PORT DESIGN - NADOR

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Preface

This thesis is the result of my graduation project concluding my Master of Science degree in Hydraulic Engineering at the Delft University of Technology. The graduation project covers various specific topics related to the design of a new port in Nador (Morocco), located in the western Mediterranean Sea. During this design project, I received much feedback and had many interesting and informative discussions with a lot of people involved in this project.

First of all, I would like to thank my Thesis Committee for all their valuable inputs and constructive feedback I received during whole the graduation project. Thank you, prof. ir. H. Ligteringen, ir. H.J. Verhagen and dr. ir. R.J. Labeur from the Delft University of Technology and ir. F.C. Vis from Alkyon (ARCADIS) for motivating and inspiring me during the process of writing of this thesis.

Furthermore, I would like to thank the people that I had to opportunity to meet and from whom I received helpful feedback during my graduation project. Amongst these are my colleagues at division Ports of ARCADIS Rotterdam and the employees of Alkyon that assisted me during my graduation project. Their opinion and assistance was highly valuable.

And last, but certainly not least, I would like to thank my parents, sister, family and friends for their continuous support during these last months. Without your attentive motivation, the hard work would have been a lot more difficult.

With these notes of thanks in mind, I wish every reader a pleasant progression while reading through this thesis.

Delft, August 2010

Robert Heuts

Summary

Along the northern coast of Morocco, order has been given for the construction of a large transshipment port in the Mediterranean Sea, at a designated project location 20 km to the west of the city of Nador. On this green-field location a new transshipment port will have to be developed for the various types of cargo: on this coastal stretch a container terminal, a liquid bulk terminal and a dry bulk terminal need to be constructed. The project location itself is characterized by a sandy beach with a length of around 8 km, enclosed between two rocky headlands. Landward of the beach, the elevation of the land increases rapidly and also the sea bed has a steep sloping bottom. Along the coastline several wades are present.

Besides the defined port's throughput specifications, additional objectives are maximizing throughput for all of the terminals, an in-phased port expansion for the terminals, incorporating possibilities for independent expansion of the bulk port and the container port, taking into account up to 20% of cargo transport to the hinterland and including enough surface space within the design for a refinery and a free trade zone.

In order to design the port masterplan layout a proper site description is indispensible. Relevant data regarding the project (location) has been identified and analysed. This data comprises of hydrodynamic data (wind, waves, currents, water levels) and environmental site data (topography, bathymetry, geology, hydrology and morphology). With the defined port's throughput specifications a plan has been formulated with the expected design vessels. Resulting from this, cargo-vessel distributions and vessel-arrival distributions have been defined and subsequently the total amount of shipping traffic.

With the determined project data, the design of the port masterplan commences with the orientation, width and depth of the approach channel, which ends in a turning basin. After an analysis from which it became clear that tugboats will have to be used for manoeuvring vessels, subsequently, all remaining manoeuvring areas, port basins and berth areas have been designed. Regarding the terminal design, a preferred allocation along the coastline has been formulated. With this decision the characteristics of the various terminals have been calculated: number of berths, quay length and surface areas. After determination of remaining surface areas for miscellaneous facilities (tugboats basin, port services), and using all port items listed above jointly, several port masterplan layouts can be drawn up.

From these layouts, the most promising port masterplan layout has been selected after comparison by means of a Multi Criteria Analysis on various criteria under which nautical ease, port zoning & efficiency, safety, expansion possibilities and costs. The selected port masterplan layout has been optimized by satisfying the cut & fill balance and the in-phase expansion of the terminals and independent port development has been outlined.

Subsequently, this port masterplan layout has been assessed regarding the topic of in-port wave penetration. For this, first of all limiting operational wave criteria have been defined and relevant wave processes have been evaluated. From a preliminary wave assessment it became clear that especially wave diffraction and wave reflection were determining processes for the in-port wave climate.

The wave simulation model DIFFRAC-2DH was applied to assess the combined influence of these two processes in more detail. After using the original port and breakwater configuration as input for the default simulation runs, it became clear that severe wave reflection could be expected, especially at the container port entrance. In order to decrease this (and thus the port's downtime), new simulation runs were carried out with an improved breakwater configuration using low-reflectivity caissons. With these wave-dampening improvements the simulation model runs yielded very positive results and the port's downtime was nearly halved. The wave study was concluded with an assessment on port oscillations as a result of earthquakes, tsunamis and meteorological forces, which could not be discarded.

After the performed wave study on the designed port masterplan layout, the port breakwaters could be designed. Two typical cross-sections are selected, one consisting of a rubble mound breakwater and one cross-section of a vertical composite breakwater. After identifying and including several construction constraints (large water depth, probability of earthquakes in the area) and determination of the design storm resulting from the maximum allowed probability of failure, the two breakwaters have been designed: the rubble mound breakwater was designed with an armour layer of $15m^3$ Accropodes II, and the vertical composite breakwater as a vertical slit caisson on a rubble mound foundation bed.
Both breakwater designs will be applied in the port masterplan layout.

The breakwater's crest heights are designed at a level that only allows a small overtopping discharge during operational conditions, creating calm in-port berthing conditions for the vessels. The breakwaters are designed to withstand limit state conditions and no failure occurs. The port facilities located in the lee of the breakwaters are thus properly protected. As a final topic, the construction methodology for the different breakwaters has been elaborated in more detail.

The designed breakwaters fulfil their function properly by providing protection from incident waves and creating sufficiently calm in-port berthing conditions. These calm in-port conditions are largely realized by the application of low-reflectivity caissons, which have proven to be indispensible within the design. Otherwise, reflection of waves against the monolithic structures would lead to large hindrance for sailing and berthed vessels and a larger (than acceptable) port downtime.

It can be concluded that with these additional wave dampening measures included, the designed port masterplan layout and its breakwaters are adequate in creating calm in-port berthing conditions: incident waves decrease considerably in height in-port, resulting in high availabilities of the berths. The original designed port layout does not have to be altered and meets all the stated specifications and requirements. However, the application of wave energy absorbing measures is a necessity in order to minimize the port downtime, and will be included in the final design. For this, the vertical slit caisson breakwater has proven to be a working solution. On other locations, the designed rubble mound breakwaters will be constructed.

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CHAPTER 1 Introduction

In order to stimulate the economic growth of the northern region of Morocco, order has been given for the construction of a large transshipment port in the western part of the Mediterranean Sea. This location is considered very favourable, as it is situated along an important intercontinental transport axis. The new port will be the largest African port present. Chapter two describes the new port its (aimed) specifications, which form the elementary basis for the design of the port.

Subsequently, additional requirements and objectives are elaborated in more detail and the exact scope of the graduation project is described. The various specific subjects focussed on in this Master of Science graduation project are partly related to Hydraulic Engineering and partly to Fluid Mechanics. Furthermore, within Hydraulic Engineering this focus is mainly on the specialism's Port & Waterways and Coastal Engineering.

The main topic comprises the design of the new port's masterplan. Within this design special attention is paid to the influence of waves and their in-port penetration and propagation. For this assessment the layout and composition of the port's breakwaters plays an essential role. Furthermore, to limit the in-port wave influence on sailing and berthed vessels, these breakwaters will have to be assessed and optimized.

In order to accomplish these goals within the timeframe of the graduation project, a certain methodology has been adopted. For this, first of all a planning of steps in time is essential. Milestones were introduced and included in this planning. One gets an overview of what needs to be done to advance through the project in a logical order.

Starting with an inventory and subsequently obtaining site data are the first steps.
From this, relevant parameters can be derived or calculated which are required for determination of the port's (wet and dry) areas. After treating specific remaining topics, several alternatives for the port's layout can be made from which the most suitable will have to be selected. After selection of this final layout a more in-depth assessment of several specific topics will be made. The port will subsequently be assessed regarding the topic of wave penetration and in-port propagation. This will yield results that can be used for the breakwater design and optimization. Concluding to this all, an evaluation of the design is made in combination with certain recommendations.

The next chapter will introduce the design project which is the basis for the graduation project.

CHAPTER 2 Project description

2.1 INTRODUCTION

The coastline of the country of Morocco is partly situated on the Atlantic Ocean, and partly on the Mediterranean Sea. It is on this last coastal section where by Mohammed VI, the king of Morocco, direct order has been given for the construction of a large transshipment port around the city of Nador (Figure 2.1 and Figure 2.2). This is in order to stimulate the economic growth of the northern region of Morocco, which is situated along an important intercontinental transport axis.

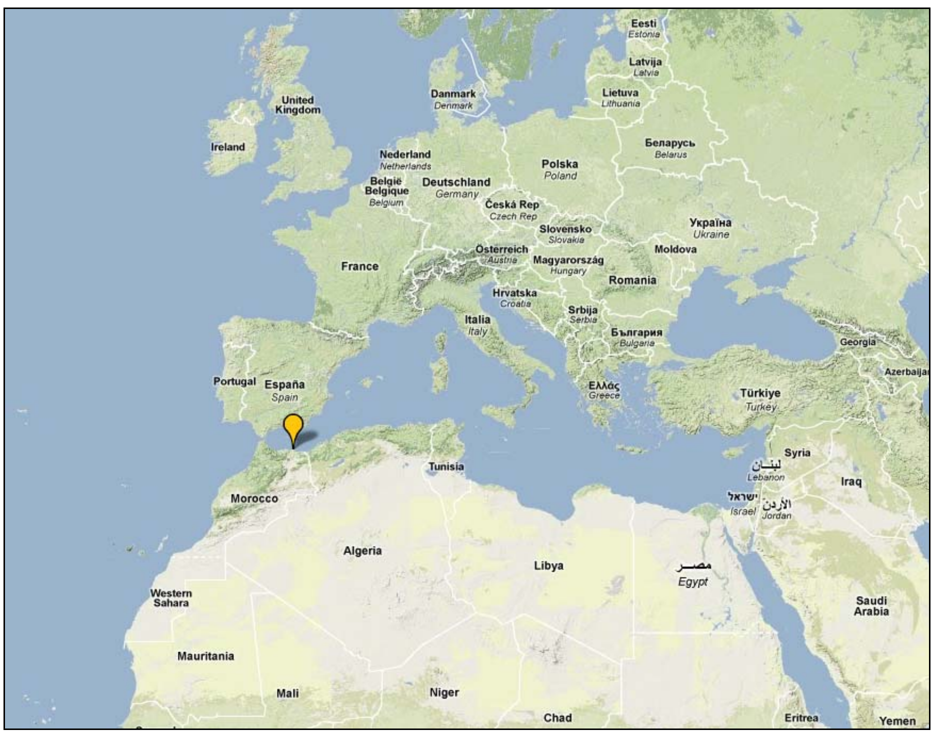


Figure 2.1

Overview Mediterranean Sea with indicated the city of Nador [GOOGLE MAPS]

This axis goes from eastern America (under which New York), crossing the Atlantic Ocean to western Europe and the Mediterranean Sea, and subsequently via the Suez Canal, to the Red Sea. From there, the transport axis crosses the Indian Ocean to Singapore and heads northwards to Shanghai. Finally, after crossing the Pacific Ocean to western America (under which Long Beach) the circle of the main liner shipping trade around the world is complete.

In the following paragraph, the project and its location will be elaborated in more detail. Additional images are presented in annex 1.

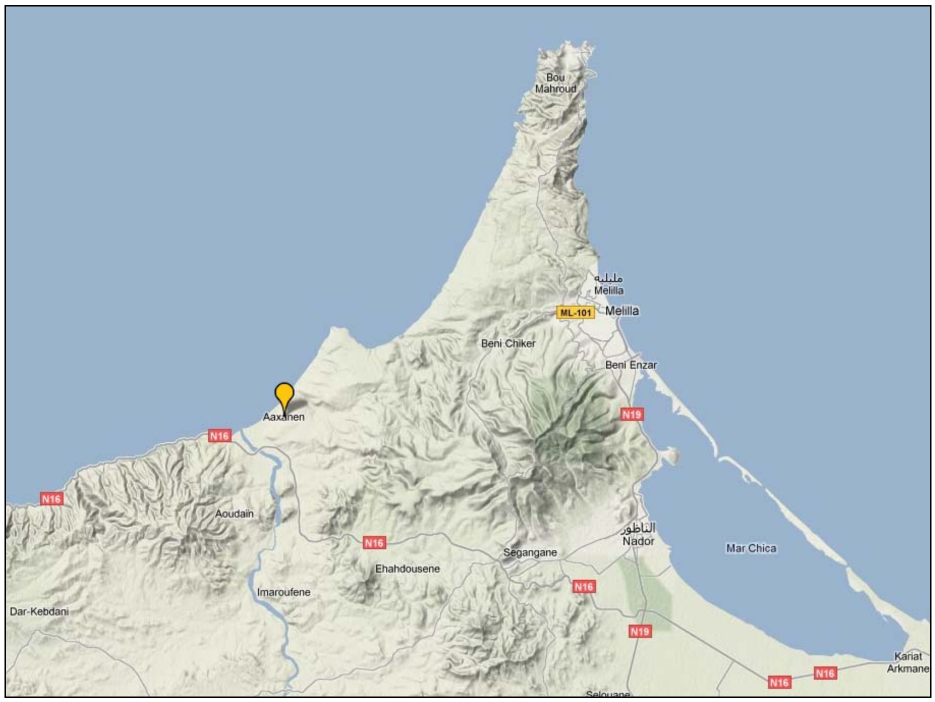
2.2 PROJECT INFORMATION

2.2.1 LOCATION

A certain location has already been designated for the development of this port. This location is approximately 20 km to the west of the city of Nador, which is situated in the Rif region. This is a mainly mountainous region of northern Morocco. From the figures below and in annex 1 it is clear that this region exhibits a certain degree of topographic relief.

Figure 2.2

Global overview of topographic relief around project location [GOOGLE MAPS]



In Figure 2.3, the exact coastal section is indicated that has been designated for the construction of the port. This coastline has a length of approximately 8 km.

Figure 2.3

Indication of coastal section available for port development [GOOGLE MAPS]

The figures in annex 1 give a somewhat more detailed overview of the project location, the bathymetry and the topography. It is emphasized that the above presented figures only give a global overview of the project location. An elaborated description of the project site itself will be outlined in chapter 3.

2.2.2 INFRASTRUCTURE

In Nador, already a Mediterranean port is present. The main port of Nador – Beni Enzar is located northwards, against the south of the autonomous Spanish exclave Melilla.
Nador has a (small) international airport with links to Morocco's main airports and some European destinations. Besides that, also several ferries sail from Melilla and Nador to the European continent. Figure 2.2 also presents a global indication of the already present (road) infrastructure. Near the designated location this is only the road named N16. There is also a railway present which runs from Selouane in the south and ends to the north in Nador.

2.2.3 SPECIFICATIONS TRANSSHIPMENT PORT

As mentioned before, in Nador already a port is present. This port is mainly used as a trade centre for fish, fruit and livestock, and for passenger transport. The new port to be developed will be a transshipment port, for the various types of cargo. On this coastal area a container terminal, a liquid bulk terminal and a dry bulk terminal need to be constructed. For the liquid bulk terminal applies that in the (near) future, space needs to be available for the construction of a refinery.

The transshipment aspect requires special attention in the port design. Transshipment has increased rapidly throughout the past decades, and the trend is that main lines call only on a few ports in their route, with feeder ships collecting and distributing the cargo within a region around such a main port [UNCTAD, 1985c], [WIKIPEDIA]. The transfer from main line vessel to feeder and vice-versa is called transshipment.

It is requested that the new transshipment port can be developed in different phases.
For these phases, the specific requirements defined by the client are outlined below.

Container terminal

Phase I: 2-3 MTEU/year incoming
2-3 MTEU/year outgoing
This amounts to 4-6 MTEU/year to be handled by the cranes.

Phase II: 7.5-15 MTEU/year incoming
7.5-15 MTEU/year outgoing
This amounts to 15-30 MTEU/year to be handled by the cranes.

Liquid bulk terminal

Phase I: 7.5-10 MT/year incoming oil products
7.5-10 MT/year outgoing oil products
Phase II: 7.5 MT/year incoming oil products
7.5 MT/year outgoing oil products
12.5 MT/year incoming crude oil
12.5 MT/year outgoing oil products (these products will then be obtained from the refinery nearby which processes the crude oil)
For this final phase, a terminal area of 200-300 ha should be available.

Dry bulk terminal

Phase I & II: 2-2.5 MT/year incoming dry bulk
Phase I & II: 2-2.5 MT/year outgoing dry bulk
It is not specified which cargo will be handled here, but the design should include a quay with 700 m. in length.

Here, Phase II equals the final phase of the port, with its required (maximum) capacity.

From the above specifications is clear that the amount of incoming cargo equals the amount of outgoing cargo: all the cargo will thus be transshipped. This will have its specific influence and poses special constraints on the design of the new port.

2.2.4 ADDITIONAL OBJECTIVES AND REQUIREMENTS

Besides the above mentioned throughput specifications, there are several additional objectives and requirements defined by the client for the new port. These are the following:

- The throughput for the total port will have to be maximized. The throughput specifications from paragraph 2.2.3 are not defined that sharply. The challenge of this project is: look at the (maximum) possibilities at the specific project location.
  Only a certain area of land is available, where as much throughput as possible should be realized.

- Another item that requires special attention is the following: the client has requested that the possibility should exist that the bulk terminals and container terminal can be developed independently from each other. So the design has to incorporate possibilities for this independent construction and future development. This is because of the fact that it is at this stage unknown whether the container terminal will be constructed in the future or not. This poses special constraints on the development of the total port, as the layout must include a certain degree of flexibility to include future expansion.

- It should be possible that 10-20% of the total container throughput can be transported to the hinterland by road and by rail. This will have its specific influence on the terminal layout (transfer areas) and its required surface area. Besides this, new infrastructure will have to be constructed to accommodate this amount of traffic.

- And finally, nearby a certain area of 1000-1500 ha should be available which will function as a Free Trade Zone. Because of this large area requirement, one has to look at the possibilities at the project location.

2.3 SCOPE OF GRADUATION PROJECT

The scope of the graduation project focuses on several specific topics. These subjects are (more or less) interlinked, but they can be divided into three main topics:

I. **Developing the new port masterplan**.
After collecting and analyzing relevant data (e.g. on cargo forecasts and site conditions), essential parameters will be derived which are required for drawing up the new port masterplan, taking into account the above mentioned additional objectives and requirements. First of all, the dimensions of the 'wet area' of the port will be determined (e.g. approach channel, basins, turning circle), and subsequently the 'dry area'. This last topic includes determining the number of berths required, the necessary dimensions for the quays for different types of cargo and the dimensions and locations of the various terminals. After considering additional aspects, various layouts of the new port can be designed, and these will be evaluated and compared to each other by means of a MCA.

II. **Performing a wave penetration study**.
The second topic will be a consideration of the wave penetration into the port.
Several (simulation) models are available for this purpose, which take into account various phenomena (e.g. diffraction, refraction). One of these will be selected and applied to the specific earlier determined layout of the new port (see 'I.'). The output of this model will be analyzed and evaluated, and (if necessary) accompanied with recommendations to improve the results.

III. **Designing a breakwater**.
In order to create calm in-port conditions for moored ships, it is essential to design a breakwater which ensures that this is accomplished. After the wave study on the selected port layout, a proper design for the breakwater will be made. From the new port masterplan (developed under 'I.') and the wave penetration study (see 'II.'), the layout and orientation of the breakwater will be clear, so subsequently the technical design can start after determination of relevant parameters required for this. Special attention will be paid to several elements in the cross-section (e.g. the armour layer).

2.4 READING GUIDE

Now that the project specifications, the objectives and the scope of the graduation project have been defined and described in detail, the core topics will be elaborated in the next chapters.

The composition of the thesis is as follows:
Chapter 3: Project data
Chapter 4: Port masterplan
Chapter 5: Wave penetration study
Chapter 6: Breakwater design
Chapter 7: Evaluation and recommendations

Additional info is presented in several annexes at the end of this report. References to various literature and other publications are made throughout the whole report and are indicated by [AUTHOR, YEAR], presented in annex 6.

CHAPTER 3 Project data

3.1 INTRODUCTION

In order to get acquainted with the project location and its characteristics, it is first of all indispensible to have a proper description of the site. Relevant data regarding the project has to be identified and determined. This data (under which environmental forces, geology, hydrology and morphology) will be presented in the following paragraphs and will prove to be essential throughout the whole (wet and dry) port design.

Besides this site data, from the previous chapter it is clear that a cargo forecast in the form of specifications for the port have been determined beforehand. These are the only specifications given for the project. The cargo forecast can be translated into the amount of shipping traffic, when taking into account the present day shipping market from which the specific characteristics of the vessels can be determined.

For the determination of the (wet and dry) port areas, the above determined design vessels in combination with the environmental conditions have to be taken into account. This will be done at the end of the chapter, where the influence of the hydrodynamic forces on the ships is assessed.

3.2 OVERVIEW PROJECT SITE

The project location is characterized by a beach consisting of medium sand and pebbles, enclosed between two headlands. The headlands are steep cliffs rising to 30 m. from the waterline. The width of the beach is in the order of several hundreds of meters with dunes covered with some vegetation. From north to south, the width of the beach increases somewhat. The total length of the beach is around 8 km.

Landward of the beach, the elevation of the land increases rapidly and mountains dominate the landscape. Along the coastline several wadis can be identified which have cut their way through the rock. The figure below presents a satellite overview of the project site, with indicated the different wadis.

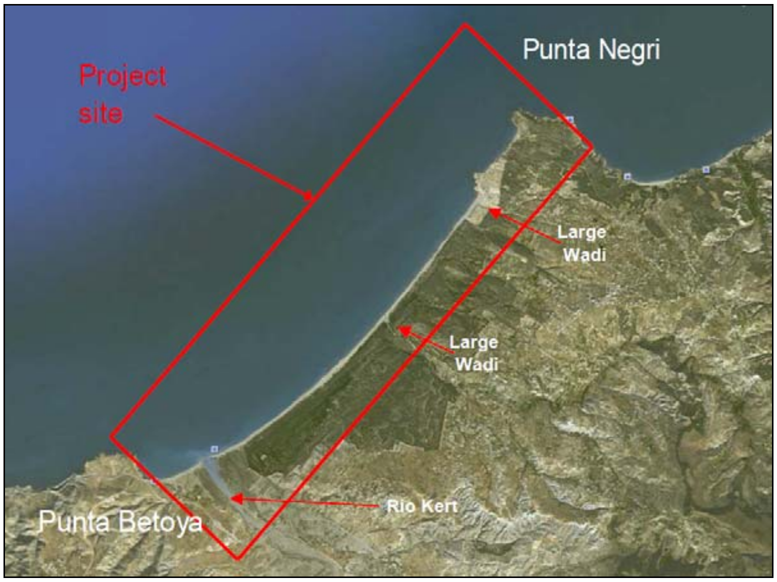


Figure 3.1
Project site with indicated the different wadis [GOOGLE MAPS]

Although no exact hard boundaries for the project site (inland as well as seawards) have been specified (only the indication above), they are mainly determined by practical limits: the terrain has a rather steep slope which makes construction of the terminals and breakwaters very expensive when situated far from the coastline. This will be clarified at the end of this paragraph. First of all, the main features of the project location are described below.

Headlands

The sandy beach at the project location is enclosed by two headlands: Punta Betoya to the south and Punta Negri to the north. The headlands have steep slopes and typical heights of about 30 m. above the water level. The pictures [ALKYON DATA] below give an impression of the headlands.

Figure 3.2
Headlands [ALKYON DATA]
left: Punta Betoya
right: Punta Negri

Near the headlands, rocky outcrops are a common feature at the sea bottom. This material is very hard and makes dredging difficult and expensive. [ALKYON DATA] states that this rock is probably too hard to dredge with a cutter suction dredger. Therefore, the location is unsuitable for (much) deepening.

Sandy Beach

The beach consists mainly of medium sand. In general, the sandy beach and sand dunes dominate the view. About 1 km. northeast of Punta Betoya, the Rio Kert flows into the Mediterranean Sea. In this area of discharge only close to the river mouth, also pebbles can be identified. The pictures below give an impression of the project site [ALKYON DATA].

Figure 3.3

Left: Looking to the north from Punta Betoya. The outflow of the Rio Kert can be identified [ALKYON DATA].

Right: Looking to the south from Punta Negri. The sandy beach with a steep inland sloping terrain is clearly visible [ALKYON DATA].

Wadis

Along the coastline, several wadis exist (including the Rio Kert). They have cut their way through the mountains and cliffs. The wadis do not carry water in their channels during the entire year, but only in times of heavy rainfall. The Rio Kert transports sediments to the beach shore, which created a beach between the two headlands.

The detailed characteristics of the project site will be addressed in 3.4. But for this, first of all an inventory and assessment of the hydrodynamic forces is a necessity in order to formulate a more detailed project site description (e.g. morphology).

3.3 HYDRODYNAMIC DATA

Throughout the whole development of a port masterplan, environmental data is indispensible. Hydrodynamic data is required for a detailed site description, the design of the access channel and manoeuvring areas, design of structures and infrastructure and the assessment of downtime, sedimentation and navigation. For this, data is required on tides, currents, wind and waves. Most of this data is made available in [ALKYON DATA].

3.3.1 WIND CLIMATE

The information on wind and waves is required for the design of the approach channel and manoeuvring areas, the design of the structures required for port infrastructure (including breakwaters), assessment of downtime and sedimentation, the wave climate study and mooring and navigation studies.

Various data sources were used in order to derive the wind climate near Nador, according to reports from [ALKYON DATA]:

1. Records of ship observations offshore Nador, during the period 1960 to 1997,
2. Directional occurrence statistics from Satellite measurements (scatterometer), based on 9190 samples from 3411 passes,
3. Directional occurrence statistics from the hind cast model WANA at model point WANA2025004 near Nador.

After comparison and evaluation of the above mentioned sources by [ALKYON DATA], the results of the satellite is deemed to be the most suitable data set for normal and extreme wind conditions. The definitive wind climate is presented below.

Table 3.1

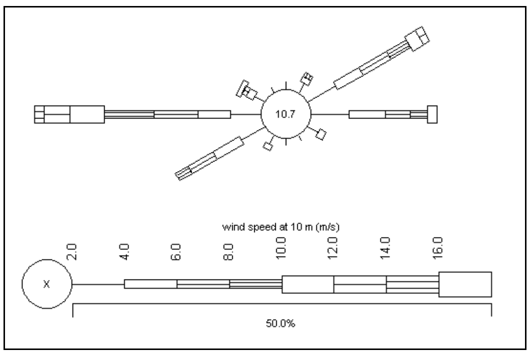
Probability of occurance (%) of wind speeds offshore Nador (35.49°N, 3.02°W) at 10 m. height [ALKYON DATA]

U10 (m/s)		wind direction (Deg)												
		-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
Lower	Upper													
<	2,0	,85	,79	1,23	1,89	,72	,46	,58	,58	,78	1,69	,59	,59	10,74
2,0	4,0	,87	1,25	3,81	4,49	1,37	,50	,63	1,08	3,22	3,69	1,36	,61	22,89
4,0	6,0	,31	,69	3,47	4,38	,86	,13	,22	,66	3,61	3,83	,90	,35	19,41
6,0	8,0	,06	,49	3,82	2,95	,27	,04	,01	,14	3,43	5,33	,58	,09	17,21
8,0	10,0	,04	,53	2,74	2,08	,09	,01	,04	,08	1,74	5,89	,49	,11	13,84
10,0	12,0	,08	,24	1,54	1,10	,01	,00	,04	,01	,39	4,19	,54	,02	8,16
12,0	14,0	,00	,26	,81	,46	,00	,00	,02	,00	,21	2,98	,33	,00	5,07
14,0	16,0	,00	,05	,27	,10	,00	,00	,00	,00	,03	1,22	,15	,00	1,83
16,0	18,0	,00	,04	,09	,03	,00	,00	,00	,00	,02	,49	,01	,00	,69
18,0	20,0	,00	,00	,00	,00	,00	,00	,00	,00	,00	,11	,01	,00	,12
20,0	>	,00	,00	,00	,01	,00	,00	,00	,00	,00	,03	,00	,00	,04
Total		2,21	4,35	17,79	17,49	3,32	1,14	1,55	2,55	13,44	29,44	4,96	1,76	100,0

And in a graphical interpretation (wind rose):

Figure 3.4

Wind Rose (offshore Nador)



The extreme wind conditions were derived using a statistical analysis tool called 'Hydrobase', by fitting the Weibull distribution to the cumulative frequency distributions of wind scatterometer data, which was executed by [ALKYON DATA]. This has resulted in the extreme wind conditions presented in the table below.

Table 3.2

Wind speeds in m/s at 10 m. height [ALKYON DATA]

U10	wind direction (°N)											
	-15	15	45	75	105	135	165	195	225	255	285	315
Return period	15	45	75	105	135	165	195	225	255	285	315	345
1 Year	8,6	16,8	17,4	16,8	9,1	6,6	7,8	9,0	13,3	20,0	17,1	9,6
5 Year	10,8	20,0	19,4	19,1	10,8	8,5	10,0	10,8	14,7	21,9	19,5	12,0
10 Year	11,7	21,3	20,2	19,9	11,4	9,3	10,8	11,6	15,2	22,6	20,4	13,0
25 Year	12,9	22,9	21,2	21,1	12,2	10,3	11,9	12,5	15,9	23,5	21,5	14,2
50 Year	13,7	24,1	21,9	21,9	12,8	11,0	12,7	13,1	16,3	24,2	22,3	15,1
100 Year	14,6	25,3	22,6	22,7	13,3	11,7	13,5	13,8	16,8	24,8	23,1	16,0
200 Year	15,4	26,4	23,3	23,5	13,9	12,3	14,2	14,4	17,3	25,5	23,8	16,9

3.3.2 WATER LEVELS

The tidal levels around Nador have been estimated on basis of astronomical constituents, available from satellite based information [ALKYON DATA]. For the tidal analysis, a large scale tidal flow model for the western Mediterranean Sea was used in combination with Tidal Analysis Software [ALKYON DATA]. This tidal analysis results in the following water levels:

Table 3.3

Water levels with respect to Chart Datum in Nador (35.25°N, 3.17°W) [ALKYON DATA]

Water levels	[m] wrt CD
HAT	+0.66
MHWS	+0.57
MHWN	+0.47
MSL	+0.35
MLWN	+0.23
MLWS	+0.13
LAT	+0.00

Wind and barometric pressure affect the water levels, leading to extreme water levels. The wind set-down has been calculated according to the following formula [d'ANGREMOND, 2001], taking into account 1:200 year wind conditions (U=26,4 m/s):

$W = c*U^2/(gd)*F$

In which:
W = wind set-up/set-down [m]
c = factor: $4*10^{-6}$ [-]
U = wind speed [m/s]
g = gravity: 9.81 [m/s^2]
d = average water depth [m]
F = length of wind influence [m]

The wind set-up/set-down has been calculated for several fetch lengths with their accompanying depths (to the north, where the slope of the sea bottom is the steepest). The results are summarized in the table below:

Table 3.4

Fetch lengths and accompanying calculated wind set-up [m]

Fetch length [m]	Average d [m]	Wind set-up [m]
5.000	25	**0,057**
10.000	45	**0,063**
15.000	70	**0,061**
20.000	110	**0,052**
30.000	190	**0,045**

Because of the steep seabed slope, the influence of wind set-up even for 1:200 year wind conditions is small: at maximum assumed to be 0,07 m.

Besides the wind set-up, there is the variation of the water level because of variations in atmospheric pressure [PIETRZAK, 2008] which adds to the extreme water level. For the height of the corresponding static rise of the mean sea level (MSL), [USACE, 2002] states that:

$z_a = 0.01*(1013-p_a)$

In which:

z_a = rise of water level [m]
1013 = mean air pressures at sea level [mbar] of [hPa]
p_a = atmospheric pressure at sea level [mbar] or [hPa]

Regarding the parameter p_a, [USACE, 2002] states that in tropical storms pressures may drop to 900 mbar. Within storm zones in somewhat higher latitudes pressure variations from 960 to 1040 mbar are common. With a safe estimation of the atmospheric pressure in accordance with meteorological data [WEBSITE METEO24], the corresponding height of static sea level rise becomes:

$z_a = 0.01*(1013-990)=0{,}23$ m.

This has been included with a safety margin and rounded off to a total of 30 centimetres.

With the above calculated parameters, the extreme water level can be determined.
Over 99% of time, water levels are lower respectively higher than LAT and HAT.
This leads to an extreme water level as calculated in table 3.5.

Table 3.5

Calculation of extreme water level [m]

Components	Water level rise [m]
Wind set-up	+0,07
Atmospheric pressure effect	+0,30
HAT	+0,66
Total water level wrt CD	**+1,0**

Sea level rise can not be predicted very accurately; however this is estimated to be a total of 0.5 m. for the next 50 years [PIETRZAK, 2008]. This additional water depth will be included in the (e.g. breakwater) designs as required throughout the next chapters.

3.3.3 CURRENTS

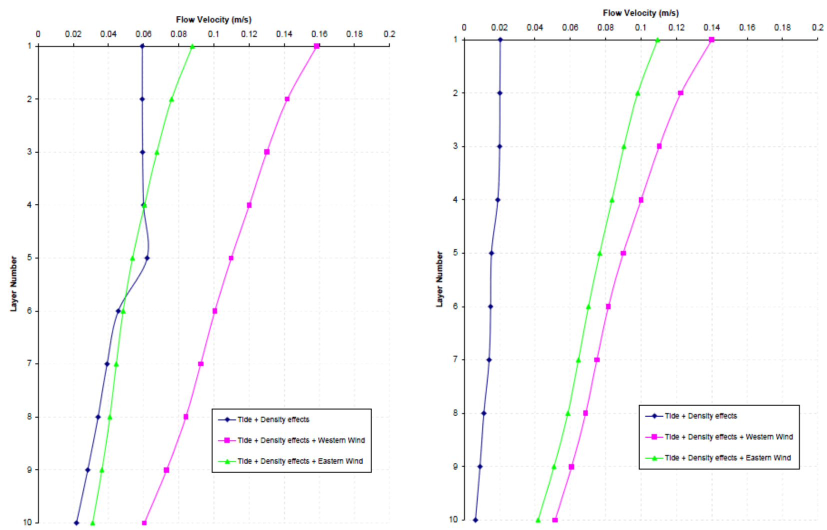
For the evaluation of the flow conditions for various situations (tidal flow only, wind induced flow and density driven flow due to differences in salinity) the Delft3D system has been applied [ALKYON DATA]. Because little data was available, several other sources (Admiralty Charts, Admiralty Tide Tables, Satellite data, Measured water levels) were used to put together to acquire the background data [ALKYON DATA].

Several 2D and 3D flow model simulations were carried out (by [ALKYON DATA]) to investigate the flow conditions at Nador. The 2D flow simulations were carried out to calibrate the western Mediterranean large scale tidal model and to determine the depth averaged flow conditions at Nador. The western Mediterranean large scale model is validated with the hindcast of the measured water levels using Delft3D.

In the 3D flow simulations besides the tidal currents, also the effects of salinity differences and strong winds have been taken into account. These results yield (somewhat) higher values for the current velocities.
The variation of the density effects-, western wind- and eastern wind- induced flow velocity profiles (during maximum flood and ebb phase of the spring tide) are shown in the figures below. The upper layer corresponds with the water surface.

Figure 3.5

Flow velocities at project location according to the western Mediterranean large scale flow model [ALKYON DATA]
Left: during max flood phase
Right: during max ebb phase



This is graphically presented in the figures on the next page, with the accompanying design situations. These occur under maximum flow velocities during flood and ebb phases, including wind and density differences that work in the direction of the flow velocities.

Figure 3.6

Western Mediterranean large scale model with maximum flow velocities during flood phase, including wind and density differences [ALKYON DATA]

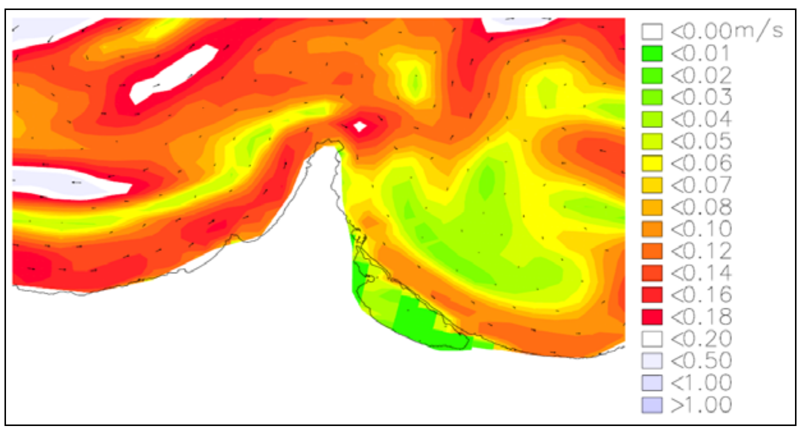
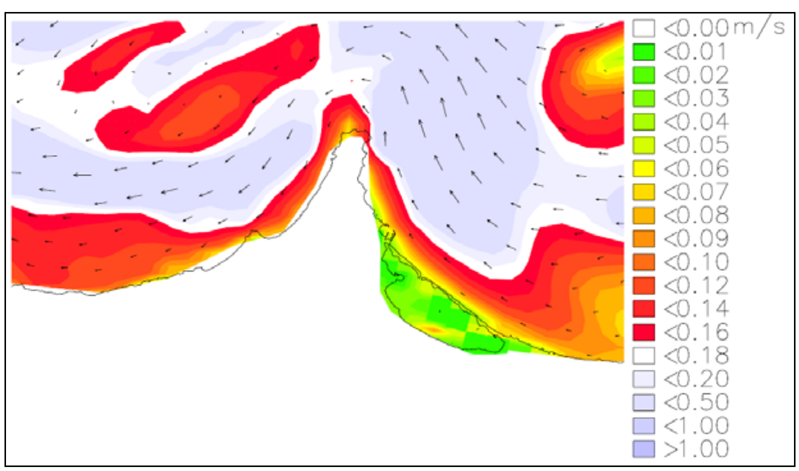


Figure 3.7

Western Mediterranean large scale model with maximum flow velocities during ebb phase, including wind and density differences [ALKYON DATA]



From the graphs and figures presented above, the results are summarized in the next table.

Table 3.6

Currents around project location in m/s

Currents	Ebb	Flood
Maximum velocity [m/s]	0.14	0.16
Direction [° from N]	225	45

Even in the worst case scenario (tidal flow + strong wind + density differences) current velocities are so small, it is assumed that they will not pose any problems for the port design. Because of this, from hereon and further currents are assumed to be negligible.

3.3.4 WAVES

The offshore wave climate is required as input for deriving the near shore normal and extreme wave conditions. The near shore normal wave conditions are required for sediment transport study, navigation and mooring studies. The extreme wave climate is required for the design of the port infrastructure, for example the breakwater (Chapter 6). Later on, the operational and limiting wave climate will be defined.

Offshore wave climate

Three data sources were compared [ALKYON DATA] to define this wave climate:

- Records of ship observations during 1960 – 1997, around the location 35.67°N, 3.57°W,
- Altimeter data from Argoss database, providing data on wave height for an area around 35.42°N, 3.19°W,
- Database of Puertos del stado, providing time series of wave heights, periods and directions for:
  - WANA hindcast model output offshore Nador (35.5°N, 2.875°W)
  - WANA hindcast model output south of Almeria (36.5°N, 2.375°W)
  - Wave measurements by offshore buoy south of Almeria (36.57°N, 2.34°W)

The ship observations data set showed very high values for the low percentages of exceedance [ALKYON DATA]. The most extreme wave heights from the ship observations can be unreliable. However, the WANA hindcast model did not reproduce the measurements from the wave buoy very accurate.

Based on the above, the offshore wave climate is based on the ships observations in terms of directional distribution, but the wave height is modified to better match the distribution for the highest and lowest wave heights from the Altimeter data from the Argoss database [ALKYON DATA]. This has lead to the following offshore wave distribution (highest of sea and swell), see the table and the figure below:

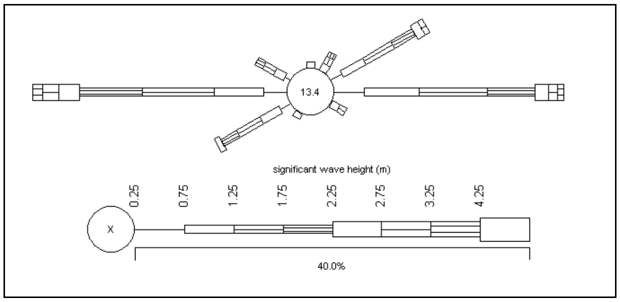
Table 3.7

Probability of exceedance (%) for the offshore wave height (highest of sea + swell) for modified ship observations offshore of Nador (35.67°N, 3.57°W) [ALKYON DATA]

Hs (m)	wave direction (Deg) -15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	10,81	3,08	12,28	24,64	2,88	,94	,81	1,82	9,61	27,22	4,69	1,21	100,00
,25	1,96	2,80	11,75	23,51	2,63	,81	,65	1,57	9,19	26,18	4,47	1,07	86,60
,75	1,48	2,35	10,49	20,50	2,14	,61	,49	1,22	8,06	23,76	3,87	,88	75,85
1,25	,79	1,56	7,89	14,89	1,30	,29	,24	,62	5,87	18,83	2,75	,58	55,63
1,75	,41	,76	4,57	7,97	,52	,09	,10	,23	3,12	11,52	1,44	,30	31,02
2,25	,19	,25	2,07	3,19	,14	,04	,02	,07	1,26	5,08	,54	,11	12,96
2,75	,10	,15	1,23	1,83	,07	,02	,02	,05	,72	2,99	,29	,05	7,51
3,25	,05	,06	,67	,90	,03	,01	,00	,02	,34	1,38	,10	,02	3,59
4,25	,01	,02	,23	,26	,01	,00	,00	,00	,09	,36	,02	,01	1,02
5,25	,00	,00	,06	,06	,00	,00	,00	,00	,02	,08	,00	,00	,23
6,25	,00	,00	,00	,01	,00	,00	,00	,00	,01	,02	,00	,00	,04

Figure 3.8

Wave rose offshore Nador (35.67°N, 3.57°W).



Following the same methodology that was applied for the extreme wind conditions, the Weibull distribution was used to derive the extreme wave heights from the offshore wave climate [ALKYON DATA] which was presented above. The extreme wave heights offshore of Nador that are expected to be exceeded for three hours during return periods of 1, 5, 10, 25, 50, 100 and 200 years are presented in the table below with their accompanying calculated peak periods [ALKYON DATA]. The peak periods are derived based on the relationship between wave height and wave period for each directional sector.

Table 3.8

Extreme wave conditions offshore of Nador [ALKYON DATA]

	0°N		30°N		60°N		270°N		300°N		330°N	
Return Period	Hs [m]	Tp [s]	Hs [m]	Tp [s]	Hs [m]	Tp [s]	Hs [m]	Tp [s]	Hs [m]	Tp [s]	Hs [m]	Tp [s]
1 year	3.41	9.3	3.85	10.9	5.4	12.3	5.72	10.6	3.82	9.6	3.02	8.7
5 years	4.34	10.3	4.61	12	6.17	13.1	6.37	11.1	4.59	10.3	3.79	9.8
10 years	4.71	10.7	4.87	12.4	6.47	13.4	6.63	11.3	4.9	10.6	4.09	10.1
25 years	5.2	11.1	5.19	12.8	6.84	13.7	6.97	11.5	5.31	10.9	4.47	10.6
50 years	5.56	11.4	5.4	13.1	7.11	14	7.23	11.7	5.61	11.1	4.75	10.9
100 years	5.91	11.7	5.61	13.4	7.37	14.2	7.47	11.9	5.91	11.3	5.01	11.2
200 years	6.25	12	5.8	13.6	7.61	14.5	7.71	12	6.2	11.5	5.27	11.4

Near shore wave climate

The wave climate in the near shore area at Nador has been obtained from the offshore wave climate using SWAN [ALKYON DATA]. This numerical model represents relevant physical phenomena such as 2D-refraction, shoaling, dissipation by bottom friction and breaking and addition of wave energy by wind. SWAN was applied to represent the wave field on a two dimensional horizontal rectangular grid [ALKYON DATA]. The depth schematisation was done according to client data.

For the directions 90°N, 105°N, 210°N, 240°N and 270°N wave generation by wind close to the site leads to double peaked spectrum, with a considerable difference between the direction of the locally generated waves and the longer waves arriving at the site from offshore. In such cases, the use of the whole wave spectrum to get wave height and average wave period and direction can be misleading, particularly when designing the layout for the breakwaters. It is because of this that two sets of computations have been carried out [ALKYON DATA]:

- The first set of the computations in which the wind was deactivated for directions 90°N, 105°N, 210°N, 240°N and 270°N within the nested grids B, C and D (for design of the breakwater layout to protect moored ships from penetrating waves).

- The second set of the computations in which wind was active for all the directions and for all the grids (for coastal studies and navigation studies).

For both sets of computations the wave conditions obtained offshore Nador were transformed to near shore locations using SWAN. This yields an output in different locations. These locations are indicated in the figures below.

Figure 3.9

Locations of several calculation points more out of the shore [ALKYON DATA]

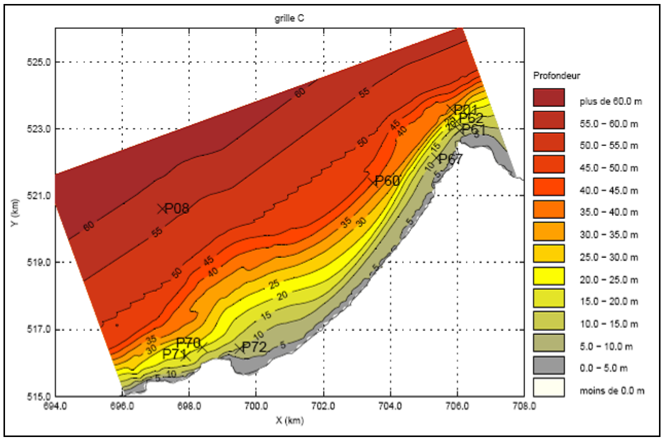
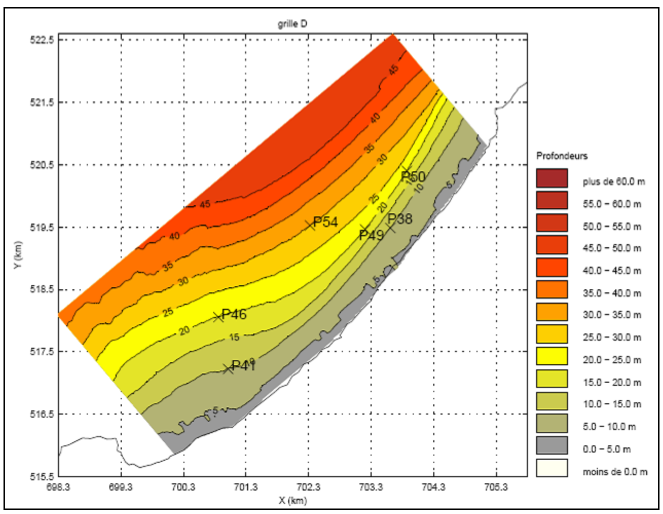


Figure 3.10

Locations of several calculation points located near shore [ALKYON DATA]



For all the available different output locations, the significant wave height H_S [m] and the spectral peak period T_P [s] are presented in annex 2.1. Here, for the port design three relevant locations along the coast have been selected in order to describe/quantify the main wave climate and it's parameters at more or less the same depth along the coast.
These are:

- P01 at a depth of CD – 33 m. north of Punta Negri
- P60 at a depth of CD – 40 m. at the northern part of the sandy beach
- P54 at a depth of CD – 30 m. at the southern part of the sandy beach.

Output point P01

Located north of Punta Negri, the influence of eastern waves is clearly visible. This is presented below in the form of wave roses for the two computational sets. This already gives a clear view of the dominant directions and the probability of occurrence. For the data tables regarding significant wave heights and peak periods, reference is made to annex 2.1.

I. For computational set 1, the wind was deactivated for directions 90°N, 105°N, 210°N, 240°N and 270°N (for design of the breakwater layout to protect moored ships from penetrating waves).

Figure 3.11

Clarification for presented wave roses of P01

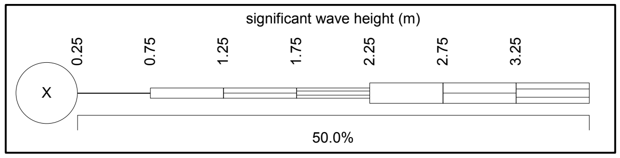
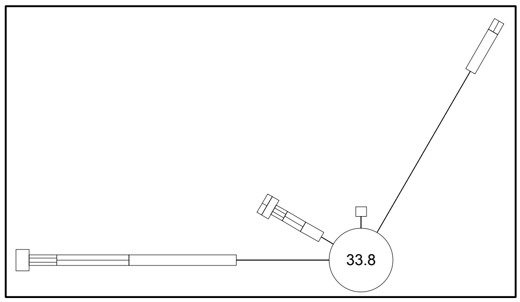


Figure 3.12

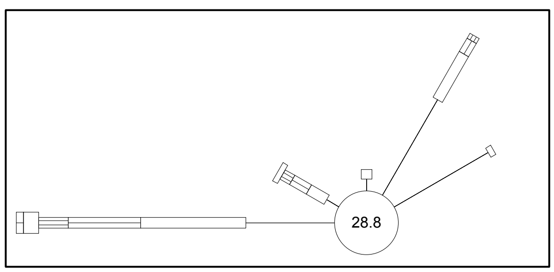
Near shore wave rose calculation point P01, set I



II. For the second set of the computations in which wind was active for all the directions and for all the grids (for coastal studies and navigation studies), see the figure below.

Figure 3.13

Near shore wave rose calculation point P01, set II



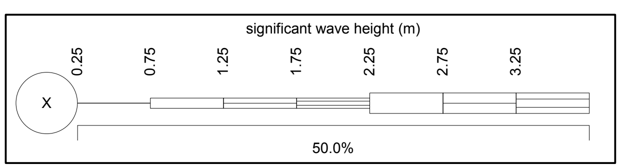
Output point P60

Located at the northern part of the sandy beach, the sheltering effect from Punta Negri is clearly visible.

I. For computational set 1, with deactivated wind for directions 90°N, 105°N, 210°N, 240°N and 270°N.

Figure 3.14

Clarification of presented wave roses for point P60, set I



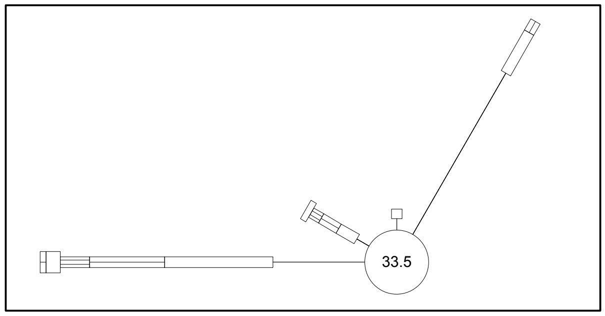


Figure 3.15
Near shore wave rose calculation point P60, set I

II. For computational set 2, in which wind was active for all the directions, see the figure below.

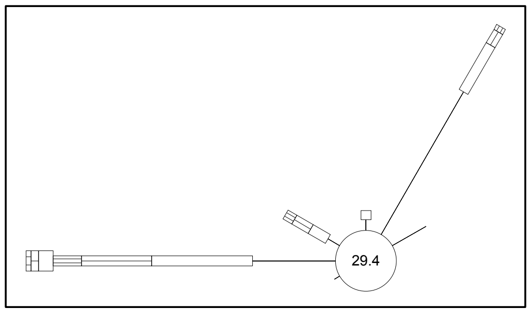


Figure 3.16
Near shore wave rose calculation point P60, set II

Output point P54

Located at the southern part of the sandy beach.

I. For computational set 1, with deactivated wind for directions 90°N, 105°N, 210°N, 240°N and 270°N.

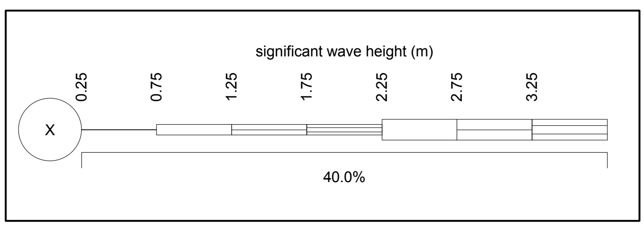


Figure 3.17
Clarification for presented wave roses of point P54

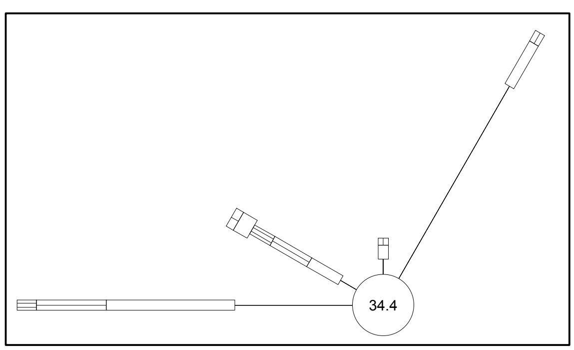
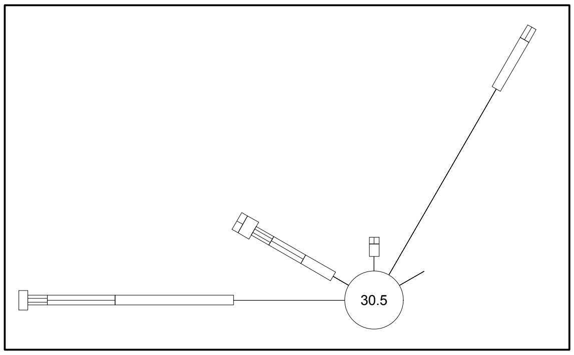


Figure 3.18
Near shore wave rose for calculation point P54, set I

II. For computational set 2, in which wind was active for all the directions, see the figures below.

Figure 3.19

Near shore wave rose for calculation point P54, set II



From the above it is evident that there are two main resulting wave directions: NNE and W-WNW. However, the waves from NNE have a smaller significant wave height and peak period than the waves arriving at the project location from W-WNW. This will be taken into consideration when designing the different port items.

The derived extreme conditions at offshore of Nador have also been transformed to near shore locations using SWAN. The extreme wave computations were carried out [ALKYON DATA] for directions 270°N, 300°N, 330°N, 0°N, 30°N and 60°N at a water level of CD + 1.0 m. (which was determined to be the extreme water level). When looking at the offshore climate and the orientation of the coast, these directions are more relevant for the design of coastal structures in Nador (e.g. the breakwater). These values are presented in the table below.

Table 3.9

Extreme wave conditions for various return periods near shore of Nador in the presented calculation points [ALKYON DATA]

Return Period	Point no.	P38	P49	P54	P01	P60	P08
	Depth(m)	9,8	19,9	30,0	33,0	40,0	56,0
1 Year	H_{mo} (m)	4,88	4,91	5,03	5,52	5,48	5,68
	$T_{m-1,0}$ (s)	9,2	8,9	8,8	9,1	9,0	9,1
	T_p(s)	11,3	10,8	10,7	11,0	11,0	11,1
	Dir (°N)	295	291	288	283	283	280
5 Year	H_{mo} (m)	5,37	5,60	5,70	6,28	6,23	6,43
	$T_{m-1,0}$ (s)	9,8	9,4	9,3	9,5	9,4	9,5
	T_p(s)	12,0	11,4	11,3	11,6	11,5	11,6
	Dir (°N)	296	292	289	284	283	280
10 Year	H_{mo} (m)	5,53	5,86	5,96	6,57	6,52	6,72
	$T_{m-1,0}$ (s)	10,1	9,6	9,4	9,7	9,6	9,7
	T_p(s)	12,3	11,7	11,5	11,8	11,7	11,8
	Dir (°N)	296	293	290	285	284	281
25 Year	H_{mo} (m)	5,70	6,20	6,29	6,95	6,88	7,08
	$T_{m-1,0}$ (s)	10,3	9,8	9,6	9,9	9,8	9,8
	T_p(s)	12,6	11,9	11,7	12,0	11,9	12,0
	Dir (°N)	297	293	290	285	284	281
50 Year	H_{mo} (m)	5,82	6,48	6,56	7,25	7,18	7,37
	$T_{m-1,0}$ (s)	10,6	10,0	9,8	10,0	10,0	10,0
	T_p(s)	12,9	12,2	12,0	12,2	12,2	12,2
	Dir (°N)	297	294	291	286	284	281
100 Year	H_{mo} (m)	5,93	6,73	6,81	7,53	7,45	7,64
	$T_{m-1,0}$ (s)	10,8	10,2	10,0	10,2	10,1	10,2
	T_p(s)	13,2	12,4	12,2	12,5	12,4	12,4
	Dir (°N)	298	294	291	286	284	281
200 Year	H_{mo} (m)	6,02	6,98	7,25	7,80	7,71	7,91
	$T_{m-1,0}$ (s)	11,0	10,3	10,1	10,3	10,3	10,3
	T_p(s)	13,4	12,5	12,3	12,6	12,5	12,5
	Dir (°N)	298	294	350	286	285	359

Seiches

Regarding seiches, no data is available at the project location [ALKYON DATA]. However, their presence and influence will be taken into account within a resonance assessment in the in-port wave penetration study (chapter 5). This is because of the possibility that natural resonant modes for the port (its basins) could fall within the same range as seiches with periods in the order of 10 minutes – 2 hours [HOLTHUIJSEN, 2007].

Excessive variations in water levels could cause large hindrance to moored vessels, which inevitably necessitates an assessment regarding seiches.

Tsunamis

These long-period gravity waves are caused by underwater earthquakes, volcanoes or landslides and they can travel vast distances across deep oceans with a height less than 1 m but a wavelength extending to 200 km. On reaching the coastline, however, the wave height may be significantly increased by shoaling, diffraction, convergence and resonance so the effect of a single event will vary at different locations [WIKIPEDIA], [WEBSITE TSUNAMI INSTITUTE], [HOLTHUIJSEN, 2007].

From the tectonic perspective, the Mediterranean Sea sets the boundary between the Eurasian and the African plate. This means that tsunamis can also occur in the Mediterranean waters, due to earthquakes caused by the African plate drifting northwards underneath the Eurasian plate, which will be described in more detail in paragraph 3.4.2. [WEBSITE TSUNAMI INSTITUTE] states that 10% of all tsunamis worldwide occur in the Mediterranean. However, on average only one disastrous tsunami takes place in the Mediterranean region every century.

The locations of occurrence of these events are indicated in the figure below.
The dots show epicentres of the earthquakes that caused tsunamis.

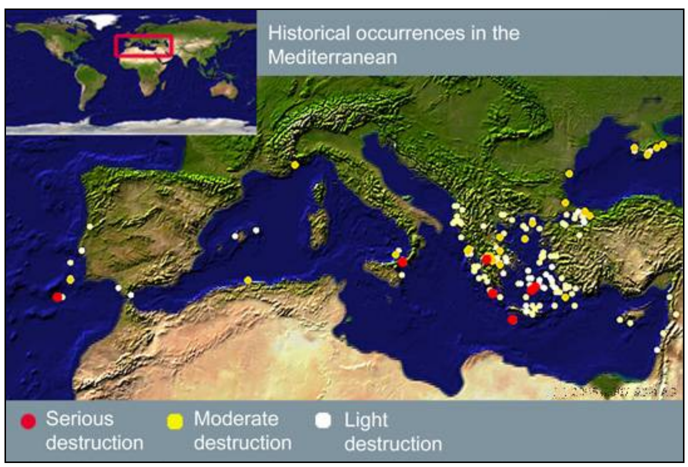


Figure 3.20

Epicentres of earthquakes that caused tsunamis in the Mediterranean Sea [WEBSITE TSUNAMI INSTITUTE]

From the above figure is evident that especially Greece, Turkey and southern Italy are mostly affected. For example, in Italy in the last four centuries 10-15 tsunamis have been recorded every 100 years [WIKIPEDIA], [WEBSITE SCIENZA GIOVANE MEDITERRANEAN].
One of the last noteworthy tsunamis took place in 2002: the tsunami of Stromboli had reached a wave height of 5-10 m. Also in 1999, the region of Izmit was hit by a wave with a height of 2 m. on average. In historic events, even wave heights up to 20-25 m. have been recorded at eastern Mediterranean locations.

Around the project location at Nador, no epicentres leading to serious tsunamis have been registered, see table A2.15 [WEBSITE TSUNAMI INSTITUTE]. This does not necessarily mean that no tsunamis occur there, as they can travel large distances at high speeds.

Nevertheless, because of the distance of the project location in relation to the chance of occurrence of epicentres that cause tsunamis in the region, this low probability qualifies as a rare event.

Tsunamis still are an important consideration for risk analysis related to the design of the port and the construction of terminals within the low-lying coastal areas at the project location. Nevertheless, it is because of the explanation above that only events that occur more frequently are to be included in order to accomplish a feasible economic design of a new port at Nador.

3.4 DETAILED SITE DESCRIPTION

By taking into account these present hydrodynamic forces as outlined above, a more thorough, detailed description of the project site will be made.

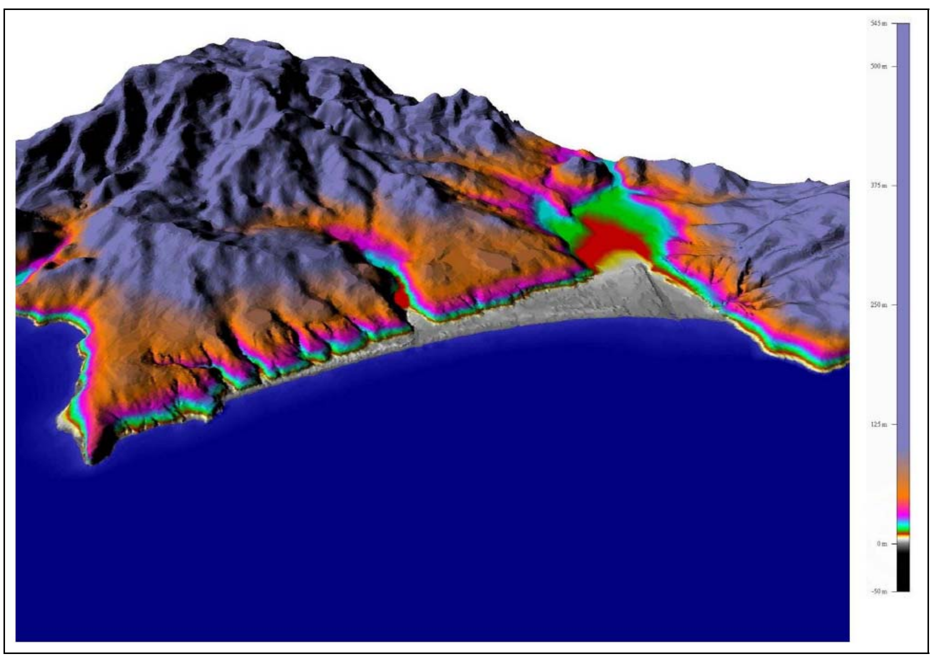
This influence of hydrodynamic forces will already become clear in the present day topography and bathymetry in paragraph 3.4.1. The terrain itself will be described in more detail in paragraph 3.4.2 where the geology and seismology will be treated. Subsequently for the assessment of hydrology and morphology the hydrodynamic forces play an important role and have its specific influence on the project location.

3.4.1 TOPOGRAPHY & BATHYMETRY

The figures in annex 1 already gave a global indication of the present topographic relief at the project location. A more visual aiding 3D overview of the topographic relief of the terrain at the project location is presented in the figure below [ALKYON DATA].

Figure 3.21

3D topographic overview of project location [ALKYON DATA]



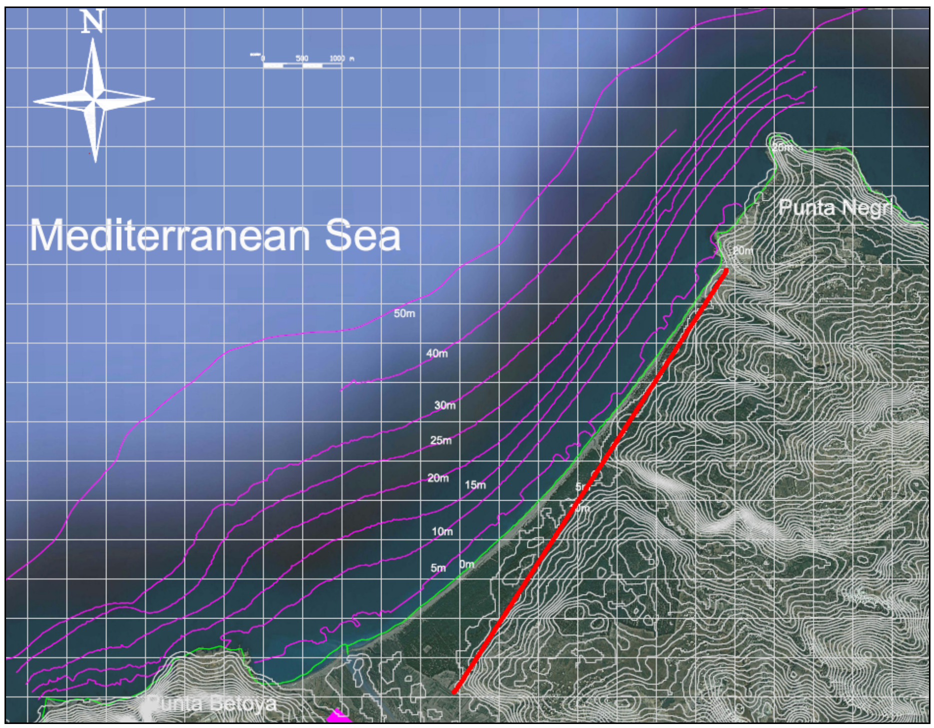
In this figure, the sandy beach is clearly visible in grey, between Punta Negri to the left and the Rio Kert and Punta Betoya to the right.

The curved beach profile is the result of wave action from waves arriving at the project location from the earlier determined dominant directions (270-300°N). The height of the beach is around +5 m CD. The yellow altitude line represents terrain heights above +10 m. CD. The subsequent altitude lines are located close to each other, which represents a steep slope land inwards. The pink colour already represents terrain heights of around +35 m. CD and orange altitudes represent terrain heights of +50 m. CD and higher.

The three wadi inlets cutting through the surrounding terrain are clearly visible in the figure above. At the location of the headlands, steep cliffs rise from the water line. The figure above also shows a steep sloping terrain inland of the sandy beach, which poses an additional challenge to the port design. A practical boundary for construction of the port (its facilities) has been indicated in the bathymetric map below (red line at CD +10 - +15 m.), which shows the bathymetry made available by the client [ALKYON DATA].

Figure 3.22

Bathymetric map provided by client, with indicated the practical inland boundary limit of the project site.



The contraction of bathymetric lines around Punta Negri indicates that the original sea bottom also has a steep slope. Because of this steep slope, also on the seaward side practical constructional limitations are introduced (e.g. regarding the breakwater construction depth and accompanying costs). Near Punta Betoya, the bathymetric lines lie further apart: here, the sea bottom slope is much less steep. This is due to the Rio Kert’s sediment discharge, elaborated in more detail in 3.4.3.

3.4.2 GEOLOGY & SEISMOLOGY

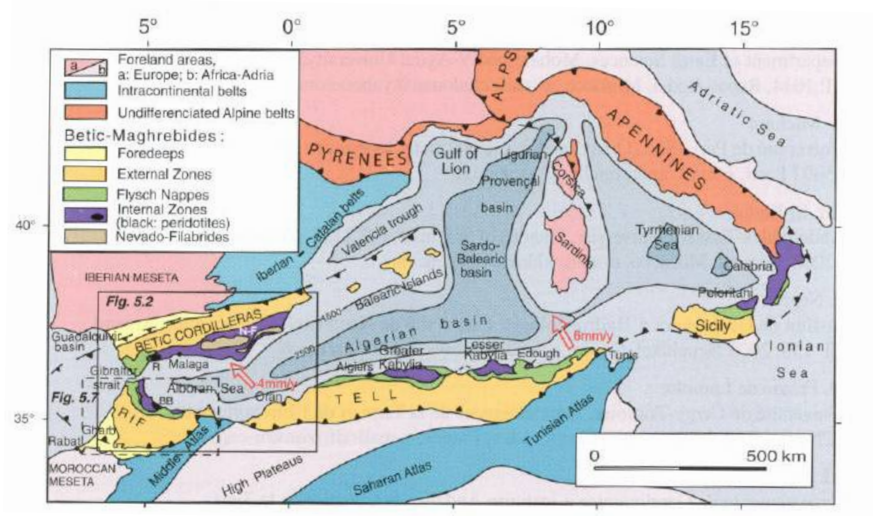
Overall geology and seismology

The project site is situated in the Rif Belt, which is part of the larger Mediterranean Alpine Belts.

The Rif Belt forms on the one hand westernmost part of the Maghrebide belt, which extends along the North African coast and continues eastward to Sicily and Calabria in southern Italy [ALKYON DATA], [GOOGLE MAPS]. On the other hand it forms the southern limb of the Gibraltar Arc, the northern limb of which correspondents to the Betic Cordilleras.

Figure 3.23

The Rif Belt [yellow], in which the project location is situated [ALKYON DATA]

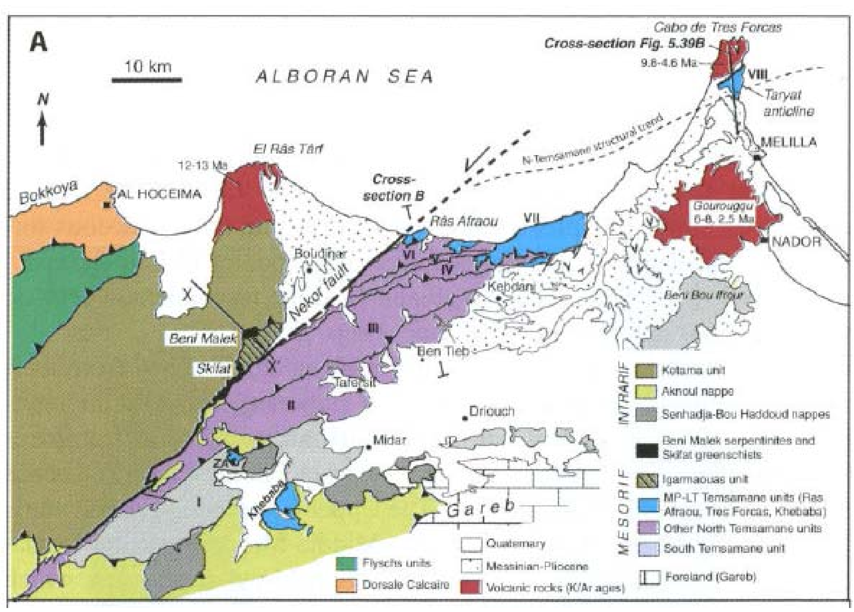


About 25 km. to the west of the project location, a major tectonic fault is present: the Nekor Fault. At the project location itself, a smaller fault is present, the Taliwine. This is between the location of Punta Betoya to the southwest and the mouth of the Rio Kert. The exact location offshore and the activity are unknown [ALKYON DATA]. It is assumed that this fault runs from north to south, extending the line northwards in the figure below.

Based on the PIANC report "Seismic Design Guidelines for Port Structures" (2001), [ALKYON DATA] states that the Peak Ground Acceleration (PGA) corresponding to a return period of 475 years for northern Morocco is between 0.05-0.15 g. It is advised to use the larger value in the preliminary design.

Figure 3.24

More detailed structural map of the Rift Belt area. The project location is situated to the right of VII. [ALKYON DATA]



The fact that there is a fault zone present around Punta Betoya means that there is a chance of earthquakes in the area. The exact area around Punta Betoya is therefore avoided on purpose as much as possible: to minimize the influence (e.g. earth movements) of it on the port. However, within the design additional constructional measures must be provided in order to follow possible uneven settlements of the construction. This is especially relevant for the liquid bulk terminal which should have as less hindrance as possible from any unwanted oscillations because of the hazardous nature of the products handled.

Geotechnical data

Not much is known about the exact geotechnical data in the area [ALKYON DATA].
A global description is presented below, which gives an indication of the soil types.
Between Punta Negri and Punta Betoya, the following types of rocks and sediments are present:

Andesite

Punta Negri consists totally of andesite, outcropping locally at the sea bottom at depths of 30-40 m. From the coastline, the rock inclines towards the northwest and is present in almost the whole bay enclosing Punta Negri. Above a height of CD +4 m. the rock consists of marl/limestone and calcified sand.

Schist

Punta Betoya consists out of a metamorphic rock called Schist, which is easily broken in contrast to andisite. Some rock outcrops are present at the sea bottom.

Beach and dune sand

Along the coastline dune and beach sands are present. The thickness of this layer differs strongly from 7-25 m. The material deposited by the wadis consists of soft compressible soils.

Wadi formation

This layer is present at the location where the wadis discharge into the Mediterranean Sea. It is a conglomerate of fine (clay, clayey silt and sand) and coarse material (gravel, pebbles). The sandy material showed SPT-values between 25-75, and the loamy materials SPT-values of 20.

Marl

From a depth of 10-20 m. marl is present. The SPT –values at some locations give results of 50-100.

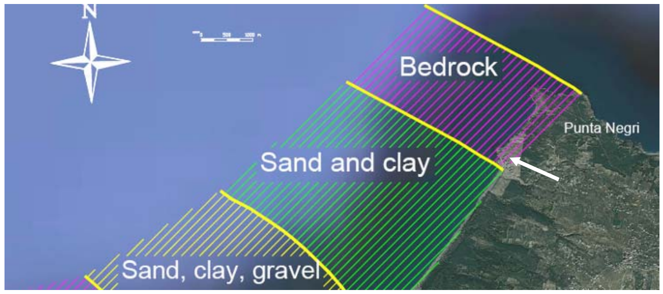
Sandstone

Cemented sand is present between the marl and the beach with its dunes. This material shows higher SPT-values than the average beach and dune sand.

The above described geotechnical data can be summarized in a figure presented below, where three different zones presenting the bed material can be distinguished.

Figure 3.25

Present bed material
Purple: bedrock present around Punta Negri and Punta Betoya
Green: mainly sand (and some clay) present
Yellow: sand and some clay and gravel present around the outflow of the Rio Kert



In zones I (purple), bedrock is present. This the case at Punta Negri and Punta Betoya. Especially the bedrock at Punta Negri (andesite) is very hard and difficult to dredge (see white arrow). Close to the coastline, this rock is present at depths less than CD -20 m.

In zone II (yellow), the influence of the wadis is clearly present. The sediments consist of clayey loam to sand with gravel, of which some parts of the sediment could be cemented. The material can be dredged.

In zone II (green), mainly beach and dune sands are present. The sediments consist of fine grained material and are even easier to dredge than in zone II. The thickness of these sediments is between 5-20 m.

3.4.3 HYDROLOGY & MORPHOLOGY

As outlined before, at the project location three main wadis can be identified: the largest (Rio Kert) to the south around 1 km. northeast of Punta Betoya, a smaller one at the middle of the sandy beach, and an ever smaller one around the tourist village at Punta Negri. The locations of the wadis are indicated in the figures presented earlier (e.g. figure 3.1). At first glance, they appear to be the main input source of sediments into the coastal system.

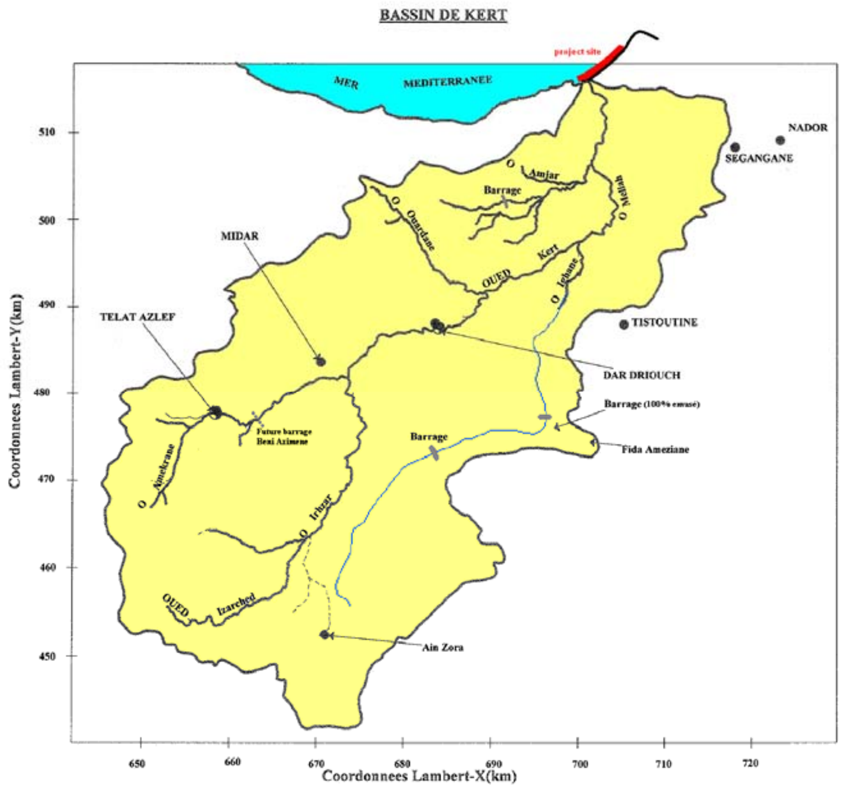
Hydrology

The Rio Kert

The Rio Kert is the largest wadi with the largest catchment area. The Kert basin is characterized by a semi-arid Mediterranean climate with very irregular precipitation throughout the year (and even between years) [ALKYON DATA]. The springs and summers are dry, and the precipitation is concentrated in the autumn and winter. Large part of the year, the wadi is completely dry. The boundaries and characteristics of the Kert basin are outlined below.

Figure 3.26

Catchment area of the Rio Kert [ALKYON DATA]



The sediment load was not known beforehand. To make a calculated estimate, this has been calculated according the morphological modelling formulae [DE VRIEND *et al.*, 2010]: $s=m*u^n$ and $S=B*s$. For the calculation, reference is made to annex 3.1. Below, the total Kert basin characteristics are presented.

Rio Kert basin characteristics [ALKYON DATA]:

- Catchment area: 2700 km^2
- Rainfall: 659.106*10^3 m^3/year (=240 mm/year)
- Runoff: 42.106*10^3 m^3/year (=6%)
- Sediment load: 492.000 m^3/year, calculated in annex 3.1
- Average discharge: 1,5 m^3/s
- Maximum discharge: 3000 m^3/s

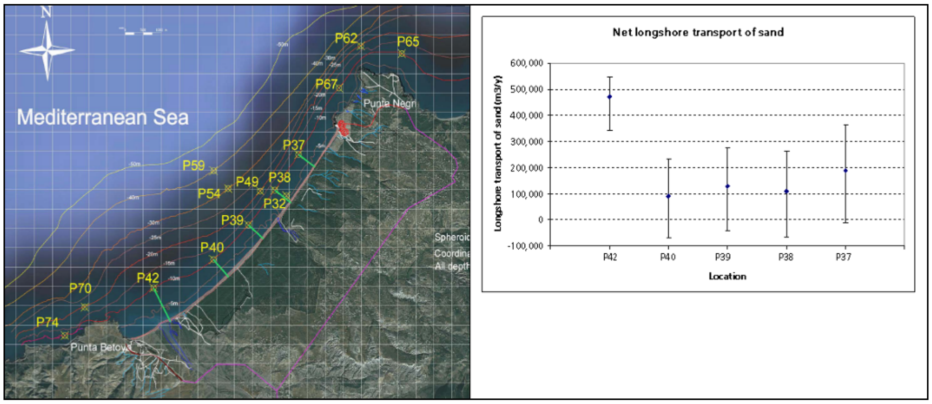
The yearly averaged <u>longshore</u> sediment transport just to the northeast of the Rio Kert (P42) is equal to 450.000 m^3/y, according to [ALKYON DATA] presented in the figure on the next page. It can be assumed that this value is more or less consistent with the earlier calculated sediment load from the Rio Kert, analogous to the following reasoning.

The rocky headlands do (almost) not erode, and do not add to the sediment balance. Besides this, the quantity of sand bypassing the headlands is negligible [ALKYON DATA]. This means that the only sediment sources in the system are the wadis, which discharge water and sediments during times of rainfall. This sediment is redistributed along the sandy beach by oblique incoming waves. The largest part of this sediment is due to wave action transported (north)east of the Rio Kert, because of the dominant wave direction from the west.

This means that the yearly averaged longshore sediment transport just northeast of the Rio Kert consists only of sediment deposits from the wadi. But not all of the sediment discharged by the Rio Kert is transported in longshore direction: there will also be some offshore losses. This is consistent with the calculations above, as the yearly averaged sediment load from the Rio Kert (S=492.000 m^3/year) is larger than the longshore transport (S=450.000 m^3). From this it is assumed that these (preliminary estimate) values are more or less consistent.

Figure 3.27

Location of calculation points along the coast where the longshore sediment transport has been determined [ALKYON DATA]



It is assumed that the Rio Kert should always be able to discharge into the Mediterranean Sea because of its large catchment area and thus discharge and sediment load. This could be realised for example with the help of an outlet stabilisation. It is deduced from this that the location around the Rio Kert outflow is not suitable for the construction of the new port.

Second (middle) wadi

The second wadi drains a much smaller catchment area than the Rio Kert, and mouths in the sandy beach at the middle of the project location. No data of this wadi is available, although the dimensions of the lower channel indicate that a flood discharge of multiple hundreds of m^3/s should be reckoned with.

A diversion of the wadi around the port site seems impossible (or very uneconomic) at this location, given the size of the new port. So this wadi will always discharge in (or around) the new port. Proper measures need to be taken here, in order to avoid sedimentation in the port because of the wadis (sediment) discharge.

In order to determine the characteristics of this wadi, several assumptions have been made. First of all, the catchment area has (roughly) been determined from [GOOGLE MAPS]. From this, the catchment area factor $\alpha=A_1/A_2$ (with A_2 is the catchment area of the Rio Kert basin) has been used with preservation of the discharge wave shape $\sqrt{(\alpha)}$ [JANSEN, 1994] to get at the maximum discharge, taking into account that [ALKYON DATA] estimated that this would be several hundreds of m^3/s. This has lead to the following characteristics:

- Catchment area (A_2): 45 km^2 [GOOGLE MAPS]
- Rainfall: 10.800*10^3 m^3/year (=240 mm/year)
- Runoff: 648*10^3 m^3/year (=6%)
- Sediment load: 38.100 m^3/year, calculated in annex 3.1.
- Maximum discharge: 385 m^3/s

Also here, a longshore transport is calculated by [ALKYON DATA] at P38: S_x=150.000 m^3/year. This amount (which is larger than the sediment load from only the middle wadi) includes the earlier picked up sediment transport to the southwest of the middle wadi (which includes a part of the longshore sediment transport originating from the Rio Kert).
The coastline is situated more perpendicular to the dominant wave directions, which results in a smaller longshore sediment transport, from which it is assumed that the values are again (more or less) consistent.

As mentioned before, allowing this wadi to discharge into the port seems inevitable.
The proposed solution to prevent sedimentation into the port is the construction of a deep stilling basin which functions as a sediment trap, and a fixed weir that prevents the sediment from flowing over the bed into the mooring basin.

Third (smallest) wadi
The smallest wadi is located south of Punta Negri, and drains an ever smaller catchment area that the second wadi. Also for this wadi, no data is available [ALKYON DATA]. However, because of its size and position, it is not expected that severe measures are required here. The methodology from above has been followed, to get a global indication of the characteristics.

- Catchment area: 3.5 km^2 [GOOGLE MAPS]
- Rainfall: 840*10^3 m^3/year (=240 mm/year)
- Runoff: 50*10^3 m^3/year (=6%)
- Sediment load: 11.300 m^3/year, calculated in annex 3.1
- Average discharge: 0,002 m^3/s
- Maximum discharge: 105 m^3/s

It is concluded that the main sediment input source is the Rio Kert. The other sediment discharging wadis do add to the total longshore transport, but much less than the Rio Kert. The sediment is transported along the coast (longshore) to the northeast. Because of the sediment input of the middle wadi and the third wadi, the average longshore sediment transport (150.000 m^3/year [ALKYON DATA]) increases somewhat to the northeast. It seems possible to let this third wadi discharge into the port of which (because of the small discharge) no severe problems are expected.

Morphological description
Considering morphological aspects is essential in order to be able to quantify the possible impact of the new port on the coastline and to give advice on the most beneficial position of the port entrance and breakwaters with respect to the sedimentation rates [LIGTERINGEN, 2007].

As was quickly described above, it is expected that the quantities sand bypassing the headlands is negligible, in accordance with [ALKYON DATA]. Furthermore, considering the fact that the headlands consist of rock, it is expected that the sediment input from these headlands is nil. This concludes that the only sediment sources in the system are the wadis at the project site as mentioned before. The sediment discharged by these wadis in times of rainfall is redistributed along the coast by wave action.

The yearly net sediment transport is directed to the northeast along the coast [ALKYON DATA], which has been visualized in the figure below.

Figure 3.28
Direction of longshore sediment transport [GOOGLE MAPS]

However, the instantaneous sediment transport depends on the environmental conditions occurring at that time: for example, waves from 0-30°N lead to an instantaneous longshore sediment transport in the opposite direction (to the southwest).

The direction of origin of wind and waves varies in time, which means that an instantaneous longshore sediment transport in both directions can occur (see directional spreading in 3.3.1 and 3.3.4). In a first (safe) approximation it can be assumed that these quantities equal the same (maximum) quantities as calculated above for the wadis sediment load, only directed to the southwest. The actual real momentary sediment transport will however always be smaller because of offshore losses.

It is expected that the morphological situation at the project site has reached more or less equilibrium with the hydrodynamics. The curved shape of the beach is the result of incoming waves and waves diffracted by the headlands. Only in periods of heavy rainfall (and subsequently high discharges from the wadis) there is considerable deposition of sediment in the coastal zone [ALKYON DATA]. This (temporarily) changes the coastline locally, until a new equilibrium is reached.

The fact that on average the resulting net longshore transport is directed northeast along the coast gives an important direction for the breakwater layout. The breakwater layout should be oriented in such a way that transport and deposition of sediments into the port is minimized, which means above all blocking the net yearly longshore sediment transport from the southwest (and the instantaneous sediment transport from the northeast).
This already necessitates the construction of two breakwaters.

3.5 SHIP CHARACTERISTICS

From the cargo forecast (see previous chapter), the amount of shipping traffic can be calculated which would be needed to arrive at the required throughput. The fact that various terminals for different commodities will be included in the new port design means that every terminal with its cargo has its own unique specifications. This also holds for the ships which transport the accompanying cargo and moor at the terminals. This results in different design vessels for the various terminals, each with their own specific characteristics. Also, the ship hydrodynamics and their manoeuvrability will be assessed.

3.5.1 DETERMINING THE (DESIGN) VESSEL DIMENSIONS

It is first of all essential to draw up a plan of expected design vessels in order to quantify the shipping traffic and cargo capacity. For determination of the design ships, it is of importance to look at the constraints of the cargo flow. As mentioned before, the global transport axis from Eastern America to Western Europe and via the Mediterranean Sea to Singapore is restricted by the Suez Canal, which connects the Mediterranean Sea to the Red Sea.
This puts constraints on the maximum vessels that can sail on these main lines, although this is also subject to change (for additional information see annex 5.2).

Because of the fact that there was limited information available about the cargo distribution, -capacity and types of the vessels, several assumptions had to be made regarding their characteristics and dimensions. For the container vessels, the following characteristics were made available [ALKYON DATA]:

Table 3.10

Container vessel cargo characteristics

% of throughput	ship class	capacity	call size
25%	Feeder	1.500 TEU	70%
42%	Panamax	3.000 TEU	50%
25%	Post-panamax	6.000 TEU	40%
8%	ULCV	14.000 TEU	30%

This division of vessel sizes is not available for liquid and dry bulk vessels, so assumptions will have to be made here as well. It is expected that oil products arrive in vessels in the range of 40.000-80.000 DWT, and leave the port in somewhat smaller product tankers of 20.000-60.000 DWT. Diesel, one of the oil products, will arrive in larger vessels up to 150.000 DWT.

In phase II, there will also be refining of incoming crude oil. This crude oil will arrive in crude oil tankers ranging from 100.000-200.000 DWT. Outgoing oil products will again be transported in the earlier mention product tankers.

The actual dimensions of the vessels have been determined in accordance with an analysis of the present day shipping market [PVE MAASVLAKTE II], [WEBSITE PORT OF ROTTERDAM], [PIANC, 2002]. The fact that Morocco can be seen more or less as a developing country which could be associated with smaller vessel sizes visiting the port is assumed to be cancelled out by the future growth of vessel sizes. Because of this, and the fact that the port will be constructed on a short time horizon, it is assumed that the present day shipping market can be considered representative.

The vessels specifications are summarized in the table below. The largest dimensions (which are often used for the design situations) are marked bold. These mentioned vessels are considered representative for the situation in Nador, and will be used for the design further on.

Table 3.11

Vessel specifications

	capacity	displacement [t]	LOA [m]	Lpp [m]	B [m]	D [m]
Container terminal	1.500 TEU	44.000	225	212	30	11,5
	3.000 TEU	67.000	294	278	32	12,5
	6.000 TEU	106.000	310	294	43	14
	14.000 TEU	**193.000**	**398**	**378**	**56**	**15,5**
Liquid bulk terminal	20.000 DWT	27.700	158	151	26	9,6
	40.000 DWT	53.600	196	189	31	11,8
	60.000 DWT	79.000	223	216	35	13,3
	80.000 DWT	104.000	245	236	40	14,4
	100.000 DWT	129.000	263	254	43	15,4
	150.000 DWT	190.000	298	290	48	17,4
	200.000 DWT	**250.000**	**327**	**318**	**53**	**18,9**
Dry bulk terminal	20.000 DWT	25.700	161	152	24	9,4
	40.000 DWT	49.400	195	186	30	11,5
	60.000 DWT	72.550	220	211	32	13,1
	80.000 DWT	95.000	238	229	35	14,2
	150.000 DWT	173.000	287	278	45	17,1

3.5.2 CARGO-VESSEL DISTRIBUTIONS AND SHIPPING TRAFFIC

Now that the dimension of the different types of vessels and their cargo capacities has been determined, the vessel-arrival distribution and the cargo-vessel distribution need to be defined.

Vessel-arrival distribution

From the foregoing, the various types of vessels with their cargo capacity are clear.
Their frequency of arrival is up until now still undetermined. These percentages of arrival have been derived from an analysis of the present-day shipping market of vessels visiting the port of Rotterdam [PVE MAASVLAKTE II], [WEBSITE PORT OF ROTTERDAM].Together with the throughput specifications they are presented in the left hand side of table 3.12.

Cargo-vessel distribution

In 3.5.1 it was emphasized that for container vessels this distribution was already given [ALKYON DATA], but this was not the case for the bulk vessels. The most important assumption is that container vessels unload their cargo, and also subsequently are loaded with (the same amount of) cargo, with quantities mentioned earlier in table 3.10. This is in contrast to the liquid and dry bulk vessels, which arrive filled up to their maximum capacity and leave the port empty. For this reason, the call size of dry and liquid bulk vessels amounts to 100%. For outgoing dry and liquid bulk alternate ships are needed, which is the reason why they are calculated separately.

As a result from the above, the total amount of shipping traffic can be calculated. The vessel-arrival- and cargo-vessel distributions can be translated into a number of ships that will arrive at and depart from the port. For this, the throughput is divided by the ships with their specific probability of arriving, taking also into account the different average call sizes of the vessels. This leads to a number of ships per year required to transport the throughput, which on its turn can be converted to the number of ships per day.

For the container terminal applies: the incoming and outgoing containers are transported in 1 ship (first unloading, than loading). So the average call size should be multiplied by 2 (as well as the service time). This also means that only half the number of ships is required to transport the throughput.

This has resulted in the table presented below: left the vessel-arrival distributions and the cargo-vessel distributions, and at the right the total amount of shipping traffic per terminal.

Table 3.12

Vessel-arrival and cargo-vessel distributions (left) &
Total shipping traffic (right)

Terminal	Phase I					Phase II				
Container terminal	2-3 MTEU/year	incoming				7.5-15 MTEU/year	incoming			
	2-3 MTEU/year	outgoing				7.5-15 MTEU/year	outgoing			
	4-6 MTEU/year	**total throughput**				**15-30 MTEU/year**	**total throughput**			
	Incoming + Outgoing					**Incoming + Outgoing**				
	% throughput	**ship class**	**capacity**		**call size**	**% throughput**	**ship class**	**capacity**		**call size**
	25%	feeder	1.500	TEU	70%	25%	feeder	1.500	TEU	70%
	42%	panamax	3.000	TEU	50%	42%	panamax	3.000	TEU	50%
	25%	post-panamax	6.000	TEU	40%	25%	post-panamax	6.000	TEU	40%
	8%	ULCV	14.000	TEU	30%	8%	ULCV	14.000	TEU	30%
	average call size		**1.828,50**	**TEU**		**average call size**		**1.828,50**	**TEU**	
Liquid bulk terminal	7.5-10 MT/year	incoming	oil products			7.5 MT/year	incoming	oil products		
	7.5-10 MT/year	outgoing	oil products			7.5 MT/year	outgoing	oil products		
	15-20 MT/year	**total throughput**				12.5 MT/year	incoming	crude oil		
						12.5 MT/year	outgoing	oil products	from refinery	
						40 MT/year	**total throughput**			
	Incoming oil products					**Incoming oil products**				
	% incoming		**capacity**		**call size**	**% incoming**		**capacity**		**call size**
	28%		40.000	DWT	100%	28%		40.000	DWT	100%
	44%		60.000	DWT	100%	44%		60.000	DWT	100%
	20%		80.000	DWT	100%	20%		80.000	DWT	100%
	8%		150.000	DWT	100%	8%		150.000	DWT	100%
	average call size		**65.600**	**DWT**		**average call size**		**65.600**	**DWT**	
						Incoming crude oil				
						% incoming		**capacity**		**call size**
						30%		100.000	DWT	100%
						45%		150.000	DWT	100%
						25%		200.000	DWT	100%
						average call size		**147.500**	**DWT**	
	Outgoing					**Outgoing**				
	% outgoing		**capacity**		**call size**	**% ougoing**		**capacity**		**call size**
	30%		20.000	DWT	100%	30%		20.000	DWT	100%
	40%		40.000	DWT	100%	40%		40.000	DWT	100%
	30%		60.000	DWT	100%	30%		60.000	DWT	100%
	average call size		**40.000**	**DWT**		**average call size**		**40.000**	**DWT**	
Dry bulk terminal	2-2.5 MT/year	incoming				2-2.5 MT/year	incoming			
	2-2.5 MT/year	outgoing				2-2.5 MT/year	outgoing			
	4-5 MT/year	**total throughput**				**4-5 MT/year**	**total throughput**			
	Incoming					**Incoming**				
	% throughput		**capacity**		**call size**	**% throughput**		**capacity**		**call size**
	40%		40.000	DWT	100%	40%		40.000	DWT	100%
	30%		60.000	DWT	100%	30%		60.000	DWT	100%
	22%		80.000	DWT	100%	22%		80.000	DWT	100%
	8%		150.000	DWT	100%	8%		150.000	DWT	100%
	average call size		**63.600**	**DWT**		**average call size**		**63.600**	**DWT**	
	Outgoing					**Outgoing**				
	% outgoing		**capacity**		**call size**	**% outgoing**		**capacity**		**call size**
	30%		20.000	DWT	100%	30%		20.000	DWT	100%
	35%		40.000	DWT	100%	35%		40.000	DWT	100%
	35%		60.000	DWT	100%	35%		60.000	DWT	100%
	average call size		**41.000**	**DWT**		**average call size**		**41.000**	**DWT**	

Phase I -	Phase I +	Phase II -	Phase II +
4.000.000 TEU	**6.000.000 TEU**	**15.000.000 TEU**	**30.000.000 TEU**
in+out in 1 ship = average callsize x2: **3.657 TEU**	in+out in 1 ship = average callsize x2: **3.657 TEU**	in+out in 1 ship = average callsize x2: **3.657 TEU**	in+out in 1 ship = average callsize x2: **3.657 TEU**
274 ships	410 ships	1.026 ships	2.051 ships
459 ships	689 ships	1.723 ships	3.446 ships
274 ships	410 ships	1.026 ships	2.051 ships
88 ships	131 ships	328 ships	656 ships
1.094 ships/y	**1.641 ships/y**	**4.102 ships/y**	**8.204 ships/y**
4 ships/d	**5 ships/d**	**12 ships/d**	**23 ships/d**
7.500.000 tons	**10.000.000 tons**	**7.500.000 tons**	**7.500.000 tons**
7.500.000 tons	**10.000.000 tons**	**12.500.000 tons**	**12.500.000 tons**
Incoming oil products	**Incoming oil products**	**Incoming oil products**	**Incoming oil products**
32 ships	43 ships	53 ships	53 ships
51 ships	67 ships	55 ships	55 ships
23 ships	31 ships	19 ships	19 ships
9 ships	12 ships	4 ships	4 ships
total **115 ships/y**	**total** **153 ships/y**	**total** **115 ships/y**	**total** **115 ships/y**
		Incoming crude oil	**Incoming crude oil**
		26 ships	26 ships
		38 ships	38 ships
		21 ships	21 ships
		total **85 ships/y**	**total** **85 ships/y**
Outgoing	**Outgoing**	**Outgoing**	**Outgoing**
56 ships	75 ships	150 ships	150 ships
75 ships	100 ships	200 ships	200 ships
56 ships	75 ships	150 ships	150 ships
total **188 ships/y**	**total** **250 ships/y**	**total** **500 ships/y**	**total** **500 ships/y**
303 ships/y	**403 ships/y**	**700 ships/y**	**700 ships/y**
1 ship/d	**2 ships/d**	**2 ships/d**	**2 ships/d**
2.000.000 tons	**2.500.000 tons**	**2.000.000 tons**	**2.500.000 tons**
2.000.000 tons	**2.500.000 tons**	**2.000.000 tons**	**2.500.000 tons**
Incoming	**Incoming**		
13 ships	16 ships	13 ships	16 ships
10 ships	12 ships	10 ships	12 ships
7 ships	9 ships	7 ships	9 ships
3 ships	3 ships	3 ships	3 ships
total **32 ships/y**	**total** **40 ships/y**	**total** **32 ships/y**	**total** **40 ships/y**
Outgoing	**Outgoing**	**Outgoing**	**Outgoing**
15 ships	18 ships	15 ships	18 ships
17 ships	21 ships	17 ships	21 ships
17 ships	21 ships	17 ships	21 ships
total **49 ships/y**	**total** **61 ships/y**	**total** **49 ships/y**	**total** **61 ships/y**
81 ships/y	**101 ships/y**	**81 ships/y**	**101 ships/y**
1 ship/d	**1 ship/d**	**1 ship/d**	**1 ship/d**
Total ships/y 1.478	**Total ships/y 2.145**	**Total ships/y 4.883**	**Total ships/y 9.005**
Total ships/d 6	**Total ships/d 8**	**Total ships/d 15**	**Total ships/d 26**

3.5.3 BASIC MANOEUVRABILITY

Besides the vessel and cargo characteristics, it is for the port design crucial to get a feel for the behaviour of the design vessels in water and to quantify their manoeuvrability

characteristics.
Many large vessels have a poor manoeuvring capability; particularly those container ships build or originally build to operate at high service speeds [LIGTERINGEN, 2007]. Turning diameters are in the order of 6-8L, when applying a rudder angle of 35°. Turning diameters for large oil and bulk carriers at service speeds in the 15-17 kn. range are in the order of 3-4L.

The stopping distance is affected by several factors: the size of the vessel, the speed at which the vessel enters the port and the stopping procedure [LIGTERINGEN, 2007]. A 200.000 DWT bulk carrier or tanker requires about 14-18L from a cruising speed of 16 knots, which is around 5000 m.

From this, it is evident that the basic manoeuvrability of the vessels beforehand proves to be insufficient: the stopping length and turning diameter are much too large. To improve this, tugboats will have to be used to accomplish fast and safe manoeuvring and (de)berthing within the port. The requirement of tug services will be elaborated further on. The use of tugboats will have its specific influence on the port layout. The total number of tugs will be determined later on in accordance with [LIGTERINGEN, 2007], [TSINKER, 1997], [HENSEN, 1997].

3.5.4 SHIP HYDRODYNAMICS

To get an indication beforehand of the behaviour of the design vessels under influence of hydrodynamic forces, several possibilities of vessel movement need to be evaluated. These are indicated in the figure below.

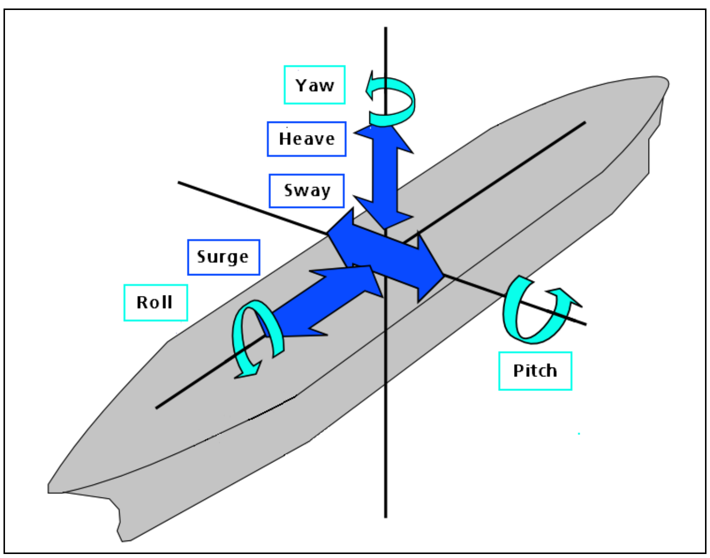


Figure 3.29

6 modes of freedom of motion: 3 lateral and 3 rotary

The wave forces on and the response of a sailing ship in waves can not be easily determined by analytical formulae. A first assessment of possible resonance can be obtained from the following reasoning (note that here only the smallest vessels are concerned: how larger the vessel, how less influence of waves on it):

- Pitching

When the ship sails in or against the direction of the waves, the pitch moment exerted by the waves is maximum for $L_{wave}=2*L_S$, with the corresponding wave period giving the highest response factor. For the smallest ships (the 20.000 DWT liquid bulk vessels) this amounts to a wavelength of $L_{wave}=2*L_S=320$ m. With a (largest) probability of exceedence of 0.1%, a wave period of $T_P=13.9$ s. occurs at P60. At a water depth of d=40 m. This means a wavelength of $L_{wave}=275$ m. (assuming relatively shallow water). This is still smaller than the wavelength for the maximum pitch of the smallest vessels, while 97.5% of the time, the wavelength is smaller or equal to L_S. Because of this, pitching will not be a real problem.

- Rolling

The eigen period of a ship for roll depends on its size, metacentric height and mass distribution. For 10.000 DWT cargo vessels this is about 7-8 s., and for 250.000 DWT vessels 12-16 s. [LIGTERINGEN, 2007]. For beam waves with periods close to the natural period, resonance will occur. While especially the lower periods occur rather often, this could result in problems for the smallest vessels (with eigen period of around 8-9 s). This happens around 9.44% of the time at P60, 8.05% at P54 and 9.49% at P01. For these larger percentages it is of importance to get the alignment/orientation of the approach channel right (taking into account directions). This is in order to minimize discomfort for visiting vessels.

- Heaving

For $L_{wave}=L_s$, the resultant vertical force of the ship is zero. For this corresponding wave period, the heave response is thus zero. With increasing wave period, and thus wave length, the incident force and the heave response will increase. With decreasing wave period the response ultimately reduces to zero. As outlined under 'pitching', 97.5% of the time the wavelength $L_{wave} \leq L_S$. From this it can be concluded that heaving will not pose real problems for the shipping traffic.

Rolling of vessels under influence of waves could turn out to be critical for vessels visiting the new port. Orientations of the approach channel under a small angle with the dominant wave direction could be a solution. Because a large share of the shipping traffic exists of small vessels (where rolling in particular could become critical), many vessels would experience this, and which should therefore be avoided. Exceedance of around 10% of time of the critical roll period for the smaller shipping traffic is rather large. Resulting from this, it is concluded that measures will need to be taken to minimize the probability of occurrence of roll on the vessels that visit the port (e.g. via alignment of the approach channel). This will be taken into account when designing the (alignment of the) approach channel, which will be a subject of the next chapter: the port masterplan.

CHAPTER 4 Port masterplan

4.1 INTRODUCTION

The sequence in which the port materplan design will be approached was already explained in chapter 2: at first, the wet areas of the port will be determined, followed by a thorough analysis of the dry areas. For this, first of all the shipping characteristics have been derived from the cargo forecast in chapter 3. This wet area comprises the approach channel, the manoeuvring space inside the port (including the turning basin) and the different basins optionally needed at the various terminals.

The design of the dry areas of the port comprises amongst others the required number of berths, the quay lengths for the terminals, the apron and the storage areas.
Sitting considerations and the construction sequence in phases will also be treated here. After this, additional (special) aspects are elaborated in more detail, for example the hinterland connections. In order to approach the masterplan in consecutive steps, here a division is made between the wet and dry port areas. Although treated independently, they are nevertheless interlinked and should be used jointly when designing the port layout.

After all the necessary elements have all been taken into account and determined, several layouts for the new port can be drawn up. Throughout the whole masterplan process, the influence of the transshipment aspect of the port is noticeable. These layouts all have their own pros and cons, and it is evident to choose a layout with the maximum possible value. This will be done by means of a MCA at the end of this chapter. From this, the final resulting port layout will be chosen, which will be subject of the wave penetration study and the breakwater design in the next chapters.

4.2 PORT'S WET AREA

4.2.1 INTRODUCTION

In the development of a port, nautical aspects play an important role: the movement of vessels in the approach channel and access areas, manoeuvring areas within port as well as mooring operations at the terminals. These manoeuvring characteristics depend on the type of ships that will visit the port and determine to large extent the space requirements.
The types of ships that will visit the new port have been deduced from the cargo forecast, in combination with a market analysis of most common ship sizes.

The wet surface of a port determines to a large extent the layout of the port. Choices made in this phase determine to a large extent the accompanying costs of the port. Besides this, it is imperative to make the right decisions regarding the port's wet area, because once the port has been built, it is very difficult to modify them [LIGTERINGEN, 2007].

The methodology for the wet port's design is as follows: first of all criteria need to be defined that allow safe and efficient port operation. From this, the influence of hydrodynamic conditions on the vessels will be assessed. Next up, with the earlier determined vessel characteristics the design of the approach channel can commence. This includes the orientation, alignment, depth and width of the approach channel, which will be elaborated further on. This will be followed by the manoeuvring areas within the port, and the port basins and berth areas. Finally, this will be concluded with an assessment of the breakwater layout.

4.2.2 OPERATIONAL AND LIMITING WAVE CRITERIA

Before starting with the approach channel design, first of all operational and limiting criteria need to be specified regarding the occurring waves and their influence on the vessels visiting the new port. It is emphasized that two different criteria play a role here:

1. Wave height outside of the port that allows port entrance
2. Wave height in-port that allows efficient loading and unloading at the berths

1. For the first criterion, the maximum allowable wave height where tugs can tie up to the vessels (while maintaining acceptable safety standards) plays a role [LIGTERINGEN, 2007]. When maintaining a maximum ship speed of 5-6 knots, the maximum wave height is about H_s=1.5 m. Because of increase in tug reliability and their capabilities of manoeuvring in bad weather conditions, and also because of the fact that a somewhat lower vessel speeds than 5 knots can be maintained, the maximum allowable wave height where tugs can tie up is set to be H_s=2.0 m.

This limiting wave height poses constrictions on moments of port entrance, as for a certain amount of time, the wave height is larger than H_s=2.0 m. It has to be assessed (later on) if this percentage of time is too large to accept as downtime, which makes an in-port length behind the breakwaters inevitable. The occurring wave heights differ from location to location at the project site, when reviewing the measurement points. Three points along the coast are considered here (the same as in the previous chapter): P54, P60 and P01, more or less along the same depth contours (see figures 3.9 and 3.10 in chapter 3.3.4) It is expected that around (on of) these point, the port entrance will be situated so they are assumed to be representative.

Output point P54

The tables where the wind was activated for all directions are considered here, as also the locally generated wind waves effects the (smaller) tug boats. From the wave data tables in 3.3.4 it is clear that the probability of exceedance of H_s>1.75 m.= 8.05%, and for H_s>2.25 m.=3.95%. In order to arrive at the probability of exceedance of H_s>2.0 m., the linear averaged of the two values has been used to be on the safe side.

Because of the logarithmic character of the probability of exceedance (decreases rapidly at first) this is a safe estimation. So this amounts to H_s>2.0 m.=6.00%.

Output point P60

Following the methodology as outlined above for the determination of H_s>2.0 m., for point P60 this gives for H_s>1.75 m.=9.44% and for H_s>2.25 m.=4.76%.
This result in H_s>2.0 m.=7.10%

Output point P01

Again, this gives for P01 for H_s>1.75 m.=9.49% and for H_s>2.25 m.=4.79%.
This result in H_s>2.0 m.=7.14%

While values towards 10% of port downtime because of the wave conditions are considered not acceptable anymore, the above determined values (which are only slightly less) have to be carefully judged later on.

2. The second criterion addresses the wave penetration in-port. Waves within the boundaries of a port may have been generated locally, or have penetrated from outside. The port layout has to satisfy two different requirements as far as wave penetration is concerned: (i) the operational conditions must allow efficient loading and unloading of the ships at berth, and (ii) for limit state conditions the ship must be able to remain at berth safely.

These operational conditions have been deducted from [LIGTERINGEN, 2007], and they differ for various vessel sizes and berth orientations. The criteria are summarized below. It should later on, (during the wave penetration study) be assessed whether the wave heights in-port are restricted to the mentioned values.

Table 4.1
Limiting wave heights for (un)loading of different vessel types

	Limiting wave height Hs [m]	
Vessel type	0 degrees (head or stern)	45-90 degrees (beam)
Container vessels	0.5 m.	-
Dry bulk vessels	1.0 – 1.5 m.	0.8 – 1.0 m.
Liquid bulk vessels	1.5 – 2.5 m.	1.0 – 1.5 m.

The above presented values indicate that for bulk vessels (provided that berths are properly oriented) the limiting wave height can be somewhat larger than for container vessels. This is due to the fact that for unloading of container vessels more crane precision is required.

While small as well as large vessels arrive at the new port, the lower criteria should be maintained when designing berths where all the vessels can (un)load efficiently. From the earlier deducted probabilities of exceedance for the wave height outside of the port, it is obvious that for smaller wave heights the probability of exceedance is even larger and simply much too large to accept as downtime:

Table 4.2
Probability of exceedance of two wave heights in several calculation points

Output point P54	Output point P60	Output point P01
H_s>1.0 m.=27.06%	H_s>1.0 m.=28.67%	H_s>1.0 m.=29.58%
H_s> 1.5 m.=12.78%	H_s>1.5 m.=14.37%	H_s>1.5 m.=14.89%

From these (large) percentages it is clear that this requires sheltered berthing within breakwaters for all the berths.

4.2.3 APPROACH CHANNEL

After this first assessment of the limiting wave conditions, the approach channel can be developed. The approach channel is defined as the waterway linking the turning circle inside a port (or an open berth at an offshore jetty) with deep water [LIGTERINGEN, 2007]. The layout of the approach channel is often largely dictated by the local sea-bed topography and other local conditions. The basic aim in approach channel design is the safe passage of all vessels from the sea to the berthing area and vice versa [PIANC, 1995a], [LIGTERINGEN, 2007]. The three design parameters are alignment, width and depth. These parameters will be determined further on, and are more or less interlinked. The approach channel has been designed according to [PIANC, 1995a] [LIGTERINGEN, 2007].

Alignment

Regarding the orientation of the approach channel, several guidelines can be given:

- Shortest possible length, taking in to account the wind and waves
- Minimum cross wind,
- Small angle with dominant wave direction,
- Minimize number of bends and avoid bends close to port entrance.
- Take into account sedimentation rates and directions.

From the previous chapter it is clear that there are roughly two main directions for the waves and winds (as currents were determined to be negligible). Their directional spreading (of origin) is summarized and presented in the figure below.

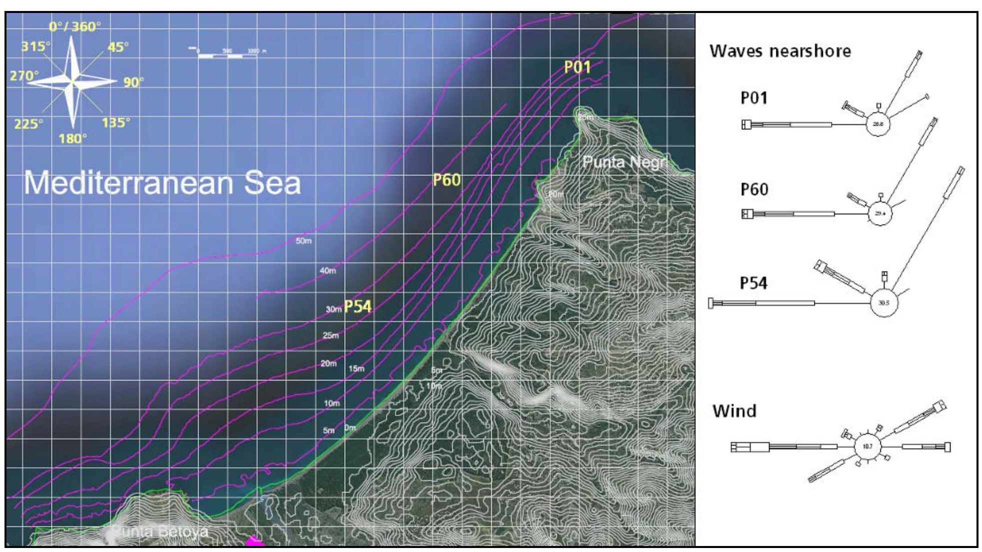


Figure 4.1

Graphical presentation spreading of near shore waves in 3 calculation points

With the above mentioned guidelines combined with the above presented environmental data, the alignment of the approach channel can be determined.

As mentioned before, a distinction has to be made between waves from the NE and waves from W-WNW. The waves from the NE are much smaller in height and period, with $H_{S;max}$=2,25 m. The waves from W-WNW are higher, with accidental waves with at maximum $H_{S;max}$=4,25 m. This last wave direction is therefore the dominant wave direction (the wave directions have more or less the same probability of occurrence, but the significant wave height from W-WNW is higher).

Because of this, it is advised to construct the approach channel under a small angle with the W-WNW dominant wave direction.

In order to protect the port from waves from this dominant direction, at first sight the entrance to the port can at best be located towards the NE. If this is possible, the approach channel and the stopping distances will be orientated such that these port items are not perpendicular to the hydrodynamic forces. Besides this, when approaching the port sailing against the dominant wind direction is advised, as this will facilitate ship control and reduce the actual stopping length required.

This leads to several possible options:

1. **Alignment approach channel: 210°N-225°N (northeast to southwest)**
   Under this angle, waves from the northeast are exactly in line with the approach channel. Nevertheless, these waves are small in height and they pose no real problems to the large(r) vessels. Besides this, the vessels sail against the dominant wave direction when entering the port. This reduces the stopping length required and facilitates ship control. Smallest (possible) angle with waves and winds. The entrance to the port can be located in such a way that not much dredging for the approach channel is required.

2. **Alignment approach channel: 180°N (north to south)**
   With this orientation of the approach channel, from the figure above it is clear that there will be very limited waves following to the channel direction, and less dredging is required. However, waves and winds from both the dominant directions (W-WNW and NE) reach the approaching vessels for a large portion of the time perpendicular to their sailing direction (beaming), which is definitely not desirable. (see paragraph 3.5.4, rolling of small vessels). Nevertheless, this orientation accomplishes easy port entrance for both shipping traffic from the east and the west.

3. **Alignment approach channel: 120°N-135°N (northwest to southeast)**
   Under this angle, there are for a considerable portion of time (30-34%) waves following the vessels in the approach channel (W-WNW) for vessels approaching the port.
   This results in a larger stopping length and it is more difficult for vessels to manoeuvre. Beaming of the vessels occurs by waves from the northeast. Although this is the lesser dominant direction, as in 3.5.4 was assessed for the smaller vessels this could give rolling problems. Also, wave penetration in-port is expected under this angle of the approach channel.

4. **Alignment approach channel: 75°N (west-southwest to east-northeast)**
   With this orientation of the approach channel, the entrance of the port is located very inconveniently for shipping traffic from the east: these vessels will need to make a large bend before entering the port, which is undesirable.

Besides this, with this orientation the largest amount of dredging is required, and waves from the west are expected to penetrate in-port. There will be following waves from the dominant direction which increases the stopping length and decreases the vessels' manoeuvring capabilities.

Several other aspects also need to be taken into account when locating the approach channel. For instance, it is emphasised that dredging should be kept as low as possible, especially around Punta Negri because of the hard rock (andesite). It is at this location advised to locate the approach channel more out of the shore if possible (parallel to the depth contour lines) in order to avoid the need for dredging.

Besides (high) dredging costs, another design consideration is breakwater costs, which are to large extent determined by the water depth. It would be preferable to construct a shore parallel breakwater, in order not to go to far into the deep water, because the sea bottom exhibits a steep slope. These two requirements (minimize dredging and breakwater costs) are however contradictory.

From the above it is evident that a consensus for the optimal approach channel orientation needs to be found. After a first consideration, option 1 (alignment 210°N-225°N) looks the most promising alternative, when taking into account the dominant directions, the ease of port entering and vessel handling. Nevertheless, the final decision can only be made when using all the port items jointly. This will be assessed later on.

Depth

The next design parameter of the approach channel is the depth. The depth of the approach channel is more or less interlinked with its width, because of the influence of the depth on the manoeuvring characteristics of the ship, which influences on its turn the required width [PIANC, 1995a]. It is because of this that first the depth of the approach channel will be determined and subsequently the width.
The depth of the approach channel is calculated using the following formula [PIANC, 1995a]:

$$d=D-T+s_{max}+r+m$$

In which:

d = guaranteed depth (with respect to Chart Datum=LAT) [m]
D = draught of the design ship [m]
T = tidal elevation with respect to CD, below which no entrance is allowed [m]
s_{max} = maximum sinkage (fore or aft) due to squat and trim [m]
r = vertical motion due to wave response [m]
m = remaining safety margin or net under keel clearance [m]

The choice for the different parameters will be outlined below.

Draught of the ship "D"

There is a large difference of 1.5 m in draught between the largest ship size (18,9 m for the 200.000 DWT liquid bulk vessel) and the second largest (17,4 m for the 150.000 DWT liquid bulk ship), which follows from paragraph 3.5.

From this it is evident that these largest vessels arrive (in phase II) only 21 times per year. Because of this low frequency it could be economically justified to introduce a tidal window to reduce the dredging costs of the approach channel. This is however not advised because of the small difference in tidal levels that follows from paragraph 3.3.2. These small differences cannot compensate the differences in draught between vessels and could only lead to a (small) reduction of the channel depth.

Because it is evident that ships with a draught of 18,9 m must be able to enter the new port ultimately in phase II, these vessels are determining for the depth of the approach channel. However, in phase I only vessels up to a draught of 17,4 m. visit the port which could lead to a smaller approach channel depth required in phase I. This depth can than be enlarged in phase II required for the 18,9 m. draught vessels. This is the proposed approach.

Tidal elevation "T"

For a first estimate, the tidal elevation is chosen to be zero. The difference between LAT and MLWS (which occurs once per month) is only 13 cm (see paragraphs 3.3.2), so with of a small increase in depth of the approach channel, it is possible for ships to enter the port at all times. The port will benefit from this in the long term, and congestion in front of the port is by means of this brought back to an absolute minimum.

Maximum sinkage "s_{max}"

The next parameter presents the maximum vessel sinkage due to squat and trim. This can be estimated on basis of experience to be around 0.5 m [LIGTERINGEN, 2007]. An alternative for this would be the squat graph from PIANC [PIANC, 1995a]. This results in a s_{max}=0.7 m, which is in accordance with [TSINKER, 1997]. For safety reasons this larger value is used. It will be checked later on if this value is correct (as for this calculation the approach channel width and depth need to be known).

Vertical vessel motion due to wave response "r"

The vertical motion by wave response is determined to be ½*Hs [LIGTERINGEN, 2007]. This is near shore around r=½*2.25 m, which takes into account waves for around 97% of the time. Because of this, r=1.13m.

Net under keel clearance "m"

For m, the value of m=1 can be adopted to give an indication for the hard soil or rock bottom to be on the safe side [LIGTERINGEN, 2007].

This yield:

$d=D-T+s_{max}+r+m=18.9-0+0.7+1.1+1=21.7$ m.

It is emphasised that the depth calculated above is the guaranteed water depth available. 99% of the time, the water levels are higher than CD (=LAT), which results in more water depth available. This is only good for the safety level for the depth and the extra reserve added depth to the approach channel.

It should further be noted that this depth is the guaranteed depth during the lowest astronomical tide, and that the bottom could be situated even somewhat lower.
This is because the dredging tolerances and the type of dredging scheme applied.

Additional Required depth

Another additional factor that could play its role is the wind set-down. The wind set-down will not be that large, because of the large average depth of the Mediterranean Sea. The calculations have been done in paragraph 3.3.2 (water levels), which resulted in a wind set-down of 0,07 m. This will be added to the required depth

Next to that, there is the variation (and, what is here important: decrease) of the water level because of variations in atmospheric pressure [PIETRZAK, 2008]. This has been included (as determined earlier in) by means of a safety margin of several decimetres.

The total required depth of the approach channel for the different phases is presented in the table below.

Table 4.3

Depth approach channel

Depth [m]	Phase I	Phase II
d	20,5 m.	22 m.

Width

After determination of the alignment and depth of the approach channel, the required interlinked width can be determined. The width of the approach channel is composed of several elements. The design width has been determined by means of the methodology according to PIANC, where the channel width is expressed in several factors times the design vessel's beam [PIANC, 1995a].

For straight sections, this design methodology is as follows:

- **For a one-way channel:**
  $W=W_{bm}+\Sigma W_i+W_{br}+W_{bg}$

  in which:

  W=width of the waterway [m]
  W_{bm}=basic manoeuvring width
  W_i=width additions according to situation and tables
  W_{br} & W_{bg}=bank clearances

- **For a two-way channel:**
  $W=2W_{bm}+2\Sigma W_i+W_{br}+W_{bg}+W_p$

  in which:

  W_p=separation distance between ships

A major consideration will be whether the channel should be wide enough to allow ships to pass in opposite directions. A typical argument for a two-lane approach channel is that if there is an accident in one lane of the channel, access to the port will still be possible via the other lane. This results in less disruption of traffic to and from the port.

Nevertheless, a two-way approach channel is much more expensive to construct and maintain. Because of the fact that the amount of shipping traffic is not that large (see 3.5.2: at maximum 26 ships/day, which is about 1 vessel/hour in phase II^{+}), it is very well not necessary to construct a two-lane approach channel.

The total result exists of the sum of several factors, expressed in a number times the design ship's beam [PIANC, 1995a]. These factors are determined, and explained in the Annex 3.2.1. The width of the approach channel has been determined for the container port and bulk port independently. This is because of the requested independent development of both ports by the client.

The calculations for the determination of the channel width according to PIANC [PIANC, 1995a], lead to the results for the design vessels are summarized in the next table:

Table 4.4

Width approach channel

Width [m]	B_{MAX} [m]	Σ-factor	W [m]
Container vessel	56	4,1	**230**
Crude oil vessel	53	4,6	**244**
Product tanker	48	5,1	**245**

The beforehand estimated squat from the PIANC graph has been calculated more precisely after determination of these design values. The calculations have been added in annex 3.2.2. From this it can be concluded that the earlier used value s_{max}= 0,7 m. can still be applied here.

As a concluding remark, it is emphasized that anchorages should be provided along the length of the channel of which the last one should be located close to the port [UNCTAD, 1985b]. Because of the fact that the approach channel is rather deep before it reaches port, it is assumed that ships can wait in the Mediterranean Sea, alongside of the approach channel.

4.2.4 MANOEUVRING AREAS WITHIN THE PORT

Now that the relevant parameters for the approach channel have been determined and assessed, manoeuvring areas within the port will be determined. This will be done by taking into account the basic manoeuvrability of the vessels, which was considered in paragraph 3.5.3. From this it was obvious that tugboats were required to increase the manoeuvring capabilities of the vessels. With this assumption, first of all the turning basin can be designed.

Turning basin

The inner approach channel should end in a turning basin or circle, from where vessels, whether small or big, are towed by tugboats to their respective basins. Because it has already earlier been outlined that manoeuvring of the large vessels without tugs becomes problematic, it is evident that enough tug support should be available. This determines the diameter of the turning circle required.

Diameter

With the use of tugboats, the diameter of the turning circle should be equal to, or greater than 2*L_s, where L_s is the largest ship length [UNCTAD, 1985b], [TSINKER, 1997], [HENSEN, 1997].

Only in ports where there is no tugboats are available, the diameter should be equal to, or larger than 3 L_s. This is subsequently not the case for the new port at Nador, as defined earlier. Currents are negligible, so no extra lengthening of the turning circle is necessary [TSINKER, 1997].

Given the adequate handling of the large vessels with tugboats, the diameter of the turning circle comes down to 2*L_s. The vessel with the largest length (over all) is the 14.000 TEU container vessel with L_s=398 m. The diameter of the turning circle should at least measure D≥2*398=796 m. in length. This results in a diameter of the turning circle of D=800 m.

This leading situation is the same for phase I, and for phase II, since the 14.000 TEU container vessel is in length the largest vessel that will enter the port. While the daily shipping traffic in phase I is still limited (at maximum 8 vessels/day), this more than triples in phase II. This places a much higher load on the one turning circle available. It is therefore advised to consider the possibility of constructing another turning basin, for at least part of the shipping traffic (e.g. for the bulk vessels in a separate basin). Besides this, it could be safer to locate a separate turning circle for the liquid bulk vessels somewhere nearby.

This secondary turning basin does not necessarily have to be as large as the primary one. For example when taking a secondary turning basin where all the bulk ships can turn, this would result in a diameter of D_2≥2*327=654 m. A vessel choice for this second turning circle could also be the 6.000 TEU container vessel, which arrives rather frequently (5-6 vessels/day in phase II$^+$). Here, the lion's share of the container vessel can turn. The diameter of this second turning circle should at least measure D_2≥2*310=620 m. in length, which is almost 200 metres smaller than the primary turning circle.

Depth

For phase I, the required depth for the turning circle can in a first estimate be taken equal to the required depth for the approach channel, which is determined by the design ship: d=CD -20.5 m. in phase I. This is in fact an overestimation of the required water depth of, because the vessel response due to waves is still included here. Nevertheless, to be on the safe side for this first estimate, a water depth for the turning basin equal to the approach channel is used, and extra safety is included for possible wave penetration in-port.

Nevertheless, for phase II the situation is somewhat different. Here, it is not necessarily required that the depth of the primary turning circle is increased, depending on its location and the presence of a secondary turning basin. This is subject to variation in the design. For example, if a second turning circle is constructed (closer to the port entrance) that will be used only by the liquid bulk vessels with the largest draught, this turning circle would only need the accompanying depth of d= CD -22 m. (but could require a smaller radius). A container terminal turning basin would only require a depth of d=D+s_{max}+m+r+extra additions= CD -18.5 m.

Considering the location of the turning basin: it should be advised not to locate the turning circle directly near hard structures [UNCTAD, 1985b], [LIGTERINGEN, 2007] (breakwater, terminals), but more spacious and if possible in the middle of the port, so that increase in diameter (taking into account the growth of vessels in the future) is still possible. Also this will be included in the final layout.

Tug Boats

The manoeuvring of small to medium vessels generally poses no problem in the sense that specific measures have to be taken in the dimensioning of the port infrastructure. For large ships this situation is different [LIGTERINGEN, 2007]. They have a much longer stopping distance and lack of course control during the stopping manoeuvre. Next to this, because of the restricted D/d ratio, their manoeuvrability behaviour becomes sluggish. As determined before, use will be made of tug boats. The number of tugs required per vessel depends on the vessel's size and on the available bollard pull of the tugs [TSINKER, 1997], [HENSEN, 1997].

First of all, the use of tugs poses the question what vessels make use of tugs, and what ships do not. The use of tugs applies to ships of about 50.000 DWT and over [UNCTAD, 1985b]. Here, it is assumed here that this is the case with the vessels of 60.000 DWT and over. This means that for the shipping traffic at Nador around 90% of the vessels require tug assistance in phase I as well as in phase II.

There should be at least two tugs available per vessel for controlled manoeuvring behaviour of larger ships: one fore and one aft [UNCTAD, 1985b]. For the smallest ships requiring assistance, two tugs could prove to be sufficient, and depending on their bollard pull, more tugs are needed for the larger vessels. The required tugs for the different ship sizes can be calculated according to the following formula [HENSEN, 1997]:

$T_B = \Delta \,/\, (100.000) * 60 + 40,$

In which:

T_B= total bollard pull required
Δ = water displacement (tons)

Because of the amount of shipping traffic, and the fact that several tugs per ship are needed for safe manoeuvring, this means that several tugs will have to be available. It is assumed that for container vessels tugs are needed for entering the port and safe manoeuvring and mooring. When they have completed this task, they can subsequently start a new one, and do not have to stay at the moored ship while unloading.

This is in contrast to the tugs used by the mooring and manoeuvring of oil tankers. Here, the tugs will continually need to be available in case of emergencies (fire, breaking of a pipe) to escort the oil tanker to a safer location [UNCTAD, 1985b], [PIANC, 2000], [HENSEN, 1997]. This poses special constraints on the total number of tugs that should be available, because tugs that tow oil tankers will stay with these vessels also during unloading (because of safety reasons and their fire fighting equipment).

A safety margin has been included in the available bollard pull for the tugs: tugboats operate with at maximum 80% of their maximum bollard pull capacity, which leaves some room for additional capacity in case of emergencies [HENSEN, 1997]. Besides this, also an extra factor regarding the total number of tugs has been added, to account for the unavailability of tugs (e.g. in case of machine failure).

The types of tugs used are again determined by means of a market analysis and in accordance with sources [PVE MAASVLAKTE II], [WEBSITE PORT OF ROTTERDAM], [HENSEN, 1997]. This has lead to two types of tugs. Their characteristics are outlined in annex 3.3, together with the calculations regarding the required number of tugs and the ships that make use of tugs. A summary of the required number of tugs is presented in the next table:

Table 4.5

Number of tugs required per phase

Required number of tugs			
	Z-peller	Schottel	Total
Phase I –	4	4	8
Phase I +	5	4	9
Phase II -	7	6	13
Phase II +	11	7	18

The division of the tugs according to the total amount of shipping traffic and cargo handling (throughput) differences in the port between the four different phases seems a little wrong: in the first phase and the second phase, the increase in tugs is not that large. This is caused by the fact that tugs for the oil tankers have to be continually available, regardless the other tugs. This adds strongly to the total tugs. This influence becomes less when the total amount of other cargo increases.

Within the port masterplan, a separate area for small craft (tugs, flats ant pilot launches) will have to be available. Because of their size, these vessels are more sensitive to wave distribution and hence the location of the small craft harbour must on one hand be well protected and on the other hand not too far from the port entrance, where they have to pick up incoming ships and let go the departing vessels [UNCTAD, 1985b], [LIGTERINGEN, 2007]. This could be achieved by creating a separate basin (with the appropriate depth) protected by its own breakwater.

Manoeuvring with tugboats

After determination of the required number of tugs, the manoeuvring of the vessels with the tugboats needs to be further elaborated. The most important factors that play a role by the slowing down and stopping manoeuvre are mentioned below, according to [LIGTERINGEN, 2007], [TSINKER, 1997], [HENSEN, 1997]:

1 Entrance speed of the ship

One very important aspect for the stopping length of the vessels is the speed at which they enter a port. It is advised that ships maintain a maximum speed (in-port) of 4.5 knots, for safety and for the ease of tugs tying up (although this can be done up to a speed of 6 knots). Nevertheless, in this way, ships sail with a speed (for ships and tugs) to establish safe manoeuvring which also reduced the required in-port stopping length (see 3.). While it is possible for vessels to sail with a speed up to 12 knots in the approach channel, it is necessary for ships to slow down their speed until 4.5 knots. This speed has to be reached at latest just outside of the breakwaters. This speed includes a safety margin for probable cross- and longcurrents, and is needed for the ship to still react to the ruder.

2 Time required to tie up the tugboats

From hereon, surrounded by calm in-port conditions, the tugs can fasten to the ship. The time required for this is on average about 10 minutes [LIGTERINGEN, 2007].

Within these 10 minutes, the ship sails continuously with speed of 4.5 knots (=4.5*0.5144=2.3 m/s). The vessel travels within these 10 minutes a distance of 10*60*2.3=1380 m.

3 Actual stopping length

Next up is the actual stopping length of the vessels by tug assistance. [LIGTERINGEN, 2007] states that the actual stopping distance (when the ship gives full astern power) is about 1.5 L_s by a speed of 4 knots. Here, it is expected that ships sail in-port with speeds up to 4.5 knots, so subsequently the stopping distance becomes 1.69 *L_s =673 m. for the largest container vessel and 1.69*318=537 m. for the largest bulk vessel.

The total required length in-port for slowing down and stopping (with assistance of tugs) totals rounded off to about 2060 m. for the container vessels and 1920 m. for the bulk vessels. At this point, the turning circle and the inner port channel can connect, so the effective length of the inner port channel is even increased (e.g. with 800 m, the diameter of the primary turning circle).

From this it can be concluded that the tugboats considerable increase the manoeuvring capabilities of the vessels, because the stopping length and turning radius are considerable reduced in contrast to the situation without tugboats.

It should be noted that if wave conditions are favourable, tugs can fasten to the vessels already outside of the port. This situation applies if the wave height at the specific location near shore amounts to $H_s \leq 2.0$ m. [UNCTAD, 1985b], [LIGTERINGEN, 2007]. This considerably reduces the required slowing down and stopping length within the port and subsequently gives more efficient handling of the shipping traffic and relieves pressure of the turning circle.

From 4.2.2 it is clear that this is the case for around 93% of the time near shore. This means that for 7% of time, wave conditions outside the port do not allow tugboats to fasten to the vessels outside of the port, and thus no port entry is possible. This percentage of time is generally somewhat high to rashly accept as port entry downtime. However, this has to be carefully assessed while taking into account the total amount of shipping traffic arriving at the ports.

It is assumed that this percentage of downtime is still acceptable for the case of the (liquid and dry) bulk vessels, because the amount of shipping traffic is simply not that large (at maximum 3 vessels/day ultimately in phase II^{+}). This means that for the bulk port an in-port stopping length is not required intrinsically. However, for the container terminals this is subsequently not the case because of the much larger amount of shipping traffic (23 ships/day in phase II), and this would result in too much downtime and congestion. Because of this it is argued that the stopping length for container vessels is required in-port. This results in a larger availability and efficiency of the container terminal.

As a concluding remark, it should be noted that the approach channel inside the breakwaters (which measures around 2000 m. in length) should be kept at its original width, even with the absence of waves. This width seems to be an overestimation. This is not necessarily the case because vessels entering the sheltered water behind the breakwaters have a certain drift angle to counteract the forces of wind and waves.

This angle may increase as the vessels reach the calm in-port water conditions and they correct their course accordingly.

4.2.5 PORT BASINS AND BERTH AREAS

While in the previous paragraph the principal manoeuvring spaces have been determined, here the size of the port basins and berth areas where the ships moor will be calculated.

Basin widths

These are however subject to variation, and depend on the alternative chosen for the layout. Since this final layout is not yet known at this stage, the determined dimension in this paragraph give indications for a possible design if these would be included in the layout, e.g. the dimensions of a basin for the dry bulk terminal has been determined and could be used in a specific alternative, but this doesn't necessarily have to be the case for all the alternatives. It simply explains the origin of the used design dimension. Besides this, the length of the basins is at this point undetermined. This is because the required quay length and number of berths is not yet known. This is a free design parameter that can be used.

In paragraph 4.2.4 the length of the approach channel in the sheltered area behind the breakwaters has been determined (with a length of around 2 km.). This channel ends in the primary turning circle of the port. These two elements already pose special constraints on the port layout, let alone when the second turning circle will be added. [TSINKER, 1997] states that also around the turning circle room has to be available for different purposes (anchorage, berthing, special purpose and manoeuvring). Next to these, port basins could be provided with sufficient width for the safe towing in and towing out of the vessels, whilst other berths are occupied.

Design considerations and calculations are outlined in annex 3.4. The result is a table with two different design widths for the basins: one for double-sided berthing in the basin, and one width for the basins if ships have to be turned in it.

This first width is roughly the same for liquid bulk and container vessels and amounts to 400 m. Also dry bulk vessel basins can make use of this dimension, e.g. if they share it with other vessels. If ships should be able to turn in the basin, larger width is required, ranging from 400 m. to over 500 m. for the container vessels.

Basin depth

It is for the bulk vessels simply not necessary to reach the deeper in-port part of the container terminal. At the container berths, only a depth is required of d=18,5-1,1 (because of absence of wave response)= CD -17.4 m. The depths of the approach channel and turning basin for container vessels is, as outlined earlier, calculated at d= CD -18.5 m.

Depending on the location of the liquid bulk berths, the (separate) bulk basin itself should be dredged to a depth of d=20,5-1,1=CD -19,4 m. in phase I and d=22-1,1= CD -20,9 m. in phase II. The required depth in front of the dry bulk berths will be somewhat smaller (because of the smaller draught): d=19,4-0,3=CD -19,1 m.

Results are summarized in the table below.

Table 4.6

Required port depths for different terminals in phase I and II

	Phase I	Phase II
Container terminal		
turn basin + channel	CD -18,5 m.	CD -18,5 m.
mooring basin	CD -17,4 m.	CD -17,4 m.
Liquid bulk terminal		
turn basin + channel	CD -20,5 m.	CD -22,0 m.
mooring basin	CD -19,4 m.	CD -20,9 m.
Dry bulk terminal		
turn basin + channel	CD -20,2 m.	CD -20,2 m.
mooring basin	CD -19,1 m.	CD -19,1 m.

As emphasized before, numerous possibilities for the arrangement of basins exists for the different ship sizes. This can be seen a degree of freedom in the design to develop specific layouts.

4.2.6 LAYOUT OF THE BREAKWATERS

In the previous paragraphs, dimensions have been determined for the various wet areas of the port. Together with the terminals, these are enclosed by breakwaters. When taking into account morphological and hydrodynamic aspects, now a layout for the breakwaters can be determined.

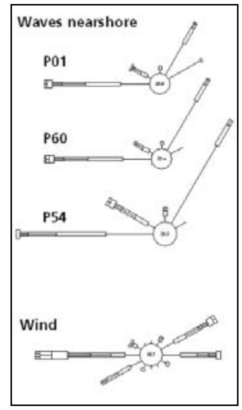
From 3.4.3 it is clear that the yearly averaged longshore sediment transport is directed to the northeast along the coast. Nevertheless, during times of heavy rainfall sediment is transported from the discharging wadis to the coastal area. Various processes (wave action, diffusion) transport these sediments momentary also into other directions. [LIGTERINGEN, 2007] states that if litoral transport is predominant in one direction, one breakwater may be sufficient. This is not necessarily always the case here, also when reviewing the probability of occurrence of the environmental forces in both directions.

So it is advised to construct two breakwaters which interrupt in the longshore sediment transport. This means that they have to be constructed sufficiently deep to avoid deposition of sediment in the port. At first approximation this is until a depth of [LIGTERINGEN, 2007]: d_b=1,6 H_s , in which H_s is the annual deep water wave height. From the extreme near shore wave data in chapter 3 and [ALKYON DATA] it is evident that the largest significant wave height at the project location arrives from 270 degrees, with significant wave height of H_s=5,7 m. This means that the breakwater should reach until a depth of at least the 1.6*5,7-0.35 (MSL)= CD-8,8 m. contour.

Because of the fact that the yearly averaged longshore sediment transport is from southwest to the northeast, the left (westerly) breakwater should be longer than the right (easterly) one, and also properly oriented to prevent waves and sediments from entering the port.
This is also especially true for the waves from the dominant direction (around 270°N).

Figure 4.2

Direction longshore sediment transport and dominant wave and wind directions



This particular breakwater orientation already limits the possibilities for the approach channel alignment as determined earlier: option 4, where the approach channel was oriented around 75°N is with this decision almost not feasible.

A compromise needs to be found between the most hydraulically effective solution and the ease of access of ships entering the port. This means a breakwater with sufficient space between the heads for easy access of ships, yet providing enough cover from the dominant wave directions [LIGTERINGEN, 2007].

Further more, it is advised to construct (more or less) shore-parallel breakwaters. The steep sloping seabed at the project location (especially around Punta Negri) already reaches large depth (d>25 m) several hundred meters off the coast. This means an expensive breakwater if it has to be constructed too far off coast.

The length of the breakwater should be minimised as these form an important cost factor [UNCTAD, 1985b]. Also, the breakwaters at the entrance of the port should not form a narrow 'sleeve' but provide space immediately behind the heads [LIGTERINGEN, 2007]. This because of the earlier mentioned reasons that ships enter a port under a drift angle (which is enlarged when reaching the calm water in-port) and so require more manoeuvring space. Besides this, open room behind the breakwater helps the diffraction effect and reduces wave penetration [LIGTERINGEN, 2007].

Besides this, immediately next to the breakwaters enough room must be available for ships manoeuvring because of the hard nature of the structure which could damage ships in case of a collision. So a spacious layout is recommended, (which also helps the diffraction effect) where there is a lot of safety manoeuvring space. Also room behind the turning circle is advised, because in case of a failed stopping manoeuvre the ship has some room left to come to a halt. Because of this, hard structures should not be present just behind the turning circle [LIGTERINGEN, 2007], [TSINKER, 1997].

Possible breakwater layouts are presented below, for the different approach channel alignment options. It should be noted that these are not final layouts but only possible orientations, as these have not yet been combined with the dry port areas. Only then, a decision for a final layout can be made.

Figure 4.3

Approach channel alignment 210°N, with breakwaters indicated. The black part indicates the part of the approach channel and/or turning circle that needs to be dredged to -22m. ultimately required in phase II.

The least amount of wave penetration from dominant directions, although close to Punta Negri.

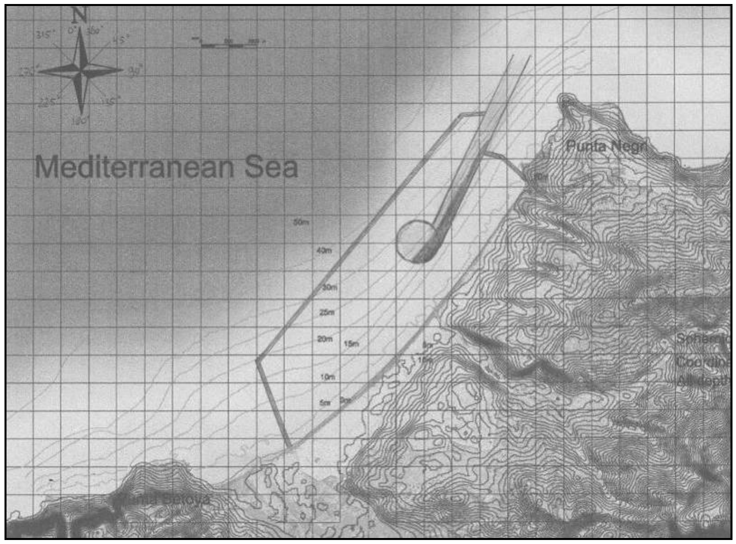


Figure 4.4

Approach channel alignment 180°N, with breakwaters.

Beaming waves from the dominant directions will be critical here, as will be the available stopping length in-port. The layout is still spacious and wave penetration from 0°N will pose no problems.

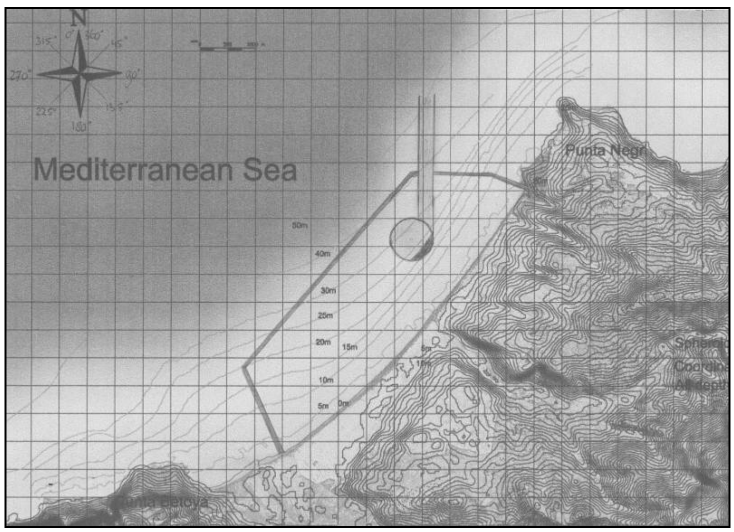
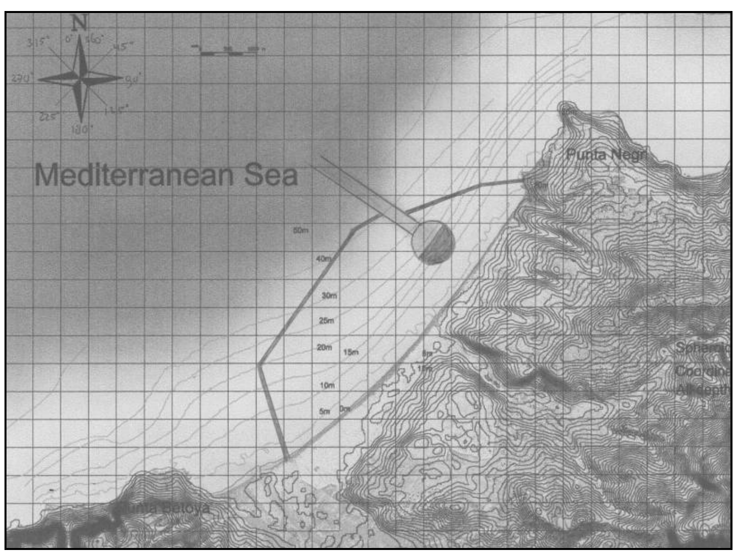


Figure 4.5

Approach channel alignment 120°N, with indicated breakwaters.

The vessel approach is perpendicular to the shore, which limits the stopping distance available. Also waves from NW pose problems to navigation and will penetrate into the port



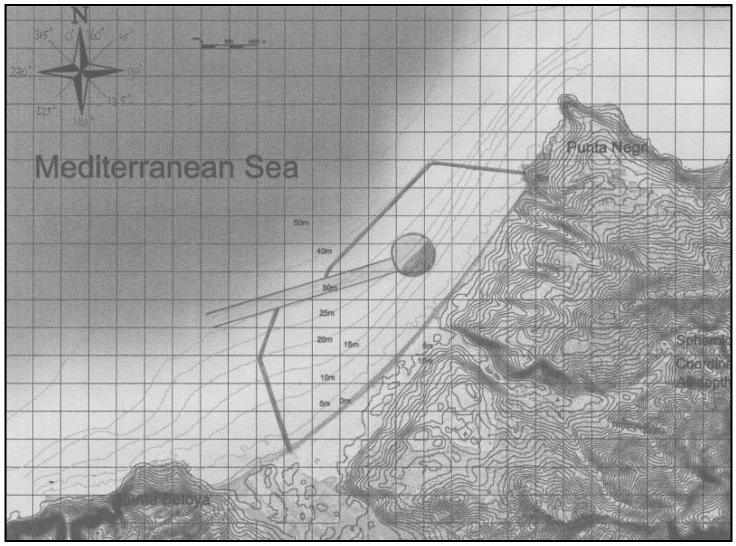


Figure 4.6

Approach channel alignment 75°N, with breakwaters indicated.

Large port entrance in width as a result of avoiding longshore sediment transport to enter. Wave penetration from the N will occur, and the waves from the dominant direction (W) can also enter the port.

Still the preferred approach channel layout is number 1, where the stopping length can be acquired by using the space inside of the breakwaters for the oil terminal. So at first glance, this direction will be more or less maintained as preferable approach channel alignment, when taking into account the layout of the breakwaters, hydrodynamic forces and morphological aspects. It is emphasized that the exact angle is still subject to variation in the design later on.

4.3 PORT'S DRY AREA

The need to pay close attention to planning land use in port areas begins at the moment the idea of port development arises and does not stop until a port is built: in other words, the planning of land use in ports is a continuous process.

Land is a limited resource which can be even more limited by the way a port is planned or administered and, once fixed, it is, with the mooring areas, the most inflexible part of a port. Increases in the size and cost of ships and the growth of international trade let to artificially constructed berths and the increased amount of cargo handled meant that wider quays were needed for efficient working.

4.3.1 INTRODUCTION

The requirements in land use planning for operational areas are directly related to the cargo capacity of vessels [UNCTAD, 1983], the possibility of the equipment to transfer the cargo fast enough to the hinterland (although this is not relevant for the new transshipment port).
The most important influences on the amount of space required for port operations are ship sizes and the storage characteristics of the cargo. It is important to qualify these factors by allowing for future demands which may increase the requirements. This will be done for each terminal later on in this paragraph.

Also here, transshipment will have its specific influence. For example, container vessels will unload part of their containers and subsequently will be loaded with containers.

Because of this, less storage space is required for the container terminals as incoming and outgoing cargo will be shipped in the same vessel. This has been clarified in the next figure.

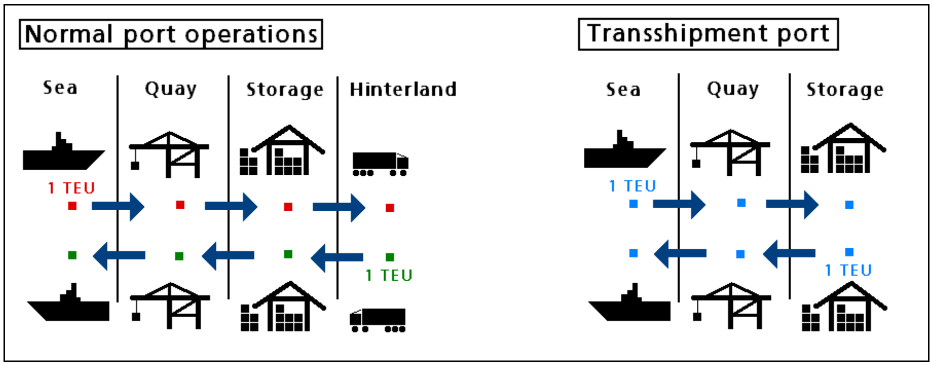


Figure 4.7
Normal port operations versus transshipment port: the cargo flows

During normal port operations, cargo is transported from the vessel via the quay and the storage to the hinterland. It will take several days (the storage time) before the cargo is transported to the hinterland. Subsequently, other cargo arriving from the hinterland will be stored several days before it is forwarded overseas. This results in considerably larger storage requirements in contrast to transshipment port: here, incoming cargo are only stored several days after arriving until this same cargo is forwarded overseas.

From the above it is clear that transshipment has its influence on various port elements and will therefore be noticeable throughout the whole design process. Besides the transshipment aspect of the port, also the in two-phase construction of the port has to be taken into account. The masterplan which will be developed here is aimed on 2 different time horizons: the short-to-medium term and the long term. This means that the required capacities should be reached at the end of the accompanying phase (if this is possible).

First of all it is important to get a proper allocation of the different items in the port design.

4.3.2 SITING CONSIDERATIONS

There is a distinct diversity possible in sitting of the new port along the coast, and within this port the sitting of the terminals and their berths. This results in numerous possible layouts. Several guidelines can nevertheless be given to ease the process in creating an effective port layout.

The Port Items

The Liquid Bulk Terminal
In the project specifications for the liquid bulk terminal it is stressed that several oil products will be transhipped, under which: crude oil, gasoline, diesel and LPG. The most important condition here is safety [UNCTAD, 1985b] [LIGTERINGEN, 2007], as all of the abovementioned products are inflammable and polluting. Hence, loading and unloading of these commodities should be done in separate basins in-port, completely isolated from the rest.

This rules out deep water jetties outside the port, not only because of reasons of safety, but also because the maximum acceptable wave height during unloading of vessels is exceeded around 10% of the time (see paragraph 3.3.4 where $p(H_s \geq 1{,}5\ m) \approx 15\%$. This is too large to accept as downtime.

Also, Single Buoy Mooring facilities (SBM's) will not be used as they are only more economical than jetties when considering small to moderate yearly throughputs. This is clearly not the case, see 2.2.3, where it is specified that the throughput will be around 40 MT/year. Another advantage for SBM's is that only tankers not larger than 50.000 DWT can be handled within 1 day [LIGTERINGEN, 2007]. Much larger vessels – up to 200.000 DWT – will visit the new port, which would result in a considerable longer service time and higher berth requirements.

So (mainly) for safety reasons the liquid bulk berths will be located in-port. Liquid bulk terminals are often located near the port entrance [UNCTAD, 1985b] [PIANC, 2000]. From this location, in case of accidents, liquid bulk tankers can quickly get out of the port and reach open water (which is considered safer, at least for the port activities). From this is can be deducted that it is convenient to locate the liquid bulk terminal downwind of other port facilities. This is because of the risk of fire spreading or liquid bulk drifting in the direction of the wind.

The above outlined considerations lead to a location which is situated around Punta Negri, to the northeast. This location is close to the entrance route (approach channel) as well as most of the time (42%) situated downwind from the other port activities. Besides this, there are (limited) possibilities for expansion available because the liquid bulk terminal increases in throughput throughout the phases. This mainly applies for the required surface area of the tank farm and the refinery. It should be taken into account that in phase II more berths will be needed. These berths will accommodate the ships with the largest draught, so it would be advised to situate the berths in deep(er) water which is already present by nature: also to the northeast.

Tugs and Port Service

The tugs should (also) preferably be located close to the port entrance. This is necessary in order to reduce time of picking up or letting go the ships that visit the port. Besides this, a location around the port entrance is convenient because here, also the liquid bulk terminal is situated. For these berths, it is advised to keep tugs at standby in case of emergencies where they can quickly evacuate liquid bulk vessels to open water. [UNCTAD, 1985b] states that these tugs should have all the necessary requisites to function as escort tugs in these type of situations, which means they should be equipped with a proper fire fighting system because of the type of (flammable) cargo handled at the liquid bulk terminal.

So tugs can at best be located close to the port entrance, to the northeast around Punta Negri. Because of their smaller size, they are more sensitive to the waves than other (larger) ships that visit the port [LIGTERINGEN, 2007], [HENSEN, 1997]. It is therefore recommended– in order to protect the tugs against this – that a (more or less) separate basin or extra breakwater is provided which would create calmer in-port conditions.

An efficient place to locate the port services is at a central location, or also around the port entrance. Here, they can keep an eye on all the events taking place in the port, registering shipping traffic, providing many additional services, and so on. The incorporation of the port services in the masterplan usually does not pose much of a problem: the area requirements are compared to other terminals not that large.

Dry Bulk Terminal

The dry bulk terminal does not increase (that much) in throughput in the different phases, which makes it suitable to locate at a rather difficult position that is later on harder to modify (because this is simply not necessary).

The ships transporting dry bulk call the least frequent in the new port, but it should be noted that these ships have the second largest draught, and should therefore also preferably be located around the port entrance (because there the depth is already larger because of the liquid bulk vessels). This way, the part of the port that requires the deepest dredging stays limited to the part of the port located close to the entrance.

The above outlined considerations could lead to a location at the centre of the port, where there is less space available to expand (along the coast to the northeast or southwest), and because there, still considerable depth is present by nature.

Container Terminal

The container terminal is the item of the port that demands the most (storage- and quay-) space, and accommodates the lion's share of the shipping traffic. So enough (storage) space, berths and water depth are required, and it can at best be located at a flexible location (that is suitable for expansion in the future to either one of the directions). Because of the large land requirements, it is convenient to locate the terminal at the most smoothened part of the coast, around the middle of the sandy beach that is. Here, there is also some space available for expansion land inwards.

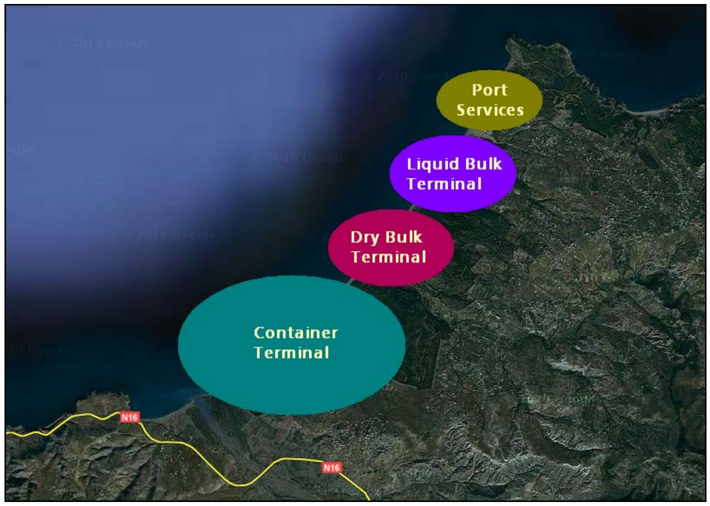
Also, here the bottom seawards to the coastline had the least steep slope. It is therefore the most convenient location to locate terminals with accompanying ships with the smallest draught (with at maximum D=15.5 m). These are the container vessels, as can be seen in annex 3.1. This in order to reduce the total amount of dredging and land reclamation required. This way, the cut & fill principle can be satisfied.

Next, a choice can be made to construct the berths and basins shore parallel or shore perpendicular. Both options will have to be investigated in the layout development. In a first glance, there is little space at the project location to situate all the berths of the container terminal. This (most of the time) results in an orientation with several basins perpendicular to the shore. In these basins several container ships can moor, and they meet the requirements of basins width given earlier. A desirable side effect with this is that the breakwater length can be shorter by the shore perpendicular orientation of berths and basins.

The above described siting considerations are summarized in the next figure.

Figure 4.8

Indicative preferred terminal allocation along the coast



4.3.3 CONSTRUCTION SEQUENCE

Looking at the time horizon for the construction of the port (and its size), it is impossible to construct the port in only one phase. In 2.2.3, a goal is determined for two different phases. This means that the port has to be designed for the (at least) required quantities at the end of phase I and II. Possibly, within these designs room has to be available for expansion (in a controlled manner).

Depending on the urge, specific terminals can be constructed (for phase I) within a short time horizon (5-10 years) to a medium-term horizon. The ultimate state of the port (phase II) is on the long time horizon (20 years, in which phase II includes phase I). This means that, Phase I comprises the completion of the three terminals for phase I capacity. The order (of construction) of these different terminals can also be done in different phases, as can the berths. For phase II this is the same. The final phase is nevertheless the same: the total required capacity for the final port.

A requirement specified by the client is that the bulk terminals and the container terminal can be developed independently from each other. It is at this stage not known which terminals will eventually be realised (and if the container terminal will be constructed at all), or which commodity is requested first, so the flexible design should include possibilities for the independent construction of the bulk port and the container port. This poses special constraints on the port layout: the bulk terminals and the container terminal are more or less different individual items of the port, but the location of the approach channel and the turning basin is the still same for all the terminals, and should therefore be included in both.

The abovementioned requirements will have their noticeable effect on the different layouts that will be drawn up later on.

4.3.4 LIQUID BULK TERMINAL

First of all, the area requirements for the oil terminal will be determined. As outlined before in 4.3.2, because of reasons of safety the loading and unloading of the vessels will be done (if possible) in a secluded basin in-port.

This should be located around Punta Negri as outlined earlier. Within this basin, several berths will have to be provided, depending on the throughput.

The landward part of the liquid bulk facilities consist mainly of storage areas. The size of the storage area depends on the number and dimensions of the storage tanks, which on its turn depend on the size of the vessels, the inter arrival times, the diversity of the products and the required available reserve. Besides this, the total surface area for the liquid bulk terminal is also determined by the construction of the refinery (phase II). For this final phase, the client requests that there should be 200-300 ha. land available [ALKYON DATA].

Specifications

To arrive at the resulting specifications, the different items will be explained below with their specific considerations.

Number of berths

For the required number of berths, several assumptions have been made. First of all [LIGTERINGEN, 2007] states that crude oil tankers smaller than 200.000 DWT can load or unload with net hourly capacities equal to roughly 10% of their deadweight tonnage. Consequently, these vessels occupy the port facilities for a short period of time only (10 hours for unloading, some time for berthing and de-berthing, so at maximum 1 day).

Here, it is expected that oil tankers arrive fully loaded and also unload this whole quantity. For the loading, the same applies: vessels arrive empty and leave the port fully loaded.

Wit the abovementioned assumptions, the required number of berths can be determined from analysing the amount of shipping traffic per day for the different phases, which already has been calculated in 3.5.2.

Length of waterfront

For safety reasons, the space between ships berthed in line should be approximately equal to the width of the biggest ship, with an added 2x15 meters extra to include the fact that the manifold is not always placed exactly amidships. The minimum centre-to-centre distance of 2 adjacent berths will be [LIGTERINGEN, 2007]:

$L_B = L_{S;MAX} + B_{max} + 2*15$ m.

In phase I, the largest ship is the 150.000 DWT oil product tanker. Here, L_B = 376 m.
This is also the largest vessel in phase II that will transport incoming oil products. Because different ships arrive in a more or less random order, it is advised apply this centre-to-centre distance for all the berths. This, in order to also accommodate the larger vessels at all times. It also leaves some reserve in the distribution of arriving ship sizes, and the increase of L_B compared to the second largest ship is not that large.

In phase II, the largest ship is the 200.000 DWT crude oil tanker. Here, L_B = 410 m.
This length should exclusively be available for the crude oil berth. Apart from the possibility of vessels berthing in line, it is also possible to berth at a (finger or T/L-) jetty.

Storage and surface areas

The storage area requirements depend on the operational and strategic storage requirements. When taking into account an operational storage in the order of 1 month (5% of total throughput). These storage requirements in tons can be translated into area requirements. The total required area follows from the necessity that the contents of the tanks in case of an accident (breaking of the tanks) could be maintained within a bund [LIGTERINGEN, 2007]. Because at this stage tank dimensions are unknown, for a first approximation the area has been calculated for the (extreme) situation that occurs when all tanks fail (e.g. because failure of one tank could lead to failure of the next one).

Miscellaneous

Besides storage of the oil (products), the terminal area will comprise several other buildings, pipelines, parking spaces, etc. Also, the unevenness of the terrain needs to be taken into account, while it has been outlined that the oil terminal will be situated around Punta Negri. This would mean a construction of the tank farm in cascades. For this, an overall safety factor of 1.4 [ALKYON DATA] has been taken into account.

This direct need for space is exclusive of the safety zone because of the hazardous nature of the products, which must be kept free of uncontrolled sources of ignition, and the distance to other objects and terminals. So it is recommended to locate the tank farm for the oil terminal at sufficient distance from any other facilities (more inland) to include this.

In the earlier mentioned 200-300 ha. for the terminal area in phase II, a refinery is included. With this refinery there will be more outgoing oil products, refined from the incoming crude oil.

The resulting specifications from the above outlined assumptions are summarized in the table below. For the detailed calculations per specific item, one is referred to annex 3.5.

Table 4.7

Specifications liquid bulk terminal

Specifications	Phase I -	Phase I +	Phase II -	Phase II +
Throughput	15.000.000 T	20.000.000 T	40.000.000 T	40.000.000 T
# of berths	2	3	5*	5*
L_B [m]	376	376	410	410
A_{LB} [ha]	35	48	200-300**	200-300**

* of which is 1 crude oil berth

** requirement from client, refinery is included

In which:

L_B= length of waterfront required per berth

A_{LB}= area requirement total liquid bulk terminal

Additional Aspects

Safety

Safety is the most important design consideration for a liquid bulk terminal because of the earlier mentioned hazardous (polluting and inflammable) nature of the products handled. For this, it is advised to locate the berths in a separate basin. This basin should preferably be fugitive [LIGTERINGEN, 2007], so the ship can stay at berth under all weather conditions.

If possible, this separate basin can also be closed off by floating booms, which prevent the liquid bulk from spreading further in the port into the direction of the wind.

The possibility of spills must be reduced to the utmost minimum. However, relatively small evens like rupture of pipes or hoses, failure of valves or flanges will occur occasionally [LIGTERINGEN, 2007]. Safety regulations should be made and measure should be taken in these cases by the terminal operators in order to adequately handle the situation. In order to prevent major accidents from occurring, the best (and only) defence is to take such precautions, both in planning, design and in operational procedures, as to bring the probability of occurrence at an extremely low level.

Terrain
The terrain around Punta Negri consists of rock and is rather rough and uneven. Inland of the beach, a natural slope of around 1:10 is present. This could lead to problems when placing the tank farm. An overall levelling of the terrain seems a very expensive solution, so an alternative needs to be found. This could be e.g. placing of the tank farm in cascades, against the sloping terrain.

Besides this, in phase II a refinery will be constructed in order to acquire more outgoing oil products retrieved from the refinery. The area requirement for this has been determined by the client, but also this refinery should be placed on the sloping terrain. This would mean additional costs for levelling of the terrain.

Seawards, in phase II a crude oil berth should be realised. For crude oil, although also inflammable and polluting, somewhat looser safety requirements than for oil products exist. So placement of the oil berth e.g. at the inside of a breakwater could also be an option. This will be taken into account when determining different layouts for the liquid bulk terminal.

4.3.5 DRY BULK TERMINAL

The project information for the dry bulk terminal states that there should be a quay wall of 700 m in length and a throughput of 4-5 MT/year (see 2.2.3). Regarding the area requirements, these depend on the handled commodity and the handling equipment. [UNCTAD, 1985b] states that the availability of land for the stockpile is limited by the natural conditions and the cost of the acquisition. While these conditions play a role at every project location, it is therefore necessary that the stockpile will be planned in such a matter that a maximum amount of material can be stored in a minimum area.

Specifications
To arrive at the resulting specifications, again several assumptions have been made. This considers:

Number of berths
From the shipping traffic in annex 3.5.2 it is clear that the traffic is not that large: on average only 101 vessels/year. The direction of the materials at the dry bulk terminal per berth is mainly one way traffic [UNCTAD, 1985b], [LIGTERINGEN, 2007].

The dry bulk terminal requires different handling equipment for loading and unloading, so it is recommended to construct (at least) two berths which serve respectively incoming and outgoing vessels. This principle will also be applied here, to ensure efficient loading and unloading of the vessels. This is also in line with the expected shipping traffic and advised occupancy [UNCTAD, 1985b].

Length of quay wall

The length of the quay wall is determined by the client and amounts to (at minimum) 700 m. This way, it is possible to construct two berths which can even accommodate two of the largest vessels (the 150.000 DWT bulk vessel), according to the formula:

L_q=1.1*n*($L_{s;average}$+15)+15 = 700 m., which is known.

This means when n=2, the $L_{S,\ average}$ can be 296 m. This is larger than the largest vessel, which means that two of the largest vessels can be accommodated at the same time at the quay, although this has a relatively small chance of occurrence.

Surface area requirements

The surface area (storage) actually depends on the commodity handled. A heavy material places special constraints on the soil conditions, while a light material places special constraints on the amount of surface area required. Here, it is assumed that around (a rather large value of) 10% of the total throughput can be stored on site. This is because there is further no information available for the commodity and this includes a certain amount of flexibility. A light material would require the largest amount of storage space, and can be considered as the design situation.

For the dry bulk terminal, an inland depth of 250-300 m. for storage of dry bulk has been adopted. Taking into account apron areas (apron depth Y=85 m [LIGTERINGEN, 2007], required over the entire quay of 700 m in length) and a factor of 1.2 for additional space requirements, the final surface area for the bulk terminal can be calculated. The detailed calculations have been added in annex 3.6.

The table below summarizes the resulting specifications for the dry bulk terminal.

Table 4.8

Specifications dry bulk terminal

# of berths	Phase I -	Phase I +	Phase II -	Phase II +
Throughput	4.000.000 T	5.000.000 T	4.000.000 T	5.000.000 T
# of berths	2	2	2	2
L_B [m]	700*	700*	700*	700*
A_{DB} [ha]	**29**	**33**	**29**	**33**

* requirement from client.

Additional Aspects

Because it is not known what dry bulk arrives at the terminal, it is also not known which storage facilities should be necessary, e.g. dry or open storage, etc. Also the handling equipment (vertical as well as horizontal) are still undetermined. The volume of material which can be stored in a given area will depend not only on the bearing capacity of the ground and the characteristics of the material, but also on the outreach and height of the stackers and reclaimers [UNCTAD, 1985b]. It is assumed that proper handling equipment will be provided.

As outlined before, a berth for the dry bulk terminal is mainly one way. Loading of the vessels virtually always happens in a continuous process, where the movable ship loaders are fed by conveyor belts which discharge into the ship under influence of gravity [UNCTAD, 1985b], [LIGTERINGEN, 2007]. Unloading happens in most of the cases with shore-based cranes on a rail track, which requires considerable surface area.

4.3.6 CONTAINER TERMINAL

The amount of land available for the container terminal depends on several design considerations mentioned previously. When the required surface area for the dry and liquid bulk terminals (and their placement) have been determined, a certain (large) area remains around the sandy beach to the southwest. Here, a throughput of as many containers as possible will have to be realised.

The container terminal accommodates the largest amount of shipping traffic and requires the most surface area. This surface area requirement is to large extent determined by the storage space required for the containers. This – on its turn – depends to a certain extent on the handling systems chosen. They are all included in order to arrive at the specifications mentioned below.

Specifications

The methodology and determination of the required areas are done according to [UNCTAD, 1985b], [LIGTERINGEN, 2007]. Below, the most important items with their considerations are outlined.

Number of berths

The determined number of berths results from the amount of shipping traffic in combination with several assumptions. For example as mentioned before, because the arriving vessels are unloaded and subsequently loaded (with the same amount of) cargo, there is less storage space required and only half of the shipping traffic needs to be taken into account (as the average call size is doubled by this assumption). Besides this, assumptions have been done for instance regarding the TEU-factor (1.5), the number of portainer cranes per vessel (3-5), the berth occupancy (maximum m_i=0.5) and the maximum number of TEU per berth per year (600.000 TEU/y). For the detailed considerations and calculations, reference is made to annex 3.7.

Length of quay walls

The quay length that must be available to accommodate the required number of berths differs also in every phase (because of the differences in berths). From a study of UNCTAD [LIGTERINGEN, 2007] it is clear that with an average berth length of 110% of the average berth length + berthing gap, no additional waiting times will occur. Because of this, the quay length is calculated (with a berthing gap of 15m.):

$$L_q=1.1*n*(L_{s;average}+15)+15$$

The average ship length $L_{s;average}$ is calculated as follows:
$L_{s;average}$=0.25*225+0.42*294+0.25*310+0.08*398=290 m..

Substituting this and the results of 3.5.1 in the formula above, the required quay length for every phase becomes:

Table 4.9

Required quay length container terminal

	Phase I -	Phase I +	Phase II -	Phase II +
# of berths	7	10	25	50
Quay length L_q	2.400 m	3.400 m	8.500 m	16.800 m

Handling methods and surface areas

To determine the surface areas, first of all the method of handling will have to be chosen. Because at the project location land is scarce (and always expensive), the stacking of containers as high as physical conditions and commercial requirements allow becomes a necessity. While the subsoil conditions are not (yet fully) taken into account, it is assumed that this high stacking height is feasible, and otherwise soil improvements will be applied to meet the requirements.

While the new modern hub port will accommodate a modern transshipment container terminal, it is assumed that besides the portainer cranes which unload the containers, Automated Guided Vehicles (AGV's) are used to transport the containers to the storage yard. Within the storage yard, gantry cranes will be used to stack the incoming containers. The advantages for the gantry cranes are a good space utilisation, low maintenance and a high productivity. Automated stacking cranes are also an option, but require high investment and maintenance costs. Gantry cranes usually [UNCTAD, 1985b] stack containers 4 high (because of reasons of inefficient movements and additional repositioning when stacked higher) and can lift 1 over 4 containers.

With the chosen handling equipment, the surface area requirements can be determined. For this, the following planning elements have to be determined and quantified:

- apron area
- storage area's (container yard, container freight station (CFS))
- container transfer area (to truck and rail)
- buildings (offices, gate and workshops)

This has been determined in detail in annex 3.7. The resulting final specifications for the container terminal are presented in the table below.

Table 4.10

Specifications container terminal

Specifications	Phase I -	Phase I +	Phase II -	Phase II +
Throughput	4.000.000 TEU	6.000.000 TEU	15.000.000 TEU	30.000.000 TEU
# of berths	7	10	25	50
Quay length L_q	2.400 m	3.400 m	8.500 m	16.800 m
A_{CT} [ha]	130	192	478	951

The throughput – net terminal (storage + CFS + apron) area ratio is around 31.000-32.000 TEU/ha, which is considered a very good value compared to other major ports [LIGTERINGEN, 2007]. It should be noted that the above-mentioned surface area for the container terminal is also exclusive the gate area. When taking into account the whole terminal surface area, this ratio is roughly halved.

Additional Aspects

From the above it is obvious that the container terminal has a large surface area requirement. This is even enlarged by the criterion from the client that the possibility must exist that 10-20% of the throughput can be transported to the hinterland by road and rail (which has been included in and additional factor for the surface area, see annex 3.7).
For this, a separate terrain will be arranged to accommodate equipment for transfer of containers to road and rail.

Because of this, the construction of (new) roads and railway tracks are a necessity. At the project location, there are no larger roads present which could accommodate trucks with their trailers. When reviewing the area and its infrastructure, it is clear that a larger road – the N16 – is already present to the south of the project location. This is a larger two-way road, which should be able to receive vehicles from two directions. So it seems convenient to connect the new roads to this already existing network.

Around the project location, there is a railway present which runs from Nador to the south. In order to connect to this network, a railway track should be constructed from the project site to Nador. This would lead through a rocky, uneven terrain from the project location to Nador, with accompanying high costs.

A rough approximation of the above mentioned infrastructure will be made after determination of the different layouts.

4.4 MISCELLANEOUS FACILITIES

In this paragraph, remaining port facilities that require additional attention are identified and elaborated in more detail. This results in additional area requirements for the new transshipment port.

4.4.1 PORT SERVICE FACILITIES

A complete port development plan must include provision for many facilities which are ancillary to the main port operations of transshipping and storing cargo. These ancillary port services pose additional surface area requirements for the port.

The ancillary services range from fire-fighting and rescue services to document-handling and data-processing systems. The services can be broadly divided into four groups: fresh water, bunkering and lubricating oils, ship repairs, and provision [UNCTAD, 1985b].

The required surface area for these miscellaneous facilities differs from port to port, because every individual port is in need of different services. Nevertheless, a rough approximation for the required surface area is necessary in order to incorporate this in the design. It has been determined [WEBSITE PORT OF ROTTERDAM] [GOOGLE MAPS] that for this area around 7-8 ha. of land should be available.

4.4.2 TUGBOATS BASIN

Port planning also requires the provision of adequate accommodation for service craft, under which the tugboats.

Tugboats are used for towing, docking, undocking and shifting vessels. This service craft has a limited draught, and generally poses no problem in the selection of a location. However, they are most sensitive to the wave distribution and sufficient calm water berthing space and suitable docking facilities must be made available. This often comprises an inner harbour – better sheltered – where the actual berths are located [LIGTERINGEN, 2007].

The tugboats can be berthed in many possible ways [LIGTERINGEN, 2007]. The most convenient solution is the two sided mooring at finger piers. This reduces the required quay length because both sides of the pier can be used for berthing. Regarding the dimensions of the finger piers, for the design situation it is advised to look at phase II+. This, because the tugboat basin will be situated in-port, and this is in a later stage difficult to expand (although the area requirement is not that large).

Besides this, for determination of the dimensions of the piers, only the largest tugboat is taken into account. This is because tugs arrive and leave at random intervals and there should always be enough space for berthing of both the tug (sizes). Determination of the required area for the tugboat basin has been done according [LIGTERINGEN, 2007], and is described below.

- The length of the slip, L_S, in most standards equals the largest length of boat that can by regulation be berthed in the slip. Here, this amounts to L_S=L_B=35 m.
- The width of the slip is determined by adding a triple clearance to W_B in case of a double berthing slip. This would result in a slip width of around (3*0.7+11.2*2=) 25 m. where two tugboats can berth.

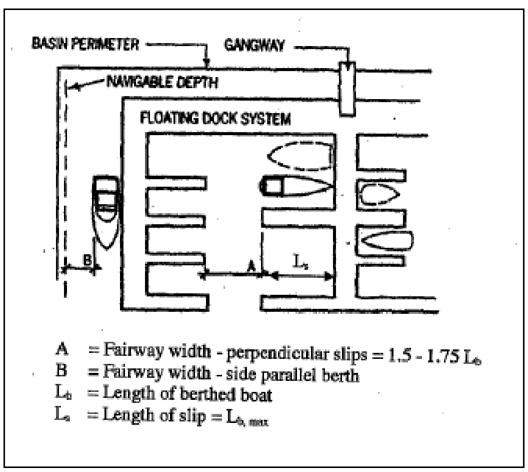


Figure 4.9
Guidelines for determination of wet surface areas for marinas

- The fairway (the water area between the slips), A, has a minimum width of 1.5*L_B, with 1.75*L_B preferred. Although the tugboats have excellent manoeuvring characteristics, the preferred value is used here: A=1.75*35=20 m.
- The length of the walkways is standardised by the manufacturers of these systems: for a walkway up to 200 m. a width of 1.8 m. is standard.
- The finger piers have a minimum width of 0.6 m., increasing to 1.5 m. for $L_B \geq 15$ m. This last criterion applies here.

The required number of tugboats per phase according to annex 3.3 is:

Table 4.11

Number of tugboats in each phase

Tugboats	Phase I -	Phase I +	Phase II -	Phase II +
# of tugboats	8	9	13	18

Because the eventually required number of berths should already (in terms of space) be available in the first phase, a surface requirement for the tugboats basin can be calculated when considering phase II+. This is, when considering two piers, each with one walkway with 3 double slips per side, with clearance A along both sides and in between.
This yields an area requirement for the tugboat basin of:

A_{TB}=L*W=(3*25+4*1.5)*[2*2*(35+0.9)+3*20] =16.492 m^2, which is around 1.6 ha.

It is clear that this surface requirement is almost negligible compared to the large terminals.

4.4.3 INFRASTRUCTURE

Setting up proper infrastructure for the new port is indispensible. The roads and railways should be connected to the already present infrastructure network, as described earlier. It is requested by the client that the possibility exists to transport 10-20% of the containers to the hinterland by road and rail. This could (especially in phase II) lead to heavy trucking traffic on the roads. Additionally, the new port will generate an extra amount of traffic on the roads because of the employment the new port will provide. The location of the new infrastructure will be outlined later on in the final port masterplan.

4.5 ALTERNATIVE LAYOUTS

In the previous paragraphs first of all the wet surface area requirements have been determined, posed by the ships and their manoeuvrability and the environmental conditions. Subsequently the dry surface area requirements for the different terminals and port facilities have been outlined. With these determined design parameters and following the earlier defined guidelines for allocation of the terminals (paragraph 4.3.2), various layouts for the port will now be realized. This creative part results in almost endless possibilities of combinations of quays, basins and their shape.

Because every item can – in principle – be varied, the necessity exits to do this within certain bounds. The methodology is as follows:

- First of all, differences in port approach (the channel) will be assessed, while taking into account wind and wave directions. When varying this item, it will become evident that these choices will have their own specific effect on the port layout (e.g. the terminal allocation along the coast) that results from this. Again, mainly the guidelines from paragraph 4.3.2 are used, in order to get an efficient layout. To ease the decision making process, the (maximum possible) throughputs for the different alternatives are not yet taken into account, but the possibilities for expansion are. The throughput for the terminals is in this first assessment for all the alternatives more or less the same.
  A Multi Criteria Analysis (MCA) will be made to select the most promising global layout in this first round.

- Secondly, now that the layout with the most promising entrance and approach channel has been selected, in this alternative other parameters will be varied. For instance, the number of berths and their locations, the breakwater construction depth, safety, the in-phase construction and port efficiency. This again leads to a number of possible port layouts. These 2nd round alternatives will be again compared to each other by means of a second MCA. After comparison, the final port masterplan will be selected.

Legend

In the next paragraphs, the different alternatives will be presented. A brief description is given for each alternative, with their specific properties. Significant details will be mentioned, and the most important benefits and drawbacks.

Every drawing has several elements in common. The approach channel is coloured grey, and ends in a turning basin. Circles with the largest diameter represent a turning basin for the largest ship (the 14.000 TEU container vessel), whereas turning basins with the smaller diameter can be used by the (liquid and dry) bulk vessels, or located at the container terminal by all vessels except the largest ones. The solid dark grey lines enclosing the port(s) represent the breakwater(s). Southwards, this breakwater is often lengthened but not drawn solid. This represents the (independently) development of the container terminals when advancing from phase I to phase II.

In every layout, the different terminals are marked with numbers. At the seaward side, the number 1 represents the liquid bulk terminal, and at the landward side this is the required surface area for the tank farm (again the solid dark grey area shows the required surface area in phase I, enclosed in the surface area for phase II). The storage farm for the liquid bulk terminal is always placed on land, to create a safety distance to other port activities. Besides this, here land area is available and avoids the need to reclaim even more land. Number 2 represents the dry bulk terminal. For the container terminal, number 3, a division is made between phase I and II. Here, 3^{I} represents the development of the container terminal in phase I, and 3^{II} in phase II. Finally, the number 4 represents the surface area required for the port services. Often, the tugboats basin is located near. The arrows along the coast show the location of the wadis where they discharge in the Mediterranean Sea.
The dashed line around Punta Negri indicates the approximate segregation of the easy dredgeable beach sand (to the left) and the hard rock (to the right).

As a concluding remark, it should be noted that the drawn ships in the layouts indicate <u>possible</u> berths. They indicate the maximum number of possible locations, but it is stressed that they are not always occupied at the same time (this, in order to meet the occupancy requirements calculated earlier). The amount of shipping traffic in phase II^{+} is at maximum 26 ships per day, but the total amount of berths will be higher. This also takes into account the service time of the vessels, which are at the berth often more than 1 day. The average parcel size of 3.657 TEU however can be handled within a day [LIGTERINGEN, 2007].
The number of berths is however used for calculation of the maximum (possible) amount of throughput.

4.5.1 ALTERNATIVES ROUND 1

Each alternative has its specific properties and therefore its own advantages and disadvantages. In order to select the most promising layout, the various alternatives have to be compared to each other in an objective, quantitative matter. This will be done by means of a Multi Criteria Analysis (MCA). The MCA is a tool developed for multi criteria problems within decision making [WIKIPEDIA].

Criteria

In order to use this tool right, first of all the relevant criteria have to be identified [UNCTAD, 1985b], [LIGTERINGEN, 2007], [MCA'S ARCADIS]. For Round 1, these are:

- Approach Channel Alignment

The approach channel is one key parameter that varies in the different alternatives. These different possibilities have to be judged on their orientation with respect to the environmental forces (wind and waves), and on the possibilities of a favourable location which for instance minimizes dredging of hard soils.

- Nautical Ease

Regarding nautical ease, again attention is paid to the environmental forces, as they influence the vessel's manoeuvring capabilities. When ships have to sail against the dominant wave direction when entering the port, this is considered more desirable. Besides this, the location of the entrance is assessed, when reviewing the shipping traffic that approaches the port. Also, the available wet space and possibilities of turning and manoeuvring will be taken into account.

- Port Zoning & Efficiency

Here, the (possible) global zoning of the port for the different terminals is evaluated. It has to be judged whether the layout is suitable for efficient handling of the cargo and port traffic. This is in order to enhance quick and efficient handling of the berthing vessels arriving at the terminals. Also it has to be assessed whether the allocation (possibilities) of the terminals is (are) convenient. One has to look also at the location of the terminals, whether they are on a logically chosen location (e.g. when considering draught or cargo hazard) or not.

- Wave Penetration & Sedimentation

The layouts all have their own unique orientation regarding the port entrance, with respect to the (dominant) wind-, wave-, and longshore sediment transport directions. The expected in-port entering of any of these should be avoided as they are considered unfavourable.

Assumptions

As mentioned before, in the following alternatives the throughput is kept constant for all the alternatives. For all the alternatives, the approach channel and turning basin have been situated in such a manner that this constant throughput can be reached. The cut & fill-balance is satisfied as much as possible by placing the terminals inland until the practical constructional boundaries (around the CD +10-+15 m. line). Also, the terminal layouts are more or less the same in all the alternatives in order to focus on the (other) important criteria for round 1 to facilitate the decision making. This already has consequences for the terminal allocation and the port efficiency.

The area around Punta Betoya has in all alternatives been avoided on purpose for the construction of the port (and its breakwaters). This is because of the location of the tectonic fault (Taliwine), and the outflow of the Rio Kert. As explained earlier, this wadi should because of its high discharge also be able to discharge into the Mediterranean Sea.

Alternatives

Below the alternatives are presented and described. In this description, the alignment of the approach channel is explained, together with its influence on the port layout. Attention is paid to a favourable location of the approach channel and port entrance. Dredging of the hard soil (bedrock) around Punta Negri is avoided on purpose as much as possible. Besides this, the terminals are all located in such a manner that the wadis outflow can take place with as less hindrance as possible.

Alternative A1

Alternative A1 is presented in the figure on the next page.

- Description

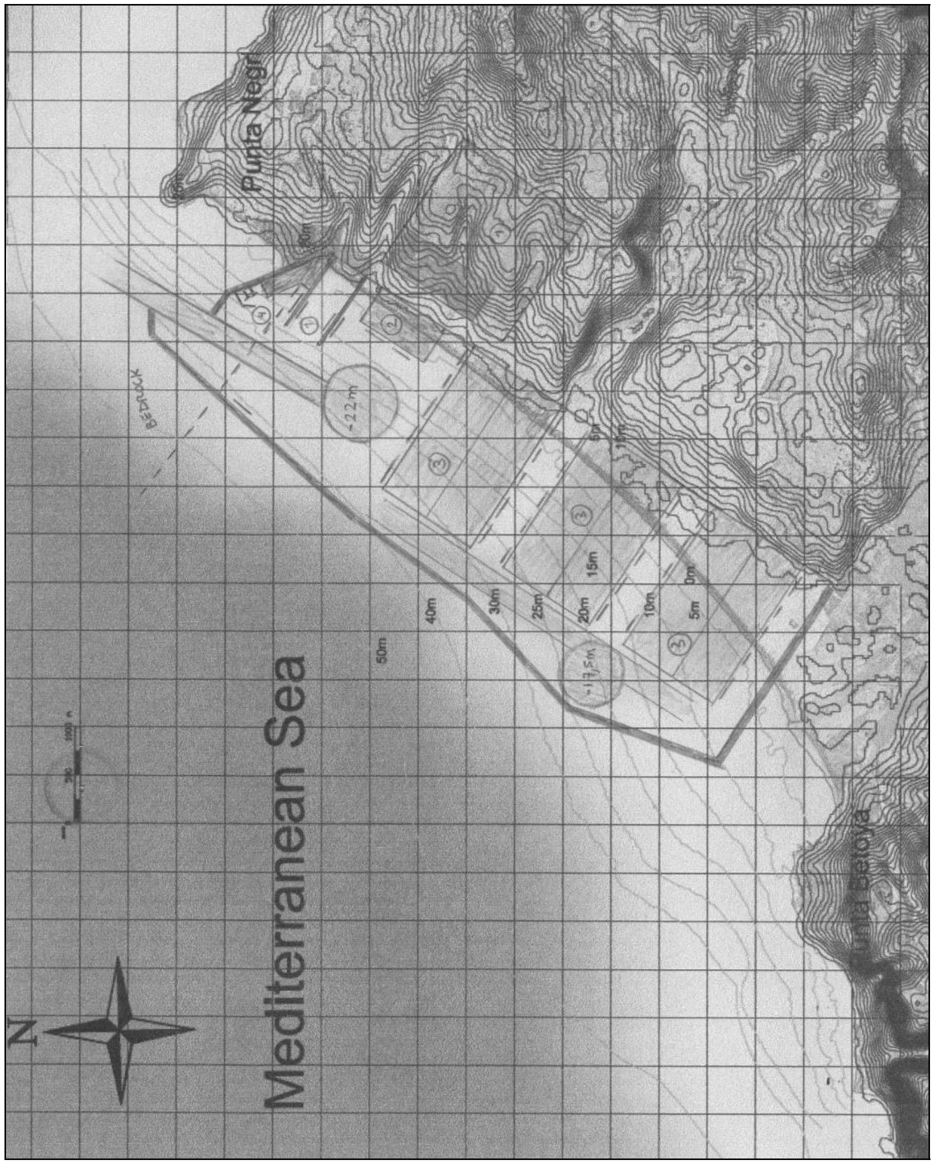
In this alternative, the approach channel is oriented at 210°N. This leads to only one possible layout with the port entrance to the northeast. With this decision of an entrance close to the Punta Negri headland, environmental influences on sailing vessels are more noticeable (e.g. because waves from different directions). However, there is no loss of 'port space' inside of the breakwaters because of the fact that the required stopping length can be used by berths located at the shore side (1), (2), (4).

Also, dredging of the hard soil around Punta Negri is avoided: the approach channel is located off shore at the CD -22 m. depth contour. Nevertheless, dredging of the turning basin is still necessary, but the soil there mainly exists of sand. Also dredging is required at the bulk berths and (mainly) at the south-western part of container terminal basins.
The channel for the container vessels is located northwards of the -17,4 m. depth contour to minimized the amount of dredging required. An (optional) extra turning circle is provided near the container terminal to provide extra manoeuvring space for the container vessels.

The terminals are located along the shore according to the guidelines from 4.3.2, which is considered favourable. The layout of the breakwaters prevents waves from the dominant direction (W and WNW) from entering the port. Also, with this layout the expected sediment transport (from SW to NE) in-port will be minimal.

Figure 4.10

Alternative A1



Alternative A2

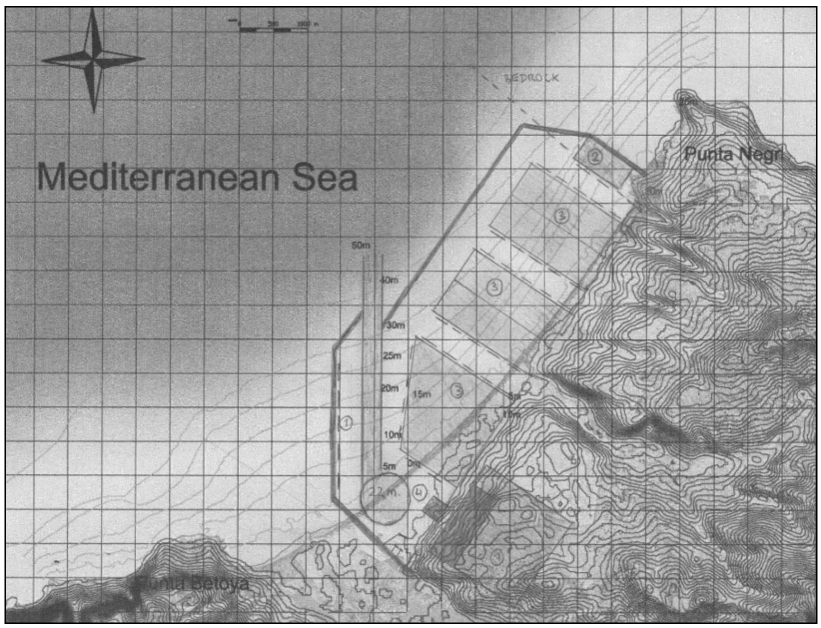
Alternative A2 is presented in the figure on the next page.

- Description

The approach channel is in this alternative situated at 180°N. With this orientation, there are several possible locations to locate the port entrance. Roughly, there are three possibilities: the port entrance to the northeast (Punta Negri), the entrance at the middle of the sandy beach, or the port entrance to the southwest. When locating the port entrance at one of these last two locations, this would only lead to more loss of space required within the breakwaters, because of the required stopping length and the inconvenient location of the turning basin. This reduces the maximum throughput of the container terminal, and gives rise to less efficient port zoning and requires towing of vessels over larger distances (see below).

Figure 4.11

180°N approach channel, located to the southwest

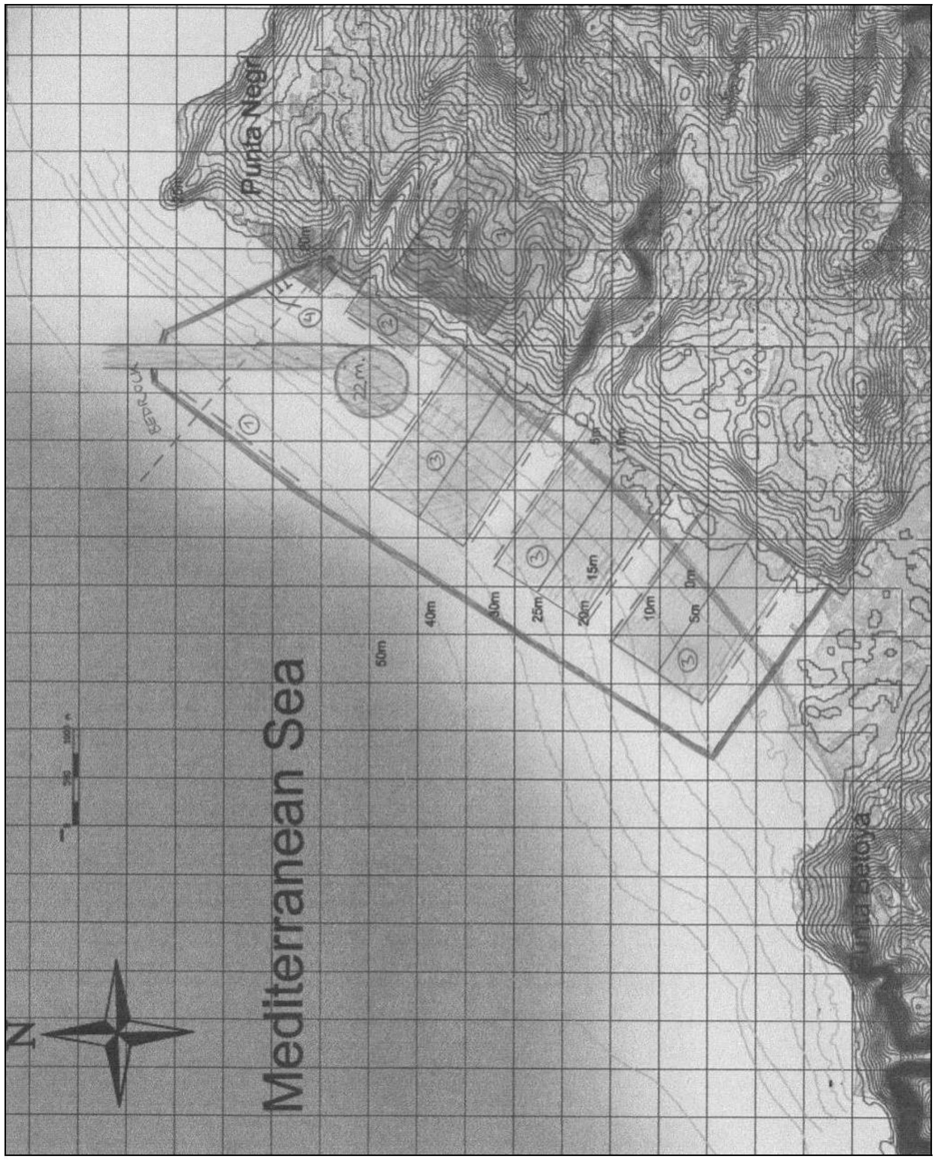


In the alternative above, to avoid towing of the liquid bulk vessels over a large distance, the liquid bulk berths are located close to the entrance, as advised in 4.3.2. However, they are now not upwind of other terminal activities anymore, and not in a secluded basin. Besides this, the tank farm (which should be located near) lies close to the fault zone around Punta Betoya which is considered less safe.

It is concluded that all locations of the 180°N channel other than NE are considered less favourable, so this last location has been used in alternative A2 (see below). Dredging of the hard rock to the northeast is minimized, but more dredging is required at the location of the turning basin (because of the required stopping length behind the breakwaters).
Besides this, the breakwaters are longer and located even deeper because of this required stopping length.

Figure 4.12

Alternative A2



The liquid bulk berths cannot be located at their former position (see A1), so they are located at the inside of the northern breakwater. Also this breakwater layout prevents waves from the dominant direction of entering in-port, but the approach channel is situated perpendicular to this direction. For the vessels visiting the port, rolling could become critical. It is expected that sediment transport in-port will be minimal.

Alternative A3

Alternative A3 is presented in the figure on the next page.

- Description

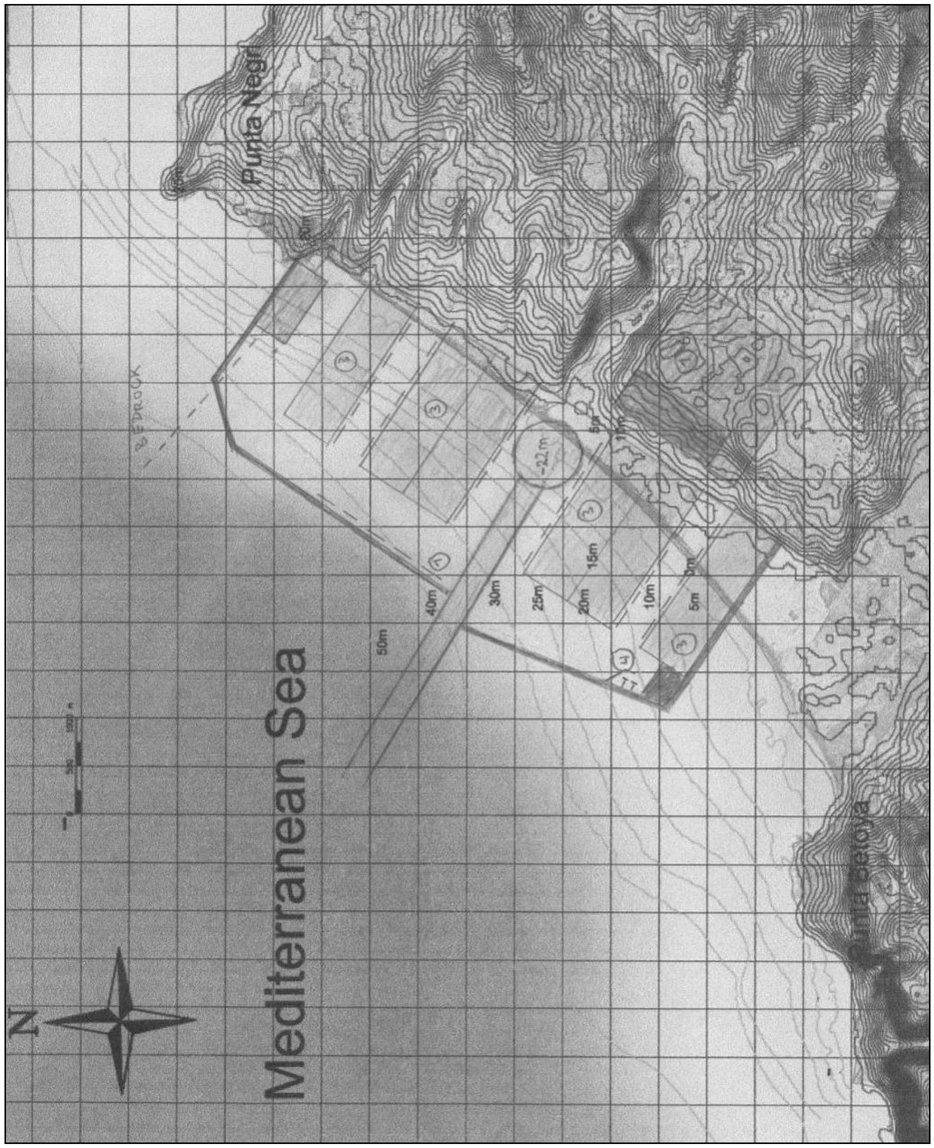
In this alternative, the approach channel is oriented under an angle of 120°N. Because of this orientation, waves from the dominant direction (W) make a small angel with the channel, but still waves from WNW do not. There are waves following the vessels in the approach channel. Also here, other positions along the coast can be chosen for the location of the approach channel. However, the required stopping length which should be available can become critical (e.g. when moving northwards along the coast) because of the perpendicular-to-shore approach.

Because of this, in alternative A3 the approach channel is located in the middle of the port, ending in the turning basin. This way, the breakwaters do not have to be constructed at depths larger than CD -45 m. (in contrast to the situation where the approach channel is located more northwards). Nevertheless, also this location of the approach channel results in a loss of in-port space at the sandy beach and decreases the maximum container terminal throughput. The liquid bulk berths need to be located again in another position than advised, which results in less favourable port zoning.

Also here, much dredging of the approach channel and the turning basin is required. Furthermore, at first glance the cut and fill balance is not completely satisfied (more fill required). The terminal allocation is not an optimal one, and the alternative has no particularly favourable influence on the construction depth and length of the breakwater. It is expected that somewhat more wave penetration into the port will occur, but sediment transport into the port will again be low.

Figure 4.13

Alternative A3



Alternative A4

Alternative A4 is presented in the figure on the next page.

- Description

In this alternative, the approach channel is oriented under an angle of 75°N. Because of this orientation, vessels need to take a large bend (to and from the east) when arriving or departing from the port. Also waves from a semi-dominant direction (WNW) are directed perpendicular to the channel, which is unfavourable for navigation. More than in any of the other alternatives, locating the 75°N channel more northwards would lead to a loss of in-port space for container terminal construction (or results in a large breakwater construction length). It is because of this that the entrance to the port is located to the southwest.

Dredging remains somewhat limited in this alternative around the northern bulk and container terminal berths, and in the approach channel. When the terminal allocation guidelines from 4.3.2 are taken into account, this again means towing of bulk vessels over large distances, and the liquid bulk berths are not located close to the port entrance.
Also in this alternative, at first glance the cut and fill balance seems a little off.

Besides this, with this orientation of the approach channel and port entrance, wave penetration and sediment transport (from the Rio Kert) inside the port could become a problem: a large entrance width is required for vessels to enter the port under this angle and at the same time avoiding sediment (which is directed from the southwest to the northeast) from entering the port.

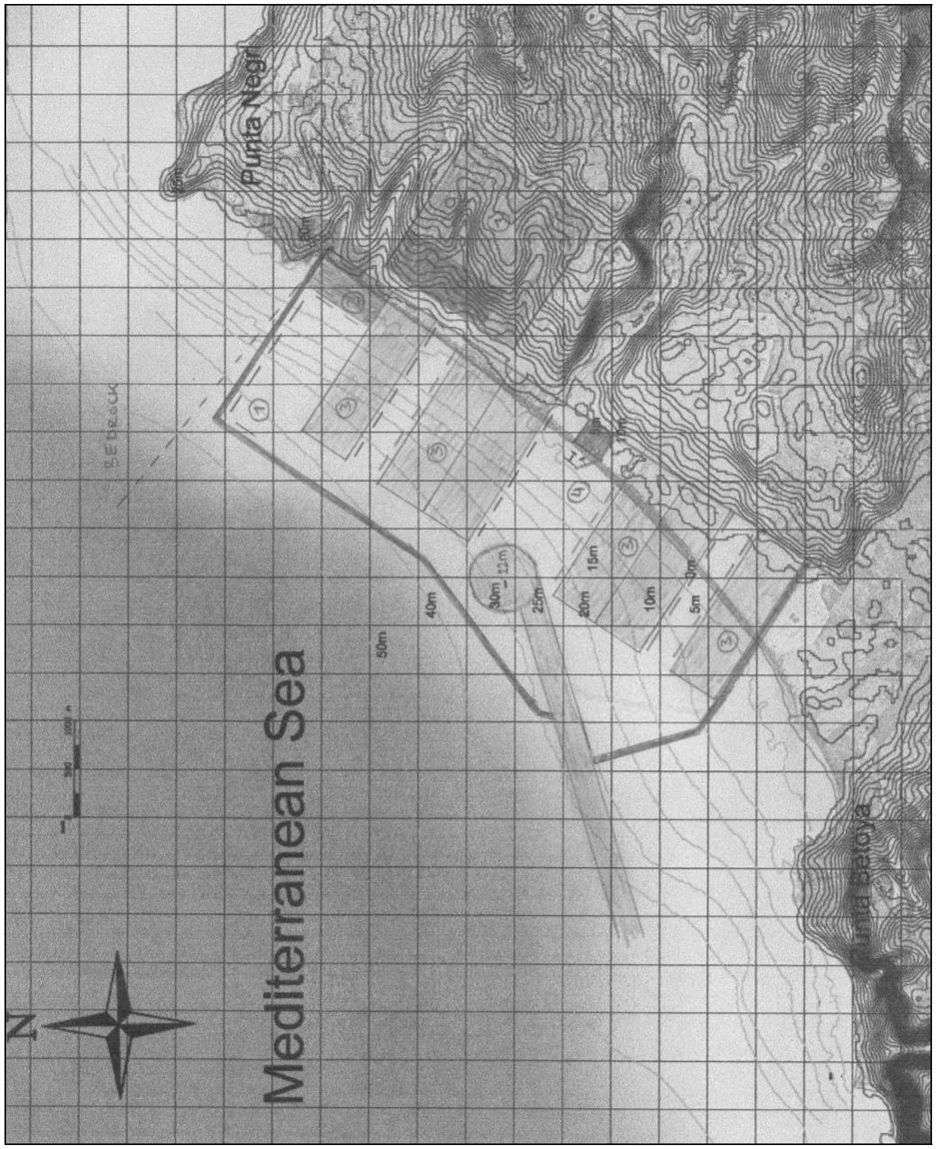


Figure 4.14

Alternative A4

Multi Criteria Analysis

The above presented alternatives will be compared to each other to select the most promising one. The results of the first round MCA are presented in the table below.

It should be noted here that the various alternatives are compared with each other.
A minus score does not necessarily mean that the specific item or alternative is bad: it is only considered less good in comparison with the other items or alternatives. The explanation for each individual score, the factors for all of the alternatives has been outlined in annex 3.8.1.

The division of scores is:

- 1 means good
- 0 means neutral
- -1 means bad

The results are presented in the table below.

Table 4.12

Results Multi Criteria Analysis, round 1

Criteria / Alternatives	Channel alignment		Nautical ease			Port zoning & efficiency		Waves & sedimentation		Total score
	Env. forces	Dredging	Env. forces	Entrance	Space	Allocation	Efficiency	Waves	Sedimentation	
A1 (channel NE to SW)	1	0	1	0	1	1	0	0	1	**5**
A2 (channel N to S)	-1	-1	-1	1	1	1	0	1	1	**2**
A3 (channel WNW to ESE)	0	-1	0	0	0	1	0	-1	1	**0**
A4 (channel WSW to ENE)	0	1	0	-1	0	1	-1	-1	0	**-1**

From the above table it is evident that alternative A1 (with the approach channel from NE to SW at around 210°N) scores clearly the best. For this alternative more layouts will be generated in order to result in an optimal port layout.

4.5.2 ALTERNATIVES ROUND 2

From the first round of MCA it is evident that the port layout with an approach channel from NE to SW looks the most promising (alternative A1). Now that this global approach and entrance direction for the port have been determined, and a decision has been made for alternative A1, the next round will commence. Here, various new alternatives to A1 will be generated and compared to each other. The alternatives will be compared on the criteria defined hereafter.

Criteria

- Costs

Here, the total construction costs of the port are considered. Although one of the objectives was to maximize throughput, this will always have to be done within reasonable boundaries. Attention is paid to the construction depth and length of the breakwater, which is an important cost item of the port (especially in this project). Besides this, it makes a large difference if the terminals can be constructed at shallow grounds or at deeper locations which require large amounts of sand for reclamation. For this, one has to take into account dredging (also of hard soils for the channel) and land reclamation: the cut & fill-balance. Often, costs are one of the most important criteria a decision for an alternative is based on.

- Nautical Ease

With this criterion, the port approach manoeuvre is again assessed under influence of environmental forces, as well as the safety of the vessels in-port. With this, congestion is taken into account, which is considered unfavourable in layouts with long basins and less space of manoeuvring. This also takes into account the amount of manoeuvring space available, the stopping length and the total amount of shipping traffic at critical points.

- Construction Phasing

With construction phasing the ease of the construction of the port in different phases is considered. For various alternatives removal of parts of a breakwater or construction of a whole new one will have to be realized, when expanding the bulk port with the container terminal: this is considered less convenient (to other alternatives). Besides this, the in phase construction of the container terminal individually (for the bulk terminals this is more or less the same for all the alternatives) is different for many alternatives, where some of them are considered better than others. Also, under this item the independent development of the bulk and container port is taken into account: if this is possible relatively easy, it results in a high score.

- Port Zoning & Location.

Here, the terminal layout will be assessed. It will be judged whether the different terminal parts are situated logically to enhance port- and cargo handling efficiency. If this is the case, a high score will be rewarded to the alternative. Their form will be judged in relation to the arriving traffic. Here, also the location of the terminal plays a role: terminals that are visited more frequently should be fairly easy to reach.

- Port Safety

With the safety of the port, several different topics are meant. One of the greatest risks is (an accident occurring at) the liquid bulk berths, which would have considerable consequences. Because of this, layouts with a separate secluded basin for the liquid bulk berths are considered safer which results in a higher score. Besides this, it is considered if the berths are fugitive and ships at berth are able to get out of the port quickly in case of accidents (this amounts especially for liquid bulk vessels): the emergency response.

- Expansion Possibilities

In the masterplan development, one of the challenges was to maximize the throughput. While in this first round assessment the throughputs for all alternatives is kept more or less constant, the maximum possible throughput ultimately reached in phase II is definitely an important criteria for the new port, which indicates the possibilities of expansion.
The alternatives that offer much space (and ease) of expansion are considered more favourable. Here, a high score has been assigned to the alternatives which ultimately reach the largest throughput.

Assumptions

Again several assumptions have been made to facilitate the decision making process in this phase.

Terminal basins

First of all, the shore parallel basins for the container terminal. With the breakwater orientation and the approach channel alignment, the perpendicular oriented basins will suffer very little from wave action. Also, this way the discharges from the various wadis can flow more easily into the port (as determined before). Besides this, the shore parallel basins give rise to high efficiency in terms of space utilisation and efficient cargo handling. Also, this way the number of ships berthed in line is limited so that congestion is minimized. Besides this, (part of) a container terminal which has been constructed in phase I can be repeated exact the same way, which eases the construction manner and could decrease construction time in the future (because of the same construction methodology already applied). As a concluding remark it is emphasized that shore parallel basins would result in a more uneven cut and fill balance, as large volumes of sand are required for land reclamation.

Cargo forecast

Here, it is assumed (when keeping an eye on the future development) that the port will have to expand considerably between the phases I and II. Because this increase in throughput is rather large (especially for containers), it is advised to design the port at the end of phase I already for the maximum capacity with accompanying phase I^{+} (which is 6.000.000 TEU throughput). This avoids an even larger required increase in throughput when advancing from phase I to phase II: it promotes a more even port development. In the alternatives, the ultimate expansion for the bulk terminals is already presented as they will always fit in.

Location

Again, in the following alternatives the area around Punta Betoya has been avoided on purpose because of the tectonic fault and the Rio Kert. Furthermore, dredging of the hard soil (rock) around Punta Negri has been minimized: the approach channel from t his direction mainly requires dredging of sand (southwards of the dashed line).

Alternatives

Below, the alternatives to A1 are presented. Per alternative, a description has been included regarding the design principles and the line of reasoning. Special attention has been paid to the expansion possibilities and the ease of which this is possible. Differences in (possible) throughput will be outlined, and the possibilities of in-phase construction of the container terminal will also be judged. Alternative A1-0 is the same as alternative A1, and is included here as comparison with the new layouts so that the differences will become clear.

Alternative A1-0

Alternative A1-0 is presented in the figure on the next page.

- **Design principles**

This alternative is the same as alternative A1, and has been included (again) as a standard alternative from which differences with the other (new) alternatives will become clear.

- **Description**

The alternative presents one total port, with the preferred approach channel alignment and terminal allocation. The terminals are places against the 'practical inland boundary', which is situated at a terrain height of about CD +10 - 15 m.

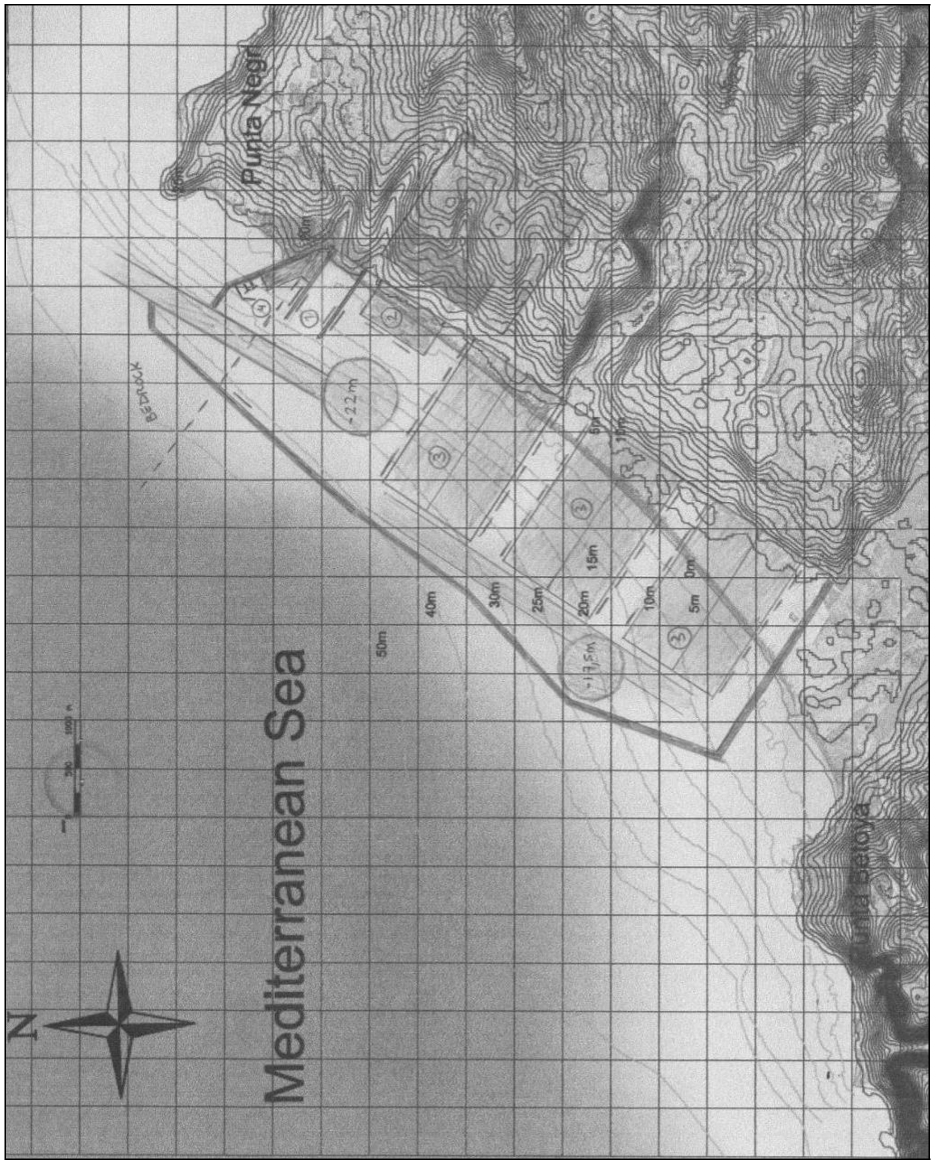
Both the dry and liquid bulk terminals reach their maximum (required) throughput in phase II^+. This is the case in all of the alternatives. Regarding the container terminal, as outlined before phase I^+ is reached in all the alternatives, but phase II^+ differs: here, 30 berths will be realized with an annual throughput of 18 MTEU at maximum.

- **In-phase Construction**

The independent development of the bulk port can be accomplished by constructing a (temporary) shore-perpendicular breakwater at the location of the most northern container terminal block. Parts of this breakwater will at a later stage have to removed to expand the port with the container terminal. This expansion can take place from the most northern container terminal to the south, with accompanying lengthening of the container terminal breakwater. However if the decision for acquiring the maximum throughput already has been made at an early stage, construction of the container terminal breakwater all at once is advised (to avoid additional costs of partly removal).

Figure 4.15

Alternative A1-0: the earlier presented standard alternative



Alternative A1-1A

Alternative A1-1A is presented in the figure on the next page.

- **Design principles**

The design principle used here is maximizing throughput. This was one of the project objectives and has been included in the following 2 alternatives (A1-1A and A1-1B).

- **Description**

As in alternative A1-0, the layout presents again one total port. To acquire the maximum throughput, additional terminals have been located seawards of the sandy beach. This is because all of the surface area along the sandy beach is already occupied with container terminals. The cut and fill balance is off: too much land reclamation is required.

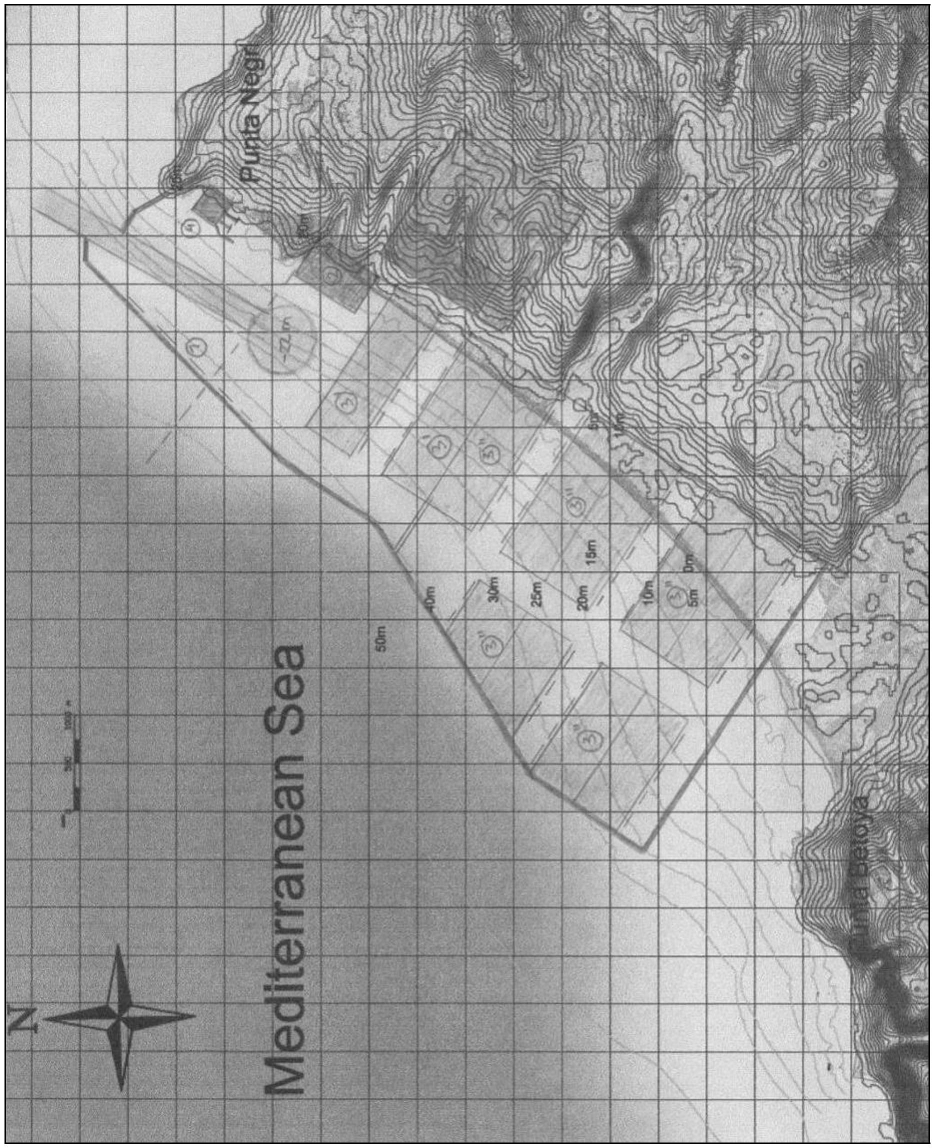
As mentioned before, the bulk terminals all reach their required maximum throughput. For the container terminal 48 berths are designed, so that an annual throughput of 28,8 MTEU can be ultimately achieved in phase II^{+}. This is almost the maximum required capacity.

- **In-phase Construction**

The independent development of the bulk port and the container port can (as in alternative A1-0) be done by constructing a temporary shore-perpendicular breakwater at the most northern container terminal block (see drawing, where e.g. phase I^{+} of the container terminal is included). Also here, parts of this breakwater will at a later stage have to remove to expand the port with more container terminals. In this alternative, there are several possibilities when deciding what part of the container terminal should be realized first, which improves expansion flexibility. Nevertheless, again if the decision for acquiring the maximum throughput already has been made at an early stage, construction of the container terminal breakwater all at once is advised (to avoid additional costs of partly removal).

Figure 4.16

Alternative A1-1A: maximizing throughput



Alternative A1-1B

Alternative A1-1B is presented in the figure on the next page.

- **Design principles**

Again in this alternative, maximizing throughput is the main design principle here.
The previous alternative showed a very uneven cut and fill balance. To improve this, here use has been made of surface area land inwards.

- **Description**

Land inwards of the sandy beach, there is much space is available. However, this terrain exhibits a steep slope, which would require massive earth movements. (It is assumed that) this material can be used for the land reclamation of the container terminal. With this, an even cut and fill balance can be achieved. The wadi at the middle of the sandy beach has been used as a basin, because at that location the least amount of dredging would be required.

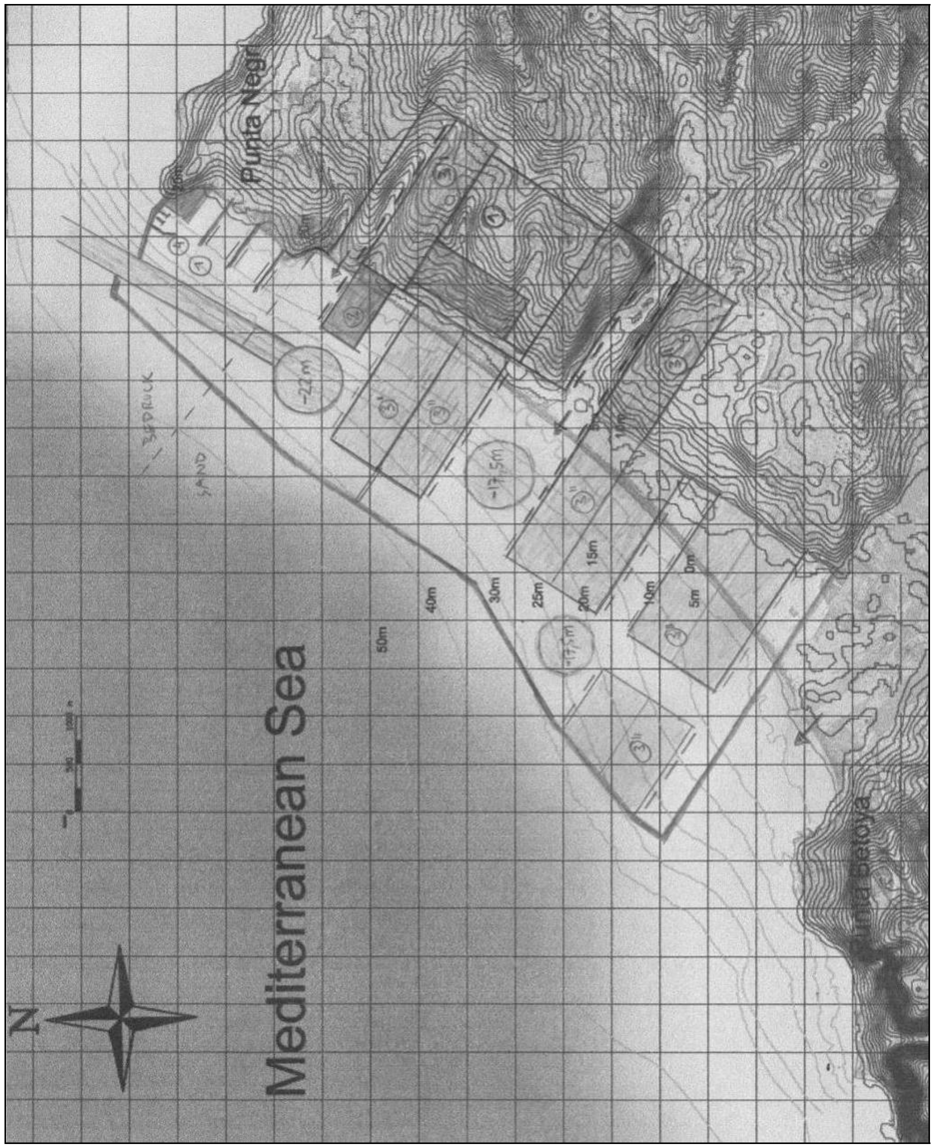
Extra manoeuvring areas have been provided near the container terminals to ease turning of the vessels. The maximum required throughput of 30 MTEU can be achieved in this alternative, with 50 berths. However, this requires large earth movements (dredging as well as land reclamation) and subsequently the alternative will be very expensive. Also the breakwaters are very long and reach considerable depth.

- **In-phase Construction**

When only the bulk port will be realized, the independent development necessitates a shore-perpendicular separate breakwater to the southwest of the dry bulk terminal. Construction in phases can be done by extending the breakwater (which divides the northern container terminal) to the south. Parts of it will have to be removed when expanding. Again, in this alternative there are several possibilities when deciding what part of the container terminal should be realized first, which improves expansion flexibility. Nevertheless, again if the decision for acquiring the maximum throughput already has been made at an early stage, construction of the whole container terminal breakwater all at once is advised (to avoid additional costs of partly removal).

Figure 4.17

Alternative A1-1B: maximizing throughput + cut & fill balance



Alternative A1-2

Alternative A1-2 is presented in the figure on the next page.

- **Design principles**

It is expected that the above mentioned alternatives reaching the maximum throughput will become very expensive. Costs will always be main criterion when choosing between alternatives, so the maximum throughput has been translated into: what is reasonably possible. In order to compare this, alternative A1-2 has been designed which aims at minimizing costs.

- **Description**

The breakwater construction depth is limited to 35 m. at maximum and with the situation of the port against the 'practical boundary', a cut and fill balance has been reached as much as possible (without large earth movements). Also, here no submarine pipelines are required for transporting the liquid bulk. The dredging of approach channel and turning basin is brought back to a minimum by locating them northwards of the CD -22 m. depth contour. Also, the liquid bulk berths have been located at a larger depth to minimize dredging.

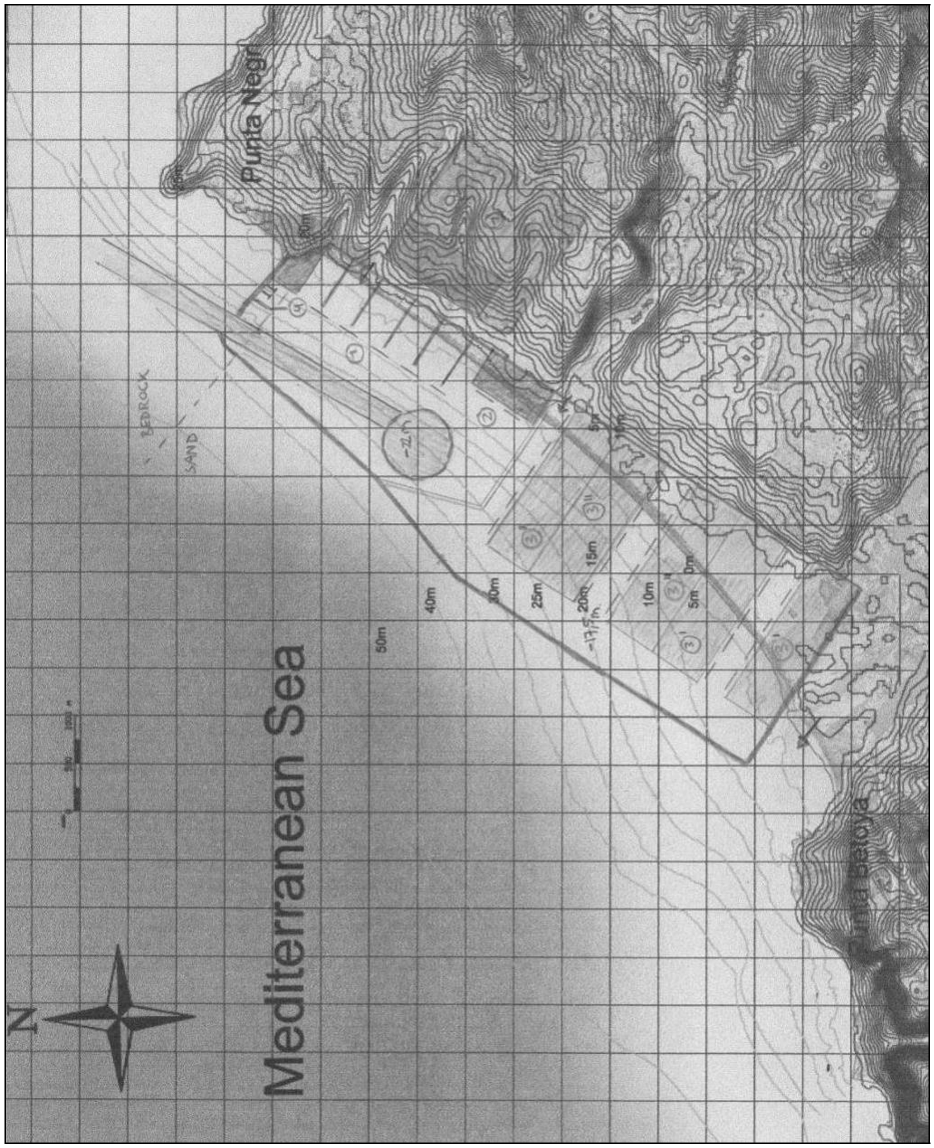
The result is a rather compact overall port. Because of the objective to minimized costs, the breakwater length and depth are limited, which inevitably results in a smaller enclosed port area. Because of this, with 25 berths only a minimal throughput of 15 MTEU can be achieved in phase II^{+}.

- **In-phase Construction**

As indicated in the layout, the independent bulk port can be realized by constructing an extra breakwater to the southwest of the dry bulk terminal. When the port will be expanded with the container terminal, parts of this breakwater will have to be removed (which entails extra costs). If the container terminal is to be realized in the future, it is advised to construct the most south-western part of the breakwater first (until a depth of around CD -20 m.), so that container terminal block 3^{I} can be realized. This way, also two independent ports exist, which can be combined (if all the container terminals are constructed) by removing parts of the dry bulk port's breakwater.

Figure 4.18

Alternative A1-2: minimizing costs



Alternative A1-3A

Alternative A1-3A is presented in the figure on the next page.

- **Design principles**

In the previous alternative, the independent construction of the bulk and container port was already (partially) attained. With the next 2 alternatives (A1-3A and A1-3B), the total independent development of both ports has been used as a main design principle. Furthermore, the emphasis is on the easy approach, where the entrance around Punta Negri has been avoided on purpose, to minimize environmental influences on ships entering the port.

- **Description**

The layout presents two different independent ports, the bulk port and the container port. They can be developed independently from each other, regardless the decision what port should be realized first. For this, two entrances have been adopted.

Because the container terminal accommodates the largest share of the shipping traffic, here the required stopping length in-port has been included in the design of this independent port. In contrast to this, the amount of shipping traffic to the bulk port is not that large. Because of this, no stopping length in-port is made available. Tugs can fasten to the vessels outside of the port most of the time (93-94%) Some downtime is accepted here because the shipping traffic is not that large (at maximum around 2 vessels per day).

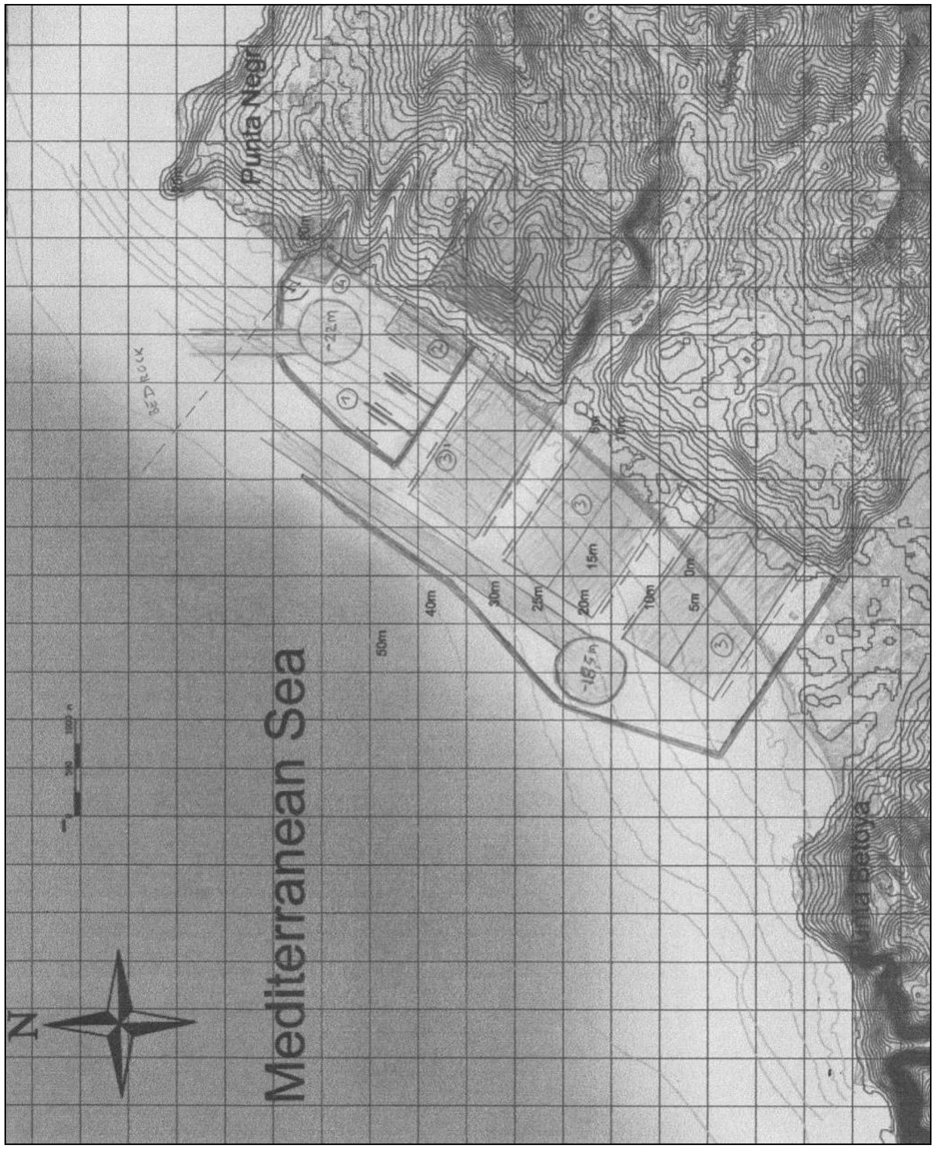
This results in a compact bulk port, and with an orientation of the approach channel at 180°N, furthermore the port entrance around the Punta Negri headland has been avoided. With this, also dredging of the hard soil around Punta Negri has been avoided. Especially here, dredging at the bulk port has remained limited. With the presented 28 berths, an annual throughput of 16,8 MTEU can be ultimately achieved in phase II^{+}. However, the cut and fill balance does not seem to be completely satisfied.

- **In-phase Construction**

As outlined before, the two different ports can be developed from each other independently. The expansion of the container terminal can at best be accomplished when starting from the southwest (near the Rio Kert) and advancing to the northeast through the phases. This way, only a part of the container port breakwater needs to be constructed in phase I (e.g. the corner with the turning circle included). The in-phase construction of the different (ports with their) terminals is considered very flexible.

Figure 4.19

Alternative A1-3A: completely independent development



Alternative A1-3B

Alternative A1-3B is presented in the figure on the next page.

- **Design principles**

The same design principles as alternative A1-3A apply here: independent port development and an easy port approach. Furthermore, in alternative A1-3B the emphasis is on creating a more even cut and fill balance.

- **Description**

While the previous alternative showed potential, the cut & fill balance is off. To improve this, the line of reasoning for alternative A1-3A has again been followed, with the difference that in alternative A1-3B again a part of the container terminal has been placed more inland. This results in an inland basin around the middle wadi where additional berths have been located. Again, completely independent development of both ports is possible.

The container terminal turning circle is centrally located, which facilitates quick turning of the vessels as most berths are located nearby. The basin at the middle wadi is somewhat larger in width, so that most of the vessels can already be turned in the basin itself. In the layout with the presented 35 berths, a maximum annual throughput of 21 MTEU can be achieved.

- **In-phase Construction**

As explained in alternative A1-3A the two different ports can be developed from each other independently. The expansion of the container terminal can at best be accomplished when starting from the southwest (near the Rio Kert) and advancing to the northeast through the phases. However, in this alternative more possibilities exist for the in-phase construction of the different container terminal blocks (e.g. first the seaward terminals and subsequently the inland basin). As in alternative A1-3A also here, only a part of the container port breakwater needs to be constructed in phase I (e.g. the south-western corner until the CD -30 m. depth contour). Expansion of the container terminal can be accomplished to the northwest.
The in-phase construction of the different (ports with their) terminals is considered very flexible.

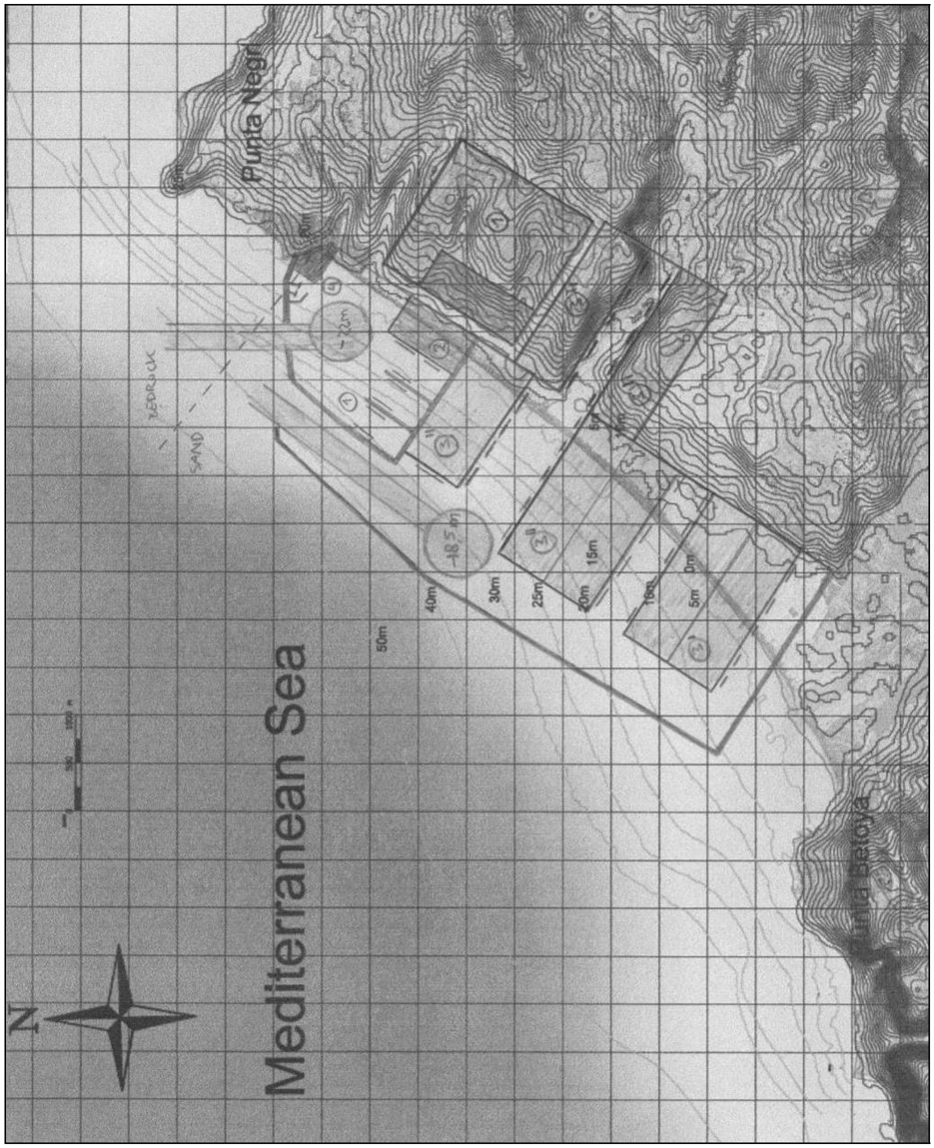


Figure 4.20

Alternative A1-3B: completely independent development + cut & fill balance

Multi Criteria Analysis

The above presented alternatives are again compared only to each other. There are several aspects that they have in common, for this they have no difference in score (neutral). When there is a clear difference between the two alternatives, this will result in a plus point for the specific alternative. The results are presented in the table below. The argumentation for each individual score has been included in annex 3.8.2.

The values represent respectively:

- 1 means good
- 0 means neutral
- -1 means bad

Table 4.13

Results Multi Criteria Analysis, round 2

Criteria / Alternatives	Costs			Nautical ease			Construction phasing		
	Breakwater	Cut & fill	Channel	Channel	Congestion	Manoeuvring	Breakwater	Terminals	Indep. Exp.
A1-0 (original)	0	0	1	0	0	1	0	0	0
A1-1A (max throughput)	-1	-1	0	0	-1	-1	0	1	0
A1-1B (max throughput + c&f)	0	0	-1	0	0	1	0	1	0
A1-2 (min costs)	1	1	1	0	0	0	0	0	-1
A1-3A (independent)	0	0	1	1	0	0	0	0	1
A1-3B (independent + c&f)	0	1	1	1	0	1	0	1	1

Criteria / Alternatives	Port zoning & location		Port safety		Expansion possibilities		Total score
	Layout	Location	Liquid bulk	Em. response	phase I	phase II	
A1-0 (original)	0	0	-1	0	1	0	2
A1-1A (max throughput)	0	-1	0	0	1	1	-2
A1-1B (max throughput + c&f)	1	0	-1	0	1	1	3
A1-2 (min costs)	1	0	-1	0	1	-1	2
A1-3A (independent)	1	0	0	1	1	-1	5
A1-3B (independent + c&f)	1	0	0	1	1	0	9

From the MCA above, it becomes clear that alternative A1-3B (independent development with cut & fill balance) is clearly the most promising alternative. From hereon and further, this alternative will be used as the final masterplan.

4.5.3 CONCLUSION

The two-round Multi Criteria Analyses have shown that alternative A1-3B looks the most promising port layout for the project location at Nador. The alternative reaches a reasonable throughput (in size), and scores relatively good compared to the other layouts. While this specific alternative was only a first layout to the specific design considerations, it is very well possible that several improvements can be made. This will be addressed in the next paragraphs. Nevertheless, the decision for the preferred alternative A1-3B is made and will be used from hereon and further.

4.6 FINAL PORT MASTERPLAN

Now that the resulting port layout has been determined, some last loose ends will be rectified and some considerations will be done regarding optimizations and improvements of the port layout from alternative A1-3B.

4.6.1 LAYOUT IMPROVEMENTS

After selection of the most suitable layout, the optimisation of the project will take place. The determination of the final characteristics of the port can be outlined, such as dimensions for port entrance, manoeuvring space, number of berths, terminal forms etc. The tools which could be used for this include computations, hydraulic model studies, navigation simulator studies and operation simulation models with as main purpose to minimize the costs.

As not all of these in-depth studies can be carried out within the scope of works, the focus will be on several specific items. Several specific design considerations are addressed and optimized, in order to minimize the costs. Furthermore, wave penetration and breakwater alignment will be assessed in the following chapters to arrive at an improved breakwater layout.

Cut & Fill Balance

It has not exactly been assessed into detail whether the cut and fill balance actually is completely satisfied. In order to accomplish this, a global consideration regarding the volumes of materials to be dredged and land to be reclaimed is necessary. This could lead to a repositioning of e.g. the container terminals more seawards or landwards.

Without inland basin

The cut & fill balance for alternative A1-3B will be assessed for the case where the inland basin is not (yet) realised. For this, first of all the cut & fill balance for the terminals seaward at the 'practical land limit' has been assessed, see the figure below. The red parts indicate land reclamation (1, 2, 3, 4, 5) and the blue parts dredging (6, 7, 8, 9, 10, 11). For the green part (12) it is assumed that this area doesn't add nor subtract from the balance, as removed soil material can be used to level the terrain in cascades for the tank farm.

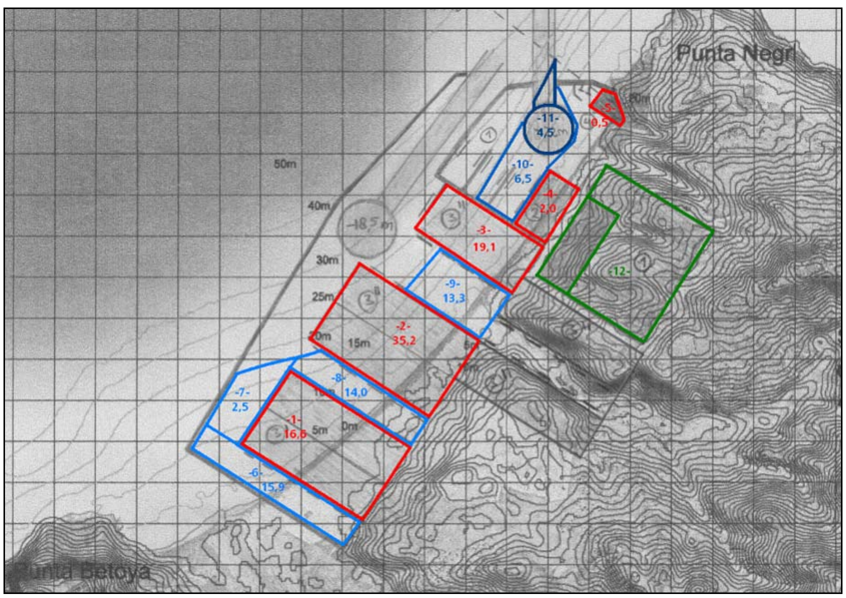


Figure 4.21

Indicated areas that require dredging (blue) and land reclamation (red)

Port items indicated with the amount of dredging or reclamation in 10^6 m^3

In a first estimate, the terminal height above MSL has been assumed at CD +3 m.
When taking the average bottom depth at the terminal locations, the volumes of sand to be reclaimed are calculated, and presented beneath the terminal number (in 10^6 m^3).
For dredging applies that the required (basin) depth minus the current sea bottom results in the amount of material to be dredged. For the container terminal, only depths of CD -17,4 m. are required, while at the bulk terminal this varies for the turning basin (CD -22 m.) and the berths. The oil berth is assumed to be at the inside of the (northern) breakwater, for which no dredging is required. The rest of the bulk basin only requires a depth of CD -19,4 m.

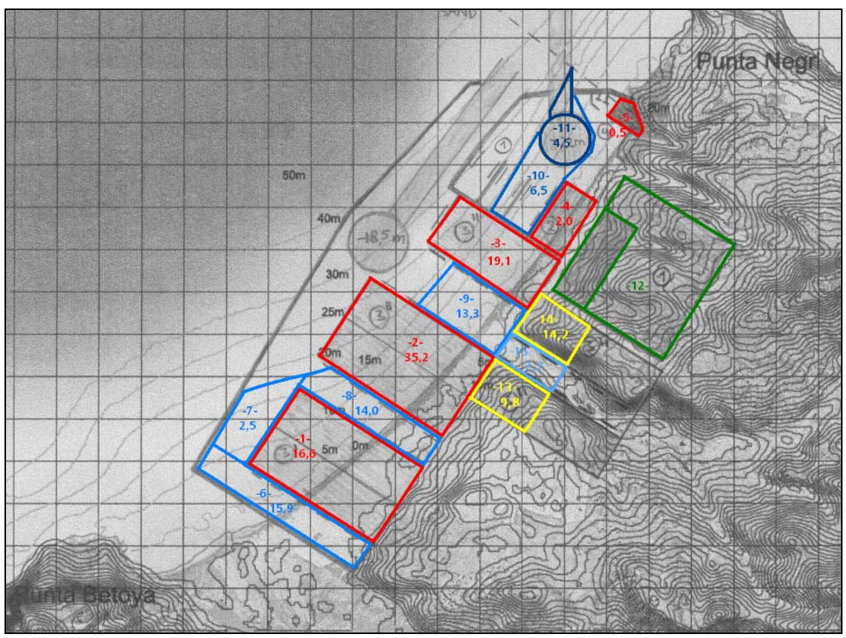
In this first (global) determination of volumes of soil that needs to be removed or reclaimed, it appears that more reclamation ($74*10^6$ m^3) needs to be done than dredging ($56*10^6$ m^3), where dredging (including inland terrain levelling) is around 75% of the reclamation.
This 'gap' of material can be filled with dredging of the inland basin.

With inland basin

When considering these basic volumes, it is expected that because of the steep inland slope dredging of the total inland basin would result in massive soil movements and a surplus of reclaimed soil. A second assessment has been done by considering 2 berth lengths inland (which is a total of 4 berths), with the required container surface area behind them.

Figure 4.22

Indicated proposed 4-berth container terminal expansion at inland basin (yellow)



This already results in an increase in 'dredging' to $88*10^6$ m^3, which is 119% of the reclamation. However, there would be large height differences present between the container terminal height at the inland basin (13, 14) around CD +3 m. and the surrounding terrain, up to CD +55 m. This would make the construction practically difficult.

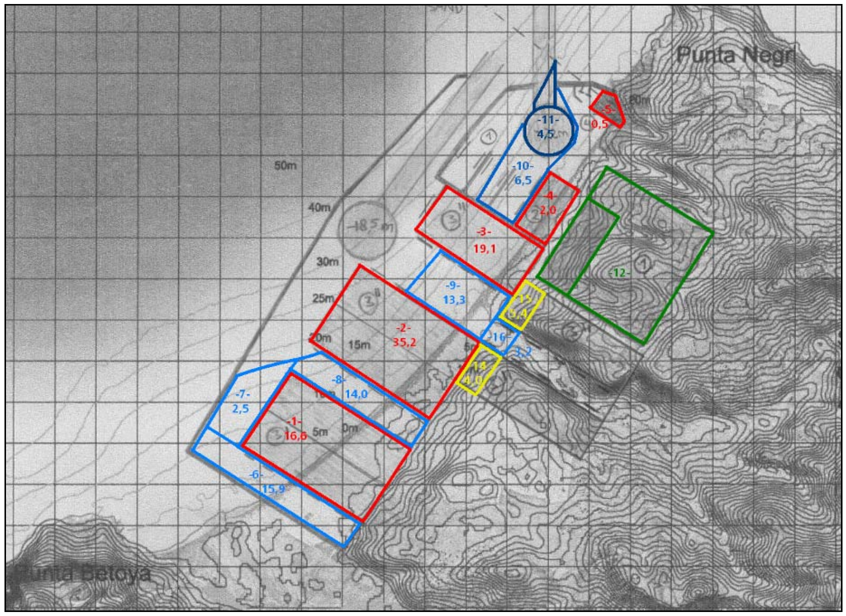
Besides this, the amount of material for levelling of the surrounding terrain (because of the height differences) has not yet been taken into account, and would also add to the total amount of 'dredging' required.

Furthermore, next to the inland basin already the oil tank farm is present, which should be constructed in cascades. In order to accomplish this, a steep slope will have to exist between the two port items (container terminal and oil farm).

Because of the above outlined arguments, it is advised to realize an inland container terminal basin with only 1 berth length (2 berths in total).

Figure 4.23

Indicated proposed 2-berth container terminal expansion (yellow) at inland basin



This results in a total volume of $69*10^6$ m^3 'dredging' of land required, which is around 93% of volume of material required for reclamation (which remains the same at $74*10^6$ m^3. The remaining percentages (soil material required for land reclamation) can be acquired by the necessity to level the surrounding terrain in an assumed slope of around 1:3.

Throughput

Now that the choice has been made for an inland basin with only 1 berth length, this would result in a considerable decrease of maximum throughput. As this was one of the points of the MCA where the alternative scored rather well, alternatives need to be found. Terminal -3- can be repositioned more efficiently, so that more berths can be realised.

To avoid the approach channel (and especially the breakwater) from being situated to far to the north (which would make the construction of the breakwater more expensive), terminal -3- will have only a depth of 4 berth lengths. The width remains the same as in the former (-1- and -2-) terminals. This way, with 30 berths a throughput can be realized of 18 MTEU/year in phase II^+.

Manoeuvring

The original alternative A1-3B offered a rather wide basin around the middle wadi, but this has decreased in the new alternative. Nevertheless, the basin is somewhat wider than required (here: around 500 m.), which makes it possible to turn vessels with a capacity up to 6.000 TEU in the basin itself (with two moored ships at both sides).

The largest container vessels (14.000 TEU) need to be towed in and out of the basin and cannot be turned here. Nevertheless, the turning circle is located nearby so this should not pose any problems. When these ULCV's arrive, it is recommended to moor them near the turning basin to establish fast berthing and deberthing of the vessels.

Breakwaters

The layout of the breakwaters has in a first preliminary design been drawn up as indicated in the layouts above. The indicated configuration prevents waves and sediment transport from entering the port. Improvements of the breakwater layout and an assessment of the present configuration is subject of the next chapters and will not (yet) be included in the final port masterplan.

4.6.2 REMAINING ASPECTS

Hinterland connections

If in a first (or later) phase the container terminal will be constructed, it will have to be possible to transport up to 20% of the throughput to the hinterland, via road and rail. It is in a first approximation assumed that the ratio between container transport via road and rail amounts to 1:1. This means that 10% of the total container throughput will occur via road and 10% via rail. It is again assumed that truck and trains unload and subsequently load cargo.

Besides this, the amount of traffic that will be generated by the various terminals and the work related traffic to the port must be able to efficiently get to and leave the port by land. In order to accomplish this, proper infrastructure is indispensible.

Roads

Before construction of the port, first of all, roads have to be constructed in order to transport material, materiel and people to the construction site. It is therefore emphasized that roads will need to be constructed at all times. However, the (ultimately) required capacity for these roads depends on the decision what port will be developed first.

For example, if only the bulk port is constructed (at first), there will be no transport of products to the hinterland. Subsequently, this requires no larger new road infrastructure, and it is assumed that the new port road infrastructure can be connected to the local road network (around Punta Negri where the tourist village was situated).

If the decision is made for the construction of the container terminal, larger road infrastructure is inevitably required in phase I as well as in phase II, as the terminal generates considerable trucking traffic (already in phase I: 10% of 6.000.000/360 = 1.700 containers per day in total to and from the hinterland, and 5.000 containers per day in phase II^{+}). This total traffic ranges from (at maximum, when assuming 1 TEU per truck) 35 trucks/h to 105 trucks/h. These roads must then be connected to the larger already present infrastructure, the N16 to the southwest of the project location. From here, a shore parallel road transport route will be constructed alongside the different terminals.

The roads will be indicated in the final port masterplan in 4.6.3. Their locations have been determined roughly, by taken into account the terrain elevation, the shortest distance to existing infrastructure (strategic routes) and the ease of infrastructure construction.

Railway tracks

As outlined before, depending on the phased development of the port railway tracks are only necessary when the decision for the construction of the container terminal has been made. If this is the case, the same amount of containers (as was calculated under 'roads') needs to be transported to the hinterland (1.700 TEU/d in phase I and 5.000 TEU/d in phase II).

The average amount of containers is around 70-90 TEU/train [WEBSITE RAIL CARGO]. This means train lengths of around 700 m. It is assumed that these trains can be accommodated at the container terminals. When assuming the somewhat higher value of around 90-100 TEU/train, this means that in phase I 8-9 trains/day will visit (incoming + outgoing in one train) the container terminal and 25 trains/d in phase II$^+$. It is assumed that because of shunting of trains several railway tracks (at least 2) will be required ultimately.

Because of the steep sloping terrain (north)east of the project location, and the limited allowable slopes for railway tracks, it is advised to construct railway track also to the southwest of the project location. From hereon, the railway tracks can connect (alongside the N16 in the direction of Nador) to the already existing rail network to the southeast. This is indicated in the final layout in 4.6.3.

As a last remark it is emphasized that the transfer areas for transferring containers to trucks and trains require additional terminal surface areas. These have been indicated as well in the final layout in 4.6.3, where there is some space reserved for this.

Free Trade Zone

In the final port masterplan, according to the specifications, an area of 1000-1500 ha should be available which will function as a free trade zone. Here, normal trade barriers such as tariffs and quotas are eliminated and bureaucratic requirements are lowered to attract new business and foreign investments [WIKIPEDIA], with as main purpose to develop the economy of that location. These zones are mainly used by transnational corporations for establishing factories for the manufacturing of several goods [UNESCAP, 2007].

The free trade zone will be located nearby the container terminal. Because no further specifications for this are given, locating the free trade zone is considered flexible. This area will be incorporated in the final port layout (4.6.3). Attention will be paid to the present terrain, and the required amount of terrain levelling. Because of this, the free trade zone can at best be located inland of the sandy beach (between the middle wadi and the Rio Kert). The present terrain is the most flat, and here, the area is available. Also, at this location the free trade zone is located nearby the existing larger infrastructure.

Downtime

The term downtime is used for the period of time that the port is unavailable and fails to fulfil its primary function [WIKIPEDIA]. Closely related is unavailability, which is the percentage of a time span that a system is unavailable. The port downtime is composed of many different elements. The downtime of the berth can be subdivided in the following two categories [THORESON, 2003]:

- Navigational unavailability: the percentage of time the ship is able to call at the port or berth safely from the open sea,
- Operational unavailability: the percentage of operational time during which the ship can operate by loading and unloading at the berth.

The above mentioned categories lead to differences in downtime between the bulk port and the container port, as different (wave) criteria have been utilized for port entrance.
The various elements are summarized in the following table. For an explanation regarding the different percentages, one is referred to annex 3.9.

Table 4.14

Downtime analysis (port unavailability in percentages)

Downtime	Bulk Port	Container Port
Navigation		
1. Ice problems	-	-
2. Excessive currents	-	-
3. Wind speed u_{10} > 16 m/s	0,85%	0,85%
4. Waves: H_s>2,0 m. ; H_s>4,0 m.	7,1%	0,5%
5. Swell & long period waves	-	-
6. Visibility less than 1000m.	0,2%*	0,2%*
7. Tugboat non-availability	-	0,05%*
Operational		
8. Wind speed u_{10}>20 m/s	0,04%	0,04%
9. Excessive ship movements	0,33%	0,33%
10. Maintenance on berth	0,5%*	0,5%*
Total downtime	**9,02%**	**2,47%**

* values adopted from [THORESON, 2003]

It is stressed that the above presented table only gives a rough approximation of the yearly average berth downtime, and more factors influence the unavailability of a berth.
For example seasonal variation has not been taken into account (differences in wind speeds and wave heights), and the actual downtime differs for various vessel sizes and various cargos.

The average estimated percentage of downtime should not be larger than about 5-10%, due to the extra cost of waiting time for ships to call at the port [THORESON, 2003]. A factor that plays a role here is the amount of shipping traffic and the importance of the cargo. It was deduced earlier that for the bulk terminal, the amount of shipping traffic per day was not that large, so that a slightly higher downtime was considered acceptable. Even with this assumption the downtime stays below the acceptable value of 10%.

In these total percentages of downtime, the joint possibilities have not been taken into account. For instance, a high wind speed leading to high waves. This means that the actual downtime will be (somewhat) lower as calculated above. A more detailed assessment of the berth downtime will be outlined in chapter 5.

4.6.3 FINAL PORT LAYOUT

The final resulting port layout is subject to variation: it depends on the decision what terminals will have to be developed. The different options are presented in this paragraph as follows: first of all the independent development (of the bulk- and container port) is made explicit, with the presented infrastructure. Besides the independent development of the different ports, the in-phase construction is made clear. Finally, the total port layout that results for the construction of all terminals will be presented.

Independent port development

The two different ports can be constructed completely independent from each other, and they are separately included in the total port design. But if, for instance, only the bulk port will be constructed (in the near future), the construction of breakwaters and terminals for the container port will not (yet) be necessary. The independent development for the two different ports is outlined below.

Bulk port

For the case where only the bulk port will be constructed, the layout results in the figure presented below. Only the bulk berth with the tank farm will be constructed, just south of Punta Negri.

The blue line represents the already existing local (secondary) infrastructure. This road can be used for the (limited) traffic to and from the bulk port in this variant. The yellow line indicates the road that needs to be constructed to accommodate traffic from the bulk port and the tank farm. To the south, the larger present road infrastructure (N16) is shown in red.

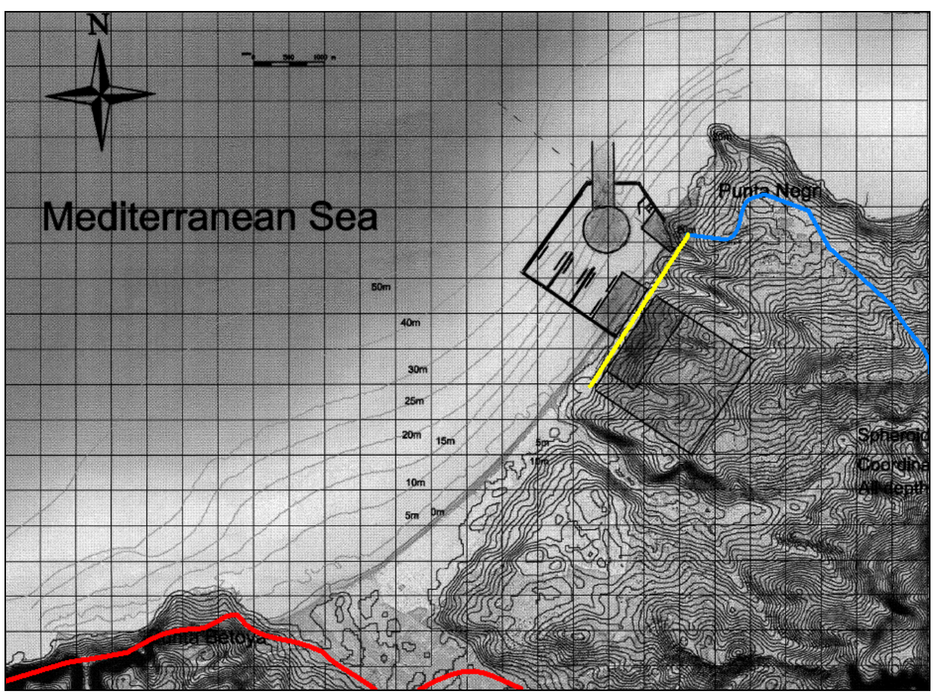


Figure 4.24
Independent bulk port construction with indicated infrastructure

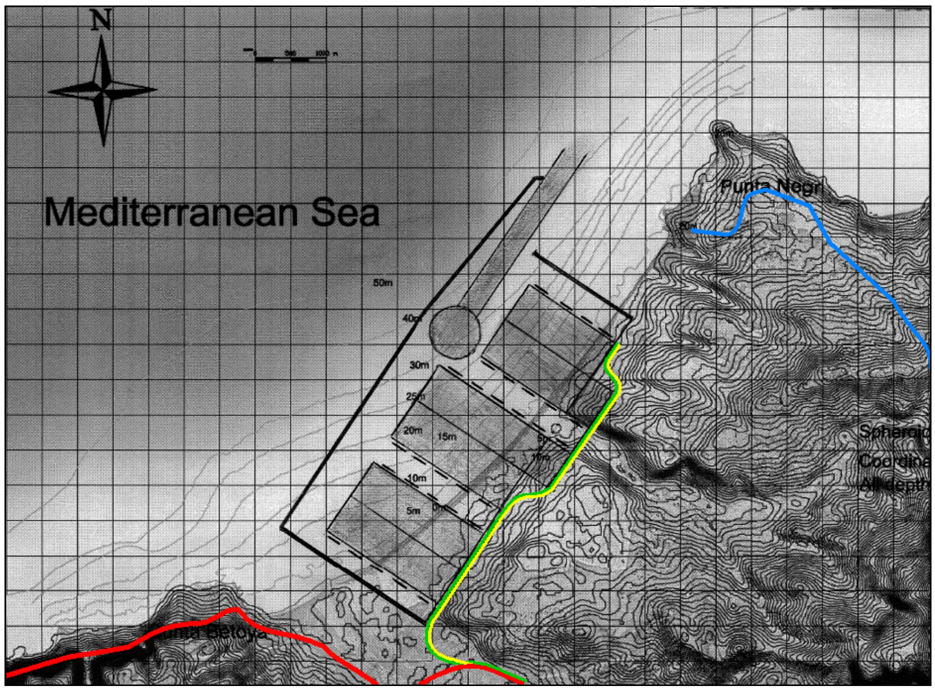
Red: N16
Blue: Local secondary road
Yellow: New (bulk) port road

Container port
When the decision has been made to only develop the container port, the resulting layout for the container terminals only can be drawn up. This has been presented in the figure below. This figure shows again the final expansion of the container port, at the end of phase II^+.

Again, the red and blue lines represent the already present infrastructure, respectively the N16 and a local secondary road. Because of the large increase in road traffic, it is advised to connect the new port infrastructure to the N16 to the south, indicated in yellow. The green line represents the railway tracks, which can follow alongside the N16 in the direction of Nador.

Figure 4.25
Independent container port construction with indicated infrastructure

Red: N16
Blue: Local secondary road
Yellow: New road
Green: New railway tracks



In-phase terminal construction
Besides the independent development of the different ports, part of the objective was to realize an in-phase development for the different ports (phases I and II). While this does not affect the bulk port layout that much, for the container terminal the situation is quite different.

Bulk port
Here, it is advised to construct the total breakwaters for the bulk port at once, as the expansion from phase I tot phase II requires only an increase in the number of liquid bulk berths from 3 to 5. The amount of dry bulk berths (2) remains the same throughout the phases. With the indicated finger jetty configuration, the jetties are already present for the easy expansion required for phase II. The berths in phase I can be at best located northwards to decrease the amount of dredging required at the south of the southern finger jetty.
The tank farm only requires the solid gray indicated surface area. The oil berth is not (yet) present at this stage.

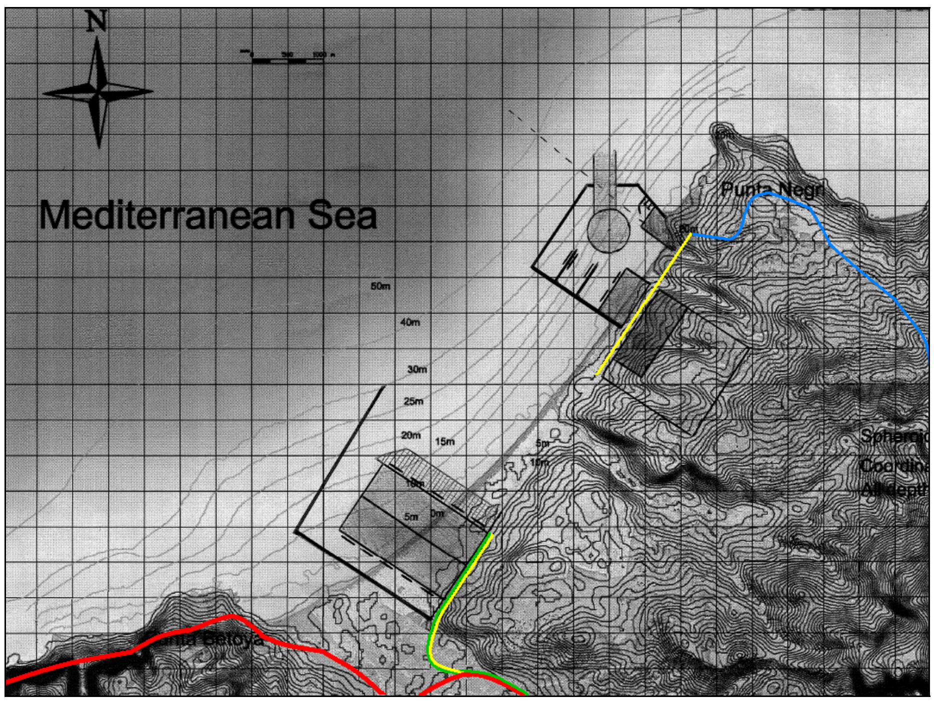
Container port
The container port can at best be expanded from the southwest to the northeast. In phase I, for this only part of the container terminal breakwater needs to be constructed, see the figure below. The achieved throughput for the phase I expansion amounts to 6 MTEU/year.

With this container terminal configuration, some wave action at (especially the northern) berths will still exist. In order to reduce this, it is advised to construct a temporary breakwater at the north of the northern container terminal basin. This is because the percentage of time that waves are actually higher than the allowed wave height is over 50% of time. This would result in too much downtime which is unacceptable. Also, there is in this phase I configuration no in-port stopping length available. This means that tugs cannot fasten to the vessels outside of the breakwaters for 7-8% of time (see annex 2, wave data tables). This container port entry downtime is considered to be acceptable, as in this stage the amount of shipping traffic (5 vessels/day) and throughput are not yet that large.

Figure 4.26

In-phase terminal construction

Southwestern container terminal part (phase I) with infrastructure and bulk port (phase I) with infrastructure



It is emphasized that the container berths required dredging until a depth of CD -17,4 m. Special attention has to be paid that also the northern container terminal berths reach the appropriate depth. This part has been hatched in the figure above.

In the expansion from phase I to phase II, the total north-western container port breakwater will be constructed. With this expansion, the in-port stopping length will be acquired and calm in-port berthing conditions will be realized. The temporary breakwater from phase I will have to be removed. The total resulting port layout will be described below.

Description total masterplan

The overall port layout is presented in the figure on the next page. A description with the resulting specifications will be presented below.

In the final port masterplan, there are two separate entrances provided: one for the container terminal and one for the bulk port. As described earlier, this has been adopted to establish independent port development. The approach channels both end in a turning basin, where for the container port an in-port stopping length has been realized, in contrast to the bulk port where this stopping length is not available. The exact specifications have been summarized in the table below.

As outlined before, the basin in front of the middle wadi is somewhat wider than required (around 500 m.), which makes it possible to turn vessels with a capacity up to 6.000 TEU in the basin itself (with two moored ships at both sides). The largest container vessels (14.000 TEU) need to be towed in and out of the basin and cannot be turned here; this has to be done in the nearby located turning basin.

Table 4.15

Summary final resulting wet port specifications

Specifications wet port	Bulk port Phase I	Phase II	Container port Phase I	Phase II
Approach channel				
alignment [°N]	180°N	180°N	230°N	230°N
width [m]	245 m.	245 m.	230 m.	230 m.
depth [m]	CD -20,5 m.	CD -22 m.	CD -18,5 m.	CD -18,5 m.
Stopping length [m]	-	-	2.060 m.	2.060 m.
Turning basin				
diameter [m]	655 m.	655 m.	800 m.	800 m.
depth [m]	CD -20,5 m.	CD -22 m.	CD -18,5 m.	CD -18,5 m.
Mooring basins				
width [m]	365 m.	365 m.	350-500 m.*	350-500 m.*
depth [m]	CD -19,4 m.	CD -20,9 m.	CD -17,4 m.	CD -17,4 m.

* lower values for 1 sides mooring basin, higher value for middle basin

The amount of throughput that will be realized in the respective phases is once again presented in the table below, in combination with the total amount of shipping traffic. The different terminal areas required per phase are included in the final masterplan.

Table 4.16

Summary final resulting dry port specifications

Specifications dry port	Dry bulk Phase I	Phase II	Liquid bulk Phase I	Phase II	Containers Phase I	Phase II
Throughput	**5.000.000T**	**5.000.000T**	**20.000.000T**	**40.000.000T**	**6 MTEU**	**18 MTEU**
# of berths	2	2	3	5*	10	30
Av. vessels/day	1	1	2	2	5	15
Quay length	700**	700**			3.400 m.	10.200 m.
Berth length			376 m.	410 m.		
Terminal area	**33 ha**	**33 ha**	**48 ha**	**250 ha****	**192 ha**	**576 ha**

* of which is 1 crude oil berth

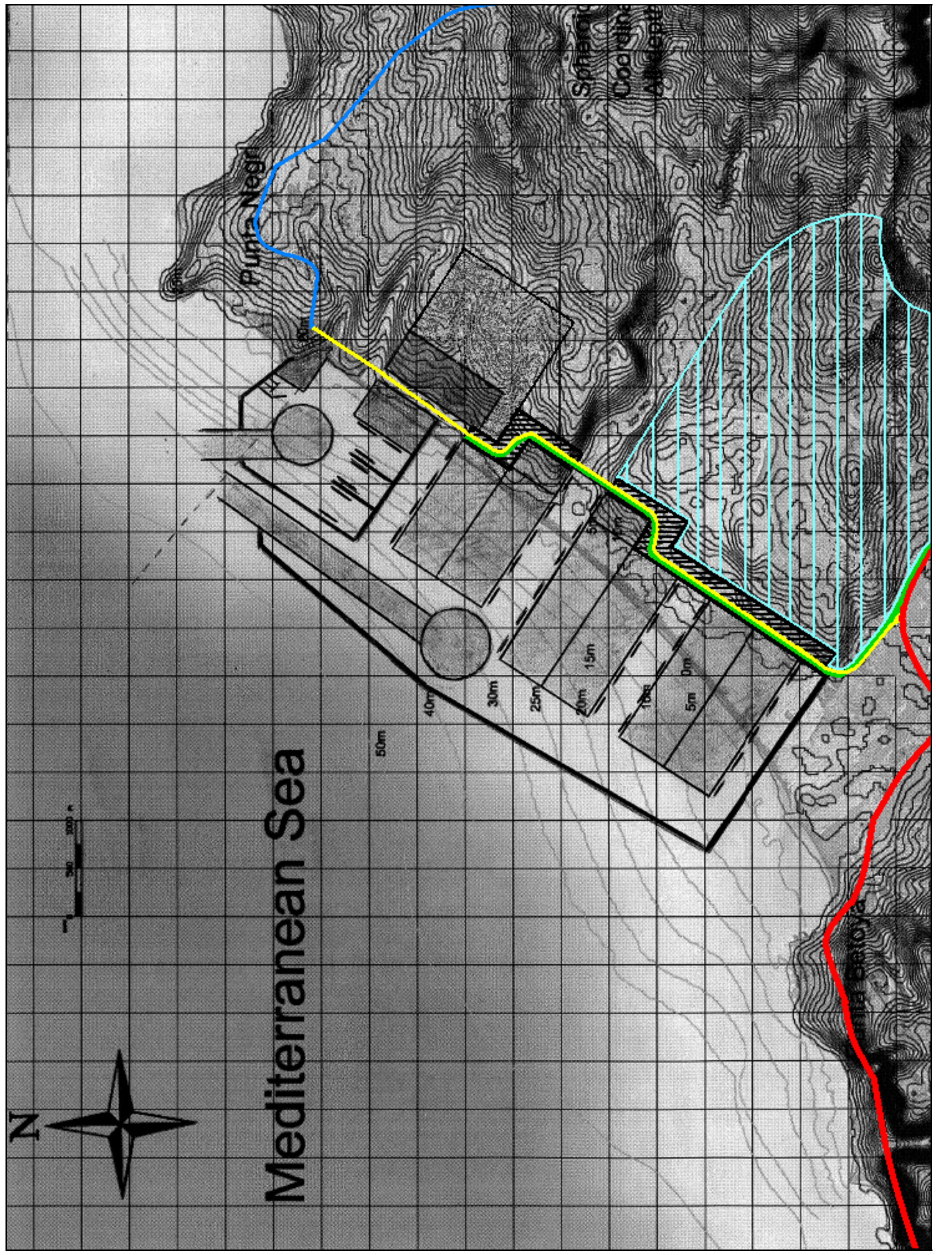
** requirements from client

For the dry bulk and container terminals, the total quay length has been calculated and presented above. For the liquid bulk terminal, only the berth length is relevant, as in the design a finger pier configuration has been adopted, as can be seen in the layout presented below.

Figure 4.27

Overall port masterplan, presented with both bulk and container ports.

Including indicated infrastructure, free trade zone (cyan) and gate areas (hatched black)



The free trade zone with a total area of around 1000-1500 ha has been located nearby the most southern part of the container terminal. At this location the present terrain is the most even levelled, and the free trade zone is located nearby the larger national infrastructure. Allocation further to the south is not advised because of the presence of the Rio Kert.
At the landward side of the container terminals, surface space for gate areas for train and trucks has been provided.

As outlined earlier, the oil berths are located shore parallel to minimize the amount of dredging required. The crude oil and 2 product berths are already at a depth of -25 m, where no extra dredging is required. The liquid bulk berths to the southeast do require some additional dredging, as is the case for the berths for the dry bulk terminal. The tank farm will be constructed in cascades, where it assumed that the ground balance can also be satisfied as a whole. The land for the port facilities will need to be reclaimed, but the area for the service craft is already on appropriate depth of around 6-7 m.

To satisfy the cut & fill-balance, the dry bulk terminal is located half onshore, and half in sea. For the total container terminal it applies that the cut & fill-balance also has been satisfied: this has been accomplished by dredging of the inland basin. This will be done inland with one berth length. The surrounding terrain will need to be partially levelled until a natural slope of around 1:3 is achieved. The area at the landward side of the container terminal can be used as a transfer area for transferring containers to trucks and trains.

The above presented resulting port masterplan fulfils the throughput specifications and meets the objectives as defined by the client, outlined in 2.2.3 and 2.2.4. Whether this port masterplan (and especially: its breakwater layout) also fulfils its function in efficiently creating calm in-port berthing conditions by preventing (too much) wave penetration will be assessed in the next chapters.

CHAPTER 5 Wave penetration study

5.1 INTRODUCTION

Now that the final port masterplan layout has been selected in the previous chapter, this resulting layout will be evaluated in more detail regarding the aspect of in-port wave penetration. The wave penetration study serves two different purposes: it aims first of all at evaluating the wave penetration in-port with the predefined breakwater layout(s) as determined in the previous chapter. Subsequently, if required it aims at optimizing the predetermined breakwater layout, so that wave penetration in-port is minimized to ensure safe loading and unloading under as many wave conditions as possible.

To get a preliminary indication of the wave penetration in-port, relevant characteristics have to be known beforehand. This applies not only to the wave parameters but also to the breakwater composition. Different types of breakwaters have different effects on the wave penetration and propagation in-port (e.g. reflection coefficients). Because of this, first of all an assessment of the breakwater type(s) and their composition will be made. With this preliminary design, the in-port wave penetration based on the wave data from 3.3.4. can be evaluated: the reflection coefficients of the different breakwaters can be imported in a simulation model.

The overall port layout will eventually be used as input for this simulation model, and it will be checked whether the wave conditions in-port do not exceed the maximum allowed operational (and limiting) wave heights at the berth. These have been outlined earlier, and it was evident that this was especially of importance for the container terminal berths, where the operational wave height should not exceed $H_s \leq 0.5$ m. If this does happen, measures are necessary, and layout improvements will have to be made. The chapter will be concluded with recommendations regarding this (new) breakwater layout.

5.2 BREAKWATER TYPE

As outlined before, first of all a (short) consideration regarding the breakwater type will be made. From the environmental data in chapter 3 it is evident that there exist some (practical) constraints for the breakwater construction. These will have their influence on the choice of the breakwater type. A more in-depth consideration is subject of the next chapter, here only a global indication of the breakwater compositions will be made.

5.2.1 CONSTRUCTION CONSTRAINTS

Water depth

From the selected final masterplan layout in the previous chapter it is clear that the designed breakwater reaches large construction depths: up to CD -45 m. This large water depth will require special measures to make an economic design. For instance, rubble mound breakwaters with a slope of 1:2 would require a bottom width of 15+2*45*2=195 m. which requires large quantities of material.

Tectonic fault

At the location of Punta Betoya and the Rio Kert, the Taliwine fault is present (see chapter 3). The offshore location of this fault is not exactly known, but the presence of this fault necessitates a special design enable the breakwater to follow (uneven) settlements and movements (flexible construction).

Use as berth

This mainly applies for the crude oil berth, as indicated in the final bulk port layout. The oil product berths exist of a finger jetty, but the crude oil berth is located at the inside of the northern bulk port breakwater (in order to save wet port area). With this decision, enough water depth (directly) along the breakwater is required.

5.2.2 RUBBLE MOUND OR MONOLITHIC

The main choice for the design of a breakwater is the choice between a structure of the rubble mound type and one of the monolithic types. These two types have their own specific advantages and disadvantages. These will be outlined below, where a choice will be made regarding the specific breakwater types.

Rubble mound breakwaters can withstand unequal settlements. Because of the presence of the Taliwine fault, this is assumed a prerequisite for the breakwater located nearby. Besides this, the water depth at the southwest of the sandy beach remains limited, so that no excessive quantities of material would have to be used. Besides this, the container terminal breakwater is not directly used as a berth. All these arguments favour the use of a rubble mound breakwater type, at least for the south-western part (corner) of the breakwater (see figure 5.1).

The longest (northern) container terminal breakwater is more or less located shore-parallel at a steep seaward bottom. The breakwater reaches depths up to CD -45 m., which would require very large quantities of (rubble) material. At these large water depths, it is more economic to use monolithic breakwaters [VERHAGEN *et al.*, 2009]. Monolithic breakwaters also have a shorter construction time than rubble mound breakwaters, which is ideal for rapid expansion of the container terminal to the northeast. The point of transition between the rubble mound and the monolithic breakwater can be made at the 3[l] expansion of the container terminal (the most south-western block), at around a depth of CD -25 m. to avoid too much wave penetration from the dominant directions (W and WNW). This point is also located somewhat further away from the tectonic fault.

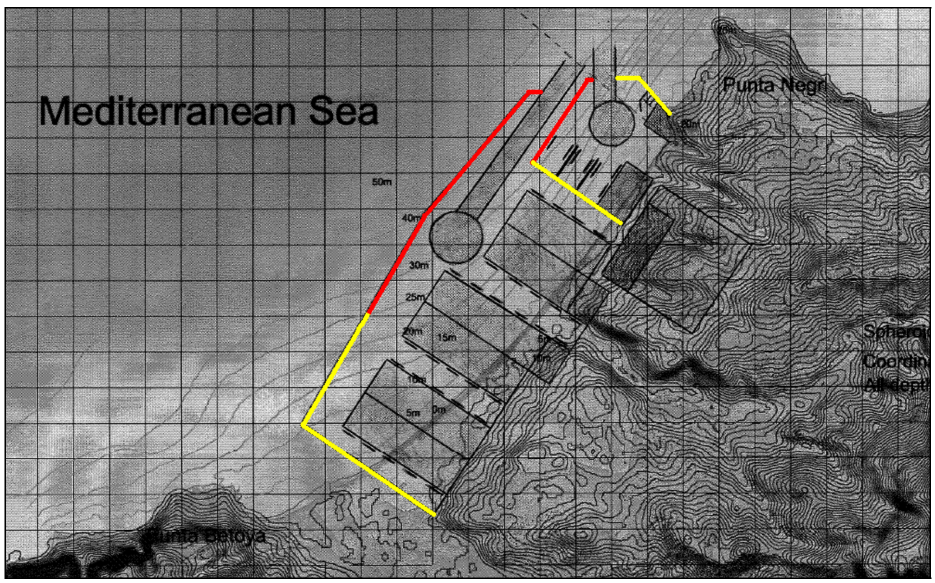
It could turn out that a caisson cannot overcome the total water depth at once, so that a combination will be used of a rubble bed with a caisson placed on top. This design concept is advised, as caissons with a uniform height can be used.

Also the north-western shore-parallel breakwater of the bulk port can at best be constructed with a monolithic type of breakwater. This is because of the earlier mentioned use as a berth for (un)loading of crude oil, and the rather large constant water depth of CD – 35m.
The south-western breakwater of the bulk port can again (as was the case for the container terminal) be designed as a rubble type, as no direct berths are located nearby and the average construction depth remains somewhat limited. These same arguments apply for the north-eastern bulk port breakwater.

The above outlined arguments result in a breakwater configuration as indicated in the figure below. Yellow indicates rubble breakwater construction types, and red monolithic breakwater types (caissons).

Figure 5.1

Breakwater configuration
Yellow: rubble type
Red: monolithic type



Regarding the exact composition of the rubble mound breakwaters (e.g. the use of armour units), a decision will be made in the next chapter, in combination with the technical design. The above outlined indications of the breakwater types are at this stage sufficient to make an in-port wave study possible.

5.3 WAVE ASSESSMENT

In order to evaluate the in-port wave climate, a certain norm has to be specified. With this standard it can be assessed what portion of time wave heights at the berth for safe (un)loading of the vessels are exceeded, which results in berth downtime. These wave criteria are specified in 5.3.1.

Various wave processes influence the propagation of waves in-port. To assess the extent of waves entering the port, an inventory regarding the wave-related processes and expected problems will be made.

These wave-related problems will be evaluated by making a global estimate of their extent according to the Coastal Engineering Manual [USACE, 2002]. From this first assessment, it will become clear whether a more detailed analysis of the in-port wave penetration will be necessary.

5.3.1 WAVE CRITERIA

Besides the breakwater composition, operational limits for wave conditions need to be specified in order to assess the in-port wave climate and overall downtime of the berths.

At the Berth

The operational wave conditions differ for respectively the liquid bulk-, dry bulk- and container terminals. The operational limiting wave heights at the berth, adopted from 4.2.2, are presented in the table below.

Table 5.1

Limiting wave heights at the berth for (un)loading of different vessel types [LIGTERINGEN, 2007]

Vessel type	Limiting wave height Hs [m] 0 degrees (head or stern)	45-90 degrees (beam)
Container vessels	0.5 m.	-
Dry bulk vessels	1.0 – 1.5 m.	0.8 – 1.0 m.
Liquid bulk vessels	1.5 – 2.5 m.	1.0 – 1.5 m.

Bulk port

For the dry bulk berths applies that wave heights smaller than H≤1.0 m. do not affect the berthed ships at all. The lower criteria of H≤0.8 m. has not been applied here because of the fact that no purely beam waves are expected to arrive at the dry bulk berths. For the liquid bulk berths applies that the wave heights should be smaller than H≤1.5 m. for loading and unloading of the tankers.

Container port

Regarding the container terminal berths, wave heights that are considerably smaller are only allowed here: H≤0.5 m. for waves head on or stern. The berths have all been designed perpendicular to the port entrance so that only head or stern waves are to be expected at the berths.

Outside the Port

With the above defined limiting wave heights at the berth, it has to be assessed what wave conditions outside of the ports will lead to exceedance of these operational wave criteria at the berth due to waves penetrating in-port (e.g. due to diffraction and reflection).
When these wave conditions have been identified, they can be translated into the port's downtime by analyzing their chances of occurrence.

Bulk port

Additionally, for the bulk port applies that vessels cannot enter the port when the wave height exceeds H≥2.0 m. As outlined before, this is because of the fact that tugboats cannot fasten to the vessels outside the port and no stopping length in-port is available. This occurs around 7% of the time (see 3.3.4).

It will have to be assessed if (un)loading of vessels at the berths is still possible under these severe wave conditions and, if not, under what wave conditions outside the port this will be the case.

Container port

Because of the availability of the in-port stopping length, container vessels can enter the container port for a larger percentage of time than the bulk port. In 4.6.3 this was expected to be up until a wave height of H≤4.0 m. Larger wave heights only occur 0.5% of time. However, here an additional criterion plays a role: directly in lee side of the breakwater, the wave height should not exceed H≥2.0 m, otherwise tugboats cannot fasten to the entering container vessels. It has to be analyzed under which wave conditions (heights and directions) this criterion is met.

Different incident wave directions will lead to different (maximum allowed) incident wave heights outside the port, because of the breakwater layout and berth orientation.
For example, for the bulk port it could turn out that wave heights of H≤3.5 m. from 270°N pose no problems for berthed vessels, while for wave heights from 180°N this could already be the case for H≤2.5 m.

The above described wave conditions at the berths and outside of the ports will need to be assessed in order to quantify the port's downtime in percentages. The probabilities of occurrence for the different wave conditions follow from the wave tables as presented chapter 3 and annex 2. As outlined before, these tables present two computational sets (see 3.3.4): one set with wind deactivated for the directions 90°N, 105°N, 240°N and 270°N, and one set with the wind activated for all the directions. This last set (which represents the reality the most accurate) will be used here, as (wind) waves from all directions are of importance when evaluating the wave penetration in-port.

5.3.2 RELEVANT WAVE PROCESSES

In order to asses the wave penetration in-port, it is first of all important to get an inventory of different processes that affect propagating waves. Processes that can affect (transform) a wave as it propagates from deep into shallow water include:

- Refraction
- Shoaling
- Diffraction
- Dissipation due to friction
- Dissipation due to percolation
- Breaking / White capping
- Additional growth due to wind
- Wave-current interaction
- Wave-wave interactions

Within the port's boundaries, additional wave processes influence the wave propagation due to the wave-structure interactions (e.g. with breakwater and quay walls), under which reflection and transmission. The most relevant processes for the assessment of the in-port wave penetration and propagation will be described below.

Diffraction

The transfer of wave energy occurs primarily in the direction of wave propagation. However, along the (variable) crest of a propagating wave there will also be a lateral transfer of wave energy, perpendicular to the direction of wave propagation. The energy transfer will be from point of greater to lesser wave height. Diffraction has a particularly significant effect on wave conditions inside the port. For example, when waves propagate past the end of a breakwater, diffraction causes the wave crests to spread into the shadow zone in the lee of the breakwater. The wave crest orientations and wave heights in the shadow zone are significantly altered. The Coastal Engineering Manual [USACE, 2002] states that the dominant process affecting interior wave conditions is usually wave diffraction. Because of this, diffraction will certainly be of major importance, and its influence will need to be assessed.

Reflection

If there is a change in water depth as a wave propagates, a portion of the wave's energy will be reflected. When a wave hits a vertical, impermeable, rigid surface-piercing wall, essentially all of the wave energy will reflect from the wall. On the other hand, when a wave propagates over a gently sloping bottom, only a very small portion of the energy will be reflected. While the water depth in front of the mound breakwaters decreases more gradually, this is not the case for the monolithic breakwater parts. Here, a reflection coefficient close to K_r=1.0 will have to be adopted. For sloping bottoms (or structures) a smaller portion of the incoming wave energy will be reflected. Wave energy that enters a port must eventually be dissipated (which occurs at the port's interior boundaries) or scattered back out (through the port entrance). At these locations, often waves arrive as a result of diffraction and wave reflection. Diffraction and reflection together are expected to be the most important wave phenomena that determine the in-port wave conditions.

Transmission

When waves interact with a structure, a portion of their energy will be dissipated, a portion will be reflected and, depending on the geometry of the structure, a portion of energy may be transmitted past the structure. If the crest of the structure is above waterline, the wave may generate a flow of water over the structure which, in turn, regenerates waves in the lee of the structure. Also, if the structure is sufficiently permeable, wave energy may transmit through the structure, which is of importance for the rubble mound breakwaters. Transmission is especially of importance for longer waves, and less for wind waves with a small period and wave length. At the project location, especially the latter class of waves occurs, so that transmission is considered to be of secondary importance. Besides this, limit state conditions for breakwater design yield breakwater construction heights high above MSL, so that wave transmission during (much less severe) operational conditions will hardly occur and thus will not be critical. Under limit state conditions it is expected that the port can not be used at all due to other restrictions, and will have to be closed.

Refraction

If a wave crest initially has some angle of approach to the shore, generally one part of the wave crest will be in shallower water than another part. Because of differences in water depth between these two parts, there exists a speed difference between the two parts (according to $c=\sqrt{(gd)}$). This speed differential causes the crest to turn more parallel to shore.

While this is an important feature in wave transformation from deep waters to near shore, within the port it is assumed that this is less important. Some variations do exist in water depth around the port entrances, however all the port basins have been designed at a uniform depth. Besides this, the bottom contours are mostly parallel to the diffracted wave crests (container port) so that the influence of refraction stays limited.

Other phenomena

Besides the above described processes, many more processes play a role when assessing in-port waves. Examples of these amongst others are vessel-generated waves and growth of wind waves inside the port. These processes are considered of less (secondary) importance than the above described processes, and have in a first assessment not been taken into account. To combine all the processes and their effects on the wave penetration and propagation in-port, extensive simulation models will need to be applied. However, this would not be in line with the required level of detail for this preliminary wave study.

Concluding to the inventory above, for a preliminary wave study the most influential wave phenomena will be analyzed to evaluate the in-port wave climate, which are diffraction and reflection (in accordance with [HOLTHUIJSEN, 2007]).

5.3.3 PRELIMINARY ASSESSMENT

To get a first indication of the extent of in-port wave penetration and propagation, a preliminary wave analysis has been made. This analysis according to the Coastal Engineering Manual is presented in annex 4. With this visual method, the independent effects of diffraction and reflection have been assessed. It turned out that they are clearly present and cannot be neglected.

However, especially their combined effect determines the in-port wave conditions.
This more complex assessment inevitably requires the use of simulation models.
Besides this, it turned out that the current applied (visual) method for approximating in-port diffraction fell short: it lacked proper ground to assess the selected masterplan layout.
Also for this, application of a wave simulation model will be a solution.

With these arguments it is clear that use will have to be made of a wave simulation model to properly evaluate the in-port wave penetration and propagation in more detail. The main focus is on diffraction and reflection of waves inside the port geometry. These processes will certainly have to be included in the wave simulation model that will be applied. For the wave analysis the output data in calculation point P60 (see 3.3.4) will be used, as around this calculation point both port entrances are located.

5.4 WAVE SIMULATION MODEL

For simulation of wave penetration in-port, several models are available. In order to select a model, one has to have an idea what phenomena need to be included. This decision is directly related to the level of detail that needs to be applied for the study.

As was outlined in the previous paragraphs, of special importance are wave diffraction and reflection, and to a somewhat lesser extent wave refraction. Preferably all, but definitely the first two in-port wave propagation phenomena need to be included in the simulation model.

The application of the wave simulation model serves different purposes: first of all with the port layout from 4.6.3 the default scenario will be assessed. Subsequently, problems can be identified and if necessary layout improvements will be made.

5.4.1 AVAILABLE MODELS

For the application goals as described above, several simulation models are available, in accordance with Alkyon and TU Delft. These models are summarized below, with their specific characteristics.

CRESS

CRESS is a collection of small routines each containing a formula or a group of formulae that are important in coastal and river engineering. The input and output are highly standardized, and are both available in numerical and graphical form. With CRESS fast and simple approximations can be achieved: for instance diffraction in the lee side of a semi-infinite breakwater, or reflection for vertical constructions or in case of slopes. Input variables can be given, and output will be computed in one specific point.

SWAN

The SWAN (Simulating WAves Nearshore) model is a third-generation (phase-averaged) wave model for the simulation of waves in waters of deep, intermediate and finite depth. SWAN can be applied to nearshore wave modeling for port design, coastal development and management, and wave hindcasting. The model simulates wave propagation in time and space including shoaling and refraction. However, diffraction is not explicitly modelled in SWAN, and neither is reflection.

DIFFRAC-2DH

The DIFFRAC program can be used to describe wave behaviour in and around structures in water of nearly uniform depth. The phenomena accounted for are diffraction and reflection. Partial reflection is modelled at reflecting edges of the schematised basins according to user defined coefficients. DIFFRAC models the behaviour of short or long crested regular waves, and can be used to compute the wave penetration into ports. With the uniform depth as input refraction is within this model not taken into account.

PHAROS

PHAROS (Program for HARbour OScillations) is a numerical wave model for the simulation of wave agitation and wave resonance in harbour basins. The model is based on the mild-slope equation, which governs linear wave propagation over a mildly sloping bathymetry, with no restrictions to the water depth. PHAROS models the following processes: diffraction, refraction (due to depth variations and ambient currents), wave dissipation (by wave breaking and bottom friction), wave reflection and transmission.
The effects of directional spreading and energy spreading over multiple wave periods can be accounted for. Furthermore, long wave resonance and seiching of ports can be computed.

5.4.2 MODEL SELECTION

The choice for the simulation model has to be consistent with the level of detail acquired before, and the level of detail that will be necessary when evaluating the in-port wave climate in the preliminary design. Besides this, the most relevant physical phenomena need to be included in the model, which are diffraction and reflection.

The first argument rules out the PHAROS model, as it too advanced for the purpose of getting an indication of the wave penetration in-port and the expected problems. On the other hand, CRESS is for this purpose too simple, and does not provide sufficient information within the whole layout, but only in specific points according to a highly schematized situation.

As described above, the SWAN model is mainly used for the transformation of waves to the near shore area, including shoaling and refraction but no diffraction and reflection (the main phenomena). Although all these processes are included in the PHAROS model, the somewhat less extensive DIFFRAC-2DH model includes the main phenomena and is at the same time consistent with the level of detail acquired and required.

As a result from these arguments, the selected wave simulation model for evaluating the wave penetration in-port will be DIFFRAC-2DH. Additional information about the model has been included in annex 5.3.

5.5 MODEL APPLICATION

The methodology for applying the simulation model is as follows. First of all, some default runs will be made. This includes the original masterplan and breakwater layout, with the previously indicated breakwater composition with reflection coefficients approaching (but still smaller than) 1 (see 5.2). From this, the resulting problems that arise can be assessed more carefully.

If it turns that for either of the ports problems are to be expected, recommendations regarding the (breakwater) improvements will be made. These improvements can consist of an altered breakwater layout, a different breakwater composition or an optimisation of the applied breakwater type. Subsequently, these breakwater improvements will be evaluated. As outlined before, the application of the simulation model thus serves different purposes.

The above described models all work on the frequency domain, which means that a simulation needs to made for each wave frequency (f=1/T) and each direction. This results in a large number of simulations. These can be put together to result in a directional spectrum. From this, the analysis can be made regarding in-port wave penetration.

5.5.1 PROCEDURE

The DIFFRAC-2DH model follows in essence the procedure described below. In annex 1 a screenshot presenting the parameter entrance screen has included, which also visualizes the calculation procedure.

The input required for DIFFRAC-2DH consists first of all of the (fixed) wave period and the incident direction of propagation (see figure A1.11). These parameters (in combination with the available water depth) primarily determine the further calculation.

After the masterplan layout has been used as input in the model (point coordinates), values need to be specified for the reflectivity of the different boundaries. Subsequently, the calculating part of the program can generate the output. For this output, several possible options and modes can be selected, of which 'isolines' has been selected. This results in a plot with the indicated (in-port) wave heights for that specific wave condition. From these plots it can be deduced whether the maximum allowable wave height at the berth is exceeded or not.

This procedure will be applied in the following paragraphs. The layout originally adopted from the resulting final masterplan (4.6.3) will be subject of the default simulation run and will be used as input for the simulation model. In order to reduce computational times and in order to assess the individual (bulk and container) ports independently, the total layout will be split up in the two different ports. First of all, the bulk port will be assessed.
The specific assumptions made for the different port parts will be elaborated in more detail below.

5.5.2 BULK PORT

For assessing the in-port wave penetration and propagation of the bulk port, the layout is first of all imported into the model. The output will be described below with possible improvements and their resulting effect.

Input data
As input, the following parameters are required:

Wave parameters
As outlined before, DIFFRAC-2DH requires one fixed wave period as input and calculates its equations with this particular period. In order to somewhat reduce the (large) amount of possible simulation runs, a fixed wave period will be chosen for each direction. However, with this criterion a problem arises.

As can be seen from the wave data tables (3.3.4 and annex 2), the wave period varies for different wave heights and directions. From the diffraction diagrams from the Coastal Engineering Manual it was clear that the wave penetration is larger for smaller wave lengths, which would be for the waves with the smallest periods, and here also the smallest height. However, larger incident waves are more critical because their (reduced) height in-port is larger (with the same diffraction coefficient $K_d=H_i/H$). So it is expected that these last waves pose the largest problems.

In order to include all wave periods for different wave heights in one direction, as a compromise, every wave period is weighed with their specific probability of occurrence in time (see table A2.11). This has been done for each relevant direction. With this approach, the emphasis is inevitably on the smaller wave heights (because of their larger probability of occurrence). However, not all wave periods per direction need to be taken into account.

It is expected that small wave heights (e.g. H< 0.5 m.) are not likely to pose much problems to berthed vessels, so these waves have not been taken into account while determining the weighed average for the wave period. On the other hand, with large wave heights vessels cannot enter the port which needs to be closed for safety measures. So only a specific, relevant range of wave periods (which defines the port's operational conditions) has been taken into account (for wave heights around 0.5 m. ≤ H_i ≤ 3.0 m.). It is expected that the simplified assumption of using a weighed average wave period is in line with the acquired level of detail in the schematizations.

With this procedure, the emphasis is on the wave conditions for the port's operational conditions. Now, a fixed wave period can be calculated for each incident wave direction. These are summarized in the table below. Here, only the main directions are indicated where problems are to be expected when the incident wave height would be too high.

Table 5.2

Values for the weighed averaged wave period T [s] for various (main) directions

300°N	330°N	0°N	30°N
7.9 s	7.6 s	7.8 s	8.9

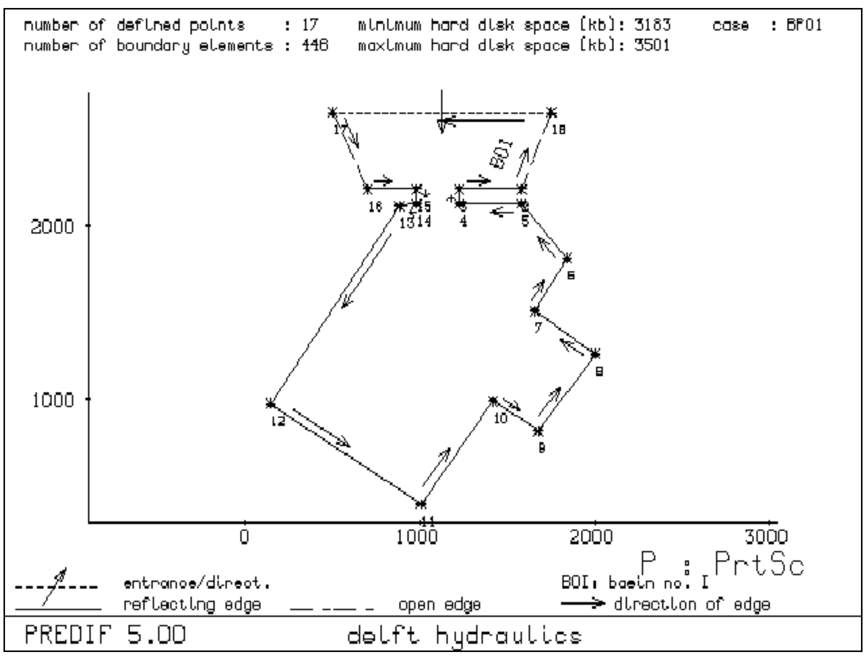
Directions other then mentioned in the table above are not likely to pose (more) problems for the berths.

Basin schematization

The next input required for the DIFFRAC-2DH model consists of the port layout. For this, point coordinates need to be specified and subsequently the connection between these points with a reflectivity coefficient. The resulting schematization is presented in the figure below.

Figure 5.2

Bulk port basin schematization in DIFFRAC-2DH with indicated point coordinates



In the schematization, several specific rules had to be taken into account. The breakwater width needs to be at least $0.5*L_{max}$ to avoid ill-posed equations [DIFFRAC-2DH MANUAL].

Because of this, the breakwater seems somewhat wide in the above presented schematization. Also, in order to reduce the calculation points (and thus calculation time), the liquid bulk jetties and port services (tugboats) breakwater have not been specified in detail in the bulk port layout. The basin form in the figure above is distorted in horizontal and vertical scales.

The outside water depth is chosen analogous to calculation point P60 at d=40 m., where the defined wave conditions from the wave tables actually do occur. The depth inside of the basin has been chosen at the entrance depth of the approach channel: here, d=35 m. This is again a rather safe assumption, as the largest part of the basin has a smaller depth. Nevertheless, by choosing this somewhat larger depth, the wave in-port wave heights give a conservative estimate by including the (larger) wave penetration in the deeper port entrance. Besides this, the differences are not that large, so this assumption has no large consequences (as the wave height in-port is mainly determined by the incident wave height).

For the reflection coefficients, beforehand determined input values will need to be used. According to [TAKAHASHI, 2002], the reflection coefficient of vertical breakwaters (caissons) is high, but nevertheless smaller than 1 because of dissipation. In accordance with [DIFFRAC-2DH MANUAL], for a first design the value of K_r=0.9 has been adopted for the monolithic breakwater types as well as the quay walls. For the rubble mound breakwaters, the value of K_r=0.5 has been adopted. Between the dry bulk terminal and the port services a beach is present. For rather steep beaches, the value of K_r=0.2 has been used because of the good wave absorbing qualities. This has been summarized in the table below.

Table 5.3

Reflection coefficients default bulk port boundaries

Edge	Description	Reflection [%]
2 – 3	Rubble mound breakwater	50%
3 – 4	Rubble mound breakwater	50%
4 – 5	Rubble mound breakwater	50%
5 – 6	Rubble mound breakwater	50%
6 – 7	Port services quay	90%
7 – 8	Port services quay	90%
8 – 9	(Steep) beach	20%
9 – 10	Dry bulk quay	90%
10 – 11	Dry bulk quay	90%
11 – 12	Rubble mound breakwater	50%
12 – 13	Caisson breakwater	90%
13 – 14	Caisson breakwater	90%
14 – 15	Caisson breakwater	90%
15 – 16	Caisson breakwater	90%
16 – 17	Open boundary edge	0%
17 – 18	Wave entrance edge	-
18 – 2	Open boundary edge	0%

Now that all data required beforehand has been determined, the default simulation runs can be made, which is done by running the processor Diffrac. The output will be described below.

Output data
The figures resulting from the simulation model runs are for all cases presented in annex 1 (figures A1.12 – A1.24). According to the simulation runs (with output in isolines for the wave height), it can be analyzed at what outside wave height the limiting operational berthing criterion at the berth is exceeded. This is facilitated by choosing a wave height analogous to the bin sizes in the wave data tables (annex 2), for example H=1.25 m., H=1.75 m., H=2.25 m. etcetera. With this it can exactly be analyzed what wave height from each direction leads to exceedance of the maximum allowable wave height at the berth.
The interpretation of the output figures is described below and concluded with a table of results which is linked to the port's downtime.

For wave direction 0°N (which is perpendicular to the bulk port entrance) it becomes clear that the occurring wave height at the dry bulk berths can become critical (see annex 1, figures A1.12 – A1.16). Even incident waves with heights smaller than the operational (un)loading wave height of H_s=1.0 m. can lead to critical situations in the most southern corner of the bulk port. At this location, wave heights larger than the incident wave height occur because of reflectional processes. Waves become trapped in this corner and are almost fully (K_r=0.9) reflected off the dry bulk berths. A standing wave pattern develops at this location, which is very unfavourable for these berths. At the liquid bulk berths no problems are expected according to these runs: the wave heights at these berth locations stay well under the maximum value of H_s=1.5 m. with incident waves up to H_s=1.75 m. However, if the wave height is further increased, the wave pattern in the corner of the dry bulk terminal and the rubble mound breakwater can lead to wave heights at the liquid bulk berths that exceed the operational berthing conditions.

When assessing waves from 330°N, it can be seen that virtually no waves higher than 1.0 m. arrive at the dry bulk terminal with an incident wave height of H_s=2.25 m (see A1.17 – A1.19). For the liquid bulk no problems at all are identified, as they are located even more sheltered from the incoming wave direction. With this approach angle, the waves are directed at the beach between the port services and the dry bulk terminal, where they are dampened considerably. Even for incident wave heights up to 3.25 m. the operational (un)loading criteria are met.

For waves from the direction of 300°N even less problems are expected. With this incident direction, the effective bulk port entrance width is already reduced considerably (as emphasized earlier), which limits the in-port wave penetration and propagation. However, with this wave direction the port services are the most susceptive to the incoming waves.

Waves from 30°N are directed more head on the liquid bulk berths (figures A1.21 – A1.24). However, due to diffraction the wave height arriving at the liquid bulk berths has already decreased in height considerably. With incident waves of H_s=2.25 m. the wave height at the location of the liquid bulk jetties is still somewhat limited, however at the location of the crude oil berth the operational wave criteria are exceeded. This is due to reflection from the inside of the north-western bulk port caisson breakwater with a high coefficient of reflectivity. However for most wave conditions (up to H_s=1.75) the berthing conditions are favourable.

These results have been summarized in the table below.

Table 5.4

Maximum allowable wave heights outside the port for which the operational berthing criteria are not exceeded

Directions	300°N	330°N	0°N	30°N
Dry bulk berths	-	H<2.25 m.	H<0.75 m.	-
Liquid bulk berths	-	-	H<1.25 m.	H<1.75 m.

This table can be extended with the limiting wave criteria at the berth and the probabilities of exceedance for the specific above-mentioned wave conditions (from wave data table for P60 in annex 2). Subsequently, the port's downtime can be determined. This has been presented in the table below.

Table 5.5

Probabilities of exceedance for limiting operational wave criteria to determine the downtime

Directions	300°N	330°N	0°N	30°N
Dry bulk berths				
Wave criterion at berth	H≤1.00 m.	H≤1.00 m.	H≤1.00 m.	H≤1.00 m.
Outside wave height $H_{outside}$	-	H<2.25 m.	H<0.75 m.	-
% of exceedance of $H_{outside}$	-	0.14%	1.77%	-
Liquid bulk berths				
Wave criterion at berth	H≤1.50 m.	H≤1.50 m.	H≤1.50 m.	H≤1.50 m.
Outside wave height $H_{outside}$	-	-	H<1.25 m.	H<1.75 m.
% of exceedance of $H_{outside}$	-	-	0.87%	0.89%

The downtime of the dry bulk berths by exceedance of limiting operational wave conditions amounts to 0.14+1.77=1.91% of time. For the liquid bulk berth this happens for 0.87+0.89=1.76% of time. This is around 1 – 1.5% larger than assumed beforehand in 4.6.2.

These calculated percentages are on the higher side. This is because of the (conservative) assumption that the lower bin-limit is chosen as limiting wave criterion, but in reality the critical wave height is in between the two bin-limits (e.g. between 1.25 m.<H<1.75 m., but the value for H<1.25 has been chosen). With this, a deviation to the real probability of exceedance is introduced; however, it is a safe starting assumption. Besides this, the deviation remains limited. This actual deviation to the real downtime percentage decreases with increasing wave height because of the smaller probability of occurrence. For example, the limiting wave height at the liquid bulk berth for direction 30°N is H<1.75 m. which is exceeded for 0.89% of time. The real percentage of exceedance is between this wave height and the next bin limit: H<2.25 m. which is exceeded for 0.34% of time (a deviation of 0.55% at maximum). Assuming an exponential decrease in probability of exceedance (see wave data tables, annex 2), this means that the actual probability of exceedance will be somewhere in between the two values, and more somewhat towards 0.34%).

However, these percentages of downtime remain limited and are acceptable as downtime for the berths. It is emphasized that the total downtime consists of more items than only the downtime for the berths, and there is some overlap in between: for example the largest contribution to the downtime is caused by the fact that tugs cannot fasten to the vessels outside the port during wave heights larger than H>2.0 m. In 4.6.2 this was determined to be 7%. In this case also the operational wave conditions for direction 330°N at the dry bulk berths would be exceeded.

Identified problems
From the simulation model runs two main problems have been identified.

1. With waves from 0°N, the operational berthing criteria at the dry bulk berths are exceeded nearly 2% of the time, which is the largest portion of time. This is due to the fact that the incident waves are directed towards the berths. Besides this, the incident waves are reflected in the corner of the dry bulk terminal which leads to a standing wave pattern, and wave heights even larger than the incident wave height.

2. Waves larger than H_s=1.75 m. from 30°N are reflected against the inside of the north-western caisson bulk port breakwater. Due to the large reflection coefficient of the vertical wall breakwater (K_r=0.9), these waves only slightly decrease in height. At the location of the crude oil berth, this leads to exceedance of the operational wave criteria at the berth for nearly 1% of time. Because of the fact that large quantities of crude oil will be transshipped here, too much unavailability of this berth is simply not allowed.

The above presented percentages may not appear to be very large. However because of the large quantities of throughput involved, every single percent of downtime less could lead to large profits for the port. This is especially true for the bulk port, where the downtime is in the range between 5 – 10%, where 10% is just around the limit of acceptable port downtime (see 4.6.2). It is clear that in this small range, every single percentage counts. Because of this, the following is aimed at minimizing the port's downtime.

Possible improvements
In order to increase the amount of wave energy dissipation in-port, measures need to be taken. An elementary solution to this is creating more wave dampening in the bulk port. This would be a good solution, especially for problem 2. If the inside of the north western breakwater dampens the incoming waves more (instead of almost fully reflecting them), the occurring wave height at the crude oil berth will be decreased. Constructing measures to increase wave absorption in-port should also have a (secondary) positive effect on the overall in-port wave propagation. This will be adopted in the improved model runs.

For problem 1, adding more in-port wave dampening measures will only have a small positive influence on the availability of the berths. This is because of the fact that waves from 0°N are directed towards the berths and are not affected by wave dampening measures before they reach these berths. Other solutions for this would be (breakwater) layout modifications. However, before applying such rigorous measures, first of all simulation runs will be made with additional wave dampening measures in-port to assess the influence of this on the downtime.

In order to incorporate these additional wave dampening measures in the design, several alterations will have to be made compared to the default alternative. These improvements can be inputted in the simulation model by modifying the reflection coefficients of the breakwaters (and thus in fact changing their detailed composition).

These new, improved reflection coefficients are presented in the table below. For the vertical composite breakwater special construction measures will be required. For low-reflectivity caissons, a (preliminary assumed) reflection coefficient of K_r=0.6 has been adopted.

According to [MARTINEZ *et al.*, 2010] and [TAKAHASHI, 2002], this reflection coefficient can actually be achieved for a specific range of waves, which the limiting operational berthing wave conditions are. Although this value of K_r=0.6 seems to be chosen somewhat arbitrarily, it indicates the possibilities for wave absorption by vertical wall breakwaters. It will be elaborated later on if this exact value is absolutely required or if somewhat looser criteria suffice.

Besides this, some of the other reflectivity coefficients have also been reduced: with minimal constructional alterations a lower reflectivity can be achieved [VERHAGEN *et al.*, 2009], [DIFFRAC-2DH MANUAL]. For example, a slope has been applied at the north-eastern edge of the dry bulk terminal, as this part is not used as berth. The improved reflection coefficients, in contrast to the original design, are presented in the table below.

Table 5.6

Reflection coefficients improved bulk port boundaries

Edge	Description	Reflection [%]
2 – 3	Rubble mound breakwater	45%
3 – 4	Rubble mound breakwater	45%
4 – 5	Rubble mound breakwater	45%
5 – 6	Rubble mound breakwater	45%
6 – 7	Port services quay	80%
7 – 8	Port services quay	80%
8 – 9	(Steep) beach	20%
9 – 10	Dry bulk quay (slope)	45%
10 – 11	Dry bulk quay	85%
11 – 12	Rubble mound breakwater	45%
12 – 13	Caisson breakwater	60%
13 – 14	Caisson breakwater	60%
14 – 15	Caisson breakwater	60%
15 – 16	Caisson breakwater	60%
16 – 17	Open boundary edge	0%
17 – 18	Wave entrance edge	-
18 – 2	Open boundary edge	0%

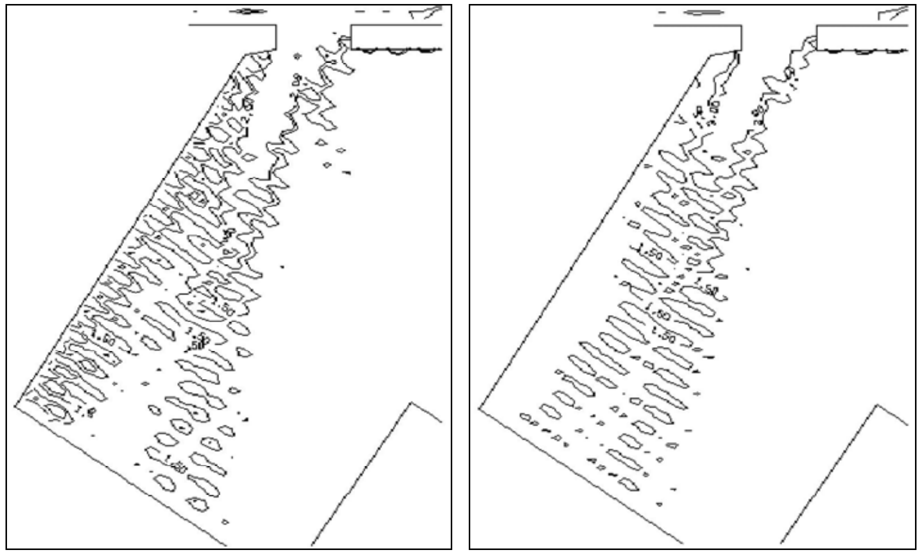
Improved bulk port simulation runs

Now that the optimized reflection coefficient as outlined above have been included in the simulation model, new simulation runs will be made. The wave climate (parameters) is kept the same as in the previous model runs. As emphasized before, these wave absorbing measures will especially have their (expected) positive influence on the availability of the crude oil berth. This will be assessed at first.

The output of the improved model runs are included in annex 1 (figures A1.25 – A1.29). From the figures below (for the wave direction of 30°N) it becomes clear that by constructing low-reflectivity caissons, dissipation of wave energy is increased and wave penetration due to reflection is decreased.

Figure 5.3

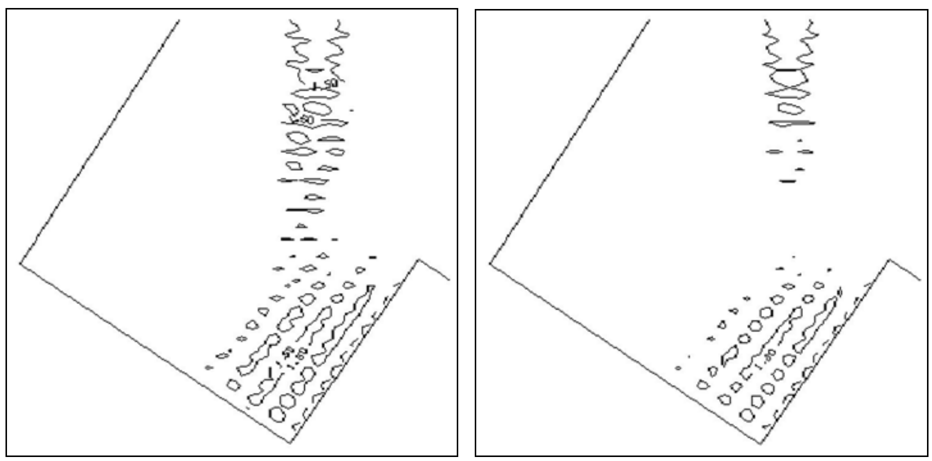
Bulk port basin
Waves from 30°N with
H=2.75 m. isolines 1.5 – 2 m.
Left: default configuration
Right: with improved low-reflectivity measures



When assessing the influence of the wave dampening measures from the direction of 0°N, also some (small) improvements are directly visible, see the figure below.

Figure 5.4

Bulk port basin
Waves from 0°N with
H=1.75 m. isolines 1 – 1.5 m.
Left: default configuration
Right: with improved low-reflectivity measures



The positive results of the breakwater construction improvements on the port's downtime in term of wave heights are summarized in the table below.

Table 5.7

Probabilities of exceedance of outside wave heights that lead to limiting operational berthing conditions for downtime analysis

Directions	300°N	330°N	0°N	30°N
Dry bulk berths				
Wave criterion at berth	H≤1.00 m.	H≤1.00 m.	H≤1.00 m.	H≤1.00 m.
Outside wave height $H_{outside}$	-	H<2.25 m.	H<1.25 m.	-
% of exceedance of $H_{outside}$	-	0.14%	0.87%	-
Liquid bulk berths				
Wave criterion at berth	H≤1.50 m.	H≤1.50 m.	H≤1.50 m.	H≤1.50 m.
Outside wave height $H_{outside}$	-	-	H<1.75 m.	H<2.25 m.
% of exceedance of $H_{outside}$	-	-	0.40%	0.34%

With application of the additional wave absorbing measures in the breakwater composition, it can be concluded that positive results are achieved.

The crude oil berth is now available a larger portion of time: the total availability is now 99.80% instead of 97.61% in the original design, even with incident waves of H_s=2.75 m. from 30°N. However, with this incident wave height the diffracted wave heights at the liquid bulk berths (jetties) will become critical. The downtime of the liquid bulk berths is now only 0.40+0.34=0.74% of time, instead of 1.76% of time in the original design.

For the direction of 0°N, incident waves of H_s=1.25 m. still lead to wave heights at the dry bulk berths of around H=1.0 m. However, wave heights are not increased anymore as a result of lower reflectivity in the southern bulk port corner: from the simulation runs it is clear that waves with a height of H=1.5 m. are not visible near the berths for this condition. Also for these berths the downtime decreases somewhat: only 0.14+0.87=1.01% instead of 1.91% of time in the original design. The fact that waves larger than H_s=1.25 m. lead to exceeded limiting operational berthing conditions does not pose much of a problem: this happens for only 0.87% of the time (see chapter 3: wave table P60). It is assumed that this can be allowed, especially when reviewing the amount of shipping traffic arriving at the dry bulk terminal which is not that large (only 1 ship/day).

The wave reflection at the port entrance will not be a severe problem to navigation: the port entrance is wide enough and enclosed by breakwaters that dissipate wave energy (the low reflectivity caisson breakwater and the rubble mound breakwater). As emphasized earlier, for port entrance the main criterion still is that tugs must be able to fasten to the vessels outside the breakwaters. For this, wave heights need to be H≤2.0 m, which is exceeded for 7.1% of time (also here, some overlap in downtime with the limiting operational berthing conditions is included for the directions 330°N and 30°N). On this percentage the bulk port design has been based (with no in-port stopping length), for which it was already concluded that certain unavailability was allowed. The total bulk port's downtime (including additional factors as outlined in 4.6.2) amounts to 9.7%.

5.5.3 CONTAINER PORT

For evaluating the wave conditions in the container port a different approach will be used. As described in the preliminary wave study, at the container port entrance (which is enclosed by monolithic breakwaters) navigational problems because of severe wave reflection can be expected. It will have to be assessed to what extent this actually does occur, and in what downtime this results. Subsequently, individual (potential problematic) berths will be assessed.

First of all, the originally designed container port entrance will be used as main focus of the first part of the in-port wave penetration and propagation study. Subsequently, if necessary, additional improvements will be made and analyzed by new model simulation runs.
Next up, the container terminal berths deeper in-port will be evaluated and. For the container port applies that a larger range of wave heights is of importance (around 0.5 m. ≤ H ≤ 4 m.). This is because of the smaller allowed wave height at the berth, but the (expected) larger allowed wave heights for port entrance.

Container port entrance
First of all, the effects of wave reflection against the caisson breakwaters at the container port entrance will be evaluated.

Input data
As input for the default model run, the following parameters are required:

Wave parameters
Analogue to the methodology as outlined in 5.5.2, in order to include all wave periods for different wave heights in one direction, as a compromise, the weighed average of the wave period depending on the probability of occurrence in time has been used (see table A2.11) for each relevant direction. However, also here, not all wave periods per direction need to be taken into account.

It is expected that small wave heights (e.g. H< 0.75 m.) are not likely to pose much problems to vessels entering the port, so these waves have not been taken into account while determining the weighed average for the wave period. This is in line with the results from the preliminary wave assessment (annex 4), where it became clear that wave heights past the container port entrance would decrease even more (but the exact extent could not be determined). On the other hand, with (too) large wave heights (H>4.0 m.) vessels cannot even enter the port. During these wave conditions the wave heights in the lee of the breakwaters is still too large for tugboats to fasten to the vessels (H>2.0 m.).

So also here, only a specific, relevant range of wave periods (which defines the port's operational conditions) has been taken into account (for wave heights around 0.75 m. $\leq H_i \leq$ 4.0 m.). It is expected that the simplified assumption of using a weighed average wave period is in line with the acquired level of detail in the schematizations.

With this procedure, the emphasis is on the wave conditions for the port's operational conditions. These values are presented in the table below. Results are summarized in the table below. Again only the main directions (that are expected to give problems) are evaluated. Other waves propagate out of the domain (e.g. waves from 270°N).

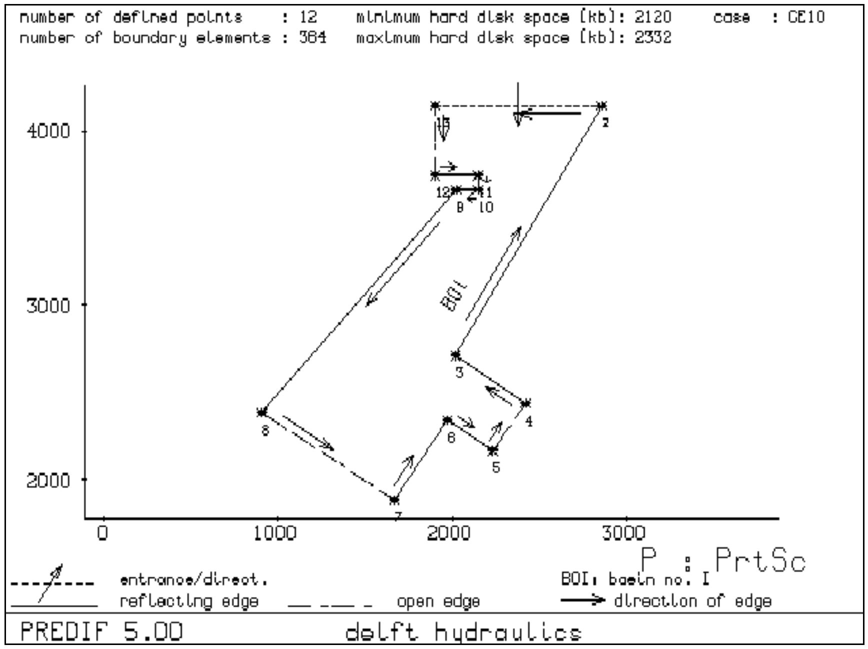
Table 5.8
Weighed averages for wave period T [s] for different directions

300°N	330°N	0°N	30°N	60°N
7.7 s	7.2 s	7.4 s	8.2 s	6.1 s

Entrance schematisation
For the container port entrance, the following schematisation is used.

Figure 5.5

Schematization container port entrance in DIFFRAC-2DH



The outside water depth is again chosen analogue to calculation point P60 at d=40 m., where the defined wave conditions from the wave tables arise. The depth inside of the container port entrance has been chosen at the actual bottom level which is around d=40 m. This is again considered to be a rather safe assumption (see 5.5.2).

The reference simulation runs according to the original design will be made with reflection coefficients according to the table below. The north-eastern and north-western breakwaters have reflection coefficients K_r=0.9 in these simulation runs, according to the reasoning as outlined for the bulk port. The rubble mound breakwaters again have reflection coefficients of K_r=0.5 according to the original design. At the end of the basins, a beach is present. This beach (where also wadis discharge) is expected to have a very low reflection coefficient. In a first estimate, the same beach reflection coefficient as for the bulk port can be applied, but for calculation simplicity, here an open edge is defined. In this first assessment, the heads of the container terminal are expected to have a reflection coefficient like the quays, which is again K_r=0.9.

Table 5.9

Reflection coefficients default container port entrance boundaries

Edge	Description	Reflection [%]
2 – 3	Caisson breakwater	90%
3 – 4	Rubble mound breakwater	50%
4 – 5	Basin (open edge)	0%
5 – 6	Container berths	90%
6 – 7	Container terminal head	90%
7 – 8	In-port container (open)	0%
8 – 9	Caisson breakwater	90%
9 – 10	Caisson breakwater	90%
10 – 11	Caisson breakwater	90%
11 – 12	Caisson breakwater	90%
12 – 13	Open boundary edge	0%
13 – 2	Wave entrance	-

Now that the input has been determined, the default model runs will be made.

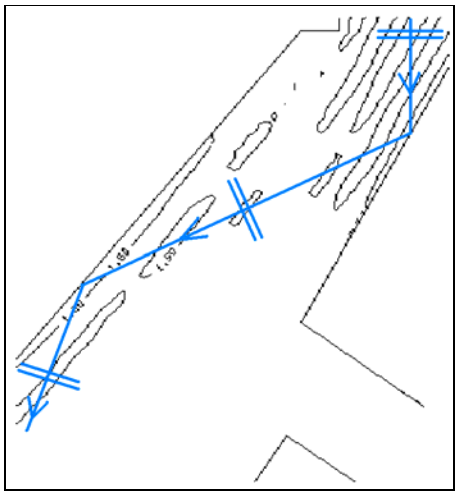
Output

The figures resulting from the default model simulation runs are included in annex 1 (figures A1.30 – A1.41). The interpretation of the output, analogues to the outlined description in the bulk port evaluation, is described below.

With incident waves from the direction 0°N, it can be clearly seen that waves are reflected from the bulk port caisson breakwater further into the container port. Here, they are again reflected against the inside of the north-western container port breakwater. This is clearly visible in the figure below. This leads to wave height larger than the incident wave height.

Figure 5.6

Direction of reflected wave crests across the container port entrance for direction 0°N

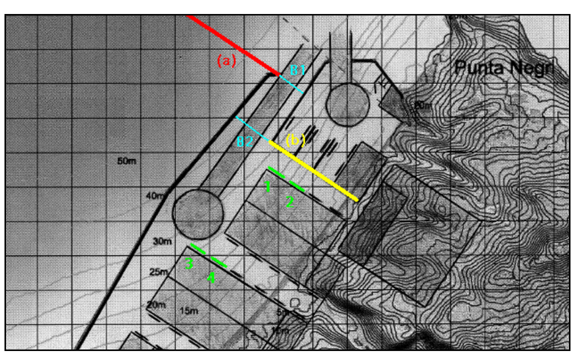


Reflection will be an important aspect to take into account when reviewing the wave conditions at the entrance. There is considerable variation in isolines, which indicates standing waves across the container port entrance. At locations close to the breakwaters, wave heights higher than the incident wave heights occur due to (almost complete) reflection. The direction of the (reflected) wave propagation in-port is clearly visible in the simulation runs.

What attracts attention is the wave climate at berths (1) and (2) (see the figure below, adopted from annex 4), which have a high availability with incident waves from this direction. Along the quay of berths (1) and (2) practically no exceedance of operational berthing conditions occur with incident wave from 0°N up to H≤1.75 m.

Figure 5.7

Indication of critical berths 1 – 4 for wave penetration assessment (adopted from annex 4)

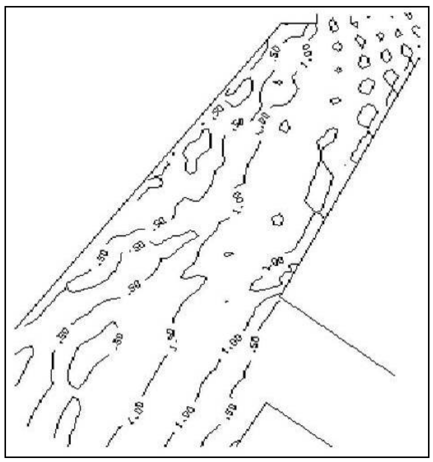


However, first of all the emphasis is primarily on the entrance wave conditions where tugs must be able to tie up to the vessels in the lee of the breakwaters with waves heights $H \leq 2.0$ m. This is the when waves are equal to or smaller than H=1.75 m. The exact results will be summarized in tables below, in combination with their probabilities of occurrence to evaluate the port its downtime.

When reviewing the output for waves from 30°N, a quite different wave pattern can be identified: the constant in-port wave penetration is clearly visible. With this direction of origin, incident waves follow the entrance almost completely. In combination with the high reflection coefficient of the breakwaters, the waves propagate almost unhindered further in-port. This can be clearly seen in the figure below, where the in-port propagating wave height does not decrease much.

Figure 5.8

Container port entrance following waves (30°N) only slightly decrease in height



This scenario will be of special importance later on when assessing berths (3) and (4) deeper in-port (see figure 5.7). When reviewing the container port entrance, it appears that incident waves from 30°N with wave heights larger than H=1.75 m. will lead to a scenario where the wave height in the lee of the breakwaters is too high for tugs to tie up (H>2.0 m.).

Wave heights larger than H=1.75 m. do not occur at all from 60°N, and smaller waves only occur for a very small portion of time (0.07%). It is expected that these waves will not be critical when considering other incident directions and heights that require improvements. Because of this, these waves are discarded.

The output figures for incident waves from 330°N show critical results: they lead to exceedance of operational entrance conditions already for wave with heights of H=1.25 m. Due to the almost complete reflection against the entrance surrounding breakwaters, a standing wave pattern develops with wave heights larger than H=2.0 m. This is visible in the large variation between isolines across the entrance (difficult for navigation). This is the most critical scenario up until now and will have to be improved.

For completion, also waves from 300°N will be evaluated. Although waves from 330°N appeared to be the most critical, this direction is only a small deviation from that direction. From the plot it appears that with incident waves from this angle practically no waves occur in-port.

The smaller waves that do penetrate more in-port arrive there because of a combination of diffraction and reflection around the tip of the north-western container terminal breakwater. A more detailed plot showing the port entrance indicates that the waves are reflected at the north-eastern breakwater, where they (largely) propagate out of the domain. However, in front of the entrance a standing wave pattern could develop because of the high reflectivity and the almost perpendicular angle of approach. Nevertheless, it is visible that waves smaller than H=2.75 m. do not pose any problems in the port entrance.

The above described results regarding the wave conditions at the entrance are summarized in the table below. Here, the emphasis is on the wave climate directly in the lee of the port entrance, where the wave height should not exceed H=2.0 m: otherwise tugboats cannot fasten to the vessels.

Table 5.10

Outside wave height criteria for port entry where tugs can fasten to the vessels (in the lee of the breakwaters H≤2.0 m.)

Directions	300°N	330°N	0°N	30°N	60°N
Port entrance	H<2.75 m.	H<1.25 m.	H<1.75 m.	H<1.75 m.	-

With these wave criteria, the downtime of the container port because of the fact that no entrance is possible can be assessed. This is presented in the table below which has been extended with probabilities of exceedance for the above-mentioned wave conditions (see wave data table for P60 in annex 2) and the limiting operational wave conditions.

Table 5.11

Probabilities of exceedance for port entry criteria for downtime analysis

Directions	300°N	330°N	0°N	30°N	60°N
Tugboats fastening	H≤2.00 m.	H≤2.00 m.	H≤2.00 m.	H≤2.00 m.	H≤2.00 m.
Outside wave $H_{outside}$	H<2.75 m.	H<1.25 m.	H<1.75 m.	H<1.75 m.	-
% of exceedance H_o	0.73%	0.51%	0.41%	0.89%	-

With these probabilities of exceedance for each relevant direction, the container port's entry downtime can be calculated. This downtime amounts to 0.73+0.51+0.41+0.89=2.54%. Analogues to the explanation in the previous paragraph, also here the downtime percentages are on the higher side. Nevertheless, for a container port with a large throughput of 18MTEU and a shipping traffic of 1 vessel per hour, this specific downtime contribution is rather large and will have to be improved.

Identified problems

From the simulation runs above several problems can be identified:

1. Due to the highly reflective caissons at the entrance with K_r=0.9, there is a lot of wave reflection, which especially for the directions 330°N and 0°N (and for 330°N in front of the entrance) causes problematic standing waves. This inevitably leads to a rough navigational climate with strongly changing wave heights across the container port entrance. This could lead to problems when fastening tugs to the entering vessels and results in a downtime of over 2.5% of time.

2. Waves from direction 30°N are almost completely aligned with the entrance, and propagate largely unhindered in-port. Their height does not decrease much during propagation, so that wave heights deeper in-port could still be (too) high for operational berthing conditions (especially berths (3) and (4)). This will have to assessed later on in more detail, when evaluating the downtime of individual berths.

It can be concluded that the original design with ordinary vertical wall breakwaters will have to be improved in order to minimize the port's entry downtime.

Possible improvements

The elementary solution to problem 1 (and 2 but to lesser extent) is again adding measures to enhance in-port wave dampening. Dissipation of wave energy must be accomplished as much as possible to realize calm in-port entrance (and berthing) conditions.
The use of low-reflectivity caissons will be a very good solution: the inside of the north western container terminal breakwater and the outside of the bulk port breakwaters will be constructed as a low-reflectivity caisson. These caissons can have reflection coefficients as low as K_r=0.6 (see explanation for the bulk port). This largely prevents waves from propagating further in-port due to reflection, which is an absolute necessity.

As outlined in 5.5.2, the reflection coefficient for some other boundary elements can be improved with minimal constructional measures. With these alterations, the reflection coefficient for the rubble mound bulk port breakwater will be somewhat lower: K_r=0.45, analogous to the bulk port assessment. Subsequently, the head of the container terminal block will be used for wave dissipation: it is assumed that here also a lower reflectivity of 45% can be achieved. The container berths itself can be equipped with low reflectivity screens and the reflection coefficients reduce somewhat. An even more effective construction methodology would be the construction of berths as a deck on piles, but this rigorous measure has not (yet) been applied in the design. The next input for the coefficients of reflection that will be used is summarized in the table below.

Table 5.12

Reflection coefficients improved container port entrance boundaries

Edge	Description	Reflection [%]
2 – 3	Caisson breakwater	60%
3 – 4	Rubble mound breakwater	45%
4 – 5	Basin (open edge)	0%
5 – 6	Container berths	85%
6 – 7	Container terminal head	45%
7 – 8	In-port container (open)	0%
8 – 9	Caisson breakwater	60%
9 – 10	Caisson breakwater	60%
10 – 11	Caisson breakwater	60%
11 – 12	Caisson breakwater	60%
12 – 13	Open boundary edge	0%
13 – 2	Wave entrance	-

Solutions to problem 2 could include altering the breakwater layout to allow for less wave penetration. This is however considered as a measure of last resort. First, the effects of the application of low-reflectivity caissons will be evaluated which could already turn out to be sufficient in creating calm port entry conditions.

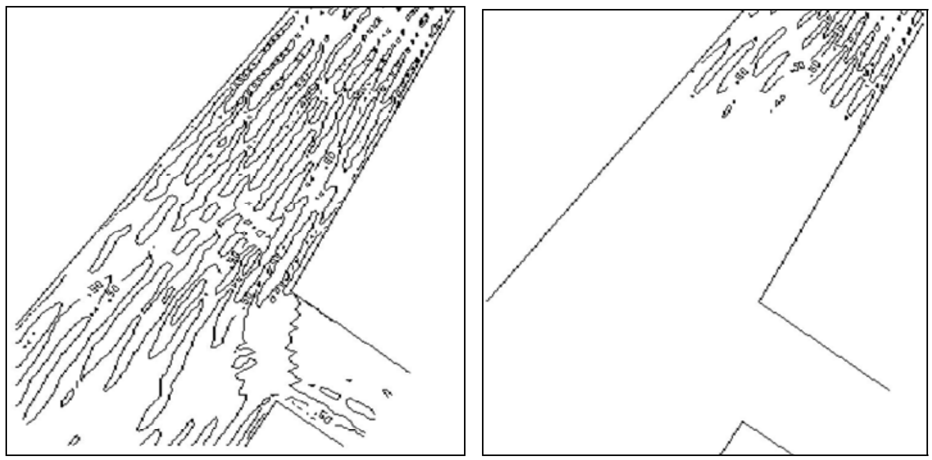
Improved container port entrance simulation runs

With the additional wave dampening as outlined in the table above, it can be concluded that there is clearly less wave action inside of the port due to reflectional wave processes. From the direction 0°N it is clear that waves reflected from the bulk breakwater have decreased so much in height (with incident wave height 1.25 m.) that no more waves higher than H=0.5 m. occur at some distance from the entrance (see figures A1.42 – A1.48). Even with incident wave heights up to H≤2.75 m. tugs can fasten to the vessels in the lee of the breakwaters. It can also directly be seen that berth (1) meets the operational berthing requirements due to the increased amount of wave dampening: the difference is again substantial.

From the critical direction 330°N, which was of special importance for the assessment of the in-port wave climate due to excessive reflection, it becomes clear that at the entrance a very large portion of wave energy is dissipated directly by the breakwaters. The comparison with the case with high reflectivity shows very large differences: directly behind the port entrance much calmer conditions are achieved (see figures below).

Figure 5.9

Container port entrance
Waves from 330°N with H=1.75 m. isolines 0.5 – 1 m.
Left: default configuration
Right: with improved low-reflectivity measures



Incident waves with a height of H=2.25 m. also do not pose problems for (all of) the berths. At the entrance itself it is clear that there inevitably is wave action present, and a particular pattern develops. The standing wave pattern could lead to hindrance to entering vessels. However wave dampening measures reduce this as much as possible, and in the lee of the breakwaters tugs can fasten to the vessels up to wave heights of H≤2.75 m.

This will also be the case for the scenario with waves from 300°N. However, it was determined from the previous assessment that these waves are not critical when assessing the berths. The wave penetration analogue to the above is even more obstructed, and waves can move out of the domain. Nevertheless, because the incoming wave angle is almost perpendicular, again a standing wave pattern can develop which is critical for entering vessels. However, these waves do not occur for a large portion of time.

As outlined before, waves from 300°N where not critical, as they propagated mostly out of the domain: from this angle practically no waves occur in-port. In front of the entrance there is some wave reflection present. Nevertheless, it is visible that outside incident waves smaller than H=3.25 m. from this direction do not pose any problems in the port entrance for tugboats tying up to the vessels.

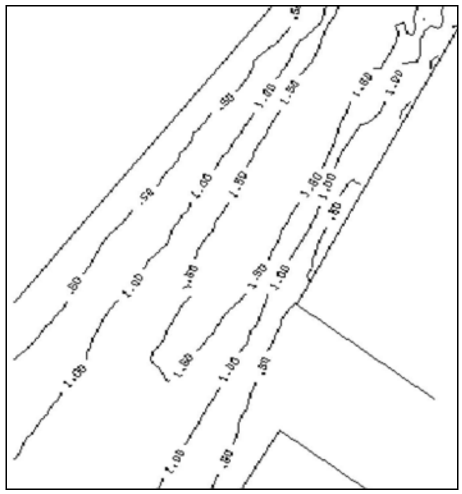
Next up, the waves from 30°N (the entrance following direction), are again assessed with the improved container port entrance. It appears that also here the wave penetration is somewhat more limited with the new low-reflective caissons. Nevertheless, deeper in-port still wave heights are noticeable with heights of H=2.0 m. with an incident wave height of H_i=2.25 m.

Although there are some minor improvements visible, still the same entrance criterion applies as before: With waves larger than H=1.75 m., tugs cannot fasten to the vessels.

This direction has to be carefully assessed during the berth analysis for the container berths, as even with dampening measures waves do not decrease that much in height (because of the incident wave direction parallel to the entrance).

Figure 5.10

Improved container port entrance
Waves from 30°N with H=1.75 m. isolines 0.5 – 1.5 m.



A more extreme case has been run with waves from 30°N and a wave height of H= 2.75 m. to assess the effect on deeper in-port locations of larger wave heights (which only exceeded 0.12% of time). It appears that at the lower end of the port boundary still waves exist with a height of 2 m. and higher. From this it is again evident that waves from this direction do not decrease much in height and will inevitably lead to downtime of berths 3 and 4.
This requires a more detailed assessment.

The results described above are summarized in the table below, in combination with their respective chances of exceedance and tugboat operational criteria.

Table 5.13

Probabilities of exceedance for improved port entry criteria for downtime analysis

Directions	300°N	330°N	0°N	30°N	60°N
Tugboats fastening	H≤2.00 m.	H≤2.00 m.	H≤2.00 m.	H≤2.00 m.	H≤2.00 m.
Outside wave $H_{outside}$	H < 3.25 m.	H < 2.75 m.	H < 2.75 m.	H < 1.75	-
% of exceedance H_o	0.37%	0.08%	0.12%	0.89%	-

The total port entry downtime has been reduced to: 0.37+0.08+0.12+0.89=1.46% instead of 2.54% without wave dampening measures. It is concluded that these measures yield very positive results for the calmness of the in-port wave climate. For almost all scenarios (98.54% of time), directly in the lee of the breakwaters the wave height is decreased enough so that tugs can fasten to the vessels (H<2.0 m.). Just outside the entrance a standing wave pattern can develop, but the exact extent of this would have to be assessed in more detail in a final design.

Subsequently, some new runs will be made for evaluating the more in-port wave conditions at several critical berth locations. The improved container port entrance will be used as input when assessing the overall container port (with added simplifications as outlined below).

Container port berths

In the foregoing analysis, the emphasis was on the container port entrance assessment. However, from these simulation runs also the availability of critical berths 1 and 2 (see figure 5.7) can be assessed as they are located within this calculation domain. From the improved simulation runs, the following criteria for berth 1 (the most critical berth of the two) have been deduced.

Table 5.14

Limiting wave criteria for the availability of berth 1 for downtime analysis

Directions	300°N	330°N	0°N	30°N	60°N
Berth criteria	H≤0.50 m.	H≤0.50 m.	H≤0.50 m.	H≤0.50 m.	H≤0.50 m.
Outside wave $H_{outside}$	-	H<2.75 m.	H<2.25 m.	H<2.75 m.	-
% of exceedance H_o	-	0.08%	0.22%	0.13%	-

The downtime of berth (1) is 0.08+0.22+0.13=0.43%, which is very low for an individual berth. For the remaining 99.57%, the berth can be used for (un)loading of vessels.
The downtime for the more sheltered berth (2) is even smaller. It can be concluded the wave dampening measures result in a very small percentage of downtime for these individual berths, which is definitely acceptable. Berths located further in the basin have subsequently even smaller downtimes.

For the assessment of the other critical berths (3) and (4), a new schematisation is required. The wave heights at these locations will be assessed according to the most critical wave conditions and directions that resulted from the previous container port entrance assessment.

Wave parameters

The wave parameters are the same as outlined before at the container port entrance.

Basin schematisation

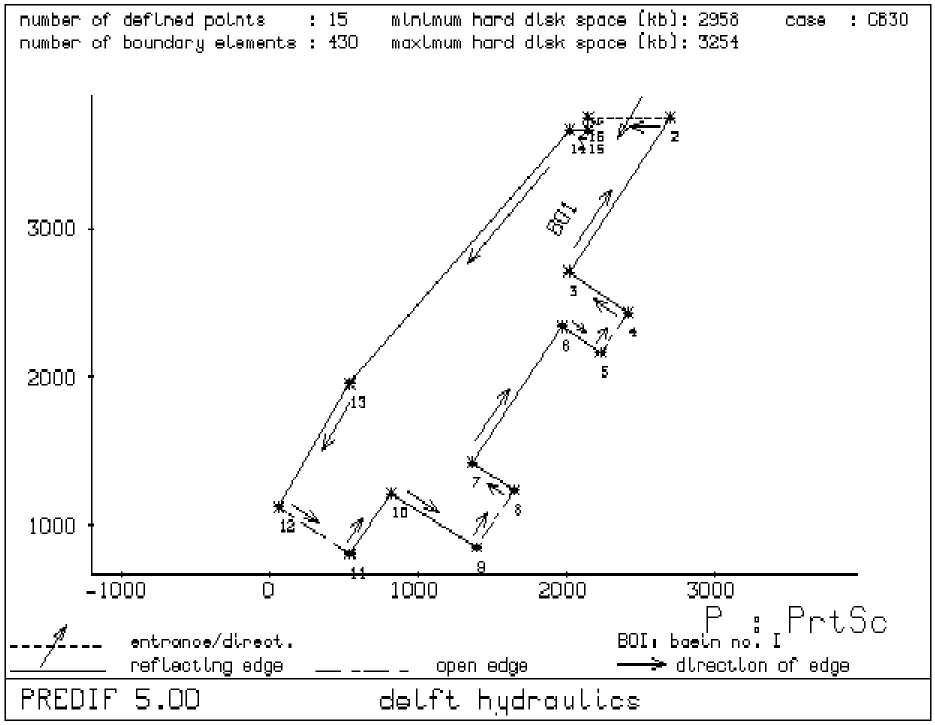
For the container port berth assessment, the schematisation presented in the figure on the next page has been used.

The outside water depth is again chosen analogous to calculation point P60 at a depth of 40 m. The basin depth in this schematization will have to be chosen somewhat smaller, as the (previously used) depth of 40 m. is not in line with the actual design depth. While the average depth for this basin is around 30 m., a somewhat more safe value of d= 35m. has been used as model input to include the effects of the deeper entrance (which results in a larger in-port wave penetration). According to several comparison simulation runs, this seems to be a safe assumption.

In order to reduce the number of computational points, the basins where the container berths are situated are schematised with an open edge, representing the further propagation of the waves (edges 4 – 5, 8 – 9 and 11 – 12). It is expected that this only has a minor influence on the schematisation, as at the end of the basins a wave energy dissipating beach is present (K_r=20%).

Figure 5.11

Schematization container port berths in DIFFRAC-2DH



For the evaluation of the wave heights at the berths, several runs will be made. These runs include the previously determined improvements of the low-reflectivity caissons.
The reflection coefficients are summarized in the table below.

Table 5.15

Reflection coefficients improved container port boundaries

Edge	Description	Reflection [%]
2 – 3	Caisson breakwater	60%
3 – 4	Rubble mound breakwater	45%
4 – 5	Basin (open edge)	0%
5 – 6	Container berths	85%
6 – 7	Container terminal head	45%
7 – 8	Container berths	85%
8 – 9	Basin (open edge)	0%
9 – 10	Container berths	85%
10 – 11	Container terminal head	45%
11 – 12	In-port edge	0%
12 – 13	Caisson breakwater	60%
13 – 14	Caisson breakwater	60%
14 – 15	Caisson breakwater	60%
15 – 16	Caisson breakwater	60%
16 – 2	Wave entrance	-

Now that the input again has been determined, the processor Diffrac has been run which leads to the following output.

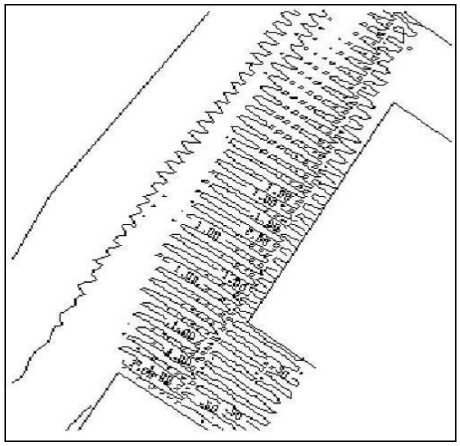
Output and identified problems

From the figures A1.49 – A1.51 in annex 1 it can clearly be seen that incident waves from the direction 30°N will lead to exceedance of operational berth criteria for berth (3) and (4), even for waves with a height of H=1.25 m.

From hereon further, waves are reflected against berths (3) and (4) to the opposite side of the basin, where other berths are located in a sheltered zone: additional berths could be unavailable because of this (see the figure below). For wave heights of H=0.75 m. it becomes clear that the allowable wave height at the berth is just around the maximum allowable operational berthing conditions. Also the reflection is not that large, directed at the other berths at the opposite side of the basin.

Figure 5.12

In-port wave penetration with waves from 30°N: waves are again reflected against berths (3) and (4).



Waves from directions other than 30°N do not penetrate far enough in-port to arrive at berths (3) and (4). So the only downtime for these berths is caused by waves from 30°N larger than H=0.75 m. This specific scenario occurs for only 7.74% of time (see wave data table P60 in annex 2) and results in an availability for these specific berth of 92.26%.

Alternatively, to improve the availability of these specific berths, additional construction measures can be adopted. If, for example, the berths are constructed as a deck on piles, wave dissipation is even more improved which results in an even smaller downtime for these berths. However this has not (yet) been adopted in the design, because it is assumed that a downtime for two to three specific berths of 7.74% is acceptable as at least 15 other berths are available for (un)loading of vessels, where also an additional safety margin was included by choosing a low berth occupancy. These specific berths should only be used when conditions allow it, which is the case for 92.26% of time at minimum. Berths situated further into the basin are located more sheltered and are available an even larger percentage of time (approaching 100%). Besides this, there exists some overlap between the downtime of port entry and the unavailability of berths (3) and (4): incident waves from 30°N are 0.89% of time larger than H=1.75, where tugs cannot fasten to the vessels.

As a last remark, it is emphasized that the actual in-port wave height is expected to be somewhat lower because of several conservative assumptions made during the process. However, outside of the entrance conditions could be worse for ships entering.
This criterion has not been assessed into detail, but there will be a large overlap with the port's entry conditions.

5.5.4 EVALUATION MODEL RESULTS

Now that both ports have been assessed individually, the interpretation of the results and concluding remarks about the port (and breakwater) layout can be defined. Again, a division will be made between the bulk port and the container port.

Bulk port

The results for the default and the improved simulation model runs with their accompanying criteria and probabilities of exceedance are presented in the summarizing table below.

Table 5.16

Bulk port results for default and improved simulation model runs with total berth downtime

	Default design				Improved design			
	Wave direction				Wave direction			
Bulk port	300°N	330°N	0°N	30°N	300°N	330°N	0°N	30°N
Dry bulk berths								
Wave height criteria at berth	H≤1.00 m.	H≤1.00 m.	H≤1.00 m.	H≤1.00 m.	H≤1.00 m.	H≤1.00 m.	H≤1.00 m.	H≤1.00 m.
Max. outside wave height Houtside	-	H<2.25 m.	H<0.75 m.	-	-	H<2.25 m.	H<1.25 m.	-
% of exceedance of Houtside	-	0.14%	1.77%	-	-	0.14%	0.87%	-
	Total downtime:			**1.91%**	**Total downtime:**			**1.01%**
Liquid bulk berths								
Wave height criteria at berth	H≤1.50 m.	H≤1.50 m.	H≤1.50 m.	H≤1.50 m.	H≤1.50 m.	H≤1.50 m.	H≤1.50 m.	H≤1.50 m.
Max. outside wave height Houtside	-	-	H<1.25 m.	H<1.75 m.	-	-	H<1.75 m.	H<2.25 m.
% of exceedance of Houtside	-	-	0.87%	0.89%	-	-	0.40%	0.34%
	Total downtime:			**1.76%**	**Total downtime:**			**0.74%**

From paragraph 5.5.2 and the table above it can be concluded that with additional wave dampening measures included the in-port wave penetration and propagation remains limited, and the availability of the bulk port berths is high:

- All the liquid bulk berths all are available for at least 99.26% of time (downtime: 0.74%). Individual berths can achieve an even higher availability.

- The dry bulk berths are available a slightly smaller portion of time. Still, for at least 98.99% of time the wave conditions allow safe (un)loading of the vessels at these berths (downtime: 1.01%).

The results of the improved simulation runs yield very positive results for the availability of the berths. However, the in-port wave conditions at the berths are not the only contribution to the port's downtime because of incident waves. In chapter 4 it was outlined that the design of the bulk port (without an in-port stopping distance for approaching vessels) was based on the acceptance of a certain downtime: the percentage of time that H>2.0 m. and tugboats cannot fasten to the vessels outside the port, which amounts to 7.1% of time. This downtime criterion overlaps, and largely supersedes the foregoing individual berth downtimes and determines to large extent the overall port's downtime. Additional factors contributing to the port's downtime due to for example crane failure were already outlined in paragraph 4.6.2.

However, it can be concluded that the design of the port and its breakwaters is adequate in creating calm in-port berthing conditions: incident waves decrease considerably in height in-port, resulting in high availability of the berths even with safe starting assumptions.

In addition to this, also the presence of the jetties in the bulk port will have a positive influence on the wave energy dissipation in-port, as these structures can be seen as an additional resistance factor which prevents wave propagation further in-port.

Container port

The results for the default and the improved simulation model runs with their accompanying criteria and probabilities of exceedance are presented in the summarizing table below.

Table 5.17

Container port results for default and improved simulation model runs with entrance and total berth downtime for critical berths

	Default design					Improved design				
	Wave direction					Wave direction				
Container port	300°N	330°N	0°N	30°N	60°N	300°N	330°N	0°N	30°N	60°N
Container port entrance										
Wave height criteria tugboats	H≤2.00 m.	H≤2.00 m.	H≤2.00 m.	H≤2.00 m.	H≤2.00 m.	H≤2.00 m.	H≤2.00 m.	H≤2.00 m.	H≤2.00 m.	H≤2.00 m.
Max. outside wave height Houtside	H<2.75 m.	H<1.25 m.	H<1.75 m.	H<1.75 m.	-	H<3.25 m.	H<2.75 m.	H<2.75 m.	H<1.75 m.	-
% of exceedance of Houtside	0.73%	0.51%	0.41%	0.89%	-	0.37%	0.08%	0.12%	0.89%	-
	Total downtime				2.54%	Total downtime				1.46%
Container port berths										
Berth 1										
Wave height criteria at berth						H≤0.50 m.	H≤0.50 m.	H≤0.50 m.	H≤0.50 m.	H≤0.50 m.
Max. outside wave height Houtside						-	H<2.75 m.	H<2.25 m.	H<2.75 m.	-
% of exceedance of Houtside						-	0.08%	0.22%	0.13%	-
						Total downtime				0.43%
Container port berths										
Berth 3 & 4										
Wave height criteria at berth						H≤0.50 m.	H≤0.50 m.	H≤0.50 m.	H≤0.50 m.	H≤0.50 m.
Max. outside wave height Houtside						-	-	-	H<0.75 m.	-
% of exceedance of Houtside						-	-	-	7.74%	-
						Total downtime				7.74%

After including additional wave absorbing improvements for the breakwaters at the container port entrance, according to the table above and paragraph 5.5.3 it can be concluded that the availability of the container port berths is also high:

- After application of low-reflectivity caissons at the container port entrance, incident waves entering the container port are dampened considerably, resulting in a port entry downtime of only 1.46% (see 5.5.3). This means that for 98.54% of time, vessels can enter the container port and tugs can fasten to the entering vessels.

- Waves from the directions 300°N, 330°N, 0°N are directed towards the low-reflectivity breakwaters surrounding the port entrance (with K_r=0.6) and are nearly completely dampened deeper in-port. With these incident wave directions, the availability for all berths amounts to nearly 100%. Berth 1 has the lowest berth availability, which amounts to 99.57% of time.

- With wave direction 30°N (which is aligned with the port's entrance), the unavailability of some berths (three at maximum) increases. For this incident wave direction, the critical berths (3) and (4) were assessed: they are available for around 92.26% of time (7.74% downtime due to waves larger than H=0.75 m). Nevertheless, other berths are located more sheltered so that this figure only accounts for these specific berths. The overall container berth availability is higher.

Because the incident wave direction poses constraints on moments that (un)loading can take place at specific berths, it is advised to take the location of vessel berthing into account.

The berths that are the most exposed should only be used when conditions allow it and when it is absolutely necessary (for instance when all other berths are occupied). Besides this it is advised to locate smaller container vessels at berths deeper in-port, and larger vessels at the 'head' berths of the container terminals. It is expected that these larger container vessels are less sensitive to the waves. This is also convenient for the easy maneuvering of larger container vessels, which do not have to be towed far into the basins.

The above described percentages show high availabilities for the container berths.
While there are many berths (18 in total), the (somewhat higher) unavailability of 7.74% for at maximum three berths does not pose much of a problem. The remaining 15 berths have an uptime approaching 100%. The overall downtime of the port is again mainly determined by the entry conditions: the time that tugs can tie up to the vessels in the lee of the breakwaters, which amounts to 98.54% of time.

Outside the port's entrance it can be expected that waves from the directions 300°N and 330°N could lead to rolling problems for entering vessels. However, this last point has not been assessed into detail, as the in-port wave penetration was subject of this chapter.
These incident waves are directed (more or less) perpendicular to the bulk port breakwater, such that a one-sided closed basin is created with a (partially) standing wave pattern.

Analogue to 3.5.4 it is clear that this exceedance of the rolling period for smaller ships (1/3 of all the vessels) could happen while approaching the port. With the chosen direction of the container terminal entrance channel this occurs only for 2.5% of time. With these percentages, it can be concluded that for 1.46%+1/3*2.5% = 2.3% of time container vessels can not enter the container port (1.46% because of tugs cannot tie up, and 0.83% because of exceedance of the vessel's rolling period). This is slightly higher than the assumed value of 0.5% in 4.6.2. The total container port's downtime amounts to 4.1%.

The simulation model runs yield good results for the container port-, berth- and breakwater layouts in terms of availability of the berths and port entry: the original layout does not have to be altered. However, the use of low-reflectivity measures is a necessity within the design, to achieve calm in-port conditions and will be adopted.

As a last remark it is emphasised that the above calculated percentages can be seen as an upper limit for the port's downtime. This is because of:

- Over-estimation of the port's depth.
- A uni-directional wave approach as a result of the use of the program DIFFRAC-2DH which does not occur in reality.
- An unfavourably (highly) reflective entrance schematization.
- Unfavourable starting assumptions of lower limiting values for the operational berthing criteria.

5.6 PORT OSCILLATIONS

The final part of the chapter on in-port wave penetration is an assessment of port oscillations. These oscillations are long-period wave motions that can disrupt harbour activities. Oscillation characteristics are generally controlled by basin size, shape and water depth. They are the most damaging when the period coincides with a natural resonant period of the port.

Port oscillations can be a significant problem for inner port components and moored vessels within basins. The oscillations can create dangerous mooring conditions and delays of loading and unloading operations. It is because of this that they will have to be assessed carefully.

5.6.1 GENERAL

These port oscillations have typical periods between 30 sec and 10 min [USACE, 2002].
In accordance with the wave records from chapter 3 it is clear that no waves with such periods are expected: infra-gravity waves and swell do not occur in the Mediterranean Sea at the project location. However, there are other factors that can cause basin resonance within the port. This will be evaluated below.

A port basin responds to external forcing and generally has several modes of oscillation with corresponding natural resonant frequencies (or periods) and harmonics. For (port) basins the forcing mechanisms include [USACE, 2002]:

- infra-gravity waves;
- eddies generated by currents moving past the entrance;
- tsunamis;
- local seismic activity;
- meteorological forces / seiches.

Regarding infra-gravity waves, no specified data records were available [ALKYON DATA], but it is expected that their influence is small. However, these waves cannot be discarded beforehand and will have to be assessed in more detail in a final design. Currents were determined to be negligible (see chapter 3), so that possibility of eddies generated by currents moving past the entrance can be discarded. [USACE, 2002] states that meteorological forces can initiate oscillations within large bays, but that they are usually not of a concern over an area as small as the size of a port. Significant changes in meteorological forces only play a role with length scales in the order of ten kilometers and much larger [PIETRZAK, 2008]. The dimensions of the port are only in the order of several kilometers (L x W = 4.8 km x 1.9 km). However, besides this, the possibility exists that seiches are generated outside the port its boundaries and subsequently force port oscillations through the port entrance.
In this preliminary assessment this has not been assessed into detail, but this will inevitably be required in a more detailed design.

The remaining two forcing mechanisms (tsunamis and seismic activities) actually can occur at the project location, although with a (very) small probability of occurrence (see chapter 3). Nevertheless, for completion an estimate regarding these frequencies will be made below.

5.6.2 CLOSED BASIN APPROACH

Port oscillations are usually initiated by forcing through the entrance, thus they deviate from a true closed basin approach. This is however not the case for seismic activity, which can create oscillations in an enclosed basin. Besides this, the closed basin approach can also be used for a narrow entrance and to transverse oscillations [LIGTERINGEN, 2007].

The different basins with their different modes of oscillation are presented in the figure below.

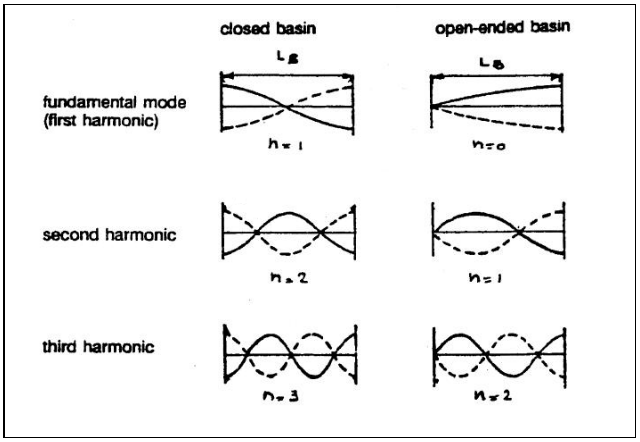


Figure 5.13
Oscillation modes for closed and open-ended basins

For closed basins, the natural free oscillating period is given by:

$T_n = 2*l_b/(n*\sqrt{(gd)})$

In which:

T_n = natural free oscillation period [s]
n = number of nodes along the long basin axis [-]
l_b = basin length [m]
g = acceleration due to gravity = 9.81 m/s^2
d = basin depth [m]

The most unfavorable situation occurs for the shortest container port berthing basin.
The length of the basin amounts to l_b= 1360 m. With a basin depth of CD -18.5 m. this means for the fundamental mode:

T_1=2*1360/(1*$\sqrt{}$(9.81*18.5))=202 s.

For the second and third harmonic (but with decreasing influence on the excitation), the oscillation periods become:

T_2=2*1360/(2*$\sqrt{}$(9.81*18.5))=101 s.
T_3=2*1360/(3*$\sqrt{}$(9.81*18.5))=67 s.

These periods are in the order of 1 – 3 minutes for the 3 most important modes of oscillation.
Moored vessels are especially sensitive to waves with periods between 30 – 150 s.
The oscillation periods for the second and third harmonic fall exactly within this range.
If during times of earthquakes these port oscillations do occur, excessive vessel movements at the berths can be expected.

Luckily, earthquakes only last for a short period of time and have a small probability of occurrence. Besides this, the amplitudes of the generated waves decrease with increasing harmonic. This means that for the second and third harmonics the wave height has already decreased somewhat in contrast to T_1, which is favourable.

5.6.3 OPEN BASIN APPROACH

The open basin approach is of importance when assessing the overall bulk port and the entrance (channel) of the container port. Again, also here a rare event can lead to port resonance: tsunamis. When considering a simple basin shape with 3 sides and 1 entrance, the free oscillation period for this approach is described by:

$T_n=4*l_b/((1+2n)*\sqrt{(gd)})$

Container port

For the container port its entire entrance length (until the southern breakwater near the Rio Kert), l_b=5550 m. and the average depth is CD -27.5 m. The first three resonance modes (with 0 = fundamental mode) give periods of:

$T_0=4*5550/((1+2*0)*\sqrt{(9.81*27.5)})=1352$ s
$T_1=4*5550/((1+2*1)*\sqrt{(9.81*27.5)})=451$ s
$T_2=4*5550/((1+2*2)*\sqrt{(9.81*27.5)})=270$ s

These are in the range of 5 – 22 minutes.

Bulk Port

For the bulk port a basin length applies of l_b=1665 m. and an average depth of CD -25 m. This results in the following free oscillation periods:

$T_0=4*1665/((1+2*0)*\sqrt{(9.81*25)})=452$ s
$T_1=4*1665/((1+2*1)*\sqrt{(9.81*25)})=142$ s
$T_2=4*1665/((1+2*2)*\sqrt{(9.81*25)})=85$ s

These are in the range of 1.5 – 8 minutes.

As outlined in 5.6.2, moored vessels are especially sensitive to waves with periods between 30 – 150 s. From this it can be concluded that for the container port no problems are expected: these periods of port oscillations are (much) larger.

For the bulk port, the oscillation periods T_1 and T_2 fall within this range, and again excessive vessel movements as a result of port oscillation forcing due to tsunamis can be expected. In 3.3.4 it was argued that the probability of tsunamis in the region is very small, because of the large distance of the project location to historically registered epicenters that caused serious tsunamis in combination with the small chance of occurrence. Besides this, also here applies that the amplitudes of the generated waves decrease with increasing harmonic, which is favourable in this case.

According to the preliminary estimations above, port oscillations cannot be completely discarded. The calculated period of the first harmonic in the open basin approach T_0 =1352 s. falls within the range of seiches, which will have to be assessed in more detail during a final design.
For a more detailed in-depth assessment of port resonance, mathematical models will have to be applied in order to also include a more realistic schematisation of the basins edges, differences in depth throughout the ports and the non-standard port (and basin) shapes.

Several specific topics have been treated in this chapter on in-port wave penetration and propagation. A summary of these findings is outlined below and will conclude this chapter.

- According to the preliminary wave assessment with the Coastal Engineering Manual [USACE, 2002], in-port wave penetration and propagation will be of large importance within the selected port masterplan layout.

- Especially the processes wave diffraction and wave reflection, and subsequently their combined influence determine the in-port wave conditions. The visual approach with the Coastal Engineering Manual appeared to fall short, and use has to be made of wave simulation models.

- Input of the original port and breakwater configuration in the simulation model showed large wave reflection, especially due to the monolithic caisson breakwater. This had to be reduced in order to minimize the port's (berths and entrances) downtime.

- The improved configuration comprises the use of low-reflectivity caissons, and was subsequently used as new input for the simulation model. The output yields positive results for the availability of the berths and entrance: the downtime is roughly halved.

- Port oscillations cannot be discarded completely: due to earthquakes, tsunamis, and seiches generated outside the port's boundaries, in-port oscillations can lead to hindrance for berthed vessels. This will have to be assessed in more detail during a final design.

CHAPTER 6 Breakwater design

6.1 INTRODUCTION

For the assessment of the in-port wave penetration in the previous chapter it was essential to get a first indication of the composition of the breakwater types. According to the wave penetration study, this original configuration can still be maintained. However, it became clear that several construction improvements will have to be made to this configuration when considering the wave-structure interaction.

For the two main breakwater types, a cross-sectional design will be made. These two types comprise the rubble mound breakwater and the vertical composite breakwater. The cross-sectional design gives an important indication of the dimensions involved and the materials to be used for the breakwater composition. For some (parts of the) breakwaters, specific measures will be necessary to improve wave energy dissipation, according to the wave study from the previous chapter. This is in order to create sufficiently calm in-port manoeuvring and berthing conditions.

This is especially of importance at the container port entrance, where wave reflection plays an important role for port entry downtime due to wave reflection. To accomplish a lower wave reflection at the container terminal entrance, specific modifications for the monolithic breakwaters at that location will have to be made.

6.2 RELEVANT PARAMETERS

For the cross-sectional design of the different breakwaters it is first of all a necessity to establish an inventory of relevant parameters required for the design. This comprises amongst others assessing critical wave conditions and their chances of occurrence and constructional constraints. These constraints consist mainly of the large water depth, the lack of space at the container port entrance, and the construction materials.

6.2.1 PROBABILITY OF FAILURE

A breakwater is constructed for a specific economic lifetime, in which the probability of failure should be sufficiently small. For breakwaters a lifetime of 50 years is very common [VERHAGEN *et al.*, 2009]. However, it is emphasized that this does not suggest the use of a once in 50 years storm for design purposes: the once in 50 years storm does not occur after 50 years, but every year there is a probability of 1/50 (=2%) that the design storm will occur, which could be next year.

The probability of serious damage during the lifetime of the construction is given by the Poisson distribution:

$p=1-\exp(-f*T_L)$

In which:

p = probability of occurrence of an event one or more times in period t_L [-]
T_L = considered period (e.g. the lifetime of the breakwater) [years]
f = average frequency of event per year [1/year]

With a common breakwater lifetime of 50 years and a storm frequency of 1/50 per year gives a probability of failure (according to the formula above) of 63%, which is clearly unacceptable as outlined before.

In order to construct a breakwater that is largely maintenance free, the failure probability (with a Poisson distribution) should be chosen sufficiently low. From economic analysis follows that this probability should be around 5%, but is still acceptable up to 20% at maximum (depending on the purpose of the structure and the risk involved) [VERHAGEN *et al.*, 2009].

As the breakwaters are designed primarily to protect the in-port facilities and berthed vessels, the (rather high) probability of failure of 20% will be too large. On the other hand, because of the large constructional depth the breakwater will be a very important cost item. Because of this, a (low) probability of 5% would lead to an expensive (high) structure.
As a consensus, a probability of failure of 10% has been adopted. This means:

$0.1 = 1-\exp(-f*50)$

From which follows that f, the frequency of the storm event per year, amounts to f=0.00211. This means a 1/475 year storm.

6.2.2 DESIGN WAVE CHARACTERISTICS

Location and direction
For determination of the design wave height at a specific location, the transformed near shore extreme wave climate will be used, which was already presented in chapter 3.
For the calculation of the design wave height, calculation point P08 (d=56 m.) will be used.

Although point P60 (d=40 m.) is situated the closest to the future breakwater's location, this would yield an underestimation for the wave height: parts of the container terminal breakwater are located in water deeper than 40 m. Due to the larger water depth at P08, wave heights are slightly larger than in calculation point P60. Differences are not that large, but nevertheless, this decision is a safe starting assumption.

Besides this location, the dominant direction will need to be specified. This is according to the wave tables (and as defined earlier) the direction 255-285° N: the dominant wave direction with the longest fetch. This critical direction will be used from hereon for the design.

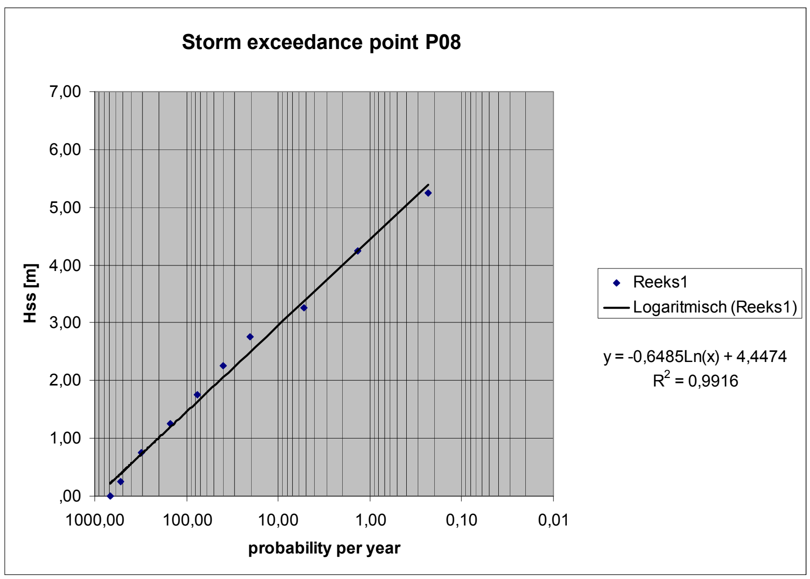
Design storm wave height and period

With the defined design storm of 1/475 year, the design storm wave height for which the breakwater will be designed can be calculated. In chapter 3, the probability of occurrence for certain specific wave heights has been defined (by [ALKYON DATA]). With the decision for the dominant direction 255-285°N, the wave table with their probability of occurrence (P) and exceedance (Q=1-P) can be put together.

While for the wave observations it is not known how many are made (only probabilities), the Peak over Threshold analysis cannot be determined. Here only random data is available from ship observations provided in probabilities. After analysing this data (see table A2.13), while assuming a storm-duration of 12 hours (thus N_s=730), an exponential relationship between the probability of occurrence of a storm and its accompanying wave height can be presented in a graph.

Figure 6.1

Storm exceedance graph (exponential) using random observations



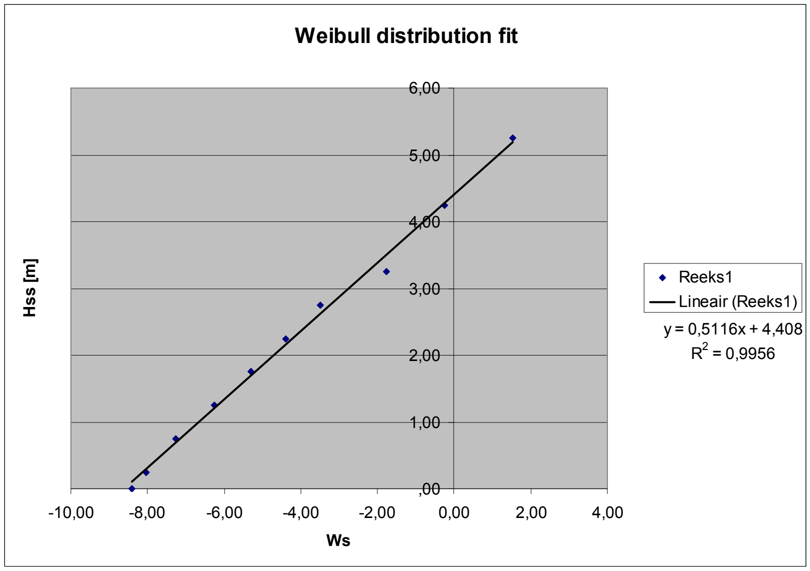
Subsequently, this analysis can be improved by using the Weibull distribution [VERHAGEN *et al.*, 2009]. This distribution has been proven to be the most robust. This has been done accordingly, and the results are presented in figure 6.2. This figure has been constructed by using the last column of table A2.13.

The duration of the storm that will be used is set to 12 hours. While a duration of 24 hours is not uncommon in the Mediterranean (often, the duration is estimated between 12-24 h.), the Weibull distribution yields larger results for smaller periods of duration. For this, the 12 hours storm duration is maintained. This gives the equation:

$y=4.408+0.5116*\ln(1/Q_s)^{1/0.88}$, see figure 6.2

Figure 6.2

Weibull distribution fit for determination of the design storm wave height



With the previously determined probability of failure p=0.1 in combination with the 1/475 year storm, the design storm wave height can be calculated on which the breakwater design will be based. This follows from:

$H_{ss}=\gamma+\beta(-\ln(W_s))^{1/\alpha}$

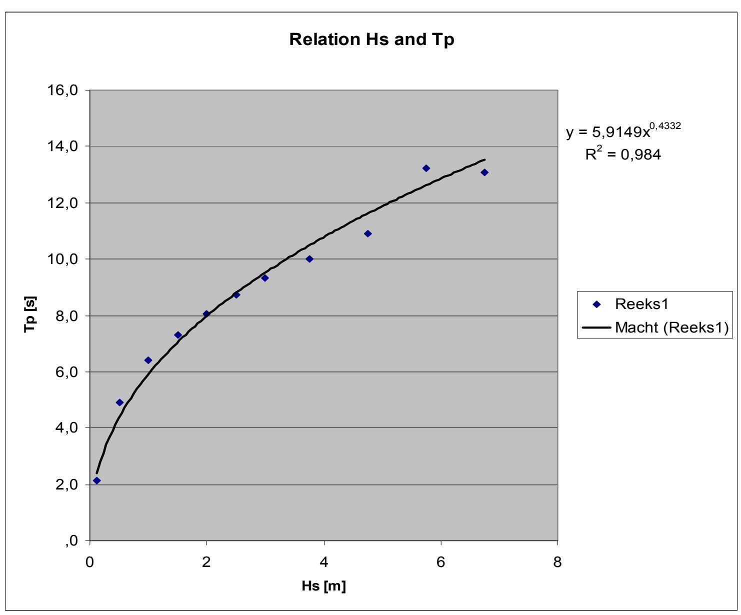
With the above derived relationship and coefficients this means that:

$H_{ss\,1/475} = 4.408+0.5116*(-\ln(1/475))^{1/0{,}88} = 8.45$ m.

In order to calculate a wave period, a relationship has been established between H_s and T_p. This has been done with a power-relationship. The figure below shows the results that have been acquired.

Figure 6.3

Relation between Hs and Tp



With the relation $y=5.9149*x^{(0,4332)}$ from the figure above and the design storm wave height of $H_{SS\ 1/475}$ = 8.45 m. the accompanying wave period can be calculated as follows:

$T_p=5.9149*8.45^{0.4332}=14.91$ s.

The peak period T_p is mostly around 10% larger than $T_{m-1,0}$ in deep water conditions [VERHAGEN *et al.*, 2009]. However for shallow water this can be completely different. Again with the extreme near shore wave climate table 3.9, a proper estimate can be made that will prove to be more reliable. It appears that for all the values applies:

$T_{m-1,0}=0.82*T_P$

This means for the design storm wave that $T_{m-1,0}=0.82*14.909=12.23$ s.

Ultimate limit state and serviceability limit state

It is emphasized that the above calculated design storm wave characteristics are valid for the ultimate limit state (ULS), under which the breakwater should not fail. For the serviceability limit state (SLS) it is of importance that vessels can still manoeuvre and (un)berth within the port's boundaries, the breakwater is accessible and that the port is fugitive for the vessels in case of severe weather conditions outside.

This means that in the SLS wave overtopping should remain limited to ensure safe manoeuvring and (un)loading of the vessels in the container port, where both typical cross-sections are located. The wave height used in the calculations of the maximum allowed overtopping during operational conditions (SLS) is the specific wave height $H_{2\%}$ which is exceeded 2% of time. This is also more or less in line with the percentage of time that container port entrance with tugboats in the lee of the breakwaters is possible (slightly over 98% of time).

The SLS design wave height $H_{2\%}$ can be estimated by a graph covering the data from the wave table for P08 in the direction of 255-285°N. While this graph is only partially valid, boundaries need to be taken into account. It is evident that wave heights smaller than H<3.25 m. occur for 97.2% of time, and H<4.25 m. for 99.3% of time. So between these two bins, the target wave height exists. The graph for determination of this wave height has been presented in the figure below.

With the presented graph it follows that $H_{2\%}=4.19$ m. For the wave data from the table of point P08, also here the earlier defined relationship between H and T applies.
This gives a peak period of:

$T_p=5.9149*4.19^{0.4332}=11.00$ s.

And, with $T_{m-1,0}=0.82*T_P$:

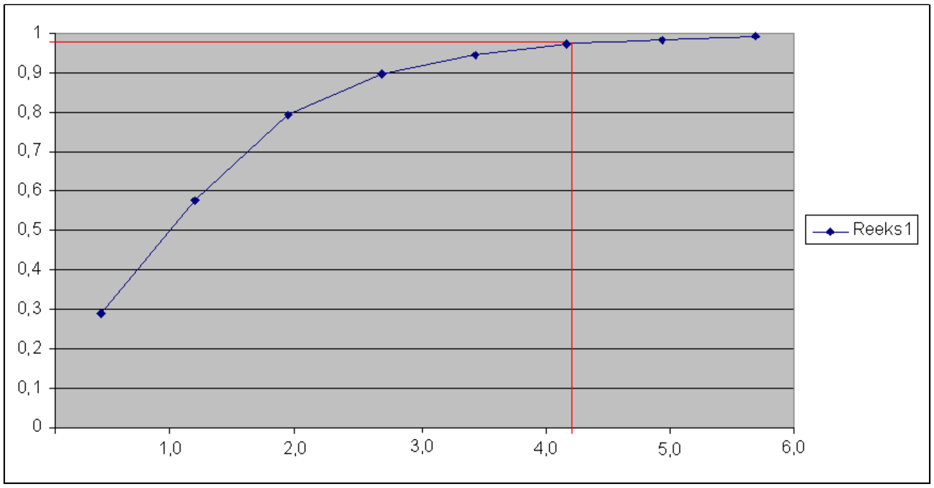
$T_{m-1,0}=9.02$ s.

Figure 6.4

Graph for estimation $H_{2\%}$

Y-axis: cumulative probability of occurrence for wave heights P08

X-axis: wave height [m]



For both limit states, the wave characteristics (height and periods) are summarized in the table below.

Table 6.1

SLS and ULS wave characteristics

Limit states characteristics	SLS	ULS
Wave height H	4.19 m.	8.45 m.
Wave peak period T_p	11.00 s.	14.91 s.
Wave period 1st negative spectrum moment $T_{m-1,0}$	9.02 s.	12.23 s.

6.2.3 MATERIAL CHARACTERISTICS

Rubble mound material

When designing a rubble mound breakwater it is of importance to investigate if there is a stone quarry nearby. This is because of the large quantities of stone material required, and it is not economical to transport this over a (too) large distance. According to reports (from [ALKYON DATA]) this is the case. The availability of rock rubble for the construction of the rubble mound breakwater will therefore not be a problem.

Sea bed material

As described in chapter 3, the exact composition of the present bed material is not exactly known and would require further investigation [ALKYON DATA]. However, from the available (global) data it can be deducted that the bed material mainly consists of sand. So from hereon, it is assumed that the sea bed consists of medium sand.

6.2.4 LOCATION CROSS-SECTIONS

With all the above determined parameters, the design of the two specific breakwater cross-sections can commence. In the next two sections, these two designs will be addressed: the rubble mound breakwater (section 6.3) and the vertical composite breakwater (section 6.4). These designs will be made for the two cross-sections as indicated in the figure below.

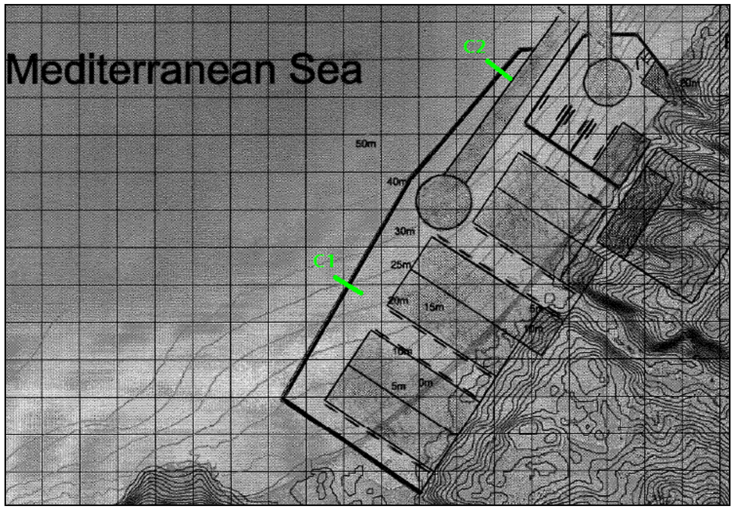


Figure 6.5

Locations for cross-sectional design of breakwaters
C1: Rubble mound breakwater at d=25 m.
C2: Vertical composite breakwater at d=45 m.

The two cross-sections are chosen at the most critical locations: where the water depth is the largest, and for C2 also the wave action on the inside of the breakwater due to wave reflection.

6.3 RUBBLE MOUND BREAKWATER

For the design of the rubble mound breakwater (cross-section C1), as a first step the required stone size for the armour layer will be determined. This armour layer has to withstand the wave attack during design conditions, with a wave height as determined in 6.2.2. Subsequently, underlying layers and other specifics can be designed.

With determined design wave heights for the ULS and the SLS, the crest level of the breakwater can be determined. For the extreme situation it is of primary importance that the structure does not fail. Run-up and overtopping are in this case allowed, as the port will be closed to any activities. In the port masterplan, no structures are located directly at the inner slope of the breakwaters so that overtopping will not pose large problems for nearby structures.

6.3.1 ARMOUR LAYER

Stone size

The stability of rock can be expressed with the dimensionless parameter $H/\Delta d_n$. The higher this relation is, the larger the waves that can be accommodated by the same stone size.
In order to get an indication of the required size of the rock under design conditions, one can use the following rewritten Van der Meer-formula [VERHAGEN *et al.*, 2009]:

$$H_{ss}/(\Delta^* d_{n50})=c_{pl}P^{0.18}(S/\sqrt{(N)})^{0.2}s_{m-1}^{0.25}\cot(\alpha)^{0.5}$$

In which:

H_{ss} = design wave height [m] = 8,45 m.
Δ = relative mass density: $(\rho_s-\rho_w)/\rho_w$ = (2650-1030)/1030=1.57 [-]
d_{n50} = nominal diameter [m]

c_{pl} = factor of 5.5 for design purposes [-]
P = notational permeability [-] = 0.6
S = damage level for quarry stone [-] = 6 (no failure but some damage, and repairs required)
N = number of waves in a storm [-] = 7500
s = wave steepness [-] = in the order of 4%
α = slope of breakwater [-]; $\cot(\alpha) = 2$

This leads to a required nominal stone diameter of d_{n50}= 2.89 m.

It appears that for the rubble mound very large stone diameters are required in the armour layer. The availability of large rock is limited, and blocks with a weight of more than 10 tons are very rare [VERHAGEN *et al.*, 2009]. With this, the use of rock for the armour layer will become difficult, as these stone sizes are not largely available. Because of this, use will be made of concrete armour units.

Armour units

Concrete armour units are available in many different forms with different characteristics. To limit the total amount of armour units used and to minimize the time and costs of construction and placement in combination with the large water depth, it is advised to construct the armour layer with a single layer armour unit.

For this single layer element, on the present day market, two good alternatives are available: the Xbloc and the Accropode II. It is emphasized that both concrete units can be used equally well for the armour layer design; a decision for this is mainly based on client and contractor considerations. Here, arbitrarily, the Accropode II has been selected.

To facilitate placement of the concrete armour units (and to reduce the accompanying costs) and at the same time increase the inter-locking, the improved Accropode II unit will be used in contrast to the original Accropode. The Accropode II is also a single layer armour unit, and can be placed under a slope of 1V:1.33H [VERHAGEN *et al.*, 2009] [WEBSITE CLI].
The Accropode II unit has been applied recently in several nearby projects: in Italy, France and Morocco [WIKIPEDIA] [WEBSITE CLI].

For single layer armour units, a design value of 2.8 for the dimensionless factor $H_s/(\Delta * d_n)$ is recommended [VERHAGEN *et al.*, 2009]. With a concrete density of ρ_c=2400 kg/m^3, the relative density becomes Δ=1.33, which results in a diameter of d_n=2.27 m. This is however not the unit height. This height can be approximated with a calculation tool [WEBSITE CLI].

It appears that for a design wave height of H_{ss}=8.45 m. Accropodes II are required with the properties defined in the table below.

Table 6.2
Properties of Accropode II armour unit under design conditions

Properties	
Theoretical volume	V=12.02 m^3
Standard volume	V=15 m^3
Unit mass	m=36 t.
Unit height	h=3.71 m.

The unit height h=3.71 corresponds with a d_n=0.7*h=2.60 m [VERHAGEN *et al.*, 2009].

It is certainly not necessary to extend the armour layer over the full water depth down to the seabed. At water depth of about one wave height below still water level the effect of wave action is limited and no heavy armour is required any more. The armour layer should then be supported by a toe. The slope of the armour layer is already predetermined: 1V:1.33H. This steep slope is favourable for the stability of the concrete armour units (due to their weight) and the limited use of underlying rock material.

Armour layer thickness

With the concrete armour units as described above, the effective layer thickness can be calculated as follows [VERHAGEN *et al.*, 2009]:

$t=n*k_t*d_{n50}$

In which:

t = layer thickness [m]
n = number of stones across the layer = 1 unit
k_t = layer coefficient = 1.29
d_{n50} = nominal diameter = 2.60 m.

This results in t=3.35 m.

6.3.2 UNDER-LAYER AND CORE

First under-layer

The first under-layer is the layer directly under the armour layer. The units forming this layer must not pass through the voids in the armour layer. The weight of the units should not be less than 1/10 of the weight of the armour units [VERHAGEN *et al.*, 2009]. With this strict criterion the filter becomes geometrically impermeable. For the quarry stone, a density of ρ_s=2650 kg/m^3 has been assumed.

This means for the first under-layer of the rubble mound breakwater a mass of m=3.6 t. with a nominal diameter of:

$d_{n50}=\sqrt[3]{(m/\rho)}=1.11$ m.

This amounts to heavy grading with a weight of 3 – 6 tons. For the layer thickness, also the earlier presented formula applies:

$t=n*k_t*d_{n50}$

However, here the number of units across the layer amounts to n=2, and the layer coefficient is according to [VERHAGEN *et al.*, 2009] k_t = 0.91 for double standard rock layers. This give a first under-layer thickness of t=2.02 m.

Core

With the above defined first under-layer, the next layer can be calculated. The next subsequent layers will consist of 1/10 to 1/25 of the material from the first under-layer. With value of around 1/15, this means for the new layer a mass of m=0.24 t. Again a density of ρ_s =2650 kg/m^3 has been assumed.

This can be used as core material with a nominal diameter of:

$d_{n50}=\sqrt[3]{(m/\rho)}=0.45$ m.

For this, the heaviest fraction of the light stone grading can be used [VERHAGEN *et al.*, 2009], which amounts to stones with a mass of around 150-500 kg.

6.3.3 CREST, TOE AND FILTER

Crest width

If the crest consists of loose armour units, its width must be at least 3 stones [VERHAGEN *et al.*, 2009], or in the form of a formula:

$B=n*k_t*d_{n50}$

In which:

B= crest width [m]
n = minimal number of loose armour units = 3
k_t = layer coefficient [-] = 1.29 for Accropodes II
d_{n50} = nominal diameter [m] = 2.60 m.

With the above outlined value this gives a crest width of B=10.1 m.

Directly on top of this rubble mound breakwater crest, no special measures are required: for example a road or pipelines on top are not required. Because of this, the calculated crest of 3 armour units wide is sufficient.

Toe material

In order to reduce the amount of the (expensive) armour units in the breakwater cross-section, the armour layer will not be extended over the full water depth down to the seabed. At a water depth of about one wave height below still water level, the effect of wave action is limited and no heavy armour is required any more.

Classic literature recommends for the size of the toe material stones that are equal to the weight of the first under-layer [VERHAGEN *et al.*, 2009]. This amounts to 1/10 of the weight of the armour units. As above, it was calculated that this would be a mass of m=3.6 t. According to:

$d_{n50}=\sqrt[3]{(m/\rho)}=1.11$ m.

This again amounts to heavy grading with a stone weight of 3-6 ton.

Filter and scour protection

It is recommended that a geometrically impermeable filter is placed under the seaward part of the breakwater. This is because under the seaward toe, large pressure gradients may exist, which can wash out material from the seabed through the structure. The pressure gradients under the centre of the structure and under the inner toe are generally much lower.

In order to account for the difference between the (large) stone diameter as core material and the virgin bottom material (medium sized sand), a geotextile will be placed underneath the toe of the structure to prevent the bottom material from being washed out. In addition, a rubble blanket will be provided in front of the breakwater as scour protection, with a width of at least 5-10 m.

6.3.4 CREST HEIGHT

Overtopping

For the height of the breakwater, the overtopping requirements are of great importance. For the serviceability limit state (SLS) the overtopping discharge must be small enough so that accessibility of the breakwater is possible. For the ultimate limit state (ULS), severe overtopping is permitted (as the port will be closed at that time), but the structure may not fail. This means also protection on the inner slope of the breakwater.

The maximum allowed discharges can be determined from the Eurotop manual [PULLEN *et al.*, 2007]. On top of the rubble mound breakwater and directly behind it there is no activity (no road is required and no pipelines on the breakwater). If there is water behind a structure, large overtopping can be allowed as this overtopping will plunge into the water again [PULLEN *et al.*, 2007]. This however generates transmitted waves, which could be unfavourable for the moored and manoeuvring vessels in the SLS, and should thus be limited. For the ULS very large overtopping criteria suffice, as in the lee of the breakwater only more water is located (e.g. no structures or roads). Because of this, the overtopping criteria for the ULS are discarded here: the crest height of the rubble mound breakwater will depend on the SLS-criterion. This overtopping criterion is:

SLS: $q<10$ l/s/m = 0.01 $m^3/s/m$.

With these criterions, the crest height freeboard Rc can be calculated [VERHAGEN *et al.*, 2009]:

$$q/\sqrt{(gH^3)}=0.2\exp(-2.3*Rc/(H*\gamma_f*\gamma_\beta)$$

In which:

q = overtopping discharge [$m^3/s/m$]
g = gravity constant = 9.81 [m/s^2]
H = wave height for specific limit state (see table 6.1) [m]
Rc = crest height freeboard [m]
γ_f = roughness factor = 0.46 for Accropodes II [-]
γ_β = effect of approach angle = 1 [-]

The result for the serviceability limit state has been summarized in the table below.

Table 6.3

Crest heights freeboards for SLS

Limit state	q	H	Rc
SLS	0.01	4.19 m.	5.30 m.

Wave run-up has always been less important for rock slopes and rubble mound structures and the crest height of these types of structures has mostly been based on allowable overtopping or wave transmissions [PULLEN *et al.*, 2007]. The cross-sectional design of the breakwater must be able to cope with large overtopping rates during the ULS (see explanation above).

Transmission

In a first design, it can be assumed that the probability of overtopping of the breakwater must be smaller than 10%, otherwise waves in the lee of the breakwater are generated (wave transmission) [PILARCZYK, 1998]. For the SLS, this can be assessed with the following formula [VERHAGEN *et al.*, 2009]:

$$P_{ov}=\exp[-(A_C*d_n/(0.19*H^2))^\wedge 1.4]=0.1$$

In which:

P_{ov} = probability of overtopping [-]
A_C = actual breakwater height above water level [m]
d_n = nominal armour diameter [m]
H = wave height for SLS [m]

This gives for a d_n=2.60 m. and a wave height of H=4.19 m. an actual crest height of A_C=2.33 m. This required crest height to prevent wave transmission (A_C) is smaller than the required crest height according to the overtopping requirements in the SLS criteria (R_C = 5.30 m.). This means that for this design transmission is not critical, but the allowable overtopping rates are. It is emphasized that here wave transmission through the port's entrance has not been taken into account, but it is expected that this deep in-port the influence is small.

Additional height

Additional crest height for the breakwater is required to account for extreme water levels, future settlement and sea level rise.

From table 3.5 it is clear that the extreme high water level is CD +1.0 m. Sea level rise can not be predicted very accurately; however this is estimated to be 0.5 m. for the next 50 years [PIETRZAK, 2008]. Additional settlement (e.g. due to earthquakes) can be taken into account by including an extra 0.8 m. [CUR, 2007].

This results in a final crest height of R_C=5.3+1.0+0.5+0.8=7.6 m. above MSL.

With this breakwater design, the width of the core at sea level is around 11 m. This is just wide enough for construction of the breakwater from land onwards: in fact it is a minimal requirement: the crest height of the breakwater should not be any lower.

6.3.5 DESIGN

In order to finalize the rubble mound breakwater design several assumptions have been made. These are outlined below.

Assumptions

- Armour units do not need to be extended down to the seabed. This is because the wave action at a water depth of around one wave height is limited, and there is no direct need for the large (expensive) concrete armour units. With a wave height of H_{ss}=8.45 m., the armour layer will be extended to a depth of CD -10 m. at the seaward side.

- At the inner side of the breakwater, the armour layer needs to be extended because of the (large) allowed amount of overtopping in the ultimate design situation. This inner armour has been extended until a depth larger than ½ *H_{ss} : a depth of CD -5 m. At this location, the armour layer is again supported by an inner toe.

- Berms and toes need to be provided in which deviations in the actual measurements can be taken into account. This has been included in the (preliminary) design by means of a larger supporting width than the upper lying layer requires.

- As wave action in-port remains limited, at larger depths the core material can be used as support for the upper layers, which limits the amount of larger first under-layer material to be used.

- The first under-layer and the core material (stones) will be constructed with a slope of 1V:1.5H, to limit the amount of material used. This also amounts for the inner slopes of the breakwater.

- The geotextile will be constructed mainly under the toe, as at that location the pressure gradient will be the largest. The geotextile will be extended for a small distance under the core.

- In general, somewhat larger supporting toe and berm widths are provided to account for the extra settlements due to earthquakes.

Rubble mound breakwater cross-section

The resulting cross-sections are presented in the figures on the next pages.

Figure 6.6

Tentative cross-section rubble mound breakwater at a depth of d=25 m.

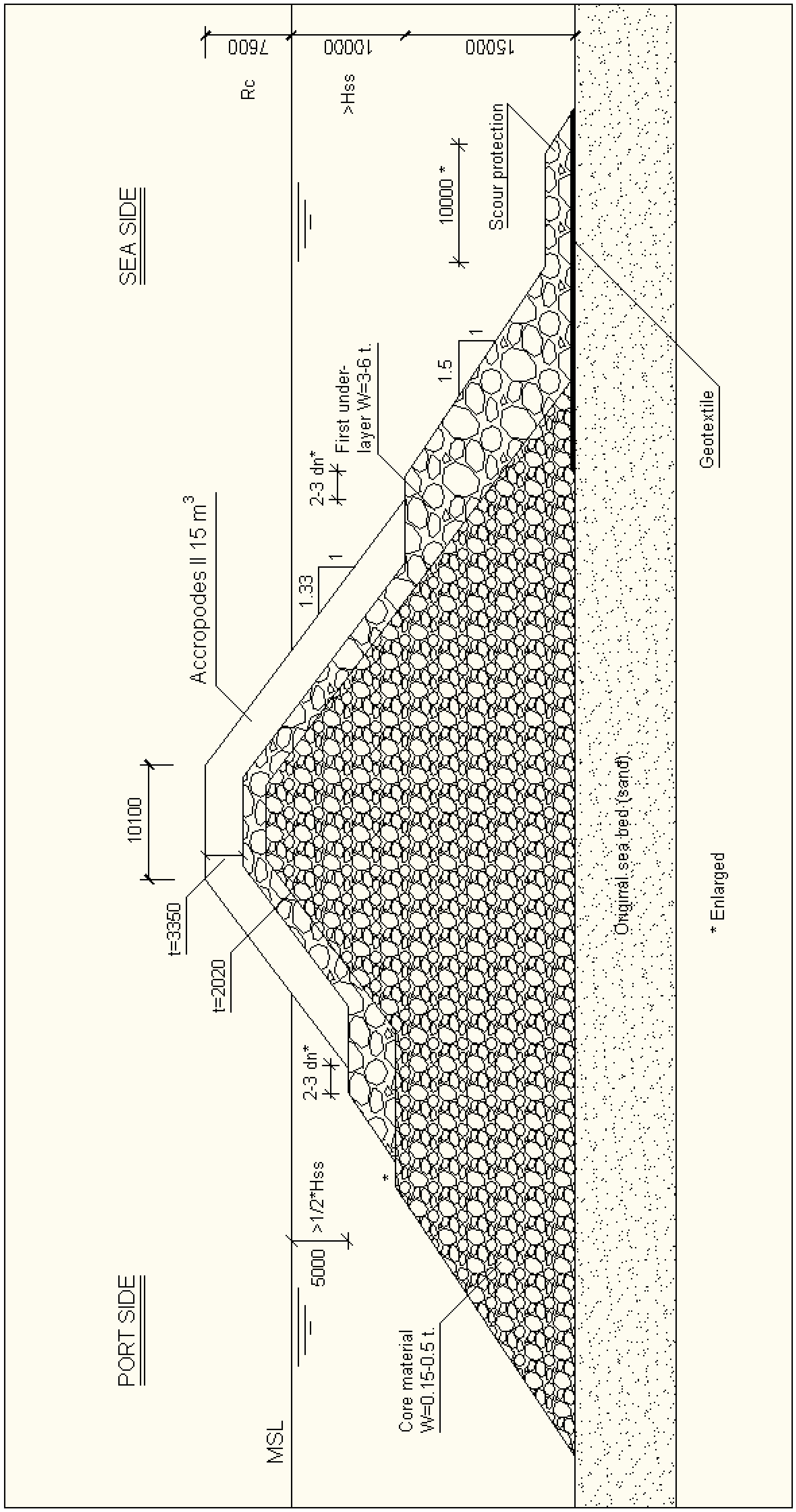
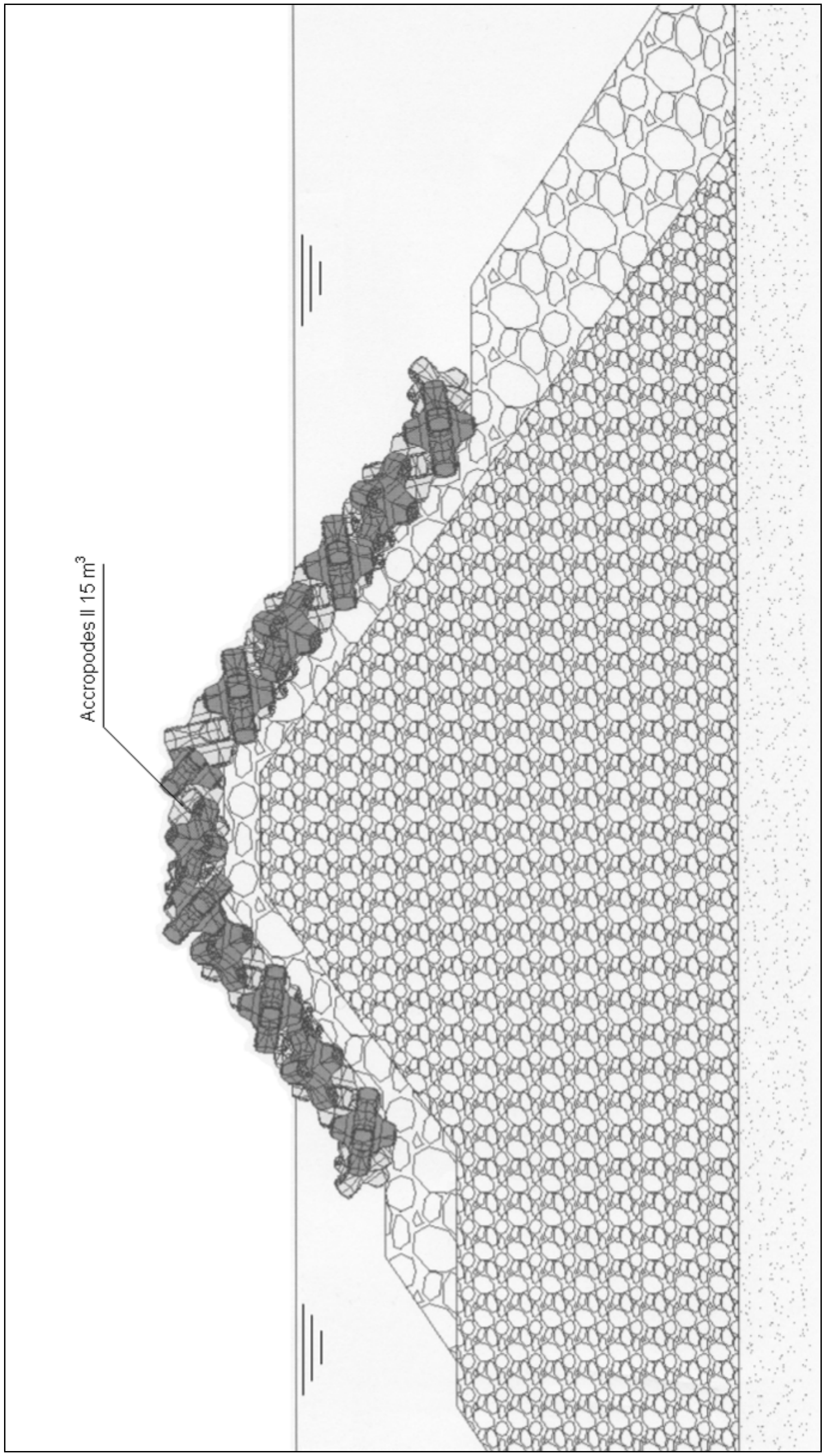


Figure 6.7

Detailed cross-section of rubble mound breakwater with indicated concrete Accropode II armour units



Final notes

As a final remark it should be checked whether the required reflection coefficient for sufficient wave dampening ($K_r \leq 0.45$) is achieved as was determined in the wave penetration study (chapter 5). The general shape of the reflection formula for straight slopes is:

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

In which:

K_r = reflection coefficient
a = 0.115 for Accropodes II [VERHAGEN *et al.*, 2009]
b = 0.87 for Accropodes II
$\xi_{m-1,0}$ = the Iribarren number based on $T_{m-1,0}$

When assuming a wave height in the SLS of H=4.19 m. and a $T_{m-1,0}$=9.02 s, this gives $\xi_{m-1,0}$ =4.13. This results in a reflection coefficient of K_r=0.38, which is smaller than required and thus more than acceptable. Even with a storm wave height of H_{SS}=8.45 m. and $T_{m-1,0}$=12.23 s. this means an $\xi_{m-1,0}$ =3.94 and a reflection coefficient of K_r=0.36.

Even under the most unfavourable conditions, the actually achieved reflection coefficient for the rubble mound breakwater is lower than the opted 45% during the wave penetration study. The actual in-port wave dampening will thus be even somewhat larger than the simulation model has shown.

This is also enhanced by the fact that the waves do not approach perfectly perpendicular to the breakwater, which makes the earlier calculated values for H_s, T_p, and H_{SS} safely assumed upper limit values.

6.4 VERTICAL COMPOSITE BREAKWATER

At the location of the container port entrance, the breakwater comprises a vertical composite breakwater, as indicated in figure 6.5. They consist of a rubble mound bed (foundation) with a caisson placed on top. A distinction can be made between low mound and high mound [TAKAHASHI, 2002]. For this, several options are available. The constructional design is subject of this paragraph.

6.4.1 CONSTRUCTION ASPECTS

For the construction of this part of the container port breakwater, several construction aspects play a role. One of these is the ease of expansion. To improve this as much as possible, a practical decision has been made for the caisson height. This height has been chosen as a uniform height (under water) of around h=25 m, the available water depth at the point of transition between rubble mound and monolithic breakwater. The remaining part of the vertical composite breakwater is located in deeper water. This eases the construction of the caissons, and decreases the total construction costs. The variation in the original sea bed will be included by adapting the height of the mound foundation where the caisson is placed on top of. Besides this, for constructional reasons a uniform caisson height is easier to connect to the adjacent caisson.

With the assumption of this uniform caisson height, also enough water depth directly alongside the caissons is maintained. Vessels only have drafts up to 22 m. at maximum (this also accounts for the north-western bulk port breakwater). Also, at the inside of the port, the need exists for additional wave energy dampening measures, as was determined in the wave penetration study. At the sea side, this direct need does not exist (for the container port breakwater).

The container port breakwater at the chosen location will be the most challenging for design. This will be outlined in the next paragraphs.

6.4.2 CAISSON DIMENSIONS

For a preliminary design of the caisson for the vertical composite breakwater, the design rules according to PIANC and [VERHAGEN *et al.*, 2009] have been used. For this design, two design wave heights have to be defined: a value H_r which is the design wave height for the ULS condition and a value H_u which is the design wave height for the SLS condition.

The values are chosen analogous to 6.3: H_u=4.19 m. and H_r=8.45 m. The ratio between the wave heights amounts to H_r/H_u=2.0

- First of all the free height of the caisson under low water level will be chosen. It was opted that the caisson would have a height of around 25 m. to use a (maximal) uniform caisson height for the construction of the container port breakwater. The low water level follows from LAT (which is CD + 0 m.). For the extreme low water level, also the wind set-down and the variation in atmospheric pressure can be taken into account. This results in the extreme low water level of CD – 0.40 m. A free height should be present of 1.5*H_r below water level, which is 1.5*8.45=12.7 m. This is only half of the actual caisson height, so this criterion is fulfilled. The free height amounts to 25-0.35-0.4=24.25 m.
- The width of the caisson should be at least 0.8 times the free height. With the above calculated free height, the required width of the caisson will be 0.8*24.25= 19.5 m.
- The toe protection against undermining should have a thickness of which is at least 0.15 times the free height. This amounts to 0.15*24.25=3.6 m. This mainly applies for the most south-western part of the vertical composite breakwater, where the water depth is limited. At the designated location for the cross-sectional design, the height of the foundation is around 20 m.
- The crest rises to an elevation of 1.3 – 1.5 times H_u above high water on the sea side and 0.5 times H_u on the port side. This first criterion means at least 1.5*4.19=6.3 m. above high water. In a first design, this height will also be maintained at the inner side.
- The scour protection extends at least 2.5*H_u in front of the wall, with a minimum of 10 – 15 m. Here, this means at least 2.5*4.19=10.5 m. in front of the caisson, and at the inner side.

Overtopping

With the above described guidelines, a first preliminary design of the caisson of the vertical composite breakwater can be made. However, especially the determination of the caisson height requires special attention in order to meet the overtopping criteria.

Overtopping of vertical walls can be analysed with the formula [VERHAGEN *et al.*, 2009]:

$q/\sqrt{(g*H^3)} = a*\exp(-b*R_c/H)$ for $0.1 < R_c/H < 3.5$

Also here, the same criteria as for the rubble mound breakwater apply. For the ULS conditions with H=8.45 m., the unit discharge of 200 l/m/s, with coefficients a=0.04 and b=2.6 give a crest height of R_c=8.9 m. For the SLS, the wave height is H=4.19 m. with a discharge of 50 l/m/s. This gives a crest height of R_c=7.54 m.

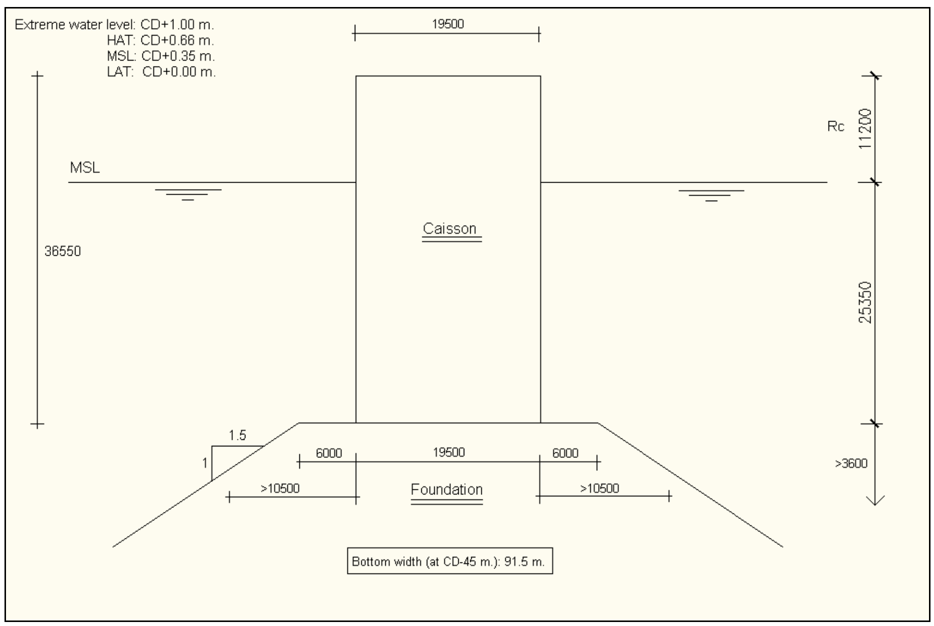
So instead of the earlier calculated R_c=6.3 m. with the PIANC guidelines, this latter (more critical) crest height R_c=8.9 m. will be used in the design. However, to account for additional required height for extreme water levels, sea level rise and settlements (as outlined in 6.3.4), this crest height is enlarged.

The final crest level will be R_C = 8.9+1.0+0.5+0.8=11.2 m. above MSL.

This gives a preliminary design as presented in the figure below.

Figure 6.8

Preliminary caisson design according to guidelines PIANC [VERHAGEN *et al.*, 2009]



The length of the caisson can be chosen somewhat arbitrarily. However, several sources (under which Goda) recommend caissons lengths up to 45 – 50 m. at maximum. To limit the number of caissons that needs to be sunken down, this latter caisson length of 50 m. will be used within the design.

The above preliminary design will be verified on a stability analysis, which is subject to the next paragraph.

6.4.3 STABILITY ANALYSIS

With the above determined caisson dimensions, the preliminary design was presented in figure 6.8. This design needs to be verified by means of a stability analysis.

For this, the methodology according to Goda has been adopted [VERHAGEN *et al.*, 2009]. With these formulas the wave pressures on the caisson are calculated, and the stability is checked by means of a safety factor. The caisson will be checked for the stability under sliding and overturning.

Storm design wave

The actual design wave heights for both limit states and their accompanying periods have already been determined in paragraph 6.3. These wave characteristics are again summarized below.

Table 6.4

Limit state wave heights and periods

Limit states characteristics	SLS	ULS
Wave height H	4.19 m.	8.45 m.
Wave peak period T_p	11.00 s.	14.91 s.
Wave period 1st negative spectrum moment $T_{m-1,0}$	9.02 s.	12.23 s.

With these values it will be assessed whether the caisson meets the criteria regarding buoyancy and uplift pressure, after H_{ss} has been translated into H_{max}. With the wave height H_{ss}=8.45 m. which occurs once in 1/475 years, a safe value will be used in accordance with Goda.

Elevation of wave pressure

The exact elevation of a wave crest along a vertical wall is difficult to assess because it varies considerably from 1.0H to more than 2.0H, depending on the wave steepness and the relative water depth. However, the following simple formula can be used:

$\eta^*=0.75^*(1+\cos\beta)H_{max}$

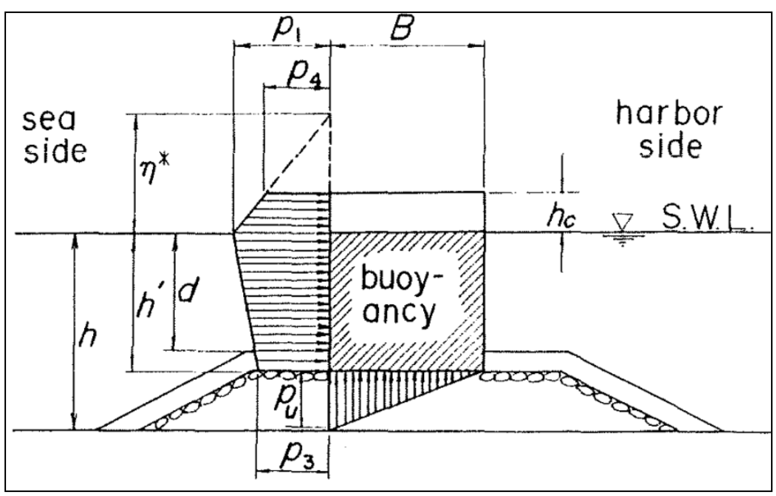
For waves of normal incidence (the most unfavourable scenario) this gives an elevation of $\eta^*=1.5^*H_{max}$

Wave pressure on front of vertical wall

The distribution of wave pressure on an upright section is sketched in figure6.9.

Figure 6.9

Wave pressures on a vertical breakwater according to Goda [VERHAGEN *et al.*, 2009]



The wave pressure takes the largest intensity p_1, at the design water level and decreases linearly towards the elevation η* and the sea bottom, at which the wave pressure intensity is designated as p_2. The largest pressure occurs at the extreme water level (CD +1.0 m.)
The intensities of the wave pressures are calculated by the following formulas:

$p_1=0.5(1+\cos\beta)(\alpha_1+\alpha_2\cos^2\beta)w_0H_{max}$
$p_2=p_1/\cosh(kh)$
$p_3=\alpha_3 p_1$
$p_4=\alpha_4\, p_1$
$p_u=0.5(1+\cos\beta)\alpha_1\alpha_3 w_0H_{max}$

In which:

$\alpha_1=0.6+0.5[2kh/\sinh(2kh)]^2$
$\alpha_2=\min\{[(h_b-d)/3h_b](H_{max}/d)^2,2d/H_{max}\}$
$\alpha_3=1-(h'/h)[1-1/\cosh(kh)]$
$\alpha_4=1-h_c^*/\eta^*$
$h_c^*=\min\{\eta^*;h_c\}$

With η^* = 1.5*8.45=12.7 m. and h_c = (11.2-1) = 10.2 m. from which follows that h_c^* = 10.2 m.

Also, h_b denotes the water depth at the location at a distance of $5H_u$ seaward of the breakwater. This is a distance of 5*4.19=21 m. From the site data, it can be deducted that the natural sea bed exhibits a slope of 1:70. This results in a depth h_b of:

h_b=45.35+1+21/70=46.65 m. under extreme water level conditions.

These values have been calculated in annex 2 (see A2.14), which has resulted in the following pressures:

Table 6.5
Resulting wave pressures on the vertical wall caisson

Pressures	[kN/m^2]
p_1	63.75
p_2	39.28
p_3	49.84
p_4	12.45
p_u	48.84

The stability of an upright breakwater against wave action is examined for the following modes of failure: sliding and overturning. For these modes, the calculation of a safety factor is a common practice of examination. The safety factors against sliding and overturning are defined by the following [VERHAGEN *et al.*, 2009]:

Sliding: S.F.=μ(W-U)/P
Overturning: S.F=(W*t-M_U)/M_P

In which:

M_P = moment of total wave pressure around the heel of upright section
M_U = moment of total uplift pressure around the heel of upright section

P = total thrust of wave pressure per unit extension of upright section
t = horizontal distance between the center of gravity and the heel of upright section
U = total uplift pressure per unit extension of upright section
W = weight of upright section per unit extension in still water
μ = coefficient of friction between the upright section and the rubble mound

The safety factors against sliding and overturning are must at least be equal to or greater than 1.2. The friction coefficient μ between the concrete upright section and the rubble stones is taken equal to 0.6 [VERHAGEN *et al.*, 2009].

Total pressure, uplift, and their moments

P=0,5*(63.75+49.84)*26.35+0.5*(63.75+48.84)*10.2=2070.8 kN/m
M_P = 20521.9 + 11777.3 = 32299.2 kNm/m
U = 0.5*19.5*48.84=476.2 kN/m
M_u = 2/3*19.5*476.2=6190.5 kNm/m

Stability of upright section

For this, the specific weight of the caissons will have to be known. As outlined before, the caissons will be perforated wall caissons to enhance wave dampening. Because of this, a wave energy dissipating chamber is present in the caissons, which adds less to the total weight of the caisson.

In a first estimate, it is assumed that 75% of the cross-sectional area is only filled with concrete with a density of 2400 kg/m^3 (the dissipation chamber takes up 25% of cross-sectional area). This means an average density for the cross-section of 1800 kg/m^3. This can be enlarged (for 50%) when adding sand which has a higher density. So the overall density of the caisson is assumed to be: 0.5*2400+0.5*2650=1925 kg/m^3. This means a specific weight of 18.9 kN/m^3. The total weight of the caisson becomes:

Wa=18.9*(26.35+10.2)*19.5=13,470.5 kN/m, dry weight
W=13,470.5-10.10*26.35*19.5=8280.9 kN/m, weight under water

With the safety factor as defined above, the stability of the caisson can be calculated:

Against sliding: S.F.=0.6*(8280.9 -476.2)/ 2070.8=2.26
Against overturning: S.F.=(8280.9*1/2*19.5-6190.5)/ 32299.2 =2.31

The above described factors are all larger than 1.2, so that the safety of the caisson against sliding and overturning is achieved, and the caisson is stable for the design wave height H_r=8.45 m. However, because of these large safety factors, some optimization of the caisson is certainly possible (more slender design).

6.4.4 WAVE DAMPENING MEASURES

The above presented stability analysis is valid when considering the design wave from the outside (the design storm wave height). From the inside other criteria are relevant: wave dampening is necessary. This can be accomplished by constructing perforated front wall caissons, with wave dissipating chambers inside.

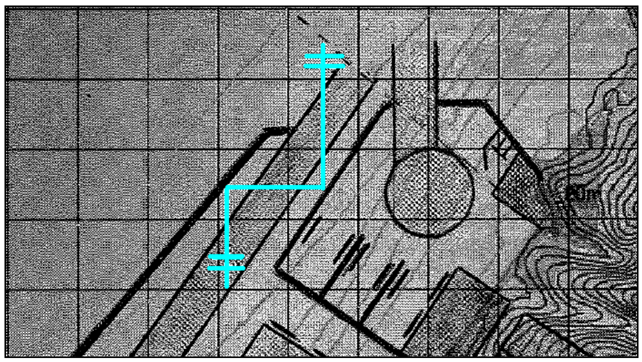
To achieve in-port wave dampening, the dampening chambers are located at the port side of the caisson. With these chambers, additional pressures occur within the caisson, which leads to different critical situations and these scenarios will need to be assessed. This is subject to this paragraph.

Target waves for wave absorption

The target waves that need to be dampened at the inside of the port follow from the model runs in the previous chapter. It was emphasized that waves from the directions 330°N and 0°N posed the largest problems (in-port wave penetration) because of their reflection on the outside of the bulk port breakwater and subsequently on the inside of the container port breakwater, see the figure below.

Figure 6.10

Directions of incoming and reflected wave rays



These waves will have to be dampened. As these waves concern the wave climate around calculation point P60, these conditions will be used here. Besides this, the scenario applies to the serviceability limit state, where vessels can manoeuvre and (un)load in the container port. From the wave penetration study, the most important results for design of the vertical breakwater were:

- After improvement of the reflection coefficients of the breakwaters, waves entering the container port are dampened very well for wave directions perpendicular to the breakwaters. Because of this, the availability of the berths for the directions 300°N, 330°N, 0°N amounts to nearly 100%. With this conclusion, the aimed reflection coefficient of K_r=0.6 will have to be achieved.
- The entrance conditions in the lee of the breakwaters allow tying up of tugs to vessels for around 98% of time. Due to diffraction, reflection and waves larger than H=2.0 m. from 30°N, for 2% of the time the wave height is exceeded for tugs tying up (30°N with H>1.75 m., 0°N with H>2.25 m., 330°N with H>2.25 m., 300°N H>2.75 m.)

Waves from the direction 30°N will notice little influenced from the dampening measures at the inside of the northwestern container port breakwater, as these waves are directed parallel. Regarding the directions 300°N, 330°N and 0°N specially waves smaller than H=2.75 m. will need to be dampened, as waves larger than H=2.75 m. will not allow tugs to tie up in the lee of the entrance breakwaters. The lower limit of the waves that pose problems (due to reflection) to the (moored) vessels starts at H>0.75 m.

So the target waves for wave absorption are waves with a height between 0.75 < H < 2.75 m.

Vertical slit caisson

To achieve dampening of these determined target waves, a caisson with a wave energy dissipation chamber will be designed. For this design, a vertical slit caisson has been chosen. Because of the large water depth, only the upper part of the caisson needs to be equipped with a wave dissipation chamber, which takes up only a small part of the caisson's cross-sectional area. This vertical slit caisson can be applied in large water depths, and has been used in several projects [TAKAHASHI, 2002]. First of all, the wave dissipation chamber will be designed.

Dissipation chamber

Openings and chamber depth

The optimal opening ratio of the front wall of the caissons has been determined in experiments [TAKAHASHI, 2002] and is usually designed between 15% and 30%. While in this case the caissons height here is rather large, (25.35+11.2=36.55 m.) the opening ratio can be on the lower side.

To get an indication for the height of the openings in the front wall, the low and high water levels are considered, and subsequently one wave height above and below them.
This results in an opening height of around 7 m.

Taking into account these two criteria, as a compromise an opening height of 8 m. has been adopted to get a higher front wall opening ratio. Earlier it was argued that the caisson measured 50 m. in length. In each caisson, 35 openings will be constructed with a width of 0.7 m. This means an opening ratio of around 12%. Considering the large depth of the caisson, it is assumed that this is sufficient.

The bottom of the wave chamber is located at around 1 wave height below LAT, which means a wave chamber depth of d'= 3.35 m. at MSL, and d'=4.35 when considering the extreme water level. From this it follows that ratio q between the chamber depth and the caisson height amounts to q=3.35/25.35=0.13. (or: q=4.35/26.35=0.17)

Chamber length

The wave dissipation chamber length can be determined after reviewing experiments according to Tanimoto and Yoshimoto [TAKAHASHI, 2002]. It appears that reflection coefficients of K_r<0.6 are possible with a relative wave chamber length of l'/L' between 0.085 – 0.385. The longest waves of L' occur at the largest (chamber) water depth of d'=4.35 m.

With the calculated values for L' this means for a lower limit L'=30.3 m. and for an upper limit wave length L'= 76.8 m. With these wave lengths, a chamber length is chosen of l' = 8.5 m., which amounts to l'/L'=0.281 for L'=30.3 and l'/L'=0.111 for L'=76.8 m. With a wall thickness of l_p= 1 m., the total chamber length becomes l=9.5 m. In the (optimal) range between 0.1< l'/L' <0.3 (the range in which the chamber dimensions are chosen), wave dissipation is even more enlarged and reflection coefficients of K_r<0.4 are achieved within this range [TAKAHASHI, 2002].

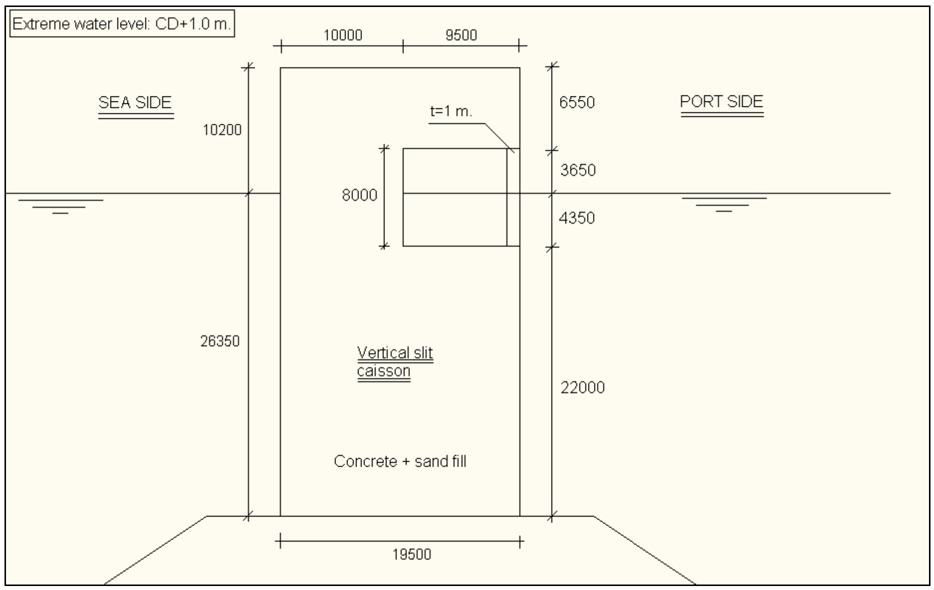
It is emphasized that the aimed wave dampening is not always exactly reached as defined by the reflection coefficient, due to the irregular character of incident wave heights and directions. However, because of the fact that the achieved reflection coefficient ($K_r \leq 0.4$) is at least 33% lower than the preliminary chosen reflection coefficient in the wave penetration study (K_r=0.6), it is assumed that this design meets the requirements to ensure sufficient wave dampening for the critical directions and the specific wave range between 0.75 < H <2.75 m.

Caisson cross-section

With the above defined parameters, the vertical slit caisson cross-section can be drawn up. This cross-section has been presented in the figure below.

Figure 6.10

Cross-section designed vertical slit caisson



As a last remark, some earlier made assumptions need to be checked. It was calculated before that the stability of the caisson was achieved even for the most severe conditions (H=8.45 m.), the ULS.

Earlier, it was assumed that roughly 75% of the cross-section would consist of concrete (see 6.4.3), as the wave dissipation chamber took up 25% of the cross-sectional area. A quick glance at the drawing presented above already shows that the wave dissipation chamber takes up much less of the cross-sectional area.

With the designed cross-section including the wave dissipation chambers, in a rough estimate it can be assumed that for a part of 9.5*8.0=76 m^2 would be empty. This means that 100-76/(19.5*36.55)*100=89% of the cross-section consists of concrete. As was outlined earlier, the weight of the caisson can even be enlarged by filling the caisson with sand. From this it can be concluded that even with he wave dissipation chambers included, the cross-section maintains its stability (with design storm waves from the outside).

6.4.5 WAVE FORCES

Nevertheless, with the addition of the wave chamber, the wave forces on the inside of the vertical slit caisson differ from the pressure calculations according to Goda. It has to be checked whether these pressures do not lead to critical situations.

For this, the wave pressure distributions at several important phases need to be evaluated: the critical forces on the sides of the caisson reach their peaks at different phases. This means that the largest possibility of the caisson sliding or overturning force does not necessarily occur when the wave crest is just in front of the caisson (largest pressures).

Generally, the wave forces on a perforated wall caisson are smaller than on conventional solid caissons [TAKAHASHI, 2002]. However, this will need to be evaluated.

Modified wave pressures

Because of the vertical slit wall in the caissons, the wave pressures on the caisson change. Even more, the pressures are different for each phase of the wave (e.g. wave crest and trough). The different (possibly critical) phases are indicated in the figure below.

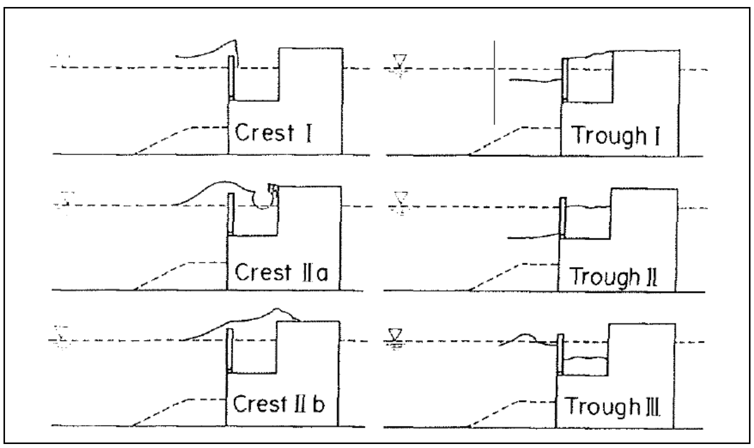


Figure 6.11

Possibly critical phases for pressure calculations [TAKAHASHI, 2002]

The extended Goda-formulas can be used in combination with several modification factors λ depending on the structural type. These factors also differ for various stages and scenarios where the e.g. the wave crest or trough hits the structure. The table below presents these factors for the different stages.

Table 6.6

Modification factors for the wave pressures in case of a vertical slit caisson [TAKAHASHI, 2002]

		Crest-I	Crest-IIa	Crest-IIb
Slit Wall	λ_{S1}	0.85	0.7	0.3
	λ_{S2}	0.4 $(\alpha^* \leqq 0.75)$ $0.3/\alpha^*$ $(\alpha^* > 0.75)$	0	0
Impermeable Front Wall	λ_{L1}	1	0.75	0.65
	λ_{L2}	0.4 $(\alpha^* \leqq 0.5)$ $0.2/\alpha^*$ $(\alpha^* > 0.5)$	0	0
Wave Chamber Rear Wall	λ_{R1}	0	$20l/3L'$ $(l/L' \leqq 0.15)$ 1.0 $(l/L' > 0.15)$	1.4 $(H_D/h \leqq 0.1)$ $1.6\text{-}2H_D/h$ $(0.1 < H_D/h < 0.3)$ 1.0 $(H_D/h \geqq 0.3)$
	λ_{R2}	0	0.56 $(\alpha^* \leqq 25/28)$ $0.5/\alpha^*$ $(\alpha^* > 25/28)$	0
Wave Chamber Bottom Slab	λ_{M1}	0	$20l/3L'$ $(l/L' \leqq 0.15)$ 1.0 $(l/L' > 0.15)$	1.4 $(H_D/h \leqq 0.1)$ $1.6\text{-}2H_D/h$ $(0.1 < H_D/h < 0.3)$ 1.0 $(H_D/h \geqq 0.3)$
	λ_{M2}	0	0	0
Uplift Force	λ_{U1}	1	0.75	0.65
	λ_{U2}	0	0	0

This leads to the following adapted expressions for the wave pressures:

$\eta^*=0.75^*(1+\cos\beta)H_{max}\lambda_1$

$p_1=0.5(1+\cos\beta)(\lambda_1\alpha_1+\lambda_2\alpha_2\cos^2\beta)w_0H_{max}$
$p_2=p_1/\cosh(kh)$
$p_3=\alpha_3p_1$
$p_4=\alpha_4p_1$
$p_u=0.5(1+\cos\beta)\alpha_1\alpha_3w_0H_{max}\lambda_3$

In which:

$\alpha_1=0.6+0.5[2kh/\sinh(2kh)]^2$
$\alpha_2=\min\{[(h_b\text{-}d)/3h_b](H_{max}/d)^2; 2d/H_{max}\}$
$\alpha_3=1\text{-}(h'/h)[1\text{-}1/\cosh(kh)]$
$\alpha_4=1\text{-}h_c^*/\eta^*$
$h_c^*=\min\{\eta^*,h_c\}$

As the cross-sectional dimensions of the caisson remain the same, h_c=8.9 m. and $\eta^*=1.5H_{max}\lambda_1$. For this extreme situation for the pressure calculation, it is not realistic to assume an incoming design storm wave height of H_{ss}=8.45 m. <u>at the inside</u> of the caisson. Waves arriving at this location (C2, see figures 6.5 and 6.10) have already been reflected by the bulk port breakwater with a reflection coefficient of K_r=0.6.

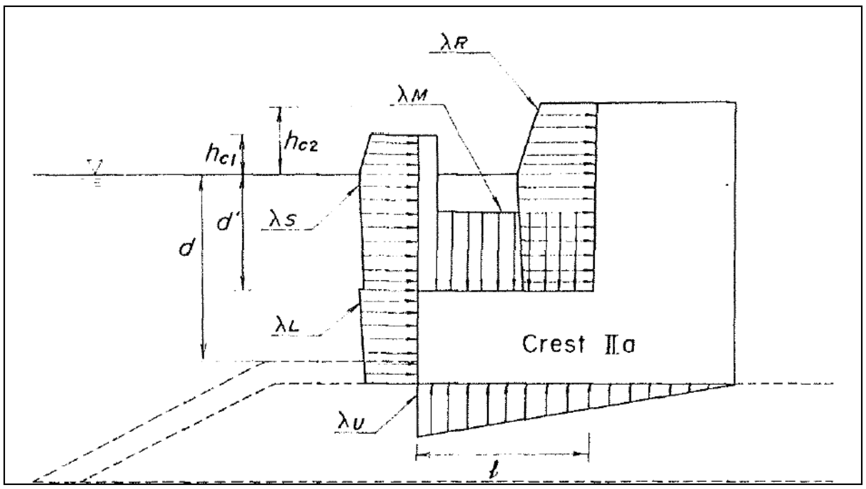
So as an upper limit, the wave height at the inside of the breakwater amounts to H_i=0.6*8.45=5.07 m. With the relationship between H and T determined in chapter 5, the accompanying period becomes $T_p=5.9149*5.07^{0.4332}$=11.95 s. It is emphasized that the waves and the pressures are the largest in this critical situation. These waves are larger than the target waves for wave dissipation in the caisson's chambers. However, in this critical situation (ULS) wave dampening is of secondary importance, and stability of the breakwater of primary importance.

Critical situation
With this smaller wave height at the inside than the design storm wave height at the outside of the caisson, the wave-induced pressures will generally also be smaller. When considering the earlier mentioned scenarios, crest-I will never be critical as all the factors are equal to or smaller than unity, resulting in lower pressures.

With a high mound (d/h=0.5) and wave heights larger than H/h>0.3, crest-IIa becomes critical [TAKAHASHI, 2002]. This is however not the case as H/h=5.07/46.35=0.11.

The peak sliding or overturning force almost always occurs at crest-IIb [TAKAHASHI, 2002]. This is also in line with tests performed by Takahashi et al [1991]. Because of this, scenario crest-IIb will be assessed in more detail. Wave troughs with their accompanying negative wave pressures are not a critical factor affecting the stability of the caisson under normal design conditions [TAKAHASHI, 2002]. The figure below presents the resulting pressures on the vertical slit caisson.

Figure 6.12
Wave pressure distribution on a perforated wall caisson for crest-IIb [TAKAHASHI, 2002]



Wave pressures on slit wall: λ_1=0.3 and λ_2=0
P_{middle}= 9.58 kN/m^2
p_{upper}= 0 and reaches to a level of +2.3 m. above extreme water level
p_{lower}= 8.99 kN/m^2

Wave pressures on impermeable wall: λ_1=0.65 and λ_2=0
p_{upper}= 19.49 kN/m^2
p_{lower}= 13.02 kN/m^2

Wave pressures on rear chamber wall: λ_1=1.4 and λ_2=0
p_{middle}= 44.73 kN/m^2
p_{upper} = 29.52 kN/m^2
p_{lower} = 41.97 kN/m^2

The total pressure (for sliding) and the moment are:

$P = P_{outside} + P_{chamber} = 409.0 + 324.08 = 733.08$ kN/m
$M_p = M_{p,outside} + M_{p,chamber} = 5470.8 + 8364.1 = 13{,}834.9$ kNm/m

These factors are all about a factor 3 smaller than calculated earlier, where the caisson was already stable against sliding (and overturning). It can safely be assumed that this will subsequently also be the case in this scenario.

Uplift

This also applies for the stability against uplift: the total uplift can only be smaller for crest-IIb than calculated for a solid wall caisson. This is because of the weight of the water inside the wave dissipation chamber that works in the positive (downwards) direction. Because of this, uplift will not occur, and the safety factor is even larger than the calculated value of 2.3 (with an assumed 75% cross-sectional area that consisted of concrete which is in fact even more).

From the above it can be concluded that not only the caisson is stable against sliding and overturning for the most severe (critical) outside wave height of H_{ss}=8.45 m., but also with wave dampening chambers included, when reviewing the most critical situation on the inside of the breakwater. These alternate wave pressure induced forces are always smaller than the wave conditions on the outside, which results in the conclusion that the designed caisson is stable.

The designed vertical slit caisson fulfils its function for wave dampening and the design is stable. As a last remark it is emphasized that the actual wave dissipation of this breakwater will however need to be investigated in precise model tests.

6.4.6 FOUNDATION BED

In the paragraphs above, the dimensions of the rubble mound bed where the caisson is placed on top have already been defined. However, the breakwater can also fail due to failure of the rubble mound foundation, which necessitates attention when designing its composition.

Weight of armour

The bed where the caisson is placed on top is located at depths larger than CD – 25 m. At this large water depth, wave action is very limited. Besides the limited wave action, currents absent, as they were negligible small. For stone stability at this depth, only the (reduced) wave action is an important factor.

Brebner and Donelly proposed a method to directly determine the necessary stone weight from the wave height [TAKAHASHI, 2002]. The stable weight of armour units W can be expressed as:

$$W=(\gamma_r H_{1/3}^3)/\{N_s^3(S_r-1)^3\}$$

In which:

γ_r = specific weight of the armour unit
$H_{1/3}$ = design wave height
N_s = stability coefficient
S_r =specific gravity of stone

N_s depends on variables such as the shape of the armour units, their manner of placement, the shape of the rubble mound foundation and wave conditions (height, period and direction). For two layers or quarry stones, the following formula applies:

$N_s = \max\{ 1.8 ; 1.3\{(1-\kappa)/\kappa^{1/3}\}(h'/H_{1/3})+1.8\exp[-1.5\{(1-\kappa)^2/\kappa^{1/3}\}(h'/H_{1/3})]$

In which:

$\kappa=\kappa_1(\kappa_2)_B$
$\kappa_1 = (2kh')/\sinh(2kh')$
$(\kappa_2)_B = \max\{\alpha_s \sin^2\theta \cos^2(kB_M \cos\theta); \cos^2\theta \sin^2 (kB_M \cos\theta)\}$

B_M = berm width = 6 m.
α_s = 0.45
θ = angle of wave incidence
k = wave number ($2\pi/L'$)

For the angle of wave incidence of θ=60°, the required weight for the armour stones is the largest [TAKAHASHI, 2002]. This direction will be used as input for the design calculations, as oblique waves are expected at the north-western container port breakwater. The stone stability becomes critical for the lowest water level CD + 0.0 m. This means h'=25 m., H_{ss}=8.45 m., k=0.02911474 and L'=215.8 m. With this, the factors become:

κ_1= 0.718
$(\kappa_2)_B$ = 0.335
κ = 0.241

Which gives N_s = 4.73 resulting in a stone mass of 3913 kg, or 3.91 tons. This means a diameter of 1.14 m. Thee effective layer thickness is calculated as follows [VERHAGEN *et al.*, 2009]:

$t=n*k_t*d_{n50}$ =2*0.91*1.14=2.1 m.

To provide a geometrically impermeable filter, the material under the armour stones will have a weight of around 1/10 of the armour stones, resulting in a core material for the rubble mound foundation of 0.15-0.5 t.

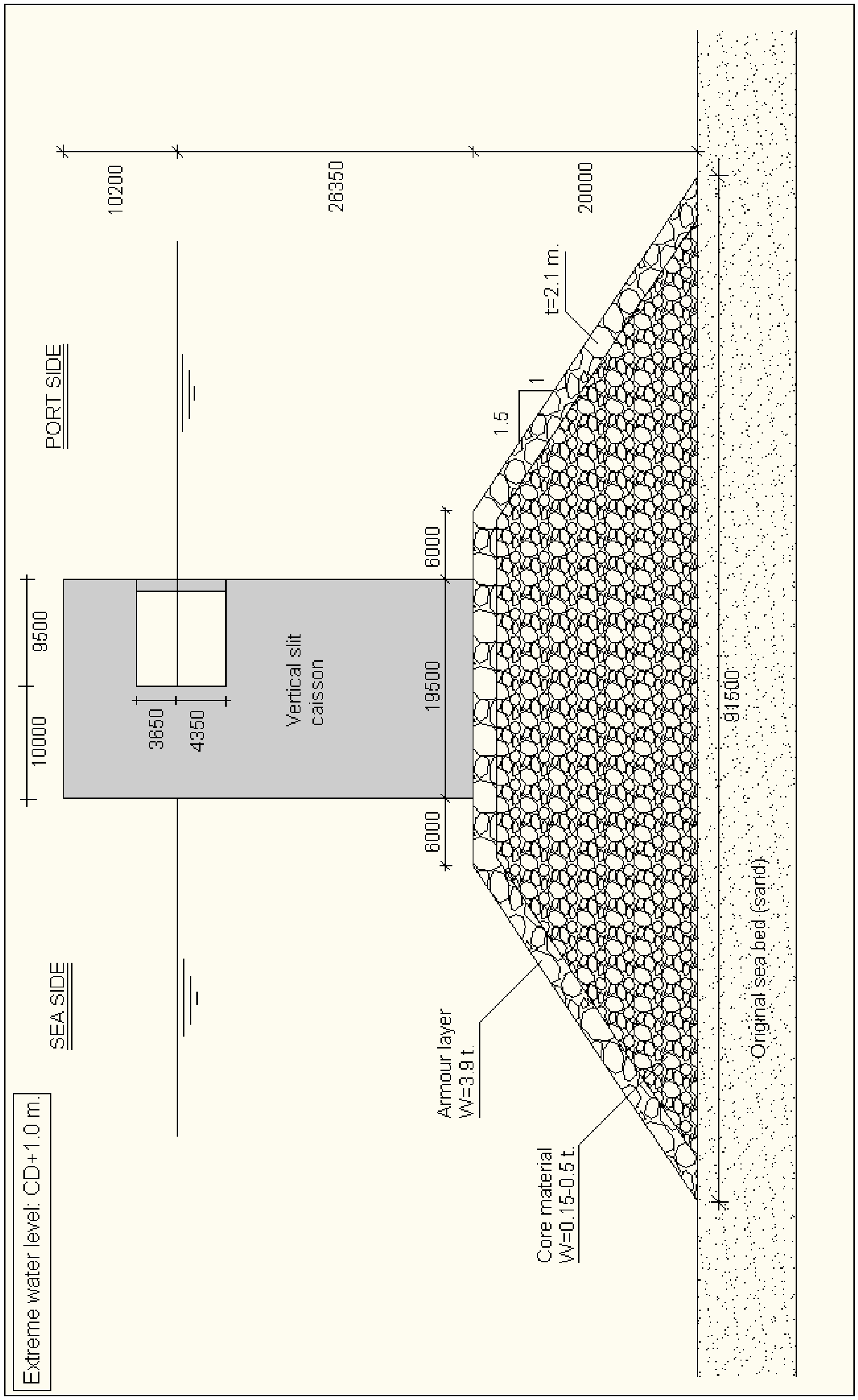
To create a geometrically impermeable filter with the present bottom material, a geotextile will be applied underneath the foundation to prevent scouring of the sand underneath through the rubble mound.

6.4.7 DESIGN

With the above calculated parameters and dimensions for the vertical slit caisson and the rubble mound foundation, the vertical composite breakwater design can be finalized.
The overall design has been presented in the figure below.

Figure 6.14

Design vertical slit breakwater on foundation bed at a depth of CD- 45 m.



6.5 CONSTRUCTION SPECIFICS

Within this concluding section, specific problems regarding the breakwater construction and layout will be addressed, under which the construction methodology and several other subjects.

6.5.1 RUBBLE MOUND BREAKWATER

Construction methodology

The order in which the rubble mound breakwater will be constructed is elaborated in detail below.

Filter

As a first step in the construction of the rubble mound breakwater, the sandtight geotextile filter will be applied on the original seabed. The sandtightness of the geotextile is required to prevent the original seabed material from being washed out. From a vessel the geotextile can be rolled out, and floated into place, where they are sunken down by (side)dumping initial ballast on the geotextile [VERHAGEN *et al.*, 2009]. The strength of the geotextile can be obtained accordingly, by using geotextile of the desired material. A considerable quantity of material is necessary to fix the sheet in position.

Core

The initial ballast placed on the geotextile to keep it into place can subsequently be replenished with more core material of W=0.15 – 0.5 t. This dumping can be done by split barges, which are usually used for bulk placement of material and realizing a controlled flow of material, where water depths are large enough for these barges.

Combined with this, the upper part of the breakwater (core) profile must be realized by dumping the rubble material with dump trucks from the landward side on, if the available water depth is smaller than the dumping vessels' draught (from water depths smaller than d<4 m.).

First under-layer

When the core has progressed sufficiently far into the seaward direction, the construction of the first under-layer will commence, beginning from the landward side onwards.

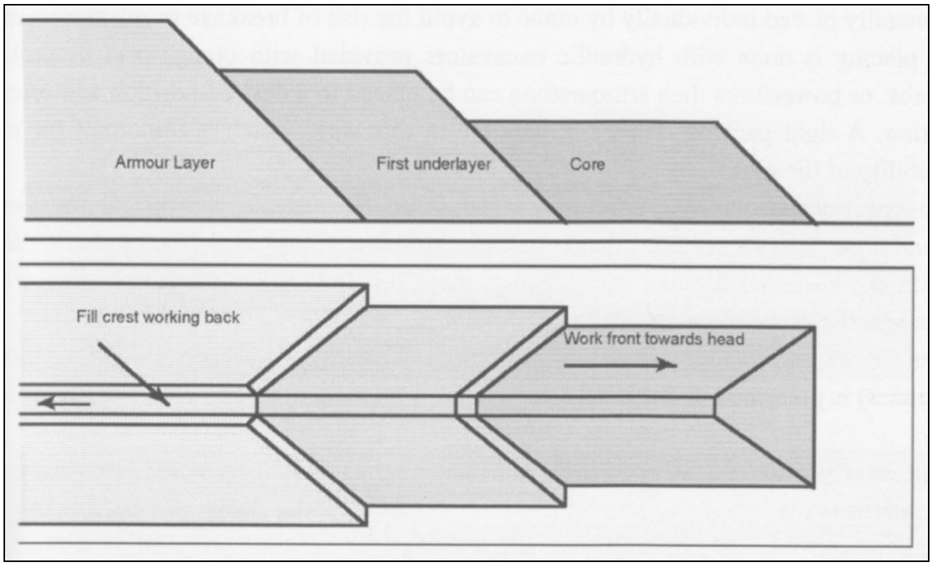


Figure 6.15
Progression of core and subsequent layers during breakwater construction [VERHAGEN *et al.*, 2009]

For this layer, controlled placement of stones with a weight of W=3-6 t. will have to be executed. For the lower cross-sectional profile of the breakwater, use can be made of side-stone-dumping vessels. The upper profile of the breakwater will again be constructed from the landside onwards, by using dump trucks and excavators for controlled placement.

Accropode II units
Construction of the armor layer with the large Accropodes II will have to be done by a combination of land-based and waterborne equipment. Under water placement of the Accropode II units can be done with cranes on pontoons using GPS. From there on, the upper part of the armor layer can be constructed from the crest, using a crane with a safe remote-release hook.

The Accropode II has a placing time of 10 minutes per unit. It is therefore recommended that several cranes will be used simultaneously for the placement, in order to minimize the construction time.

With the application of concrete armor units, a concrete mixing plant is required as well as a block casting area and a storage area for the armour units. These should all be located nearby.

Risks during construction
One of the main problems during breakwater construction will be the stability (or vulnerability) of the structure during every construction phase, the accessibility of the work front and the safety of the equipment [VERHAGEN *et al.*, 2009].

In the semi-arid climate in this part of Morocco, the rainfall is concentrated in autumn and winter. The rainfall events are generally short and intense. The highest yearly rainfall for 30 years was measured in the season of 2008 – 2009, which also applies for the monthly rainfall: in October 2008 the highest monthly precipitation was appointed at 268 mm, which approximately equals the average yearly rainfall.

In these times of (heavy) rainfall (autumn and winter) it is certainly not advised to execute construction works of the breakwater. This is not only because of the more severe weather conditions during these seasons (higher wind speeds → higher waves), but also because of the fact that in times of rainfall the wadis discharge large amounts of water and sediment. The large discharge from the Rio Kert generates large currents, which will cause serious problems for the stability of the work front, which cannot withstand the design storm. Besides the (unprotected) core material large hindrance will be caused to the work equipment (e.g. stability of pontoons) and personnel.

To minimize these problems and the related construction risks, it is advised to start the construction of the breakwater just after the winter, during spring and summer seasons. It is expected that the whole structure can be completed within this calm period, which considerably reduced the risk. If it turns out that this is not the case, construction can be interrupted during the rough seasons on the condition that the work front is well protected.

With this in mind, it is also advised to keep the distance between work fronts as small as possible. If damage should occur, it will only be restricted to a small stretch of the structure.

6.5.2 VERTICAL COMPOSITE BREAKWATER

Construction methodology

The sequence of construction of the vertical composite breakwater with several specifics will be elaborated in detail below.

Vertical slit caisson

According to the cross-sectional design as outlined in the previous paragraph, the vertical slit caisson will be made of concrete. The dimensions of the caisson are large: 50x36.6x19.5 m^3. Because of this, the most convenient caisson construction system will be the production of large prefabricated units, instead of for example construction out of smaller elements, or in-situ casting.

The caissons will have to be constructed in a (nearby located) building yard. Assumed that the buoyancy of the caissons is sufficient, they can be constructed in the dry and brought afloat by allowing water to enter the building dock. The openings in the caisson will have to be (temporarily) closed off by means of steel hatches to prevent water from entering the caisson.

Transport

After completion of the caissons, they have to be transported to the project location. It is essential that sufficient depth and keel clearance is available throughout the route from the dock to the site. This will necessitates dredging around the construction dock, but the large average depth of the Mediterranean Sea will generally pose no problems for transportation of the floating caissons. Even at the project location itself, depths are larger than 25 m., which should be more than enough.

The transport (or: towing) of the caissons generally occurs with vessels provided with sufficient propulsion and a good manoeuvring capability. For this, use can be made of tugboats.

Floating stability

Although the caissons have been designed to withstand forces in the final state after breakwater construction (ULS), also the dynamic stability of the caisson during transport will need to be evaluated [VERHAGEN *et al.*, 2009]. For this analysis, a caisson cross-section as presented in the figure below has been designed.

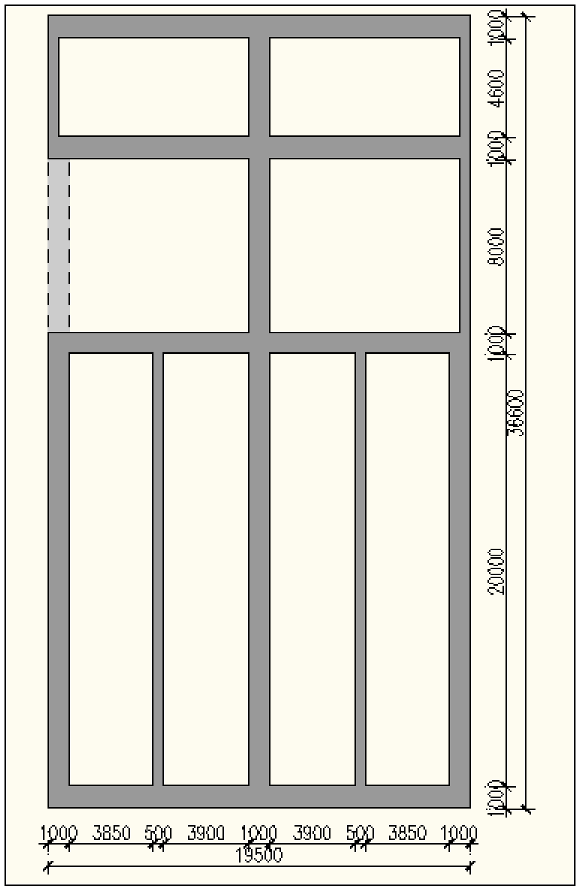


Figure 6.16

Designed vertical slit caisson cross-section with indicated dimensions (in mm)

The wave dissipation chamber is clearly visible. As it was defined below, the wall thickness on the outside (slit wall) amounts to 1 m. However, over half of the caisson length, this wall is perforated. Nett, this result in an (average) wall thickness of 0.5 m., the same wall thickness as the rear wall (for calculation simplicity and stability). Also, the caisson has been divided in several chambers, which can be filled with sand or water, which results in additional caisson weight for stability and sinking of the caisson.

First of all, the centre of gravity (g_b) above the bottom has been calculated. With the cross-sectional area of the concrete, the weight per meter caisson length has been determined (G). Subsequently, the moment of inertia (I) has been calculated, and together with the displacement (V) of the caisson in salt Mediterranean water, this results in MC=I/V.

The calculation of the moment of inertia I_y has been split up in contributions of several rectangles, according to figure 6.16. From this it follows that:

I_y=1/12*1*19.5^3 + 1/12*20*1^3 + 2*(1/12*20*0.5^3 + 20*0.5*4.65^2) + 2*(1/12*20*1^3 + 20*1*9.25^2) + 1/12*1*19.5^3 + 2*(1/12*8*0.5^3 + 8*0.5*9.5^2) + 1/12*8*1^3 + 1/12*1*19.5^3 + 1/12*4.6*1^3 + 2*(1/12*4.6*0.5^3 + 4.6*0.5*9.5^3) + 1/12*1*19.5^3 = 7470.45 m^4

Other calculated characteristics of the cross-section per meter length are presented in the table below.

Table 6.7

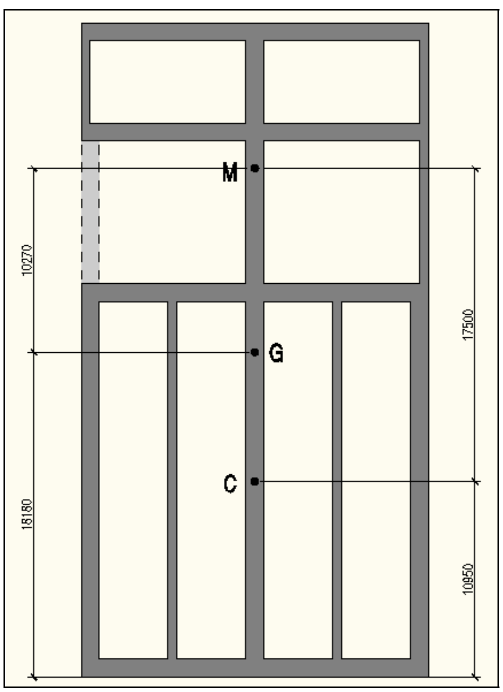
Dynamic stability characteristics for the vertical slit wall caisson. The metacentric height mc is positive, which results in a stable caisson during transport

Calculation of dynamic stability			
Cross-sectional area	A	183,2	m2
Weight of caisson	G	4396,8	kN
Centre of gravity above bottom	gb	18,177	m
Moment of inertia	Iy	7470,45	m4
Displacement	V	426,8738	m3
Draught of the caisson	dr	21,89096	m
Centre of buoyancy above bottom	cb	10,94548	m
Distance M-C	MC	17,50037	
Metacentric height	mc	10,26885	

The caisson is dynamically stable during transport, which can be verified as follows. The metacentric height amounts to m_c=MC+c_b-g_b=17.50+10.95-18.18=10.27 m. Because this value is positive (and large), this results in a dynamically stable caisson during transport [VERHAGEN *et al.*, 2009]. For clarification, these parameters have been presented in the figure below.

Figure 6.17

Locations of the parameters M,G and C (in mm) within the cross-section



Placing of the caissons

The vertical slit caissons will be placed on a beforehand prepared rubble foundation (see paragraph 6.4). This foundation bed should be constructed as flat as possible, so that the load of the caisson will be well spread over the bed. Uneven support may lead to failure of the bottom of the caisson by bending moments.

The construction methodology for the rubble foundation is analogue to the methodology for the rubble mound breakwater. First of all, the geotextile will be floated into place and subsequently sunk by applying more initial ballast. This initial ballast will be replenished with more core material, on which the armour layer can be constructed.

Because this part of the breakwater is an extension of the rubble mound breakwater, the execution of the construction work will be mostly done waterborne. Dumping vessels can construct the rubble foundation, and the caissons will be floated over them. By ballasting the chambers in the caisson with water or sand, and by removing the watertight hatches, the caisson will be sunk.

It is emphasized that this sinking operation should happen in a controlled matter, and during calm sea state conditions. It is not easy to keep the unit in position during sinking. It is recommended [VERHAGEN *et al.*, 2009] that at least two winches should be available to make a connection with for example the previously placed caisson, or tugboats that keep the caisson into place. The (large enough) under keel clearance of the caisson allows placement of the caissons during the whole tidal range.

Connections between caissons

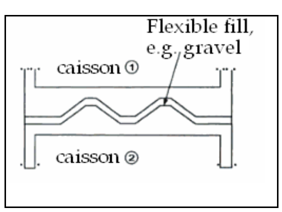
Because of the nearby located fault zone, there is a probability of earthquakes in the direct area of the project location (chapter 3). This necessitates special construction measures for the vertical composite breakwater.

The foundation bed, consisting of rubble material is because of its loose composition able to follow uneven settlements by (minor) displacements of the rubble stones. The vertical slit caisson placed on top follows these vertical settlements, as it rests on the foundation bed. However, at the transition of successive caissons, this will lead to height differences. The joints connecting two caissons at this location must be able to absorb these uneven settlements. Also horizontal displacements of the caisson can occur, and should be avoided in order to prevent meandering failure of the vertical caisson breakwater.

With these requirements for the caisson connection, a joint as presented below will be applied [AGERSCHOU, 2004]. The seam can be filled with gravel.

Figure 6.18

Joint between two adjacent caissons with flexible fill



As a last remark it is emphasized that the actual design of breakwaters for the bulk port depends strongly on the envisaged port expansion. If it is already decided up front that both ports will be built (including container terminal expansion), the bulk port breakwater be constructed less heavy (as it is party protected by the container port breakwater).
Before this, the large container port breakwater can be constructed at first, and subsequently the less heavy bulk port breakwater.

CHAPTER 7 Conclusions & Recommendations

In this final chapter resulting conclusions and recommendations are presented, mainly categorized according to the three main topics of the graduation project.

7.1 CONCLUSIONS

7.1.1 PORT MASTERPLAN

This designed masterplan layout meets all the objectives, requirements and throughput specifications as defined by the client (see 2.2.3 and 2.2.4):

Throughput specifications

- The dry bulk terminal achieves a throughput of 5,000,000 T/year in construction phases I and II, which was the maximum required throughput defined.

- The liquid bulk terminal achieves a throughput of 20,000,000 T/year in construction phase I and 40,000,000 T/year in phase II. These are both the maximum specified throughputs defined by the client.

- The container terminal achieves a throughput of 6,000,000 TEU/year in construction phase I, and 18,000,000 TEU in phase II. For phase I, the maximum required throughput is achieved. In phase II a throughput is achieved which lies between the minimum required and maximum required throughput (15-30 MTEU).

Additional objectives and requirements

- The port its throughput has been maximized on the available coastal stretch, even during every single construction phase. However for phase II, some economic considerations have been taken (partly) into account: in phase II a throughput of 30 MTEU is ultimately possible. However, this would require massive earth movements, resulting in an uneconomic design.

- Independent bulk and container port development is possible: the port masterplan layout provides a clear distinction between the two ports by a separating breakwater. Both ports have independent entrances and can be constructed independently from each other in different order, and even if the decision is made that one of both ports will not be constructed at all. For all these possible combinations, the cut & fill-balance has been satisfied.

- The hinterland connections have been indicated in the (various phases of the) total port masterplan layout, which can be connected to the already existing infrastructure depending on the construction phase and the road- and rail traffic. Besides this, surface area has been incorporated in the design for required liquid bulk refinery (in phase II), the gate areas and the Free Trade Zone.

Besides these required objectives and specifications defined by the client, a port masterplan has been designed that scores good on criteria as nautical ease, safety, preventing in-port sedimentation, protection against waves, port zoning and expansion possibilities (see 4.5).

It can thus be concluded that the designed port masterplan layout provides a good design for the construction of a large transshipment port at the specified project location with its practical construction constraints, and which at the same time meets all the specifications and objectives as defined by the client.

7.1.2 WAVE PENETRATION STUDY

In chapter 5, this resulting masterplan layout was assessed on the topic of in-port wave penetration and propagation. The wave study served two different purposes: first of all evaluating the in-port wave penetration with the predefined breakwater layout(s) and subsequently optimizing this predetermined breakwater layout, so that in-port wave penetration is minimized.

A preliminary wave penetration assessment was made according to the Coastal Engineering Manual [USACE, 2002]. From this it could already be concluded that:

- Because of the varying direction of wave incidence, in-port wave penetration and propagation is clearly of importance within the selected port masterplan layout.
- Especially the processes wave diffraction and wave reflection and their combined influence determine the in-port wave conditions.

After it became clear that the visual methods of the Coastal Engineering Manual fell short, a wave simulation model was used (which is in accordance with the required level of detail): DIFFRAC-2DH. From these simulation runs, the following became clear.

- Simulation runs with the original breakwater composition showed large wave reflection (especially at the container port entrance) due to monolithic caisson breakwater.

It was concluded that this had to be minimized in order to reduce the port's downtime. Use was made of wave dampening measures (low-reflectivity caissons), and a new set of simulations runs was carried out. From these it was evident that:

- Wave reflection against the monolithic caisson breakwaters is considerably reduced with the use of low-reflectivity caissons, and in-port wave penetration is substantially smaller (especially at the container port entrance)

- The improved breakwater configuration yields positive results for the availability of the liquid bulk berths and the container berths, as well as for the container port entrance: the downtime is roughly halved. For the dry bulk berths, also some (minor) improvements became clear.

The downtime for the dry bulk berths is in the order of 1%, and for the liquid bulk berths even less: 0.74%. However, bulk port entry not possible for 7% of time (tugboats cannot fasten to vessels outside the port). Container port entry is not possible for 1.46% of time. Besides this, it appears that there is one critical wave condition for 2-3 container berths: waves from 30°N lead to a downtime of 7.74% for these berths. The other berths (at least 15 in total) have a downtime of 0.43% and smaller.

These are good figures and acceptable as downtimes for berths and port entry.
The unavailability of 7.74% of 2-3 berths will not pose much of a problem, as at least 15 other berths are available (with an uptime approaching 100%) for (un)loading of vessels, where also an additional safety margin was included by choosing a lower berth occupancy. These specific berths should only be used when conditions allow it, which is the case for 92.26% of time at minimum.

With these wave dampening measures included, it can be concluded that the design of the port and its breakwaters is adequate in creating calm in-port berthing conditions: incident waves decrease considerably in height in-port, resulting in high availabilities of the berths, even with safe starting assumptions. The original port layout does not have to be altered. However, the proposed use of wave energy absorbing measures is a necessity to achieve calm in-port conditions (and thus a low downtime), and will have to be adopted in the design.
As a last remark it is concluded that port oscillations cannot be discarded completely: due to earthquakes, tsunamis, and seiches generated outside the port's boundaries, resulting in-port oscillations can lead to hindrance for berthed vessels. This will certainly have to be assessed in more detail during a final design with the use of mathematical models.

7.1.3 BREAKWATER DESIGN

In chapter 6, two critical breakwater cross-sections have been designed: a rubble mound breakwater with concrete armour units and a vertical slit caisson (to stimulate wave absorption) on a rubble mound foundation bed. In these designs certain construction constraints have been taken into account, such as the use as berth, the possibility of earthquakes in the area and the large water depth present.

Rubble mound breakwater

According to stone stability calculations [VERHAGEN *et al.*, 2009], the design has been based on not failing during design conditions (ULS), with a 1/475 year storm and a wave height of H_{ss}=8.45 m. The designed breakwater with 15 m^3 Accropode II units meets these requirements.

The designed crest height of the rubble mound breakwater (R_c=7.6 m.) allows a maximum overtopping rate of q=10 l/s/m during operational conditions. With this, the probability of overtopping of the breakwater is smaller than 10%, and no waves in the lee of the breakwater are generated (transmission).

It can be concluded that this specific location (taking into account construction constraints) a rubble mound breakwater design has been made that fulfils its primary functions: creating sufficiently calm in-port wave conditions and protection for berthed vessels during operational conditions (small wave overtopping and no transmission), and at the same time providing protection of port facilities during ultimate limit state conditions. Designed with single layer armour units, this design is a possible solution.

Vertical slit caisson on rubble foundation bed

With the stability calculations according to Goda [VERHAGEN *et al.*, 2009], also the designed vertical slit caisson breakwater does not fail during design conditions (ULS). As outlined above, also this design only allows a small overtopping discharge during design conditions, so that calm in-port wave conditions are realized even during the SLS.

With the cross-sectional design of the vertical slit caisson as presented in chapter 6, theoretically, reflection coefficients in the order of K_r=0.4 and smaller can be realized. This is smaller than the (preliminary) assumed value of K_r=0.6 during the wave penetration study, which is favourable: the actual wave dampening with this design can even be somewhat larger. However, it is emphasized that this value for the reflection coefficient is only valid for a very specific wave range.

The cross-sectional design of the vertical composite breakwater posed more of challenge because of the large water depth present (d=45 m.) and the required low reflectivity measures. However, with the designed vertical slit caisson it can be concluded that also this cross-sectional breakwater design fulfils its functions regarding protection of berthed vessels, creating calm in-port wave conditions (due to a small reflection coefficient K_r<0.6 and only allowing a small overtopping discharge) during operational conditions, and protection of port facilities during ULS conditions. The vertical slit caisson is a possible solution in this situation.

7.1.4 OVERALL

Resulting from the above, it can be concluded that it is certainly possible to construct a large transshipment port on de designated project location, which meets all objectives and specifications as defined by the client and all the environmentally induced constraints.
The new port's location is considered to be very favourable, as it is situated along an important intercontinental transport axis from America through the Mediterranean Sea to Asia. This new port will then be the largest African port present.

This large port will most likely attract new business and foreign investments (with the presence of a Free Trade Zone), which on its turn would lead to economic development of the northern region of Morocco. However, it remains to be seen if the aimed throughputs will actually be achieved. In the Mediterranean many other large ports are present where main line vessel call and goods are transshipped.

Besides this, these ports have a more lively and developed hinterland. Nevertheless, the construction of this port will be a very ambitious and prestigious project for Morocco and northern Africa and put will put this region on the world's shipping and port maps for sure.

7.2 RECOMMENDATIONS

When proceeding to a more final design, the level of detail increases and even more factors will play a role. Because of this, numerous recommendations can be given if the preliminary design proceeds into a more final design and assumptions need to be verified.
The recommendations are listed below, where a distinction in specific topics is made as much as possible.

7.2.1 ENVIRONMENTAL DATA

Because of the fact that partly only global and incomplete environmental was data available, several assumptions had to be made inevitably. Before proceeding with a final design, it is first of all recommended to complete this missing or incomplete environmental data.

- Although rather detailed data was available (estimated by [ALKYON DATA]) on wind and waves, it is recommended to equip the project location with measuring equipment to get the exact project site characteristics.

- More detailed soil characteristics are required (e.g. with soundings to determine the bearing capacity of the subsoil). This is not only relevant for the landward side, but also for the seaward side, where hard soils are present around the headlands. This could cause hindrance to dredging.

- It is recommended to assess the wadis at the project location in more detail: their catchment area, possible water discharge and maximum sediment load.

- The possibility of occurrence of rare events such as earthquakes and tsunamis will have to be determined, evaluated and subsequently included in a final design. In line with this, the activity of the nearby fault will have to be analysed, as well as the offshore location of the tectonic fault.

- For the vessel characteristics, several assumptions have been made regarding the expected types and sizes of vessels that will call at the new port. Throughout the whole design it will have to be taken into account that this represents only an approximation and not the real amount of shipping traffic, which can strongly differ in number and vessel size.

- It will have to be assessed whether the assumed vessel-arrival- and cargo-vessel distributions are realistic throughout the whole design: for example it is assumed that the call size of dry and liquid bulk amounts to 100%, and that all container vessels unload and subsequently are loaded with cargo. It is likely that this will not be the case for all vessels.

- Simulation models will have to be used to assess the vessels' manoeuvrability and hydrodynamics more precisely. This could lead to different operational and limiting wave criteria.

7.2.2 PORT MASTERPLAN

- The approach channel has been designed according to preliminary guidelines, resulting in a static required water depth. The influence of dynamic differences for various parameters will have to be analysed in more detail (e.g. maximum sinkage and the vessel motion due to wave response).

- Several assumptions were made regarding tugboats, such as the types, sizes and bollard pull. Exact specifications will have to be examined, which most likely leads to differences in tugboat requirements.

- A more detailed assessment of (adequate) possibilities of vessels manoeuvring and turning (with tugboats) inside the port's boundaries, basins and turning circles is also recommended. In line with this is the application of simulations models to evaluate the shipping traffic and possible congestion (especially around the container port entrance).

- The specifications of the various terminals will have to be determined in more detail, for example by applying queuing theory, simulation of the shipping traffic, and efficiency of terminal operations (equipment). Also, optimizing the used equipment between container ship, quay and storage yard can its influences in the required number of berths or terminal surface area. Also, the exact arrangement of terminals and hinterland connections has not been taken into account in large detail.

- The design of the tank farm of the liquid bulk terminal requires special attention: this tank farm will be constructed in cascades. Special attention will have to be paid to safety considerations.

- A comparison between various layouts can be done more accurately by using a more sophisticated approach (e.g. a more detailed MCA or with the aid of mathematical models).

- It is recommended to evaluate the port design in different phases of expansion considering topics of downtime, safety and nautical ease.

- Although the selected masterplan layout has been designed with satisfying the cut & fill balance in mind, it will have to be evaluated if the dredged material (from sea bottom or landward side) can actually be used as fill for the terminal construction.

- The fact that the outflow of the middle wadi discharges into the port certainly requires more in-depth study on the topics of outflow velocity, sedimentation and erosion rates. Especially in times of heavy rainfall it will have to be evaluated if problems are to be expected and for example the specific basin will need to be closed.

- Other considerations besides mainly technical (and efficiency) considerations have not been taken fully into account while designing the port masterplan. For example it is highly likely that for aesthetic reasons several layout improvement can be made, or a beach and natural reserve can be set up for recreational purposes.

7.2.3 WAVE PENETRATION STUDY

- In line with the required level of detail, many simplifications have been made with the application of the simplified model DIFFRAC-2DH and subsequently by the schematization of the port and its basins. Examples are uniform basin depths, no refraction and shoaling, absorbing boundaries and a uni-directional wave approach.

- Application of a more sophisticated wave-penetration model is recommended: e.g. a model according to the mild-slope-equation to account for non-uniform basin depths and including refraction, shoaling and wave dissipation as well as the exact spreading in wave direction and height. This output will yield more detailed results, and subsequently a more reliable berth and entrance downtime assessment can be made.

- A more detailed assessment of wave reflection at container port entrance and in dry bulk corner is recommended, as these locations turned out to be the most critical for wave reflection.

- It is advised to construct (especially the critical) container berths as a deck on piles. With this construction methodology, even more in-port wave dampening is achieved, resulting in a higher uptime of even the most critical berths.

- Port oscillations (e.g. forced through the port's entrance due to seiches, by earthquakes or tsunamis) cannot be discarded and will have to be analyzed in detail in a more final design.

7.2.4 BREAKWATER DESIGN

Rubble mound breakwater

- The rubble mound breakwater design has been made with the (arbitrarily) chosen Accropodes II armor units. It is recommended to analyze alternative designs which use other single layer amour units (e.g. the Xbloc). This could turn out to be more favourable for this situation in terms of for example concrete use.

- A rather large scour protection has been placed in front of the breakwater. However because of the large water depth it could be that the effects of wave action at the sea bottom are negligible, and a heavy scour protection is not necessary.

Vertical slit caisson breakwater

- The low-reflectivity caisson breakwater has been designed as a vertical slit caisson. Various other possibilities exist; it has to be analyzed in detail what alternative will be selected for the design.

- It is recommended that the actual achieved reflectivity of the vertical slit caisson breakwater will be assessed with model tests. This is due to the fact that the aimed (low) reflectivity of K_r<0.6 in this design is in reality not guaranteed: this strongly depends on occurring environmental conditions.

- The maximum allowed overtopping discharge in the ULS has been chosen somewhat conservatively: q<200 l/s/m. Because of the fact that only water is situated behind the breakwater, much larger discharges can be allowed. This will result in a smaller required crest height and a cheaper breakwater.

- For the ease of expansion and construction, caissons with a uniform height have been chosen. However, this means that the height of the foundation bed increases with increasing depth. Eventually, half of the water depth consists of a rubble mound foundation. It will have to be evaluated if the (expensive) caisson placed on top is actually cheaper than completing the total cross-section as a rubble mound breakwater.

- The design of the vertical slit breakwater can be further optimized, as the safety factors of the caisson are twice as large as required. For instance, a crown wall can be placed on top of the caisson to acquire the crest height.

- The bearing capacity of the stones of the rubble mound foundation should be verified, as the designed caisson is large and heavy. This has to be subject of a more final design.

Overall

- An assumption has been made for a probability of failure of the breakwater of 10%, resulting in a design wave height of H_{ss}=8.45 m. for a 1/475 year storm. It will have to be evaluated in detail if this (rather safe) assumption is an absolute necessity.

- Model tests will have to be performed to assess the actual overtopping discharge of both the breakwaters during operational conditions.

- The stability of both designed breakwaters will have to be assessed in more detail and for all possible failure modes.
- A geotextile has been applied under the toe of the rubble mound structures. It is recommended to analyze the possibility to construct a granular filter at this location: for example if it turns out that only a small layer of gravel will be sufficient, the construction costs will be lower.

- A complete risk assessment during breakwater construction will have to be made and the execution of work should be done accordingly.

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A1.14 – Default bulk port run 3, H_s=1.25 m, waves from 0°N, isolines: 1.5 m.
A1.15 – Default bulk port run 4, H_s=1.75 m, waves from 0°N, isolines: 1.5 m.
A1.16 – Default bulk port run 6, H_s=2.75 m, waves from 0°N, isolines: 1.5 – 2 m.
A1.17 – Default bulk port run 7, H_s=1.75 m, waves from 330°N, isolines: 1 – 1.5 m.
A1.18 – Default bulk port run 8, H_s=2.75 m, waves from 330°N, isolines: 1 – 1.5 m.
A1.19 – Default bulk port run 9, H_s=3.25 m, waves from 330°N, isolines: 1.5 – 2 m.
A1.20 – Default bulk port run 11, H_s=2.75 m, waves from 300°N, isolines: 1.5 – 2 m.
A1.21 – Default bulk port run 12, H_s=1.75 m, waves from 30°N, isolines: 1 – 1.5 m.
A1.22 – Default bulk port run 13, H_s=2.25 m, waves from 30°N, isolines: 1.5 m.
A1.23 – Default bulk port run 14, H_s=2.75 m, waves from 30°N, isolines: 1.5 – 2 m.
A1.24 – Default bulk port run 15, H_s=1.75 m, waves from 30°N, isolines: 1.5 m.
A1.25 – Improved bulk port run 1, H_s=2.25 m, waves from 30°N, isolines: 1.5 m.
A1.26 – Improved bulk port run 2, H_s=2.75 m, waves from 30°N, isolines: 1.5 – 2 m.
A1.27 – Improved bulk port run 3, H_s=1.75 m, waves from 0°N, isolines: 1.5 m.
A1.28 – Improved bulk port run 4, H_s=1.25 m, waves from 0°N, isolines: 1 – 1.5 m.
A1.29 – Improved bulk port run 5, H_s=2.25 m, waves from 0°N, isolines: 1.5 m.
A1.30 – Default container port entrance run, H_s=1.25 m, waves from 0°N, isolines: 0.5 – 1 m.
A1.31 – Default container port entrance run, H_s=1.25 m, waves from 0°N, isolines: 1.5 m.
A1.32 – Default container port entrance run, H_s=2.25 m, waves from 0°N, isolines: 0.5 – 1 m.
A1.33 – Default container port entrance run, H_s=1.75 m, waves from 0°N, isolines: 1 – 2.5 m.
A1.34 – Default container port entrance run, H_s=1.25 m, waves from 30°N, isolines: 0.5 – 1 m.
A1.35 – Default container port entrance run, H_s=1.25 m, waves from 30°N, isolines: 1 – 2 m.
A1.36 – Default container port entrance run, H_s=1.75 m, waves from 30°N, isolines: 0.5 – 2 m.
A1.37 – Default container port entrance run, H_s=2.75 m, waves from 30°N, isolines: 0.5 – 2 m.
A1.38 – Default container port entrance run, H_s=1.25 m, waves from 330°N, isolines: 0.5 - 1 m.
A1.39 – Default container port entrance run, H_s=1.75 m, waves from 330°N, isolines: 1 – 2 m.

A1.40 – Default container port entrance run, H_s=1.25 m, waves from 300°N, isolines: 0.5 – 1 m.
A1.41 – Default container port entrance run, H_s=2.25 m, waves from 300°N, isolines: 0.5 – 1.5 m.
A1.42 – Improved container port entrance run, H_s=1.25 m, waves from 0°N, isolines: 0.5 – 1 m.
A1.43 – Improved container port entrance run, H_s=2.25 m, waves from 0°N, isolines: 0.5 – 2 m.
A1.44 – Improved container port entrance run, H_s=1.25 m, waves from 330°N, isolines: 0.5 – 1 m.
A1.45 – Improved container port entrance run, H_s=1.75 m, waves from 330°N, isolines: 0.5 – 2 m.
A1.46 – Improved container port entrance run, H_s=2.25 m, waves from 330°N, isolines: 0.5 – 1.5 m.
A1.47 – Improved container port entrance run, H_s=1.75 m, waves from 30°N, isolines: 0.5 – 1.5 m.
A1.48 – Improved container port entrance run, H_s=2.75 m, waves from 30°N, isolines: 0.5 – 2 m.
A1.49 – Improved container port berths run, H_s=1.75 m, waves from 30°N, isolines: 0.5 – 1.5 m.
A1.50 – Improved container port berths run, H_s=1.25 m, waves from 30°N, isolines: 0.5 – 1 m.
A1.51 – Improved container port berths run, H_s=0.75 m, waves from 30°N, isolines: 0.5 – 1 m.
A1.52 – Water levels in Nador

Figure A1.1

Satellite image with indicated project location (A) [GOOGLE MAPS]

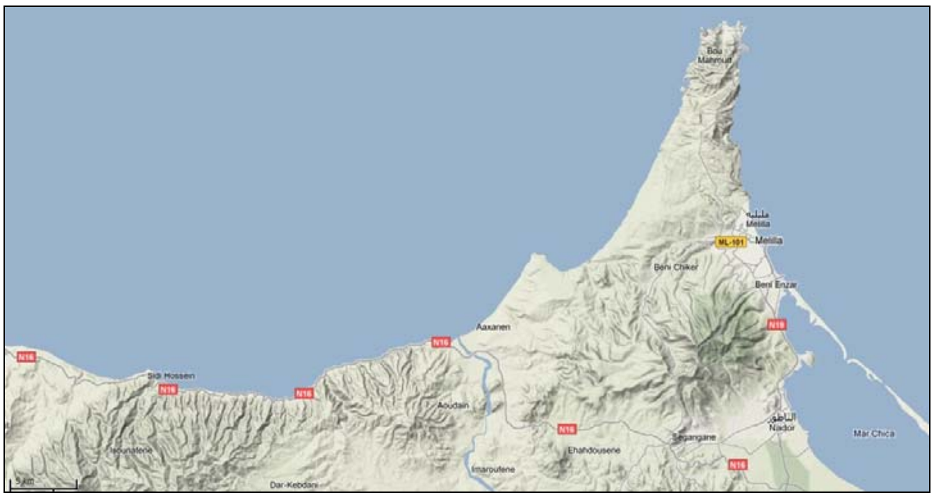


Figure A1.2

Relief of the region around Nador [GOOGLE MAPS]

Figure A1.3

Indication of the present (larger) infrastructure [GOOGLE MAPS]

Figure A1.4

Bathymetric map of project location from client

1 square is 500x500m^2

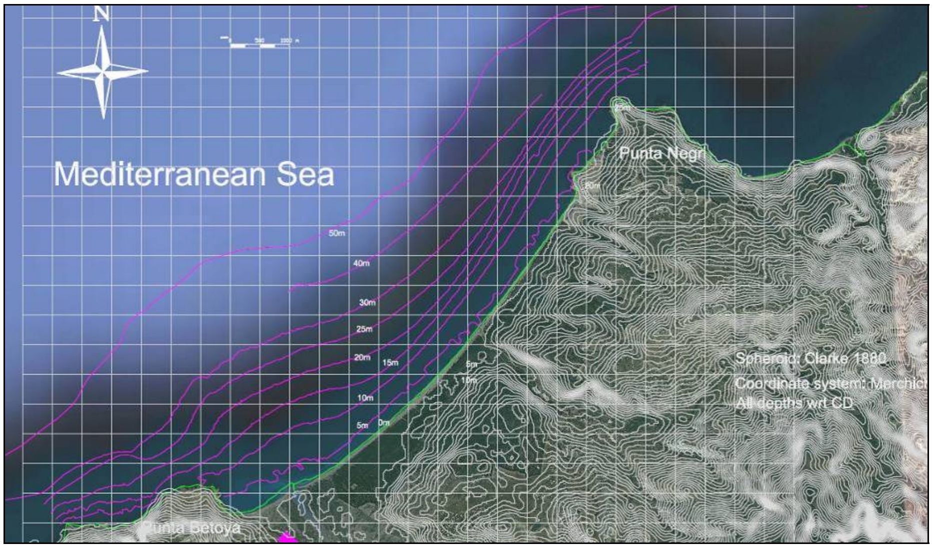


Figure A1.5

Bathymetric map Nautical Charts [m]

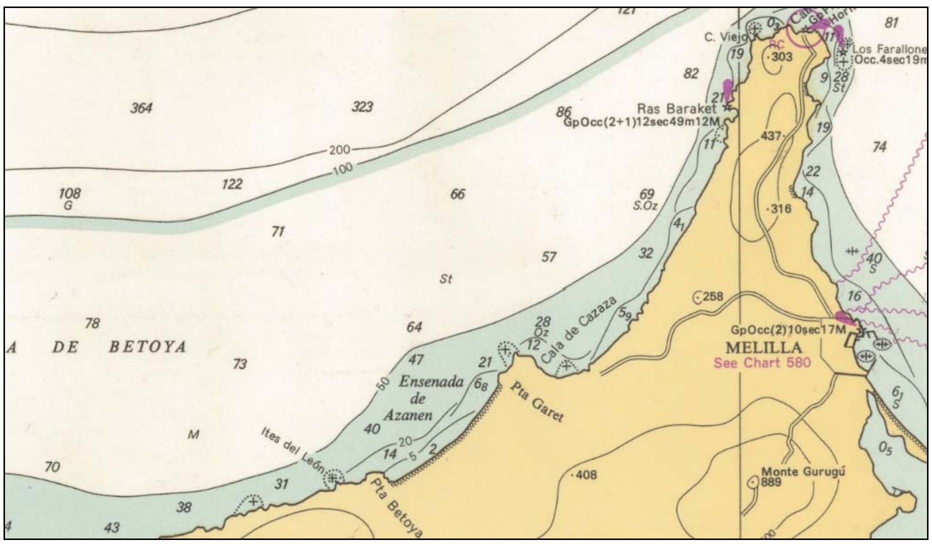


Figure A1.6

Diffraction diagram combined with bulk port layout.

Values of K_d for

S_{max}=10; B/L=2.0

Waves from 300°N

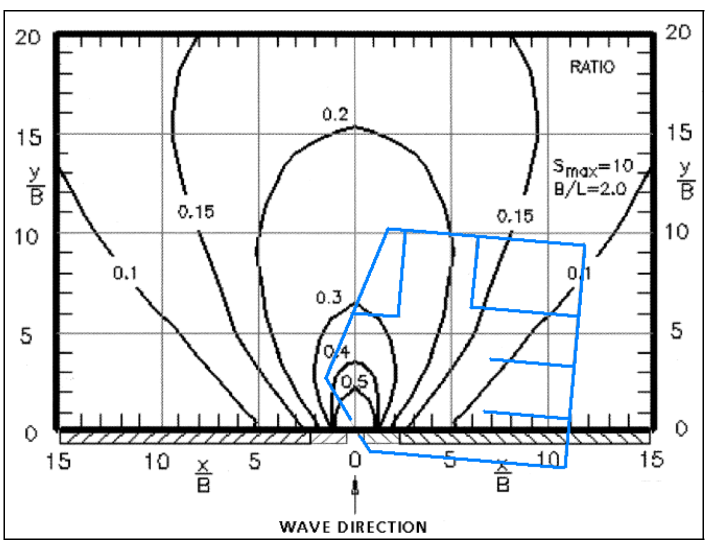


Figure A1.7

Diffraction diagram combined with bulk port layout.
Values of K_d for
S_{max}=10; B/L=4.0
Waves from 330°N

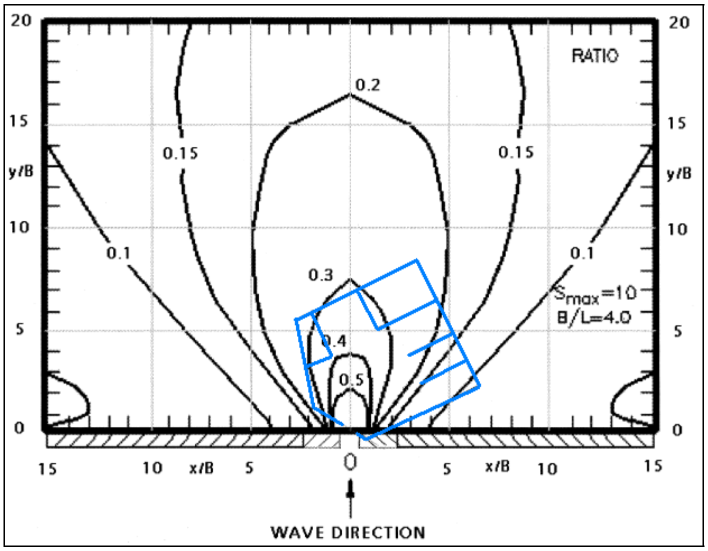


Figure A1.8

Diffraction diagram combined with bulk port layout.
Values of K_d for
S_{max}=10; B/L=4.0
Waves from 0°N

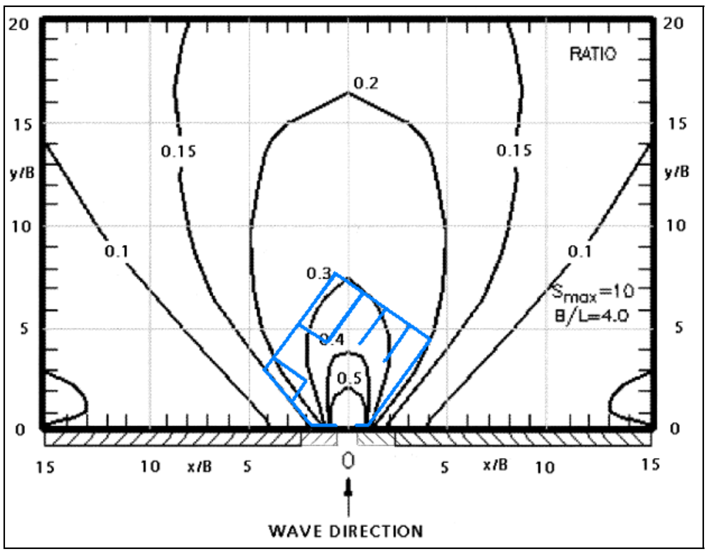


Figure A1.9

Diffraction diagram combined with bulk port layout.
Values of K_d for
S_{max}=10; B/L=2.0
Waves from 30°N

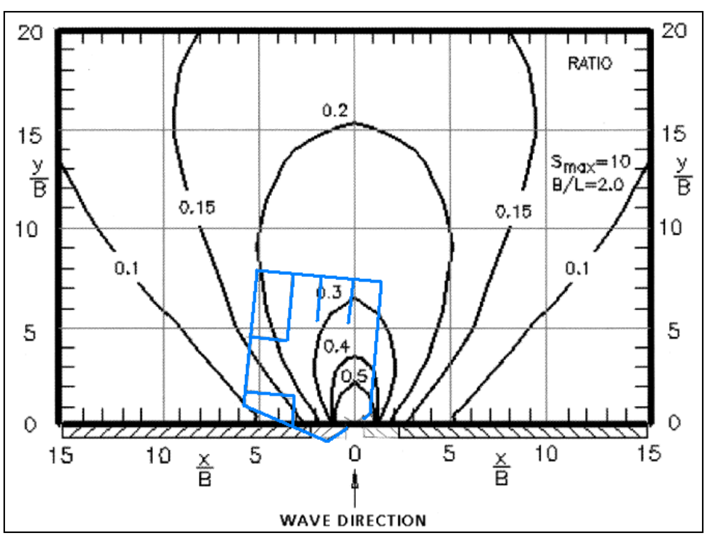


Figure A1.10

Diffraction diagram combined with bulk port layout.
Values of K_d for
S_{max}=10; B/L=4.0
Waves from 60°N

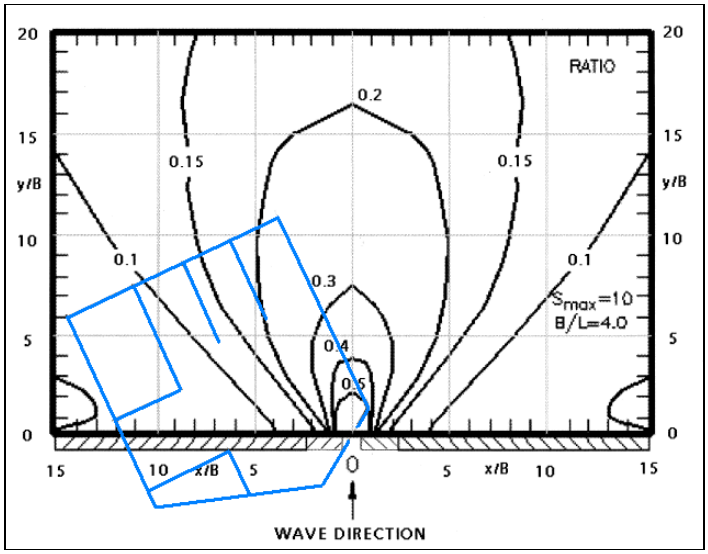


Figure A1.11

DIFFRAC-2DH
Predif basic screen, with indicated the various input steps before running Diffrac

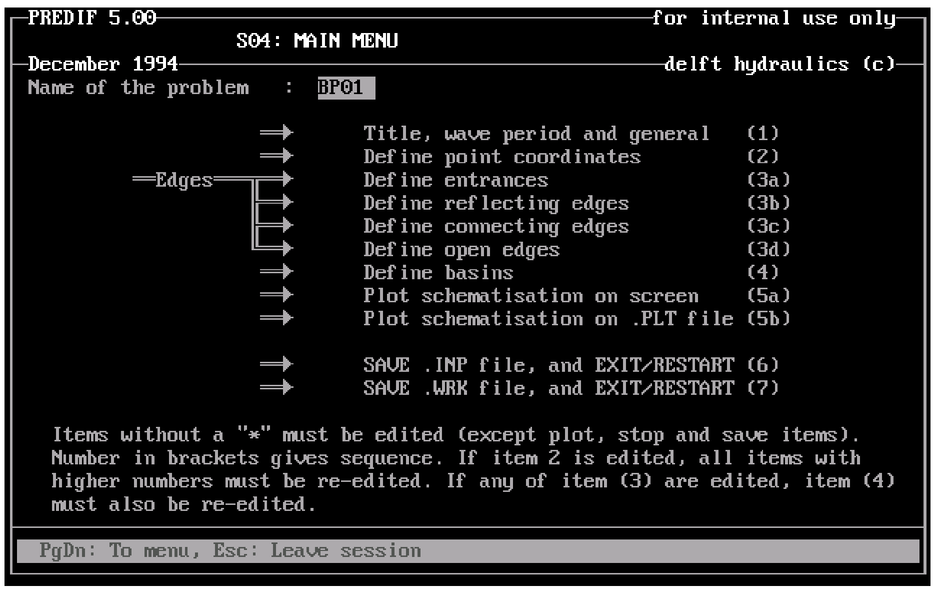


Figure A1.12

Bulk port run

Wave direction: 0°N

Wave height: H_s=1.75 m

Contour lines: 1, 1.5 m

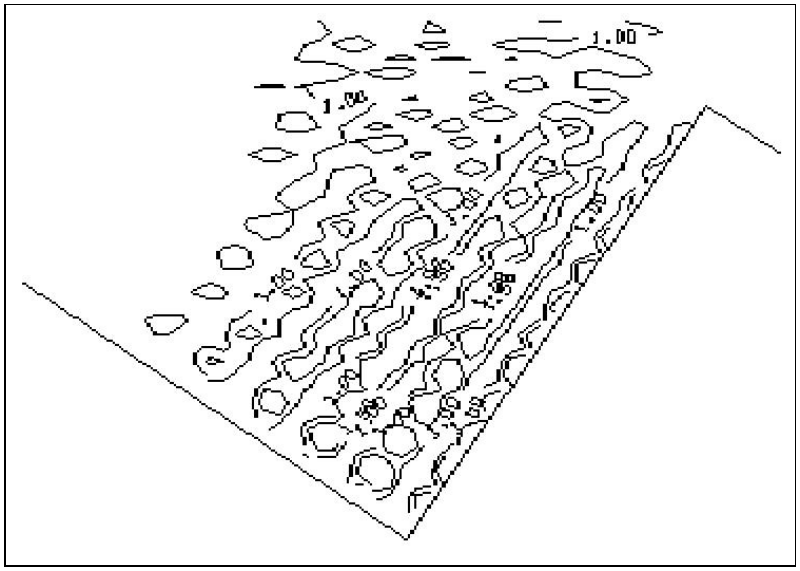


Figure A1.13

Bulk port run

Wave direction: 0°N

Wave height: H_s=1.75 m

Contour lines: 2 m

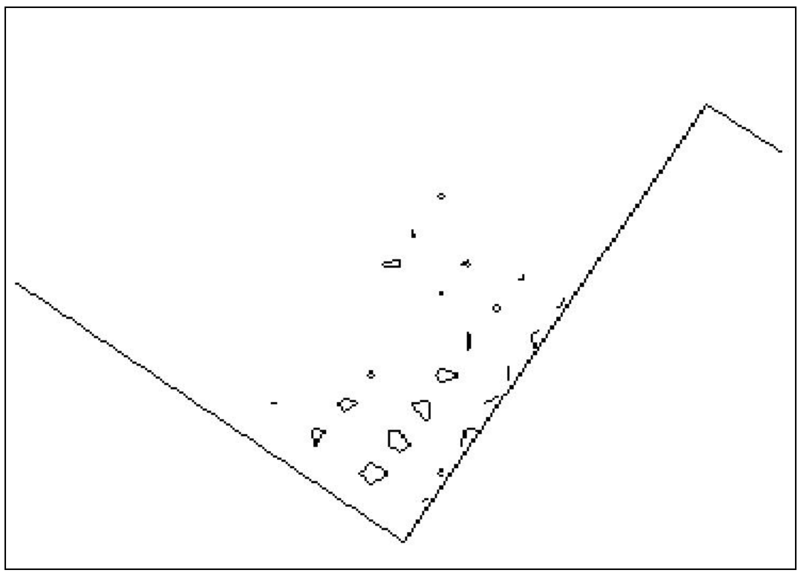


Figure A1.14

Bulk port run

Wave direction: 0°N

Wave height: H_s=1.25 m

Contour lines: 1.5 m

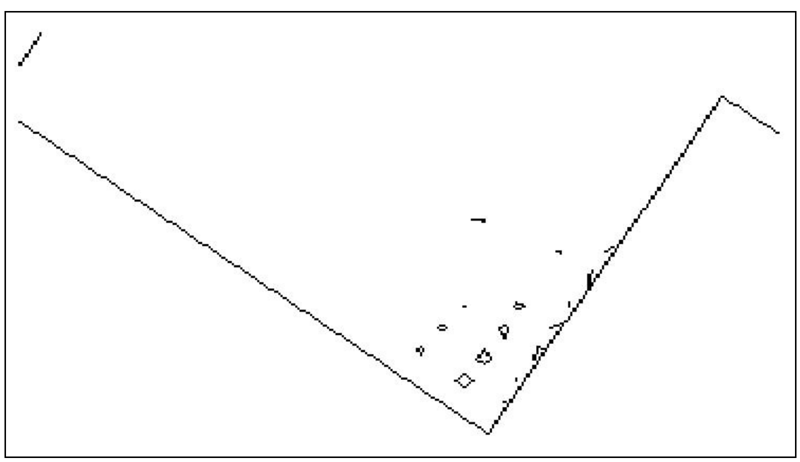


Figure A1.15

Bulk port run

Wave direction: 0°N

Wave height: H_s=1.75 m

Contour lines: 1.5 m

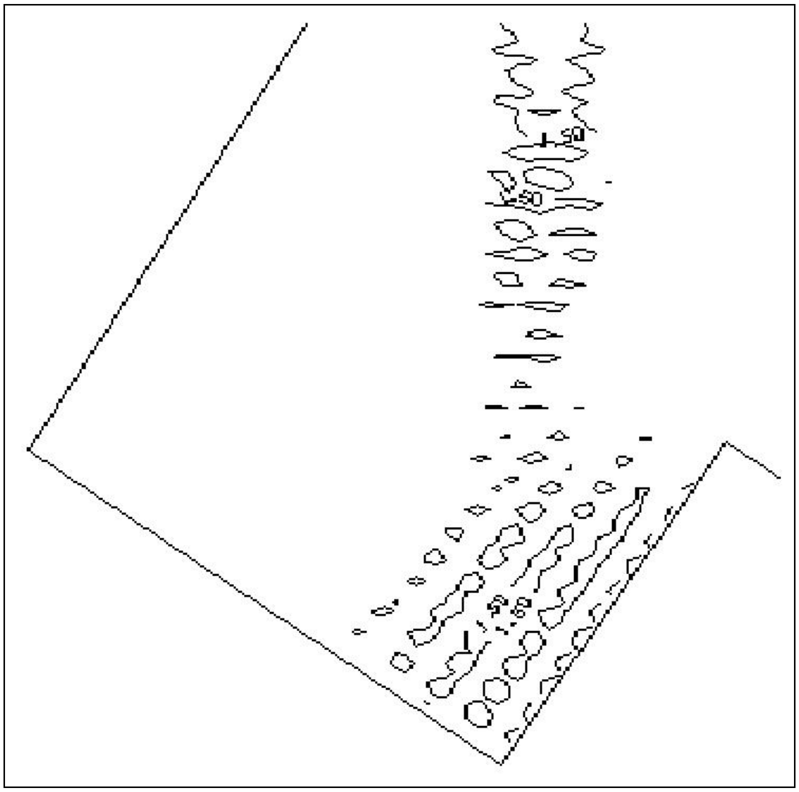


Figure A1.16

Bulk port run

Wave direction: 0°N

Wave height: H_s=2.75 m

Contour lines: 1.5, 2 m

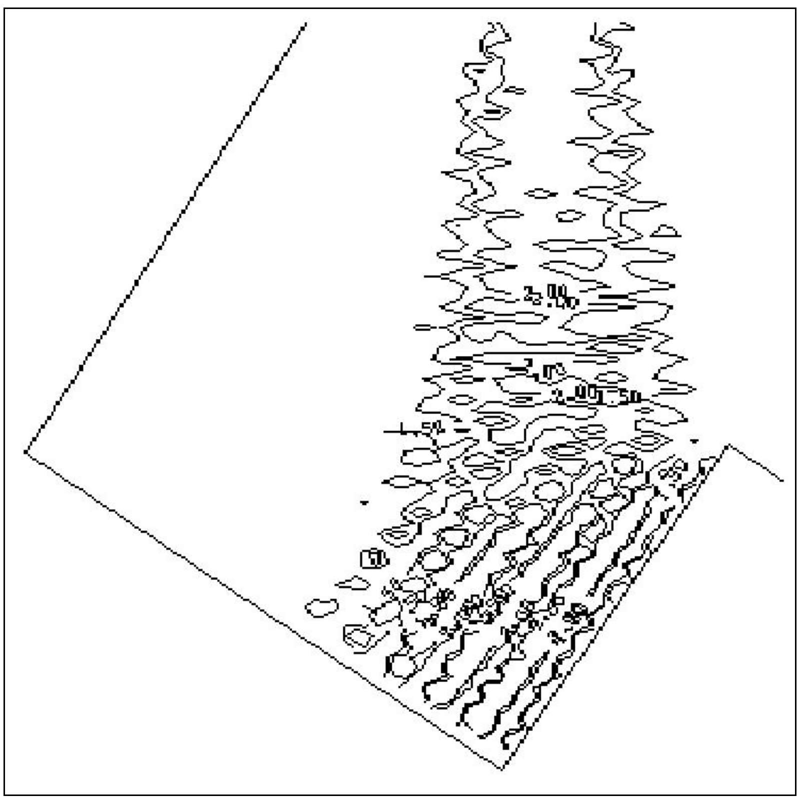


Figure A1.17

Bulk port run

Wave direction: 330°N

Wave height: H_s=1.75 m

Contour lines: 1, 1.5 m

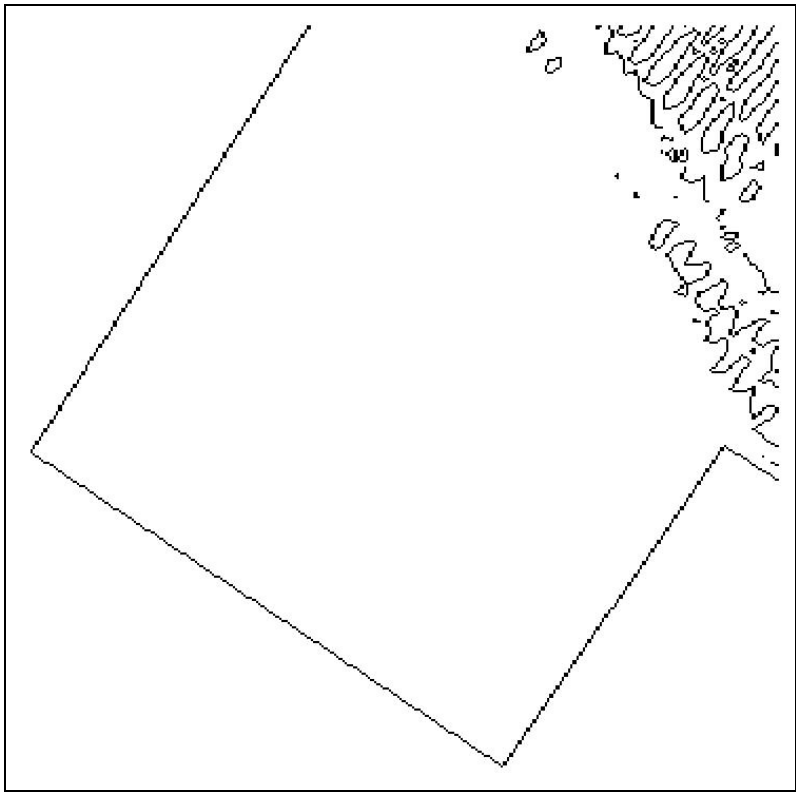


Figure A1.18

Bulk port run

Wave direction: 330°N

Wave height: H_s=2.75 m

Contour lines: 1, 1.5 m

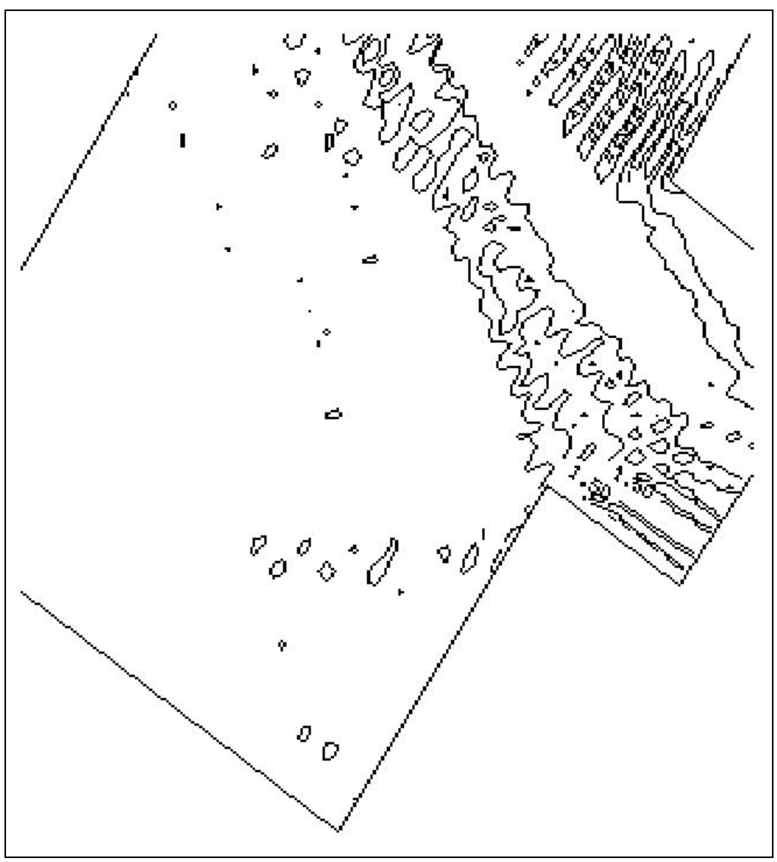


Figure A1.19

Bulk port run

Wave direction: 330°N

Wave height: H_s=3.25 m

Contour lines: 1.5, 2 m

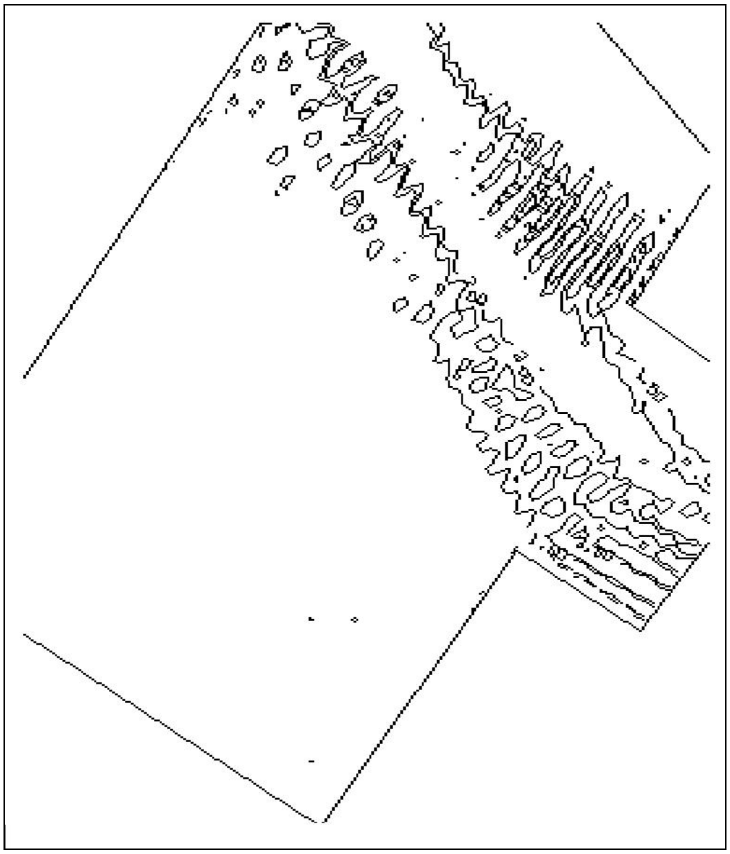


Figure A1.20

Bulk port run

Wave direction: 300°N

Wave height: H_s=2.75 m

Contour lines: 1.5, 2 m

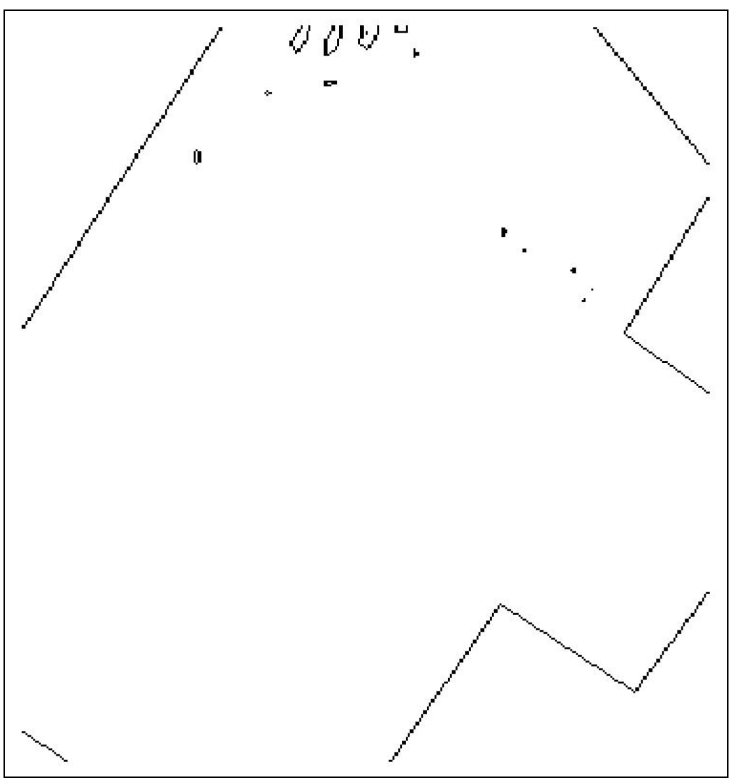


Figure A1.21

Bulk port run

Wave direction: 30°N

Wave height: H_s=1.75 m

Contour lines: 1, 1.5 m

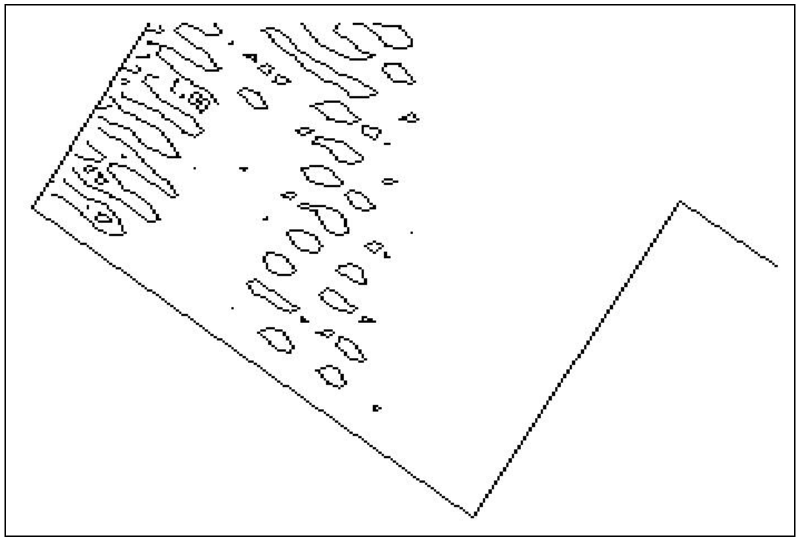


Figure A1.22

Bulk port run

Wave direction: 30°N

Wave height: H_s=2.25 m

Contour lines: 1.5 m

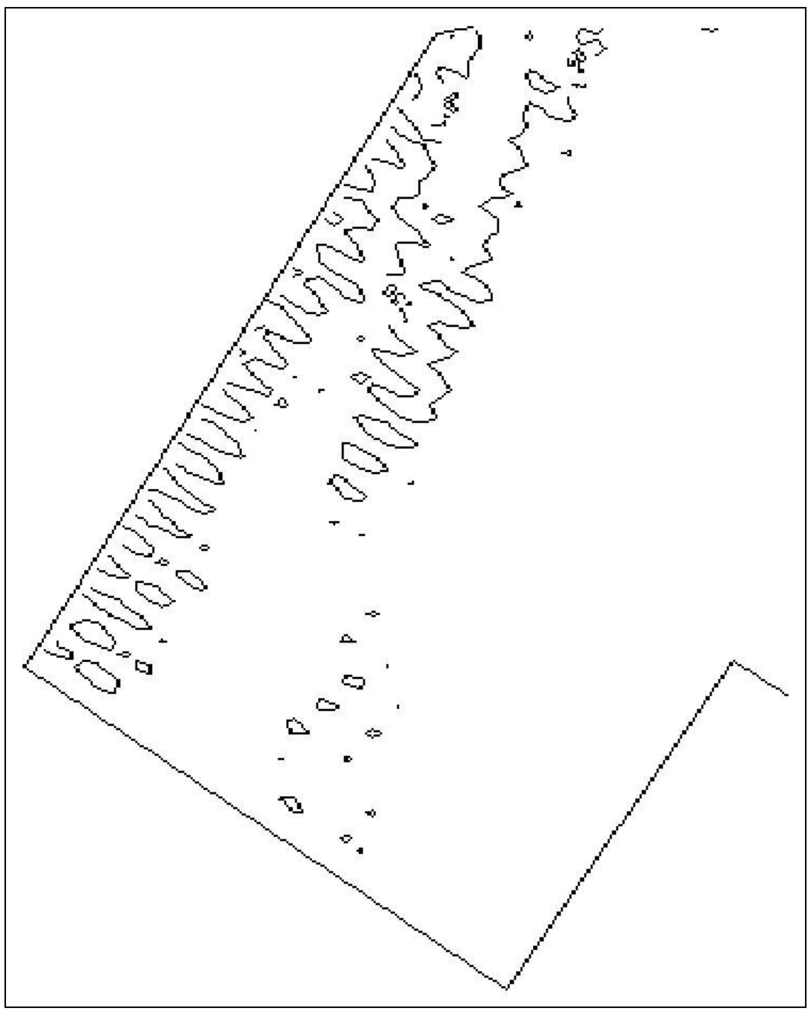


Figure A1.23

Bulk port run

Wave direction: 30°N

Wave height: H_s=2.75 m

Contour lines: 1.5, 2 m

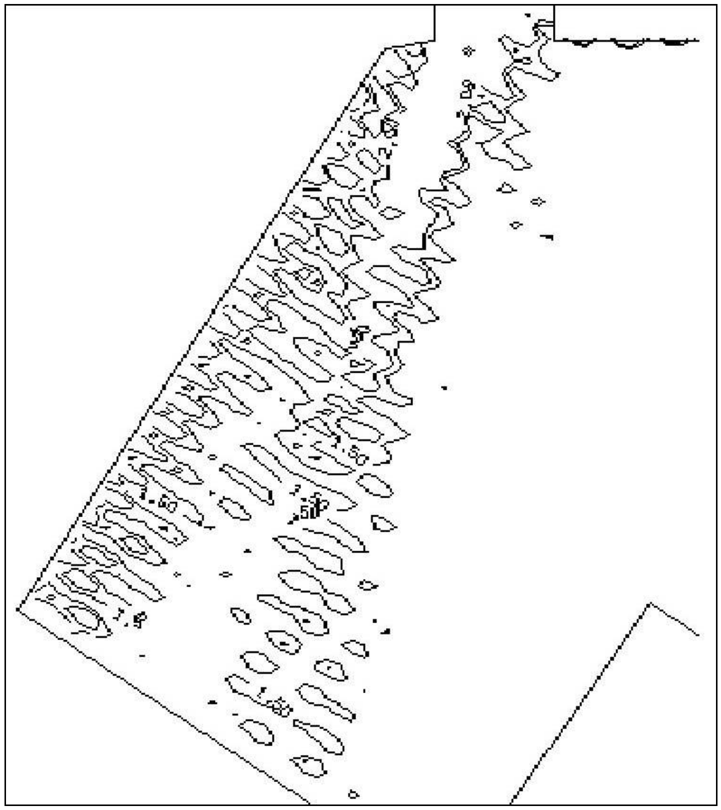


Figure A1.24

Bulk port run

Wave direction: 30°N

Wave height: H_s=1.75 m

Contour lines: 1.5 m

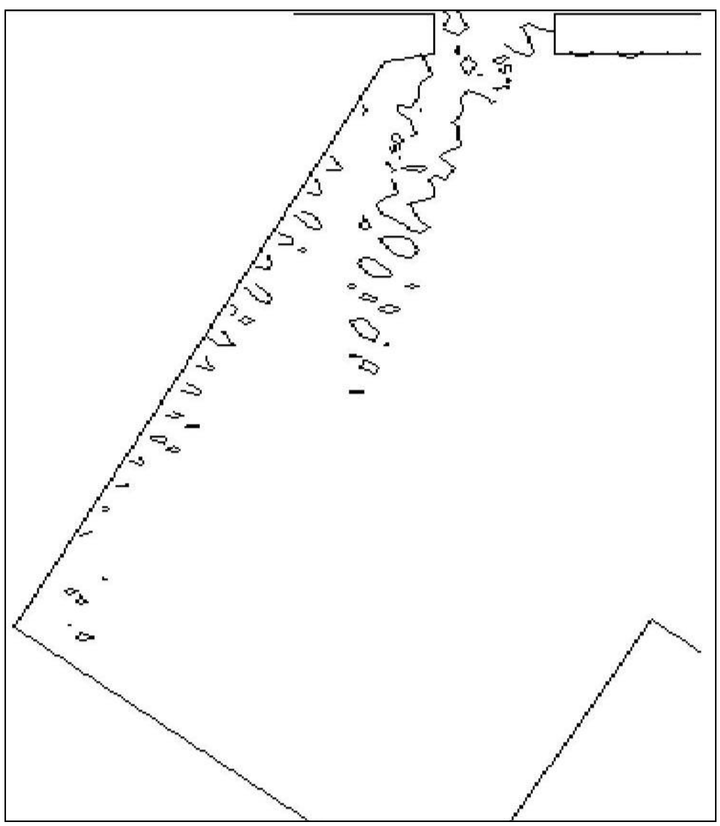


Figure A1.25

Improved bulk port run

Wave direction: 30°N

Wave height: H_s=2.25 m

Contour lines: 1.5 m

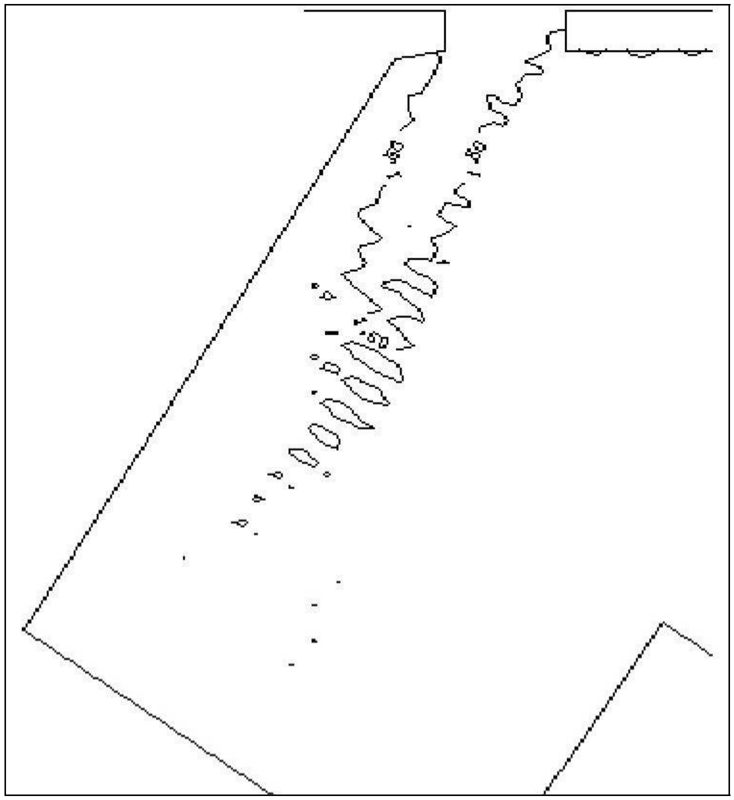


Figure A1.26

Improved bulk port run

Wave direction: 30°N

Wave height: H_s=2.75 m

Contour lines: 1.5, 2 m

Figure A1.27

Improved bulk port run

Wave direction: 0°N

Wave height: H_s=1.75 m

Contour lines: 1.5 m

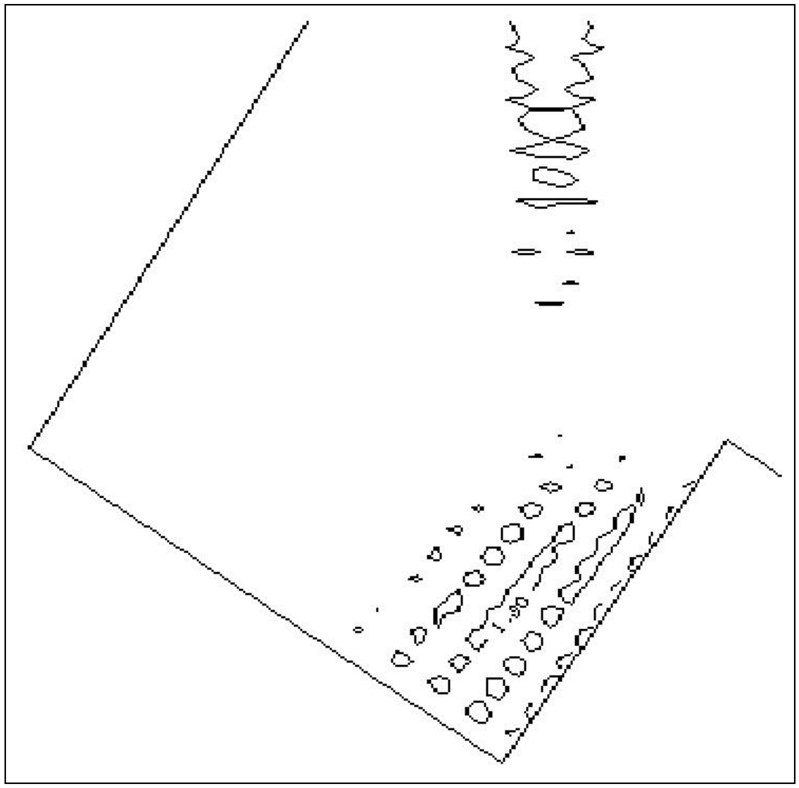


Figure A1.28

Improved bulk port run

Wave direction: 0°N

Wave height: H_s=1.25 m

Contour lines: 1, 1.5 m

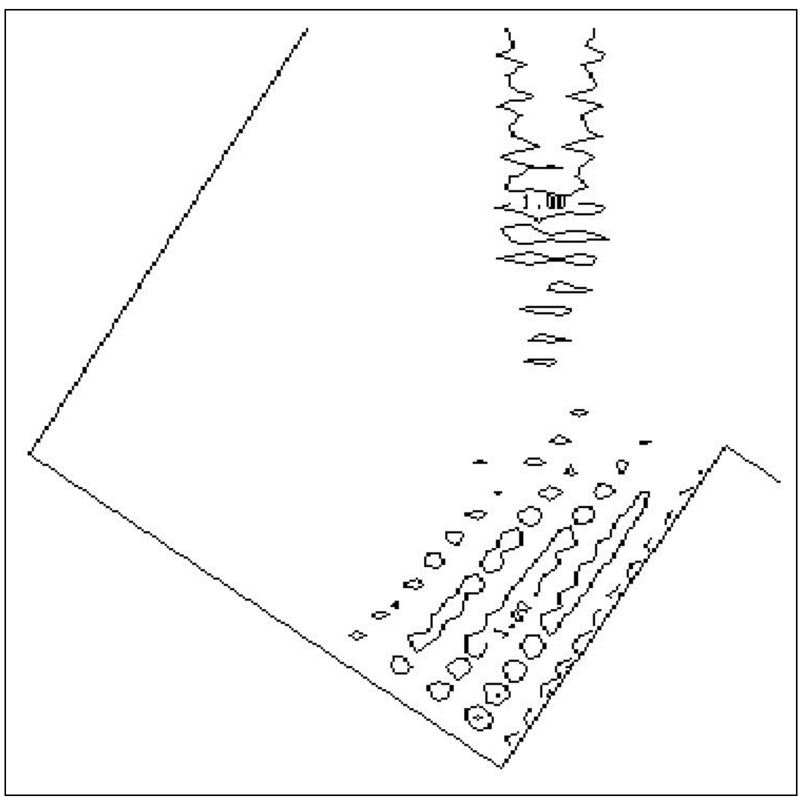


Figure A1.29

Improved bulk port run

Wave direction: 0°N

Wave height: H_s=2.25 m

Contour lines: 1.5 m

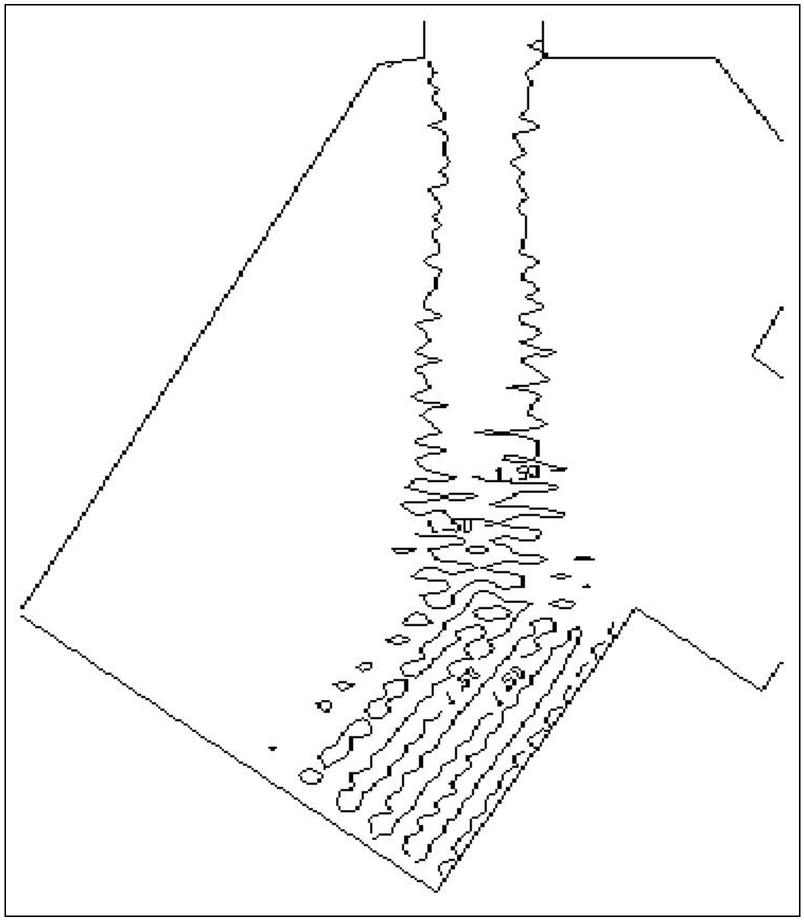


Figure A1.30

Container port entrance run

Wave direction: 0°N

Wave height: H_s=1.25 m

Contour lines: 0.5, 1 m

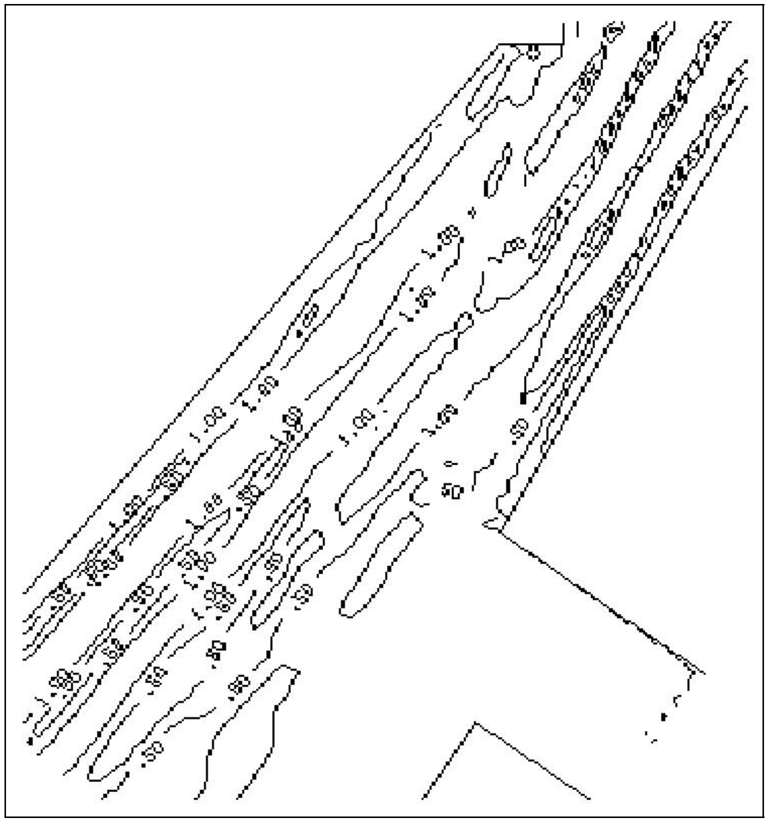


Figure A1.31

Container port entrance run

Wave direction: 0°N

Wave height: H_s=1.25 m

Contour lines: 1.5 m

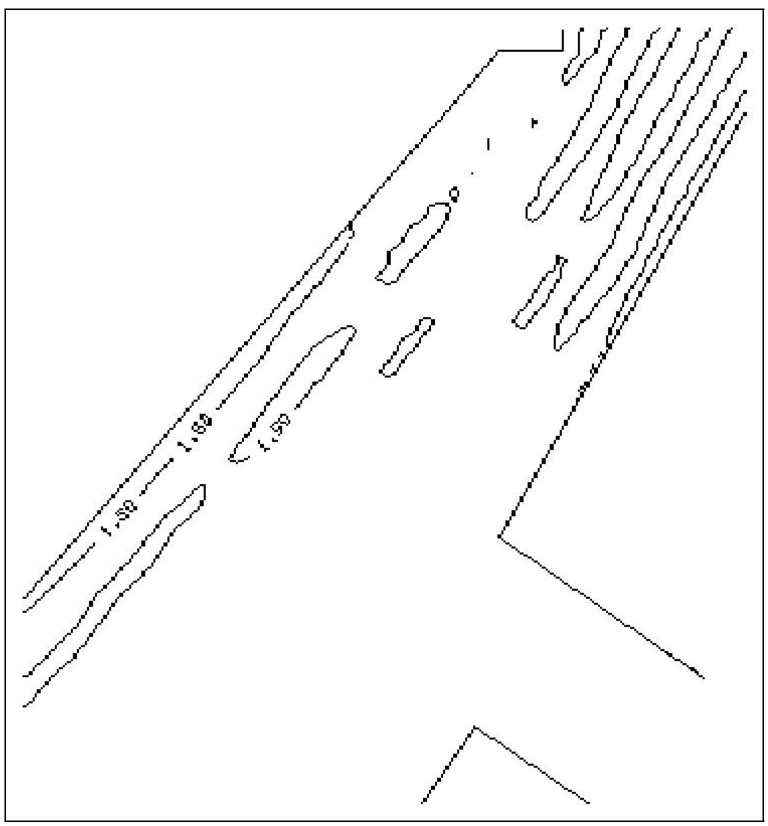


Figure A1.32

Container port entrance run

Wave direction: 0°N

Wave height: H_s=2.25 m

Contour lines: 0.5, 1 m

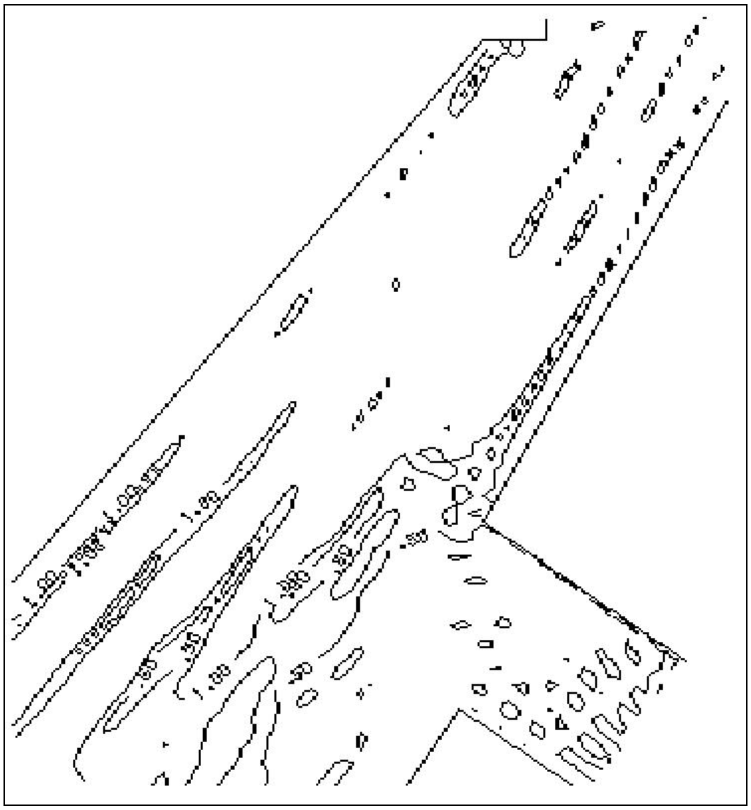


Figure A1.33

Container port entrance run

Wave direction: 0°N

Wave height: H_s=1.75 m

Contour lines: 1 - 2.5 m

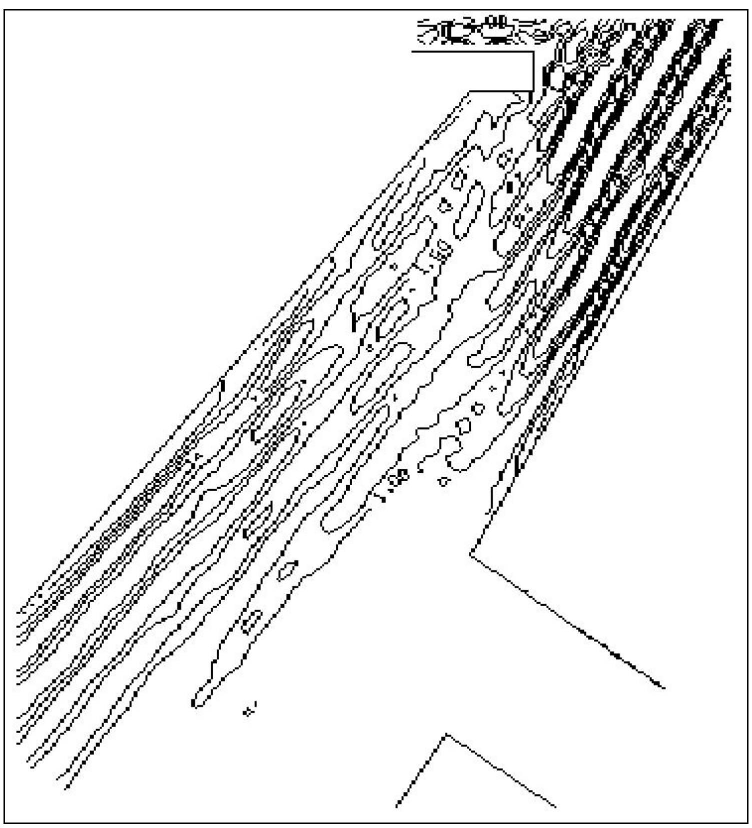


Figure A1.34

Container port entrance run
Wave direction: 30°N
Wave height: H_s=1.25 m
Contour lines: 0.5 - 1.5 m

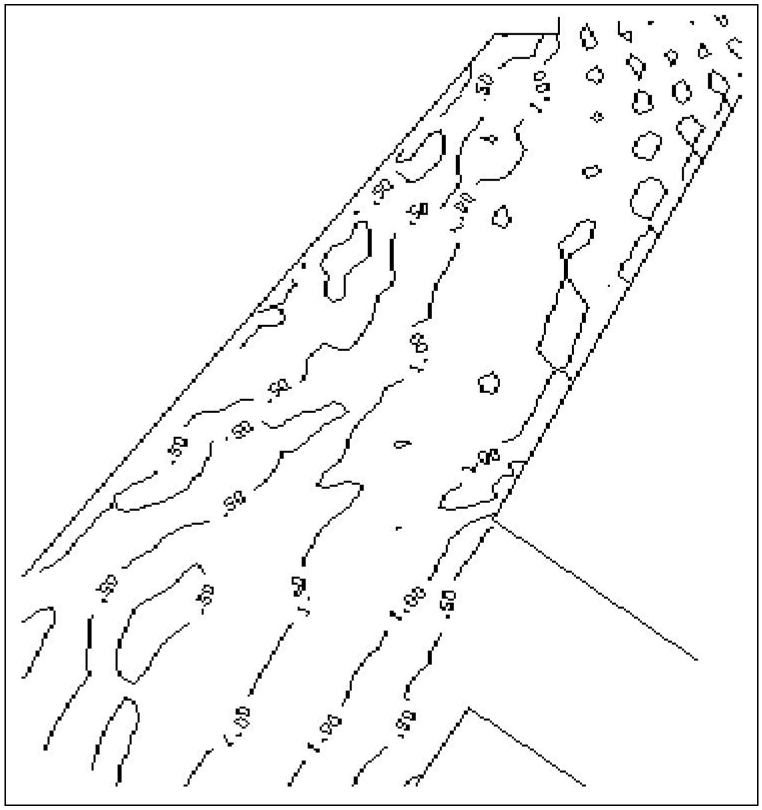


Figure A1.35

Container port entrance run
Wave direction: 30°N
Wave height: H_s=1.25 m
Contour lines: 1 - 2 m

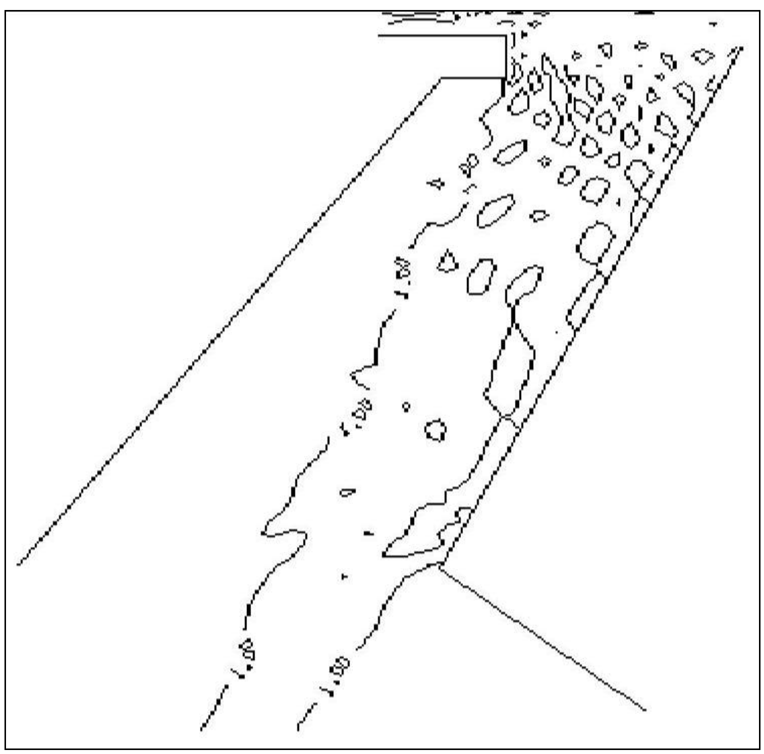


Figure A1.36

Container port entrance run

Wave direction: 30°N

Wave height: H_s=1.75 m

Contour lines: 0.5 - 2 m

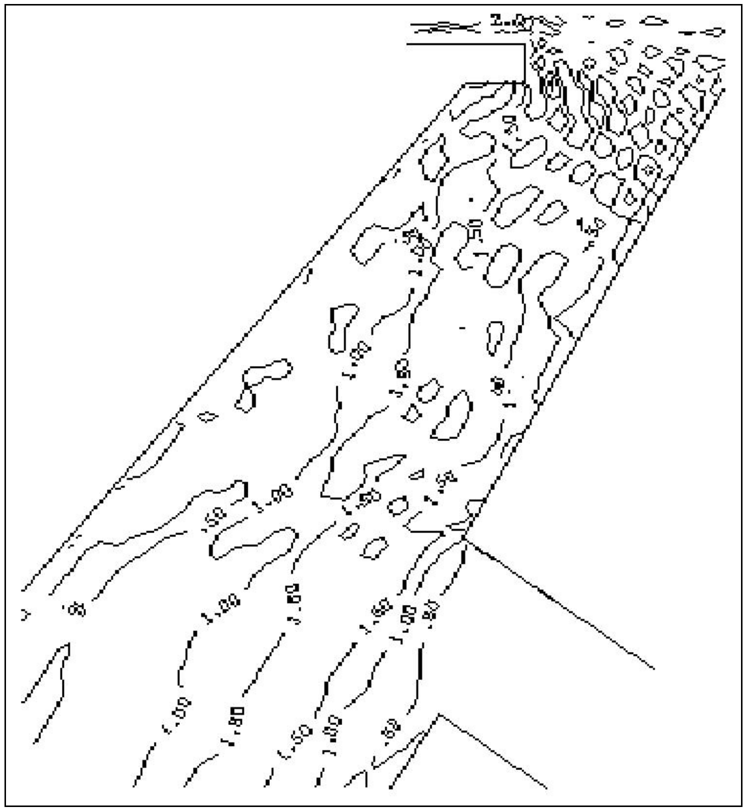


Figure A1.37

Container port entrance run

Wave direction: 30°N

Wave height: H_s=2.75 m

Contour lines: 0.5 - 2 m

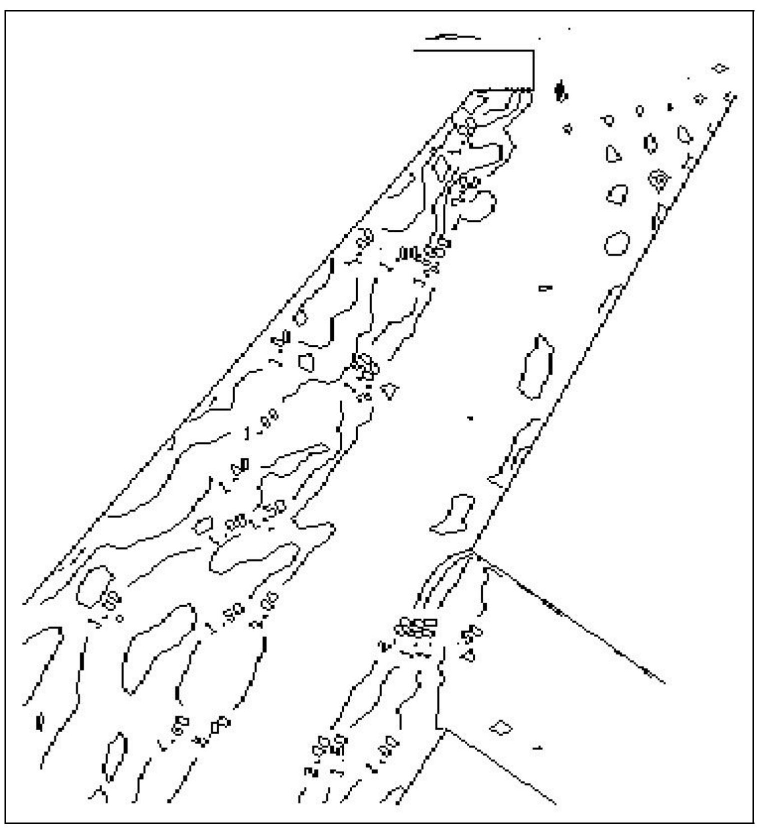


Figure A1.38

Container port entrance run

Wave direction: 330°N

Wave height: H_ς=1.25 m

Contour lines: 0.5, 1 m

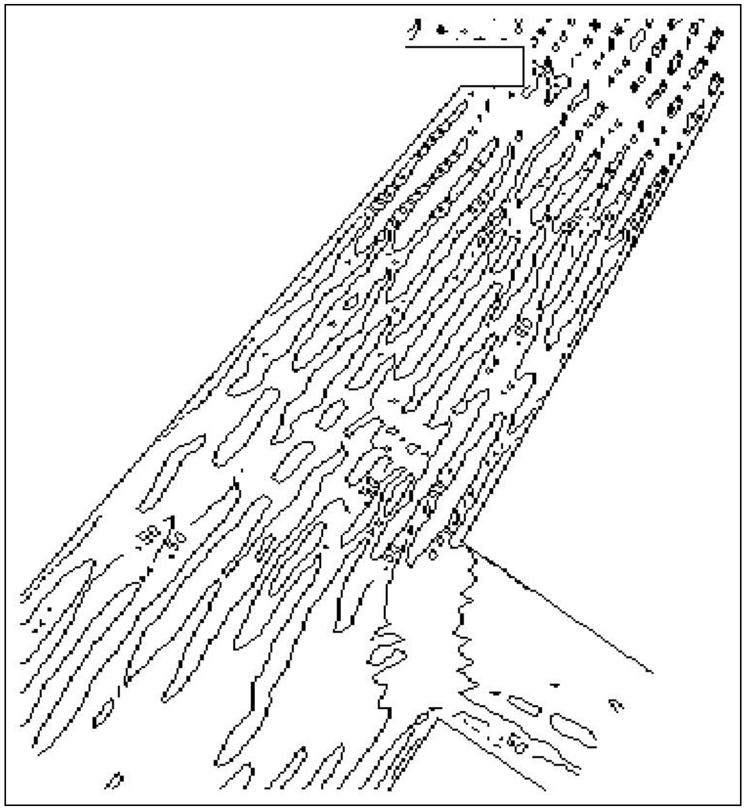


Figure A1.39

Container port entrance run

Wave direction: 330°N

Wave height: H_ς=1.75 m

Contour lines: 1 - 2 m

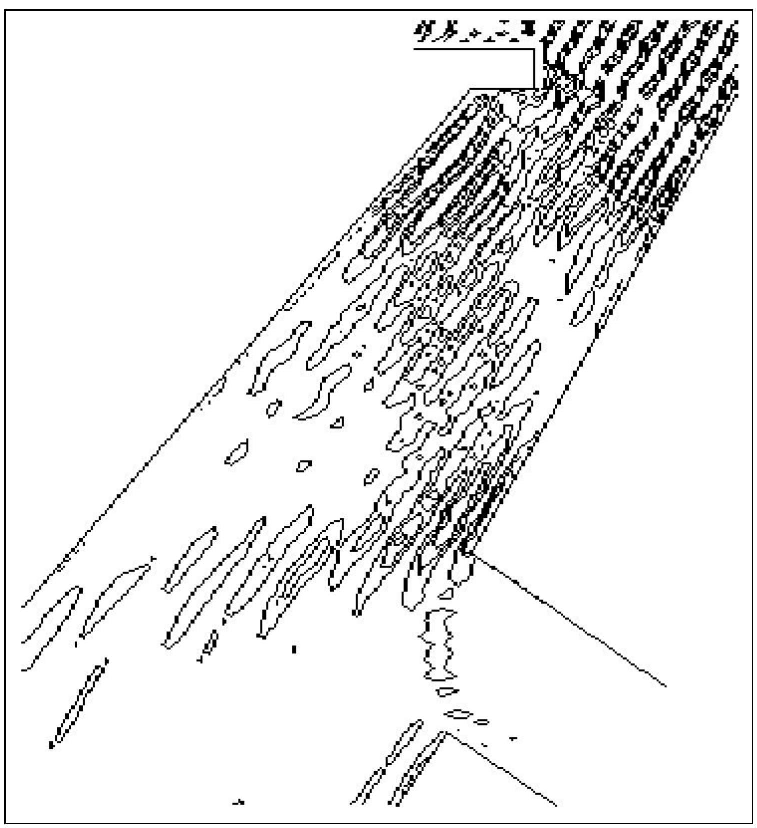


Figure A1.40

Container port entrance run

Wave direction: 300°N

Wave height: H_s=1.25 m

Contour lines: 0.5 - 1 m

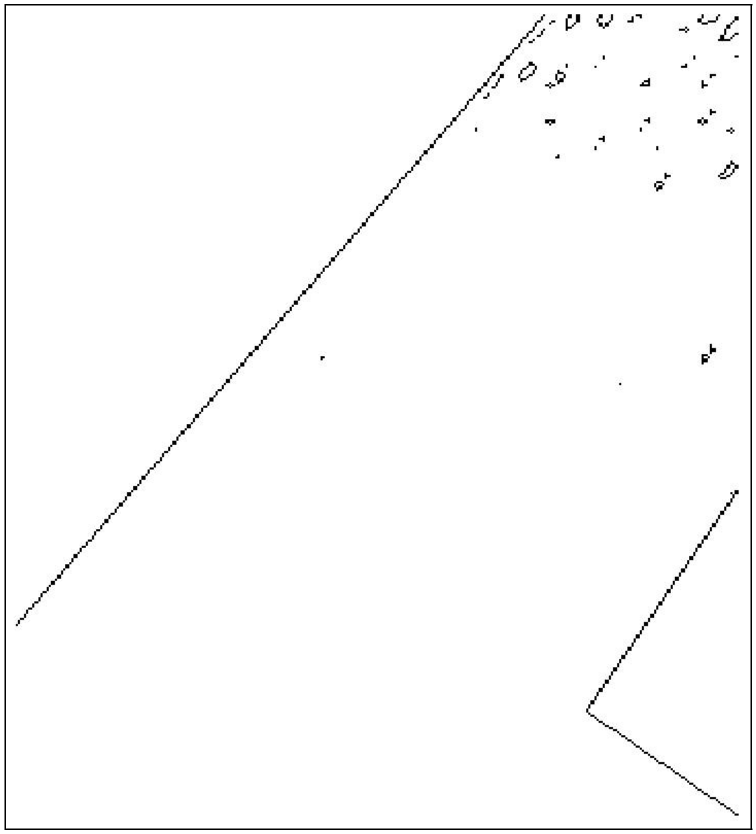


Figure A1.41

Container port entrance run

Wave direction: 300°N

Wave height: H_s=2.25 m

Contour lines: 0.5 – 1.5 m

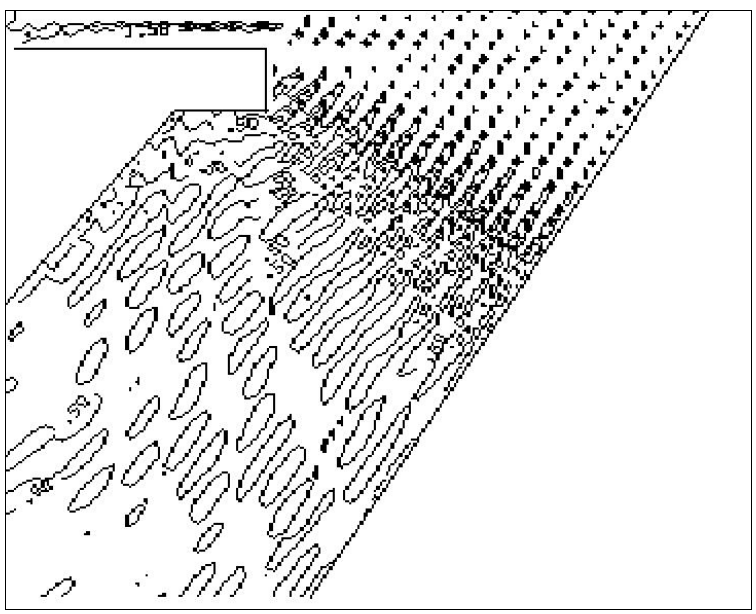


Figure A1.42

Improved container port entrance run
Wave direction: 0°N
Wave height: H_s=1.25 m
Contour lines: 0.5 - 1 m

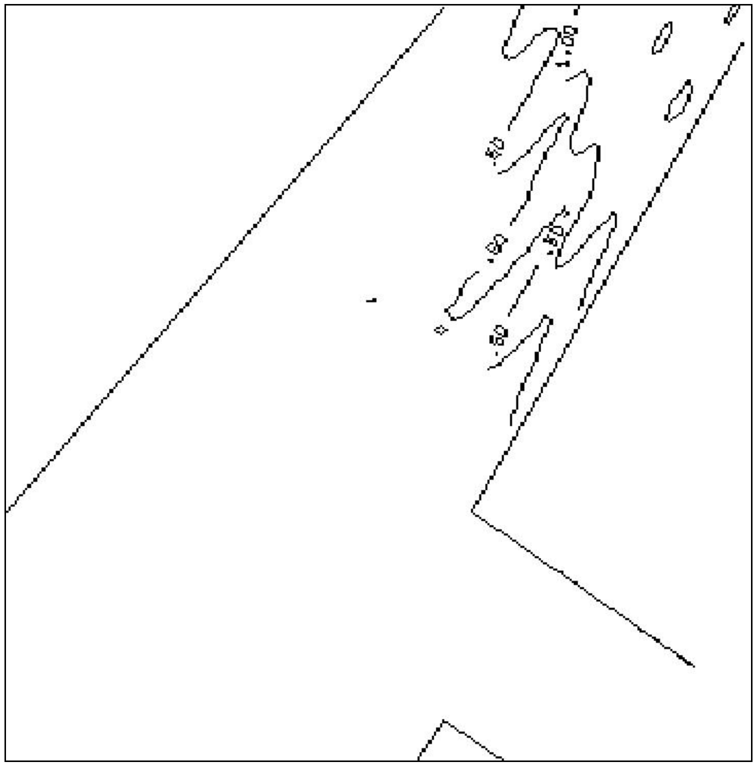


Figure A1.43

Improved container port entrance run
Wave direction: 0°N
Wave height: H_s=2.25 m
Contour lines: 0.5 - 2 m

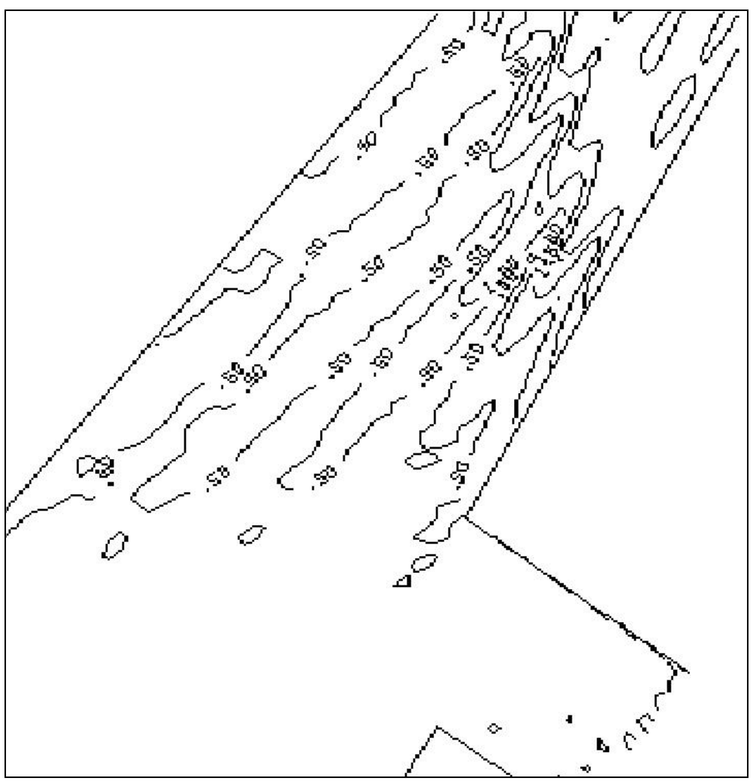


Figure A1.44

Improved container port entrance run
Wave direction: 330°N
Wave height: H_s=1.25 m
Contour lines: 0.5 - 1 m

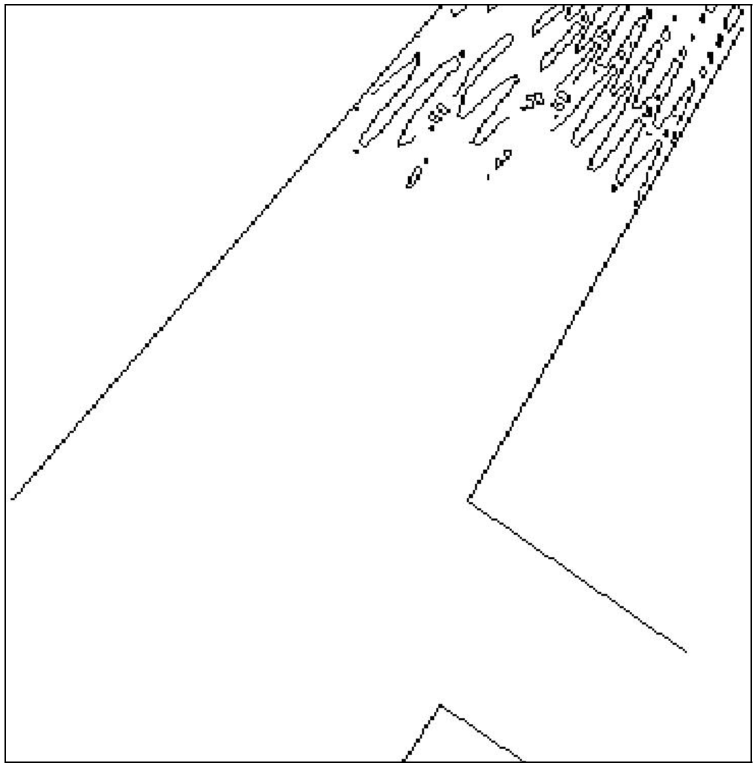


Figure A1.45

Improved container port entrance run
Wave direction: 330°N
Wave height: H_s=1.75 m
Contour lines: 0.5 - 2 m

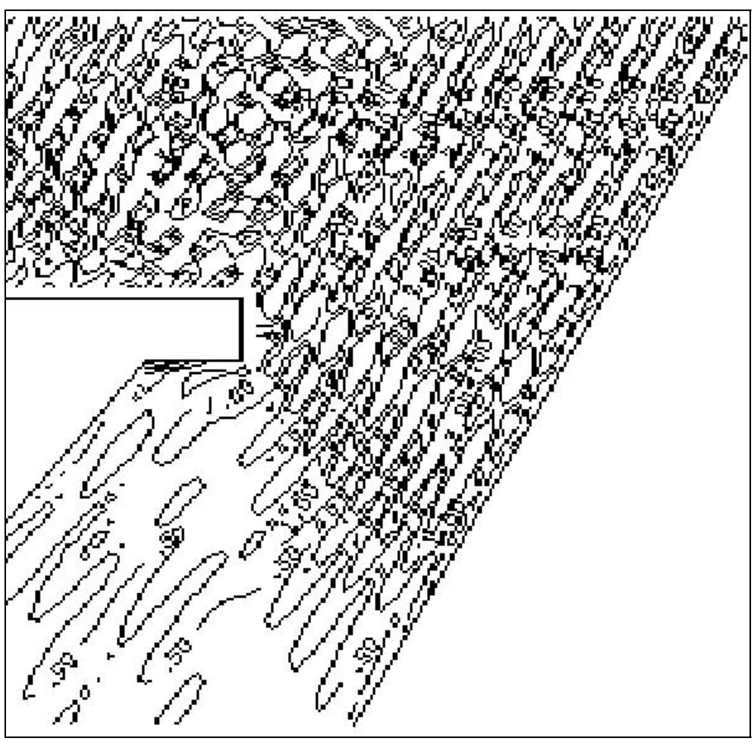


Figure A1.46

Improved container port entrance run
Wave direction: 330°N
Wave height: H_s=2.25 m
Contour lines: 0.5 – 1.5 m

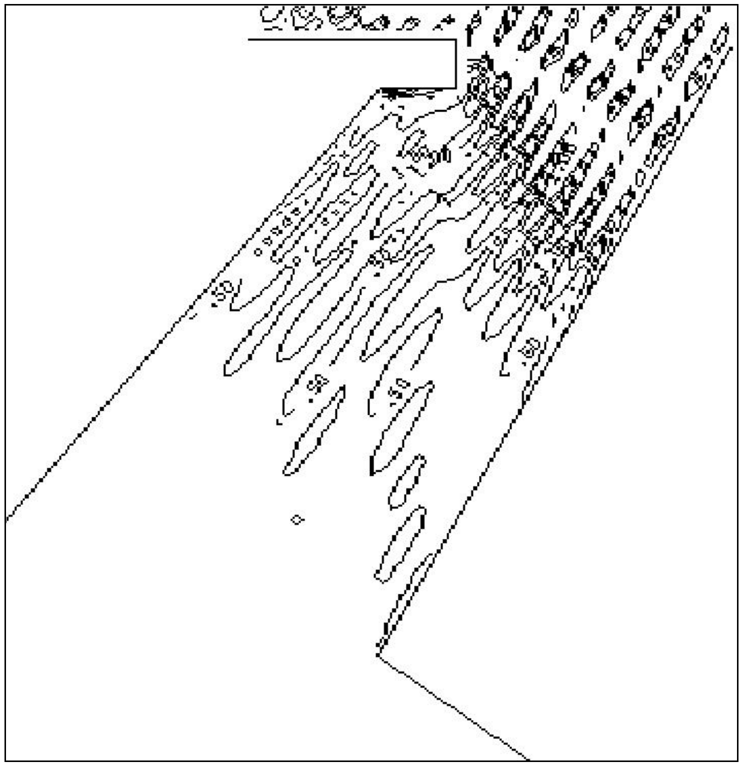


Figure A1.47

Improved container port entrance run
Wave direction: 30°N
Wave height: H_s=1.75 m
Contour lines: 0.5 – 1.5 m

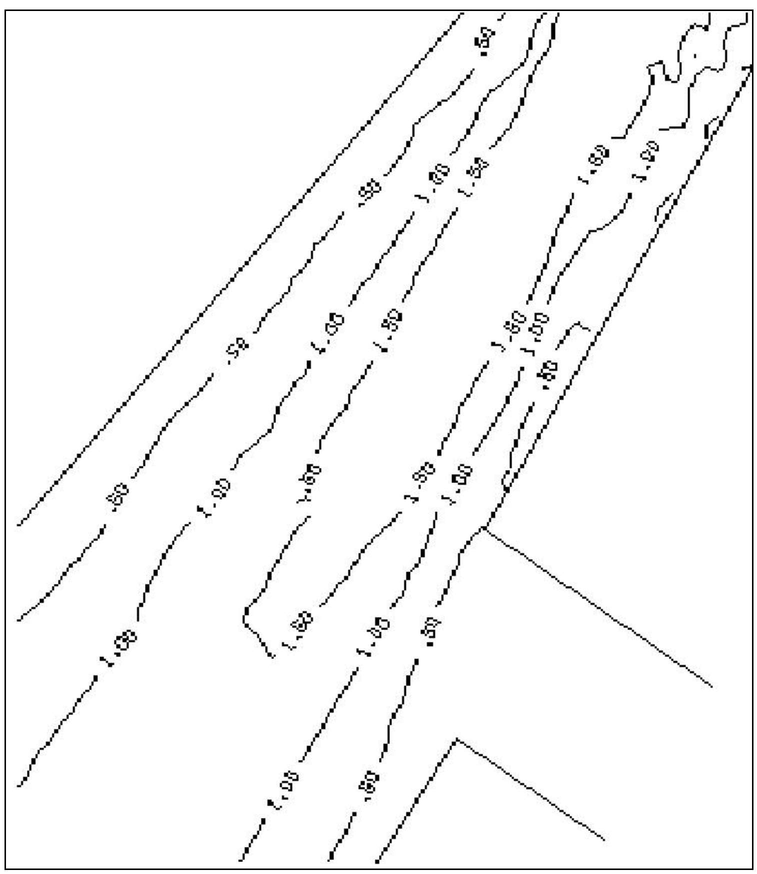


Figure A1.48

Improved container port entrance run

Wave direction: 30°N

Wave height: H_s=2.75 m

Contour lines: 0.5 - 2 m

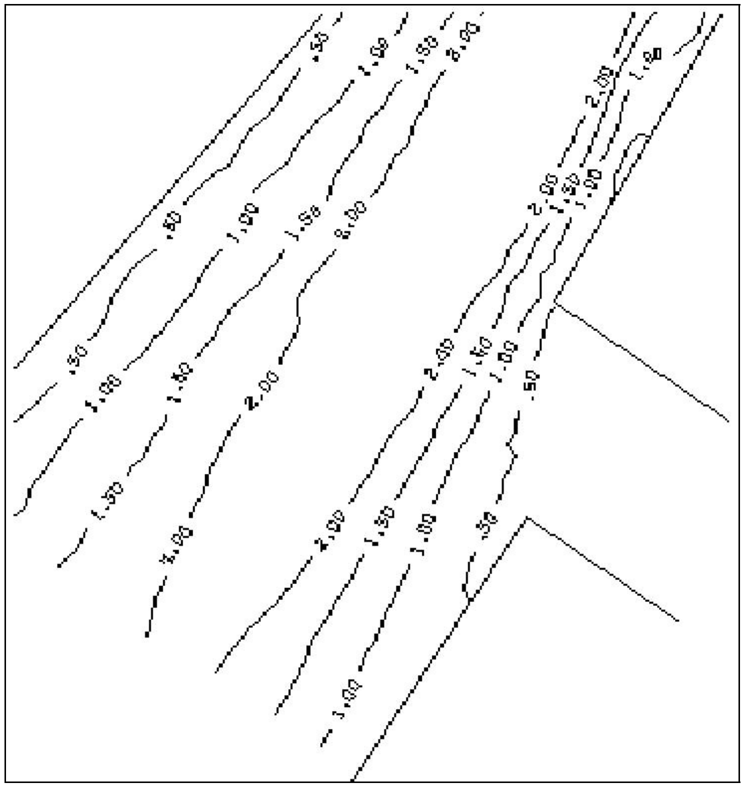


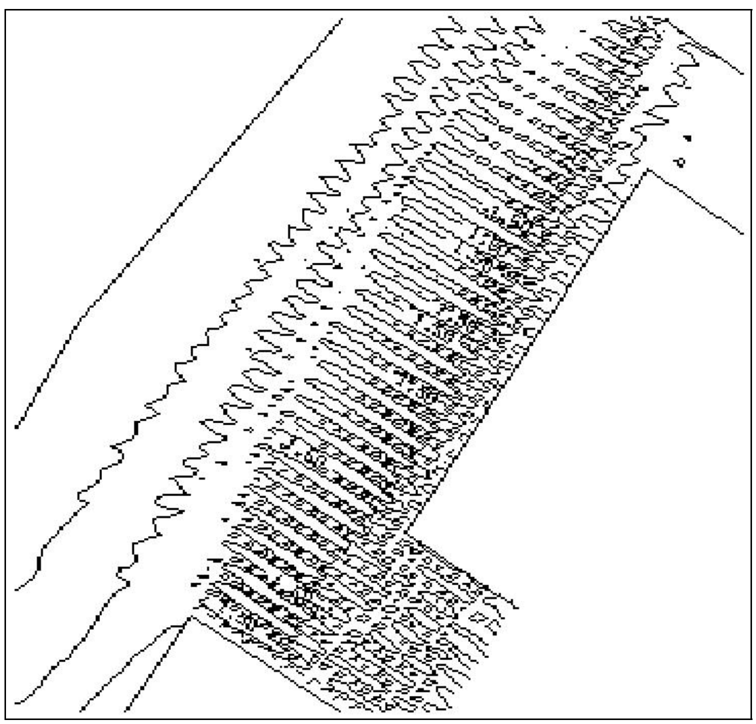
Figure A1.49

Improved container port entrance: berths run

Wave direction: 30°N

Wave height: H_s=1.75 m

Contour lines: 0.5 – 1.5 m



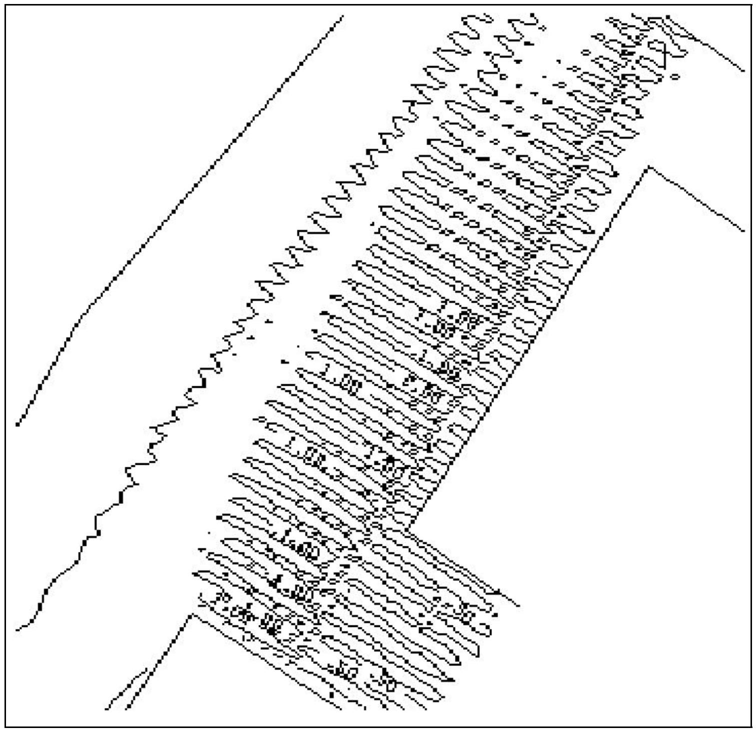


Figure A1.50

Improved container port entrance: berths run
Wave direction: 30°N
Wave height: H_s=1.25 m
Contour lines: 0.5 – 1 m

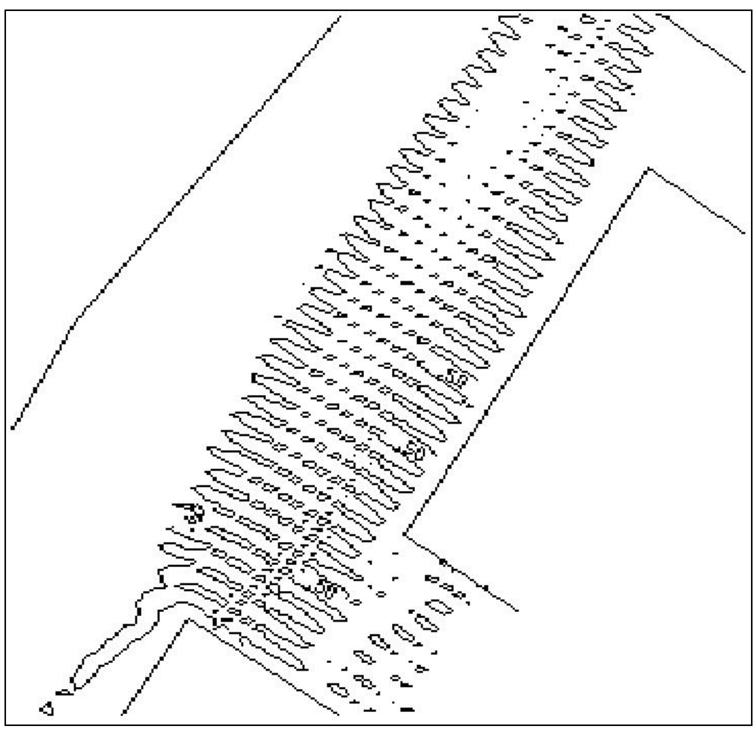
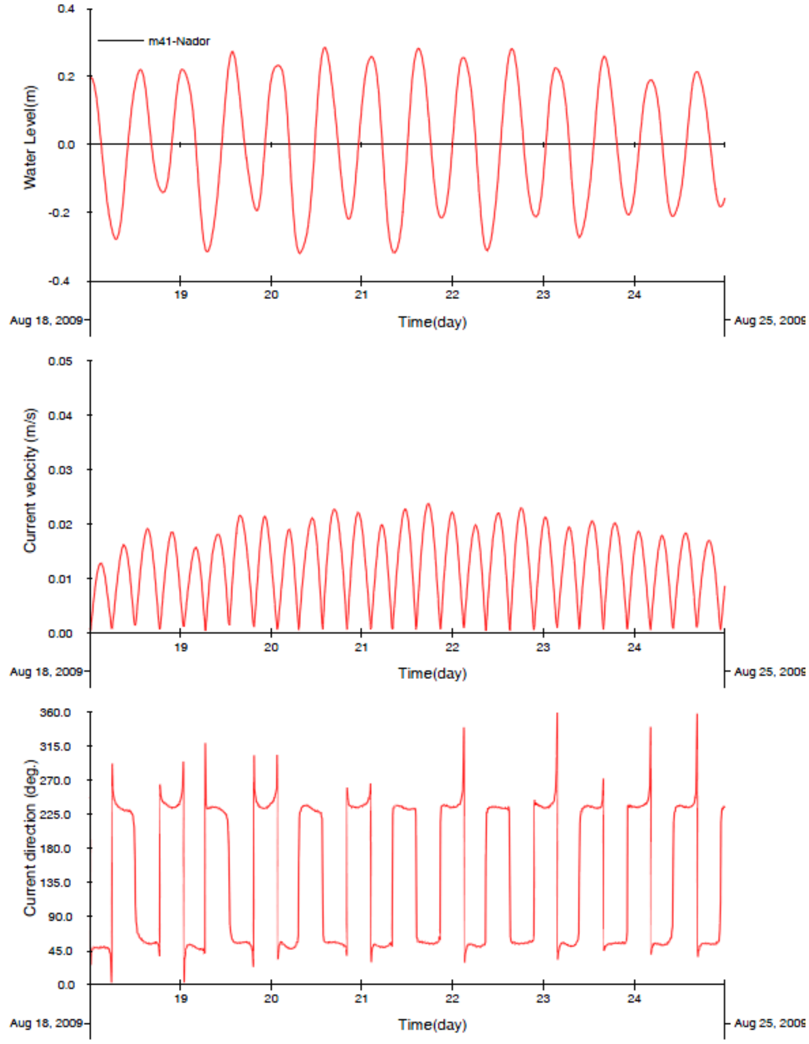


Figure A1.51

Improved container port entrance: berths run
Wave direction: 30°N
Wave height: H_s=0.75 m
Contour lines: 0.5 – 1 m

Figure A1.52

Water levels, current velocities and directions



ANNEX 2 Annex 2: Tables

2.1 Wave tables

Complementary to paragraph 3.3.4, here the tables and wind roses for the wave climate are presented.

Table A2.1

Hs at output point P38

Hs (m)	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	23,42	25,38	,70	,57	1,25	,93	,81	,40	,21	22,11	20,68	3,55	100,00
,25	13,55	6,14	,00	,00	,00	,00	,00	,00	,01	18,46	19,92	2,05	60,15
,75	3,35	,00	,00	,00	,00	,00	,00	,00	,00	9,12	17,57	1,46	31,49
1,25	,73	,00	,00	,00	,00	,00	,00	,00	,00	1,84	11,74	,82	15,12
1,75	,19	,00	,00	,00	,00	,00	,00	,00	,00	,16	5,55	,43	6,33
2,25	,05	,00	,00	,00	,00	,00	,00	,00	,00	,00	2,71	,22	2,98
2,75	,01	,00	,00	,00	,00	,00	,00	,00	,00	,00	1,24	,14	1,39
3,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,59	,06	,65
4,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,13	,02	,15
5,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,02	,00	,02

a) Computation set#1
wind was deactivated for directions 90°N, 105°N, 240°N and 270°N within the nested grids B, C and D

Hs (m)	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	20,16	28,48	,77	,62	1,26	,93	,81	,41	,30	22,51	20,17	3,57	100,00
,25	10,71	11,73	,04	,00	,00	,00	,00	,00	,05	18,98	19,33	2,04	62,89
,75	3,54	,34	,00	,00	,00	,00	,00	,00	,00	9,61	16,84	1,44	31,78
1,25	,85	,01	,00	,00	,00	,00	,00	,00	,00	2,55	11,28	,81	15,50
1,75	,22	,00	,00	,00	,00	,00	,00	,00	,00	,48	5,62	,43	6,75
2,25	,05	,00	,00	,00	,00	,00	,00	,00	,00	,09	2,98	,22	3,34
2,75	,01	,00	,00	,00	,00	,00	,00	,00	,00	,01	1,51	,14	1,66
3,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,74	,06	,80
4,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,20	,02	,21
5,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,04	,00	,04

b) Computation set#2
wind was active for all the directions and for all the grids

Probability of exeedance(%) for the wave height for output point P38 at 9.8m depth

Hs (m) Lower	Hs (m) Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	9,87	19,24	,69	,57	1,25	,93	,81	,39	,20	3,65	,76	1,50	39,85
,25	,75	10,20	6,14	,00	,00	,00	,00	,00	,00	,01	9,34	2,35	,60	28,65
,75	1,25	2,62	,00	,00	,00	,00	,00	,00	,00	,00	7,29	5,83	,64	16,37
1,25	1,75	,54	,00	,00	,00	,00	,00	,00	,00	,00	1,67	6,19	,39	8,80
1,75	2,25	,14	,00	,00	,00	,00	,00	,00	,00	,00	,16	2,83	,21	3,35
2,25	2,75	,03	,00	,00	,00	,00	,00	,00	,00	,00	,00	1,47	,09	1,59
2,75	3,25	,01	,00	,00	,00	,00	,00	,00	,00	,00	,00	,66	,08	,74
3,25	4,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,45	,04	,50
4,25	5,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,12	,02	,13
5,25	6,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,02	,00	,02

a) Computation set#1
wind was deactivated for directions 90°N, 105°N, 240°N and 270°N within the nested grids B, C and D

Hs (m) Lower	Hs (m) Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	9,45	16,75	,73	,62	1,26	,93	,81	,41	,25	3,52	,84	1,53	37,11
,25	,75	7,16	11,38	,04	,00	,00	,00	,00	,00	,05	9,37	2,48	,60	31,10
,75	1,25	2,70	,33	,00	,00	,00	,00	,00	,00	,00	7,05	5,57	,63	16,28
1,25	1,75	,63	,01	,00	,00	,00	,00	,00	,00	,00	2,07	5,66	,38	8,75
1,75	2,25	,17	,00	,00	,00	,00	,00	,00	,00	,00	,40	2,64	,21	3,41
2,25	2,75	,04	,00	,00	,00	,00	,00	,00	,00	,00	,08	1,47	,09	1,68
2,75	3,25	,01	,00	,00	,00	,00	,00	,00	,00	,00	,01	,77	,08	,86
3,25	4,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,54	,04	,59
4,25	5,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,16	,02	,18
5,25	6,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,04	,00	,04

b) Computation set#2
wind was active for all the directions and for all the grids

Probability of occurance(%) for the wave height for output point P38 at 9.8m depth

Table A2.2

Hs at output point P49

Hs (m)	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	16,24	34,59	,71	,58	1,25	,93	,81	,41	,22	27,26	15,38	1,62	100,00
,25	6,70	16,73	,00	,00	,00	,00	,00	,00	,02	23,61	14,77	1,39	63,23
,75	3,90	1,59	,00	,00	,00	,00	,00	,00	,00	14,12	13,10	,98	33,68
1,25	1,44	,14	,00	,00	,00	,00	,00	,00	,00	4,87	9,28	,59	16,32
1,75	,56	,01	,00	,00	,00	,00	,00	,00	,00	,93	4,80	,34	6,63
2,25	,20	,00	,00	,00	,00	,00	,00	,00	,00	,16	2,49	,18	3,04
2,75	,08	,00	,00	,00	,00	,00	,00	,00	,00	,01	1,08	,11	1,28
3,25	,04	,00	,00	,00	,00	,00	,00	,00	,00	,00	,48	,06	,57
4,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,10	,02	,13
5,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,01	,01

a) Computation set#1
wind was deactivated for directions 90°N, 105°N, 240°N and 270°N within the nested grids B, C and D

Hs (m)	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	15,33	34,84	1,20	,72	1,29	,93	,81	,42	,41	27,42	15,01	1,62	100,00
,25	5,81	20,44	,36	,01	,00	,00	,00	,00	,07	23,95	14,34	1,39	66,36
,75	3,35	2,90	,00	,00	,00	,00	,00	,00	,00	14,43	12,60	,98	34,26
1,25	1,36	,45	,00	,00	,00	,00	,00	,00	,00	5,54	8,78	,59	16,72
1,75	,56	,06	,00	,00	,00	,00	,00	,00	,00	1,54	4,63	,34	7,13
2,25	,21	,00	,00	,00	,00	,00	,00	,00	,00	,46	2,61	,18	3,45
2,75	,08	,00	,00	,00	,00	,00	,00	,00	,00	,13	1,31	,11	1,63
3,25	,04	,00	,00	,00	,00	,00	,00	,00	,00	,02	,66	,06	,77
4,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,17	,02	,19
5,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,03	,01	,04

b) Computation set#2
wind was active for all the directions and for all the grids

Probability of exeedance(%) for the wave height for output point P49 at 19.9m depth

Hs (m) Lower	Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	9,53	17,86	,70	,58	1,25	,93	,81	,40	,20	3,64	,62	,23	36,77
,25	,75	2,80	15,14	,00	,00	,00	,00	,00	,00	,02	9,50	1,67	,41	29,55
,75	1,25	2,46	1,45	,00	,00	,00	,00	,00	,00	,00	9,25	3,81	,39	17,35
1,25	1,75	,88	,14	,00	,00	,00	,00	,00	,00	,00	3,94	4,48	,25	9,69
1,75	2,25	,35	,01	,00	,00	,00	,00	,00	,00	,00	,77	2,31	,16	3,60
2,25	2,75	,12	,00	,00	,00	,00	,00	,00	,00	,00	,15	1,42	,07	1,76
2,75	3,25	,05	,00	,00	,00	,00	,00	,00	,00	,00	,01	,60	,05	,70
3,25	4,25	,03	,00	,00	,00	,00	,00	,00	,00	,00	,00	,38	,04	,45
4,25	5,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,10	,01	,12
5,25	6,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,01	,01

a) Computation set#1
wind was deactivated for directions 90°N, 105°N, 240°N and 270°N within the nested grids B, C and D

Hs (m) Lower	Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	9,52	14,40	,84	,71	1,29	,93	,81	,42	,34	3,47	,67	,23	33,64
,25	,75	2,46	17,54	,36	,01	,00	,00	,00	,00	,07	9,52	1,74	,41	32,11
,75	1,25	1,99	2,45	,00	,00	,00	,00	,00	,00	,00	8,89	3,81	,39	17,53
1,25	1,75	,80	,39	,00	,00	,00	,00	,00	,00	,00	4,01	4,16	,25	9,60
1,75	2,25	,35	,06	,00	,00	,00	,00	,00	,00	,00	1,08	2,02	,16	3,67
2,25	2,75	,13	,00	,00	,00	,00	,00	,00	,00	,00	,33	1,30	,07	1,82
2,75	3,25	,05	,00	,00	,00	,00	,00	,00	,00	,00	,11	,65	,05	,85
3,25	4,25	,03	,00	,00	,00	,00	,00	,00	,00	,00	,02	,49	,04	,58
4,25	5,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,14	,01	,15
5,25	6,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,03	,01	,04

b) Computation set#2
wind was active for all the directions and for all the grids

Probability of occurance(%) for the wave height for output point P49 at 19.9m depth

Table A2.3

Hs at output point P54

Hs (m)	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	12,51	38,41	,74	,60	1,26	,93	,81	,41	,23	30,24	12,38	1,48	100,00
,25	3,26	22,55	,01	,00	,00	,00	,00	,00	,03	26,70	11,80	1,26	65,61
,75	2,07	5,28	,00	,00	,00	,00	,00	,00	,00	17,51	10,34	,88	36,08
1,25	1,01	1,45	,00	,00	,00	,00	,00	,00	,00	7,49	7,54	,54	18,03
1,75	,46	,50	,00	,00	,00	,00	,00	,00	,00	2,05	4,22	,30	7,53
2,25	,23	,17	,00	,00	,00	,00	,00	,00	,00	,55	2,40	,14	3,50
2,75	,13	,05	,00	,00	,00	,00	,00	,00	,00	,11	1,14	,09	1,52
3,25	,08	,01	,00	,00	,00	,00	,00	,00	,00	,01	,52	,05	,66
4,25	,01	,00	,00	,00	,00	,00	,00	,00	,00	,00	,10	,01	,13
5,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,01

a) Computation set#1
wind was deactivated for directions 90°N, 105°N, 240°N and 270°N within the nested grids B, C and D

Hs (m)	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	12,47	33,48	5,31	,92	1,34	,93	,81	,44	,67	30,33	11,82	1,48	100,00
,25	3,22	24,10	2,36	,02	,00	,00	,00	,01	,20	27,11	11,21	1,26	69,49
,75	2,04	6,51	,03	,00	,00	,00	,00	,00	,00	18,00	9,71	,88	37,17
1,25	1,00	1,87	,00	,00	,00	,00	,00	,00	,00	8,29	6,90	,54	18,60
1,75	,45	,66	,00	,00	,00	,00	,00	,00	,00	2,75	3,88	,30	8,05
2,25	,23	,23	,00	,00	,00	,00	,00	,00	,00	1,13	2,22	,14	3,95
2,75	,13	,06	,00	,00	,00	,00	,00	,00	,00	,41	1,18	,09	1,88
3,25	,08	,02	,00	,00	,00	,00	,00	,00	,00	,12	,63	,05	,89
4,25	,01	,00	,00	,00	,00	,00	,00	,00	,00	,01	,17	,01	,21
5,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,03	,00	,04

b) Computation set#2
wind was active for all the directions and for all the grids

Probability of exeedance(%) for the wave height for output point P54 at 30.0m depth

Hs (m) Lower	Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	9,25	15,86	,73	,60	1,26	,93	,81	,41	,20	3,54	,58	,22	34,39
,25	,75	1,19	17,28	,01	,00	,00	,00	,00	,00	,03	9,19	1,45	,38	29,52
,75	1,25	1,06	3,83	,00	,00	,00	,00	,00	,00	,00	10,02	2,80	,34	18,06
1,25	1,75	,55	,95	,00	,00	,00	,00	,00	,00	,00	5,44	3,32	,24	10,50
1,75	2,25	,23	,33	,00	,00	,00	,00	,00	,00	,00	1,50	1,82	,16	4,03
2,25	2,75	,10	,12	,00	,00	,00	,00	,00	,00	,00	,44	1,26	,06	1,97
2,75	3,25	,05	,04	,00	,00	,00	,00	,00	,00	,00	,10	,63	,04	,86
3,25	4,25	,07	,01	,00	,00	,00	,00	,00	,00	,00	,01	,42	,04	,54
4,25	5,25	,01	,00	,00	,00	,00	,00	,00	,00	,00	,00	,10	,01	,12
5,25	6,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,01

a) Computation set#1
wind was deactivated for directions 90°N, 105°N, 240°N and 270°N within the nested grids B, C and D

Hs (m) Lower	Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	9,25	9,37	2,95	,90	1,34	,93	,81	,43	,47	3,21	,61	,22	30,51
,25	,75	1,18	17,59	2,34	,02	,00	,00	,00	,01	,19	9,12	1,49	,38	32,32
,75	1,25	1,04	4,64	,03	,00	,00	,00	,00	,00	,00	9,71	2,81	,34	18,57
1,25	1,75	,54	1,21	,00	,00	,00	,00	,00	,00	,00	5,54	3,02	,24	10,55
1,75	2,25	,22	,43	,00	,00	,00	,00	,00	,00	,00	1,62	1,66	,16	4,09
2,25	2,75	,10	,16	,00	,00	,00	,00	,00	,00	,00	,72	1,04	,06	2,07
2,75	3,25	,05	,05	,00	,00	,00	,00	,00	,00	,00	,30	,56	,04	,99
3,25	4,25	,07	,02	,00	,00	,00	,00	,00	,00	,00	,10	,46	,04	,68
4,25	5,25	,01	,00	,00	,00	,00	,00	,00	,00	,00	,01	,14	,01	,17
5,25	6,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,03	,00	,04

b) Computation set#2
wind was active for all the directions and for all the grids

Probability of occurance(%) for the wave height for output point P54 at 30.0m depth

Table A2.4

Hs at output point P01

Hs (m)	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	12,06	38,86	,75	,61	1,26	,93	,81	,41	,23	33,79	8,85	1,44	100,00
,25	2,89	23,64	,01	,00	,00	,00	,00	,00	,03	30,15	8,31	1,22	66,25
,75	1,80	6,12	,00	,00	,00	,00	,00	,00	,00	21,43	6,99	,84	37,18
1,25	,88	1,87	,00	,00	,00	,00	,00	,00	,00	11,35	5,00	,52	19,62
1,75	,41	,69	,00	,00	,00	,00	,00	,00	,00	4,56	3,03	,29	8,98
2,25	,22	,27	,00	,00	,00	,00	,00	,00	,00	1,95	1,81	,15	4,39
2,75	,12	,10	,00	,00	,00	,00	,00	,00	,00	,73	1,10	,09	2,14
3,25	,07	,04	,00	,00	,00	,00	,00	,00	,00	,25	,59	,04	,99
4,25	,02	,00	,00	,00	,00	,00	,00	,00	,00	,02	,17	,01	,22
5,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,03	,00	,04

a) Computation set#1
wind was deactivated for directions 90°N, 105°N, 240°N and 270°N within the nested grids B, C and D

Hs (m)	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	12,06	19,65	18,96	1,42	1,46	,93	,81	,44	1,17	33,63	8,04	1,44	100,00
,25	2,89	17,63	10,80	,11	,00	,00	,00	,01	,51	30,53	7,49	1,22	71,20
,75	1,79	7,26	,60	,00	,00	,00	,00	,00	,02	22,18	6,16	,84	38,86
1,25	,88	2,30	,06	,00	,00	,00	,00	,00	,00	12,30	4,22	,52	20,29
1,75	,41	,87	,01	,00	,00	,00	,00	,00	,00	5,49	2,42	,29	9,49
2,25	,22	,33	,00	,00	,00	,00	,00	,00	,00	2,72	1,37	,15	4,79
2,75	,12	,12	,00	,00	,00	,00	,00	,00	,00	1,28	,86	,09	2,47
3,25	,07	,04	,00	,00	,00	,00	,00	,00	,00	,62	,45	,04	1,22
4,25	,02	,00	,00	,00	,00	,00	,00	,00	,00	,15	,16	,01	,34
5,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,03	,04	,00	,08
6,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,01	,00	,00	,01

b) Computation set#2
wind was active for all the directions and for all the grids

Probability of exeedance(%) for the wave height for output point P01 at 33.0m depth

Hs (m) Lower	Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	9,17	15,22	,74	,61	1,26	,93	,81	,41	,20	3,64	,54	,22	33,75
,25	,75	1,10	17,52	,01	,00	,00	,00	,00	,00	,02	8,72	1,32	,38	29,06
,75	1,25	,92	4,25	,00	,00	,00	,00	,00	,00	,00	10,08	1,99	,33	17,56
1,25	1,75	,47	1,18	,00	,00	,00	,00	,00	,00	,00	6,79	1,97	,23	10,64
1,75	2,25	,19	,42	,00	,00	,00	,00	,00	,00	,00	2,61	1,22	,14	4,59
2,25	2,75	,10	,17	,00	,00	,00	,00	,00	,00	,00	1,22	,70	,06	2,25
2,75	3,25	,05	,06	,00	,00	,00	,00	,00	,00	,00	,49	,52	,05	1,16
3,25	4,25	,06	,04	,00	,00	,00	,00	,00	,00	,00	,23	,42	,03	,77
4,25	5,25	,01	,00	,00	,00	,00	,00	,00	,00	,00	,02	,14	,01	,18
5,25	6,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,00	,03	,00	,04

a) Computation set#1
wind was deactivated for directions 90°N, 105°N, 240°N and 270°N within the nested grids B, C and D

Hs (m) Lower	Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	9,17	2,02	8,16	1,30	1,46	,93	,81	,43	,66	3,10	,54	,22	28,80
,25	,75	1,10	10,37	10,20	,11	,00	,00	,00	,01	,49	8,35	1,33	,38	32,34
,75	1,25	,91	4,96	,54	,00	,00	,00	,00	,00	,02	9,88	1,94	,33	18,57
1,25	1,75	,47	1,43	,06	,00	,00	,00	,00	,00	,00	6,81	1,80	,23	10,80
1,75	2,25	,19	,54	,01	,00	,00	,00	,00	,00	,00	2,78	1,04	,14	4,70
2,25	2,75	,10	,21	,00	,00	,00	,00	,00	,00	,00	1,43	,51	,06	2,31
2,75	3,25	,05	,08	,00	,00	,00	,00	,00	,00	,00	,67	,41	,05	1,25
3,25	4,25	,06	,04	,00	,00	,00	,00	,00	,00	,00	,47	,29	,03	,88
4,25	5,25	,01	,00	,00	,00	,00	,00	,00	,00	,00	,11	,12	,01	,26
5,25	6,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,03	,04	,00	,07
6,25	7,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,01	,00	,00	,01

b) Computation set#2
wind was active for all the directions and for all the grids

Probability of occurance(%) for the wave height for output point P01 at 33.0m depth

Table A2.5

Hs at output point P60

Hs (m)	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	12,05	38,86	,74	,62	1,26	,93	,81	,41	,23	34,34	8,33	1,41	100,00
,25	2,86	23,77	,01	,00	,00	,00	,00	,00	,04	30,80	7,79	1,20	66,45
,75	1,78	6,29	,00	,00	,00	,00	,00	,00	,00	22,18	6,47	,83	37,55
1,25	,88	1,91	,00	,00	,00	,00	,00	,00	,00	11,99	4,49	,51	19,79
1,75	,41	,71	,00	,00	,00	,00	,00	,00	,00	4,95	2,59	,29	8,94
2,25	,22	,28	,00	,00	,00	,00	,00	,00	,00	2,19	1,50	,14	4,34
2,75	,12	,11	,00	,00	,00	,00	,00	,00	,00	,87	,91	,08	2,08
3,25	,07	,05	,00	,00	,00	,00	,00	,00	,00	,31	,48	,04	,95
4,25	,02	,00	,00	,00	,00	,00	,00	,00	,00	,03	,14	,01	,20
5,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,01	,01	,00	,03

a) Computation set#1
wind was deactivated for directions 90°N, 105°N, 240°N and 270°N within the nested grids B, C and D

Hs (m)	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	12,04	31,08	8,10	,96	1,35	,93	,81	,45	1,31	33,63	7,92	1,41	100,00
,25	2,85	23,99	3,88	,03	,00	,00	,00	,01	,60	30,69	7,36	1,20	70,61
,75	1,77	7,74	,07	,00	,00	,00	,00	,00	,02	22,50	6,03	,83	38,96
1,25	,88	2,39	,01	,00	,00	,00	,00	,00	,00	12,55	4,05	,51	20,39
1,75	,41	,89	,00	,00	,00	,00	,00	,00	,00	5,63	2,22	,29	9,44
2,25	,22	,34	,00	,00	,00	,00	,00	,00	,00	2,84	1,21	,14	4,76
2,75	,12	,13	,00	,00	,00	,00	,00	,00	,00	1,39	,73	,08	2,45
3,25	,07	,05	,00	,00	,00	,00	,00	,00	,00	,67	,37	,04	1,21
4,25	,02	,00	,00	,00	,00	,00	,00	,00	,00	,17	,13	,01	,33
5,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,04	,03	,00	,08
6,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,01	,00	,00	,01

b) Computation set#2
wind was active for all the directions and for all the grids

Probability of exeedance(%) for the wave height for output point P60 at 40.0m depth

Hs (m) Lower	Hs (m) Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	9,19	15,10	,74	,62	1,26	,93	,81	,41	,19	3,54	,54	,22	33,55
,25	,75	1,08	17,48	,01	,00	,00	,00	,00	,00	,03	8,62	1,32	,37	28,90
,75	1,25	,90	4,38	,00	,00	,00	,00	,00	,00	,00	10,19	1,97	,32	17,76
1,25	1,75	,47	1,20	,00	,00	,00	,00	,00	,00	,00	7,04	1,90	,23	10,85
1,75	2,25	,19	,43	,00	,00	,00	,00	,00	,00	,00	2,76	1,09	,14	4,60
2,25	2,75	,10	,17	,00	,00	,00	,00	,00	,00	,00	1,32	,60	,06	2,25
2,75	3,25	,05	,06	,00	,00	,00	,00	,00	,00	,00	,56	,42	,04	1,13
3,25	4,25	,06	,04	,00	,00	,00	,00	,00	,00	,00	,28	,34	,03	,75
4,25	5,25	,01	,00	,00	,00	,00	,00	,00	,00	,00	,02	,13	,01	,17
5,25	6,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,01	,01	,00	,03

a) Computation set#1
wind was deactivated for directions 90°N, 105°N, 240°N and 270°N within the nested grids B, C and D

Hs (m) Lower	Hs (m) Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	9,19	7,09	4,23	,93	1,35	,93	,81	,44	,71	2,94	,56	,22	29,39
,25	,75	1,08	16,25	3,81	,03	,00	,00	,00	,01	,58	8,19	1,33	,37	31,65
,75	1,25	,90	5,35	,06	,00	,00	,00	,00	,00	,01	9,95	1,98	,32	18,56
1,25	1,75	,47	1,50	,01	,00	,00	,00	,00	,00	,00	6,92	1,83	,23	10,95
1,75	2,25	,18	,55	,00	,00	,00	,00	,00	,00	,00	2,79	1,01	,14	4,68
2,25	2,75	,10	,22	,00	,00	,00	,00	,00	,00	,00	1,45	,48	,06	2,31
2,75	3,25	,05	,08	,00	,00	,00	,00	,00	,00	,00	,72	,36	,04	1,25
3,25	4,25	,06	,04	,00	,00	,00	,00	,00	,00	,00	,51	,24	,03	,88
4,25	5,25	,01	,00	,00	,00	,00	,00	,00	,00	,00	,13	,09	,01	,25
5,25	6,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,03	,03	,00	,07
6,25	7,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,01	,00	,00	,01

b) Computation set#2
wind was active for all the directions and for all the grids

Probability of occurence(%) for the wave height for output point P60 at 40.0m depth

Table A2.6

Hs at output point P08

Hs (m)	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	11,50	39,28	,80	,71	1,29	,93	,81	,41	,26	35,68	7,00	1,33	100,00
,25	2,41	28,56	,09	,01	,00	,00	,00	,00	,06	32,50	6,54	1,13	71,32
,75	1,48	10,22	,00	,00	,00	,00	,00	,00	,00	24,34	5,36	,79	42,18
1,25	,79	3,63	,00	,00	,00	,00	,00	,00	,00	14,20	3,69	,49	22,81
1,75	,40	1,47	,00	,00	,00	,00	,00	,00	,00	6,63	2,05	,27	10,82
2,25	,21	,70	,00	,00	,00	,00	,00	,00	,00	3,15	1,09	,14	5,29
2,75	,12	,32	,00	,00	,00	,00	,00	,00	,00	1,48	,69	,08	2,69
3,25	,08	,15	,00	,00	,00	,00	,00	,00	,00	,65	,36	,04	1,27
4,25	,03	,03	,00	,00	,00	,00	,00	,00	,00	,14	,11	,01	,31
5,25	,01	,00	,00	,00	,00	,00	,00	,00	,00	,03	,01	,01	,06
6,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,002	,00	,00	,002

a) Computation set#1
wind was deactivated for directions 90°N, 105°N, 240°N and 270°N within the nested grids B, C and D

Hs (m)	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	11,50	13,87	24,54	2,13	1,53	,93	,81	,47	1,76	34,43	6,69	1,33	100,00
,25	2,41	12,71	18,58	,46	,00	,00	,00	,01	1,03	31,82	6,24	1,13	74,40
,75	1,48	8,12	4,28	,01	,00	,00	,00	,00	,12	24,43	5,09	,79	44,31
1,25	,79	3,49	,88	,00	,00	,00	,00	,00	,02	14,57	3,45	,49	23,69
1,75	,40	1,59	,25	,00	,00	,00	,00	,00	,01	7,13	1,87	,27	11,51
2,25	,21	,81	,07	,00	,00	,00	,00	,00	,00	3,60	,95	,14	5,77
2,75	,12	,39	,01	,00	,00	,00	,00	,00	,00	1,87	,59	,08	3,06
3,25	,08	,18	,00	,00	,00	,00	,00	,00	,00	,96	,30	,04	1,55
4,25	,03	,03	,00	,00	,00	,00	,00	,00	,00	,25	,09	,01	,41
5,25	,01	,00	,00	,00	,00	,00	,00	,00	,00	,06	,02	,01	,10
6,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,01	,00	,00	,01

b) Computation set#2
wind was active for all the directions and for all the grids

Probability of exeedance(%) for the wave height for output point P08 at 56.0m depth

Hs (m)		wave direction (Deg)												
Lower	Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	9,09	10,72	,71	,70	1,29	,93	,81	,41	,20	3,17	,46	,20	28,68
,25	,75	,93	18,35	,09	,01	,00	,00	,00	,00	,06	8,16	1,19	,34	29,13
,75	1,25	,68	6,59	,00	,00	,00	,00	,00	,00	,00	10,14	1,67	,30	19,37
1,25	1,75	,40	2,16	,00	,00	,00	,00	,00	,00	,00	7,57	1,64	,22	11,99
1,75	2,25	,19	,77	,00	,00	,00	,00	,00	,00	,00	3,48	,96	,13	5,53
2,25	2,75	,09	,38	,00	,00	,00	,00	,00	,00	,00	1,67	,40	,06	2,60
2,75	3,25	,04	,17	,00	,00	,00	,00	,00	,00	,00	,83	,33	,04	1,41
3,25	4,25	,05	,12	,00	,00	,00	,00	,00	,00	,00	,52	,25	,03	,96
4,25	5,25	,02	,03	,00	,00	,00	,00	,00	,00	,00	,11	,09	,01	,25
5,25	6,25	,01	,00	,00	,00	,00	,00	,00	,00	,00	,03	,01	,01	,06
6,25	7,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,002	,00	,00	,002

a) Computation set#1
wind was deactivated for directions 90°N, 105°N, 240°N and 270°N within the nested grids B, C and D

Hs (m)		wave direction (Deg)												
Lower	Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	9,09	1,16	5,96	1,67	1,53	,93	,81	,46	,73	2,61	,45	,20	25,60
,25	,75	,93	4,59	14,30	,45	,00	,00	,00	,01	,91	7,39	1,16	,34	30,09
,75	1,25	,68	4,63	3,41	,01	,00	,00	,00	,00	,10	9,86	1,64	,30	20,62
1,25	1,75	,40	1,90	,63	,00	,00	,00	,00	,00	,01	7,44	1,58	,22	12,18
1,75	2,25	,19	,78	,18	,00	,00	,00	,00	,00	,01	3,53	,91	,13	5,73
2,25	2,75	,09	,42	,06	,00	,00	,00	,00	,00	,00	1,73	,36	,06	2,72
2,75	3,25	,04	,21	,01	,00	,00	,00	,00	,00	,00	,91	,29	,04	1,50
3,25	4,25	,05	,15	,00	,00	,00	,00	,00	,00	,00	,71	,21	,03	1,14
4,25	5,25	,02	,03	,00	,00	,00	,00	,00	,00	,00	,18	,07	,01	,31
5,25	6,25	,01	,00	,00	,00	,00	,00	,00	,00	,00	,05	,02	,01	,09
6,25	7,25	,00	,00	,00	,00	,00	,00	,00	,00	,00	,01	,00	,00	,01

b) Computation set#2
wind was active for all the directions and for all the grids

Probability of occurence(%) for the wave height for output point P08 at 56.0m depth

Table A2.7

Tp at output point P38

Hs (m)		wave direction (Deg)												
Lower	Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	1,7	3,2	4,6	5,1	5,5	5,0	4,5	5,0	4,5	2,5	2,9	9,7	3,2
,25	,75	7,1	4,7	11,9	-	-	-	-	9,1	7,9	5,2	5,5	5,4	5,8
,75	1,25	8,4	-	-	-	-	-	-	-	-	6,1	7,4	7,1	7,0
1,25	1,75	9,9	-	-	-	-	-	-	-	-	6,4	7,8	7,6	7,7
1,75	2,25	11,5	-	-	-	-	-	-	-	-	7,3	8,6	9,0	8,7
2,25	2,75	11,3	-	-	-	-	-	-	-	-	8,0	9,2	9,4	9,3
2,75	3,25	12,9	-	-	-	-	-	-	-	-	-	10,3	11,1	10,4
3,25	4,25	14,9	-	-	-	-	-	-	-	-	-	11,4	10,8	11,3
4,25	5,25	-	-	-	-	-	-	-	-	-	-	13,8	12,6	13,7
5,25	6,25	-	-	-	-	-	-	-	-	-	-	16,2	12,3	15,9

a) Computation set#1
wind was deactivated for directions 90°N, 105°N, 240°N and 270°N within the nested grids B, C and D

Hs (m)		wave direction (Deg)												
Lower	Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	1,3	2,9	3,8	4,8	5,5	5,0	4,5	4,9	3,6	2,6	2,9	9,7	3,0
,25	,75	6,9	5,3	4,9	9,6	-	-	-	9,7	6,9	5,3	5,4	5,4	5,7
,75	1,25	8,2	5,7	-	-	-	-	-	-	9,2	6,4	7,2	7,0	6,9
1,25	1,75	9,5	6,9	-	-	-	-	-	-	-	7,0	7,8	7,5	7,7
1,75	2,25	11,8	-	-	-	-	-	-	-	-	7,7	8,6	8,8	8,7
2,25	2,75	11,2	-	-	-	-	-	-	-	-	8,0	9,2	9,4	9,2
2,75	3,25	12,9	-	-	-	-	-	-	-	-	9,9	10,0	11,1	10,2
3,25	4,25	14,9	-	-	-	-	-	-	-	-	-	11,0	10,8	11,0
4,25	5,25	-	-	-	-	-	-	-	-	-	-	13,1	12,6	13,0
5,25	6,25	-	-	-	-	-	-	-	-	-	-	15,8	12,3	15,7

b) Computation set#2
wind was active for all the directions and for all the grids

Mean value of spectral peak wave period (s) at P38

Table A2.8

Tp at output point P49

Hs (m)		wave direction (Deg)												
Lower	Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	2,2	4,5	4,4	5,1	5,5	5,0	4,5	5,0	4,6	2,5	2,6	1,9	3,7
,25	,75	6,7	6,2	10,9	-	-	-	-	9,1	7,7	5,3	4,9	4,7	5,9
,75	1,25	8,4	6,8	-	-	-	-	-	-	-	6,5	7,3	6,2	6,9
1,25	1,75	9,0	7,7	-	-	-	-	-	-	-	6,7	8,0	7,1	7,5
1,75	2,25	9,9	10,2	-	-	-	-	-	-	-	7,3	8,7	8,4	8,5
2,25	2,75	9,8	-	-	-	-	-	-	-	-	7,8	9,2	9,7	9,2
2,75	3,25	11,1	-	-	-	-	-	-	-	-	9,2	10,2	10,1	10,2
3,25	4,25	12,1	-	-	-	-	-	-	-	-	-	11,3	9,2	11,2
4,25	5,25	14,3	-	-	-	-	-	-	-	-	-	14,1	11,8	13,8
5,25	6,25	-	-	-	-	-	-	-	-	-	-	13,8	12,4	12,8

a) Computation set#1
wind was deactivated for directions 90°N, 105°N, 240°N and 270°N within the nested grids B, C and D

Hs (m)		wave direction (Deg)												
Lower	Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	2,2	3,4	3,4	4,7	5,6	5,0	4,5	4,9	3,4	2,6	2,6	1,9	3,1
,25	,75	5,9	5,7	4,8	7,9	-	-	-	9,5	6,6	5,3	4,9	4,7	5,5
,75	1,25	7,7	7,5	-	-	-	-	-	-	7,5	6,6	7,1	6,2	6,9
1,25	1,75	8,9	8,0	-	-	-	-	-	-	-	7,0	7,8	7,1	7,6
1,75	2,25	10,0	7,8	-	-	-	-	-	-	-	7,8	8,7	8,4	8,5
2,25	2,75	9,8	11,1	-	-	-	-	-	-	-	8,2	9,2	9,7	9,1
2,75	3,25	11,1	-	-	-	-	-	-	-	-	8,8	10,1	10,1	10,0
3,25	4,25	12,1	-	-	-	-	-	-	-	-	9,5	11,0	9,2	10,9
4,25	5,25	14,3	-	-	-	-	-	-	-	-	-	13,0	11,8	12,9
5,25	6,25	-	-	-	-	-	-	-	-	-	-	14,9	12,4	14,5

b) Computation set#2
wind was active for all the directions and for all the grids

Mean value of spectral peak wave period (s) at P49

Table A2.9

Tp at output point P54

Hs (m) Lower	Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	2,2	4,5	5,4	5,6	5,6	5,0	4,5	5,0	4,5	2,4	2,5	1,8	3,7
,25	,75	4,5	6,6	11,3	-	-	-	-	-	7,4	5,2	4,6	4,6	5,9
,75	1,25	7,1	7,8	-	-	-	-	-	-	-	6,5	6,7	6,1	6,8
1,25	1,75	7,8	8,8	-	-	-	-	-	-	-	6,9	7,9	7,0	7,4
1,75	2,25	9,2	9,4	-	-	-	-	-	-	-	7,6	8,9	8,0	8,4
2,25	2,75	9,3	10,3	-	-	-	-	-	-	-	7,8	9,4	9,9	9,1
2,75	3,25	9,5	10,8	-	-	-	-	-	-	-	8,8	10,0	9,6	9,8
3,25	4,25	11,1	9,5	-	-	-	-	-	-	-	9,7	11,1	9,6	10,9
4,25	5,25	12,7	-	-	-	-	-	-	-	-	-	13,9	11,8	13,6
5,25	6,25	13,9	-	-	-	-	-	-	-	-	-	13,7	11,6	13,0

a) Computation set#1
wind was deactivated for directions 90°N, 105°N, 240°N and 270°N within the nested grids B, C and D

Hs (m) Lower	Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	2,2	3,8	3,9	4,9	5,6	5,0	4,5	4,9	3,4	2,4	2,5	1,8	3,3
,25	,75	4,4	6,3	5,4	6,8	-	-	-	9,1	5,7	5,3	4,7	4,6	5,8
,75	1,25	6,9	7,5	5,4	-	-	-	-	-	8,5	6,7	6,9	6,1	6,9
1,25	1,75	7,7	8,4	-	-	-	-	-	-	7,6	7,2	7,8	7,0	7,5
1,75	2,25	8,9	9,1	-	-	-	-	-	-	-	7,9	8,7	8,0	8,4
2,25	2,75	9,0	10,3	-	-	-	-	-	-	-	8,4	9,2	9,9	9,0
2,75	3,25	9,5	10,7	-	-	-	-	-	-	-	8,9	10,1	9,6	9,7
3,25	4,25	11,1	9,6	-	-	-	-	-	-	-	9,6	11,0	9,6	10,7
4,25	5,25	12,7	-	-	-	-	-	-	-	-	12,0	12,7	11,8	12,6
5,25	6,25	13,9	-	-	-	-	-	-	-	-	-	14,1	11,6	13,8

b) Computation set#2
wind was active for all the directions and for all the grids

Mean value of spectral peak wave period (s) at P54

Table A2.10

Tp at output point P01

Hs (m) Lower	Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	2,2	4,4	5,7	5,7	5,6	5,0	4,5	5,0	4,7	2,4	2,4	1,7	3,7
,25	,75	4,3	6,5	11,3	11,2	-	-	-	9,0	7,9	5,1	4,5	4,5	5,9
,75	1,25	6,9	7,9	-	-	-	-	-	-	9,1	6,6	6,3	6,1	6,9
1,25	1,75	7,6	9,0	-	-	-	-	-	-	-	7,3	7,4	6,9	7,5
1,75	2,25	8,8	9,8	-	-	-	-	-	-	-	8,1	8,4	7,5	8,4
2,25	2,75	8,5	10,6	-	-	-	-	-	-	-	8,5	9,3	9,6	8,9
2,75	3,25	9,2	10,7	-	-	-	-	-	-	-	8,8	10,4	10,0	9,7
3,25	4,25	10,4	11,1	-	-	-	-	-	-	-	9,5	11,3	9,3	10,6
4,25	5,25	12,1	12,1	-	-	-	-	-	-	-	10,6	12,9	11,3	12,5
5,25	6,25	13,7	-	-	-	-	-	-	-	-	12,3	15,4	11,4	14,6

a) Computation set#1
wind was deactivated for directions 90°N, 105°N, 240°N and 270°N within the nested grids B, C and D

Hs (m) Lower	Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	2,2	3,3	3,2	3,3	5,4	5,0	4,5	4,9	3,2	2,2	2,4	1,7	3,0
,25	,75	4,3	6,2	5,5	5,2	-	-	-	8,2	5,8	5,1	4,5	4,5	5,5
,75	1,25	6,8	7,8	7,6	10,4	-	-	-	-	7,4	6,6	6,3	6,1	6,9
1,25	1,75	7,6	8,7	5,4	-	-	-	-	-	8,0	7,4	7,4	6,9	7,5
1,75	2,25	8,7	9,2	6,0	-	-	-	-	-	7,6	8,1	8,3	7,5	8,3
2,25	2,75	8,5	9,9	-	-	-	-	-	-	-	8,7	8,9	9,6	8,8
2,75	3,25	9,2	10,4	-	-	-	-	-	-	-	9,2	10,1	10,0	9,6
3,25	4,25	10,4	11,1	-	-	-	-	-	-	-	9,9	11,2	9,3	10,4
4,25	5,25	12,1	12,1	-	-	-	-	-	-	-	10,6	12,6	11,3	11,7
5,25	6,25	13,7	-	-	-	-	-	-	-	-	13,9	14,1	11,4	13,9
6,25	7,25	-	-	-	-	-	-	-	-	-	14,5	12,6	-	14,3

b) Computation set#2
wind was active for all the directions and for all the grids

Mean value of spectral peak wave period (s) at P01

Table A2.11

Tp at output point P60

Hs (m) Lower	Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	2,2	4,4	5,4	5,6	5,6	5,0	4,5	5,0	4,6	2,5	2,4	1,8	3,6
,25	,75	4,2	6,5	11,5	11,4	-	-	-	9,0	7,7	5,1	4,5	4,5	5,9
,75	1,25	6,7	7,9	-	-	-	-	-	-	9,1	6,6	6,3	6,0	6,9
1,25	1,75	7,5	9,0	-	-	-	-	-	-	-	7,4	7,5	7,0	7,6
1,75	2,25	8,8	9,9	-	-	-	-	-	-	-	8,2	8,4	7,7	8,4
2,25	2,75	8,5	10,3	-	-	-	-	-	-	-	8,7	9,1	9,8	8,9
2,75	3,25	9,2	10,6	-	-	-	-	-	-	-	9,1	10,2	9,8	9,6
3,25	4,25	10,5	10,8	-	-	-	-	-	-	-	10,0	11,2	9,4	10,6
4,25	5,25	12,4	11,8	-	-	-	-	-	-	-	12,8	13,0	11,3	12,8
5,25	6,25	13,7	-	-	-	-	-	-	-	-	15,1	12,6	11,6	13,5

a) Computation set#1
wind was deactivated for directions 90°N, 105°N, 240°N and 270°N within the nested grids B, C and D

Hs (m) Lower	Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	2,2	3,6	3,4	5,0	5,6	5,0	4,5	4,8	3,2	2,3	2,4	1,8	3,2
,25	,75	4,2	6,2	5,7	6,3	-	-	-	8,2	6,1	5,1	4,5	4,5	5,7
,75	1,25	6,7	7,8	6,2	-	-	-	-	-	7,4	6,6	6,3	6,0	6,9
1,25	1,75	7,5	8,6	5,2	-	-	-	-	-	8,0	7,4	7,4	7,0	7,6
1,75	2,25	8,6	9,2	-	-	-	-	-	-	7,6	8,2	8,4	7,7	8,4
2,25	2,75	8,5	9,9	-	-	-	-	-	-	-	8,7	8,8	9,8	8,9
2,75	3,25	9,2	10,3	-	-	-	-	-	-	-	9,4	9,8	9,8	9,6
3,25	4,25	10,5	10,8	-	-	-	-	-	-	-	10,1	10,9	9,4	10,4
4,25	5,25	12,4	12,0	-	-	-	-	-	-	-	11,1	12,5	11,3	11,7
5,25	6,25	13,7	-	-	-	-	-	-	-	-	13,8	14,3	11,6	13,9
6,25	7,25	-	-	-	-	-	-	-	-	-	13,5	12,6	-	13,4

b) Computation set#2
wind was active for all the directions and for all the grids

Mean value of spectral peak wave period (s) at P60

Table A2.12

Tp at output point P08

Hs (m) Lower	Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	2,2	4,0	5,6	5,8	5,6	5,0	4,5	5,0	4,2	2,2	2,5	1,7	3,4
,25	,75	4,0	6,5	7,7	10,7	-	-	-	9,6	6,9	5,0	4,4	4,4	5,9
,75	1,25	6,3	7,7	-	-	-	-	-	-	9,1	6,4	6,3	6,1	6,8
1,25	1,75	7,3	8,6	-	-	-	-	-	-	7,6	7,3	7,2	6,9	7,5
1,75	2,25	8,4	9,4	-	-	-	-	-	-	-	8,1	7,9	7,3	8,2
2,25	2,75	8,3	9,9	-	-	-	-	-	-	-	8,7	8,5	9,4	8,9
2,75	3,25	8,6	10,6	-	-	-	-	-	-	-	9,4	9,6	10,4	9,6
3,25	4,25	9,6	10,8	-	-	-	-	-	-	-	9,9	10,8	9,2	10,2
4,25	5,25	10,2	11,6	-	-	-	-	-	-	-	11,8	12,5	10,7	11,9
5,25	6,25	13,6	13,4	-	-	-	-	-	-	-	12,9	15,0	11,4	13,4
6,25	7,25	-	-	-	-	-	-	-	-	-	13,9	-	-	13,9

a) Computation set#1
wind was deactivated for directions 90°N, 105°N, 240°N and 270°N within the nested grids B, C and D

Hs (m) Lower	Upper	-15 to 15	15 to 45	45 to 75	75 to 105	105 to 135	135 to 165	165 to 195	195 to 225	225 to 255	255 to 285	285 to 315	315 to 345	Total
<	,25	2,2	2,6	3,6	4,3	5,5	5,0	4,5	4,8	3,1	2,1	2,5	1,7	3,1
,25	,75	4,0	5,8	6,1	5,0	-	-	-	8,3	5,9	4,9	4,5	4,4	5,6
,75	1,25	6,3	7,3	7,8	7,9	-	-	-	-	7,7	6,4	6,3	6,1	6,8
1,25	1,75	7,3	8,3	9,0	-	-	-	-	-	9,5	7,3	7,2	6,9	7,5
1,75	2,25	8,4	9,4	9,2	-	-	-	-	-	8,0	8,1	7,9	7,3	8,2
2,25	2,75	8,3	9,8	9,5	-	-	-	-	-	-	8,7	8,3	9,4	8,9
2,75	3,25	8,6	10,6	9,7	-	-	-	-	-	-	9,3	9,3	10,4	9,5
3,25	4,25	9,6	10,9	9,9	-	-	-	-	-	-	10,0	10,3	9,2	10,1
4,25	5,25	10,2	11,4	-	-	-	-	-	-	-	10,9	12,7	10,7	11,3
5,25	6,25	13,6	13,3	-	-	-	-	-	-	-	13,2	15,1	11,4	13,6
6,25	7,25	-	-	-	-	-	-	-	-	-	13,1	-	-	13,1

b) Computation set#2
wind was active for all the directions and for all the grids

Mean value of spectral peak wave period (s) at P08

Table A2.13

Calculation table design storm wave heights

For direction 255N - 285N in P08

alfa 0,88

Hs bin [m]		P.table	P.norm	P.cum	Q (1-P.cum)	LN(Q)	t.s [h]	Qs s/y	LN(Qs)	LN(Hs.max)	Ws.calc	Ws.def
,00	,25	2,61	0,07587	0,07587	0,92413	0,079	12	674,61	6,514142	-1,38629	8,410809	-8,4108
,25	,75	7,39	0,21467	0,29054	0,70946	0,343		517,90	6,249788	-0,28768	8,024027	-8,0240
,75	1,25	9,86	0,28624	0,57679	0,42321	0,860		308,95	5,733168	0,22314	7,274651	-7,2747
1,25	1,75	7,44	0,21610	0,79288	0,20712	1,574	Ns	151,19	5,018569	0,55962	6,253361	-6,2534
1,75	2,25	3,53	0,10261	0,89550	0,10450	2,259		76,29	4,334503	0,81093	5,294131	-5,2941
2,25	2,75	1,73	0,05017	0,94567	0,05433	2,913	730	39,66	3,680362	1,01160	4,395998	-4,3960
2,75	3,25	,91	0,02651	0,97218	0,02782	3,582		20,31	3,011007	1,17865	3,499376	-3,4994
3,25	4,25	,71	0,02070	0,99288	0,00712	4,945		5,20	1,648462	1,44692	1,764739	-1,7647
4,25	5,25	,18	0,00526	0,99814	0,00186	6,287		1,36	0,306087	1,65823	0,260455	-0,2605
5,25	6,25	,05	0,00154	0,99968	0,00032	8,048		0,23	-1,4549	1,83258	1,531222	1,5312
6,25	7,25	,01	0,00032	1,00000	0,00000	0		0,00	0	1,98100	0	0,0000

Table A2.14

Calculation pressures on solid vertical breakwater

Solid Vertical Breakwater			
Hmax	**8,45**	**m**	
cosB	1		
n*	12,675	> hc=10.2	
w	0,421407465		
w2	0,177584252	gktanhkh	
k	0,02298246		
L	273,3904598	m	
h	46,35		
kh	1,065237021		
hb	46,65		
d	26,35		
h'	26,35		
a1	0,731770555		
a2	0,014916788	min	
	6,236686391		
a3	0,781758413		
w0	10,1043	kN/m3	
p1	63,75316214	kN/m2	
p2	39,2789703	kN/m2	
p3	49,83957088	kN/m2	
p4	12,44884231	kN/m2	hc=10.2 m.
pu	48,84391143	kN/m2	

Table A2.15

Historic recordings serious tsunamis in the Mediterranean [WEBSITE TSUNAMI INSITUTE]

Date	Sea region	Affected region	Magnitude	Max. average wave height	Casualties
21.05.2003	Algeria	N. Algeria, Mallorca	6,8	2,00 m	k. A.
17.08.1999	Turkey	Kocaeli, Turkey	7,6	2,50 m	k. A.
31.12.1995	Greece	W. Corinthos Gulf	-	2,00 m	k. A.
20.04.1988	Italy	La Fossa Volcano, Vulcano Is.	-	5,50 m	k. A.
19.02.1968	Greece	Lemnos Island, North Aegean Sea	7,1	1,20 m	k. A.
06.07.1965	Greece	North Corinth Gulf, Greece	6,9	20,00 m	k. A.
02.11.1956	Greece	Thessaloniki (Salonica), Greece	-	1,20 m	k. A.
09.07.1956	Greece	Amorgos Island, Aegean Islands	7,5	20,00 m	53
22.04.1948	Greece	Ionian Islands	6,5	1,00 m	k. A.
09.02.1948	Greece	Dodecanese, Karpathos Is.	7,1	k. A.	k. A.
26.09.1932	Greece	Gulf Of Hierissos	7	k. A.	k. A.
11.09.1927	Ukraine	Black Sea	6,8	1,00 m	k. A.
27.11.1914	Greece	Ionian Islands	6,3	3,30 m	k. A.
28.12.1908	Italy	Messina	7,2	1,00 m	75.000
27.04.1894	Greece	Central Greece	7	3,00 m	k. A.
09.02.1893	Greece	Samothrace	6,5	1,00 m	k. A.
27.08.1886	Greece	Peloponnesus, Greece	7,5	1,00 m	k. A.
20.09.1867	Greece	Peloponnesus, Greece	7,1	k. A.	12
26.12.1861	Greece	Corinth Gulf	7,3	2,50 m	k. A.
28.02.1851	Turkey	Fetiye, Asia Minor	7,2	1,00 m	k. A.
23.01.1838	Romania	Romania	6,9	1,00 m	k. A.
26.10.1802	Romania	Romania	7,5	1,00 m	k. A.
10.07.1688	Turkey	Smyrna, W. Turkey	7	k. A.	k. A.
26.09.1650	Greece	Thera Island (Santorini)	-	16,00 m	k. A.
07.03.1629	Greece	Islands Of Cythera & Crete	7	k. A	k. A.
14.09.1509	Turkey	Constantinople	7,6	k. A	k. A.
03.10.1481	Greece	Island Of Rhodes	7,1	k. A.	k. A.
1427	Black Sea	Black Sea	7	2,00 m	k. A.
1341	Turkey	Sea Of Azov	6,5	1,00 m	k. A.
08.08.1303	Greece	Lybian Sea	8	k. A.	k. A.
26.10.740	Turkey	Marmara Sea	7,3	k. A.	k. A.
15.08.557	Turkey	Bali Sea	7,5	2,00 m	k. A.
15.08.554	Turkey	Mandelia, Turkey	7	3,00 m	k. A.
543	Bulgaria	Bulgaria	7,4	4,00 m	k. A.
08.11.447	Turkey	Marmara Sea	7,2	k. A.	k. A.
21.07.365	Greece	Crete, Egypt, Sicily	8	k. A.	50.000
142	Greece	Island Of Rhodes	7	k. A.	k. A.
-227	Greece	Dodecanese	7,2	k. A.	k. A.
-373	Greece	Helike, Golf of Korinth	8	k. A.	k. A.
-426	Greece	Euboean Greece	7,1	k. A.	k. A.
-1628	Greece	Santorini, Crete,	k. A.	k. A.	k. A.

ANNEX 3 Annex 3: Calculations & considerations

3.1 Sediment transport

The sediment load of the different wadis was not known beforehand. To make a calculated estimate, this has been calculated according the morphological modelling formulae [DE VRIEND *et al.*, 2010]:

$s=m*u^n$ and $S=B*s$.

In which:

s = sediment load [m^2/s]
$m = 0.05/(\sqrt{(g)}*C^3*\Delta^2*D_{50})$ [m]
u = flow velocity [m/s]
n = constant factor (=5)
S = sediment load [m^3/s]
B = river width [m]
d = water depth [m]

Rio Kert (around Punta Betoya)

To make a calculated estimate, this has been calculated according the morphological modelling formulae [DE VRIEND *et al.*, 2010]:

$s=m*u^n$ and $S=B*s$.

In which the parameters are:

$m = 0.05/(\sqrt{(g)}*C^3*\Delta^2*D_{50}) = 0.05/(\sqrt{(9,81)}*50^3*1,65^2*(0,2*10^{-3})) = 2,3*10^{-4}$ [m]
$u = Q/(B*d) = 3000/(200*4)= 3,75$ m/s
$n = 5$
$B = 200$ m. from [GOOGLE MAPS].
$d = 4$ m. (estimate from bridge height over de Rio Kert [GOOGLE MAPS])

This results in S=34,1 m^3/s.

To find the yearly sediment transport, one has to take into account that the maximum discharge will not occur all the time. Only 42.106*10^3/(3000*3600)= 4 hours are needed to discharge the whole yearly runoff into the Mediterranean Sea. With the peak discharge it is assumed that the sediment transport capacity is at its maximum because of the high flow velocities. So this situation is considered the most important. The maximum yearly sediment transport from the Rio Kert follows from:

S=34,1*3600*4=492.000 m^3/y.

Middle wadi (around the middle of the sandy beach)
The sediment load has been calculated according to the same methodology as for the Rio Kert basin: $s=m*u^n$ and $S=B*s$.

In which:

$m = 0.05/(\sqrt{(g)}*C^3*\Delta^2*D_{50}) = 0.05/(\sqrt{(9,81)}*50^3*1,65^2*(0,2*10^{-3})) = 2,3*10^{-4}$ [m]
$u = Q/(B*d) = 385/(80*1,8)= 2,7$ m/s
$n = 5$
B = 80 m. from [GOOGLE MAPS].
d = 1,8 m. (estimate from [ALKYON DATA], [GOOGLE MAPS])

This results in S=38.100 m^3/year.

Smallest wadi (around Punta Negri)
Again, the sediment load is calculated following the same methodology:
$s=m*u^n$ and $S=B*s$.

In which:

$m = 0.05/(\sqrt{(g)}*C^3*\Delta^2*D_{50}) = 0.05/(\sqrt{(9,81)}*50^3*1,65^2*(0,2*10^{-3})) = 2,3*10^{-4}$ [m]
$u = Q/(B*d) = 105/(35*1,2)= 2,5$ m/s
$n = 5$
B = 35 m. from [GOOGLE MAPS].
d = 1,2 m. (estimate from [ALKYON DATA], [GOOGLE MAPS])

This results in S=11.300 m^3/year.

3.2 Approach channel

Width
The different design parameters are determined from the following tables, directly taken from the preliminary guidelines for approach channel design of PIANC. The chosen values are elaborated in more detail below. Conveniently, all values are expressed in a factor times the width of the ship's beam. Besides this, the values for the design of the outer channel design are chosen from the tables, to ensure enough manoeuvring space even within the port area. It is emphasized that the differences are not that large compared to the inner channel design.

Table A3.1

Approach channel width additions according to PIANC

Ship Manoeuvrability	good	moderate	poor
Basic Manoeuvring Lane, W_{BM}	1.3 B	1.5 B	1.8 B

WIDTH w_i	Outer Channel exposed to open water	Inner Channel protected water
(a) Vessel speed (knots)		
- fast > 12	0.1 B	0.1 B
- moderate > 8 - 12	0.0	0.0
- slow 5 - 8	0.0	0.0

WIDTH w_i	Vessel Speed	Outer Channel exposed to open water	Inner Channel protected water
(b) Prevailing cross wind (knots)			
- mild ≤ 15 (≤ Beaufort 4)	all	0.0	0.0
- moderate > 15 - 33 (> Beaufort 4 - Beaufort 7)	fast	0.3 B	-
	mod	0.4 B	0.4 B
	slow	0.5 B	0.5 B
- severe > 33 - 48 (> Beaufort 7 - Beaufort 9)	fast	0.6 B	-
	mod	0.8 B	0.8 B
	slow	1.0 B	1.0 B
(c) Prevailing cross current (knots)			
- negligible < 0.2	all	0.0	0.0
- low 0.2 - 0.5	fast	0.1 B	-
	mod	0.2 B	0.1 B
	slow	0.3 B	0.2 B
- moderate > 0.5 - 1.5	fast	0.5 B	-
	mod	0.7 B	0.5 B
	slow	1.0 B	0.8 B
- strong > 1.5 - 2.0	fast	0.7 B	-
	mod	1.0 B	-
	slow	1.3 B	-
(d) Prevailing longitudinal current (knots)			
- low ≤ 1.5	all	0.0	0.0
- moderate > 1.5 - 3	fast	0.0	-
	mod	0.1 B	0.1 B
	slow	0.2 B	0.2 B
- strong > 3	fast	0.1 B	-
	mod	0.2 B	0.2 B
	slow	0.4 B	0.4 B
(e) Significant wave height H_s and length λ (m)			
- $H_s \leq 1$ and $\lambda \leq L$	all	0.0	0.0
- $3 > H_s > 1$ and $\lambda \approx L$	fast	≈ 2.0 B	
	mod	≈ 1.0 B	
	slow	≈ 0.5 B	
- $H_s > 3$ and $\lambda > L$	fast	≈ 3.0 B	
	mod	≈ 2.2 B	
	slow	≈ 1.5 B	

WIDTH w_i	Outer Channel exposed to open water	Inner Channel protected water
(f) Aids to Navigation		
- excellent with shore traffic control	0.0	0.0
- good	0.1 B	0.1 B
- average, visual and ship board, infrequent poor visibility	0.2 B	0.2 B
- average, visual and ship board, frequent poor visibility	≥ 0.5 B	≥ 0.5 B
(g) Bottom surface		
- if depth ≥ 1.5 T	0.0	0.0
- if depth < 1.5 T then		
- smooth and soft	0.1 B	0.1 B
- smooth or sloping and hard	0.1 B	0.1 B
- rough and hard	0.2 B	0.2 B
(h) Depth of waterway		
- ≥ 1.5 T	0.0	≥ 1.5 T 0.0
- 1.5 T - 1.25 T	0.1 B	< 1.5 T- 1.15 T 0.2 B
- < 1.25 T	0.2 B	< 1.15 T 0.4 B
(i) Cargo hazard level		
- low	0.0	0.0
- medium	≥ 0.5 B	≥ 0.4 B
- high	≥ 1.0 B	≥ 0.8 B

Table A3.1

Approach channel width additions according to PIANC (continued)

PASSING DISTANCE w_p	Outer Channel exposed to open water	Inner Channel protected water
Vessel speed (knots)		
- fast > 12	2.0 B	-
- moderate > 8 - 12	1.6 B	1.4 B
- slow 5 - 8	1.2 B	1.0 B
Encounter traffic density		
- light	0.0	0.0
- moderate	0.2 B	0.2 B
- heavy	0.5 B	0.4 B

WIDTH for BANK CLEARANCE (w_{Br} or w_{Bg})	Vessel Speed	Outer Channel exposed to open water	Inner Channel protected water
Sloping channel edges and shoals :			
	fast	0.7 B	-
	moderate	0.5 B	0.5 B
	slow	0.3 B	0.3 B
Steep and hard embankments, structures :			
	fast	1.3 B	-
	moderate	1.0 B	1.0 B
	slow	0.5 B	0.5 B

CATEGORY	CARGO
Low	Dry bulk, break bulk, containers, passengers, general freight, trailer freight
Medium	Oil in bulk
High	Aviation spirit, LPG, LNG, chemicals of all classes

CATEGORY	TRAFFIC DENSITY (vessel/hour)
Light	0 - 1.0
Moderate	> 1.0 - 3.0
Heavy	> 3.0

Basic width, W_{bm}

The factors for the basic manoeuvring width are presented by table A3.1. For the basic manoeuvring lane, a factor of 1.8B is chosen. This is because the large container and bulk vessels require tug assistance when proceeding through the approach channel and their manoeuvrability can be classified as poor to moderate. To be on the safe side for the preliminary design, the value for 'poor' is chosen, which results in a factor of 1.8B.

It is assumed that this is a reasonable estimate for the large design vessels which require tugs to manoeuvre in confined space. This is also because the manoeuvring characteristics of ships change noticeably in shallow water.

Additional widths, W_i

These are indicated in table A3.1.

First of all, the ship with the largest beam is chosen: the 14.000 TEU container vessel

(a) vessel speed: it is assumed (to be on the safe side, notwithstanding the fact that ships cannot sail that fast because of the tugs) that ships sail through the approach channel with at maximum a moderate speed of 8-12 knots = 4.1-6.2 m/s

This results in an additional width of 0.0B.

(b) prevailing cross wind: from the data it is clear that 1% of the time, the wind is larger than 33 knots = 17 m/s. Because of this, with a moderate speed, the additional width is 0.4B.

The probability of exceedance is less than 1%. When this occurs it is advised for the design ships to wait outside of the approach channel in the Mediterranean Sea. It has to be judged whether it is safe enough for smaller ships to enter the approach channel.
Besides this, the chance of occurrence of both the design ship entering, and the conditions begin unfavourable at that specific time is very small.

(c) & (d) prevailing currents: from the data (with tide, density and wind-generated currents) it is clear that the maximum current amounts to 0.16 m/s. Because of the orientation, this means actually a (rather low) longshore current. Because of this, the total additional required with for longshore and cross current is 0.0B.

(e) again analysing the data, it is obvious that 1% of the time the waves are higher than H_s=3 m. This results in an extra 1.0B. For the 1% of the time exceedance, two design ships can not enter in the approach channel at the same time, but it is safe to say that 1 still can. Besides this, unloaded ships sail on the right side of the approach channel which is situated close to the rocky Punta Negri. This results in an extra safety margin because these ships have a smaller draught. This is however not relevant for container ships, because they leave with the same draught. Optionally, it is advised to maintain a lower speed than 'moderate' (lower than 8 kn), which should reduce the required width of the approach channel with 0.5B.

(f) it is assumed that by modern vessels, making long intercontinental journeys, the aids to navigation are rather good. It is because of this that the additional width is only 0.1B.

(g) the depth of the channel in phase I & II is smaller than 1.5*D, and the bottom is smooth (covered with sand/rock) but rather hard. Because of this, the additional width here amounts to 0.1B.

(h) as mentioned before, the depth is smaller (at most points) than 1.25*D, so an additional 0.2B is added. This is also the appropriate criterion to be used for the inner channel.

(i) While the cargo hazard level for the design ship (container vessel) is low, this is not the case for the liquid bulk vessels. While the crude oil can be considered medium hazard (0.5B added), the oil products (most of the throughput) can be considered as high cargo hazard (1.0B added), according to table A3.1. For now, the calculation has been done without the added cargo hazard (containers are low cargo hazard, 0.0B added). Nevertheless, this has to be investigated later on.

Passing distance, W_p

When a two-way approach channel will be constructed, the passing distance between ships needs to be taken into account (because of the following chance of accidents, and the necessity for one lane to always be available). The required distance between two passing ships is which encounter at a moderate speed (with moderate traffic density, derived from number of ships is at maximum 1.1 ship/hour in Phase II+. It follows from table A.3.6 and A3.1 that 1.6B+0.2B=1.8B should be added.

Width for bank clearance, W_{br}/W_{bg}

The bottom consists of a channel with (originally present) a sloping edge of rock, covered by sand. With a moderate speed this gives an additional required width of 0.5B according to table A3.1.

With a beam of the container vessel of 56 m, this results for a one-way channel in a total of: W=4.1*B=230 m.

Next, the second largest vessel (but with increased cargo hazard) is chosen
The factors as above mentioned are the same, but the beam of a 200.000 DWT liquid bulk vessel (which transports crude oil) is somewhat smaller, 53 m. The cargo hazard is somewhat larger; this adds additional width to the total: increasing cargo hazard means an additional 0.5B (so 2*Wi for a two-lane channel).

With a beam of the crude oil vessel of 53 m, this results for a one-way channel in a total of: W=4.6*=244 m.

Finally, the third largest vessel (in beam, but with high cargo hazard) is chosen
The factors as above mentioned remain the same, but the beam of the 150.000 DWT liquid bulk vessel (which transports diesel) is somewhat smaller: 48 m. The cargo hazard is again larger; this adds additional width to the total: high cargo hazard with an additional 1.0B (so 2*Wi for a two-lane channel).

With a beam of the liquid bulk vessel of 48 m, this results for a one-way channel in a total of: W=5.1*B=245 m.

From the above it is clear that the width of the one-way approach channel is governed by the oil product tanker, and the two-way channel by the crude oil vessel.

The width for the two-way approach channel has been calculated for the case when 2 design vessels are using it. This may be not reasonable when considering Phase I, where two design container vessels would use the approach channel (because their chance of arriving at the same time is rather small). In the design, it is to be carefully judged if a two-way lane is absolutely necessary. At first glance, this is not the case, but it could still improve safety and efficiency of the port.

Squat calculations

The squat calculations are done analogue to [PIANC, 1995a] and [LIGTERINGEN, 2007], with two different methodologies. The largest vessels are used to determine the maximum squat. This differs for the container vessels and the bulk vessels, as they (could) use two different approach channels.

- The first method, according to [PIANC, 1995a] calculates squat via $s=2.4^* (\blacktriangledown/L_{pp}^{2})^* F_{nh}^{2}/\sqrt{(1-F_{nh}^{2})}$,
  and $F_{nh}=v/\sqrt{(gh)}$

  In which:

  F_{nh} = Froude Depth Number
  v = vessel speed [m/s]
  g = acceleration due to gravity = 9,81 m/s^2
  h = water depth [m]

▼ = volume of water displacement [m^3] = C_B*L_{PP}*B*D
L_{PP} = length of ship between perpendiculars [m]
B = vessel beam [m]
D = vessel draught [m]
C_B = block coefficient [-]

In the design of the approach channel, it has been assumed that vessels sail through the channel with a speed of at maximum 8 knots. This results in:

Table A3.2
Parameters and squat calculation according to [PIANC, 1995a]

Parameter	Container vessel	Liquid bulk vessel	Crude oil vessel
Vessel speed, v_s	8 kn = 4,2 m/s	8 kn = 4,2 m/s	8 kn = 4,2 m/s
Water depth, h	18,5 m	20,5 m	22 m
Froude Depth Number, F_{nh}	0,32	0,30	0,29
Block coefficient, C_B	0.6	0,8	0.8
Vessel length between pp, L_{PP}	378 m	290 m	318 m
Vessel beam, B	56 m	48 m	53 m
Vessel draught, D	15,5 m	17,4	18,9 m
Displacement, ▼	196.862 m^3	193.766 m^3	254.832 m^3
s_{max}	**0,36 m**	**0,52 m**	**0,54 m**

- The second method, according to [LIGTERINGEN, 2007] states that s=C_B/30* $S_2^{2/3}$*$v_s^{2.08}$

  In which:

  s = squat [m]
  v_s = vessel speed [kn]
  C_B = block coefficient [-]
  S_2 = S/(1-S) [-]
  S = blockage factor = A_S/A_{ch} [-]

  Vessels travel through the approach channel with a speed at maximum 8 kn. (as determined by the approach channel design). This value has been used as a first estimate in the following table.

Table A3.3
Parameters and squat calculation according to [LIGTERINGEN, 2007]

Parameter	Container vessel	Liquid bulk vessel	Crude oil vessel
Vessel speed, v_s	8 kn	8 kn	8 kn
Block coefficient, C_B	0.6	0.8	0,8
Cross-sectional area vessel, A_S	868 m^2	836 m^2	1002 m^2
Cross-sectional area channel, A_{ch}	4532,5 m^2	5023 m^2	5390 m^2
Blockage factor, S = A_S/A_{ch}	0,19	0,17	0,19
S_2 = S/(1-S)	0,24	0,21	0,23
s_{max}	**0,58 m**	**0,71 m**	**0,76 m**

From method 1 and 2 it is evident that the on experience based s_{max}=0,5 m. is too low, as it is exceeded in almost all the calculations. It appears that the estimated 0,7 m. from the PIANC graph is rather close to the calculated (maximum) value of 0,76 m. It is for vessels not advised to sail with a speed of 8 kn., as the available stopping length is limited (for the bulk vessels). The 0,7 m. can be considered as the extreme design situation, as vessels (are advised to) sail most of the time with a smaller speed than v_s= 8kn.

With a speed of v_s = 6 kn., the maximum squat for the liquid bulk vessel already reduces to s_{max} = 0.45 m. Nevertheless, the largest value will still be used in the design, to be on the safe side.

3.3: Tugboats

The types of tugs used are again determined by means of a market analysis and in accordance with the sources [PVE MAASVLAKTE II], [WEBSITE PORT OF ROTTERDAM], [HENSEN, 1997]. This has lead to two types of tugs. Their characteristics as well as the division of tugs per ship are summarized in the tables below.

Table A3.4

Tug characteristics

	bollard pull [t]	LOA [m]	B [m]	D [m]	speed [kn]	speed [m/s]
Z-peller tug	54	30	9,9	4,2	12,5	6,4
Schottel tug	71	35	11,2	4,9	14	7,2

Table A3.5

Division of tugs per vessel that uses them, with indicated bollard pull

	capacity	displace-ment [t]	required Tb [t]	Division Z-peller	Division Schottel	Available Tb [t]
Container terminal	1.500 TEU	44.000	67	2		108
	3.000 TEU	67.000	81	1	1	125
	6.000 TEU	106.000	104	2	1	179
	14.000 TEU	193.000	156	2	2	250
Oil terminal	60.000 DWT	79.000	88	1	1	125
	80.000 DWT	104.000	103		2	142
	100.000 DWT	129.000	118	3		162
	150.000 DWT	190.000	154		3	213
	200.000 DWT	250.000	190	2	2	250
Dry bulk terminal	60.000 DWT	72.550	84	2		108
	80.000 DWT	95.000	97	1	1	125
	150.000 DWT	173.000	144	2	1	179

From this, the calculations for the number of tugs required in the different phases have been done, and are presented below in table A3.6. It should be noted that in the table there is some reserve capacity taken into account: the tugs work on average with around 80% of their full capacity. At first sight, this reserve seems a conservative safe choice, but is seems necessary in case of accidents where tugs could make use of extra reserve capacity to get vessels out of dangerous situations [PIANC, 2000], [HENSEN, 1997].

It is not wise to dimension the required number of tugs on the total required capacity they can offer: there is no room for reserve in case of emergencies. For this reason, the total available bollard pull for the different ship sizes is around 25-30% larger than the required bollard pull (the higher value here amounts for the oil tankers, because of their cargo hazard). For the (larger) container vessels, as stated before, the criterion of difficulty with large vessels applies, and because of their high wind catchment area, the safety margin is larger because more tugs are applied.

Assumptions number of tugs needed

Primary assumptions: the time spent in the port for berthing and departure is stipulated at 2 hours (one for each manoeuvre). This means that there are per tug 24/2=12 movements (= mooring and unmooring) can be accomplished, based on a 100% occupancy.

This is far from realistic because tugs also have to sail to and from the ships and berths, which decreases this amount of movements per day considerably. Also taking into account the time for fuelling and changing shifts, the actual occupancy comes down to about 2/3 (66.67%) of that, which is only 8 movements per tug per day.

Besides this, the (considerable) possibility of failures in the steering machine or propulsion unit of the tugs and vessels during port transits cannot be neglected [UNCTAD, 1985b], [HENSEN, 1997]. This has been included in a safety margin (of 20% reserve) in case of failure of machinery.

The calculations have been done for all the 4 different phases, with their different specifications. This leads to substantial different requirements for harbour tugs, as outlined in the table below at the bottom line.

Table A3.6

Calculation sheet number of tugs required for the different phases

	capacity	displacement [t]	required Tb [t]	Division Z-peller	Division Schottel	Available Tb [t]
Container terminal	1.500 TEU	44.000	67	2		108
	3.000 TEU	67.000	81	1	1	125
	6.000 TEU	106.000	104	2	1	179
	14.000 TEU	193.000	156	2	2	250
Dry bulk terminal	60.000 DWT	72.550	84	2		108
	80.000 DWT	95.000	97	1	1	125
	150.000 DWT	173.000	144	2	1	179
Oil terminal	60.000 DWT	79.000	88	1	1	125
	80.000 DWT	104.000	103		2	142
	100.000 DWT	129.000	118	3		162
	150.000 DWT	190.000	154		3	213
	200.000 DWT	250.000	190	2	2	250

Phase I -	Z-pel.	Schot.	Total	Phase I +	Z-pel.	Schot.	Total	Phase II -	Z-pel.	Schot.	Total	Phase II +	Z-pel.	Schot.	Total
274 ships	2	0	**2**	410 ships	3	0	**3**	1.026 ships	6	0	**6**	2.051 ships	12	0	**12**
459 ships	2	2	**4**	689 ships	2	2	**4**	1.723 ships	5	5	**10**	3.446 ships	10	10	**20**
274 ships	2	1	**3**	410 ships	3	2	**5**	1.026 ships	6	3	**9**	2.051 ships	12	6	**18**
88 ships	1	1	**2**	131 ships	1	1	**2**	328 ships	2	2	**4**	656 ships	4	4	**8**
27 ships	1	0	**1**	33 ships	1	0	**1**	27 ships	1	0	**1**	33 ships	1	0	**1**
7 ships	1	1	**2**	9 ships	1	1	**2**	7 ships	1	1	**2**	9 ships	1	1	**2**
3 ships	1	1	**2**	3 ships	1	1	**2**	3 ships	1	1	**2**	3 ships	1	1	**2**
# tugmoves/day	10	6	**16**	# tugmoves/day	12	7	**19**	# tugmoves/day	22	12	**34**	# tugmoves/day	41	22	**63**
Possible # tug-movemens/day	8	8		Possible # tug-movemens/day	8	8		Possible # tug-movemens/day	8	8		Possible # tug-movemens/day	8	8	
Number of tugs/day:	2	1	**3**	Number of tugs/day:	2	1	**3**	Number of tugs/day:	3	2	**5**	Number of tugs/day:	6	3	**9**
Tug reserve	20%			Tug reserve	20%			Tug reserve	20%			Tug reserve	20%		
Container + Bulk tugs	**3**	**2**		Container + Bulk tugs	**3**	**2**		Container + Bulk tugs	**4**	**3**		Container + Bulk tugs	**8**	**4**	
Oil tugs required	**1**	**2**		Oil tugs required	**2**	**2**		Oil tugs required	**3**	**3**		Oil tugs required	**3**	**3**	
Tugs required:	**4**	**4**	**8**	**Tugs required:**	**5**	**4**	**9**	**Tugs required:**	**7**	**6**	**13**	**Tugs required:**	**11**	**7**	**18**

3.4 Basin Widths

Port quay basins may be placed in an unlimited number of ways. In conventional port design, there are two principal systems: the parallel quay system and the pier system [TSINKER, 1997]. A combination of both is also used (L=shaped or T-shaped piers). It is suggested that the basin length due to land traffic congestion should not exceed about 10 average ship lengths. However, the minimum length of a pier should be sufficient to provide berth for the longest ship expected to arrive. With respect to basin width, the following criteria are recommended for pier basins with two, three, and four berths [TSINKER, 1997]:

Two-berth basin:	$W=2B_{max}+30$ m
Three-berth basin:	$W=2B_{max}+40$ m
Four-berth basin:	$W=2B_{max}+50$ m

Here, B_{max} is the beam of the largest ship (m). This applies for short basins. For long basins, with quays at both sides, the following approach is used:

1. The width of the basin is determined as equal to the beams of the maximum-sized ships located at both sides of the basin with two rows of lighters on the outer side of each ship and a fairway twice the beam of the largest ship between moored ships
2. The average ship size located at both sides of the basin with one row of lighters on the outer side of each ship and a fairway four times the beam of the average ship, so that two average ships are able to pass one another.

This comes down to about: $W_B=6B_{max}+4*B_{tug,max}$
A simple calculation (e.g. for the container terminal) this gives: W=6*56+4*11,2=381 m
As opposed to this method, [LIGTERINGEN, 2007] states that for port basins a sufficient width for safe towing in and towing out of the vessels (with occupied berths) one must use:

$W=5B_{max}+100$
This would give for the container terminal: W=5*56+100=380 m.

It is obvious that both approaches can be used and that they yield more or less the same results. The latter can also be used in case of big tankers and bulk carriers in combinations with two-sided use of the basin (actually: W=4 to $6B_{max}$+100, where the lower value applies to favourable wind conditions, and the higher to frequent and strong winds. For safety reasons, the middle value is chosen (5B), because strong winds do not occur that frequently).

When the possibility needs to exist that ships can be turned in the basin (e.g. when the basin is longer than 1000 m), this would yield for the largest ships:

$W=L_{max}+B_{max}+50$, or $W=8B_{max}+50$.
This would give for the container terminal: W=398+56+50=504 m, or W=8*56+50=498 m.
The first would be a more severe criterion.

The values for the different required widths for the basins are calculated for several standard situations, and they are subsequently presented in the table below.

Table A3.7

Basin widths for various vessels and terminals
Left: required basin widths for double sided berthing (towing in and out)
Right: required basin widths for double sided berthing (exclusive berthed vessels) when vessels need to be turned in the basin

Basin Width [m]		Required 5B+100	6B+4Bt	Turn basin L+B+50	8B+50	Length diff.
Container Terminal	1.500 TEU	**250**	220	**305**	290	55
	3.000 TEU	**260**	237	**376**	306	116
	6.000 TEU	**315**	303	**403**	394	88
	14.000 TEU	380	**381**	**504**	498	123
Liquid Bulk Terminal	20.000 DWT	**230**	156	234	**258**	28
	40.000 DWT	**255**	186	277	**298**	43
	60.000 DWT	**275**	255	308	**330**	55
	80.000 DWT	**300**	285	335	**370**	70
	100.000 DWT	**315**	298	356	**394**	79
	150.000 DWT	**340**	333	396	**434**	94
	200.000 DWT	**365**	363	430	**474**	109
Dry Bulk Terminal	20.000 DWT	**220**	144	235	**242**	22
	40.000 DWT	**250**	180	275	**290**	40
	60.000 DWT	**260**	232	302	**306**	46
	80.000 DWT	**275**	255	323	**330**	55
	150.000 DWT	**325**	315	382	**410**	85

The above table gives far from all the possible combinations. Combinations such as where for example 6.000 TEU vessels can turn in a basin (403 m required), can be combined with the towing in and out of a 14.000 TEU vessel (381 m required). This is also beneficial for the costs, because widening of the basin for turning a 14.000 TEU vessel seems at first sight (in the first phase) not necessary.

The total required width for double-sided use of the basins is especially of importance when viewing phase II, where the larger ships (e.g. largest container vessels) arrive more frequent. This future development needs to be taken into account now already, because once determined and constructed, the wet layout of the port is very hard (and extremely costly) to modify later on. Space can in this stage be reserved for expansion in the future beforehand.

Besides this, the port layout has to satisfy two different requirements as far as wave penetration is concerned [LIGTERINGEN, 2007]: operational conditions must allow efficient loading and unloading of the ships at berth, and for limit state conditions the ship must be able to remain at berth safely.

This first requirement is often expressed in allowable ship motions and limiting wave heights. It will be checked later on (in the wave penetration study) if the design is successful in adequately creating calm in-port conditions. It is expected that from this wave penetration study it becomes clear if the design suffices and no harbour basin resonance will occur, although this is not expected because this problem manifests particularly along the borders of oceans, because of the long period swell [LIGTERINGEN, 2007], [PIETRZAK, 2008].

As a concluding remark, it should be noted that there need to be lay-bys (anchorages) available for ships awaiting change in weather conditions, or when queuing for service at the port [UNCTAD, 1985b]. Special anchorages available for ships carrying explosives or dangerous cargo are separately provided and such areas should be so designated on charts. They are usually located away from the marine terminal and adjacent to main channels so that they are near deep water, but clear of other ships movements.

Because the slope of the seabed consists mainly of rock following a steep slope, the approach channel already reaches considerable depth close to the land. Because of this availability of enough water depth, the areas alongside of the approach channel can all be used as proper anchorages.

3.5 Liquid bulk terminal

The considerations and calculations are elaborated in detail below.

3.5.1 Number of berths

For the different phases, the throughput and shipping traffic has been summarized in the table below (note that phase II^- = phase II^+):

Table A3.8

Liquid bulk vessels per year

Ships per year	Phase I -	Phase I +	Phase II -	Phase II +
Throughput	15.000.000 T	20.000.000 T	40.000.000 T	40.000.000 T
# of ships	303	403	700	700

From this, the required number of berths can be calculated from (when considering the fact that the oil tankers can all be unloaded with 10% of their deadweight tonnage per hour):

$n=N_v/(360*m_i)$

In which:

n= number of berths required [-]
N_v = number of vessels per year [-]
m_i = desired occupancy of the berth [-]

The desired occupancy m_i, determined from [UNCTAD, 1985b], differs from phase to phase and depends on the number of berths. This can be calculated iteratively. It should be noted that that different types of handling equipment are required for unloading crude oil and oil products (and even possibly between different oil products) [UNCTAD, 1985b]. While in phase I only oil products are transhipped, it is assumed that the berth can handle al types of oil products.

In phase II however, there is a separate berth needed for the incoming crude oil. Per year, 85 ships arrive at the liquid bulk terminal unloading crude oil. With one berth, the occupancy would be 0.24, which is a decent value (often for one berth an occupancy of at maximum 0.35 is demanded [LIGTERINGEN, 2007]). In reality, actually this is even lower: ships do not occupy the berth for one whole day, but roughly only half of it. The remaining ships for incoming and outgoing oil products (700-85=615 ships), require 4 berths, which would result in a maximum occupancy of 0.43.

Again, in reality this means a lower occupancy because ships do not occupy the berth for a whole day (10% of DWT unloading capacity amounts to 10 hours).

The results are presented in the next table:

Table A3.9

Number of berths and occupancy liquid bulk terminal

# of berths	Phase I -	Phase I +	Phase II -	Phase II +
Throughput	15.000.000 T	20.000.000 T	40.000.000 T	40.000.000 T
Maximum m_i	0,50	0,55	0,60	0,60
# of oil product berths	2	3	4	4
# of crude oil berths	-	-	1	1

3.5.2 Storage areas

For the storage of oil products and crude oil different tanks are to be used. [UNCTAD, 1985b] states that the products can be split into two different categories, which require different storage:

1 Black Oils, under which the crude oil

2 White Oils, under which the gasoline, diesel etc.

The storage requirements depend on the operational and strategic storage requirements. When taking into account an operational storage in the order of 1 month (5% of total throughput), the total required storage for the different phases becomes:

Table A3.10

Required liquid bulk storage

Storage required	Phase I -	Phase I +	Phase II -	Phase II +
Throughput	15.000.000 T	20.000.000 T	40.000.000 T	40.000.000 T
Oil products	750.000 T	1.000.000 T	1.375.000 T	1.375.000 T
Crude oil	-	-	625.000 T	625.000 T

These storage requirements in tons can be translated to area requirements. The total required area follows from the necessity that the contents of the tanks in case of an accident (breaking of the tanks) could be maintained within a bund [LIGTERINGEN, 2007]. For oil products, an averaged density of 0.75 T/m^3 is used, and for crude oil 0.85 T/m^3 [WIKIPEDIA]. Further, a height of 5 meters is assumed for the bund (of which 4 m effective). Dividing the required storage in cubic meters by this height gives the required storage area.

The calculation sheet is presented on the next page. This has resulted in the area requirements displayed on the bottom row, and summarized in the table below.

Table A3.11

Summary calculations liquid bulk terminal

Storage required	Phase I -	Phase I +	Phase II -	Phase II +
Throughput:				
Oil products	15.000.000 T	20.000.000 T	27.500.000 T	27.500.000 T
Crude oil	-	-	12.500.000 T	12.500.000 T
Storage:				
Oil products [T]	750.000	1.000.000	1.375.000	1.375.000
Crude oil [T]	-	-	625.000	625.000
Oil products [m^3]	1.072.000	1.429.000	1.965.000	1.965.000
Crude oil [m^3]	-	-	736.000	736.000
Storage area [ha]	25	34	65	65
A_{LB} [ha]	**35**	**48**	**200-300***	**200-300***

* requirement from the client

In which:

A_{LB}= total surface area required for the inland liquid bulk terminal

Table A3.12

Calculation sheet surface area requirements liquid bulk terminal

Storage Oil Products – Incoming	Phase I -	Phase I +	Phase II -	Phase II +	Outgoing	Phase I -	Phase I +	Phase II -	Phase II +
Throughput	7.500.000	10.000.000	7.500.000	7.500.000		7.500.000	10.000.000	20.000.000	20.000.000
Storage	5,00%	5,00%	5,00%	5,00%		5,00%	5,00%	5,00%	5,00%
Storage [T]	375.000	500.000	375.000	375.000		375.000	500.000	1.000.000	1.000.000

Storage Crude Oil – Incoming	Phase I -	Phase I +	Phase II -	Phase II +	Outgoing	Phase I -	Phase I +	Phase II -	Phase II +
Throughput			12.500.000	12.500.000					
Storage			5,00%	5,00%					
Storage [T]			625.000	625.000					

Oil Products	750.000	1.000.000	1.375.000	1.375.000
Crude Oil			625.000	625.000
Total Storage [T]	750.000	1.000.000	2.000.000	2.000.000

	Oil Products	Crude Oil
Tank Size [m3]	100.000	100.000
Per tank needed [m2]	50.000	25.000
Average Density	0,75	0,85

	Phase I -	Phase I +	Phase II -	Phase II +
Storage [m3]				
Oil products [m3]	1.000.000	1.333.340	1.833.340	1.833.340
Crude oil [m3]	0	0	735.300	735.300
Bund height	4			
Oil products	250.000	333.335	458.335	458.335
Crude oil			183.825	183.825
Surface area [m2]	250.000	333.335	642.160	642.160
Surface area [ha]	25	34	65	65

Total storage area [ha]	25	34	65	65	
LxW [m]	500	584	807	807	
Refinery [ha]			20	20	included in demands client
Total surface area [ha]	25	34	85	85	
LxW [m]	500	584	922	922	
Factor for slope + buidlings	1,4				
Total surface area [ha]	35	48	119	119	
			200-300	200-300	requested by client
Resulting surface area [ha]	35	48	200-300	200-300	

3.6 Dry bulk terminal

The calculations and considerations regarding the dry bulk terminal are elaborated in more detail below.

3.6.1 Number of berths

In combination with the shipping traffic, the number of berths for the dry bulk terminal is calculated for the different phases with the following the formula:

$$n=N_v/(360*m_i)$$

Results are again summarized in the following table.

Table A3.13

Number of berths dry bulk terminal

# of berths	Phase I -	Phase I +	Phase II -	Phase II +
Throughput	4.000.000 T	5.000.000 T	4.000.000 T	5.000.000 T
# of ships/yr	81	101	81	101
Maximum m_i	0,40	0,40	0,40	0,40
# of berths	2	2	2	2

An (low) occupancy of 0.4 (for each individual berth) has been chosen here because of the fact that the incoming cargo as well as the outgoing cargo berth must be treated separately considering their traffic. The maximum real occupancy for the incoming cargo berth will be: 40/360=0.11, and for the outgoing cargo berth: 0.17*1.6=0.41.

3.6.2 Surface area requirements

Here, it is assumed that around (a rather large value of) 10% of the total throughput can be stored on site. This means:

Table A3.14

Required dry bulk storage

Storage required	Phase I -	Phase I +	Phase II -	Phase II +
Throughput	4.000.000 T	5.000.000 T	4.000.000 T	5.000.000 T
Dry bulk	400.000 T	500.000 T	400.000 T	500.000 T

A light material would require the largest amount of storage space, and can be considered as the design situation. From the client, an estimate is made available of around 250-300 m. [ALKYON DATA] land inward (seen from the quay) for the storage of dry bulk.

Taking into account apron areas (apron depth Y=85 m [LIGTERINGEN, 2007], required over the entire quay of 700 m in length) and a factor of 1.2 for additional space requirements, the final surface area for the bulk terminal can be calculated. The calculation sheet is presented below.

Table A3.15

Calculation sheet dry bulk terminal surface area requirements

Storage bulk Products				
Throughput				
	Phase I -	**Phase I +**	**Phase II -**	**Phase II +**
Throughput	4.000.000	5.000.000	4.000.000	5.000.000
storage [%]	10,00%	10,00%	10,00%	10,00%
storage [T]	400.000	500.000	400.000	500.000
terminal length [m]	700	700	700	700
terminal depth [m]	250	300	250	300
storage area O [ha]	17,5	21	17,5	21
Apron areas				
Apron depth Y =	85			
nr of berths, n=	2			
max ship length=	350			
Total Apron area [ha]	6			
Terminal surface	23,5	27	23,5	27
factor included	1,2			
Total surface area [ha	**29**	**33**	**29**	**33**
LxW [m]	539	575	539	575

And, summarized in the next table:

Table A3.16

Summary dry bulk surface areas

Storage required	Phase I -	Phase I +	Phase II -	Phase II +
Throughput	4.000.000 T	5.000.000 T	4.000.000 T	5.000.000 T
Dry bulk	400.000 T	500.000 T	400.000 T	500.000 T
A_S [ha]	17,5	21	17,5	21
A_{AA} [ha]	6	6	6	6
A_{DB} [ha]	**29**	**33**	**29**	**33**

In which:

A_S= storage area dry bulk terminal [ha]
A_{AA} = apron area [ha]
A_{DB}= surface area dry bulk terminal (included factor of 1,2) [ha]

3.7 Container terminal

For the preliminary design study for the port layout, the method for area estimation is used according to [UNCTAD, 1985b] [LIGTERINGEN, 2007].

3.7.1 Number of berths

The number of berths can be calculated from:

$n= C_s/c_b$

In which:

C_s = total number of TEU entering and leaving the terminal by seagoing vessels (including empties),
c_b = average annual number of TEU per berth [TEU/yr]

and $c_b=p*f*N_b*t_n*m_b$

In which:

p = gross production per crane [moves/hr]
f = TEU – factor [-]
N_b = number of cranes per berth [-]
t_n = number of operational hours per year [hrs/yr]
m_b = berth occupancy factor [-]

The gross (effective) productivity per crane (p) is assumed to be 25, because the new terminal will be equipped with modern portainer cranes for handling, suitable to function at a modern transshipment port (but still, taking into account reserves and time losses between shifts, repairs, etc).

The TEU – factor is defined as: $f = (N_{20'}+2*N_{40'})/N_{tot}$. [LIGTERINGEN, 2007] mentions that in developing countries a lower TEU – factor is encountered, while the main line traffic shows a shift towards 40 ft containers. While this last argument is by far the most important one (because Nador will be on the main trade routes), it has to be taken into account that the region of Africa can be seen as a developing one. It is because of this that the TEU – factor is assumed to be 1.5. This means that 50% of the containers are 40 ft containers.

The number of cranes per berth, N_b, depends on the ship size. The division of the number of cranes that can be used per ship is displayed in the table below (assuming 75-80 m ship length working area per crane):

Table A3.17

Number of portainer cranes per vessel

# Portainer cranes	L_s	N_b
1.500 TEU vessel	225	3
3.000 TEU vessel	294	4
6.000 TEU vessel	310	4
14.000 TEU vessel	398	5

On average this means that there are 4 portainer cranes per ship at work (which also can be calculated taking into account the division of the vessels: actually 3.8 on average).

The number of operational hours per year, t_n, is (as used for the throughput calculations):
t_n= 24 * 360=8640 hours per year.

Berth occupancy depends on the number of berths that is available at the terminal. Main lines have relatively tight schedules on their fixed routes, which makes the assumption of random arrivals somewhat conservative [LIGTERINGEN, 2007]. It is because of this that the berth occupancy can be somewhat higher than the often proposed value of 0.35.
These two arguments result in the assumption for the berth occupancy of 0.5.

This yields:
c_b=25*1.5*4*8640*0.5=648.000 TEU per berth per year.

In practice, several modern hub terminals with a throughput of around 500.000 TEU per berth per year exist, although the above calculated value is rather high [LIGTERINGEN, 2007]. This is because the larger average ship- and call sizes result in more cranes operating per ship, in combination with a TEU – factor of 1.5. Still, to be on the safe side this value will not be used, and by taking into account 9-10% reserve, it is assumed that per berth at max 600.000 TEU per year can be handled. This means that the required number of berths for the different phases are (calculated from n=C_s/600.000):

Table A3.18

Number of berth container terminal

	Phase I -	Phase I +	Phase II -	Phase II +
Throughput	4.000.000	6.000.000	15.000.000	30.000.000
# of berths	7	10	25	50

3.7.2 Surface areas and handling methods

For storage, a substantial amount of surface is needed, divided into separate stacks for outgoing containers, incoming containers, reefers, hazardous cargo, empties and a (limited, because it is a transshipment port,) Container Freight Station (CFS). The CFS will be treated separately later on. The required surface area for the other different stacks can be calculated as follows, and depends on the stacking height and the handling method used via ‘F’ and ‘r’:

$A_{SY}=(C_i*t_d*F)/(r*360*m_i)$

In which:

A_{SY} = storage yard area required [m^2]
C_i = number of containers movements per year per type of stack in TEU’s
t_d = average dwell time [days], which differs for incoming, outgoing and empty containers
F = required area per TEU, equipment travelling lanes inclusive [m^2]

r = average stacking height / nominal stacking height (0.6 – 0.8)
m_i = acceptable average occupancy rate (0.65 – 0.7)
360 = the number of working days per year.

While the new modern hub port will accommodate a modern transshipment container terminal, it is assumed that besides the portainer cranes which unload the containers, Automated Guided Vehicles (AGV's) are used to transport the containers to the storage yard. Within the storage yard, gantry cranes will be used to stack the incoming containers. With this decision for the handling equipment, the parameters F and r are more or less determined: in accordance with [UNCTAD, 1985b], [LIGTERINGEN, 2007]:

F, the required area per TEU amounts to 8.5 m^2/TEU. This (rather low) value is deducted from the fact that gantry cranes with large spans are used which can stack containers more efficiently in terms of space utilisation, which is needed because of the scarceness of land. On the other hand, this requires good planning of containers which remain at the yard for a while or should be transported further immediately, to minimize repositioning.

The factor r (between 0.6-0.9) is chosen to be 0.75, which means that the average stacking height approaches the nominal stacking height of 4, in order to make good use of the space of the available land. Still, taking into account an increase in repositioning because of an increase in stacking height, the average stacking height will be 3 containers high.

m_i , or the acceptable average occupancy rate lies between 0.65 – 0.70 [LIGTERINGEN, 2007]. Because of the transshipment aspect of the port, containers arrive in a rather large batch and there is considerable variation in the arriving number of containers per ship. It is because of this that the factor m_i will be at the lower boundary, so: m_i = 0.65.

The dwell time t_d differs for incoming, outgoing and empty containers. It is not good policy to encourage long storage of containers at ports with relatively limited open storage space. It is because of this that the dwell times for storage should be minimized. In order not to underestimate the time containers stay at the yard, the dwell time for containers will be, according to [UNCTAD, 1985b], [LIGTERINGEN, 2007]*:* 10 days for incoming containers, 7 days for outgoing containers and 6 days in the CFS.

For the different phases the required surface areas are calculated and summarized in the tables below. Also the surface area for the CFS will be included, which follows the equation:

$A_{CFS}=(C_i*V*t_d*f_1*f_2)/(h_a*m_i*360)$

In which:

C_i = number of TEU moved through CFS, ≈ 15% of throughput [TEU/yr], assumption
V = contents of 1 TEU container = 29 m^3
f_1 = gross area / net area, ≈ 1.4 [-]
f_2 = bulking factor, ≈ 1.15 [-]
h_a = average height of cargo in the CFS, = 3 [m]
m_i = acceptable occupancy rate, = 0.65 [-]

Besides the storage yard and CFS, there is another surface demand which takes considerable space displayed in the table: the apron area. This is the area in which the portainer cranes are situated, and the equipment for picking up the containers for transportation to the storage yard.

The surface area requirements are:

- A service lane between the front crane rail and the quay, here: 5 m,
- When 5 portainer cranes are working on one ship, each has transport equipment lining up in their own lane for reasons of safety. This is the case here, so 5 lanes amount to (container width is 2.5 m, lane width of 4 m, extra lane(s), so rounded off to) 30 m,
- A back reach area of 15 m. for the portainer cranes,
- When AGV's are used, this means a lane width required of the width equal to the crane track, which is 30 m.

Total per berth this means that an area inland is required of:
Y=5+30+15+30=80m.

The total apron area per phase is calculated by A_{AA}=Y*L_q, and is per phase summarized below:

Table A3.19

Number of container terminal berths and apron areas

	Phase I -	Phase I +	Phase II -	Phase II +
# of berths	7	10	25	50
Quay length L_q	2.400 m	3.400 m	8.500 m	16.800 m
A_{AA} [ha]	20	28	68	135

The total required area for the container terminal is the cumulated total of the above values (for storage, CFS and apron areas), and taking into account a factor of around 1.15 (for marshalling areas, vehicles parking, rail and road access, reefers, staff, administration and dangerous goods). This factor has been determined after analyzing various terminals (e.g. ECT Rotterdam). [UNCTAD, 1985b] recommends around 30.000 m2 per berth, which is around a factor of 1.1. This seems in agreement with the earlier mentioned value of 1.15. For further clarification of all the specific areas and items, the calculation sheet is presented on the next page.

The total required surface area for the container terminal is summarized in the table below.

Table A3.20

Total surface area container terminal

Total Area	Phase I -	Phase I +	Phase II -	Phase II +
Throughput	4.000.000 TEU	6.000.000 TEU	15.000.000 TEU	30.000.000 TEU
O_{CT} [ha]	**130**	**192**	**478**	**951**

The throughput –terminal area ratio is around 31.000-32.000 TEU/ha, which is considered a very good value compared to other major ports [LIGTERINGEN, 2007].

It should be noted that by the above calculated terminal area, the gate area is not yet included. In a first approximation, a factor of 1,2 can be used for these additional surface requirements.

Table A3.21

Calculation sheet container terminal

	Throughput	Throughput	Throughput	Throughput
	4.000.000	6.000.000	15.000.000	30.000.000

Storage Yard

Incoming	Phase I -	Phase I +	Phase II -	Phase II +	Outgoing	Phase I -	Phase I +	Phase II -	Phase II +
Ci	1.000.000	1.500.000	3.750.000	7.500.000	Ci	1.000.000	1.500.000	3.750.000	7.500.000
td	10	10	10	10	td	7	7	7	7
F	8,50	8,50	8,50	8,50	F	8,50	8,50	8,50	8,50
r	0,75	0,75	0,75	0,75	r	0,75	0,75	0,75	0,75
days/yr	360	360	360	360	days/yr	360	360	360	360
mi	0,65	0,65	0,65	0,65	mi	0,65	0,65	0,65	0,65
Asy [m2]	484.330,48	726.495,73	1.816.239,32	3.632.478,63	Asy [m2]	339.031,34	508.547,01	1.271.367,52	2.542.735,04
Asy [ha]	**49**	**73**	**182**	**364**	**Asy [ha]**	**34**	**51**	**128**	**255**

CFS

Incoming	Phase I -	Phase I +	Phase II -	Phase II +	Outgoing	Phase I -	Phase I +	Phase II -	Phase II +
Ci	1.000.000	1.500.000	3.750.000	7.500.000	Ci	1.000.000	1.500.000	3.750.000	7.500.000
[% of Ci]	15%	15%	15%	15%	[% of Ci]	15%	15%	15%	15%
V	29	29	29	29	V	29	29	29	29
td	6	6	6	6	td	6	6	6	6
f1	1,4	1,4	1,4	1,4	f1	1,4	1,4	1,4	1,4
f2	1,15	1,15	1,15	1,15	f2	1,15	1,15	1,15	1,15
ha	3	3	3	3	ha	3	3	3	3
mi	0,65	0,65	0,65	0,65	mi	0,65	0,65	0,65	0,65
days/yr	360	360	360	360	days/yr	360	360	360	360
Acfs [m2]	59.858,97	89.788,46	224.471,15	448.942,31	Acfs [m2]	59.858,97	89.788,46	224.471,15	448.942,31
Acfs [ha]	**6**	**9**	**23**	**45**	**Acfs [ha]**	**6**	**9**	**23**	**45**

Atotal [m2]	544.189,46	816.284,19	2.040.710,47	4.081.420,94	Atotal [m2]	398.890,31	598.335,47	1.495.838,68	2.991.677,35
Atotal [ha]	**55**	**82**	**205**	**409**	**Atotal [ha]**	**40**	**60**	**151**	**300**

Total Storage Area + CFS

	Phase I -	Phase I +	Phase II -	Phase II +
As in+out [ha]	83	124	310	619
LxB [m]	912	1.114	1.761	2.488
Acfs in+out [ha]	12	18	46	90
LxB [m]	347	425	679	949
As+Acfs [ha]	**95**	**142**	**356**	**709**
LxW [m]	975	1.192	1.887	2.663
incl. factor of 1,15	**110**	**164**	**410**	**816**

Apron Area

	Phase I -	Phase I +	Phase II -	Phase II +	
Apron depth Y [m]	80	80	80	80	determined from *[5]*
# of berths	7	10	25	50	
Ls,average	290	290	290	290	
Lquay	2.400	3.400	8.500	16.800	rounded off to +100 m.
Aaa [ha]	**20**	**28**	**68**	**135**	

Act,total [ha]	**130**	**192**	**478**	**951**
LxW [m]	1.141	1.386	2.187	3.084
# TEU/ha	30.769	31.250	31.381	31.546
# TEU/m quay	1.667	1.765	1.765	1.786

3.8 Multi Criteria Analysis

The assignment of scores to the alternatives is explained here for each criterion.

3.8.1 Round 1

- Channel alignment

A1: small angle with (almost) all dominant directions, stopping length available, and channel can be located offshore (parallel to shore) to minimize dredging, → score: 1, 0

A2: environmental forces perpendicular to channel, because of required stopping length much dredging needed → score: -1, -1

A3: small angle with dominant directions although not with NW, approach perpendicular to shore requires the most dredging → score: 0, -1

A4: small angle with dominant directions but almost beaming waves from NW, much space available for stopping length → score: 0, 1

- Nautical ease

A1: when entering port sailing against dominant direction, facilitates ship control and decreases stopping length, entrance easy for ships from east but somewhat less from west, much space at in-port entrance → score: 1, 0, 1
A2: sailing perpendicular to environmental forces (wind + waves), rolling becomes critical, easy entrance from any direction of approach, much space behind breakwaters for maneuvering → score: -1, 1, 1
A3: sailing under small angle with dominant waves, but following waves do not facilitate ship maneuvering, easy access for ships from the west, somewhat less for vessels from the east, neutral amount of space behind breakwaters → score: 0, 0, 0
A4: angle with environmental forces, but waves from NW could pose rolling problems, entrance located unfavorable for ships from east, neutral amount of space behind breakwaters → score: 0, -1, 0

- Port zoning & efficiency

A1: allocation terminals according to guidelines, bulk north, containers south, efficient for bulk but less for far away container berths → score: 1, 0
A2: allocation terminals according to guidelines, efficient for bulk vessels, but less for container berths far away → score: 1,0
A3: allocation terminals more or less according to guidelines, turning basin in middle of the port, more favorable but bulk situated far from entrance → score: 1, 0
A4: allocation terminals according guidelines, turning basins somewhat more central, but still far away from bulk, and bulk located far from entrance → score: 1, -1

- Waves & sedimentation

A1: left breakwater is longer to avoid waves from dominant direction to enter, but NE waves can still enter, sedimentation because of longshore transport from SW to NE not possible → score: 0, 1
A2: only waves from the north can enter, but these are low in occurrence and height, small entrance and practically no sedimentation because of longshore transport → score: 1, 1
A3: waves from a semi-dominant direction (W-WNW) can enter the port and reach the berths, not much sedimentation expected, left breakwater is longer → score: -1, 1
A4: waves from dominant direction can enter (W and WNW), aimed on minimizing sedimentation, but this leads to a large entrance width → score: -1, 0

3.8.2 Round 2

- Costs

A1-0: compared to the other alternatives, the breakwater has a medium length and reaches around CD -40 m. Somewhat more land reclamation is required, the channel is located somehwat offshore to minimize dredging of hard soil → score: 0, 0, 1
A1-1A: long breakwater length at reasonable depth, very uneven cut & fill balance, too much reclamation required, channel requires some dredging of hard soils, but not much → score: -1, -1, 0
A1-1B: breakwater less deep but still long, better cut and fill balance but much dredging oil berths, channel requires some dredging of hard soil → score: 0, 0, -1

A1-2: breakwater not that long and at relatively small depth, reasonable cut and fill balance, channel located so that no dredging is required → score: 1, 1, 1
A1-3A: breakwater not that long, but at medium depth, somewhat uneven cut & fill balance, both channels oriented tot minimize dredging (of hard soils), → score: 0, 0, 1
A1-3B: breakwater again not that long and medium depth, more even cut and fill balance, both channels oriented to minimize dredging (of hard soils), → score: 0, 1, 1

- Nautical Ease

A1-0: one way channel that ends in a spacious situated turning basin, some congestion possible at container terminals, but reasonable spacious layout with 2 possibilities for turning → score: 0, 0, 1
A1-1A: one way channel which ends in a spacious situated turning basin, congestion could become critical at container terminals, not that much turning space → score: 0, -1, -1
A1-1B: one way channel ending in spacious turning basin, much space for maneuvering and turning, possible congestion at some points → score: 0, 0, 1
A1-2: one way channel which ends in a spacious situated turning basin, some congestion possible for terminals situated further away, reasonable maneuvering space → score: 0, 0, 0
A1-3A: two one way channels, both ending in turning basins located in the middle of port activities, less chance on congestion because separation in traffic, average maneuvering space → score: 1, 0, 0
A1-3B: two one way channels, both ending in a favorably located turning basin, somewhat less chance on congestion due to somewhat more maneuvering space as A1-3A →
score: 1, 0, 1

- Construction Phasing:

A1-0: no unambiguous expansion of breakwater possible, but for the container terminals this can be done, independent construction of bulk and container ports is not very easy possible, but a breakwater could be placed/removed → score: 0, 0, 0
A1-1A: also no clear expansion possibilities in phases, although container terminals could be included in the phase I breakwater, independent development not easy possible →
score: 0, 1, 0
A1-1B: small lengthening of breakwater possible, several container terminals can be included, small breakwater part needs to be removed, no easy independent development of bulk and container terminals → score: 0, 1, 0
A1-2: separate breakwater necessary which should be removed for expansion, terminals can be expanded, but not that easily, independent development not easy possible →
score: 0, 0, -1
A1-3A: breakwater construction for independent container terminal port, lengthening to the northeast possible but also not that easy, independent development completely possible →
score: 0, 0, 1
A1-3B: breakwater construction for independent container terminal port, expansion also not particularly easy, but in phase container construction is independently possible →
score: 0, 1, 1

- Port Zoning & location

A1-0: standard layout of terminals, different terminals located logically (bulk to the northeast, containers to the south) → score: 0, 0
A1-1A: average terminal layout, different terminals located according to guidelines, but oil berths against breakwater → score: 0, -1

A1-1B: terminals and port items on favorable location, efficient layout → score: 1, 0
A1-2: very compact layout, terminals on logically chosen location → score: 1, 0
A1-3A: efficient layout, division of traffic, terminals located conveniently → score: 1, 0
A1-3B: efficient layout, division of traffic, terminals located conveniently → score: 1, 0

- Port Safety

A1-0: liquid bulk berths located around other terminals, vessels can get relatively quick out of the port in case of accidents → score: -1, 0
A1-1A: liquid bulk berths at inside of breakwater, away from other traffic, but along approach channel, container vessels deep in-port can not leave the port that fast, but bulk vessels can → score: 0, 0
A1-1B: liquid bulk berths located around other terminals, vessels can get relatively quick out of the port in case of accidents, although somewhat problematic deep in-port → score: -1, 0
A1-2: liquid bulk berths amongst other terminals, vessels can leave port relatively quick → score: -1, 0
A1-3A: liquid bulk berths clearly separated from largest shipping traffic, can leave port relatively easy → score: 0, 1
A1-3B: liquid bulk berths clearly separated from largest shipping traffic, can leave port relatively easy → score: 0, 1

- Expansion Possibilities

A1-0: for phase I, the required throughput is easily reached, for phase II around 18 MTEU, which is average → score: 1, 0
A1-1A: for phase I, the required throughput is easily reached, for phase II around 28,8 MTEU, which is high → score: 1, 1
A1-1B: for phase I, the required throughput is easily reached, for phase II around 30 MTEU, which is high → score: 1, 1
A1-2: for phase I, the required throughput is easily reached, for phase II around 15 MTEU, which is low, but still acceptable → score: 1, -1
A1-3A: for phase I, the required throughput is easily reached, for phase II around 16,8 MTEU, which is rather low → score: 1, -1
A1-3B: for phase I, the required throughput is easily reached, for phase II around 21 MTEU, which is average → score: 1, 0

3.9 Downtime analysis

The factors for the downtime analysis are described below, in accordance with [THORESON, 2003].

Navigation

1. Ice problems
These are not to be expected in the Mediterranean Sea during average yearly conditions, so the downtime because of this is 0.
2. Excessive currents
In chapter 3 it was already deduced that currents (even in combination with strong winds and density differences) around the project location are negligible, let alone excessive currents.

3. Wind speed [navigational]

According to [THORESON, 2003], wind speeds higher than 7 times the vessel speed (4.5 kn) make the vessel maneuvering more difficult. This is around 16 m/s. The probability (according to the project data from chapter 3) that wind speeds are higher than 16 m/s amounts to 0,85%.

4. Wave height

For the bulk port, wave heights above H_s>2.0 m make it impossible for vessels to enter, as tugs can not tie up outside the bulk port. This happens 7,1% of the time at P60. For the container port, it is assumed that with waves higher than H_s>4.0 m, vessels will not enter. Because of diffraction, the wave height behind the breakwaters is assumed to decrease to 0.5 times the incoming wave height (H=2.0 m.). This is exceeded in P60 around 0.5% of time.

5. Swell & long period waves

Swell is not expected to pose any problems in the Mediterranean Sea, and long period waves have been included in water level variations, determined from LAT. There will be no downtime because of this.

6. Visibility

As a general rule, most oil and gas terminals will close for arrival and berthing or unberthing and departure of tankers if the visibility is less than 1000-2000 m. This value has been adopted from [THORESON, 2003].

7. Tugboat non-availability

For the bulk port it is stressed that there should always be enough tugs available (also included in a safety margin for the number of present tugs), so these berths tug preference and thus 'tug-certainty'. For the container terminal this somewhat differs, and a value from [THORESON, 2003] has been adopted.

Operation

8. Wind speed [operational]

With a wind speed higher than 10 m/s, [THORESON, 2003] states that crane operation could become critical. From the environmental data it is clear that this happens around 0,04% of the time.

9. Excessive ship movements

These values have been deduced from the amount of time that wave heights exceed around H_s>4.0 m. It is expected that these higher waves will influence the berthed vessels because they have not enough decreased in height when arriving at the berth. This happens for 0.5% of time.

10. Maintenance on the berth

These values have been adopted from [THORESON, 2003], and includes the downtime because of for example repairs, and crane unavailability.

3.10 Wave analysis bulk port

In the table below, the relevant combined wave heights and their periods are presented for calculation point P60.

Hs (m)		wave direction (Deg)													
		-15 to 15		15 to 45		45 to 75		225 to 255		255 to 285		285 to 315		315 to 345	
Lower	Upper	Hs [%]	Tp [s]	Hs [%]	Tp [s]	Hs [%]	Tp [s]	Hs [%]	Tp [s]	Hs [%]	Tp [s]	Hs [%]	Tp [s]	Hs [%]	Tp [s]
1,25	1,75	0,47	**7,5**	1,50	**8,6**	0,01	**5,2**	0,00	**8,0**	6,92	**7,4**	1,83	**7,4**	0,23	**7,0**
1,75	2,25	0,18	**8,6**	0,55	9,2	0,00	-	0,00	**7,6**	2,79	8,2	1,01	8,4	0,14	7,7
2,25	2,75	0,10	8,5	0,22	**9,9**	0,00	-	0,00	-	1,45	**8,7**	0,48	**8,8**	0,06	**9,8**

These parameters are used as input for the wave parameters, calculated in the next table.

In the table below, also the variation in depth has been considered: at the bulk port entrance, the water depth it not exactly CD -40m. as is the case for calculation point P60. Here, the depth is only CD -35 m. The differences are however relatively small, and do certainly not lead to a difference in the B/L-ratio (as can be seen at the bottom of the table).
The calculated values at a depth of CD -35 m. are however somewhat larger, so these values are critical and will be used in further calculations. The directional spreading is maintained the same for both depths.

		Hs							
	L0 [m]	1,25 - 1,75	**87,8**	**115,5**	**42,2**	**99,9**	**85,5**	**85,5**	**76,5**
		1,75 - 2,25	**115,5**	132,1		**90,2**	105,0	110,2	92,6
		2,25 - 275	112,8	**153,0**			**118,2**	**120,9**	**149,9**

		Hs	TW	TW	DW	TW	TW	TW	TW / DW
d= 35 m	**d/L0**	1,25 - 1,75	0,398526	0,303097	0,829034	0,350267	0,409369	0,409369	0,457491
	35	1,75 - 2,25	0,303097			0,388107			
		2,25 - 275		0,228722			0,29617	0,289477	0,233414

		Hs							
d= 40 m.	**d/L0**	1,25 - 1,75	0,455458	0,346397	0,947467	0,400305	0,467851	0,467851	0,522847
	40	1,75 - 2,25	0,346397			0,443551			
		2,25 - 275		0,261397			0,338479	0,33083	0,266759

	Hs	w^2	w	w^2	w	w^2	w	w^2	w	w^2	w	w^2	w	w^2	w
w = 2π/T	1,25 - 1,75	0,7018	0,8378	0,5338	0,7306	1,4600	1,2083	0,6169	0,7854	0,7209	0,8491	0,7209	0,8491	0,8057	0,8976
	1,75 - 2,25	0,5338	0,7306					0,6835	0,8267						
	2,25 - 275			0,4028	0,6347					0,5216	0,7222	0,5098	0,7140	0,4111	0,6411

d= 35m	**k**	**0,0724**	**0,0565**	**0,1488**	**0,0643**	**0,0743**	**0,0743**	**0,0826**
		0,0565			**0,0707**			
			0,0448			**0,0554**	**0,0543**	**0,0455**
d= 40m	**k**	**0,0720**	**0,0557**	**0,1488**	**0,0637**	**0,0739**	**0,0739**	**0,0824**
		0,0557			**0,0702**			
			0,0436			**0,0545**	**0,0534**	**0,0444**

		Hs	-15 to 15	15 to 45	45 to 75	225 to 255	255 to 285	285 to 315	315 to 345
d= 35 m.	**c**	1,25 - 1,75	11,56	12,92	8,12	12,22	11,43	11,43	10,86
		1,75 - 2,25	12,92			11,70			
		2,25 - 275		14,17			13,03	13,14	14,09
d= 40 m.	**c**	1,25 - 1,75	11,64	13,12	8,12	12,34	11,49	11,49	10,90
		1,75 - 2,25	13,12			11,78			
		2,25 - 275		14,54			13,24	13,36	14,45

		Hs	-15 to 15	15 to 45	45 to 75	225 to 255	255 to 285	285 to 315	315 to 345
d= 35 m.	**L**	1,25 - 1,75	86,73	111,14	42,22	97,72	84,56	84,56	76,03
		1,75 - 2,25	111,14			88,91			
		2,25 - 275		140,27			113,38	115,63	138,03
d= 40 m.	**L**	1,25 - 1,75	87,28	112,82	42,22	98,70	86,04	86,04	76,29
		1,75 - 2,25	112,82			89,53			
		2,25 - 275		143,98			115,20	117,59	141,57

Entrance width (standard) B= **245 m.**

		Hs	0 degrees 0,0000 rad	30 degrees 0,5236 rad	60 degrees 1,0472 rad	120 degrees 2,0944 rad	90 degrees 1,5691 rad	60 degrees 1,0472 rad	30 degrees 0,5236 rad
		Bnew	**245,0 m.**	**212,2 m**	**122,5 m**	**-122,5 m**	**0,4 m**	**122,5 m**	**212,2 m**
d= 35 m.	**B/L**	1,25 - 1,75	2,82	1,91	2,90	-1,25	0,01	1,45	2,79
		1,75 - 2,25	2,20			-1,38			
		2,25 - 275		1,51			0,00	1,06	1,54
d= 40 m.	**B/L**	1,25 - 1,75	2,81	1,88	2,90	-1,24	0,01	1,44	2,78
		1,75 - 2,25	2,17			-1,37			
		2,25 - 275		1,47			0,00	1,04	1,50
Resulting B/L ratio:			4	2	4	0	0	2	4

In the table above, the B/L-ratios for waves from the directions 120°N and 90°N are smaller than or equal to zero. This is because of the fact that cos(90°)=0 and cos(120°)<0. This results in zero or negative entrance widths, with a resulting B/L ratio of 0. These directions are considered to be not critical when determining the wave penetration into the bulk port.

3.11 Wave analysis container port

In the table below, the relevant combined wave heights and their periods are presented for calculation point P60.

Hs (m)		wave direction (Deg)							
		-15 to 15		15 to 45		45 to 75		315 to 345	
		Hs	Tp	Hs	Tp	Hs	Tp	Hs	Tp
Lower	Upper	[%]	[s]	[%]	[s]	[%]	[s]	[%]	[s]
0,75	1,25	0,90	**6,7**	5,35	**7,8**	0,06	**6,2**	0,32	**6,0**
1,25	1,75	0,47	7,5	1,50	8,6	0,01	**5,2**	0,23	7,0
1,75	2,25	0,18	8,6	0,55	9,2	0,00	-	0,14	7,7
2,25	2,75	0,10	8,5	0,22	9,9	0,00	-	0,06	9,8
2,75	3,25	0,05	9,2	0,08	10,3	0,00	-	0,04	9,8
3,25	4,25	0,06	10,5	0,04	10,8	0,00	-	0,03	9,4
4,25	5,25	0,01	**12,4**	0,00	**12,0**	0,00	-	0,01	**11,3**

In the table below, the results of the calculations for the relevant directions are presented. The entrance depth at the container terminal entrance is located at CD – 40m.

L0

L0 [m]	Hs	-15 to 15		15 to 45		45 to 75		315 to 345	
	0,75 - 1,25	[m]	**70,1**	[m]	**95,0**	[m]	**60,0**	[m]	**56,2**
	1,25 - 1,75		87,8		115,5		**42,2**		76,5
	1,75 - 2,25		115,5		132,1				92,6
	2,25 - 2,75		112,8		153,0				149,9
	2,75 - 3,25		132,1		165,6				149,9
	3,25 - 4,25		172,1		182,1				138,0
	4,25 - 5,25		**240,1**		**224,8**				**199,4**

d/L0
d=40m.

d/L0	Hs	-15 to 15	15 to 45	45 to 75	315 to 345
40,00	0,75 - 1,25	**0,5707**	**0,4211**	**0,6665**	**0,7117**
	1,25 - 1,75	0,4555	0,3464	**0,9475**	0,5228
	1,75 - 2,25	0,3464	0,3027		0,4321
	2,25 - 2,75	0,3546	0,2614		0,2668
	2,75 - 3,25	0,3027	0,2415		0,2668
	3,25 - 4,25	0,2324	0,2196		0,2899
	4,25 - 5,25	**0,1666**	**0,1779**		**0,2006**

w

w	Hs	-15 to 15	15 to 45	45 to 75	315 to 345
w=2π/T	0,75 - 1,25	0,9378	0,8055	1,0134	1,0472
	1,25 - 1,75	0,8378	0,7306	1,2083	0,8976
	1,75 - 2,25	0,7306	0,6830		0,8160
	2,25 - 2,75	0,7392	0,6347		0,6411
	2,75 - 3,25	0,6830	0,6100		0,6411
	3,25 - 4,25	0,5984	0,5818		0,6684
	4,25 - 5,25	0,5067	0,5236		0,5560

w^2

w^2	Hs	-15 to 15	15 to 45	45 to 75	315 to 345
	0,75 - 1,25	0,8794	0,6489	1,0270	1,0966
	1,25 - 1,75	0,7018	0,5338	1,4600	0,8057
	1,75 - 2,25	0,5338	0,4664		0,6659
	2,25 - 2,75	0,5464	0,4028		0,4111
	2,75 - 3,25	0,4664	0,3721		0,4111
	3,25 - 4,25	0,3581	0,3385		0,4468
	4,25 - 5,25	0,2568	0,2742		0,3092

k

k	Hs	w^2 -15 to 15	k	w^2 15 to 45	k	w^2 45 to 75	k	w^2 315 to 345	k
	0,75 - 1,25	0,8794	**0,0898**	0,6489	**0,0668**	1,0270	**0,1047**	1,0966	**0,1118**
	1,25 - 1,75	0,7018	**0,0720**	0,5338	**0,0557**	1,4600	**0,1488**	0,8057	**0,0824**
	1,75 - 2,25	0,5338	**0,0557**	0,4664	**0,0494**			0,6659	**0,0685**
	2,25 - 2,75	0,5464	**0,0569**	0,4028	**0,0436**			0,4111	**0,0444**
	2,75 - 3,25	0,4664	**0,0494**	0,3721	**0,0409**			0,4111	**0,0444**
	3,25 - 4,25	0,3581	**0,0397**	0,3385	**0,0380**			0,4468	**0,0476**
	4,25 - 5,25	0,2568	**0,0310**	0,2742	**0,0325**			0,3092	**0,0354**

c

c=w/k	Hs	-15 to 15	15 to 45	45 to 75	315 to 345
[m/s]	0,75 - 1,25	10,4455	12,0621	9,6758	9,3656
	1,25 - 1,75	11,6368	13,1184	8,1187	10,8989
	1,75 - 2,25	13,1184	13,8235		11,9210
	2,25 - 2,75	12,9942	14,5430		14,4456
	2,75 - 3,25	13,8235	14,9086		14,4456
	3,25 - 4,25	15,0776	15,3184		14,0393
	4,25 - 5,25	16,3594	16,1335		15,6860

L

L=c*T	Hs	-15 to 15	15 to 45	45 to 75	315 to 345
[m]	0,75 - 1,25	69,98	94,08	59,99	56,19
	1,25 - 1,75	87,28	112,82	42,22	76,29
	1,75 - 2,25	112,82	127,18		91,79
	2,25 - 2,75	110,45	143,98		141,57
	2,75 - 3,25	127,18	153,56		141,57
	3,25 - 4,25	158,32	165,44		131,97
	4,25 - 5,25	202,86	193,60		177,25

Entrance width container port **Channel B=230 m.**
Total width W=440 m.

When considering the B/L-ratios, two widths need to be taken into account: the approach channel width and the total entrance width. This last width is critical when determining the B/L-ratio.

ANNEX 4 Annex 4: Preliminary Wave Study

In order to acquire a first indication of the extent of the in-port wave penetration, a preliminary wave study has been performed. This study has been performed according to graphs from the Coastal Engineering Manual [USACE, 2002]. The most important processes comprise wave diffraction and reflection. These effects will be elaborated independently from each other in more detail below.

From the preliminary wave study, it will become clear that the effects of in-port wave penetration and propagation will be clearly noticeable when assessing the in-port wave climate. This will have to be analysed in more detail with a wave simulation model (paragraph 5.4). For the wave analysis the output data in calculation point P60 (see 3.3.4) will be used, as around this calculation point both port entrances are located.

4.1 Diffraction

As outlined before, diffraction will be of major importance when considering wave propagation in-port. For a preliminary assessment of its effect, use will be made of the diffraction diagrams adopted from the Coastal Engineering Manual [USACE, 2002]. These graphs represent the resulting diffraction coefficients in the lee side of the breakwaters, at a certain distance from the entrance gap. The diffraction coefficient K_d is defined as the ratio between the diffracted wave height and the normal incident wave height: $K_d=H/H_i$

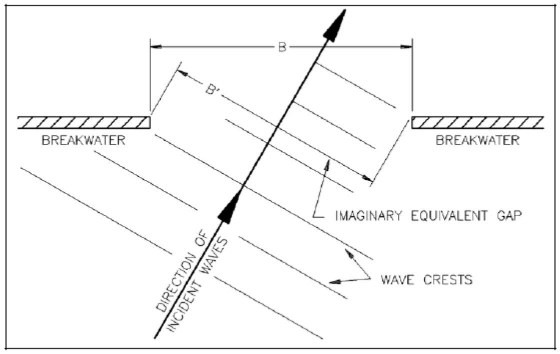
The effects of wave diffraction on an individual wave depend on the incident wave frequency and direction. Each component of a directional wave spectrum will be affected differently by wave diffraction and has a different K_d value at a particular point in the lee of a breakwater. Use will be made of the diffraction diagrams representing irregular waves [USACE, 2002], as in reality waves are far from monochromatic. As outlined before, at the project location mainly waves occur with periods between 7 – 9 s. These waves can be classified as wind waves and they have a large directional spreading. This directional spreading will be taken into account by using the diffraction coefficient graphs representing wind waves with s_{max} = 10. Here, s_{max} is the directional concentration parameter which characterizes the directional spread of a wave spectrum. This value for wind waves is in contrast to swell with a long decay distance (with s_{max} = 75), where the directional spread is quite limited.

The diffraction diagrams represent standard cases with incoming waves normal to the entrance gap. From the directional spreading of the waves it is clear that this situation only occurs for a small portion of time. Nevertheless, the diagrams can still be used for different incoming wave directions by modifying the gap width [USACE, 2002]. This is necessary in order to select the proper diffraction diagram, where the ratio between the gap width (B) and the wave length (L) has to be calculated.

The gap width between the breakwaters (B) is a function of the angle of the incoming waves. This has been clarified in the figure below. For incoming waves normal to the gap, the factor cos(0°)=1 will be used which results in the original entrance width. When the waves make an angle of for example 30° with the normal to the gap, the width B is reduced cos(30°)=0.87 its original width (B'). With this, also the horizontal and vertical dimensions of the diagrams change as well, which requires special attention.

Figure A4.1

Wave incidence oblique to breakwater gap, with the resulting imaginary equivalent gap (B') [USACE, 2002]



For the determination of the wave length L, the methodology is as follows.
Assuming transitional water depth (as d/L lies between 1/20 < d/L < ½, see table A4.1), the following formula will be used [HOLTHUIJSEN, 2007]:

Table A4.1

Classification of water waves [USACE, 2002]

Classification of Water Waves

Classification	d/L	kd	tanh (kd)
Deep water	1/2 to ∞	π to ∞	≈ 1
Transitional	1/20 to 1/2	π/10 to π	tanh (kd)
Shallow water	0 to 1/20	0 to π/10	≈ kd

$L=c*T=g*T^2/(2*\pi)*\tanh(k*d)$

In which:

L = wave length [m]
c = wave celerity [m/s]
T = wave period [s]
g = gravity constant = 9,81 [m/s^2]
k = wave number = 2*π/L [1/m]

With the known wave periods T and water depth d, k can be calculated through iteration between the formulas L= 2*π/k and $L=c*T=g*T^2/(2*\pi)*\tanh(k*d)$.

With these calculated wave lengths, the ratio between the (varying) gap width B and the wave length L can be calculated. These ratios have been rounded off to above to comply with the accompanying graphs from [USACE, 2002], which are only available for certain predefined ratios (1, 2, 4 and 8). Rounding off is a conservative assumption because of the relationship between the B/L-ratio and the wave penetration: if the B/L-ratio is larger, the wave penetration in-port is also larger.

Bulk Port

First of all the wave diffraction into the bulk port will be assessed with the use of the diffraction diagrams. For this, all relevant criteria and parameters need to be determined beforehand. As described in 5.3.1, the analysis is only relevant for a certain range of wave heights and periods. For example, waves smaller than H_i<1.0 m. do not affect the berthed vessels in the bulk port at all. Because of the diffracted wave height in-port decreases with increasing distance from the gap, after a first glance at the diffraction graphs it is assumed that even waves smaller than H_i<1.25 m. do not affect berthed vessels. This is because of the fact that the contour where K_d=1.0/1.25=0.8 is already located close to the entrance gap where no berths are situated. With this, the lower limit for wave analysis according to the wave table for point P60 (see annex 2) is H=1.25 m.

Besides this, it has to be assessed at what wave height the in-port wave penetration is so large that (un)loading of vessels cannot take place. Again after a first glance at the diffraction diagrams it is expected that with a wave height larger than H>2.75 m. this will not be the case (K_d=1.0/2.75=0.36).

With these two criteria the boundaries are known between which the in-port wave diffraction is of special importance: these limits are roughly between 1.25 m. < H < 2.75 m. The calculation of the different parameters and the resulting ratios are presented in annex 3.10, which has lead to results presented below for the different directions.

Table A4.2

B/L-ratios for various incoming wave directions

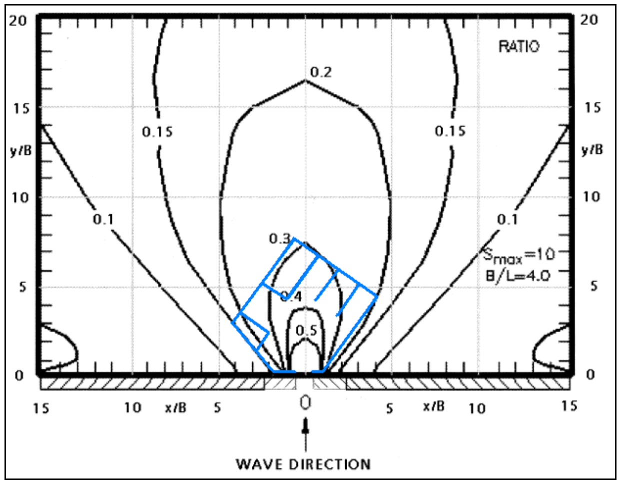
-15° – 15°	15° – 45°	45° – 75°	225° – 255°	255° – 285°	285° – 315°	315° – 345°
4	2	4	0	0	2	4

With the above presented ratios for the various directions, it can be concluded that there are several design situations which will need to be assessed. In the table above, the B/L-ratios for waves from the directions 240°N and 270°N are equal to zero. This is because of the fact that cos(≥90°)= ≤0. This results in zero or negative entrance widths, with a resulting B/L ratio of 0. These directions are considered to be not critical when determining the wave penetration into the bulk port.

To make a preliminary assessment, the final bulk port layout has been overlain onto the specific diffraction diagrams, and this way an estimation of K_d can be made. An example of this is presented in the figure below. For the remaining cases, all the figures have been added in annex 1.

Figure A4.2

Wave diffraction diagram [USACE, 2002] with bulk port layout for B/L=4.0, and waves from 0°N. The figure presents values for K_d



It should be noted that in these diffraction diagrams from [USACE, 2002], reflection of (diffracted) waves has not yet been taken into account. Besides this, the diffraction diagrams are valid for breakwaters with unrestricted space in the lee side behind the gap. This is not exactly the case in the bulk port layout. Also, the diagrams apply for an orientation of the two breakwater tips at 180°.
For an angle of around 120° (as is the case for the bulk port), values for K_d will be around 15% higher [USACE, 2002]. In a first assessment it is assumed that these effects are (at least partly) balanced by choosing the larger B/L ratios beforehand (which was a safe assumption). These, in combination with the roughly visually approximated upper values, the determined values are assumed to be consistent. The figures from annex 1 lead to the K_d values that are presented in the table below.

Table A4.3

Maximum values of K_d at the dry and liquid bulk berths for different wave directions. Reflection is not (yet) included.

Max values		Wave	direction		
of K_d at berth	300°N	330°N	0°N	30°N	60°N
Dry bulk	0.15	0.31	0.36	0.24	0.09
Liquid bulk	0.11	0.26	0.37	0.34	0.25

With the above determined values in combination with the maximum allowed wave height at the berth as defined in 5.3.1, the wave height can be calculated at which the berthing criteria (in terms of allowable wave height) are just met. It must be noted that in this determination only diffraction has been taken into account (although in a relatively safe approach), and no reflection. This was not possible due to restrictions of the visual method used [USACE, 2002].

Maximum values for the incoming wave height from different directions have been summarized in the table below. For the dry bulk this means that $H_{i,max}=H_{allowed}/K_d=1.0/K_d$ and for the liquid bulk berths $H_{i,max}=H_{allowed}/K_d=1.5/K_d$.

Table A4.4

Maximum incoming wave heights to allow (un)loading of berthed vessels

Max values		Wave	direction		
of H_s	300°N	330°N	0°N	30°N	60°N
Dry bulk	6.67 m.	3.23 m.	**2.78 m.**	4.17 m.	11.11 m.
Liquid bulk	13.64 m.	5.77 m.	4.05 m.	4.41 m.	6.00 m.

From the table above it is evident that for every wave direction except 60°N, the dry bulk berths are critical for the maximum allowed wave heigpostht in-port.

This is because of the smaller allowed wave height at the berth (see 5.3.1). It appears that in a first glance waves with heights of $H_{s,i}$≤2.75 m. do not pose any problems at the berths for safe loading and unloading of vessels. Only 0.12% of time (see wave tables) waves from 0°N are larger than $H_{s,i}$=2.75 m. This is considered acceptable as downtime. Besides this, the only other addition to the downtime is the waves arriving from 330°N, where $H_{i,i}$>3.25 m. is exceeded for 0.04% of time.

So for (100%-0.12%-0.04%=) 99.84% of time, vessels can safely load and unload at the berths with this bulk port- and breakwater layout. In 4.6.2 it was assumed that this downtime was 0.33%. This means that the actual downtime will be somewhat lower than calculated in 4.6.2. However, for 7.1% of time (see 4.6.2) vessels can not enter the bulk port because of the fact that tugboats can not fasten to the vessels outside the port. This last criterion still determines the actual downtime.

In this preliminary study, at first glance it appears that the bulk port layout adequately creates calm in-port berthing conditions. This will however be investigated in more detail with the simulation model in paragraph 5.5.

Container port

In the wave diffraction assessment of the bulk port, the entrance gap width could be used according to the graphs of [USACE, 2002]. For the container port, a different methodology has to be used, as the entrance gap is not perpendicular to the basin: its form is more complex and far from standard. Incoming waves at this entrance will be subject to various processes, of which diffraction and reflection are the most important ones. A schematisation regarding the container port entrance will be made. For this, first of all an inventory will be made for the most unfavourable situations.

It is expected that waves from the dominant directions 270°N – 300°N will not pose the largest problems for the berthed container vessels. These waves will generally not enter the port because of the breakwater layout (and the reflection off the northern bulk port breakwater). The wave penetration due to diffraction into the container port is assumed to be the largest for waves from the direction 30°N, as the approach channel (entrance) is practically aligned with this direction. Waves from this direction are the least subject to the diffraction and reflection processes at the entrance, which would decrease the propagating wave height (as is the case for waves from 0°N).

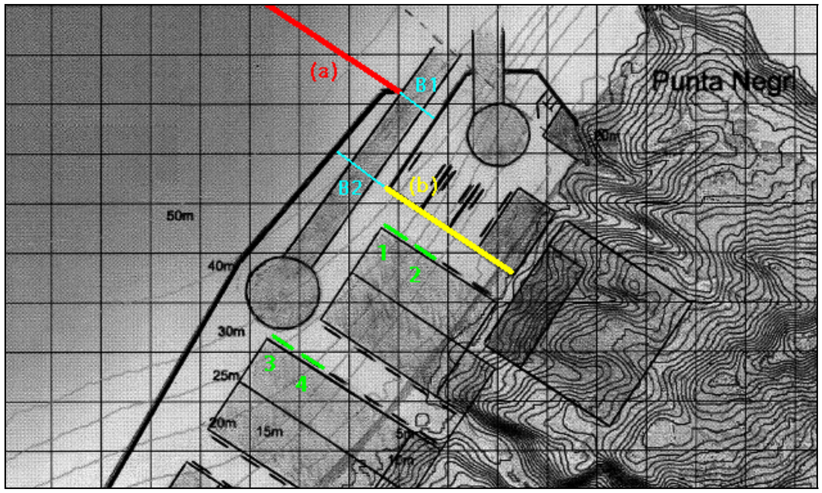
Following the same methodology as outlined earlier, the container port wave parameters have been calculated and are presented in annex 3.11. It is assumed that waves smaller than H_i<0.75 m. will not pose any problems for the berthed vessels.

The schematisation of the container port entrance is as follows, see the figure below.
The north-western container terminal breakwater will be schematised in two ways: both as a semi-infinite breakwater (a) at entrance location B1 and with an entrance with a breakwater gap. It is expected that especially waves from 30°N will give the largest diffraction coefficients around width B2, as these waves are practically aligned with the container port approach channel. From width B2 onwards, again a semi-infinite breakwater schematisation (b) is used, here for the southern bulk port breakwater. This results in a K_d consisting of two parts. The waves will diffract further in-port.

The berths which are (expected to be) the most vulnerable to incoming waves are indicated in the figure below in green (1-4). Especially for these locations the extent of wave diffraction has been determined.

Figure A4.3

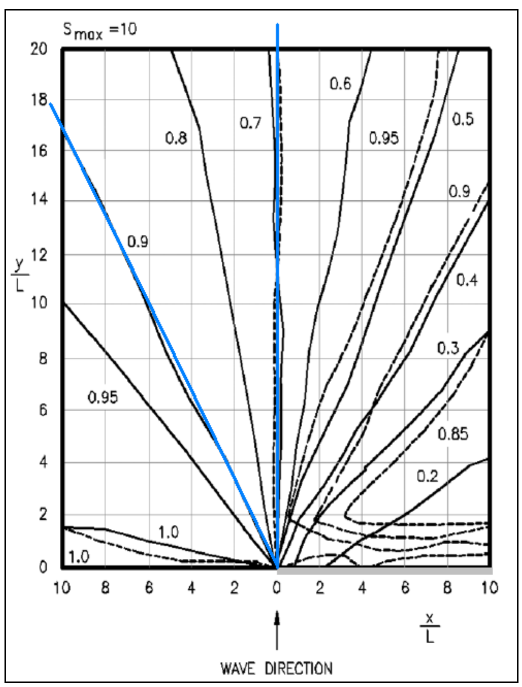
Schematization container port entrance with (semi-infinite) breakwaters (a) and (b)



The analysis with a semi-infinite breakwater (a) at B1 gives diffraction coefficients at B2 between $0.7 \leq K_d \leq 0.9$ (see figure below). Along the left line (depending on the wave length L) the location of the tip of breakwater (b) is situated. Here, it is in a first estimate assumed that the breakwater tip of (a) was oriented perpendicular to the wave direction.

Figure A4.4

Diffraction coefficients semi-infinite breakwater (a)

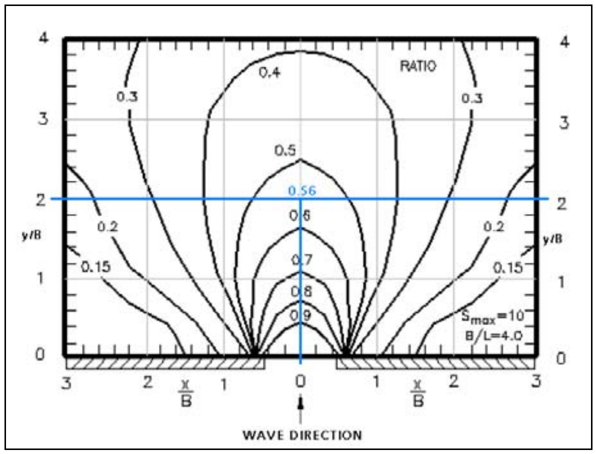


This approach can be seen as an upper limit purely for the diffraction coefficients (no reflection), and the approach with the breakwater gap width as a lower limit.
This estimation will be done below. Again it is assumed that the breakwaters gap is perpendicular to the wave direction. The gap width of the approach channel is W=230 m., but the gap width of the total entrance B1 amounts to 440 m.

The waves that enter the container port at a depth of 40 m. from 30°N have lengths varying between 94 m. and 194 m. This gives for the ratio B/L values of around 2-4. This last value will be used to be on the safe side (B/L=4.0). The y/B ratio amounts to 2, as y=890 m. (the distance from B1 to B2). The result is presented below.

Figure A4.5

Diffraction diagram of a breakwater gap [USACE, 2002] for B/L=4.0



From the figure above it is clear that with this approach the diffraction coefficient K_d is around 0.56. However, this is a (too) low value. Due to reflection and the limited space behind the breakwater this value should actually be higher. At this point, the current visual approximation method from the Coastal Engineering Manual falls short: it lacks proper ground to assess the far from standard container port entrance layout.

It is expected that the real value for the reflection coefficient lies somewhere between the boundary limits $0.56 \leq K_r \leq 0.8$, of which K_d=0.7 is the averaged value which has been adopted in this preliminary assessment. This means that diffracted waves through B1 arriving at B2 still have 0.7 times their incident wave height.

From B2 further in-port, again the schematisation with the semi infinite breakwater will be used. It is assumed that waves propagate further in-port by diffracting around the bulk port breakwater corner (b). From this location (which can be considered as the breakwater tip) berth 1 (see figure A4.3) can be evaluated by checking the diffracted wave height under an angle of 50 degrees, which amounts to K_d=0.31. The combined diffraction coefficient for waves arriving at berth 1 will be: $K_{d,tot}$=0.7*0.31=0.22. With the maximum allowed wave height at the berth of H=0.5 m., the incoming wave height will be H_i<2.27 m. This height is exceeded for 0.34% of time, which is considered acceptable for this one particular berth. Especially because of the fact that all the other berths are located more sheltered.
For example, berth 2 (see figure A4.3) is located under an angle of 25 degrees in the lee side of (b), which results in a combined diffraction coefficient $K_{d,tot}$=0.7*0.23=0.16, and incident waves at B1 with a height of H_s=3.13 m.

Berths 3 and 4 (as in figure 5.4) are located much further in-port but almost perpendicular to the incoming wave direction. According to the approach of the semi-infinite breakwater, the diffraction coefficient would remain the same: K_d=0.7. This is considered to be not realistic, as because of wave energy dissipation due to diffraction and reflection (at the inside of the breakwaters and at the heads of the container terminals), the wave height should decrease further in-port because of this energy loss.

From the diffraction diagrams it can be seen that the diffraction coefficient decreases with increasing distance from the gap. However with this estimation, the diffraction coefficient at berth 3 and 4 (y/B=1.889/610=3) would still be K_d=0.46 (so that $K_{d,tot}$=0.7*0.46=0.32), and the maximum allowable wave height $H_{allowed}$=0.5/0.32=1.56 m. This means that at berth 3 more stringent wave criteria exist than for berth 1.

These results are summarized in the table below, for berths 1 – 4 as indicated in figure A4.3. Other berths are located more sheltered and no additional problems are expected.

Table A4.5

Maximum allowable wave heights outside container port for which the criterion H≤0.5 m. at the berth is just met.

	$H_{i\ outside}$
Berth 1	< 2.27 m.
Berth 2	< 3.13 m.
Berth 3	< 1.56 m.
Berth 4	< 1.56 m.

However, the distance from the port entrance to berths 3 and 4 is so large (around 2.000 m.), that in reality it is expected that at this location the wave penetration will be smaller (because of the earlier mentioned wave dissipation at the inside of breakwaters and heads of the terminals and thus decreased reflection and diffraction).
As at this point the current applied visual method according to [USACE, 2002] falls short, this exact effect will have to be assessed in more detail with a wave simulation model (paragraph 5.5).

4.2 Reflection

As emphasized before, wave energy that enters a port must eventually be dissipated or scattered back out. This dissipation occurs at the port's interior boundaries. It may be necessary, because of excessive wave reflection, to decrease the reflection of certain boundary structures in order to keep interior wave agitation at acceptable levels. This will also be a subject of the wave assessment with a simulation model in 5.5. Generally, rubble mound breakwaters have lower reflection coefficients than monolithic breakwaters.

According to [USACE, 2002], the reflection coefficient for sloping structure forms can be given by the following equation:

$$K_r=(a*I_r^2)/(b+I_r^2)$$

In which:

a, b = coefficients depending on structure geometry and the (ir)regularity of the waves [-]
I_r = Iribarren number = $\tan(\alpha)/\sqrt{(H_i/L_0)}$ [-]
α = angle of slope form with horizontal [°]
H_i = incident wave height [m]
L_0 = incident wave length in deep water [m]

However, reflection coefficients for a monolithic breakwater cannot be calculated with the formula presented above: the Iribarren number gives for a vertical slope infinite values of I_r. Vertical wall breakwaters without special wave dampening measures generally have reflection coefficients approaching unity, e.g. K_r=0.9.

Problems are to be expected at the entrance of the container port. At this location, the somewhat narrow port entrance is surrounded by monolithic breakwaters (the caissons, see the figure below), which have large reflection coefficients (e.g. K_r=0.9). This means that not only incident waves are almost fully reflected, but also diffracted waves within the port will largely be reflected.

It could turn out that around the container port entrance the incident and reflected waves lead to a difficult navigational situation for entering and departing vessels. A (too large) portion of time that because of these conditions the container port entrance is unavailable is considered to be not acceptable. This will be an important subject of the wave simulation model.

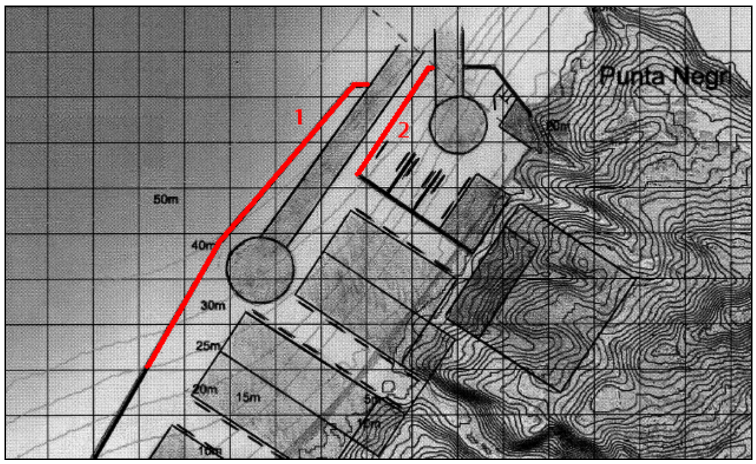


Figure A4.6

Entrance container terminal surrounded by monolithic breakwater types with reflection coefficients $K_r \approx 1.0$

It is expected that proper measures for wave dissipation will need to be taken at these problematic locations. Wave energy that penetrates a port entrance should be dissipated locally as soon as possible, to prevent its subsequent further reflection and propagation. Sloping planes at the interior boundary of the port are effective at absorbing wave energy, and can be applied at necessary locations (e.g. at the inside of the northwestern container terminal breakwater (1), or the northwestern edges of the container terminal blocks). Also, monolithic breakwaters which absorb wave energy can be applied [MARTINEZ *et al.*, 2010]. This is often the case with caisson breakwaters with grates. This seems at first glance a good option for the monolithic breakwaters at the container port entrance (2).

Within the bulk port the maximum allowable wave height is larger than in the container port. Besides this, the use of rubble mound breakwater (to the northeast and the southwest) is favorable for wave energy dissipation. Reflection coefficients for rubble mound breakwaters are smaller: [USACE, 2002] states that the highest reflection coefficients for these structures are around K_r=0.5. However, waves can still be reflected from the inside of the monolithic breakwater to the northwest (2). Also here it could turn out that additional measures will have to be taken.

For the container port it is expected that diffracted waves from the 30°N direction will pose the most problems for the berthed vessels. For the bulk port it is expected that this will be the case with waves from 0°N. For the considerations regarding reflection this is somewhat different. Here, the directional spectrum will need to be taken in to account, as incident waves from various directions will penetrate in-port leading to different scenarios.

Besides this, diffracted waves are reflected at the interior port's boundaries, which lead to numerous different wave conditions.

The combination of diffraction and reflection determines (too large extent) the wave conditions in-port. As outlined earlier, in the foregoing a consideration for both phenomena has been made independently, but especially their combined effect will have to be evaluated. This more complex assessment inevitably requires the use of simulation models. Besides this, it turned out that the current applied (visual) method for approximating in-port diffraction fell short: it lacks proper ground to assess the masterplan layout. Also here, application of a wave simulation model could be a solution. This will be the subject of paragraphs 5.4 and further.

ANNEX 5 Annex 5: Additional information

5.1 Detailed description of steps and tasks in time

- The first period that starts in November comprises a start-up phase ***(1)***, with a description of the project and a definition of the subject. After the first meeting with the thesis committee ***(a)*** the work plan regarding the graduation project will be formulated and finalized around the end of week 49, December 4th.

- Meanwhile, from week 48 onwards, an orientation and inventory phase commences ***(2)***. During this period, relevant data will be determined, collected and analyzed, which gives a clearer view of the situation on site. From this, relevant parameters (e.g. wave heights and directions) can be deducted which will be needed in the next phases. Furthermore, more in-depth knowledge is needed for certain aspects of the MSc. project (e.g. transshipment ports, or the masterplan process itself), which requires some literature study. Week 53 is (lightly) greyed out gray, because of reduced productivity during holidays (Christmas and New Year).

- After this phase, there will be enough data available to start the design of the port's 'wet area' ***(3)*** (basins, channels, turning circles). For this, first of all the ship characteristics (e.g. draught, size, etc.) need to be determined or calculated. Here again, the greyed out periods indicate exams and holidays.
- The next step, which starts halfway through February, will be an assessment of the required 'dry area' of the port ***(4).*** Dimensions of quays and areas for different types of terminals and cargo will be determined. Special attention will be paid to the transshipment aspect of the port, and to the fact that the port will be constructed in two phases.
- The transition to the subsequent activities will be around the beginning of March, although it is not defined that sharply. Here, additional aspects are (more or less broadly) taken into account ***(5)***, such as morphology, hinterland connections, tugs and safety considerations for dangerous cargo. All these aspect have their influence on the layout of the port.
- From all the preceding activities, multiple alternatives can be developed for the port layout, and these will be evaluated and compared to each other by means of a Multi Criteria Analysis ***(6)***. A rough cost estimate will have to be made. This results in one preferential port layout with its orientation and arrangement of the breakwaters, the wet- and the dry areas.

- Now that the layout of the port has been determined, a (short) study can be done to the penetration of waves into the port under influence of this layout. For this, different models are available which take into account several phenomena (e.g. diffraction, refraction). One of these models needs be selected and learned ***(7)***.
- Next up is the actual application of this model on the specific situation ***(8)***. The determined port layout will be imported, and waves will be modelled. The model will yield results of the wave penetration into the port.

- As a last step, the results of this model simulation will be analyzed and evaluated ***(9)***. A judgment will be made whether these results are acceptable or not, and what can be altered to improve these results if needed. The results will be used for a proper breakwater design.

- It is expected that in one of the first weeks of May (week 18), the design of the breakwater can commence. At first, relevant parameters need to be determined for this ***(10)*** (e.g. wave characteristics near shore and probability of occurrence). Furthermore, after analysis of the characteristics and (local) conditions, a decision will be made for the resulting type of breakwater to be designed.
- After this step, the actual technical design of the breakwater will be done ***(11)***, where special attention will be paid to important aspects (e.g. armour layer, filter, etc.). The design of the breakwater will take several weeks.

- Halfway through June (week 23/24), it is expected that the finalization phase of the MSc. project will commence ***(12)***. During this period, the thesis will be completed and the graduation trajectory will be rounded off.
- This comprises (amongst others) handing in the final thesis and giving a concluding presentation ***(13)***. This will be somewhere around the end of June and the beginning of July

5.2 Suez Canal characteristics
From 2001 onwards, after a large project to deepen the Suez Canal [WEBSITE SUEZ CANAL] the canal could accompany ships of 62 ft (19 m.). The cross-sectional area increased.
The canal depth will reach 66 ft (20 m) in 2009 and it is planned to reach a depth of 72 ft (22 m) from 2010 onwards [WEBSITE SUEZ CANAL].

For one way traffic, the canal is already 133 m in width, which is (more than) large enough to accompany the largest possible ships that will visit the new port. The next limiting factor will be the Malacca Strait around Malaysia [LIGTERINGEN, 2007], [WEBSITE GLOBAL SECURITY], [WIKIPEDIA]. It is expected that ships larger than this size will not moor in Nador.

Another limiting factor could be the height of the ships to pass under the Suez Canal Bridge. The bridge has a 70 m over water clearance [WIKIPEDIA], [WEBSITE SUEZ CANAL]. This will not be a large problem for the vessels visiting the new port: dry and liquid bulk vessels have rather small freeboards and the largest container vessels (e.g. the Emma Maersk, fully loaded) has a maximum height above the water surface of around 40 m.

5.3 DIFFRAC-2DH Model
The DIFFRAC packet consists of 3 programs:
A preprocessor (PREDIF)
- A processor (DIFFRAC)
- A postprocessor (POSDIF)

PREDIF is used to make input suitable for the main computational program, DIFFRAC.
This is done using a menu controlled, whole screen data entry system.
The program DIFFRAC computes the solution to the wave penetration problem.
POSDIF arranges print and graphical output in the required specified form.

The program DIFFRAC-2DH computes the diffraction effect of waves in ports and around structures. The mathematical model is based on the theory of linear harmonic water waves. Simplifications made in the mathematical formulations are:

- The fluid is ideal, no viscosity or turbulence effects are taken into account,
- The fluid motion is irrotational, therefore a potential formulation can be used,
- There is no energy dissipation in the wave propagation area,
- The formulation is linearized, therefore only waves with a small wave steepness can be considered,
- The wave motion is harmonic,
- The water depth must be constant in each basin,
- Boundaries of the propagation area must be schematized to vertical walls, but may have partial reflection properties,
- The boundaries at which the incoming wave field is defined must be situated on one straight line.

The equation ultimately solved by the program is the Helmholtz equation:

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} + k^2 \phi = 0$$

In which:

ϕ = two dimensional wave potential (complex)
k = wave number = $2\pi/L$

The boundary conditions in the horizontal plane for the vertical walls are:

$$\frac{\partial \phi}{\partial n} + ika\phi = 0$$

In which:

a = reflection property of the wall in terms of amplitude reduction and phase shift

The program package gives the possibility to schematize the port into two or more basins width a different water depth for each of them. These basins are coupled by means of an internal boundary, on which continuing conditions are given for ϕ and $\frac{\partial \phi}{\partial n}$.

The practical restrictions of the DIFFRAC-2DH model are:

- No special facilities have been arranged in the program for using backup storage by solving the system of equations. Therefore, the size of computational area is limited by a certain maximum size, depending on the wave period and the computer capacity.
- The program can be used to estimate resonance frequencies of ports. However, the computed wave heights at a resonance frequency are not reliable due to restrictions of the model (e.g. no energy dissipation).
- In case of close opposite boundaries (relative to wave length), the results of the calculations can be inaccurate. Only with a very fine source distribution at the boundaries can reasonable results be obtained. For such cases it is better to use a one dimensional model.

ANNEX 6 Annex 6: References

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