Comandatuba Beach Resort

Coastal dynamics and shoreline protection

Modeling study

WATER • ENVIRONMENT • HEALTH

Biomonitoramento
Final Report
February 2009

Comandatuba Beach Resort

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1 INTRODUCTION AND BACKGROUND

Since several years beach erosion has been observed along the property of the Trans américa Hotel located on Ilha da Comandatuba-BA. Previous analyses (Ref /1/, Ref /2/) have shown that the beach erosion is episodic and an analysis of historical photos showed a general tendency of mild accretion along the island. Figure 1.1 shows the beach in front of the hotel during a storm surge in September 2006.

Figure 1.1 Photographs showing the beach in front of the hotel during the surge in September 2006. Source: Ref /1/

The photographs show that the water level was so high that waves could pass across the backshore and attack the dune. This event has caused damage to the coastal infrastructure and has left a large beach scarf that is clearly visible in the photographs.

Beach erosion is a process that acts on various time scales. Storm surges (ressacas) act on short term time scales of hours or days. When large-scale meteorological phenomena, such as cold fronts, coincide with high tide and large waves then considerable shoreline erosion can occur during a short time. On a longer time scale effects such as variations in meteo-marine conditions may become important. From several locations it is known that time variations in offshore wave conditions occur. Such variations may occur on time scales of seasons, years or event decades.

One of the key problems along the shore in front of the hotel is that there is an insufficient volume of sand above the high water line in front of the property (buildings, pools and other facilities) that acts as a natural buffer and can absorb the natural fluctuations of the shoreline.

Possibly the observed beach erosion is also related to the dynamics of the delta in the mouth of Rio Comandatuba in the northern end of the Island, Ref /2/. The river mouth has migrated along the shore during the last decades. In case of a sudden breach of the river channel, the ebb tidal delta in front of the mouth will undergo changes. The old ebb delta will erode as no new sediment is brought to it by the river. A new delta is formed in front of the new breached channel. This process may cause variations in the volume of sediment that is bypassed along the river mouth. The delta acts as a form of barrier for littoral transport, similar to a coastal structure. When the delta erodes, its resistance to the passage of the littoral transport will decrease. This results in shoreline erosion along the updrift river margin. The erosion is gradually migrating towards South and may affect the shoreline behavior at the resort. The erosion will gradually decrease and will halt when the new delta has reached its equilibrium form and a new balance in littoral sediment transport has been established.

In this analysis a number of shoreline protection concepts have been identified and tested through mathematical modeling. The study was supported by field data collected along the Comandatuba Island during the course of the study. Extensive use was made of the findings of the site visit by Prof. Landim and reported in Ref /2/ and Ref /3/.

2 OFFSHORE WAVE CONDITIONS

The basis for the present analysis of wave conditions consists of offshore wave data provided by the global wave model, operated by the United Kingdom Meteorological Office (UKMO). In this model wave conditions are simulated based on the variation of wind fields and air pressure. The data used in this analysis are based on the model grid point located at 16.2^{o}S, 38.1^{o}W, see Figure 2.1. The data covered the time period from June 1991 to September 2008 and provide wave parameters at six-hour intervals.

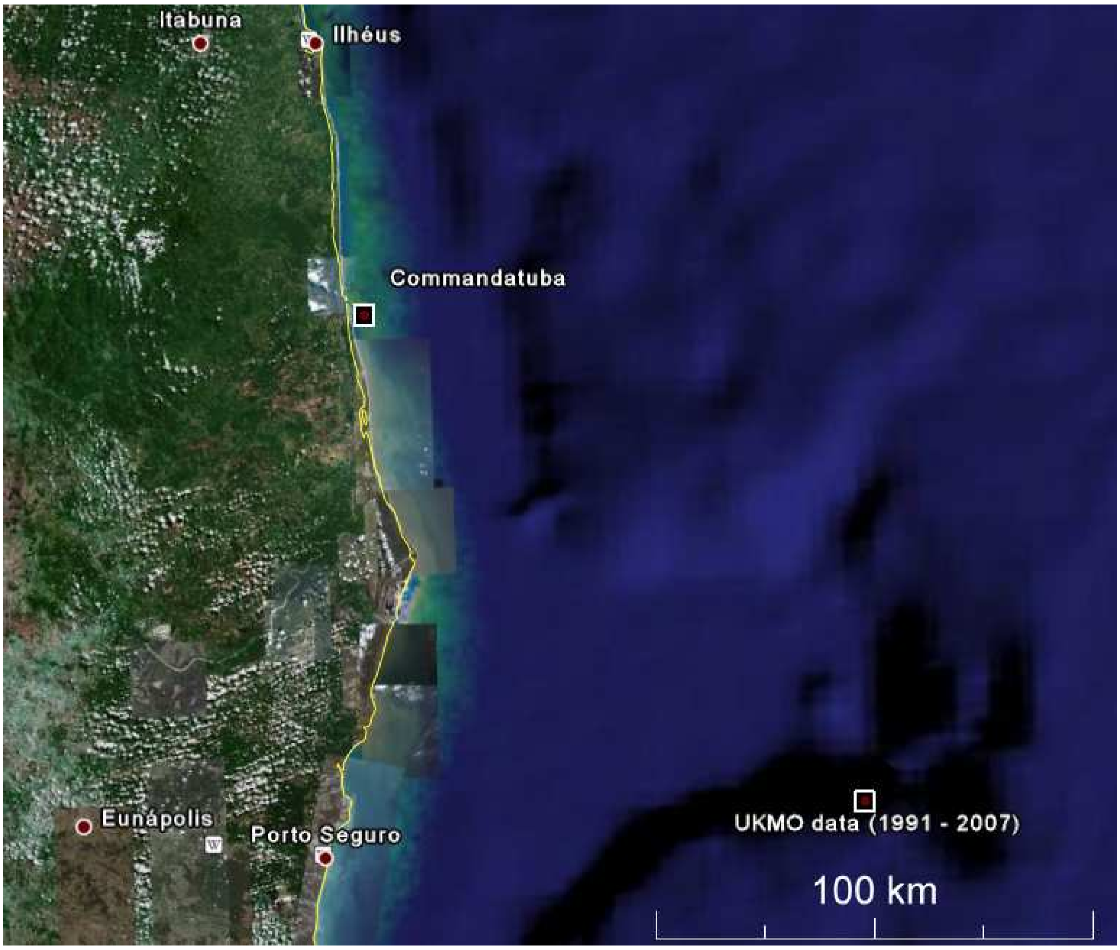


Figure 2.1 Position of the UKMO offshore grid point (16.2 ^{o}S, 38.1 ^{o}W) and the project site

2.1 Offshore Wave Statistics

Figure 2.2 shows the distribution of offshore wave energy as function of the wave height. The values represent the contribution to the time-averaged wave energy in the offshore zone calculated over the entire period covered by the data. The figure shows that most offshore wave energy is carried by waves with heights between 1.5 and 2 m. Furthermore, it is noted that only a small part of the total offshore wave energy is represented by waves higher than 4.0 m.

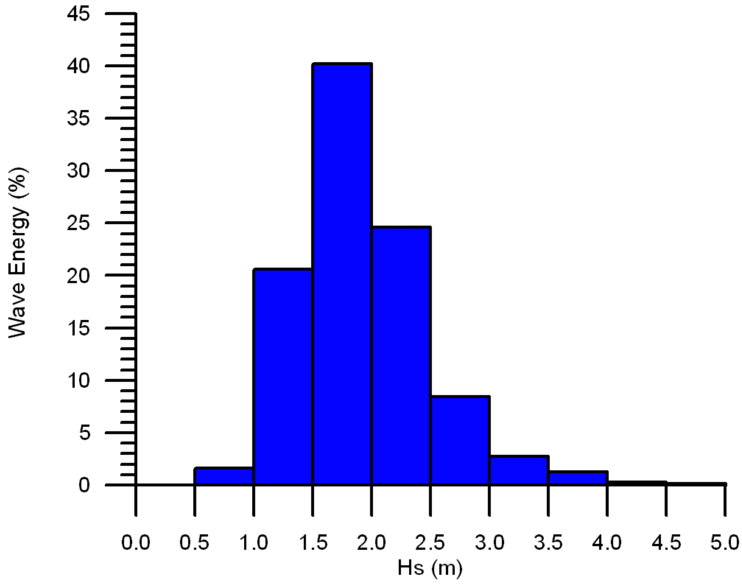


Figure 2.2 Offshore Wave energy distributions per wave height interval

Similarly, the statistics for the mean wave periods, Tz, were calculated, see Figure 2.3. The total amount of wave energy was calculated for discrete wave period intervals. It was found that most of the offshore wave energy occurs for waves with periods between 7s and 8s. Only a small percentage of the annual offshore wave energy is carried by waves with periods longer than 10s.

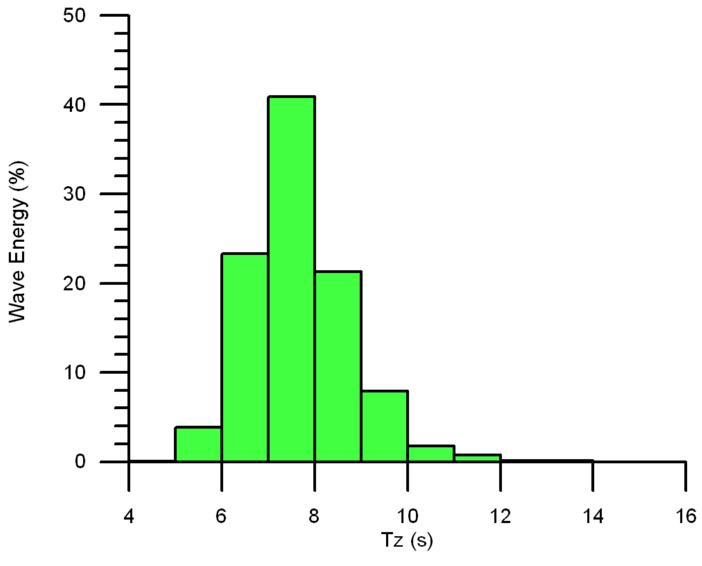


Figure 2.3 Time-averaged distribution of offshore wave energy per wave period interval

The distribution of wave energy per wave direction interval is shown in Figure 2.4. The data indicate that the dominant offshore wave direction is SSW. A small but significant local maximum in wave energy is observed for waves from NE-ENE.

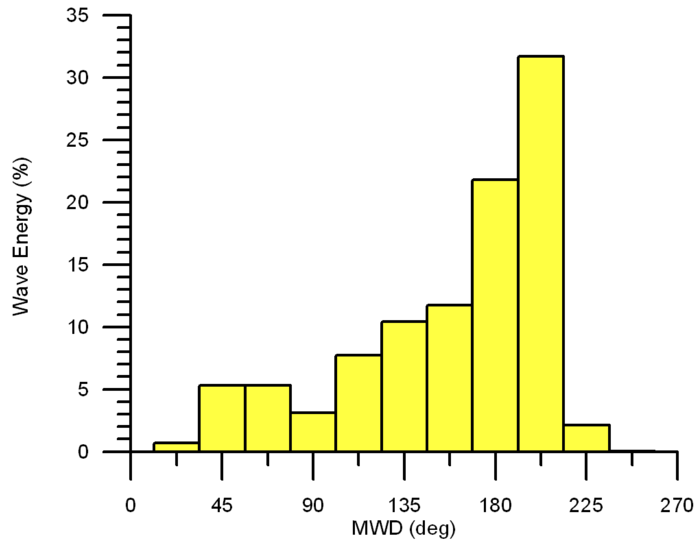


Figure 2.4 Time-averaged distribution of offshore wave energy per wave direction interval

2.2 Time Variations in Offshore Wave Parameters

From other DHI studies performed along the coast of Brazil, it is known that temporal variations in wave conditions occur on time scales that are important for the present project. In order to illustrate such time variations, a few representative wave parameters were calculated for each year during the period of data coverage. The mean wave height H_R, wave period T_R and wave direction α_R were calculated as:

$$H_R = \sqrt{\overline{H_s^2}} \qquad (2.1)$$

$$T_R = \frac{\overline{T_z H_S^2}}{\overline{H_S^2}} \qquad (2.2)$$

$$\alpha_R = \frac{\overline{\alpha H_S^2}}{\overline{H_S^2}} \qquad (2.3)$$

The representative wave period and wave angle were thus calculated as averaged values, weighed by the wave energy. The calculated time variation of the representative wave height, H_R, is shown in Figure 2.5.

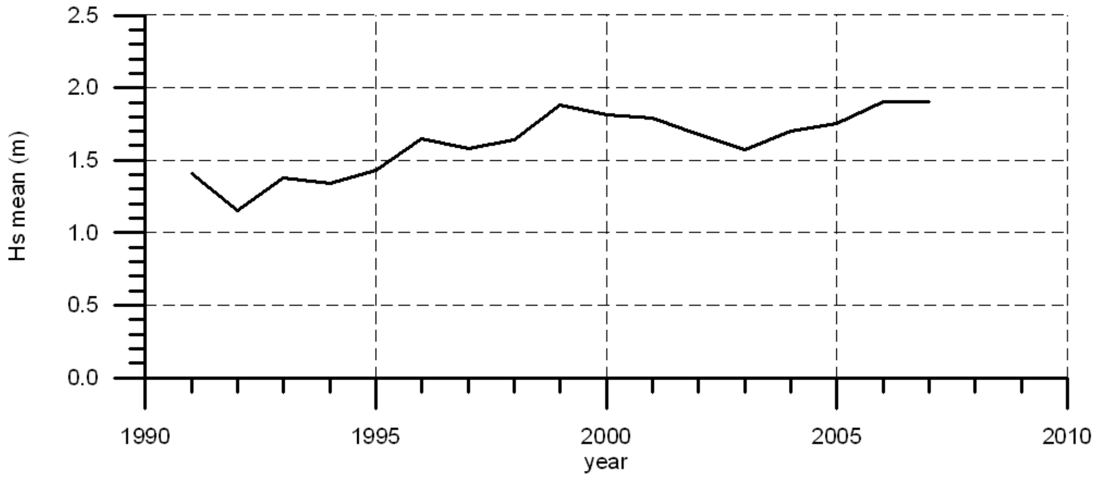


Figure 2.5 Average offshore significant wave height during the period 1991-2007

The time covered by the wave data (16 years) is too short to draw conclusions about trends towards permanent changes in wave height, but clearly some fluctuations on time scales of 5 to 7 years can be observed. The observed fluctuations in the offshore wave height are in the order of 0.3 m. A gradual increase in wave height of approximately 3 to 4 cm/yr was observed during the period covered by the data.

The average representative mean wave period, T_R, is shown in Figure 2.6. The time variations are small, with amplitudes smaller than 0.5s, and are not expected to have a significant impact on the project site. On average the mean wave period has increased with approximately 1s over the past 16 years.

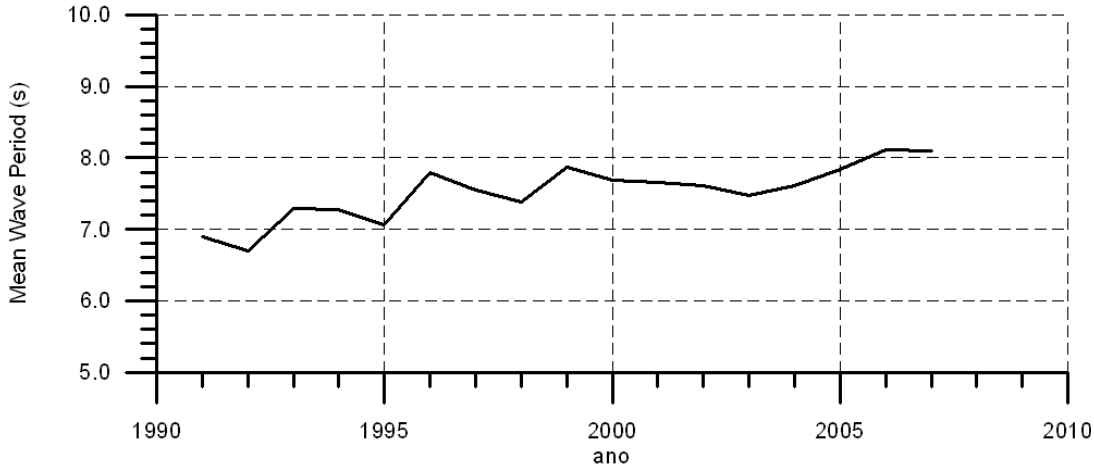


Figure 2.6 Average mean wave period T_R for the period 1991-2007

The time variation of the representative wave direction, α_R, is shown in Figure 2.7. The blue line represents a linear fit through the data. The analysis indicates significant fluctuations in the offshore wave direction. The fluctuations have amplitudes of ± 5 to 10 degrees. Besides the fluctuations the mean offshore wave direction seems to have shifted clockwise by approximately 25 degrees during the period covered by the data. This corresponds to an average change of around 1.5 degree/year, which is a considerable change and will have an impact on the shoreline.

On the basis of previous studies performed along the coast of Brazil, it is believed that the observed changes are, at least partly, due to periodic variations in wave conditions. Such variations appear on time scales of years to decades and are related to phenomena that cause variations in climatic conditions such as El Niño and La Niña. The effect of climate changes due to global warming cannot be confirmed, nor excluded on the basis of this analysis. The effect of the gradual shift in wave direction is mainly important for the plan stability of the beaches adjacent to the project site.

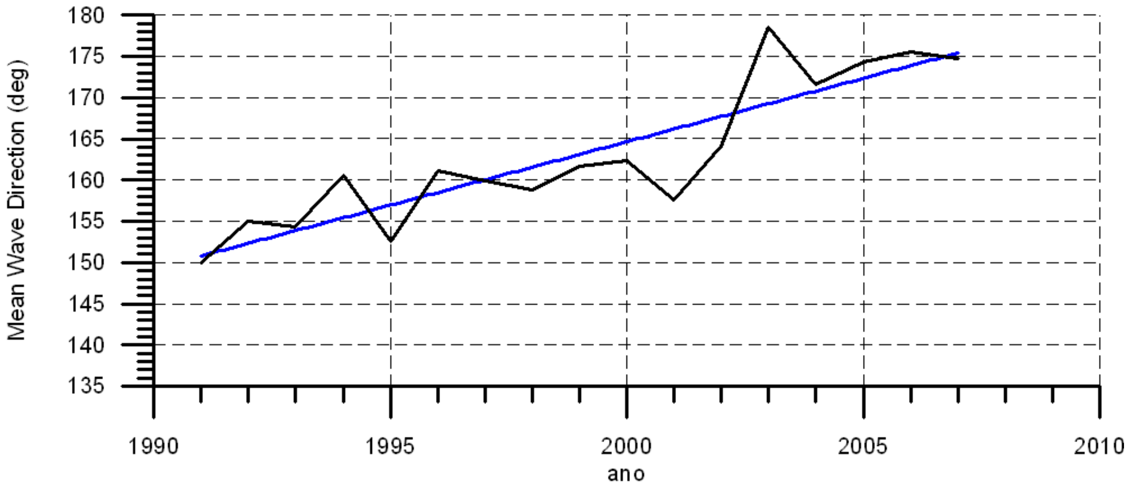


Figure 2.7 Average offshore wave direction for the period 1991 – 2007

Fluctuations in offshore wave conditions were also observed on shorter time scales. Figure 2.8 to Figure 2.10 show the monthly averaged wave height, - period and – direction. Average offshore wave heights seem to vary from around 1.5 m in December to February to around 1.8 m in May to September. Similarly, the average wave period varies from around 7s to 8s. Very clear seasonal variations are observed in the offshore wave direction. In the summer months the average wave direction is SE. In the winter the average direction is SSE to S.

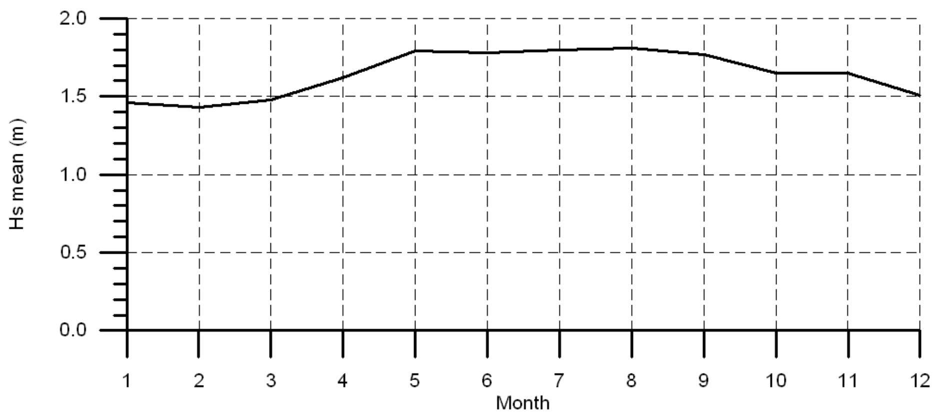


Figure 2.8 Monthly averaged offshore wave height

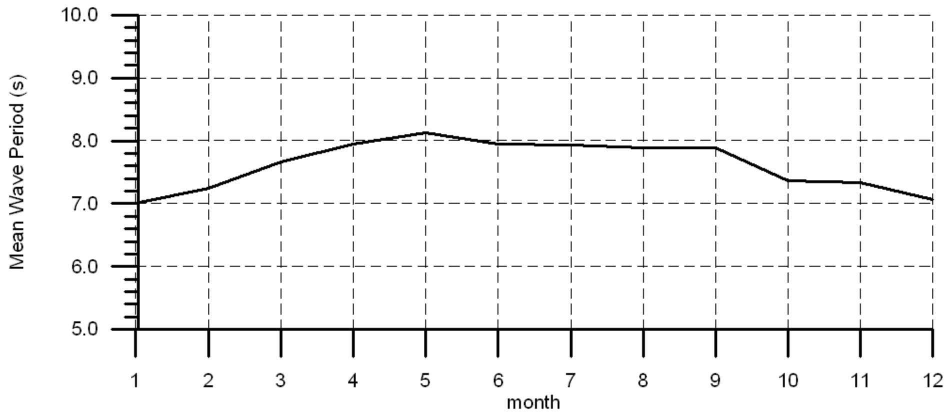


Figure 2.9   Monthly averaged offshore wave period

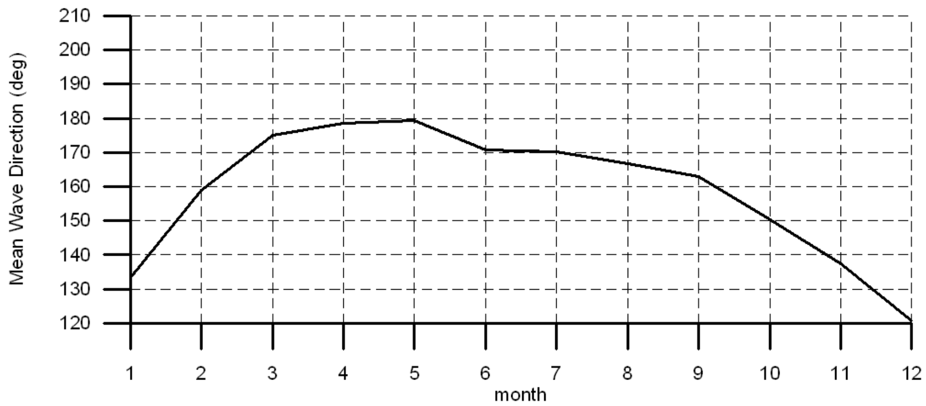


Figure 2.10   Monthly averaged offshore wave directions

3 WAVE TRANSFORMATION STUDY

The UKMO data covered the period 1991 – 2008 and consisted of continuous time series of sea and swell parameters (wave height, - period and mean direction).The validity of the UKMO data is limited to deep water. In order to retrieve nearshore wave conditions in front of the resort a wave transformation study was performed using DHI's spectral wave model MIKE-21 SW. This is a state-of-the-art model for waves in oceans and coastal areas. The wave transformation model includes all physical mechanisms that are relevant for the present application such as refraction, shoaling, and energy dissipation due to bed friction and breaking, and wind growth. More information about DHI models can be obtained at www.dhi.dk or upon request.

3.1 Model Set Up

Bathymetric data were derived from nautical charts, supplemented by recent bathymetric survey and beach profile data. The model bathymetry shows that the shoreline in front of the hotel is almost perfectly straight and that the depth contours are parallel to the coastline, at least until the 5 m depth contour. Figure 3.1 shows the model bathymetry used in the wave transformation study. The model covered an area of roughly 130km x 230km. A total of approximately 35,000 computation cells were used. Figure 3.2 shows a zoom of the project area.

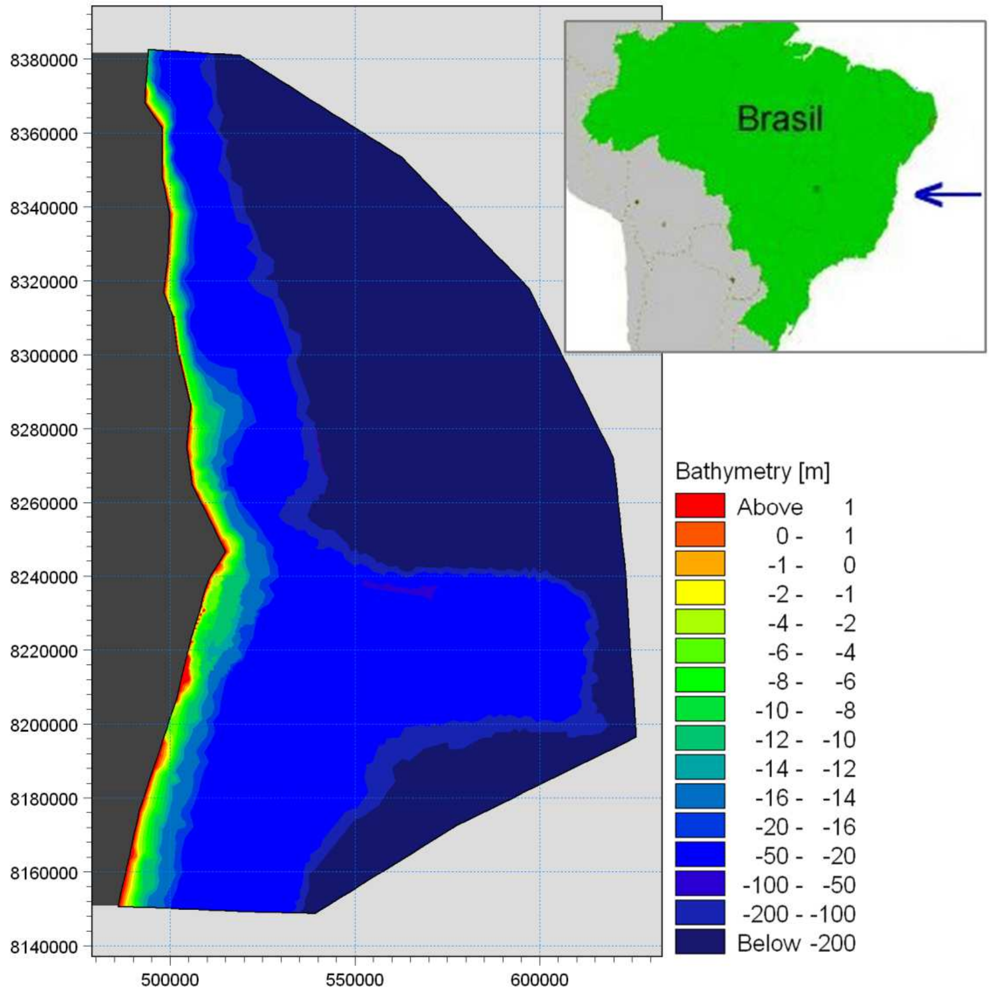


Figure 3.1 Wave model area

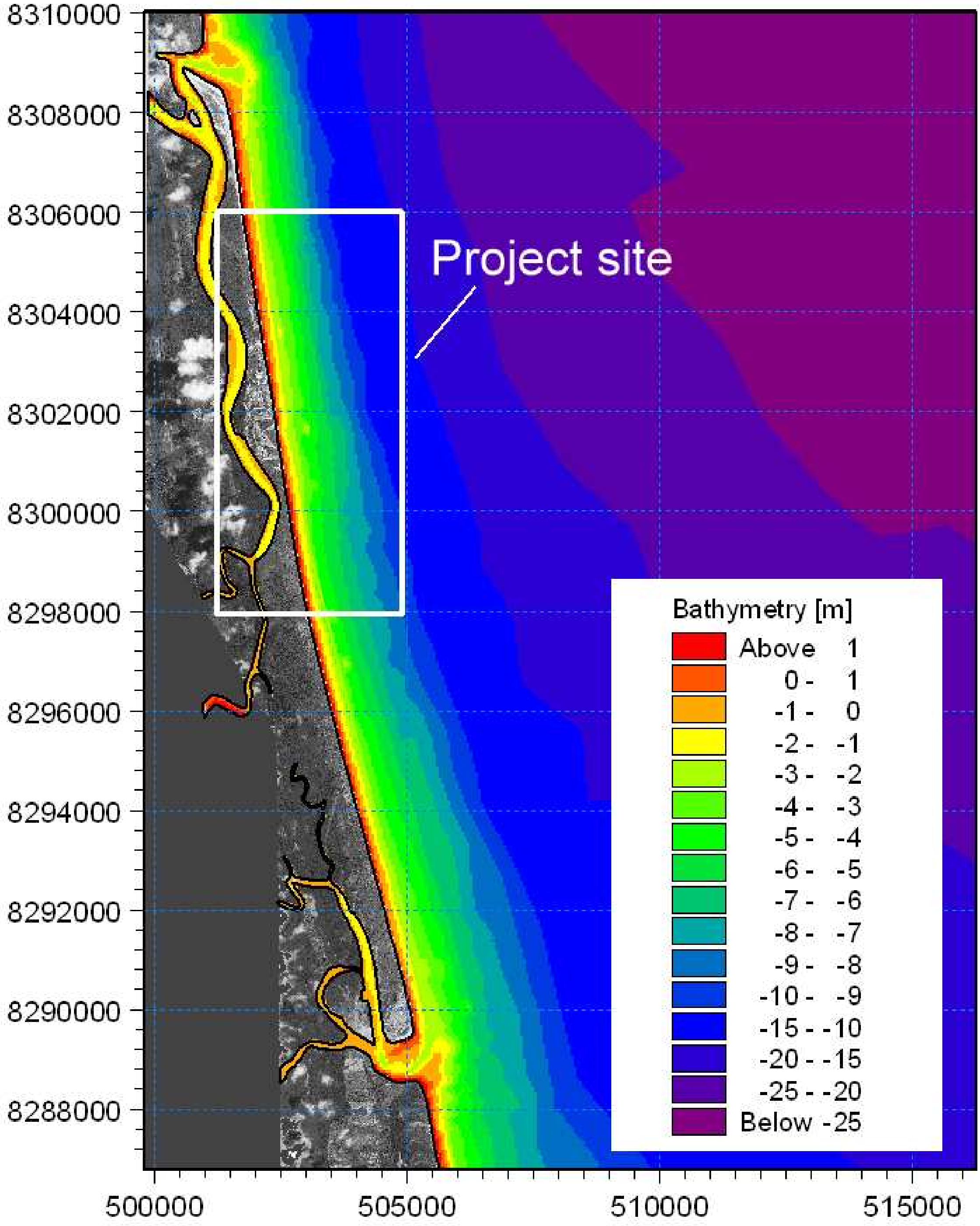


Figure 3.2 Zoom of the model bathymetry around the present project site

At the offshore model boundary, wave conditions derived from the UKMO data were specified. The nearshore wave climate was established by transforming the entire offshore time series of wave parameters to the nearshore zone. Sea- and swell components were transformed separately in the model. In the output locations the resulting wave was calculated by superposing the transformed sea and swell components. The resulting wave height was calculated from the sum of the sea- and swell contributions to the total wave energy:

$$H_{s,res} = \sqrt{H_{s,sea}^2 + H_{s,swell}^2} \qquad (3.1)$$

The resulting period was estimated as an averaged value of the sea and swell components, weighed using their respective contributions to the total wave energy:

$$T_{z,res} = \frac{T_{z,sea} H_{s,sea}^2 + T_{z,swell} H_{s,swell}^2}{H_{s,sea}^2 + H_{s,swell}^2} \qquad (3.2)$$

Finally, the resulting wave direction was estimated in a similar way from:

$$\tan\alpha_{res} = \frac{\sin\alpha_{sea} H_{s,sea}^2 + \sin\alpha_{swell} H_{s,swell}^2}{\cos\alpha_{sea} H_{s,sea}^2 + \cos\alpha_{swell} H_{s,swell}^2} \qquad (3.3)$$

It is noted that the calculated wave angle has only limited physical importance in cases with simultaneous occurrence of sea and swell where the energy carried in both components is more or less equal while the angles between the two components are large. In such cases, the wave direction was taken as the wave direction corresponding to the dominant component. The transformation simulations were carried out including wave breaking. The nearshore wave parameters were derived in a location in front of the hotel along the 15 m depth contour.

3.2 Model Calibration

Model calibration was performed upon measured wave parameters measured south of the mouth of the Jequitinhonha River in the period July/August 2006. The wave data were collected using an Acoustic Doppler Current profiler (ADCP). Model parameters were adjusted until a satisfactory agreement was found between model results and measurements. Figure 3.3 shows the comparison between simulated- and measured wave parameters.

Differences between model results and measurements were found not systematic. Deviations between model results and measurements may be caused by inaccuracies in three independent sources: 1) - The UKMO data specified at the offshore model boundary, 2) - The wave transformation model itself, and, 3) - The measurements.

It is important to note that the input data along the model boundaries do not consist of measured wave data but rather synthetically generated wave data, as derived from the UKMO model. Any disagreement between offshore boundary data with the true values in nature will be reflected in the present model results. The wave conditions were assumed uniform along the entire model boundary. Despite some discrepancies the agreement was generally found satisfactory for the present purpose.

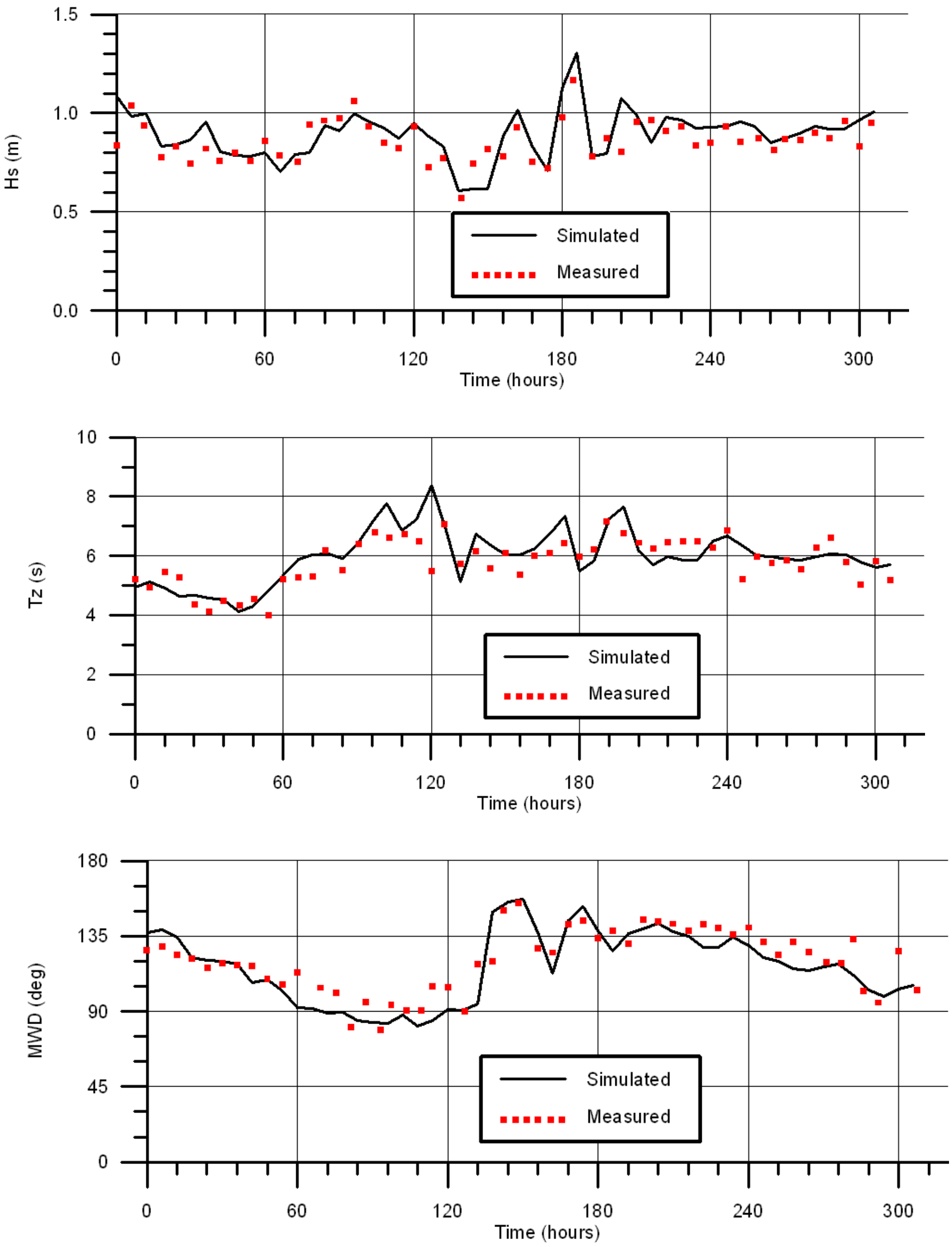


Figure 3.3 *Comparison between model results and measurements for July 2006. Top: Significant wave height. Middle: Peak wave period. Bottom: Mean wave direction*

3.3 Nearshore Wave Conditions

The calibrated model was applied to transform the time series of offshore wave parameters provided by the UKMO data to a nearshore position along the 15 m depth contour in front of the resort.

3.3.1 Annual Nearshore Wave Climate

Figure 3.4 shows the distributions of significant wave height, mean wave period and mean wave direction as calculated from the transformed time series. On an annual basis most wave energy reaches the shoreline through waves with a height between 1.00 m and 1.25 m. Waves higher than 2.5 m are rare. Similarly, most wave energy is carried by waves with mean periods of 6s-7s, periods higher than 10s occur rarely. The wave energy distribution per wave direction interval shows a maximum for waves coming from ESE.

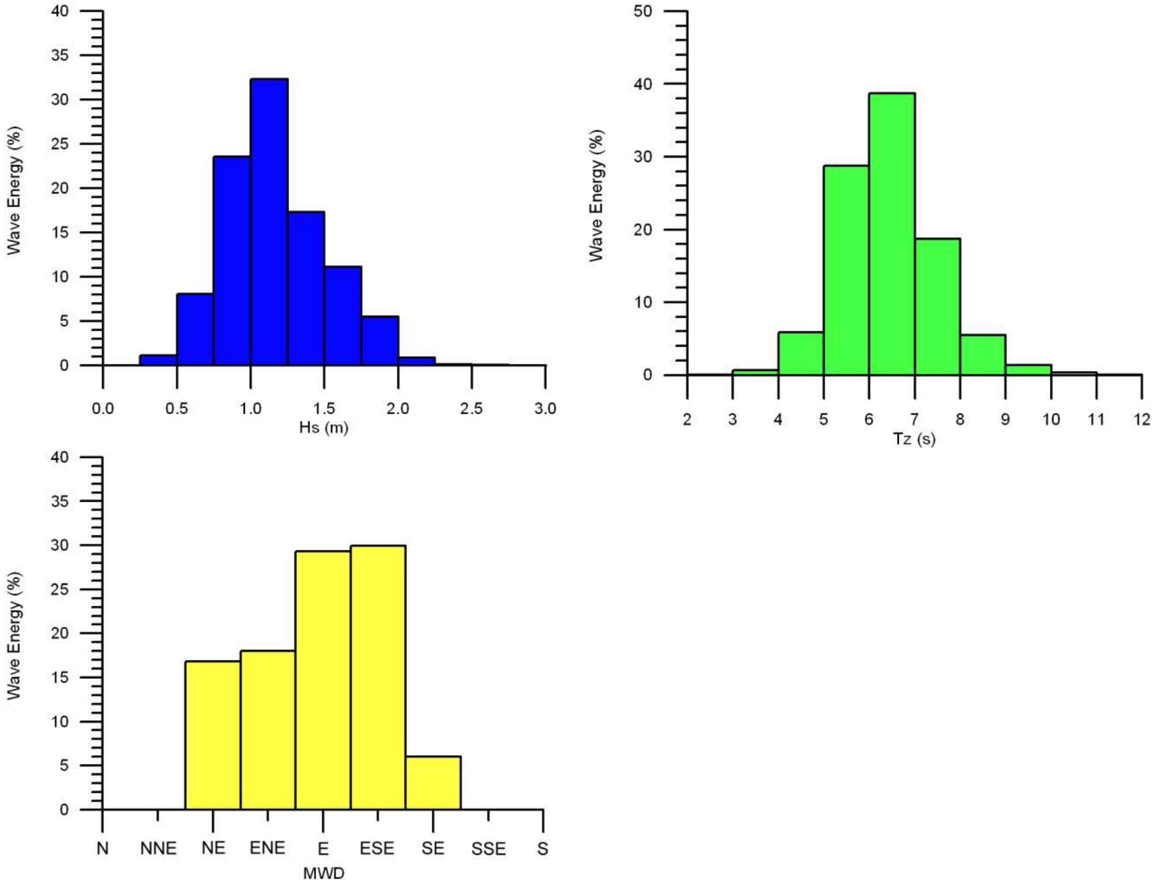


Figure 3.4 Time averaged distribution of wave energy per wave height interval (upper left), wave period interval (upper right), and wave direction interval (lower left)

3.3.2 Annual Variation in Nearshore Wave Conditions

In order to analyze possible fluctuations in wave conditions throughout the period covered by the data, representative wave parameters were calculated for each year in the period 1991-2007 (annual average data from 2008 were not included as only data until September 2008 were available at the time of writing). Figure 3.5 shows the time variation of the annually averaged significant wave height, mean wave period and mean wave direction.

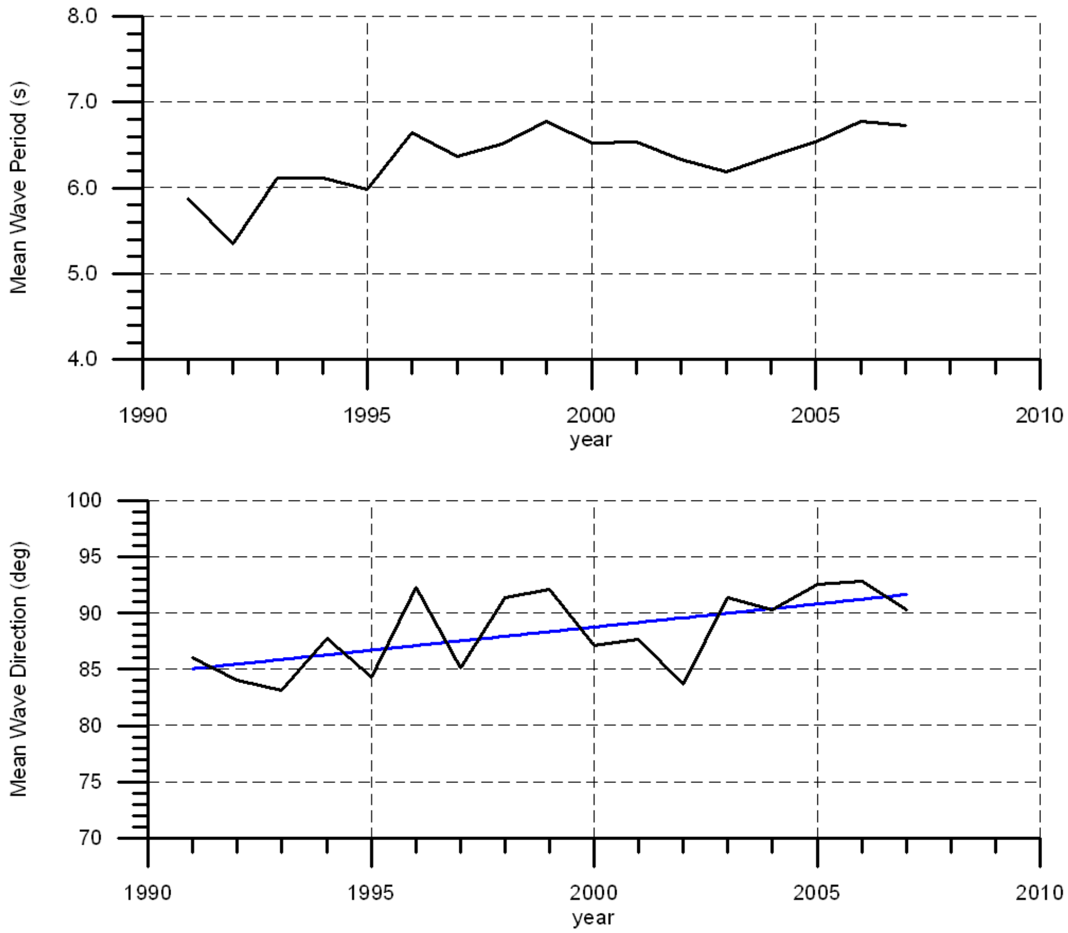


Figure 3.5 Average near shore wave parameters for each year during the period 1991-2007. Top: Significant wave height. Middle: Mean wave period. Bottom: Mean wave direction

The calculations indicate an average wave height of approximately 1.0 m to 1.2 m. Slight variations with amplitudes in the order of 0.2 m were observed during the period covered by the data. Similarly, the calculated average wave periods indicate some fluctuations in the mean wave period with maximal amplitudes of approximately 0.5s. Some significant changes were observed in the mean wave direction. In the early 1990s the mean wave direction was around 85 degrees. During the following decade, the wave direction gradually shifted clockwise and presently the mean wave direction is approximately 95 degrees. This is a change in wave direction of 10 degrees, which is considerable and is important for the shoreline dynamics of the present project site.

3.3.3 Seasonal Variations in Nearshore Wave Conditions

Also on shorter time scales significant fluctuations in wave conditions were observed. Figure 3.6 shows the wave roses for the four trimesters. A clear dominance of waves from SE is observed during the period April until September. During the summer months a relatively strong ENE component was observed. The seasonal variation in wave direction, and associated littoral sediment transport is very important for the behavior of the shoreline and the effects of human interventions such as groins.

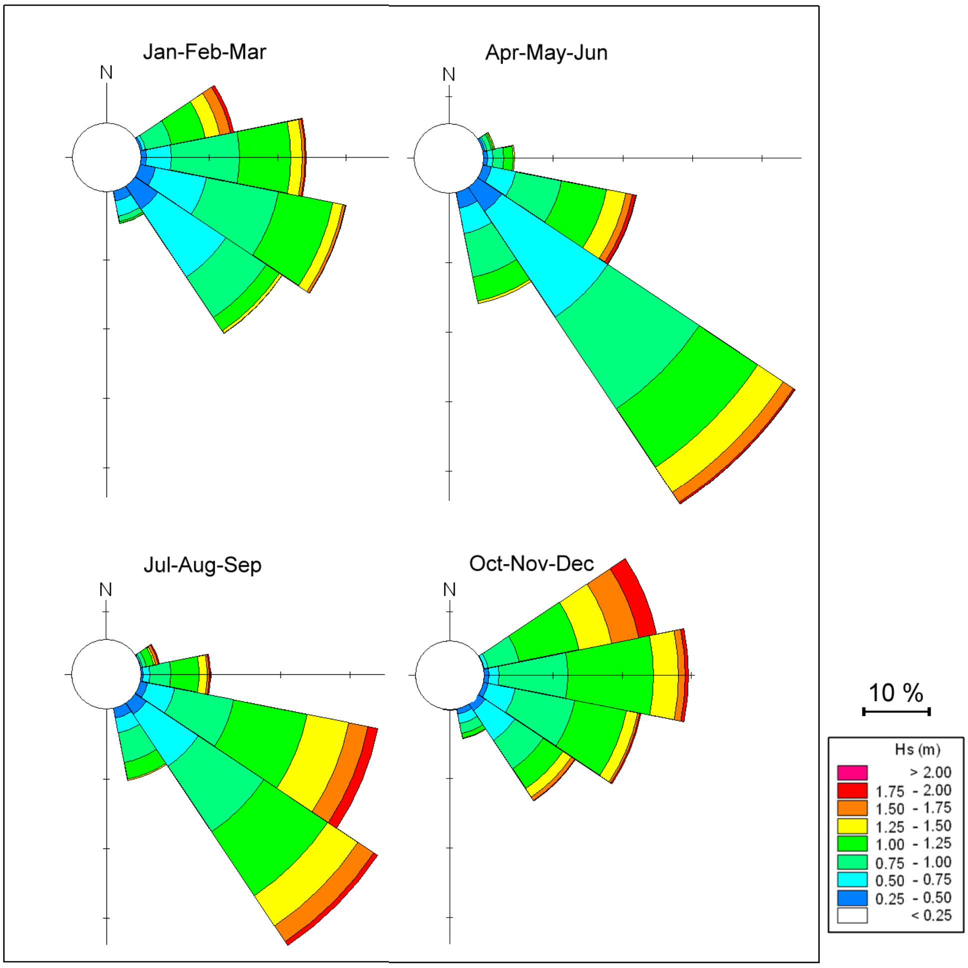


Figure 3.6 Wave roses for 4 seasons along the 15 m depth contour in front of the resort

Further analysis has shown that the highest average wave height occurs in the period August-November, the lowest wave heights occur in March and April as shown in Figure 3.7. The wave period also shows some variations throughout the year. The most important variation was observed for the mean wave direction. In the summer months, the average wave direction is around between 70° and 90°, whereas during the winter months it reaches values between 100° and 110°. This is a considerable difference, which is of major importance for the choice of human interventions.

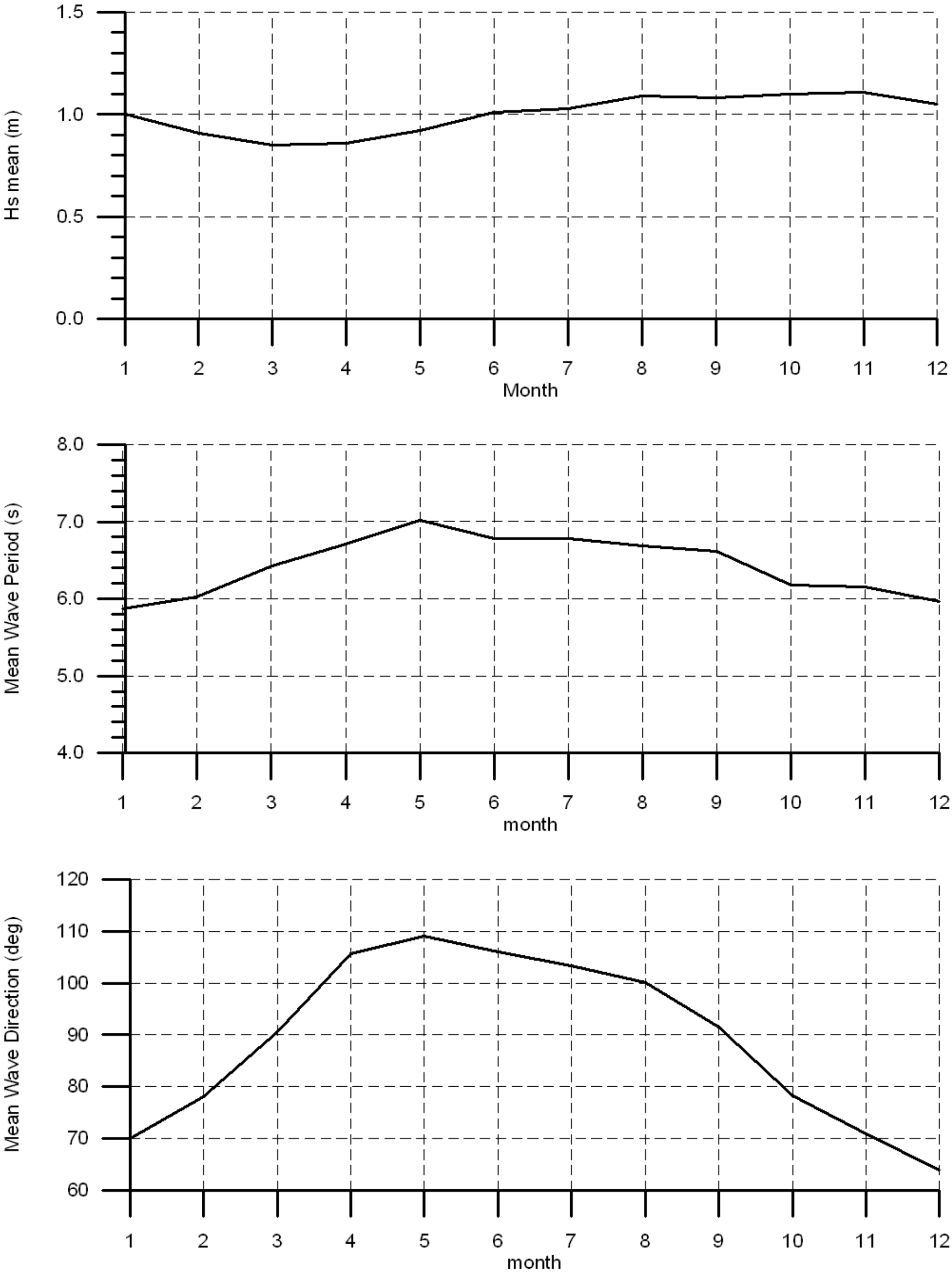


Figure 3.7 Monthly variation in nearshore wave parameters. Top: Significant wave height. Middle: Mean wave period. Bottom: Mean wave direction

3.3.4 Extreme Wave Heights

Extreme wave heights are important for the design of coastal structures. In order to derive estimates of extreme wave height a statistical analysis was performed on the simulated wave data. The exceedance probability of the wave height (from all directions) is shown in Figure 3.8.

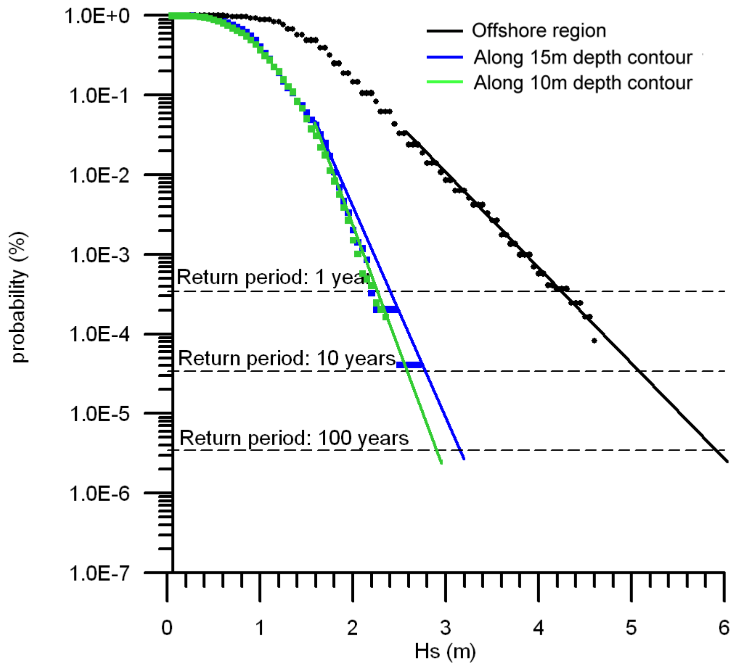


Figure 3.8 Exceedance probability of Hs for the offshore region (black), along the 15 m depth contour (blue), and 10 m depth contour (green)

The offshore wave heights with return periods of 1year, 10 years and 100 years were estimated as 4.2 m, 5.1 m, and 5.9 m, respectively. Similarly, for the inshore zone (water depth of 15 m) wave heights of 2.4, 2.8, and 3.2 were calculated for return periods of 1, 10 and 100 years, respectively. The extreme wave heights are listed in Table 3.1.

Table 3.1 Estimated extreme wave heights in various water depths and return periods

	Water depth		
Return period	Offshore	15m	10m
1 year	4.2	2.4	2.3
10 years	5.1	2.8	2.6
100 years	5.9	3.2	2.9

4 DYNAMICS OF CROSS-SHORE BEACH PROFILE

Short-term beach erosion often occurs during periods with combinations of high water levels (ressaca) and high waves. During these short period events relative large volumes of sand can be eroded from the beach, and deposited in the deeper parts of the beach profile, leading to a sudden retreat of the shoreline. During the following period with calm weather, the sand is gradually transported back to the shore and the original shoreline is re-established.

4.1 Local Water Level Statistics at the Project Site

The water level plays an important role in the process of coastal flooding and beach erosion during storms. The main effect of an increased water level is that the larger water depths allow larger waves to reach the shore, where they break and cause erosion. For the present project, the water level variations are caused by the effects of astronomical tide and wind on a regional scale and waves on a local scale.

Water level variations caused by waves include the wave set-up and wave run-up. Set-up is the variation of the mean water level that is caused by wave breaking. Wave run-up is the maximal level that an individual wave reaches while it rushes up the beach after breaking. The wave-induced water level variations are well understood and various expressions can be found in the literature. In the present analysis the definitions presented in the guidelines of the American FEMA are applied (FEMA, 2004). These guidelines are used to assess the risk of coastal erosion along the coast of California and are widely accepted.

The model calculates the wave run-up and set-up and the derived inshore wave data were used to calculate the statistics for the wave-induced water level variations. Unfortunately, no measured data of water level variations were available. Usually these water level variations are small compared to the wave generated water levels, and were neglected in this analysis. Water level variations due to astronomical tide were calculated from the tidal constituents.

Figure 4.1 shows the statistical distributions of water levels. The blue curve indicates the water levels generated by the astronomical tide, the green curve indicates the wave-induced water levels and the red curve represents the water levels due to the combined effect of waves and tides. Levels shown here are related to the mean water level according to tidal station Canavieras of FEMAR.

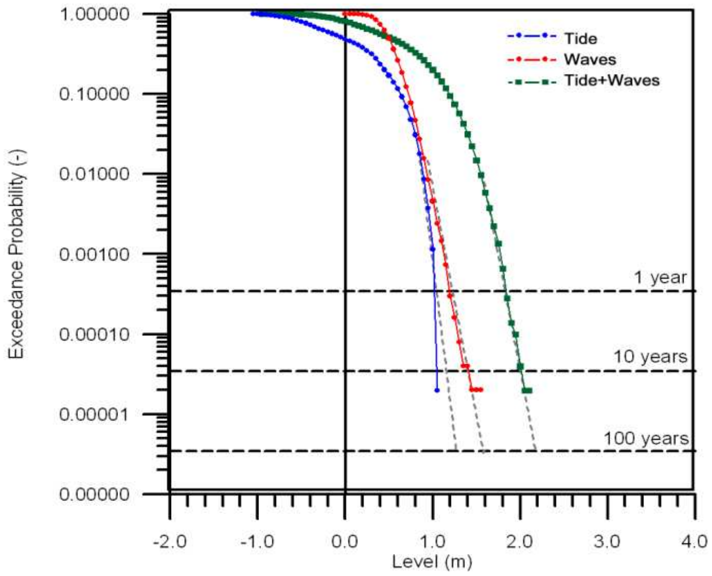


Figure 4.1 Statistical distribution of water levels due to astronomical tide, waves and combined tide + waves, related to Mean Water Level of FEMAR tidal station at Canavieras

From the statistical analyses, extreme water level statistics were derived. This was done by extrapolating the derived distributions manually to the values corresponding to return periods of 10 years and 100 years. The water levels for different return periods are listed in Table 4.1.

Table 4.1 Maximal water level elevation above mwl

Return Period	Tide	Waves	Combined
1 yr	1.05 m	1.20 m	1.80 m
5 yr	1.12 m	1.32 m	1.92 m
10 yr	1.15 m	1.40 m	2.00 m
50 yr	1.22 m	1.52 m	2.12 m
100 yr	1.25 m	1.60 m	2.20 m

4.2 Short- and Medium Term Profile Evolution

During recent years considerable effort has been put in developing mathematical models that describe the dynamic behavior of the cross-shore profile.

The present analysis was based on the model presented in Kriebel and Dean (1993). This model provides solutions to time-dependent beach profile response to storms in the form of a convolution integral. The model includes a time-varying erosion-forcing function and an exponential erosion-response function. The erosion-function includes wave- and water level data for the present project site. In order to derive analytical solutions, Kriebel and Dean (1993) represented a storm by means of an idealized hydrograph. In the present work the governing equations are solved numerically where no simplifications regarding wave- and water level conditions were made.

The basis for the convolution method is the observation that beach response to steady-state forcing conditions is approximately exponential in time. A linear differential equation governing the profile response to variations in water level is assumed to have the form:

$$\frac{dR(t)}{dt} + \alpha R(t) = \alpha R_{\infty} f(t) \qquad (4.1)$$

R = the shoreline position and t = time. R∞ represents the maximum potential shoreline advance or retreat if the beach were allowed to reach a new equilibrium relative to the water level and breaking wave conditions. The factor α = the characteristic rate parameter of the system, defined as α = 1/Ts. The expression for the characteristic time scale, Ts, was derived on the basis of measurements:

$$T_s = C_1 \frac{H_b^{3/2}}{g^{1/2} A^3} \left(1 + \frac{h_b}{B} + \frac{m x_b}{h_b} \right)^{-1} \qquad (4.2)$$

C_1 = constant, C_1 = 320, based on laboratory results (Kriebel and Dean, 1993)

Hb= Breaking wave height

G = Acceleration due to gravity

A = Profile constant

h_b = Depth at wave breaking

B = Height of the berm

M = Beach slope at the waterline

x_b = Width of the breaker zone

The profile constant, A, is based on the assumption that the shape of the cross-shore profile can be simplified according to the following expression:

$$h = AX^{2/3} \qquad (4.3)$$

Here h = water depth, X is cross-shore distance from the waterline. Dean (1987) found an empirical expression for A, entirely determined by the sediment properties, based on field data:

$$A = 0.067 w_s^{0.44} \qquad (4.4)$$

Here w_s is the sediment fall velocity (in cm/s).

The sediment fall velocity increases with grain sizes so for coarse sand a high factor A is calculated, which results in a steep profile. For fine sand, A is small and the profile will correspondingly be gentler. Figure 4.2 shows the shape of the cross-shore profile for various grain diameters.

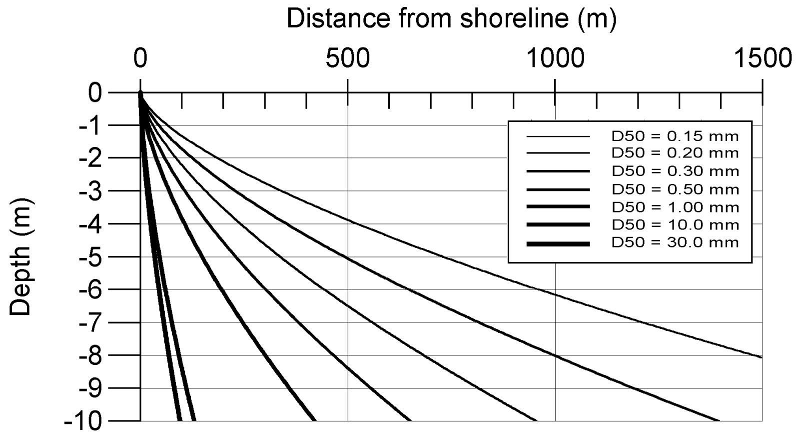


Figure 4.2 Equilibrium profiles for different grain sizes (from Mangor 2004)

The profile parameter A is derived from the measured profile using equation 4.3. The sediment grain size as such does not appear explicitly in the equations. The derived value for A corresponds to a grain size around 0.2 mm. This is in agreement with the grain size observed in the field.

The breaker height H_b and breaker depth h_b are calculated in the model using a simple wave transformation model that assumed linear refraction and shoaling. The breaker index was taken as 0.78, which corresponds to a commonly used value. The height of the berm B and the beach slope were derived directly from the measured beach profile.

The shape of the cross-shore profile is assumed to remain constant but the active part of the profile shifts according to the water level elevations (surge level). The speed of this profile shift is determined by the wave conditions. It is assumed that no sediment is lost across the shore, but that it is relocated due to the variations of wave conditions and water levels. The concept is illustrated in Figure 4.3.

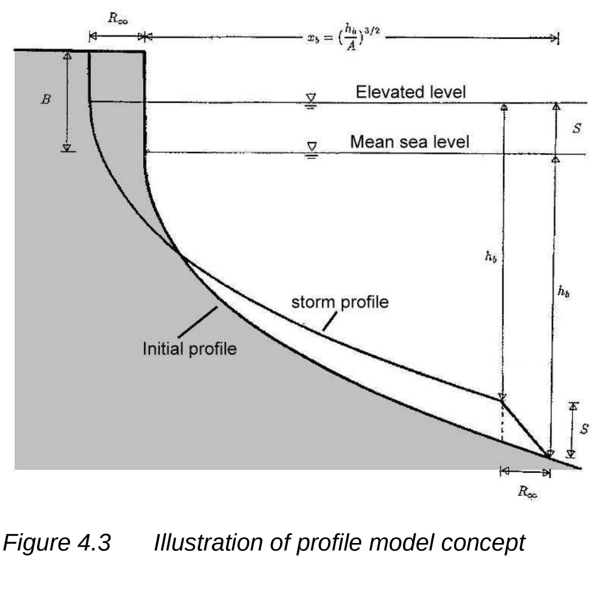


Figure 4.3 Illustration of profile model concept

From the above definitions the equilibrium shoreline position, which will be approached if the actual conditions were to remain constant, can be derived as:

$$R_{\infty} = \frac{S\left(x_b - \frac{h_b}{m}\right)}{B + h_b - S/2} \tag{4.5}$$

where S = water surface elevation (due to tide, surge, wave set-up and – run up).

In the simulations wave parameters were taken from the derived inshore wave conditions. The total water level elevation consists of 2 components: 1) astronomical tide and 2) wave set-up and run-up as presented in the previous section. The wave set-up and run-up were estimated from Ruggiero et Al. (2001). Equation (4.1) was solved numerically for the period 1991 until 2007. The model simulates the continuous movement of the shoreline. Figure 4.4 shows the simulated shoreline dynamics during the entire period covered by the data. Figure 4.5 shows a detail of the shoreline movement since 2005.

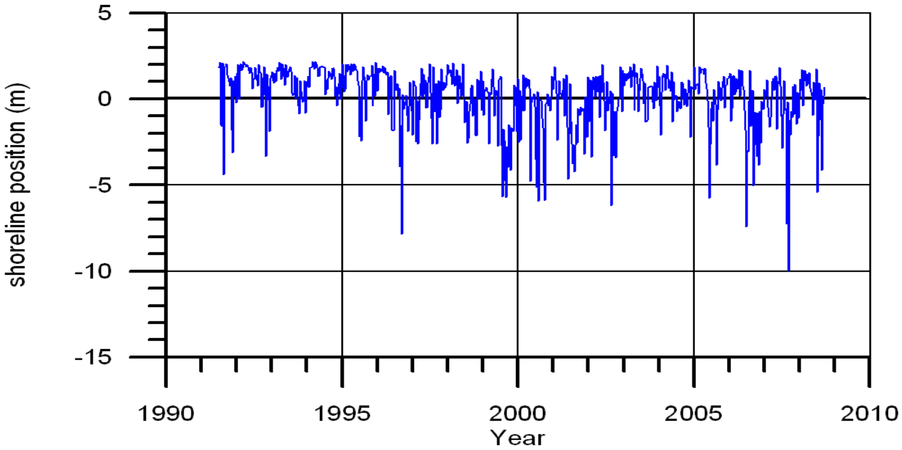


Figure 4.4 Time history of simulated shoreline movement due to cross-shore sediment transport mechanisms (June 1991 –Sep 2008)

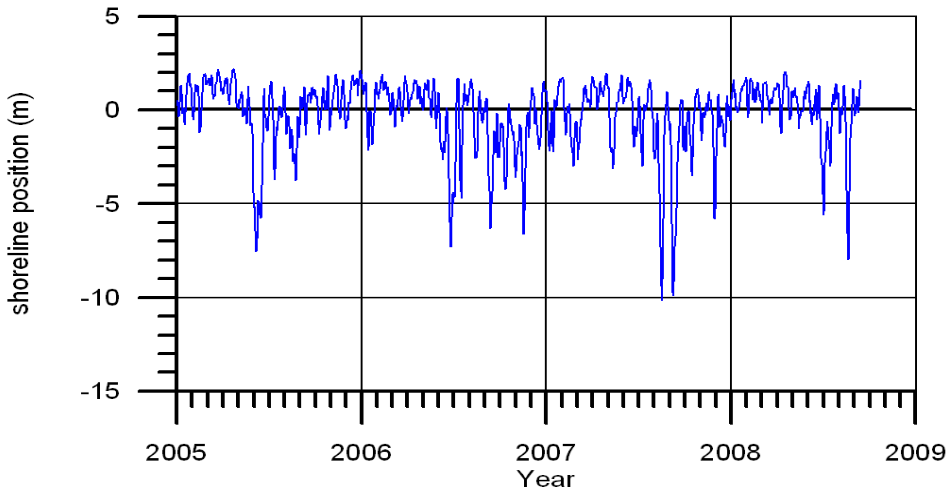


Figure 4.5 Time history of simulated shoreline movement due to cross-shore sediment transport mechanisms. Period: Jan2005 - Sep 2008

The simulations indicate that the amplitude of shoreline motions is typically in the order of a few metres. The shoreline erosion rarely exceeds 10 m. It is noted that the shoreline dynamics presented here only represent the movement of the water line due to cross-shore sediment transport mechanisms. Shoreline erosion can also occur due to long-shore gradients in the littoral transport as will be present in the next section. It is important to note that the largest beach erosion typically occurs in the period between July and September. During the remaining part of the year, the beach is generally somewhat wider.

It is noted that these simulations only represent the shoreline retreat due to pure cross-shore sediment transport mechanisms. This implies that in these simulations the total volume of sand is kept constant. The simulations represent the shoreline dynamics due to the cross relocation of sand caused by the action of waves, combined with water levels. The sediment losses due to longshore sediment transport must be superposed to the simulated shoreline variations due to cross-shore sediment transport processes.

Exceedance statistics of the shoreline fluctuations were derived from the model simulations (see Figure 4.4). The black dots represent model results, the blue curve represents a best fit. The maximal shoreline erosion for various return periods was derived by extrapolating the derived distributions manually to the values corresponding to return periods of 10 years and 100 years. The results are shown in Table 4.2.

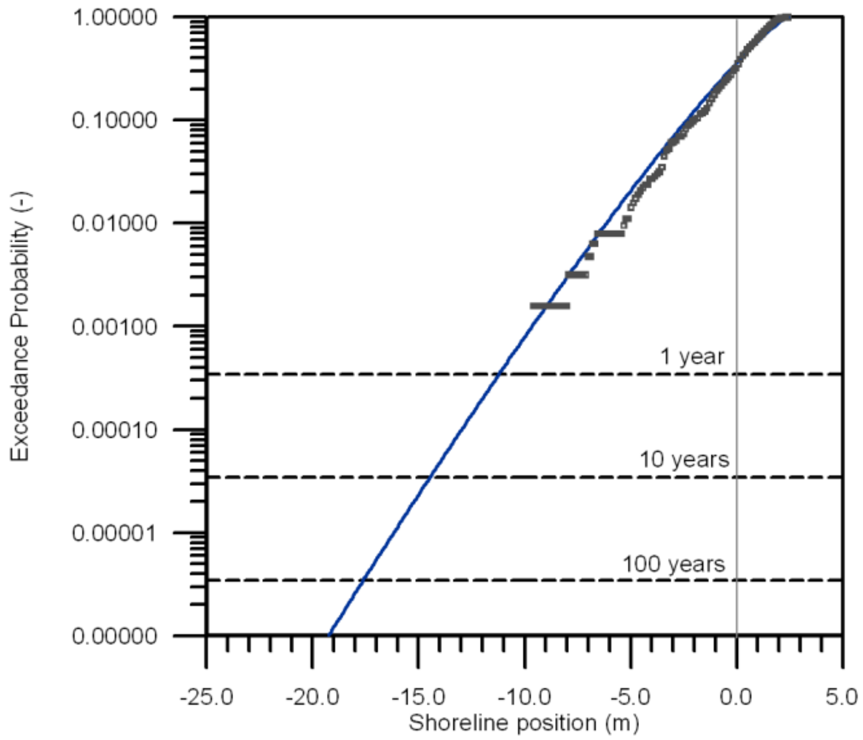


Figure 4.6 Exceedance statistics for shoreline dynamics due to profile development

The maximal shoreline retreat for a return period of 50 years was approximately 18 m. This indicates that the shoreline is expected not to retreat with more than 18 m during the lifetime of the project provided that no sediment is lost from the beach due to long-shore sediment transport, and that the meteo-marine conditions remain unchanged in the region.

Table 4.2 Estimated maximal shoreline retreat (m) for various return periods

Return Period	*Shoreline Retreat*
1yr	11.9m
5yr	14.5m
10yr	15.6m
50yr	18.2m
100yr	19.3m

5 LITTORAL SEDIMENT TRANSPORT

The derived inshore wave statistics were used to calculate the annual sediment transport along the shore. Sediment transport calculations were made using DHI's sediment transport modeling system LITPACK. The measured beach profiles and sediment characteristics were used as input to the model.

Figure 5.1 shows the calculated cross-shore variation of the littoral sediment transport in front of the hotel. The figure shows that the bulk of the sediment transport occurs within a distance of 200 m from the shoreline. The transport is practically zero for water depths larger than 4.5 m + MWL. The net transport was calculated as approximately $1x10^5$ m^3/year directed towards North.

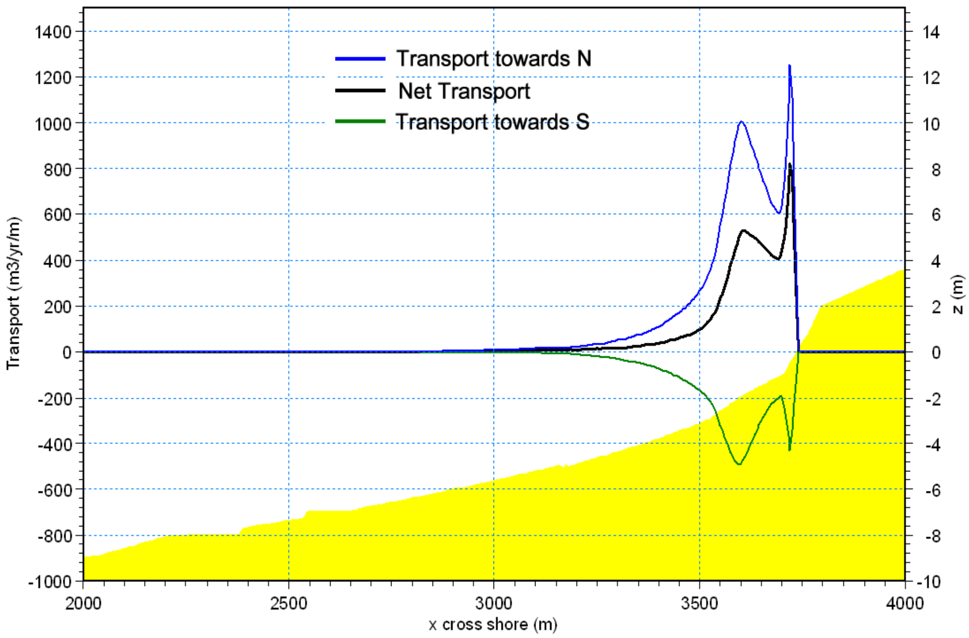


Figure 5.1 Cross-shore distribution of annual littoral drift

An important aspect of littoral transport is the shoreline's orientation compared to the so called equilibrium orientation, which is defined as the shoreline orientation where the annual drift towards South is of equal magnitude as the annual drift towards North. The shoreline orientation is defined as the angle between shore normal (i.e. line perpendicular to the shore) and North. Figure 5.2 shows the calculated relation between littoral transport and shoreline orientation. The model simulations indicate an equilibrium shoreline orientation of approximately 88 °N. The shoreline orientation in front of the hotel is around 78 °N, thus indicating a difference of approximately 10 ° with the equilibrium orientation. This difference is of major importance for the shoreline response to intrusive coastal structures such as groins, jetties or breakwaters as will be demonstrated in the next sections.

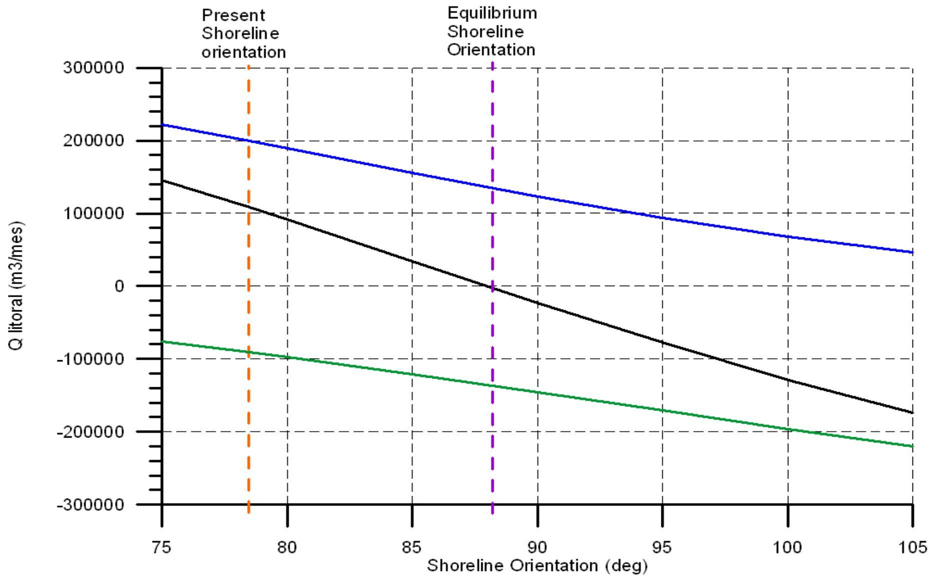


Figure 5.2 Variation of annual littoral drift with shoreline orientation

In order to analyze possible temporal variations in sediment transport patterns an analysis was made of the littoral transport for each year covered by the data. The results are shown in Figure 5.3. It was found that significant fluctuations have occurred in the annual sediment transport during the last decades. In the early 1990s, the net transport was practically zero with northward and southward components in the range of 100,000 m^3/year. During the following period the magnitudes of the northward directed transport have increased significantly. Also, a shift from southward to northward was observed in the direction of the net transport. The model simulations indicate fluctuations of littoral transport on a time scale of 5 to 10 years.

The littoral transport varies with the shoreline orientation and is a function of the wave conditions. Along the coastal stretch in front of the hotel, the shoreline is practically straight with parallel depth contours until at least the closure depth. The wave conditions were not found to vary significantly along the beach. Shoreline erosion - or accretion related to littoral sediment transport can only occur due to longshore gradients in littoral transport. The observations above indicate no clear direct reason for shoreline erosion due to such gradients. Gradients in littoral transport may occur locally in the vicinity of the two inlets that border the Comandatuba Island. Morphodynamics changes in the tidal delta in front of the river mouth will cause some variations in sediment bypass along the delta. It is possible that such variations cause shoreline fluctuations along the updrift beach and may have some effect on the shoreline dynamics in front of the hotel.

On the basis of the field observations and model results it is expected that the observed shoreline erosion is mainly related to short term profile dynamics that act on time scales varying from a single storm event to a season.

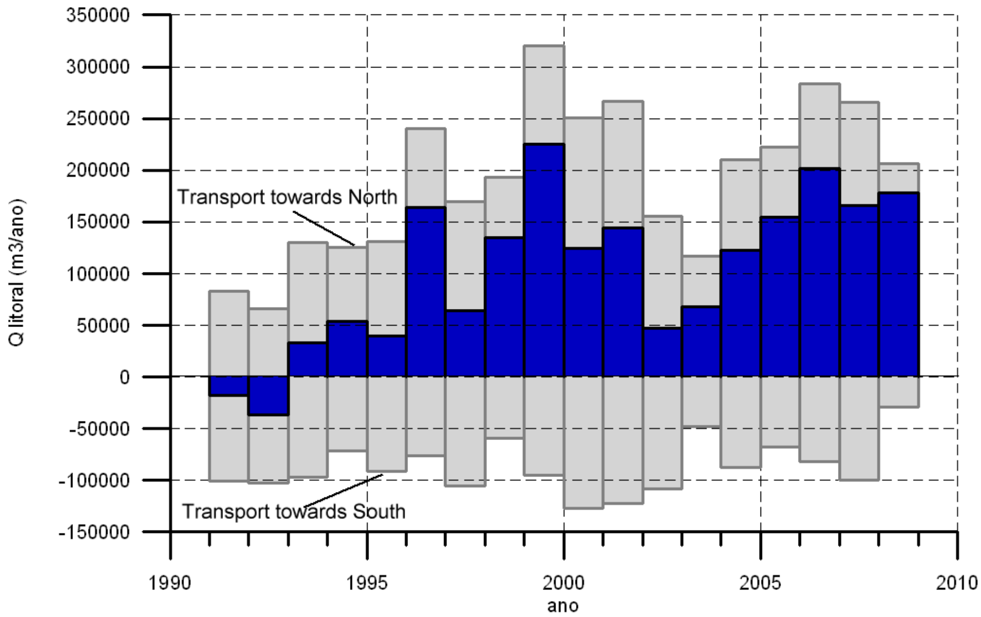


Figure 5.3 Time history of annual drift (1991 – 2008)

The variation of littoral transport throughout the year is shown in Figure 5.4. Positive values indicate transport towards North, negative values towards South. The grey areas indicate the north- and southward components of the littoral transport, the blue boxes indicate the net annual transport. A clear seasonal variation in littoral sediment transport was observed. During the period March – September the net littoral transport is directed towards North. During the rest of the year it is directed towards South. The highest magnitudes of littoral transport were found to occur in June and July, when average transport rates of approximately 30,000 m^3/month were observed. The lowest transport rate was found in February.

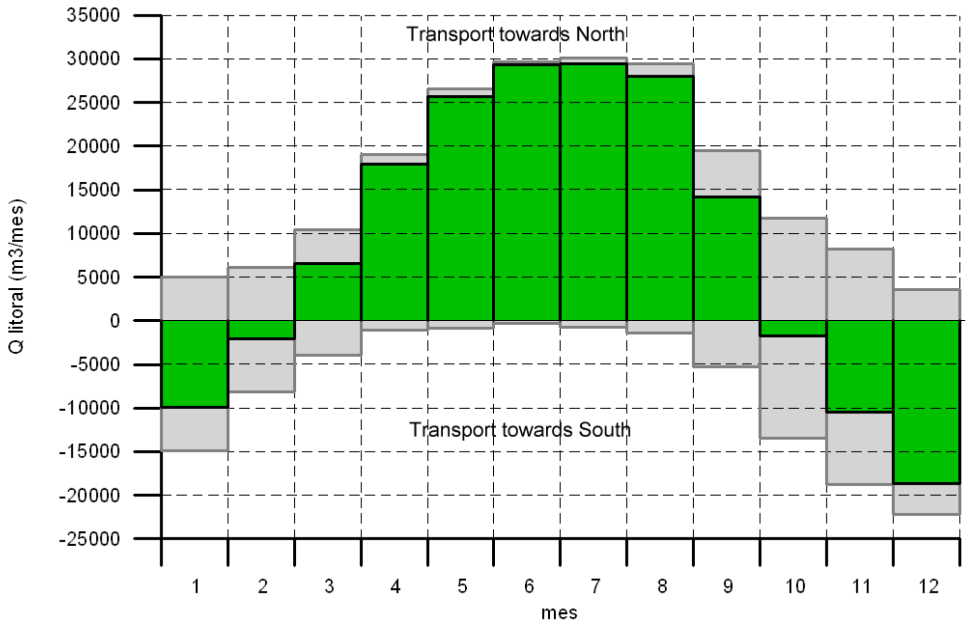


Figure 5.4 Monthly variation of littoral drift

6 CONCEPTS OF HUMAN INTERVENTIONS

Many coastal protection schemes are based on the idea to reduce the amount of wave energy that reaches the shore. Such schemes include emerged- or submerged breakwaters, sills and artificial reefs. The wave conditions along the project site are not extreme and are actually very suitable for recreational purposes. In order to guarantee a good quality beach, its exposure to wave energy must not be reduced too much, as this would have a negative impact on the quality of the beach (accumulation of fine material and debris, poor circulation, reduced water quality). It is neither necessary nor desirable to reduce the level of wave energy for the present project. Therefore coastal protection schemes based on reducing wave energy are not recommended and have not been considered in this study.

In order to establish a shoreline protection scheme that fulfills the above requirements several different concepts are possible. On the basis of the performed analysis three main types have been defined: 1)-Intrusive structures, 2)-Non-intrusive structures and 3) - Beach nourishment. The term “intrusive” indicates that a coastal structure interferes with the littoral current and, to some degree, blocks the littoral sediment transport. Each type of intervention is outlined in the following sections.

6.1 Intrusive Coastal Structures

The idea behind an intrusive coastal structure is to (partly) block the littoral sediment transport. This blocking leads to sediment accretion at the updrift side but also to beach erosion along the downdrift side.

Often a number of structures are established in a regular grid with a more or less constant spacing between the structures. The function of the structures is to provide a number of fixed non-erodible points along the shore, which allows the beaches in between these structures to attain their natural equilibrium orientation.

The beach in between two structures must be wide enough to allow the seasonal variations in shoreline position, both with regard to plan shape and with regard to profile changes caused by cross-shore sediment transport mechanics.

In case of seasonal variations in wave conditions, the equilibrium shoreline orientation varies during the year, thus leading to corresponding seasonal variations in shoreline orientations in between two structures. The larger the variations in equilibrium shoreline orientation, the larger will be the amplitude of shoreline fluctuations in between two structures. This amplitude can be decreased efficiently by reducing the spacing between two structures. However, this will increase the cost of the project considerably, it is aesthetically unattractive, and may compromise the safety of swimmers as dangerous currents may develop around these structures.

Traditionally, the supporting structures for coastal restoration schemes have mainly been groins and breakwaters. However, various unwanted effects associated with these structures do occur as demonstrated in Figure 6.1. Recently, possible modifications to the layout of traditional structures to minimize these unwanted effects have been developed by Mangor (1998, 2001, 2004). A basic feature of these improvements is the use of one larger artificial headland instead of a number of small-scale coastal structures such as groins and breakwaters.

The philosophy of the artificial headland is as follows:

1. To improve the bypass, minimize the offshore loss as well as the lee side erosion

2. To eliminate dangerous rip currents as well as lee areas that might otherwise trap debris

3. To enhance the aesthetic appearance and to gain some useful land

An artificial headland acts more or less like a shore-connected breakwater; the only difference is a smoother transition of the longshore current (and the littoral transport) at the upstream end of the structure. This is caused by the smooth transition between the coast and the structure in the case of the headland. This is also the case if beach fill is included in the simulation.

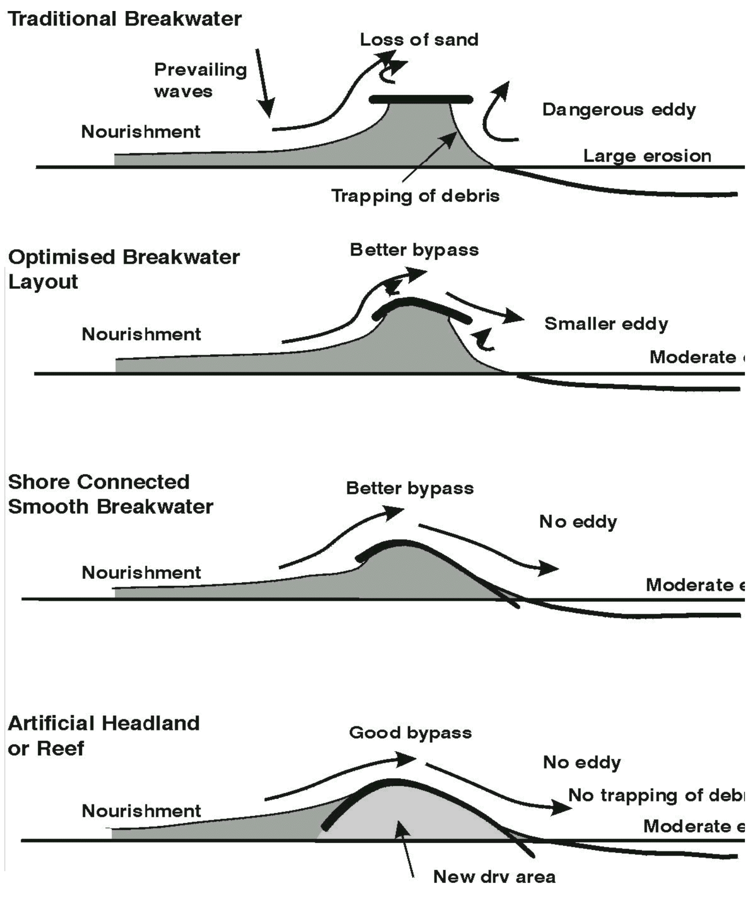


Figure 6.1 Characteristics of different shoreline management structures, Mangor (2001, 2004)

At the downstream side no large-scale vortices are generated, which is the case for other types of coastal structures. This improves the safety for swimmers and reduces the trapping of seaweed and debris. If the reclaimed area is sufficiently elevated, it can be used for permanent recreational installations. The headland can also be partly submerged, whereby it will act as a headland continuing into a reef. Careful design will make such a headland appear almost natural. The structures have a streamlined shape, which enhances the natural bypass of sediment to adjacent beaches and minimizes the risk of dangerous rip currents and vortices in the lee side of the structures. Between the headlands, curvy beaches are constructed that provide a safe and attractive environment for recreational purposes.

A number of model simulations have been performed to study the shoreline response to one or more intrusive structures. The following items were varied systematically in order to evaluate the schemes against each other:

1. Number of structures

2. Distance to the shoreline (degree of blocking)

3. Spacing between two structures

Figure 6.2 shows the result of a simulation with one structure, extending 80 m into the sea, measured from the still water line at mean water level (MWL). The simulation of shoreline evolution was performed for the entire period covered by the data (1991 – 2008). The instantaneous, transformed inshore wave conditions were used as model input. The figure shows the maximal, minimal and mean shoreline position observed during the entire period covered by the simulations. The initial shoreline is shown for reference. The model simulation showed that the beach has accreted south of the structure with an average width of approximately 15 m. North of the Structure the beach has eroded on average 10 m .

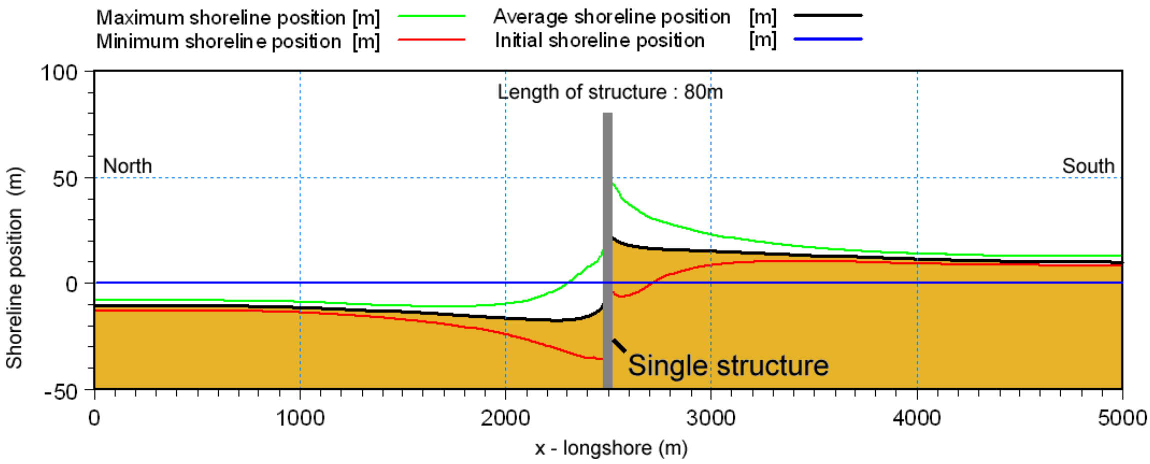


Figure 6.2 Simulated shoreline evolution (1991-2008) for a scheme with one intrusive structure (Note: Distorted y axis)

During the year, the shoreline is fluctuating between its maximal and minimal positions as a result of seasonal variations in wave conditions. The maximal amplitudes of these seasonal shoreline fluctuations occur along the two sides of the structure. Along the southern side of the structure, the shoreline was found to fluctuate between -10 m during events with transport towards South and approximately +50 m during periods with transport towards North. The extreme positions of the shoreline along the northern side of the structure are -35 m and +20 m, corresponding to events with transport towards North and towards South, respectively.

An important aspect of this scheme is that maximal beach accretion occurs during the period between July and September. This period corresponds to the period where the largest shoreline erosion occurs due to cross-shore sediment transport mechanisms. The highest risk of shoreline retreat due to cross-shore profile dynamics thus occurs when the beach is widest due to sediment accumulation caused by littoral sediment transport. Correspondingly, the strongest shoreline retreat occurs in February, when the risk of erosion due to cross-shore sediment transport is lowest. This indicates that the scheme helps reducing the risk of damage due to shoreline erosion.

By increasing the cross-shore length of the structure, the accumulation south of the structure will increase but also the erosion north of it. Table 6.1 to Table 6.4 show the maximal-, minimal- and mean shoreline position along both sides of the structure. A distinction is made between the shoreline in the vicinity of the structure (0-100 m) and the shoreline further away from the structure (at 1 km and 2 km).

Table 6.1 Overview of extreme – and mean shoreline position for one single structure of 40 m

$L_{structure}$ = 40m	*North of the structure*			*South of the structure*		
Distance	-2km	-1km	0-100m	0-100m	1km	2km
Max. shoreline	-2	-1	+10	+18	+2	+2
Min. shoreline	-3	-2	-13	- 7	+1	+1
Mean shoreline	-2	-2	-1	+ 3	+2	+2

Table 6.2 Overview of extreme – and mean shoreline position for one single structure of 60 m

$L_{structure}$ = 60m	*North of the structure*			*South of the structure*		
Distance	-2km	-1km	0-100m	0-100m	1km	2km
Max. shoreline	-3	-4	+17	+32	+8	+6
Min. shoreline	-5	-9	- 26	-11	+4	+4
Mean shoreline	-4	-6	- 6	+9	+6	+5

Table 6.3 Overview of extreme – and mean shoreline position for one single structure of 80 m

$L_{structure}$ = 80m	*North of the structure*			*South of the structure*		
Distance	-2km	-1km	0-100m	0-100m	1km	2km
Max. shoreline	-8	-11	+20	+50	+17	+13
Min. shoreline	-13	-17	-36	- 6	+10	+ 9
Mean shoreline	-10	-14	-10	+20	+13	+10

Table 6.4 Overview of extreme – and mean shoreline position for one single structure of 100 m

$L_{structure}$ = 100m	*North of the structure*			*South of the structure*		
Distance	-2km	-1km	0-100m	0-100m	1km	2km
Max. shoreline	-15	-19	+18	+66	+28	+22
Min. shoreline	-23	-29	-48	+1	+18	+14
Mean shoreline	-18	-23	-16	+34	+23	+17

6.1.1 Schemes with Multiple Intrusive Structures

Model tests were performed to analyze the shoreline evolution in case two or more intrusive structures were used. Figure 6.3 shows the maximal – and minimal shoreline positions for a scheme with twin structures at a distance of 450 m from each other. Both structures have a length of 80 m, measured from the still water line at MWL.

The model results show that the shoreline evolution along the updrift (southern) and downdrift (northern) side of the twin structures are very similar to the scheme with one single structure. The twin structures seem to have a slightly increased effect on both sides of the scheme: slightly stronger beach accretion along the beach south of the structures and slightly stronger beach erosion along the beach north of it.

In between the two structures, the shoreline positions are not more favourable than they are for the one-structure scheme. Along the northern side of the southern structure, the maximal beach accretion is larger than for the one-structure scheme. However, this wider beach occurs during periods with transport towards South, while the highest risk for storm events occurs during periods with transport towards North. In the same location, the maximal beach erosion is stronger than for the one-structure scheme. This beach erosion occurs during period with transport towards North (i.e. April-September). During this period the risk of shoreline erosion during storm events is highest. Therefore the beach must be as wide as possible during this period in order to reduce this risk of damage to the coastal facilities.

On the basis of the model simulations it was concluded that multiple-structure schemes will not improve the shoreline protection provided by a single-structure scheme and are therefore not recommended.

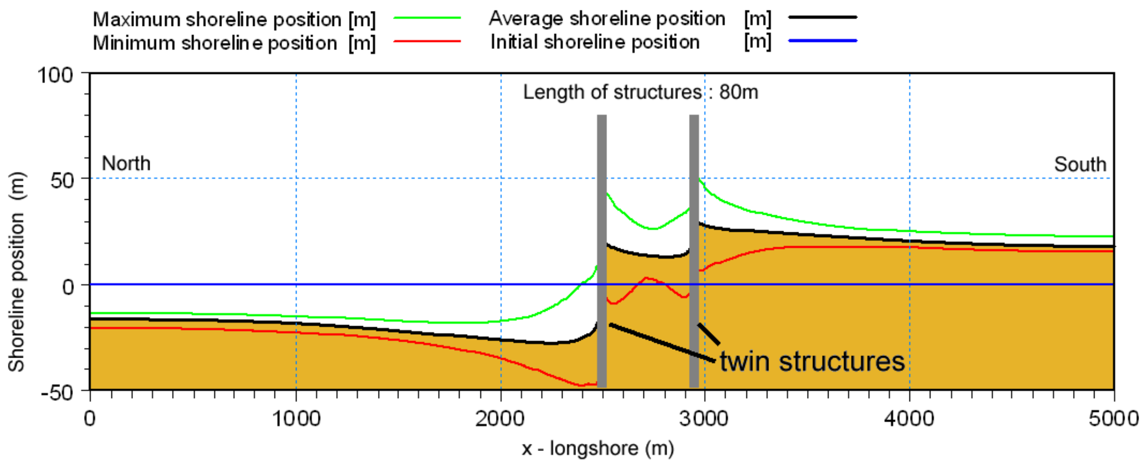


Figure 6.3 Simulated shoreline evolution (1991-2008) for a scheme with twin structures (Note: Distorted y axis)

6.1.2 Shoreline Evolution under different Meteo-marine Conditions

One of the important issues to address in the case of intrusive structures is their response to changes in wave conditions. If wave directions should shift towards North, then littoral transport towards South would increase, which could lead to an inversion of the sediment transport pattern. If that should happen (e.g. a shift in net annual drift from northwards to southwards) then the risk appears that the structure might have a negative impact on the shoreline in front of the hotel. In order to minimize such risk a solution must be sought that will function properly, even under changing wave conditions. Often this would mean the construction of a multiple-structure scheme that would have a positive effect regardless of the direction for the littoral transport. However, multi-structure schemes are not recommendable here due to the strong seasonal variations in wave direction.

In order to verify the sensitivity of the scheme to changes in wave conditions, a number of model simulations were performed where it was assumed that all offshore wave directions were shifted 10 degrees towards North. It is noted that this is quite an extreme assumption, especially taking into account that recent observations indicate a shift in the opposite direction (e.g. towards South).

The simulated extreme positions of the simulated shoreline are shown in Figure 6.4. The results indicate that, in case of a 10 degrees shift in offshore wave direction, the net annual drift would be practically zero. This is reflected in the fact that almost no net beach accretion or erosion would occur along the beaches on both sides of the structure. Close to the structure, the amplitude of the shoreline fluctuations would be similar to the amplitude for the situation without shift in wave direction. During periods with transport towards North (April-September) the beach will be wider than average in front of the resort. During the remaining period with transport towards South the beach is narrower. As the highest risk of damage occurs during the winter, the simulations indicate that even in case of a drastic shift in offshore wave directions, the scheme would have a beneficial effect.

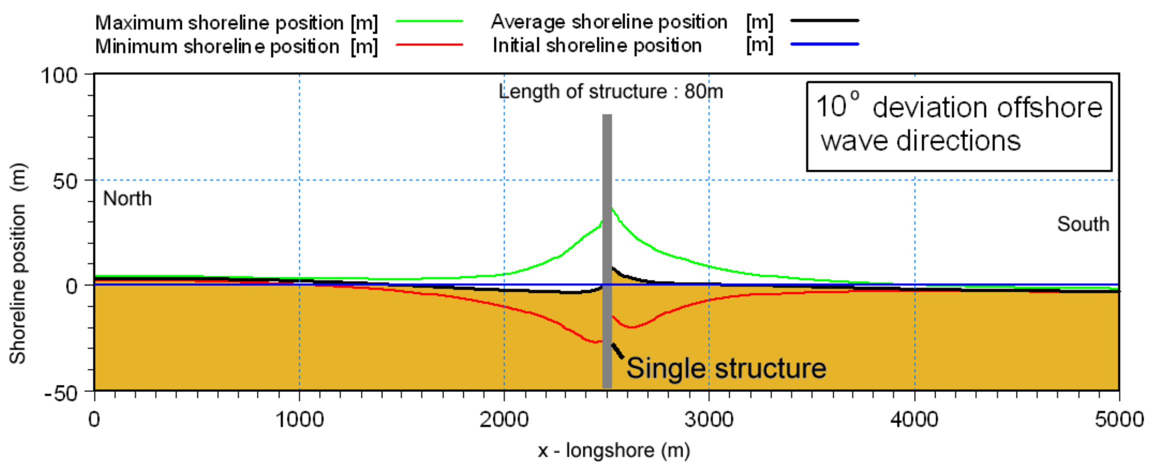


Figure 6.4 Simulated shoreline evolution (1991-2008) for a scheme with one intrusive structure. Anti-clockwise rotation of offshore wave directions: 10° (Note: Distorted y axis)

The same simulation was performed for a two-structure scheme. The results are shown in Figure 6.5. The results along the beaches at both sides of the scheme are very similar to the single-structure scheme. Within the two structures the amplitude of the shoreline movements is slightly smaller than for the single-structure scheme. However, the maximal shoreline erosion along the northern side of the southern structure may create critical conditions as it will occur during the period with highest risk for storm surges.

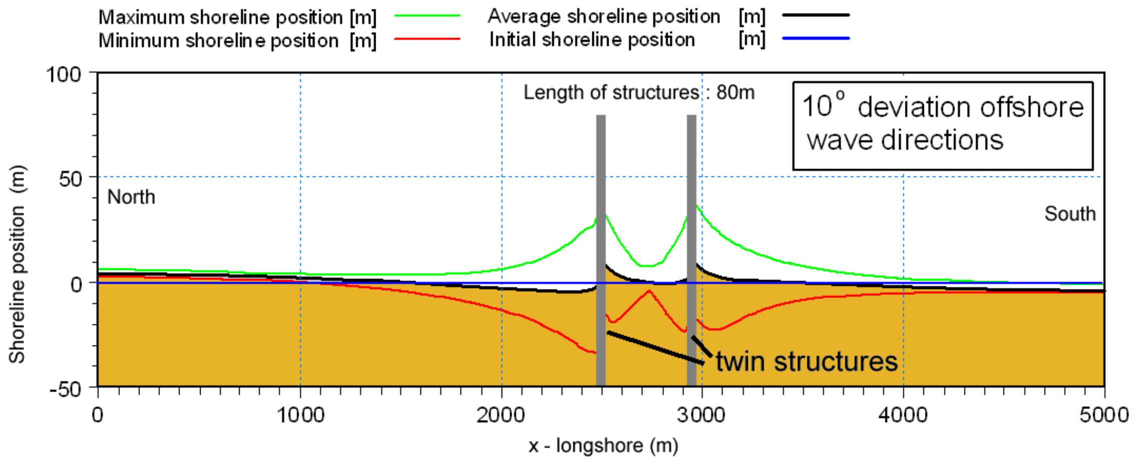


Figure 6.5 *Simulated shoreline evolution (1991-2008) for a scheme with twin- structures. Anti-clockwise rotation of offshore wave directions: 10°(Note: Di storted y axis)*

6.1.3 Advanced Modeling of Flow around Structures

A number of model simulations were performed in order to study the details of the flow in the vicinity of the coastal structures. The simulations were performed using MIKE 21 HD FM, which is an advanced state-of-the-art modeling tool for flow in coastal waters. The model includes the combined effects of tide -, wind – and wave driven flow.

The simulations were performed for a classic groin and an artificial headland. The simulations were covered by a tidal cycle of 12 hours.

Constant offshore wave conditions were assumed during the simulation (Hs=1.0m, Tz = 8s, MWD=120° N). The tidal conditions corresponded to spring tide. The objective of the modeling is to study the flow patterns around the heads of the structures and to verify whether undesired offshore directed flow (rip currents) would occur. Such flow phenomena would create dangerous situations for swimmers. Rip currents can possibly cause loss off sediment as the strong currents may transport it and deposit it in water depths where waves and currents no longer can transport the sediment back to the shore.

Specific model parameters that determine bed friction and eddy viscosity were given typical values based on DHI experience in other, similar applications. A formal model calibration is not required for this analysis as its objective is to perform a qualitative evaluation of the schemes under typical hydrodynamic conditions.

The simulated wave field during high water and low water are shown in Figure 6.6. The model results show that the heads of the structures are located outside the surf zone, which is the area where waves break and rapidly lose their height and energy. Under these circumstances the structures will significantly block the wave generated longshore current, which has its maximal strength inside the surf zone.

At low water the breaker zone extends well beyond the heads of the structures. This indicates that under these conditions the structures will only have a limited effect on the littoral current and sediment transport. The bulk of the sediment will bypass the structures towards the beaches north of them.

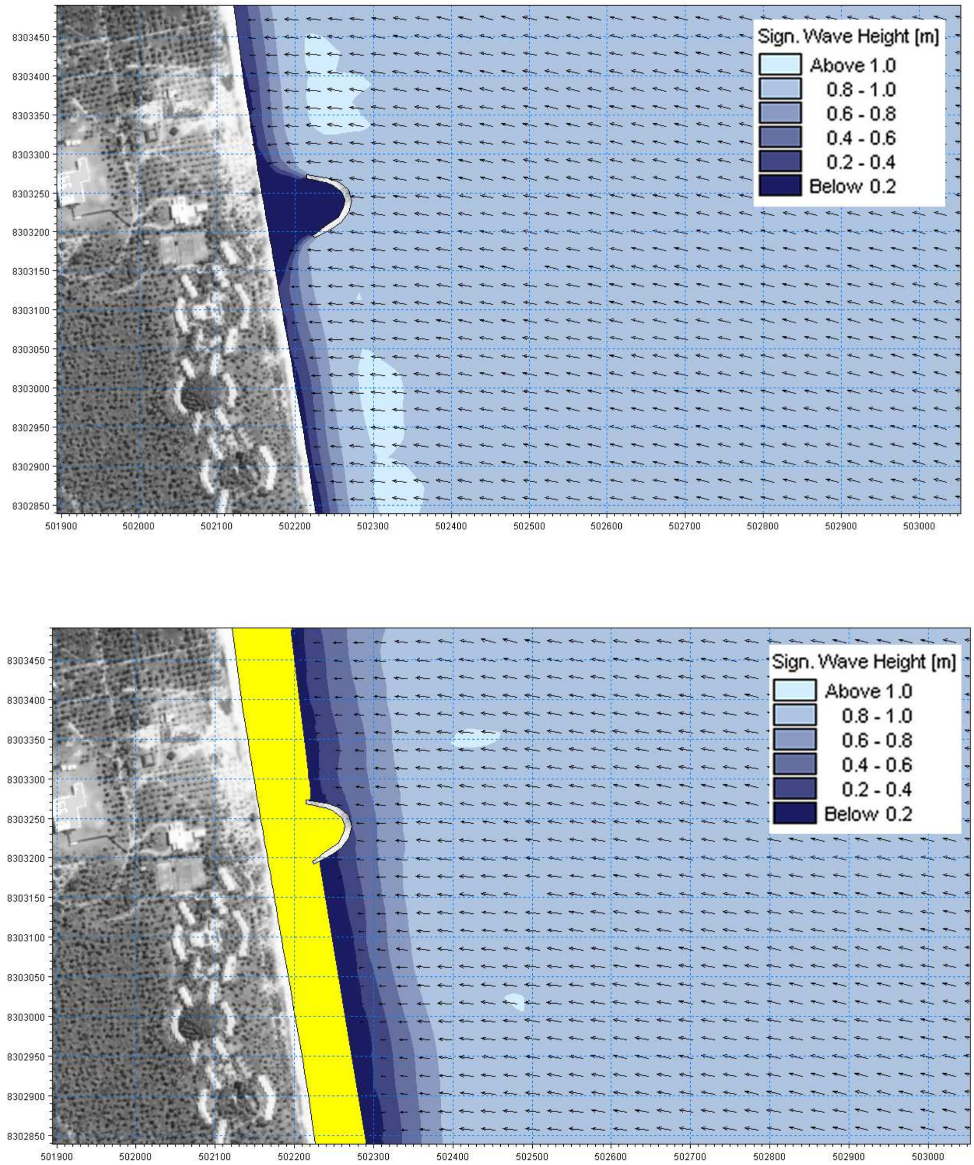


Figure 6.6 Simulated wave field around an artificial headland. Top: MWHS, Bottom: MLWS

Figure 6.7 shows the simulated flow fields for both structures at high water. The flow around the groin was found to separate around the head of the structure. The flow around the artificial headland stays attached to the shoreline and does not create vortices or rip currents.

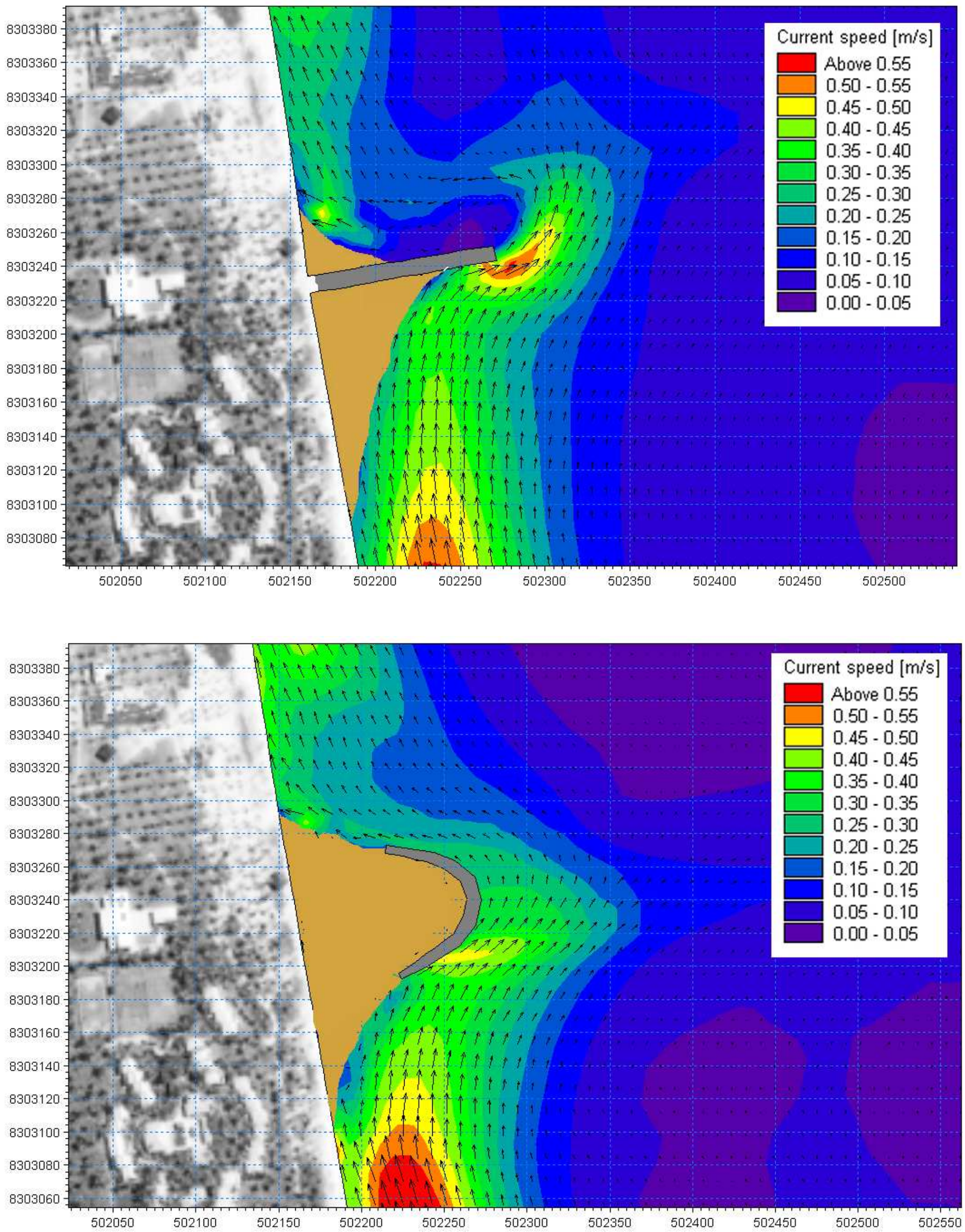
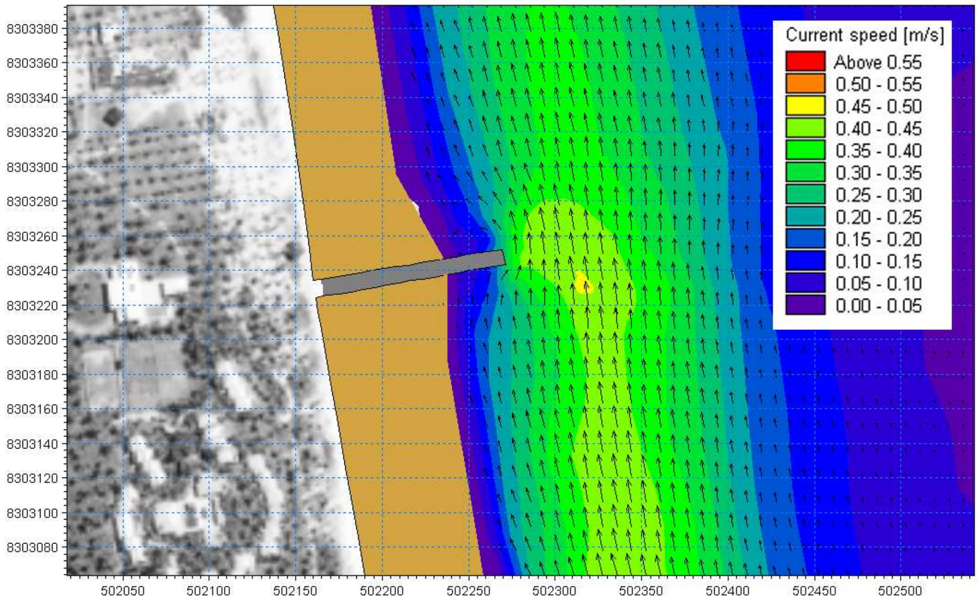


Figure 6.7 Simulated wave driven flow around two coastal structures at high water. Top: classic groin, bottom: artificial headland

At low water, the structures have only a very limited effect on the flow, which indicated that sediment is transported towards North unhindered by the structures. This is shown in Figure 6.8.

The model simulations have shown that the structures have a significant blocking effect at high water and are practically neutral to the littoral current at low water. This indicates that beach accretion will occur but that the accumulation of sediment is mainly confined to the upper part of the beach profile. In the lower parts of the profile sediment is transported without being hindered by the structures. This bypass of sediment is crucial for the stability of the beach along the downdrift side of the structure.



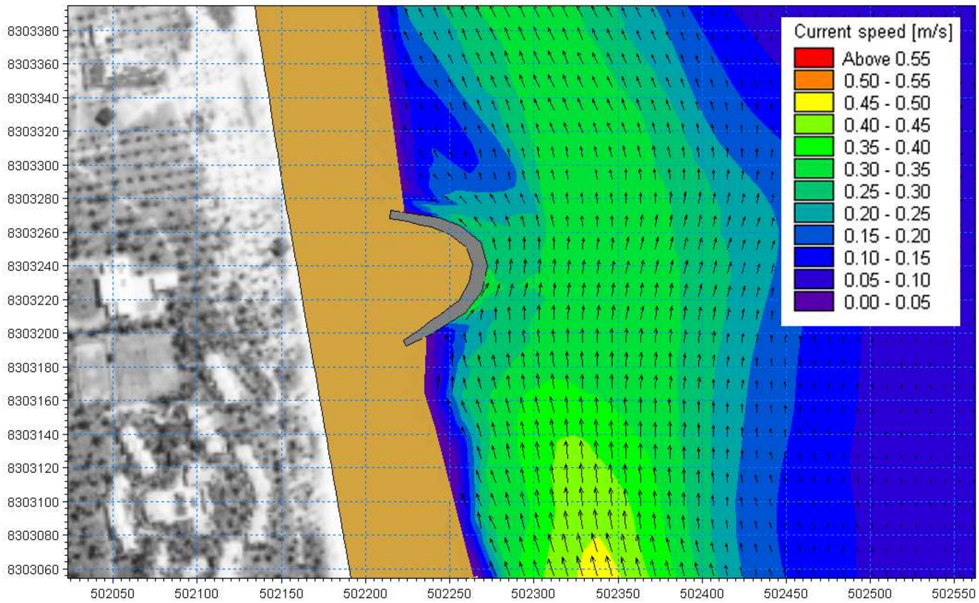


Figure 6.8 Simulated wave driven flow around two coastal structures at low water. Top: classic groin, bottom: artificial headland

The model simulations have also shown that the artificial headland creates a smoother flow pattern around the head of the structure than the groin. This may be associated with some sediment loss in case of the groin. On the other hand the flow gets reattached to the shore over a relatively short distance. Therefore, besides the detailed flow patterns around the heads, the structures are expected to perform quite similar to each other.

6.2 Non-intrusive Coastal Structures

The idea behind a non intrusive coastal structure is that it normally does not interfere with the natural transport of sediment along the shore but only becomes active in case of extreme conditions with strong sea advance. In this way they act as so-called sleeping defense systems, only to be activated under extreme conditions. For the present application two types of non-intrusive coastal structures can be considered as effective shoreline protection measures in front of the resort: 1) Revetment (enrocamento) and 2) Artificial dunes. The two types of protection are described in the following sections.

6.2.1 Revetment

A revetment is a facing of stone, concrete units or slabs etc., built to provide protection against erosion by wave action, storm surge and currents. A revetment is not protecting against flooding. Revetments are normally used for lightly to moderately exposed locations as is the case for Comandatuba. The structure is constructed on the backshore and is usually covered by sand.

Revetments are always made as a sloping structure, and very often constructed as a permeable structure by natural stones or concrete blocks, whereby wave energy absorption is enhanced and reflection and wave run-up are minimized. However, revetments can also consist of different kinds of concrete slabs, some of them permeable and interlocking. In this way their functionality is increased in terms of absorption and strength. Net mesh stone filled mattresses, such as Gabions, are also used; however, they are only recommendable at fairly protected locations. Revetments can also consist of sand-filled geotextile fabric bags, mattresses and tubes; such structures must be protected against UV-light to avoid weathering of the fabric. Sand-bagging is often used as emergency protection. Geotextile fabric revetments are fragile against mechanical impact and vandalism, and their appearance is not natural.

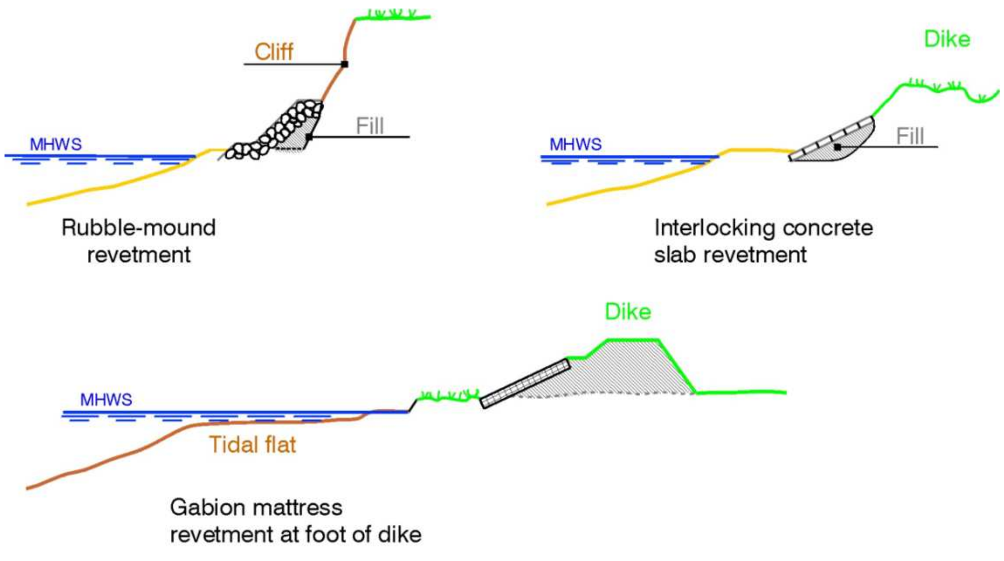


Figure 6.9 Examples of revetments

A buried revetment can be constructed as part of a soft protection, e.g. as a hard emergency protection built into a strengthened dune which acts as shore protection and/or sea defense, see Figure 6.10.

Figure 6.10 Example of emergency revetment constructed in concrete blocks, the revetment will later be buried into an artificial dune. (Danish Coastal Authority)

As a buried revetment does not interfere with the natural sediment transport it does not have an impact on the shoreline. It only becomes exposed to waves during extreme conditions where a large part of the beach in front of the revetment has been eroded. If designed carefully, buried revetments can be integrated in the natural landscape without being aesthetically unattractive.

6.2.2 Artificial Dune

A natural dune is nature's own flexible protection against coastal erosion and flooding. Artificial dunes are applicable as combined protection measure in areas with natural dunes that suffer from wind and coastal erosion, beach degradation and/or flooding.

An artificial dune is constructed by importing sand from outside the project area. The dune is normally constructed on the backshore close to the coastline. In order to protect the dune and to enhance the natural dune building process, an artificial dune is normally planted with marram grass and protected by spruce fences.

The main functions of an artificial dune are listed in the following:

1. Enhances the natural dune growth processes and is an environmentally sound and sustainable protection method

2. Provides flexible protection against coastal erosion. As the dune is gradually eroded, sand is released to the littoral processes, and the impact on adjacent beaches is therefore positive. The volume and quality of the dune sand determines the durability of the protection

3. Helps to maintain a wide sandy beach

4. If sufficiently high, a continuous artificial dune line protects against flooding

In order to act as reliable shoreline protection measure, artificial dunes must be maintained and restored after significant erosion events that occur during periods with high water levels. The functionality depends on the volume, height and longshore extension of the dune as well as the quality of the sand. The minimal volume- and height of the dune depend on the maximal shoreline retreat that can occur during storm surges (ressacas). This maximal shoreline retreat is determined by the maximal water levels, the duration- and wave conditions during these storm events as well as the characteristics of the sand.

Figure 6.11 Planting of Marram grass and the placing of fences on artificial dunes (Danish Coastal Authority)

6.2.3 Beach Scraping

Although beach scraping is not a coastal structure as such, it is included in this section because it can be functional in combination with an artificial dune and does not include beach material that originates from external sources such as beach nourishment, which is presented in the next section.

Beach scraping is recovering material from the berm at the foreshore and placing it on the backshore at the foot of the dunes or the cliff. A beach berm consisting of coarse sand or gravel is sometimes formed during relatively mild wave conditions, which tend to transport seabed material towards the beach. Beach scraping is normally performed using front loaders.

The purpose of beach scraping is to strengthen the upper part of the beach profile and the coastal dune or cliff. The material is placed in a position that enhanced the natural capacity to withstand beach erosion occurring during storm surge conditions. This method can be used for beaches, which are mainly exposed to seasonal erosion, whereas it is probably not feasible for locations, which are exposed to long-term erosion. One disadvantage of the method is that the material used for strengthening the upper part of the beach profile is taken from the lower part of the same profile, which means that the method only contributes insignificantly to the overall stability of the beach profile. Another issue is that the operation may disturb recreational activities.

6.3 Beach Nourishment

Nourishment can be regarded as a natural way of combating coast erosion and shore erosion as it artificially replaces a deficit in the sediment budget over a certain stretch with a corresponding volume of sand. However, if the cause of the erosion is not eliminated, the erosion will continue in the nourished sand. This means that nourishment as a stand-alone method normally requires a long-term maintenance effort. In general, nourishment is only suited for major sections of shoreline; otherwise the sand loss to neighboring sections will be too large.

The success of a nourishment scheme depends very much on the grain size of the nourished sand, the so-called borrow material, relative to the grain size of the native sand. The characteristics of the sand determine the overall shape of the coastal profile expressed in the equilibrium profile concept. Furthermore, in nature the hydrodynamic processes tend to sort the sediments in the profile so that the grain size is decreasing with increasing water depth.

When borrow sand is placed in a coastal profile then nature will attempt to re-establish a new equilibrium profile so that changes will always occur in the nourished profile. Immediately after establishment of the nourishment, the waves start to rework the sand the nourishment will gradually decay. It is necessary to re-nourish at regular intervals. This requirement for regular maintenance is sometimes found hard to accept by the public, the environmental authorities and the owners of the beach. On the other hand, as environmental concern and requirements for sustainability are gaining in importance, nourishment has over the last 20 years gradually increased its share of shoreline management schemes. Presently, beach nourishment is the most commonly applied measure to combat beach erosion in Denmark, and in many other parts of the world.

If the nourishment material is coarser than the native sand, it will tend to form a steeper profile than the natural profile. This means that a wider beach will tend to be formed, see Figure 6.12 (lower part). Furthermore, coarser sand will be more stable in terms of longshore loss.

If the nourishment material is finer than the native sand, it will tend to form a flatter profile than the natural one. The equilibrium reshaping of the nourished sand will reach out to the closure depth. If the objective of the nourishment is to obtain a wider beach, this will require very big volumes of sand as illustrated in the upper part of Figure 6.12.

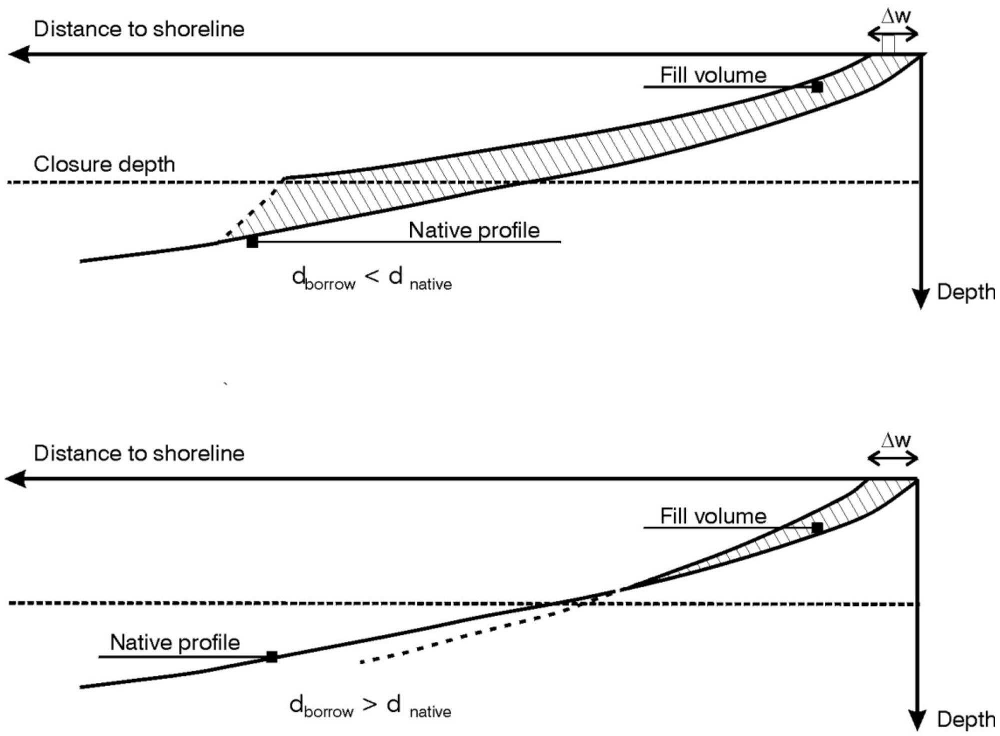


Figure 6.12 Equilibrium conditions for nourished beaches required to obtain an additional beach width of Δw with borrow sand which is finer and coarser than the native sand (upper and lower, respectively)

It is evident that the sand volume needed to obtain a certain beach width is increasing drastically with decreasing grain size of the nourished sand. Nourishment can be divided into three types: backshore nourishment, beach nourishment and shoreface nourishment. The three different nourishment methods will be discussed briefly in the following.

Backshore nourishment is strengthening of the upper part of the beach by placing nourishment on the backshore or at the foot of the dunes.

The main objective of backshore nourishment is to strengthen the backshore/dune against erosion and breaching during extreme events. The material is stockpiled in front of the dunes and acts as a buffer, which is sacrificed during extreme events. This kind of nourishment is working more by volume than by trying to restore the natural wide beach. The loss is normally large during extreme events, whereby steep scarps are formed. Backshore nourishment can be characterized as a kind of emergency measure against dune setback/breach; it can therefore not be characterized as a sustainable way of performing nourishment and it does not normally look very natural.

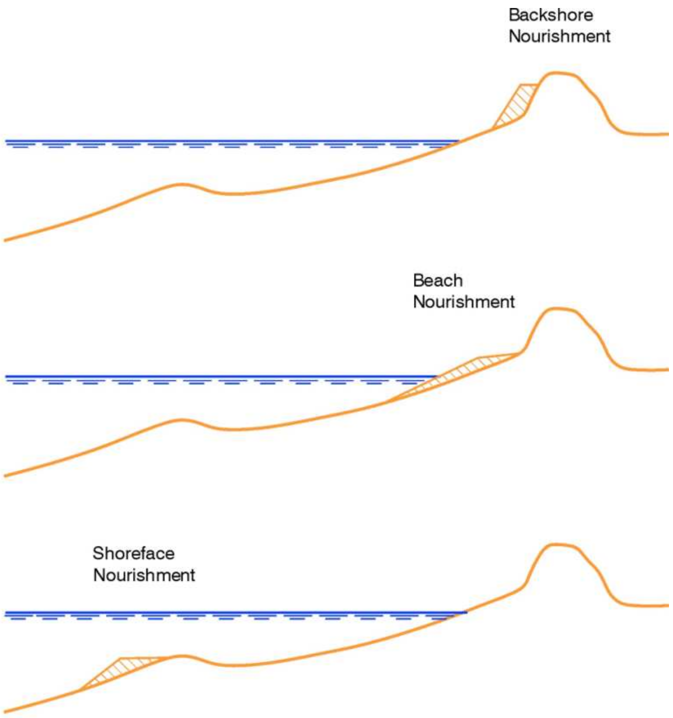


Figure 6.13 Principles in backshore nourishment, beach nourishment and shoreface nourishment

Backshore nourishment can be performed by hydraulically pumping sand through pipes discharging at the foot of the dunes and later adjusted by dozer. The sand source can be either offshore supply via a cross-profile pipeline, floating or buried or it can be supplied along the shore from e.g. a sand bypassing plant. The sand can also be supplied via land transport by dumpers.

Beach nourishment is to supply sand to the shore to increase the recreational value and/or to secure the beach against shore erosion by adding sand to the sediment budget. It is not a coast protection measure, as the beach will normally be flooded during extreme events, but it will support possible coastal protection measures. When performing beach nourishment, the borrow sand must be similar to the native sand to adjust smoothly to the natural profile. It may be an advantage to use slightly coarser sand than the natural beach sand, as this will enhance the stability of the resulting slightly steeper profile. Finer sand will very quickly be transferred to deeper water and will thus not contribute directly to a wider beach. However, the fine sand will help building up the outer part of the profile.

Shoreface nourishment is supply of sand to the outer part of the coastal profile, typically on the seaside of the bar. It will strengthen the coastal profile and add sediment to the littoral budget in general. This type of nourishment is used in areas where coast protection measures have steepened the coastal profile or in areas of long-term sediment deficit. Shoreface nourishment is sometimes used in combination with beach nourishment in order to strengthen the entire coastal profile. It is recommended for obtaining a nourished profile close to the equilibrium profile. Stand-alone shoreface nourishment acts only indirectly as a shore protection measure through slightly decreased wave exposure and as a shore restoration measure with considerable delay and little efficiency.

Shoreface nourishment is often performed using split barges. The unloading is fast and the unit price therefore low. Shoreface nourishment can profitably be used in connection with large beach nourishment schemes where borrow material, which does not fulfill the requirements for beach nourishment, can be used in the outer part of the profile where it belongs naturally.

Figure 6.14 Nourishment methods in practice by the Danish Coastal Authority. Beach nourishment by pipe discharge on the beach and over the bow pumping and shoreface nourishment by split barge

A number of model simulations were performed to study the response of the shoreline after establishment of beach nourishment. The simulations were performed using DHI's shoreline evolution model LITLINE and covered the entire period covered by the data. The following two key parameters were varied systematically in order to compare the different nourishment schemes with each other:

1 – Total volume of beach nourishment

2 – Initial placement of the nourishment

The sediment characteristics of the nourishment material were assumed equal to the sediment presently available on the dry beach (with a mean grain size of 0.18 mm). Figure 6.15 shows the shoreline configuration at the end of the model simulations. The initial placement of the nourishment is indicated for reference. The simulated nourishment schemes are listed in Table 6.5.

Table 6.5 *Simulated beach nourishment schemes*

Scheme	*Total Volume*	*Longshore length of nourishment (m)*	*Width of nourish-ment (m)*
1	400,000 m^3	1600 m	40 m
2	200,000 m^3	1600 m	20 m
3	200,000 m^3	800 m	40 m

The model simulations indicate a gradual decay of the initial nourishment. At the end of the simulation (17 years) the width of the nourishment has reduced to approximately 10 m for scheme 1 and to 5 m for schemes 2 and 3. It is noted that the initial shape of the nourishment has no significant effect on the future shoreline evolution (e.g. comparison between scheme 2 and 3 which had an equal nourishment volume).

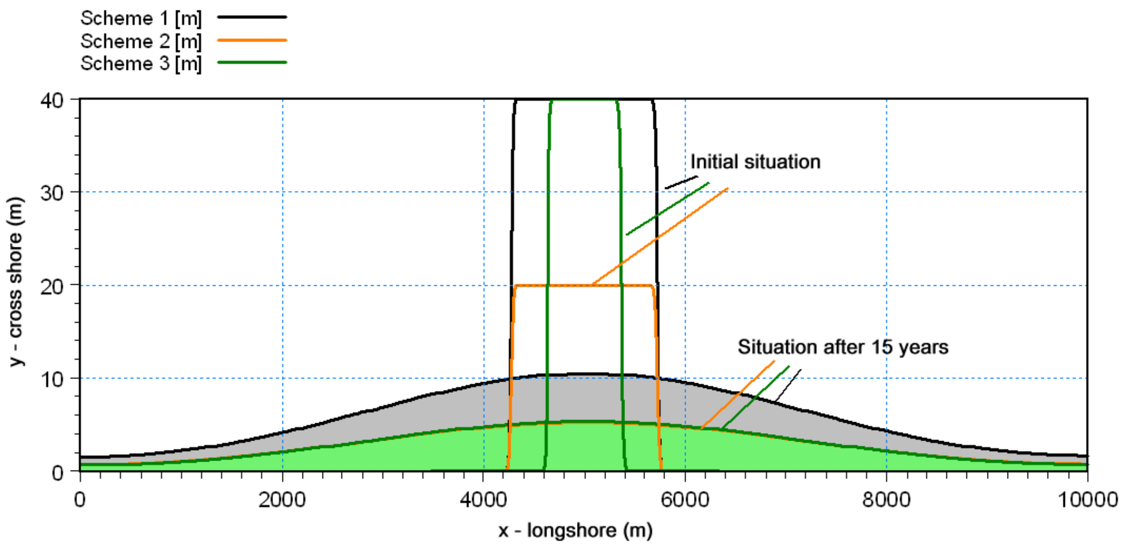


Figure 6.15 Simulated shoreline response for nourished beaches. (Note: Distorted y axis)

7 RECOMMENDED SHORELINE PROTECTION

In this section the results of the analyses are integrated into recommendations for a sustainable shoreline management scheme for the resort. The scheme must fulfill the following requirements:

1. Provide protection against damage to coastal infrastructure on the property of the hotel
2. Technically – and economically feasible
3. Aesthetically attractive
4. Provide a safe and good quality beach in front of the hotel

In order to define the recommendations for the optimal shoreline management scheme, an overview of advantages and disadvantages of the analyzed solution concepts is given below.

7.1 Overview of Advantages and Disadvantages of Selected Concepts

7.1.1 Intrusive Coastal Structures

Advantages:

1. “Permanent” solution that only requires maintenance after many years
2. By choosing a streamlined structure, detached from the beach, no dangerous situations are created for swimmers
3. The structure can be integrated in the landscape and, if designed properly, can contribute positively to the natural scenery.

Disadvantages:

1. Creates variations in shoreline position during the year and permanent beach erosion along its downdrift (northern) side
2. Sensitive to changes in wave conditions. If the direction of the annual sediment transport should change from Northward to Southward the structure would have a negative impact on the beach in front of the hotel
3. A structure creates a wider beach but does not necessarily enhance the vertical level of it. If beach erosion is associated with short term events with high water levels (storm surges), then an intrusive structure as stand-alone solution will not be sufficient to prevent damage to coastal infrastructure.

7.1.2 Non-intrusive Coastal Structures

Advantages:

1. Does not interfere with the natural shoreline dynamics, only becomes active during events with extreme shoreline erosion

2. If a buried structure is used, it can be integrated in the natural landscape (construction of artificial dune)

3. Can be constructed entirely on land.

Disadvantages:

1. If the beach is generally eroding then the structure will not halt the erosion until it becomes exposed

2. In case the structure is exposed the retreat of the coastline will be halted, but the beach in front of the structure will be lost.

7.1.3 Beach Nourishment

Advantages:

1. If suitable sand is used, beach nourishment does not compromise the quality of the beach and is not aesthetically unattractive

2. Beach nourishment will always have a positive effect, even if wave conditions should change drastically.

Disadvantages:

1. Requires regular maintenance (re-nourishment). The time period in between re-nourishments is depending on the nourishment volumes

2. If not managed properly, nourishment material may be eroded from the beach and create accumulation problems elsewhere, for example in dredged port access channels or tidal inlet

3. If no good quality sand is available then the nourishment may cause negative effects on the quality of the beach. In case of too much coarse material, steep slopes and beach scarps will develop, which would make the beach less suitable for swimmers, especially children. In case of too much fine material, wind may cause undesired transport and accumulation of the finer fractions into other areas of the hotel.

7.2 Recommended Solution for the Resort

The analyses performed in this work indicate that the shoreline is exposed to mild erosion that occurs during events with elevated water levels and relatively high waves. Possibly the variations in the shoreline position along the resort are partly related to the morphodynamics of the shoal in front of the river mouth at the northern end of the Comandatuba Island. No indications of severe long-term erosion processes were observed in the data analysis and modeling studies.

Given these circumstances, it is believed that, from a coastal engineering point of view, beach nourishment would be a recommendable solution. This method is environmentally friendly, aesthetically attractive and has no negative effects as long as proper nourishment material is applied. Model simulations have indicated that beach nourishment can provide a durable solution, but regular maintenance in the form of periodic renourishment would be required.

However, from a practical- and economic point of view beach nourishment may not be the optimal solution because no good quality nourishment sand is available in the area in front of the resort. The seabed in the offshore region is mainly covered by mud or other material that is inappropriate as nourishment material. Beach nourishment material would have to be brought in from relative large distance, which makes this option economically less attractive.

The area in- and around the mouth of the Comandatuba River was considered as a possible source of nourishment material. However, removal of sand from this area and placing it along the hotel property is not a sustainable solution, because it would create shoreline erosion in a similar way as for an intrusive structure. Extracting sand from – or near the tidal delta could even provoke the breaching and sudden migration of the river mouth, which would create a quite unpredictable morphological response. In his report, Prof. Landim (REF /3/) observed a relation between morphodynamics near the river mouths and shoreline erosion further updrift.

Besides this, the removal of sand from the tidal delta in the mouth of the Comandatuba River would have to be done using dredgers and transported over a long distance to reach the target area. In case of a single intrusive structure sand can be removed from land (by bulldozers) very close to the location where it is needed for the establishment of the artificial dunes.

On the basis of the above considerations it was concluded that beach nourishment would a good solution as long as the nourishment material originates from an area outside the sediment cell of the project site. Extracting sand from the river mouth will create similar erosion problems as an intrusive structure and is considerably more expensive.

Since the wave conditions in the region are relatively mild the establishment of a buried revetment as some form of sleeping defense along the entire property is not regarded necessary. The establishment of such structure would be expensive and would not guarantee a safe and attractive beach in front of the resort. On a smaller scale, the establishment of a buried revetment in front of buildings and other valuable property could be considered. In principle, sufficient protection can be obtained from artificial dunes, but such dunes require regular maintenance (after each dune erosion event). If such maintenance could be problematic or if there is not enough space to construct the dunes that are wide enough to provide protection, the establishment of buried revetments could be considered as an additional safety measure.

The establishment of a relatively small intrusive structure will enhance the beach in front of the hotel during the period with the highest risk of shoreline erosion. Therefore this type of solution is considered viable for the present case. Due to the strong seasonal variations in wave conditions, the establishment of intrusive structures will provoke relatively strong variations in shoreline position, especially in the vicinity of the structure itself. Beach accretion will occur along the updrift side of the structure, beach erosion occurs on the downdrift side. For the present case it is recommended to establish one single structure in the northern end of the area to be protected.

The present analysis has shown that the use of more structures would cause negative effects in the form of downdrift erosion in front of the hotel. Therefore use of schemes with multiple structures is not recommended.

The recommended length of the structure is 80 m, measured from the position of the still water line at mean water level (MWL). A structure with this length will create a sufficiently wide beach during the period with transport predominantly towards North. On the other hand, due to its limited length, it will not block the littoral transport entirely. It will mainly be effective for water levels higher than MWL. During periods with low water levels, the littoral current and sediment transport will bypass the structure practically unhindered.

The structure can be established as a classical groin or as a more sophisticated artificial headland. As the beach is not extensively used by swimmers, the use of an artificial headland could be considered less attractive due to its higher cost. However, from an aesthetical point of view an artificial headland is preferred as it could be easier integrated in the coastal landscape and would not be such an obstacle for passage along the beach as a classical groin would. The groin should extend on to the dry beach until the high water line. The artificial headland could be constructed starting further seawards and thus leaving a wider passage for transport along the beach. In terms of shoreline protection the two structures will have a very similar effect. The location of the headland and the groin are shown in Figure 7.1.

The establishment of the structure will lead to a gradual increase of the beach width in front of the hotel and a gradual shoreline retreat north of the structure. The long-term beach accretion in front of the hotel is expected to be in the range of 20 m to 50 m, depending on the time of year. The shoreline retreat north of the structure is expected not to exceed 15 m.

The establishment of an intrusive structure will enhance the width of the beach, but will not automatically increase the height of the backshore to a secure level. The beach widening in front of the hotel as stand-alone solution is not sufficient to avoid the risk of damage to coastal infrastructure that occurs during events with extraordinary high water levels. Therefore the establishment of the structure must be combined with the establishment - and maintenance of an artificial dune on the backshore in front of the property.

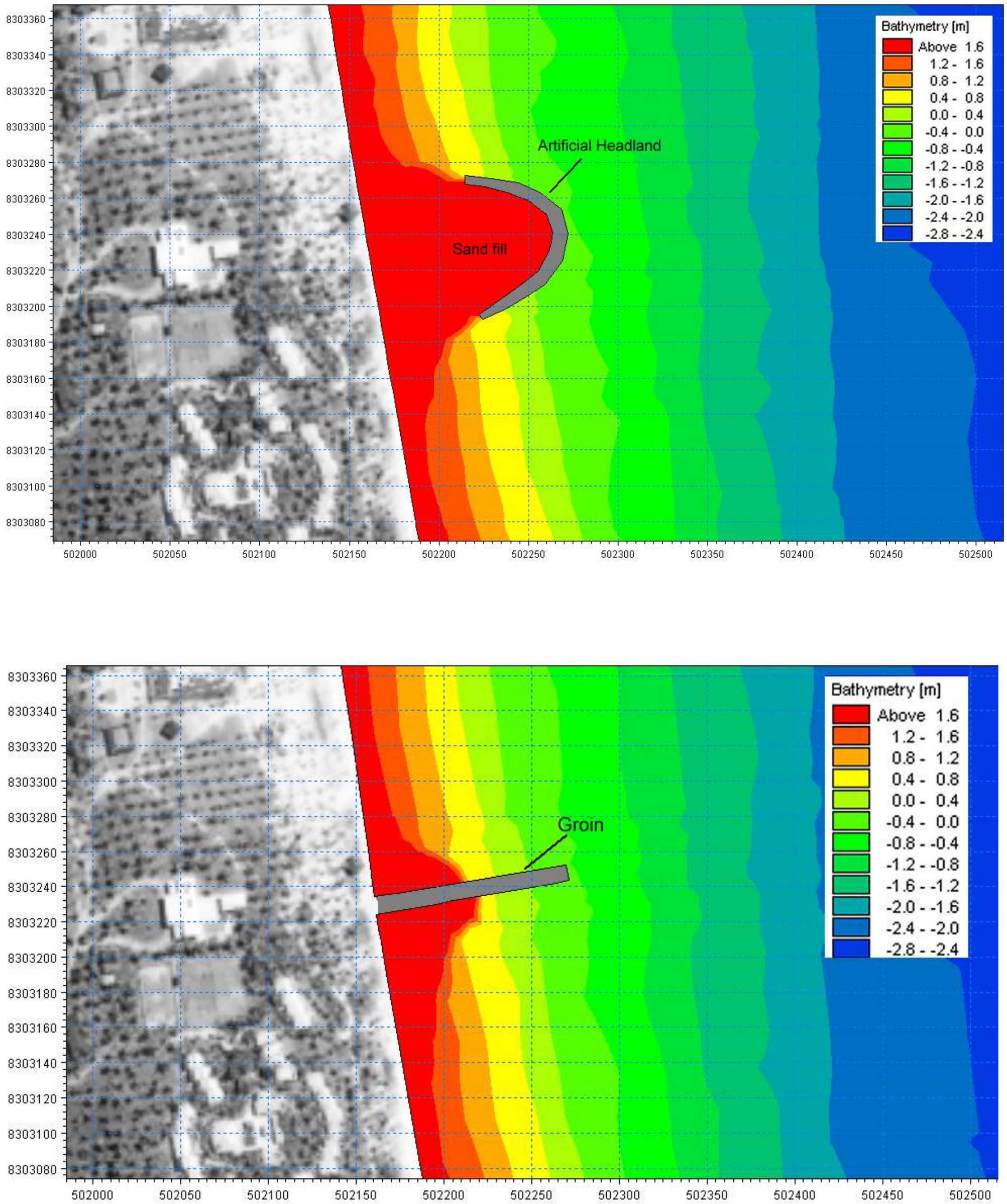


Figure 7.1 Location of the intrusive structures. Top: Artificial Headland, Bottom: Groin

The material for this dune can be derived from the beach through beach scraping during the months when the beach is accreting. The height of the dune must be at least 2.20 m above MWL, which corresponds to an extreme water level with a return period of 100 years. The width of the dune should be in the order of 10 to 20 m, corresponding to an extreme event of shoreline erosion with a return period of 50 years. The dimensions of scheme are shown graphically in Figure 7.2.

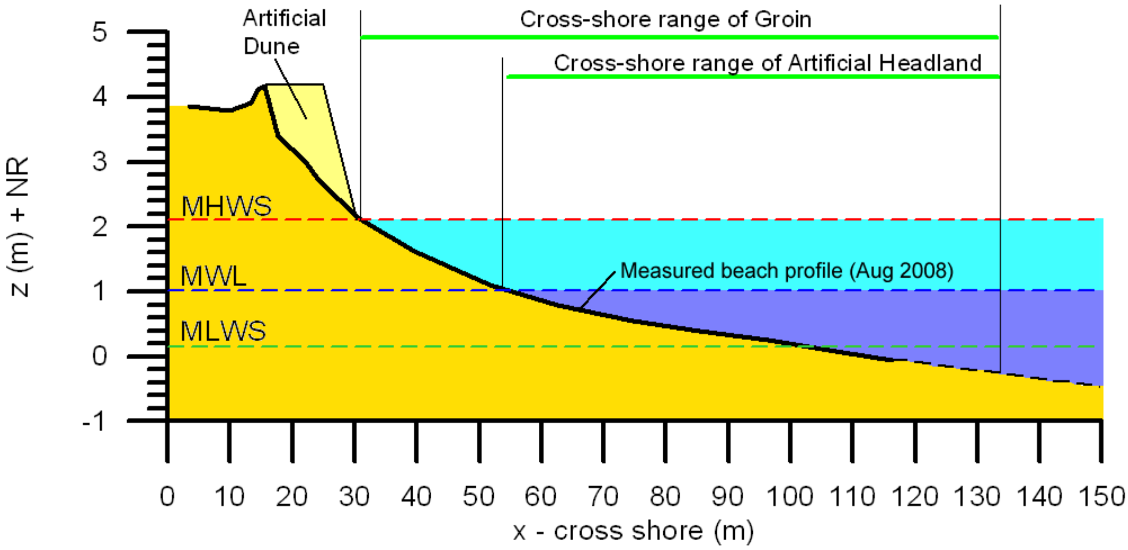


Figure 7.2 Location and properties of the intrusive structure and artificial dune

After establishment of the structure some of the sand accumulated along its southern side can be removed during the period June – October. It must be removed gradually in order not to compromise the safety of the beach. The volume accumulated during one single season will not be sufficient to construct artificial dunes along the entire property at once. Therefore the removal of sand must be done gradually, spread over a couple of years. It is recommended to start the establishment of the artificial dunes in front of the buildings and continue its establishment along the rest of the property later on.

To a certain extent the creation and maintenance of the artificial dune is presently already practiced along parts of the resort. The condition-, height- and width- of the existing dunes must be evaluated. If necessary, the dunes must be reinforced according to the scheme below. It is recommended to fix the dune as much as possible by means of planting the proper dune vegetation. It may be necessary to increase the height of the dune according to the requirements for the chosen type of vegetation's tolerance of saline water. It is recommended to obtain guidelines for planting of the dune vegetation from botanic experts. The characteristics of the recommended scheme are listed in Table 7.1 (intrusive structure) and Table 7.2 (artificial dune).

Table 7.1 Outline of recommended intrusive structure

	Type of structure	
Characteristic	Groin	Artificial headland
Extension seaward	80m from waterline at MWL	80m from waterline at MWL
Extension landward	High Water Line (+1.88m NR)	Mean Water Line(+1.02m NR)
Extension longshore	Not applicable (<10m)	100m
Height	3.25 m (+NR)	3.25m (+NR)
Construction period	April-May	April-May

Table 7.2 Outline of recommended artificial dune

Characteristic	*Artificial Dune*
Toe level	+1.88m (+NR) = MHWL
Top level	+3.24m (+NR) or higher
Width	10m - 20m
Extension longshore	Along buildings and other valuable facilities along the shore
Recommended period for construction and maintenance	June-October (at least one year after establishment of the intrusive structure)

8 CONCLUSIONS

8.1 Wave conditions

1. Time series of offshore wave data was analyzed. The data covered the period 1996-2007. A mathematical wave transformation model (MIKE 21 SW) was set-up and calibrated upon field data. The model was applied to transform the time series of offshore wave data to a number of selected nearshore locations

2. In the nearshore zone (15 m depth) waves are typically around 1.0 to 1.5 m. The wave height rarely exceeds 3.0 m

3. On average the dominant wave directions are E and ESE

4. During the last 16 years, the mean wave direction has shown a gradual clockwise shift of approximately 15°

5. Strong seasonal variations in mean wave direction have been observed, with waves predominantly from ENE during the summer and from ESE during the winter.

8.2 Cross-shore Profile Response

1. The maximal water level elevation (above Mean Sea Level) is estimated around 2.20 m + MSL (100 year event)

2. Maximal shoreline retreat due to cross-shore sediment transport mechanisms is around 20 m (100 year event)

3. A strong seasonal variation in shoreline dynamics was observed. The largest shore-line erosion occurs in the period June – September.

8.3 Littoral Sediment Transport

1. The net annual transport is approximately 1.0×10^5 m^3/year and is directed towards North. The annual northward- and southward components of the littoral transport are in the order of 2.0×10^5 m^3/year and 1.0×10^5 m^3/year, respectively

2. The bulk of the sediment transport occurs within a distance of 200 m. The closure depth is estimated as 3.5m +NR

3. The equilibrium shoreline orientation is calculated as 87 °N, which is approximately 10° away from the present shoreline orientation

4. The magnitude of northward directed transport has increased significantly during the last 2 decades. This has resulted in a shift in net transport direction from South (early 1990s) to North (present)

5. Strong seasonal variations in littoral drift were observed. During the period March – September the net transport is directed towards North, during the remaining part of the year it is directed towards South.

8.4 Concepts of Human Interventions

8.4.1 Intrusive Coastal Structures

From the model simulations performed with intrusive structures, the following conclusions were drawn:

1. Due to strong seasonal variations in wave conditions intrusive coastal structures have a relatively large impact on the shoreline and will cause significant variations in shoreline positions during the year

2. Schemes of multiple structures are not recommended because they cause considerable downdrift erosion during periods with transport towards North

3. In case of one single structure, strong shoreline variations occur only in the vicinity of the structure. The structure should be established approximately 200 m north of the area of prime interest, where natural downdrift erosion will not cause any damage

4. A single intrusive structure located north of the Hotel will provide beach accretion in front of the hotel in the period July – September. This coincides with the period with the highest risk of shoreline erosion due to cross-shore sediment transport mechanisms

5. Model simulations indicate that a single structure can be constructed such that it has a significant blocking effect at higher water levels and allows sediment to bypass the structures almost unhindered at lower water levels.

8.4.2 Non-intrusive Structures

1. Buried revetments can act as a so-called “sleeping defense” which will provide effective protection against flooding and damage due to advancing shoreline

2. The establishment of a revetment as stand-alone solution will not guarantee a stable beach in front of the resort

3. Artificial dunes are an effective and ecologically sustainable way to provide protection against flooding and damage to coastal infrastructure during short-term beach erosion events with elevated water levels such as storm surges

4. Artificial dunes as stand-alone solution do not halt shoreline erosion due to longshore gradients in the littoral sediment transport

5. Beach scraping can be an efficient and sustainable method to increase the natural storage of sand on the backshore, which helps reducing the risk of damage due to beach erosion during storm surges

6. Beach scraping must not be used to extract sand from the beach (sand mining) but only to relocate sand within the coastal sediment cell.

8.4.3 Beach Nourishment

1. Beach material placed initially on the beach in front of the hotel will gradually be transported mainly towards North and partly towards South

2. Beach nourishment has a relative long lifetime (their effect is still noticeable after 15 years)

3. If suitable sand is used, beach nourishment does not compromise the quality of the beach and is not aesthetically unattractive

4. Beach nourishment will have a positive effect even if wave conditions should change drastically

5. Suitable nourishment sand is not available nearby the project site.

8.5 Recommended Solution for the Resort

1. The recommended solution exists of one single intrusive structure, combined with the establishment of an artificial dune. The structure can be established as a classic groin or a more sophisticated artificial headland. The structure must be placed in the northern end of the area to be protected

2. An important aspect of this scheme including one single intrusive structure is that the maximal beach accretion occurs during the period between July and September. This period corresponds to the period where the largest shoreline erosion occurs due to cross-shore sediment transport mechanisms. Correspondingly, the strongest shoreline retreat occurs in February, when the risk of erosion due to cross-shore sediment transport is lowest. This indicates that the scheme helps reducing the risk of damage due to shoreline erosion

3. The advantage of a groin is its relative simple layout and low price. The disadvantages are 1) - risk of flow separation and sediment loss at its head and 2) blocking of passage along the shore as it must be extended on the beach until the high water line and 3) unaesthetic appearance

4. The advantages of the artificial headland are 1) - a more streamlined flow pattern around its head and 2) safer conditions for swimmers and 3) better passage along the beach as the headland does not need to be extended as far up the dry beach as the groin. Its disadvantage is its higher cost

5. Seawards, the structure must extend to 80 m from the mean shoreline position at Mean Water Level

6. The artificial dune must be established on the backshore, with a minimal height of 3.23 m + NR and a width of 10 m to 20 m. The dune must be maintained by supplying sand after event of dune erosion. The sand can be taken from the beach (beach scraping) in front of the dune during periods of shoreline accretion

7. The recommended period for construction of the groin or headland is April-May. The recommended period for establishment of the artificial dune is June-October.

9 REFERENCES

/1/ IBAMA, 2007 Termo de Referencia

/2/ Dominguez, J,M,L. 2008 Relatório Comandatuba Geologia

/3/ Dominguez, J,M,L, 2008 Diagnóstico da Erosão Costeira entre Canavieiras e a Ilha de Comandatuba- Vulnerabilidade da Linha de Costa em Frente ao Hotel Ilha de Comandatuba

/4/ Dean, R.G. 1987. “Coastal Sediment Processes: Towards Engineering Solutions”. Proc. Coastal Sediments ´87, ASCE, New York, N.Y. 1 – 24

/5/ Elfrink, B., D. Accetta and K.H. Mangor. 2006. Innovative shoreline protection scheme for the city of Marataízes-ES. Proceedings International Conference on Coastal Engineering. San Diego, USA

/6/ Elfrink, B., D. Accetta and K.H. Mangor. 2008. Shoreline Protection Scheme at Conceição da Barra-ES, Brasil. Proceedings International Conference on Coastal Engineering. Hamburg, Germany

/7/ Hallemeyer, R.J., 1981. “A Profile Zonation for Seasonal Sand Beaches from Wave Climate”. Coastal Engineering, Vol. 4, 253-277

/8/ Kriebel, David, L. and Robert G. Dean., 1993. “Convolute Method for time-dependent Beach-Profile Response”. Journal of Waterway, Port, Coastal and Ocean Engineering. Vol. 119, No. 2. ASCE

/9/ Mangor, K. H., Shoreline Management Guidelines. (2004). DHI Water and Environment. ISBN 87981950-5-0. 290 pp

/10/ Ruggiero, P., P.D. Komar, W.G. McDougal, J.J. Marra and R. A. Beach, (2001). Wave run up, Extreme water levels and the erosion of properties Backing Beaches. Journal of Coastal Research, Vol. 17, No. 2, pp. 407 - 419