Delft University of Technology

CT5318
FIELDWORK HYDRAULIC ENGINEERING

Preliminary Design of a Floating Marina

December 2009

Preface

This report represents the outcome of the fieldwork of the course 'CT5318 Fieldwork Hydraulic Engineering' held between the Delft University of Technology and the University of Architecture, Civil Engineering and Geodesy, Sofia, Bulgaria. From the 4th until the 11th of October 2009 fifteen students went to Bulgaria to execute field measurements in the coastal zone area of Varna. The aim of this fieldwork was to become familiar with a measuring campaign.

The students sincerely would like to thank Mr. Boyan Savov of the Black Sea Coastal Association, and his son Boyan Savov Jr. for the assistance in executing the measurements and showing us around in Varna. We also would like to thank the University of Sofia for providing the measurement equipment and in particular Mr Kristjo Daskalov. And of course, hotel Azalia for the hospitality.

At last, we would like to thank Mr Henk Jan Verhagen and the Delft University of Technology for making the fieldwork possible.

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Summary

Bulgaria is famous for its long coastline of white and golden sandy beaches. It is therefore a popular destination for tourists during summer. It is expected that tourism will increase in the future. St. Constantine and Elena is a small touristic village along this coastline located on the north of Varna. Some parts of the coastline here are suffering from a retreating coastline which could be a hazard for the hotels on the beach in future.
In 2009 a group of 15 students of the Delft University of Technology went to St. Constantine and Elena to execute some coastal field measurements. The main goal was to become familiar with executing coastal field measurements and apply them on real problems. The analysis of these measurements is the main subject of this report. The report contains different topics, each described below.

The beach in front of the hotel Sirius is suffering from a retreating coastline. Bathymetry and beach profiles were measured. With the data from previous years the long term evolution of the coast was analyzed. The main conclusions were that the beach in front of the Sirius hotel is still suffering from erosion and if this erosion trend continues it will damage the hotel itself in just a few years. More to the south the beach was actually a little bit accreting. The reason why only the beach in front of the hotel is eroding is probably because the beach there is at the leeside of the breakwater, which causes a gradient in the longshore sediment transport

The hotel owner of Azalia, located at the Azalia beach north of Sirius beach, observed a retreating coastline during some months in the year. Therefore also here the bathymetry and beach profiles were measured. Unfortunately there were no previous measurements to compare the data with. From the beach profile we can conclude that there is probably a seasonal variation in the coastline. During winter the coastline is retreating due to the more severe wave climate. Yet in summer the beach is accreting. It can be possible that sometimes the beach has not accreted enough at the moment the touristic season starts. This can be a problem for the hotel owner because the beach is not wide enough then. A solution then can be found by some minor dredging operations in the form of beach nourishments. The sand can be recovered from deeper water or from shoals at the same beach. It can be assumed that the nourishment is not necessary at the moment but might be a good idea in the next years.

Furthermore the owner of hotel Azalia wants to build a new marina a few kilometers to the south of St Constantine. Due to difficulties in obtaining a building permit it will be a floating marina which can be removed in winter, which must be protected by a breakwater. By a Multi Criteria Analysis a floating breakwater that will be removed during winter proved to be the best solution in this case. This breakwater can be designed with the milder wave conditions during summer which is a serious advantage.

During the fieldtrip more measurements have been carried out. Quarries were visited to analyze if the rock from these quarries is suitable for a conventional breakwater. However for the design and construction of a floating breakwater rocks are not necessary. But the rock of the quarry is suitable for a breakwater along the Bulgarian coastline because the wave climate is rather mild.

Also sand samples were taken and analyzed from Azalia beach and Sirius beach. There is no direct link with these measurements and the purpose of the report except for educational reasons. Finally the evolution of the St. Constantine breakwater is analyzed.

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1. Introduction

Bulgaria is located in the southeastern part of Europe and is connected to the Black Sea in the east. Bulgaria is famous for its 378 kilometer long coast line with sandy white and golden beaches. In summer, Bulgaria is therefore a popular destination for tourists. During the summer period, marinas are full with yachts.

It is expected that the tourism will increase in the future; competition between the hotel and marina owners is very high. Preservation of their beaches is important to hotel owners to maintain their valuable position and status in the market.

To the north of Varna a small village called St. Constantine i Elena is located. This village is famous for its sandy golden beaches and tourism is therefore the main source of income.

To guarantee enough capacity in the future for an increasing number of tourists, new hotels and marinas could be constructed. Yet the town has to cope with a locally retreating coastline, which could cause serious problems for the future economical prospects.

To shed some light on these matters a group of 15 students of the Delft University of Technology went to Bulgaria to carry out some field measurements in October 2009. These measurements had different objectives which are shortly described below.

1.1 Obtain information about the evolution of the coastline

In the last decades many parts of the coast of Varna and St. Constantine I Elena have shown signs of deterioration or suffer from a retreating coastline. The beach in front of the Sirius hotel, which has almost completed been washed away, has been measured in the past already. To identify a trend in the erosion of this beach long term measurements are necessary, therefore also in 2009 the bathymetry and coastal profile are determined at this location and compared to previous data.

Azalia beach, a beach to the north of Sirius beach, also shows a retreating coastline for some months per year and the hotel owner is afraid that erosion problems might be a hazard for the hotel in the future, similar to Sirius beach. Therefore measurements on the bathymetry, beach profiles and coastline were executed to give qualified information.

1.2 Preliminary design of a floating marina

Hotel Azalia wants to extend its activities by developing a marina a few kilometers to the south of the Azalia hotel. Obtaining permits for coastal structures is a problem in Bulgaria. It was proposed to construct a floating marina. The advantage is that for the construction of a floating marina different permits are needed; these are easier to obtain. Furthermore a floating marina can be removed in winter, so it does not have to withstand the severe winter wave climate. A preliminary design for this marina has to be made. For the design boundary conditions are required. Field measurements were carried out to determine the bathymetry and the wave characteristics at the desired location.

The marina has to be protected by a breakwater. The Bulgarian wave climate is rather mild in summer, yet heavy waves may occur in winter. Strong breakwaters have to be designed hence information is gathered about the available rock from a quarry to explore the possibilities for the construction of a conventional or submerged breakwater.

Another possible type of breakwater might be a floating breakwater which can be removed in the winter.

With the aid of the obtained data, a preliminary design of a floating marina and a breakwater is made.

1.3 Outline report

In this report first the some general information about Bulgaria and its coastline is given in chapter 2. In detail, the coastline to the north of Varna is described. Also the locations of the areas where the fieldwork was executed are given. In chapter 3 the evolution of the St. Constantine breakwater is treated. In chapter 4 the morphology of Sirius beach and Azalia beach is analyzed with the obtained data from the measurements. Also sand samples were taken and the characteristics of the sand were determined, this is elaborated in chapter 5.

Subsequently in chapter 6 it is investigated if stones are available from quarries in the vicinity of Varna and if these are suitable as construction material for a possible conventional breakwater. After this a preliminary design of the floating marina is presented in chapter 7. The design is established with the collected boundary conditions and design criteria including a Multi Criteria Evaluation of the possible breakwater types and an economical consideration. Finally in chapter 8 the overall conclusions of the fieldwork are presented and some recommendations are given for future measuring campaigns.

2. Location

In this chapter first a general overview will be given of Bulgaria and the investigated area in section 2.1. In the subsequent paragraphs a characterization of the coastline and the coastal protection structures near Varna will be given. Finally the Varna port and the future development plans are treated in section 2.5.

2.1 Bulgaria

Bulgaria is a country in the southeastern part of Europe. In the north, west and south side Bulgaria borders Romania, Serbia, Republic of Macedonia, Greece and Turkey. In the east it is connected to the Black Sea. The 378 km of coastline, also called the Bulgarian Black Sea Riviera, covers the entire eastern bound of Bulgaria. Approximately 130 km of the coastline is occupied by white and golden sandy beaches. This region is one of the most popular tourist destinations of the country, especially during summer. One of the two largest cities and main sea ports on the Bulgarian Riviera is Varna. Varna is the third largest city in the country and is located in the northern part of the coast.

Figure 2-1: Map of Bulgaria

2.1.1 Black Sea

The Black Sea is an inland sea and is connected to the Aegean Sea and the Mediterranean Sea through the Bosphorus strait and the strait of the Dardanelles. Currently the Black Sea water level is relatively high due to a high river run-offs and water is exchanged with the Mediterranean Sea. Consequently there hardly is a tidal difference in the Black Sea.

2.1.2 Overview of measurements carried out at Varna and St Constantine I Elena

To the north of the city Varna is a little town named St. Constantine I Elena. The field measurements have been carried out here. An overview of the measurements that were carried out within this area is given in figure 2-2.

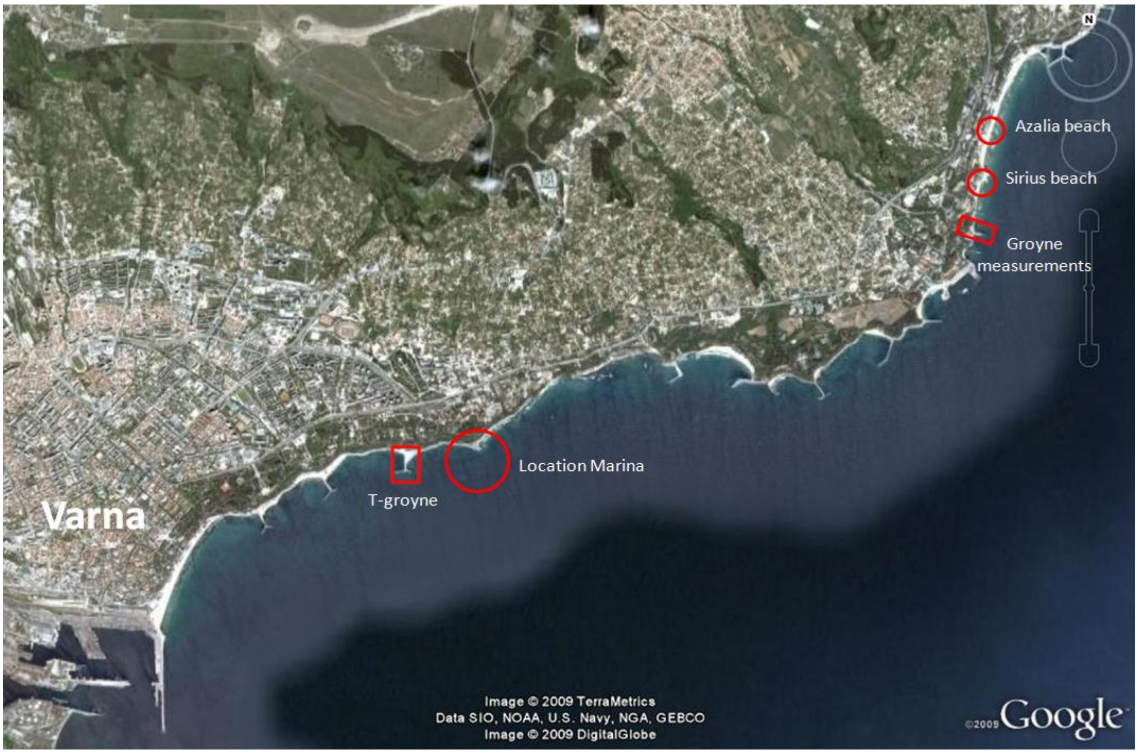


Figure 2-2: Overview of the measurements carried out around Varna and St. Constantine I Elena

2.2 Varna beach

Varna Beach is here considered as the part of the beach close to the city center of Varna. The south end of the beach is adjacent to the eastern Breakwater, which shelters the port of Varna. Going north along the beach several groynes are found. An overview of Varna beach is represented in figure 2-3.

Figure 2-3: Overview of Varna beach

The sand that is available comes from landslides and eroding cliffs. Most of the sediment originates from cliffs. These cliffs exist of lime stone and are slightly eroding during the north eastern storms in the winter season.
In the past sand was also supplied by the river Devnya. However, in the past some reservoirs functioning as a sand trap were created. The high river discharges which were accompanied with high sediment concentrations do not occur anymore due to the controlled flow at the reservoirs. Hence the river does not provide sediment anymore.
Considering the above, it can be concluded that due to a decrease of available sediment erosion of the beach is very likely to happen. This erosion pattern of Varna beach can be recognized in different ways.

First of all by a track along the waterline recorded with a GPS device. This track has been imported into "Google Earth" which gave the opportunity to see changes of the waterline. In the figure below the red line represents the track of the waterline, recorded on October the 5th 2009. The picture of Google Earth originates from May 3rd 2007. A remark can be made about the beach erosion at two locations, pointed out in figure 3. In front of the eroding parts of the beach, the blue watercolor is darker than at other places, which indicates deeper water over there. A consequence of the deeper areas in front of the coastline can be that the waves will not break until they reach the beach and the wave attack on the beach is more severe.

Figure 2-4: Waterline Varna Beach

Secondly, to show the Varna beach erosion several pictures are made during beach walks along the coastline. These pictures are shown below and are, if possible, compared with pictures of previous years (from other fieldwork groups).
Halfway through the beach there is a photo series of one building at three moments in time, which clearly shows the erosion. Three years later, in 2008, (figure 2-6) the balcony has been shortened and finally in 2009 (figure 2-7) even the foundation has started to become visible.

Figure 2-5: Year 2005

Figure 2-6: Year 2008

Figure 2-7: Year 2009

Figure 2-8: Visible roots

On several other pictures signs of erosion are visible as well. Trees cannot hold the soil between their roots anymore (figure 2-8). In figure 2-9 a building is displayed which has been partly removed already. Also figure 2-10 shows a foundation which has come above the surface.

Figure 2-9: Partly removed building

Figure 2-10: Visible foundation

2.3 Varna beach T-Groyne

The T-groyne, as depicted in figure 2-10, stretches out into the sea for about 150 meters with a 150 meters wide head parallel to the coast. In principal the structure was designed to prevent erosion of the lime stone rocky coast of Varna bay, to prevent littoral sediment transport and to create a beach over the full length of the coastline. The groyne should trap the sediment, which would create a beach upstream of the groyne. Many groynes have been built along the Black Sea coast for this purpose.

Since most of the cliffs upstream are stabilized by rock-revetments, the net littoral sediment transport is very low. Accretion on the east (right) side of the groyne is therefore very low over the last 30 years (figure 2-11).

Figure 2-11: Overview T-groyne

The initial plan to create a beach over the full length between the land and the off-shore breakwater did not succeed for two reasons. The first one was the collapse of the communism; there was no funding from the government for this project anymore. The second reason was that the municipality did not give permission to the contractors to transport sand through the city. In the present situation the inner side of the groyne functions as a mooring place for small ships.
The T-groyne consists of two parts. First a concrete wall was built, but solely this wall was not able to resist the wave attack during the winter. Therefore an armour layer of Tetrapods was placed at the sea side to absorb most of the wave energy. The two parts together make a solid yet over dimensioned structure. If a cost-benefit analysis would be carried out, this would probably turn out to be an unattractive construction.

2.3.1 Calculation of the design wave height

The amour layer of the t-groyne exists of Tetrapods. These Tetrapods will be attacked most heavily during the wintertime. The winter storm comes from the north-east direction which gives the governing waves for the design wave height.
To determine for which waves this structure has to be able to resist, a calculation can be made, since we know the sizes of the Tetrapods. Two different sizes have been applied: Tetrapods with a leg length of 85 cm and 115 cm.

Mass of Tetrapods

With the formulas from the Shore Protection Manual (1984), other dimensions of the Tetrapods can be determined.

First relation:
$C = 0.477*H$

In which C is the leg length and H is the total height. See figure 2-12.

Figure 2-12: Dimensions Tetrapod

Second relation:

$$V = 0.280 * H^3$$

In which V is the volume of the Tetrapod and H is the total height.
For calculation of the mass we assume the density of concrete to be 24 00 kg/m^3
Applying these formulas results in table 2-13.

	Leg length C	Height	Volume	Weight	Weight
Small Tetrapod	0.80 m	1.68 m	1.32 m^3	3170 kg	31.1 kN
Large Tetrapod	1.15 m	2.41 m	3.92 m^3	9400 kg	92.2 kN

Table 2-13: Sizes T-groyne Tetrapod

2.3.2 Calculation design wave

To verify the design wave, applied to the T-groyne, several methods can be used, the method of Hudson, Van der Meer or Hanzawa. In this calculation the “Large Tetrapod” is considered with a height of 2.42 m.

<u>Hudson</u>

The formula:

$$\frac{H_{sc}}{\Delta d} = \sqrt[3]{K_d \cot\alpha}$$

This formula takes the slope and a damage factor Kd into account. The damage factor makes use of the following values from the Shore Protection Manual (SPM 1984):

K_d-values	Breaking waves	Non-breaking waves
Trunk	7	8
Head	5	6

Table 2-14: K_d^- values for the Hudson formula

For this breakwater the K_d factor is 5, because the wave will break at the head of the breakwater. The other parameters to be determined:

$$\Delta = \frac{\rho_s}{\rho_w} - 1 = \frac{2400}{1025} - 1 = 1.34$$

$$\cot\alpha = 1.5$$

$$d = 1.15$$

Inserting these parameters in the general Hudson formula gives the design wave height:
H_{sc}= 3,01 m.

Van der Meer

According to Van der Meer a distinction should be made between plunging and surging waves attacking the breakwater. Therefore a breaker index is used which determines the method of approach.

Breaker index:

$$\xi = \frac{\tan\alpha}{\sqrt{s_m}} = 2.98$$

s_m = wave steepness for standards waves at shallow water.
The transition area of waves plunging or surging is about 3 according to the breaker index. If $\xi<3$ then the wave will plunge, if $\xi>3$ then the waves will surge. So in this case both methods will be calculated.
Some parameters needed:
N_{od}=0.2 (Damage level is low, so 0.2 is sufficient)
N = 2700 (number of waves with a period of 8 seconds in a storm of 6 hours)

Plunging waves: $\frac{H_s}{\Delta d} = (8.6(\frac{N_{od}}{\sqrt{N}})^{0.5} + 3.94)s_m^{0.2} = 2.46$ → H_s = 3.79 m
Surging waves: $\frac{H_s}{\Delta d} = (3.75(\frac{N_{od}}{\sqrt{N}})^{0.5} + 0.85)s_m^{-0.2} = 1.97$ → H_s= 3.03 m

Van der Meer's method shows that it is important to know if the waves will surge or plunge, since there is a considerable difference between the two values for the design wave height.

Hanzawa

Hanzawa developed a method of approach especially for Tetrapods.

$\frac{H_s}{\Delta d} = 2.32(\frac{N_{od}}{N_z^{0.5}})^{0.2} + 1.33 = 2.10$ → H_s = 3.23 m

2.3.3 Conclusion design wave

As shown in the last paragraph different approaches gave different values. The design wave heights vary between 3.01 m and 3.79 m. Which method has been applied is not sure. It can be concluded that this breakwater design is very solid, since it can withstand a 6 hour storm with at least 3 m high waves. The significant wave height at the toe of the structure is approximately 1,5m (see chapter 7.2.3) and thus it seems that the whole design is over-dimensioned.

2.4 Varna Eastern Breakwater

The sea port of Varna East is connected to the Black Sea. In wintertime north-eastern storms can cause severe wave action where the port has to be protected from. For this purpose the eastern Breakwater was constructed.

The eastern breakwater creates a sheltered area for ships to moor. Right on the inside an area for yachts has been reserved. At the southern tip of the breakwater Saint Nicholai's ornament labels the light house positioned there. Saint Nicholai is a patron saint of sailors, fishermen, ships and sailing.

Figure 2-15: Eastern breakwater

Figure 2-16: St. Nicholail light house

At the northern end, where most of the ships are moored, the breakwater is higher than at the head. This has been done to reduce overtopping and to provide more shelter. The initial breakwater looks like a very big steep concrete stair, this unusual design had many upgrades to fulfill the requirements. After several upgrades a combination of rocks, Tetrapods and a concrete crown wall form the structure at present, which is clearly visible in figure 2-17.

Figure 2-17: North end upgrades

An upgrade which cannot be noticed in figure 2-17 is the installation of concrete boxes at the bottom in front of the break water. They were placed in order to break the waves before they reach the breakwater. In figure 2-18 a sketch of a cross section (the initial profile with the later placed concrete boxes) of the northern part of the breakwater is given.

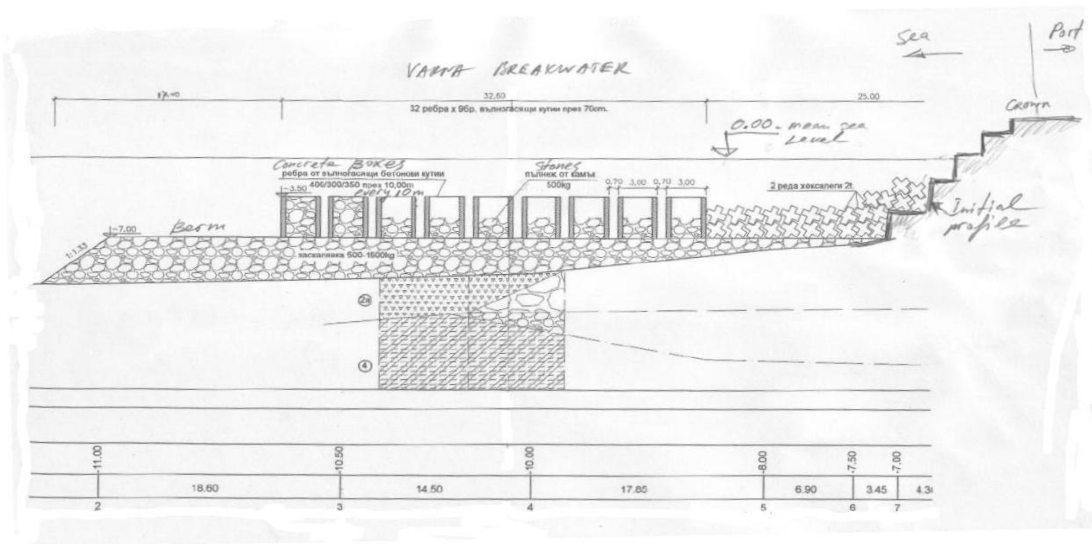


Figure 1-18: Cross section Northern part breakwater

At the middle section of the breakwater it consists of only a concrete core. No ships have to moor at the inner side there, so overtopping does not seem to be a problem. Figure 1-19 represents the construction at the middle section. Figure 1-20 displays a picture with quite some overtopping at the breakwater during a winter storm.

Figure 2-19: Halfway Eastern breakwater

Figure 2-20: Overtopping during storm

At the tip head the breakwater additional Tetrapods have been applied again, probably for more protection of the light house. In figure 2-21 there is a picture of these Tetrapods taken from the light house.

Figure 2-21: Tetrapods around light house

Figure 2-22: Close-up Tetrapods

The Tetrapods at the head of the breakwater were not of supreme quality. Figure 2-22 shows a close-up of the Tetrapods. Various Tetrapods are missing a leg, probably due to bad construction, or they have wide cracks, which can be the result of applying concrete of insufficient quality. Another reason for the broken Tetrapods is that the mould was filled with concrete on two different days resulting in a bad connection between the two casts and low tensile strength.

2.5 Varna's port development

2.5.1 Current situation

The port is situated in a lake that has an elongated shape. The south shores are high, steep and wooded and the northern shores are slant. Lake Varna was formed in a river valley. The bottom is covered by a thick alluvium of slime and hydrogen sulphide mud in the deepest parts.

Lake Varna is connected in the west to Lake Beloslav. The two main rivers discharging into Lake Beloslav are the Devnya and the Provadiyska. These rivers are visible on the left side in figure 2-23.

Figure 2-23 Overview Port Varna

The sea port of Varna East is located close to the city center. The eastern port has two canals, which connect Lake Varna with the Black Sea. The two channels have draughts of respectively 11.5 m and 11.0 m. The island in between these two channels is occupied with a deepwater oil terminal. Vessels calling at the port have a maximum capacity of 50.000 gross tons. Due to safety requirements the vessels may have a draught of at most 9.9 meters.
Further the Port of Varna offers full service: loading, discharging, stevedoring, forwarding, storage and various intermodal services.

2.5.2 Future Plan

In 1999 a general plan for the Port of Varna was approved by the government. The plan consisted of a development, which should be executed within 2 decades, until 2020. The major projects for new construction, reconstruction and modernization include a deepwater container terminal and a ro-ro terminal.
Other projects are the a grain terminal on the north shore of Lake Varna, a liquid chemicals terminal and a cement and clinker terminal at Varna West, and a modernization of the passenger and ro-ro terminals at Varna East.
New plans were disclosed in 2007. The container terminal will be relocated from Varna East to a new larger basin on the north shore of Lake Varna. The old port of Varna East, located in the city center will be redeveloped into a large marine attractions zone with a new cruise terminal, yacht marina and other tourist facilities.

Figure 2-24: Location soil measurements

2.5.3 Geotron

For the realization of the new larger container terminal at the north shore of Lake Varna (west of the city Kazashko) new berths have to be constructed. A Dutch company Geotron is investigating the bottom characteristics for part of the construction of the port extension. In figure 2-24 the location of the soil measurements is shown. Geotron was doing two types of measurements; A Cone Penetration Test (CPT) and the Hollow Stem Auger.

The CPT is done to determine the soil geotechnical property of bearing capacity. The test gives the total penetration resistance when pushing a tool with a conical tip into the soil.

When visiting the pontoon a CPT has been carried out. Geotron explained that during this test, the CPT equipment pressed until a maximum pressure of 10 tons was reached. Before the Cone was lifted back up, it was left in position for some time to measure the time it takes for the pore water pressure to reduce to a steady level again, because during the tests extra pore pressure develops as the cone penetrates deeper into the soil. In figure 2-25 a picture of the CPT device positioned on the pontoon is visible.

Figure 2-25: Cone penetration Test at pontoon

The second measurement Geotron was performing at another pontoon is called the Hollow Stem Auger. This test determines the types of soil in the bottom of the lake. The Hollow Stem Auger will cut and retain a solid plug of material and haul it to the surface for easy removal. The holes require support during the drilling operations; therefore a steel casing is used to prevent the holes from collapsing during the drilling operations. In figure 2-26 a Hollow Stem Auger is being emptied. From the measurements it followed that the top layer consists of contaminated sludge with underneath mud and clay and finally a layer of rock.

Figure 2-26: Removal of solid plug

3. Groyne Measurement St. Constantine

To determine the state and the rate of deterioration of the St. Constine groyne several cross sections have been measured. After a general introduction in section 3.1 and a site description in section 3.2, the measuring method is explained in section 3.3. The cross sections are compared in section 3.4 which is followed by the conclusions and recommendations in section 3.5 and 3.6.

3.1 Introduction

Groynes are continuously exposed to hydrodynamic forces, such as waves and currents. By regularly analyzing the state of the groyne it can be investigated if the groyne is designed according to the right design criteria; whether it still sufficiently fulfills its function or if restoration is necessary.
To the south of Sirius beach in St. Constantine I Elena there is the so called St. Constantine groyne. In previous years measurements of the cross-profiles of the groyne were carried out. By measuring the profiles of this groyne periodically the current state of the groyne with respect to the past can be identified. Qualified information of the behavior of the groyne on hydrodynamics forces can be given.
Profile measurements (of a groyne or breakwater) are also carried out to determine the as-built state of the structure and to be able to recalculate the amount of stones placed by the contractor for payment. Therefore it can be stated that it is an important task that should be done and understood correctly. In this case it is a good opportunity to see and experience how a dumped stone structure survey is performed.

3.2 Location

The groyne is located on the south-side of the St. Constantine I Elena resort, down south along the beach from the hotel Sirius. The groyne is composed of a core of small caissons protected with armour rock at the sides. In the centre of the caissons a water conduit is implemented, meant to serve as a water inlet or outlet, but it never served its purpose.

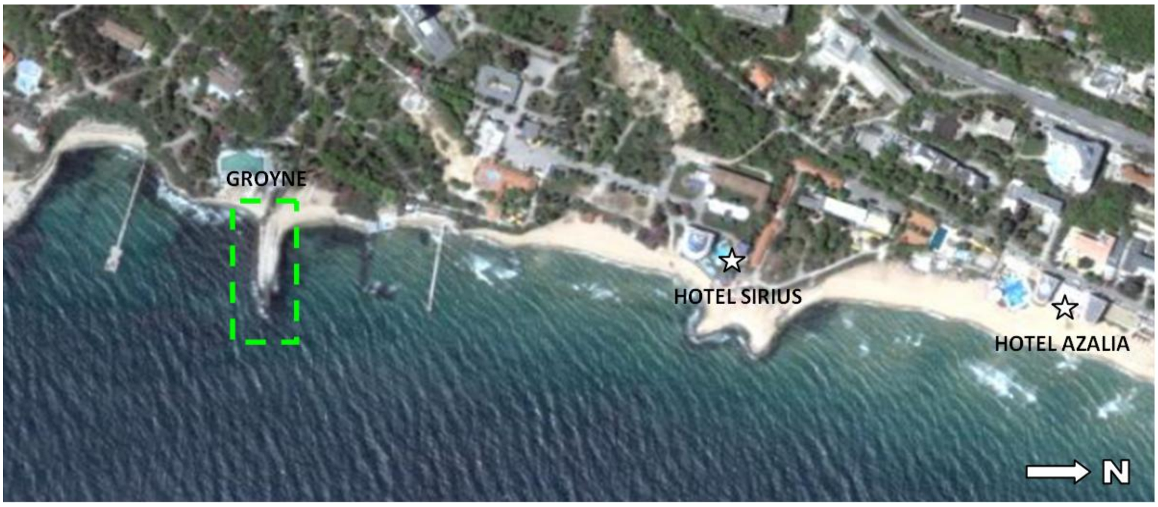


Figure 3-1: Location of the groyne with respect to the Hotels Azalia and Sirius

Figure 3-2: Armour rock North-side (left), Caisson core (middle), Armour rock South-side (right)

3.3 Measurement setup

In order to get a correct topographical survey of the structure it is important to stipulate a methodical measurement plan in advance of the measurements. The total measurement consists of 12 cross-sectional profile measurements with an in-between distance of 5m of both the north and south side of the armour stone protection. These cross-sections lay perpendicular on a baseline along the groyne. The baseline is drawn straight along the crest of the groyne towards the coast from a base point which was defined in 2002. The location of the base point is considered to be stable and not to vary over the years. The width of the crest of the groyne is 9.5 m. The baseline lies 1.5 m from the southern edge in y-direction. The following pictures show a sketch of the co-ordinate system used to set-up the survey and pictures of the location of the base point.

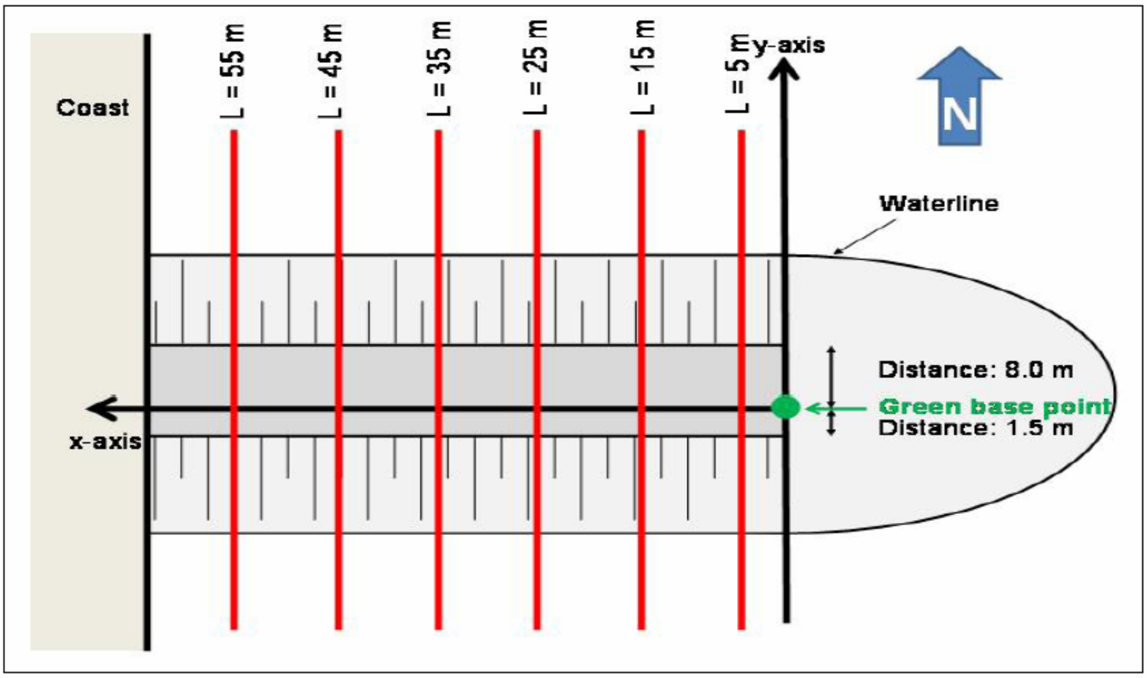


Figure 3-3: Co-ordinate system applied on the groyne (taken from Fieldwork Hydraulic Engineering report 2008)

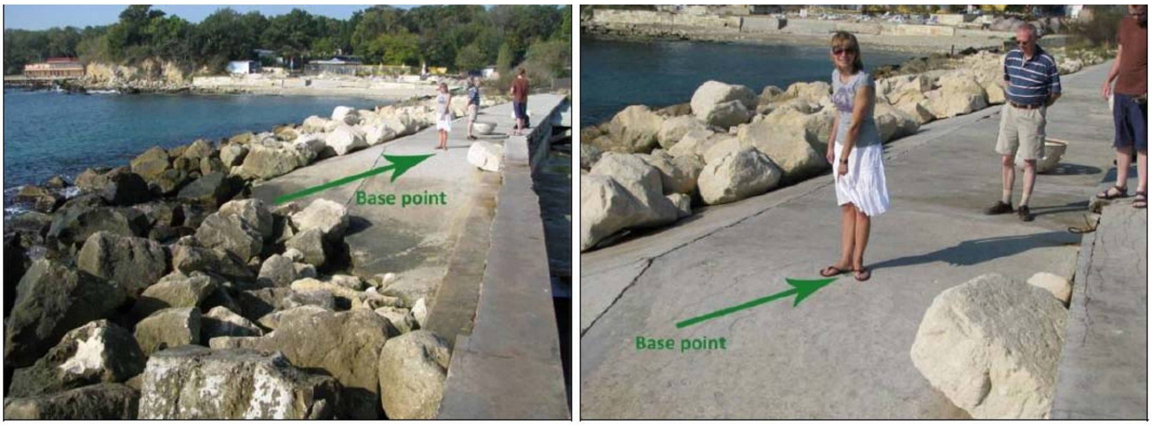


Figure 3-4: Location of the base point

With the baseline set up the survey of the cross-sectional profiles could start. The first measured cross-sectional profile was at L = 0 m, after that every 5 m a cross-profile was measured perpendicular to the baseline, ending at L = 55 m.
In the measurement of 2002 and 2004 6 cross-sections were measured, starting at L=5 m with an in-between distance of 10 m, ending at L=55 m.
The groyne measurements performed in the other years than 2002 and 2004 were of either a different groyne, had other cross-sections, or were incomplete or unavailable. Therefore from now on, even though 12 cross-sections were surveyed, only the cross-sections of L=5 m, L=15 m, L=25 m, L=35, L=45 m and L=55 m are considered. This makes it possible to make a comparison for the years 2002, 2004 and 2009.

With the inconsistent surveys of the previous years it is advised to follow from now on the above described setup (profile measurements for L=5 m, L=15 m, L=25 m, L=35, L=45 m and L=55 only) in the following years of the Fieldwork. By doing so consistency is created and the already largest database is further expanded.
The measurements are performed using a theodolite, a measuring rod fixed to a hemisphere and a measuring tape. Measurements are done relative to the height of the considered stable base point. The hemisphere at the end of the measuring rod is used to smoothen the measured profile since it prevents that the rod is being positioned in a gap between two stones.

3.4 Cross-section comparison

In the following graphs the profiles of the various cross-sections are plotted. The baseline is located at x=0. The caisson is considered stable over the width and lies between -1.5 m and 8.0 m. The negative values along the x-axis represent the southern side of the groyne and the positive values represent the northern side. The positive values along the y-axis are depths below the base point. Negative values indicate points where rocks have a larger elevation than the base point.

3.4.1 Cross-section at L = 5 m

Figure 3-5 represents the cross profile of the groyne at a distance of five meters from the base point.

Southern part

From the lines representing the cross-sections of the different years it is easy to see that some stone at the top of the revetment have moved down slope. Since the upper stones seemed relatively dry and unaffected by direct wave forces the cause for this can be found in the fact that stones further down the slope have moved at first and that sliding occurred. The overall picture is more or less the same over the years. With an exception of a single stone moved at the bottom of the slope, where the waves constantly impact on the stones. The caisson structure is still well-protected.

Northern side

Data from 2002 is unavailable for the northern slope of the cross-section at L=5 m. Comparison of the 2009 profile to the 2004 profile clearly shows the movement of stones from the upper part to the lower part. The amount of volume lost and gained in the different parts is almost equal, so no stones are directly lost from the profile, rather relocated. But the caisson is not directly protected by an armour stone layer in the upper part, which might lead to problems in the future.
The stones on the northern side have smaller dimensions for an unknown reason. The difference in dimensions partly explains the bigger change of the profile with respect to the south side.

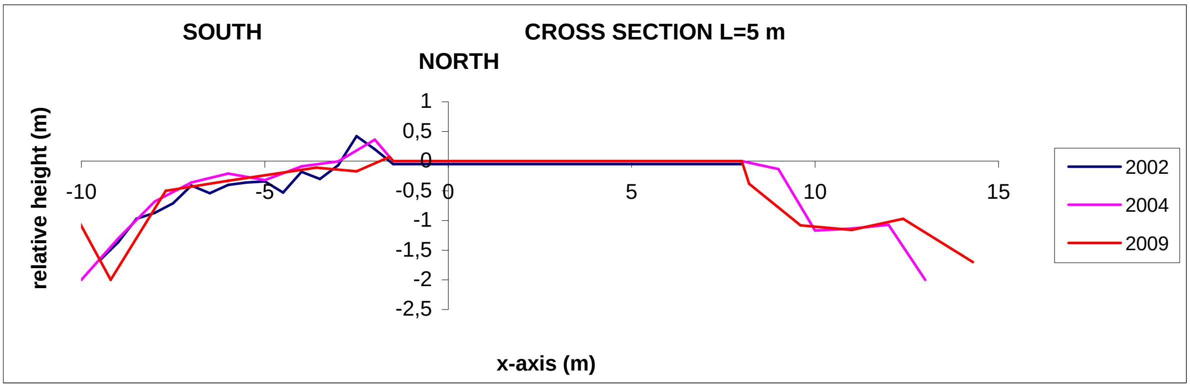


Figure 3-5: Cross-section at L=5 m

Figure 3-6: South side (left) and north side of the groyne (right) at L=5 m

3.4.2 Cross-section at L = 15 m

Figure 3-7 represents the cross profile of the groyne at a distance of fifteen meters from the base point.

Southern part

The profile of 2009 clearly shows an increase in the average level of stones on the south side, especially the upper part but a reason of this is unknown. Storms may have transported the stones up in the cross-section, but this is unlikely. Maybe people changed the position of the stones manually. Or maybe a contractor filled in an exposed area some time ago. If stones are removed from the bottom part of the structure this might lead to sliding and adjustment in the future again. For now the caisson structure is well-protected.

Northern part

In the northern part of the profile stones are shifted a little bit from the top to the bottom part. The overall profile stayed constant over the years though and the caisson structure is well protected.

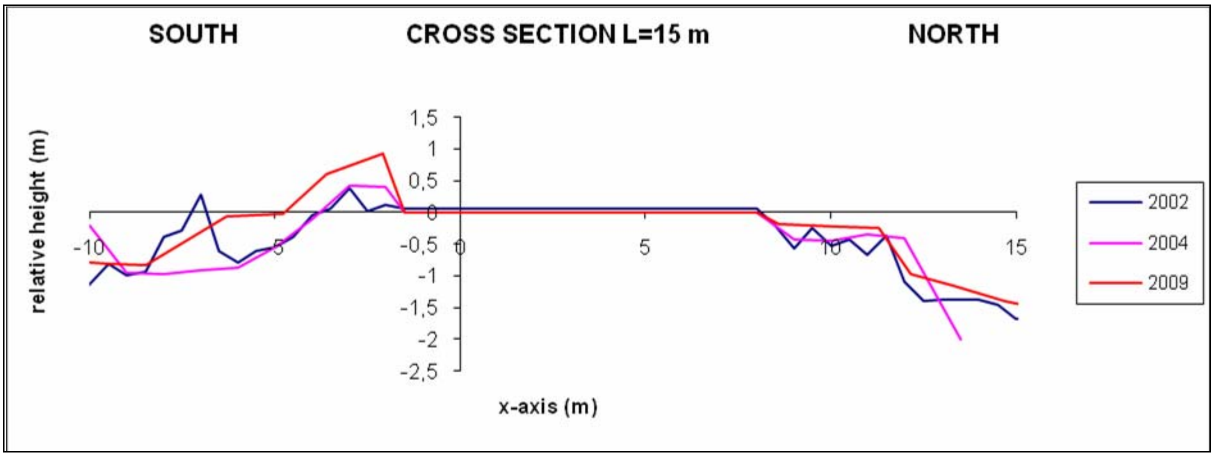


Figure 3-7: Cross-section at L=15 m

Figure3-8: South side (left) and north side of the groyne (right) at L=15 m

3.4.3 Cross-section at L = 25 m

Figure 3-9 represents the cross profile of the groyne at a distance of twenty five meters from the base point.

Southern part

Both the 2004 and the 2009 profile showed a decrease of the overall level of the caisson with respect to 2002. The 2002 profile of the caisson is located above the base point. This seems wrong, therefore for the analysis it is accepted that there is a small overall difference. The general profile is identical with a small shift here and there. It can be stated that the overall profile has not changed much and that it protects the caisson structure well.

Northern part

The northern profile also has not changed much over the years and is considered stable.

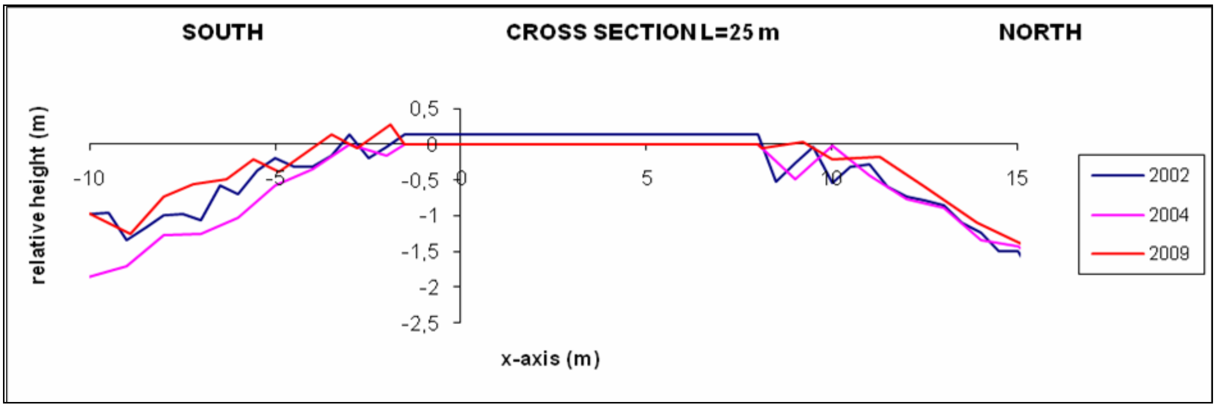


Figure 3-9: Cross-section at L=25 m


Figure 3-10: South side (left) and north side of the groyne (right) at L=25 m

3.4.4 Cross-section at L = 35 m

Figure 3-11 represents the cross profile of the groyne at a distance of thirty five meters from the base point.

Southern part

In the overall profile there is a significant shift visible. This shift can be explained with an error during the repositioning of the theodolite and writing this shift down, a mistake of a novice surveyor. When this shift is neglected it can be concluded that the profile is almost perfectly similar to the previous measurements with the exception of a small surface smoothening in the lower part of the revetment possible due to the equipment used (different hemisphere), due to a different distance in-between measurements or as a result of the reshaping of the profile by wave action.

Northern part

Also for the northern part, neglecting the level shift, the profile stayed the same over the years with an exception of some stone movement downwards from the upper part of the revetment. Overall the protection of the caisson structure is good.

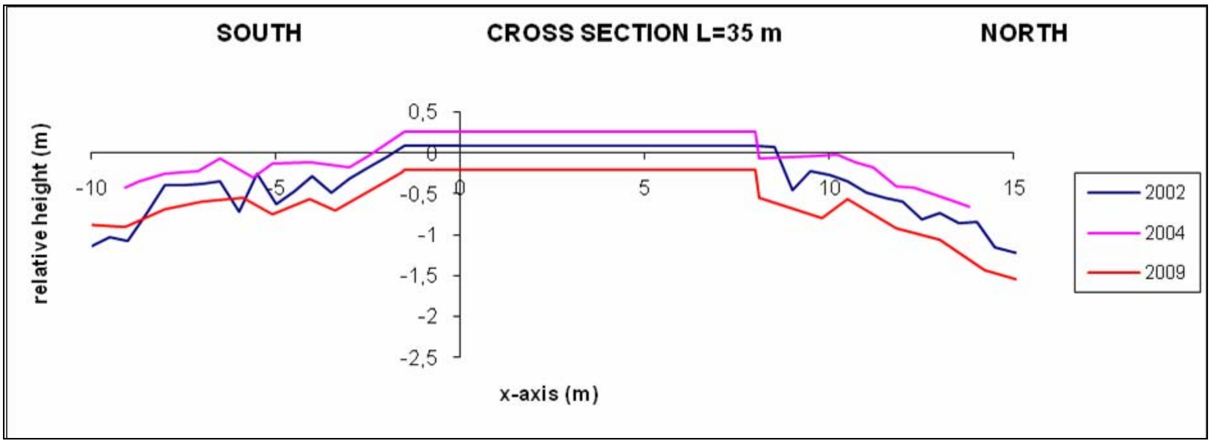


Figure 3-11: Cross-section at L=35 m

Figure 3-12: South side (left) and north side of the groyne (right) at L=35 m

3.4.5 Cross-section at L = 45 m

Figure 3-13 represents the cross profile of the groyne at a distance of forty five meters from the base point.

Northern and southern part

The shift also applies for the cross-section at L=45 m. For both the south side and the north side it can be said that, if the shift is neglected, the overall profile did not change much over the years and that the caisson is well protected.

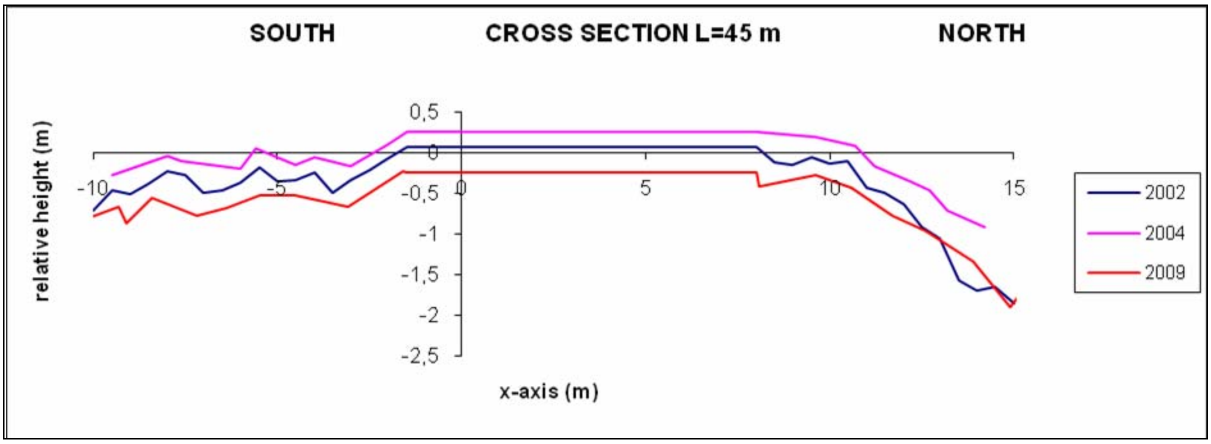


Figure 3-13: Cross-section at L=45 m

Figure 3-14: South side (left) and north side of the groyne (right) at L=45 m

3.4.6 Cross-section at L = 55 m

Figure 3-15 represents the cross profile of the groyne at a distance of fifty five meters from the base point.

Northern and southern part

Finally, the shift also applies for the cross-section at L=55 m. For both the southern and the northern part it can be said that, if the shift is neglected, the overall profile did not change much over the years and that the caisson is well protected.

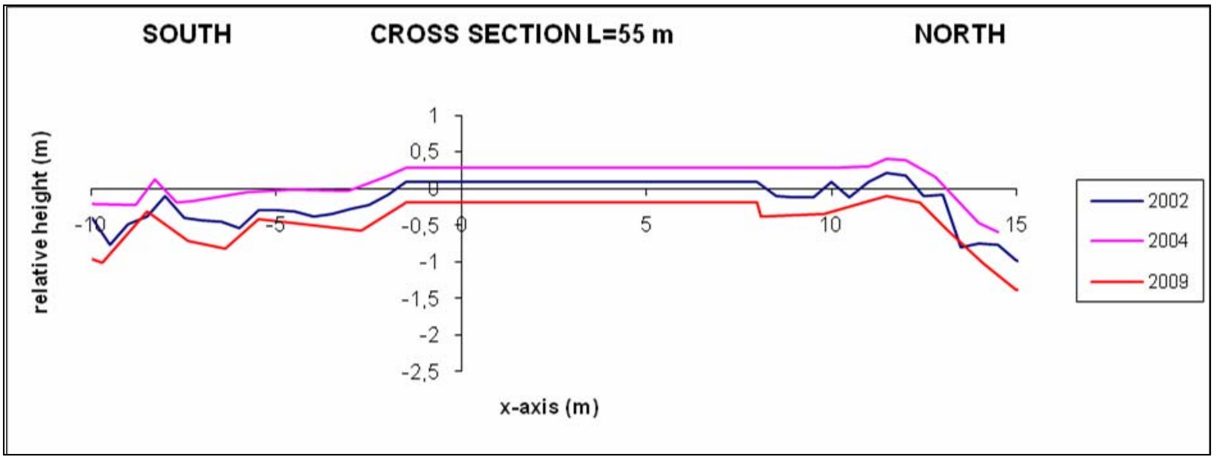


Figure 3-15: Cross-section at L=55 m

Figure 3-16: South side (left) and north side of the groyne (right) at L=55 m

3.5 Overall conclusions

From the cross-sectional comparison it can be said that the overall profile of the groyne has not changed much over the years. And it can be stated that the groyne is relatively stable. Some individual stones have moved down the slope though but small movement is allowed in stone-dumped structures. On the northern side stones appeared to have moved slightly down in the cross-sections and visual observations even showed some individual stones that moved out of the revetment. This leads to individual loose stones at a small distance of the toe of the structure.
The fact that the stones on the northern side are smaller than the stones on the southern side and that storm conditions could just as well originate from the north as from the south might hint that the design of the southern side is probably over-dimensioned, (since both the sides look intact).

3.6 Recommendations

Continuation of the monitoring of the entire groyne is necessary. Although the profile of the groyne has not changed much over the last seven years, a severe storm might still influence big profile changes. By continuous monitoring damage can probably still be repaired in time before the entire groyne fails.

It is recommended to firstly investigate the functioning of this groyne. The beaches in the direct area of the groyne are not particularly attractive or well-maintained. From figure 3-17 it seems that littoral sediment transport from the north to the south causes accretion on the northern side and erosion on the southern side of the groyne. In figure 3-18 the narrow beaches on the south of the groyne are displayed. With a hotel located directly on the southern side of the groyne this might lead to problems. Perhaps the seaward stretch of the harbor might block sediment transport sufficiently from all the beaches north of the harbor.
Finding out the purpose and true functioning of the groyne together with the total littoral sediment transport will help decide whether the groyne should be maintained, improved or even removed.

Figure 3-17: Overview of the T-groyne

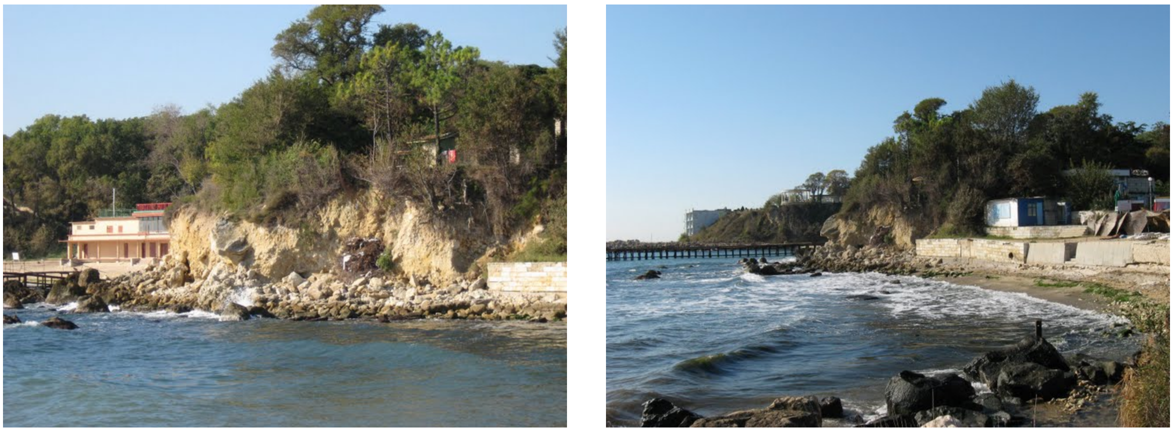


Figure 3-18: Erosion on the south side of the groyne

4. Morphology

The coastal area around Varna is important because it is a rapidly developing touristic area. Especially the white and golden beaches in front of the hotels are very valuable. The competition among the hotels on the coastline around Varna is considerably high the last few years. There is a rapid development of hotels with high quality services, which mainly attract tourists during the summer months by the beautiful seashore. Numerous tourists are spending their holiday on the sandy beaches. Their preservation is therefore of great importance.
However, during the last decades the coastline around Varna has shown some signs of deterioration. The coastline is retreating (figure 4-1) and coastal protection works are damaged. The sea is coming closer to the hotels and the width of the beach is decreasing. The hotel managers in St. Constantine I Elena, a small town consisting of hotels, to the north of Varna, are concerned about the future of their beach and consequently of the hotels itself, since it seems that they are losing their greatest advantage.
To give qualified information about the evolution of the coastline, a thorough investigation of the beach dynamics as well as the coastal morphology of the area is necessary. By executing hydraulic field measurements and analyzing these data it is possible to give a qualified estimation of the evolution of the coastal zone and if necessary some possible solutions.
In this chapter first the equipment needed for the measurements is treated in section 4.1, subsequently two beaches are investigated: Sirius beach and Azalia beach. Sirius beach is treated in section 4.2 followed by Azalia Beach in section 4.3.
By measuring the coast line, bathymetry and cross shore profiles of the beach and compare the results with data from previous years a conclusion can be drawn about the evolution of the coastal area.

Figure 4-1: Sirius beach: retreating coastline

Figure 4-2: Azalia beach: value of the sandy beaches

4.1 Measurements

In order to gain insight into the morphological behavior of Sirius and Azalia beach, it is necessary to measure it frequently. The frequency has to be high enough to record not only the long term morphological tendencies, but also the morphological variations during a yearly cycle (formation of winter and summer profiles).
The beach in front of Azalia though has not been adequately measured in the past. The only measurement available is a GPS walk along the waterline of October 2008 and some beach profiles in the southern part of the section. For the time being, it is therefore not easy to draw some safe conclusions based only in the past measurements.

There are several ways to measure the beach depending on the availability of equipment, their necessity and of course money. The measurements done for this course were chosen according to the availability of equipment and the necessity. There are three types of records created; these will be discussed in the following sections. Firstly a walk with a GPS device along the waterline, secondly measuring beach cross profile with the use of simple surveying instruments (a theodolite and a measuring rod) and at last measuring the bathymetry of the area with an echo sounder device connected with a GPS. Also some sand samples were collected; these are treated in Chapter 5 on Sand Sampling.

4.1.1 Waterline with GPS

The position of the shoreline has been measured by walking along the waterline with a Garmin GPS recorder. The data can be compared to data of previous years and also with aerial photographs of Google Earth. In combination with the beach profiles, it provides an insight into the long-term local coastal evolution.

Figure 4-3:Measurement with GPS

Figure 4-4 shows a graph with the zero GPS-point observations, the average GPS-point is 0582483 E, 4787694 N. This average zero GPS-point is obtained by taking the average of the GPS points during a time period set by the GPS recorder.
This process was performed in order to determine also the accuracy of the GPS device and consequently, to find out how safe it is to draw any conclusions from the measurements. The accuracy was found to be in the order of 5-10m as can be observed directly from the following graph.

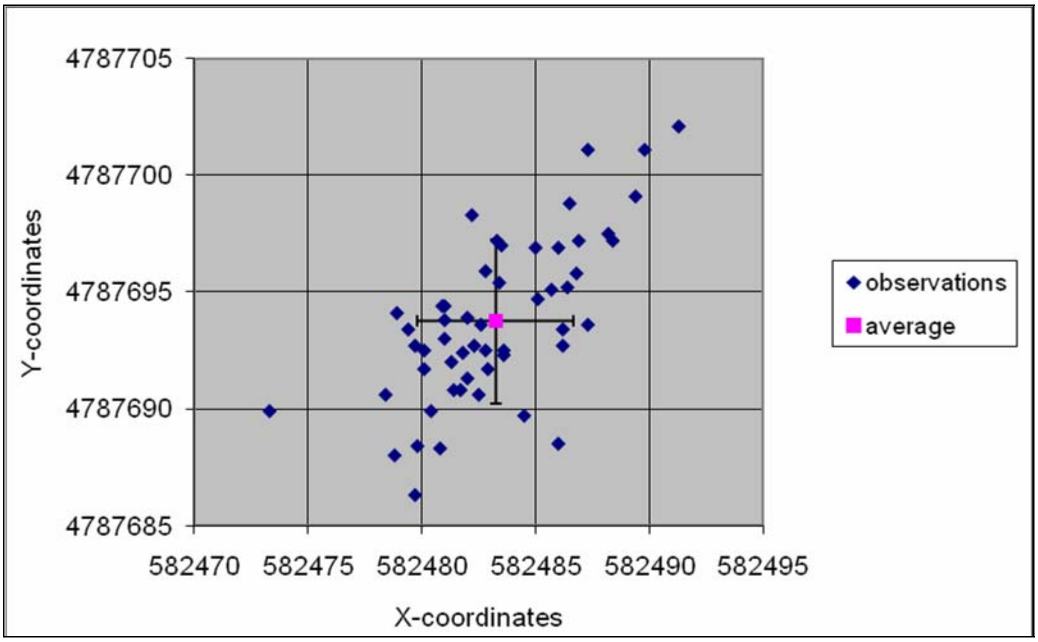


Figure 4-4: Average zero GPS-point- Azalia beach

4.1.2 Beach profiles with Theodolite

A beach can be schematized by making profiles of the cross-section of the beach. By measuring these cross profiles of the beach on a yearly basis it is possible to compare the profiles and give qualified information on the dynamic evolution of the beach in time. It is possible to identify if a beach is stable, eroding of accreting and to give an estimation of the trend line of the change of the beach.

Reference point and base line

To compare beach profiles of several years it is necessary to have some requirements that have to be equal during every measurement. In the case of measuring beach profile the requirements consist of a standard reference height and a base line. For defining the reference height, take a point in the area which certainly won't settle down in future. The baseline has to be almost parallel to the coastline. The different cross profile will be measured perpendicular to this baseline. For defining the baseline, take two reference points on each side of the intended baseline. Be sure that the reference points will be there next time you will execute the field measurements.

Equipment and execution of the measurements

To measure cross shore beach profiles simple tools can be used. The equipment needed is an Angle mirror (figure 4-6a), a Theodolite (figure 4-6b), jalons, a tape measure and a scale (figure 4-6c).

Figure 4-6: Measurement equipment

The angle mirror is necessary to define the base line. With the angle mirror it is possible, while standing on the baseline, to see both reference points on each side of the baseline at the same time. To find the baseline search for both reference points in the angle mirror. If they are both visible a point on the baseline is found. With jalons the baseline can be marked at the beach.
After defining the baseline a certain starting point should be chosen. From there on a point at the baseline is marked every 25 meter.
The theodolite can estimate the height referring to a reference level with accuracy up to cm. After defining a reference height the height can be estimated at every single point. Determining the height at any interesting point can be done by placing the scale upright and looking through the theodolite to the scale and reading out the number. Comparing this with the reference height gives the height at that point.

4.1.3 Bathymetry using an echo sounder and GPS

An echo sounder measures the distance to the bottom (water depth) at a certain point by using sound pulses directed vertically from the surface to the bottom by means of sound waves.
With the aid of merely a small rubber boat, the bathymetry can already be determined.

Figure 4-7: The measuring equipment. Left: rubber boat. Right: Fishfinder echosounder and Garmin GPS device

On the back of this boat, a single beam Fishfinder echo sounder has been mounted. This echo sounder is connected to the control panel and the Garmin GPS device which can be seen in figure 4-7. The GPS device will determine the X and Y coordinates of the boat and the echo sounder will measure the depth in the Z direction. Importing these data in the computer software Surfer 9 makes it possible to plot the bathymetry of an area.

4.2 Sirius Beach

4.2.1 Location Sirius Beach

Sirius beach is a sandy beach and located on the beach of Saint Constantine I Elena, a few kilometers to the north of Varna and directly south to the Azalia Beach (figure 4-8). Hotel Sirius is located on the northern side of the beach. The hotel makes use of a part of the beach, lying directly in front of the hotel itself.

Figure 4-8: Location of Sirius and Azalia beach

There are major morphological variations in front of hotel Sirius (figure 4-9). The past few years a landward shift of the coastline was noted, resulting in a decrease of the width of the beach. The coastline is already so close to hotel Sirius, that during a severe storm the hotel could be damaged. In 2003 there has been a nourishment to increase the width of the beach, but unfortunately all the sand of the nourishment is already gone. This was proved by previous beach measurements.

Because of the retreating coastline it is necessary to model the behavior of the coast. By executing new measurements and comparing them with previous measurements it is possible to describe a trend line in the behavior of the coast and suggest a possible solution.

Figure 4-9: Sirius beach, retreating waterline and eroding beach

4.2.2 Waterline by GPS

In 2003, 2004, 2007, 2008 and in 2009 the waterline of Sirius beach was measured by GPS device. Figure 4-10 shows all the measured waterlines in the coordinate system.

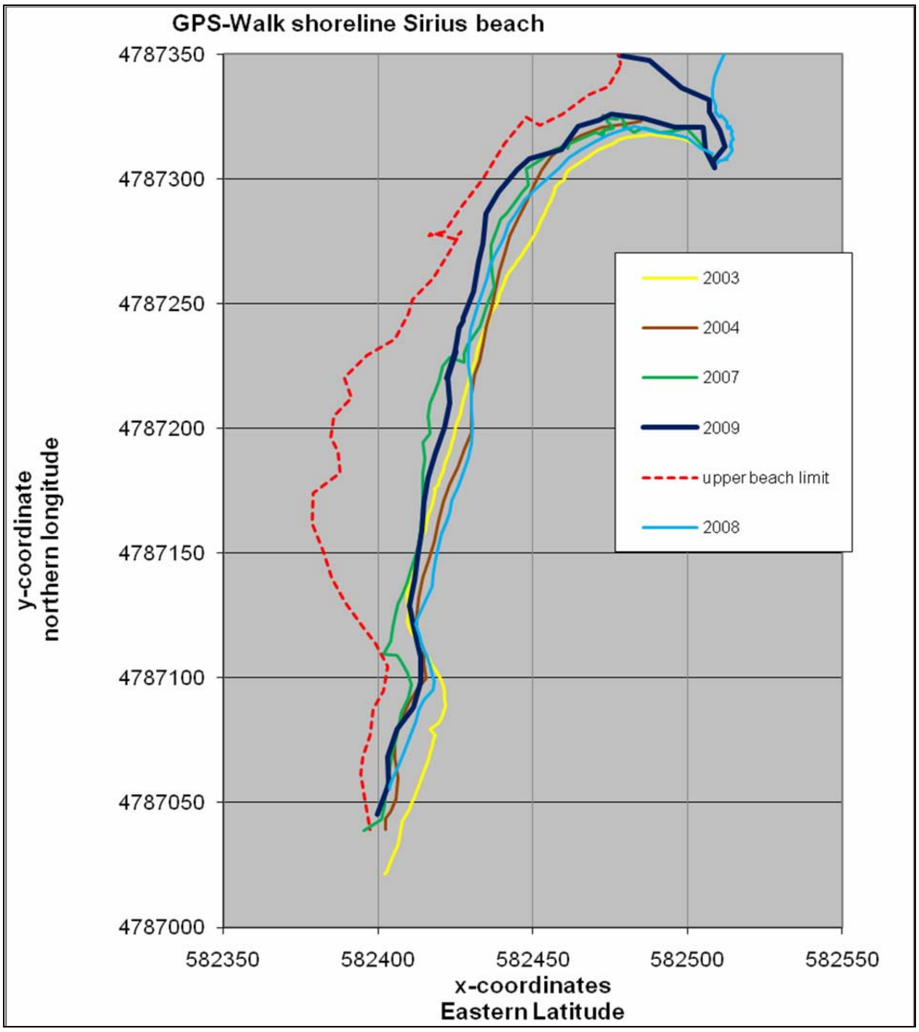


Figure 4-10: Sirius beach

Comparing the waterlines in figure 4-10 clearly shows that that the coastline is very dynamic. A clear erosion trend of the entire coast is not observed, in contrast with the beach just in front of the Sirius Beach hotel where an erosion trend is clearly visible. To compensate for the beach erosion in front of the Sirius beach hotel, beach nourishment was carried out in 2003. This seemed a good solution for the short term, but as can be seen from figure 4-10 it is apparent that the sand of the nourishment already disappeared. Therefore it can be concluded that beach nourishment on this scale is not a good solution for the long term, unless it is repeated regularly.

4.2.3 Beach profiles Sirius Beach

Since 2003 beach-profile measurements are carried out at Sirius Beach. All data of previous years is collected and compared with the new data of 2009. All the measurements were done with the same reference points and baseline. When the reference points are determined and a baseline is drawn between the two reference points, the beginning point can be determined.

Explanation of Reference points, baseline and reference height.

The area of interest is given in figure 4-11 and schematized in figure 4-12. In the figures the reference points (RP1 and RP2) and the dashed baseline can be found.
In the schematization there are also lines perpendicular to the baseline. Those lines are the cross-section profiles that are measured.
To give a clear understanding of the position of reference points Rp1 and RP2, photos were taken. In RP1 also the reference height can be determined. It is the level on the top of the concrete wall as shown in the photo.

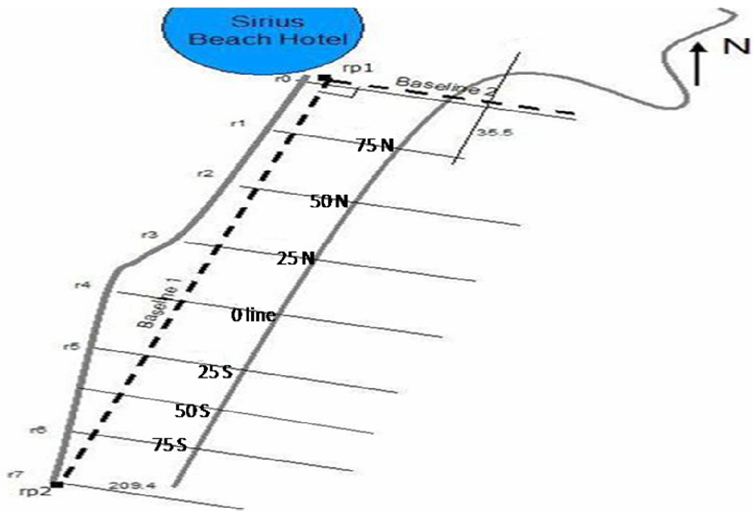


Figure 4-11: Sirius beach overview

Figure 4-12: Schematization of Sirius beach, with references points, baseline and cross-section profiles.

Figure 4-13: references points RP1 and RP2

Starting point

Before measuring is started, the starting points need to be determined. The starting point is rather important, especially if the measurements over the years need to be compared.
The starting point of every year is recorded with a GPS devise. Those points are plotted in Figure 4-14. The yellow dashed line in this figure is *the baseline*. Perpendicular to that baseline, in blue are the *profiles of 2008*. From the legend also *RP1 and RP2* can be found.

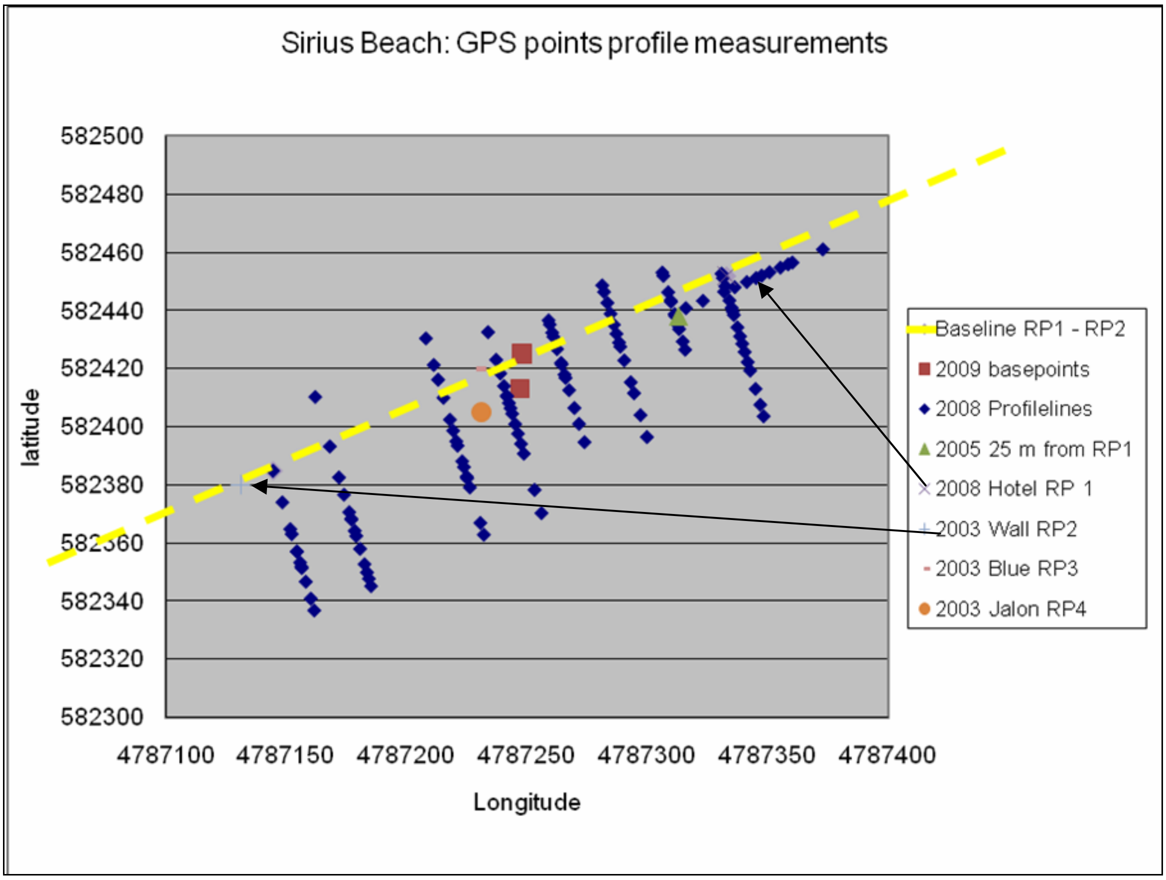


Figure 4-14: GPS data plotted (starting/reference and measuring points)

There are some minor differences between the starting points of each year. This will be briefly explained.
2003 starts from the 'a corner' of the beach house. By 'a corner' is meant that there is no way to find it at this moment. In that year the house had a blue color. Nowadays the house is painted orange; see figures 4-15 and 4-16. This makes it quite difficult to find the same spots before starting up the measurements.

Figure 4-15: Blue Beach house (Sirius beach)

Figure 4-16: Orange Beach house (Sirius beach)

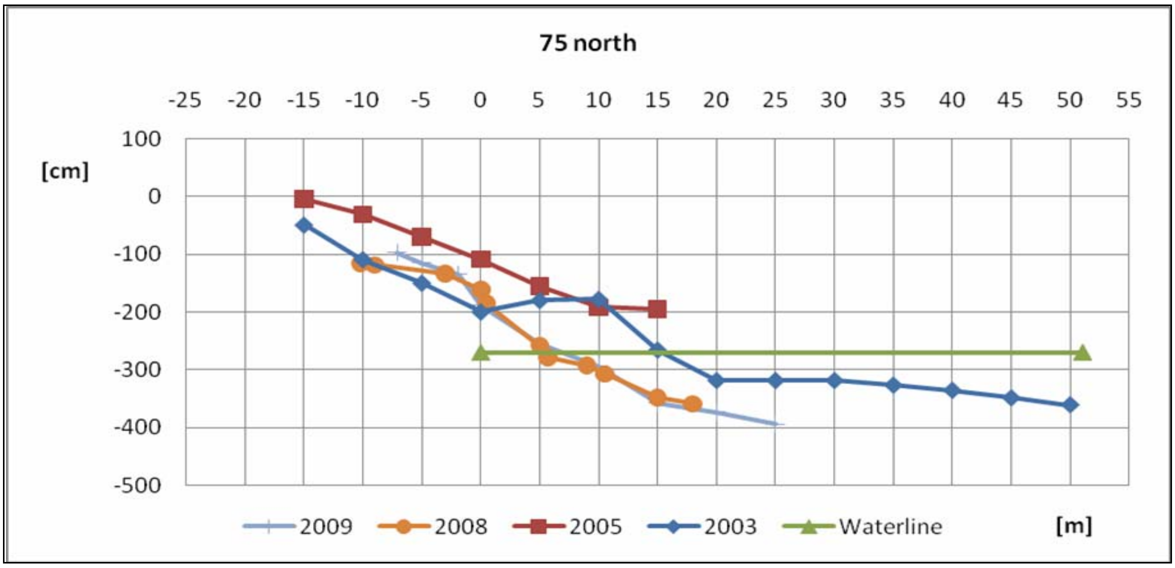
2005 starts 25 m from RP1 and has profiles every 25 meter up to 150 m from RP1. The starting point is the green triangle in figure 4-15.
2008 positioned every measure point with GPS, which can clearly be seen in figure 4-16.
2009 also started from the (orange) Beach house, this year also has GPS points and those are plotted also in Figure 4-16.

To illustrate the differences in starting points they are shown in one photograph, see figure 4-17. It is easy to see that every year another starting point is taken. Between 2003 and 2009 there is a difference of approximately 15 m.

Figure 4-17: Beach house on Sirius beach, different starting points for measuring

Comparison of cross-section profiles

The yearly beach profiles will be compared to give some information about the evolution of the coastline and an estimate for the future. There is data of 7 profiles of 4 different years: 2003, 2005, 2008 and 2009. The data of each of these profiles is put together. The 7 profiles are presented below and show the beach profile from the Sirius Hotel in the north up to 200 meters southward.
Note that because of the changes in starting points the profiles are not really taken at the exact same point but vary in the order of 5 to 10 m. Nevertheless these profiles give a clear understanding about the movements of the coast.
An explanation about the graphs is given at the end of this paragraph. The first graph is called 75 N and gives the profile in front of the Sirius hotel. The names of the graphs refer to figure 4-12 (schematization of Sirius beach)


75 north
-25 -20 -15 -10 -5 0 5 10 15 20 25 30 35 40 45 50 55
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-100
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2009 2008 2005 2003 Waterline
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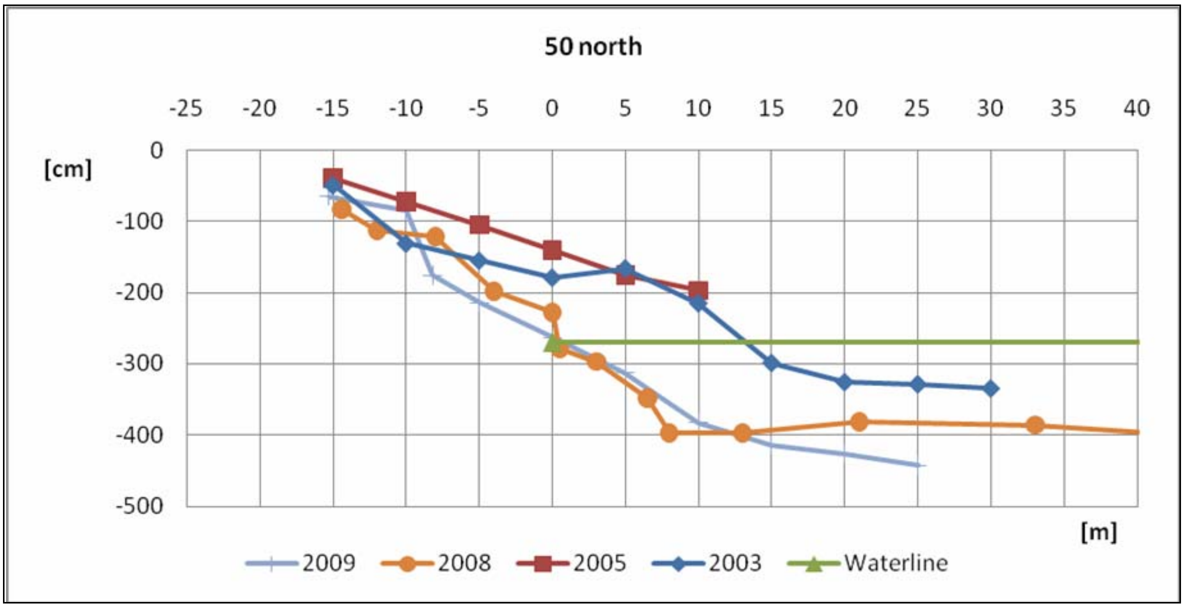
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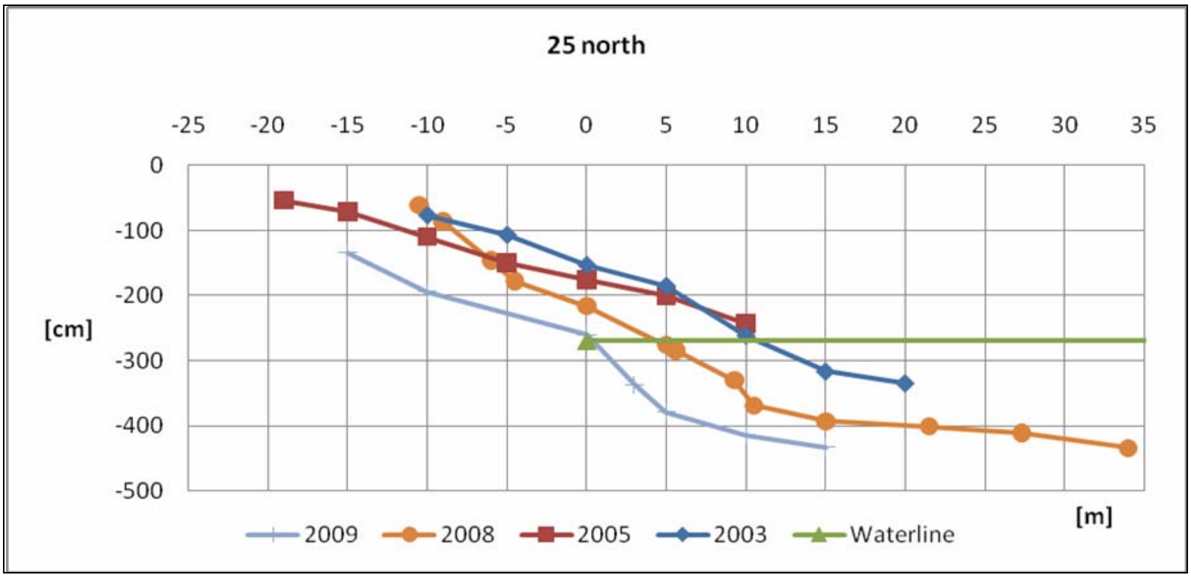
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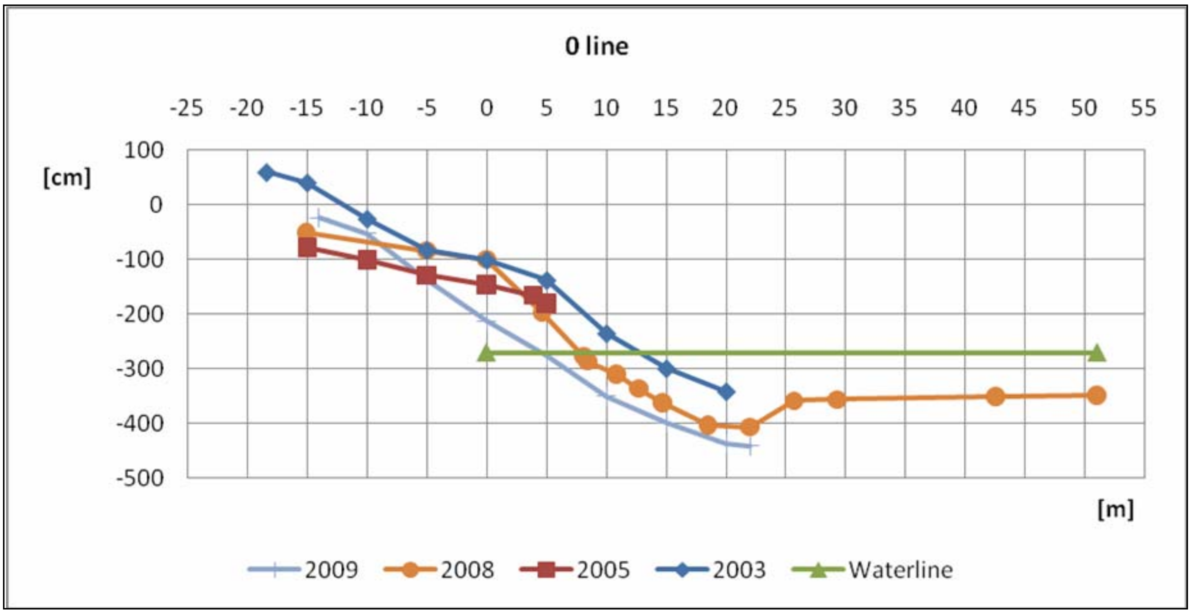
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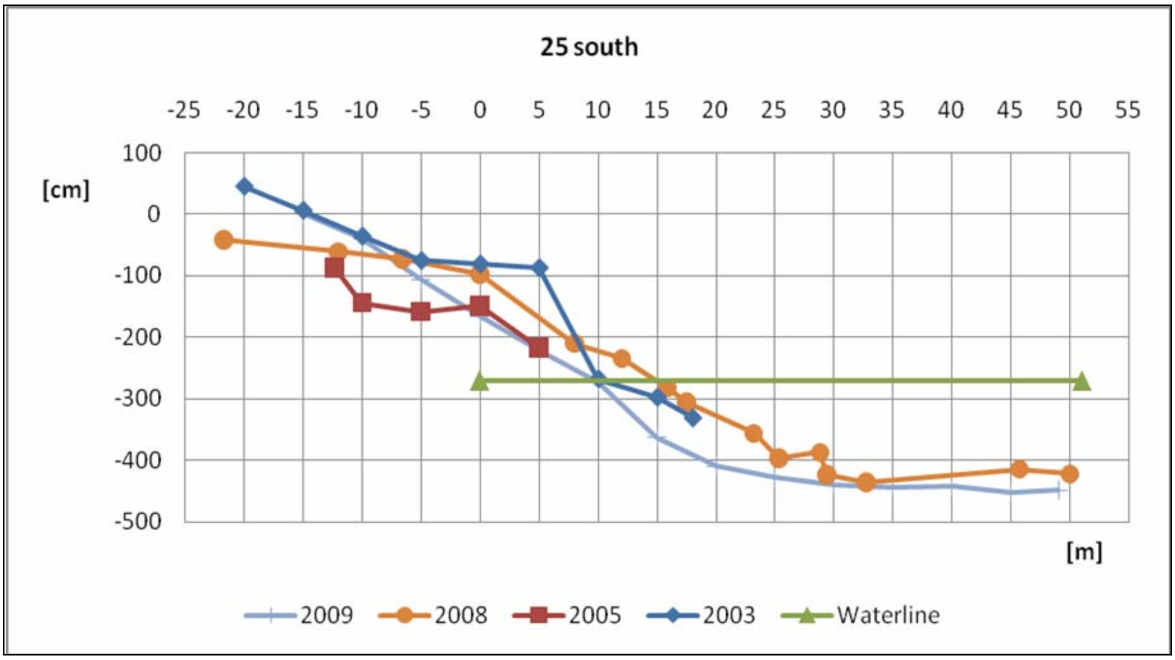
25 south
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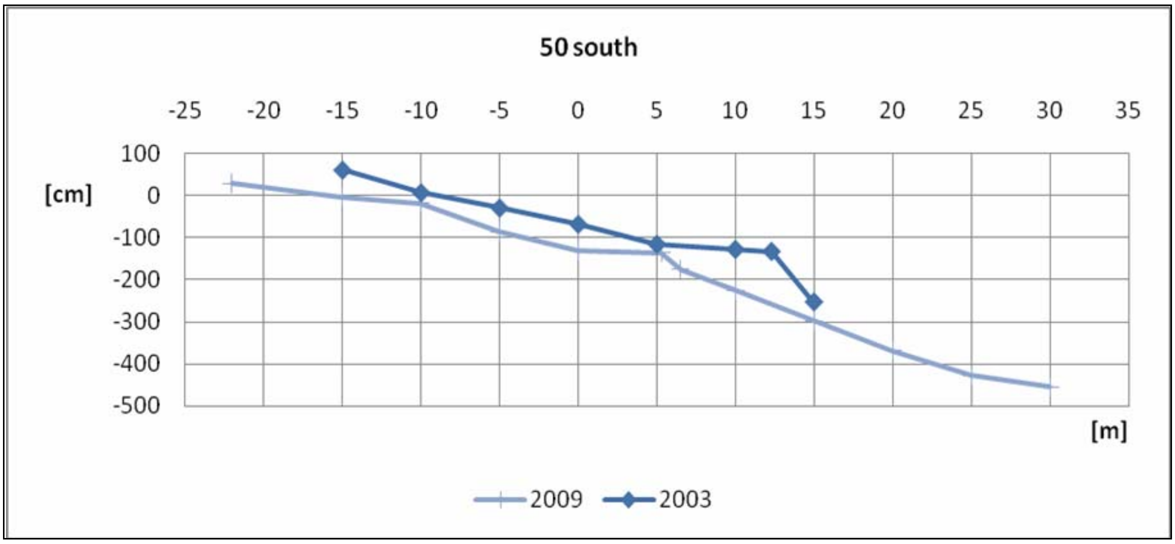
2009
2008
2005
2003
Waterline


50 south
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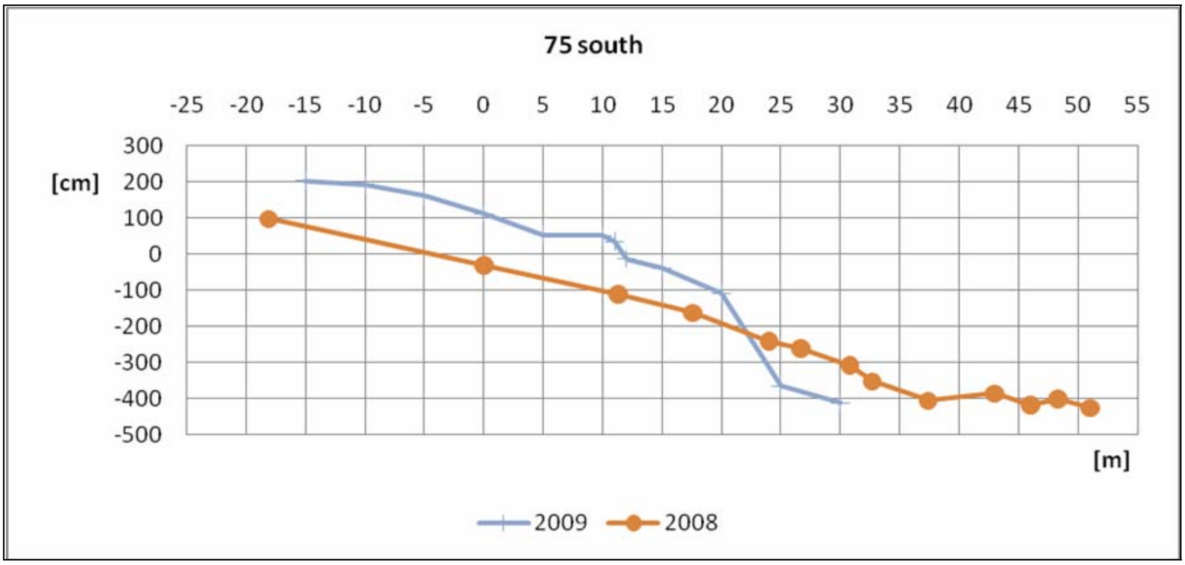
2009
2003



Comments on beach profiles

In the last year quite some data is gathered. There really are big changes in the profiles. The biggest changes can be seen in the northern part of the beach, just behind the groyne in front of the hotel.
Right after the construction of Sirius hotel in 2002 the beach profile started to change a lot. In 2003 there was a nourishment just in front of the hotel to widen the beach. Unfortunately, already a few years after the nourishment all the sand has disappeared. The photograph in figure 4-18 (below) of the beach in front of Sirius hotel clearly shows the erosion and the retreating behavior of the beach. There are major changes between 2003 and 2009, the beach is almost completely gone.
The underwater bed profiles are also lowering, probably due to heavy wave attack, but are not changing as much as the upper part of the profiles. The underwater profile is quite stable because the bed consists of more rocky stones.
In the southern part of the beach the profiles start to change the other way around. The sand that is gone in the northern part is moved to this area. The beach is even growing at the most southward profiles.

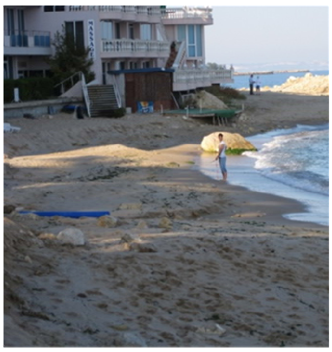


Figure 4-18: profile 75 N, complete beach disappeared, waterline is retreating
Figure 4-19: profile 25 N, complete 'bite' taken from the slope due to a heavy storm
Figure 4-20: profile 75 S, growing of the beach

4.2.4 Conclusion about the profile measurements

The beach is certainly not stable and changes in profiles will go on if nothing is done. Every year more sand will be taken from the area in front of the hotel and will be put in the southern area.
There are two reasons for this, first of all due to heavy storms and rainfall, the beach is attacked many times a year, mainly in the winter period. In the summer period the beach has to restore to the old profile.
The second reason is because of the longshore current. The sand is transported by the longshore current from North to South, due to the main northeast wind direction in winter. When this transport is stopped by a groyne, behind it erosion will take place. The eroded sand will be transported southward and is deposited on the southern part of Sirius beach, which is why the beach is growing in that part.

4.2.5 Solution

The best way to deal with the erosion of the northern part of Sirius beach would probably be to regularly redistribute the sand using small dredging equipment.

4.2.6 Recommendation

It cost the group of 2009 great effort to find out the difference between the data and starting points of 2003 and 2008, mostly because it was difficult to interpret the data in a good way. Therefore it is recommended to keep to one starting point, so that in the coming years the data can easily be compared with the data of 2009.
The coordinates of the 2009 starting point are (E 582425, N 4787250), with an inaccuracy of order 5 m. From that point it is recommended to take measurements every 25 m away from the zero point in order to gain 7 or 8 profiles.

4.3 Azalia Beach

4.3.1 Location Azalia beach

Azalia beach is located on the beach of Saint Constantine i Elena, a few kilometers to the north of Varna main port and directly to the north of Sirius Beach (figures 4-8 and 4-11). Azalia hotel is located on the Azalia beach, in the dynamic coastal zone. The hotel makes use of a part of the beach, lying directly in front of the hotel itself. The beach is sandy and has a length of about 300 m and a width of about 40 m (figure 4-21). Some morphological variations are expected on yearly basis and in the long term. The existence of the hotel itself is also responsible for the morphological evolution of the beach, since it is located on a dynamic coastal zone.

Figure 4-21: Area of interest in Azalia beach

4.3.2 Waterline Azalia beach by GPS

Figure 4-22 shows the GPS-walks along the shore line in 2008 and 2009 of the beach in front of the Azalia hotel. Besides the GPS lines, the line of the upper beach limit is presented. In figure 4-23 the GPS recorded waterline is drawn in a Google Earth photograph of the 3th of May 2007. The impact of a shift of the coastline at the beach width is clearly visible.

Figure 4-22: GPS walk shoreline Azalia beach 2009

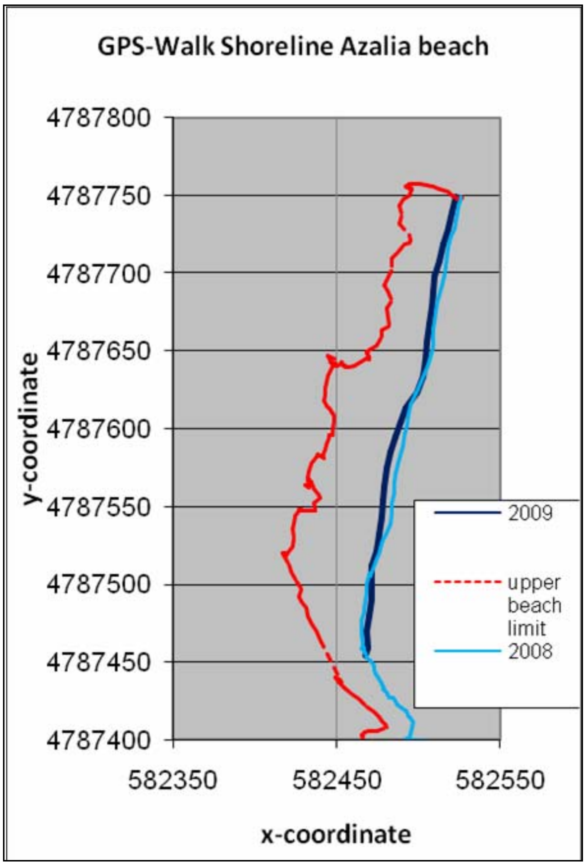


Figure 4-23: Comparison shorelines Azalia beach

As far as it concerns the comparison of the waterlines for the years 2008 and 2009, it is not safe to draw a solid conclusion as a two-year data record cannot be considered sufficient, especially if the accuracy of the measurements and the yearly (weather) specificity is taken into consideration. However, this comparison can provide a hint about the evolution and show the tendency of the beach.

In this case it seems that the beach tends to erode since the waterline of 2009 is in the inland-side compared with the 2008 measurement.

It is important to note that the Google Earth photograph serves only as a background and its scope is mainly to help in identifying the broader area. Thus it cannot be used in this analysis since it is taken during late spring (May) when the summer profile has already started being formed and the beach is retreating.

4.3.3 Beach Cross profiles Azalia Beach

Reference points and base line

As mentioned before the different cross profiles will be measured perpendicular to a baseline. For defining a baseline, take two reference points on each side of the intended baseline. For Azalia beach the following reference points are created:

Reference point A is at the outside swimming pool of the Azalia hotel, at the corner of the deck platform (see picture 4-24). Point A was created on a position with the coordinates of 0582478 E, 4787668 N.

Reference point B is at the concrete semi-circular platform of the outside bar of the Sunny Day hotel (see picture 4-25). This reference point was created with the coordinates of 0582588 E, 4788009 N

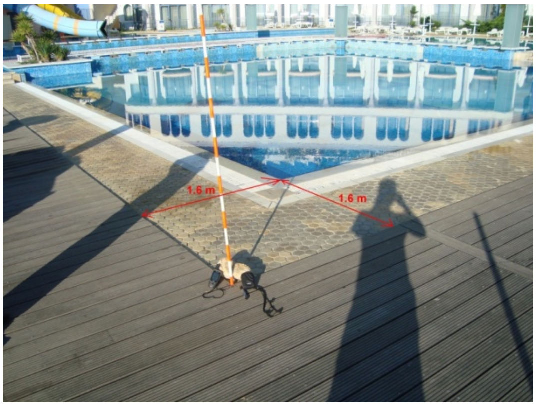


Figure 4-24: Reference point A - Azalia beach

Figure 4-25: Reference point B - Azalia beach

A zero gridline was created on the baseline. The reference point for this beach lies on the bottom of a concrete pile of the southern wing of the Azalia hotel (see picture 4-26).

Figure 4-26: Zero reference point - Azalia beach

Figure 4-27 gives an overview of the locations of the 3 measured cross sections on the baseline. The three cross sections are taken directly at the zero line, at -100m and at +50m. This Figure is only an indication of the positions.

Figure 4-27: Location of reference points (Azalia beach)

Reference height Azalia beach

Since there was no reference height for the Azalia beach from previous years, the first step was to define a certain reference height. This height corresponds with the ground level of the balcony which is on the top of the southern arcaded wing of Azalia hotel. For better understanding this point is marked (in red) in the next Google Earth picture (figure 4-28a) and additionally, it is also shown in the following photographs.

Note: The figure 4-28c is a photograph of the relative anti-symmetric point in the northern wing of the hotel.

Figure 4-28a: Reference point of Azalia beach

Figure 4-28b: Reference point of Azalia beach

Figure 4-28c: Reference point of Azalia beach

In order to be able to correlate and compare (if necessary) the measurements of Sirius and Azalia beach, the height difference of the reference points had to be measured. The Sirius reference height (which is shown in figure 4-29) was found to be -1.13m lower than the relative reference height of Azalia. The height of the northern wing balcony was also measured and found to be +4cm higher from the reference point.

Figure 4-29: Reference point of Sirius beach

Cross profiles Azalia beach

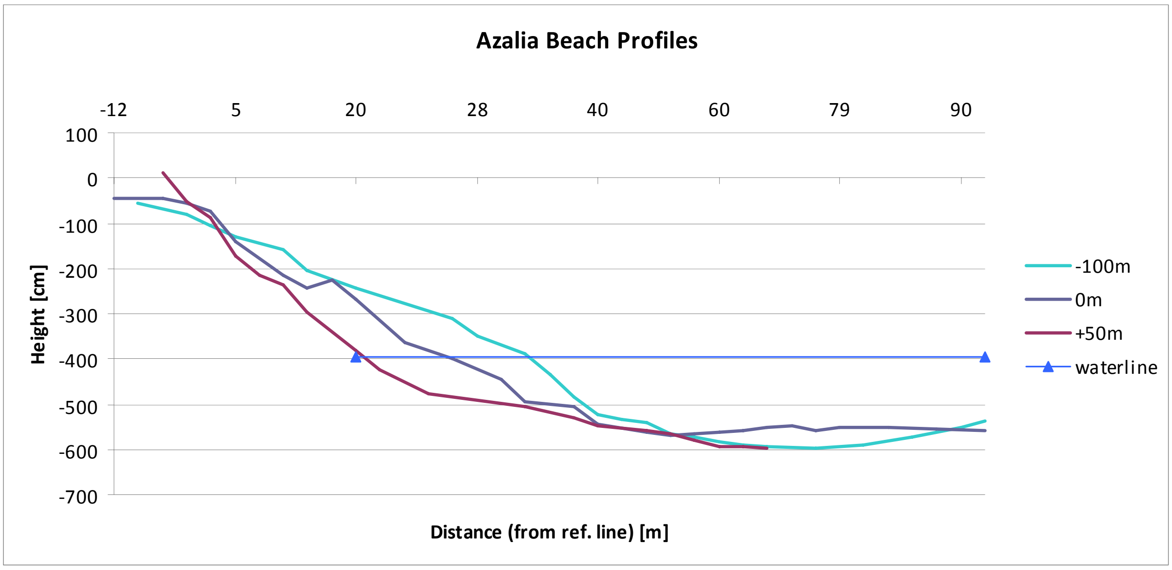


Figure 4-30: Beach profiles (Azalia beach)

The above presented graph shows the three beach profiles measured in front of the hotel Azalia. The first observation is that all the profiles have identical shape which means that this beach section can be considered uniform (in terms of cross-section shape). This also implies that there are no local (physical) particularities which affect directly the formation of the profile and since the wave action is dominant, the evolution is more predictable and thus, conclusions can be drawn more safely.

The main aspect which can be observed from the above presented figure is the rather steep slope of the beach zone alongside the waterline. Contrary, the underwater bottom slope seems very mild, forming a broad shoaling just in front of the waterline. This description corresponds to the winter profile which is formed due to the increase of the wave action during autumn and winter.

4.3.4 Bathymetry of Azalia Beach (with Echo soundings)

The original idea in this type of measurements is to move in lines perpendicular to the coast in order to create a net of measurements (with X, Y coordinates and Z the corresponding water depth) and finally simulate graphically the seabed using the appropriate software.

The following figure presents the route followed during the measurements (with echo-sounder) of the bathymetry of the shore section in front of hotel Azalia which consists of four parallel lines distanced (roughly) every 100m (green lines). Additionally, a set of extra measurements were performed locally, near the waterline, where a shoal was formed (marked in purple in the following figures).

For the identification of this shoaling, the contour line of -1.1 m was mapped (walking on a route with depth -1.1 m and marking with GPS the coordinates).

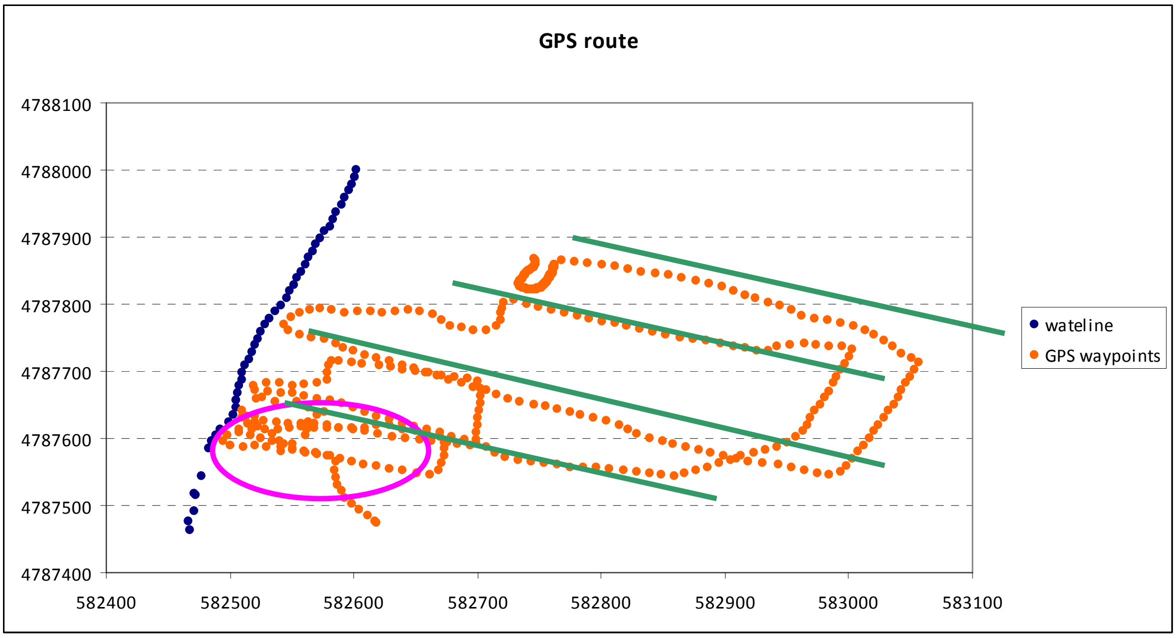


Figure 4-31: Measurement route of Azalia beach

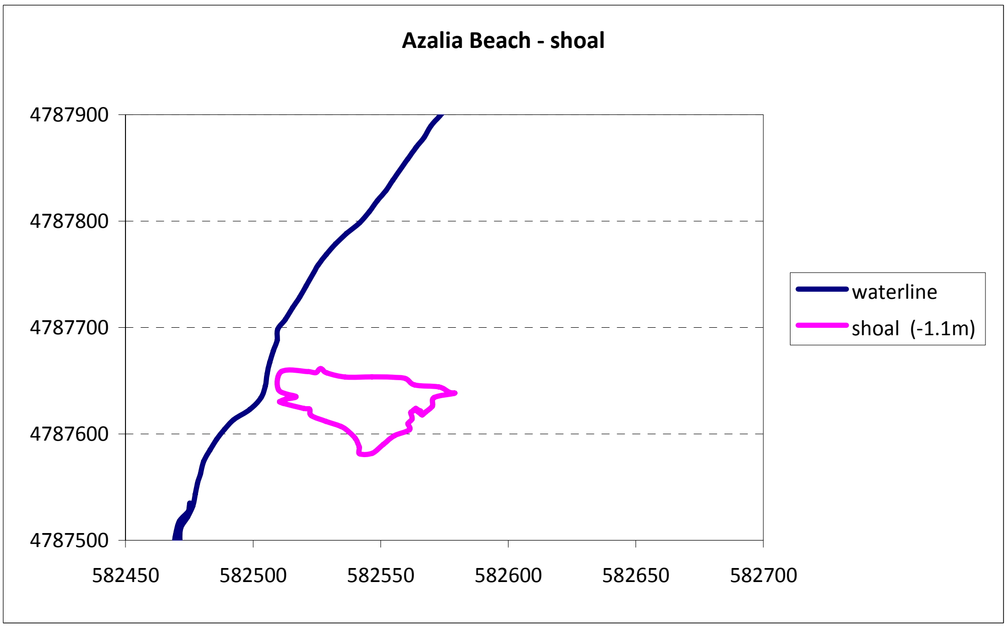


Figure 4-32: Shoaling in front of Azalia

The collected data was processed and inserted to Surfer32 (Golden Software) in order to obtain a simulation of seabed of this section. The main process of Surfer32 is to interpolate linearly the water depth measurements, form the contour lines and finally create a 2D image and a 3D representation of the shore section.

In the next figures, the output of the Surfer32 is presented. The position of the shoal is also noted.

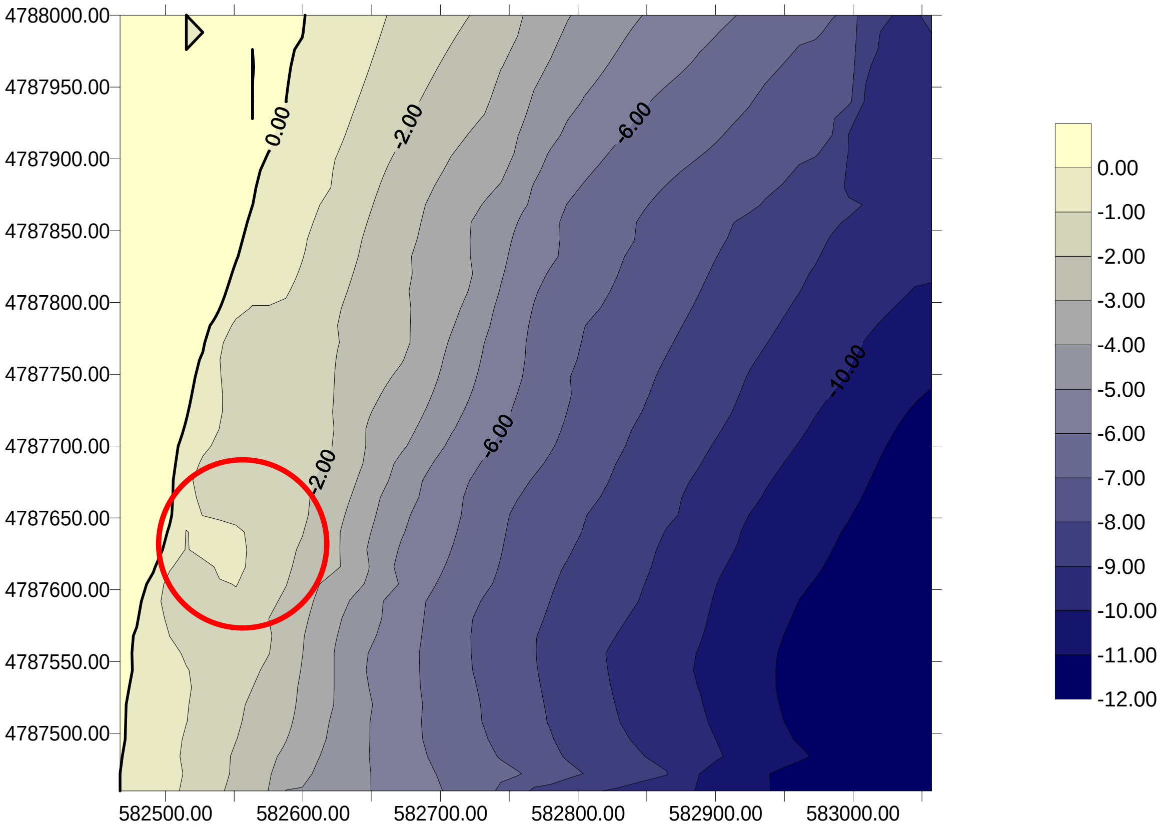


Figure 4-33: Map of Azalia seabed (Surfer32 output image)

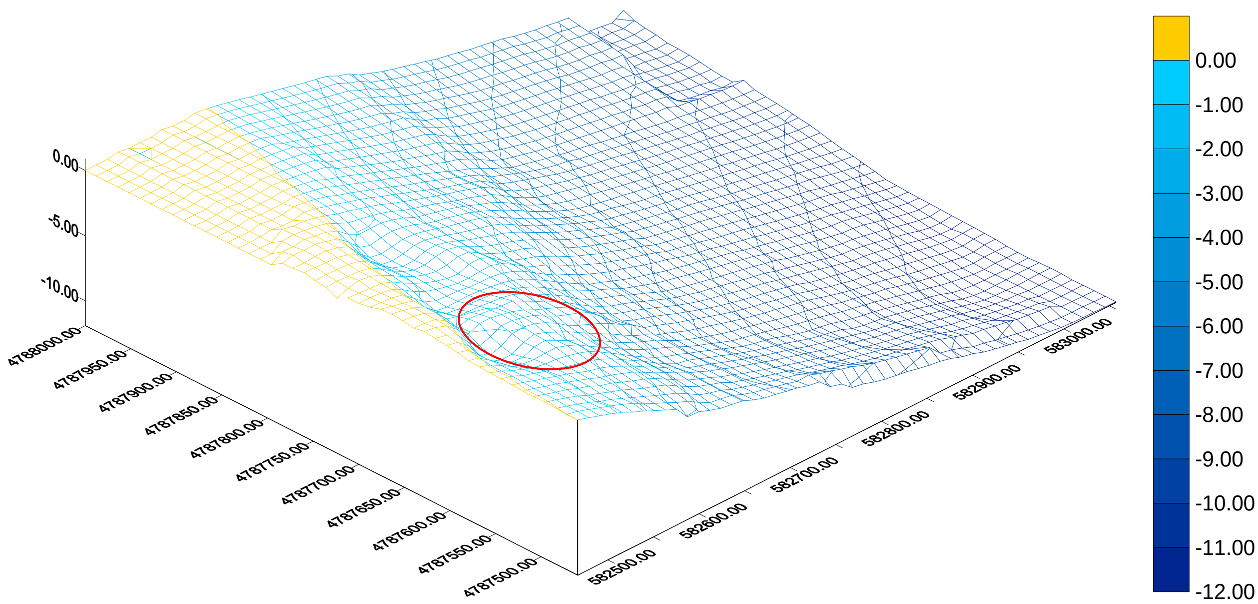


Figure 4-34: 3D simulation of Azalia seabed (Surfer32 output image)

4.3.5 Conclusion Azalia Beach

The first thing which can be observed directly from the beach profiles (figure 4-30) is the accordance of the actual measurements with the theoretical seasonal profiles. The typical profiles predicted by the theoretical approach are presented in the following figure.

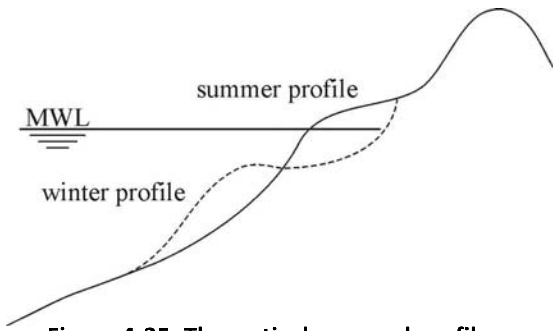


Figure 4-35: Theoretical seasonal profiles

Even though the measurements were performed during autumn, it is apparent that the winter profile is already created: a part of the sand has moved to the seaside, just front of the beach, forming an extended shoaling and finally creating a steeper beach profile along the waterline zone. It is expected that the presented profiles will continue to evolve in this way until early spring.

This observation can lead to the conclusion that the beach will follow the seasonal evolution and consequently, it will start to retreat after spring and form again the summer, mild profile with the extended beach zone.

Generally, it can be noted that the wave action is dominant in this area, affecting directly the whole beach section.

4.3.6 Recommendation Azalia Beach

Starting with the measurements, as it is mentioned previously, they can be useful for a first-level analysis but they cannot be considered sufficient for a thorough scientific approach. More beach profiles should be measured and a more dense net of GPS tracks should be considered in order to create an adequate database which will be able to provide solid conclusions and an accurate visualization of the beach and the seabed.

Solution

As far as it concerns the problem of beach narrowing after a winter with heavy storms, the solution is to perform minor dredging activities and recover the removed sand back to the beach. This procedure is recommended only if after the winter time it is observed a considerable erosion of the beach. In this case, before the summer season, a beach nourishment operation can be performed. This procedure is relatively simple and not so time-consuming and can be accomplished using simple equipment like sand pumps shown in the following pictures. An advantage of this solution is that the whole operation doesn't require a thorough coastal analysis by experts or the cooperation with a major dredging company. A local, minor dredger is sufficient to precede this operation.

Figure 4-36: Dredging pumps

5. Sand samples of Sirius and Azalia beach

In order to gain some insight into the morphological behavior of the beaches in front of the Sirius and Azalia hotels, an investigation of the type of existing sand is necessary, as well as a comparison with the sand found in the same locations the past years. The procedure for determining the type of existing sand is to first get some samples from the area of interest which is treated in section 5.1.

Subsequently carrying out gradation measurements in the laboratory is required, this is discussed in section 5.2. The results of the laboratory tests can in the end be compared to the corresponding results of previous years, this is the subject of sections 5.3 and 5.4

5.1 Sampling

A sand sample has to be representative of the area of interest in order to be good. This means that the samples have to be well distributed in the examined area, while their number and size must be small enough to allow an easy and unburdened analysis, and large enough to allow the derivation of safe conclusions.

The choice of the sample locations this year has been made in correspondence with the locations of the previous years, so that a comparison of the results would be possible. In total, 24 different samples have been collected from 9 different beach profiles. From the majority of profiles three different samples have been taken (A, B, C – see figure 5-1). Samples of type A are located in the water, at a distance of 3 to 4 meters from the waterline, samples of type B are on the waterline, while samples of type C are located about 10 m landward. Apart from this there were also 3 samples of deeper soil taken with a piston sampler, at 10-20 meters from the waterline (into the sea) at Sirius beach.

Figure 5-1: Sample locations

The average weight of the samples is about 250 gr. The only criterion used for the choice of this value, was the easy transfer of the samples from Bulgaria to the Netherlands, where they would later be analyzed.

5.2 Sieving

The most important piece of information about the type of sand on Sirius and Azalia beach is its gradation and its median grain size (D_{50}). The means to get this information via the collected sand samples is a sieve analysis in the laboratory.

This analysis follows some standard steps. First of all the net weight of the sieves has to be measured, in order to be subtracted later from the total weight of the graded sand. Secondly the sieves are placed on a vibration machine with an increasing mesh size from bottom to top (see figure 5-2), and on the top of them the sand is placed. After a few minutes of vibration the sand particles are distributed over the different sieves, with the coarsest material remaining in the top sieves and the finest material moving downwards to the bottom sieve. By measuring the amount of sand passing from each sieve, useful information is gained about the gradation of the tested sample. It is important to be noted that the sand samples have to be dry in order the test to be successful.

Figure 5-2: Vibration machine for sieving

The sieving took place in the Geo-Lab of the Faculty of Civil Engineering and Geosciences of TU Delft. In total 9 sieves were used with mesh sizes from 0.063 to 6.3mm. Their net weights can be seen in the following table:

Mesh size (mm)	net weight (gr)
0	346
0,063	388
0,15	364
0,212	432
0,3	389
0,6	460
1,18	518
2	549
3,35	597
6,3	521

Table 5-1: Net weight of sieves

Prior to the sieving process some of the samples had to be dried in the oven. They were mostly samples taken from the wet spots (samples A and B). In order to get totally dry they were put in an oven at 105°C for 24 hours.

5.3 Results

The results taken directly from the sieving process are the total weight of each sieve together with the amount of sand that is on it. In order to extract a distribution curve for each sample, these results had to be slightly adjusted. First the net weight of the sieve had to be subtracted, and then the weights had to be added to get a cumulative distribution. With this a distribution curve could be made that depicts the percentage of sand that passes from each sieve. The passing sand percentages that have been found for each sample can be seen in the following table.
From the distribution curves the median grain size (D_{50}) and other characteristics can be determined.

Mesh size (mm)	1a	1b	1c	2b	2c	3a	4c	5a	5b	5c	6a	6b	6c	7a	7b	7c	8a	8b	8c	9a	9b	9c
0.063	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
0.15	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.55%	0.33%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
0.212	0.00%	0.00%	0.00%	0.00%	0.35%	0.00%	0.00%	0.40%	0.60%	0.36%	0.00%	0.59%	0.56%	0.00%	1.10%	0.66%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
0.3	0.00%	0.00%	0.00%	0.00%	2.10%	0.00%	0.00%	0.80%	4.50%	2.51%	0.00%	7.06%	1.13%	0.00%	4.95%	2.32%	0.00%	0.47%	1.04%	0.72%	0.00%	1.49%
0.6	0.00%	0.00%	0.00%	6.47%	42.31%	3.16%	8.50%	22.31%	33.93%	48.03%	4.71%	82.94%	35.59%	1.05%	47.25%	31.46%	17.61%	20.00%	56.25%	12.90%	1.49%	34.82%
1.18	28.31%	7.47%	39.44%	94.03%	97.90%	22.11%	71.66%	62.55%	96.70%	97.85%	49.21%	98.24%	95.48%	32.98%	96.70%	93.38%	50.83%	96.74%	91.67%	49.46%	65.35%	98.21%
2	59.93%	68.05%	96.48%	97.01%	100.00%	70.53%	95.14%	77.69%	99.70%	99.64%	74.35%	99.41%	97.18%	59.69%	99.45%	97.68%	59.80%	99.07%	94.27%	70.25%	99.50%	99.11%
3.35	76.84%	100.00%	99.30%	99.00%	100.00%	94.74%	99.19%	85.66%	100.00%	100.00%	86.39%	100.00%	97.74%	74.87%	100.00%	98.68%	67.77%	99.53%	97.40%	81.00%	100.00%	99.40%
6.3	84.56%	100.00%	100.00%	100.00%	100.00%	99.47%	100.00%	92.83%	100.00%	100.00%	94.76%	100.00%	98.31%	82.72%	100.00%	100.00%	79.07%	99.53%	98.96%	90.32%	100.00%	99.70%
>6.3	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%

Table 5-2: Passing sand of each sieve

5.3.1 Comparison of the different profiles (locations 1-9)

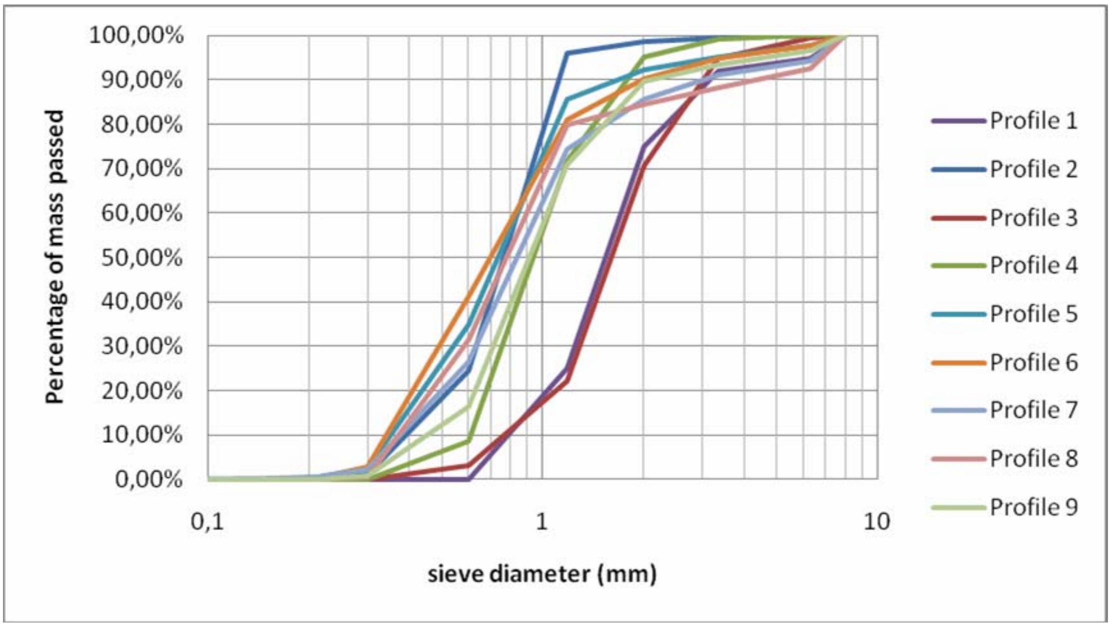


Figure 5-3: Averaged distribution curves per profile

In order to see if there is any variation in the composition of the beach in the longshore direction the average distribution of every profile was calculated. These distributions are compared in figure 5-3, from which can be seen that there is not much variation between them, except for profile 1 and 3, both on Sirius beach. These deviations are probably caused by the fact that these samples contained some gravel, while the others didn't. Apart from this it can be concluded that the sand has practically the same grading over the total investigated area.

5.3.2 Comparison of the different points with regard to distance from the waterline (points A, B, C)

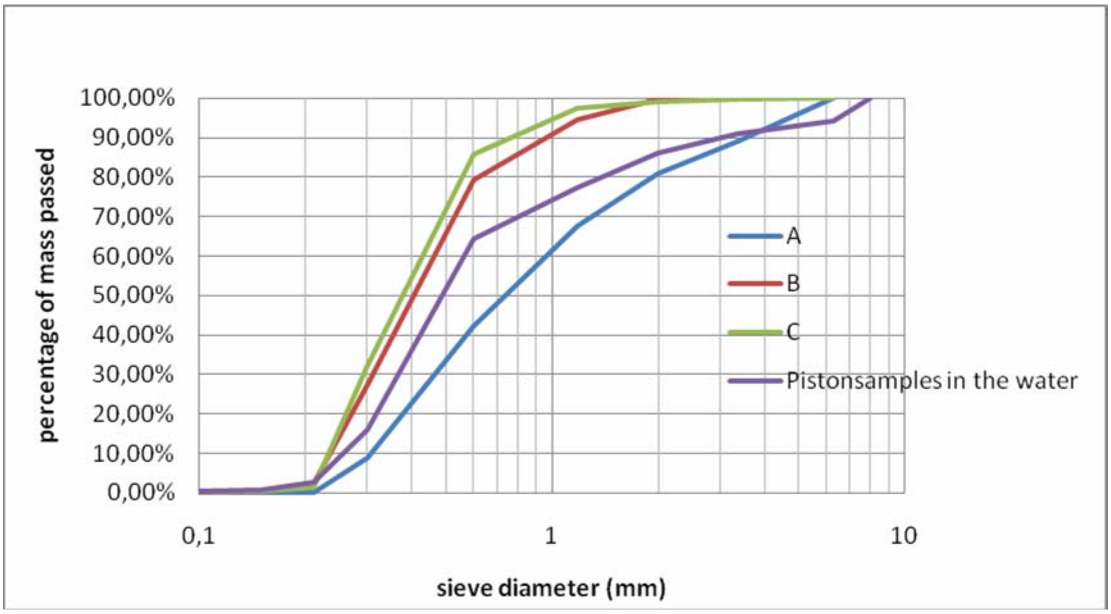


Figure 5-4: Averaged distribution curves with regard to distance from waterline

	A	B	C	piston
d10	0.31	0.23	0.23	0.26
d50	0.75	0.4	0.38	0.49
d60	0.98	0.46	0.43	0.57
d60/d10	3.16	2.00	1.87	2.19

Table 5-3: Values for grading of sand in mm.

To see if there is any difference in the grading at different distances from the waterline, all the A, B and C samples are grouped and averaged, as well as the piston samples from deeper into the sea. These distributions can be seen in figure 5-4. Also the values for d10, d50 and d60 were determined; these are shown in table 5-3. These results show that the average grain size is largest for the samples at points A, just below the waterline. This can be explained by the fact that these points are most exposed to the wave action, which washes away the finer particles. These points also have the largest value for d60/d10, and thus the widest grading.

5.3.3 Comparison with previous years

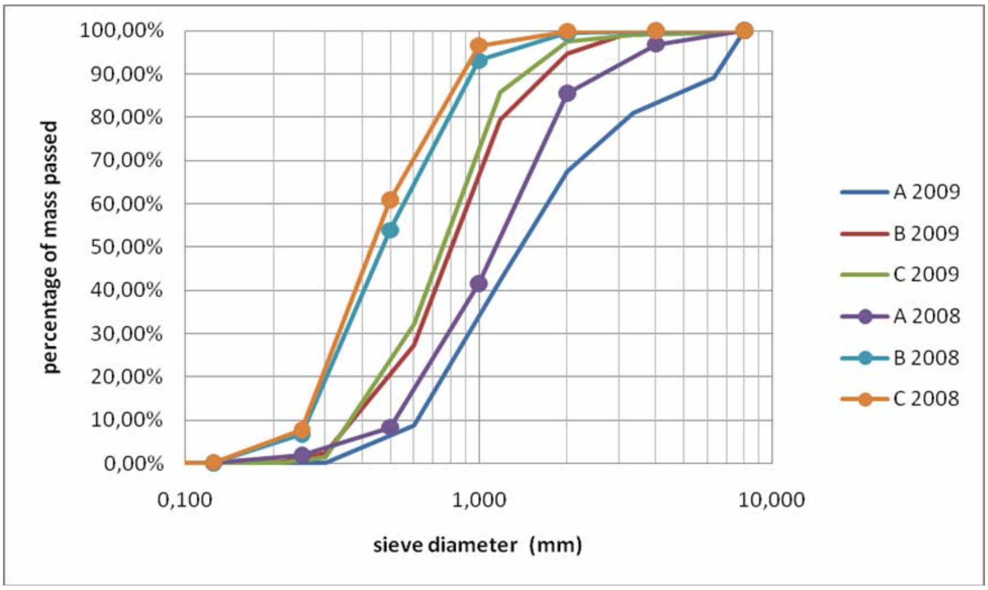


Figure 5-5: Comparison of distribution curves 2009 and 2008

		A	B	C
d10	2009	0.31	0.23	0.23
	2008	0.26	0.13	0.13
d50	2009	0.75	0.4	0.38
	2008	0.60	0.24	0.22
d60	2009	0.98	0.46	0.43
	2008	0.71	0.29	0.25
d60/d10	2009	3.16	2.00	1.87
	2008	2.71	2.16	1.90

Table 5-4: Comparison of sand grading 2009 and 2008

In order to see if the beach changes in time the results of paragraph 3.2 are compared with results of the samples of 2008. This is shown in figure 5-5 and table 5-4. It can be seen that the average grain sizes of this year are much higher over the whole profile than those of last year. Since there were no human interventions or large changes in the local conditions it seems unlikely that the composition of the whole beach changed that much in only one year. Therefore it is assumed that this change is caused by a difference in the sieving method or calculation method.

5.4 Conclusions

What can be seen from the results is that the beaches have a pretty narrow grading, with most of the grains having a diameter between 0.6 and 2 mm. There is not much variation in longshore direction, so both beaches have roughly the same type of sand. Due to the difference with the 2008 results not much can be said about how the composition of the beach changes in time, but since there is not much sediment transport no substantial changes are expected. There was however a mention of a beach nourishment in the 2008 report which might be of influence, so it is advised that samples are taken in the same places in the coming years.

6. Quarry Measurements

During the fieldwork two quarries were visited. In the Marciana quarry near Devnya 20 stones were arbitrarily selected. Subsequently the weight and five dimensions of the stones were measured. With these facts basic stone properties, like nominal diameter, elongation rate and blockiness, can be determined. With the latter two parameters the design significant wave height can be determined. Furthermore the amount of heavy stones, 1-3 tons, was estimated in a heap of stones.

First of all the operational aspects of the quarry will be described in section 6.1. The stone properties are determined in section 6.2.

6.1 Quarry operation

In a quarry stony material is extracted from the earth, sorted out and sold (figure 6-1) The material is usually extracted by drilling holes near the edge of a bench of about 20 meters high (1). These holes are filled with explosives. After ignition cracks will form and the stony material will fall onto the lower bench. The distance between the holes, the amount of explosives and the order of blasting determines the various dimensions of the stones.

Subsequently the stony material is transported to the jaw crusher (4). In here the stony material is crushed into smaller particles and then transported by conveyor belts to a sorting machine (6, 9). Sometimes there is an additional sorting directly after the jaw crusher (as in the picture below) to improve following crushing and sorting processes (5). Large stones (1-3 ton) will not be crushed, but used for example in hydraulic engineering. When using producing aggregates the dust has to be removed by means of washing (8). After sorting the material will either be deposited in a stockpile (7, 11) or transported directly to a customer via rail or road (12).

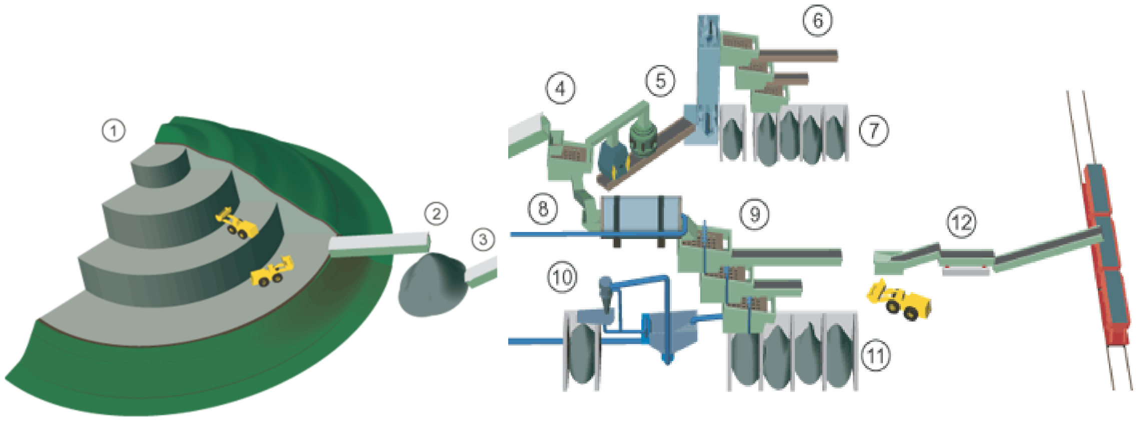


Figure 6-1: Quarry operation

The Marciana quarry produces besides aggregates and cementations materials also additives for the pharmacy and food for animals (200-300 µm). Therefore a ball crusher and a dust separation system are added to the process. The dust is separated by means of an oven which lifts the light particles and a skimmer separates the air from the light particles by the centrifugal principle. (figure 6-2)

By sorting the stony material into various sizes almost the complete output of the sorting operation can be sold.

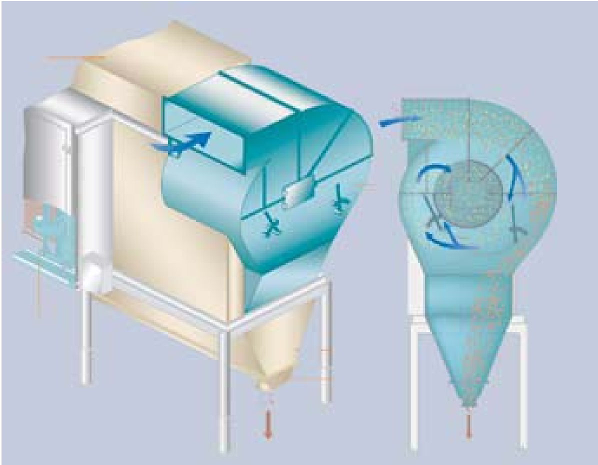


Figure 6-2: Skimmer

6.2 Stone properties

6.2.1 Specific density

Since no sample from the quarry was taken, the specific density can't be verified in Delft as described in the NEN 5186. Fortunately in previous years samples were taken and the specific density was determined. Averaging the results of previous years gives a specific density of about 2356 kg/m^3. In the following calculations we will take the rounded specific density of 2350 kg/m^3.

Year [-]	Density [kg/m^3]
2002	2284
2003	2349
2004	2352
2006	2400
2007	2405
2008	2345
Average	2355.83

Table 6-1: Estimated specific density per year

6.2.2 Nominal diameter (Dn50)

With this averaged specific density and the measured weight the nominal diameter can be determined. The quarry measurements this year were carried out twice with two groups. For the weight and dimensions the average of these two groups is taken. With some stones the measured value of only one group is taken because the value of the other group seems rather unrealistic, probably due to problems with the weighing scale.

The nominal diameter can be calculated as:

$$d_n = \sqrt[3]{V} = \sqrt[3]{M/\rho}$$

V=Volume [m3]
M=Mass [kg]
P=Density [kg/m3]

To find the distribution of the nominal diameter the stones are ranked by their weight. With the number of stones a frequency of exceedance is determined. Subsequently this is presented in a graph to check whether the material is well graded (S-curve) and normally distributed.

Weight (kg)	Volume (m^3)	D_n50 (m)	D_n50 (mm)	Exceedance Frequency (%)
19.5	0.008	0.202	202	5
21.5	0.009	0.209	209	10
21.5	0.009	0.209	209	15
26.0	0.011	0.223	223	20
26.5	0.011	0.224	224	25
28.0	0.012	0.228	228	30
29.5	0.013	0.232	232	35
30.0	0.013	0.234	234	40
30.5	0.013	0.235	235	45
34.5	0.015	0.245	245	50
35.5	0.015	0.247	247	55
41.5	0.018	0.260	260	60
41.0	0.017	0.259	259	65
44.0	0.019	0.266	266	70
44.0	0.019	0.266	266	75
49.0	0.021	0.275	275	80
51.0	0.022	0.279	279	85
52.0	0.022	0.281	281	90
60.0	0.026	0.294	294	95
71.0	0.030	0.311	311	100

Table 6-2: Stones ranked by weight

From the first graph one can see that the material is poorly graded. The selection process also has an influence on gradation. Hence from the graph one can recognize that the stones were not completely arbitrarily selected. Logically a pre-selection was made by those picking the stones out of the bunch. This statement is confirmed by the fact that the ratio between D85 and D15 is rather small:

$$\frac{D_{85}}{D_{15}} = \frac{279\ mm}{209\ mm} = 1.34 \leq 1.5$$

Figure 6-3: Measurement of the weight and five stone dimensions

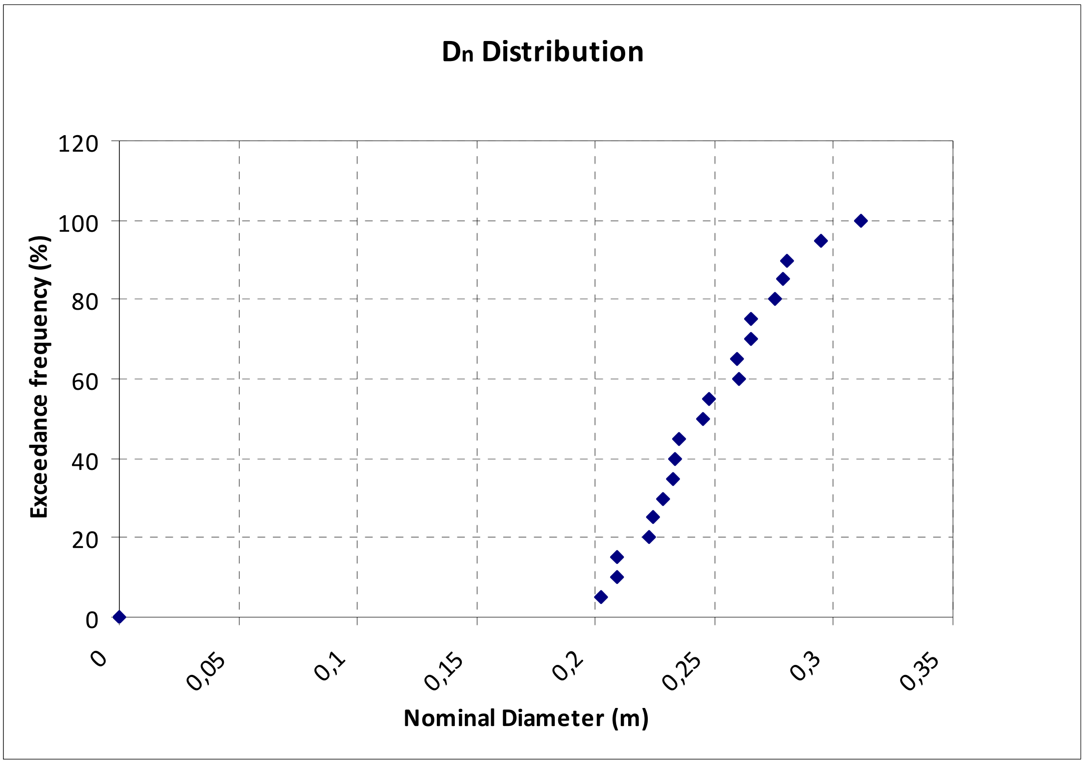


Figure 6-4: Gradation curve of the stones

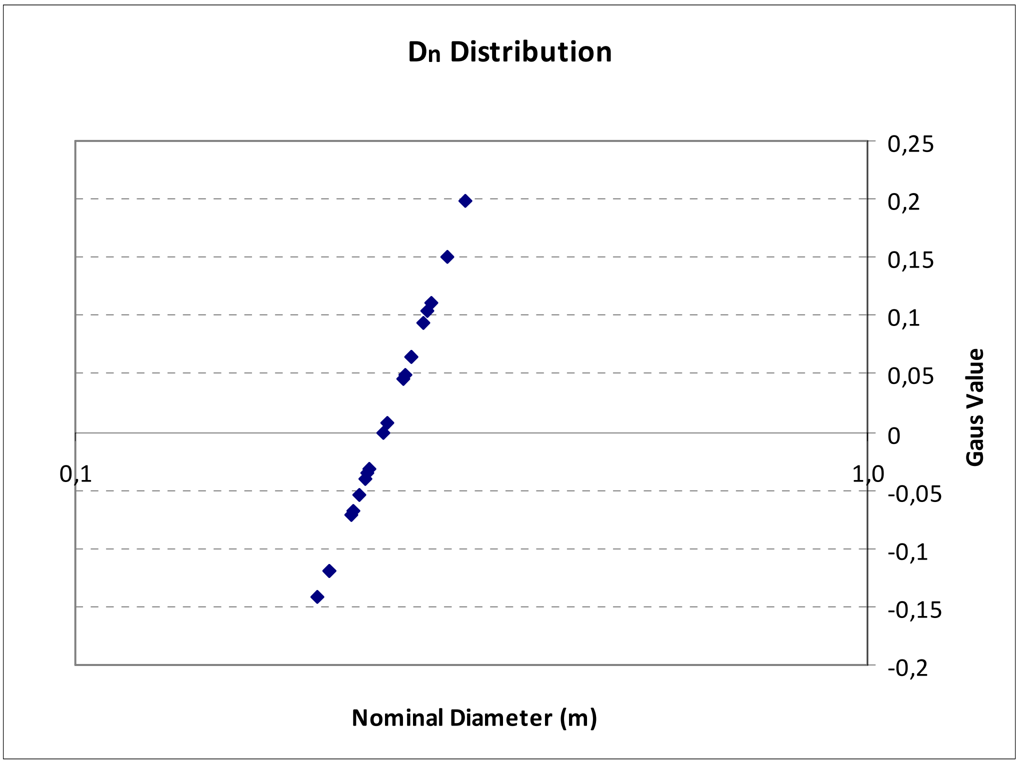


Figure 6-5: Logarithmic gradation curve of the stones

6.2.3 Elongation rate

The elongation rate is defined as the ratio of the longest axial length (l) over the smaller axial length (d). These lengths are measured relatively close to the centre of mass of the stones.

$$\frac{l}{d} = \frac{Longest\ axial\ length}{Shortest\ axial\ length}$$

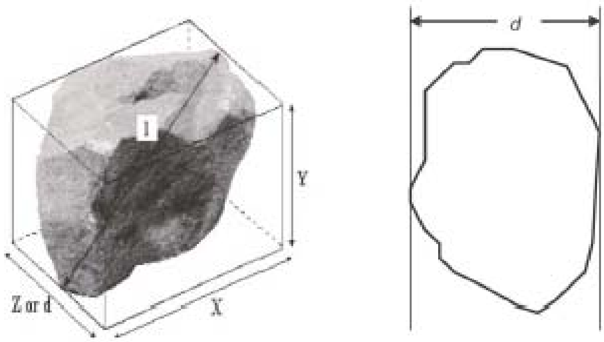


Figure 6-6: Definition of the elongation rate

In the table below the elongation rate is presented. The dimensions are averaged from the two measurements of the group and sorted by stone number. The average elongation rate is 2.11

Smallest Width (m)	Largest Distance (m)	Elongation (-)
0.25	0.52	2.08
0.28	0.52	1.86
0.27	0.44	1.63
0.25	0.46	1.82
0.15	0.49	3.27
0.27	0.54	2.00
0.24	0.56	2.33
0.40	0.54	1.34
0.24	0.47	1.96
0.33	0.54	1.64
0.32	0.47	1.45
0.31	0.48	1.55
0.41	0.58	1.40
0.26	0.48	1.83
0.23	0.50	2.17
0.15	0.54	3.57
0.18	0.43	2.36
0.18	0.44	2.44
0.21	0.50	2.36
0.15	0.49	3.23

Table 6-3: Elongation rate per stone

6.2.4 Blockiness

The blockiness is defined as the ratio of the volume of the stone (weight divided by specific density) over the volume of a box in which this stone fits exactly.

$$Bl = \frac{Volume\ of\ the\ stone}{Volume\ of\ the\ box} * 100\%$$

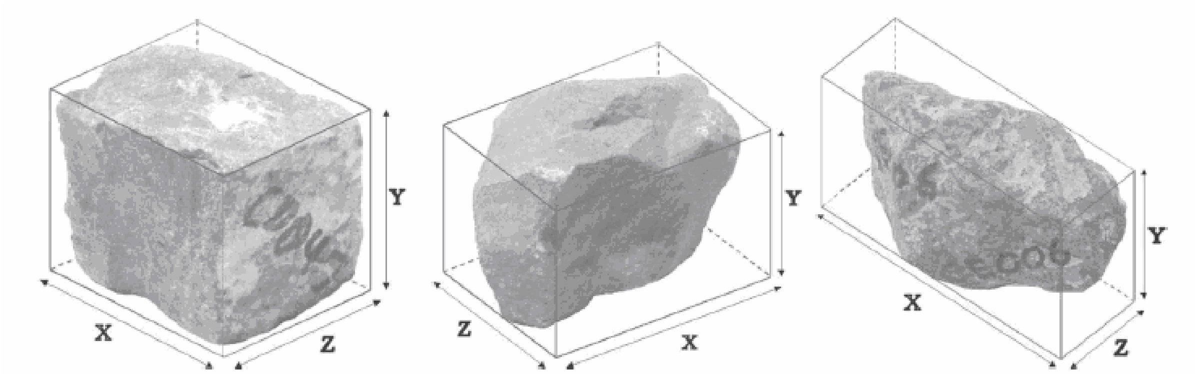


Figure 6-7: Definition of the blockiness

In the table below the blockiness per stone is presented. The dimensions are averaged from the two measurements of the group and sorted by stone number. The average blockiness is 34.055.

Volume Stone (m3)	Volume box (m3)	Blockiness (%)
0.026	0.043	59.961
0.019	0.054	34.732
0.011	0.028	40.159
0.017	0.061	28.430
0.011	0.032	34.498
0.022	0.065	34.041
0.018	0.060	29.638
0.022	0.077	28.006
0.012	0.040	29.428
0.021	0.070	29.664
0.019	0.055	33.943
0.015	0.047	31.154
0.030	0.094	31.979
0.015	0.043	35.161
0.013	0.046	27.323
0.008	0.023	35.583
0.009	0.029	31.821
0.009	0.027	34.330
0.013	0.041	31.662
0.013	0.032	39.594

Table 6-4: Blockiness per stone

6.2.5 Significant wave height

With the above calculated stone properties the maximum allowable significant wave height for the large stones (1-3 ton) from the Marciana quarry on a certain breakwater can be determined. The maximum significant wave height can be calculated with the Hudson and Van der Meer formula.

Hudson

The Hudson formula is defined as follows:

$$\frac{Hs}{D_{n50}} = \sqrt[3]{K_D * \cot(\alpha)}$$

Since for a breakwater probably only the large stones of the Marciana quarry can be considered, first the nominal diameter of these large stones has to be determined. Sadly from only four stones the dimensions were measured:

L (m)	B (m)	D (m)	BLx(est)	V (m^3)	D_{n50} (m)
1.7	0.8	0.75	65	0.663	0.871976
1.6	0.9	0.7	60	0.6048	0.845676
1.4	1.4	0.5	70	0.686	0.881945
1.4	0.9	0.7	45	0.3969	0.734898

Table 6-5: Determination of the nominal diameter of some random selected large stones

The average nominal diameter of these four stones is D_{n50}=0.83 m.
The remaining parameters are:

$\Delta=(\rho_s - \rho_w)/\rho_w$	1,55
KD	3,5
Cot(α)	3

Table 6-6: Parameters Hudson formula

This results in a maximum allowable significant wave height of about Hs = 2.50 m.

Van der Meer

The Van der Meer formula is more accurate than the Hudson formula because more parameters are included and is defined as follows.

$$\frac{H_s}{\Delta D_{n50}} = 6.2P^{0.18}\left(\frac{S_d}{\sqrt{N}}\right)^{0.2}\xi^{-0.5}$$ (Plunging breakers)

$$\frac{H_s}{\Delta D_{n50}} = 1.0P^{-0.13}\left(\frac{S_d}{\sqrt{N}}\right)^{0.2}\xi^{P}\sqrt{\cot\alpha}$$ (Surging breakers)

Where the transition of these formula is given by the transition coefficient:

$$\xi_{Transition} = [6.2 * P^{0.31} * \sqrt{\tan\alpha}\,]^{\frac{1}{P+0.5}} = [6.2 * 0.1^{0.31} * \sqrt{1/3}\,]^{\frac{1}{0.1+0.5}} = 2.55$$

The Iribarren number for the imaginary breakwater:

$$\xi = \frac{\tan\alpha}{\sqrt{H_s / L_0}} = \frac{1/3}{\sqrt{0.05}} = 1.49 \leq 2.55$$ → So the formula for plunging breakers has to be used

With the previous calculated elongation rate and blockiness probably a higher stability parameter can be used which results in a higher maximum significant wave height.
The elongation rate and the blockiness of the 20 middle size stones are used for this calculation, since it was not possible to measure the weight of the larger stones (and so calculate the blockiness).
First of all the single layer porosity, single layer thickness and the double layer porosity have to be determined with the following regression formula:

Parameter = A + B·BLc_m + C·l/d_m + D (BLc)

Mean value of blockiness BLcm (%)	34,05
Mean value of elongation (l/d)m	2,11
Standard deviation of blockiness σ(BLc)	7,035

Table 6-7: Mean value of the blockiness, - elongation and standard deviation of the blockiness

Then the appropriate parameter can be chosen with the table below (bold):

Parameter	slope	A	B	C	D	Value
Singe layer porosity n_v	1:1.5	42.38	-0.2177	3.695	-0.4128	39.8593
	1:2	42.9	-0.2204	3.74	-0.4179	40.34644
	1:3	**43.46**	**-0.2233**	**3.789**	**-0.4233**	**40.87309**
Single layer thickness k_t	1:1.5	1.1375	-0.0026	-0.1588	-0.0003	0.711791
	1:2	1.0736	-0.0024	-0.1499	-0.0003	0.67348
	1:3	**1.1038**	**-0.0025**	**-0.1541**	**-0.0003**	**0.691413**
Double layer porosity n_v	1:1.5	34.53	-0.2137	3.446	0.1852	35.82764
	1:2	35.94	-0.2224	3.586	0.1928	37.29028
	1:3	**36.2**	**-0.224**	**3.613**	**0.1942**	**37.56262**

Table 6-8: Parameters for regression formula to calculate the porosity and the layer thickness

Subsequently the stability parameters C_{pl} and C_{sur} defined as "6.2" and "1.0" in the Van der Meer formula is taken from the table below.

BLc-range	l/d range	Armour Porosity (%)	Placement method	"6.2"	"1.0"
40%-50%	**1.3 - 3.0**	**38.7**	**standard**	**7.09**	**-**
40%-50%	1.3 - 3.0	36.1	dense	6.68	1.67
50%-60%	1.3 - 3.0	37.1	standard	6.44	1.51
50%-60%	1.3 - 3.0	35.2	dense	7.12	2.08
60%-70%	1.3 - 3.0	35.5	standard	7.71	2.63
60%-70%	1.3 - 3.0	34.4	dense	10.85	-
50%-60%	1.0 - 2.0	36.1	standard	8.50	1.45
50%-60%	1.0 - 2.0	34.6	dense	8.80	-

Table 6-9: Stability Parameters

The remaining parameters in the Van der Meer formula are assumed as follows:

Permeability P = 0.1
Number of waves N =7500
Damage level S = 10
Finally the maximum allowable wave height is estimated:

$$H_s = C_{pl} P^{0.18} \left(\frac{S_d}{\sqrt{N}} \right)^{0.2} \xi^{-0.5} * \Delta * D_{n50} = 7.09 * 0.1^{0.18} \left(\frac{10}{\sqrt{7500}} \right)^{0.2} * \sqrt{1.49} * 1.35 * 0.83 = 4.15\, m$$

6.2.6 Estimation of the available stones

In the previous chapter the wave height for the large stones in the heap of the Marciana quarry was determined. It could be interesting to find what kind of a structure can be constructed with these stones. Therefore it is important to find out how many large stones are available in the heap.
First of all the surface of the heap is determined by measuring the perimeter. This results in a surface of about 1250 square meters. The nominal diameter of the large stones has been determined; one can easily determine the average surface of the stones which is about 1 square meter. Furthermore it has been assumed that about 60 percent of the surface is covered with large stone in a single layer. This results in a total number of about 750 stones (1250*60%).

Figure 6-8: Heap of large stones in the Marciana quarry

In water depth of about 10 meters with this heap of stones a breakwater with a length of 10 meters can be constructed. This implies that for a breakwater of 100 meters length 7000 stones more are required. Though probably the stone quality is insufficient for a breakwater.

7. Feasibility Study Floating Marina

A feasibility study of a floating marina is the subject of this chapter. After a discussion of the desired location in section 7.1 the local conditions are treated in section 7.2. With these local conditions and the design criteria, which are described in section 7.3, a breakwater type can be chosen in section 7.4. After this a preliminary design can be made, this is the topic of section 7.5. the chapter is concluded with some conclusions and recommendations in section 7.6.

7.1 location

One of the objectives of the Fieldwork Coastal Engineering 2009 is to perform a study for the construction of a new marina. To make this possible data regarding depths, position of the coastline, soil information and wave data have been collected. With the aid of these data, a preliminary design of a floating marina with floating breakwater is made.
The location of the new marina has already been chosen. The new marina will be located on the east coast of Bulgaria, near the city of Varna. The location is indicated in figure 7-1.

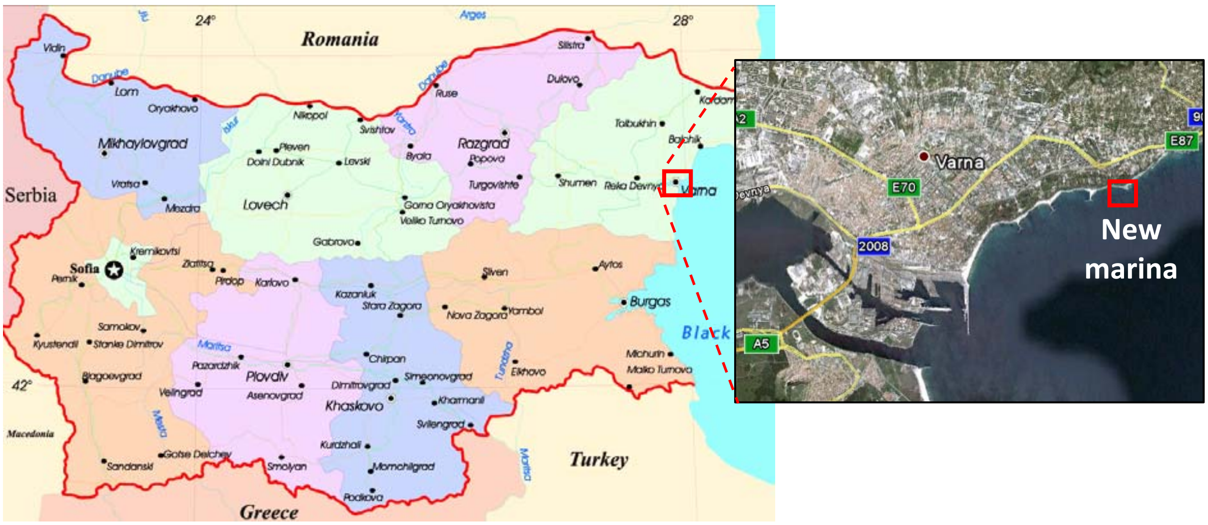


Figure 7-1: Location of the new marina near Varna, Bulgaria.

One of the options to construct a marina is to apply a removable floating marina in combination with a floating removable breakwater. This can be desirable as the marina can have a slender structure and the breakwater can be designed for mild conditions; it will only be in place in the summer months. This option is further investigated in this chapter; first a synergy with an existing project will be given. Thereafter the local environmental conditions regarding the properties of the coast, the bathymetry and wave conditions, and the design criteria are considered. With these criteria a Multi Criteria Evaluation could be executed to determine the type of breakwater. After this a preliminary design will be made. Subsequently, the implications of the chance of failure of the marina a are going to be discussed. In the last section, conclusions and recommendations will be given.

7.1.1 Synergies with existing projects

During the field trip there was also a visit arranged to the manufacturer Ship Machine Building. The company is mainly active in fabrication of ship equipment, fabrication of floating reinforced-concrete vessels, prefabrication and site erection of metal structures, ship repair activities, machining and non-destructive and mechanical testing. Until now, the company has produced

more than 180 sets of hatch covers, 350 floating reinforced-concrete vessels and 75000 tons metal structures. It is really an experienced manufacturer in producing floating structures, which are needed for a floating marina.
During the visit a pontoon built by that company was displayed. The pontoon is also built for floating structures. The outside layer of the concrete can survive in salt water to protect the inner steel from corrosion. The size of the pontoon is 31 meters in length, 15 meters in width and the draught is 4.5 meters. In our design of floating breakwaters, this length and draught are used. Moreover, because SMB is a local producer, the construction cost will be quite reasonable.

7.2 Environmental conditions

As it is desired to apply a floating marina only the environmental conditions prevailing in summer are relevant to the design.

7.2.1 Location marina

General

The new marina will be located in the bay in front of the city of Varna. The bay itself is quite sheltered from waves. For the design of the marina only the waves from the east and southeast are able to reach the marina and have to be accounted for. Along this part of the coast almost no sediment transport takes place, simply because there is hardly any sediment available to transport.

Description of the local coastline

In this section a qualitative description of the coastline is given. This is necessary to be able to determine where the access from the land to the marina can best be situated.

Figure 7-2: Map of marina location

At the location of the new marina the position of the coastline has been recorded with a Garmin GPS device. The actual waterline could not be recorded due to the presence of revetments. With

the GPS device the road which follows the coast has been tracked. Later with the aid of a Google Earth image, the position of the waterline has been estimated. These data have been used to draw a map of the project location (see figure 7-2). With the aid of this map it is possible to make a layout and to determine the optimal location for the new marina.
Also the type of revetments and other coastal structures near the location of the new marina have been investigated. On the map descriptions of the different kind of structures can be found. Below a more detailed description of the coast from west to east will be given.

From the T-groyne in the west until the small groyne the coast consists of a stone revetment which has a width of about 5 to 7 m (see figure 7-3). The small groyne has an armor layer consisting of Tetrapods (see figure 7-4). The width of this revetment is 8 to 10 m.

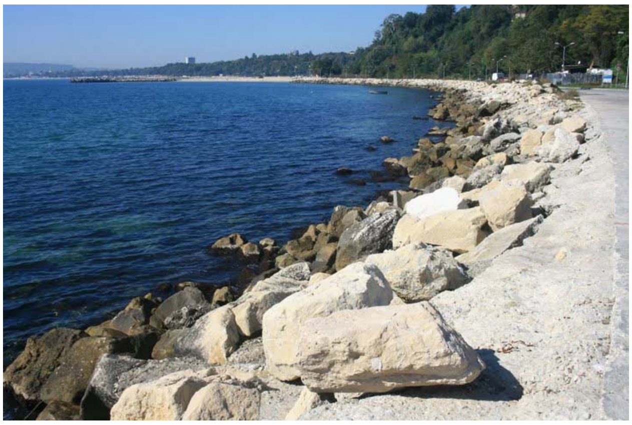


Figure 7-3: Rock revetment west of small groyne

Figure 7-4: Small groyne with Tetrapods as seen from the water

East of the small groyne (see figure 7-5) there is a small sandy beach. East of the groyne the rock revetment continues. Here the rock revetment has a width of 4 to 6 m. In front of this revetment are some collapsed groynes consisting of concrete blocks placed behind each other. In front of the revetment some sand is present, as can be seen on the right photo in figure 7-5.

Figure 7-5: Beach east of small groyne, stone revetment and collapsed groynes. Left: view from small groyne towards the east. Right: view on the small groyne from the west.

7.2.2 Bathymetry

The bathymetry at the location of the new marina is one of the boundary conditions that have to be taken into account during the design of the marina. It is necessary to determine whether the water is deep enough to sail and the depth is an important design parameter for the breakwater and the anchorage. The bathymetry is also necessary to determine the local (shallow water) design wave conditions. During the fieldwork the bathymetry at the location of the new marina has been measured.
In figure 7-6 the sailed track of the boat is displayed. A quite dense sailing pattern has been followed. This is convenient, because the more measuring points, the more accurate the resulting bathymetric chart will be.

Figure 7-6: Sailing track

Results

The result of the measurements is a long list of points with X and Y coordinates and the measured depth Z at those points. With the aid of the computer program Surfer, the measured data points have been interpolated on a regular grid. Next a bathymetric map with contour lines has been made. The result can be found in figure 7-7. In this figure the contour map is displayed on a Google Earth image of the area. It should be noted that the bathymetric chart has been cut of just east of the T-groyne, because in the interpolation process the presence of the T-groyne could not be taken into account.
In the bathymetric chart one can see that at two places, namely at the sandy beach east of the T-groyne and at the tip of the small groyne, the map intersects with land, which is not correct.

These are interpolation errors, because the position of the coastline has not been taken into account. That is why in the vicinity of the coast the map might be displaying wrong depths.

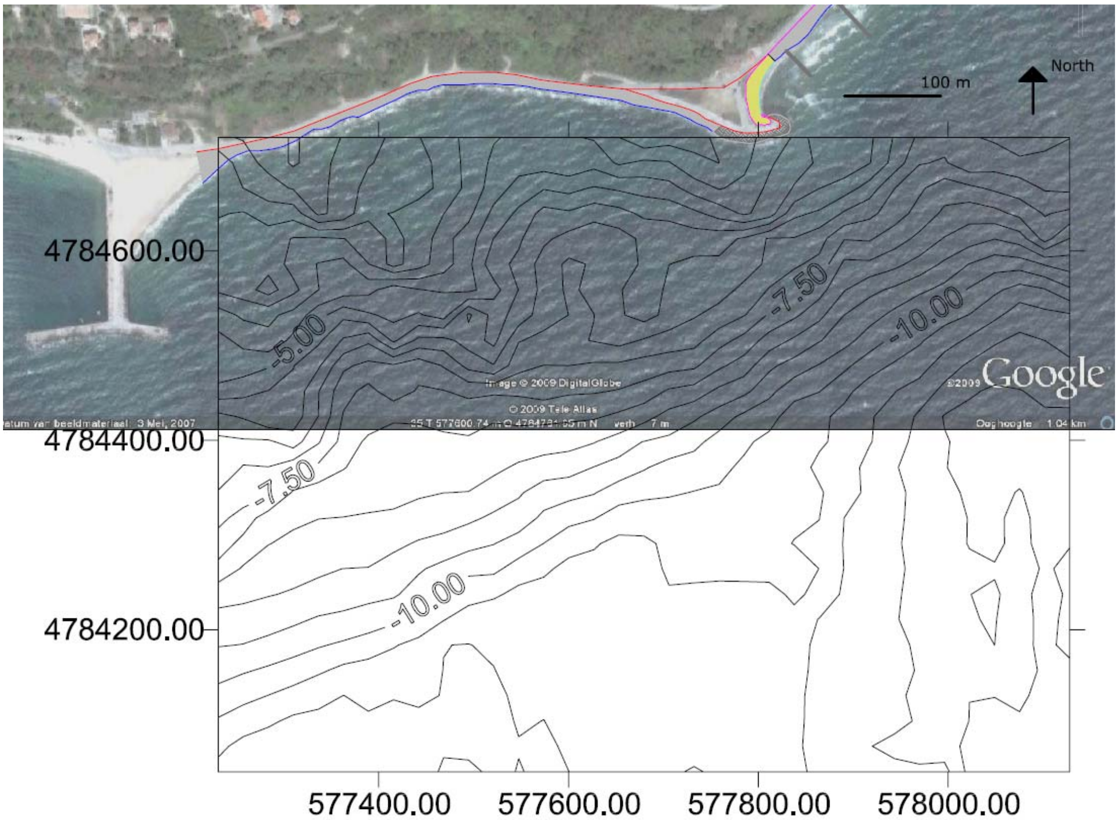


Figure 7-7: Bathymetric chart with Google Earth underlay. Depths are in m with respect to mean sea level.

As stated before, the contour map will be used to determine the location of the marina. It is also used to make a representative cross shore profile as input for a SWAN calculation, which will be executed later in this chapter.

7.2.3 Wave climate

Using long-term statistics (Argoss 2009) the following wave characteristics have been determined:

In winter

- The deep water significant design wave height during winter H_s is 8 meter.
- The design wave period during winter is 10 seconds.

In summer

- The deep water significant design wave height during summer H_s is 2 meter.
- The design wave period during summer is 6 seconds.
- The dominant wave direction is from the east.
- No tides.

The summer wave characteristics and the dominant wave direction have been used as SwanOne input to calculate the significant design parameters (see also Appendix 1).

The SwanOne calculation has revealed that at the designated location of the marina the wave parameters are as follows:

- Significant wave height H_{mo}= 1.5m
- Average wave period T_{m1}= 4.83s
- Water depth d= 7.5m
- Wave length L= 32.6m

The wave length has been calculated using the first three parameters.

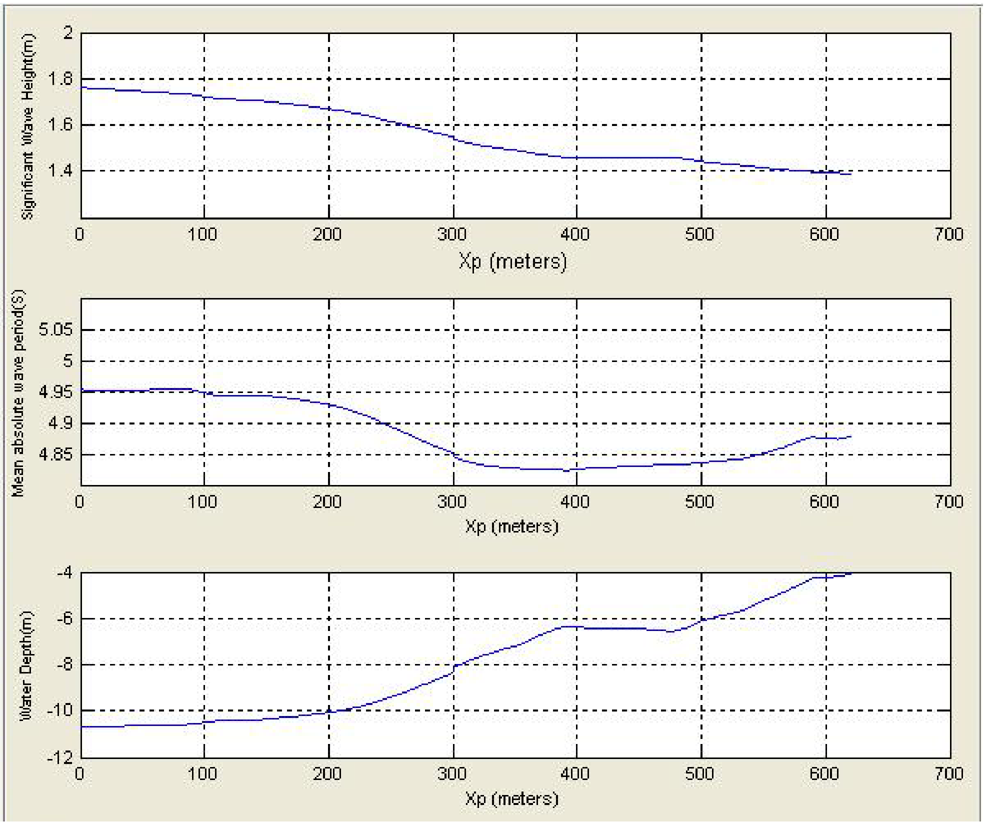
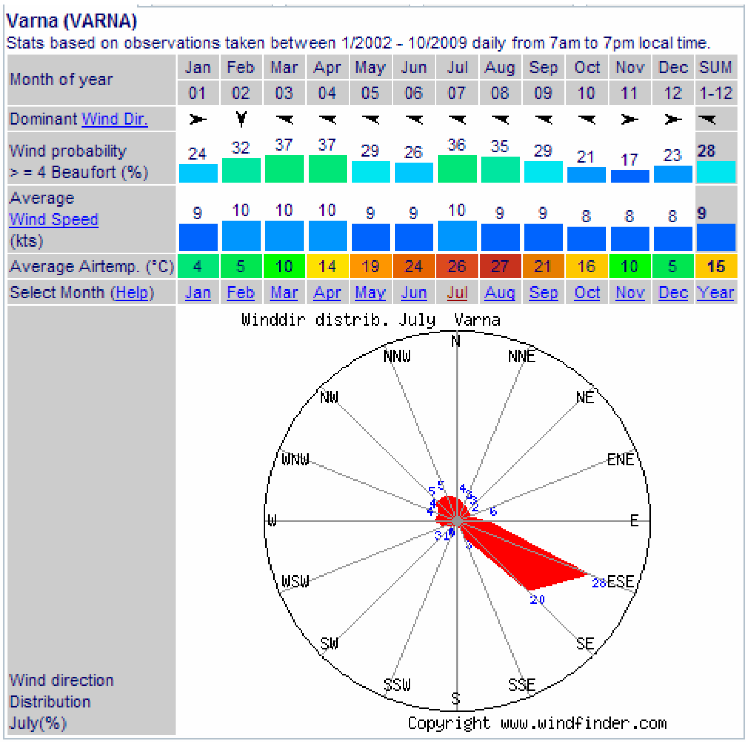


Figure 7-8: SwanOne output

7.2.4 Climate

According to the wind statistics in the figure below the wind mainly comes from an ESE direction (112.5°) with a velocity of 10 knots (5 m/s) (Windfinder 2009). These data have been used as SWAN input. With around 40 mm/month precipitation is comparatively low in the summer months. The air temperature on a summer day is around 25°C and on a summer night 15° on average (Weather Network 2009).



Varna (VARNA)
Stats based on observations taken between 1/2002 - 10/2009 daily from 7am to 7pm local time.

Month of year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	SUM
	01	02	03	04	05	06	07	08	09	10	11	12	1-12
Dominant Wind Dir.													
Wind probability > = 4 Beaufort (%)	24	32	37	37	29	26	36	35	29	21	17	23	28
Average Wind Speed (kts)	9	10	10	10	9	9	10	9	9	8	8	8	9
Average Airtemp. (°C)	4	5	10	14	19	24	26	27	21	16	10	5	15
Select Month (Help)	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year

Figure 7-9: Wind statistics Varna. Source: www. windfinder.com

7.3 Design criteria

To assure that the design satisfies the project owner's wishes to the highest possible degree and assure a high quality of the technical design a list of design criteria has been drawn. These criteria will be used as guidelines during the design process. Furthermore, they will serve as a quality check of the design at the end of the process. For further detailing of the environmental conditions mentioned in the criteria (wave climate, climate, location) see the following section.

7.3.1 Marina

The design criteria of the marina are based on the size of the yachts and the proposed capacity of the marina.
The capacity of the marina should fulfill the growing numbers of the yachts in Varna. The capacity is based on the number of vessels in some nearby marinas. In summer the number of vessels in a marina ranged from 50 to 70. Because it is a new port, generally the capacity of the port should be bigger than the old ones. So 100 vessels is taken as a design capacity of the marina.
The size of every berth in the marina should hold the most common seen yachts and also the yachts size in the near future. The size of the yachts is estimated from the Google earth image. The overall length and the beam of the yachts in the nearby marina are determined. The draught is taken as the typical value of the yachts.
Each berth should accommodate a yacht. Also extra space is necessary between the floating dock and the yacht. The width of the berths is calculated as: 5m+5m+0.4m*2+0.6m=11.4 m Which is twice the beam of a yacht and the additional space
According to the California Department of Boating & Waterways (2005) the width of the channel in a marina should be 1.75 x $L_{b,}$, where L_b is the length of the longest berth perpendicular to the channel.
Inside the marina, the wave conditions should be mild enough for the mooring of yachts.

As the marina is relocated to sheltered waters all the loads like uniform live load, as it freezes in winter in Bulgaria, ice and snow load are considered. Others like wind loads, unusually heavy live point loads are also important.

Design criteria	
Marina capacity	100 vessels
Length of the yachts	15 m
Beam of the yachts	5 m
Draught of the yachts	3.5 m
Length of the berths	15 m
Width of the berth	11.4 m

Table 7-1: Design criteria

7.3.2 Breakwater

- Sufficient stability in climate and wave conditions (crucial for functions as walking on the breakwater or mooring ships to it)
- Functioning (limit wave action and overtopping) only in summer (since the marina operates only in summer)
- Stability during its lifetime
- Sufficient wave (energy) dissipation (see table 7-2)
- No current
- Durable material
- Easy implementation
- Little maintenance
- Low costs
- Esthetics

Wave direction relative to vessel	Peak Wave Period T [s]	Return Period of the Event		
		1 in 50 yrs	1 in 1 yr	1 in ea. week
Head seas	T < 2		Hs 0.31m	Hs 0.30m
Head seas	**2 < T < 6**	**Hs 0.61m**	**Hs 0.30m**	**Hs 0.15m**
Head seas	6 < T	Hs 0.61m	Hs 0.30m	Hs 0.15m
Beam seas	T < 2		Hs 0.30m	Hs 0.30m
Beam seas	**2 < T < 6**	**Hs 0.23m**	**Hs 0.15m**	**Hs 0.08m**
Beam seas	6 < T	Hs 0.23m	Hs 0.15m	Hs 0.08m
For Excellent wave climate multiply criteria by 0.75 *For Moderate wave climate multiply criteria by 1.25*				

Table 7-2: Provisionally recommended criteria for Hs, in a small craft marina. Source: PIANC report "Floating Breakwaters; A Practical Guide for Design and Construction".

7.4 Multi Criteria Evaluation of different breakwater types

There are several types of breakwaters. Each with its own advantages and disadvantages depending on the design criteria and boundary conditions. Next three types of breakwaters are described in general. Following by a Multi Criteria Evaluation to determine the type of breakwater to design in this preliminary design study on a floating marina. Followed by a more in-detail description of the chosen breakwater type.

7.4.1 Type of breakwater

Conventional breakwaters

Fixed breakwaters work on the basis of wave energy reflection and are bottom founded. There are three types of fixed breakwaters, the rubble mound breakwater, the caisson breakwater or monolithic breakwater, and the composite breakwater. The composite breakwater is actually a combination of a rubble mound sill on which a caisson is placed. For a caisson breakwater totally reflected waves in front of the caisson have a negative effect to shipping. The ships should be strong enough to survive those large waves.

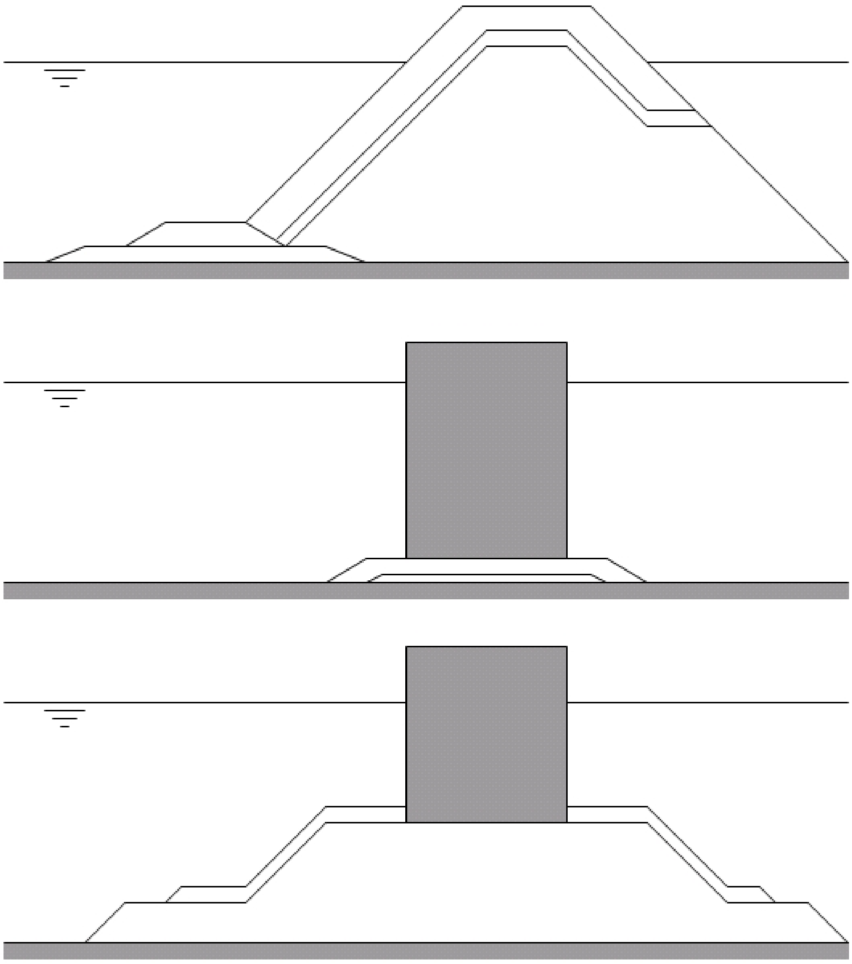


Figure 7-10 Types of different fixed breakwaters, from above to below the rubble mound, the caisson, and the composite one

Floating breakwaters

Floating breakwaters work on the basis of interference to either cancel out wave trains or by using turbulence and friction to remove energy from waves. A combination of both processes is also possible.

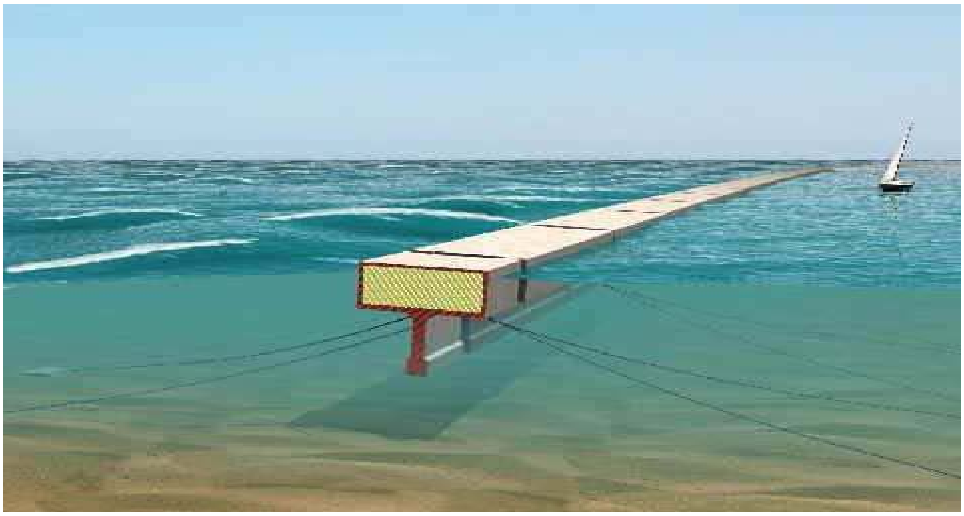


Figure 7-11: An example of floating breakwater

If categorized in more detail there are three main types of floating breakwaters namely the reflecting floating breakwaters, the displacement floating breakwaters and the dissipative floating breakwaters.
In this study a removable floating breakwater is preferred because its functioning suits the location. For the displacement type there are still two possibilities. It is an option to make the floating breakwater strong enough to withstand the severe wind and wave conditions of summer and winter. Therefore the breakwater does not have to be removed during winter times.
The other possibility is to remove the floating breakwater during winter times. There will be extra costs to move and install every year. In comparison with a permanent breakwater the total initial costs will be much less, but the maintenance costs are higher.

On the other hand floating breakwaters also have some obvious disadvantages.
Though floating breakwaters are more applicable if the subsoil is not of a good quality, they still have to be anchored into the sea bottom. Unlike a conventional breakwater, which fails gradually, during a severe storm, the mooring system of the floating breakwater will fail suddenly which can cause a lot of damage or loss. Moreover, the dislodged elements of the floating breakwaters could also damage the harbor facilities.
Floating breakwaters are only effective to a certain band of wave frequencies. So when the wave conditions are very complicated, the damping effect of the floating breakwater diminishes significantly.
The mooring system is always a complicated problem of floating breakwaters (chapter 7.5.4).

Submerged breakwaters

Submerged breakwaters are a special type of breakwaters distinguished from other emerged offshore ones. They are built with their crests submerged in the water. With this advantage they can not only avoid generating significant reflected waves and affecting the nearby shoreline, as other kinds of breakwaters do, but also save large quantities of engineering resources from the view of economics. In this case the advantage of submerged breakwater is not that much, because it is not in very deep water, so the construction cost will be larger than the floating breakwaters. Moreover, submerged breakwaters cannot be further used as a mooring point.

7.4.2 Multi Criteria Evaluation Breakwater

Several types of breakwaters can possibly applied: a year-round floating breakwater, a floating breakwater for summer and a submerged breakwater. As this is a preliminary study for a breakwater a Multi Criteria Evaluation (MCE) is used at an early stage to compare the types mentioned above. The three types are qualitatively compared using a list of criteria that indicate the value the breakwater has for the project owner.

Criteria	Floating breakwater removable	Floating breakwater permanent	Submerged breakwater
Degree of difficulty construction	+	+	-
Construction permit	+	+	-
Maintenance costs	-	0	0
Maintenance effort	-	0	-
Failing chance	+	-	0
Life time	0	-	+
Construction cost	+	+	-
Using comfort (wave penetration etc.)	+	+	-
Multi-functionality (use as mooring spot, fishing pier etc.)	+	+	-
Esthetics	-	-	+

Screening the table above it is evident that a floating breakwater that is removed in summer will be a good option. The main disadvantage of a removable breakwater is that it has to be brought into calm water when winter approaches. This requires a certain degree of planning and management. The project owner will have to decide himself if he can set up a reliable management that cares to do its task on a long term. It must be mentioned though that a submerged breakwater with floating jetties poses the same problem.

7.4.3 Advantages of a removable floating breakwater

The properties of a removable breakwater are discussed in this section in depth. Usually, there are two main aspects why floating breakwaters are used instead of conventional type of breakwaters: economical reasons and technical reasons.

Economical reasons

Compared to conventional breakwaters floating breakwaters usually have low initial costs, especially when it comes to very deep water. In [d'Angremond and Tutuarima, 1998] it is stated that, from an economic point of view, a rubble mound conventional breakwater is only preferable in water depth up to 8 meters. As can be seen from figure 7-12 the investment costs can thus not be the main reason for the construction of a floating breakwater in Varna. The water is too shallow to make it an economical advantage.

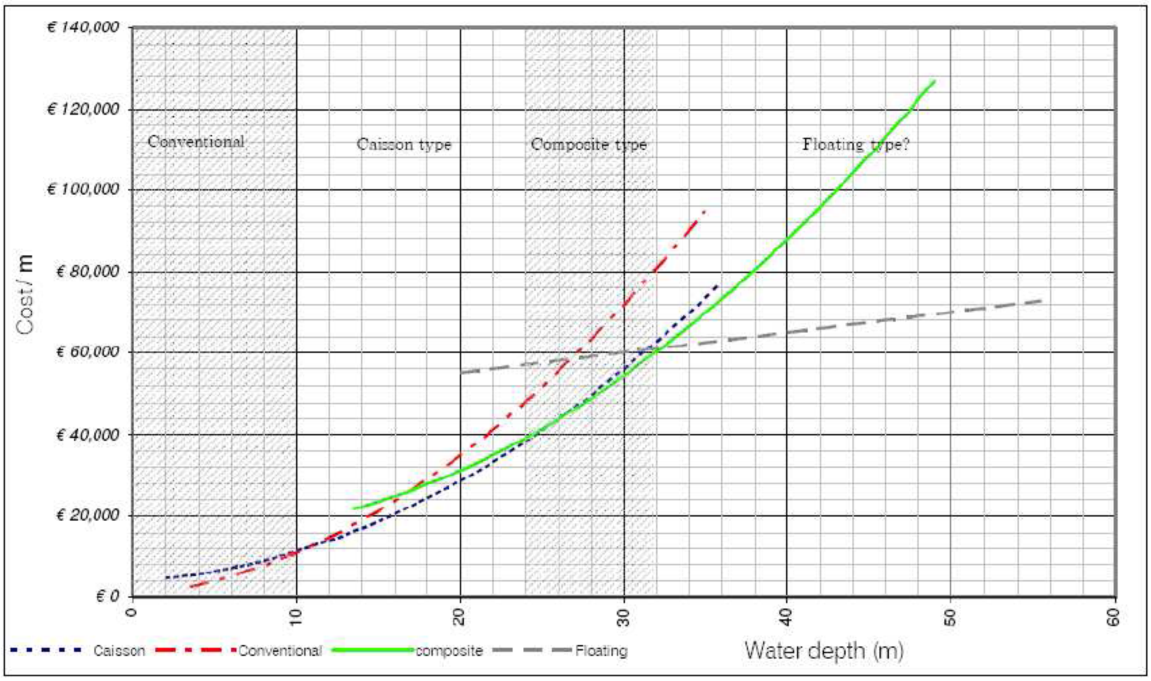


Figure 7-12: Comparison of construction cost per running meter depending on water depth as of 2006. Source: van Tol (2008), p. 8.

Though in reality many factors will influence the final cost of a floating breakwater. The mooring system strongly depends on the wave conditions and subsoil conditions. The floating breakwater only works effectively in a certain wave frequency band, so the local wave length is the determining factor of the size of the floating breakwater. Generally speaking, the initial cost of floating breakwaters is much lower than conventional breakwaters.

In Bulgaria, application for construction permits is a complex procedure, which is both time and money consuming. To construct a conventional breakwater construction permit is needed. Moreover, in Bulgaria, the department where you should apply for the permission is also not legally distinguished. But for a floating breakwater only mooring permission is needed which is simpler to apply for than for a construction permit.

When, like in this case, the floating breakwater is designed to be relocated to a lake during winter, the pontoons can directly be used for a temporary mooring place for the yachts during the winter period. This is an advantage, because for insurance companies, the yachts should be inaccessible from the land during the winter time. The port can also make money by stalling of yachts in winter. This all results in a cut down in the maintenance cost of the floating breakwaters.

In this special case the marina is only functioning during the summer time. The removable floating breakwater does not have to be designed for the winter period when severe wave conditions could happen. Instead of conventional breakwaters which should be strong enough to withstand the more severe wave or storm action during the winter.

Technical reasons

Soil condition should be good enough to support the vertical load generated by the weight of a conventional breakwater. In case of a floating breakwater the construction site is much more flexible than for a conventional breakwater. However, the floating breakwater still has to be anchored to the bottom.

The performance of a floating breakwater compared to the conventional is also quite different. Floating breakwaters are only effective for a certain band of wave frequencies. When other frequency waves are added, the functioning of the breakwater deteriorates quickly.
Scouring of the foundation is always a considerable problem when building a conventional breakwater. Because the conventional breakwater occupies the whole water depth, it acts as a partial dam which disturbs the littoral drift and traps the sediment at one side of the breakwater.
Floating breakwaters can be rearranged relatively easily during their lifetime. It is much easier to transport and rearrange according to specific needs and requirements.

Conclusion

From an economic point of view and in this specific case, a floating breakwater has lower initial costs and there can be a cut down in maintenance cost by using pontoons during the winter period.
From a technical point of view, a floating breakwater is also preferred because it can be more easily rearranged during the design life time. It is much more flexible than a conventional breakwater, when for example the capacity of the marina needs to be expanded.

7.5 Preliminary design

7.5.1 Layout marina

The set up of the marina is based on the values of the sizes of the berth determined in chapter 7.3 about the design criteria. In figure 7-13 below the detailed dimensions of each element is given.

Figure 7-13: marina layout in meters

Layout of the marina

To prevent dredging operations, the entire marina will be situated outside the -3.5 m depth contour. This is the maximum draught of the design vessel.
Three piers with 34 berths are necessary to accommodate all vessels. As a base layout, three piers next to each other are chosen. Between these piers, a certain free channel width is required.
The length of a berth will be 15 m, so the width of the channel will be 1,75 x 15 m = 26,25 m according to the California Department of Boating & Waterways (2005). For simplicity this will be rounded of to 26,5 m. In figure 7-14 an overview of the designed layout has been displayed.

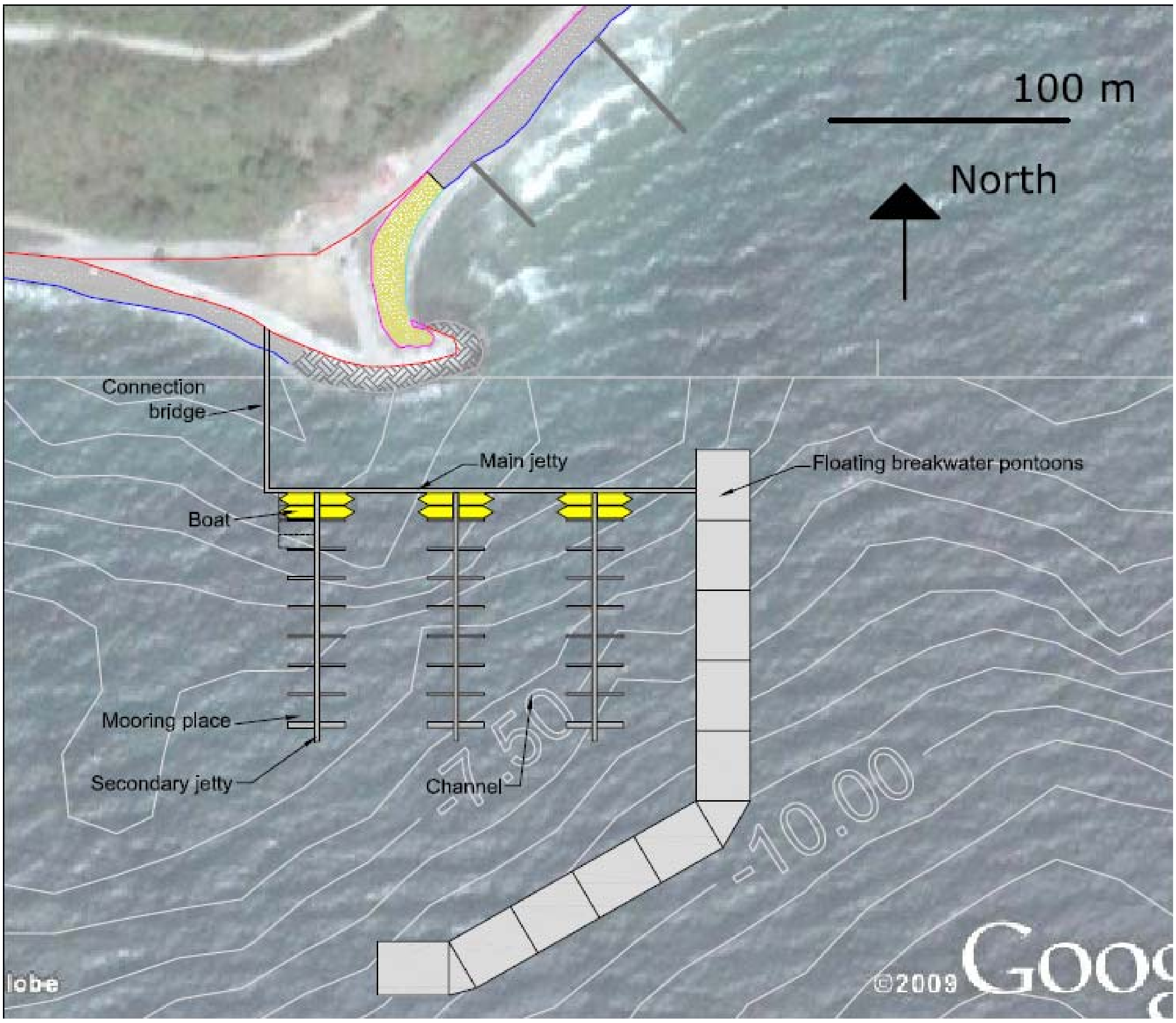


Figure 7-14: Overview of marina design

The floating breakwater will be situated east and south of the marina to block summer waves from the east. The wing to the south will prevent that diffracted waves will enter the marina.
A connection bridge or floating jetty will serve as connection between the marina and the main land. The landside of the connection will be situated just west of the Tetrapod revetment, above the stone revetment. The other side of the connection is connected to the main jetty wherefrom one can walk to one of the three piers. The distance from the main jetty to the coast is approximately 50 m.

7.5.2 Pontoons

In both the floating marina and the floating breakwater pontoons are used to provide buoyancy to support all the loads during the design life time. When the pontoons are serving as berthing systems in a floating marina both the dead load and live load should be taken into account. Examples of the dead load are the weight of the decking, the bolts and other equipment and examples of the live load are loading induced by people and during mooring and the lateral loads caused by, wind, waves and currents. The bigger the loads, the more capacity the pontoon should provide. Excess weight is required to achieve the designed freeboard and draft.

Marina pontoons are typically manufactured in concrete. The pontoon material selection should always take the influence of the environment into account. Especially due to the salt water the outer concrete layer should be strong enough to resist corrosion. Other environmental effects are also important, such as currents, waves, tides, flooding, wind, storms, extreme temperatures, ultraviolet exposure, impacts, and potential seismic activity.

The size of the pontoons is not only determined by the design criteria, but also by the manufacturer. In the preliminary design the size of the pontoon is offered by a local manufacturer in Varna. The size for the floating pontoon is 31 meters in length and the 4.5 meters in draught which was adapted from the manufacturer, but the width of the pontoon depends on the local wave length.

7.5.3 Breakwater

As discussed in chapter 7.4 the choice of the type of breakwater has been narrowed down to a breakwater which is removable in winter. Since it is neither feasible nor necessary to completely damp the incoming wave (energy) the floating breakwater will be a displacement floating breakwater instead of a reflective or dissipative breakwater. Depending on the geological conditions the breakwater might have to be divided into sections to reduce the forces onto the mooring system. Furthermore the length will also be limited to roughly 100 m by the requirement of transportability in the winter season. It is recommended to make these sections free-floating because it is technically not feasible to make sufficient connections between these sections.

Dimensioning

As this is a preliminary design the simplest shape of a floating breakwater is a simple pontoon, which is assumed to be rectangular in its cross-section. Double pontoons could be applied as well; double pontoons have the advantage of additional reduction of the wave field, but with the relatively calm wave conditions in summer double pontoons are not needed. In the MSc thesis of van Tol (2008) five parameters that characterize the properties of a floating breakwater are indicated:

- Beam width (W)
- Draught (D)
- Beam-to-draught ratio W/D
- Position of Centre of gravity (COG)
- Viscous roll damping

According to van Tol the wave transmission mainly depends on underflow, wave generation by the combined roll/sway motion and wave generation by the heave motion (Figure 7-15). In addition he proceeds to show that in terms of stability and wave transmission it is optimal for floating breakwaters to have a beam-to-draught ratio equal to 5.0 and a COG positioned as high as technically feasible. Furthermore he demonstrates that the beam and draught dimensions depend on the longest occurring wave. Figure 7-15 can be used to match the given wave length with the appropriate beam and draught dimensions (van Tol 2008: p. 49-55).

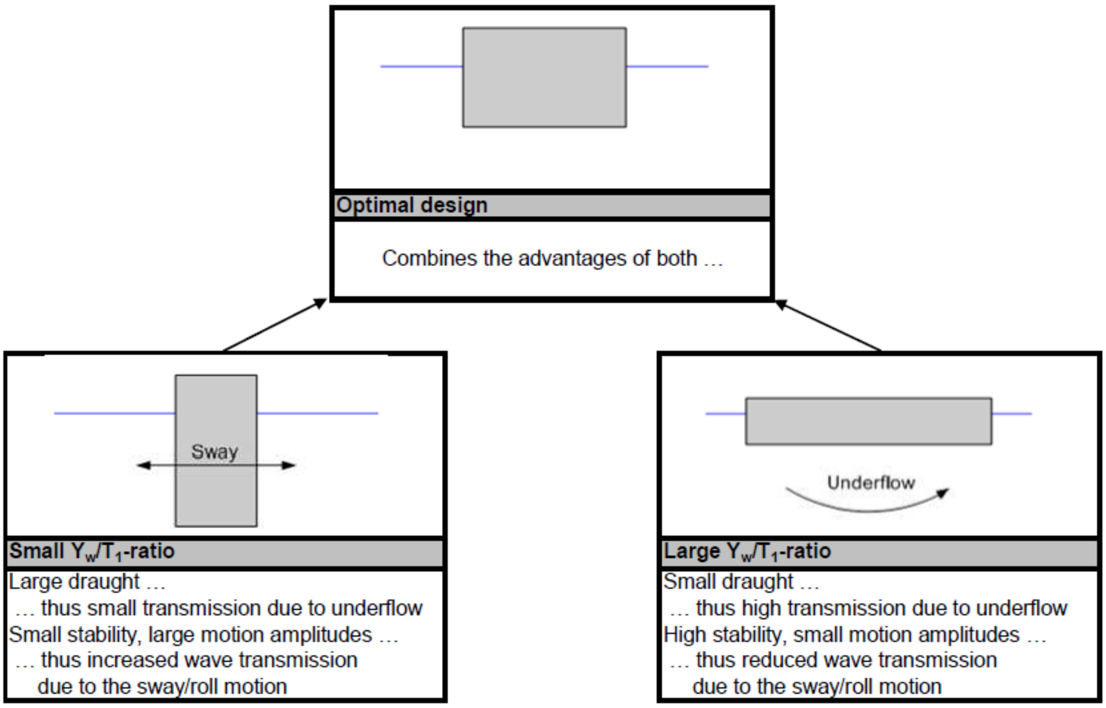


Figure 7-15: The most optimal design combines the advantages of a small and a large beam-to-draught ratio. Source: van Tol (2008), p. 51.

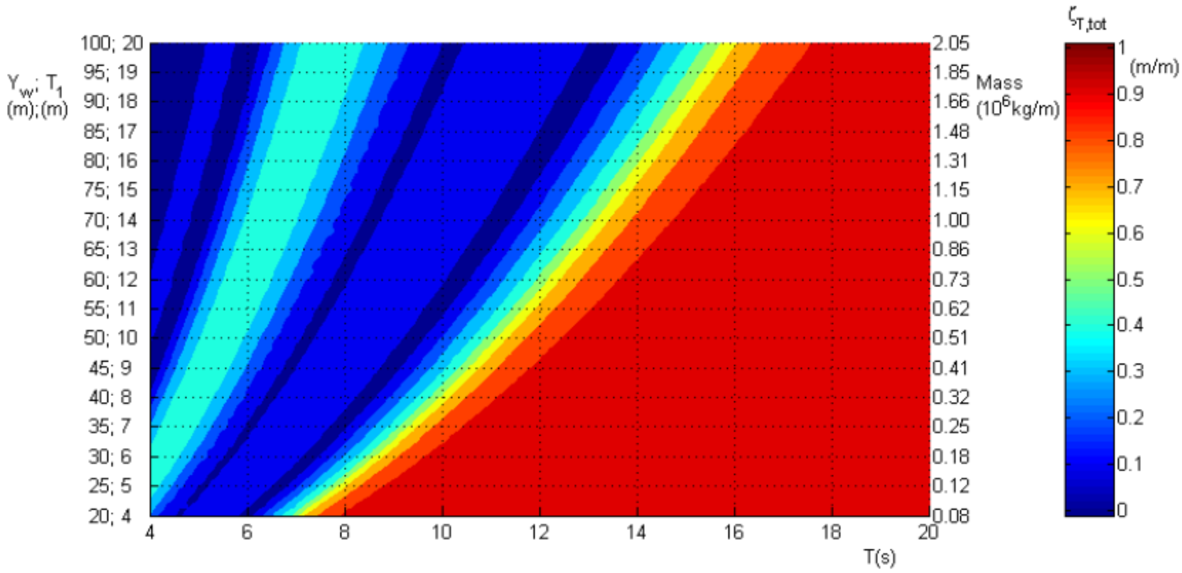


Figure 7-16: The effect of the beam draught on the wave-attenuating properties, beam-to-draught ratio=5.0 and the COG is as high as assumed to be technical feasible, on the wave transmission RAO. Source: van Tol (2008), p. 54.

In this case the longest occurring wave length is 32.6 m with a period of 4.83 seconds. The significant wave height H_s is 1.5 m and the required maximal wave height in the marina is H=0.15 m (see Chapter 7.3). The allowed wave transmission is in this case 10% ($\zeta_{T,tot}$=0.1).In combination with the beam-to-draught ratio and with the help of figure 7-16 this results in a draught D of 4.5

m and a beam width W of 22.5 m. According to the design criteria the freeboard should be about 1.0 m. The total height of the breakwater would be 5.5 m.

Design of breakwater section

After determining the rough dimensions of the floating breakwater some thought should be given to the design of each of these free-floating breakwater sections. The length of these sections depends on the geographical conditions and cannot be definitively determined until some soil measurements are carried out at the designated location of the marina. It is likely though that the length will be in the order of 100 m for transport reasons.

Each section will be divided into several compartments. These compartments do not offer the section structural stability but also make the breakwater less vulnerable to damage caused by for example ship collisions. One flooded compartment will not make the breakwater sink and loose its functionality. The inner walls provide the breakwater with stiffness and bear the ballast that might be needed to reach the required draught. Ballast can also be used to influence the position of the center of gravity beneficially; it should be positioned as high as possible.

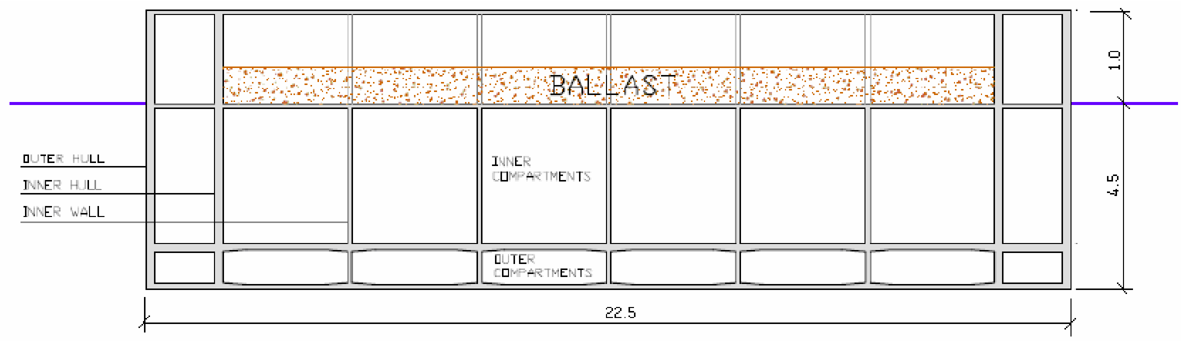


Figure 7-17: Layout concept for floating breakwater section. Source: van Tol (2008), p.79.

7.5.4 Mooring System

In poor soil, floating breakwaters are also applicable, but anchoring them to the sea bottom becomes more complicated. The mooring system is responsible for holding the floating breakwater element on its place. An equilibrium solution has to be sought between a very stiff system with small deflections and large forces, and a very soft system with large deflections and small forces. In the mooring system various ropes including synthetic and steel ropes are used. Below a table is given of synthetic ropes used for mooring systems.

		Polyester	HMPE	Circular braided nylon
Material		High tenacity polyester	High modulus polyethylene	Nylon
Specific weight		1,38 [kg/m^3]	0,99 [kg/m^3]	1,14 [kg/m^3]
Elongation				
	At 20% of MBL	3 [%]	0,8 [%]	16 [%]
	At 50% of MBL	6 [%]	2 [%]	22 [%]
	At break	12 [%]	4 [%]	>40 [%]
Tensile strength (MBL)				
	Diameter 120 mm	6057 [kN]	3924 [kN]	3345 [kN]
	Diameter 144 mm	8529 [kN]	5396 [kN]	4679 [kN]
	Diameter 168 mm	11301 [kN]	6867 [kN]	6023 [kN]

Table 7-3: Material properties for mooring lines

In higher wave conditions the floating breakwater will move and in case of shallow waters mooring lines are extremely loaded. In order to allow navigation and to reduce the maximum horizontal movements, the chain length is necessarily short compared to deep water installations. Shocks due to full line extension are expected and must be tolerated.
In the absolute absence of design guidelines long-term practice has suggested, at least for past installations, to use chains widely available in the market. Extreme loads do not only affect the mooring system but may also affect the structure durability. For example concrete structures may be heavily stressed and even crack in an extreme event. Such cracking may lead to accelerated corrosion of steel reinforcement.

Although the effect of wave direction on loads and motions of long structures had been studied for a long time, the accurate evaluation of mooring loads is complicated by the combination of the non-linear and irregular nature of the wave loads, by the non-linear reaction forces provided by the chains and by the presence of critical 3D effects (like for instance the interconnecting module constraints). In case of possible shocks on the mooring, inertia issues set further hurdles in the evaluation process. More specifically, most authors approach the problem of defining the mooring forces by describing the floating breakwater movements. Stiffness coefficients are derived from basic catenary equations, added mass and hydraulic stiffness and damping by potential theory and, hence, mooring forces are obtained from the equations of motion. As a conclusion maximum mooring forces may now probably be accurately evaluated by simulations but only in the absence of shocks.
In the present case, simply an I-shaped connection of floating breakwaters is assumed and in the curvature of the breakwaters a J-shaped connection is used.

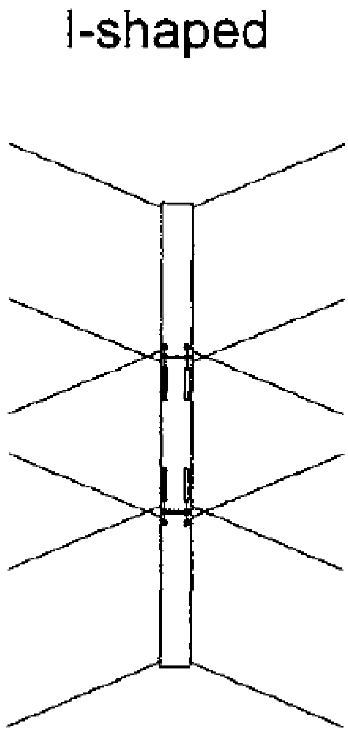


Figure 7-18: I-shaped layout

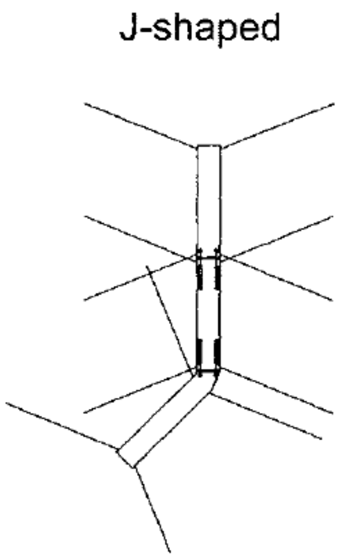


Figure 7-19: J-shaped layout

In order to get a rough estimation of the mooring force, the results of the paper P.Ruol et al (2008) is used. In the literature, the maximum mooring force is derived by the Australian Standards 3692. The floating breakwater should resist the wave load with a 1/100 characteristic value relative to the design storm. This is certainly an extreme condition, which is suited to be represented by experimental investigation: while being representative of the maximum, it has a much greater statistical reliability.

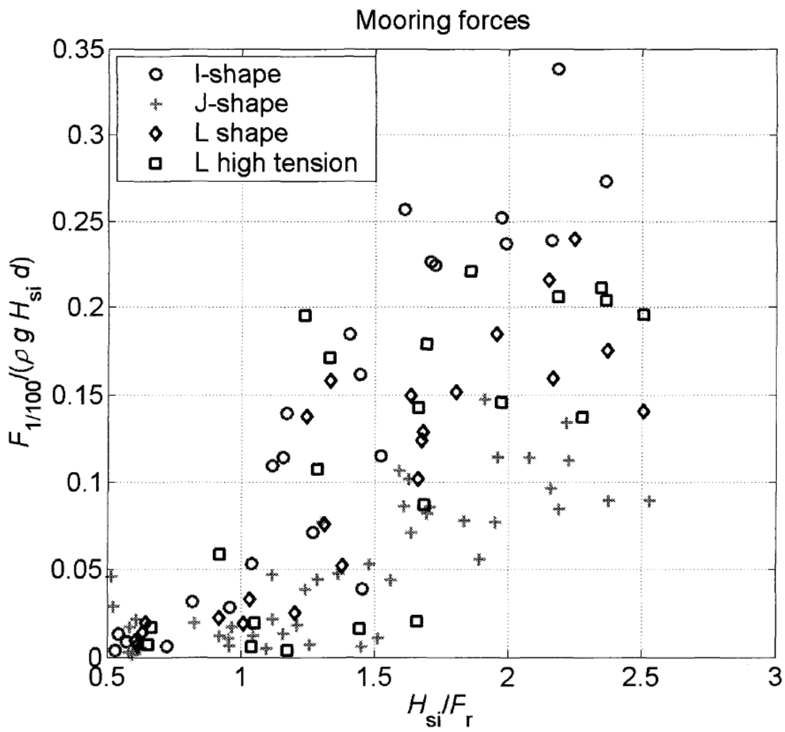


Figure 7-20: Maximum non-dimensional load on the moorings VS degree of overtopping from P.Ruol et al (2008)

The graph can be considered as an important design tool as it allows for the evaluation of the maximum load on the mooring required by standards. In practice, the design of the floating breakwater crest freeboard should roughly satisfy the condition: $H_{si}/Fr < 1.0$; but a safe mooring design should be based on an upper limit: $H_{si}/Fr \approx 1.5$.
For an I-shaped connection assuming $H_{si}/Fr \approx 1.5$: the average value of $F_{1/100}/\rho g H_{si} d$ should be around 0.1 and the extreme value is 0.17, where H_{si} is the incident wave height, d is the height of the structure, Fr is the freeboard.
So as a rough estimation, the result from the SWAN simulation was used. In 7.5 meter water depth the incident wave height H_{si} is 1.5 m, then Fr=1 m, d=4.5+1=5.5m, then:
$F_{1/100}=0.1*1000*9.81*1.5*5.5=0.81\times 10^4 N/m$

And in the extreme case:
$F_{1/100}=0.17*1000*9.81*1.5*5.5=1.38\times 10^4 N/m$

Similarly for J-shaped connection, $F_{1/100}/\rho g H_{si} d$ should be around 0.05.
$F_{1/100}=0.05*1000*9.81*1.5*5.5=0.49\times 10^4 N/m$

This is a rough estimation, and all the parameters are chosen for the most dangerous case. The value maybe is too safe and not beneficial from a economical point of view. More detailed modeling is needed to get an optimal value.

7.6 Conclusion and recommendations

In this chapter the general properties of the location and the governing boundary conditions have been described. The advantages and disadvantages of several types of breakwaters have been given. A preliminary design for a floating marina with floating breakwater is made. Also economical considerations regarding the construction of a floating breakwater have been made.
After considering all these aspects, it can be concluded that a floating marina with floating breakwater is technically feasible.
Also the following recommendations can be given:

- As a conclusion of the multi criteria evaluation and the economical considerations given in this chapter it is advised to use a floating breakwater which can be removed in winter. The main advantage is that in this way exposure to severe winter waves is prevented, so the initial costs for the construction of a removable breakwater will be considerably lower than a permanent floating breakwater. The initial costs for a floating breakwater will also be lower than for a conventional (submerged) breakwater.
- Removing the breakwater every winter will of course increase the maintenance costs. These can be reduced by allowing people to store their boat on the floating breakwater in winter while it is stored on Varna Lake. People have to pay a certain price to place their boat on the breakwater.
- It is recommended that the type of soil on the local bottom will be investigated, because during the fieldwork no equipment was available to take soil samples at great depths. The type and quality of the soil is relevant for the design of the anchoring system of the floating breakwater.
- It is suggested to further optimize the exact location of the marina. It might be located closer to the coast, by dredging the bottom near the coast until the design depth has been reached.
- To reduce construction costs to a minimum, it is recommended to use the services of the local company which constructs concrete pontoons.

8. Conclusions and recommendations

In the previous chapters the results of the field measurements that were carried out in Bulgaria in 2009 have been presented. The following conclusions (section 8.1) can be drawn and recommendations (section 8.2) can be given.

8.1 Conclusions

- The overall profile of the St. Constantine groyne has not changed much over the year; the groyne is thus relatively stable.
- The coastline of Sirius Beach is very dynamic. In front of the Sirius Beach hotel just behind the groyne an erosive trend is clearly visible: a retreating coastline is observed the past years. The largest changes in the beach profiles are observed here, which started after the construction of the groyne.
- The underwater profile of Sirius Beach is quite stable as the bed consists of mostly rocky stones. In the southern part of the beach the profiles increase in volume. The sand that is eroded at the northern part is partly moved to the southern part of the beach.
- Beach nourishment is not the ideal long term solution unless it is repeatedly regularly. Therefore the best way to deal with the erosion of the northern part of Sirius beach is to regularly redistribute the sand using small dredging equipment.
- The apparent erosion of Azalia beach is not a direct threat. Azalia beach follows the seasonal evolution and consequently, it will start to retreat after summer forming the winter profile with a shallow foreshore. After this, the summer profile will be formed again which results in a wider beach with a steeper foreshore.
- Both beaches have a quite narrow grading, most of the grains having a diameter between 0.6 and 2 mm. Also there is not much variation in longshore direction.
- The general properties of the location and the governing boundary conditions for the marina have been described. Also the advantages and disadvantages of several types of breakwaters have been given and a preliminary design for a floating marina with a floating breakwater is made. After considering all these aspects, it can be concluded that a floating marina with floating breakwater is technically feasible.

8.2 Recommendations

- Finding out the purpose and true functioning of the St. Constantine groyne together with the total littoral sediment transport is desirable. This will help decide whether the groyne should be maintained, improved or even removed.
- Erosion of Azalia Beach is not a direct problem, the sand is only relocated. However if the seasonal evolution is disturbed or the beach is not wide enough yet at the start of the touristic season, minor performing dredging activities can be considered to widen the beach.
- It is highly recommended to keep one starting point for the baselines, so that in the coming years the data can easily be compared with the data of 2009.
- It is advised to take sand samples at the same locations in the coming years.
- As a conclusion of the multi criteria evaluation and the economical considerations given in this chapter it is advised to use a floating breakwater which can be removed in winter.
- Removing the breakwater every winter will of course increase the maintenance costs. These can be reduced by allowing people to store their boat on the floating breakwater in winter while it is stored on Varna Lake.
- It is recommended to carry out soil investigations at the location of the new marina to determine the type of soil on the local sea bottom. The type and quality of the soil is relevant for the design of the anchoring system of the floating breakwater.
- It is suggested to further optimize the exact location of the marina. It might be located closer to the coast, by dredging the bottom near the coast until the design depth has been reached.
- To reduce construction costs (of floating marina) to a minimum, it is recommended to use the services of the local company which constructs concrete pontoons.

9. References

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Software

- Google Earth
- Surfer, version 6.01
- SWANOne

Appendix

A.1 SwanOne model for floating marina

The software SwanOne has been used to do an one-dimensional analysis of the wave conditions at the coastal location designated for the floating marina. The screenshots below show the show the input concerning the wave and wind conditions, the coast orientation and the bottom profile. It has been assumed that there is no current present at the modeled location.

The cross-section used to define the bottom profile has been derived from the echosounder data which has been measured during a boat trip at the location in question during the field trip. This has been done as follows. The echosounder data consists of longitudinal and latitudinal coordinates and water depth. Using this data as input for a software called Surfer a contour map has been made (see Chapter 7.2.2). With the command "slice" in Surfer a representative cross-section has been derived (figure A-1). This cross-section was then used as input for the bottom profile in SwanOne.

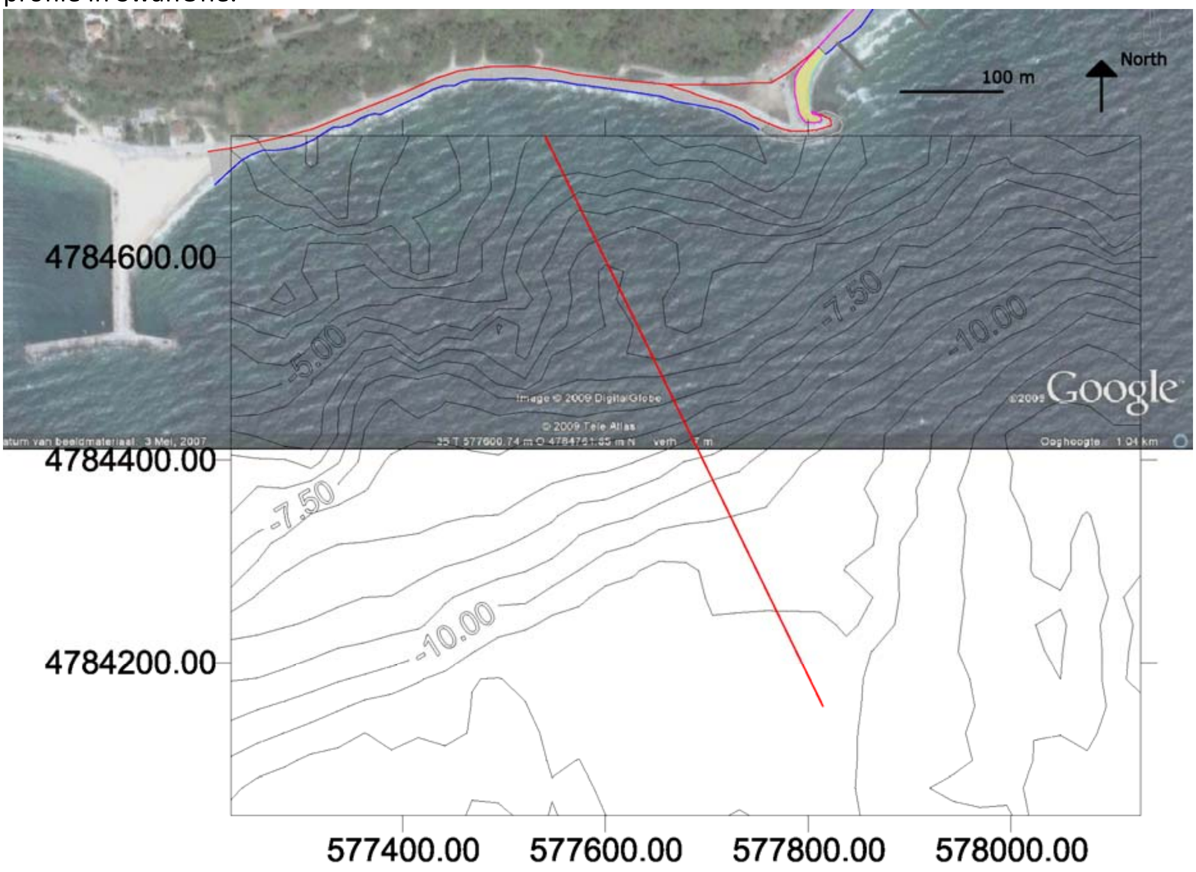

Figure A-1: Indication of cross-section used as Swan1D input

The following screen shots show the input of the wave parameters at deep sea into SwanOne:

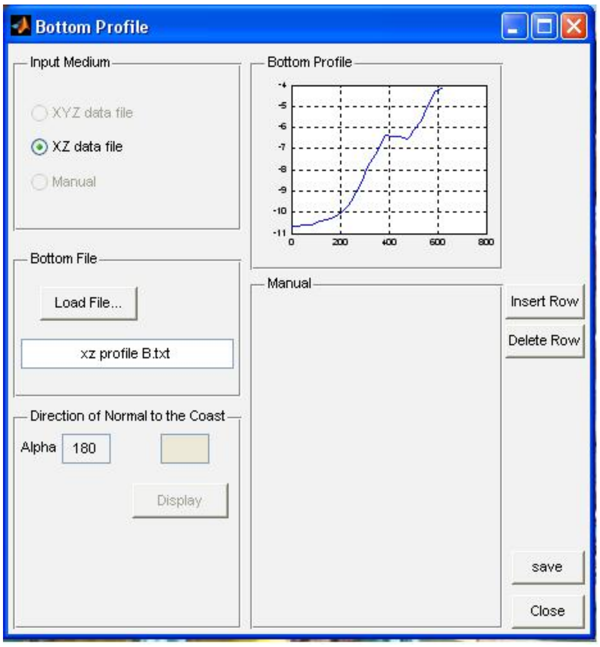

Figure A-2: Input SwanOne - Bottom Profile

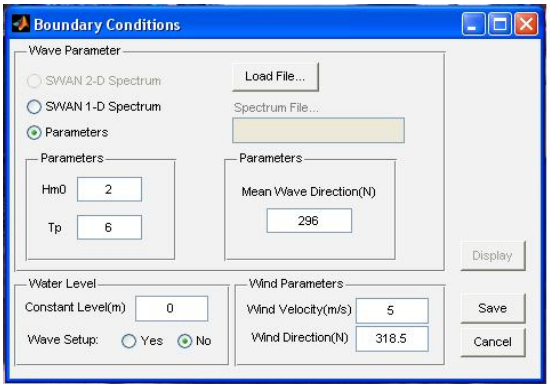

Figure A-3: Input SwanOne - Boundary Conditions

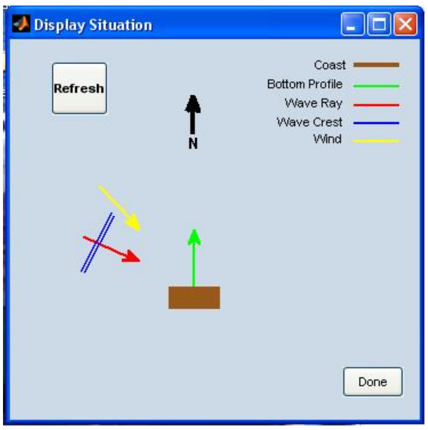

Figure A-4: Resulting Situation in SwanOne

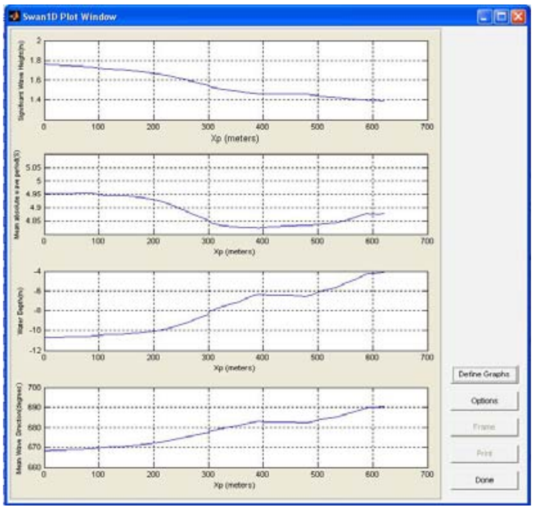

Figure A-5: Output of SwanOne - Ray Plots for significant wave height, average wave period, water depth and average wave direction

From the Ray Plots shown in figure A-5 all the needed parameters for the design of the marina, the floating breakwater and the mooring system can be extracted.