Port Engineering and Maritime Works International Workshop 2017

Design of a Commercial Port in Nador West, Morocco

Group A: Annalisa, Christelle, James, Juan Manuel, Marios, Matthias, Sara, Stendert, Tessa & Yichuan

21st of June 2017

Acknowledgements

We would like to acknowledge all the professors and visiting professionals, who gave lectures, good advice or helped in other ways to realize our project. We want to give special thanks to Mr. Mouaze and Mr. Safari, who let us test our breakwater in their laboratory wave flume, and Mr. Carpentier for his commitment to this workshop.

Executive Summary

This report includes a design from the new Nador West Med Port near Nador, Morocco. Given requirements are an entrance to the northeast and minimal dredging works to be conducted. A layout of the harbour is made for wave directions during storm from the northeast and northwest. A maximum downtime of the harbour of less than 1% of the year is allowed. The harbour will accommodate smaller vessels, such as ro-ro, general cargo, and container feeders up to larger vessels for bulk transport, crude oil and product tankers, and container mother ships. Structural designs are made for two cross-sections of breakwaters, a rubble mound and a vertical caisson, a mooring structure for tankers with mooring and breasting dolphins, and a typical cross-section of a quay wall.

Contents

List of Figures

List of Tables

1 Introduction

The new Port of Nador in Morocco has to be designed to handle a higher traffic capacity to allow for further development of the Orientale Region. Since there is not enough space in the old harbour, a new harbour will be constructed on the west coast of the headland, around 30 kilometers from Nador (**Error! Reference source not found.**).**Error! Reference source not found.**). The considered area, Aaxanen, is an uninhabited coast with enough space for the development of the new Nador West Med Port (NWMP).

FIGURE 1-1- PORT LOCATION

Many possibilities for the construction of NWMP exist. This report focusses on the option with an entrance facing to the Northeast and the need to minimise dredging during construction. Future development and expansion of the port will be considered.

First of all, the design criteria are determined, focusing on design ships and number of berths. Then the design conditions are set, considering the offshore and nearshore wave conditions, a geotechnical analysis, and the sediment transport. The next step is to base conditions for the layout on the previously defined design criteria and conditions. Afterwards a layout can be determined. After which the wave agitation in the harbour can be modelled. After several runs with different layouts a final layout is found. A structural design is made for the breakwaters (both rubble mound and vertical caissons), a cross-section of a quay-wall and a mooring and breasting structure for crude oil and product tankers. The final steps include a cost estimate and an overview of the project management, leading to a final conclusion to summarise the final design.

2 Design Criteria

2.1 Design Ships

The dimensions of the design ships are needed to calculate the number and dimensions of the berths and the dimensions of the navigational channel and basin.

To determine the design ship for each classification of ship, several graphs were created from a database of ships currently in service. These graphs include length, breadth and draught versus deadweight in tons or the number of TEU's for container carrying vessels.

Once these graphs were plotted, anomalous and extreme values were cut off via a threshold, to ensure the port was not being designed for a single ship with much larger dimensions which would have a very low frequency of entering the port. Instead these ships will not enter the port by design, as the dimensions of the berths and other structures will be too small.

In the figures below are an example of the process used for Product tankers:

FIGURE 2-1- LENGTH VS DEADWEIGHT TANKER SHIPS

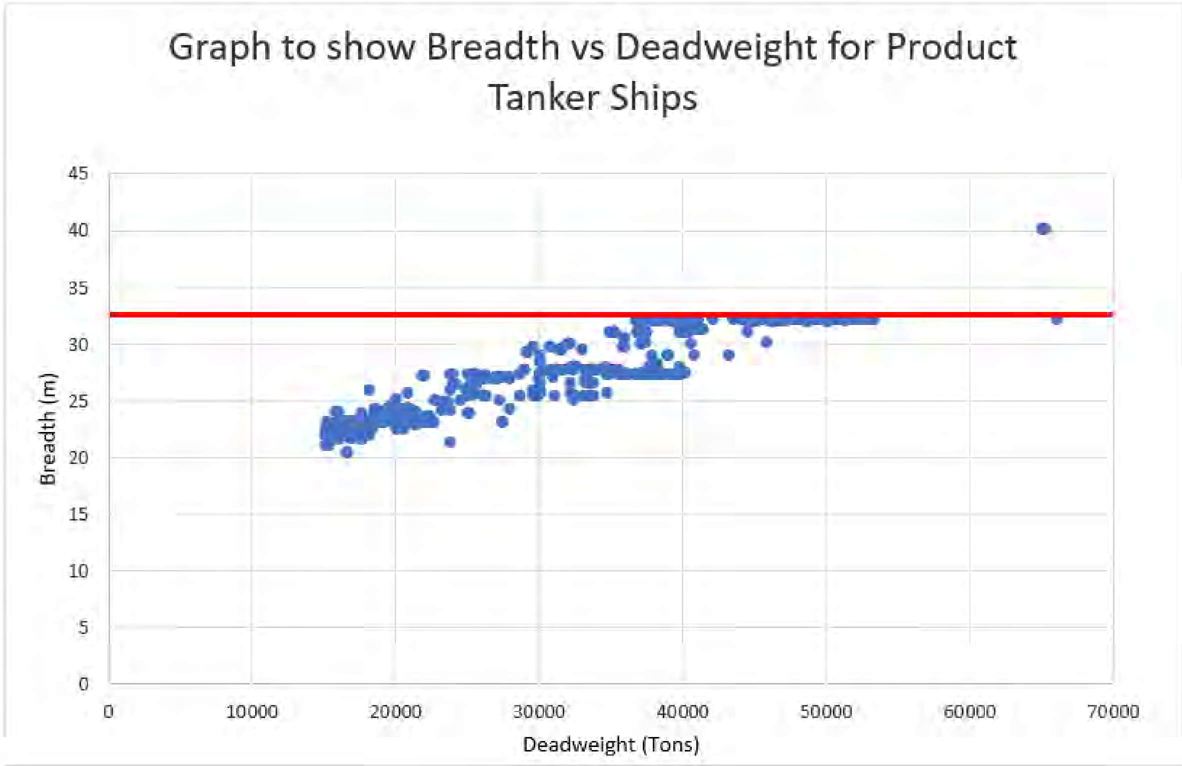


FIGURE 2-2- BREADTH VS DEADWEIGHT TANKER SHIPS

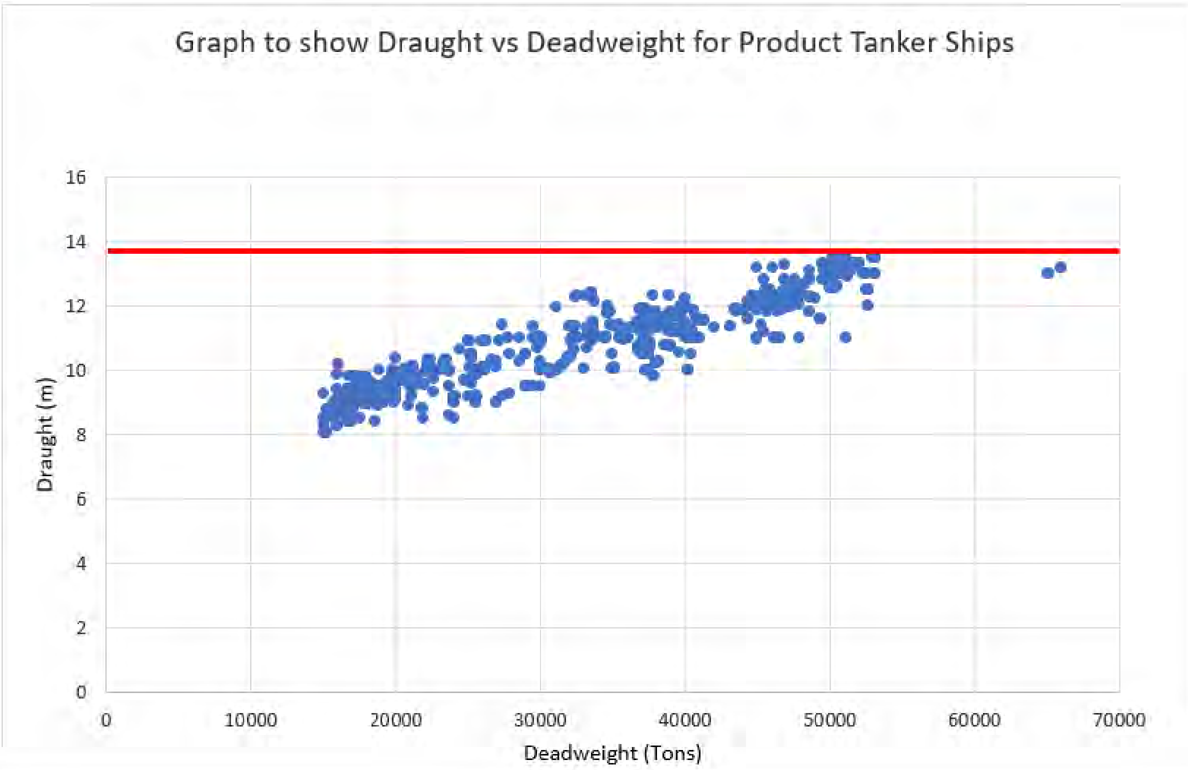


FIGURE 2-3- DRAUGHT VS DEADWEIGHT TANKER SHIPS

From the above graphs, it can be seen by the threshold (red line) that the design product tanker ship is 195.3m long, 32.3m wide and has a draught of 22m.

The above process to determine the design ship was carried out for all classifications of ships which will be needed according to the specified traffic capacity. The results are summarised below: (Table 2-3Table 2-1):

TABLE 2-1- SUMMARY SHIP RESULTS

	Length	Draught	Breadth	Deadweight	TEUs
General Cargo	185.5	12	29.4	40085	-
Bulk Carrier	295	18.3	47	185920	-
Crude Oil Tankers	343.7	22	60	300000	-
Ro-Ro Ships	200	7.5	26.6	14565	-
Product Tankers	195.3	13.5	32.3	53187	-
Container Mother	399.2	16	59	199273	19224
Container Feeder	294.9	13.65	32.3	70000	5303

From this table, the extreme values of the design can be seen:

- Length of 399.2m from the container mother ships
- Breadth of 60m from the crude oil tankers
- Draught of 22m from the crude oil tankers

These extreme design values will determine the dimensions of the navigational channel and harbour entrance.

2.2 Number of Berths

Two separate methods were used to determine the number of berths for each cargo type based upon the traffic capacity specified in the brief and the design ships.

2.2.1 Number of Berths – General Formula

In order to calculate the capacity of each berth and to determine the number of berths, it is possible to use the general method shown in this chapter, as a preliminary way.

This method consists of applying the following general formula to each type of terminal in the port.

$$V_y = N_c C_c N_h O_f E_f$$

EQUATION 2-1- GENERAL FORMULA NUMBER OF BERTH

Where V_y is the yearly traffic of one berth (Tons or TEU per year), N_c is the number of (un)loading equipment used by one berth, C_c is the (un)loading rate (Tons or TEU per hour), N_h is the number of operational hours worked in a year, O_f is the occupancy factor which represents the ratio of time which a berth is occupied and E_f which is an efficiency factor equal to 80%.

Assuming 350 days of each year will be worked (allowing for holidays and bad weather) and that the port will operate 24 hours per day. This gives a total number of operational hours per year as 8400.

The Occupancy factor, the ratio of time in which the berth if occupied, depends on the type of terminal and the number of berths and has to be estimated. In order to help the designer to choose a realistic Occupancy factor value, it is possible to use the following Table, (Table 2-2), which shows an acceptable range of Occupation factors.

TABLE 2-2- BERTH OCCUPANCY FACTORS IN PERCENTAGE (THORESEN, 2003)

Number of berths	Berth occupancy factor in percentage Control of arrival of ship to berth		
	None	Average	High
1	25	35	45
2	40	45	50
3	45	50	55
4	55	60	65
5	60	65	70
6 or more	65	70	75

In this preliminary method, once the designer assumes the number of berths and the Occupancy factor to use in the equation, the designer needs whether the yearly capacity calculated meets the design criteria for total yearly traffic.

To calculate the number of berths, the number of (un)loading equipment and the equipment's productivity need to be estimated:

- For Container Terminals, it was assumed that 4 cranes would operate on each berth, each crane having a productivity of 25 containers per hour.
- For Liquid Terminals, it was assumed that 80% of the transported liquids are crude oil and 20% are refined products. Only 1 pipe would be used to (un)load each vessel with a productivity of 12500 tons/hour.
- For Dry Bulk (Coal) Terminals, 2 pieces of (un)loading equipment will be used per berth, with a productivity for importing (unloading only) of 1250 tons/hour.
- For General Cargo Terminals, assuming 3 cranes will be used each with a productivity of 60 tons per hour.

- For Ro-Ro Terminals, one ramp will be used loading 600 vehicles per hour or 13500 tons per hour. (Assuming an average weight for each vehicle of 22.5 tons (Department for Transport GOV UK, 2013)

The results for the number of berths for each cargo terminal are summarised in the following Table. (Table 2-3):

TABLE 2-3- SUMMARY NUMBER OF BERTHS

Terminals Quantities	Container	Oil	Coal	General Cargo	Ro-Ro
Nc	4/berth	1/berth	2/berth	3/berth	1/berth
Cc	37.5 TEUs/hour	12 500 tons/hour	1 250 tons/year	60 tons/hour	13 500 tons/hour
Nh	8 400 hour/year	8 400 hour/year	8 400 hour/year	8 400 hour/year	8 400 hour/year
Of	0.65	0.35	0.4	0.55	0.35
Ef	0.8	0.8	0.8	0.8	0.8
Vy	655 200 TEUs/year	29 400 000 tons/year	6 720 000 tons/year	665 280 tons/year	31 752 000 tons/year
Design annual throughput	3 000 000 TEUS/year	25 000 000 tons/year	7 000 000 tons/year	2 000 000 tons/year	1 000 000 tons/year
Number of berths	**5**	**1**	**2**	**3**	**1**

2.2.2 Number of Berths – Queuing Theory

The second method used to determine the number of berths was queuing theory, in this method the service time and the time between each ship arriving at its respective berth were taken into consideration. As well as the loading rates and occupancy factors mentioned in the general formula method.

To determine the number of berths a number of equations were used to determine important parameters such as calls per year, arrival rate, service rate, berth occupancy and the utilization factor. These equations have been detailed below (Table 2-4):

Table 2-4- Summary Number of Berths, Queuing Theory

Variable	Equation	Units
Calls Per Year	$\frac{Yearly\ Cargo\ Traffic}{Average\ DWT\ Per\ Ship}$	No units
Inter-arrival time	$\frac{1}{\lambda} = \frac{Operational\ Hours}{Calls\ Per\ Year}$	Hours
Arrival Rate	$\frac{1}{Interarrival\ Time}$	Hours^{-1}
Service Time	$\frac{1}{\mu} = \frac{2\ x\ Average\ DWT\ Per\ Ship}{(un)loading\ rate} + Mooring\ Time$	Hours
Service Rate	$\frac{1}{Service\ Time}$	Hours^{-1}
Berth Occupancy	$\rho = \frac{\lambda}{\mu} = \frac{Arrival\ Rate}{Service\ Rate}$	No units
Berth Utilsation	$u = \frac{\rho}{n} = \frac{Berth\ Occupancy}{Number\ of\ Berths}$	No Units

This method involved a number of assumptions:

- All ships will use the same queuing method M/E_1/n
- The maximum acceptable wait time was determined as 10% for all traffic types other than container traffic, containers are considered to be more valuable than the other traffic types therefore its maximum wait time was selected as 5%. (Wait Time is the percentage of time spent which is spent waiting for a berth of the total service time).
- The mooring time was assumed to be 2 hours for all ships.
- The loading rate for Ro-Ro terminals is 600 vehicles per hour, with an average DWT of 22.5 tons calculated from standards set out by the UK government (Department for Transport GOV UK, 2013)
- The ratio of the two separate liquid products is 80% oil: 20% Refined Products.
- Two separate berths are needed for the two liquid products (Crude Oil and Refined Products).

To determine the number of berths using Queuing theory, the Berth Utilisation factor was calculated using an initial guess of the number of berths, and then a table, Average Waiting Time of Ships in the Queue M/E/n (Groenveld, 1993) was used to find the wait time of each vessel. If the wait time was less than the established limits stated above the initial guess of the number of berths is correct.

The results of this method are displayed in the table Appendix A: Number of Berths Validation

Both the general and queuing theory methods give the same value for the number of berths, summarised below (Table 2-5):

TABLE 2-5- BERTH VALIDATION

Cargo	Number of Berths
General	3
Ro-ro	1
Bulk	2
Liquid (Oil)	1
Liquid (Product)	1
Container	5
Total	13

2.3 Quay Lengths

The lengths for each quay were calculated using the following formulas:

$$L_q = L_D + 2 \times 15 \text{ For 1 berth}$$

$$L_q = 1.1 \times n \times (L_D + 15) + 15 \text{ For more than 1 berth}$$

EQUATION 2-2- QUAY LENGTH

Where L_D is the design ship length and n is the number of berths.

These equations apply to quays where each berth are in a single line, which is the case for General, Ro-Ro and Container berths.

For the Oil and Bulk berths, it was assumed that L-Jetty's will be used for each isolated berth, these berths require a clearance of 50m either side of the design ships length.

Below is a table (Table 2-6) showing the lengths of quays for each respective cargo type:

TABLE 2-6- SUMMARY QUAY LENGTH

Cargo	Number of Berths	Design Ship Length	Quay Type	Quay Length	Terminal Length
-	-	m		m	m
General	3	186	Line	720	1010
Ro-ro	1	200	Line	290	
Bulk	2	295	L shape	510	510
Liquid (Oil)	1	343.7	L shape	858	1608
Liquid (Product)	1	195.3	L Shape	235	
Container Feeder	3	295	Line	1090	2130
Container Mother	2	400	Line	1040	

2.4 Storage Areas

Different methods were used to calculate the storage area required for each type of cargo.

2.4.1 General Cargo

General cargo terminals require a certain length behind the quay to allow space for cranes to unload/load cargo, warehouse and open areas for cargo storage vehicle access/parking.

This length was determined as 345m by taking the average of the upper and lower limits for these lengths.

Multiplying this length by the quay length of 720m gives a total storage area of 97200m^2 (9.72 Hectares).

2.4.2 Ro-Ro

As a general rule Ro-Ro terminals require around 10 hectares per berth, therefore only 10 hectares are needed for Ro-Ro Storage.

For a quay length of 290m, the Ro-Ro storage area will need to extend 345m behind the quay.

2.4.3 Containers

The storage area required depends on the storage configuration of the containers which also determines the type of equipment needed to operate the terminal. The following equation (Equation 2-3) was used to determine the storage area taken up by the containers per berth:

$$S = \frac{N_{TEU} \times T_{st} \times S_{TEU}}{F_u \times 365 \times F_{occ}}$$

EQUATION 2-3- CONTAINERS STORAGE AREA

Where N_{TEU} is the yearly traffic, T_{st} is the time a container is stored, S_{TEU} is the surface occupied by 1 TEU (dependant on storage configuration), F_u is the utilisation factor (from 0.78 to 1 depending on how high containers are stacked), 365 is the number of days in a year and F_{occ} is the occupation factor of the terminal (assumed as 0.8).

This component of the storage is, as a general rule, 70% of the total storage behind the quay so the values calculated in the above equation will need to be divided by 0.7 to account for this. Also, a length will be needed behind the quay for cranes to operate this length was assumed to be 47.5m, multiply this by the quay length (2130m) requires a total of 10.12 hectares

To find a reasonably sized storage area and to determine the storage configuration, all configurations were compared to find a reasonable value of approximately 25-30 hectares per berth. The results are displayed in appendix B.

From Appendix B, an acceptable value for the total storage area per berth was found as 30.70 Hectares, using a RMG/RTG storage configuration with 4 levels of containers. This gives a total container storage area of 153.52 hectares and a depth which the storage area extends behind the quay of 720.7 m.

2.4.4 Liquid Storage Area (Oil terminal)

The oil volume for one year is calculated to be 25 million tons which represent 20 million m^3. The operational storage capacity is, generally, in the order of 1 month consumption and it is chosen from the annual consumption. The storage capacity of the terminal must to be almost 1 670 000 m3 In case of a site accident, the oil have to be retained inside the schemed boundaries (generally 5m high bund for a useful high of 4m). If the product is stocked in 100 000 m3 tank, 17 reservoirs are needed. The needed surface is 25 000m² per unit so we require 425 000m². But we can consider that this storage area represent 70 % of the oil terminal. We have to take into account pipes, technical buildings, administrative buildings, exchange between trucks/terminal, parking.

The total surface is then to 638 143 m², almost 62 ha.

2.4.5 Bulk Storage Area

The storage of bulk material such as coal doesn't need to be just behind the quay or jetty of the coal terminal, because these kind of materials can be transferred with conveyors. According to TU Delft, « Port and Terminals – Planning & Functional design » 2014, an estimate of total length and width required for the stockpiles can be made with the following equation:

$$V = b\frac{1}{2}h\,I\,m_b$$

V= 1167 000 tons which is the equivalent of 898 000 m3.

Assuming that there is needed a storage capacity for 2 months which is 1/6 of the annual importation (7 000 000 tons).

b : width of stockpile = 20 m (angle of repose of bulk material between 35° and 40°).

h : height of stockpile = 7,5m

l : total length of stockpile

m_b = 1

The length of the piles is l = 12 000 m.

The surface for the bulk storage area that is needed is 240 000m². But it is possible to consider that this storage area represent 70 % of the Bulk storage terminal. Additionally conveyors, technical buildings, administrative buildings, exchange between trucks/terminal, parking are needed to be considered. The total surface is then to 343 000 m², almost 34, 3 ha.

2.4.6 Storage Area Summary

A summary of the storage areas for each cargo type is displayed below (Table 2-7):

Table 2-7- Summary Storage Area

Terminal	Area		Quay Length	Length Behind Quay
	m²	hectares	m	m
Container	1535100	153.51	2130	720
General	97200	9.72	720	135
Ro-Ro	100000	10	290	345
Bulk	343000	343	-	-
Oil	638143	63.8	-	-

3 Design Conditions

3.1 Offshore Waves Statistical Analysis

To analyze the offshore wave conditions a peak over threshold method has been used. As a threshold, a value for the significant wave height is chosen to reach an average of 10 storms per year. Most storms reach the coast under an angle around 24° or 303°. This is clear in the wave and wind roses.

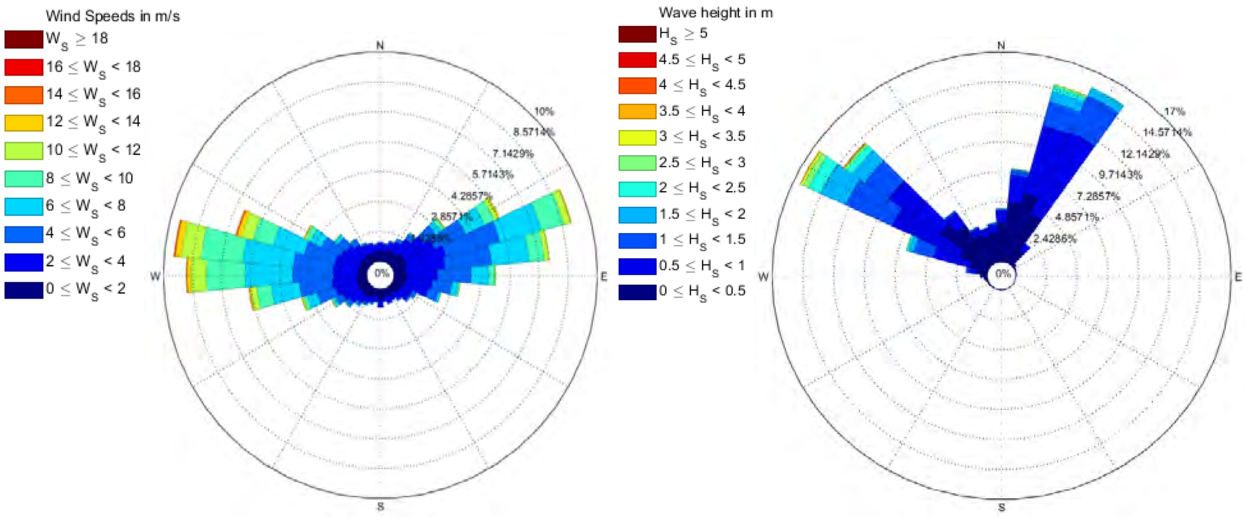


FIGURE 3-1- WIND AND WAVE ROSE

Therefore, the data is split over two quadrants, the northwest and northeast. Both quadrants give an average 10 storms per year.

The resulting significant wave heights under storm conditions H_{ss} are plotted against different distributions. For both directions, the Weibull distribution seems to be the best fit (R^2=0.99). Consecutively the significant wave height has been calculated for the two given return periods R of 100y and 1y. With a trend line under the assumption of the relation $T_p=a*H_{ss}^b$ the corresponding peak period T_p is found. The results from Table 3-1 are found and can be translated to nearshore data with a wave propagation model.

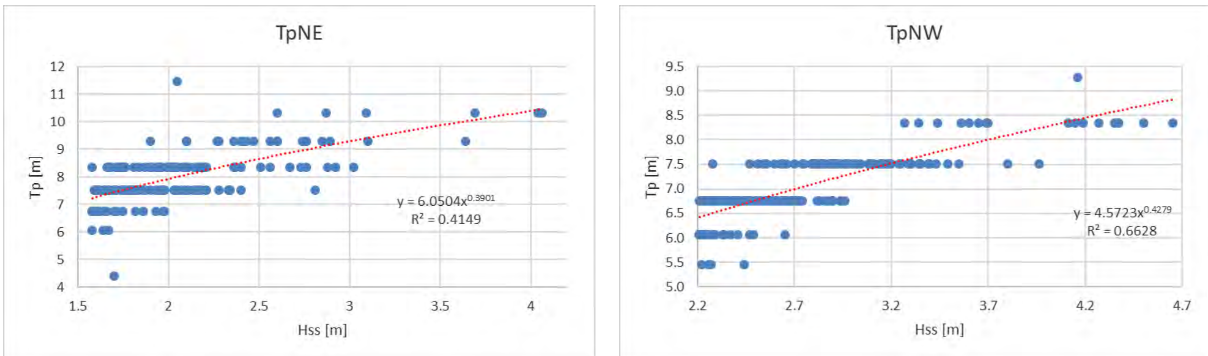


FIGURE 3-2- PEAK PERIOD

TABLE 3-1- STORM CONDITIONS

	NW (303°)		NE (24°)	
Return period	100 y	1 y	100 y	1 y
H_{ss}	5.5 m	3.4 m	4.7 m	2.6 m
T_p	9.5 s	7.7 s	11.1 s	8.8 s

3.2 Geotechnical Analysis

Drilling samples and examined and assessed, in order to determine soil conditions and geotechnical properties for the area of interest. Several samples have been collected in a large area in and outside the harbour. Due to the soil stratigraphy existed under the construction area of the port the soil investigation determined the final layout and the position of the two breakwaters. In the following Figure 3-3 the bathymetry of the area can be seen with the positions of the borehole drilling profiles.

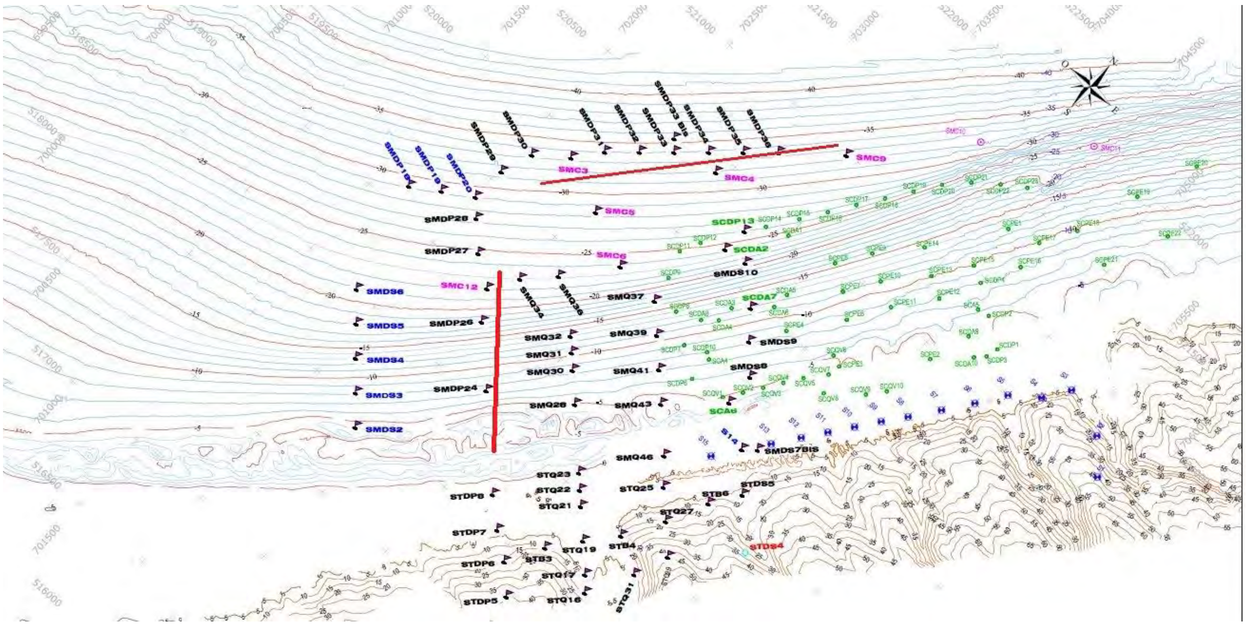


FIGURE 3-3- BOREHOLE POSITIONING AND BATHYMETRY OF THE PORT AREA

In the area of interest we can easily detect from our soil profiles that a relatively thick layer of soft clay is underlying the whole area that may cause a lot of problems in the stability of our structures. In the following Figure 3-4 we can see that from the position SMDP24 up to SMDP26 we have a layer of yellow sand with friction angle of 33 degrees from 12 meters to 10.5 meters and 9.5 meters and in the bathymetry (Figure 3-3) can be represented by the vertical red line.

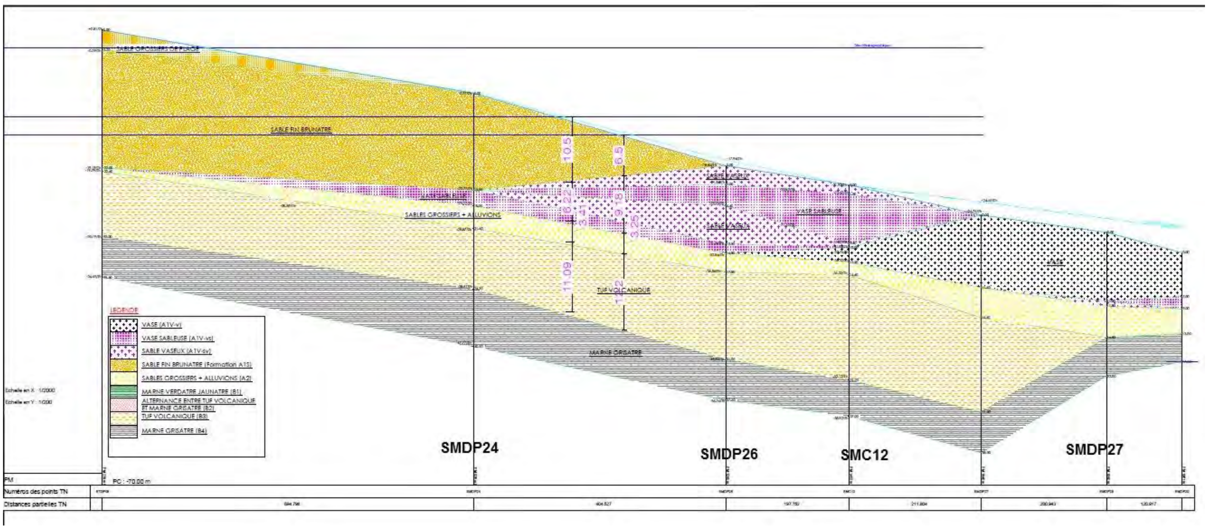


FIGURE 3-4- SOIL PROFILE FROM SPD24 TO SMDP27

The soil conditions of the these profiles seems to be ideal up to SMDP26 where a thick layer of clay appears that is needed to be treated by excavation of the half amount and reinforce the remaining clay.

Also on the horizontal red line in Figure 3-3 where the water depths are approximately -30 to -35 meters it is possible to detect from the given soil profiles that the 1st soil layer is a thick layer of soft clay that lies from SMDP29 up to SMC09 as can be seen in Figure 3-3 and the soft clay is varying from -12 meters to -4 meters as can be seen in the next figure.

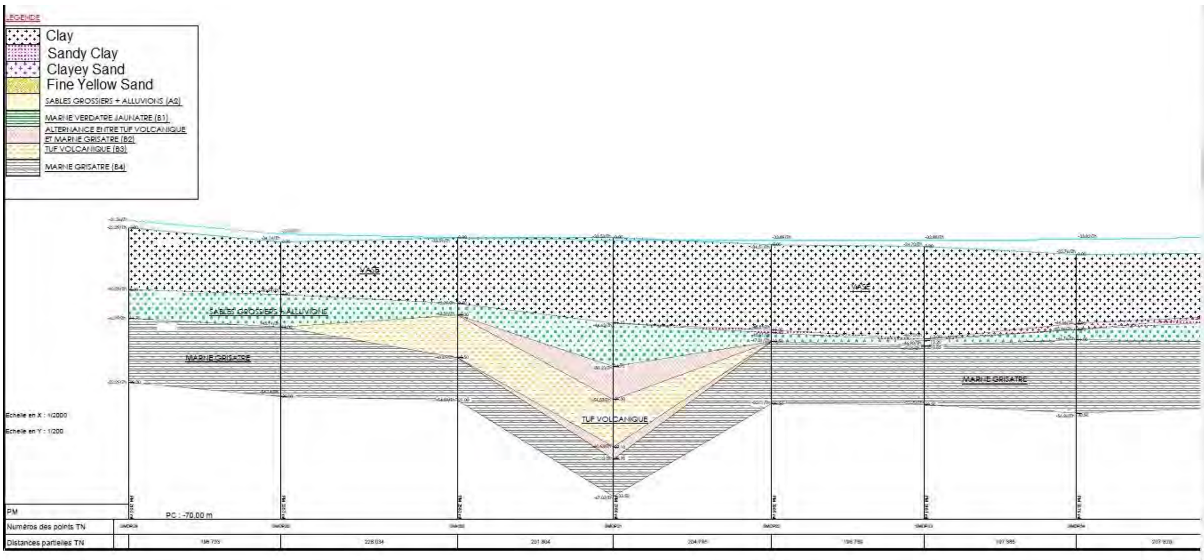


FIGURE 3-5: SOIL PROFILE IN DEEP WATER

3.3 Soil Reinforcement

The current soil conditions where the port is laying are not entirely reliable and after analysing the given geotechnical profiles a solution for a bigger soil bearing capacity are being introduced.

Firstly it is needed to take into consideration that the in the most critical conditions as can be seen in figure 3-5 that there is a layer of soft clay that reaches -9 meters and a combined solution of excavation of half of the soil and replace it with sandy mixtures from the area and reinforce the rest of the soil with rigid inclusions (solid columns). The suggested rigid inclusions should have the following parameters

The rigid columns should «attract » some load share and refine the stress distribution within the soil and the stress to be reduced to an allowable level with regards to the soil bearing capacity or the limiting values of settlements as shown in the following figure.

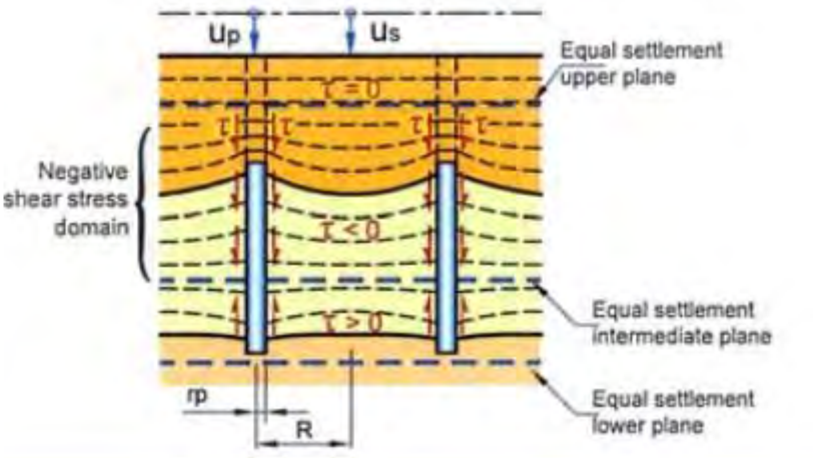


Figure 3-6 Impact of the soil reinforcement rigid inclusions

The impact of the reinforced soil will lead to a set of varied objectives:

- Increase short term stability
- Decrease ultimate value of the acceptable displacements
- Decrease settlement during lifetime
- Reduce soil lateral displacements
- Speed up the consolidation process.

3.4 Sediment Transport

3.4.1 Sediment supply

Most of the year the Oued Kert is dry. During an average year, three flows occur which last no more than 6 days. These flows appear twice in autumn and once in winter. The amount of sediment transported during these flows differs from year to year. On average, the transported fines will be 200,000 t/year with peak years of 2,200,000 t/year. Exceptional events of a supply of 3,000,000 t/year may occur. The transported amount of sand is 50,000 m^3/year on average.

Due to the turbulence induced by the breaking waves near the coast, fines will not have time to settle. The heavier sand particles with a higher fall velocity will have enough time to settle. Therefore, the deposition of fines is expected to be more offshore, where the deposition of sands will be on and near the shore.

3.4.2 Building timeframe
The deposition of a high volume of fines offshore may cause difficulties during the construction of a breakwater. Before the construction of a breakwater, some dredging will have to be conducted. When a high sediment supply occurs from the Oued Kert this dredged part might fill up again. Therefore, there is a maximum building timeframe between the dredging activities and the next flow in the river. Ideally, dredging is conducted after the high flow in winter. Afterwards, the construction of the underwater part of the breakwater con be conducted during spring and summer, before the first high flow in autumn.

3.4.3 Sedimentation inside the port basin
The plume of fine sediment transported offshore will be transported mostly towards the NE, due to the dominating wave direction from the NW. Since the entrance to the harbour is also located on the NE, it seems unlikely that a significant amount of fines will enter the harbour and settle there. Some maintenance dredging will always have to be conducted, but the volume to be dredged is significantly smaller than for a harbour with the entrance on the SW.

Settlement of fines in the access channel towards the entrance of the port needs to be considered. Hard structures are not preferred, therefore likely a sediment trap will be used. The sediment trap will effectively collect the sediment before entering the channel. Use of a trap will be more cost efficient and likely be effective to minimize maintenance dredging. Depending on the final layout of the harbour, a sediment transport model may be used to confirm the effectiveness of the deposition mitigation.

3.4.4 Coastal changes
Before the construction of the harbour the coastline is in equilibrium. With a dominating wave direction from the NW, the longshore sediment transport is directed towards the NE. The construction of a harbour will fully block this transport. The resulting gradient in the transport rate will lead to accretion on the SW and erosion on the NE of the harbour. The accretion causes no problem, as long as it does not reach the offshore end of the breakwater. The erosion on the other hand has to be reduced or mitigated if this starts causing a problem. This can be done by constructing shore-normal or detached breakwaters.

3.5 TOMAWAC: Wave Transfer
We used Tomawac software to propagate the offshore waves to the coast. The input data used within the software are:

- Geometry conditions
- Boundary conditions (contour)
- Bottom friction dissipation (activated)
- Boundary main direction
- Boundary peak frequency
- Boundary significant wave height
- Depth-induced breaking dissipation

- Initial still water level
- Minimal frequency

That data needed has been obtained previously by statistical analysis of the offshore waves. There are two main directions (northeast and northwest) and for each of them there are data for extreme waves and yearly waves. Also for the northeast direction there is data for operational waves.

For each of these combinations we have done two simulations changing the still water level from the higher (0.66m) to the lower level (0m) in order to find the most unfavorable results. Analyzing the results obtained we conclude that the most unfavorable conditions are when the still water level is at a maximum.

The bathymetry given is showed in the picture below (Figure 3-7).

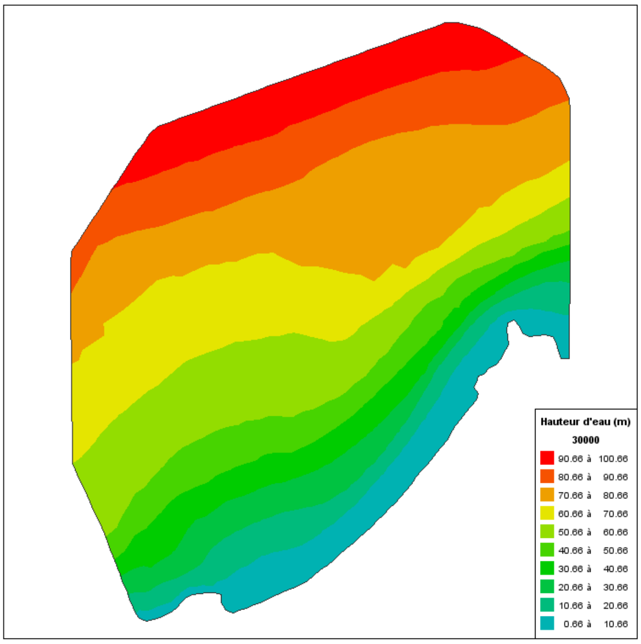
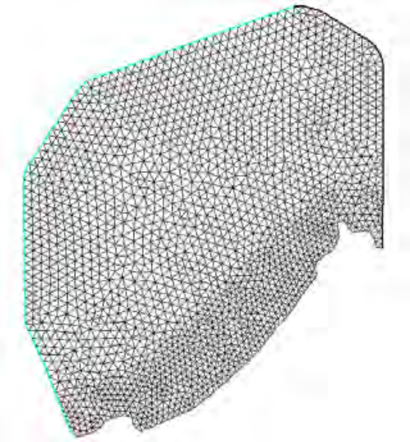
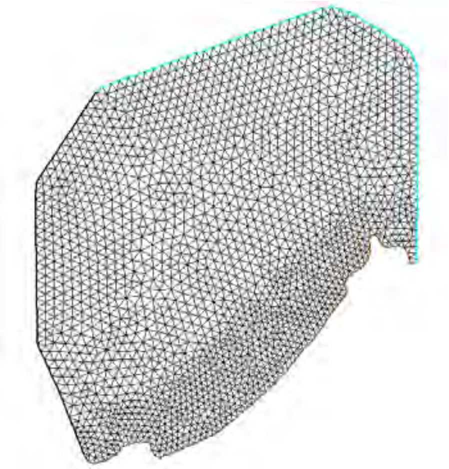
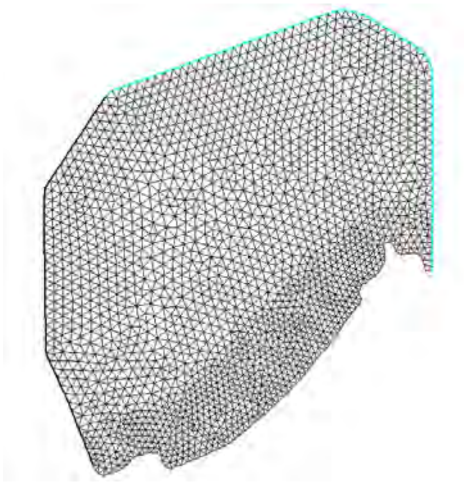
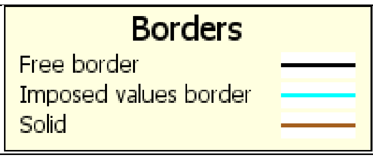


FIGURE 3-7- BATHYMETRY

Boundary conditions for the simulations depend on the main direction of offshore waves are shown in the table below (Table 3-2).

TABLE 3-2- BOUNDARY CONDITION IN TOMAWAC

	Boundary conditions for extreme and yearly waves coming from northwest direction (303º)
	Boundary conditions for extreme and yearly waves coming from northeast direction (24º)
	Boundary conditions for operational waves coming from northeast direction (10º)
	Legend Borders in blue represent the area where the waves are coming, the black borders

	represent a free border and borders in brown represent the coast.

Results of the wave propagation simulation for Extreme Waves are the most critical and they are used for the design of the elements of the port. Also, Operational Waves are important for checking the agitation inside the basin of the port. Those results are showed below.

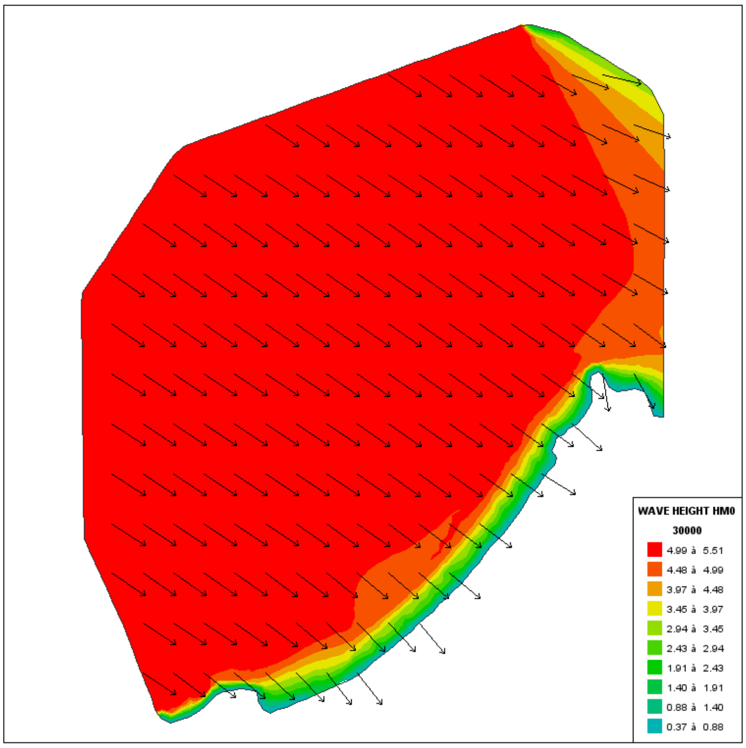


FIGURE 3-8 WAVE HEIGHT AND MEAN DIRECTION FOR NORTH WEST EXTREME WAVES

Input data for Northwest Extreme Waves (100 Year Return Period)					
Main direction (°)	Hs (m)	Tp (s)	Fp (Hz)	Fmin (Hz)	Still water level (m)
303	5,5	9,5	0,1	0,05	0,66
303	5,5	9,5	0,1	0,05	0

See *Appendix C Wave Propagation Profiles* for more graphic information about properties of Northwest Extreme Waves

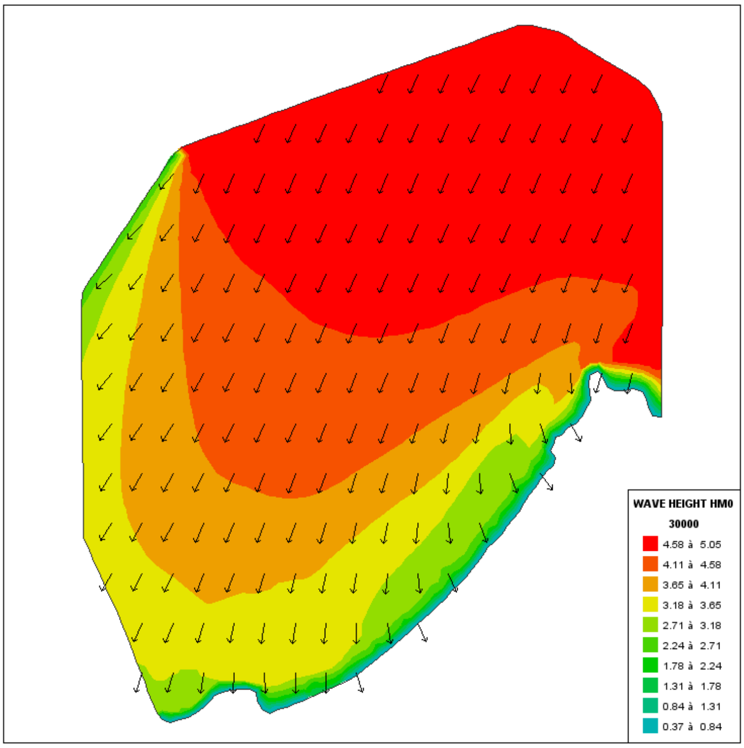


FIGURE 3-9 WAVE HEIGHT AND MEAN DIRECTION FOR NORTHEAST EXTREME WAVES

Input data for Northeast Extreme Waves (100 years return period)					
Main direction (°)	Hs (m)	Tp (s)	Fp (Hz)	Fmin (Hz)	Still water level (m)
24	4,7	11,1	0,09	0,045	0,66
24	4,7	11,1	0,09	0,045	0

SEE APPENDIX 12.2 WAVE PROPAGATION PROFILES FOR MORE GRAPHIC INFORMATION ABOUT PROPERTIES OF NORTHEAST EXTREME WAVES

Input data for Northeast Operational Waves (1 years return period)					
Main direction (°)	Hs (m)	Tp (s)	Fp (Hz)	Fmin (Hz)	Still water level (m)
10	2,6	8	0,13	0,065	0,66
10	2,6	8	0,13	0,065	0

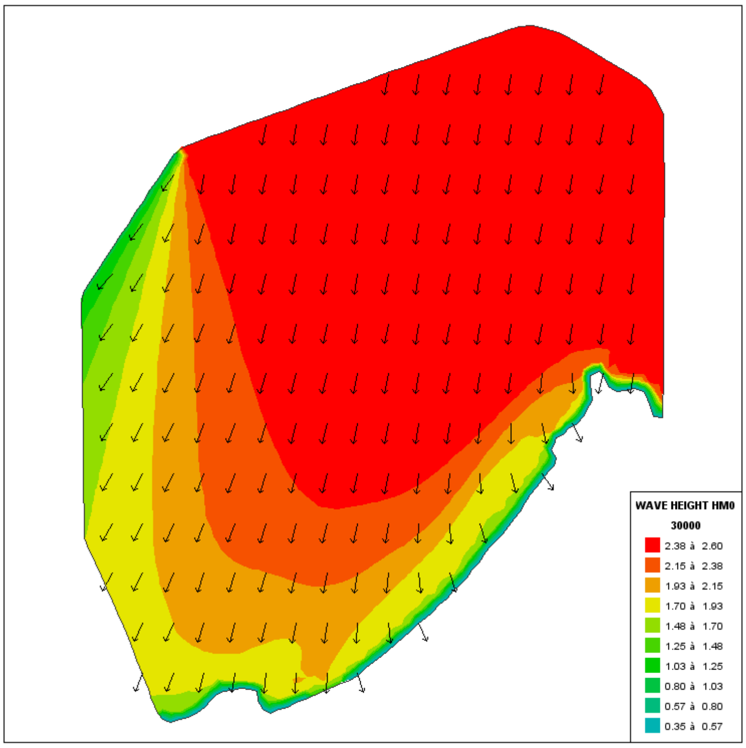


FIGURE 3-10 WAVE HEIGHT AND MEAN DIRECTION FOR NORTHEAST OPERATIONAL WAVES

SEE APPENDIX C WAVE PROPAGATION PROFILES FOR MORE GRAPHIC INFORMATION ABOUT PROPERTIES OF NORTHEAST OPERATIONAL WAVES

SEE APPENDIX D WAVE PROPAGATION YEARLY WAVES FOR RESULTS OF WAVES WITH ONE YEAR RETURN PERIOD

4 Environmental Impact

The construction of the port will have a large impact on the environment. In this section the key environment factors will be defined and their impact will be taken into consideration during the design. We know that some of these points will have no influence in our case but we don't know enough the region and the environment of the project.

4.1 Critical Points

4.1.1 Pollution

Contamination with the oil: impact on the water. It is one the most important risk of pollution. As we could see, oil leaked can arrive easily by accident.

Pollution of the air: the trucks and ships circulation will improve so it will increase the air pollution.

Pollution with the building site: this part can cause lots of pollution. It will depend of the way they will use. For the villages around, it will bring noise pollution too.

4.1.2 Sedimentation

Incidence on the sedimentation: changing sea currents could have a bad influence on them.

4.1.3 Fauna and Flora

Marine Fauna: Some species will be killed by this industry or will leave because of the activity. It will change the environment and can have bad influence for the future.

Cape Three Forks: protected zone not really far away, the risk is that the pollution arrived in this zone which is protected.

4.1.4 Socio-economic Activities

Transformation of the landscape: actually, this is countryside with forest. This change can impact tourism for example.

Fishermen: the noise will cause all the animals to flee. For villagers who live here, it will make a change in their way of life.

Increase of the employment: the port will create new jobs which will improve the local economy and develop villages around. With the population increase, new buildings like hospital and schools will certainly appear and it will bring some modernity in the area.

4.2 SWOT Analysis

SWOT analysis (alternatively SWOT matrix) is
an acronym for strengths, weaknesses, opportunities, and threats and is a
structured planning method that evaluates those four elements of an
organization, project or business venture.

Strengths: characteristics of the business or project that give it an advantage over others

Weaknesses: characteristics of the business that place the business or project at a disadvantage relative to others

Opportunities: elements in the environment that the business or project could exploit to its advantage

Threats: elements in the environment that could cause trouble for the business or project

TABLE 4-1- SWOT MATRIX

	STRENGTHS	WEAKNESSES
INSIDE	• Improve the local economy • Creation of jobs	• Air pollution • Impact on the local species • Transformation of the landscape • Change on the local economy
	OPPORTUNITIES	THREATS
OUTSIDE	• Relation with other regions/countries	• Oil leaked • Storm

The most harmful impact that this port can have are:

A. Oil leaked

B. Air pollution
C. Impact on the local species
D. Impact on the local populations
E. Transformation of the landscape

4.3 Risk indicator: Criticality

For each risk, the criticality must be indicated. The criticality can be calculated with this formula: C = P x G x D

- Probability: The higher the number, the higher the probability which the phenomenon will happen.
- Gravity: Higher the number is, higher the probability gravity is.
- Non-detection: Higher the number is, higher the non-detection is.

TABLE 4-2- RISK INDICATORS

Risk	Probability : P (1 to 5)	Gravity : G (1 to 5)	Non-detection : D (1 to 3)	Criticality
A. Oil leaked	3	5	3	45
B. Air pollution	4	3	1	12
C. Impact on the local species	4	3	2	24
D. Impact on the local populations	4	2	1	12
E. Transformation of the landscape	5	3	1	15

In the following diagram (Figure 4-1) we can see the criticality for each risk can be seen:

- C1 (0 to 10): the risk level is acceptable
- C2 (11 to 29): It is still acceptable but the risk needs to be controlled
- C3 (30 60): the risk is not acceptable. These risks must be avoided

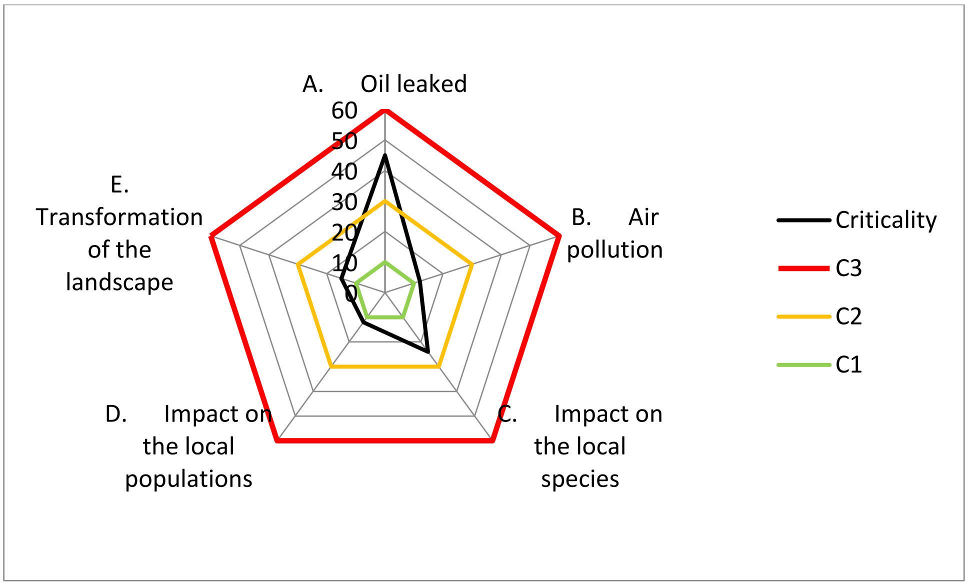


FIGURE 4-1- RISK INDICATORS DIAGRAM

It can be seen that the most important risk is oil. This risk has an influence on the local population and species, on the water pollution and can impact protected

5 Layout Discussion

5.1 Navigational Dimensions

To help determine the dimensions and orientation of the layout, information on the navigational requirements for ships entering the port is needed.

5.1.1 Turning Circle

The turning circle should be the highest value of either:

- 2 times the design ship length of the least manoeuvrable ship.
- 1.7 times the design ship length of the most manoeuvrable ship.

The least manoeuvrable ships are crude oil tankers with a design ship length of 345m, this gives a turning circle of 700m (690m).

The most manoeuvrable ships are container motherships, with a design ship length of 400m. This gives a turning circle of 680m.

Therefore the diameter of the turning circle is 700m.

This turning circle is specific to ships which are manoeuvring under tug boat assistance (Thoresen, 2003), and the turning circle should be placed inside the protected harbour zone. The entire turning circle will need to be dredged for the largest design ship draught multiplied by a factor of 1.2, which gives a depth of 26.4m (1.2 multiplied by the design draught of 22m for a crude oil tanker).

5.1.2 Navigation Channel

To minimise dredging, a curved channel will enter our harbour from the North East, the curve is shallow with a change in orientation less than 10 degrees, this will minimise the width of the channel needing to be dredged.

The channel width has been designed to allow 1 way traffic due to the number of ships entering the port per day being only 1, therefore a two ways channel is not necessary.

The channel width consists of several components (Thoresen, 2003):

- Manoeuvring width equal to 2 times the largest design ship breadth of 60m.
- Yaw width equal to half of the design ship breadth.
- Channel clearance equal to 1.5 times the design ship breadth.

This gives a navigational channel width of 240m.

When the navigational channel enters the port there should be a clearance of 50m either side to the nearest breakwater, therefore the harbour entrance should be 340m.

The navigation channel should have a minimum length to allow a stopping distance for ships, this should be in total 8 times the largest design ship length, and 4 times this length should be inside the harbour. Giving a total stopping distance of 3200m, and inner stopping distance of 1600m.

5.1.3 Depth of Berthing Locations

The depth of water at the locations of each berth should be 1.2 times the design draught of the respective ship using the berth (Thoresen, 2003), the results are displayed below (Table 5-1):

TABLE 5-1-SUMMARY DEPTH OF BERTHING LOCATIONS

Cargo	Berth Depth
-	m
General	14.4
Ro-ro	9
Bulk	21.96
Liquid (Oil)	26.4
Liquid (Product)	16.2
Container Feeder	16.8
Container Mother	19.2

5.2 Layout Design

For the layout of the port some criteria are set based on the design criteria and design conditions.

Due to the poor geotechnical conditions near the Oued Kert (see the red area in **Error! Reference source not found.**), the harbour will have to be built at a location of 2000 m north of the river (the yellow part near the coast in **Error! Reference source not found.**). In this way the southwestern, primary breakwater can be built in such a way that a minimal amount of dredging and some small soil reinforcements are necessary (see **Error! Reference source not found.**). Furthermore, from the **Error! Reference source not found.** one can conclude that the significant waves come from the northeast and northwest. It must be taken into account that the entrance of the harbour shelters the quays from these wave angles. Another criterion for the location of the harbour is the depth of the breakwater. In the deepest parts, a vertical breakwater can be chosen. To limit the costs, the deepest breakwater will have to be no deeper than 30 m. For the shallower parts a rubble mound breakwater is a better option.

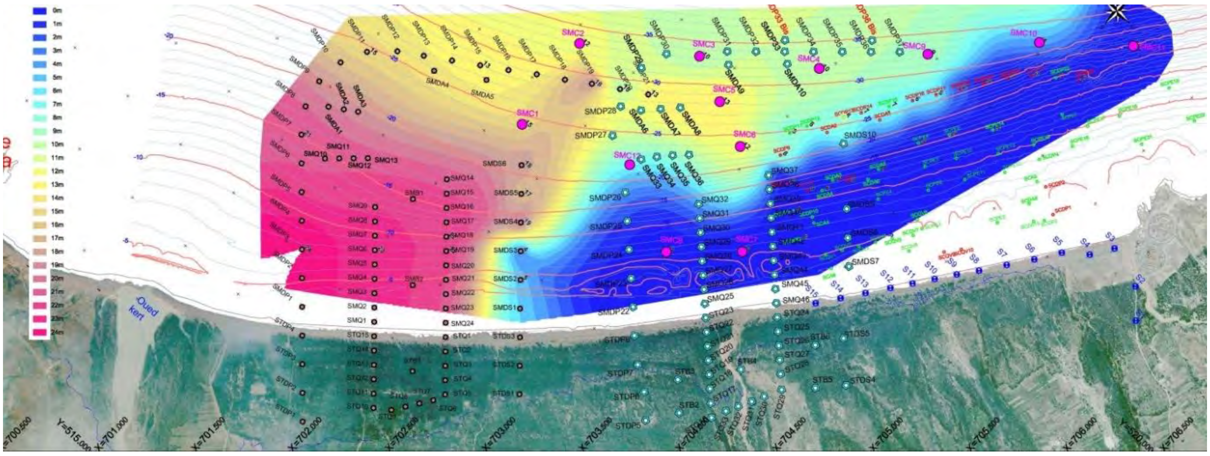


FIGURE 5-1: GEOTECHNICAL CONDITIONS AT MOUTH OF OUED KERT

For the general dimensions of the harbour, some numbers are calculated in **Error! Reference source not found.**. The turning point should have a diameter of at least two times the size of the least manoeuvrable ship (700 m). The entrance should be of the size of the access channel plus 50 m on both sides to accommodate the toe of the breakwaters and to reduce the risk of a ship collision with parts of the breakwater above and below the water level. Further dimensions of the quay and channel can be found under **Error! Reference source not found.** and **Error! Reference source not found.** respectively. No quay should be placed at the end of the entrance channel to limit the risk of a ship collision with the quay.

The liquid and bulk terminals can be constructed inside the harbour along the deep breakwater. This reduces the required amount of dredging for the deep draught of these vessels. The container terminals can be placed in further relatively deep water, to limit dredging, where the Ro-Ro and general cargo can be accommodated in the shallower parts of the harbour. Another measure to minimize dredging is to go as far offshore as possible, but taking the maximum depth of the breakwater into account.

With an entrance to the northeast, as asked for by the client, enough room for future expansion of the harbour has to be available between the entrance and the headland north of the port.

6 Finalisation of Layout

6.1 Wave Agitation

Once a final layout was found, it had to be checked to ensure the wave agitation in the port would not be too high for berthing ships. For this, we used the software BlueKenue.

6.1.1 General criteria

The following general criteria was used for the simulation:

TABLE 6-1: GENERAL WAVE CRITERIA

Simulation	Hs (m)	Tp (s)
WavA	5,5	9,5
WavB	4,7	11
WavC	2,6	9
WavD	2,6	8

Lowest Tp to model is Tp = 2.6 s and average depth = 20 m

- Tp = 8s → Tmin = 4s and Tmax = 10.4 s
- Lp = 89 m and Lmin = 25 m
- Number of periods = 5
- Direction of wave propagation = 280°
- Number of directions = 7

Mesh size was calculated from the requirement of at least three nodes per wavelength for the lowest period (Lmin). Lmin/3 = 25m/3= 8.33m, therefore a mesh size of 8m was used.

Wave thresholds may be considered as follows from the design brief:

- Hs = 1.00m for oil and coal berths
- Hs = 0.70m for container berths
- Hs = 0.50m for general cargo and ro-ro berths

TABLE 6-2: REFLECTION COEFFICIENTS

	Open Berth	Close Berth
Reflection coefficient	0.2	0.9

6.1.2 Bathymetry

It is necessary to adjust the bathymetry layout because there are some dredged areas.

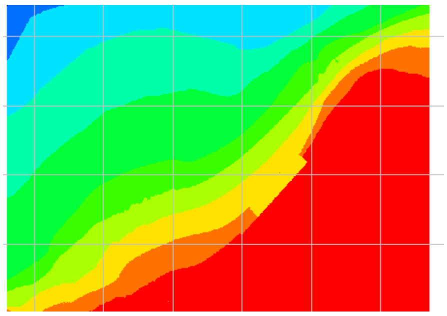


FIGURE 6-1: LAYOUT 1 AND 2 BATHYMETRY

6.1.3 Layout 1

The first design utilised a composite breakwater along the outer walls, and a rubble mound breakwater for the close to shore sections. The entire quay is solid and closed.

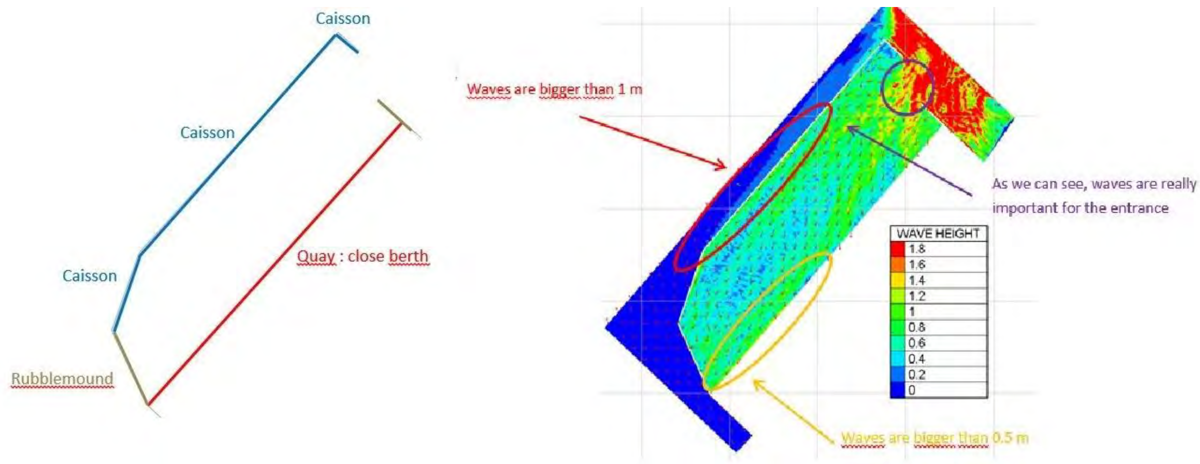


FIGURE 6-2: LAYOUT 1 STRUCTURE AND RESULTS

It can be seen that the wave thresholds are not respected. Therefore the layout must be changed and tested.

6.1.4 Layout 2

The following changes have been made to layout 1 to reduce the wave agitation:

- Change the design of the entrance
- Decide to put a rubble mound instead of the left hand side caisson
- Decide to use open berth for Ro-Ro and general ships and closed berth for container cargo ships. Open berths will decrease the reflections in the port

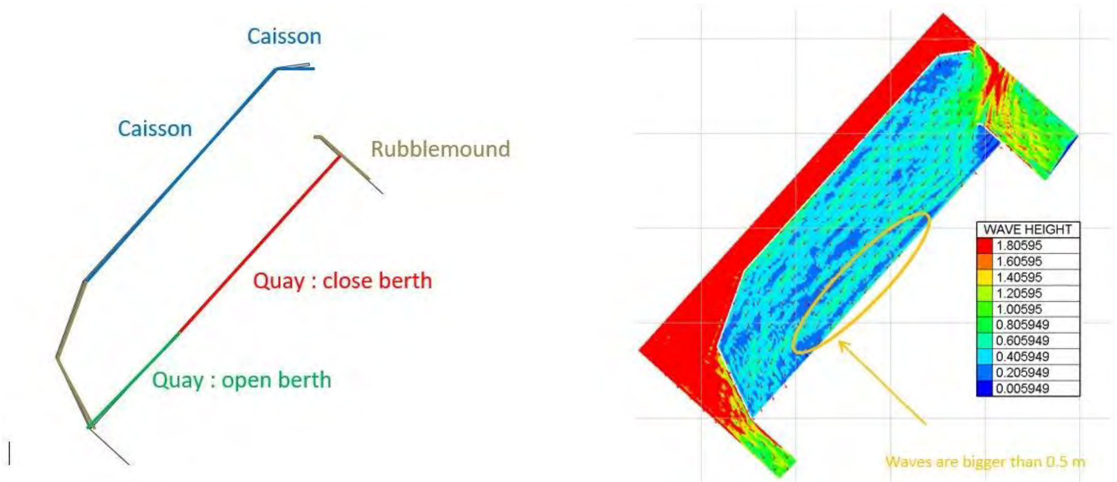


FIGURE 6-3: LAYOUT 2 STRUCTURE AND RESULTS

Again the wave height threshold has been reached, therefore the design needs to be interated further.

6.1.5 Layout 3

Layout 2 has been iterated to further reduce wave agitation:

- Change the design of the entrance to the port
- Dredged basin for ro-ro and general cargo berthing

Implementing these changes it is necessary to change the dredged area and consequently the bathymetry.

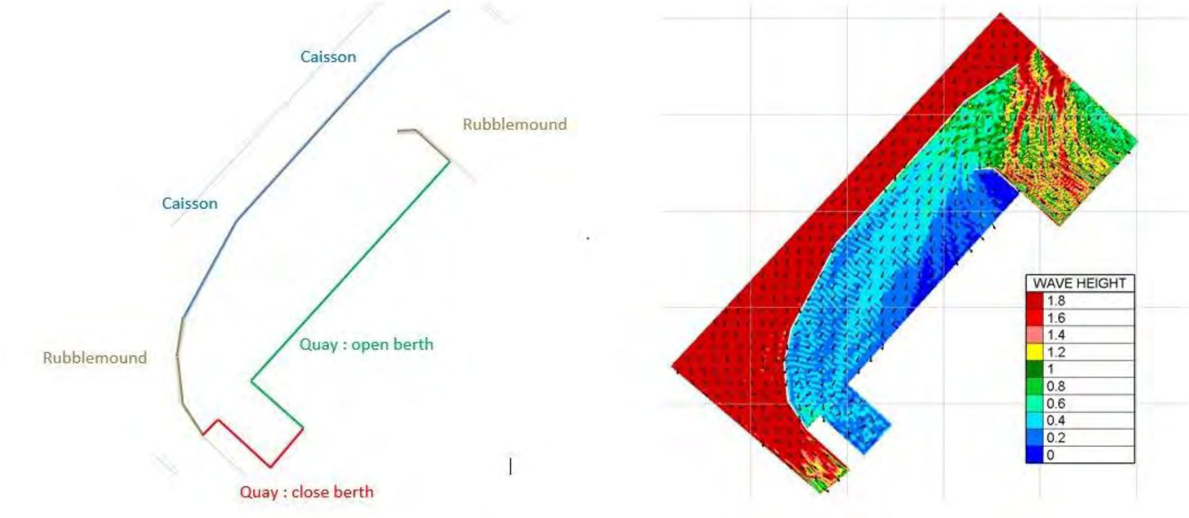


FIGURE 6-4: LAYOUT 3 STRUCTURE AND RESULTS

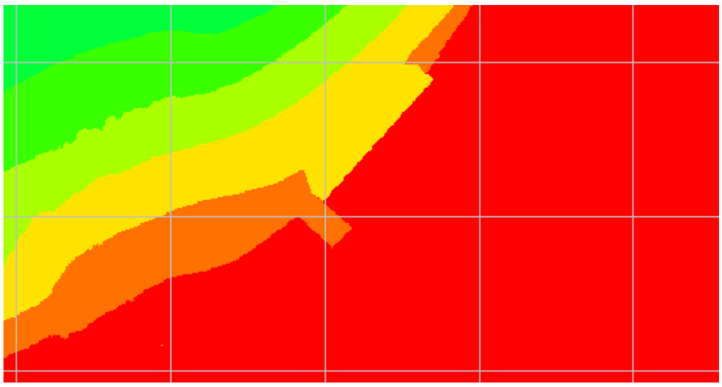


FIGURE 6-5: LAYOUT 3 AND 4 BATHYMETRY

From the results it can be seen that this layout layout is much better than the previous iterations in terms of wave agitation inside the port. The wave heights threshold has not been exceeded anywhere inside the port, therefore it is safe for berthing ships.

6.1.6 Layout 4

To optimise this design, the type of quay wall for each berth has been modified to include an open berth at the southern end of the port to minimise the reflections in that area.

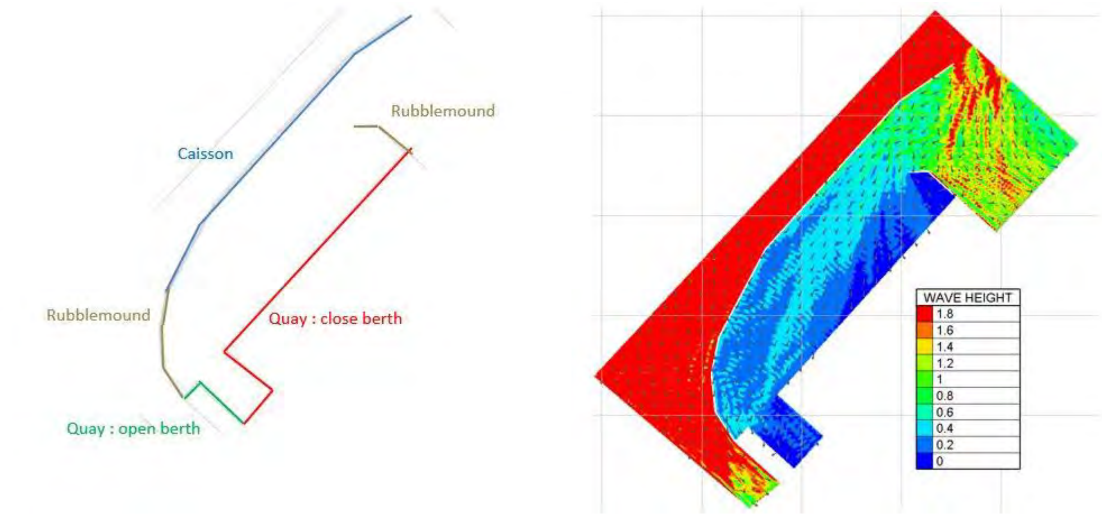


FIGURE 6-6: LAYOUT 4 STRUCTURE AND RESULTS

This has led to a final design of the ports layout as seen below:

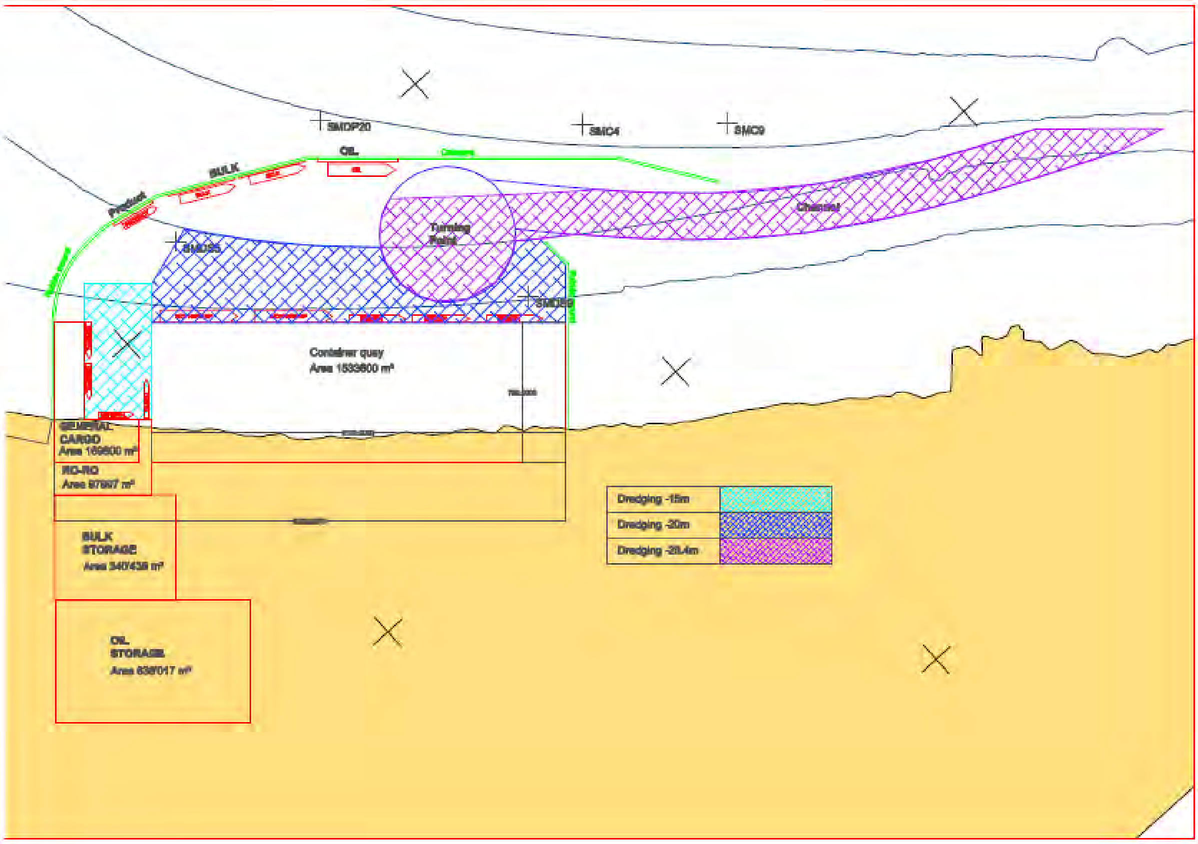


FIGURE 6-7: LAYOUT FINAL DESIGN

6.2 Dredging

For the dredging, 3 different areas had been selected with differing depths needed to be dredged: dredging until 15m, until 20m and until 26.4m

Those depth had been selected to provide enough space for the ships and a 1.2 factor had been added.

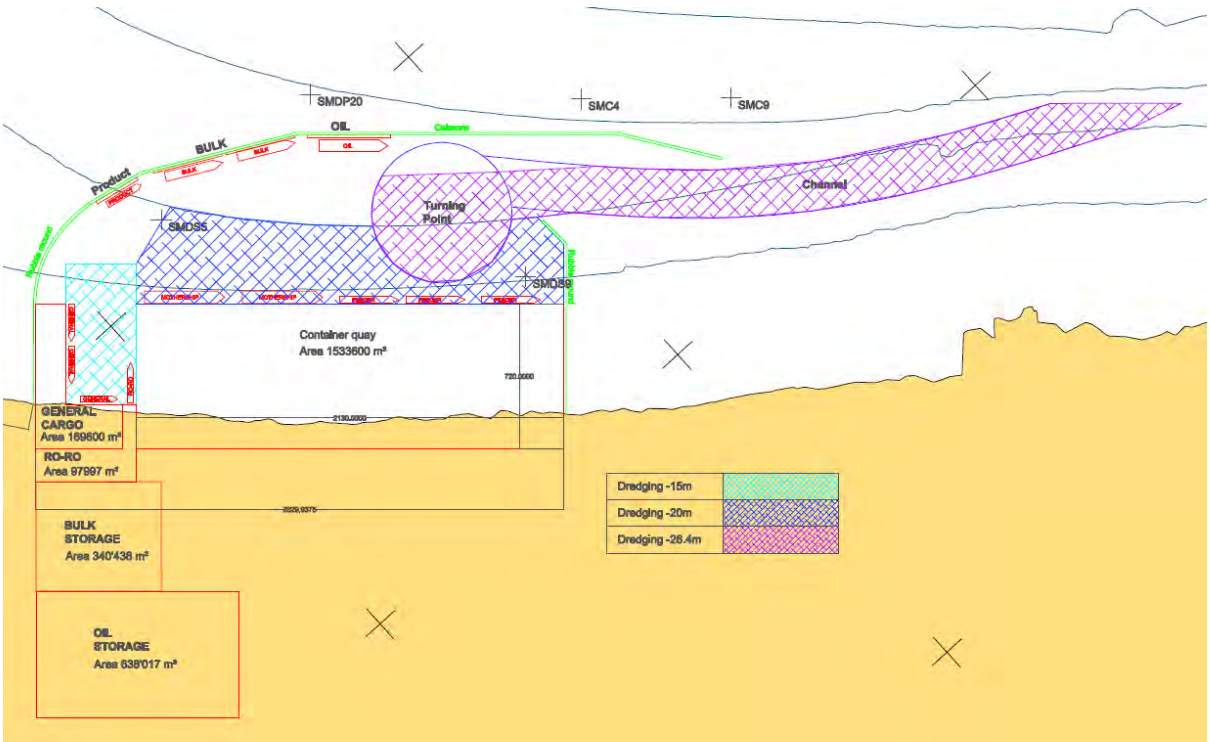


FIGURE 6-8: LAYOUT 3 AND 4 DREDGED AREAS

Using the bathymetry and the areas given by Autocad it permitted to calculate the exact amount of dredging.

After calculation, the total of dredged cubic meter is: 13 748 000 m^3

	A	B	C	D	E	F	G	H	I	J	K
1	Dredging -20 m					Dredging -15 m					
2											
3		Area (m^2)	Depth to dredge (m)	Cubic to dredge (m^3)			Area (m^2)	Depth to dredge (m)	Cubic to dredge (m^3)		
4	A1	713 506,0	5,5	3 924 283,0		A1	245 000,0	7,5	1 837 500,0		
5											
6											
7			Total	3 924 283,0				Total	1 837 500,0		
8											
9	Dredging -26 m					Under BW					
10											
11		Area (m^2)	Depth to dredge (m)	Cubic to dredge (m^3)			Linear (m)	Width (m)	Area (m^2)	Depth to dredge (m)	Cubic to dredge (m^3)
12	A1	1064722	5,0	5323610		Caissons	3533	94,45	333691,85	5	1668459,25
13						Rubble mound	2106	94,45	198911,7	5	994558,5
14											
15			Total	5 323 610,0						Total	2 663 017,8
16											
17							TOTAL amount of dredging (m^3)		13 748 410,8		

FIGURE 6-9: VOLUME OF DREDGED MATERIAL CALCULATIONS

7 Structural Design

7.1 Rubble Mound Breakwater Design

For the shallow parts of the breakwater normal to the coastline, partially protecting the reclaimed area, a rubble-mound breakwater can be constructed. This is cost-efficient compared to the composite breakwater with caissons for the required part in deeper water. From the TOMAWAC and ARTEMIS simulations same basic parameters can be found: $H_s = 5\ m$, $T_p = 9.55\ s$ and $\beta = 60°$ (primary breakwater).

7.1.1 Rock

A first calculation is done for regular rock with a density of $\rho_s = 2.65\ t/m^3$. Since $\xi_m < \xi_{m,cr}$ and $0.03 < s < 0.06$, plunging waves occur. Therefore, the following Van der Meer formula (Equation 7-1 can be used:

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

EQUATION 7-1- VAN DER MEER FORMULA

In this formula, the following design values have been used: $c_{pl} = 5.5$, $P = 0.4$, $S = 2$ (needs no repair), $N = 8h * 3600s\ /\ T_p \approx 3000\ waves/storm$. For a slope of 1:2 this gives a stone mass of $M_{50} = 22.6\ tons$, which will be too expensive. A slope of 1:4 leads to $M_{50} = 8.0\ tons$, but this slope will be to gentle to leave enough depth for navigation.

7.1.2 Concrete Armour Units

An alternative is the use of concrete armour units instead of rock. For the design, a high-quality concrete of $\rho_c = 2.4\ t/m^3$ is considered. Recently it has gotten more common to use armour units in a single layer.

A first calculation is done with Core-Loc (2:3 slope). Van der Meer gives a design calculation for no repair needs of $\frac{H_s}{\Delta D_n} = 2.8$, leading to Core-Loc units of $D_n = 1.3\ m$, $V = 2.4\ m^3$ and $M_{50} = 5.7\ tons$. However, this unit is very slender and is more likely to break due to rocking. Most projects prefer to use more bulky units for their breakwater design.

A calculation for Accropode ($\frac{H_s}{\Delta D_n} = 2.5$) gives units of $D_n = 1.5\ m$, $V = 3.3\ m^3$ and $M_{50} = 8.0\ tons$. This is a normal, not too expensive size. In practice Accropodes are made in standard molds. In this case that would lead to $D_n = 1.6\ m$, $V = 4.0\ m^3$ and $M_{50} = 9.6\ tons$. This means that a cheaper concrete can be used, up to $\rho_c = 2.0\ t/m^3$.

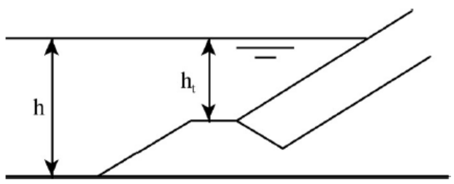


FIGURE 7-1- DIMENSION OF TOE

7.1.3 Toe

For a toe of 1.5m high at a water depth of 20 m, $ht/h = 0.9$ (see **Error! Reference source not found.**), $\frac{H_s}{\Delta D_{n50}} = 7.0$ is required for stability. This leads to rocks of $D_{n50} = 0.45\ m$. The toe must be 2 to 3 stones high and 3 to 5 stones wide. For Accropode it is very important that the toe is well connected to the first row. If this is not the case, the first row can displace and will no longer interlock correctly with the rest of the breakwater, leading to more displacement. It is advised to place two rows of Accropodes at the toe.

7.1.4 Overtopping

Overtopping has been calculated according to the EurOtop II manual. For this rubble-mound breakwater a mean discharge of $q = 1\ l/s/m$ is allowed for no damage to occur. Since the area behind the breakwater is used for storage and economic activities zero overtopping might be required ($q = 0.4\ l/s/m$). For the calculation, the following formula (Equation 7-2) is used:

$$\frac{q}{\sqrt{gH_{m0}{}^3}} = \frac{0.026}{\sqrt{\tan(\alpha)}}\gamma_b\xi_{m-1,0} * \exp\left(-\left(2.5\frac{R_c}{\xi_{m-1,0}H_{m0}\gamma_b\gamma_f\gamma_\beta\gamma_v}\right)^{1.3}\right)$$

EQUATION 7-2- OVERTOPPING FORMULA

The slope for Accropodes is 3:4. Furthermore $\gamma_b = 1$ (no berm), $\gamma_f = 1$ (concrete), $\gamma_v = 1$ (no crown wall) and $\gamma_\beta = 1 - 0.0033|\beta|$ (with $\beta = 0°$ in flume and $\beta = 60°$ or more in real situation). Designing for less than $1\ l/s/m$ of overtopping leads to a free board of at least $R_c = 7.5\ m$.

To reduce the overtopping at the reclaimed area the crest will be made 6 Accropodes wide with 2 Accropodes at the landward slope, according to given recommendations for this type of armour unit.

7.1.5 Filter layer

Accropodes of $4.0\ m^3$ lead to an armour thickness of 2.05 m and require an under layer of at least 1.31 m. This under layer should be designed according to the rules of Terzaghi to function as a filter. Every layer must be at least 0.4 m thick. Therefore, two under layers are placed under a ratio for D_{n50} of 1:2.2-2.5 (given for Accropode). The core is scaled in the same way.

7.1.6 Wave flume test

For a wave flume test these values have been scaled (1:75), as given in Table . Due to the unavailability of certain materials, a small deviation from the design is used. The crest and landward slope are covered with Core-Loc units, because there were not enough Accropodes available. Also the double layer of Accropodes at the toe was replaced by the rocks fromTable , due to the same reason. A side view of the situation in the wave flume is given in.

TABLE 7-1 - SCALED DIMENSIONS

Scale 1:75	Real size	Scaled
T_p	9.55 s	1.1 s
H_{ss}	5.0 m	7.0 cm
h	20 m	27 cm
h_{toe}	1.5 m	2.0 cm
R_C	8.5 m	11 cm
$d_{accropde}$	2.74 m	3.65 cm
$M_{accropde}$	16.9 tons	44.01 g
$V_{accropode}$	7.1 m^3	16.7 cm^3
$d_{n50,filter}$	0.45-0.75 m	6-10 mm
$d_{n50,core}$	0.3-0.45 m	4-6 mm
$d_{n50,toe}$	1.62 m	2.16 cm
$M_{50,toe}$	11.3 tons	27.77 g
density	0.15 units/m^2	821.8 units/m^2

FIGURE 7-2- SIDE VIEW WAVE FLUME TEST

The result of the wave flume test was satisfactory. Up to an agitation of 90% of the design wave height, no rocking of any units was observed. At the design wave height, the rocks for the toe started rocking, but no displacement of units occurred. Even at 120% of the design wave height no failure, nor displacement occurred. This is probably due to the overdesigned armour units in the wave flume (7 m^3 instead of 4 m^3). Also, the overtopping was very small. Just small splashes of water drops were observed, estimated to be below $1\ l/s/m$ in a real situation.

One can conclude that smaller armour units can be used, probably the previously calculated 4 m^3 units. Also, the freeboard is high enough, since in the real situation the significant waves will be obliquely incident. The coarser core material increases the dissipation of energy and have a positive effect on the overtopping.

A second test with a lower packing density of the armour units (745.5 units/m^2) lead to failure of the construction at the design wave height. Probably a density between the two used values is optimal.

7.1.7 Final design

For a final design, a combination of the calculated values and the observations from the breakwater in the wave flume is selected. This leads to the design in Figure 7-3: Rubble Mound Breakwater Design:

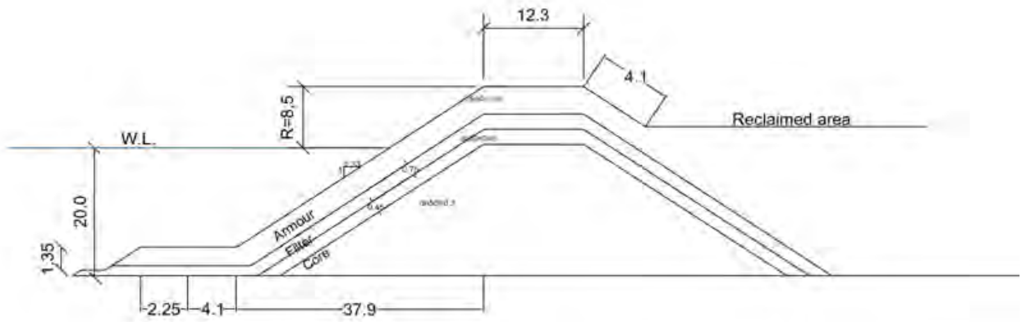


FIGURE 7-3: RUBBLE MOUND BREAKWATER DESIGN

7.2 Vertical Breakwater Design

7.2.1 Vertical Breakwater

Vertical caisson breakwaters will be utilized as the primary breakwater to provide protection for the harbour as well as berthing structures for the bulk cargo and oil terminal. Vertical caissons are preferred over rubble mound breakwaters in this case because they are more economic after a depth of 25m, as seen in Figure 7-4, as well as provide a deep berthing structure for the larger vessels while fulfilling functional requirements, including providing surface for access to the quay, conveyors for the bulk cargo, and sufficient height to minimize overtopping. Floating caissons will be used and can be filled with dredged material and placed on the rubble mound foundation which is suitable for the less firm soil conditions. The berm extension is intended to increase the foundation area and by means of this the resultant of the vertical loads is kept within the core of the structure's cross-section.

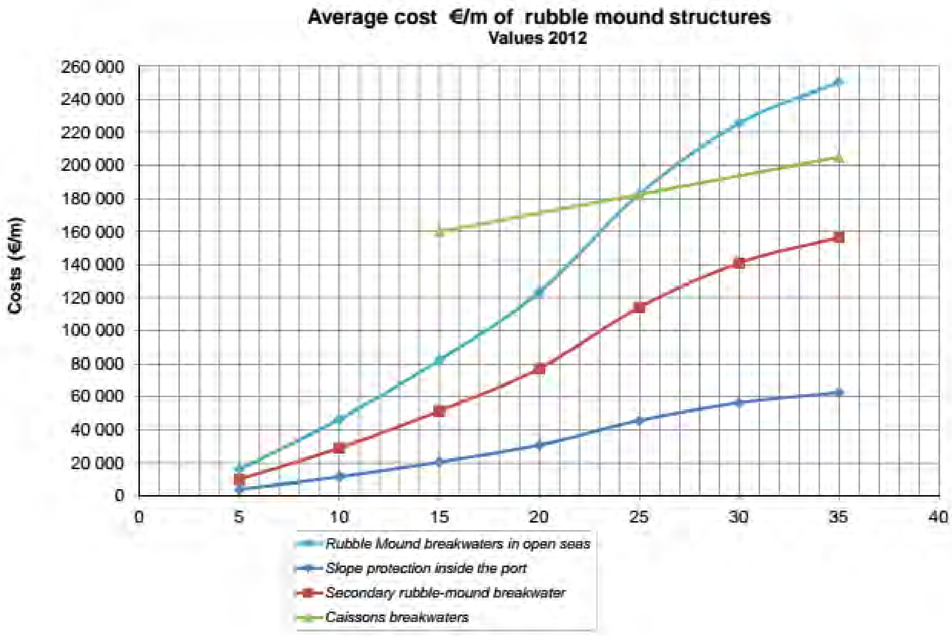


FIGURE 7-4: COST COMPARISON FOR VERTICAL AND RUBBLE MOUND BREAKWATERS

7.2.2 Initial Dimensions

The PROVERBS parameter map, shown in Figure 7-5, was utilized to design for no impulsive forces on the breakwater. This map utilizes non-dimensional input parameters which allow for design decisions to be made at the corresponding level.

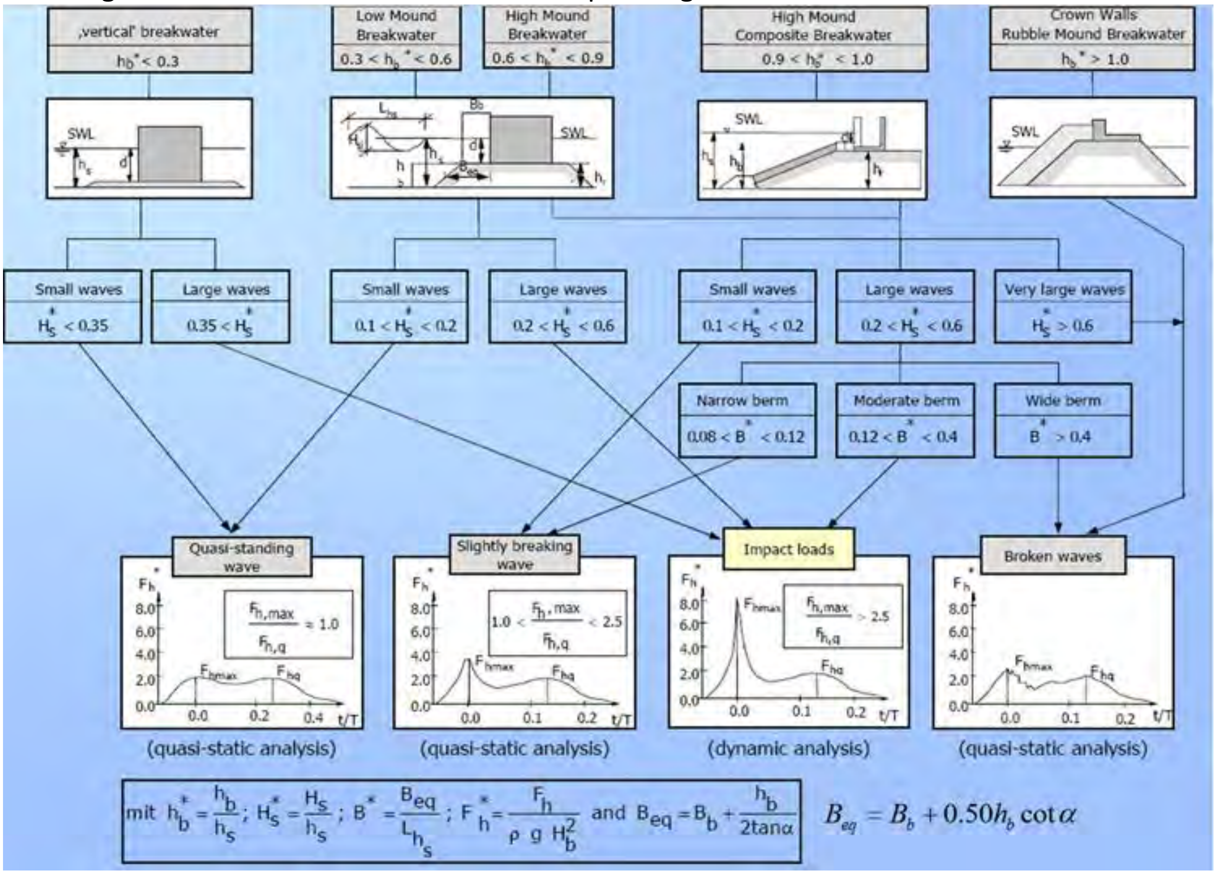


FIGURE 7-5: PROVERBS PARAMETER MAP (OUMERACI ET AL 2001)

At the first decision level, the relative height of the rubble mound foundation h_b* governs the type of structure from the wave loading point of view, distinguishing between either a vertical, composite or crown wall on a rubble mound breakwater. h_b* is given by the equation h_b*=h_b/h_s, where h_b is the height of the rubble mound and h_s is the water depth at the toe of the mound.

At the next two decision levels following the parameter map, the loading case is determined using the relative wave height H_s*, which is given by the equation H_s*=H_s/h_s where H_s is the significant local wave height. Parameters used and results are given below in Table 7-2 Table 7-3 respectively and highlighted in the PROVERBS map.

7.2.3 Stability: Goda Method

In order to assess the horizontal and vertical loads needed to assess stability and necessary dimensions for the vertical breakwater, the Goda method was used (Goda, 2000). Results are shown below in Table 7-4 and illustrated in **Error! Reference source not found.**. Explanation of variables and formulas are given in the Appendix E.

Once the pressure figure around the caisson was defined, the resulting horizontal force F_H, the uplift force F_U and the resulting overturning moments around the heel of the structure, M_H and M_U, were calculated to assess the safety factors against sliding and overturning.

Dimensions were adjusted until acceptable safety factors (>1.2) were obtained. Overall dimensions and safety factors are given below in Figure 7-4.

TABLE 7-2: PARAMETERS USED FOR VERTICAL BREAKWATER DESIGN

h_s	29.00	m
H_s	5.50	m
T_p	5.00	s
β_o	15.00	deg
β_o	0.26	rad
γ_{H20}	1.03	ton/m^3
μ	0.60	
kh	4.67	
Lo	39.00	m
L	39.00	m
L/h	0.74	
tan(α)	0.50	

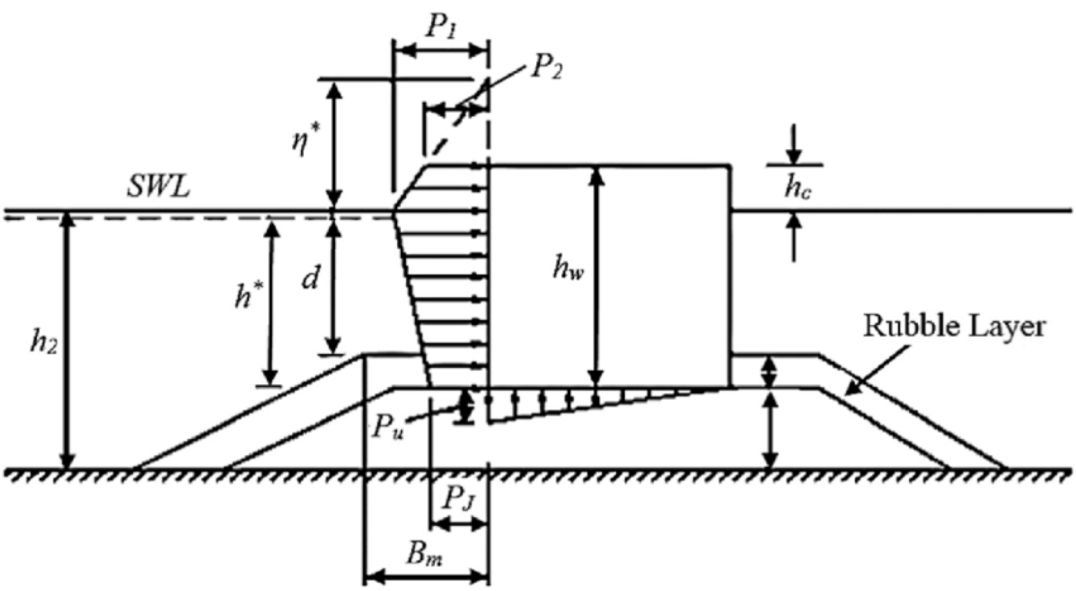


FIGURE 7-6: ILLUSTRATION OF DIMENSIONS AND PRESSURE FIELDS USED IN GODA FORMULA

TABLE 7-3: VERTICAL BREAKWATER DIMENSIONS AND DETAILS

d	25.0	m
h_b	4.0	m
h'	27.50	m
h_c	4.0	m
B	15.00	m
B_b	2.0	m
h_b*	0.14	
H_s*	0.17	
B*	0.06	
B_{eq}	5.00	m
Total Height	32.50	m

Table 7-4: Goda's Method Calculations

Goda's formulae		
H_{max}	9.21	m
$H_{1/3}$	5.12	m
η*	13.58	m
p_1	10.34	
p_2	3.94	
p_3	4.39	
p_4	3.91	
p_u	1.77	
P	287.70	
U	26.52	
MU	212.20	
$S_{f.sliding}$	1.47	>1.2
$S_{f.turn}$	1.60	>1.2

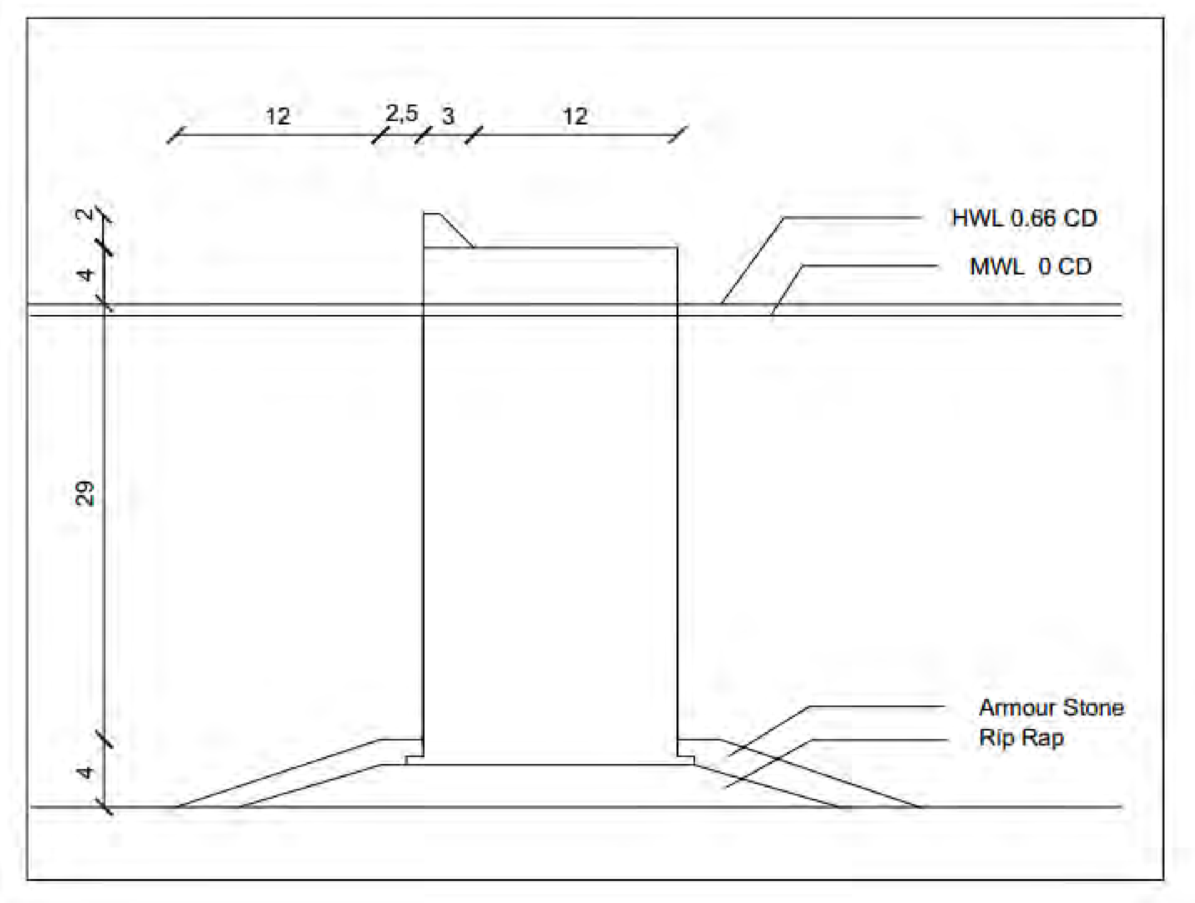


FIGURE 7-7: FINAL BREAKWATER DIMENSIONS

7.2.4 Overtopping

Overtopping discharge q was checked to ensure no overtopping conditions (maximum of .4 l/s) were met. The latest Eurotopping Manuel gives q as:

$q=(9.81*H_s^3)*0.047*\exp(-((2.35*h_c/H_s)^{1.3}))$

The no over topping criteria of q<.4 l/s was satisfied for the maximum high water level with a calculated q of .26 l/s.

With a mid-range projected sea level rise (RCP6.5) of .66m, there is some overtopping, with a q of .41 l/s. We therefore advise the addition of a superstructure to the caisson for future conditions.

7.2.5 Foundation

The stone size used for the armour of the rubble mound foundation was calculated using the Hudson-type formula proposed by Tanimoto (1982), and is given in Table__. The average stone diameter is defined from the stability number:

$$\frac{H_{1/3}}{\Delta D_{n50}} = N_s$$

Where the stability number Ns is defined as:

$$N_s = \max\left\{1.8, \left(1.3\frac{1-\kappa}{\kappa^{1/3}}\frac{h'}{H_{1/3}} + 1.8\exp\left[-1.5\frac{(1-\kappa)^2}{\kappa^{1/3}}\frac{h'}{H_{1/3}}\right]\right)\right\}$$

Foundation bearing capacity and failure mechanisms should be assessed with further investigation. The hydraulic force exerted on the caisson, plus the weight, determine the local pressures in the interface between the caisson and the foundation, which need to be assessed to ensure that these pressures will not lead to soil mechanical failure.

TABLE 7-5: FOUNDATION ARMOUR RESULTS

Foundation Armour		
Ns	9.16	
κ	0.22	
L'	116.45	m
γ_r	2.65	
Sr	2.59	
W	0.12	tons
V	0.04	m^3
D_{n50}	0.35	m

7.3 Quay Design

The main usage and function of a quay wall is to allow vessels to moor and sustain the impact and loads of the vessels. The choice of quay type is based on geotechnical parameters, vessel types and several parameters on the onshore work.

In general the three different types of quay walls are:

- Gravity wall
- Open berth quay
- Sheet pile wall

7.3.1 Gravity walls

Gravity walls are used when the soil profile of the construction area is mostly composed of rocks or firm soil that do allow the installation of piles (open berth quay or sheet pile wall). This type of quay is suitable for big bearing capacities of the soil.

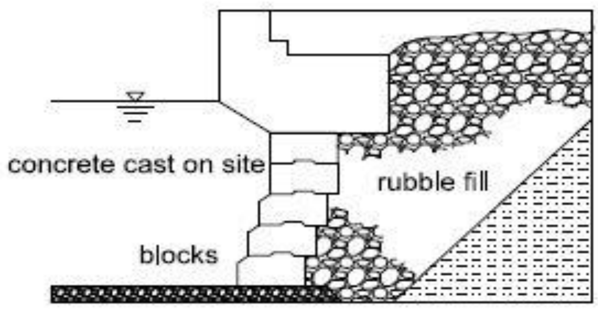


FIGURE 7-8 EXAMPLE OF GRAVITY WALL [CUR, 2005]

7.3.2 Open Berth quay

This type of structure has a horizontal deck that its foundation lays on vertical piles that are built on a slope. Because of its design the open Berth quay is used not for retaining the soil but to permit the water to drive through the piles and to permit the birth of the ships. Open berth quays are suitable for poor soil conditions and low bearing capacity soil.

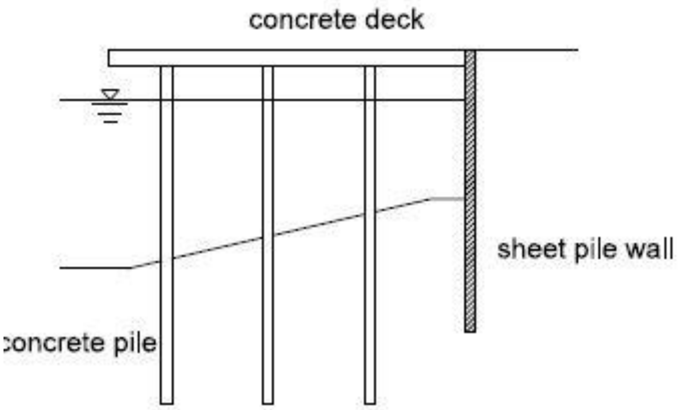


FIGURE 7-9 OPEN BERTH QUAY [CUR 2005]

7.3.3 Sheet pile wall

The function of the sheet pile wall is to retain the soil and can be found by obtaining the soil pressure combined with the resistance of the bending. This type of quay can be used in most soil types as long as the sheet pile is possible to surpass all of the soil layers.

Two types of sheet pile walls can be designed free standing or anchored. Combined walls can be used in order to carry considerable loads and big retaining heights.

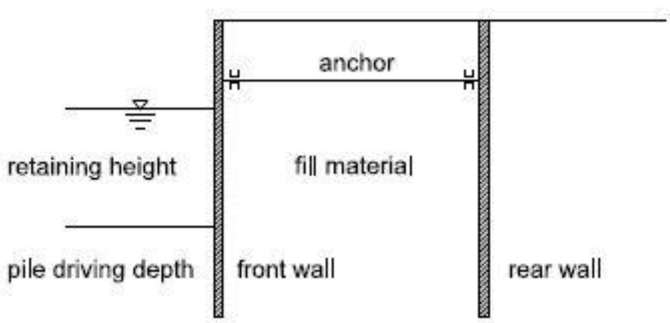


FIGURE 7-10 SHEET PILE WALL (COFFERDAM) [CUR 2005]

The choice of the quay wall that is going to be designed in the current project depends on several parameters. Since we design the quay wall in poor soil conditions and we want to retain soil at the back of the quay the choice of the sheet pile wall with two anchors is the one that is going to be examined.

7.3.4 Design of the quay wall.

The design of the quay wall was performed with the K-Rea software. The quay wall lays close to the shore as can be seen on the following figure.

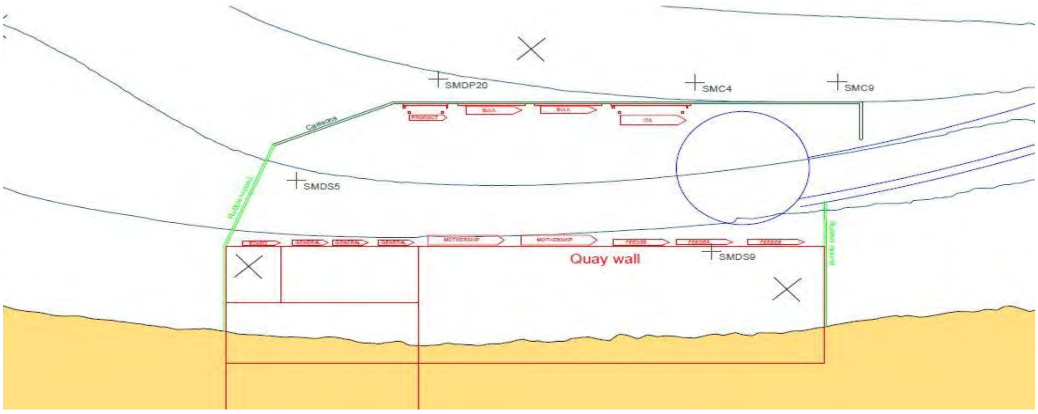


FIGURE 7-11 QUAY WALL VIEW FROM THE LAYOUT

The water depth at the area of interest is 5 meters and soil profile is described from the point SMDS9 in the following figure (Figure 7-12).

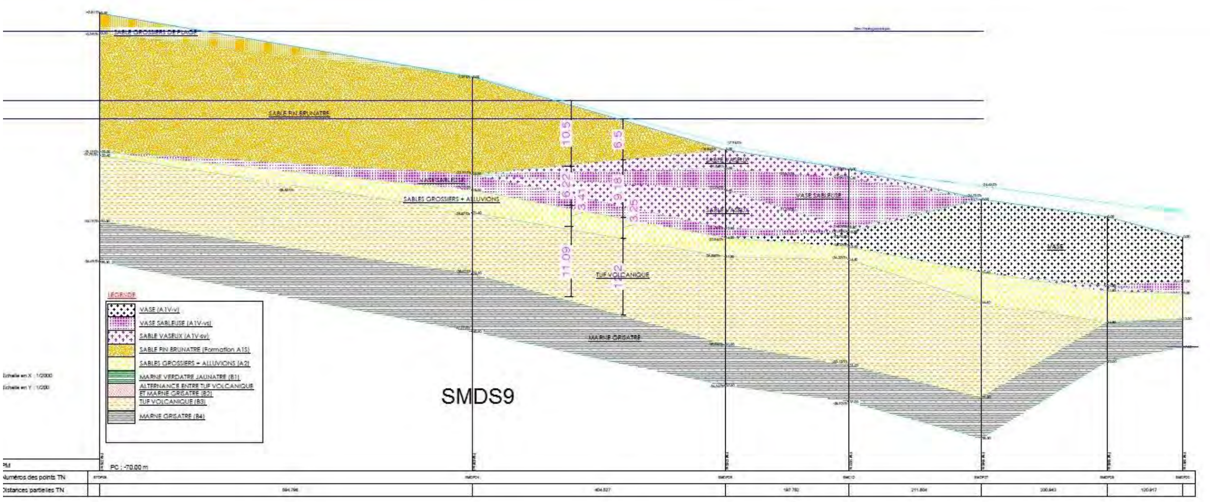


FIGURE 7-12- SOIL STRATIGRAPHY AT SMD9

So the soil profile in the area of interest is described on the next table (Table 7-6).

TABLE 7-6- SUMMARY SOIL STRATIGRAPHY

Height	Soil Stratigraphy SMDS9	SCA09
0 to -8	Water	Water
-8 to -27.5	Sable fin Brunarre (Yellow fine sand)	Soft clay
-27.5 to -36.6	Sables grossiers alluvions (Coarse sand)	Coarse sand
-36.6 to -41.5	Marne Grisarne (Marly)	Silty clay

And the provided soil parameters can be seen in the following Table 7-7- Geotechnical Parameters:

TABLE 7-7- GEOTECHNICAL PARAMETERS

		Etat en place		Paramètres de résistance mécanique			Paramètres Œnométriques					Paramètres de rigidité		
				Court terme	Paramètres effectifs		Tassement			Consolidation		Module élastique	Module pressio.	Coef. Poisson
Sol	Profondeur	$\gamma_{sèche}$ (kN/m3)	γ_{sat} (kN/m3)	Cu (kPa)	Φ' (°)	C' (kPa)	eo	Cc	Cs	Cv (cm2/s)	Cr (cm2/s)	E (MPa)	Em (MPa)	v
Vase (silt argileux)		13	18.7	11.5 + 0.385.Z	23	9	1.09	0.3	0.05	0.001	0.005		--	--
Sable grossier		--	18.2	--	33	0	--	--	--	--	--		--	--
Marne verdâtre		14	19	120	21	35	0.83	0.18	0.06	0.0015	0.007		30	0.2
Marne grisâtre		17	20	170	30	50	0.6	0.11	0.05	0.0015	0.007		90	0.2
Marne grisâtre dure		17	20	200	30	50	0.6	0.11	0.05	0.0015	0.007		90+ 2.5.Z	0.2
Tuf volcanique	de 0 à 10m	19	22	1000	--	--	--	--	--	--	--		130 +11.Z	0.2
	> 10m	19	22	1000	--	--	--	--	--	--	--		240 +3.Z	0.2
Sable pour drains		--	20	--	30	--	--	--	--	--	--	20		
TV 1 à 500		17	20	--	45	--	--	--	--	--	--			
Enrochements inférieurs à 1T		17	20	--	45	10	--	--	--	--	--			
Enrochements supérieurs à 1T		17	20	--	45	10	--	--	--	--	--			
BCR		15	18.4	--	45	20	--	--	--	--	--			
TETRAPODES		12	17	--	45	20	--	--	--	--	--			
ACCROPODES		12	17		46	20	--	--	--	--	--			

For the K-rea Calculations it is needed to calculate the soil coefficients k_a, k_p and k_0 that can be obtained with the Rankine formula based on Mohr Coulomb approach and based on geotechnical data like friction angle soil cohesion and angle between lateral earth pressure and the normal to the wall.

$$k_{a\gamma} = \tan^2\left(\frac{\pi}{4} - \frac{\varphi}{2}\right) \qquad k_{p\gamma} = \tan^2\left(\frac{\pi}{4} + \frac{\varphi}{2}\right) \qquad k_{a\gamma} = \left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\varphi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\varphi}}\right]\cos\beta$$

EQUATION 7-3- RANKINE FORMULAS

Then it is important to calculate the subgrade reaction coefficient kh were the elastic behaviour if the soil and the interaction of soil and the structure are considered.

And the parameters used can be seen in the following Table 7-8.

TABLE 7-8- PARAMETERS USED IN K-REA

		EI (kNm^2/ml)	Steel section	Young's modulus E	Horizontal spacing	Diameter	Thickness	Height
Wall 1	Piles	4663617	Hollow circular steel section	2,1E+8 kN/m^2	2,85 m	2030 mm	19,84 mm	32 m
	Between piles	137		2,1E+8 kN/m^2			19,84 mm	26,5 m
	EI (kNm^2/ml)	Steel section	B	I	W	G	H	Height
Wall 2	76083	AZ 18-700	700 mm	378000 cm^4/m	1800 cm^3/m	109 kg/m^2	420 mm	15 m

The linking anchors are to depth of -1 and -11 meters and the elastic constant is K=16000 and the length of them is 30 meters.

The 1st wall is designed as a composite wall and then the 2nd one as a sheet pile wall and the structure profiles can be seen in the next figure (Figure 7-13 Figure 7-14)

FIGURE 7-13- COMPOSITE WALL

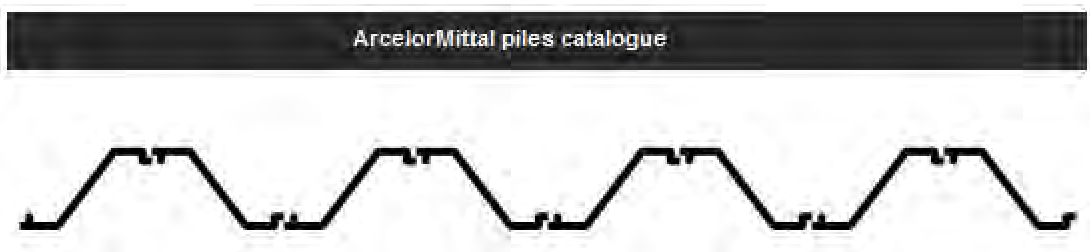


FIGURE 7-14- SHEET PILE WALL

The model that was designed in K-rea can be seen in the following figure (Figure 7-15. And the dimensioning was performed by iteration. The dimensions of the wall needed to be put up to 30 meters in order to minimise the displacements and to drive the retaining sheet pile up to -30 and – 25 meters from the sea level.

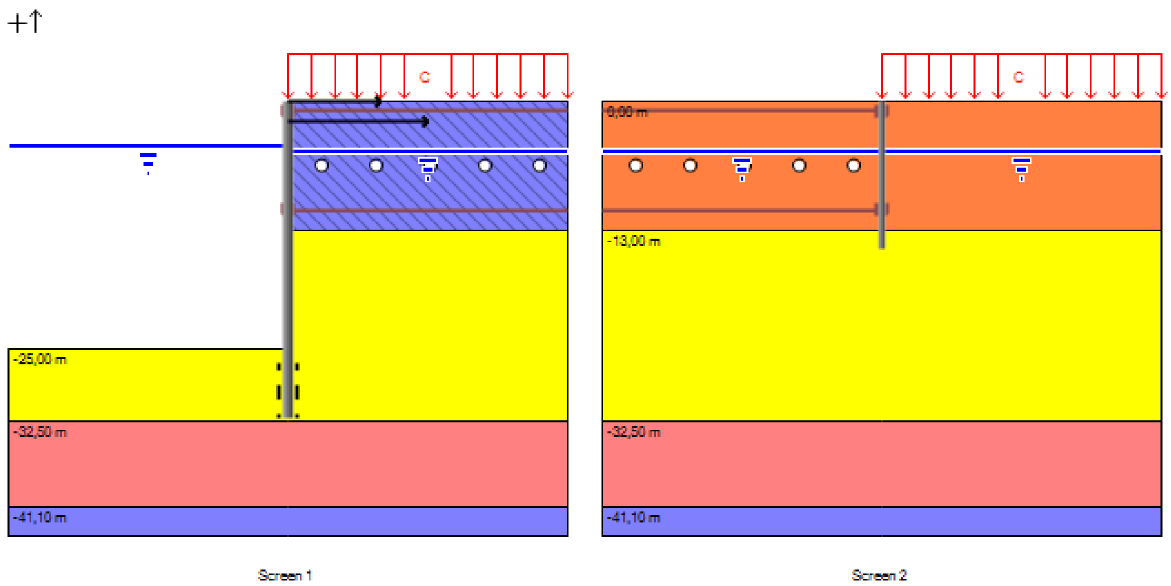


FIGURE 7-15- MODEL AT K-REA

The phases of the construction and the calculations that were performed in K-rea can be seen in the next table (Table 7-9).

TABLE 7-9- PHASES IN K-REA

Phase	Description
0	Construction of both walls
1	Placement of two linking anchors (-1 and -11)
2	Fill with sand Surcharge in back of wall 2 (30kN/m/ml)
3	Improve properties of filled sand (friction angle=35°) - Vibration
4	Surcharge in back of wall 1 (30kN/m/ml)
5	Dredge to -25 m
6	Hydraulic action because of the tides (+ 0,66 m)
7 **SLS-1**	Apply horizontal linear force because of shiploader live load (-17 kN/ml) Apply horizontal linear force because of bollards live load (-70 kN/ml) Apply moment because of bollards live load (35 kNxm/ml)
8 **SLS-2**	Apply horizontal linear force because of shiploader live load (+17 kN/ml) Apply horizontal linear force because of fenders live load (130kN/ml)

The displacements, moments, shear forces and earth and water pressures gained after running K-Rea can be seen in the next figures according to the depths.

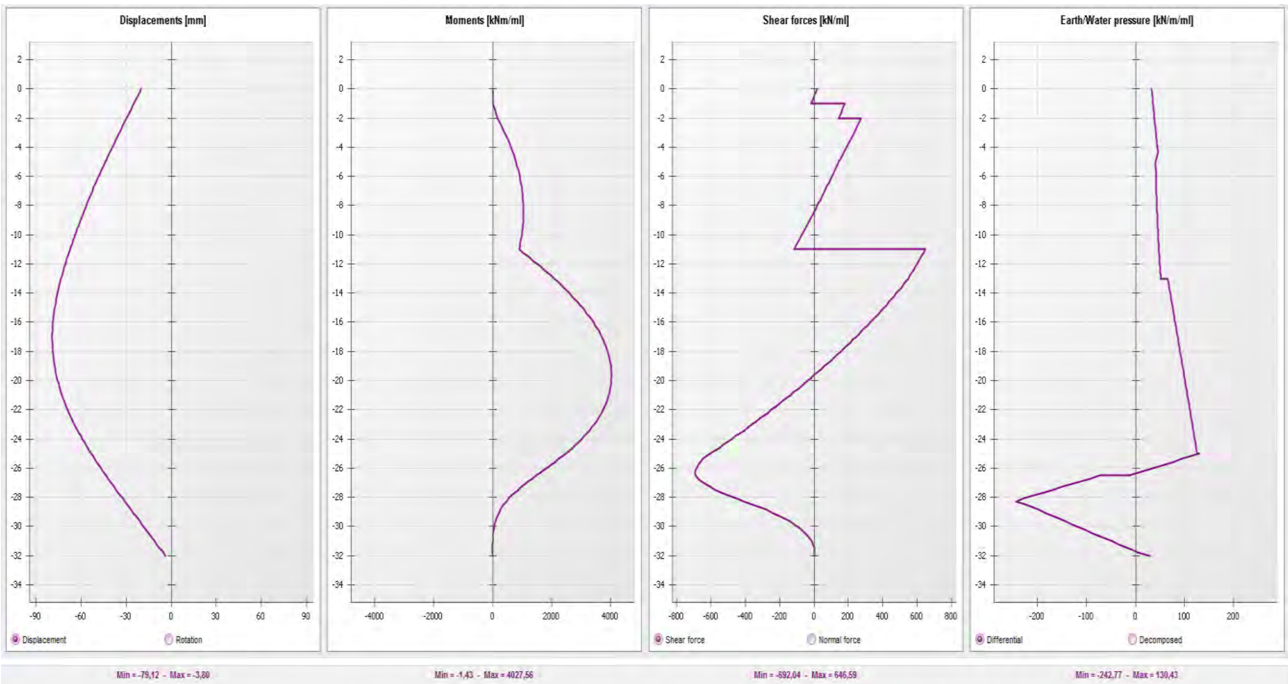


FIGURE 7-16- WALL 1

TABLE 7-10- SUMMARY WALL 1

Wall 1	Min	Max
Displacements [mm]	-79.12	-3.8
Moments [kNm/lm]	-1.43	4027.56
Shear forces [kN/m]	-692.04	646.59
Earth/ water Pressure	-242.77	130.43

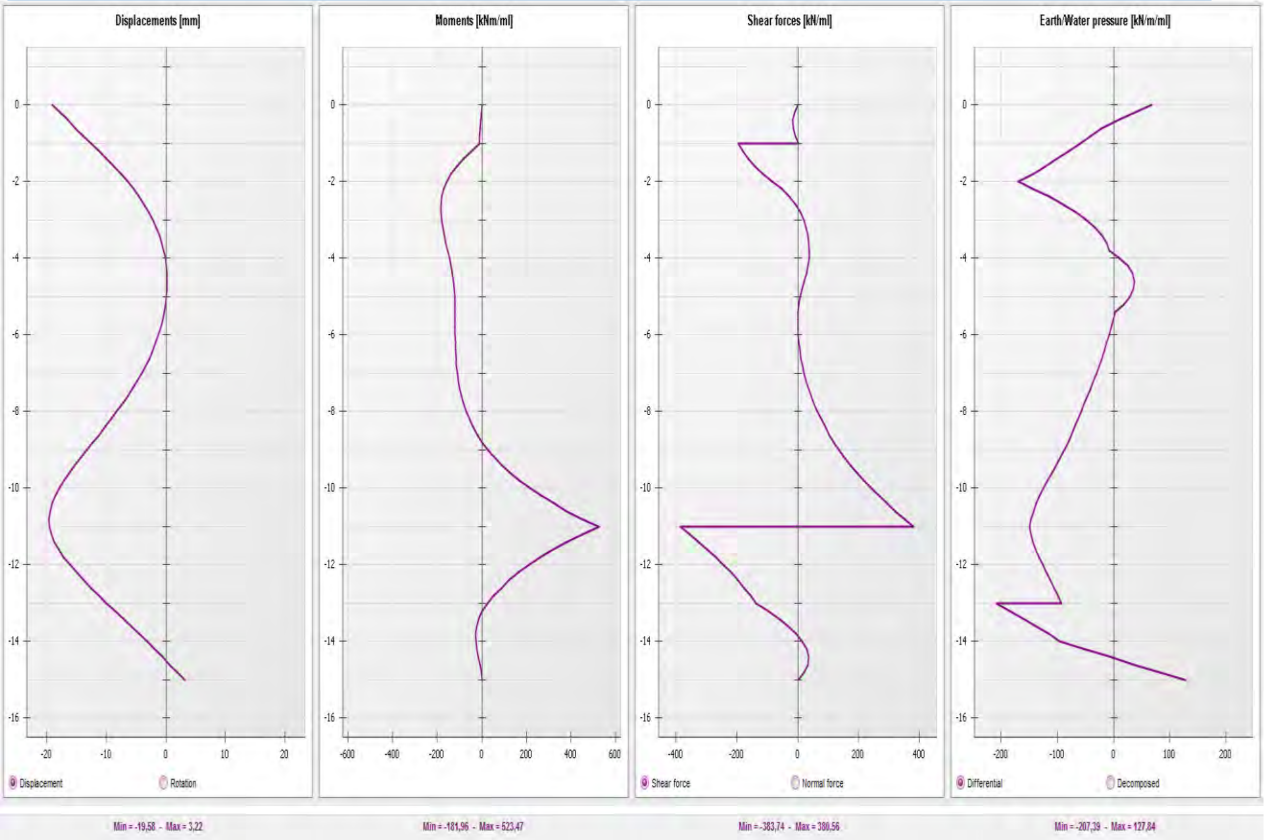


FIGURE 7-17- WALL 2

TABLE 7-11- SUMMARY WALL 2

Wall 2	Min	Max
Displacements [mm]	-19.58	3.22
Moments [kNm/lm]	-181.96	523.47
Shear forces [kN/m]	-383.74	380.56
Earth/ water Pressure	-207.39	127.84

7.4 Mooring Dolphin Design

7.4.1 Mooring lay-out

For the design of the mooring facilities, at first a general lay-out should be defined. A design is made for 4 different design vessels, varying in size. In Figure 7-18**Error! Reference source not found.**, the maximum angles of the mooring lines are given. These angles depend on the distances between the mooring dolphins and the dimensions of the ship. As a rule of thumb for the distance between the two breasting dolphins, a value of 0.3 to 0.4 times the overall length of the ship is used. To build as few structures as possible, 6 mooring dolphins and 4 breasting dolphins are needed. The exact dimensions are given in Table 7-12, Table 7-13, leading to the situation in **Error! Reference source not found.**. Some angles are slightly out of range (**Error! Reference source not found.**), but the values are still regarded as acceptable.

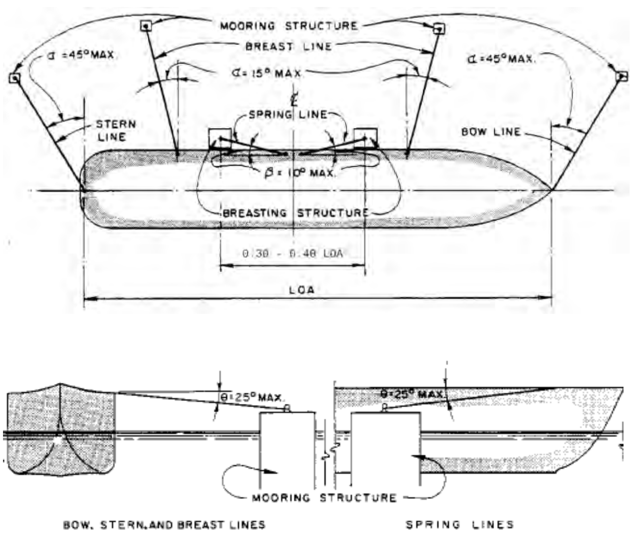


FIGURE 7-18- MOORING LINES

TABLE 7-12- DISTANCES BETWEEN BREASTING DOLPHINS

B_{fender}	4.5	m
$L_{dolphin}$	40	m
L_{stern}	210	m
$L_{breast/stern}$	143	m
L_{breast}	80	m
L_{breast}	80	m
$L_{breast/bow}$	143	m
L_{bow}	210	m

TABLE 7-13 - DISTANCES BETWEEN MOORING DOLPHINS

DWT	B [m]	LOA [m]	0.3LOA [m]	0.4LOA [m]	$L_{breasting}$ [m]
300000	60	345	103.5	138	105
150000	48	274	82.2	109.6	105
50000	32	182	54.6	72.8	55
15000	21	143	42.9	57.2	55

TABLE 7-14- ANGLES OF MOORING LINES

DWT	B [m]	LOA [m]	Stern line [°]	Breast line [°]	Spring line [°]	Breast line [°]	Bow line [°]
300000	60	345	118.2	73.0	4.9	107.0	61.8
150000	48	274	138.8	106.0	4.9	74.0	41.2
50000	32	182	132.9	79.6	9.3	100.4	47.1
15000	21	143	144.8	105.9	9.3	74.1	35.2
MAXIMUM ANGLE			120-50	75-105	5-10	75-105	30-60

The number of lines for largest vessels is estimated to be 4 3 2 2 3 4 from left to right and for small vessels 2 2 2 2 2 2 from left to right. For safety reasons, quick release hooks are favored over regular bollards. These hooks must be able to resist a load higher than the minimum breaking load (MBL) of the mooring lines. This means that the following deck fitting is required:

- Quadruple QRH for 150 tons on the exterior mooring dolphin for large vessels
- Triple QRH for 150 tons on the mooring dolphin for large vessels
- Double QRH for 150 tons on the mooring dolphin for small vessels
- Double QRH for 150 tons on the breasting dolphins

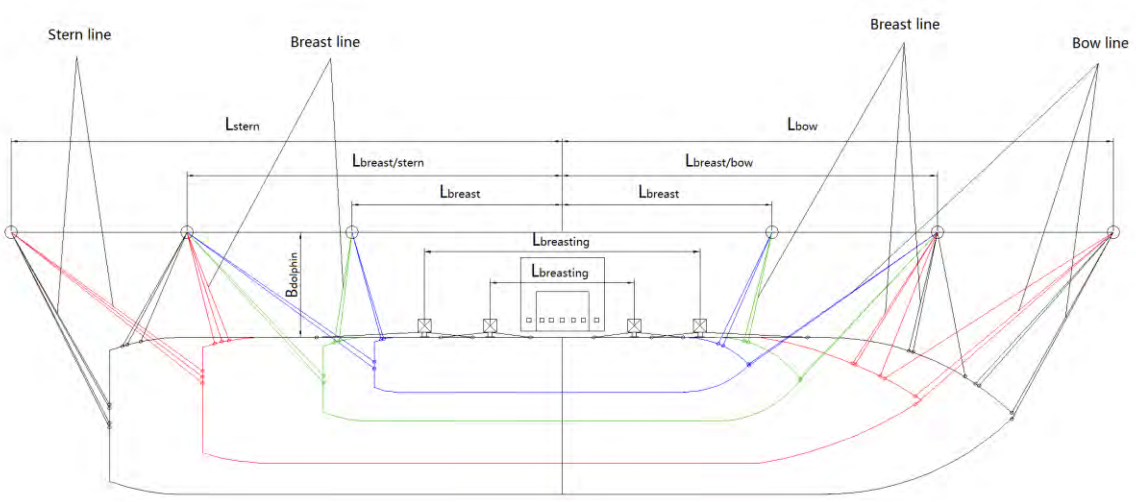


FIGURE 7-19- DESIGN OF STRUCTURE

Fender Design

To determine the necessary properties of the fender on the breasting dolphins, the following equations (Equation 7-4and assumptions are used:

$$E_N = 0.5 * M * v_B{}^2 * C_M * C_E * C_C * C_S$$

$$E_A = F_S * E_N$$

EQUATION 7-4- ENERGY ON FENDERS

with:

- the normal energy E_N
- the abnormal energy E_A
- a factor of safety $F_S = 1.25$ or 1.75
- $M = 300\,000$ or $50\,000\ DWT$
- $v_B = 0.06$ or $0.1\ m/s$ for difficult berthing in a sheltered area
- $C_M = 1.8$
- $C_E = \frac{K^2+R^2 cos^2\varphi}{K^2+R^2}$ with:
  - $R = \sqrt{y^2 + \left(\frac{B}{2}\right)^2}$
  - $\varphi = \frac{\pi}{2} - \alpha - \text{asin}\left(\frac{B}{2R}\right)$
  - $K = (0.19 * C_B + 0.11) * LOA$ with:
    - $C_B = \frac{M_{displaced}}{LOA*B*D*\rho_{sea}}$
- $C_C = 1.0$ for open structures
- $C_S = 1.0$ for soft fenders

TABLE 7-15- SUMMARY FENDERS

Fender	SCN1300 (E0.9)	SCN900 (E0.9)
E_R [kJ]	743	248
R_R [kN]	1103	527
A_{fender} [m^2]	5.5	2.6
Possible dimensions [m x m]	2 x 3	1.5 x 2

All values are given for large and small tankers respectively. The result is an abnormal energy of $E_A = 730\ kJ$ for large tankers and $E_A = 235\ kJ$ for small tankers. The outer fenders have to be designed for large tankers, but to save money the inner fenders can be designed for the largest type of small tankers. This leads to two types of fenders; the SCN1300 with E0.9 rubber and the SCN900 with E0.9 rubber. Assuming a hull pressure of 200 kN/m^2, one can calculate the required area of the fender (see **Error! Reference source not found.** for further details).

7.4.2 Monopile structure

The breasting and mooring dolphins are designed as monopile structures. For the structural checking of all dolphins the mooring forces are leading. The following three checks are done:

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - \frac{f_a}{F_e'}\right) F_b} \leq 1.0$$

$$\frac{f_a}{0.6F_y} + \frac{f_b}{F_b} \leq 1.0$$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0\ if\ \frac{f_a}{F_a} \leq 0.15$$

EQUATION 7-5- CHECKS ON DOLPHIN

with:

- $C_m = min\left(1 - 0.4\frac{f_a}{F_e'}; 0.85\right)$
- $f_a = \frac{N}{A}$, with:
  - o $N = sin(25°) * MBL * g = 622\ kN$
  - o $A = \frac{\pi}{4} * (D^2 - (D - 2t)^2)$
- $f_b = \frac{M}{W}$, with:
  - o $M = cos(25°) * MBL * g * (L + 2.5D) = 47\ 543\ kNm$ (M_{max} at $2.5D$ below bed)
    - ▪ $L = 26.4 + 5 = 31.4\ m$ (water depth and part above water)
  - o $W = \frac{I}{D/2}$, with $I = \frac{\pi}{64} * (D^4 - (D - 2t)^4)$
- $F_y = min(F_{xc}, F_{xe})$ for local buckling, with:
  - o $F_{xe} = 2CE\frac{t}{D}$, with $C = 0.3$ and $E = 210\ 000\ MPa$
  - o $F_{xc} = F_y$ for $\frac{D}{t} \leq 60$ or otherwise $F_{xc} = F_y * \left(1.64 - 0.23 * \left(\frac{D}{t}\right)^{1/4}\right) \leq F_{xe}$
  - o with an initial yield stress for S450 steel of $F_y = 450\ MPa$
- $F_a = F_a * f_{allowable\ stress}$ for column buckling:
  - o $F_a = \frac{\left(1 - \frac{(KL/r)^2}{2C_c^2}\right)F_y}{\frac{5}{3} + \frac{3(KL/r)}{8C_c} - \frac{(KL/r)^3}{8C_c^3}}$ for $KL/r < C_c$
  - o $F_a = F_e' = \frac{12\pi^2 E}{23(KL/r)^2}$ for $KL/r \geq C_c$
    - ▪ $K = 2.1$ (design value for rotation and translation fixed at bottom, free at top)
    - ▪ $r = \sqrt{\frac{I}{A}}$
    - ▪ $C_c = \left(\frac{2\pi^2 E}{F_y}\right)^{1/2}$
  - o $f_{allowable\ stress} = 1.33$ for extreme cases
- $F_b = F_b * f_{allowable\ stress}$ for bending:
  - o $F_b = 0.75F_y$ for $\frac{D}{t} \leq \frac{10\ 340}{F_y}$
  - o $F_b = \left(0.84 - 1.74\frac{F_y D}{Et}\right)F_y$ for $\frac{10\ 340}{F_y} < \frac{D}{t} \leq \frac{20\ 680}{F_y}$

- $F_b = \left(0.72 - 0.58\frac{F_y D}{Et}\right) F_y$ for $\frac{20\,680}{F_y} < \frac{D}{t} \leq 300$

The three checking criteria are fulfilled with a diameter $D = 1.7\ m$ and a steel thickness $t = 0.0635\ m = 2.5\ inches$, allowing a corrosion of $3\ mm$. The monopile would have to be placed at least $2.5D$ under the sea bed. Therefore, a depth of $5\ m$ is considered to be safe.

8 Cost Estimate

8.1 Unit Cost analysis

A compromise should be found to satisfy environmental, technical and economic requirements. In this section the economic aspects of the design will be considered.

To satisfy the design brief, only a preliminary estimation of the total cost has been carried out. Not taking into consideration, for example, the type of breakwater's artificial rocks or the material of the quay.

Using the dimensions of the port design from an AutoCad layout, and referring to the Appendix F: Unit Cost, the cost to install the port was calculated in Euros.

8.1.1 Breakwater cost

It can be seen from the appendix, it is necessary to distinguish which type of breakwater we are considering: rubble mound in open seas, secondary, inner port protection (not our case), or caissons.

From the coast to a depth of 27 m a rubble mound open sea breakwater will be used. From this depth to 30m the breakwater is built by caissons. The structure on the left side of the image below is a secondary breakwater.

The results are shown in the table:

type of breakwater	depth	€/m	length	€
open seas breakwater	5	20000	535	10700000
	13	70000	601	42070000
caissons breakwater	20	170000	302	51340000
	24	180000	800	144000000
	27.5	185000	1595	295075000
	27	185000	538	99530000
secondary breakwater	6	10000	849	8490000
	13	40000	164	6560000
total				658 million €

8.1.2 Quay cost

Following the previous steps the following cots were reached for the quay wall:

	quay depth	€/m	length	€
containers	18	32000	2130	68160000
gen. Cargo	14	55000	854	46970000
ro ro	8	20000	500	10000000
total				126 million €

8.2 Dredging cost

In order to reach the sufficient depth for navigation, it's necessary to dredge the sand present in the inner port. This soil will be reused to backfill the quay wall of terminal container and Ro- Ro. Dredging must be done also under the breakwater, to remove the thin soft clay layer that has poor load-bearing capacity characteristic.

type of soil	€/m³	m³	€
sand	7	4000000	28000000
backfilling	3	4000000	12000000
under breakwater soft clay	10	2670000	26700000
total			40 million €

8.2.1 Jetty cost

In the appendix, the cost per metre of each structure that composes the jetty's and walkways is available.

	unit	€/unit	€
mooring dolphins	6	1700000	10200000
berthing dolphins	2	2100000	4200000
platform	8	6500000	52000000
fender system	4	150000	600000
		total	67 million €

	m	€/m	€
walkways	1522	9000	13698000
		total	14 million €

8.2.2 Total Cost

A preliminary estimate of the total cost for the port design gives approximately 905 million €.

Below for the two layouts 2 and 3 is a table to show a cost comparison between them, it can be seen that the updated design is nearly 100 million €.

	layout 2	layout 3
breakwater	760 million €	658 million €
quays	100 million €	126 million €
dredging	98 million €	40 million €
jetty	81 million €	81 million €
	1039 million €	905 million €

9 Project Management

A work breakdown strcuture in the form of a Gantt chart was created to monitor progress of the design phase during the project. This allowed the team to divide the tasks and split up into smaller teams to complete each component of the project. The smaller teams were made of 2-4 people, and were selected based on eavh memebers respective experience in the are of intererest of the sub-task. The work Breakdown structure can be seen in Appendix G.

A project review was conducted each week with a mentor to review progress. Also around 2-3 group meetings were conducted each week to review and discuss progress.

10 Discussion

In future Port Engineering design projects, the design team have discussed what has been learnt during this design

10.1 Discussion offshore wave analysis

In the offshore wave analysis, a longer data set could have been acquired to be able to make an extrapolation for the significant wave heights with a higher certainty. The used dataset of 18 years is quite long, but most probably more data is available for this area in the Mediterranean. Also, the given return period of $R = 100\ years$ is relatively small. Considering the relationship for the probability of failure during the lifetime of the structure$p_{f,TL} = 1 - e^{-T_L/R}$, there is a 18% probability of failure for a short lifetime of 20 years. For a more realistic lifetime of 30 to 50 years, this would be 26% to as high as 39%. Usually the failure probability for a breakwater lies between 5% and 20%. If the lifetime of the breakwater should be more than 20 years a higher return period is recommended.

Furthermore, the operational wave conditions could have been split up over different angles. In this analysis, operational waves are determined over all directions together and an average angle is used to find the wave agitation. In reality, a distinction will be visible in the height of operational waves from different directions. This could have an impact on the final layout.

10.2 Discussion rubble mound breakwater

The final design is based on an engineering judgement of the design calculations and the wave flume results. These combined give a reasonable design. It is advisable to redo a wave flume test with the final design to properly check armour stability. Also, the overtopping could be measured with an overtopping chute in a larger wave flume. This will give more reliable results than the current ones, done by visual inspection.

Furthermore, the construction method of the breakwater should be thought over. This might lead to changes in the final design. For example, the use of a crown wall can be useful to create a road for the construction works. This might be cheaper than placement by use of vessels. Also, a quarry should be found. The properties of this quarry can influence the stone sizes in the design, since these can be adapted to the outcome of the quarry yield curve.

10.3 Discussion Dolphin design

The design of mooring arrangement and dolphins are not perfect because of the shortage of detailed available data and limited time. Some improvements could still be made if more detailed design is needed.

Because of the lack of detailed characteristics of each vessel, the locations of the attachment of the mooring lines on the vessels are not accurate. This could have some influence on the mooring layout and line distribution, which could be improved if more data for coming ships is given.

As a preliminary design, all of the six mooring dolphins are designed on one line. In reality, these dolphins are positioned like a wing shape. So, our mooring layout could be optimized depending on the requirement of clients.

When designing the fenders, we chose the fender type according to the energy to be absorbed by the fender. Actually, for one number of the energy, more than one type of fender could be chosen because they all may satisfy the energy requirement. High grade fender which is softer will cost more but give smaller reaction force which is beneficial to pile structures. Low grade harder fender is cheaper but give larger reaction force on the dolphin piles. Which kind of fender to be chosen depends on the total cost of fenders and the piles structure and could be discussed with the clients.

Mooring and berthing aid systems may include lasers, display board, line tension monitors and computer workstation, which can help ensuring a safe berthing and mooring. The systems can be chosen depending on the clients' preference which are not included in our preliminary design.

11 Conclusion

For the construction of the port of Nador the conceptual design of several structural and managing aspects have been considered. Initially from the provided wind and wave data a statistical analysis was conducted and the results that were extracted were that the most critical waves arrive at the area of interest are from North East and North West. Aditionally operational waves are defined for maximum 1% downtime of the harbour per year. These data were transformed to near shore conditions in order to retrieve a numerical analysis by the Artemis software inside the harbour.

The harbour can receive ro-ro, general cargo, bulk, product, crude oil and container vessels. All facilities to receive these ships are installed. A general overview of the harbour is given in the layout in FIGURE. As can be seen, ro-ro and general cargo terminals are located in the south-western corner of the harbour, container terminals are located in the middle and north-western stretch of the coastline, where the mother ships are located near to the entrance, where the bathymetry is the deepest, and bulk carrier, product tanker and crude oil tanker terminals are located on the breakwater in deeper water, where the latter is nearest to the entrance. Rubble mound breakwaters with Accropodes are constructed up to a depth of about 20 m. This includes the first half of the primary breakwater and the entire secondary breakwater. For the deeper parts, up to 30 m, vertical breakwaters out of caissons are used.

A final cost estimation gives a construction cost for the harbour of 905 million euros.

References

12 References

Department for Transport GOV UK. (2013, July 15). *Guide to Lorry Types and Weights.* Retrieved from GOV.UK: https://www.gov.uk/government/uploads/system/uploads/attachment_data/file/211948/simplified-guide-to-lorry-types-and-weights.pdf

Groenveld, I. R. (1993). *Servive Systems in Ports and Inland Waterways.* Delft.

Thoresen, C. A. (2003). *Port Designer's Handbook: Recommendations and Guidelines.* Thomas Telford.

Goda, Y. 2000. ‘Random seas and design of maritime structures” (2nd Edition), Advanced Series on Ocean Engineering vol. 15, World Scientific (2000) 443 pp.

Oumeraci, H., Kortenhaus, A., Allsop, W., de Groot, M., Crouch, R., Vrijling, H., and Voortman, H. 2001. Probabilistic Design Tools for Vertical Breakwaters.

Tanimoto 1982. Irregular Wave Tests for Composite Breakwater Foundations. Coastal Engineering Journal (1990).

Appendices:

Appendix A: Number of Berths Queuing Theory

Cargo	Traffic Per Year	Traffic Type	Number of Berths	No of Handling Equipment	(un)Loading Rate	Calls per Year	Inter Arrival Time	Service Time	Berth Occupancy	Berth Utilisation	Calculated Wait Time	Wait Time limit
-	Tons or TEU	-	-	-	Tons or TEU per hour	-	Hours	Hours	-	-	%	%
General	2000000	Import/Export	3	3	60.00	72	118	157	1.34	0.50	8	10
Ro-ro	1000000	Import/Export	1	1	13500.00	51	168	4	0.02	0.45	10	10
Bulk	7000000	Import	2	2	1250.00	40	211	143	0.34	0.02	2.5	10
Liquid	25000000	Import/Export	2	1	10400.00	61	139	42	0.30	0.17	2	10
Containers	3000000	Import/Export	5	4	37.50	105	80	192	2.41	0.48	4	5
Total	-	-	13	-	-	-	-	-	-	-	-	-

Appendix B: Container Storage Area

Storage System	Levels	S_{teu}	F_u	S_{st}	S_{st}	S (No Quay Storage)	S Total (Including Quay Storage)	S Total Per Berth	Depth Behind Quay	Acceptable
	-	M2/Teu	-	m2	Hectares	Hectares	Hectares	Hectares	m	
Tractor	1.00	57.50	1.00	5907534.25	590.75	843.93	854.05	170.81	4009.64	no
Straddle Carrier	2.00	17.50	0.96	1872859.59	187.29	267.55	277.67	55.53	1303.62	no
Straddle Carrier	3.00	11.50	0.91	1298359.18	129.84	185.48	195.60	39.12	918.31	no
								0.00		
Reach Stacker and Forklift	2.00	37.50	0.96	4013270.55	401.33	573.32	583.44	116.69	2739.18	no
Reach Stacker and Forklift	3.00	27.50	0.91	3104771.94	310.48	443.54	453.66	90.73	2129.85	no
RMG/RTG	2.00	17.50	0.96	1872859.59	187.29	267.55	277.67	55.53	1303.62	no
RMG/RTG	3.00	11.50	0.91	1298359.18	129.84	185.48	195.60	39.12	918.31	no
RMG/RTG	4.00	8.50	0.87	1003778.93	100.38	143.40	153.52	30.70	720.74	yes
RMG/RTG	5.00	7.00	0.82	877046.44	87.70	125.29	135.41	27.08	635.74	yes

Appendix C: Wave propagation Yearly Waves

Results of the propagation for Yearly Waves are showed below. Maximum still water level is used because is also the most critical situation.

North West Yearly Waves (1 year return period)					
Main direction (°)	Hs (m)	Tp (s)	Fp (Hz)	Fmin (Hz)	Still water level (m)
303	3,4	7,7	0,13	0,065	0,66
303	3,4	7,7	0,13	0,065	0

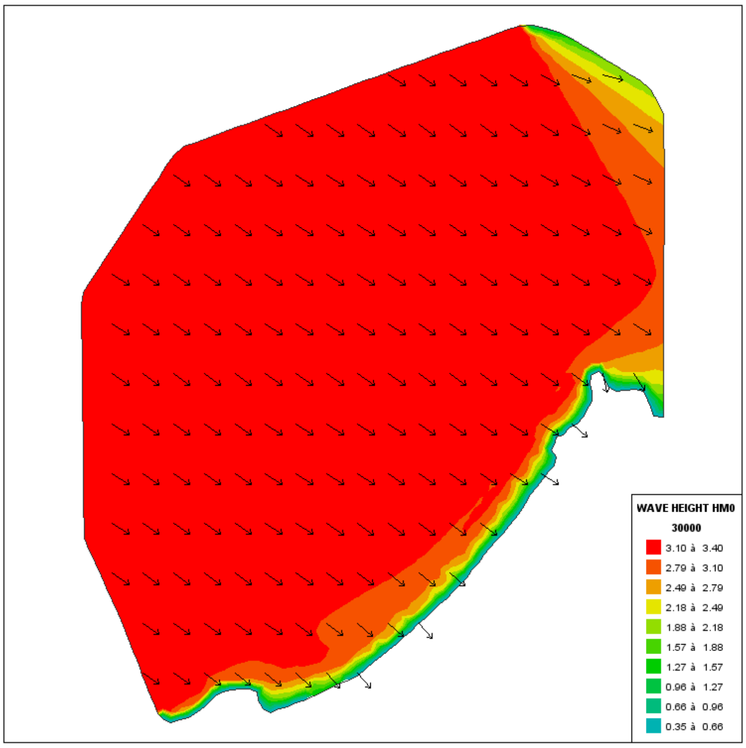


FIGURE 0-1 WAVE HEIGHT AND MEAN DIRECTION FOR NORTH WEST YEARLY WAVES

Northeast Yearly Waves (1 years return period)					
Main direction (°)	Hs (m)	Tp (s)	Fp (Hz)	Fmin (Hz)	Still water level (m)
24	2,6	8,8	0,11	0,055	0,66
24	2,6	8,8	0,11	0,055	0

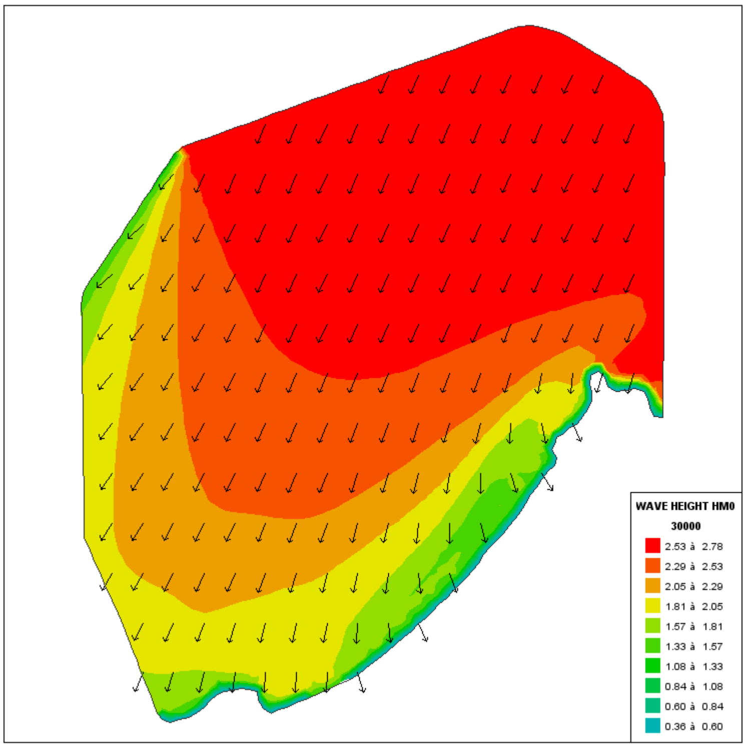


FIGURE 0-2 WAVE HEIGHT AND MEAN DIRECTION FOR NORTHEAST YEARLY WAVES

Appendix D: Wave propagation Profiles

Below six profiles are shown in the area of interest running along the main direction of the waves, two for each one. These profiles are used to check water depth, peak period (TPR5), wave height (HM0) and the mean direction.

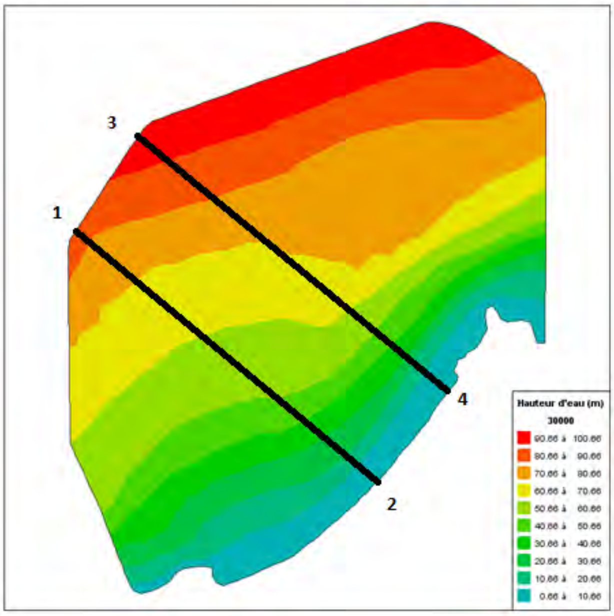


Figure 0-3 Placement of profiles for North West Extreme Waves

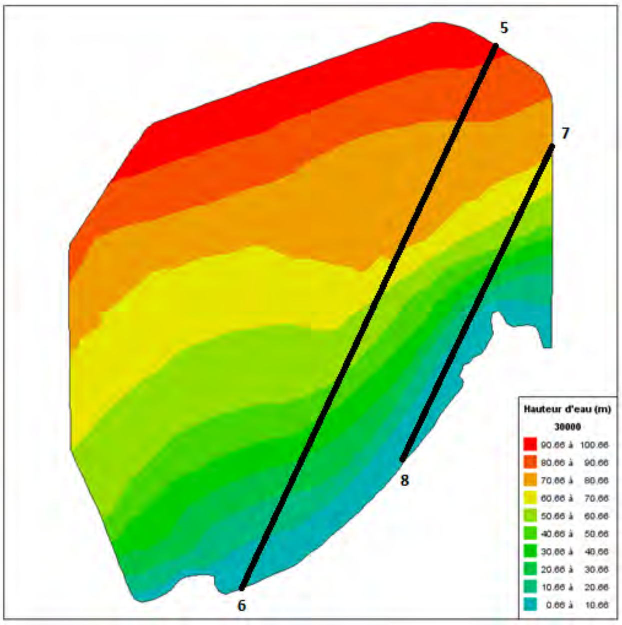


Figure 0-4: Placement of the profiles for Northeast Extreme Wave

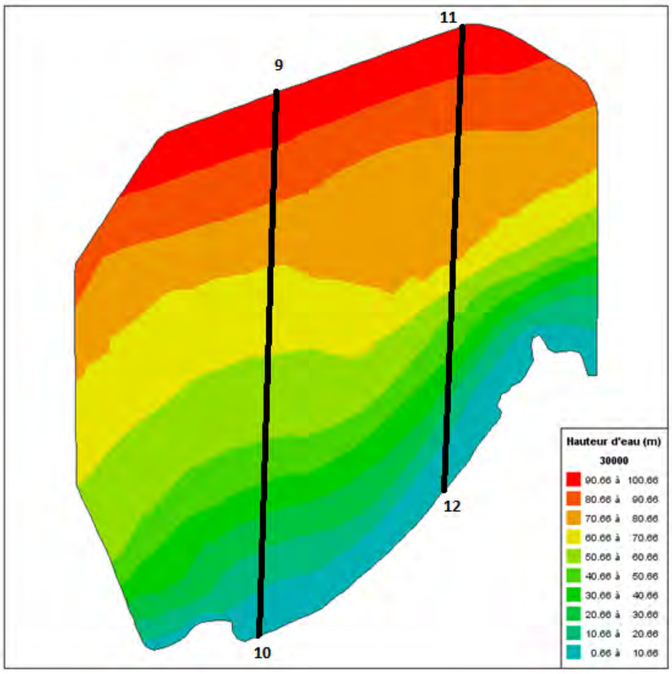


FIGURE 0-5: PLACEMENT OF THE PROFILES FOR NORTHEAST OPERATIONAL WAVES

North West Extreme Waves

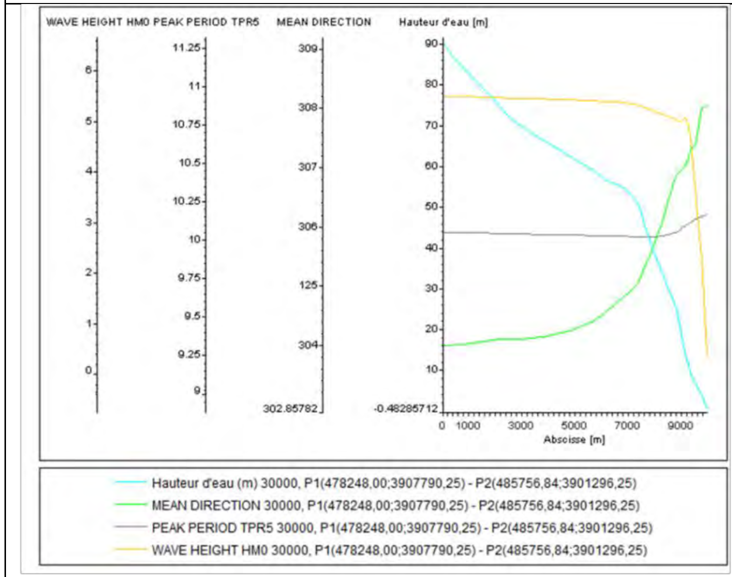


FIGURE 30 PROFILE 1-2

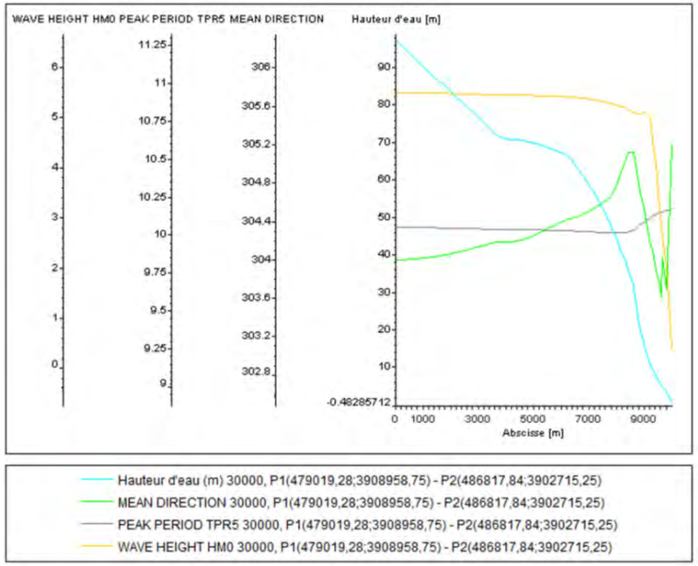


FIGURE 31 PROFILE 3-4

Northeast Extreme Waves

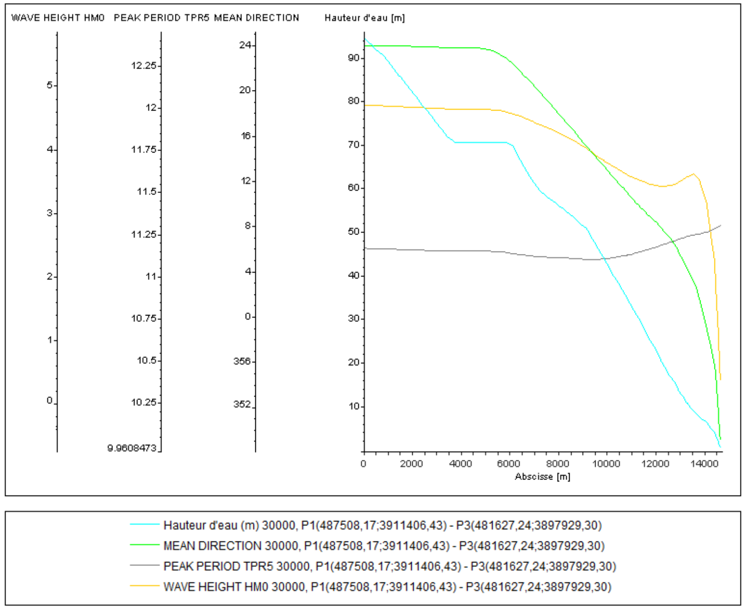


FIGURE 34 PROFILE 5-6

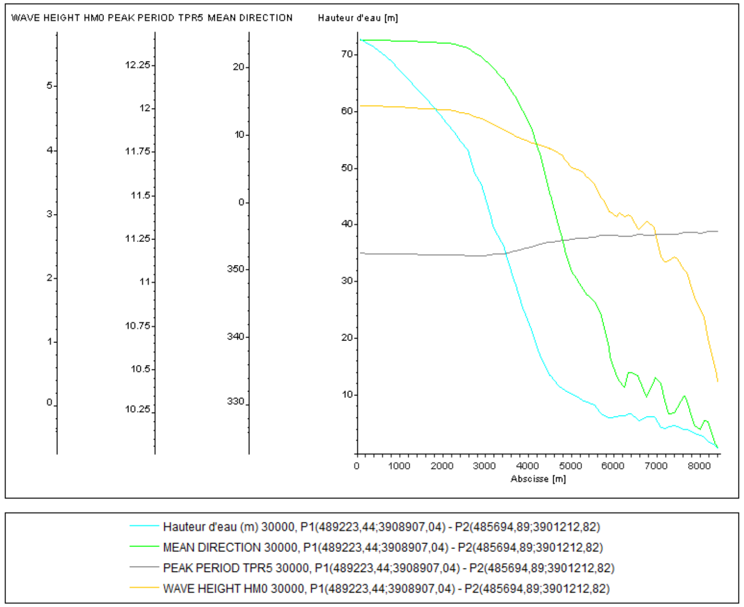


FIGURE 0-6 PROFILE 7-8

Northeast Operational Waves

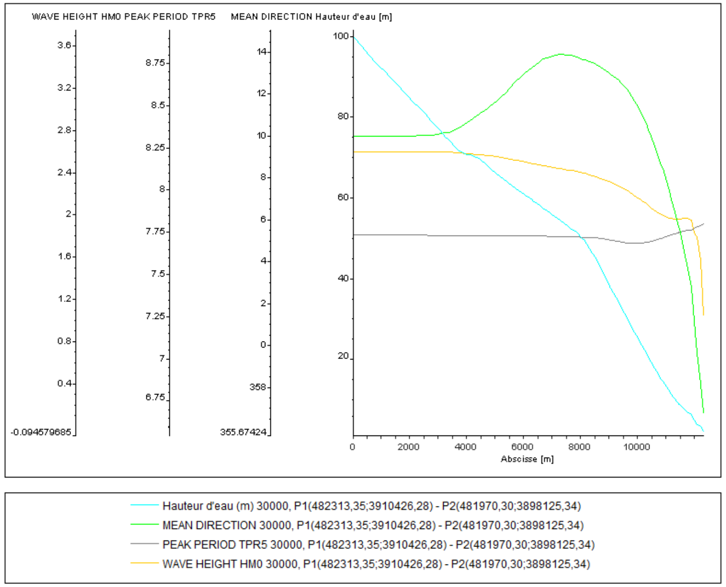


FIGURE 0-7 PROFILE 9-10

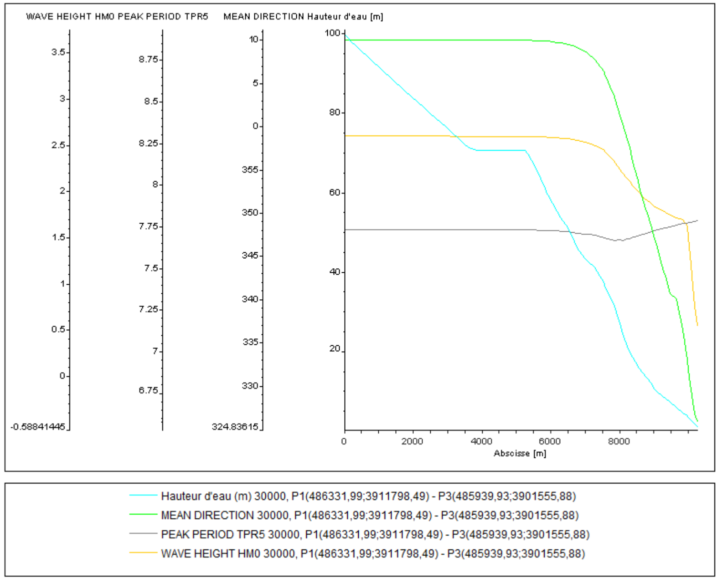


FIGURE 8 PROFILE 11-12

Appendix E: Vertical Breakwater

Goda defines a design wave height H_{max} as 1.8 times the significant wave height, or a maximum breaking wave height if the caisson is located in the surf zone. Goda defines the pressure distribution as trapezoidal on the seaward side of the caisson. The theoretical maximum elevation of wave pressure as η^* given by the equation:

$$\eta^* = 0.75\left(1 + \cos\beta\right) H_{max}$$

Where β is the angle of incoming waves relative to the normal direction of the caisson. As an additional safety factor, the wave direction is recommended to be reduced by 15 degrees towards the normal line, if greater than 15 degrees, to account for the uncertainty of estimating the incoming wave angle (Goda 2000).

Goda determines the wave pressures in front of the caisson with the maximum wave pressure (p_1) occurring at the design water level, as shown in the figure, and reduces linearly to a value at the bottom given by p_2:

$$p_1 = \frac{1}{2}\left(1 + \cos\beta\right)\left(\alpha_1 + \alpha_2 \cos^2\beta\right)\rho g H_{max}$$

$$p_2 = \frac{p_1}{\cosh kh}$$

Where k is the local wave number, L is the local wave length based on $T_{1/3}$, at depth h. This wave length was calculated from the dispersion relation at shallow water.

The caisson does not extend all the way to the sea bottom but to a level h_0 below design water level, therefore the pressure at the toe of the caisson is calculated by interpolation between p1 and p2 :

$$p_3 = \alpha_3 p_1$$

$$\begin{aligned} p_4 &= p_1\left(1 - \frac{h_c}{\eta^*}\right) && \text{if } \eta^* > h_c \\ &= 0 && \text{if } \eta^* \le h_c \end{aligned}$$

The values of the model coefficients are given by:

$$\alpha_1 = 0.6 + \frac{1}{2}\left(\frac{2kh}{\sinh 2kh}\right)^2$$

$$\alpha_2 = \min\left\{\frac{h_b - d}{3h_b}\left(\frac{H_{max}}{d}\right)^2, \frac{2d}{H_{max}}\right\}$$

$$\alpha_3 = 1 - \frac{h'}{h}\left(1 - \frac{1}{\cosh kh}\right)$$

The uplift pressure distribution is given by:

$$p_u = \frac{1}{2}\left(1 + \cos\beta\right)\alpha_1\alpha_3\rho g H_{max}$$

The resulting horizontal force F_H, the uplift force F_U can be defined from the surrounding pressure field:

$$F_H = \frac{1}{2}(p_1 + p_3)h' + \frac{1}{2}(p_1 + p_4)h_c^*$$

$$F_U = \frac{1}{2}p_u B$$

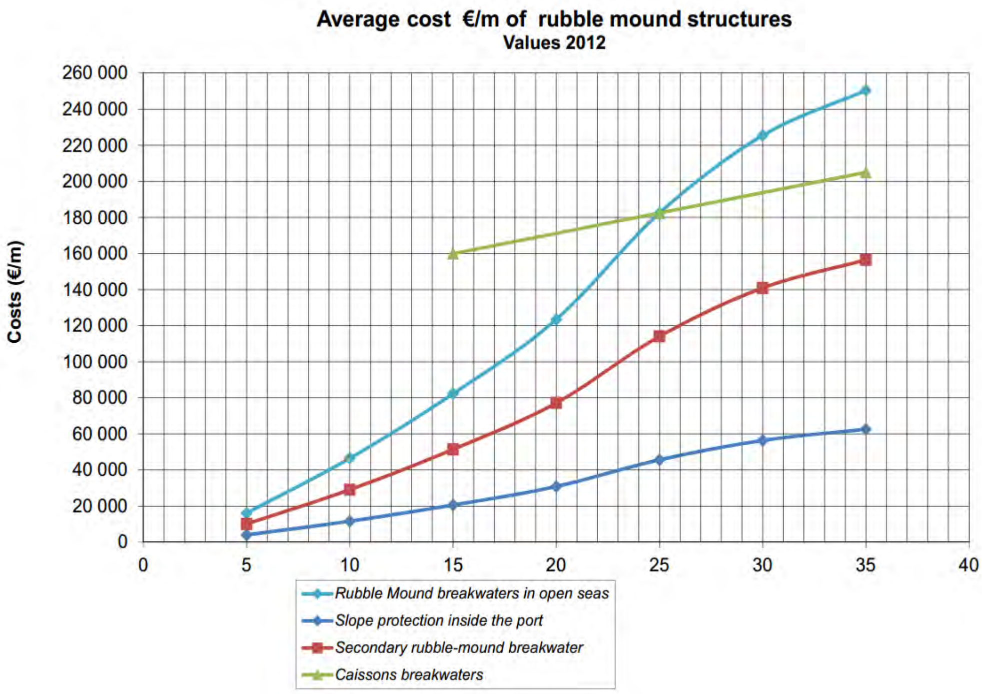
The resulting overturning moments around the heel of the structure M_H and M_U were calculated using:

$$M_H = \frac{1}{6}(2p_1 + p_3)h'^2 + \frac{1}{2}(p_1 + p_4)h'h_c^* + \frac{1}{6}(p_1 + 2p_4)h_c^{*2}$$

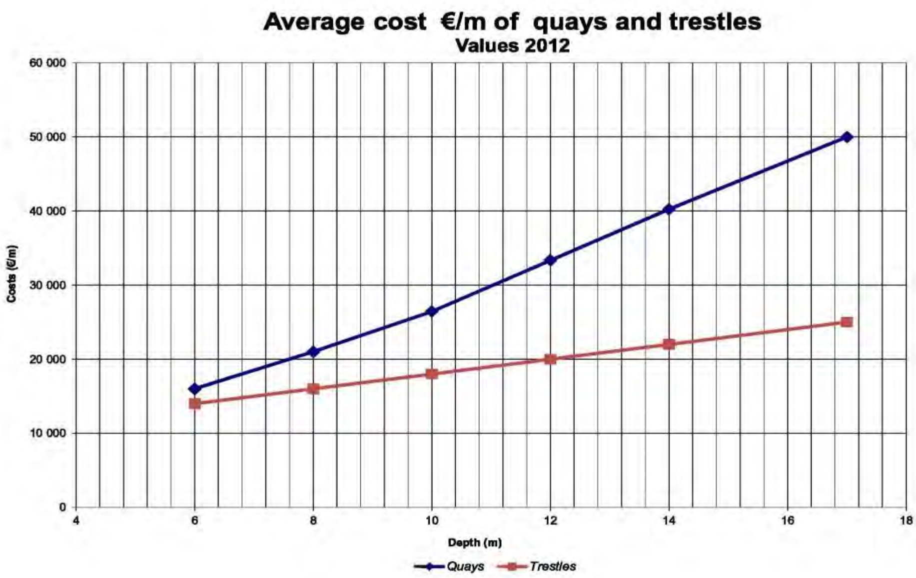
$$M_U = \frac{2}{3}F_U B$$

Appendix F: Unit Costs

1) Breakwater – rubble mound and caissons



2) Quay



3) Dredging fill

Item	Unit of measurement	Cost
Dredging in sand	€/m3	7
Dredging in soft clay	€/m3	10
Dredging in hard clay	€/m3	50

Dredging in rock	€/m3	80
Backfilling with dredged material	€/m3	3 in addition to dredging
Backfilling with sand dredged offshore the port	€/m3	15
Backfilling with quarry run (inshore)	€/m3	30
Backfilling with material excavated in land	€/m3	12
Substitution with quarry run (offshore)	€/m3	40
Mobilisation/demobilisation of one dredger	€	2 000 000

4) Jetty

JETTY - JETTY HEAD - Costs		
	Unit price (EURO)	
Mooring dolphins	1 700 000	each
Berthing dolphins	2 100 000	each
Platform	6 500 000	each
Walkways	9 000	per meter
Fender system	150 000	Each

Appendix G: Work Breakdown Structure

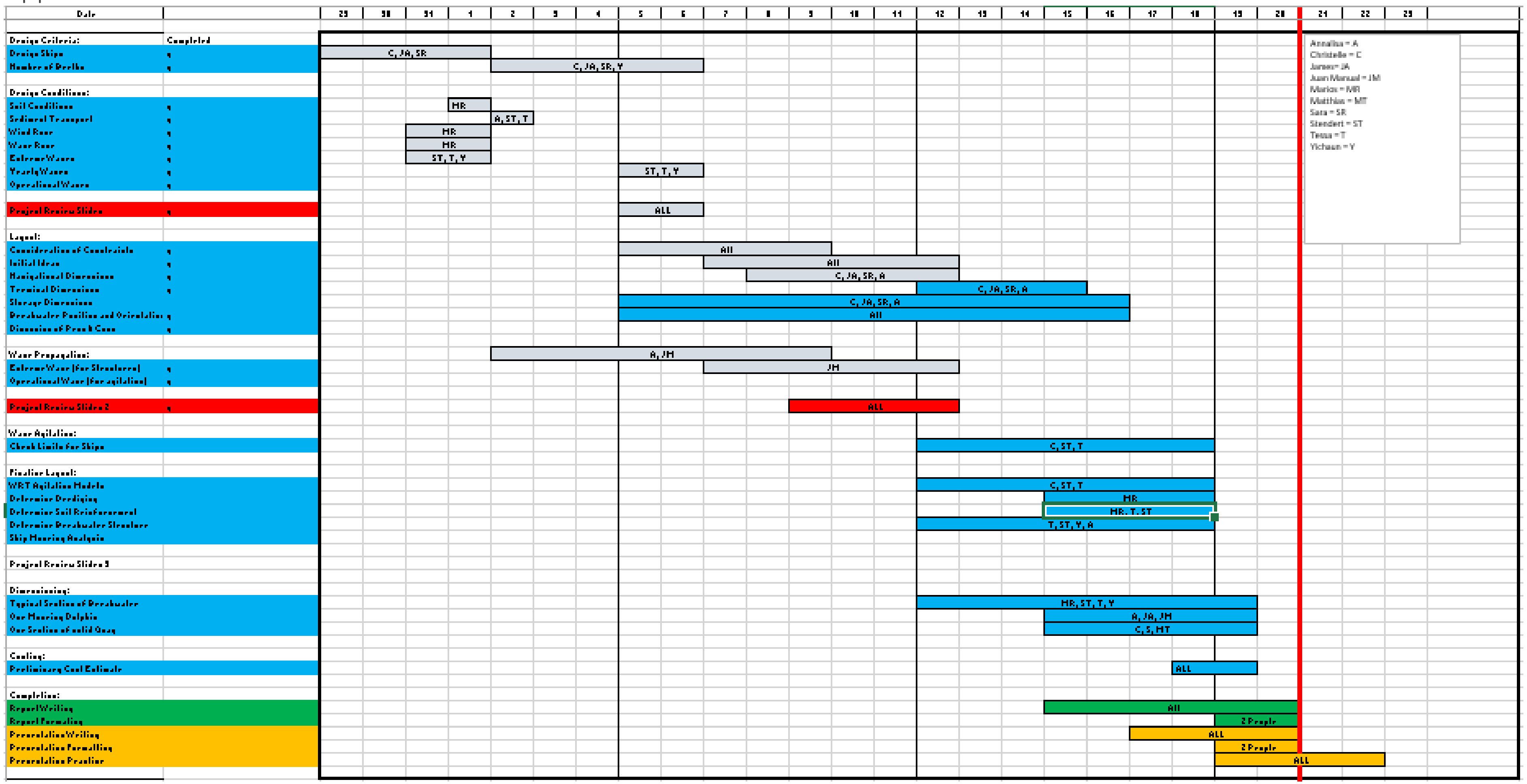


FIGURE 0-8: WORK BREAKDOWN STRUCTURE

Appendix A: Wave propagation

In this appendix, the steps followed in TOMAWAC are shown, as well as the graphic output for the different design situations.

Steps:

1. Bathymetry meshing -> input file was provided by ESITC Caen
2. Imposing boundary conditions on the direction of wave propagation to simulate the behaviour of water particles:
    - fixed - in the direction of wave propagation.
    - free - in all other directions
    - solid – the coast
3. Examining different cases based on the two main wave directions as determined by Wave analysis (which part) – NE and NW
    - different return periods: 100 years (extreme wave) and 1 year (yearly wave)
    - 1% operational waves
4. Parameters taken for all cases:
    - Hs – significant height from Wave analysis
    - Tp – peak period from Wave anaylis
    - direction from wind analysis
    - NW was taken 315° - 180° = 135 °
    - NE was taken as 20° + 180° = 200 °
5. Initial conditions at T0
    - initial still water level = 0.66 (Highest High water spring)
    - main direction = 0
    - Initial peak frequency = 0.01
    - minimal frequency = 1/2Tp
    - Hs= 0
    - type of initial directional spectrum = 6
6. Boundary conditions
    - boundary peak frequency = 1/Tp

- boundary significant wave height - depends on the case
- type of boundary direction spectrum = 6
- Discretisation
- number of directions = 24
- number of frequencies = 21
- minimal frequency = 1/2Tp

7. General parameters

- time step = 15 s
- number of time steps = 2000
- period of listing printout = 20s
- variables for 2D graphic printouts – Hm0 (wave height), Dmoy (wave direction), TRP5 (peak period), WD (water height)
- period for graphic printouts = 20s
- depth induced breaking dissipation = 1 (Battjes and Janssen model)
- number of breaking time step = 5
- bottom friction dissipation = 1

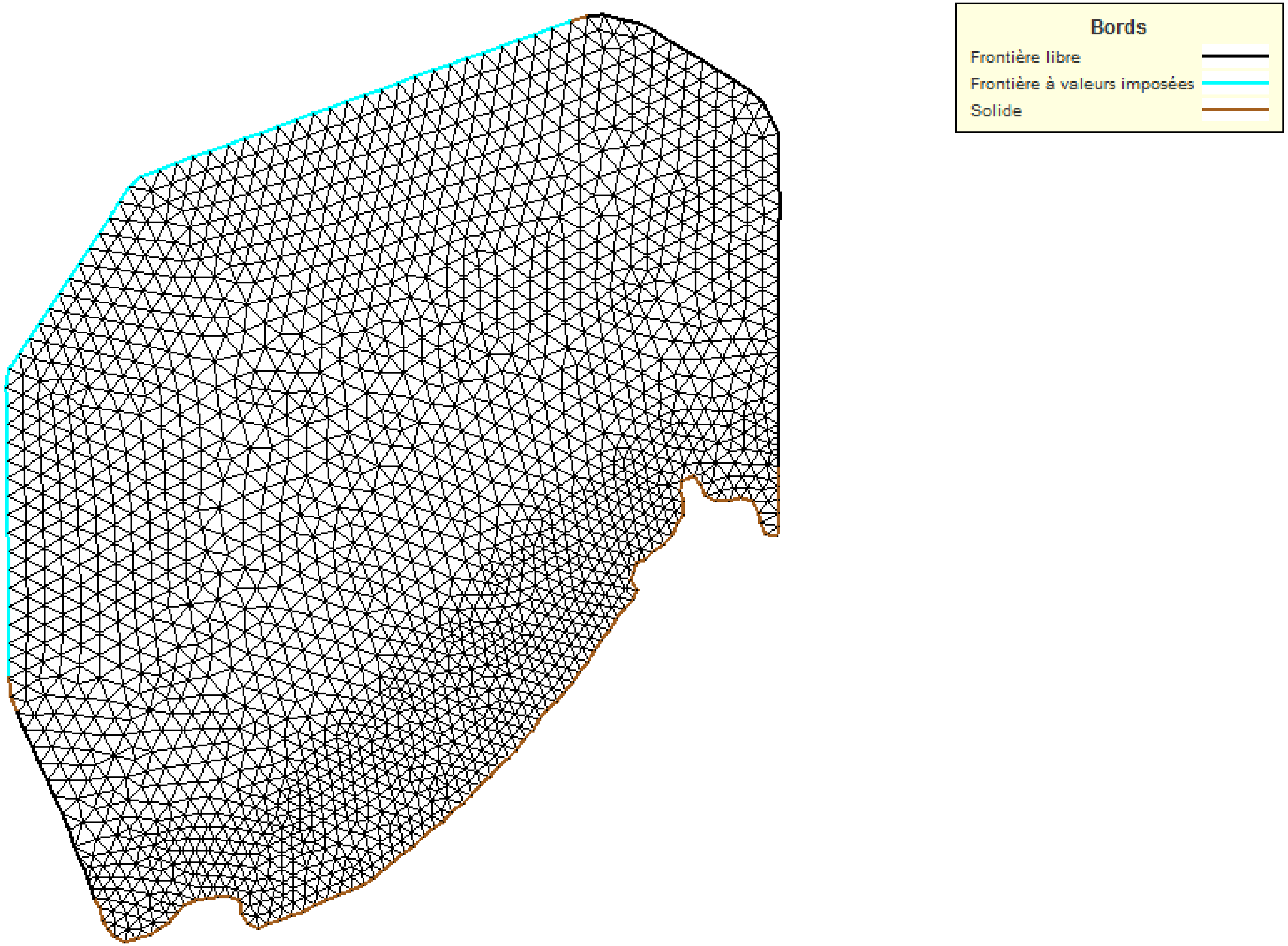
The obtained results show the water depth, mean direction of propagation, wave height and the peak period in both NE and NW directions.

Analysis of wave propagation

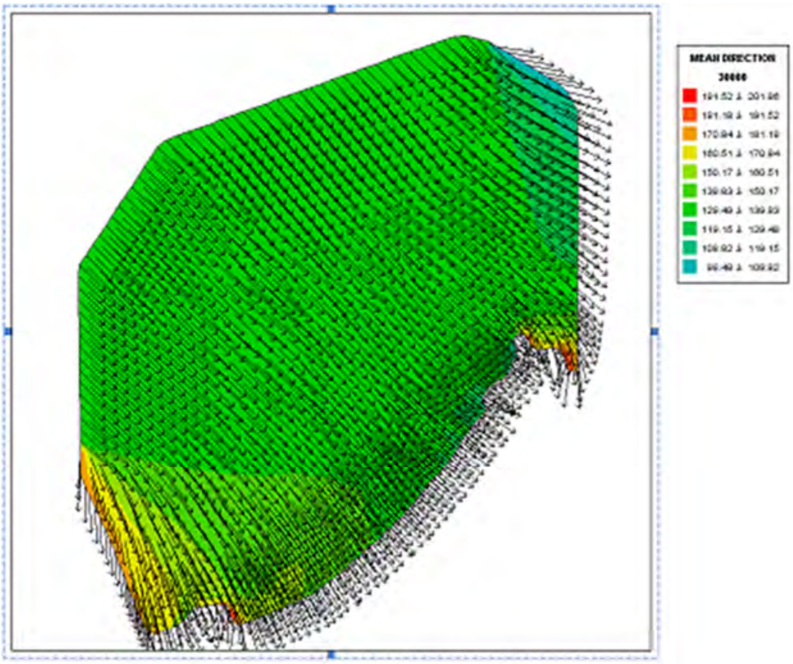
One of the design conditions is minimising the breakwater depth, therefore only depths up to -25 m are considered in this analysis.

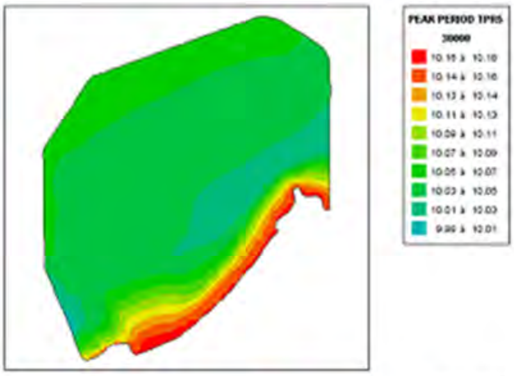
Sector 1 direction

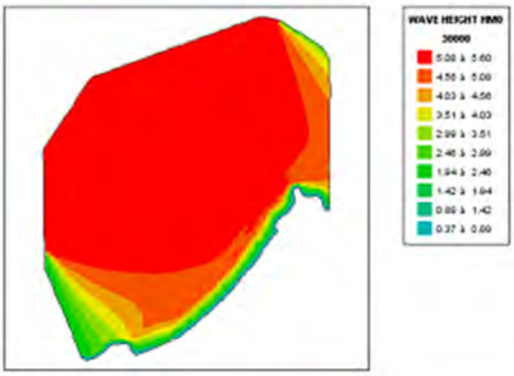
Mesh of bathymetry and boundary condition

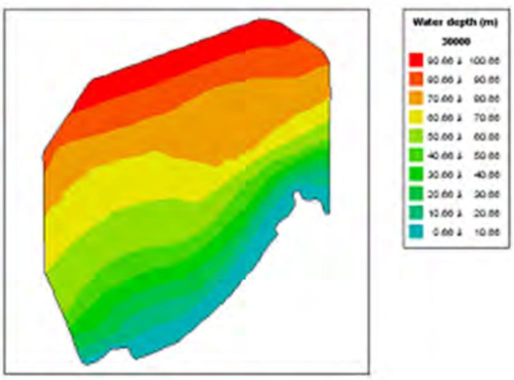


Graphics result about return period of 100 years

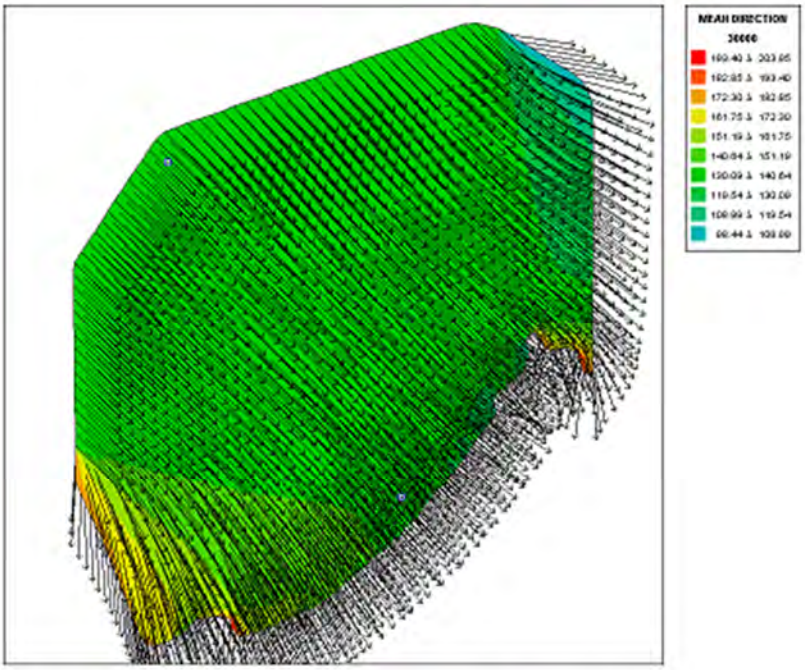


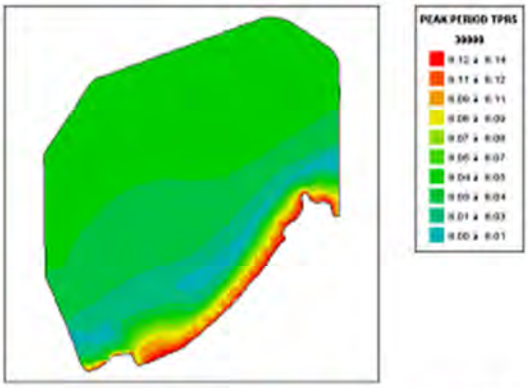


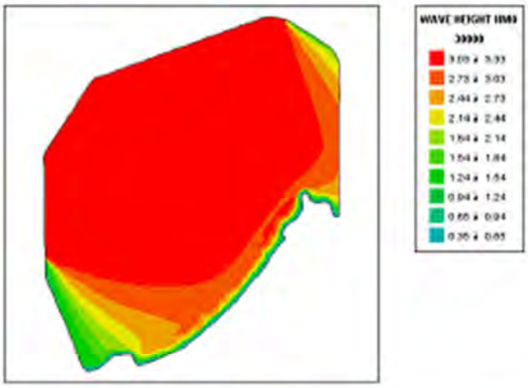


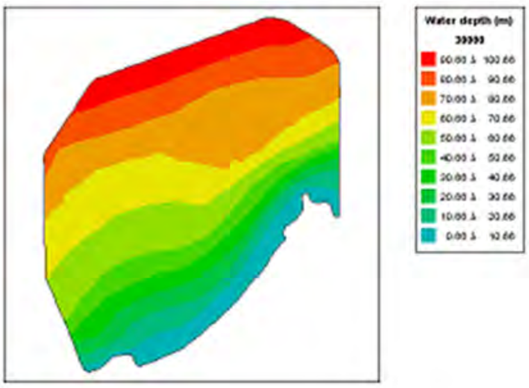


Graphics result about return period of 1 year

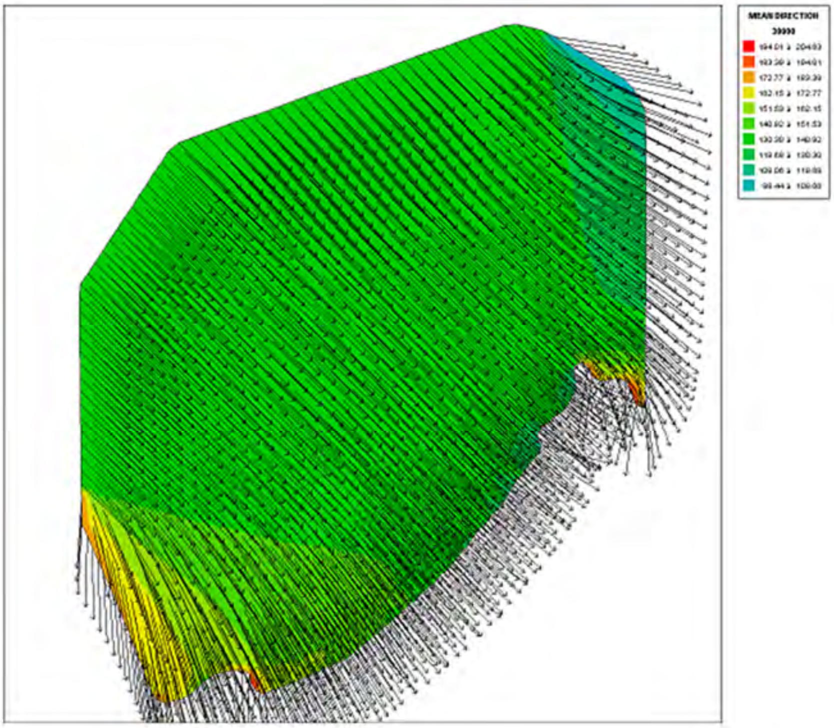


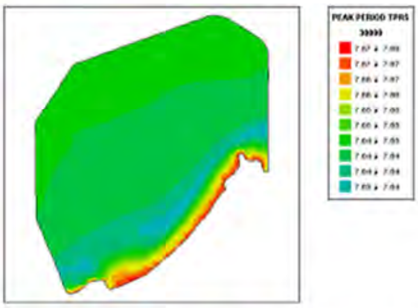


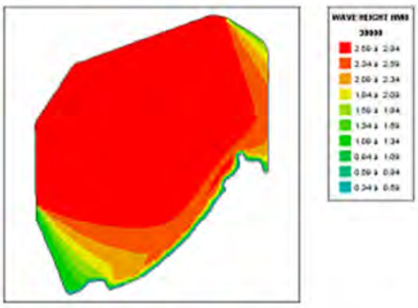


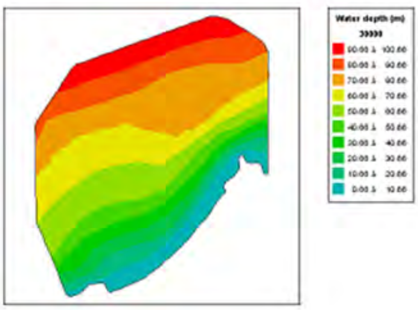


Graphics result about 1%



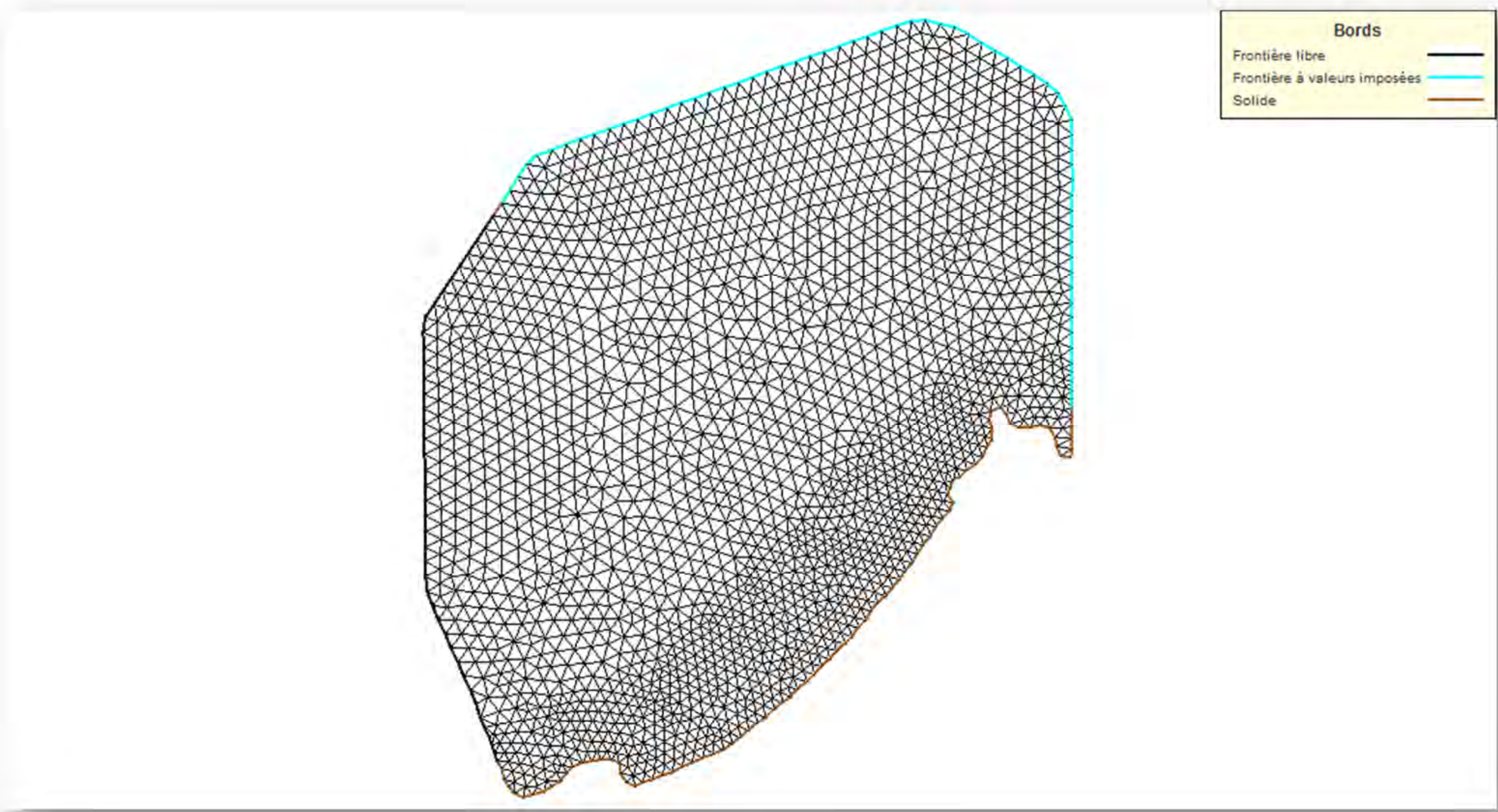




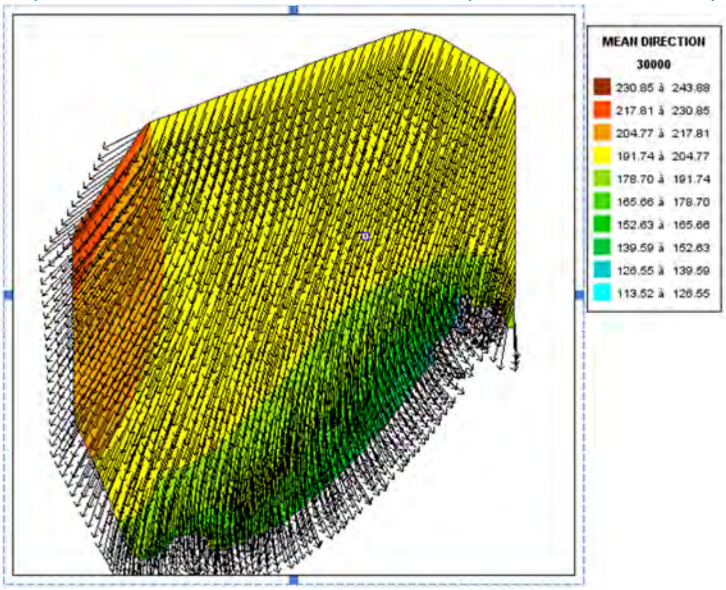


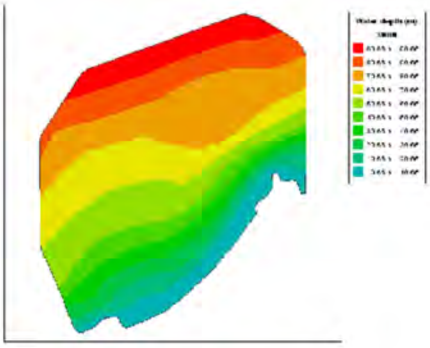
Sector 2 direction

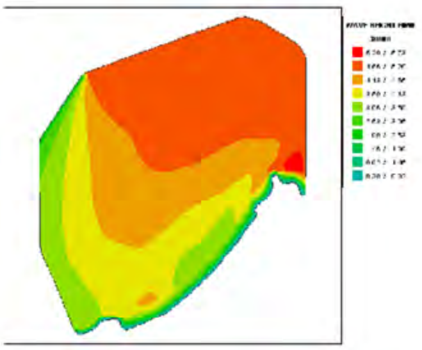
Mesh bathymetry and boundary conditions

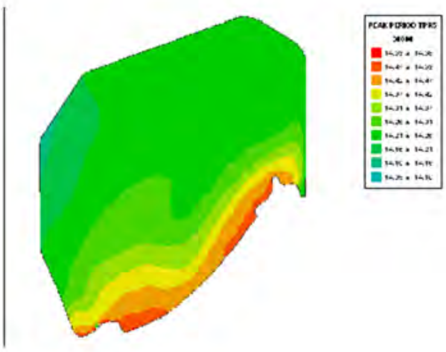


Graphics result about return period of 100 years

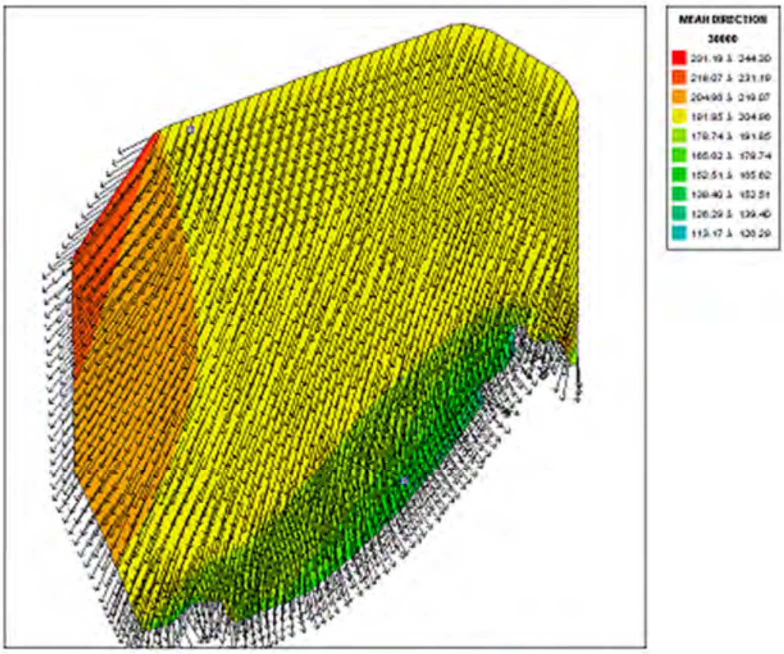


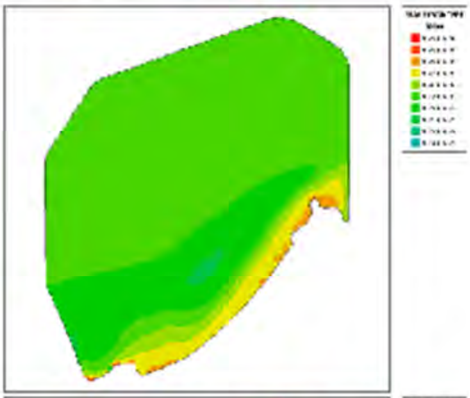


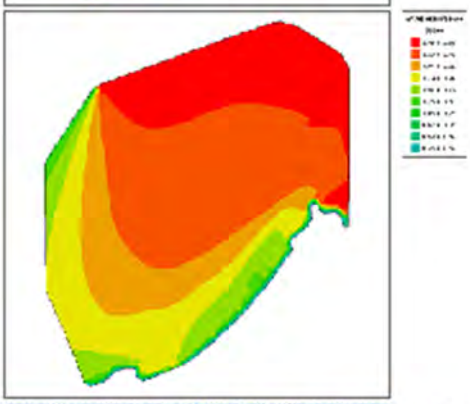


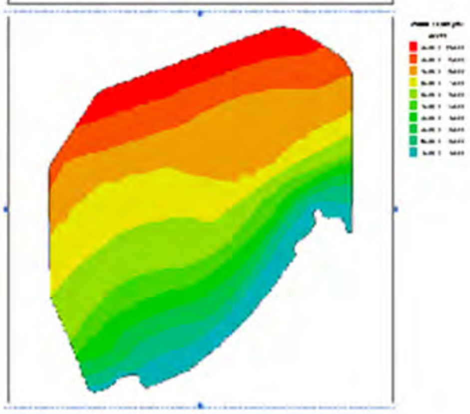


Graphics result about return period of 1 year

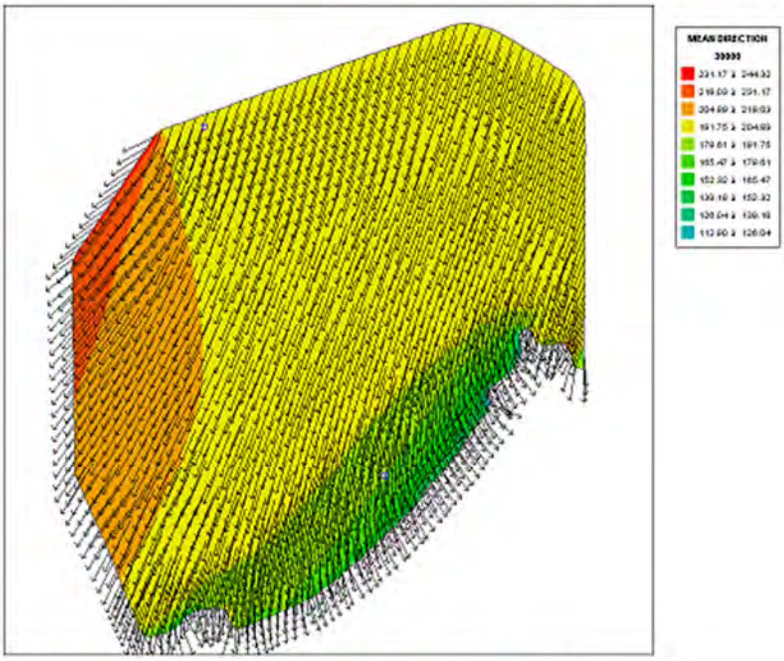


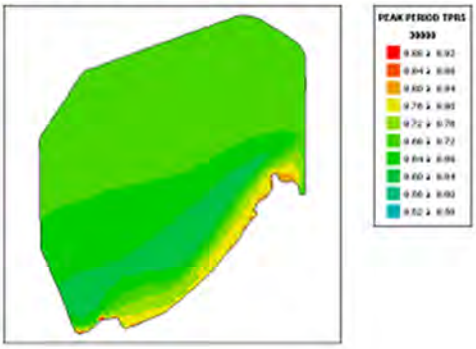


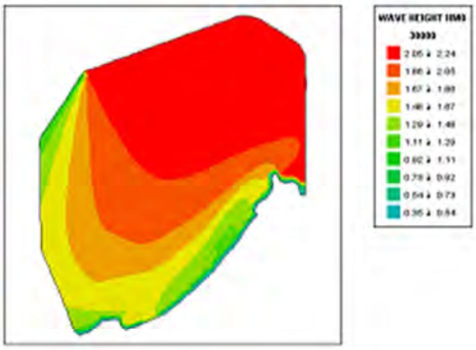


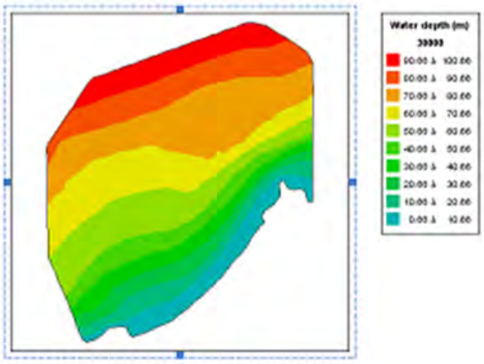


Graphics result about return period of 1%



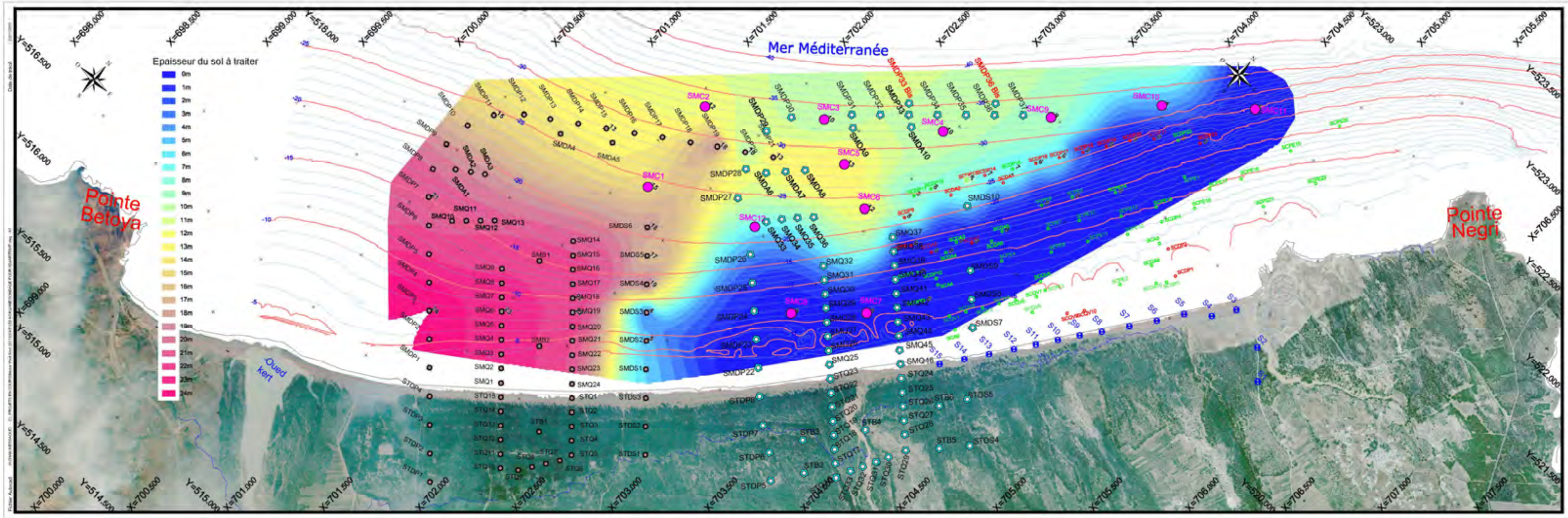






Appendix B

Clay deposits

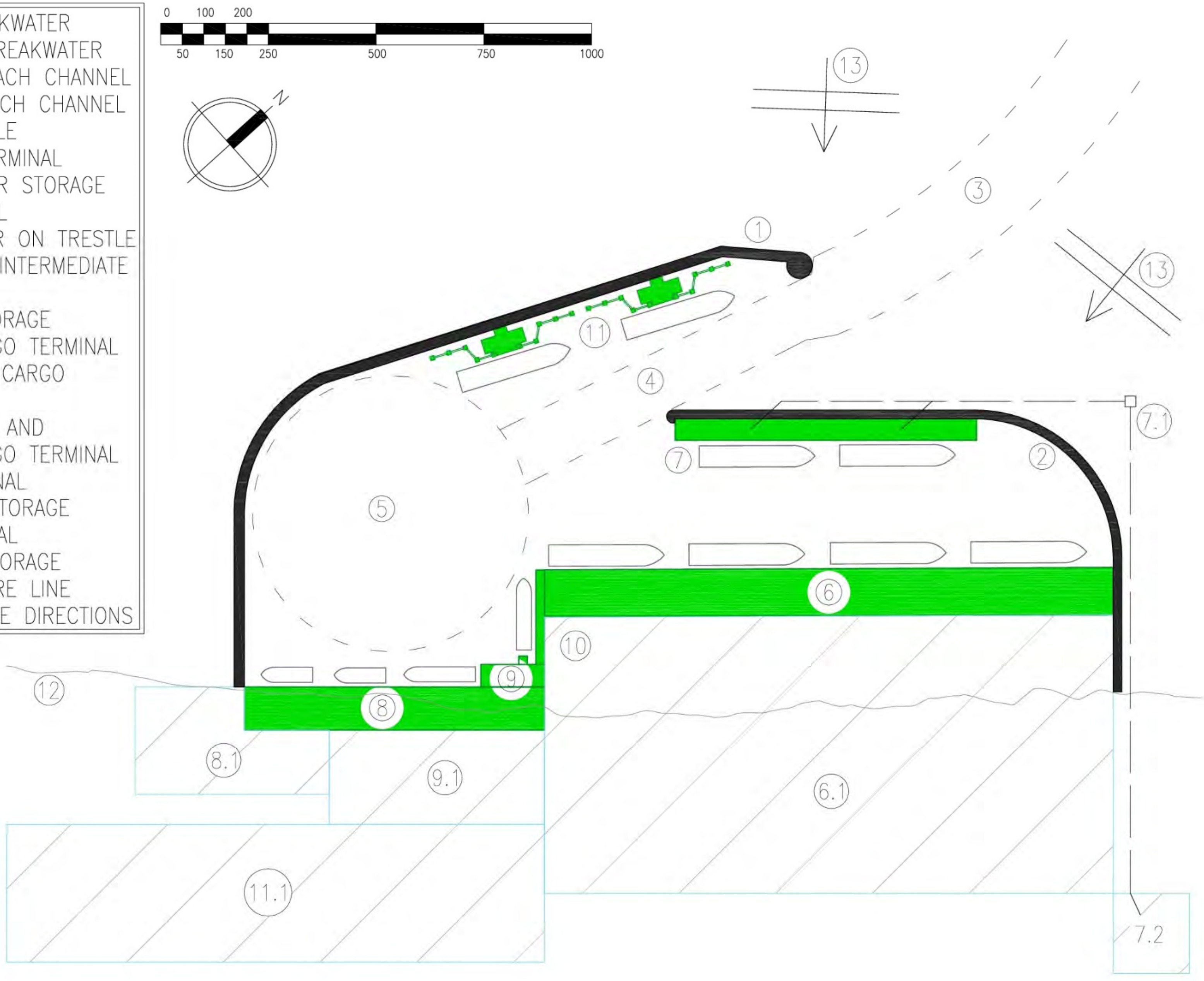


Appendix C

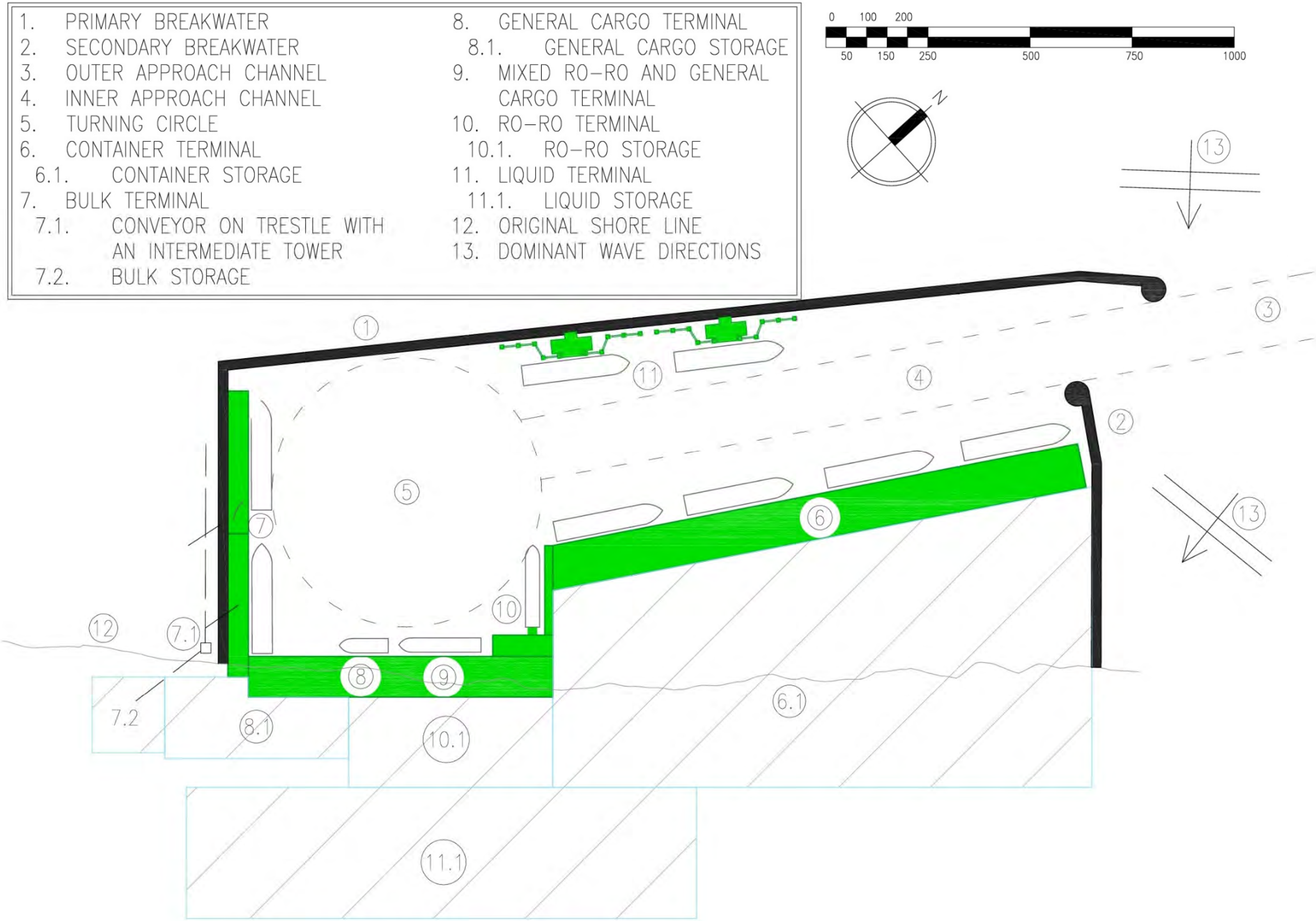
Layouts

1

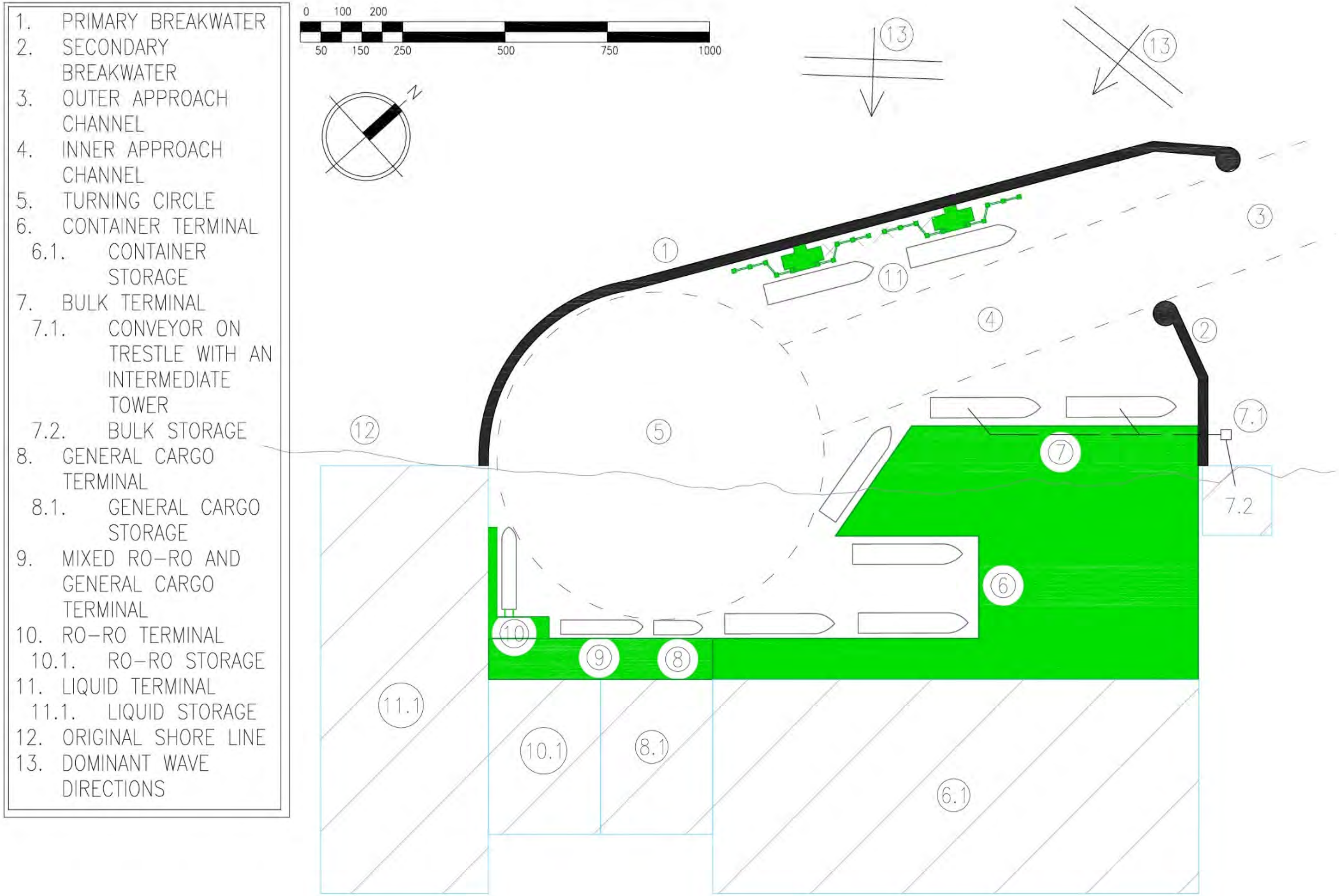
1. PRIMARY BREAKWATER
2. SECONDARY BREAKWATER
3. OUTER APPROACH CHANNEL
4. INNER APPROACH CHANNEL
5. TURNING CIRCLE
6. CONTAINER TERMINAL
   6.1. CONTAINER STORAGE
7. BULK TERMINAL
   7.1. CONVEYOR ON TRESTLE WITH AN INTERMEDIATE TOWER
   7.2. BULK STORAGE
8. GENERAL CARGO TERMINAL
   8.1. GENERAL CARGO STORAGE
9. MIXED RO-RO AND GENERAL CARGO TERMINAL
10. RO-RO TERMINAL
    10.1. RO-RO STORAGE
11. LIQUID TERMINAL
    11.1. LIQUID STORAGE
12. ORIGINAL SHORE LINE
13. DOMINANT WAVE DIRECTIONS



2


1. PRIMARY BREAKWATER
2. SECONDARY BREAKWATER
3. OUTER APPROACH CHANNEL
4. INNER APPROACH CHANNEL
5. TURNING CIRCLE
6. CONTAINER TERMINAL
6.1. CONTAINER STORAGE
7. BULK TERMINAL
7.1. CONVEYOR ON TRESTLE WITH AN INTERMEDIATE TOWER
7.2. BULK STORAGE
8. GENERAL CARGO TERMINAL
8.1. GENERAL CARGO STORAGE
9. MIXED RO-RO AND GENERAL CARGO TERMINAL
10. RO-RO TERMINAL
10.1. RO-RO STORAGE
11. LIQUID TERMINAL
11.1. LIQUID STORAGE
12. ORIGINAL SHORE LINE
13. DOMINANT WAVE DIRECTIONS
0 100 200
50 150 250 500 750 1000
N
13
1
2
3
4
5
6
7
7.1
7.2
8
8.1
9
10
10.1
11
11.1
12
6.1
13

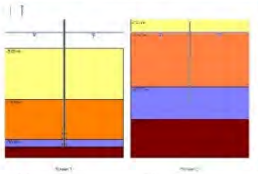
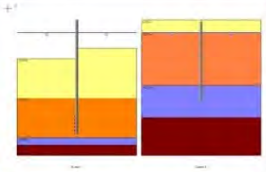
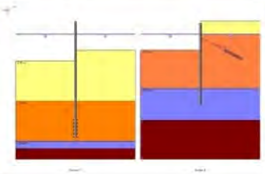
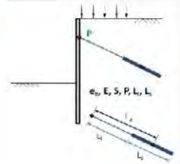
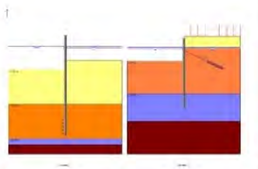
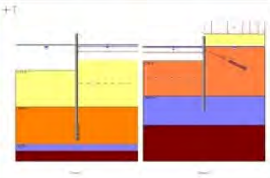
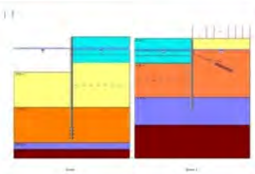
3


1. PRIMARY BREAKWATER
2. SECONDARY BREAKWATER
3. OUTER APPROACH CHANNEL
4. INNER APPROACH CHANNEL
5. TURNING CIRCLE
6. CONTAINER TERMINAL
6.1. CONTAINER STORAGE
7. BULK TERMINAL
7.1. CONVEYOR ON TRESTLE WITH AN INTERMEDIATE TOWER
7.2. BULK STORAGE
8. GENERAL CARGO TERMINAL
8.1. GENERAL CARGO STORAGE
9. MIXED RO-RO AND GENERAL CARGO TERMINAL
10. RO-RO TERMINAL
10.1. RO-RO STORAGE
11. LIQUID TERMINAL
11.1. LIQUID STORAGE
12. ORIGINAL SHORE LINE
13. DOMINANT WAVE DIRECTIONS
0 100 200
50 150 250 500 750 1000
N
13
13
1
2
3
4
5
6
7
7.1
7.2
8
9
10
11
12
11.1
10.1
8.1
6.1

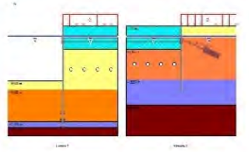
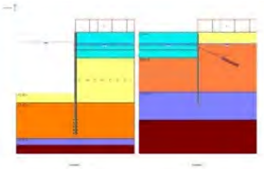
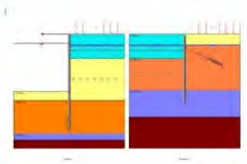
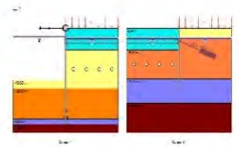
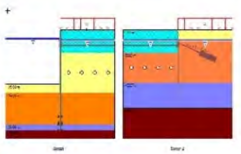
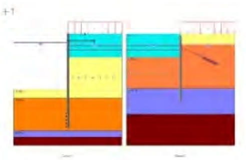
Appendix D

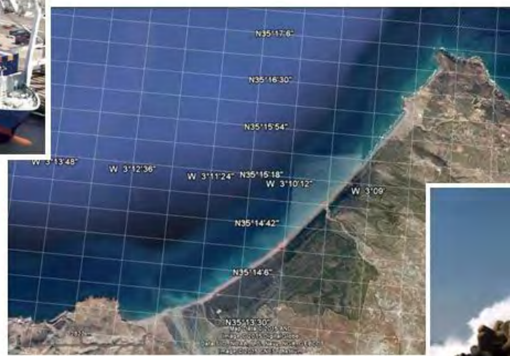
Quay wall phases

Phases 0-5

N°	Phases	Description	Data	Verifications Wall 1/Wall2		
0		Input data about soil, title and options and retaining wall have been entered		**Checking safety against failure on the passive side of the wall** Mobilised passive earth pressure: Characteristic value: Bt,k = 804,82 kN/ml Design value: Bt,d = 1086,51 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 13963,09 kN/ml Design value: Bm,d = 12693,71 kN/ml	**Checking safety against failure on the passive side of the wall** Mobilised passive earth pressure: Characteristic value: Bt,k = 1330,48 kN/ml Design value: Bt,d = 1796,15 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 22639,88 kN/ml Design value: Bm,d = 20581,71 kN/ml	Bt,d < Bm,d **Ok**
1		An excavation of 8 meters on the left of the first wall has been done		**Checking safety against failure on the passive side of the wall** Mobilised passive earth pressure: Characteristic value: Bt,k = 804,82 kN/ml Design value: Bt,d = 1086,51 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 13963,09 kN/ml Design value: Bm,d = 9973,63 kN/ml	**Checking safety against failure on the passive side of the wall** Mobilised passive earth pressure: Characteristic value: Bt,k = 957,33 kN/ml Design value: Bt,d = 1292,39 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 7463,90 kN/ml Design value: Bm,d = 5331,38 kN/ml	Bt,d < Bm,d **Ok**
2		An other excavation of 7,36 meters oh the left of the wall 2 has been done. An anchor to the second pile in order to increase the stability has been placed	 	**Checking safety against failure on the passive side of the wall** Mobilised passive earth pressure: Characteristic value: Bt,k = 804,82 kN/ml Design value: Bt,d = 1086,51 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 13963,09 kN/ml Design value: Bm,d = 12693,71 kN/ml	**Checking safety against failure on the passive side of the wall** Mobilised passive earth pressure: Characteristic value: Bt,k = 1124,30 kN/ml Design value: Bt,d = 1517,81 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 7463,90 kN/ml Design value: Bm,d = 6785,37 kN/ml	Bt,d < Bm,d **Ok**
3		The first linking anchor has been placed and the first consolidation on the right of the wall 2 has been considered. The distance between the 2 anchors is 30 meters.The prestress is about 10 kN on the liking anchor.		**Checking safety against failure on the passive side of the wall** Mobilised passive earth pressure: Characteristic value: Bt,k = 804,84 kN/ml Design value: Bt,d = 1086,53 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 13963,09 kN/ml Design value: Bm,d = 12693,71 kN/ml	**Checking safety against failure on the passive side of the wall** Mobilised passive earth pressure: Characteristic value: Bt,k = 1124,30 kN/ml Design value: Bt,d = 1517,81 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 7463,90 kN/ml Design value: Bm,d = 6785,37 kN/ml	Bt,d < Bm,d **Ok**
4		The characteristic of the land has been increased with new soil properties. An other linking anchor with the same characterisitc as the previous linking anchor has been placed. The distance between these 2 linking is about 1,5m.		**Checking safety against failure on the passive side of the wall** Mobilised passive earth pressure: Characteristic value: Bt,k = 1370,74 kN/ml Design value: Bt,d = 1850,50 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 13963,09 kN/ml Design value: Bm,d = 9973,63 kN/ml	**Checking safety against failure on the passive side of the wall** Mobilised passive earth pressure: Characteristic value: Bt,k = 1744,35 kN/ml Design value: Bt,d = 2354,87 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 20206,77 kN/ml Design value: Bm,d = 14433,41 kN/ml	Bt,d < Bm,d **Ok**
5		The characterisitc of the land have been improved with the following data.		**Checking safety against failure on the passive side of the wall** Mobilised passive earth pressure: Characteristic value: Bt,k = 1370,74 kN/ml Design value: Bt,d = 1850,50 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 13963,09 kN/ml Design value: Bm,d = 9973,63 kN/ml	**Checking safety against failure on the passive side of the wall** Mobilised passive earth pressure: Characteristic value: Bt,k = 1744,35 kN/ml Design value: Bt,d = 2354,87 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 20206,77 kN/ml Design value: Bm,d = 14433,41 kN/ml	Bt,d < Bm,d **Ok**

Phases 6-11

N°	Phases	Description	Data	Verification Wall 1/ Wall 2		
6		A second excavation of 8 meters has been realized on the left on the wall 1. A second consolidation on the right of the wall 2 has been done.		Checking safety against failure on the passive side of the wall Mobilised passive earth pressure: Characteristic value: Bt,k = 1230,50 kN/ml Design value: Bt,d = 1661,17 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 5501,31 kN/ml Design value: Bm,d = 3929,51 kN/ml	Checking safety against failure on the passive side of the wall Mobilised passive earth pressure: Characteristic value: Bt,k = 1897,61 kN/ml Design value: Bt,d = 2561,77 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 23842,79 kN/ml Design value: Bm,d = 17030,56 kN/ml	Bt,d < Bm,d Ok
7		The hydraulic action have been insered		Checking safety against failure on the passive side of the wall Mobilised passive earth pressure: Characteristic value: Bt,k = 1197,94 kN/ml Design value: Bt,d = 1617,21 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 5501,31 kN/ml Design value: Bm,d = 3929,51 kN/ml	Checking safety against failure on the passive side of the wall Mobilised passive earth pressure: Characteristic value: Bt,k = 1893,56 kN/ml Design value: Bt,d = 2556,30 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 23842,79 kN/ml Design value: Bm,d = 17030,56 kN/ml	Bt,d < Bm,d Ok
8		The shiploader live load have been put		Checking safety against failure on the passive side of the wall Mobilised passive earth pressure: Characteristic value: Bt,k = 1198,13 kN/ml Design value: Bt,d = 1617,47 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 5501,31 kN/ml Design value: Bm,d = 5001,20 kN/ml	Checking safety against failure on the passive side of the wall Mobilised passive earth pressure: Characteristic value: Bt,k = 1895,78 kN/ml Design value: Bt,d = 2559,31 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 23842,79 kN/ml Design value: Bm,d = 21675,26 kN/ml	Bt,d < Bm,d Ok
9		The bollards live load have been inserted		Checking safety against failure on the passive side of the wall Mobilised passive earth pressure: Characteristic value: Bt,k = 1198,85 kN/ml Design value: Bt,d = 1618,45 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 5501,31 kN/ml Design value: Bm,d = 5001,20 kN/ml	Checking safety against failure on the passive side of the wall Mobilised passive earth pressure: Characteristic value: Bt,k = 1904,44 kN/ml Design value: Bt,d = 2571,00 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 23842,79 kN/ml Design value: Bm,d = 21675,26 kN/ml	Bt,d < Bm,d Ok
10		Desactivation of (AM) and (PR) and the opposite direction shiploader live load (PR) of phase 8 have been put		Checking safety against failure on the passive side of the wall Mobilised passive earth pressure: Characteristic value: Bt,k = 1198,11 kN/ml Design value: Bt,d = 1617,44 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 5501,31 kN/ml Design value: Bm,d = 5001,20 kN/ml	Checking safety against failure on the passive side of the wall Mobilised passive earth pressure: Characteristic value: Bt,k = 1897,68 kN/ml Design value: Bt,d = 2561,87 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 23842,79 kN/ml Design value: Bm,d = 21675,26 kN/ml	Bt,d < Bm,d Ok
11		Fenders live load have been put		Checking safety against failure on the passive side of the wall Mobilised passive earth pressure: Characteristic value: Bt,k = 1197,63 kN/ml Design value: Bt,d = 1617,07 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 5501,31 kN/ml Design value: Bm,d = 5001,20 kN/ml	Checking safety against failure on the passive side of the wall Mobilised passive earth pressure: Characteristic value: Bt,k = 1878,83 kN/ml Design value: Bt,d = 2536,42 kN/ml Limiting passive earth pressure: Characteristic value: Bm,k = 23842,79 kN/ml Design value: Bm,d = 21675,26 kN/ml	Bt,d < Bm,d Ok



NADOR WEST PORT (MOROCCO)

Scarlett Beauducel, Giovanni Salomone, Simona Bofelli, Juan Antonio Rebollo, John C.L. Mayson, Jochem Roubos, Eva Alcolea, Wojciech Serwatka

ESITC Caen – Workshop 2017

Preface

This is the final report describing the New Port design on the Mediterranean coast of Morocco, approximately 15 miles to NW from the city of 'Nador'. This study has been undertaken by an international group of 10 undergraduate students from 6 European universities as a summer programme at ESITC Caen. The organisers provided with a wide range of data and criteria which were taken into account to produce functional, economical and sustainable design.

The report is targeted towards our supervisors at ESITC Caen and anyone interested in our port design. The design, together with the conclusions and recommendations should be useful for anyone interested in designing and/or building a similar port.

Our gratitude goes to the supervisors at the department of Port Engineering at ESITC Caen for organising and coordinating the workshop. Special thanks to the workshop coordinator Mr. Carpentier, technical coordinator Mr. Silva and Management coordinator Mr. Sibony. We are thankful for all the teachers and professors from different universities for the lectures they prepared. We would also like to thank the cooperating companies for their financial support.

Abstract

Government of Morocco has planned the construction of a new port in Baie Betoya, west of the city of Nador. This project is considered part of a regional development plan that aims to substantially support the economic development of the area. Under this context, this port is expected to involve a mixed cargo port including terminals for containers, hydrocarbons, bulk (including coal) and Ro-Ro. Additionally, further development of the port is expected.

This team faced this project with a main objective; to study a conceptual design that included a general treatment of every major element of the project. Consequently, this project serves as a first approach and estimation for a hypothetic constructive project.

Project philosophy takes shape as numerous different studies and design calculations, that can be summarized as statistical, quantitative and qualitative analysis of the site conditions, definition of the design basis according to the requirements stablished by the contractor, discussion and definition of an initial port layout and sample designs of the main port structures, i.e. main breakwater, solid quay walls and dolphins.

These calculations were supported by an organization plan and an additional economical study, taking into account a first approach regarding volumes and materials of the construction works, as well as the associated costs.

This project concludes that an approximate investment of 460 million euros is required for the first stage of the port. This expense would translate in a major construction operation including 2500 metres in terms of rubble mound breakwaters, 2900 metres of solid quay wall and 2 dolphin platforms. Additionally, earthworks up to a total of 22 million cubic meters would be required.

These structures would allow for the exploitation of 305 hectares in terms of storage area, handling traffic volumes of 3 million TEU (containers), 25 million DWT (oil), 7 million DWT (Coal) and 3 million DWT (general cargo).

The queuing theory is used to determine the number of berths. In the final lay-out there are 4 container berths, 2 oil jetty's, 2 coal berths, 2 general cargo berths and 1 Ro-Ro berth. One of the general cargo berth is multifunctional so it can be used for general cargo and Ro-Ro.

Table of Contents

List of Figures

List of Tables

1. Introduction

The government of Morocco is developing a major port and an industrial compound. The new port is to be placed on the Mediterranean coast of Morocco, approximately 15 miles to NW from the city of Nador (see left side of Figure 1 below), in a site called ‘Baie de Betoya’. The proposed site is a 5 miles long stretch of relatively straight coast, with a dry river bed (oued) to the SW. There is also a permanently dry river bed approximately in the middle of the area studied.

The location for the new port has been chosen to be approximately 1.5 mile NE from the mouth of oued (see right side of Figure 1 below) basing on the combination of geological and topological conditions.

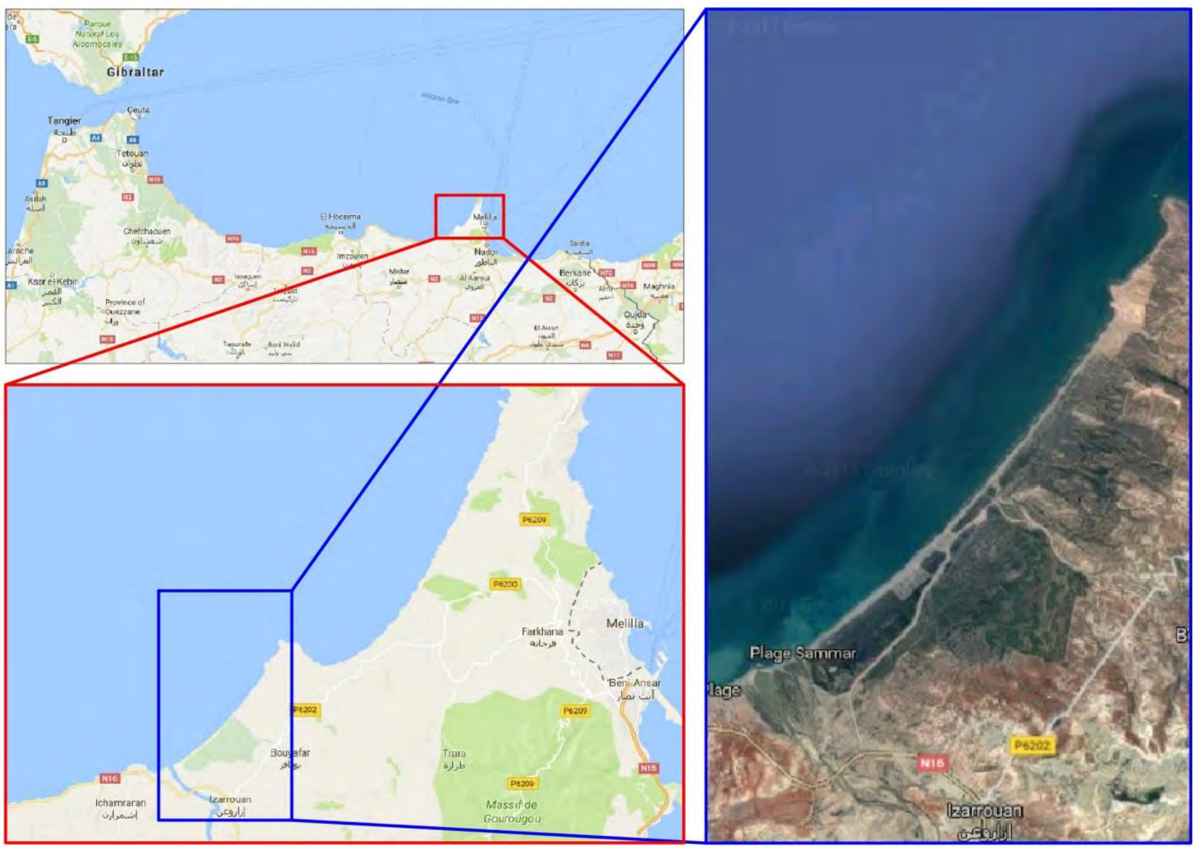


Figure 1: Port of Nador West

There is an existing national road 1.5 mile to SE, with a stretch of a local road running parallel to the dry river bed. Following the satellite photos analysis it has been noticed that the dry river bed is currently being used by the local community as a dirt road, which suggests that it could be upgraded to a tarmac road with relatively low costs.

The main purpose of the Nador West port is to become a new hub for container transport in this part of the Mediterranean Sea. The client also requires a certain capacity for shipment of hydrocarbons (both crude and refined products, coal, general cargo and Ro-Ro mode).The required capacities are given in Table 1 below. Nador West is required to remain operational for 350 days a year and working for 24 hours a day.

Traffic Typology	Unit	Volume	Note
Containers	TEU	3,000,000	1 TEU – twenty-foot equivalent unit
Oil	Tons	25,000,000	
Coal	Tons	7,000,000	
General cargo	Tons	3,000,000	1,000,000 of which transported in Ro-Ro mode

Table 1: Traffic data

A variety of data and design conditions have been analysed and the results presented in this report. On this basis 3 preliminary layouts were prepared and compared using multi-criteria analysis. Future expansion was considered to be an important factor impacting the decision of the final design. Following the decision on the final layout, wave agitation software (ARTEMIS) was used to run a check if the port layout (namely breakwaters and terminals arrangement) are adequate and fulfil requirements for various types of vessels.

As a part of this report the team prepared analysis and design of typical structural components, such as breakwaters, quay walls and mooring dolphins using variety of software, such as MS Excel, TALREN V5 and more.

The main design criteria specified by the client are:

- Port entrance to North-East
- Minimise the breakwater costs
- Maximise the dredging
- Ease of future expansion

Additional design criteria employed in the design:

- Safety for navigation and operation of port

2. Design criteria

Introduction

In the previous chapter the design criteria is given for the design of the lay-out. First the dimensions of the design ships are given by using the ‘Scope of Works’ and the Excel file ‘design_ship’. The number of berths are calculated by two approaches; the simple approximation and the queuing theory. The quay length is determined by using the average length of the ships. At the end of the chapter the storage areas per terminal are given.

Design ship

For the container ships an assumption have to be made to find the amount of motherships and feeders. Also for the oil tankers an assumption is needed to determine the amount of crude oil tankers, refined product tankers and feeders. For coal, general cargo and ro-ro ships there is only one kind of ship that has to be taken into account.

Container ships

The container terminal needs to be designed for two kind of ships; mother ships up to 18,000 TEU and Feeders up to 5,000 TEU.

Excel file:
Average capacity motherships = 14,239 TEU
Average capacity feeders = 3,663 TEU
Average capacity of a container ship = 6,402 TEU
$L_{s,mother}$ =372 m
$L_{s,feeder}$ = 250 m

$$L_q = 0.259 * 372 + 0.741 * 250 = 281.6\ m$$

An assumption is made that 50% of the volume will be imported by motherships and 50% of the volume will be exported by feeders.

Amount of motherships = 1,500,000/14,239 = 106 ships (25.9%)
Amount of feeders = 1,500,000/3,663 = 410 ships (74.1%)
Average capacity ship = (25.9 * 14,239 + 74.1 * 3,663) / 100 = 6402 TEU

Oil

These tankers need to be able to transport crude oil and refined products (up to 65,000 DWT). The products may be handled at the same berth.

Excel file:
Average capacity of crude oil tankers = 206,777 DWT
Average capacity of product tankers = 36,211 DWT

An assumption is made that 50% of the volume is crude oil, 25% of the volume is transported to smaller carriers and 25% of the volume is used to refine the product.

25,000,000 * 0.5 = 12,500,000 tons of crude oil (import)
Amount of ships = 12,500,000 / 206,777 = 61 ships (22.2%)
25,000,000 * 0.25 = 6,250,000 tons of refined product
Amount of ships = 6,250,000 / 36,211 = 173 ships (62.9%)
25,000,000 * 0.25 = 6,250,000 tons of crude oil (export)
Amount of ships = 6,250,000 / 154,988 = 41 ships (14,9%)

Average capacity of a ship = (0.222 * 206,777 + 0.629 * 36,211 + 0.149 * 154,988) = 91,774 DWT

Ship	Average capacity	Average length (L_q) [m]	Max draught (D) [m]	Max breadth (B) [m]
Container	6,402 [TEU]	281.6	16.5	59
Oil	91,774 [DWT]	295	22.2	60
Coal	175,549 [DWT]	290	18.6	50
General cargo	14,679 [DWT]	139	13.3	46.8
Ro-Ro	10,041[DWT]	170	7.5	28

Table 2 *All these values are determined by using the 'Design_Ships.xls'

Number of berths

Assumptions

- 1 Ton of oil = 1.165 m^3 (Hofstrand, 2008)
- We store the oil for 1 month. Actually, during that month a part of the oil will be distributed so the storage of 1 month is the maximum space needed. (Ligteringen & Velsink, Ports and Terminals, 2012)
- Angle of repose of coal: 37 DEGS (Ligteringen & Velsink, Ports and Terminals, 2012)

First estimation

An approximation of the number of berths and hence the quay length is made on the basis of an estimated berth productivity. For determining the first estimation of the yearly traffic per berth for a container terminal the following formula is used:

$$V_y = N_c \cdot C_c \cdot N_h \cdot O_f \cdot E_f$$ (Silva P. , Terminal Typology, 2016)

with:

$V_y = yearly\ traffic\ per\ berth\ \left[\frac{TEU}{yr}\right]$ or $\left[\frac{\text{DWT}}{\text{yr}}\right]$
$N_c = number\ of\ cranes$
$C_c = crane\ hourly\ capacity\ \left[\frac{TEU}{hour}\right]\ or\ [\frac{DWT}{hour}]$
$N_h = number\ of\ operational\ hours\ per\ year$
$O_f = occupancy\ factor$
$E_f = efficiency\ factor$

Crane hourly capacity is determined as follows (Silva P. , Terminal Typology, 2016) *:*

Container

- 25 containers/hour * 1.5 = 37.5 TEU/hour

Oil

- Crude oil: 5,000 – 20,000 DWT/hour →Average = 12,500 DWT/hour
- Refined product: 1,000 – 3,000 DWT/hour → Average = 2,000 DWT/hour
- (un)loading-rate average over the ships = 0.629 * 2,000 + 0.371 * 12,500 = 5,896 DWT/hour

Coal/General Cargo/Ro-Ro

- Unloading bulk solid cargo: 500 – 20,000 → Average = 12,500 DWT/hour

Traffic Typology	N_c	C_c	N_h	O_f	E_f	V_y
Container	4	37.5	8400	0.60	0.80	604,800
Oil	1	5,896	8400	0.35	0.80	13,867,392
Coal	2	1,250	8400	0.35	0.80	5,880,000
General Cargo	1	1,250	8400	0.35	0.80	2,940,000
Ro-ro	1	1,250	8400	0.35	0.80	2,940,000

Table 3: First estimation

The number of berths can be calculated with:

$$n = \frac{V}{V_y} \text{ (Silva P. , Terminal Typology, 2016)}$$

with:
$n = number\ of\ berths$
$V = Volume\ \left[\frac{TEU}{yr}\right] or\ [\frac{DWT}{yr}]$

Traffic Typology	V	V_y	n	Number of berths
Container	3,000,000 [TEU/yr]	604,800	4.96	5
Oil	25,000,000 [Tons/yr]	13,867,392	1.80	2
Coal	7,000,000 [Tons/yr]	5,880,000	0.60	1
General Cargo	2,000,000 [Tons/yr]	2,940,000	0.68	1
Ro-ro	1,000,000 [Tons/yr]	2,940,000	0.34	1

Table 4: number of berths, first estimation

Queuing theory

The queuing theory is a model which can predict the queue length and waiting time at a port. The factors determining the behaviour of such a system are:

- The customers arrivals
- The service times of customers
- The service system (queue-discipline, number of berths)

For the new port area the number of berths will be determined assumed that a year consists of 8400 operational hours. The transhipment contains three types; containers, oil, coal and general cargo.

For these types of transhipment the following data have to be calculated (Groenveld, 1993):

Arrival rate: $\lambda = \frac{calls/year}{operational\ hours/year}$

(un)loading time : $\frac{(un)loading/call}{(un)loading\ rate}$

Service rate: $\mu = \frac{1}{service\ time} = \frac{1}{(un)loadingtime + mooring\ time}$

Berth occupancy: $\rho = \frac{\lambda}{\mu}$

Berth utilization: $u = \frac{\lambda}{\mu \cdot n} = \frac{\rho}{n}$

	Container	Oil	Coals	General Cargo	Ro-Ro

Arrival rate (λ)	0.05583	0.03251	0.00476	0.01631	0.01190
(un)loading time	42.68 hours	15.57 hours	141 hours	11.74 hours	8.03 hours
Service rate (μ)	0.02238	0.05693	0.00702	0.07278	0.09970
Berth occupancy (ρ)	2.49	0.57094	0.67828	0.22410	0.11936

Table 5: queuing theory values

Containers

For container terminals we use a queuing-system and the time required for mooring.

Additional data container vessels (Groenveld, 1993)*:*
Queuing-system: E2/E2/n
Time required for mooring: 2 hours
Maximum acceptable waiting time: 10% of the service time

Throughput [TEU]	Calls/year	(un)loading/call [TEU]	(un)loading-rate [TEU/hour]
3,000,000	469	6402	150

Table 6: Containers, queuing theory

Calls/year = 3,000,000 / 6402 = 469
(Un)loading/call (average capacity) = 6,402 TEU
(Un)loading-rate = 37.5 TEU/hour * 4 cranes = 150 TEU/hour

These numbers can be used to determine the average waiting time of ships in the queue E2/E2/n. With Table 5 in the book 'Service Systems in Ports and Inland Waterways' (Table 7) the berth occupancy and the berth utilization can be used to get the average waiting time. If this waiting time is higher than 0.1 (10% of the service time) the assumed number of berths is not ok, so a higher amount of berths is needed. Table 8 shows the iterative process to determine the number of berths. **Four berths are needed for the container terminal.**

	Number of servers (n)									
Utilisation (u)	1	2	3	4	5	6	7	8	9	10
0.1	0.0166	0.0006	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
0.2	0.0604	0.0065	0.0011	0.0002	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
0.3	0.1310	0.0235	0.0062	0.0019	0.0007	0.0002	0.0001	0.0000	0.0000	0.0000
0.4	0.2355	0.0576	0.0205	0.0085	0.0039	0.0019	0.0009	0.0005	0.0003	0.0001
0.5	0.3904	0.1181	0.0512	0.0532	0.0142	0.0082	0.0050	0.0031	0.0020	0.0013
0.6	0.6306	0.2222	0.1103	0.0639	0.0400	0.0265	0.0182	0.0128	0.0093	0.0069
0.7	1.0391	0.4125	0.2275	0.1441	0.0988	0.0712	0.0532	0.0407	0.0319	0.0258
0.8	1.8653	0.8300	0.4600	0.3300	0.2300	0.1900	0.1400	0.1200	0.0900	0.0900
0.9	4.3590	2.0000	1.2000	0.9200	0.6500	0.5700	0.4400	0.4000	0.3200	0.3000

Table 7: Waiting time

Number of berths (n)	Occupancy (ρ)	utilization (u)	Waiting time (W*)	W<0.1
3	2.49	0.83	0.46	No
4	2.49	0.6225	0.0639	Yes

Table 8: number of berths containers, queuing theory

Oil

Additional data oil vessels (Groenveld, 1993)*:*
Queuing-system: M/D/n
Time required for mooring: 2 hours
Maximum acceptable waiting time: 15% of the service time

Throughput [Tons]	Calls/year	(un)loading/call [DWT]	(un)loading-rate [Tons/hour]
25,000,000	273	91,774	5,896*

*The average of the given values in the PowerPoint 'Terminal Typology'

Table 9: Oil, queuing theory

(Un)loading/call (average capacity) = 91,774 DWT
Calls per year = 25,000,000 / 91,774 = 273

For determining the number of berths Table 2 in the book 'Service Systems in Ports and Inland Waterways' is used. **Two berths are needed for the oil terminal.**

Number of berths (n)	Occupancy (ρ)	utilization (u)	Waiting time (W*)	W<0.15
1	0.5709	0.5709	0.6750	No
2	0.5709	0.2855	0.0506	Yes

Table 10: number of berths oil, queuing theory

Coal

Additional data coal vessels (Groenveld, 1993)*:*
Queuing-system: M/E2/n
Time required for mooring: 2 hours
Maximum acceptable waiting time: 20% of the service time

Throughput [Tons]	Calls/year	(un)loading/call [DWT]	(un)loading-rate [DWT/hour]
7,000,000	40	175,549	1,250*

*The average of the given values in the PowerPoint 'Terminal Typology'

Table 11: Coal, queuing theory

(Un)loading/call (average capacity) = 175,549 DWT
Calls per year = 7,000,000 / 175,549 = 40

Two berths are needed for the coal terminal, see Table 12.

Number of berths (n)	Occupancy (ρ)	utilization (u)	Waiting time (W*)	W<0.2
1	0.67828	0.67828	1.58932	No
2	0.67828	0.33914	0.10348	Yes

Table 12: number of berths coal, queuing theory

General Cargo

Additional data general cargo vessels (Groenveld, 1993)*:*
Queuing-system: M/E2/n
Time required for mooring: 2 hours
Maximum acceptable waiting time: 20% of the service time

Throughput [Tons]	Calls/year	(un)loading/call [DWT]	(un)loading-rate [DWT/hour]
2,000,000	137	14,679	1,250*

*The average of the given values in the PowerPoint 'Terminal Typology'

Table 13: General cargo, queuing theory

(Un)loading/call (average capacity) = 14,679 DWT
Calls per year = 2,000,000 / 14,679 = 137

Two berths are needed for the general cargo terminal, see Table 14.

Number of berths (n)	Occupancy (ρ)	utilization (u)	Waiting time (W*)	W<0.2
1	0.22410	0.22410	0.2169	No
2	0.22410	0.11205	0.0122	Yes

Table 14: number of berths general cargo, queuing theory

Ro-Ro

Additional data general Ro-Ro vessels (Groenveld, 1993)*:*
Queuing-system: M/E2/n
Time required for mooring: 2 hours
Maximum acceptable waiting time: 20% of the service time

Throughput [Tons]	Calls/year	(un)loading/call [DWT]	(un)loading-rate [DWT/hour]
1,000,000	100	10,041	1,250*

*The average of the given values in the PowerPoint 'Terminal Typology'

Table 15: ro-ro, queuing theory

(Un)loading/call (average capacity) = 10,041 DWT
Calls per year = 1,000,000 / 10,041 = 100

Only one berth is needed for the Ro-Ro terminal, see Table 16.

Number of berths (n)	Occupancy (ρ)	utilization (u)	Waiting time (W*)	W<0.2
1	0.11936	0.11936	0.0919	Yes

Table 16: number of berths ro-ro, queuing theory

*In the following design steps the amount of berths according to the queuing theory will be used.

Quay length

For multiple berths along a straight continuous quay front the quay length is based on the average vessel length, as follows:

$$L_q = \begin{cases} L_{s,max} + 2 \cdot 15 & for\ n = 1 \\ 1.1 \cdot n \cdot (L_s + 15) + 15 & for\ n > 1 \end{cases}$$ (Ligteringen & Velsink, Ports and Terminals, 2012)

with:
$L_q = quay\ length$
$L_s = average\ ship\ length\ (Excel\ file)$
$n = number\ of\ berths$ (queueing theory)

Traffic Typology	Number of berths (n)	Length ship (L_s) [m]	Quay length (L_q) [m]
Container	4	281.6	1,320
Oil	2	295	697
Coal	2	290	686
General Cargo	2	139	353.8
Ro-ro	1	170	218.5

Table 17: quay length

Container:
$L_{s,mother} = 372\ m\ (Excel\ file)$
$L_{s,feeder} = 250\ m\ (Excel\ file)$

$$L_q = 0.259 * 372 + 0.741 * 250 = 281.6\ m$$

Storage Area

Different terminals have different requirements for the storage areas. The following section describes the detailed calculation of the storage area.

Container Terminal

The storage area of container terminals is calculated according to the following equation (Silva P. , Terminal Typology, 2016):

$$S_{st} = \frac{N_{TEU} \times T_{st} \times S_{TEU}}{F_u \times 365 \times Focc}$$

In which:

N_{TEU}	=	yearly traffic in TEU
T_{st}	=	average time of one container in the park
S_{TEU}	=	surface occupied from one TEU, depending on the operational system
F_u	=	utilisation factor of the available height (= 1 for storage at one level, not more than 0.78 for storage at various levels)
F_{occ}	=	occupation factor of the terminal

N_{TEU} is known in provided information, which is 3 million TEU. T_{st} is assumed to be 8 days. RMG system is used at port and the container can be put at 5 levels. So, the S_{TEU} is taken as 8. Because containers are stored at different levels, F_u is taken as 0.78. The occupation factor of the terminal F_{occ} is assumed as 0.8. Therefore, the storage area of containers is calculated to be 84.3 ha.

Parameters	**Value**
N_{TEU}	3000000 TEU
T_{ST}	8 days
S_{TEU}	8
F_u	0.78
$Focc$	0.8
S_{st}	84.3 ha

Table 18: Storage area of the container terminal Figure

Oil Terminal

Enough places are needed for tank farm on oil terminals. In our project, there is 25 million tons oil per year.

Oil tanks are used for the storage of oil. First, the tons of oil are transferred to volume. The average density of 1.165 m^3/ ton is assumed. Furthermore, it is assumed that there are 20 days for oil to be stored at port. According to 'Ports and terminals', a tank of 100000 m^3 surrounded by a 5 m high bound (4 m useful) requires a surface of 25000 m^2. Such tanks are used in our case. Therefore, number of tanks and area of tanks can be filled in the following table.

Parameters	**Value**
Capacity	25000000 tons
Volume	29125000 m^3
Days	20 days

Number of tanks	17
Storage area	42.5 ha

Table 19: Storage area of storage area

Coal Terminal

Coal is transported at dry bulk terminal. The total capacity of coal is transferred to volume at first. The average density is assumed to be 1.3 m3/ton. 10 days are assumed for the storage of coal. The utilization rate is assumed to be 0.8. The angle of repose varies from 30 to 45 degrees. If the height of stockpiles is 15 m, the width of stockpile is taken as 40 meters which meets the requirement of repose angle. The estimation of volume of total stockpile can be made with the following equation. (Ligteringen & Velsink, 2012)

$$V = b \times \frac{1}{2} \times h \times l \times m_b$$

In which:

V	=	maximum volume of cargo in storage
b	=	width of stockpile
h	=	height of stockpile
l	=	total length of stockpile
m_b	=	utilization rate

The total volume of coal is known, so the length of total stockpiles can be calculated. 5 stockpiles are used, and the length of each stockpile is about 150 m. Results are shown as follows.

Parameters	**Value**
Capacity	7000000 tons
V	5384615 m^3
Days	10 days
b	40 m
h	15 m
m_b	0.8
Number of stockpiles	5
L(total)	641 m
L(single)	150 m

Storage area	3 ha

Table 20: Storage area of coal terminal

General Cargo Terminal

The required area for storage has to be determined from the annual throughput and the average transit time of the goods as main parameters. For a transit shed, the required floor area A_{gr}can be calculated as (Ligteringen & Velsink, 2012):

$$A_{gr} = \frac{f_{AREA} \cdot f_{BULK} \cdot N_C \cdot \bar{t}_d}{m_C \cdot h_s \cdot \rho_{CARGO} \cdot 365}$$

In which:

N_C	=	total annual throughput which passes the transit shed
$\bar{t}_d$	=	average dwell time of the cargo in days
ρ_{CARGO}	=	average relative density of the cargo as stowed in the ship
h_s	=	average stacking height in the storage
f_{AREA}	=	ratio gross over net surface, accounting for traffic lanes for FLTs etc
f_{BULK}	=	bulking factor due to the stripping and separately stacking of special consignments, damaged goods, etc.
m_C	=	average rate of occupancy of the transit shed or storage

The table below shows the values used for the calculation.

Table 21: Storage area of general cargo terminal

N_C	**2000000**	**t/year**
$\bar{t}_d$	10	days
ρ_{CARGO}	0.6	t/m^3
h_s	2	m
f_{AREA}	1.5	-
f_{BULK}	1.2	-
m_C	0.7	-
A_{gr}	11.7	ha

Five warehouses of 100 m * 250 m are needed.

Ro-Ro Terminal

The parking area at a Ro-Ro terminal is a function of the number of vehicle movements per year, the average transit time in days and the area requirement per vehicle.

Assumption:

- 30 tons per trailer
- Transit time = 2 days
- Area requirement = 40 m^2 per trailer unit
- Capacity of the terminal = 1,000,000 tons/year

Number of trailer/year = 1,000,000 / 30 = 34,000
Number of trailer/month = 34,000 / 12 = 2850
Area required = 2,850 * 40 = 11.3 ha

Summary

The design of the ship dimensions are obtained by using the Excel file, which gives the following data:

Ship	Average capacity	Average length (L_q) [m]	Max draught (D) [m]	Max breadth (B) [m]
Container	6,402 [TEU]	281.6	16.5	59
Oil	91,774 [DWT]	295	22.2	60
Coal	175,549 [DWT]	290	18.6	50
General cargo	14,679 [DWT]	139	13.3	46.8
Ro-Ro	10,041[DWT]	170	7.5	28

Table 22: Ship design

For the number of berths two approaches are uses; one simple estimation and a more complicated one (queuing theory). In the rest of the report the number of berths, estimated by the queuing theory will be used.

Traffic Typology	Number of berths (1st estimate)	Number of berths (Queuing theory)
Container	5	4
Oil	2	2
Coal	1	2
General Cargo	1	2
Ro-Ro	1	1

Table 23: Number of berths

The different quay lengths are obtained by using the average lengths of the design ships (see Table 22).

Traffic Typology	Quay length (L_q) [m]
Container	1,320
Oil	697
Coal	686
General Cargo	353.8
Ro-ro	218.5

Table 24: Quay length

A total area of 152.6 hectares is needed for the final port lay-out, see Table 25.

Traffic Typology	Value	Unit
Container	84.3	[ha]
Oil	42.5	[ha]
Coal	3	[ha]
GENERAL CARGO	11.7	[ha]
RO-RO	11.3	[ha]
TOTAL AREA	152.6	[ha]

Table 25: Storage area

3. Site Conditions

Introduction

In the previous chapter the maritime climate will be discussed. To determine the extreme values the peak over threshold (POT) method is used. For this project, propagation of waves is done by Finite Element Method, using the software TOMAWAC. This allows for a 2D analysis of wave propagation. The geological data has been provided for the area between Pointe Betoya and Pointe Negri. This data will be analysed in this chapter. The best solution will be given to reduce the amount of accumulated sediment in front of the port.

Maritime climate

One of the main design bases is the marine climate model, which defines the behavior of the waves and wind in the area surrounding the port, and ultimately how they affect the structures and the ship operation.

The analysis is based on information collected at an offshore point northwest of Nador West Med. This point has the following main characteristics:

Northing (UTM)	520,512 meters
Easting (UTM)	702,369 meters
Depth	60 meters
Time range	1992 - 2009
Measurement freq.	3 hours
Wave data type	H_s,T_m,T_p,D_m,D_p
Wind data type	V_v,D_v

Table 26: Data source characteristics

This point records every 3 hours accurate data of the current climate and performs a short term statistical analysis, storing the climate conditions as significant values.

Climate is assessed by performing an extremal, long term analysis of the given data. Extremal wave height and wind speed are considered stochastic variables accurately described by theoretical statistical distributions. Data is fitted to these distributions to determine the particular solutions that define the variables.

The objective of this section is to determine the wave climate associated to the required conditions. These conditions are specified as return periods T_R of 100 years and 1 year for extremal calculations, and 1% exceeded wave height $H_{1\%}$ for operational requirements.

This analysis allows to determine the significant wave height Hs, peak period T_p and mean period T_m for the different design conditions.

Direction Analysis

Overall marine climate is a combination of different processes, like wind generated waves, tides or swell that have different properties (direction, period, wave height, etc.), origins and effects on the coast. An extremal analysis requires a prior decoupling of the marine climate into its different elements. For this particular case, a direction-based analysis is done.

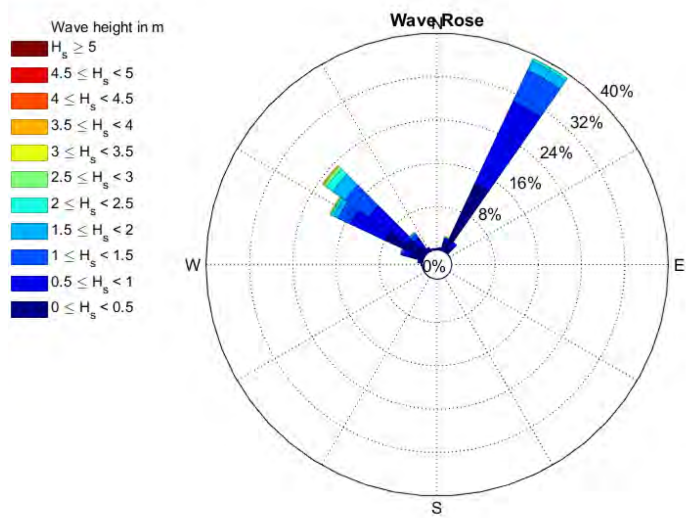


Figure 2. Wave rose

As it can be seen, Nador West climate is clearly produced by two different wave processes. One of the processes has an NW origin (direction 315-330º) from the port, while the other comes from NNE (direction 20-30°).

This information is used to extract the wave data corresponding to these directions. This data is analyzed separately to obtain its extremal regime and design values.

As for the wind data, following the same procedure the following wind rose is obtained:

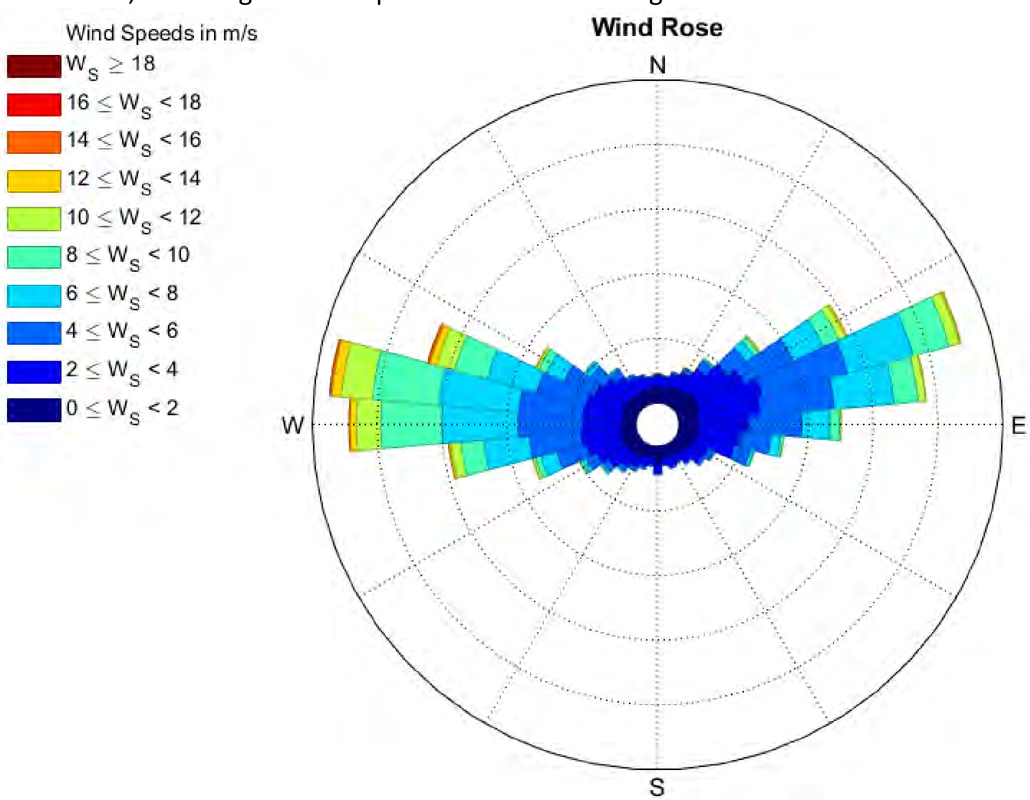


Figure 3. Wind rose

Main wind comes from West and East directions, being the West direction the only able to produce waves. Comparing with the wave rose, it is clear that there is no correlation between wind and waves direction, which suggests that wave processes have a strong swell component.

Extremal data methodology

Several data sets can be used to determine the extremal climate. The most common ones are:

- Complete set: The whole data is taken into consideration.
- Maximum annual value: Only the maximum value of each year is considered.
- Peak Over Threshold (POT): Only peaks (i.e. local maximum) of events that surpass an established threshold are considered.

This project makes use of POT method for extremal calculations. Contrary to the complete set, it avoids the lower and most frequent values of the data set. This allows for a better fitting of the higher, extreme values. Additionally, POT method evaluates the highest events of the whole set, as opposed to the maximum annual value, which does not account for different storm concentration between years.

Hs
Year 2005 Year 2006
time
Peak Over Threshold

Hs
Year 2005 Year 2006
time
Annual Maximum

Legend
Wave set
Extremal events

Figure 4. POT-Annual Maximum comparison

Threshold is chosen in order to get a reasonable amount of data to work with. As the POT is used for a posterior fitting, a sensitivity study is done to observe how the change in the threshold affects the fitting error and the significant values. The chosen threshold is the one that gives a stable significant value with minimum fitting error.

As for the operational waves, i.e. $H_{1\%}$, POT method is not applied. Distribution is therefore fitted to the complete data set. This is necessary as this wave height aims to describe the limitations of the operational situation, not extreme events.

Extremal data is fitted to a 3-parameter Weibull distribution

$$F(x) = 1 - \exp\left(-\left(\frac{\boldsymbol{x} - B}{A}\right)^k\right)$$

Realizations of the variable *x* are obtained from the extremal data, while the non-exceedance probability *F(x)* for each realization is determined as:

$$F_i = 1 - \frac{i}{n+1}$$

For each extremal value *i* of the population *n*. Fitting is done by obtaining the parameters *A, B* and *k* that better adjust to the relation *F(x) - x*.

As this distribution has 3 degrees of freedom, it cannot be linearized, it requires an iterative process. For this analysis, Maximum Likelihood Method is chosen. Parameters *k* and *A* are found by Newton-Raphson iteration, while *B* is optimized by maximizing a Log-likelihood function.

Finally, design conditions are related to the probability of occurrence by adjusting sample intensity λ:

$$\lambda = \frac{n^{\underline{o}}\ of\ extreme\ data}{n^{\underline{o}}\ of\ years\ of\ observation}$$

$$T_R = \frac{1}{\lambda\big(1 - F(x)\big)}$$

As for the operational design condition, it is enough to define:

$$F(x) = F(X < x) = F(X > x) - 1 = 0.99$$

Wave period

Wave periods are obtained by assuming a direct correlation Period-Height. This is a crude estimation, but useful as a first approach in simple wave climate. The correlation follows a power distribution as:

$$T_{p,m} = a_{p,m} \cdot {H_s}^{b_{p,m}}$$

Sensitivity analysis and threshold selection

Threshold criteria for POT analysis is done based on a sensitivity analysis. By choosing a threshold and following the whole calculation process a significant wave height can be obtained. Besides, the data given by POT analysis has a fitting goodness given by the fitting error. The sensitivity analysis studies how the threshold variation affects the wave height and the fitting error. This helps to choose a threshold that gives:

- **Stable results**, i.e. wave height does not suffer significant variation with a slight threshold change.

- **Best fitting**, i.e. data that offers a minimum fitting error.

For sector NW, it can be seen that the data set is mostly stable in the threshold range of 1.5-3.2 meters. The minimum fitting error at the stable is in 2.5 meters.

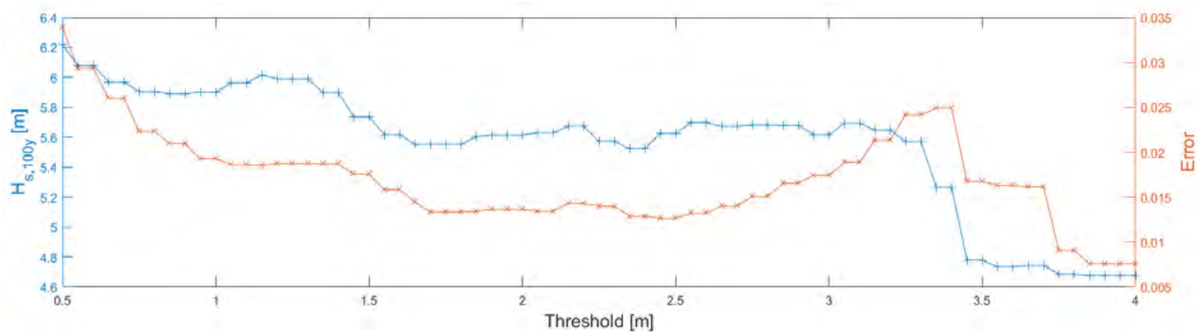


Figure 5. Sensitivity analysis for sector NW

As for sector NNE, is mainly stable in the threshold range from 1 to 3 meters. Minimum fitting error occurs with a threshold of 1.7 meters.

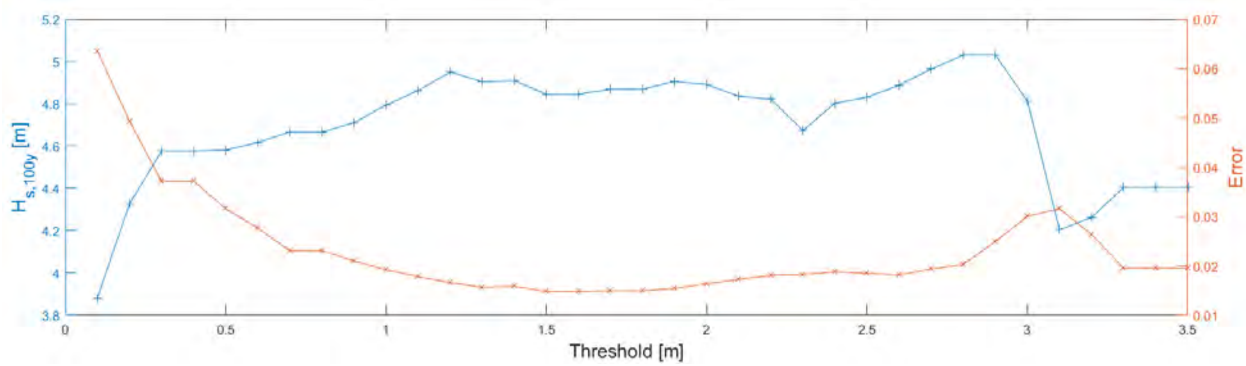


Figure 6. Sensitivity analysis for sector NNE

To summarize, the values to be used as threshold in the POT analysis are:

Sector	Threshold	Nº of peaks
NW	2.5 meters	130
NNE	1.7 meters	136

Table 27. Threshold values

Results

Wave height

For extremal design values, applying the POT analysis with the given threshold and fitting the data through a 3-parameter Weibull function gives the following parameters:

Sector	A	B	k	λ
NW	0.5226	2.4827	1.0216	5.2
NNE	0.4315	1.6804	0.9154	5.44

Table 28. Weibull parameters for extreme analysis

For operational values, fitting the whole data set to a 3-parameter Weibull function gives the following parameters:

Sector	A	B	k
NW	0.7659	1.1964	0.0995
NNE	0.6156	1.3012	0.0956

Table 29. Weibull parameters for operational analysis

Finally, wave data is obtained as:

	Design conditions	H_s
Sector NW 315-330º	T_R 100 years	5.6 m
	T_R 1 year	3.33 m
	1% exceeded	2.84 m
Sector NNE 20-30º	T_R 100 years	4.87 m
	T_R 1 year	2.44 m
	1% exceeded	2.1 m

Table 30. Design wave height

Wave period

After fitting the whole data set by Least Square method, the parameters that relate wave height and period are as follows:

Sector	Peak period		Mean period	
	a	b	a	b
NW	4.8421	0.4120	3.9183	0.3930
NNE	6.0822	0.4813	4.3701	0.4221

Table 31. Parameters for H_s-T_p and H_s-T_m relation

As it can be seen, the fiting procures an acceptable relation between the variables:

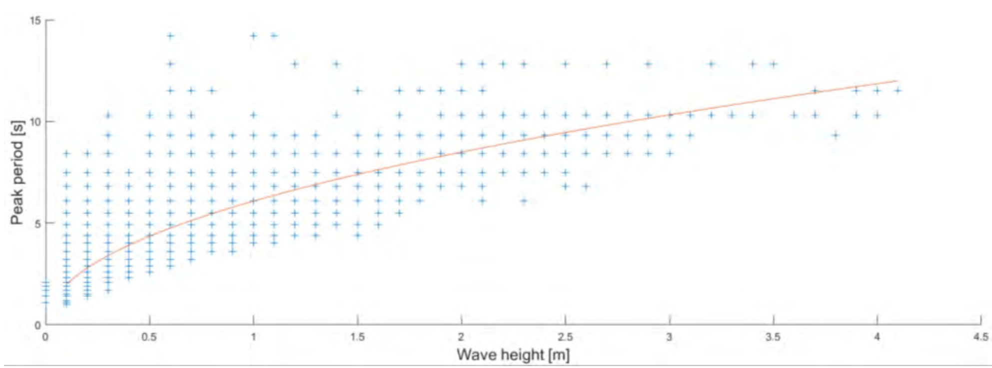


Figure 7. H_s - T_p relation

Being the resultant periods:

	Design conditions	T_p	T_m
Sector NW 315-330º	T_R 100 years	10 s	7.7 s
	T_R 1 year	8 s	6.3 s
	1% exceeded	7.4 s	5.9 s
Sector NNE 20-30º	T_R 100 years	13 s	8.5 s
	T_R 1 year	9.34 s	6.4 s
	1% exceeded	8.66 m	6 s

Table 32. Design values for peak and mean period

Water level

Reference for water level calculation is taken at Chart Datum (CD). Astronomic water level at the site varies at follows:

Water Level	Value
Highest High Water Spring	0.66 m
Mean High Water Spring	0.57 m
Mean High Water Neap	0.47 m
Mean Sea Level	0.35 m
Mean Low Water Neap	0.23 m
Mean Low Water Spring	0.13 m
Lowest Astronomical Tide	0.00 m

Table 33. Astronomic water levels

Wave propagation

Once the offshore marine climate is defined, it is necessary to obtain the values in the area surrounding the port. This process is obtained by propagating the wave properties until the objective points.

For this project, propagation is done by Finite Element Method, using the software TOMAWAC. This allows for a 2D analysis of wave propagation.

The main results obtained are the spectral significant wave height, the average direction of origin, and the mean and peak frequencies which allow propagating the wave from the offshore to the sites of interest. These values are associated to the design conditions specified before.

Results from this analysis are as follows:

	Direction	T_R	Depth	H_s	T_m
Sector 1 315°	317°	100 years	20 m	5 m	10.1 s
	317°	1 year	20 m	2.9 m	8.15 s
	316°	1%	20 m	2.5 m	7.6 s
Sector 2 20°	10°	100 years	20 m	3.45 m	14.3 s
	2°	1 year	20 m	1.65 m	9.34 s
	5°	1%	20 m	1.45 m	8.66 s

Table 34. Design values after propagation

Detailed explanation of the process followed to obtain these results can be obtained at the Appendix A, as well as graphic expamples of the propagation for the different design conditions.

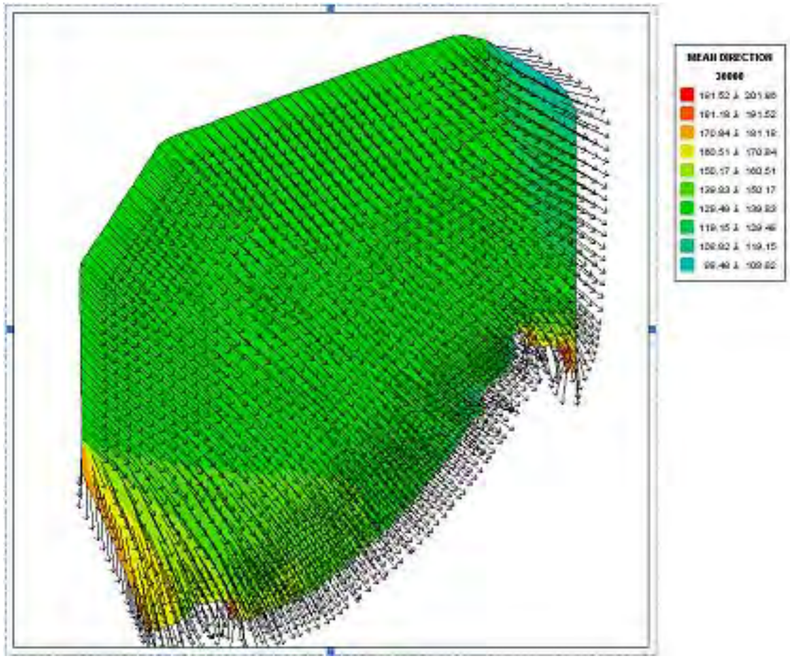


Figure 8. Variation of wave direction for Sector NW and T_R =100 years

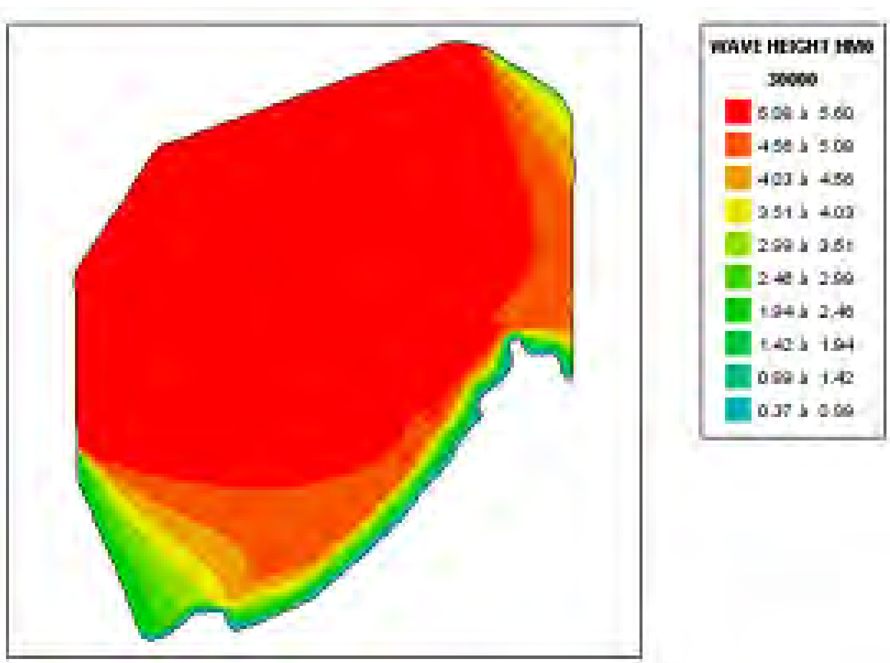


Figure 9. Variation of wave height for Sector NW and T_R =100 years

Geotechnical Data

The area subject to this preliminary geotechnical analysis is located on the Mediterranean shore of Morocco, near the city of Nador. The geological data has been provided for area between Pointe Betoya and Pointe Negri. The client provided:

- Map showing clay stratum thickness for approximately 10km length of the shore
- Map showing several boreholes studies along the coast and land
- The selected sections obtained from boreholes data

Clay deposits

See Figure 10 below for the thickness of clay strata across most of the given study area. The area to the left shown in red has deep deposits of clay, while area in blue has sand as a topmost layer. The new port would ideally be located away from the clay zone for the ease of design, construction and maintenance. However, the topography study suggests that the areas closer to Pointe Negri (right side of the map below) have much steeper slope, hence increasing the cost of the breakwater.

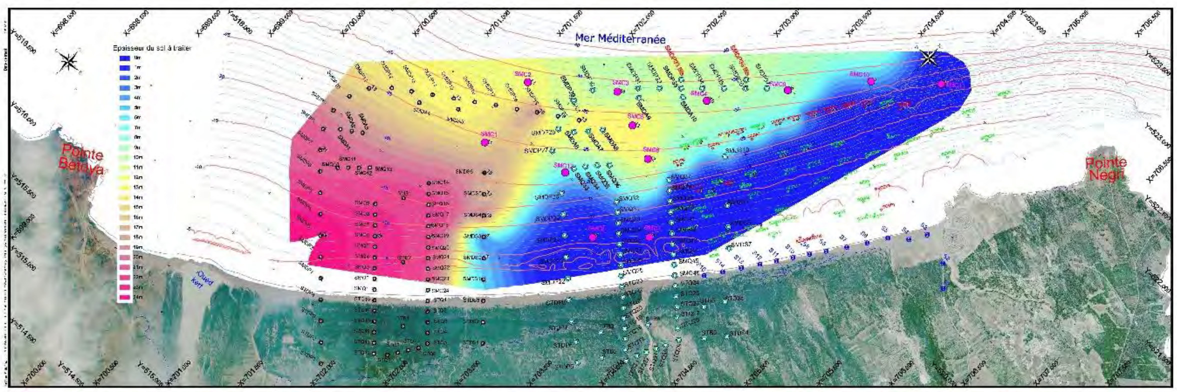


Figure 10: Thickness of clay strata

Taking into account data described above, the preliminary location is chosen to be entirely in the area of sandy deposit, but away from Pointe Negri. See Figure 11 for an example of such location.

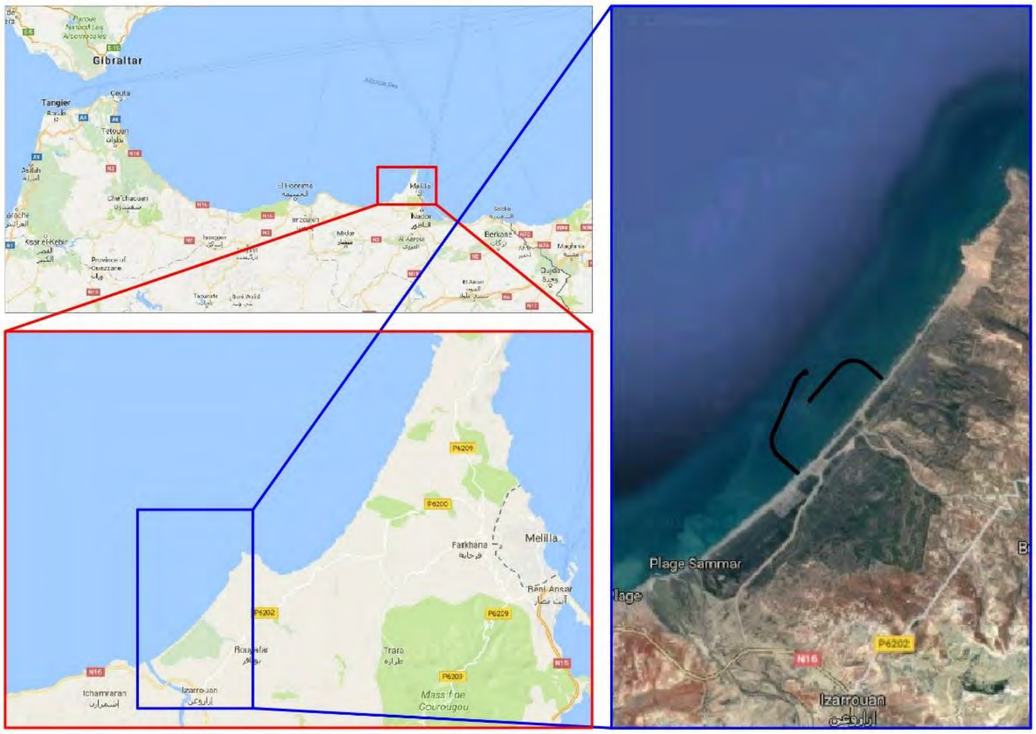


Figure 11: Preliminary location of the new port (see right side, outline in black)

Boreholes

See Figure 12 for locations of boreholes and sections. Profiles parallel to the coastline are indicated with letters A and B, while profiles running perpendicular to the coastline are indicated with numbers 1-3.

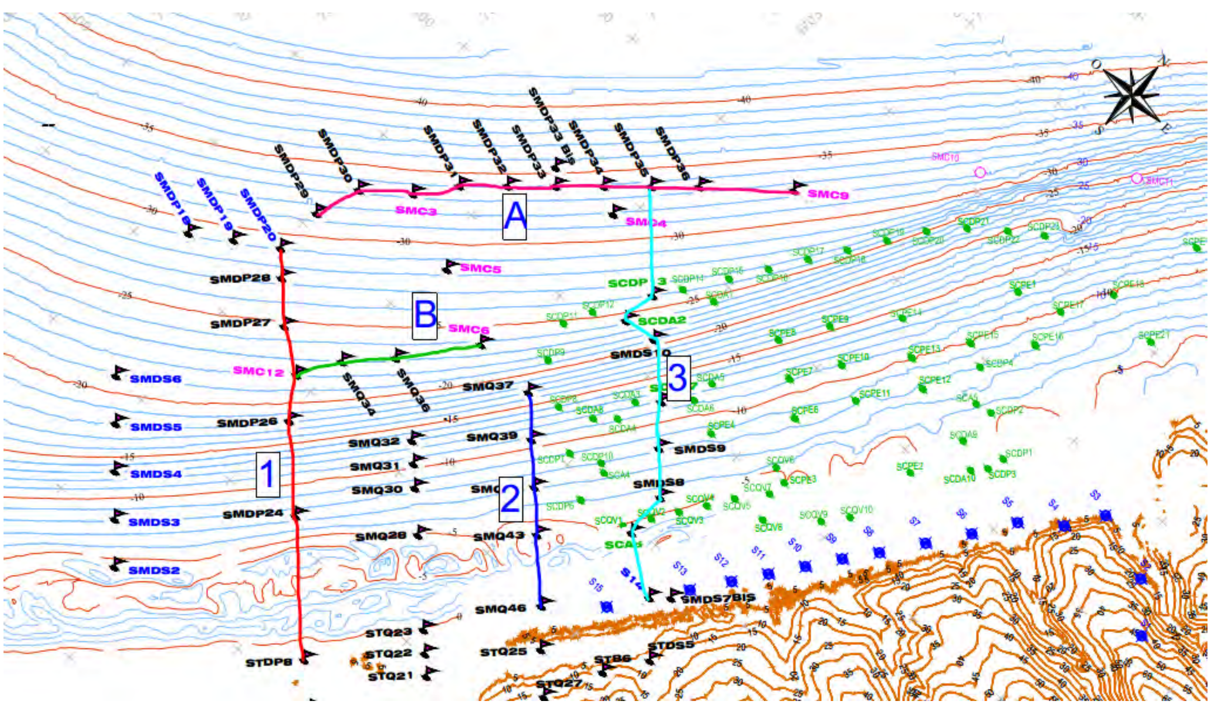


Figure 12: Boreholes and profiles' markers

Section 1

There is fine brown sand stratum with maximum depth of approximately 20m and offshore length of approximately 1000m (Figure 13). Below the above mentioned sand layer there is a layer of sandy clay with depth varying between 0 and 3 metres. 1000m offshore the clay stratum thickens and is the topmost layer. Going further offshore the sediments become finer, with the topmost layer being clay.

Below the above mentioned sediments there are a continuous strata of volcanic tuff (10m thick) and grey mudstone (8m thick).

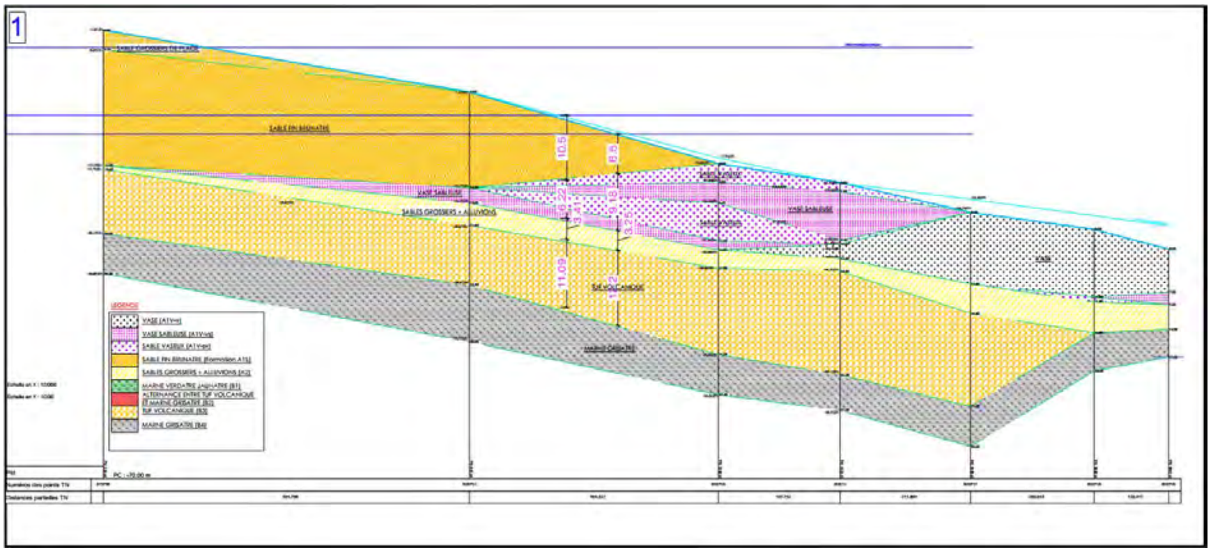


Figure 13: Section 1

Section 2

For the first 350m the topmost stratum is the fine brown sand of approximate thickness of 26m. It is underlain by a 8m thick grey mudstone layer.

Further offshore there is an additional 30m deep volcanic tuff layer introduced between above mentioned layers (350m - onwards).

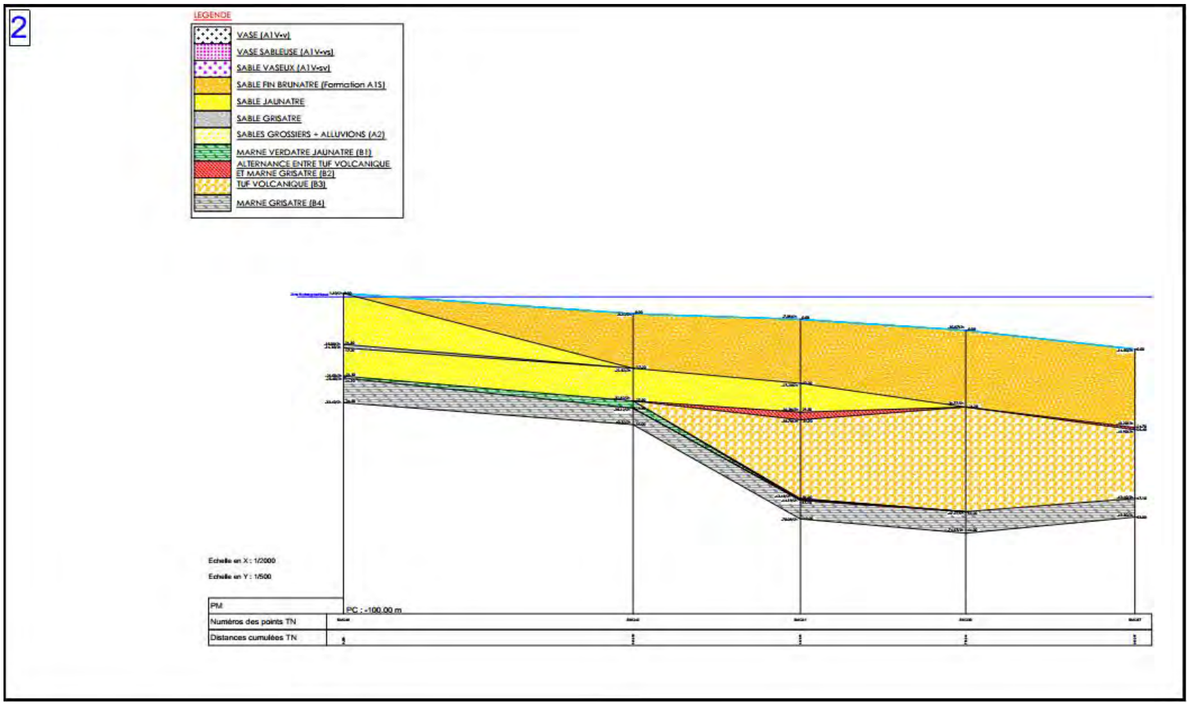


Figure 14: Section 2

Section 3

For 0-900m, the topmost layer is fine brown sand approximately 15-20m underlaid by coarse sands with alluvia of a maximum depth of 15m. The sand layers are underlaid by a mixture of volcanic tuff, green and grey mudstone.

900m onwards the topmost layer becomes sandy clay (10m), underlaid by coarse sand (8m), volcanic tuff (20m) and grey mudstone (10m).

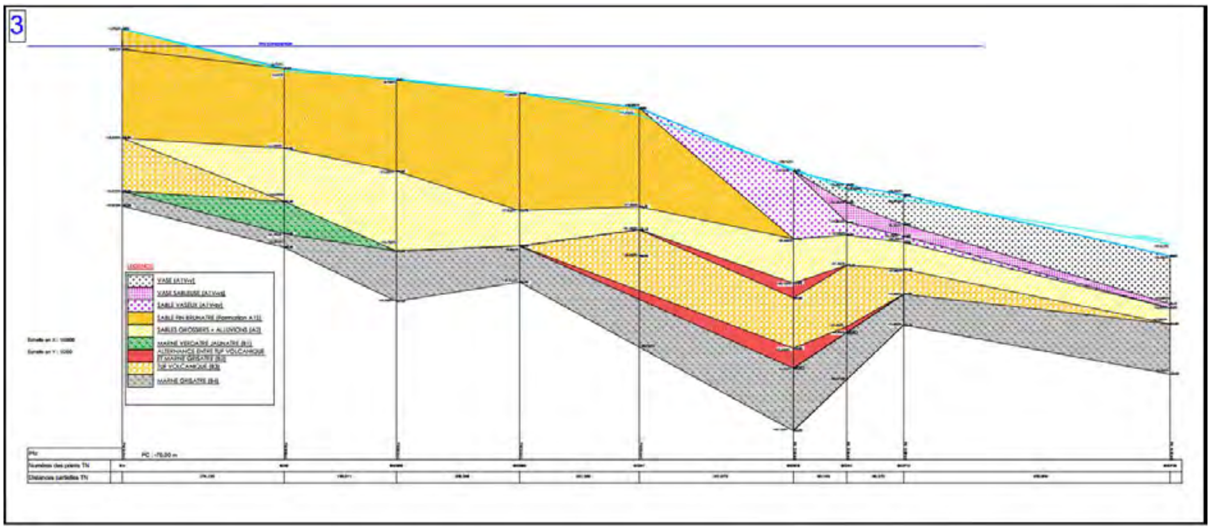


Figure 15: Section 3

Section A

In the 2nd section there is a clay stratum throughout with approximate thickness of 10m. It is underlaid by grey mudstone of approximate thickness of 10m.

There are two zones with volcanic tuff trench-like intrusions of maximum depth of 8m (at SMDP31 and SMDP36). The sea bed has a constant depth of approximately 33m.

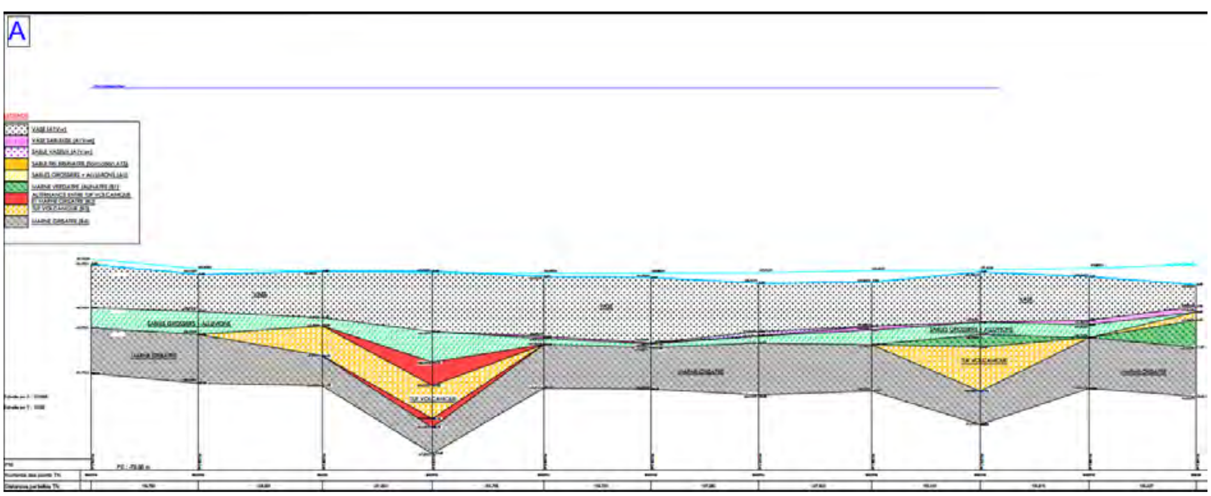


Figure 16: Section A

Section B

In the 3rd section there is a clay stratum throughout with approximate thickness between 7-11m. It is underlaid by sands of varying thickness between 5-35m and grey mudstone of approximate thickness of 6m.

The sea bed has a constant depth of approximately 24m.

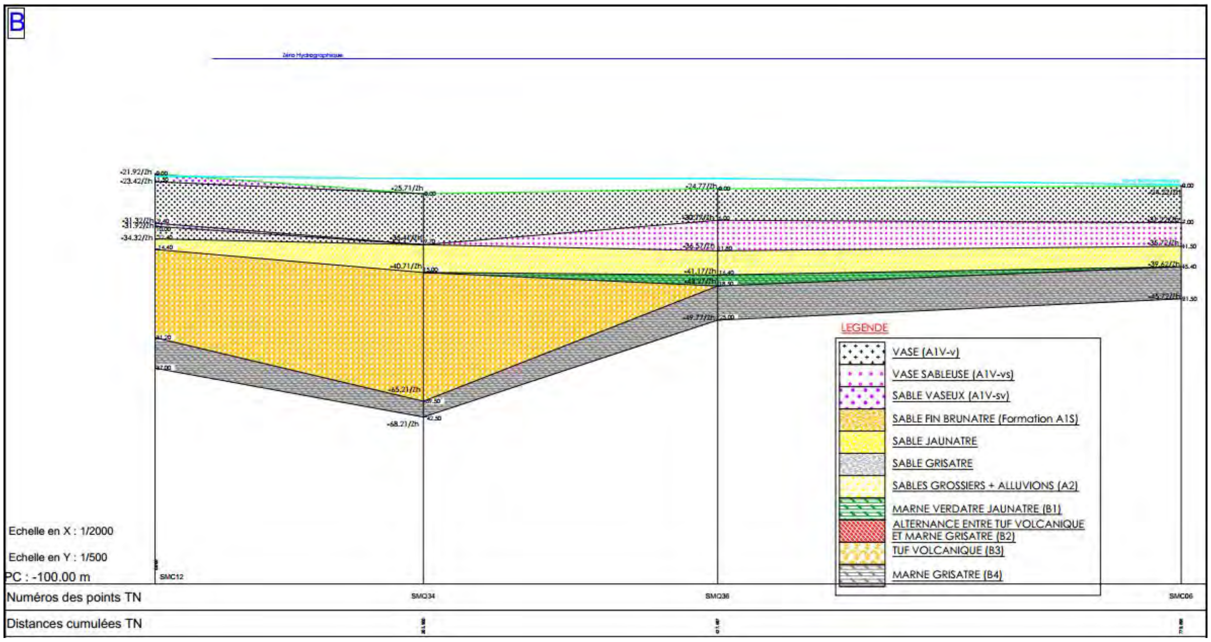


Figure 17: Section B

Summary

The extreme wave analysis is done by using the peak over threshold method, with the threshold numbers of Table 27.

For the two different wave propagations, NW and NNE, the significant wave heights are found in Table 30. The significant wave heights are given for the following design conditions; T_R 100 years, T_R 1 year and 1% exceeded. For the same design conditions the peak period T_p and the mean period T_m are given in Table 32.

For this project, propagation is done by Finite Element Method, using the software TOMAWAC. This allows for a 2D analysis of wave propagation. The design values after wave propagation can be seen in Table 34.

The geological data has been provided for area between Pointe Betoya and Pointe Negri. For sections perpendicular to the shore (1, 2 and 3), there is a stratum of sand for the first 900m offshore. The sediments are then becoming finer, going through sandy clays to clays deeper offshore. Volcanic tuff and grey mudstone are underlying the above mentioned state in almost all locations on those sections.

In sections parallel to the shore (A and B), the topmost statum is a clay sediment with constant depth. It is underlain by finer clay sediments, volcanic tuff and grey mudstone. The depth of the sea bed is constant throughout (24m and 34m respectively).

The area close to the shore (0-500m offshore) is suitable for construction of marine structures without extensive soil reinforcement necessary. Dredging of the channels is expected to be relatively cheap due to soft sediments up to 40m depth in all locations. The sediments are distributed in a very predictable pattern with no major variations between profiles. It is therefore expected that the geotechnical conditions can be extrapolated from above mentioned profiles at any point enclosed by perimeter of boreholes.

4. Layout discussion

Introduction

The purpose of this chapter is to present and compare 3 layouts that have been prepared by the team following the iterative process of complying with several design criteria:

- Design drivers (namely minimising cost of breakwater)
- Bathymetry
- Geotechnical conditions
- Sediment conditions
- Wave and wind conditions
- Risk
- Ease of future expansion
- Ease of navigation
- Construction and operation feasibility

Multi-criteria analysis

In order to decide upon the final design, a multi-criteria analysis was performed. For the purpose of this preliminary design a linear additive model was chosen. (Department for Communities and Local Government: London, 2009)

“In a Multicriteria analysis, the “preferable” solution is the one with the highest measured effectiveness, relative to the set of goals or assessment criteria. In the public works decisions makers must indentify assessable and quantifiable objectives representing the impacts due to different alternatives, from the planning, transportation, spatial, economic-financial and environmental points of view.” (Cappelli & Libardo, 2008)

The criteria are listed and quantified for each design proposal. Once every criterion is measured in all proposed layouts, those are multiplied by their respective weighting and added together to produce total score. (Libardo & Parolin, 2012)

For this design the most important factors are considered to be:

- Breakwater cost – 1.0 factor
- Navigation – 0.7 factor
- Safety – 1.0 factor
- Future expansion – 0.6 factor

The score in each category is given on the scale 1-5, with 1 meaning worst conditions and 5 meaning most desirable conditions.

Scores are then added and the layouts can be compared numerically. Annotated layouts 1, 2 and 3 are presented in Appendix C of this report. See below Table 35, Table 36 and Table 37 for full MCA for all 3 layouts.

Criteria	Subcriteria	Weighting	Score	Total score	Explanation
Costs	Dredging	0.2	3	0.6	Large volume of dredging – in some areas as much as 25m (turning circle) from initial water depth. However the basin does not cut inland.
	Breakwater	1.0	4	4	Relatively short and shallow (max depth 22m, average depth much less), since the port is pushed in-land
	Reclamation	0.2	3	0.6	Relatively small area of reclamation required only for container berths and part of container storage.
	Quay length	0.4	3	1.2	Length of quay is comparable with other layout proposals
Sedimentation		0.5	3	1.5	Curved primary breakwater ensures no excessive accumulation of sediment at the outer face of breakwater. Relatively short primary breakwater may potentially cause problems with quick accumulation of sediment at the entrance.
Navigation	Inside port	0.35	4	1.4	Relatively easy navigation inside the port since all berths within terminals are collinear. Every terminal is easily accessible from the turning circle. The approach channel within the port is not excessively long.
	At entrance/outside	0.35	4	1.4	Vessels entering the port are approaching at the smallest angle (compared with other layouts), to the dominant wave direction.
Safety		1.0	4	4	There are no structures (except for primary breakwater) in the extension of approach channel axis, therefore there is no risk of crashing into terminal in case of loss of control over a vessel.
Expansion		0.6	5	3	There could be future expansion in both directions (no storage obstructions, such as liquid or bulk). If expansion would go to the left, the primary breakwater may be used as secondary breakwater for the new basin. If expansion would go to the right, container terminal may be using the

					same storage and create one larger terminal
Construction and operation feasibility		0.3	3	0.9	All of the dredging is done in soft soil. There is no reclamation at large depth. Accurate construction of curved breakwater may be potentially problematic and skilled contractor should be hired.
Total				**18.6**	

Table 35: MCA for Layout 1

Criteria	Subcriteria	Weighting	Score	Total score	Explanation
Costs	Dredging	0.2	2	0.4	Very large volume of dredging – in some areas as much as 25m (turning circle and inner approach channel) from initial water depth. However the basin does not cut inland.
	Breakwater	1.0	4	4	Relatively long but very shallow (max depth 17m, average depth less), since the port is pushed in-land.
	Reclamation	0.2	2	0.4	Significant area of reclamation required for container berths and large part of container storage.
	Quay length	0.4	3	1.2	Length of quay is comparable with other layout proposals
Sedimentation		0.5	2	1	Straight primary breakwater will cause large volume of sediment accumulating at the outer face of breakwater. Long primary breakwater extending beyond secondary breakwater ensures that there is limited sediment accumulation in the entrance of the port
Navigation	Inside port	0.35	3	1.05	Relatively easy navigation inside the port since all berths within terminals are collinear. Every terminal is easily accessible from the turning circle. However, the inner approach channel is very long and berthing time is therefore longer than in other designs.
	At entrance/out side	0.35	2	0.7	Vessels entering the port are approaching at the steepest angle (compared with other layouts), to the dominant wave direction.

Safety		1.0	2	2	There is a bulk terminal in the extension of approach channel axis, therefore this design presents a risk of crashing into the terminal in case of loss of control over a vessel.
Expansion		0.6	3	1.8	The expansion in both direction is limited. If expansion would go to the left, the primary breakwater may be used as secondary breakwater for the new basin, but the bulk storage must be moved and conveyor belts re-erected on the bridge. If expansion would go to the right, the new container terminal may be using the same storage, but the berths are unlikely to be collinear.
Construction and operation feasibility		0.3	5	1.5	All of the dredging is done in soft soil. There is no reclamation at large depth. Since the breakwater is using only straight segments, the ease of construction is greatest compared with other designs.
Total				**14.1**	

Table 36: MCA for Layout 2

Criteria	Subcriteria	Weighting	Score	Total score	Explanation
Costs	Dredging	0.2	1	0.2	Extremely large volume of dredging – in some areas as much as 30m (turning circle build in-land) from initial level. The basin cut inland very significantly, with 40% of turning circle and majority of terminals dredged inland.
	Breakwater	1.0	5	5	Relatively short and very shallow (max depth 15m, average depth much less), since the port is pushed in-land.
	Reclamation	0.2	4	0.8	Relatively small area of reclamation required only for multi-purpose berths.
	Quay length	0.4	3	1.2	Length of quay is comparable with other layout proposals
Sedimentation		0.5	4	2	Curved primary breakwater ensures no excessive accumulation of sediment at the outer face of breakwater. Long primary breakwater extending beyond secondary breakwater ensures that there is limited sediment accumulation in the entrance of the port
Na vig ati	Inside port	0.35	2	0.7	Potentially difficult navigation since berths within terminals are not

					collinear. Some berths of container terminal are not easily accessible from the turning circle. The inner approach channel is relatively long and berthing time may be longer than acceptable.
	At entrance/out side	0.35	3	1.05	Vessels entering the port are approaching at the moderate angle (compared with other layouts), to the dominant wave direction.
Safety		1.0	3	3	There is a small chance of crashing into Ro-Ro berth in case of loss of control over a vessel.
Expansion		0.6	1	0.6	The expansion in both direction is very limited. If expansion would go to the left, the primary breakwater may be used as secondary breakwater for the new basin, but the liquid storage must be moved (very difficult) and pipework must be re-routed underground. If expansion would go to the right, the bulk storage must be moved and new conveyor must be erected on a bridge. The new container will not be directly connected to the one currently designed
Construction and operation feasibility		0.3	4	1.2	All of the dredging is done in soft soil. There is no reclamation at large depth. The breakwater is constructed at much shallower depth than other designs. Accurate construction of curved breakwater may be potentially problematic and skilled contractor should be hired.
Total				**15.8**	

Table 37: MCA for Layout 3

Results

The results of the Multi-Criteria Analysis is presented in Table 38 below.

Criteria	Subcriteria	Layout 1	Layout 2	Layout 3
Costs	Dredging	0.6	0.4	0.2
	Breakwater	4	4	5
	Reclamation	0.6	0.4	0.8

	Quay length	1.2	1.2	1.2
	Subtotal	6.4	6	7.2
Sedimentation		1.5	1	2
Navigation	Inside port	1.4	1.05	0.7
	At entrance/outside	1.4	0.7	1.05
Safety		4	2	3
Expansion		3	1.8	0.6
Construction and operation feasibility		0.9	1.5	1.2
Total		**18.6**	**14.1**	**15.8**

Table 38: Summary of Multi-Criteria Analysis

Layout 1 achieved the highest score of 18.6, compared with 14.1 for Layout 2 and 15.8 for Layout 3. Thus, the further analysis in this report will be based on this design.

Summary

A multi-criteria analysis was performed for 3 distinct layout proposals. Detailed analysis is presented in Table 35, Table 36 and Table 37. A simplified summary is presented in Table 38.

Layout 3, with the breakwater pushed extremely inland was found to be most cost efficient.

Layout 1 was found to be the safest for navigation as well as the best for future expansion of the port.

In other categories (which are considered less important than ones mentioned above) there is no distinct advantage of any layout over another.

Overall, layout 1 was found to be the most suitable for the given design conditions. It achieved the highest score of 18.6, compared with scores of 14.1 and 15.8 for Layouts 2 and 3 respectively.

5. Lay-out definition

Introduction

In the previous chapter the final layout have been chosen using multi-criteria analysis from 3 preliminary designs. The purpose of this chapter is to evaluate the performance of the layout in wave agitation, as well as provide with navigation analysis and general operational plan.

Wave agitation

Based on the multi-criteria analysis one layout is chosen. The wave agitation model is done for this layout.

The port of the layout shall be such to protect berths from incoming waves. For the wave exceeded 1% of the time, the wave threshold values shall not be exceeded.

Wave thresholds may be considered as follows:

- Hs = 1.00m for oil and coal berths
- Hs = 0.70m for container berths
- Hs = 0.50m for general cargo and ro-ro berths

Modelling

Software: ARTEMIS (Agitation and Refraction with TElemac on a Mild slope)

This software takes into account:

- Mono-directional or multidirectional random waves;
- Diffraction by obstacles;
- Bottom refraction;
- Reflection by walls/breakwaters;
- Depth induced wave breaking.

To obtain the new bathymetry in the port the program 'AutoCad' is used to make different dredging contours. In BlueKenue these three dredging areas were adapted to make a more reliable agitation model, see Figure 18.

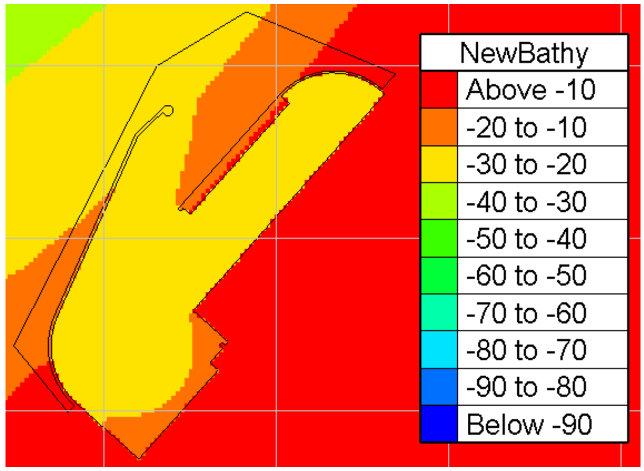


Figure 18: Bathymetry of the new port

The new port of Nador is exposed to two different wave directions. In our design the entrance of the port is at the NE side. For wave agitation only the second wave is leading. The orientation of the wave angle differs from the orientation in TOMAWAC. In ARTEMIS the leading direction of propagation corresponds to 260°, see Figure 19.

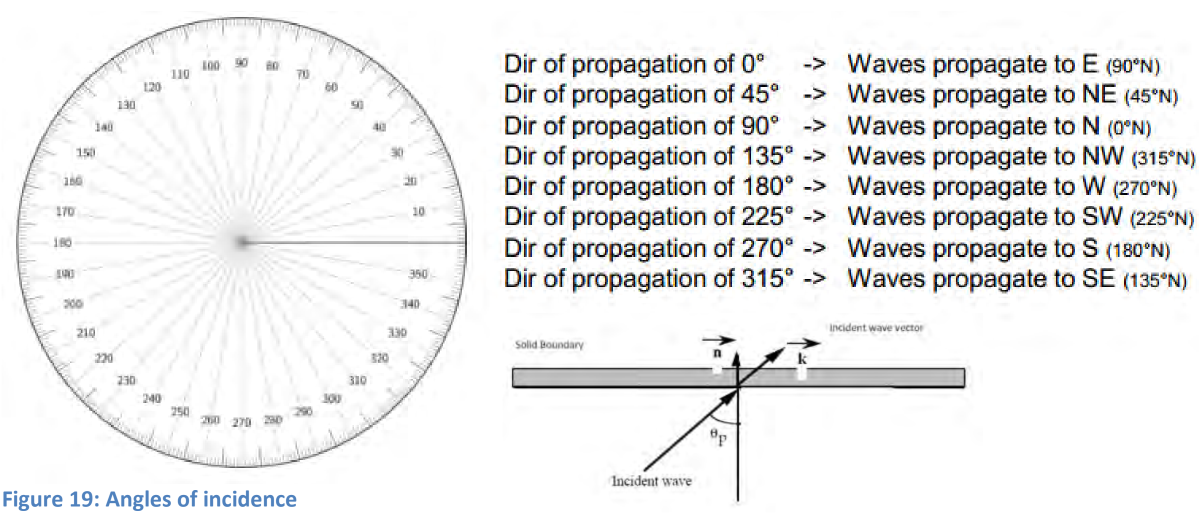


Figure 19: Angles of incidence

The quay walls (black), the breakwater (red) and the wave border need to be specified as a boundary condition, seeFigure 21: Contour port with hammer **Fout! Ongeldige bladwijzerverwijzing.**. To run the model the software needs the angle of incidence (see Figure 19), which can be obtained from the leading direction of propagation.

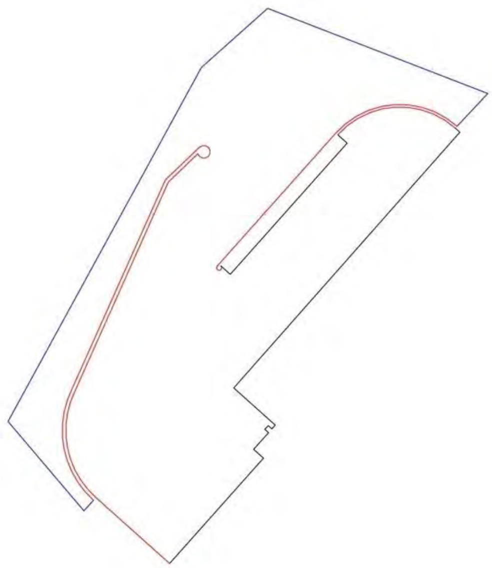


Figure 20: Contour port original

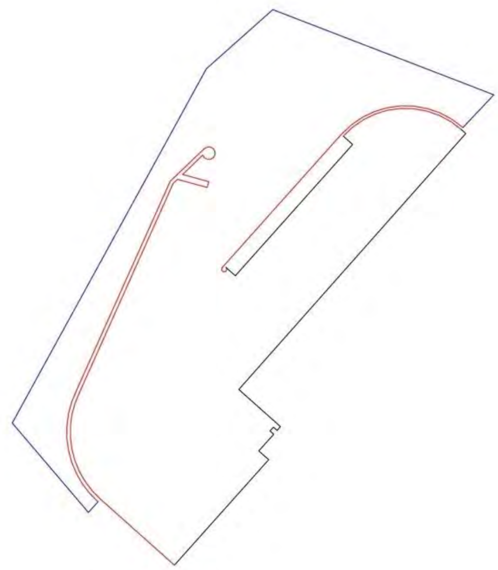


Figure 21: Contour port with hammer

Input

- Significant wave height H_s at the entrance of the port = 1.45 m
- Peak period T_p = 8.65 s
- Minimum spectral period = 0.5 * T_p = 4.33 s
- Maximum spectral period = 1.3 * T_p = 11.25 s
- Direction of wave propagation = 260 degrees (leading)
- Reflection coefficient quay wall = 0.9
- Reflection coefficient breakwater = 0.45

Output

After running the software an agitation model is obtained, see Figure 22

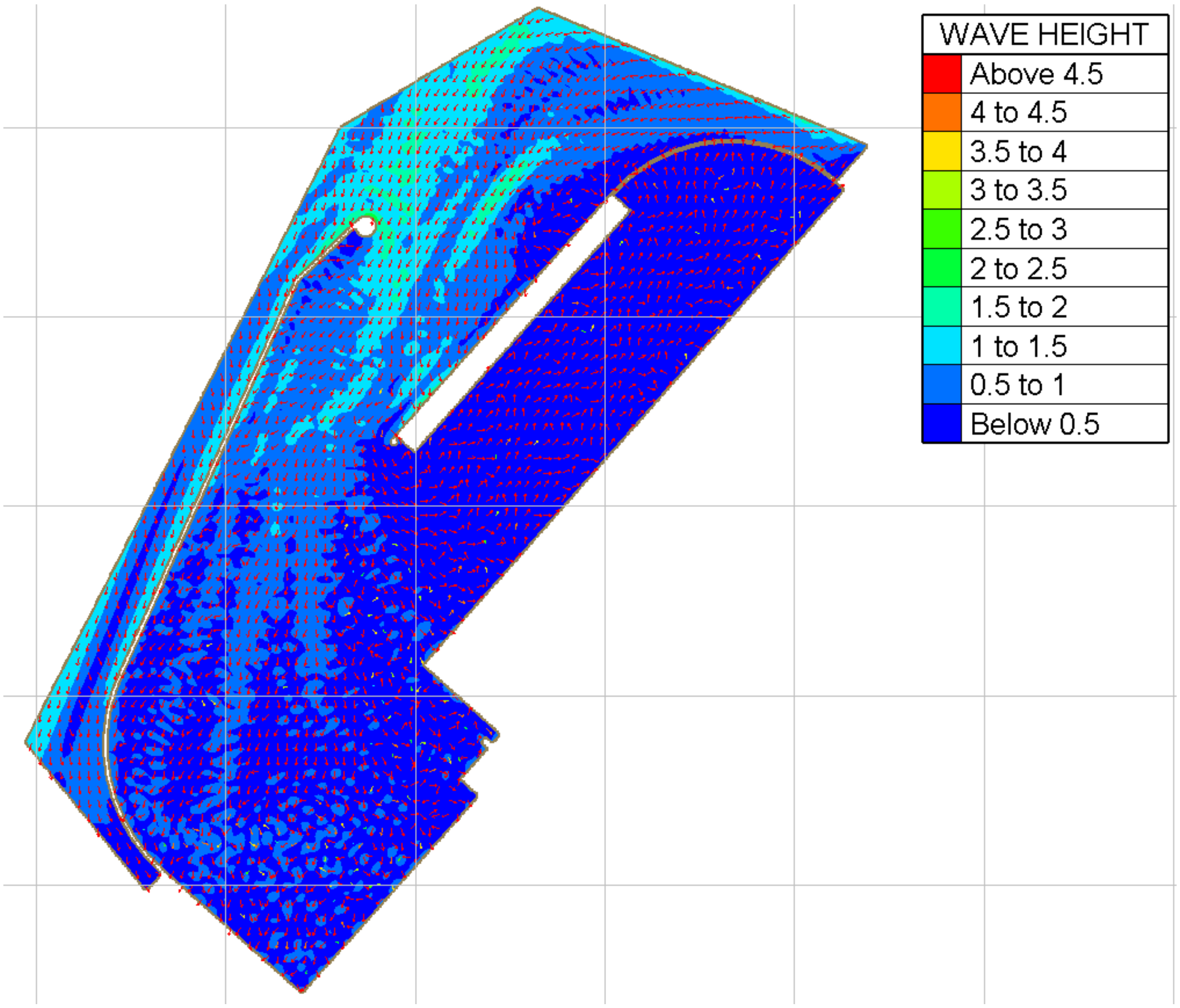


Figure 22: Wave agitation model output

Results

Only a few waves between 0.5 and 1.0m are travelling into the port to the container terminal. The container terminal need to meet the requirement of Hs = 0.70m and the coal Hs = 1.0m. The general cargo and ro-ro terminals may be exposed to Hs = 0.50m, which is the case in the most situations. The only problems will occur at the primary breakwater near the entrance of the port where the oil ships will berth.

The wave agitation requirements according to PIANC can be seen in Figure 23. Using these requirements the wave agitation in the New Port will be fulfilled.

Ship	Wave agitation requirements		Unavailability
Container	0.50m	1.00m	< 1 to 2%
Tanker	1.50m	2.00m	2 to 7%
LNG	1.00m	1.50m	2 to 5%
Break bulk	1.00m	1.50m	2 to 7%
Ferries, ro-ro	0.40m	0.80m	< 1 to 2%
Service port	0.40m	0.40m	-
Fishing	0.30m	0.30m	-

Figure 23: (PIANC, 2014)

To meet the requirements of the scope of works the breakwater need to be extended or an extra breakwater ('hammer') should be designed to reduce the waves in the port, see Figure 21: Contour port with hammer. Due to lack of time this adjustment is not tested for wave agitation.

Navigation analysis

Approach channel design

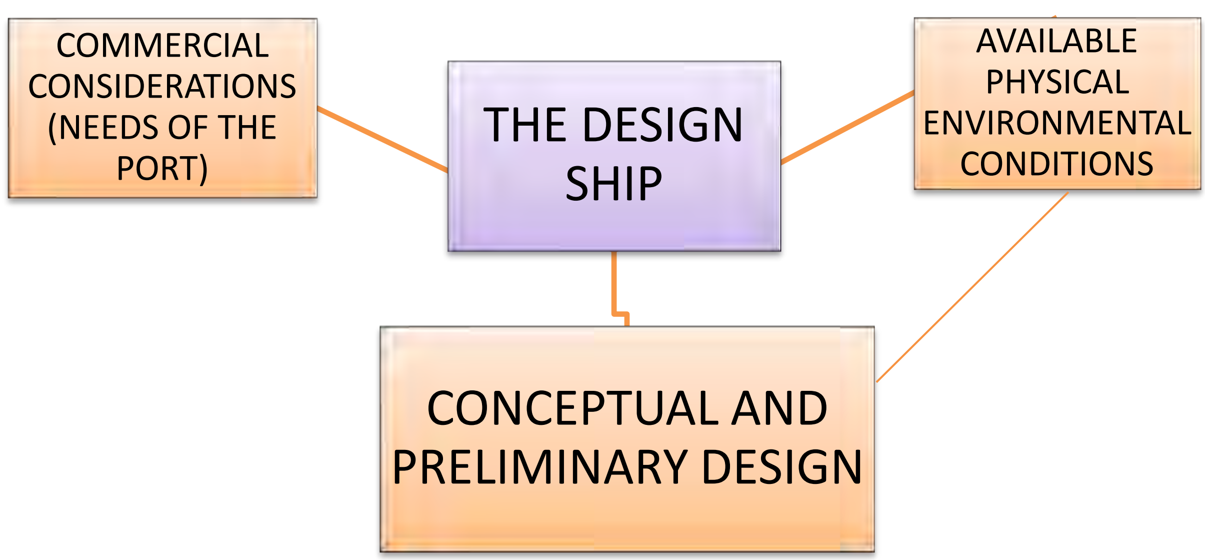


Figure 24: Flow chart of basic processes involved

Elaboration on the basic design considerations of an Approach Channel

The above flow chart (see Figure 24) gives a brief description of the prerequisite steps involved in an approach channel design. Below is a bullet point explanation of the design considerations.

- Commercial consideration: the requirements of the newly developed or modified Port.
  - Expected volume of cargo per year
  - Transhipment

- Expected Services to be rendered

- The design ship: the choice of the design ship depends on the below listed criteria:
  - General Governing Criteria
    - The right type (considering the needs of the Port)
    - All other ships likely to use the channel can do so safely
  - Technical Criteria
    - Manoeuvrability
    - Size in the context of port operations
    - Windage Area
    - Type of cargo transported
  - Hindrance to design ship selection
    - When the channel is to serve a mix of traffic containing both deep draughted ships and those with high windage
    - Sometimes you have to choose more than one design ship for ensure that all type of ships considered can use the channel safely
- Available physical environmental conditions: the existing natural conditions in the Port and its surroundings.
  - Bathymetry
  - Seabed features
  - Winds
  - Waves
  - Tides
  - Currents
  - Channel bottom conditions

- Conceptual and preliminary design: provides initial estimates of the overall physical parameters of the proposed channel. They are all linked by the approach velocity of the ship.
    - Width – multiple of the Ship's Beam
    - Depth – function of the Ship's draught
    - Alignment (Bend radii) – multiple of the Ship's length
    - Length
  - Operational Criteria: aids to navigation
    - Tugboats
    - Radars
    - Position of Navigational Towers
  - Operational limits
    - Vessel Speed limits
    - Weather condition limits
  - Marine traffic and Risk Analysis
    - Estimation of probable frequency of vessel collision and its consequences

Preliminary Design of the Approach Channel

In the following sections, the methodology used in obtaining the dimensions of the channel's parameters will be discussed. The reference used was the PIANC working group 2002 guidelines for the design of Approach Channels. Due to the variety of cargo the proposed port will handle, it was not feasible to select one design ship the design of the channel. Therefore, the worst-case scenario was considered in each case (width, depth, alignment etc). For the width of the channel, the ship with the widest beam was used as the design ship; for the depth, the ship with the deepest draught was used; and for the channel's alignment and bend radius, the ship with the biggest overall length was used.

Due to the project's objective of minimizing the cost of breakwaters and maximizing dredging, the overall layout of the port is closer to the coast, in intermediate and shallow waters. This limitation required a better use of the space allocated for the layout, positions, and depth of the external breakwaters protecting the harbour from waves. As a result, it was decided that a one-way channel is preferable to a two-way. Furthermore, as the emphasis is on the sheltered area of our harbour, an inner, straight approach channel was selected.

1. The Channel's Width

Using PIANC guidelines, the formula for the width of a one-way channel is as follow:

$$w = w_{BM} + \sum_{i=1}^{n} w_i + w_{Br} + w_{Bg}$$

Where:

w	=	the width of the channel
w_{BM}	=	the width of the basic manoeuvring Lane (function of the channel's location and the beam of the ship)
w_i	=	additional widths for straight channel sections (a function of the vessel speed allowed in the channel, the beam of the ship, and the channel's location and several other parameters such as: cross winds, cross and longitudinal currents, significant wave height and wavelength, aids to navigation, bottom surface, depth of waterway, and cargo hazard level)
w_{Br}	=	additional width for bank clearance on the right-side due to sloping channel edges and shoals, and steep and hard embankments: structures (a function of the channel's location and the ship's beam)
w_{Bg}	=	additional width for bank clearance on the left-side due to sloping channel edges and shoals, and steep and hard embankments: structures (a function of the channel's location and the ship's beam)

Results

Channel Width							
		Values					
Description	Symbol	Tanker	Container Ship (Large)	Bulk	RO-RO	General Cargo	Unit
Total Width - One way channel	w	189	105	112	49	41	m

Table 39:Results showing the calculated channel width required for different ships. The highlighted cell indicates the width chosen for the design

2. *The Channel's Depth*

In this section, the water depth shown on the bathymetry was taken as the distance from the MSL (+0.35 CD) to the seabed. To know the lowest water depth expected in the channel, the water depth at the Lowest Astronomical Tide (+0.00 CD) level was used in the calculations, assuming that the tidal variations in our area of focus are negligible.

$$h = h_B - (MSL_{CD} - LAT_{CD})$$

Where:

h = the water depth used in the calculations
h_B = the water depth obtained from the bathymetry
MSL_{CD} = Mean Sea Level with reference to the Chart Datum
LAT_{CD} = Lowest Astronomical Tide with reference to the Chart Datum

To calculate the depth required in the channel, PIANC guidelines proposes the formula below:

$$h_C = T - h_T + S + UC + A$$

Where:

h_C = is the required channel depth below the MSL
T = the admissible Draft of the Design Ship
h_T = Tolerance for dredging
S = Effects due to squat/vertical trim
UC = Under-keel Clearance including
A = Depth factors due waves, atmospheric pressure, sounding accuracy, allowance for sediment deposit between maintenance dredging operations, and character of bottom rock

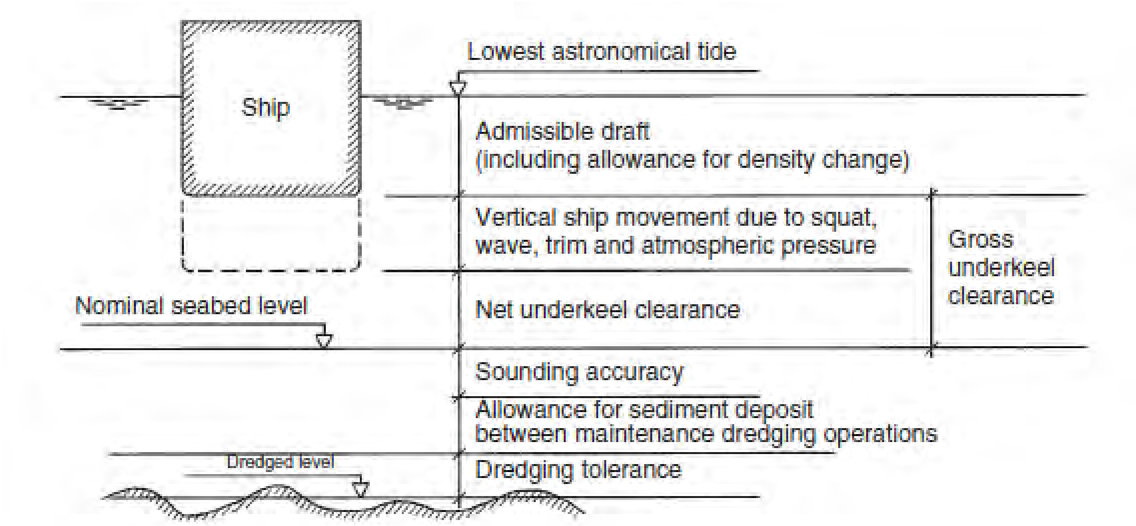


Figure 25: Graphical description of the required channel depth from the Port Designer's Handbook

Results

Channel Depth								
		Values						
Description	Symbol	Tanker	Container Ship (Large)	Bulk	RO-RO	General Cargo	Unit
Channel Depth	Dc	26	16	21	9	15	m

Table 40: Results showing the calculated channel depth required for different ships. The highlighted cell indicates the depth chosen for the design

3. Channel Alignment

Based on the limited space in our layout, the navigation of ships within the port will be aided with the use of tugboats. This allows for the reduction of the Turning Circle diameter and radius. A rudder angle of 20 degrees and a bend angle of 45 degrees will be used respectively.

Bend Radius

The total alignment of the channel includes a bend from the area exposed to waves, to the part sheltered by the external breakwaters. The length of this bend is called the bend radius.

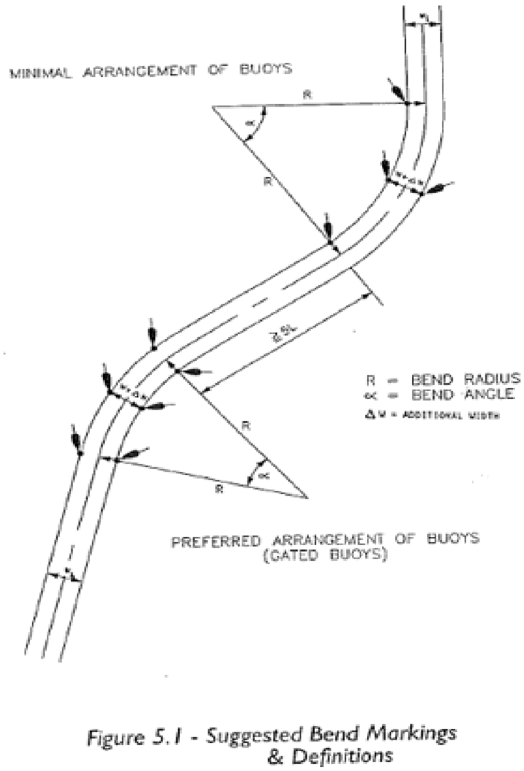


Figure 26: Graph showing bend radius and angle

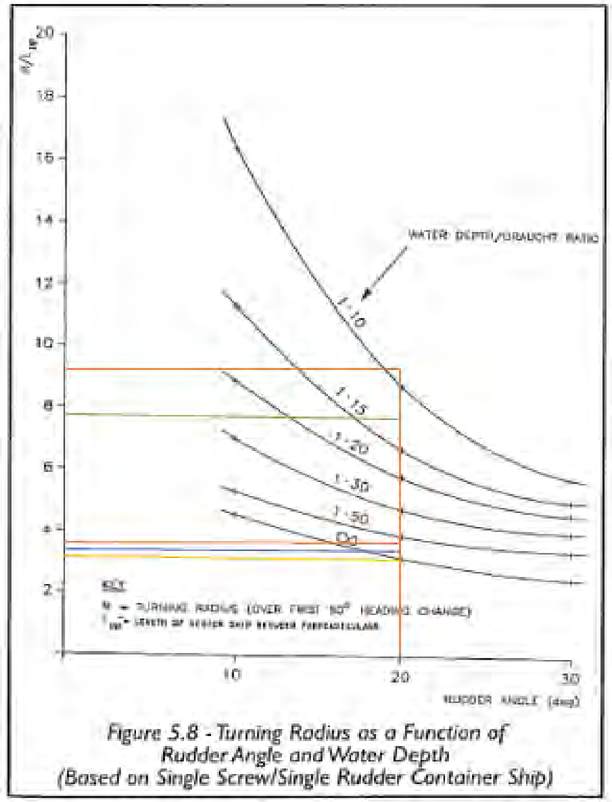


Figure 27: Turning Radius estimation graph (PIANC 2002)

By finding the water depth to draught ratio, the Bend Radius was calculated using the chart in the figure below. The coloured lines represent the extrapolated values obtained and used for our project.

$$BR = L_{pp} * Extrapolated\ Values\ from\ the\ Chart$$

Where:

BR = the bend radius of the channel

L_{pp} = the length between perps of the ship

Width of Swept Track

This is defined as the track swept out by the extremities of the ship while manoeuvring. It is calculated as a function of the water depth to draught ratio, the rudder angle, and the ship's beam. An extrapolation of the values was done using the graph below. The coloured lines represent the extrapolated values obtained and used for our project.

$$Ws = B * Extrapolated\ Values\ from\ Chart$$

Where:

Ws = the width of swept track

B = the Beam of the Ship

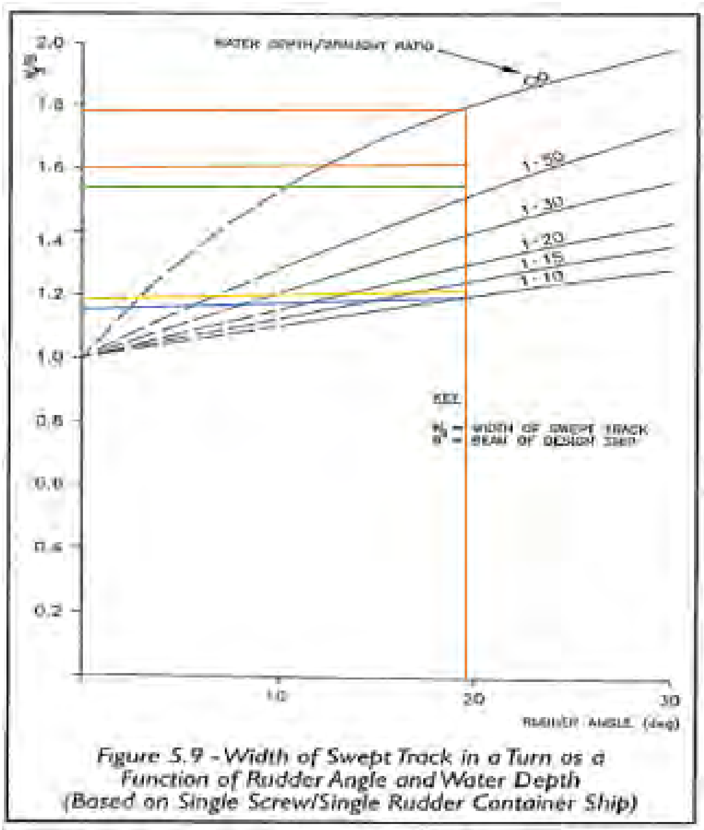


Figure 28: Swept Track Width Estimation (PIANC 2002)

Results

Channel Alignment

Description	Symbol	Value					Unit
		Tanker	Container Ship (Large)	Bulk	RO-RO	General Cargo	
Rudder Angle	R	20	20	20	20	20	deg
Bend Radius	BR	3003	1011	2087	580	439	m
Width of swept track	Ws	68	64	53	46	35	m

Table 41: Channel alignment results

4. Manoeuvring Areas within the Port

To enable safe manoeuvring of vessels in the port areas, several parameters need to be determined. The total bollard-pull required and the subsequent number of tugboats required for each type of vessel, and the total length within the protection of the breakwater.

Tugboat Assistance

In order to maximize the limited space available for our harbour, the use of tugboats is compulsory in enabling a safe navigation of the vessels. The calculation of the total bollard pull per ship was done using a formula from the “Ports and Terminals” (Ligteringen & Velsink, Ports and Terminals, 2014) book. It is a function of the size of the vessel. The capacity is expressed as the maximum bollard pull of a tugboat.

$$TB = \frac{\Delta}{100000} * 60 + 40$$

Where:

TB = the maximum bollard-pull of the tugboat in tons

Δ = the ship's displacement

Tugboats with a capacity of 60 tons were selected for the project. Therefore, determining the number of tugs required for safe navigation of a vessel is achieved by dividing the total bollard pull by the capacity of the tug.

$$N_{tugs} = \frac{TB}{C_{tugs}}$$

Where:

N_{tugs} = the number of tugs required per vessel for safe navigation

C_{tugs} = the capacity of the tugs in tons

Stopping length

This is basically the total length required to bring the vessel to a stop from the entrance of the inner channel to the turning circle. It is determined by summing the entrance speed, the time required to tie up the tugboats and to manoeuvre them in position, and the final stopping length. Normally, the vessel speed used in these calculations is the minimum required vessel speed, usually 4 knots or 2.06 meters per seconds.

Entrance Speed

The length needed to slow down is found using the formula below. (Ligteringen & Velsink, Ports and Terminals, 2014)

$$L_1 = (v_s - 2) * \frac{3}{4} L_s$$

Where:

L_1 = the total length required for the vessel to slow down

v_s = the minimum vessel speed required in the channel (4 kn or 2.06 m/s)

L_s = the required channel width using the minimum vessel speed

5. Corresponding Length required to tie up tugboats

The time required for tying up the tugboats is a function of the minimum vessel speed required, the environmental conditions present during navigation, and the experience and expertise of the tug crew. In practice, an approximate time of 10 minutes is used, which is converted to seconds in the formula. (Ligteringen & Velsink, Ports and Terminals, 2014)

$$L_2 = 10 * 60 * v_s$$

Where:

L_2 = the corresponding length required to tie up the tugboats

6. The final stopping distance

The final stopping distance is then determined by multiplying the width of channel (considering the minimum vessel speed) by a coefficient of 1.5. (Ligteringen & Velsink, Ports and Terminals, 2014)

$$L_3 = 1.5 * L_s$$

Where:
L_3 = the final stopping distance

Finally, the total length within the protection of the breakwater is determined by finding the sum of the previous 3 lengths.

$$L_{tot} = L_1 + L_2 + L_3$$

Results

Manoeuvring Areas within the Port							
		Values					
Description	Symbol	Tanker	Container Ship (Large)	Bulk	RO-RO	General Cargo	Unit
Bollard Pull	BP	247	100	168	53	52	tons
Tugboat Capacity	TgC	60	60	60	60	60	tons
Number of tugboats required	Tugs	4	2	3	1	1	nbr
Length needed to slow down	L_1	8	5	5	2	2	m
Time required to tie up tugboats	Treq	10	10	10	10	10	min
Corresponding length required to tie up tugboats	L_2	1235	1235	1235	1235	1235	m
Final Stopping distance	L_3	283	157	168	74	62	m
Total length within the protection of a breakwater	L_{TOT}	1526	1396	1408	1311	1299	m

Table 42: Results of navigation analysis

Organization plan

In this section an organizational plan of the major terminals are given, showing the most important operational zones of each one.

Container terminal

Ship-to-shore gantry cranes will be used for unloading the ship immediately after the ship has arrived at the berth. The cranes are provided with a trolley and a cabin, which moves with it, from which the crane driver guides the trolley and the spreader to the right container on the ship. The container is picked up and transported to the space between the seaward and landward leg of the crane, where it is lowered and placed on the straddle carrier which is in use between the quay and stack.

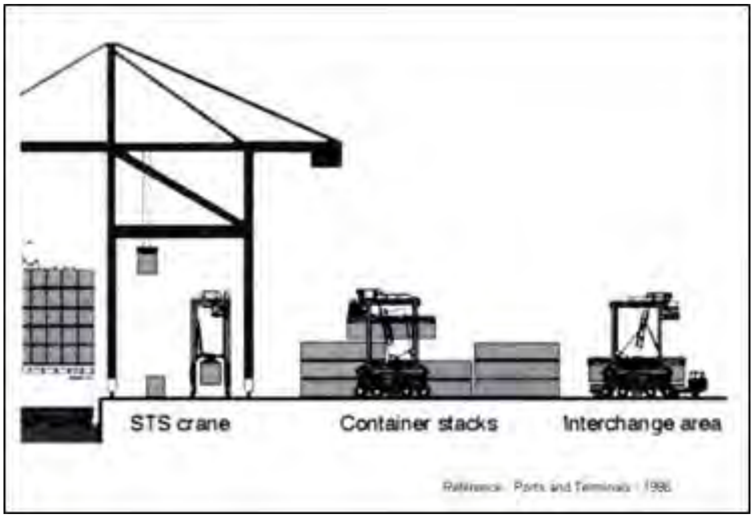


Figure 29: s-t-s gantry cranes (Silva P. , Terminal Typology, 2016)

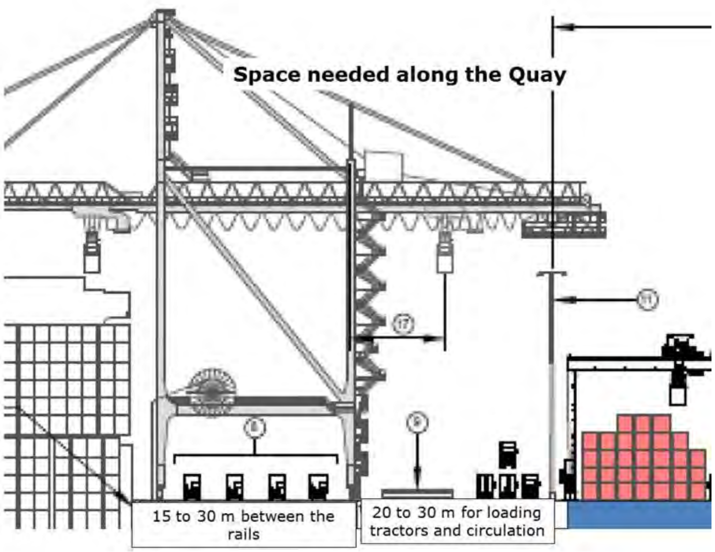


Figure 30: s-t-s gantry cranes (Silva P. , Terminal Typology, 2016)

6. Structural dimensioning

Introduction

The purpose of this chapter is to present the design of typical structural members, namely the primary breakwater, mooring dolphins with fenders and a quay wall.

Breakwater

In this chapter the design of the breakwater will be presented. After having decided the port layout with the exact location of the breakwaters, the next step will be to design the breakwaters and all of its dimensions. The breakwater is an essential part of the port. Its function is to dissipate the wave energy in order to protect an area from waves. In the case for this project the breakwaters are used to keep the inner part of the port calm. The breakwater should be dimensioned based on the local boundary conditions (wave conditions), but also on the requirements given by the port client. The local wave conditions are given by the results of the wave propagation model.

OVERVIEW LAYOUT

To ensure a calm swell in the inner part of the port there are two breakwaters. There is the primary breakwater protecting the harbor to the directions South-West and North-East with a total length of 1.97 km. The secondary breakwater protects the port to the directions North-East and South-East with a length of 1.54 km. As you can see in **Fout! Verwijzingsbron niet gevonden.** the primary breakwater is more offshore with a maximal water depth of 22 m. The secondary breakwater is closer to the shore with a maximal water depth of 11 m. The primary breakwater has a maximal distance of 1.1 km to the originally shoreline.

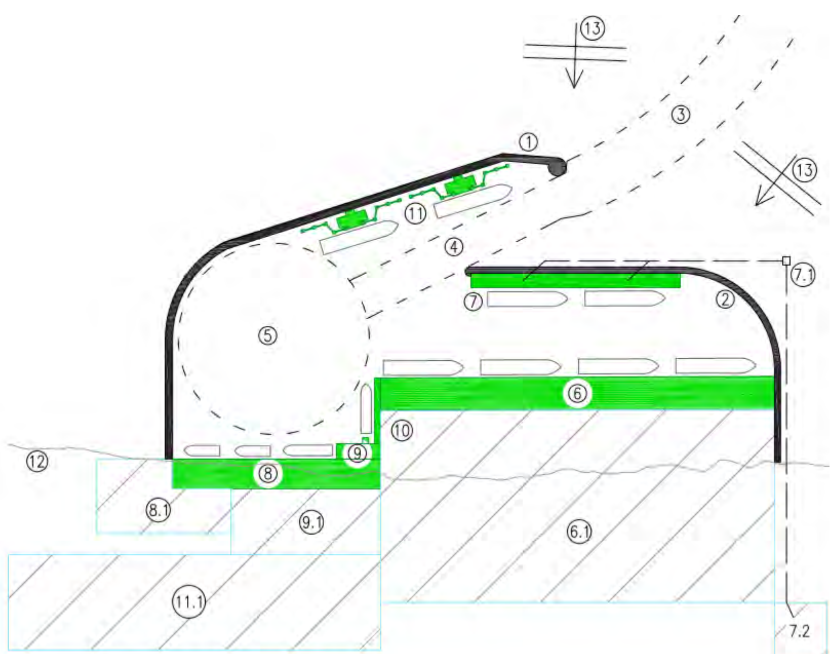


Figure 31: Layout

First of all, the decision for the type of the breakwater should be made. Knowing that the maximal water depth of the breakwater is 22 m, the decision of building a rubble mound breakwater was made. Both breakwaters will be designed with cubic concrete armour units. In total, one cross-section is calculated for the primary breakwater at the most inconvenient position, being the position directly in front of the round-head.

REQUIREMENTS

The design of the breakwater depends of course on the wave conditions, but also on the functional requirements like allowable damage and allowable overtopping.

Allowable damage

The breakwater should be stable for the extreme wave conditions (once per 100 year wave height), but some damage are unavoidable. Besides, a very conservative breakwater will result in small damages, but the investments will be high. A compromise must therefore be found between economics (the short term as well as the long term) and safety.

For this design it is assumed that during the extreme wave conditions some damage is allowed. In those cases, some repairs are accepted.

Allowable overtopping

The amount of the overtopping determines the crest height of the breakwater.
Knowing that behind the breakwater there will be the liquid terminal, the allowable overtopping in this case is 0.02 l/s per meter structure length for the design (safety criteria for operations as it can be seen in Figure 32)

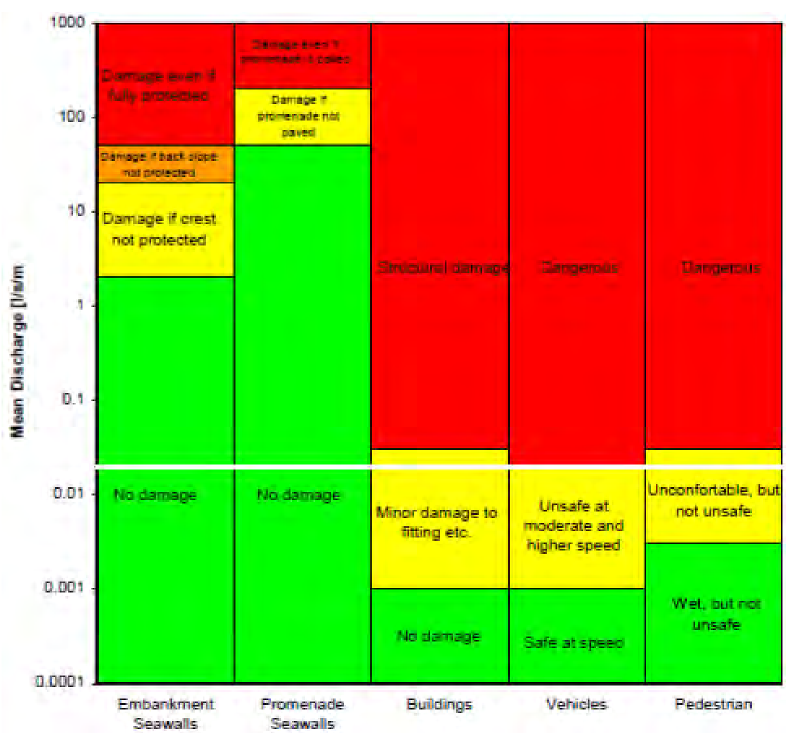


Figure 32: Limiting mean overtopping discharge (M.W., 1980)

Local boundary conditions

From the analysis made in chapter 4 the maritime conditions of the port are determined and the design of the breakwater can be done. The design parameters are taken for a 100 year return period and are shown in Table 43. These values are used for the preliminary design of the structures.

Water depth	Hs (1/100 year)	Tp (1/100 year)	Direction
22 m	5.0 m	10.1 s	317.6°N

Table 43: Wave parameters from long-term wave statistics and spectral wave's model

Dimensioning of layers

The typical cross-section of the rubble mound breakwater is shown in Figure 33. The design has to fulfil hydraulic performances and structural responses. Firstly, the structural parameters are calculated; these include the stability of the armour layer, the under layer, the core and the crest height of the breakwater. Then, the hydraulic parameters are determined for wave overtopping. Finally, a preliminary geometry of the breakwaters is done and an experimental analysis is performed to check the preliminary designed structures.

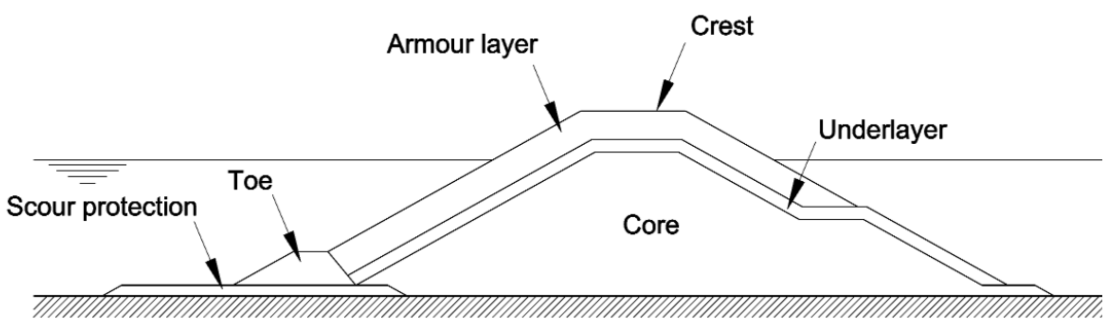


Figure 33: Cross-section of a typical rubble mound breakwater with superstructure (CIRIA, 2007)

To design the section of the breakwater, we first considered different options in order to compare them and choose the most suitable one for the armour layer. In the beginning concrete cubes and rocks were considered. It was found that these rocks were very heavy as it would weight 28.4 t each rock. So for this project the verdict was in favour of the concrete cubes. The seaward slope of the structure will be designed with a slope of 1:1.5.

Armour layer

The armour layer is designed in such a way that the stability is fulfilled when the seaward side of the structures is under wave attack. For the calculation of the required stone sizes the Hudson formula will be used. Hudson developed a simple expression for the minimum armour weight required to resist a (regular) wave height, H, which is given as:

$$M = \frac{\rho_s \cdot H^3}{K_D \cdot cot\alpha \cdot \Delta^3}$$

In which:

ρ_s = Mass density of concrete [kg/m^3]

K_D = Stability coefficient [-]

α = Slope angle of the structure [°]

Δ = relative mass density [-]

The relative mass density is given as:

$$\Delta = \frac{\rho_s}{\rho_w} - 1$$

In which:
ρ_s = Mass density of concrete = 2400 [kg/m³]
ρ_w = Mass density of water = 1025 [kg/m³]

The relative mass density becomes therefore Δ = 1.34.

The value of K_D for concrete should be taken as 6.5, according to the rock manual for slopes between 1:1.5 - 1:3.

According to the SPM, it's better to consider the ten-year wave height for the design:

$$H_{1/10} = H \ = 1.27 \cdot H_s$$

This will be 6,35 m for the design wave height.

So finally we obtain a concrete mass per cube of M = 26.1 t.

And the equivalent cube size of the block is defined as:

$$D_n = \left(\frac{M}{\rho_s}\right)^{1/3}$$

In this case we obtain a cube length of 2.22 m for the armour layer.
As it's explained in chapter XX, the concrete mass will be increased by 21 % after the analysis made during the wave flume test.

In Table 44, the results for the dimensions of the armour layer of the primary breakwater in front of the roundhead and the modified values are summarized.

	Material	**Mass**	**diameter**	**Number of layers**	**Layer thickness**
Armour layer (calculated)	Concrete cubes	26.1 t	2.22 m	2	4.4 m
Armour layer (modified)	Concrete cubes	31.6 t	2.36 m	2	4.7 m

Table 44: Dimensions of the armour layer

Underlayer

Beneath the armour layer, one or more progressive granular underlayer must be placed in order to assure the stability of the breakwater. Underlayers should be designed to prevent the washout of fine material. But on the other hand the underlayer should allow the transport of water since our breakwater will be permeable. The Shore Protection Manual (Engineers, 1984) recommends for the relation of the stone mass of the underlayer $M_{underlayer}$, and of the armour M_{armour}, should be between:

$$\frac{M_{50,underlayer}}{M_{armour}} = \frac{1}{15}\ to\ \frac{1}{10}$$

For this design, the material of the underlayer is 1/12.5 of the mass of the armour blocs. The underlayer will be made of rocks with a rock mass density of 2650 kg/m³. The equivalent cube length of median rock can be obtained by the following equation:

$$D_{n50,underlayer} = \left(\frac{M}{\rho_s}\right)^{1/3} = 0.98\ m$$

After careful consideration and consulting Mr. Safari we have come up to use only one underlayer by using the thickness of two underlayers. And additionally a relatively large stone size in the underlayer gives more interlocking with the armour because of its larger surface.

The relation between the diameters of the blocks and the rocks should be checked as well since it should fulfil the filter criteria. The relation between both diameters should be between 2.2 and 2.5. Otherwise, finer particles will be extracted from the inner part of the breakwater. This criterion is stated in the Rock Manual:

$$\frac{D_{n,armour}}{D_{n50,underlayer}} = \frac{2.36\ m}{0.98\ m} = 2.4$$

It is considered as a sufficient value for the relation.

The results are given in the following table.

	Material	Mass	diameter	Number of layers	Layer thickness
underlayer	rocks	2.5 t	0.98 m	4	3.9 m

Table 45 Dimensions of the underlayer

The range of masses is chosen by experience and it is between half of the nominal mass and the double of it.

Core

In order to reduce the costs, sand material was selected for the core as it can be obtained from the dredging process. After careful consideration and consulting Mr. Silva we have come up to not use sand for the core. To prevent the sand, an additional system would have to be taken into account. This could have been a geotextile layer, which would make the core less permeable and further which would be very complicate to be used and placed during the construction under water. So finally, the core will be filled with quarried rock material which will fit better with the progressive layer material diameter criterion. In order to assure the stability of the core for it not to be washed out, the weight relation exposed before will be used too:

$$\frac{M_{50,core}}{M_{50,underlayer}} = \frac{1}{15}\ to\ \frac{1}{10}$$

For this design, the material of the core is 1/15 of the mass of the underlayer rocks. Thus $M_{50,core}$ is 169 kg.

The diameter of these particles will be obtained as follows:

$$D_n = \left(\frac{M}{\rho_s}\right)^{1/3} = 0.4\ m$$

The filter criterion is checked again:

$$\frac{D_{n50,underlayer}}{D_{n50,core}} = \frac{0.98\ m}{0.4\ m} = 2.45$$

Which is also in the acceptable range.

	Material	Mass	diameter
core	rocks	169 kg	0.4 m

Table 46: Dimensions of the core

Toe

The toe is built to protect the bottom and support the seaward armour layer as you can see in the Figure 34.

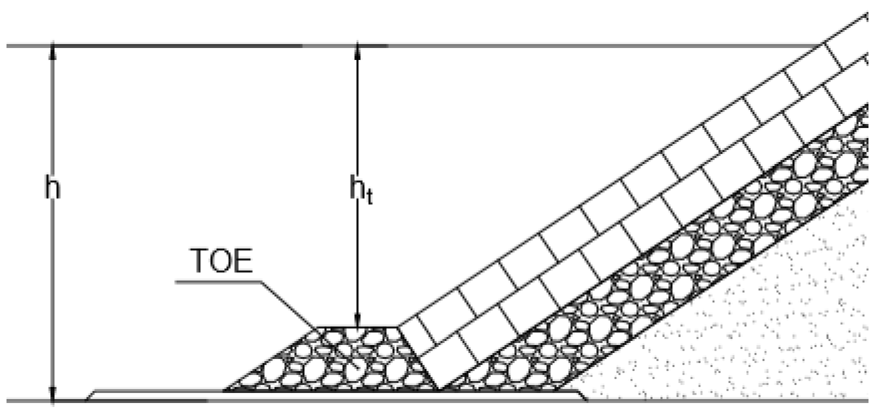


Figure 34: Cross-section with toe protection

The purpose of it is to ensure the stability of the armour, in order to avoid sliding as well as to offer an additional reinforce concerning the seabed scour that would affect the breakwater.

Toe stability is essential because failure of the toe will often lead to failure throughout the entire structure, and his cost is small compared with the cost of the armour.

The formula for the toe stability is given by the following equation:

$$\frac{H_s}{\Delta D_{n50}} = 7.8 \cdot \left(\frac{h_t}{h}\right)^{1.43} \cdot N_{od}^{0.15}$$

In which:

H_s	=	Significant wave height in front of the breakwater [m]
Δ	=	relative mass density [-]
D_n	=	Diameter of the stones [m]
h_t	=	Water depth at the toe on the seaside [m]
h	=	Water depth [m]
N_{od}	=	Damage number [-]

In which N_{od} is the damage parameter. For this design a N_{od}-value of 0.5 will be used which corresponds to a 'start of damage' being a safe figure for the design after the Rock Manual.

In this equation the diameter D_n is function of the water depth over the toe and the water depth is function of the diameter D_n so that means that an iteration has to be carried out to determine both values. For h_t a depth of 17.5 m is obtained and D_n will be 0.62 m. To reduce the costs it's chosen to use for the toe the same stones of the underlayer so the new diameter is 0.98 m.

The rock manual is advising to use between three and five D_n for the width. For this project a width of 5 stones and an height of 6 stones will be calculated.

The results for the toe are given in the following table.

	Material	**Mass**	**diameter**	**height**	**width**
toe	rocks	2.5 t	0.98 m	3.9 m	4.9 m

Table 47: Dimensions of the toe

Scour protection

Also the scour protection is required, it has a function of preventing the erosion of the seabed surface, and a sufficient amount of protection has to be placed. After consulting Mr. Silva we have come up to construct a 0.6 m thick and 31.2 m long scour. The used material will be gravel with a diameter between 2 and 3 cm.

	Material	**diameter**	**height**	**width**
scour	gravel	2-3 cm	0.6 m	31.2 m

Table 48: Dimensions of the scour protection

DIMENSIONING OF THE CREST LEVEL

The height of the crest level is determined by the allowable amount of overtopping.

Overtopping

Overtopping is a phenomenon that occurs when a wave is passing over the crest of a structure. Nowadays, two different parameters are used to account for this phenomenon.

The average overtopping volume, q, is measured in litres per second per running metre and represents the whole time-domain discharge. The crest height, which represents the breakwater's height above the water level, is associated with the average overtopping. For the calculation of the required crest height, the EurOtop formula is used (al, 2016). This formula is given as:

$$\frac{q}{\sqrt{g{H_s}^3}} = a \cdot exp\left[b\,\frac{R_c}{H_{m0}\gamma_\beta}\right]^{1.3}$$

In which:

H_s	=	Significant wave height in front of the breakwater [m]
a	=	Parameter (see below) [-]
b	=	Parameter (see below) [-]
R_c	=	Crest level [m]
H_{m0}	=	Spectral wave height [m]
γ_β	=	Reduction coefficient for oblique incoming waves [-]

With using the two following parameters:

$$a = 0.09 - 0.01(2 - cot\alpha)^{2.1} \ = \ 0.09$$

$$b = 1.5 + 0.42(2 - cot\alpha)^{1.5} = \ 1.65$$

As mentioned above the corresponding maximum overtopping is 0.02 l/s/m. The angle of obliquity (angle with respect to the perpendicular of the breakwater) is 24.4°, so the reduction coefficient becomes 0.95. With all these data, the obtained crest level R_c is 6.6 m.

DIMENSIONING CROWN WALL

The breakwater will be designed with a crown wall along the whole length. The crown wall makes it possible to access the breakwater for maintenance and it reduces the overtopping. For the dimensioning of the crown wall and the stability the method of Pedersen is used (see Figure 35).

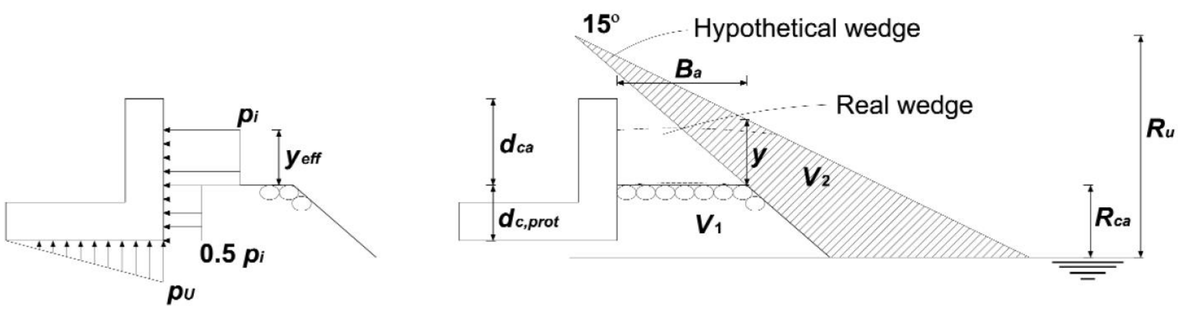


Figure 35: pressure distribution from Pedersen (CIRIA, 2007)

The horizontal impact pressure is given as:

$$p_i = g\rho_w(R_{u\,0.1\%} - R_{ca})$$

In which:

ρ_i	=	Horizontal impact pressure [kN/m^2]
$R_{u,0,1\%}$	=	0,1% wave run-up level [m]
R_{ca}	=	Height of crest of armour berm [m]

The equation for the wave run-up level according to Van der Meer and Stam (1992) is given as:

$$\frac{R_{u\,0.1\%}}{H_s} = b \cdot \xi_m{}^c \quad for\, \xi_m > 1.5$$

The coefficients b and c are given with 1.34 and 0.55. And ξ_m is the Iribarren breaking parameter:

$$\xi_m = \frac{\tan \alpha}{\sqrt{s_m}}$$

ξ_m is in this case 3.16 with α=33.7° and s_m=0.045.

The wedge thickness is given as:

$$y = \frac{R_{u\,0.1\%} - R_{ca}}{\sin \alpha} \frac{\sin 15°}{\cos(\alpha - 15°)}$$

Where α is the slope angle of the armour layer (°).

And the effective height of the impact zone y_{eff} is given by the following equation:

$$y_{eff} = \min\left\{\frac{y}{2}\ ;\ d_{ca}\right\}$$

In which d_{ca} is the height of the crown wall above the armour crest. In this design, the vertical parapet of the crown wall is completely protected by the armour units. The value of d_{ca} and therefore the value of y_{eff} becomes thus 0.

The horizontal force on the crown wall is given by the following equation:

$$F_{H\,0.1\%} = 0.21 \sqrt{\frac{L_{om}}{B_a}} \left(1.6\, p_i\, y_{eff} + V \frac{p_i}{2} d_{c,prot}\right)$$

In which:

$F_{H,0.1\%}$	=	the horizontal force with a probability of exceedance of 0.1% [kN/m]
L_{om}	=	deep water wave length based on T_m [m]
B_a	=	Berm width in front of the crown wall [m]
V	=	Min {V_2/V_1; 1} (see figure XX for the determination of V_1 and V_2 [-]
$d_{c,prot}$	=	Height of the protected part of the crown wall [m]

For the determination of the horizontal forces at each cross section, the deep water wave length L_{om}, the width of the berm in front of the wall, the value of V and the value of $d_{c,prot}$ should be determined. The width of the berm is determined as 3 times the D_n of the armour unit. The deep water wave lengths and berm widths are summarized in Table 49.

The turning moment, generated by the wave, $M_{H,0.1\%}$ is given by:

$$M_{H\,0.1\%} = 0.55\,(d_{c,prot} + y_{eff})\, F_{H\,0.1\%}$$

And finally the uplift pressure, generated by the waves $p_{U,0.1\%}$, is given by:

$$p_{U\,0.1\%} = 1.0\, V\, p_i$$

All the results are given in Table 49.

$R_{u,0,1\%}$	y	y_{eff}	L_{om}	D_n	B_a	V_1	V_2	V_2/V_1	V

12.6 m	2.99 m	0 m	112.5 m	2.4 m	7.1 m	89.5 m²	228.8 m²	2.56	1

Table 49: Calculation of several values

Now, the horizontal and vertical forces and the moments can be calculated with the above described formulae. The results of those calculations are given in Table 50.

$F_{H,0.1\%}$	$p_{u,0.1\%}$	$M_{h,0.1\%}$
191.1 kN	60.9 kN/m²	788.4 kNm

Table 50: Calculation of the forces on the crown wall

In the final step it is now possible to design the crown wall. The width of the crown wall (d) has to be at least 8 m, because that is the minimal width required for maintenance cranes to access the breakwater. In this case the crown wall width will be 10 m. The other dimensions of the crown wall will first be based on practical considerations. After that, the corresponding volume and weight of the crown wall will be calculated to fulfil the stability against sliding. The final dimensions of the crown wall are given in Figure 36.

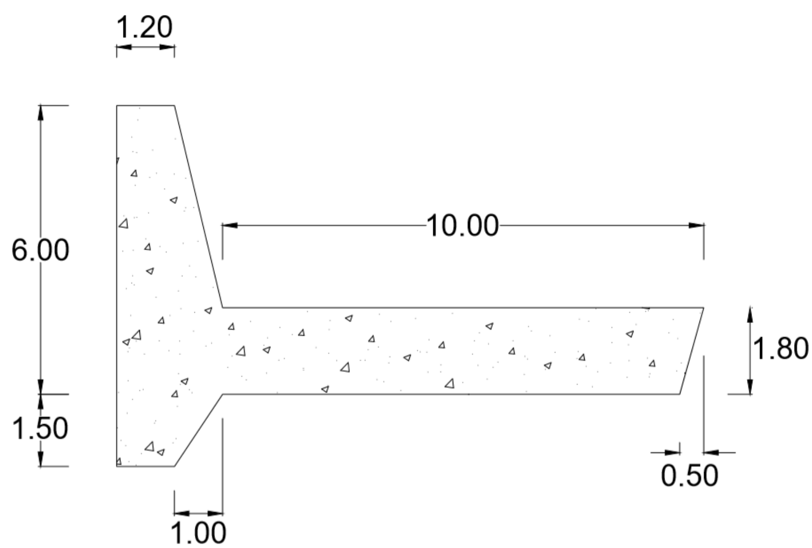


Figure 36: Details of the crown wall

The stability against sliding can be checked with the following equation:

$$f\ (F_G - F_U) \geq F_{H\ 0.1\%}$$

In which:

f	=	Friction coefficient [-]
F_G	=	weight of the crown wall [kN/m]
F_U	=	wave induced uplift force [kN/m]
F_H	=	wave induced horizontal force [kN/m]

The friction coefficient can be assumed to be approximately 0.5. The horizontal forces are already calculated (see Table 50). The uplift force can be calculated as being the area of the under pressure (see Figure 35). The weight of the crown wall depends on the dimensions of the crown wall (see Figure 36). The crown wall will be designed above SWL, so there is no buoyancy effect. A summary of the calculations is given in Table 51. If the value of $f(F_G – F_U)$ is larger than the horizontal force, the crown wall can be assumed to be stable against sliding. It can be seen that stability against sliding is provided.

Volume crown wall	Weight crown wall	Wave induced uplift force	Wave induced horizontal force	$f(F_G – F_U)$
31.2 m³/m	780 kN/m	371.6 kN/m	191.1 kN/m	204.2 kN/m

Table 51: Calculation of the forces on the crown wall

Crest width

The crest width will be composed of the number of concrete cubes being side by side horizontally on the top of the crest and the width of the crown wall. In this case the total width will be 19.8 m, consisting out of 3 concrete cubes with a total width of 7.1 m and the crown wall width of 12.7 m.

Inner/rear armour

The inner slope of the breakwater is more or less protected against the outer wave action. Therefore, this slope can be designed only with the rocks. The armour layer of the inner slope will be designed with the same material as the under layer at the offshore part of the breakwater. The slope of the inner side will be the same slope as the offshore part that means 1:1.5.

Global Stability

Final breakwater section

A preliminary design of the primary breakwater is illustrated on Figure 37. It is designed by taking into consideration the results from the previous subsections and by engineering judgment.

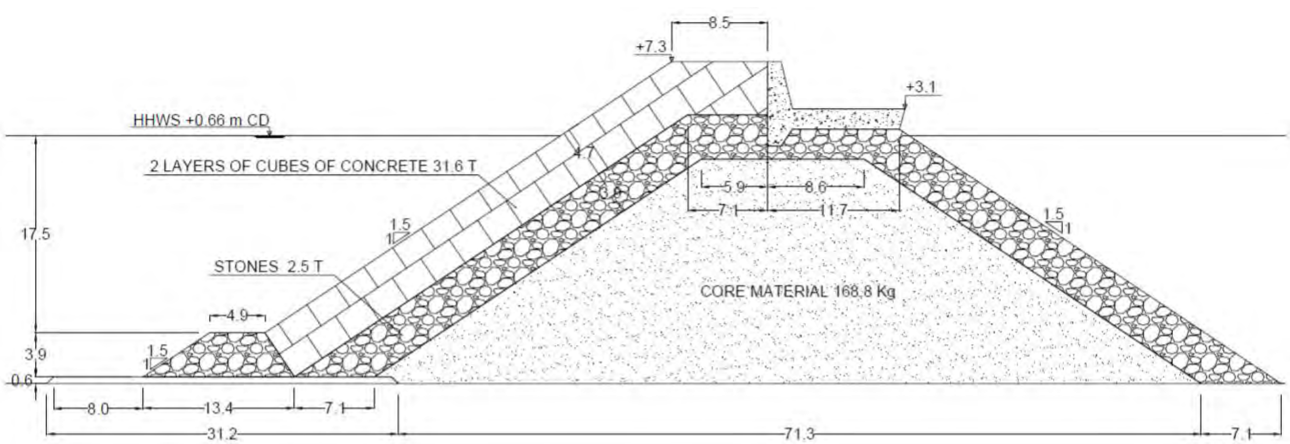


Figure 37: Cross-section of the primary rubble mound breakwater

Summary

For the breakwater, the following designs are proposed:

	weight	diameter	thickness	height	width
armour	31.6 t	2.36 m	4.72 m	-	-
underlayer	2.5 t	0.98 m	3.94 m	-	-

core	169 kg	0.4 m	-	-	-
toe	2.5 t	0.98 m	-	3.9 m	4.9 m
scour	-	2-3 cm	-	0.6 m	31.2 m

Table 52: Dimensions for the primary breakwater at the water depth of 22m

In general a global sea level rise of 0.003 m/year has to be taken into account for the structure life but in this case it's not applicated for the design of this project.

WAVE FLUME TEST

In order to know if our designed breakwater is stable a wave flume test was carried out. In the following chapters there will be explained the preparation, the carrying out and finally the obtained conclusions of the wave flume test.
Generally, scaled models are tested for the purpose of validating the expected behaviour of large structures before investing large amount of money in constructing them. The aim of this chapter is to validate the design formula used for the design of the breakwater structures by analysing datasets of overtopping, transmission and reflection during tests done in the coastal laboratory at ESITC University of Caen. Besides, the stability of the armour layer is checked by analysing its damage.

Preparation

The breakwater scaling is performed by means of scale ratios for Froude models. The scale ratio is defined as λ and is, in this project, calculated by relating prototype water depth at the breakwater and model water depth.

Geometry similarities hold a direct relation. This fact considerably simplifies the calculation of dimensions of the model. Relations for geometric, kinematic and dynamic similarities can be found in Table 53.

similarity	parameter	Froude Relation
geometric	length	λ
kinematic	time	$\sqrt{\lambda}$
dynamic	mass	λ^3

Table 53: Scale ratios for Froude models (London, 2012)

A scaling factor of λ = 75 is used in order to construct the model that is tested in the wave flume. A comparison of the prototype and the model is shown in Table 54. These values are not those that are found with the preliminary dimensioning (with Hudson formula); as there are only cubes with a length of a side of 35 mm and a weight of 75 g in the laboratory, the calculated values are increased by 20% in order to compare the model with the prototype. With the model values shown in the following table the breakwater model is built.

		prototype	model
H_s		5.0 m	6.7 cm
T_p		10.1 s	1.2 s
Water depth		22 m	29.3 cm
Height of the breakwater		28.6 m	38.1 cm
Slope		1:1.5	1:1.5
Armour layer	**M**	31.6 t	75 g
	D_{n50}	2.36 m	3.2 cm
	thickness	4.7 m	6.3 cm
underlayer	**M_{50}**	2.5 t	6.0 g

	D_{n50}	0.98 m	1.3 cm
	thickness	3.9 m	5.3 cm
core	M_{50}	0,17 t	0.4 g
	D_{n50}	0.4 m	0.5 cm

Table 54: Design values for the model

Carrying out/Procedure

The wave flume test is carried out with two different models. These two models will be described in the following.

- **Model A**

The first experiment is observed with the above described values. The finished model is shown in Figure 59 and Figure 60.

Table 55: Waveflume test model A (1)

Table 56: Waveflume test model A (2)

The simulation is run with gradual increases in H_s. This experiment starts with 60% of the significant wave height until the design H_s is reached. From 60 to 90% the structure resists wave conditions with almost no problem. There is only one overturning of a concrete block of the armour layer observed which had no interlocking with neighbouring blocks. But after this overturning the block is situated in a much more stable position and doesn't move any more. With 100% of H_s there is observed some sliding of the armour layer and one stone is up-lifted, but not removed. But all in all the structure is quite stable. With reaching 110% of H_s the overtopping increases. Taking everything into account, the breakwater toe is very stable and the damage is in the allowable range. There is no technical complete failure.

- **Model B**

Additionally a second experiment is run with only one armour layer of concrete blocks. This modification is made in order to see, if the structure is over designed. The new model is shown in Figure 0 and Figure 38.

Figure 38: Wave flume test model B

Table 57: Wave flume test model B

In the second run there is much more moving of observed. At 90% of the significant wave height there are four blocks moving a little bit. At 110% of Hs there is some sliding of the whole armour layer. Furthermore, there is one block extracted and the underlayer is also moving. In fact in this experiment there is more overtopping noticed as in the previous one. And finally there are regular waves applicated on the structure. In this case are the following damages observed: Sliding of the armour layer, concrete blocks are removed, group of blocks are extracted, extraction of the underlayer and finally rapid flow of the underlayer (see Figure 39 and Figure 40).

Figure 39: Removed concrete blocks and extraction of the underlayer

Figure 40: Before and after the test. In the second picture it can be seen the sliding of the armour layer.

Analysis

- Since there is less overtopping and less damages the breakwater with two concrete armour layers will be chosen for the construction.
- Knowing that the structure of model A resists very well with heavier cubes than calculated they will be applicated for the construction. Knowing that a model cube weighs 75 g the cube weight of the carried out cubes has to be 31.6 t and the diameter is 2.36 m.
- It is advisable to place the concrete blocks randomly to increase the porosity and to reduce much more the amount of overtopping.
- The toe of the model consists of two concrete blocks side by side and additionally rocks of the same width. The carried out consists only out of rocks.
- The damage study in the armour layer provides insight on the stability of it. Model A has no substantial damage even if more extreme wave conditions are considered. The structure is therefore found to be stable and is the proposed breakwater structure.

CONCLUSION

The primary breakwater in front of the roundhead is situated at the water depth of 22 m. By assuming that the maximal water discharge q for the overtopping is 0.02 l/s/m structure length the freeboard R_c= 6.6 m is calculated. The Hudson formula is used in order to determine the weight of the concrete blocks. For the underlayer and the core stone weights relations from the Rock Manual are used. The breakwater has in total a height of 28.6 m and the bottom length is 109.6 m. In the wave flume test, the designed breakwater is justified and afterwards some modifications are applied.

Dolphin design

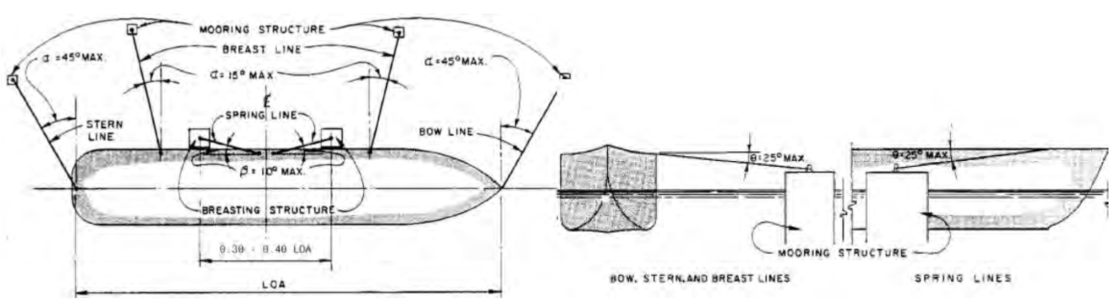


Figure 41: Example of the dolphin design (Rolland, 2017)

Moorings are designed to accommodate crude oil tankers and product tankers. These vessels have different characteristics, such as draft, length, width and capacity. When vessels berth at the terminal, there are various forces acting on the breasting structures directly. Through breast line, stern line and bow line, mooring structures are under forces as well. Therefore, structures are designed specifically in following sections.

Breasting structures

A minimum of two breasting structures are normally required at a fixed mooring. Extra dolphins may be added due to vessels. Breasting spacing from 30 to 40 percent of overall length (LOA) of the vessel are recommended. (Ligteringen & Velsink, 2012)

Following tables show the distance between moorings for vessels with minimum and maximum length, as well as deepest and heaviest vessels.

Crude oil tankers

	Capacity (m3)	Capacity (t) (DWT)	Displacement (t)	Draft (m)	LOA (m)	Breadth (m)	0.3 LOA (m)	0.4 LOA (m)	Distance Between Outer Berthing Dolphins
Deepest	329,882	296,894.00	342,442.00	22.20	274.20	58.00	82.26	109.68	106
Longest	172,329	155,096.00	205,705.00	16.00	345.00	48.00	103.50	138.00	106
Heaviest	333,332	299,999.00	341,550.00	21.50	269.10	60.00	80.73	107.64	106
Shortest	333,332	299,999.00	339.39	21.30	269.20	60.00	80.76	107.68	106
									Need 2 breasting structures

Table 58: Distance between breast dolphins for crude oil tankers

Product tankers

	Capacity (m3)	Capacity (t) (DWT)	Draft (m)	LOA (m)	Breadth (m)	0.3 LOA (m)	0.4 LOA (m)	Distance Between Outer Berthing Dolphins
Deepest	56,764	51,088	13.54	183.0	32.00	54.90	73.20	70
Longest	44,463	40,017	13.17	228.6	32.00	68.58	91.44	70
Heaviest	59,097	53,187	13.50	183.20	32.30	54.96	73.28	70
Shortest	16,683	15,015	9.27	129.	22.00	38.70	51.60	46

				00				

Need 4 breasting structures

Table 59: Distance between breast dolphins for product tankers

In summary, considering different tankers, 6 breasting structures are needed. The number of mooring dolphins is determined according to the requirement of angles.

Mooring structures

Mooring structures should be placed symmetrically about the transverse centerline of the mooring in order to obtain a balanced distribution of mooring load. Mooring dolphins are positioned behind the breasting dolphins at a distance of 35 to 50m (Ligteringen & Velsink, 2012). Forty meters are assumed in our case. The requirements of angles of lines are given (Ligteringen & Velsink, 2012). The distance between mooring dolphins is assumed first. The distance can be found in **Fout! Verwijzingsbron niet gevonden.**. Then, the angles of mooring lines can be checked.

Four kinds of vessels are used for checking the horizontal angles in the following table:

	Capacity(dwt)	**Draft (m)**	**LOA (m)**	**Breadth (m)**	**Stern line**	**Breast line**	**Sping line**	**Breast line**	**Bow line**
Shortest	15,015	*9.27*	129	22	38.5	7.8	10	7.8	38.5
Longest	155,096.00	*16*	345	48	2.2	46.7	4.4	46.7	2.2
Heaviest	299,999.00	*21.5*	269.1	60	22.9	56.1	4.4	56.1	22.9
Deepest	296,894.00	*22.2*	274.2	58	24.9	57.2	4.4	57.2	24.9
Recommended value(to vertical axis)					<45	<75	<10	<75	<45

Table 60 Angles of mooring lines

From the above table, the angles of lines at mooring dolphins can meet the requirements. There are two lines for each dolphin. In the view of safety, quick release hook is used.

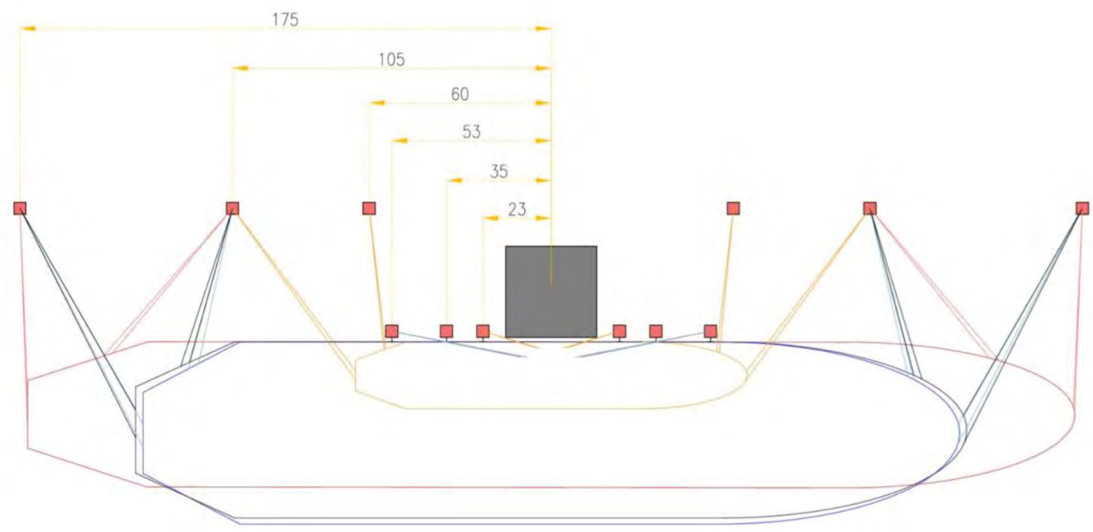


Figure 42: Mooring design

Determination of the breasting dolphin pile

In this section, the detailed design of breasting dolphin pile is given. The length of pile is related to the water depth. The diameter of pile is determined according to the forces on the dolphins through fenders.

The length of pile

The dolphin is going to be built near the primary breakwater (see layout). The local water depth is 22 meters. The level of top is assumed to be 1.5 m above the water level. So, L= 23.5 meters. 2.5D is estimated to be under the seabed.

Determination of the diameters of piles

Circular steel piles are used for mooring and breast dolphins. Three formulas are needed to check the compression and bending of piles. (Rolland, 2017)

$$\frac{f_a}{F_a} + \frac{C_m\sqrt{{f_{bx}}^2 + {f_{by}}^2}}{\left(1 - \frac{f_a}{F_e{}'}\right)F_b} \leq 1.0$$

$$\frac{f_a}{0.6F_y} + \frac{\sqrt{{f_{bx}}^2 + {f_{by}}^2}}{F_b} \leq 1.0$$

$$\frac{f_a}{F_a}+\frac{\sqrt{f_{bx}{}^2+f_{by}{}^2}_b}{F_b}\leq 1.0\ \ when\ \frac{f_a}{F_a}\leq 0.15$$

For the parameters in above equations, following processes are followed.

$$f_a=\frac{N}{A}$$
$$N=\rho\cdot V\cdot g=24\cdot D^2$$

$$A=A=\frac{\pi}{4}*(D^2-(D-2t)^2)$$

$$f_b=\frac{M}{W}$$
$$M=F\cdot d=4543.6 \qquad W=\frac{I}{D/2}\quad I=\frac{\pi}{64}*(D^4-(D-2t)^4)$$

$$F_y=min(F_{xc},F_{xe})$$
$$F_{xe}=2CE\frac{t}{D}$$

$F_{xc}=F_y$ or

$$F_{xc}=F_y*\left(1.64-0.23*\left(\frac{D}{t}\right)^{1/4}\right)\leq F_{xe}$$

$F_a=\dfrac{\left(1-\frac{(KL/r)^2}{2C_c{}^2}\right)F_y}{\frac{5}{3}+\frac{3(KL/r)}{8C_c}-\frac{(KL/r)^3}{8C_c{}^3}}$ for $KL/r<C_c$

$F_a=F_e'=\dfrac{12\pi^2E}{23(KL/r)^2}$ for $KL/r\geq C_c$

$$C_c=\left(\frac{2\pi^2E}{F_y}\right)^{1/2}$$

Factor K is taken as 2.1 (Rotation free and translation free) $r=\sqrt{\frac{I}{A}}$

- $F_b=0.75F_y$

$$F_b = \left(0.84 - 1.74\frac{F_y D}{Et}\right) F_y$$
$$F_b = \left(0.72 - 0.58\frac{F_y D}{Et}\right) F_y$$

By intergration in matlab program, the results are obtained in the following graph. The limitation condition is the black soild line. Above this line, conditons meet the requiement. Oppositely, area below the black line can not meet the requirement of compression and bending. In order to minimize the space, 1 m can be taken as the outer diameter of the piles. The thickness is about 0.03m. The inner diamerter is around 0.07m.

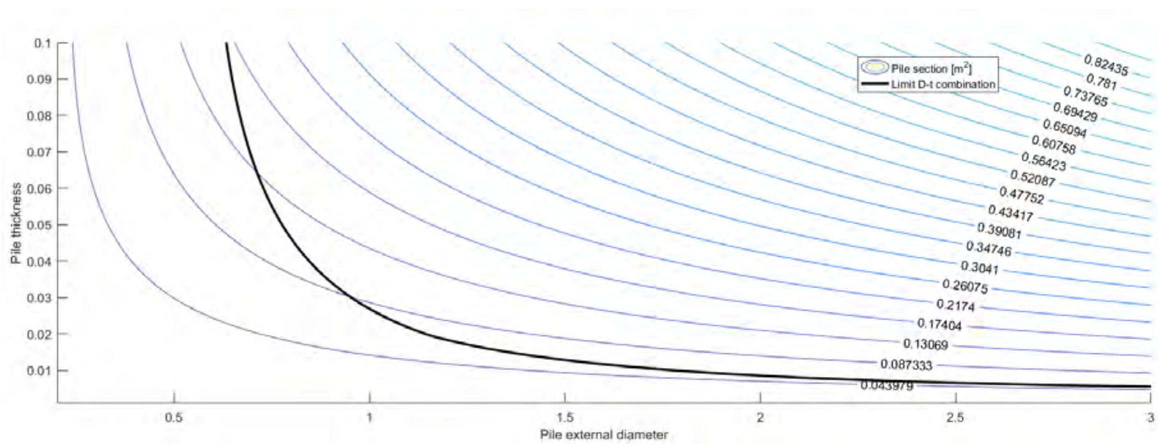


Figure 61: Results of integration

Aid system

Aid system contains berthing aid system and mooring aid system.

In berthing aid system, two lasers are installed on each side of the loading platform. A display board is used for pilot. A computer workstation in control room for display record the information.

In mooring aid system, load cell is installed in the hook, and the Line tension monitoring is used.

Energy absorbed by Fenders on breasting dolphin

The total energy absorbed by the fender is determined using the Kinetic theory method used in the PIANC 2002 guidelines and other Port design books. It is a function of the component of the ship's approach velocity perpendicular to the berthing line, the displacement of the ship, and some empirical factors. These factors are: the added mass coefficient, eccentricity coefficient, berth configuration coefficient, and the softness coefficient. However, due to safety requirements, a factor of safety considering abnormal berthing scenarios is multiplied with the absorbed energy to give the total energy used in the selection and design of fender units.

$$E_A = F_S * E_N$$

Where:

E_A = the abnormal energy to be absorbed by the fender

F_S = the safety factor considering extreme/abnormal berthing scenarios
E_N = the total energy absorbed by the fenders using the kinetic energy method

The Kinetic Energy Method

This method simply described the energy transferred by the ship to the berthing structure due to the motion of the ship before and during berthing. In principle, the kinetic energy exerted on the berthing structure will be absorbed and/or reflected to the ship's hull and/or transmitted to the berthing structure, by the fenders. The complexity of the process described in the previous sentence is dependent on the type of fender unit chosen (hard or soft). The formula below is used to determine the normal energy absorbed by the fenders:

$$E_N = 0.5 * M * {V_B}^2$$

Where:
E_N = kinetic energy in kNm
M = virtual mass in tons, which is the sum of the ship displacement (M_d and the hydrodynamic mass M_h
V_B = the component of the ship's approach velocity perpendicular to the berthing line in $^m/_s$
M_d = the mass of the design ship or the displacement (fully loaded ships) in tons
M_h the additional mass of water pressing the ship against the berth

For simplicity, the ship with the largest displacement (expected at certain berth within the harbour) will be used as the design ship in these calculations, and the displacement value obtained from the PIANC guidelines and the catalogue given in this project is assumed to be the total virtual mass of the ship.

The Approach Velocity (V)

As ships approach the berth, the speed at which they entered the approach channel slowly decreases, whether they are aided by tugboats (or not) while manoeuvring. The approach velocity is defined as the speed of the ship at the initial point of berthing contact. For reasons of safety and to reduce the probability of damage to the fender systems, PIANC recommends that when designing fender systems for larger ships the berthing velocities with use of tugboat assistance should not be less than:

Berthing Conditions	Values	Units
Very favourable conditions	0.1	m/s
In most cases	0.15	m/s
Very unfavourable conditions with cross-current and/or much wind 25 cm/s.	0.25	m/s

Table 62: PIANC's Recommendation for the approach velocity of the vessel during berthing

However, most or all ships berth at an angle to the berthing line, and the resultant of the approach velocity corresponds to the approach angle at which the ship berths. The resultant velocity has both horizontal and vertical components, the latter of which is used in the determination of the total energy absorbed by the fender. This vertical component is aligned perpendicular to the berthing line, and is found by multiplying the approach velocity by the sine of the sum of the angles it forms with the berthing line.

$$V_B = V * \sin(\phi + \alpha + asin\frac{B}{2R})$$

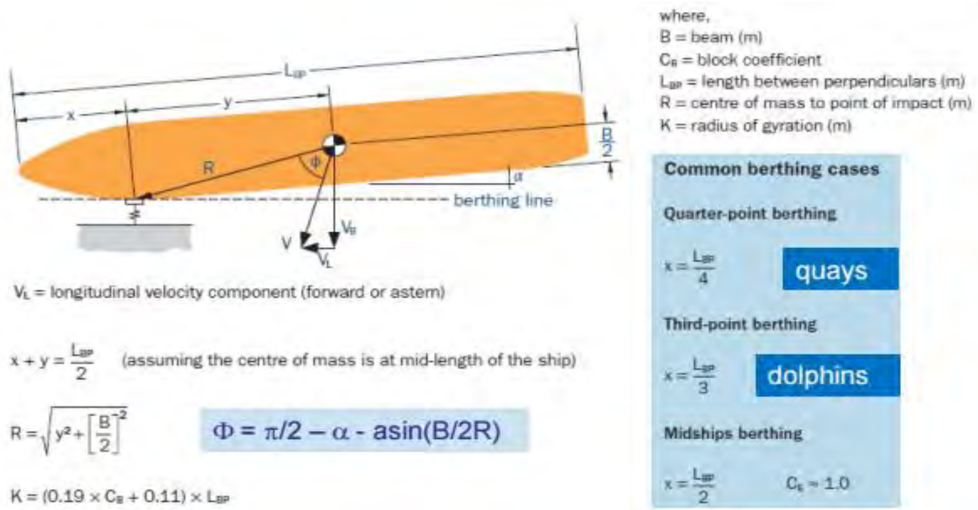


Figure 43: Slides taken from SAIPEN/ESITC Caen (Fixed mooring-structure design presentation). (ROLLAND, 2017)

Berthing Coefficient

However, an adjustment factor or berthing coefficient added to the kinetic energy formula to account for the different hydrodynamic processes occurring before and during berthing. This added coefficient is a sum of other coefficients which are discussed below.

$$E_N = 0.5 * M * {V_B}^2 * C$$

Where:

C = the sum of the coefficients ($C = C_M * C_E * C_C * C_S$)
C_M = the hydrodynamic mass coefficient
C_E = the eccentricity coefficient
C_C = the berthing configuration coefficient
C_S = the softening coefficient

7. *The Hydrodynamic Mass Coefficient* (C_M)

This parameter allows the movement of water around the ship to be taken into account when we calculate the total energy of the vessel by increasing the mass of the system. It can be calculated from the following equation provided in the British Standards (BSI, 2014):

$$C_M = 1 + \frac{2D}{B}$$

Where:

D = the displacement of the ship in tons
B = the ship's Beam in m

The hydrodynamic mass coefficient can also be determined using PIANC's guidelines (PIANC Working Group - 33, 2002), using the tale below:

PIANC (2002)		Shigera Ueda (1981)	Vasco Costa* (1964)
for $\frac{K_c}{D} \leq 0.1$	$C_M = 1.8$	$C_M = \frac{\pi \times D}{2 \times C_B \times B}$	$C_M = 1 + \frac{2D}{B}$
for $0.1 \leq \frac{K_c}{D} \leq 0.5$	$C_M = 1.875 - 0.75\left[\frac{K_c}{D}\right]$		
for $\frac{K_c}{D} \geq 0.5$	$C_M = 1.5$		

* valid where $V_B \geq 0.08$m/s, $K_c \geq 0.1D$

where,
D = draft of vessel (m)
B = beam of vessel (m)
L_{BP} = length between perpendiculars (m)
K_c = under keel clearance (m)

Figure 44: PIANC Working Group Recommendation for calculating the hydrodynamic mass coefficient. Slides taken from SAIPEN/ESITC Caen (Fixed mooring-structure design presentation). (ROLLAND, 2017)

The eccentricity factor is due to the consideration of the energy dissipation which arises from the rotational motion after berthing around the contact point at either the bow or at the stern. (Thorensen, 2014)

$$C_E = \frac{K^2 + R^2 cos^2 \phi}{K^2 + R^2}$$

Where:
K = the ship's radius of gyration
R = the distance of point of contact from the centre of mass
ϕ = the angle shown in figure 1

If the angle between R and V is 90 degrees, the equation becomes:

$$C_E = \frac{1}{1 + \frac{R^2}{K^2}}$$

The radius of gyration of the ship calculated from this formula:

$$K = (0.19C_B + 0.11)L$$

Where:
L = the length of the hull between perpendiculars
C_B = the block coefficient estimated from the table below

Table 3 — Typical ranges of C_b

Vessel type	Range of C_b
Tanker/bulk	0.72 to 0.85
Container	0.65 to 0.70
Ro-Ro	0.65 to 0.70
Passenger	0.65 to 0.70
Dry cargo/combi	0.60 to 0.75
Ferry	0.50 to 0.65

Table 63: Typical range of Block Coefficient Values (BSI, 2014)

8. *Berthing Configuration Coefficient* (C_C)

Also called the water cushioning effect, the berthing configuration coefficient accounts for water pressure generated between the hull of the ship and the berthing structure. This occurs in the case of solid berth structures; the water pressure reduces the energy exerted by the ship on the berth structure. For open berth structures, the value is one as the water find its way between the vertical supports, instead of building up. The table below gives the factors to be used in different scenarios.

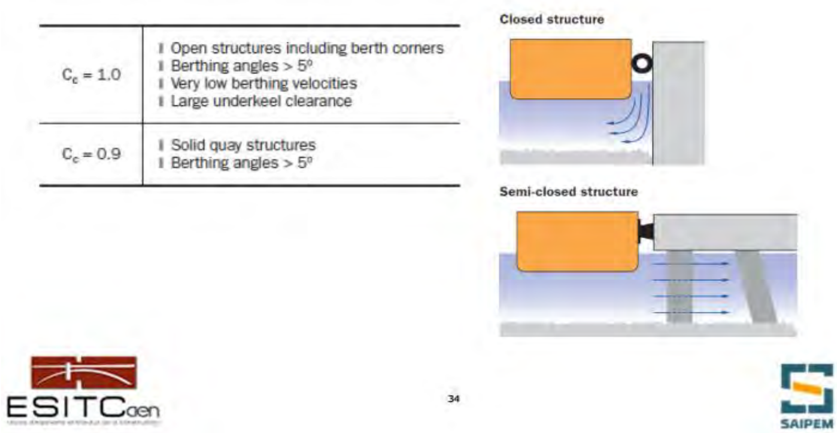


Figure 45: Estimation of Berthing Configuration Coefficient Values from PIANC. Slides taken from SAIPEN/ESITC Caen (Fixed mooring-structure design presentation). (ROLLAND, 2017)

9. *The Softening Coefficient* (C_S)

This factor is determined by the ratio between the elasticity and/or the flexibility of the ship's hull and that of the fender system or berth structure. Therefore, part of the berthing kinetic energy will be absorbed by elastic deformation of the ship's hull and/or flexibility of the berth structure. For a small ship C_S is generally taken to be 1.0. For hard fenders and larger ships (e.g. large tankers or flexible wood piers) C_S is 0.9–1.0. (Thorensen, 2014)

$C_S = 1.0$	Soft fenders ($\delta_f > 150mm$)
$C_S = 0.9$	Hard fenders ($\delta_f \leq 150mm$)

Figure 46: Estimation of Softness Coefficient Values from PIANC. Slides taken from SAIPEN/ESITC Caen (Fixed mooring-structure design presentation) (ROLLAND, 2017)

The Total Energy absorbed by the Fenders (E_N)

After determining the coefficients describing the processes during berthing (from the initial point of impact between the ship's hull and the fenders up to the time the ship leaves the berth structure), the total energy absorbed by the fenders can be calculated using the expanded kinetic energy formula below:

$$E_N = 0.5 * M * {V_B}^2 * C_M * C_E * C_C * C_S$$

Safety Factor (F_s)

The Factor of Safety takes into consideration the abnormal impacts transferred to the berth structure by the ship during berthing. In the below table are values recommended by PIANC.

PIANC Factors of Safety (F_S)

Vessel type	Size	F_S
Tanker, bulk, cargo	Largest Smallest	1.25 1.75
Container	Largest Smallest	1.5 2.0
General cargo		1.75
RoRo, ferries		≥2.0
Tugs, workboats, etc		2.0

Source: PIANC 2002; Table 4.2.5.

Table 64: PIANC's recommendation for Safety Factor values. Slides taken from SAIPEN/ESITC Caen (Fixed mooring-structure design presentation. (ROLLAND, 2017)

The Total Abnormal Energy absorbed by the Fenders then becomes the product of the absorbed energy and the safety factor.

$$E_A = F_S * E_N$$

NB: For this project, the total energy exerted on the breasting dolphin by the Crude Oil Tanker exceeded the values listed in the catalogue provided by Trelleborg Marine Systems. Therefore, it was decided that 2 fenders be placed closed to each other for simplicity purposes, and to obtain a reaction value which enables the calculation of vertical load on the pile of the mooring dolphin.

In practice, the Client or Port owner may request that a custom fender is designed and manufactured.

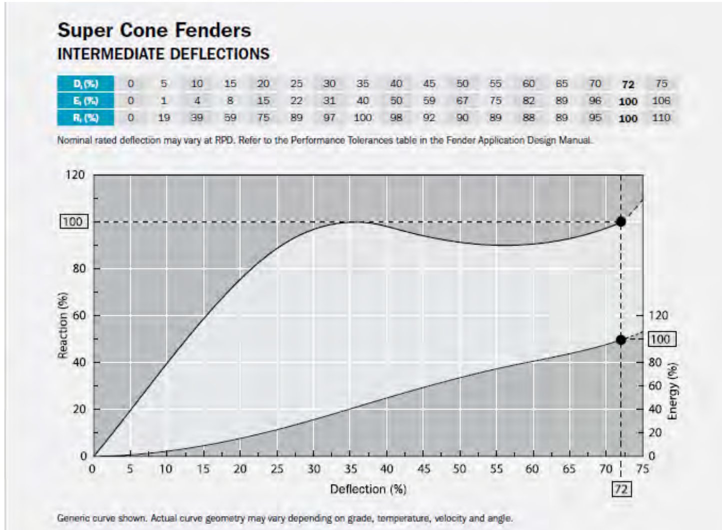
Results

Properties of Design Ships								
		Value						
Description	Symbol	Tanker	Container Ship (Large)	Bulk	RO-RO	General Cargo	Unit
Dead-weight	DWT	298033	70000	174505	11089	17500	tons
Displacement	Displ	344175	100000	213200	21552	20750	tons
Overall Length	LOA	334	280	282	197	143	m
Length between perps	Lpp	330	266	271	187	133	m
Beam	B	59	42	45	26	22	m
Draught	D	22	14	19	8	13	m
Approach Velocity	V	0	0	0	0	0	m/s
Block Coefficient	CB	1	1	1	1	1	(-)
Added Mass Coefficient	CM	2	2	1	2	2	(-)
Coefficient of Eccentricity	CE	1	1	1	1	1	(-)
Berth Configuration Coefficient	Cc	1	1	1	1	1	(-)
Softness Coefficient	Cs	1	1	1	1	1	(-)
Energy absorbed by the fender	EN	10095	2919	3529	654	616	Knm
Safety Factors	Fs	1	2	1	2	2	(-)
Abnormal Energy absorbed by the fender	EA	12619	4379	4411	1308	1077	Knm
Reduced Abnormal energy assuming 2 fenders placed closed to each other	REA	6309				539	Knm

Table 65: Results displaying the total energy to be absorbed by the fenders.

Fender Reaction

To obtain the reaction from the fender, a catalogue provided by Trelleborg Marine Systems (Trelleborg AB, 2017) can be used. The relationship between the total abnormal absorbed energy of the fender unit and the reaction value is a function of the expected deflection based on the type of fender chosen. The figure below shows a representation of this relationship.



Super Cone Fenders

INTERMEDIATE DEFLECTIONS

D_i (%)	0	5	10	15	20	25	30	35	40	45	50	55	60	65	70	**72**	75
E_i (%)	0	1	4	8	15	22	31	40	50	59	67	75	82	89	96	**100**	106
R_i (%)	0	19	39	59	75	89	97	100	98	92	90	89	88	89	95	**100**	110

Nominal rated deflection may vary at RPD. Refer to the Performance Tolerances table in the Fender Application Design Manual.

Generic curve shown. Actual curve geometry may vary depending on grade, temperature, velocity and angle.

Figure 47: Relationship between the percentages of energy absorbed and the subsequent reaction (Trelleborg AB, 2017)

		E0.9	E1.0	E1.1	E1.2	E1.3	E1.4	E1.5	E1.6	E1.7	E1.8	E1.9	E2.0
SCN 300	E_R	7.7	8.6	8.9	9.2	9.5	9.8	10.1	10.4	10.6	10.9	11.2	11.5
	R_R	59	65	67	68	70	72	74	75	77	79	80	82
SCN 350	E_R	12.5	13.9	14.4	14.8	15.3	15.7	16.2	16.7	17.1	17.6	18	18.5
	R_R	80	89	91	93	96	98	100	102	104	107	109	111
SCN 400	E_R	18.6	20.7	21.4	22.1	22.8	23.5	24.2	24.8	25.5	26.2	26.9	27.6
	R_R	104	116	119	122	125	128	131	133	136	139	142	145
SCN 500	E_R	36.5	40.5	41.9	43.2	44.6	45.9	47.3	48.6	50	51.3	52.7	54
	R_R	164	182	187	191	196	200	205	209	214	218	223	227
SCN 550	E_R	49	54	56	58	59	61	63	65	67	68	70	72
	R_R	198	220	226	231	237	242	248	253	259	264	270	275
SCN 600	E_R	63	70	72	74	76	78	80	82	84	86	88	90
	R_R	225	250	257	263	270	276	283	289	296	302	309	315
SCN 700	E_R	117	130	134	137	141	144	148	151	155	158	162	165
	R_R	320	355	365	374	384	393	403	412	422	431	441	450
SCN 800	E_R	171	190	196	201	207	212	218	223	229	234	240	245
	R_R	419	465	478	490	503	515	528	540	553	565	578	590
SCN 900	E_R	248	275	282	289	296	303	310	317	324	331	338	345
	R_R	527	585	601	617	633	649	665	681	697	713	729	745
SCN 950	E_R	291	322	331	339	348	356	364	373	381	390	398	407
	R_R	588	653	671	688	706	724	742	759	777	795	813	830
SCN 1000	E_R	338	375	385	395	405	415	425	435	445	455	465	475
	R_R	653	725	745	764	784	803	823	842	862	881	901	920
SCN 1050	E_R	392	435	447	458	470	481	493	504	516	527	539	550
	R_R	720	800	822	843	865	886	908	929	951	972	994	1015
SCN 1100	E_R	450	500	514	527	541	554	568	581	595	608	622	635
	R_R	788	875	899	923	947	971	995	1019	1043	1067	1091	1115
SCN 1200	E_R	585	650	668	685	703	720	738	755	773	790	808	825
	R_R	941	1045	1073	1101	1129	1157	1185	1213	1241	1269	1297	1325
SCN 1300	E_R	743	825	847	869	891	913	935	957	979	1001	1023	1045
	R_R	1103	1225	1258	1291	1324	1357	1390	1423	1456	1489	1522	1555
SCN 1400	E_R	927	1030	1058	1085	1113	1140	1168	1195	1223	1250	1278	1305
	R_R	1278	1420	1459	1497	1536	1574	1613	1651	1690	1728	1767	1805
SCN 1600	E_R	1382	1535	1577	1618	1660	1701	1743	1784	1826	1867	1909	1950
	R_R	1670	1855	1905	1955	2005	2055	2105	2155	2205	2255	2305	2355
SCN 1800	E_R	1967	2185	2244	2303	2362	2421	2480	2539	2598	2657	2716	2775
	R_R	2115	2350	2413	2476	2539	2602	2665	2728	2791	2854	2917	2980
SCN 2000	E_R	2700	3000	3080	3160	3240	3320	3400	3480	3560	3640	3720	3800
	R_R	2610	2900	2978	3056	3134	3212	3290	3368	3446	3524	3602	3680

*in accordance with PIANC. [Units: kNm, kN]

Table 66: Catalogue of the absorbed energy and their corresponding reaction (Trelleborg AB, 2017)

		E2.1	E2.2	E2.3	E2.4	E2.5	E2.6	E2.7	E2.8	E2.9	E3.0	E3.1	E/R (ε)
SCN 300	E_R	11.8	12.1	12.4	12.7	13.0	13.3	13.5	13.8	14.1	14.4	15.9	0.138
	R_R	84	86	89	91	93	95	97	100	102	104	114	
SCN 350	E_R	19	19.4	19.9	20.3	20.8	21.3	21.7	22.2	22.6	23.1	25.4	0.163
	R_R	114	117	120	123	126	129	132	135	138	141	155	
SCN 400	E_R	28.3	29	29.7	30.4	31.1	31.8	32.5	33.2	33.9	34.6	38.1	0.186
	R_R	149	153	157	161	165	169	173	177	181	185	204	
SCN 500	E_R	55.4	56.7	58.1	59.4	60.8	62.2	63.5	64.9	66.2	67.6	74.4	0.232
	R_R	233	239	246	252	258	264	270	277	283	289	318	
SCN 550	E_R	74	76	77	79	81	83	85	86	88	90	99	0.256
	R_R	283	290	298	305	313	320	328	335	343	350	385	
SCN 600	E_R	93	96	99	102	105	108	111	114	117	120	132	0.290
	R_R	324	332	341	349	358	366	375	383	392	400	440	
SCN 700	E_R	169	173	177	181	185	189	193	197	201	205	226	0.364
	R_R	462	474	486	498	510	522	534	546	558	570	627	
SCN 800	E_R	252	258	265	271	278	284	291	297	304	310	341	0.414
	R_R	606	621	637	652	668	683	699	714	730	745	820	
SCN 900	E_R	355	364	374	383	393	402	412	421	431	440	484	0.466
	R_R	765	785	805	825	845	865	885	905	925	945	1040	
SCN 950	E_R	418	429	440	451	463	473	485	496	507	518	570	0.492
	R_R	853	875	897	919	941	963	986	1008	1030	1052	1158	
SCN 1000	E_R	488	501	514	527	540	553	566	579	592	605	666	0.518
	R_R	945	969	994	1018	1043	1067	1092	1116	1141	1165	1282	
SCN 1050	E_R	565	580	595	610	625	640	655	670	685	700	770	0.544
	R_R	1042	1069	1096	1123	1150	1177	1204	1231	1258	1285	1414	
SCN 1100	E_R	652	669	686	703	720	737	754	771	788	805	886	0.571
	R_R	1145	1174	1204	1233	1263	1292	1322	1351	1381	1410	1551	
SCN 1200	E_R	847	869	891	913	935	957	979	1001	1023	1045	1150	0.622
	R_R	1361	1396	1432	1467	1503	1538	1574	1609	1645	1680	1848	
SCN 1300	E_R	1074	1102	1131	1159	1188	1216	1245	1273	1302	1330	1463	0.674
	R_R	1597	1638	1680	1721	1763	1804	1846	1887	1929	1970	2167	
SCN 1400	E_R	1341	1376	1412	1447	1483	1518	1554	1589	1625	1660	1826	0.725
	R_R	1853	1901	1949	1997	2045	2093	2141	2189	2237	2285	2514	
SCN 1600	E_R	2003	2056	2109	2162	2215	2268	2321	2374	2427	2480	2728	0.830
	R_R	2418	2480	2543	2605	2668	2730	2793	2855	2918	2980	3278	
SCN 1800	E_R	2851	2926	3002	3077	3153	3228	3304	3379	3455	3530	3883	0.932
	R_R	3060	3139	3219	3298	3378	3457	3537	3616	3696	3775	4153	
SCN 2000	E_R	3904	4008	4112	4216	4320	4424	4528	4632	4736	4840	5324	1.039
	R_R	3778	3876	3974	4072	4170	4268	4366	4464	4562	4660	5126	

*in accordance with PIANC. [Units: kNm, kN]

Table 67: Catalogue of the absorbed energy and their corresponding reaction (Trelleborg AB, 2017)

However, if the calculated energy is more than the values in the catalogue (available fenders on the market), the Client or Port owner can request for a custom design of the required fender. In this project, the displacement of the design ship chosen (a tanker) for the jetty/dolphin exerts an energy on the fender which cannot be found in the catalogues from Trelleborg.

Type of fenders

There are different types of fenders, and determining which fender is the most appropriate depends on parameters such as the berthing energy, subsequent fender reaction, and performance curve.

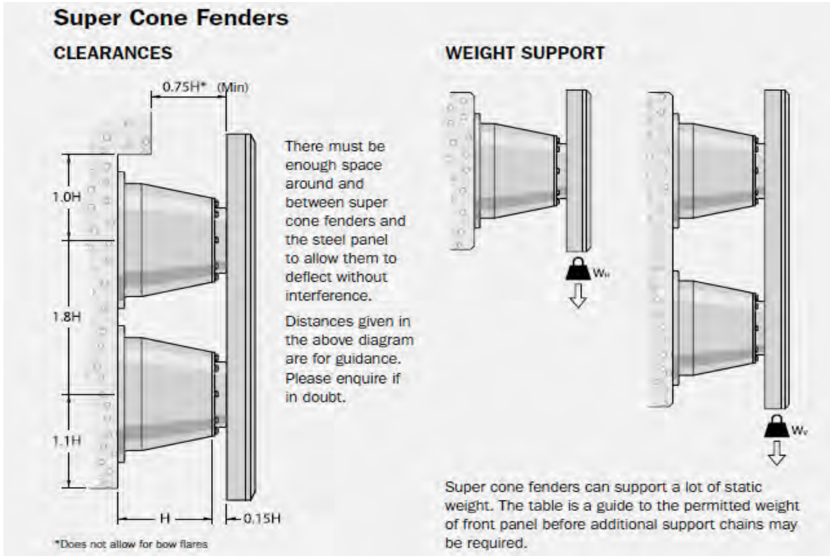


Figure 48:Basic dimensions of Super Cone Fenders (Trelleborg AB, 2017)

According to the figure above, the formula for determining the height of the fender panel is:

$$H_{TOT} = 1.0 * H + 1.8 * H + .1 * H$$

Where:

H	=	the distance between the berthing structure and the fender panel (m)
H_{TOT}	=	the total height of the fender panel, excluding bevels (m)

The width of the panel is calculated using the formula below:

$$P = \frac{R}{W * H}$$

Where:

P	=	the average hull pressure (kN/m²)
W	=	the panel width, excluding bevels (m)
R	=	the total fender reaction (kn)
H	=	the panel height, excluding bevels (m)

Hull Pressures and Beltings

HULL PRESSURES

Allowable hull pressures depend on hull plate thickness and frame spacing. These vary according to the type of ship. Refer to the table on the right for PIANC's guidelines on hull pressures.

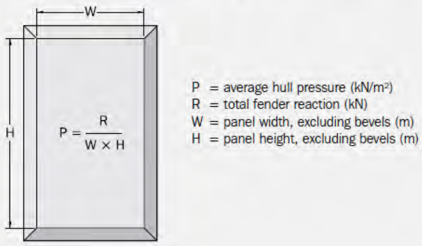


Figure 49: Hull pressures (Trelleborg AB , 2016) p.48

VESSEL TYPE	SIZE/CLASS	HULL PRESSURE (kN/m²)
Container ships	< 1,000 teu (1st/2nd generation)	< 400
	< 3,000 teu (3rd generation)	< 300
	< 8,000 teu (4th generation)	< 250
	> 8,000 teu (5th/6th generation)	< 200
General cargo	≤ 20,000 DWT	400–700
	> 20,000 DWT	< 400
Oil tankers	≤ 20,000 DWT	< 250
	≤ 60,000 DWT	< 300
	> 60,000 DWT	150–200
Gas carriers	LNG/LPG	< 200
Bulk carriers		< 200
RoRo		Usually fitted with beltings (strakes)
Passenger/cruise		
SWATH		

Source: PIANC 2002; Table 4.4.1

The average hull pressure can be found using the table below:

Table 68:Trelleborg's table for determining the hull pressures for different kinds of vessels

Results of Fender Dimensions

Fender Properties and Dimensions							
Description	Symbols	Values					
		Oil tanker **	Container Ships (Large)	Bulk carrier	RO-RO *	General Cargo	Units
Size/Class	DWT	298033	70000	174505	11089	17500	tons
Abnormal energy absorbed by the fenders	EA	6309.5	4379.0	4411.0	1308.0	1077.1	kN.m
Abnormal absorbed fender energy chosen from	Eac	6458.6	4469	4469	1319.8	1091.1	

catalogue							
Reaction fender (RPD - soft fenders)	R	5137.7	3969.7	3969.7	1831.5	1673.6	kN
Type of super cone fenders chosen	F	2.5	2.4	2.4	2.6	2.8	(-)
Distance between panel fender and berth structure	H	2.3	2.0	2.0	1.3	1.2	m
Height of fender panel	Htot	8.8	7.8	7.8	5.1	4.7	m
Hull pressure on the fender	P	150	200	200	Usually fitted with beltings	400	kN/m^2
Width of the fender panel	W	3.9	2.5	2.5	Use values for General Cargo	0.9	m

** for 2 fenders

Quay Wall

Definition

Quay wall are mainly used for vessels 'berthing and also to have a space where trucks and cranes can approach the vessels.

Theory method

In this project a composite wall piles and sheet piles will be used. The review has to verify the risk of erosion and siphoning feet to protect the only constraint that this piece has. The problem to solve is statically determined thanks to 2 unknowns (embedment depth D and d nil) and 2 equations of equilibrium (the horizontal translation and rotation). It will be necessary to use tie rods because the height of the work is very important.

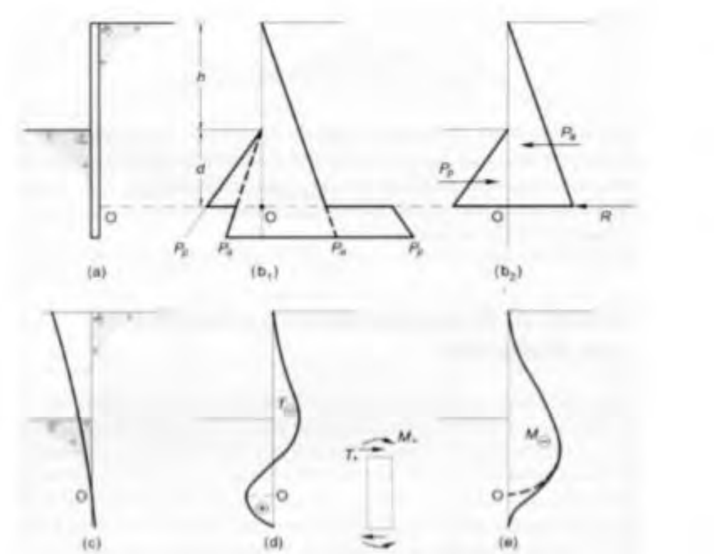


Figure 50: Blum method (Lo Presti, 2016)

Calculation principle

The software allows the modelling of the quay wall. The rigidity matrix used to calculate the model contains:

- beam elements that represent the wall
- springs that simulate the soil in an elastic phase
- external links

Thus, K-REA calculates the internal forces and the deformations of a retaining wall through a number of calculation phases, as well as the external forces including the soil reactions and the external links.

Soil data

In this part, the geotechnical data given in the previous section will be used in the software. The modeling of the quay wall will be done in the Section 3 (refer to the previous part about geotechnics data).For each wall, the geotechnical data of each type of soil present in the section 3 will be entered. In the following tables, these data are represented.

Wall 1								
NAME	z	φ (°)	c (kPa)	ϒ (kN/m3)	ϒ' (kN/m3)	δa/φ	δp/φ	kh
Fine Sand	-5,00	**33,00**	**0,00**	**19,00**	**9,00**	**0,00**	**-0,66**	**47736,00**
Core sand and alluvions	**-19,00**	**35,00**	**0,00**	**19,00**	**9,00**	**0,00**	**-0,66**	**14609,00**
Sand 3	**-30,00**	**35,00**	**0,00**	**20,00**	**10,00**	**0,00**	**-0,66**	**34460,00**
Marne	**-32,00**	**35,00**	**50,00**	**19,00**	**9,00**	**0,00**	**-0,66**	**21192,00**

Table 69

Wall 2								
NAME	z	φ (°)	c (kPa)	ϒ (kN/m3)	ϒ' (kN/m3)	δa/φ	δp/φ	kh
Fine Sand	**2,93**	**33,00**	**0,00**	**19,00**	**9,00**	**0,00**	**-0,66**	**47736,00**
Core sand and alluvions	**-0,57**	**35,00**	**0,00**	**19,00**	**9,00**	**0,00**	**-0,66**	**14609,00**
Sand 3	**-15,57**	**35,00**	**0,00**	**20,00**	**10,00**	**0,00**	**-0,66**	**34460,00**
Marne	**-24,37**	**35,00**	**50,00**	**19,00**	**9,00**	**0,00**	**-0,66**	**21192,00**

Table 70

Retaining wall

In this part, the information about the composition of the walls:

Pile in the Wall 1:

E(Kn/m^2)	D (mm)	t (mm)	L (mm)	Horizontal space (mm)
2.1 * 10^8	2020	20	29000	5100

Table 71

Pile in the Wall 2:

E(Kn/m^2)	D (mm)	t (mm)	L (mm)	Horizontal space (mm)
2.1 * 10^8	2020	20	24000	5100

Table 72

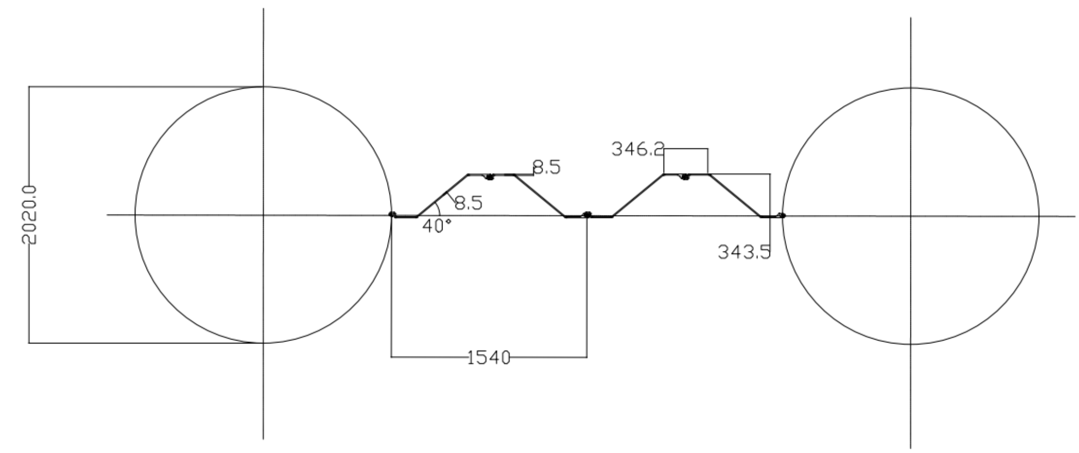


Figure 51: Composite Wall 1 - Pile and sheet pile section AZ 12-770

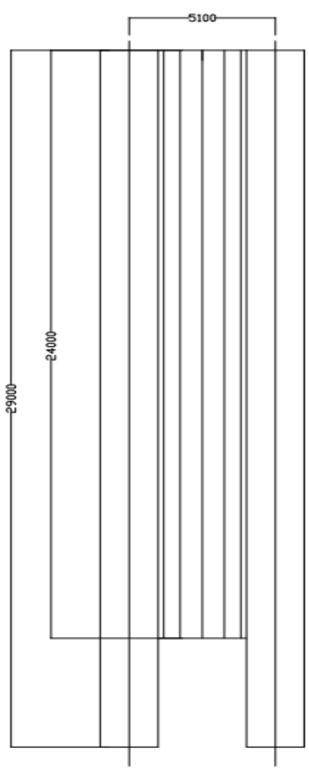


Figure 52: Section of the composite wall 1

Loads

In the scope of word it is suggested to use the following data: "For the quay, dimensioning factors are berthing energy to fenders (spacing = 20 m) and vertical loads + loads on bollards, for which the following values may be considered.

- MOF harbour for zone C
  - Vertical load = 4 tons/m2 on all the surface
  - Dynamic loads on bollards : 50 tons each, spacing 20m
- Container quay
  - Vertical load = 6 tons/m2
  - Vertical load of crane =
    - Weight = 2 500 tons
    - Linear load max on one rail = 100 tons/m (along 10m)
    - Rails spacing = 30 m
- Dynamic loads on bollards : 150 tons each, spacing 20m "

Indeed, to have the better conditions on the quay it is necessary to define these loads:

Bollards loads: P=25 kN

Table 12.1 Bollard load *P* and approximate spacing

Ships with a displacement up to: tonnes	Bollard load *P*: kN	Approximate spacing between bollards: m	Bollard load normal from the berth: kN/m berth	Bollard load along the berth: kN/m
2 000	100	10	15	10
5 000	200	15	15	10
10 000	300	20	20	15
20 000	500	20	25	20
30 000	600	25	30	20
50 000	800	25	35	20
100 000	1000	30	40	25
200 000	1500	30	50	30
>200 000	2000	35	65	40

Figure 53: Bollard load P and approximate spacing (Thoresen, 2014)

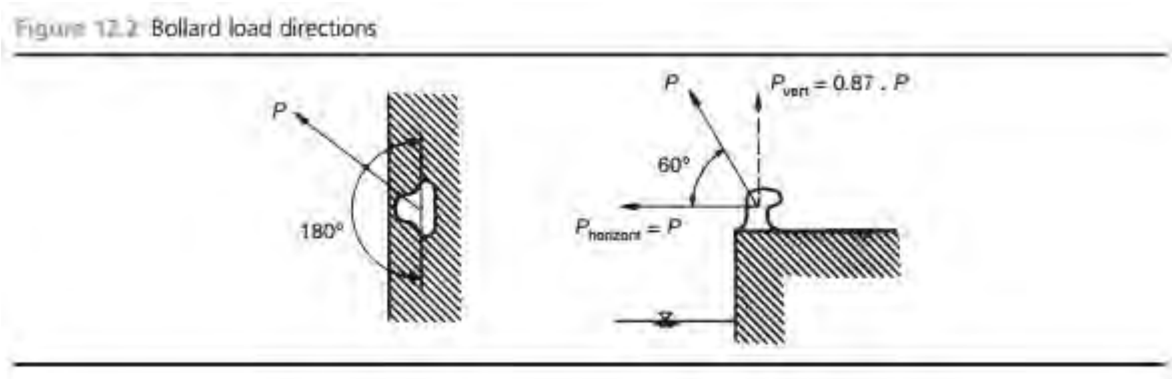


Figure 54 : Bollard load direction (Thoresen, 2014)

Fender reaction load:

$$P = \frac{R}{H * W} = \frac{4593}{10 * 2.5} = 184\ kN$$

Different modelling phases

For the model, several phases have been created phases to take into account for the model. The work is divided in 11 different phases in order to maintain the stability and the safety of the area. The first step is to place the piles with a distance of 30 meters. After, a first excavation is made on the right of the first wall with a depth of 8 meters. A second excavation is made during the phase 2 on the left of the second wall and an anchor is placed to reinforce the wall 2. Then, it is necessary to put 2 linking anchor between the two walls to increase the stability of the soil. During the fifth phase, the excavation is covered by a new soil between the two walls in order to increase the characteristic of the soil. In the phase 7 to arrive to a necessary depth for the ships, an excavation of 16 meters has been done. For the rest of the phases all of the loads have been entered.

All of these phases are detailed in the respective tables (Appendix D)

Modeling's results

After create all of the phases, it is necessary to check whether the modelling complies with the standards imposed by Approach 2 -Eurocode 7.

All of these results are presented in the Figure 0 and Figure 49. The following picture gives the results about the first wall and the second wall.

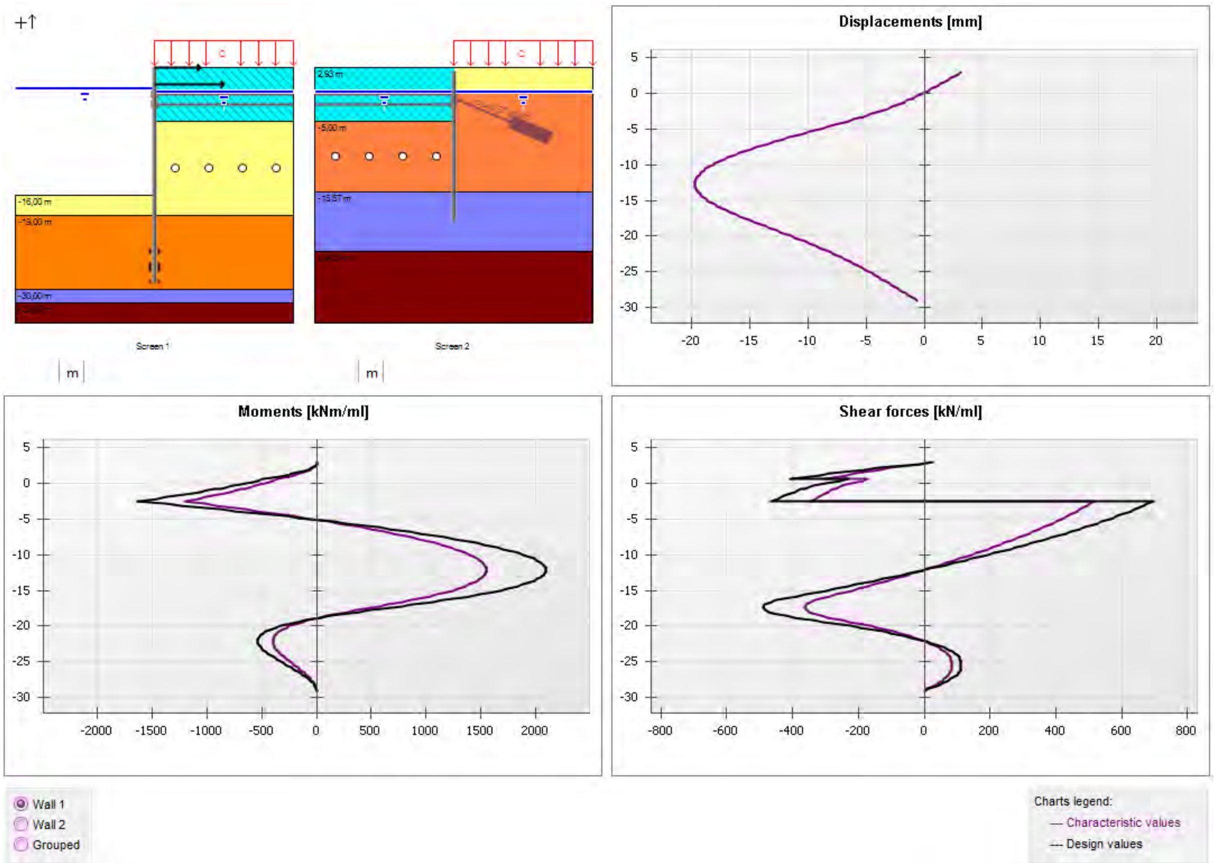


Figure 55 : Results of K-REA for the WAll 1

At the end of the modelling after calculating, the model for the first wall seems to be verified. Indeed, according to the SSL requirement, the displacement of the model is

$$\frac{d}{h} < \frac{1}{200}.$$

At the top of the pile the displacement must not exceed 145 millimetres. In this case, the maximum displacement on the top is 20 millimetres.

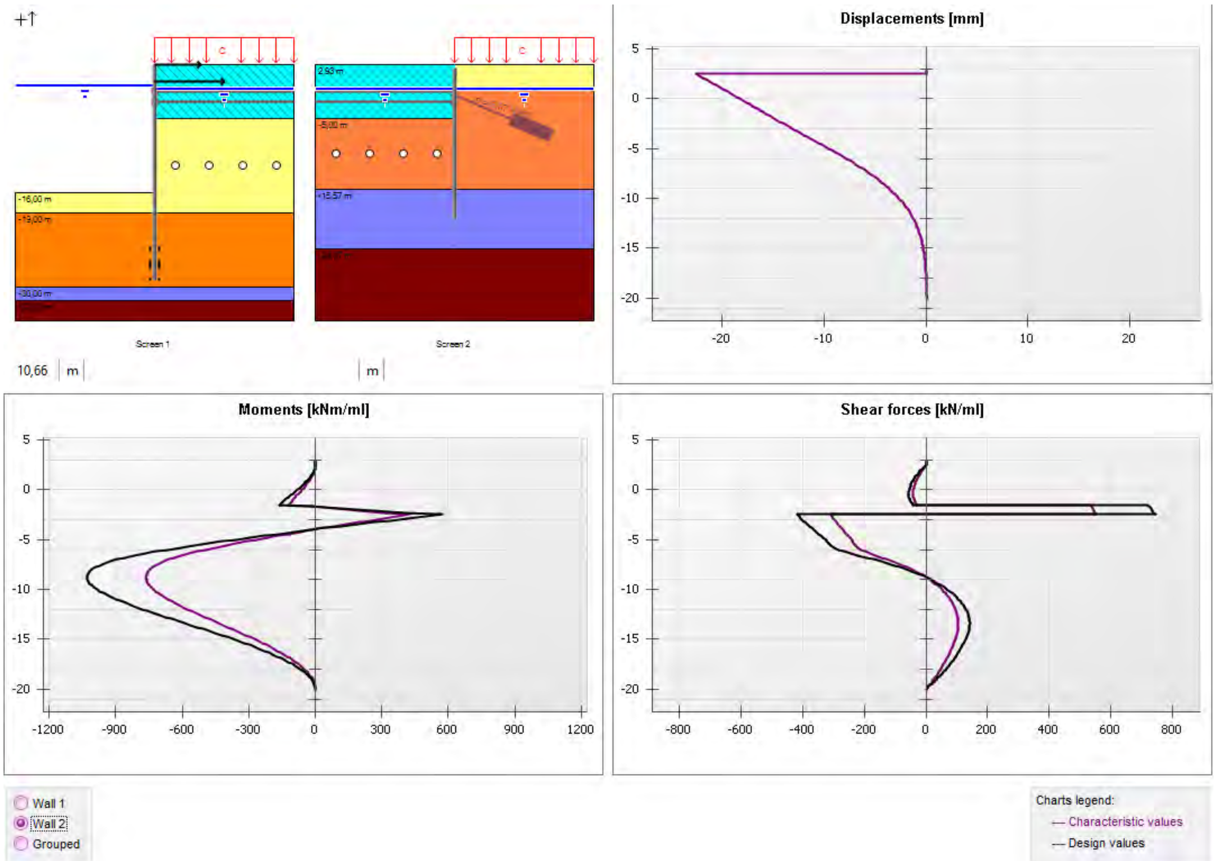


Figure 56 : Results of K-REA for the WALL 2

At the end of the modelling after calculating, the model for the second wall seems to be also verified. Indeed, at the top of the pile the displacement must not exceed 100 millimetres and for this case, the maximum displacement is 25 millimetres.

Conclusion

The structure of the quay wall is verified to accept all of the loads about the logistic and the operability of the container ships. The quay wall is designed to accept all of the container ships of 100 000 tons.

Summary

The rubble mound breakwater was designed analytically and proven to work experimentally in a flume test. The final design was adjusted following the discussion of the group with the laboratory coordinator. The final annotated design can be found in Figure 37.

The mooring dolphin has been designed for the liquid terminal. The designers took into account different sizes of vessels for both crude oil and product tankers. It was ensured that all kinds of vessels specified by the client are able to berth at the terminal. For the final mooring design see Figure 42.

Two sheet pile walls have been designed to carry appropriate load. Several phases of construction have been modelled and checked against failure (see Appendix D, Figure 55 and Figure 56).

7. Financial Analysis

Introduction

AutoCAD is used to make 24 cross-sections of the port area. Looking to the amount of dredging and the amount of backfilling a conclusion can be made about the dredging costs.

In this chapter, there is a preliminary cost estimation of different parts of the project, such as:

- Breakwaters
  - Primary breakwater
  - Secondary breakwater
- Mooring infrastructures
  - Quays
  - Jetty
- Dredging

Dredging

In order to calculate the amount of dredging, it is used the bathymetry AutoCAD file.

This file is overlapped with the port layout and the total surface of the port and this area is divided into 24 cross sections with a separation of 100 meters amount them.

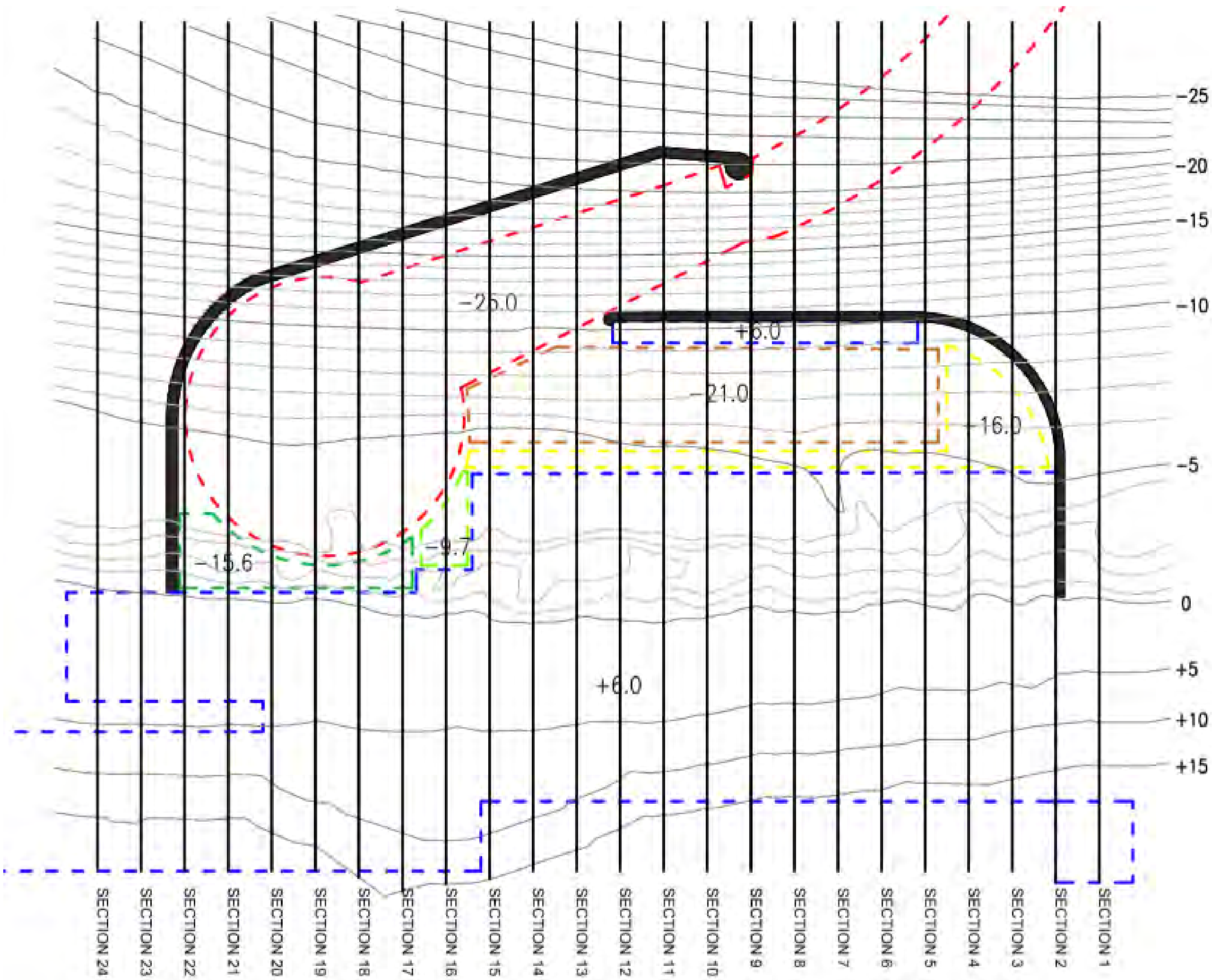


Figure 57: Layout of the cross sections

Once all the cross sections are created with their different bathymetry and heights for the terminals, the different dredging and backfilling areas are obtained. All these measures are done by using Autocad.

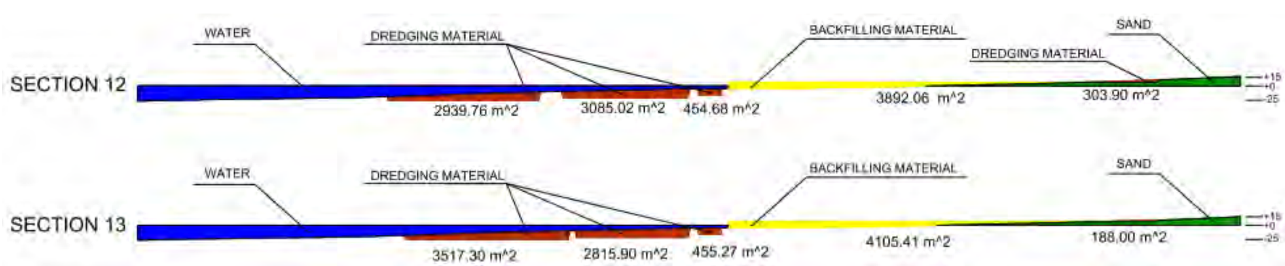


Figure 58: Examples of cross sections

Once they are finished, in order to obtain the amount of volume of dredging and backfilling the method of the medium-areas is followed.

$$V = \frac{1}{2}(A_1 + A_2) \cdot 100$$

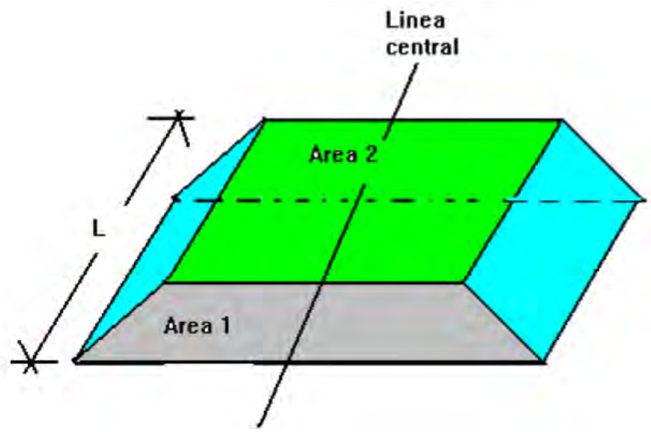


Figure 59: Method of the medium areas

It is deduced from this method the following amounts:

Total amount of dredging volume	Total amount of backfilling volume
16380660 m^3	6675707 m^3

Table 73: Total volumes

For the purpose of dredging sand material, 3 Hopper Dredgers are used. The dredged material will be pumped onshore for land reclamation.

If the dredged material is used for the backfilling, there is a leftover of 9,704,953 m^3 of dredged material.

Cost Estimate

Breakwater costs

Considering the layout and the bathymetry, it is possible to evaluate the breakwater costs.

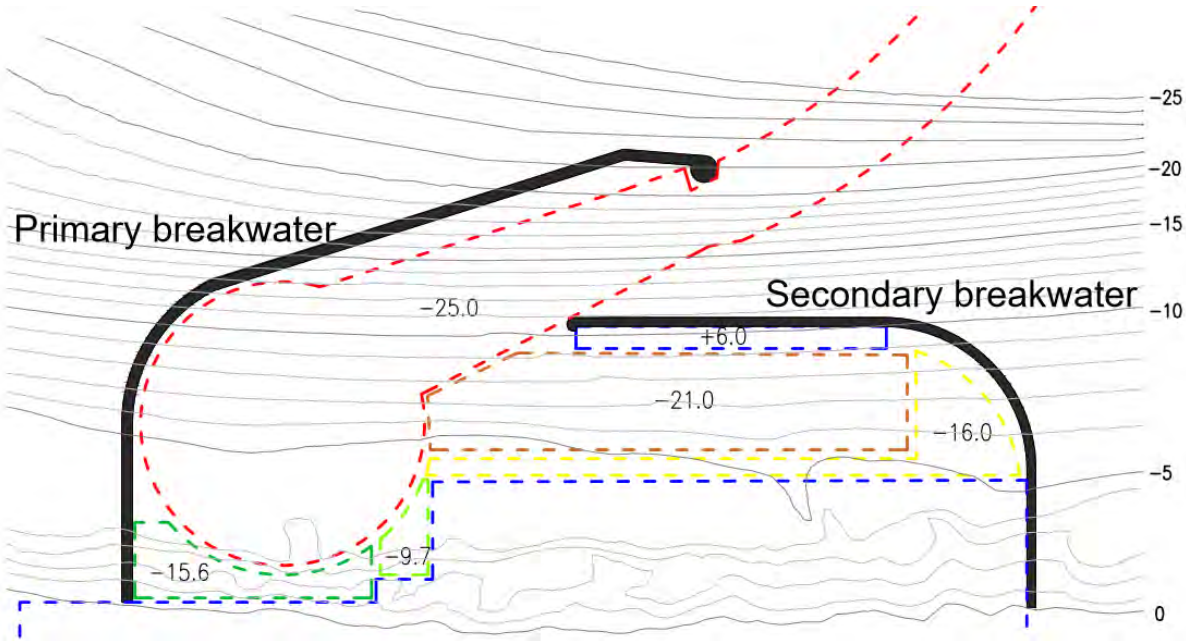


Figure 60. Layout and bathymetry

How it can be seen in the following Figure 61. Costs for the different breakwater, for the estimation of the cost of the breakwaters, the cost per meter is provided by knowing the depth of the various breakwater sections. For each meter of depth, the length of the corresponding breakwater section and the unit costs are calculated. Afterwards, by multiplying them by the length of each breakwater section and adding all the parts, the cost has been met.

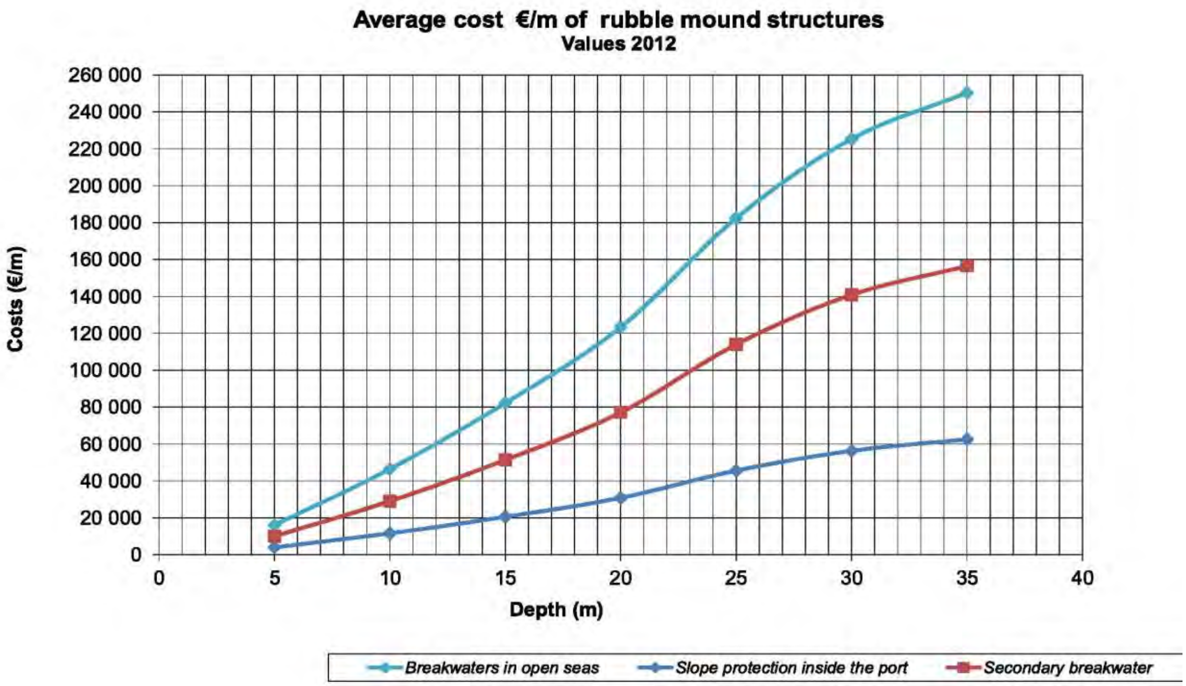


Figure 61. Costs for the different breakwater

In Table 74. `Calculation of the costs of the breakwater`, there is a summary of the data required for the calculation and the final cost.

Breakwater	Depth	Length	Total Costs
Primary Breakwater	0-22 m	1,970 m	154.9 millions
Secondary Breakwater	0-11 m	1,540 m	35.4 millions

Table 74. Calculation of the costs of the breakwater

Quay costs

The procedure for calculating the cost of the quay is the same as for the breakwater. By knowing the length and the depth of the quay wall, it is possible to evaluate the total cost.

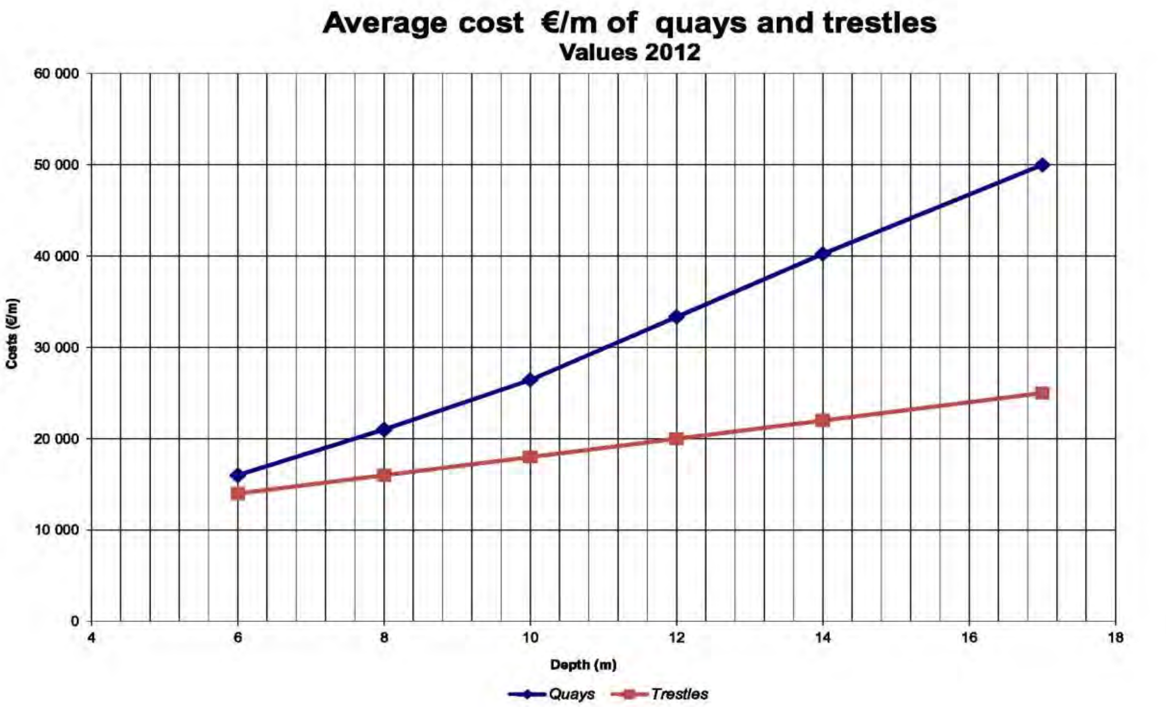


Figure 62. Costs for the quays

For General Cargo the unit costs will be reduced by 15% because the quay is built on land.
In the Table 75. Calculation of the costs of the quay, the data required for the calculation and the final cost obtained are summarized.

	Depth	Length	Total Costs
Quay	9.7-21 m	2,940 m	134.2 millions

Table 75. Calculation of the costs of the quay

Jetty costs

	Unit price	Cost	N°	Costs
Mooring dolphins	1,700,000 €	each	12	20.4 millions
Berthing dolphins	2,100,000 €	each	12	25.2 millions
Platform	6,500,000 €	each	2	13 millions
Walkways	9,000 €	per meter	680 m	6.1 millions
Fender system	150,000 €	each	12	1.8 millions
QRMH	80,000 €	each	24	1.9 millions

Table 76. Calculation of the costs of the jetty

Dredging costs

In this case, the costs are provided per cubic meter; they are function of the type of ground and the volumes to be dredged.

Item	Unit of measurement	Unit cost	Amount	Costs

Dredging in sand	€/m3	7	16,380,660 m^3	114.7 millions
Backfilling with dredged material	€/m3	3 (in addition to dredging)	6,675,707 m^3	20 millions
Mobilisation/demobilisation of one dredger	€	2,000,000	3	6 millions

Table 77. Calculation of the costs of the dredging

Summary

The total amount of dredging volume is 16,380,660 m^3 and the total amount of backfilling volume is 6,675,707 m^3. If the dredged material is used for the backfilling, there is a leftover of 9,704,953 m^3 of dredged material.

The leftover dredged material will be stored at the right side of the port. Because of the sediment transport there will be erosion just behind the secondary breakwater. If there is a huge storage of dredging material it is easy to nourish the beach with this amount of sand.

The different costs are summarized over here:

	Costs
Breakwater	190.3 millions
Quay	134.2 millions
Dredging	140.7 millions
Jetties	68.4 millions
Total	533.6 millions
5 % markup	26.7 millions
TOTAL	560.3 millions

The dimensions of the gantry cranes (Ligteren & Velsink, 2012):

- Outreach: 35m (enough for the design container vessel)
- A service lane (between the coping and the front crane rail): 3-5m
- Rail gauge(distance between seaward and landward leg): max. 35m
- Back reach: min. 15m

For straddle carriers (SC) two lanes are usually sufficient. The average width of a SC is 5m. A two-way lane will assumed to be 20m (Ligteren & Velsink, 2012). An overview is given in Figure 63

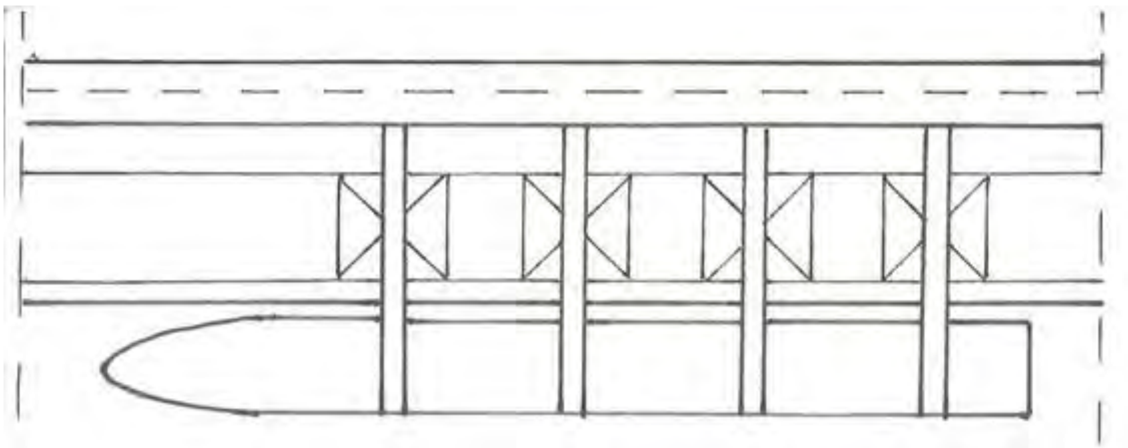


Figure 63 : Apron area

Oil terminal

The oil ships will berth inside the breakwaters of the port. The berth mainly consists of a jetty (Figure 64) and dolphins. It is called a L-jetty.

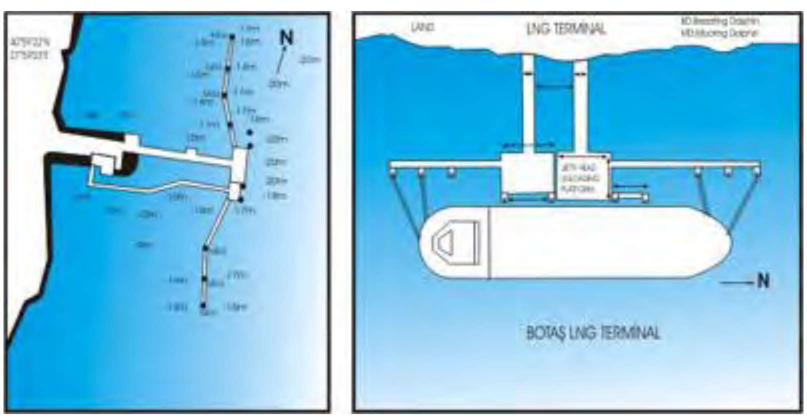


Figure 64 : oil terminal general layout (company, 2003)

The crude oil will be transported by pipes, on top of the breakwater. The crude oil is stored in the storage area for a maximum time of 20 days. The refinery is designed inside the storage area, where a part of the crude oil is going to be transformed into refined products. The other part will be transported by feeder ships.

Coal terminal

The main goal of the coal terminal is to bring the coal to the storage area with at least two conveyers, one for unloading and one for loading the coal vessel. For (un)loading the ships grabs are used, see Figure 65. The grab is used for picking up material from the vessel hold and place it onto a belt conveyer. A bulk cargo terminal for a range of commodities will require a set of 2 or 3 grab buckets per crane.

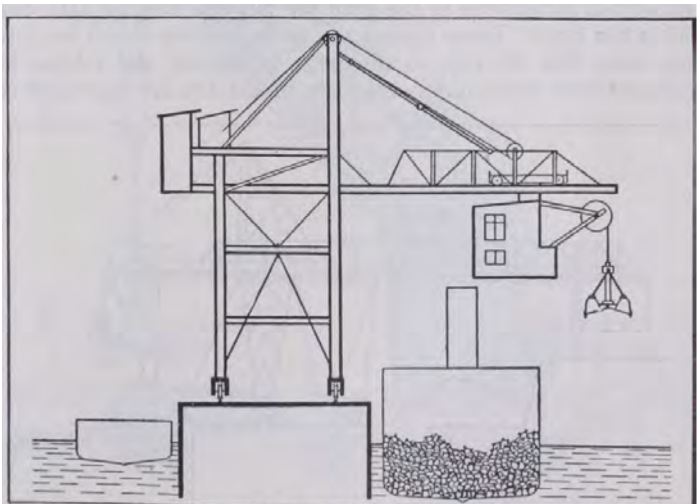


FIG. 15.—ELECTRIC TRANSPORTER CRANE FOR DISCHARGING COAL
In this type the front boom is first raised to permit the entry of the ship; the trolley and control cabin then move out till over the ship's hatch

Figure 65 : loading and unloading equipment for bulk terminal (overheadcraneskit, 2014)

The storage area will be placed in line with the electrical plant, which is designed outside of the storage area.

General cargo and Ro-Ro

The general cargo and the Ro-Ro terminal are combined in the layout in order to reduce the waiting time for the vessels (see queuing theory). One of the general cargo berths is a multifunctional berth, at the back of the ship there is place to (un)load the ro-ro and at the side there is a possibility to (un)load the general cargo.

The storage area is divided in three parts; transit shed, open storage and warehouses. Depending on the type of general cargo a detailed layout can be made for this part.

Summary

The layout presented in Chapter 4 was tested in the modelling software for agitation and the output was compared with the maximum allowable values for all designed terminals. The values obtained from the model exceeded criteria specified in the scope of work for one terminal, namely the liquid cargo located by the primary breakwater. However, the obtained values do not exceed limitations outlined in PIANC, therefore the design is considered to remain valid. In case the client would like to lower the agitation for this terminal to meet more strict requirements, an additional hammer was specified to further protect the liquid terminal.

The approach channel dimensions were calculated and presented above. The maximum bend radius, channel width and stopping length were found to be less than those initially specified in the layout, therefore the layout design is appropriate and does not need adjustments. All results for the approach channel are presented in Table 39:Results showing the calculated channel width required for different ships. The highlighted cell indicates the width chosen for the design

, Table 40: Results showing the calculated channel depth required for different ships. The highlighted cell indicates the depth chosen for the design

, Table 41: Channel alignment results

and Table 42: Results of navigation analysis

above. Tugboat assistance is recommended for a safe and efficient operation of the port. Those are specified and presented in Table 42: Results of navigation analysis

Table 42: Results of navigation analysis

above.

For the container terminal Ship-to-shore gantry cranes are used, 4 per berth. The apron area will be approximately 90 meters. The containers are transported to the storage area and stacked by straddle carriers. The oil will be transported by pipe lines, on top of the breakwater. For the coal terminal grabs are used to place the coal on the conveyers.

8. Conclusion

Design process

Like any other large maritime project, it was not a close end problem where the set of entry data produces defined output; the process was iterative instead. First crude assumptions were made by applying engineering judgement and employing solutions known from existing ports around the world. Then progressive and iterative application of given data allowed the team to produced first models, which were improving with every iteration. The final design presented in this report does not represent whole iterative process, but rather most important steps that led to the complete solution and its thorough explanation.

Design drivers

The main drivers for this design were specified by the client:

- Entrance to North-East
- Minimise the cost of breakwater
- Maximise the dredging
- Allow for future expansion

Those drivers were taken into account at every stage of this iterative design.

Final solution

The final solution is designed, analysed and presented in this report. The total cost of the new port was calculated to be approximately €560 million, which is within the expected range for this size of the port.

The proposed layout of the new port is simple, yet utilises the strength and minimises the weaknesses of the site conditions. One of the main design criterion was the possibility of the future expansion. The group put special emphasis on the possible extension of the container terminal, with plausible subsequent minimisation of general cargo shipment. Therefore not only new terminals can be build (effectively enlarging container terminal capacity), but also the general cargo terminal can be re-adapted easily to serve as a secondary container terminal.

Safety of navigation and port operation was another important factor which was fulfilled in the final design with great care. One of the main issues when preparing different layout options was creating a well-utilised space, yet avoid possible collision zones between approaching and berthing vessels. The efforts were very successful and full clearance between manoeuvring areas and terminals has been achieved. Nevertheless, tugboats are recommended for the operation and are specified within this report.

Problems

The design process is never ideal and issues were present on the way. Most of them were minor and were effect of model inaccuracies, misunderstandings between team members etc. Those were usually solved within the group, occasionally with a help of one of the coordinators.

Unfortunately the final design still carries uncertainties associates with the issues that are considered not to be resolved fully

Wave agitation

As seen in Chapter 5, the agitation model prepared for the final layout produced waves inside the port higher than expected. The liquid terminal located at the inner side of the primary breakwater could potentially experience waves up to 1.0m in height during operation. The obtained value does not meet the client requirements for the operational wave high for this type of terminal. However, the PIANC allows for heights of waves to be maximum 1.5m for oil terminals.

The terminal was later found to be inappropriately sheltered in the design. A small adjustment to the solution was proposed (short “hammer” breakwater attached to the primary structure near the port entrance). Unfortunately the time allowed for preparation of the agitation model was highly limited, therefore no additional simulation could have been run. It is recommended for the future investigation to prepare more agitation models to meet client’s requirements.

Complying with contradictory requirements

On the stage of designing layout options the team has encounter a problem in the form of contradictory design requirements. Those were:

- Entrance to NE
- Shallow angle between vessels’ path in outer approach channel and dominant wave direction
- Not more than 30° turn in the approach channel
- No turn in the entrance of the port

Those issues have not been fully resolved, but were minimised by multiple iterative design process. 3 proposed layouts provide different level of compliance with conditions specified above.

The most reasonable solution to this problem would be not complying with the entrance direction and place in the SW or W direction. This would increase maintenance (due to sedimentation from nearby Oued river), but would ensure safe and economical design.

Fender

An example of a minor, yet unsolved problem is a fender design for the liquid terminal mooring dolphin.

The client specified the vessels which will be using the new port. In the process of analytical solution of the problem, the team found out that the manufacturer does not specify fenders of the required (large enough) size. This is an example of a very typical problem in real, large engineering projects – lack of pre-designed, pre-manufactured structures. The recommendation is to design and produced a custom fender, probably by outsourcing it to a specialist company. Nevertheless, this design is beyond the scope of the project.

Summary

Overall the design is considered valid, however further investigation is recommended before commencing any works. Due to time limitations some aspects of the project may have been overlooked or not iterated enough, therefore increasing the design cost. This report can serve as a basis for the further study of construction of Nador West Med port.

Bibliography

al, D. M. (2016). *Manual on wave overtopping of sea defences and related structures.*

BSI. (2014). *BS 6349 Maritime works – Part 4: Code of practice for design.* London: BSI Standards Limited.

Cappelli, A., & Libardo, A. (2008). A CBA and MCA as DSS in Transportation Engineering. *Transportation decision making: issues, tools, models and case studies*, (pp. 50-62). Venice.

CIRIA, C. &. (2007). *The Rock Manual, the use of the rock in hydraulic engineering.* London.

company, S. s. (2003). *selimshipping*. Opgeroepen op 06 15, 2017, van http://www.selimshipping.com/eng/pages/marmara2.htm

Department for Communities and Local Government: London. (2009). *Multi-criteria analysis: a manual.* London: Communities and Local Government Publications.

Engineers, U. A. (1984). *Shore Protection Manual.*

Groenveld, R. (1993). *Service Systems and Inland Waterways.* Delft: TU Delft, Department Hydraulic Engineering.

Hofstrand, D. (2008, Extension). *Liquid Fuel Measurements and Conversions*. Opgehaald van Extension: https://www.extension.iastate.edu/agdm/wholefarm/html/c6-87.html

Libardo, A., & Parolin, A. (2012). Multicriteria analysis evaluating Venice port development. *Procedia - Social and Behavioural Sciences*, 2545-2554.

Ligteren, H., & Velsink, H. (2012). *Ports and Terminals.* Delft: Delft Academic Press.

Ligteringen, & Velsink. (2012). *Ports and Terminals.*

Ligteringen, H., & Velsink, H. (2012). *Ports and Terminals.* Delft: Delft Academic Press/VSSD.

Ligteringen, H., & Velsink, H. (2014). *Ports and Terminals.* Delft: Delft Academic Press / VSSD.

Lo Presti. (2016). *Mauale di geotecnica.* Pisa.

London, I. C. (2012). *Model-Prototype Simillarity.*

M.W., O. (1980). *Design of sea walls allowing for wave overtopping.* Wallinford.

overheadcraneskit. (2014). *grabs*. Opgehaald van overheadcraneskit: http://www.overheadcraneskit.com/tag/grab-bucket

PIANC. (2014). *PIANC.*

PIANC Working Group - 33. (2002). *Guidelines for the design of Fender systems.* Brussels: International Navigation Association.

Rebollo, J. A. (2017). *Nothing, really.*

Rolland, G. (2017, June). Fixed-mooring structure design.

ROLLAND, G. (2017, June 20). *Group B.* Opgehaald van Google Drive: https://drive.google.com/drive/folders/0B4vK2GyIQnk1MHFiaDFaOFFiLWs

Silva, P. (2016, November). Terminal Typology.

Silva, P. (2016). Terminal Typology.

Silva, P. (2016, November). Terminal Typology.

Thorensen, C. A. (2014). *Port Designer's Handbook - Third Edition.* London: ICE Publishing.

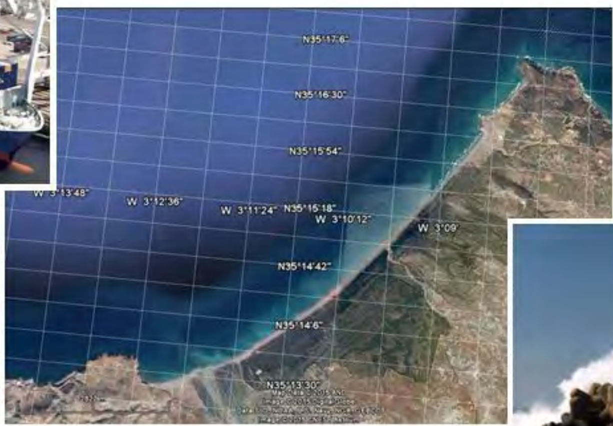
Thoresen, C. A. (2014). *Port Designer's Handbook 3rd Edition .* London.

Trelleborg AB . (2016). *Fender Apllication-Design Manual .* Trelleborg.

Trelleborg AB. (2017). *Fender Systems.* Trelleborg.

Port design

New port and industrial complex design, around 30km from the town of Nador, on Morocco's Mediterranean coast



GROUP C:

Mathijs Mann (Netherlands)
Claude la Hausse (Maurice)
Lelde Zimele (Latvia)
Elena Delmas (Spain)
Emmery Beauperthuy (France)
Francesco Roda (Italy)
Giannis Karampetsos (Greece)
Maxime Lamour (France)
Stephane Delbreil (France)
Verdiana Verdiglione (Italy)

WORKSHOP on Port Engineering - June 2017

EXECUTIVE SUMMARY

The main aim of the report is to present a port layout design, as well as the design of the related main structural elements, for the location in the Mediterranean basin, 30 km from Nador, Morocco.

As requested by client (the government of Morocco), different layout options are investigated for the particular geographical requirements – the entire bay is available for construction, as well as the other design criteria, such as the traffic needs, design ship dimensions, the number of berths required. The main constraints of the design are as follows: wave agitation in the port, sedimentation criteria, as well as the potential for further expansion of the port. There were two non-negotiable aspects that had to be incorporated in the design. First, the entrance of the port must be located facing the SW direction. Second, dredging must be reduced to minimum, leading to the main breakwater located in deep water, at about 35m depth.

The main driving forces for the choice of the location of the proposed port layout: increased water depth in NE and poor geotechnical conditions in SW (due to increase in sediment transport from River Kert).

The recommended solution consists of a caisson breakwater at a right angle, which is replaced by a rubble-mound breakwater with decreased depth, and is thoroughly described in later sections of the report. Queuing theory has been used to determine the number of berths required, while design wave conditions have been acquired from statistical wave data.

Finally, design optimisation is suggested, considering that the design criteria can be revisited.

TABLE OF CONTENTS

1. INTRODUCTION

The project of NMW is part of the overall strategy of the development of this part of Morocco, which is currently regressive in terms of economic and human development. The proposed port will be built in Betoya Bay, approximately 30 km from the city of Nador. It will consist of a deep-water port and industrial platform open to investors.

The port has a very advantageous position in the Mediterranean, which is a potential position of the container transhipment. The future site is well connected to already existing road networks, motorway and airport (either existing or planned).

The physical requirements for the port are as follows: terminals for containers, hydrocarbons, and bulk materials (including coal), ro-ro terminal, service quay and industrial investment zone (1500 ha free zone and 2500 ha industrial area). The main functions of the port are identified to be the following (but not limited to): transhipment hub for crude oil, transhipment hub for import and export of the containers, import and export of the general cargo, including ro-ro mode, and import of coal for the future electrical plant. The port might need to adapt additional functions during its lifetime, as further expansion of the port is likely and must be considered in the design process.

The first chapter of the report gives an overview of the design criteria that must be met for the port design. The report then describes the design conditions, including statistical wave analysis, wave propagation, site conditions and sediment transport. The following chapter describes two preliminary layout designs, and discusses their advantages and disadvantages. Then, the final design layout is thoroughly described, and checked for the previously set design criteria. The final chapter of the report summarises the findings and main design features, as well as gives recommendations for potential design optimisation.

2. DESIGN CRITERIA

The chapter outlines all the design requirements for the Baie de Betoya. First it describes the geographical limitations and traffic needs. This is followed by the description of the procedure for both the determination of the dimensions of the design ships and the number of berths. The chapter then moves on to outlining the wave agitation and sedimentation criteria. Finally, the chapter outlines the active load considerations for the dimensioning of the coastal structures, such as breakwater, dolphin and the quay.

2.1 Geographical Limitations

1. River Kert in SW;
2. Cape Garet in NE.

2.2 Traffic Needs

The traffic needs for the development of the port are summarised in Table 2.1 below.

Table 2.1. Traffic volumes.

Traffic Classification	Unit	Volume (Units/year), 10^6
Containers	TEU[1*]	3
Oil	Tons	25
Coal	Tons	7
General Cargo	Tons	3[2*]

[*1]: 1TEU = 20 equivalent units;

[*2]: 1 million are transported by trucks in a ro – ro mode.

2.3 Design Ships

The required design sizes of the ships to be hosted in the port are summarised in Table 2.2 for the ships carrying containers, coal, general cargo, and ro–ro ships. However, additional analysis for the large crude oil ships (from statistical data available) must be carried out to determine the allowable tanker dimensions to be allowed in the port.

Table 2.2. Required ship dimensions to be hosted in the port.

Traffic Classification	Subcategory	Units	Maximum Size
Containers	Mother ships	TEU	18 000
	Feeders		5 300
Coal carriers	Capesize	DWT	170 000 - 180 000
General cargo	Ships	DWT	40 000
Ro-Ro	Ships	DWT	15 000
Tankers*			

*: Size determined by statistical analysis of the variety of the ship sizes, and is described below.

2.4 Number of Berths

Based on the determined dimensions of the design ships and the set traffic needs, the number of berths must be further calculated using queuing theory. For this, see section 4.1.2. The following assumptions for the capacity of each berth were in place:

- Berth must work 24/7;
- Berth must be available for 350 days/year;
- Efficiency of each berth (with respect to the nominal capacity of each crane) equals to 80 %;
- Occupation factor of each berth depends on the total number of berths.

2.5 Wave Agitation

The threshold values for the wave heights at the berths, with non-exceedance rate of 1 %, are summarised in the Table 2.3.

Table 2.3. Allowable wave heights at the berths.

Berth Classification	Acceptable Wave Height, m
Oil and coal	1.0
Container	0.7
General cargo and ro-ro	0.5

2.6 Sedimentation Criteria

The effects of the sediment transport from the River Kert on the port layout are analysed and quantified in this section. Then, the recommendations for the layout are made, considering the sedimentation constraints.

2.7 Space for Further Expansion

Even though the current design of the port is governed by the particular conditions provided, possibility of future expansion must be considered.

2.8 Active Loads for Dimensioning of the Coastal Structures

Table 2.4 summarises the active loads that need to be considered for the design of different coastal structures.

Table 2.4. Active loads acting on the coastal structures.

Coastal Structure	Active factors for the design
Breakwater	Wave
Dolphin	Berthing energy to fenders
	Stress induced by mooring lines to bollards or quick release mooring hooks
Quay	Berthing energy to fenders
	Vertical loads and loads on bollards

The vertical loads and loads on bollards are summarised in Table 2.5 for the different zones of the port.

Table 2.5. Vertical loads and loads on bollards.

Harbour Zone	Vertical Load (whole surface), t/m^2	Vertical Load (crane), t	Dynamic Load (on bollards*2), t
C	4	-	50
Container Quay	6	2 500*1	150

*1: Maximum linear load on one rail equals to 100 t/m (along every 10 m), rail spacing equals to 30 m;

*2: Spacing equals to 20 m.

(marine operational forces)

2.9 Conclusions

The main design criteria for the port development have been identified in the section, according to which the port layout is designed in the following sections. This includes the geographical limitations, traffic needs, design ship and wave agitation requirements, sedimentation criteria, as well as the summary of the active loads for dimensioning of the coastal structures.

3. DESIGN CONDITIONS

3.1 Topography and Bathymetry

Source of all the figures: Google Earth.

The following figures depict the topography of the bay where the port development is proposed. The general project area is illustrated in Fig.3.1. A closer look at the topography profiles available reveals that the area identified for the port location is not flat; the topography profiles 1 and 2 are shown in Fig.3.2 and 3.3 respectively. The height changes, on average, from 15 until 70 meters in this area. Moreover, it can be observed in Fig.3.4 and 3.5 (topography profiles 3 and 4 respectively) that there is a linear increase in height inland from the coast; hence, it is considered a reasonable location for the port development.

Even though there is a flat area available at the south side of the bay, with an almost constant height of 70m (the darker area at the bottom left corner in Fig.3.1), one of the main requests by client is to have the port entrance at the SW. If the port was moved to the flat area, more complications would arise for the sediment accumulation that gets transported in larger amounts from River Kert. For this, it was decided that the elevated area higher up in the North would be farther investigated as the location for the port development.

Figure 3.1: topography of the project area (Google Earth).

Figure 3.2: Topography profile 1 (see Fig.3.1).

Figure 3.3: Topography profile 2 (see Fig.3.1).

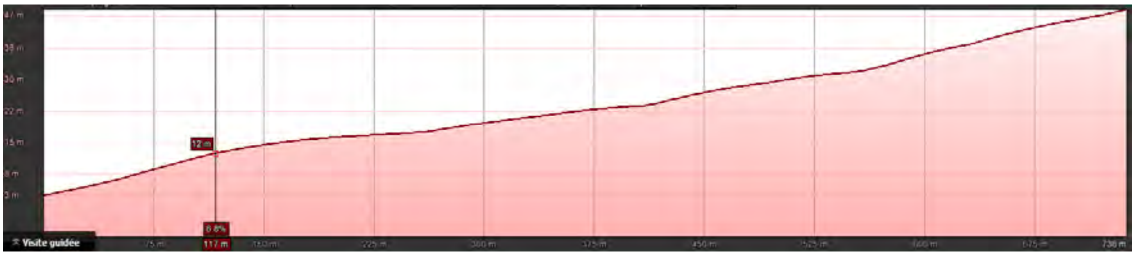


Figure 3.4: Topography profile 3 (see Fig. 3.1).

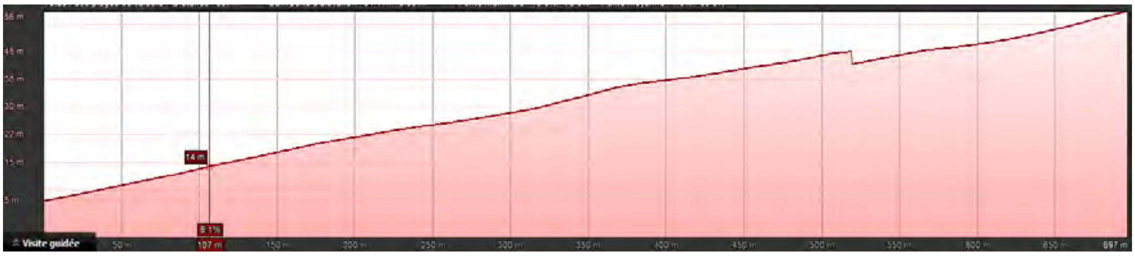


Figure 3.5: Topography profile 4 (see Fig. 3.1).

The bathymetry of the location of the port development is depicted in Fig.3.6. It is noticeable that the more to the North an area is located, the deeper the water level; the contour lines are spaced more frequently in the North than in the South of the bay. At the river outlet, the contour lines are spaced considerably less frequently, therefore, the depth is increasing not that fast in the southern area. Also the contour lines in depths larger than 20 meter are close to each other in the northern part. Therefore the depth is increasing quite fast in the area. This is another argument in favour of moving the port more to the North, as to comply with the requirement to minimise the dredging needed for the construction and operations of the port.

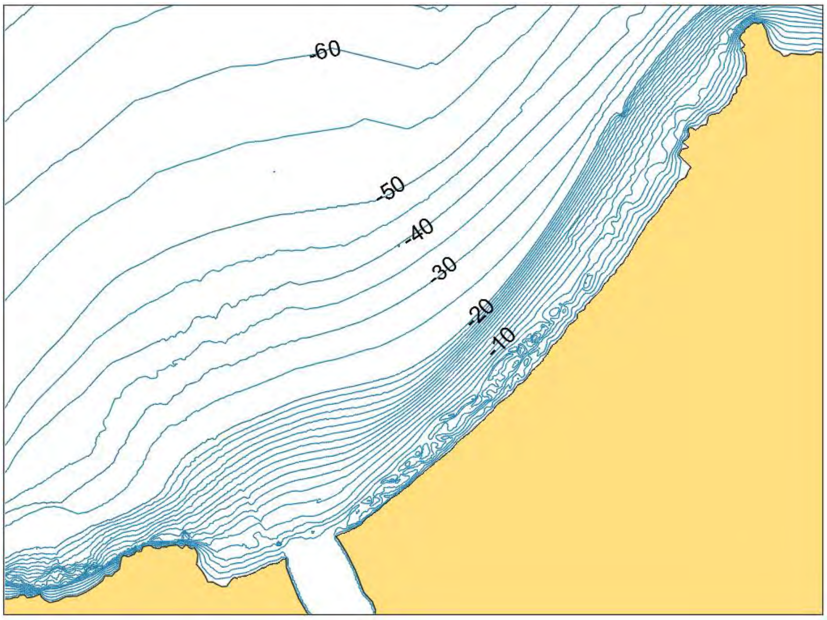


Figure 3.6: The water depth profile of the project area; values shown in metres (Scope of Works).

The bathymetry of the bay allows for the deep water port that will be able to accommodate new generation container ships (up to 18 000 TEU) and tankers of up to 170 000 tons, with limited amount of dredging involved, as long as the port is moved farther up to the North.

3.2 Wave Analysis and Wave Propagation

3.2.1 Sea water level

The Intergovernmental Panel on Climate Change (IPCC) has given sea level rise predictions based on forecasting models, which use predictions for greenhouse gas emissions as an input. The RCP4.5 (Fig. 3.7) was used for the sea level rise predictions. Since a 60-year lifetime has been chosen for the structure, a 0.4m sea level rise is defined for the project.

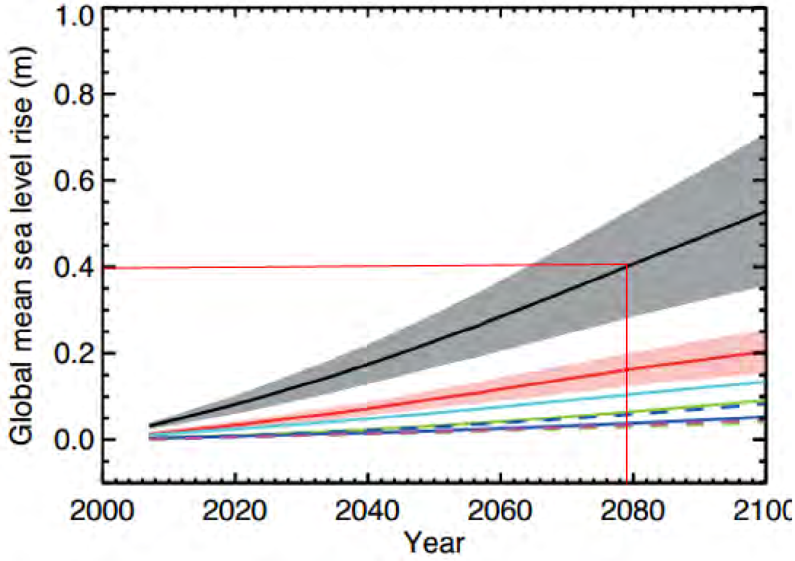


Figure 3.7. RCP4.5 global mean sea level rise predictions (Church et al., 2013).

3.2.2 Wave analysis

Time series of offshore waves and winds were acquired from the Nador West buoy, of coordinates 70.2369N, 52.0512E. The analysis for the basis of the project relied on 18 years of data made available, from 1992-2009.

The 'wave rose' in Fig. 3.8 represents the direction of the incoming waves and their respective wave height distribution. Similarly, the wind direction frequencies of occurrence and speeds are shown on the 'wind rose' in Fig. 3.9. As seen in Fig. 3.8, the highest and most frequent waves originate from the WNW and NNE directions. The wind also indicates two main directions of approach: the W and ENE. The westerly winds are slightly more frequent, and of considerably higher speeds.

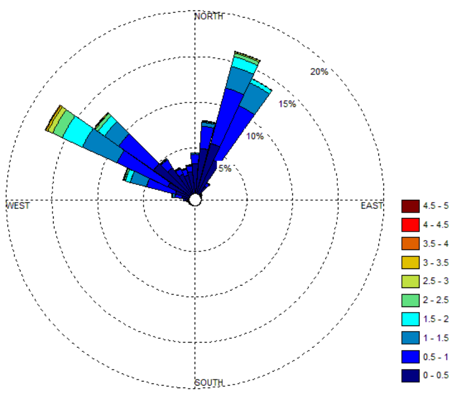


Figure 3.8: Wave rose (values in m).

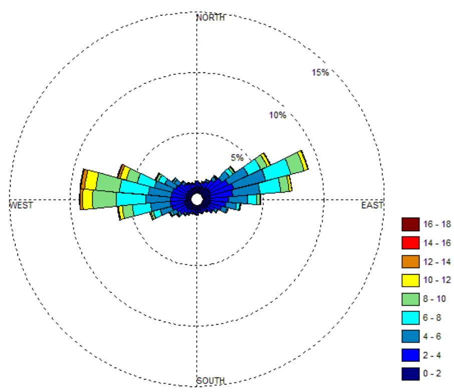


Figure 3.9: Wind rose (values in m/s).

To acquire the design inputs of the port infrastructure design, the extreme wave heights, periods and directions for given return periods were determined, as indicated in the following procedure.

Wave heights

To determine a reasonable significant wave height for design purposes, a wave height dependency on a given return period was determined. The first step involved extracting storm data from given time series. Since the area of study is in the Mediterranean, storms were defined as events for which the wave height exceeded a threshold of 1.8m (approx. 5% of occurrence). Each storm event has a minimum duration of 24 hours of wave heights exceeding the indicated threshold. An average of 5 to 10 storms per year were retrieved. A Weibull curve was then plotted to the data, and provides an equation relating the wave height to the return period, shown in Fig. 3.10 below.

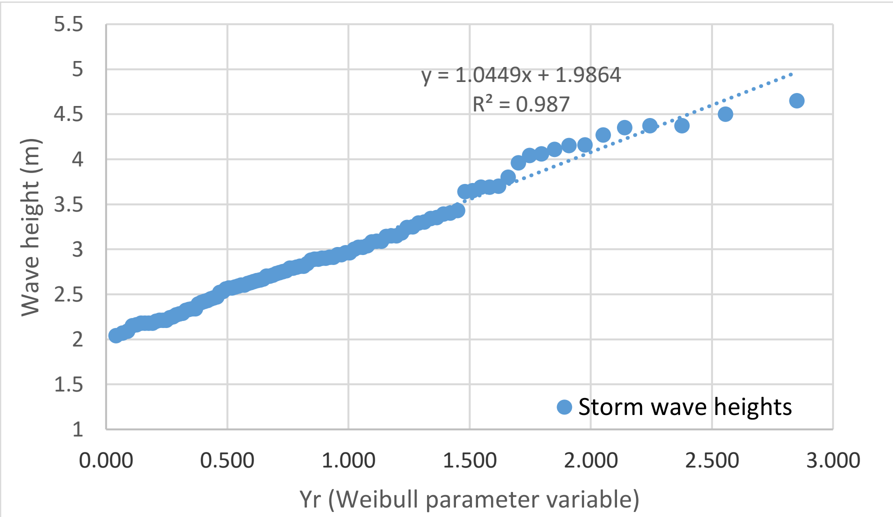


Figure 3.10. Storm wave heights plotted on a Weibull distribution.

Then, a relation between the peak period and the wave height was determined using the storm periods and wave heights, and fitting a line (Fig.3.11).

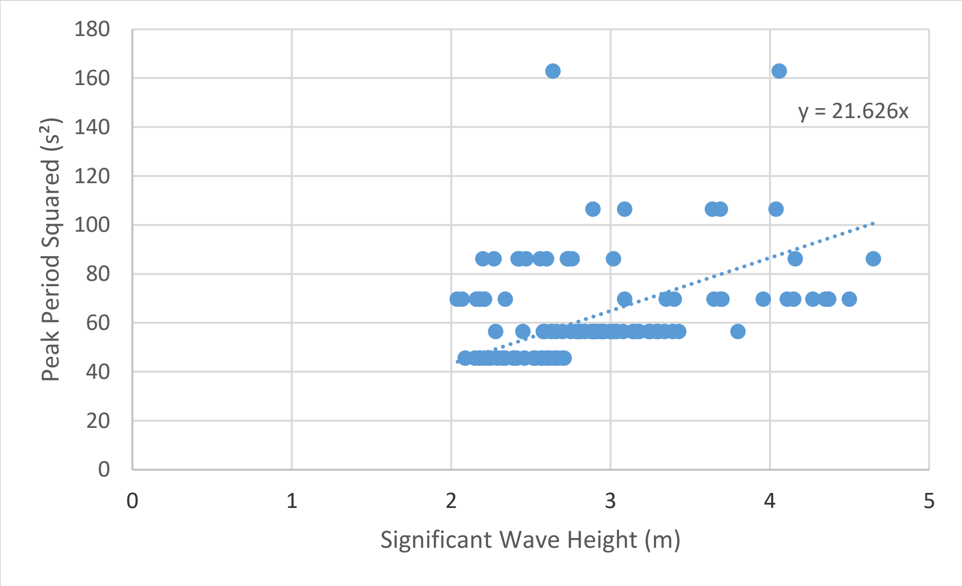


Figure 3.11: Linear relation between the wave height and peak period squared.

Since the analysis included waves from all directions, a directional coefficient was determined to provide the expected design waves in respective directions for a given return period, all summarised in Table 3.1 below.

Table 3.1: Directional Coefficients for storm events

Direction	**Total Wave Count**	**Wave Count for 99%**	**Average of Highest percentile waves (H1/99), m**	**Directional Coefficient**
N	5827	58.27	2.19	0.81
NNE	17238	172.38	2.7	**1**
NE	1579	15.79	1.4	0.519
W	866	8.66	1.84	0.681
WNW	11402	114.02	2.39	**0.885**
NNW	3495	34.95	1.73	0.641

As noted in Table 3.1, the highest waves are expected to approach from the NNE and WNW directions, with directional coefficients equal to 1 and 0.885 respectively.

Design life results

As the design life of the port is 60 years, the design wave return period is therefore determined by taking into consideration the probability of failure of the structure within its lifetime. Based on a normal level of importance an indicative acceptable probability of failure is 0.05. The limits are summarised in Table 3.2 below, which shows the indicative values of acceptable probabilities of failure within structure lifetime.

Table 3.2. Acceptable probabilities of failure for a structure's lifetime (PIANC, 2002).

Limit State	Safety Class			
	Very Low	Low	Normal	High
SLS	0.4	0.2	0.1	0.05
ULS	0.2	0.1	**0.05**	0.01

Based Eq. 3.1, the return period for a lifetime of 60 years is equal to 1170 years. The specification criteria nevertheless call for a return period of 100 years. Hence, to be consistent throughout the project, the specified return period of 100 years is used for the design of the port.

$$T_R = -\frac{T_L}{\ln(1 - P_f)}$$

Equation 3.1.

Where T_R is the return period, T_L – lifetime, and P_f – probability of occurrence.

The Table 3.3 below presents the wave heights and directions at different depths for different return periods.

Table 3.3. Wave height and period for different return periods

Input			Offshore	
Lifetime	Return Period	Direction	Wave Height	Peak Period
TL	TR	-	Hs	Tp
(yr)	(yr)	(°)	(m)	(s)
N/A	1	NNE	3.53	8.74
		WNW	3.12	8.22
	100	NNE	5.67	11.07
		WNW	5.02	10.42
60	1170	NNE	6.58	11.93
		WNW	5.82	11.22

3.2.3 Wave Propagation

The values for different wave heights, their period and direction are provided at an offshore location. In order to acquire information about the waves at the location of the main breakwater, Tomawac software is used to propagate the offshore waves to the coast. In such way, a simplified equation for the spectro-angular density of wave action is solved using finite element method; steady state conditions are assumed. The following physical phenomenon are considered:

1. Wind-generated waves;
2. Refraction at the bottom of the seabed;
3. Dissipation through bathymetric wave breaking;
4. Maximum tidal value.

The propagated values are used for the design of the main coastal structures of the port, as well as performing the wave agitation into the port. The propagated values from the NNE and WNW directions are displayed in Table 3.4 and 3.5 respectively. The values are recorded for the water level depth of 15m and 35m.

Table 3.4 Propagated wave values coming from the NNE direction.

	35m depth			15m depth		
	Direction	**Wave height**	**Peak period**	**Direction**	**Wave height**	**Peak period**
NNE 1 year	10.72	2.73	8.98	174.23	2.24	9
NNE 100 Years	7.41	4.24	11.14	169.75	3.59	11.23

Table 3.5. Propagated wave values coming from the WNW direction.

	35m depth			15m depth		
	Direction	**Wave height [m]**	**Peak period [s]**	**Direction**	**Wave height [m]**	**Peak period [s]**
WNW-1year	296.65	2.87	8.99	117.77	2.78	9
WNW-100 years	299.34	4.55	11.19	118.38	4.51	11.19

The propagated wave heights, peak periods, and main wave directions for the NNE direction for one year return period are shown in Fig. 3.11, 3.12, 3.13 respectively. Figures showing the propagated values for the NNE for 100 years return period, as well as the propagated values for tne WNW direction for both return periods of interest are included in the Appendix A.

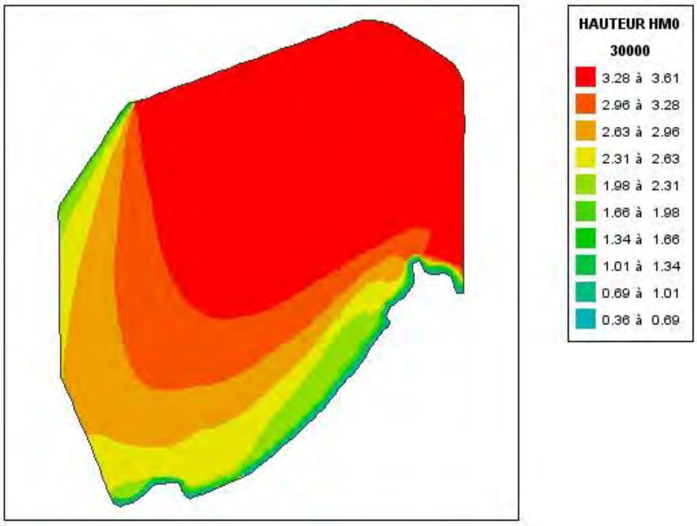


Figure 3.11. Propagated wave heights (shown in m).

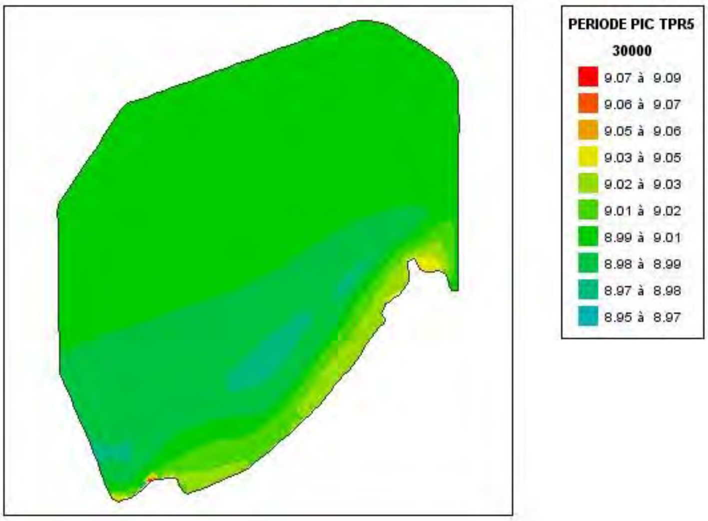


Figure 3.12. Propagated peak periods (shown in s).

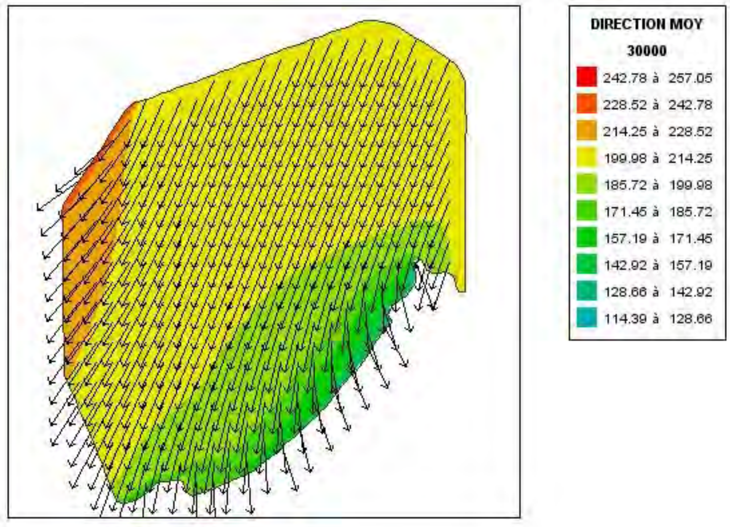


Figure 3.13. Propagated wave directions (shown in degrees clockwise from the N).

3.3 Site Conditions

3.3.1 Boreholes

The geotechnical study revealed that the soil characteristics more to the South are worse (very weak clay layers – marked red in Fig.3.14) than those closer to the North; hence, it was decided that the analysis will be based on the location surrounding the potential location of the port (marked green in Fig.3.14.).

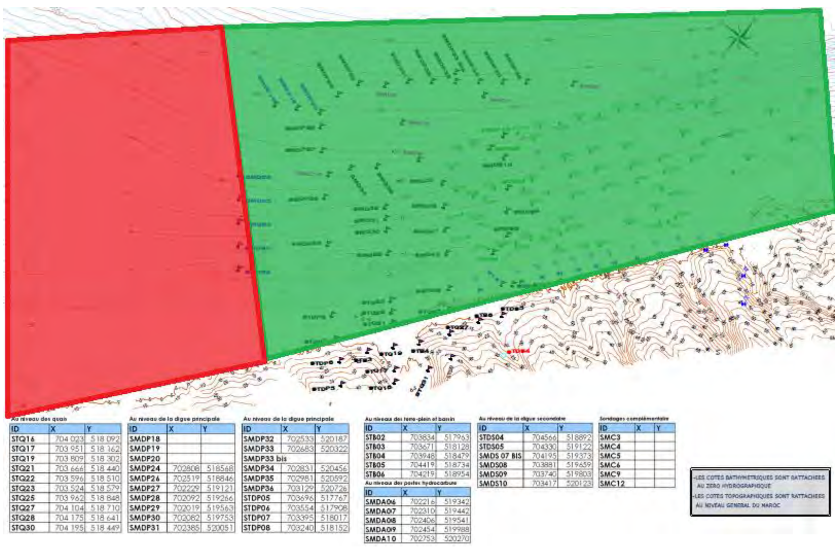


Figure 3.14. Borderline between a better and worse geotechnical conditions for construction.

Geotechnical profiles have been acquired form the borehole data provided (see borehole mesh and the different soil profiles in Appendix B). They give a valuable insight into the different strata soil layers in both longitudinal (away from the shoreline) and cross-sectional (parallel to the shoreline) directions. The following main conclusions can be drawn:

1. There is a substantial layer of fine, brown sand close to the coast. The seabed is sandy up until 8 m depth.
2. The seabed gradually transforms into layers of different types of clay. Between 8 and 18 m depth the bed can still be classified as sandy, however, with a significant fraction of fine contents (silts and clay);
3. Further offshore, beyond about 20 m depth, the seabed is composed of clay (from the given borehole data). The height of the clay and sandy clay layers is, on average 10-12 m.

Generally, the seabed becomes finer with distance offshore; hence, careful consideration of the soil reinforcement must be done, as the breakwater would be built in deep water to minimise the dredging. The considered soil improvement techniques are described in Appendix F.

3.3.2 Seismic data

The area was identified from the very beginning to be of low seismicity; hence, maximum ground accelaration equal to 0.18g must be considered in the design.

3.4 Sediment Transport

The aim of the study is to manage the sediment transport in order to reduce sediment deposition in the port and its channel. The results of hydro-sedimentary study using a software from Artelia shows the presumed sediment transport along the coast at the potential location of the port, as shown in Fig. 3.15.

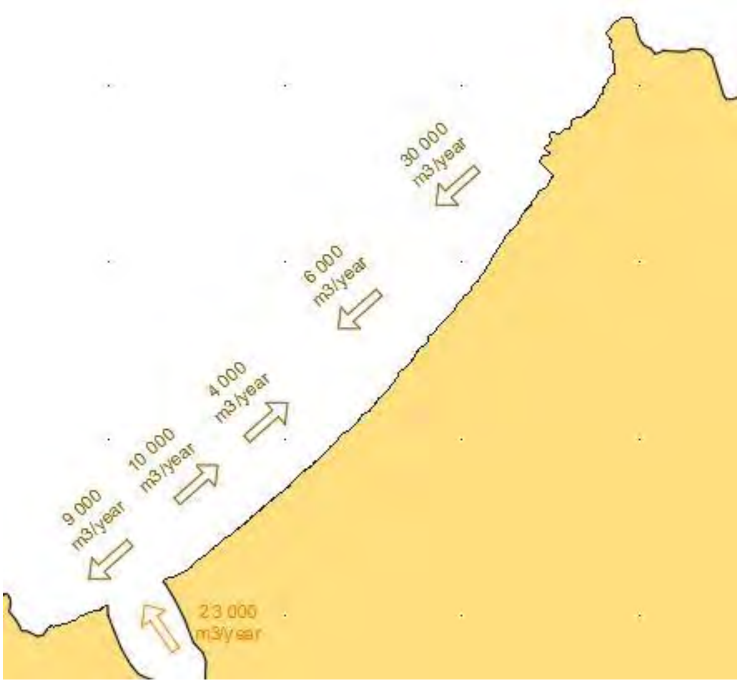


Figure 3.15. Sediment transport along the coast.

The following conclusions can be drawn from the above figure:

1. The amount of sediment is relatively small; the preventative solutions should be adapted accordingly;
2. The two sources providing the sediments are as follows : the cape at the Northeast and the Oued at the Southwest. The sediments move in the following two directions: from the NE to the SW and from the SW to the NE;
3. The sediment transport is mainly carried out by bedload along the coast and not by suspension.

Literature review of the problem suggests the following list of the probable technical solutions for managing the sediment transport:

1. Sand trap;
2. Submerged rubble-mound;
3. Detached breakwater;
4. Groin;
5. Sand by-pass.

The location and of the port can be added to Fig. 3.15. leading to Fig. 3.16 ;this allows the sediment transport to be observed relative to the port.

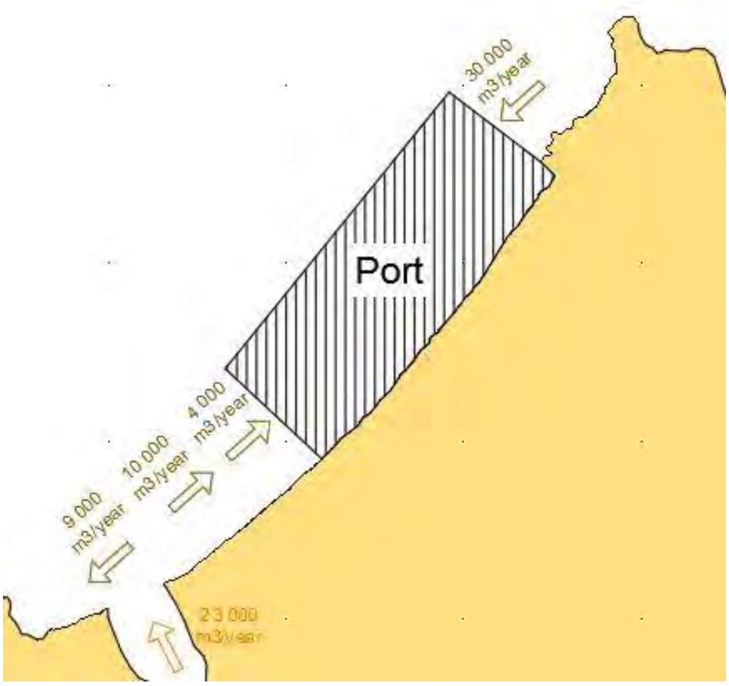


Figure 3.16. Sediment transport relative to the location of the port.

The following conclusion can be drawn from the figure above:

1. There is a very low arrival of sediments (4 000 m^3 / year) in front of the port entrance positioned in the SW direction;
2. The port is also affected by the arrival of larger quantities of sediments (30,000 m^3 / year) from the NE.

Several issues must be therefore considered. The following preventative measures have been proposed that will be put in place either separately or in combination with others:

1. Annual dredging will suffice, as the amount of the accumulated sediment is low – no special protection is needed;
2. Release the passage of sediments using solutions that will remove sediment from the port entrance (combination of a detached breakwater and a sand trap, sand by-pass system) ;
3. Block completely the sediment transport (submerged rubble mounds, breakwaters, groins).

The choice of the solution depends on the precise port location, hence, the design layout. The particular solutions are described in Chapter 5.

4. INITIAL LAYOUT PROPOSALS

4.1 Preliminary Layout Study

The design of the proposed port layouts strongly depends on the dimensions of the design ship that the port must accommodate, as well as the number of berths, quay lengths and storage area needed for the particular traffic requirements. This section therefore describes the analysis and its main findings for all the previously mentioned parameters.

4.1.1 Design Ships

First, different categories of the ships entering the port were investigated separately; the capacity of the ships (expressed in either TEU or DWT) was set as the common reference criteria. Additional data investigated were the length, draught and breadth of the ships. All three parameters are plotted against the capacity of the ships in Fig. 4.1, Fig.4.2, and Fig. 4.3 respectively for general cargo ships. The values chosen for the farther analysis correspond to the maximum value within the area with the largest concentration of the data.

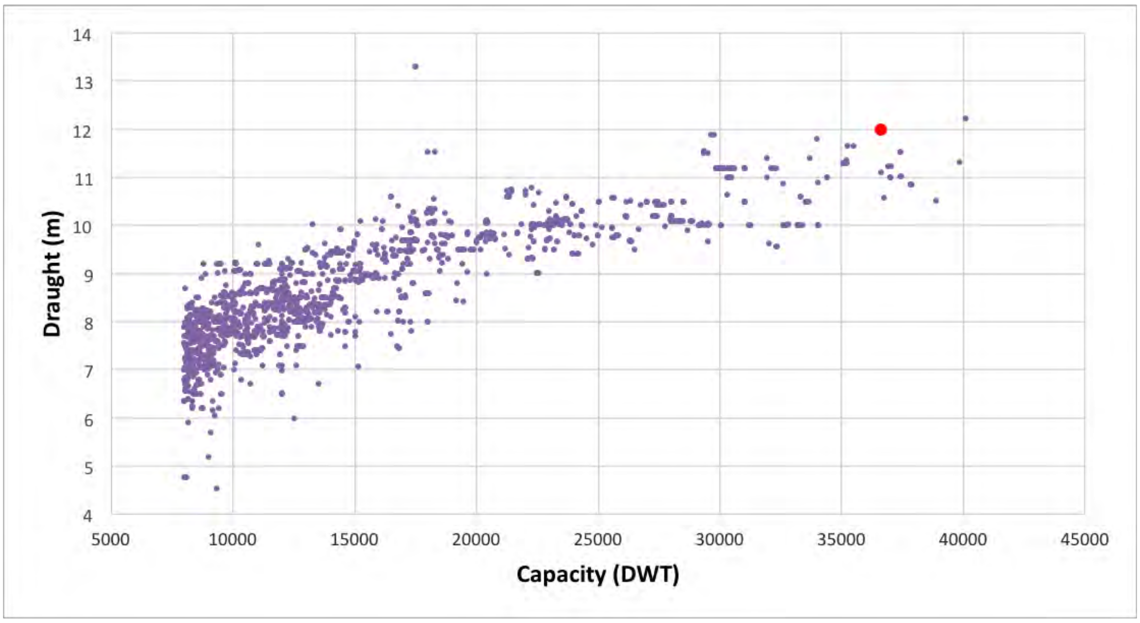


Figure 4.1. Ship draught vs. ship capacity.

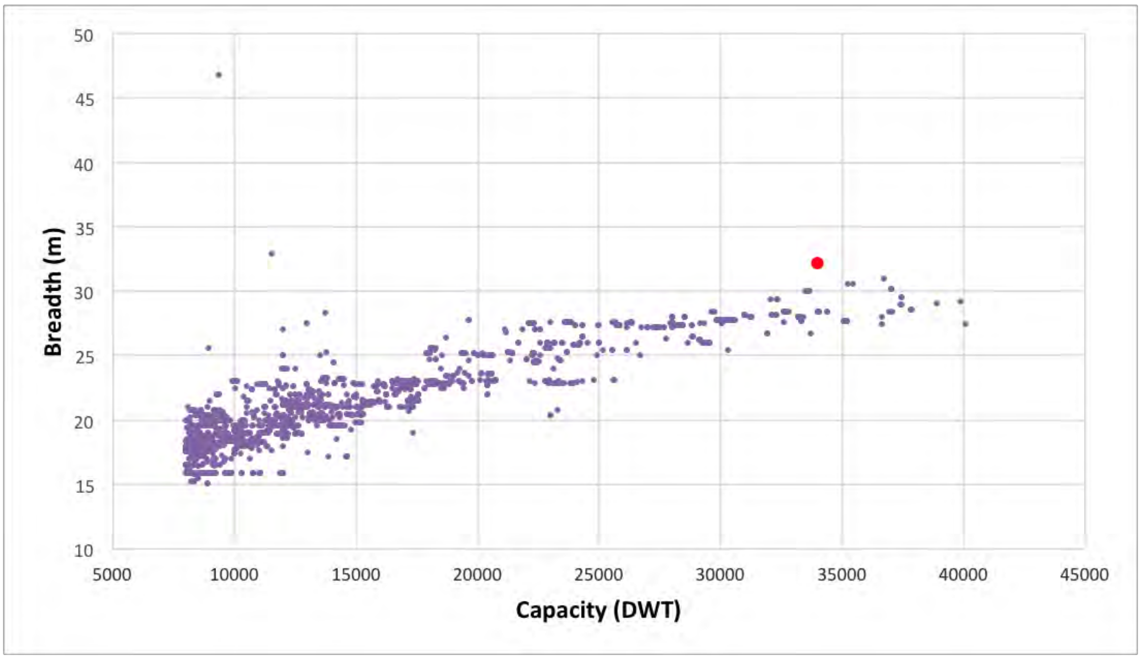


Figure 4.2. Ship breadth vs. ship capacity.

Additionally, the mean values for all the given parameters above were calculated to determine the number of berths necessary for the port. The anomaly values that correspond to 1% of all the values were disregarded in the analysis. This is shown for the previously mentioned general cargo characteristics in Fig 4.1, Fig. 4.2, and Fig. 4.3 respectively.

For the large crude oil tankers two sets of dimensions were chosen due to the two different concentrations of values that are obvious in the Fig.4.4, 4.5 and 4.6 for ship length, breadth and draught respectively.

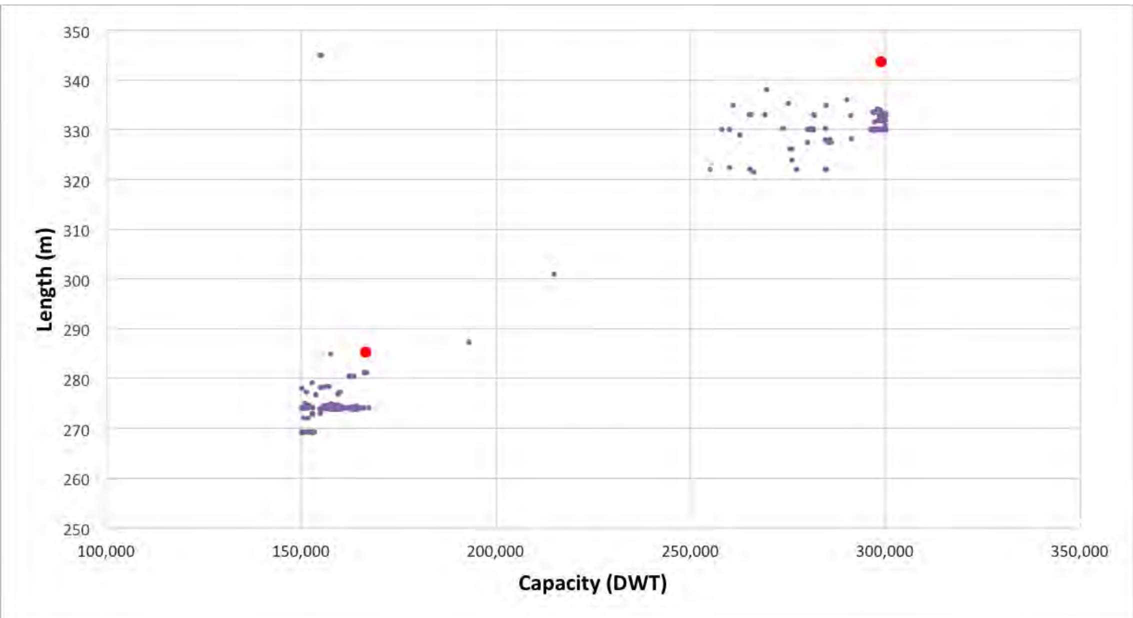


Figure 4.4. Ship length vs. ship capacity.

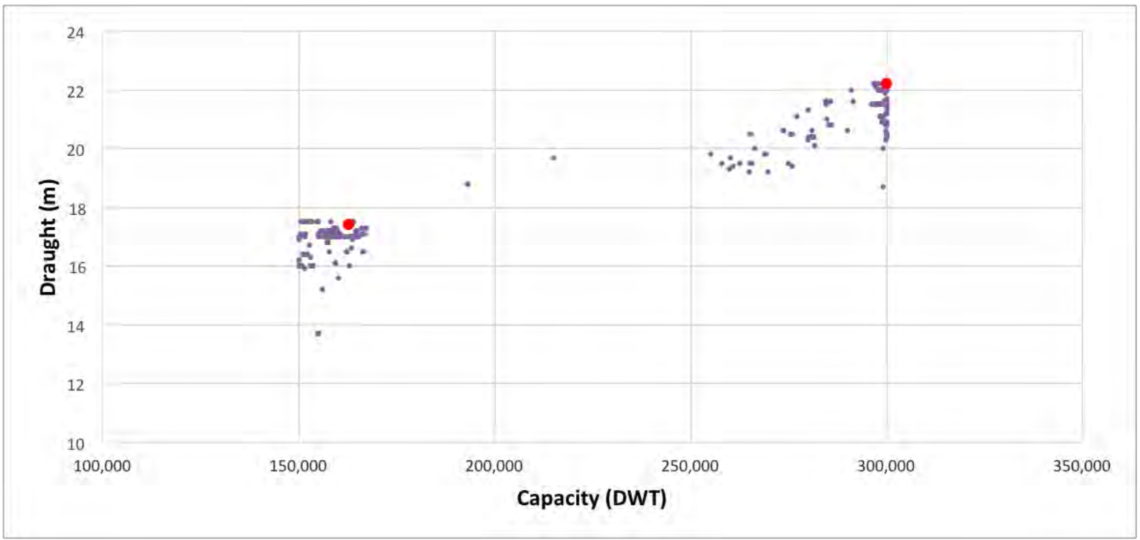


Figure 4.5. Ship draught vs. ship capacity.

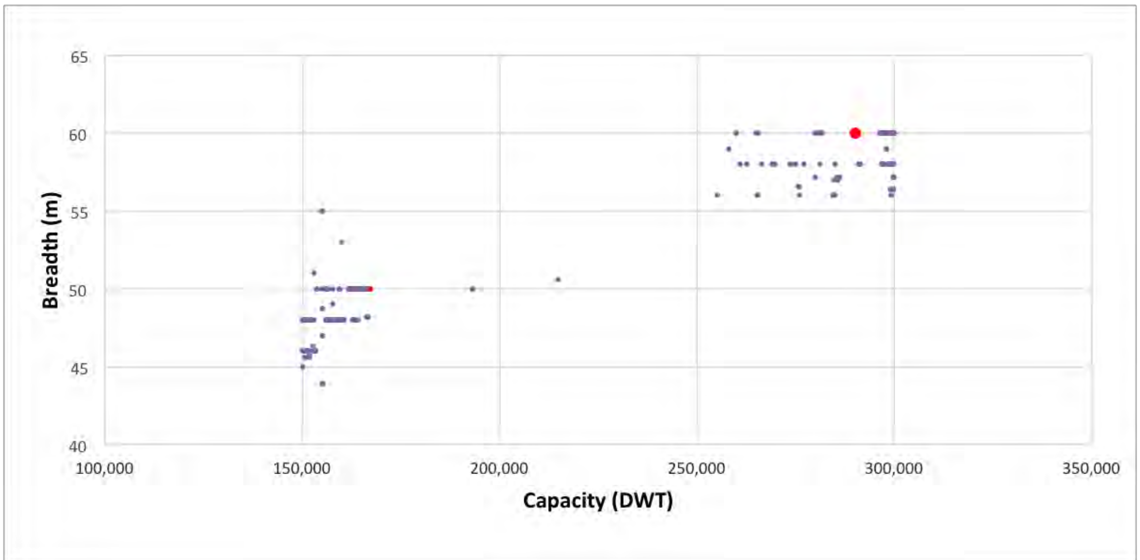


Figure 4.6. Ship breadth vs. ship capacity.

The design values for the different kind of ships are summarized in Table 4.1, 4.2, 4.3, 4.4, 4.5, 4.6 for container ships, coal carriers, general cargo ships, ro-ro ships, panamax and large crude oil tankers respectively.

Table 4.1. Design dimensions for a container ship.

CONTAINER SHIPS	Feeders	Mother ships
Length (m)	294	398
Draught (m)	13.65	16.02
Breadth (m)	32.3	54
Mean length (m)	250.17	372.71

Table 4.2. Design dimensions for a coal carrier.

COAL CARRIERS	
Length (m)	295
Draught (m)	18.6
Breadth (m)	48
Mean length (m)	289.9

Table 4.3. Design dimensions for a general cargo ship.

GENERAL CARGO SHIPS	
Length (m)	201.54
Draught (m)	12
Breadth (m)	32.21
Mean length (m)	138.56

Table 4.4. Design dimensions for a ro-ro ship.

RO-RO SHIPS	
Length (m)	229.8
Draught (m)	7.5
Breadth (m)	26.5
Mean length (m)	170.4

Table 4.5. Design dimensions for a Panamax tanker.

PANAMAX TANKERS	
Length (m)	195.3
Draught (m)	13.5
Breadth (m)	32.26
Mean length (m)	171.95

Table 4.6. Design dimensions for a large crude oil tanker.

LARGE CRUDE OIL TANKERS	1st group	2nd group
Length (m)	347.3	285.4
Draught (m)	22	17.5
Breadth (m)	60	50
Mean length (m)	331.15	272

4.1.2 Number of Berths and Quay Lengths

Availability of an adequate berth capacity is of major significance in the operations of a port. Too few berths lead to delays, due to increased waiting time within the port. Too small berths limit the maximum ship size that the port can accommodate, on the other hand too many berths lead to increasing building and maintenance costs of the port.

The calculations of the number of berths required are based on the queuing theory. The method is outlined in accordance with R. Groenveld (2001). The queuing theory describes the waiting system in front of a port. This theory takes 3 different factors into account, namely:

1. The arrival pattern of vessels;
2. The service times at the terminal;
3. The service system at the terminal.

The arrival pattern and the service times can be described by the arrival rate, λ, and the service rate, μ. The two factors are expressed as the following statistical distributions:

1. The negative exponential distribution (M);
2. The Erlang distribution (Ek);
3. The deterministic distribution (D);
4. The general distribution (G).

Each transshipment is assigned to a specific distribution that takes the form of letter/letter/number. The first letter corresponds to the inter arrival time distribution; the second letter - to the service time distribution, and the number corresponds to the number of servers see .

The delivered scope of works provided not enough information to do the calculations for the queuing theory. Therefore, some assumptions are made to get all the required parameters. All assumptions are explained later in this chapter. The procedure for the queuing theory is described in Appendix E.

Main functions of the designed port

The port will have four different functions. For the calculations some assumptions were made for the amount of traffic within the port:

1. Import of crude oil and export of products (after refining in a refinery). Assumed is that 50% of the volume is crude oil and the other 50% are refined products.
2. Transhipment of containers consist of mother and feeder ships. The transhipment is or between those ships or the import and export to the Moroccan region. Assume is that 66% of the traffic comes from mother ships and 34% from the feeders.
3. Import and export of cargo for the region with 2 million tons by general cargo ships and 1 million by Ro-Ro ships.
4. Import of coal for an Electrical Plant in the industrial area in the future.

To calculate the availability of the port the operational hours are needed:
24 hours a day during 7 days a week
350 days per year (15 days off in case of holydays)
Efficiency factor (with respect to the nominal capacity of each crane) = 80%

The total number of hours the terminals operates during one year is: H= 24 * 350 * 0.8 = 6720 hours.

Table 4.7 gives the assumed (un)loading-rates of each transhipment.

Containers (moves/hour)	37.5
Oil (tons/hour)	20 000
Coal (tons/hour)	2000
Cargo (tons/hour)	60
RO-RO (tons/hour)	2100

Table 4.8 gives the assumptions needed for the queuing theory.

Traffic typology	Berthing time (hours)	Distribution	Length 1	Length 2	Mean Length 1	Mean Length 2
Containers	2	M/E/N	400	295	375	255
Oil	2	M/D/3	350	200	335	175
Coal	2	M/E/2	295		290	
Cargo	0,5	M/E/4	205		140	
RO-RO	0,5	M/E/1	230		170	

See Table 4.9 for an overview of the amount of berths and quay lengths of each transhipment. The prior given assumptions are used in the steps. In the last row the quay lengths for each transhipment are given.

Table 4.9. Summary of the amount of berths and quay lengths.

Traffic typology	CONTAINERS [TEU]		OIL [Tons]		COAL [Tons]	CARGO [Tons] 3MLN	
Volumes [TEU, TONS]	3000000		25000000		7000000	2000000	1000000
Ship	MOTHERS	FEEDERS	LARGE CRUDE OIL	PANAMAX	BULK CARRIER	GENERAL CARGO	RO-RO
Average capacity	14240	3663	206777	36211	175462	14626	10041
% of income	66%	34%	50%	50%			
Arrival rate λ [Calls/year]	139,045	278,460	60,452	345,199	39,895	136,743	99,592
Service time [Hours]	46,19259259	20,28	14,33885	5,81055	91,731	82,25555556	5,781428571
Effective calls µ	145,4778704	331,3609467	468,6568309	1156,517025	73,25767734	81,69660948	1162,342476
Utilization ratio	0,955780721	0,84035334	0,128989046	0,298481531	0,544580166	1,673787782	0,085681868
N° berths	3	3	1	2	2	4	1
New utilization ratio	0,318593574	0,28011778	0,128989046	0,149240766	0,272290083	0.418446946	
% waiting time	3	2	0,7	1	8	3	8
Quay Lenght	1302	906	400	433	686	697	260
			380	230	330		

4.1.3 Storage Areas

Reference : Ports and terminals (H. Ligteringen)

The calculations are done according to the book of H. Ligteringen called Ports and Terminals.

Container Terminal

According to the Doc-11 Terminals Typology given in the scope of works the storage area of the container terminal can be calculated through the following equation:

$S_{tot} = S_{st} + S_{tb} + S_{ab} + S_{ech}$ = 120,4 ha

S_{st} : storage area (70% of the excluding quay)
S_{ctb} : surface of the technical buildings (10% of the excluding quay)
S_{sv} : surface of the administrative buildings (5% of the excluding quay)
S_{var} : Surface for exchange between train/trucks and terminal (15% of the excluding quay)

The storage area is calculated in the following way :

$S_{st} = \frac{N_{teu} \times T_{st} \times S_{teu}}{F_u \times 365 \times F_{oc}}$ = 843 000 m^2 = 84,3 ha

N_{TEU} = 3 000 000 TEU (yearly traffic in TEU)
T_{st}= 8 days (Assumption on the average time of one container in the terminal)
S_{TEU} = 8 m^2 (Surface occupied from one TEU)
Fu = 0,78 (for storage at various levels)
Focc = 0.80 (occupation factor of the terminal)

The value of the unit surface for one TEU in the storage area given a RMG/RTG system for 5 levels of storage (Port and Terminals, 1996).

according to TU Delft, « Port and Terminals – Planning & Functionnal design » 1996.

General Cargo Terminal

According to the Doc-11 Terminals Typology given in the scope of works:

The surface area for a general cargo terminal is determined from the length necessary for the activities between the quay and the border of this terminal.

L = a + b + c + d = 40 + 60 + 50 + 30 = 180 m

a : Quay lanes for cranes
b : Warehouses
c : Short term storage and truck lane
d : Parking

The lenght of the quay berth is 700m, so the total area needed for the general cargo terminal is 126 000 m^2 (126 ha).

Ro-Ro terminal

The area needed behind the berths for the ro-ro is about 10 hectares for one berth.

Coal terminal

The area for the storage of coal does not need to be just behind the quay or jetty of the coal terminal, because the raw material is moved with conveyors.
According to TU Delft, « Port and Terminals – Planning & Functionnal design » 2014, an estimate of total lenght and width required for the stockpiles can be made with the following equation :

$V= b * \frac{1}{2} * h * l * m_b$

Whereby:

- V= 1 167 000 tons which is the equivalent of 898 000 m3.
  The assumption is made that the storage capacity is needed for 2 months which is 1/6 of the annual importation (7 000 000 tons).
- b : width of stockpile = 20 m (angle of repose of bulk material between 35° and 40°).
- h : height of stockpile = 7,5m
- l : total length of stockpile
- m_b = 1

The total length of the pile is l = 12 000m.
The needed surface for the coal terminal is 240 000m^2 . Considered is that this storage area represent 70 % of the coal terminal. It is needed to take into account conveyors, technical buildings, administrative buildings, exchange between trucks/terminal, parking.
The total surface is then increased to 343 000 m^2 , given almost 34,3 ha.

Oil terminal

The transferred oil volume for one year is 25 million tons, which represent 20 million cubic meter. The operational capacity is, generally, in the order of one month consumption, out of strategic storage. The storage capacity of the terminal must to be almost 1 670 000 m^3.

In case of damage, the oil have to be contained within the bund (generally 5m high bund for a useful height of 4m). If the product is stocked in 100 000 m^3 tank, there is need for 17 reservoirs.

The needed surface is 25 000m^2 per unit so there is 425 000m^2 required. But it is considered that this storage area represent 70% of the oil terminal. Other things to take into account are pipes, technical buildings, administrative buildings, exchange between trucks/terminal, parking.
The total storage area for the oil is 607 143 m^2 , therefore almost 61 ha.

Table 4.9. Surface area required.

Type of terminal	Surface area required (m^2)
Container	1 204 000
General cargo	126 000
Ro-Ro	100 000
Coal	343 000
Oil	610 000

4.1.4 General Conclusions

Two preliminary layout concepts were developed for the project location. Different options were discussed according to the previously determined design criteria and design conditions. The common aspects for both layouts are as follows: Anchorage area is located just outside of the port. The vessels then can wait there if there is a waiting time to approach one of the berths inside the port. The required length and width of the entrance channel is indicated with the black rectangle at the entrance of the port, followed by the required turning circle specified by the black circle. The storage areas and design ships of each shipment category are indicated with their specific colour.

As the major design criterion requested by the client is to minimize dredging and to have the port entrance looking at the SW, both layouts are located at the more northern side of the bay, however, not reaching the small cape, farther from which a mountainous region begins. The seabed soil conditions are better at the location than more to the S, where there is very weak soil strata layer present. Furthermore, there is sediment transport that must be addressed if the port developement is moved farther to the S. Additionally, the cape can be utilised in the port design to reduce the breakwater length. These constraints, including the fact that the water depth is greater farther in the N (dredging must be minimised), were the driving forces on the final location determination. Moreover, this lead to a conclusion that the main breakwater must be located at about -35 m depth. This subsequently led to the caissons use in the breakwater design. the breakwater will mostly consist of caissons.

Considering the parameters displayed in Section XX and XX, preliminary Layout A and Layout B were developed.

4.2 Layout A

4.2.1 General Layout

The main breakwater is designed as a curve (Fig. 4.7) in Layout A. The main reasons for this were to reduce the wave impact on the breakwaters well as avoid a substantial accumulation of sediments at that side of the breakwater; most of the sediments are directed pass the breakwater. A curved breakwater can also reduce the amount of reflection in the harbour. To minimize dredging, the port is partly constructed in the water, therefore, there is not that much dredging needed for the entrance channel. As for the preliminary layout development, the main aim was set to search a good ratio of dredging vs. filling. However, the quays are made at a depth of 10 meters; some dredging is needed there. The container terminal is mostly constructed in one line to provide the best efficiency for the cranes. Next to the container terminal, the ro-ro and general cargo berths are placed. On the inner side of the breakwater the oil berths are placed to create a sheltered location. The coal is constructed in the curved part of the breakwater. It may increase the reflection within the port, however, the increase is considered insignificant. An area for port offices is reserved between the container and ro-ro terminals.

Attached to the large breakwater there is a designated place for the tugboats. For this layout, a coastline of almost 4km length is needed.

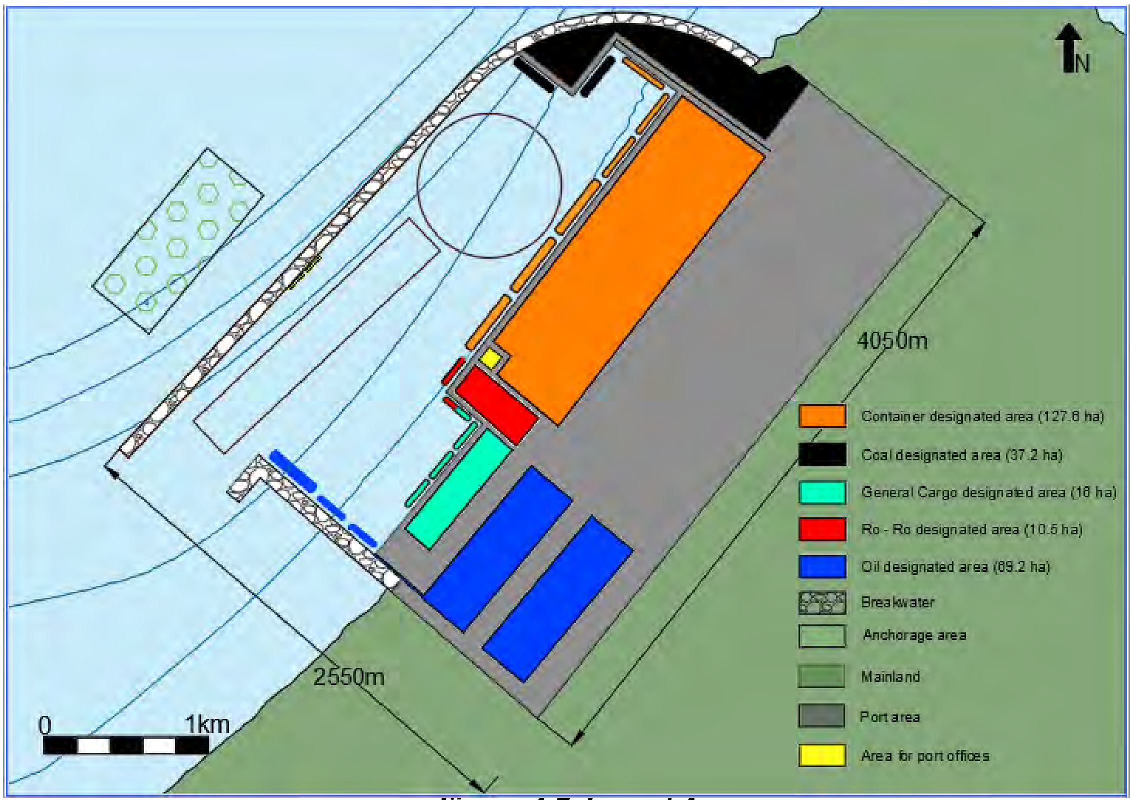


Figure 4.7. Layout A.

4.2.2 Solution for the Sediment Transport

Initially, to manage the sediments coming from the NE a combination of a sand trap and a detached breakwater is employed. The sand trap prevents the sediments from penetrating the port and its channel. In such way, the sediments accumulate upstream of the entrance. Furthermore, the sand trap would become unblocked during strong currents carrying the trapped sediment beyond the entrance channel, as the position of the detached breakerwater allows a narrowing of the passage and consequently an acceleration of the fluid.

In fact, the breakwater has a binary functionality. In addition to being used in sediment management, it protects the entrance of the port from waves coming from the NW direction.

Sediments from the SW will be blocked by the secondary port dike. In order to minimize the sediment entry into the harbor, a submerged rubble mound may be constructed along the channel. Care must be taken not to extend the submerged rubble mound too far as it would cause blockage of sediments coming from the NE within the channel. There will then be an accumulation of sediments between the coast, the secondary breakerwater and the submerged rubble mound. Considering future extension of the port, the collected sediment deposits can be used to extend the storage area of the containers. However, it must be noted that this sediment accumulation will certainly lead coastal erosion down the coast further to the south. The initial stages of the port protection against the sediment are depicted in Fig. 4.8 below.

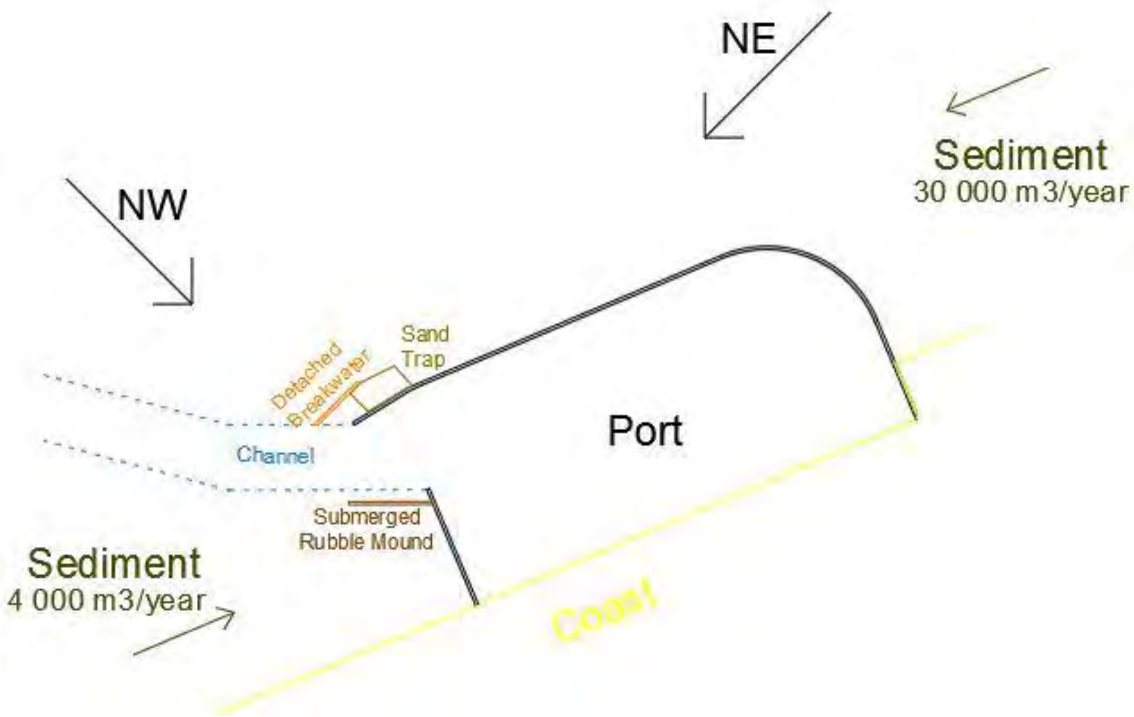


Figure 4.8. Schematic diagram of protection against the sediment deposition at the entrance of the port.; Layout A.

4.3 Layout B

4.3.1 General Layout

The main breakwater is constructed at 90 degrees angle (Fig. 4.9) with the breakwater at the NE. This allows for more space inside the harbour, easier transportation of the goods within the port, as well as simpler construction methods (what is the difference?). Attention, however, must be paid to reflection by the quay walls. The rectangular shape can also cause some reflection and therefore resonance, which would lead to higher wave heights inside the harbour. The issue can be addressed with building a few of the quay walls on piles and sheet piles. The oil berths are located next to the main breakwater, as the water depths are larger here; the largest oil tankers can berth close to the entrance, leading to less dredging is required. In this design, the quay lengths are all straight to provide the best efficiency to handle the cargo. The maximum depth of the longest quay wall is around 10 meters, hence, reasonable amount of dredging is needed in the area. An area close to the entrance is designated for the tugboats. To sum up, the layout is relatively simple and therefore very efficient. All the berths are made next to each other, except from the coal berths. Also, the storage areas are made in simple rectangular shapes to increase their efficiency. For this layout, a coastline length of around 3.8km is needed; advantageous in terms of the cost of the breakwater.

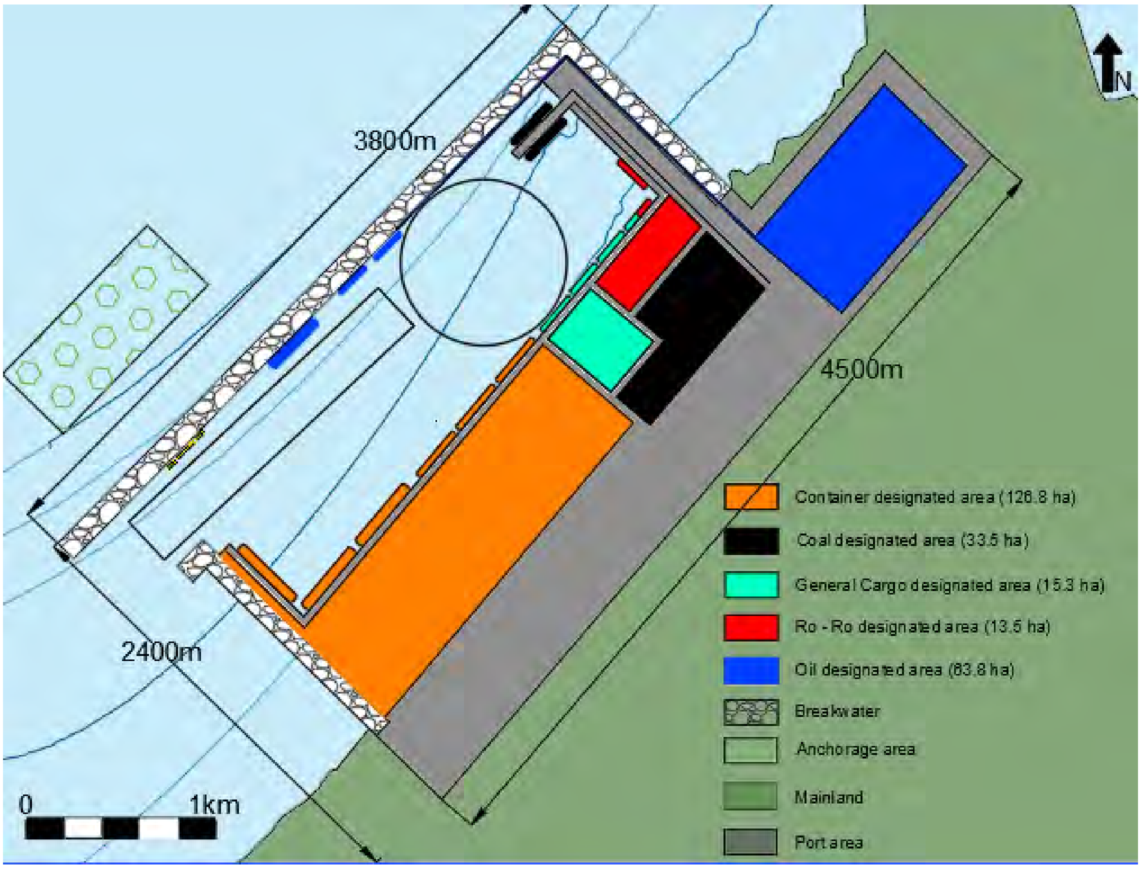
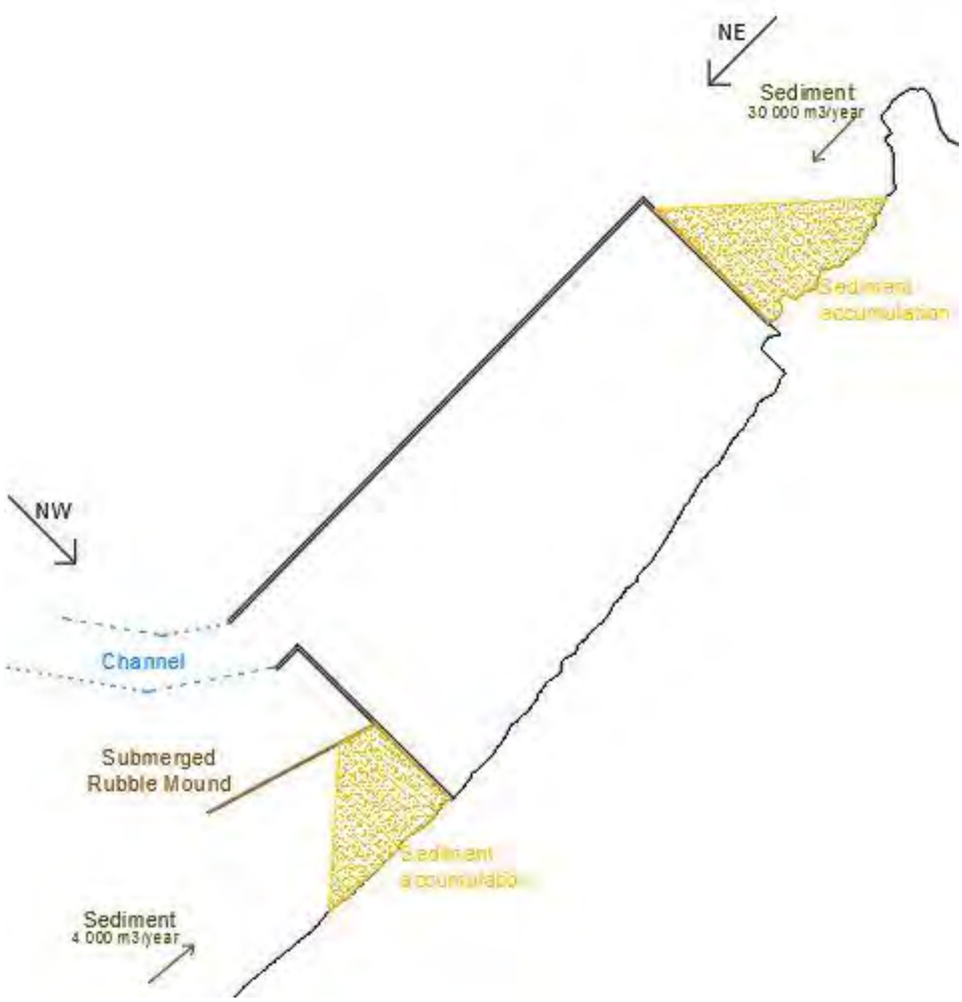


Figure 4.9. Layout B.

4.3.3 Solution for the Sediment Transport

The main solution for the sediments coming from the SW is as per Layout A. However, the management approach of the sediments coming from the NE has changed. The main breakwater at a right angle acts like a groin. In fact, we have a bedload transport close to the coast, while the breakwater extends offshore more than 1 km. Hence, the sediment will accumulate against the breakwater. The sediment may be reused after dredging. The sand can be either cleaned and sold, or it can be used for the the potential port extension. The initial stages of the port



protection against the sediment are depicted in Fig. 4.10 below.

Figure 4.10. Schematic diagram of protection against the sediment deposition at the entrance of the port.; Layout B.

4.4 Comparison Between the Two Layouts

4.4.1 Advantages/ Disadvantages

For both layouts the advantages and disadvantages are summarised in Table 4.1 below. Possible solutions for the identified drawbacks are included in the same table.

Table 4.1. Advantages, disadvantages and possible solutions for both Layout A and B.

<table>
<tr><th colspan="4">LAYOUT A</th></tr>
<tr><th>CRITERIA</th><th>PROS</th><th>CONS</th><th>SOLUTION</th></tr>
<tr><td>Aesthetics & Environmental Impact</td><td>Port looks organised and clear</td><td>A lot of dredging needed. Especially at the berths for the oil ships</td><td rowspan="2">Place the oil berths at another location, maybe next to the breakwater</td></tr>
<tr><td>Health & Safety</td><td>Enough space to navigate into the port</td><td>Oil berths are not separately placed from the other terminals</td></tr>
<tr><td rowspan="3">Costs</td><td rowspan="3"></td><td>The length of the port is quite long</td><td rowspan="2">Try to place the berths more efficiently</td></tr>
<tr><td>Huge amount of fillings</td></tr>
<tr><td>A round breakwater it is hard to construct by using caissons</td><td>Make the breakwater with a 90 degrees angle</td></tr>
<tr><td>Geographical matters</td><td></td><td>Huge amount of dredging needed in the mountain area</td><td>Rearrange the storage areas</td></tr>
<tr><td rowspan="2">Storage areas</td><td>Enough space for future expansion of the storage areas</td><td rowspan="2">The storage area of the coal is not efficient</td><td rowspan="3">Try to place the berths more efficiently</td></tr>
<tr><td>There is space for port offices</td></tr>
<tr><td>Further expansions</td><td></td><td>The placement of the container berths is not efficient in this layout</td></tr>
</table>

<table>
<tr><th colspan="4">LAYOUT B</th></tr>
<tr><th>CRITERIA</th><th>PROS</th><th>CONS</th><th>SOLUTION</th></tr>
<tr><td rowspan="2">Aesthetics & Environmental Impact</td><td>Clear and efficient division of berths and storage areas</td><td rowspan="2">Straight breakwater can create sedimentation at the left</td><td rowspan="2">Check the amount of sediment if it is a risk or not</td></tr>
<tr><td>Not much dredging needed</td></tr>
<tr><td rowspan="3">Health & Safety</td><td>Enough space to navigate into the port</td><td rowspan="3"></td><td rowspan="3"></td></tr>
<tr><td>Oil berths are placed separately from the other terminals</td></tr>
<tr><td>Good protection against the waves</td></tr>
<tr><td>Costs</td><td></td><td>The length of the port is still quite long</td><td>Try to place the berths more efficiently (use of peers/jettyes)</td></tr>
<tr><td>Geographical matters</td><td></td><td>Storage area of the oil is at mountain area (not flat)</td><td>Place the storage area of the oil somewhere else</td></tr>
<tr><td>Storage areas</td><td></td><td>The storage area of the coal is blocking future expansion of the ro-ro and general cargo</td><td>Place the storage area of the coal further away</td></tr>
<tr><td>Further expansions</td><td>Enough space for future expansion of the container shipment</td><td></td><td></td></tr>
</table>

4.4.2 Multi-criteria Analysis

To define the leading layout of the port development, a multi criteria analysis was performed. Different criteria are specified and graded from 1 (very bad) to 5 (very good). The scores are shown in Table 4.2 below. The layout with the highest score was then identified as the better layout.

Table 4.2. Scores for the multi-criteria analysis of the different layouts proposed.

Criteria	Layout A	Layout B
Health and Safety	3	5
Construction	3	4
Navigability inside the port	2	4
Total space needed for the port	2	3
Costs	2	2
Amount of dredging/filling	2	4
Environmental impact	3	2
Aesthetics	3	2
Further expansion	3	4
Geographical problems	4	2
Total	27	32

The health and safety is identified to be better in Layout B, as the oil and container berths are placed in a slightly more hazardous area in Layout A. Also, the construction of the curved breakwater is a more difficult process in Layout A. Considering the navigability, Layout B is again superior to Layout A.

The location of the containers in Layout B is better thought of, and the quays are arranged in a straight line to allow the vessels for an easier approach to the berths. The area of Layout A is slightly larger than that for Layout B; it will not affect the costs significantly. The aim is to reduce, however, the total length of the port in the final layout as for the environmental impacts on the local area.

Dredging to filling ratio is lower in Layout B, hence desirable, even if there is a significant amount of dredging needed in the mountainous region, where the port is located, that would lead to a different position of the oil refinery in Layout B. Moreover, a substantial amount of dredging is needed for the oil berths in Layout

A. This has an environmental impact and will result in geographical problems for both layouts.

'Space for further expansion' is better addressed in Layout B; and also the container terminals are better placed in this layout. The only obvious drawback is the location of the coal storage area; this must be placed further away to give some space to the general cargo and ro-ro for farther expansions opportunities.

After thorough consideration of the advantages and disadvantages of both initially proposed layouts and the multi criteria analysis, it can be concluded that both layout concepts have their advantages and disadvantages. However, Layout B is overall the better layout to built on the final layout of the port development.

4.4.3 Recommendations for the Final Layout Development

According to Section XX (the topography) the area is not very suitable for structures at the right side of the bay, however, as mentioned before, it is considered less agreeable to build the port farther to the South due to the interaction with the river Kert, as well as the criteria to minimise the dredging in the project. Hence, the most appropriate location identified for the port is shown in Fig. XX. in the next chapter. In the area the coast is linearly increasing land inwards, but is quite rough looking in the parallel direction to the coast; dynamite explosions are needed here. This has already been successfully done in other projects previously, including in the surroundings of the current project location.

Additional improvements to be done on the final layout:

1. Make the breakwater straight to create larger space in the port. Furthermore, an angle of 90 degrees in the breakwater is easier to construct with caissons. By doing this, there has to be some attention to reflection and resonance in the quay wall design;
2. Shorten the port length to minimum by using peers for the berths of the general cargo. Peers are not, however, efficient for the container berths;
3. Move the turning circle towards the left side of the port; this will subsequently lead to a reduced total length of the port;
4. Create space for future expansion by placing the storage areas further into the land, especially the storage areas allocated for oil and coal. In fact, the

oil storage area and the refinery have to be relocated, as it is located in an uneven area in Layout B;

5. Place the container terminal at the left side of the port to create space for future expansion;
6. Shorten the width of the quay wall at the right side of Layout B – in the area designated for coal. It does not require that much space and large water depths; this will subsequently reduce the filling costs;
7. Reserve some space for port offices;
8. Split the breakwater into two physical parts. Vertical caisson breakwater is obviously more suitable for large water depths, while rubble mound breakwater is encouraged at a water depth of 15 m; it will ease the construction of caissons, as different caisson design will not be required for decreasing water depth;
9. Changed the end of the short breakwater to a round end as it should be on rubble mound breakwaters.

5. FINAL LAYOUT DESIGN

5.1 General Layout

The chosen area for the port is indicated in Fig. 5.1 for the final design.

Figure 5.1: The location of the designed port in the Bae bay.

All the recommendations made in Section 4.4.3 have been addressed for the final layout and are displayed in Fig. 5.2.

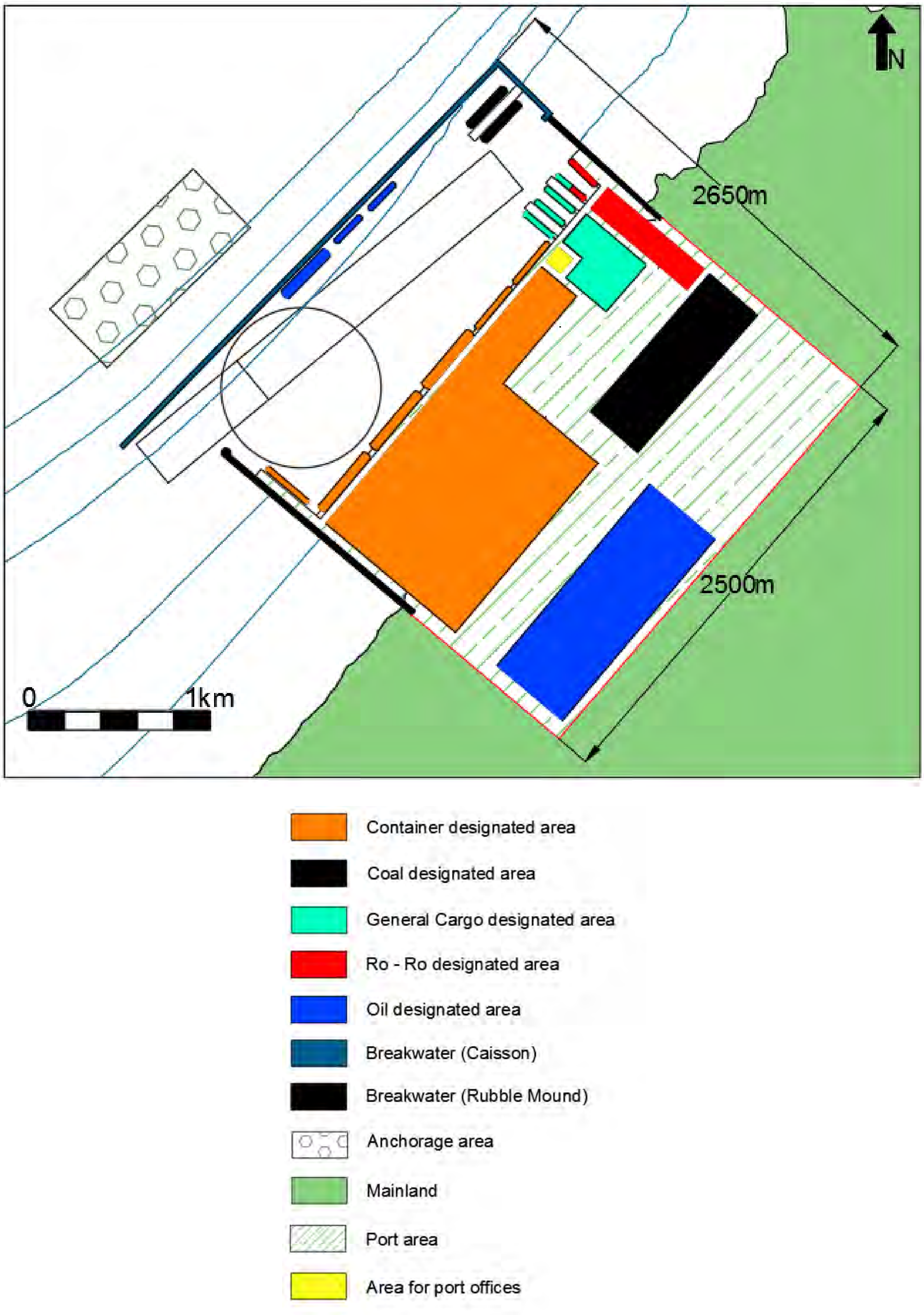


Figure 5.2 Final layout.

5.2 Main Element Design

5.2.1 Soil Reinforcement

The considered soil reinforcement techniques, as well as the all potential soil improvement measures adapted to the site have been thoroughly described in Appendix F and G. After a quick price study we conclude the best solution to reinforce the soil under the breakwater is the carrying out of rigid inclusions for almost 57 500 000 €, as shown in Fig. 5.3 below.

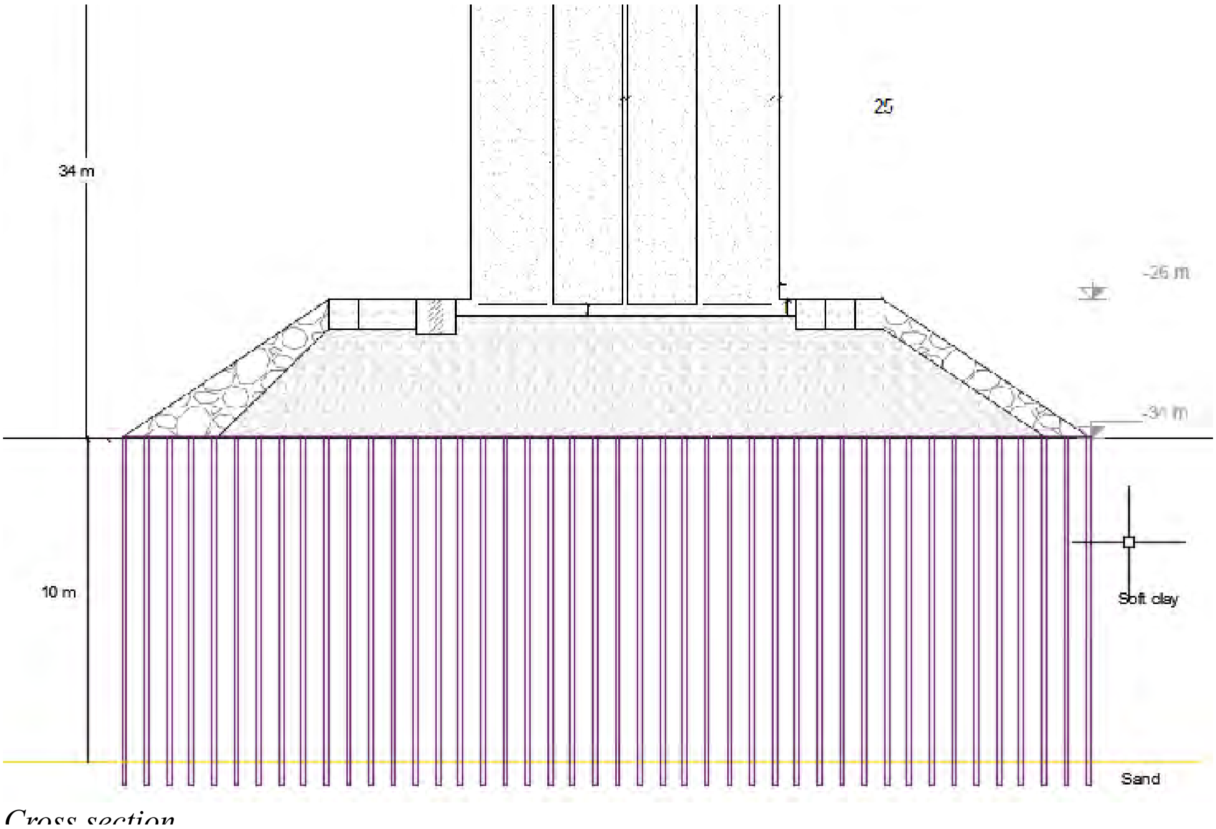


Figure 5.3. The cross section of the soil reinforcement chosen for the final design layout.

5.2.2 Quay Wall

Introduction

In order to have a good understanding about the geology of the land where the project is about to be implemented, various drilling operations and land surveys have been carried out. The results confirm the geotechnical layering shown in Appendix B. The soil parameters can be observed in the same Appendix B. Considering the available information, it has been concluded that the port will be compiled of two different docks: A and B (Fig.5.4). The different platforms will be located on varying soil strata layers (Fig.5.5).

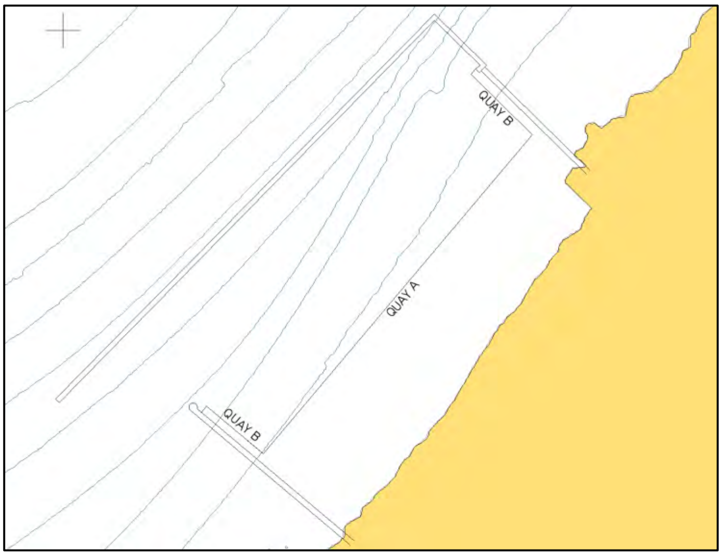


Figure 5.4. Layout for different quays in the final layout design.

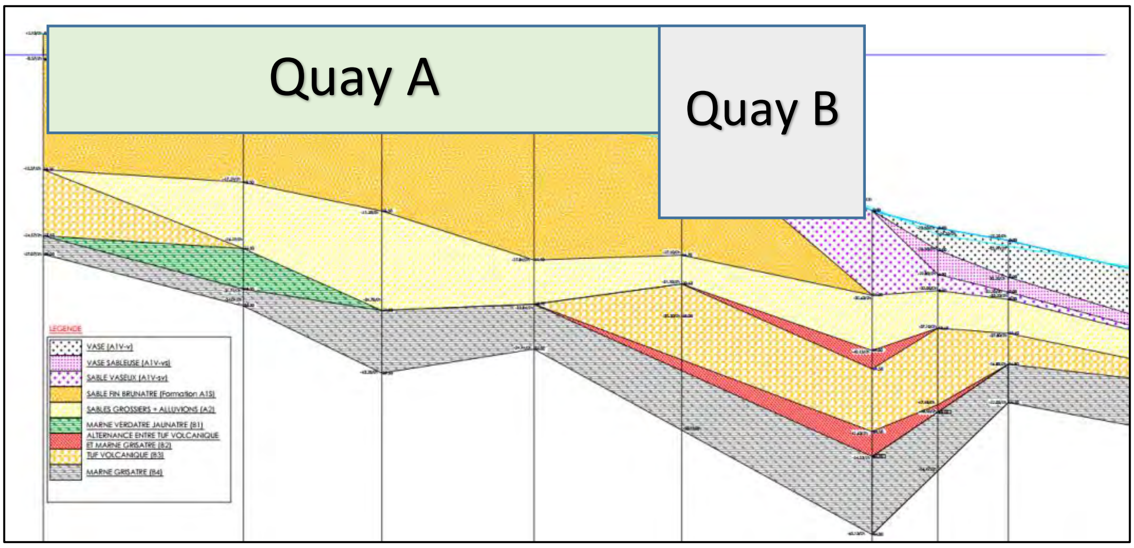


Figure 5.5. Varying soil strata profiles for bot quay A and B.

As limited amount of geotechnical data for different soil profiles is available, assumptions about these are made (the course sand layer has the Young's Modulus equal to 20 MPa) to continue with the design of the quay wall. To do this, K-REA software developed by TERRASOL is used. The concepts of the analysis procedure are described in Appendix C.

The results for Quay A are summarised in Fig. 5.6 and 5.7 for pile and sheet pile displacements respectively.

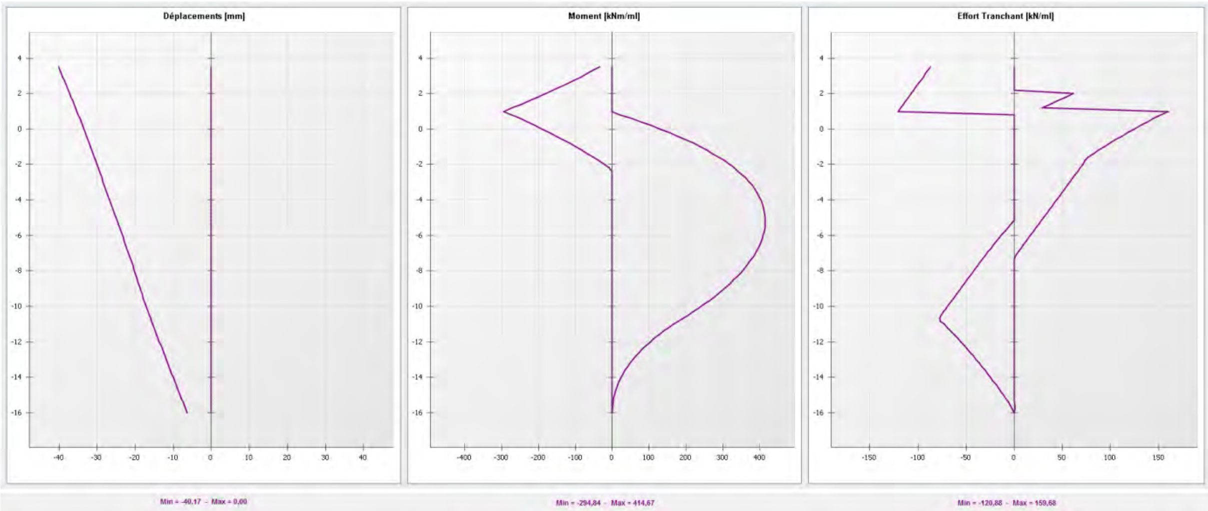


Figure 5.6. Displacement of the pile; Quay A.

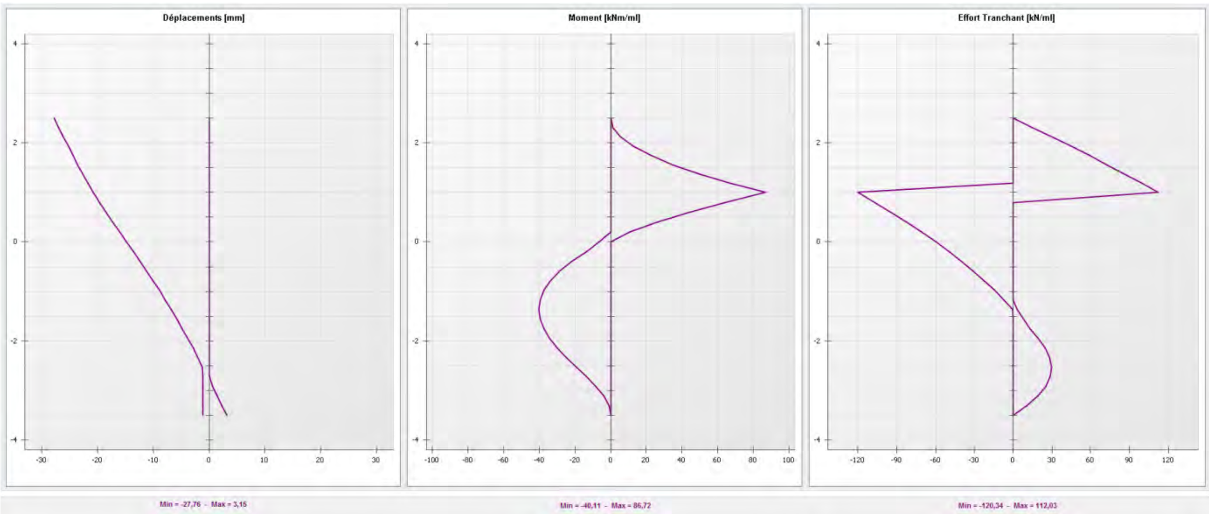


Figure5.7. Displacement of the sheet pile; Quay B.

It can be observed that the maximum displacements are 40.17 mm and 27.76 mm for the pile and sheet pile respectively.

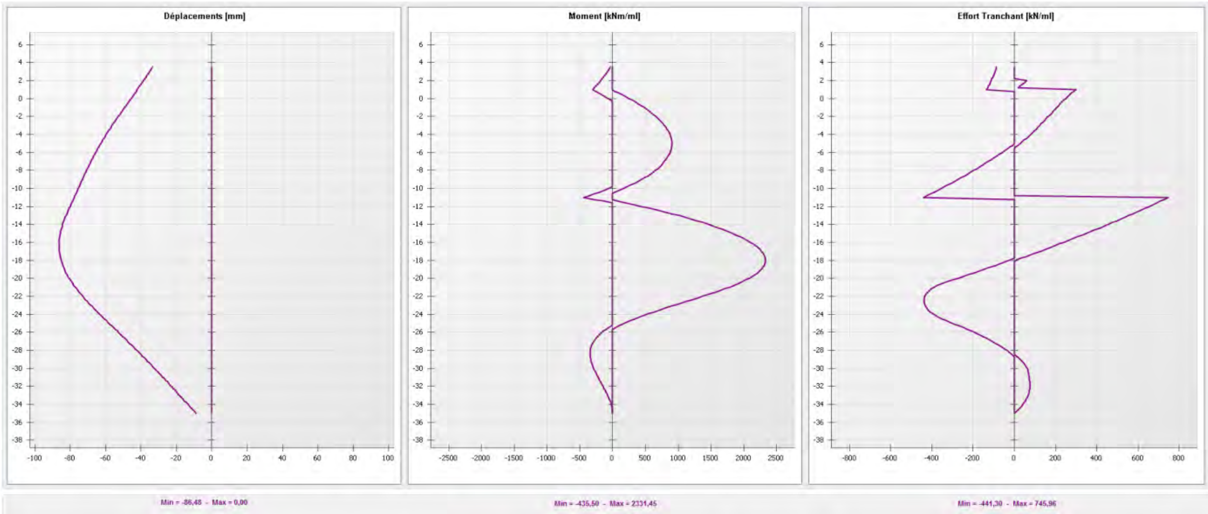


Figure 5.8. Displacement of the pile; Quay B.

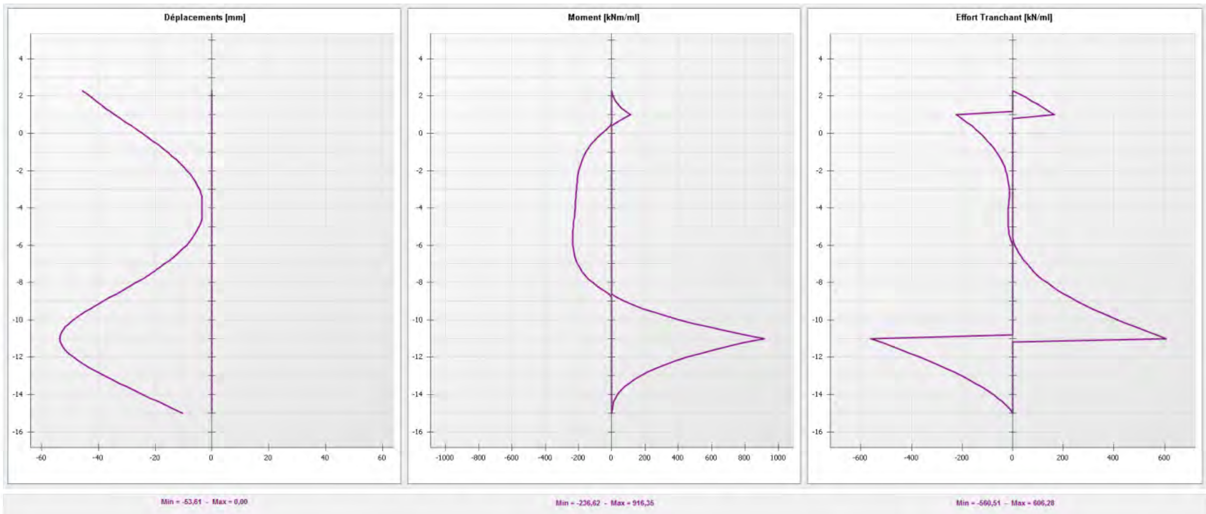


Figure 5.9. Displacement of the sheet pile; Quay B.

It can be observed that the maximum displacements are 86.48 mm and 53.61 mm for the pile and sheet pile respectively.

All the displacements are within the acceptable limits according to the design codes.

5.2.3 Rubble Mound Breakwater

Two rubble mound breakwaters were used in the shallower waters, on the northeast and the southwest of the port. At the NE, the breakwater extends to 15 m, and connects with a vertical breakwater. On the Southwest, the breakwater ends inside the entrance channel at 24 m.

Since both rubble mound breakwaters have a quay built on it, overtopping was minimized to the maximum. Since the breakwater to the north also requires a conveyor, pipelines and traffic, minimization of overtopping was crucial.

Since the breakwater to the north connects to the caisson breakwater, both breakwater crown walls were designed to match elevations.

For design purposes, we considered the northeastern breakwater extending to 15 m depth. The overtopping was designed for less than $q = 0.00002\, m^3/s/m$ and the slope of our breakwater was chosen to be $\alpha = 1{:}\,2$.

Armor units

The armor size was first designed using rough quarry stones. The Van der Meer equation below was used.

$$Dn_{50,armour} = \frac{H_S}{8.7 * P^{0.18} * (\frac{S}{N})^{0.2} * \xi_m^{-0.5} * \Delta} \quad \text{if } \xi_m < \xi_{mc} \text{ for plunging waves}$$

$$Dn_{50,armour} = \frac{H_S}{1.4 * P^{-0.13} * (\frac{S}{N})^{0.2} * \sqrt{tan\alpha} * \xi_m^P * \Delta} \quad \text{if } \xi_m \geq \xi_{mc} \text{ for surging waves}$$

Input

- Hs= 4.24 m propagated at 15 m for a 100-year return period
- P=0.4 porosity coefficient (for permeable breakwaters)
- N= 3000 number of waves (maximum number 7500)
- S= 4 damage level (range between 4 to 6)
- α= 26.57° slope of the breakwater
- Δ= 1.585 relative buoyant density

An armor stone of $Dn_{50} = 1.33m$ was obtained for the given input, of M_{50}=$(Dn_{50})^3 * 2650$= 6223 kg. As the armor layer is composed of two stones, the thickness of the layer was calculated to be 2.66m.

Filter layer

Similarly, the filter layer weights and dimensions are found and shown below.

$M_{50} = 414.15kg$ $\qquad\qquad Dn_{50} = 0.54m$

The thickness of the filter layer is calculated from the following equation

$t_f = 2 * Dn_{50filter} = 2.67m \approx 1.08m$

Crest and Core

Base on the allowable overtopping the crest elevation was calculated.

In the case of surging waves the following equation applies:

$$\frac{q}{\sqrt{g * H_s^3}} = 0.2 * e^{-2.6 * R_n}$$

$$R_n = \frac{R_c}{\gamma * H_s}$$

Input:

- $q = 0.00002\, m^3/s/m$ (allowable overtopping)
- $\gamma = 0.385$ reduction factor for surface roughness and shallow water conditions
- $R_c = 7.87m$ - crest elevation

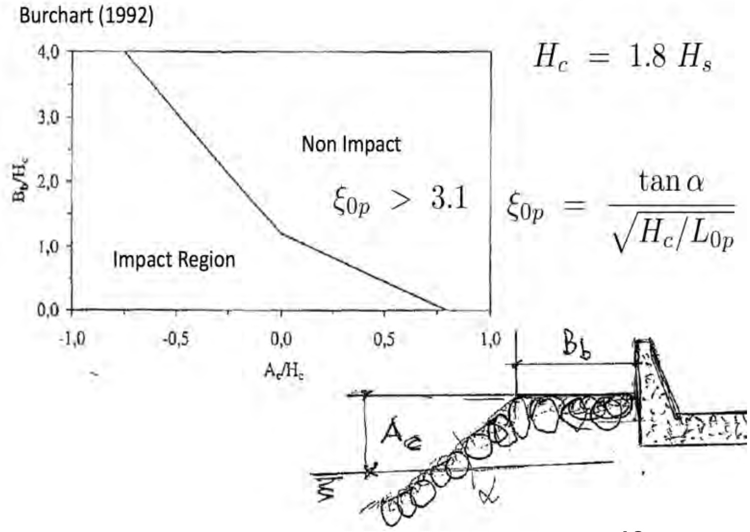
And the thickness of the core is calculated according to the following equation

$$t_c = R_c + h_{max} - t_f - t_a = 17.33m \approx 17.5m$$

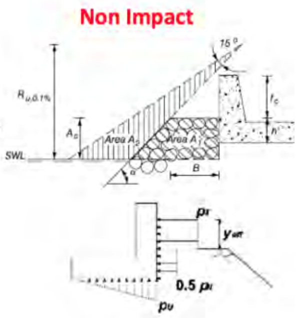
The maximum water level includes the expected sea water level rise (0.4m)

<u>Crown wall</u>

The crown wall was kept at the same level as the caisson crown wall. Overturning and sliding were checked and met the safety factor of 1.5. We are in non impact (□ $_{0p}$=3,7) and the model used is from Pedersen.



Pedersen (1996)



$$F_{h,0.1\%} = 0.21\sqrt{\frac{L_{0m}}{B}}\left(1.6\, p_m y_{eff} + A\frac{p_m}{2}h'\right)$$

$$M_{h,0.1\%} = a \cdot F_{h,0.1\%} = 0.55\,(h' + y_{eff})\, F_{h,0.1\%}$$

$$R_{u,0.1\%} = \begin{cases} 1.12 H_s \zeta_m & \zeta_m \le 1.5 \\ 1.34 H_s \zeta_m^{0.55} & \zeta_m > 1.5 \end{cases}$$

$$\zeta_m = \tan\alpha / \sqrt{H_s / L_{om}}$$

$$y = \begin{cases} \frac{R_{u,0.1\%} - A_c}{\sin\alpha}\frac{\sin 15^0}{\cos(\alpha - 15^0)} & y > 0 \\ 0 & y \le 0 \end{cases}$$

$$A = \min\{A_2/A_1, 1\}$$

$$y_{eff} = \min\{y/2, f_c\}$$

$$p_m = \rho_w g (R_{u,0.1\%} - A_c)$$

factor in the formulae	0.21	1.6	0.55	1.00
standard deviation σ	0.02	0.10	0.07	0.30

The crown wall dimensions are the following:

Crown height = 8 m, crown thickness = 2 m, crown width= 15 m (in order for the trucks, conveyors and pipelines to fit)

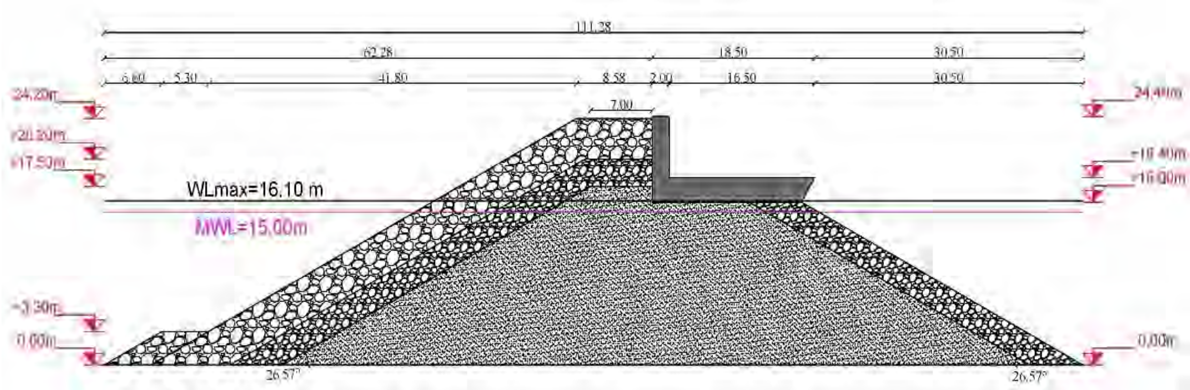


Figure 7: Cross section of the Rubble mound breakwater

Design review of the breakwater

After reviewing calculations and performing a flume test the stone size was found to be too small, and the armor layer of the breakwater was modified (see flume test section).

Tetrapods armor layer

The design assumed no damage of the armor layer. As provided in the equation below, the armor diameter was deduced.

$$N_s = \frac{H_s}{\Delta D_n} = \begin{cases} 3.7 & \text{no damage} \\ 4.1 & \text{failure} \end{cases}$$

The slope facing the sea was set to 1:1.33.

D_n =1,8 m, M_{50} = 14.0 tn

The armor layer is given as:

$t_a = n * k_\delta * Dn_{50}$=2,5 m

The resulting core thickness was determined:

$$t_a = 20.6\ m$$

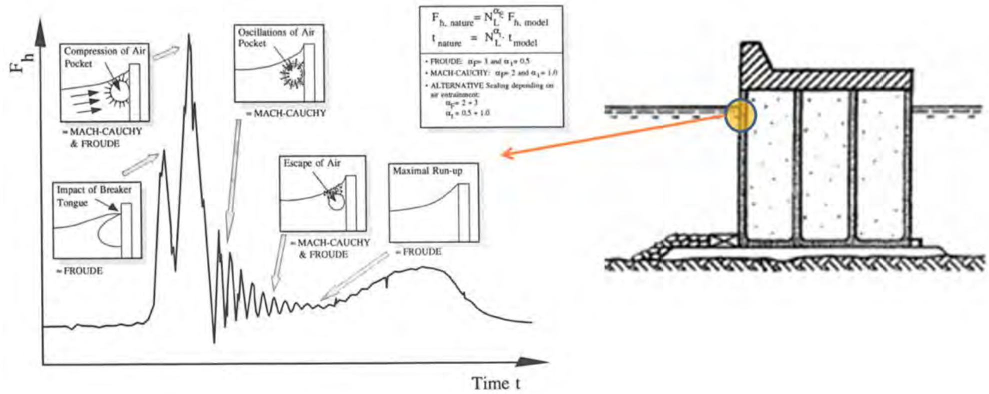
Advantages of the tetrapods versus the quarry stone armor

There are 2 main advantages of choosing the artificial blocks relative to using the natural stones. Firstly, since an interlocking factor is used for the artificial blocks, the angle of the slope can be reduced, the sub grade underlying the breakwater is also reduced and only one layer of armor is required over the filter layer. Secondly, the mass of the armor is considerably smaller.

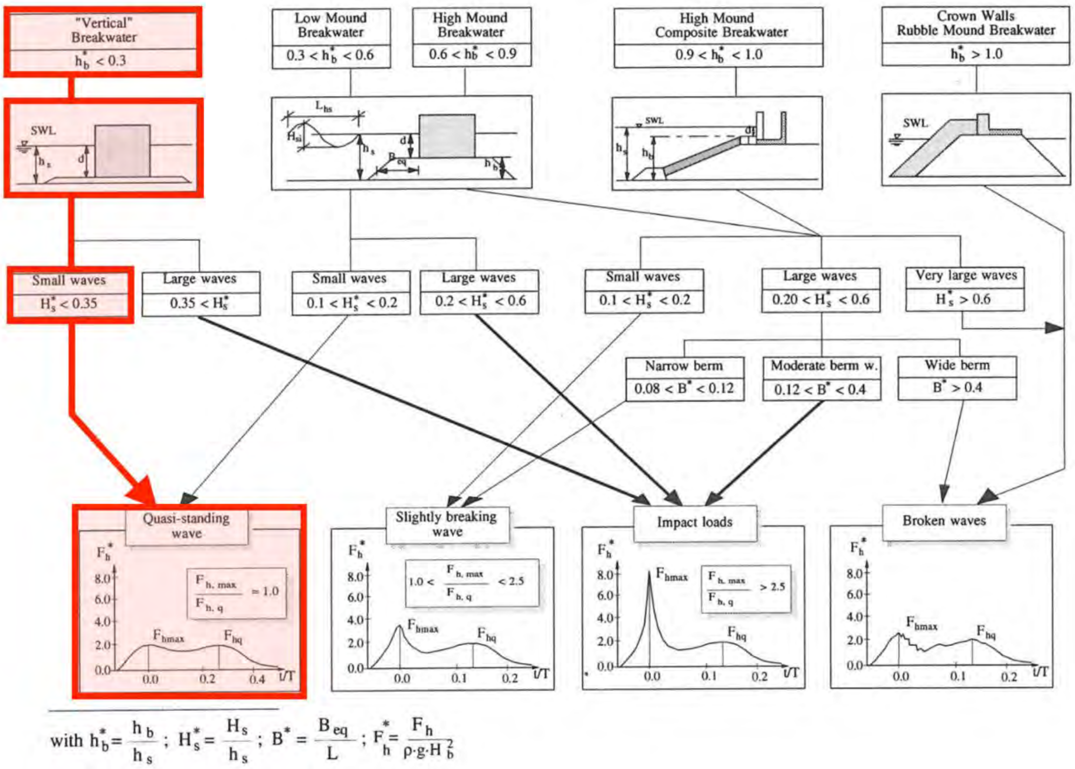
5.2.4 Caisson Breakwater

Goda Method

In order to find the dimensions for the needed caisson that will compose the vertical breakwater, the method used to find the pressure scheme is the Goda method.
The main advantage of this method is that inspite it’s a quick and simple for a pre-dimensioning work, it takes into account of the impulsive pressures that can be generated if the wave breakes in front of the breakwater.



The first step is to verify if the depth in front of the breakwater is deep enough not to make the wave break, in particular the depth to check is the one immediately in front of the breakwater, between the basement and the mean water sea level. This aim can be achieved with the following table which led to the assumption for the low mound to be smaller than one third of the local depth (8 meters).



Another advantage of the Goda method is that it deletes all of the uncertainties about the choice of the significant wave height. In fact the Hs can be calculated with the following formula:

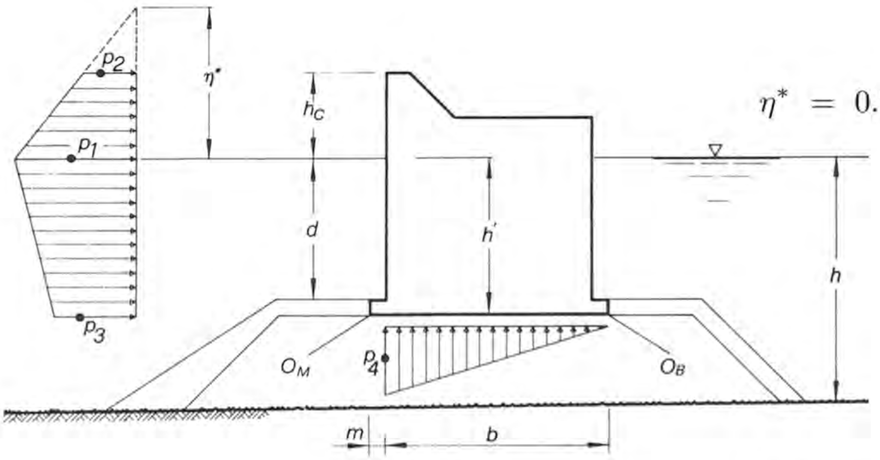
$$H = \min\left(1.8H_s; H_b\right)$$

$$H_b = 0.18\frac{gT^2}{2\pi}\left\{1-\exp\left[-\frac{3\pi^2 h_b}{gT^2}\left(1+15s_b^{4/3}\right)\right]\right\}$$

Hs (100y)	4,55	5Hs(100y)	22,75
T (100y)	11	L0(100y)	189,014331
hb (100y)	34,31	L (100y)	125,866302
sb (100y)	0,0133	K (100y)	0,04989421
Hb (100y)	20,1258912		
h	34	Hdesign	8,19

where all the parameters referred to H_b have to be calculated at a distance of $5H_s$ from the breakwater.

Once the design wave height is determined is possible to calculate the pressure on the breakwater wall according to Goda's formulas as shown below:



$$\eta^* = 0.75\,(1+\cos\beta)\,H$$

$$p_1 = 0.5\,(1+\cos\beta)\,(\alpha_1+\alpha_2\cos^2\beta)\,\rho g H \qquad \alpha_1 = 0.6+0.5\left(\frac{2kh}{\sinh(2kh)}\right)^2$$

$$p_2 = p_1\frac{\eta^*-h_c}{\eta^*} \qquad p_3 = \alpha_3 p_1 \qquad \alpha_2 = \min\left\{\frac{h_b-d}{3h_b}\left(\frac{H}{d}\right)^2, \frac{2d}{H}\right\}$$

$$p_4 = 0.5\,(1+\cos\beta)\,\alpha_1\alpha_3\rho g H \qquad \alpha_3 = 1-\frac{h'}{h}\left(1-\frac{1}{\cosh kh}\right)$$

In order to get the values of pressure, more assumptions were needed and they're highlighted in violet in the table below, furthermore the results calculated for the pressure led to a high similarity between the head and the trunk of the breakwater so in further calculations are taken into account only the pressures of the head caisson that, however, are a little higher.

Hbasement	8
h'	26
beta	45
hc	7
hr	2
Ro Water	1000
Ro Concrete	2200
Ro CAISSON	2100

BODY	
Eta	9,369
Alfa 1	0,626
Alfa 2	0,011
Alfa 3	0,507
P1	38553,615
P2	9749,373
P3	19530,495
P4	19433,765

PRESSURE	[N/m^2]
f1	68245,61
f2	100814,85
f3	247300,56
f4	507792,88
F0	924153,90
Arm	17,05

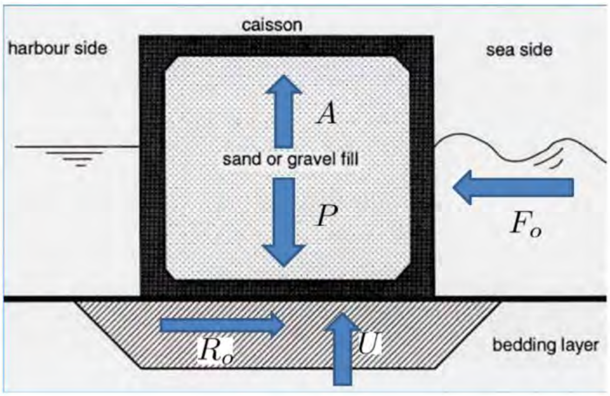
HEAD	
Eta	9,369
Alfa 1	0,626
Alfa 2	0,012
Alfa 3	0,507
P1	38564,366
P2	9752,092
P3	19535,941
P4	19433,765

PRESSURE	[N/m^2]
f1	68264,64
f2	100842,96
f3	247369,52
f4	507934,48
F0	924411,60
Arm	17,05

Security checks

The last and most important parameter to determine is the caisson width (B) that can be obtained from the security checks.
For this project, as it is in a preliminary phase, only 3 most important verifications are taken into account:

- Sliding check



$$\frac{\mu R_v}{R_o} > C_s \qquad \begin{aligned} R_o &= F_o \\ R_v &= P - A - U \end{aligned}$$

with

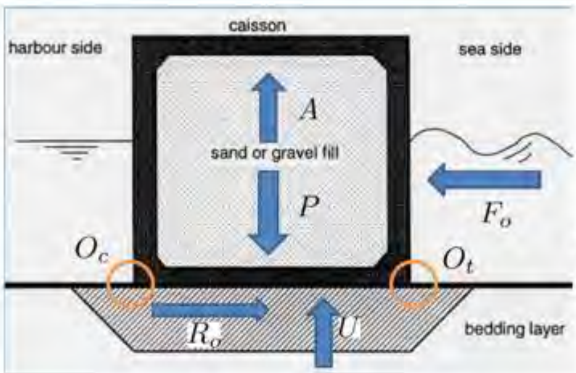
$$\mu = 0.6$$

Friction coefficient

$$C_s = 1.3$$

Security Factor

- Overturning check



$$\frac{M_s}{M_r} > C_r \qquad \begin{aligned} R_o &= F_o \\ R_v &= P - A - U \end{aligned}$$

With

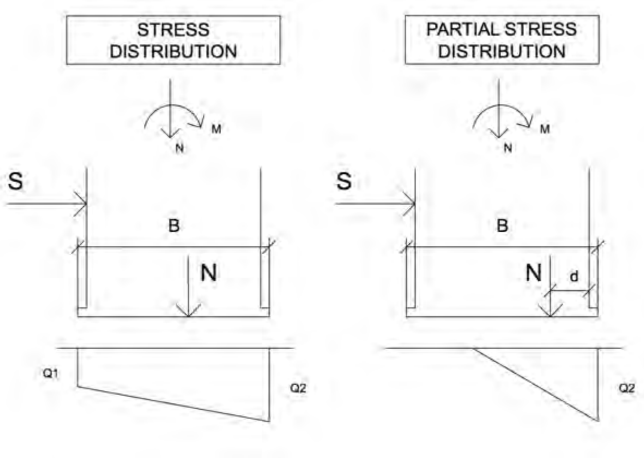
$$\begin{aligned} C_r &= 1.5 \quad H_D = H_{1/20} \\ C_r &= 1.3 \quad H_D = H_{max} \end{aligned}$$

Security Factor

O_c Turning point in crest conditions

O_t Turning point in trough conditions

- Soil bearing capacity check



Navier general formula $$Q_{1,2} = \frac{N}{B} \pm \frac{M\,{}^{B}/_{2}}{J}$$

Partial stress formula $$Q_2 = \frac{2N}{3d}$$

Each one of these checks resulted stricter than the previous one and below are shown the results:

SLIDING	
mu	0,6
Sfactor	1,3
m	1
U	67300,85
P	3581165
A	1511530,7
EQ sliding	0
b=	6,93

OVERTURN	
Sfactor	1,5
M turning	15757516,72
M resistent	6934257,785
Ratio	0,44
U	119761,6071
P	6843716,369
A	2888581,584
Mres*	23635690,84
Ratio*	0
b=	12,33

WEIGHTS	
Base	20
Ro CAISSON UW	1100
Ro CAISSON OW	2100
CAISSON WIDTH	20
H underwater	26
H emerged	2
Weight underwater [N]	112226400
Weight overwater [N]	16480800
Total Weight [N]	128707200
Weight underwater [N/lm]	5611320
Weight overwater [N/lm]	824040
Total Weight [N/lm]	6435360

BEARING CAPACITY	
W	6435360
U	194337,6476
N	6241022,352
F0	924153,8999
Arm	17,05074958
M	15757516,72
J	666,67
e	2,52
Verify if e < b/6 , else parzial stress and need to increase b or use partiartial stress formula	
B/6	3,33
Q1	558130,7508
Q2	85405,25
Qmax	600

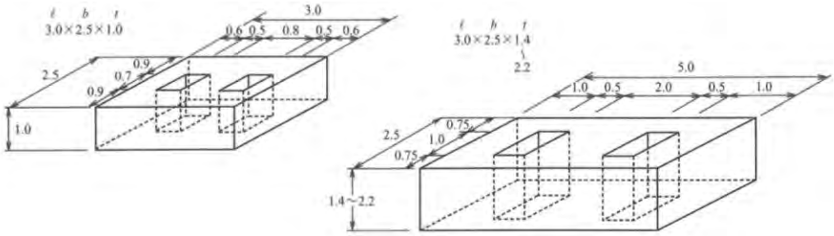
As can be seen from these tables the bearing capacity check is satisfied with a caisson width of at least 20 meters, taking into account the assumption of a basement bearing capacity of 600 Kn/m^2.
The next steps are just to determine the dimensions of other minor parts but not less important than the caisson itself.

Toe Rocks

Below are shown the formulas and the tables used to get the dimensions of the toe rock, essential to prevent scour phenomena of the low mound basement:

$$t = d_f \left(\frac{h'}{h}\right)^{-0.787} H_{1/3}$$

d_f 0.18 for the trunk
0.21 for the trunk



Required thickness of foot protection blocks t(m)	Dimensions ℓ(m)×b(m)×t(m)	Mass (t/unit)	
		Block with openings	Block without openings
0.8 or less	2.5×1.5×0.8	6.23	6.90
1.0 or less	3.0×2.5×1.0	15.64	17.25
1.2 or less	4.0×2.5×1.2	24.84	27.60
1.4 or less	5.0×2.5×1.4	37.03	40.25
1.6 or less	5.0×2.5×1.6	42.32	46.00
1.8 or less	5.0×2.5×1.8	47.61	51.75
BODY 2.0 or less	5.0×2.5×2.0	52.90	57.50
HEAD 2.2 or less	5.0×2.5×2.2	58.19	63.25

Protection armour

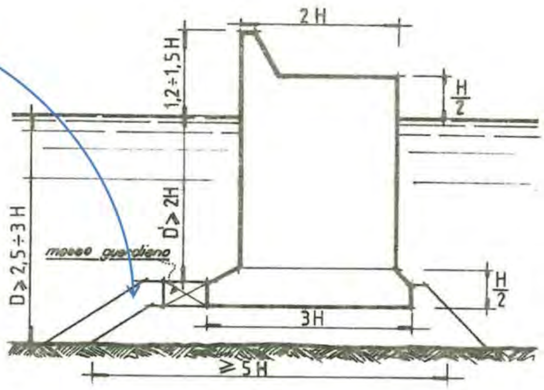
Another important part is the protection armour, which is different from the rock toe mainly for one reason: the toe rocks are holed in order to dissipate the underpressure in trough conditions that could lift them.
The values of Dn50 and M50 for the protection armour were calculated with the Madrigal and Valdes formula taking into account the possible execution of a quarry stone rock armour or a concrete block one.

Protection Armour

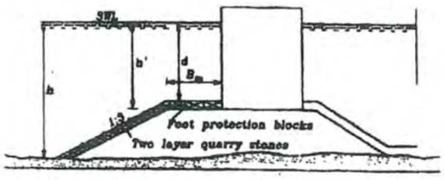
Madrigal & Valdes (1995)

$$N_s = \frac{H_s}{\Delta D_{n50}} = \left(5.8\frac{d}{h} - 0.60\right) N_0^{0.19}$$

$$N_0 \begin{cases} 0.5 & \text{start of damage} \\ 2.0 & \text{acceptable damage} \\ 5.0 & \text{severe damage} \end{cases}$$



$\Delta = 1.65$
$0.5 < h'/h < 0.8$
$7.5 < h'/D_n50 < 17.5$
$0.30 < B_m/h < 0.55$, B_m is berm width.



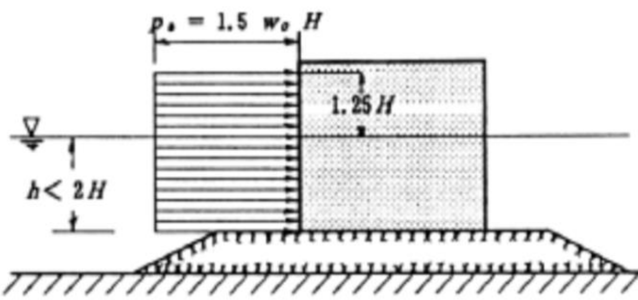
Figur 4.5: Definition sketch

ARMOUR	Q.STONES	ARMOUR	BLOCKS
Ro stone	2700	Ro block	2350
Ro water	1030	Ro water	1030
Delta	1,621	Delta	1,282
N0(damage)	0,5	N0(damage)	0,5
Dn50	1,50	Dn50	1,90
M50	9157	M50	16140

The last part needed to be dimensioned is the crown wall that usually doesn't bring so many stability problems. The only case that could lead to a failure is when the crown wall is not much higher than the mean sea water level, as in this case, and it has been checked with the Hiroi method with a security factor equal to 1 because the formula itself is already a lot conservative.

CROWN WALL (HIROI FORMULA)

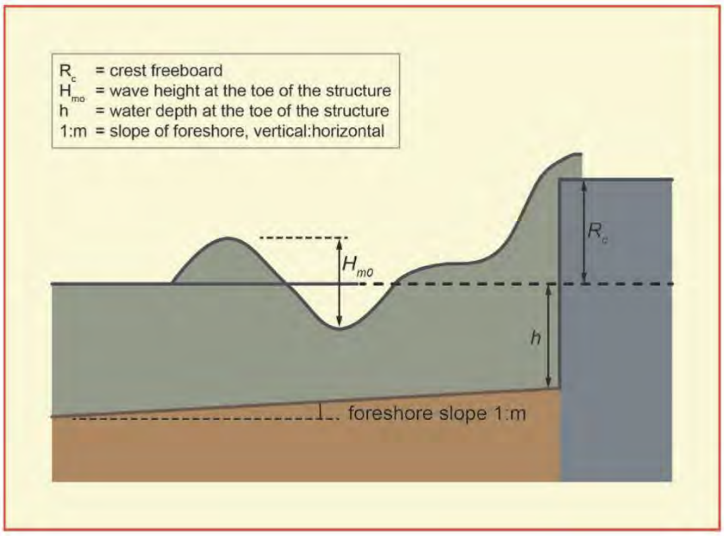
$$p = 1.5\,\rho_w\,g\,H$$



P	75782,25
MU	0,6
RO C.WALL	2300
SF	1
H cw	5
T slab	1
B cw	20
W cw	631518,75
Tcrown =	2,00
Eq sliding	0

Overtopping discharge

After dimensioning, all of the structural parts there is a need to verify the amount of overtopping allowed from the breakwater, to reach this aim it was used the EurOtop manual 2016, as shown in the images below.
The overtopping discharge had been calculated for both formulas, but just the value coming from the second formula has been taken into account to be checked on the 'disturb table', because, as written on the EurOtop manual, is the most conservative value in a dimensioning phase.



To take a mean value approach, Equation 7.1 is derived from measured data and should be used for predictions and comparisons with measurements. This is the same as Equation 5.18 with cot α = 0. The reliability of Equation 7.1 is given by σ(0.047) = 0.007 and σ(2.35) = 0.23.

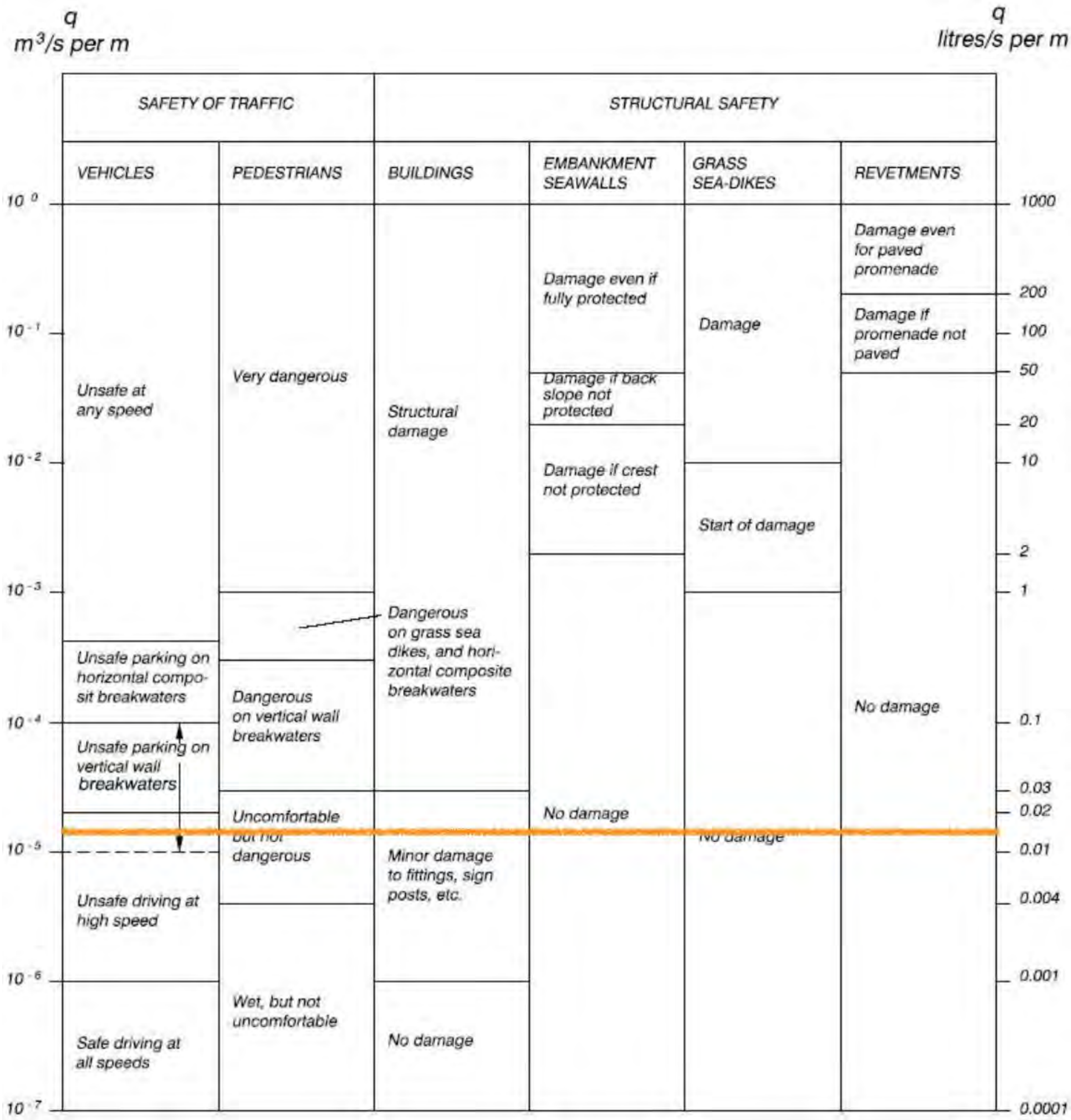
$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.047 \cdot \exp\left[-\left(2.35 \frac{R_C}{H_{m0}}\right)^{1.3}\right] \qquad 7.1$$

For a design or assessment approach, it is strongly recommended to increase the average discharge by about one standard deviation. Thus, Equation 7.2 should be used in design and safety assessments, where the effectiveness and scatter associated with this prediction can be gauged from Figure 7.8:

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.054 \cdot \exp\left[-\left(2.12 \frac{R_C}{H_{m0}}\right)^{1.3}\right] \qquad 7.2$$

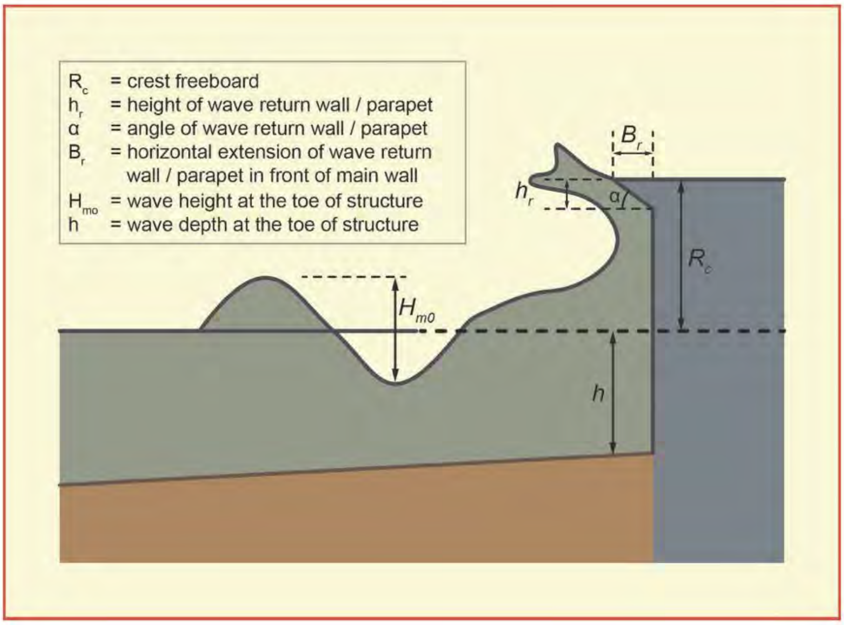
The results are shown in the table below, and for the value got from the 7.2 formula it has been found out that the calculated overtopping discharge is going to disturb only pedestrians, which are not supposed to be there because that breakwater is just designed for a commercial and specialized use.

OVERTOPPING		
	Q [l*m/s]	Q [mc*m/s]
7.1	0,007	7,0E-06
7.2	0,016	1,6E-05



Bullnose (overtopping reduction)

In the end, it has also been considered the possibility to insert a bullnose in front of the crown wall, in order to reduce the overtopping discharge without increasing furthermore the height and the weight of the structure.
Unfortunately, the bullnose has not been dimensioned because of the lack of time and also because it needs detailed simulation in flume in order to find the correct behavior of the waves with this layout, but it has been calculated anyway the parameter gamma from the table below of the EurOtop, which, for the design configuration of the breakwater leads to a real gain in terms of overtopping discharge, and for so, it could deserve further analysis and calculations.



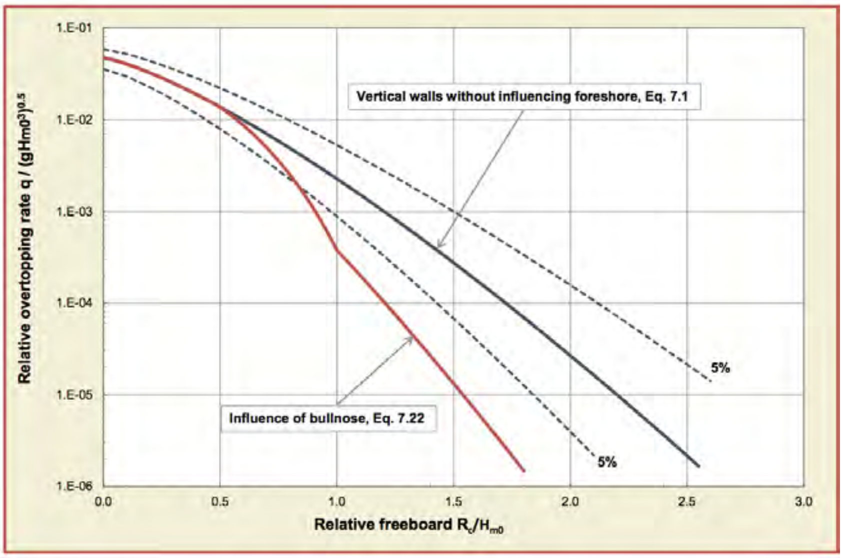
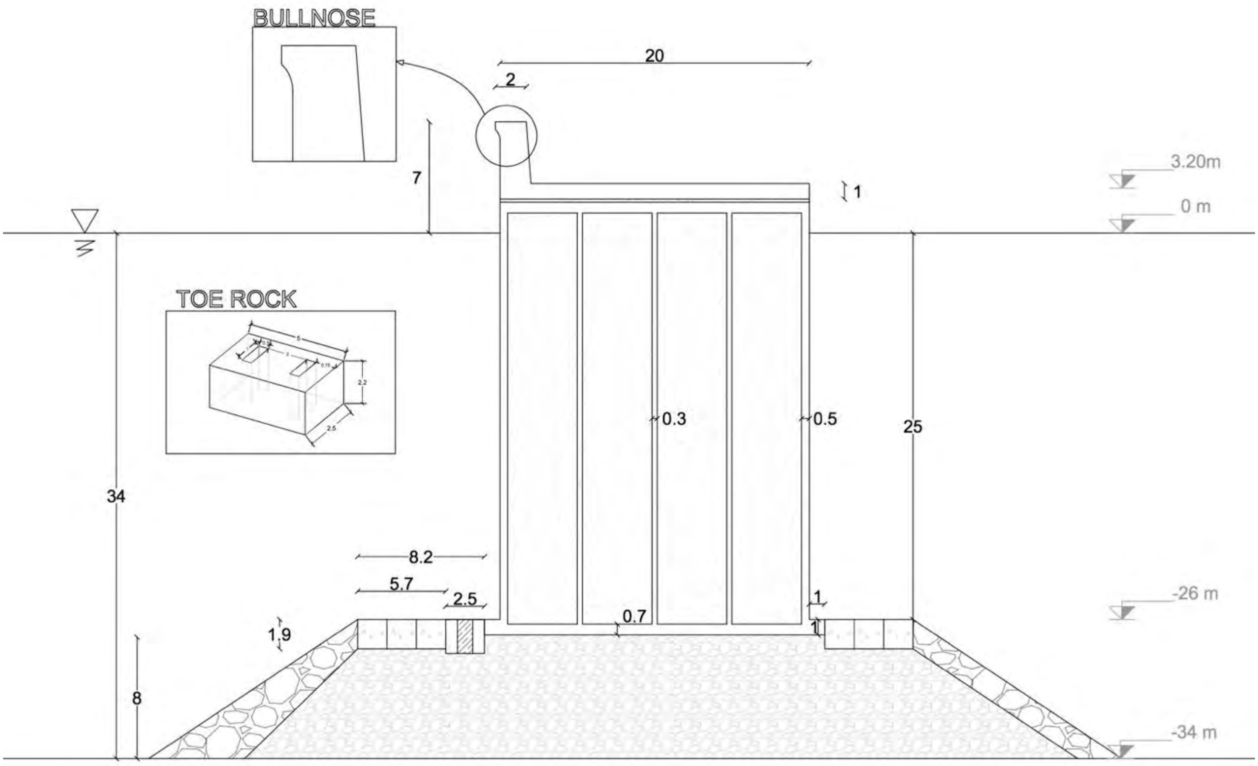


Figure 7.22: Graph showing the three regimes of effectiveness of a bullnose / wave return wall (TAW, 2003)

$$\gamma = 1.0 \quad \text{for } R_c/H_{m0} \leq 0.5$$
$$\gamma = 1.3 - 0.6\,R_c/H_{m0} \quad \text{for } 0.5 < R_c/H_{m0} \leq 1.0 \qquad 7.22$$
$$\boxed{\gamma = 0.7 \quad \text{for } R_c/H_{m0} > 1.0}$$

Final design

In this last part is just shown the final layout of a section of the caisson breakwater with the detailed measures of the toe rock of the head section and with a simple zoom on a hypothetical bullnose shape.



In conclusion it's provided a quick summary for the cost of the caissons breakwater, calculating the cost at each depth with the sphere of influence theory usually used for the loads distribution of buildings and also a table for the weight of the heaviest section for the geotechnical verifications and soil reinforcement techniques.

COSTS (LENGHTS CALCULATED WITH SPHERE OF INFLUENCE)			
Depth (m)	Unit Cost (€/ml)	NEEDED LENGHT	JOINT
15	160000	80	60
20	170000	120	
25	180000	120	
30	192500	60	
33	200000	2960	
35	205000	0	
		TOTAL COST €	
		667950000	668MLN

BREAKWATER WEIGHT	
Total Weight (N/m^2)	321768
Total Weight (KN/m^2)	321,8
Gamma sat Tout Venant	20
Gamma sat TV underwater	10
Hbasement	8
Basement weight (KN/m^2)	80
TOTAL WEIGHT (KN/m^2)	401,8

5.2.5 Mooring and breasting structure design

An analysis of the mooring and breasting structures were carried out for two terminals; the oil terminal and the mother container ship terminal.

Mooring structure arrangement

The quay assigned to berthing the mother ships, in the container terminal, should be capable of handling ships of dimensions presented in the table below.

Table 4: Ship dimensions berthing the container terminal

	Design vessels: Container Mother ship					Berthing Requirements		
	TEU	**Displ/. (Tons)**	**Draft (m)**	**LOA (m)**	**Breadth (m)**	**> 0,3 LOA (m)**	**<0,4 LOA (m)**	**D berth (m)**
Max values	18270	249000	16.02	398	54	119.4	159.2	**124**
Mean values	14240	204208	15.7	378	51	113.4	151.2	**124**
Min values	12400	169799	15.5	322	48.2	96.6	128.8	**124**

Similarly, the different crude oil tankers considered for the berthing structures are shown in the table below.

Table 5: Ships dimensions berthing the oil terminal

	Design vessels: Crude Oil tanker					Berthing Requirements		
	DWT (tons)	**Displ/. (Tons)**	**Draft (m)**	**LOA (m)**	**Breadth (m)**	**>0,3 LOA (m)**	**<0,4 LOA (m)**	**D berth (m)**
Max values	299999	348760	22	343.7	60	103.11	137.48	**116**
Mean values	299985	334589	21.25	331.15	59.17	99.345	132.46	**116**
Min values	255087	289561	18.7	322	56	96.6	128.8	**116**

In both the Mother ship container terminal and the oil tankers terminal, the fender spacing proposed can handle all the design ships. This is explained from the fact that the berthing ships have a similar length.

As a broken fender can prevent the functionality of a berth, 2 added fenders are proposed. These were placed towards the inside of the 2 proposed fenders, which will be of use in the case of a broken fender, and can also benefit the mooring of smaller feeder ships, in the case that the container terminal for feeder ships is fully occupied.

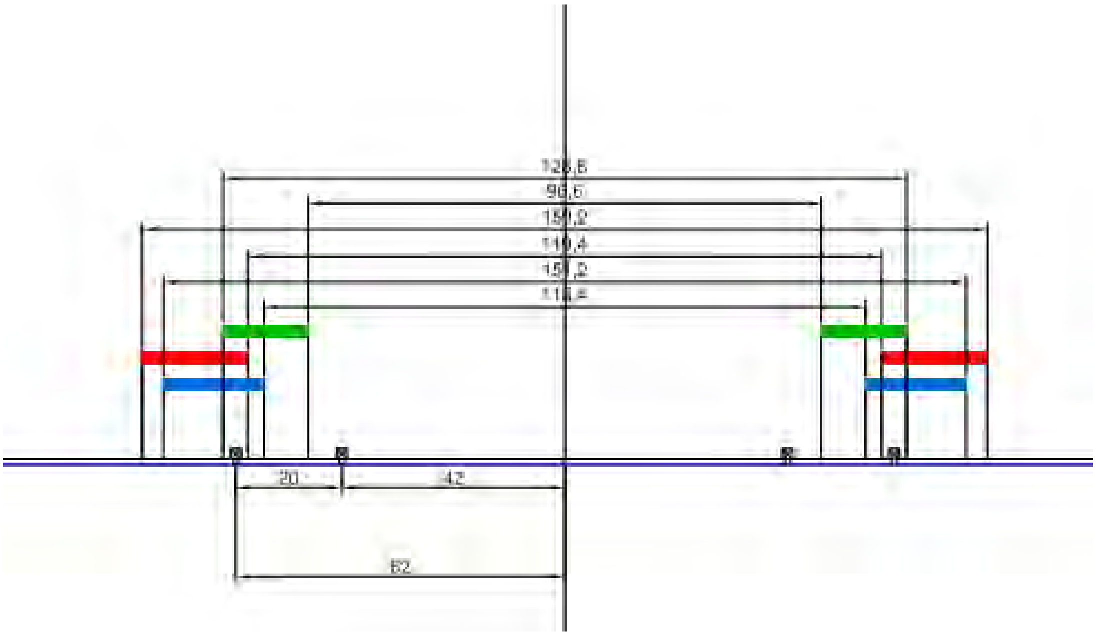


Figure 8: Mooring layout for the container mother ships

The drawing above represents the range in which the fenders can be placed, according to the 0.3 LOA and the 0.4 LOA limits, for each of the smaller, average and larger ships (in green, blue and red, respectively). As seen on the figure above, the location of the two designed fenders is chosen at a point where all three overlap, enabling the berthing of all the ships considered for design.

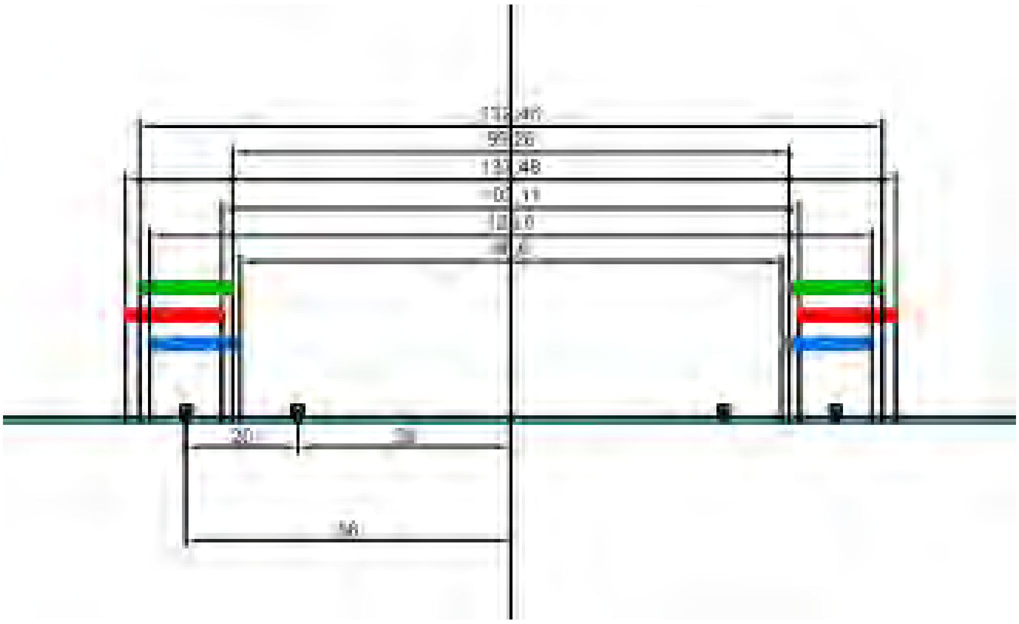


Figure 9: Oil terminal fender location

Similarly, for the oil tankers, the fenders were placed in the area of acceptable berthing location for all design ships. (Thorensen, 2014)

Design of the berthing fender

As the ships approach the berth, the vessels kinetic energy is absorbed by the fenders. To minimize the horizontal force on the quay and prevent damage to the ship as well as to the quay, fenders capable of handling the approaching energy are designed.

The kinetic energy of the ship to be absorbed by the fender (E_f) can be estimated using the following equations:

$$E_f = C \times (0.5 \times M \times V_s^2)$$

$$C = C_M \times C_E \times C_C \times C_S$$

M = Mass of the vessel (displacement in tonne) at chosen confidence level.*
V_B = Approach velocity component perpendicular to the berthing line† (m/s).
C_M = Added mass coefficient
C_E = Eccentricity coefficient
C_C = Berth configuration coefficient
C_S = Softness coefficient

Our calculations are based on the Port Designers Handbook, (Thorensen, 2014), and presented in the appendix of the report.

The table below presents the results for the energy required to be absorbed by the fenders at berthing.

Table 6: Results of the energy per fender for each quay

Terminal	M (Mass of the Vessel)	V^2 (Approach velocity squared)	CM (Added mass coef)	CE (Eccentricity coef)	CC (Berth configuration coef)	CS (Softness coef)	Ef (Normal energy) Kn/m
Container	249000	0.0196	1.8	0.86	0.9	1	**3372**
Oil tankers	348760	0.0169	1.8	0.99	0.9	1	**4727**

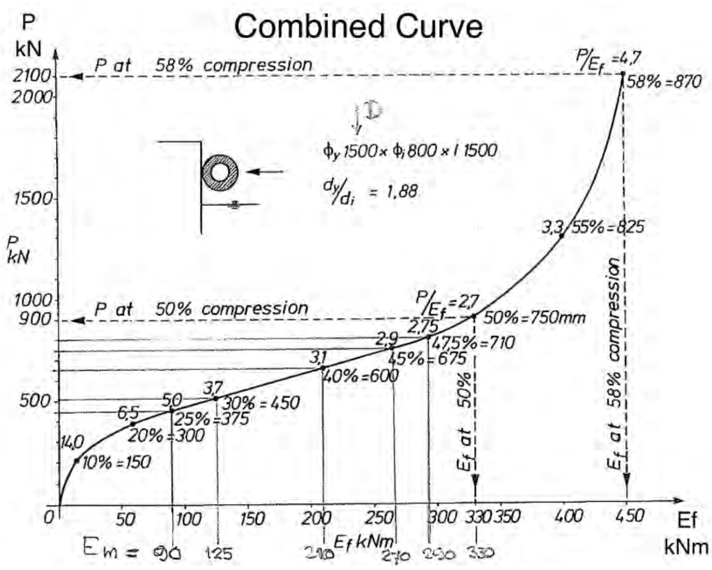
Safety factor on the normal energy of the fender

The usual safety factor for largest container ships terminal, should be 1,25, but the maximal safety factor value is 2.0. It is preferable to consider the worst situation to avoid a failure of one fender which could make the terminal unserviceable. (Pianc, 2002)

Choice of fenders

The choice of the fender is defined by the capability to resist the load of the berthing ships, and by the capability to dissipate the horizontal force which affects the quay. The higher the fender deflection, the more energy is dissipated, and the higher the load on the quay.

As can be seen from the graph below, the optimum compression for the fender is around 50%. If the load is increased further than 50% fender deflection the pressure on the quay would increase considerably. Therefore, we design the fenders so that these undergo 50% deflection under design conditions.



According to the previous calculations, the impact energy for the container terminal is Ef = 6744 [kN*m] and for the oil tankers is Ef = 9454 [kN*m].Two supercone fenders were chosen to absorb the load on the quay walls. The choice of supercone dimensions was based on the product capacity for each quay (see Appendix).

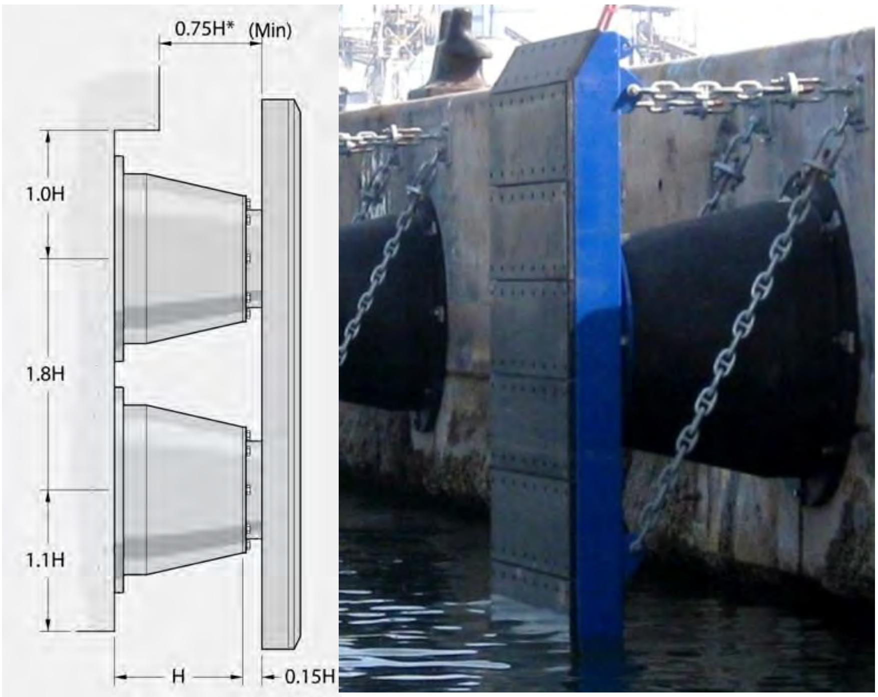


Figure 10: Picture and sketch of the supercone and fender plate

Upon dimensioning of the cones, the panel was dimensioned to resist the design loads. (See appendix H)

Table 7: Capacity of fender panels and cones

	Capacity of fender elements (kN)			
	Container Terminal		Oil tankers	
	Cone	Panel	Cone	Panel
R (kN)	2875	5749	3618	7237
H (m)	2.0	7.8	2.3	8.8
Ø , W (m)	3	4	3	4.5
P (Kn/m²)	-	184.28	-	183.26

Bollard design

To determine the Bollard capacity and spacing, the maximum wind force acting on the design ship is required.

$P_w = C_w \times (A_w \times sin^2\varphi) + (B_w \times cos^2\varphi) \times \frac{V_w^2}{1600}$ (Thorensen, 2014)

From our wind analysis, the strongest winds oriented closest to the perpendicular to the length of the berthed ship come from the East.

Φ = 30degrees, Vw= 16 m/s, Bw = 12000 m^2, Aw = 1800 m^2 P_w=3216kN.

For simplicity of this analysis, the table below is used over the calculation. As a rule of thumb, the figure below can be used to determine the necessary bollards spacing and load combinations. For a 250000ton and 300000ton vessels, 30 and 35 m, and 50 kN/m and 65 kN/m are required respectively. The bollard capacity should be set to 1500 kN and 2275 kN for

Ships with displacement in tons up to	Bollard load P in kN	Approximate spacing between bollards in metres	Bollard load normal from the berth in kN/m berth	Bollard load along the berth in kN/m berth
2000	100	10	15	10
5000	200	15	15	10
10000	300	20	20	15
20000	500	20	25	20
30000	600	25	30	20
50000	800	25	35	20
100000	1000	30	40	25
200000	1500	30	50	30
>200000	2000	35	65	40

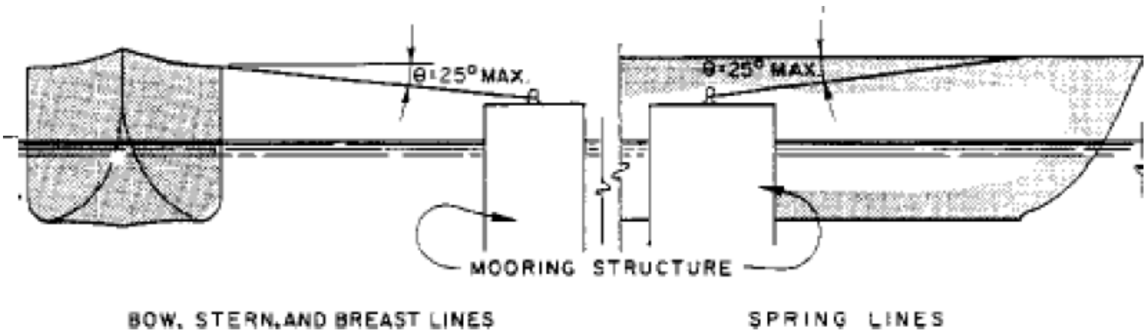
the oil tanker and container terminal respectively.

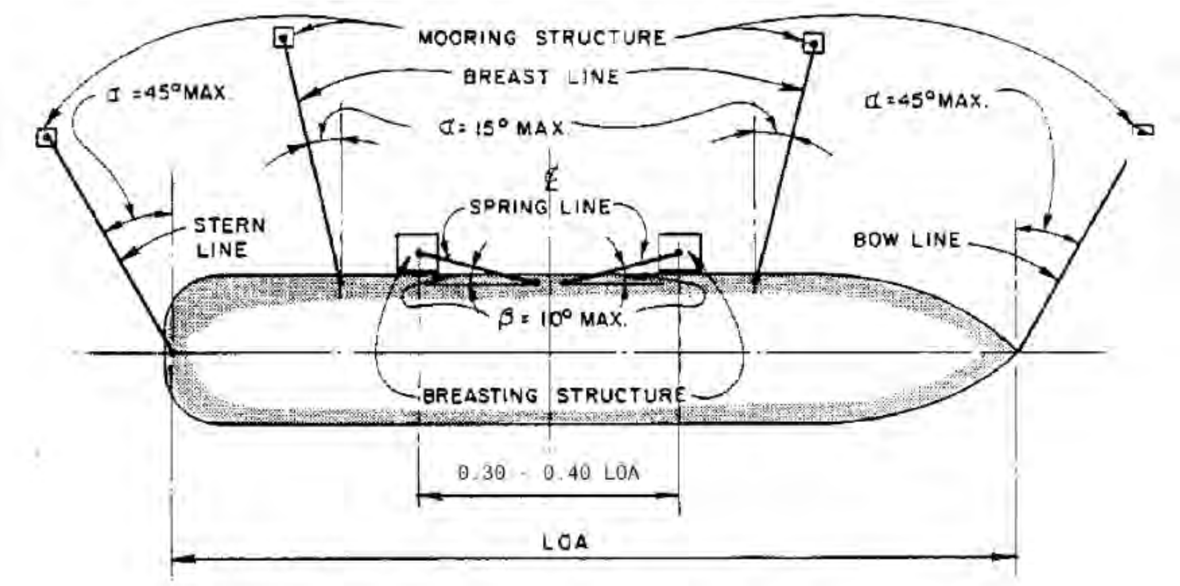
Table 8: Relation table between ship weight and bollard specifications (Nyvoll Consult, NTNU, 2016)

Mooring lines

The mooring lines depends upon mooring usage, space available for mooring, size of the ships, mooring conditions and the lines available on the vessels.

In the following figures a typical fixed mooring and mooring layout is shown, including the maximum spring line angles.





The bow, stern and breast lines have a good behavior under lateral loads while the spring lines are more efficient for longitudinal loads.

The OICMF recommendation for the mooring equipment are:

- Bollard capacity: 100% of MBL mooring lines
- Safe working load: 55% of MBL

MBL: Minimum breaking load of the mooring line which is equal to the hook/bollard capacity.

The design mooring loads can be obtained with the following expressions:

- Operating load:

$$Fm, op = N \cdot 60\% \cdot MBL$$

For our conditions, the max operating loads on for mooring lines are:

- 60%x1500= 900 kN for the oil terminal,
- 60%x2275= 1665kN and container terminal

5.3 Design Checks. Wave Agitation.

Artemis software enabled the design team to check for wave agitation inside the harbor. To run the model, first the bathymetry of the area was inputted. Then, the contour of the port was imported from AutoCad and placed at the precise location with respect to the bathymetry of the bay. After that, the boundaries (liquid/ solid) were defined. At the liquid boundaries, wave conditions were inputted.

The wave conditions (direction, significant wave height, peak period) were obtained from the offshore waves propagated using the TAWAMAC model, as discussed in the section above.

Model runs

Specific wave conditions were chosen from the statistical wave data. The highest wave, of relatively low occurrence, and of critical directions was chosen. For a port entrance facing the SW direction, the most aligned direction of the waves was the WNW direction.

Table 9: Frequency of occurrence of waves per direction and per wave heights

	Interval		N	NNE	NE	ENE	SSW	SW	WSW	W	WNW	NW	NNW	
Sig Wave Heights Hs (m)	0	0.5	7.89%	11.44%	1.90%	0.17%	0.07%	0.16%	0.27%	0.98%	3.90%	9.43%	4.93%	
	0.5	1	2.64%	13.70%	0.97%	0.00%	0.00%	0.00%	0.03%	0.53%	8.57%	7.08%	1.44%	
	1	1.5	0.40%	5.37%	0.14%	0.00%	0.00%	0.00%	0.00%	0.12%	5.32%	2.89%	0.22%	
	1.5	2	0.09%	1.60%	0.00%	0.00%	0.00%	0.00%	0.00%	0.02%	2.33%	1.61%	0.04%	
	2	2.5	0.04%	0.45%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.97%	0.77%	0.02%	
	2.5	3	0.01%	0.15%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.40%	0.33%	0.00%	
	3	3.5	0.01%	0.02%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.14%	0.13%	0.00%	
	3.5	4	0.01%	0.02%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	3 0.04%	0.04%	0.00%	
	4	4.5	0.00%	1 0.01%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	2 0.01%	0.05%	0.00%	
	4.5	5	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	
			11.08%	32.77%	3.00%	0.17%	0.07%	0.16%	0.30%	1.65%	21.68%	22.34%	6.64%	100%

The table above shows the three cases selected to run with in the agitation model. As very few conditions exceed the wave heights of the three cases, it was assumed that if the agitation requirements were met for the given cases, then the operational port requirements would also be satisfied.

The figure presents our input into the model.

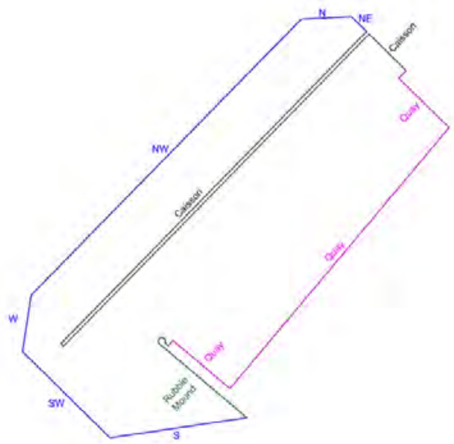


Figure 11: Port outline and boundaries included in the model

In case 1, the waves approaching from the northern directions did not affect the port.

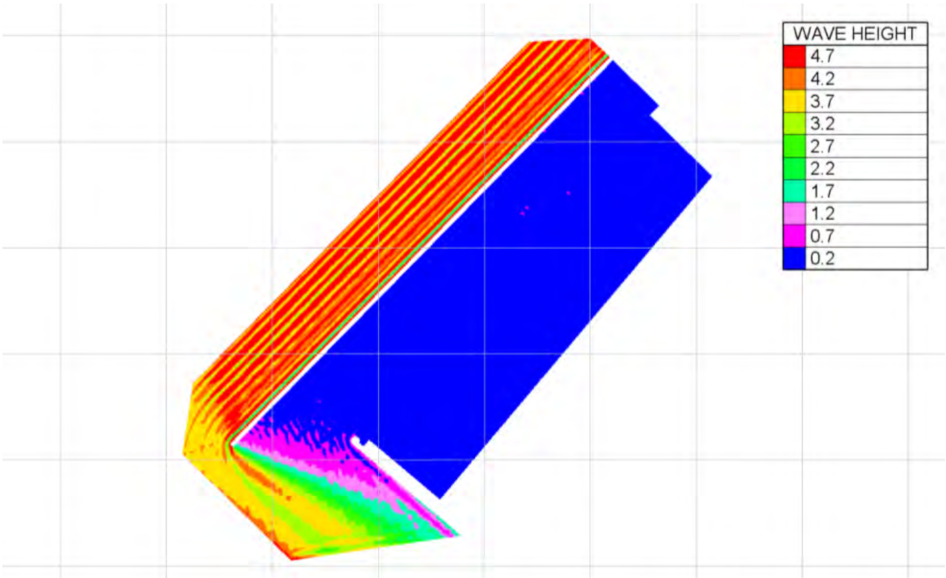


Figure 12: Case 3 - Wave height agitation model - Input Hs=4m, Tp=6.8, Dir=295deg

As seen in the figure of case 3, the wave height criteria limiting operational use of the port is not exceeded for none of the oil tanker berths, for the container terminal nor the ro-ro terminal.

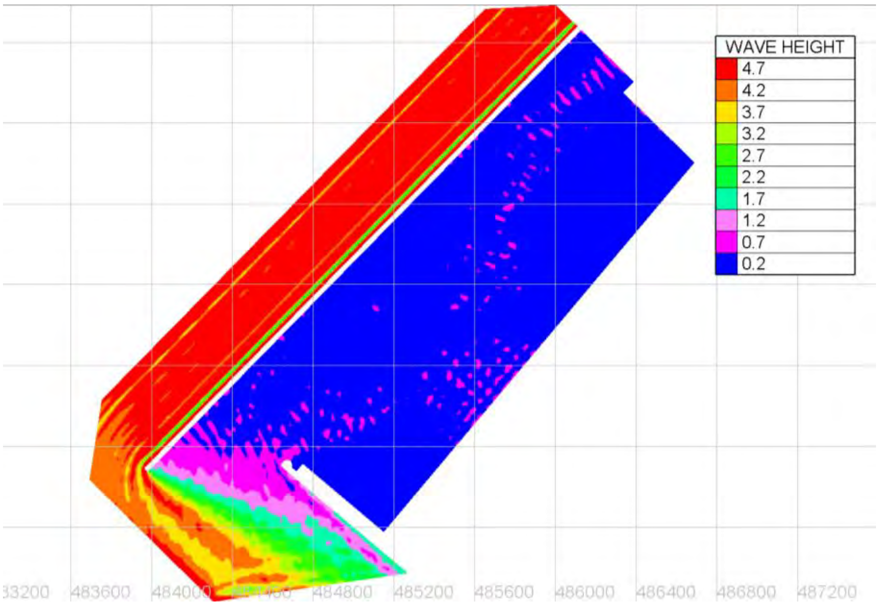


Figure 13: Case 2 - Wave height agitation model - Input Hs=4.4m, Tp=8.4s, Dir=295deg

Model 3 shows some localized areas, which have waves that exceed the set design criteria, of 0.7 m for the oil tanker terminal. These conditions are nevertheless very localized and rare; therefore, the port configuration is not compromised.

Based on the models investigated, the port operations may be halted 0.01% of the time or less.

Breakwater length optimization

Different port entrance configurations were considered while evaluating the agitation inside the port. We optimized the length of the vertical breakwater which protects the entrance of the port from incoming waves, by reducing it as much as possible, without allowing the entrance of waves which exceed the agitation requirement.

5.4 Design Check. Wave Flume.

Modelling and scaling

A model was built in the lab to verify the stability of the design of the rubble mound breakwater located on the northern side of the port. Since gravity are the governing forces acting on the model, the Froude scaling was implemented. The scale applied was governed by the depth of the flume limited to 30 cm. The dimensioning of the stone weight was obtained from equating the model and prototype stability number.

Stability number

$$\boxed{\left(\frac{H_D}{\Delta D_n}\right)_m = \left(\frac{H_D}{\Delta D_n}\right)_p}$$

Froude scaling table

Physical Parameter	Unit	Multiplication factor
Length	[m]	λ
Structural mass:	[kg]	$\lambda^3 \cdot \rho_F / \rho_M$
Force:	[N]	$\lambda^3 \cdot \rho_F / \rho_M$
Moment:	[Nm]	$\lambda^4 \cdot \rho_F / \rho_M$
Acceleration:	[m/s²]	$a_F = a_M$
Time:	[s]	$\sqrt{\lambda}$
Pressure:	[Pa=N/m²]	$\lambda \cdot \rho_F / \rho_M$

Froude scaling table

Scale 1:50.

Parameter	Unit	Symbol	Prototype	Model	Scale applied	Check & Comments
Stone Density	kg/m³	ρs	2650	2650	-	Same in model and reality
Water Density	kg/m³	ρw	1030	1000	-	Salt water vs fresh water
Peak Period	s	Tp	11.14	1.575	√ λ	ok 1s < tpm < 2s
Wave height	m	Hs	4.24	0.085	λ	ok Hsm < 7cm
Depth	m	d	16.1	9.758	λ	ok d < 30cm
Diameter Armor	m	Dn50 armor	1.33	0.0254	Stability Number	
Armour Mass	kg	Marmor	6234.49	0.0432	Stability Number	
Armor Weight	kN	Warmor	61160.33	0.4238	Stability Number	
Diameter Filter	m	Dn50 filter	0.58	0.0111	Stability Number	
Filter Mass	kg	Mfilter	517.05	0.0036	Stability Number	
Filter Weight	kN	Wfilter	5072.23	0.0351	Stability Number	
Relative density	-	Δ	1.57	1.65	-	
Crest height	m	hc	8.30	0.166	λ	

Flume test

The flume test performed on the scaled preliminary rubble mound breakwater design showed high levels of erosion. The quarry stones calculated in the initial design were too light resulting in high erosion at 70% of the design wave height.

At 100% of the significant wave height the damages observed were very severe as seen on the pictures below.

Figure 14: Damage observed on the wave flume model

The wave flume model proved that once the armor layer failed, and exposed the filter layer, the destruction of the breakwater was exponentially faster. The eroded material took the profile of an S shape was observed after the test.

The problem was that the stone size would have to be considerably larger to resist the wave height and duration according to Van der meer formula. Since the armor stones were of too large, the design was changed to include accropodes.

5.5 Cost

From the given unit costs (the breakdown of all the costs and the process of the cost estimation is given in Appendix D), estimation of the total cost of the designed port is carried out.

The costing includes the following parts of the port:

1. Breakwater
    - Rubble mount breakwater
    - Caisson breakwater
2. Quay wall
3. Jetty structure
4. Amount of dredging
5. Soil reinforcement

Table 5 summarises the main costs associated with the construction of the port:

Table 5. Main costs associated with the construction of the port.

Summary				
		Length (m)	Cost (€)	Total (€)
Breakwater				734 684 000 €
	Rubble mound breakwater in open sea	788	24 609 000 €	
	Slope protection inside the port	0	0 €	
	Secondary rubble mound breakwater	1143	42 350 000 €	
	Caissons breakwater	120	667 725 000 €	
Quays				103 977 200 €
	Quays	3902	92 042 200 €	
	Trestles	385	11 935 000 €	
		Quantity (u or m3)	Cost (€)	Total (€)
Jetty				46 800 000 €
	Walkways	4000	36 000 000 €	
	Fender system	72	10 800 000 €	
Dredging filing				190 754 200 €
	Dredging in sand	4198511	29 389 577 €	
	Dredging in soft clay	5837900	58 379 000 €	
	Backfilling with dredged material	4198511	12 595 533 €	
	Backfilling with sand dredged offshore the port	2266413	33 996 189 €	
	Backfilling with material excavated in land	4532825	54 393 902 €	
	Mobilisation/demobilisation of dredger	1	2 000 000 €	
Soil reinforcement				52 479 000 €
	Rigid inclusions	107100	52 479 000 €	
			TOTAL	**1 128 694 400 €**

The estimated total cost for the port is **€ 1 128 694 400**.

5.6 Construction

The following constructive method concerning the construction of the harbour is divided into six main phases according to the sequence of constructing the port.

Phase 1: Excavation and general preparation of the existing site

1. Excavation of the site to prepare terminals platforms and the accesses to this new industrial harbour. Depending of the quality of in-situ soil, they could be reused as construction materials in order to fill the polder area.
2. Reinforcement of the soil under the rubble-mound structure by rigid inclusions.

Phase 2: Construction of the two rubble-mounds breakwaters

1. Implementation of the several layers (sand core, filter, accropods armour layer) of the rubble mound structures.
   The work and the materials supply will be done with machinery (trucks, excavators) operating outside of the water.
2. Construction of the breakwater crown.

Phase 3: Construction of the caisson breakwater

1. Reinforcement of the soil under the caisson breakwater structure by rigid inclusions
2. Implementation of the basement with dredged sand brought by barges.

3. The armour and toe rock of the basement will be installed at the advancement using a crane on a barge.
4. Transport of the prefabricated caissons on site and positioning on their locations by sinking them by gravity with a filling of sand.
5. Implementation of the precast crown from sea with a crane on a barge.
6. Start dredging the entrance channel, and along the future quay wall in order to assure the navigability of yard ships.

<u>Phase 4:</u> Construction of quay wall and dredging

1. Construction of the quay wall by implementation of steel piles and sheet piles from a specialised ship.
2. Then at the same time, the seabed is dredged at the correct depth to assure navigability for commercial ships and dump the excavated material in a specific zone at sea.
   Then one the way, the ship will dredge sandy materials to fill the polder, at the same side the wall is being built.
   At the end of the operation, after the stabilisation of the platform, the second wall sheet piles and tierods can be implemented from the stabilised platform.
3. After all the space left between the two walls is filled and stabilised.

<u>Phase 5:</u> Prepare the storage areas

The bearing capacity of the storage areas are insured by a heavy compaction method.
The terminals slabs, access roads and technical/administrative buildings could be built after the tierods are placed.

<u>Phase 6:</u> Equipment

1. Implementation of bollards, fenders, buoy, etc.
2. Implementation all terminals facilities (cranes, load carriers...)

6. CONCLUSIONS

The preliminary report of the proposed Port in Nador, Morocco, provided hereby satisfies all the recommended design criteria. The port layout and proposed improvements satisfy; the necessary traffic needs, the provided port limits, no sedimentation concerns, allow for future expansions. The agitation within the port basin is also kept within acceptable range and the infrastructure is stable from a geotechnical standpoint.

This preliminary design, shines the light on the need for further research and expertise analysis prior to moving forward with the development. Including:

- Further geotechnical investigations should be carried out along the proposed foundation of the breakwaters and quays, to prevent any unforeseen soils.
- A detailed land survey should be provided and inland geotechnical information, to allow for best choice of the placement of the infrastructure and storage area. Reorganize the storage areas more efficiently. A more detailed look at the placement of the different storage areas;
- An in depth study of the organisation of the port basin could help optimize the space use. There is quite a large area in the port to navigate with the ships. Probably there is a design possible where the breakwater could be placed closer to the coast, whereby the costs will be smaller.

Given the guideline requiring the entrance to the SW and minimisation of dredging work, the proposed port layout was developed. This criteria imposed certain limitations, which made the port design more costly than if these conditions were not imposed.

7. REFERENCES

Church, J. a., Clark, P. U., Cazenave, a., Gregory, J. M., Jevrejeva, S., Levermann, a., … Unnikrishnan, a. S. (2013). Sea level change. *Climate Change 2013: The Physical Science Basis. Contribution of Working Group I to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change*, 1137–1216. https://doi.org/10.1017/CB09781107415315.026

Construction Industry Research, Information Association, Civieltechnisch Centrum Uitvoering Research en Regelgeving (Netherlands), & Centre d'études maritimes et fluviales (France). (2007). *The Rock Manual: The use of rock in hydraulic engineering* (Vol. 683). Ciria.

Groenveld, R. (2001). *Service systems in ports and inland waterways*. VSSD.

Ligteringen, H., & Velsink, H. (2012). *Ports and terminals* (pp. 199-220). Delft, Netherlands: VSSD.PIANC (2002)

Scope of works

Thoresen, C. A. (2003). *Port designer's handbook: recommendations and guidelines*. Thomas Telford.

8. APPENDICES

Appendix A. Wave Propagation.

NNE direction; 100-years return period1.

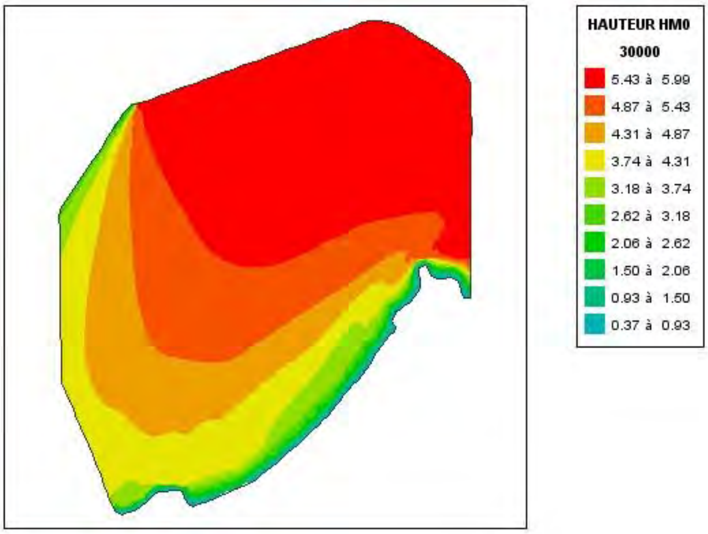


Figure 1. Wave height.

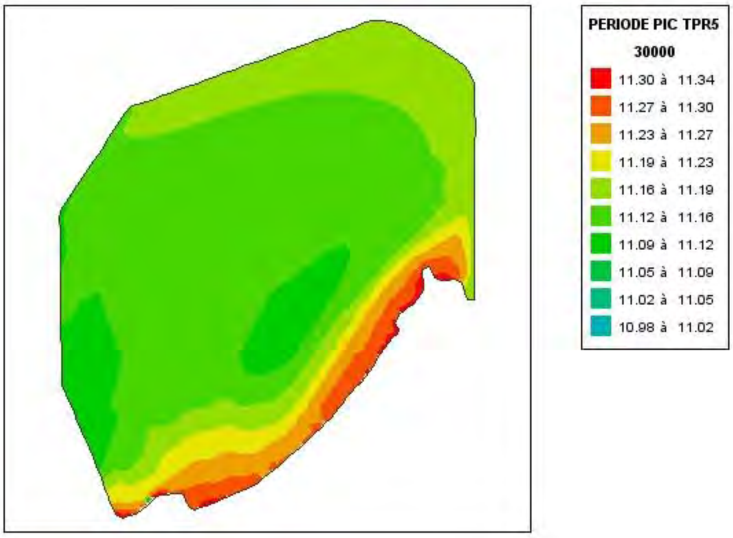


Figure 2. Peak periods.

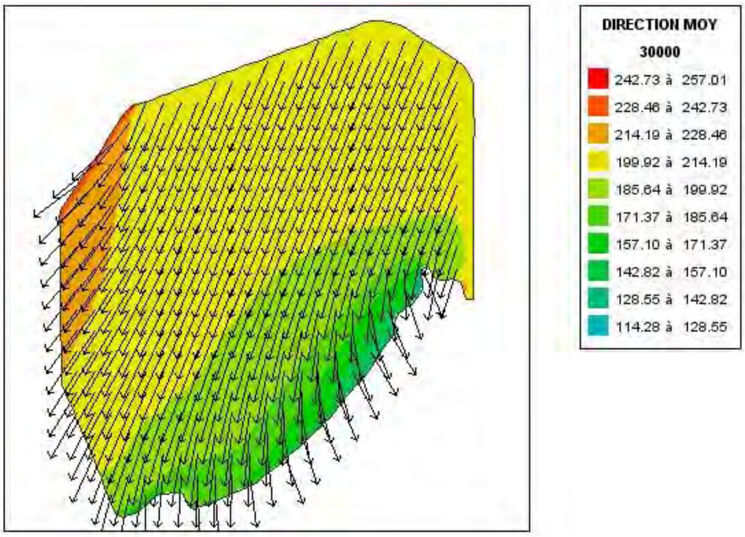


Figure 3. Main wave direction.

WNW direction; 1-years return period.

1-year return period

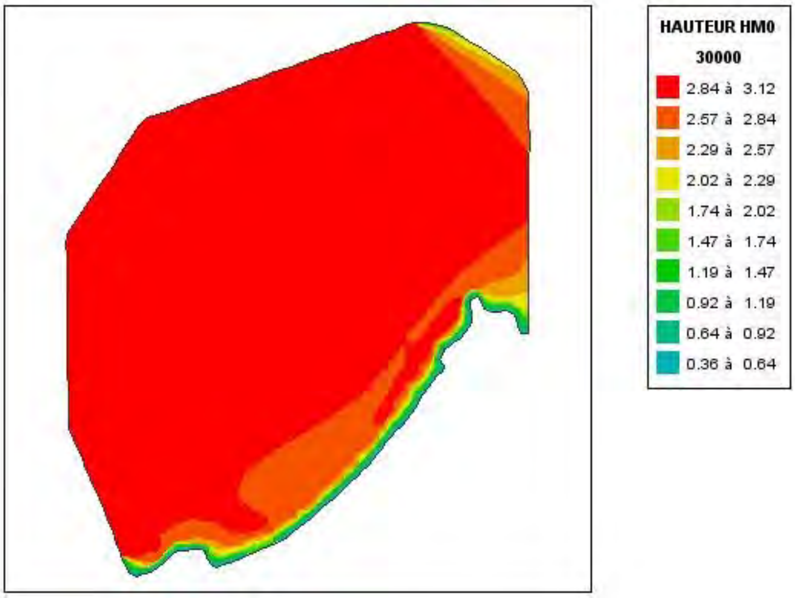


Figure 4. Wave height.

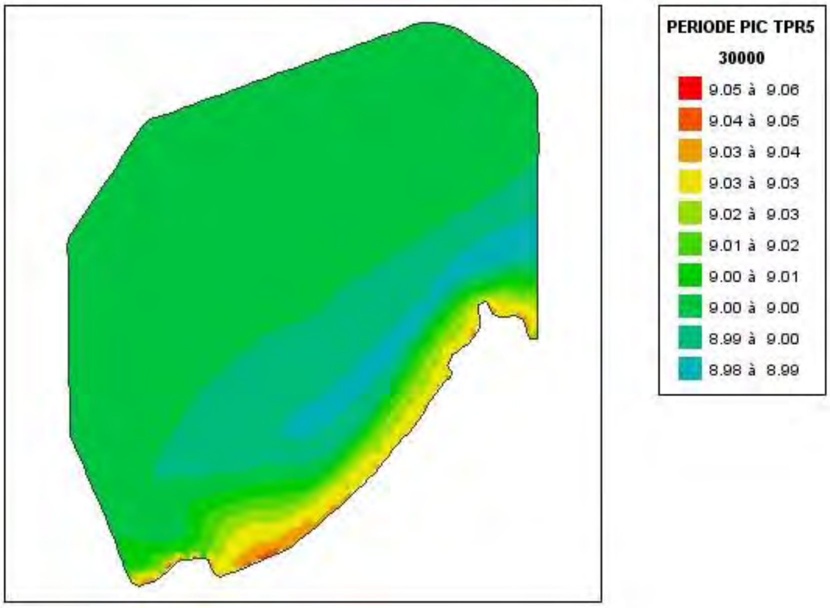


Figure 5. Peak period.

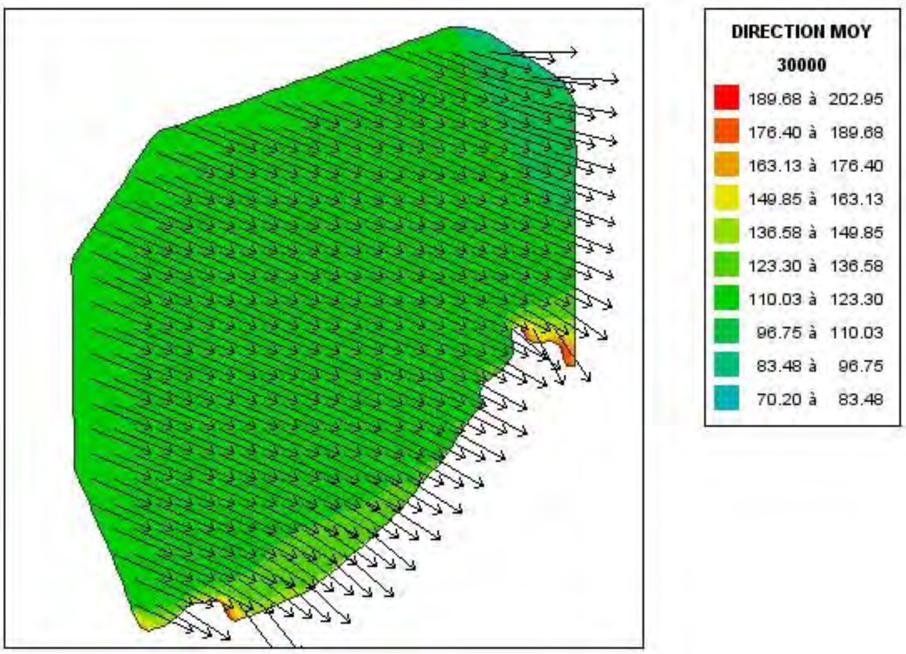


Figure 6. Main wave direction.

WNW direction; 100-years return period.

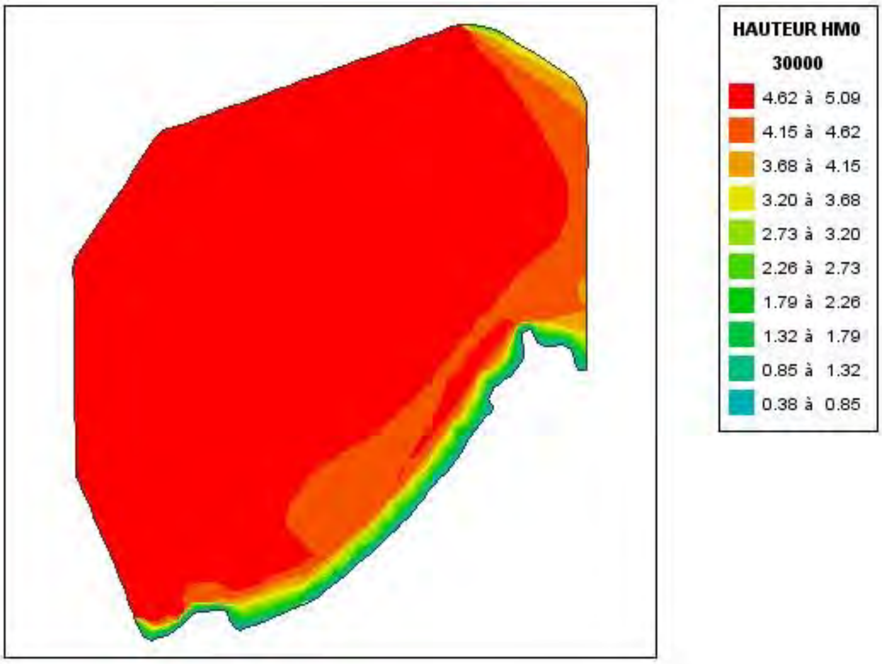


Figure 7. Wave height.

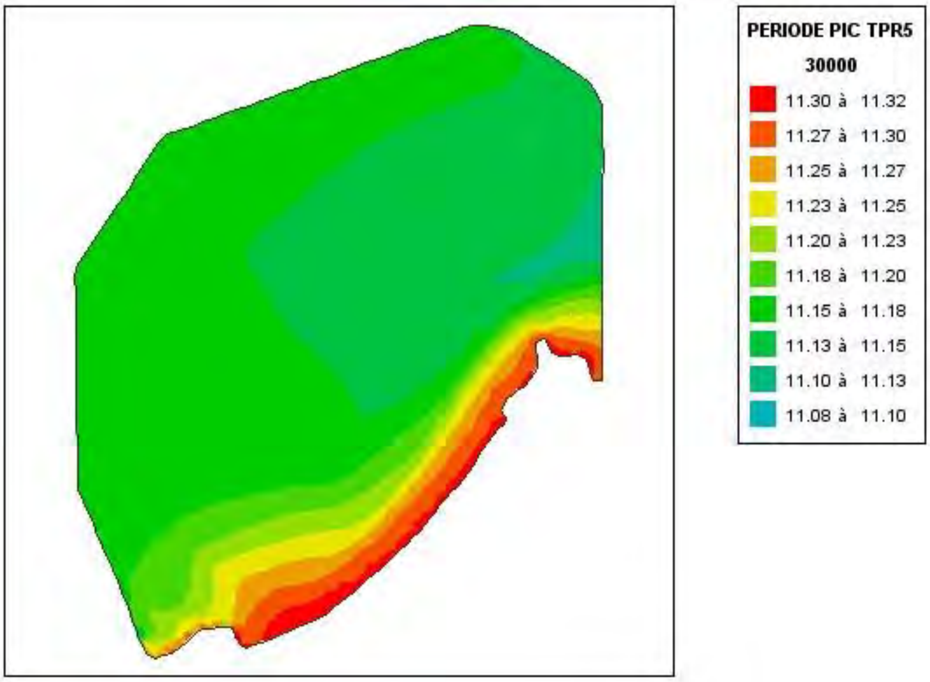


Figure 8. Peak period.

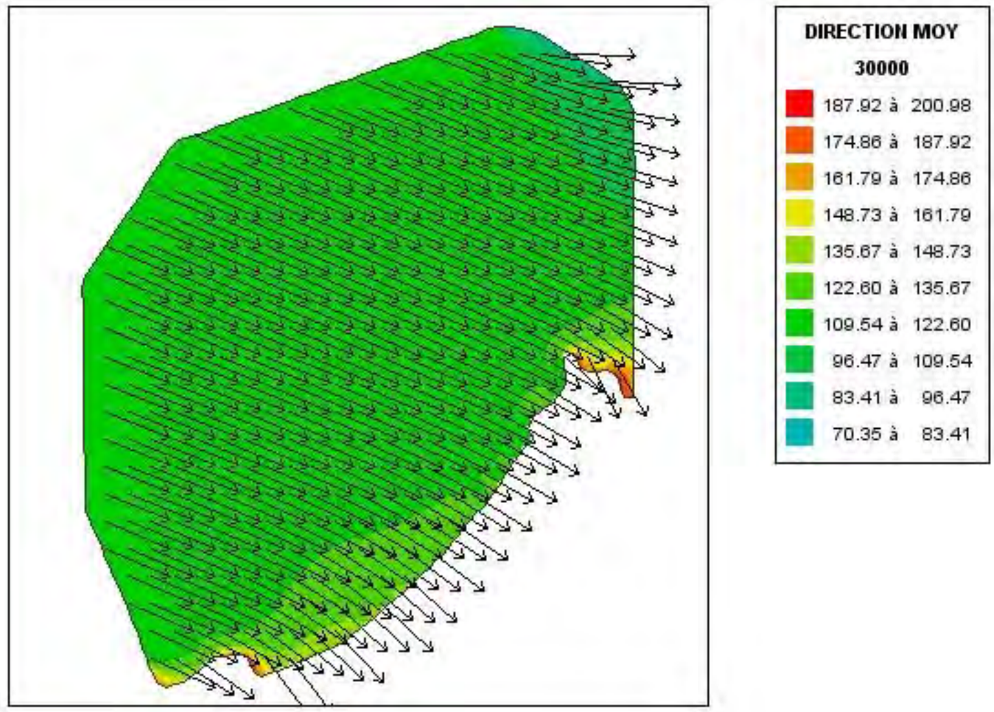
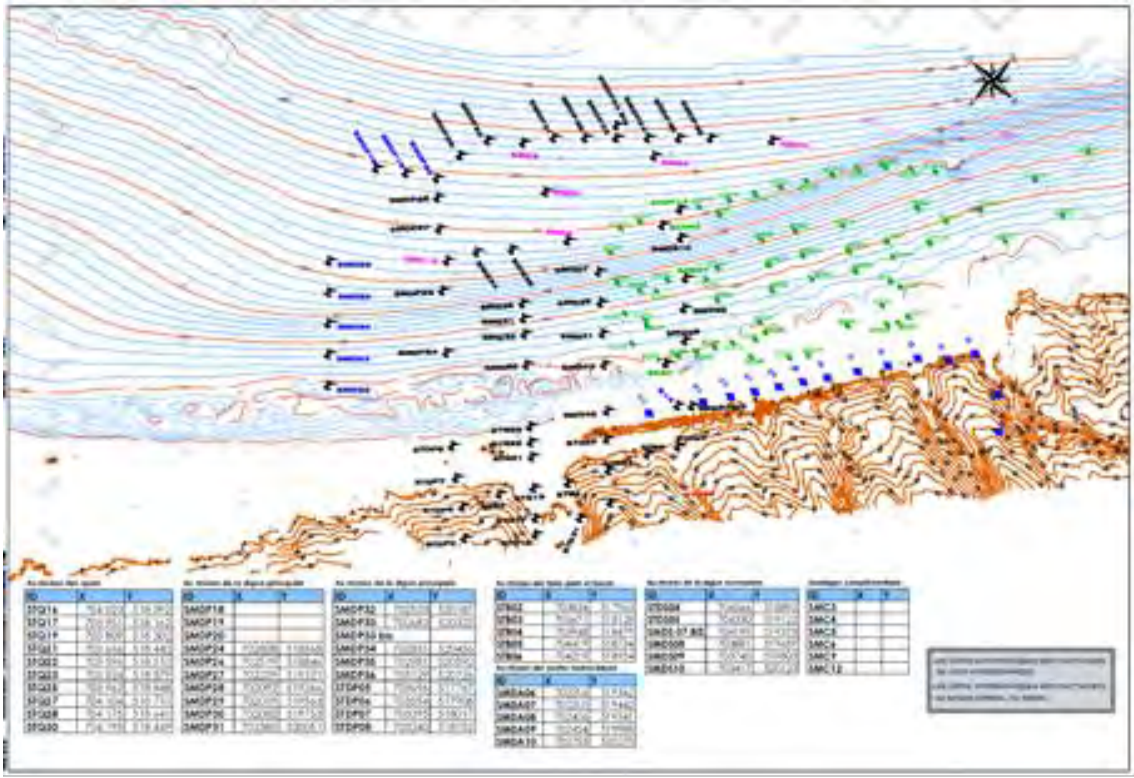
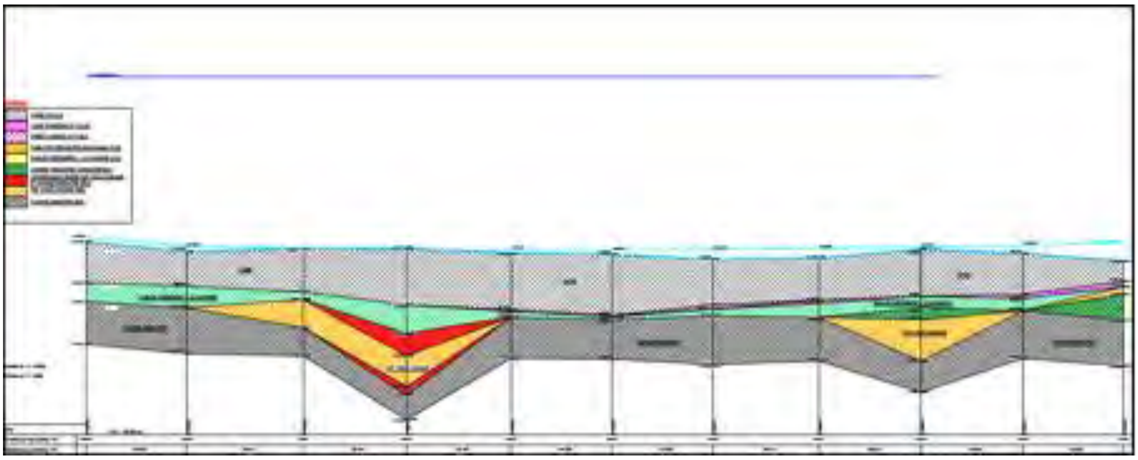
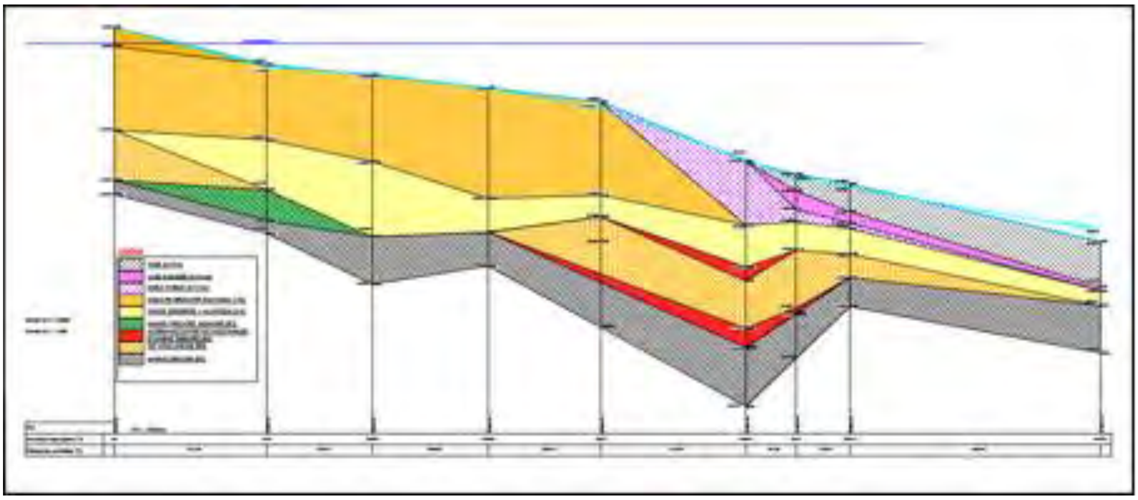


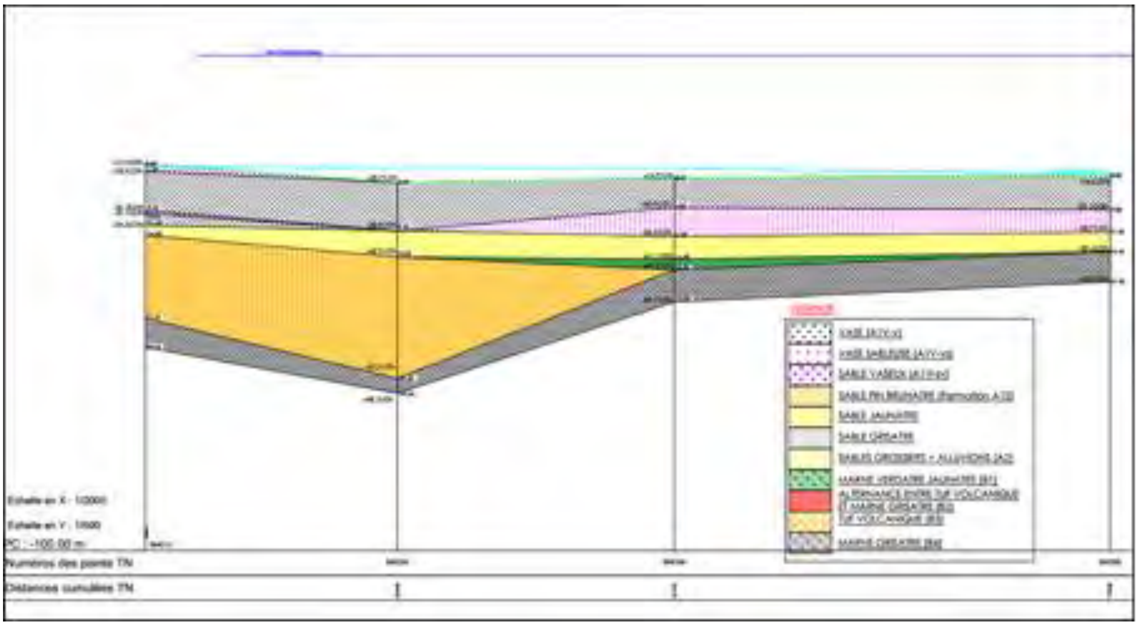
Figure 9. Main wave direction.

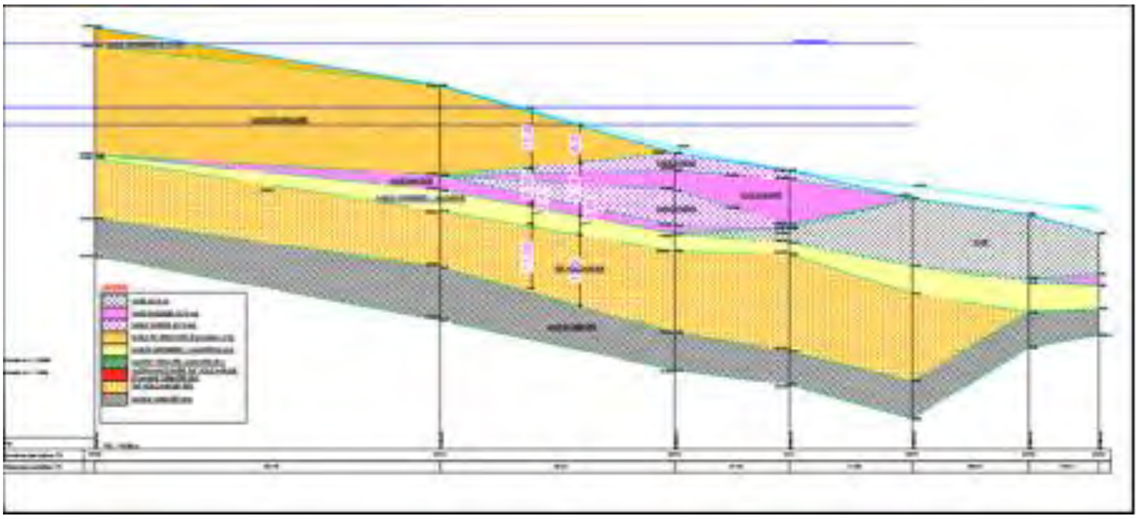
Appendix B. Site Conditions.

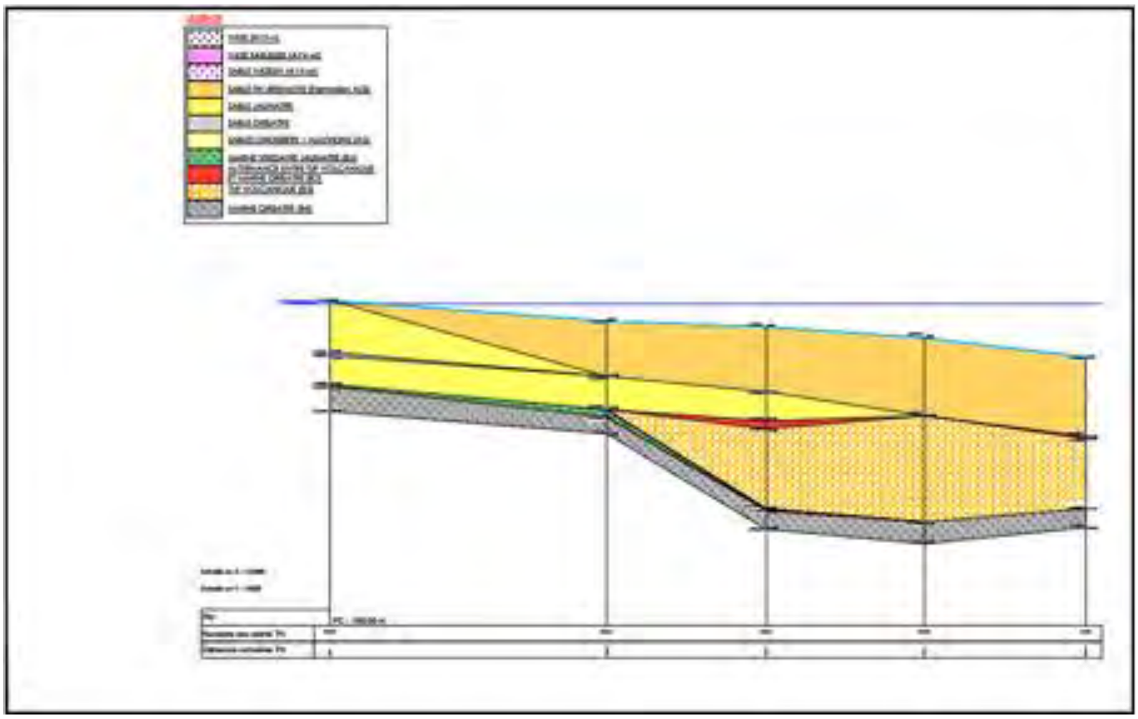







Numéros des points TN
Distances cumulées TN





		Etat en place		Paramètres de résistance mécanique			Paramètres Œnométriques					Paramètres de rigidité		
				Court terme	Paramètres effectifs		Tassement			Consolidation		Module élastique	Module pressio.	Coef. Poisson
Sol	Profondeur	γsèche (kN/m3)	γsat (kN/m3)	Cu (kPa)	Φ' (°)	C' (kPa)	eo	Cc	Cs	Cv (cm2/s)	Cr (cm2/s)	E (MPa)	Em (MPa)	v
Vase (silt argileux)		13	18.7	11.5 + 0.385.Z	23	9	1.09	0.3	0.05	0.001	0.005		--	--
Sable grossier		--	18.2	--	33	0	--	--	--	--	--		--	--
Marne verdâtre		14	19	120	21	35	0.83	0.18	0.06	0.0015	0.007		30	0.2
Marne grisâtre		17	20	170	30	50	0.6	0.11	0.05	0.0015	0.007		90	0.2
Marne grisâtre dure		17	20	200	30	50	0.6	0.11	0.05	0.0015	0.007		90+ 2.5.Z	0.2
Tuf volcanique	de 0 à 10m	19	22	1000	--	--	--	--	--	--	--		130 +11.Z	0.2
	> 10m	19	22	1000	--	--	--	--	--	--	--		240 +3.Z	0.2
Sable pour drains		--	20	--	30	--	--	--	--	--	--	20		
TV 1 à 500		17	20	--	45	--	--	--	--	--	--			
Enrochements inférieurs à 1T		17	20	--	45	10	--	--	--	--	--			
Enrochements supérieurs à 1T		17	20	--	45	10	--	--	--	--	--			
BCR		15	18.4	--	45	20	--	--	--	--	--			
TETRAPODES		12	17	--	45	20	--	--	--	--	--			
ACCROPODES		12	17		46	20	--	--	--	--	--			

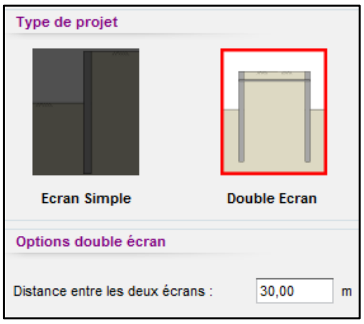
Figure 1. Soil parameters.

Appendix C. Quay Wall Design.

QUAY A

To start the project related modelling, phasing analysis must be made. For this, it is first necessary to have a project with a double screen type allowing to facilitate and to understand the realization of a retaining wall in a maritime environment.

The distance seen between these two screens are set to be 30m. An assumption about this length have been made and it has been decided to take the same value that have been input for the presentation model.



It is essential to perfectly enter soil data in the software according to their intrinsic parameters. Some parameters will be calculated with the following formulas:

Total vertical constraint

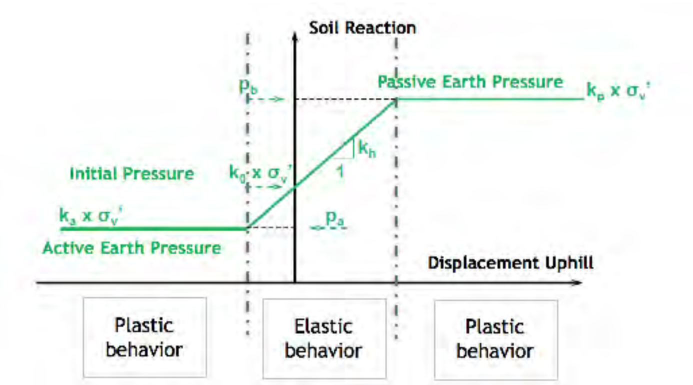
$$\sigma v' = y'.z = (ysat - yw).z$$

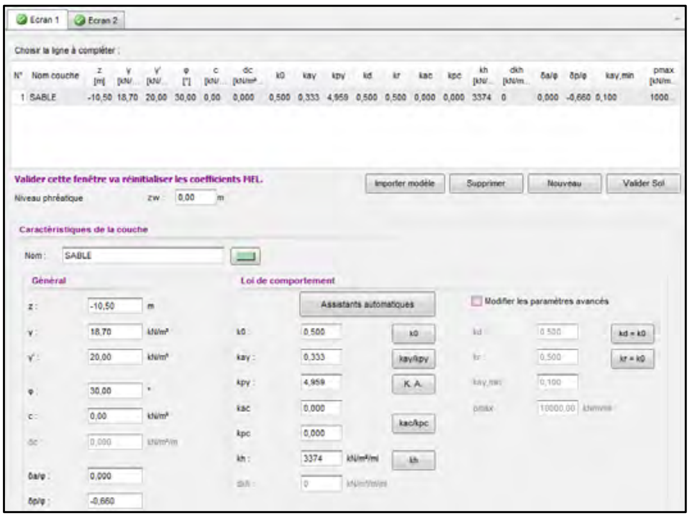
Active earth pressure

$$ka = \tan(\frac{\pi}{4} - \frac{\varphi}{2})$$

Passive earth pressure

$$kp = \tan\left(\frac{\pi}{4} + \frac{\varphi}{2}\right) = \frac{1}{ka}$$





Upgrade reaction coefficient

$$kh = \frac{2,0 * (\frac{Em}{\alpha})^{\frac{3}{4}}}{(EI)^{\frac{1}{3}}}$$

Live Load are as follows:

Ship Load on Fenders : H = 130kN/ml at +2m

Ship Load on Bollards : H = 70kN/ml, M = 35kN.m/ml at +3.5m

Live Load on Quay : Q = 50kN/m/ml at +3.5m

Shiploaders Loads : PR = +/- 17kN/ml at +3.5m

The data are then as follows:

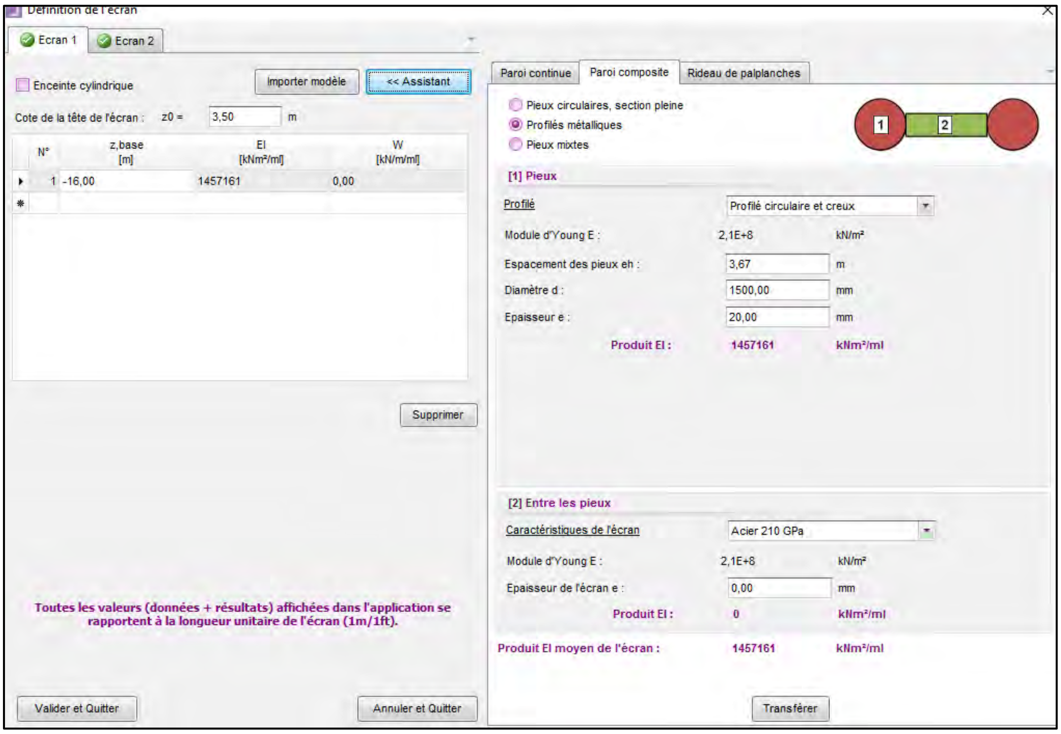


Figure 1 - Screen 1 parameters for quay A

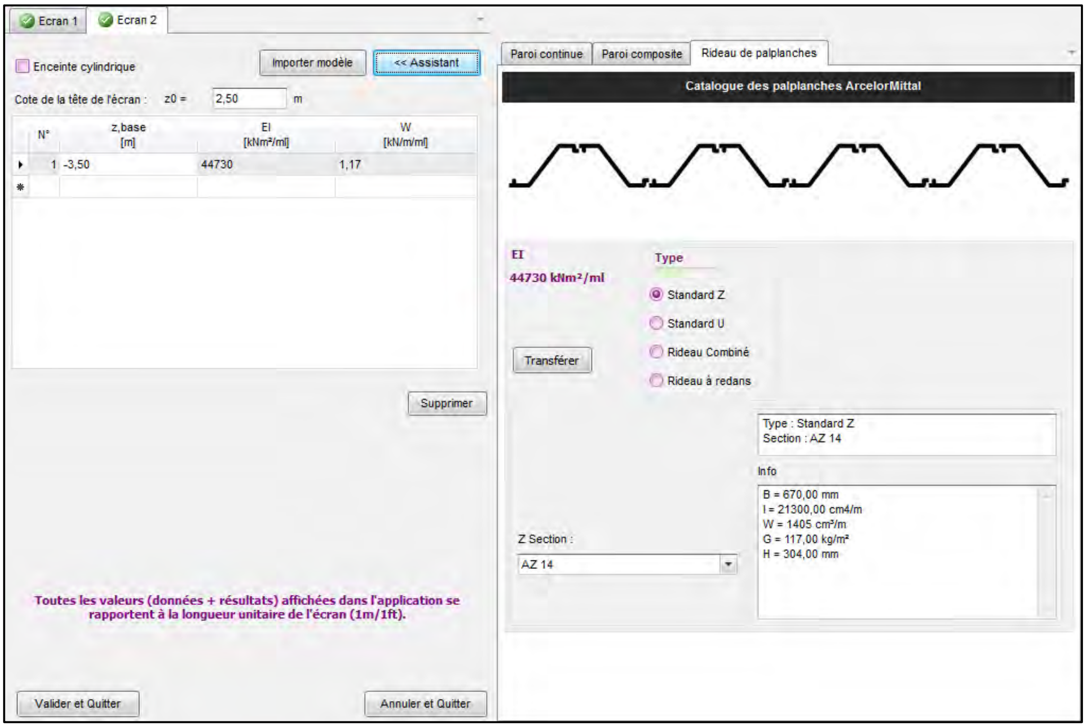


Figure 2 - Screen 2 parameters for quay A

STEPS	DESCRIPTION
1	Reduce the pressure that you have at the bottom of the pile
2	Backfill the screen 2 at both side to create the tie rod from the first screen
3	Create the tie rode between the pile and the sheet piles and apply a Caquot load of 50kN on the right side of the sheet pile
4	Fill the right side of the pile with Sand
5	Change the characteristic of the backfilling soil between the pile and the sheet pile
6	Apply a Caquot load of 50kNon the right fill of the pile
7	Apply a hydraulic action on the quay wall
8	Apply the force of the tenders: 130 kN
9	Apply the bollard strength and moment
10	Make a combination for the different force
11	Apply the ship load loads of -17kN

Construction phases have been created to scale more precisely and correctly our retaining walls.

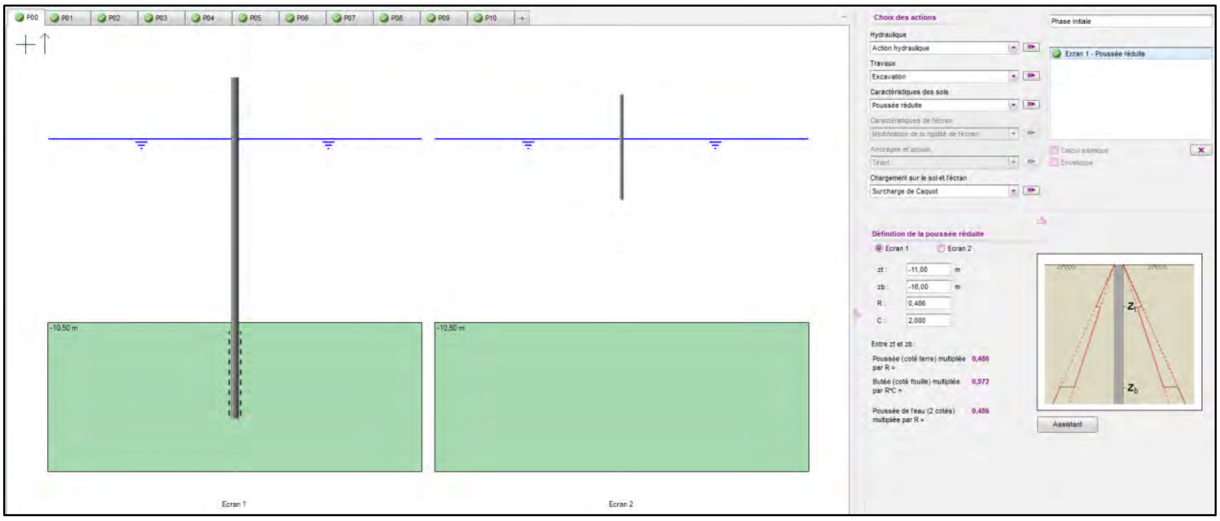


Figure 3 - Reduced pressure

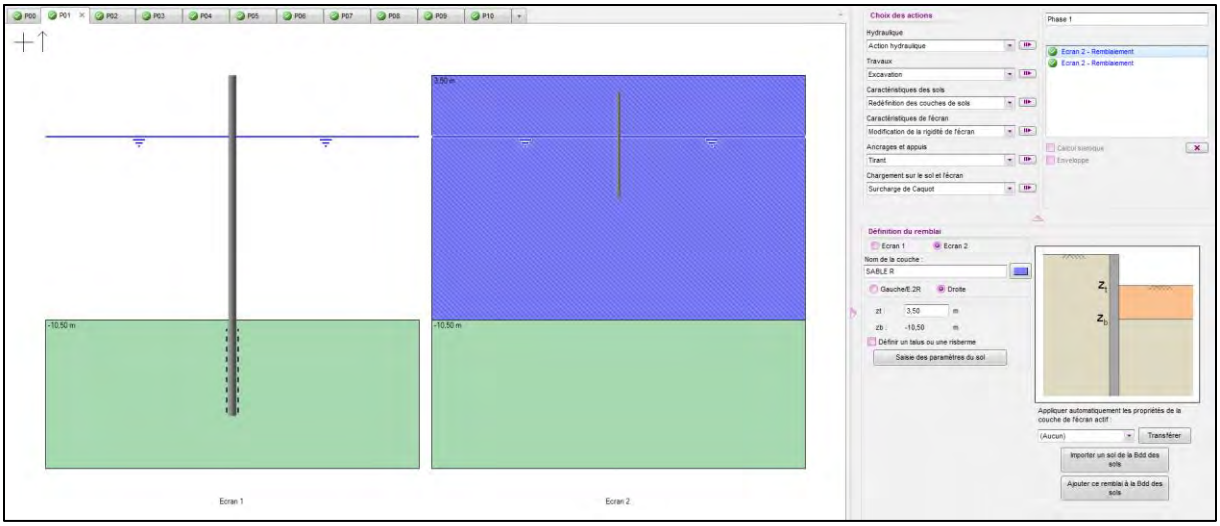


Figure 4 - Fill the screen 2 at both side

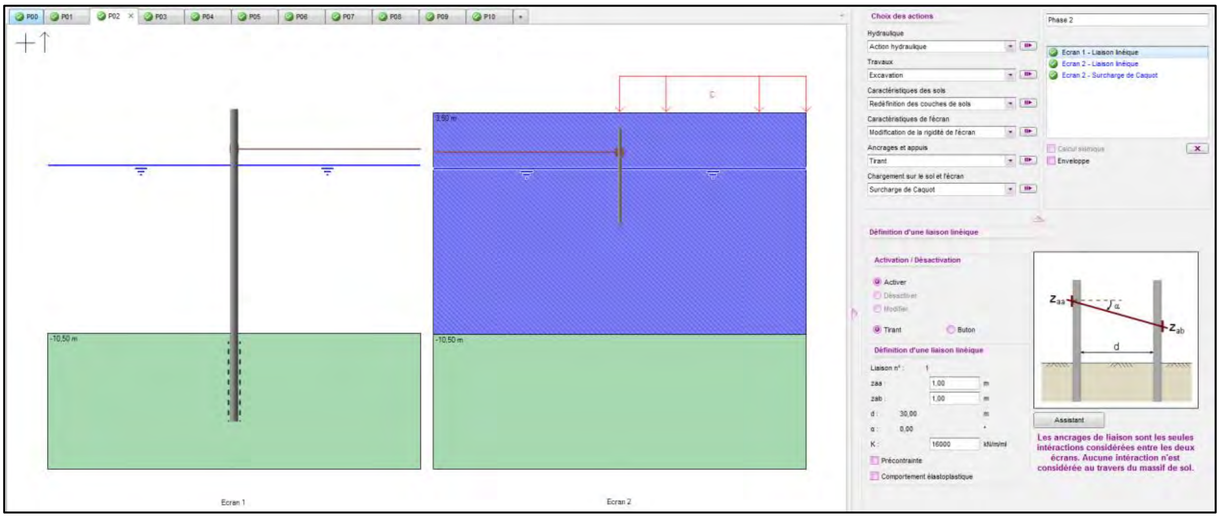


Figure 5 - Tie rode between pile and sheet pile

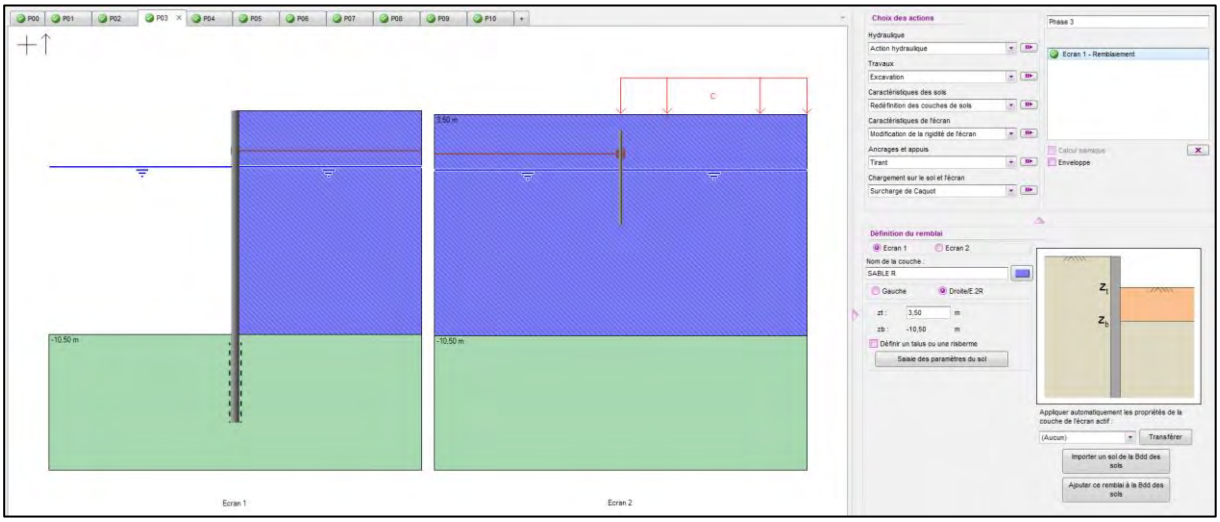


Figure 6 - Fill at the right of the pile

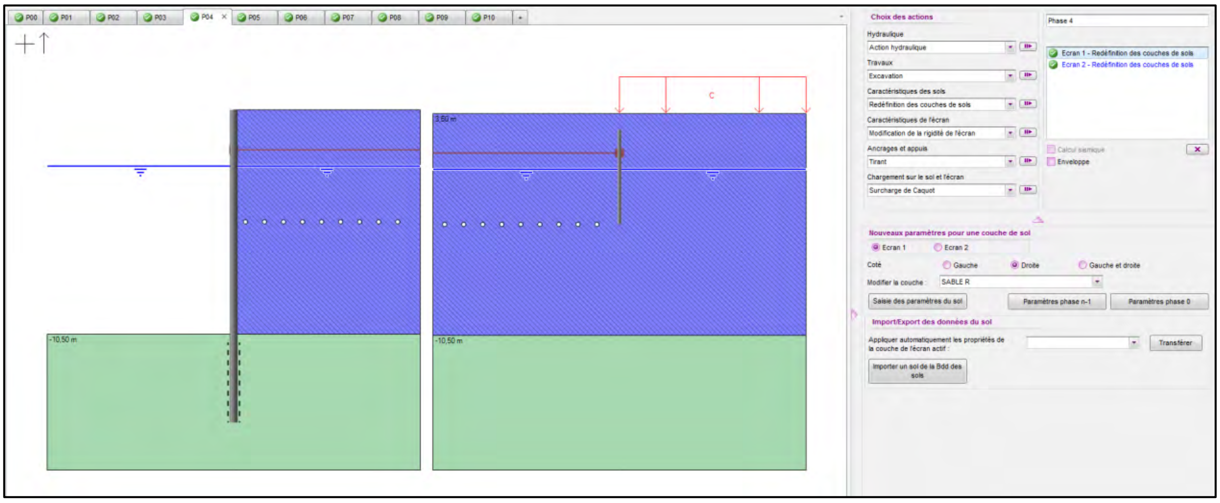


Figure 7 - Change the characteristics of the backfilling soil

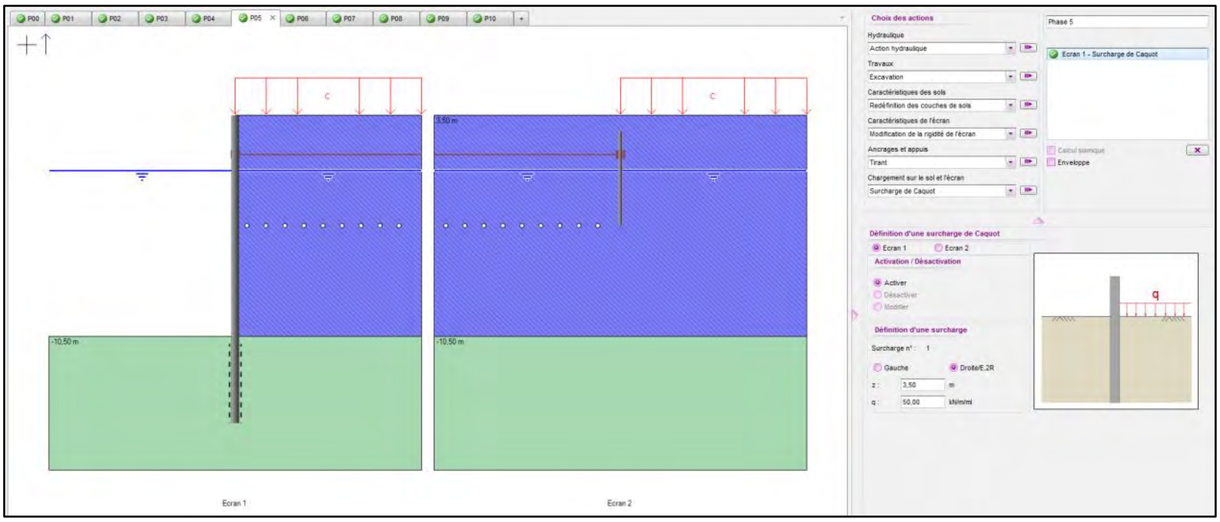


Figure 8 - Caquot load 50kN at the right side of the pile

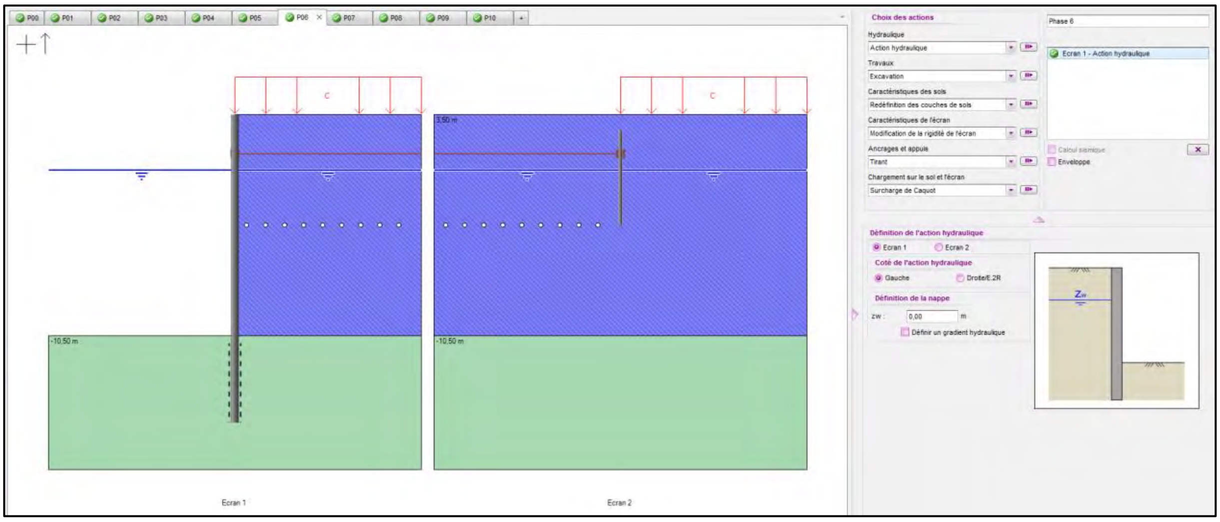


Figure 9 - Hydraulic action on the quay wall

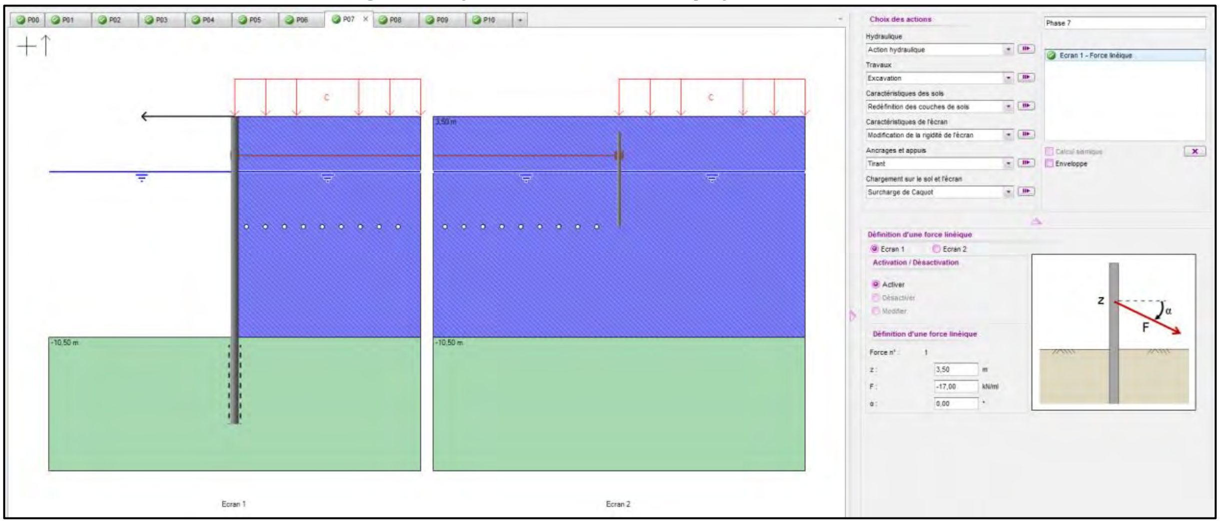


Figure 10 - Apply the fenders force of 130kN

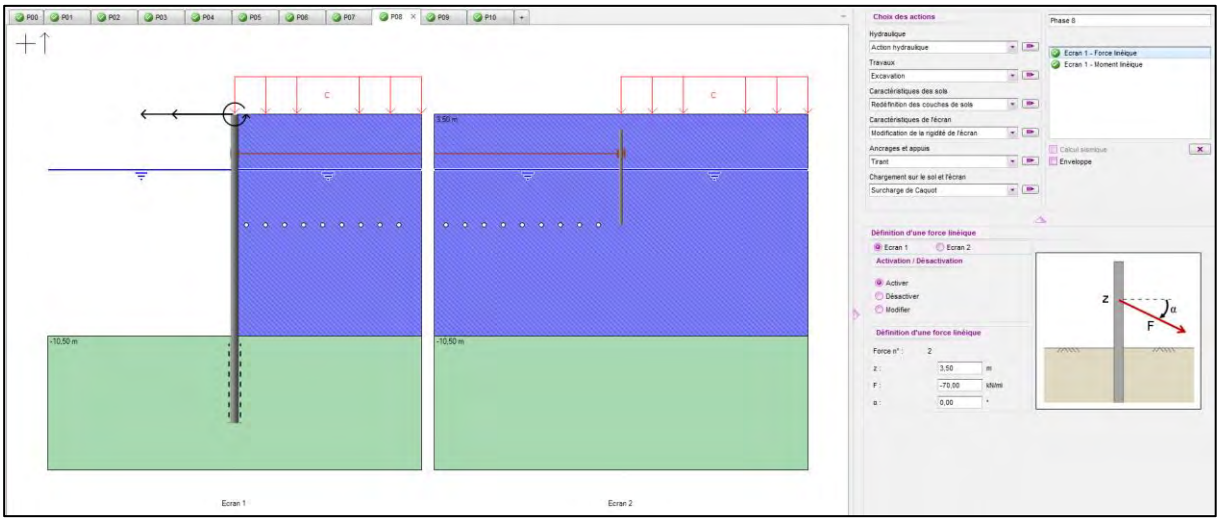


Figure 11 - Apply the Bollard information

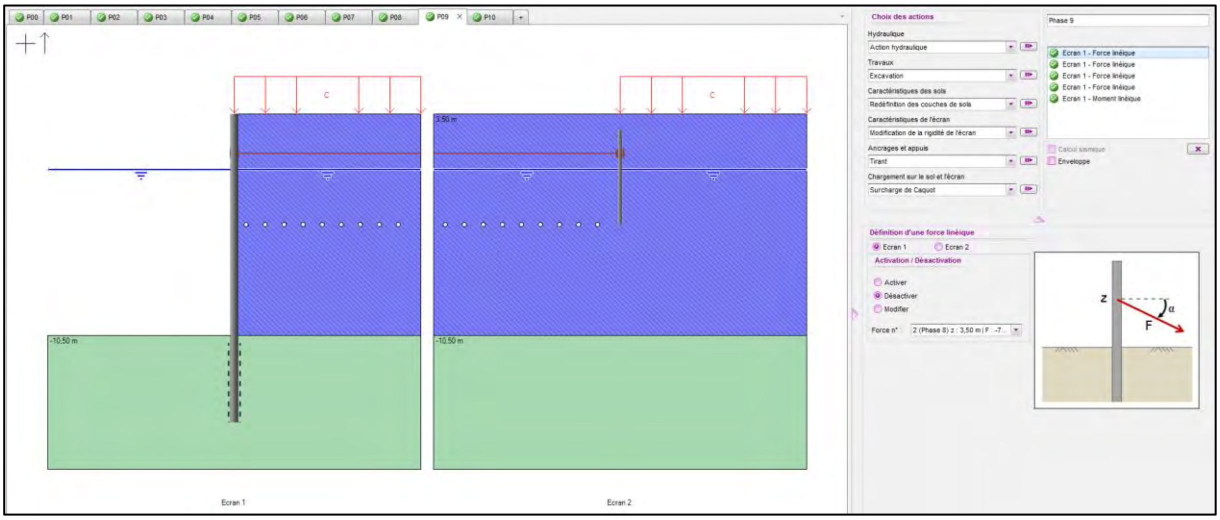


Figure 12- Combination of the different forces

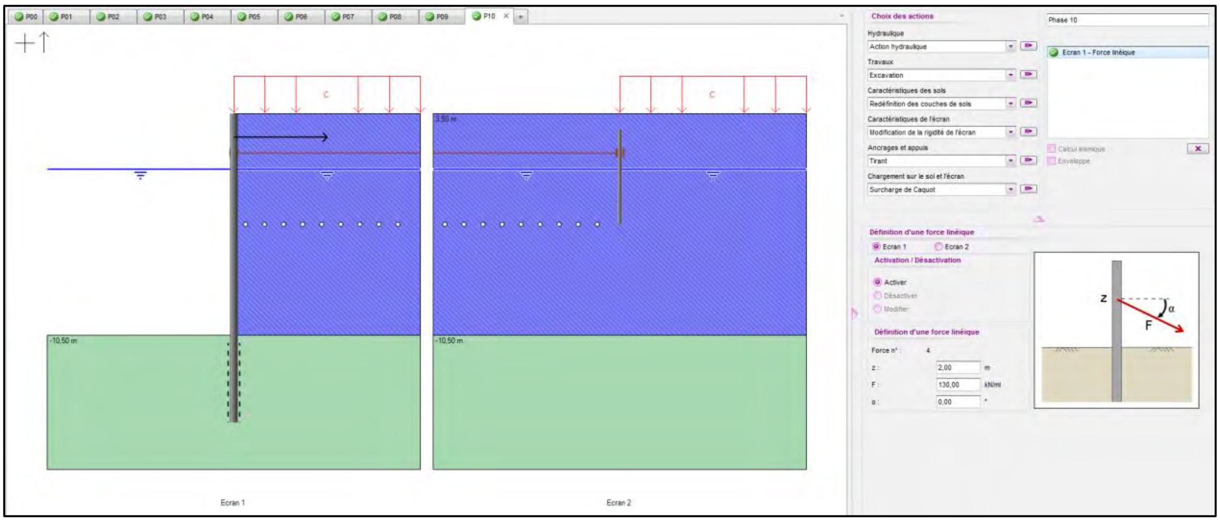


Figure 13 - Apply ship loader load

QUAY B

For the quay B, we have done the same step to dimension our quay wall. Some changes about the pile have be done (pile & sheet pile parameters). Also, the quay B have a different support of soil then the quay A, so we have to change the layers.

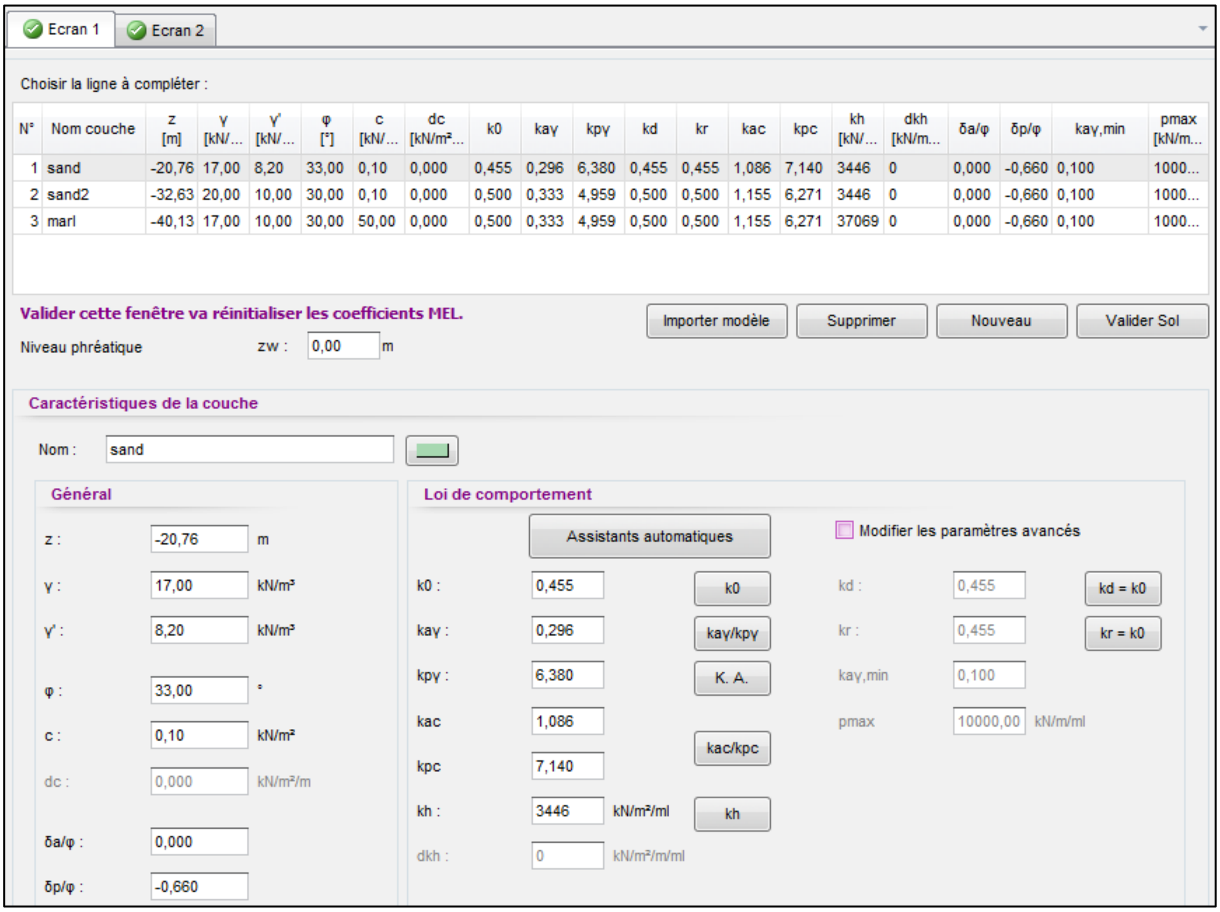


Figure 15 - Soil parameters for the quay B

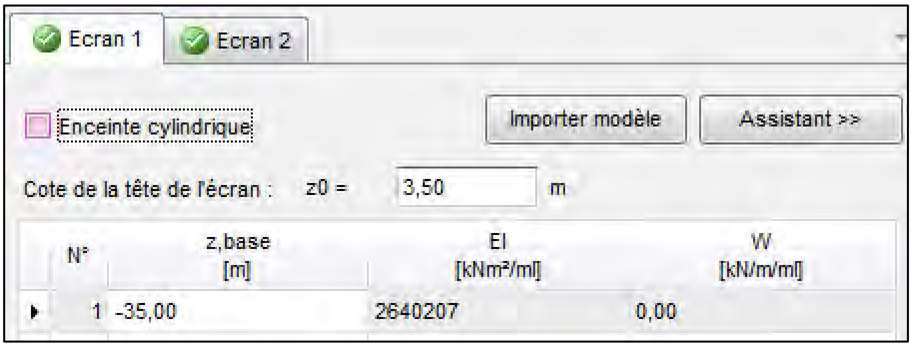


Figure 16 - Screen 1 for the quay B

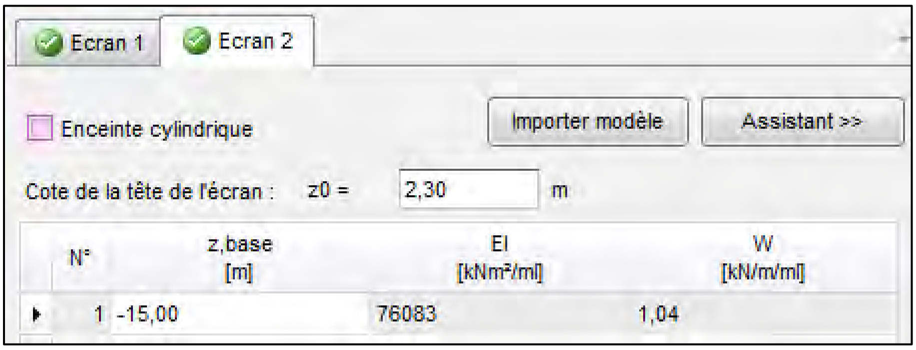


Figure 17 - Screen 2 for the quay B

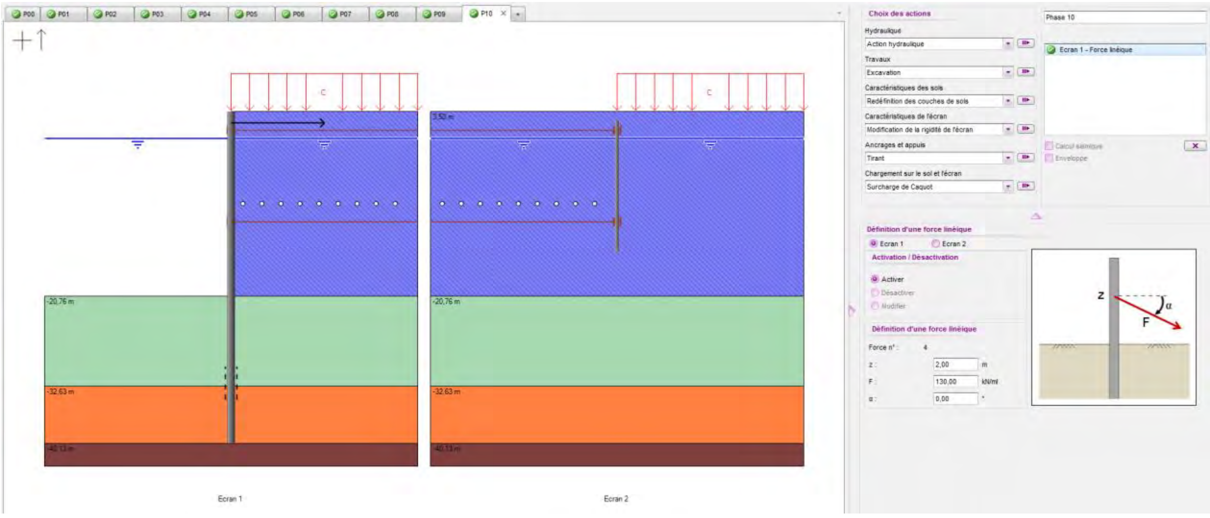
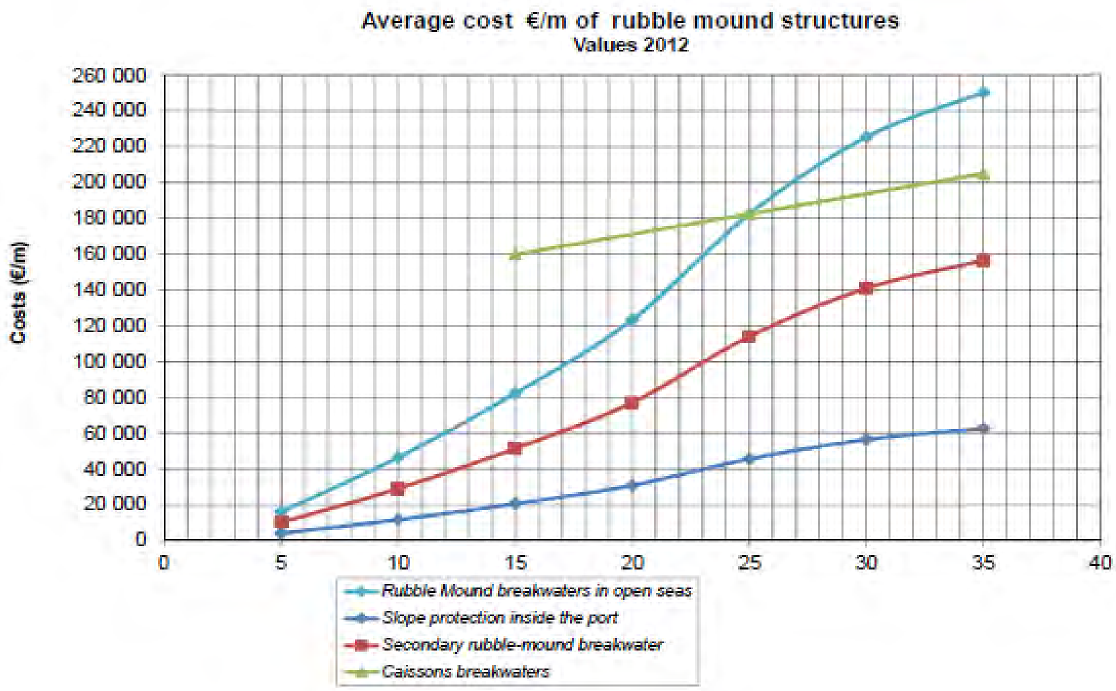


Figure 18 - Final step of the quay B dimensioning

Appendix D. Costing.

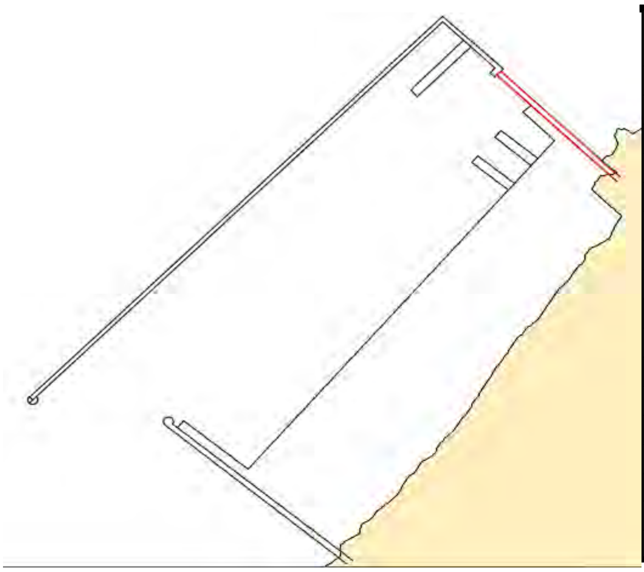
The design consists of multiple breakwaters. There are two rubble mound breakwaters: one in open sea and one sheltered by the main caisson breakwater.

In figure below the average costs are given for rubble mound structures.



Rubble mound breakwater in open sea

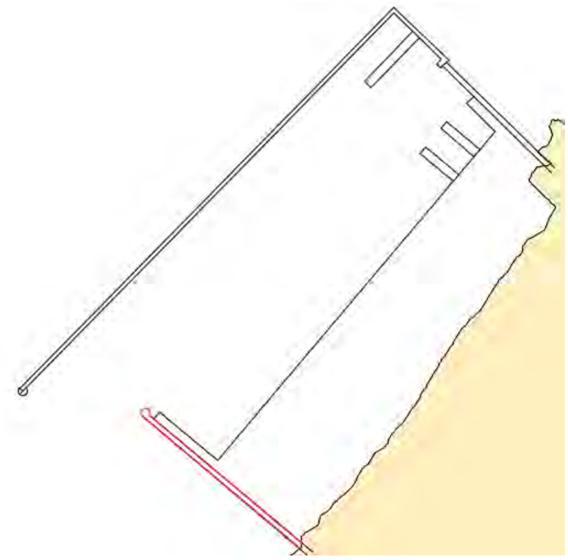
In figure on the next page the location of the breakwater in open sea is indicated. And from table on the next page the costs per depth can be obtained. The cost of the rubble mound breakwater in open sea is € 24 609 000.



Rubble mound breakwater in open sea			
Depth (m)	Unit Cost (€/ml)	Length of our project (m)	Cost (€)
0	15000		
		294	4410000
5	15000		
		331	9930000
10	45000		
		163	10269000
15	81000		
		0	0
20	122000		
		0	0
25	181000		
		0	0
30	222000		
		0	0
35	250000		
	TOTAL	788	24609000

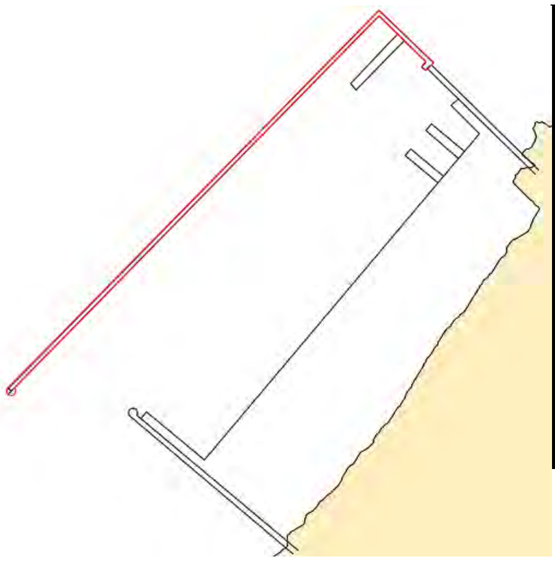
Secondary rubble mound breakwater

In the figure below the location of the secondary breakwater is indicated. And from the table below the costs per depth can be obtained. The cost of the rubble mound breakwater in open sea is € 42 350 000.



Caissons breakwater

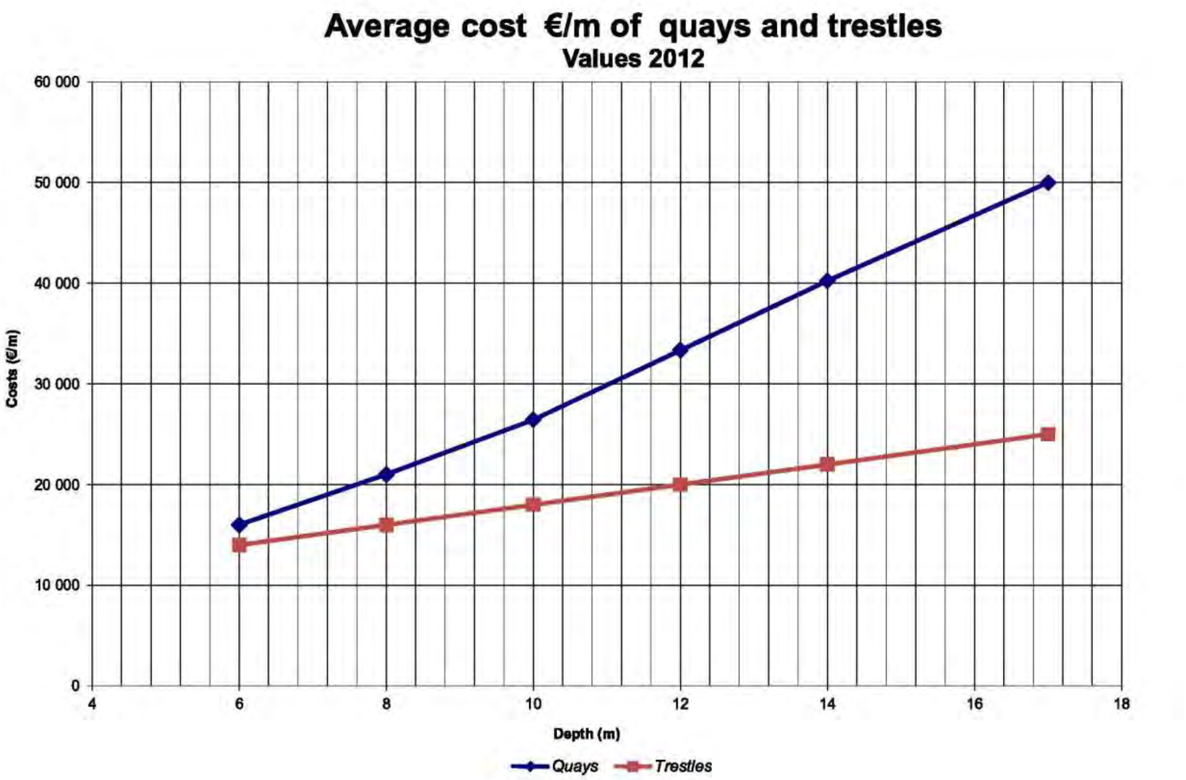
In figure below the location of the caisson breakwater is indicated. And from table below the costs per depth can be obtained. The cost of the rubble mound breakwater in open sea is € 667 725 000.



Caissons breakwater			
Depth (m)	Unit Cost (€/ml)	Length of our project (m)	Cost (€)
15	160000		
		80	13200000
20	170000		
		120	21000000
25	180000		
		120	22350000
30	192500		
		60	11775000
33	200000		
		2960	599400000
35	205000		
	TOTAL	3340	667725000

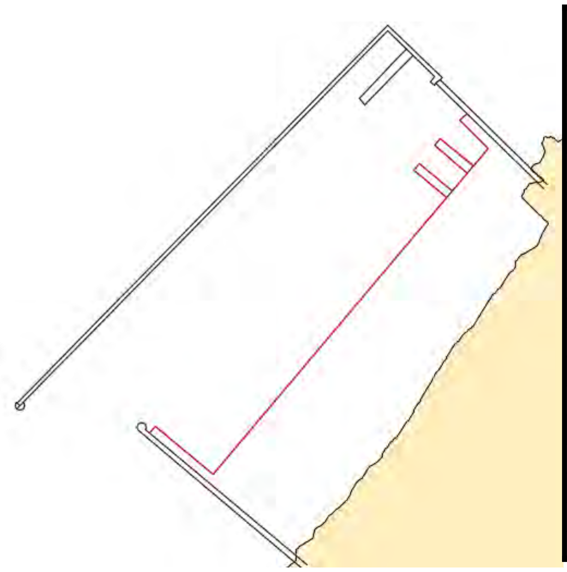
Quay and jetty costs

Figure below gives the costs for the quay wall and the jetty.



Quay

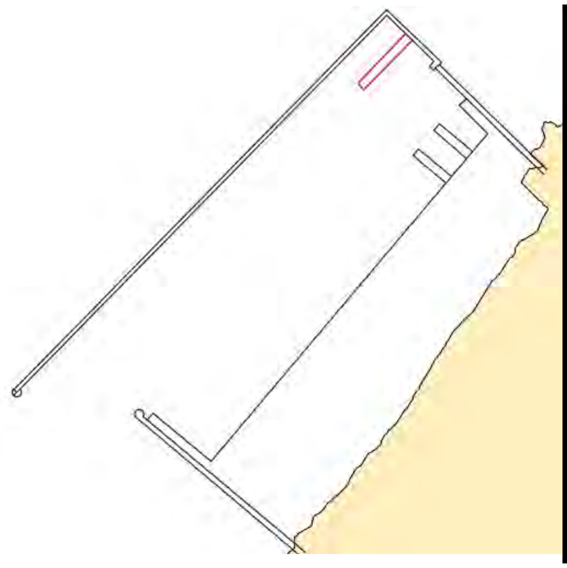
In figure below the location of the quay wall is indicated. And from Table xx the costs per depth can be obtained. The cost of the rubble mound breakwater in open sea is € 92 042 200.



Quays			
Depth (m)	Unit Cost (€/ml)	Length of our project (m)	Cost (€)
6	16000		
		1808,5	33457250
8	21000		
		1658,7	38979450
10	26000		
		109,6	3288000
12	34000		
		64,9	2401300
14	40000		
		56,9	2446700
16	46000		
		25	1200000
17	50000		
		178,6	10269500
22	65000		
	TOTAL	3902,2	92042200

Jetty

In figure below the location of the jetty is indicated. And from Table below the costs per depth can be obtained. The cost of the rubble mound breakwater in open sea is € is 11 935 000.



Trestles			
Depth (m)	Unit Cost (€/ml)	Length of our project (m)	Cost (€)
6	14000		
		0	0
8	16000		
		0	0
10	18000		
		0	0
12	20000		
		0	0
14	22000		
		0	0
16	24000		
		0	0
17	25000		
		0	0
22	30000		
		385	11935000
24	32000		
	TOTAL	385	11935000

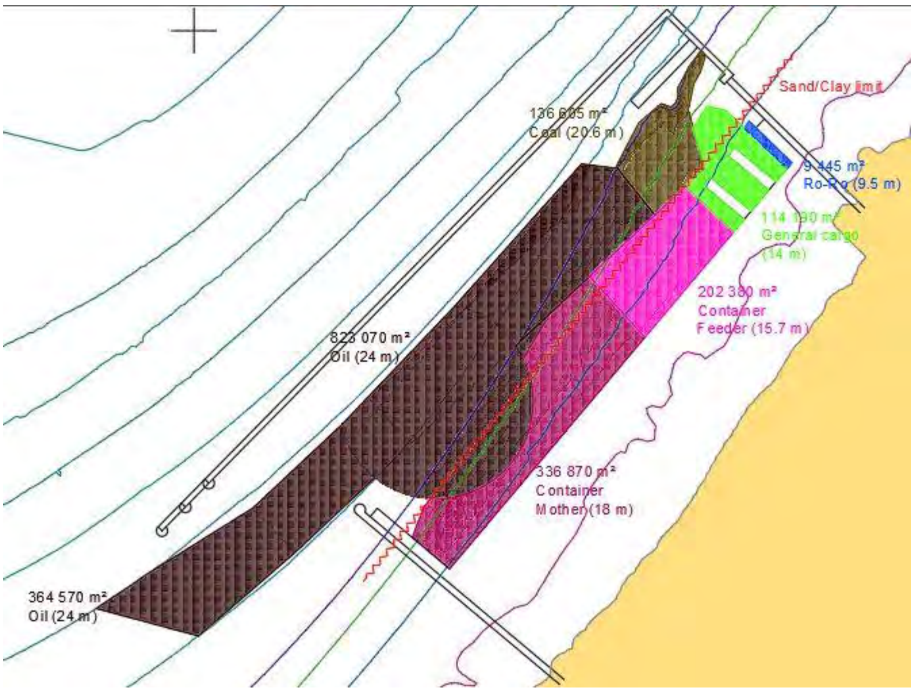
Jetty – Head jetty

Jetty – Jetty head			
Item	Unit Cost (€/u or ml)	Quantity in our project (u or m)	Cost (€)
Mooring dolphins	1700000	0	0
Berthing dolphins	2100000	0	0
Platform	6500000	0	0
Walkways	9000	4000	36000000
Fender system	150000	72	10800000
QRMH	80000	0	0
		TOTAL	46800000

The quantity of the fender system was calculated with 4 fenders per berth (16 berths) plus 2 fenders per tug boat (4 tugs boats).

Dredging

In figure below the dredged areas are indicated with different colours. For each transhipment the ships have different draughts. Therefore some areas need to be deeper than others.



Dredged material quantity

In the following two tables the amount of dredged material is calculated for sand and mud.

Ship		SAND				
Type	Draught + 2m	Depth min	Depth max	Dredging mean witdht (m)	Surface (m²)	Volume (m3)
Container Feeder	15,7	7,5	13	5,45	178640	973588
Container Mother	18	9	14,5	6,25	291613	1822581,25
Oil large	24	11	17	10	89585	895850
Coal	20,6	11	12	9,1	1175	10692,5
Roro	9,5	6	9,5	1,75	9445	16528,75
Cargo	14	6	11	5,5	87140	479270
						4198511

Ship		MUD				
Type	Draught + 2m	Depth min	Depth max	Dredging mean witdht (m)	Surface (m²)	Volume (m3)
Container Feeder	15,7	13	15,7	1,35	23740	32049
Container Mother	18	14,5	18	1,75	45257	79199,75
Oil large	24	17	22	4,5	1098055	4941247,5
Coal	20,6	12	18,6	5,3	135430	717779
Roro	9,5	0	0	9,5	0	0
Cargo	14	11	12	2,5	27050	67625
						5837900

Backfilling quantity

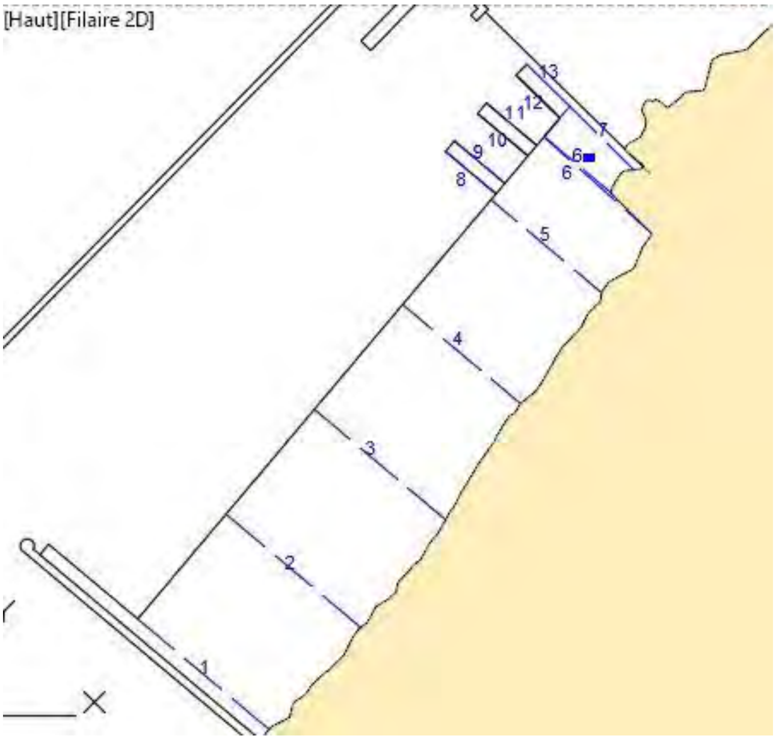
In the following table belowthe volumes are given for backfilling the quay walls. In figure below the sequence of the filling is given.

Perfil	Perfil length (m)	Quay altitude (m)	Surface (m², between 0mCD and the bathy)	Surface (m², 0 mCD and Quay)	Mean distance between perfils (m)	Volume (m3)
1	626,5	3	3109,7	1879,5		
					500	2643775
2	644,3	3	3653	1932,9		
					500	2572975
3	620,7	3	2843,9	1862,1		
					500	2221925
4	561	3	2498,7	1683		
					500	1934612,5
5	517,75	3	2003,5	1553,25		
					301	958466,775
6	529,6	3	1223	1588,8		
6 bis	320,9	3	1230,6	962,7		
					131,5	281666,425
7	330,9	3	1097,9	992,7		
8	240	3	2040,5	720		
					50	136340
9	240	3	1973,1	720		
10	240	3	1865,7	720		
					50	128467,5
11	240	3	1833	720		
12	223,6	3	1556,7	670,8		
					55	119520,5
13	219,4	3	1460,5	658,2		
						10997748,7

Dredging costs

In the following Table the costs are given for the dredging works.

Dredging filing			
Item	Unit Cost (€/m3 or u)	Quantity in our project (u)	Cost (€)
Dredging in sand	7	4198511	29389577
Dredging in soft clay	10	5837900	58379000
Dredging in hard clay	50	0	0
Dredging in rock	80	0	0
Backfilling with dredged material	3	4198511	12595533
Backfilling with sand dredged offshore the port	15	2266413	33996189
Backfilling with quarry run (inshore)	30	0	0
Backfilling with material excavated in land	12	4532825	54393902
Substitution with quarry run (offshore)	40	0	0
Mobilisation/demobilisation of dredger	2000000	1	2000000
		TOTAL	190754200



Some assumptions are done concerning the backfilling :

- 100 % of the dredged sand is reused
- the backfilling = 40% dredged material (the totality of the dredged sand) + 40% material excavated in land + 20% sand dredged offshore the port

The costs of the dredging works are € 190 754 200

Soil reinforcement

The total costs of the soil reinforcement is given in the following table.

The costs for the soil reinforcement are € 9 639 000.

	Rigid Inclusions
Quantity	
Length of the breakwater (m)	3400
Surface to reinforce (m²)	214200
Depth of the column (m)	10
A_{mesh} (m²)	1,4
Number of columns	153 000
Diameter of the column (m)	0,3
Surface of the column (m²)	0,07
Total volume of the columns (m3)	107100
Construction cost	
Drilling price / column	280
Total drilling price	42 840 000
Filling with concrete (offshore) €/m3	90
Total filling price	9 639 000

Appendix E. Queuing Theory.

The following procedure is followed for the queuing theory for each transhipment:

1. Arrival rate λ (calls/year)

The arrival rate for the container terminal depends on: the annual volumes (TEU), the % of income, the kind of ship (mothers/feeders) and the average capacity.

Lambda is calculated in the following way:
λ = (Volume * % income) / Average capacity

2. Service time (hours)

The service time is the number of hours the ship needs to be berth and unload/ load.

$$service\ time = \frac{average\ capacity}{1.5\ \times (unloading\ rate) \times (number\ of\ cranes)} + 2$$

Assumed is that there are 6 cranes per berth for mothers ships and 4 cranes per berth for feeders ships. Also, the berthing time is 2 hours for containers ships.

3. Service rate (μ)

The effective calls is the number of containers ships, which could be welcomed in the harbour. This depends on the specific category (mothers/feeders), the service time and the number of operating hours of the terminal.
The service rate $\mu = \frac{operational\ hours\ per\ year}{service\ time\ per\ vessel}$

4. Utilization ratio (p)

The utilization ratio of a berth is determined by the comparison the arrival rate λ and the effective calls rate μ for one berth. If the value is too high, another berths are needed in order to decrease the percentage of waiting time of the ship. This value can't exceed 3% for containers mothers and feeders vessels and 8% for other ships.

Utilization ratio $= p = \frac{\lambda}{\mu}$
New utilization ratio = Utilization ratio / N° berths

Specific tables (see Figure 1) are necessary to check the waiting time according to number of berths and waiting time. For example for containers mothers ships a statistical distributions like $M/E_2/6$ is assumed.

TABLE IV

Average waiting time of ships in the queue **$M/E_2/n$**
(In units of average service time)

utilisation (u)	number of servers (n) 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
0.10	0.08	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.15	0.13	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.20	0.19	0.03	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.25	0.25	0.05	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.30	0.32	0.08	0.03	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.35	0.40	0.11	0.04	0.02	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.40	0.50	0.15	0.06	0.03	0.02	0.01	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.45	0.60	0.20	0.08	0.05	0.03	0.02	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.50	0.75	0.26	0.12	0.07	0.04	0.03	0.02	0.01	0.01	0.01	0.00	0.00	0.00	0.00	0.00
0.55	0.91	0.33	0.16	0.10	0.06	0.04	0.03	0.02	0.02	0.01	0.01	0.01	0.00	0.00	0.00
0.60	1.13	0.43	0.23	0.14	0.09	0.06	0.05	0.03	0.03	0.02	0.02	0.01	0.01	0.01	
0.65	1.38	0.55	0.30	0.19	0.12	0.09	0.07	0.05	0.04	0.03	0.03	0.02	0.02	0.02	
0.70	1.75	0.73	0.42	0.27	0.19	0.14	0.11	0.09	0.07	0.06	0.05	0.04	0.03	0.03	
0.75	2.22	0.96	0.59	0.39	0.28	0.21	0.17	0.14	0.12	0.10	0.08	0.07	0.06	0.05	
0.80	3.00	1.34	0.82	0.57	0.42	0.33	0.27	0.22	0.18	0.16	0.13	0.11	0.10	0.09	
0.85	4.50	2.00	1.34	0.90	0.70	0.54	0.46	0.39	0.34	0.30	0.26	0.23	0.20	0.18	
0.90	6.75	3.14	2.01	1.45	1.12	0.91	0.76	0.65	0.56	0.50	0.45	0.40	0.36	0.33	

Figure1: The used table for the queuing theory(Groenveld, 2001)

Finally the quay lengths for each terminal can be calculated by using the amount of berths needed.

- Ships in row: L = 1.1* (N° berths)* (Mean length + 15) +15
- Ship alone (one berth): L = Length of the ship + 30

Appendix F. Ground improvement techniques

The clay is cohesive soils which can just be improved with those methods:

➢ **Direct action on soil structure (No material added):**

6. Consolidation by vertical drains + preloading
7. Vacuum consolidation

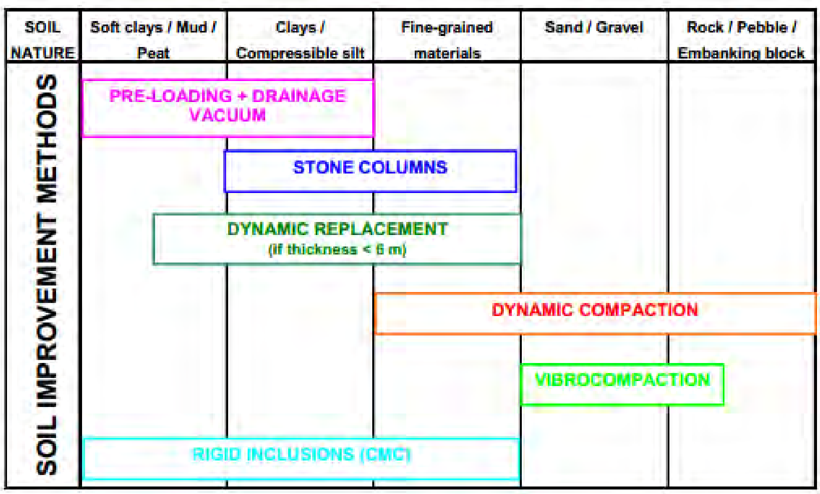
4. Reinforcement by inclusions (Material added):

Inclusions of granular material:

8. Dynamic replacement
9. Stone columns

Rigid inclusions:

10. Pile like inclusions such as CMC's, VCC's
11. Jet grouting columns
12. Soil mixing columns



Treatment methods depending on the soil nature

In our geotechnical condition, the technical solution which can be carried out are pre-loading + drainage vacuum, stone columns, dynamic replacement (impossible because thickness need to be inferior to 6m) or rigid inclusions.

1. **The pre-loading + drainage vacuum**

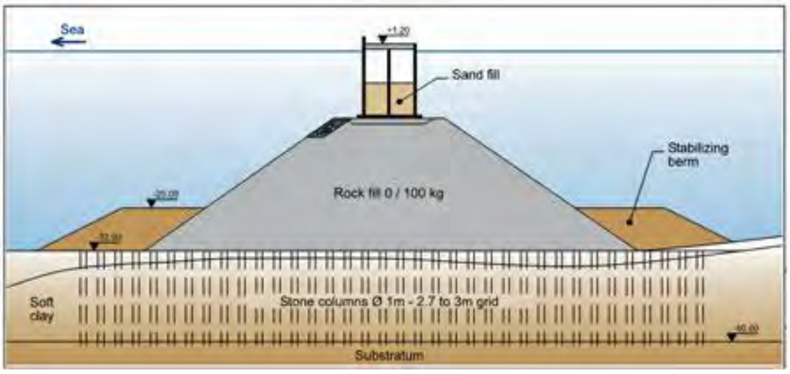
The pre-loading technique of ground improvement is an effective treatment for soils having high compressibility and low shearing strength. The methodology process allowed starting the settlement of the ground before the construction by the implementation of pre-load with earth fills, water lowering the utilization of vacuum technique under impervious membrane. The pressure on the weak soil usually rages from 1.2 to 1.3 times of the future structural pressure of the platform. Once the settlement under the pre load is completed, this one is removed and the construction process is started. A drainage vaccum is not possible on the seabed, so we can just study the possibility of a pre loading. This proposed technical solution can be applied by providing materials for generating a pre loading at low costs. It our case we will have to bring materials from the sea by dredging, which go against our aim to minimize this one. Then we have to take into account a sufficient time for the pre loading program.

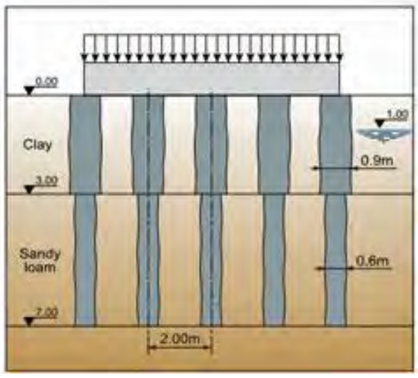
2. **The dynamic replacement**

The dynamic replacement is not a technical solution available for our project because we have a layer of 10m clay and it can only work if the thickness of the layer is inferior to 6m. The rigid inclusions

3. **The stone columns**

A soil can be reinforced with a stone columns technical solution which is the introduction of porous elements with good engineering properties on a regular grid.





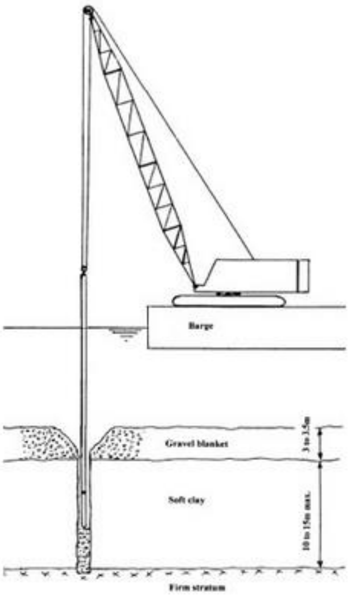
The effects on the soil treated are:

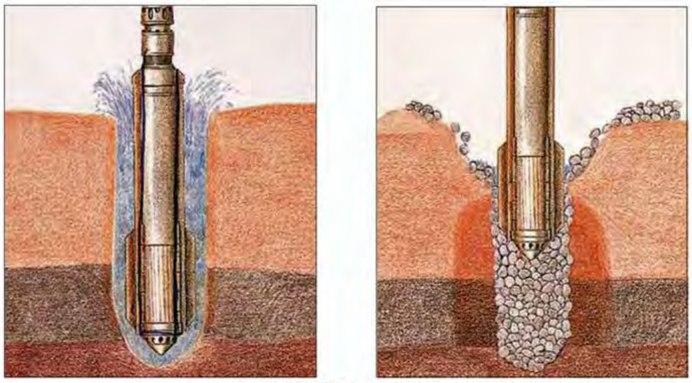
- the improvement the modulus of deformation of the whole treated mass,
- the increasing of the average angle of internal friction and the overall shear strength,
- the increasing of the coefficient of earth pressure at rest (K0),
- the significant increasing of the rate of consolidation, with most of the settlement occurring within a short time after construction.

Most of the stone columns reinforcement methods are available for loads spread over a large area on land or off-shore.

In clayed and silty soils two methods, wet top feed or dry bottom feed can be executed to form stone columns.

- In Wet-top feed method, vibrator is lowered down to the desired depth with the help of the water jet which creates a space around itself. After this operation, the hole is filled by stones from the surface of the soil.

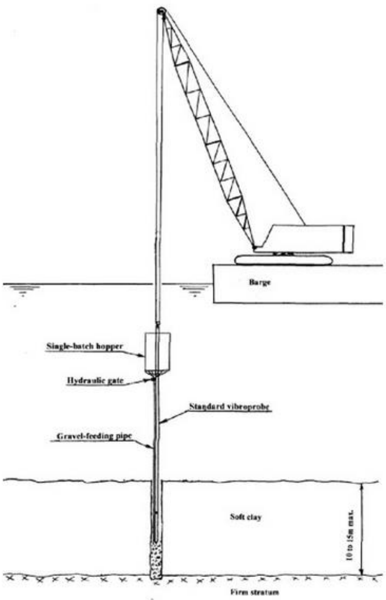


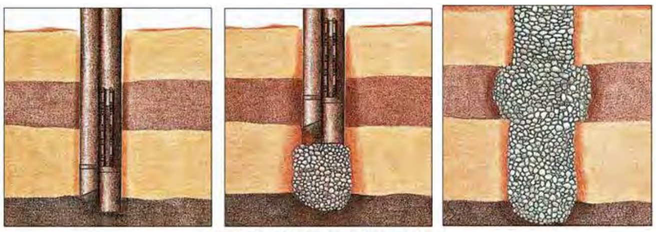


Wet-top feed stone column method

- In Dry-bottom feed method, vibrator is lowered down to the desired depth with the help of the help of the air jet. After this operation, stones are placed from the bottom of the drilling area with the help of the pipe to form up the stone column to the surface

  The diameter of the stone column depend of the density of the surrounding soil.





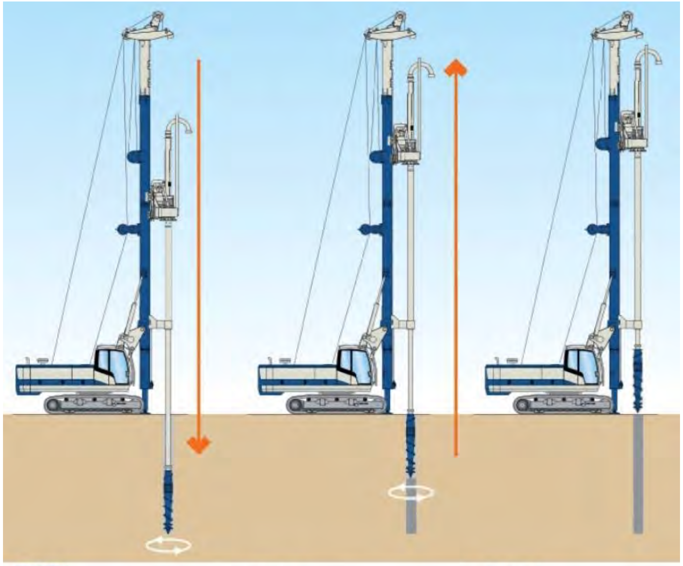
Dry-bottom feed stone column method

4. **The rigid inclusions**

The rigid inclusions can be piles (concrete or mortar columns), jet grouting, and soil mixing columns which are installed through weak, highly compressible soils to reduce settlement and increase bearing capacity. They are not directly connected to the foundations. A load transfer platform is installed between the foundation and the rigid inclusions.

The system use either a rotary displacement hollow auger or a vibrated driven steel tube which can tighten the surrounding soil. When the design depth is achieved, a concrete is bottom fed by pumping continuously at positive pressure through the hollow tube during the extraction to form the rigid inclusion. We have a combined effect of densification and reinforcement which improve the performance of the soft soil directly after the operation.

The productively is quite high so the cost is competitive.



STAGE 1.
DISPLACEMENT AUGER PENETRATES TO DESIGN DEPTH

STAGE 2.
AUGER EXTRACTED AND GROUT/CONCRETE IS PUMPED UNDER CONTROLLED PRESSURE

STAGE 3.
COMPLETED PENCOL™ INCLUSIONS

Appendix G. Soil Reinforcement.

The Design of the Reinforced Soil Column

- **The long term predictable settlement**

$\delta = \Delta\sigma \cdot H / E_{oed}$
$\Delta\sigma = 401{,}72\ kN/m^2$ and E_{oed} =2 MPa for clay soil (Assumption)
H=10 m, height of the clay layer

$\delta = \Delta\sigma \cdot H / 2000 = 2\ m$

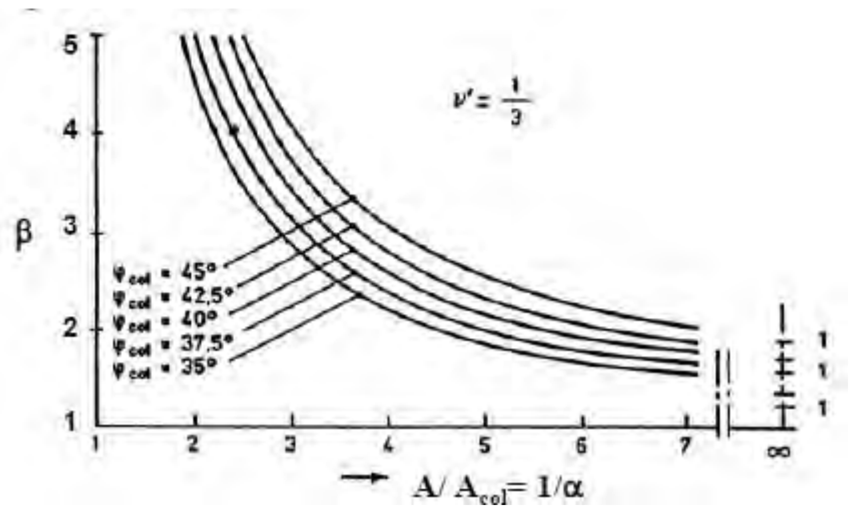
- **Reinforcement with ballast column**

The utilization of ballast column to reinforce the weak soil, allowed to divide the settlement by an average efficiency factor β=3, that we assumed as the concentration factor n=10 (maximal value).
$B = 1 + (n - 1)\ \alpha$ so $\alpha = 0{,}22 = A_{column}/A_{mesh}$

Or the diameter of the stone columns have to superior to 80cm and the coverage area ratio α=10 to 35%. If we choose a column of diameter 90cm, A_{column}= 0.64m², with φ=40°.

Check of overall stability:

According to Priebe's method, Amesh / Acol = 3.4 and A_{mesh} = 2.16m²
It represents a grid of square with a side of 1,47m.



- **Reinforcement with rigid inclusion**

The utilization of rigid inclusion to reinforce the weak soil, allowed to divide the settlement by a minimum efficiency factor β=10, that we assumed as the concentration factor n=10 (maximal value).

Or the diameter of the stone columns have to superior to 25cm and the coverage area ratio α=2 to 10%. If we choose a column of diameter 30cm, A_{column}= 0,07m² and 5 % coverage area, we have Amesh = 1,4 m². It represents a grid of square with a side of 1,19m.

	Ballast column	Rigid inclusion

Quantity		
Length of the breakwater	3 400 m	3 400 m
Surface to reinforce	214 200 m²	214 200 m²
Depth of the column	10 m	10 m
A_{mesh} (m²)	2,16	1,4
Number of columns	99 167	153 000
Diameter of the column	0.9 m	0.3 m
Surface of the column (m²)	0.64	0.07
Total volume of the columns	634 669 m^3	107 100 m^3
Construction cost		
Drilling price / column	280 €	280 €
Total drilling price	**27 766 760 €**	**42 840 000 €**
Filling with quarry stone (offshore) €/m3	**40**	**x**
Filling with concrete (offshore) €/m3	**x**	**90**
Total filling price	**25 386 760 €**	**9 639 000 €**
Total price	**53 153 520 €**	**52 479 000 €**

Dredging and Substitution

Besides the solutions of consolidating the layer of mud not strong enough to support our breakwater, we can consider the possibility of removing this layer to rest on a "good soil".

This solution would involve dredging the clay layer on an average depth of 9 m, 63 m wide and 3.9 km long (3,4 km under the main breakwater at -35 m depth, 250 m under this same dike towards the coast and 250 m under the secondary dike).

The extraction of the mud is not sufficient, it must be replaced in order to constitute a stable support to our dike. This substitution must be carried out by a material of good quality and with suitable mechanical characteristics.

- **What dredger ?**

Our needs and constraints :

- type of dredging works : capital dredging
- production : high
- swells : to 1,5-2 m
- soil type : < weak rock (mud, soft clay)
- boulders : small
- pump ashore : no (the mud may be discharged offshore)
- precision : medium
- free sailing : yes (there is no maritim traffic currently but we want an unload offshore)

	Hopper	Cutter	Backhoe	Grab	Bucket chain

Production	Very high	Very high	Low-medium	Low-medium	Low-medium
Swells	< 2,5 m	< 1,5 m	< 1 m	< 1 m	< 1 m
Soil types	< weak rock	< medium hard rock	< medium hard rock	< weak rock unless broken	< weak rock
Boulders	Small	Small	Large	Large	Small
Pump ashore	Yes or No	Yes or No	No	No	No
Precision	Medium	High (unless fluidized)	High	Medium-high	High
Free sailing	Yes	No	No	No	No

Comparison of dredgers

After the comparison between 5 different dredgers (hopper, cutter, backhoe, grab and bucket chain), it seems that the best machine is the hopper. The use of an hopper dredger implies a trailing suction fully adapted to the soil type. Regarding the emptying it is possible to discharge the hopper offshore thanks to its bottom disposal.

To optimise the dredge cycle, we may contemplate that after its emptying offshore, the dredger collects good material and then brings it to use it as backfilling.

- **How much does this solution cost ?**

	Unit cost	Quantity	Cost
Dredging on soft clay	10 €/m^3	2 211 300 m^3	22 113 000,00 €
Mobilisation / demobilisation of dredger	2 000 000 €/u	1	2 000 000,00 €
Backfilling with sand dredged offshore the port	15 €/m^3	2 211 300 m^3	33 169 500,00 €
		TOTAL	**57 282 500,00 €**

Appendix H. Fender design.

Fender design - Mother container ships and oil tankers

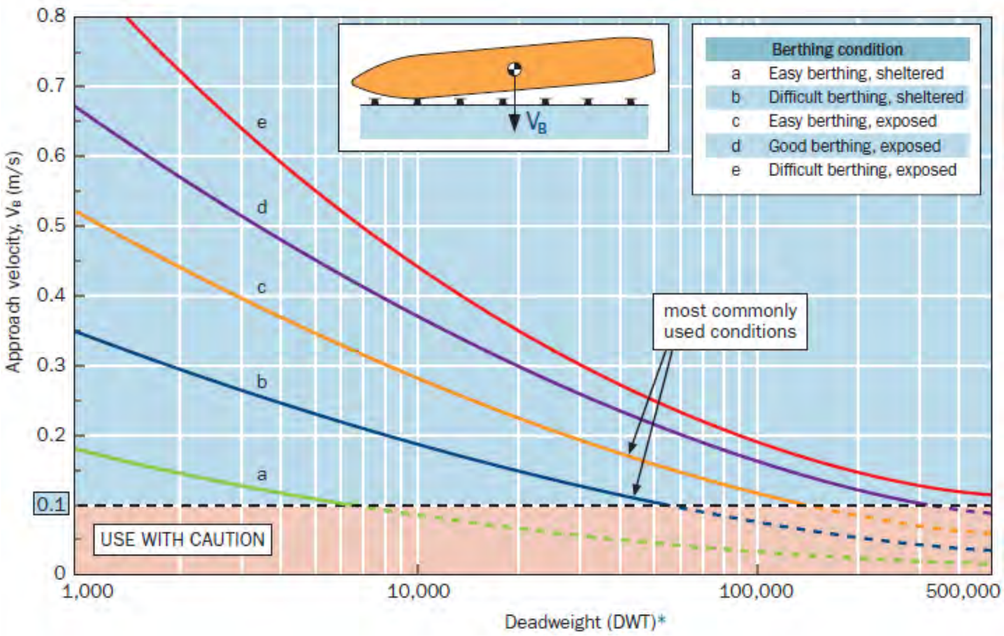
The normal energy to be absorbed by the fender can be calculated as:

$$E_N = 0.5 \times M \times V_B^2 \times C_M \times C_E \times C_C \times C_S$$

Where,
E_N = Normal berthing energy to be absorbed by the fender (kNm)
M = Mass of the vessel (displacement in tonne) at chosen confidence level.*
V_B = Approach velocity component perpendicular to the berthing line† (m/s).
C_M = Added mass coefficient
C_E = Eccentricity coefficient
C_C = Berth configuration coefficient
C_S = Softness coefficient

- M: Mass of the vessel, displacement: 249 000 tons
- Vb: approaching velocity according to the displacement of the vessel.

When the berthing condition is difficult and exposed, which is the worst condition of the graph give the value : **v_b =0.14/s**.



Brolsma Table

- **Block coefficient C_B:**

$$C_B = \frac{M_D}{L_{BP} \times B \times D \times \rho_{SW}}$$

where,
M_D = displacement of vessel (t)
L_{BP} = length between perpendiculars (m)
B = beam (m)
D = draft (m)
ρ_{SW} = seawater density ≈ 1.025t/m³

The input datas corresponding at our biggest ship are: M_D = 249 000 tons

B = 54m

D = 16m

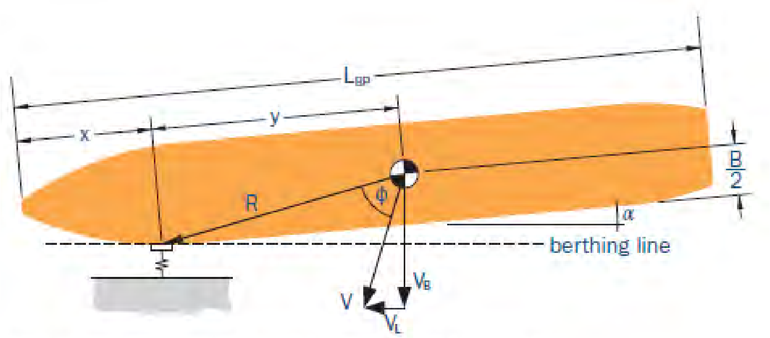
The block coefficient is: **C_B = 0.742**

- **Added Mass coefficient C_M:**
  Kc/D = So according to PIANC,

  $$\text{for } 0.1 \leq \frac{K_c}{D} \leq 0.5 \quad C_M = 1.875 - 0.75\left[\frac{K_c}{D}\right]$$

  C_M= 1,7925
  With an under keel clearance: Kc= 2m and a draft of the ship D = 16m.

- **Eccentricity coefficient C_E:**



V_L = longitudinal velocity component (forward or astern)

$$x + y = \frac{L_{BP}}{2} \quad \text{(assuming the centre of mass is at mid-length of the ship)}$$

$$R = \sqrt{y^2 + \left[\frac{B}{2}\right]^2}$$

$$K = (0.19 \times C_B + 0.11) \times L_{BP}$$

$$C_E = \frac{K^2 + R^2\cos^2\phi}{K^2 + R^2}$$

where,
B = beam (m)
C_B = block coefficient
L_{BP} = length between perpendiculars (m)
R = centre of mass to point of impact (m)
K = radius of gyration (m)

According to the design ship, L_{BP}= 378m and in case of quarter point berthing, x = L_{BP}/ 4 = 94,5m and y = (L_{BP}/ 2) – x = 94,5m.

The maximum breadth is 54m so we obtained the following datas :

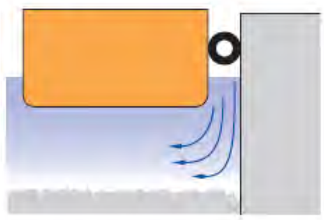
R= 98.28m
α= 8.13°
Φ= -6.838°
K= 94.935
C_E = 0.857

- **Berth configuration coefficient C_C:**

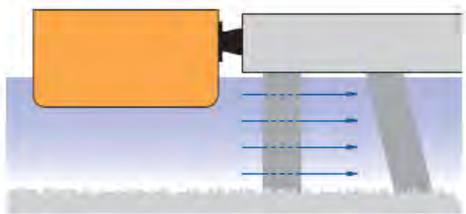
C_c = 0.9 because the caisson is a solid quay structure with berthing angle > 5°.

$C_c = 1.0$	Open structures including berth corners Berthing angles > 5° Very low berthing velocities Large underkeel clearance
$C_c = 0.9$	Solid quay structures Berthing angles > 5°

Closed structure



Semi-closed structure



- **Softness coefficient C_S:**

The choice of the structure defense has fallen upon, the hard fenders with a coefficient Cs=1.0

$C_s = 1.0$	Soft fenders ($\delta_f > 150$mm)
$C_s = 0.9$	Hard fenders ($\delta_f \leq 150$mm)

- **Normal energy of the fender**

These parameters allowed us to calculate the normal energy to be absorbed by the fender.

$$E_N = 0.5 \times M \times V_B^2 \times C_M \times C_E \times C_C \times C_S$$

E_N= 3372 Kn.m

- **Safety factor on the normal energy of the fender**

The usual safety factor for largest container ships terminal, should be 1,25, but the maximal safety factor value is 2.0. It is preferable to take into account the worst situation to avoid a failure of one fender which could make the terminal unserviceable.

The abnormal energy to be absorbed by the fender can be calculated as:

$$E_A = F_S \times E_N$$

E_A= 2.0 x 3372 = 6744 Kn.m

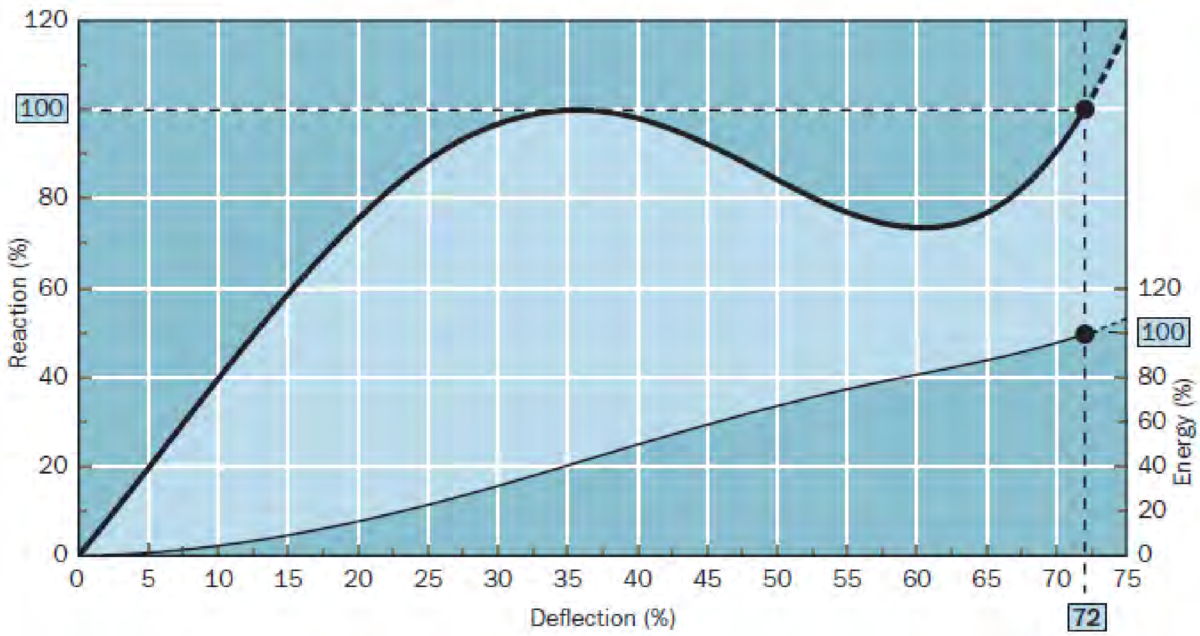
ANC Factors of Safety (F_S)

essel type	Size	F_s
Tanker, bulk, cargo	Largest Smallest	1.25 1.75
Container	Largest Smallest	1.5 2.0
General cargo		1.75
RoRo, ferries		≥2.0
Tugs, workboats, etc		2.0

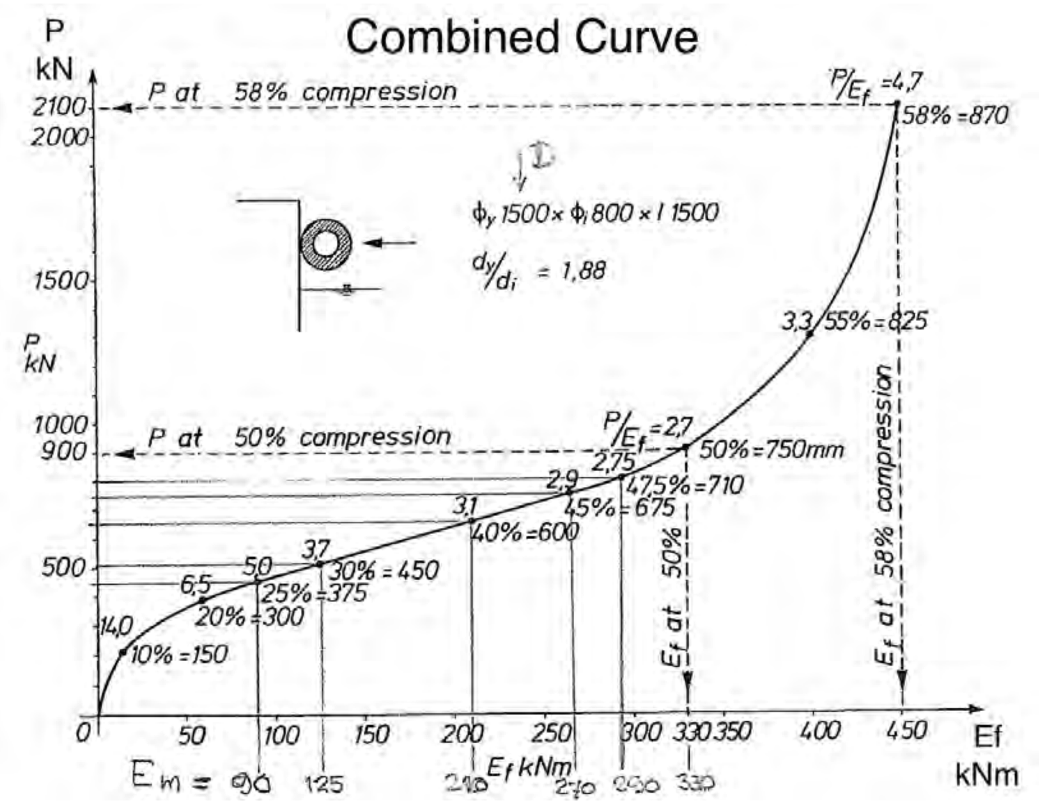
Source: PIANC 2002; Table 4.2.5.

- **Selection of the fender type**

The fender of the oil terminal are elastomeric fenders with the following reaction to pressure.



As can be seen from the graph below, the optimum compression for the fender is around 50%, more than 50% would lead to a big increase of pressure on the quay, less than 50% would lead to a big decrease of fender absorbing energy Ef that can be solved only increasing the size of the fender itself.



So according to the impact energy Ef = 6744 [kN*m] the choice has been made taking into account of two different super cones in order to reduce the pressure on the caisson wall:

- **Super cone fenders**

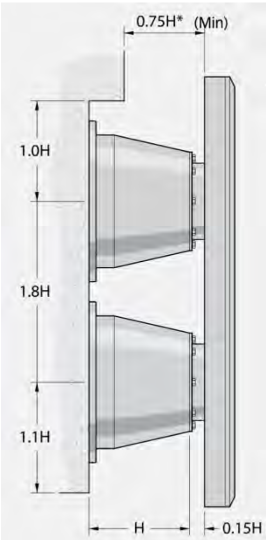
Then this data would be used to determine the geometry of the elastomeric fender panel as shown on the images below:

2000	CV	E	2700.0	3000.0	3080.0	3160.0	3240.0	3320.0	3400.0	3480.0	3560.0	3640.0	3720.0	3800.0
		R	2368.9	2511.0	2578.0	2645.0	2712.0	2779.0	2846.0	2941.8	3037.6	3133.4	3229.2	3325.0
	RPD	E_R	2862.0	3180.0	3264.8	3349.6	3434.4	3519.2	3604.0	3688.8	3773.6	3858.4	3943.2	4028.0
		R_R	2511.0	2661.7	2732.7	2803.7	2874.7	2945.7	3016.8	3118.3	3219.9	3321.4	3423.0	3524.5

Super Cone Fenders

DIMENSIONS

	H	ØW	ØU	C	D	ØB	ØS	F0.9- 1.8 ANCHORS / HEAD BOLTS ^	F1.9- 3.1 ANCHORS / HEAD BOLTS ^	Z_{min}	WEIGHT
SCN 300	300	500	295	27 – 37	20 – 25	440	255	M16	M16	77	40
SCN 350	350	570	330	27 – 37	20 – 25	510	275	M16	M16	77	50
SCN 400	400	650	390	30 – 40	20 – 28	585	340	M16	M20	82	76
SCN 500	500	800	490	32 – 42	30 – 38	730	425	M20	M24	95	160
SCN 550	550	880	540	32 – 42	30 – 38	790	470	M20	M24	95	210
SCN 600	600	960	590	40 – 52	35 – 42	875	515	M20	M30	115	270
SCN 700	700	1120	685	40 – 52	35 – 42	1020	600	M24	M30	120	411
SCN 800	800	1280	785	40 – 52	35 – 42	1165	685	M24	M30	120	606
SCN 860	860	1376	845	40 – 52	35 – 42	1250	735	M24	M30	130	750
SCN 900	900	1440	885	40 – 52	35 – 42	1313	770	M30	M30	135	841
SCN 950	950	1520	930	40 – 52	40 – 50	1390	815	M30	M30	142	980
SCN 1000	1000	1600	980	50 – 65	40 – 50	1460	855	M30	M36	150	1125
SCN 1050	1050	1680	1030	50 – 65	45 – 55	1530	900	M30	M36	157	1360
SCN 1100	1100	1760	1080	50 – 65	50 – 58	1605	940	M30	M36	165	1567
SCN 1200	1200	1920	1175	57 – 80	50 – 58	1750	1025	M30	M42	180	2028
SCN 1300	1300	2080	1275	65 – 90	50 – 58	1900	1100	M36	M42	195	2455
SCN 1400	1400	2240	1370	65 – 90	60 – 70	2040	1195	M36	M42	210	3105
SCN 1600	1600	2560	1570	65 – 90	70 – 80	2335	1365	M42	M48	240	4645
SCN 1800	1800	2880	1765	75 – 100	70 – 80	2625	1540	M42	M56	270	6618
SCN 2000	2000	3200	1955	80 – 105	90 – 105	2920	1710	M42	M56	300	9560
SCN 2250	2250	3600	2205	100 – 120	100 – 110	3285	1930	M48	M56	335	13,500
SCN 2500	2500	4000	2450	120 – 150	100 – 120	3650	2150	M48	M64	375	18,500



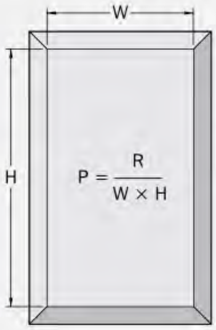
	CONE	PANEL
R (Kn)	2874.7	5749.4
H (m)	2000	7,800
Ø , W (m)	2,920	4
P (Kn/m^2)		184,28

According to PIANC table below, the pressure value gotten from the assumption of 4 meters width is perfect.

Hull Pressures and Beltings

HULL PRESSURES

Allowable hull pressures depend on hull plate thickness and frame spacing. These vary according to the type of ship. Refer to the table on the right for PIANC's guidelines on hull pressures.



P = average hull pressure (kN/m²)
R = total fender reaction (kN)
W = panel width, excluding bevels (m)
H = panel height, excluding bevels (m)

VESSEL TYPE	SIZE/CLASS	HULL PRESSURE (kN/m²)
Container ships	< 1,000 teu (1st/2nd generation)	< 400
	< 3,000 teu (3rd generation)	< 300
	< 8,000 teu (4th generation)	< 250
	> 8,000 teu (5th/6th generation)	< 200
General cargo	≤ 20,000 DWT	400–700
	> 20,000 DWT	< 400
Oil tankers	≤ 20,000 DWT	< 250
	≤ 60,000 DWT	< 300
	> 60,000 DWT	150–200
Gas carriers	LNG/LPG	< 200
Bulk carriers		< 200
RoRo		Usually fitted with beltings (strakes)
Passenger/cruise		
SWATH		

Source: PIANC 2002; Table 4.4.1

Fender design - Oil tankers

The kinetic energy of the approaching vessel have to be absorbed by the potential energy of the fender unit itself. This transfer will bring a reaction load, call berthing load, to the structure supporting the fender.

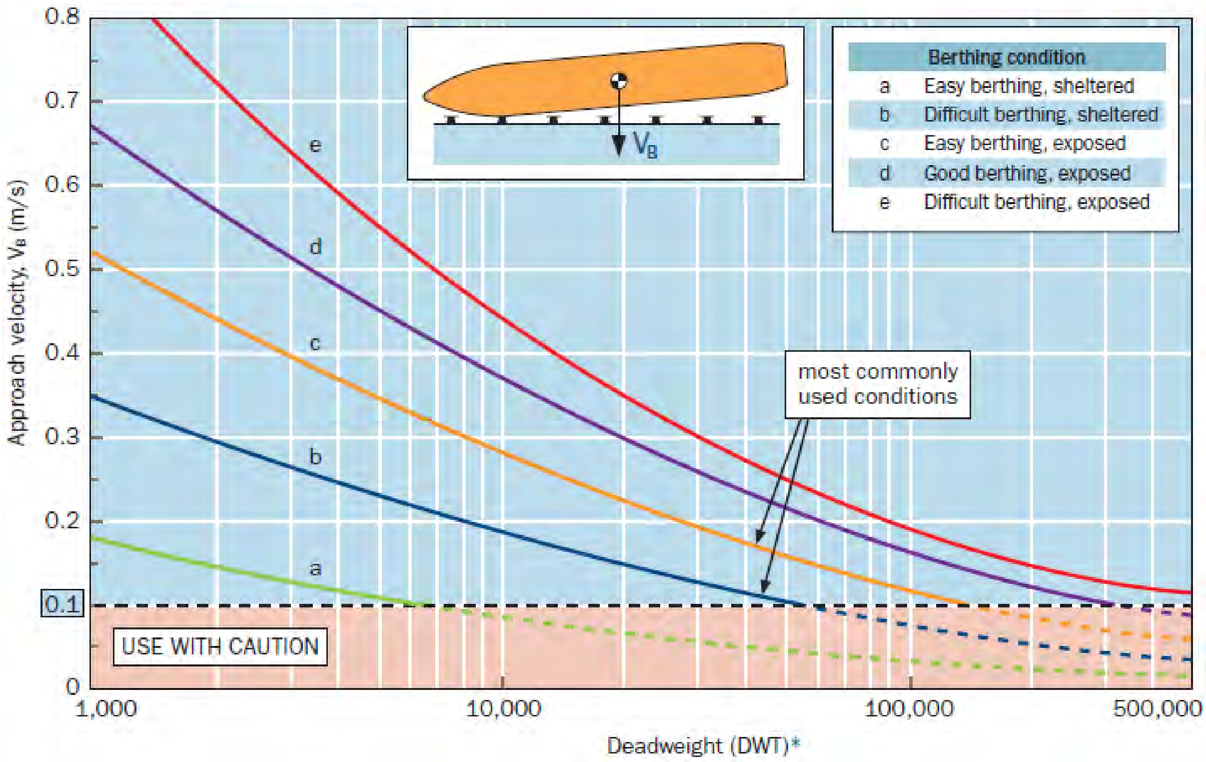
The normal energy to be absorbed by the fender can be calculated as:

$$E_N = 0.5 \times M \times V_B^2 \times C_M \times C_E \times C_C \times C_S$$

Where,
E_N = Normal berthing energy to be absorbed by the fender (kNm)
M = Mass of the vessel (displacement in tonne) at chosen confidence level.*
V_B = Approach velocity component perpendicular to the berthing line† (m/s).
C_M = Added mass coefficient
C_E = Eccentricity coefficient
C_C = Berth configuration coefficient
C_S = Softness coefficient

- M: Mass of the vessel, displacement: 348 760 tons
- Vb: approaching velocity according to the displacement of the vessel.

When the berthing condition is difficult and exposed, which is the worst condition of the graph give the value : **v_b =0.13m/s**.



Brolsma Table

- **Block coefficient C_B:**

$$C_B = \frac{M_D}{L_{BP} \times B \times D \times \rho_{SW}}$$

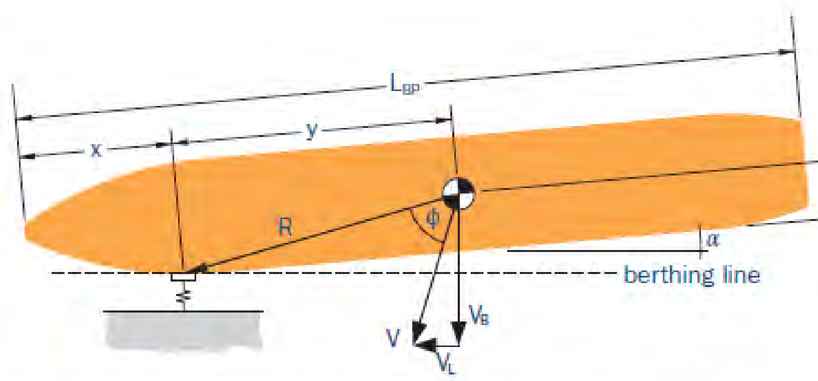
where,
M_D = displacement of vessel (t)
L_{BP} = length between perpendiculars (m)
B = beam (m)
D = draft (m)
ρ_{SW} = seawater density ≈ 1.025t/m³

The input datas corresponding at our biggest ship are: M_D = 348 760 tons
B = 60m
D = 22m

The block coefficient is: **C_B = 0.796**

- **Added Mass coefficient C_M:**
According to PIANC, Kc / D < 0.1 so **C_M = 1.8**
With an under keel clearance: Kc= 2m and a draft of the ship D = 22m.

- **Eccentricity coefficient C_E:**



V_L = longitudinal velocity component (forward or astern)

$$x + y = \frac{L_{BP}}{2}$$ (assuming the centre of mass is at mid-length of the ship)

$$R = \sqrt{y^2 + \left[\frac{B}{2}\right]^2}$$

$$K = (0.19 \times C_B + 0.11) \times L_{BP}$$

$$C_E = \frac{K^2 + R^2\cos^2\phi}{K^2 + R^2}$$

where,
B = beam (m)
C_B = block coefficient
L_{BP} = length between perpendiculars (m)
R = centre of mass to point of impact (m)
K = radius of gyration (m)

According to the design ship, L_{BP}= 323.7m and in case of quarter point berthing, x = L_{BP}/ 4 = 80.925m and y = (L_{BP}/ 2) – x = 80.925m.

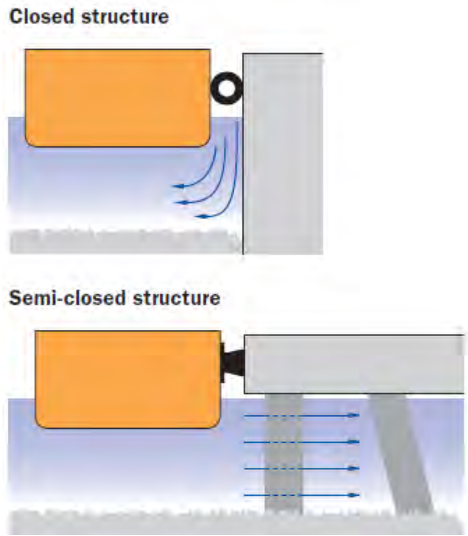
The maximum breadth is 60m so we obtained the following datas :

R= 86.31m
α= 10.5°
Φ= -9.285°
K= 84.583
C_E = 0.99

- **Berth configuration coefficient C_c:**

C_c = 0,9 because the caisson is a solid quay structure with berthing angle > 5°.

$C_c = 1.0$	Open structures including berth corners Berthing angles > 5° Very low berthing velocities Large underkeel clearance
$C_c = 0.9$	Solid quay structures Berthing angles > 5°



- **Softness coefficient C_S:**

The choice of the structure defense has fallen upon, the hard fenders with a coefficient Cs=1.0.

$C_s = 1.0$	Soft fenders ($\delta_f > 150$mm)
$C_s = 0.9$	Hard fenders ($\delta_f \leq 150$mm)

- **Normal energy of the fender**

These parameters allowed us to calculate the normal energy to be absorbed by the fender.

$$E_N = 0.5 \times M \times V_B^2 \times C_M \times C_E \times C_C \times C_S$$

E_N= 4727 Kn.m

- **Safety factor on the normal energy of the fender**

The usual safety factor for largest oil tankers terminal, should be 1,25, but the maximal safety factor value is 2.0. It is preferable to take into account the worst situation to avoid a failure of one fender which could make the terminal unserviceable.

The abnormal energy to be absorbed by the fender can be calculated as:

$$E_A = F_S \times E_N$$

ANC Factors of Safety (F_S)

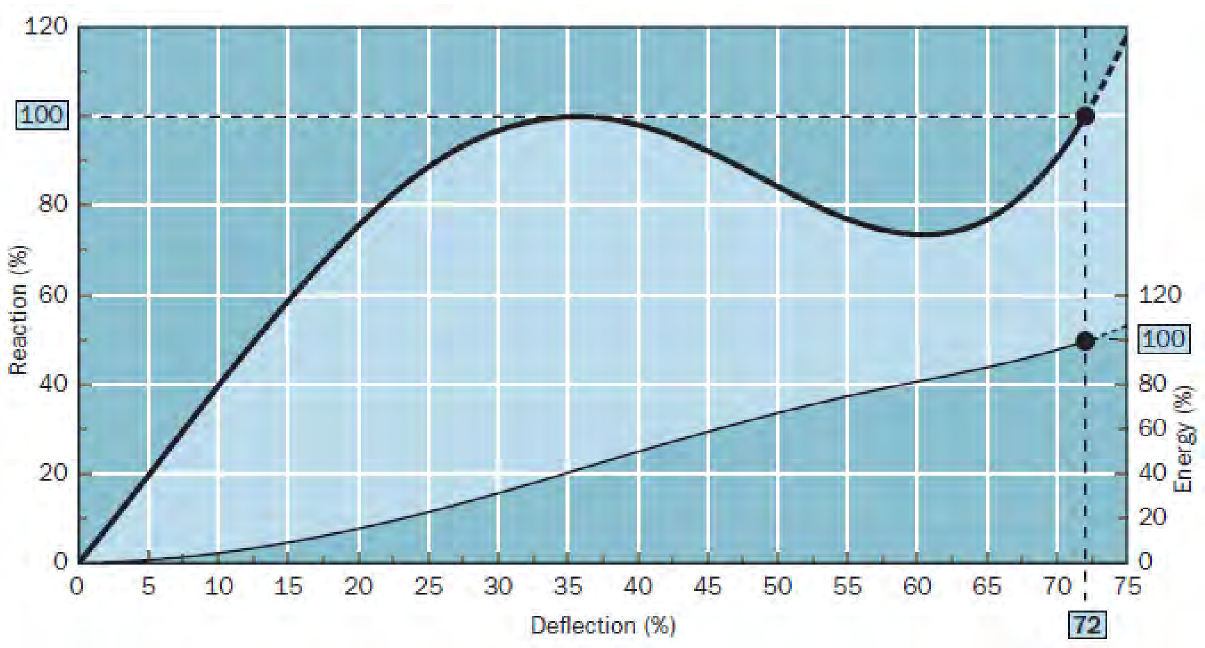
essel type	Size	F_S
Tanker, bulk, cargo	Largest Smallest	1.25 1.75
Container	Largest Smallest	1.5 2.0
General cargo		1.75
RoRo, ferries		≥2.0
Tugs, workboats, etc		2.0

Source: PIANC 2002; Table 4.2.5.

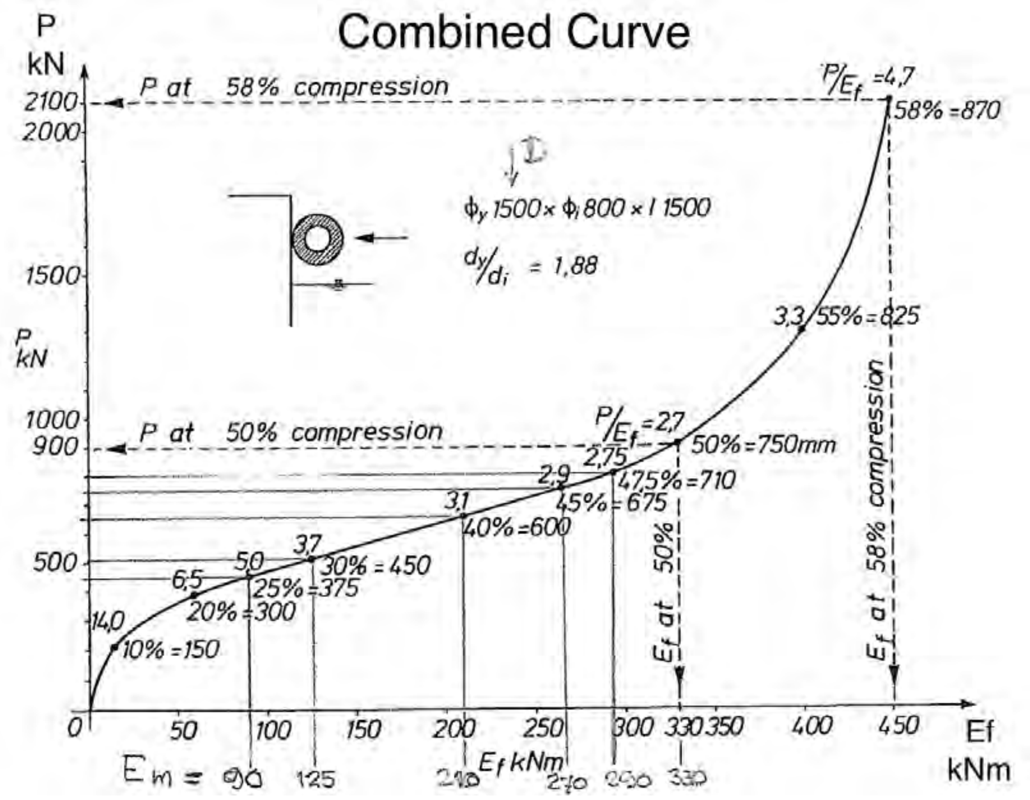
E_A= 2.0 x 472.7 = 9454 Kn.m

- **Selection of the fender type**

The fender of the oil terminal are elastomeric fenders with the following reaction to pressure.



As can be seen from the graph below, the optimum compression for the fender is around 50%, more than 50% would lead to a big increase of pressure on the quay, less than 50% would lead to a big decrease of fender absorbing energy Ef that can be solved only increasing the size of the fender itself.



So according to the impact energy Ef = 9454 [kN*m] the choice has been made taking into account of two different super cones in order to reduce the pressure on the caisson wall.

- **Super cones fenders:**

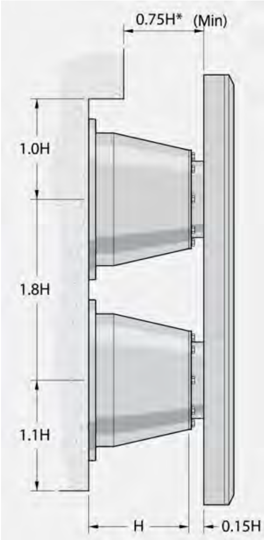
Then this data would be used to determine the geometry of the elastomeric fender panel as shown on the images below:

2250	CV	E	3844.0	4271.0	4385.0	4499.0	4613.0	4727.0	4841.0	4955.0	5069.0	5183.0	5297.0	5411.0
		R	3010.4	3191.0	3276.0	3361.0	3446.0	3531.0	3616.0	3738.0	3860.0	3982.0	4104.0	4226.0
	RPD	E_R	4036.2	4484.6	4604.3	4724.0	4843.7	4963.4	5083.1	5202.8	5322.5	5442.2	5561.9	5681.6
		R_R	3160.9	3350.6	3439.8	3529.1	3618.3	3707.6	3796.8	3924.9	4053.0	4181.1	4309.2	4437.3

Super Cone Fenders

DIMENSIONS

	H	ØW	ØU	C	D	ØB	ØS	F0.9- 1.8 ANCHORS / HEAD BOLTS ^	F1.9- 3.1 ANCHORS / HEAD BOLTS ^	Z_{min}	WEIGHT
SCN 300	300	500	295	27 – 37	20 – 25	440	255	M16	M16	77	40
SCN 350	350	570	330	27 – 37	20 – 25	510	275	M16	M16	77	50
SCN 400	400	650	390	30 – 40	20 – 28	585	340	M16	M20	82	76
SCN 500	500	800	490	32 – 42	30 – 38	730	425	M20	M24	95	160
SCN 550	550	880	540	32 – 42	30 – 38	790	470	M20	M24	95	210
SCN 600	600	960	590	40 – 52	35 – 42	875	515	M20	M30	115	270
SCN 700	700	1120	685	40 – 52	35 – 42	1020	600	M24	M30	120	411
SCN 800	800	1280	785	40 – 52	35 – 42	1165	685	M24	M30	120	606
SCN 860	860	1376	845	40 – 52	35 – 42	1250	735	M24	M30	130	750
SCN 900	900	1440	885	40 – 52	35 – 42	1313	770	M30	M30	135	841
SCN 950	950	1520	930	40 – 52	40 – 50	1390	815	M30	M30	142	980
SCN 1000	1000	1600	980	50 – 65	40 – 50	1460	855	M30	M36	150	1125
SCN 1050	1050	1680	1030	50 – 65	45 – 55	1530	900	M30	M36	157	1360
SCN 1100	1100	1760	1080	50 – 65	50 – 58	1605	940	M30	M36	165	1567
SCN 1200	1200	1920	1175	57 – 80	50 – 58	1750	1025	M30	M42	180	2028
SCN 1300	1300	2080	1275	65 – 90	50 – 58	1900	1100	M36	M42	195	2455
SCN 1400	1400	2240	1370	65 – 90	60 – 70	2040	1195	M36	M42	210	3105
SCN 1600	1600	2560	1570	65 – 90	70 – 80	2335	1365	M42	M48	240	4645
SCN 1800	1800	2880	1765	75 – 100	70 – 80	2625	1540	M42	M56	270	6618
SCN 2000	2000	3200	1955	80 – 105	90 – 105	2920	1710	M42	M56	300	9560
SCN 2250	2250	3600	2205	100 – 120	100 – 110	3285	1930	M48	M56	335	13,500
SCN 2500	2500	4000	2450	120 – 150	100 – 120	3650	2150	M48	M64	375	18,500



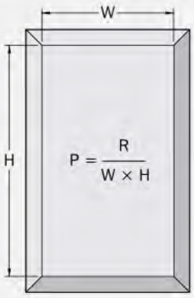
	CONE	PANEL
R (Kn)	3618,3	7236,6
H (m)	2,25	8,775
Ø , W (m)	3,285	4.5
P (Kn/m^2)		183.26

According to PIANC table below, the pressure value gotten from the assumption of 4.5 meters width is perfect.

Hull Pressures and Beltings

HULL PRESSURES

Allowable hull pressures depend on hull plate thickness and frame spacing. These vary according to the type of ship. Refer to the table on the right for PIANC's guidelines on hull pressures.



$$P = \frac{R}{W \times H}$$

P = average hull pressure (kN/m²)
R = total fender reaction (kN)
W = panel width, excluding bevels (m)
H = panel height, excluding bevels (m)

VESSEL TYPE	SIZE/CLASS	HULL PRESSURE (kN/m²)
Container ships	< 1,000 teu (1st/2nd generation)	< 400
	< 3,000 teu (3rd generation)	< 300
	< 8,000 teu (4th generation)	< 250
	> 8,000 teu (5th/6th generation)	< 200
General cargo	≤ 20,000 DWT	400–700
	> 20,000 DWT	< 400
Oil tankers	≤ 20,000 DWT	< 250
	≤ 60,000 DWT	< 300
	> 60,000 DWT	150–200
Gas carriers	LNG/LPG	< 200
Bulk carriers		< 200
RoRo		Usually fitted with beltings (strakes)
Passenger/cruise		
SWATH		

Source: PIANC 2002; Table 4.4.1

NADOR WEST MED NEW PORT

Design proposal

ESITC CAEN
Ecole Supérieure d'Ingénieurs des Travaux de la Construction

Contents

List of figures

Acknowledgements

We would like to thank Mr. Carpentier for organising the whole PEMW Workshop and allowing us to have a great experience of Caen and ESITC. We would also like to give our thanks to Mr. Silva for unknowingly creating a wonderful group of 10 people who during these 4 weeks turned into a great team. Your support during our project has been invaluable.

Finally, we would like to thank all the amazing lecturers who took time from their busy schedules to come and share some of their detailed knowledge with us. Everything we've learned has been appreciated, and we are sure that the knowledge we gathered throughout this workshop will be beneficial for us in the future.

EXECUTIVE SUMMARY

This report presents a design proposal for the construction of a new port at Baie de Betoya on Morocco's Mediterranean coast. The project belongs to a wide industrial development scheme that will serve all type of ships and will contain an 1500 ha industrial area providing economic benefits to the country.

For an economically successful and sustainable project, the port design needed to abide by a set of essential design criteria. The main client requirement was a port entrance from the South-West direction and minimising the depth of the breakwater. The limited options for project location and different requirements from the stakeholders were considered initially, together with a sustainability consideration. The design ship dimensions were determined as 370 m length, 60 m width, 22.2 m draught, which influenced the number of berths for each type of terminal. Consequently, the total quay length was found to be 2620 m and the total surface storage area required is 155 ha.

Several design conditions were also present at the site. Initially an analysis of the ground and bathymetry conditions was performed to determine possible locations for the port. Three different locations at the coast were considered. It was found that the middle section of Baie de Betoya contains the most favourable geotechnical conditions (little to no clay) and the water is shallow enough to allow construction of an economical breakwater. Therefore, this location was selected as the future location of the port.

A thorough analysis of the wind and wave data was performed. It was found that the wind direction is primarily from West and East. The waves with the highest frequency came from North-West and North-East directions. The extreme waves at the port were taken as the 100-year return period waves, while the operational ones were taken as the 1% exceedance probability waves. After comparison of the waves from both directions, it was observed that the waves from North-West are more critical, since they also are in the direction of the port entrance.

A sedimentation study revealed that there are 2 locations at the bay where sediment currents balance out, thus reducing the amount of sediment transport. A location was selected such that for a 25 year period, 90% of the time the sediment transport from the river Kert to the basin is negligible and easy to manage, and in rare cases it would require dredging of the fine sediment laying on top of the seabed.

In order to produce layout options, the dimensions of the navigation channel were found. The approach channel needs to be 2383 m long, 285 m wide and 24.1 m deep to allow the largest design ships to safely berth at the port. The turning circle has a diameter of twice the maximum design ship length, and the quay wall needs to be at 2.80 m above water level.

After determining these design dimensions, 2 initial layout proposals were conceptualised. They were compared using a multi-criteria analysis (MCA) with 9 criteria, including cost, ship manoeuvrability, environmental impact and future expansion. It was found that alternative 2 had the higher score, but was subjected to several improvements. Therefore, a modified version of layout 2 was created and scored on the MCA matrix, obtaining much higher score than the previous alternatives. This layout was consequently chosen as the final layout and was further detailed.

Two rubble mound breakwaters with accropodes were designed which limit the wave exposure of the port. To confirm that the wave height in the port is below the acceptable limits, a precise wave agitation model was created. The zone near the port location is highly seismic, therefore a horizontal acceleration was considered when designing the port structures. The breakwater sections were dimensioned in detail. An anchored sheet pile quay wall was chosen to sustain the vertical and horizontal loads at the quay. Finally, a breasting dolphin was designed for the oil terminal.

After determining the construction sequence, a thorough cost analysis of the construction phase was performed. The total cost of the project was calculated as 625,000,000 €. An environmental and social impact assessment was carried out to identify possible risks and mitigation measures. The rising sea water level (0.6 m prediction in 100 years) was accounted for in the project design, therefore making the port suitable for future climate changes. The selected layout can also be expanded further in land, allowing for future port expansion if required.

INTRODUCTION

Introduction

The aim of this report is to present a complete design proposal for constructing the new port of Nador West Med in Baie de Betoya. This chapter describes the project scope, as well as the leading team of project designers involved in the works. The following chapters contain a detailed description of the design criteria and conditions at the site, followed by three conceptual port layout designs. After performing a thorough multi-criteria analysis of the alternatives, the final layout has been described with an appropriate level of detail, supported by relevant calculations and drawings. Moreover, a cost-benefit analysis and an environmental assessment are presented to support the design proposal. Finally, recommendations for future possible expansion of the port are made.

Project description

The current project concerns the development of a major new port and industrial complex around 30 km from the town of Nador, at the Baie de Betoya on Morocco's Mediterranean coast (Figure 1). It is part of a wider development scheme for the Nador region and will act as a measure to reduce the regional disparities and support the economic and infrastructural development of the Oriental Region.

Figure 1 Large-scale view of the Nador region, Morocco

The development will be undertaken by a dedicated Company called 'Nador West Med' (NWM) and will include terminals for:

- transhipment hub and import/export of containers
- transhipment hub for crude oil and import/export of refined crude products from a Refinery to be built in the Industrial zone
- import of coal for an Electrical Plant to be built in the Industrial Zone
- import/export of general cargo (Ro-Ro mode)

The development will also include a service quay and an industrial investment zone comprising a total area of 4000 ha, with a potential to attract international investments. A more detailed assessment of the port design criteria, including design ship measurements and required number of berths is presented in Chapter I.

The port needs to have an entrance towards the South-West (SW) direction. An additional design criterion has been imposed by the funding body and demands that the port elements (breakwater, quays, dolphins) are constructed in as shallow waters as possible, thus avoiding the costs for construction in deep water. This implies that the costs of dredging will increase.

The design team

The following report has been produced by group 'Design Your Dream'. The team consists of specialists in different areas of port engineering and maritime construction. The team members have worked in smaller divisions to produce an economical, sustainable and feasible design proposal (See Appendix A for detailed planning). The next page lists the team members together with their origin university and specialism area.

Team members

Alexander van der Hoek

Project Leader
Delft University of Technology

Alejandro Ruiz Aguilera

Maritime Analyst
ETSICCP de Granada

Abdigani Hassan

Wave/wind Analyst
Aalborg University

Alan Kerveillant

Planning & Cost Manager
ESITC Caen

Arnaud Bégué

Model Specialist
ISA BTP Anglet

Marius Varnelis

Geotechnical Specialist
University of Edinburgh

Maya Lambert

Port Specialist
Universidad de la República

Milena Velikova

Communications Manager
University of Edinburgh

Raúl Maldonado

Maritime Analyst
ETSICCP de Granada/ ESITC Caen

Stefania Viaggio

Designer
Universita degli studi di Genova

CHAPTER 1

DESIGN CRITERIA

1. Design Criteria

For an economically successful and sustainable project, the port design needs to abide by a set of essential design criteria, presented in the forthcoming chapter. This section includes a description of the project location, a stakeholder analysis, determination of the design ship dimensions, as well as a calculation of the number of berths, quay length and the required surface area. Finally, a summary of environmental and safety concerns is presented, together with considerations for dredging and future port expansion.

1.1 Location

The project is to be constructed at Baie de Betoya (coordinates: N35° 14’ 42’’, W 3 °10’ 12’’), 35 km north from the town of Nador (**Error! Reference source not found.**). The port location is limited to the 7.1 km coastline of the bay between the Kert river (Oaed Kert) and Garet Cape. The south west coast presents poor geotechnical conditions and increased sedimentation due to the presence of the Kert river, while the north east coast is dominated by deep water, which could cause high structural expenses. Therefore, the location of the port needs to be selected carefully.

Figure 2 Baie de Betoya location

1.2 Stakeholder analysis

Before any design efforts are made, a stakeholder analysis must be performed to determine all interested parties and the level of their significance. The degrees of interest and influence are divided into 'low to none' and 'medium to high'. Figure 3 below shows the stakeholder analysis as determined by Group Design Your Dream.

As it can be seen from the figure, the most important stakeholder is the Moroccan

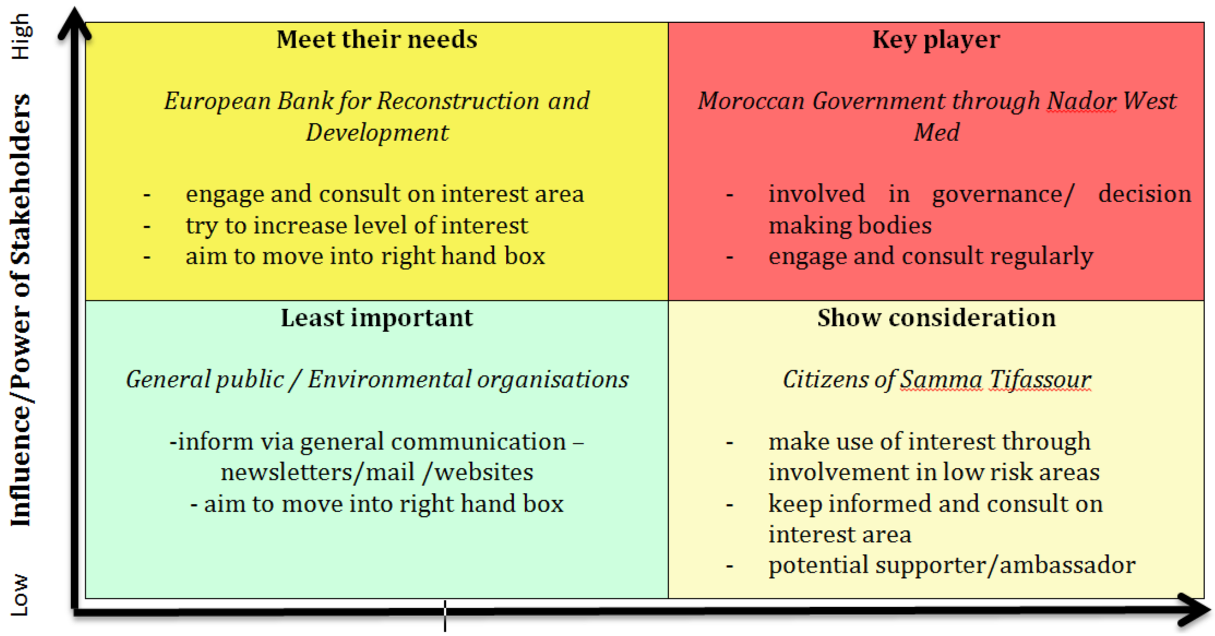


Figure 3 Stakeholder analysis for the new port at Nador West Med

government, who has both large interest and influence over the outcome of the project. Therefore, its needs have to be considered first when making design decisions regarding the port.

However, there are two other stakeholders who need to be thoroughly considered. The citizens of the nearby town of Samma Tifassour would be directly affected by everything happening at the port locations. Thus, they should be informed of any decisions in a timely manner and their concerns and needs have to be understood. The European Bank for Reconstruction and Development has a major power over the project

1.3 Design ship analysis

The port infrastructure must be dimensioned in accordance with the size of the ships that will berth at the port. Therefore, an analysis of all the expected incoming vessels was conducted to determine the required design length, draught and breadth. These parameters are required for the calculation of number of berths, quay length, size of navigation channel and the depth of the port basin.

The analysis consisted of determining the design ship for each type of category (container, coal, general cargo, ro-ro and tankers). Afterwards, the design characteristic, e.g. length, was plotted against the number of twenty-equivalent units (TEU) or deadweight tonnes (DWT). The following section represents the main conclusions of the analysis. A detailed determination of all design ship dimensions is presented in Traffic requirements

The traffic requirements at the new port are presented in Figure 4.

Figure 4 Traffic requirements at the new port

Traffic typology	Unit	Volume (Unit/year)	Notes
Containers	TEU	3 million	
Oil	Tons	25 million	
Dry bulk (coal)	Tons	7 million	
General cargo & ro-ro	Tons	3 million	1 million of which transported in ro-ro mode

For each traffic category, the design ships are the following:

- Container: mother ships up to 18,000 TEU and Feeders up to 5,300 TEU
- Coal carriers: capesize of 170,000 to 180,000 DWT
- General cargo ships: up to 40,000 DWT
- Ro-Ro ships: up to 15,000 DWT
- Tankers: Panamax tankers up to 65,000 DWT for refined products

1.3.1 Design ship length

The container mother ships are the largest vessels expected in the port. Therefore, their length was taken as a representative for the maximum design ship length. The design value of the length was determined by plotting all the available lengths for container mother ships against their TEU capacity, as shown in Figure 5.

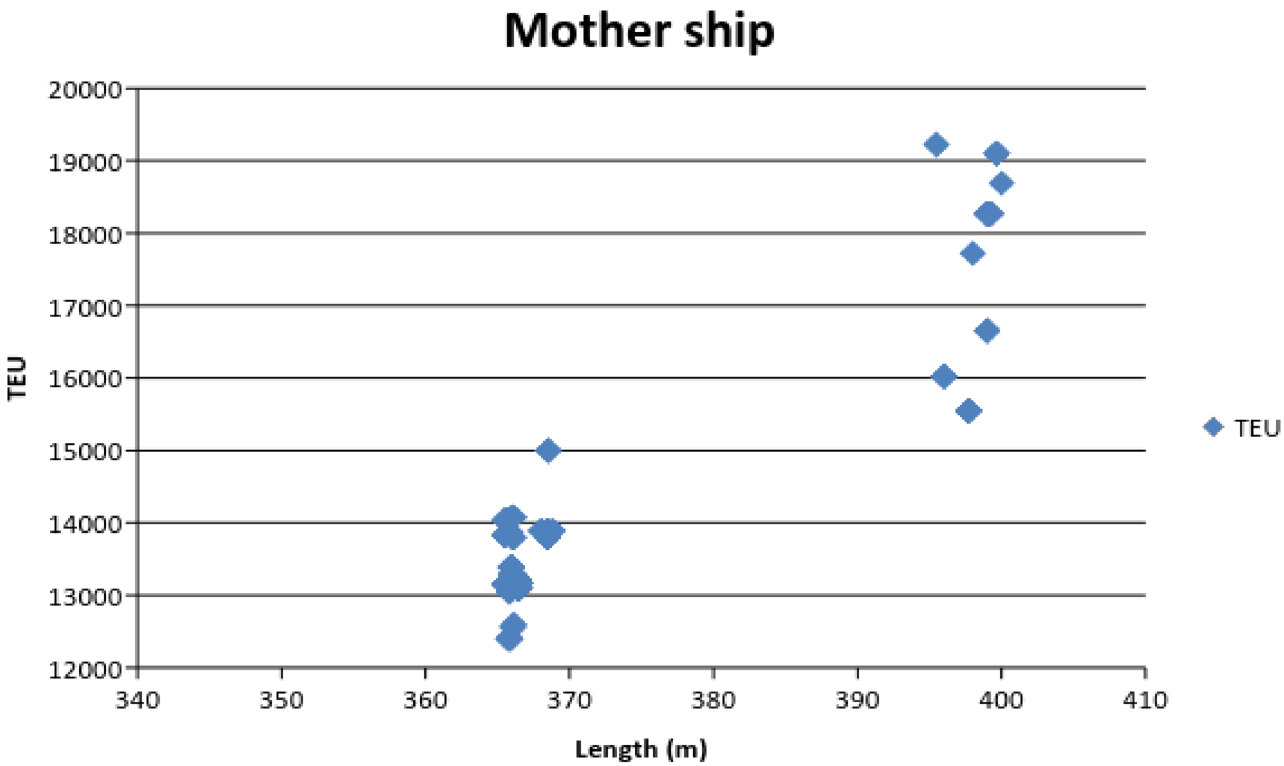


Figure 5 Container mother ship lengths

It can be seen from the graph that most of the ships have a length overall (LOA) of less than 370m. Their container capacity is up to 15 000 TEU. Only 20% of the ships have an overall length nearing 400 m, and higher TEU of up to 19 500 TEU.

Since only 9 ships are longer than 370 m, this value was used for dimensioning of the terminal, as it was assumed the other 80% of ships represent a good sample of the expected vessels in the new port.

Any larger container ships will be redirected to another transportation hub which has the required capacity to fit their larger dimensions.

1.3.3. Design ship draught

The crude oil tankers are the vessels with the largest draught expected in the port. Therefore, their draught was taken as a representative for the maximum design ship draught. The design value of the draught was determined by plotting all the available draughts for crude oil tankers against their DWT capacity, as shown in 6.

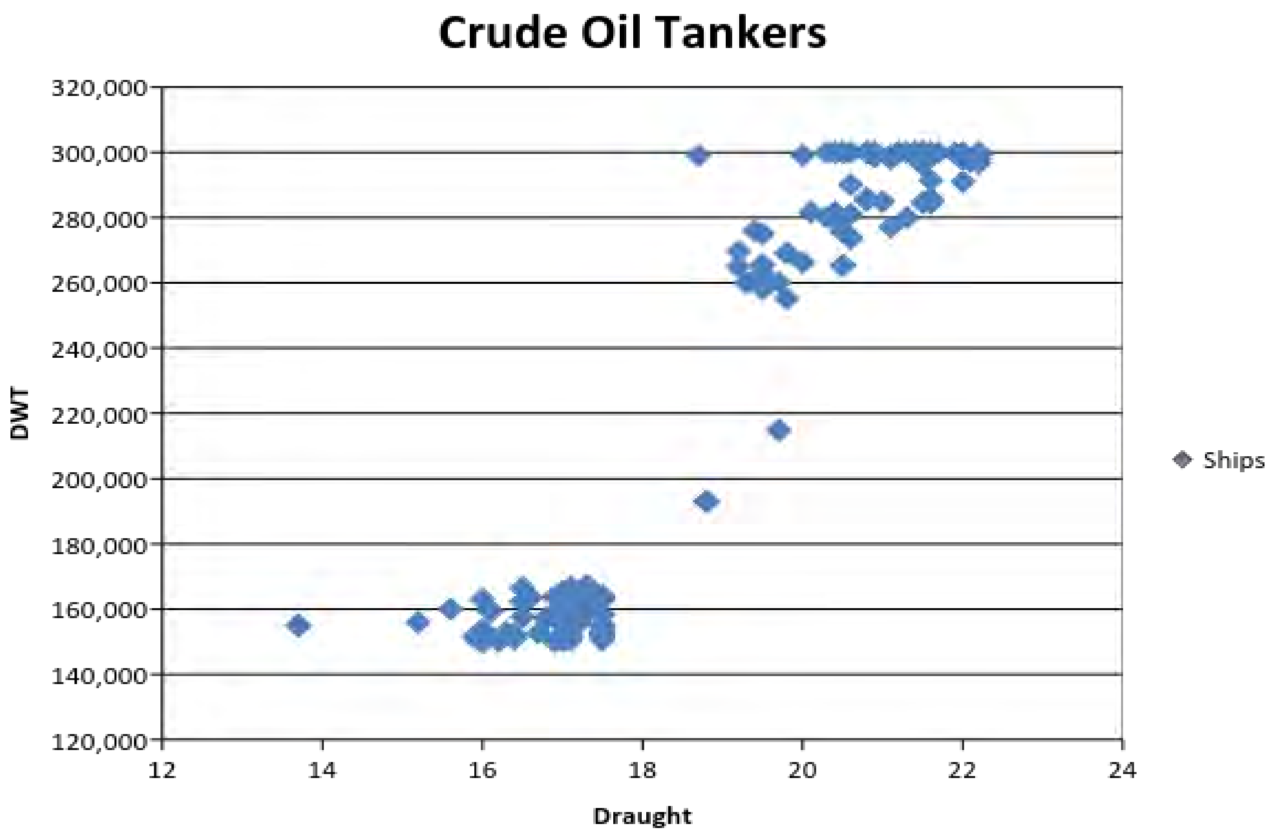


Figure 6 Crude oil tankers draught

It can be seen from the graph that there are two major groups of oil tankers: ones with DWT of around 160,000 with draught of 16-18 m, and larger vessels with DWT of up to 300,000 with draughts in the range of 19-22 m.

Since the channel needs to allow all ships to enter the port, the maximum ship draught of 22.2 m was taken as a design value. Thus, it will ensure that all available oil tankers can safely enter the port. Any larger container ships will be redirected to another transportational hub which has the required capacity to fit their larger dimensions.

1.3.4 Design ship breadth

The crude oil tankers are the vessels with the largest breadth expected in the port, together with the largest mother ships. However, the mother ships with TEU capacity of over 15,000 were not considered for the analysis since their length is larger than the maximum design one. Therefore, the breadth of crude oil tankers was taken as a representative for the maximum design ship breadth. The design value of the breadth was determined by plotting all the available breadths for crude oil tankers against their DWT capacity, as shown in Figure 7

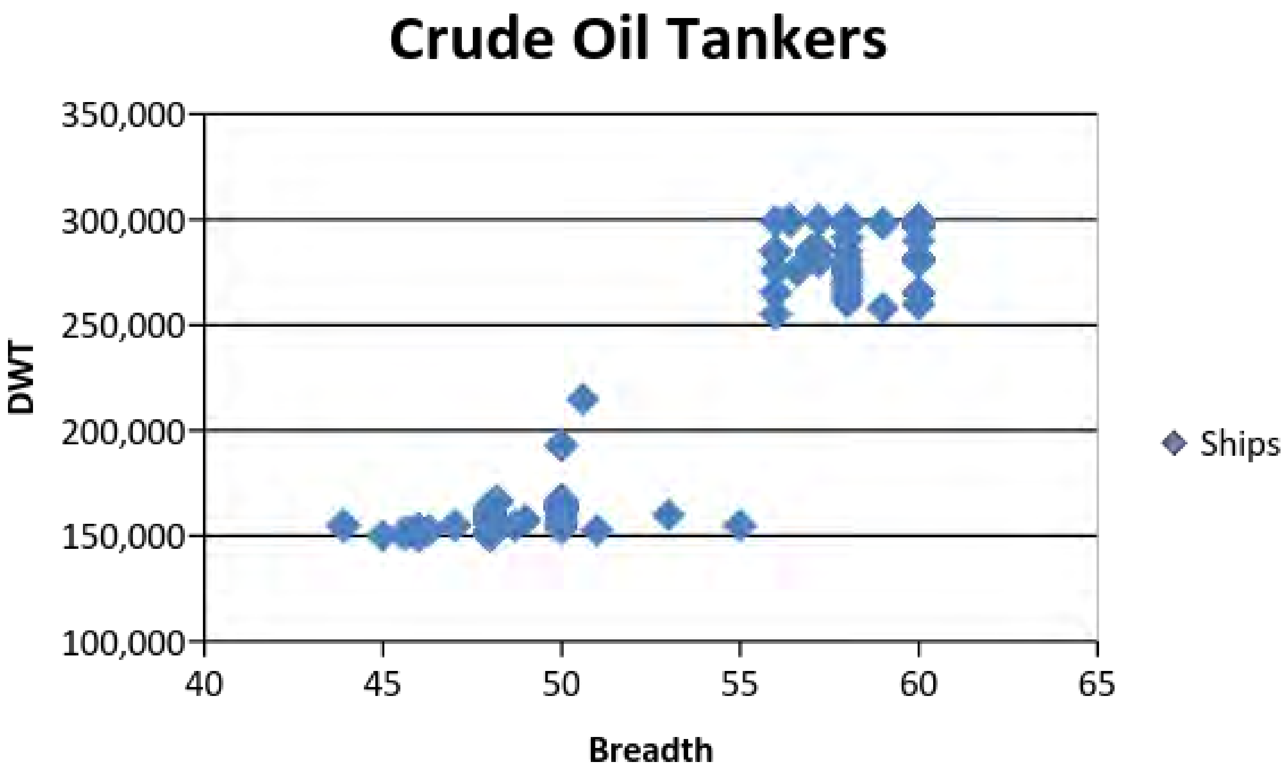


Figure 7 Crude oil tankers breadth

It can be seen from the graph that there are two major groups of oil tankers: ones with DWT of around 160,000 with breadth of 45-52 m, and larger vessels with DWT of up to 300,000 with breadths in the range of 55-60 m.

Since the channel needs to allow all ships to enter the port, the maximum ship breadth of 60m was taken as a design value. Thus, it will ensure that all available oil tankers can safely enter the port. Any larger container ships will be redirected to another transportation hub which has the required capacity to fit their larger dimensions.

1.3.5 Summary of design ship dimensions

Figure 8 showcases the maximum length, breadth and draught for each type of vessel entering the port. A designated graph for the dimensions of each type of ship is presented in Appendix A.

	Length (m)	Draught (m)	Breadth (m)
Container Mother ship	400 / 370	16	60/52
Container Feeder	290	13.6	32.3
Coal Carrier	293	18.6	45
General Cargo	200	12.0	30
Ro-ro ship	200	7.5	27
Crude oil tanker	340	22.2	60
Product tanker	195	13.6	33
Design Value	**370**	**22.2**	**60**

Figure 8 Summary of design ship dimensions

It can be seen that the design ship length was selected to be 370 m as 80% of the ships entering the port have lengths less than this value. The design ship draught was chosen to be 22.2 m, and the design ship breadth for dimensioning the navigation channel was selected to be 60 m due to the large size of the crude oil tankers.

1.4 Number of berths and quay length

The number of berths was calculated basing on two different approaches: the queueing theory and a general formula for berth productivity. The queueing theory was used as a primary source for determining the number of required berths, while the general formula was used for verification of the obtained values. Consequently, the quay length was calculated using the number of berths and design ship dimensions obtained from section 1.3.5.

The following assumptions were made:

- the port operates 24 hours a day
- the operational hours are 8400/year, allowing for 15 days of not working due to holidays
- the efficiency factor with respect to the nominal capacity of each crane was taken as 80%
- the occupational factors for each berth depend on the number of berths
- the mooring time per vessel is taken as 2 hours

1.4.1 Queuing theory

First the queuing theory was applied per terminal. Appendix B contains an explanation of the formulas used in the calculations.

When the number of berths is known, the quay length along a straight continuous quay front can be calculated (formula 7.5, Ligteringen & Velsink, 2014). In this formula the L_s = LOA and the average Length overall is 80% of the maximum LOA.

$$L_q = \begin{cases} L_{s,max} + 2 * 15, & n = 1 \\ 1.1 * n * (\bar{L}_s + 15) + 15, & n \geq 1 \end{cases}$$

1.4.1.1. Container terminal

Important to note is that the ratio 20ft. – 40ft. of the containers to be handled (TEU – FEU) is 50% so in total 1.000.000 TEU containers and one time 1.000.000 FEU makes in total 3.000.000 TEU but 2.000.000 containers to be handled.

The calls per year are calculated by assuming that 30 % of the ships account for the motherships (14000 TEU) and 70 % of the ships are feeders (3600 TEU) resulting in an average of 6720 TEU per ship. The total amount of throughput divided by this average gives: 446 calls/year.

Figure 9 presents the results of the calculation for the container terminal.

Transhipment	Throughput	Calls/year	(Un)loading/call	(Un)loading-rate
Container	3.000.000 TEU	446	6726 TEU	100 moves/hour

Figure 9 Container terminal berths calculation

In figure 10 the arrival rate (**λ**) , (un)loading time, service rate (**μ**) and berth occupancy (**ρ**) are respectively calculated. Because the berth occupancy is about 3,7, the first estimate of the number of berths is n = 4. This gives an utilisation ratio of u = 0.92. The queuing system is E2/E2/n so from table V (Groenveld, 2001) it can be found that the average waiting time of ships in the queue (in units of average service time) is 92%. This is too high to meet the realistic waiting time requirements.

A second estimate is made with n = 5. This gives a value u = 0.73 which gives an average waiting time of ≈ 0,10. A 10% waiting time, expressed in total time of the ship in the port, is sufficient.

λ	0,053 calls/hour
(un)loading time	67 hours
μ	0,01444 service/hour
ρ	3,677

Figure 10 using formulas of appendix B

<u>Therefore, the number of container ship berths is 5</u>. Since n > 1, the following formula is used to calculate the quay length. The average length of a feeder container ship is taken as

80% of 290 m is 232 m. Rarely a mothership will enter the port so for financial reasons it is not useful to design the port with 5 mothership berths. For this reason, when a mothership will enter the port it can use 2 berths and the cranes can move to the ship.

The required quay length of the container terminal is 1374 m, which was rounded to 1400 m.

1.4.1.2 Oil terminal

Transhipment	Throughput	Calls/year	(Un)loading/call	(Un)loading-rate
LNG	25.000.000 t	121	206.777,4 t	15.000 t/hour

Figure 11 Oil terminal berths calculation

The average tonnage per oil ship is 206.777.4 t (see figure 6 in previous section) so the calls per year become 25.000.000/206.777,4 = 121 calls/year. There is import for crude oil and after refining in a refinery, it will be exported by smaller carriers and distributed to other smaller ports.

Using the data above

Λ	0,014 calls/hour
(un)loading time	14 hours
M	0,063 service/hour
ρ	0,227

Figure 12 using formulas of appendix B

The first estimate for the number of berths (occupancy is 0,227) is n = 1, which gives a value of u = 0.227 for the utilisation. The queuing system is M/D/n and the maximum acceptable waiting time is 0,05 -0,10. The value u = 0,227 gives a value of about 0,12 and therefore 1 berth is sufficient.

According to the theory just 1 berth is sufficient so the quay length can be calculated with the following formula. From section 1.3.5 we can see that almost all ships are smaller than 340 m (except from two ships) so with this length we can calculate the quay length.

$$L_q = L_{s,max} + 2 * 15$$

The required quay length of the oil terminal is 370 m. Alternatively, a jetty of comparable length can be used. Because the crude oil will be refined and exported to smaller carriers, there is a need for 2 berths. The quay length of the second berth will be calculated for smaller carriers with a design length of 290 m (see section 1.4.1.1). Using the same formula as above, the smaller quay length was calculated as 320 m.

1.4.1.3 Dry bulk (coal) terminal

Transhipment	Throughput	Calls/year	(Un)loading/call	(Un)loading-rate
Dry bulk	7.000.000 t	41	170.000 t	4.000 t/hour

Figure 13 Dry bulk terminal berths calculation

Loading coal goes faster than unloading but the overall (un)loading-rate is assumed to be 4.000 t/hour. Using the data above and the formulas described in Appendix C:

λ	0,004881 calls/hour
(un)loading time	42,5 hours
μ	0,022 service/hour
ρ	0,22

Figure 14 using formulas of appendix C

A first estimate of n = 1 gives an utilisation of 0,22. Knowing the queuing system $M/E_2/n$ table IV shows an average waiting time of ≈ 0,19 so it is too large. The maximum acceptable waiting time for bulk is 0,10.

The second estimate of n = 2 gives an utilisation of 0,11. Table IV shows an average waiting time of ≈ 0,01. So when two (n=2) berths are applied the ships almost have no waiting time: just 1 percent of the total time the ship is served in the port.

The number of berths is 2 so the following formula is used to determine the quay length. The average length of the ship will be 80% of 292 m (table 3) is 234 m.

$$L_q = 1.1 * n * (\bar{L}_s + 15) + 15$$

The required quay length of the dry bulk terminal is 563 m ≈ 560 m.

1.4.1.4 General cargo & ro-ro terminal

Transhipment	Throughput	Calls/year	(Un)loading/call	(Un)loading-rate
Dry bulk	3.000.000 t	200	15.000 t	1.000 t/hour

Figure 15 General cargo and ro-ro terminal berths calculation

From the total throughput 1/3 is transported by trucks in a ro-ro mode (about 800 vehicles per hour) 800 t/hour but 2/3 is transported by a crane resulting in a (un)loading-rate of about 1000 t/hour. Using the data above and the formulas described in Appendix B:

λ	0,024 calls/hour
(un)loading time	15 hours
μ	0,059 service/hour
ρ	0,40

Figure 16 using formulas of appendix C

A first estimate of n = 1 gives an utilisation of 0,40. Knowing the queuing system $M/E_2/n$ table IV shows an average waiting time of ≈ 0,50 so it is too large. The maximum acceptable waiting time for cargo is 0,10.

The second estimate of n = 2 gives an utilisation of 0,20. Table IV shows an average waiting time of ≈ 0,03. So when two (n=2) berths are applied the ships only have to wait 3 percent of the total time in the port.

The number of berths is 2 so the following formula is used to determine the quay length. The average length of the ship will be 80% of 200 m (table 3) is 160 m.

$$L_q = 1.1 * n * (\bar{L}_s + 15) + 15$$

The required quay length of the general cargo terminal is 400 m.

1.4.2 Verification by berth productivity

The first approximation of the number can be verified on the basis of estimated berth productivity. As showed in Appendix C these values correspond to the values calculated in the previous section.

1.4.3 Summary number of berths and quay length per terminal

Transhipment	Number of berths	Quay length [m]
Container terminal	5	1400
Oil terminal	2	370/320
Dry bulk (coal) terminal	2	560
General cargo & ro-ro terminal	2	400

Figure 17 Summary number of berths and quay length

1.5 Surface storage area

In this section the needed land area for the facilities of the port is calculated. This is done for the container area, the RO-RO & General cargo area, solid bulk and the area for the oil terminal.

1.5.1. Container Area

Firstly, the container surface area was calculated using a formula giving the total storage area. It is the sum of the storage area, surface for technical buildings, administrative buildings and for exchange – tracks and train terminals.

The assumption that the average berthing time of one container is 8 days was made. Another assumption is that the operational system is RMG (Rail Mounted Gantry) with 5 levels of containers and twenty-equivalent units (TEU) area of 6 m^2/TEU .

Figure 18 is showing the distribution, as percentage, of the different areas. The total needed area for containers terminal is 90 has.

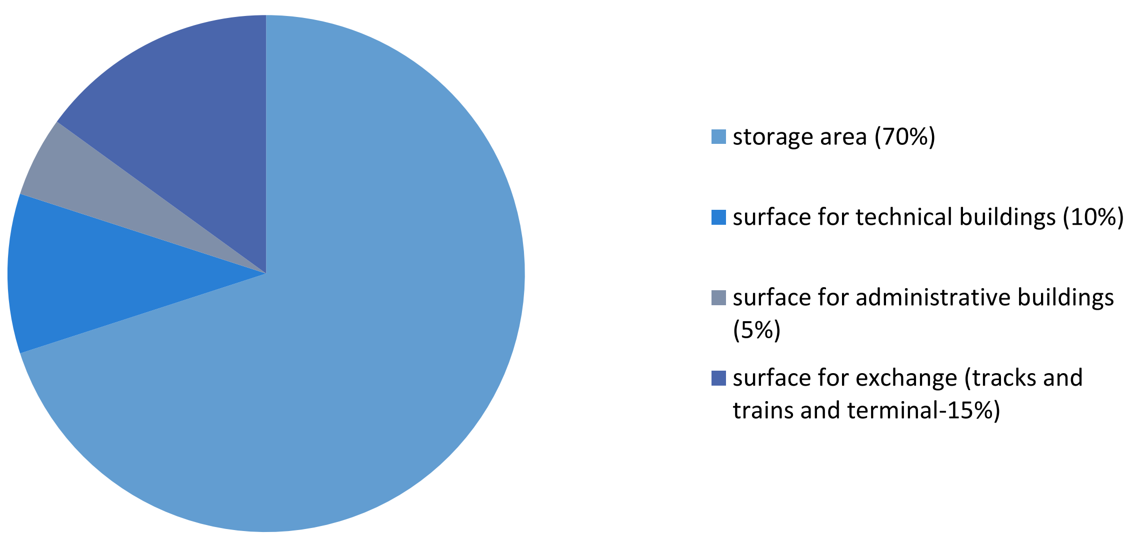


Figure 18 Distribution of container areas

1.5.2 RO-RO areas & General cargo

For the ro-ro terminal, we have made the assumption that the needed storage space is 10 ha per berth. The total area is calculated as the number of berth multiplied by the needed area. Therefore, 20 ha for the two ro-ro berths are required.

For general cargo, the width of the berths was maximised. Instructions from the Terminal Typology databook were taken, therefore and the larger values from the recommendations were selected. The total width of the general cargo terminal is 150m. This value is multiplied by the length of the berth, thus yielding that the general cargo terminal requires 6 ha area. Since Ro-Ro and General cargo will be berthing at the same terminal, only the larger terminal area of 20 ha was considered.

1.5.3 Solid bulk

To define the needed area, the volume of two biggest ships, which is 360.000 tons was taken. A storage coefficient of 1,08 (recommended for coal) and an average height of two meters were considered; that means mound about 4 m height with an angle of 45 °. The needed area is 20 has.

1.5.4 Liquid bulk

First, the liquid bulk was split into crude oil and refined products. For the crude oil terminal, 8 tanks of 100 000 m^3 capacity are required. For the refined products, 4 tanks of the 30 000 m^3 capacity are needed. In total, we need 25 ha for storing liquid bulk.

1.5.5. Summary

The needed area in land for all the terminals is 155 ha. Figures 19 and 20 summarise the results.

Tipe of terminal	Area (has)
Containers	90
General cargo & RO-RO	20
solid bulk	20
liquid bulk	25
TOTAL	155

Figure 20 Needed terminal area

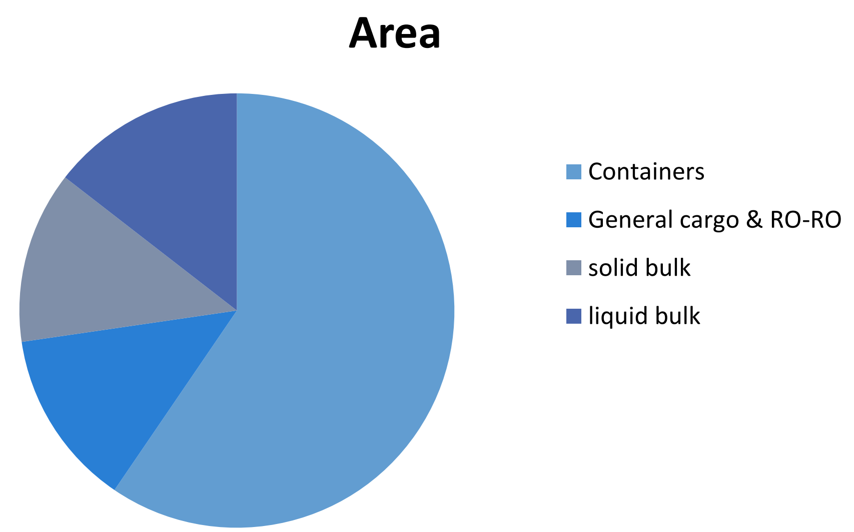


Figure 19 Required terminal area

1.6 Sustainability and safety

The new port at Nador West will significantly impact the surrounding environment both during the construction and operation phases. Moroccan law *[Law No. 12-03 on EIA enacted by Dahir No. 1-03-60 10 rabii I 1424 (12 May 2013)]* requires the commissioning of an environmental impact assessment (EIA) which identifies potential hazards and devises a plan to mitigate risks. Besides environmental issues, Design your Dream is also concerned with the social implications of the project. Consequently, ten areas of social and environmental hazards were identified:

- loss of marine/terrestrial habitat during construction
- management of dredged materials
- water and waste management, including the possibility of spills
- marine and road traffic management
- contractor management
- occupational health and safety (during construction and operation)
- air, noise and dust management
- communal health and safety
- economic displacement resulting from the loss of agricultural lands and decreased fishing along the beach of Betoya Bay due to turbidity
- fire safety hazard due to the presence of easily ignitable/ explosive substances such as crude oil

The identification of specific risks in each of these categories will be explored after the completion of the layout proposal. A detailed environmental and social impact assessment (ESIA) will be conducted and possible mitigation measures will be proposed to ensure safety during construction and operation of the port.

1.7 Dredging

The main design requirements for the project are dredging the largest amount towards land and creating a breakwater in as shallow water as possible. Dredging is also used for establishing the breakwater foundations. However, disposing of the dredged material at sea is a sensitive issue that needs to be handled with care in order to prevent environmental catastrophes. Identifying a suitable disposal site is based on bathymetry, currents and ecological sensitivity. Sediment quality has been analysed for 8 heavy metals and it was concluded that the sediment is safe to deposit at sea. An area just off the edge of the Betoya Bay was identified to have suitable physical characteristics that will limit the impacts of spoiling on marine wildlife. However, an appropriate method for dredging the site needs to be selected.

CHAPTER 2

DESIGN CONDITIONS

2. Design Conditions

2.1 Bathymetric conditions

The coastline of approximately 8 km starting from River Kert and finishing at Garet Cape is considered for development of new port.

At the south of the coastline, near the River Kert, the bathymetric lines are more apart of each other than in the north coastline. This indicates that south bed slope is lower and is varying more gradually. While the bed slope at the north of the coastline is varying more steeply.

In the south, the 10m bathymetric line (BL) is roughly 950m perpendicularly away from the coastline. At the middle of the coastline (~4km from the River Kert), 10m bathymetric line is roughly 690m away. At the north coastline, 10m bathymetric line is only 230m away from the coastline. In overall, south coastline is much more shallow than north coastline. Consequently, south coast is more suitable for port designs with small/short breakwater. Shorter breakwaters will result to minimization of structural expenses. However, since the approximate maximum design ship draught is 24m, more dredging will be required in the south. Similarly, less amount of near-shore dredging will be required at the north coast due to deep water conditions.

Bathymetric Line	South	Middle	North
10	1.0	0.7	0.2
20	1.8	1.0	0.6
30	2.5	1.5	0.8
40	3.2	2.0	1.1
50	4.1	2.5	1.5

Figure 21 Approximate perpendicular distances of bathymetric lines from the coastline (in kilometres)

2.2 Geotechnical conditions

Most of boreholes show that the sea bed is comprised of silt, sand, mudstone and volcanic tuff, where mudstone and volcanic tuff are settled at the bottom layers of the sea bed.

In general, silty or clayish soil, which is at the top layers, is considered to be a poor soil for supporting foundations of structures (for breakwater). Because of the small particles it contains internal moisture very well. This property might cause a severe soil expansion leading to uplifting and potential damage of foundations. Moreover, silty and clayish wet soil is usually very flexible, which is also not a suitable property for foundations.

Sand and gravel are considered to be a good soil for foundations because of its relatively large particles. Soil with large particles does not retain water content very well and is consolidated faster. When sand/gravel is wet, the particles hold together quite well, however it might lose interlocking friction which might cause a wash-away of particles leading to gaps beneath the foundation. Due to this precaution, soil reinforcement might be required. Rock, mudstone or volcanic tuff can be considered as a great soil for foundations because they have a high bearing capacity, and no consolidation needs to be considered.

Following the existing soil conditions, all the silt or silty sand layers need to be dredged. Sand, gravel, mudstone and volcanic tuff layers will be kept, except where stated otherwise.

2.3 Seismic conditions

The port is located in the Alboran Sea, which is a well-known seismic zone of the Mediterranean region. The delimit tectonic plate between Europe and Africa is pushed northwest by the African plate, therefore in the region there are a lot of geological faults. Due to the presence of this plate, the United States Geological Survey (USGS) estimated that this movement between plates is around 4-10 mm/year.

The project location is in a zone of higher seismic hazard, with 10 notable earthquakes having stricken in the last 12 months (Appendix E). In conclusion, the project is in a seismic zone so a horizontal acceleration (=0.18g) needs to be taken into account when calculating geotechnical stability verifications of the breakwater and of the quay designs.

2.4 Wind analysis

An analysis of the given wind measurements is illustrated in a wind rose in Figure 22. It can be seen from the wind rose that wind is most frequent from west and east directions. For the layout, it is important to consider the wind direction in order to determine the most critical waves.

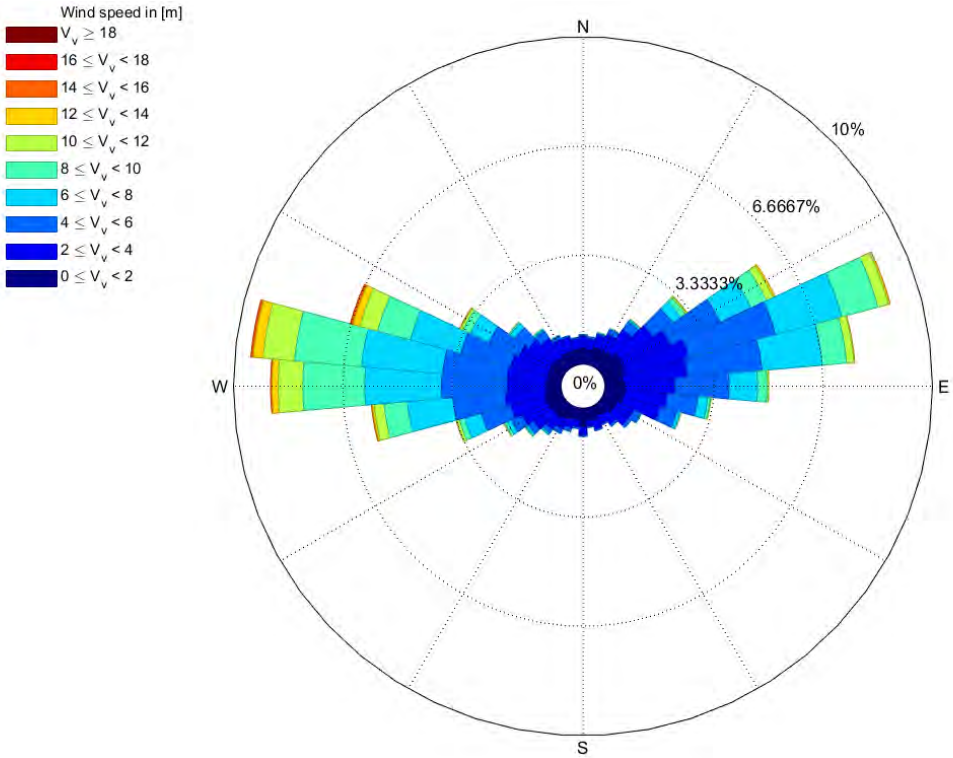


Figure 22 Wind rose indicating the main wind directions

2.5 Design waves

The provided time series of wave data and the considered directions are shown in the wave rose in Figure 23. Data for both the mean direction and the peak direction was provided, but values for the peak direction are used in further considerations.

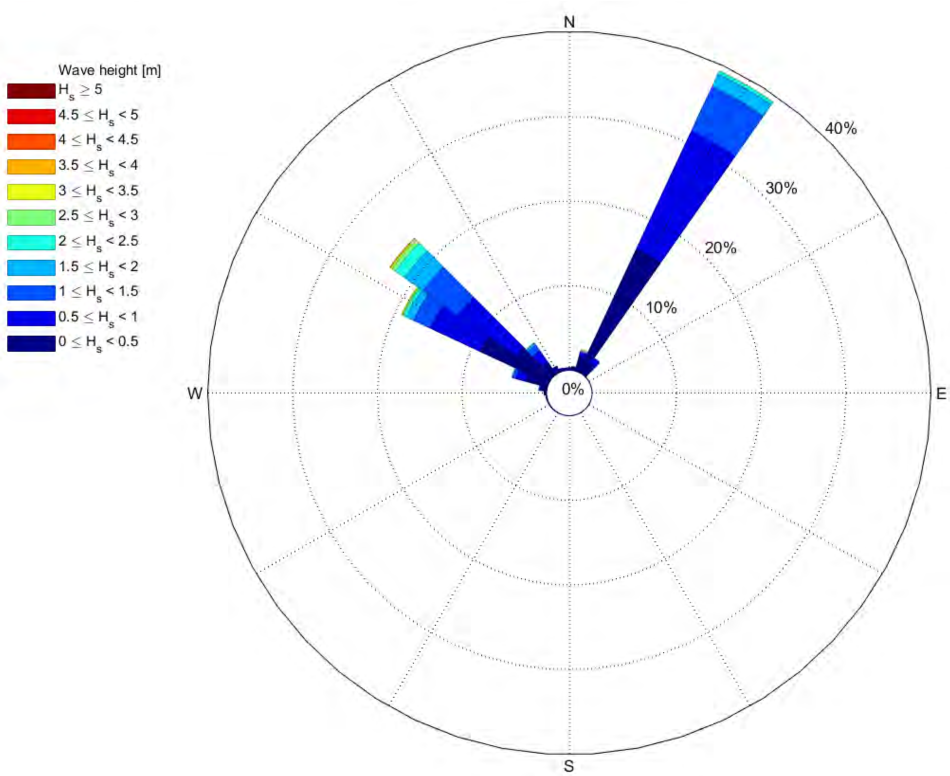


Figure 23 Design wave rose

The wave heights were analysed by firstly separating the data into the two main directions Northwest (NW) and Northeast (NE). In order to determine the design value of the significant wave height, H_s an extreme- and a statistical analysis was applied. For the selection of extreme data, the POT-method (peak over threshold method) was used. In this context, a threshold was chosen which is so high that identified peaks are extreme events and so low that sufficient number of storms are identified. Choosing a threshold which for both directions leads to 25-30 peaks is often considered sufficient. The extreme data should fulfil the independence criteria meaning that 24 hours distance between the peaks (storms) should be chosen.

The extreme data for the considered directions are fitted to a Weibull distribution, which often is used for significant wave heights in design. The least error occurs by fitting the extreme data through maximum likelihood method and so this method is used to determine the wave height with a return period of 100 years.

2.5.1 Extreme waves

The design wave heights for the main directions are given in Figure 24 and determined by the formulae introduced in Appendix F.

Direction	H_s	T_p
NE (22º-67º)	4.95	9.99
NW (292º-337º)	6.15	11.13

Figure 24 Extreme wave parameters

2.5.2 Yearly waves

Design wave characteristics with 1 year return period are determined and the obtained results are given Figure 25.

Direction	H_s	T_p
NE (22º-67º)	2.55	7.17
NW (292º-337º)	3.56	8.47

Figure 25 Yearly wave parameters

2.5.3 Operational waves

The operational waves are defined as the ones exceeded 1% of the time. Thus by taking the given wave heights for 18 years and determining the 99% quantile, the operational waves are found. In this context, it is assumed that the operational waves propagate in a direction of 10°. Figure 26 shows the operation wave parameters.

$H_{1\%}$	$T_{1\%}$
2.64	7.29

Figure 26 Operational wave parameters

2.6 Wave propagation

Wave propagation analysis was used to determine the wave height, peak period and direction in the area of interest where the possible breakwater locations are. The varying bathymetry contributes to the increased complexity of analysis. Therefore, a 2D modelling software (TOMAWAC) was used to produce a realistic representation of ocean waves from depths up to 90 m (around 10 km from the coastline). The changes both in time and spatial domain were calculated using the wave energy direction spectrum and the bathymetry data.

For the modelling purposes, the bathymetry was taken as the same described in section 2.1. The limits of the area of interest were set far enough from the potential port location to prevent the results from being affected by boundary errors.

Three types of boundary lines were considered:

- Imposed value – the boundary which meets the coming wave (approximately 45° each side from the mean wave direction)
- Free – all others boundaries in the water
- Solid – earth boundary

A *JONSWAP* spectrum was used for the propagation analysis. For a realistic representation, it also considered bottom friction.

The two main wave directions are NE and NW. Three types of waves for each of these directions were analysed:

- 100 year return period
- 1 year return period
- 1% exceeded wave height.

The 100 year return period wave is used in further calculations for the design of the breakwaters. The 1 % exceeded wave height is used to verify agitation inside the port, ensuring port operability 99% of the time. The 1-year waves were analysed but not taken into consideration when determining operability, since the 1% exceedance waves were seen as a more useful representation ofwaves needed for port design. .

As it can be seen from section 2.5, the most critical 100 year wave direction is NW. Therefore, only waves from this direction were considered for the breakwater design when determining the characteristics of the 100 year return period waves. The operational wave height was determined by the 1% exceedance wave propagation graph. Its direction was taken as 10° NNE as a recommendation based on the analysis of wave frequency. For a complete set of results, refer to Appendix G.

2.6.1 Extreme waves

For a 100-year return period wave, it can be seen that at approximately -40 m depth, the wave height starts to decrease. The wave direction also changes, becoming more perpendicular to the shore as waves approach the coast.

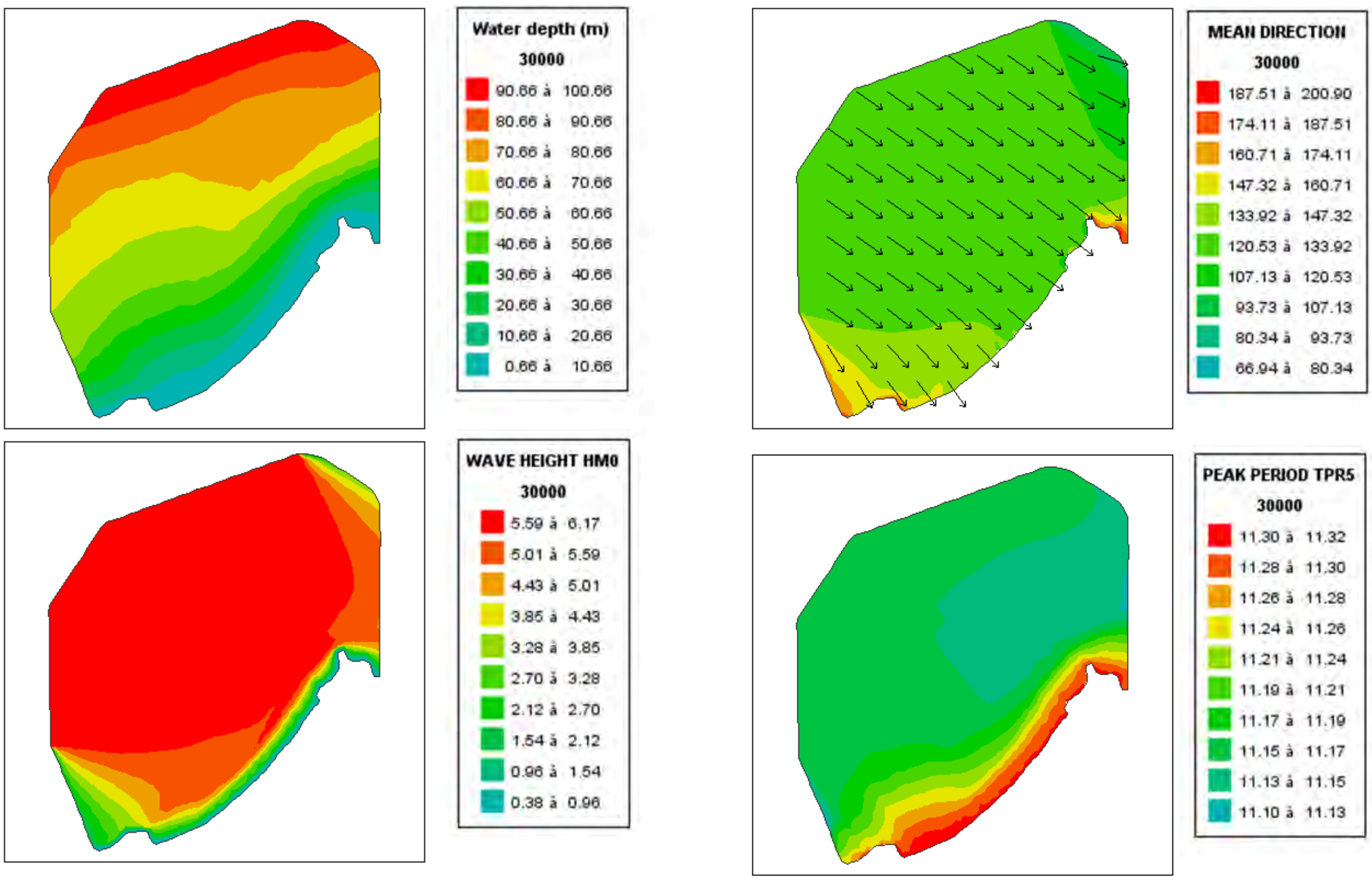


Figure 27 Propagation of waves from North-West, 100-year return period

2.6.2 Operational waves

The operational waves are calculated as the 1% exceedance waves, coming at an angle of 190 °. Since the port entrance is from north-west, these waves do not enter the port directly and are diffracted by the breakwater. However, it can be seen from Figure… that the wave height at 30m depth is about 2 m, which is twice the maximum allowed wave height for container ships and 4 times higher than the allowance for ro-ro ship. Therefore, the wave height needs to be reduced by the use of breakwaters.

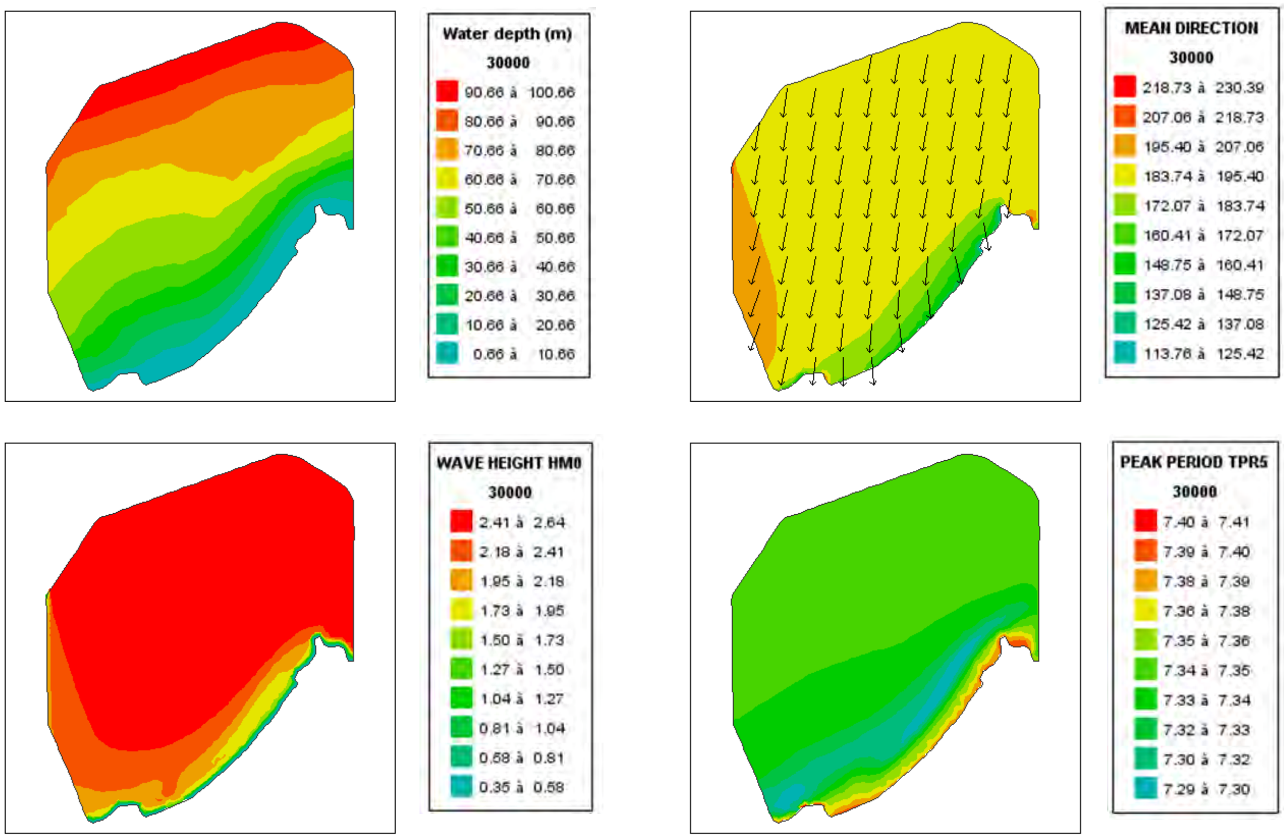


Figure 28 Propagation of waves from North-West, 1% exceedance

2.7 Rising sea water level

According to the latest report conducted by the IPCC (Intergovernmental Panel on Climate Change) the worldwide mean sea water level is expected to continually rise during the 21st century. This is expected to happen at a faster rate than that observed in 1971 to 2010. In the worst case scenario, the mean sea water level is expected to rise between 45 and 82 cm, with an average of 63 cm. The latter is used for further design under the quay and breakwater design. Furthermore, the astronomical tides are also considered in this project. For breakwater design, the high tides are the most critical and for the quay wall the low tides result in the worst case scenario.

2.8 Sediment transport

The waves propagate to the shore and during this way they suffer some transformation due to refraction, shoaling or diffraction. All those change the length, the height, the direction and the energy of the wave. As waves bring a huge amount of energy toward the coast, we need to dissipate it with a coastal protection, in our case with a breakwater.

In this project, there is an input of sediments coming from the Oued Kert. Sediment transport occurs due to the coastal currents along our coast. These currents are influenced by tides and morphological features of the region, and generally the sediments transported are of fine nature.

The sediment transport along the littoral of Betoya bay has been calculated in ARTELIA and is shown in Figure 29. The 2D simulations show that tidal currents are almost zero in the Betoya Bay (about 0.02 m/s). The maximum flow surface current induced by the effects of spring tide comes from a wind from West (0.16m/s) and a wind from East (0.09m/s). Consequently, the coastal currents reach a maximum speed of 0.16 m/s in extreme conditions of wind and tide.

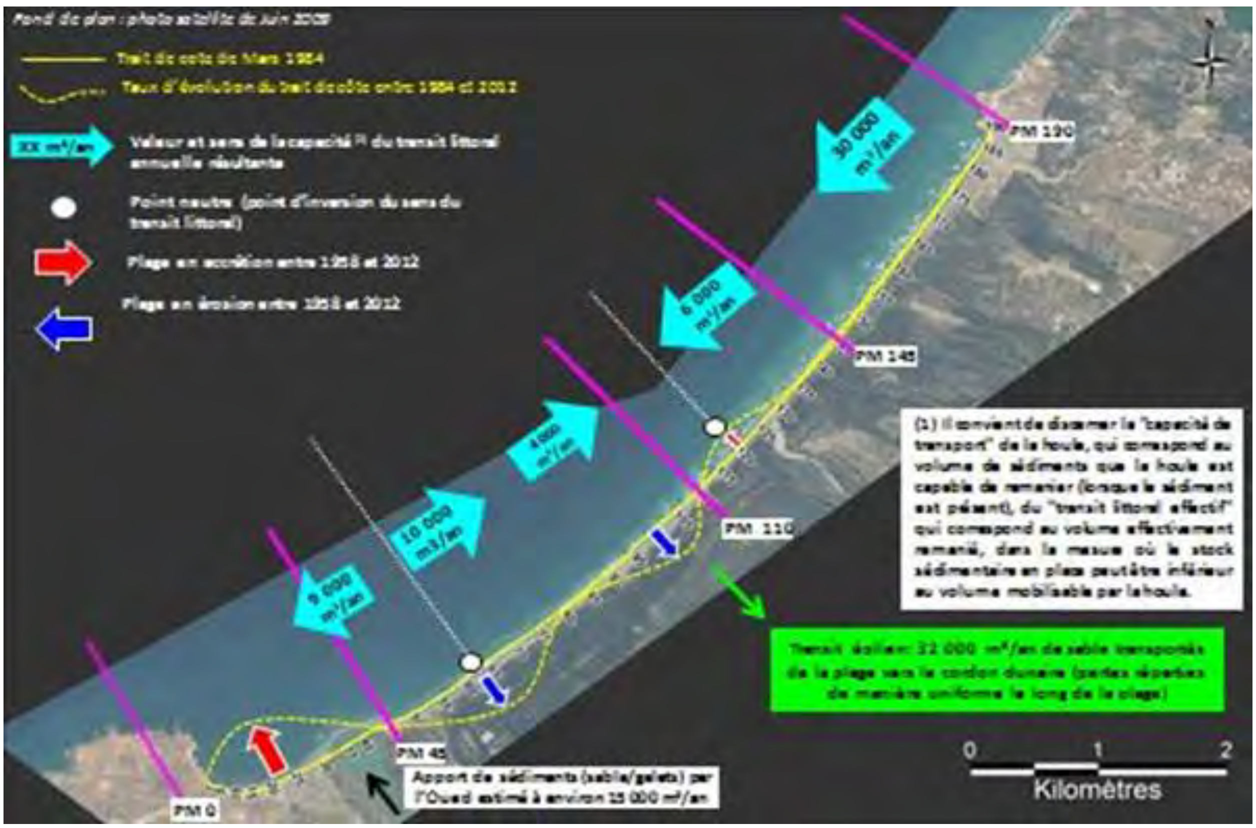


Figure 29 Sediment transport in Betoya Bay

The amount of sediment that the currents can transport is not very high. The currents are coming from North East and South West making the bay mostly in equilibrium. The main flow of sediments is by the OUED KERT, which creates an accumulation in the mouth and erosion in the right side of the river. However, due to the currents coming from North East, erosion turns into accretion. There are two neutral points that show us this phenomenon.

Based on the figure above, we can notice that with port entrance on South West, the port can't be put in the South West part because there will be a problem with sedimentation. At the same time, if we are going in the North East part, they will have a lot of sediment going on our breakwater (30 000 m^3/year). However, if we don't want to have accumulation inside our port, we don't want either accumulation outside our port, in the breakwater line.
Consequently, for our port, the best locations to avoid problems with sedimentation will be to place our port in the middle part of the Baie de Betoya, around the PM110 and the neutral point on the right of PM110.
In Figure 30 below, you can see the amount of sediments defined by the different hydrological years.

	Amount of floods	Percentage of occurrences	Amount of sediments
Weak Hydrological years	3	45,50%	200 000 tons/year
Average hydrological year	3	43%	200 000 tons/year
Strong hydrological year	1	6,80%	2 200 000 tons/year
Exceptional Rise	1	4,70%	3 000 000 tons/year

Figure 30 Amounts of sediments in different years

As we can see, almost 90 % of the years, we have around 200 000 Tons of sediments coming from the river. Based on the figure, we have assumed than 50 % of the sediments are going toward South West and the other 50 % will go toward our port. Furthermore, we can assume that our port, including the navigation channel, will have an area of more than 2 000 000 $m^{2.}$ If it is the case, and if all the sediments are going in the port, it will represent a deposition of 4 mm in a year everywhere in the port.

Consequently, we can conclude that 90 % of the year we won't have problem with sedimentation. There is only the 10 % left of the year which can represent a problem. We will have to take into account this problem for our layout and be preparing to have:

- A sediment trap near the entrance of our port,
- Or be prepared to have some dredging operation in some years – sediments are fine and dredging won't be a problem; however port needs to be active 350 days/year.

CHAPTER 3

LAYOUT PROPOSALS

3. Layout proposals

3.1 Location selection

According to the Document 6 (Geotechnical Soft Clay Thickness), the south of the coast (near the River Kert), up to 25m BL contains thick layer of clay, while the middle (up to 20m) and north (up to 30m) coasts contain minimal amount of clay. Moreover, the middle coast, further up the 20m BL, contains moderate amount of clay. Since the clay is not a suitable soil for foundations, it must be adequately treated – dredged or reinforced.

In overall, port location at the south side should be avoided due to large amounts of clay (20 – 24m thickness) and potential further severe sediment build-up due to River Kert. Dredging or soil reinforcement at this location would be uneconomic. In comparison with south coast, middle and north coast have the best geotechnical conditions. Even though north clay has larger extent of non-clay bed and requires less dredging for providing draught depth compared to middle coast, deep water conditions at the north coast will complicate construction of breakwaters and consequently will raise construction costs. All the above considerations lead to the conclusion of middle coast (~4km from the River Kert) being the most optimal location for the new port.

The majority of key length is situated at the 0 water depth line because we need to minimise our breakwater costs. This dredging in shallow water by backhoe dredger. Some terminals may be in deeper water which makes sure that there is sufficient place for berthing and the port length is kept optimal (not too long).

In the middle of our port, there is a dried out river that needs to be split into two. On both sides of the port a small river will then flush the sediment to deeper areas, therefore solving the problem.

3.2 Approximate estimation of dredging volumes and costs

Approximate estimation of dredging in south, middle and north coast has been conducted. It was assumed that the amount of dredging will be required depending on clay thickness and maximum design ship draught required. At this stage, no other soil than clay has been considered. The calculations were carried out in accordance to Document 6.

Location	Volume of dredged material (m^3)	Dredging Cost (EUR)
South Coast	35,100,000	351,000,000
Middle Coast	29,400,000	205,000,000
North Coast	17,500,000	123,000,000

Figure 31 Approximate estimation of dredging volume and costs

Assumptions used:

- Only topsoil (clay) must be removed in order to provide sufficient support for breakwater foundations.
- Entrance channel and the whole area of inner port must be dredged up to 25 m to provide sufficient water depth for maximum design ship (24 m).

3.3 Port dimensions: flotation and navigation areas

3.3.1 Approach channel

It is recommended that the orientation of the channel must be parallel to the main winds and currents. In case of a dredged channel, the shortest possible length is used to minimize the dredging costs. Minimise the angle between the channel and dominant wave direction, and minimum number of bends and avoid bends close to the port entrance.

Using the formulas in Appendix I, the dimensions of the approach channel were calculated. They can be summarised in Figure 32.

Length	Width	Depth
2383	285	24.1

Figure 32 Approach channel dimensions

3.3.2 Turning circle and tugboats

The inner approach channel ends in a turning circle from where the vessels are towed by tugboats to their respective berth location. The diameter of this turning circle should be larger than 2 times Ls (LOA). The diameter will be 2 x 370 m = 740 m.

The distance of the circle until the closest point of the terminal is 190 m. See plan view in figure 33.

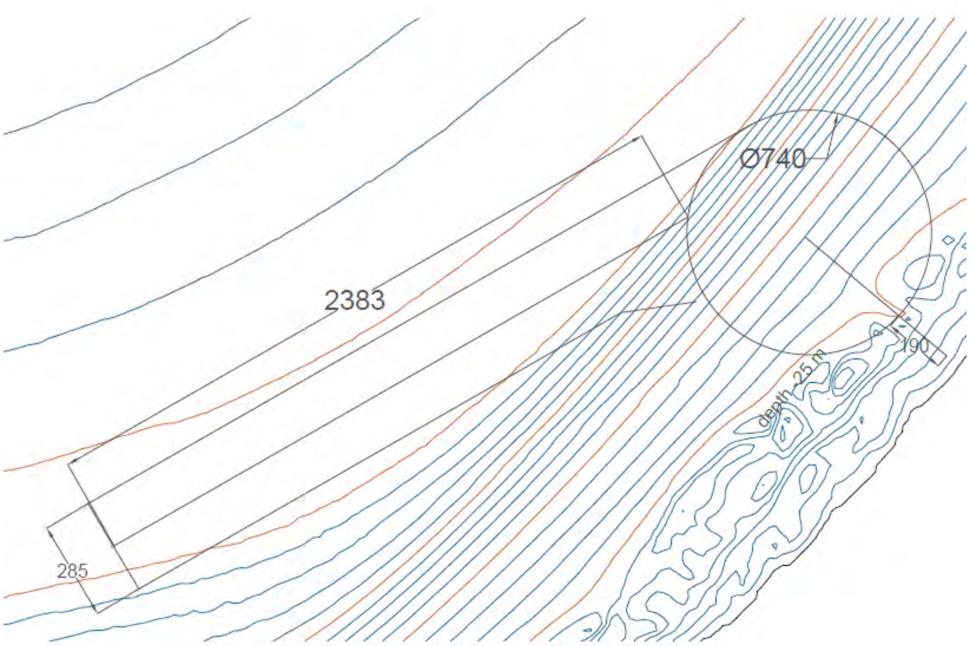


Figure 33 Plan view of the turning circle

3.3.3. Level of the quay

The level of the quay is calculated so that in no circumstances it is under the water. The designers considered the high tidal level; 0.66 m, the increase of the level in the useful life of the port; 0.6 m and a freeboard for safety of 0.5 m. The result is 1.76 mZ.

For the Ro-Ro ramps the recommendation is to take at least 1.5 m above the sea surface in such a way the slope of the ramp allows the exchange between ship and port area. So the minimum quay level is 2.8 mZ, this level was taken for all the quays in the way to keep the surface horizontal and without steps.

3.4 Layout alternatives

3.4.1 Layout 1

For all of the layouts in this chapter, the entrance is considered from the South-West and the breakwater was constructed in as shallow water as possible. Figure 34 presents the first layout proposal.

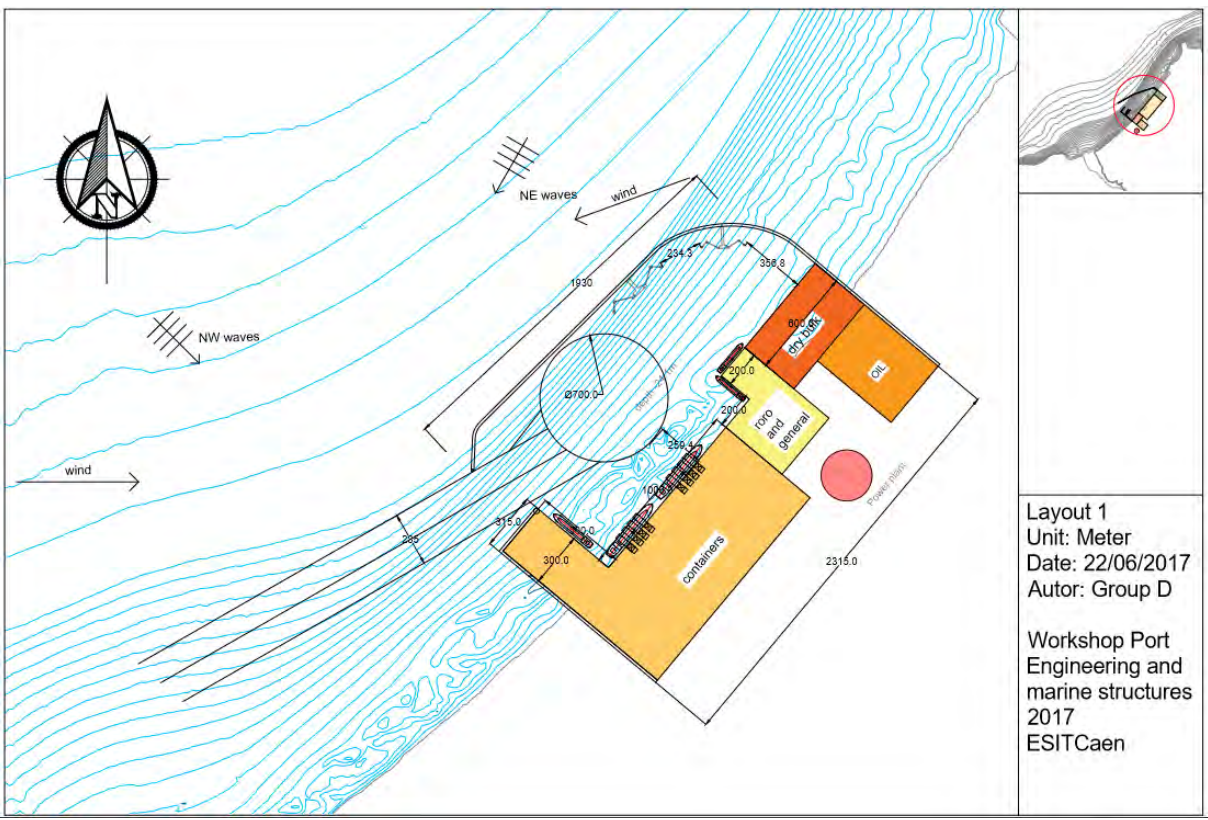


Figure 34 Layout Option 1

In this first layout the two jetties with mooring dolphins are situated in deep water in the inner bend of the breakwater. The oil terminal is situated behind the dry bulk terminal and for safety reasons there is a wall of containers, filled with (dredged) sand, in between the terminals. The rounded shape of the breakwater is possible because rubble mound is used but it can be more expansive than a straight breakwater. The ro-ro and general bulk terminal is situated in the middle with a corner in the quay in order to place the ramp for the roll-on roll-of vehicles. The container terminal is situated in such a way that expansion is possible in south west direction. The turning circle is close to the breakwater but there is a safety distance of 250 m between the container ships and the circle.

3.4.2 Layout 2

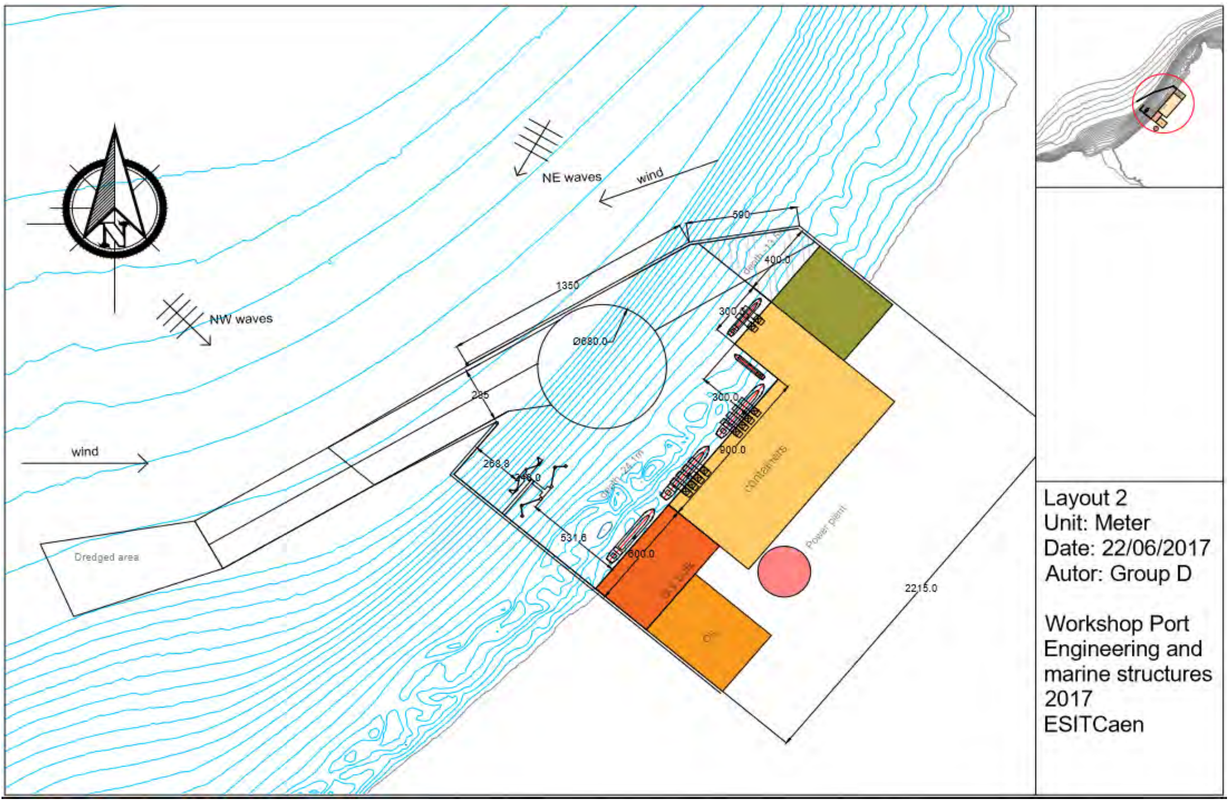


Figure 35 Layout Option 2

In this layout (Figure 35) the breakwaters are straight without curves and the end of the approach channel end up into the slower area at the breakwater. The roro and general cargo are placed at the shallower part in the north east and the dry bulk (deeper vessels) terminal is placed in the deeper waters. In this deeper part of the port also the oil tankers can berth at the mooring dolphins. There is a safety distance between the oil tankers and the dry bulk ships of about 530 m. The container terminal can only extend landward but that would not be a problem because there are already five berths and not the number of ships are increasing but only the size of the ships. Our cranes are connected with rails on the quay. The corner in the container terminal may therefore be not efficient. At the end of the approach channel an extra area is dredged for ships to make their turn into the channel.

3.4.3 Layout 2modified

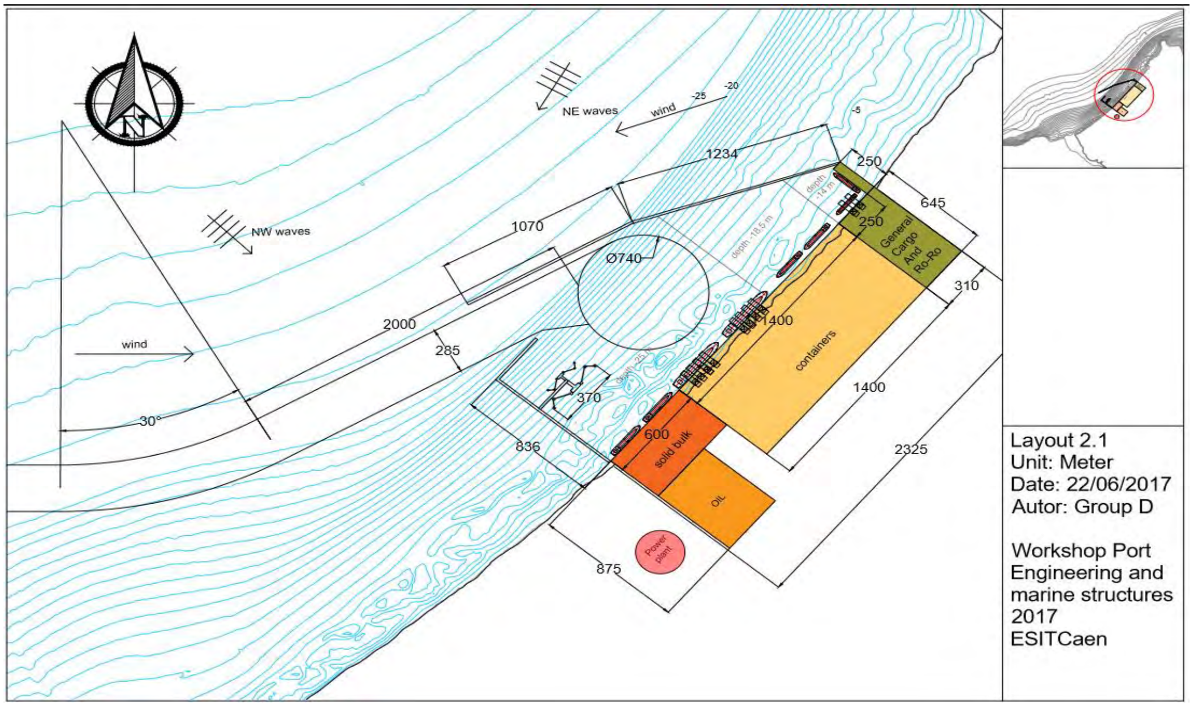


Figure 36 Modified Layout Option 2

The previous layouts both have different advantages. The main idea of the second layout (straight breakwaters) had the preference. For this reason, the second layout is modified by adding some elements from the first layout. Next to this, the upper breakwater is somewhat longer and in the approach channel we added a small bend (30 degrees and diameter of 2400m) which turns into deeper water. Because of manoeuvrability reasons the terminal are situated at one line. This line is situated at 0 m water depth because this is cheaper (dredging) than having a majority of the quay at -5 m water depth, which was initially the plan (see Appendix J). In this modified layout only the ro-ro and general cargo terminal (green part, see Figure 36) has a corner where the ramp is situated. A large improvement compared with layout one is the direction of the approach channel which is pointed at a breakwater instead of a quay. In case a ship is not able to break it will not end up in collision with another ship.

3.5 Wave Agitation

Criteria for agitation near berths:

The next criteria are given for a wave exceeding 1% of the time.

-Hs=1.00m for oil and coal berths

-Hs=0.70m for containers berths

-Hs=0.50m for general cargo and ro-ro berths.

For calculating the wave height inside the harbour, we use the soft called ARTEMIS, powered by EDF.

In the next pictures, you can see two blue bands, going to land. That bands show where the coastline is.

3.5.1 Layout 1

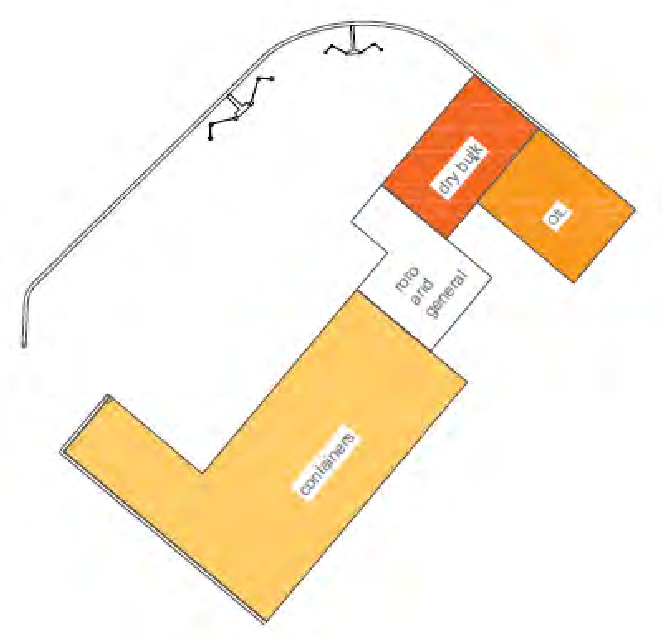


Figure 37 Layout 1 for Agitation

With this long breakwater, the layout 1 quite well protected from waves.

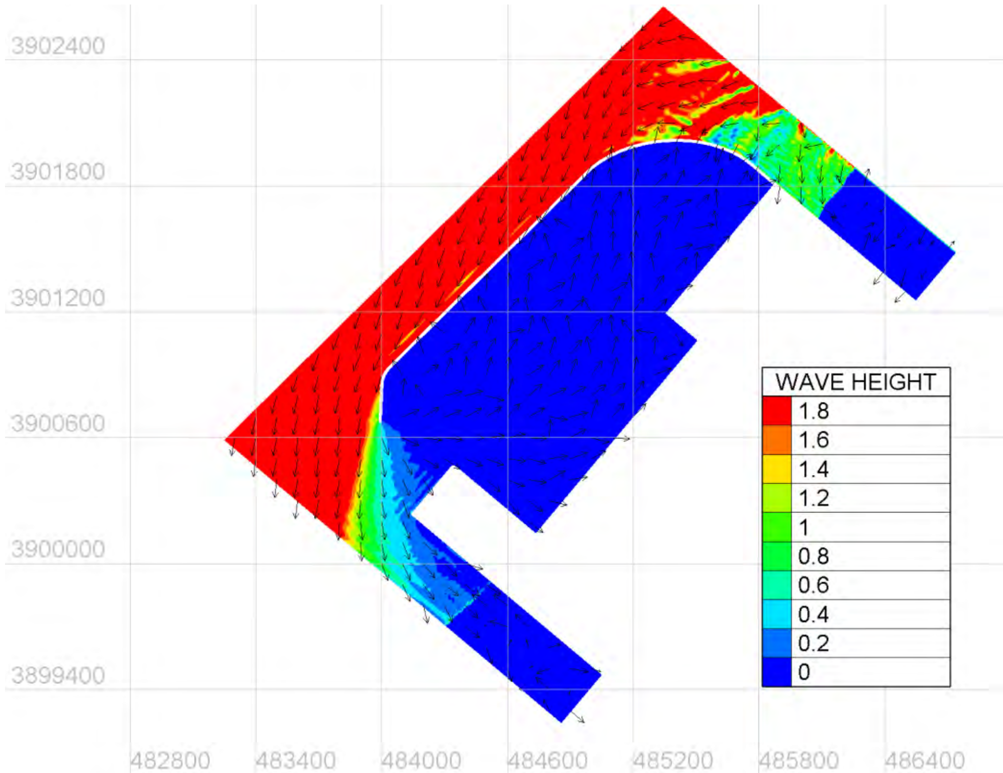


Figure 38 Wave heights are spreaded 2.64m, coming from 10°N (Layout 1)

In this layout (Figure 38) and with that sea state, our port agitation is correct. Maybe we can reduce our main breakwater length.

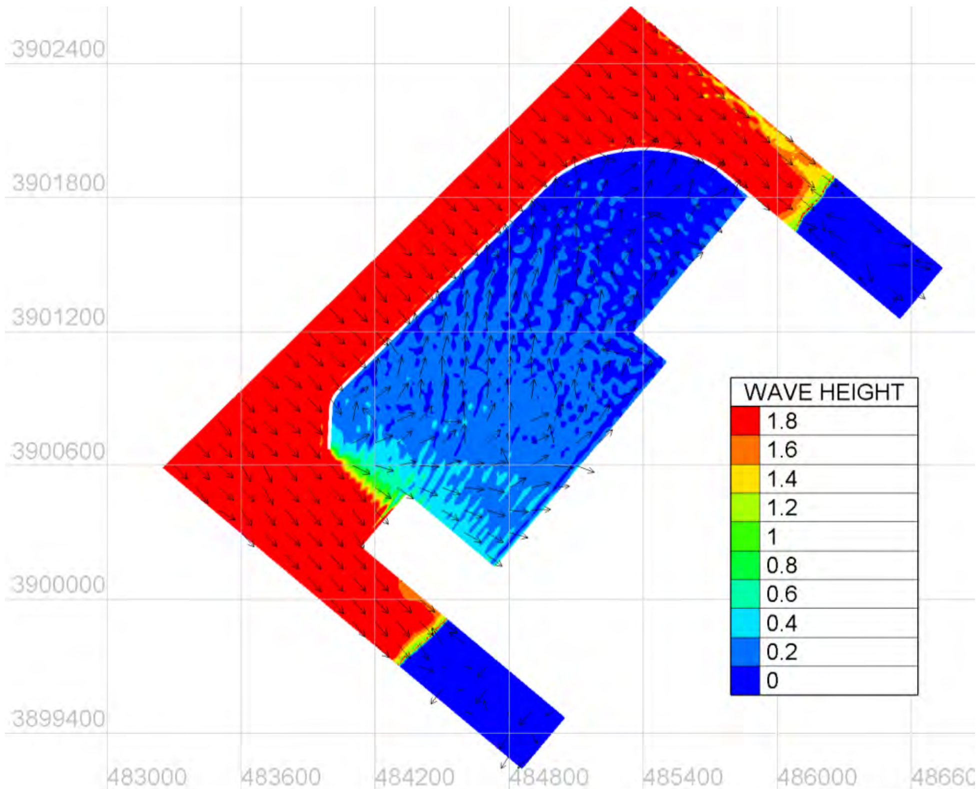


Figure 39 Wave height spreaded 2.64 m, coming from 315°N (Layout 1)

In the layout (Figure 39) and with that sea state, our port has quite big waves. For example, the main wave height for the container terminal next to the entrance is 0.6m. Those waves are admissible for containers berths.

For agitation conditions, that layout is suitable.

3.5.2 Layout 2

Figure 40 Layout 2 for Agitation

In layout 2, we tried to reduce our breakwater length in the deeper part., maybe a bit too much.

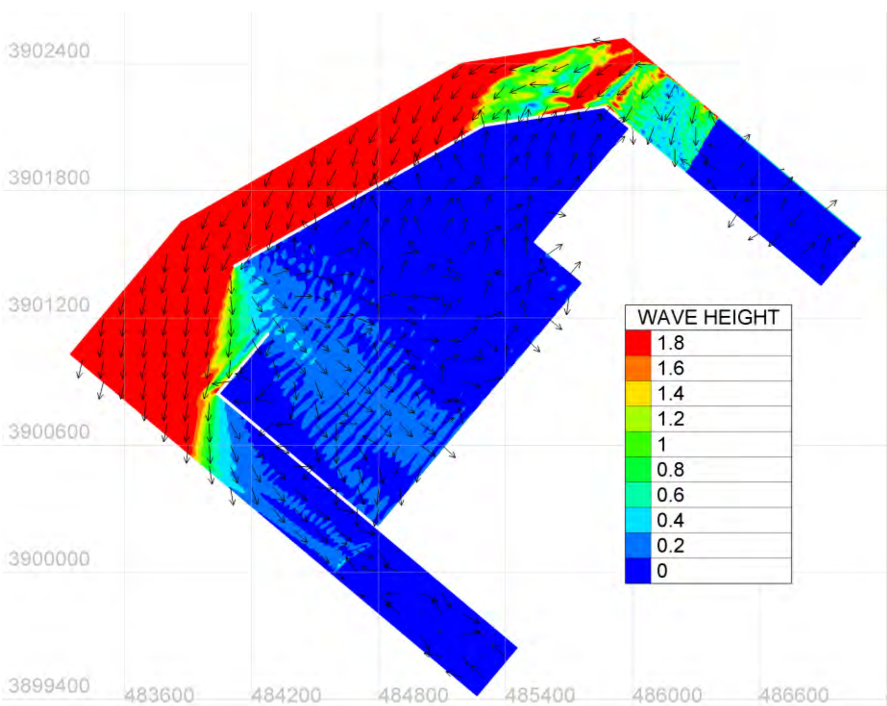


Figure 41 Waves height spreaded 2.64m, coming from 10°N (Layout 2)

In that layout (Figure 41) and with that sea state, our port agitation is correct, there are only little waves.

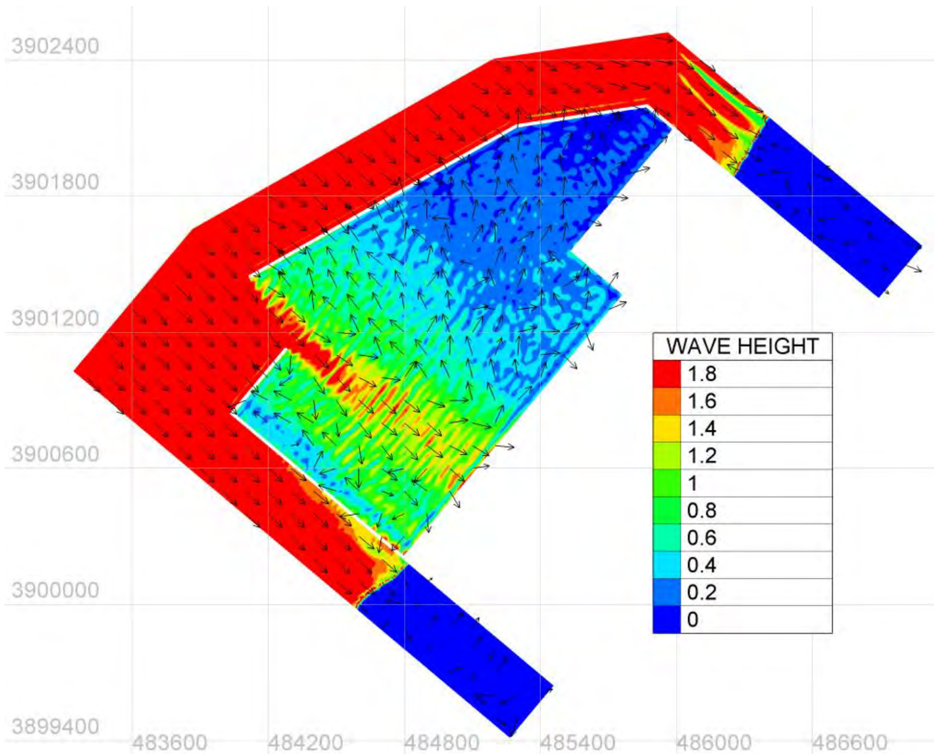
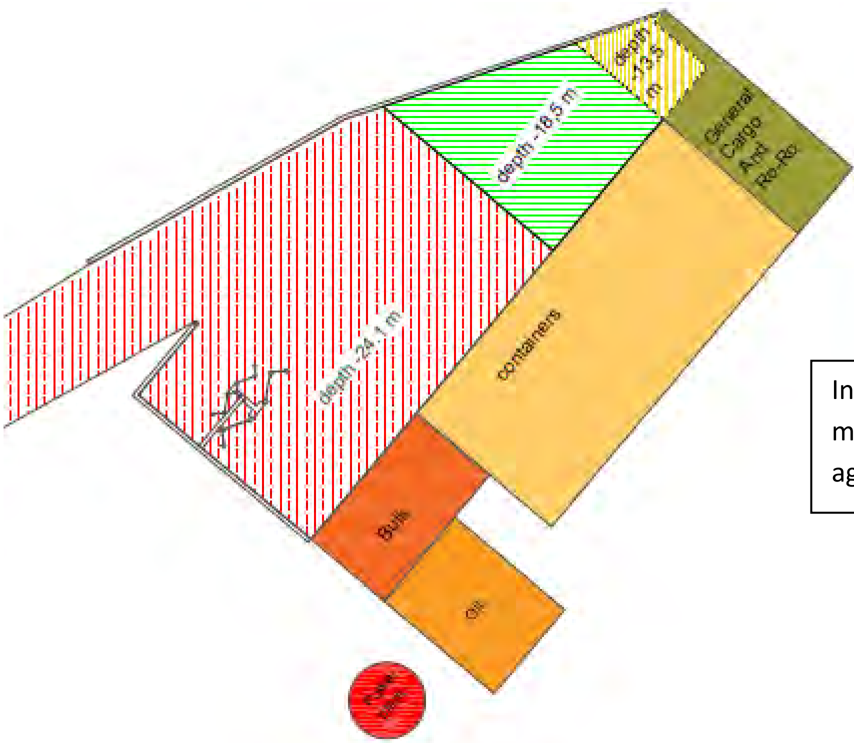


Figure 42 Waves height spreaded 2.64 m, coming from 315°N (Layout 2)

In this layout (Figure 42) and with that sea state, our port agitation is not good. Due to the length of our main breakwater, waves are coming into the port. For that reason, in the next layout, we increase the length of our main breakwater to reduce agitation.

That layout is not receivable; wave agitation is too high for waves coming from North-West.

3.5.3 Layout 2modified



In layout 2modified, we lengthen our main breakwater to reduce wave agitation into the port.

Figure 43 Layout 2 Modified for Agitation

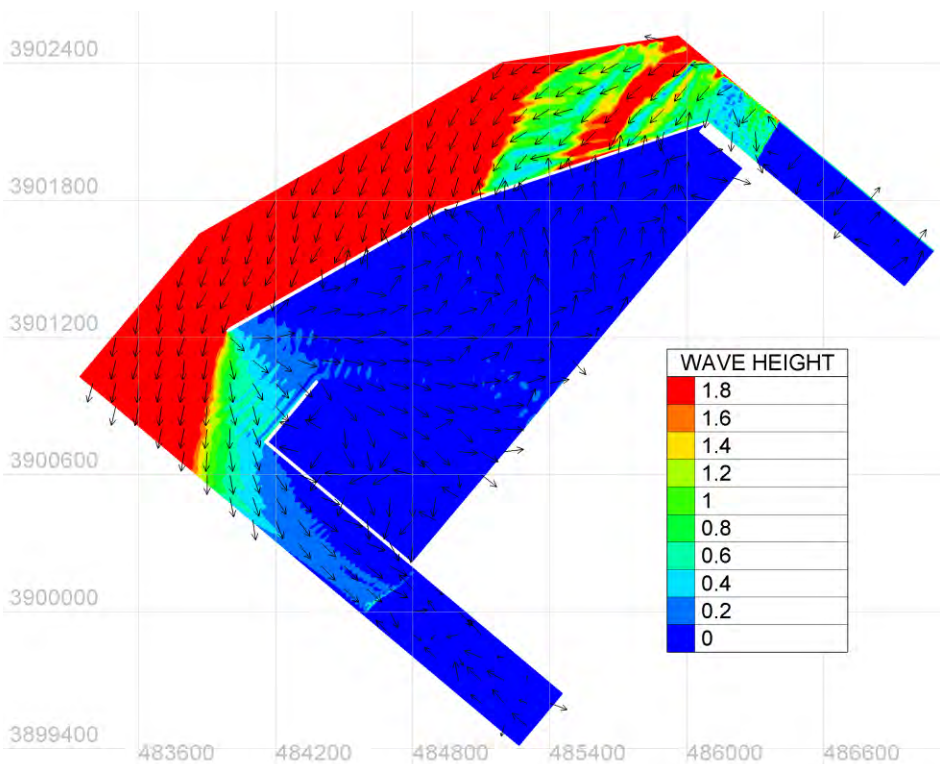


Figure 44 Waves height spreaded 2.64m, coming from 10°N (Layout 2 Modified)

In that layout (Figure 44) and with that sea state, our port agitation is correct. Only a few waves are entering in the port.

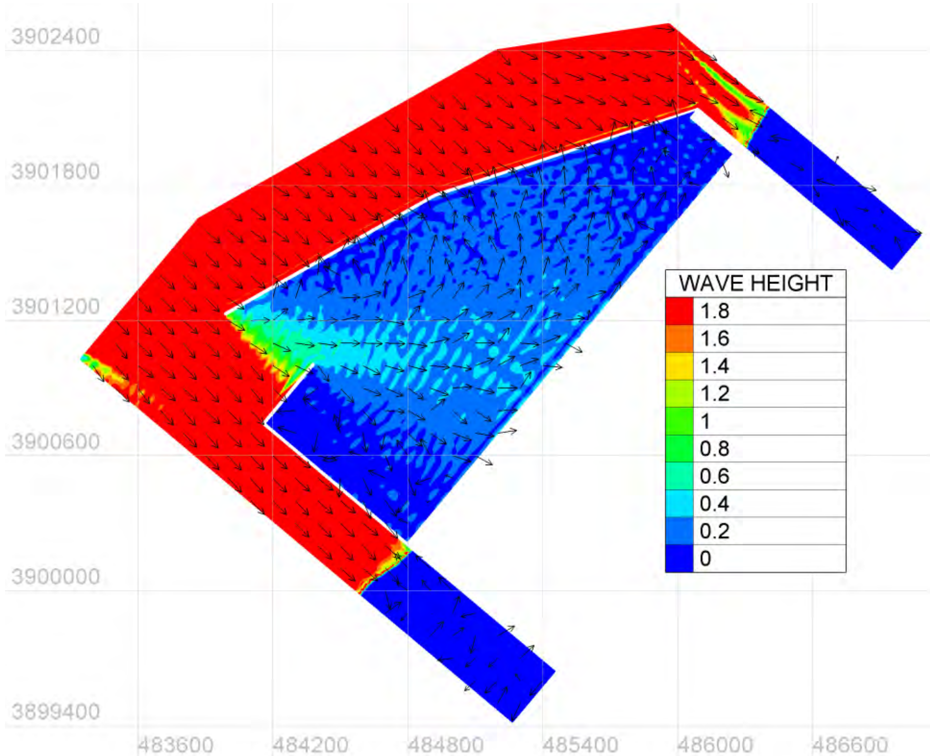


Figure 45 Wave height spreaded 2.64m, coming from 315°N (Layout 2 Modified)

In that layout (Figure 45) and with that sea state, our port agitation is correct. There are a few waves entering in the port, but there are admissible according to the criteria.

That layout is receivable, even if there are little waves.

CHAPTER 4

4. Alternative analysis

4.1 Criteria Selection for Multi-Criteria Analysis

CRITERION WEIGHT

The scale between 0 and 1 point to assign CRITERION WEIGHT has been use. Descriptions of criterion weight selection and importance are presented further into the section.

Costs: the cost of the port is the most important element in order to have a successful project and for this reason a weigh factor of 0.9 was chosen. The choice has been made according to so the size of the breakwater (width, high, length and shape) and the amount of dredging. All of these aspects will are different in each layout. However, designated surface ares of the terminal are the same.

0.9 point

Ship manoeuvrability; this factor is important in terms of safe environment in the port. Ideally, If the entrance of the port should be designed in such a way that ships can enter the port easily without difficult manoeuvres. This will lead to high the score. If we are able to avoid difficult manoeuvres, the susceptibility of accidents will reduce and ships will pass the navigation channel more quickly. The latter will increase the productivity of the whole port.

0.8 point

Wave penetration; this factor is considered to be essential due to the fact that the security of entering ships depend on inclination angle of the breakwater and the orientation of the entrance. The effects of agitation on the moored ships could decrease, if the orientation of both breakwaters is designed properly.

0.8 point

Safety; the lay-out of the port also influences the safety. As written in the Ports book (Vellinga) for safety reasons the end of the approach channel may not be pointed at a quay with berths. This precaution is made in case ship brake failure, which might be defined as crash into quay (or even worse, another ship). The end of the approach channel must be pointed at a shallower sandy part of the port, if possible. In addition to this criterion, the position of the mooring dolphins and oil terminal also influences the safety.

0.8 point

Environment: like the criterion “costs”, the location of the port is quite important due to two aspects:

a. The amount and type of dredging. Due to client specifications, the amount of dredging needs to be maximized, so certain quantity of dredged material might be reused for building terminals and for that it is necessary to study the geotechnical report and the type of soil. Check whether there are no contaminants.

b.	Environmental impact: since the port is to be built in natural environment, all the animal species in the building zone have to be considered. Therefore, in the project it is important to add an environmental impact study to know what kind of marine' species and terrestrial species there are and how they can be affected by the construction and port existence.

0.7 point

Re-using the dredged material; this factor relates to costs and environment. Ideally, the dredged sand should be re-used for building platforms facilitating required terminals.

0.6 point

Building method: The building method of the port will vary according to the kind of breakwaters and types of machinery (dredging ships, cranes, etc.) are used. In addition, the construction period will be different if the sizes and surfaces of the elements are different and according to the types of construction materials (for example, the kind of concrete, using prefabricated beams or not).

0.6 point

Efficiency of terminal position; this factor has a considerable importance due to the fact that our production will depend on the time ships have to use port in order to unload; arrive to dolphins and berths, etc. If the terminal position is efficient, ships will be served more quickly in the port and the production will increase. For instance, the score will be higher if the dry bulk (coal) terminal is located close to the energy plant or when the pipelines for the oil terminal are shorter.

0.6 point

Future extension; the factor is considered to be important because client might desire to have expand the capacity of the port in the future. This potential requirement should be fulfilled at some extent. The easiest way of terminal enlargements should be soaked according to the layout. Since the port has been designed to be already relatively large, the criterion is not considered to be very important in this case.

0.4 point

4.2 Alternative analysis

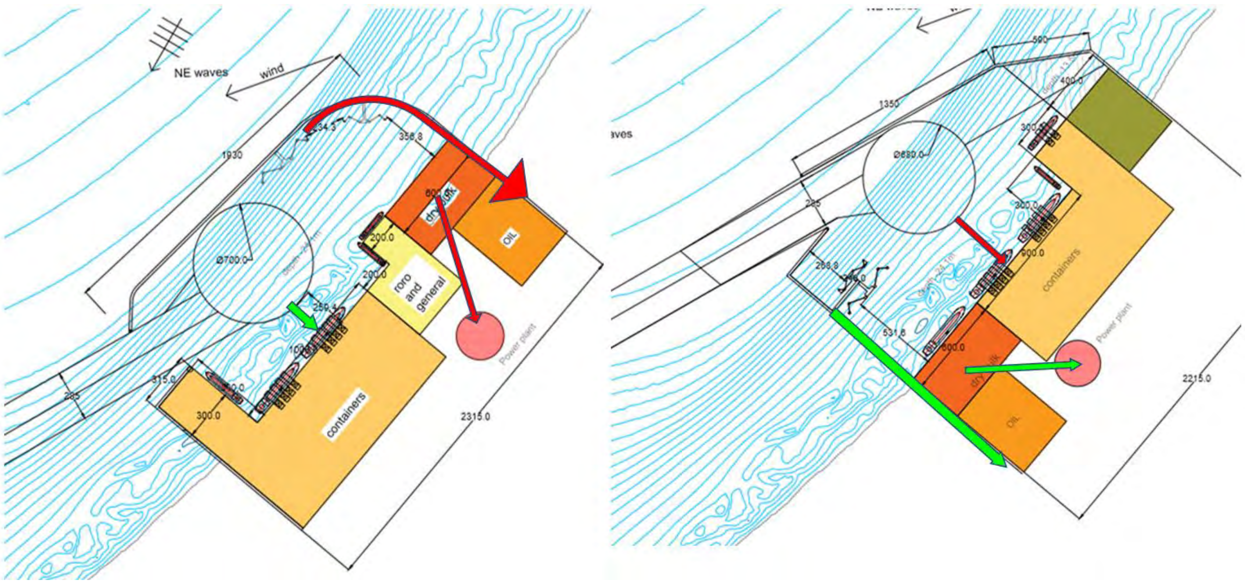


Figure 46 Comparison of Layout 1 and 2

Multi criteria analysis: analysis of alternatives					
CRITERIA	ALTERNATIVE 1	ALTERNATIVE 2	CRITERION WEIGHT	TOTAL	
	WEIGHT			ALT.1	ALT. 2
Costs	2	3	0.9	1.8	2.7
Ship maneuverability	4	3	0,8	3.2	2.4
Wave penetration	2	1	0.8	1.6	0.8
Safety	2	5	0.8	1.6	4
Environment	3	4	0,7	2.1	2.8
Re-using the dredged material	1	2	0.6	0.6	1.2
Building method	3	4	0.6	1.8	2.4
Efficiency of terminal position	4	4	0.6	2.4	2.4
Future extension	4	3	0.4	1.6	1.2
				16.7	19.9

Figure 47 Multi-criteria analysis of Layout 1 and 2

WEIGHTING OF ALTERNATIVES

In this case, the scale to score each weight alternative will be between 0 – 5 points. The evaluation of each alternative is explained below:

Costs: According to estimations, breakwater costs are around 264 million € for Layout 1 and 323 million € for Layout 2. However, this is only the preliminary estimation for Layout 1´s breakwater. Due to the fact that the pipelines on the main breakwater, this estimation is quite wrong because we will need non-significant overtopping on more than 1 km; the price of breakwater will increase. This might indicate the insight of layout 2 ´s breakwater being cheaper.

However, according to dredging costs material we can confirm Layout 2 (268 million €) would be cheaper that Layout 1 (348 million €) as like just like we can check in our calculations.

Due to the both reasons, the score of 3 is selected for Layout 2, while Layout 1 would have 2 points.

Ship maneuverability: layout 1 has easier maneuverability due to the fact that when the ships enter layout 1´s port, they are able to drive until terminals more efficiently than in the port of layout 2. The example more representative is oil terminal. It can be seen clearly that the ships can dock oil terminal without any difficult maneuver when they go in layout 1. However, ships, which want to dock in oil terminal of layout 2, must to turn around 135°. Because of that layout 1 will have 4 points while layout 2 will be valued with 3 points.

Wave penetration; According to the orientation of the entrance we can explain why we gave low score for this criterion for the layout 1. We have a undesirable situation in this layout due to the fact that the orientation of entrance is too rectilinear so we could have three big troubles with this orientation; we would dredge more, the security would decrease in the entrance and the ships would have to do more maneuvers to go into the port.

On the other hand, we can explain a difference between layout 1 ´s agitation and Layout 2´s agitation regarding our calculations. We will have bigger agitation in Layout 2 than in layout 1. Furthermore, the biggest agitation that we have in Layout 2 is in the place where we have designed our oil terminal, it could decrease our security.

Due to the both reasons explained we have decided to score Layout 1 with 2 points and Layout 2 with 1 point.

Safety: According to Ports book (Vellinga), the most important factor to get the safety in our port will be that the end of the approach channel may not be pointed at a quay with berths otherwise the ships will crash into a quay (or even worse, another ship) in case the ships fail to brake. We can check that in layout 1 we have two different terminals at the end of the approach channel (RO-RO and DRY BULK). Nevertheless, we can see that our channel would finish in the breakwater in Layout 2.

Furthermore, another important security factor could be the position of the mooring and oil terminal. We can see how the ships can find dolphins in front of their navigation channel in the Layout 1. However, oil terminal is sufficiently far of our navigation channel in Layout 2 so it will be more secure in order to avoid crashes.

Due to the both reasons which we have explained, we have estimated Layout 2 will be safer than Layout 1 so we are going to value layout 1 with 2 points and layout 2 with 5 points.

Environment:

a. Respect the amount and type of dredging, in the both layouts we can not use all of material we have dredged due to the fact that the material's qualities are not enough good to build our terminals. This is a negative aspect to environment criteria. Even though we have to dredge more in Layout 1 than in Layout 2, we consider a big difference to value this criterion due to the fact that in the Layout 1 we have to dredge 34.819.368 m^3 while in the Layout 2 just 26.809.200 m^3.

b. With respect to animals, in the both layouts we have the same situation, we have decided to not design any platform to care the environmental impact about animals. We could build something if we consider in the future that it can be necessary.

Due to these facts we have decided value Layout 2 with 4 points and Layout 1 with 3 points.

Re-using the dredged material; According to our calculations, we would need to dredge in layout 1 around 35 millions of m^3 while in layout 2 we would need around 27 millions of m^3. On the other hand, we know that we can re-use just some of dredged material in the both layouts due to the fact that the majority of dredged material will not have a good quality to be re-used. Furthermore, we have calculated the volume of our terminals where we can re-use our dredged material, it is around 960,000 m^3 in Layout 1 and 626,000 m^3 in Layout 2. Nevertheless, we should indicate this volume is a big dredged material volume.

Depending on our calculations, we can explain that we will be able to re-use enough dredged material to build all of terminals area which are inside to the sea. However, we can check that we will have more surplus dredged material with good qualities in Layout 1 than in layout 2. Besides, we know that we should dredge more in layout 1 than in layout 2. Due to these facts, we have scored with 1 points the layout 1 and layout 2 with 2 point. The both of them have a low score because we have to dredge too much material just like we have to design our port.

Building method: We have wanted to use the same breakwater for the both of layouts, a rubble mound breakwater. However, we should use a complex building method to build the curve corner of main breakwater in Layout 1 so this design to increase the breakwater cost.

We are going to use the same kind of breakwater for all of different sections of breakwater because we have considered that it will be easier in order to decrease the difficulty of building method.

Besides, we are going to suppose that we use the same dredging machinery for the both of layouts so we have to score with 4 points Layout 2 and 3 points Layout 1.

Efficiency of terminal position; If we observe each layout we can see that according to each layout, there are different distances between the turning circle and the terminals. As we have already explained, a longer distance between the both of areas would decrease our productivity. In accordance with this explication, we can check that in layout 1 we have less distance between turning circle and terminals than in the layout 2. We should indicate also that the distance difference between the both layouts is not too much big. In spite of this fact, we have to know that the productivity of our port will be affected in a long term.

On the other hand, we should add that the both distances are too big.

However, if we consider the distance between the dry bulk (coal) terminal and the energy plant we can see that this distance is bigger in layout 1 than in layout 2. Below we indicate the both distances:

(LAYOUT 1); DRY BULK TERMINAL - POWER PLANT = 780 m.

(LAYOUT 2); DRY BULK TERMINAL - POWER PLANT = 651 m.

We can check also the length of oil terminal pipelines. In the layout 1 we have 1535 meters of pipelines while in the layout 2 we have 1294 meters of pipelines for oil terminal. In layout 2 the pipelines that we would use would be shorter.

Due these facts we have decided to value layout 1 with the same score that layout 2, 4 points.

Future extension: If we are going to enlarge our port in the future we have to know which way would be the best. We have to know also that the main terminal enlargement is usually for containers terminal. In our project, the potential of container terminal enragement is not considered because current number of berths will be sufficient for future as well according to our applied studies.

However, if we had to enlarge this terminal, we should enlarge this in the same direction as the coast line. Due this fact, we have considered that layout 1 has a better design to the future expansion than layout 2 which can not enlarge in coast line direction due to the fact that there are other terminals next to container terminal. Therefore, we have decided to score layout 1 with 4 points and layout 2 with 3 points.

CONCLUSION

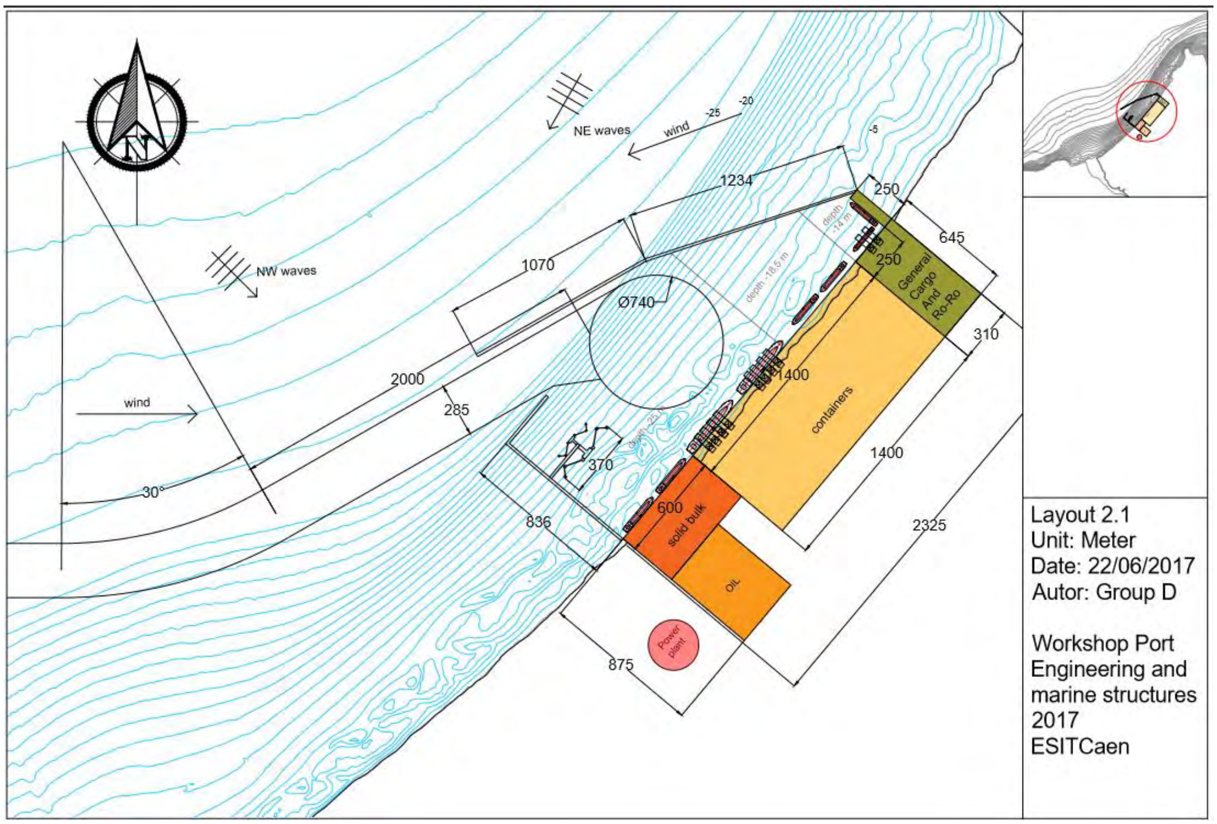
As we can see in the table, the alternative 2 has better score. It means that our Layout 2 would be the best choice to design our port.

However, we have wanted to go further and we have designed a new layout due to the fact that we have checked the Layout 2 has some significant drawbacks. We have designed the LAYOUT 2 Modified, it is a mix between the both layouts with the benefits of each layout. Below, we are going to explain the drawbacks and benefits of our new layout and we are going to compare with the other layouts with the same multi criteria analysis; analysis of alternatives.

4.3 Modified layout 2

Layout 2 has been compared using the same MCA against the two previous alternatives.

LAYOUT 2 MODIFIED



WEIGHT ALTERNATIVE; LAYOUT 2 MODIFIED

Costs:

We have tried two different locations for this layout:

- With the quays mostly at -5 m ZH
- With the quays mostly at 0 m ZH

To decide which one is the best, we have made a cost analysis (Appendix L). We decided to compare both of this locations with the Breakwater costs and the additional dredging we will have with the quays mostly at 0 m ZH. We obtained this cost:

- With the quays mostly at -5 m ZH: Breakwater cost = 448 839 500 €

- With the quays mostly at 0 m ZH: Breakwater + additional dredging cost = 414 858 500 €

Based on this estimated cost, we have decided to use the layout with the majority of the quays at 0 m ZH.

We have decreased the Layout 2´s breakwater. We have enlarged the breakwater in the entrance of our port. However, we have shortened the distance between the turn circle and our terminals so we have decreased the amount of dredging and the breakwater´s length. Due these facts the LAYOUT 2 MODIFIED´s price is the cheapest of our alternatives and we are going to score with 5 points.

Ship maneuverability: We have considered that our new layout will have the oil terminal in the same place than layout 2 due to the security. However, we have an easier manoeuvrability in layout 1 due the fact that the ships are able to drive until terminals more efficiently when they arrive to port of layout 1 than in the port of layout 2 and 2 MODIFIED.

However, we have decreased the distance between turn circle and terminals as we have already explained so in the same way, the ship manoeuvrability will decrease. On the other hand, the entrance of our port has been modified and we have a curve entrance in the new layout where our ships will be able to go in easier.

Due to the first reason we cannot score LAYOUT 2 MODIFIED with the maximum, however we think 4.5 points would be a fair score.

Wave penetration; According to the orientation of the entrance we can explain why we have scored with a high score this criterion for the LAYOUT 2 MODIFIED. The orientation of entrance is 30 degrees so we have a good orientation for decreasing the dredged and improving the security in the entrance.

On the other hand, we have modified the breakwaters of Layout 2 to decrease the agitation. We have decided to enlarge the both of breakwater; main and secondary breakwater. In this way, we have calculated that this way will be the best to design our port in order to improve the security for the ships which go in our port. Like we can see in the calculations, the LAYOUT 2 MODIFIED is the layout with less agitation.

Due to the both reasons explained, we have decided to score LAYOUT 2 MODIFIED with 5 points.

Safety: According to Ports book (Vellinga), the most important factor to get the safety in our port will be that the end of the approach channel may not be pointed at a quay with berths otherwise the ships will crash into a quay (or even worse, another ship) in case the ships fail to brake. In LAYOUT 2 MODIFIED we have kept the same design of layout 1 and we can see that our channel would finish in the General Cargo and Ro-ro Terminal. However, we have modified something in favor of security; we have a depth of 14 meters at the end of channel, the oil ships need around 22 meters of depth so they would not crash to any of the terminals in the case brake failure. In that case ship would simply stop due to shallow seabed depth.

Furthermore, another important security factor could be the position of the mooring and oil terminal. In the LAYOUT 2 MODIFIED like in layout 2, oil terminal is sufficiently far away from the navigation channel so it will be more secure in order to avoid accidents.

Due to the both reasons which we have explained, we have estimated that LAYOUT 2 will be safer so we are going to value LAYOUT 2 MODIFIED with 4.5 points.

Environment:

a. With respect to the amount and type of dredging we have decided to give the same score because in all of layouts we cannot use all of material we have dredged due to the fact that the material quality is poor. This is a negative aspect to environment criteria.
b. With respect to animals, in all of layouts we have the same situation, we have decided to not design any platform to care the environmental impact about animals. We could build something if we consider in the future, if necessary.

Due to these facts we have decided value with the same score, 3 points.

Re-using the dredged material; We have get to increase the dredged material in the LAYOUT 2 MODIFIED shortening the distance between turn circle and terminals. However, we should dredge more in our entrance due to the new orientation which will ensure safety. In spite of this, the Layout with less dredging is LAYOUT 2 MODIFIED.

On the other hand, we have calculated the volume of our terminals where we can re-use our dredged material, it is around 52,000 m^3 in LAYOUT MODIFIED. Depending on our calculations, we can explain that we will be able to re-use enough dredged material to build all of terminals area which are inside to the sea. However, we can check than in LAYOUT 2 MODIFIED we will have more surplus dredged material with good qualities than in the other layouts.

We should indicate also that this volume is not a very large compared to dredged material volume due to the fact that this channel is enough far of the river mouth, it is a place with a big amount of sediments.

According to all of our calculations, we have to say that LAYOUT 2 MODIFIED should be scored with 2.5 points.

Building method: We have wanted to use the same kind of breakwater as Layout 2, a rubble mound breakwater because we have considered that it will be easier in order to decrease the difficulty of building method. Also, we assume that we use the same dredging machinery for all of layouts so we have to score with the same, 4 points.

Efficiency of terminal position; A longer distance between the both areas will decrease our productivity. In accordance with this explication, we can check that in LAYOUT 2 MODIFIED we have decrease the distance between turning circle and terminals so this distance is shorter than the distance of previous layouts.

The distance between the dry bulk (coal) terminal and the energy plant in LAYOUT 2 MODIFIED is bigger than layout 2 and shorter than in layout 1. Nevertheless, the location of our power plant has been changed due to the fact that the previous locations were not effective places in order to increase the port needs because it was in the same way than our leaving way terminals.

The oil terminal pipelines will have the less distance than layout 2. We have tried to change the dolphin position for big oil ships with dolphin position of small ships. However, we have decided to use the same position as layout 2 due to the fact that we have to consider our breakwater to be rubble mound breakwater and this kind of breakwater is too wide so we have preferred to use the widest dolphin for big oil ships on the inner side.

Due these facts we have decided to value LAYOUT 2 MODIFIED with 5 points.

Future extension: If we had to enlarge containers terminal, we should enlarge this in the same direction than the coast line. However, in LAYOUT 2 MODIFIED we have the containers terminal in the middle of the port so we should enlarge our terminal in the direction of leaving way terminal. We have enlarged our port in order to have containers terminal following coast line as Prof. Piero advised us. We have decided to score this criterion with 3.5 points.

Finally, Figure 48 shows that Layout 2 modified is the best option for the port design.

Multi criteria analysis: analysis of alternatives							
CRITERIA	**ALTERNATIVE 1**	**ALTERNATIVE 2**	**ALTERNATIVE 2 MOD.**	**CRITERION WEIGHT**	**TOTAL**		
	WEIGHT				**ALT. 1**	**ALT. 2**	**ALT. 2 MOD**
Costs	**2**	**3**	**5**	**0.9**	**1.8**	**2.7**	**4.5**
Ship maneuverability	**4**	**3**	**4.5**	**0,8**	**3.2**	**2.4**	**3.6**
Wave penetration	**2**	**1**	**5**	**0.8**	**1.6**	**0.8**	**4**
Safety	**2**	**5**	**4.5**	**0.8**	**1.6**	**4**	**3.6**
Environment	**3**	**4**	**3**	**0,7**	**2.1**	**2.8**	**2.1**
Re-using the dredged material	**1**	**2**	**2.5**	**0.6**	**0.6**	**1.2**	**1.5**
Building method	**3**	**4**	**4**	**0.6**	**1.8**	**2.4**	**2.4**
Efficiency of terminal position	**4**	**4**	**5**	**0.6**	**2.4**	**2.4**	**3**
Future extension	**4**	**3**	**3.5**	**0.4**	**1.6**	**1.2**	**1.4**
					16.7	**19.9**	**26.1**

CHAPTER 5

FINAL DESIGN

5. Final Design

5.1 Wave agitation

The wave agitation analysis is the same as presented in section 3.5.3. It is visible from Figure 49 that the waves at the port do not exceed 0.4 m height, which is below the allowable limit of 0.7 m. Only few larger waves are entering the port. Therefore, the port can safely accommodate all different types of ships no matter their sensitivity level.

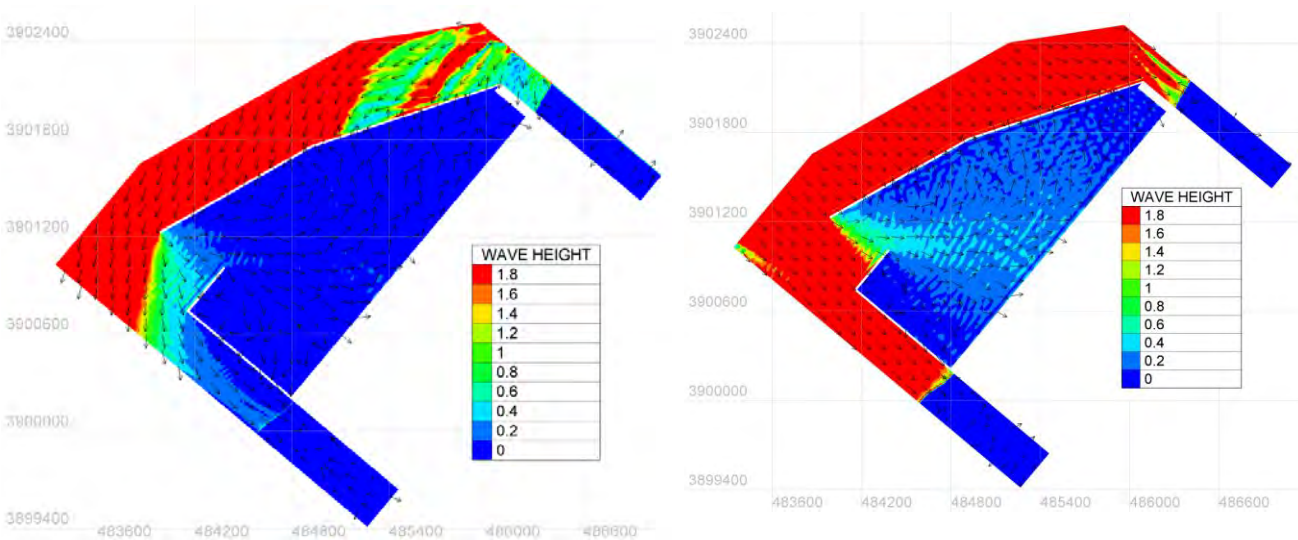


Figure 49 Wave agitation for layout 2 modified

5.2 Channel design

The channel dimensions are as described in 3.3. The channel was curved to avoid ships entering the shallow water on the west side of the port and having navigation problems because of it.

Length	Width	Depth	Turning circle
2383 m	285 m	24.1 m	740 m diameter

Figure 50 Final approach channel dimensions

5.3 Sedimentation

The channel goes towards South West, so it is going to getting closer to the river, where the sediments problems are more important. In that case it is necessary to take care because the channel cannot suffer accretion of sediments. However, as we can see in the design condition, a solid discharge of the Oued is very rare and essentially made of very fine material (clay), which is easy to dredge.

This allows us to say that 90 % of the years, we won't have any problem with sedimentation. The other 10 % will be different. One year each 10 years, we will have huge floods with a lot of sediments. Consequently, after the floods, one dredging operation will be necessary and we will to take it into account for the maintenance of the port.

Figure 51 shows the situation of the outer part of the channel.

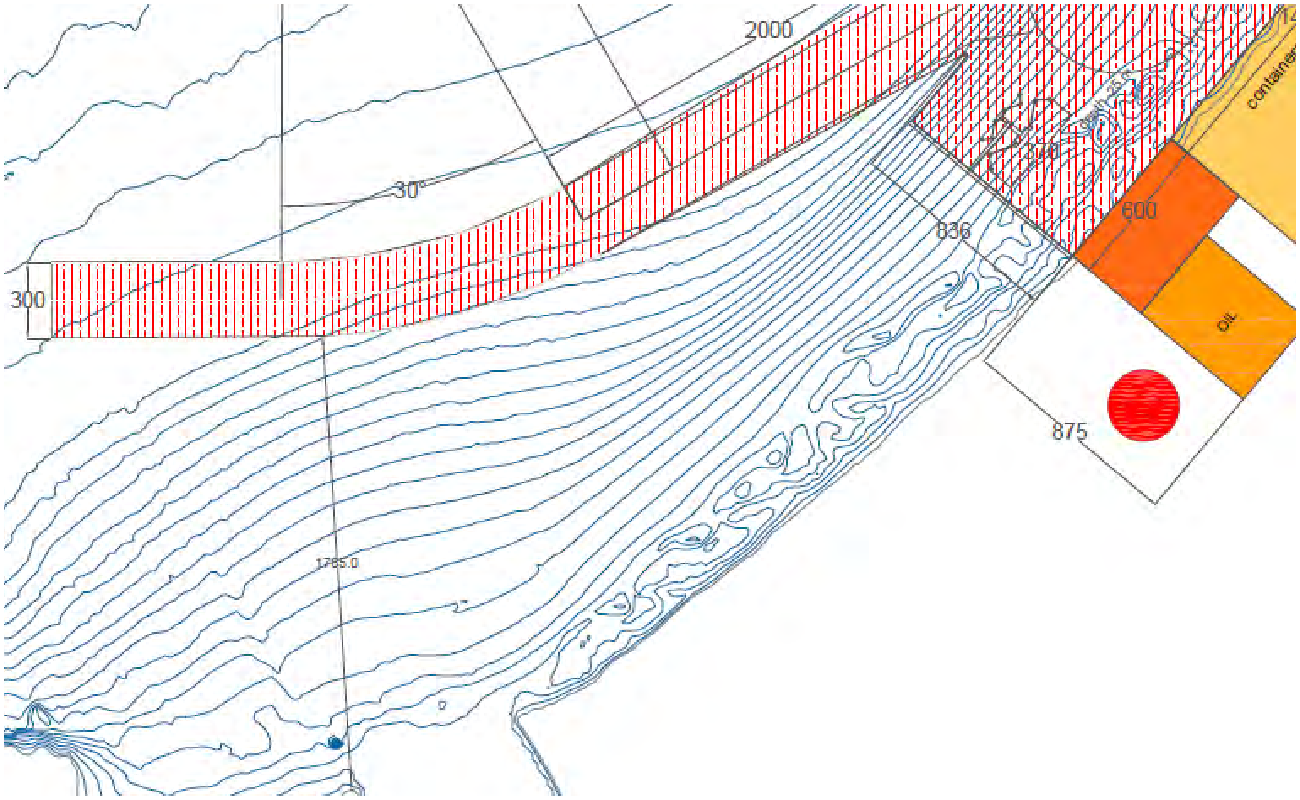


Figure 51 Outer part of the channel

5.4 Breakwater

5.4.1 Selection of rubble mound breakwater

This chapter is presenting the breakwater design of the different sections in Nador West New Port.The design is a probabilistic method; it considered the wave of 100 year return period.

The breakwater was separated into two different sections as it can be seen on figure 52. We studied a rubble mound solution and a vertical breakwater solution. The design condition is to maximize the dredging and minimize the breakwater depth, so the breakwater is seated in shallow water (maxim depth around 20 m). Moreover, the length of the breakwater is not long enough for constructing a vertical breakwater because the fixed cost is high, so this solution is better choice for long breakwaters. Therefore, the design team selected rubble mound for all the breakwater sections.

The group considered different armour possibility: cubes, accropodes and stones. In the cases of stone the breakwater needs to be of higher weight than is reasonable (around 40 tons), so this option was discarded. Comparing cubes and accropodest the first one is easier to construct and put it in place and it is not patented, so it is cheaper. However, the slope of the breakwater must be lower and we need at least two layers for the armour. In accropode design, the slope can be higher and we can put only one layer, decreasing the required material for the construction.

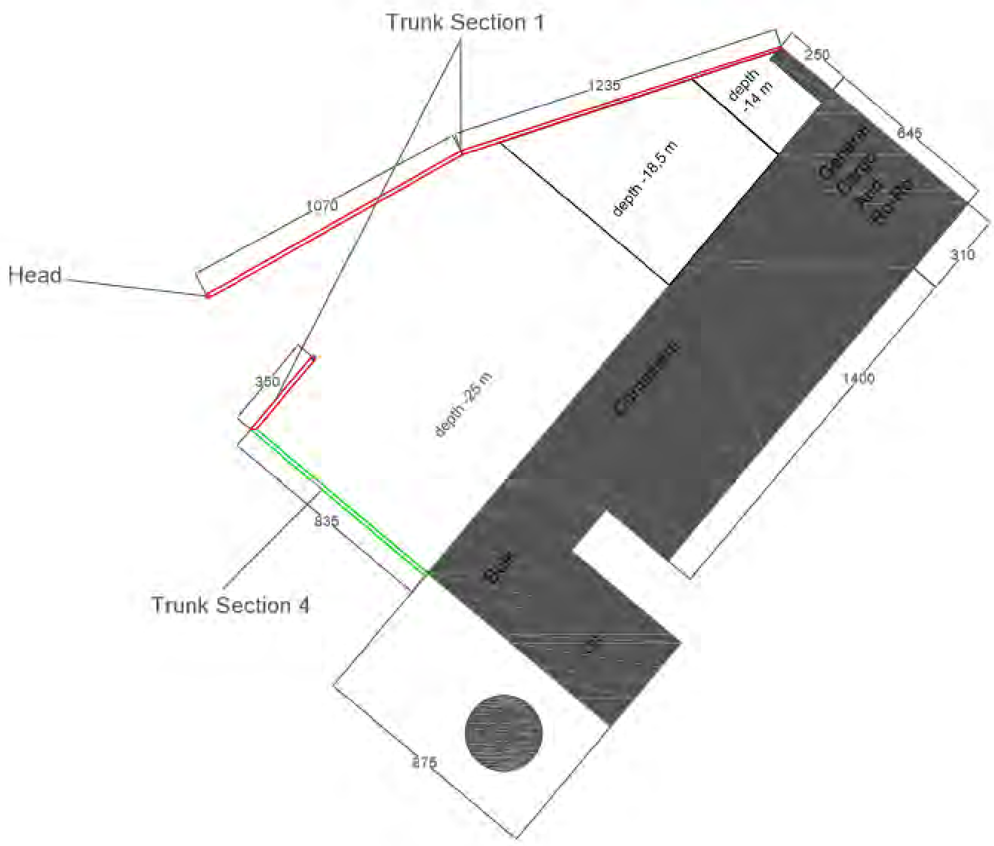


Figure 52 Breakwater sections

5.4.2. Failures modes

Figure 53 represents a typical cross-section of a rubble mound breakwater.

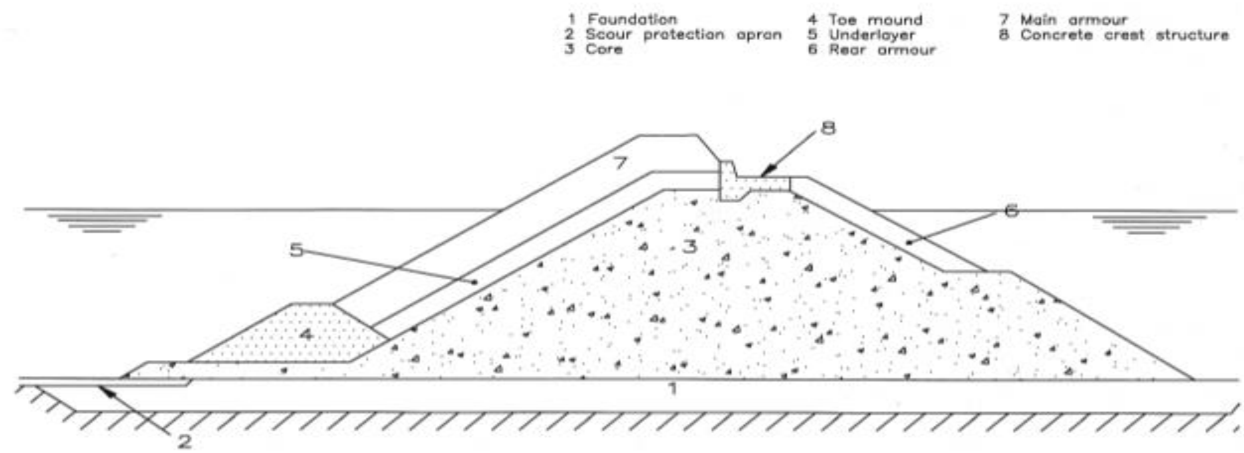


Figure 53 Breakwater cross-section

We considered as failure modes; armour stability (showing as 7 in the figure), rear-side stability (showing as 6 in the figure) and toe berm erosion (showing as 4 in the figure). The loss of units in the armour layer was considered as failure mode for the design because the armour pieces are the main element for the reduction of the high wave. The loss of the rear-side units goes to global instability, and the loss of berm units implies erosion in toe of the breakwater and a global stability loss.

Each of This failure modes were calculated with the same wave (100 years return period), and the Highest High Water Spring for armour and rear-side stability, and Mean Low Water Spring for berm erosion. Armour, rear-side and toe berm stability were checked in Appendix L.

5.4.3 Primary breakwater dimensions

At first the breakwater was separated in four sections and the head haw is showing in the Figure 54. After dimensioning, section 1,2 and 3 turned out to have the same dimensions, so only 1 section is shown here.

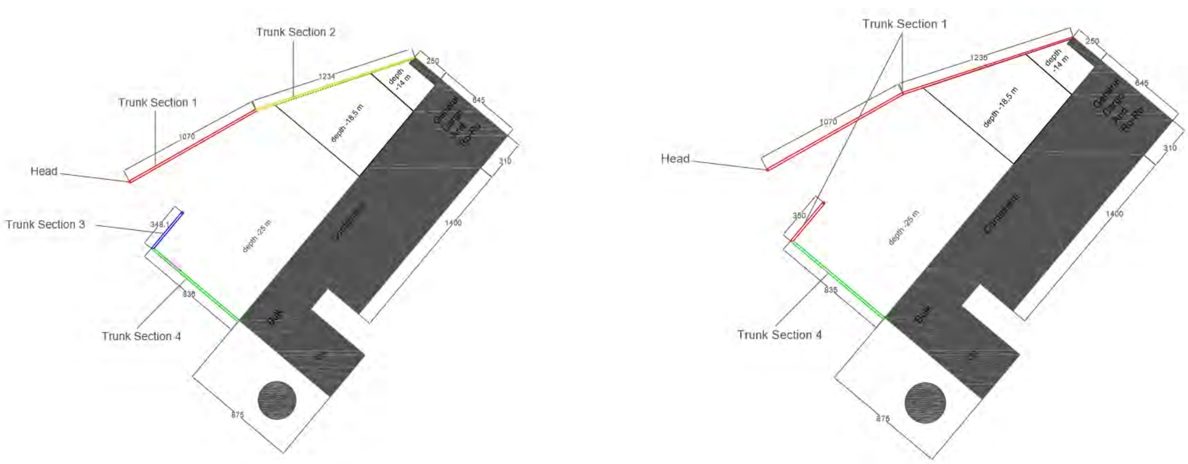


Figure 54 Trunk Section 1: Principal breakwater and section NE-SW of the secondary breakwater

The main parameters used in the design are presented in the Figure 55. The width of the crown is calculated as the necessary to keep three units of accropodes. For the seaside it was taken a slope of 4:3 that is a recommendation from CLI, and 1:2 to the portside slope because the rear-side is constructed with stone. The needed volume from the accropode is 4 m^3 each unit that correspond to 9.4 tons, the size of the under layer units is given as a recommendation by CLI; it is 1 ton the mean weight.

The crown level was defined using the Overtopping Manual equation as was presented in the last chapter; it was calculated for an overtopping of 10 l/s/m that is just under the structural damage overtopping. The needed freeboard is 6.4 m, the high water level is 0.66 m and the expected increase from the water level is 0.6 m, so, the crown level of the breakwater is +7.7 mZ.

The breakwater has berm only in the sea side, this berm was calculated with the Rock Manual formula, the necessary mean weight is 1.7 ton and the geometry is defined taken at least 3 times the thickness layer in the width and 2 times the thickness layer in the height, but taking in account that is mandatory that the height is enough to put up with the accopode layer. Finally the height is 3 meters and the width is 3 meters too.

The results of crown level was used to calculate the rear-side weight unit, it was designed in stone in the wave to save money, because the accropodes are expensive. The mean weight of the units is 3 tons.

Return period (years)	100
Maximal depth (mZ)	-25
Minimal depht (mZ)	0
Maximal water level (mZ)	0,66
Minimal water level (mZ)	0,13
Significan wave design (m)	5,60
Peak period design (s)	11

Parameter	Section 1	Unit	Description
B	6	m	Minimum width from the crown
Alfa	36,86	°	Slope from the seaside
Phi	26,56	°	Slope from the portside
Volume	4	m³	Volume of one accropode unit
Warmour	9.4	ton	Weight of one accropode unit
Wunderlayer	1	ton	Mean weight of underlayer units
Crown level	7.7	mZ	Crown level of the breakwater
Wrear-side	3	ton	Mean weight of the rear-side units
W berm	1.4	ton	Mean weight of the berm units
Width berm	3	m	Width of the top of the berm
Height berm	3	m	Height of the berm

Figure 55 Breakwater design parameters

Figure 56 presents trunk section 1 with the main characteristics and dimensions. The detailed section and in a larger scale is in the Appendix L.

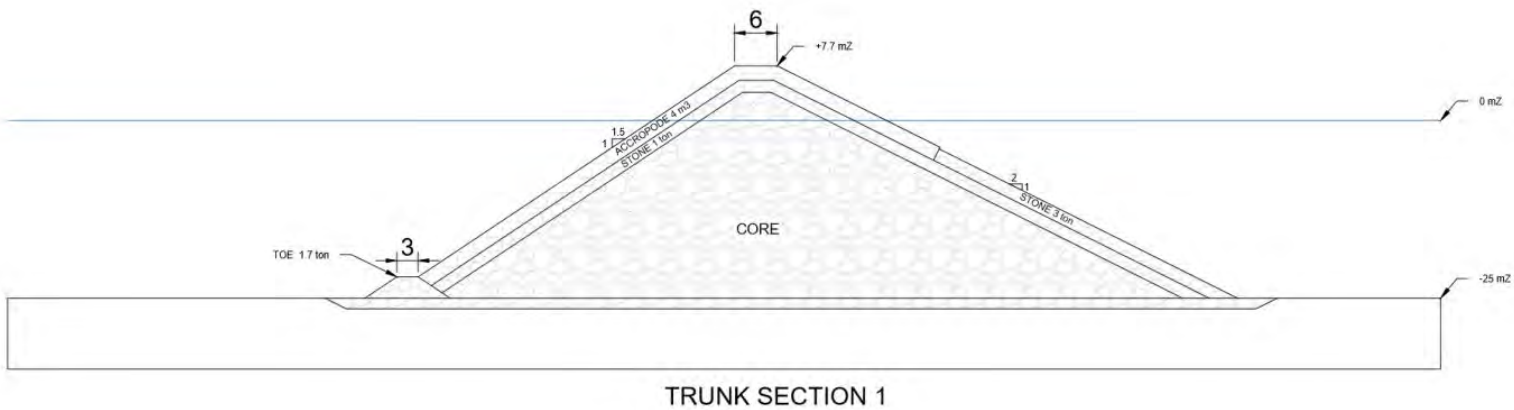


Figure 56 Main Breakwater cross-section

5.4.4. Secondary breakwater perpendicular to the coastline

The main parameters used in the design are presented in Figure 57. The width of the crown is calculated as the necessary to keep three units of accropodes. For the seaside it was taken a slope of 4:3 that is a recommendation from CLI, and 1:2 to the portside slope because the rear-side is constructed with stone. The needed volume from the accropode is 3 m^3 each unit that correspond to 7.0 tons, the size of the underlayer units is given as a recommendation by CLI; it is 1 ton the mean weight.

The crown level was defined using the Overtopping Manual equation as was presented in the last chapter; it was calculated for an overtopping of 10 l/s/m that is just under the structural damage overtopping. The needed freeboard is 4.2 m, the high water level is 0.66 m and the expected increase from the water level is 0.6 m, so, the crown level of the breakwater is +5.5 mZ. This difference is because the waves come approximately parallel to the breakwater.

The breakwater has berm only in the sea side, this berm was calculated with the Rock Manual formula, the necessary mean weight is 1.2 ton and the geometry is defined taken at least 3 times the thickness layer in the width and 2 times the thickness layer in the height, but taking in account that is mandatory that the height is enough to put up with the accropode layer. Finally the height is 3 meters and the width is 3 meters too.

The results of crown level was used to calculate the rear-side weight unit, it was designed in stone in the wave to save money, because the accropodes are expensive. The mean weight of the units is 2 tons.

Return period (years)	100
Maximal depth (mZ)	-16
Minimal depht (mZ)	0
Maximal water level (mZ)	0,66
Minimal water level (mZ)	0,13
Significan wave design (m)	5,0
Peak period design (s)	11

Parameter	Section 1	Unit	Description
B	6	m	Minime width from the crown
Alfa	36,86	º	Slope from the seaside
Phi	26,56	°	Slope from the portside
Volume	3	m^3	Volume of one accropode unit
Warmour	7.0	ton	Weight of one accropode unit
Wunderlayer	1	ton	Mean weight of underlayer units
Crown level	5.5	mZ	Crown level of the breakwater
Wrear-side	2	ton	Mean weight of the rear-side units
W berm	1.2	ton	Mean weight of the berm units
Width berm	3	m	Width of the top of the berm
Height berm	3	m	Height of the berm

Figure 57 Secondary breakwater parameters

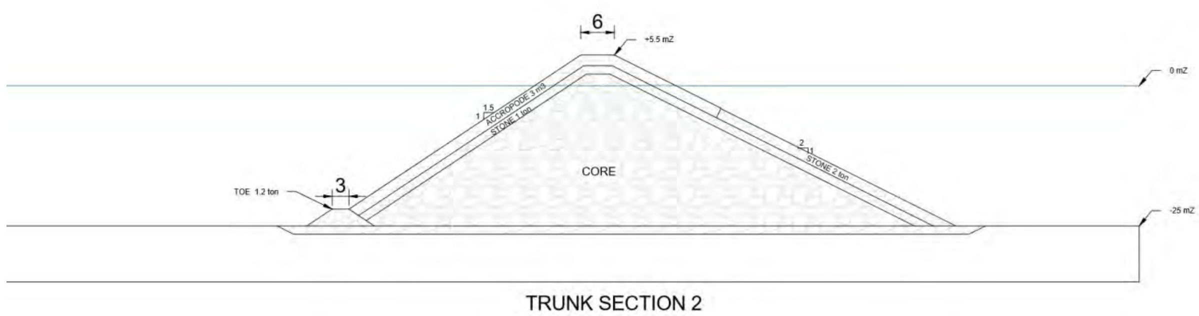


Figure 58 Secondary breakwater cross-section

5.4.5 Head Section

The main parameters used in the design are presented in Figure 59. The width of the crown is calculated as the necessary to keep three units of accropodes. For the seaside it was taken a slope of 4:3 that is a recommendation from CLI, and 1:2 to the portside slope because the rear-side is constructed with stone. The needed volume from the accropode is 5 m^3 each unit that correspond to 12.0 tons, the size of the underlayer units is given as a recommendation by CLI; it is 1 ton the mean weight. The crown level is the same as the trunk section.

The breakwater has berm in both side port and sea. The necessary mean weight is 1.7 ton and the geometry is defined taken at least 3 times the thickness layer in the width and 2 times the thickness layer in the height, but taking in account that is mandatory that the height is enough to put up with the accropode layer. Finally the height is 3 meters and the width is 3 meters too.

Return period (years)	100
Water depth (mZ)	-25
Maximal water level (mZ)	0,66
Minimal water level (mZ)	0,13
Significan wave design (m)	5,6
Peak period design (s)	11

Parameter	Section 1	Unit	Description
Alfa	36,86	°	Slope from the seaside
Phi	36,86	°	Slope from the portside
Volume	5	m³	Volume of one accropode unit
Warmour	12	ton	Weight of one accropode unit
Wunderlayer	1	ton	Mean weight of underlayer units
Crown level	7,7	mZ	Crown level of the breakwater
W berm	1.7	ton	Mean weight of the berm units
Width berm	3	m	Width of the top of the berm
Height berm	3	m	Height of the berm

Figure 59 Head design parameters

5.4.5 Slope stability of breakwater

Not only should the breakwater be designed for hydraulic and

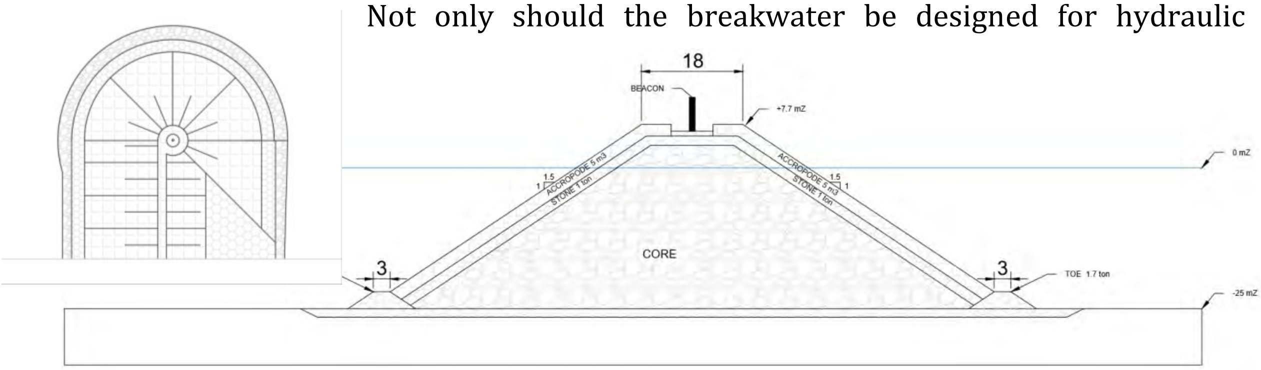


Figure 60 Head cross-section

structural responses, but also a geotechnical analysis is necessary. It is therefore important to investigate possible geotechnical failure modes. In this project, the failure of the breakwater foundation is not considered. Rather, the failure mode considered in this regard is the slip failure. Bishop's simplified method is used in the program Talren where the factor of safety F_{min} is determined. The factor of safety is defined as the ratio between the moments resisting movements and moments motivating movements. The motivating moments includes gravitational weight of soil and water. For the resisting moment, the frictional and cohesional strengths are included. The setup for the model is shown in the figure below and the soil parameters are taken from the provided table of different parameters. These can be seen in the upper left corner of the figure.

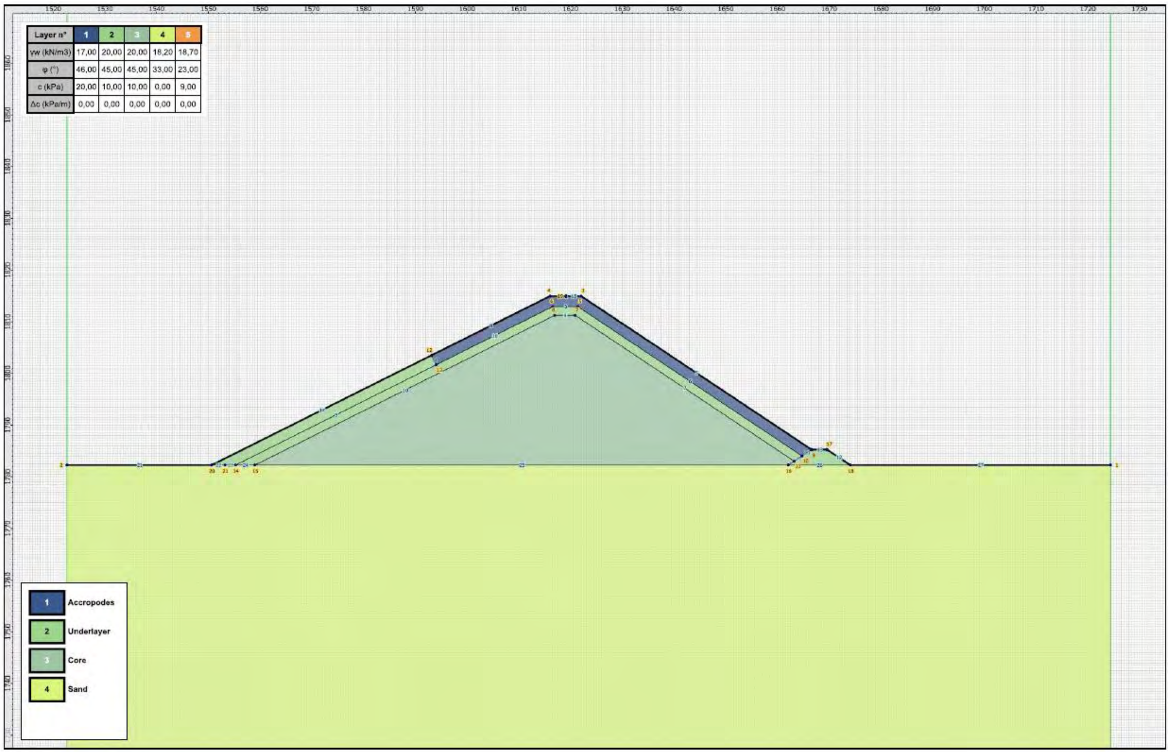


Figure 61 Slope stability

For the main breakwater, which in this case is considered, the underlying soil of the breakwater is coarse sand and the parameters for this layer is given in the provided table of different parameters for different soil layers. As already described in section 2.2, silty or clayish soil are the top layers and considered poor soils for foundation of the breakwater. For that reason, these layers are dredged and replaced with sand material as good as the coarse sand. The parameters are given in the below table.

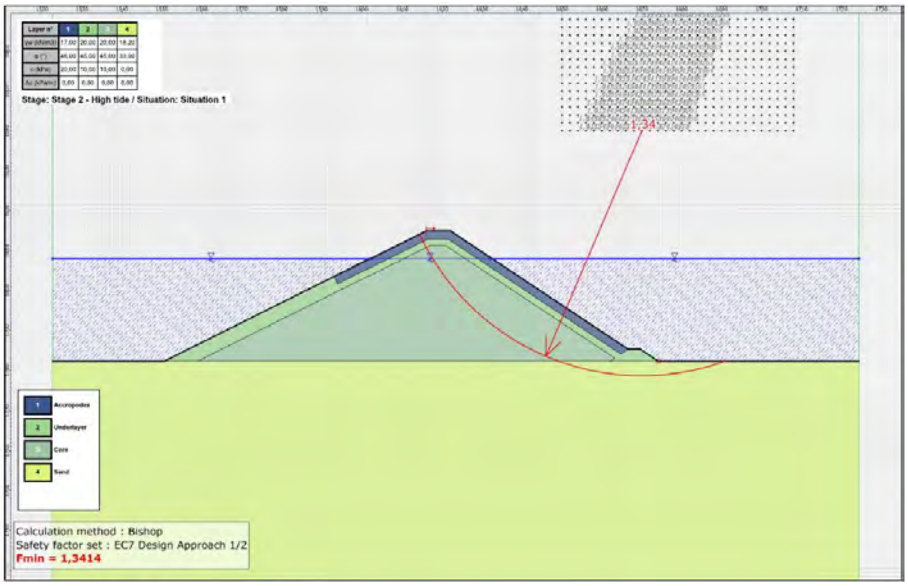
Layer	1	2	3	4
γ_w (kN/m3)*	17	20	20	18.1
φ' (°)	46	45	45	33
c' (kPa)	20	10	10	0

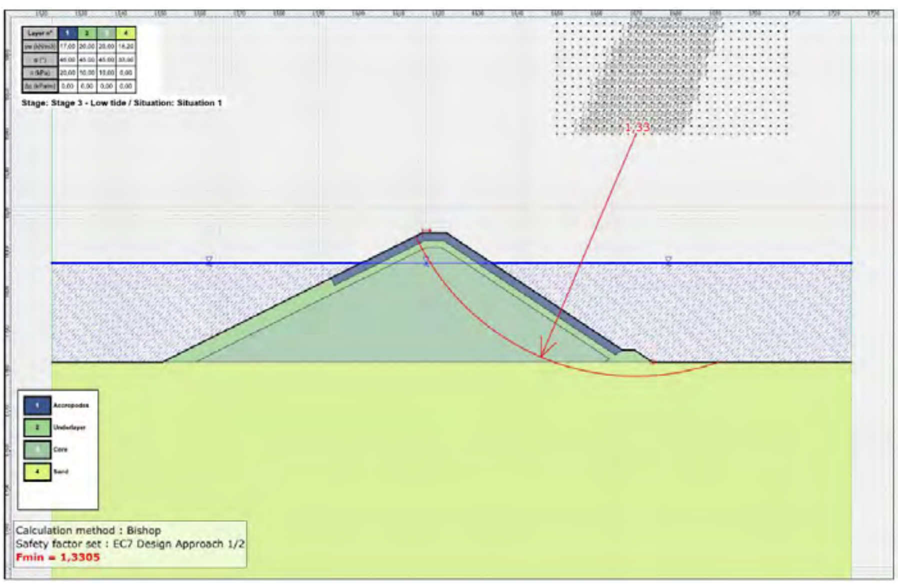
*Saturated soil unit weight

The partial safety factors used in the design are taken from Eurocode 7. For slope stability analysis, most of the partial safety factors are equal to those found when Ultimate Limit State is analyzed for drained situation. This is called ‘Design approach 1/2' in the program Talren. In Bishops simplified method the factor for variable actions, $\gamma_Q = 1.5$, and the factor of safety should be equal to or larger than 1.25. The partial safety factors are given in the below table.

γ_G	γ_φ	γ_Q
1.0	1.25	1.5

It is important to mention, that slope stability methods are usually based on static case (no external loads considered) with constant pore pressures. This will be case where the stability of slope on the seaside is analyzed for high and low tides. The results are illustrated in the below figures.





For both situations it can be seen, that the factor of safety in above 1.35 which means, that the slope of the breakwater is stable. The slip surface illustrated with red is important to consider for maintenance dredging. It can be seen for both situations, that the footing moves and for that reason it is important to consider the necessary distance from the breakwater when dredging in order not dredge away a part of the breakwater.

5.5 Quay design

Since the quay is located at the border between the coastline and the sea, the creation of a soil profile for the 0m depth line was necessary. The soil profile shown in Figure… was built basing on data for three boreholes: STDP8, SMQ46 and S14.

The following chapter presents a design of a container ship quay.

The quay walls were designed in KREA-V4 – software for modelling earth-retaining structures. It required a stratified soil profile; therefore the following assumptions for the soil were made:

- Clay was only found in borehole STDP8 and had depth of only 1m, therefore a clay layer was not assumed in the general profile
- the average depth for a specific soil type was taken, for example volcanic tuff has a 8 m wide layer, 0 m wide layer and 4 m wide layer -> average of 3 m layer was taken
- water table was assumed to be at 0m

The following soil layers were identified:

- from 0 to 2 m depth, the soil is coarse sand with D50 of 0.37mm
- from 2 to 20 m depth, the soil is fine sand
- from 20 to 25 m depth, coarse sand + alluvium
- from 25 to 28 m depth, volcanic tuff
- from 28 to 35 m depth, mudstone
- from 35m depth, solid rock

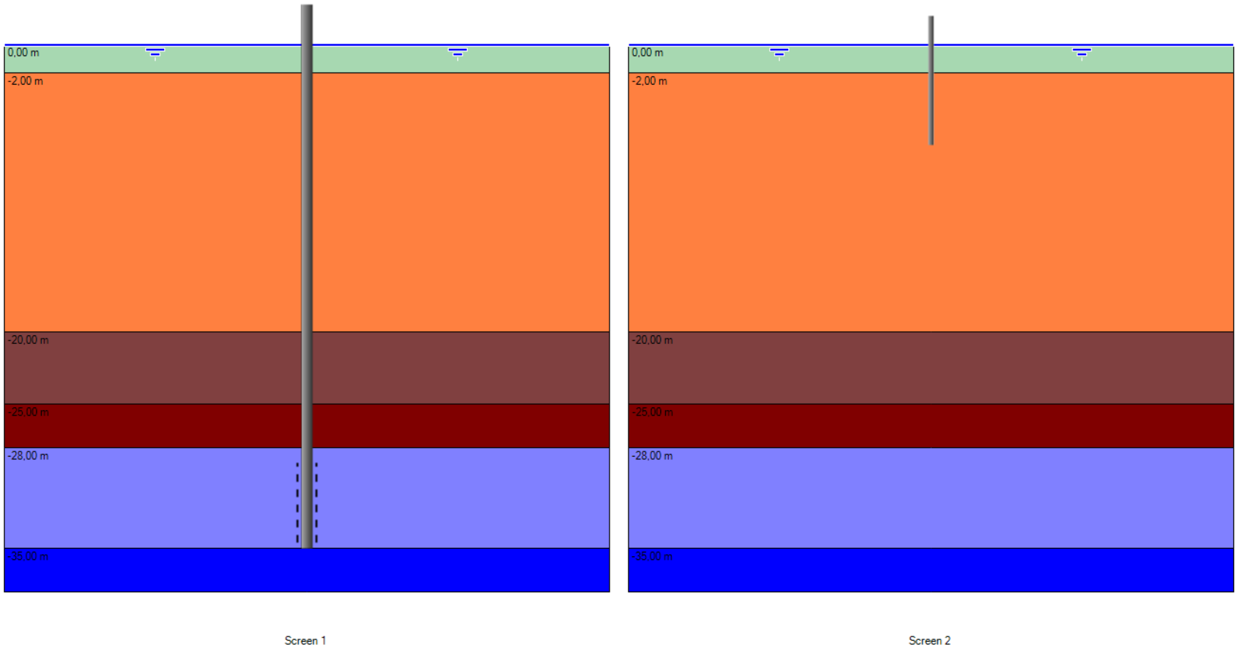


Figure 62 Soil profile for quay design

The simplified soil profile in Figure 62 was used for all further modelling.

Figure 63 shows the soil properties that were input into the model. Some characteristics were assumed based on a typical soil (for instance, the unit weight γ was taken from a soil databook).

Wall 1 | wall 2

Select the line to edit:

N°	Layers names	z [m]	γ [kN/m³]	γ' [kN/m³]	φ [°]	c [kN/m²]	dc [kN/m²/m]	k0	kaγ	kpγ	kd	kr
1	COARSE SAND	0,00	18,20	8,20	33,00	0,00	0,000	0,455	0,296	6,380	0,455	0,455
2	FINE SAND	-2,00	15,70	5,70	30,00	0,00	0,000	0,500	0,333	4,959	0,500	0,500
3	COARSE SAND AND ALLUVIUM	-20,00	20,00	10,00	35,00	0,00	0,000	0,426	0,271	7,301	0,426	0,426
4	VOLCANIC TUFF	-25,00	22,00	12,00	30,00	0,00	0,000	0,500	0,333	4,959	0,500	0,500
5	MUDSTONE	-28,00	20,00	10,00	30,00	50,00	0,000	0,500	0,333	4,959	0,500	0,500
6	ROCK	-35,00	20,00	10,00	45,00	10,00	0,000	0,293	0,172	17,191	0,293	0,293

Wall 1 | wall 2

Select the line to edit:

N°	Layers names	kac	kpc	kh [kN/m²/ml]	dkh [kN/m²/m/ml]	δa/φ	δp/φ	kaγ,min	pmax [kN/m/ml]
1	COARSE SAND	0,000	0,000	47736	0	0,000	-0,660	0,100	10000,00
2	FINE SAND	0,000	0,000	30805	0	0,000	-0,660	0,100	10000,00
3	COARSE SAND AND ALLUVIUM	0,000	0,000	14069	0	0,000	-0,660	0,100	10000,00
4	VOLCANIC TUFF	0,000	0,000	821338	0	0,000	-0,660	0,100	10000,00
5	MUDSTONE	1,155	6,271	263382	0	0,000	-0,660	0,100	10000,00
6	ROCK	0,828	13,864	0	0	0,000	-0,660	0,100	10000,00

Figure 63 Soil properties

ka, kp, k0 are obtained using the Rankine formula based on the Mohr-Colomb approach.

δa and δp are the angles between the lateral earth pressure and the normal of the wall

φ is the soil friction angle

$\frac{\delta a}{\varphi}$ was taken as 0, $\frac{\delta p}{\varphi}$ was taken as -0.66 because the wall is vertical (90 °)

kh was calculated by the software using the Schmitt formula which requires Em, α and (EI)

After the soil was input, a retaining wall with 2 sides was created in the software.

Wall 1 separates the sea water from the soil. It was selected to be a composite wall since it's much lighter as a structure, and therefore easier to install and maintain. A hollow circular steel section of diameter 2020 mm and thickness of 22 mm from the ArcelorMittal sections databook was selected for the piles. They are spaced at 4m centres. Between the piles, a steel wall was put.

The distance between the water and the quay was taken as 2.8m, therefore the top of the fill needs to be at 2.80 m above the water level. The pile length was taken as the maximum possible of 37.8 m, since a deeper pile will require drilling in the rock. Also, the rock level is approximate, so a deeper pile is not possible.

Wall 2 is used to stabilise the retaining wall. It is located at a distance of 35m from Wall 1. It is a sheet pile of a standard Z section from the ArcelorMittal sections databook (AZ 24-700). The sheet pile has a length of 10m, starting from 2m above ground level to -8m depth.

Phasing:

Initial Phases:

- Initial phase: Driving of the Front Wall Piles (-35m CD) + Sheetpiles (-29.1m CD) and driving of Anchor Wall Sheetpiles (-8m CD)
- Phase 1: set of tie rod (tyrant) at 0.5 CD
- Phase 2: Backfill from (0 CD) to (2.8 m CD) with fine sand
- Phase 3: Vibration of the first layer of soil in order to increase φ from 30 to 35°
- Phase 4: Caquot Live Load of 30 kN/m/ml
- Phase 5: Dredging from (-25.1 CD) to (0m CD)
- Phase 6: Hydrostatic action due to different water level sea side (0 m)

SLS

- Phase 7: Shiploader Live Load (PR): H= 25 kN (taken from Port Designer Databook)
- Phase 8: Bollard Live Load (AM): H= -73.6 kN/ml, M=36.8 kNm/ml at +2.8m CD

ULS (using approach 2 – EC7 – NF P94-282)

- Phase 9: Deactivation of (AM) and (PR) and adding Shipload Live Load H = 20 kN/ml
- Phase 10: Fenders Live Load (AC) H= 206 kN/ml

K-REA V4 was used to analysed the input data and the quay walls behavior under the given loads. The following results were obtained:

Wall 1:

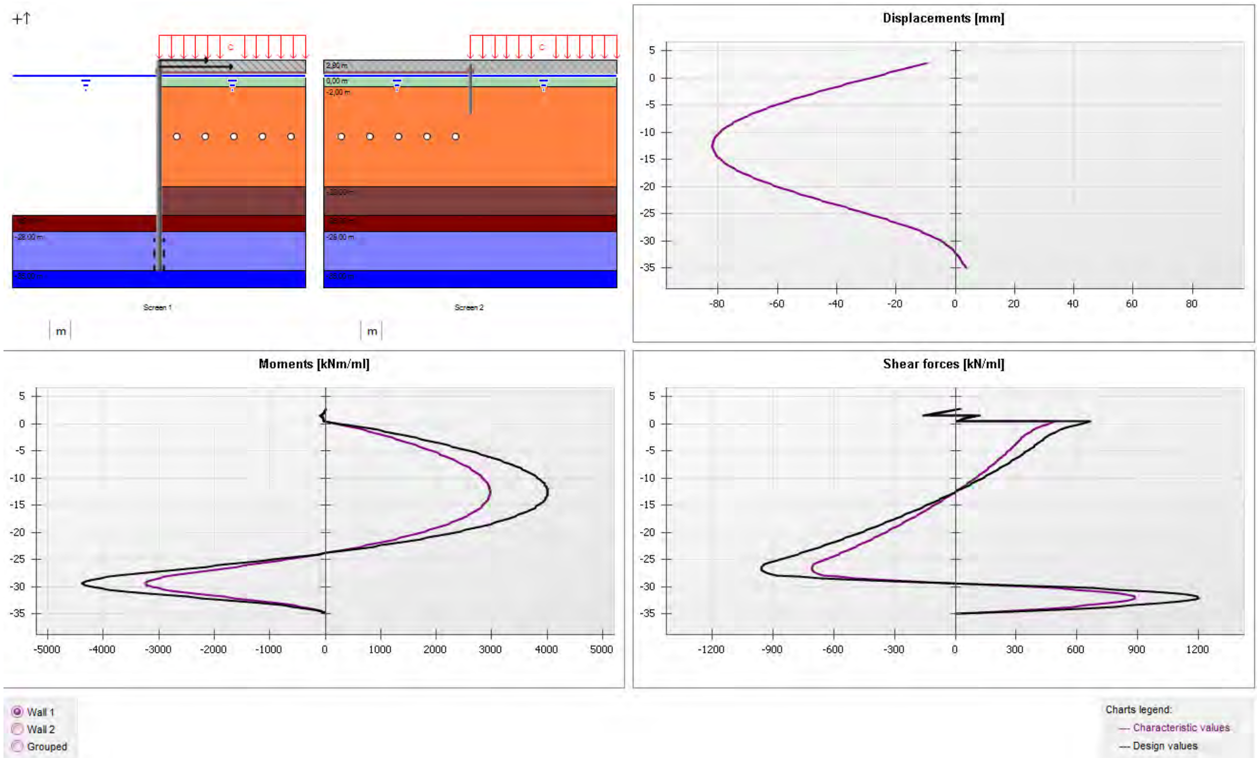


Figure 64 Front wall stability check

For SLS , the displacement needs to be checked against the maximum allowable displacement at the top. For Wall 1, the distance between the maximum bending moment in seabed and the top of the pile is 32m, therefore the maximum allowed displacement is 32 000/200 = 160 mm. The calculated displacement at the top of the pile is only 13 mm, which is less than the allowed value. Therefore, the pile design passes the checks.

Wall 2:

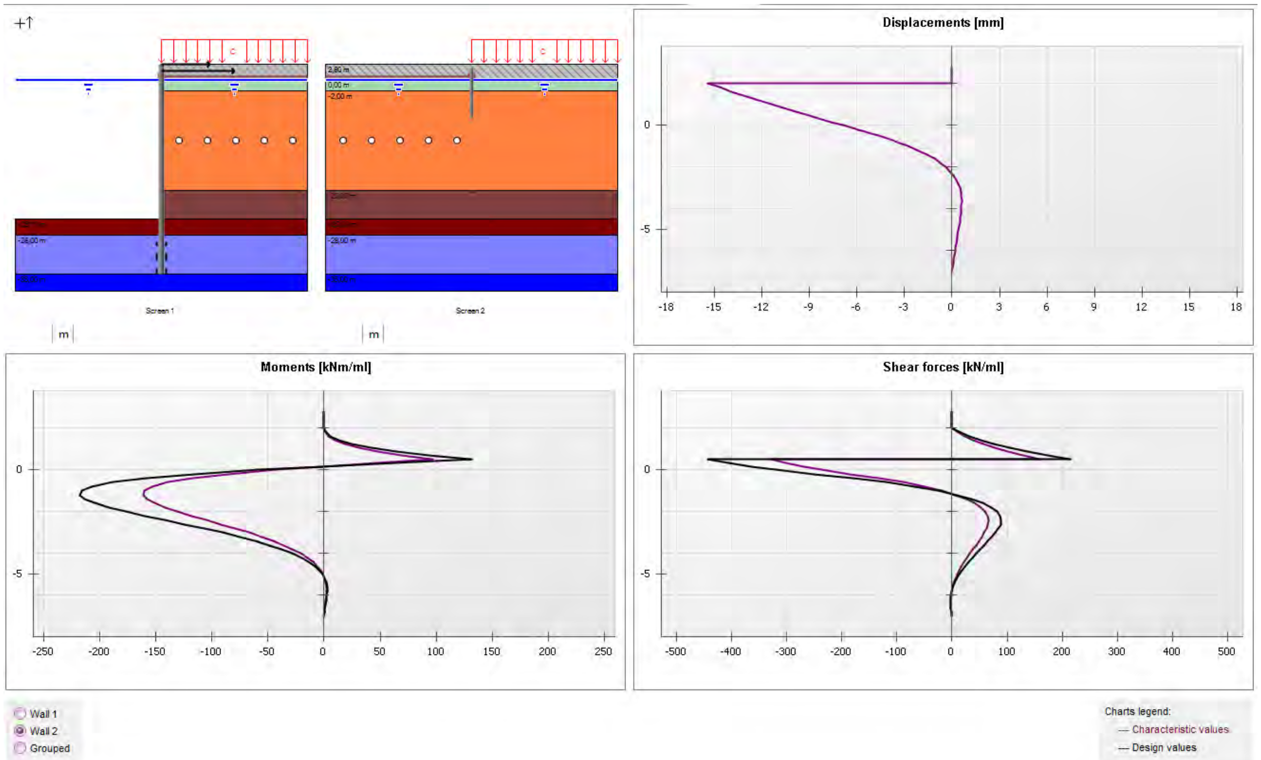


Figure 65 Back wall stability check

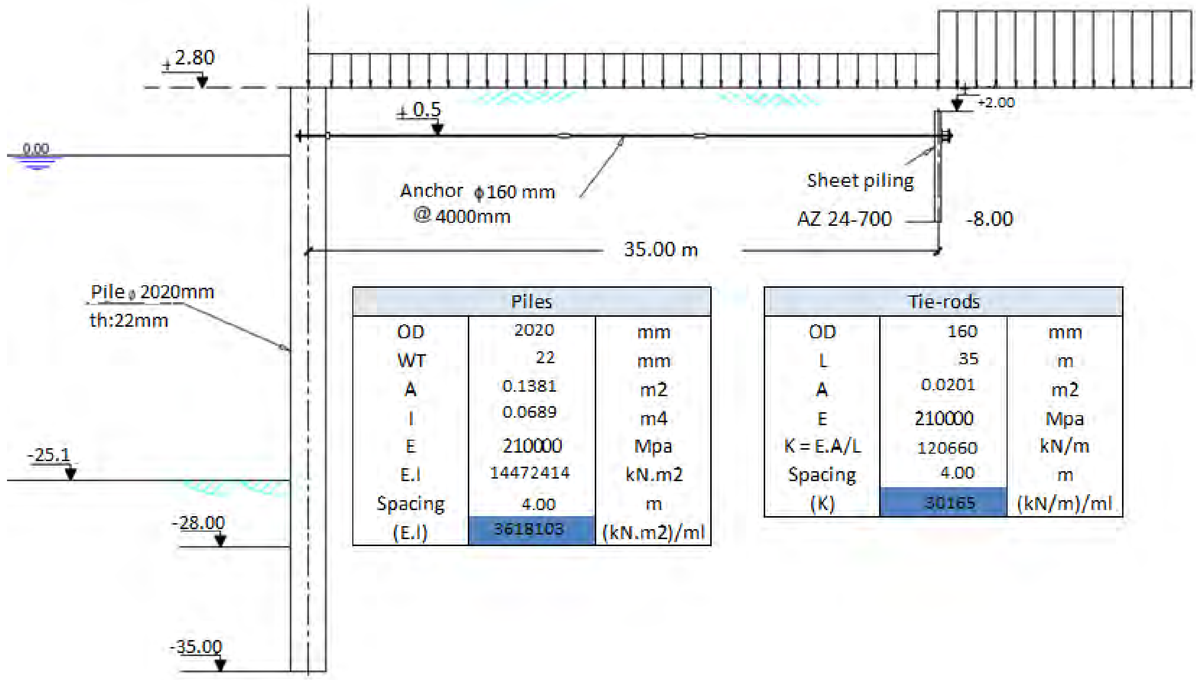
For SLS, the displacement needs to be checked against the maximum allowable displacement at the top. For Wall 2, the distance between the maximum bending moment in seabed and the top of the pile is 3.2 m, therefore the maximum allowed displacement is 3 200/200 = 16 mm. The calculated displacement at the top of the pile is 15 mm, which is less than the allowed value. Therefore, the pile design passes the checks.

Two ULS checks were performed:

- ULS Earth Pressure ratio
- ULS Kranz check

Wall 1 and 2 were checked against the two criteria, as well as an EC7 check. The results of these checks can be found in Appendix .. .

The final design was as shown in figure :



Piles		
OD	2020	mm
WT	22	mm
A	0.1381	m2
I	0.0689	m4
E	210000	Mpa
E.I	14472414	kN.m2
Spacing	4.00	m
(E.I)	3618103	(kN.m2)/ml

Tie-rods		
OD	160	mm
L	35	m
A	0.0201	m2
E	210000	Mpa
K = E.A/L	120660	kN/m
Spacing	4.00	m
(K)	30165	(kN/m)/ml

Figure 66 Quay design cross-section, side view

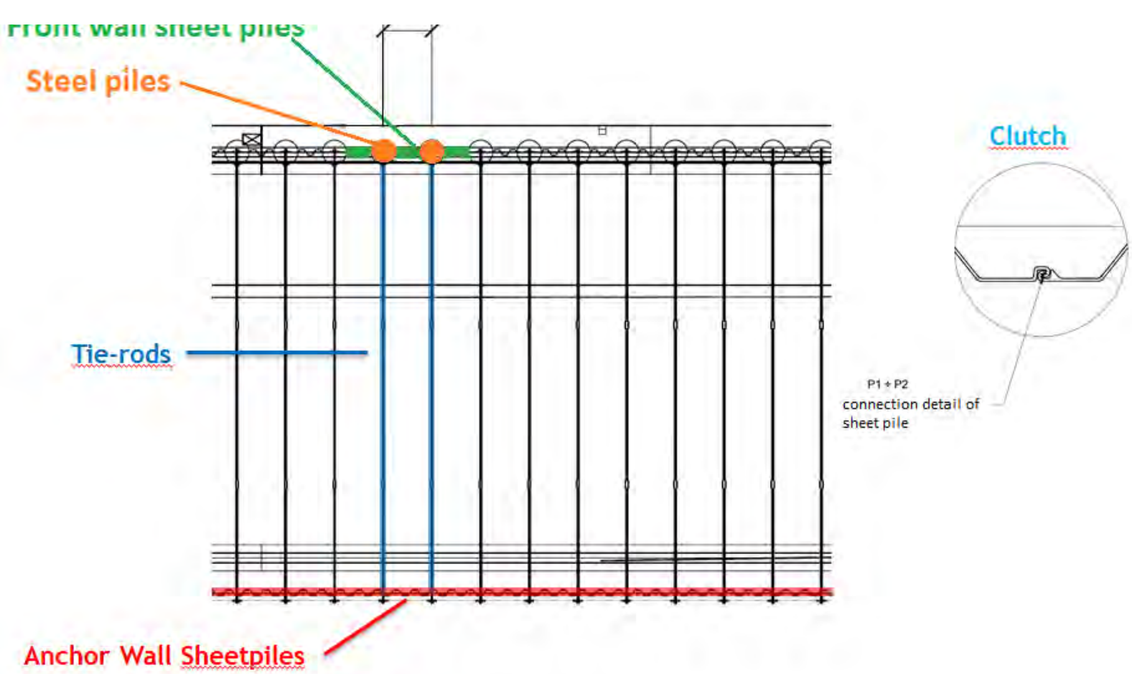


Figure 67 Quay wall, top view

5.6 Mooring dolphin

The port will contain two jetties providing breasting and mooring abilities for crude oil and product tankers, as required. Jetties will be designed to be connected to the south west breakwater. The location for breasting and mooring of crude oil and product tankers has been chosen for safety reasons – as further as possible from the other port facilities or ships and to minimize susceptibility to wave or current motions.

5.6.1 Distances of Breasting and Mooring Structures

Ships with the largest length, DWT, draft and smallest length were considered for defining the distance between breasting and mooring dolphins. Only the longest and shortest ships had influence to determination of dolphin distances. Figure 68 and 69 give a summary of vessels characteristics to be considered.

In order minimize structural costs, ideally same dolphin group (front and back dolphins) should be used for each design ship. This criterion was achieved for crude oil tankers since the middle breasting areas of shortest and longest ships were overlapping. Therefore, for all types of crude oil tankers only 2 breasting dolphins will be required with maximum required spacing of 106 m. However, as can be seen in Figure 69, in total 4 breasting dolphins will be required since dolphin arrangement for longer ships will have too large spacing for shorter ships to berth. The spacing between inner (smaller) dolphins is 39 m, while the distance between outer dolphins is 71 m. All dolphins will be placed symmetrically as shown in Figures 74 and 75.

Crude Oil Tankers (Mother Ships)					
Ship Characteristics	**Design Ships For Breasting and Mooring**				
	Universal Prime	**Lijmiliya**	**Cerigo**	**Eliza**	**Front Warrior**
Capacity (m^3)	333,317	172,399	332,321	333,332	170,364
Capacity (t) (DWT)	299,985.00	155,159.00	299,089.00	299,999.00	153,328.00
Displacement (t)	342,998.00	205,705.00	348,760.00	344,410.00	175,181.00
Draft (m)	22.20	13.70	22.00	21.50	17.50
LOA (m)	331.00	345.00	332.00	333.00	269.20
Breadth (m)	58.00	55.00	58.00	60.00	46.00
	(Deepest)	*(Longest)*	*(Heaviest)*	*(Heaviest)*	*(Shortest)*
0.3 LOA (m)	99.30	103.50	99.60	99.90	80.76
0.4 LOA (m)	132.40	138.00	132.80	133.20	107.68
Distance Between Outer Breasting Dolphins	106	106	106	106	106
	2 Breasting Dolphins Are Required				

Figure 68 Dolphin design 1

Product Tankers (Feeder Ships)				
Ship Characteristics	Design Ships For Breasting and Mooring			
	High Venture	Moray	Conger	Liquid Power
Capacity (m^3)	56,764	73,328	73,328	16,683
Capacity (t) (DWT)	51,088.00	65,995.00	65,995.00	15,015.00
Displacement (t)	29,942.00	44,067.00	44,067.00	9,488.00
Draft (m)	13.54	13.17	13.17	9.20
LOA (m)	183.00	228.60	228.00	129.00
Breadth (m)	32.20	32.20	32.20	22.00
	(Deepest)	(Longest/Heaviest)	(Heaviest)	(Shortest)
0.3 LOA (m)	54.90	68.58	68.40	38.70
0.4 LOA (m)	73.20	91.44	91.20	51.60
Distance Between Outer Breasting Dolphins	70.89	70.89	70.80	38.70
	4 Breasting Dolphins Are Required			

Figure 69 Dolphin design 2

5.6.2. Mooring Layout of dolphins for crude oil tankers

The jetty for the oil tankers consist of mooring and breasting dolphins. As can be seen in the drawings of the previous section the two jetties will differ in size. In this section an orientation of the dolphins is designed in such a way that the shortest (crude oil tankers) and the longest (crude oil tankers) vessel can moor on the same type of jetty. This can also be done for the product tankers in the same way.

First the distance between the breasting dolphins is determined. Looking at the different sizes of crude oil ships a distance of 106 m is sufficient because the 30%-40% areas overlap each other. This means that 2 breasting dolphins (fenders) are sufficient but for safety reasons it is possible to add a third one in the middle. For the product tankers 4 breasting are needed because not al areas overlap each other.

Secondly, the position of the mooring dolphins is determined. After looking at some reference jetties, the distance between the middle and outer dolphin is halfway the distance between the breasting and outer mooring dolphin (vertical distance in the drawing). The distances along the ship (Figure 74) is calculated by looking at the smallest ship.

A requirement according to the book (Vellinga) is that the angle of the bow and stern lines may not exceed 45 degrees ($\alpha < 45°$).

Step 1, assume a distance between the front of the smallest ship and the most right mooring dolphin of about 50 m; refer to Figure 70. The vertical distance in the drawing is 20 + 0,5 * 46 (width of smallest ship) = 43 m, together with the horizontal distance of 50 m the angle becomes 50° which is larger than 45°.

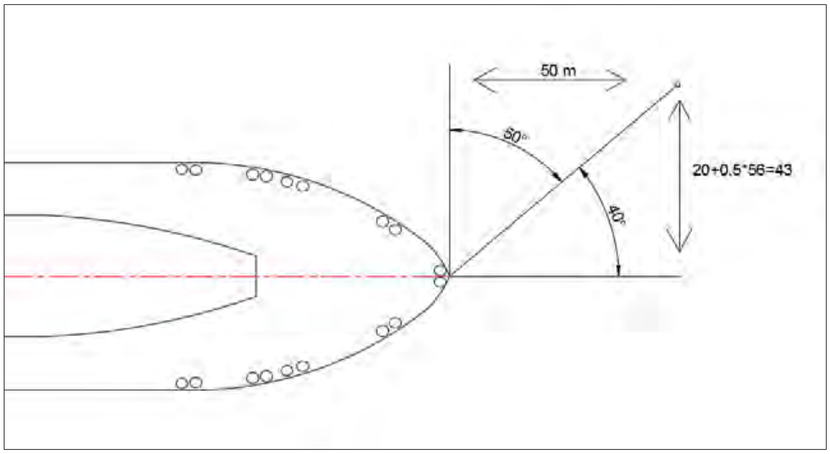


Figure 70 first try with a distance of 50 m

Step 2, now use a <u>horizontal distance of 40 m</u> so the mooring dolphin will be almost at the top front of the largest ship. The angle of the mooring lines for the smallest ship will now be <u>43° which is sufficient</u>.

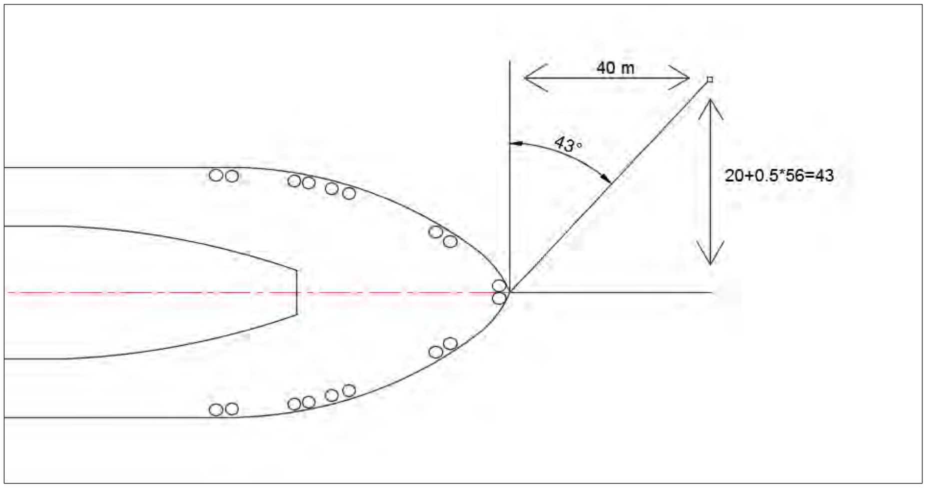


Figure 71 second try 40 m, sufficient

A requirement according to the book (Vellinga) is that the angle of the middle lines may not exceed 15 degrees: $\alpha < 15°$. Those lines are connected at about ¼ of the total length of the ship. The limits are set by the smallest ships so from the middle of the ship until the attachment point will be ¼ * 270 = 68 m so the total distance between the most right breasting dolphin and the middle mooring dolphin will be 80. This gives a horizontal distance of 80 – 68 = 12 m (see drawing). Together with the vertical distance of 40 m this will give an angle of 17° which is too large.

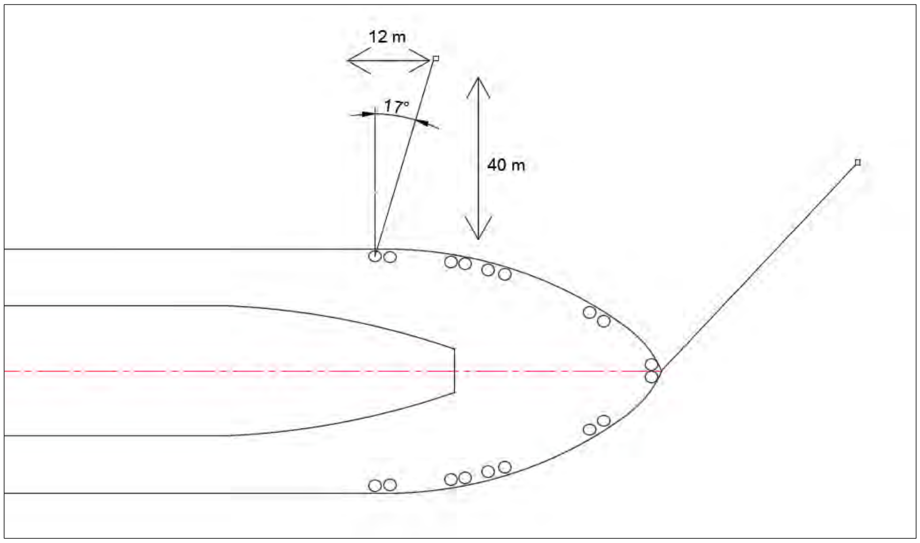


Figure 72 first try 12 m

Now the distance between the most right breasting dolphin and the middle mooring dolphin will be changed to 75 m instead of 80 m, which makes the horizontal distance 75 -68 = 7 m, as can be seen in Figure 73. Together with the vertical distance of 40 m this will give an angle of 10° which is sufficient.

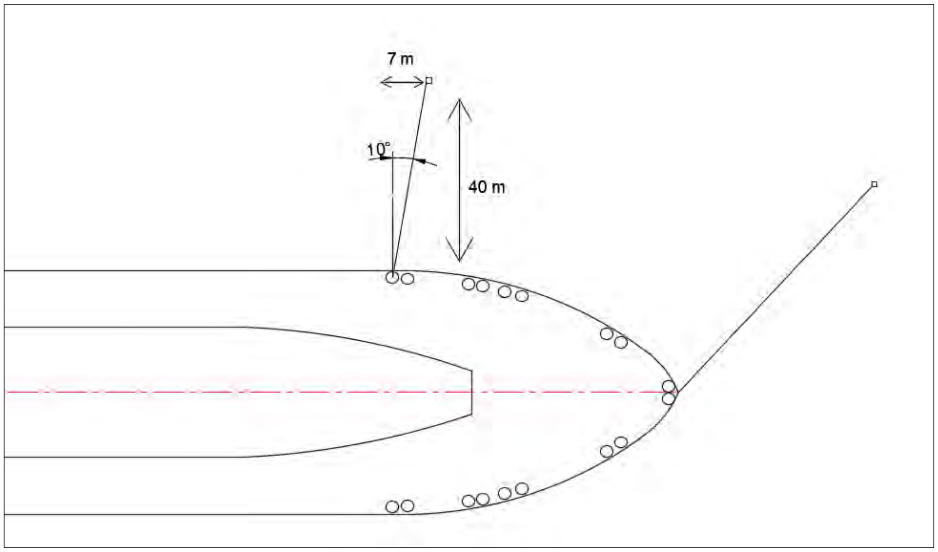


Figure 73 second try 7 m, sufficient

Those steps can also be done for the smaller product oil tankers. The result of the hand calculations above give a good estimate for the first design of the oil tanker jetty consisting of 6 (maybe 7 for safety reasons) dolphins.

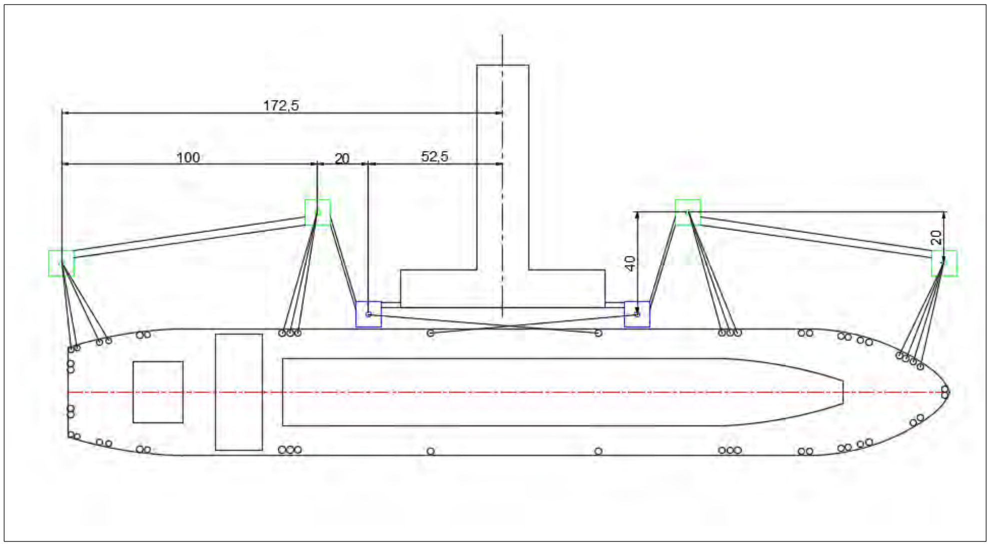


Figure 74 Total layout of crude oil jetty with a large ship

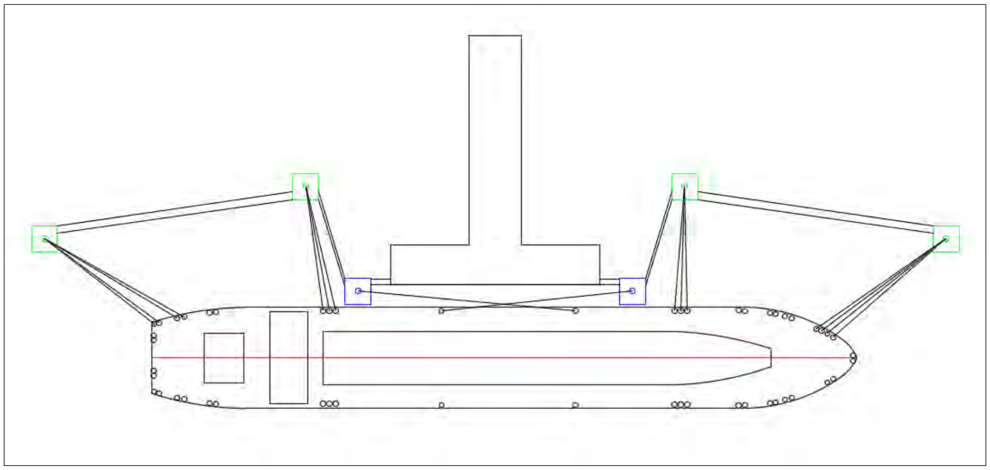


Figure 75 Total layout of the crude oil jetty with a small ship

5.6.3. Berthing Energy

In order to design fender and the actual breasting dolphins, the maximum induced ship breasting energy is required. To define the energy, same ships as above were analysed. As can be seen from Figure 76 and 77 show ship that have largest DWT or displacement produces the largest breasting energy that needs to be absorbed by fenders.

Main assumptions used:

- LBP (Length Between Perpendicular) is 20 m less than LOA (Overall Ship Length).

- Berthing angle is 10° to the parallel line of jetty.
- There is enough clearance around jetty and breakwaters for ship mooring.
- Berthing configuration for crude oil tankers – B. Mooring will occur in sheltered area by breakwaters. Larger DWT ships mainly depends on current. Almost no currents will be presented inside the port, therefore assuming B condition (Difficult berthing/sheltered). According to port breasting conditions, A configuration could be chosen. However, B configuration was selected for safety reasons.
- Berthing configuration for product tankers – C. Smaller DWT ships mainly depend on wind. Therefore assuming C condition (Easy berthing/exposed). Similar as for above, B berthing condition could be selected for product tankers. However, C is chosen for safety reasons.
- Berthing case – third-point berthing (0.3% of LOA is the middle ship breasting length).
- Softness coefficient – C_s = 0.95. In between soft and hard fender.
- Berth configuration – open structures (jetty) and berthing angles >5°. Therefore, taking C_c = 1.
- Taking F_s = 1.25 for large tankers and 1.75 for smaller tankers (According to PIANC).
- The average temperature in port location is 30°C.

For more detailed design, ship breasting should be evaluated more accurately than using Brolsma table. Especially, smaller tankers, that are more dependent on wind, should be analysed using computer aided systems.

Figure 76 indicates the approach to the breasting dolphin.

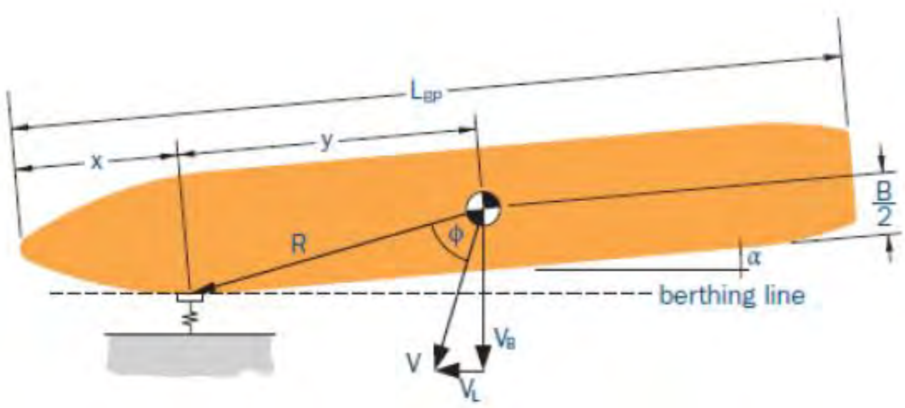


Figure 76 approach

Crude Oil Tankers					
		Lijmiliya	Front Warrior	Eliza	Universal Prime
		Longest	Shortest	Largest DWT	Largest Draft
Water Density	**(t/m3)**	1.025	1.025	1.025	1.025
DWT	**t**	153,328	153,328	299,999	299,985
Capacity	**m3**	172,399	170,364	333,332	333,317
Displacememt	**Md (t)**	205,705	175,181	344,410	342,998
Overall Length	**LOA (m)**	345	269.2	333	331
Length Between Perpendicular	**LBP (m)**	331.2	258.432	319.68	317.76
Max Width	**B (m)**	55	46	60	58
Draft	**Te (m)**	13.7	17.5	21.5	22.2
Mini Water Depth	**Wd (m)**	24.1	24.1	24.1	24.1
Clearance	**Kc (m)**	10.4	6.6	2.6	1.9
Bloc Coefficient	**Cb**	0.80	0.82	0.81	0.82
Radius of Giration	**k (m)**	90.66	71.63	88.18	87.85
Angle Ship/Quay	**alpha (°)**	10.00	10.00	10.00	10.00
Berthing Configuration	**Broslma**	B	B	B	B
Broslma Berthing Velocity	**Vb (m/s)**	0.07	0.07	0.05	0.05
Design Berthing Velocity	**Vb (m/s)**	0.07	0.07	0.05	0.05
Added Mass Coefficient	**Cm**	1.36	1.52	2.19	2.60
Berthing Point of Impact	**(% LBP)**	0.34	0.30	0.34	0.33
Bething Point of Impact	**x (m)**	113.10	76.72	107.34	106.38
	y (m)	52.50	52.50	52.50	52.50
	R	59.27	57.32	60.47	59.98
	gamma (°)	52.35	56.34	50.26	51.08
Eccentricity Coefficient	**Ce**	0.81	0.73	0.81	0.81
Softness Coefficient	**Cs**	0.95	0.95	0.95	0.95
Berth Configuration Coefficient	**Cc**	1.00	1.00	1.00	1.00
Normal Berthing Energy	**En (MNm)**	0.53	0.48	0.79	0.93
Abnormal Berthing Factor	**Fs**	1.75	1.75	1.25	1.25
Abnormal Berthing Energy	**Ea (MNm)**	0.92	0.85	0.98	1.16
Abnormal Berthing Energy	**Ea (MNm)**	1.05	0.96	1.12	1.31

Figure 77 Dolphin berthing energy

Figure 77 indicates that the maximum energy of 1.31 MNm at crude oil dolphins is induced by Universal Prime having 300,000 DWT. The maximum energy value will be used for the dolphin design.

Product Tankers				
		Moray	Liquid Power	High Venture
		Longest/Heaviest	Shortest	Largest Draft
Water Density	**(t/m3)**	1.025	1.025	1.025
DWT	**t**	65,995.00	15,015.00	51,088.00
Capacity	**m3**	73,328	16,683	56,764
Displacememt	**Md (t)**	44,067.00	9,488.00	29,942.00
Overall Length	**LOA (m)**	228.60	129.00	183.00
Length Between Perpendicular	**LBP (m)**	219.456	123.84	175.68
Max Width	**B (m)**	32.2	22	32.2
Draft	**Te (m)**	13.17	9.2	13.54
Mini Water Depth	**Wd (m)**	24.1	24.1	24.1
Clearance	**Kc (m)**	10.93	14.9	10.56
Bloc Coefficient	**Cb**	0.46	0.37	0.38
Radius of Giration	**k (m)**	45.21	23.24	33.39
Angle Ship/Quay	**alpha (°)**	10.00	10.00	10.00
Berthing Configuration	**Broslma**	C	C	C
Broslma Berthing Velocity	**Vb (m/s)**	0.15	0.26	0.16
Design Berthing Velocity	**Vb (m/s)**	0.15	0.26	0.16
Added Mass Coefficient	**Cm**	1.34	1.27	1.35
Berthing Point of Impact	**(% LBP)**	0.26	0.08	0.20
Bething Point of Impact	**x (m)**	57.23	9.42	35.34
	y (m)	52.50	52.50	52.50
	R	54.91	53.64	54.91
	gamma (°)	62.95	68.17	62.95
Eccentricity Coefficient	**Ce**	0.53	0.27	0.42
Softness Coefficient	**Cs**	0.95	0.95	0.95
Berth Configuration Coefficient	**Cc**	1.00	1.00	1.00
Normal Berthing Energy	**En (MNm)**	0.33	0.10	0.22
Abnormal Berthing Factor	**Fs**	1.75	1.75	1.75
Abnormal Berthing Energy	**Ea (MNm)**	0.58	0.18	0.38
Abnormal Berthing Energy	**Ea (MNm)**	0.66	0.21	0.43

Figure 78 Maximum energy

Figure 78 indicates that the maximum energy of 0.65 MNm at product oil dolphins is induced by Moray having 65,000 DWT. This maximum value will also be used in designing dolphins.

5.6.4. Fender Design

Each fender for both crude oil and product oil tanker berthing has been designed to absorb the maximum induced energy. The fender selection is based on PIANC guidelines. The width of each fender was assumed to be 2 m. The height has been calculated according to assumed allowable hull pressure, fender reaction force and assumed width. The final height of the fender has been factored by 1.1 for manufacturer requirements.

Maximum induced energy	Ea (MNm)	1.31
Fender Type		SCN1400/E2.1
Absorbed Energy by Fender	Er (MNm)	1.34
Total Fender Reaction	Rr (MN)	1.85
Allowable Hull Pressure	Pr (MN/m2)	0.15
Fender Width	W (m)	2
Fender Height	H (m)	6.2
Factore Fender Height	H (m)	6.8

Figure 79 Crude oil tankers

Maximum induced energy	Ea (MNm)	0.66
Fender Type		SCN1200/E2.2
Absorbed Energy by Fender	Er (MNm)	0.67
Total Fender Reaction	Rr (MN)	1.17
Allowable Hull Pressure	Pr (MN/m2)	0.3
Fender Width	W (m)	2
Fender Height	H (m)	2.0
Factore Fender Height	H (m)	2.1

Figure 80 Product tankers

5.6.5. Structural Design of Breasting Dolphins

Breasting dolphin has been chosen to be flexible – cylindrical steel monopile. This specific choice was made because flexible dolphins are more desirable for large vessel mooring, which is the case for designing jetty in the discussing port. Moreover, flexible dolphins are more efficient in deep waters since the longer pile length increases energy absorption capacity. This would minimize the cost of fenders because less soft fenders will be required. Furthermore, flexible dolphins are less susceptible to damage comparing to rigid dolphins. This would also eliminate the need of extra dolphins for safety reasons.

The size of mooring dolphins will be the same as breasting dolphins because mooring dolphins exhibit less loads than breasting dolphins. In order to estimate more accurately the size of mooring dolphins and minimize costs, detailed analysis of mooring dolphins should be conducted.

Main assumptions used for structural breasting dolphin design:

- Load exerted by lines is equal to 150 kN.
- Thickness of piles is 60 cm.
- The total bending moment is induced by line loads and breasting force, which is taken to be equivalent to fender reaction force.
- The maximum bending moment depth is $2.5D_0$ under seabed surface.
- All 4 dolphins for product tankers will be the same size. Although shorter product tankers will induce less energy to inner dolphins, it was assumed that it will be economically feasible to install and have all 4 dolphins the same size.

Crude Oil Tankers		
Pile Dimensions/Structural Characteristics		
D_0 (m)	2.90	(diameter)
t (m)	0.06	(thickness)
L_e (m)	34.3	(unbraced/effective pile length)
A (m^2)	0.54	(cross-section area)
Material properties		
F_y (MPa)	450	(steel yield strength of steel)
E (MPa)	210,000	(elastic modulus of steel)
Structural Loads		
N (kN)	150	(load of lines)
Rr (kN)	1,850	(breasting Load)
M (kNm)	68,681	(total bending moment)

Figure 81 Pile loads

Product Tankers		
Pile Dimensions/Structural Characteristics		
D_0 (m)	2.300	(diameter)
t (m)	0.060	(thickness)
L_e (m)	32.841	(unbraced/effective pile length)
A (m^2)	0.422	(cross-section area)
Material properties		
F_y (MPa)	450	(steel yield strength of steel)
E (MPa)	210,000	(elastic modulus of steel)
Structural Loads		
N (kN)	150	(load of lines)
Rr (kN)	1,170	(breasting Load)
M (kNm)	43,350	(bending moment)

Figure 82 Pile loads product

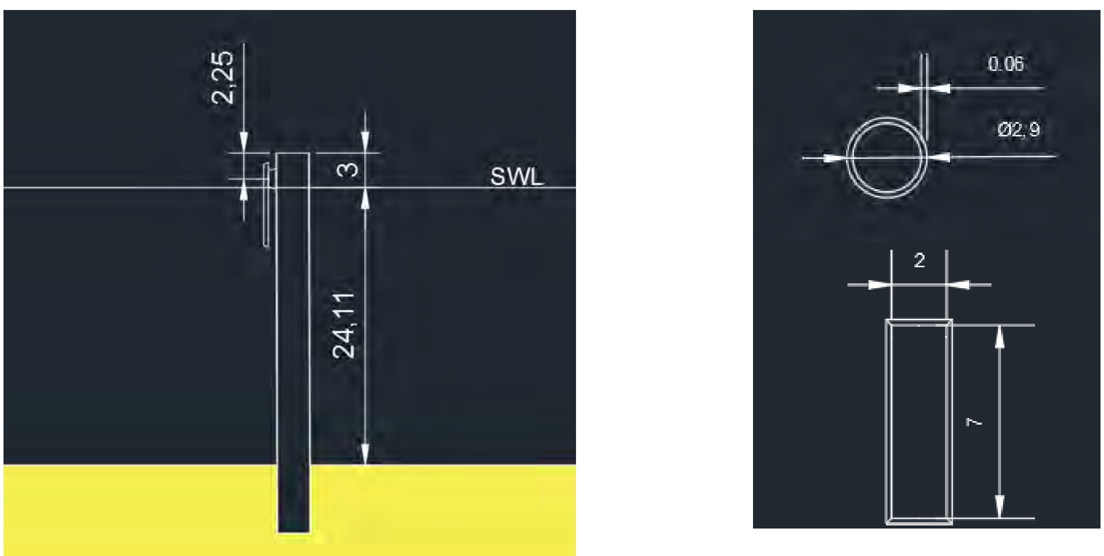


Figure 83 : The design of monopile and fender for crude oil tankers

Figure 84 The design of monopile and fender for product tankers

5.7 Verification of the design

The final section of the main breakwater was verified in a flume model test. The determined breakwater parameters were scaled through Froude scaling since the governing forces are related to inertia. Froude scaling table illustrated below is used for the different parameters. The weight of accropodes in the laboratory is known to 42.5 gr. In addition, since the real weight of accropodes is known the scaling factor can be determined. The value of the scaling factor is $\lambda = 60$. The remaining parameters are then scaled after these values.

Physical Parameter	Unit	Multiplication factor
Length	[m]	λ
Structural mass:	[kg]	$\lambda^3 \cdot \rho_F / \rho_M$
Force:	[N]	$\lambda^3 \cdot \rho_F / \rho_M$
Moment:	[Nm]	$\lambda^4 \cdot \rho_F / \rho_M$
Acceleration:	[m/s^2]	$a_F = a_M$
Time:	[s]	$\sqrt{\lambda}$
Pressure:	[Pa=N/m^2]	$\lambda \cdot \rho_F / \rho_M$

For the first experimental test conducted in wave flume with the scaled 100-year return period for the wave height resulted in displacement of stones in both the armour layer and the toe. Due to insufficient amount of accropodes the armour layer of the breakwater model consists of both Accropodes and Corelocs. This is illustrated in the below figure.

After the test, following displacement of different blocks was observed:

- 2 corelocs
- 3 accropodes
- 4 toe units

Furthermore, the corelocs had after the test moved somewhat significantly. This can be seen in the figure below.

In the second test the package density was considered especially since corelocs in the armour layer during the first test had moved significantly. More corelocs were placed in order to have a denser a armour layer and hence follow the recommendation for the package density. In the right figure the breakwater before test 2 is shown. After the test following observations were made:

- Displacement of one accropodes
- Displacement of 2 toe units

From the observations it can be noticed, that the armour layer of the breakwater during the second test was denser.

5.8 Construction sequence

For our port, we will have this specific construction sequences:

1. Preparation of the work, make every implementation plan, detailed planning and also do every grid for general Earthwork, dredging, quays (piles, sheet piles and tie-rods).
2. Site installation, achievement of the approach road, preparation of all the storage area.
3. We will start the construction by dredging the deeper part with one hopper dredger and one cutter dredger. In the same time, we will remove the clay of the future breakwater section. A backhoe dredger will join after 2-3 weeks to do the dredging as describe on the dredging method.
4. Backfilling the breakwater section with good sand. We will make a little hill on the section to compact the sand.
5. In the same time than phase 3., we will start the general Earthwork on land to prepare our platforms.
6. 6-7 weeks after the end of phase 4., we will do the sea bed slope for the breakwater. We will use the cutter dredge due to the very high precision of this one.
7. Almost on the same time that the phase 6., we will start to build our breakwater as describe on the breakwater sequences.
8. At the end of phase 7., we will be able to start the quays, also as describe on the quays sequences before.
9. When phase 5. will be done, we will start to do the two layer, starting by the land going to the quay. This way, we will be able to start this phase in the same schedule than the preceding's one. This phase will finish after the previous phase because we cannot finish the layer without the quays.
10. Still in the same timing, we will be able to build the mooring and breasting dolphin. After this, we will build the platforms
11. When phase 9. and 10. will be done, we will build the jetty between the platforms for oil tankers and the land.
12. After finishing phase 9., we will be able to do the three hot bituminous mixture layer.

In the same schedule, we will start to put all our protection equipment, QMRH on the mooring dolphin and fenders on the quays and the breasting dolphin.

Detailed construction sequence can be found in Appendix Q.

5.9 Economic analysis

For the economic analysis of our layout, we made the cost of the all project and not just the breakwater and the dredging as we have done before to choose our layout.

We have divided our cost in different part:

- Breakwater Cost
- Dredging Cost
- Quays Cost
- Trestle Cost
- Protection Equipment Cost
- Jetty Cost
- Platforms Cost (Optional : Based on assumption)

5.9.1. Breakwater Cost

To do the breakwater cost, we have used the Unit Cost given in the Scope of Work.

In this file, we have an average cost according to the depth. Consequently, we calculated the length of the Breakwater at each depth. We started at – 5 m ZH and we finished at the end of our two Breakwater, at – 25 m ZH for the main one and at – 17 m ZH for the second one.

After, we just multiplied the length of the breakwater for every depth by the unit cost of a rubble mound according to this depth, which give us the Breakwater cost : 266 858 500,00 €

5.9.2. Dredging Cost

For the dredging cost, we have used the Autocad file, and we did different area of dredging:

- Between 0 m ZH and -5 m ZH,
- Between -5 m ZH to -10 m ZH,
- Between -10 m ZH to -15 m ZH,
- Between -15 m ZH to -20 m ZH,
- Between -20 m ZH to -25 m ZH.

After, we assume that we will have to dredge from an average value for each area. For example, between 0 m ZH and – 5 m ZH, we assume that we will start to dredge at the average value of -2.5 m ZH. Consequently, for the deep part, we will have to dredge from -2.5 m ZH to -24.1 m ZH, for the shallow part, from -2.5 m ZH to -13.5 m ZH and for the intermediate part, from -2.5 m ZH to -18 m ZH.

Here is all the average value we took based on the AutoCad file:

Depth (m ZH)	Average depth
0 m ZH to – 5 m ZH	-2.5 m ZH
-5 m ZH to – 10 m ZH	-7.5 M ZH
-10 m ZH to – 15 m ZH	-12.5 m ZH
-15 m ZH to – 20 m ZH	-18.5 m ZH
-20 m ZH to – 25 m ZH	-22 m ZH

After this, to do the amount of dredging of the different area, we had to do some assumption in the soil:

- Between 0 m ZH and -5 m ZH, we have to dredge in sand until -22 m ZH and after in we Volcanic Tuff (Soft Rocks) for 2 meters.
- Between -5 m ZH and -15 m ZH, we have just sand to dredge.
- Between -15 m ZH to -20 m ZH, we have sand until – 19.5 m ZH and after we have soft clay
- After – 20 m ZH, we have soft clay.

Based on this depth and the soil, we made our volume of dredging (Appendix K).

Furthermore, we have to dredge for our breakwater: After -19 m ZH, there is clay everywhere. Consequently, before doing our breakwater, we have to dredge all this clay at a depth of 10 m. We will also have to backfill after with some sand or Volcanic Tuff. You can find the cost in our final cost estimation.

Our dredging cost is 271 083 233 €.

5.9.3. Quays Cost

Due to the position of our quays, mostly at 0 m ZH, we don't have the average price of them. Consequently, we assumed that each meter will cost 10 000 €, which give us a final cost of 25 300 000 €

5.9.4. Trestles Cost

We have one Trestle going from the quays to the Oil Terminal. To calculate the cost, we did the same method we used for the Breakwater. For each depth, we calculated the length and we multiplied it by the unit cost according to this depth, which give us our Trestles cost: 9 935 000 €.

5.9.5. Protection Equipment

We have two different kind of protection equipment:

- QMRH (Quick Release Mooring Hook), used for the mooring dolphin. We have 8 Mooring dolphins, so there is 8 QMRH.
- Fenders : We decided to put one Fender each 50 meters

With this 2 different equipment, we obtain a cost of 9 340 000 €.

5.9.6. Jetty

For the Jetty cost, we just need the number of Mooring and Berthing Dolphins. We have 8 mooring dolphins and 4 berthing dolphins. We also have 2 platforms, one for each terminal and we have a walkways starting to the platforms and going to land. We finally have a jetty cost estimation of 42 875 000 €

5.9.7. Total Cost Estimation

With all this different cost, we can calculate our cost estimation which is 625 391 733 € (Appendix S). We think this is quite a realistic cost and it is a verification for our layout which make us confident of the design.

5.9.8. Optional Cost

In our cost estimation, we also made an estimation of the cost of the different platforms. To do it, we used the total area we had: 155 Ha (1 550 000 M2).

According to the geotechnical profile in the sea, we made the assumption that the soil is not too bad but because we need a highly resistant platform, we will to two Layer of materials treated with hydraulic binder.

We can also see in these profile that the ground level is going up and we made the assumption that the average ground level is at +6 m ZH. According to our two layer of 35 cm thickness and our quay level at + 2.80 m, we will have to excavate an average of 3.9 m per square meter, which give us a quantity of 6 045 000 M3 to excavate.

We made two layer of 35 cm because the engine you used to treat the soil can treat 35 cm at the maximum.

For the asphalt concrete structure, we have the assumption that we need 3 different layers:

- One base layer composed of specific hot bituminous mixture. It will be a specific one to prevent the rutting. The thickness will be 10 cm.
- One binder course compose with the same specific hot bituminous mixture. Same Thickness.
- One surface course compose with hot bituminous mixture like EB 0/10 roul/laison in France.

We based our price with French prices you can find around Caen.

- D21 materials = 30 €/M3: Because of the huge quantity, the carriage cost will be expensive, that's why the price is quite high.
- Hydraulic binder = 300 €/Ton so with 4%, we will have 12 €/M3 + the carriage price : 3 €/M3 of treated material = 15 €/M3

5.10. Environmental and Social impact assessment

The impact assessment of the project was divided into 3 sections: positive, environmental and social, to discuss the different impacts of the project and possible management strategies.

5.10.1. Positive impact

The project is part of the Royal Development Initiative for the Oriental region, which is lagging behind the rest of Morocco in several socio-economic parameters, including the level of poverty and vulnerability. It was predicted by NWM that the port and the corresponding free zone will create around 100,000 work positions in the long term. The construction phase itself is expected to employ around 2,500 workers, and another 1,700 could be employed for continuous port operation. Therefore, the project has a large positive impact on the region, as it brings new foreign investments and creates job opportunities for local low educated people.

5.10.2. Environmental impact

After a sensitivity analysis of all risks associated with the port activities, the key environmental and social issues were divided into 3 main sections: construction, operation and dredging as shown in Figure 85.

Type	Risk	Mitigation measures
Construction	Reforested dunes needs to be cleared, so terrestrial flora and fauna will be affected	Avoid clearing beyond the marked parts; After construction re-vegitate the area using plants
	Increased turbidity in the sea due to dredging and disposal of dredged material	Use dredging techniques which minimize the turbidity; monitor the turbidity
	Degradation of water quality and affected marine flora and fauna	Monitor physical and chemical water quality
	Health and safety of workers	Ensure that work doesn't present any risks to the works and nearby population
	Safety of residents during the material transportation phase	Use appropriate signaling form to inform road users
	Acoustic impact on marine wildlife and population	Maintain transport vehicles in good working order to minimize exhaust gases and noise
	Improper disposal of construction waste	Ensure the waste is disposed at the correct specified location with minimal impact on the marine wildlife
Operation	Increased maritime traffic can influence the safety and productivity of fishermen	Limit traffic to the designated footprint area
	Improper dredging and disposal of dredged material during the operational phase	An area at the tip of the Baie de Betoya, SW of the port site, was identified which allows the safe disposal of the the material
	Safety of residents during the material transportation phase	Ensure the safety of occupants in the bordering work area by providing a fence and monitoring
	Potential impact of oil spills, especially in the nearby Ramsar site	All fuel and oil handling should be performed under constant surveillance to prevent spilling
	Environmental impact and activities associated with the industrial activities in the free zone	Provide air quality monitoring stations; develop a dust management programme
Dredging	Disposal of the material from dredging of the harbor to create an access channel with the required depth (up to 24 m) and establishing the breakwater foundations	A portion of the dredged material with good quality can be reused for embankment of terminals
		An area at the tip of the Baie de Betoya, SW of the port site, was identified which allows the safe disposal of the the material

Figure 85 Environmental impact

5.10.3 Socio-economical

The project has been planned to avoid or minimize the impacts on people and property. The selected site results in no displacement, and most of the acquired land is uncultivated. The western limits of the free zone need to be adapted to avoid impact on the primary residents in the populated areas of Iaazzanene. The port will affect around 30 property owners, who need to be compensated. The free zone construction will affect around 150 property owners. Finally, around 100 fishermen will be affected by the construction and operation of the port, since their main fishing location has been transformed into a port. They will be moved into a new location near the Chamlalla beach landing.

It is important to note that no permanent residents are affected by the project. The tourists of the nearby El Bouyaffer Kallat may be potentially affected by noise, dust and visual impacts during construction and operation, however for these citizens the impact is minimal and temporary.

CHAPTER 6

CONCLUSIONS

6. Conclusions and Recommendations

1. Conclusions

The combination of the design conditions and the design criteria at the Baie de Betoya provide a unique combination of opportunity and difficulty. The selected port location tries to balance the most favourable geotechnical conditions together with the least amount of accumulated sediment. The added design condition of the port entrance from the SW direction, which is also a direction of the incoming waves, added difficulty to the design. However, the design team overcame this obstacle by carefully designing two breakwaters to protect the port from the incoming high waves.

The design was completed with a design of the container quay cross-section, as well as a breasting/mooring dolphin section. These are representative for their respective areas, however they do not represent values for all quays. The construction sequence indicates that the project requires careful long-term planning to complete the port on time. However, with good management, the port can become a beneficial part for the Nador region.

2. Future expansion and recommendations

Looking at the future, the amount of containers will probably increase. In our modified layout the container terminal can expand landward and there are already five berths. The size of the ships will increase, but this increase can be solved by placing the cranes at the berth of the bigger ship. These cranes can be replaced on rails and therefore the quay productivity is flexible.

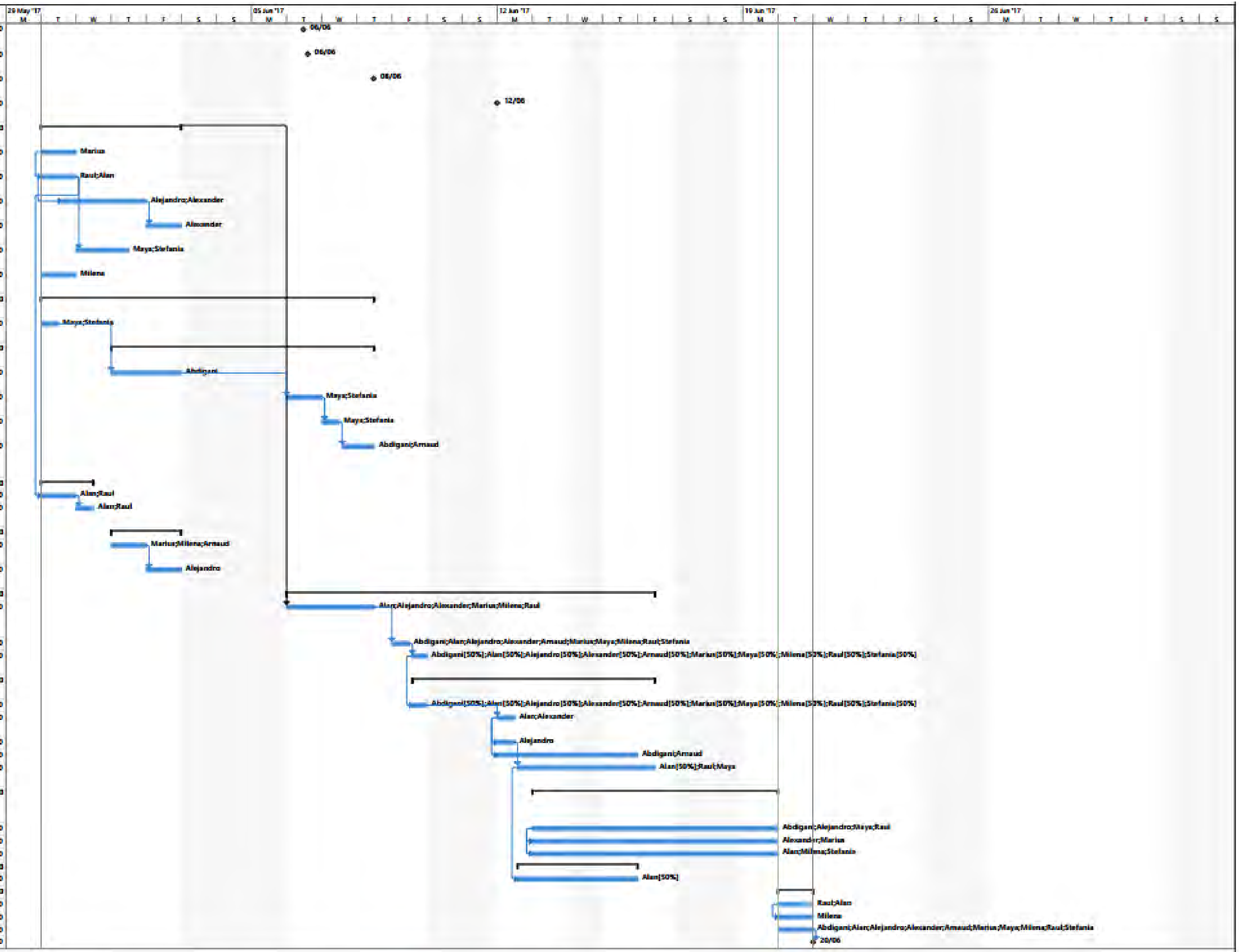
If the oil terminal must be increased there is no problem because at the moment we actually need one berth but we designed two. The frequency of crude oil tankers is smaller than refined oil tankers and can therefore use this jetty to moor.

This modified design is design for the future by ‘Design Your Dreams’.

APPENDICES

Appendix A: Determination of planning

ID	Nom de la tâche	Duration	Start	Finish
1	Project follow (BESSAN : Project management)	0 days	06/06/17 11:30	06/06/17 11:30
2	Project follow (SILVA : technical questions)	0 days	06/06/17 14:30	06/06/17 14:30
3	Project follow (ALLSOP : Coastal Structure)	0 days	08/06/17 11:30	08/06/17 11:30
4	Project follow + review (SILVA + CARPENTIER)	0 days	12/06/17 08:00	12/06/17 08:00
5	Design criteria	3,5 days	30/05/17 08:00	02/06/17 18:00
6	List of criteria	1 day	30/05/17 08:00	30/05/17 18:00
7	Analysis design ships	1 day	30/05/17 08:00	30/05/17 18:00
8	Calculate of Nr of berth	2 days	30/05/17 14:00	01/06/17 18:00
9	Calculate Quay Lenght	1 day	02/06/17 08:00	02/06/17 18:00
10	Calculate needed surface area	1 day	31/05/17 08:00	01/06/17 12:00
11	Safety requirements + Environments + Dredging	1 day	30/05/17 08:00	30/05/17 18:00
12	Design conditions	6 days	30/05/17 08:00	08/06/17 12:00
13	Bathymetry	0,5 days	30/05/17 08:00	30/05/17 12:00
14	Study of offshore waves/wind	4,5 days	01/06/17 08:00	08/06/17 12:00
15	Create the waves statistic + Analyse of wind	2 days	01/06/17 08:00	02/06/17 18:00
16	Create wave models (TOMAWAC)	1 day	06/06/17 08:00	06/06/17 18:00
17	Find extreme waves and design waves	0,5 days	07/06/17 08:00	07/06/17 12:00
18	Water level and protection of the water level in the future	1 day	07/06/17 14:00	08/06/17 12:00
19	Sedimentation	1,5 days	30/05/17 08:00	31/05/17 12:00
20	Study of currents	1 day	30/05/17 08:00	30/05/17 18:00
21	Describe sedimentation	0,5 days	31/05/17 08:00	31/05/17 12:00
22	Geotechnical data	2 days	01/06/17 08:00	02/06/17 18:00
23	Analyse and localise differents profil	1 day	01/06/17 08:00	01/06/17 18:00
24	Describe seismic conditions	1 day	02/06/17 08:00	02/06/17 18:00
25	Layout concept	7 days	06/06/17 08:00	16/06/17 12:00
26	Study of 2-3 concept with general organizational plan + wave agitation general design	2,5 days	06/06/17 08:00	08/06/17 12:00
27	List of criteria	0,5 days	09/06/17 08:00	09/06/17 12:00
28	Choice of the layout (Advantages + Drawbacks)	0,5 days	09/06/17 14:00	09/06/17 18:00
29	Finalisation of the layout	4 days	09/06/17 14:00	16/06/17 12:00
30	Improvments	0,5 days	09/06/17 14:00	09/06/17 18:00
31	Dimensioning the channel and the turning circle	0,5 days	12/06/17 08:00	12/06/17 12:00
32	Design for the future	0,5 days	12/06/17 08:00	12/06/17 12:00
33	Wave agitation	3 days	12/06/17 08:00	15/06/17 18:00
34	Design an organizational plan + futur approach	3 days	12/06/17 14:00	16/06/17 12:00
35	Dimensioning (including earthquake design + carbon footprint)	4 days	13/06/17 08:00	19/06/17 18:00
36	Breakwater	4 days	13/06/17 08:00	19/06/17 18:00
37	Dolphin	4 days	13/06/17 08:00	19/06/17 18:00
38	Solid Quay	4 days	13/06/17 08:00	19/06/17 18:00
39	Cost Estimation	2,5 days	12/06/17 14:00	15/06/17 18:00
40	Estimated cost	2,5 days	12/06/17 14:00	15/06/17 18:00
41	Optional	1 day	20/06/17 08:00	20/06/17 18:00
42	Construction method	1 day	20/06/17 08:00	20/06/17 18:00
43	Environmental assessment	1 day	20/06/17 08:00	20/06/17 18:00
44	Report + PWP	1 day	20/06/17 08:00	20/06/17 18:00
45	End of the project	0 days	20/06/17 18:00	20/06/17 18:00



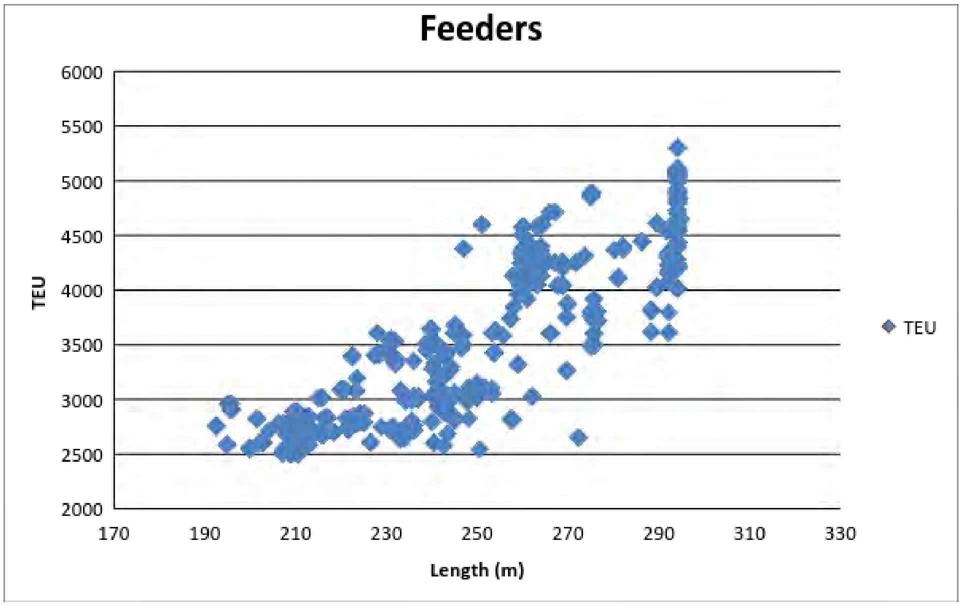
Projet : Planning Workshop
Date : 20/06/17 16:40

Tâche, Fractionnement, Jalon, Récapitulative, Récapitulatif du projet, Tâche inactive, Jalon inactif, Récapitulatif inactif, Tâche manuelle, Durée uniquement, Report récapitulatif manuel, Récapitulatif manuel, Début uniquement, Fin uniquement, Tâches externes, Jalons externes, Échéance, Avancement, Progression manuelle

Appendix B: Determination of the design ship parameters

LENGTH

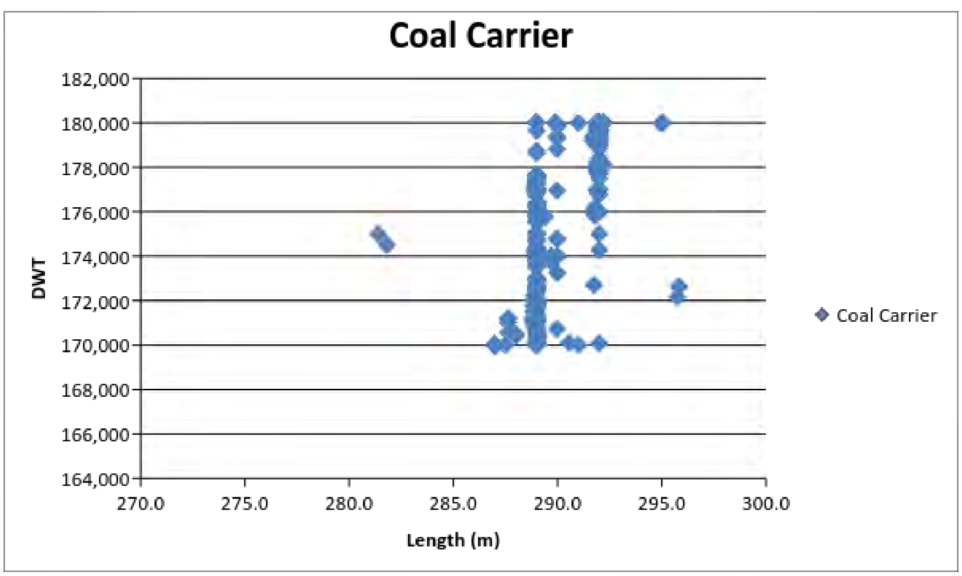
1. Feeders



The maximum LOA is 295 meters and 25 % of the feeders have a LOA of more than 270 meters, so, the design ship for feeders must be 290m because 25% of the ships is enough to consider.

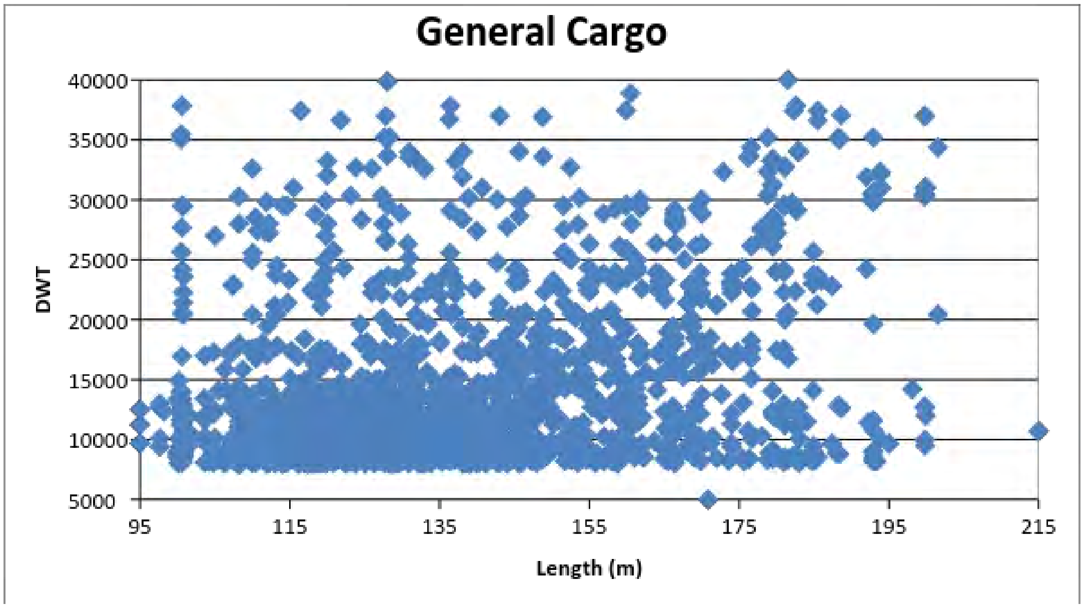
2. Coal carrier (Bulk carrier)

The capesize was of 170 000 to 180 000 DWT and we deleted the biggest and the smallest to study these.



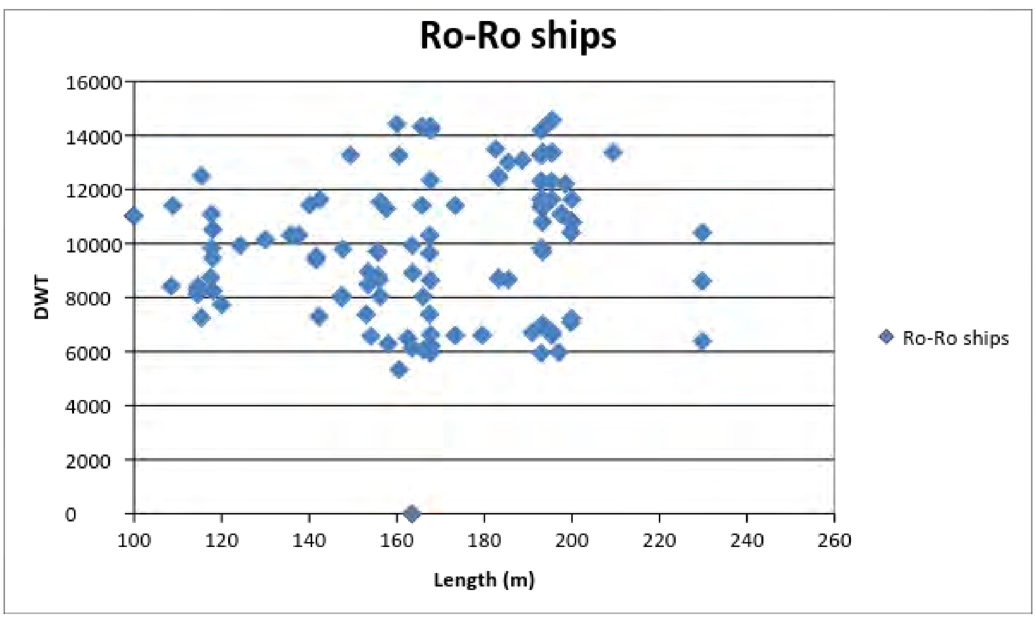
Only 2 % of the bulks carrier have a length of 293 meters or more,so in that case, the design ship will be 293m.

3. General Cargo Ships



For the general cargo, the maximum length is 215m, but it is only one ship. We take the one immediately below that have enough ships, so, the design ships will be 200m.

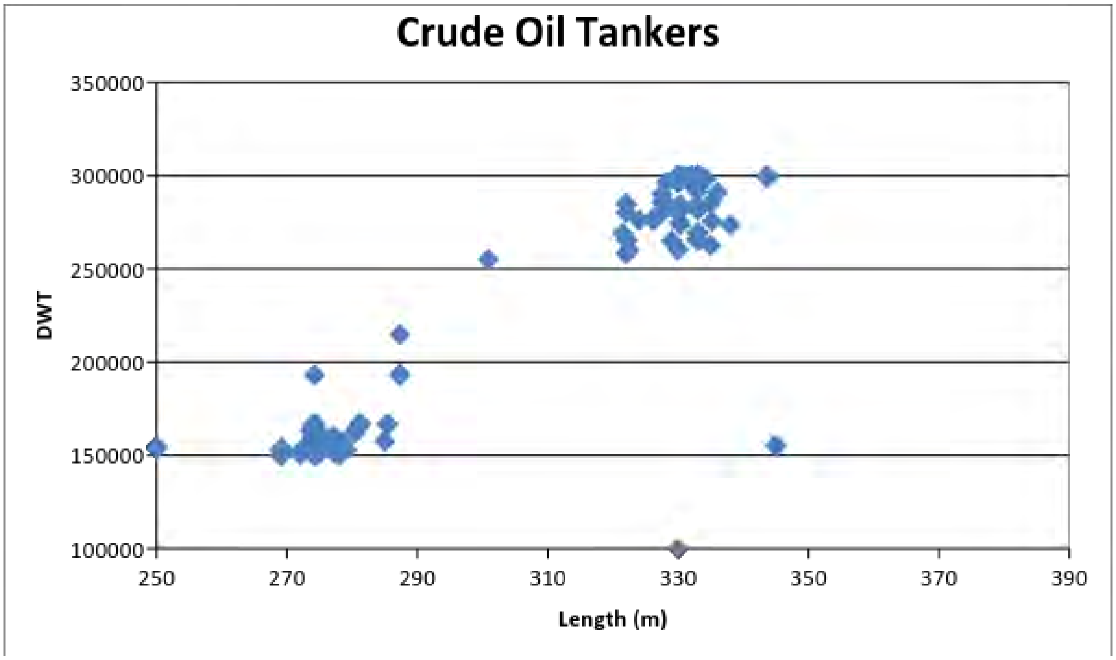
4. Ro-Ro ships



Only 3 % of these ships have a length of 200 meters or more. There is not a lot of Ro-Ro in the port so we won't take care of them. Our design ship will be 200m.

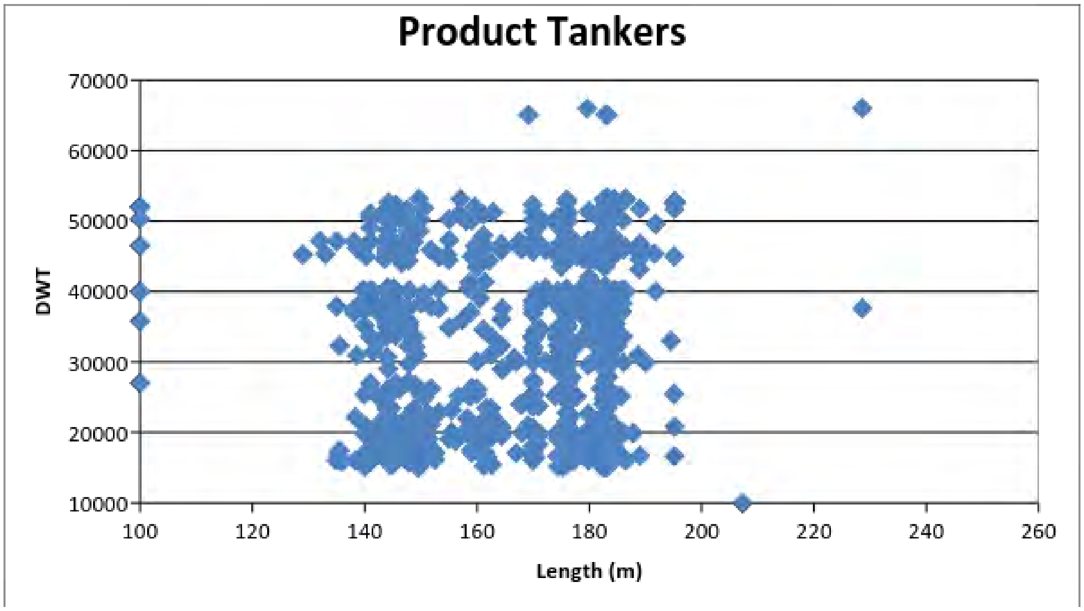
5. Tankers

- Large crude oil



The most of the crude oil tankers are bigger than 310 meters, so we take the maximum. Design ship: 340m.

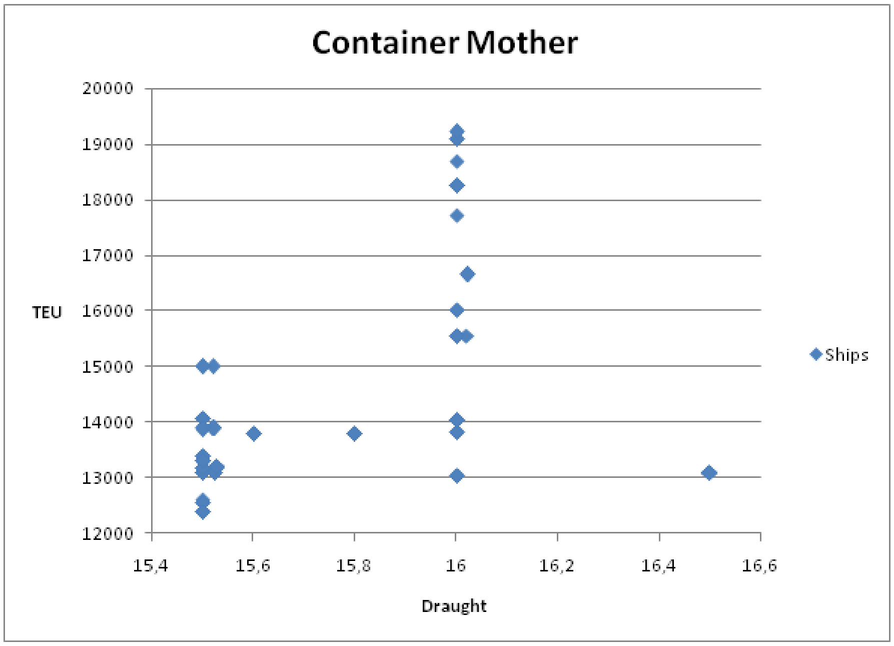
- Product Tankers:



0.15% of all the product tankers are bigger than 195 meters, so we take the ship with 195 m of length and the biggest deadweight tonnes to sizing the berths of this terminal. The DWT is going to affect to the draft of the port, because to more weight, more draft will have the ship.

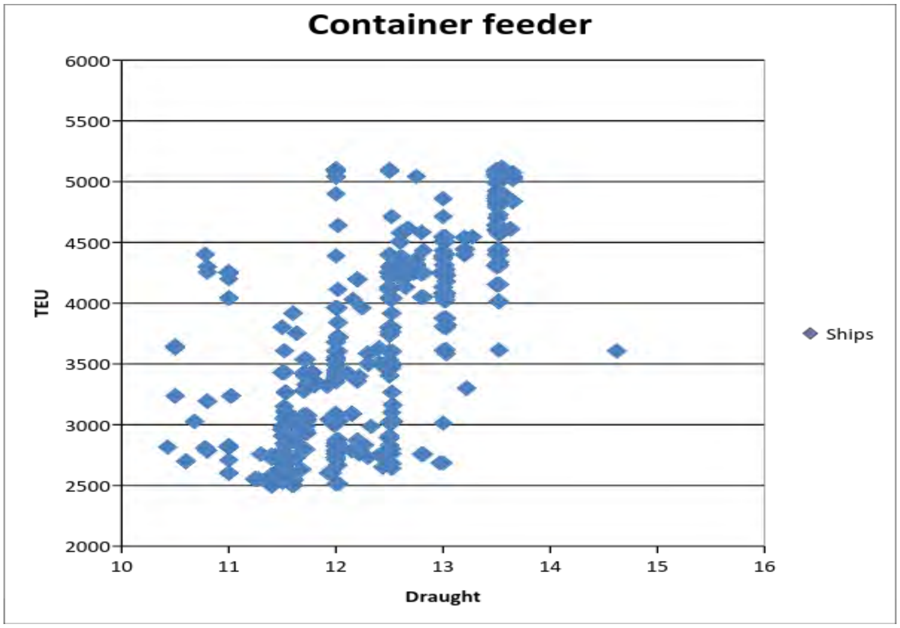
DRAUGHT

1. Container
    - Mother ships



6% of the ships have more than 16m of draught, so we choose 16 meters as the design ship for containers mother.

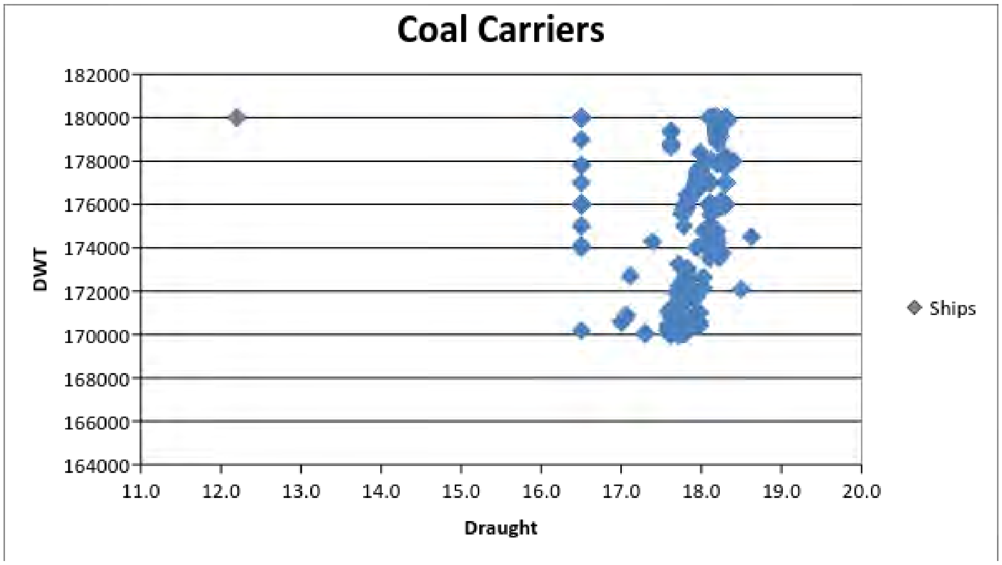
- Feeders:



The deepest feeder have 14,62 meters of draught but just a 0.13% of them are deeper than 14 m, so our design ship will be 13,65 m, the one immediately below.

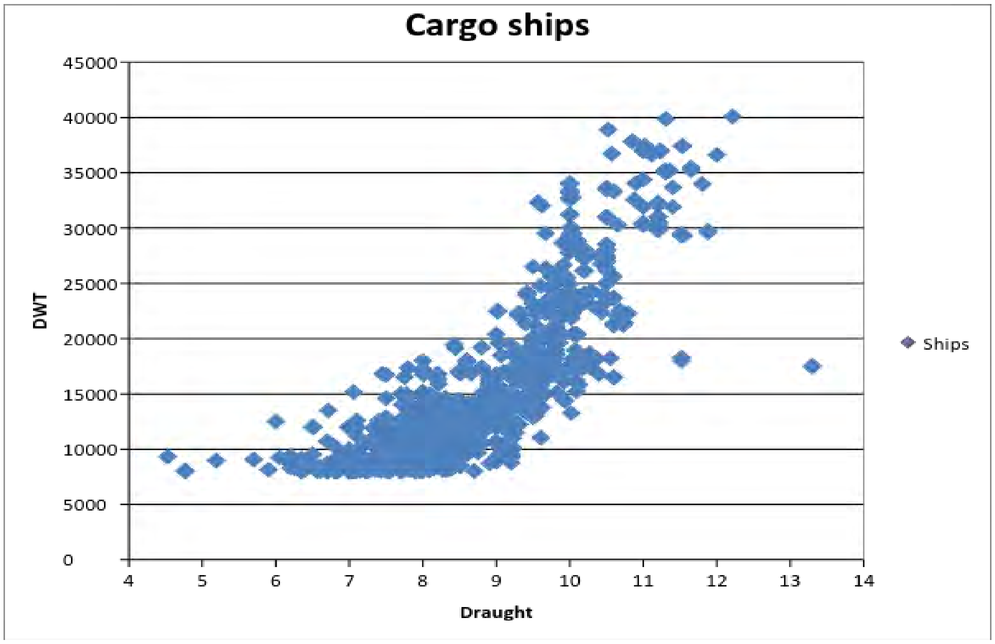
2. Coal carrier (Bulk carrier)

We have a CAPESIZE of 170000 to 180000 DWT so we deleted the biggest and the smallest.



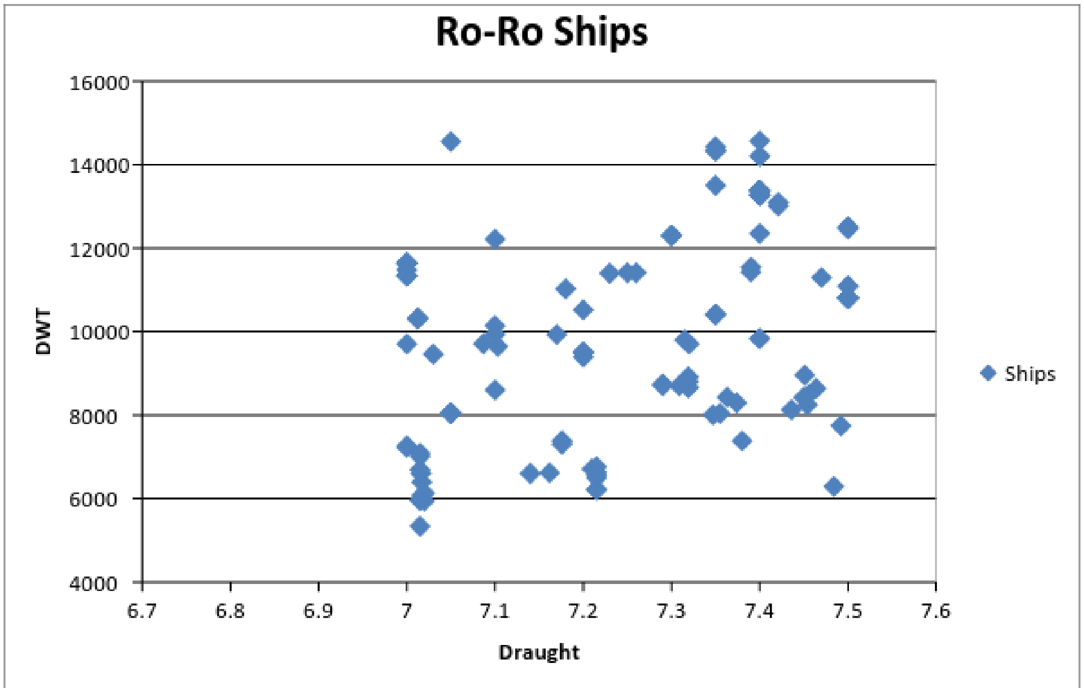
For the coal carriers the maximum is 18,6 meters and we take that because the majority of them are between 17.5 and 18,6 m, with a limit DWT of 18000

3. General Cargo Ships



The maximum is 13.3 m. 14.6% of the ships have more than 10 m of draught and just 0.17% of them have more than 12m,so the design ship is 12 m.

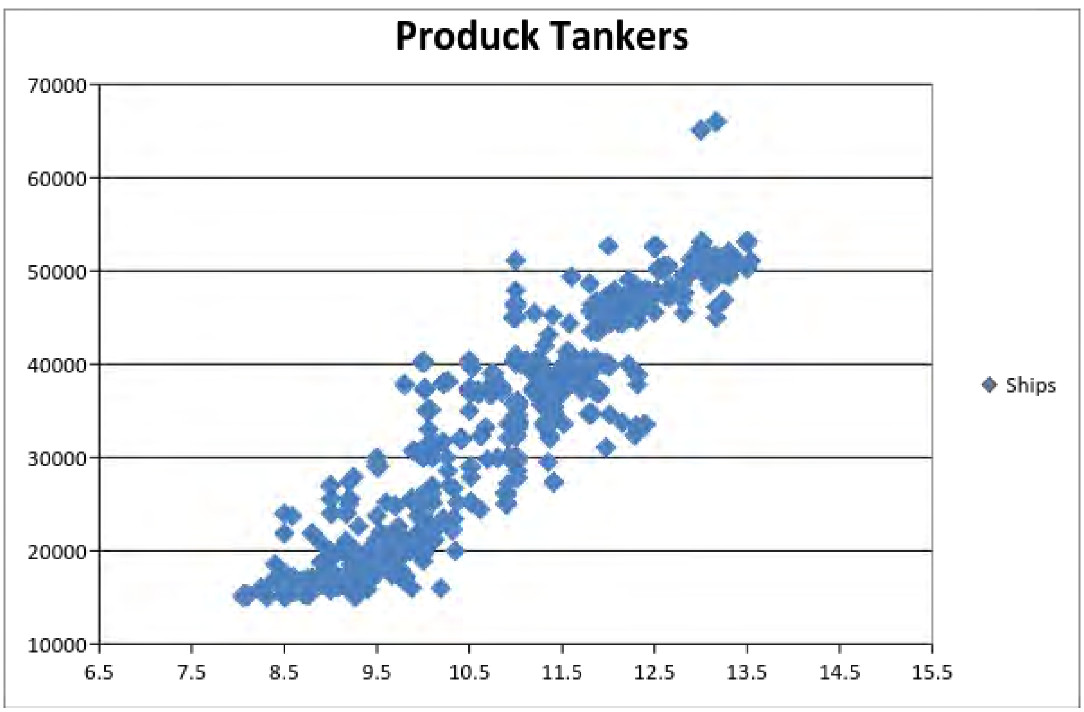
4. Ro-Ro ships



The maximum is 7.5 m and 12.12% of the ships have 7.5m of draught,so the design draught will be 7.5 m.

5. Tankers

- Large crude oil

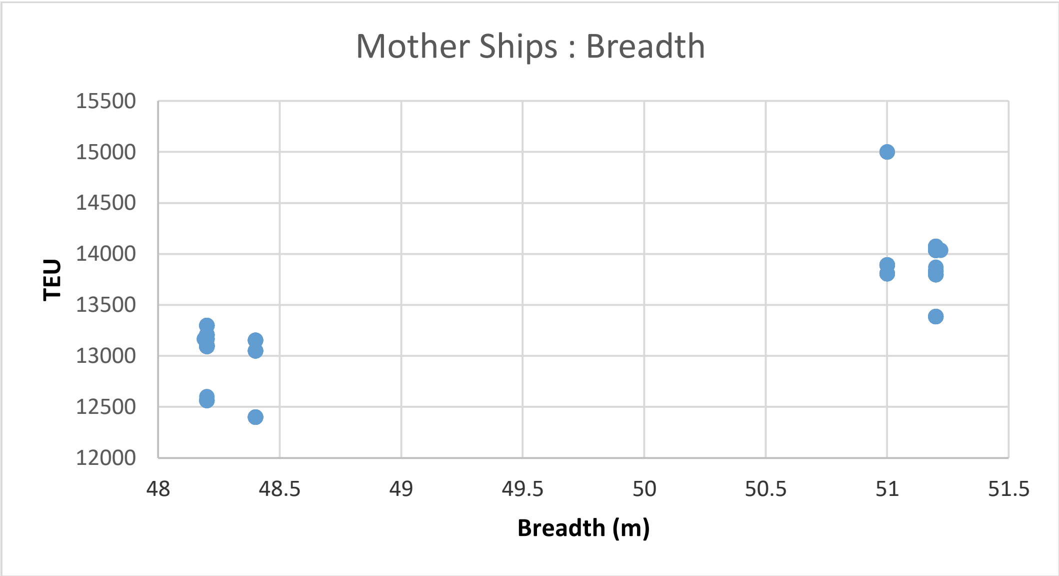


We have a ship with more than 13.5m of draught and twelve ships with 13.5 so we can choose 13.54m (the maximum).

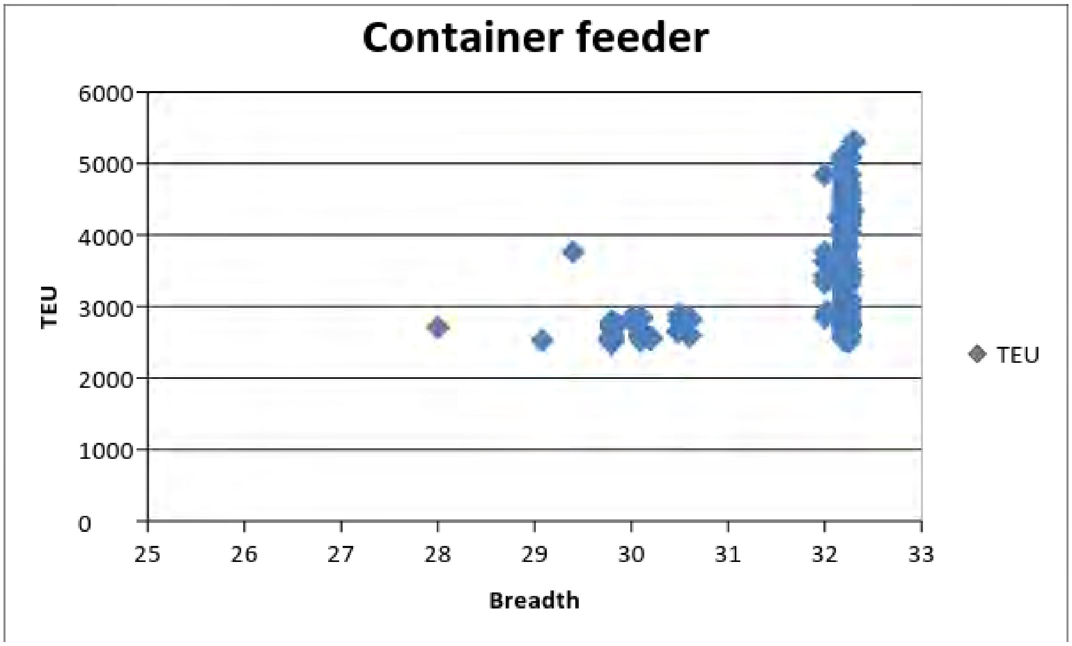
BREADTH

1. Mothership

According to our maximum length of our Mother Ships, we did not take into account the biggest ships. With this data, we have a maximum breadth of 51.2 m. This is the breadth that we will take for our next calculations.



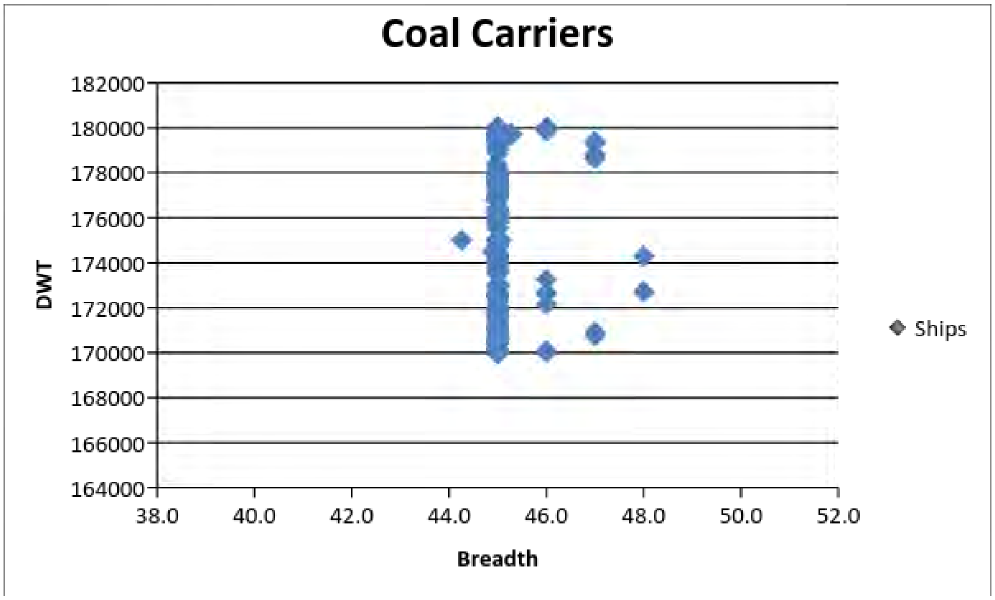
2. Feeders



As we can see the most of the ships have more than 32m of breadth with a maximum of 5300 TEU, so we choose 32,3 m (the maximal breadth).

3. Coal carrier (Bulk carrier)

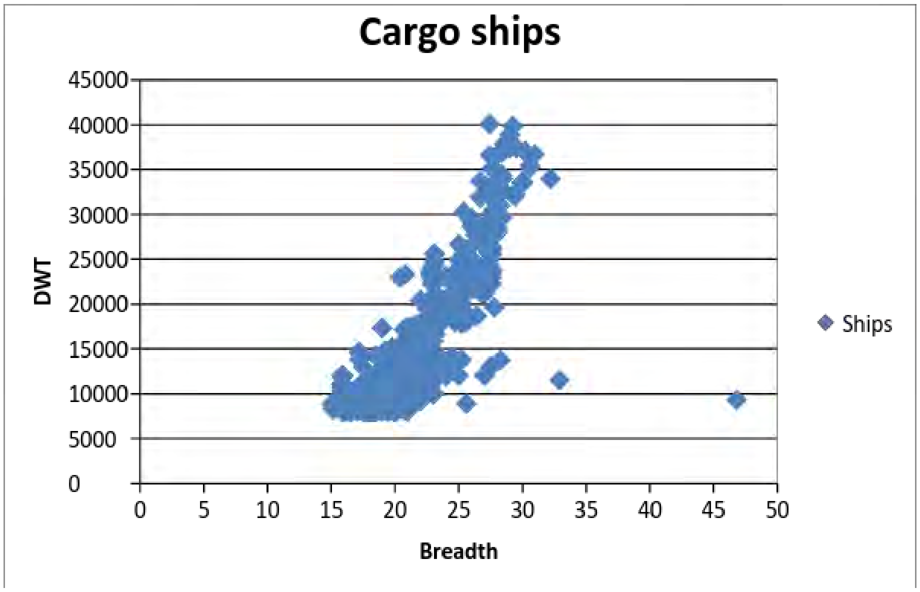
We have a CAPESIZE of 170000 to 180000 DWT so we deleted the biggest and the smallest.



10.3% of the ships have more than 45m of breadth, so we choose 45m of breadth because as we can see in the graphic, the most of the ships have 45m of breadth. The maximum is 48 m.

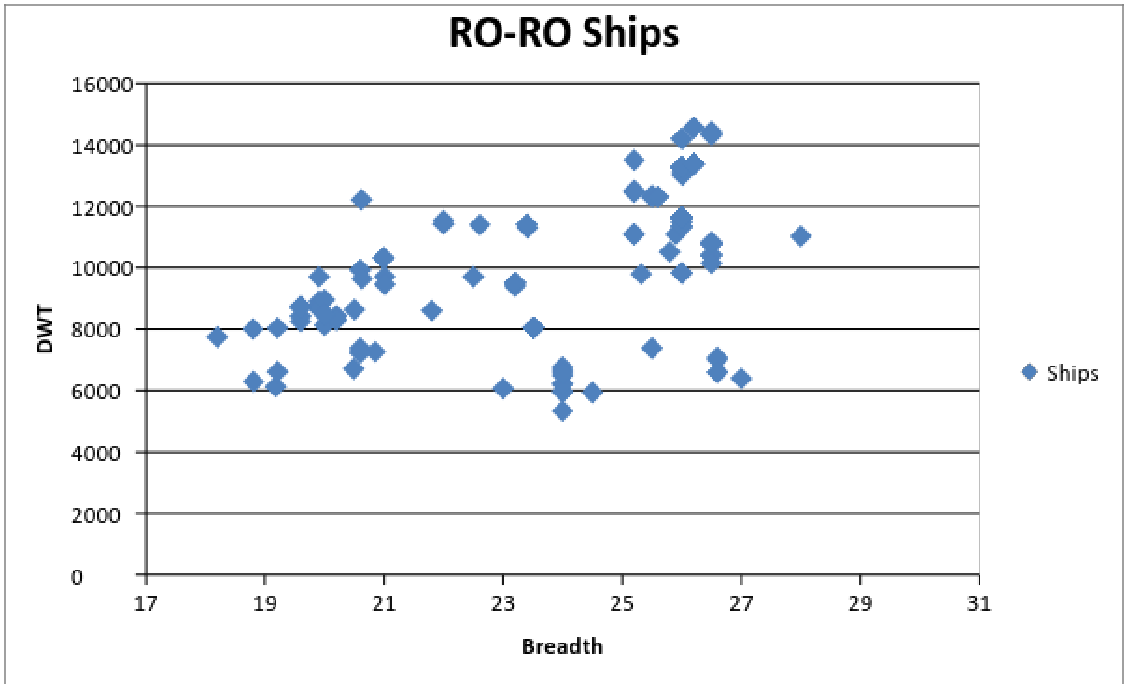
Design ships : 45 meters

4. General Cargo Ships



The maximum breadth is 46.8 m but it's only one ship. 99.4% of the ships have between 18 and 30m of breadth and 16.7% have more than 25 m, so, our design ship will be 30 m.

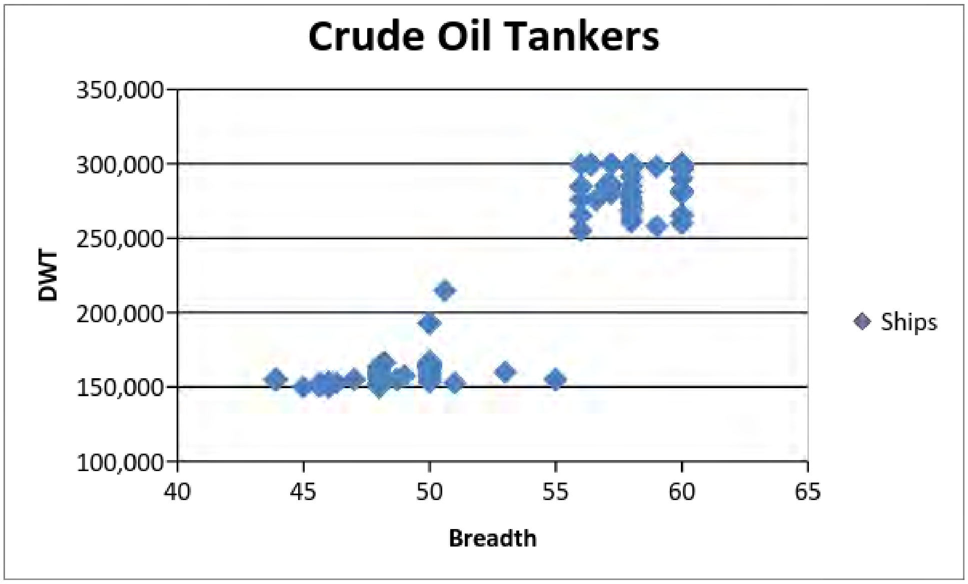
5. Ro-Ro ships



For the Ro-Ro ships we will use 27 meters of breadth because there is just one deeper.

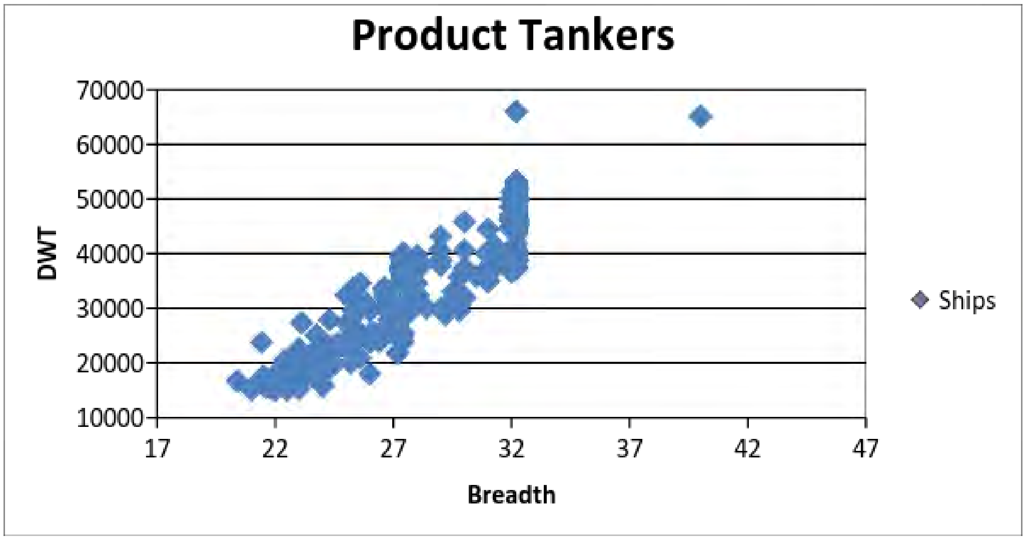
6. Tankers

- Large crude oil



The maximum is 60 m and 36% of ships are wider than 55 m, so we use a breadth of 60m.

- Product Tankers



The maximum is 40 m but it is not representative, so we choose 33 meters.

Appendix C: Calculation of the number of berths and quay length

The queuing theory will be applied. Different queuing systems are possible per terminal. The factors determining the behaviour of such a system are:

1. The vessels arrivals
2. The (un)loading times (+ time required for mooring = service times)
3. The service system (queue-discipline, number of berths)

The queue-discipline is taken as 'first in first out' (FIFO). The combination of the three-part code tells something about the distribution of the inter arrival times, service times, and the number 'n' of servers. For every queuing system there is a different table to find the average waiting time of ships in the queue in units of average service time (Groenveld, 2001). In order to find this waiting time, the arrival rate, service rate and berth occupancy should be calculated first.

$$arrival\ rate: \lambda = \frac{calls/year}{operational\ hours/year}$$

$$(un)loading\ time = \frac{(un)loading/call}{(un)loading\ rate}$$

$$service\ rate: \mu = \frac{1}{(un)loading\ time + \ mooring\ time}$$

$$berth\ occupancy: \rho = \frac{\lambda}{\mu}$$

With this berth occupancy first estimates of the number of berths 'n' can be made. This first estimate can be used to start calculating the utilisation.

$$utilisation: u = \frac{\lambda}{\mu * n}$$

Because the queuing system is known, it is now possible to find the average waiting time of ships in the queue (in units of average service time) by using the tables described above. If this waiting time meets the requirements (for containers = ~ 1 to 5 %, for oil, bulk and general cargo = ~ 5 to 10 %), <u>the number of berths is sufficient</u>. Otherwise, more estimates of the number of berths 'n' should be tried.

2. Verification by berth productivity

Such an estimate is made as follows:

$$c_b = P * f_{TEU} * N_{cb} * n_{hy} * m_b * E_f$$

c_b = average annual productivity per berh ....(TEU/yr or tones/yr)
P = net production per crane ....(containers/hr)
f_{TEU} = TEU factor ....(-)
N_{cb} = number of cranes per berth ....(-)
n_{hy} = number of operational hours per year ....(hrs/yr)
m_b = berth occupancy factor..(-)
E_f = efficiency factor...(-)
C = total number of TEU entering and leaving the terminal.......(TEU/yr or tones/yr)

2.1 CONTAINER TERMINAL

P = 25 containers/hour (1 container = 1.5 TEU) for a container gantry crane.
f_{TEU} = (1 container = 1.5 TEU) = 1.5 TEU
N_{cb} = 4 cranes
n_{hy} = 24 hours * 350 days = 8400 hours/year
m_b = 0.6
Ef = 0.8
C = 3000000 TEU

$$c_b = 25 * 1.5 * 4 * 8400 * 0.6 * 0.8 = 604800 \text{ TEU/year}$$

Subsequently the number of berths (n) is calculated as:

$$n = \frac{C}{Cb} = \frac{3000000\,TEU}{604800\,TEU} = 4{,}96 = 5$$

<u>We will have to build **5 BERTHS**</u>

2.2 OIL TERMINAL

P = 12500 tones/year
N_{cb} = 1 crane
n_{hy} = 24 hours * 350 days = 8400 hours/year
m_b = 0.3
Ef = 0.8
C = 25000000 tones/yr

$$c_b = 12500 * 1 * 8400 * 0.3 * 0.8 = 25200000 \text{ tones/year}$$

Subsequently the number of berths (n) is calculated as:

$$n = \frac{C}{Cb} = \frac{25000000\,Tones/yr}{25200000\,Tones/yr} = 0{,}99 = 1$$

<u>We will have to build **1 BERTH**</u>

2.3 DRY BULK (COAL) TERMINAL

P = 4000 tones/year

N_{cb} = 1 crane

n_{hy} = 24 hours * 350 days = 8400 hours/year

m_b = 0.45

Ef = 0.8

C = 7000000 tones/yr

c_b = 4000 * 1 * 8400 * 0.45 * 0.8 = 12096000 tones/year

Subsequently the number of berths (n) is calculated as:

$$n = \frac{C}{Cb} = \frac{12096000\ \ Tones/yr}{7000000\,Tones/yr} = 1{,}73 = 2$$

<u>We will have to build **2 BERTHS**</u>

2.4 GENERAL CARGO & RO-RO TERMINAL

P = 1000 tones/year

N_{cb} = 2 cranes

n_{hy} = 24 hours * 350 days = 8400 hours/year

m_b = 0.45

Ef = 0.8

C = 3000000 tones/yr

c_b = 1000 * 2 * 8400 * 0.45 * 0.8 = 6048000 tones/year

Subsequently the number of berths (n) is calculated as:

$$n = \frac{C}{Cb} = \frac{6048000\ \ Tones/yr}{3000000\,Tones/yr} = 2$$

<u>We will have to build **2 BERTHS**</u>

Appendix D: Surface area calculations

Containers

To calculate the needed area for container terminal it was used next formula:

$S_{tot} = S_{st} + S_{tb} + S_{ab} + S_{ech}$

Where,

S_{tot} is the total area needed,

S_{st} is the needed area for the storage of the containers

S_{tb} is the needed area for technical buildings

S_{ab} is the needed area for administrative buildings and

S_{ech} is the needed area for exchange

The surface storage is calculated as $S_{st} = \frac{\text{NTEU X Tst X STEU}}{\text{Fu X 365 X Focc}}$ and the other areas are calculated as a percentage of this area.

It takes in account the traffic volume (3 Millions TEU per year), the average time of the containers in the park (8 days), the surface occupied by one TEU and the level and crane type used to storage the containers (5 levels, RMG).

Variable	Description	Unit	Value
Stotal	total storage area	ha	90
Sst	storage area (70%)	m2	632244
Sctb	surface for technical buildings (10%)	m2	90321
Ssv	surface for administrative buildings (5%)	m2	45160
Svar	surface for exchange (tracks and trains and terminal-15%)	m2	135481

Nteu	yearly traffic in TEU	TEU	3000000
Tst	average time of 1 container in the park (8-12 days)	days	8
Steu	surface occupied for one TEU, depending of the operational system- RMG	m2/TEU	6
Fu	utilisation factor of the available height (=1 for storage at 1 level, not more than 0,78 for various levels)	-	0,78
Focc	occupation factor of the terminal	-	0,8

General cargo and Ro-Ro

RO-RO			
Variable	**Description**	**Unit**	**Value**
N° berths	number of berths	-	2
area	needed area	has	20

GENERAL CARGO			
Variable	**Description**	**Unit**	**Value**
Lberth	length of the berths	m	400
Wgc	width of the storage	m	150
a	quay for crane (30-40)	m	40
b	warehouses (50-60)	m	60
c1+c2	storage and trucks (40-50)	m	50

Solid bulk

Variable	Description	Unit	Value
Volume	Volume of two ships	Tons	360000
Area	Needed area	has	19
Storage coefficiente		m3/ton	1,08
Maximum height	Maximum height of the storage	m	6

Angle	Angle of the mound	°	45
Average Hight		m	2

Liquid bulk

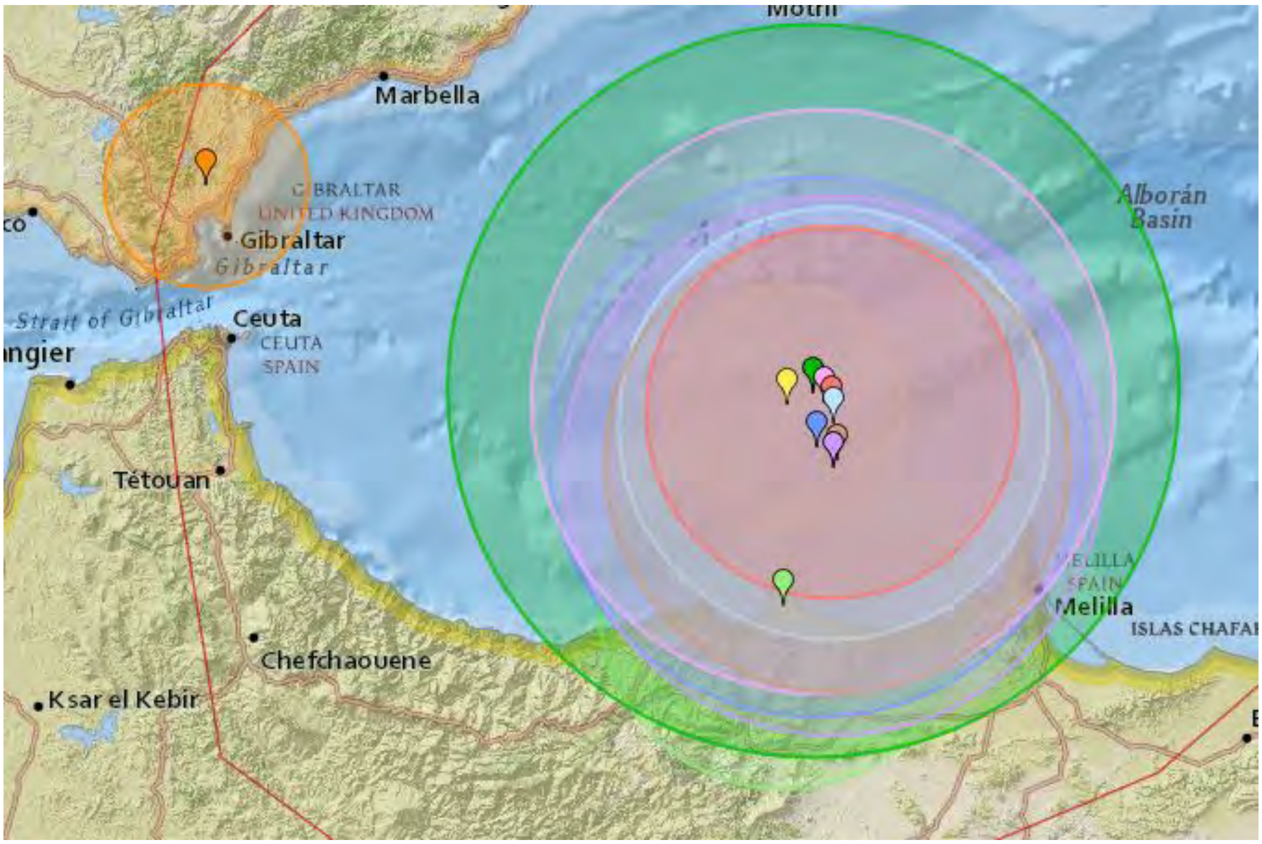
CRUDE OIL			
Variable	**Description**	**Unit**	**Value**
Volume per month	Needed capacity considering that the oil « will be in the the tank » for a month	ton	600000
Density	density of oil	ton/m3	0,825
tank capacity	capacity of one tank	m3	100000
N° tanks	number of tanks	-	8
Needed area	needed area for tanks	has	20

REFINED PRODUCTS			
Variable	**Description**	**Unit**	**Value**
Volume per month	Needed capacity considering that the oil « will be in the the tank » for a month	ton	100000
H	height of tank	m	12
D	diameter of tank	m	55
Density	density of oil	ton/m3	0,825
tank capacity	capacity of one tank	m3	30000
N° tanks	number of tanks	-	4
Needed area	needed area for tanks	has	2,25

Appendix E: Seismic activity

Below we are going to indicate which are the most important earthquake that Morocco has suffered in the last year, we can see that all of them are near to Nador, Morocco. Lastly, we are going to indicate the last closer earthquakes of our project zone. We are using the Richter scale.

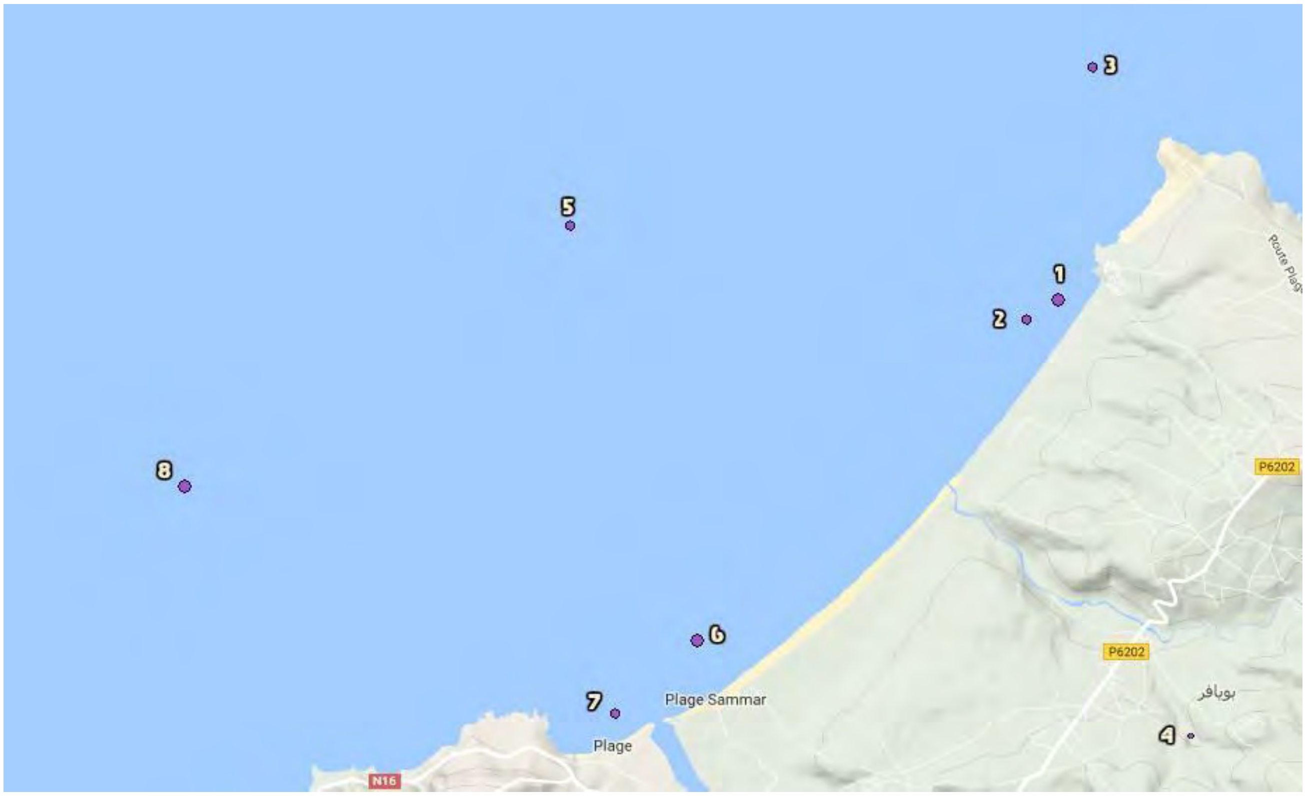
EARTHQUAKES IN THE LAST YEAR



- 9 months ago 3.8 magnitude, 5 km depth San Roque, Andalusia, Spain	- A year ago 5.2 magnitude, 10 km depth Al Hoceïma, Taza-Al Hoceima-Taounate, Morocco
- A year ago 4.1 magnitude, 10 km depth Al Hoceïma, Taza-Al Hoceima-Taounate, Morocco	-A year ago 5.3 magnitude, 10 km depth Al Hoceïma, Taza-Al Hoceima-Taounate, Morocco
- A year ago 4.7 magnitude, 10 km depth Imzoûrene, Taza-Al Hoceima-Taounate, Morocco	- A year ago 4.9 magnitude, 10 km depth Al Hoceïma, Taza-Al Hoceima-Taounate, Morocco

- A year ago 5.6 magnitude, 10 km depth Al Hoceïma, Taza-Al Hoceima-Taounate, Morocco	- A year ago 5.0 magnitude, 10 km depth Al Hoceïma, Taza-Al Hoceima-Taounate, Morocco
- A year ago 5.2 magnitude, 10 km depth Al Hoceïma, Taza-Al Hoceima-Taounate, Morocco	- A year ago 4.7 magnitude, 10 km depth Al Hoceïma, Taza-Al Hoceima-Taounate, Morocco

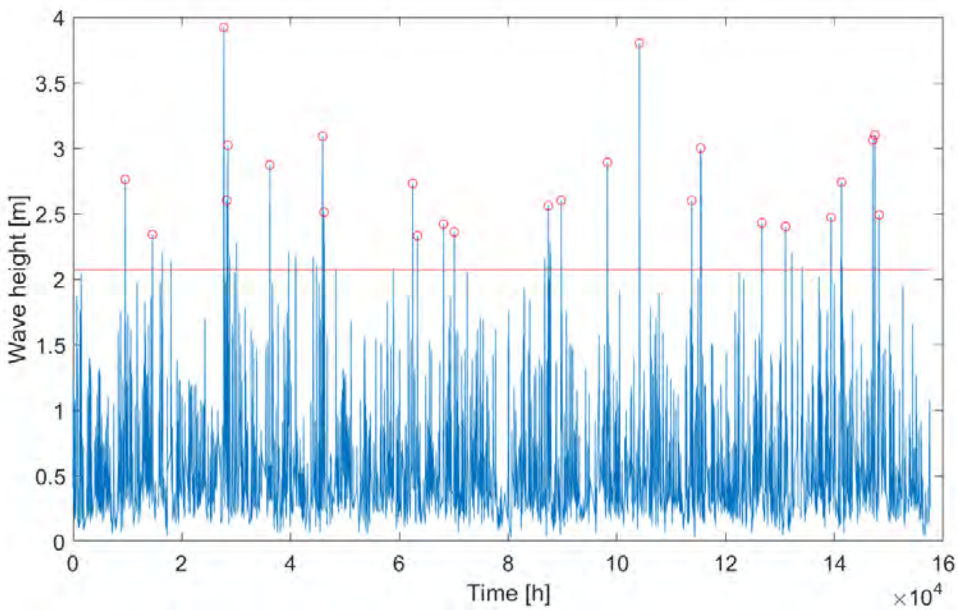
LAST EARTHQUAKES IN OUR PROJECT ZONES



Mag	Depth km	Day	Time UTC	Lat	Lon	Dist km
3.7	10	2004-12-26	07:58:34	35.27	-3.15	0
2.8	0	2004-12-02	23:02:19	35.27	-3.15	0
3.2	10	2004-12-05	10:45:16	35.29	-3.14	2
2.2	22.7	2001-07-13	02:56:23	35.23	-3.13	5
3	0	2004-12-04	12:06:03	35.28	-3.21	5
3.6	10	2004-12-09	07:50:38	35.23	-3.19	5
3	0	2004-12-07	12:04:24	35.23	-3.2	6
3.5	24.2	2000-01-24	00:26:08	35.25	-3.25	9

Appendix F: Wave analysis

An example of the chosen threshold for NE is illustrated in the figure below. The number of peaks is chosen as 25 for each direction.



For a 100-year return period, the design wave height through a Weibull distribution is determined by following formula:

$$x^T = A\left(-\ln\left(\frac{1}{\lambda T}\right)\right)^{1/k} + B$$

Where
$x^T = wave\ height\ corresponding\ to\ return\ period\ T$
$A, k, B = distributions\ parameters$
$\lambda = sample\ intensity$
$T = return\ period$

A fitting method is chosen in order to fit the extreme data to the chosen distribution. This can be done by either applying Maximum Likelihood method or Least Square Method. Through these methods the distributions parameters are also found. In the following the formulas to determine these is introduced for both methods:

<u>Maximum Likelihood Method</u>
The maximum likelihood estimate *k* is obtained by solving the following equation in iterative manner:

$$N + k\sum_{i=1}^{N}\ln(x_i - x') = N\,k\,\sum_{i=1}^{N}((x_i - x')^k \ln(x_i - x'))\left(\sum_{i=1}^{N}(x_i - x')^k\right)^{-1}$$

This has been done I the program MATLAB. The different parameters are defined as:

$N = number\ of\ extreme\ data$
$x_i = extreme\ data$
$x'(B) = threshold\ value$

The distribution value of *A* is estimated by:

$$A = \left[\frac{1}{N}\sum_{i=1}^{N}(x_i - x')^k\right]^{1/k}$$

<u>Least Square method</u>

For the least squared method, the fitting is done by:

$$X = AY + B$$

Where

$$A = \frac{COV(Y,X)}{Var(Y)}, \quad B = \bar{X} - A\bar{Y}$$

$$Var(Y) = \frac{1}{n}\sum_{i=1}^{n}(y_i - \bar{Y})^2$$

$$COV(Y,X) = \frac{1}{n}\sum_{i=1}^{n}(y_i - \bar{Y})(x_i - \bar{X})$$

$$Y = (-\ln(1 - F))^{1/k}$$

The different parameters are defined as:

$\bar{X}, \bar{Y} = average\ values\ of\ X\ and\ Y$
F = *non-exceedance probability* , $F_i = 1 - \frac{i}{n+1}$
$n = number\ of\ data\ pairs$

The fitting goodness of both methods these is evaluated through relative error determined by:

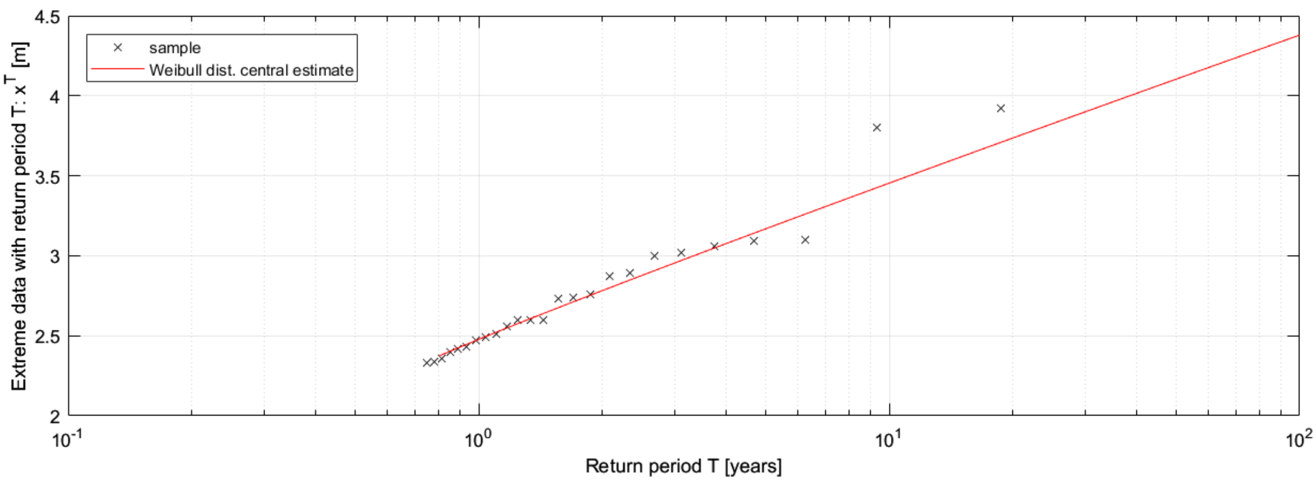
$$E = \frac{1}{n}\sum_{i=1}^{n}\frac{\left|x_{i,estimated} - x_{i,observed}\right|}{x_{i,estimated}}$$

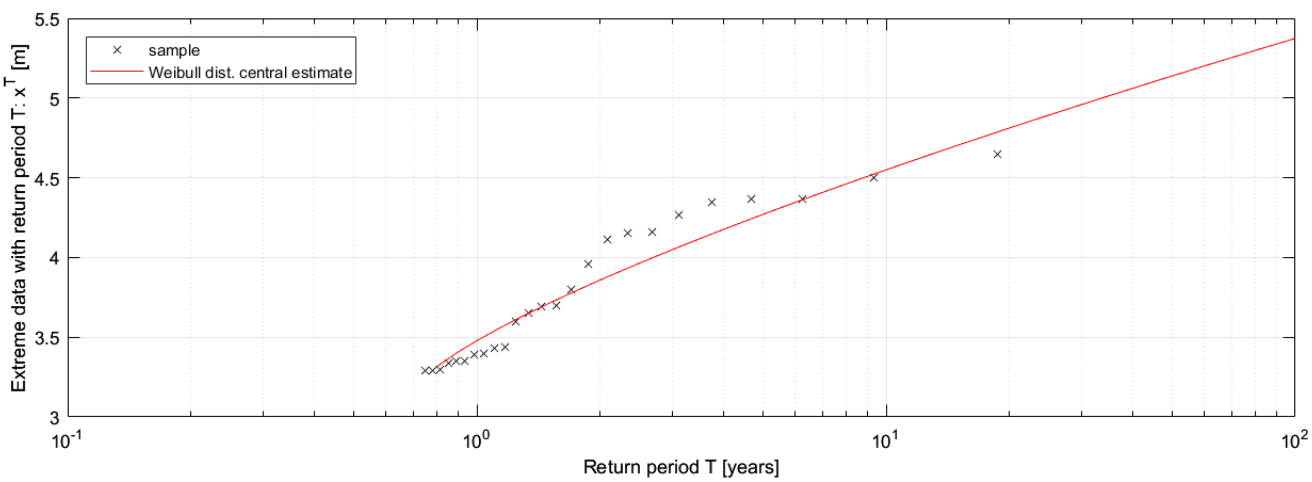
The obtained results are given in the table below:

	Maximum Likelihood Method	Least Squared Method
NE	1.66%	4.64%
NW	2.22%	3.32%

Design conditions

The plot of extreme data against the return period, *T*, is shown in the figure below. Firstly the main direction NE is shown and afterward NW.





The distributions parameters are given in the following table.

Direction	*A* [-]	*B* [-]	*k* [-]
NE $(22^o\text{-}67^o)$	0.453	2.32	1.05
NW $(292^o\text{-}337^o)$	0.648	3.2	1.32

Direction	x^T [m]
NE $(22^o\text{-}67^o)$	4.38
NW $(292^o\text{-}337^o)$	5.37

Since there may be some uncertainties regarding the measured wave data, these will be taken into account by a Monte-Carlo simulation. A numerical simulation is applied and the result is a design wave height (H_s), which corresponds to a 90% one-sided confidence interval. The simulation results in higher values of the wave heights. There is no theory, which describes how to determine the design wave theory corresponding to the design wave height obtained by extreme analysis. This is due to the complexity and locality of the joint distribution between wave height and wave period. In DS449, the range of peak period is described as:

$$\frac{\sqrt{130\,H_s}}{g} < T_p < \frac{\sqrt{280\,H_s}}{g}$$

A mean value of the range is used.

Matlab code: Wave and Wind data for Nador West

```
%--------------------------------------------------------------

% CAEN Workshop

clc; clear all;close all
```

Input

```
load(strcat('WavesWind','.mat'));   % Data for 18 years

waves=WavesWind(:,5);               % Wave height data

wind=WavesWind(:,10);               % Wind data

%dir_m=WavesWind(:,8);               % mean direction

dir_p=WavesWind(:,9);               % peak direction

dir_v=WavesWind(:,11);

tp=WavesWind(:,7);

hours=[0:3:157799]';                % time in hours

dir_m=dir_p;

% figure(1)            % timeseries for wave data

% plot(hours,waves);

% %hold on

% %scatter(time3(locations),wind(locations),10,'filled')

% %hold off

% % title('Wind speed')

% xlabel('Time [h]') ; ylabel('Wave height [m]');

% set(gca, 'fontsize', 16);

%

% figure(2)           % timeseries for wind data

% plot(hours,wind);

% %hold on

% %scatter(time3(locations),wind(locations),10,'filled')

% %hold off

% % title('Wind speed')

% xlabel('Time [h]') ; ylabel('Wind speed [m/s]');
```

```
% set(gca, 'fontsize', 16);
```

Plotting the wave- and windrose

```
[figure_handle,count,speeds,directions,Table]=WindRose...

    (dir_m,waves,'FreqLabelAngle',45,'nFreq',4,'TitleString',{'';''});

[figure_handle,count,speeds,directions,Table]=WindRose...

    (dir_p,waves,'FreqLabelAngle',45,'nFreq',4,'TitleString',{'';''});

[figure_handle,count,speeds,directions,Table]=WindRose...

    (dir_v,wind,'FreqLabelAngle',45,'nFreq',4,'TitleString',{'';''});
```

Seperating into main direction NE

```
dirNE=[];

for i=1:length(waves)-1

   if (22<dir_m(i)) && (dir_m(i)<=67)

     dirNE(end+1)=i; % save index of crossing points

   end

end

% Finding the directions

 dir_NE=zeros(23084,1);

  for n = 1:23084

      dir_NE(n)=dir_m(dirNE(1,n));

  end

% The corresponding wind speeds in m/s

 H_NE=zeros(23084,1);

for n=1:23084

    H_NE(n)=waves(dirNE(1,n));

end

% The corresponding time in days

 t_NE=zeros(23084,1);

for n=1:23084

    t_NE(n)=hours(dirNE(1,n));

end

% Finding number of peaks and ensuring independence criteria
```

```
[peakNE,locNE] = findpeaks(H_NE, 'MinPeakDistance', 24, 'MinPeakHeight', 2.3);

Hs_NE=flipud(sort(peakNE));

figure(3)           % timeseries for wind data

plot(t_NE,H_NE);

hold on

plot(t_NE, 2.07*ones(1,length(t_NE)),'r');

hold on

scatter(t_NE(locNE),H_NE(locNE),40,'ro')

hold off

xlabel('Time [h]') ; ylabel('Wave height [m]');

set(gca, 'fontsize', 16);
```

Seperating into main direction NW

```
dirNW=[];

for i=1:length(waves)-1

   if (292<dir_m(i)) && (dir_m(i)<=337)

     dirNW(end+1)=i; % save index of crossing points

   end

end

% Finding the directions

 dir_NW=zeros(26769,1);

  for n = 1:26769

      dir_NW(n)=dir_m(dirNW(1,n));

  end

% The corresponding wind speeds in m/s

 H_NW=zeros(26769,1);

for n=1:26769

    H_NW(n)=waves(dirNW(1,n));

end

% The corresponding time in days

 t_NW=zeros(26769,1);
```

```
for n=1:26769

    t_NW(n)=hours(dirNW(1,n));

end

% Finding number of peaks and ensuring independence criteria

[peakNW locNW] = findpeaks(H_NW, 'MinPeakDistance', 24, 'MinPeakHeight', 3.25);

Hs_NW=flipud(sort(peakNW));

% figure(3)           % timeseries for wind data

% plot(t_NW,H_NW);

% hold on

% plot(t_NW, 3.25*ones(1,length(t_NW)),'r');

% hold on

% scatter(t_NW(locNW),H_NW(locNW),40,'ro')

% hold off

% xlabel('Time [h]') ; ylabel('Wave height [m]');

% set(gca, 'fontsize', 16);
```

Appendix G: Wave propagation

The Telemac system was developed in France by EDF R&D and is a system for analysing waves, sedimentology and hydrodynamics and water quality. It is based on the finite element method based on the Fortran 90 source code. TOMAWAC software is used for 2D analysis of wave propagation. It models the generation and propagation of sea states from the ocean to coastal areas. The changes both in time and spatial domain can be calculated using the wave energy direction spectrum and the bathymetry data.

The following parameters were used as an input for analysing the waves at Baie de Betoya.

NW angle was taken -45° + 180° = 135 °

NE was taken as 30° + 180° = 210 °

Operational waves 10°+180°=190° for both NW and NE

Initial conditions at T0

- initial still water level = 0.66 (Highest High water spring)
- main direction = 0
- Initial peak frequency = 0.01
- minimal frequency = 1/2Tp
- Hs= 0
- type of initial directional spectrum = 6

Boundary conditions

- boundary peak frequency = 1/Tp
- boundary significant wave height - depends on the case
- type of boundary direction spectrum = 6

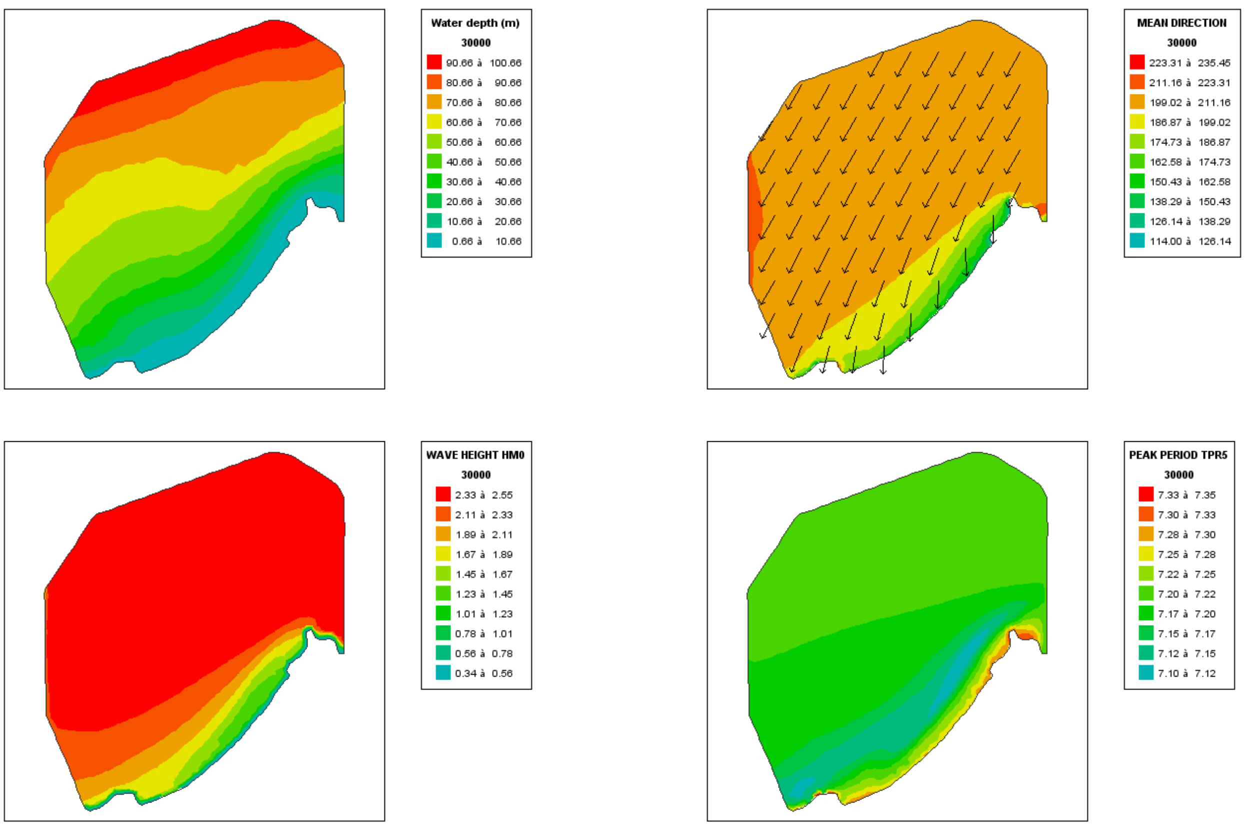
Discretisation

- number of directions = 24
- number of frequencies = 21
- minimal frequency = 1/2Tp

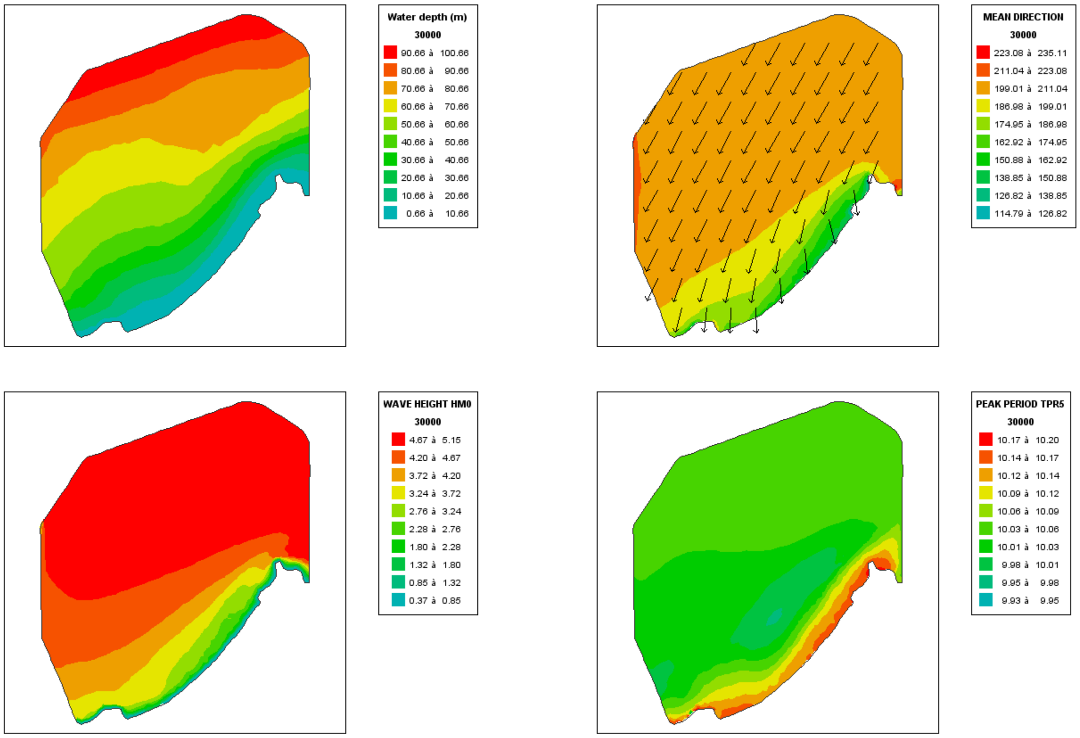
General parameters

- time step = 15 s
- number of time steps = 2000
- period of listing printout = 20s
- variables for 2D graphic printouts – Hm0 (wave height), Dmoy (wave direction), TRP5 (peak period), WD (water height)
- period for graphic printouts = 20s
- depth induced breaking dissipation = 1 (Battjes and Janssen model)
- number of breaking time step = 5
- bottom friction dissipation = 1

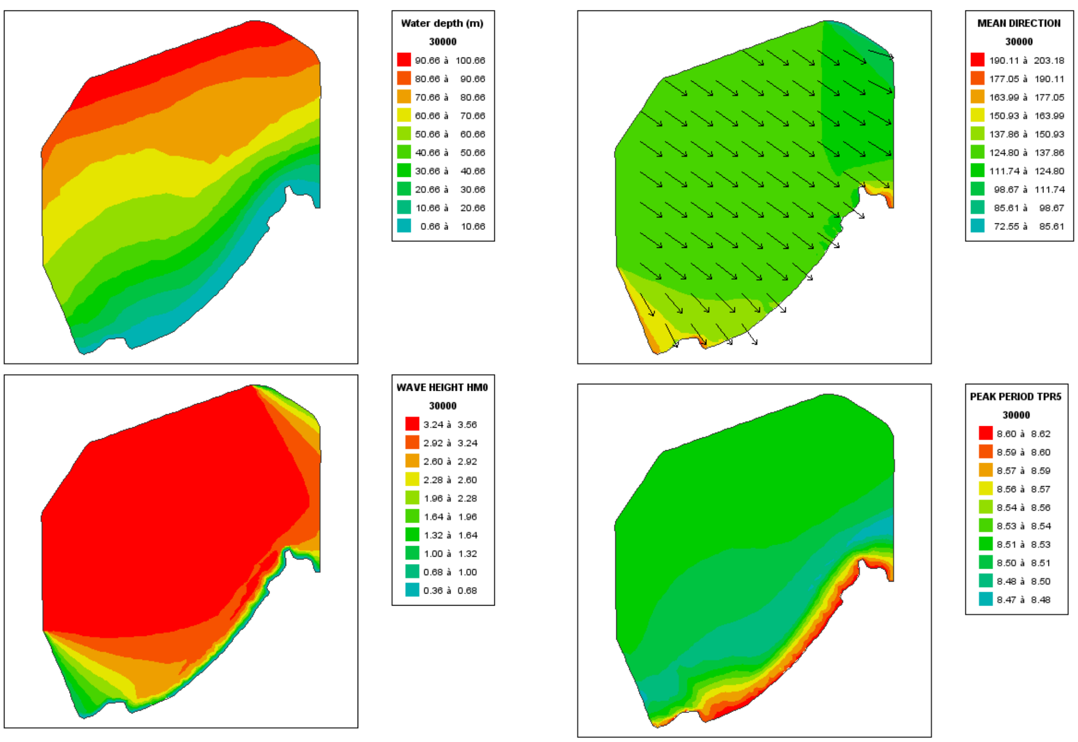
Results



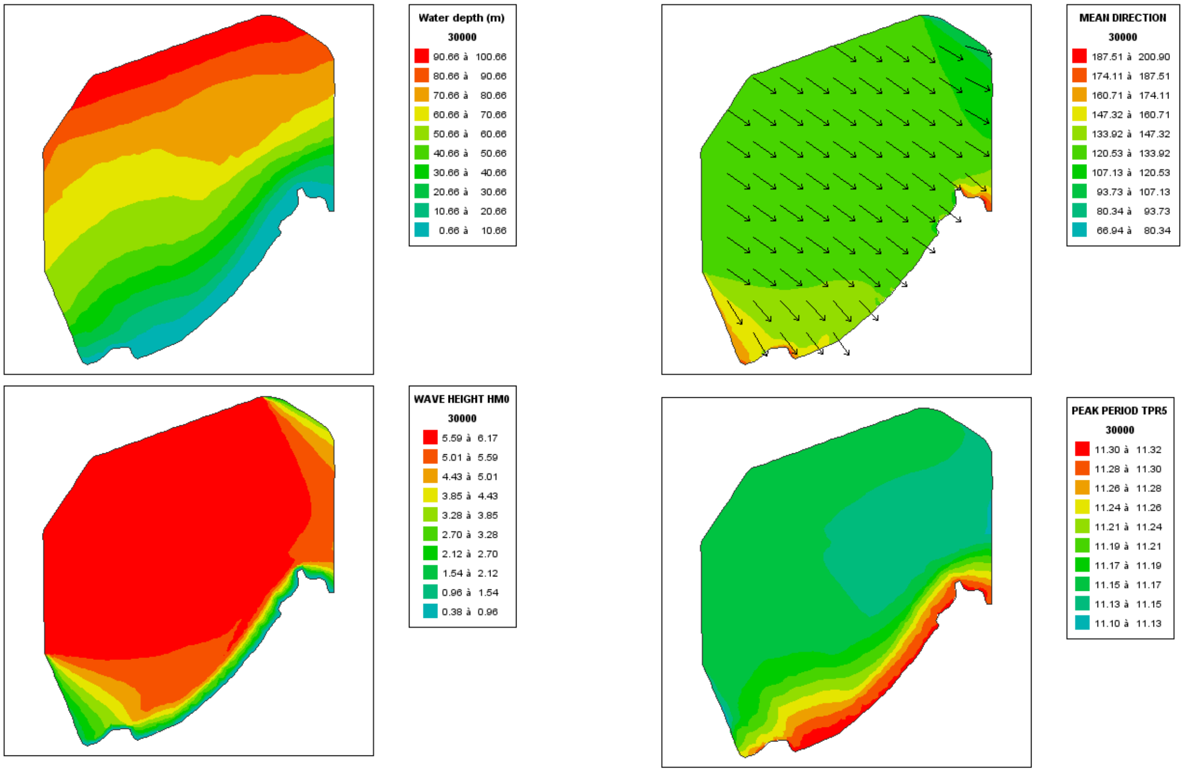
NE 1-year return period waves



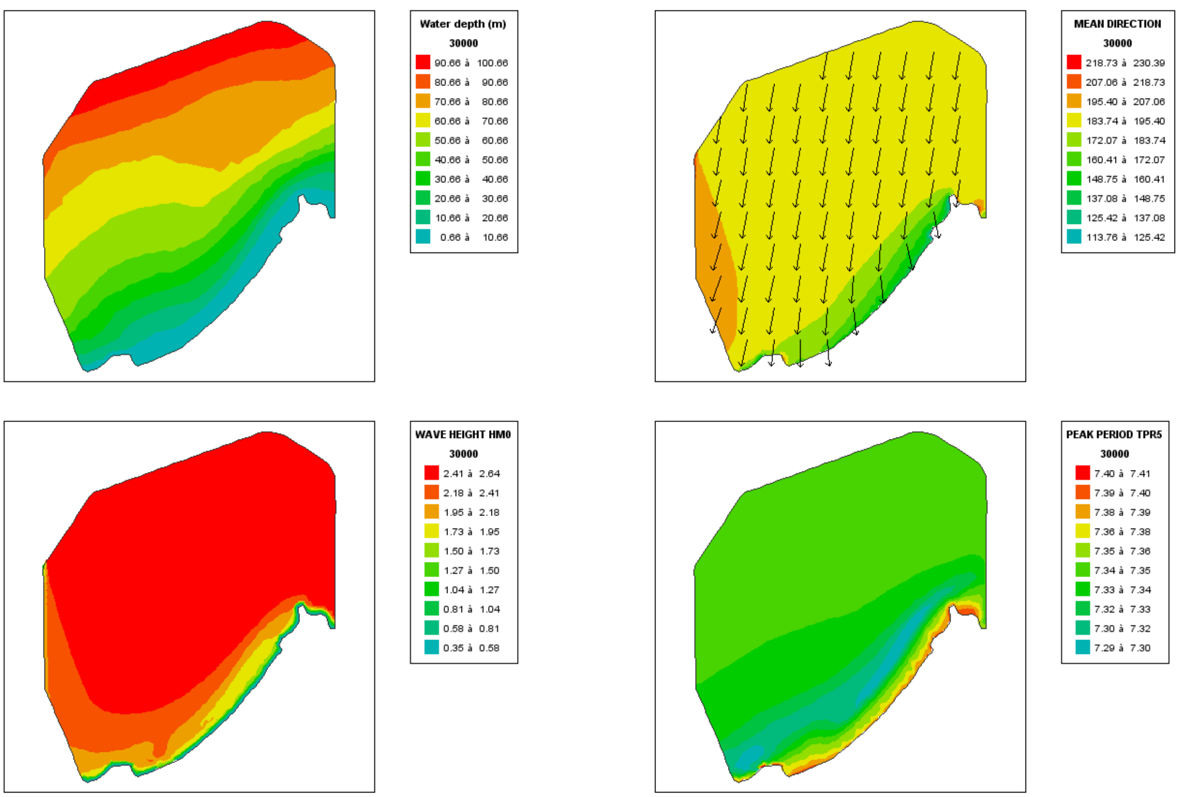
NE 100-year return period waves



NW 1-year return period waves



NW 100-year return period waves



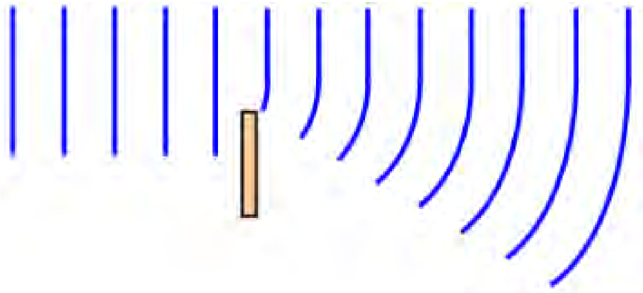
1% exceedance period waves

Figure 86 1% exceedance period waves - 190 degrees

Appendix H Sediment transport

In any case, we need to avoid that the overtopping waves contains sediments, the sediment infiltration through the breakwater, the coastal erosion and the coastal accretion inside the basin and to ensure that, we have to consider:

1. The diffraction of the waves when they arrive to the breakwater (to dimensioning the breakwater length).



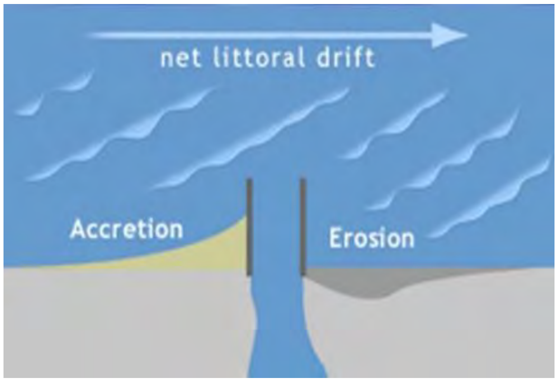
2. The orientation of the breakwater to avoid that the current can put sediments inside.

Here we have some examples about what we need to avoid:

1. Accumulation inside the port

Here the breakwater is badly orientated or not long enough because we can see that there are big amounts of sediment accumulated inside the basin, on the sides of the boats. That is going to cause a dredging operation.

2. Coastal erosion



In that case we can see a channel which has accumulated sediment due to the current direction but this accumulation is in the correct side (outside of the channel), so this means that the coastal protection is well positioned. However, in the other side, we see coastal erosion due to the currents and protections.

In our project we can have this case too, and therefore we propose two possible solutions:

a. Dredging of the offshore zone and put this material in the coastal.
b. Using a by-pass system to transfer accumulated sediments in the accretion zone towards the erosion zone below the channel. This solution will be more expensive.

A possible case of having coastal erosion for our project can be seen in the figure below. The particular position of breakwater (illustrated with brown) and wave direction of NE will lead to an erosion zone and accumulation zone.

Appendix I: Navigation channel design

Length

The total length of the approach channel needed for the ships to entre safe into the port is calculated in three parts. In the first part (L1) the ships need to slow down to 4 knots (2 m/s) because tugs can only fasten a vessel at a maximum of 4 knots. Also not slower than 4 knots because then the rudder control worsens. The second part (L2) is the necessary length to fasten the vessel and the third part (L3) is the length in which the vessel needs to stop. These components of the total length can be calculated with the following formulas (Ligteringen & Velsink, 2014):

$$L = L_1 + L_2 + L_3$$
$$L_1 = \Delta V_s * {}^3/_4 * L_s$$
$$L_2 = t_{fastening} * V_{s,min}$$
$$L_3 = 1.5 * L_s$$

Where:

ΔVs = change of velocity of the vessel is Vs.eff (taking the current velocity into account) minus Vs.min [m/s]
$t_{fastening}$ = time required for tying up tugboats [s]
Ls = length of the design vessel (LOA) [m]

The largest vessel is 370 m long (LOA) so this will be the length of the design vessel. To calculate ΔVs the $V_{s.eff}$ must be calculated with the $V_{s.min}$ which is 4 knots (2 m/s). The parameter 'u' stands for the maximum current velocity [m/s] which is in this case 0.16 m/s.

The change of velocity of the vessel becomes Vs.eff - Vs.min = 2.64 m/s. The time required for tying up the tugboats determines on the expertise of the crew and the environmental conditions. According to the textbook (Ligteringen & Velsink, 2014) the time will be in the range of about 10 minutes. Using the formulas above these results in a total length of the approach channel of:
673 m + 1200 m + 510 m = 2383 m .

Width

The PIANC-method give us this formula for the width of a channel:

W = Wbm + Wi + Wb

where,

W = total width [m]
WBM = basic width [m]
Wi = additional width [m]
WB = bank clearance [m]

Bs = width of the vessel [m]

WIDTH COMPONENT	OIL TANKER
Bs	**60 m**
Wbm = Bs*1.5	**90 m**
Wb = Bs*0.5	**30 m**
Wi = Bs*i:	**165**
Wia	6
Wib	24
Wic	15
Wid	0
Wie	30
Wif	12
Wig	6
Wih	12
Wii	60
Total W	**285 m**

Depth

The depth of the channel depends on a number of factors (draught, squat, trim, water level, bottom factors etc.). For this design phase the following formula can be used. (Ligteringen & Velsink, 2014):

$$h_{gd} = D - h_T + s_{max} + a + h_{net}$$

In which:

hgd = guaranteed depth [m]
D = draught of the ship [m]
hT = tidal elevation above reference level, below which no entrance is allowed [m]= 0.13 in our case.
Smax = maximum shrinkage due to squat and trim [m]
a = vertical motion due to wave response [m]
hnet = remaining safety margin or net under keel clearance [m]

In order to calculate the channel depth hgd the draught of the design vessel needs to be used. In this case 22.2 m of the oil tankers.

The parameter Smax can be calculated with the following formula:

$$s = 3{,}98 \cdot \frac{C_b}{30} \cdot k^{0{,}81} \cdot v_s{}^{2{,}08}$$

Where::
s = squat [m]
vs = vessel speed [m/s]
Cb = block coefficient (≈ 0,8 is assumed) [-]
k = blockage coefficient (As/Ach) [-]

The vessel speed is 3.2 m/s as recommendation and As = 22.2*60 = 1332 m.

The vertical motion of the ship due to wave response a = 1 m and the dredging tolerance (safety margin) which must be added is 0,75 m.

With that we have putting the squat formula inside the guaranteed depth formula with all the parameters, and we can see that the equation is iterative, so it was calculated with a programmable calculator:

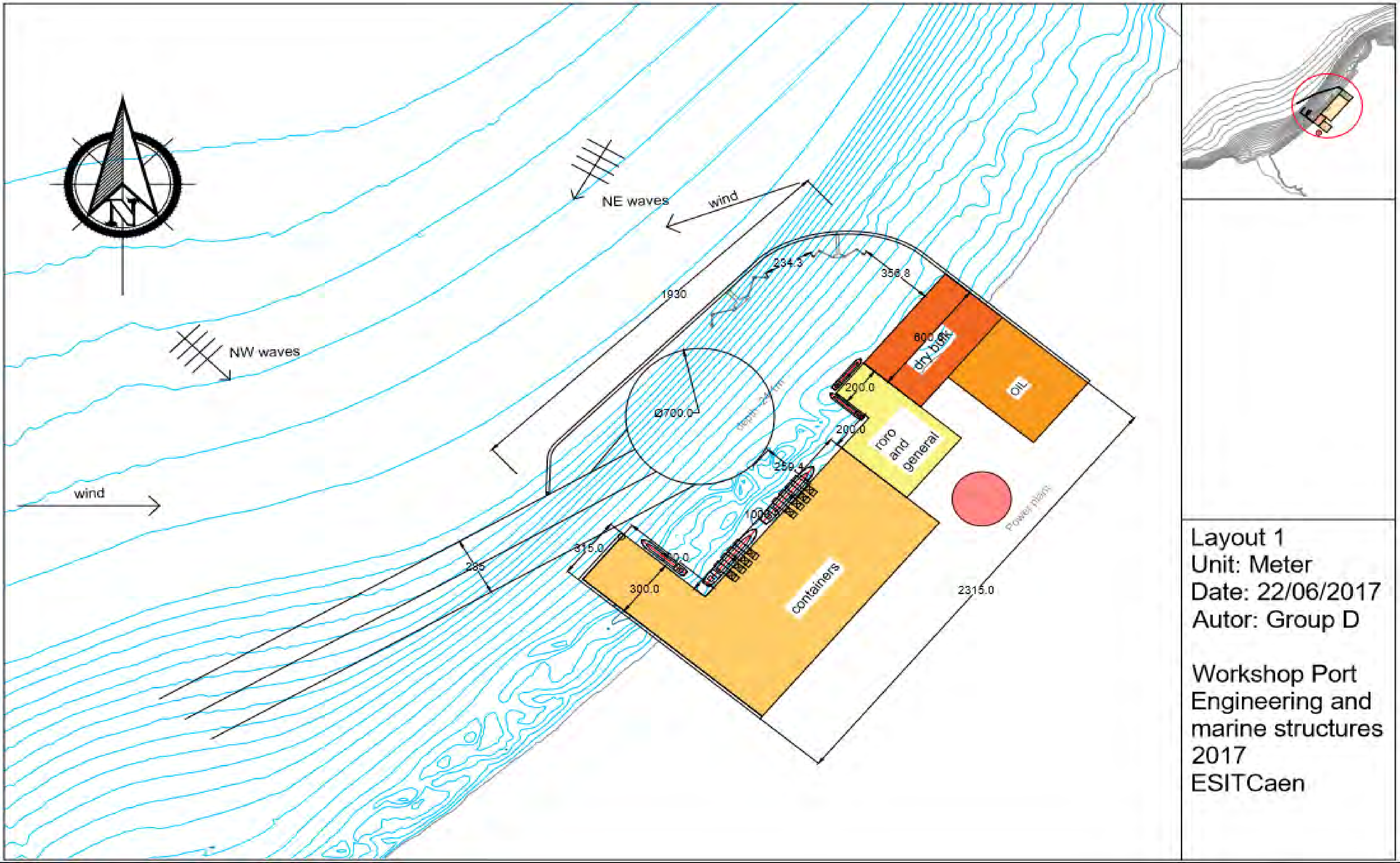
$$hdg = 22.2 - 0.13 + Smax + 1 + 0.75$$

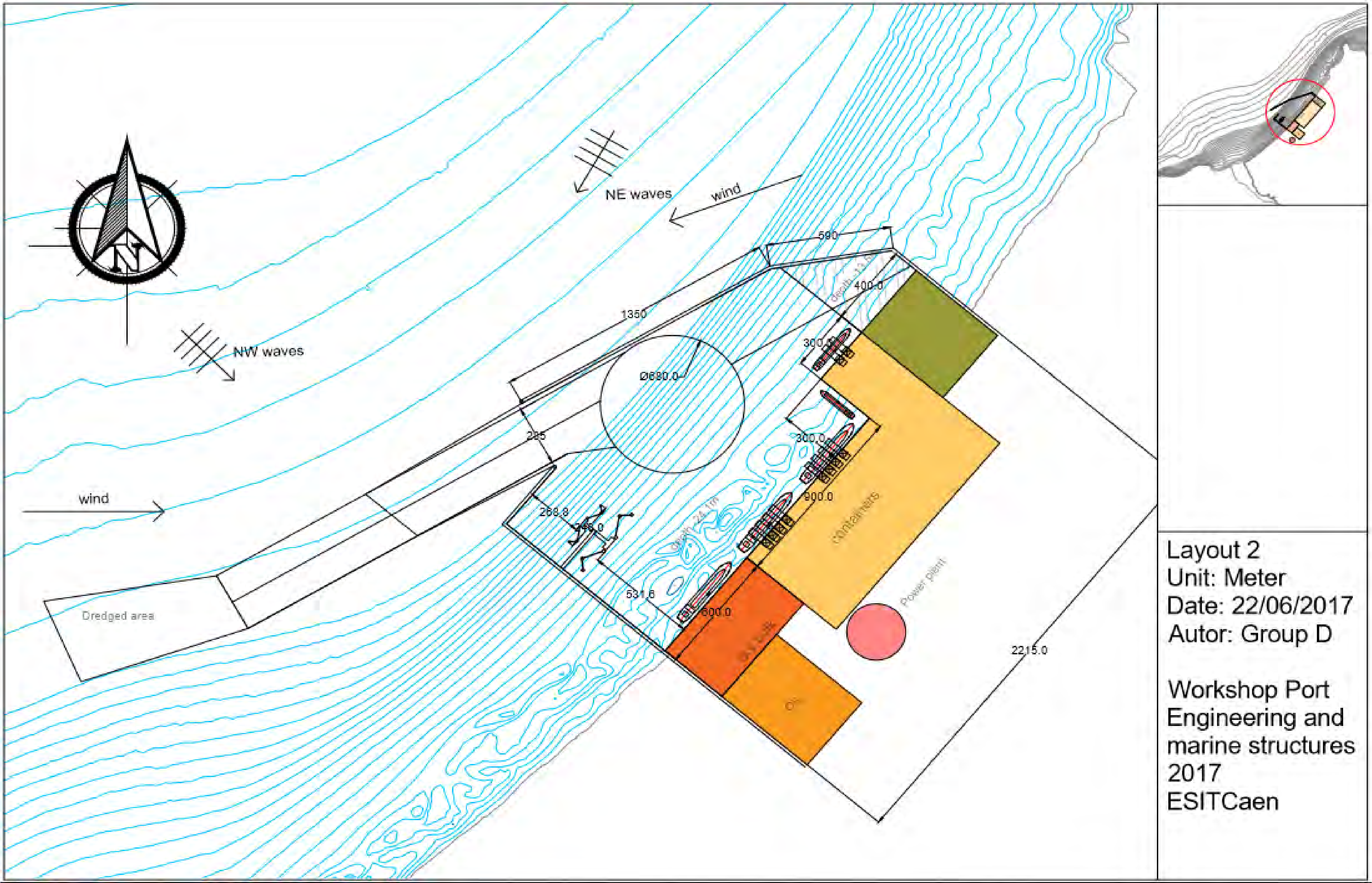
Finally, the depth of the channel is at least 24.1 m.

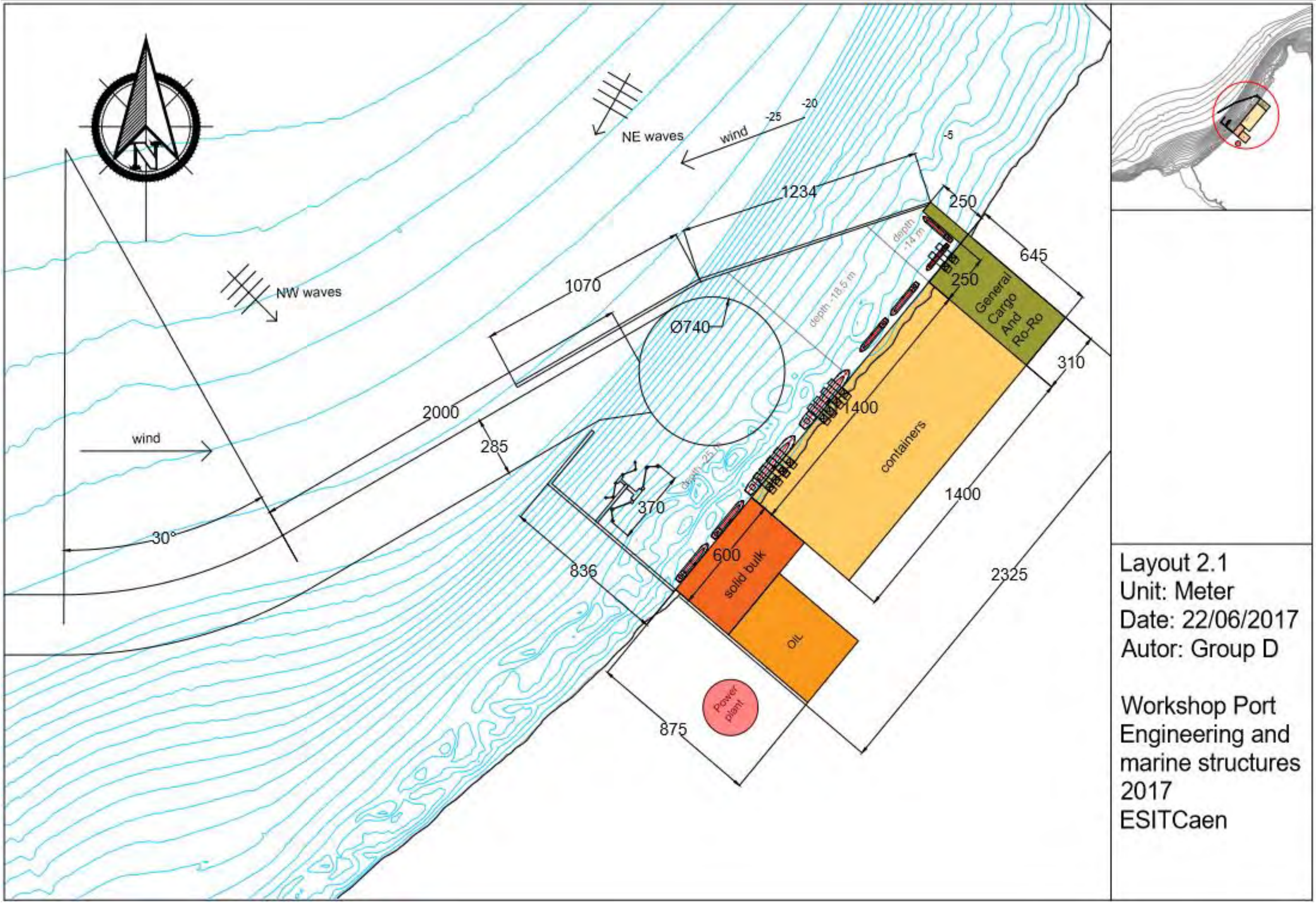
Summary

Dimensions	**[m]**
Length	2383
Width	285
Depth	24.1

Appendix J: Layout options




NE waves
wind
NW waves
wind
590
400.0
1350
Ø680.0
300.0
300.0
285
268.8
900.0
Containers
531.6
600.0
Dredged area
Power plant
2215.0
Oil
Layout 2
Unit: Meter
Date: 22/06/2017
Autor: Group D
Workshop Port
Engineering and
marine structures
2017
ESITCaen


NE waves
wind
-25
-20
-5
1234
250
645
250
1070
NW waves
Ø740
General Cargo And Ro-Ro
310
2000
1400
285
wind
containers
370
1400
30°
600
solid bulk
836
2325
OIL
Power plant
875
Layout 2.1
Unit: Meter
Date: 22/06/2017
Autor: Group D
Workshop Port Engineering and marine structures 2017
ESITCaen

Appendix K: Dredging Quantity

		Shallow Part	Intermediate Part	Deep Part
Surface (M2)	Zone (0;-5)	83 000,00	174 000,00	453 000,00
	Zone (-5;-10)	3 500,00	187 000,00	400 000,00
	Zone (-10;-15)		28 000,00	240 000,00
	Zone (-15;-20)			328 000,00
	Zone (20;-25)			1 035 000,00
Dredging volume (M3)	Zone (0;-5)	913 000,00	2 697 000,00	9 784 800,00
	Zone (-5;-10)	27 300,00	1 963 500,00	6 640 000,00
	Zone (-10;-15)		154 000,00	2 784 000,00
	Zone (-15;-20)			1 836 800,00
	Zone (20;-25)			2 173 500,00
Dredging volume in Rocks (M3)	Zone (0;-5)	0,00	0,00	906 000,00
	Zone (-5;-10)	0,00	0,00	0,00
	Zone (-10;-15)		0,00	0,00
	Zone (-15;-20)			0,00
	Zone (20;-25)			0,00
Dredging volume in Sand (M3)	Zone (0;-5)	913 000,00	2 697 000,00	8 878 800,00
	Zone (-5;-10)	27 300,00	1 963 500,00	6 640 000,00
	Zone (-10;-15)		154 000,00	2 784 000,00
	Zone (-15;-20)			328 000,00
	Zone (20;-25)			0,00
Dredging volume in Clay (M3)	Zone (0;-5)	0,00	0,00	0,00
	Zone (-5;-10)	0,00	0,00	0,00
	Zone (-10;-15)		0,00	0,00
	Zone (-15;-20)			1 508 800,00
	Zone (20;-25)			2 173 500,00

Total Dredging Volume in Rock (M3)	906 000,00
Total Dredging Volume in Sand (M3)	24 385 600,00
Total Dredging Volume In Clay (M3)	3 682 300,00
Total Dredging Volume (M3)	**28 973 900,00**

Appendix L: Total costs for MCA & Specified dredging cost

	Layout 1	Layout 2	Layout 2mod
Breakwater Cost	264 000,00 €	322 000,00 €	267 000,00 €
Dredging approximative cost	348 000,00 €	268 000,00 €	290 000,00 €
Total of Breakwater and Dredging	612 000,00 €	590 000,00 €	557 000,00 €

Notes			
Price Notes	2	3	5

		Layout 1			Layout 2			
		Shallow water	Intermediate Water	Deep Water	Shallow water	Intermediate Water	Deep Water	S
Surface (M2)	Zone (0;-5)			464 730,00			482 000,00	
	Zone (-5;-10)			640 000,00	119 000,00		541 000,00	
	Zone (-10;-15)			700 000,00	12 000,00		317 000,00	
	Zone (-15;-20)			850 000,00			229 000,00	
	Zone (20;-25)			267 000,00			928 000,00	
Dredging volume (M3)	Zone (0;-5)			10 038 168,00			10 411 200,00	
	Zone (-5;-10)			10 624 000,00	714 000,00		8 980 600,00	
	Zone (-10;-15)			8 120 000,00	30 000,00		3 677 200,00	
	Zone (-15;-20)			5 610 000,00			1 511 400,00	
	Zone (20;-25)			427 200,00			1 484 800,00	
Total Dredging Volume (M3)		34 819 368,00			26 809 200,00			
Cost Estimation (€)		348 193 680,00 €			268 092 000,00 €			

	Layout 2mod		
	Shallow water	Intermediate Water	Deep Water
0	83 000,00	174 000,00	453 000,00
0	3 500,00	187 000,00	400 000,00
0		28 000,00	240 000,00
0			328 000,00
0			1 035 000,00
0	913 000,00	2 697 000,00	9 784 800,00
0	27 300,00	1 963 500,00	6 640 000,00
0		154 000,00	2 784 000,00
0			1 836 800,00
0			2 173 500,00

	28 973 900,00

	289 739 000,00 €

Appendix M: Breakwater design

Armour stability

For armour stability we used Hudson equation modified by CLI.

$$V = \frac{H_S^3}{K_D \times \Delta^3 \times \cot \propto}$$

Where,

Hs is the significant wave height

K_D is the stability coefficient

Δ is the relative density

α is the slope of the breakwater and

V is the needed volume of one accropode unit

Rear-side stability

The rear-side stability was calculated with the Van Gent formula

$$Dn_{50} = 0,008 \left(\frac{Sd}{\sqrt{N}}\right)^{-1/6} \left(\frac{u_{1\%}T_{m-1}}{\sqrt{\Delta}}\right) cot^{2,5/6}(\alpha) \left(1 + 10exp\left(\frac{-R_{cr}}{H_s}\right)^{1/6}\right)$$

Where,

Hs is the significant wave height

Δ is the relative density

Sd is the damage level

N is the number of waves

$u_{1\%}$ is the maximum velocity that is exceeded by 1% of the incident waves

T_{m-1} wave period

α is the rear-slope of the breakwater

Rcr is the freeboard and

D_{n50} is the mean diameter of the rear-side units

Berm stability
To dimensioning the size of the berm units we used the formula 5.190 from the Rock Manual.

$$\frac{H_s}{\Delta D n_{50}} = \left(-0,6 + 5,8\left(\frac{h_t}{h}\right)\right) N_{od}^{0,19}$$

Where,

Hs is the significant wave height

Δ is the relative density

ht is the distance between water level and the top of the berm

h is the depth of the water

Nod is the damage number and

D_{n50} is the mean diameter of the berm units

General parameters			
Tr (años)	100	100	100
Profundidad máxima (mW)	-25	-16	-25
Profundidad mínima (mW)	0	0	0
Tr nivel (años)	1	1	1
Nivel máximo (m Wh)	0,66	0,66	0,66
Nivel mínimo (m Wh)	0,13	0,13	0,13
Tr oleaje (años)	100	100	100
Hs diseño (m)	5,60	5,00	5,60
Tp ap (s)	11	11	11
Increase water level in 85 years	0,6	0,6	0,6

Rubble mound breakwater						
Parameter	Section 1	Section 2	Head	Unit	Commentary	Source
Hs	5,60	5,00	5,60	m	Significant wave height	ROM 1.0 p 107
hm	25,66	16,66	25,66	M^-1	Water depht	
L0	1928	1928	1928	m	Longitud de onda en AP	Teoría lineal
k	0,01	0,01	0,01	M^-1	Número de onda	Teoría lineal
L	440	440	440	m	Longitud de onda AS	Teoría lineal
L	440	440	440	m	Longitud de onda AS	Teoría lineal
aux	0,00	0,00	0,00	m	Auxiliar para iteración	-
Fc/H_*	1	1	1	-	>= 1,0	ROM 1.0 p 108
Ft/H_*	0,60	0,60	0,60	-	>=0,60	ROM 1.0 p 108
n	3	3	3	-	numero de piedras (minimo recomendado 3)	CEM p VI-5-131
k	1,1	1,1	1,1	-	coeficiente de capa	CEM T VI-5-51

B	5,24	4,76	5,72	m	Anchura mínima de coronamiento	CEM ec VI-5-116
Slope	0,40	0,40	0,40	m/m	Pendiente de enrocado a barlomar	ROM 1.0 p 106
Alfa	36,86	36,86	36,86	º	Ángulo de enrocado a barlomar	ROM 1.0 p 106
cot(alfa)	1,33	1,33	1,33	-	Cotangente of alfa	Cálculo
Rho w	1025	1025	1025	kg/m³	Water density	RM p 564
Rho r	2350	2350	2350	kg/m³	Croncreteunits density	RM p 564
Delta hormigón	1,29	1,29	1,29	-	Relative density	RM p 564
Kd	15,0	15,0	15,0	-	Kd tronco, non breaking, accropode, 1 layer	CLI
H1/10	7,11	6,35	7,11	m	H1/10 = 1,27*Hs	RM p 564
Weight	9	7	12	ton	Weight of the acropode unit	RM p 564
Volume	4,06	2,89	5,00	m³	Volume of accropode unit	Cálculo
Height	32,09	20,89	30,55	m	height of the breakwater	Cálculo
Iribarren	6,65	7,04	6,65			ROM 1.0

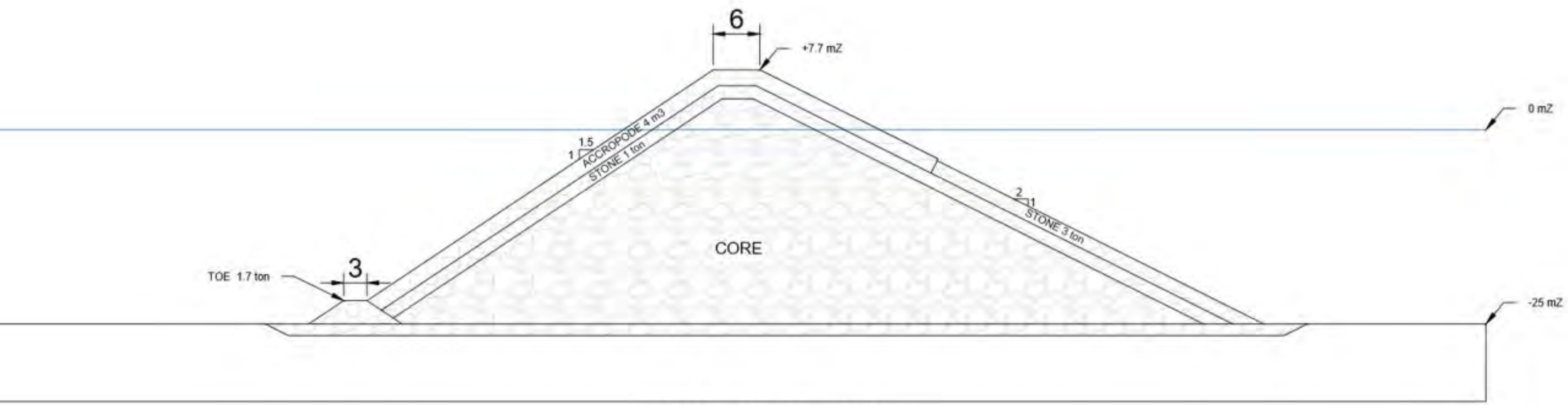
Toe (Rock Manual)						
Parameter	Section 1	Section 2	Head	Unit	Commentary	Source
Hs	5,60	5,00	5,60	m	Significant wave height	RM p 623 Ec 5.187
Delta	2	2	2	-	Relative density	RM p 623 Ec 5.187
Dn50	0,69	0,67	0,69	m	Nominal mean diameter to toe units	RM p 623 Ec 5.187
W 50	0,86	0,80	0,86	ton	Nominal mean weight to toe units	RM p 623 Ec 5.187
Alt. Berma	3,00	3,00	3,00	m	Toe height	Constructivo
Ancho Berma	3,00	3,00	3,00		Toe width at the top	ROM 1.0 p 107
kt	0,91	0,91	0,91		Layer coefficient	RM p 126 Tab 3.9
n	1	1	1		Number of layers	RM p 124 Ec 3.25
r	0,63	0,61	0,63		Tickness of one layer	
d	22,13	13,13	22,13		Distance between the top of the toe and the water surface	
h	25,13	16,13	25,13		Distance between the surface water and the bottom of the toe	
Nod	2	2	2		Damage level	

Parameter	Section 1	Section 2	Head	Unit	Commentary	Source
Underlayer (CLI)						
Hs	5,60	5,00	5,60	m	Significant wave height	RM p 623 Ec 5.187
Volume of unit	4	3	5	m3	Volume of the unit	CLI
Unit height	2,27	2,07	2,45	m	height of the unit	CLI
Unit mass	9400	7050	11750	kg	Mass of the unit	Calculation
ECS	1,59	1,44	1,71	m	Equivalent cube size	CLI
r_armour	2,05	1,86	2,21	m	Armour thikness	CLI
Packing density	0,64	0,65	0,64		Armour concrete consumption and coverage	CLI
consumption	1,021	0,93	1,098	m3/m3	Armour concrete consumption and coverage	CLI
Number of units	0,255	0,31	0,22	u/m3	Armour concrete consumption and coverage	CLI
Porosity	50,12	50,00	50,24	%	Armour concrete consumption and coverage	CLI
NLL Standard	0,67	0,50	0,84	tons	Standard Nominal lower limit of Filter stone underlayer	CLI
NLL min	0,50	0,40	0,60	tons	Minimal Nominal lower limit of Filter stone underlayer	CLI
NLL max	0,90	0,70	1,10	tons	Maximal Nominal lower limit of Filter stone underlayer	CLI
NUL Standard	1,34	1,01	1,68	tons	Standard Nominal upper limit of Filter stone underlayer	CLI
NUL min	0,90	0,70	1,20	tons	Minimal Nominal upper limit of Filter stone underlayer	CLI
NUL max	1,70	1,30	2,20	tons	Maximal Nominal upper limit of Filter stone underlayer	CLI
Thickness for underlayer	1,30	1,30	1,30	m	Thickness of standard underlayer. Thickness coefficient kt=1,15	CLI

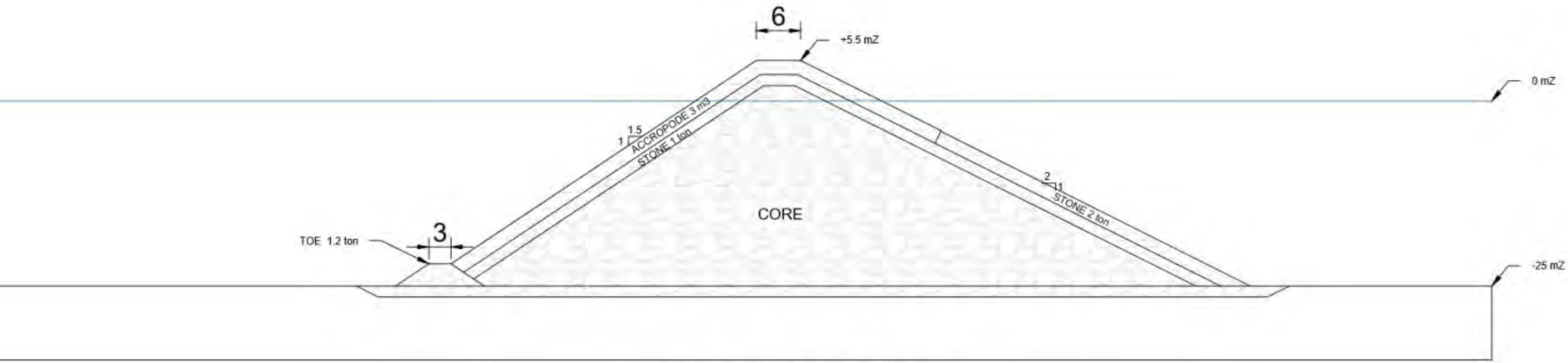
Parameter	Section 1	Section 2	Head	Unit	Commentary	Source
Core						
M50 upper	300	300	300	kg		CEM F VI-5-55
M50 lower	5	5	5	kg		CEM F VI-5-55

Parameter	Section	Section	Head	Unit	Commentary	Source
Overtopping						

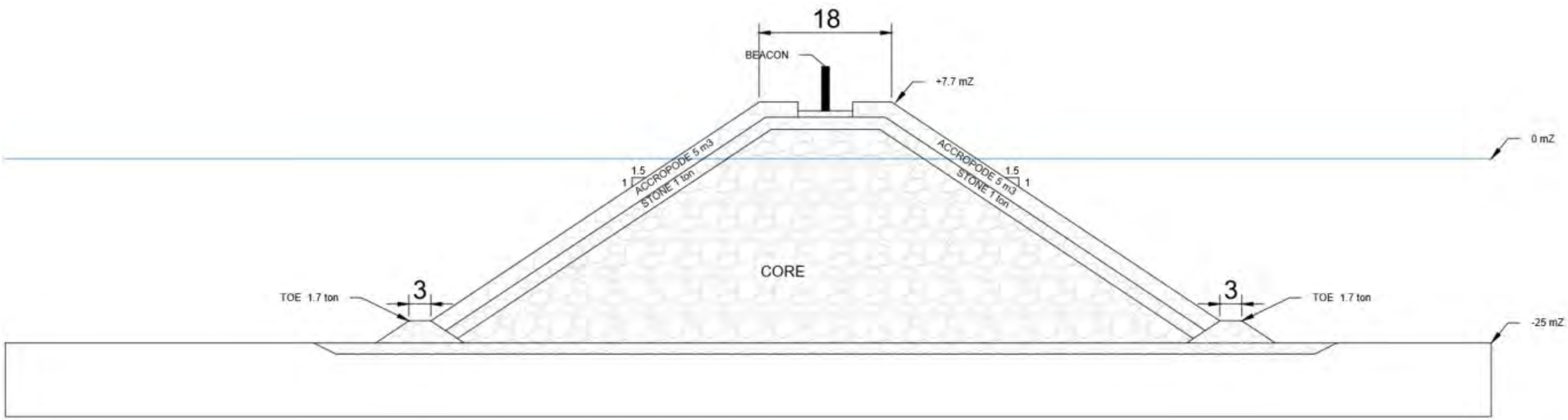
	1	2				
Rc	6,43	4,2	4,9	m	Freeboard (distance between the top of the breakwater and the water level)	Overtopping manual
Hmo	5,6	5,0	5,6	m	Significant wave height	Overtopping manual
Gamma f	0,5	0,5	0,5		Coefficient asociated to the material	Overtopping manual
Gamma betta	0,9472	0,7195	0,7195		Coefficient asociated to the direction of the waves	Overtopping manual
q	0,01	0,01	0,01	l/sec	Discharge	Overtopping manual
Angle	16	85	85	°	Angle between the direction of the wave and the perpendicular to the breakwater	Overtopping manual
Crown level	7,7	5,5	6,1	mZ	Crown level	Overtopping manual
Rear side						
Parameter	**Section 1**	**Section 2**	**Head**	**Unit**	**Commentary**	**Source**
Dn50	1,04	0,92	0,93	m	Nominal mean diameter of the rear side units	Van Gent
M50	2,97	2,06	2,14	tons	Nominal mean weight of the rear side units	Van Gent
phi	0,46	0,46	0,46	rad	Rear side slope	Van Gent
Sd	2	2	2		Damage level	Van Gent
n	1500	1500	1500		Number of waves	Van Gent
Tm-1	11	11	11	s	Peak period	Van Gent
u1%	9,39	7,98	8,12	m/s	velocity exceeded by 1% of the waves	Van Gent
Delta	1,59	1,59	1,59		Relative density	Van Gent
g	9,81	9,81	9,81	m/s2	gravity	Van Gent
Gammaf-c	0,45	0,45	0,45			Van Gent
z1%	10,56	7,22	8,02	m		Van Gent
p	1,76	1,76	1,76		Parameter to compare with Iribarren number	Van Gent
c0	1,45	1,45	1,45		Empirical coeficient	Van Gent
c1	5,10	5,10	5,10		Empirical coeficient	Van Gent
c2	4,48	4,48	4,48		Empirical coeficient	Van Gent
r	2,1	1,8	1,9	m	Layer tickness	Van Gent



TRUNK SECTION 1



TRUNK SECTION 2


18
BEACON
+7.7 mZ
0 mZ
ACCROPODE 5 m3
STONE 1 ton
1.5
1
ACCROPODE 5 m3
STONE 1 ton
1.5
1
CORE
3
TOE 1.7 ton
3
TOE 1.7 ton
-25 mZ

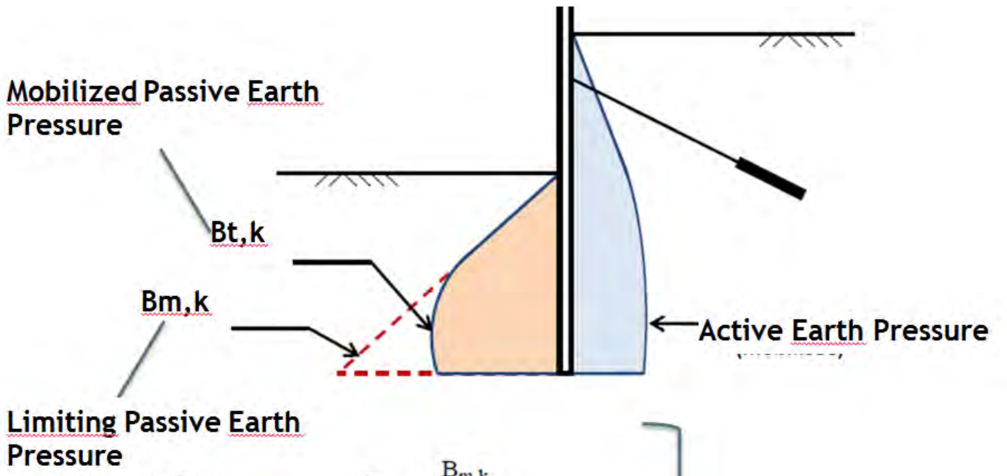
Appendix N: Quay wall checks

Type
SLS
ULS

Wall1							
PHASE N°	Type	M,d maximal [kNm/ml]	V,d maximal [kN/ml]	Design force link. anchor n°1...	Check Pass. press.	Check Vert. Eq. [kN/ml]	Check Kranz
1	SSIM	0,00	0,00	0,00	OK	52,56	OK
2	SSIM	-107,68	-73,49	28,46	OK	-144,03	OK
3	SSIM	-107,68	-73,49	28,46	OK	-144,03	OK
4	SSIM	-171,81	-120,44	43,35	OK	-274,67	OK
5	SSIM	-4283,32	1170,60	603,00	OK	-1178,98	OK
6	SSIM	-4283,32	1170,60	603,00	OK	-1178,98	OK
7	SSIM	-4283,32	1170,60	603,00	OK	-1178,98	OK
8	SSIM	-4282,21	1170,46	713,01	OK	-1176,74	OK
9	SSIM	-4328,07	1183,52	695,16	OK	-1218,73	OK
10	SSIM	-4372,73	1199,60	658,37	OK	-1323,94	OK
Extrema		-4372,73	1199,60	713,01		-1323,94	

Wall 2							
PHASE N°	Type	M,d maximal [kNm/ml]	V,d maximal [kN/ml]	Design force link. anchor n°1...	Check Pass. press.	Check Vert. Eq. [kN/ml]	Check Kranz
1	SSIM	0,00	0,00	0,00	OK	14,46	OK
2	SSIM	10,94	18,56	28,46	OK	0,92	OK
3	SSIM	10,94	18,56	28,46	OK	0,92	OK
4	SSIM	15,87	24,92	43,35	OK	-0,79	OK
5	SSIM	149,16	-364,70	603,00	OK	-249,16	OK
6	SSIM	149,16	-364,70	603,00	OK	-249,16	OK
7	SSIM	149,16	-364,70	603,00	OK	-249,16	OK
8	SSIM	-226,30	-474,71	713,01	OK	-321,11	OK
9	SSIM	-223,45	-464,49	695,16	OK	-314,96	OK
10	SSIM	-217,58	-443,41	658,37	OK	-304,03	OK
Extrema		-226,30	-474,71	713,01		-321,11	

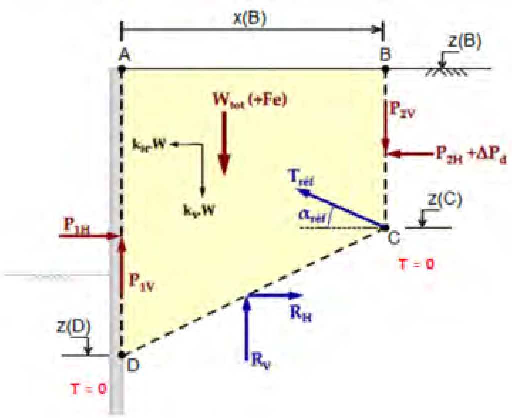
ULS checks



$$B_{t,d} = \gamma_E \cdot B_{t,k} \qquad B_{m,d} = \frac{B_{m,k}}{\gamma_R}$$

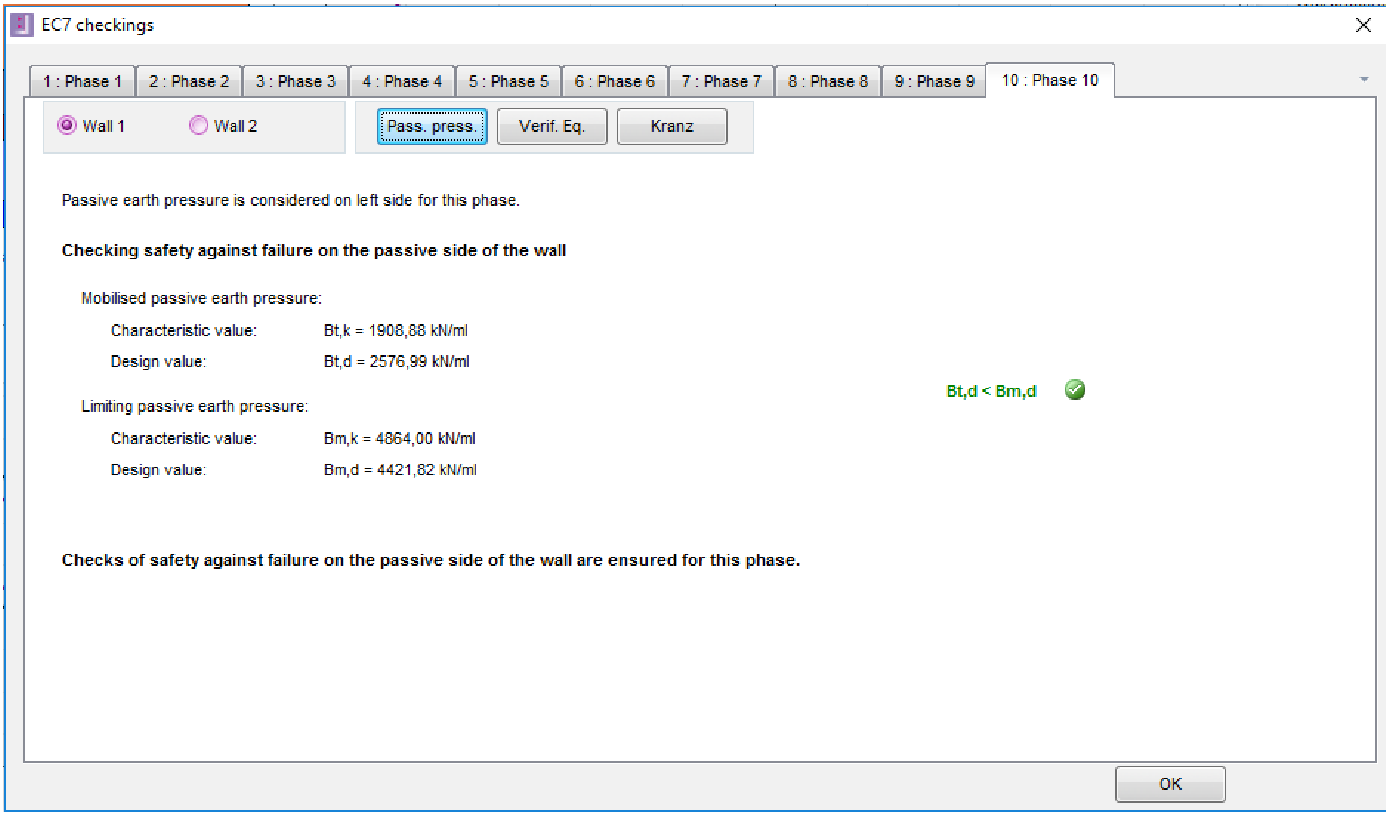
Phase	YE	YR	SF
Transitory Phase	1.35	1.1	1.485
Durable Phase	1.35	1.4	1.89

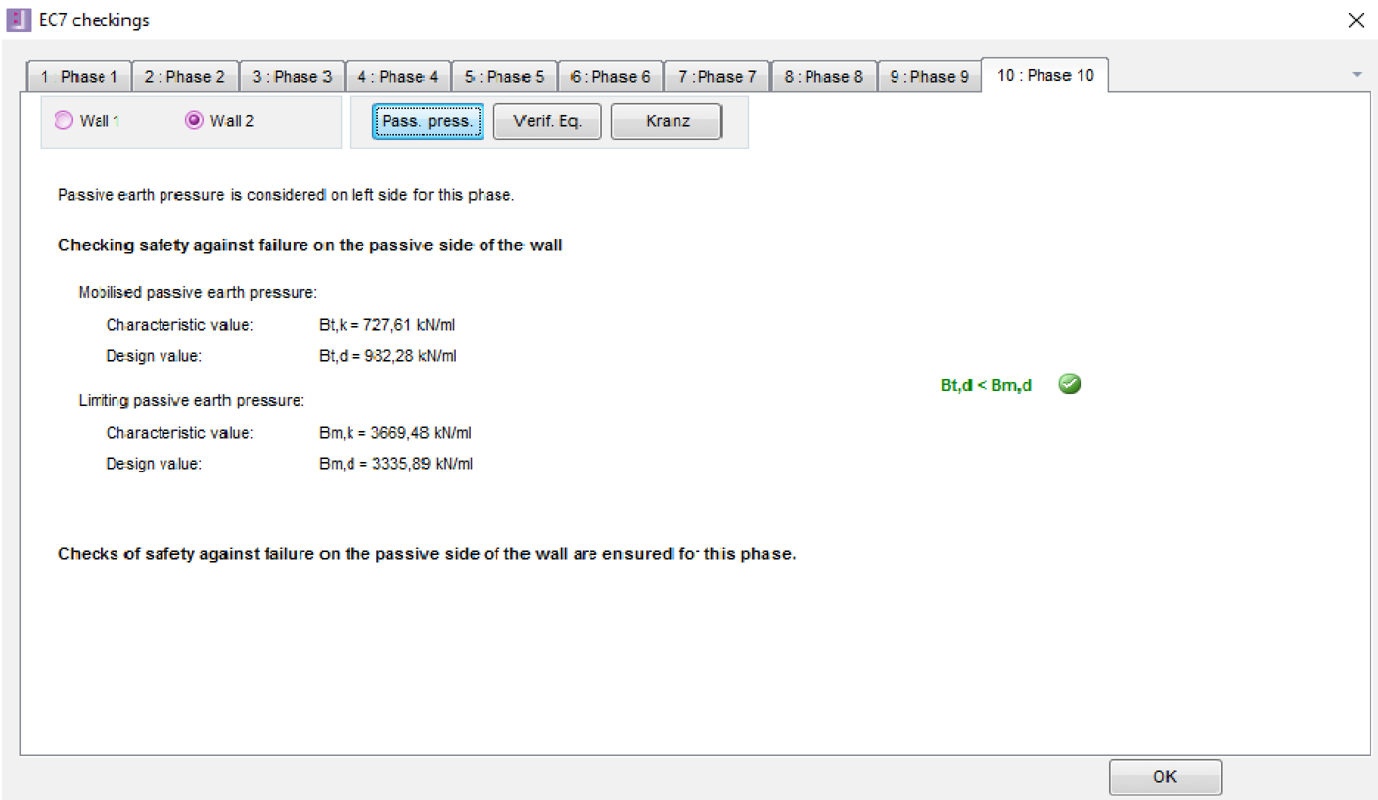
$$\frac{B_{t,k}}{B_{m,k}} \leq \gamma_E \cdot \gamma_R = SF$$



$$T_{réf,d} = \gamma_E \cdot T_{réf} \leq T_{dsb,d} = \frac{T_{dsb}}{\gamma_R}$$

$\gamma_R = 1{,}10 \quad \gamma_E = 1{,}35$


EC7 checkings
1 : Phase 1
2 : Phase 2
3 : Phase 3
4 : Phase 4
5 : Phase 5
6 : Phase 6
7 : Phase 7
8 : Phase 8
9 : Phase 9
10 : Phase 10
Wall 1
Wall 2
Pass. press.
Verif. Eq.
Kranz
Passive earth pressure is considered on left side for this phase.
Checking safety against failure on the passive side of the wall
Mobilised passive earth pressure:
Characteristic value: Bt,k = 1908,88 kN/ml
Design value: Bt,d = 2576,99 kN/ml
Bt,d < Bm,d
Limiting passive earth pressure:
Characteristic value: Bm,k = 4864,00 kN/ml
Design value: Bm,d = 4421,82 kN/ml
Checks of safety against failure on the passive side of the wall are ensured for this phase.
OK


EC7 checkings
1 : Phase 1
2 : Phase 2
3 : Phase 3
4 : Phase 4
5 : Phase 5
6 : Phase 6
7 : Phase 7
8 : Phase 8
9 : Phase 9
10 : Phase 10
Wall 1
Wall 2
Pass. press.
Verif. Eq.
Kranz
Passive earth pressure is considered on left side for this phase.
Checking safety against failure on the passive side of the wall
Mobilised passive earth pressure:
Characteristic value: Bt,k = 727,61 kN/ml
Design value: Bt,d = 932,28 kN/ml
Bt,d < Bm,d
Limiting passive earth pressure:
Characteristic value: Bm,k = 3669,48 kN/ml
Design value: Bm,d = 3335,89 kN/ml
Checks of safety against failure on the passive side of the wall are ensured for this phase.
OK

Appendix O : Breasting and Mooring Dolphin

Distances of Breasting and Mooring Structures

For crude oil tankers:

$$Average = (0.3\ x\ L_{Longest}; 0.4\ x\ L_{Shortest})$$
$$Average = (0.3\ x\ L_{Lijmiliya}; 0.4\ x\ L_{Front\ Warrior}) = 106\ m$$

For product tankers:

$$Dolphin\ Distance_1(for\ longer\ ships) = \ Average = (0.3\ x\ L_{Longest}; 0.4\ x\ L_{Deepest})$$
$$Average = (0.3\ x\ L_{Moray}; 0.4\ x\ L_{High\ Venture}) = 71\ m$$

$$Dolphin\ Distance_2(for\ shorter\ ships) = \ 39\ m$$

Berthing Energy Calculations

The values for calculations are presented in the main report.

Abnormal Energy:

$$E_N = 0.5 \times M \times V_B{}^2 \times C_M \times C_E \times C_C \times C_S$$

$$E_A = F_S \times E_N$$

F_s – Global safety factor

Block Coefficient:

Typical values for tankers is 0.85.

$$C_B = \frac{M_D}{L_{BP} \times B \times D \times \rho_{SW}}$$

Added Mass Coefficient:

$$C_m = 1.2 + 0.12\ x \frac{D}{W_d - D}$$

Eccentricity Coefficient:

$$C_E = \frac{K^2 + R^2\cos^2\phi}{K^2 + R^2}$$

$$R = \sqrt{y^2 + \left[\frac{B}{2}\right]^2}$$

$$\Phi = \pi/2 - \alpha - \mathrm{asin}(B/2R)$$

$$x + y = \frac{L_{BP}}{2}$$

$$x = 0.33\ x\ L_{BP}$$

Correction for Manufacturer Tolerance:

The correction is based on berthing angle temperature factor. AF was taken to be 1.00 (10°) and TF to be 0.969 (30°).

$$E_A = 1.1 \cdot \frac{SF \cdot E_N}{A_F \cdot T_F}$$

Fender Design Calculations

Allowable hull pressures for different ship types (P_R):

Allowed Hull Pressures	
Type of Vessel	**Hull Pressure KN/m^2**
Tankers	150~250
ULCC & VLCC(Coastal Tankers)	250~350
Product & Chemical Tankers	300~400
Bullk Carriers	150~250
Post-Panamax Container Ships	200~300
Panamax Container Ships	300~400
Sub-Panamax Container Ships	400~500
General Cargo	300~600
Gas Carriers	100~200

Fender Height Estimation:

$$H = \frac{R_R}{W \ x \ P_R}$$

Structural Dolphin Design

Structural Pile Parameters:

The structural life of dolphins was assumed to be 50 years for corrosion allowance consideration.

$$f_a = \frac{N}{A} \qquad A = \frac{\pi}{4} \cdot [D^2 - (D - 2t)^2]$$

$$f_b = \frac{M}{W} \qquad W = \frac{I}{D/2} \qquad I = \frac{\pi}{64} \cdot [D^4 - (D - 2t)^4]$$

$$r = \sqrt{\frac{I}{A}}$$

Stress parameters required to check structural stability:

$$F_{xe} = 2CE\ t/D$$

$$F_{xc} = F_y \times [1.64 - 0.23(D/t)^{1/4}] \leq F_{xe}$$

$$F_{xc} = F_y \qquad \text{for } (D/t) \leq 60$$

$$F_y = Min(F_{xc}, F_{xe})$$

$$F_a = \frac{\left[1 - \frac{(Kl/r)^2}{2C_c^2}\right] F_y}{5/3 + \frac{3(Kl/r)}{8C_c} - \frac{(Kl/r)^3}{8C_c^3}} \quad \text{for } Kl/r < C_c$$

$$F'_e = F_a = \frac{12\pi^2 E}{23(Kl/r)^2} \quad \text{for } Kl/r \geq C_c$$

$$C_c = \left(\frac{2\pi^2 E}{F_y}\right)^{\frac{1}{2}}$$

$$F_b = 0.75\, F_y \qquad \frac{D}{t} \leq \frac{10{,}340}{F_y}$$

$$F_b = \left[0.84 - 1.74\frac{F_y D}{Et}\right] F_y \qquad \frac{10{,}340}{F_y} < \frac{D}{t} \leq \frac{20{,}680}{F_y}$$

$$F_b = \left[0.72 - 0.58\frac{F_y D}{Et}\right] F_y \qquad \frac{20{,}680}{F_y} < \frac{D}{t} \leq 300$$

Structural Checking:

$$\frac{f_a}{F_a} + \frac{C_m\sqrt{f_{bx}^2 + f_{by}^2}}{\left(1 - \frac{f_a}{F_e'}\right) F_b} \leq 1.0 \qquad (3.3.1\text{-}1)$$

$$\frac{f_a}{0.6F_y} + \frac{\sqrt{f_{bx}^2 + f_{by}^2}}{F_b} \leq 1.0 \qquad (3.3.1\text{-}2)$$

Since the cross-section is circular – $f_{bx} = f_{by} = f_b$

$$\frac{f_a}{F_a} + \frac{\sqrt{f_{bx}^2 + f_{by}^2}}{F_b} \leq 1.0 \qquad (3.3.1\text{-}3) \quad \text{When } \frac{f_a}{F_a} \leq 0.15$$

The Figure below presents the values for structural calculations.

Pile Dimensions/Structural Characteristics		
D_0 (m)	2.90	(diameter)
t (m)	0.06	(thickness)
Δ_c	0.004	(corrosion allowance)
D_1 (m)	2.896	(corroded diameter)
t_1 (m)	0.056	(corroded thickness)
L_e (m)	34.341	(unbraced/effective pile length)
D_1/t_1	51.489	(diameter-thickness ratio)
A (m^2)	0.535	(area)
W (m^3)	0.350	(section modulus/Z)
I (m^4)	0.506	(second moment of area)
r (m)	0.972	(radius of gyration)
Material properties		
F_y (MPa)	450	(steel yield strength of steel)
E (MPa)	210,000	(elastic modulus of steel)
Coefficients/Factors		
C	0.3	(critical elastic buckling coefficient)
K	2.1	(slenderness factor)
ff	1.0	(allowable stress factor)
Structural Loads		
N (kN)	150	(vertical-axial force)
M (kNm)	71,723	(bending moment)
Local Buckling Check		
f_{xe} (MPa)	2447	
f_{xc} (MPa)	450	for D/t<=60
F_y (MPa)	450	(local buckling stress)
Column Buckling Check		
C_c	96	
L_e/r	35	(slenderness)
Kl_e/r	74	(factored slenderness)
F_a (MPa)	166	(global buckling stress)
Bending Check		
F_b (MPa)	295	(bending stress)
f_a (MPa)	0.280	(intermediate stresses)
f_b (MPa)	205	
F'_e (MPa)	197	
C_m	0.85	
Factorial Check		
UC (3.3.1-1)	0.838	<=1
UC (3.3.1-2)	0.984	<=1
UC (3.3.1-3)	0.985	<=1
Max UC	0.985	<=1

Appendix P: Flume test

Parameter	Value	Unit	Scale value	Scaled value		Scaled value	Unit
Top	6	m	60	0,1		**10**	**cm**
Bottom	-20,66	m	60	-0,344		**-34,4**	**cm**
Berm height	3	m	60	0,05		**5**	**cm**
Berm width	3	m	60	0,05		**5**	**cm**
Underlayer tickness	1,3	m	60	0,02		**2,2**	**cm**
Armour tickness	2,05	m	60	0,03		**3,4**	**cm**
Distance between water level and starting of stones	4	m	60	0,07		**6,7**	**cm**
Slope	1,33						
Rear slope	2						
Berm slope	1,5						
Wave height	5,6	m	60	0,09		**9,3**	**cm**
Peak period	11	s	7,745966692	1,42		**1,4**	**s**
Unit armour mass	9400	kg	220695,6522	0,04		**42,6**	**gr**
Underlayer mass stone	1000	kg	216000	0,00		**4,6**	**gr**
Rearside mass stone	2600	kg	216000	0,01		**12,0**	**gr**
Berm mass stone	1640	kg	216000	0,01		**7,6**	**gr**
min Core mass stone	5	kg	216000	0,00		**0,0**	**gr**
max Core mass stone	300	kg	216000	0,00		**1,4**	**gr**
Crest level	7,7	mZ	60	0,13		**12,8**	**cm**

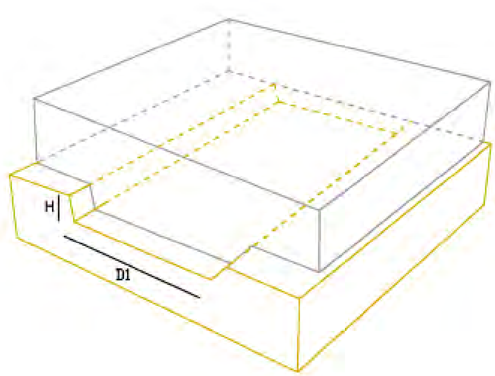
Appendix Q: Construction method (sequence)

Breakwater method:

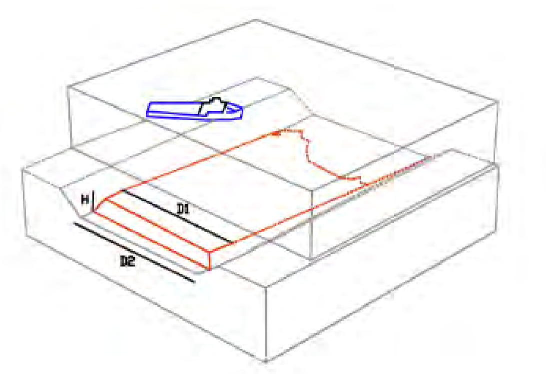
1. - Dredged. It need:

a. 1 hopper dredger to dredge the clay.
b. 1 hopper dredger to backfill with sand.
c. 1 cutter dredger to do a precise sea bed for the breakwater.

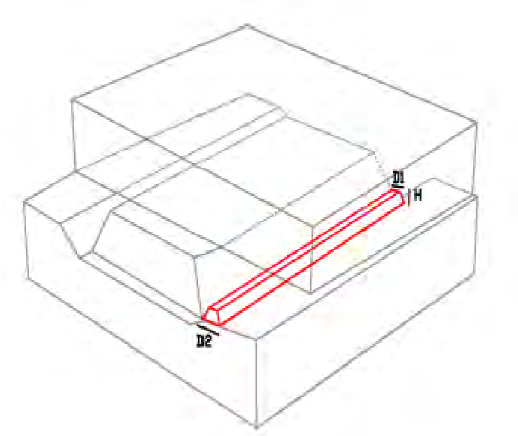
The soil to dredge must be flat to put the foundation there.



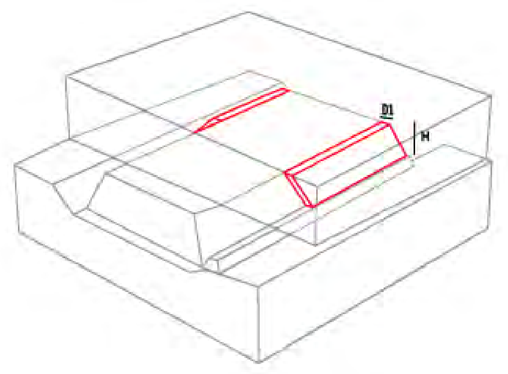
2. - Core maritime: after the foundation, the core is putted by layers with the material that has been dredge before, because it is enough valid.



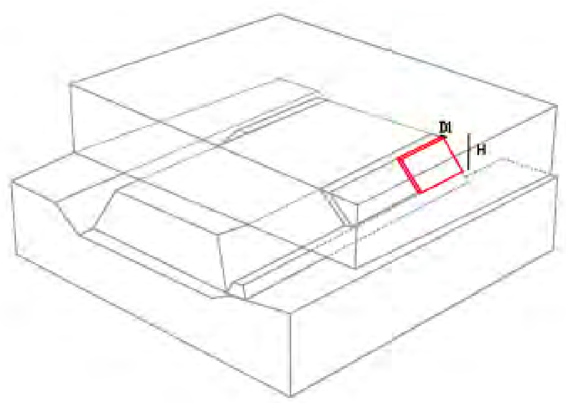
3. - Toe mound: it's an important part of the breakwater to decrease the energy of the waves what clash with the breakwaters. That's built with rocks.



4. - Under layer: the first layer after the core and it is a significant element to cover the core against possible landslides.

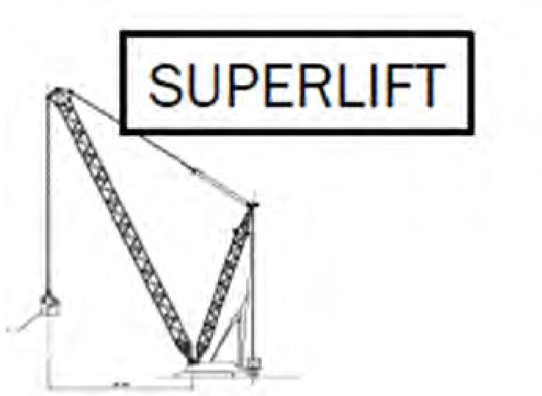


5. - The main armour (outer side) and the rear armour (inner side) is the last layer to protect our structure and must be built with special machinery.

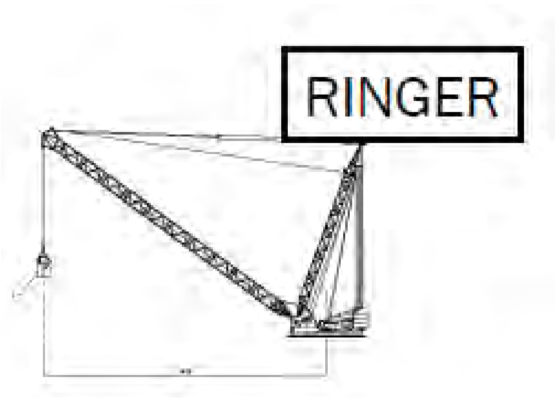


There is two different kind of machinery we can used for the main armour:

1. Superlift

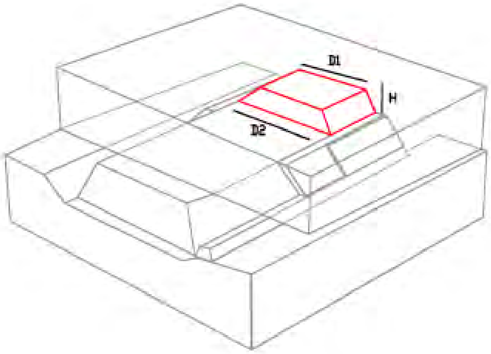


2. Ringer: Reacher than the superlift

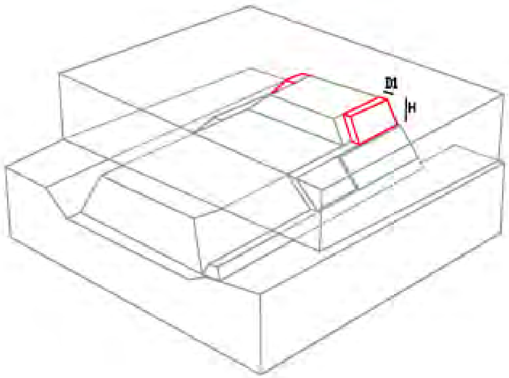


After these steps, we are out of water. Consequently, we have to do the same steps but out of the water

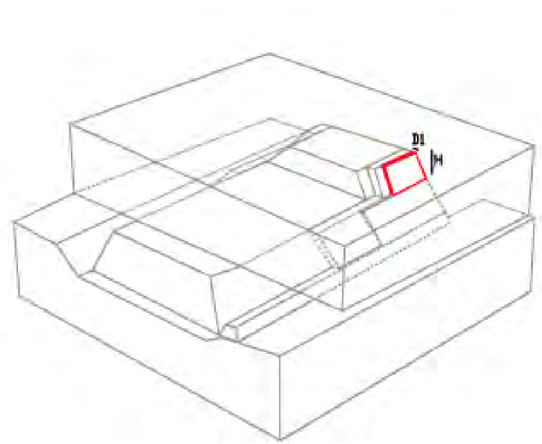
6. - Core terrestrial



7. - Under layer terrestrial



8. - Main armour and rear armour terrestrial



Dredging Method and Sequence

In order to dredge all the areas, we will use 3 different dredgers:

- Hopper Dredger
- Cutter Dredger
- Backhoe Dredger

According to the soil we have on our project, the hopper dredger is the best one. It doesn't need a barges to store the dredging materials, it cans deal with large swells compare to other dredger and furthermore, it has a very high productivity. Wi will use a 17 000 M3 THSD which have a productivity of more than 800 000 M3 per week. The draught loaded is around 11 meters so we will need another dredger to dredge from -13 m ZH to 0 m ZH.

For this area, we will use two different dredgers:

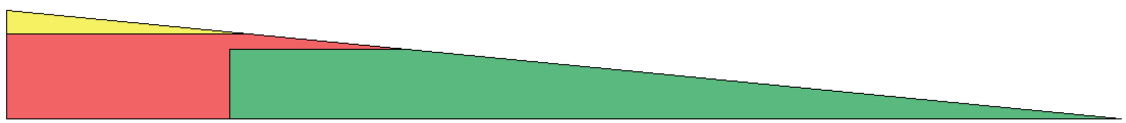
First, we will use a Backhoe dredger between – 7 m ZH and 0 m ZH. We will use a large one because there is a lot of dredging along the coast side. We will take one with a 40 M3 bucket which is the biggest we can find. It will need barges to store the dredged soil and the productivity is not as high as a hopper dredger: around 100 000 M3 for the one we will take.

After -7 m ZH, we will use one cutter dredger. Compare to the backhoe dredger, the cutter dredger has a very high production, more than 750 000 M3 per week. It has a draught of 5.5 m so it will be able to work with a depth of more than -7 m ZH.

Also, along the coast, there is some soft rocks (Volcanic Tuff) and consequently, the cutter dredger became even more important. And finally, it has a very high precision and this will be important to do the breakwater foundation. Effectively, we will dredge the clay and backfill after with sand but we will need to do a flat sea bed for the rubble mounds.

Consequently, we will use 3 different dredgers (you can see where they are going to work in the port in the graphic below (not on scale)):

- One Backhoe dredger along the coast between – 7 m ZH and 0 m ZH (yellow area),
- One Cutter dredger along the coast for the rocks, for the depth from -13 m ZH to 0 m ZH (Red Area),
- One Hopper dredger to remove the clay and all the soil after the depth of – 13 m ZH (Green Area)



In quantity, the backhoe dredger will dredge around 5 % of the quantity and they will need 15 weeks to do all the dredging at 100 000 M3 per week and without delays.

The cutter dredger will do around 40 % of the amount of dredging. Without delays and at 750 000 M3 per week, it will need 16 weeks and it will also need one more week to prepare the breakwater sea bed.

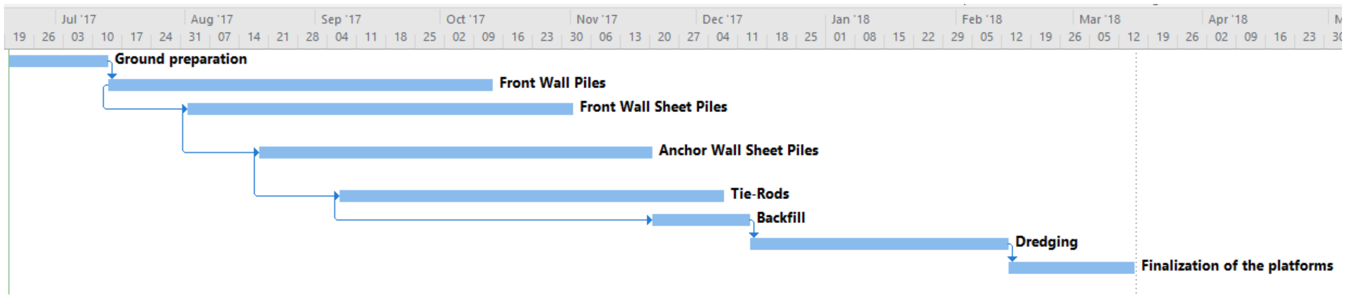
The Hopper dredger will dredge around 55 % of the quantity. Without delays and at 800 000 M3 per week, it will need 20 weeks. It will need also 1 more week to dredge the clay in the future location of our location.

However, we can dredge more with the cutter dredger and consequently, we will need 18 weeks to dredge without delays and 1 more weeks to do the breakwater sea bed.

Quay's construction sequence:

To do the quays we will follow an usual method because our quays are mostly in land:

1. Ground preparation for the engines: Levelling the ground and soil compaction.
2. Pile driving with a drummer. Because we are too close to water, we will use a drummer and not a drilling machine (we will need to use bentonite and it's not good for the environment). Driving until -35 m ZH. The top of every piles needs to be at + 2.80 m ZH.
3. Sheet pile driving with a drummer between each pile. Driving until -29.1 m ZH with the top of each sheet pile at +2.80 m ZH.
4. Sheet pile driving with a drummer for anchor wall. The top of every sheet piles is at +2.00 m ZH and the bottom is at – 7.00 m ZH;
5. Set of tie-rods at +0.5 m ZH between every piles and the anchor wall made of sheet piles.
6. Backfill from 0 m ZH to the top of the piles
7. Dredging from 0 to – 24.1 m ZH or – 18 m ZH (it depends if it is in Deep part or in Intermediate part)
8. Finalize the platforms, put the bollards on and also the fenders every 50 m.



On the planning above, you can see how we will do on the construction site. We will start every phase from phase 2 to phase 6 with a delay between every task. This way, every task will start one after the other but we won't waste a lot of time waiting that the previous task is completely done.

Appendix R: Layout 2mod Cost Analysis

	Main Breakwater			Small Breakwater		
Depth (mZH)	Length (m)	Cost per meter (€/m)	Cost (€)	Length (m)	Cost per meter (€/m)	Cost (€)
-5	248,5	16 000,00 €	3 976 000,00 €	44,5	16 000,00 €	712 000,00 €
-6	126,5	22 000,00 €	2 783 000,00 €	45	22 000,00 €	990 000,00 €
-7	132	28 000,00 €	3 696 000,00 €	47,5	28 000,00 €	1 330 000,00 €
-8	91	34 000,00 €	3 094 000,00 €	52	34 000,00 €	1 768 000,00 €
-9	103	40 000,00 €	4 120 000,00 €	51	40 000,00 €	2 040 000,00 €
-10	88,5	46 000,00 €	4 071 000,00 €	45,5	46 000,00 €	2 093 000,00 €
-11	79	53 000,00 €	4 187 000,00 €	40	53 000,00 €	2 120 000,00 €
-12	71,5	60 000,00 €	4 290 000,00 €	37,5	60 000,00 €	2 250 000,00 €
-13	60	67 000,00 €	4 020 000,00 €	33,5	67 000,00 €	2 244 500,00 €
-14	53	75 000,00 €	3 975 000,00 €	27	75 000,00 €	2 025 000,00 €
-15	49	82 000,00 €	4 018 000,00 €	27	82 000,00 €	2 214 000,00 €
-16	46	90 000,00 €	4 140 000,00 €	198	90 000,00 €	17 820 000,00 €
-17	45,5	98 000,00 €	4 459 000,00 €	175	98 000,00 €	17 150 000,00 €
-18	61	106 000,00 €	6 466 000,00 €	0	106 000,00 €	0,00 €
-19	59,5	114 000,00 €	6 783 000,00 €	0	114 000,00 €	0,00 €
-20	542	122 000,00 €	66 124 000,00 €	0	122 000,00 €	0,00 €
-25	450	182 000,00 €	81 900 000,00 €	0	182 000,00 €	0,00 €
		Total	**212 102 000,00 €**		Total	54 756 500,00 €

Total breakwater costs :	**266 858 500,00 €**

Additional Cost of Dredging	
Total Port Surface (M2)	2 960 000
Additional depth to dredge (M)	5
Volume (M3)	14 800 000
Price of Dredging (€/M3)	10
Additional cost	**148 000 000,00 €**

	Main Breakwater			Small Breakwater		
Depth (mZH)	Length (m)	Cost per meter (€/m)	Cost (€)	Length (m)	Cost per meter (€/m)	Cost (€)
-5	115	16 000,00 €	1 840 000,00 €	44,5	16 000,00 €	712 000,00 €
-6	107	22 000,00 €	2 354 000,00 €	44,5	22 000,00 €	979 000,00 €
-7	72	28 000,00 €	2 016 000,00 €	47	28 000,00 €	1 316 000,00 €
-8	123	34 000,00 €	4 182 000,00 €	52	34 000,00 €	1 768 000,00 €
-9	108,5	40 000,00 €	4 340 000,00 €	51	40 000,00 €	2 040 000,00 €
-10	90	46 000,00 €	4 140 000,00 €	45,5	46 000,00 €	2 093 000,00 €
-11	51	53 000,00 €	2 703 000,00 €	40	53 000,00 €	2 120 000,00 €
-12	56	60 000,00 €	3 360 000,00 €	37,5	60 000,00 €	2 250 000,00 €
-13	73,5	67 000,00 €	4 924 500,00 €	33,5	67 000,00 €	2 244 500,00 €
-14	52,5	75 000,00 €	3 937 500,00 €	27	75 000,00 €	2 025 000,00 €
-15	48,5	82 000,00 €	3 977 000,00 €	27	82 000,00 €	2 214 000,00 €
-16	48,5	90 000,00 €	4 365 000,00 €	29,5	90 000,00 €	2 655 000,00 €
-17	46	98 000,00 €	4 508 000,00 €	33	98 000,00 €	3 234 000,00 €
-18	43	106 000,00 €	4 558 000,00 €	37,5	106 000,00 €	3 975 000,00 €
-19	41,5	114 000,00 €	4 731 000,00 €	46	114 000,00 €	5 244 000,00 €
-20	191,5	122 000,00 €	23 363 000,00 €	396	122 000,00 €	48 312 000,00 €
-25	787	182 000,00 €	143 234 000,00 €	125	182 000,00 €	22 750 000,00 €
-30	535	225 000,00 €	120 375 000,00 €		225 000,00 €	0,00 €
		Total	**342 908 000,00 €**		**Total**	**105 931 500,00 €**

Total breakwater costs :	**448 839 500,00 €**

Layout 2 with quays at 0mZH		Layout with quays at -5mZH	
Main breakwater cost	212 102 000,00 €	Main breakwater cost	342 908 000,00 €
Small breakwater cost	54 756 500,00 €	Small breakwater cost	105 931 500,00 €
Total breakwater cost	266 858 500,00 €	Total breakwater cost	448 839 500,00 €
Additionnal dredging cost	148 000 000,00 €		
Cost estimation	**414 858 500,00 €**	**Cost estimation**	**448 839 500,00 €**

Cost estimation = Breakwater + additional dredging only

Appendix S: Costs of the project

Cost of Nador West Med Port : *Design your Dream* Company

	Working	Unit	Quantity	Unit Price (€)	Total Amount (€)
Quays	Containers Quays at 0mZH	ML	1 400	10 000,00 €	14 000 000,00 €
	Ro-ro and General Cargo Quays from 0 mZH to -2 mZH	ML	400	10 000,00 €	4 000 000,00 €
	Ro-ro and General Cargo Quays from -2 mZH to -4 mZH	ML	100	13 000,00 €	1 300 000,00 €
	Bulk Quays at 0mZH	ML	600	10 000,00 €	6 000 000,00 €
				Total Quays Cost	**25 300 000,00 €**
Jetty	Mooring Dolphins	U	8	1 700 000,00 €	13 600 000,00 €
	Berthing Dolphins	U	4	2 100 000,00 €	8 400 000,00 €
	Platforms	U	2	6 500 000,00 €	13 000 000,00 €
	Walkways	ML	875	9 000,00 €	7 875 000,00 €
				Total Jetty Cost	**42 875 000,00 €**
Trestle	Trestle at :				
	-7 m ZH	ML	320	15000	4 800 000,00 €
	-6 m ZH	ML	43	14000	602 000,00 €
	-5 m ZH	ML	41	13000	533 000,00 €
	-4 m ZH to the terminals	ML	400	10000	4 000 000,00 €
				Total Trestle Cost	**9 935 000,00 €**
Breakwaters	Main Breakwater at a depth of :				
	- 5 m ZH	ML	248,5	16 000,00 €	3 976 000,00 €
	- 6 m ZH	ML	126,5	22 000,00 €	2 783 000,00 €
	- 7 m ZH	ML	132	28 000,00 €	3 696 000,00 €
	- 8 m ZH	ML	91	34 000,00 €	3 094 000,00 €
	- 9 m ZH	ML	103	40 000,00 €	4 120 000,00 €
	- 10 m ZH	ML	88,5	46 000,00 €	4 071 000,00 €
	- 11 m ZH	ML	79	53 000,00 €	4 187 000,00 €
	- 12 m ZH	ML	71,5	60 000,00 €	4 290 000,00 €
	- 13 m ZH	ML	60	67 000,00 €	4 020 000,00 €
	- 14 m ZH	ML	53	75 000,00 €	3 975 000,00 €
	- 15 m ZH	ML	49	82 000,00 €	4 018 000,00 €
	- 16 m ZH	ML	46	90 000,00 €	4 140 000,00 €
	- 17 m ZH	ML	45,5	98 000,00 €	4 459 000,00 €
	- 18 m ZH	ML	61	106 000,00 €	6 466 000,00 €
	- 19 m ZH	ML	59,5	114 000,00 €	6 783 000,00 €
	- 20 m ZH	ML	542	122 000,00 €	66 124 000,00 €
	-25 m ZH	ML	450	182 000,00 €	81 900 000,00 €
	Secondary Breakwater at a depth of :				
	- 5 m ZH	ML	44,5	16 000,00 €	712 000,00 €
	- 6 m ZH	ML	45	22 000,00 €	990 000,00 €
	- 7 m ZH	ML	47,5	28 000,00 €	1 330 000,00 €
	- 8 m ZH	ML	52	34 000,00 €	1 768 000,00 €
	- 9 m ZH	ML	51	40 000,00 €	2 040 000,00 €
	- 10 m ZH	ML	45,5	46 000,00 €	2 093 000,00 €
	- 11 m ZH	ML	40	53 000,00 €	2 120 000,00 €
	- 12 m ZH	ML	37,5	60 000,00 €	2 250 000,00 €
	- 13 m ZH	ML	33,5	67 000,00 €	2 244 500,00 €
	- 14 m ZH	ML	27	75 000,00 €	2 025 000,00 €
	- 15 m ZH	ML	27	82 000,00 €	2 214 000,00 €
	- 16 m ZH	ML	198	90 000,00 €	17 820 000,00 €
	- 17 m ZH	ML	175	98 000,00 €	17 150 000,00 €
				Total Breakwater Cost	**266 858 500,00 €**