FIELDWORK 2017

CIE5318 Fieldwork Hydraulic Engineering

December 2017

PREFACE

As part of the Master program at the Delft University of Technology, an opportunity was provided to the students to take the course CIE5318 (Fieldwork Hydraulic Engineering). During the fieldwork we, as civil engineers, experienced the connection with the ideal theoretical study environment and the unpredictable practical real-life situation. The near winter conditions of the Black sea have shown us that measurements are not always as simple as they look. The famous saying: “measuring is knowledge” captures the essence of the trip, in which careless assumptions can lead to false precision or large uncertainty. The result of this process and cooperation is presented in this report.

Our appreciation goes out to H.J. Verhagen and M. Voorendt for organizing the trip to Varna and supporting the group during the fieldwork. We also would like to thank ir. Boyan Savov and his wife Tsanka Savov for their help, support and highly skilled tips. The information concerning the local characteristics and general knowledge of measurements have proven very valuable.

Thanks to the four architectural students of Free University Varna, who participated in the course, for their cooperation in the fieldwork measurement activities.

We would like to thank Svasek for their contribution to the fieldwork data collection. Their metrological and wave data provided us with the tools to validate the measured data.

Alejandra Ares
Daan Bader
Vassia Dagalaki
Koos Dijkhuis
Maria Georgiou
Xenofon Grigoris
Danny Janssen
Igor Koevoets
Vasio Kralli
Teni Maroudi
Lina Nikolaidou
Ioanna Saxoni
Thom Olsthoorn
Floris Sijbesma
Alexia Sotiriadou
Sebastiaan Woerlee

SUMMARY

This report is the result of the Fieldwork Hydraulic Engineering of 2017. The course has been performed for the past 15 years in St. Konstantine, a resort in the coastal area of Varna city, Bulgaria. The objective of this year's fieldwork can be summarised to the following: Plan and conduct a measuring campaign in order to collect sufficient data and assess last year's design solution for this coastal area. Within this report many references are made to previous years. A short summary is given, structured in accordance with the topics in this report (beach, soil sampling, bathymetry, marina, groyne and breakwater), incorporating data collection and feasibility analysis for the topics considered.

During the fieldwork, the elevations at different points at both the north and the south beach were measured, in order to determine those beaches' cross sections and the change in the volume of sand (accretion or erosion) compared with last year. The raw data was processed and the new contours, as well as the net result of the nearshore processes at those beaches was determined.

Soil samples were solely taken from the Southern Beach from the land onshore as well as from the bed offshore. A first estimation of the mean grain size is obtained by using a sand ruler. After transportation to Delft, the nominal diameter of the South beach sand, calcium content and other characteristics were determined in the laboratory.

Within this report the groyne structure situated norther of the north beach is investigated. The objective is to assess the current state of the structure with both visual and in situ methods. The data collected were used in comparison with the relevant data from previous years in order to recreate the evolution of the structure through the years and assess the deteriorating process initiated by the local weather conditions. Based on these results suggestions on the incorporation of the groyne within the general rehabilitation plan were made.

Wave conditions, bathymetric survey and inventory list of materials were calculated and checked in order to assess the Marina design in relation to the existing design that was presented in the last year's report. This was done by ensuring that the wave penetration inside marina can be considered minimum and safe navigation will exist. The phenomena of reflection and diffraction were assessed by using wave data from Svasek hydraulics. Concerning the bathymetric survey, the water depth inside the marina was measured with the help of an echo sounder with GPS in order to assess if the required depth for navigation was reached and possible dredging that may be needed. Last but not least, the available materials in the area were determined and can be reused for the construction of several structures inside marina.

Additionally, wave measurements corresponding to winter conditions were taken using different methods (visual observations, Svasek forecast, wave loggers). They were afterwards processed to derive the significant wave height which is used for the feasibility study and further checks to improve the design of the breakwater. Regarding the breakwater itself, its dimensions were measured and a report of its status was made (material inventory and current condition assessment). All the above provided the necessary data to perform the feasibility study on last year's design and based on its deficiencies to compute and propose an improved breakwater design that in in accordance with the client's wishes.

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1. INTRODUCTION

1.1. Background

The tourism sector in Varna, Bulgaria requires hotel owners to create and maintain beaches to strengthen their position in the market. The large seasonal changes in the wave climate of the Black Sea have proven that hard measures are required to maintain beaches along the coast line.

The coastal development area discussed in this report is situated in Varna, Bulgaria along the Black sea and depicted in Figure 1 The development area Saint Constantine consist of a marina st. Elias and beaches north and south of the marina, which belong to the owners of Grand hotel Varna. The marina and beaches have to be protected by groynes and breakwaters due to the sometimes severe storm conditions of the Black Sea. The original design for the marina and beaches was never fully completed due to financial difficulties. The state of the breakwater and groyne, protecting the marina and beaches, is deteriorating with time due to harsh storm condition and an unfinished structure.

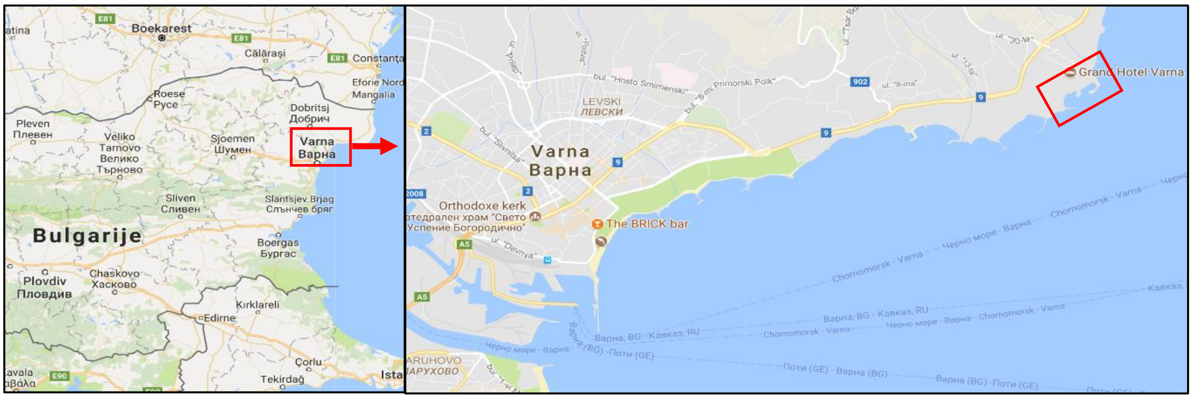


Figure 1: Location development area St. Konstantine

The development area has been measured in the course Hydraulic Engineering Fieldwork (CIE5318) in the study years of 2014 to 2017. Besides the measurements of the coastal characteristic, the previous course work also shows a new conceptual design for the marina and beaches in cooperation with architectural students of the Free University of Varna. This report is based on this conceptual design.

Figure 2: Current state (left) and original design (right) of marina st. Elias and beaches.

The current state and original design are presented on the left and rights in Figure 2, respectively. The picture clearly shows that the large breakwater is not completed and heavily damaged. The beach to north of the marina has a significantly narrower beach compared to the original design.

1.2. Problem definition

The hotel owner of Grand hotel Varna wants to use the beaches and marina of the development area to their full potential, which is now prevented by the uncompleted design and damages of the last years. The damages also show what kind of influence the Black Sea has on the current design state, which provides a valuable aspect for the future design. Given the conceptual design of 2016 Fieldwork course, the research question is:

What is the feasibility of the Fieldwork Bulgaria 2016 conceptual design and how is the necessary data acquired?

The sub question’s that will contribute to solving the problem are:

1. Which variables are necessary in order to assess the feasibility of the concept design?
2. How are the variables gathered using measurements?
3. How to organize the data collection and processing?
4. In what way can the measurement data be used to achieve a feasibility study of the design?
5. How is external data validated by measurements in the field?
6. Which design requirements are used in the summer and winter use cases?

1.3. Objective

The aim of this report is first, to research variables and characteristics of the marina and beaches of Grand hotel Varna using a measurement campaign and secondly to use these variables to perform a feasibility study on the design of the 2016 Hydraulic Engineering Fieldwork Report.

1.4. Reading guide

In Chapter 1, a concise summary of the conceptual design for the St Elias marina and the neighbouring beaches is presented, as proposed in the Fieldwork Report of 2016. Chapter 2, focusses on summarizing key elements in the previous design of the 2016 Hydraulic Engineering Fieldwork course, these elements form the basis for the feasibility study preformed in Chapter 6. The required variables for the assessment presented in Chapter 3, which acts as an inventory for the fieldwork. Chapter 4 elaborates on the method and precision used during the measurement campaign to obtain these variables. The results of the measurements is presented in Chapter 5. Chapter 6, contains the second part of the feasibility study, where the variables are used to assess the feasibility of the key elements in the concept design. Moreover, Chapter 7 continues with a preliminary design of the breakwater. Finally, the conclusion and recommendations are discussed in Chapter 8.

2. CONCEPT DESIGN 2016

In this chapter the basic elements of the design solutions proposed by the hydraulic engineering and architecture students in 2016 will be presented. The design requirements that stirred the students to those solutions will also be discussed.

2.1. Design requirements

In the coastal area of the St. Konstantin and Elena resort different areas of interest were distinguished, presented in the Figure 3. For each area of interest, the design requirements were identified, based on which the proposed solutions were structured. The design requirements and aims for each area of interest are presented below.

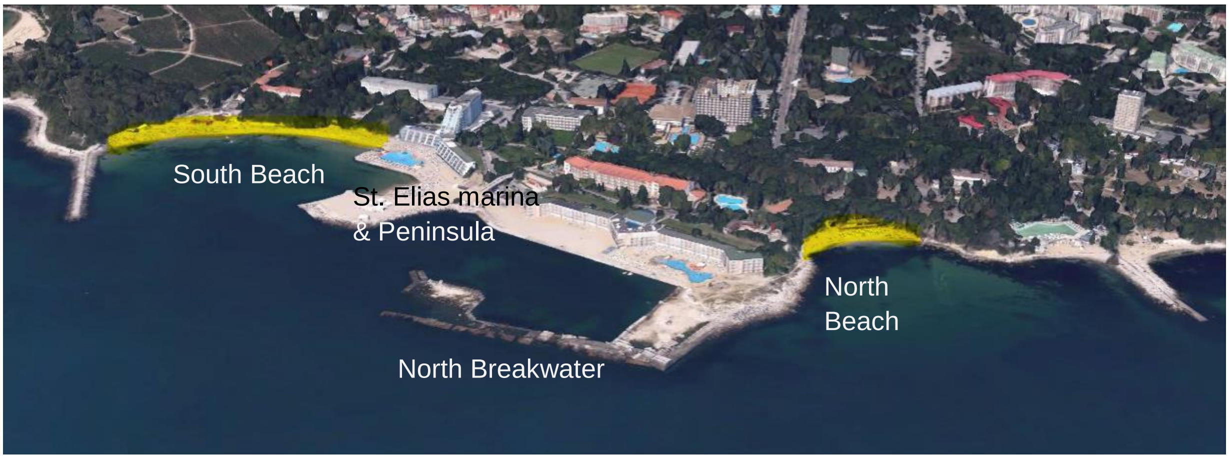


Figure 3: Areas of interest

In the North Beach the aim is to increase the beach width and create a stable profile not only for the summer period but also for the more variable spring and autumn seasons. Different options to this end reconstruction of the existing slope protection of the northern breakwater, natural accumulation and artificial nourishment were considered.

The marina is expected to operate only in the summer. The wave penetration through the marina entrance and wave reflection should be decreased while a slip (boat ramp, slipway) suitably positioned is considered as an essential element. Lastly, the design considers the removal and reuse of any available material in the area.

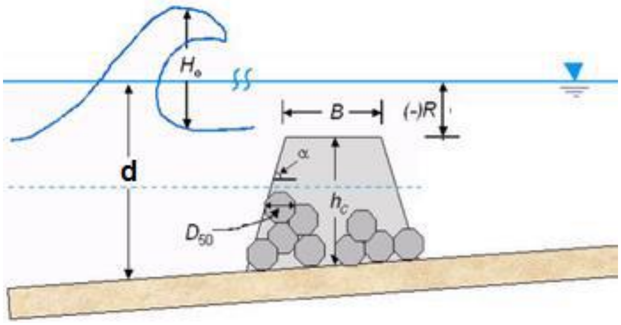
The breakwater should provide sufficient shielding for recreational shipping during the summer period. At the same time, it should be able to survive the winter storms protecting the marina facilities from damage. From a landscaping point of view the breakwater should be as low and wide as possible in order to achieve a maximum recreational potential.

As far as the South beach is concerned, the aim concentrates in a design that includes elements of recreational interest e.g. swimming pools. There is need to create more waterline but not necessarily a wide (and desert-like) beach. Additionally, the wave energy that is reflected from the quay wall next to the swimming pool and attacks the beach should be reduced.

2.2. Design aspects

A design plan was developed to address the aforementioned design requirements. The most essential parts of this design proposal are presented below.

In order to mitigate the sand loss during storm events in the North Beach the construction of a submerged breakwater (reef) was proposed. A statically stable rubble mound structure was chosen for this case positioned at the equilibrium closure depth of the beach profile (approximately 2.5m) spanning for approximately 120m at a distance of 100m from the waterline.



R=-1.5m
B=2.4m
D_{n50}=0.6m (560kg)

Table 1: Proposed dimensions for submerged breakwater in the North beach

The proposed layout for the marina can be seen in Figure 4 (left). The main elements of this solution are the following:

- A wide and low breakwater made of concrete slabs, reusing the slabs present on the breakwater. The full length of the outer side of the breakwater will be protected by riprap or tetrapods. The width of the breakwater is increased in order to facilitate recreational purposes and balance the height decrease;
- An additional groyne will restrict the entrance opening and block the sediment transport towards the marina basin (Figure 4-right);
- A ramp will be installed next to the new groyne;
- It was estimated that all the available material in the area of the breakwater will be reused in the construction of the new marina structures.

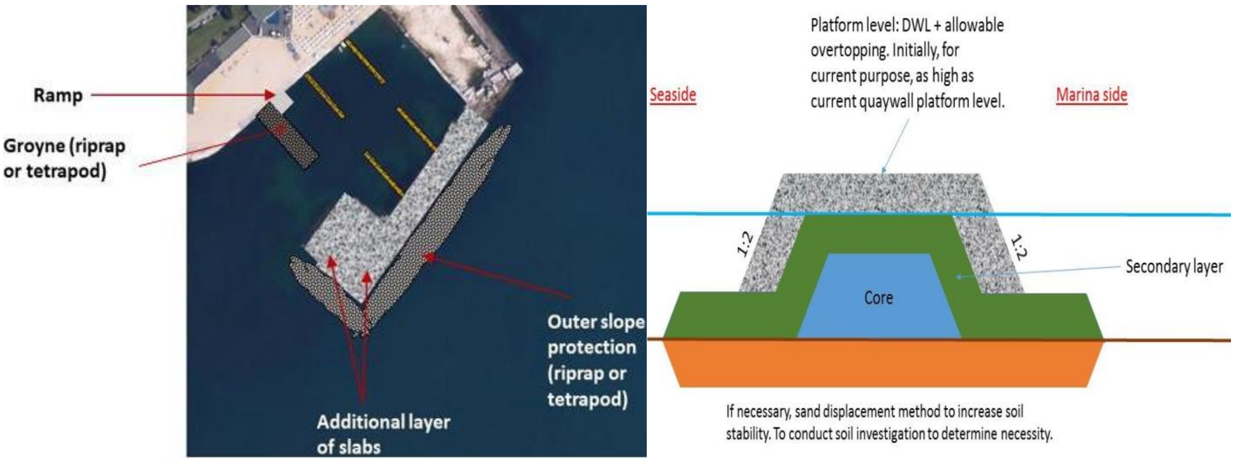


Figure 4: Sketch of the proposed layout for the marina (left), cross-section of the proposed groyne (right).

The area of the peninsula was completely redesigned to accommodate the abovementioned requirements (see Figure 5). It was proposed that the peninsula is removed and replaced by an artificial beach. The breakwater at the end of the peninsula is to remain and be extended in the southeast direction. This will reduce the amount of energy reflected from the quay wall that attacks the South beach and at the same time create the conditions for a salient in the shadow zone of the breakwater that will increase the waterline span.

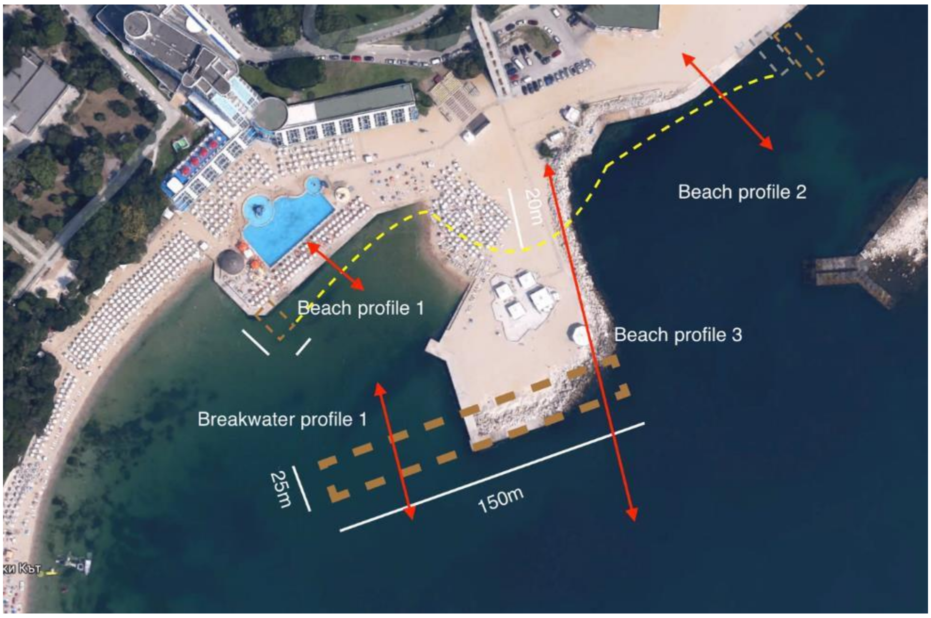


Figure 5: Overview of the design in the peninsula area

More specifically, the proposed design of the breakwater is presented in Figure 5. The total length of the breakwater will be doubled (in order to satisfy the condition for the creation of a salient) while the height will be decreased significantly, from 3-4m above MSL to 1m above MSL, so that it is partially submerged and does not pose a significant obstruction for the eye. Consequently, the salient will be affected by winter storms and thus will require maintenance every few years. The armour layer and core layer of this breakwater will be constructed using solely recycled material from the old breakwater and peninsula.

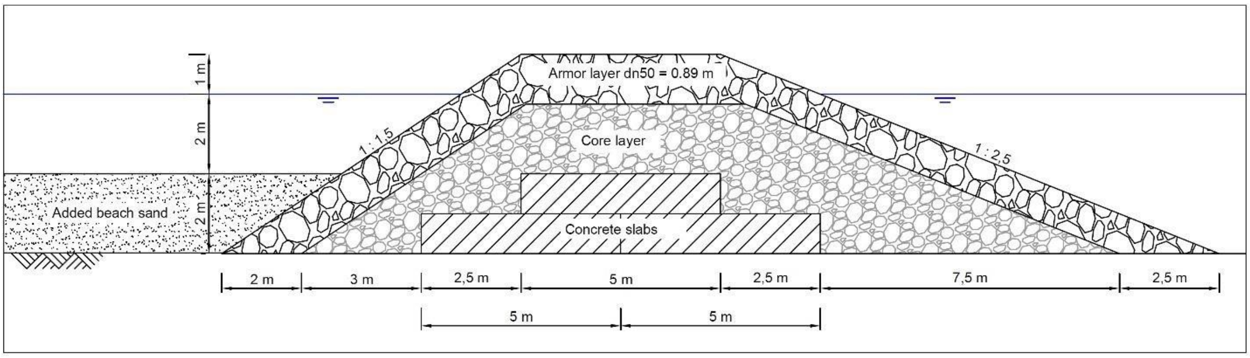


Figure 6: New peninsula breakwater

A salient is preferred over the creation of a tombolo for swimmer safety reasons. The length of the designed salient, reinforced by additional sand deposition, was defined 20 m and the total amount of sediment needed was calculated.

The profiles of the new coastline that will be created can be seen in Figure 7 and Figure 8. The new beach adjacent to the marina entrance will help reduce the energy that entered the basin having previously reflected on the existent quay wall. The total amount of sand needed was estimated using the equilibrium beach profile, 1200m^3 in total.

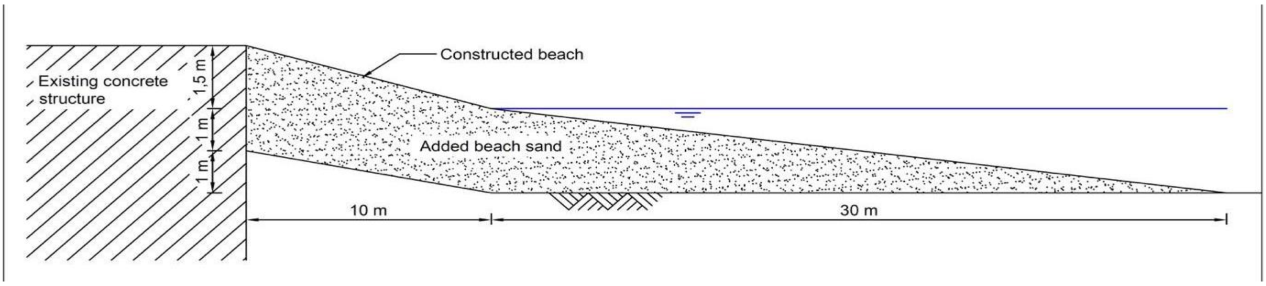


Figure 7: Beach profile 1

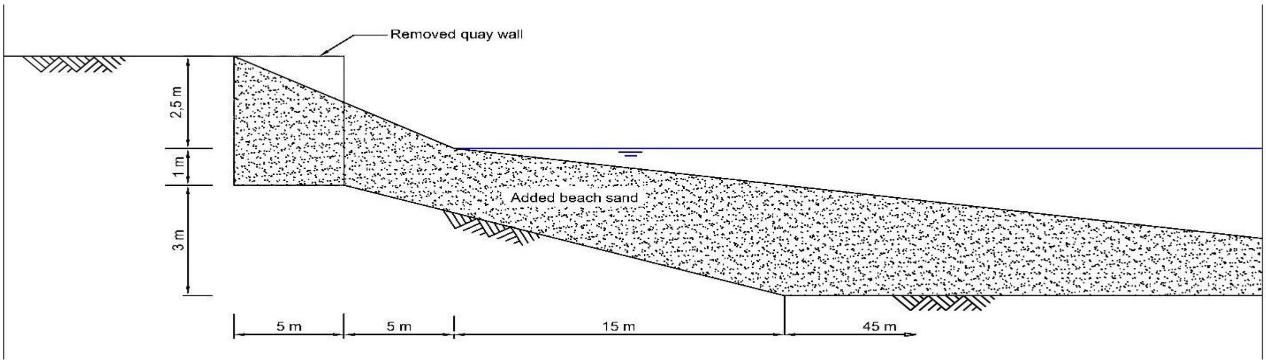


Figure 8: Beach profile 2 (inside the marina)

As can be seen in Figure 7 the southern part of the quay wall that spans along the length of the swimming pool will be retained behind the beach profile. Lastly, one additional groyne was proposed in order to prevent wash-off of the beach. It will be located at the southern tip of the quay wall in front of the pool. Sediment transport in the marina will be prevented by the groyne at the northern end of the new beach that was presented in Figure 4 (right).

Some additional/alternative aspects of the design presented in the architectural report of 2016 are presented in Figure 9 and summarised below:

- Landscape development in the zone connecting the marina and the North beach and creation of an infinity pool including a bar and locker rooms in front of hotel Albatross. Additional extension of the sandy part of the North beach until the pool which will act as a sediment barrier preventing sediment to leave the North beach towards the south;
- Construction of a building that will house the marina administration and custom offices as well as a restaurant with panoramic view. The building is situated at the northwest corner of the breakwater where the tetrapod armour layer is higher so as to be protected by the rough winter conditions;
- It is stated that the demolition of the peninsula is inadvisable as it has a different ownership status from the adjacent beach. In the alternative proposed design, the main peninsula structure is maintained. At the end, the existing breakwater is modified to take a slightly curved form with dimensions 25*150m. At both sides of the peninsula short pocket beaches are formed. A submerged breakwater (reef) protects the eastern beach from waves caused by ships, while an array of pilots with nets will keep pollution out of the swimming area. An extra T-shaped breakwater will keep the sediment of the west beach from being transported away by the current. The area above the peninsula will be reformed for recreational purposes while the construction of a cafe, public showers and changing rooms as well as a parking were proposed.

Figure 9: Architectural proposal

3. DATA INVENTORY

3.1. What kind of data?

In order to fulfil this fieldwork's requirements, the acquisition of site specific data is required. These data may be structure and/or aim specific and are characterised by their acquisition method and the intended/succeeded accuracy. With the aim of assessing the feasibility of 2016 design, the information acquired during the visit in Varna, Bulgaria, can be grouped in the following categories:

- Beach data: pertaining to waterline measurement, beach profiles, sand samples.
- Quarry data.
- Marina data: pertaining to wave, flow characteristics in the marina region, present structure characteristics.
- Bathymetry data.
- General data.

3.2. Data and accuracies

In the table below the types of data per topic are presented. For each type of data, the desired accuracy in terms of maximum allowable error and the acquisition method are indicated.

Type	Data	Desired accuracy	Acquisition method
Marina Data	Significant wave height	± 10 cm	wave logger, visual observation, Svasek information
	Significant wave period	± 1 sec	wave logger, visual observation, Svasek information
	Wave direction	± 22.5 degrees	Visual observations, Svasek information
	Wind speed	± 2 m/s	Svasek information, anemometer
	Wind direction	± 22.5 degrees	Visual observations, Svasek information
	Setup from MSL	± 5 cm	Wave logger
	Tetrapod/rock dimensions	± 3 cm	Measuring tape
	Rock blockiness	±5%	Visual observation
	Amount of available material for reuse	±100 tn	Visual estimation, Tape line, Google maps
Beach data	Waterline position	±0.5m	GPS device, Viewranger
	Beach height distribution	±0.5m	Levelling instrument, Range pole, Theodolite, Level staff, Measuring tape, Tape line, Tripod, Prism, Rope, Poles
	Soil characteristics	-	Visual observations, (Sampling for further insight)
	Sand diameter	±1mm	Visual observations, Sand ruler
Bathymetry data	Depth	±0.1m	Echosounder, measuring rope with attached weight

Quarry data	Nominal rock diameter	±0.1m	Tape line, measuring tape, visual observations, scales,
	Blockiness	±5%	Measuring tape, Visual observations
	Elongation	?	Measuring Tape
	Rock Density	±100 kg/m3	Sample processing, Visual estimation
	Rock quality	-	Visual estimation
	Stockpile quantity	±100 tn	Measuring tape, visual estimations Google maps, Viewranger
Groyne Data	Profile height distribution	±0.5m	Levelling instrument, Range pole, Hemisphere base, Measuring tape, Tape line, Tripod, Prism
	Damage assessment data	-	Visual observations, camera
General data	Weather conditions	-	Visual observations, windmeter, internet weather information
	Social/economic information	-	Visual observations, Local experts, online research

Table 2: Data inventory

3.3. Variables

The post measurement processing of data will yield the value to variables that will be used directly in the process of assessing the proposed design and its feasibility. These variables are presented in the table below

Type	**Variable**	**Value source**
Marina	Wave characteristics (height, direction, period)	Svasek information, SWAN model
	Diffraction coefficient	CRESS calculator
	Standard yacht dimensions	Internet research
	Dimensions of the basic elements of the marina	2016 report, 2017 design
Groyne	Slopes	Profile measurements
	Armour elements' dimensions (rocks, tetrapods)	In situ measurements
	Blockinsess factor	Visual estimation
	Bathymetry	Navionics maps
Breakwater	Elements of design proposal	2016, 2015 reports
	Wave characteristics (height, direction, period)	Svasek information, SWAN model
	Damage requirements (for armour layer, toe stability, overtopping)	Rock manual, EurOtop manual

Table 3: Variable invetory

4. MEASURING METHODS

In this chapter the methods followed in order to obtain each type of the required data (Table 2) are going to be described in detail. The aim of the following paragraphs extends beyond the mere recording of this specific measuring campaign. Additionally, it is attempted to create a handbook of detailed instructions for the future fieldwork students, discuss the unforeseen circumstances and their effect on the resulting accuracy.

4.1. Beach

For the two beaches that frame the marina, the North and South beach, a range of data were acquired in situ. In the following paragraphs the methodology followed to define the position of the waterline, the beach height distribution as well as the soil characteristics will be described.

4.1.1. Equipment

The availability and budget for the equipment was limited in Varna. Most of the equipment was already on site and in relative useable state. For the cross-shore beach profile measurements the following equipment was used: Levelling instrument, Jalon, tape, range pole, theodolite, tripod, prism.

The theodolite and levelling instruments were used to measure the height differences between each of the given points. These devices were mounted on a tripod in order to level out the instrument. The level staff was used in cooperation with these devices to read the height differences. The prisms were used to align each of the grid points and lines so that they are perpendicular to each other. In the following pictures (Figure 10, Figure 11, Figure 12) the above mentioned equipment is presented while being used on the field.

Figure 10: North beach – measuring height differences with levelling instrument and level staff

Figure 11: North beach – determining the measuring points with a tapeline

Figure 12: south beach - Levelling the theodolite by adjusting the tripod legs and using the bulls-eye level

4.1.2. Dolphin new level (DNL)

Previous years have used a corner on the ground level of the Dolphin Marina Hotel as a reference point (Figure 13). Even though, the Fieldwork of 2017 does not consider this a wise and sustainable reference point, this point was used nonetheless. This choice was made due to comparability considerations. For more information regarding the method that is applied to require this location, see the report of Fieldwork 2015.

Figure 13: Dolphin new level reference point

4.1.3. South beach

The method for the measurements on the south beach cross profile is described as follows.

Primarily the three reference points on the south beach must be defined. The reference points are indicated with green graffiti paint (Figure16, Figure17, Figure 18). These points must be marked with a jalon at each reference point S1, S2 and S3, which will be elaborated in the following section. An imaginary baseline from the jalons, standing at reference points S1/S2/S3, must be drawn to construct a grid. The base line can be interpreted as an imaginary line that has the shortest distance between the reference points. The baseline does not necessarily follow the coastline. The measured cross sections are perpendicular to the baseline with distances from each other as shown in the figure bellow. The choice of these cross sections (C0 to C7) is made to make a comparison with the results from the previous year, as these were the cross sections they measured. This year, two additional ones were added in the points S1 and S3, called C-1 and C8, respectively.

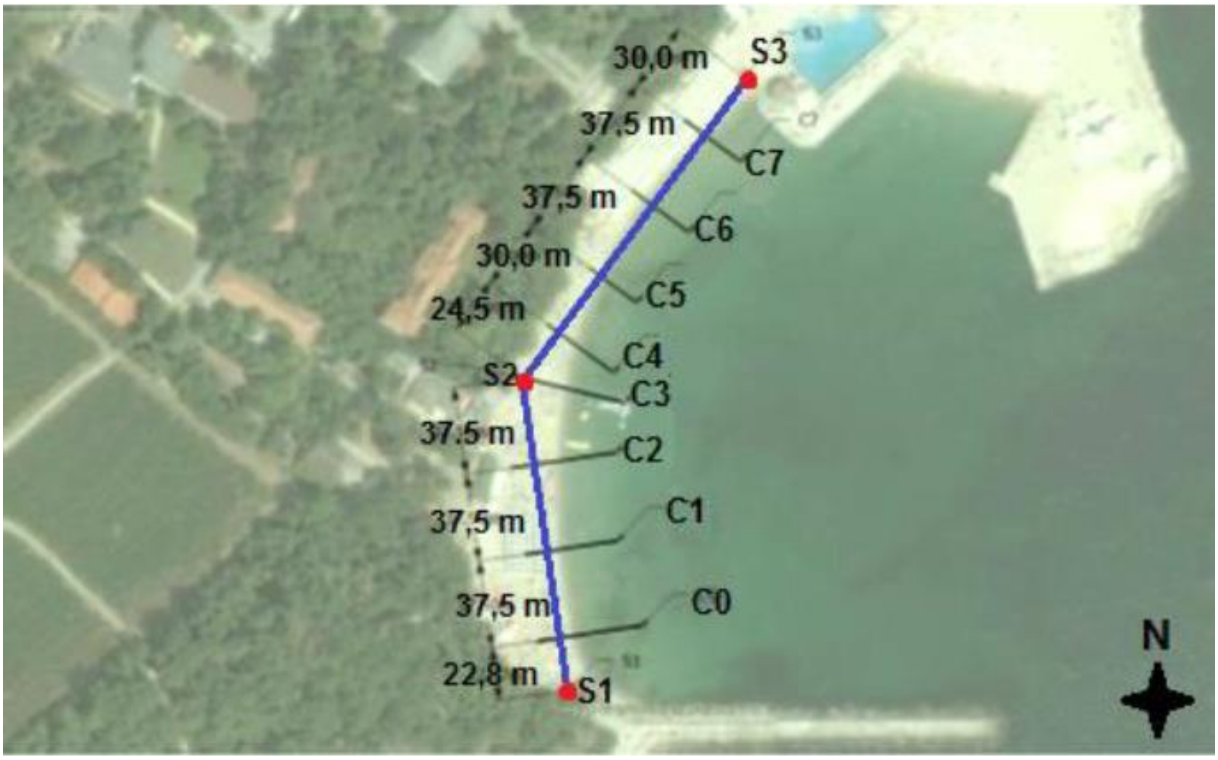


Figure14: South beach baseline

The position of the theodolite can be chosen arbitrarily, based upon own judgement. Position the theodolite, level it out so that it is horizontal to the ground. The theodolite is used as a levelling instrument. The cross of the levelling instrument must meet the horizon. The tape line and prisms are used to move along the baseline from S3 to S2 to S1. The position of the theodolite must be stationary during the whole experiment. A few points, along each cross section, are taken. The cross section is the line that is perpendicular to the base line. A 5-meter interval between each point is taken, towards the sea and land inwards, starting from the intersection point between the baseline and the cross section. The height is measured at each interval using the level staff or jalon. It must be noted that enough points must be taken sea inwards to overlap with the bathymetry measurements. In each cross section, measurements were taken in onshore and offshore direction, as far as it was possible. However, in 2016 they did not take as much measurements as this year, so a comparison cannot be made with furthest away points from the baseline, but it will serve for a good comparison with the following year measurements.

4.1.3.1. Baseline and Reference Points

For the Southern beach, the points used to set up the baseline are shown in Figure15: Reference points. As described above, three points were defined, the first at the most southern point of the beach, the second on the large retaining wall in the middle of the beach and the third at the most northern part of the beach on a small retaining wall.

Figure15: Reference points

Figure16: Reference point S1

Figure17: Reference point S2

Figure 18: Reference point S3

4.1.3.2. Reference Point for the heights and MSL

The reference points determined for the south beach are not yet compared to the Mean Sea Level (MSL). To compare the beach profile and the heights of the grid and reference points, the MSL must be determined. The reference points for determining the heights have therefore been chosen differently. Three points are needed to determine the heights of the reference points compared to the MSL.

Primarily a central place is chosen for the theodolite or levelling instrument. One of the reference points is chosen to be reference point S2 of the south beach reference points. The second reference point is chosen to be the Dolphin New Level Point. The final point is chosen to be just inside of the peninsula across from the point the theodolite is situated on. This point has been chosen due to its relative calm water level circumstances. This is desired, because the MSL needs to be determined and large waves compromise the accuracy of the measurements.

The first measurement is to determine the height of the theodolite, measured with the level staff. Consequently, the height of the reference points are measured, after which the MSL is estimated.

All the heights were measured with respect with the mean sea level (MSL). However, in order to make a comparison with last year results, these measurements have to be referred to the Dolphin New Level (DNL), so the height of reference point S3 was measured, because the relation between this height and the DNL is known (S3 height =-1,3m with respect to DNL).

4.1.4. North beach

To measure to cross sections, all the basepoints were marked with sticks and jalons, thus, to have the baseline marked. The levelling instrument was set up at a position from where most of the points of the beach were visible. Subsequently, it was calibrated with respect to the height reference point at the edge of the stairs of the restaurant, which can be seen at Figure 19. The levelling instrument was placed 0.025m above the reference point and 2.93 above MSL, Figure 19Figure 20. The reference point is located 0.25m above the Dolphin New Level (DNL) (Figure 21).

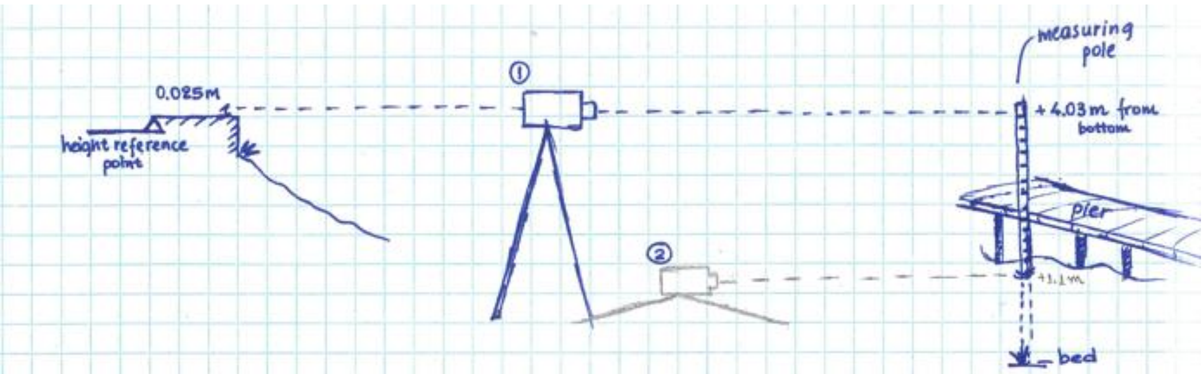


Figure 19: Mean sea level compared with reference level

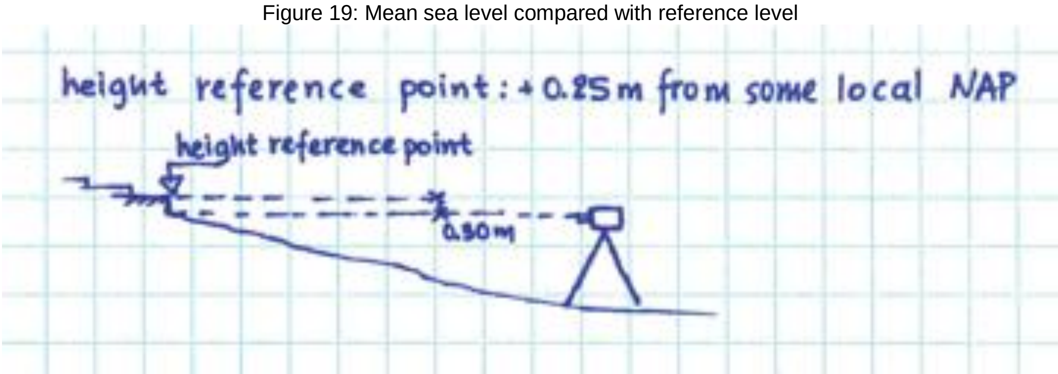


Figure 20: height reference point and dnl

A GPS application from a smartphone was used to determine the coordinates of the two basepoints at the two edges of the beach, as well as of the reference point, with an accuracy of 1-2 m. The results are summarized in the table below:

Point	X	Y
R1	43.227600	28.013395
R4	43.228339	28.013371
Height reference point	43.227772	28.013133

Subsequently the heights of several points on the beach were measured. For that purpose, 7 cross sections, perpendicular to the baseline, at intervals of 10 m were taken. On each cross section the height of the profile was measured every 2 metres perpendicular to the baseline, reaching maximum distances of 10m land inwards and 12m seawards. For some cross sections fewer points were measured, since some points had to be ignored, because they were located too deep in the water. To ensure that the cross sections were perpendicular to the baseline, the prism was used. The distances landwards are positive, whereas the distances seawards from the baseline are negative. The grid of the measurement points is shown in the sketch below:

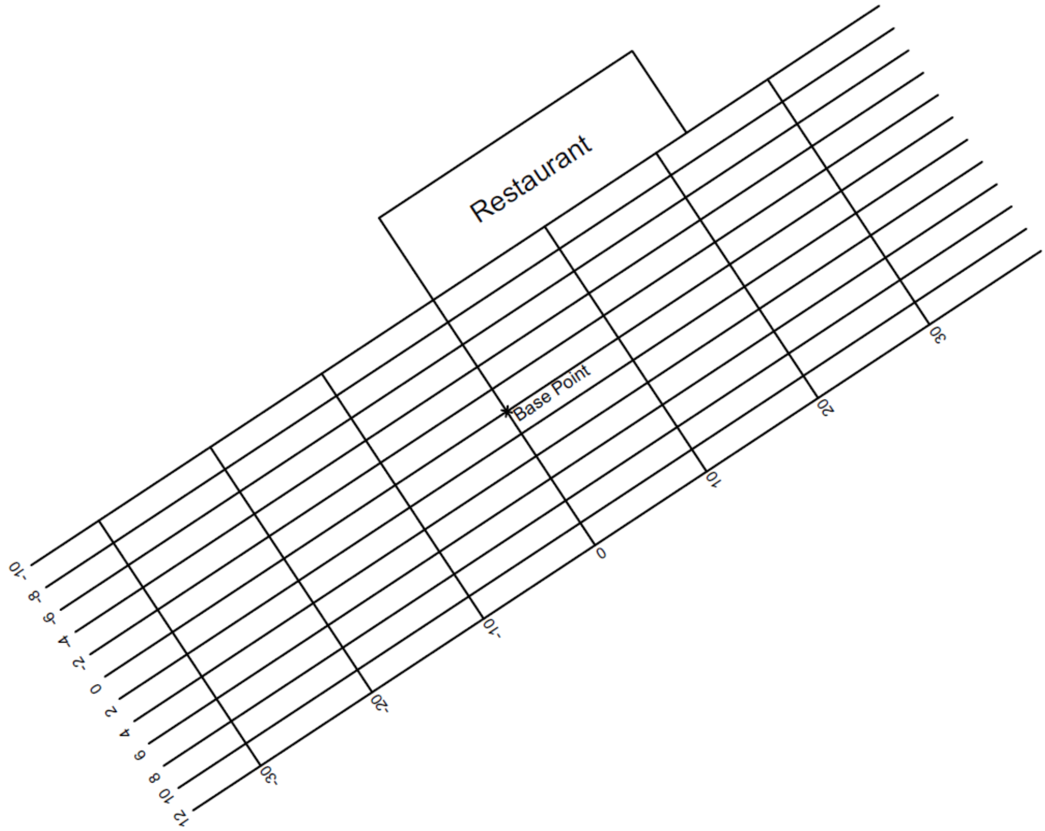


Figure 21: Grid of measurement Points at north beach

4.1.4.1. Baseline and Reference Points

For the Northern beach, the points used to set up the baseline are shown in the sketch below. For that purpose, 4 points were defined, the first at the side of the pier, second at the most south edge of the small retaining wall and the other two at the edge of the restaurant. It is important to note here that the baseline of 2017 is located 10 m from the stairs of the restaurant to the direction of the waterline, whereas the baseline of 2016 was located at 11.7m.

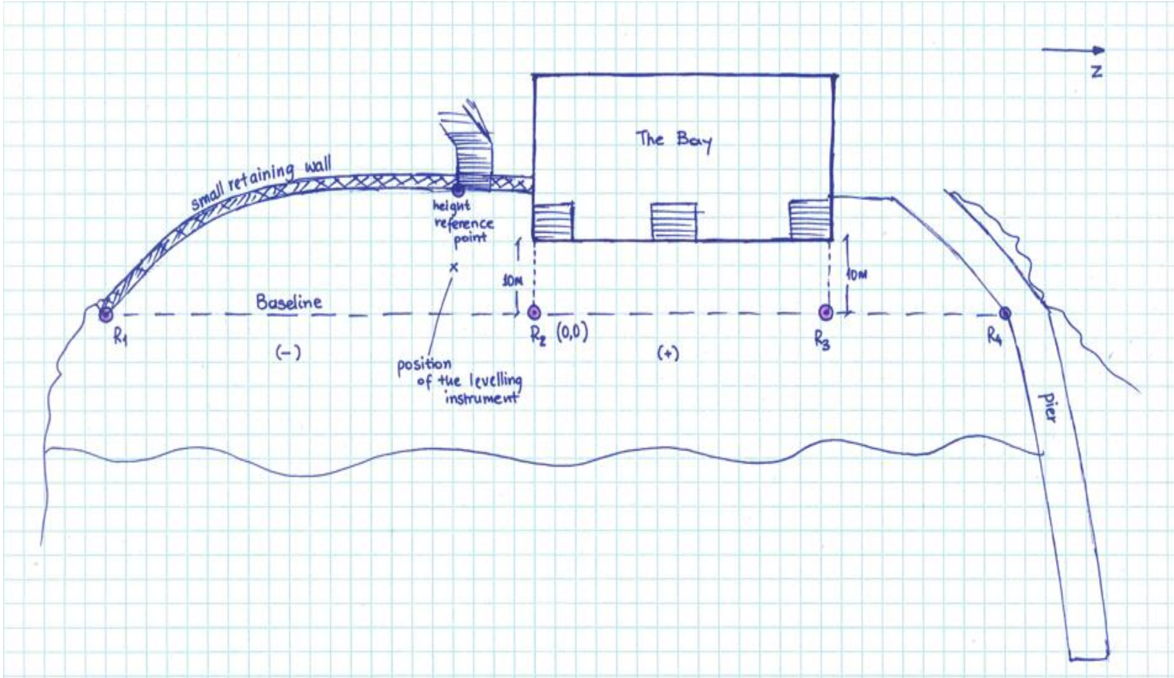


Figure 22: Baseline and height reference point

4.1.5. Shoreline measurements

For shoreline measurements and analysis two different measurement techniques were conducted. During the fieldwork measurements using GPS devices were done. Afterwards, for comparison reasons, an analysis of satellite imagery was also done. Both methods are described.

4.1.5.1. GPS devices

Primarily the use of GPS (Global Positioning System) devices is used to determine the waterlines. This is done by walking along shore with the devices, which would then measure their position on earth, by measuring the distance to a group of satellites. The devices record their position in time and store these in their memory.

4.1.5.2. Data collection

The shoreline was measured on 29 September 2017, using multiple devices. One measurement was done with the GPS device, GARMIN GPS 76. To verify measurements and accuracy, 3 measurements were done by using the GPS app, View Ranger. Different settings were used to compare the accuracy. The most important setting was the minimum tracking distance, for which 10, 15,20 m were used. The unit type was set in meters.

The waterline measuring procedure went as follows:

- The students started from one end of the beaches
- After which, the students walked in a single file along the average run-up line of the shoreline, with one student following.
- The structures located along the route were avoided by pausing the recording of the measurements, when a structure was encountered.
- At the end of each shoreline section, students turned off the devices.

After the measurements were completed, the data collected with the smartphones, was first emailed and then added to the common Google Drive. The data collected from the Garmin GPS76 was retrieved later when access to a computer was available.

Figure 23:Garmin GPS76 (left) ViewRanger app (right)

As stated before, along the route of the shoreline multiple large structures were encountered making it impossible to carry on the track. This resulted in multiple tracks. After these measurements, the tracks should naturally be processed to combine the results of the different devices and tracks. The full route and recording has been divided into seven tracks. They have been depicted as follows in the map (Figure 24).

Figure 24: Track overview

The processed tracks are provided as .gpx as well as .kml files.

4.1.5.3. Google satellite images

Historical images of google earth could be used to find the shoreline position according to the suitable years/dates available for each beach, and then compare the shoreline at different dates. The procedure starts with determining two control/reference points for each beach in positions that can be considered unchanged and present over all the years examined. This has to be done because Google Satellite images are not precisely georeferenced from year to year. Afterwards, selecting the year in historical imagery we create new path and draw a line over the shoreline observed and we repeat it for other years/dates. Using QGIS, a free and open source Geographic Information System, we convert the lines drawn in google earth into shapefiles and we move each shoreline to match the two control points using as a background the latest image of each beach imported from Google Satellite. It is important to convert the WGS84 coordinate system in a metric on which for the zone of Bulgaria is the UTM 35N, so that the files created can be used in AutoCAD. Finally, we save it as DXF file and we import it to AutoCAD to manage the shorelines and present an efficient comparison between them.

4.2. Soil sampling

The soil samples were solely taken from the Southern Beach. One of the main consideration for taking the samples solely from the South Beach, is that a sufficient yet small amount could be carried back to the laboratory in Delft. The samples were both taken from the land onshore as from the bed offshore. Before transportation to the TU Delft, a first estimation of the mean grain size is obtained by using a sand ruler. The locations where chosen so they would be representative enough but at the same time the number of points had to be restrained so the samples could be transported to TU Delft for further analysis.

After transportation to Delft, the samples were primarily, weighed and oven-dried. Consequently, the calcium content is determined, after which a sieve analysis is conducted. This process is executed in the following order:

- Name samples
- Weigh samples
- Oven dry samples
- Dissolve calcium content
- Oven dry samples
- Weigh samples
- Sieving
- Determine weight fractions.

A summary has been made in the paragraphs below on the extraction of the soil samples and their locations, determining the calcium content, the execution of the sieve analysis and the determination of the mean grain size.

4.2.1. Location and depth of the soil samples

The location of the soil samples should be carefully chosen. A fair consideration of both the vertical as the horizontal planes must be made. The top layer of the soil could be contaminated with organics or waste. Samples should therefore be taken from a good distance from the surface.

4.2.2. Extraction of soil samples

When extracting the different and multiple soil samples, it should be considered from where these samples are taken, onshore or offshore. Two different techniques are described, firstly for onshore samples and secondly for offshore samples.

The onshore samples are obtained using a tubular soil sampler (Figure 13, Figure 14). The sampler works based on vacuum, directing the soil inside the tube, comparable to a piston. By pushing the sampler straight down it will ensure the sediment below the surface to be trapped in the lower part of the tube. The sample must be obtained in wet circumstances otherwise the vacuum will not work. The samples are taken as close to the waterline as possible due to the severe wave conditions.

Figure 25: Tubular soil sampler (right), sampling moment (left)

Samples from offshore needed a more intensive effort to be extracted. Besides the ponar grab (Figure 26) also a boat is required. The boat is used to travel offshore to the desired sampling locations. The ponar grab, a heavy grabbing device, is operated from the boat, letting it fall from above water level until it reaches the bed, from which it grabs an amount of soil. It must be noted, that it is difficult to determine the sample depth.

Figure 26: Ponar grab

4.2.3. First estimation of grain size using sand ruler

In every sampling location pictures were taken as it is shown in Figure 28. The reason was to define the soil layering, the sample condition and the length of the sample. The sand ruler was used to get a first indication of the diameter of the sand particles at the top and at the bottom level of the sample (Figure 29). A representative part of the sample to be tested with the sand ruler is rubbed dry with the fingers in the palm of the hand. The sample is then placed in the hollow area in the centre of the ruler. The average grain size is now judged by comparing the average grain size of the sample with the specimen in the ruler. The sand rulers contain different sizes of sand in different air pockets that can be compared to the sample. The sizes in the sand ruler used range from 2mm to 0.063mm. This measuring technique however is not considered accurate enough to determine the actual mean grain size as it is only a visual interpretation of a few grains.

Figure 27: Procedure of extracting the soil of the tubular soil sampler

Figure 28: Sample taken with tubular soil sampler

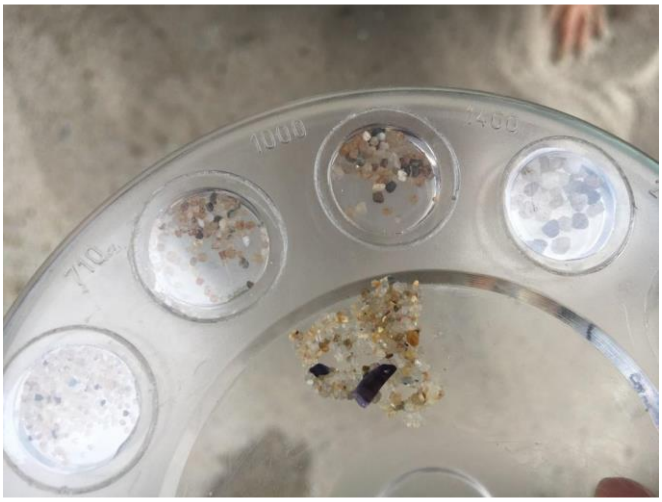


Figure 29: Sand ruler

4.2.4. Packing and transport of soil samples

The samples extracted from the beach and the bed, had to be transported back to TU Delft for further analysis. To minimize the weight and size of the samples, the samples were kept to a limited amount. The preferred amount of soil is put in a sealable bag, eg. a zip lock bag. It should be made sure the bags are airtight and waterproof and must above all not be vulnerable to breaking whilst being transported. The samples and therefore bags must be carefully marked.

4.2.5. Preparing soil samples for sieve analysis

When arriving back in Delft, the soil samples should be examined and analysed further and as quick as possible. The soil samples must be taken from their bags in portions from approximately 100 grams and put into small ceramic bowls. These should also be carefully marked and stored before drying.

4.2.6. Sieve Analysis

4.2.6.1. Drying of the soil samples

The samples are put in heat resistant ceramic bowls of two different sizes as shown in Figure 30 that can contain approximately 80g and 90g respectively. Consequently, they are put in the oven and in which they are dried for 24 hours at a temperature of 105 degrees Celsius.

Figure 30: Samples after being dried for 24 hours

4.2.6.2. Weighing of the samples

The samples are extracted from the oven and are weighed on a scale as shown in Figure 31. Accuracy of the scale used can be considered around 0.01g, with an error of 0.005g.

Figure 31: Scale used to weigh the samples

4.2.6.3. Extracting the calcium content

After weighing and drying, the calcium content (shells and shell particles) should be extracted from the samples, otherwise this could lead to errors and “pollute” the grainsize distribution. The soil samples are removed from the previous bowls and are carefully put into glass containers, keeping in mind that all the volume of the sand had to be transferred. Then a solution of HCl is added, which reacts with the calcium and extracts it in the form of CO2. Demineralized water should then be used to clean the samples and should be left for an adequate amount of time to make sure that the acid is washed out. Using pipettes the water should be extracted from the containers, very carefully so that sand does not flow out as well.

Figure 32: Waiting for solution to react with calcium

4.2.6.4. Drying the samples

Due to the addition of water, the samples must be dried again, at the same settings as the previous drying phase.

4.2.6.5. Weighting dry samples without calcium content

After the second drying, the samples should be weighed again, from which the original weight of the sand can be determined, without the shell content.

4.2.6.6. Sieving

To assess the particle size distribution, a sieving should be carried out. The sieves that were available in the lab and were considered appropriate to form a representative sieve curve have the following dimensions of openings:

- 3.4 mm
- 2 mm
- 1 mm
- 0.6 mm
- 0.425 mm
- 0.212 mm
- 0.125 mm
- 0.09 mm

Each sieve is weighted to obtain its net weight. The sieves are put in that order on the sieve machine, each sample is placed in the top sieve and the machine starts vibrating for a minimum duration of 5 minutes with an intensity higher than 7 so that the soil will be sieved accurately and efficiently (Figure 33). After that, each sieve containing sand particles is weighted. Using the weight of the individual grain fractions, the sieve curves can be constructed.

Figure 33: Sieving machine

4.3. Bathymetry

During the fieldwork, the bathymetry is measured from Boyan’s boat with an echo sounder with GPS. The result of the offshore bathymetry can be combined with the beach profile measurements and the waterline measurements to get a good view of the morphology of the beach.

The bathymetry survey consists of four parts, the preparation, sailing, storage and visualizing. These phases are shortly described below.

Preparation

1. After placing the boat in the water and installing the echo sounder, the distance between the echo sounder and the still water level has to be determined.
2. Before sailing, the track has to be recorded, this can be done with the option ‘save track’ at the echo sounder.

Sailing

3. The bathymetry survey can be done through sailing a track of free choice, to cover the whole area it is easy to sail a zigzag pattern. It is advisable to sail first in the area near the breakwater and the northern beach if this is possible. Sailing in these areas is hard with some small waves.
4. Check during sailing if the track is recorded.
5. Different tracks can be sailed and stored separately.

Storage

6. The data has to be saved as a .gpx file, this file will contain the coordinates, water temperature and the local depth. Of course, only the water depth and coordinates are relevant for the bathymetry survey.

Visualizing bathymetry

7. The .gpx files from the bathymetry survey and the waterline survey have to be converted to coordinates and depth values only. The result of the beach profile measurements has also been represented as coordinates with height with respect to the water level.
8. The data can be analysed with several software. The data in this report is analysed with MATLAB.

4.4. Groyne

The variable inventory showed that a geometry and damage assessment needs to done in order to validate the hydraulic effects on the current design. During the fieldwork in 2017 both the geometry measurements and a visual inspection are performed, as is specified in the paragraphs below.

4.4.1. Height/slope measurement

For the height measurement the levelling instrument, a theodolite and probe are used. The hemisphere tool is used to make sure the range pole will not penetrate in between the rocks.

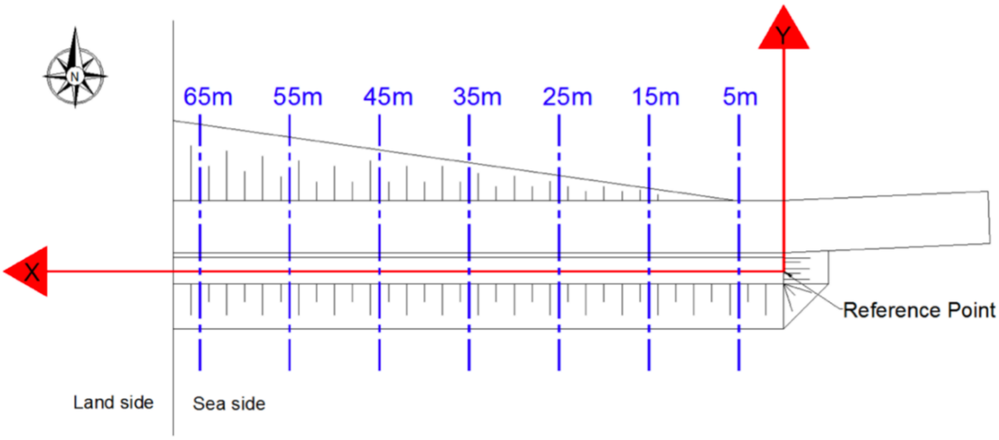


Figure 34 Topview of Groyne(source: Fieldwork Report 2013)

1. The reference point as described in the report of 2015 was retrieved in order to measure the same cross sections as the previous years and enable the comparison, as is depicted in Figure 34. The reference point is made at the following coordinates: 43°13'46.6" N 28°00'55.7" E.
2. Measure the height at the reference point.
3. From the reference point, the x direction is taken perpendicular to the beach.
4. From the reference point, the y direction is taken parallel to the beach.
5. The groyne is subdivided into sections, see Figure 34, first 5m from the reference point in x direction, then 15m, 25m, 35m, 45m , 55m, 65m from the reference point.
6. At every x location(5,15,25,35,45,55,65m) measurements are performed in both –Y and +Y direction. Taking measurement in both Y directions: Y=-3, Y=-2, Y=-1m, Y=0m, then Y=1m, Y=2m, Y=3m and Y=-1, Y=-2m, Y=3m etc. Note: in some cases it is impossible to go into the sea due to safety issues, please perform these measurements with caution. Perform the measurements until a point that can be reached safely

In the following steps an example is given on how to perform the measurements that should be performed to measure the slope, y direction measurements (e.g. 35m). The steps below can be repeated for every x location. The measurements are performed with the levelling instrument, range pole and relative height to the reference point.

7. Start at Y=0
8. Measure height at Y=0
9. Measure height at Y=1m
10. Measure height at Y=2m
11. Go further until the measurements cannot be performed safely anymore
12. Also do these measurement in –Y direction
13. Repeat steps at different x location

4.4.2. Damage Assessment

Following the quantitative measurements that served to record the profile evolution of the groyne, a qualitative investigation was carried out with the aim of visually estimating the state of damage on the groyne at present and compare it with the information of previous years. For that reason, photos were taken on 7 points on the breakwater at defined distances from the reference point facing North and South. These points were selected to be the same as those used in the fieldwork of 2015 so that it is possible to compare the pictures and assess the damage of the last years.

The groyne was subdivided into the same subsections as in the previous paragraph. Now at each of these subsections pictures are taken in the following way:

1. Start at reference point, coordinates: 43°13'46.6" N 28°00'55.7" E
2. Go to X=5m
3. Take a picture in +Y direction at the specific X location
4. Take a picture in –Y direction at the specific X location
5. Go to X=15m and repeat step 3 and 4
6. Go to X=25m and repeat step 3 and 4
7. Go to X=35m and repeat step 3 and 4
8. Go to X=45m and repeat step 3 and 4
9. Go to X=55m and repeat step 3 and 4
10. Go to X=65m and repeat step 3 and 4

4.5. Marina

A marina needs multiple types of data in order to make a complete assessment. This originates from the many elements that a marina contains, such as a required depth and maximal wave height inside the marina, In order to achieve these requirements, structures such as breakwater are needed, which is dependent on the wave climate in the Black Sea. Wave data were deemed the most necessary for the assessment of the marina design. In the following paragraphs the measurements that were performed to acquire this kind of data are discussed.

Wave measurements can be divided into many different methods. During the fieldwork, we used several of them. First of all, we did visual wave measurements to get an idea of the wave characteristics. Secondly, a wave logger was used to retrieve the wave characteristics from the pressure. Ultimately, offshore wave data from a measurement of Svašek Hydraulics is used to retrieve onshore data by making use of SWAN. All three methods are discussed in this chapter.

4.5.1.1. Visual wave measurements

These measurements have been performed at two locations by making use of different techniques. The detached concrete jetty north of the north beach was used as measurement point, since over here waves could be easily compared with the total height of the jetty. The measurements were performed at two locations. The first location is situated just south of the north beach, on the highest point of the part of the coastline that is protected by concrete tetrapods. Since this location is situated 460m away from the jetty, visual wave observations have been done using a theodolite as binoculars. The second location is situated at the groyne just north of the north beach. The distance to the measurement point is 187m, so the theodolite was not used as binoculars. Instead, measurements are done with the naked eye. In Figure 35, the measurement points are visualised, where Location 1 corresponds to the location at the tetrapods and Location 2 to the location at the groyne.

Figure 35: Locations for visual wave measurements at jetty

At Location 1, six series of measurements were performed. The first series consisted of a sequence of 106 consecutive waves, whereas the other five series consisted of a sequence of thirty consecutive waves each. Zero-up crossings were used, meaning that the wave height is defined as the range between a crest and the succeeding trough. The total duration of the measurements is equal to 29 minutes, which is performed within an hour. Since the tide at the Black Sea is minimal, this duration is short enough to not allow for temporal variations of the water level (stationary). The average duration per series is about five minutes, which results in a frequency resolution of about 0.003 Hz. This gives an almost continuous plot, but the error is significant, namely about 45% (error=1/p, with p = 5).

At Location 2, three series of measurements were performed. The first series consisted of a sequence of 40 consecutive waves, the second one of 44 consecutive waves and last one of 50 consecutive waves. Zero-down crossings were used, since the front of the wave was best visible when looking from the coast. The wave height was in this case defined as the range between a trough and the succeeding crest. The total duration of the measurements is 14 minutes, performed within about 75 minutes. This is just long enough to obtain an accurate result. Again, the average duration per block is about five minutes, resulting in the same frequency resolution, but this time with an error of 58% (error=1/p, with p = 3).

Figure 36:Visual wave measurements at Location 1 (left) and Location 2 (right)

There are some general comments to these observations. Firstly, low waves were quite hard to observe, especially when they were succeeding larger waves. This was due to the angle of view, since we had to look from the shore towards both locations. Furthermore, since the measurements were performed for onshore conditions, we observed cnoidal waves. This means the crests are short and high, whereas the troughs are long and low. Wave heights have been compared to the height of the jetty, which was about MSL +1.7m (including the wind set-up of 0.3m at both moments). The wave crest heights are expressed in percentages of the height until where the wave crests reach. Assumed is that the wave troughs are twice as low as the wave crests (due to the cnoidal character, corresponding to an elliptic parameter *m* of 0.9). Also, wind set-up should be taken into account. Figure 37llustrates the tip of the jetty and its relative height with respect to the Mean Sea Level.

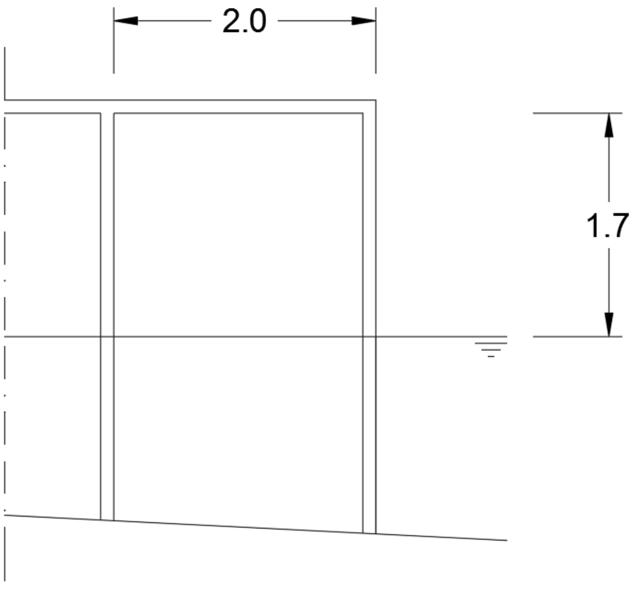


Figure 37: Jetty tip and position with respect to MSL

When for example a 50% wave is observed, the crest height is equal to 0.65m (wind set-up included). The total wave height is then 0.98m, when assuming the trough height is half the crest height.

4.5.1.2. Svašek forecasts and SWAN model

For the entire period of the fieldwork, we received weather forecasts from Svašek Hydraulics, which included wave heights, periods and directions, even as wind speeds and directions. This data was retrieved from a weather station just offshore of Sveti Konstantin, as can be seen in the figure below.

Figure 38: Location of the Svašek weather station

The weather station is situated at 15m water depth, which is corresponding to intermediate water for most wave periods. It is assumed that the wave and wind conditions are equal along the depth contours at a specific moment. Using all wave and wind characteristics at the weather station, the onshore wave conditions can be determined using SWAN. In order to succeed, also a depth profile from the weather station is needed, which is retrieved using Navionics.

The SWAN calculation is performed at two locations. The first location is cross-shore from the jetty, to compare the results to the visual wave measurements. One computation is done for severe conditions (corresponding to the first days of the fieldwork and serving as boundary condition for the summer). The other computation is done for mild conditions (corresponding to the last days of the fieldwork). The second location is cross-shore from the breakwater, to check for the breakwater design. Over here, three computations are done. The first two computations are based on the severe and mild conditions as described above. The third calculation corresponds to the extreme conditions, which are retrieved from Valchev, Andreeva and Prodanov (2014). The severe conditions correspond to the extreme summer conditions, whereas the extreme conditions correspond to the extreme winter conditions. The cross-shore distances for both locations can be found back in the figures below.

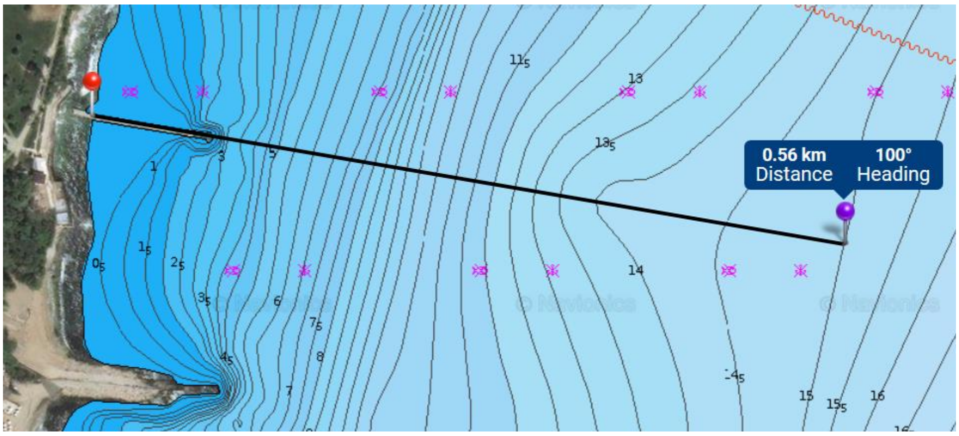


Figure 39: Cross-shore distance at the location of the jetty

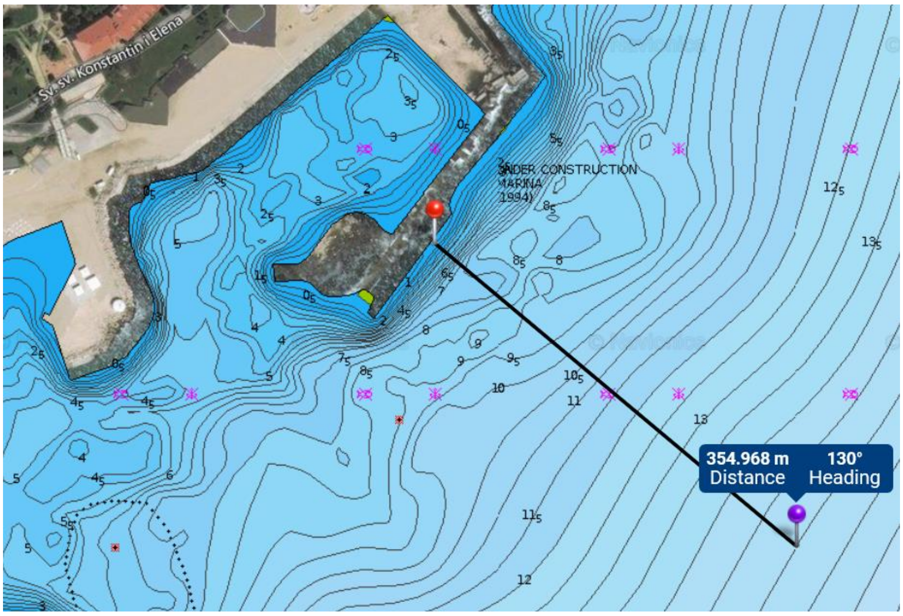


Figure 40: Cross-shore distance for the location of the breakwater

4.5.1.3. Measurements with wave loggers

Two wave loggers were used for the measurement of the free surface elevation due to waves in the vicinity of the breakwater. Measuring tape was also used to measure the dimensions of the logger and determine distances based on the place where the sensor is located. A rope and a tie rap were used to help define the depth in front of the marina and also to attach the protective cylinder case of the sensor to one of the wave loggers. Furthermore, a boat was used to deploy the loggers in the middle and outside of the marina. Finally, buoys were used to accompany the wave loggers and make it easier to spot and to retrieve them.

Figure 41: Wave logger 1 (left side) and wave logger 2 (right side).

As can be seen from the picture Figure 41 in case of wave logger 1, the tubular PVC case could not be fitted in the base of the logger and it was therefore attached to it.

All the necessary dimensions where measured before the deployment of the wave.

	Wave pressure logger 1	Wave pressure logger 2
Height of sensor (m)	0.23	0.23
Height of the bottom concrete base	0.16	0.18
Distance from buoy bottom to sensor	0.52	0.56
Distance from top of sensor to bottom	0.75	0.82

Table 4: Wave logger measurement information

Three wave pressure records were obtained with two different pressure loggers during the field campaign. The wave pressure loggers where deployed in key positions, such that the resulting wave data would be used in feasibility analysis phase of the project.

The first deployment position was near the inner south end of the quay wall (record 1). That would facilitate the extraction of the incoming wave height, the wave penetration into the marina and the feasibility analysis of the quay wall. The second deployment position was outside of the marina, windward from the breakwater in order to record the incoming wave height (record 2). The last one was deployed inside the marina with the purpose to record the transmitted wave height (record 3).

Figure 42: Overview of deployment positions

The measurements took place on a day when the weather was good i.e. little wind and small waves. To acquire the first record, the logger was deployed in the sea for one hour and 15 minutes, while for the second and third records, loggers kept continuous measurements for about 1 day and 2 hours.

At each location where the sensors were positioned, the coordinates were measured with Google maps. The water depth was calculated by placing a tie rap on the point of the rope that is exactly at the sea surface level and then measuring the distance from that point to the tip of the sensor capsule. Moreover, each sensor is secured in a concrete buoy, in order to remain at a fixed position, with the pressure sensor closer to the bottom end. To deploy the pressure loggers for records 1 and 3, the use of the boat was necessary at the target locations. For the case of record 1 the deployment was realized from the quay wall of the peninsula and a rope connected to the wave logger was attached to a stable steel rod at the quay wall. All the above information of the locations where the buoys where positioned as well as the distances from the buoy base are presented in Table 5.

	Record 1	Record 2	Record 3
Location (short description)	Near quay wall	Outside of marina	Inside of marina
Location (coordinate UTM zone 35T)	582221E 4786365N	582460E 4786254N	582344E 4786348N
Water depth (m)	3.5	10.4	3.0
Wave pressure logger type	2	1	2

Table 5: Overview of wave loggers location

Figure 43: Deployment of wave logger for record 1 (left) and 2 (right)

It must be noted that the sensor is placed downwards so that it is closer to the bed, to minimize sediment infiltration into the wave logger. In addition, the original idea was to record the diffraction patterns around the tip of the breakwater but due to presence of rocks on the bottom of the marina close to the breakwater, which could result in the tilting of the buoy or its trapping, the sensor was placed close to the middle of the marina. Moreover, even if the positioning of the sensor very close to the breakwater was possible, the measuring of the diffraction pattern would still be difficult. That is because, due to the good weather there was absence of wind and small waves thus the occurring variations were expected to be not significant enough to be captured by the pressure sensor.

Concerning the preparation of the loggers before their deployment, several steps must be executed with extra attention. Firstly, the user must decide on the values that best fit the scope of their measurements. The sensor is then connected to the appropriate software by means of a USB cable and all the necessary settings (described in the next paragraph) are defined. Afterwards the sensor is placed in the protective PVC case and it is closed in a waterproof way. That being said, grease compound is applied on the external case to ensure that no intrusion of water will happen during their recording underwater. A possible water leakage would not only destroy the measurements but the equipment itself as well. Finally, to achieve an accurate measuring record, the wave logger should maintain a stable position. In any case severe wave conditions are a serious threat on the stability of the wave loggers.

As explained, before their deployment, the wave loggers had to be configured with the necessary settings. The pressure record (duration of recording, interval between recordings) must be determined very carefully such that it is short enough to describe a steady-state wave condition by a single value and long enough to include a statistically correct data processing. The sampling duration should have at least 100 waves to allow a decent spectral analysis. This means that the sampling time has to be 20-30 min. The sampling frequency was fixed and equal to 4 Hz for all the wave loggers.

Regarding the setup of the device, the applied settings are shown on Table 6.

Sampling frequency (Hz)	4
Number of samples	8192
Approximate duration of single record (min)	30
Real duration of single record (min)	33.6
Time interval between consequent records (hr)	1 (including the 33.6 min of measurement)

Table 6: Wave loggers setup for wave measurements.

The wave logger designated to measure record 1, was deployed on September 29, 2017 during not so mild conditions (1.8m wave height). At 14:32h it was logged on and at 15:03h it was deployed to the sea. It was extracted on 16:15h the same day. For this measurements wave logger 2 was used.

Wave loggers for records 2 and 3 were deployed the same day, at October 3, 2017 and retrieved the next day. Both wave logger 1 and 2 were used.

4.5.1.4. Calibration of wave loggers

Before the post-processing of the wave logger data, calibration is needed. The calibration of the logger took place after the measurement of record 1, on September 29. A second calibration process to accompany the measurements of October 4, was not possible neither at the marina due severe wave conditions, nor at the pool since it was emptied by then. The purpose of the calibration is to define the relation between the pressure and the water depth. The type of the relation is depending on the device. In this case, it is linear.

For calibration purposes the logger was dropped in the swimming pool outside of the hotel. This position was selected because of the fixed and sufficient water depth. One aspect to take into account is the difference in salinity between the sea water and the fresher water from the pool. Nevertheless, a position inside the marina was not a choice since the waters were far from calm.

To lower the device in the water and make it as stable as possible a small rock with a smooth surface was attached to the logger with a rope and several plastic cable ties (Figure 4). A measuring tape was also attached to the sensor which enabled us to measure 5 water depths every 20cm starting from the bottom upwards. Of course, all the related distances were measured with a tape measure. An overview of the process in presented in Table 7:

Location	Swimming pool outside hotel
Number of measuring points	5
Duration of measurements	4min
Distance from bottom of rock to sensor	5cm
Distance from sensor to zero point in measuring tape	35cm
Depth 1	165cm
Depth 2	145cm
Depth 3	125cm
Depth 4	105cm
Depth 5	85cm

Table 7: Overview of calibration procedure

Figure 44: DIY set-up for calibration purposes

The output of the wave logger is a .txt file. Plotting the output water pressure in Voltage with measurements in time one notices the linear relation between the water depth and the pressure in Voltage. The average of the resultant wave data per water depth (5 depths) is taken. The resultant mean voltage values can be plotted with the water depth and the coefficients of the linear function of the form $y=\alpha x+\beta$ can be found. Then mean voltage values can be transformed to pressure values in Pa. The density of the pool and the gravity constant need to be prescribed for the transformation.

4.6. Breakwater

To perform an assessment of the present condition of the breakwater the existing material types as well as their quantity should be identified. This process is relevant in order to estimate the amount of material that can be available for removal and reuse. Thus, selection of the proper methodology is crucial in order to achieve accurate results. The material types that were observed consist of tetrapods, concrete blocks, rock and concrete debris. Different methodologies were used in order to identify the quantity of every material type.

In the case of tetrapods, the quantity was obtained by visual inspection on site. Due to the bad weather conditions an overall inspection in all breakwater sections was difficult and dangerous to be conducted, so satellite images were also used as an assisting method in order to perform a complete investigation. For the existing amount of rock, sampling was carried out. Additionally, measurement of the dimensions of each rock sample was performed so as to calculate the nominal diameter and identify a sample distribution with a sieve curve.

The number of concrete slabs existing in the breakwater were measured. These blocks can be re-used since the construction of the breakwater stopped and there still presented there. The estimation of their dimensions and the number of blocks were not possible to be done accurately since unfavourable weather conditions were present at the day of measurements. For that reason, images from google Earth were also used.

Finally, an estimation of the concrete debris was made. There are several hotels in the region which are to be demolished and therefore the material resulting from the demolition could possibly be recycled and reused for core material for the new breakwater. The aim was to obtain the total volume of concrete used for the construction of the hotels so that a rough estimate can be obtained regarding the volume of concrete that will remain after the demolition.

4.7. Quarry

During the Fieldwork we visited two quarries near Varna. The first quarry was used to get a feeling for the characteristics of the stones that are produced in the quarry, the second quarry was to show the process of gaining the stones. At the first quarry we collected some stones and tried to get an overview of the available material. In this chapter, we're going to elaborate on the characteristics of the stones. For the data gathered in the quarry, multiple measuring methods were used. In the following paragraphs we present a measuring method, while in the next chapters the results will be presented and discussed.

The first test carried out at the quarry was the determination of the elongation and blockiness of the stones.

1. Pick a number of stones of the available piles of stones. A feasible number should be picked, neither too large nor too small. You want the number of stones to be representative, but also workable. The group was divided in two to compare results gathered at each of the exercises. The two groups together picked a total of 39 stones.
2. Determine with a scale the weight of each stone.
3. Determine with a ruler / measuring tape the main dimensions of the stones.
4. Determine with a ruler / measuring tape the longest axial dimension and the shortest axial dimensions of the stones.

Everything was written down during the test. Note has to be made that the two groups used different scales (a traditional floor scale and a luggage scale used with a plastic bag). In this way we could average out the results of the scale and the other measuring device. This cancels out some accuracies in both measuring devices.

The blockiness is determined by the ration between the volume of the rock and the smallest volume defined by the axis of the rock. The elongation is the ratio between the longest and shortest axial length of a rock. The following formulas can be applied

$$\text{Blockiness} = \text{volume of rock} / (x*y*z)$$

$$\text{Elongation} = \text{longest axial dimension}/\text{shortest axial dimension}$$

The second test that we had to carry out in the quarry was the determination of the availability of the rocks. In the quarry a large area was covered with a lot of large rocks and we had to get an impression of the number of rocks to make sure that a design could be carried out. When building you don't want to be confronted with a shortage of rocks. The procedure followed to assess the stockpile capacity is presented below:

1. With a GPS Device (or ViewRanger Application on mobile phone) a circumference of the stock of rocks could be determined. Just walk around the stock and you get an impression of the area that is covered with rocks. Our tracks can be seen in Figure 1
2. Pick a sample position
3. Determine an imaginary box, it should be workable and representative for the average porosity of the stock
4. Determine the amount of volume of the rocks within the imaginary box. This can be done by measuring the dimensions of the rocks with a ruler / measuring tape
5. Divide the volume of the rocks by the volume of the imaginary box
6. The so called "1 - porosity" is now known. This is equal to % of rocks in a certain volume.
7. Determine the average height of the stock of rocks
8. Determine the average volume of the stock by multiplying the average height by the area
9. Determine the number of rocks available by multiplying the average volume of the stock by the "1-porosity" value.

Once again everything was written down during the tests. This test has also been carried out twice to get two different sample imaginary boxes to compare the results. The average result presented us with ~21600 rocks available.

Figure 45: Periphery of the stockpile as captured by the GPS application (ViewRanger)

Last part of the quarry test that had to be carried out was back in Delft. We picked some sample stones from the quarry to determine the density of the stones and the dn50.

1. Determine with a scale the weight of the stones
2. Submerge the stones in a known volume of water
3. Determine the volume displacement of the water
4. Determine the density of the stones with the determined mass and the displaced volume by applying the following formula:

$$\rho = m/V$$

5. RESULTS

In the present chapter the results of the measurements presented above will be presented. With the aim of keeping this report short and concise only the final results will be discussed here. In the case that these results do not come directly from the measurements but there is some post processing required, it will be included in the appendix and the reader will be referred to it. A discussion of the succeeded accuracy and the usefulness of these results in the feasibility assessment is also included in the present chapter.

5.1. Beach

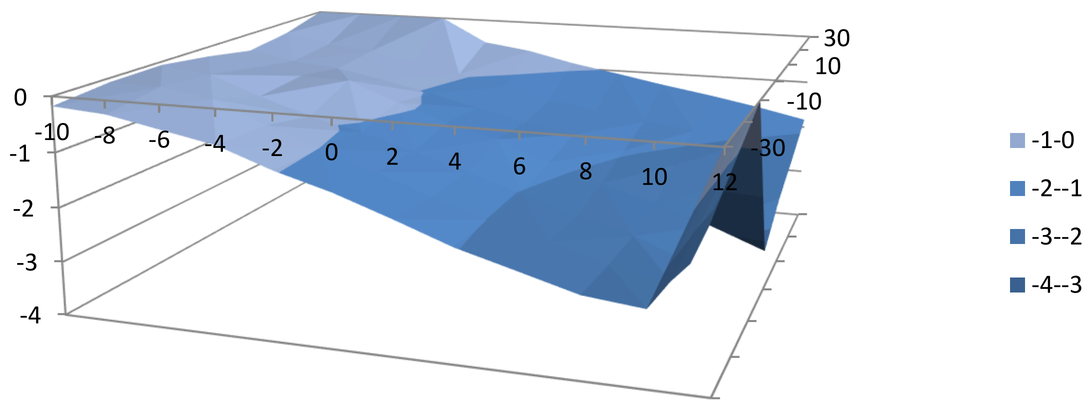
This section contains the results of the profile measurements of the North and South beach as well as the shoreline position results.

5.1.1. South beach

The results of the cross shore profile measurements of the South Beach are shown at the table below. The cross sections (C-1 to C8) and distances from S2 reference point are shown in the horizontal axis, and the measured points along them (-7 to 5) with their distance to the baseline are shown in the vertical axis.

		C-1	C0	C1	C2	C3	C4	C5	C6	C7	C8
	[m]	-172,8	-150	-112,5	-75	-37,5	24,5	54,5	92	129,5	159,5
-7	-35						0,85				
-6	-30						0,59				
-5	-25					-0,12	-0,57				
-4	-20					-0,32	-0,67	0,88	0,99	0,63	
-3	-15				0,38	-0,61	-0,8	0,65	0,77	0,41	
-2	-10	-0,92	0,17	-0,03	-0,37	-1	-0,95	0,27	0,53	0,29	
-1	-5	-1,78	-0,62	-0,69	-0,85	-1,54	-1,51	-0,37	0,06	-0,31	-1,53
0	0	-1,81	-0,98	-0,96	-1,2	-2,17	-2,16	-0,81	-0,53	-0,67	-1,57
1	5	-2,09	-1,38	-1,35	-1,85			-0,84	-0,78	-1,09	-1,61
2	10	-2,28	-1,87					-1,37	-1,08	-1,6	-1,87
3	15		-2,03						-1,36	-1,84	-2,1
4	20								-1,9	-2,11	
5	25								-2,11		

Figure 46: measured heights in South beach, with respect to DNL



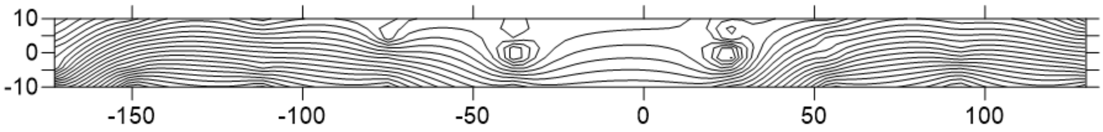


Figure 47: south beach contours – 2017

5.1.2. North beach

The results of the cross shore profile measurements of the North Beach are shown at the table below. The distances of the cross sections from R2 reference point are shown in the horizontal axis, and the distance from the baseline of the measured points along them are shown in the vertical axis.

[m]	-30	-20	-10	0	10	20	30
-10	-0,175	-0,075	-0,065	-0,195	-0,365	-0,245	
-8	-0,245	-0,095	-0,195	-0,375	-0,535	-0,455	
-6	-0,435	-0,255	-0,555	-0,475	-0,635	-0,545	
-4	-0,625	-0,665	-0,875	-0,505	-0,825	-0,755	
-2	-0,975	-1,005	-1,105	-0,885	-1,045	-0,955	-0,515
0	-1,255	-1,355	-1,405	-1,215	-1,285	-1,155	-0,655
2	-1,605	-1,715	-1,705	-1,475	-1,525	-1,395	-0,885
4	-1,975	-2,015	-2,025	-1,855	-1,695	-1,575	-1,065
6	-2,255	-2,305	-2,355	-2,185	-1,945	-1,725	-1,255
8	-2,545	-2,605	-2,615	-2,515	-2,165	-1,855	-1,425
10	-2,655	-2,785	-3,025	-2,825	-2,455	-2,055	-1,615
12				-3,225	-2,665	-2,275	-1,835

Figure 48: measured heights in North beach, with respect to DNL

5.1.3. Accreting/eroding tendency in South and North beach

The 3D sketches of the Southern Beach produced by the data measured in 2017 and 2016 cannot be compared, as lesser points were measured in 2016 fieldwork and only a comparison can be made with the central points and not with the points further away from the baseline.

By comparing with last year measurements, the sand height difference is determined and shown in Table 8. Also, the Surfer software package was used and the contours of the beach were determined both for 2017 and 2016 and can be seen at Figure 49: South beach contours – 2017Figure 49 and Figure 50 respectively. Additionally, the change in sand volume was calculated, which was found to be 28 m^3, meaning that the beach gained 28 m^3 of sand between 09/2016 and 09/2017. But it should be noted that the number of points that could be compared was not very high. More specifically, the grid shown in includes only points at the cross sections at -10, -5, 0, 5 & 10 m from the baseline. It should be noted that at the other cross sections there were many points missing, which made it impossible for us to include them and getting a reliable result. Thus, this is the reason why only a part of the south beach is represented in the figures above.

In conclusion, it can be said that not only that there has been an overall accretion in the beach, but also this accretion has mostly occurred in the sides of the beach and erosion has occurred in the centre. It is also probable that not only the offshore sand was deposited in the sides of the beach but also the sand which was eroded in the centre.

	0	1	2	3	4	5	6	7
-7								
-6								
-5								
-4							1,96	
-3							2,65	1,76
-2	1,1	0,875					1,93	2,1
-1	0,535	0,38	0,045		-0,06	1,18	1,82	1,28
0	0,31	0,19	-0,13	-0,96	-0,58	0,94	2,32	1,51
1	-0,035	0,92	-0,715			1,92		2,15
2	0,065					2,13		
3	0,815							
4								
5								

Table 8: Sand height difference (M) between 2016 & 2017

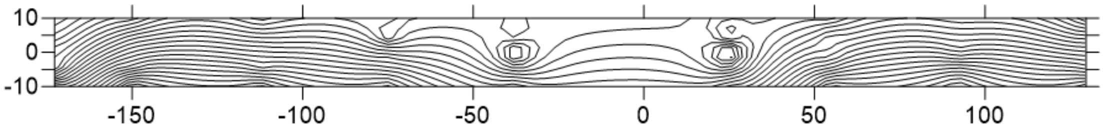


Figure 49: South beach contours – 2017

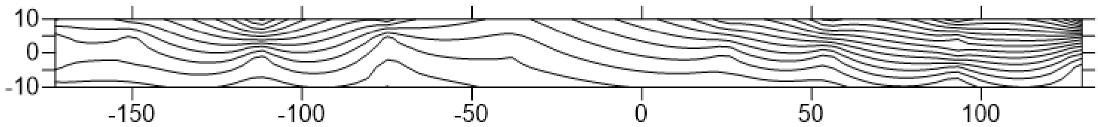


Figure 50: South beach contours – 2016

The 3D sketches of the Northern Beach produced by the data measured in 2017 and 2016 can be seen at below, respectively.

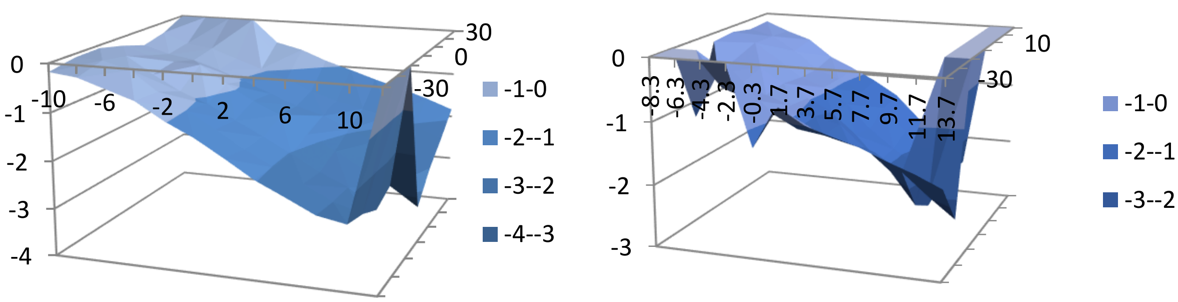


Figure 51: Sketch of North beach 2017 (left), 2016 (right)

By interpolating last year's measurement points, which were different than this year's, the sand's height difference at this year's measurement points is determined and shown in Table 9. Also, the Surfer software package was used and the contours of the beach were determined both for 2017 and 2016 and can be seen at figures 7 and 8 respectively. Additionally, the change in sand volume was calculated, which was found to be -24 m3, meaning that the beach lost 24 m3 of sand between 9/2016 and 9/2017. In conclusion, it can be said that not only sand has eroded from the beach and disappeared further offshore, but also that sand has moved from the southern side towards the northern side of the north beach.

	-30	-20	-10	0	10	20	30
-10							
-8				-0,1975	-0,411	-0,27	
-6			-0,034	-0,067	-0,3345	-0,2405	
-4		-0,084	-0,211	0,0425	-0,253	-0,2085	
-2		-0,275	-0,284	-0,092	-0,214	-0,124	0,232
0	-0,0925	-0,4235	-0,444	-0,205	-0,296	-0,175	0,275
2	-0,959	-0,596	-0,6115	-0,244	-0,255	-0,2075	0,251
4	-1,1055	-0,83	-0,8025	-0,272	-0,04	-0,1165	0,3335
6	-1,257	-1,02	-0,784	-0,212	0,004	0,1175	0,535
8	-1,4195	-1,196	-0,6785	-0,289	0,05	0,324	0,7545
10	-1,3625	-1,104					
12							

Table 9: Sand height difference (M) between 2016 & 2017

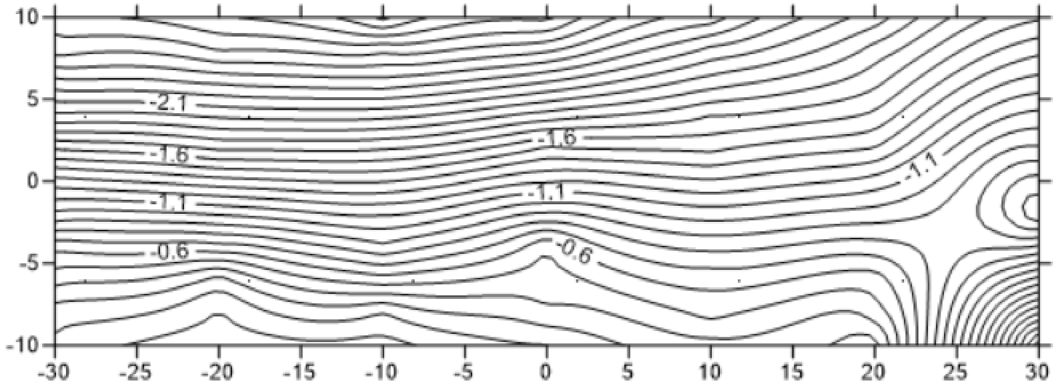


Figure52: North beach contours – 2017

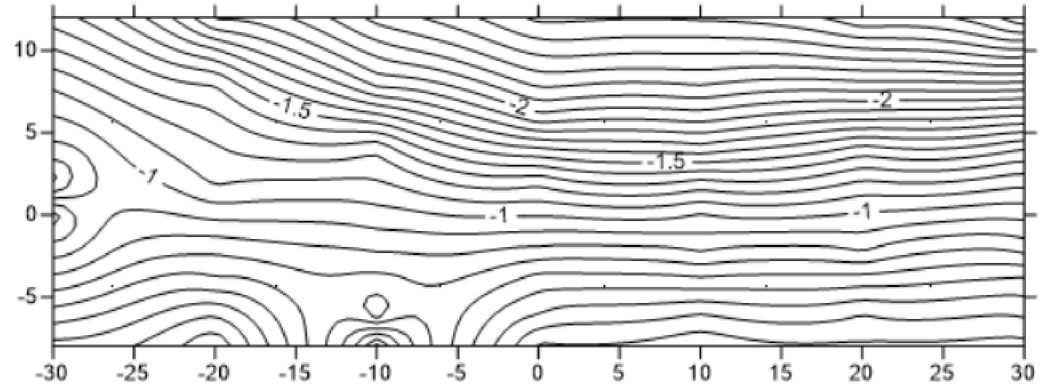


Figure53: north beach contours – 2016

5.1.4. Discussion cross shore profiles

As far as the accuracy of the equipment is concerned, the following deviations can be considered. For the measuring tape, there are 2 kinds of errors, the one is a reading error (systematic), which is approximately 10 cm, and the other is an error, because the tape was not perfectly horizontal, especially during the days when the wind conditions were severe. The total error can be approximated at about 0.5 m. For the levelling instrument and pole measurements, there are also 2 kinds of deviations, the one resulting from the sinkage into the sand of the vertical measuring pole and amounting to 5 cm, and the other being a reading error of approximately 1-2cm.

The measurements, in order to determine the height of the Mean Sea Level were conducted during rough wind and wave conditions in the surfzone, so there is significant error in the order of 0.5m.

To estimate the height difference between the height reference point and the MSL we used the levelling instrument the next days in 2 positions/heights because it was not possible to see at the same time the water surface and above the height reference point through the levelling instrument. See sketch for full process. This was done the next day of the beach profile measurements, and it was windy. The height of the wave at the point of the measurements was estimated through the levelling instrument as 0.4-0.8 m. It is obvious that the accuracy of this as well as the MSL measurement is limited under these conditions.

The final accuracy of the measurements for the cross shore beach profiles is:
Slope: 10 degrees (compared to 5 degrees initially)
Width: 2m (compared to 1)
Length: 7m (compared to 5)
Height: 50cm-1m (compared to 20-50 cm)

NOTE:
Northern Beach – Day 2 – 2.10.2017
The measurements conducted at Day 2 at the Northern beach are not elaborated further, since they were done during a day with very severe weather, wind and wave conditions, thus, there is very high uncertainty and inaccuracy in the measurements. Furthermore, the cross sections are not perpendicular to the baseline, but to the shoreline, which is not a constant line and, thus, not helpful for comparison with the other years.

5.1.5. Shoreline position

To compare shorelines from 2017 with 2016 and 2015 also the data of the latter two years must be processed. First an inventory was made. The processed tracks are provided as GPX as well as KML files.

Then the different shoreline measurements were compared to each other as per track. A distinction has been made between the different dates of measuring, since the second day of measuring, the weather was calmer. After which, the measurements of 2017 are compared to the measurements of 2015 and 2016. With respect to 2015 only the North Beach was compared. Note, that data for 2015 is only available as excel files (.xlsx), whereas from 2016 and 2017 is available as .gpx and/or .kml. This only determine the way of comparison, mostly google earth is used to compare. However, for some measurements also excel is used.

The shoreline measurements are used to determine the shoreline retreat. It may be said that the shoreline retreat directly corresponds to the beach profile change. If the shoreline retreat is determined, the total loss of beach volume can be determined as a measure for a first estimation. The average shoreline retreat can be determined by dividing the area of the beach in question by its representative length.

5.1.5.1. North Beach

The shoreline of the north beach has been analysed using the GPS devices and the Google Earth satellite images. The results per method are presented below:

Regarding the north beach several measurements, using the GPS devices, have been made, these are shown in Table 10. The table depicts the measurements done by each person on that specific date for each different track. The recordings which have not been used for reasons such as a defect or sporadically working device or non-useable files, have been marked red in in the table. The measurement techniques applied to collect and process the data has been summarized in chapter Shoreline measurements4.1.5.

	Upper North Beach	**Lower North Beach**	**Mid North Beach**	**Bay North Beach**
Measurer 29-09-2017	T01 [UNB]	T02 [LNB]	T03 [MNB]	T04 [BNB]
Sebastiaan W.	-	X	X	x
Daan B.	-	X	X	x
Lina		X	X	x
Measurer 04-10-2017				
Teni M.	x [2x]	-	-	x [2x]
Ioanna S.	-	x [2x]	-	x
Alejandra A.	x [2x]	x [2x]	-	x [2x]
Christian	-	-	-	-
Kamelie	-	-	-	x
Number of samples	4	6	2	8

Table 10: Measurements conducted for the North Beaches

5.1.5.1.1. Overview of results of all tracks

Primarily the results of the all the measurements of 2017 are overviewed in excel. These are the results of all the tracks walked / made during the fieldwork in 2017. The first graph depicts the tracks walked on 20/09/2017 and the second of the tracks walked on 04/10/2017. The graphs are shown in Appendix A, in Figure 98 and Figure 99.

From these graphs, not a straight conclusion can be drawn. It simply shows the tracks walked and nothing more. Furthermore, the detail which these graphs provide is too little to determine the actual accuracies of the tracks.

To show the results of the tracks in more detail the results of the all the measurements of 2017 are overviewed in excel separately. These are the results of all the tracks walked / made during the fieldwork in 2017 for the northern beaches. They show the tracks walked on that specific beach per person (Figure 100, Figure 101, Figure 102).

These tracks are only the results of the measurements made on 04/10/2017. They incorporate the bay of the north beach, the southern and northern north beach. The tracks walked by Ioanna, Teni and Alejandra are somewhat shifted from one another. The difference is best seen in the tracks of the bay of the north beach.

It must be noted that the tracks walked up and down by one device, for example two tracks of Alejandra, give a more aligned result. The difference between the different tracks walked by different people, is mainly cause by the different shorelines they've walked. For instance, Alejandra has walked the interpreted maximum shoreline, considering the maximum wave run up as well. Whereas, Teni and Ioanna have walked the actual shoreline, not considering the wave run up. It must be noted, however, that still a difference can be seen with Alejandra. This difference may be due to the difference in settings or the accuracy of the different telephone / app devices. Also, the results of Ioanna were walked by one of the Bulgarians, who might not have walked in a straight line, but next to Teni, which would explain the slight difference.

From these graphs, not a clear conclusion can be drawn, regarding the overall change in beach volume. Somethings can be said regarding the wave run up. However, this will be discussed in the next paragraph. Furthermore, the detail which these graphs provide is too little to determine the actual accuracies of the tracks.

To draw a comparison between the two days a display in google earth has been made and the full overview of all the tracks have been overviewed in excel between the results of 29/09/2017 and 04/10/2017.

Again, a clear difference is shown between the different tracks that have been measured by different people. These differences show in each of the figures. The differences are best seen on Figure 101, since these show a more zoomed in result. The differences will therefore be explained using this specific figure. Note that the waterline measurements conducted on 29/09/2017 are by far the most land inward. They have been conducted by Sebastiaan and Daan and both lines correspond well to each other. They have measured the interpreted mean wave run up shoreline. If they are compared to the maximum wave run up shore line, measured by Alejandra on 04/10/2017, they have a slight difference of approximately 0.5 to 1 meters. Furthermore, if both results are compared to the measurements conducted by Teni and Ioanna on 04/10/2017, the difference between the measurements is approximately 3 to 4 meters. From a comparison between these results it may be said that the maximum wave run up level is approximately 5-7 meters during harsh weather conditions and 3 - 4 meters during more calm weather conditions. The waterline measurements of Teni may be used to other data to compare the beach volume in- or decrease.

To analyse the evolution of the shoreline, this year's results were compared to GPS records from year 2015 and 2016. Availability of GPS recordings from 2015 and 2016 can be found in their reports.

The recordings of 2015 have only been stored in excel and no google earth files were stored. Furthermore, the excel files have been stored in a full overview of total measurements conducted, so not separately per beach track. This way the comparison to the measurements conducted in 2017 can only be done on a full beach overview in excel (Figure 54).

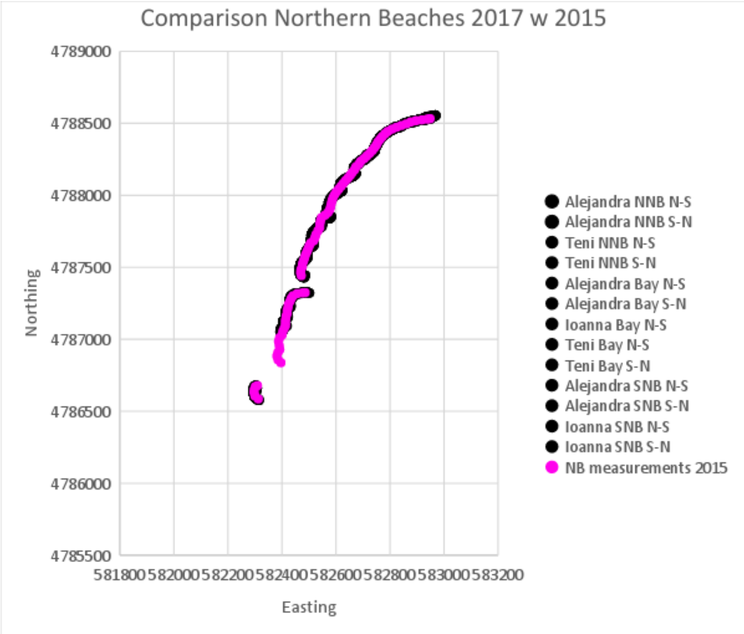


Figure 54: Comparison of results 2015 with those of year 2017

At first sight it can be concluded that the shoreline measurements conducted in 2015 and 2017 have a large correspondence. They do not seem to differ immensely from each other. It seems that the pink line is precisely in the middle of the black line. This may be explained because in 2015 the shoreline measurements were done along the interpreted average shoreline. On the contrary the shoreline measurements in 2017, done by Alejandra, Teni and Ioanna have been done along the maximum shoreline and minimum. This means the black line incorporates a larger range of shoreline. It may therefore be concluded that it is indeed quite natural that the pink line would be the average of the black line. It should however, be noted that the weather conditions may not have been totally the same and the device accuracies may deviate. Consequently, a conclusion that no shoreline retreat or increase has occurred cannot be made with full conviction.

The recordings of 2016 could not be compared in Google Earth since the measurements are elevated and not thus not vertically aligned with the measurements conducted in 2017. They have therefore been extracted to excel files and compared in graphs with the measurements conducted in 2017 (Figure 108, Figure 109, Figure 110)

The shoreline measurements conducted in 2016 have been compared in excel files. In Figure 108 and Figure 109 the shoreline measurements seem to correspond quite well to the shoreline measurements conducted by 2017. These two graphs however show little detail. The third graph Figure 110, shows more detail. This means that this graph is used to compare the different measurements in more detail. It shows that the waterline measurements of 2016 corresponds mostly to the waterline measurements done by Teni. Previously it has been mentioned that Teni has done the waterline measurements without considering the wave run up. Moreover, the waterline measurements done by 2016 consider the average wave run up shoreline. It has also been shown by usage of Google Earth that the difference between the shoreline of Teni and Alejandra and Ioanna is approximately 2 to 4 meters. Therefore, it either may be concluded that the shoreline has retreated with approximately 1 to 2 meters, or the weather conditions and therefore the wave run up were slightly more calm during the measurements in 2016. Furthermore, the accuracy of the devices may deviate with a few meters, which could cause the difference in shoreline measurements. Overall, it is hard to draw a conclusion regarding the shoreline retreat or increase. The accuracy of the measurements are of the order as the differences and the weather conditions are an unknown factor with a large impact on both the measurements and accuracy, as the interpretation of the average shore line is an inaccuracy in itself.

Historical images of google earth are used to find the shoreline position of the North beach over the suitable years/dates available and compare the different positions at the various dates. Ten images are used to make the comparison, five belonging to the ‘’winter’’ months (October to May) and five to the ‘’summer’’ months (June to September). Two control points were used to have the same reference field, one being the roof of a short building and the other being one angle of a pool, both being present in all the years examined.

Three comparisons are presented, all three of which have as a background the latest Google Satellite image (9/10/2016).

In Figure 55 the maximum retreat and advance over the years examined is presented, considering months that are close to each other even if they belong to different periods. It can be observed that the shoreline moved more onshore around 9m. This is an extreme value that is not considered very representative, since in 2014 there was a great advance of the beach, both during winter and summer months, that was later restored by the beach retreating again.

Figure 55: North beach min- max excursions

In Figure 56 and Figure 57: North beach winterFigure 57 the shoreline positions during summer and winter months are shown. The North beach can be considered very dynamic since it is highly unstable, and in the summer as well as in the winter months we can observe differences in the positions of the beach. It can also be observed that there is a small seasonality, since the width of the beach on the summer months seems to be smaller than in the winter months. However, this contradicts the theory which indicates that the summer profile is wider than the winter profile. This could partially be explained by the fact that during the winter, the storms could be strong and the sand transported offshore is not returned to the beach in spring and summer, because its moved in deeper water. These means that we have a loss of sand in the system.

Nevertheless, we cannot draw a clear conclusion about the seasonality, neither the retreat of the beach because only ten images could be used and considered representative (they had adequate resolution and the weather conditions such as clouds did not influence the clearness of the image). This number is insufficient to determine the average shoreline each year. Therefore, we expect great inaccuracies since the images are not average shorelines over the whole winter/summer period, but snapshots of a single moment in the winter/summer months.

Additionally, we should keep in mind that weather conditions play an important role on the shoreline. When storms are present a wave set up will be induced and waves will be generated, which will cause strong return currents that will move sediment offshore. The before mentioned mechanisms, will move the shoreline inland temporarily, per the duration of the storm event. All in all, even small changes in weather conditions can have a large influence on the waterline.

Figure 56: North beach summer

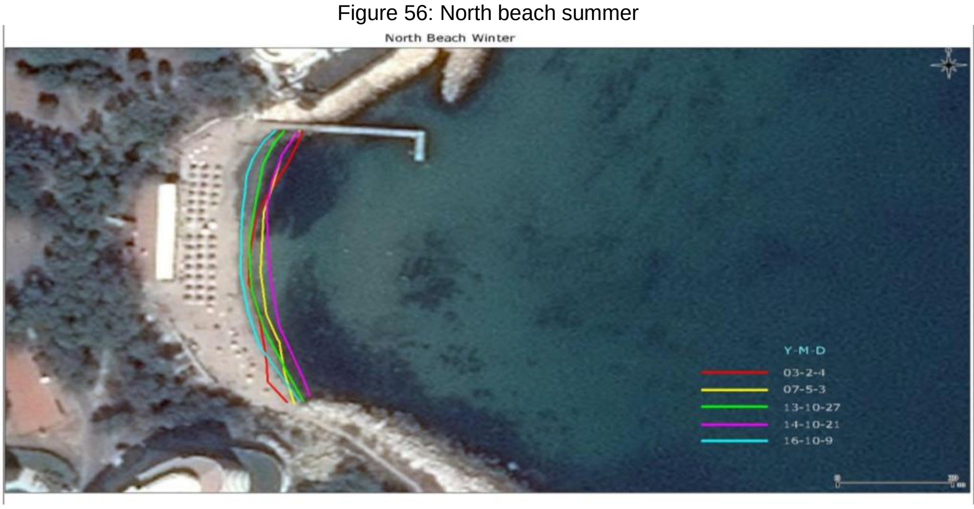


Figure 57: North beach winter

5.1.5.2. Marina

The track recorded, T04, represents the quay wall of the marina. The exact area can be observed in . More specifically, as it can be also seen in Figure (focused on Marina part) there are no significant damages in the examined area of the marina. Therefore, there is no reason to compare with last years' measurements. Note that Lina used a GPS device provided at the location, the Garmin GPS receiver. This device was not working well, so this data has not been used in the measurements.

	Marina
Name of Measurer	T04 [M]
Daan B.	X
Lina	X
Sebastian W.	X

Number of samples	2

Table 11: Measurements conducted in Marina

Figure 58: General picture of the areas/tracks recorded

5.1.5.3. South beach and Peninsula

The shorelines of the south beach and the peninsula have been analysed using the GPS devices and the Google Earth satellite images. These methods have been summarized below.

Regarding the South beach and Peninsula several measurements have been made, these are shown in Table 12. The table depicts the measurements done by each person on that specific date for each different track. The recordings which have not been used for reasons such as a defect or sporadically working device or non-useable files, have been marked red in in the table. The measurement techniques applied to collect and process the data has been summarized in Chapter 4.1.5.

	Peninsula	South Beach
Measurer 29-09-2017	T06 [Pen]	T07 [SB]
Sebastiaan W.	x	x
Daan B.	x	x
Lina	x	x
Measurer 04-10-2017		
Teni M.	-	x [2x]
Ioanna	-	X
Alejandra A.	-	x [2x]
Christian	-	X
Kamelie	-	X
Number of samples	2	8

Table 12: Measurements conducted on South Beach

Primarily the results of the all the measurements of 2017 are overviewed in excel. These are the results of all the tracks walked / made during the fieldwork in 2017. The graph depicts the tracks walked on 04/10/2017. The graphs are shown in Appendix A in addition to the measurements of 29/09/2017.

From these graphs, not a straight conclusion can be drawn. It simply shows the tracks walked and nothing more. Furthermore, the detail which these graphs provide is too little to determine the actual accuracies of the tracks. It does however, show the slight difference between the tracks walked by Ioanna, Alejandra and Teni. The difference between these tracks have been previously explained and show the same results in the south beach measurements.

Just as for the results of the North Beach, unfortunately also the results of Sebastian W. and Daan B. for the South Beach could not be extracted to excel files. To draw a comparison between the two days a display in google earth has been made between the results of 29/09/2017 and 04/10/2017 (Figure 112, Figure 113)

The legend shows which measurements were conducted by who on what day. Again, a clear difference is shown between the different tracks that have been measured by different people. Again, the measurements by Sebastiaan and Daan correspond well to each other and are again more land inward. Furthermore, the correspondence to the measurements by Alejandra on 04/10/2017, differentiate with approximately 0.5 to 1 meters. If both results are compared to the measurements conducted by Ioanna and Teni on 04/10/2017, the difference between the measurements is approximately 3 to 4 meters. From a comparison between these results it may be said that the maximum wave run up level is approximately 5-7 meters during harsh weather conditions and 3 - 4 meters during more calm weather conditions.

The waterline measurements regarding the peninsula give totally different results than they show in the observed waterline on Google Earth. This may be due to different weather conditions, since the measurements conducted were conducted on 29/09/2017 were during tough weather conditions. Therefore, no clear conclusions can be drawn from the measurements of the peninsula.

To analyse the evolution of the shoreline, this year's results were compared to GPS records from year 2015 and 2016. Availability of GPS recordings from 2015 is depicted in Table 14 and from 2016 in Table 15 in the corresponding reports. These recording also cover areas between the locations used for this report, however the analysis was restricted to the locations given in Figure 21 (item 3.3.1).

The recordings of 2015 have only been stored in excel and no google earth files were stored, also not by Boyan. Furthermore, the excel files do not contain the measurements of the South Beach. Therefore, no comparison of the south beach from 2015 can be made with the recording of 2016 or 2017.

The recordings of the Fieldwork group 2016 could not be compared in Google Earth since the measurements are elevated and not thus not vertically aligned with the measurements conducted in 2017. They have therefore been extracted to excel files and compared in graphs with the measurements conducted in 2017. However, the measurements made by Boyan in 2016 have been stored in Google Earth files, they have thus been compared to the measurements conducted in 2017 (Figure 59: South Beach track comparison with BoyanFigure 59, Figure 60).

Figure 59: South Beach track comparison with Boyan

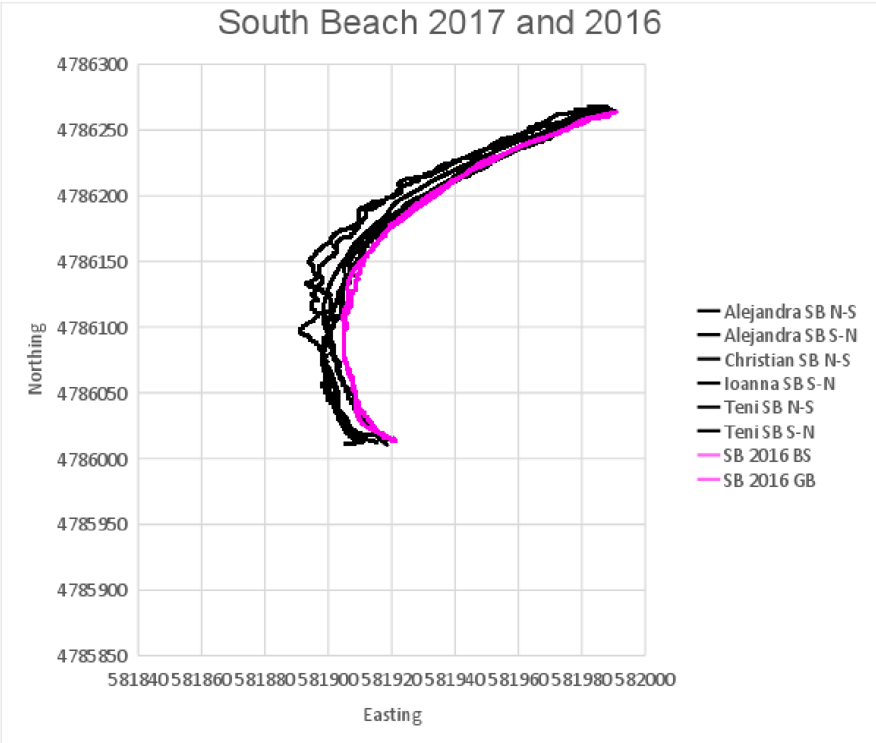


Figure 60: Comparison South Beach with results 2016

Primarily the results of the south beach measurements are compared to the results of Boyan conducted in 2016 in Google Earth (Figure 59). The red lines represent the measurements conducted on 04/10/2017, the black lines the measurements conducted on 29/09/2017 and the turquoise line the measurements conducted by Boyan on 2016. It shows that the measurements conducted by Boyan correspond the most to the measurements conducted by Teni. It must be noted that the measurement techniques of Boyan remain uncertain, e.g. was the maximum wave run up shore line measured or like Teni, was the wave run up not considered. If the respective latter option is the case the shoreline corresponds very well to the measurements of 2017, meaning no shoreline retreat or increase. This is most likely the case as can be derived from the measurements conducted by the fieldwork group in 2016, seen in Figure 60. These measurements seem to correspond best to the measurements of Teni as well.

The measurements may have been conducted in the same way as Teni. However, they might have also considered the mean wave run up shoreline. In that case, would mean that the shoreline did retreat with 1 to 2 meters. It must also be noted that the weather conditions may not have been the same, which means that the difference would be explained because of the wave run up. Also, the inaccuracies of the different devices may explain the differences.

Historical images of google earth are used to find the shoreline position of the South beach over the suitable years/dates available and compare the different positions at the various dates. Ten images are used to make the comparison, five belonging to the ''winter'' months (October to May) and five to the ''summer'' months (June-September). Two control points were used to have the same reference field, one being the edge of the breakwater and the other being one angle of a pool, both being present in all the years examined.

Three comparisons are presented, all three of which have as a background the latest Google Satellite image (9/10/2016).

In Figure 61 the maximum retreat and advance over the years examined is presented, considering months that are close to each other, otherwise the comparison would not make sense. More than two lines are presented because the north and south part of the beach have different extremes. In the northern part of the South beach, the difference between the extremes is around 11 m (difference between the oldest and the latest record). As mentioned before there are inadequate data between 2003 and 2011, so the comparison made above cannot be easily interpreted. In the southern part of the beach there is a smaller range of fluctuations of the shoreline, that is around 8m. This can be interpreted by the reduction of the littoral transport on the northern part caused by the presence of the peninsula.

Figure 61: South beach min- max excursions

In Figure 62 and Figure 63 the shoreline positions during summer and winter months are shown. The South beach can be considered relatively stable, as especially in the summer months the displacement of the shoreline is small. The shoreline shows both retreat and advance over the years and depending on the season, it slightly changes. The shoreline as presented for the latest date (9/10/2016) is in an average position between the shorelines obtained. From the data examined, we can conclude that the beach waterline is not eroding in the long term.

It can also be observed that there is a small seasonality, since the width of the beach on the summer months seems to be smaller than in the winter months. This contradicts the theory which indicates that the summer profile is wider than the winter profile. However, the differences are small, and they could be attributed to the lack of data (only ten images could be used and considered representative), that does not allow to determine the average shoreline every year, and the shorelines presented are snapshots of a single moment in the winter/summer months.

Additionally, we should keep in mind that weather conditions play an important role on the shoreline. When storms are present a wave set up will be induced and waves will be generated, which will cause strong return currents that will move sediment offshore. The before mentioned mechanisms will move the shoreline inland temporarily, per the duration of the storm event. All in all, even small changes in weather conditions can have a large influence on the waterline.

Figure 62: South beach summer

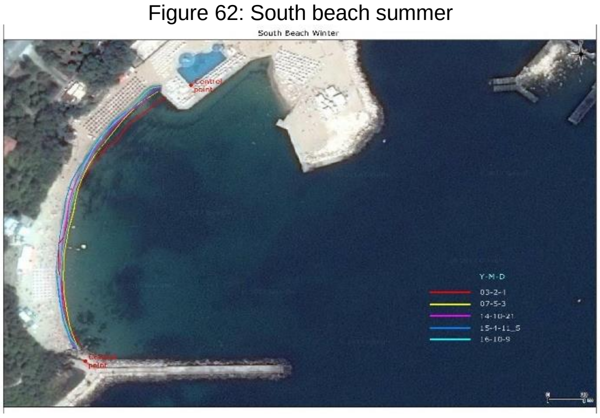


Figure 63: South beach winter

5.1.6. Discussion

The GPS devices provide the raw data of the measurements. Often this data provides inaccuracies, faults or flaws. The raw data should, therefore, primarily be manually processed by removing points or tracks which could negatively influence the clean shoreline (processed raw shoreline).

The accuracy regarding the GPS devices maybe influenced by the following aspects:

- The Garmin GPS76 has been used and worn out and known to have some functionality issues. It may have errors due to satellite shading, signal multi-path, orbital errors, receiver clock errors, etc. It is also known to shut down occasionally. The accuracy is usually in order of 5 m.
- To improve the accuracy interpretation, the value of the error and the raw data processing, the GPS devices can be placed in a known position. It may then collect data and points of its location. The error can thus be extracted from the collected data.
- The iPhone application, ViewRanger, follows an accuracy is of approximately 1-20 m and measures in a range of 1 to 10 s.
- Despite the fact that the students have chosen and performed the measurement in in single file, the results of each device still deviate amongst each other.
- The route the students chose was along the line of the average run up. However, this is and estimated and arbitrary line. It must therefore be considered in the global inaccuracy of the measurements.

The accuracy of this method using the Google satellite images can be reduced because:

- The Google Satellite pictures are not always clear (either they don't have a high resolution or weather conditions as clouds).
- There are inadequate pictures for each year, especially in the period between 2003 and 2013, that lead to inaccurate interpretations of the shoreline advance or retreat over the years.
- The images are not taken in the same date every year, so an image of October would have to be compared with a picture of March, considering that they both represent ''winter'' months.
- The determination of the shoreline visually involves a lot of inaccuracies because the set-up levels are unknown, and assumptions have to be made depending what period of the year is examined.

5.2. Grain size

Sand samples are extracted from the points in the beach indicated in Figure 64 and they are taken as close to the waterline as possible. However, the points seem to be displaced more landwards according to the google satellite picture because during the sampling process, storm conditions where prevailing.

Figure 64: Points where sand samples were extracted – South beach

5.2.1. First estimation of grain size with sand ruler

The samples taken using the tubular soil samples were long enough, so they could represent more accurately the soil layering. The sand ruler was used to get a first indication of the diameter of the sand particles at the top and at the bottom level as it can be shown in Table 13, where the grain size of the upper layer and the lower layer of the sand samples are determined. That procedure was conducted to demonstrate the extent in which the sand is inhomogeneous. These values will be later compared with the values determined from the sieve analysis.

Point	Upper grain size (mm)	Lower grain size (mm)	Length sample (mm)
1	1.4 - 2	0.5 - 0.71	650
2	0.5 - 0.71	1	1140
3	0.71 - 1	1.4 - 2	840
4	0.71 - 1	1.4	960
5	0.71 - 1	1	960

Table 13: Sand ruler estimation of grain sizes

5.2.2. Determination of grain size –sieve analysis

Characterization of particles is based on their size. Grain size can be typified by nominal diameter. The aperture of sieves that lets a particle through identifies the nominal diameter. The nominal diameter of smaller particles is defined as the diameter of a ball of the same material as the particle with the same sedimentation speed in a liquid. The widely varying range of nominal diameter is divided into fractions.

Soils of natural origin consist of a variety of particle sizes, or even several fractions. Thus, the size of particles can be characterized by grain size distribution showing the probability of occurrence of particle sizes. The most expressive way of illustrating grain size distribution is by drawing a graph that which shows the proportion of the mass of particles smaller than a certain diameter compared to the full particle cluster mass. Since particle size varies widely, the horizontal axes of the graphs have a logarithmic scale. The cumulative mass percentage of the vertical axes of the graphs is the mass percentage of sand that passed the sieve.

The smallest sieve diameter was 0.09 mm and the largest sieve diameter was 3.4 mm. Some samples may have included finer sand than sand with a diameter of 0.09 mm. The sand that would be found behind the finest sieve is smaller than 0.09 mm. Sand in the sieve of 3.4 mm is graded to 3.4 mm. Therefore, sand with a diameter below 0.09 mm and above 3.4 mm is not analysed. To improve the results more sieves with different diameters can be used.

The results of the sieve analysis are presented in Figure 65. The size of particles can be characterized by grain size distribution showing the probability of occurrence of particle sizes. From the sieve analysis it can be concluded that there are no big differences in the grain size distribution, as it was also expected since it was not possible to take samples from the bottom of the sea due to the storm conditions. The dn50 is raging from 0.7mm-1mm approximately for the points examined. More specifically, in Table 14 the dn50 for each sample is presented, obtained by reading the sieve curves. As a representative grain size for each location dn50 is used, which is the diameter of the particle that 50% of a sample's mass is smaller than and 50% of a sample's mass is larger than. It should be also noted in this point that these values represent substantiated estimates and are not values that were actually measured.

Analysing the distributions of the grain sizes it can be observed that in every examined point in the beach there is coarse sand (raging from approximately 0.5-1mm).

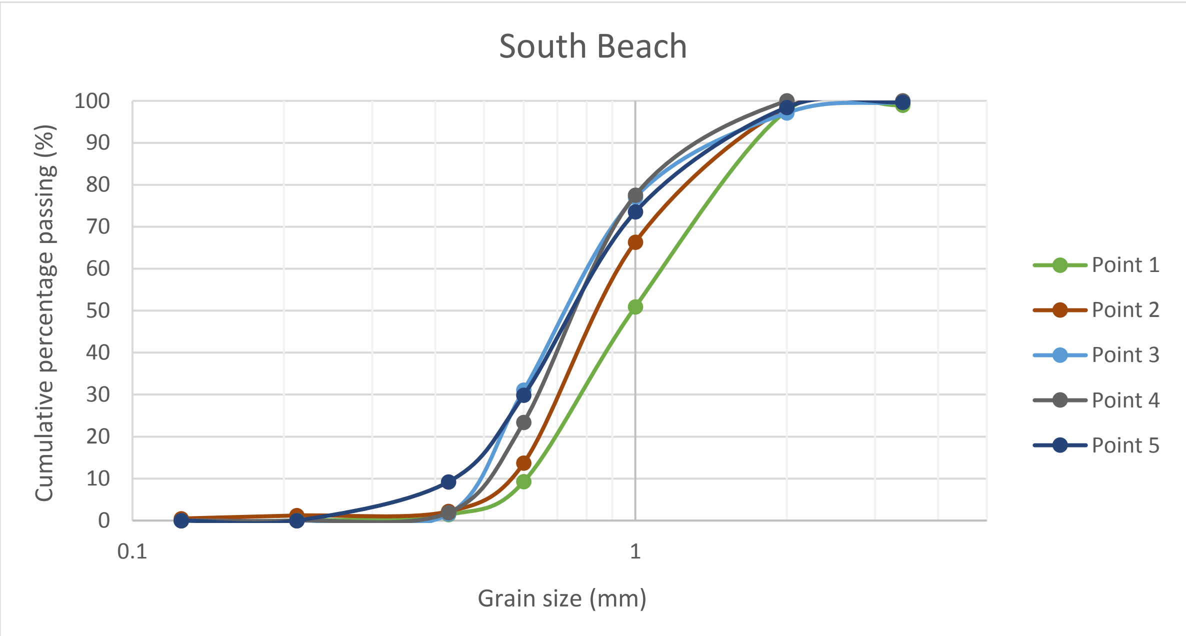


Figure 65: Sieve curves - South beach

Sampling point	d_{n50} (mm)
1	1
2	0.83
3	0.71

4	0.72
5	0.72

Table 14: Dn50 for each point examined in South beach

5.2.3. Comparison of grain size determined from sand ruler and sieve analysis

The first estimation of the grain size using the sand ruler is not expected to be very accurate since it is based on a visual interpretation and it requires a sufficient experience of the use of the sand ruler. Furthermore, a source of inaccuracy/deviating factor in the diameter defined in the sieving analysis would be that only a part of the amount that was contained in the tubular soil sampler was used and since the soil is inhomogeneous, the samples used may not be representative enough.

In order to obtain comparable values, the lower and higher grain sizes will be determined from the sieve curves (Table 16). They can now be compared to the values estimated from the sand ruler. As it is expected, there are some deviations, but most of them fall between the estimated limits.

Sampling point	d_{n10} (mm)	d_{n90} (mm)
1	0.6	1.75
2	0.57	1.65
3	0.49	1.5
4	0.51	1.4
5	0.42	1.52

Table 15: Determination of lower and higher grain sizes from sieving analysis

5.2.4. Cross-shore comparison

It would be useful to compare the distribution of the grain sizes of point 4 and point 9 which are approximately on the same line perpendicular to the coast. It can be observed that the representative dn50 is quite similar for the beach as well as the underwater location. However, there is an obvious difference in the slopes of the sieve curves that indicate the gradation of the sediment. The underwater point shows a narrower gradation and it occupies a smaller range of the aggregate. In the beach location there is a denser gradation that refers to a sample that has amounts of various sizes of aggregate that have closer values to each other. That means that on the beach we can find sediment that is more well graded than it is as we move offshore. Of course, it is obvious that two points are not enough to draw that generalized conclusion.

Figure 66: Points on the same line perpendicular to the coast – beach and offshore

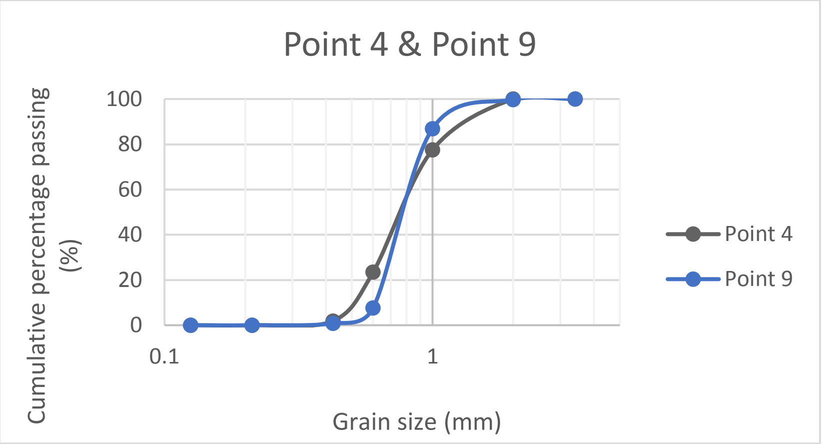


Figure 67: Sieve curves of points on the same line perpendicular to the coast – beach and offshore

5.2.5. Underwater samples

Soil samples were taken underwater in some representative points indicated in Figure 68: Points where sand samples were extracted - underwaterFigure 68. A sieve analysis is conducted as well for the underwater points, from which sieve curves are determined. There are small differences in the dn50 of the samples, since it is raging from 0.78mm to 0.86mm. More specifically, in Table 16 the dn50 for each sample is presented, obtained by reading the sieve curves. As a conclusion, the sediment in the points examined can be considered coarse sand and its size does not vary a lot depending on the position and the depth.

Figure 68: Points where sand samples were extracted - underwater

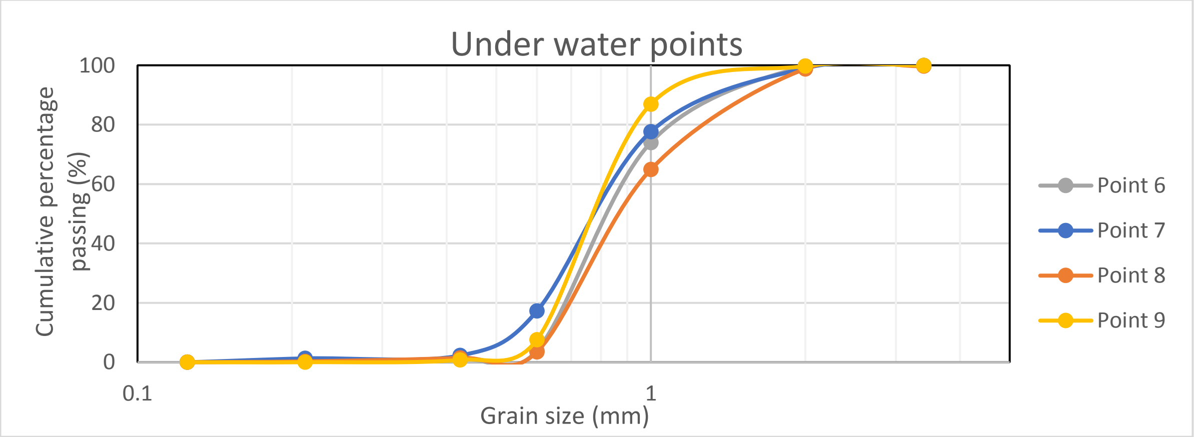


Figure 69: Sieve curves – underwater points

Sampling point	d_{n50} (mm)
6	0.82
7	0.78
8	0.86
9	0.78

Table 16: Dn50 for each point examined underwater

5.2.6. Comparison with last year results

The nominal diameter (dn50) of last year assignment in the same point (approximately) where this year's Point 8 is located, is equal to 0.79mm. This year we calculated in the same point a value of dn50 equal to 0.85mm. There is a difference of 0.06mm which can be attributed to some inaccuracies during the data processing as well as to the different sorting of the material due to the different weather conditions that were prevailing when the two samples were taken from the same point.

5.2.7. Discussion

During the process of the sieving analysis, some errors could be present that might affect the accuracy of the results. These errors need to be taken into account, so that the results can be justly interpreted. Some common error and the respective inaccuracies are listed below:

- As it was described above, the sand is removed from the ceramic bowls after the drying and is placed into glass containers in which the extraction of the calcium content will take place. After that the wet sand is placed again into the ceramic bowls in order to be dried again. The procedure of the transport of the sand from one container to another requires caution in order to make sure that the whole amount initially examined is taken into account. However, it may be expected that a small amount of soil can be disregarded during this procedure. This amount could lead to an error in the order of 0.01g.
- The extraction of the remaining HCl solution and of the demineralized water using the pipettes may lead to some soil particles flowing out too (especially the finer ones that are expected to need more time to settle in the bottom of the container). If this procedure is implemented with caution the error can be expected to be less than 0.05g.
- While handling the samples, sand from one sample could fall in the other and contaminate it. Normally, less than 0.05 g of soil is expected to contaminate another.
- During the sieving procedure, soil can get stuck in the various parts of the equipment used. The largest error can be expected to come from the sieves since fine particles of sand can easily get stuck in the openings of the sieves and if the cleaning is not performed with care, this could result in inaccuracies of up to 0.05g.
- The number of sieves used can alter the distribution of the resulting grain sizes. Using more sieves is obviously increasing the accuracy of the measurements. Additionally, the intensity of the vibrations of the machine is of an important role to the accuracy of the results.
- The quality of sieves plays also play an important role in the accuracy of the results. There are risks that the sieves have been contaminated by either wet soil, or damaged by wire sponges. This would lead to influences in the size of the openings in some parts of the sieves. However, the majority of the sieves are expected to be unaffected, therefore the inaccuracies can be considered very low, and may affect soil in the order of 0.18mm, but also smaller fractions of soil.
- The scale used when weighting the samples will influence the results. There might be measuring errors or rounding errors in the order of 0.01g and 0.005g respectively. Additionally, the fluctuation of the scale should be taken into account and it results in inaccuracies of up to 0.02g.
- The diameter defined in the sieving analysis was measured using only a part of the amount that was contained in the tubular soil sampler and since the soil is inhomogeneous, the samples used may not be representative enough.

5.3. Bathymetry

The bathymetry was only done on Tuesday 3 October and Wednesday 4th of October. The other days it was not possible to sail due to large waves. Two tracks are saved and presented in Figure 70. The blue track was sailed on Tuesday and the green track on Wednesday.

As can be seen in the figure the bathymetry in front of the northern beach was not measured. The waves were on both days too large to sail safely in the area in front of the northern beach. It was possible to sail in front of the southern beach, in front of the breakwater and the marina self. The bathymetry of these three areas will be analysed and are indicated in red in Figure 70.

Figure 70: Sailed tracks

5.3.1. South Beach

The bathymetry of the south beach is connected with the beach profiles measured on the south beach. The contour map can be seen in Figure 71. In this figure, three dashed lines are plotted. Along these lines, depth profiles are determined. These profiles are presented in Figure 72 are interpolated. The contour lines are plotted on the google earth image. In the figure can be seen that the waterline measured during the fieldwork different is then the waterline on google earth.

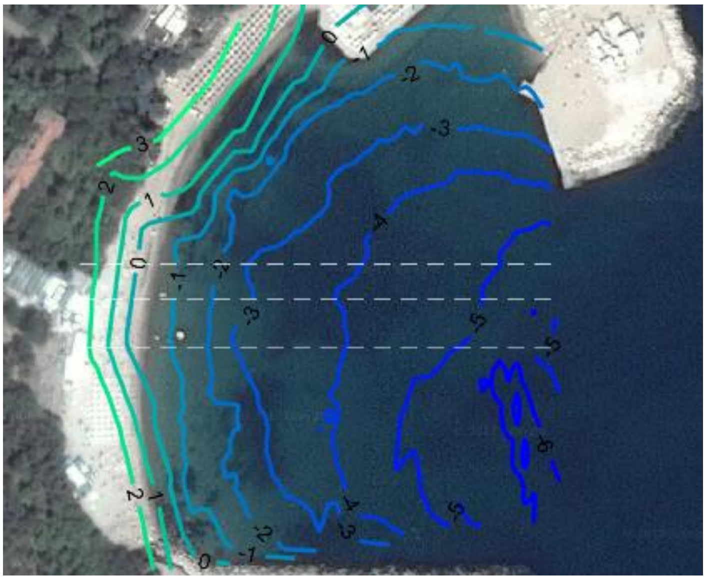


Figure 71: Contour map south beach

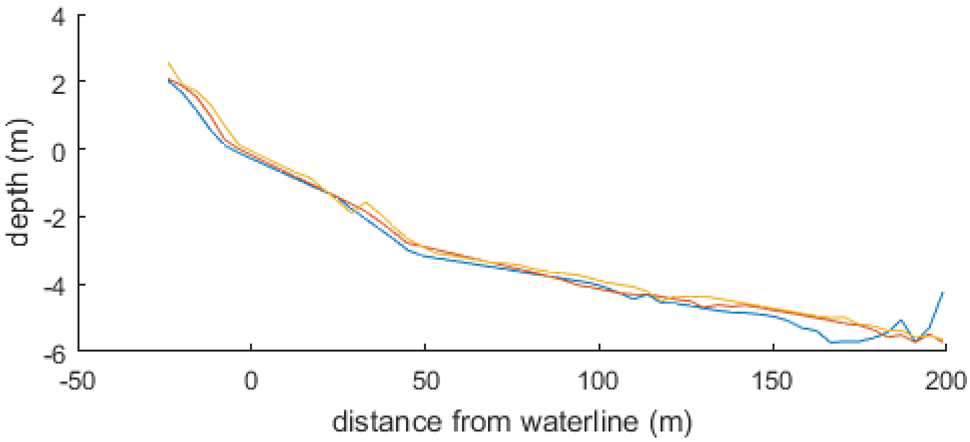


Figure 72: Beach profiles

5.3.2. Marina

The largest part of the marina is covered during the bathymetry survey. The depth map of the marina is shown in Figure 73. In this figure can be seen that in the largest part the depth is larger than 3.0 m, but in the northern corner and next to the breakwater the depth is less than 2.0m, which is indicated in yellow in the figure.

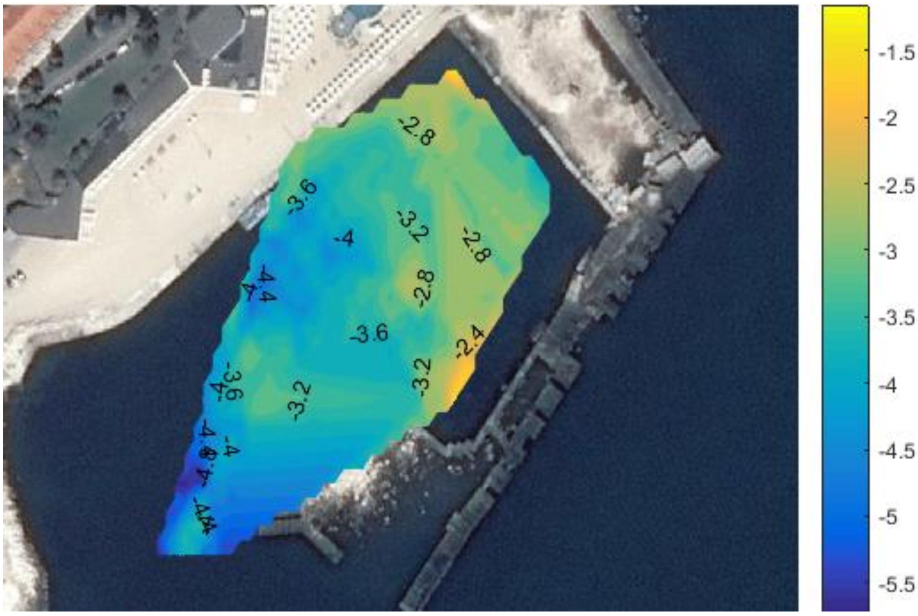


Figure 73: Depth map in marina

5.3.3. In front of the breakwater

Sailing close to the breakwater was not possible due to reflected and breaking waves. However, a reliable bathymetry map can be made for the area in front of the breakwater. This map is shown in Figure 74. The maximum measured depth is around 5m.

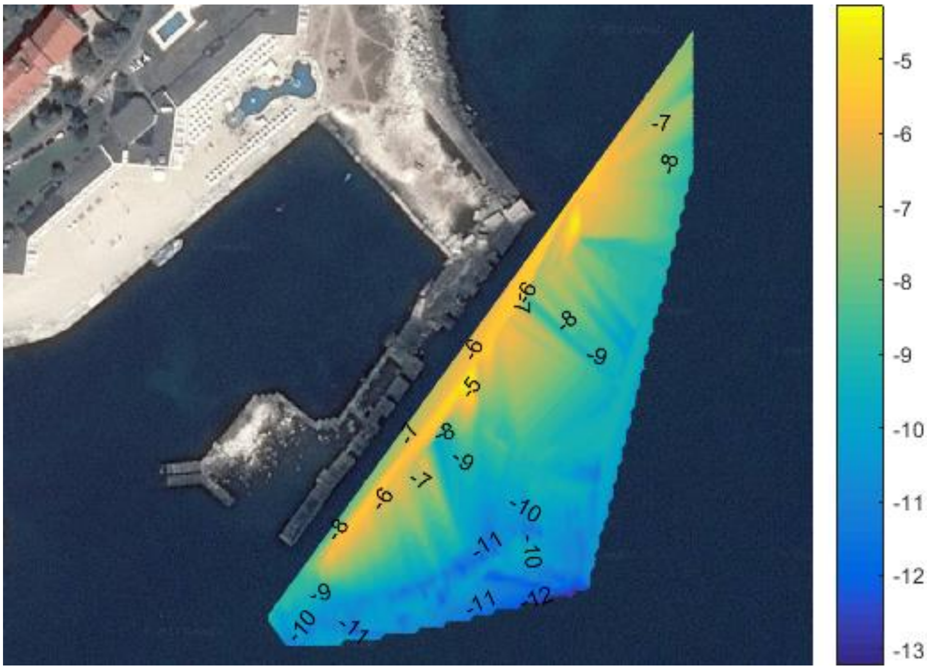


Figure 74: Bathymetry in front of the breakwater

5.3.4. Discussion

The bathymetry results cannot be compared with results of previous years because there are no good results obtained in previous years. Only some recommendation for further fieldwork can be done.

With the sailed tracks during the fieldwork, some bathymetric map can be made. However, a better bathymetry result could be obtained if the horizontal distance between the track lines was less. To decrease the distance between the track lines more lines in the alongshore direction are needed, which is only possible if there is enough time to sail safely in the several areas.

In front of the northern beach is a lot wave shoaling. This leads to high waves in this area. Sailing in front of the northern beach is often not possible even with a small offshore wave height. In the black Sea, the wave heights are often less in the morning than in the afternoon. Sailing close to the breakwater is also hard. This is because of reflection and break of waves on the breakwater.

During further fieldwork, it is better to start with the survey in front of the northern beach and close to the breakwater if the weather conditions are good in the morning.

5.4. Groyne

Following the profile measurement and the damage assessment of the present state of the groyne carried in situ as described in chapter 4.4, the results are presented below.

5.4.1. Profiles

In the pictures below the graphical results of the groyne can be found. It can be obtained that overall the results are approximately the same as previous years. In the following paragraphs the changes are mentioned.

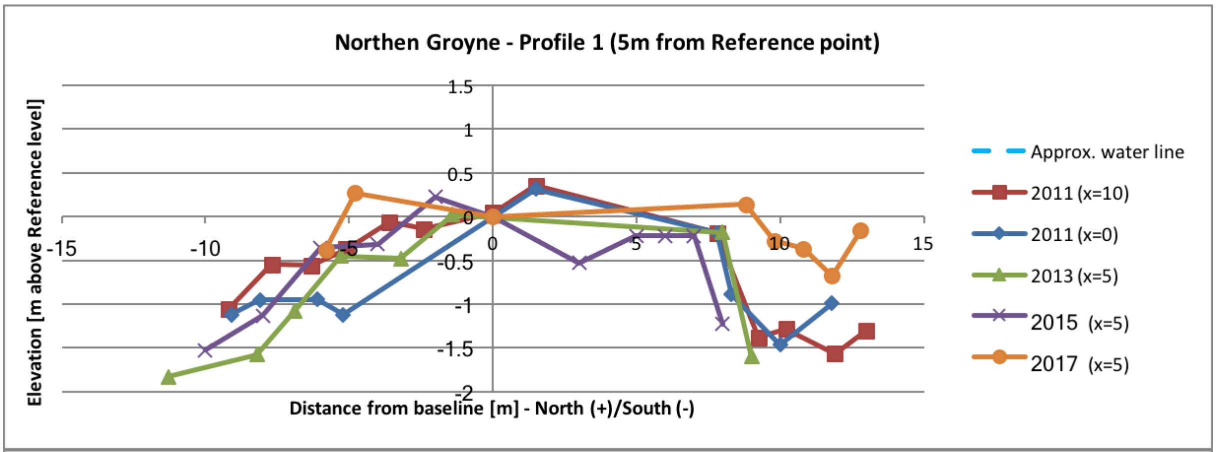
First at x = 5m, the measurements at + 8.8m and further to the north direction are higher than previous years. In the south direction a slight increase in height is noticed as well.

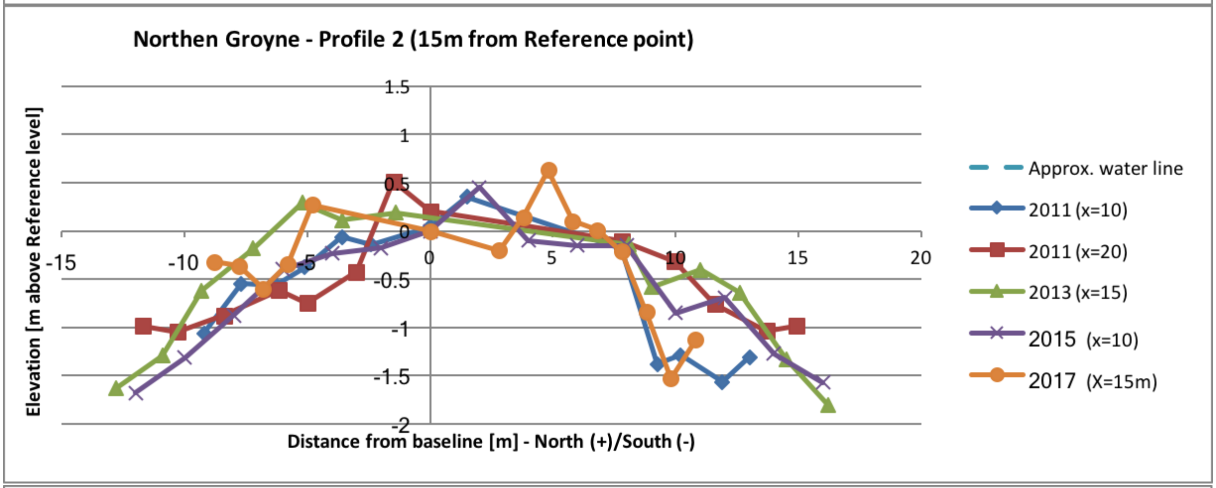
Secondly at x=15m, at +5 m from the baseline there is a peak in the measurements. Further the line seems to comply with previous years.

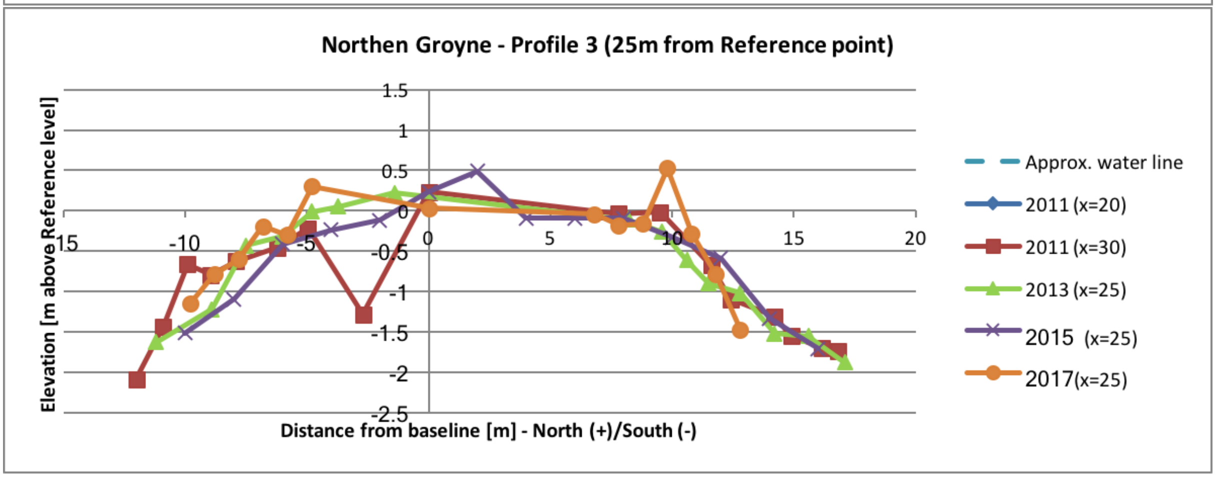
Thirdly at x=25m, at +10m from the base line there is a peak in the measurements. +0.5m elevation instead of the approximate 0 meter in previous years.

Fourthly at x=35, the line from 2017 is similar as the those from previous years, a small hump at -5m from the baseline is measured.

Fifthly at x=45m, no noticeable changes are measured in this line.
Sixthly at x=55m, line is similar.


Northen Groyne - Profile 1 (5m from Reference point)
Elevation [m above Reference level]
Distance from baseline [m] - North (+)/South (-)
Approx. water line
2011 (x=10)
2011 (x=0)
2013 (x=5)
2015 (x=5)
2017 (x=5)


Northen Groyne - Profile 2 (15m from Reference point)
Elevation [m above Reference level]
Distance from baseline [m] - North (+)/South (-)
Approx. water line
2011 (x=10)
2011 (x=20)
2013 (x=15)
2015 (x=10)
2017 (X=15m)


Northen Groyne - Profile 3 (25m from Reference point)
Elevation [m above Reference level]
Distance from baseline [m] - North (+)/South (-)
Approx. water line
2011 (x=20)
2011 (x=30)
2013 (x=25)
2015 (x=25)
2017(x=25)

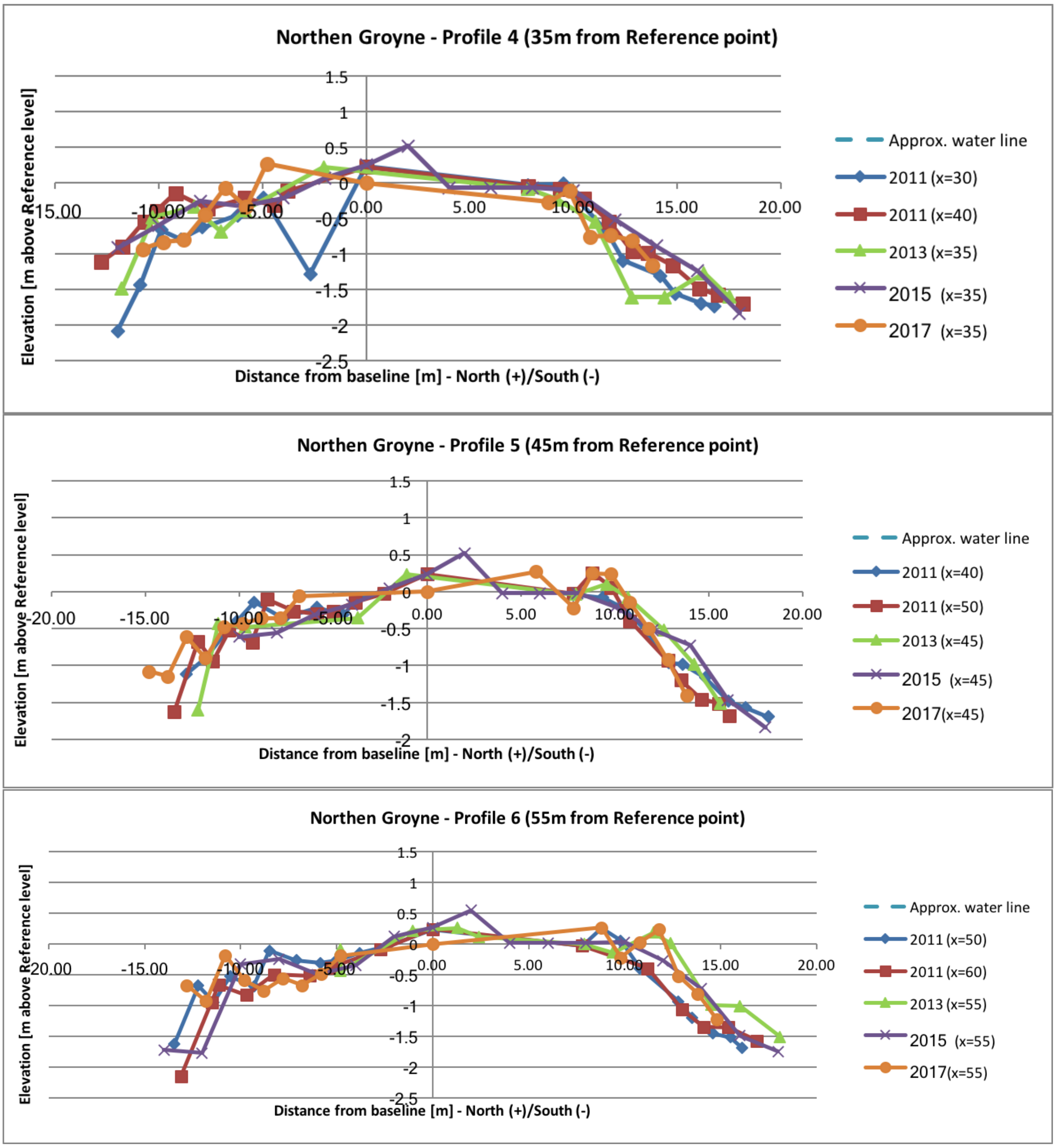


Figure 75: Measured groyne profiles of 2017 and earlier

5.4.2. Damage assessment

Following the visual inspection some remarks can be drawn and will be presented below. As stated also in the report of 2015 most visible damage has occurred from 2009 onwards. In this report, this year's photos were compared with the photos of 2015 and 2013 that were captured from the same positions.

- The southern side of the breakwater features a sloped concrete slab with rocks-tetrapods on it while the northern side consists of a vertical wall and a stack of rocks-tetrapods adjacent to it.
- On the breakwater rocks and tetrapods are mainly found on the southern side. The rocks present have a wide variety of diameters probably as a result of larger rocks breaking. On the northern side of the breakwater only larger pieces of rocks can be seen scattered on top of the breakwater.
- The seaward side of the breakwater is evidently heavily damaged. The concrete slabs there as well as many concrete elements that framed the sluice have been moved from their position,

while the remaining elements are severely corroded. The slope of the rocks on that side is very steep as the smaller rocks have been carried away. Especially at the northern end, where the wave attack is expected to be most severe most rocks have disappeared, tetrapods have been moved further below the waterline thus leaving the concrete caissons unprotected.

- In the photos that capture the tip of the groyne a significant advance of the damage can be observed. In 2013 the concrete deck was still in place, in 2015 there is a part of the deck missing, while in 2017 a bigger part seems to have been carried away under the wave attack. Let it be noted that pictures of the tip of the breakwater have not been captured from the same spot.
- Additionally, along the 1st profile concrete slabs have been lifted off the southern slope and transported to other parts of the breakwater. However, this part does not seem to have significantly deteriorated since 2015.
- The landward side of the groyne is better preserved. There the slope on both sides (northern and southern) is milder and smaller rocks are still in place.
- At several locations, especially on the southern side broken tetrapods can be observed, with the missing part scattered around as well as cracked rocks. The rock material used in this breakwater is of low quality and the required dimensions have been underestimated. As a result of rock cracking, the smaller rock parts cannot withstand the design wave attack or fulfil the structural function and are drifted away.
- Along the breakwater a sluice was constructed to accommodate the discharge of the local creek. The slabs that were covering this creek seem to have moved. The situation has deteriorated since 2015, now with a longer part of this sluice open. On the other hand, the rocks that were observed to have been carried in this sluice in 2015 and 2013 were not observed this year. This can be either attributed to a large storm event or to human intervention in order to deblock the sluice.

5.4.3. Discussion

Two different measurements have been performed, profile measurements and the damage assessment. Within this discussion the accuracy and recommendations for these measurements is provided

First, profile measurements for the accuracy depends on the equipment and the expertise of the person conducting the measurements. Also the weather conditions and visibility are limiting factors. It was the first time the sphere was used as the base of the Jalon combined with the levelling instrument. Because the person conducting the experiment was familiar with the use the sphere, some errors are expected to appear in the results. Within this year's profiles there are more peaks than in the profiles of previous years. This can be traced back either to natural or artificial transfer of material within the groyne profiles or to mistakes in the measuring process.

Second, damage assessment since it is done visually the accuracy is not that high. Because you cannot take the same photo's every year. The visual part makes it hard to see all parts of the groyne, due to some large tetrapods, stones and due to the slope itself. Damage assessment is based not only on the photo's but also experience because on this the results depend on the expertise of the person conducting the measurements. There are two limiting factors during the measurements both weather and the state of the deck of groyne. Due to high waves it is impossible to get on every part of the groyne since it is too dangerous.

In the case of increased accuracy demand, it is recommended to use a combination of methods to assess the current state of the groyne. For example, satellite images and remote sensing software could be used in combination with in situ measurements to obtain reasonable estimation of the groyne profile. Also the use of a "GO PRO" camera is advisable for data collection on the current underwater status of the groyne. In this way, both damages and ecology could be assessed.

5.5. Marina

As described in chapter **Error! Reference source not found.**, three ways of wave height assessment w ere used: visual estimation, wave height information provided from Svasék and combined with a SWAN model of the area as well as in situ measurements using wave loggers. The results of the first two will be presented in the following paragraphs. Unfortunately, no results from the wave loggers will be included in this report due to the malfunction of the instruments.

5.5.1. Visual wave measurements

The obtained results can be found in the appendix for the two separate measurements. The wave height is most likely Rayleigh distributed. The probability density distribution function for waves in deep water is (Holthuijsen, 2007):

$$p(H) = \frac{4H}{H^2_{1/3}} \exp[-2(\frac{H}{H_{1/3}})^2]$$

Where H1/3 is the measured significant wave height and H a variable. The H1/3 is the average wave height of the 1/3th highest waves. Doing this for the achieved wave data results in the probability density function in Figure 76. On day 1 the time in between two measurements is not too big, for that reason all of the different wave blocks has been taken as one single measurement. On day 2, the measurements were divided into blocks of 40 minutes. The significant wave height and period are presented in Table 17.

Measurementblock		1	2	3
Day 1	H1/3 (m)	1.57		
	Tm (s)	6.73		
Day 2	H1/3 (m)	1.84	1.82	1.63
	Tm (s)	7.76	7.53	8.37

Table 17: Measured Hs different measurements

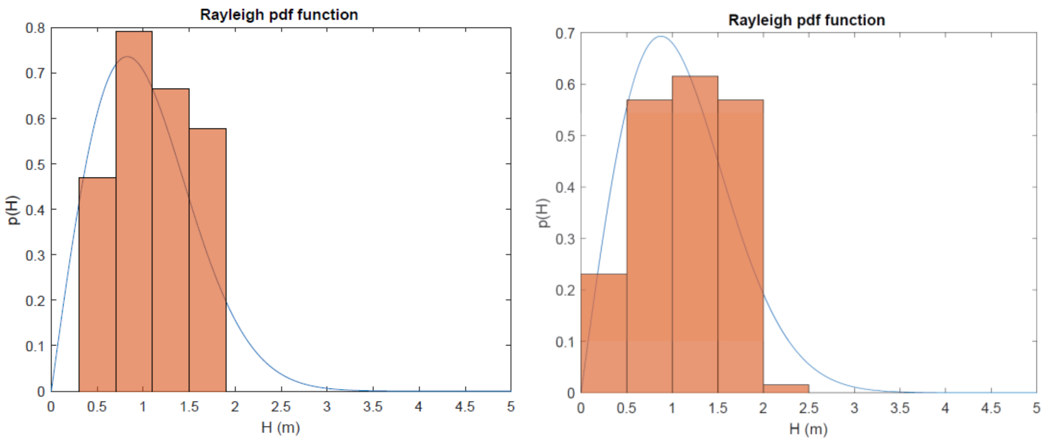


Figure 76: Rayleigh pdf Day 1 (left) and Day 2 (right)

The cdf function for Rayleigh in shallow water equals (Holthuijsen, 2007):

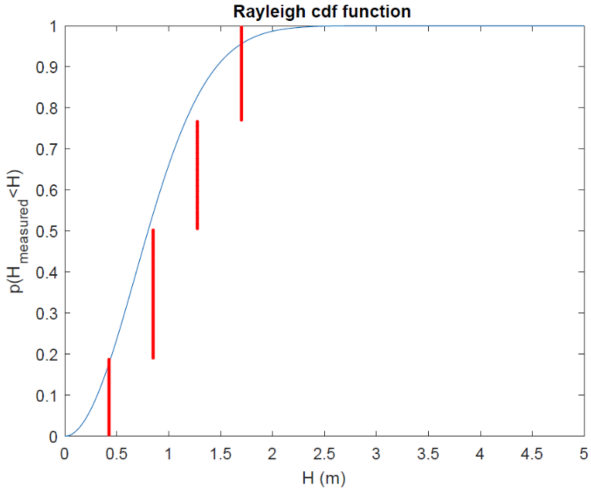
$$\Pr\{\underline{H} < H\} = 1 - \exp[-2(\frac{H}{H_{ch,i}})^{ki}]$$

Where a k value of 2 is recommended for wave height smaller than the transition wave height and a value of 3.6 is recommended larger than H_{tr}. H_{tr} is the breaking wave height is approximately 0.88 h_b for linear wave theory and 0.78 h_b for non-linear wave theory (Bosboom, 2016), where h_b is the water depth at the break point. The waterdepth at the measure point is set to 5 meters (Navionics, 2017). The characteristic wave height before breaking equals the root mean squared measured wave height and the characteristic wave height after breaking equals the transition wave height. In **Error! Reference s**

ource not found. he cumulative distribution function of the first day can be found. The probability of the data points can be determined using:

$$\Pr\{\underline{H} < H\} = \frac{number}{totalwaves + 1} \qquad MSE = \frac{1}{n}\sum_{i=1}^{n}(\hat{X}_i - X_i)^2$$

To check whether it was right to choose for a Rayleigh distribution, one has to determine the mean squared error. This results in a mean squared error of 0.0170, which is quite high, thus the Rayleigh distribution is doubtful. The mean squared error for day 2 for this distribution equals 0.0037, which is close to zero, thus the Rayleigh distribution seems to be a good estimation.



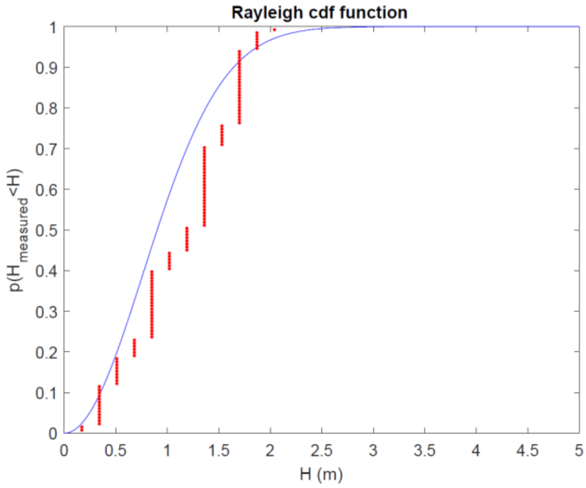


Figure 77: cdf Rayleigh pdf day 1 (right) and day 2 (left)

The maximum wave height can be estimated using the distribution (Holthuijzen, 2007):

$$\Pr\{\underline{H}_{\max} \leq H\} = \Pr\{\underline{H} < H\}^{N}$$

Where N is the total number of waves in the dataset. Plotting these results using the cdf function mentioned in the previous paragraph results in Figure 78, from which follows that the maximum expected wave height at this water depth, according the obtained visual data is approximately 2.5 to 3 meters.

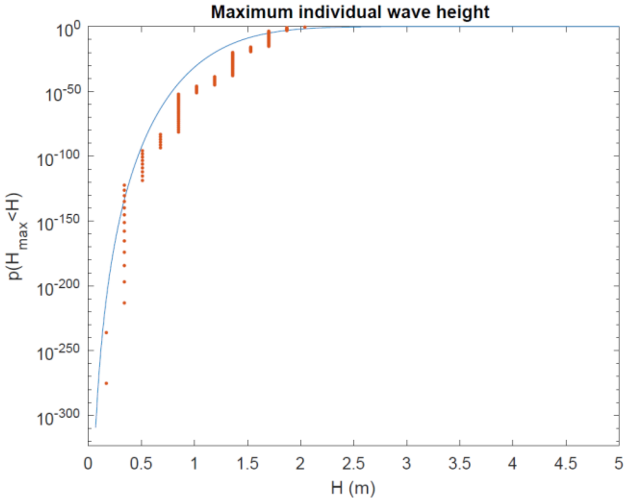


Figure 78: Maximum individual wave height from data

5.5.2. Wave height from SWAN analysis and Svasek results

The Svašek weather station is located 530m offshore at a depth of 15m. We assume constant wind and wave conditions over depth contours. Offshore variables are determined from the data for severe conditions at the 15m depth contour (corresponding to 28 and 29 September 2017) and mild conditions at the 15m depth contour (corresponding to 3 and 4 October 2017). Using Valchev, Andreeva and Prodanov (2014), also extreme conditions at the 15m depth contour have been assumed. These variables have been put into SWAN, to generate the significant wave height, mean wave period, water depth and the wave height of the highest 2% of the waves at both the jetty (location of visual wave measurements) and the breakwater. Also, the energy spectra have been made up at several cross-shore locations.

5.5.2.1. Severe conditions

Variable	Value
Water depth	15 m
Significant wave height	2.0 m
Peak period	9 s
Wind direction	NE
Wave direction	ENE
Wind set-up	0.4 m
Wind speed	11 m/s

Table 18: Variables for severe conditions at 15m depth contour

The wind set-up has been measured at the quay wall in the marina and is assumed to be equal at the 15m depth contour.

Variable	Value
Shore normal angle	280ºN
Offshore distance	560m

Table 19: Variables for the proportion of the location of the jetty to the 15m depth contour

A depth profile has been determined using Navionics. From the figures in Appendix G, it can be seen that the waves break approximately at the tip of the jetty. The wave height slowly decreases when a wave approaches the coast and then suddenly increases due to the decrease in water depth. The visual wave measurements resulted in a significant wave height of about 1.8m, which is almost equal to the wave height resulting from the data from the weather station. The same holds for the spectral wave period. Therefore, it can be said that the visual wave measurements are a good approximation for the determination of the wave characteristics.

The variance-density spectrum corresponding to severe conditions at the jetty is relatively wide, corresponding to a wind sea state. At 470m from the offshore boundary, the waves break and therefore lose energy. Also, due to triad wave-wave interactions, a second peak is formed at the higher frequencies when moving in onshore direction.

Variable	Value
Shore normal angle	300ºN
Offshore distance	355m

Table 20: Variables for the proportion of the location of the breakwater to the 15m depth contour

The same kind of wave progress can be seen at the location of the breakwater. However, the shoaling process is abruptly cut off by the breakwater, leading to plunging breakers. The energy spectra are almost the same as those at the location of the jetty. The significant wave height at the breakwater is about 0.7m, but the wave energy at the breakwater is still very high. This will lead to a lot of run-up when constructing a new breakwater.

5.5.2.2. Mild condition

Variable	Value
Water depth	15 m
Significant wave height	0.6 m
Peak period	7 s
Wind direction	SSE
Wave direction	E
Wind set-up	0 m
Wind speed	3 m/s

Table 21: Variables for severe conditions at 15m depth contour

The wind set-up for mild conditions was negligibly small and is therefore not taken into account. The increase of wave height just before breaking is higher than for severe conditions, caused by a lower height to depth ratio. Waves contain a lot less energy and the second spectral peak due to triads is less visible. The significant wave height at the breakwater is only a little lower than for severe conditions (0.6m), but the broken wave contains only half of the energy for mild conditions as for severe conditions.

5.5.2.3. Extreme conditions

Variable	Value
Water depth	15 m
Significant wave height	5.0 m
Peak period	10 s
Wind direction	70ºN
Wave direction	70ºN
Wind set-up	1.0 m
Wind speed	30 m/s

Table 22: Variables for severe conditions at 15m depth contour

The wind set-up of 1.0m is an assumption based on the proportion of other wind characteristics with the mild and severe conditions. Due to the extreme wave heights, the wave heights only decrease before breaking. The significant wave height at the breakwater is still equal to 2.2 metres and the waves still contain a huge amount of energy when arriving at the breakwater (about eight times more than for severe conditions). Therefore, the waves collapse into the breakwater. The run-up for the new breakwater will therefore be extremely large.

5.5.2.4. Svasek data validation

For the completion of the preliminary design of the breakwater and the marina, significant wave height is of undeniable importance. During the fieldwork wave height measurements were taken by visual observations as well as deployment of wave loggers. However, it was not possible to keep a daily wave record throughout all the period of our stay in Bulgaria due to the many measurement tasks to be performed in a short time span. For that reason, the wave data considered for the design calculations were derived from Svasek.

Svasek has points all over the world where observations are made based on which a daily forecast is provided. The wave data we used refer to a point 530 m offshore in the Black Sea at a depth of 15 m. Among others, this report aims to validate the accuracy of the Svasek predictions using our own wave measurements. It should be noted that only the visual observations are used for the data validation since due to technical difficulties the wave logger records were damaged thus not usable.

The visual observations were acquired during the first two days of our stay in Bulgaria, namely on September 29 and 30. They refer to the waves reaching the jetty. After processing they result in a significant wave height of 1.57 m with a mean wave period of 6.73 s for the first day and a wave height of 1.76 m with a significant wave period of 7.9 s for the second day respectively.

The Svasek data don't distinguish for those two days and they give a significant wave height of 1.9 m with a period of 8.1s. It should be mentioned that the above result is obtained after transforming the waves at the offshore point in the nearshore using the program SWAN.

The aforementioned values are summarized in Table 23.

WAVE CHARACTERISTICS	HS (M)	TS (S)
VISUAL (DAY 1)	1.57	6.73 (mean)
VISUAL (DAY 2)	1.76	7.9
SVASEK	1.9	8.1

Table 23: Wace characteristics from Svasek and visual observations.

As one can see, the values are in the same order of magnitude and quite close. In particular day 2 is considered more accurate since its results are based on multiple blocks. In conclusion, based on the available data, the Svasek data are in accordance with the measurements.

5.5.3. Wave height from wave loggers

Since the wave loggers did not work properly there are no results from these wave loggers. The reason for this is discussion in chapter 5.5.3.

5.5.3. Discussion

Already from the post-processing that started in Varna, it was evident that something with the wave data was wrong. The resulting data log files where either empty or irregularly small, making it impossible to retrieve any wave information from them, not to mention accurate ones.

Upon our arrival in Delft we requested from Boyan to send us the remaining calibration data that we there not able to acquire. Unfortunately, the results were also unsatisfactory. After running several tests, he discovered that the problem was a loose connection in the slots of both SD cards. Because of this, the log file was either too short or missing.

More specifically, after a number of opening and closing the slot, firm contact was lost. It must be noted that these loggers are low cost devices and most components are simple and cheap. This applies for the type of the slot for the SD card as well.

Afterwards, the SD cards in the slots were fixed in such a way that was difficult to take them out. Finally, the loggers were ready for a one-time deployed at the breakwater.

Two wave loggers were deployed in October 25, by Boyan both offshore and onshore of the breakwater aiming to get new datalog files. Due to technical problems and misfortunes the procedure (safe retrieval of the loggers, post-processing) could not be completed, so that wave date from the loggers are not available on the current report.

When it comes to field measurements one can face many difficulties related to the human factor or not. In this case, a preliminary inspection of the equipment could reveal possible malfunctions of the loggers.

5.6. Breakwater

The assessment of the existing breakwater condition was conducted through identification of the current material types in combination with estimation of the quantity and quality of the materials available on the shore and on the vicinity of the breakwater that should be removed. Moreover, the notion of reusing a part of those materials was explored for the construction of the new breakwater. The material types that were observed consist of tetrapods, concrete blocks, rock and concrete debris.

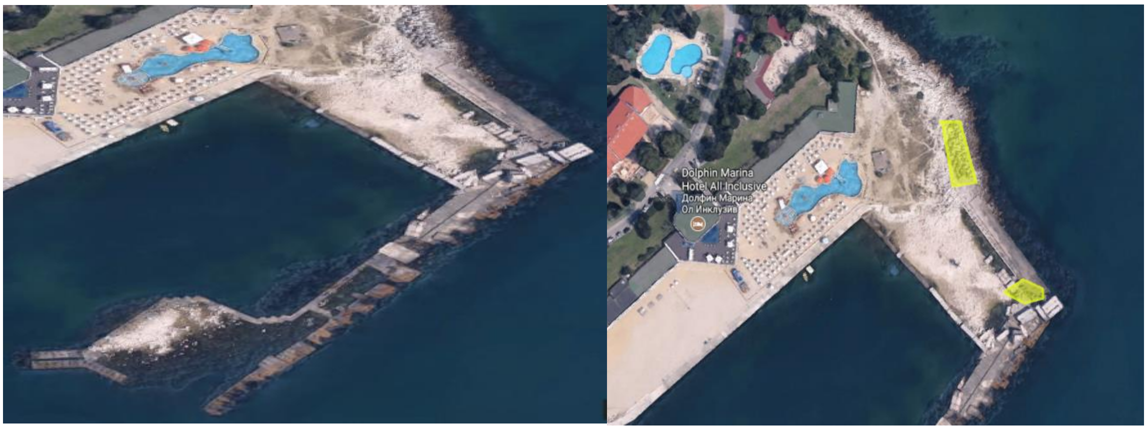


Figure 79: Top view of existing breakwater (left) tetrapod locations (right)

5.6.1. Tetrapods

After visual inspection it was observed that tetrapods are located on the outer edge of the most landward part of the breakwater as well as in its curve. The areas can be observed highlighted in the following aerial image.

The total amount of tetrapods was estimated visually in combination with Google Earth satellite images, since the weather conditions did not enable a closer approach. This explains the difference in the total amount of tetrapods that were inspected compared to previous years, as probably buried or semi-submerged tetrapods were not able to be identified. In the outer edge of the most landward end of the breakwater approximately 80 tetrapods were counted, allowing for a marginal error of 5 tetrapods.

Two characteristic dimensions of the tetrapods were measured with the use of a measuring tape, for the purpose of conducting a volumetric and a mass calculation. The tetrapod height in the outer edge (H) was measured 2.40 m while the length of the leg (C) was 1.20 m. According to the Shore Protection Manual (1984) formulas the volume (V) and mass (M) of the tetrapods can be computed as follows:

$$V = 0.28 \cdot H^3 \quad [m^3]$$
$$M = \rho \cdot V \qquad [tn]$$

The resulting amounts per tetrapod were found to be V=3.87 m3/tetrapod and M=8.7 tn/tetrapod, with a density value equal to ρ=2.25 tn/m3, since the existing aggregate type is limestone which is lighter than the usual aggregate. The aforementioned results refer to the most landward part. For the second location where tetrapods were observed, it must be noted that it was not possible to access the outer edge of the breakwater. Since the tetrapods located there were noticeably smaller than the ones measured, their volume was visually estimated to be 80% of the volume of the measured ones. Approximately 15 of these tetrapods were observed at the second location.

In terms of quality inspection it should be noted that the quality of the tetrapods was not considered good enough for reuse in their original form. Either due to weathering damage or due to poor construction techniques, the concrete cover thickness was found to be very thin or the aggregates (including shells) were exposed. The tetrapods located at the elbow of the breakwater appear to be smoother and generally in a better condition. There were tetrapods with evident damage (leg cut-off, split in the middle) as can be seen in images 3-5. Additionally, close to the shore tetrapods were attached on concrete that was allegedly dumped there after the construction of the neighbouring hotels. During the inspection there was a wind surge (roughly estimated at 30 cm) and the tetrapods were partly submerged. However, from Google Earth inspection it seems that these tetrapods are on the dry during calm wave conditions.

Figure 80: Tetrapod leg cut-off

Figure 81: Visual aggregates on tetrapods

Figure 82: Broken tetrapod leg

Taking all the aforementioned observations into account, it was deemed more appropriate to reuse the tetrapods grinded as core material for the breakwater. Assuming that 60% of the entire amount of material will be utilized, the total volume of the material that can be used and the total mass are estimated to be $V_{tot} = 0.6 \cdot 387 = 232.2\, m^3$ and $M_{tot} = 0.6 \cdot 8700 = 5220\, tn$, considering a total number of 100 tetrapods for both locations.

5.6.2. Rock

Rocks were located at the most landward part of the breakwater, with a wide spread over the area. The material was identified as limestone (Baltic rock). In order to determine the amount of rock located at the area, the length, width and height of 27 samples were measured by measuring tape, giving the following results, displayed in Table 24.

Number	L (m)	W (m)	H (m)	V (m^3)	d_{50} (m)	%
1	20	38	10	0.0076	0.197	4
2	45	21	20	0.0189	0.266	7
3	40	20	30	0.024	0.288	15
4	42	25	30	0.0315	0.316	15
5	40	27	30	0.0324	0.319	19
6	52	30	25	0.039	0.339	22
7	40	34	29	0.03944	0.34	26
8	70	40	15	0.042	0.348	30
9	100	55	10	0.055	0.38	33
10	46	40	30	0.0552	0.381	37
11	59	25	38	0.05605	0.383	41
12	90	25	25	0.05625	0.383	44
13	52	29	44	0.066352	0.405	48
14	53	45	30	0.07155	0.415	52
15	60	32	38	0.07296	0.418	56
16	60	50	25	0.075	0.422	59
17	64	34	39	0.084864	0.439	63
18	56	34	47	0.089488	0.447	67
19	53	67	29	0.102979	0.468	70
20	48	60	40	0.1152	0.487	74
21	110	70	15	0.1155	0.487	78
22	80	60	25	0.12	0.493	81
23	62	55	47	0.16027	0.543	85
24	60	55	52	0.1716	0.556	89
25	49	60	62	0.18228	0.567	93
26	164	45	26	0.19188	0.577	96
27	56	80	48	0.21504	0.599	100

Table 24: Rock characteristics

The level of accuracy of the conducted method of volume calculation is significantly low, due to the fact that the maximum dimensions of the rocks were used, thus, overestimating the result. In addition, the measurements were conducted under storm conditions with high wind and waves, so the precision further decreases. Moreover, the number of samples is considered low and only surface stones were taken into account. The volume and the sieve diameter d50 were calculated per rock. The results are summarized in the following table. Finally, a “sieve curve” of the rock sample was determined and is presented in the graph below (Figure 83). The average diameter of the sample was found to be 0.45 m.

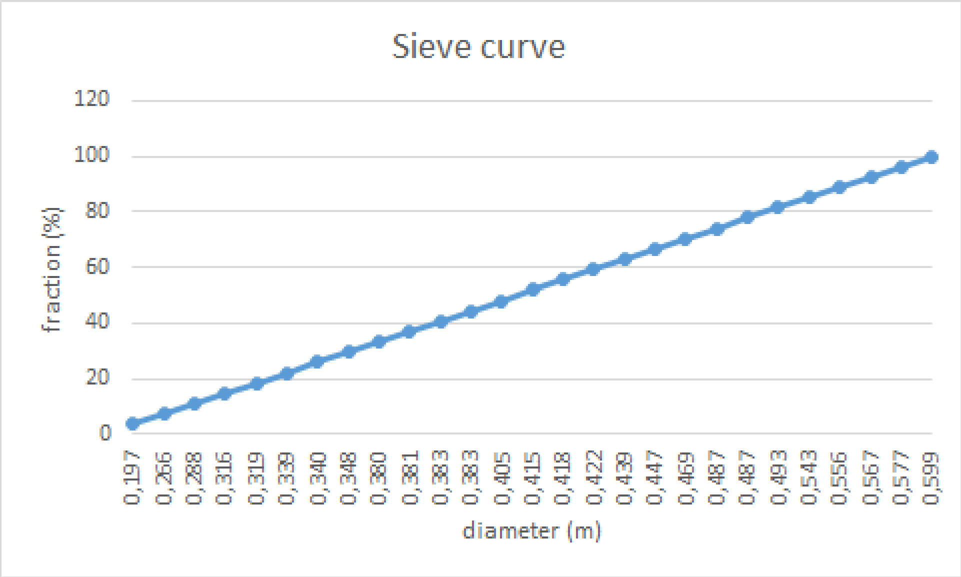


Figure 83: Sieve curve of rock samples

5.6.3. Concrete blocks

The concrete blocks were obtained from visual observations can be seen with yellow colour in Figure 84 below. Furthermore, their dimensions and associated volumes were calculated as can be seen in Table 25 below.

Figure 84: Overview of concrete blocks on the breakwater

Type of concrete blocks	Amount	Dimensions (m)	Concrete volume(m^3)/per block	Accuracy (m)
Large	31	10*3*0.9	27	+/- 0.2
Small	11	3*3*0.9	8.1	+/- 0.2

Table 25: Inventory of concrete blocks on the breakwater

The concrete blocks were damaged (some, cut in half) and with insufficient concrete cover. The rebars could be seen and were corroded. The amount of usable material was estimated up to 550m^3 (12237.5 tn) which represents the visible and counted concrete blocks while a percentage of about 40% is assumed to be the losses of materials.

5.6.4. Concrete debris

In order to estimate the amount of reusable material from the hotels that will be demolished, a trial was made to count the number of columns and slabs from the outside and measure their corresponding dimensions (length, width and thickness). However, since the blueprint of the internal of the hotel was not accessible there was no way of knowing the internal format of the building therefore a different method was applied. An estimate of the overall dimensions of the Dolphin Marina Hotel was made using Google Earth and on-site measurement which will be afterwards related to the volume of concrete of the structure. The indicated area can be seen in Figure 85 and Table 26 provides the data extraction.

For the coupling of structure volume with the concrete volume, the ratio of concrete volume over total structural volume was roughly estimated equal to 13%. This result is based on the examination of a simple model structure common values were chosen for the spacing between the columns and slabs as well as for the dimensions of the structural elements. Moreover, assuming that from the total volume of the concrete of the structure 60% remains after the demolition (40% is lost in the air or remains wrapped around the reinforcement) the final estimation is 3337.35m^3.

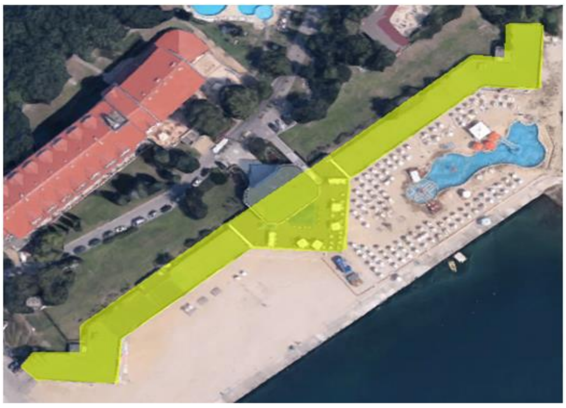


Figure 85: Overview of Dolphin Marina Hotel

Estimated total surface (m^2)	3688.5
Total floor height (m) including slabs	11.6
Total volume (m^3)	42786.6
Concrete Factor	0.13
Total concrete volume (m^3)	5562.258

Table 26: Inventory of concrete debris

5.7. Quarry

In the following paragraphs the characteristics of the quarry rocks will be presented as they resulted through procedures of rock sampling and analysis. A short discussion will follow.

As far as the elongation and blockiness characteristics of the rock samples, the reader is referred to Table 39 in Appendix H. Main results are that the elongation of the 39 stones picked is on average 3.1 and the blockiness is equal to 34.1%.

The average density of the stone samples that we took is 2477.33 [kg/m3], the calculation sheet can be found on appendix H. One should note that item Quarry_2 is an extreme outlier and therefore is neglected. This value can be different to the other due to measurement or human error. The material of which the stones are made is limestone. The density of limestone is, according to ThoughtCo. (2017), within 2300-2700 [kg/m3]. So, we could say that the density is within the theoretical limits of limestone. The dn50 of the stones is 0.18 [m]. The calculation-sheet is presented in Appendix A.

The results over the last few years have been presented in Table 27. The density that was acquired during the Fieldwork 2017 is in line with the densities found in previous years. In 2016 a relatively high density has been found, it isn't really clear why this has occurred. But the density which was determined during this year's fieldwork seems reasonable if 2016 has been taken out of consideration. One point that has to be highlighted is the d_{n50} of this year's samples is one of the smallest over the last few years. A simple conclusion is that smaller stones have been picked as samples compared to last year. Another interesting point is that the blockiness is the smallest over the last few years and the elongation one of the largest, which can be elaborated by the fact that we seem to have chosen rectangular shaped stones.

YEAR	DENSITY [KG/M^3]	BLOCKINESS [%]	ELONGATION	D_{n50} [M]
2017	2477	33.1	3.10	0.18
2016	2737	50.0	2.35	0.15
2015	2426	43.9	2.04	0.28
2014	2347	53.0	2.50	0.20
2013	2537	38.4	3.33	0.20
2012	2272	45.4	2.00	0.21

Table 27: Results over the last few years

6. FEASIBILITY DESIGN

In the 6th chapter of this report the assessment of the feasibility of the design presented in chapter 2 will be carried out based on the data collected and processed during this course. This feasibility assessment will be conducted mainly from a structural and functional point of view as we want to ensure a rehabilitation plan that will fulfil the desired functions with structural integrity under the existing loads. Aspects that are also significant are the constructability of the proposed structures as well as the potential effects of them on the ecological status of the region. Due to data shortage the economic feasibility of the proposed solution will not be assessed in the present report.

6.1. Groyne

The visual damage inspection data and the design wave height calculations based on the available information of the rock and tetrapods on site, led us to the conclusion that the damage has progressed a lot over the last 6 years, and the present condition of the breakwater is not resistant to impact during storms.

More specifically, both sizes of tetrapods appear to be undersized for the depth limited wave that attacks the armour layer. This is revealed also by the visual observations of severed and displaced tetrapods. Most of the tetrapods near the tip of the breakwater are observed below the waterline, a fact that suggests either failure of the toe or shifting of the top tetrapods due to wave attack. Additionally, the concrete used for the tetrapods and the casting method do not live up to the quality standards.

The trunk of the breakwater is protected by rocks that are at present too small to withstand the wave attack. Since no information from the initial design is available, it is possible that the initial decided size was sufficient but due to the weak nature of this specific rock, more fractioning than expected has taken place. Of course, the fractioning accelerates the damage even more. The fractioned parts are swept away from their position during energetic wave conditions resulting in profiles with slope that is getting steeper especially on the seaside and towards the tip of the breakwater.

The caissons that comprise the skeleton of this structure have withstood the wave attack of the last decade without compromising the structural integrity of the groyne. Damage from wave attack evidence is obvious. Many concrete pieces of the deck of the structure have been blown off, the reinforcement is uncovered in several places and there is extensive corrosion. However, the continuously weakening armour layer is bound to put more stress on the caissons. There are already signs of weakening of the concrete parts due to abrasion. Additionally, stability problems due to scour holes may occur.

Taking into consideration the fact that this groyne is part of the coastal front of a touristic resort it goes without saying that keeping this badly maintained and damaged structure downgrades the landscape of the general area and reduces the touristic/ recreational value. Additionally, from the available evidence it is expected that the structure will sustain severe damage in the close future that may also lead to loss of structural stability.

The northern groyne was designed to protect the Northern Beach from energetic wave conditions and interrupt the alongshore sediment transport. According to previous reports the beach has been retreating for at least 20 years and this retreat can be evidenced in the structural interventions that have taken place there over the years: the construction of the revetment and the extension of the stairs at the northern part of the beach. Whether or not the groyne in its present damaged condition offers reduced protection is not clear. For this a model of the wave and flow around the groyne would be required. However, it is certain that if the groyne is not amended, the continuous and accelerated deterioration of the structure will reduce the protection of the area.

The repair of the Northern groyne should therefore be included as a part of the rehabilitation plan of the northern beach.

6.2. Marina

The marina design in the report of 2016 was based on the following objectives:

- Minimize wave penetration into the marina in summer seasons
- Withstand winter storm conditions with minimum damage
- Safe navigation for motorized luxury boat

The proposed design in the report of 2016 (Fieldwork Coastal Engineering, 2016) is shown in the **Error! Reference source not found.**. Concrete slabs will be used to repair the existing breakwater. A new groyne will be made of riprap or tetrapods, which are already available, inside the marina. This groyne will reduce the entrance width and it will reduce the wave penetration into the marina. The groyne will also block sediment transport and it prevents sedimentation into the marina. Next to the groyne a ramp will be created. Four piers are presented in the design to berth some motorized boats.

6.2.1. Wave conditions

Utilization of the local wave field is crucial in order to obtain the necessary wave data for the assessment. The marina design of 2016 was based on the BTM ARGOSS wave data that cover the period 1992-2013 and contain the seasonal and inter-annual wave height variation, as well as the wave height rose plot.

The dominant phenomena inside the marina are reflection and diffraction and a visible demonstration is displayed in Figure 86. The image presents the existing wave conditions of the marina and proves the existence of increased wave heights inside the marina. This is because the waves diffract in the tip of breakwater and also reflected in the marina entrance. The layout of the new design must restrict these phenomena.

Figure 86: Dominant wave directions in the area of the marina

The wave data that are used for the design assessment were obtained from SVASEK Hydraulics and concern the South and South-East wave directions, which served as the basis for the design, representing the worst scenarios that create the worst penetration inside the marina.

The associated wave conditions were obtained as representatives for the summer period which is the most demanding time in terms of use for the marina. For every wave direction a single wave was selected. The obtained wave conditions are presented in the table below.

Wave direction	Wave height [m]	Wave period [sec]
S	1.5	6
SE	1.5	6

Table 28: Representative waves for assessment of the marina design

Waves start to diffract in the tip of the breakwater, which is considered as an impermeable and semi-infinite structure. To obtain the diffracted wave height inside the marina, a diffraction coefficient needs to be calculated, for each of the representative waves.

The diffraction coefficient was calculated using the CRESS calculator software by considering the exact position given from last year's report. The entrance points from the tip of the breakwater was considered equal to 63m and 10m in y and x direction respectively. The incidence wave angles were found using the SWAN model equal to 180°S and 135°SE. The obtained results are depicted in the table below.

Wave direction	Wave height [m]	Diffraction coefficient K_d	Diffracted wave height [m]
S	1.5	0.1	0.15
SE	1.5	0.14	0.21

Table 29: Swan results for incidence wave angles 180°S and 135°SE.

Even though the diffraction phenomenon plays an important role inside the marina the reflection phenomenon should also be considered. Inside the marina the wave height increases due to reflection and decreases due to diffraction. Thus, the above calculated diffracted wave heights could be larger and for that reason one can conclude that inside the marina unfavourable conditions occur.

6.2.2. Marina design

With the design that was proposed by the students that carried out the Bulgarian Fieldwork in 2016 (Fieldwork Coastal Engineering, 2016) and the Google Earth measuring tool (Google Earth, 2017), an estimation of the dimensions of the design has been made. The students of last year didn't specify the dimensions, so to verify the design we make an estimation. The estimated dimensions are shown in Figure 87. The dimensions of this design will be checked for a motorized luxury boat with a draft of 2 meters, like proposed in the report of last year. The dimensions of the standard yacht are:

- Length of 12.5m;
- Width of 4.0m; and
- Draft of 2.0m

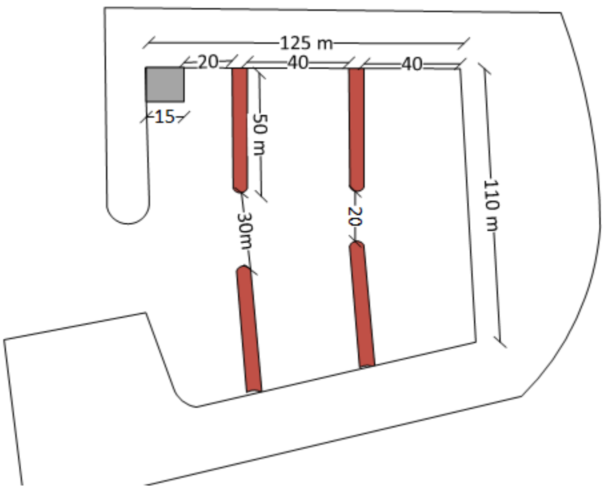


Figure 87: Estimated dimensions of the Marina

6.2.2.1. Piers

In the design four piers are presented. there are no dimensions given of these piers in the report. It is not clear if these are floating or fixed piers. From the above estimated dimensions and the standard design guidelines of Ports and Terminals (2012) is followed:

- The length of the piers is 50 meters, following the guidelines only a maximum length is specified. This maximum length of 200 meters isn't reached so this is sufficient.
- The standard with of piers is 1.8 meters, so this number will be taken into account.
- Due to the fact that the maximum possible water level difference during summer is smaller than 1.0 meters fixed piers are allowed.

6.2.2.2. Main Interior channel

The interior channel of a marina should be 1.5-1.75 times the length of a yacht or at least 20m for standard yachts. Preferred is width of 25m for the interior channel. The smallest part of the main interior is between the two northern. This distance is 20m. The width of the main interior channel is thus sufficient.

6.2.2.3. Fairways near berths

The minimum design width of fairways near berths is 1.5 times the length of a yacht. For this marina holds thus a minimum width of 18.75m. The total width between the two piers is 40m. This means that it is not possible to berth perpendicular to the piers two in the design. A minimum width of 43.75m is necessary to make this possible.

6.2.2.4. Depth

The require depth in the marina is assumed as 2.0m. From the bathymetry survey follows that the current depth is larger than 2.0m except in the part near the main breakwater and in the northern corner. In these areas dredging is needed to reach the required depth.

6.2.3. Improved design

The fairway width is not sufficient in the design. A new design need to be made. The new design is shown in Figure 88. In this design the fairway widths are sufficient. The width of the interior channel is equal to the preferred width of 25m. The total capacity of this design is 70 yachts. This is about the same as the design that was made in 2016.

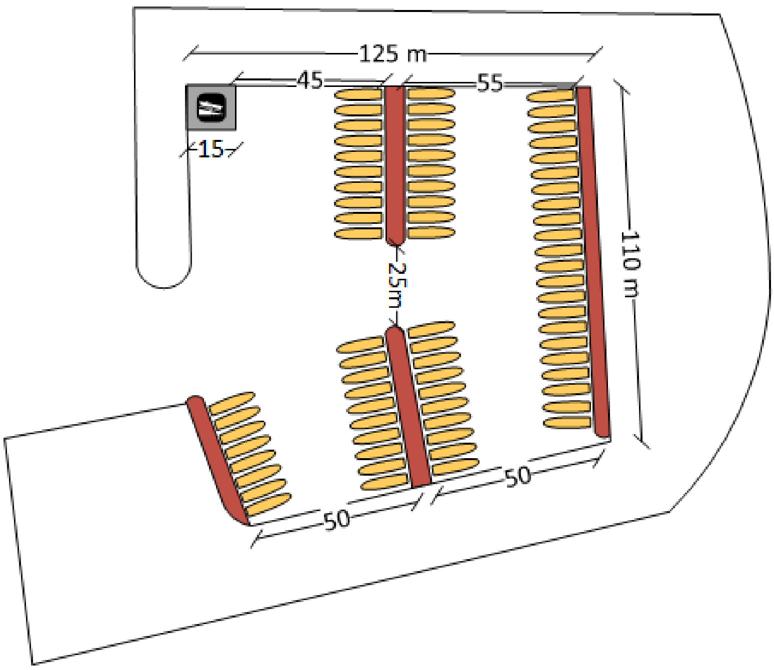


Figure 88: New marina design

6.2.4. Constructability

The design of last year was mainly based upon the present situation. The entire outline of the marina is being used. Keeping in mind the experience in Bulgaria the seaside of the marina outline needs to be strengthened. The available stacked concrete plates could be used to provide shelter for the moored vessels. The possibility is available of using fixed piers, these need to be casted and located. This can't be done with the already available materials, so investments must be made. The hotel owner do not want his summer season to be interfered by constructions works, but the ideal moment of constructing the marina would be in summer conditions. An option could be to start constructing at the start of the summer season and finish it during the end of the season. By this, nuisance during high season is prevented, but the possibility of working under summer conditions is increased.

6.3. Breakwater preliminary design

The breakwater of the marina constitutes the main structure that will ensure the proper functionality of the marina for the three quarters of the year. The main questions to be answered are:

- Can the proposed design withstand the winter conditions?
- Can it ensure calm wave conditions in the marina during the operation period?
- Does it satisfy the recreational and architectural visions for the area?
- Does it provide sufficient protection for the recreational areas/buildings that are proposed to be constructed on it?
- What are the possible effects on the environment?
- Is the construction feasible taking into account the location, availability of material and equipment?

In the report of 2016 the design presented in Figure 89 as suggested for the layout of the marina. It is proposed to repair the existing breakwater by constructing on top of it an additional layer of concrete slabs. An outer slope made of tetrapods or riprap will protect the breakwater from storms and extreme weather conditions. This outer slope will be constructed along the whole length of the main breakwater (along the L shape) including the area around the tip that looks into the marina entrance. The tip of the breakwater will be wide enough, in order to be used for recreational activities.

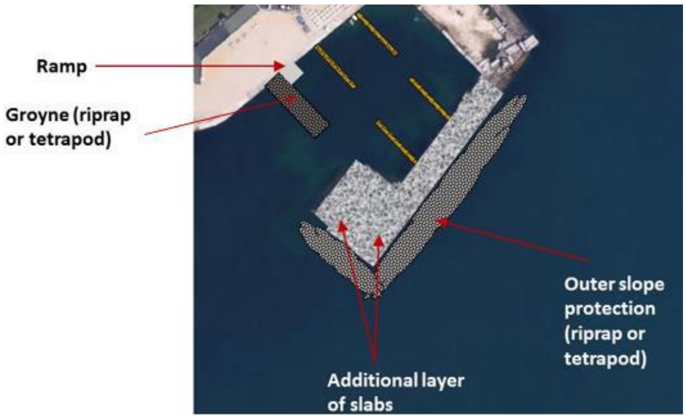


Figure 89: Sketch of the proposed layout for the marina (report of 2016)

However, this can only be considered as a preliminary sketch since it provides no information about the material used and the dimensions of the features (e. number of layers, layers' thickness, dn,50 etc.). Therefore, for the feasibility study of the design, also the design of 2015 was also taken into as it was the basis for the report of 2016.

In the report of 2015, a design is proposed for the breakwater which appears to have been checked for winter conditions. The aforementioned design is displayed in the figures below (Figure 90, Figure 91, Figure 92, Figure 93).

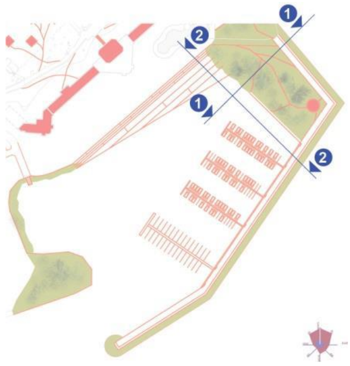


Figure 90: Cross-section lines on the masterplan

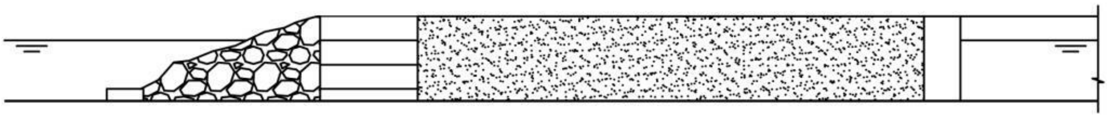


Figure 91 Cross-section 1-1: condition in 2015

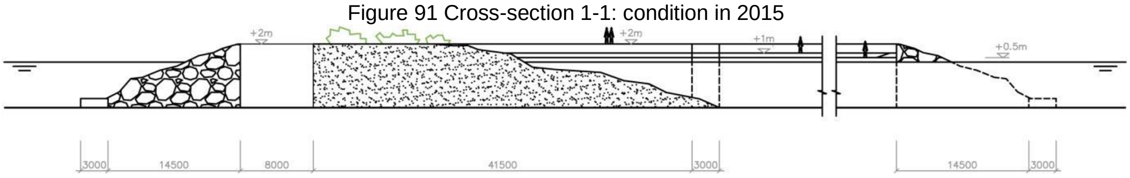


Figure 92 Cross-section 1-1: design proposal

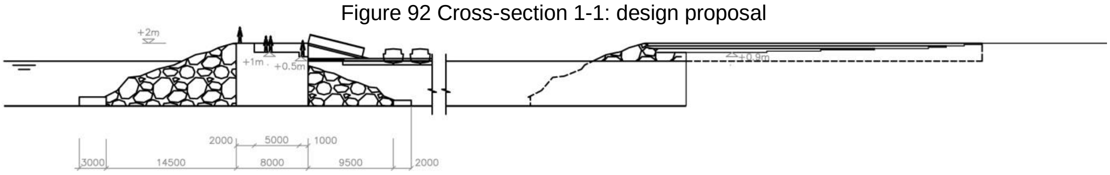


Figure 93 Cross-section 2-2: profiles of the breakwater and the terraced embankment

The aim of this year's report is to assess the existing design and propose modifications were the last one fails to comply with the design criteria. In this procedure there are several problems to be dealt with as important dimensions and features are not provided in the drawings. As can be seen in Figure 91, Figure 92, Figure 93 the seaward part of the breakwater consists of an amount of rocks of various nominal diameters. The different layers, e.g. armour layer, first under layer, core etc. are not indicated and there is no indication of filter layer or geotextile at the bottom in the seaward part. The water depth and the height of the toe are not mentioned in the drawings. The materials used to construct the breakwater are not clearly indicated. In conclusion there is great difficulty in assessing the design as important information is missing. To cope with that hindrances, assumptions are made in order to propose a more detailed design based on the existing one.

According to 2015 proposal the breakwater can be used as a walking pier. The designed walking paths on three levels of the breakwater allow to have an overview of the sea and the marina or to enjoy the terraces of the embankment. As a result, there can't be any overtopping allowed over the crest of the breakwater as this poses a disturbance or even a danger to the pedestrians. Therefore, as part of the feasibility assessment, the proposed design will be checked for compliance with the overtopping criterion.

6.3.1. Boundary conditions

In this chapter, a preliminary design of the Mariana breakwater will be performed. The input wave parameters are equal to the parameters mentioned in chapter 5.5.2. (Wave height from SWAN analysis and Svasek results), transformed in shallow conditions. The normative wave parameters are shown in Table 30:

Condition	H_s (m)	T_p (s)	RP (years)
ULS	4.0	9	1/50
SLS	1.7	9	1/1

Table 30: Input parameters design

Three failure modes will be checked in the preliminary design of the breakwater. The armour layer of the breakwater is allowed to have repairable damage after the normative storm, the same holds for the toe design. During summer, pedestrians should be able to walk over the breakwater without getting wet feet, since the marina will only be in use during summer. For that reason, more overtopping is allowed during the winter season.

Failure modes		Limit state	Return period (years)	Design method	Design value
No damage requirements (winter period)	Armour layer stable under extreme conditions	ULS	50	Armour layer stability formula	Sd=3
	Toe stability	ULS	50	Van Leeuwen (1996)	Nod=0.5
	Overtopping	ULS	50	EurOtop Manual	Q=1 l/m/s
Summer period	overtopping	SLS	1	EurOtop Manual	Q=0.05 l/m/s

Table 31: Design criteria

6.3.2. Design results and constructability

The following design results are based upon the calculation provided in Appendix G. In Figure 95, a sketch of the breakwater can be found. In the armour layer of the breakwater the biggest rock gradation (6,000 – 10,000) is used. According to the supervision of the fieldwork this rock class is available on the market but it is really hard to obtain. As a big rock size is chosen the breakwater can be constructed with relatively steep slope of 1:2, according to the proposal of the previous fieldwork. A possible solution to reduce the required stone gradation is to increase the relative density of the stones, by searching for another quarry with stones with the desirable property. Another possible solution is to design the breakwater with a milder slope e.g. 1:2.5 or 1:3, but this will lead to increase of the breakwater width. Another approach would be to use concrete elements for the armour layer in the sea side slope of the breakwater instead of rock. In such a case, some of the tetrapods already existing at the site can be used. The choice of concrete elements is not considered the optimum as it results in several gaps in the protection. Since the marina is used for recreational purposes, children may fall in these gaps which leads to unsafe situations.

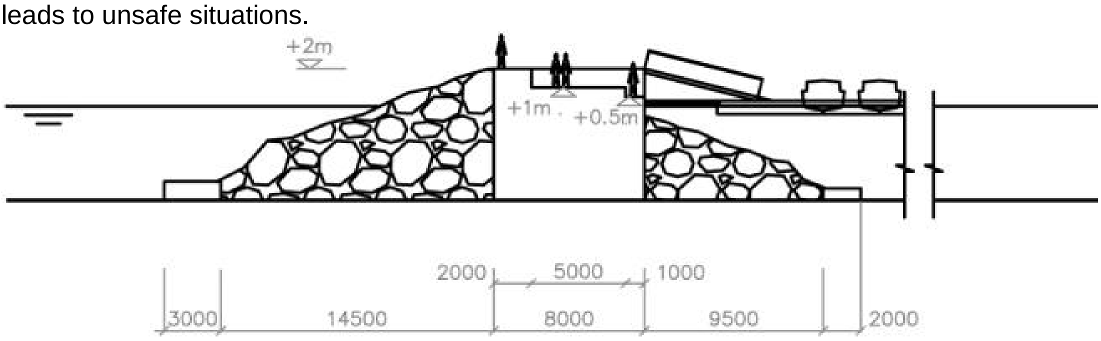


Figure 94: Cross section A-A of the breakwater in 2016 Design

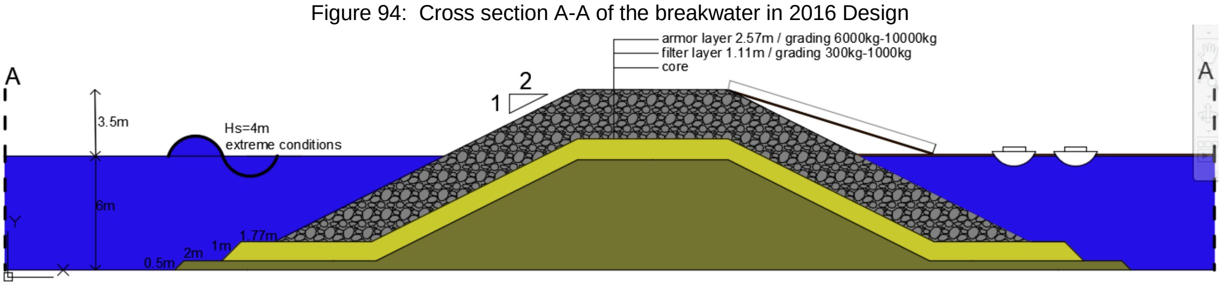


Figure 95: Cross-section A-A new design (2017)

Until now, only the ocean facing side of the breakwater has been considered, however before the construction of the entire breakwater, the rear side of the breakwater has to be designed properly as well. In the figure, the rear side is covered with the same rock class as the sea side. The seaside is dimensioned for the load of the waves coming from the Blacksea. For the inner slope this is not the case and therefore a smaller rock class can be used.

In every design one should also take into account how feasible is to construct the structure. To obtain the optimum execution plan a more detailed investigation on the equipment provided in Bulgaria is required. The possibility of using equipment from other countries could also be considered in case this option can lead to faster and relatively cheap execution.

The site preparation is necessary before the actual construction operations in order to specify the location for storage of materials, the stockpile areas, the loading and offloading facilities. It is obvious that sufficient space is required for stockpiling of materials in designated grades. For the transport of stones from the quarry, trucks can be used. Near the marina area it may be needed to construct temporary access roads. The granular filter layer can be placed by dump trucks. According to the rock manual the stones of the toe side can be placed by a stone-dumping vessel or a barge. The placement of big rocks of the armour layer can be done with wire rope crawler crane which has a lifting capacity 20t for wire and its reach is much more than the hydraulic excavator. The crane can be positioned on a pontoon that can have access to the area by the sea.

The construction of the breakwater cannot take place during the summer period when tourists are visiting the area. On the other hand, an execution during winter can be difficult due to bad weather conditions. The climate of the area should be studied in detail. A possible solution would be to start the construction before the tourist season, for example in March and finish in May. So, only for one year the tourist period will last less months. In order to define the exact period, one should make an execution plan. This will include in detail the construction stages, their order and the required weather conditions for each stage.

6.3.3. Discussion

For the completion of the breakwater design there were some challenges encountered which eventually were successfully overcome. First of all, the acquisition of the breakwater dimensions was quite difficult since in the first days of measurements there was a storm. The large waves that were smashing onto the breakwater and its surface being extremely slippery due to years of water overtopping made it impossible for us to physically approach the breakwater edge and measure its dimensions. Eventually, Google Earth images were used. Secondly, the design was required to comply with different wave height conditions (summer and winter conditions). However, the actual wave conditions could be measured only during the time that we were on site which coincided with the beginning of the winter period. So, for the summer conditions assumptions had to be made. Moreover, regarding the chosen winter conditions, the available data comes from Svasek's database as our measurements were unusable due to equipment failure and they covered a time span of a single storm. Therefore, without a wave climate available, assumptions were made to determine the extreme conditions. We also had to make more use

of the visual wave measurements, since they provided us of the only onshore measured wave data. By combining Svasek and visuals, we could still get a good view of the wave conditions in the end.
Last but not least, the exercise's aim was to check the feasibility and stability of last year's design. However, the team of 2016 focused on designing the groyne rather than the breakwater so, for the last, only a general sketch was provided. That meant for us that we had to assume an initial breakwater design in order for the checks to be possible. The final design is also based on this preliminary design assumption with alterations in order for several design criteria to be fulfilled.

Besides those hindrances, everything else went smoothly. The report was made in a structured way and we manage to collaborate well despite the different schedule of the team members. We are confident that the proposed final design is safe, economic, constructible and complies with the client's wishes.

7. ECOLOGY

Each living organism has a specific relation with the environment and the aspect of ecology shows which mutual relations are required to sustain and/or improve the habitat for the organisms. For the design of the feasibility check and preliminary design of the new marina the current functioning ecosystem needs to be protected.

The current marina St. Elias and beaches positioned north and south of the marina provide an important environmental function for land and water and is part of a larger ecosystem. The marina and beaches do not only contain a sheltered water and land environment but also the transition between them. The transition provides a habitat for a narrow band of organisms, due to the large seasonal changes in the wave climate, but is important nevertheless an important contribution to the ecosystem.

Another environmental aspect is the interface between the sand and rock protected areas. Each type of area harbours and protects a different ecological habitat, in which multiple species can find shelter. Any changes in the interface occurring in the design could have consequences for the population of species or habitat and impact the ecosystem in that area. For example, the owner of the owner of Grand hotel desires larger beaches in the new design, which has an impact on the interface between rock and sand. One needs to protect/maintain the necessary conditions of a healthy population which requires the environment of a rock protection.

The ecology can be included into the coastal design by evaluating the current ecosystem and analyse key aspects for maintaining the inherent character and functional integrity of the ecosystem. This approach does allow monitoring of the ecosystem and intervene of necessary. However, this approach does not guarantee no harm will be done to the ecosystem as they are delicate and complicated processes.

In Appendix F, the type of habitat and an inventory of species (plants and animals) in the beaches and marina St. Elias are determined. The fieldwork analyses showed that there are six habitats:

- Forest area
- Vegetation and rocks
- Flora and fauna in sand
- Sea life
- Fauna between rocks in water
- Small vegetation

Below are four examples of habitats occurring the marina st. Elias and bordering beaches.

Figure 96: Habitat forested area (left) flora and fauna in sand (right)

Figure 97: Fauna between rocks in water (left), small vegetation (right)

In the different habitats multiple examples of plants and animals are found, which are documented in the Appendix F. The analysis shows the current characteristic of the ecosystem. The next step is to identify the critical processes using the eleven ecological principles. This list forms a check list for each part of the design that is evaluated:

1. Pursuit of continuity of physical processes, like water and sediment flows and land-water interfaces in the ecosystem;
2. Minimisation of direct human disturbance of the ecosystem, because direct disturbance may affect the health of the ecosystem;
3. Minimisation of endogeneity (level of invasion of an ecosystem by exotic species), which is preferred above invasive colonisation;
4. Preserve viability of populations to prevent extinction;
5. Create opportunities for endangered populations and particular species;
6. Pursuit of trophic web integrity to maintain a healthy interaction of all species in an ecosystem;
7. Create or maintain opportunities for ecological succession of species from pioneer stage to climax stage;
8. Aim at zone integrity aims to ensure that the natural mosaic of the ecosystem is fully represented;
9. Preserve characteristic (in)organic cycles to secure the integrity of the throughputs of carbon, nitrogen, phosphorous and silicon in an ecosystem to support and enable the ecosystem character and functioning;
10. Maintain or optimise the characteristic physical-chemical water quality over time and space to prevent triggering atypical, unwanted events;
11. Strive for resilience that enables the ecosystem to withstand and even benefit from reasonable, foreseeable disturbances.

An eco-friendly design or eco-dynamic design should incorporate elements of sustainability into the design objectives. The ecological design objectives are formed from the sustainability requirements. The sustainability requirements in the previous design are not determined, so the existing design is evaluated with the check list to highlight possible harm for the ecosystem.

7.1. Groyne

The groyne structure is currently in a deteriorating position, however, a new plan was not included in the concept design of 2016. In this case, the ecology analysis is added to highlight potential pitfalls and risk regarding the present ecological value.

The structures core exists of a concrete element and is protected by large rock elements. The concrete element also functions as a drainage channel for a small channel on the mainland. The groyne in its current shape offers possibilities for marine life such as clamshells to nest in the space between the rocks. During the fieldwork some vegetation was also found between the rocks as sand/soil was either placed there or transported by alongshore sediment transport. Moreover, the main function of the groyne(blocking alongshore sediment transport) has created a beach on the north side, which formed a new piece of the ecosystem when it was constructed. The removal of the structure would endanger the current ecosystem. Using the checks for the critical processes of the eleven ecological principles, some recommendation can be made for the new design:

1. The main function of the current breakwater should remain in a similar form, as otherwise the water and sediment flows are changed in a way that affect the current ecosystem in a negative way.
2. On the south side of the groyne new opportunities could be created for vegetation, as the current structure blocks the sediment transport there ones was.
3. The drainage channel, although not always present in the past, requires attention in the new design in order to maintain or optimise the characteristic physical-chemical water quality.

7.2. Marina

The new design of the marina shows more mooring places for ships and a new pier structure. In general, the wave climate inside the marina will be calmer, as the new breakwater blocks waves to pass through and limits overtopping. This results in a more suitable environment marina life to settle, however, the increased activity of the marina with regards to the current activity could cancel out the just mentioned positive effects.

The design influences the following process with respect to the current situation in the checklist:

1. Create opportunities for endangered populations and particular species;
2. Pursuit of trophic web integrity to maintain a healthy interaction of all species in an ecosystem;
3. Create or maintain opportunities for ecological succession of species from pioneer stage to climax stage;
4. Preserve characteristic (in)organic cycles to secure the integrity of the throughputs of carbon, nitrogen, phosphorous and silicon in an ecosystem to support and enable the ecosystem character and functioning;
5. Maintain or optimise the characteristic physical-chemical water quality over time and space to prevent triggering atypical, unwanted events;

The points 1 to 3 refer to the now calmer wave climate as this is an ideal opportunity to create a more resilient place for the occurring species. The points 4 and 5 refer to the effects of the decrease in the water quality from the increase in marine traffic in the marina. The materials that could enter the water are for example oils and trash. These materials will settle on the subsoil and create a toxic environment inside the marina.

7.3. Breakwater

The breakwater is an element of the marina but receives a separated section as this part receives the largest changes in the concept design of 2016. The current breakwater provides a very rough environment for flora and fauna as it functions as an almost submerged breakwater. The majority of the waves break on the remains of the original breakwater design. When restoring the full function of the breakwater in the concept design, the breakwater will increase in height and provide calmer conditions, which increase the ecological activity. This change does not cause harm to the processes that occur on the checklist. The only downside is that less water is passed through to the marina and thus less flow inside the marina to refresh the water.

8. CONCLUSION AND RECOMMENDATIONS

The processing and analysis performed in the previous chapters led to several conclusions and recommendations. In this chapter the most important conclusions and recommendations per topic will be presented.

8.1. Conclusions

The research question is answered by solving the different sub-questions, which are presented below. An answer to those questions is provided, after which the feasibility is described in accordance with the structure of topics in this report (beach, soil sampling, bathymetry, marina, groyne and breakwater). The feasibility results are supported by the measurements.

The sub-questions that will contribute to solving the problem are:

1. Which variables are necessary in order to assess the feasibility of the concept design?
2. How is the value of each variable defined using measurements?
3. How to organize the data collection and processing?
4. In what way can the measurement data be used to achieve a feasibility study of the design?
5. How is external data validated by measurements in the field?

The necessary variables are discussed in Chapter 3 including their desired accuracy. The final accuracy is mentioned in the measurement techniques and was in general higher than the desired accuracy. Chapter 3 also stated which measurement techniques/methodologies are used to acquire the variable and are described in more detail in Chapter 4. In Chapter 5 is shown how the measurement data is used to assess the feasibility of the conceptual design. The external wave data is validated by the visual wave measurements, due to the errors in the wave logger data.

The beach profiile measurements have shown that there is a variation in the long term evolution of the North and South Beach. The first has an erosive tendency with sand not only going offshore, but also moving from the southern to the northern side of the beach. On the contrary, in the South beach, the central part is eroding, whereas the sides are accreting with the net result being accretion.

The measurements of the shore lines and google sattelite images support the conclusion drawn from the beach profile measurements. The South beach shows both retreat and advance over the years and depending on the season, it slightly changes. In the North beach, we can observe large differences in the positions of the beach shoreline since it is highly unstable. Shoreline measurements over the past two years showed that the beach and its shoreline is very sensitive to the weather conditions. This may also be due to the very large slope of the beach. It must be noted that the difference in shoreline positions can also be due to weather conditions, device settings and inaccuracies, or differences in the yearly chosen measurement positions.

In general, additional measures need to be taken (nourishment, hard structures) to stop the North beach from eroding and, therefore, make it more suitable for tourism and recreation purposes. There is no imperative need to take measures for the South Beach, as it seems to be sufficiently stable for the time being and compared with last year. In both cases, a more detailed coastal analysis is recommended at a later stage of the feasibility assessment. Long term trends as well as seasonal variations and sea level rise effects should be taken into account.

The grain size measurements in the Southern beach showed no considerable differences in the grain size distribution among the different locations. This was to be expected as the weather conditions did not allow for samples to be collected from the sea bed. All the samples collected on the beach as well as underwater contained coarse sand (d_{n50}=0.7-1mm) with the size not varying a lot depending on the position and the depth. Further conclusions are not possible to be drawn from this amount of samples.

The bathymetry and the measurements around the marina indicate possible wave height amplifications due to reflection and reduction due to diffreaction. In summer conditions these effects are expected to almost cancel out. From the bathymetry survey follows that, to provide a 2 meter draught for vessels, dredging must be performed. The feasibility assessment resulted in a more indepth design, dimensions were included and fairway widths were taken into account.

The feasibility calculations of the marina breakwater showed that the proposed breakwater solution is indeed feasible, however the required rocks dimensions are very hard to obtain in that area. A review of the current design (changing the slope, applying other rocks or change to concrete elements) could solve these problems and yield a design that is both functional and constructible.

The groyne measurement were used to assess the current status of the groyne. Although there are some considerations about the accuracy of these measurements, it is obvious that the groyne has sustained serious damage in the last couple of years since its construction. The current condition suggests that the intitial design was insufficient to withstand the physical loads in the area or that the material used was of insufficient quality. Based on these remarks it is expected that the groyne will not be able to fulfil its purpose in the coming years, which is to protect the north beach from waves, current and structural erosion. Also, taking into account that this area is part of a general rehabilitation plan aiming to increase the touristic influx, the presence of a deteriorated structure is unacceptable as it lowers the touristic value of the area. It is suggested that the groyne is incorporated in the integrated coastal rehabilitation plan for the region.

The above topics conclude the feasibility part of the Fieldwork Bulgaria 2016 conceptual design and answer the main research question of this report. Chapter 7 deals with the integration of ecology in hydraulic engineering design and demonstrates the necessity for monitoring and evaluation of the current ecosystems present in the area with the aim of maintaining their inherent character and functional integrity of the ecosystem in an anthropogenically changing coastal environment.

The aim of this report was to research variables and characteristics of the marina and beaches of Grand hotel Varna using a measurement campaign, which was been proven successful, except for the wave logger data. The validation of the external data was performed consequently using the (less accurate) visual wave data. The second objective was to use these variables to perform a feasibility study on the design of the 2016 Hydraulic Engineering Fieldwork Report. Despite the previous year's design lack of important information This objective was also completed and a more complete design was presented and assessed.

8.2. Recommendations

Concerning the measurements, a more extensive analysis of the shore line measurement of the North beach should be performed using the google satelite images. Currently, one cannot draw a clear conclusion about the seasonality, neither the retreat of the beach, because the images are not average shorelines over the whole winter/summer period, but snapshots of a single moment in the winter/summer months. Even small changes in weather conditions can have a large influence on the waterline. Another aspect of the measurement is the repetition of the failed wave logger measurements and validation of the data used in this report.

In further studies, the conclusions from the assesment can be incoperated into the new design. For the preliminary design of the break water, the rear side of the breakwater, facing the shore, has to be designed in more detail as the current design is very conservative.

Lastly, there are elements of last year's architectural design that were not incorporated in this report for lack of time: the design of the infinity pool that was proposed, the new peninsula, the two new beach stretches and the new breakwaters. These are elements that could be assessed in the future from a coastal engineering, ecology and spatial planning point of view.

9. LITERATURE

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Appendix A: Shoreline measurements

A. North Beach

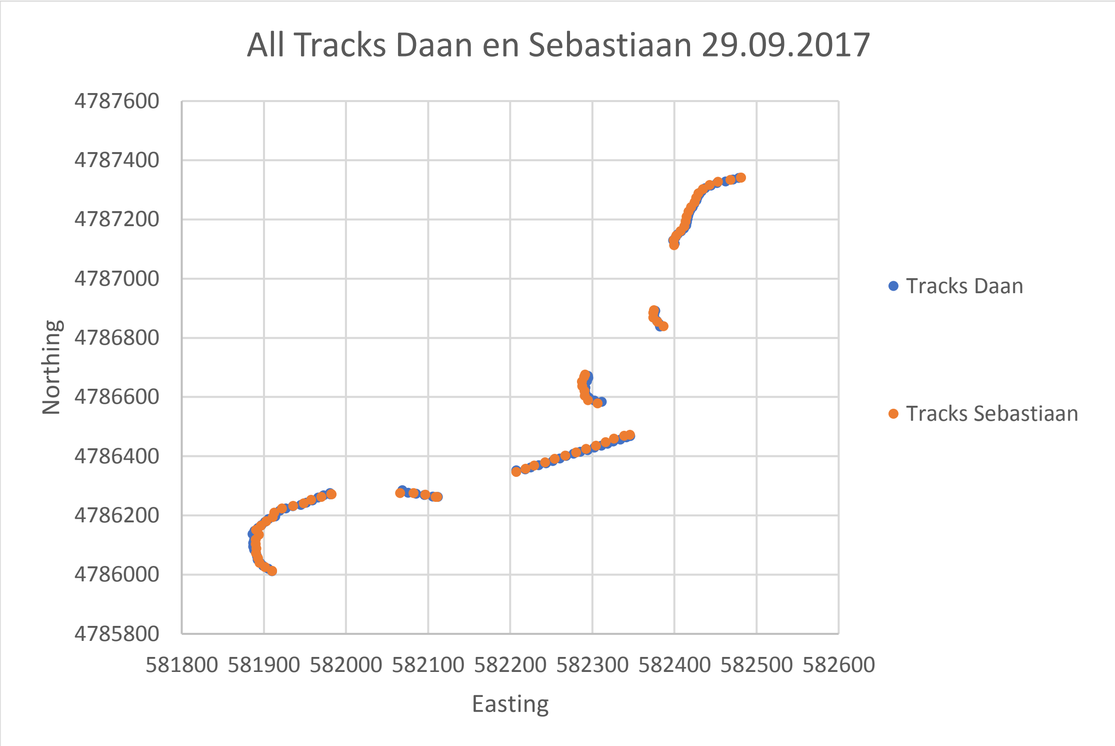


Figure 98: All shoreline tracks on 29.09.2017

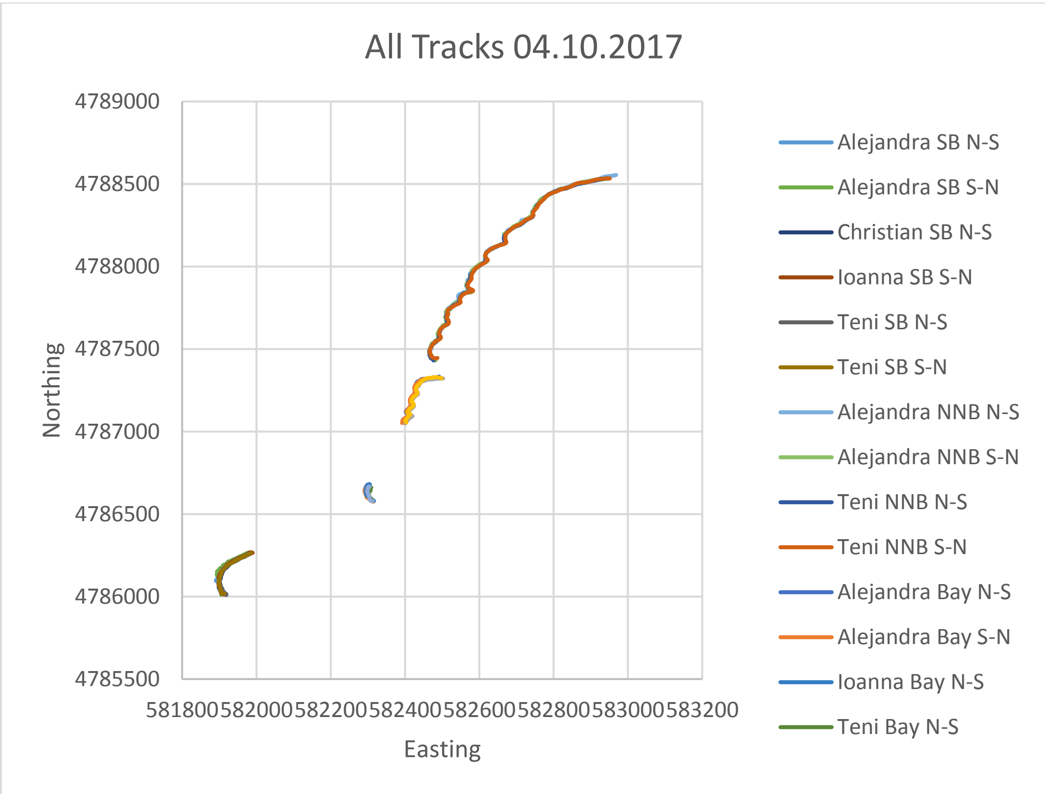


Figure 99: All shoreline tracks on 04.10.2017

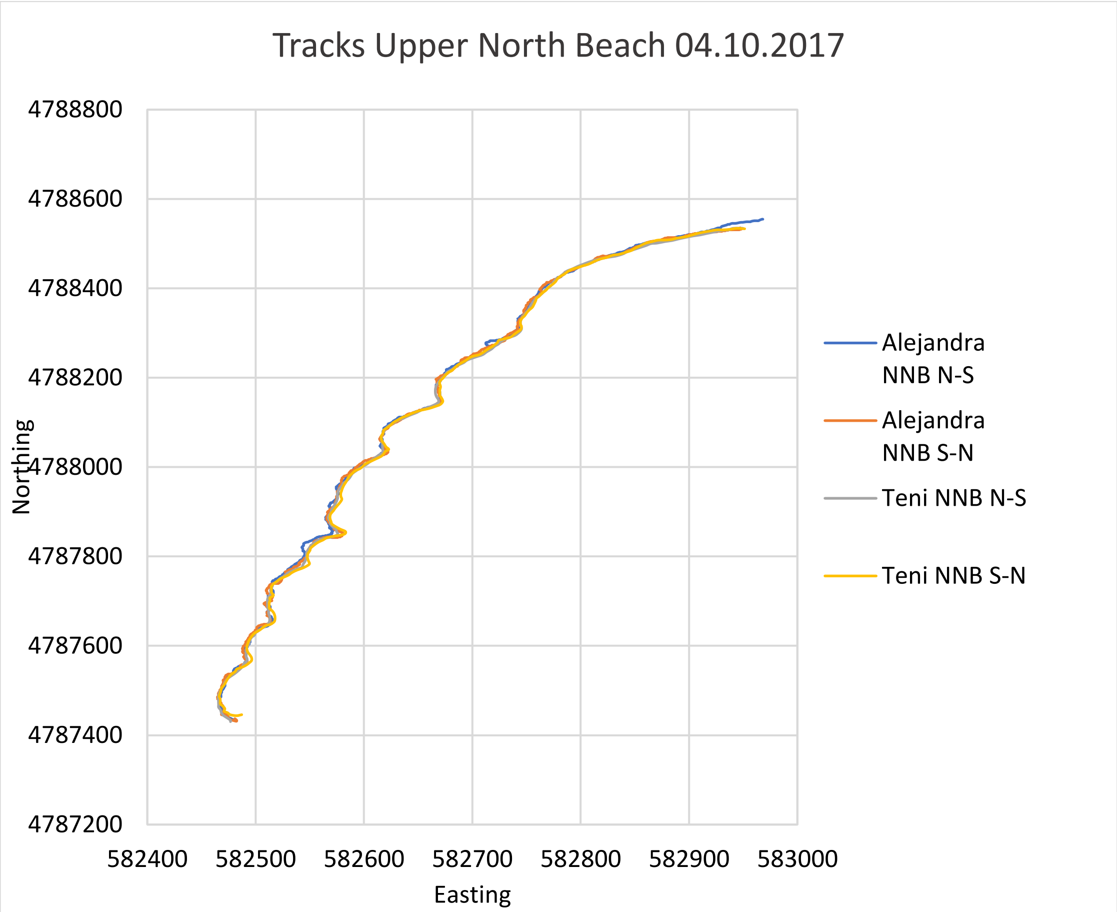


Figure 100:Tracks Upper North Beach 04.10.2017

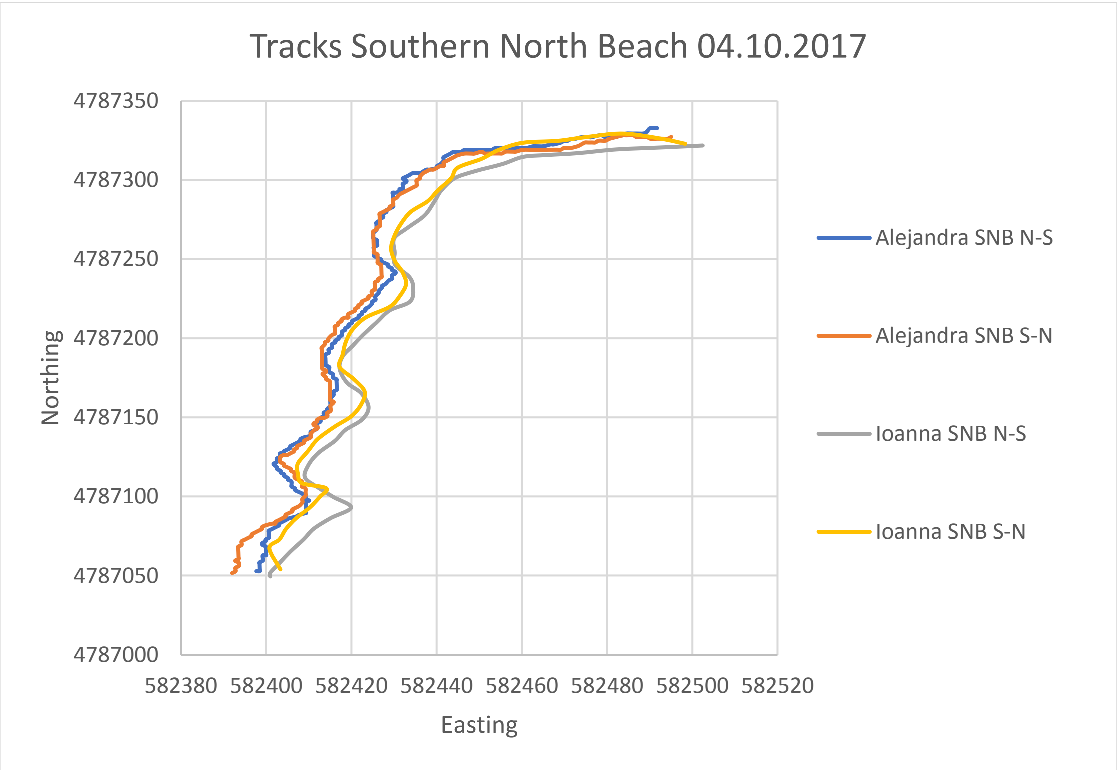


Figure 101: Tracks Southern North Beach 04.10.2017

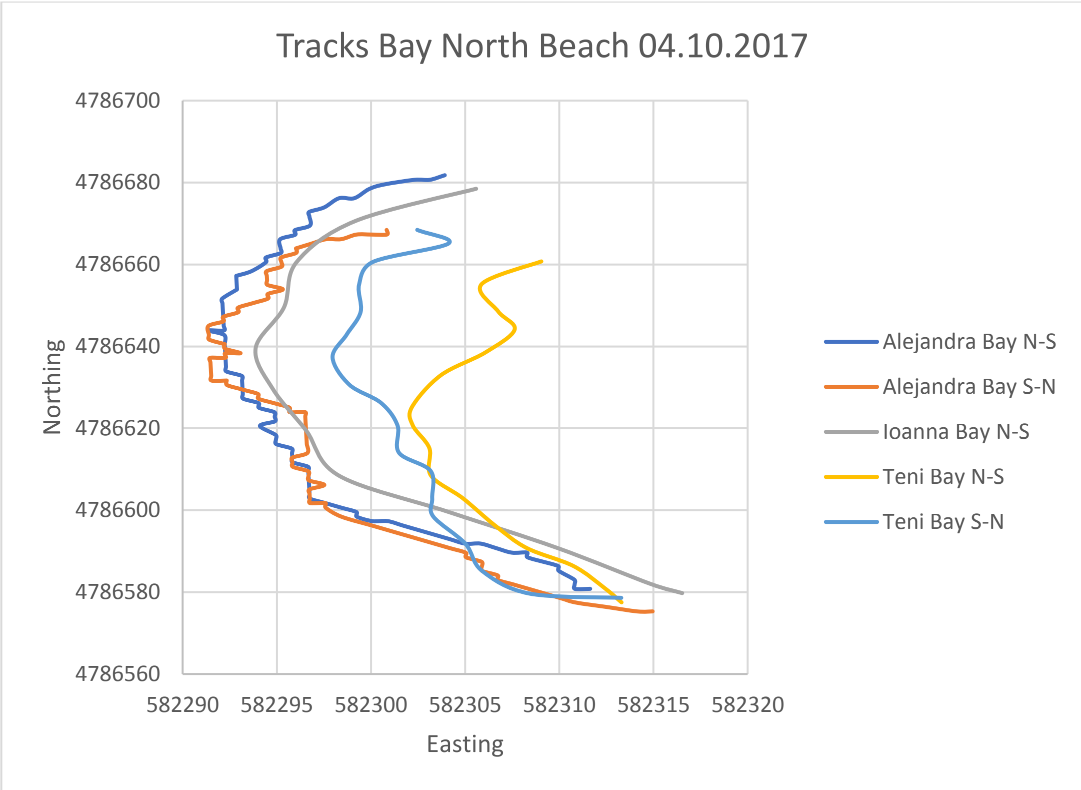


Figure 102: Tracks The Bay North Beach 04.10.2017

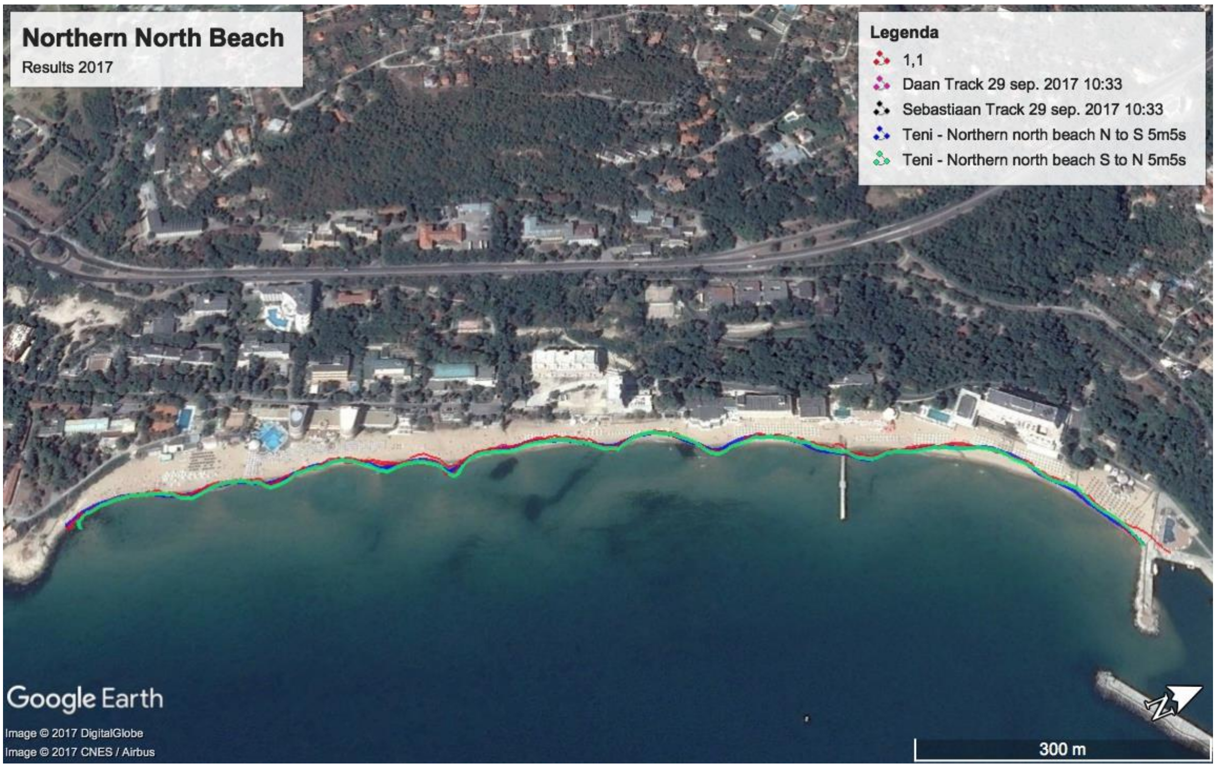


Figure 103: Tracks Google Earth view Northern North Beach

Figure 104: Tracks Google Earth view Southern North Beach

Figure 105: Tracks Google Earth view Beach Near the Groyne

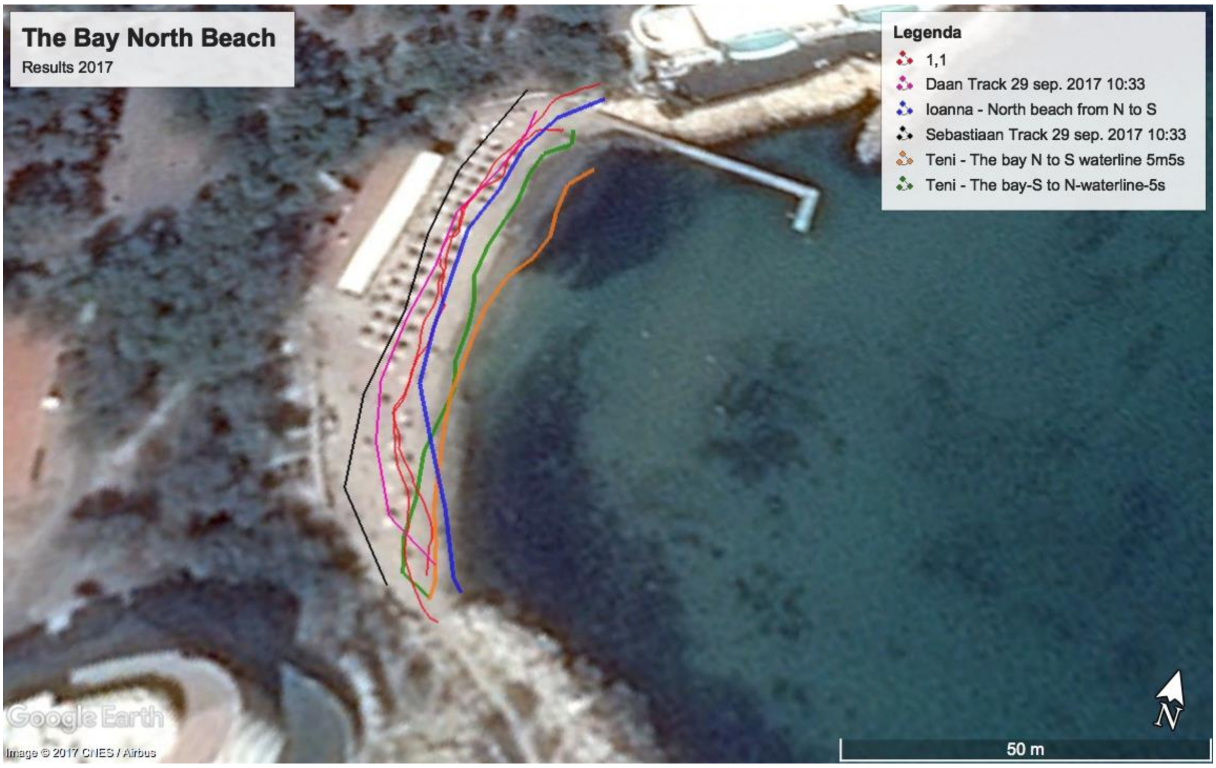


Figure 106: Tracks Google Earth view The Bay north beach

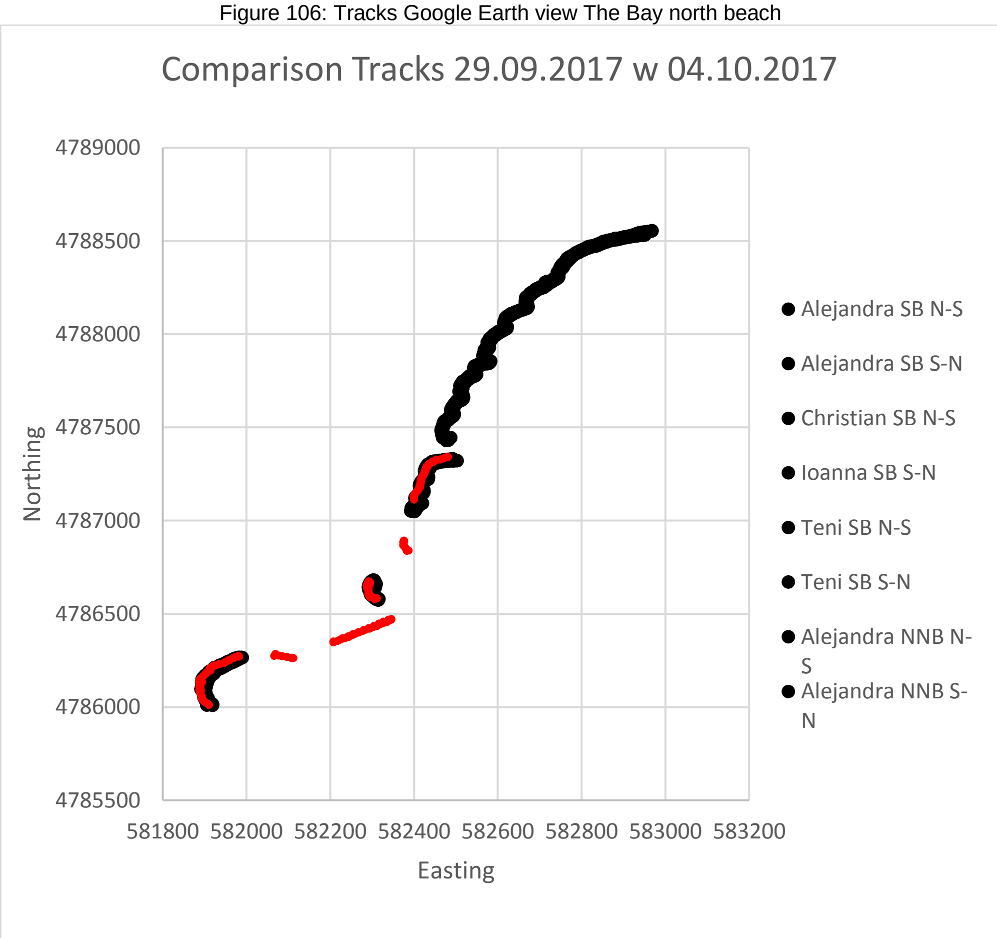


Figure 107: Comparison tracks 29/09/2017 and 04/10/2017

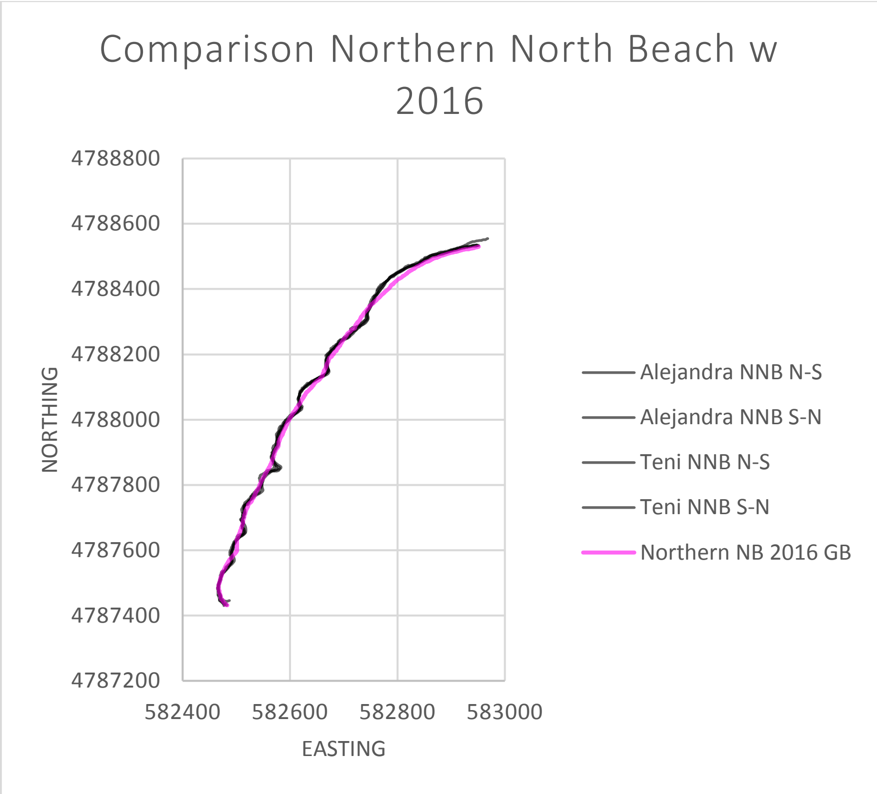


Figure 108: Comparison Northern North Beach with 2016

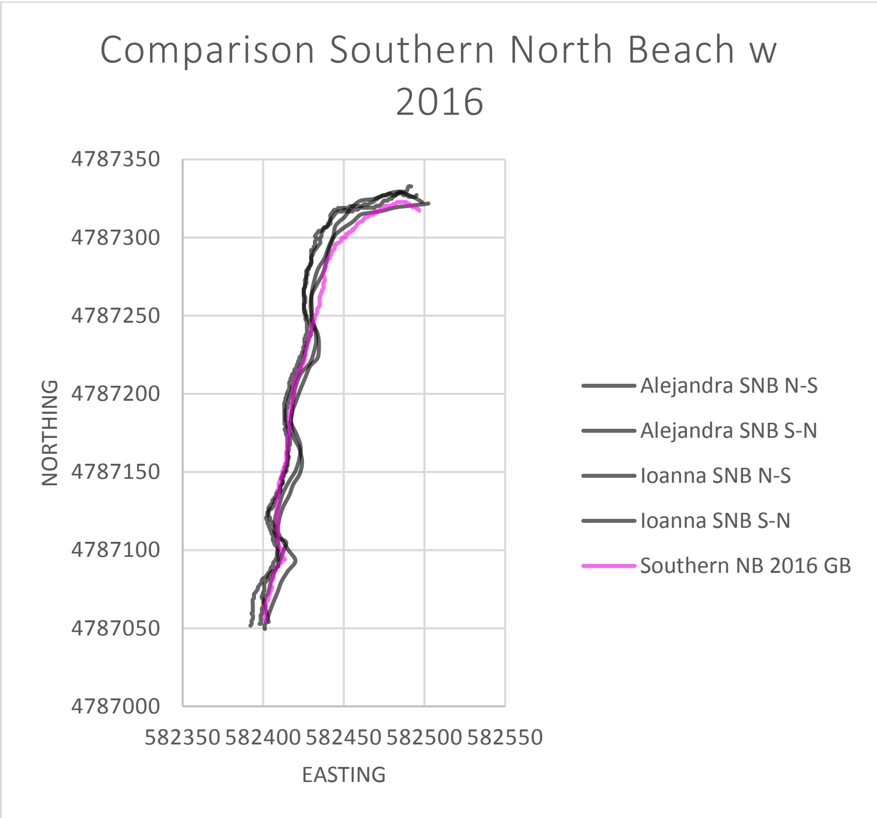


Figure 109: Comparison of Southern North Beach with 2016

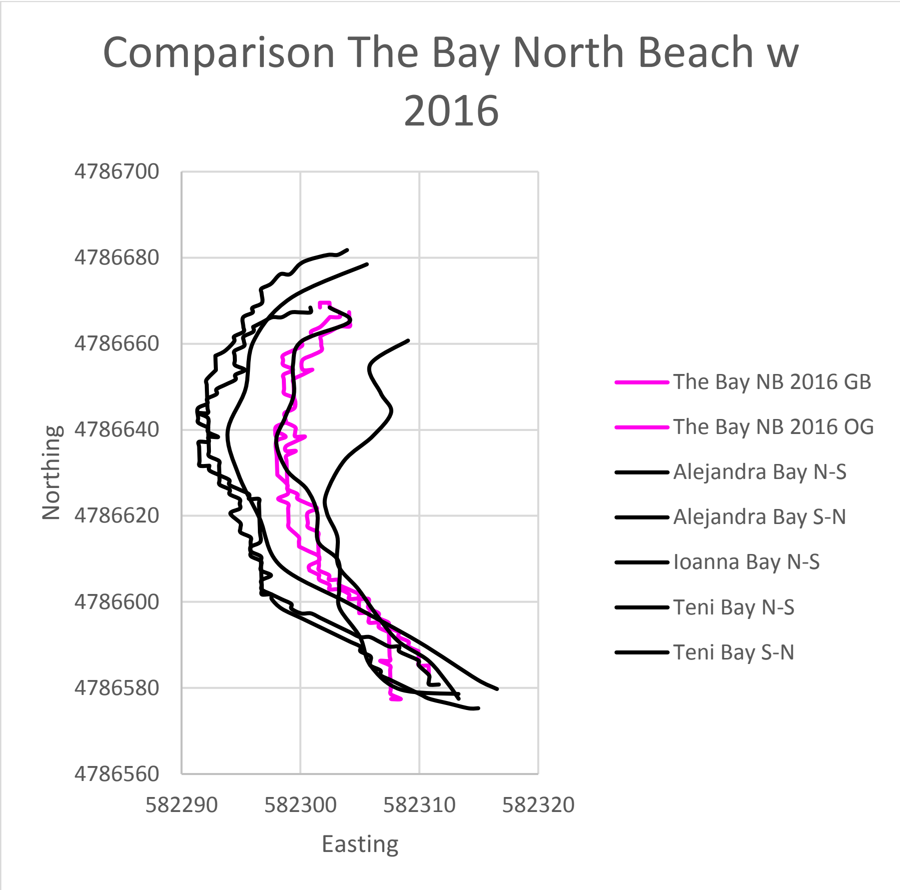


Figure 110: Comparison The Bay North Beach with 2016

B. South beach

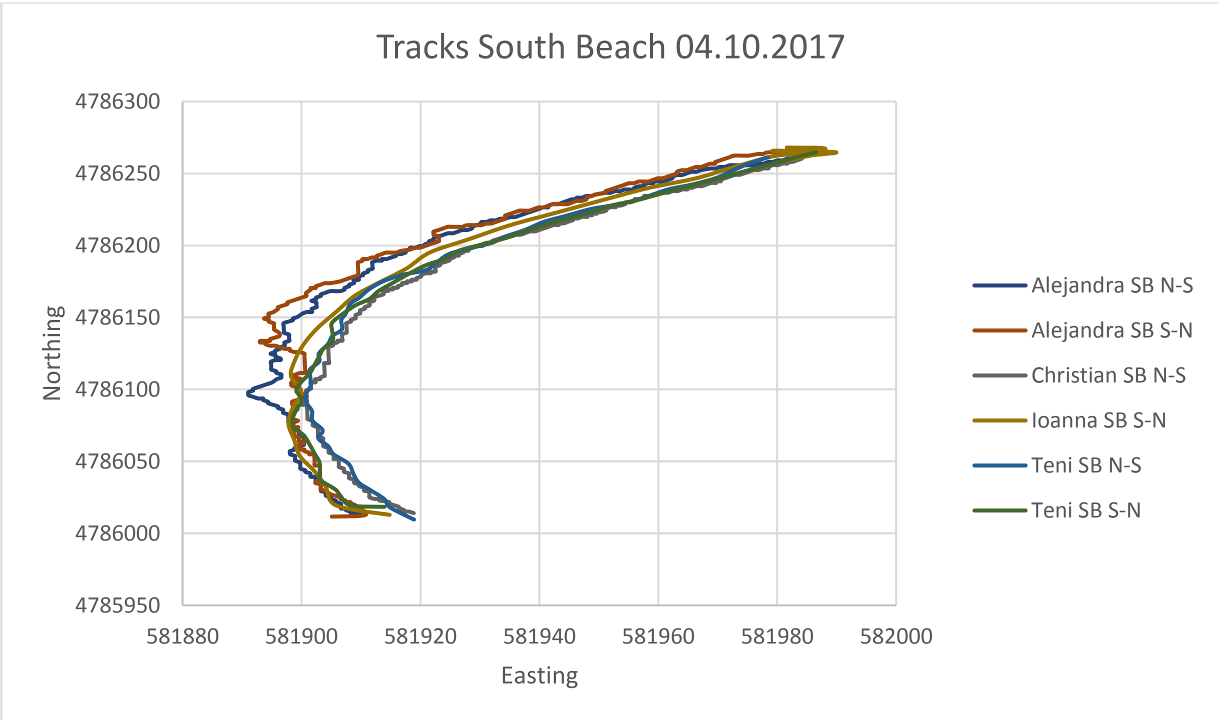


Figure 111: Tracks South Beach 04.10.2017

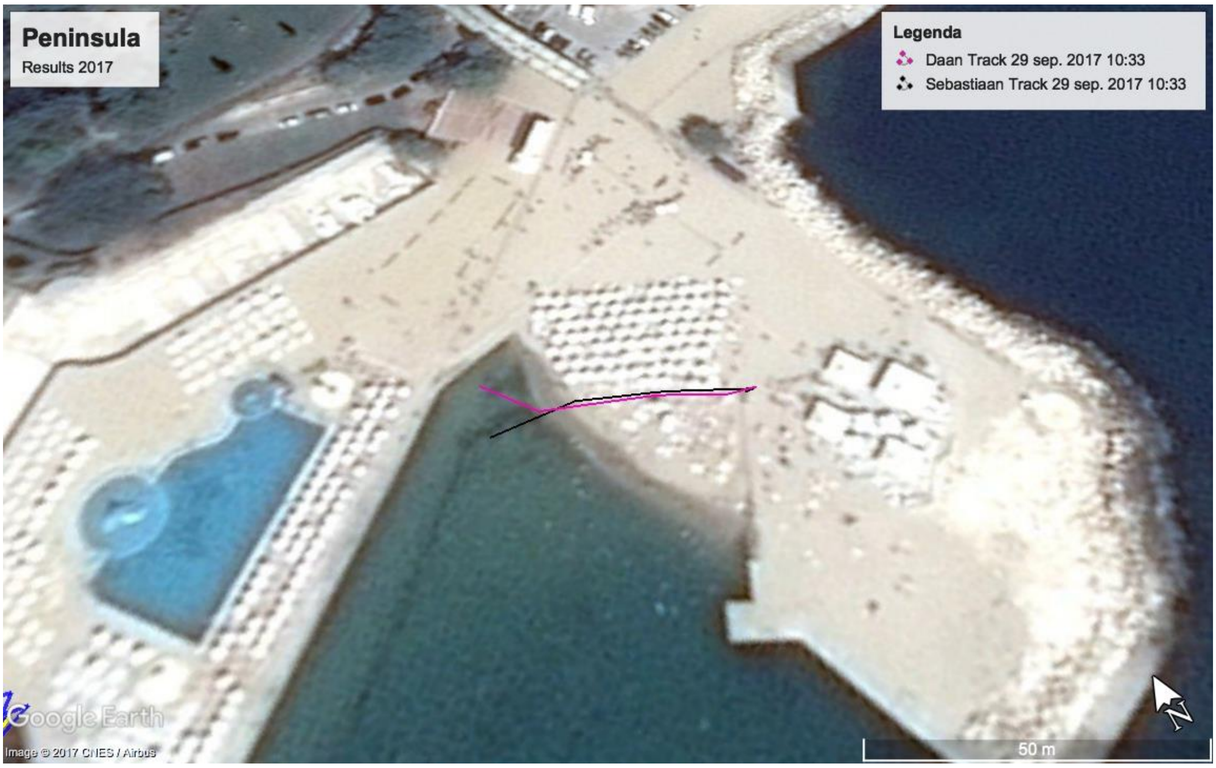


Figure 112: Peninsula tracks

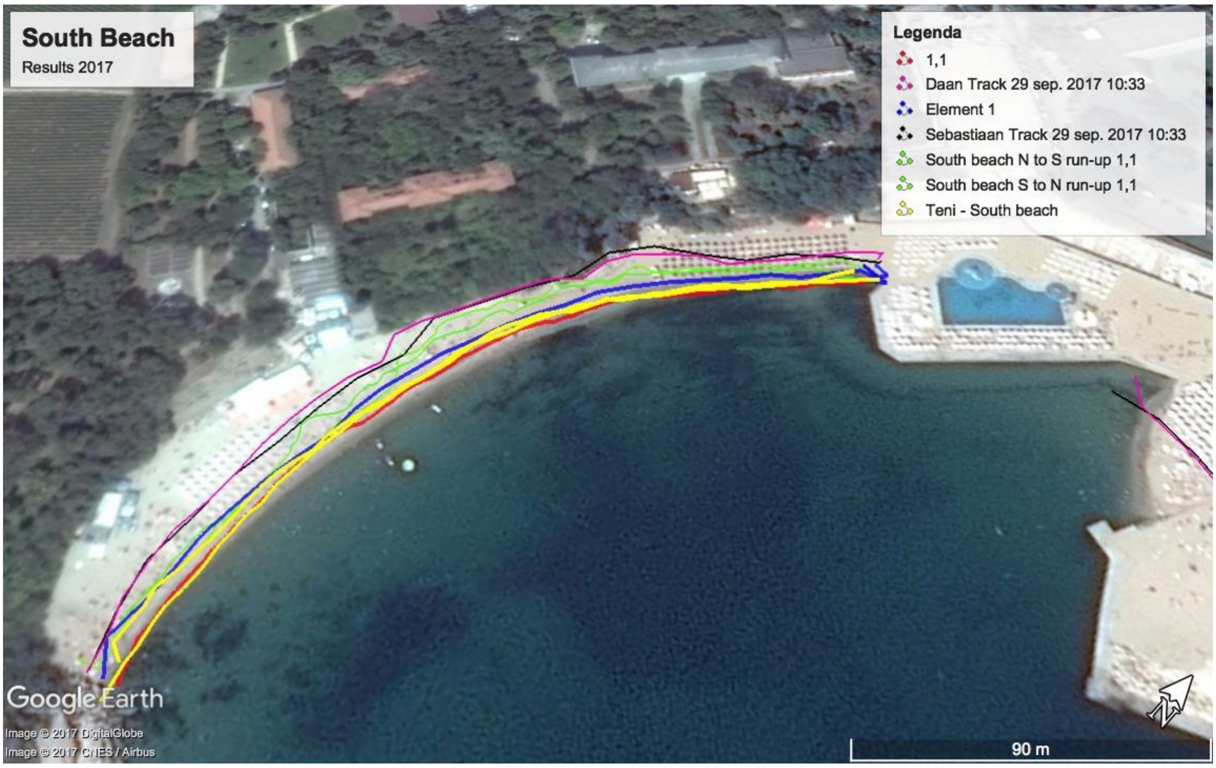
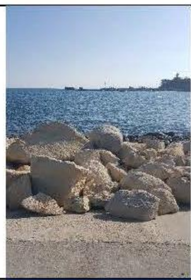


Figure 113: South Beach tracks 2017

Appendix B: Northern groyne visual inspection

x	Direction	2013	2015	2017
5	N			
5	S			
15	N			
15	S			
25	N			

25	S			
35	N			
35	S			
45	N			
45	S			

55	N			
55	S			
65	N			
65	S			

Table 32: Visual inspection of northern groyne 2013, 2015, 2017

Appendix C: Northern groyne assessment

Additionally to the visual inspection, a random set of 50 rocks were measured in order to get an indication of the average dimensions available onsite. A blockiness factor of 0.6 was visually estimated and applied in order to calculate the volume of the stones, while the rock density in the region was estimated 2300 kg/m3. The rock weight for which 50% of the total sample weight is equal or less was calculated as 148.8 kg. The dn50 of this sample can be calculated as follows:

$$d_{n50}=(W_{50}/\rho)^{1/3}=0.4m$$

This results in smaller median dimensions than those calculated in previous years. This may happen due to the added strain and damage over the last years but can also be attributed to a non-representative sample either for the last or the previous measurements. Therefore, no final conclusions can be drawn on that measurements.

Lastly, an inventory of the scattered tetrapods that were on the breakwater was made. Two different classes according to their size were distinguished. The volume was calculated using the well-known relation (U.S. Army Corps of Engineers) V=0.280H3. Additionally, the specific density of the concrete used to calculate the weight was considered 2400kg/m3 taking into account the poor quality of the specific elements. Lastly, Dn, the length of the side of a cube with the same volume as the tetrapod, was calculated for each class of tetrapods.

Tetrapods						
	foot(m)	height(m)	volume (m^3)	weight (tn)	D_n (m)	number
small:	1	1.7	1.385	3.30	1.1	9
large:	1.35	2.05	2.41	5.79	1.34	5

Table 33: Observed tetrapods and their characteristics

In order to judge the stability of the armour layer of the groyne (stones and tetrapods) the design wave height is calculated using the Hudson formula and the present armour characteristics:

$\frac{H_S}{\Delta D_n} = \sqrt[3]{K_D cot a}$, where:

- H_s[m], the wave height at the toe of the structure
- D_n[m], equivalent cube length
- Δ[-]=(ρ-ρ_w)/ρ_w
- a[-], the slope angle
- K_D[-], stability coefficient

Allowing for 0 to 5 % of the stones/ tetrapods to be displaced from the region between the crest and a level of one wave height below still water (no damage condition). For rough, angular, randomly placed armour stone KD=3.5 for breaking (depth-limited) waves and KD=4.0 for non-breaking waves. For ρw=1018 kg/m3, ρs=2300 kg/m3, Δ=1.26. For a slope of approximately 1:3 on the outer edge of the breakwater and Dn=0.4m, the resultant Hs is approximately 1.1m.

Assuming the following relation between the depth and the depth limited wave height, Hs=1.5h only the part of the breakwater that is situated at depth 1.1/1.5=0.73m or lower can be assumed to be well designed. For the rest part the incident (depth limited wave height) will exceed the 'no damage' threshold more frequently than desired. Of course, it should not be ignored that Hudson formula leads to over dimensioned rocks compared to other usually used formulas such as SPM. However, here it is used to indicate the insufficient protection of the concrete caissons that form the core of the groyne.

As far as the tip of the groyne is concerned the design wave condition is assessed using the Hudson formula with $H_{1/10} \approx 1.27H_s$, $K_D=5.0$ for breaking (depth-limited) waves, slope 1:1.5, $D_n=1.34m$, $\rho_w=1018$ kg/m3, $\rho_s=2400$ kg/m3, $\Delta=1.36$ resulting in $H_s=2.80m$. Assuming depth limited waves and a relation of depth to wave height $H_s=1.5h$, if the tip of the breakwater is situated at depth equal or smaller than 2 m then the design can be assessed as sufficient. Otherwise, the tetrapod size is not sufficient to withstand damage and protect the concrete caissons.

We do not have any measurements of the depth to which the northern groyne reaches. According to the Navionics naval map of the area presented below, the tip of the breakwater is situated at a depth of 4 to 5 m.

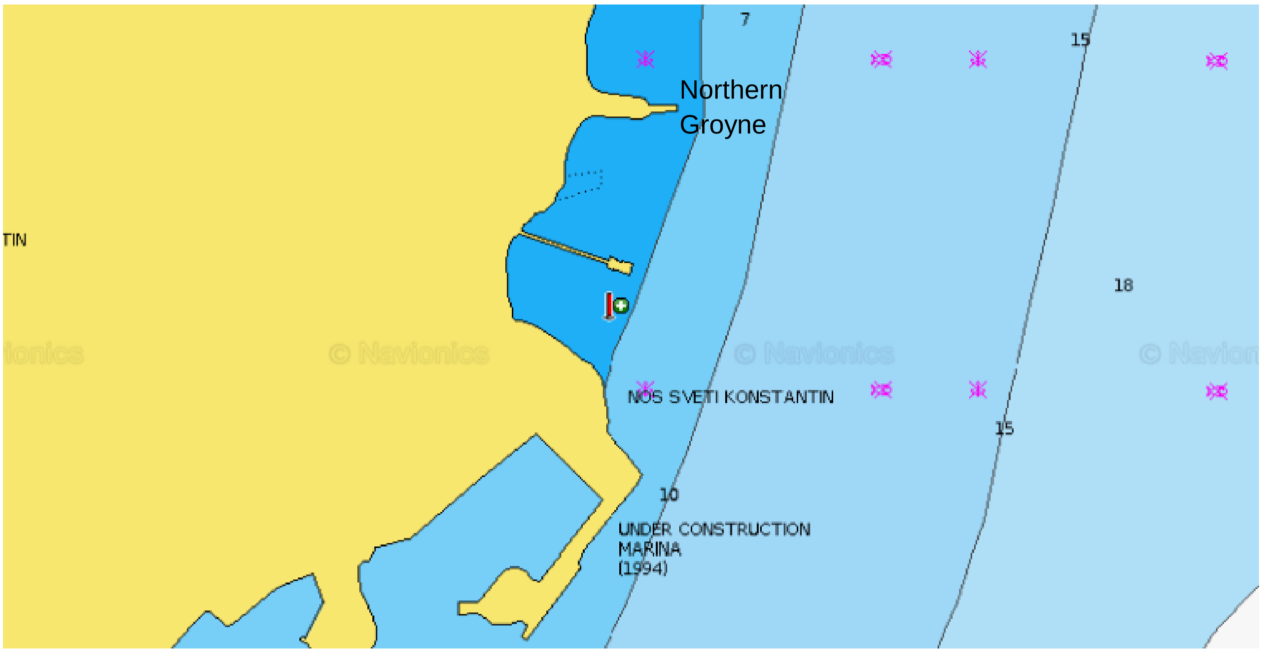


Figure 114: Navionics naval map of the area of interest

Appendix D: Visual Wave data

A. Day 1 (29-09-2017)

	1	2	3	4	5	6
Start time	15:00	15:10	15:29	15:40	15:47	15:52
No waves	30	30	106	30	30	30
25% height	12	11	17	4	2	2
50% height	11	10	32	7	11	10
75% height	5	4	29	11	9	10
100% height	2	5	28	8	8	8
Time	165	142	787	214	207	208

B. Day 2 (30-09-2017)

	1st			2nd			3rd		
# wave	% pier height	period (s)	wave height (m)	% pier height	period (s)	wave height (m)	% pier height	period (s)	wave height (m)
1	100	6,15	1,7	100	1,66	1,7	80	6,11	1,36
2	90	2,33	1,53	70	5	1,19	20	3,95	0,34
3	100	6,42	1,7	70	5,37	1,19	80	6,09	1,36
4	20	7,35	0,34	80	7,06	1,36	80	2,89	1,36
5	30	2,86	0,51	90	4,24	1,53	80	6,9	1,36
6	50	8,58	0,85	50	7,91	0,85	50	3,49	0,85
7	100	11,82	1,7	80	8,41	1,36	100	7,43	1,7
8	80	8,17	1,36	50	3,53	0,85	40	8,13	0,68
9	80	4,87	1,36	90	5,72	1,53	10	6,03	0,17
10	90	7,93	1,53	100	5,87	1,7	20	4,35	0,34
11	100	9,46	1,7	100	6,77	1,7	40	9,35	0,68
12	60	7,59	1,02	50	4,2	0,85	50	8,78	0,85
13	100	6,55	1,7	70	8,59	1,19	50	9,09	0,85
14	80	8,54	1,36	80	4,06	1,36	80	10,41	1,36
15	80	7,94	1,36	30	4,25	0,51	30	8,39	0,51
16	100	10,7	1,7	50	9,1	0,85	20	5,72	0,34
17	100	8,27	1,7	80	5,32	1,36	100	7,62	1,7
18	30	7,69	0,51	60	9,36	1,02	110	8,43	1,87
19	20	2,12	0,34	80	7,73	1,36	110	8,65	1,87
20	20	5,89	0,34	50	8,85	0,85	30	3,49	0,51
21	40	2,2	0,68	110	8,42	1,87	50	7,47	0,85
22	100	2,42	1,7	60	7,72	1,02	40	8,29	0,68
23	110	7,52	1,87	50	6,26	0,85	20	4,92	0,34
24	80	3,45	1,36	60	6,42	1,02	50	2,67	0,85
25	30	3,28	0,51	50	5,44	0,85	20	7,62	0,34

26	50	3,16	0,85	20	4,88	0,34	100	9,62	1,7
27	60	5,46	1,02	50	3,73	0,85	100	6,88	1,7
28	70	1,85	1,19	110	7,55	1,87	50	9,12	0,85
29	100	6,47	1,7	100	8,87	1,7	80	8,99	1,36
30	100	9,1	1,7	60	10,13	1,02	80	9,67	1,36
31	50	3,59	0,85	70	2,24	1,19	70	3,77	1,19
32	120	5,8	2,04	80	7,11	1,36	70	7,09	1,19
33	80	8,39	1,36	80	9,19	1,36	90	6,56	1,53
34	20	5,78	0,34	90	11,38	1,53	30	4,41	0,51
35	80	3,75	1,36	20	3,19	0,34	20	4,15	0,34
36	70	9,16	1,19	100	7,26	1,7	100	6,26	1,7
37	10	6,89	0,17	100	8,21	1,7	20	4,12	0,34
38	80	3,58	1,36	110	8,95	1,87	80	8,06	1,36
39	90	2,03	1,53	50	6,2	0,85	80	6,47	1,36
40	50	5,74	0,85	100	4,04	1,7	50	5,84	0,85
41				100	8,12	1,7	30	3,2	0,51
42				100	8,34	1,7	30	4,39	0,51
43				110	8,8	1,87	40	8,1	0,68
44				90	8,43	1,53	50	5,39	0,85
45							40	3,23	0,68
46							50	7,33	0,85
47							80	8,84	1,36
48							80	7	1,36
49							50	6,89	0,85
50							80	0,62	1,36
Average		6,02	1,41		6,68	1,50		6,45	1,16
Record duration (s)		242,74			293,69			322,27	
Mean period		6,07			6,53			6,45	

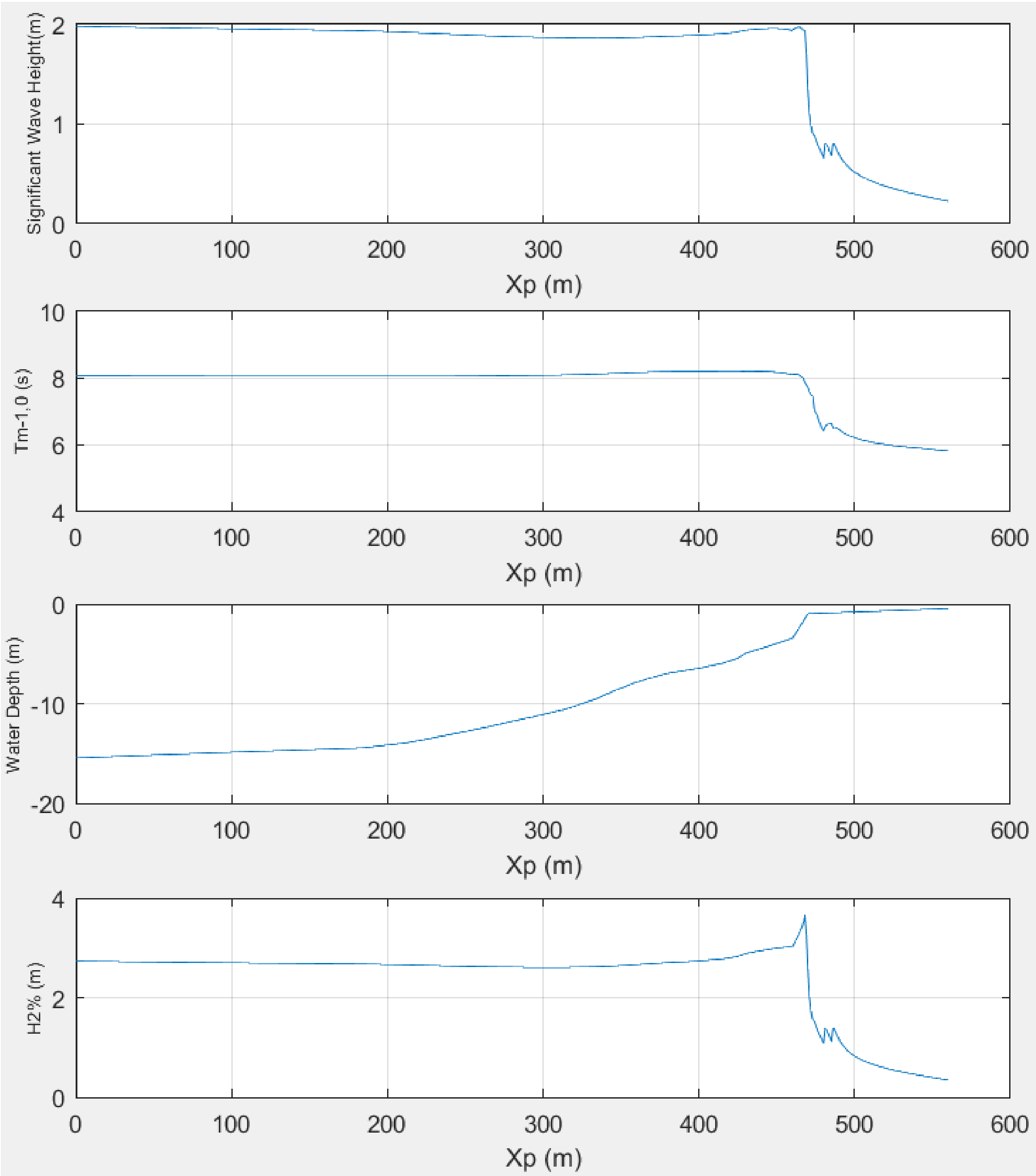


Figure 115: Cross-shore progress of several variables for severe conditions at the jetty

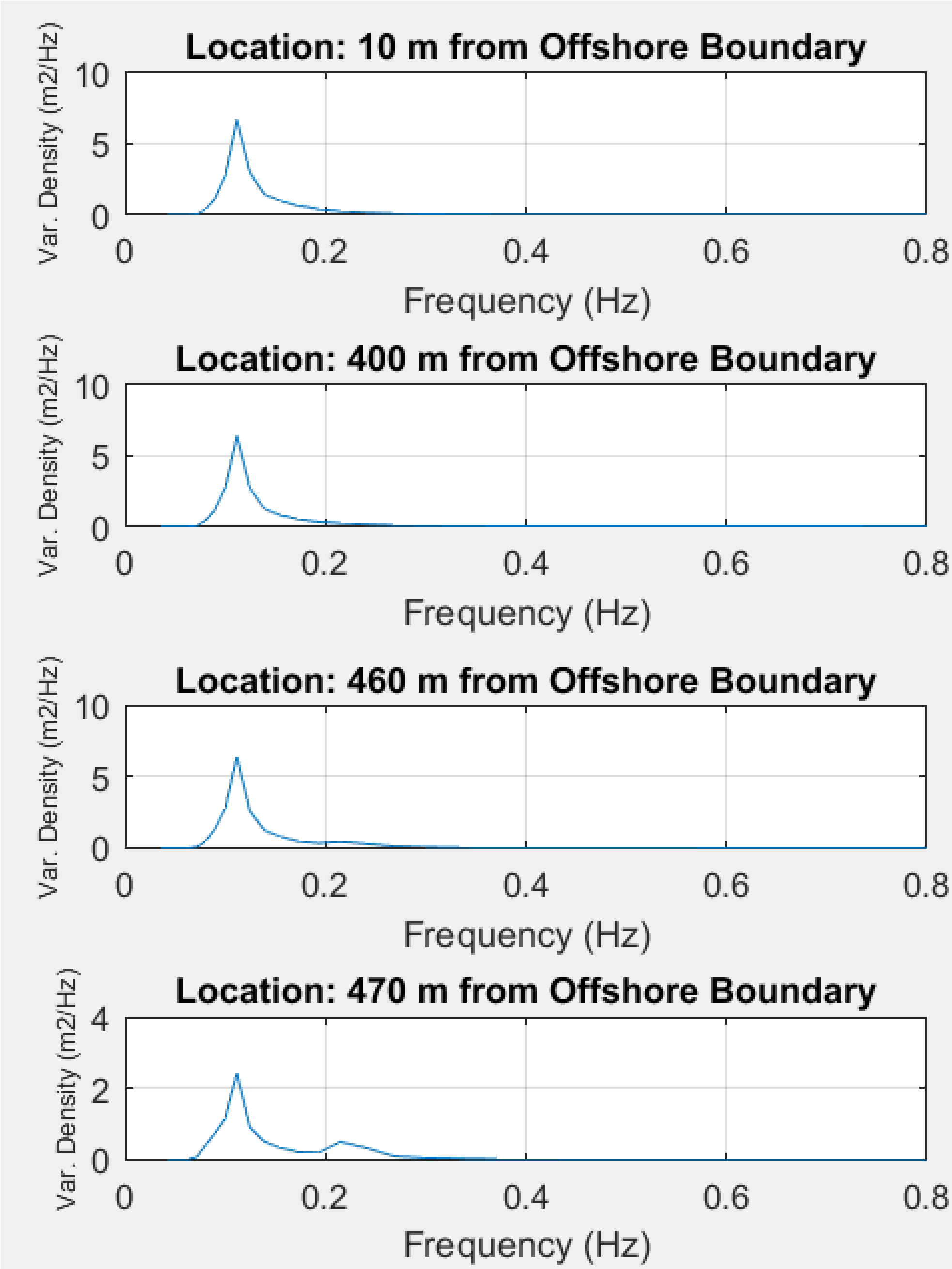


Figure 116: Energy spectra at several locations cross-shore from the jetty for severe conditions

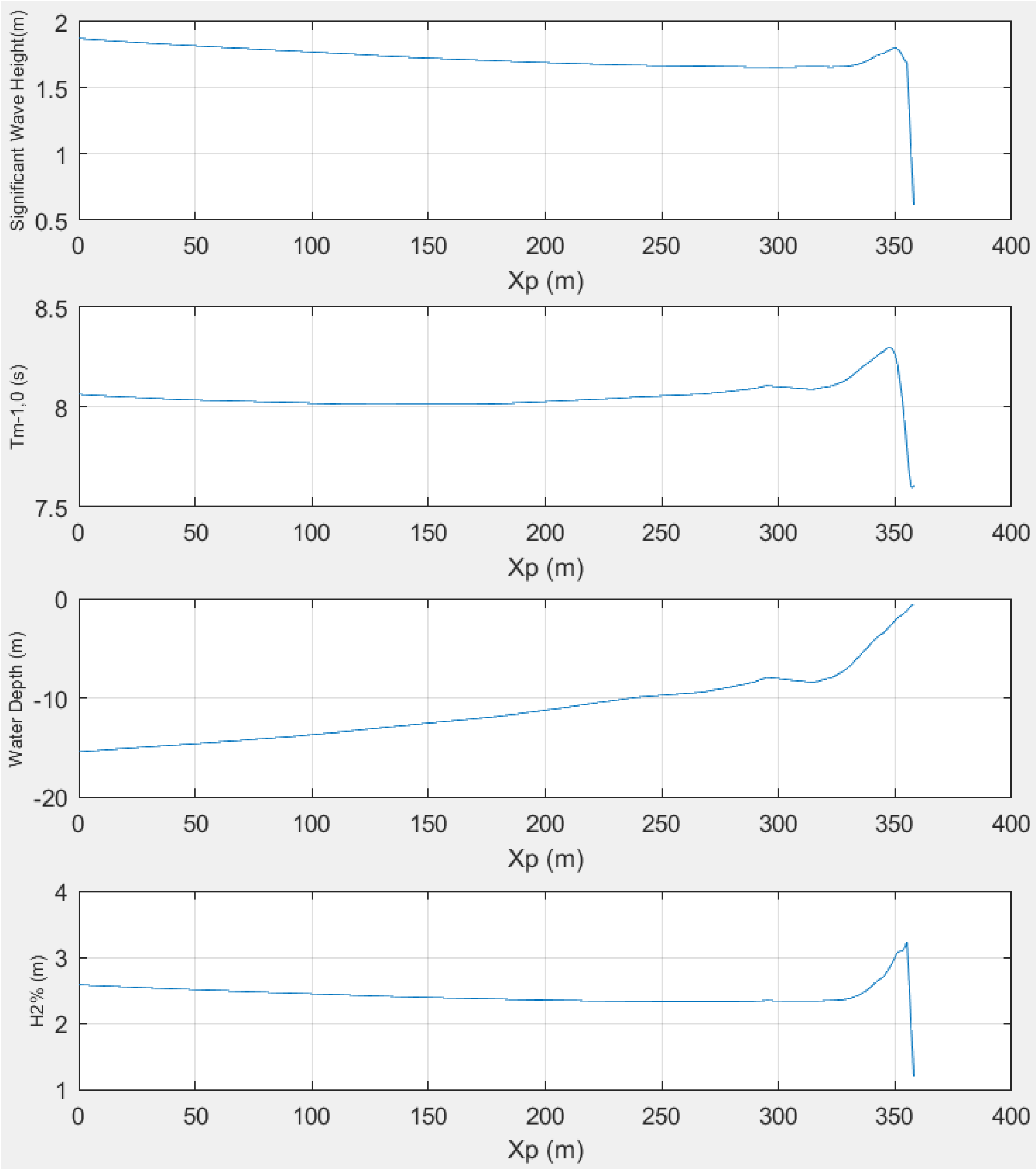


Figure 117: Cross-shore progress of several variables for severe conditions at the breakwater

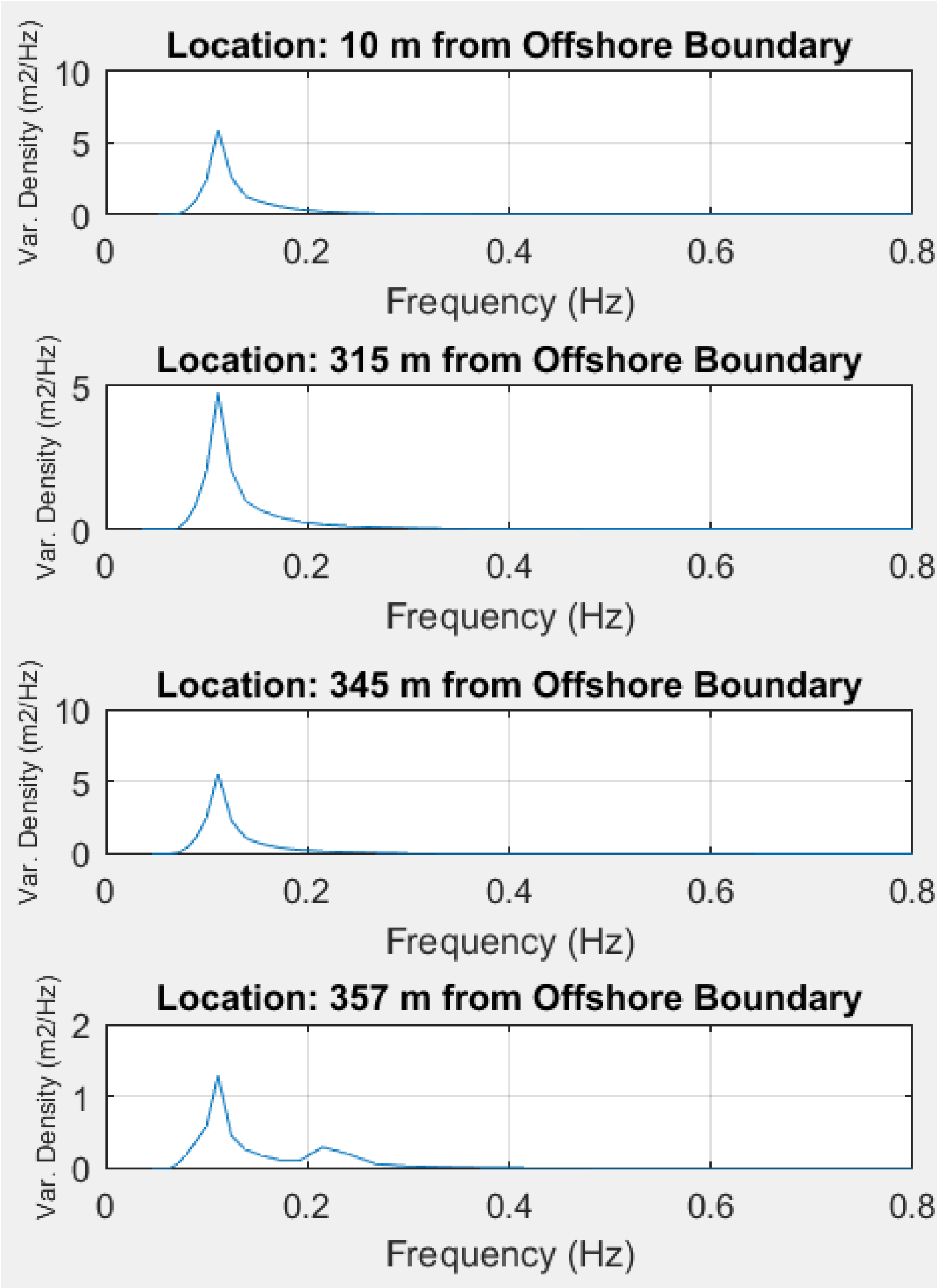


Figure 118: Energy spectra at several locations cross-shore from the breakwater for severe conditions

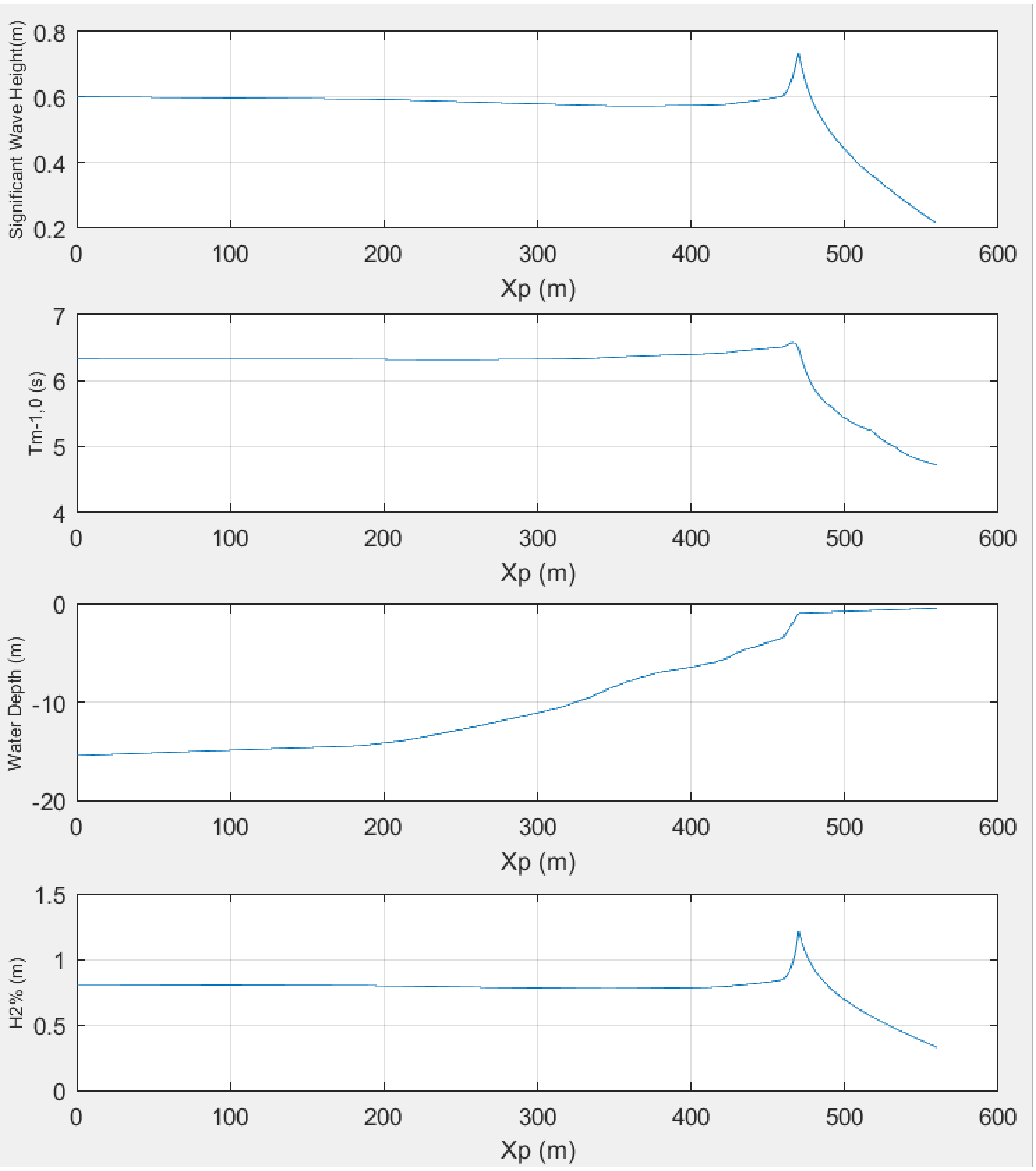


Figure 119: Cross-shore progress of several variables for mild conditions at the jetty

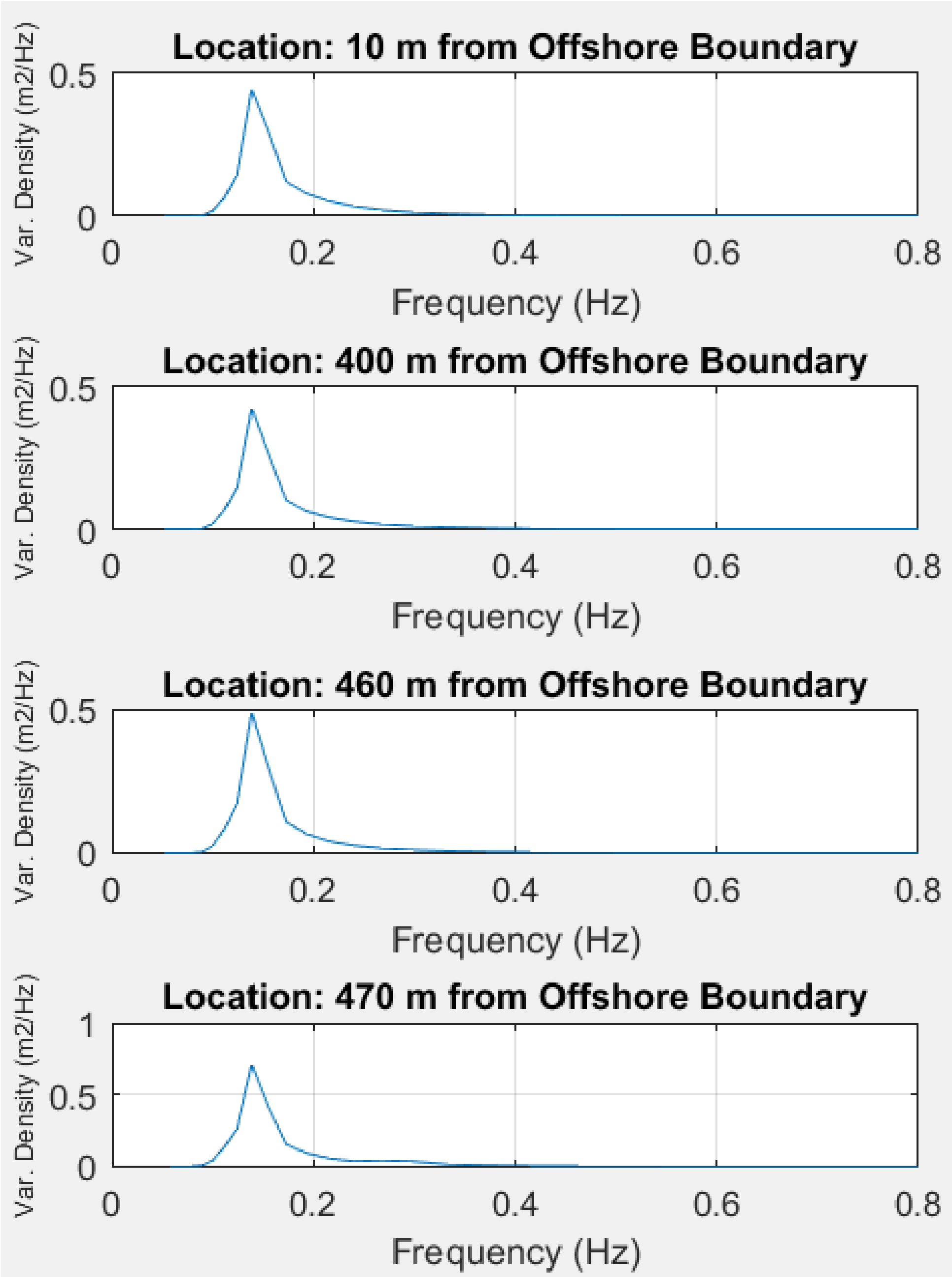


Figure 120: Energy spectra at several locations cross-shore from the jetty for mild conditions

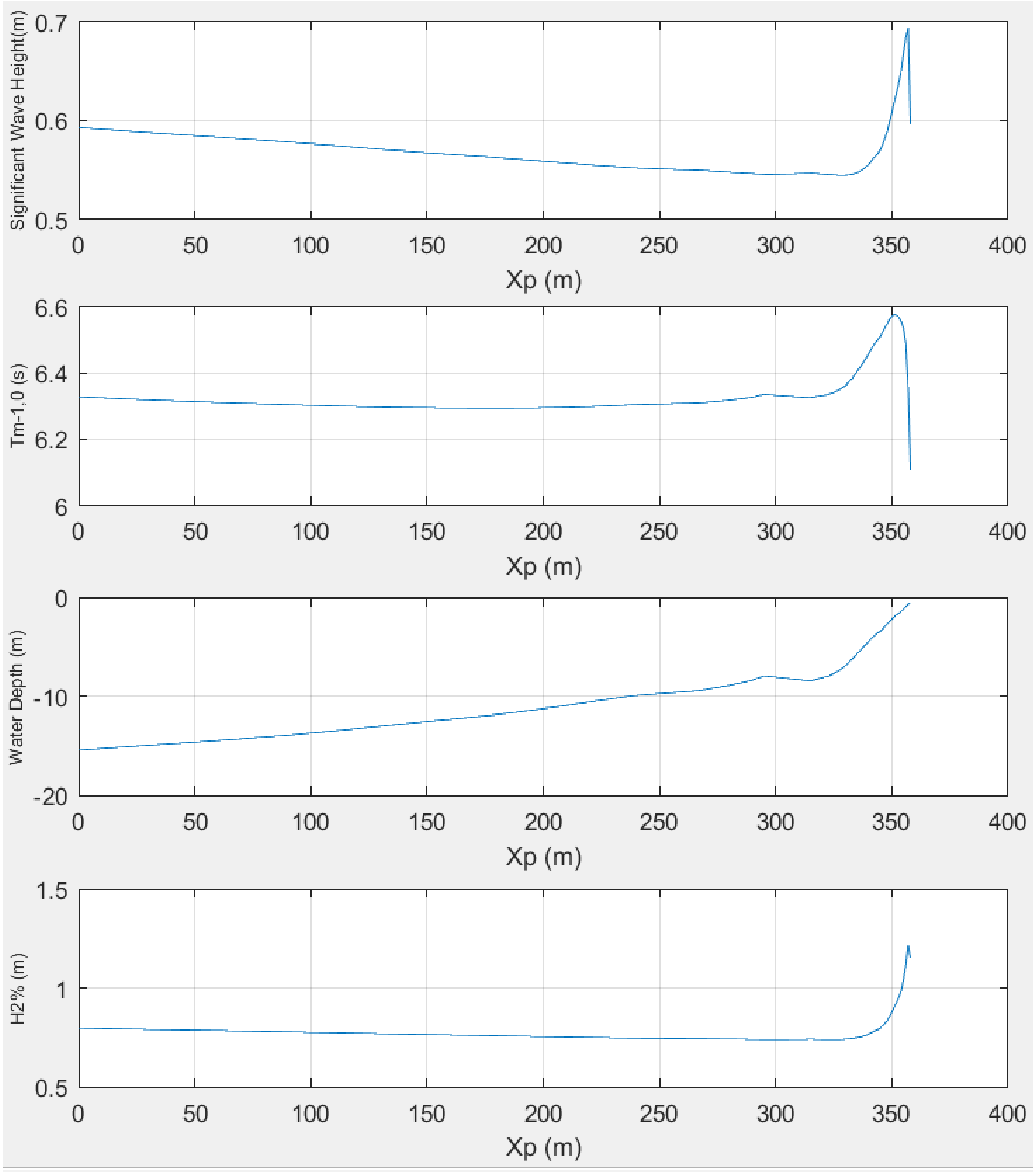


Figure 121: Cross-shore progress of several variables for mild conditions at the breakwater

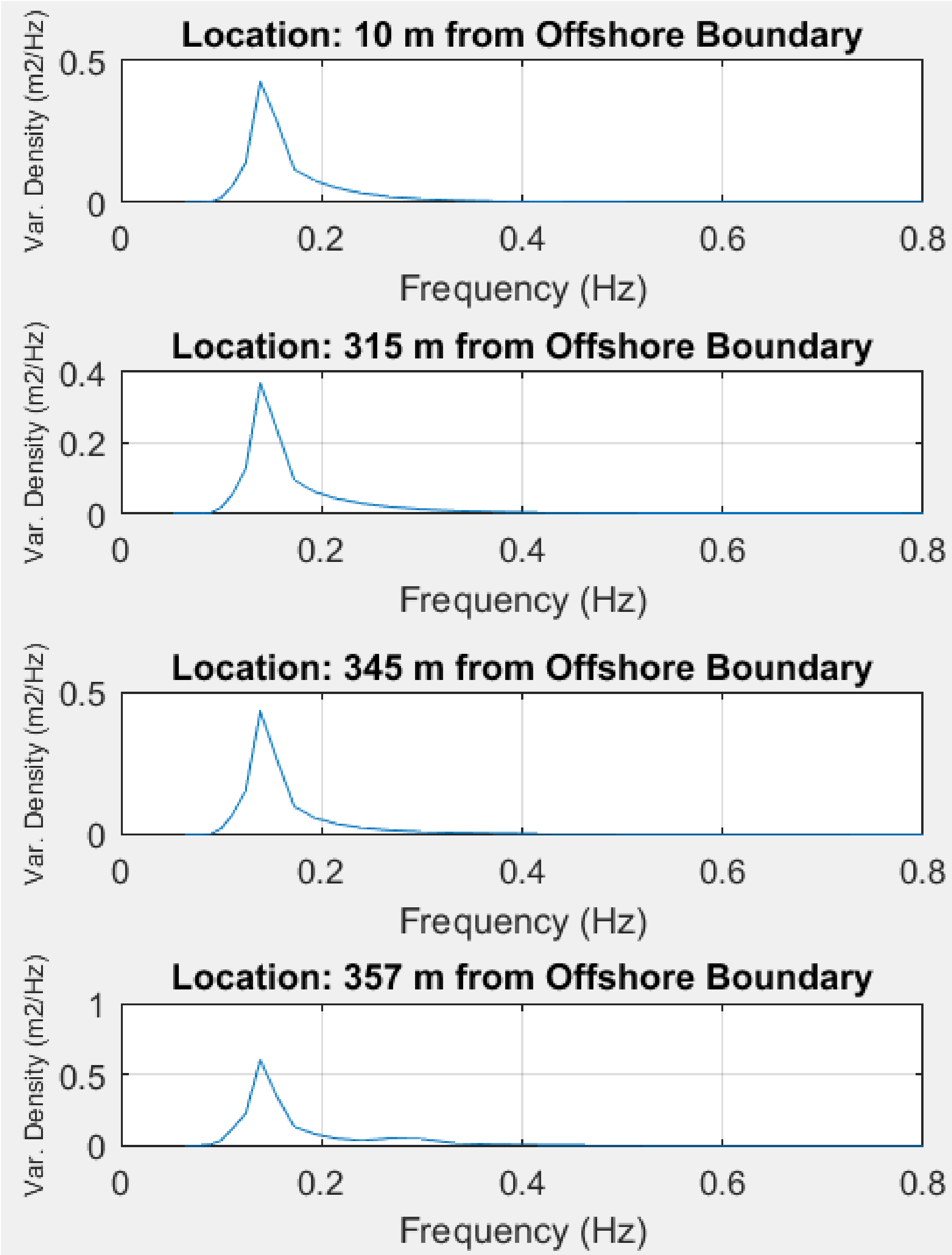


Figure 122: Energy spectra at several locations cross-shore from the breakwater for mild conditions

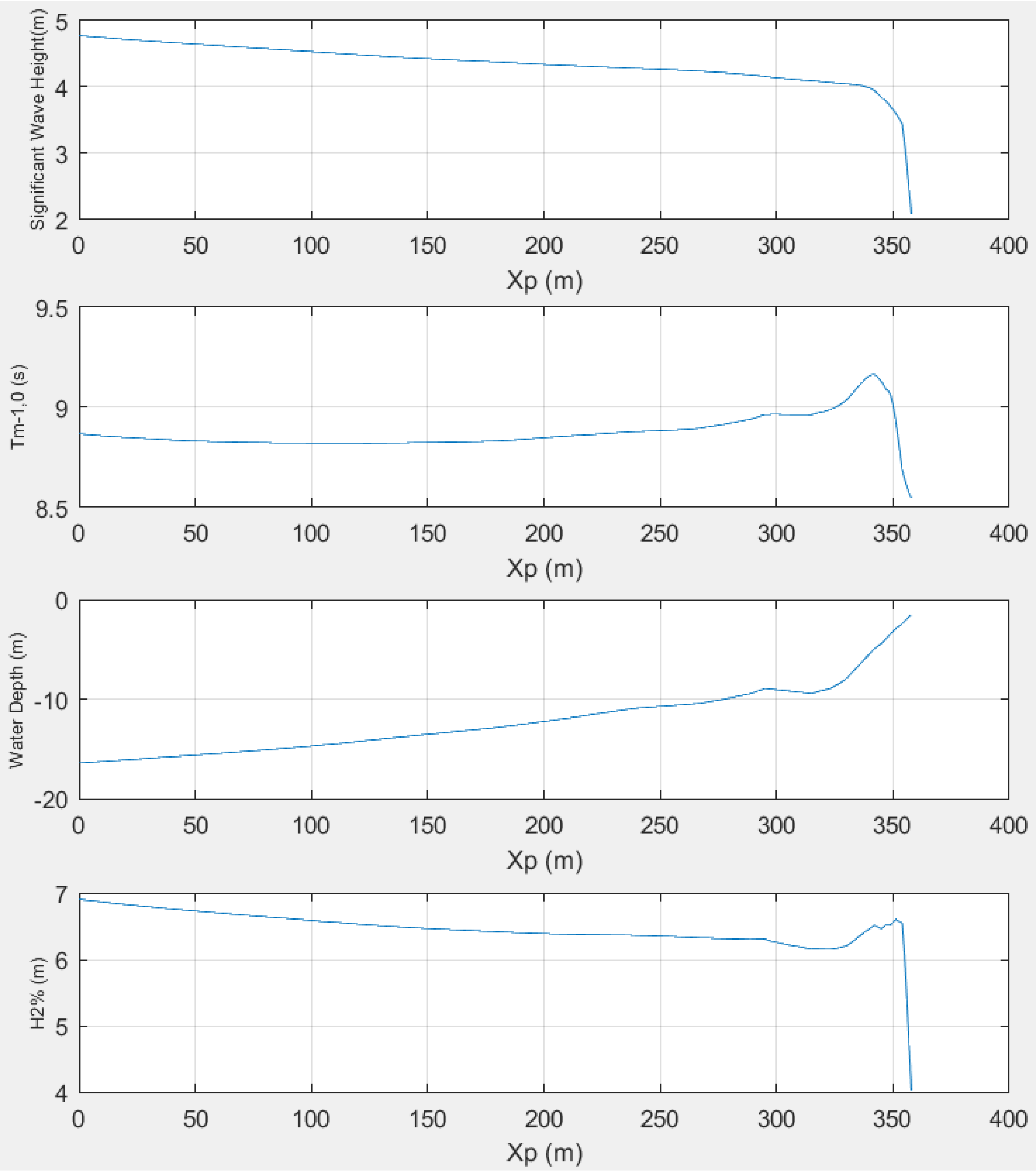


Figure 123: Cross-shore progress of several variables for extreme conditions at the breakwater

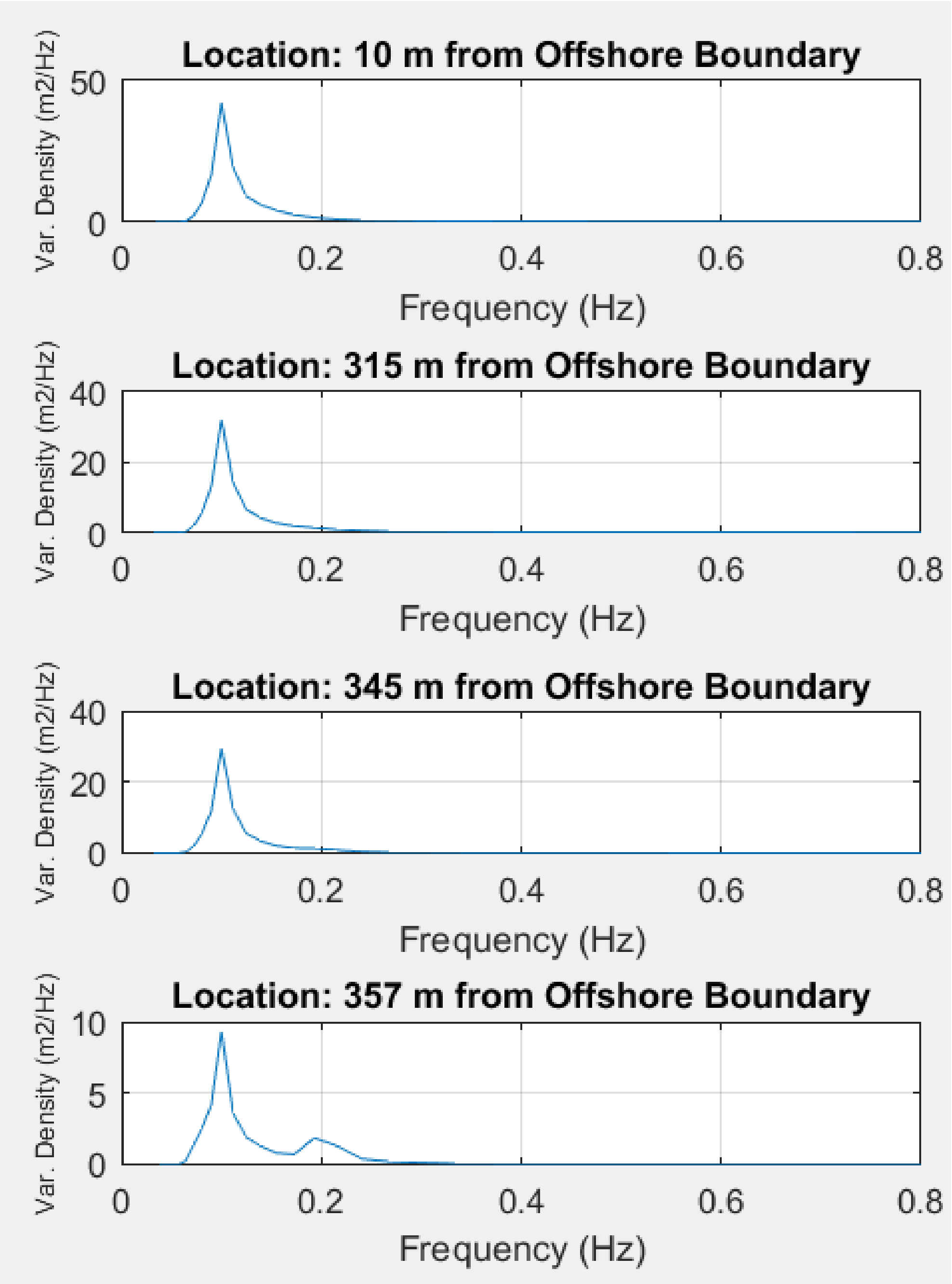


Figure 124: Energy spectra at several locations cross-shore from the breakwater for extreme conditions

Appendix F: Evaluation of habitats in marina st. Elias and bordering beaches

In this appendix an evaluation of the ecosystems is performed in the fieldwork of 2017-09-28. The evaluation exists of determining the types of habitat in the area around Grand hotel Varna. By taking a closer look at the occurring organisms and environment six different habitats could be defined. In each habitat multiple examples of plants and animals are provided.

Figure 125: Areas of ecological interest close to the marina

The ecology is an important factor to take into account when designing or constructing hydraulic structures. The current state needs to be maintained as much as possible or mitigated. In the case of protected species, no alteration of the environment is allowed. To centre the attention to the beach and marina of the hotel group, several areas of interest are defined in figure .. .

The green area represents a forested environment, which connects to the beach. The forest mostly consists of pinecone trees where a variety of fauna is present. For instance, squirrels and birds can be found. An impression is depicted in figure .. .

Figure 126: Flora in the green area

The southern forest includes a drainage channel, which provides a wider variety of species in that area.

The white area is composed by rocks, which function as an erosion protection for the coast. The rocks are surrounded by short vegetation.

Figure 127: Flora with white area

The blue area describes the marine life present in the water close to the shore. Multiple types of fauna where detected near the water line such as jelly fish, small fish and clamshells.

Figure 128: Flora in the blue area

The carbonated beach area is represented by the purple area, in which different animals are found. The life on the beach is dependent on the season, where the summer and winter differs considerably. After the summer period, the following species can be found: crabs, worms, seagulls and black birds. The worms can be found in holes present in the sand. Regarding the flora on the beach, a great amount of seaweed can be found.

Figure 129: fauna and flora in the purple area

The rock protections in the water are a habitat for clamshells located in the red area.

Figure 130: Clamshels in the red area

The yellow area entails a grass and small plants environment where birds feed. The plants are characterised by having attached several snails, as can be seen in the figure below.

Figure 131: Fauna and flora in the yellow area

Appendix G: Preliminary design of breakwater

In this appendix, a preliminary design of the marina breakwater will be performed. The input wave parameters are equal to the parameters mentioned in chapter 5.5.2. (Wave height from SWAN analysis and Svasek results), transformed in shallow conditions. The normative wave parameters are shown in Table 34:

Condition	H_s (m)	T_p (s)	RP (years)
ULS	4.0	9	1/50
SLS	1.7	9	1/1

Table 34: Input parameters design

Three failure modes will be checked in the preliminary design of the breakwater. The armour layer of the breakwater is allowed to have repairable damage after the normative storm, the same holds for the toe design. During summer, pedestrians should be able to walk over the breakwater without getting wet feet, since the marina will only be in use during summer. For that reason, more overtopping is allowed during the winter season.

Failure modes		Limit state	Return period (years)	Design method	Design value
No damage requirements (winter period)	Armour layer stable under extreme conditions	ULS	50	Armour layer stability formula	Sd=3
	Toe stability	ULS	50	Van Leeuwen (1996)	Nod=0.5
	Overtopping	ULS	50	EurOtop Manual	Q=1 l/m/s
Summer period	overtopping	SLS	1	EurOtop Manual	Q=0.05 l/m/s

Table 35 Design criteria

A. Armour Stones

The stone size of the primary armour layer of the breakwater will be determined using the formulae of van der Meer. When calculating the primary armour layer, the Hudson formula is also applicable. The difference between the van der Meer formulae and the Hudson formulae is the complexity of both formulas. The formula of van der Meer includes the effects of storm duration, wave period, the structure's permeability and a clearly defined damage level. For this reason, the formula of van der Meer will be taken into account for the calculation of the rock armour layer of the rubble mound breakwater instead of Hudson's formula.

For the determination of the armour stone units, the normative storm conditions will be taken into account. The boundary conditions, as a result of these normative storm conditions, are shown in Table 36. The relative density of the stones is based on the data from the quarry.

Boundary	Description	Value
H_s	Significant wave height	4 m
T_p	Peak wave period	9.0 s
T_m	Mean wave period	7.0 s
A	Structure slope angle	26.5° (Slope 1:2)
G	Gravitational acceleration	9.81 m/s^2
ρ_{water}	Specific density seawater	1025 kg/m^3
ρ_{stone}	Specific density stones	2475 kg/m^3
S_{0m}	Fictitious wave steepness mean wave period	0.053
ξ_m	Surf similarity parameter (mean wave)	2.17
T_{storm}	Duration normative storm	6 hours

Table 36: Boundary conditions for determining the rock armour layer

The wave steepness (s_{0m}) and the surf similarity parameter (ξ_m) can be determined with (CIRIA; CUR; CETMEF, 2007):

$$s_{0m} = \frac{2\pi \times H_s}{g \times {T_m}^2}$$

$$\xi_m = \frac{\tan\alpha}{\sqrt{s_{0m}}}$$

For the critical wave conditions, the van der Meer method for determining the armour stone stability is separated into plunging and surging wave conditions. Plunging waves will occur when $\xi m < \xi cr$ and surging waves when $\xi m \geq \xi cr$. The critical surf similarity parameter (ξcr) is determined with (CIRIA; CUR; CETMEF, 2007):

$$\xi_{cr} = (\frac{c_{pl}}{c_s} \times P^{0.31} \times \sqrt{\tan\alpha})^{\frac{1}{P+0.5}}$$

Where:

C_{pl}	Empirical coefficient (No requirement for failure)	6.2
C_s	Empirical coefficient (No requirement for failure)	1.0
P	Permeability structure (Armour layer and filter layer)	0.4
α	Angle of crest (1:2)	26.5°

$$\xi_{cr} = 3.77$$

$\xi_m \leq \xi_{cr}$ (2.17 ≤ 3.77),plunging waves will occur.

Therefore, the formula of van der Meer is equal to (CIRIA; CUR; CETMEF, 2007):

$$\frac{H_s}{\Delta \times D_{n50}} = c_{pl} \times P^{0.18} \times \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \times \xi_m{}^{-0.5}$$

Where:

D_{n50}	Median nominal diameter	Variable (m)
H_s	Significant wave height	4.0 m
Δ	Relative buoyant density $\left(\frac{\rho_{stone}}{\rho_{wat}}\right) - 1$	1.43
C_{pl}	Empirical coefficient plunging waves (No requirement failure)	6.2
P	Permeability structure (Armour layer and filter layer)	0.4
S_d	Damage parameter (Start of damage)	3
N	Number of waves at toe $\left(\frac{Storm\ duration\ (6\ hr)}{T_m\ (7.7\ s)}\right)$	2792
ξ_m	Surf similarity parameter (mean wave)	2.17
α	Angle of crest (1:2)	26.5°

According to the above equation (for the normative storm conditions),

$$D_{n50} = 1.53\ m$$

The required M50 (mass of particle for which 50% of the granular material is lighter), can be calculated using the following equation (CIRIA; CUR; CETMEF, 2007):

$$M_{50} = \rho_{stone} \times D_{n50}{}^3$$

Where:

ρ_{stone}	Specific density of stone	2475 kg/m^3
D_{n50}	Median nominal diameter	1.41 m

$$M_{50} = 6950\ kg$$

For this average weight, a standard gradation of 6,000-10,000 kg is required. The nominal diameter of this gradation is $D_{n50} = 1.44\ m$.

The relation between the diameter of the filter layer and the armour layer is approximately 2.2 to 2.5, dependent of the standard grading ratio. The D_{n50} of the armour layer is 1.44 m. This means the required D_{n50} of the filter layer is (CIRIA; CUR; CETMEF, 2007):

$$\frac{D_{50a}}{D_{50u}} = 2.2\ to\ 2.5$$

Where:

D_{50u} Nominal gradation diameter filter layer Variable (m)

D_{50a} Nominal gradation diameter armour layer 1.48 m

$$\frac{D_{50a}}{D_{50u}} = 2.2 \rightarrow D_{50u} = 0.67\ m$$

$$\frac{D_{50a}}{D_{50u}} = 2.5 \rightarrow D_{50u} = 0.59\ m$$

The M_{50} for these nominal diameters is:

$$M_{50} = \rho_{stone} \times {D_{n50}}^3$$

$$M_{50,u} = 751 - 512\ kg$$

This result in a filter layer with the gradation 300-1000 kg, the D_{n50} of this standard gradation is 0.59 m.

B. Core

The relation between the diameter of the filter layer and the core is approximately 2.2 to 2.5, dependant of the standard grading ratio. The D_{n50} of the filter layer is 0.59 m, so the required D_{n50} of the core is:

$$\frac{D_{50u}}{D_{50c}} = 2.2\ to\ 2.5$$

Where:

D_{50u} Nominal gradation diameter filter layer 0.59

D_{50c} Nominal gradation diameter core Variable (m)

$$\frac{D_{50u}}{D_{50c}} = 2.2 \rightarrow D_{50c} = 0.29\ m$$

$$\frac{D_{50u}}{D_{50c}} = 2.5 \rightarrow D_{50c} = 0.26\ m$$

The M_{50}, for these nominal diameters is equal to:

$$M_{50} = \rho_{stone} \times {D_{n50}}^3$$

$$M_{50,c} = 41 - 61\ kg$$

This results in a core with a quarry run gradation of 1-500 kg, the $M_{50}$of this quarry run is 50 kg. The quarry run is a waste product of a quarry and for this reason, less expensive than a more specific rock gradation.

C. Layer thickness

The layer thickness can be determined using the equation (CIRIA; CUR; CETMEF, 2007):

$$t_d = n \times k_t \times D_{n50}$$

Where:

t_d	Theoretical layer thickness	variable (m)
n	Number of layers	2
k_t	Layer thickness coefficient	0.87
D_{n50}	Nominal stone diameter	1.48/0.59 m (Armour and Filter)

The k_t factor is defined using a general guidance from the Rock Manual (CIRIA; CUR; CETMEF, 2007). The double layer will be placed using a standard degree of compaction. The survey of the layers will be done using the spherical foot staff. The applied rock is quite irregular with a Blockiness of 0.33 (CIRIA; CUR; CETMEF, 2007).

Using the above equation, the layer thickness of the armour layer ($t_{d,a}$) and the filter layer ($t_{d,u}$) are:

$$t_{d,a} = 2 \times 0.87 \times 1.19 = 2.57\ m$$

$$t_{d,u} = 2 \times 0.87 \times 0.63 = 1.11\ m$$

Summary	Grading (kg)		Layer thickness (m)
Armour layer	6000	10000	2.57
Filter layer	300	1000	1.11
Core	Quarry	run	na

Table 37: Results armour layer

D. Toe design

The toe dimensions will be calculated using the following boundary conditions, the water depth is the current water depth in front of the breakwater:

Boundary	Description	Value
Hs	Significant wave height	4.0 m
ρ_{water}	Specific density seawater	1020 kg/m^3
ρ_{stone}	Specific density stones	2475 kg/m^3
Δ	Relative buoyant density $\left(\frac{\rho_{stone}}{\rho_{wat}}\right) - 1$	1.42
H	Water depth	6 m

Table 38: Boundary conditions toe design

The minimum required toe armour size can be determined using the equation shown below (CIRIA; CUR; CETMEF, 2007), the formulae may be applied in the range of 3 < h_t/D_{n50} < 25:

$$\frac{H_s}{\Delta D_{n50}} = \left(2 + 6.2\left(\frac{h_t}{h}\right)^{2.7}\right) N_{od}{}^{0.15}$$

Where:

N_{od}	Damage number (Start of Damage)	0.5
h_t	Depth of toe below water level	Variable (m)

A safe boundary for applying the equation for calculating the toe armour size is (CIRIA; CUR; CETMEF, 2007): $\frac{h_t}{H_s} < 2$. The relative toe depth (h_t) is not determined yet. The significant wave height is 4.0 m. A safe value for h_t can be determined:

$$h_t < 2 * H_s \rightarrow h_t < 2 * 4 \rightarrow \; h_t < 8\,m$$

Since the value is higher than the water depth, this relative toe depth is not applicable. For that reason a value for h_t is for that reason taken as 4.5, resulting in a toe with a height of 1.5 meters.

$$\frac{4.0}{1.43 \times D_{n50}} = \left(2 + 6.2\left(\frac{5}{6}\right)^{2.7}\right)0.5^{0.15}$$

$$\frac{H_s}{\Delta D_{n50}} = 4.37$$

$$D_{n50} = 0.64\,m$$

And,

$$M_{50} = \rho_{stone} \times {D_{n50}}^3$$

$$M_{50} = 652\,kg$$

The minimum required toe grading is 300-1000 kg, with a D_{n50} of 0.59 m. This gradation is equal to the filter layer gradation.

$\frac{h_t}{D_{n50}} = 8.47$ → 3 < h_t/D_{n50} < 25 → 3 < 7.02 < 25, the formula may be applied.

$\frac{h_t}{H_s} < 2$ → $\frac{4.5}{4.0} < 2$ →$1.125 \leq 2$, a safe boundary for the toe dimensions is applied.

The relative toe height is set to 4.5 m. The total height of the toe is (Depth – relative toe height) 6.0 – 4.5 = 1.5 m.

The thickness of the toe can be defined as (CIRIA; CUR; CETMEF, 2007):

$$t_{d,toe} = 2 \times k_t \times D_{n50}$$

Where:

k_t	Layer thickness coefficient (See part Layer thickness)	0.87
D_{n50}	Nominal diameter toe gradation	0.59 m

$$t_{d,toe} = 1.0\,m$$

The remainder height of the toe (1.5 – 1.0 = 0.5 m) has to be completed using the core material of the breakwater.

The minimal toe width, conform the rock manual, is equal to 3 times the nominal toe diameter. So the toe width is designed at 1.77 meters.

The shoulder width is, conform the Rock manual, not less than 2 meters. Since, the size of the toe is only 1.77 meters; the shoulder will be constructed equal to its minimal width.

The slopes of the toe are 1:1. A schematic drawing of the toe structure can be seen in **Error! Reference source not found.**.

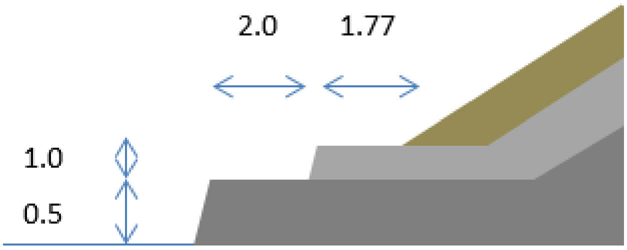


Figure 132: Schematic Toe

E. Crest height

Overtopping should be checked for the extreme winter conditions. This can be done using the following equation (Van den Bos & Verhagen, 2017):

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.2\exp\left(-2.3\frac{R_c}{H_{m0}\gamma_f\gamma_\beta}\right)$$

The mean overtopping discharge (q) can be determined for ULS and SLS requirements. In this case, since the breakwater will be well protected, SLS requirements are leading. For extreme conditions, pedestrians cannot be allowed onto the breakwater. Only trained staff should be able to have access, therefore a mean overtopping discharge of 1 l/s/m is chosen (most unfavorable situation). The significant wave height refers to the location of the toe of the breakwater. The reduction factor for roughness and permeability of the slope (γ_f) is determined based on the most unfavorable situation regarding pitched quarry stone blocks (Van den Bos & Verhagen, 2017). The maximum storm set-up is assumed to be 1.0m. The parameters are summarized below:

Parameter	Description	Value
H_{m0}	Significant wave height	4.0 m
SS_{max}	Maximum storm set-up	1.0 m
Q	Mean overtopping discharge	1 l/s/m
γ_f	Slope roughness influence factor	0.9
B	Angle of wave incidence	50°

Figure 133: Breakwater overtopping parameters for extreme winter conditions

The only parameter that is missing in order to calculate the required crest height of the breakwater is the influence factor for oblique wave attack (γ_β). This factor can be calculated with the following formula:

$$\gamma_\beta = 1 - 0.0033|\beta| = 1 - 0.0033 \cdot 50 = 0.835$$

This results in a crest height of 2.1m above the design water level. Accounting for a meter of storm set-up and 40cm of surcharges (tidal variations, temporal/local water level changes and some surplus height), the crest height should be equal to SWL +3.5m.

Considering the run-up, there should be checked for the summer conditions. During summer, no overtopping is allowed, since the area is really touristic at that moment and the marina is filled with boats. The wave run-up height should therefore be lower than the freeboard of the breakwater. The wave run-up height ($R_{u2\%}$) can be computed using the following formula from J.P. van der Meer:

$$\frac{R_{u2\%}}{H_{m0}} = \gamma_f\gamma_\beta\left(B - \frac{C}{\sqrt{\xi_{m-1,0}}}\right)$$

In this formula, the Iribarren parameter ($\xi_{m-1,0}$) is described using the following equation:

$$\xi_{m-1,0} = \frac{\tan\alpha}{\sqrt{H_{m0}/(1.56 \cdot T_{m-1,0}^2)}}$$

Summer boundary conditions are equal to the severe conditions stated in the SWAN calculations. The significant wave height is again taken equal to the one at the toe of the breakwater, even as the spectral wave period. The influence factors remain the same.

Parameter	Description	Value
H_{m0}	Significant wave height	1.7 m
SS_{max}	Maximum storm set-up	0.4 m
tan α	Tangent of the seaward slope angle	1 : 2
$T_{m-1,0}$	Spectral wave period	8.2 s
B	Curve fitting parameter	4.3
C	Curve fitting parameter	1.6

Figure 134: Breakwater run-up parameters for summer boundary conditions

The resulting Iribarren parameter is equal to 3.93, which leads to a wave run-up height of 4.8m above design water level. The corresponding value for the crest height (by summing up the maximum storm set-up) is much larger than for the overtopping in winter. This arises the question if every form of overtopping in summer is indeed undesirable. During a storm in summer, most importantly the ships should not be damaged. Furthermore, it should be still possible for pedestrians to walk on the breakwater. However, it should be expected from pedestrians that when they enter the breakwater during storm conditions, they at least are aware of possible danger and tolerate to get wet. Therefore, the mean overtopping discharge can be as high as 1 l/s/m, resulting in a crest height above design water level of 2.7m. This corresponds to a total crest height of SWL +3.1m, which is lower than for winter conditions. The dominant crest height is therefore SWL +3.5m.

The reflection in the seaward part of the breakwater is checked for the summer conditions. A low value of reflection coefficient is important for the safe sailing of the ships approaching the marina. To determine the reflection coefficient Kr the Zanuttigh and Van der Meer formula (2007) is used.

$$K_r = \tanh(a * {\xi_{m-1,0}}^b)$$

In which:
a=0.167* [1-exp(-3.2*γf)]
b=1.49*(γ_f−0.38)^2 +0.86

The roughness coefficient differs from the one used for run-up and overtopping. It is now equal to 0.55 in the most unfavorable situation (impermeable rock). This results in a reflection coefficient of 0.44. The reflected significant wave height is therefore equal to 0.75m during summer boundary conditions.

According to the EurOtop manual a very small transmitted wave height is only found if the wave overtopping is at least 30-50 l/s/m. In our case the design discharge for overtopping is 1 l/s/m. It is obvious that transmitted wave height is going to be extremely small and will not influence our structure or the land behind it.

From the preliminarily calculation, one can conclude that it is possible to construct the breakwater with a slope of 1:2 consisting of stone armour, using the biggest available stone class. However,

Item nr.	Weight [kg]	Dn [m]	Cum. Perc[%]	x [cm]	y [cm]	z [cm]	max [cm]	min [cm]	Elongation	Density [m3/kg]	Blockiness [%]
26	1,05	0,075	0,203	15	10	9	15	6	2,50	2477,33	31,4
38	1,3	0,081	0,453	14	14	7	16	3	5,33	2477,33	38,2
25	1,55	0,086	0,752	21	10	8	21	7	3,00	2477,33	37,2
39	3,45	0,112	1,418	19	17	12	21	7,3	2,88	2477,33	35,9
23	4,05	0,118	2,199	24	19	14	22	11	2,00	2477,33	25,6
22	5,7	0,132	3,299	26	10	19	26	8	3,25	2477,33	46,6
14	6	0,134	4,456	19	27	12	31	12	2,58	2477,33	39,3
16	6	0,134	5,614	20	25	11	31	11	2,82	2477,33	44,0
18	6	0,134	6,771	24	29	17	32	10	3,20	2477,33	20,5
21	6,95	0,141	8,112	32	23	16	31	8	3,88	2477,33	23,8
15	7	0,141	9,463	31	24	18	35	15	2,33	2477,33	21,1
27	8,9	0,153	11,180	33	18	21	30	10	3,00	2477,33	28,8
11	9	0,154	12,916	28	23	19	30	16	1,88	2477,33	29,7
19	9	0,154	14,652	25	28	12	34	10	3,40	2477,33	43,2
33	9,2	0,155	16,427	31	19	18	32	11	2,91	2477,33	35,0
29	9,9	0,159	18,337	23	22	20	28	12	2,33	2477,33	39,5
4	10	0,159	20,266	33	22	18	41	9	4,56	2477,33	30,9
31	10,45	0,162	22,282	33	17	24	30	10	3,00	2477,33	31,3
35	11	0,164	24,404	34	21	20	28	12	2,33	2477,33	31,1
8	12	0,169	26,719	23	37	16	42	15	2,80	2477,33	35,6
12	12,5	0,172	29,131	32	15	20	33	14	2,36	2477,33	52,6
34	12,65	0,172	31,571	31	22	17	32	11	2,91	2477,33	44,0
37	12,7	0,172	34,021	40	16	24	40	7	5,71	2477,33	33,4
32	13,4	0,176	36,607	31	25	26	30	14	2,14	2477,33	26,8
30	13,55	0,176	39,221	33	27	16	37	8	4,63	2477,33	38,4
7	14	0,178	41,921	31	32	18	37	17	2,18	2477,33	31,6
10	14	0,178	44,622	35	22	23	36	18	2,00	2477,33	31,9
13	15	0,182	47,516	42	25	28	45	16	2,81	2477,33	20,6
20	15	0,182	50,410	34	30	16	39	8	4,88	2477,33	37,1
9	20	0,201	54,268	34	47	17	48	14	3,43	2477,33	29,7
3	21	0,204	58,320	31	40	23	40	15	2,67	2477,33	29,7
36	21,05	0,204	62,381	44	27	25	47	18	2,61	2477,33	28,6
2	22	0,207	66,625	45	26	21	47	14	3,36	2477,33	36,1

6	23	0,210	71,062	47	20	29	49	16	3,06	2477,33	34,1
28	23	0,210	75,499	33	29	23	36	13	2,77	2477,33	42,2
1	24	0,213	80,129	40	29	25	40	16	2,50	2477,33	33,4
17	30	0,230	85,917	43	33	30	46	25	1,84	2477,33	28,4
5	34	0,239	92,476	55	29	16	60	14	4,29	2477,33	53,8
24	39	0,251	100,000	53	31	34	58	12	4,83	2477,33	28,2

Table 39: Rock samples' characteristics determined in situ

ITEM	**WEIGHT [G]**	**H_0 [CM]**	**H_{END} [CM]**	**D_H [CM]**	**VOLUME [CM^3]**	**DENSITY [KG/M^3]**
QUARRY_1	100,6	16,31	16,57	0,26	35,58	2827,398
QUARRY_2	18	16,12	16,3	0,18	24,63	730,7389
QUARRY_3	539,2	16,44	18,2	1,76	240,85	2238,718
QUARRY_4	825,6	16,4	18,95	2,55	348,96	2365,875
GROYNE_1	206,8	16,21	16,7	0,49	67,06	3084,017
GROYNE_2	233,8	16,08	16,77	0,69	94,42	2476,04
GROYNE_3	131,8	16,06	16,42	0,36	49,27	2675,316

Table 40: Average density of stone samples calculation sheet