Feasibility study of port project Clifton Point, New Providence, Bahamas

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Preface

The final step before graduation at the faculty of Civil Engineering and Geosciences of the University of Technology in Delft is to write a master thesis. This MSC thesis contains a feasibility study of the relocation of the current port of Nassau, Bahamas. The study was performed in cooperation with Port Management Consultants and Lievense Consulting Engineers.

PMC and Lievense provided me with a working space and put all their knowledge and advise at my disposal, for which I am very grateful.
I really appreciated the working atmosphere of an office to finish my thesis.
I also would like to thank the following members of the graduation committee for guiding my during the process of graduation:

Prof. ir. H. Ligteringen	DUT, Hydraulic Engineering
Ir. R. Groenveld	DUT, Hydraulic Engineering
Ir. R.J. Labeur	DUT, Fluid Mechanics
Dr. Ir. W. Veldhuyzen	Port Management Consultants

Finally I want to thank my family and friends, especially Marianne, for the support over the last year. Without it the road would definitely be a few paces longer.

Edwin Schaap
Den Haag, October 2007

Abstract

The current port of the city of Nassau, the capital of the Bahamas, encounters a capacity problem. The increasing distribution of goods through the city centre causes traffic congestion. By relocating the port the hindrance for Nassau can be minimised. Relocating the port will also provide opportunities for the tourist industry.

A previous study for the Bahamian Government concluded that the area of Clifton Point is most suitable for locating the new port structure. This report will focus on the feasibility of the appointed project site.

The objective of the report is to relocate the total throughput of the container and dry bulk cargo. A feasible design has to be generated for a minimum of costs and a minimal amount of annual downtime for the port's activities.

First the conditions of the project site are determined. With a relatively small amount of wave data, the operational and extreme conditions of the port have to be determined. Because the project is situated in the Caribbean, the influence of hurricanes has also to be checked. The site conditions show that the conditions at Clifton Point are relatively calm.

The next step is to determine the characteristics of the relocated vessels and cargo. Therefore a forecast is performed. This forecast has three referential years, namely 2020, 2028 and 2035. All alternatives will be designed for these years of reference.
With the forecast the functions of berths and terminal areas are determined. For this reason a capacity study is performed.

After the general dimensions of the port are determined, several alternatives are generated for the new port design. Because of the restricted availability of surface area at the project site, the alternative options are limited. The limited surface area is caused by the presence of an industrial area and the fact that the shore in front of the coastline is relatively steep.

After performing a multi criteria analysis it is clearly that an inland port design is inefficient in relation to a port which is based on land reclamation.
To create sufficient space for the port a small part of liquid bulk storage area has to be relocated.

In a detailed study the configuration of the breakwaters is optimised; variant layouts are elaborated to select the best option of construction costs in relation to the occurring annual downtime. Also the manoeuvring of the vessels arriving at the new port structure has to be checked.

Finally the breakwater will be designed in more detail. The geometry and composition of armour protection are part of this design. Since two breakwaters are required, two individual designs are generated.

Also details of the structure like for example the armour layer at the head of the breakwater are determined.

Finally it can be concluded that the chosen design is technically feasible. All design vessels are able to berth in a port, which provides a sufficient capacity and a safe turning basin for the navigation of the vessel.

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1 Introduction

This thesis covers the planning and feasibility study for the new port of the Bahamas, located at the south-western coast of the island of New Providence. In this chapter first of the background of the current problems is presented, followed by the problem analysis and the objectives of this study.

1.1 Background of the study

In the middle of the Caribbean the Bahamas are situated. The Bahamas consist of a large group of islands with the capital Nassau located on the relatively small island of New Providence. Financial services and the tourist sector are the two main drivers of the nation's economy.

New Providence is inhabited by approximately 266.000 people, which is about 70 percent of the nation's population. For this reason the island can be seen as the business and communication centre of the archipelago.

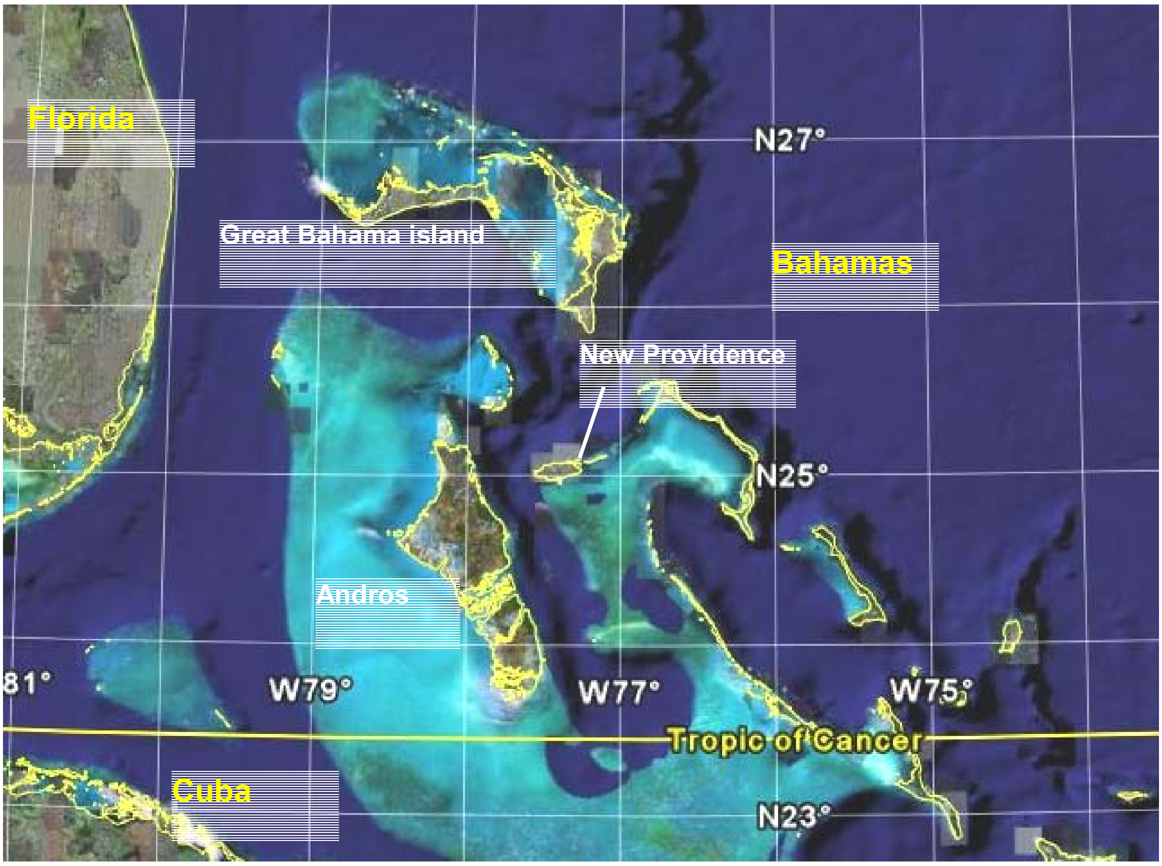


Figure 1-1: Map of the Bahamas

Commercial vessels with destination New Providence arrive at the island at two locations. These are the port of Nassau, located northeast, and Clifton Point, located southwest.

In the current situation the major part of transport overseas takes place at the port of Nassau. This transport is divided over the locations indicated as Downtown Nassau and Arawak Cay as presented in figure 1-2.

Previous studies point out the problem of traffic congestion in downtown Nassau. The increasing commercial activity causes the distribution of goods to block the historical centre of Nassau. Relocation of the major part of commercial activities of the port of Nassau is a possible solution to the problem of traffic congestion.

Tourist activities in Nassau will also benefit from relocating the port. A decrease of industrial activity will increase the attraction for tourist activities. The new available quays also provide new opportunities for the cruise industry, which is expecting future growth. It also provides development opportunities for additional recreational vessels.

In a preliminary study, the EIA report, five possible locations along the southern coast of New Providence have been evaluated. As the result of this Environmental Impact Analysis (EIA) the location of Clifton Point has been selected as the best option for relocation.

Figure 1-2: Map of New Providence

In the EIA report it was concluded that in relation to the other options the location of Clifton Point has the best environmental conditions. This is based on the following considerations:

Since the distance to deep water is minimal, the impact on the marine environment is minimised. Also because of the presently industrial area, the impact on water and air quality is minimized. For terrestrial environment this alternative has a relatively unsuccessful score, this is due to the fact that the preliminary design is fully integrated in the island, no balance between ‘’cut and fill’’ is applied. This is apart from an environmental view also a disadvantage in an economical way.

For the long term master planning of New Providence the location near the Power Plant is optimal; the port is located on land utilized for industrial purposes and the area is large enough for future developments. Because of the industrial zone the port traffic is separated from traffic with residential and tourist destinations.

Also with respect to construction and engineering criteria the Power Plant alternative scores best; the acquisition of surrounding lands is feasible and construction costs are relatively low. The relatively low wave climate is expected to be causing little problems.

According to the study the location is also the best option in a socioeconomic sense; it has a minimal impact on the current tourism industry and sites of archaeological significance.

1.2 Clifton Point

The most distinctive features of the project area are the steep slope of the coast and the limited available working space for future terminal areas.

The steep slope forces the design solution of the new port to be located within close range of the current shoreline. Constructing the port at a distance to large would increase the construction costs. Port structures like quay walls and breakwaters which are constructed at a relatively large water depth would increase the construction costs substantially.

Since the current structures of the Power Plant and Brewery are not relocated, the available surface directly located at the shore line is limited.
For the new port design the terminal areas have to use the available terrain in a optimal way.

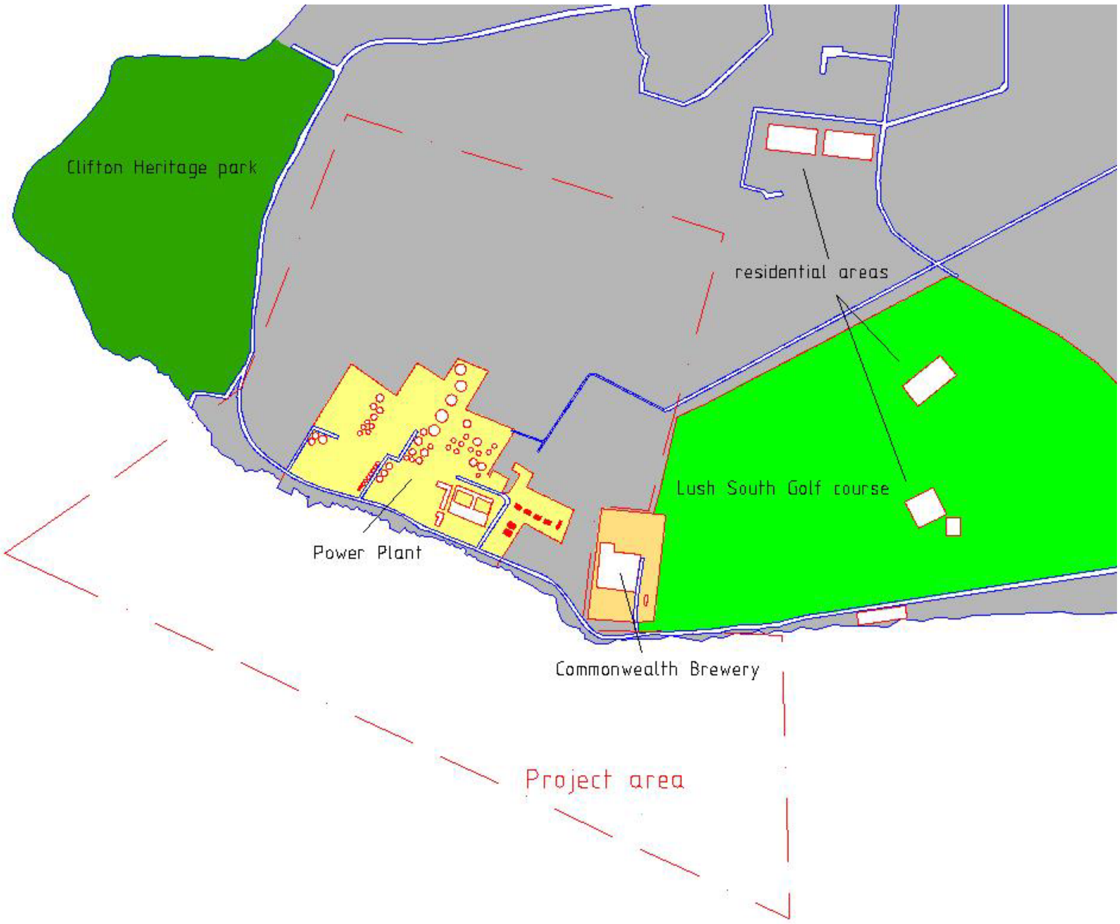


Figure 1-3: Map location Clifton Point

The area surrounding Clifton Point has the lowest population density of New Providence. In figure 1-3 can be seen that the nearest residential areas are located from the Power Plant at a distance of approximately 1200 m.

The Clifton Heritage Park, which is located west of the project site, is the nature reserve of New Providence. When a controversial plan for building luxury residences at this location was presented in 1998, political and environmental activists were triggered to resist this plan.
The opposition was successful. In 2004 the Bahamas parliament passed an Act for the establishment of a corporate body, the Clifton Heritage Authority, with the responsibility for owning, managing and preserving the area.

The largest industrial area is the land owned by Bahamas Electricity Corporation and petrol storage of various petrol companies (including Shell, Texaco and Exxon).
Connected to this area are three berthing piers, which are accessible to break bulk vessels, oil tankers and LPG tankers. The location of these berthing piers is presented in figure 1-4.

In eastern direction of the Power Plant complex the Commonwealth Brewery is situated. The land is property of the Bahamian government.
Currently the brewery is the solely user of the alcohol berth.

A larger image of the project area is presented in Appendix A.

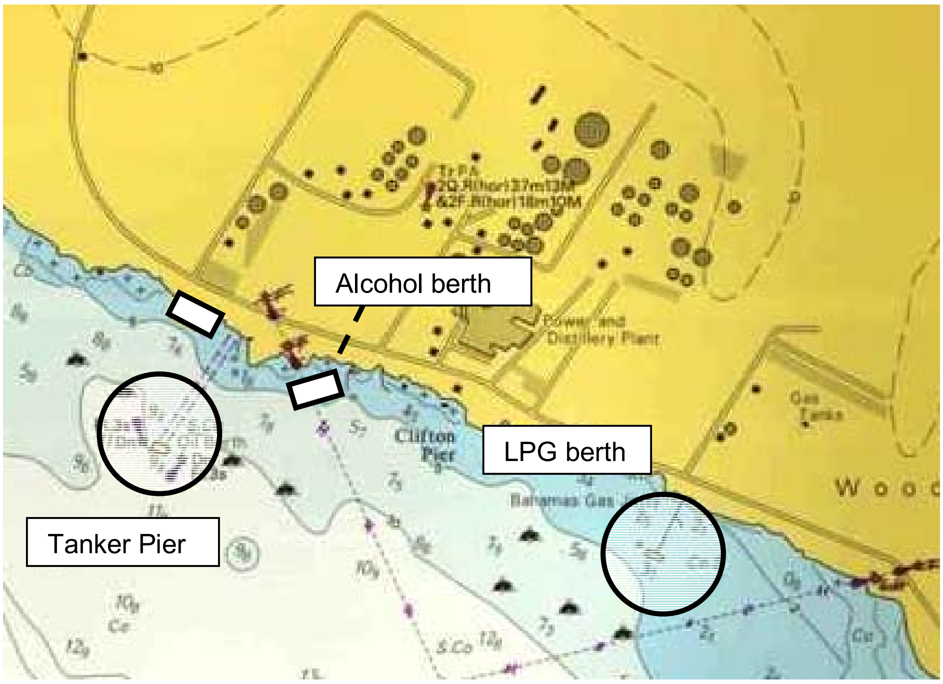


Figure 1-4: Current port structure of Clifton Point

1.3 Structure of report

The report is structured in the way as presented in the methodology of figure 1-5.

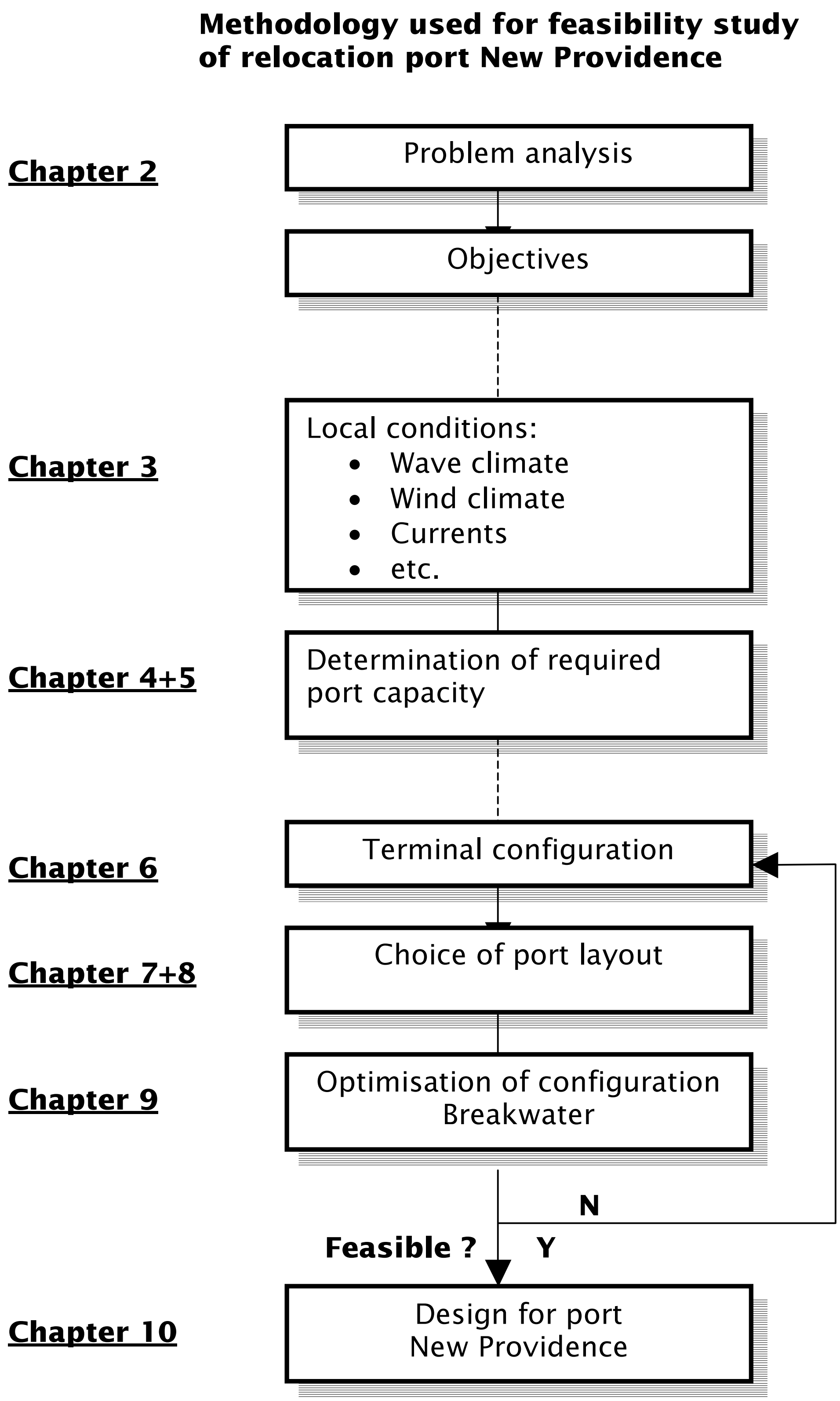


Figure 1-5: Methodology of feasibility study

2 Study scope

2.1 Problem analysis

The EIA report concludes that the present port facilities at the location of Nassau, New Providence, are inadequate. The traffic density around Downtown Nassau causes congestion. With an expected growth of future port operations the traffic problems will even increase.

The Bahamian authorities desire a limitation of the occurring traffic congestion. The growth of the tourist sector and especially the cruise industry also demands a solution which creates additional space.

Because of the limited space available at the water side, expansion of the port is not a feasible solution. Expansion will also not solve the traffic problem.

Therefore the options of relocating the major part of the port of Nassau have to be investigated. By relocating the port several opportunities are generated for the growing tourist industry. Besides the increasing availability of land, the port of Nassau also becomes more attractive in a tourist perspective.

The best suitable location according to EIA study is the location of Southwest New Providence, also known as Clifton Point. Since many design variables are unexplored, a feasibility study is required for this location.

The port functions of Nassau to be relocated are the container and dry bulk terminals. More details about these functions are presented in paragraph 4.1.

2.2 Objective

The main objective of this thesis study is to examine the technical feasibility of the Clifton Point for relocation a share of the port of Nassau.
The feasibility study has to provide an answer for the following questions:

- Is the development of a port at Clifton Point viable?
- What is the optimal lay-out of the chosen alternative?
- What are the most important influences and risks during the design of a port lay-out?

2.3 Design requirements

A port design which is considered technically feasible should comply to the following requirements:

- The new port provides space for the terminals as described in table 4-1 and the terminals presently located at Clifton Point.
- Under operational conditions the manoeuvring area for the vessels within the port has to be sufficient.
- At extreme weather conditions the berthing facilities should be protected in an optimal way.
- The berths of the port's terminals have to provide a capacity in such a way that the mean waiting time is below limits.
- The construction costs of the design alternative have to be as low as possible.
- The loss of capacity due to downtime of the port has to be within acceptable limits.

When the port turns out to be non feasible recommendations are required for additional solutions. An example could be to redirect a part of the new port's functions to the port of Nassau.

2.4 Design criteria

The new port has to provide sufficient capacity for a period up to the year 2035. The capacity is judged by the amount of mean waiting time.
The vessels used to transport the specific types of cargo the acceptable waiting time varies:

- The maximum mean waiting time for vessels transporting containers, RoRo, break bulk and cars is 40 % of the gross service time.

- The maximum mean waiting time for vessels transporting dry bulk and liquid bulk is 20 % of the gross service time.

- The current mean annual downtime for the berthing facilities at Clifton Point is 20 days. The maximum of the mean annual downtime for the new port structure is 20 days.

3 Site conditions

For the generation of adequate port alternatives at the location of Souhwest New Providence sufficient data is necessary about the local conditions.
This data can be split into the following components:

- Bathymetry
- Tidal data
- Currents
- Waves
- Winds
- Hurricanes
- Storm surge
- Sedimentation
- Soil conditions

In this chapter all these components are treated.

3.1 Bathymetry

The island of New Providence is located in the centre of the Bahamas. The shallow waters surrounding the islands are called the Great Bahama Bank. These shallow waters with a minimum bottom level of -5.0 fathoms (about -8.90 m. MSL) are presented by the blue areas in figure 3-1. The mean bottom level of these areas is about -4.50 m. MSL.

The Tongue of the Ocean is the only deepwater area within the Bahamas. It has a maximum depth of 900 fathoms (about 1600 m.). This area enters the Bahamas from northwest and north-eastern direction.
This area is presented as the white area in figure 3-1.
Bathymetry on a smaller scale is presented in Appendix A.

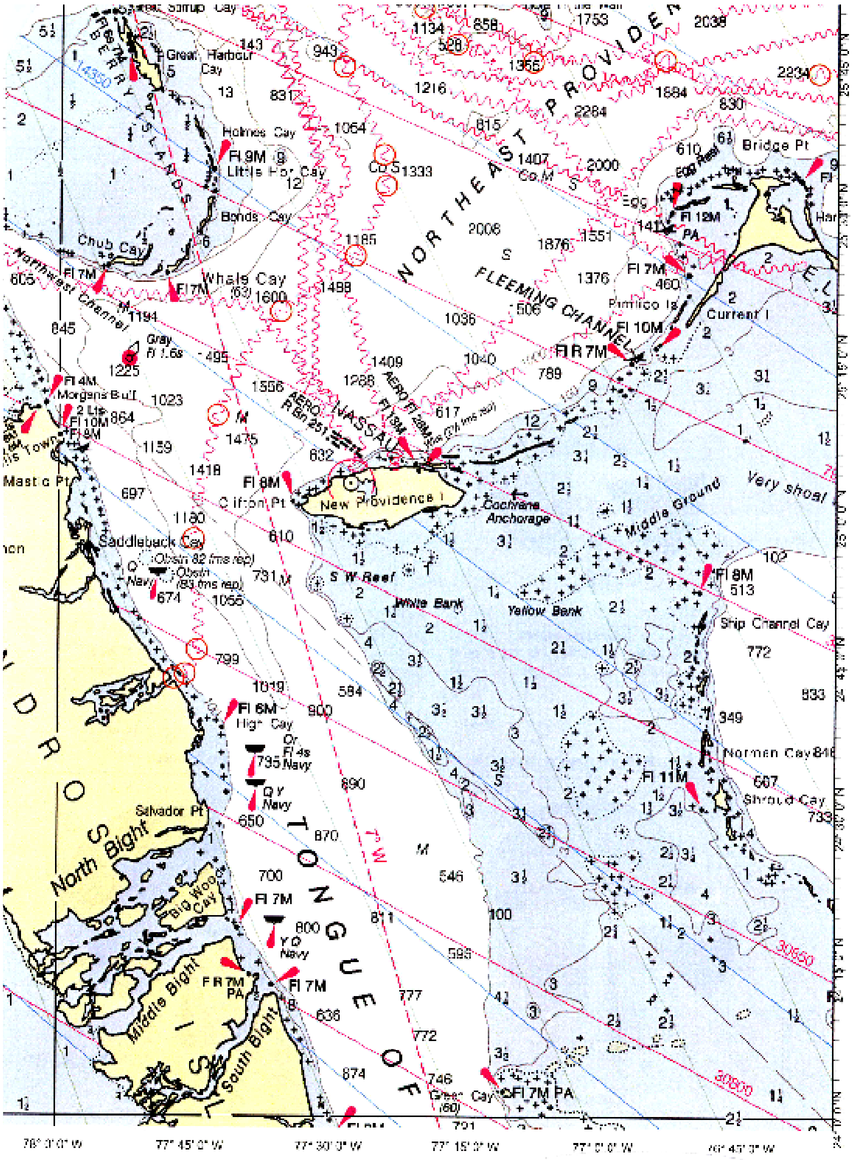


Figure 3-1: Bathymetry of vicinity of New Providence

3.2 Tidal data

Tides in the vicinity of New Providence Island are predominately semi-diurnal with an average range of 0.75 meters. The tidal period is approximately 12.4 hours. The following tide levels were obtained from the EIA report.

Mean High Water Spring (MHWS):	+0.40 meters
Mean High Water Neap (MHWN):	+0.30 meters
Mean Sea Level (MSL):	+0.00 meters
Mean Low Water Neap (MLWN):	- 0.30 meters
Mean Low Water Spring (MLWS):	- 0.50 meters

Possible future rise of the water level due to for example climate change will not be included in this report.

3.3 Currents

In the vicinity of New Providence the magnitude of the currents is ranging from 0.50 to 0.75 knots. These currents are dominated by tidal variation along with a general current to the northwest feeding the Florida Current and the Gulf Stream. Wind generated surface and wave induced currents caused by wave breaking are experienced locally as well.

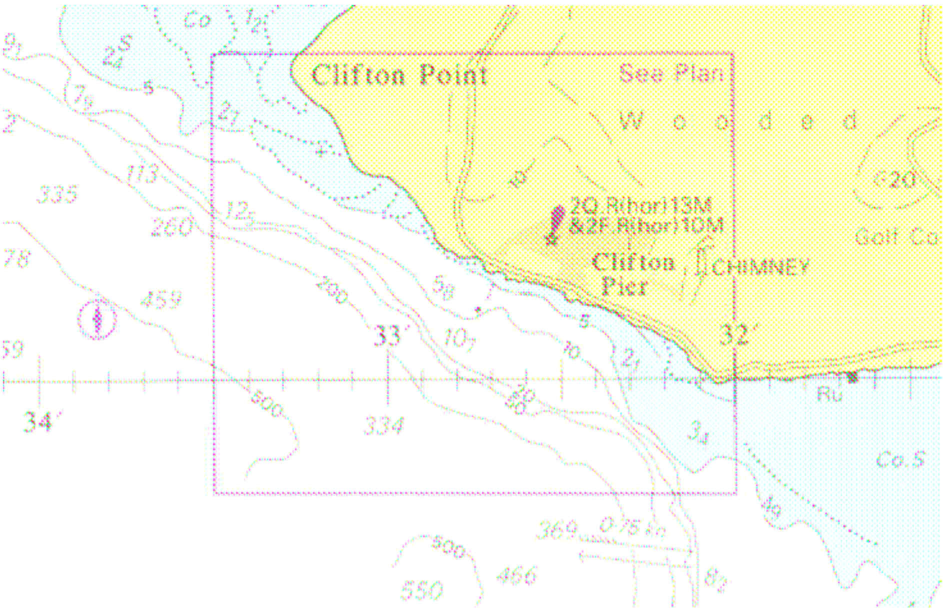


Figure 3-2: Currents in vicinity of New Providence

3.4 Wind climate

In the EIA report a statistical analysis of wind data was performed. The data was obtained from NOAA from 1999 to 2004 at a grid point approximately 25 km northwest of Nassau. In figure 3-3 the annual wind rose following from the statistical analysis is presented.

At the southern side of New Providence the wind rose varies. The island will divert the winds from northern and eastern direction. Winds in the Caribbean usually come from north-eastern direction. With no other wind data available the wind rose of figure 3-3 is considered as an upper limit for the wind climate at Clifton Point. Since Clifton Point is located at the lee side of New Providence the wind velocities are expected to be lower than the wind velocities of the wind rose of figure 3-3.

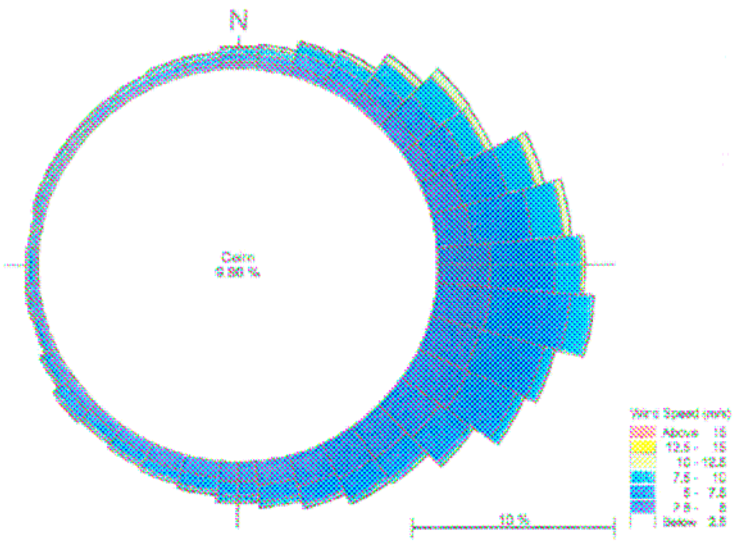


Figure 3-3: Annual windrose at New Providence

A table containing the total percentage of annual occurrence is available in Appendix B.

3.5 Hurricanes

In the Caribbean hurricanes and storm surges have a significant impact on local ports and coastal structures. The storm surge caused by the hurricane also determines the extreme wave condition. These conditions are required to design the coastal structures, like for instance breakwaters. It is therefore critical to evaluate design storms and hurricanes in the project’s vicinity.

First of all the design storms are determined from historical hurricane tracking records, which are collected by NOAA.

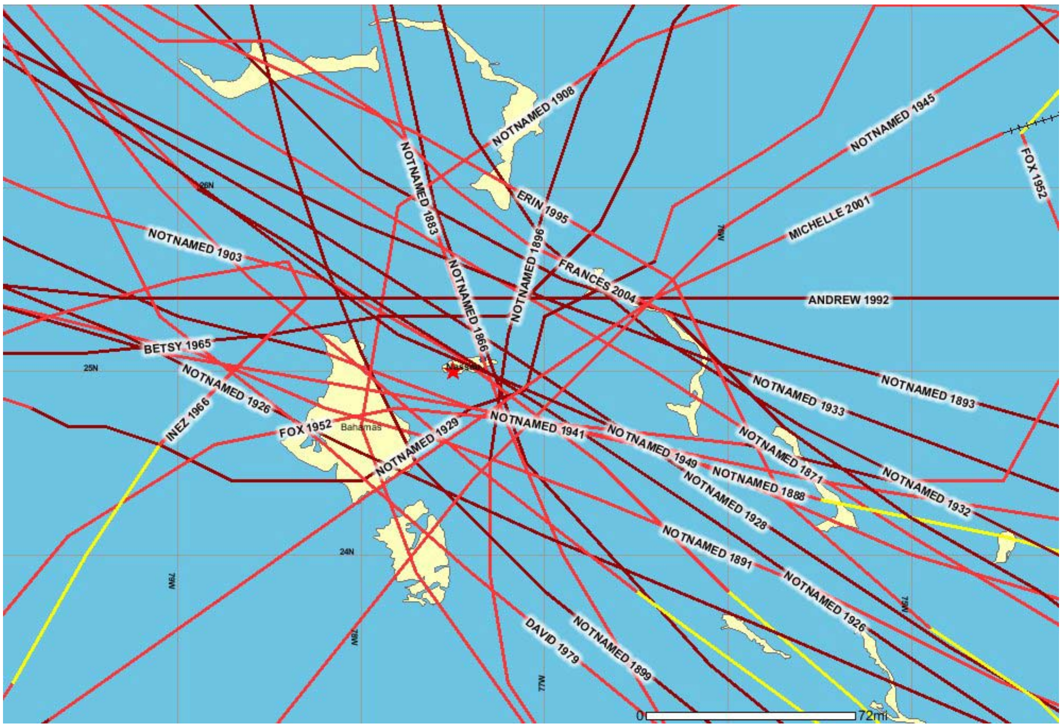


Figure 3-4: Hurricanes arriving at New Providence within 65 sea miles

For a time period of 155 years hurricanes within a radius of 65 sea miles of New Providence are analysed. According to NOAA an area which is struck by a hurricane is defined by an area with a radius of 65 sea miles. In the area of influence, which is presented in figure 3-4, 28 hurricanes of category 1 and higher were found. The categories of a hurricane are defined by the Saffir-Simpson scale, presented in table 3-1.

Category	Wind speed	Indication of damage

	(kt)	
H1	64-82	No real damage to building structures. Damage primarily to unanchored mobile homes, shrubbery and trees. Also some coastal road flooding and minor pier damage.
H2	83-95	Some roofing material, door, and window damage to buildings. Considerable damage to vegetation, mobile homes, and piers. Coastal and low-lying escape routes flood 2-4 hours before arrival of centre. Small craft in unprotected anchorages break moorings.
H3	96-113	Some structural damage to small residences and utility buildings with a minor amount of curtain wall failures. Mobile homes are destroyed. Flooding near the coast destroys smaller structures with larger structures damaged by floating debris. Terrain continuously lower than 5 feet ASL may be flooded inland 8 miles or more.
H4	114-135	More extensive curtain wall failures with some complete roof structure failure on small residences. Major erosion of beach. Major damage to lower floors of structures near the shore. Terrain lower than 10 feet ASL may be flooded requiring massive evacuation of residential areas inland as far as 6 miles.
H5	> 136	Complete roof failure on many residences and industrial buildings. Some complete building failures with small utility buildings blown over or away. Major damage to lower floors of all structures located less than 15 feet ASL and within 500 yards of the shoreline. Massive evacuation of residential areas on low ground within 5 to 10 miles of the shoreline may be required.

Table 3-1: Categorizing of hurricanes on Saffir-Simpson scale

Tropical storms which are below the category of a hurricane are labelled by the following definitions:

Category	Abbreviation	Wind speed (kt)	Features
Tropical depression	TD	< 34	Tropical cyclone
Tropical storm	TS	34 - 64	Tropical cyclone
Subtropical storm	SS	< 34	Subtropical cyclone
Extratropical	E		The energy source driving the storm has lost its characteristics. The storm can still retain winds of its original force

Table 3-2: Categorizing of tropical storms below category of hurricane

Rec	YEAR	MONTH	DAY	STORM NAME	CATEGORY	PRESSURE(MB)	WIND SPEED (KTS)	WIND SPEED (m/s)
1	1859	10	16	-	TS	-	60	26,8
2	1866	10	1	-	H4	-	120	53,6
3	1877	9	26	-	TS	-	40	17,9
4	1878	8	25	-	TS	-	40	17,9
5	1880	8	20	-	TS	-	40	17,9
6	1871	8	24	-	H2	-	90	40,2
7	1883	9	8	-	H2	-	90	40,2
8	1888	8	16	-	H2	-	90	40,2
9	1888	9	7	-	TS	-	40	17,9
10	1888	9	7	-	TS	-	45	20,1
11	1891	8	23	-	H2	-	85	38,0
12	1893	8	26	-	H3	-	105	46,9
13	1893	10	21	-	TS	-	35	15,6
14	1896	9	5	-	H3	-	100	44,7
15	1897	10	19	-	TS	-	55	24,6
16	1897	10	23	-	TS	-	50	22,3
17	1898	10	23	-	E	-	35	15,6
18	1899	8	12	-	H3	-	105	46,9
19	1901	8	9	-	TS	-	40	17,9
20	1903	9	10	-	H1	-	70	31,3
21	1908	10	1	-	H2	-	95	42,5
22	1909	8	28	-	TS	-	40	17,9
23	1910	8	26	-	TD	-	30	13,4
24	1923	10	20	-	E	-	35	15,6
25	1926	7	26	-	H4	-	115	51,4
26	1926	9	14	-	TS	-	35	15,6
27	1926	9	17	-	H4	-	125	55,9
28	1927	11	1	-	TS	-	35	15,6
29	1928	8	6	-	H1	-	70	31,3
30	1928	9	16	-	H4	-	135	60,3
31	1929	9	26	-	H3	-	105	46,9
32	1932	8	29	-	TS	-	50	22,3
33	1932	9	5	-	H5	-	140	62,6
34	1933	9	3	-	H4	-	120	53,6
35	1933	11	3	-	TS	-	40	17,9
36	1936	7	28	-	TS	-	40	17,9
37	1937	8	3	-	TS	-	35	15,6
38	1937	8	28	-	TS	-	35	15,6
39	1939	8	11	-	TS	-	55	24,6
40	1941	10	5	-	H2	-	90	40,2
41	1945	10	13	-	H1	-	65	29,1
42	1946	11	1	-	TS	-	40	17,9
43	1947	10	6	-	TS	-	45	20,1
44	1949	8	26	-	H3	-	100	44,7
45	1952	10	26	FOX	H2	-	95	42,5
46	1958	10	6	JANICE	TS	997	55	24,6
47	1961	9	12	-	TD	-	25	11,2
48	1961	10	18	GERDA	TD	-	30	13,4
49	1965	9	7	BETSY	H3	957	110	49,2
50	1966	7	18	CELIA	TD	-	25	11,2
51	1966	10	3	INEZ	H1	986	70	31,3
52	1969	9	6	GERDA	TD	1015	25	11,2
53	1970	9	12	FELICE	TD	-	25	11,2
54	1974	10	6	SUBTROP4	SS	-	40	17,9
55	1979	9	2	DAVID	H1	979	75	33,5
56	1984	9	26	ISIDORE	TD	1002	30	13,4
57	1987	8	10	ARLENE	TD	1009	25	11,2
58	1988	8	27	CHRIS	TD	1008	30	13,4
59	1992	8	23	ANDREW	H5	922	140	62,6
60	1995	8	1	ERIN	H1	985	75	33,5
61	2001	11	5	MICHELLE	H1	972	80	35,8
62	2004	9	3	FRANCES	H2	960	90	40,2
63	2005	8	24	KATRINA	TS	1006	35	15,6

Table 3-3: Historical Tropical Storms and Hurricanes within radius of 65 sea miles from Clifton Point

To conduct an extreme probability analysis a Gumbel distribution was used to fit the accumulative probabilities of the corresponding extreme events with respect to the associated winds. The result of this Gumbel distribution can be seen in table 3-4. The Gumbel distribution is defined as

$$V = A[-\ln(\ln(\frac{1}{P})] + B$$

Where V is the wind speed, P the accumulative probability that the wind speed is not above the considered wind speed. A en B are fitting coefficients to determine the trend line. P is determined in the following way:

$$\frac{T_r}{r} = \frac{1}{1-P}$$

Where T_r is the return period and r is the mean interval time between two successive storm events.

The maximum wind velocities provided by the hurricane records are of an one-minute interval. In the following paragraphs the wind velocities are used for the generation of waves. For wave generation the maximum mean wind velocity has to be determined over a larger time interval. The maximum mean wind velocity is conversed to a one hour interval using the following formulae:

$$\frac{U_t}{U_{3600}} = 1.277 + 0.296\left[\tanh 0.9 \log \frac{45}{t}\right]$$ for 1s < t ≤ 3600 s

$$\frac{U_t}{U_{3600}} = 1.533 - 0.15 \log t$$ for 3600 s < t ≤ 36.000 s

Return period (year)	One-minute maximum sustained wind speed (m/s)	One-hour maximum sustained wind speed (m/s)
25	51.4	41.4
50	60.8	48.9
100	70.1	56.4

Table 3-4: Design wind speeds, New Providence

The direction of the maximum winds caused by hurricanes depend on the propagating direction of the hurricane centre. For the northern hemisphere the wind velocities within the hurricane can be schematised by figure 3-5.

It can clearly be seen in the figure that winds occurring at the eastern side of the centre have the highest wind velocity.

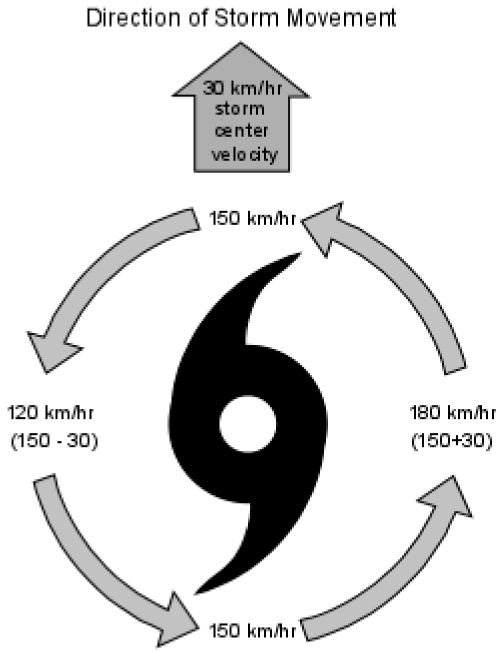


Figure 3-5: indication of wind distribution within hurricane at northern hemisphere

When the hurricane paths of figure 3-4 are evaluated it can be concluded that almost all hurricanes travel in the direction of 230° to 310°.
The part of the hurricanes propagating in a direction of 300° can be estimated on 70 %. Since the hurricane data does not provide a direction for the maximum wind velocity, the direction of 300° is selected as the decisive direction.

3.6 Wave climate

In the EIA report offshore wave data of NOAA's Wave Watch III model is used to generate a wave climate. The wave data in the report is only published as a single figure (figure 3-7). To retrieve quantified data of the wave climate, the data of the EIA report has to be adapted.
This is accomplished in this paragraph.

First the available data will be discussed. From this data of the wave climate as determined in the EIA report a wave climate at deep water in the vicinity of the new port's location will be generated.

Besides the use of the NOAA data several other sources were consulted. These are the following sources:

- ARGOS data, Alkyon
- Wave atlas: "Global wave statistics", British Marine Technology 1985

The ARGOS data was of an area located in southern direction of the project's site; more specific the sea between the Bahamas and Cuba, presented in figure 3-6. This was the result of the fact that the minimum number of samples necessary for a statistical analysis is 5000 observations.
The area surrounding the island of New Providence only provided 600 samples. Since the locations differ largely in water depth and protection in relation to fetch, the ARGOS data is insufficient.

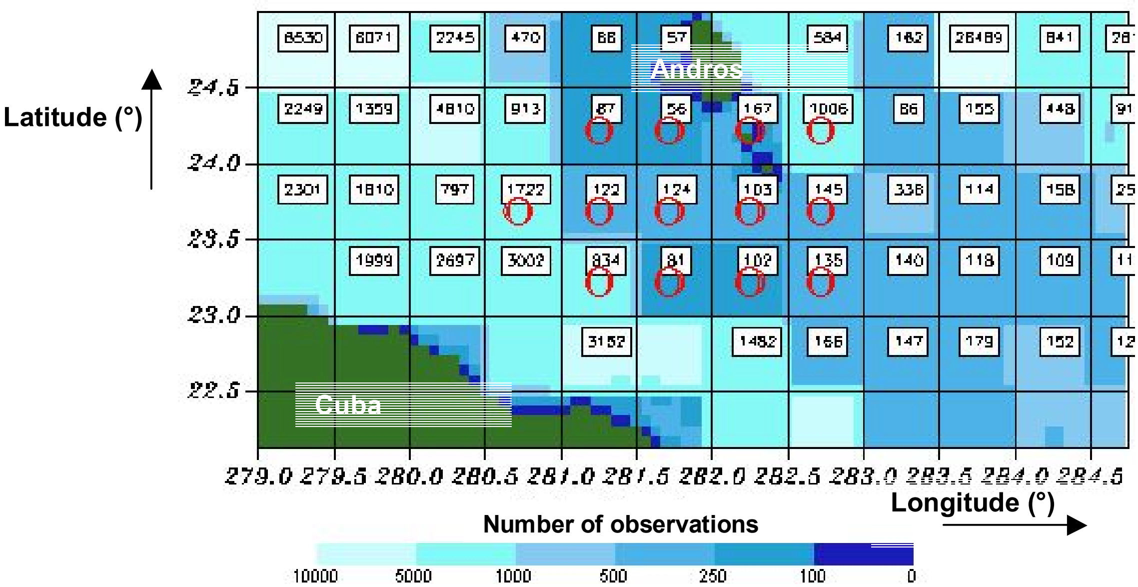


Figure 3-6: Origin of ARGOS data

The wave atlas of the British Marine Technology covers a relatively large area. Namely an area with a longitude from 20° to 30° N and latitude from 60° to 81° W. Therefore this atlas can only be used as broad indication of the wave climate.

3.6.1 Deep water wave statistics

The wave roses presented in figure 3-7 show three locations where the distribution of wave directions and wave heights are known. This data was provided by NOAA through NOAA's Wave Watch III model. The data acquired was part of a five year program from the time period of 1999 to 2004.

However the data accumulated through this program was not available for use from the NOAA. Therefore the data had to be subtracted from the published wave roses in the EIA report. The presented wave roses show the recorded data of wave heights above 0.50 m.

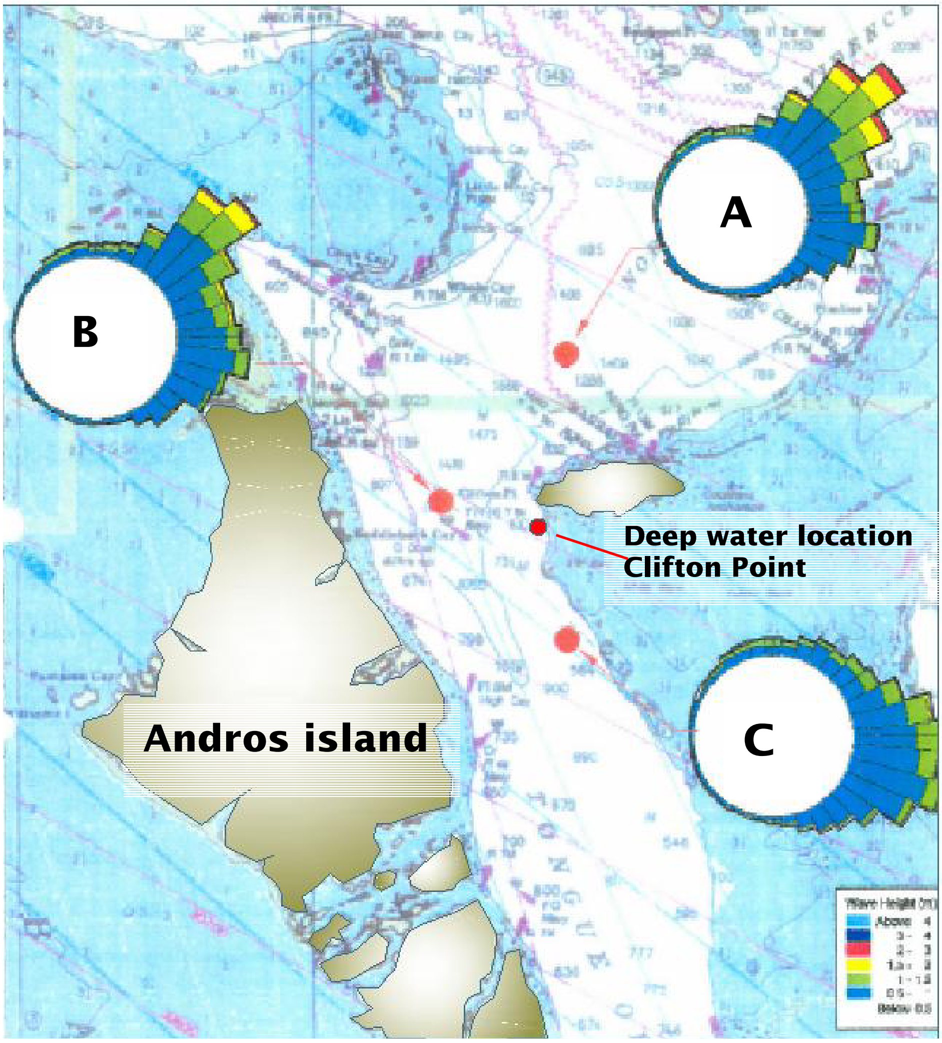


Figure 3-7: Location of available wave data

A wave climate with wave heights below 0.50 m. is considered to be calm. For each wave rose the mean occurrence of calm waves is published in the EIA report.

The data of wave height and wave directions were obtained through 3 offshore placed buoys. These buoys acquired the data on a 3 hour interval. Since figure 3-7 only provides data about height and direction of the waves, the wave period and wave length are unknown parameters.

The location of the buoys are the following:

- Buoy A: 77.50° W, 25.25° N - approx 25 km north of Nassau.
- Buoy B: 77.75° W, 25.00° N - approx 20 km west of Clifton Point.
- Buoy C: 77.50° W, 24.75° N - approx 25 km south of Clifton Point.

To determine the wave climate at a deep water location in front of the new port, a conversion of the currently available wave data to the deep water location of Clifton Point is performed.
This conversion is determined in the next paragraph.
First some general characteristics of the wave roses are presented.

The wave roses from figure 3-7 are presented using 36 segments corresponding with wave directions of 10°. For the reason of surveyability the following graphs are presented with segments corresponding to wave directions of 30°. The total derived dataset can be found in Appendix C.

Wave rose A

The north coast of New Providence is exposed to both long period swells from the Atlantic Ocean and locally wind-generated waves.
In relation to wave rose B and C the waves from the Atlantic Ocean result in the highest wave height of all buoys; 3.2 % of the recorded waves has a wave height larger than 2.00 m. Calms occur for 25.0 % of the total time. The wave height distribution of wave rose A is presented in figure 3-8.

The south coast of the island is partially sheltered by the island itself from the northerly wave climate. To determine the amount of influence the most important factor is the wave length.

Wave lengths in deep water are related to the wave period by $L_0 = \frac{gT^2}{2\pi}$.

Swell, which has a larger wave period in relation to the locally wind generated waves, will diffract around both sides of the island. With the new port located at the southwest coast of the New Providence the influence of swell has to be verified. The effect of the western coast on swell is determined in the next paragraph.

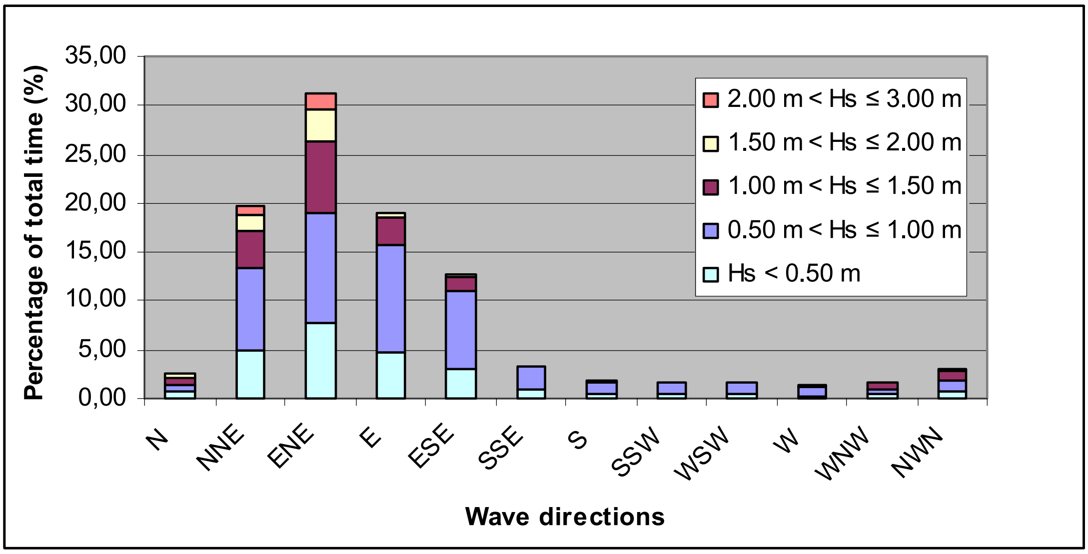


Figure 3-8: Wave climate at location of wave rose A

<u>Wave rose B</u>

The wave climate at wave rose B is clearly influenced by wave rose A. Most waves are from north eastern direction. Because the sea at the location of rose B is less open than rose A, other wave directions are of less influence.

Especially from western direction the wave climate is minimal. In relation to wave rose A the wave height of waves from north eastern direction also decreases. It can be concluded that the recorded waves from western direction (225° to 315°) only cover about 4.8 % of the total wave climate.

A calm wave climate is encountered by 11.3 % of the time.
The wave height distribution of wave rose B is presented in figure 3-9.

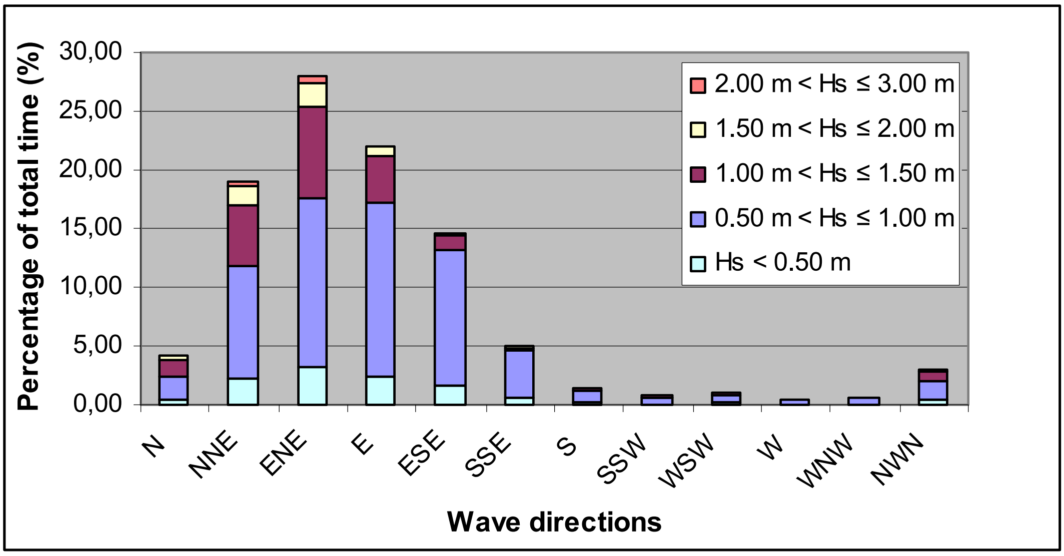


Figure 3-9: Wave climate at location of wave rose B

Wave rose C

The recorded waves at the location of wave rose C have a relatively low wave height. This is caused be the protective character of the location. Waves from northern and north eastern direction are limited by the island's location.
The majority of the waves has its origin from eastern and south eastern direction. These waves travel over the shallow Great Bahama Bank and are therefore limited in their wave height. A large amount of wave energy from this direction is dissipated by wave breaking and bottom friction.

For 10.2 % of the time the wave climate is considered to be calm.
The wave height distribution of wave rose C is presented in figure 3-10.

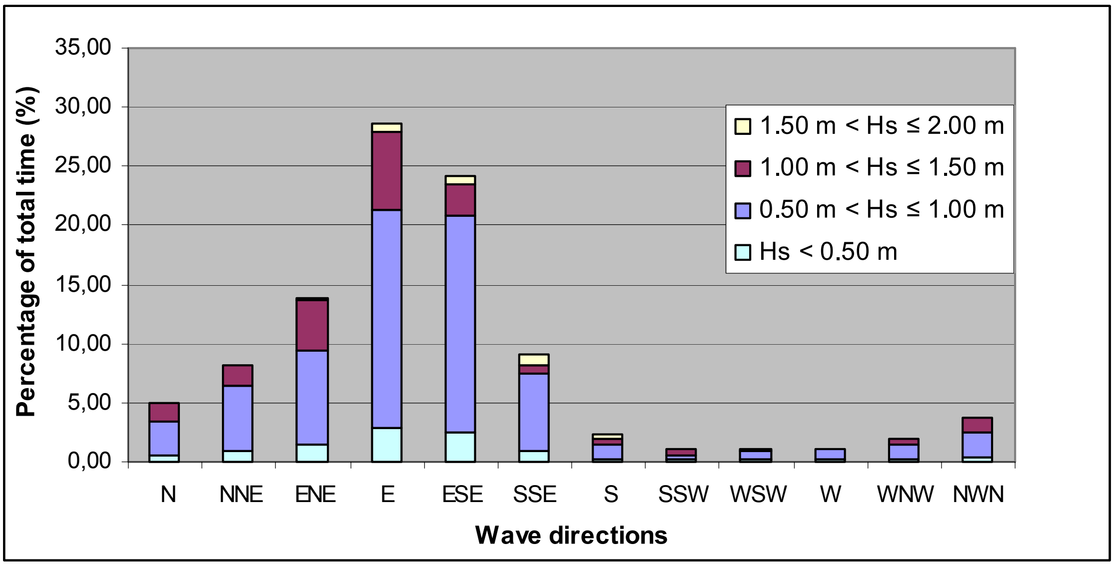


Figure 3-10: Wave climate at location of wave rose C

3.6.2 Conversion wave climate to location of Clifton Point

The deep water wave climate in front of Clifton Point will be determined with the data obtained from the 3 wave roses mentioned in paragraph 3.6.1. Based on the characteristics of these wave roses the wave climate of Clifton Point is expected to have the following features:

The southwest coast of New Providence is relatively well protected. The shallow Great Bahama Bank limits the wave height from eastern direction. The wave climate from northern direction is expected to be blocked by the island itself. The influence of swell is checked in the this paragraph.

Above description corresponds with the wave climate of rose C, which is located closest of all three wave roses to the new port's location. Wave rose C will therefore be chosen as basis of the local deep water wave climate.

The local wave climate will also show deviation from rose C. From western direction the location is less protected by Andros island. Therefore the local influence from western side is expected to be larger in relation to rose C. The influence of the western wave climate of wave rose B is checked in this paragraph.

Influence of swell at Clifton Point

The swell originating from the Atlantic Ocean is caused by the subtropical winds, generated in between latitudes of 40° to 60°.
The swell will propagate towards the equator in south-western direction.
This can be seen in figure 3-11.

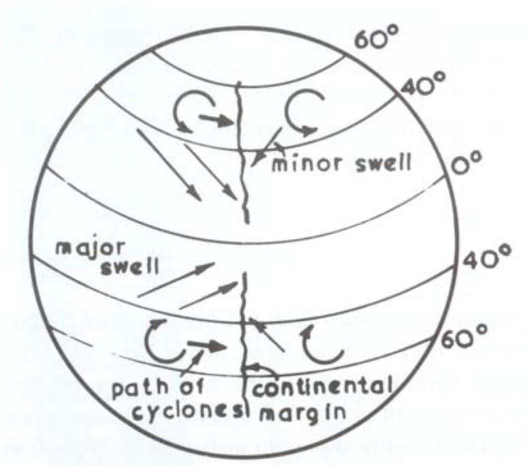


Figure 3-11: Global movement of swell caused by subtropical winds

Swell propagating from north-eastern direction can have an direct effect on the wave climate at Clifton Point. The swell encountering the western shore of New Providence changes direction due to diffraction and refraction.
A part of the energy of the waves will be dissipated and causes the waves to decrease in height. This paragraph researches if swell from northern direction is affecting the location of Clifton Point.

Wave diffraction describes the wave mechanism of the wave height in the shadow region of nontransmitting semi-infinite barrier obstructing the propagating wave. When the water depth in the lee of the structure is not constant the wave crest pattern is also affected by refraction.
This is the case for swell travelling around the west coast of New Providence.

Refraction considers waves travelling from deep water to shallow water. Due to the transition of water depths the wave height and wave direction changes. The angle of the wave crest parallel with the coast decreases for decreasing water depth. Wave diffraction and refraction are both schematised in figure 3-12.

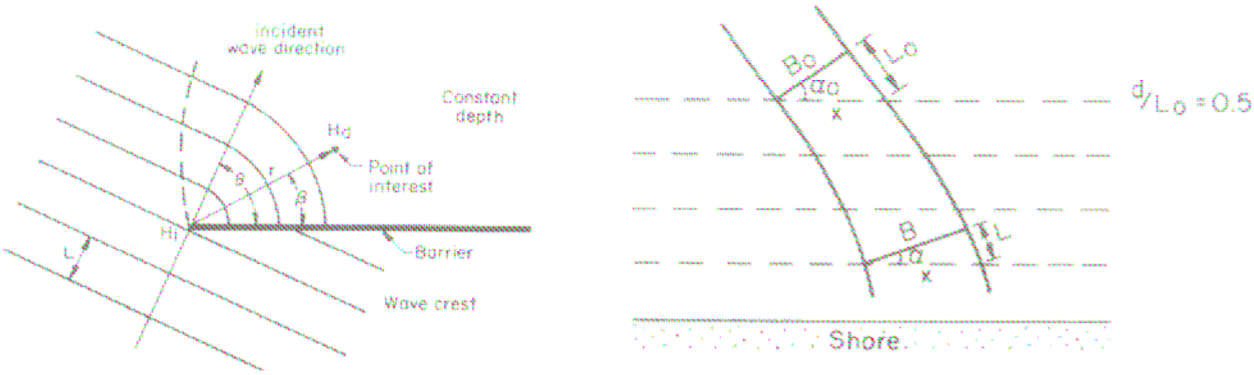


Figure 3-12: Wave diffraction and refraction

In figure 3-12 it can be seen that the waves travelling along the western coast follow various paths and transform in several stages.
The first path is the one of waves passing Lyford Cay. These waves first refract due to the shallow waters at the northern side of the island. From there they travel through the shallow waters in the lee of Lyford Cay. There they are first affected by diffraction, followed by refraction. From that point the waves diffract around the western shore of Clifton Point. This is the most direct way from the northern side of the island to the location of the new port.
It is clear that much dissipation of energy is encountered.

The second path is around the small island of Goulding Cay.
The swell will diffract around the island before arriving at Clifton Point.

To provide a first indication of the wave diffraction, the following assumptions are made. If diffraction plays an important role, these effects require a more detailed study.

- The barrier of Lyford Cay does not reflect or transmit wave energy.
- The depth of the shallow water areas is constant.
- The shoreline is assumed to be linear for determining the effect of refraction.

It has to be noted that the propagating waves do not all follow a path as presented in figure 3-13. In practise the waves travelling from a certain direction are influenced by directional spreading of the waves. When for instance waves travel from the direction of 20°, it is possible that swell from a direction of 60° interferes with the primarily direction at the same time.
The effect of directional spreading will not be included.

To determine the amount of dissipating energy in a further phase a computer model will be used in most conditions. These models solve the effects of diffraction and refraction of the waves numerically. Before a numerical model is used rough calculations are performed by hand; the diffraction and refraction coefficient are first determined independently.

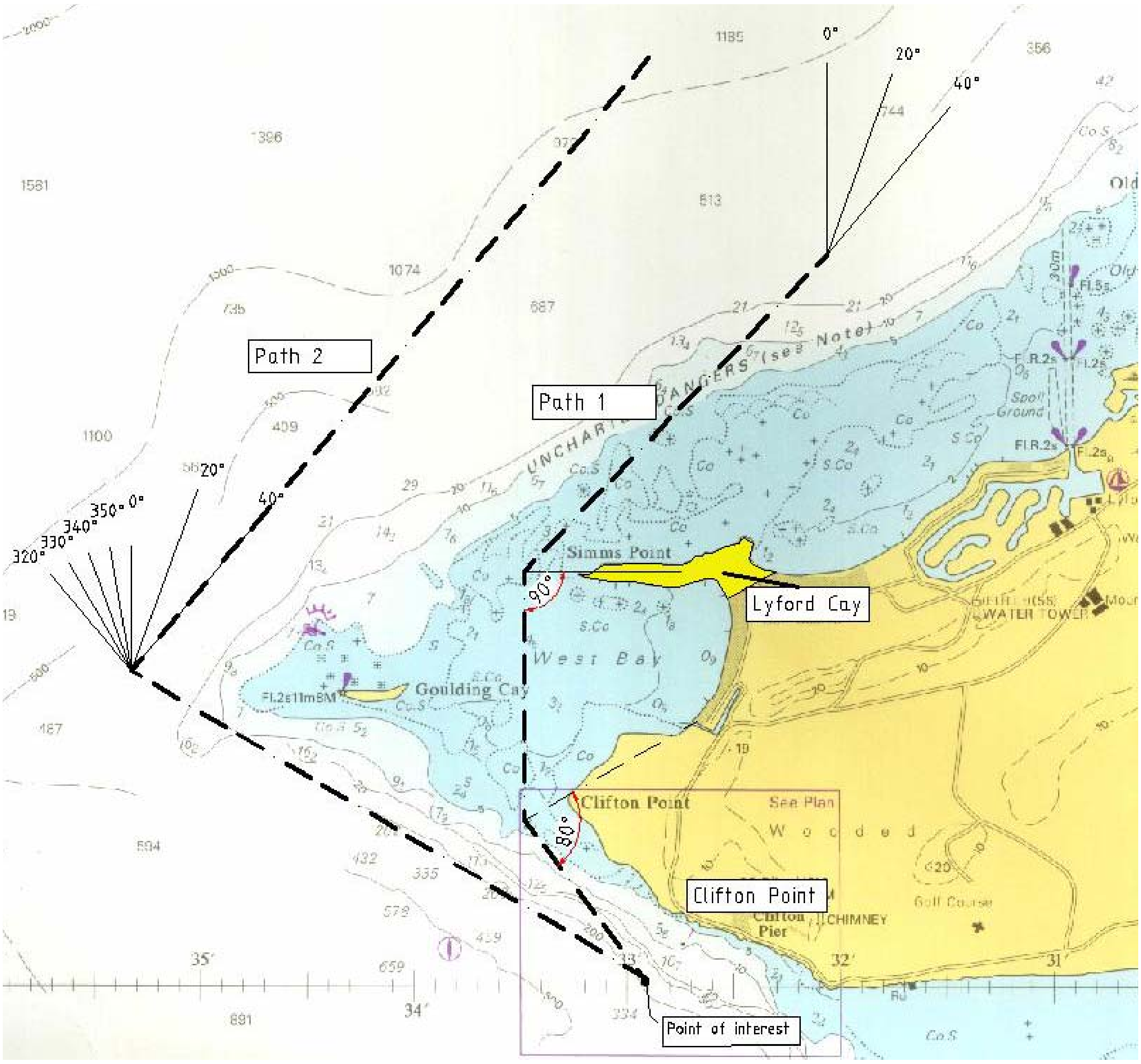


Figure 3-13: Nautical conditions at western side of New Providence

The swell period is approximated by a value of 15 s. This results in a wave length in deep water of about 350 m. To determine the influence of swell a distinction between swell and short period waves is required.
The amount of swell as part of the total wave climate is however unknown.
To check the influence all wave climate is considered as swell.
This is however unrealistic.

The results of the swell calculation is presented below.
The calculations themselves can be found in Appendix D.

Waves following Path 1 of figure 3-13 are first refracted at Lyford Cay.
When they travel over the shallows of West Bay water depths of 1.00 m. are encountered. There a large amount will break.
It is assumed that the wave height -depth ratio (H_s/d) is 0.50.
After the shallow water depths refraction occurs from shallow to deep water at Clifton Point. The direction of the waves will thereby alter to southwestern direction and will not be able to reach the project's location. The project site can only be reached by diffracted waves around Clifton Point. The amount of diffraction is relatively high. Therefore the influence of swell from waves travelling by path 1 can be neglected.

Swell travelling along Path 2 will diffract at the small island of Goulding Cay in front of the western coast. The result of diffraction on the swell arriving from the directions of 320° to 20° is summarised in table 3-5.

Wave direction		H_s at Wave Rose A (%)				expected H_s at Clifton Point (%)		
	K_d	< 1.00 m	1.00-1.50	1.50-2.00	2.00-3.00	< 0.50 m	0.50-1.00	>1.00
320°	0.50	0.42	0.42	0.14	-	0.50	0.48	-
330°	0.30	0.42	0.42	0.14	-	0.88	0.10	-
340°	0.15	0.56	0.42	0.14	-	1.13	-	-
350°	0.13	0.28	0.28	0.14	-	0.70	-	-
0°	0.10	0.28	0.28	0.28	-	0.84	-	-
20°	0.08	1.69	0.56	0.14	0.14	2.53	-	-

Table 3-5 Influence of swell, according to Path 2

For table 3-5 the wave height distribution is used which represents the total wave climate from a certain direction. The swell is included in this data.
The results of both tables show the influence of swell when all waves are categorized as swell. This is however not realistic.
When the amount of swell is approximated by 20 % of the total wave climate, the swell with H_S larger than 0.50 m. only occurs for 0.10 %.
This relatively small amount can be neglected.

From the above it can be concluded that swell has no influence at the location of Clifton Point.

Sensitivity analysis

To check the sensitivity of the determination of the amount of swell, the influence of some parameters used in the calculation has to be analyzed.

One of the most important parameters is the swell period. In the calculation a swell period of 15 s. is assumed. By increasing the swell period to 22,5 s. (factor 1.5), the wave length is expected to increase by a factor of 2,25 to L= 790 m. ($L \cong T^2$).

For path 1 the r/L ratio alters in 2, the refraction factor K_r only shows a minor difference. The wave celerity in deep water will increase by a factor of 1.5 ($c = \frac{gT}{2\pi}$). The angle of the wave crest with the shore line as a result of the refracted wave only shows a minor decrease; K_r decreases by approximately 1 %. But still swell is of no influence at Clifton Point.

Also for Path 2 the diffraction factor increases by roughly 20 % for the higher K_d (320° to 330°) and 50 % for the lower K_d (340° to 20°). The effect on the total influence of swell can still be neglected.

Influence of waves from western direction

When the waves from western direction of rose B and C are compared, it can be expected that waves at the location of rose B are of higher influence due to the more open area of the location.

When the percentages of wave height occurrence of both wave roses are compared, it can be seen that the influence of waves from western direction is higher for rose C. Therefore it can be stated that the influence from western direction is more dominating for wave rose C. This results in a more active wave climate. With the uncertainty of the local wave climate, the western waves from rose C are selected for a safer solution.

For percentage of total time;	W			WNW			NNW		
H_s> 0.50 m	260°	270°	280°	290°	300°	310°	320°	330°	340°
Wave rose B	0.16	0.16	0.16	0.16	0.16	0.32	0.64	0.95	1.43
Wave rose C	0.28	0.42	0.42	0.56	0.70	0.70	0.70	1.41	1.55

Table 3-6: Comparison of wave height for wave rose B and C

3.6.3 Deep water wave climate Clifton Point

The former two paragraphs show that swell from the Atlantic has no significant influence on the local deep water wave climate at Clifton Point.
Also the percentages of wave climate for western direction remain constant.
The wave climate of Clifton Point will mainly correspond with the wave climate of rose C.

The local deep water wave climate is presented in figure 3-14.
The entire data set of the local wave climate is presented in Appendix C.

For all wave roses rendered from the NOAA data the percentage of calm wave conditions is known. For the deep water wave climate at Clifton Point an assumption is required. For the calm wave conditions a percentage of 10 % is assumed. This percentage is lower than the percentage of wave rose C, thereby increasing the safety of the approximated wave climate at deep water.

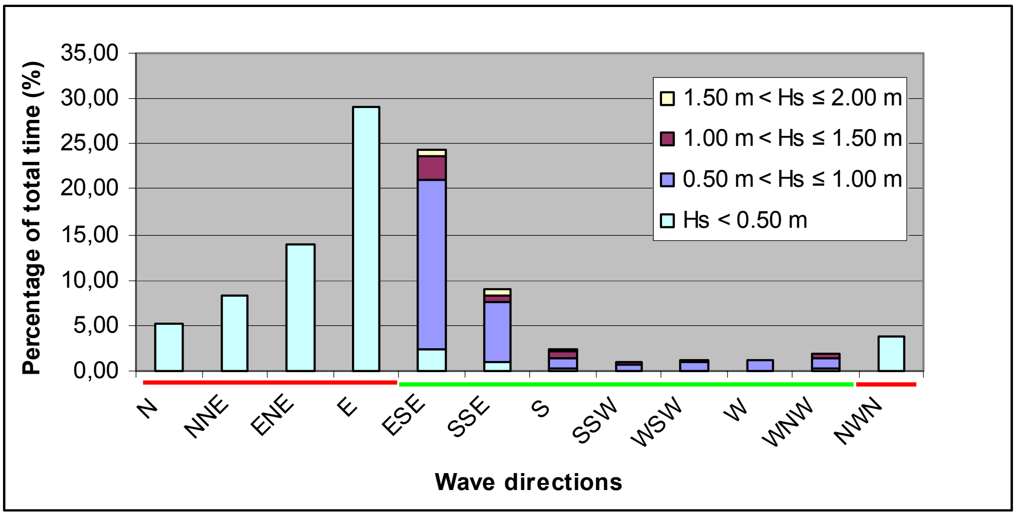


Figure 3-14: Statistics of deep water wave climate at Clifton Point

Certain wave directions which occur at the location of wave rose C, will occur at the location of Clifton Point under limited conditions.
These directions can be seen in figure 3-14. At the location of Clifton Point only wave directions occur, which are coming from the directions in the segment of 115° to 315°. The waves from the directions in the segment of 315° to 115° don't occur at Clifton Point at all. These last waves, which have a part of 58 % of the wave total at buoy C, are therefore transformed to insignificant waves with a wave height smaller than 0.5 m.

The wave segments which are considered insignificant are coloured red.
The green segments are the parts which remain unchanged.

By translating the data corresponding with wave rose C directly to the location of interest, the following can be noted:

The waves propagating over the shallow banks are of a limited wave height. The influence of altering the route of the travelling waves as presented in figure 3-15 is therefore assumed to be insignificant to the wave statistics.

Under different conditions the varying influence of refraction and changing bottom levels has to be checked.

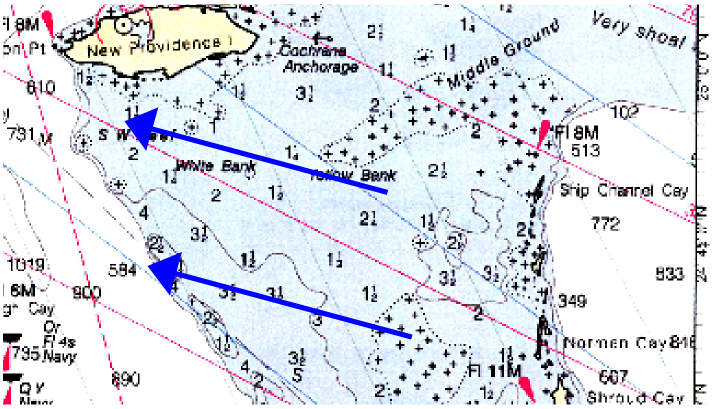


Figure 3-15: Waves travelling over Great Bahama Bank

3.6.4 Deep water extreme design waves

The design conditions for the port structures are all based on extreme conditions. Besides the limitations of the construction material all structures have to withstand extreme weather conditions.

The project site is well protected against high waves from northern and eastern direction. Only from western and southern direction it is possible that waves enter with a wave height larger than 3.00 m. In the five year time series of the available data there are only waves in the category of a H_s between 1.50 m. and 2.00 m. These waves contain 0.27 % of the total time for a southern direction. The rest of the waves from a western and southern direction are smaller than 1.50 m. (figure C-4, Appendix C). It can therefore be concluded that only extreme waves occur during hurricane conditions.

Because of the occurrences of these two extreme events within the 5-year span, it is unrealistic to conduct an extreme probability study using the NOAA wave data to determine the design waves of long-term return periods. In this study, a windwave hindcasting technique was utilized to predict extreme design waves.
The technique, documented by Silvester and Hsu (1993), includes the effects of wind speed, fetch, water depth, and varying wind durations.

When a hurricane passes by, the anticlockwise cyclone may generate winds and waves from all directions. Also, depending on the speed and the size of the hurricane itself, the effective fetch and wind duration may vary.

Within the eye of the hurricane the surface winds are absent and the atmospheric pressure. The maximum wind velocity occurs at the edge of the hurricane’s eye. From that point the wind velocity reduces radially.

The fetch length is determined by the formula of:

$$\frac{F}{R'} = -0.002175 \cdot U_{\max}^2 + 0.01506 \cdot U_{\max} V_F - 0.1223 V_F^2 + 0.219 \cdot U_{\max} + 0.6737 \cdot V_F + 0.7980$$

where $R' = 22.500 \log_{10} R - 70.800$

The graph of this function is presented in figure 3-16.
Since the radius of the eye of each individual hurricane is unknown, a mean radius is assumed. For each hurricane mentioned in table 3-3, a radius of 40 km is assumed, based upon the dimensions of a hurricane of figure 3-17. The forward velocity of the hurricane's eye (V_F) for each hurricane is 10 m/s.

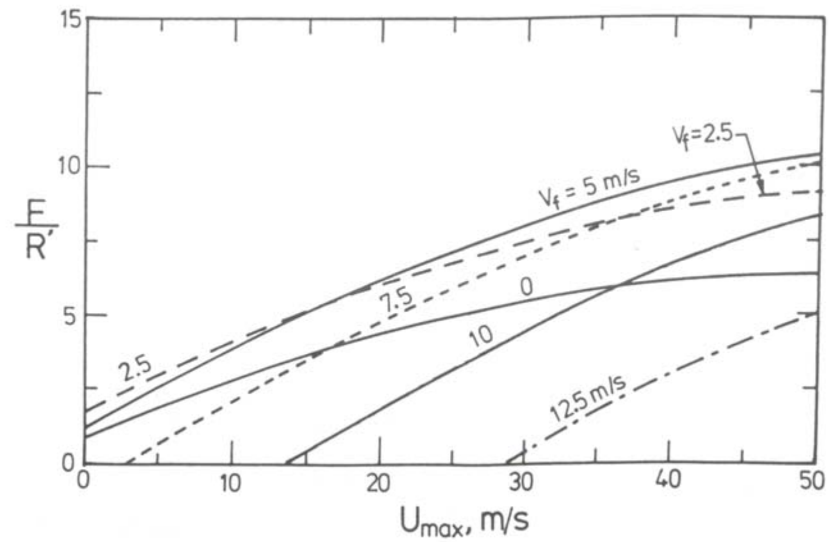


Figure 3-16: Fetch length as function of wind velocity

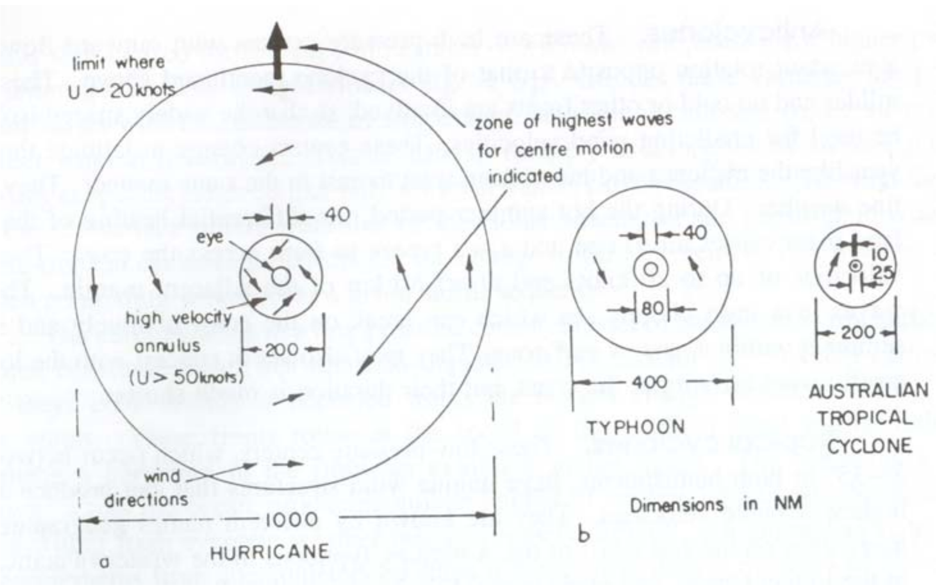


Figure 3-17: Dimensions of cyclone winds

As a result the fetch length of each hurricane is calculated. For all hurricanes the fetch length is larger than the actual fetch length allowed by local geometry. Therefore an upper limit of 40 km is chosen for the local fetch length. This length is determined in figure 3-18.

Since the hurricane will travel in westward direction the wind field and thereby the resulting wave field from southern direction cannot by fully developed. Although the open sea in southern direction shows a potential fetch larger than 200 km, the effective fetch is relatively small. It is expected that the effective fetch originating from the hurricane is less than 40 km. The water level elevation from western direction will be larger than the elevation from southern direction.

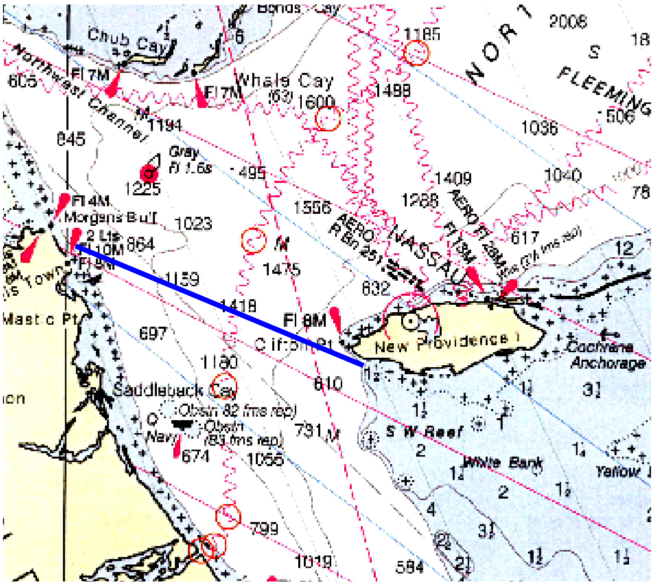


Figure 3-18: Fetch of normative hurricane at Clifton Point

The wave height as part of the wave field developed by the hurricane is a function of the effective fetch and maximum wind speed of the hurricane. The wave height is determined by the following formula:

$$\frac{gH_{\max}}{U_{\max}^2} = 0.0016\sqrt{\frac{gF}{U_{\max}^2}}$$

For the design conditions the significant wave height is required instead of the maximum wave height. For the conversion the H_{max}/H_s ratio is determined by the formula of:

$$\left[\frac{H_{\max}}{H_s}\right]_{mean} = \sqrt{\frac{\ln N}{2}} + \frac{\gamma}{2\sqrt{2\ln N}}$$

where: N = number of waves during storm conditions (= 1000 waves)
γ = constant of Euler (= 0.5772)

The one minute maximum of the wind speed and the corresponding return period is not equal to the determined wind speeds of table 3-7.
This is because the effective fetch also has a lower limit of the wave height. Tropical storms which feature a maximum wind speed smaller than 35 knots result in a H_{max} of the wave field of 0 m. The data set used to determine the maximum wave height is therefore limited by waves in the range of 35 knots and higher. The NOAA data set decreases from 63 to 46 samples.

The maximum wave height at deep water in front of Clifton Point therefore has the following results.

Return period (year)	One-hour maximum sustained wind speed (m/s)	Significant wave height at deep water (m)
25	37.8	2.1
50	45.5	2.6
100	53.0	3.2

Table 3-7: Design effective wind speeds, New Providence

3.7 Water level during extreme weather conditions

The most extreme water level occurs when a hurricane passes New Providence. A hurricane causes a storm surge which arrives at the new port's location. The storm surge represents a set up of the water surface caused by the winds and waves of the hurricane. The water level will also be affected by the local decrease of atmospheric pressure.

The tidal elevation of the water level also contributes to the extreme water level. The chance of spring tide occurring while a storm surge arrives at the target location has to be included.

A hurricane which passes the target location at a distance of several 100 km can still cause a considerable rise of the water level. In that case the target location itself is not affected by relatively high wind speeds.
Also waves and wind from other locations can have a possible effect on the target location. In case of hurricanes influencing Clifton Point it is assumed that the increase of water level is solely caused by the hurricane. Influence of waves and wind from other directions are considered to have no effect on the extreme conditions.

3.7.1 Pressure setup

During a hurricane a pressure drop occurs in the vicinity of the hurricane's eye. This results in a maximum pressure drop in the eye itself. The pressure drop correlates with a local rise of the water level.
As a rule of thumb the water level rises with 1 cm. for an decrease of 1 hPa (mbar) of the atmospheric pressure.

Return period (year)	Hurricane Eye Pressure (mbar)	Maximum Pressure Setup (m)
25	956	0,44
50	945	0,55
100	935	0,65

Table 3-8: Maximum Pressure Setup for New Providence Island

3.7.2 Wind setup

The increase of the mean water level due to the water piling up on the shore is a result of friction between the hurricane wind and the water surface.
The friction of the wind speed over the water surface results in inclination of the water level in situations with limited water depths.
The wind set-up is not only dependent on the wind speed, but also on the fetch length and the water depth.

To determine the effects of wind set-up, winds caused by the hurricane from all wind directions have to be evaluated.

By using a Hydrodynamic modal in MIKE 21 (HD) the amount of wind setup was determined in the EIA report. Since the wind velocities for this report vary, the wind set up is translated by using the relation of:

$$\Delta h_w = c_w F_w \frac{U^2_{\max}}{gd}$$

The fact that the EIA report only uses tropical storms, starting at class H1, explains the difference of wind velocities. Their data set also has a different radius (160 km) for recording a hurricane. Finally the range of 1852 to 2004 differs, a difference of two years.
The ratio of the wind velocities (U_{wmax} EIA/ U_{wmax} paragraph 3.6.4) used to determine the maximum wind setup is presented in table 3-10.

Since the wind setup and wind velocity are quadratic related, the ratio of table 3-9 has to be squared.

The most extreme situation is presented in figure 3-19, where the wind reaches New Providence from a southern direction. The data and calculations, where this table is based on, are not available. The wind speed used for the modelling of figure 3-19 is unknown. The figure can therefore only be used as an indication.

Return period (year)	Ratio of U_{wmax} (-)	Maximum Wind Setup EIA (m)	Maximum Wind Setup (m)
25	0.95	0.44	0.40
50	1.00	0.50	0.50
100	1.06	0.55	0.61

Table 3-9: Maximum Wind Setup

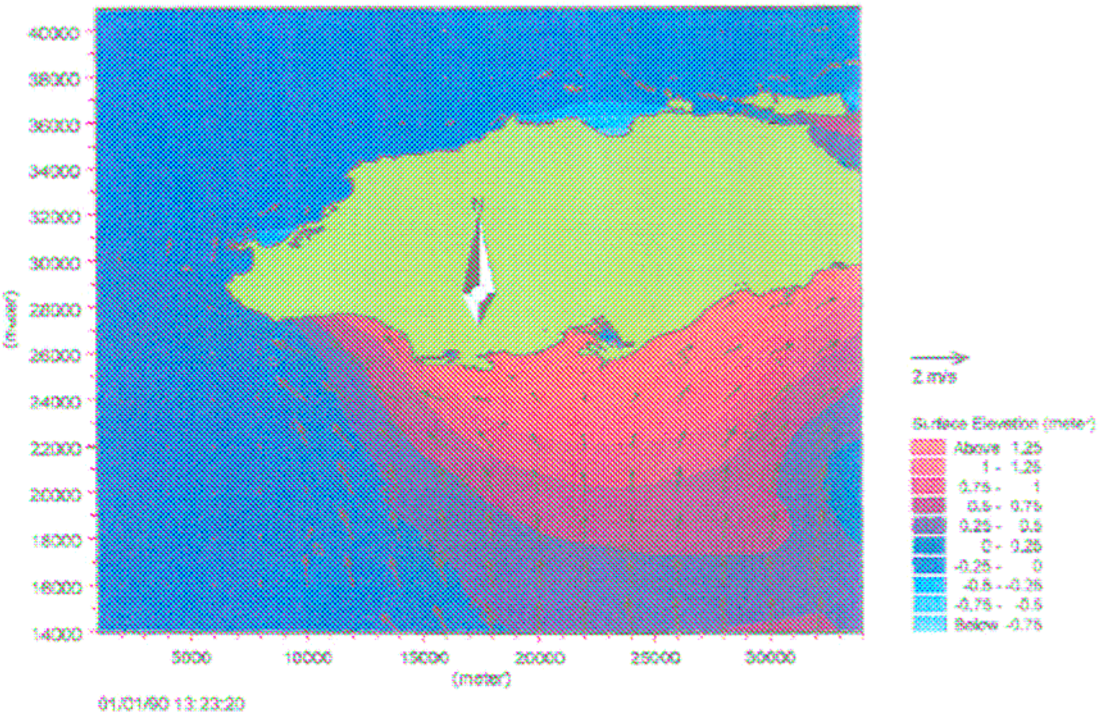


Figure 3-19: Indication of maximum wind setup from southern direction

3.7.3 Wave setup

Wave setup is the superelevation of the water level caused by wave climate. The waves breaking before the point of interest are causing the water level to rise. At the breaker point, the location where a wave breaks, a local setdown occurs; from that point the water level will rise.
It is important to determine the breaker depth including the water elevating components of pressure setup, wind setup and the influence of tides.
The local setup is determined by:

$\overline{\eta}_b = -1/16\gamma^2 d_b$ and $\overline{\eta} = \overline{\eta}_b + \frac{d\overline{\eta}}{dx}\Delta x$, where $\frac{d\overline{\eta}}{dx} = \left[\frac{1}{1+\frac{8}{3\gamma^2}}\right]\tan\beta$

where:	$\tan\beta$ = slope of shore	(°)
	γ = breaker parameter	(-)
	d_b = depth of breaking	(m)

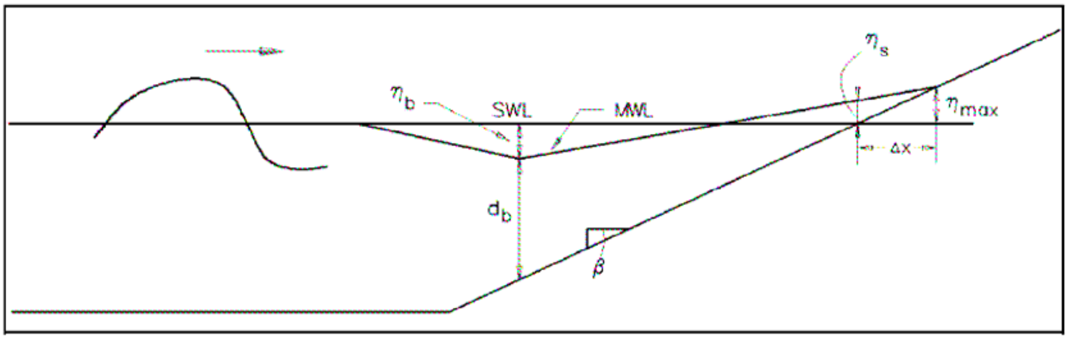


Figure 3-20: Wave setup

When it is assumed that waves will break at $H_s/h = 0.5$, it can be concluded that the extreme design wave of 1:100 years will not break at deep water.
In paragraph 3.8 the wave setup for smaller water depths will be determined.

The breaker parameter of $H_S/h = 0.5$ is based on $H_{max} \cong 0.14 L \tanh\left(\frac{2\pi h}{L}\right)$.

When the exact wave length at deep water (L_0) is unknown, $H_S/h = 0.5$ provides a proper indication.

3.7.4 Tides

The maximum increase of the water level occurs at spring tide.
The events of a passing hurricane and spring tide don't occur simultaneously.
It is assumed that during a hurricane event in the vicinity of Clifton Point High Astronomical Tide (HAT) occurs, a water level elevation of + 0.35 m. MSL.

The probability of an occurring spring tide is neglected.
This because the difference between HAT and springtide is relatively small, a difference of 0.05 m.

3.7.5 Total water level elevation during extreme weather conditions

The total elevation of the water level during extreme weather conditions is not a simple add up of the 4 components described in the last paragraphs. The maximum value of wind and wave set up and the local fluctuation of atmospheric pressure at the new port's location is not simultaneous; The maximum setup of wind and waves will arrive when the maximum fluctuation of atmospheric pressure has passed.
This can be seen in figure 3-21, where a sample distribution of the drop of atmospheric pressure is presented.

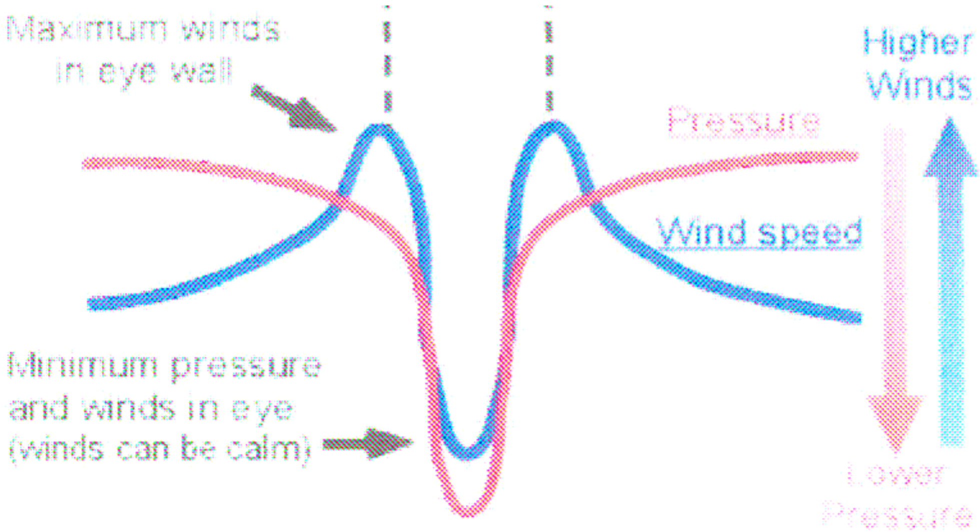


Figure 3-21: Distribution of atmospheric pressure during hurricane presence

Since the effect of the setup of wind and waves is larger than the influence of atmospheric pressure, the decisive moment of water level elevation is when the set up of wind and waves is maximal. For the atmospheric pressure component an assumption is required. It is therefore assumed that the contribution of the atmospheric pressure is 70 % of its maximum.

However it has to be noted that for the design conditions the situation is schematised. In real time the wind and pressure setup will show complex dynamics due to the interaction with the characteristics of the shallow water depths. It is therefore possible that a larger phase differences occur, causing a smaller total setup. The percentage of 70 % is therefore a safe estimation.

In relation to the return period of all components the astronomical tide is independent. As mentioned in paragraph 3.7.4 the occurring of the event of a hurricane and spring tide are not related. The effect of the tide on the total water elevation is integrated by using a deterministic method. It is assumed that during hurricane conditions high tide occurs.
The final result is presented in table 3-10.

Return period (year)	Maximum pressure setup (m)	Maximum wind setup (m)	Design extreme water level elevation, incl. HAT (m)
25	0.31	0.40	+ 1.06
50	0.39	0.50	+ 1.24
100	0.46	0.61	+ 1.42

Table 3-10: Design water level elevation

3.8 Extreme design conditions at Clifton Point

In the previous two paragraphs the extreme conditions were determined for deep water. Since the wave height and elevation during extreme conditions are affected by smaller water depths the deep water conditions are translated to local conditions at shallow water.

Waves arriving at shallow water depths will break.
It is assumed that these waves will break according to a H_s/h ratio of 0.5.

3.8.1 Local water level elevation

In addition to the elevation of the water level at deep water, as determined in paragraph 3.7, the effect of wave setup at the shallow water location has to be checked. The setup depends on the breaking depth of the incoming waves. For the local conditions the setup is determined at the bottom level of -10.00 m. and -5.00 m. MSL.

The distance from deep water to the shoreline as described above is insignificant in relation to the scale of the fetch and the radius of pressure fluctuation. The influence on the water level rise can therefore be neglected for these components.

The slope in front of the shore is approximated by a constant value of 1:40 ($\tan \beta$= 0.025). The breaker parameter is estimated by a value of 0.5.

It can be concluded that the waves will only break at the location of a bottom level of – 5.00 m. MSL. under the conditions of a return period of 100 years.

For a coast with a constant slope of 1:40 the setup can be neglected.
The setup from the bottom line of -5.00 m. MSL is increasing by 0.002 m. per meter. However when structures are implemented to prevent the port from hindrance due to incoming waves, the structures are expected to be designed with steeper slopes. In that case the influence of wave setup has to be checked. This will be part of the breakwater design in chapter 9.

Return period (year)	Extreme wave height (m)	Water depth at -5.00 m. MSL (m)	Breaker parameter (-)	Wave will break at bottom level (m)
25	2.1	6.06	0.35	-3.14 MSL
50	2.6	6.24	0.42	-3.96 MSL
100	3.2	6.42	0.50	-5.00 MSL

Table 3-11: Local elevation of water level

3.8.2 Local extreme wave height

In the previous paragraph the effect of wave setup can be neglected for the current bathymetry of the project site. Therefore the extreme wave heights as determined in paragraph 3.6.4 remain unchanged.
The extreme wave heights are presented in table 3-12.

Return period (year)	Local H_s during storm conditions (m)
25	2.1
50	2.6
100	3.2

Table 3-12: Local extreme wave height

3.9 Sediment transport

Breaking waves in the surf zone, combined with nearshore currents, cause sediment movement in the coastal zone. Sediment movement is a complex process and depend on many environmental factors.
Understanding sediment movement is critical in design and maintenance of these coastal structures.

For the port location at Clifton Point limited sediment transport is expected due to the lack of sediment and deep water depth.
Therefore, the maintenance dredging frequency for the port's basin is expected to be low.

3.10 Soil conditions

The south coast of New Providence mainly exists of a limestone shore.

Limestone is a sedimentary rock composed principally of calcite dolomite.
It is commonly composed of tiny fossils, shell fragments and other fossilized debris. Some varieties of limestone have an extremely fine grain.
Limestone can be considered as a soft rock.

The material has many varieties of composition, like hardness and porosity.
To determine the exact composition of the rock soil samples of the area are required. The composition of soil at bottom level is presented in figure 3-22.
It is expected that the sea grass and sand soil are covering the limestone soil.
The depth of the soil body is assumed to be relatively small.

Since no soil samples are available it is assumed that the whole area surrounding Clifton Point exists of limestone rock.

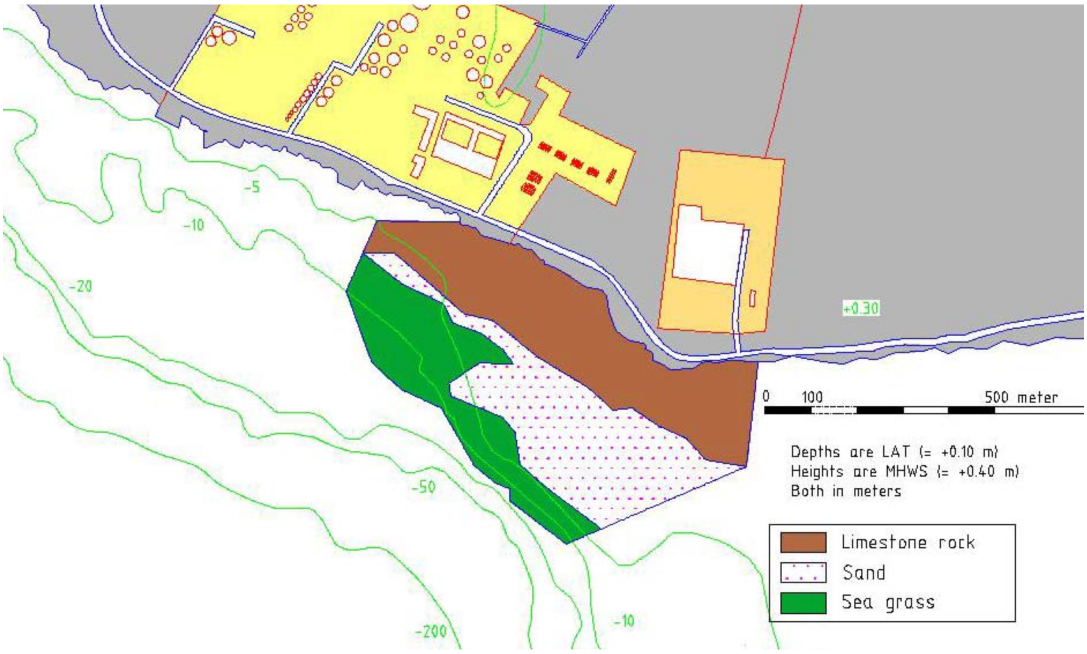


Figure 3-22: State of sea bottom at Clifton Point

4 Current port structure of New Providence

By creating an overview of the current port activities in the port of Nassau, the functions required for the new port at Clifton Point become clear.

In the present situation several types of commodity enter the island of New Providence. Each type has its own arrangement of land use, also the requirement for loading/unloading equipment differs.
In general can be said that all cargo transported to the island is domestic cargo.

In the following overview the major shipping companies are summarized. With each company the location, the type of commodity and shipping schedule is mentioned. Furthermore the locations of all companies are presented in figure 4-1. The companies related to the orange marked numbers of table 4-1 are planned to be relocated to the new port facility.

Number on map	Shipping company/ shipper	dedicate terminal	Other terminal used	Main type of cargo	shipping schedule interval
1	Betty K Line	Kelly's Dock	None	Containers lo-lo / break-bulk	weekly
2	Pioneer Shipping	Union Dock	None	Containers ro-ro	2 p/wk
3	Tropical Shipping	John Alfred Dock	Arawak Cay	Containers lo-lo	4 times daily, Arawak 1-2 p/wk
4	Seabord Marine	Symonettes Dock	Arawak Cay	Containers ro-ro	weekly
5	Crowley	None	Arawak Cay	Container lo-lo	2 p/wk
6	MSC	None	Arawak Cay	Containers lo-lo from Freeport	3 p/wk
7	Inter-island feeder service	None	Arawak Cay	Containers lo-lo / break-bulk	?
8	Mosko	Mosko Terminal	Martin Marietta Dock, Arawak Cay	Cement, limestone, granite, sand	irregular
9	Cars ro-ro	Prince George Dock West	None	Cars	monthly
10	Water & Sewage Corporation	Own facility west of Arawak Cay	None	Water	?
11	MailBoat Cy	None	Arawak Cay, Potters Cay	General cargo and passengers inter-island	25-40 p/wk
12	Sailing Boats	Arawak Cay	None	Break-bulk	?
13	Cruise Lines	Prince George Dock	None	Cruise passengers	2-7 daily
14	Passengers Paradise Island	Passenger ferry terminals near Prince George Dock	None	Passengers	?
15	Passengers Inter-island	Potters Cay, near MailBoat services	None	Passengers	?

Table 4-1: Summary of present companies using the port of Nassau

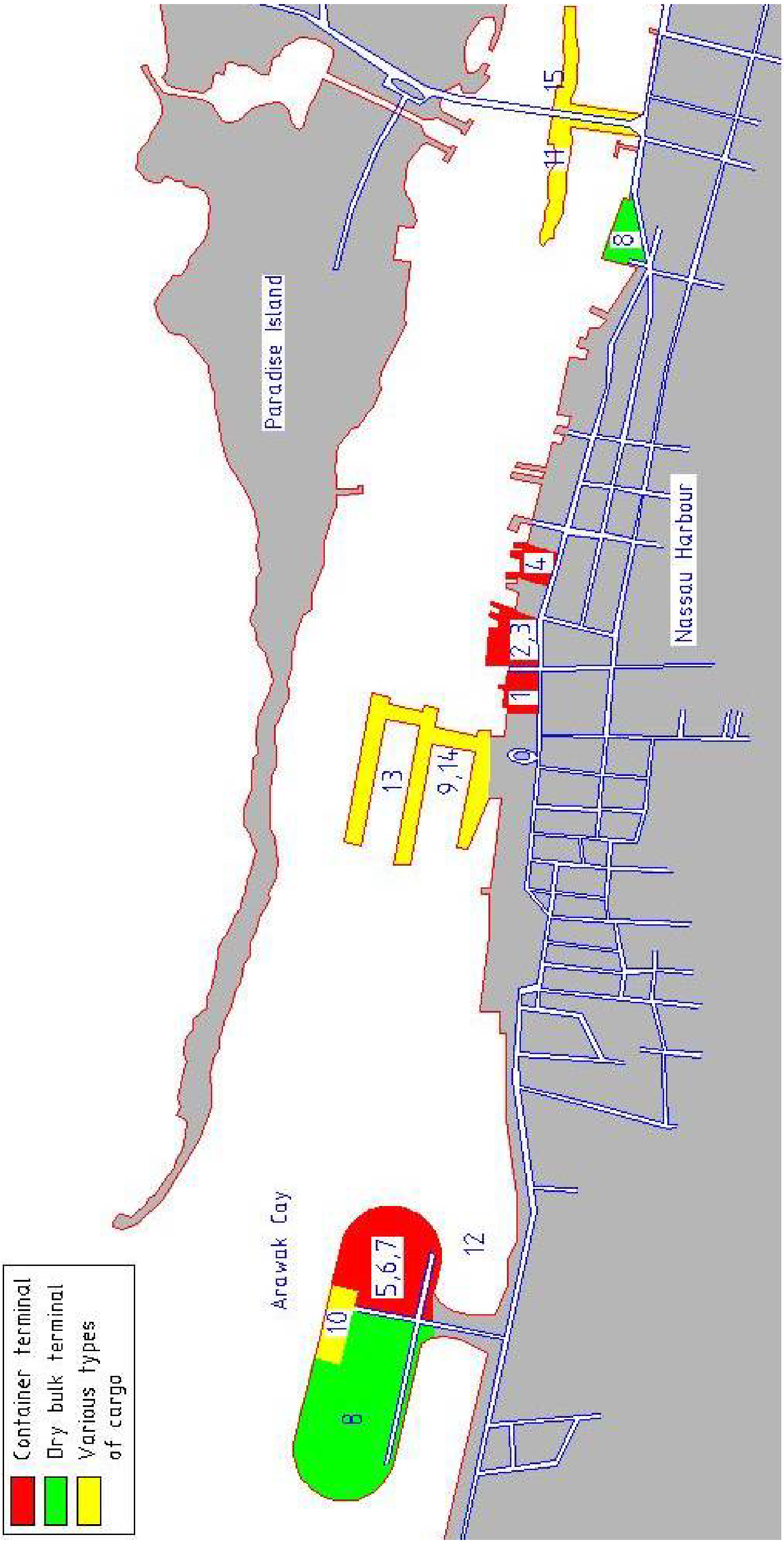


Figure 4-1: Overview of present companies using the port of Nassau

4.1 Containers and break bulk.

In the port of Nassau the major part of the cargo exists of containers.
In table 4-2 the terminals are mentioned corresponding with the shipping companies from table 4-1. The total throughput at Arawak Cay includes the cargo of the shipping companies of MSC, Crowley and a share of Tropical Shipping. The terminals of Tropical Shipping and Arawak Cay contribute to a share of 80 % of the total throughput of cargo of the 5 terminals.

2006

Container handling activity per terminal				TEU share in		
Terminal	Throughput '000 tons	Calls	Containers TEU	Total	Share 40 ft	Call size in TEU
Betty K.	33	116	1.398	2%	56%	12
Seaboard Marine	69	153	7.596	10%	83%	50
Tropical Shipping	328	236	33.295	46%	83%	141
Pioneer Shipping	70	105	5.738	8%	68%	55
Arawak Cay	169	408	25.115	34%	84%	62
All terminals	670	1.018	73.142	100%	81%	72

Table 4-2: Container handling activity per terminal in Nassau

The cargo handling equipment varies for each shipping company and is mentioned in table 4-3. Some companies handle the cargo with self unloading ship cranes, others use mobile cranes on the quay.
All companies use forklifts for terminal operations. Several companies also own a reach stacker.

TERMINAL	SHIP TO SHORE EQUIPMENT	TERMINAL OPERATIONS EQUIPMENT	FCL - LCL	DWELL TIME
Tropical shipping	Heavy mobile crane	• 10 forklift 2.3 t • 1 forklift 5.4/6.8/13.6 t • top loaders • 2 reach stackers		
Pioneer shipping	Mobile crane – 65 t	• forklift	60-40	7
Seaboard Marine	3 tractors RoRo	• forklift 2.3 – 4.1 t • 1 forklift 23.6 lbs		15-20 5 reefer
Betty K Line	Ship's crane – 25t Side doors	• 8 forklifts 2.3 t • 1 reach stacker 45 t	C-GC 20-80	
Arawak Cay	Mobile crane	• heavy forklifts • 1 reach stacker		

Table 4-3: Container handling equipment per terminal in Nassau

Compared with the rest of the world, the amount of break bulk transport in the Caribbean is relatively high. Because of the worldwide decrease in common use of break bulk vessels during the last decades, no efficiency of scale occurs by increasing the vessel's dimensions. This is the explanation for the relatively small vessels, with exception of the combined pallet/container vessel. An example of these small vessels can be found by the vessels of Betty K Line in table 4-5.

Table 4-2 shows that the smallest terminals have the highest rate of break bulk cargo. The majority of the break bulk cargo is carried by Betty K Line, which operates between Nassau and Miami with two small general cargo vessels. It is possible for ships to have a combination of break bulk and container cargo.

A large part of the container ships is also capable of transporting RoRo containers. These RoRo (Roll on Roll off) containers are TEU, which are loaded on truck transport continuously, also during sea transport. Ships with a combination of RoRo and LoLo containers call the port frequently.

4.2 Cars

In the current situation the import of cars for the domestic market occurs in several ways. Besides the bulk import which uses dedicated car carriers, cars are also transported on deck of the container ships of Pioneer Shipping or in containers of different shipping companies. The share of cars transported by car carriers in 2005 was around 70 %. Car carriers call the port of Nassau each month and berth at the Prince George Wharf, where Pioneer Shipping is responsible for the stevedoring operations. The car carriers are operating on a basis of a fixed delivery schedule; a small part of the total cargo of the carrier is unloaded at the various ports of call.

4.3 Liquid bulk

On the island of New Providence the liquid bulk cargo can be divided in several categories; these comprise petroleum products, LPG (Liquefied Petroleum Gas), water, molasses and alcohol. These subcategories are handled at Clifton Point, with the exception of the water.

The petroleum products and the LPG bulk both have a separate berthing jetty. The molasses and alcohol, belonging to the Commonwealth Brewery, are shipped at a small quay wall at Clifton Pier. The exact location can be seen in figure 4-2. Water is shipped by the Water and Sewerage corporation from Andros Island to an open storage west of the Marin Marietta Dock on Arawak Cay. For the location see figure 4-1.

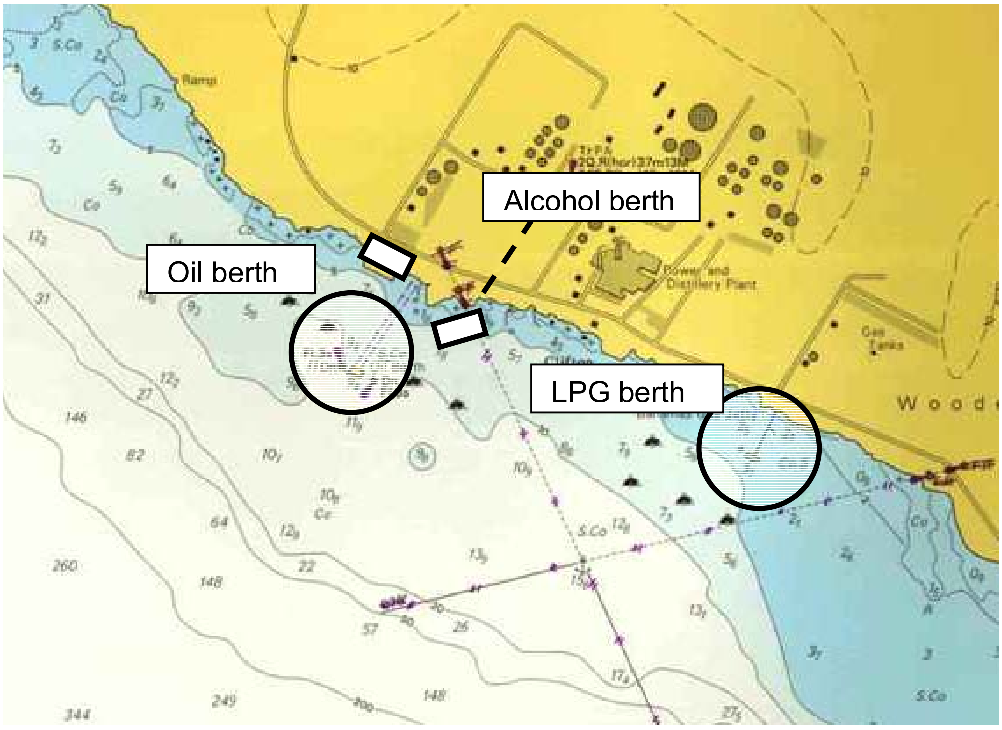


Figure 4-2: Current port structure of Clifton Point

The following text is subtracted from Lloyd's port register and include the specifications of the berths presented in figure 4-2.

- Clifton Pier: a small break bulk facility, used for shipping the alcohol and molasses cargo of the Commonwealth Brewery.
- Clifton tanker pier: accommodation to vessels with a maximum Length of 192 m and a maximum draft of 10.9 m. Maximum DWT of 40.460 ton, maximum arrival disp. 48.768 mt. No berthing within 1 hour of low water. Night berthing permitted, 1 to 2 tugs required. Min parallel body 67 m.
- Clifton LPG berth: anchor and buoys to seabed LPG pipeline, taken on starboard side, discharging to nearby shore pressure vessels.

Piloting services for the berths are obligatory and should be contacted at least 24 hours in advance. Tankers are not permitted to berth if wind speeds exceed 20 knots. With winds larger than 15 knots 2 tug vessels are recommended. The vessels must have onboard anchor available to use during berthing, also to assist in deberthing. With currents larger than 2 knots and waves or swell exceeding a significant wave height of 1 m., cargo operations must be suspended.

The petroleum products are further distributed over the islands by small coastal tankers in the range of 300 -1000 dwt, making about 560 calls annually.

For the storage of LPG there are 32 tanks available; 22 tanks have a capacity of 113.000 liters and 10 tanks with a capacity of 200.000 liters.
The total storage has a volume of 4.5 million liters. Since the density of LPG is 0.6 ton per cube of LPG, the total storage is 2700 tons.

	Boiling Point (°C)	Liquid Density at 15 °C (kg/m3)	Gas Density at 15 °C (kg/m3)
Butane	0.5	570-580	1.9 -2.1
Propane	-42	500-510	1.4 -1.55

Table 4-4: Characteristics of fuel gasses

The surface area appointed for storage of LPG has the dimension of 2.9 ha, where 1.6 ha is currently used.

The current capacity of the storage facilities for the use of petroleum products is however unknown. It is assumed that the area of Clifton Point retains sufficient space for optional extension of the storage volume.

4.4 Dry bulk

Dry bulk can be divided in the following subcategories: cement and aggregates e.g. limestone, granite and sand. In the present situation the company of MOSKO Cement is responsible for the discharge of all the categories mentioned above; these operations are executed at the Martin Marietta Dock and at Arawak Cay, as seen in figure 4-1. Unloading is done by two pay loaders with 5 ton capacity buckets.
Further distribution is done by lorry or by barges with a capacity of 2000 to 3000 tons.

Another cement importer, Bahamas Cement, is located at Clifton Point.
In the current situation the total annual cement throughput of New Providence is divided 50-50 between MOSKO Cement and Bahamas Cement.

4.5 Cruise

In the current situation Prince George dock has three possible accommodations for cruise vessels. Each day up to a maximum of 7 ships berth along the pier, handling 900 vessels a year. Near the cruise terminal several small ferry boat terminals are located, which lines connect the centre of Nassau with Paradise Island. Passengers for inter-island travel board from Potter's Cay with high speed catamarans and other types of craft.

4.6 Other trades

Some small volumes of cargo concern inter-island shipments in small boats operated by Mailboat Cy, mixed cargo/passenger ship operators and non-liner ship operators. This trade is small in volume and is mainly handled at Potter's Cay. Because of their relatively small draft they are able to enter the sea in eastward direction.

Finally, there is also freight transport with Haiti by sailing ships operated by Haiti nationals under Bahamian flag. The ships are anchoring east of Arawak Cay. This trade has the full characteristics of an affair in the informal sector.

4.7 Example of current vessels in port of Nassau

Table 4-5 provides an indication of the vessels entering the port in the present situation. The vessels will be used as design vessels for the new port.

Sector	type	GRT	NRT	DWT	cars	TEU	LOA	knots	draft
Container vessels									
Tropical Shipping									
Tropic Jade	LO/RO	1.827	548	2.563		176/44	84	15,5	4,80
Tropic Lure	LO/RO	1.827	548	2.563		176/45	84	15,5	4,80
Tropic Mist	LO/RO	1.827	548	2.563		176/46	84	15,5	4,80
Tropic Sun	LO/RO	6.536	1.961	7.450		400/87	110	15,0	6,20
Seaboard Marine									
Seaboard Spirit	RR	2.295	778	2.596		234	96	10,0	4,10
Discovery Sun	RR	11.979	4.005	2.337		100	134	18,0	5,50
Betty K.									
Betty K VI	Break bulk Ship	1.457	994	1.070		37	62		3,90
Betty K VII	MP			1.250			71	12,5	
Pioneer Shipping									
Baltic Breeze						200	87		
Pioneer Star	Lo-Lo					80	75		
Arawak Cay									
Free Port Flyer						64			
United Spirit-inter-island						28			
KCT inter-island						18			
Fiesta Mail	Ferry	2.485	910	710			69		3,00
Wesertor	MP	4.180	2.108	4.985		408	109	15,6	6,00
Dry bulk vessels									
Bahama Spirit: limestone aggregates	dry bulk carrier	26.792	13.616	44.389			187	14,0	11,40
Glory Sun: bulk cement	dry bulk carrier	15.879	6.086	24.938			160	13,5	10,20
CSL Argosy: granite aggregates	dry bulk carrier	46.409	17.235	74.423			245		13,50
Tug barge: Dredged sand	dry bulk carrier			3.000					
Tankers									
Stella Azzurra	tanker			19.985			161	15,0	8,50
Ficus	tanker			32.229			171	15,5	11,40
Stolt Sincerity LPG	tanker	20.013	11.545	31.943			177	17,0	11,60
Jo Spirit (Bacardi)	tanker	4.425	1.456	6.248			107	13,5	7,00
Car carriers									
Eurasian Brilliance	car carrier	26.746	8.023	9.763	2583		159	16,0	7.60
Cruise ships (top size)									
Navigator of the Seas	cruise ship	138.279	105.131	9.616			311	22,0	8,60
Voyager of the Seas	cruise ship	137.276	105.011	11.132			311	22,1	8,80
Millennium	cruise ship	90.228	53.239	11.928			294	24,0	8,30

Table 4-5: Samples of current vessel types

5 Forecast per commodity

The design of the new port infrastructure will have to provide the required capacity. To determine this capacity a detailed forecast of the commodities processed by the new port is necessary. The forecast published in this chapter is based on the forecast presented by ECORYS.

The ECORYS report uses the years of 2005, 2015, 2025 and 2035 as reference. For this report the years of 2005, 2020, 2028 and 2035 are used.

This choice of design is based on the assumption that three design phases are elaborated. Each phase is accompanied with a realistic date of the year in which the construction phase will end. The design capacity of all construction phases are linked to the forecast of this chapter. Constructing a port with a capacity for the year 2015 for instance will not be practical, due to the expected end date in the year 2012 of the first construction phase.

It is however highly possible that certain construction phases are combined and executed in one phase. This is decided on the aspect of efficiency. It can be expected that increasing a quay with a relatively short length of 10 m. is not feasible from an economical perspective. It will also cause hinder to the vessels of the current port. This aspect will be reviewed in chapter 6.

	Start date	End date	Sufficient for capacity of year:
Design phase	-	2009	-
Construction phase 1	2010	2012	2020
Construction phase 2	2018	2020	2028
Construction phase 3	2026	2028	2035

Table 5-1: Construction phases of port design Clifton Point

The throughputs of the several commodities obtained from the forecast of ECORYS are translated to the referential years . This is accomplished by interpolated the available data.

The annually throughput of containers is the only commodity which uses three different scenarios of growth. For the design of the alternatives the medium scenario is used.

5.1 Expected growth of GDP

A region's gross domestic product (GDP) is one of the ways for measuring the size of its economy. The GDP of a country is defined as the market value of all final goods and services produced within a country in a given period of time. For the Bahamas the GDP is expected to grow according to the following rates:

Period	Expected growth of GDP
2005 – 2015	3.0%
2015 – 2025	2.5%
2025 – 2035	2.0%

Table 5-2: Expected annual growth of GDP

5.2 Containers

For the future expectations of the container transport of New Providence the growth of the throughput is related with the growth of the GDP.
During a period of 25 years a growth ratio between the annual growth of TEU throughput and GDP with a factor of about 2 has been found.

Period	TEU annual growth	GDP annual growth	Ratio of growth rates
1990-95	0,1%	-0,4%	-0,3
1995-00	9,5%	4,3%	2,2
2000-06	3,1%	1,5%	2,1
1990-2006	4,1%	1,7%	2,4

Table 5-3: Annual growth in relation to annual growth GDP

Expectations are that the ratio of growth rates will gradually decrease to about 1 at the end of the period of forecasting. This is based on the economical theory that a ratio higher than 1 cannot be maintained for a relatively long period of time.

	2005	2020	2028	2035
Scenario: medium				
Loaded containers '000 TEU	**66**	**142**	**193**	**220**
Annual growth rate (trend)		5,2 %	3,9 %	1,9%
Throughput/ GDP growth ratio		2.1	2.0	1.0
Scenario: high				
Containers high forecast	**66**	**177**	**251**	**302**
Annual growth rate (trend)		6,8 %	4,5 %	2,7 %
Scenario: low				
Containers low forecast	**66**	**113**	**146**	**160**
Annual growth rate (trend)		3,6 %	3,3%	1,3 %

Table 5-4: Forecast of throughput containers

The throughput determined for the medium scenario is reflected with more detail in the table 5-5. Since the call size of 73 TEU in 2005 is rather small, it seems reasonable that this will increase in time. In this way the ships improve their economies of scale. It is assumed that the number of calls remain constant. Since the port of New Providence is not the only port of call for the shipping companies, it is assumed that the call size is 24 % of the ship size. The percentage of RoRo transport is assumed to remain constant.

Imported containers	Category	2005	2020	2028	2035
	'000 TEU	66	142	193	220
	ship calls	905	905	905	905
	call size TEU	73	157	213	243
	ship size TEU	300	649	880	1004
	ship length m.	110	134	155	165
	share ro-ro	10%	10%	10%	10%

Table 5-5: Forecast of throughput containers of chosen scenario

5.3 Break bulk

It is expected that the amount of break bulk will decrease, following the global behaviour of this cargo in a delayed rate. It is therefore expected that the break bulk cargo will gradually be containerised. The amount of break bulk is assumed to be insignificant in the year 2028. The break bulk will then be transformed in 13000 TEU annually. This is already included in table 5-6.

Break bulk	Category	2005	2020	2028	2035
	'000 tons	127	127	0	0
	ship calls	100	100	0	0
	call size tons	1270	1270	-	-
	ship size dwt	1300	1300	-	-
	ship length m.	70	70	-	-

Table 5-6: Forecast of throughput break bulk cargo

5.4 Cars

As mentioned in paragraph 4.2, cars are imported by several methods. The major part (around 70 % in 2005) is imported by car carriers. Other cars are imported as cargo placed on deck or in containers of container vessels. It is expected that with the construction of a new port the share of the car carriers will cover the complete range of imported cars, a total of 7649 cars in 2005.

Because the economic state of the Bahamas can be considered as an upcoming development country, the percentage of car owners is expected to grow. In developed, industrialised countries the level of car ownership has been saturated and will increase with the growth of the population. Since the economy of the Bahamas is expected to grow, the grow rates of the delivered cars are set higher than the GDP, respectively 4 %, 2.8 % and 2 %.

Cars	Category	2005	2020	2028	2035
	units	7649	12949	15892	18254
	ship calls	40	68	83	95
	call size cars	191	191	191	191
	ship size dwt	9762	9762	9762	9762
	ship length m.	160	160	160	160

Table 5-7: Forecast of throughput cars

5.5 Dry bulk cargo

The demand for bulk materials on New Providence shows very large fluctuations. This is explained by the variation of the flow of projects on the road and building industry. Based on the data of the MOSKO Group the average throughput volume for each bulk type is presented in table 5-8.

2006	Volume in metric tons		
	average	min	max
Arawak Cay			
limestone aggregates	522000	365400	679000
granite aggregates	102000	71400	132600
dredged sand	295000	206500	383500
MOSKO dock			
bulk cement	45500	31850	59150
Bahamas Cement			
bulk cement	45500	31850	59150
Paradise Island			
Temporary Cement Plant	23000	16100	29900

Table 5-8: Variance of throughput dry bulk cargo, 2006

It is therefore assumed that the average amounts of annually throughput of the aggregates and sand bulk has a standard deviation of 30 % for determining the high and low scenario. The annually cement throughput has a standard deviation of 20 %.

It is assumed that the growth rate of the amount of bulk is positioned between the growth rate of the GDP and the population; therefore it is set on respectively 2.4 %, 2.0 % and 1.6 %.

Since the call size of the cement vessels at Clifton Point is relatively small, it is assumed that the size of these vessels will increase.
The call size of the vessels of MOSKO cement are expected to remain constant, because of the draft limitations.

The call size of both carriers of aggregates will also not increase due to the draft limitations. These draft limitations are determined by the maximum harbour depth of 10 m. It is an option to increase the maximum depth to allow larger vessels enter the new port. This option will be discussed in chapter 8.

Dry bulk cargo	Category	2005	2020	2028	2035
MOSKO Cement	'000 tons	45	64	73	82
	ship calls	8	11	13	15
	call size tons	5500	5500	5500	5500
	ship size dwt	9000	9000	9000	9000
	ship length m.	115	115	115	115
Bahamas Cement	'000 tons	45	64	73	82
	ship calls	52	52	52	52
	call size tons	873	1230	1404	1581
	ship size dwt	1048	1482	1692	1897
	ship length m.	70	78	81	85
Limestone aggregates	'000 tons	575	805	932	1041
	ship calls	23	32	37	42
	call size tons	25000	25000	25000	25000
	ship size dwt	44000	44000	44000	44000
	ship length m.	187	187	187	187
Granite aggregates	'000 tons	113	158	182	204
	ship calls	5	6	7	8
	call size tons	25000	25000	25000	25000
	ship size dwt	74000	74000	74000	74000
	ship length m.	245	245	245	245
Sand	'000 tons	325	455	526	589
	ship calls	108	152	175	196
	call size tons	3000	3000	3000	3000
	ship size dwt	3000	3000	3000	3000
	ship length m.				

Table 5-9: Forecast of throughput dry bulk cargo

5.6 Liquid bulk cargo

On the Bahamas the dominant use of petroleum products is based on the need for transportation and residential and industrial use.
Due to increases in fuel efficiency and a shift to more efficient products such as the shift from fuel oil to diesel oil in the past, it is expected that the growth rate of petroleum products is lower than the growth of GDP.
It is assumed that the growth of petroleum products and LPG is therefore half of the growth of GDP, respectively 1.5 %, 1.25 % and 1.0 %.

The growth of the alcohol and molasses cargo is presumed to act between the growth rate of the population and GDP. This results in a growth rate of respectively 2.4 %, 2.0 % and 1.6 %.

Because the oil tanker berthing at Clifton Point is already bound to the maximum water depth of 10 m., it will be clear that the number of calls will increase with a growing annual throughput. Since the vessels for LPG and alcohol are not limited in the current situation, it is assumed that the size of these vessels will increase due to economies of scale, although the condition of a maximum water depth of 10 m. still obtains.

The forecast of ECORYS mentions the amount of calls of the petrol distribution. A constant number of 580 of calls are made. This forecast however does not contain the amount of petrol used for distribution to the other islands. It is therefore assumed that an average of 60 % of the total import is distributed.
The mean call size of the vessels is adapted to this assumption.

The Water & Sewage corporation, which controls an open storage area on Arawak Cay, is planned to be gradually phased out.
This because a pipeline to New Providence or own desalination facilities prove to be economically more feasible. It is therefore not included in this forecast.

Liquid bulk cargo	Category	2005	2020	2028	2035
Petroleum products	'000 tons	563	694	762	817
import	ship calls	38	47	51	55
	call size tons	14816	14816	14816	14816
	ship size dwt	47500	47500	47500	47500
	ship length m.	185	185	185	185
distribution	'000 tons	338	416	457	490
	ship calls	580	580	580	580
	call size tons	580	720	800	850
	ship size dwt	1200	1200	1200	1200
	ship length m.	60	60	60	60
LPG	'000 tons	38	47	52	55
	call size tons	2.000	2.474	2.737	2.903
	ship size dwt	10.703	13.240	14.648	15.536
	ship length m.	127	136	141	144
Molasses and alcohol	'000 tons	48.0	67.2	77.8	87.0
	ship calls	24	24	24	24
	call size tons	2.000	2800	3240	3630
	ship size dwt	7.000	9800	11350	12700
	ship length m.	110	117	121	125

Table 5-10: Forecast of throughput liquid bulk cargo

6 Determination of dimensions terminals and water area

With the prospects of the new port determined in the former chapter, the next phase can be started. Before different lay-out alternatives can be generated, it is necessary to determine the amount of terminals, each with its own specific functions. The next step will be the determination of the quay length and the dimensions of the terminal land surface.

Some alternatives are situated directly at the coastline or are located in an even more seaward direction. Other alternatives will be based on an inland lay-out, requiring a network of waterways and turning basins. These waterworks are discussed in paragraph 5.4, where some general design recommendations are presented.

The berths and terminals are based on the forecast scenario with a medium annual growth rate. At the end of the study the effects of a different scenario of growth on the final design are investigated.

6.1 Functions of the terminals

To facilitate the different shipping companies in the new port it is required to divide the port in separate terminal areas each with its own distinctive functions. These terminals are the following:

Container terminals
At the container terminals all vessels holding containers and break bulk will be berthed. Each shipping company receives its own surface area at the terminal. In this way all terminals have to be placed in such an order that future takeovers are executable in an efficient way.

The berths in front of the container terminals are however not assigned to a single user. Application of single user berths would result in additional quay length. The intensity of usage for each container berth will also decrease. This will result in berthing facilities which are less efficient from an economical perspective.

Since the container berths require a ramp facility for RoRo transport, it is efficient to locate the berths for car carriers at this terminal.

Dry bulk terminal
By concentrating all dry bulk cargo at one terminal, a situation arises which is most efficient for several reasons.
The dry bulk vessels are the largest vessels accommodated in the new port; this fact requires that the depth of the bottom is rather large. Also the forces on the fender constructions are relatively large. A separate terminal for dry bulk is also effective for minimising the hindrance of noise and dust.
Summarised: the terminal will accommodate all aggregate and cement transport.

Petroleum bulk terminal
The petroleum berth is a berth that is solely used for the transhipment of petroleum products which are stored at the facilities of the Power Plant. This berth will need strict safety requirements. The berth solely handles the export of the petrol by smaller vessels, with a size within a range of 300 and 1000 dwt. The import of petrol is accomplished by an offshore terminal, the SBM, because of the local water depth.

LPG bulk terminal
The berth with the highest level of safety requirements is the LPG berth.
This berth needs specific equipment, like pumping installations.
In the case of an occurring disaster, the impact is relatively large.
Therefore the berths are usually located with a safety clearance from existing installations for reasons of safety precautions. These safety distances are presented in paragraph 7.1.2.

Alcohol and molasses terminal
The transhipment of alcohol and molasses occurs at the quay presented in figure 4-2. Since the client of this commodity is the Commonwealth Brewery, the berth is positioned in the surroundings of the Brewery.

Each terminal is evaluated for three different phases of development.
In this way the flexibility of the port structure is tested. The three phases are the referential years with the capacity of the years 2020, 2028 and 2035 as mentioned in table 5-1.

From economical perspective it is wise to design for different phases of the period of forecast. It is required to design the port in a flexible way, so in the case of a lack of growth or an unexpected increase of growth the design is easily adjustable. When the number of construction phases is too large, it will cause too much hindrance for the existing shipping traffic. Therefore a phase interval of eight years is chosen to mediate between flexibility and hindrance during construction.

In some cases terminals can be expected to turn out sufficient for all three design phases and will not require separate construction phases.
In this case it is decided to construct the terminal in a single construction phase. Also the start date of all phases of construction is an early assumption, because these durations are not constant in time.
This will be treated in further paragraphs.

6.2 Berths assessment

Before the individual terminals mentioned in paragraph 6.1 are discussed in this paragraph, some general design methods are discussed to solve a particular capacity problem.

6.2.1 Design methods

In between the phases of determining the forecast and the dimensioning of the port lay-out, is the phase of the determination of an optimal configuration for the port’s capacity.

The solution for this capacity problem can be accomplished with several design methods:

- rules of thumb
- queuing theory
- simulation models

The most simple way are the use of empirical rules of thumb, which are applied in capacity problems with a low traffic intensity and where it is possible to obtain a good insight into the prevailing conditions.

When the intensity of the interactions starts to play an important role, for example in case of increasing traffic, it is necessary to use a queuing theory to estimate the basic throughputs. Different distribution methods are available, like the Erlang or a negative exponential distribution. These distributions are in most of the cases used to determine the arrival and service time.

Simulation models present the next level of determining the port’s capacity with an increasing complexity. Models are used in cases when queuing models aren’t sufficient. For example a non-negligible sailing time in the harbour channel in relation to the service time or the case of tidal conditions affecting the system. In this study the capacity problem is solved by the queuing theory.

The queuing theory consists of a queuing system. Within this system several components are used, each with it’s own type of distribution. In the case of this port a queuing system is used with the following 3 components:

- Arrival of vessels
- Service time of the vessels in the port
- Service system (number of berths)

These components are noted in the Kendall notation. This particular notation consists of 3 letter code, where each letter represents a different component: the first letter describes the distribution of arrival times, the second the distribution of service times and the last describes the number of berths. The terminals of New Providence are presented by the code M/ E_k /n.

The M is assigned to the negative exponential distribution, presented in the formula of $M(t) = \mu \cdot e^{-\mu \cdot t}$, where 1/μ is the expected mean interval of arrival time.

The E_k distribution of the service time is formed by the formula of
$E_k(t) = \frac{(k \cdot \mu)^k \cdot t^{k-1} e^{-k \cdot \mu \cdot t}}{(k-1)!}$, where k is the Erlang parameter (k>0).

The Erlang parameter determines the deviation of the resulting distribution. If k=1 the distribution turns into the negative exponential distribution. For k→ ∞ the distribution results in a constant value (1/μ is constant).

The permitted total service time a year is determined by the net working day (21 hours) multiplied by the amount of working days a year (360 days). When the occupancy rate is near the recommended value further analysis is required. In that case an application of a queuing method is necessary.

The capacity of the transport system consists of several components, presented in figure 6-1. The system can be considered serial; the capacity of the system as a whole is determined by the smallest capacity.

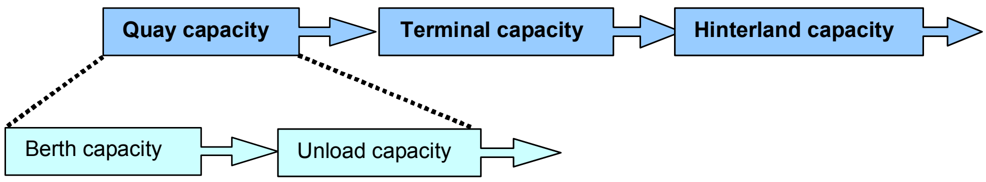


Figure 6-1: Schematisation of capacity of transport system

The quay capacity exists of two elements: the capacity of the berth and the capacity of the loading and unloading operations. The berth capacity is determined by the berth's occupancy. It is stated that when the occupancy rate reaches its limit, the capacity is at its maximum.
The capacity of the loading and unloading operations is based on the transport capacity at the location of the quay's apron.
This transport capacity is determined by the efficiency and number of personnel and available equipment. This concept of quay capacity will determine the number of berths in the following paragraphs.

The capacity of the terminal is determined by the available equipment and personnel at the terminal and the availability of the storage area. In this study the surface of each terminal will be determined by various factors. Each type of commodity is related to a different density of cargo storage.

The surface area required at the terminal for all functions will be determined individually. The space required by the transportation of the commodities is determined by a percentage of the net storage area. The equipment and personnel required to operate a terminal in an economically efficient way will not be part of this study. It is assumed that this aspect is non-normative in an economical perspective. The hinterland capacity is determined by the ability of transporting the cargo stored at the terminal to the point of destination. There are several options for the transportation of the commodities. These options include the use of trucks or a relatively short railway. It is assumed that transportation by trucks is best from an economic perspective. The design for an optimal transportation system of the island's infrastructure is behind the scope of this report.

6.2.2 Container berth

In paragraph 5.2 it can be read that it is assumed that the number of port calls of container vessels remains constant in the period till 2035. Since the throughput of TEU's will increase, this will result in an increasing of the container vessel's call size. The mean call size changes from 72 TEU present to 157 TEU (2020), 213 TEU (2028) and 243 TEU (2035).

The call size of the car carriers is assumed to remain constant. For this type of vessel the number of calls will increase. For servicing the car carriers at the berths the service capacity is 100 cars/hour. More information can be read in paragraph 5.3.

The mean service times of these vessels are presented in table 6-1. The idle service time comprehends the time when the vessel is berthing/deberthing. For both type of vessels the mean idle time is 1 hour.

The service capacity of container vessels is based on the assumption that one crane of 20 moves/hour services a container berth. With a TEU factor of 1.80 (80% of all containers are of a size of 40 ft.) there will be moved 36 TEU/hour. A part of 10 % of the cargo on the terminal exists of RoRo trailers; the service capacity of RoRo is 25 trailers/hour, where each trailer contains 2 TEU, resulting in a total of 50 TEU/hour. The overall service capacity of the container terminal is therefore 38 TEU/hour (0.9*36+0.1*50).

As can be seen in table 6-1 the gross service time is not constant for the container vessels, since the size of the vessel increases in time. The mean call size of the car carriers remains constant.

Service time (ST)	Service capacity	Nett ST (hour)	Idle ST (hour)	Gross ST (hour)
Container	38 TEU/hour	8,3-12,8	1	9,3-13,8
Car carrier	100 cars/hour	1,9	1	2,9
Break bulk	75 ton/hour	16,9	2	18,9

Table 6-1: Gross service time of container vessel and car carrier

The total service time (ST) for the whole terminal is determined in table 6-2.

	2020			2028			2038		
	Call size (ton)	calls	ST (hrs)	Call size (ton)	calls	ST (hrs)	Call size (ton)	calls	ST (hrs)
Container	157	905	8417	213	905	11041	243	905	12489
Break bulk	1270	100	1890	0	0	0	0	0	0
Car	191	68	265	191	83	324	191	95	371
Total		1073	10572		988	11365		1000	12860

Table 6-2: Total service time of container terminal

Currently the arrivals of larger container vessels are divided over several days of the week. Since the distribution over time in the future for all vessels is unknown, the distribution is considered random.
This is represented by a negative exponential distribution.
The handling of the cargo at the berth occurs at a less randomly rate and is chosen as a E_2 distribution.

The utilisation is determined by the following formula: $u = \frac{ST_{total}}{n \cdot t_{available}}$,

where n is the number of berths at the terminal.

In table 6-3, the waiting time as part of the total service time is presented as function of the utilisation and number of berths. These values are limited to a M/ E_2 /n distribution. The maximum value of the waiting time is set on 30 % of the total service time. Therefore starting from the year 2028 a triple berth configuration will be sufficient. A mean waiting time larger than 4.5 hours will be insufficient.

utilisation (u)	number of servers (n) 1	2	3	4	5	6	7	8
0,10	0,08	0,01	-	-	-	-	-	-
0,15	0,13	0,02	-	-	-	-	-	-
0,20	0,19	0,03	0,01	-	-	-	-	-
0,25	0,25	0,05	0,02	-	-	-	-	-
0,30	0,32	0,08	0,03	0,01	-	-	-	-
0,35	0,40	0,11	0,04	0,02	0,01	-	-	-
0,40	0,50	0,15	0,06	0,03	0,02	0,01	0,01	-
0,45	0,60	0,20	0,08	0,05	0,03	0,02	0,01	-
0,50	0,75	0,26	0,12	0,07	0,04	0,03	0,02	0,01
0,55	0,91	0,33	0,16	0,10	0,06	0,04	0,03	0,02
0,60	1,13	0,43	0,23	0,14	0,09	0,06	0,05	0,03
0,65	1,38	0,55	0,30	0,19	0,12	0,09	0,07	0,05
0,70	1,75	0,73	0,42	0,27	0,19	0,14	0,11	0,09
0,75	2,22	0,96	0,59	0,39	0,28	0,21	0,17	0,14
0,80	3,00	1,34	0,82	0,57	0,42	0,33	0,27	0,22
0,85	4,50	2,00	1,34	0,90	0,70	0,54	0,46	0,39
0,90	6,75	3,14	2,01	1,45	1,12	0,91	0,76	0,56

Table 6-3: Waiting time as function of service time for M/ E_2 /n distribution

YEAR	n	Utilisation	W	Waiting time mean (hour)	Ship length mean (m)	Quay length (m)
2020	~~2~~	~~0,699~~	~~0,73~~	~~7,93~~	~~86~~	~~204~~
""	3	0,466	0,09	0,98	86	314
2028	3	0,501	0,12	1,38	96	347
2035	3	0,567	0,18	2,31	106	380

Table 6-4: Required number of berths quay length, container terminal

The quay length is determined by the formula of:

$$L_{quay} = n \cdot 1.1 \cdot L_{vessel\_mean} + (n-1) \cdot 15$$

Based on the theory that each vessel requires 10 percent additional berthing space and the space between each vessel is 15 m.

6.2.3 Dry bulk terminal

All dry bulk will be unloaded at the dry bulk terminal.
The different commodities are mentioned in table 6-6, related to their service time. Since the call size of each cargo type is not constant, the totals of the service time differ in time. The sizes of the different type of vessels are presented in table 4-5.

With exception of the cement carriers of Clifton Point (Bahamas Cement), all ship sizes remain constant during the period from 2005 to 2035.
The call size of the Bahamas cement carriers increase in the following way: 1230 ton (2020), 1404 ton (2028) and 1581 ton (2035).

	2020			2028			2038			
	Call size (ton)	calls	ST (hrs)	Call size (ton)	calls	ST (hrs)	Call size (ton)	calls	ST (hrs)	Ship length (m)
Aggregates										
Granite	25000	6	148	25000	7	172	25000	8	193	245
Limestone	25000	32	756	25000	37	870	25000	42	973	187
Cement										
Bahamas	1230	52	426	1404	52	468	1581	52	515	160
MOSKO	5500	11	342	5500	13	391	5500	15	440	160
Sand	3000	152	649	3000	175	752	3000	196	837	60
Total		253	2321		284	2653		313	2958	

Table 6-5: Characteristics of forecasted dry bulk cargo vessels

Service time (ST)	Service capacity (ton/hour)	Nett ST (hour)	Idle ST (hour)	Bruto ST (hour)
Limestone carrier	1100	21,1-21,7	3	24-25
Granite carrier	1100	22,2-22,6	3	25-25,5
MOSKO Cement	200	6,2-7,9	2	8,2-9,9
Bahamas Cement	200	27,3-29,1	2	29-31
Sand barge	1100	2,7	2	4,7

Table 6-6: Gross service time of dry bulk vessels

In table 6-5 the total service time is determined. The next step is to determine the utilisation. The maximum value of the waiting time is set on 30 % of the total service time. The distributions of the queuing system are presented by a negative exponential (M) distribution for the arrival and an E_2 distribution for the service time at the berth.

It can be seen that during all phases of construction a single berth will not be sufficient. Therefore a double berth structure is chosen. However the double berth, which is determined by the mean length of the terminal's vessels, is shorter in length than a single berth. The reason is that the single berth is determined by the length of the maximum vessel. In this case the granite carrier with a length of 245 m. is normative and for all phases the quay length will be 285 m. The service time of the granite carrier as part of the total service time of the terminal is 6.5 %. The granite carrier can therefore be seen as the dominating factor of the quay length.

YEAR	n	Utilisation	W	Waiting time mean (hour)	Ship lenght mean (m)	Quay length multiple berth (m)	Quay length single berth (m)
2020	~~1~~	~~0,307~~	~~0,33~~	~~3,19~~	~~86~~	~~95~~	~~285~~
""	2	0,154	0,02	0,20	86	~~204~~	**285**
2028	2	0,176	0,02	0,20	106	~~248~~	**285**
2035	2	0,196	0,03	0,30	105	~~246~~	**285**

Table 6-7: Required number of berths quay length, dry bulk terminal

6.2.4 Liquid bulk terminals

The liquid bulk terminals exist of three berths. Since the transferred commodities each require specific loading equipment, each commodity remains its facility of a separated single berth. The service time for each type of commodity is presented in table 6-8.

Service time (ST)	Service capacity (ton/hour)	Nett ST (hour)	Idle ST (hour)	Bruto ST (hour)
Petrol				
Import	4000	3,7	2	5,7
Export	1000	1,2-1,4	1	2,2-2,4
LPG carrier	1300-1500	1,9	2	3,9
Alcohol and molasses transport	1000	2,5-2,9	1	3,5-3,9

Table 6-8: Gross service time of liquid bulk vessels

	2020			2028			2038			
	Call size (ton)	calls	ST (hrs)	Call size (ton)	calls	ST (hrs)	Call size (ton)	calls	ST (hrs)	Ship length (m)
Petrol										
Import	14816	47	268	14816	51	291	14816	55	314	185
Export	720	580	1276	800	580	1334	850	580	1392	60
		627	1544		631	1625		635	1706	
LPG	2474	19	74	2737	19	74	2903	19	74	177
Alcohol and molasses transport	2800	24	84	3240	24	89	3630	24	94	125

Table 6-9: Total service time of liquid bulk vessels

Petroleum berth

Current facilities are able to sustain the increasing import and distribution of petroleum products. From the perspective of the berth's capacity the current berth can remain on the same location. The expected call size and ship length mentioned in table 6-9 are for the tankers used for import. The sizes of these vessels vary enormously in relation to the vessels used for distribution. This can be seen in table 5-10.

YEAR	Throughput (ton/year)	Call size expected (ton)	Calls	Ship length (m)	Total service time (hour)	Utilisation
2020	694000	14816	627	177	1544	0,204
2028	762000	14816	631	177	1625	0,215
2035	817000	14816	635	177	1706	0,226

Table 6-10: Utilisation of petroleum berth

LPG berth

According to the forecast of the LPG tankers the total service time only shows a small increase during the forecast period. This can be seen in table 6-11; the increasing call size has no influence on the total service time.
This is caused by the increase of the discharging rate, which is expected to grow linear with the increase of the vessel size. The occupancy rate of the berth is very small and doesn't reach the maximum of 30 %.

YEAR	Throughput (ton/year)	Call size expected (ton)	Calls	Ship length (m)	Total service time (hour)	Utilisation
2020	47000	2474	19	177	74	0,010
2028	52000	2737	19	177	74	0,010
2035	55000	2903	19	177	74	0,010

Table 6-11: Utilisation of LPG berth

In the current situation the LPG berth is sufficient for vessels of 10.000 DWT. This berth has a length of 40 m. To facilitate the LPG tanker of 32.000 DWT mentioned in table 4-5, the bottom level of the current facility is insufficient.

Alcohol and molasses berth

The intensity of vessels transporting alcohol and molasses to Clifton Point is relatively low. The current facility is able to handle the commodity for future use.

YEAR	Throughput (ton/year)	Call size expected (ton)	Calls	Ship length (m)	Total service time (hour)	Utilisation
2020	59300	2471	24	125	84	0,011
2028	65000	2708	24	125	89	0,012
2035	69700	2903	24	125	94	0,012

Table 6-12: Utilisation of alcohol and molasses berth

6.2.5 Pilot and tugboat area

For assisting the vessels with entering the port pilots and tugboats are necessary. The expected length of pilot service vessels is 15 m. for the tugboats have a length of 30 m.
The area used for locating these vessels is assumed to be 2000 m^2.

6.3 Terminal area

In this paragraph the dimension of the surface area of the various terminals is determined. These areas are directly related to the throughputs determined in chapter 5. The design parameters for each terminal are based on current conditions and technologies.

6.3.1 Surface area container terminals

The surface area of the container terminal consist of several parts.
These can be distinguished into the following elements:

- Apron area
- Container stacking area
- Container transfer lanes
- Buildings (container freight station, office, gates, etc.)

The major part of the area is the container stacking area, which surface is calculated in table 6-13. Each type of container has its own stack density. The transferring lanes for container transport are included as part of the stacking density.

All containers currently shipped in the port of Nassau are shipped at the new port accommodation. Containers which are shipped by other ways or which life cycle is ended on the island can be neglected.
Since New Providence has no exporting sector all exported containers are considered empty; a share of 50 % of the annual container moves.

For further determination of the surface areas a distinction between regular containers (89%), RoRo (10%) and reefers (1%) is made.
For the determination of the required surface area the empty regular containers are separated. The stack density of these empties is higher in relation to the imported containers. Because of the lower container weight the containers can be stored in higher stacks.

For the RoRo containers and reefers no distinction is made between a full nd empty container in relation to stack density. They are therefore not separately shown in table 6-13.

In the current configuration the average dwell time of the container cargo is 7 days. This relatively low dwell time can be explained by the just-in-time character of the cargo. Most container loads are required on a short time span for the island’s tourist industry. A short dwell time is a positive aspect of a port, since it limits the required land surface with the associated high costs.

Item	Unit	Year		
		2020	**2028**	**2035**
Containers				
Import		142000	193000	220000
Export		142000	193000	220000
Annual moves		284000	386000	440000
Regular containers (44,5%)		126380	171770	195800
Empty containers (44,5%)		126380	171770	195800
RoRo containers (10%)		28400	38600	44000
Reefer boxes (1%)		2840	3860	4400
Average dwell time	days	7	7	7
Required slots				
Regular containers		2424	3294	3755
Empty containers		2424	3294	3755
RoRo		78	106	121
Reefers		54	74	84
Stack density				
Regular containers	TEU/ha	700	700	700
Empty containers	TEU/ha	1000	1000	1000
Reefers	TEU/ha	300	300	300
Surface area				
Regular containers	ha	3,5	4,7	5,4
Empty containers	ha	2,4	3,3	3,8
RoRo	ha	0,6	0,8	0,9
Reefers	ha	0,2	0,2	0,3
Hazardous	ha	0,2	0,3	0,4
+ outsized cargo				
Total area	ha	6,8	9,3	10,7

Table 6-13: Surface area of container stacking area

The apron area is located directly to the quay wall. This area is used for transferring the commodities; cranes pick up or drop off the containers. The dimensions are based on the different traffic lanes. Next to track spacing of the cranes, traffic lanes are required for transporting the cargo to the storage areas. This transport can be accomplished by several alternatives, like straddle carriers, a multi trailer system (MTS) or automated guided vehicles (AGV). The selection of terminal equipment is done in a further design phase. This is behind the scope of this report.

Another possibility is to link up the apron area directly with the hinterland by a landward rail connection. This is behind the scope of this report. The apron area also provides access to the ships for the crew and for supplies and services.

Some containers require a container freight station (CFS), where containers are ‘stripped’ to divide the cargo into portions for different destinations. It is assumed that 10 percent of the imported containers is stripped at the freight station.

The office building is planned as a two storey building. The building acts as a central office for the port and will contain all administrative services for the port and the operators.

The total surface area of the container terminal is calculated in table 6-14.

Item	Year					
	2020		2028		2035	
Apron area	600 x 50	30000	600 x 50	30000	600 x 50	30000
Container stacking area	200 x 300	68000	250 x 380	93000	280 x 420	107000
CFS		7800		10600		12100
Loading area CFS	(50 % of CFS)	3900	(50 % of CFS)	5300	(50 % of CFS)	6050
Entrance gate	10 x 20	200	10 x 20	200	10 x 20	200
Weigh bridge	30 x 10	300	30 x 10	300	30 x 10	300
Maintenance	50 x 70	3500	50 x 70	3500	50 x 70	3500
General office buildings	20 x 25	500	20 x 25	500	20 x 25	500
Warehouses	2 x (60 x 100)	12000	2 x (60 x 100)	12000	2 x (60 x 100)	12000
Car parking: car carrier	1,2x 360x 20 m^2	8700	1,2x 440x 20 m^2	10600	1,2x 500x 20 m^2	12000
Parking trucks	29 x (2 x 36)	2100	39 x (2 x 36)	2800	44 x (2 x 36)	3200
Total container terminal		137.000 m^2 13,7 ha		168.800 m^2 16,9 ha		186.850 m^2 18,6 ha

Table 6-14: Total surface area of container terminal

6.3.2 Surface area dry bulk terminal

Aggregates and sand

The surface area which is reserved for the dry bulk cargo has merely the purpose of storing the bulk. For each type of bulk commodity the surface area is determined using the following method:

it is assumed that the all ship calls are divided according to a normal distribution. The average arrival interval (t_a) is then: $t_a = \dfrac{365days}{\#calls}$.
When t_a> 20 days a surface area providing storage for one shipload will be sufficient. In the case t_a< 20 days, the surface area will provide storage for a commodity's throughput of 20 days, representing a dwell time of 20 days. As a result the only type of cargo with a terminal capacity of a single shipload are the granite aggregates.

The amount of bulk to be stored is then translated from tons to a certain volume. The next step is to determine the optimal configuration for the piles of dry bulk. For all types of dry bulk the storage areas are designed with a retaining wall with a height of 2 m. This increases the storage capacity of the terminal area. Every type of dry bulk is stacked as a strip with a maximum length. The terminal configuration and the cross section of a stack are schematized in figure 6-2.

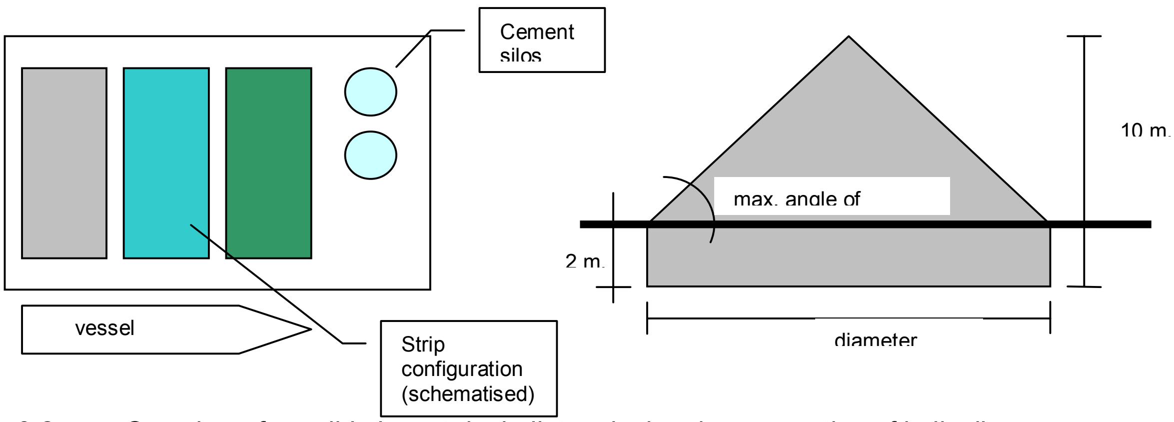


Figure 6-2: Overview of possible layout dry bulk terminal and cross section of bulk pile

For each type of dry bulk the angle of repose is different. This angle depends on the type of material. The maximum angle of repose is presented in table 6-15.

	Height max (m)	Angle of repose	Height wall (m)	Width (m)	Total storage pile (m^3/m)
Limestone	10	40	2	19,1	114
Granite	10	40	2	19,1	114
Sand	10	30	2	27,7	166

Table 6-15: Dimensions of bulk pile for maximum stored cargo

The surface area for each dry bulk commodity is determined in the tables 6-16 to 6-18.

Limestone

Item	Unit	Year		
		2020	**2028**	**2035**
Annual throughput limestone	tons	805000	932000	1041000
Calls		29	36	42
Dwell time	days	20	20	20
Density	ton/m^3	1,1	1,1	1,1
Volume	m^3	40100	46426	51856
Angle of repose	°	40	40	40
Width silo	m	19,1	19,1	19,1
Storage volume per m length	m^3/m	114,4	114,4	114,4
Length of storage stroke	m	350	406	453
Nett surface	ha	0,67	0,77	0,86
Total surface area	ha	0,80	0,93	1,04

Table 6-16: Surface area of dry bulk terminal for storage of limestone aggregate

Granite

Item	Unit	Year		
		2020	**2028**	**2035**
Annual throughput granite	tons	158000	182000	204000
Calls		6	7	8
Dwell time	days	40	40	40
Density	ton/m^3	1,6	1,6	1,6
Volume	m^3	15625	15625	15625
Angle of repose	°	40	40	40
Width silo	m	19,1	19,1	19,1
Storage volume per m length	m^3/m	114,4	114,4	114,4
Length of storage stroke	m	137	137	137
Nett surface	ha	0,26	0,26	0,26
Total surface area	ha	0,31	0,31	0,31

Table 6-17: Surface area of dry bulk terminal for storage of granite aggregate

Sand

Item	Unit	Year		
		2020	2028	2035
Annual throughput sand	tons	455000	526000	589000
Calls		137	167	196
Dwell time	days	20	20	20
Density	ton/m^3	1,65	1,65	1,65
Volume	m^3	15110	17468	19560
Angle of repose	°	30	30	30
Width silo	m	27,7	27,7	27,7
Storage volume per m length	m^3/m	166,3	166,3	166,3
Length of storage stroke	m	91	105	118
Nett surface	ha	0,25	0,29	0,33
Total surface area	ha	0,30	0,35	0,39

Table 6-18: Surface area of dry bulk terminal for sand storage

Cement

At the port location of Southwest Providence the two cement companies MOSKO Cement and Bahamas Cement are present.
Each company has its own silo and transfer equipment. Since the throughput of both companies is equal, both are equipped with a 10.000 ton silo.
The size of the silo is not influenced by the different dwell times of the companies. It is decided to invest in one size of silo for the minimum time period of 35 years.

To generate a sufficient terminal area a part of the terminal is used for transferring the commodities. For this area a multiplication factor of 1.2 is used. The total surface area of the total dry bulk terminal consists of the elements presented in table 6-19.

	Year		
Surface area	**2020**	**2028**	**2035**
Aggregate storage (ha)			
Limestone	0,80	0,93	1,04
Granite	0,31	0,31	0,31
Total	1,11	1,24	1,35
Sand storage (ha)	0,30	0,35	0,39
Cement silos (ha)			
MOSKO cement	0,40	0,40	0,40
Bahamas cement	0,40	0,40	0,40
Total	0,80	0,80	0,80
Nett storage area (ha)	3,08	3,25	3,40
Transferring lanes, 10 % of storage	0,22	0,24	0,25
Apron area (width 30 m.)	0,86	0,86	0,86
Total surface dry bulk terminal (ha)	3,30	3,49	3,65

Table 6-19: Total surface area of dry bulk terminal

6.3.3 Surface area of liquid bulk terminals

<u>LPG terminal</u>

The current capacity of the storage tanks of LPG is determined on 2700 ton. Since the yearly throughput increases, the capacity is also required to grow. For the determination of the costs and the required surface area, it is assumed that all additional storage tanks provide a capacity of 200.000 litres.

As mentioned before the appointed surface area for the LPG storage is 2.9 ha. In the current situation 1.6 ha of the surface area is covered by tanks.

Each tank uses a surface area of roughly 500 m^2. For safety distances and transport activities a surface factor of 1.2 is chosen to determine the total surface area.

Figure 6-3: Detailed view of LPG storage area at current Power Plant

With the assumed number of 19 calls a year, the mean arrival time is smaller than 20 days. A minimum storage capacity of 30 days is required. The total capacity and the accompanying surface area is presented in table 6-20.

	Units	2005	2020	2028	2035
Throughput	ton/year	38.000	47.000	52.000	55.000
Capacity required	ton	3123	3863	4274	4521
Number of tanks	-	41	48	52	55
Nett surface area	ha	2,1	2,4	2,6	2,8
Gross surface area	ha	2,5	2,9	3,1	3,3

Table 6-20: Surface area of LPG storage

The current 2.9 ha of surface area will be sufficient for locating LPG tanks with a capacity of the year 2020. From that point of time the storage tank area has to expand.

Petroleum terminal

The current capacity of the storage facilities for the use of petroleum products is unknown. It is assumed that the current facilities are able to process the throughput of 2005, namely 563.000 tons. The gross area currently used by the storage tanks is approximately 10.3 ha. The area available to the Power Plant, excluding the LPG and power plant facilities, is 18.5 ha.

The throughput of petrol will increase in 2035 in relation to 2005 with 45 %. In the case this is correlated to the growth of required land, the utilized land surface will be 14.9 ha. It is therefore assumed that no additional expansion is required.

In case expansion is still desirable the land north of the current Power Plant is still available. It can be stated that the growth of the throughput of petroleum will not cause problems for the petroleum terminal.

Alcohol and molasses terminal

About the current use of the alcohol and molasses berth little information is available. It is assumed that cargo delivered at the berth is directly transported to the Commonwealth Brewery by the road connection. The land belonging to the brewery is expected to be able to store the relatively small throughput of alcohol and molasses.

6.3.4 General facilities

Some facilities of the new port structure have no fixed location. They require a centred position in relation to the terminals and can either be located next to the container or dry bulk terminal. The surface area is reserved for the following general facilities:

Building port authority

The Port authority of Clifton Point requires a building.
It is most likely that the building is several stories high.

Custom office building

To control the import and export through the port customs are required, which will need a dedicated building.

Fire Fighting Station

The fire Fighting Station shall provide housing for all fire fighting equipment and personnel. The station shall also house first-aid medical facilities.

Item	Year					
	2020		2028		2035	
Office building Port Authority	30 x 50	1500	30 x 50	1500	30 x 50	1500
Custom office building	20 x 20	400	20 x 20	400	20 x 20	400
Parking 50 cars	50 x (10 x 4)	2000	50 x (10 x 4)	2000	50 x (10 x 4)	2000
Fire fighting station	20 x 25	500	20 x 25	500	20 x 25	500
Total area general facilities		4400 m^2 4,4 ha		4400 m^2 4,4 ha		4400 m^2 4,4 ha

Table 6-21: Total surface area of general facilities

Besides the buildings mentioned above the new port also requires some general utilities and services. The required utilities are electricity, drinking water system and drainage system. Also waste collection utilities are required.

6.4 Water area

The water area within the port is also bound to various design conditions.

Each berth requires a sufficient water depth. Also the wave climate in front of a berth is limited caused by several operational conditions. These berthing conditions are reported in this paragraph.

To provide an efficient and safe port area from navigational perspective several features may be required, like a navigation channel and/or turning basin. This paragraph results in a guideline used for the optional usage of these components.

6.4.1 Maximum significant wave heights at berth

The primary function of a port is providing the vessels a safe location for unloading and loading their cargo. For this purpose relatively calm mooring conditions area required.

Table 6-22 can be accepted as a general guideline for the maximum significant wave height for berthed vessels. It is clear that waves with the direction to the ship's beam show a smaller maximum H_s.
This can be explained by the fact that movements perpendicular to the berth are less favoured than movements parallel to the berth, due to the inertial momentum of the cross section.

The wave heights presented in table 6-22 are significant wave heights in front of the quay with a wave period in the range of about 7 to 12 s.
The maximum acceptable wave height always has to be checked in respect to the ship's movement at the berth.

Type of ship	Limiting wave height H_s (in m.)	
	0^o (head or stern)	$45^o – 90^o$
General cargo	1.0	0.8
Container, RoRo ship	0.5	
Dry bulk 30.000 – 100.000 DWT Loading	1.5	1.0
Unloading	1.0	0.8 – 1.0
Tankers		
30.000 DWT	1.5	
30.000 – 200.000 DWT	1.5 – 2.5	1.0 – 1.2
> 200.000 DWT	2.5 – 3.0	1.0 – 1.5

Table 6-22: Mooring limiting wave heights

6.4.2 Minimum bottom level in port

Within a commercial port grounded vessels are an unacceptable feature.
A grounded vessel causes unnecessary downtime for the port. Also the vessel itself can receive possible damage.
To prevent the vessel from grounding a maximum bottom level is required.
The maximum bottom level is determined by several components.

<u>Bottom level at berth</u>

For the bottom level at the berths these components are schematised in figure 6-4 and are described in the following text.

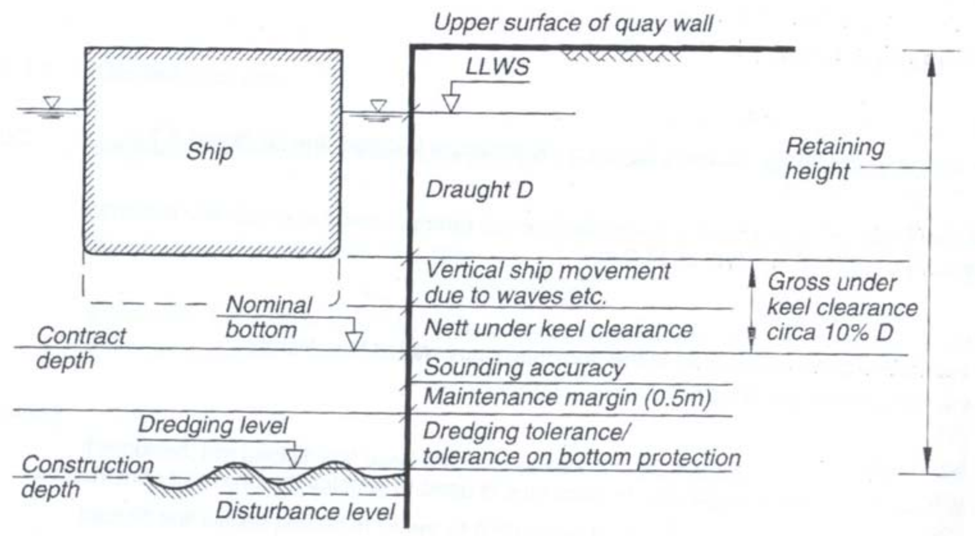


Figure 6-4: Determination of contract and construction depth

The maximum draft of the vessels entering the new port of Southwest New Providence is calculated with the following sum.

Minimum construction level is the total of:

- Vessel's draught
- Under keel clearance
- Dredging parameter
- Sounding accuracy

Vessel's draught
The minimum construction level of both terminals is determined as function of the maximum draft of the expected vessels. The draft of a vessel is determined by its call size as function of its displacement tonnage:

$$d_s = \frac{callsize + LWT}{DT} D$$

where:	D = fully loaded draft	(m)
	DT = displacement tonnage	(ton)
	LWT = Lightship weight (LWT= DT-DWT)	(ton)

The displacement tonnage is determined by:

$$DT = C_B \cdot \rho \cdot LBP \cdot B \cdot D$$

where:	C_B = block coefficient	(-)
	LBP = Length between Perpendiculars	(m)
	B = beam	(m)
	ρ = density of water (=1025 kg/m^3 ≈ 1 ton/m^3)	

For large vessels like the dry bulk carriers the block coefficient is approximated to be 0,9. The LBP of the vessel is expected to be LOA - 5.0 m. The beam of the vessel is expected to be 1/6 of the vessel's LOA.

Under keel clearance
For safety measures the nominal bottom level of the vessel is determined. This is done by adding an under keel clearance of 10 % of the vessel's draught. The gross under keel clearance has a maximum of 1.00 m.

Dredging parameter
The difference between the construction depth and contract depth exists of a dredging parameter, which is composed of a maintenance margin and a minimum dredging tolerance. The maintenance margin and the minimum tolerance are set on respectively 0.50 m. and 0.30 m. In this way the dredging parameter is 0.80 m. in total.

Sounding accuracy
The construction depth is determined as part of the new design. The accuracy for the soundings is only required for existing berths.

The displacement tonnage of both aggregate carriers is:

Type of vessel	LBP (m)	Beam (m)	Draft (m)	DT (ton)
Granite carrier	240	40	13.50	117
Limestone carrier	182	31	11.40	58

Table 6-23: Displacement tonnage of aggregate carriers

For the draft at the container terminal the maximum draft of the car carrier is chosen. The draft of both aggregate carriers is defined by using the call size of paragraph 6.2.3.
When the shipping companies demand a larger call size of the dry bulk carriers additional dredging is required. A decrease in calls reduces time spent in port. The possible benefits can be researched by an economical analysis.

The construction level following from the relevant drafts are presented in table 6-24. It is decided that the minimum construction level op the entire port is -10.00 m. MSL.

Berth	LLWS (m)	Expected draft vessel (m)	Under keel clearance (m)	Dredging parameter (m)	Construction level (m)
Container terminal					
Car carrier	-0.50 MSL	7.60	0.80	0.80	- 9.70 MSL
Dry bulk terminal					
Granite carrier	-0.50 MSL	7.90	0.80	0.80	- 10.00 MSL
Limestone carrier	-0.50 MSL	7.70	0.80	0.80	- 9.80 MSL

Table 6-24: Mooring limiting wave heights

Bottom level of waterways

The bottom level of the port's waterways require an additional increase of water depth. Because the waterways, like the port's entrance and navigation channel are expected to be subject of a higher wave climate, the bottom level is determined by:

$\frac{h}{d} \geq 1.5$ for waves of H_s > 1.00 m,

where:	h = water depth	(m)
	d = draft of vessel	(m)

With a maximum draft of 8.00 m. the bottom level of the port's waterways results in a bottom level of -12.50 m. MSL (water depth and LLWS).

6.4.3 Turning basin

The primary function of the port's turning basin is to allow the vessels to manoeuvre themselves in an optimal position before the operation of mooring. Therefore a certain amount of open space is required. The size of the turning basin will be a function of the manoeuvrability and the length of the vessel. It also depends on the time permitted for execution of the turning manoeuvre.

The minimal diameters of the basin are based on the following conditions, presented in table 6-25. In practice it can be noted that relatively large vessels do not enter the port without tugboat assistance. In most cases the turning basin is designed with a radius of two times the maximal vessel length.

Condition			Number of ship lengths (L_s)
Use of bow thrusters	Tugboat assistance	Good weather	
			4
		•	3
	•		2
	•	•	1.6
•			1.5
•	•	•	1.2

Table 6-25: Minimal radius turning basin as function of maximum L_s

The conditions of Clifton Point are considered as good weather. The wave climate outside the port doesn't exceed a H_s of 1.50 m. for 94 % of the time. The wind climate can also be considered calm, wind speeds above 10 m/s only occur 5 % of the time.

Therefore the diameter of the turning basin is based on 1.6 L_s.
The maximum length corresponds with the car carrier (L_s= 160 m.), resulting in a turning diameter of 260 m. When the dry bulk terminal requires a turning basin, the diameter is 390 m. (L_s= 245 m.)

It is determined that the large dry bulk carriers arriving at the port are forced to use tugboats to enter the port's basin. Because of the relatively calm weather conditions, the carriers are able to enter the port for most of the time.
As part of the final design the manoeuvrability of the bulk carriers will be checked for the selected configuration of breakwaters.

6.4.4 Navigation channel

The use of a navigation channel within the design is an option.
Only by designing an inland port a navigation channel is applied,
s can be seen in the following chapter. Nevertheless an entrance of the port is always necessary. The width of the entrance is determined with the same method of the navigation channel.

In the case of some alternatives an inland navigation channel will be required to transfer the vessels to their berth of destination.
This channel can be designed as a single or two lane channel.
The channel length is limited, since no a relatively short distance is required to travel inland. The length is assumed to be 500 m.

Therefore a vessel navigating the channel with an average speed of 8 kn takes 2 minutes to travel the channel. With an expected annually total of 1970 calls in the year 2035, included liquid bulk transport, the arrival time of a single vessel can be estimated at less than 1 per hour.
The navigation time of the channel in relation to the estimated arrival time shows that a two lane channel is not required. Because of the short distance no curves are expected within the channel.

The width of a single lane channel is determined with the following formula:

$$W_{channel}= W_{BM} + \sum W_i + 2W_B$$

where : W_{BM} = basic width = 1,6B, since the h/d = 1.4
$\sum W_i$ = additional widths = 1.5B
(0.2B for prevailing cross current, 1.0B for prevailing wave height, 0.2B for hard seabed, 0.1B for good aids of navigation)
W_B = bank clearance = 1.0B, for a steep and hard embankment

Component	Criterion	Width in B_S (width of vessel)
W_{BM} = basic width	1.25 D < d < 1.5 D	$W_{BM} = 1.6\ B_S$
	d ≤ 1.25 D	$W_{BM} = 1.7\ B_S$
W_i = additional width		
Transversal winds	15 – 33 kn	$W_i = 0.4\ B_S$
	33 – 48 kn	$W_i = 0.8\ B_S$
Transversal current	0.2 – 0.5 kn	$W_i = 0.2\ B_S$
	0.5 – 1.5 kn	$W_i = 0.7\ B_S$
	1.5 – 2.0 kn	$W_i = 1.0\ B_S$
Parallel current	1.5 – 3.0 kn	$W_i = 0.1\ B_S$
	> 3.0 kn	$W_i = 0.2\ B_S$
Wave height	1 – 3 m.	$W_i = 1.0\ B_S$
	> 3 m.	$W_i = 2.2\ B_S$
Navigational aids	VTS	$W_i = 0.0\ B_S$
		$W_i = 0.1\ B_S$
Bottom characteristics	Soft	$W_i = 0.1\ B_S$
	Hard	$W_i = 0.2\ B_S$
Risk of cargo	Medium	$W_i = 0.5\ B_S$
	High	$W_i = 1.0\ B_S$
W_B = bank clearance	Faint slope	$W_B = 0.5\ B_S$
	Steep slope, hard embankment	$W_B = 1.0\ B_S$

Table 6-26: Width factors to determine minimal width of navigation channel

The minimum channel width is therefore 4.1B, where B is the maximum width of a vessel entering the navigation channel. This results in a channel width of 100 m. ($B_{car\ carrier}$ = 24 m). As mentioned before the minimum width of the port's entrance is equal to the channel width of 100 m.

To optimize the usage of land, it is an option to locate a berth next to the navigation channel. In case of a dry bulk berth located in the channel, the navigation channel has to be designed on the width of the dry bulk carrier, also an additional width of 1.2 B_b is expected to be sufficient for berthing an additional vessel. The total width then results in 5.3B, which is 160 m.

6.4.5 Anchorage

When no berths are available, waiting areas (also called anchorages) have to be provided. Since the amount of space within the port boundaries is limited, due to the steep shore, the vessels therefore have to wait outside the port.

Outside sufficient space is available. The surface required for anchoring is a function of the ship length and water depth at the location.
The area indicated as anchorage has to take into account the environmental conditions of the sea bottom. The damage to coral reefs etc. in the vicinity of the port has to be limited.

7 Alternatives of future port lay-outs

With the requirements of the new port determined in preceding chapters the next phase is the design of several alternatives for the final layout. Before the alternatives are generated, all requirements are recapitulated.

7.1 Design requirements

The determination of the required quay lengths and surface areas from chapter 6 results in the following design requirements.
The requirements can be divided in the following design aspects.

- Dimensions of terminals and quays
- Safety distances for LPG and petrol berth

7.1.1 Dimensions of terminals and quays

From the previous chapters the following dimensions are accumulated for the new design. For the design of the alternatives some additional choices are made. The CFS as part of the container terminal is positioned independently from the rest of the container area. Because a high rate of distribution activities is expected at the CFS, it requires a location at the edge of the terminal. To allow future expansions of the terminal area it is highly possible that in the initial phase the CFS is not located directly next to the terminal.

	2020		2028		2035	
	L_{quay} (m)	$A_{terminal}$ (ha)	L_{quay} (m)	$A_{terminal}$ (ha)	L_{quay} (m)	$A_{terminal}$ (ha)
Container terminal						
Container area		12.5		15.3		16.8
CFS		1.2		1.6		1.8
General facilities		0.4		0.4		0.4
	314	14.1	347	17.3	380	19.0
Dry bulk terminal	285	3.3	285	3.5	285	3.7
Petroleum bulk terminal		-		-		-
LPG terminal		2.9		3.1		3.3

Table 7-1: Design requirements of new port, Clifton Point

The general facilities of which the surface area is relatively small are considered part of the container terminal. This to simplify the design procedure. For detailed design of all terminal's surface area it is possible that the general facilities are divided over the total port area.

LPG currently provides a storage area of 2.9 ha, which is not fully utilized. Therefore the surface area currently used will firstly be optimized when possible before it is expanded.

The capacity of the storage area required for petroleum products in the current situation is an unknown factor. For this reason the surface area required for extension of the amount of storage can't be determined. It is therefore assumed a maximum storage area of 10 % of the current storage area remains available for expansion. This possible expansion will be located at the borders of the current Power Plant installation.

7.1.2 Safety conditions LPG and petrol berth

Both the LPG berth and petroleum berth are a substantial part of the new port of Southwest New Providence. Since the new port structure will be positioned between the Power Plant and the Commonwealth brewery it is most likely that parts of the LPG and petroleum berth accommodations need to be relocated.

When transporting LPG and petrol there are 4 possible failure components in the vicinity of a LPG berth. All failure components are part of the commodity's supply chain of transport from vessel to land based storage tank. They are presented in figure 7-1.

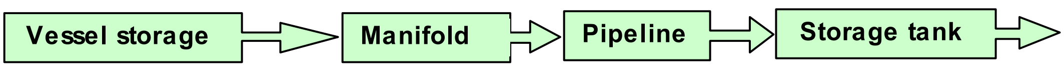


Figure 7-1: Failure components of LPG supply chain

Failure of the vessel's storage tank has large consequences, because it results in a large quantity of escaping gas. Therefore the chance of failure, mostly caused by collision of grounding of the vessel, need to be minimized. Therefore safety distances are introduced, presented in table 7-2.

The most vulnerable part of the transport system is the manifold. This is caused by the fact that the procedure is performed during dynamic interaction of the incoming waves. The interaction occurs between the vessel and the shore based pipeline. Because of this dynamic behaviour the joint structure of transport has to be designed elastically.

Because of the vulnerability of the transportation from vessel to shore a lot of safety measures are available, like non-returning valves and auto-disconnects etc. In case of a calamity the escaping amount of gas is small compared to other failures, like for example a rupture of a vessel's storage tank.

The pipeline connection between the manifold and the onshore storage tanks is also relatively safe. In case of a pipeline rupture the amount of escaping gas is not enough to cause a calamity on a large scale. Since the length of the pipeline increases the risk of failure, precautions are required like non returning valves to decrease the total risk.

In respect to the other components storage tanks are relatively safe. In the case of the new port a storage tank is most likely to be land based.

In this study the following requirements in relation to the safety can be noted.

The pipeline, manifold and storage tanks each use their own standard regulations of design. These are not recited and are beyond the scope of this stage of design. There are however certain assumptions required:

- The pipeline will be connected in the most direct path to the storage tanks. However the length of the pipeline underneath the existing Power Plant facility is limited. The pipeline would otherwise receive problems of accessibility; the pipeline has to remain open for required inspections etc.

Since LNG accommodations are not required, the distances in table 7-2 are of a mere comparative nature.

	Distance between two moored vessels	Distance of passing ship to moored tanker	Distance of installations to berth
Oil tankers	100 m	100 m	unknown
Gas tankers			
LPG	100 m	200 m	150 m
LNG	300 m	300 m	300 m

Table 7-2: Safety distances in port for vessels of petroleum and gas transport

7.2 Generation of the alternatives new port

By using the design requirements from paragraph 6.4 and 7.1 numerous alternatives for the new port design can be generated.

Each alternative is designed according to the end situation, the construction phase of 2035. From this final design the designs for the two earlier phases are extracted. In this way the flexibility of a design alternative will be optimal. For all alternatives only the construction phase of 2020 and 2035 are presented in this chapter. Unless the construction phase of 2028 is problematic for the design.

A general design choice is to construct the total quay length of 380 m. for the container terminal during the first construction phase. The planned additional length of 66 m. will be included at the initial phase of the project. This decision is made in favour of minimizing the hindrance for the shipping. Extending the quay during operational activities of the berth causes disturbance for the established shipping operations. his can result in an intolerable increase of the port’s downtime.

It can therefore be concluded that the berths belonging to the container and dry bulk terminal are constructed during the initial phase of the project. The only aspect of the port structure which is variable during the phases of construction is the surface area of all terminals.

This variability results in the design aspect that required facilities, like warehouses and offices, are constructed at a location which are at a distance from the storage areas already present. This leads to an inefficient usage of the surface area.

Positioning the new port in the project area as mentioned in figure 1-3 leaves a little amount of options. Eventually the surface located in between the Clifton Heritage park and the Power Plant offers an possibility. This site however clearly has disadvantages in relation to the other location. The first problem is the steep shore, a bottom level of -10.00 m. MSL is already reached within a distance of 50 m. Also the location directly next to the Heritage park is expected to cause environmental problems.

The second option is the area in between the Power Plant and Commonwealth Brewery. Although the entrance to the hinterland is smaller and the LPG storage tanks are a possible obstacle, the water in front of the local shore is workable.

In all alternatives the location of the petroleum and LPG berth remains intact. Since the available space within the port is limited both berths are excluded from the new port structure. However due to the limited space some transitions are required. The LPG berth is transited for a relatively small distance in western direction. In some alternatives the LPG storage area is relocated to provide additional space. The total relocation of these existing berths is presented in paragraph 7.3.

Important considerations for the generation of alternatives are the following:

- Inland port vs. port in seaward direction
- Optimal usage of basin vs. manoeuvrability within port
- Orientation and layout of breakwater
- Relocation of LPG storage facilities

Inland port vs. port in seaward direction

A port structure is considered to be an inland port when the land behind the current shoreline is digged down to form a basin and berth structure. The closed character of the port results in a protective barrier against wave energy entering the port. The port will require less protection from breakwaters and other structures in relation to a seaward port structure. Since the soil of the project site is limestone the costs for digging down the soil is relatively high.

A seaward port structure will extend in seaward direction. For these alternatives land will be reclaimed to provide space for port facilities. A land reclaimed will also require a suitable protection. For the protection breakwaters are implemented in the design. These breakwaters are expected to be a major factor of the total construction costs.

Because the shore in front of Clifton Point shows a relative steep profile, the radius of the seaward extension of the island is limited. It can be stated that the larger the water depth, the larger the volume required for the land reclamation will be. Within a radius of 500 m. of the proposed project site, the bottom level reaches a depth of -50.00 m. MSL.

The port’s basin can however be affected to seiches. They belong to the subdivision of the long waves and have a wave period in the order of 10 to 20 minutes. Seiches are resonances in a body of water which is disturbed by one or more factors. These factors are mostly meteorological factors like swell, but local seismic activity or tsunamis can also be the cause. The period of the seiche is fully determined by the shape of the port’s basin. In the case that alternative A1 or A2 turns out to be the best option, a further research of the chosen bathymetry is required.
An method to estimate the influence of swell is introduced in Appendix D.

Optimal usage of basin vs. manoeuvrability within port

The larger the water area within a port structure the more comfortable the port will be to navigate. To construct a port including a large turning basin and berths which are optimal to access from the waterside the construction costs will increase. An inland port for instance where all berths are located around the turning basin requires a relatively large area to be dug out.

To increase the optimal use of the available water area in the port an option is to include finger piers to the design. This option is chosen for alternative A2 and B2. A finger pier also causes the terminal area to be located more directly to the terminal berths.

Orientation and layout of breakwater

The breakwaters will provide shelter for the port basin from excessive wave energy. To create an optimal layout some recommendations are required.

The orientation of the port entrance is directed out of line to the dominant direction of waves and currents. The site conditions in chapter 3 show that the majority of these aspects arrive from an eastern to south eastern direction (90° to 145°). By placing the port entrance in the sector of 220° to 270° the influence of waves and currents is minimized.

Since the littoral transport of sediment is relatively small, the influence of sediment transport can be neglected. Sedimentation within the port is not expected to be causing problems.

Relocation of LPG storage facilities

The land available for port operations is limited at the location of Clifton Point. Relocation of the storage facilities of LPG would therefore improve the accessibility of the terminal space. The costs for relocating the tanks are expected to be relatively high. Therefore an aspect of design is the option to let the tanks remain stationary, partially or fully.

Relocating the tanks includes the following operations:

Creating temporary storage
The storing of LPG should not be interrupted for a long period of time.

Relocation of pipeline to storage tanks
By relocating the LPG storage, the connecting pipeline will also need to be adapted. Because the space in direction of the LPG berth is insufficient, the length of the pipeline needs to be increased. For construction of new facilities the pressurised pipeline is bound to a maximum radius. Beyond the radius additional pumping stations are required. It is assumed that by locating the new facility at the border of the current facilities of the Power Plant, no additional pumping stations are necessary.

7.2.1 Alternative A1

Alternative A1 is designed as a port with an inland basin.

Disadvantages to the port design are the large basin which has to be cut in the coral coast. Since the soil exists of limestone, the costs for dredging are relatively high. Another disadvantage is the fact that not all sides of the basin have to function as a quay wall. A quay length of 665 m. is required, while the available length is 1500 m. Using the full potential of quay length is also not possible because of the unavailable hinterland at the eastern side of the port's basin. This side however has the possibility of locating smaller vessels, like tugboats etc.

To minimize the inland turning basin the dry bulk terminal is located at the western side of the navigation channel. As mentioned in paragraph 6.4.4 the required width of the channel, namely 160 m., is available.

Noted that the granite carrier is the largest vessel for the new port, tugboats have to be applied to allow the carrier to enter the port. Design conditions for this procedure and the required space will be determined in a further phase.

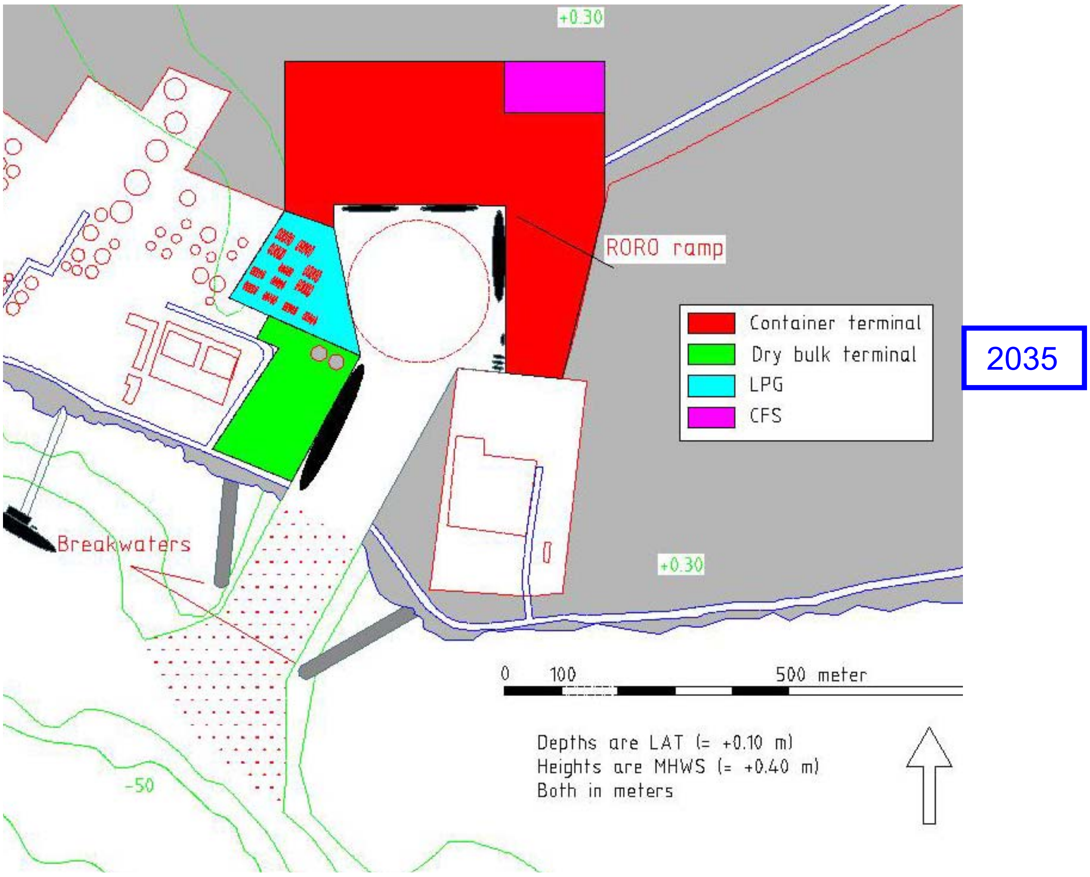
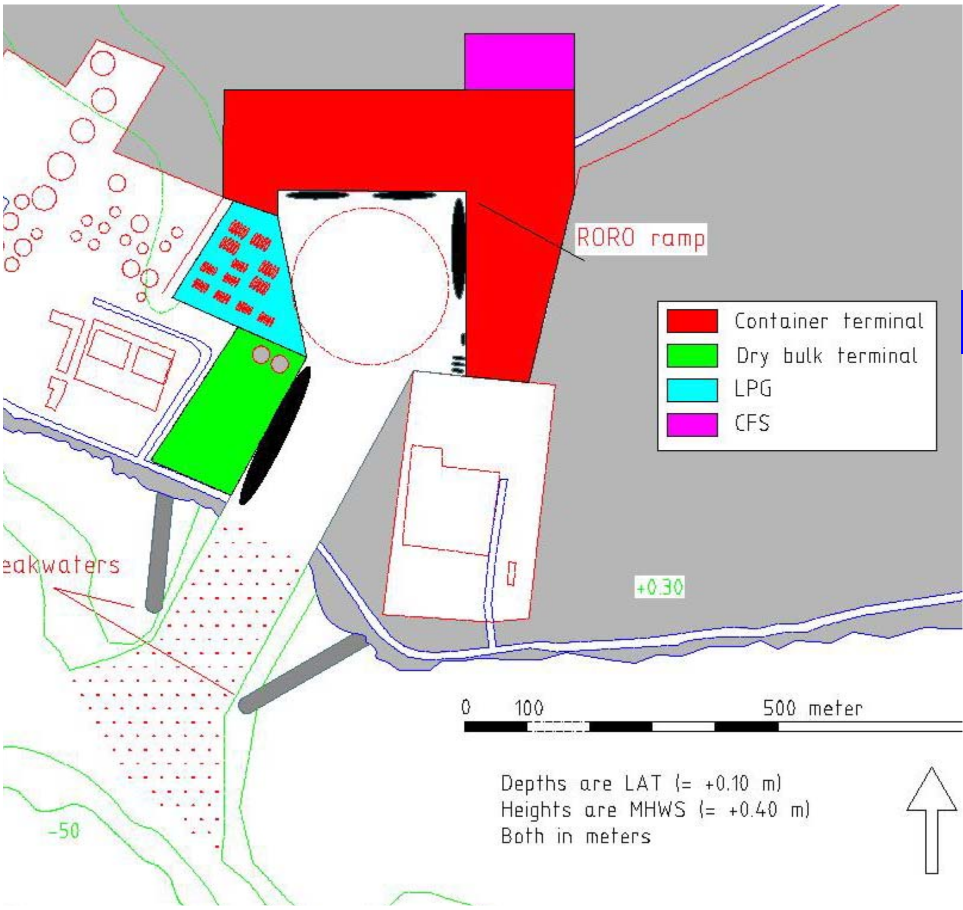


Figure 7-2: Layout alternative A1: 2035



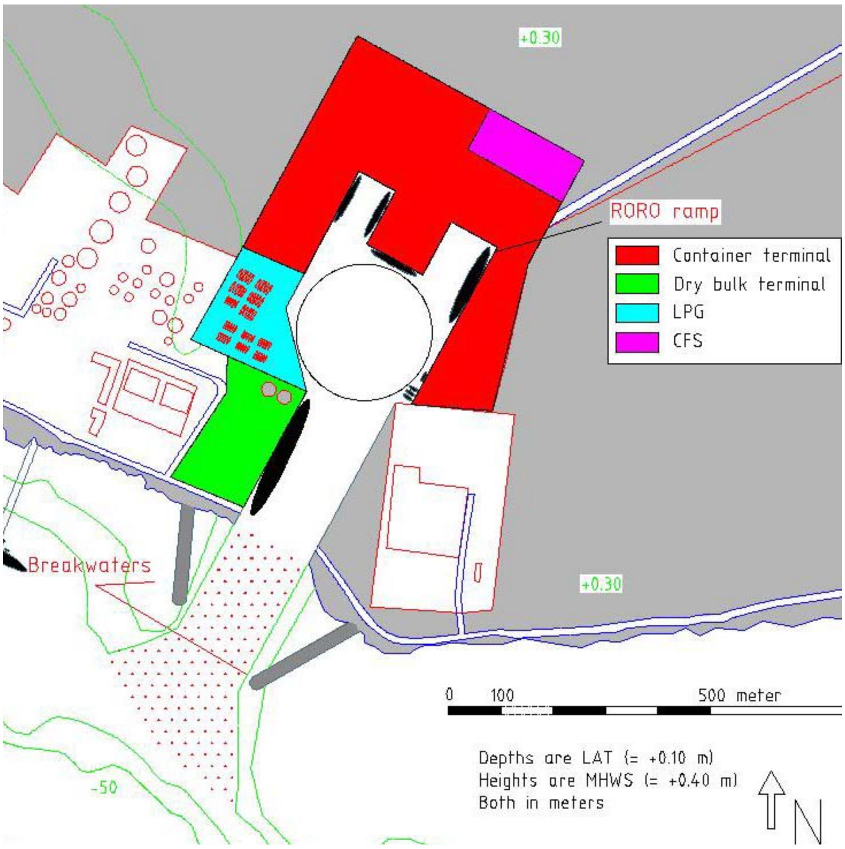
2020

Figure 7-3: Layout alternative A1: 2020

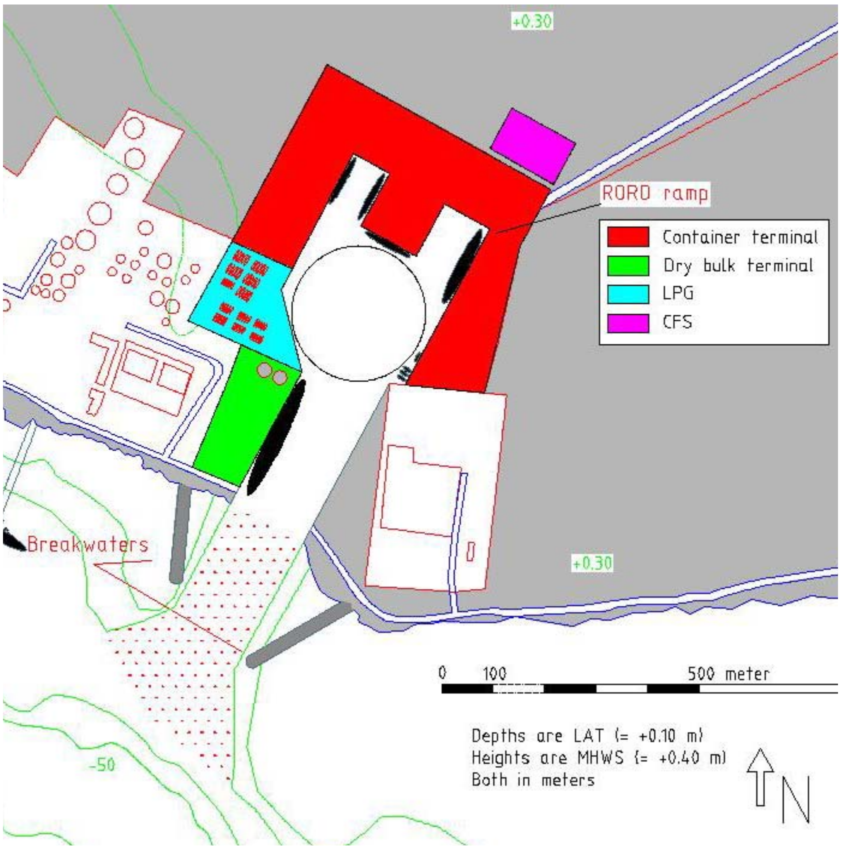
7.2.2 Alternative A2

The base of alternative A2 is also a port with an inland basin like alternative A1. The major difference is the shape of the port's basin. For alternative A2 the shape of the inner basin is clearly optimized. A major difference is the choice for a single pier structure for the vessels moored at the container terminal. This also generates an option to apply space at the northern quay wall for constructing an expansion. This increases the flexibility of this alternative.

Also with this alternative the effect of seiches has to be further researched. The same advantages and disadvantages also apply as alternative A1, like the less wave energy entering the port and the large area of limestone to be cut. In relation to alternative A1 manoeuvring the vessels will be more difficult.



2035



2020

Figure 7-4: Layout alternative A2, 2020 and 2035

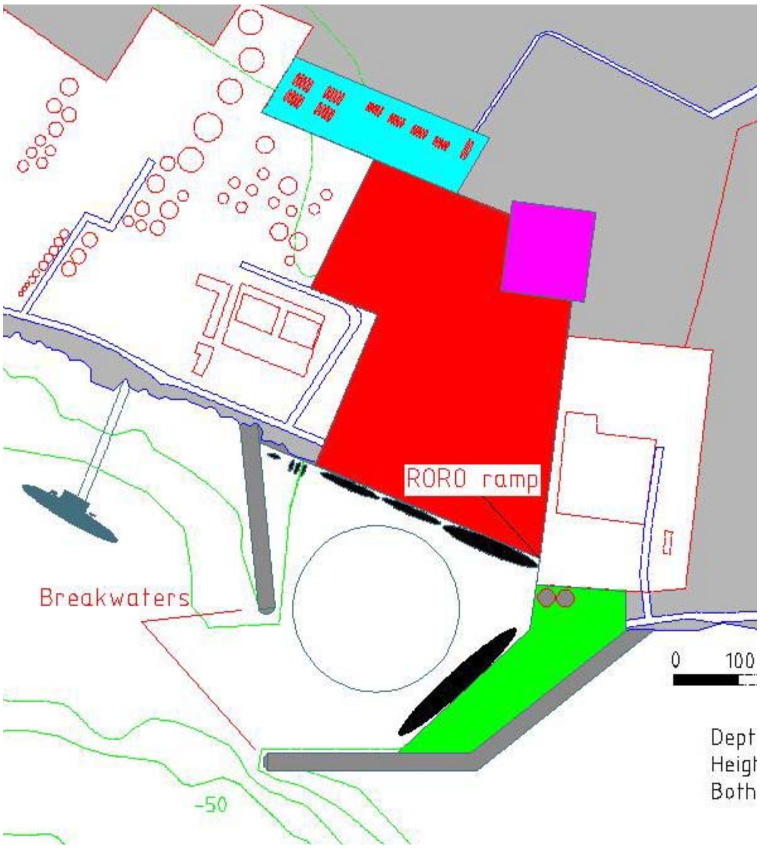
7.2.3 Alternative B1

The objective of alternative B1 is to remove a minimal amount of land. Therefore the quay walls are directly located at the shoreline, instead of the inland berths of alternative A1 and A2.

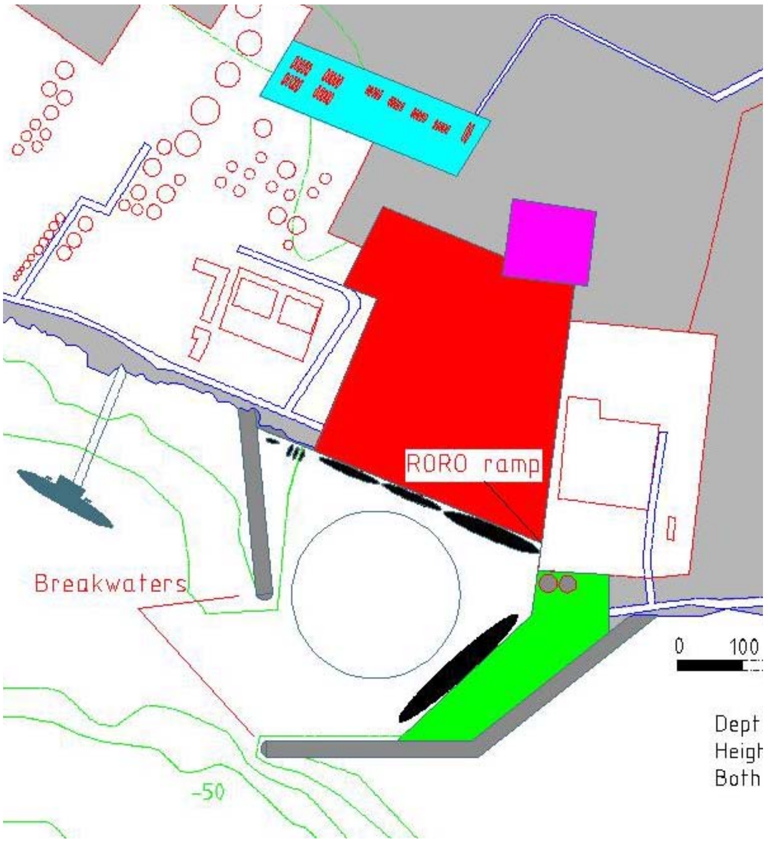
The width of the coast between the Power Plant and Commonwealth Brewery is limited and insufficient to locate the combined quay length of both terminals of 665 m. A design solution is to locate the dry bulk terminal seawards in a relatively shallow area of the sea. This new land reclamation will combine the function of terminal and breakwater. The area behind the breakwaters will have to be protected against the design wave heights, mentioned in table 6-22.

The width of the land area between the power plant and brewery will be fully used as surface area for the container terminal. This to apply an optimal usage of the available land behind the berths. The travel distance from berth to storage has to be minimised. Larger distance will increase the transport costs for terminal operations.

By positioning the terminal close to the shoreline, the entire LPG storage facility has to be relocated. For the reason of safety conditions the LPG tanks are placed at the edge of the new port structure.



2035



2020

Figure 7-5: Layout alternative B1, 2020 and 2035

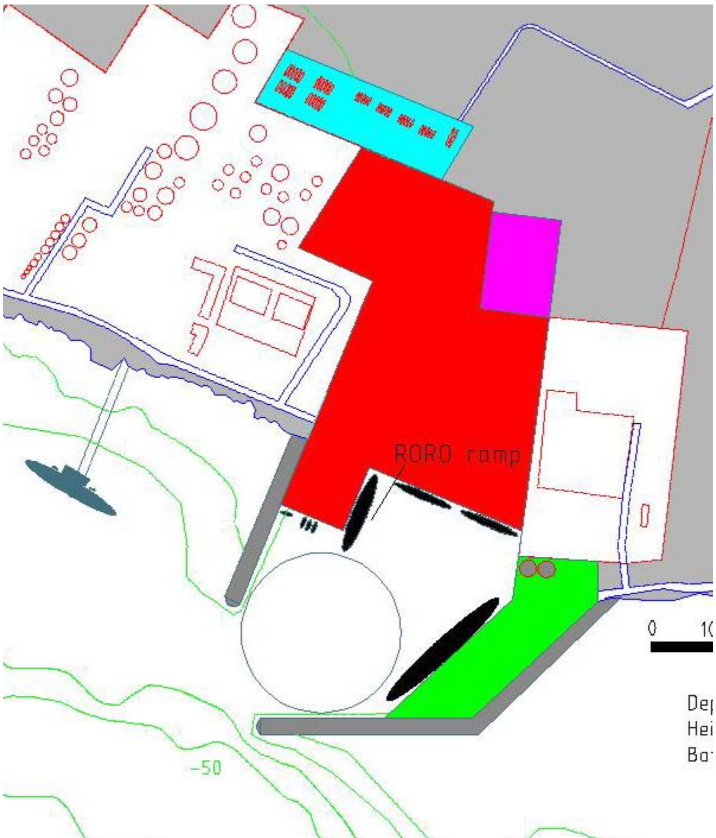
7.2.4 Alternative B2

The port structure of alternative B2 resembles a large part of alternative B1. Also in this alternative the dry bulk terminal is integrated with the eastern breakwater and the container terminal is located directly to the coastline.

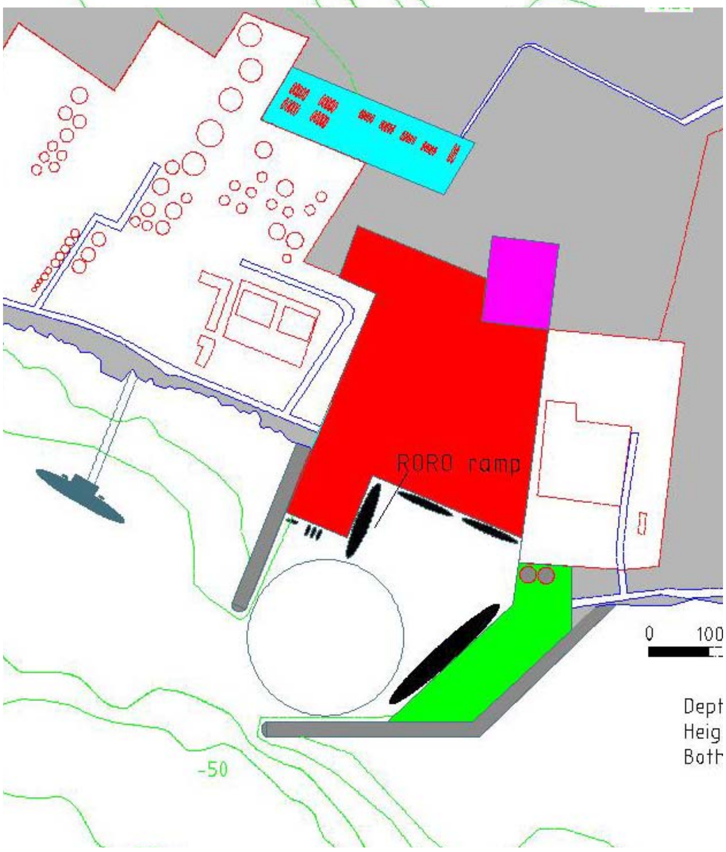
The difference with alternative B1 is the expansion into the sea, instead of an expansion to the fallow land west of the terminal area. With the seaward expansion the manoeuvrability of the vessels will decrease. Also the costs for the reclamation of additional land are expected to be higher than an expansion to the west. It can be expected that the accessibility of the terminal will increase more than with the western expansion.

On the contrary the length of the breakwaters is expected to be smaller.

As in the case of alternative B1 the entire LPG storage facility will have to be relocated. The CFS is located with a distance of 100 m.
from the container terminal in the first construction phase. In this way the CFS allows the container terminal to expand in an efficient way.



2035



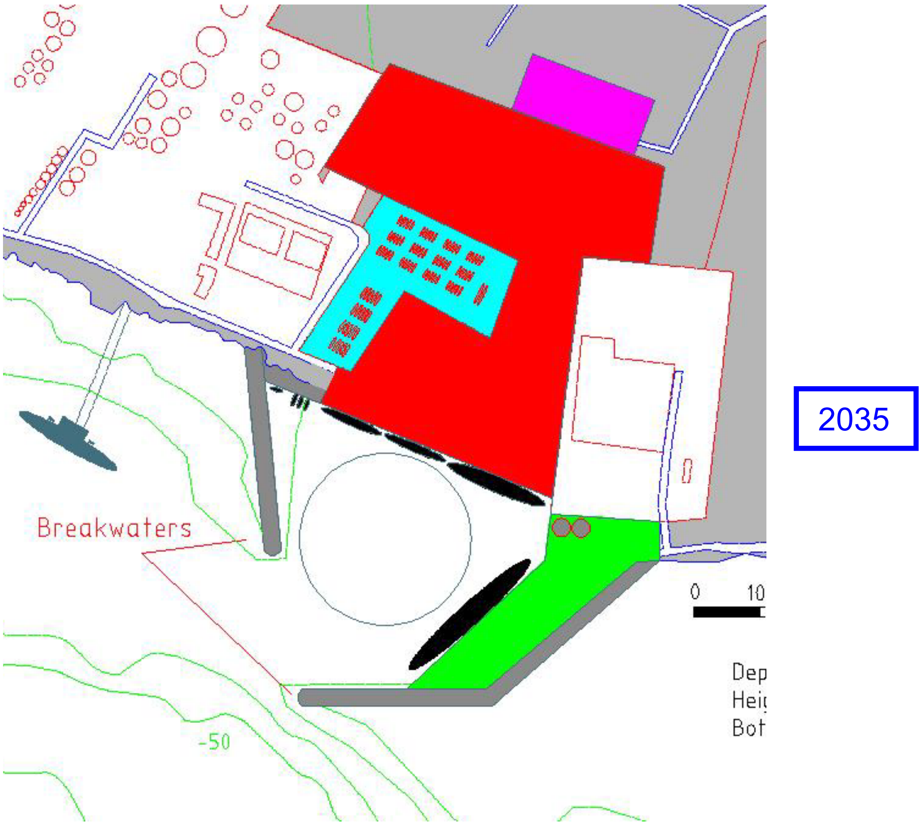
2020

Figure 7-6: Layout alternative B2, 2020 and 2035

7.2.5 Alternative C

The relocation of the LPG storage facility will increase the alternative's construction costs. Therefore in alternative design C the LPG storage tanks remain stationary. Since the required surface area for the LPG tanks grows from currently available 2.9 ha to 3.1 ha, a small expansion should still be possible.

The implication of keeping the LPG storage stationary is the fact that the required space for the container terminal is limited.
Between the LPG facility and the Brewery a distance of 100 m. exists.
When this distance is proved to be insufficient, it is possible to relocate a part of the LPG facility. This will be elaborated in a further stage when the alternative is proven to be worthwhile.



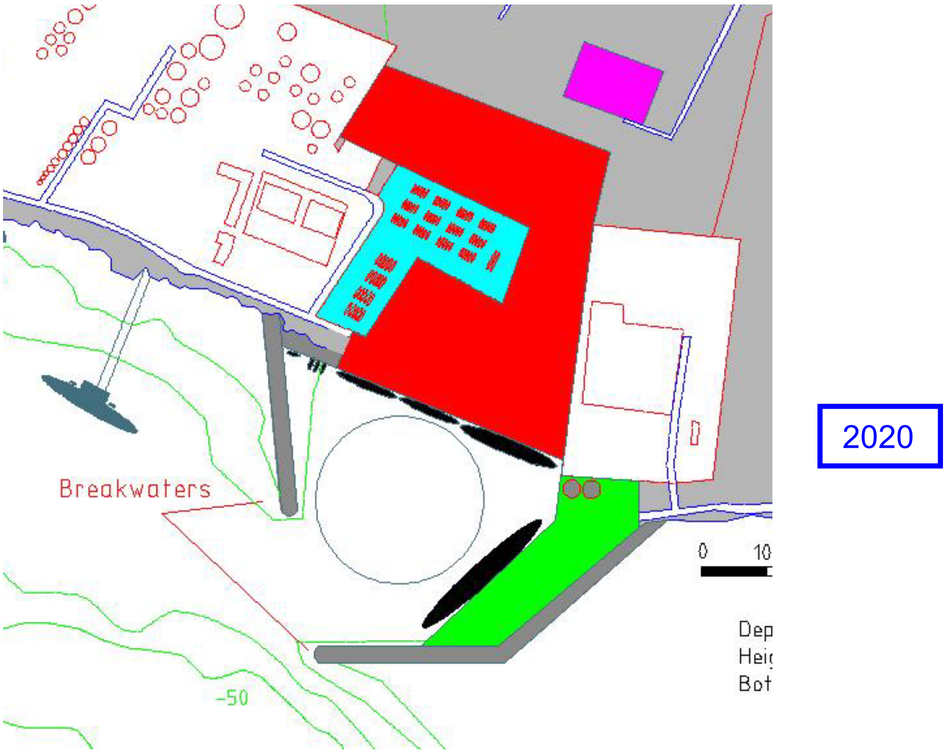


Figure 7-7: Layout alternative C, 2020 and 2035

7.3 Design of petroleum and LPG berths

Due to the generated alternatives of the new port at the location of the Power Plant, the present facilities for the liquid bulk cargo require to be relocated. In figure 7-8 the possible relocation is presented. Along the coastline the influence of the coastal structures of the generated alternatives, like breakwaters and navigation channel, is also presented.

The purple areas, including the hatched areas, represent the area in which no other vessels are allowed to pass when the berth is occupied. The blue areas represent the areas in which no other installations besides a jetty and pipelines are allowed.

It is therefore possible that the situation occurs of a LPG tanker located at its berth and an arriving vessel with a cargo of alcohol and molasses. This situation will result in additional waiting time for the carrier of alcohol and molasses. This is also the case for an already moored alcohol and molasses carrier with an arriving a LPG tanker.
The probability of both events occurring at the same time is relatively very small. The use of the berths at the yearly basis in the year 2035 is 74 hours (LPG) and 94 hours (alcohol and molasses); resulting in a probability of 1.22 x 10^{-4}. For this reason and the occurring waiting time this situation is neglected.

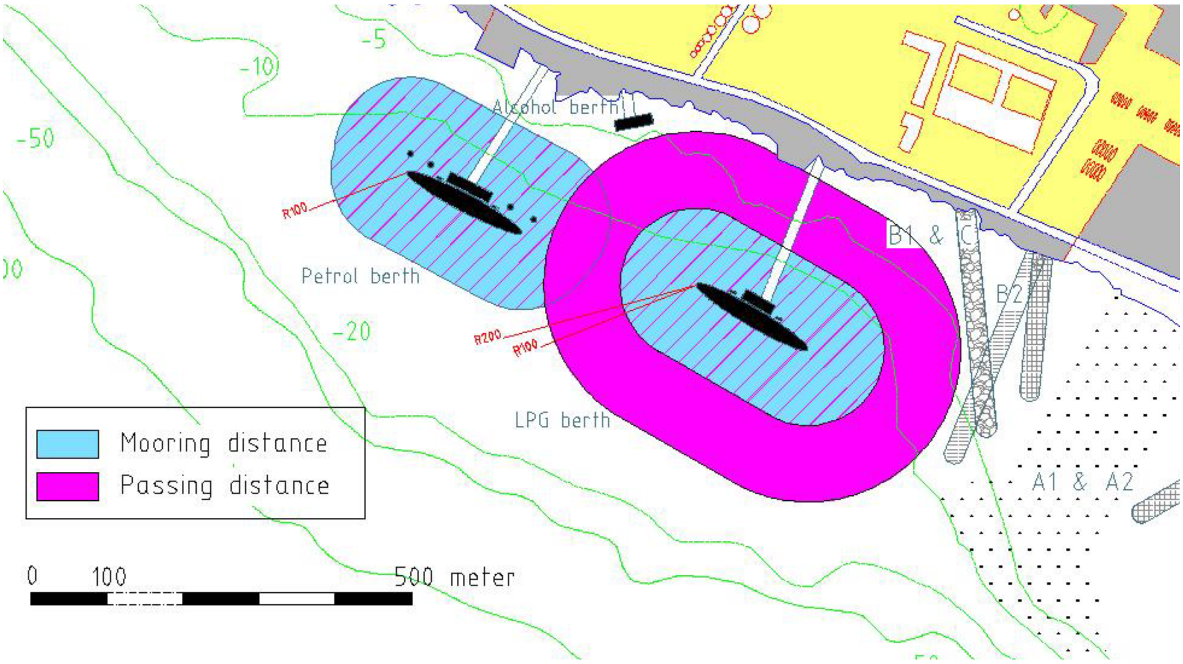


Figure 7-8: Relocation of LPG berths

As stated in paragraph 6.2.4 the facilities at its current form are able to cope with the increasing of the number of calls (petrol) or the increasing of the vessel's draft (LPG and alcohol/molasses).
For the new design of the petroleum and LPG berths the LPG berth is shifted in western direction along a distance of 200 m.

At the current location the minimum bottom level surrounding the liquid bulk berths is -10.00 m. MSL, presented in figure 4-2.
The expected vessels will have an increasing draft. According to table 4.5 the maximum draft is 11.60 m. (LPG) and 11.40 m. (petrol).
The minimum bottom level is then determined on -17.50 m MSL.
For the location of the liquid bulk berths additional dredging is required.

8 Multi Criteria Analysis

The best alternative is selected by applying an multi criteria analysis (MCA). Several design criteria will be used to test the five alternatives from paragraph 7.2. First all criteria used for selection are elaborated. These criteria contain for instance environmental and functional conditions of the different designs.

For the next step all alternatives receive a value for each criterion. By using weight factors a total score per alternative is obtained. To select the best alternative the construction costs are estimated. By combining the MCA score and the construction costs the lowest cost per value point is determined. The score which obtains the lowest cost per value point is selected as the best design alternative for the new port layout.

8.1 Evaluation criteria

The criteria are divided in 4 main categories, namely functionality, environment, preservation and development. Each of these categories has one or more criteria, which can be seen below.

<u>Functionality</u>
- Accessibility
- Re-arrangement
- Safety
- Downtime

<u>Environment</u>
- Disturbance
- Environmental harm

<u>Preservation</u>
- Hindrance

<u>Development</u>
- Phasing

<u>Functionality</u>
The functionality of the port structure can be described by several criteria. The criterion of **accessibility** measures the manoeuvrability of the port. The maritime accessibility determines the quality of the port's turning basin and additional water infrastructure. The terminal accessibility reviews

the surface area of the terminals at the quality of access to berths.
The criterion of **re-arrangement** indicates the flexibility of the destination of the port's surface area. When certain functions are bound to a location, the flexibility of the port as a whole will decrease.

For the **safety** criterion the safety for the vessels entering and leaving the port are checked. When the basin is intensely used and many crossings between vessels occur, the safety of the port decreases. It can therefore be noted that the criteria of accessibility and safety are linked, since a relatively bad access means lower safety conditions.

Although an upper limit of the downtime is stated, there's a difference between the expected **downtime** between the alternatives. It can be expected that the alternatives with an inland basin result in a smaller downtime.
For the consumers of the port the downtime is of major importance.

<u>Environment</u>
Some of the port's operations **disturb** the environment: dry bulk piles can create dust, machines generate noise.
These aspects have to be minimized for the local people, tourists and flora and fauna living in the vicinity.

The construction of a new port causes **environmental harm**. The bottom of the sea will be ruffled and some parts will be destroyed.
The sea bottom at the area of the new port has however already suffered damage from recent port operations. The marine environment on the shallow coast east of the port location has a high quality. For this reason constructing a breakwater has a large impact.
Next to the marine environment the terrestrial environment will also be affected. The EIA study notes that the flora and fauna present at the new project site does not have uncommon or rare characteristics.

<u>Preservation</u>
The port requires regularly maintenance to remain operational.
Aspects of maintenance are repairs and dredging activity, where dredging will be most common. Since these operations can cause downtime for parts of the port, the effects are elaborated under the criterion of **hindrance**.

<u>Development</u>
All alternatives are constructed in 3 construction phases. When alternatives are not easy to expand, because of unavailability of land, or cause a lot of irregularities during expansion, the **phasing** of the design will be badly graded.

8.2 MCA

All alternatives of the new port are given a value between 1 and 10 for each criteria, where 1 presents a negative aspect and 10 presents the most positive.

8.2.1 Weight factors

The level of importance for each criterion is unequal. Therefore criteria receive a weight factor varying from 1 to 3. The final score for each alternative is obtained by multiplying the values with the weight factor. A larger weight factor implies a higher importance of the criterion.

Each user of the port has a different set of priorities for the new port design. Terminal operators require an efficient handling and storage area for the cargo. Environmental specialists are mainly interested in the prevention of environmental damage. These varying perspectives and the accompanying selection of alternatives are determined in paragraph 8.5.

The decisive selection of the final design alternative is based on the following overall design perspective.

Accessibility can be seen as the most important factor for a port structure. The phasing of the port has a smaller importance because the difference in requirements between the three phases of construction is relatively low.

Because of this difference in importance each criterion receives a weight factor. The weight factors of the overall design perspective are displayed in table 8-1.

8.2.2 MCA

The result of the multi criteria analysis is presented in table 8-1. The values assigned to the criteria are qualitative indicators. Explanation for the assigned values can be found below.

An open port structure is desirable for the **maritime accessibility** of the port. Because A2 is easier to enter, it receives a higher grade than A1. Because of the seaward expansion B2 receives a lower value than B1. The narrow access to the berths of alternative C are the cause of a low value in **terminal accessibility**.

The factor of **re-arrangement** decreases when the port structure is designed more compact. The **safety** of an open port structure will be higher due to the missing of an inland turning basin.

The **disturbance** of port activities to the environment will be higher for A1 and A2. For alternatives A1 and A2 the terrestrial surface area is larger than the influenced area at the sea bottom, where the quality of the marine life is of higher value. The total environmental value of the total affected area by port construction shows that alternatives B1, B2 and C receive a higher value (**environmental harm**)

The port structure with the inland basin receives more **hindrance** from maintenance than an open port structure.
Especially in the case of maintenance in the navigation channel, some downtime will occur.

	Weight factor	A1	A2	B1	B2	C
Functionality						
Maritime accessibility	2	5	6	7	6	7
Terminal accessibility	2	8	8	6	6	4
Re-arrangement	1	8	7	7	6	4
Safety	2	5	6	7	6	7
Downtime	2	9	9	7	7	7
Environment						
Disturbance	2	6	6	7	7	7
Environmental harm	2	6	6	7	7	7
Preservation						
Hindrance	2	5	5	7	7	7
Development						
Phasing	1	6	6	5	5	4
Total score		102	105	108	103	100

Table 8-1: Multi criteria analysis

According to table 8-1 alternative B1 is the most optimal design for the new port structure. With a score of 103 B2 proves to be second best. These optimal designs however are judged from a technical point of view; the port with the most optimal aspects for users and locals. Another important selection criterion, the construction costs, are excluded from the MCA. To include the financial factor, the construction costs are determined in paragraph 8.3.

8.3 Construction costs

The construction cost for each alternative is roughly estimated on the dimensions of the design. After the MCA and approval of the authorities, the selected alternative will require a more detailed study, including determination of the total construction costs.

Alternative C turns out to be the alternative with the lowest construction costs. This is due to the fact that the design is as compact as possible and no relocation of the LPG storage tanks takes place.

Some important aspects of the new port design are not taken into account. For instance the cargo handling equipment at the terminals are excluded. It is assumed that the costs for terminal equipment is constant for all alternatives.

All land used occupied by the layout of the alternatives is already owned by the state. Land acquisition is therefore not required.
Since all soil layers are expected to exist of limestone rock the costs for dredging are relatively high.

Alternatives	Unit price	Costs per alternative (in 1000 €)				
		A1	A2	B1	B2	C
Preparation of site	5 €/m2	1.280	1.280	1.280	1.280	1.115
Dredging						
port basin	20 €/m3	42.100	51.300	22.130	19.580	23.360
land reclamation	5 €/m3	-	-	875	1.130	875
Quay wall						
main berths	20.000 €/m	15.000	15.000	13.300	13.300	13.300
berths smaller vessels	10.000 €/m	6.000	7.500	-	-	-
minimal shore protection	5.000 €/m	2.000	-	-	-	-
Terminal						
pavement: heavy duty	150 €/m2	27.900	27.900	27.900	27.900	27.900
pavement: asphalt	100 €/m2	7.000	7.000	7.000	7.000	3.700
offices, warehouses		10.000	10.000	10.000	10.000	10.000
Breakwater	50.000 €/m	21.000	21.000	45.000	44.000	45.000
Relocation LPG terminal						
LPG tanks	120.000 €/tank	2.760	2.760	2.760	2.760	-
LPG pipeline	2.000 €/m	400	400	800	800	-
LPG berth		3.000	3.000	3.000	3.000	3.000
Public roads and utilities						
Roads, power supply, water supply fire fighting		5.000	5.000	5.000	5.000	5.000
Total Direct costs		143.440	152.140	139.045	135.750	133.250
Indirect costs						
Profit, risk and insurance	20 % of total direct costs	28.688	30.428	27.809	27.150	26.650
Total Construction costs	per 1000 €	172.128	182.568	166.854	162.900	159.900

Table 8-2: Indication of construction costs

8.4 Selection of best alternative

The best alternative from an overall design perspective is selected by determining the cost per point of value. These are presented in table 8-3. It is concluded that the best alternative from the MCA is alternative B1. It has the lowest cost per quality point of value.

Total costs different alternatives			
	Costs (€)	Points MCA	Cost per point (€)
A1	172.128.000	102	1.687.529
A2	182.568.000	105	1.738.743
B1	166.854.000	108	1.544.944
B2	162.900.000	103	1.581.553
C	159.900.000	100	1.599.000

Table 8-3: Cost per point method

8.5 Selection of alternative from perspective of specific participant

To check the MCA for varying results the selection will be repeated for specific participants of the new port design. For the final score the distributed values for each criterion remains constant. The weight factors differ in relation to the perspective of the participant.

The participants used for a secondary selection are:

<u>Terminal operator</u>
The operator of the terminal will be mainly interested in the efficiency of the terminal area. The accessibility of the port has its primary interest. Especially the downtime of the port has to be minimised.

<u>Nautical expert</u>
The nautical expert, representing the captains of the visiting vessels, will concern the maritime accessibility and the safety of all vessels within the port. Hindrance because of maintenance has to be prevented.

<u>Economical expert</u>
For the economical process the downtime is the main importance to the economical expert. Hindrance which influences the total downtime has to be minimised.

<u>Environmental expert</u>
The main concern for the environmental expert are the environmental criteria of the different port designs. Since occurring disasters and future expansion of the port have an influence on the environment, the criteria of safety, hindrance and phasing are of secondary importance.

The weight factors of each participant are presented in table 8-4.

	Overall designer's point	Terminal operator	Nautical expert	Economical expert	Environmental expert
Functionality					
Maritime accessibility	2	2	3	1	1
Terminal accessibility	2	2	1	1	1
Re-arrangement	1	2	1	1	1
Safety	2	1	3	1	2
Downtime	2	3	1	3	1
Environment					
Disturbance	2	1	1	1	3
Environmental harm	2	1	2	2	3
Preservation					
Hindrance	2	1	3	3	2
Development					
Phasing	1	2	1	2	2

Table 8-4: Weight factors of specific participants of the new port design

By applying the cost per point method of paragraph 8.4 the best alternative is selected for each perspective. It can be concluded that with the exception of the economical perspective B1 is the best alternative. From an economical perspective alternative B2 is favoured. This is explained by the difference in construction costs and the minor difference in scores of the MCA.

Total costs different alternatives								
	Terminal operator		**Nautical expert**		**Economical expert**		**Environmental expert**	
	Score	Cost per point (€)	Score	Cost per point (€)	Score	Cost per point (€)	Score	Cost per point (€)
A1	103	1.671.146	94	1.831.149	98	1.756.408	96	1.793.000
A2	104	1.755.462	99	1.844.121	99	1.844.121	98	1.862.939
B1	99	1.685.394	109	1.530.771	100	1.668.540	107	1.559.383
B2	93	1.751.613	102	1.597.059	99	1.645.455	103	1.581.553
C	87	1.837.931	103	1.552.427	93	1.719.355	100	1.599.000

Table 8-5: Cost per point method for specific participants

8.6 Sensitivity analysis of MCA

The result of the multi criteria analysis is determined by a large amount of factors. Varying criterion values, weight factors or a different indication of construction costs will influence the final result. This paragraph presents the sensitivity of these factors for the selection of the final alternative. The margins between the selection of a certain alternative are determined.

A sensitivity analysis can exist of several levels of complexity. When for instance criterion values as well as weight factors are altered the flexibility of the analysis and thereby the complexity increases. For this particular sensitivity analysis only one degree of freedom is applied.

<u>Sensitivity of alternative B1 in relation to alternative B2</u>

In the current MCA the margin of the score values between alternative B1 and B2 is 2.5. In the case that two criterion values (for example Maritime accessibility and Safety) change with a single point the selected alternative will alter. When changing the weight factors it will not be possible to alter the selected alternative due to the constant criterion values.
An increase of 2.4 % of the construction costs of alternative B1 in relation to alternative B2 alters the selection of the final design alternative.

<u>Sensitivity of alternative B1 in relation to alternative C</u>

Between alternative B1 and C a margin of the score values of 3.5 occurs. In relation to alternative C, the design of alternative B1 has a higher quality. Therefore the ranking of the criterion values remains constant. The only criteria which are able to change are the terminal accessibility and re-arrangement. Increasing both criterion values with 2 results in selecting alternative C as final design alternative.

By altering the weight factors it is impossible to generate a solution which has alternative C as the best alternative. It has to be noted that the value distribution of weight factors of 1 to 3 remains constant.
An increase of 3.5 % of the construction costs of alternative B1 in relation to alternative B2 will alter the selection of the final design alternative.

As a conclusion of the sensitivity analysis the following can be noted:

Varying the criterion values has a relatively small influence on the selected alternative. By altering one criterion value the selection is not affected. Altering the weight factors has almost no influence. When the scaling of 1 to 3 remains intact the selection only differs if an economical perspective is used. The option of using a different method of weighting the criteria is not reviewed.

For an increase of 2.4 % in construction costs of alternative B1, alternative B2 becomes the best alternative. However increasing the costs with 2.4 % (around € 4 million) is a large amount in relation to alternative B2, because the designs have a large resemblance.
With a difference of almost € 4 million in the current cost indication the difference will have to increase by 100 %.

8.7 Conclusion

The following can be noted:
although the costs of alternative C are the lowest construction costs of all alternatives, the alternative of B1 is selected as the most adequate design for the new port accommodation. This is because the quality in relation to the cost is the highest for alternative B1.

The sensitivity analysis shows that the result of the MCA is relatively stable. Applying different perspectives of participants has a relatively small effect on the result. Also altering the criterion values has no effect when one value is changed. The required margins of construction costs are also relatively large.

9 Preliminary design of breakwaters

The design of the new port at Clifton Point contains various structures of hydraulic engineering. Among those structures the breakwaters have a relatively large impact on the layout of the final design and the surroundings of the port as a whole. A sufficient design of both breakwaters will result in a feasible port, because of the reduction of wave energy penetrating the port basin.

Therefore a preliminary design of the breakwaters is presented in this chapter. The design mainly focuses on two aspects. The first is a research on the amount of penetrating wave energy, which is the cause of the port's downtime. The objective states a maximum of the mean annual downtime of 20 days.

The second aspect is the production of an economic design. The indication of the construction costs presented in paragraph 8.3 shows that the costs for the breakwater are 28 % of the total construction costs (table 8-2). It is therefore desirable to optimize the breakwater design.

This chapter is structured in the following way:

In the first paragraph the functions of the breakwater are determined. Secondly the optimal configuration for the breakwater is presented. The optimal design is selected by compromising between occurring downtime and construction costs. Finally the cross-section of the breakwater is designed.

The navigational aspect of the configuration will also be evaluated.

9.1 Functions of breakwater

9.1.1 Functions of breakwater

To assure the capacity of the port as stated in chapter 5, the percentage of operational time per year should be as high as possible.

When the port is out of operation due to meteorological causes, accidents etc. this is called downtime. Breakwaters provide protection for the port activities. They give shelter against the wave climate outside of the port.

The breakwater has to fulfil the following functions:

- The breakwater has to protect the inner basin from the deep water wave climate in such a way that the mean annual downtime is below 20 days.

- The structure has to be able to withstand the extreme weather and wave conditions of an annual probability of 1:100.

- The structure will not function as a walkway for vehicles and pedestrians.

9.1.2 Design requirements breakwater

The design functions have to be translated into the design requirements:

- The occurring downtime caused by operational wave conditions has a maximum of 20 days.

- The extreme design conditions of the breakwater have a return period of 100 years.

- During extreme conditions the structure may not fail.

- The construction costs of the breakwater have to be as low as possible.

- For navigational purpose the breakwater has to be visible during operational conditions.

- No traffic over breakwater after completion.

9.1.3 Design assumptions breakwater

- When the critical wave height (H_{cr}) at a berth is exceeded, the total berth is considered to be out of operation.

- For the preliminary design phase the bottom levels of the breakwater's cross section are assumed to be constant:
    - Western breakwater: - 4.00 m. MSL
    - Eastern breakwater: - 5.00 m. MSL

9.2 Determination of configuration breakwater

The configuration of breakwaters as presented in the design of alternative B1 in chapter 7 requires optimization. When the layout is optimally adapted to the wave climate the downtime within the port and the length of the breakwater structures can be minimised.

The breakwaters have to be placed in such a way that the annual downtime for each terminal is acceptable. In table 6-22 (paragraph 6.4.1) the maximum H_s was determined under which the various vessels were able to safely perform their operations at the berth. For waves from the angle of 45° to 90° affecting the container and RoRo vessels no recommended value is provided. Therefore the maximum wave height is assumed to be equal to the maximum head and stern waves, a H_s of 0.50 m.

With this information the following maximum wave conditions are allowed within the port as presented in figure 9-1.

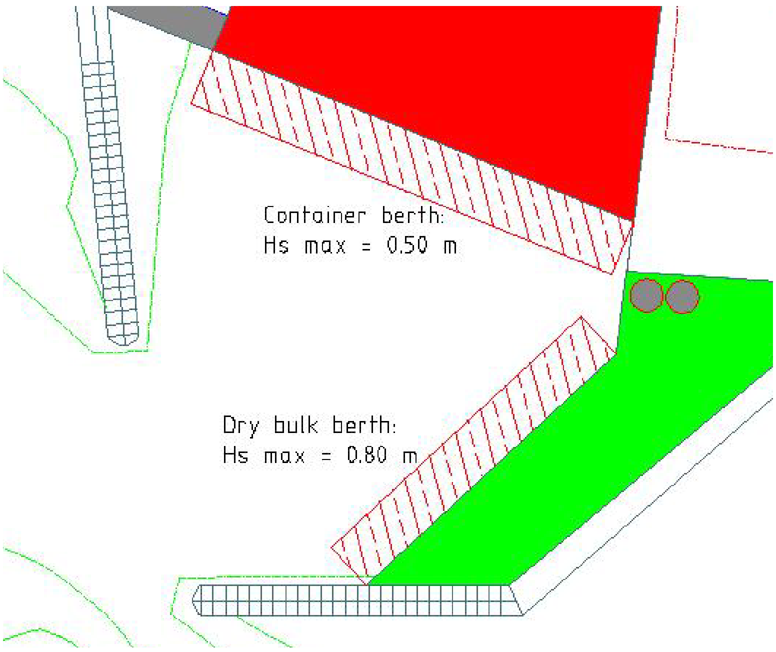


Figure 9-1: Critical wave heights in front of particular berths

To determine the downtime the operational wave conditions are used according to paragraph 3.6.3. The maximum mean annual downtime within the port as result of the local wave climate is 20 days; this is 5.50 % annually.

The methodology to determine the amount of downtime is presented in figure 9-2. Each element will be described in detail in this paragraph.

Methodology used for downtime determination for each breakwater configuration

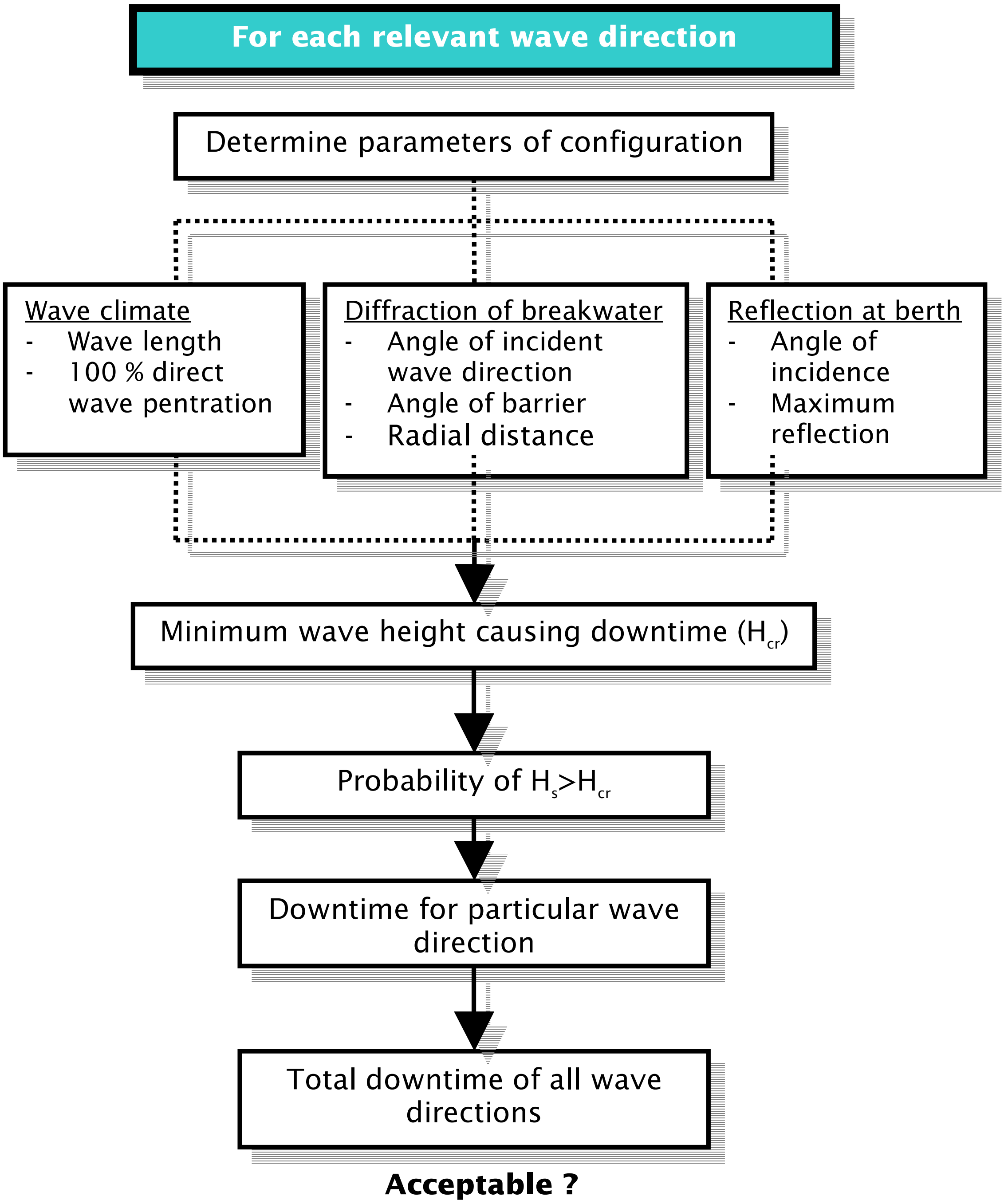


Figure 9-2: Critical wave heights in front of particular berths

This method of determining the annually downtime caused by wave climate is demonstrated by using the breakwater configuration of alternative B1.

9.2.1 Determine parameters of configuration

The entrance enclosed by the breakwaters of the chosen alternative B1 was focused in western direction. This because the main direction of the local wave climate was the sector from 90° to 180°.

Figure 9-3: Breakwater configuration, alternative B1

<u>Wave climate</u>

To determine the downtime the operational wave climate as derived in paragraph 3.6 is used.

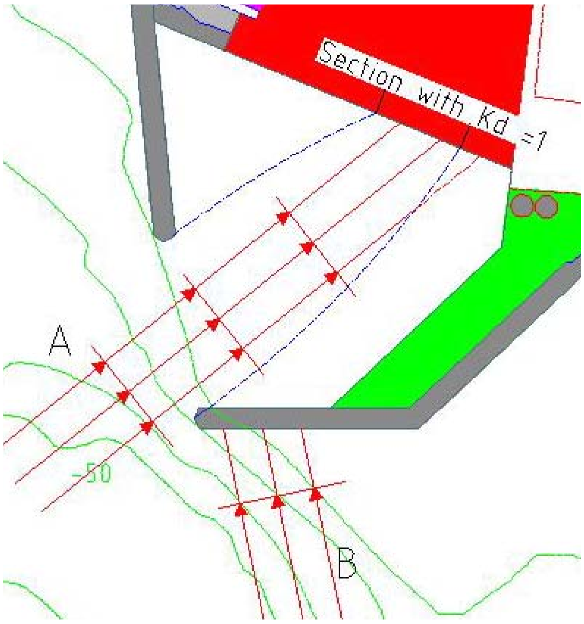


Figure 9-4: Diffraction at breakwater

Waves from all wave directions arriving at the berths of the new port structure collide into the breakwaters surrounding the port. Some wave directions however can partially reach the berth without significant hinder of the breakwaters (example: wave train A).
Waves from other directions, that do not reach the berths directly, are also able to transfer their wave energy into the port's basin (example: wave train B).
Logically these wave heights are smaller in relation to wave train A.
In the case that the wave heights of blocked waves are substantial, they can still cause critical wave conditions at the berths.

One of the aspects determining the amount of diffraction is the radial distance to the berth from the breakwater in relation to the wave length.
The wave length varies for each wave, depending on its wave height and wave period. To determine the critical wave height in front of the berth a constant wave length will be used. Including a variable wave length results in an increase of the complexity without increasing the accuracy of the method. In relation to varying α and β, it can be concluded that the wave length has a minor influence on K_D.

As an assumption wave lengths from all directions have a length of 80 m., corresponding with a wave period of 7.2 s. This assumption is based on the following general relation of wave period and wave height:

$3.6 \cdot \sqrt{Hs} < T_p < 8 \cdot \sqrt{Hs}$, with $T_s \approx 0.9 T_p$ $\rightarrow T_s = 5 \cdot \sqrt{H_s}$ and $L_0 = \dfrac{g T_s^2}{2\pi}$

As an effect of diffraction the wave height in front of the berth will decrease are evenly increase. The effect of diffraction on the wave height is therefore determined by the factor K_D. For each wave direction a profile of wave height along the berth is determined. Both the effects of diffraction and reflection are included.

<u>Diffraction of breakwater</u>

The K_D profile which is used to determine the wave height at the berths is presented as a function of r/L (radial distance to quay divided by the wave length), angle of incidence (α) and angle of radial distance with the median of the breakwater (β).
The diffraction factor follows from the table of Wiegel [1962].

Points at the lee side of the breakwater which are least affected by the waves are represented by their low β. Therefore the smaller β, the smaller K_D with constant r/L and α. For the configuration of breakwaters of alternative B1, the angle β for both breakwaters is presented in the images of figure 9-5 and 9-6. This figure can be used as indication for the variant configurations.

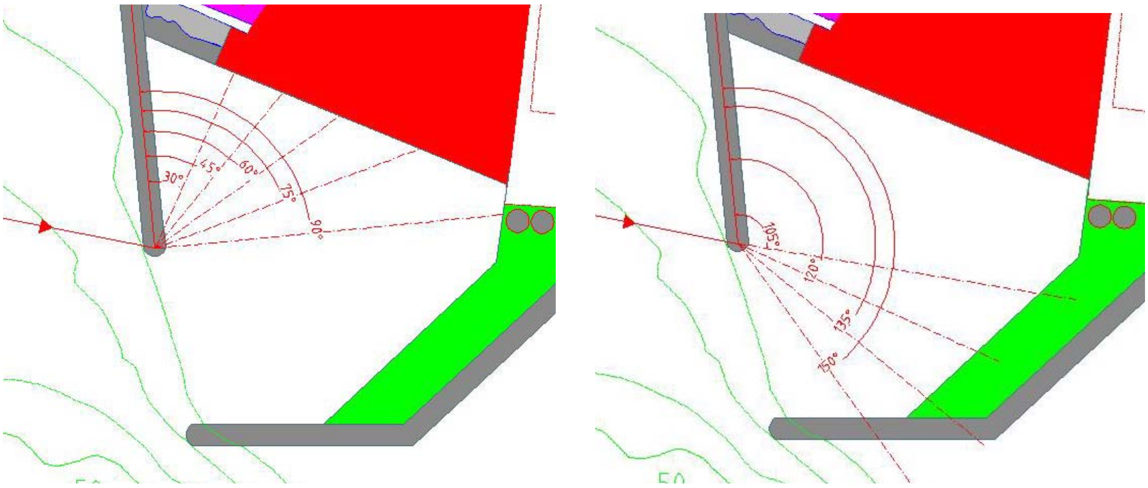


Figure 9-5: Diffraction around western breakwater

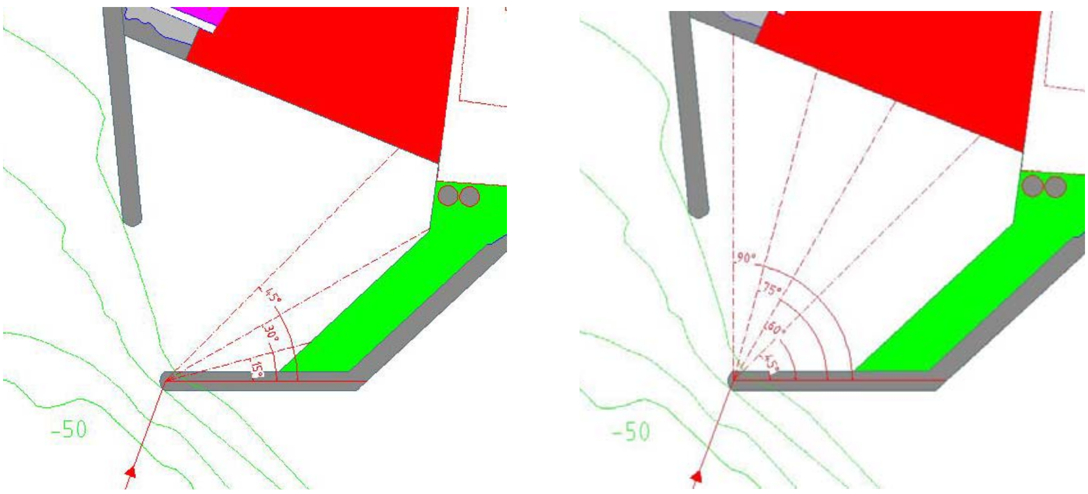


Figure 9-6: Diffraction around eastern breakwater

Wave reflection at berth

For the quay structures vertical walls are selected.
When a wave is considered two dimensional vertical walls result in standing waves in front of the berth. Standing waves are 100 % reflections of the incoming waves. This doubles the wave height in front of the quay wall, as can be seen in figure 9-7.

For three dimensional waves the waves are only reflected 100 %, when they arrive perpendicular to the quay wall. With a different angle of income the maximum reflection is determined the maximum value of the sum of the incoming (η_i) and reflected (η_t) water level elevation:

$\eta_s = \eta_i + \eta_t$, with $\eta_i = a_i \sin(\omega t - kx)$

$$\eta_t = a_t \sin(\omega t + kx + \Delta\varphi)$$

and $\Delta\varphi = (1 - \cos(2(90^o - \alpha)) \cdot \pi / 2$

where
- ω = angular wave frequency = $2\pi/T$ (s^{-1})
- k = wave number = $2\pi/L$ (m^{-1})
- $\Delta\varphi$ = phase difference (-)
- α = angle of incidence (°)

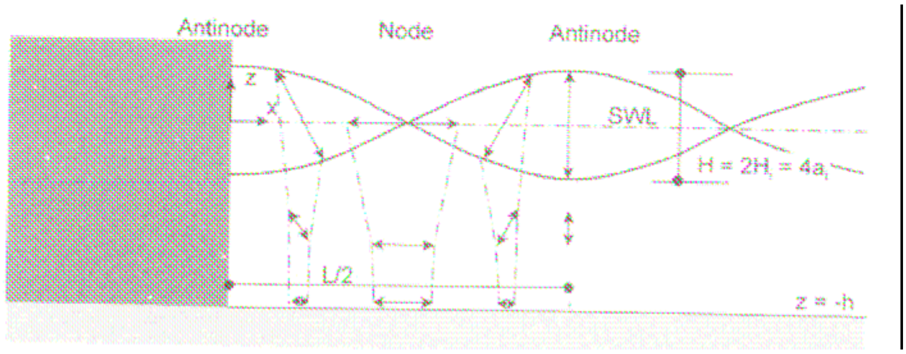


Figure 9-7: Principle of standing waves

The reflection is only determined for the terminal of interest. Waves reflecting at the other quay wall are beyond the scope of this downtime calculation. The influence of reflected waves from the other quay wall to the berth of interest should be determined in a numerical model.

Minimum wave height causing downtime (H_{cr})

All transitions of wave energy result in varying wave heights along the berths. The critical wave heights, which if exceeded are causing downtime, are mentioned in table 9-1. With the determined influence of diffraction and reflection the critical wave height at the berth can be translated to the wave condition outside the port basin:

$$H_{cr} = \frac{H_{cr-berth}}{K_D K_{RL}}$$

To illustrate the determination of the waves causing a critical wave height at the terminals, the breakwater configuration of alternative B1 is used.

First the wave profile along the berth of the container terminal is inspected.

Wave direction (°)	100% wave penetration ?	radial distance (m)	r/L (-)	α (°)	β (°)	Kd (-)	Kr (-)	KdKr (-)	Hcr (m)
170		425	5,3	100	45	0,10	1,92	0,19	2,60
"		410	5,1	"	60	0,10	2,00	0,20	2,50
"		405	5,1	"	75	0,13	2,00	0,26	1,92
"		430	5,4	"	90	0,35	1,97	0,69	0,73
180	•	425	5,3	90	45	0,12	1,92	0,23	2,17
"	•	410	5,1	"	60	0,17	2,00	0,34	1,47
"	•	405	5,1	"	75	0,26	2,00	0,52	0,96
"	•	430	5,4	"	90	0,52	1,97	1,02	0,49
190	•	425	5,3	80	45	0,15	1,92	0,29	1,74
"	•	410	5,1	80	60	0,25	2,00	0,50	1,00
"	•	405	5,1	80	75	0,56	2,00	1,12	0,45
"	•	430	5,4	80	90	1,05	1,97	2,07	0,24

Table 9-1: Sample of H_{cr} at container terminal for configuration alternative B1

It can be seen that for H_s > 0.73 m. of wave direction 170° downtime will arise even though the berths are not directly affected by waves from that direction.

All wave directions influencing the container berth are shown in table 9-2. For example the H_{cr} corresponding with 270° will never be exceeded, because the maximum H_s of this direction is 1,00 m.

Wave direction (°)	Kd (-)	Kr (-)	Hcr (m)
150	0,13	2	1,92
160	0,19	2	1,32
170	0,35	1,97	0,73
180	0,52	1,97	0,49
190	1,05	2	0,24
200	1,05	2	0,24
210	1,05	2	0,24
220	1,05	1,97	0,24
230	1,05	1,84	0,26
240	1,05	1,59	0,30
250	0,52	1,2	0,80
260	0,35	1	1,43
270	0,13	1	3,85

Table 9-2: H_{cr} at container terminal for configuration alternative B1

9.2.2 Calculation of downtime

With the critical wave height determined it is now possible to calculate the probability of exceeding H_{cr} and thereby causing downtime for the terminal reviewed.

Probability of $H_s > H_{cr}$

To calculate the downtime for each wave direction the exceeding percentage of $H_s > H_{cr}$ is required. Since the wave heights are distributed in ranges of 0.50 m. to 1.00 m. a fitting distribution is necessary. For this procedure a Weibull distribution is selected. A Weibull distribution is often used for long term wave statistics. An alterative is the log normal distribution.
The Weibull distribution contains three fitting coefficients.
The degree of freedom is therefore relatively large. Since the distribution and available wave data is ‘fitted by eye’, the amount of freedom is decreased by applying a two parameter Weibull distribution by selecting A=0.

The cumulative function of the Weibull distribution is presented by the formula of:

$$P(H_s > H_{cr}) = 1 - e^{-\left(\frac{H_S - A}{B}\right)^C}$$

where A, B and C are the fitting parameters.

An example of the Weibull distribution is presented in figure 9-8.

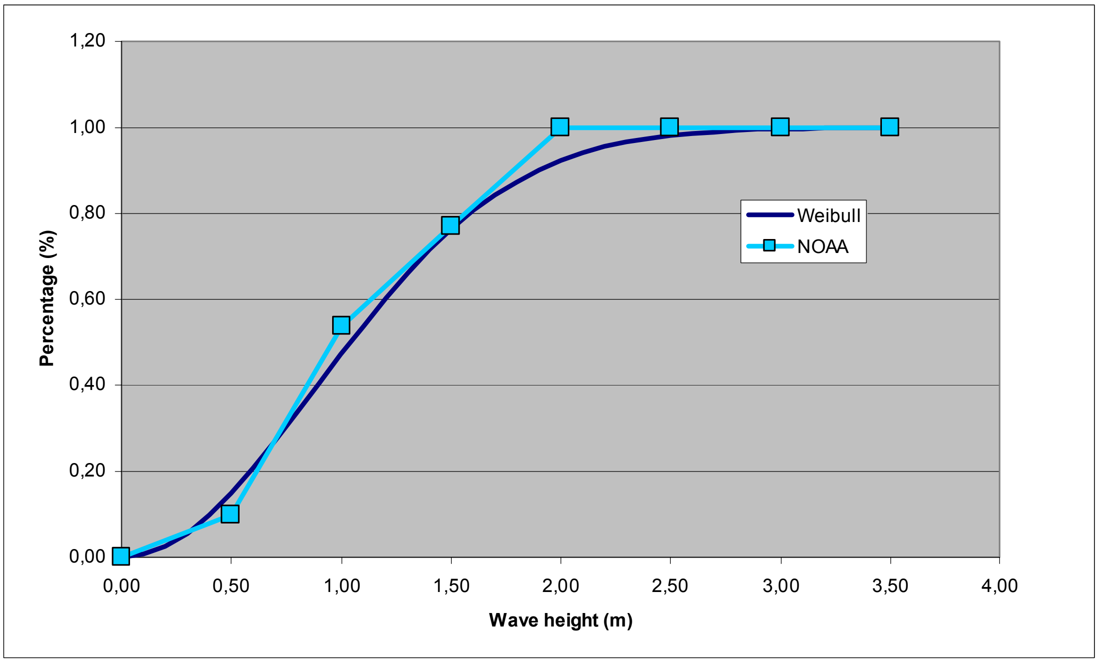


Figure 9-8: Cumulative distribution of H_s from a wave direction of 170°

Applying the Weibull distribution results in the following side effects.
These effects are general for waves from all wave directions.
The effects are attended by the distribution of waves
from a wave direction of 170°. For the fitting parameters of this direction
the following values are selected:

B= 1.25 C= 2

The percentage of waves with a relatively small H_s is overestimated
by the Weibull distribution. The data from the wave climate as determined
in paragraph gives a percentage of H_s < 0.50 m. of 10 %.
The Weibull distribution provides a percentage of 15 %.

The percentage of waves with a relatively large H_s is underestimated.
The data from the wave climate as determined in paragraph shows
that no H_s occurs above 2.00 m..
The Weibull distribution provides a percentage of 92 %.

It can be concluded that the Weibull distribution provides additional safety
for larger H_s. For lower H_s the risk exists that smaller H_{crit} are used.
This has to be taken into the evaluation of the method.

Determine downtime for particular wave direction

With the wave percentage known of the waves exceeding H_{cr} the downtime can be determined. By multiplying $P(H_s > H_{cr})$ with the probability of wave occurrence from a certain direction (table C-3), the downtime for the given direction is determined. The total of downtime of all wave directions results in the annual mean downtime of the chosen terminal.

For the container berth the annual downtime is presented in table 9-3. The total annual downtime is 4,33 %, which corresponds with 16 days.

From figure 9-9 can be seen that especially the waves from southern direction (160° to 180°) are the main cause of the total downtime. These directions cause 40 % of the total downtime.

Wave direction (°)	100 % wave penetration ?	$P(H_s > H_{cr})$	wave direction (% /year)	Downtime (% /year)
150°		0,05	3,09	0,15
160°		0,24	2,25	0,54
170°		0,71	1,13	0,81
180°	•	0,82	0,70	0,57
190°	•	0,95	0,42	0,40
200°	•	0,96	0,28	0,27
210°	•	0,96	0,28	0,27
220°	•	0,95	0,42	0,40
230°	•	0,94	0,42	0,40
240°	•	0,92	0,42	0,39
250°	•	0,45	0,28	0,13
260°		0,08	0,28	0,02
270°		0,00	0,42	0,00
Total				4,33

Table 9-3: Downtime calculation for container terminal, alternative B1

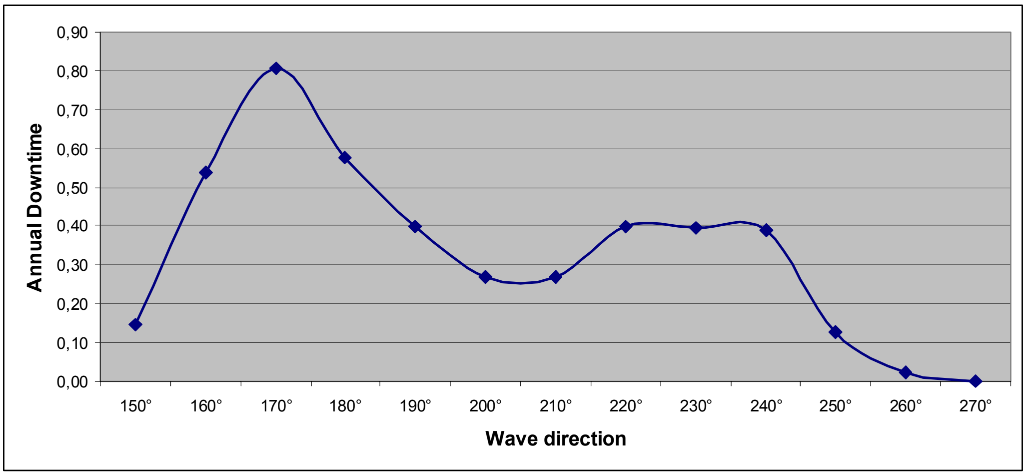


Figure 9-9: Downtime distribution for container terminal, alternative B1

For the dry bulk berth the annual downtime is presented in table 9-4.
The total annual downtime is 2,42 %, which corresponds with 9 days.

From figure 9-10 can be seen that especially the waves from north western direction (290° to 310°) are the main cause of the total downtime.
These directions cause 62 % of the total downtime.

Wave direction (°)	100 % wave penetration ?	P (H_s > H_{cr})	wave direction (% /year)	Downtime (% /year)
230°		0,00	0,42	0,00
240°		0,14	0,42	0,06
250°	•	0,49	0,28	0,14
260°	•	0,49	0,28	0,14
270°	•	0,61	0,42	0,26
280°	•	0,75	0,42	0,32
290°	•	0,81	0,56	0,45
300°	•	0,88	0,70	0,62
310°	•	0,64	0,70	0,44
320°		0,00	0,70	0,00
Total				2,42

Table 9-4: Downtime calculation for dry bulk terminal, alternative B1

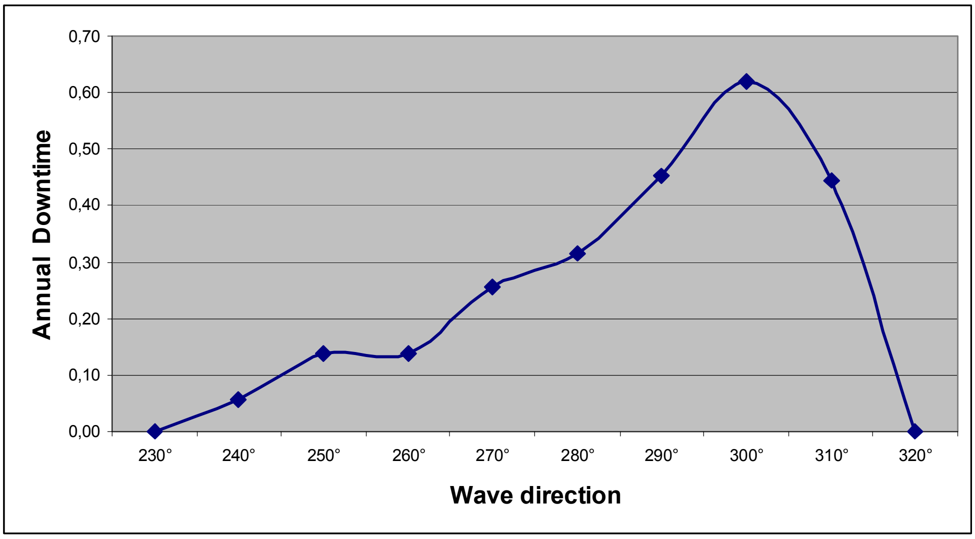


Figure 9-10: Downtime distribution for dry bulk terminal, alternative B1

9.2.3 Limitations of method to determine annual downtime

This method to determine the mean annually downtime of the new port tries to be as accurate as possible. It does however also has its limitations and assumptions.

- The whole berth is assumed to be out of order when effected by $H_s > H_{critical}$. In practise a part of the berth is still accessible.
- The distortion of wave energy caused by diffraction around both breakwaters is not included. The distortion influences the waves and increases the complexity of the method. For further research the use numerical modelling is an option.
- L_0 is determined for one wave height (increasing H_s causes smaller r/L and therefore higher K in general (also here restrictions)
- L_0 is based on mean wave length ($Ts = 5 \cdot \sqrt{Hs}$ → $3.6 \cdot \sqrt{Hs} < T_p < 8 \cdot \sqrt{Hs}$, $T_s \approx 0.9 T_p$), not L_{max}
- Distribution of wave heights is approached by a Weibull distribution. The 'real' distribution of wave heights is different.
- Friction of waves within the port is neglected.
- Reflection is checked in front of own berth. The effect of reflecting waves from one berth to the other is not checked.
- Tides and currents are not taken into account.

9.3 Optimized design breakwater

The design configuration of alternative B1 reaches an acceptable annual downtime of 16 days (4,33 %). By shifting the breakwaters the configuration can be probably be optimised to result in less downtime or a possible decrease of the costs.

In this paragraph additional variants of the configuration are presented. The configuration of breakwaters with the best combination of downtime and expected construction costs is selected. It is however important that vessels visiting the port can still manoeuvre within the turning basin and entrance of the selected variant. This is a design condition and will be checked.

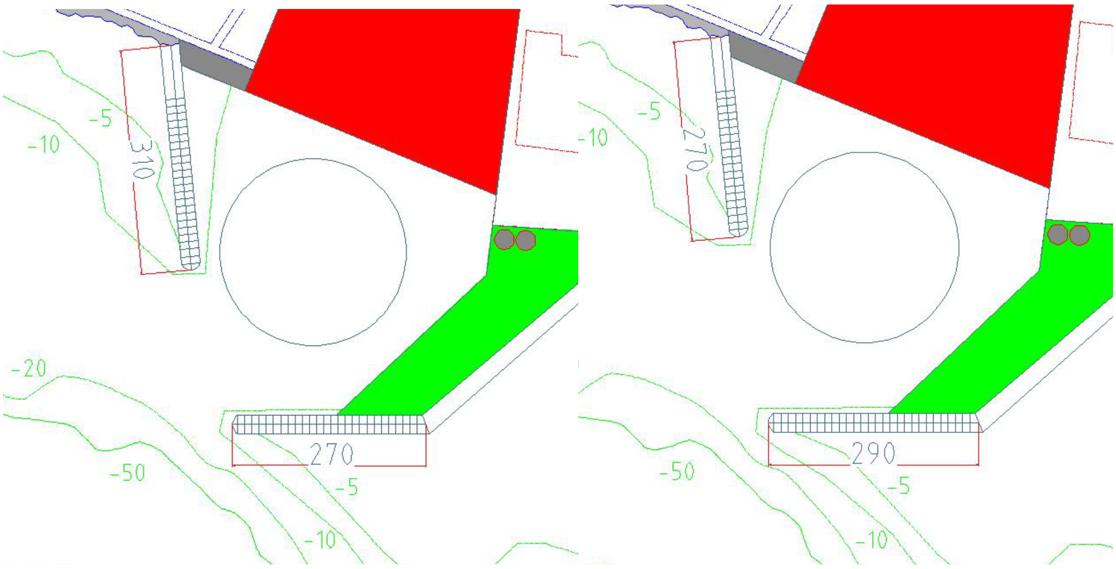


Figure 9-11: Breakwater configuration Alt. B1 and BW 2

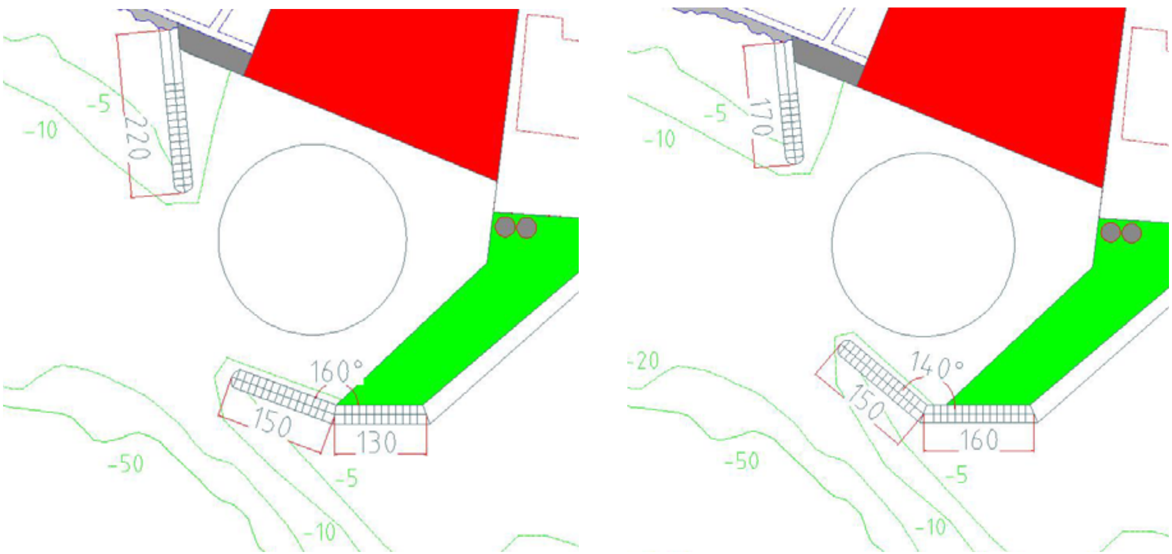


Figure 9-12: Breakwater configuration BW 3 and BW 4

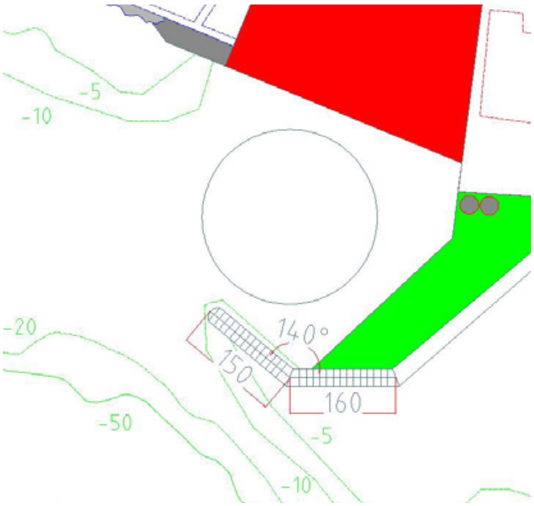


Figure 9-13:Breakwater configuration BW 5

9.3.1 Optimisation of downtime and construction costs

Variant BW 2 is an adaptation of the configuration of design B1.
Since B1 receives the most annual downtime from southern direction the southern breakwater is lengthened by 20 m. The western breakwater is shortened by 40 m. The annual downtime for both terminal decreases.

To decrease the annual downtime of the container terminal even more the influence of waves from southern direction for BW 3 has to be decreased. Increasing the breakwater's length in western direction results in a large increase of the volume. This because the depth of the encountered bottom level increases. The western breakwater is shortened even more.

For variant BW 4 the rotation of the port's entrance in northern direction is increased. This results in less annual downtime. Shifting the orientation of the entrance can result in a harder manoeuvrability for the arriving vessels.

Variant BW 5 is a conversion of variant BW 4. the difference is that the entire western breakwater is deleted from the design. This variant will show if the influence of waves from western direction is insignificant to the total downtime of the port.

As an indication of the reduction of annual downtime of the described variants the graph of figure 9-14 and 9-15 can be used.

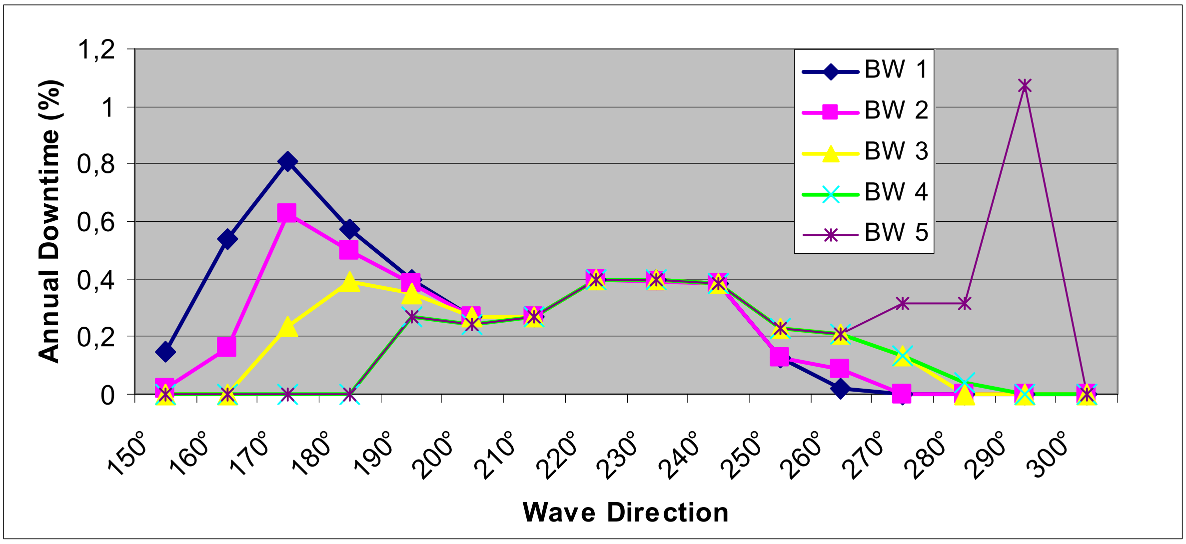


Figure 9-14: Downtime distribution container terminal for all alternatives

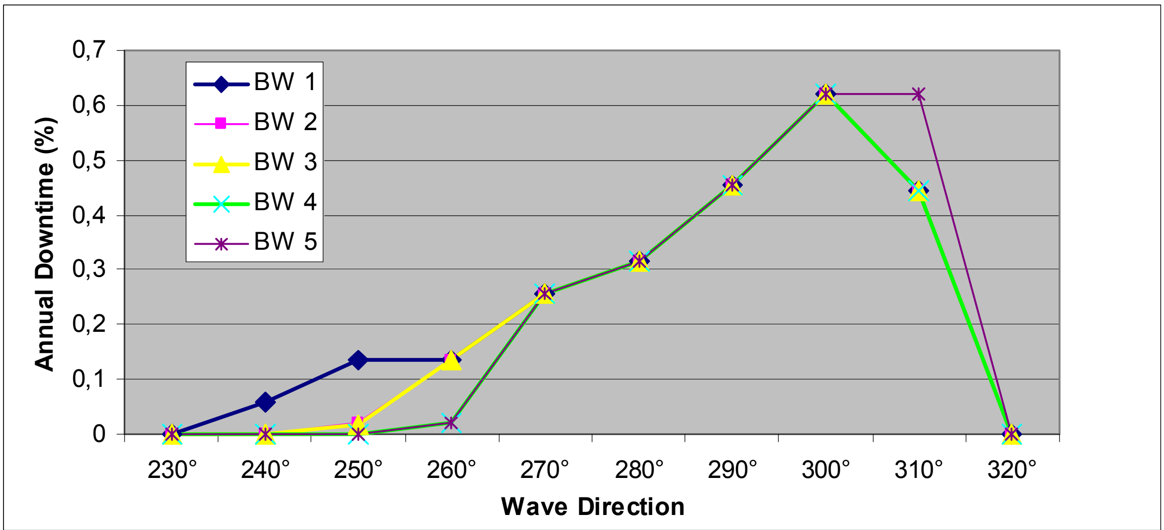


Figure 9-15: Downtime distribution dry bulk terminal for all alternatives

The total downtime of the variants is presented in table 9-5.
A detailed overview of downtime per wave direction is presented in Appendix E.

	T_d container (days)	T_d dry bulk (days)	L_{west} (m)	L_{south} (m)
Alt. B1	16	9	310	270
BW 2	13	8	270	290
BW 3	12	8	220	290
BW 4	9	8	170	310
BW 5	15	8	0	310

Table 9-5: Total downtime and breakwater length for all alternatives

To select the most optimal configuration an indication of the costs has to be generated. First the volume of the breakwaters is adapted for the varying level of sea bottom. For the design of the cross sections it was assumed that the total section of a particular breakwater has a constant depth. The volume of the cross section at the heads of the breakwater are substantially affected by the bottom level. Therefore the volume of the heads is adapted.
This is shown in table 9-6 and 9-7, where the section are divided per bottom level.

For an indication of the construction costs of the breakwater some general assumptions of the breakwater's cross sections are required.
For the top of the breakwater's crest a level of + 2.00 m. is assumed. The final crest level is determined in paragraph 9.4. The outer slope is 1:2 and the inner slope is 1:1.5.

The bottom level of the structure has a large influence on the volume used. For instance: in relation to a cross section of a bottom level of -4.00 m. MSL the volume per running meter increases with 31 % for a bottom level of -5.00 m. MSL.

To provide an approach for the determination of costs per variant it is assumed that a volume of 114 m^3/m costs $ 50.000. The costs for varying cross sections are altered linearly. The total costs are presented in table 9-8. It can be concluded that the configuration of BW 4 has the most optimal costs and downtime for Clifton Point.

	BW west		BW south		
	L_{west} – 5.00 MSL (m)	L_{west} – 7.00 MSL (m)	L_{south} – 4.00 MSL (m)	L_{south} – 5.00 MSL (m)	L_{south} – 7.00 MSL (m)
Alt. B1	270	40	200	50	20
BW 2	270	-	200	50	40
BW 3	220	-	180	110	-
BW 4	170	-	200	110	-
BW 5	-	-	200	110	-

Table 9-6: Distribution of breakwater sections in relation to construction depth

Bottom level	Cross section (m^3/m)	Cost per m^1 ($/m)
- 4.00 m. MSL	87.0	38.200
- 5.00 m. MSL	113.8	50.000
- 7.00 m. MSL	177.8	78.100

Table 9-7: Estimated costs per cross section

	Total volume (m^3/m)	Cost BW west ($ in million)	Cost BW south ($ in million)	Total costs ($ in million)
Alt. B1	64.500	16,6	11,7	28,3
BW 2	60.900	13,5	13,3	26,8
BW 3	53.200	11,0	12,4	23,4
BW 4	49.300	8,5	13,1	21,6
BW 5	29.900	-	13,1	13,1

Table 9-8: Estimated costs per breakwater configuration

By using the method above some important assumptions are done. Also some aspects are excluded. It is assumed that the costs of the cross sections are related linearly. Thereby the aspect of a varying thickness of the armour layers is excluded. Larger units of armour result in higher costs.

Also shifting the orientation of the port's entrance results in a varying area to be dredged. The amount of excavated soil is left out of the determination of costs. The eastern part of the southern breakwater is excluded from the costs. The volume of the eastern part is constant for all variants included.

From table 9-8 it can be concluded that variant BW 5 has the lowest construction costs. The difference with variant BW 4 is € 8.5 million. When the criterion of total downtime is evaluated variant BW is the best choice. In relation to variant BW 5 the difference is approximated by 6 days of downtime. This results in the costs of € 1.4 million per day of downtime.

To be able to compare both variants the annually costs of a day of downtime will be roughly determined. The breakwaters will be designed for a lifetime of 100 years. When the maintenance is approached by an annually 5 % of the construction costs, the benefits of selecting BW 5 per day of downtime is determined by:

$$\frac{\Delta(initial\_costs + total\_maintenance\_costs)}{days\_of\_annual\_downtime \bullet life\_time} = \frac{8.5 + 0.05*100*8.5}{6*100} = \text{€ } 85.000$$

The costs of a day of downtime for the port because of a missing financial input is expected to be larger than € 85.000 a day. BW 4 is therefore the best alternative, because its relatively small costs and related downtime.

For instance the missing berthing dues of the port can be roughly approximated by € 1000 a day per running meter of quay length.
When the port is utilised as determined in table 6-4 and 6-7, 270 m. of quay length is occupied on a daily base.
(utilisation container terminal → 0.57 of 380 m., utilisation dry bulk terminal → 0.20 of 280 m., → a total quay length of 270 m.)

The missing revenues therefore result in € 270.000, which is higher than the benefits of € 85.000.

9.3.2 Alternative with less reflective quay wall

By constructing the quay wall as a 'deck on piles' structure the height of the reflected waves will decrease. This is caused by the slope beneath the concrete deck. The reflection factor is determined by $K_R \approx 0.1\xi^2$.
As an example a slope of 1:4 results in a K_R of 0.26. For a deck on piles such a slope is relatively large. Combining a slope with the concrete deck of the terminal results in a quay structure which costs double or triple the indicated cost of a vertical quay wall. The decrease of downtime by selecting a 'deck on piles' structure is estimated on a 2 or 3 days per terminal.

The costs of the quay wall are estimated to increase by at least € 13 million. The alternative of a 'deck on piles' structure is therefore considered inadequate.

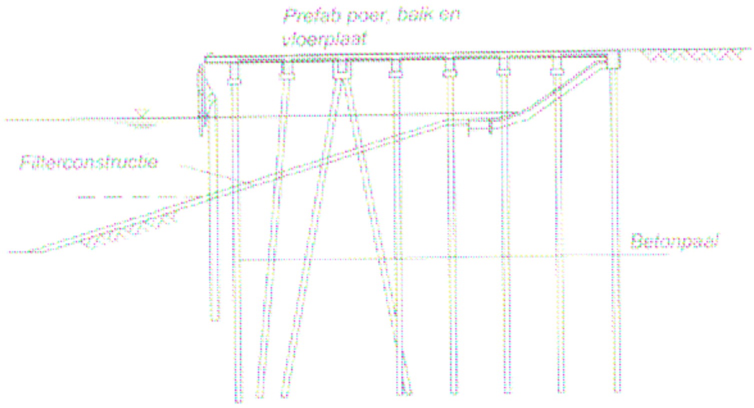


Figure 9-16: Example of deck on piles

9.3.3 Manoeuvrability of vessels in port configuration

In paragraph 9.3.1 the variant is selected which is used as configuration of the breakwaters. The variant has the lowest annual downtime and the construction costs in relation to the amount of downtime is low.

The final step before the particular variant is selected as final breakwater configuration is to check the manoeuvrability of the vessels within the port structure. Until this stage the manoeuvrability of the vessels was only guaranteed by the general design rules of paragraph 6.4.
In this paragraph a toolbox is used to perform a navigation simulation with the selected configuration.

The Toolbox is a designing tool which can be used for simulating the manoeuvring behaviour of vessels in port entrances, approach channels and turning basins. The Toolbox was designed by dr. ir. W. Veldhuyzen and has various possibilities.

Since the human behaviour of captains on a vessel visiting a port is very complex for programming into a modelling environment, the best solution for checking the manoeuvrability of a port is real time simulation. These simulations are mostly performed by full bridge simulators. These devices are relatively time consuming and expensive. In relation to the options above the Toolbox is a flexible alternative. The input of the simulations can be easily adapted.

This paragraph shows a single simulation. The operation of the model will be explained by presenting the result for the port of Clifton Point.

Bathymetry and weather conditions

First the depth lines have to be implemented to generate a modelling space. The depth lines are primarily used as restrictions for the draft of the vessels. The water depth is for example of no influence to the wave climate. The next step is to insert the conditions of wind, waves and currents for the duration of the simulation. For the simulation of Clifton Point the total influence of the conditions of the environment are orientated in south-eastern direction to create a modelling conditions which is the least favourable for the simulation.

The following conditions occur:

Wind
- Velocity: 7 m/s
- Direction: south east

Wave
- Height: 0.75 m.
- Period: 5 s.
- Direction: south east

Current
- Velocity: 0.75 kn.
- Direction: south east

For the conditions of the ocean current coordinates have to be selected. These coordinates are related to a certain velocity. During the simulation the currents are interpolated from these reference points. The coordinates and their velocity for the simulation are presented in figure 9-17.

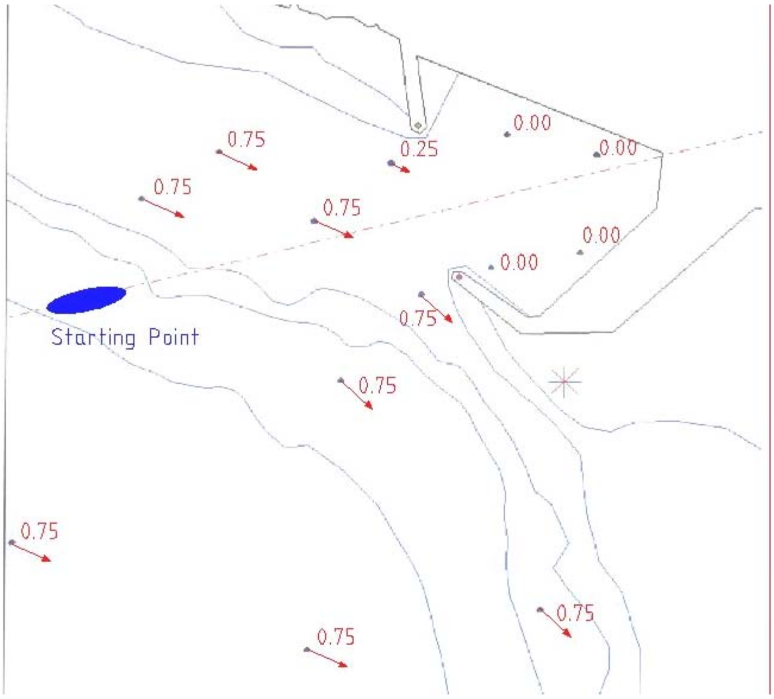


Figure 9-17: Bathymetry and ocean currents of Toolbox

The vessel which is most difficult to handle in the current layout is the granite bulk carrier, named CSL Argosy. The characteristics of this vessel can be found in table 4-5. The length of the vessel is 245 m. In paragraph 6.4.2 it was determined that the maximum draft of the vessel was 7.90 m.

The objective of the simulation is to navigate the CSL Argosy within the turning basin of the new port. The velocity of the vessel within the port has to be insignificant. Since the turning basin is not designed for the autonomously entering of vessels with a length of larger than 160 m., both of aggregate carriers require tugboat assistance. With the Toolbox it is possible to assign two tugs in front and two tugs behind a vessel.

The simulation performed with the toolbox is visualised in figure 9-18. The toolbox allows the designer to navigate in real time. The following instruments of the navigated vessel can be controlled:

- Rudder angle
- Power of ship engine
- Power of bow and stern thrusters (when present)
- Length of anchor chains
- Pulling force and direction of tugboats

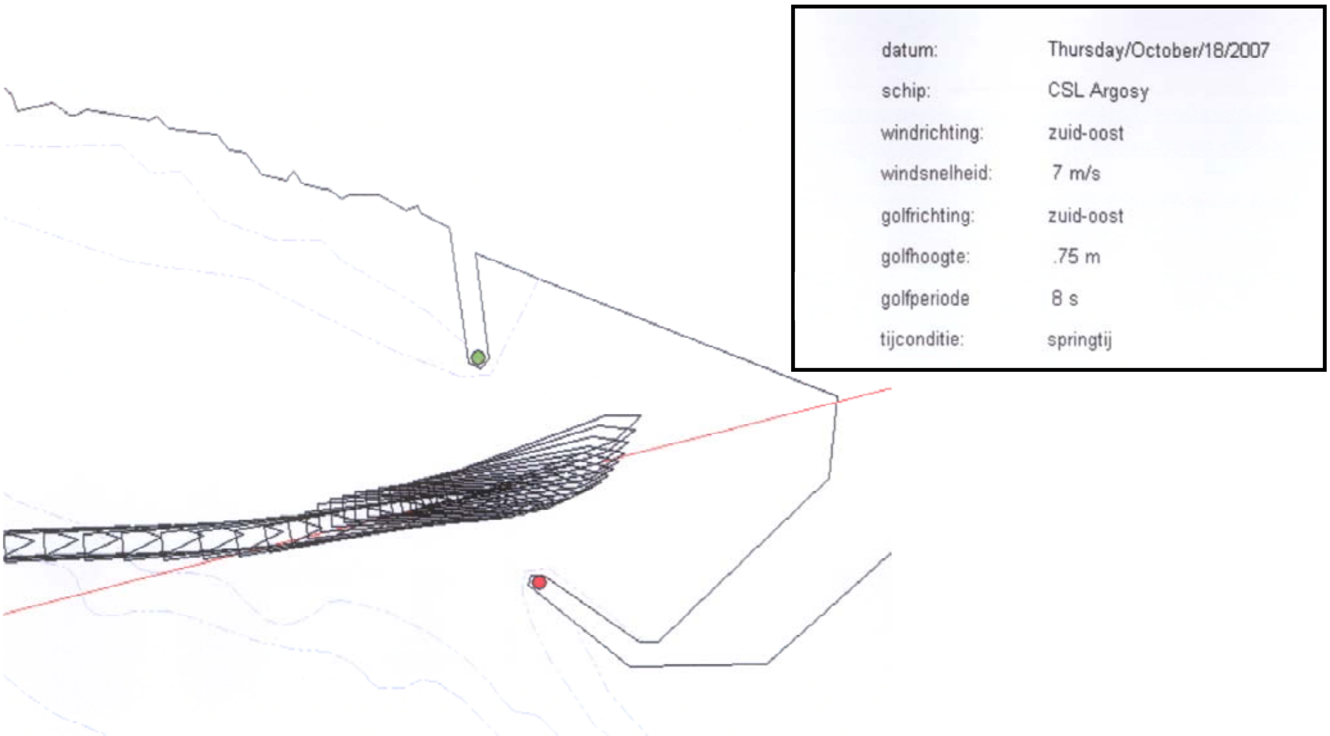


Figure 9-18: Toolbox simulation of CSL Argosy

In figure 9-18 it can be seen that the vessel with an initial velocity of 8 kn. is able to enter the turning basin under the most extreme operational conditions. When the vessel is navigated within the port the tugboats are able to manoeuvre the aggregate carrier to the dry bulk berth by pushing the vessel in the correct direction.

9.4 Structural design of breakwater

In the design of alternative B1 two breakwaters are located in front of the new harbour. In this paragraph the cross section of the western and southern breakwater structure are determined. Since various parameters influence the design, it is divided in two cross sections.
Both locations of the cross sections can be seen in figure 9-19.

Figure 9-19: Location of cross sections of breakwaters

The cross section of the eastern breakwater, which will be an integrated design with the quay of the dry bulk terminal, will be behind the scope of this thesis. It can be noted that the structure requires a smaller volume per running meter. This is caused by the smaller wave attack from the relatively shadow eastern bank.

Each cross section is located with distinctive parameters.
Among these parameters are the water depth and the wave climate.

Since the definitive configuration of the breakwaters is unknown the cross section of both breakwater are determined by using a constant bottom level. The breakwaters have a more or less fixed orientation.

The western breakwater will not be placed further in western direction; this to avoid the large increase of the water depth due to the relatively steep slope. To guarantee a navigatonable turning basin a transfer of the breakwater in eastern direction is also impossible.
For the mean level of the construction depth a value of -5.00 m. MSL is chosen.

Lengthen the southern breakwater in western direction is also limited by the relatively steep slope of the sea bottom. The mean construction depth of the core body is chosen on a level of -4.00 m. MSL. It can be expected that the head of the breakwater is located at a deeper construction level.

9.4.1 Western cross section

The water depth at the western cross section is assumed to be constant with a level of -5.00 MSL. As determined in paragraph 9.1 the breakwater will be designed for extreme conditions with a return period of 100 years. The extreme wave climate dictates a condition for a local wave height of 3.20 m. This in combination with water level elevation, including the storm surge.

With an increase of the slope angle, the wave setup will affect the total water level elevation. The wave set up is determined by:

$$\overline{\eta}_b = -1/16\gamma^2 d_b \quad \text{and} \quad \overline{\eta} = \overline{\eta}_b + \frac{d\overline{\eta}}{dx}\Delta x, \qquad \text{where} \quad \frac{d\overline{\eta}}{dx} = \left[\frac{1}{1+\frac{8}{3\gamma^2}}\right]\tan\beta$$

where: tan β = slope of shore (°)
γ = breaker parameter (-)
d_b = depth of breaking (m)

The waves with a wave height of 3.20 m. will break at a water depth of -5.00 m. MSL. The dη/dx for a 1:2 slope is 0.043 m/m^1. After 10 m. the wave setup will be 0.43 m. and is significant for the total setup.

The total water level elevation for storm with a return period of 100 years is thereby + 1.80 m. MSL.

Stone diameter of outer slope

For the material used to construct the breakwater several options arise. Every type of material has its own advantages and disadvantages.

First of all there is quarry stone, which is a material easy to use. The most important condition of using quarry stone is the availability of the stone. When the quarry is located at a relatively large distance the use of another material could be more feasible from an economic view.

Another option is the use of concrete blocks. They can be of various forms from simplistic cubes to shapes with a higher complexity Acropods, dolos, etc. These specialized forms all have their own design standards.
They each require a nearby construction site, where a sufficient number of moulds is present. For these moulds a balance has to be found between mould costs and turn around time.

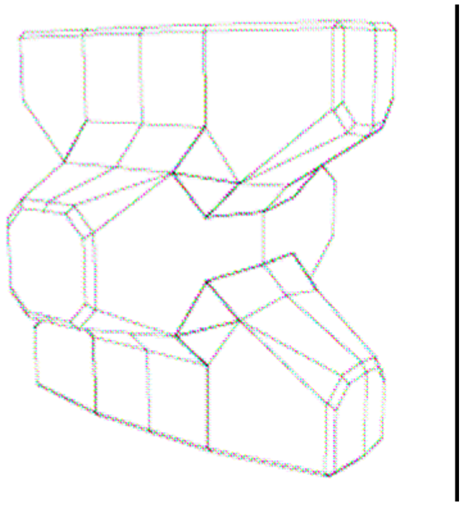


Figure 9-20: Acropod, armour protection

For the location of the new port there is no detailed information about the available material. It is expected that the foundation of limestone rock within the surroundings of the Power Plant provide sufficient material for the filling of the breakwaters. The limestone subsoil has to be dredged to generate a sufficient water depth in all conditions.

The material of which the breakwater is constructed are part of several stone layers, each with its own diameter; the core material and the top layer.
The core will be constructed with stones of a small diameter of D_{n50} = 230 mm. The top layer will have to resist the extreme waves as determined in paragraph 3.6. The larger blocks of the top layer protecting the breakwater are firstly chosen as concrete blocks.

The D_{n50} of the top layer is designed using the method of Van der Meer. This method is selected because of the conservative approach of the Hudson method.

$$\frac{H_s}{\Delta D_{n50}} = 6.2P^{0.18}\left(S/\sqrt{N}\right)^{0.2} 1/\sqrt{\xi_m} \qquad \text{for plunging waves}$$

$$\frac{H_s}{\Delta D_{n50}} = 1.0P^{-0.13}\left(S/\sqrt{N}\right)^{0.2} \xi_m{}^{P}\sqrt{\cot\alpha} \qquad \text{for surging waves}$$

where S = damage level
N = number of waves during extreme conditions
P = permeability coefficient
Δ = relative mass density
ξ = breaker parameter
D_{n50} = nominal stone diameter
H_s = significant wave height

The Van der Meer formulas have a variant for plunging and for surging waves. The variant is selected using the transition parameter ξ_{mcrit}. When the breaker parameter ξ is smaller than ξ_{mcrit} the waves are considered to be plunging, otherwise they are considered to be surging. The wave's breaker types are presented in figure 9-21.

$$\xi_{mcrit} = \left[6.2P^{0.31}\sqrt{\tan\alpha}\right]^{\frac{1}{P+0.5}}$$

For a slope of 1:2 the transition parameter is 3,54.

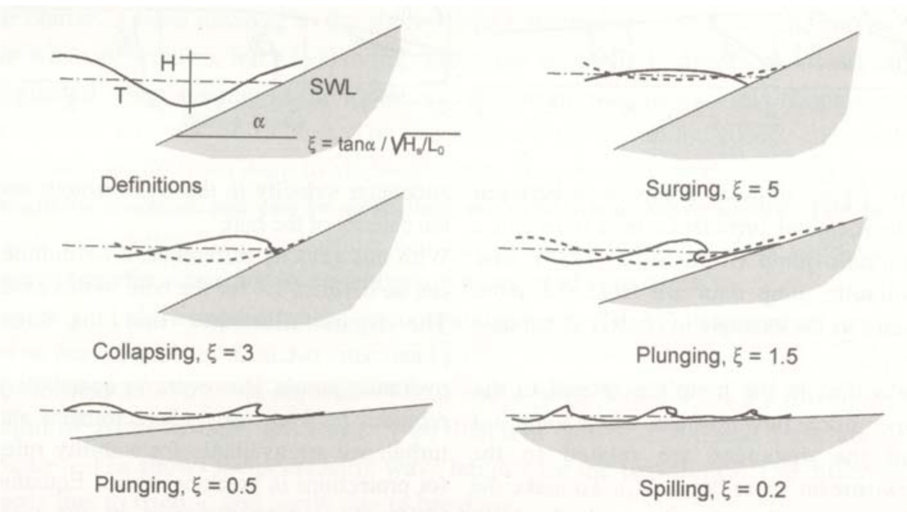


Figure 9-21: Breaker types

The breaker parameter is determined by $\xi = \frac{\tan\alpha}{\sqrt{H_{si} / L_0}}$.

Because the L_0 of the design wave is expected to be in the range of 140 to 200 m. (T_s from 9 s. to 11 s.), the breaker parameter of Irribarren is in the range of 3,16 to 3,78.

The waves arriving at the structure can therefore be either plunging or surging. Plunging waves are normative for the structure and result in a larger stone diameter.

The factor of 6.2 of VD Meer formula is dependent on the shape of the quarry stone. For shapes with an higher irregularity the factor increases. 6.2 is chosen for a standard situation.

For a slope of 1:2 the damage level for which total failure of the structure occurs is 8. Total failure of the breakwater by a design wave with a return period of 100 years is not allowed. The breakwater will therefore be designed on a damage level of S=2, causing relative minor damage which doesn't require repair during the structure's lifecycle.

A structure consisting of two layers has a permeability coefficient of 0.5.
The number of waves during a design storm of 6 hours is around 2000 waves.
The mass density of concrete is 2650 kg/m^3 (ρ_m).

The relative density is determined as $\Delta = \frac{\rho_m - \rho_w}{\rho_w}$ with ρ_w as water density in the Caribbean (ρ_w = 1025 kg/m^3). Therefore the relatively density is 1.58 [-].
An extreme wave with a chance of 1:100 year results in D_{n50} = 1340 mm.

Concrete cubes with D_{n50} = 1340 mm. have a weight of 5.5 ton (ρ_S x D_{n50}^3, where ρ_S = 2650 kg/m^3).

By adjusting the slope to 1:1,5 the waves are considered as surging waves, reducing the diameter to 700 mm. This is only valid for L_0 > 140 m.
Since the spectrum of waves of H_s = 3.20 m. is unknown, it is considered unsafe to design a stone diameter on surging waves. Also the angle of tanα= 1,5 is minimal. Therefore the slope of 1:2 will be maintained.

General shape of cross section

The height of the crest level is determined by the freeboard of the breakwater design. Although the crest can have a certain height above MSL, wave energy can still be transmitted through and over the breakwater.
Therefore the transmission coefficient has to be determined for the chosen cross section.

For the design it is decided that during operational port conditions no wave energy may be transmitted through or over the cross section. During storm conditions waves are allowed to be transmitted and are free to enter the port.

For the operational conditions the transmission coefficient K_T has to be 0.
K_T is determined by:

$$K_T = a\frac{R_c}{H_{si}} + b$$

where $K_T = \frac{H_{si}}{H_t}$, $a = -0.4$ and $b = c\left(\frac{B}{H_{si}}\right)^{-0.31}\left(1 - e^{-0.5\zeta}\right)$

R_c= crest freeboard in relation to SWL	(m)
H_{si} = incoming significant wave height	(m)
H_t = transmitted significant wave height	(m)
B= crest width	(m)
ξ = breaker parameter of Irribarren	(-)
c = 0.64 for permeable structures and 0.80 for impermeable structures	(-)

The outer slope of the cross section is 1:2. A small slope will cause a smaller volume of the cross section.

The inner slope of the breakwater will not be encountered by high wave attack and receives the minimum slope of 1:1.5. The stability of the inner slope will be checked in a later stage.

The breakwater is permeable, so c= 0.64.The width of the structure has a minimum dimension of 3 block sizes. A width of 4.00 m. is chosen.

When the maximum wave height under operational conditions is chosen to be 1.00 m. the freeboard has a value of 0.9 m. The H_{Smax} of 1.00 m. is selected, since a larger wave height would still cause downtime to the port through the port entrance, even though the breakwaters would prevent the relevant wave.

When spring tide is included, the top of the crest is situated at a level of + 1.25 m. MSL. This results in a relatively small cross section. With the given angles of the slope the volume of the cross section per running meter is 93 m^3/m.

When storm conditions occur, the waves are transmitted by the breakwater structure. Because the vessels have sufficient time to evacuate for storm conditions, transmitted waves caused by extreme conditions are allowed in the current port design.

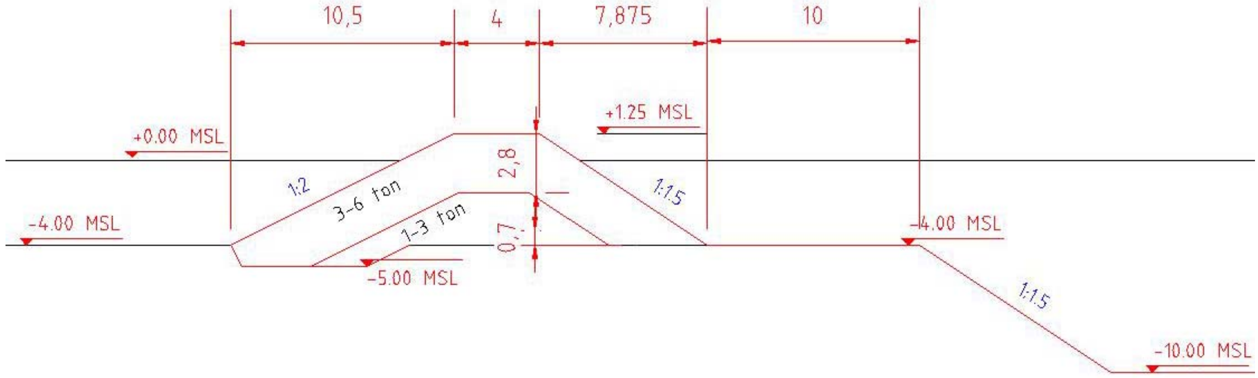


Figure 9-22: Cross section A-A' of western breakwater

Filter layer and filling material of core structure

The amounts of limestone which are dredged from the project site are used to generate sufficient fill for the filter layers of the breakwater. The amount of material dredged is sufficient. Research has to prove if the quality of the material also passes the restrictions.

The combination of a limestone core protected by the layer of concrete blocks is limited to several design conditions. These design conditions are based on experiments.

The water in front of the structure is flowing through the granular medium, causing loads on the granular material. To prevent the grains moving from the core out of the structure, the space between the grains of the filter layer have to be small enough to withhold the core grains from passing the filter. This is called the stability of the filter, presented by the limitation of $\frac{d_{15F}}{d_{85B}} < 3$ and $\frac{d_{50F}}{d_{50B}} < 10$.

It has to be noted that the condition for stability between the different layers has a higher safety for the first filter layer of the structure. For filter layers behind the first layer a more flexible condition of $D_{15F}/D_{85B}= 5$ is used. When only armour units and a filter layer are applied, the condition of $D_{15F}/D_{85B}= 5$ is unused.

In the case the filter is too closely packed, pressure is building up against the grains. This load can result in the failure of the structure.
Therefore a condition of permeability is applied to the filter structure.

This condition is presented by $\frac{d_{15F}}{d_{15B}} > 5$.

Also the stability of the layer on its own has to be guaranteed. For this reason a condition is used for the internal stability. The limitation of internal stability is presented by $\frac{d_{60}}{d_{10}} < 10$.

Until now only the nominal diameter of the grains was used. For further use new characteristics of the grain layers are required. The grading of the layers is represented by the diameters D_{15} and D_{85}, where $D_{\%}$ presents the percentage of grains with a smaller diameter.

The diameter of both filter grains are assumed to have a common grading distribution. Quarry stone generally has a grading
in a range of 1.5 $<D_{85}/D_{15}<$ 2.5.
For blocks in the range of 100 to 1000 kg a ratio of D_{85}/D_{15}= 2 is given.
This results in the following table:

	D_{85} (mm)	D_{n50} (mm)	D_{15} (mm)
Blocks top layer	1340	1340	1340
Filter layer (D_F)	480	360	240

Table 9-9: Composition of granular filters, cross section A-A'

The conditions of the filter structure are presented below.

Filter check		Armour layer (d_F) vs. filter layer (d_B)
Stability between layers	$\frac{d_{15F}}{d_{85B}} < \alpha$	2.79
	$\frac{d_{50F}}{d_{50B}} < 10$	3.72
Permeability	$\frac{d_{15F}}{d_{15B}} > 5$	5.58

Table 9-10: Filter check for each interacting layer, cross section A-A'
α = 3 for outer layer, α = 5 for inner layer

The material used for the structure has to comply to the condition of internal stability. This can only be checked when the actual material is available.

Outer slope

As a rule of thumb the top layer needs to be applied to the level of one H_s below MSL. In the case of this design the armour layer will reach the bottom level at the sea side of the structure at – 5.00 m. MSL.
It is therefore not possible to apply smaller sized stones at the outer slope.

To provide lower support for the armour layer a toe berm will be required. For the toe protection the following formula is used:

$$\frac{H_s}{\Delta D_{n50}} = \left(0.24 \frac{h_t}{D_{n50}} + 1.6\right) * N_{od}^{0.15},$$

for the validity range of : $0.4 < h_t / h, < 0.9$ and $3 < h_t / D_{n50} < 25$,

where N_{od} = damage level (0.5 = start of damage, 1.0 = acceptable, 4.0 = failure)

The nominal diameter of the toe protection results in 650 mm. for a damage level of 1.0. The toe protection is construction as a two layer structure with a slope of 1:2.

The toe protection is tested with the method of Van der Meer for longshore transport. The longshore transport is expressed in transported stones per wave (S(x)). The maximum transport occurs when waves arrive with a wave angle from 15 to 35 degrees.

For H0*Top > 105, otherwise S(x)= 0

$$S(x) = 0.00005\left(H_0 T_{op} - 105\right)^2$$

$$H_0 = \frac{H_s}{\Delta D_{n50}} \qquad T_{op} = T_p \sqrt{\frac{g}{D_{n50}}}$$

The critical diameter of the toe protection where longshore transport initiates is a Dn50 critical = 790 mm.

Inner slope

When waves are passing the crest of the breakwater damage can occur in relation to the discharges overtopping the structure.
From research in the past [Van der Meer 1993] a discharge with a maximum of around 0.100 $m^3/s/m$ will not cause damage to the inner slope.

The discharge of waves overtopping the structure is determined by using the TAW (2002a) method. This method is formulated as:

$$\frac{q}{\sqrt{gH_{m0}^{\ 3}}} = \frac{A}{\sqrt{\tan\alpha}}\gamma_b \xi_{m0} \exp\left(-B\frac{R_c}{H_{m0}}\frac{1}{\xi_{m0}\gamma_b\gamma_f\gamma_\beta}\right)$$

with a maximum of:

$$\frac{q}{\sqrt{gH_{m0}^{\ 3}}} = C\exp\left(-D\frac{R_c}{H_{m0}}\frac{1}{\gamma_f\gamma_\beta}\right)$$

where:	q = overtopping discharge	($m^3/s/m$)
	γ_β = reduction factor angle of wave attack, 1- 0.0022α ≈ 1	(-)
	γ_f = reduction factor of slope roughness, 0,47 for 2 layer of rock	(-)
	γ_b = reduction factor of berm, no berm → 1	(-)
	H_{m0} = spectral wave height	(m)
	ξ_{m0} = breaker parameter	(-)

The coefficients which include a safety margin for deterministic use are:

A = 0.067, B = 4.30, C = 0.20 D = 2.30

The crest element will almost stop the waves from overtopping the structure. The overtopping discharge is determined on 0.010 $m^3/s/m$. This won't cause damage at the inner slope.

After completion the cross section of the western breakwater is presented in the figure 9-19.

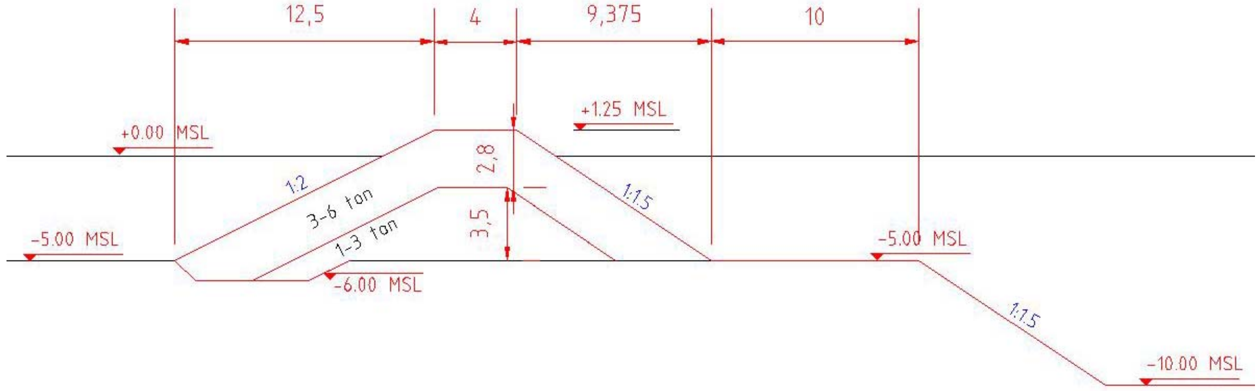


Figure 9-23: Cross section B-B' of eastern breakwater

Type of blocks	**D_{n50} (mm)**	**Volume of cross section per m (m^3/m)**
Armour blocks	1340	69.2
Filter layer	330	25.1
Toe protection	500	7.2
Total volume		101.5

Table 9-11: Volume per running meter of all layer types, western breakwater

Head of breakwater

The head of the breakwater is relatively vulnerable since, owning to the curvature, the armour units are less supported and less interlocking. In general damage occurs at the inner quadrants of the head.
To increase the strength the head is often reinforced by using larger armour units, increasing the unit's density or a reduction of the slope at the head.

For this design it is chosen to increase the size of the armour units.
Hudson recommends a K_D value of 1.6 (armour stone, slope 1:2), resulting in

$$\frac{H_s}{\Delta D_{n50}} \leq (K_D \cot\alpha)^{1/3} = 1.47 \qquad \text{(Rock Manual)}$$

Since the bottom level in front of both heads is -12.50 m. MSL, the stone diameter of the blocks used have to be designed for a H_s of 7.25 m.
This design wave is almost equal to the maximum deep water wave (H_s = 7.50 m.).
Following from the formula above, the stone diameter results in 3120 mm.

These blocks are very large. For Tetrapods a K_D of 4.5 is recommended:

$$\frac{H_s}{\Delta D_{n50}} \leq (K_D \cot\alpha)^{1/3} = 2.08$$

For the heads of the breakwater tetrapods with D_n= 2130 mm. are selected on a slope of 1:1.5.

The layer of these larger armour units will be constructed in the yellow indicated area of figure 9-24.

Since the nominal diameter of the Acropods increases at the breakwater's head the filter layer beneath the armour layer requires a new filter check.
This check is performed in table 9-13. It can be concluded that besides the Acropods the filter layer also increases in nominal diameter.

	D_{85} (mm)	D_{n50} (mm)	D_{15} (mm)
Tetrapods	2130	2130	2130
Filter layer (D_F)	750	560	375

Table 9-12: Composition of granular filters, cross section head

The conditions of the filter structure are presented below.

Filter check		Armour layer (d_F) vs. filter layer (d_B)
Stability between layers	$\frac{d_{15F}}{d_{85B}} < \alpha$	2.84
	$\frac{d_{50F}}{d_{50B}} < 10$	3.80
Permeability	$\frac{d_{15F}}{d_{15B}} > 5$	5.68

Table 9-13: Filter check for each interacting layer, cross section head
α = 3 for outer layer, α = 5 for inner layer

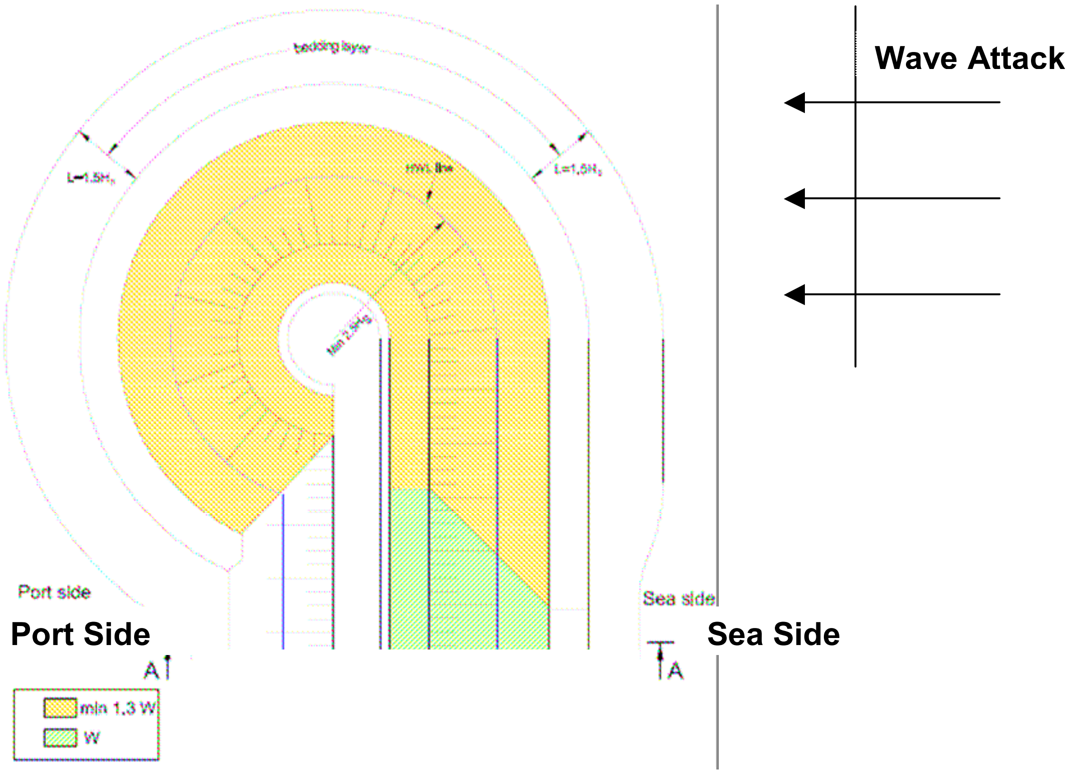


Figure 9-24: Composition of top layers as located at the breakwater's head

9.4.2 Southern cross section

The design of the southern breakwater is not identical to the design of the western cross section. The most notable feature is a lower bottom level. For the design of the southern breakwater a mean bottom level of -4.00 MSL is assumed.

For the armour layer of the outer slope the wave height of 3.20 m. is chosen as design wave. This results in the same nominal diameter of 1340 mm.

All layers of the structure are identical to the western breakwater.
The cross section of the southern breakwater with it's corresponding volumes of layers per running meter is shown in figure 9-25.

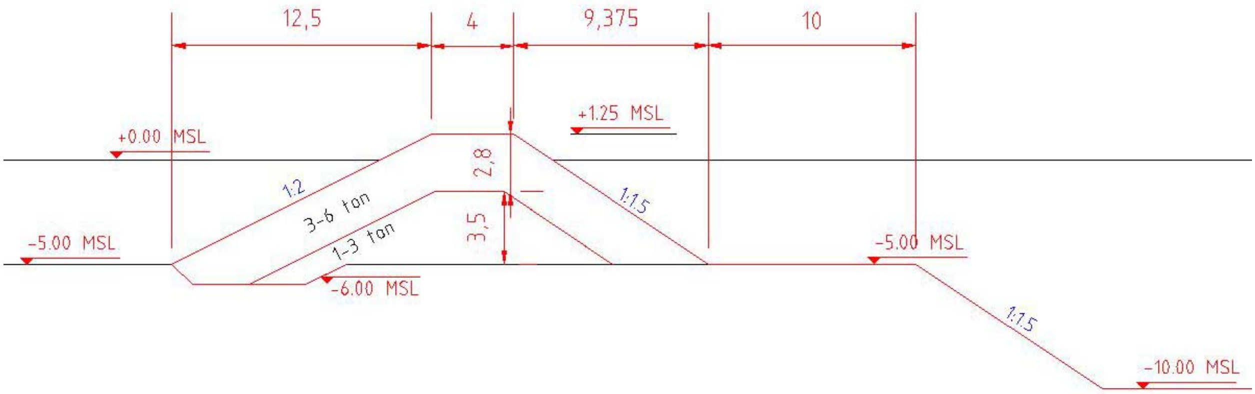


Figure 9-25: Cross section B-B' of southern breakwater

Type of blocks	D_{n50} (mm)	Volume of cross section per m (m^3/m)
Armour blocks	1340	93.4
Filter layer	330	40.7
Toe protection	500	7.2
Total volume		141.3

Table 9-14: Volume per running meter of all layer types, southern breakwater

10 Final layout

In this chapter the final design layout is presented.
Besides some general design notes about transportation routes, some important considered are discussed. In two separate paragraphs the influence of applying different throughput scenarios and the opportunities of expanding the port after 2035 are presented.

10.1 Final layout

With the chosen breakwater configuration of the layout of the new port at Clifton Point will change. The total layout of the new port is presented in detail in figure 10-1. The shown layout is for the expected forecast in the end phase of 2035. The medium scenario is used, which is determined in chapter 5.

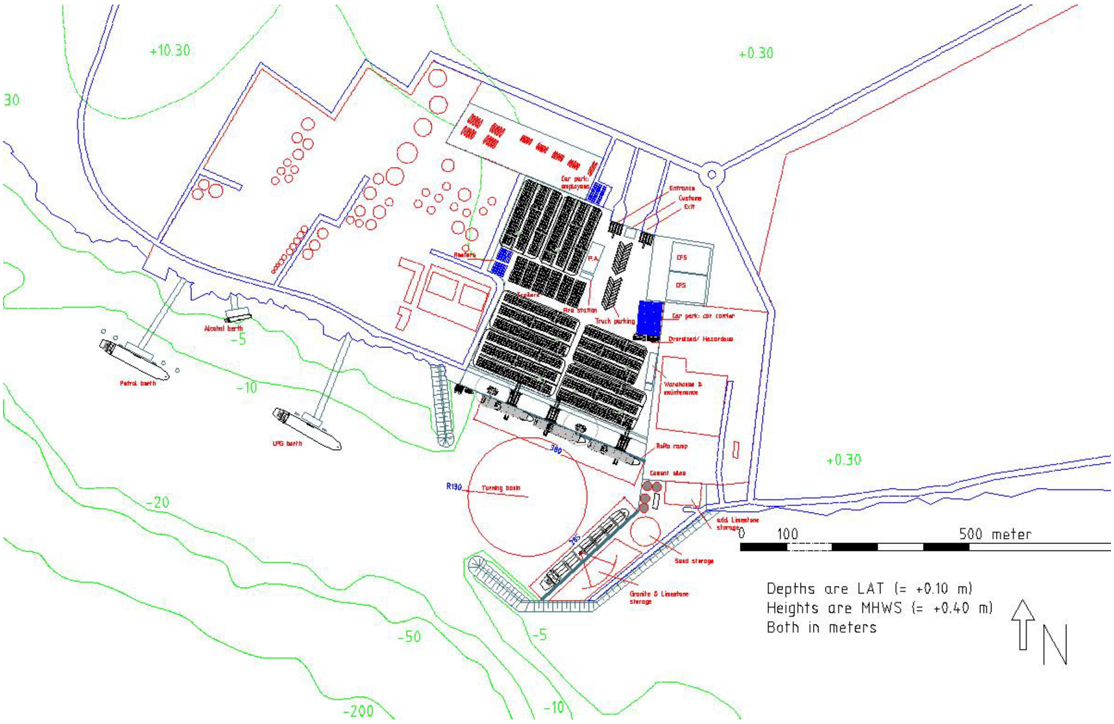


Figure 10-1: Final layout of port design, Clifton Point

The distribution of the terminal surface area is an approach of the expected situation. For a further design phase it is recommended to determine the optimal configuration for both terminals.

Besides the configuration of the terminal which includes the shore based infrastructure of traffic lanes etc. the connection with the infrastructure outside of the port area also requires attention. In the current situation a road is located along the coast at the southern side of the power plant.
A new road will have to connect the southern ring way of the island.

A larger version of the final layout is presented in Appendix F.

10.2 Influence of scenario of throughput on design

As mentioned for this design the forecast with a medium scenario was selected. The consequences of a deviation of the throughput are determined in the following text. When the actual throughput runs according a lower scenario, the effects are of minor influence.
Since the surface area of the terminals is expanded in 3 phases, a fall down of expected throughput can be absorbed easily by adapting the masterplan.

This can be done by expanding the terminal for a smaller surface or delay the start of the following phase.

A lower scenario has no effect on the total required quay length.
For the container terminal an expected low throughput of 160.000 TEU annually in 2035 still requires a terminal with 3 container berths.
Since the length of the quay for dry bulk cargo is determined by a single granite carrier, a decrease of throughput is not affecting the quay length.

When the throughput of the new port is based on the high scenario the port will prove to be insufficient for its current design. To provide capacity for a throughput of 302.000 TEU an additional berth is required.
The utilisation of the berth is increased to 0,76, resulting in a mean waiting time of 59 % of the service time. This is unacceptable. When this scenario occurs, expansion of the port is required before 2035.

To surface area of both terminals also needs to be increased. For the situation at the end of 2035 a terminal surface of the container terminal of 14,5 ha is required. This is an increase of almost 4 ha. At all events the fixed location of the CFS facility prevents an efficient expansion of the terminal area for such a large surface area. Additional research has to show if a relocation of the CFS of the current design is required.

For a larger throughput of dry bulk the terminal is proven to be inflexible. The area is determined by the given throughput. The fixed position of the breakwater prevents the terminal from expanding in eastern direction. The only possibility is acquiring land from the Commonwealth Brewery.

For the liquid bulk facilities no additional improvements are required. This because the limit of the capacity is not reached by far by the current forecast. For expansion sufficient space is available in northern direction.

10.3 Expansion of the port beyond 2035

The objective of the report was to design a feasible port structure for a forecast up to the year 2035. After 2035 the port is considered to be able to expand its port activities. In this paragraph the options of expanding the port are discussed.

When the expected throughput of the port increases the port capacity will have to grow in a parallel way. To increase the port's capacity the following options arise:

- Increase of cargo handling efficiency
- Increase of demand of usable terminal area

An increase of the cargo handling efficiency is assumed to require no additional space for terminal operations. The port is designed by current technologies. Future developments like a more efficient storage of empty containers has to be watched. This developments can also have an influence on the final design of 2035. Future developments could cause to decrease the demand of required surface area for the related terminals.
It is however important that without an increase of crane efficiency the quay capacity becomes insufficient. This will be discussed in the following subsection.

The port's capacity can also be increased by expansion of the terminal areas. When the throughputs of the different terminals increases it will also be required to construct additional quay walls for berthing the vessels. It is expected that for a throughput of 275.000 TEU an additional berth is required. For a medium scenario of container growth this throughput is expected in 2047, assuming that an annual growth rate of 1.9 % remains constant.

When the length of vessels arriving at the dry bulk terminal remains constant, the length of the dry bulk berth will be adequate for a relatively long period. The current forecast present a mean waiting time of 3 % of the service time, where 20 % is allowed.

Areas in the vicinity of the port of Clifton Point which are optional for expansion of port activities are presented in figure 10-2.

The area of E1 offers very limited opportunities. The area is positioned besides the Clifton Heritage park. It is expected that the amount of preventing measures against environmental harm is relatively large. A quay in front of the coast line is obstructed by the presence of the petrol berth. This berth will have to be relocated. Also the space for locating breakwaters is limited.

The area of E2 located at the eastern side of the current design is a better option for future expansion. Since the land behind the coastline is currently used by tourist accommodation the port expansion has to be located on new land reclamation. The presence of marine life on the shallow banks which are part of the southern side of New Providence can be damaged. This has to be checked by an environmental study.

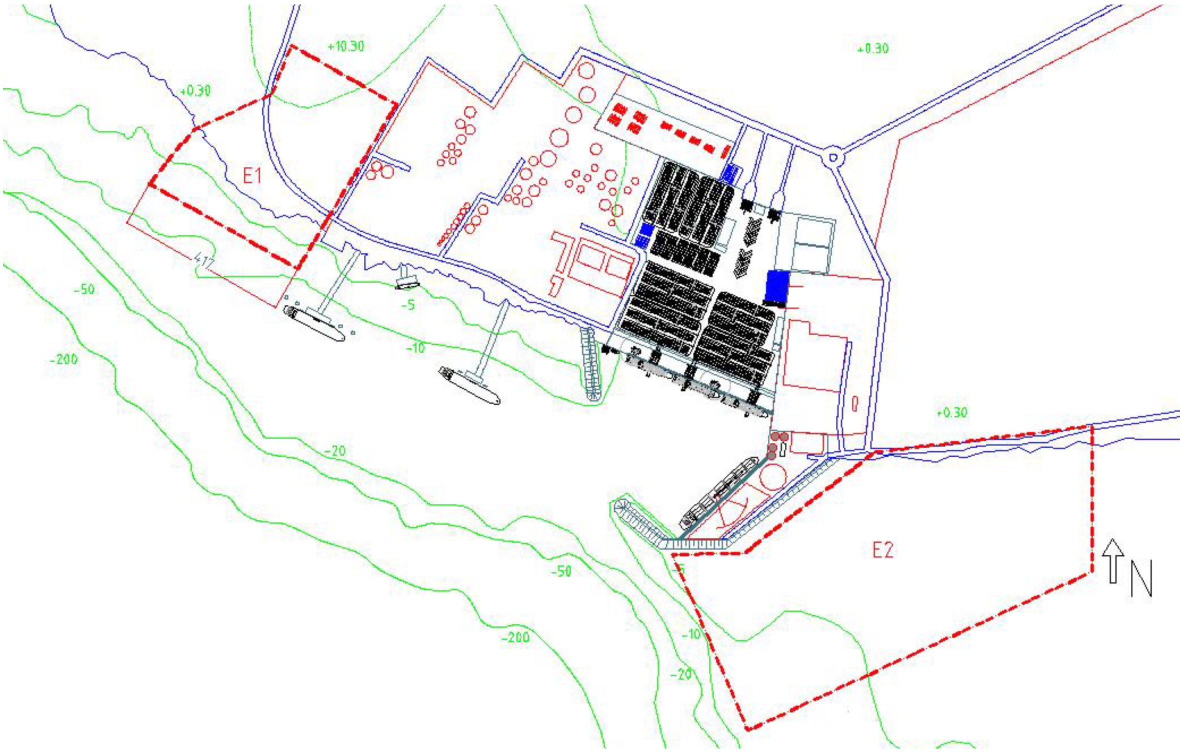


Figure 10-2: Optional expansion areas in vicinity of final design

11 Conclusions and recommendations

In this final chapter of the study of the port relocation at Clifton Point a final judgement will be provided about the feasibility of the project. Also recommendations required for future continuation of the project are given.

11.1 Conclusion

The final design layout provides a port structure which is technically feasible. For the vessels visiting the new port the capacity provided by the layout of berths is acceptable. The mean waiting times are below limits.

The configuration of breakwaters results in a port basin which is manoeuvrable by all vessels of the expected sizes. The occurring downtime due to wave disturbance of 9 days per year is acceptable.

The total construction costs are $ 166.9 million. These costs are excluding the costs required for the purchase of terminal equipment, like cranes and other cargo handling vehicles.

11.2 Recommendations

To realize the project some additional studies are recommended. Some assumptions which were made during this feasibility study may change in the future due to new developments. Therefore the effect of changing conditions on the final design has to be reviewed. The following actions are recommended:

- Detailed information about the soil conditions is required. It is currently assumed that all layers exist of limestone rock.
  The condition has a large influence on the dredging method and costs.

- For the surface area of the terminals only the area for the individual components are determined. A detailed layout is not yet available.
  The layout of all components like stacking areas, bulk piles and transferring areas requires optimisation.

- During the breakwater's design conditions it is expected that no damage occurs. It has to be investigated if decreasing the damage level is optimal in relation to the required maintenance costs. Nevertheless the safety of the structure as a whole has to be guaranteed at all times.

- To determine the profitability of the port a cost benefit analysis is required. This economic analys shows if the investments necessary for the new port are worthwhile .

- For this feasibility study one of the design requirements was to relocate the cargo types as determined in table 4-1. With the dominating presence of the aggregate carriers in the new port, the option of restructuring the current dry bulk facilities at Arawak Cay can be proven to be worthwhile. The effect of leaving out the aggregate carriers on the port of Clifton Point has to be investigated. It is expected that the quay length of the required dry bulk decreases. Also a smaller surface area for the dry bulk terminal is necessary. A future study has to prove if the expected decrease in costs in relation to the remaining presence of the terminal at Arawak Cay is satisfying. Also the feasibility of Arawak Cay in relation to the expanding terminal has to be investigated.

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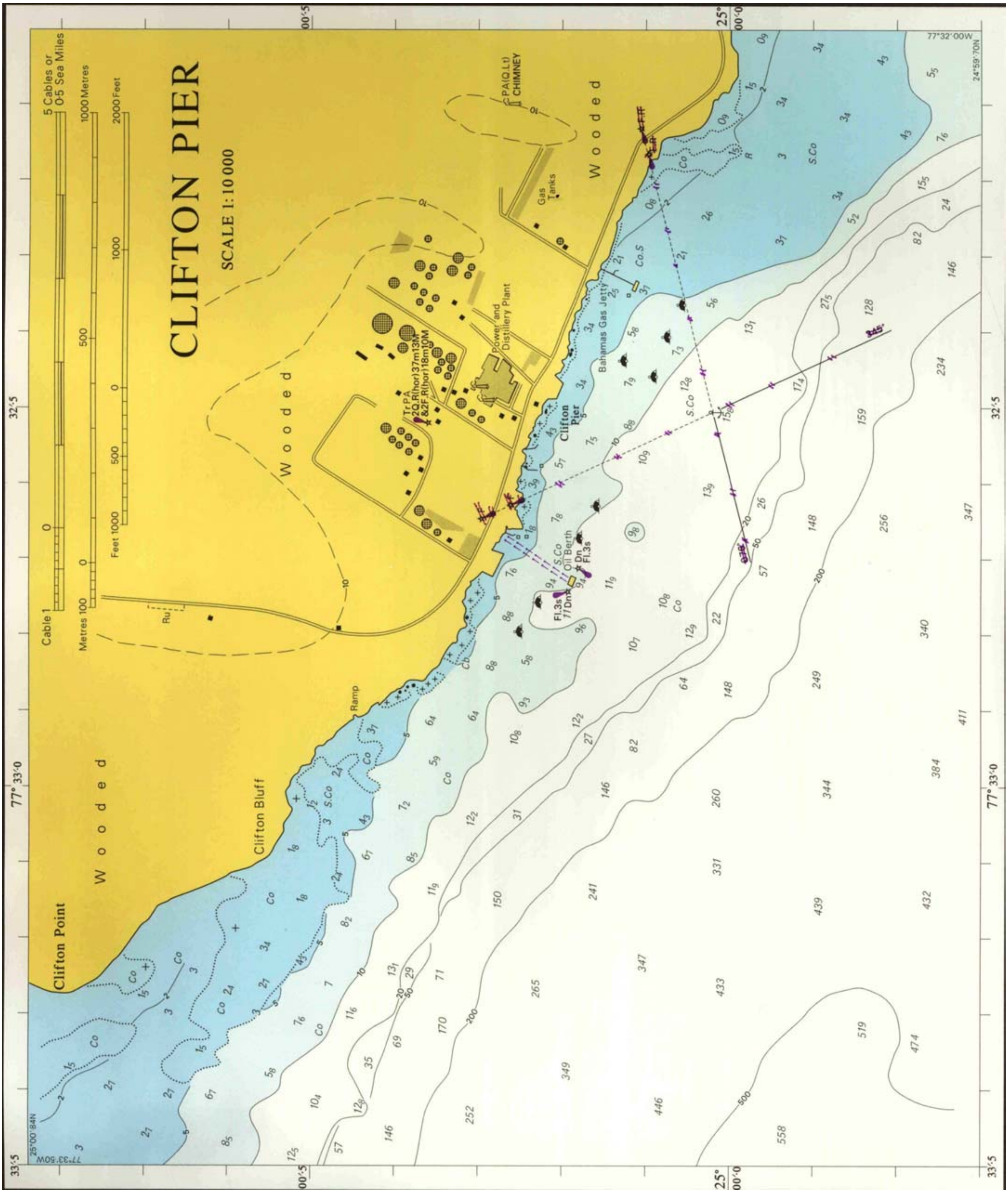
Appendix A: Bathymetry of project location



Appendix B: Data of Wind velocity versus. Wind direction

Wind speed (m/s)		wind direction																Total
Lower	Upper	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	
0,00	5,00	1,07	1,42	2,54	3,86	6,20	7,21	5,68	4,67	2,90	2,21	1,72	1,03	0,80	0,70	0,75	0,77	43,53
5,00	10,00	1,39	2,55	4,92	8,05	10,47	8,34	4,11	2,67	2,23	1,53	1,42	0,85	0,70	0,78	0,77	1,13	51,91
10,00	15,00	0,22	0,38	0,95	0,93	0,41	0,10	0,03	0,07	0,13	0,11	0,11	0,10	0,09	0,25	0,19	0,22	4,29
15,00	20,00	0,03	0,02	0,02	0,01	-	-	-	-	0,01	0,01	0,02	-	0,01	0,01	0,01	-	0,15
20,00	25,00	-	-	-	-	-	-	-	-	-	0,01	-	0,02	-	-	-	0,01	0,04
25,00	30,00	0,01	0,01	-	0,01	-	-	-	-	-	-	-	-	-	-	-	0,01	0,04
30,00	35,00	-	-	-	-	-	-	-	-	-	-	0,01	0,01	-	-	-	-	0,02
35,00	40,00	-	-	-	-	-	-	-	-	-	-	-	-	0,01	0,01	-	0,01	0,03
Total		2,72	4,38	8,43	12,86	17,08	15,65	9,82	7,41	5,27	3,87	3,28	2,01	1,61	1,75	1,72	2,15	100,00

Table B-1: Wind climate at distance of approximately 25 km of Nassau in north-western direction

Appendix C: Data of Wave height versus. Wave direction

Hs (m) Lower	Hs (m) Upper	wave direction (Deg) -15 to -5	-5 to 5	5 to 15	15 to 25	25 to 35	35 to 45	45 to 55	55 to 65	65 to 75	75 to 85	85 to 95	95 to 105	105 to 115	115 to 125	125 to 135	135 to 145	145 to 155	155 to 165
<	,50	0,18	0,21	0,25	0,63	1,37	2,92	3,59	2,60	1,62	1,30	1,58	1,86	1,62	1,02	0,53	0,39	0,21	0,25
,50	1,00	0,21	0,21	0,32	1,27	2,64	4,64	4,64	3,27	3,38	2,64	3,69	4,64	3,80	2,74	1,37	1,05	0,63	0,74
1,00	1,50	0,21	0,21	0,21	0,42	0,95	2,43	3,38	2,74	1,16	1,16	0,95	0,84	0,95	0,32	0,21	0,11		
1,50	2,00	0,11	0,21	0,21	0,11	0,32	1,16	1,79	1,27	0,21	0,11	0,11	0,11	0,11					
2,00	3,00				0,11	0,21	0,53	0,95	0,53	0,11									
Total		0,70	0,84	0,98	2,53	5,49	11,67	14,35	10,41	6,47	5,20	6,33	7,45	6,47	4,08	2,11	1,55	0,84	0,98
		N			NNE			ENE			E			ESE			SSE		

165 to 175	175 to 185	185 to 195	195 to 205	205 to 215	215 to 225	225 to 235	235 to 245	245 to 255	255 to 265	265 to 275	275 to 285	285 to 295	295 to 305	305 to 315	315 to 325	325 to 335	335 to 345	Total
0,21	0,14	0,14	0,14	0,14	0,14	0,21	0,11	0,11	0,11	0,11	0,14	0,14	0,14	0,14	0,25	0,25	0,28	25,00
0,53	0,32	0,32	0,42	0,42	0,42	0,53	0,32	0,32	0,32	0,32	0,21	0,21	0,21	0,21	0,32	0,32	0,42	48,00
0,11	0,11	0,11				0,11					0,21	0,21	0,21	0,21	0,32	0,32	0,32	18,46
															0,11	0,11	0,11	6,12
																		2,43
0,84	0,56	0,56	0,56	0,56	0,56	0,84	0,42	0,42	0,42	0,42	0,56	0,56	0,56	0,56	0,98	0,98	1,13	100,00
S			SSW			WSW			W			WNW			NNW			

Table C-1: Wave climate at location of Wave rose A

Hs (m)		wave direction (Deg)																	
Lower	Upper	-15 to -5	-5 to 5	5 to 15	15 to 25	25 to 35	35 to 45	45 to 55	55 to 65	65 to 75	75 to 85	85 to 95	95 to 105	105 to 115	115 to 125	125 to 135	135 to 145	145 to 155	155 to 165
<	,50	0,14	0,16	0,18	0,23	0,56	1,35	1,53	0,93	0,70	0,74	0,81	0,93	0,70	0,63	0,32	0,32	0,18	0,05
,50	1,00	0,56	0,56	0,85	0,99	2,54	6,20	6,49	4,65	3,38	3,81	4,94	5,92	4,79	4,37	2,40	2,26	1,27	0,42
1,00	1,50	0,42	0,56	0,42	0,71	1,41	2,96	3,38	2,40	1,83	1,55	1,27	1,27	0,56	0,42	0,14	0,14	0,14	
1,50	2,00	0,14	0,14	0,14	0,14	0,42	1,13	1,55	0,28	0,28	0,42	0,14	0,14	0,14	0,14		0,14		
2,00	3,00						0,28	0,56											
Total		1,27	1,43	1,59	2,07	4,93	11,92	13,51	8,27	6,20	6,52	7,15	8,27	6,20	5,56	2,86	2,86	1,59	0,48
			N			NNE			ENE			E			ESE			SSE	

165 to 175	175 to 185	185 to 195	195 to 205	205 to 215	215 to 225	225 to 235	235 to 245	245 to 255	255 to 265	265 to 275	275 to 285	285 to 295	295 to 305	305 to 315	315 to 325	325 to 335	335 to 345	Total
0,07	0,05	0,04	0,04	0,02	0,04	0,05	0,02	0,04	0,02	0,02	0,02	0,02	0,02	0,04	0,07	0,11	0,16	11,30
0,42	0,28	0,28	0,14	0,14	0,28	0,28	0,14	0,28	0,14	0,14	0,14	0,14	0,14	0,28	0,42	0,56	0,71	61,34
0,14	0,14		0,14			0,14									0,14	0,28	0,42	21,01
																	0,14	5,50
																		0,85
0,64	0,48	0,32	0,32	0,16	0,32	0,48	0,16	0,32	0,16	0,16	0,16	0,16	0,16	0,32	0,64	0,95	1,43	100,00
	S			SSW			WSW			W			WNW			NNW		

Table C-2: Wave climate at location of Wave rose B

Hs (m) Lower	Upper	wave direction (Deg) -15 to -5	-5 to 5	5 to 15	15 to 25	25 to 35	35 to 45	45 to 55	55 to 65	65 to 75	75 to 85	85 to 95	95 to 105	105 to 115	115 to 125	125 to 135	135 to 145	145 to 155	155 to 165
<	,50	0,19	0,17	0,16	0,24	0,29	0,30	0,36	0,42	0,63	0,92	1,00	1,00	1,09	0,85	0,53	0,37	0,32	0,23
,50	1,00	1,01	0,88	0,88	1,52	2,15	1,89	2,15	2,27	3,66	5,43	6,32	6,69	7,83	6,44	4,04	2,78	2,27	1,52
1,00	1,50	0,63	0,63	0,51	0,63	0,38	0,76	1,01	1,39	1,77	2,27	2,27	1,89	1,52	0,76	0,38	0,25	0,25	0,25
1,50	2,00									0,13	0,38	0,25	0,25	0,25	0,25	0,25	0,25	0,25	0,25
2,00	3,00																		
Total		1,83	1,69	1,55	2,39	2,81	2,95	3,52	4,08	6,19	9,00	9,85	9,85	10,69	8,30	5,20	3,66	3,09	2,25
		N			NNE			ENE			E			ESE			SSE		

165 to 175	175 to 185	185 to 195	195 to 205	205 to 215	215 to 225	225 to 235	235 to 245	245 to 255	255 to 265	265 to 275	275 to 285	285 to 295	295 to 305	305 to 315	315 to 325	325 to 335	335 to 345	Total
0,11	0,07	0,04	0,03	0,03	0,04	0,04	0,04	0,03	0,03	0,04	0,04	0,06	0,07	0,07	0,07	0,14	0,16	10,20
0,51	0,38	0,25	0,13	0,13	0,25	0,25	0,25	0,25	0,25	0,38	0,38	0,51	0,38	0,38	0,38	0,88	0,88	66,56
0,25	0,25	0,13	0,13	0,13	0,13	0,13	0,13						0,25	0,25	0,25	0,38	0,51	20,46
0,25																		2,78
																		0,00
1,13	0,70	0,42	0,28	0,28	0,42	0,42	0,42	0,28	0,28	0,42	0,42	0,56	0,70	0,70	0,70	1,41	1,55	100,00
S			SSW			WSW			W			WNW			NNW			

Table C-3: Wave climate at location of Wave rose C

Hs (m) Lower	Hs (m) Upper	-15 to -5	-5 to 5	5 to 15	15 to 25	25 to 35	35 to 45	45 to 55	55 to 65	65 to 75	75 to 85	85 to 95	95 to 105	105 to 115	115 to 125	125 to 135	135 to 145	145 to 155	155 to 165
<	,50	1,85	1,70	1,56	2,42	2,84	2,98	3,55	4,12	6,25	9,09	9,94	9,94	1,08	0,84	0,53	0,37	0,31	0,23
,50	1,00													7,93	6,52	4,09	2,81	2,30	1,53
1,00	1,50													1,53	0,77	0,38	0,26	0,26	0,26
1,50	2,00													0,26	0,26	0,26	0,26	0,26	0,26
2,00	3,00																		
Total		1,85	1,70	1,56	2,42	2,84	2,98	3,55	4,12	6,25	9,09	9,94	9,94	10,80	8,38	5,26	3,69	3,13	2,27
		N			NNE			ENE			E			ESE			SSE		

Hs (m) Lower	Hs (m) Upper	165 to 175	175 to 185	185 to 195	195 to 205	205 to 215	215 to 225	225 to 235	235 to 245	245 to 255	255 to 265	265 to 275	275 to 285	285 to 295	295 to 305	305 to 315	315 to 325	325 to 335	335 to 345	Total
<	,50	0,11	0,07	0,04	0,03	0,03	0,04	0,04	0,04	0,03	0,03	0,04	0,04	0,06	0,07	0,07	0,71	1,42	1,56	62,41
,50	1,00	0,51	0,38	0,26	0,13	0,13	0,26	0,26	0,26	0,26	0,26	0,38	0,38	0,51	0,38	0,38				29,92
1,00	1,50	0,26	0,26	0,13	0,13	0,13	0,13	0,13	0,13						0,26	0,26				5,24
1,50	2,00	0,26																		1,79
2,00	3,00																			0,00
Total		1,14	0,71	0,43	0,28	0,28	0,43	0,43	0,43	0,28	0,28	0,43	0,43	0,57	0,71	0,71	0,71	1,42	1,56	100,00
		S			SSW			WSW			W			WNW			NNW			

Table C-4: Wave climate at location of Clifton Point

Appendix D: Influence of swell at Clifton Point

Path 1
The waves entering the lee side of Lyford Cay are first affected by refraction. Besides the change of the wave's direction, the height of the wave will also decrease. This effect is characterised by the refraction coefficient (K_r).

$$K_r = \sqrt{\frac{\cos\alpha_0}{\cos\alpha}}, \qquad \text{since } \frac{\sin\alpha}{c} = \text{constant} \rightarrow \frac{\sin\alpha_0}{c_0} = \frac{\sin\alpha}{c}$$

where: α_0= angle between wave crest and contour of constant water depth at deep water
α = angle between wave crest and contour of constant water depth at location
c_0= wave celerity at deep water
c = wave celerity at location

c_0 at deep water is determined by $c_0 = \dfrac{gT}{2\pi}$.

c at shallow water is determined by $c = \sqrt{gd}$.

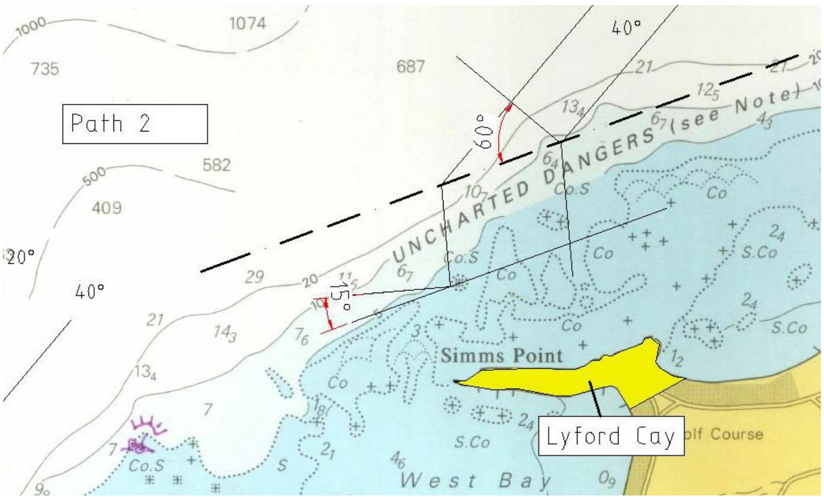


Figure D-1: Refraction at northern shorelines of New Providence, alpha!

The contour of constant water depth follows a path of 70°. This results in a refraction factor of:

Wave direction	α_0	α	Kr	Corrected wave direction
0°	20°	5,9°	0.95	350°
20°	40°	11,1°	0.78	350°
40°	60°	15,0°	0.52	0°
60°	80°	17,1°	0,18	0°

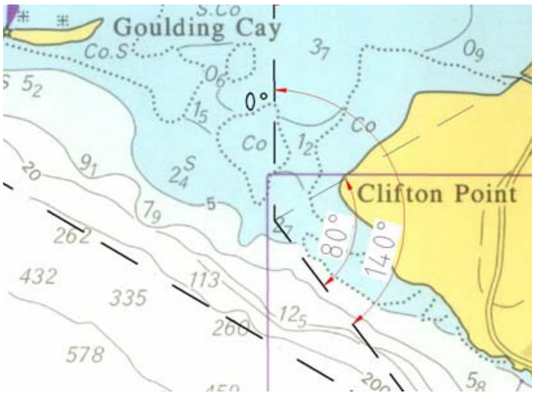
Table D-1: Refraction factor north shore, New Providence

The wave height of the incoming waves will alter in the way presented in table D-2. Due to the shallow coast line waves will break by a $H_{S/d}$ ratio of 0.5. Because of the minimum water depth of 1.00 m. waves with $H_S > 0.50$ m. will break.

With the transformation of the angle of incidence of the waves due to refraction, the effect of diffraction around Lyford Cay (figure D-1) can be neglected. The corrected direction of the waves is rounded off in one digit.

Waves arriving at the southern shore of New Providence will refract in southwest direction, the direction of the open sea. The project site can therefore only be reached by waves diffracted at Clifton Point as presented in figure D-2.

The diffraction around the western shore of Clifton Point is represented by the diffraction coefficient K_d. This coefficient is determined by the method of Wiegel (1962). The following parameters can be discerned, which are presented in figure D-2:



α = angle between barrier and incident wave direction

β = angle between barrier and point of interest

r= radial distance between diffraction point and point of interest (m)

L= wave length (m)

Figure D-2: Diffraction of waves around western coast of Clifton Point

For the wave directions presented in table D-1 the diffraction coefficient is determined around the western shore of Clifton Point in table D-2. The radial distance (r) from the point of diffraction and Clifton Point is approximately 1750 m, resulting in r/L = 5.

Original wave direction	Corrected wave direction	α	β	Kd
0°	350°	150°	80°	0.10
20°	350°	150°	80°	0.10
40°	0°	140°	80°	0.11
60°	0°	140°	80°	0.11

Table D-2: Diffraction factor along western coast of Clifton Point

It can be concluded that the waves travelling over the shallow banks have a H_{Smax} of 1.00 m. Diffraction of these waves will result in a H_S in the order of 0.10 m. Swell with this wave height can be neglected.

Path 2

Path 1 resulting in a relatively large loss of energy. The length of path 2 is longer. The expected loss of energy is smaller. Since the radial distance is parallel to the schematised shoreline the influence of refraction is neglected. Swell arriving at Goulding Cay will only diffract. The wave directions of 320° and 340° represent only a small part of the total wave climate. Also the swell caused by wave climate from the Atlantic in these directions can be questionable. These directions are however included to review every option of wave climate at Clifton Point.

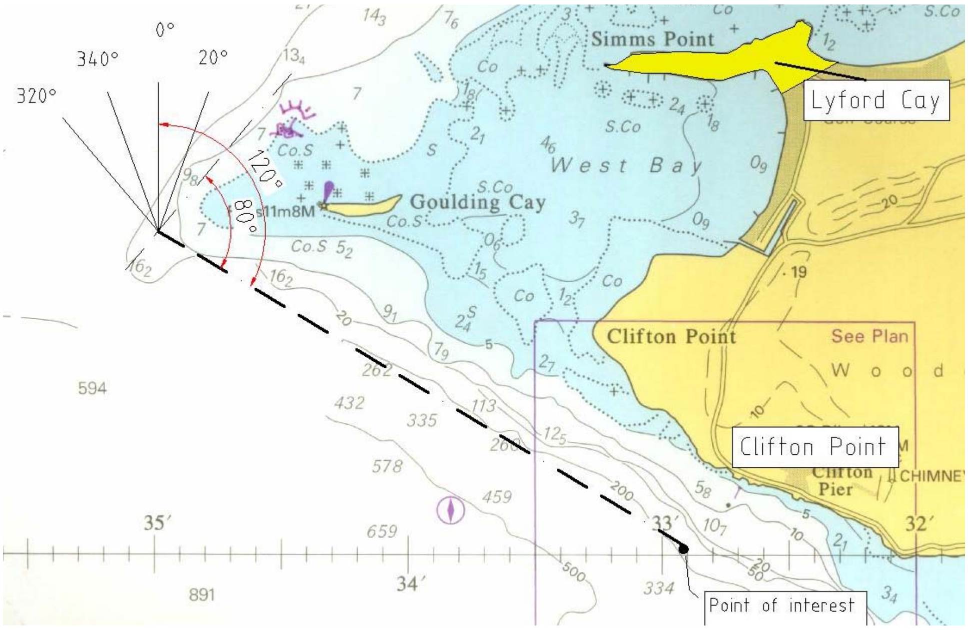


Figure D-3: Diffraction of waves around Goulding Cay

The radial distance between the diffraction point and the point of interest is r= 4500 m. resulting in a r/L ratio of 12.9. The diffraction factor is presented in table D-3.

Wave direction	α	β	Kd
320°	80°	80°	0.50
330°	90°	80°	0.30
340°	100°	80°	0.15
350°	110°	80°	0.13
0°	120°	80°	0.10
20°	140°	80°	0.08

Table D-3: Diffraction along Goulding Cay

As a result of the waves curving around Goulding Cay, the only swell reaching the deep water location of Clifton Point originates from the directions of 320° and 330°. This can be seen in figure D-4.

For this path it is also presumed that the total wave climate consists of swell; this is not realistic. By assuming that 20 % of the total wave climate is indicated as swell, the percentage of swell with $H_S > 0.50$ m.
reaching Clifton Point from a direction of 320° is smaller than 0.10 %.
This amount of swell can be neglected for the determination
of the port's downtime.

Wave direction		H_s at Wave Rose A (%)				expected H_s at Clifton Point (%)		
	K_d	< 1.00 m	1.00-1.50	1.50-2.00	2.00-3.00	< 0.50 m	0.50-1.00	>1.00
320°	0.50	0.42	0.42	0.14	-	0.50	0.48	-
330°	0.30	0.42	0.42	0.14	-	0.88	0.10	-
340°	0.15	0.56	0.42	0.14	-	1.13	-	-
350°	0.13	0.28	0.28	0.14	-	0.70	-	-
0°	0.10	0.28	0.28	0.28	-	0.84	-	-
20°	0.08	1.69	0.56	0.14	0.14	2.53	-	-

Table D-4: Influence of swell from north western direction at Clifton Point location

Appendix E: Optimisation of layout breakwater

BW 2

Wave direction (°)	100 % wave penetration ?	P (H_s > H_{cr})	wave direction (% /year)	Downtime (% /year)
150°		0,01	3,09	0,02
160°		0,07	2,25	0,16
170°		0,55	1,13	0,63
180°		0,72	0,70	0,50
190°	•	0,92	0,42	0,38
200°	•	0,96	0,28	0,27
210°	•	0,96	0,28	0,27
220°	•	0,94	0,42	0,40
230°	•	0,94	0,42	0,39
240°	•	0,91	0,42	0,38
250°	•	0,45	0,28	0,13
260°		0,32	0,28	0,09
270°		0,00	0,42	0,00
Total				3,62

Table E-1: Annual downtime for container terminal of variant BW 2

Wave direction (°)	100 % wave penetration ?	P (H_s > H_{cr})	wave direction (% /year)	Downtime (% /year)
230°		0,00	0,42	0,00
240°		0,00	0,42	0,00
250°	•	0,07	0,28	0,02
260°	•	0,49	0,28	0,14
270°	•	0,61	0,42	0,26
280°	•	0,75	0,42	0,32
290°	•	0,81	0,56	0,45
300°	•	0,88	0,70	0,62
310°	•	0,64	0,70	0,44
320°		0,00	0,70	0,00
Total				2,25

Table E-2: Annual downtime for dry bulk terminal of variant BW 2

BW 3

Wave direction (°)	100 % wave penetration ?	P (H_s > H_{cr})	wave direction (% /year)	Downtime (% /year)
150°		0,00	3,09	0,00
160°		0,00	2,25	0,00
170°		0,21	1,13	0,24
180°		0,56	0,70	0,39
190°	•	0,83	0,42	0,35
200°	•	0,96	0,28	0,27
210°	•	0,96	0,28	0,27
220°	•	0,95	0,42	0,40
230°	•	0,94	0,42	0,40
240°	•	0,92	0,42	0,39
250°	•	0,82	0,28	0,23
260°	•	0,76	0,28	0,21
270°	•	0,32	0,42	0,13
Total				3,27

Table E-3: Annual downtime for container terminal of variant BW 3

Wave direction (°)	100 % wave penetration ?	P (H_s > H_{cr})	wave direction (% /year)	Downtime (% /year)
230°		0,00	0,42	0,00
240°		0,00	0,42	0,00
250°	•	0,07	0,28	0,02
260°	•	0,49	0,28	0,14
270°	•	0,61	0,42	0,26
280°	•	0,75	0,42	0,32
290°	•	0,81	0,56	0,45
300°	•	0,88	0,70	0,62
310°	•	0,64	0,70	0,44
320°		0,00	0,70	0,00
Total				2,24

Table E-4: Annual downtime for dry bulk terminal of variant BW 3

BW 4

Wave direction (°)	100 % wave penetration ?	P ($H_s > H_{cr}$)	wave direction (% /year)	Downtime (% /year)
150°		0,00	3,09	0,00
160°		0,00	2,25	0,00
170°		0,00	1,13	0,00
180°		0,00	0,70	0,00
190°		0,65	0,42	0,27
200°	•	0,87	0,28	0,24
210°	•	0,96	0,28	0,27
220°	•	0,95	0,42	0,40
230°	•	0,94	0,42	0,40
240°	•	0,92	0,42	0,39
250°	•	0,82	0,28	0,23
260°	•	0,76	0,28	0,21
270°	•	0,32	0,42	0,13
280°	•	0,09	0,42	0,04
Total				2,54

Table E-5: Annual downtime for container terminal of variant BW 4

Wave direction (°)	100 % wave penetration ?	P ($H_s > H_{cr}$)	wave direction (% /year)	Downtime (% /year)
230°		0,00	0,42	0,00
240°		0,00	0,42	0,00
250°		0,00	0,28	0,00
260°	•	0,07	0,28	0,02
270°	•	0,61	0,42	0,26
280°	•	0,75	0,42	0,32
290°	•	0,81	0,56	0,45
300°	•	0,88	0,70	0,62
310°	•	0,64	0,70	0,44
320°		0,00	0,70	0,00
Total				2,11

Table E-6: Annual downtime for dry bulk terminal of variant BW 4

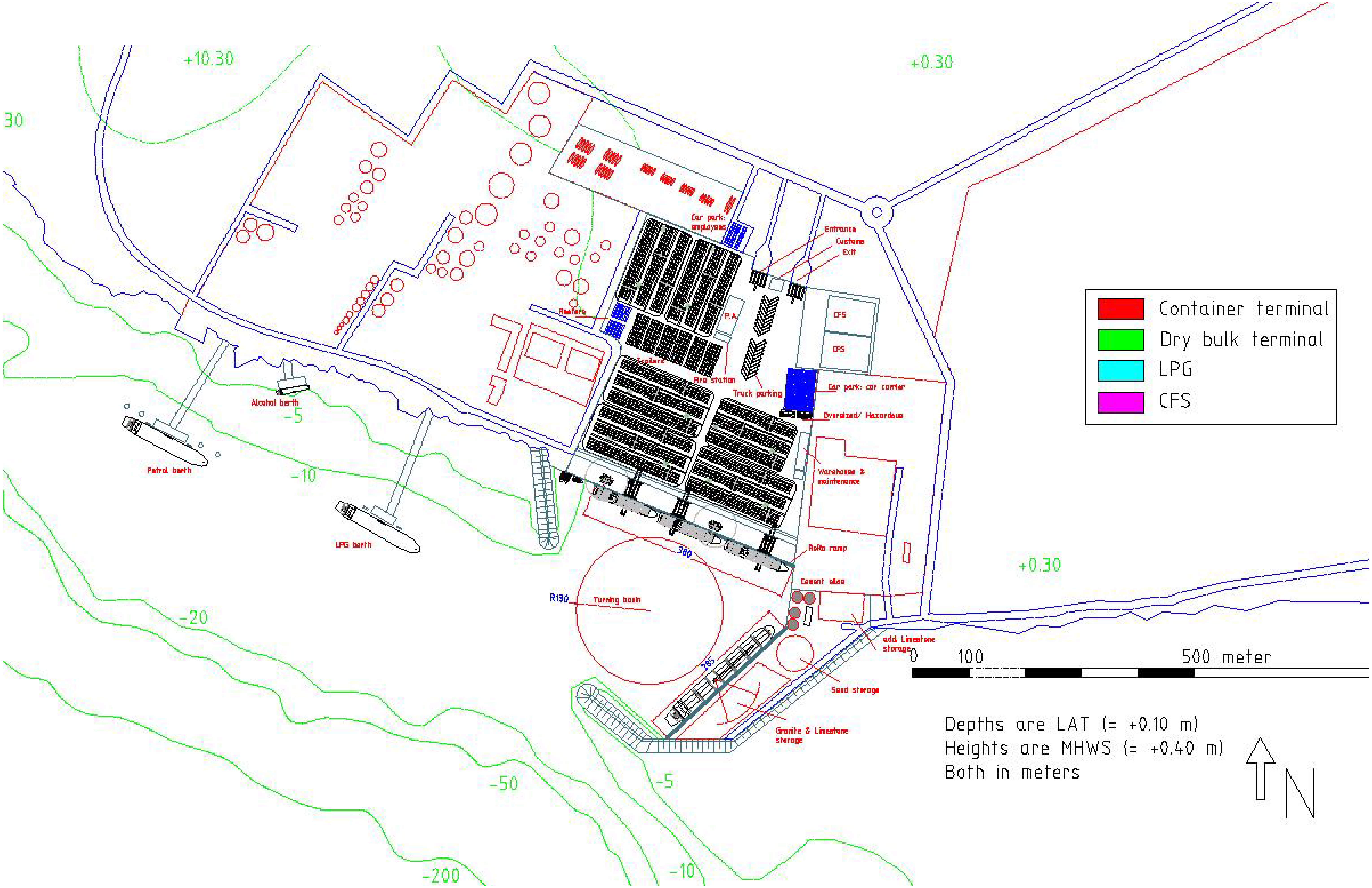
BW 5

Wave direction (°)	100 % wave penetration ?	P (H_s > H_{cr})	wave direction (% /year)	Downtime (% /year)
150°		0,00	3,09	0,00
160°		0,00	2,25	0,00
170°		0,00	1,13	0,00
180°		0,00	0,70	0,00
190°		0,65	0,42	0,27
200°	•	0,87	0,28	0,24
210°	•	0,96	0,28	0,27
220°	•	0,95	0,42	0,40
230°	•	0,94	0,42	0,40
240°	•	0,92	0,42	0,39
250°	•	0,82	0,28	0,23
260°	•	0,76	0,28	0,21
270°	•	0,76	0,42	0,32
280°	•	0,76	0,42	0,32
290°		0,76	1,42	1,07
300°		0,00	2,42	0,00
Total				4,11

Table E-7: Annual downtime for container terminal of variant BW 5

Wave direction (°)	100 % wave penetration ?	P (H_s > H_{cr})	wave direction (% /year)	Downtime (% /year)
230°		0,00	0,42	0,00
240°		0,00	0,42	0,00
250°		0,00	0,28	0,00
260°	•	0,07	0,28	0,02
270°	•	0,61	0,42	0,26
280°	•	0,75	0,42	0,32
290°	•	0,81	0,56	0,45
300°	•	0,88	0,70	0,62
310°	•	0,89	0,70	0,62
320°		0,00	0,70	0,00
Total				2,28

Table E-8: Annual downtime for dry bulk terminal of variant BW 5


Container terminal
Dry bulk terminal
LPG
CFS
Depths are LAT (= +0.10 m)
Heights are MHWS (= +0.40 m)
Both in meters
0
100
500 meter
N
+10.30
+0.30
+0.30
30
-5
-10
-20
-50
-200
-5
-10
Car park: employees
Entrance
Customs
Exit
CFS
CFS
P.A.
Reefers
Fire station
Truck parking
Car park: car carrier
Oversized/ Hazardous
Warehouse & maintenance
RoRo ramp
Cement silos
Turning basin
R130
380
Sand storage
Granite & Limestone storage
Alcohol berth
Petrol berth
LPG berth