$H_{rs,0}$ (m)	T_s (sec)	Δt (hrs)
W	1.59	5.07	5.3	0.94	3.90	0.5	0.76	3.50	1.3	1.63	5.13	6.4
WNW	1.05	4.12	462.5	0.94	3.89	544.0	1.07	4.16	323.5	1.08	4.17	368.5
NW	1.08	4.17	391.4	0.86	3.72	149.2	1.18	4.36	119.9	1.05	4.11	221.2
NNW	1.03	4.09	100.1	0.72	3.41	113.1	0.81	3.62	172.3	1.19	4.39	124.0
N	0.91	3.83	66.7	0.58	3.06	16.3	0.70	3.36	41.4	1.13	4.27	13.5
NNE	0.83	3.66	14.4	0.70	3.35	17.0	0.63	3.20	22.4	0.84	3.67	28.6
NE	0.80	3.58	3.8	0.64	3.20	2.4	1.82	5.42	1.9	1.10	4.21	6.7
ENE	1.18	4.37	14.4	0.74	3.47	7.3	0.86	3.73	11.3	1.07	4.15	10.9
E	0.89	3.79	3.5	0.58	3.05	0.9	1.17	4.35	3.5	1.08	4.18	3.5
ESE	0.79	3.56	32.6	0.65	3.25	56.0	0.73	3.42	31.8	0.77	3.53	33.0

Table 1. Representative wave heights, corresponding periods and seasonal occurrence durations from all directions for each season

The sediment transport formulas are mostly based on various laboratory and field measurements. As the accuracy of these formulas may change from one site to another depending on the wave characteristics, grading of sediment, existing currents (tidal or wind induced) and sea bottom topography, direct use of these formulas may not reflect the actual sediment transport rates for the site studied. Therefore, calibration of these kinds of numerical models needs to be performed beforehand. For this study, the shoreline retreat at the downdrift of Groin-8 from 2003 till 2007 is used to calibrate the model.

The sea level variations of the Black Sea, where semidiurnal tides are dominant with a spring tidal range of 8-12 cm, has been largely controlled by a seasonal variation (inter-annual change) of 20–40 cm maximum in June (Alpar, 2009; Vigo et al., 2005; Bondar, 2007). The inter-decadal sea level variations have a period of 30 years and are in the order of 16 cm (Trifonova & Grudeva, 2002). The sea level variations produced by the atmospheric pressure changes or by sudden changes of wind direction could reach 10-20 cm with periods of 1-5 hours (Bondar, 2007). In the computations, the effects of these sea level variations are disregarded. The results of the simulations are given in Fig. 6.

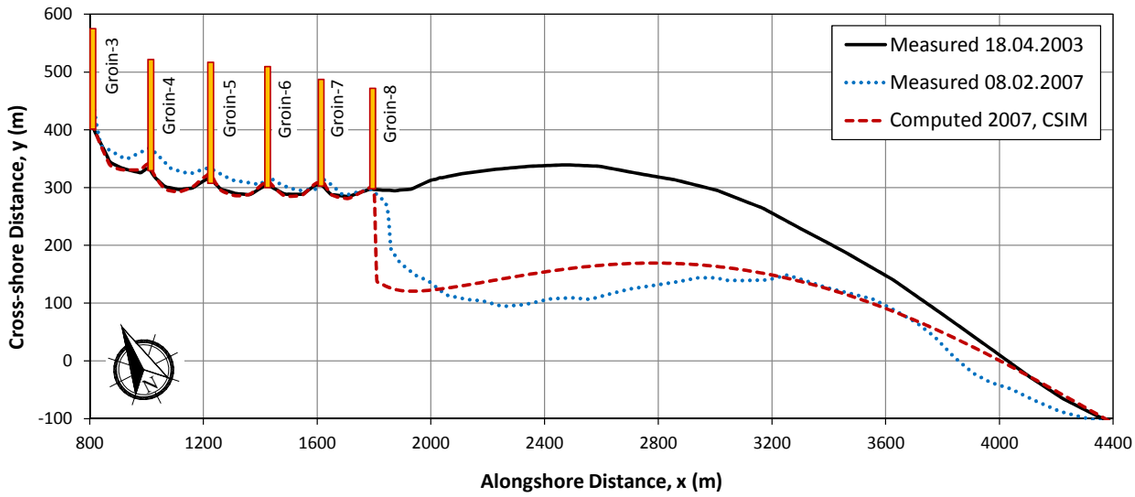


Fig. 6. Comparison of the site measurements with the numerical simulation (Groin series and downdrift of Groin-8)

3.3 Results and discussion: Kızılırmak River mouth

In the design of coastal defense structures such as shore perpendicular groins as in the case of the Kızılırmak River mouth, adverse effects of the defense structures at the adjacent shores have to be studied before the implementation of the project. In this case study, as an application of the numerical model, the shoreline changes both between coastal defense structures and adjacent shore are studied together.

Fig.6 shows that the shoreline changes between groins could only be simulated qualitatively; yet, the shoreline retreat after Groin-8 is in good agreement with the measurement quantitatively. For the groin system, the numerical model results reflect the shoreline changes qualitatively but not quantitatively. A reason for the inconsistency between the measured and computed shoreline positions for the year 2007 at the groin series might be due to the actual bypassing amount of sediment from the first 2 Y-type and 1 I-type groin system is not well defined. Also the small amount of the sediment carried by the river is neglected in the computations as no reliable knowledge or measurements regarding the amount of sediment carried by the river exist. As for the downstream shore adjacent to Groin-8, the agreement between the numerical model results and measurements are in good agreement both qualitatively and quantitatively.

4. Case study-2: Coastal erosion problem in Side, Perissia Hotel Beach

The second example for the adverse effects of anthropogenic activities on geomorphological evolution of coastal areas is from one of the touristic areas of Turkey; Perissia Hotel Beach, located in Side in Antalya at southern coasts of Turkey (Fig. 7). Side is one of the resort towns in Antalya, heavily utilized by touristic activities with numerous hotels. The closest rivers to Side are the Manavgat River (at 93 km in length, app. 12.5 km to Perissia Hotel Beach) and Köprü Çay stream, one of the finest rafting sites in Mediterranean region (at 14 km in length, app. 19 km to Perissia Hotel Beach). There are two dams on the Manavgat River; Oymapınar (1984) and Manavgat (1987) dams. The Manavgat River drains a

topographic basin of 928 km² extending to the northern Taurus mountain range and with the addition of several closed basins (Mesozoic limestone aquifers), it has a total drainage area of 9,100 km², one of the major catchment basins in the Mediterranean coastal area (Yurtsever & Payne, 1986). The direction of the net longshore drift is from East to West.

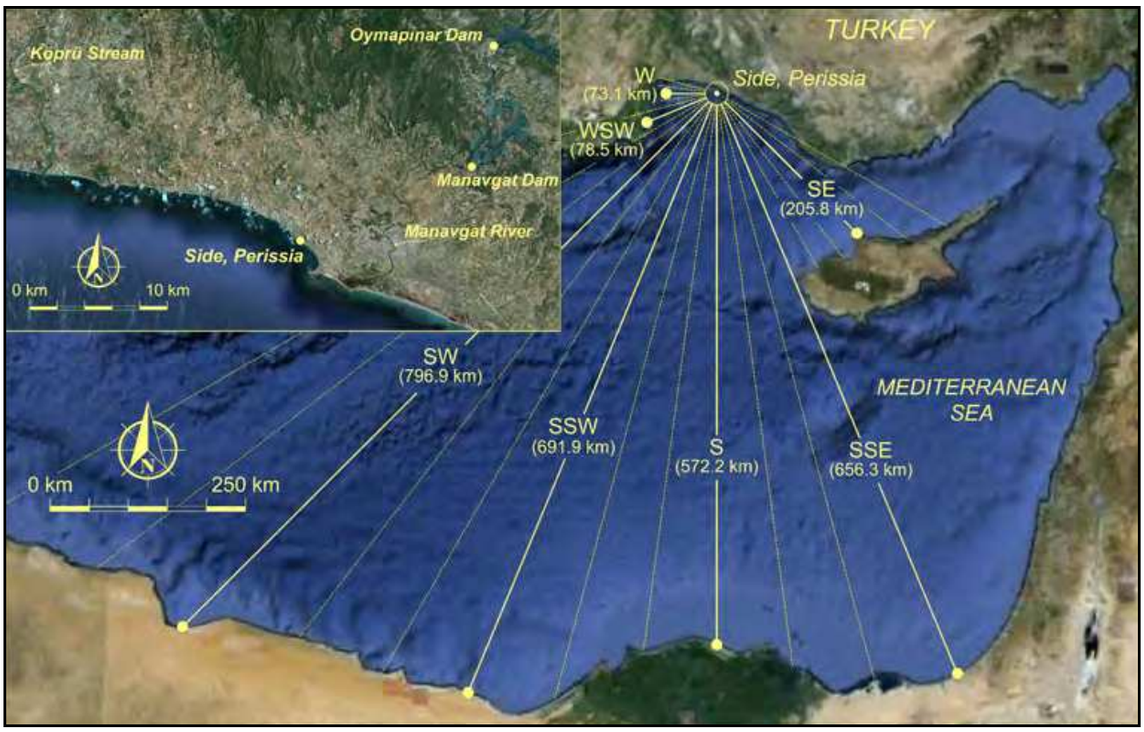


Fig. 7. Location of Side, Perissia Hotel beach and effective fetch distances for all directions (Google Earth, 2011)

The beach in front of Perissia Hotel is approximately 300 meters long and the width of the beach was approximately 50 meters before 1999. It is protected by naturally existing rocky formations at the east and west boundaries of the beach which extend 25 meters into the sea and a 120 meter long pier at the center of the beach, rocky formations lay underneath. The major protection for the beach against erosion due to wave attack was the rocky formations which were located at approximately 1-1.5 meter water depths and 60-100 meters offshore acting as submerged breakwaters and dissipating the energies of the approaching waves. However, the rocky formations in the western part of the beach were removed from the sea bottom in respond to customer needs in 1999. Altering the characteristics of the sea bottom topography, a serious erosion problem (retreating approximately 30 meters) was initiated at the beach mostly due to offshore movement of sands under wave action especially in winter season (Fig. 8). Remedial attempts utilizing artificial nourishment and construction of a groin of 25 meter long and 2 meter wide behind the pier could provide some amount of accretion at eastern part of the beach but did not work properly for the rest of the beach.

For the purpose of finding a consistent and effective remedial measure for the coastal erosion problem at Perissia Beach, site investigations and bathymetrical surveys were performed, wave climate of the region was determined and numerical modeling studies

were carried out for alternative solutions by Güler et al. (2008). The wave climate of the region was determined carrying out a wave hindcasting study using the hourly average wind data measured by the nearest coastal meteorological station (Alanya Meteorological Station) for the years 1993-2004 which is obtained from DMİGM. The dominant wave directions were found to be South-South-East, South and South-South-West directions. Alternative solutions for the coastal erosion problem including hard and soft measures were discussed in detail in view of effectiveness.

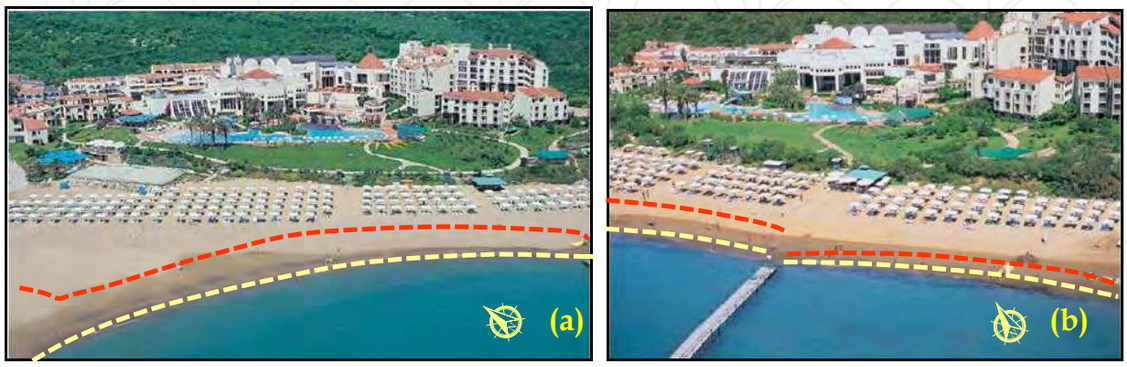


Fig. 8. Perissia Hotel Beach at Side; shoreline retreat due to erosion over the years, red line shows the shoreline position approximately in August, 2006 (a: view of the western part of the beach from sea, b: view of the eastern part of the beach from sea; pictures from 1999; Güler et al., 2008)

The coastal protection system proposed was composed of several measures including both hard and soft measures, considering geomorphological features of the beach, sediment trapping capacity, possible harm to neighboring beaches and also scenic beauty. It was planned to be applied in three stages. The first stage is the construction of a 40 meters groin, 15 meters of which is at the land part, at the western border of the beach in addition to the existing groin at the mid-beach. This groin was planned to be a low crested structure, which will be at mean sea level at high tides and during storm events. It was actually planned to use the existing rocky formation at the western edge of the beach and strengthen it up to 25 meters seaward from the beach. The second stage includes the construction of a 40 meters submerged breakwater which is 30 meters away from pier and 60 meters offshore at the western part of the pier with a crest height 1 meter below still water level. The final stage was the nourishment of the western part of the beach between to groins (one existing and one constructed) at least 30 meters from the shoreline having a median grain size of around 1 mm (approximately 7000 m^3 of sand). It was left optional to utilize gabion structures in front of the nourished area against wave action in cross-shore direction. The median grain size diameter (D_{50}) was found as 0.15 millimeters from the sieve analyses of the sediment samples taken from the site (Ergin et al., 2006). The summary of the proposed solution is shown in Fig. 9.

A recent site investigation was performed in January, 2010. Applied measures are investigated and the current shoreline position was measured. During the site investigation, it is found that the implemented remedial measures differ than the proposed coastal defense system presented above slightly in dimension and construction sequence:

- Western groin is extended to 15 meters towards the sea.
- Existing 25 meters long groin underneath the pier was removed for aesthetic reasons.
- Approximately 6000 m^3 sand artificially added to the beach.
- The construction of the submerged offshore breakwater is delayed. The existing 25 meters long groin underneath the pier was removed.

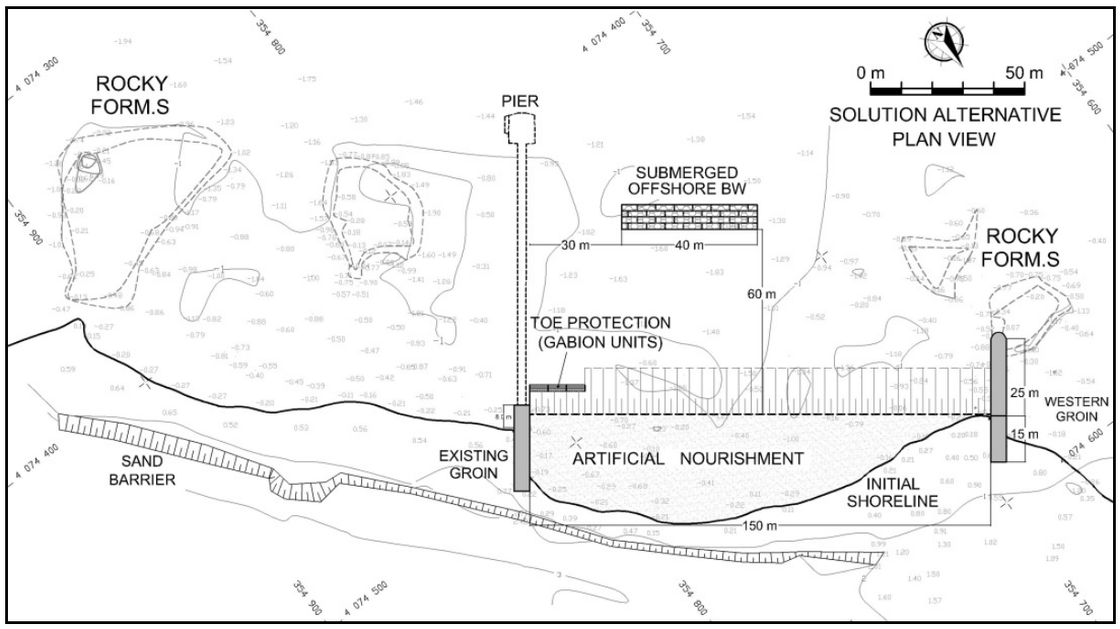


Fig. 9. Proposed remedial measures for the shoreline erosion at Perissia Hotel Beach at Side and nearshore bathymetry (18.11.2006)

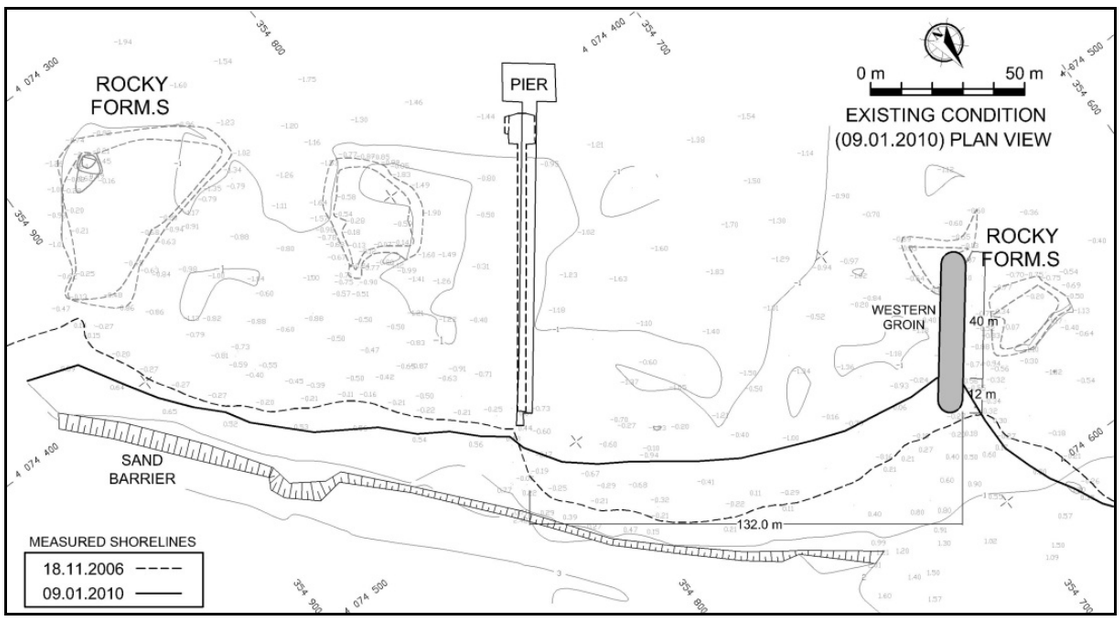


Fig. 10. Condition of Perissia Hotel Beach at Side at 09.01.2010, applied coastal defense structures and nearshore bathymetry (18.11.2006)

The implemented remedial structures and condition of the beach during the site investigation are shown in Fig. 10. In this study, the implemented measures are discussed and shoreline changes between the years 2006-2010 are studied with one-dimensional numerical shoreline change model, CSIM.

4.1 Wave climate study: Side, Perissia Hotel Beach

To obtain the wave climate in the last ten years at Side, a wave hindcasting study is performed using the using hourly average wind data measured at 10 m above ground level by Alanya Meteorological Station between the years 1993-2004, obtained from DMİGM. For each direction, the effective fetch distances are determined for each direction with an effective generation area as a sector from -22.5° to +22.5° totally covering an area of 45° with 7.5° intervals, from South-East to West using the navigation maps of SHODB. The effective fetch directions and the effective fetch distances are shown in Fig. 7.

Using the effective fetch distances and the wind data obtained from Alanya Coastal Meteorological Station, deep water wave parameters (H_{s0} : deep water significant wave height, T_s : significant wave period) are obtained for the storms occurred during 12 years (1993-2004) by using the numerical model, W61, developed at Middle East Technical University, Department of Civil Engineering, Ocean Engineering Research Center. The characteristic deep water wave steepness value (H_{s0}/L_0) for the project area is obtained as 0.042 from deep water significant wave heights and deep water wave lengths computed from corresponding significant wave periods of each individual storm.

The wave conditions in Mediterranean Sea are moderate compared to the Black Sea. There is not a very significant difference between the wave characteristics and their occurrences between the seasons. Therefore, for Side, an annual based wave data input sequence is prepared for the numerical model to carry out long term wave statistics and determination of representative deep water significant wave characteristics for each direction. The wave data input for the model consisting of representative wave heights, corresponding periods and annual frequencies for all directions are presented in Table 2.

Directions	$H_{rs,0}$ (m)	T_s (sec)	Δt (hrs)
W	0.77	3.42	11
WSW	0.84	3.59	119
SW	0.88	3.67	53
SSW	1.01	3.93	137
S	0.97	3.84	229
SSE	1.04	3.99	26
SE	0.88	3.67	4

Table 2. Representative wave conditions and annual occurrence durations for all directions (Güler et al., 2008)

4.2 Numerical modeling study: Side, Perissia Hotel Beach

For the numerical modeling of the applied coastal defense measure system, the shoreline at western part of the beach is assumed to be nourished till the end of the newly constructed groin at land side which makes approximately 5600 m² of nourishment area. It is assumed that the shoreline remained same till the construction of the western groin. The length of the western groin is 52 meters from the nourished shoreline of 2008. The rocky formations shown in Fig. 9 and 10 are modeled as submerged offshore breakwaters at various crest heights, widths and distances from shoreline. Submerged breakwaters or such rocky formations close to still water level force the waves to break and dissipate their energies and create sheltered wave fields behind the formations. The wave heights behind such structures are computed using diffraction terms and transmission coefficients ($C_t=H_t/H_i$, ratio of transmitted wave height to incoming wave height) in CSIM. Depending on geometrical properties of the rocky formations given in the bathymetry measured in 18.11.2006, the transmission coefficients are roughly computed using below given equation (CIRIA, CUR, CETMEF, 2007) where R_c is the crest height from still water level and H_s is the significant wave height in at the toe of the structure.

$$C_t = 0.46 - 0.3R_c/H_s \qquad \text{for } -1.13 < R_c/H_s < 1.2 \tag{9}$$

The berm height (B), the landward end of the active profile, is assumed as 1.5 meters according to the site investigations. The sequence of wave data input (Table 2) in the simulation starts from W to SE directions for each year. For the calibration of the numerical model, the shoreline retreat after the construction of the groin till January, 2010 has been used.

Alpar et al. (1995) gives a 0.21 meters spring range of semi-diurnal tidal variations together with a seasonal variation of 0.18 meters sea level variations for Antalya based on sea level measurements of Antalya station for the years 1935-1976. The effects of sea level variations are disregarded in the computations.

The computed position of shoreline using CSIM and measured shoreline positions for the dates 18.11.2006 and 09.01.2010 are given in Fig. 11.

4.3 Results and discussion: Side, Perissia Hotel Beach

As seen from Fig. 11, the shoreline change computed by the numerical model in the eastern part of the beach is in agreement with the measured shoreline both qualitatively and quantitatively. The erosion at the eastern part of the beach is due to the removal of the 25 meters long groin underneath the pier.

For the western part of the beach, the shoreline changes are in agreement qualitatively but not quantitatively. The accretion and erosion at the toe of the groin both on the eastern and western sides are in very good agreement with the measured shoreline in 2010 both qualitatively and quantitatively. The main reason for the quantitative disagreement between measured and computed results for the western part of the beach is due to disregarding the cross-shore sediment transport in the computations. In this area, which is bounded with the pier and the groin, the cross-shore sediment transport appears to be more effective and has to be included in the computations at the further stages of investigations. The excessive erosion in the western part of the beach could be reduced with the construction of the proposed submerged offshore breakwater (Fig. 9) which will reduce the cross-shore sediment transport.

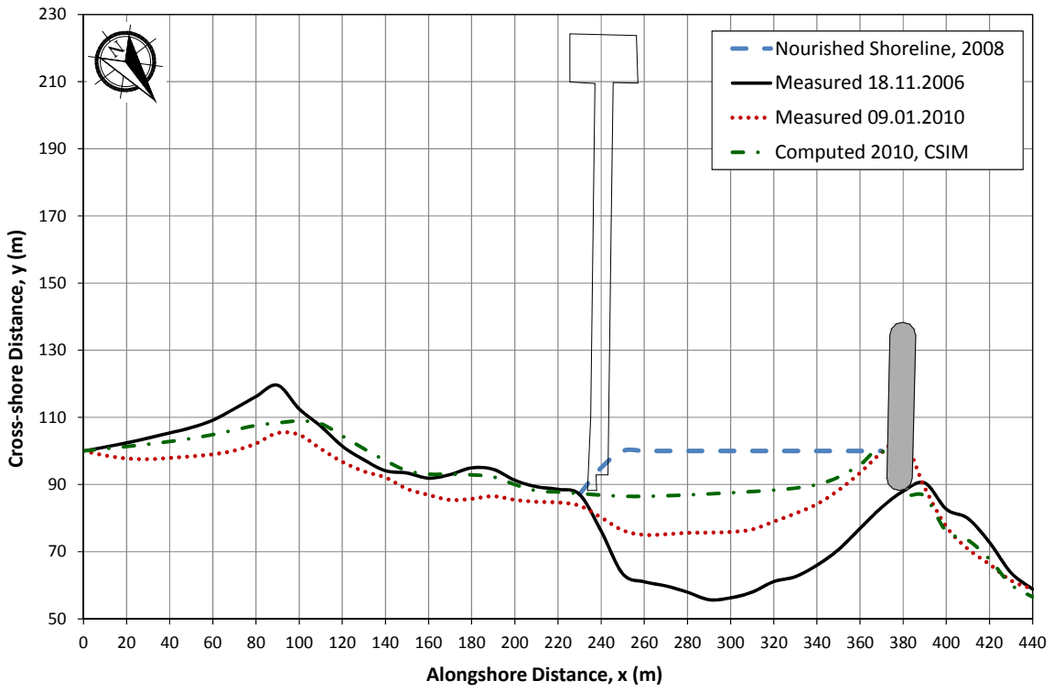


Fig. 11. Perissia Hotel Beach at Side; shoreline retreat due to erosion over the years, red line shows

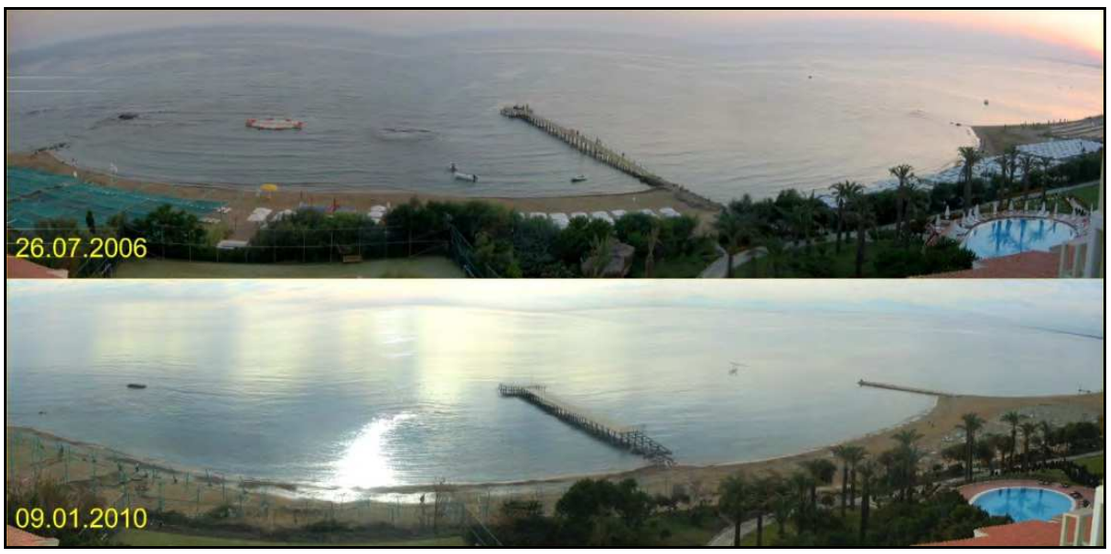


Fig. 12. Two panoramic views of the Perissia Hotel Beach at 26.07.2006 and 09.01.2010 showing the before and after remedial measures

The coastal erosion problem faced with in Perissia Hotel Beach is a very important example of the erosion caused by alteration and harming of the natural balance in a coastal area, which results from damaging and removing the rocky formations, naturally acting as submerged breakwaters or reefs and thus, protecting the beach against wave action. Such an application has initiated and accelerated the erosion at Perissia Hotel Beach. In Fig. 12, the

panoramic views of the beach at two different dates are given. The restoration of the western part of the shoreline is clearly seen in these pictures. However, the shoreline in eastern part started to retreat in return due to the removal of the groin underneath the pier. This retreat may be controlled by the implementation of the proposed remedial structures such as submerged offshore breakwater.

5. Conclusion

In this chapter, governing parameters of the geomorphological evolution of the coastal areas are briefly discussed in the view of sediment budget concept. Numerical modeling of the sediment transport mechanisms in coastal areas is discussed putting emphasis on one-dimensional numerical modeling which has been used as an effective tool by scientists and engineers for years to understand and predict shoreline changes in long term due to coastal erosion problems. In the case studies given above, the general approach for predicting shoreline changes at coastal areas, where human induced coastal erosion has become a chronic problem, is discussed. The Kızılırmak River mouth is a typical example of the effects of flow regulation structures on rivers and wetlands of alluvial plains fed by these rivers and resulting in severe coastal erosion problems in return. While silting of the reservoirs decreases the economic life of them, the coastal areas at downstream of these reservoirs hunger the silted material and degrades day by day. Perissia beach case is again another typical example of direct human intervention on the geomorphological and coastal-hydrodynamic processes at the beaches and resulting in erosion problems. These two cases stress the importance of the sustainable use of the resources in the long term for the benefit of mankind and of developing more sophisticated tools and methodologies to provide a better understanding and prediction capabilities of the adverse effects of anthropogenic activities at coastal areas.

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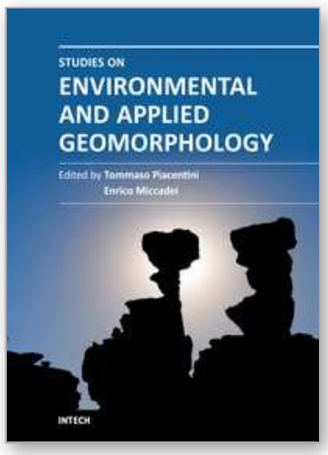
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