Preface

This document is the final report for my thesis project required as part of my MSc degree programme at Delft University of Technology, Faculty of Civil Engineering & Geosciences, Sections Ports & Waterways and Civil Management. This thesis has been carried out in co-operation with Posford Haskoning, Peterborough, England.

Six months working in the Maritime Division of Posford Haskoning at their Peterborough Office proved to be a very instructive period. Besides, living and working in England was an invaluable experience.

First of all I would like to thank Henry Rowe and Simon Harries for making this thesis project possible. Many thanks goes to my colleagues from Posford Haskoning. Especially Adrian Hoyle and Wim Welvaarts have been very helpful in providing me with information. Special thanks go to Jim Marriage for his guidance and efforts.

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Abstract

This Final Report presents the results of research on the sensitivity of outcome construction costs due to varying site investigation data. The study concentrated on the design for the construction of a new port and dry-dock complex at Duqm in the Al Wusta Region of the Sultanate of Oman. The study was carried out in co-operation with Posford Haskoning Ltd., Peterborough, United Kingdom.

This report presents designs for two design phases, the preliminary design phase and the detailed design phase. In the scope of this research, a port lay-out already prepared by Posford Haskoning was adopted for this study. Following port components have been designed: breakwaters, quay walls, dredging works and land reclamation. Next costs and designs of the different port components are compared between the preliminary and the detailed design phase.

In the first design phase, referred to as the preliminary design phase, a design is presented on basis of limited data. A lower bound (optimistic) and upper bound (realistic pessimistic) design are also presented. Subsequently cost estimations for the designs are established using a unit rate technique.

In the second design phase, referred to as the detailed design phase, a design is presented on basis of more detailed data. Again lower bound and upper bound were indicated. Same unit rate technique was used to establish a cost estimate for the detailed design phase. The protective structures consist of two breakwaters. The main breakwater is protected with Core-Loc, the lee breakwater with rock as primary armour unit. A block-work wall is preferred as quay wall structure.

In port projects like Duqm the geotechnical and wave data are of uttermost importance. On basis of the data available it has been indicated what design margins could realistically be expected for the breakwater design, the quay wall design and the dredging works and land reclamation.

With the available information the sensitivity on costs of variations in the wave climate have been analysed. The results of this analysis are only valid for Duqm but they provide a good view for comparable projects. Contrary, the limited geotechnical information available made it only possible to give an indication of the margins to be expected at Duqm.

For determination of the inshore wave climate same offshore wave data were used in both design stages. Although same offshore wave data were used in both design stages, the inshore wave climate changed considerably. In the first design phase hand calculations were used to determinate the inshore wave climate while in the second design phase the inshore wave climate was determined with a SWAN-analysis. The difference in inshore wave climate can be explained by the fact that the SWAN-analysis does take account of the bathymetric profile and the shape of the land.

Small changes in wave climate can have great influence on the breakwater costs. Especially at the point where rock of sufficient weight to be stable is not available anymore and the breakwater has to be protected with Core-Loc primary armour.

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Symbols and abbreviations

Names and Organisations

ARGOSS	Advisory and Research Group on Geo Observation Systems and Services
CRESS	Coastal and River Engineering Support System
IHE	International Institute for Infrastructural, Hydraulic and Environmental Engineering
PIANC	Permanent International Association of Navigation Congresses
SWAN	Simulating WAves Near shore

Other abbreviations

dwt	dead weight ton
e.g.	for example
i.e.	that is
h	height
l	length
no	number (units); as in 5 no
%	per cent
£	pound sterling
ALWC	Accelerated Low Water Corrosion
BS	British Standard
CD	Chart Datum
D_{n50}	median nominal diameter
H_s	design wave height
K_d	stability coefficient
K_r	factor of refraction
K_s	factor of shoaling
L_o	wave length
MHHW	Mean High High Water
MWL	Mean Water Level
NE	North East
S	type of steel as in S355
SI	Site Investigation
SW	South West
SWL	Still Water Level
T_s	design wave period
UK	United Kingdom

W_{50}	mean weight of armour stones
ρ_m	density material

Units of measurement

mm	millimetre (s)
m	metre (s)
km	kilometre (s)
km^2	square kilometre
m^2	square metre
m^3	cubic metre
m^3/m^2	cubic metre (s) per square metre
g	acceleration of gravity, m/s^2
hr	hour
l/s/m	litre (s) per second per metre
m/s	metres per second
m/s^2	metres per second squared (acceleration)
mm/y	millimetre (s) per year
s	second
kg	kilogram
kg/m^3	kilograms per cubic metre
t	ton = kg x 10^3
^oC	degrees Celsius

Arabic words

khareef	monsoon (from south west)
ra's	cape
sabkha	salt flats – seaside marshy area with thin crust of salt
wilayat	administrative district

1 General Introduction

Background of the project

In recent years the Government of Oman has made some investment in fisheries with the construction of a fish-processing factory at Ra's ad Duqm where shelter exists for small fishing boats in the lee of the Ra's ad Duqm headland.

The Government now intends to build a new port including a dry-dock facility and fisheries harbour at Duqm. This will help to stimulate development of the region and bring employment opportunities to the area. The new port will also be used for the benefit of the Royal Navy of Oman, the Royal Yacht Squadron, the Royal Omani Police and the coastguard.

The Wilayat of Duqm, approximately 10,000 km^2, is part of the Al Wusta region on the South East Coast of Oman but is sparsely populated with some 3,200 people. The small town of Duqm is situated near the coast some 600-km from Muscat. A map of Oman is shown in Appendix I.

Posford Haskoning was awarded the consultancy services for the design of a new port and dry-dock complex at Duqm. At the moment they put a finishing touch to their final report.

Motivation of this research

Especially in deserted regions like Duqm there is a lack of accurate site data at the beginning of the design process. These data are normally refined during the detailed design stage. However these data, for instance geological conditions and wave climate, have a great influence on the construction costs.

During the preliminary design stage, at the beginning of the design process, it is important to be aware of the uncertainties in the site data. To control the inaccuracy in the site data it is important to know what the sensitivity of the influence in the variation of the site data is on design and costs. Therefore uncertainties in the site data should be translated in design and costs. This is the starting point of this study.

Structure of the report

Section 2 of this report deals with the problem description of this study resulting in a plan of research. The content of this section was already presented in a preliminary study phase report. Section 3 provides the boundary conditions for the preliminary design. Section 4 presents the preliminary design of the port components. Section 5 translates the preliminary design into preliminary cost estimates for the different port components.

With the start of Section 6 a new design phase starts. This section provides the boundary conditions for the detailed design phase. Section 7 discusses the detailed designs of the port components. Next the cost estimates for the detailed design phase are presented in Section 8.

Section 9 brings together and provides a comparison between both design phases. Conclusions and recommendations regarding this comparison are presented in Chapter 10. Finally, in Chapter 11 a brief description of the developments of the Duqm port and dry-dock complex project during this study, are presented.

More detail on the studies carried out is given in the Appendices, which are presented in a separate volume.

2 Problem description

2.1 Problem analysis

At the start of a project the client or his consultant will prepare an initial estimate of the costs. The early estimate is particularly important as it influences the client's intentions and can determine the viability of the entire project. Despite the lack of accurate data and the presence of unquantifiable factors it is therefore important to make the first estimate realistic.

The first estimate that is published for review and approval has a particularly crucial role to play because it is the basis for the release of funds for further studies and estimates, and because it becomes the marker against which subsequent estimates are compared. Moreover, early estimates are important because of the need to know, for the purposes of economic appraisal, the capital cost of the project[Lit 24].

2.2 Problem definition

At the beginning of the design stage of a project the available site investigation (SI) data are often not very detailed, these data are normally refined during the detailed design stage. However these data, for instance geological conditions and wave climate, have a great influence on the construction costs.

2.3 Objective

Research on the sensitivity of the outcome construction costs due to varying site investigation data.

2.4 Research limits

Due to the available time, the study will concentrate on the evaluation of construction costs for the port of Duqm. This project is outlined in the Inception Report of this study[Lit 12]. Detailed information can be found in the Draft Final Report on the Port of Duqm prepared by Posford Haskoning[Lit 19].

2.5 Plan of research

Posford Haskoning has already prepared a general layout for the port at Duqm. The phase 1 layout will form the basis for the research; the ship repair yard has been left aside, the lay-out can be found in Appendix II. In order to get some insight on the sensitivity of the outcome construction costs due to varying SI data the construction costs will be analysed in two different design stages, the preliminary design stage and the detailed design stage.

For each stage two designs and cost calculations will be made. In the preliminary design stage a design will be made on the basis of limited data. In the detailed design stage a design will be made on base of new data which are obtained at this time and will be available in a later stage of the study.

3 Boundary conditions preliminary design

3.1 Introduction

In this section the boundary conditions for the preliminary design phase will be presented. First Section 3.2 describes the meteorological conditions at Duqm. In Sections 3.3 and 3.4 the wave climate and the tides and currents are presented. Finally Section 3.5 describes the geotechnical conditions.

3.2 Meteorological data

Temperature

Temperature records for the period 1956 to 2001 were obtained from the weather station at Masirah, see Figure 3-1. Mean air temperature varies from 22°C to 30°C. Maximum and minimum temperatures over the 45-year period were 45°C and 9°C respectively[Lit 19].

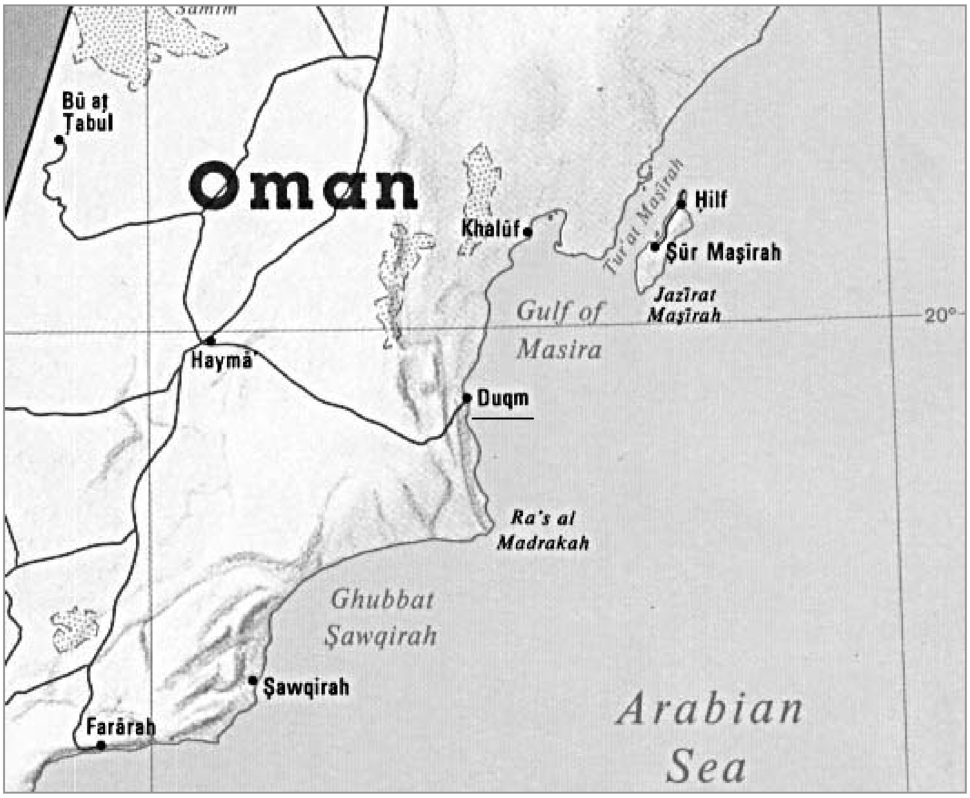

Figure 3-1 Duqm and Masirah

Rainfall

Rainfall data for the period 1956 to 2001 were obtained from the weather station at Masirah. The average annual rainfall is 64mm. However there is a significant variation over the years[Lit 19].

Wind

Wind data were obtained from the UK Meteorological Office[Lit 19]. The wind climate is dominated by the north-east and south-west monsoons. The wind speeds corresponding to a return period of 1 in 100 years were calculated using an Extreme Value Analysis, Table 3-1.

Return period (years)	Wind direction (degrees north) (knots)							
	0	**30**	**60**	**90**	**120**	**150**	**180**	**210**
1:100	58.3	52.1	48.3	57.4	46.9	52.0	72.2	76.3

Table 3-1 wind speeds, 1:100 year return period

Water level changes due to wind set-up are computed with CRESS, Table 3-2. CRESS (Coastal and River Engineering Support System) is a computer program developed by Rijkswaterstaat and IHE Delft that supports calculations on basic coastal and river engineering processes.

Wind set-up, 1:100 years						
Degrees north	30	60	90	120	150	180
Angle of approach Duqm	60	30	0	30	60	90
Wind speed (knots)	52.1	48.3	57.4	46.9	52.0	72.2
Wind speed (m/s)	26.1	24.2	28.7	23.5	26.0	36.1
Set-up (m)	0.17	0.26	0.42	0.24	0.17	-

Table 3-2 wind set-up in front of main breakwater

Cyclones

Tropical cyclones generally develop off the western coast of India and track westwards across the Arabian Sea. Most of these storms diminish before reaching land. A cyclone passed over the northern tip of Masirah Island in 1977 with wind speeds up to 45 m/s. Baird and Associates (2001) estimated the return period of such an event to be between 200 and 500 years. Estimations of the offshore significant wave height associated with the event were in the range of 9.5m to 13.5m. The return period of a cyclone occurring at Duqm would be higher:

- All cyclones cross the coast between Ra's al Hadd and the Yemen border (1500-km).
- The centre of the cyclone must pass within 200-km of Duqm for the waves to have a significant impact.

It is concluded the occurrence of cyclones is not decisive for the design of the port components.

3.3 Wave climate

Offshore wave climate

Wave data were obtained from the UK Meteorological Office (visual observations), ARGOSS satellite measurements and the Meteorological Office at Seeb International Airport (forecasting models). An extreme value analysis of the offshore data was undertaken to calculate return period waves from different directions. Table 3-3 summarises the extreme wave heights for various wave directions and return periods[Lit 19].

Return Period (years)	Wave Direction (Degrees North)								
	0	30	60	90	120	150	180	210	All
1:Month	1.49	2.03	1.91	1.62	1.11	1.75	3.74	4.46	5.18
1:1	3.17	3.67	3.35	3.28	2.96	3.75	5.95	7.00	7.28
1:5	4.23	4.72	4.27	4.32	4.12	5.02	7.35	8.43	8.64
1:10	4.68	5.17	4.67	4.77	4.62	5.57	7.96	9.05	9.22
1:20	5.14	5.62	5.07	5.21	5.13	6.12	8.56	9.67	9.81
1:50	5.74	6.21	5.59	5.81	5.79	6.84	9.36	10.48	10.58
1:100	6.19	6.66	5.99	6.25	6.29	7.39	9.96	11.10	11.17

Table 3-3 offshore return period wave heights (m)

Inshore wave climate

As the offshore waves propagate into shallower water the wave heights and directions change by the influence of the bathymetry[Lit 1]. With CRESS the refraction and shoaling factors were computed in order to determine the design wave height H_s, see Table 3-4. In accordance to practice and design lifetime a return period of 100 years was used[Lit 7]. The depth is not a limiting factor in front of the main breakwater; the depth is approximately 15m.

Computation of influence of bathymetry on the design wave height 1:100 years					
Deep water angle	60	30	0	30	60
H_s, deep water (m)	6.66	5.99	6.25	6.29	7.39
L_o, wave length (m)	222	200	208	210	246
T_s, wave period (s)	11.9	11.3	11.6	11.6	12.6
K_s, shoaling (-)	0.73	0.94	1	0.94	0.73
K_r, refraction (-)	1.12	1.07	1.07	1.07	1.12
Inshore design wave height (m)	5.45	6.02	**6.70**	6.33	6.04

Table 3-4 inshore design wave height

Lower and upper bound wave conditions

Assumptions have to be made on the range of sensitivity of starting wave and wind conditions. A margin of 20% as lower and upper bound on offshore wave and wind conditions has been set. The inshore wave climate and wind set-up have been determined, Table 4-9.

The relatively wide margin is the result of the assessment of the inaccuracy in offshore wave and wind climate data and of the simplified formulas used in preliminary design. Historical data are used to predict a future situation. Some of these data are from visual observations. The record time of the most accurate data, the satellite observations, is limited (15 years). The formulas used to determine the inshore wave and wind climate do not take account of the shape of the land and the bathymetric profile.

Offshore		Lower bound	Upper bound
Deep water wave height	m	5.0	7.5
Mean wave period, T_s	s	10.3	12.7
Inshore			
Significant wave height, H_s	m	5.3	7.95
Wind set-up	m	0.27	0.6
Design water level	m + CD	2.9	3.3

Table 3-5 lower and upper bound wave and wind conditions

3.4 Tides and currents

The tidal range in the Arabian Sea is generally small. Halcrow established tidal levels at Duqm as part of a previous study [Lit 10], Table 3-6.

Tidal levels to Chart Datum (m)	
Mean high high water level	2.53
Mean low high water level	2.29
Mean high low water level	1.69
Mean low low water level	0.90

Table 3-6 tidal levels near Duqm

For the design an extreme water level of CD + 3.1m has been adopted. This combines MHHW with a storm surge of approximately 0.6m. The storm surge is a combination of the wind set-up and atmospheric pressure reduction.

The tidal currents are generally weak, with a mean rate of 0.5 m/s in a north-east direction during SW monsoon (khareef), and a mean rate of 0.25 m/s in south-west direction during NE monsoon.

3.5 Geotechnical data

Site location

The site is located on the East Coast of Oman near the small village of Duqm some 600-km from Muscat. It includes the headland of Ra’s ad Duqm, a smaller headland some 2-km to the north west of Ra’s ad Duqm and a small bay in between, see Appendix I and III.

Site geology

To the south of the Ra’s ad Duqm headland the topography is characterised by 75-km of cliffs at an elevation of 70 to 90 metres. From the 1992 Geological Map of Duqm and Madracca, the Duqm Formation would appear to be less resilient to erosion than the limestone outcrops in the region; this explains the existence of the bay in the first place. Appendix IV shows a geological section of the Duqm formation at Ra’s ad Duqm.

The small bay between the two headlands has a sandy beach, with some rocky outcrops and sabkha (seaside marshy area with thin crust of salt). The site is underlain with rock covered with a layer of marine deposits, consisting of loose material. The thickness of this marine deposit layer is likely to vary from a few centimetres up to several meters. Except for the ridge forming Ra’s ad Duqm, the shore topography is very flat and low-lying.

The coastline immediately to the north of the smaller headland consists of a large area of sabkha leading to a long low-lying bay with a sandy beach[Lit 15, 19].

Implications for breakwater and quay wall design

A great variety of subsoil conditions is to be found beneath much of the site. Some of the hard outcrops of the Duqm Formation may preclude driven piles. Contrary some sandy layers might be causing bearing capacity problems, particularly for the quay walls.

The marine deposits of recent years are considered to be too soft to support a quay wall and will have to be removed. This dredged material is too fine to use for reclamation.

The subsoil under the breakwater will mainly consist of bedrock covered with sand. Settlements of the subsoil will be negligible. Because of the presence of bedrock a deep slip circle is unlikely to occur.

Material sources

The breakwaters have the highest demand for local construction materials. A minor amount of material will be needed for the reclamation.

At the moment there are no existing quarries in the vicinity of the proposed site. Earlier visual site investigations have pointed out two possible quarry locations [Lit 16].

- The first location is the ridge running inland from the Ra's ad Duqm. Blocks have been seen up to 4 ton, however it is more likely the average block weight will be about 1.3 ton.

- The second quarry location can be found along the access road to Sidarah, about 40-km from the site. Fallen blocks up to 1.6 ton have been found. Larger sized in situ blocks in the 5 – 10 ton range are probable ($\rho_m \approx 2600$ kg/m^3).

Probably some of the dredged materials from basins and entrance channel can be used for the reclamation.

Seismic activity

According to seismic design guidelines for port structures of PIANC, Oman lies in earthquake intensity zone 0. This results in values of acceleration between 0 – 0.05g at a return period of 475 years. Normal good building practice should ensure adequate protection.

4 Preliminary design of port components

4.1 Introduction

This section describes the preliminary design of the port components. For the breakwater design as well as for the quay wall design a range of possible solutions will be discussed. The most appropriate solutions are adopted for further design.

In Section 4.2 the preliminary breakwater designs are presented. Lower and upper bound designs are also presented. For the quay wall two suitable solutions were adopted for further design, see Section 4.3. Upper and lower bound are again indicated. Finally dredging works and land reclamation are discussed in Section 4.4.

4.2 Preliminary breakwater design

4.2.1 Introduction

Section 4.2.2 discusses the range of structures available for the breakwater at the new Duqm port. A short description of each type is given. For each type is indicated if it is an appropriate solution for Duqm.

Based on the design conditions, Section 3, designs have been made for the main and lee breakwater, Sections 4.2.3 and 4.2.4. Subsequently designs for the upper and lower bound conditions are presented.

4.2.2 Breakwater options

Monolithic breakwater

Monolithic breakwaters have a cross-section designed in such a way the structure acts as a solid block. Main differences between the mound types and the monolithic type of breakwater are caused by the interaction between the structure and the subsoil and by the behaviour of failure. A major drawback on the use of monolithic breakwaters is in case of exceeding a critical load value: the structure will lose stability at once.

Most common type of a monolithic breakwater is a caisson structure. Besides the behaviour of failure there are a number of other drawbacks that make a caisson breakwater structure inappropriate for Duqm:

- A special dock is needed to build the caissons. This dock will either have to be dredged or a floating dock is required since there are no shipyards or slipways near the port site suitable for the caissons to be built.

- Specialist techniques would be required to prepare the rock bed and subsequently place the caissons.

- During construction the caissons are fully exposed to tide and wave action. With the Duqm tidal range and waves, conditions are highly unfavourable for an accurate and swift placement of the caissons.

Rock mound breakwater

The stability of the exposed slope of the mound depends on the ratio between load and strength i.e. wave height on one hand and size and relative density of the elements on the other hand. For several slopes mean rock sizes have been determined using the Van der Meer approach based on irregular waves (1998)[Lit 23], see Appendix V.

Seaward slopes	W_{50} (t)	
	8-hour storm	10-hour storm
1 : 2	37	40
1 : 2.5	27	28
1 : 3	20	22

Table 4-1 primary rock armour sizes main breakwater

Because of geological limitations of local quarries it is not possible to obtain blocks of quarry stone with sufficient weight to be stable.

Bermed breakwater

Bermed breakwaters allow some deformation of the slope without endangering the integrity of the structure as a whole. This diminishes the required rock sizes significantly. Armour units move out of their place and find a more stable position within the same cross-section. The longshore transport of armour units should be limited. Burchard and Frigaard (1988) give some recommendations on minimum armour sizes so the longshore transport remains within reasonable limits. Their approach has been used to determine minimum rock sizes (not valid for the breakwater head)[Lit 7].

For a bermed breakwater the minimum recommended size of armour units is 15 ton. Local quarries can not be used to obtain blocks of quarry stone of sufficient weight.

Concrete armoured breakwater

When local quarries cannot produce blocks of the required size, concrete armoured breakwaters are a good and frequently used substitute. Lots of different concrete armoured units are available. Two main types can be distinguished to achieve stability: weight and interlock. The interlock type is most suitable for Duqm[Lit 7].

Interlocking types that could be used at Duqm are the Tetrapod (double layer), Accropode and Core-Loc[Lit 5] (single layer). The decision which type is most appropriate for Duqm depends on the costs. The amount of concrete is decisive for the costs. The design wave height has been used to calculate the size of the armour units, see Appendix V. To provide a fair and transparent comparison the volume of concrete per m^2 of the breakwater slope is also presented, Table 4-2.

Armour unit	Size (t)	Number of units per m^2	Concrete volume (m^3/m^2)	Comparison (%)
Tetrapod	29	0.194	2.34	100
Accropode	17	0.167	1.18	50
Core-Loc	13	0.182	0.98	42

Table 4-2 comparison of concrete armour units

The double layered Tetrapod units require twice as much concrete as single layered armour units and are therefore, from an economic point of view, considered unsuitable. Compared to Accropode units the Core-Loc units will save up to 17% of concrete. The limited size of the Core-Loc units makes them easier to produce and handle. Since royalty payments (per m^3) for both units are equal the Core-Loc armour unit is most favourable for Duqm.

4.2.3 Breakwater design: main breakwater

Armour layer

The major part of the main breakwater is in relatively deep water and under severe wave attack. Therefore same size armour units will be used along the full length of the breakwater. The round head of the breakwater needs special attention; larger armour units will be needed. The Hudson formula has been used for the design of stable weight units, with breaking wave conditions and a no-damage criterion (less than 2% displacement), Table 4-3.

Primary armour sizes		
	Trunk	Head
Stability coefficient[Lit 5], K_d	16	13
Structure slope	1 : 1.5	1 : 1.5
Weight of individual armour unit (ton)	12.8	15.7

Table 4-3 Core-Loc primary armour sizes, using the Hudson formula

12.8 ton Core-Loc armour units are recommended for the main breakwater. The head will be protected with 15.7 ton armour units, Table 4-4, Appendix VI-a.

Recommended primary armour unit			
		Trunk	Head
Stable weight of individual armour unit	ton	12.8	15.7
Volume of armour unit	m^3	5.4	6.7
Layer thickness	m	2.7	2.9

Table 4-4 Core-Loc recommended armour unit, trunk and head

Crest

The crest level of the breakwater has been determined under consideration of overtopping conditions on a 1 in 100 year return period. On basis of investigations by Gerloni (1991) and interpretation by Van der Meer (1993) overtopping rates have been limited to 10 l/s/m. Guidelines and formula by HR Wallingford Ltd (1999)[Lit 2] have been used to calculate the necessary crest height, Table 4-5.

The Core-Loc armour units require a minimum shoulder width. The width is dependent on whether or not a cap-block is used. A parapet has been considered. Because of the necessary minimum berm width, required with Core-Loc units, a parapet does not give a substantial reduction of the overtopping rates.

Deviations from one location to another can be expected along the breakwater. It is assumed that during design lifetime settlements of subsoil (bedrock) and breakwater are negligible. Because of the expected deviations some tolerance has been taken into account in the design.

Main breakwater			
Crest wall / cap-block		Yes	No
Design water level	m + CD	3.1	3.1
Significant wave height, H_s	m	6.7	6.7
Mean wave period, T_s	s	9.2	9.2
Discharge	l/s/m	9.9	9.5
Shoulder width (to cap-block)	m	5.2	10.4
Freeboard on top of slope relative to SWL	m	10.4	9.6
Tolerance	m	0.30	0.30
Crest height, relative to CD	m	13.8	13.0

Table 4-5 height and width of crest, main breakwater

Considerations concerning the use of a cap-block (crest wall):

- A crest wall allows access along the structure and provides a platform for maintenance.
- The primary armour stops at the crest wall, instead of being carried over the crest to the leeward slope.
- The width of the shoulder will be smaller; this will give a reduction on materials.
- The freeboard relative to CD will be 0.8m higher i.e. an increase in use of material.

The advantages outweigh the disadvantages; therefore the crest level will be at CD +13.8m, with a crest wall (cap-block) to support the primary armour units. Calculations on the necessary freeboard can be found in Appendix VI-b.

With considerable amounts of water overtopping the structure or when the core of the structure is very permeable and the wave period is very long, wave transmission is possible. Wave transmission is the phenomenon in which wave energy passing over and trough a breakwater creates a reduced wave action in the lee of the structure. With the relatively high crest level and the wide berm, wave transmission is negligible.

First under-layer

The layer under the armour layer is called the first under-layer. The weights in the under-layer should not be less than 1/10 of the weight of the primary armour. Compared to the filter rules of Terzaghi this may seem a very strict rule, and can even be contradictory. However, one must remain conservative because of the consequences of failure of the filter mechanism. Quarry stone with a grading (Dutch standard) of 3 to 6 ton will be used for the first under-layer. The layer thickness will be $2*D_{n50}$.

These 3 to 6 ton quarry stones will also be used as armour on the leeward side of the main breakwater to ensure protection against overtopping and ship waves.

A second under-layer, consisting of 0.3 to 1.0 ton stones, between the first under-layer and the core is necessary.

Core

Standard class of rock grading with a range of 10 to 200 kg will be used.

Filter, toe berm and scour protection

Large pressure gradients may exist and wash out the seabed material, specifically under the seaward toe. The loss of bed material directly in front of the toe may cause a soil mechanical stability problem.

To ensure stability a toe berm is implemented in the design. The toe berm is the lower support for the armour layer. The weight of quarry stones that is recommended for the toe berm has been determined on basis of investigations by Gerding (1995). The weight of the stones used as first under-layer satisfies as toe material. The toe berm is placed on a bed of core material.

Dependent on the soil conditions a filter and/or scour protection may be needed (in case of soft material). For a typical cross-section of the main breakwater see Figure 4-1.

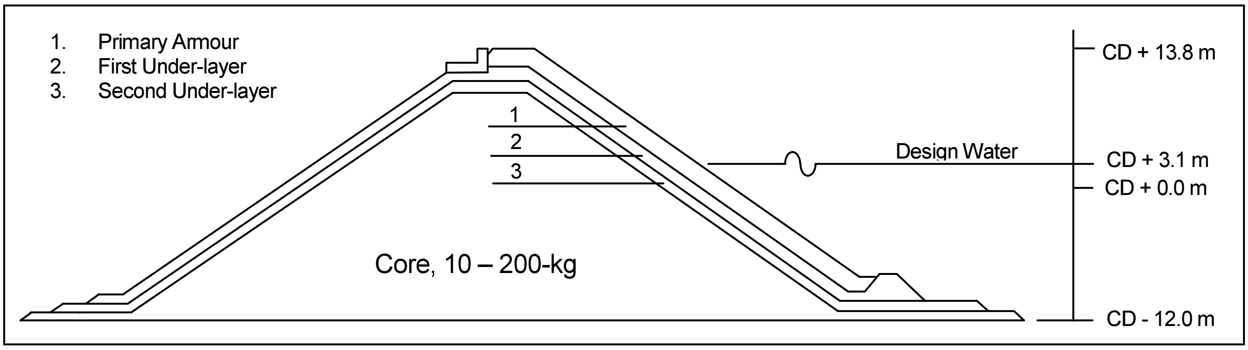

Figure 4-1 typical cross-section main breakwater

4.2.4 Breakwater design: lee breakwater

Design criteria

Because of decreasing depth landwards, wave conditions vary considerably along the lee breakwater. For the most severe wave attack the lee breakwater gets some shelter from the main breakwater. The lee breakwater has been split-up in 8 sections. The inshore wave climate has been determined for a return period of 1 in 100 year, Table 4-6.

The wind set-up in front of the lee breakwater will be limited with a maximum expected set-up of 0.26m. The design water level will be CD + 2.8m.

Bottom level (m – CD)	Depth (m)	Wave height (m)	Wave period (s)
2	4.8	3.74	6.65
3	5.8	4.52	6.93
4	6.8	4.74	7.16
5	7.8	4.74	7.42
6	8.8	4.74	7.60
7	9.8	4.74	7.80
8	10.8	4.74	7.97
9 (head)	11.8	4.74	8.09

Table 4-6 inshore wave climate lee breakwater, 1 in 100 year return period

Armour layer

The lee breakwater is less exposed to wave attack. Different slopes have been considered for the lee breakwater. Steeper slopes save material but require larger stones as primary armour units to be stable. Mild slopes (angle exceeding 1:3) at the end of the breakwater are undesirable, because it could affect navigation safety.

Where possible quarry rock will be used as primary armour. The parts of breakwater in deeper water will be armoured with Core-Loc units. To determine the weight of the Core-Loc units the Hudson formula has been used. The van der Meer formula is best suited to estimate the weight of rock armour units, Table 4-7, Appendix VI-c.

Location along breakwater (m)		Storm	Slope	Armour weight W_{50}	Primary armour	Recommended (Dutch grading)
Start – 2.0	**S1**	8-hr.	1 : 3	3.7 t	Quarry stone	3 – 6 t
		10-hr.		4.0 t		
2.0 – 7.0	**S2**	8-hr.	1 : 3	7.6 t	Quarry stone	6 – 10 t
		10-hr.		8.1 t		
7.0 – 9.0	**S3**		1 : 1.5	4.5 t	Core-Loc	5.6 t
Head				5.6 t		

Table 4-7 primary armour lee breakwater

Crest height

The crest level of the breakwater has been determined under consideration of overtopping conditions on a 1 in 100 year return period. Overtopping rates have been limited to 10 l/s/m, Table 4-8. A crest wall will be implemented, see Appendix VI-d.

Location along breakwater		**S1**	**S2**	**S3**
Depth relative to CD	m – CD	**Start – 2**	**2 – 7**	**7 – head**
Design water level	m + CD	2.8	2.8	2.8
Significant wave height, H_s	m	3.74	4.74	4.74
Mean wave period, T_s	s	6.65	7.80	8.09
Discharge	l/s/m	8.8	9.5	9.8
Shoulder width (to crest wall)	m	3.9	4.7	4.0
Freeboard on top of slope relative to SWL	m	2.6	3.8	5.7
Tolerance	m	0.3	0.3	0.3
Crest height, relative to CD	m	5.7	6.9	8.8

Table 4-8 height and width of crest, lee breakwater

The portside of the lee breakwater will be armoured with 1 to 3 ton quarry stones on a 1:1.5m slope, to protect washing out of core material due overtopping and ship waves.

First under-layer

The weights in the under-layer should not be less than 1/10 of the weight of the primary armour. Quarry stone with a grading (Dutch standard) of 0.3 to 1.0 ton will be used for the first under-layer under sections S1 and S3. Under section S2 stones with a grading ranging from 0.6 to 1.0 ton will be used.

Core

Standard class of rock grading with a range of 10 to 200 kg will be used.

Toe berm, filter and scour protection

Along most part of the lee breakwater the depth is equal to the breaker depth. To ensure stability of the toe the armour units are placed in a dredged trench. This will anchor them and prevent them from moving seawards under pressure of the breaking waves. For the rock armour this means a width of at least $3*D_{n50}$.

Dependent on the soil conditions a filter and/or scour protection may be needed (in case of soft material). When quays are immediately behind the breakwater a geotextile will be placed on the inner slope of the breakwater.

4.2.5 Lower bound breakwater design

Main breakwater

A margin of 20% has been set as lower bound on offshore wave and wind conditions. The inshore wave climate and wind set-up have been determined, Table 4-9.

Offshore		
Deep water wave height	m	5.0
Mean wave period, T_s	s	10.3
Inshore		
Significant wave height, H_s	m	5.3
Wind set-up	m	0.27
Design water level	m + CD	2.9

Table 4-9 lower bound design water level and wave height

With a decrease in the significant wave height, required weight of rock as primary armour units decreases to 12 ton. Nevertheless it is still not possible to obtain blocks of quarry stone with sufficient weight to be stable. Core-Loc units will be used as primary armour. The Hudson formula has been used for the design of stable weight units, with breaking wave conditions and a no-damage criterion (less than 2% displacement), Table 4-10 and Appendix VII-a.

Primary armour sizes		
	Trunk	Head
Stability coefficient, K_d	16	13
Structure slope	1 : 1.5	1 : 1.5
Weight of individual armour unit (ton)	6.3	7.8

Table 4-10 lower bound primary armour sizes

The crest level of the breakwater has been determined under consideration of overtopping conditions on a 1 in 100 year return period. Overtopping rates have been limited to 10 l/s/m, Appendix VII-b. A cap-block has been implemented in the design, Table 4-11.

Main breakwater		
Design water level	m + CD	2.9
Significant wave height, H_s	m	5.3
Mean wave period, T_s	s	8.6
Discharge	l/s/m	9.6
Shoulder width (to cap-block)	m	4.2
Freeboard on top of slope relative to SWL	m	8.2
Tolerance	m	0.3
Crest height, relative to CD	m	11.4

Table 4-11 lower bound height and width of crest, main breakwater

The portside of the main breakwater will be protected with stones of 1.0 to 3.0 ton. Quarry stone with a grading of 0.6 to 1.0 ton will be used as first under-layer. The core, situated directly under the first under-layer will consist of quarry stones with a grading of 10 to 200 kg. A standard toe structure, similar to the one used for the original main breakwater design, has been applied.

Lee breakwater

The inshore wave climate in front of the lee breakwater has been determined for a return period of 1 in 100 year. The maximum wave height is 3.9m. The wind set-up in front of the lee breakwater will be limited with a maximum expected set-up of 0.15m. For the design of the lower bound lee breakwater an extreme water level of CD + 2.7m has been adopted.

With the limited maximum wave height and wave period the lee breakwater can be protected with rock armour units along its full length, Table 4-12 and Appendix VII-c.

Location along breakwater (depth – CD, m)		Storm	Slope	Armour weight W_{50}	Primary armour	Recommended (Dutch grading)
start – 2.0	**S1**	8-hr.	1 : 3.0	3.6 t	Quarry stone	3 – 6 t
		10-hr.		4.3 t		
2.0 – head	**S2**	8-hr.	1 : 2.5	6.8 t	Quarry stone	6 – 10 t
		10-hr.		7.3 t		

Table 4-12 Primary armour on lower bound lee breakwater

The crest level of the breakwater has been determined under consideration of overtopping conditions on a 1 in 100 year return period. Overtopping rates have been limited to 10 l/s/m. A cap-block has been implemented in the design, Table 4-13. The portside of the breakwater will be armoured with 1 to 3 ton quarry stones on a 1:1.5 slope.

Location along breakwater		**S1**	**S2**
Depth relative to CD	m – CD	**Start – 2.0**	**2.0 – head**
Design water level	m + CD	2.7	2.7
Significant wave height, H_s	m	3.74	3.93
Mean wave period, T_s	s	6.55	8.01
Discharge	l/s/m	9.6	9.6
Shoulder width (to crest wall)	m	4.7	4.7
Freeboard on top of slope relative to SWL	m	3.2	4.5
Tolerance	m	0.3	0.3
Crest height, relative to CD	m	6.2	7.5

Table 4-13 crest height, lower bound lee breakwater

Quarry stone with a grading of 0.6 to 1.0 ton will be used as first under-layer. The core, situated directly under the first under-layer will consist of quarry stones with a grading of 10 to 200 kg. A toe structure, similar to the one used for the original lee breakwater design, has been applied.

4.2.6 Upper bound breakwater design

Main breakwater

A margin of 20% as upper bound of offshore wave and wind conditions has been set. The inshore wave climate and wind set-up have been determined, Table 4-14.

Offshore		
Deep water wave height	m	7.5
Mean wave period, T_s	s	12.7
Inshore		
Significant wave height, H_s	m	7.95
Wind set-up	m	0.60
Design water level	m + CD	3.3

Table 4-14 upper bound design water level and wave height

Core-Loc units will be used as primary armour. The Hudson formula has been used for the design of stable weight units, with breaking wave conditions and a no-damage criterion (less than 2% displacement), Table 4-15 and Appendix VIII-a.

Primary armour sizes		
	Trunk	Head
Stability coefficient, K_d	16	13
Structure slope	1 : 1.5	1 : 1.5
Weight of individual armour unit (ton)	21.4	26.3

Table 4-15 upper bound primary armour sizes

The crest level of the breakwater has been determined under consideration of overtopping conditions on a 1 in 100 year return period. Overtopping rates have been limited to 10 l/s/m, Appendix VIII-b. A cap-block has been implemented in the design, Table 4-16.

Main breakwater		
Design water level	m + CD	3.3
Significant wave height, H_s	m	7.95
Mean wave period, T_s	s	9.7
Discharge	l/s/m	9.4
Shoulder width (to cap-block)	m	6.3
Freeboard on top of slope relative to SWL	m	12.4
Tolerance	m	0.3
Freeboard, relative to CD	m	16.0

Table 4-16 upper bound height and width of crest, main breakwater

The first under-layer consists of 3.0 to 6.0 ton quarry stones. These 3 to 6 ton quarry stones will also be used as armour on the leeward side of the main breakwater to ensure protection against overtopping and ship waves. A second under-layer, 0.3 to 1.0 ton, will be necessary. The core material, situated under the second under-layer consists of 10 to 200 kg ranging quarry stones. A standard toe structure, similar to the one used for the original main breakwater design, has been applied.

Lee breakwater

The inshore wave climate in front of the lee breakwater has been determined for a return period of 1 in 100 year. The maximum wave height is 5.9m. The wind set-up in front of the lee breakwater will be limited with a maximum expected set-up of 0.24m. For the design of the upper bound lee breakwater an extreme water level of CD + 2.9m has been adopted.

Only the first part of the upper bound lee breakwater can be protected with quarry stones, Appendix VIII-c. Rest of the breakwater will be protected with Core-Loc armour units, on the head heavier units are necessary to be stable, Table 4-17.

Location along breakwater (depth – CD, m)		Storm	Slope	Armour weight W_{50}	Primary armour	Recommended (Dutch grading)
Start – 3.0	**S1**	8-hr.	1 : 3.0	7.0 t	Quarry stone	6 – 10 t
		10-hr.		7.5 t		
3.0 – 8.5	**S2**		1 : 1.5	9.7 t	Core-Loc	9.7 t
Head	**S3**		1 : 1.5	11.9 t	Core-Loc	11.9 t

Table 4-17 Primary armour on upper bound lee breakwater

The crest level of the breakwater has been determined under consideration of overtopping conditions on a 1 in 100 year return period. Overtopping rates have been limited to 10 l/s/m. A cap-block has been implemented in the design, Table 4-18. The portside of the breakwater will be armoured with 1 to 3 ton quarry stones on a 1:1.5m slope.

Location along breakwater		**S1**	**S2, S3**
Depth relative to CD	m – CD	**Start – 3.0**	**3.0 – head**
Design water level	m + CD	2.9	2.9
Significant wave height, H_s	m	4.60	5.90
Mean wave period, T_s	s	7.68	9.02
Discharge	l/s/m	0.90	9.9
Shoulder width (to crest wall)	m	4.7	4.8
Freeboard on top of slope relative to SWL	m	4.4	9.2
Tolerance	m	0.3	0.3
Crest height, relative to CD	m	7.6	12.4

Table 4-18 crest height, upper bound lee breakwater

Quarry stone with a grading of 0.6 to 1.0 ton will be used as first under-layer under section S1. Under the Core-Loc units an under-layer of 1.0 to 3.0 ton grading will be used. The core, situated directly under the first under-layer will consist of quarry stones with a grading of 10 to 200 kg. A toe structure, similar to the one used for the original lee breakwater design, has been applied.

4.3 Preliminary quay wall design

4.3.1 Introduction

Generally, in quay wall design the most important factors in determining the optimum structure type at a given location are the subsoil conditions and water depth. In Section 4.3.2 different structure types will be discussed.

After some considerations about corrosion, Section 4.3.3, two structure types were adopted for further design. A design is presented for the 10, 8 and 5m depth quays, Sections 4.3.5 and 4.3.6.

Design criteria

From the Draft Final Report[Lit 19] by Posford Haskoning the following quay dimensions have been derived, Table 4-19.

Depth (m – CD)	Length (m)	Height (m + CD)
– 5	300	5
– 8	600	5
– 10	600	5

Table 4-19 lengths of –10, –8 and –5m depth quay walls

4.3.2 Quay wall options

Subsoil conditions and water depth are important factors in determining the optimum structure type. Two main structure types can be distinguished: solid and open-piled quay wall structures.

Solid quay wall structures can be divided in two main types: gravity based structures and retaining structures. Three types of gravity walls (block-work wall, caisson wall and cellular sheet pile wall), one type of retaining walls (bulkhead) and one type of open-piled structures (suspended deck) will be discussed in this Section.

Caisson wall

Caissons are built in a building yard. Either the whole caisson or the base of the caisson is constructed in the dry. The construction yard must be kept dry until the structures are ready to float. A special dock is needed to build the caissons. After completion the caissons are floated to the port site and sunk in their place. A bed of rock will be needed for the caissons to be stable. Caisson quay wall structures are especially suitable for retaining heavy loads and high vertical banks. There are a number of drawbacks on the use of a caisson quay wall:

- A special dock is needed to build the caissons. This dock will either have to be dredged or a floating dry-dock is required since there are no shipyards or slipways near the port site suitable for the caissons to be built.
- Specialist techniques would be required to prepare the rock bed and subsequently place the caissons.
- Risk of overturning failure from overdredging or scour.
- Threat of corrosion to the reinforcement steel

Block-work wall

Block-work walls are built up with either natural stone blocks (nowadays not anymore) or non-reinforced concrete blocks. The dimension of the blocks must be determined in consideration of the available construction materials, the fabrication potentials, the capacity of the cranes to place the blocks and placing and operating conditions. The wall can be constructed both water-born and land-based.

Block-work structures can be built successfully only if load-bearing soil is present below the base of the foundation. Reliability, in terms of proven track record and long term stability is very high and maintenance is limited to a minimum. Disadvantages of a block-work quay wall are:

- Specialist techniques would be required to prepare the rock bed and place the blocks.
- Huge amounts of concrete would be required. This could result in relatively high construction costs.
- Risk of overturning failure from overdredging or scour.

Cellular sheet pile wall

A cellular sheet pile wall is a composition of sheet piles and fill-material that acts as a gravity wall. The piles are driven in a cellular shape. All cells, which are connected by small diaphragms, are constructed individually and filled, and are therefore independently stable. The nature of cell fill-material must be carefully specified.

Cellular sheet pile walls offer the advantage that they can be designed as stable gravity walls without anchoring. There are some drawbacks on the use of cellular sheet pile walls:

- They are very vulnerable regarding potential damage that may occur following a vessel impact.

- Compared to other solutions they are less reliable.

- Vulnerable to corrosion, resulting in a short design lifetime and high maintenance costs.

Dependent on corrosion rates, a cellular steel sheet pile wall could be a suitable solution, see Section 4.3.3.

Bulkhead

Bulkheads are vertical wall structures typically constructed of steel, concrete or timber sheet piles anchored by tie rods to anchor walls. Anchored bulkheads derive their support from a combination of passive earth support and anchoring. Duqm soil conditions preclude timber or concrete sheet pile walls.

With limited retaining heights steel sheet piling is frequently an ideal solution both as regards structural considerations and driving conditions. When hard grounds or rock are to be expected (Duqm) driving conditions may require heavier piles than derived from design considerations. Two important advantages of steel sheet pile walls are:

- In general construction costs are relatively low. Although with hard grounds costs can increase dramatically.

- Some maintenance dredging in front of the structure is allowed without endangering the integrity of the structure as a whole.

Corrosion rates have to be investigated to check the suitability of steel sheet pile walls, see Section 4.3.3.

Suspended deck

Open-piled structures have a suspended deck supported on piles. The structure can either be flexible with only vertical piles and without external horizontal restraint or constructed more rigidly with raking piles or with struts to the shore. A flexible type of structure may be unsuitable since it has to accommodate cranes and bulk handling equipment.

Suspended deck structures may be of steel, concrete or timber. They will usually be the most suitable type in circumstances when ground consisting of weak upper materials overly a stronger stratum or ground immediately below seabed consists of suitable material for bearing piles. They become increasingly competitive with bulkheads or other fill-type structures for greater water depths. Duqm ground conditions preclude timber or concrete piles.

Corrosion rates have to be investigated to check the suitability of steel piles, see Section 4.3.3.

4.3.3 Corrosion of steel sheet piles

When in contact with water and at the same time in the presence of oxygen, steel is subject to corrosion. Material abrasion depends on local conditions as well as on the position (vertical) regarding the waterline. The degree of corrosion intensity is the decrease in wall thickness (rusting in mm). The following consequences, as a result of corrosive action, are to be taken in consideration [Lit 4, 22, 6]:

- Decrease in thickness results in reduced bearing / retaining capacity.
- Local corrosion through the piles can occur. The soil behind the piles can be flushed out causing settlement damage.

There are a number of guidelines on corrosion in temperate climates. The main corrosion in marine environments in the main attack zone is about 0.12 mm/y. There are cases where reductions of over 0.5 mm/y have been experienced. For warmer areas these figures should be adjusted upwards.

In addition there are other corrosion accelerating mechanisms which occur in marine environments. One of them is Accelerated Low Water Corrosion (ALWC). This phenomenon appears to be the result of bacterial respiration processes. Corrosion rates to be expected are as least as high as 1.0 mm/y.

There are some protection measures which can reduce corrosion rates or delay the start of corrosion:

- Corrosion protection from coatings. Coatings can delay the start of corrosion and are especially effective as protection for reinforcement steel.
- Cathodic corrosion protection. Corrosion under the water line can be substantially eliminated by electrolytic means by the installation of cathodic protection. Cathodic corrosion protection is widely used to protect offshore structures.
- Alloy additives. Alloy additives do not protect against corrosion in the submerged zone. However in and above the splash zone lifetime can be increased.
- Corrosion protection by oversize. With regard to exceeding the retaining capacity it is possible to select oversized profiles.
- Use of higher yield steel. If with the same section modulus S355 steel would be used, instead of S270 steel, lifetime would increase with 30%.

Post construction protection measures or complete renewal are extremely difficult and are usually accompanied with disproportionate high costs.

Corrosion implications for Duqm

The Duqm conditions with high salinity seawater and high temperatures are most unfavourable; a mean rusting speed of 0.14 mm/y is to be expected. In the marine environment of Oman accelerated low water corrosion is likely to occur, increasing the mean rusting speed up to 1.0 mm/y. When rating the rusting speed the design period must also be stated, the design lifetime for the quay walls is 50 years.

When comparing retaining heights at Duqm and similar other quay wall structures, sheet piles with a leg thickness of 14mm would satisfy. Without any protection measurements the sheet piles would just have been rusted through after only 14 years.

In discussion with specialists it appeared that a cathodic protection at Duqm is not feasible from a practical and economic point of view. The use of alloy additives will not give protection at a location on the sheet piles where corrosion is most severe. Protection with special coatings will delay the corrosion process. There are a number of different coatings available; start of corrosion can be delayed with about 4 years. The last two mentioned protection measures are comparable. Either one of them could be applied.

With the use of special coatings (increase of lifetime with 4 years) and higher yield steel (30 % increase in lifetime) the piles will be rusted through after 23 years. When the thickness of the sheet piles decreases, the retaining capacity of the sheet piles decreases also. With a decreased pile thickness the critical load value will probably already have been exceeded before the piles are completely rusted through. Therefore it is concluded that steel structures are unsuitable for the quay walls at the Duqm port.

4.3.4 Selection of preferred quay wall option

With the exclusion of steel structures, the only options left are either a caisson wall or a block-work wall. Their behaviour and reliability are similar:

- They are designed in such a way the structure acts as a solid block.
- Specialist techniques would be required to prepare the rock bed and place the blocks or caissons.
- Risk of overturning failure from overdredging or scour.

The major difference between both solutions is the production process of the construction elements.

A caisson solution would require a floating dry-dock to build the caissons in. A caisson could also be vulnerable to corrosion. Good building practice and special coatings on the reinforcement steel should give the caisson structures sufficient protection against corrosion.

The block-work wall is constructed of non-reinforced concrete blocks. These blocks are constructed on land and placed by floating cranes. In the past block-work walls have encountered problems in the Arabian region because of concrete blocks of poor quality.

The costs will be decisive in the selection process to find the most suitable solution. Therefore both the caisson and the block-work wall will be adopted for further design.

4.3.5 Quay wall design: caisson wall

In the design process of caisson three important stages can be identified, the stability of the caisson quay wall, floating stability of the caissons and strength of the individual caissons. To determine the dimensions for the caissons, for the different retaining heights, the following iterative design cycle has been used:

First the overall stability of the placed caissons is reviewed, meeting a number of stability criteria (sliding, overturning, deep slip). Then the floating stability of the caissons is checked. Finally the strength of the concrete walls is checked.

Calculations on (floating) stability and concrete strength can be found in Appendix IX-a.

Caissons

The caissons are designed so that, after placing, the top is just above low water level. The advantage is that filling and finishing work can be done ‘in the dry’ with caisson height limited to minimum. The base of the caisson will be 1m below the required basin depth, taking account of dredging. The thickness of the maintenance-dredging zone is 0.5m, the dredging tolerance is set on 0.5m[Lit 22].

Inside the caissons walls have been placed. This will spread the loads on the external walls. This will also improve the floating stability of the caissons. In the 10 and 8m depth caissons 3 walls parallel to the front wall are implemented in the design; in the 5m depth caissons 2 walls. Perpendicular to the front wall 7 internal walls per caisson are used.

The cells of the caissons are filled with sand. The seaward compartments are filled with lean-mix concrete because conditions are such that the front wall could seriously deteriorate and to provide increased resistance to vessel impacts[Lit 3].

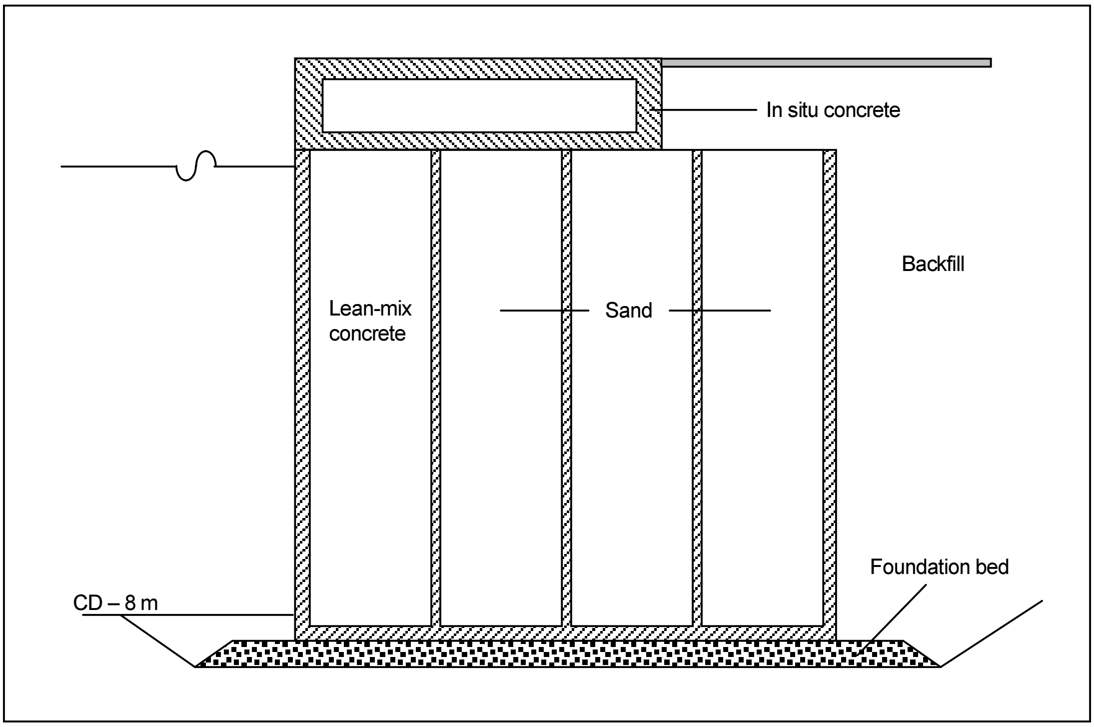

Figure 4-2 typical cross-section caisson quay wall, CD – 8m

The superstructure consists of a solid in situ capping which is back-filled. The in situ capping will be cast after the caissons have been filled and the joints completed. In situ concrete keyed joints are used, placed within vertical recesses formed in the outer walls of each caisson.

The caissons are placed on a foundation bed of small graded quarry stone, 1m thick. The bed must be carefully levelled with special gear and the aid of divers.

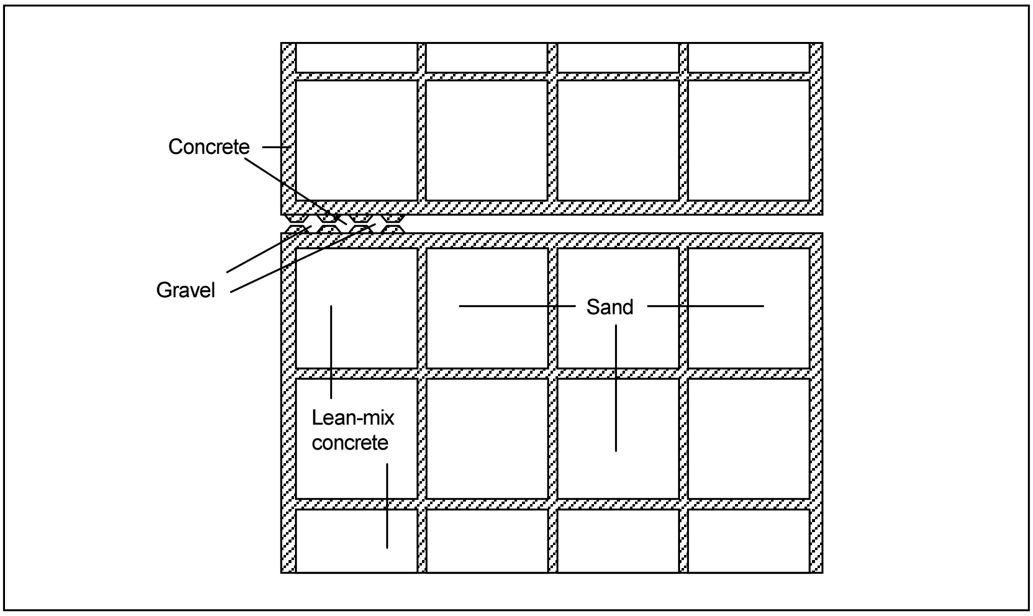

Figure 4-3 caisson-view from above with joints

Stability of caisson wall

- Sliding. A factor of safety of 1.5 has been applied[Lit 13]. In the design passive resistance in front of the toe has not been taken into account because of the possibility of material being removed by scour or dredging.

- Overturning. A safety factor of 1.3 has been applied. With their rectangular shape and equal spread of pressure overturning is not a critical factor of safety.

- With the assumption of the presence of rock deep slip circles are unlikely to occur.

Floating stability caissons

The caissons will have to be dynamically stable during transport. When the metacentric height is below the mass centre of the concrete, relative to the bottom of the caisson, the caisson can easily tilt. Tugs, waves and currents can cause this instability.

This instability can easily be prevented by adjustment of the design, for example by increasing the width of the caissons (increase of moment of inertia) or increasing the mass of the caisson floor (lowering the concrete mass centre). There are also some additional measurements preventing tilt, for example by placing ballast on the bottom of the caisson, using stability pontoons or joining two caissons during transport.

The –5m and the –8m caissons are dynamically stable, see Appendix IX. The –10m caissons will be weighted with ballast. Normally solid ballast is advantageous (e.g. sand). When water is used the liquid ballast starts moving if the caisson gets a little tilt so the centre of gravity shifts, increasing the tilt. In the design internal walls have been implemented, this will minimise the shift of the centre of gravity, making water a suitable ballast.

Concrete strength

The last stage in the preliminary caisson design is to verify the thickness of the internal and external walls and bottom. In the calculations account has been taken of a cover layer on the concrete and some extra tolerance because of the inability to do proper maintenance once the quay structure has been completed.

		Caissons		
		-5m	-8m	-10m
Caisson dimensions				
- Height	m	7.5	10.5	12.5
- Width	m	7.0	8.5	10.0
- Length	m	20	20	20
Wall thickness				
- External walls	mm	400	400	510
- Internal walls	mm	300	300	300
- Bottom	mm	500	500	500
Number of caissons		15	30	30

Table 4-20 caisson dimensions

4.3.6 Quay wall design: block-work wall

Block-work

The maximum weight of the blocks is limited by the load capacity of the block-placing crane. A decrease in dead load of the blocks means an increase in the working radius of the crane. But the blocks must be sufficiently large to withstand the expected stresses. The maximum block weight is limited to 200 tons. The blocks have been designed to the recommendations of British Standard 6349, part 2[Lit 3] and some rules of thumb[Lit 13].

Dimensions of the blocks are determined after an iterative process, satisfying a number of stability criteria:

- Sliding. A factor of safety of 1.5 has been applied[Lit 13]. In the design passive resistance in front of the toe has not been taken into account because of the possibility of material being removed by scour or dredging.

- Overturning. A factor of safety of 1.3 has been applied.

- Deep slip. Deep slip circles are not likely to occur when the subsoil consists of rock.

The resultant of the forces, from blocks and surcharge, must be in the middle third of the base to reduce the point bearing pressure. In the initial design an overdredge of 0.5m has been taken into account for all quay walls. Calculations for determination of the block dimensions can be found in Appendix X-a.

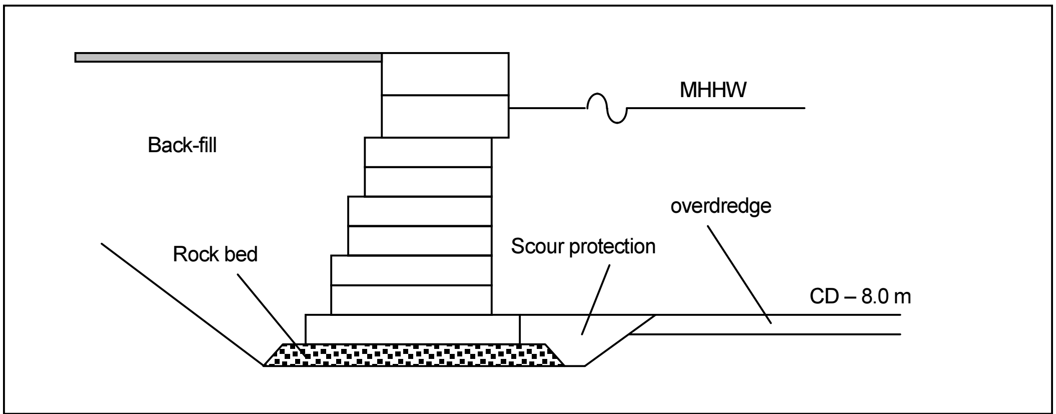

Figure 4-4 typical cross-section block-work quay wall, CD – 8m

To provide efficient utilisation of formwork, same sized blocks have been used for the different quay walls. The blocks are stacked in vertical columns. The main advantage of this way of construction is that it can accommodate differential settlement.

Each block column is 5m wide. With increasing depth the length of the blocks increases. To distribute the ground pressures at the foundation joint as uniformly as possible the base block has a spur of 1m (from the edges of the block above) on front and backside.

The blocks are provided with a tongue and groove interlock, see Figure 4-5. The blocks are placed with the tongue uppermost so that the groove of succeeding blocks can be engaged and the blocks slid into place. To improve the resistance to sliding the bottom of the base block will be made rough, in order to create a serrated effect.

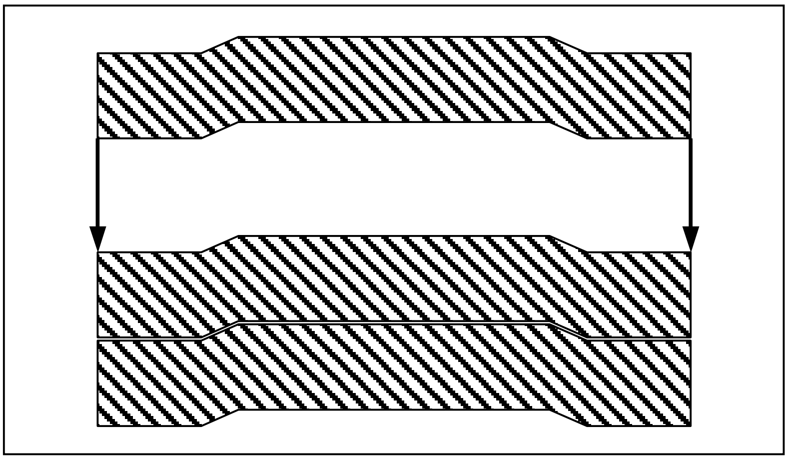

Figure 4-5 tongue and groove interlock

	- 5m		- 8m		- 10m	
	Dimensions h x l (m^2)	Weight (ton)	Dimensions h x l (m^2)	Weight (ton)	Dimensions h x l (m^2)	Weight (ton)
Block 1	2.0 x 5.5	132	2.0 x 6.5	156	2.0 x 6.5	156
Block 2	2.0 x 5.5	132	2.0 x 6.5	156	2.0 x 6.5	156
Block 3	1.5 x 5.5	99	1.5 x 6.5	117	2.0 x 6.5	156
Block 4	1.5 x 5.5	99	1.5 x 6.5	117	1.5 x 7.0	126
Block 5	1.5 x 5.5	99	1.5 x 7.0	126	1.5 x 7.0	126
Block 6	1.5 x 5.5	99	1.5 x 7.0	126	1.5 x 7.5	135
Block 7	1.0 x 7.5	90	1.5 x 7.5	135	1.5 x 7.5	135
Block 8	-	-	1.5 x 7.5	135	1.5 x 8.0	144
Block 9	-	-	1.0 x 9.5	114	1.5 x 8.0	144
Block 10	-	-	-	-	1.0 x 10	120

Table 4-21 block dimensions and weights

Foundation bed

A foundation bed of small graded quarry stone, 1m thick, will be placed between the subsoil and the base block. The bed must be carefully levelled with special gear and the aid of divers.

Scour and erosion

The effects of ships' propellers and bow-thrusters and waves on the stability of the seabed near structures often lead to the failure of block-work walls. To ensure stability the base block and foundation bed are placed in a trench. In the initial design an overdredge of 0.5m has been taken into account. The base of the block column is 0.5m below this overdredge depth. After placing the foundation bed and the quay wall blocks, the trench will be filled up with stones. These stones are assumed to act as scour protection. Scour is considered to be likely along the full length of the quay walls.

The subsoil in front of the quays and trench is assumed to consist of rock, scour and erosion will be negligible. Therefore no additional scour protection will be needed.

4.3.7 Lower and upper boundaries quay wall design

The assumption of rock under the quay walls satisfies the bearing capacity requirements. Besides, deep slip circles are unlikely to occur. This implies that lower and upper boundaries are not to be set on the subsoil conditions.

Besides uncertainties in the subsoil conditions there can also be uncertainties in the characteristics of the back-fill material. Upper and lower boundaries can be set on the characteristics of the back-fill material. The factor applied for active earth pressure depends on the angle of internal friction. In Table 4-22 the upper and lower bounds are presented, calculations of the lower bound quay wall dimensions can be found in Appendix IX-b respectively Appendix X-b and upper bound quay wall dimensions can be found in Appendix IX-c respectively Appendix X-c.

	Angle of Internal friction	Active earth pressure coefficient
Lower bound	30.0	0.33
Original design	32.5	0.30
Upper bound	35.0	0.27

Table 4-22 lower and upper boundaries of quay wall design

4.4 Dredging works and land reclamation

4.4.1 Dredging works

The characteristics of the material to be dredged is of uttermost importance to determine the dredging costs, the suitability of the dredged materials to be used for land reclamation and the type of dredging equipment. The soil under the site is assumed to consist of rock, covered with a layer of marine deposits.

Dredging volumes

The capital dredging works account for the development of basins, approach channel and turning circle. In addition trenches will have to be dredged for the foundation of the quay walls and for the first row of primary armour at the lee breakwater. The costs of the trench dredging will be included in the costs for the quay wall or the breakwater.

To provide a first estimation of the dredge volumes, all areas to be dredged have been split up in several parts. Per part the average depth is determined. With subtracting the design depth the volumes are known, Table 4-23. For all depths an overdredge of 0.5m has been taken into account. The method used is very coarse; the estimates of the different volumes are therefore likely to be subject to some variation. However it is likely that an overestimation may counterbalance an underestimation.

	Dredge volume (m^3)
Approach channel, –12m	**168,000**
Basin –5m quay	**94,900**
Basin –8m quay	**510,500**
Basin –10m quay and general harbour area	**7,984,200**
Approach channel, basins, turning circle	**8.8 million**

Table 4-23 volumes to be dredged

Equipment

The intended utilisation of the removed soil is also relevant to the choice of dredging equipment. The cutting and transportation process can break down the rock in small particles and make it less suitable for land reclamation. The marine deposits will most likely be dumped at sea by barges. The rock will be pumped ashore by discharge pipelines. This material can either be dumped at the reclamation area or on a temporary onshore placement site.

4.4.2 Land reclamation

Reclamation volumes

The method used to provide a first estimation of the volume to be reclaimed is comparable to the one used for the dredging works. The area to be reclaimed has been split up in smaller parts. Per part the average depth is determined. With adding the design level of the reclamation areas the volumes are known, see Table 4-24.

	Volumes (m^3)
Reclamation below MWL	1.2 million
Reclamation above MWL	2.5 million
Total volume	3.7 million

Table 4-24 volumes to be reclaimed

Soil characteristics under the reclamation area

The bedrock subsoil is covered with a layer of marine deposits. The marine deposits of recent years are too weak to use for land reclamation. The suitability of the bedrock as reclamation material is at this design stage, with very little information on the soil characteristics available, still unclear.

Suitability of dredged materials for land reclamation

The marine deposits under the reclamation areas will not have to be removed. It is assumed that long term settlements deriving from consolidation of these marine deposits are unlikely to be excessive. Most consolidation will take place during construction after placement of the reclamation material.

4.4.3 Lower and upper boundaries for dredging works and land reclamation

The costs of both dredging works and land reclamation depend principally on the soil characteristics. However the influence of the soil characteristics on the costs for dredging works and land reclamation can rather be contradictory. When dealing with rock its appears that materials showing a great resistance to dredging are usually reasonably suitable as reclaim material, while in contrast materials less resistant to dredging prove to be less suitable as reclaim material.

In the Duqm case it could well be possible that the dredged material is unsuitable as reclamation material. If the dredged materials from the port site are indeed unsuitable for reclamation, material will have to be obtained from an offshore borrow area or a land-based borrow site.

Probably those materials that appear to be more resistant to dredging, implicating higher dredging costs per m^3, are more suitable as reclamation material, implicating lower reclamation costs per m^3. Because of this dependency lower and upper bounds can not unequivocally be identified.

Besides the uncertainty of the suitability of the dredged material for land reclamation there is also no certainty about the thickness of the marine deposit layer. The thickness of this layer is assumed to vary from a few centimetres up to several meters. A lower and upper boundary on the average thickness of this marine deposit layer has been set, Table 4-25.

Marine deposit Layer thickness (m)	Marine deposits (million m^3)	Rock (million m^3)	Reclamation (million m^3)
1.0	1.2	7.6	3.7
1.5	3.1	5.7	3.7
2.0	3.6	5.2	3.7

Table 4-25 volumes of available dredged materials and required reclamation material

With a thickness of 2m of the marine deposit layer, the upper bound, there would still be enough rock available to use for the land reclamation.

5 Cost estimation preliminary design

5.1 Introduction

In this Section estimates of the costs for the port components designed in Section 4 are presented. This is also done for the lower and upper boundary designs.

To determine the costs a unit rate technique has been used. This technique is based on the traditional bill of quantities approach. During the preliminary design stage a detailed bill of quantities is not available. Instead quantities of the main items of work have been estimated and these are priced using rates that take account of the associated minor items.

The unit rate estimating technique relies on historical data. The associated risks of using historical data in estimating are important to recognise, see Section, 9.5. The rates used are based on estimates for earlier projects in the region and other projects with similar structures as Duqm. Earlier studies on following port projects provided most suitable information:

- Duqm, Oman, 2002, new port and dry-dock complex
- Aden, Yemen, 2002, flour-mill jetty
- Aden, Yemen, 2000, container terminal
- Hidd, Bahrain, 2000, new port and industrial area
- Ajman, United Arabian Emirates, 2000, Al Zora creek dredging and dry-dock complex

5.2 Cost estimation breakwater

With the above mentioned method cost estimations have been established for the main and lee breakwater. In Table 5-1, Table 5-2 and Table 5-3 are the costs for respectively the original, the lower bound and the upper bound breakwater design presented. Calculation of the volumes can be found in Appendix XI.

The amounts of material required for the breakwaters are determined by dividing the breakwater in a number of sections. Per section the amounts of material in a typical cross-section are calculated. By multiplying the amounts of material with the length of the section and adding the amounts for all sections, the amount of material needed for the breakwater is known. Heavy grading rock armour ranges from 1 up to 10 ton. The heavy and light rock armour amounts are the sum of materials required for primary armour, under-layers, toe and harbour protection.

Item	Quantity (m^3)	Rate (£)	Total (£)
Main breakwater			
Core-Loc	182,700	58	10,596,600
Rock armour, heavy grading	449,100	9	4,041,900
Rock armour, light grading	245,500	7.5	1,841,300
Core	1,227,400	5	6,137,000
Cap-block	4,900	40	196,000
Subtotal			**22.8 million**
Lee breakwater			
Core-Loc	19,100	58	1,107,800
Rock armour, heavy grading	179,700	9	1,617,300
Rock armour, light grading	52,700	7.5	395,200
Core	232,000	5	1,160,000
Geo-textile (m^2)	5,800	2	11,600
Cap-block	4,300	40	172,000
Trench dredging	39,100	6	234,600
Subtotal			**4.7 million**
Total cost breakwaters			**27.5 million**

Table 5-1 costs preliminary design, main and lee breakwater

Item	Quantity (m^3)	Rate (£)	Total (£)
Main breakwater			
Core-Loc	140,400	58	8,143,200
Rock armour, heavy grading	113,600	9	1,022,400
Rock armour, light grading	212,400	7.5	1,593,000
Rock core	1,103,200	5	5,516,000
Cap-block	4,300	40	172,000
Subtotal			**16.4 million**
Lee breakwater			
Rock armour, heavy grading	217,200	9	1,914,300
Rock armour, light grading	58,300	7.5	437,200
Rock core	246,600	5	1,233,000
Geo-textile (m^2)	6,500	2	13,000
Cap-block	4,200	40	168,000
Trench dredging	44,200	6	265,200
Subtotal			**4.0 million**
Total cost breakwaters			**20.4 million**

Table 5-2 costs lower bound preliminary design, main and lee breakwater

Item	Quantity (m^3)	Rate (£)	Total (£)
Main breakwater			
Core-Loc	243,100	58	14,099,800
Rock armour, heavy grading	497,400	9	4,476,600
Rock armour, light grading	267,200	7.5	2,004,000
Core	1,482,400	5	7,412,000
Cap-block	5,300	40	212,000
Subtotal			**28.2 million**
Lee breakwater			
Core-Loc	71,000	58	4,118,000
Rock armour, heavy grading	157,500	9	1,417,500
Rock armour, light grading	61,800	7.5	463,500
Core	404,600	5	2,023,000
Geo-textile (m^2)	7,800	2	15,600
Cap-block	4,400	40	176,000
Trench dredging	45,200	6	271,200
Subtotal			**8.5 million**
Total cost breakwaters			**36.7 million**

Table 5-3 costs upper bound preliminary design, main and lee breakwater

5.3 Cost estimation quay wall

With the unit rate technique costs have been calculated for the block-work quay wall and the caisson quay wall. In Table 5-4 to Table 5-9 costs are presented for the original, lower bound and upper bound design.

Caisson original	Item	Quantity	Rate (£)	Costs (£)
Wall – 5m	Dredging (m^3)	24,400	6	146,400
	Rock bed + levelling (m^2)	2,700	40	108,000
	Concrete + casting (m^3)	4,000	130	520,000
	Core fill (m^3)	7,800	15	117,000
	Lean mix fill (m^3)	3,900	30	117,000
	Top block (m^3)	1,800	50	90,000
	Floating, placing, finishing (no.)	15	50,000	750,000
	Subtotal			**1.8 million**
Wall – 8m	Dredging (m^3)	86,400	6	518,400
	Rock bed + levelling (m^2)	6,300	40	252,000
	Concrete + casting (m^3)	12,800	130	1,664,000
	Core fill (m^3)	30,600	15	459,000
	Lean mix fill (m^3)	10,200	30	306,000
	Top block (m^3)	3,700	50	185,000
	Floating, placing, finishing (no.)	30	62,000	1,860,000
	Subtotal			**5.2 million**
Wall – 10m	Dredging (m^3)	128,900	6	773,400
	Rock bed + levelling (m^2)	7,200	40	288,000
	Concrete + casting (m^3)	16,800	130	2,184,000
	Core fill (m^3)	43,600	15	654,000
	Lean mix fill (m^3)	14,500	30	435,000
	Top block (m^3)	3,700	50	185,000
	Floating, placing, finishing (no.)	30	74,000	2,220,000
	Subtotal			**6.7 million**
Total cost caisson quay wall, original design				**13.7 million**

Table 5-4 caisson quay wall cost estimation

Caisson lower bound	Item	Quantity	Rate (£)	Costs (£)
Wall – 5m	Dredging (m^3)	23,400	6	140,400
	Rock bed + levelling (m^2)	2,600	40	104,000
	Concrete + casting (m^3)	3,900	130	507,000
	Core fill (m^3)	71,000	15	106,500
	Lean mix fill (m^3)	3,600	30	108,000
	Top block (m^3)	1,800	50	90,000
	Floating, placing, finishing (no.)	15	46,000	690,000
	Subtotal			**1.7 million**
Wall – 8m	Dredging (m^3)	83,700	6	502,200
	Rock bed + levelling (m^2)	6,000	40	240,000
	Concrete + casting (m^3)	13,800	130	1,794,000
	Core fill (m^3)	27,500	15	412,500
	Lean mix fill (m^3)	9,200	30	276,000
	Top block (m^3)	3,700	50	185,000
	Floating, placing, finishing (no.)	30	58,000	1,740,000
	Subtotal			**5.1 million**
Wall – 10m	Dredging (m^3)	125,400	6	752,400
	Rock bed + levelling (m^2)	6,900	40	276,000
	Concrete + casting (m^3)	16,700	130	2,171,000
	Core fill (m^3)	40,900	15	613,500
	Lean mix fill (m^3)	13,600	30	408,000
	Top block (m^3)	3,700	50	185,000
	Floating, placing, finishing (no.)	30	70,000	2,100,000
	Subtotal			**6.5 million**
Total cost caisson quay wall, lower bound design				**13.3 million**

Table 5-5 lower bound caisson quay wall cost estimation

Caisson upper bound	Item	Quantity	Rate (£)	Costs (£)
Wall – 5m	Dredging (m^3)	25,300	6	151,800
	Rock bed + levelling (m^2)	2,900	40	116,000
	Concrete + casting (m^3)	4,100	130	533,000
	Core fill (m^3)	85,000	15	127,500
	Lean mix fill (m^3)	4,300	30	129,000
	Top block (m^3)	1,800	50	90,000
	Floating, placing, finishing (no.)	15	54,000	810,000
	Subtotal			**2.0 million**
Wall – 8m	Dredging (m^3)	91,800	6	550,800
	Rock bed + levelling (m^2)	6,900	40	276,000
	Concrete + casting (m^3)	13,100	130	1,703,000
	Core fill (m^3)	35,100	15	526,500
	Lean mix fill (m^3)	11,700	30	351,000
	Top block (m^3)	3,700	50	185,000
	Floating, placing, finishing (no.)	30	70,000	2,100,000
	Subtotal			**5.7 million**
Wall – 10m	Dredging (m^3)	132,300	6	793,800
	Rock bed + levelling (m^2)	7,500	40	300,000
	Concrete + casting (m^3)	17,000	130	2,210,000
	Core fill (m^3)	46,300	15	694,500
	Lean mix fill (m^3)	15,400	30	462,000
	Top block (m^3)	3,700	50	185,000
	Floating, placing, finishing (no.)	30	78000	2,340,000
	Subtotal			**7.0 million**
Total cost caisson quay wall, upper bound design				**14.7 million**

Table 5-6 upper bound caisson quay wall cost estimation

Block-work original	Item	Quantity	Rate (£)	Costs (£)
Wall – 5m	Dredging (m^3)	25,300	6	151,800
	Rock bed + levelling (m^2)	2,900	40	116,000
	Concrete + casting (m^3)	18,750	70	1,312,500
	Placing, finishing blocks (no.)	420	1,000	420,000
	Subtotal			**2.0 million**
Wall – 8m	Dredging (m^3)	91,800	6	550,800
	Rock bed + levelling (m^2)	6,900	40	276,000
	Concrete + casting (m^3)	59,100	70	4,137,000
	Placing, finishing blocks (no.)	1080	1,000	1,080,000
	Subtotal			**6.0 million**
Wall – 10m	Dredging (m^3)	128,900	6	773,400
	Rock bed + levelling (m^2)	7,200	40	288,000
	Concrete + casting (m^3)	69,900	70	4,893,000
	Placing, finishing blocks (no.)	1200	1,000	1,200,000
	Subtotal			**7.2 million**
Total cost block-work quay wall, original design				**15.2 million**

Table 5-7 block-work quay wall cost estimation

Block-work lower bound	Item	Quantity	Rate (£)	Costs (£)
Wall – 5m	Dredging (m^3)	24,400	6	146,400
	Rock bed + levelling (m^2)	2,700	40	108,000
	Concrete + casting (m^3)	17,100	70	1,197,000
	Placing, finishing blocks (no.)	420	1,000	420,000
	Subtotal			**1.9 million**
Wall – 8m	Dredging (m^3)	89,100	6	534,600
	Rock bed + levelling (m^2)	6,600	40	264,000
	Concrete + casting (m^3)	54,900	70	3,843,000
	Placing, finishing blocks (no.)	1080	1,000	1,080,000
	Subtotal			**5.7 million**
Wall – 10m	Dredging (m^3)	125,500	6	753,000
	Rock bed + levelling (m^2)	6,900	40	276,000
	Concrete + casting (m^3)	65,100	70	4,557,000
	Placing, finishing blocks (no.)	1200	1,000	1,200,000
	Subtotal			**6.8 million**
Total cost block-work quay wall, lower bound design				**14.4 million**

Table 5-8 lower bound block-work quay wall cost estimation

Block-work upper bound	Item	Quantity	Rate (£)	Costs (£)
Wall – 5m	Dredging (m^3)	26,200	6	157,200
	Rock bed + levelling (m^2)	3,000	40	120,000
	Concrete + casting (m^3)	20,400	70	1,428,000
	Placing, finishing blocks (no.)	420	1,000	420,000
	Subtotal			**2.1 million**
Wall – 8m	Dredging (m^3)	94,500	6	567,000
	Rock bed + levelling (m^2)	7,200	40	288,000
	Concrete + casting (m^3)	63,300	70	4,431,000
	Placing, finishing blocks (no.)	1080	1,000	1,080,000
	Subtotal			**6.4 million**
Wall – 10m	Dredging (m^3)	132,300	6	793,800
	Rock bed + levelling (m^2)	7,500	40	300,000
	Concrete + casting (m^3)	74,700	70	5,229,000
	Placing, finishing blocks (no.)	1200	1,000	1,200,000
	Subtotal			**7.5 million**
Total cost block-work quay wall, upper bound design				**16.0 million**

Table 5-9 upper bound block-work quay wall cost estimation

The costs for the different quays and the total quay wall costs per scenario are now known. To be able to make a fair comparison between the different quay walls (structure type as well as quay wall depth) the costs per metre quay length should be known. The costs per metre quay length are presented in Table 5-10.

		- 5m	- 8m	- 10m
Caisson	Lower	£ 5,800	£ 8,600	£ 10,800
	Original	£ 6,200	£ 8,700	£ 11,200
	Upper	£ 6,500	£ 9,500	£ 11,600
Block work	Lower	£ 6,200	£ 9,500	£ 11,300
	Original	£ 6,700	£ 10,100	£ 11,900
	Upper	£ 7,100	£ 10,600	£ 12,100

Table 5-10 costs per meter quay length.

5.4 Cost estimation dredging works and land reclamation

As mentioned in Section 4.4.3 lower and upper boundaries still have to be determined for the soil characteristics concerning the material to be dredged. Because of the influence of the soil characteristics on dredging costs as well as on reclamation costs it is not possible to determine these boundaries unambiguous.

To derive a cost estimation for the dredging works and land reclamation, first lower and upper boundaries for the dredging works have been determined. The resistance to dredging is the norm for the dredging costs. The resistance to dredging is therefore expressed in costs. Hence a greater resistance will give higher costs. Three levels have been identified, lower, original and upper, Table 5-11.

Costs for (de-) mobilisation of a cutter-dredger and other dredging equipment are incorporated in the dredging costs per m^3.

For all levels two situations have been identified: the dredged materials are suitable for reclamation or the dredged materials are unsuitable for reclamation. A complex relation exists between the resistance to dredging and the suitability of the dredged material as reclamation material. It is beyond the scope of this project to determine this relationship.

		Cost per m^3 (£)			
Dredging	**Reclamation**	**Dredging**	**Disposal**	**Reclamation**	
				– MWL	+ MWL
Lower	Suitable	1.5	0.5	0.5	1
	Unsuitable			1.5	2
Original	Suitable	3	0.5	0.5	1
	Unsuitable			1.5	2
Upper	Suitable	4.5	0.5	0.5	1
	Unsuitable			1.5	2

Table 5-11 rates per m^3, identification of the boundaries

In Section 4.4 the volumes to be dredged, disposed and reclaimed were identified. With the rates provided in Table 5-11 the total costs per item can be determined, see Table 5-12.

Dredging costs (£)			
Lower bound		13.1 million	
Original		26.3 million	
Upper bound		39.4 million	
Disposal and reclamation costs (£)			
Use of dredged materials		Suitable	Unsuitable
Reclamation	+ MWL	2.6 million	5.1 million
	– MWL	0.6 million	1.8 million
Disposal dredged materials		2.5 million	4.4 million

Table 5-12 dredging, disposal and reclamation costs

By adding the dredging costs and costs for disposal and reclamation the total costs for the dredging works and reclamation are known, see Table 5-13. For each scenario two sums of total costs are provided. One column presents the total costs if the dredged materials are suitable for reclamation and the other column the total costs in case the dredged materials are unsuitable as reclamation material.

Total costs of dredging and reclamation works (£)		
Use of dredged materials	Suitable	Unsuitable
Lower bound	18.8 million	24.4 million
Original	32.0 million	37.6 million
Upper bound	45.1 million	50.7 million

Table 5-13 total dredging, disposal and reclamation costs per scenario

During the design of a port normally a balanced dredge-reclaim volume ratio is favoured. If dredge and reclaim volumes are out of balance, either a borrow area or a disposal area has to be indicated. By definition of the research plan for this study a port layout developed by Posford Haskoning was adopted as the base for this study, therefore this aspect has not been taken into consideration.

6 Boundary conditions detailed design

6.1 Introduction

In this Section the boundary conditions for the detailed design stage will be established. In the original research plan it was assumed that new site investigation data, mainly regarding the soil conditions, would be available. Owing to circumstances this is unfortunately not the case.

At the start of this study this has been taken into consideration. Therefore information from a soil investigation carried out by Fugro for the Ministry of Agriculture and Fisheries in 1989 and 1990 has not been used in the preliminary design. This investigation, which was for a proposed fishing harbour at Duqm, will provide new geotechnical information for the detailed design phase.

Meteorological data such as temperature and rainfall are not subject to changes affecting the design. Tides and currents are the same as in the preliminary design, see Section 3. Cyclones and wind set-up will be discussed within the wave climate section.

6.2 Wave climate

Offshore wave climate

The data to establish the offshore wave climate for the detailed design are the same as the data used for the preliminary design. These data can be found in Section 3.3, or in more detail in the Draft Final Report by Posford Haskoning[lit 19].

Inshore wave climate

For determination of the inshore wave climate in the detailed design phase same offshore wave data are used as in the preliminary design phase. Refinement of the inshore wave climate, in comparison with the wave climate in the preliminary design phase, can be obtained by the use of more advanced calculation methods.

The numerical model SWAN (Simulating WAves Near shore) was used by Posford Haskoning to simulate the key processes (refraction, shoaling, wave breaking and diffraction) and transform the selected offshore waves to a number of inshore locations. SWAN is a 2-dimensional full spectral coastal wave model developed by Delft University of Technology.

The SWAN models were run with a water level of 3.2m CD. This corresponds to MHHW plus an allowance of 0.67m for a meteorological surge. A number of SWAN runs was used to transform waves with return periods of 100 years, 1 year and 1 month to inshore locations along the main and lee breakwater. The results of the analysis are shown in Table 6-1.

Return period	**Offshore wave direction**							
	0	30	60	90	120	150	180	210
1:1 month	0.85	1.44	1.76	-	-	-	-	-
1:1 year	2.04	2.66	2.51	2.43	1.90	2.09	2.46	1.78
1:100 year	-	-	3.36	4.21	3.54	3.55	3.57	2.7
Cyclone (H_s = 10m)	-	-	-	5.64	-	-	-	-
Cyclone (H_S = 15m)	-	-	-	5.82	-	-	-	-

Table 6-1 inshore wave height (- :scenario not modelled, other scenarios are more severe)

Cyclones

In the preliminary design the influence of the occurrence of cyclones was assumed to be negligible, this primarily on basis of a study by Baird and Associates (2001). However in an earlier study by Halcrow (1992), based on historical data and eyewitnesses, the return period of a cyclone was estimated at approximately 100 year. Therefore the occurrence of cyclones has been taken count of in the SWAN analysis and will be so in the detailed design phase.

The significant wave heights at the entrance to the harbour corresponding to offshore wave heights of 10m and 15m were calculated using the numerical model SWAN to be 5.6m and 5.8m respectively. These results demonstrate that the relatively shallow (<50m deep) 60-km wide shelf between Duqm and deep water has a significant depth limiting effect on incoming waves greater than 10m. Therefore, the uncertainties in the calculation of offshore wave height are not critical. It is recommended that an inshore significant wave height of 5.8m will be used for the purposes of design.

The cyclones generally develop off the western coast of India and track westwards across the Arabian Sea. When reaching the Arabian Peninsula orientation of the cyclones is west to south-west. In case of a cyclone attack the lee breakwater gets shelter from the main breakwater.

Lower and upper bound wave conditions

Margins will have to be indicated for the lower and upper bound breakwater design. A margin of 10% has been set on the inshore wave climate. Compared to the previous design stage a tighter margin can be used because the SWAN analysis does take account of the shape of the land and the bathymetric profile.

Inshore			**Lower bound**	**Upper bound**
Significant wave height, H_s	**1:1 year**	m	2.2	2.8
	1:100 years	m	3.8	4.6
	Cyclone	m	5.2	6.4
Wind set-up		m	0.60	0.74
Design water level		m + CD	3.1	3.3

Table 6-2 lower and upper bound wave and wind conditions

For the SWAN analysis an extreme water level of CD + 3.2m was adopted. This combines MHHW with a storm surge of approximately 0.7m. Same as for the wave climate, the margin on the wind set-up will have to be set on the inshore wave climate. A margin of 10% has been set on the inshore wind set-up and storm surge, Table 6-2.

6.3 Geotechnical data

Fugro soil investigation

In 1989-1990 Fugro Middle East carried out a soil investigation for the Ministry of Agriculture and Fisheries. This investigation, which was for a proposed fishing harbour, consisted of ten offshore boreholes, see Appendix XII. These boreholes were carried out at locations adjacent to the Ra's ad Duqm headland.

The boreholes show varying thickness of marine deposit overlying interbedded siltstones, mudstones and sandstones. The marine deposits were generally described as very loose to medium dense silty sand. The reported thickness of the marine deposits varied between 0.05m and 3.15m, with an average of approximately 1.1m. As an example Appendix XIII shows borehole logs no. 4 and no. 8 from the Fugro report.

The underlying bedrock was described as very weak to moderately weak (classification BS 5930: 1981) laminated interbedded siltstone and mudstone. Occasional layers of weak to moderately strong sandstone and gypsum were found.

One part of the proposed port area that is not covered by the Fugro analysis is the small bay. The subsoil underlying this bay will mainly have to bear the quay wall structures. From the boreholes close to the bay and from the 1992 Geological Map of Duqm it is assumed that the bay itself is underlain by the interbedded green marls, gypsum and calcarenites of the Duqm Formation, Appendix IV.

Applicability and validity of soil investigation

The results do provide new information but still do not give a definite answer regarding the soil conditions. The size of the fishery port investigated in earlier studies is much smaller than the size of the port in this study. The Fugro site investigation covers only part of the proposed port site, it is therefore not necessarily fully representative of the site as a whole and the small shallow bay in particular.

The boreholes were penetrated to a maximum depth of only 9m below the existing seabed. From the 1992 Geological Map of Duqm it was expected to find rock under most of the site, due to the limited penetration depth this can not be confirmed. It is likely that for most of the site rock will be found under the layers of siltstones and mudstones.

Implications for breakwater and quay wall design

The seabed surface layer of marine deposits is too soft to support the quay wall. This surface layer will have to be removed before construction. Probably it will be possible to leave the marine deposits in place beneath the breakwater and reclamation areas; long term settlements deriving from consolidation of the marine deposits are unlikely to be excessive. The average thickness of the marine deposit layer is only 1.1m and most consolidation will take place right after the reclamation material is put on.

The siltstone and mudstone layers may create bearing capacity problems for the foundation of the quay wall structure. The quay wall design will have to be reviewed.

On basis of the Fugro site investigation it appears that most of the dredged material from the seabed consists weak to moderately weak siltstones and mudstones. These materials will probably break down during dredging to a point that they are no longer suitable as reclamation material.

7 Detailed design of port components

7.1 Introduction

In the previous design stage a preliminary design for the port components has been presented. In this detailed design phase the components will be defined at a more specified level. The level of detail can be obtained by the use of new data and by the use of more accurate design methods. These designs are based on the boundary conditions provided in Section 6.

First the breakwater designs will be presented, again lower and upper bound designs are also indicated, Section 7.2. Following the quay wall design will be reviewed in Section 7.3. Finally dredging works and land reclamation will be discussed in Section 7.4.

7.2 Detailed breakwater design

7.2.1 Introduction

Changes in the inshore wave climate and geotechnical data will have their reflections on the breakwater design. The design wave height is of the same order as in the previous design stage. Therefore same type of breakwater structure, concrete armoured breakwater with Core-Loc primary armour units, will be adopted for further design.

On basis of the borehole logs it is assumed that the layer of weak marine deposits under the main breakwater ranges from 5cm up to 50cm. No additional scour protection or soil improvement will be needed under the main breakwater or in front of its seaward toe.

No Fugro boreholes were carried out at locations near the proposed lee breakwater. The lee breakwater runs from the smaller headland into deeper water. Under the deepest part of the lee breakwater subsoil conditions will be comparable with conditions under the main breakwater, with a layer of weak marine deposits ranging up to 50cm. No additional scour protection or soil improvement will be needed under the deepest part of the lee breakwater or in front of its seaward toe.

The part of the lee breakwater in shallow water is cause of more concern. The smaller headland is the northern limit of the bay. Within this bay the marine deposit layer ranges from 1 up to 3m. The average thickness of the marine deposit layer near the shallow part of the lee breakwater is assumed to be in the order of 1.5m. Scour protection or soil improvement will be needed to ensure stability of the seaward toe.

7.2.2 Breakwater design: main breakwater

Armour layer

The major part of the main breakwater is in relatively deep water and under severe wave attack. Therefore same size armour units will be used along the full length of the breakwater. The round head of the breakwater needs special attention; larger armour units will be needed. The Hudson formula has been used for the design of stable weight units, with breaking wave conditions and a no-damage criterion (less than 2% displacement), Table 7-1. The design wave height for the main breakwater is 5.8m.

Primary armour sizes		
	Trunk	Head
Stability coefficient[Lit 5]**, K_d**	16	13
Structure slope	1 : 1.5	1 : 1.5
Weight of individual armour unit (ton)	8.3	10.2

Table 7-1 Core-Loc primary armour sizes, using the Hudson formula

8.3 ton Core-Loc armour units are recommended for the main breakwater. The head will be protected with 10.2 ton armour units, Table 7-2 and Appendix XIV-a.

Recommended primary armour unit			
		Trunk	Head
Stable weight of individual armour unit	ton	8.3	10.2
Volume of armour unit	m^3	3.5	4.3
Layer thickness	m	2.3	2.5

Table 7-2 Core-Loc recommended armour unit, trunk and head

Crest

The crest level of the breakwater has been determined under consideration of overtopping conditions on a 1:1 year return period, a 1:100 return period and for the occurrence of a cyclone. On basis of investigations by Gerloni (1991) and interpretation by Van der Meer (1993) overtopping rates have been established, Table 7-3. Guidelines and formula by HR Wallingford Ltd (1999)[Lit 2] have been used to calculate the necessary crest height, Table 7-4 and Appendix XIV-b.

Return period	Design wave height Hs (m)	Wave period $_{Ts}$ (s)	Inshore direction	Overtopping rates (l/s/m)
1:1 year	2.50	7.79	99	1.0
1:100 years	4.34	8.42	102	10.0
Cyclone 15m	5.82	9.39	102	100.0

Table 7-3 design conditions crest height main breakwater

Deviations from one location to another can be expected along the breakwater. It is assumed that during design lifetime settlements of subsoil and breakwater are negligible. Because of the expected deviations some tolerance has been taken into account in the design.

Main breakwater		
Design water level	m + CD	3.2
Crest wall / cap-block	m	Yes
Shoulder width (to cap-block)	m	4.6
Freeboard on top of slope relative to SWL	m	6.2
Tolerance	m	0.30
Crest height, relative to CD	m	9.7

Table 7-4 height and width of crest, main breakwater

First under-layer

The layer under the armour layer is called the first under-layer. The weights in the under-layer should not be less than 1/10 of the weight of the primary armour. Quarry stone with a grading of 1 to 3 ton will be used for the first under-layer. The layer thickness will be $2*D_{n50}$.

1 to 3 ton quarry stones will also be used as armour on the leeward side of the main breakwater to ensure protection against overtopping and ship waves.

Core

Standard class of rock grading with a range of 10 to 200 kg will be used.

Filter, toe berm and scour protection

To ensure stability a toe berm, designed according investigations by Gerding (1995), is implemented in the design. The toe berm is the lower support for the armour layer. The weight of the stones used as first under-layer satisfies as toe material. The toe berm is placed on a bed of core material.

7.2.3 Breakwater design: lee breakwater

Design criteria

In the SWAN analysis the wave heights were calculated at three points just in front of the lee breakwater (S1, S2, S3) and at one spot a few hundred meters in front of the lee breakwater. The lee breakwater is sheltered from direct cyclone wave attack by the main breakwater. The most severe conditions are produced by waves from the north-east. In the SWAN-analysis same water level was used for the lee breakwater as for the main breakwater. The design water level is CD + 3.2m.

The SWAN analysis does not provide information about the deepest part of the lee breakwater. The main breakwater shelters this part. To derive a design wave height for this part of the lee breakwater (H) the results of the SWAN analysis have been extrapolated to deeper water and compared with results at spots in front of the main breakwater with same depth.

		S1	S2	S3	H
1:1 year	Wave height, H_s	0.96	1.94	2.38	2.5
	Wave period, T_s	4.85	5.43	5.56	5.5
1:100 year	Wave height, H_s	0.98	2.10	3.27	4.0
	Wave period, T_s	5.61	7.15	7.65	8.4

Table 7-5 inshore wave climate lee breakwater

Armour layer

Along the full length of the lee breakwater it is possible to use quarry rock as primary armour. The van der Meer formula is best suited to estimate the weight of rock armour units, Table 7-6, Appendix XIV-c.

Location along breakwater		Length (m)	Storm	Slope	Armour weight W_{50}	Primary armour	Recommended (Dutch grading)
Start – 1.0	**S1**	450	8-hr.	1:2.5	190 kg	Quarry stone	0.3 – 1.0 kg
			10-hr.		210 kg		
1.0 – 2.5	**S2**	485	8-hr.	1:2.5	1.4 t	Quarry stone	1.0 – 3.0 t
			10-hr.		1.5 t		
2.5 – 5.5	**S3**	400	8-hr.	1:2.5	4.2 t	Quarry stone	3.0 – 6.0 t
			10-hr.		4.5 t		
5.5 - head	**H**	380	8-hr.	1:2.5	7.5 t	Quarry stone	6.0 – 10.0 t
			10-hr.		8.0 t		

Table 7-6 primary armour lee breakwater

Crest height

The crest level of the breakwater has been determined under consideration of overtopping conditions on a 1:1 year return period and a 1:100 return period, Table 7-7. In case of a cyclone attack the main breakwater shelters the lee breakwater. For a 1 in 1 year return period overtopping rates have been limited to 1.0 l/s/m (S3, H), with quays immediately behind the breakwater overtopping rates are limited to 0.1 l/s/m (S1, S2). For a 1 in 100 year return period overtopping rates have been limited to 10 l/s/m, Appendix XIV-d. A crest wall will be implemented.

Location along breakwater		**S1**	**S2**	**S3**	**H**
Depth relative to CD	m – CD	Start – 1	1 – 2.5	2.5 – 5.5	5.5 – head
Design water level	m + CD	3.2	3.2	3.2	3.2
Shoulder width (to crest wall)	m	1.95	2.85	3.75	4.5
Freeboard on top of slope relative to SWL	m	1.8	3.7	3.8	5.0
Tolerance	m	0.3	0.3	0.3	0.3
Crest height, relative to CD	m	5.3	7.2	7.3	8.5

Table 7-7 height and width of crest, lee breakwater

The portside of the lee breakwater will be armoured with 1 to 3 ton quarry stones on a 1:1.5m slope, to protect washing out of core material due overtopping and ship waves.

First under-layer

The weights in the under-layer should not be less than 1/10 of the weight of the primary armour. Quarry stone with a grading of 0.6 to 1.0 ton will be used for the first under-layer under section H. Under sections S2 and S3 stones with a grading ranging from 0.3 to 1.0 ton will be used. Section S1 does not need a filter layer between primary armour and core material.

Core

Standard class of rock grading with a range of 10 to 200 kg will be used.

Toe berm, filter and scour protection

Under the seaward toe of the first part of the lee breakwater the layer of soft material will be removed and replaced by core material. The layer of soft material varies along this part of the breakwater but will approximately be 1.5m thick. To ensure stability the primary armour units are extended on bed level. They are placed on the bed of core material.

To ensure stability of the toe at the second (deepest) part of the lee breakwater, the armour units are placed in a dredged trench. This will anchor them and prevent them from moving seawards under pressure of the breaking waves. For the rock armour this means a width of at least $3*D_{n50}$. Except for the dredged trench the marine deposit can stay in its place. The armour units have to be placed in a trench otherwise armour units on the top row of the toe could face stability problems.

Where quays are immediately behind the breakwater a geotextile will be placed on the inner slope of the breakwater. The toe at the portside of the breakwater does not need special protection. The portside armour units are placed on a bed of core material.

7.2.4 Lower bound breakwater design

Lower bound main breakwater

A margin of 10% has been set as lower bound on inshore wave and wind conditions to determine the lower bound design criteria, Table 7-8.

Return period	Design wave height Hs (m)	Wave period T_s (s)	Overtopping rates (l/s/m)
1:1 year	2.24	7.42	1.0
1:100 years	3.79	7.73	10
Cyclone 15m	5.24	8.82	100

Table 7-8 lower bound design criteria main breakwater

With the design criteria weight of the primary armour units, Table 7-9, and crest level, Table 7-10, have been determined. Core-Loc units will be used as primary armour, Appendix XV-a. For calculations on overtopping see Appendix XV-b.

Recommended primary armour unit			
		Trunk	Head
Stable weight of individual armour unit	ton	6.0	7.4
Volume of armour unit	m^3	2.5	3.1
Layer thickness	m	2.1	2.2

Table 7-9 lower bound Core-Loc primary armour units, main breakwater

Lower bound main breakwater		
Design water level	m + CD	3.1
Crest wall / cap-block	m	Yes
Shoulder width (to cap-block)	m	4.2
Freeboard on top of slope relative to SWL	m	5.1
Tolerance	m	0.30
Crest height, relative to CD	m	8.5

Table 7-10 lower bound height and width of crest, main breakwater

Measures of other breakwater elements have been determined and are shown in Table 7-11.

Lower bound main breakwater	
First under-layer	1.0 – 3.0 ton quarry stone
Core material	10 – 200 kg quarry stone
Inner slope	1.0 – 3.0 ton quarry stone
Toe protection	Same as original design

Table 7-11 breakwater elements, lower bound main breakwater

Lower bound lee breakwater

A margin of 10% has been set as lower bound on inshore wave and wind conditions to determine the lower bound design criteria, Table 7-12.

		S1	S2	S3	H
1:1 year	Wave height, H_s	0.86	1.75	2.14	2.3
	Wave period, T_s	4.62	5.10	5.19	5.1
1:100 year	Wave height, H_s	0.88	1.89	2.94	3.6
	Wave period, T_s	5.38	6.81	7.22	7.9

Table 7-12 inshore wave climate lower bound lee breakwater

With the design criteria weight of the primary armour units, Table 7-13, and crest level, Table 7-14, have been determined. Calculations on armour weight and overtopping can be found in Appendix XV-c respectively Appendix XV-d.

Location along breakwater		Length (m)	Storm	Slope	Armour weight W_{50}	Primary armour	Recommended (Dutch grading)
Start – 1.0	**S1**	450	8-hr.	1:2	200 kg	Quarry stone	0.3 – 1.0 t
			10-hr.		220 kg		
1.0 – 2.5	**S2**	485	8-hr.	1:2	1.5 t	Quarry stone	1.0 – 3.0 t
			10-hr.		1.6 t		
2.5 – 5.5	**S3**	400	8-hr.	1:2	4.3 t	Quarry stone	3.0 – 6.0 t
			10-hr.		4.6 t		
5.5 - head	**H**	380	8-hr.	1:2	7.6 t	Quarry stone	6.0 – 10.0 t
			10-hr.		8.2 t		

Table 7-13 primary armour lower bound lee breakwater

Location along breakwater		S1	S2	S3	H
Depth relative to CD	m – CD	**Start – 1**	**1 – 2.5**	**2.5 – 5.5**	**5.5 – head**
Design water level	m + CD	3.1	3.1	3.1	3.1
Shoulder width (to crest wall)	m	1.95	2.85	3.75	4.5
Freeboard on top of slope relative to SWL	m	1.7	3.5	3.4	4.5
Tolerance	m	0.3	0.3	0.3	0.3
Crest height, relative to CD	m	5.1	6.9	6.8	7.9

Table 7-14 height and width of crest, lower bound lee breakwater

Measures of other breakwater elements have been determined and are shown in Table 7-15.

Lower bound lee breakwater		
First under-layer	**S1**	-
	S2, S3	0.3 – 1.0-t quarry stone
	H	0.6 – 1.0-t quarry stone
Core material		10 – 200 kg quarry stone
Inner slope	**port**	1.0 – 3.0 ton quarry stone
	quay	Geotextile
Toe protection		Same as original design

Table 7-15 breakwater elements, lower bound main breakwater

7.2.5 Upper bound breakwater design

Upper bound main breakwater

A margin of 10% has been set as upper bound on inshore wave and wind conditions to determine the upper bound design criteria, Table 7-16.

Return period	Design wave height Hs (m)	Wave period T_s (s)	Overtopping rates (l/s/m)
1:1 year	2.75	8.16	1.0
1:100 years	4.63	8.68	10
Cyclone 15m	6.40	9.93	100

Table 7-16 upper bound design criteria main breakwater

With the design criteria weight of the primary armour units, Table 7-17, and crest level, Table 7-18, have been determined. Core-Loc units will be used as primary armour, Appendix XVI-a. For calculations on overtopping see Appendix XVI-b.

Recommended primary armour unit			
		Trunk	Head
Stable weight of individual armour unit	ton	11.1	13.7
Volume of armour unit	m^3	4.6	5.7
Layer thickness	m	2.5	2.7

Table 7-17 upper bound primary armour units, main breakwater

Upper bound main breakwater		
Design water level	m + CD	3.3
Crest wall / cap-block	m	Yes
Shoulder width (to cap-block)	m	5
Freeboard on top of slope relative to SWL	m	6.9
Tolerance	m	0.30
Crest height, relative to CD	m	10.5

Table 7-18 upper bound height and width of crest, main breakwater

Measures of other breakwater components have been determined and are shown in Table 7-19.

Upper bound main breakwater	
First under-layer	1.0 – 3.0 ton quarry stone
Core material	10 – 200 kg quarry stone
Inner slope	1.0 – 3.0 ton quarry stone
Toe protection	Same as original design

Table 7-19 breakwater elements, upper bound main breakwater

Upper bound lee breakwater

A margin of 10% has been set as upper bound on inshore wave and wind conditions to determine the upper bound design criteria, Table 7-20.

		S1	S2	S3	H
1:1 year	Wave height, H_s	1.06	2.13	2.62	2.8
	Wave period, T_s	5.07	5.74	5.91	5.9
1:100 year	Wave height, H_s	1.08	2.31	3.60	4.4
	Wave period, T_s	5.83	7.48	8.06	8.9

Table 7-20 inshore wave climate upper bound lee breakwater

With the design criteria weight of the primary armour units, Table 7-21, and crest level, Table 7-22, have been determined. Calculations on armour weight and overtopping can be found in Appendix XVI-c respectively Appendix XVI-d.

Location along Breakwater		Length (m)	Storm	Slope	Armour weight W_{50}	Primary Armour	Recommended (Dutch grading)
Start – 1.0	**S1**	450	8-hr.	1:3	190 kg	Quarry stone	0.3 – 1.0 kg
			10-hr.		210 kg		
1.0 – 2.5	**S2**	485	8-hr.	1:3	1.4 t	Quarry stone	1.0 – 3.0 kg
			10-hr.		1.5 t		
2.5 – 5.5	**S3**	400	8-hr.	1:3	4.3 t	Quarry stone	3.0 – 6.0 t
			10-hr.		4.6 t		
5.5 – head	**H**	380	8-hr.	1:3	7.5 t	Quarry stone	6.0 – 10.0 t
			10-hr.		8.0 t		

Table 7-21 primary armour upper bound lee breakwater

Location along breakwater		**S1**	**S2**	**S3**	**H**
Depth relative to CD	m – CD	**Start – 1**	**1 – 2.5**	**2.5 – 5.5**	**5.5 – head**
Design water level	m + CD	3.3	3.3	3.3	3.3
Shoulder width (to crest wall)	m	1.95	2.85	3.75	4.5
Freeboard on top of slope relative to SWL	m	1.9	3.7	3.9	5.1
Tolerance	m	0.3	0.3	0.3	0.3
Crest height, relative to CD	m	5.5	7.3	7.5	8.7

Table 7-22 height and width of crest, upper bound lee breakwater

Measures of other breakwater elements have been determined and are shown in Table 7-23.

Upper bound lee breakwater		
First under-layer	**S1**	-
	S2, S3	0.3 – 1.0-t quarry stone
	H	0.6 – 1.0-t quarry stone
Core material		10 – 200 kg quarry stone
Inner slope	**port**	1.0 – 3.0 ton quarry stone
	quay	Geotextile
Toe protection		Same as original design

Table 7-23 breakwater elements, upper bound main breakwater

7.3 Detailed quay wall design

7.3.1 Introduction

The quay wall structures selected in the previous design stage are both very vulnerable to variations in the soil conditions. With changes in the soil conditions, see section 6.3, other possible quay wall options will have to be reconsidered.

In the previous design stage two suitable quay wall structures, a block-work wall and a caisson wall were adopted for further design. Only one structure should be adopted for the detailed design phase. Finally upper and lower bound design are presented.

7.3.2 Review of quay wall options

The quay wall structures will mainly be build where the bay is at present. Under this part of the port-site the subsoil consists of a thick (1m to 3m) first layer of marine deposits on a layer of weak mudstone and siltstone (approximately 5 meters thick). These layers are underlain with a moderately strong layer of sandstone.

With considerable changes in soil characteristics earlier rejected quay wall solutions should be reconsidered. With the assumption of rock in the previous design stage, concrete piled structures were precluded because of poor driving conditions. The borehole logs show that the subsoil consists of weak mudstone and siltstone, therefore concrete piled structures should again be taken into consideration.

Nevertheless there are some drawbacks that makes concrete pile driving unsuitable:

- The composition and thickness of the mudstone and siltstone layer itself varies widely.
- In weaker layers thin bands and lenses of strong material are present on an extensive scale.
- The underlying stronger layers still preclude pile driving. The composition of this layer is too diverse and unpredictable to connect (grout) concrete piles to them.

As the load-bearing capacity of the subsoil below the base foundation is a key-factor for both suggested quay wall structures (caisson wall and block-work wall), these designs will have to be reviewed. Probably weaker layers will have to be removed or improved.

7.3.3 Selection of preferred quay wall structure

In the preliminary design two possible solutions for the quay wall structure were adopted for further design, a caisson wall and a block-work wall. It was suggested that the construction costs would be decisive in the selection process of the preferred quay wall structure. A caisson quay wall structure appeared to be cheaper, however differences in construction costs were marginal.

The margins on differences in construction costs are too tight to make the choice for either one of the solutions. The choice for either the caisson quay wall or the block-work quay wall will also depend on their buildability and their reliability during lifetime. Similarities and differences between both structure types were already briefly discussed in Section 4.3.4.

Reliability, in terms of proven track record and long term stability is for both options very high. The block-work wall has a better ability to accommodate small settlements. Both types are vulnerable to overturning failure from overdredge or scour.

In the past block-work walls have encountered problems in the Arabian region because of concrete blocks of poor quality. Poor quality concrete is an even bigger threat to the reinforced concrete caisson structures. In Section 4.3.3 the threat of corrosion to steel structures in marine environments has been outlined.

The phase 1 layout by Posford Haskoning, which was adopted for this study, included a dry-dock complex, Appendix II. A floating dry-dock able to facilitate 100,000-dwt bulk carriers was proposed. This floating dry-dock could ideally be used as construction dock for the caissons. If a 100,000-dwt floating dock would be available it would be possible to construct and store at one time half the number of caissons needed. The construction of the caissons can only start after the floating dry-dock has arrived at the construction site. I.e. after completion of the main breakwater and dredging has progressed to a level that the floating dry-dock has access to the port.

The construction of the blocks for the block-work wall could start after the land-based building yard would be set-up. As soon as the main breakwater would be completed construction of the quays could start. There is plenty of space near the port-site to set up a building yard.

On the basis of the above the block-work wall has been adopted as quay wall structure for the new port at Duqm. Summarised the major reasons are:

- Better ability to accommodate small settlements.
- Less vulnerable to poor quality concrete.

- Start of construction of blocks not dependent on other activities.
- Immediately after completion of the main breakwater construction of the quay wall can start.
- Uncertainty if investments in a large floating dock can be justified at the start of the project.

These mentioned reasons regarding buildability and reliability outweigh the higher construction costs.

7.3.4 Design of quay wall

The block-work quay wall in the detailed design phase has been designed to the recommendations of British Standard 6349, part 2, and Recommendations of the Committee for Waterfront Structures Harbours and Waterways, EAU 1996.

Block-work

To provide efficient utilisation of formwork, same sized blocks have been used for the different quay walls. The blocks are stacked in vertical columns. The main advantage of this way of construction is that it can accommodate differential settlement. Each block column is 5m wide. With increasing depth the length of the blocks increases. The maximum block weight is limited to 200 tons, Table 7-24.

The principal modes of failure of a block-work quay wall are:

- Deep slip. The subsoil under the quay walls consists of rock. Deep slip circles are not likely to occur.
- Overturning. Overturning is most times caused by failure from overdredging or scour. The factor of safety against overturning is 1.3.
- Sliding. Unbalanced hydrostatic pressure and loss of toe embedment are the main reasons for sliding failure. A factor of safety of 1.5 has been applied.

The cross-section of the wall and the size of individual units are designed in such way that stability criteria are met both at foundation level and at horizontal joint levels. Stability calculations of the different quay walls can be found in Appendix XVII.

	- 5m		- 8m		- 10m	
	Dimensions H x l (m^2)	Weight (ton)	Dimensions h x l (m^2)	Weight (ton)	Dimensions h x l (m^2)	Weight (ton)
Block 1	2.0 x 5.5	132	2.0 x 5.5	132	2.0 x 5.5	132
Block 2	2.0 x 5.5	132	2.0 x 5.5	132	2.0 x 5.5	132
Block 3	2.0 x 5.5	132	2.0 x 5.5	132	2.0 x 5.5	132
Block 4	2.0 x 6.0	144	2.0 x 6.0	144	2.0 x 5.5	132
Block 5	2.0 x 6.0	144	2.0 x 6.0	144	2.0 x 6.0	144
Block 6	1.0 x 8.0	96	1.5 x 6.5	117	2.0 x 6.0	144
Block 7	-	-	1.5 x 7.0	126	1.5 x 7.0	126
Block 8	-	-	1.0 x 9.0	108	1.5 x 8.0	144
Block 9	-	-	-	-	1.0 x 10.0	120

Table 7-24 block dimensions detailed design

Foundation

To distribute the ground pressures at the foundation joint as uniformly as possible the base block has a spur of 1m (from the edges of the block above) on front and backside. The resultant of the forces, from blocks and surcharge, must be in the middle third of the base to reduce the point bearing pressure. In the initial design an overdredge of 0.5m has been taken into account for all quay walls.

A foundation bed of small graded quarry stone, 1m thick, will be placed between the subsoil and the base block. The bed must be carefully levelled with special gear and the aid of divers.

The marine deposits and weaker layers will have to be removed. They will be replaced with suitable material. This is only the case for the –5m quay wall. The other two block-work walls (–5m and –8m) extend to the stronger layers.

Blocks

In the preliminary design stage the blocks were provided with a tongue and groove interlock. It appears that in practice the blocks do not slid easily into place but hang up on each other. This and the experienced poor quality of concrete in other projects in the region have come to the decision that rectangular shaped blocks will be used. The critical load on the blocks will be during transport and lifting.

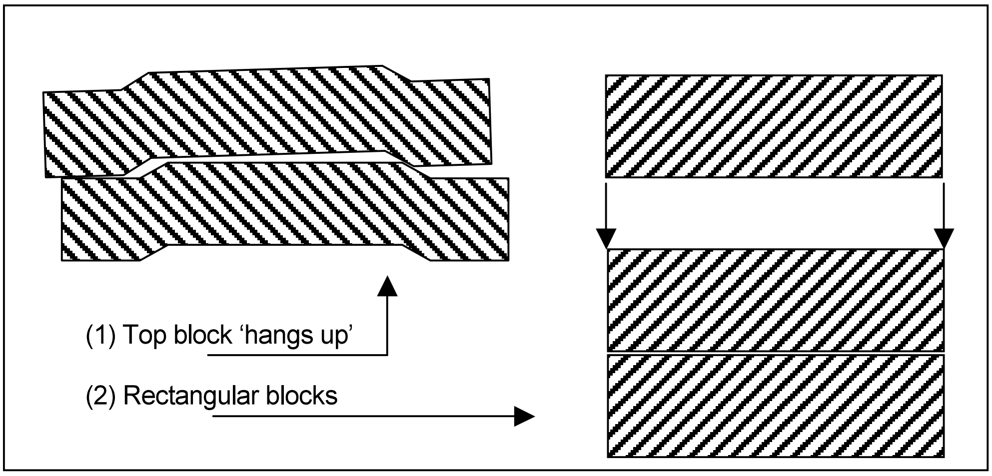

Figure 7-1 groove and interlock blocks hang up (1) and rectangular blocks (2)

7.3.5 Lower and upper bound detailed quay wall design

In the previous design stage the assumption of rock under the quay walls satisfied the bearing capacity requirements. Boundaries were set on the backfill material. It appeared that this had a limited effect on design and costs.

The Fugro analysis introduced uncertainties in the bearing capacity of the layers underlying the marine deposits. The boundaries in this design phase will be set on the subsoil. No boundaries will be set on the backfill material.

As lower bound the assumption has been made that the siltstone and mudstone layer appears to be strong enough to bear the quay wall. The marine deposits will still have to be removed. As upper bound the thickness of the siltstone and mudstone layer (which in this case has to be removed) is assumed to be 9m, instead of 5m. These changes do affect the foundation of the block-work wall, and therefore the construction costs. The block-work structure itself does not change.

7.4 Dredging works and land reclamation

7.4.1 Dredging works

At present the best data available on the bathymetry in the immediate vicinity of the site is a survey undertaken by Nortech Surveys Ltd in 1989. For an area of 2500 by 2500 meters, which covers the proposed port, depth contour lines have been covered with a grid of 50x50 m^2. For each section the depth has been averaged. These data have been worked out in a spreadsheet, resulting in a bathymetric profile of the area, Figure 7-2. For all areas an overdredge of 0.5m has been taken into account.

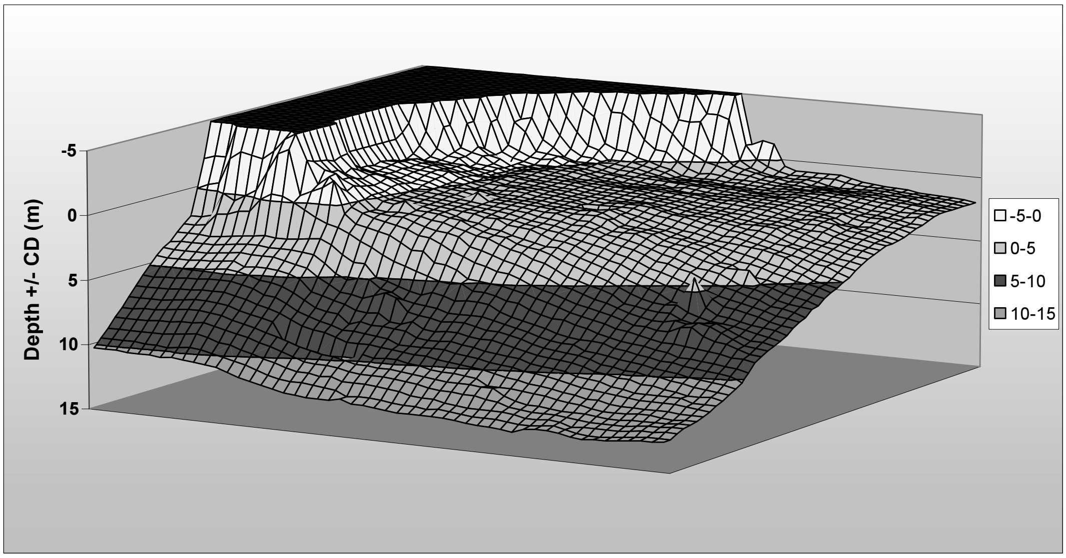

Figure 7-2 bathymetric profile of Duqm site location

By subtracting or adding the required depth profile for the port layout the spreadsheet gives an indication of the volume to be dredged for the construction of the port. The capital dredging works account for the development of basins, approach channel and turning circle, Table 7-25.

	Dredge volume (m^3)
Approach channel, –12m	141,300
Basin –5m quay	93,700
Basin –8m quay	650,500
Basin –10m quay and general harbour area	7,779,500
Total volume to be dredged	**8.7 million**

Table 7-25 volumes to be dredged, design

In the previous design stage the subsoil under the marine deposits was assumed to consist of rock. From the Fugro boreholes it appeared that weaker materials are to be found, see Section 6.3. The weaker materials will have a lower resistance to dredging, this expresses itself in a lower rate for the dredging costs.

7.4.2 Land reclamation

Reclamation volumes

The volumes to be reclaimed have been determined in the same way as the dredging volumes have been calculated.

	Volumes (m^3)
Reclamation below MWL	1.1 million
Reclamation above MWL	2.7 million
Total volume	3.8 million

Table 7-26 volumes to be reclaimed

Soil characteristics under the reclamation area

The marine deposits under the reclamation areas will not have to be removed. It is assumed that long term settlements deriving from consolidation of these marine deposits are unlikely to be excessive. Most consolidation will take place during construction after placement of the reclamation material.

The layer of weak mudstones and siltstones underlying the marine deposits is suitable as foundation layer for the reclamation and future port activities.

Suitability of dredged materials for reclamation

The rock subsoil is covered with a layer of marine deposits. The marine deposits of recent years are too weak to use for land reclamation. From the Fugro site investigation it appears that most of the other materials to be dredged from the seabed consist weak to moderately weak siltstones and mudstones. These materials will probably break down during dredging to a point that they are no longer suitable as reclamation material.

Nevertheless it is still possible that some of the dredged material is suitable for land reclamation. Therefore the option of the use of the dredged material for land reclamation is not ruled out in the cost estimation, see Section 8.4.

7.4.3 Lower and upper boundaries dredging works and land reclamation

Dredging works

The Fugro site investigation covers only part of the proposed port site, it is therefore not necessarily fully representative of the site as a whole and the small shallow bay in particular. But it does tighten the margins to be set on the rates for the dredging costs.

From the Fugro boreholes it appeared that materials weaker than earlier assumed in the preliminary design stage are to be found. In the preliminary design stage it was already taken into consideration that weak materials could be found. Therefore the lower bound for the dredging costs remains the same as in the preliminary design.

The upper bound rate for the dredging costs increases considerably compared to the rate in the preliminary design. The rates and costs can be found in the cost estimation section for the detailed design phase, section 8.4.

Land reclamation

The new information available from the Fugro analysis leads to the conclusion that the possibility that dredged materials are suitable for reclamation decreases. In the cost estimation same rates are used as in the preliminary design.

Unfortunately it is in the scope of this research not possible to give an indication of the chance that dredged materials can be used as reclaim material.

8 Cost estimation detailed design

8.1 Introduction

In this Section estimates of the costs for the port components designed in Section 7 are presented. This is also done for the lower and upper boundary designs.

In the detailed design stage again the unit rate technique has been used to determine the costs. The bill of quantities is still not very detailed. Instead quantities of the main items of work have been estimated and these are priced using rates that take account of the associated minor items.

For the detailed breakwater and quay wall designs same methods are used as in the preliminary design to determine the quantities. To make a transparent comparison between the two design phases same rates have been used to estimate the costs. Besides no new information was available concerning the rates.

For the dredging works and land reclamation a refined method has been used to determine the quantities, see Section 7.4. Besides historical data, the rates for the dredging works depend largely on soil conditions. The changes in the soil conditions will affect the rates.

8.2 Cost estimation breakwater

With the above mentioned method cost estimations have been established for the main and lee breakwater. In Table 8-1, Table 8-2 and Table 8-3 are the costs for respectively the original, the lower bound and the upper bound breakwater design presented. Calculation of the volumes can be found in Appendix XVIII.

Item	Quantity (m^3)	Rate (£)	Total (£)
Main breakwater			
Core-Loc	135,800	58	7,876,400
Rock armour, heavy grading	290,700	9	2,616,300
Core	957,600	5	4,788,000
Cap-block	4,500	40	180,000
Subtotal			**15.5 million**
Lee breakwater			
Rock armour, heavy grading	101,800	9	916,200
Rock armour, light grading	70,700	7.5	530,200
Core	367,700	5	1,838,500
Geo-textile (m^2)	9,200	2	18,400
Cap-block	3,300	40	132,000
Trench dredging	27,600	5	138,000
Subtotal			**3.6 million**
Total cost breakwaters			**19.1 million**

Table 8-1 costs main and lee breakwater

Item	Quantity (m^3)	Rate (£)	Total (£)
Main breakwater			
Core-Loc	112,200	58	6,507,600
Rock armour, heavy grading	271,400	9	2,442,600
Core	845,200	5	4,226,000
Cap-block	4,300	40	172,000
Subtotal			**13.3 million**
Lee breakwater			
Rock armour, heavy grading	87,800	9	790,200
Rock armour, light grading	55,700	7.5	417,800
Core	297,900	5	1,489,500
Geo-textile (m^2)	8,800	2	17,600
Cap-block	3,300	40	132,000
Trench dredging	27,300	5	136,500
Subtotal			**3.0 million**
Total cost breakwaters			**16.3 million**

Table 8-2 costs lower bound main and lee breakwater

Item	Quantity (m^3)	Rate (£)	Total (£)
Main breakwater			
Core-Loc	157,400	58	9,129,200
Rock armour, heavy grading	304,400	9	2,739,600
Core	1,034,100	5	5,170,500
Cap-block	4,800	40	192,000
Subtotal			**17.2 million**
Lee breakwater			
Rock armour, heavy grading	114,000	9	1,026,000
Rock armour, light grading	85,900	7.5	644,200
Core	429,200	5	2,146,000
Geo-textile (m^2)	9,400	2	18,800
Cap-block	3,300	40	132,000
Trench dredging	28,000	5	140,000
Subtotal			**4.1 million**
Total cost breakwaters			**21.3 million**

Table 8-3 costs upper bound main and lee breakwater

8.3 Cost estimation quay wall

The same unit rate technique with same rates has been used to establish the cost estimations for the detailed quay wall design . The costs for the different block-work quay wall designs are presented in Table 8-4, Table 8-5 and Table 8-6.

Block-work original	Item	Quantity	Rate (£)	Costs (£)
Wall – 5m	Dredging (m^3)	44,600	5	223,000
	Rock bed + levelling (m^2)	3,500	40	140,000
	Trench filling (m^3)	4,400	10	44,000
	Concrete + casting (m^3)	19,500	70	1,365,000
	Placing, finishing blocks (no.)	360	1,000	360,000
	Subtotal			**2.1 million**
Wall – 8m	Dredging (m^3)	112,100	5	560,500
	Rock bed + levelling (m^2)	7,500	40	300,000
	Concrete + casting (m^3)	51,800	70	3,626,000
	Placing, finishing blocks (no.)	960	1,000	960,000
	Subtotal			**5.4 million**
Wall – 10m	Dredging (m^3)	162,500	5	812,500
	Rock bed + levelling (m^2)	8,100	40	324,000
	Concrete + casting (m^3)	60,300	70	4,221,000
	Placing, finishing blocks (no.)	1080	1,000	1,080,000
	Subtotal			**6.4 million**
Total cost block-work quay wall, original design				**13.9 million**

Table 8-4 block-work quay wall cost estimation

Block-work lower bound	Item	Quantity	Rate (£)	Costs (£)
Wall – 5m	Dredging (m^3)	31,500	5	157,500
	Rock bed + levelling (m^2)	3,500	40	140,000
	Concrete + casting (m^3)	19,500	70	1,365,000
	Placing, finishing blocks (no.)	360	1,000	360,000
	Subtotal			**2.0 million**
Wall – 8m	Dredging (m^3)	112,100	5	560,500
	Rock bed + levelling (m^2)	7,500	40	300,000
	Concrete + casting (m^3)	51,800	70	3,626,000
	Placing, finishing blocks (no.)	960	1,000	960,000
	Subtotal			**5.4 million**
Wall – 10m	Dredging (m^3)	162,500	5	812,500
	Rock bed + levelling (m^2)	8,100	40	324,000
	Concrete + casting (m^3)	60,300	70	4,221,000
	Placing, finishing blocks (no.)	1080	1,000	1,080,000
	Subtotal			**6.4 million**
Total cost block-work quay wall, lower bound design				**13.8 million**

Table 8-5 lower bound block-work quay wall cost estimation

Block-work upper bound	Item	Quantity	Rate (£)	Costs (£)
Wall – 5m	Dredging (m^3)	119,600	5	598,000
	Rock bed + levelling (m^2)	3,500	40	140,000
	Trench filling	30,800	10	308,000
	Concrete + casting (m^3)	19,500	70	1,365,000
	Placing, finishing blocks (no.)	360	1,000	360,000
	Subtotal			**2.8 million**
Wall – 8m	Dredging (m^3)	186,500	5	932,500
	Rock bed + levelling (m^2)	7,500	40	300,000
	Trench filling	20,400	10	204,000
	Concrete + casting (m^3)	51,800	70	3,626,000
	Placing, finishing blocks (no.)	960	1,000	960,000
	Subtotal			**6.0 million**
Wall – 10m	Dredging (m^3)	172,100	5	860,500
	Rock bed + levelling (m^2)	8,100	40	324,000
	Trench filling	1,900	10	19,000
	Concrete + casting (m^3)	60,300	70	4,221,000
	Placing, finishing blocks (no.)	1080	1,000	1,080,000
	Subtotal			**6.5 million**
Total cost block-work quay wall, upper bound design				**15.3 million**

Table 8-6 upper bound block-work quay wall cost estimation

As has been done in the preliminary design stage the costs per metre quay wall length have been determined, see Table 8-7.

		- 5m	**- 8m**	**- 10m**
Block work	Lower	£ 6,700	£ 9,100	£ 10,700
	Original	£ 7,100	£ 9,100	£ 10,700
	Upper	£ 9,200	£ 10,000	£ 10,800

Table 8-7 costs per meter quay length.

8.4 Cost estimation dredging and land reclamation

Based on the Fugro analysis new rates for the dredging works have been established, Table 8-8. The margin between lower and upper bound has tightened. The rate for the reclamation costs is the same as in the preliminary design stage. Although the rates are the same the Fugro analysis does have its influence on the reclamation costs. The chance that the dredged material is suitable for reclamation has diminished.

		Cost per m^3 (£)			
Dredging	**Reclamation**	**Dredging**	**Disposal**	**Reclamation**	
				– MWL	+ MWL
Lower	Suitable	1.5	0.5	0.5	1
	Unsuitable			1.5	2
Original	Suitable	2	0.5	0.5	1
	Unsuitable			1.5	2
Upper	Suitable	3	0.5	0.5	1
	Unsuitable			1.5	2

Table 8-8 rates per m^3, identification of the boundaries

In Section 7.4 the volumes to be dredged, disposed and reclaimed were identified. With the rates provided in Table 8-8 the total costs per item can be determined, see Table 8-9.

Dredging costs (£)			
Lower bound		13.0 million	
Original		17.3 million	
Upper bound		26.0 million	
Disposal and reclamation costs (£)			
Use of dredged materials		Suitable	Unsuitable
Reclamation	+ MWL	2.7 million	5.5 million
	– MWL	0.5 million	1.6 million
Disposal dredged materials		2.4 million	4.3 million

Table 8-9 dredging, disposal and reclamation costs

By adding the dredging costs and costs for disposal and reclamation, the total costs for the dredging works and reclamation are known, see Table 8-10. For each scenario two sums of total costs are provided. One column presents the total costs if the dredged materials are suitable for reclamation and the other column the total costs in case the dredged materials are unsuitable as reclamation material.

Total costs of dredging and reclamation works (£)		
Use of dredged materials	Suitable	Unsuitable
Lower bound	18.6 million	24.5 million
Original	22.9 million	28.7 million
Upper bound	31.6 million	37.4 million

Table 8-10 total dredging, disposal and reclamation costs per scenario

9 Comparison preliminary and detailed design

9.1 Introduction

In this section costs and design of the different port components are compared between the preliminary and the detailed design phase. First the breakwaters will be looked at in Section 9.2. The influence of the wave climate and the influence of the geotechnical conditions will be discussed in separate sections. Next there are some comments on the design techniques used in the breakwater design.

The quay wall structure costs and designs for the two design stages will be compared in Section 9.3. It will be pointed out what the uncertainties are and what their influence is on the design. Section 9.4 presents the uncertainties in the dredging works and land reclamation. Finally there are some thoughts concerning the cost estimate technique in Section 9.5.

9.2 Comparison breakwater designs and costs

9.2.1 Comparison of the results of the different design stages

When the costs in the different design stages are compared the figures show a striking decrease in costs. In Table 9-1 the costs as calculated in Sections 5 and 8 are presented (in million pound). The major reason for the difference in construction costs is the reduced design wave height in the detailed design phase.

	Main breakwater		Lee breakwater	
	Preliminary design phase	Detailed design phase	Preliminary design phase	Detailed Design phase
Lower bound	16.4	13.3	4.0	3.0
Original	22.8	15.5	4.7	3.6
Upper bound	28.2	17.2	8.5	4.1

Table 9-1 summary of breakwater costs (all costs are in million pound sterling, £)

For determination of the inshore wave climate same offshore wave data were used in both design stages. Although same offshore wave data were used in both design stages, the inshore wave climate changed considerably, this remarkable variation will be explained in Section 9.2.4.

First it will be indicated what caused the major differences in the design, and subsequently the costs, of the main breakwater, respectively the lee breakwater, Section 9.2.2. Section 9.2.3 discusses the influence of the variation in wave data and the influence of the variation in geotechnical data on the breakwater design.

In order to have a clear view on the phenomena that play a role the following structure to describe them has been adopted:

I Qualitative description → **II Quantitative description** → **III Translation into costs**

9.2.2 I Qualitative description of results

Comparison main breakwater design

The differences in cost and design of the main breakwater depend solely on changes in the inshore wave climate. The inshore wave climate is a combination of a certain wave height and a certain wave period.

In two flowcharts the influence of an increased wave height and an increased wave period are presented, Figure 9-1 and Figure 9-2. These flowcharts are valid for the main breakwater, protected with Core-Loc primary armour units.

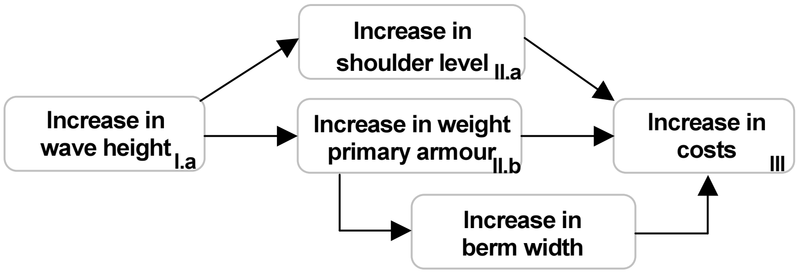

Figure 9-1 influence of an increase in wave height on the construction costs

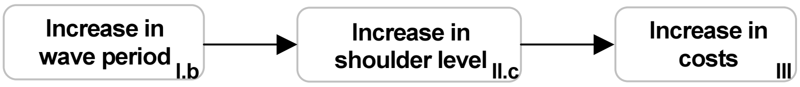

Figure 9-2 influence of in increase in wave period on the construction costs

In essence all six main breakwater designs are equal. Only the size of the breakwater (height, width) and the size of its elements varies between the different design stages and scenarios.

For determination of the inshore wave climate same offshore wave data were used in both design stages, what make the differences even more striking. When taking a close look at the estimated main breakwater costs there are two eye-catching phenomena:

1. the decrease in costs between the preliminary design stage and the detailed design stage;

2. the decrease of the margin between the different scenarios, within a design phase, between the two design stages.

Both phenomena can be explained by the fact that in the detailed design phase a more accurate method was used to determine the inshore wave climate. For the first phenomenon this means that the SWAN-analysis comes to a lower design wave height than what would be derived from a first estimate with a hand calculation, see also Section 9.2.4. The second phenomenon finds its explanation in the fact that margin chosen for the inaccuracy in the wave data can be decreased in the detailed design phase.

It is important to note that a direct comparison between both original breakwater designs can be made. In contrast the lower and upper bound designs depend on values derived from the original design values. Therefore a direct comparison between the two lower bound designs, respectively the two upper bound designs, *can not* be made.

Comparison lee breakwater design

The differences in the lee breakwater designs, and subsequently the costs, depend on changes in the inshore wave climate as well as on changes in the geotechnical conditions. Besides the changes in size of the breakwater and its elements there are also fundamental changes in the lee breakwater design. These changes, which have their reflection on the costs, are:

1. change of size breakwater (shoulder level, width) and its elements;

2. change of type of primary armour units;

3. changes in the toe structure.

1. The change of size of the breakwater and its elements depends on changes in the inshore wave climate. In two flowcharts the influence of an increased wave height and an increased wave period are presented, Figure 9-3 and Figure 9-4. These flowcharts are valid for the lee breakwater, protected with rock primary armour units.

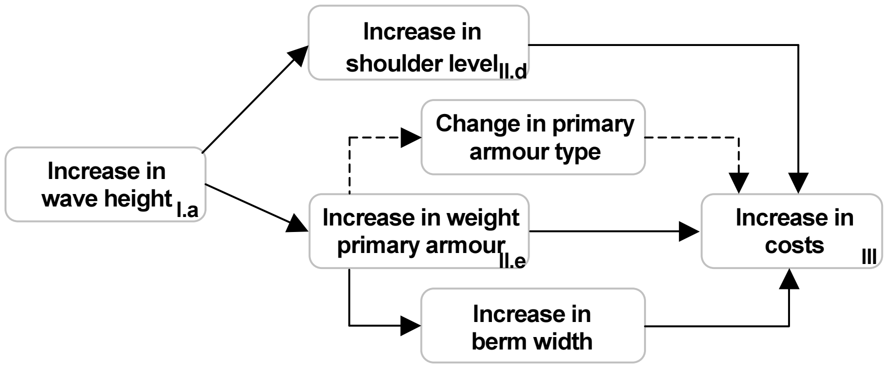

Figure 9-3 influence of an increase in wave height on the construction costs when using rock

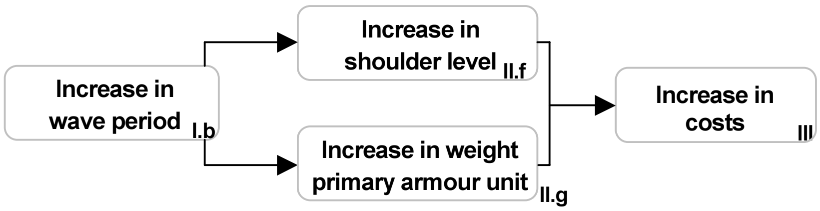

Figure 9-4 influence of an increase in wave period on the construction costs when using rock

2. The change of primary armour unit, from rock to Core-Loc, finds its origin in an increase in wave height and wave period. Rock primary armour units are much cheaper (see Chapter 5) than artificial primary armour units but because of geological limitations of local quarries it is not possible to obtain blocks of quarry stone (rock) with sufficient weight to be stable under all conditions.

When calculating the required weight of rock primary armour units the Van der Meer formula is preferred. This formula does take the wave height and the wave period into account. When calculating the required weight of Core-Loc primary armour units the Hudson formula is preferred. The Hudson formula does not take count of the wave period.

Core-Loc primary armour units were only required in the original (on a small scale) and the upper bound preliminary lee breakwater design. The dramatic increase in costs (by 80%) for the upper bound design finds its explanation in the extensive use of Core-Loc armour units in the upper bound lee breakwater design.

3. The changes in the toe structure depend on changes in the geotechnical conditions as well as on changes in the wave climate. In the preliminary design the primary armour units were placed in a dredged trench. This because the depth in front of the breakwater was equal to the breaker depth of the incoming waves.

In the detailed design two toe structures were applied. The marine deposits under the first part, near the small headland, of the lee breakwater needed to be replaced by core material. The primary armour units were placed on this bed of core material. For the second (deepest) part of the lee breakwater same toe structure as in the preliminary design was applied. For more detail see Section 7.2.

Soil improvement under a breakwater is most times accompanied with a substantial increase in costs. This does not come forward very clear in the cost estimates. This will be further discussed in Section 9.2.5.

9.2.3 Sensitivity analysis to changes in the wave climate

Throughout this study it has become clear that design and costs differ for Core-Loc primary armour units and rock primary armour units. First the sensitivity to changes in the wave climate on breakwater design and costs with Core-Loc armour units will be discussed. Following this will be done when rock is used as primary armour.

Core-Loc primary armour

As shown in the flowcharts in the previous sections a change in the wave climate influences design and costs in several ways. In the following subsections it will discussed what the sensitivity of changes in wave climate on shoulder level and primary armour unit are. Subsequently this will be translated into costs.

- Sensitivity of uncertainties in the wave height and wave period on the shoulder level

The shoulder level depends on the wave height and the wave period. With a known wave height the wave period depends on the steepness of the waves. A steeper wave represents a shorter wave period. An overtopping criterion of 10 l/s/m with a 1:1.5 slope has been used to determine the shoulder levels in Figure 9-5.

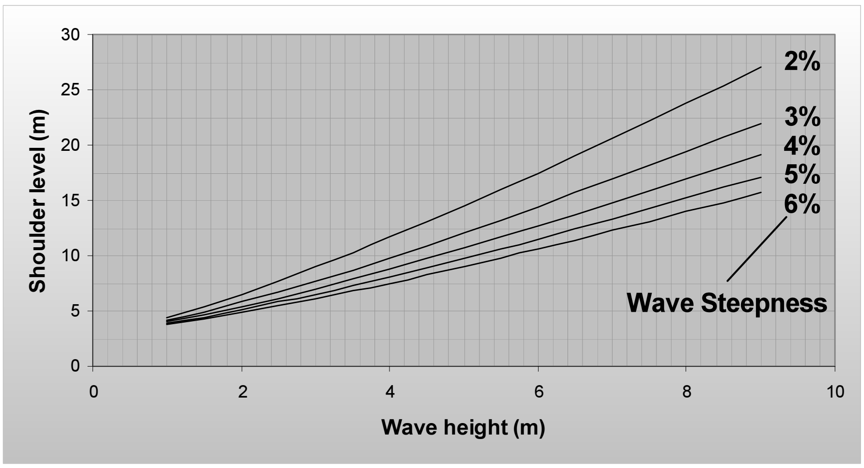

Figure 9-5 wave height - shoulder level ratio for different wave steepness **II.a / II.c**

This graph is applicable for Duqm. A water level of CD + 3.2m was adopted. It shows that a 1-metre increase of the incoming wave can lead to an increase of several metres of the shoulder level. It also shows the importance to know the wave steepness.

- Sensitivity of uncertainties in wave height on type and size of primary armour units

Besides the shoulder level the wave height also plays a dominant role in the determination of size and type of primary armour unit. In Figure 9-6 the ratio between increasing wave height and increasing weight of Core-Loc primary armour units (implicating higher costs) is plotted.

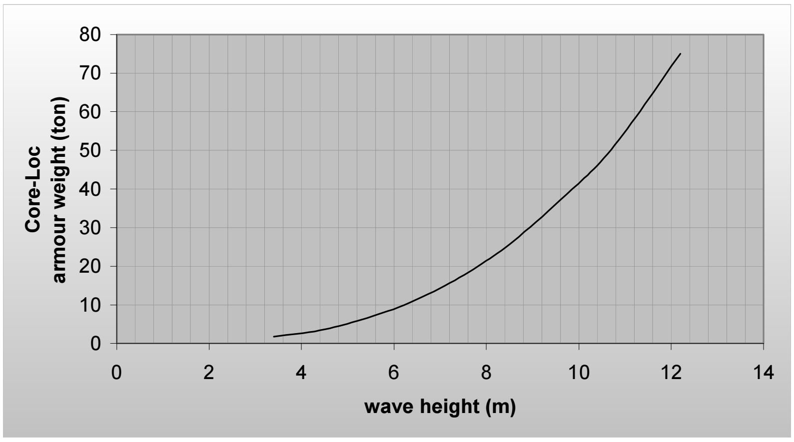

Figure 9-6 Core-Loc primary armour ratio between wave height and armour weight **II.b**

- **III** Translations of uncertainties in wave climate into costs

In Sections 9.2.2 the relations between changes in the design conditions and how they influence the costs were presented in four flowcharts. Following it was shown how changes in the wave conditions influence the design. The results were presented in several graphs. These changes in design will now have to be translated into costs. The most important rates used for the cost estimates are summarised in Table 9-2.

	Rate £ per m^3
Core Loc primary armour	58
Rock armour, heavy grading (> 1000 kg)	9
Rock armour, light grading (< 1000 kg)	7.5
Core material	5

Table 9-2 Summary of cost-rates for breakwater design

In Figure 9-7 are the results presented that are valid for the first two flowcharts (Core-Loc primary armour). A slope of 1:1.5 has been adopted for the designs. The breakwater has been designed in the same way as the main breakwater designs in Sections 4 and 7.

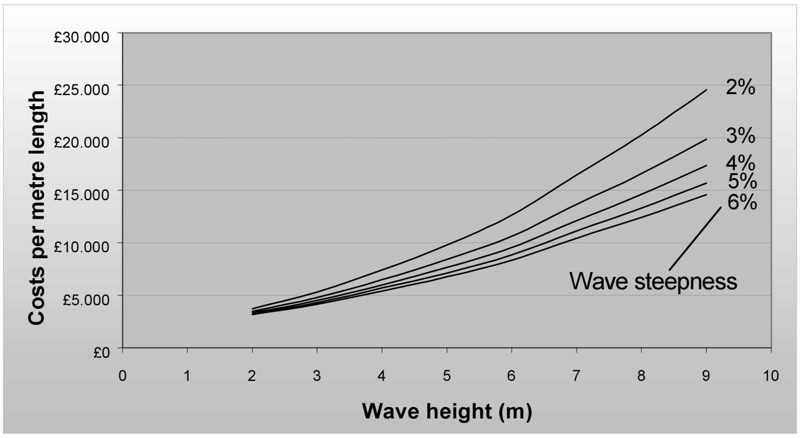

Figure 9-7 effect of an increase in shoulder level on increase of construction material

The sensitivity of a change in wave climate on the shoulder level and on primary armour units is now known. It has also been shown what effect the combination of these two phenomena have on design and costs. Last step to make is to show what their share is in the total construction costs.

- Sensitivity of increase in water depth on breakwater costs

For a breakwater designed in the same way as the main breakwater for this study the costs per metre length have been plotted for several depths with different wave heights, see Figure 9-8. The breakwater is protected with Core-Loc primary armour units. For shoulder level calculations a wave steepness of 3% was adopted.

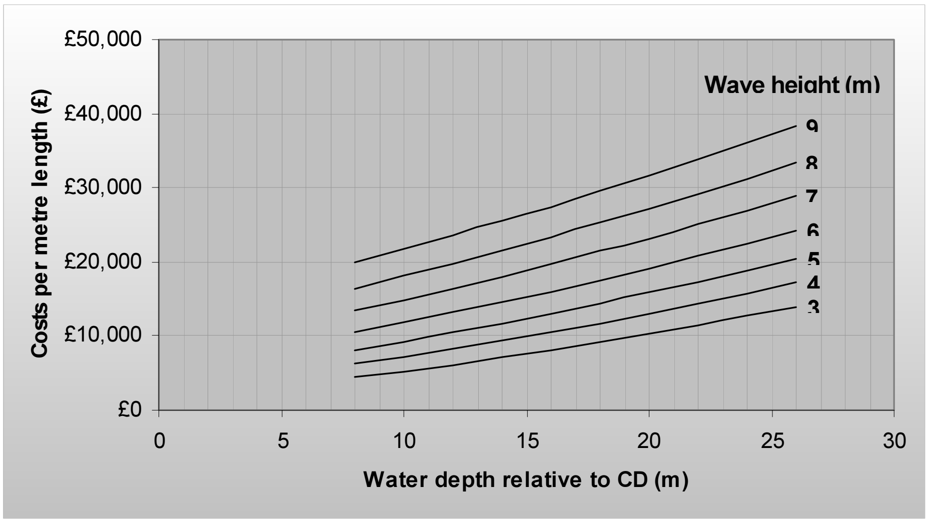

Figure 9-8 breakwater costs per metre length for different water depths and wave heights

As one should expect the costs increase with increasing water depth. To control the costs in future designs it is interesting to know what element of the breakwater dominates the costs. The primary armour and core material together take account for 80% of all costs with a water depth of 8m. This share increases to over 90% for a depth of 26m. In Table 9-3 are the shares in the costs for the primary armour units and core material shown.

		Water depth (m)									
		8	**10**	**12**	**14**	**16**	**18**	**20**	**22**	**24**	**26**
		Share in costs of primary armour									
Wave height (m)	**3**	42	40	38	37	35	34	32	31	30	29
	6	51	49	48	47	45	44	43	42	41	40
	9	53	52	51	50	49	48	47	46	45	44
		Share in costs of core material									
	3	48	50	53	55	57	59	60	62	63	65
	6	38	40	42	44	45	47	48	50	51	52
	9	33	34	36	37	38	40	41	42	43	44

Table 9-3 share in total breakwater costs per metre length, in percentage

With increasing depth the share of the primary armour units in the total costs decreases. In contrast the share of the costs of core material increases with increasing water depth, see

Figure 9-9. This can logically explained by the fact that with increasing depth the width at bottom level of the breakwater increases, see Figure 9-7.

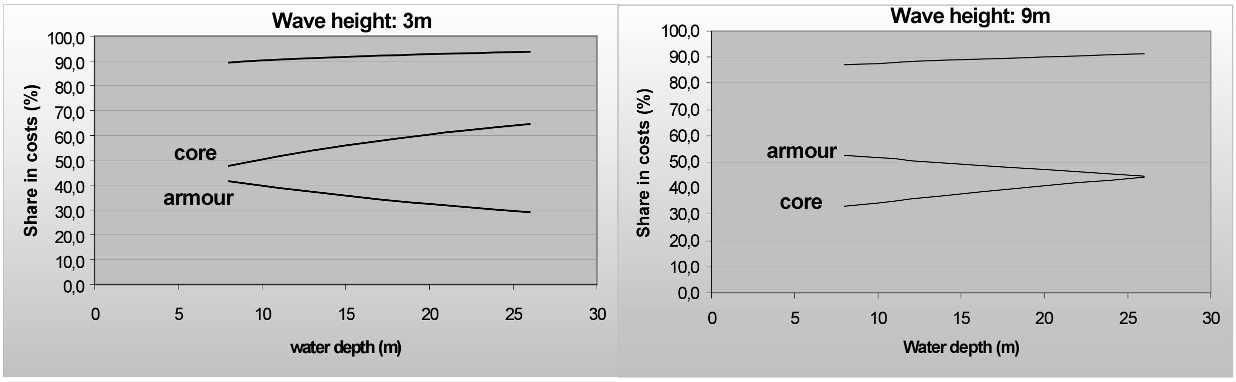

Figure 9-9 share of components in total breakwater costs

Rock

- Sensitivity of uncertainties in the wave height and wave period on the shoulder level

The shoulder level depends on the wave height and the wave period. With a known wave height the wave period depends on the steepness of the waves. A steeper wave represents a shorter wave period. An overtopping criterion of 10 l/s/m with a 1:3 slope has been used to determine the shoulder levels in Figure 9-10.

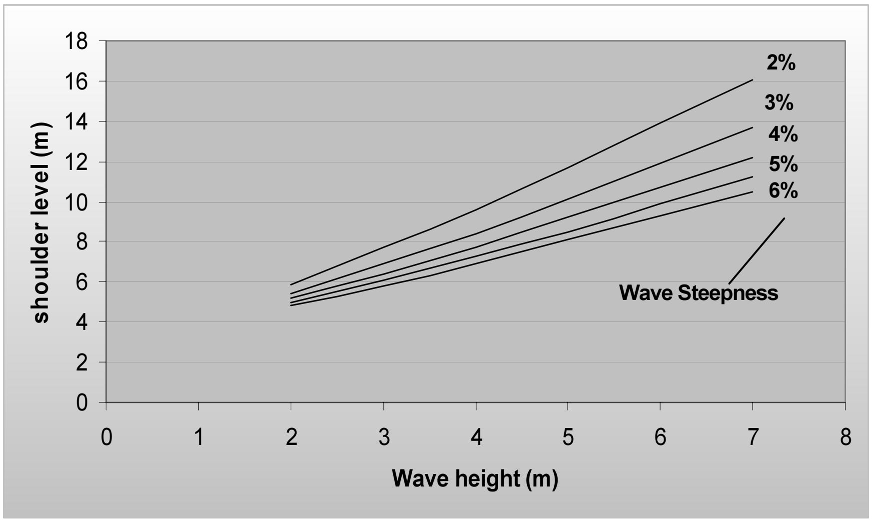

Figure 9-10 wave height - shoulder level ratio for different wave steepness **II.d / II.f**

- Sensitivity of uncertainties in wave height on type and size of primary armour units

As discussed earlier rock armour units depend on wave height as well as wave period. In Figure 9-11 the ratio between wave height and rock armour units is presented for several wave periods. The wave period is presented as the wave steepness. A slope angle of 1:3 has been applied.

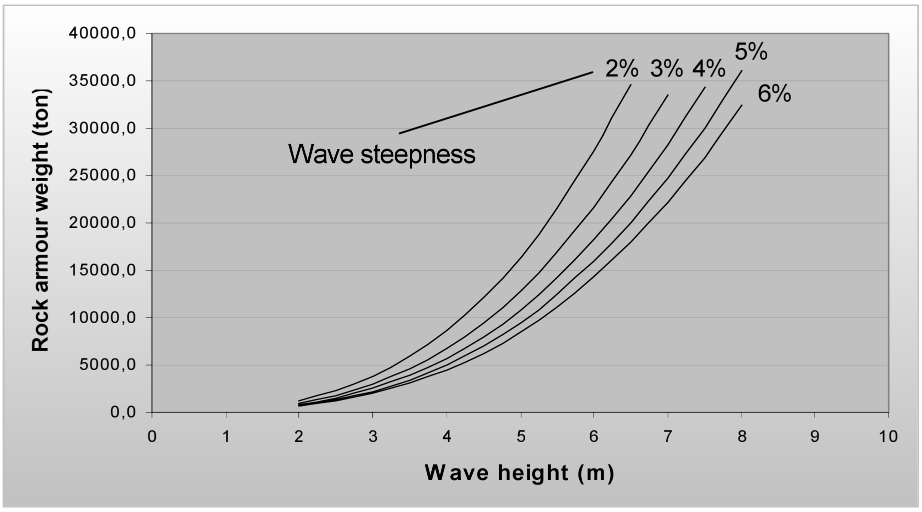

Figure 9-11 rock primary weight with different wave heights and wave periods **II.d**

Rock is a product of nature. Therefore it is not always possible to obtain blocks of sufficient size to be stable under the design conditions. With a milder slope smaller blocks are possible with same wave conditions. The ratio between rock primary armour and the wave height is shown in Figure 9-12 for different slopes. With a steeper slope the run-up of the incoming waves increases, therefore the shoulder level also increases to meet the same overtopping criterion.

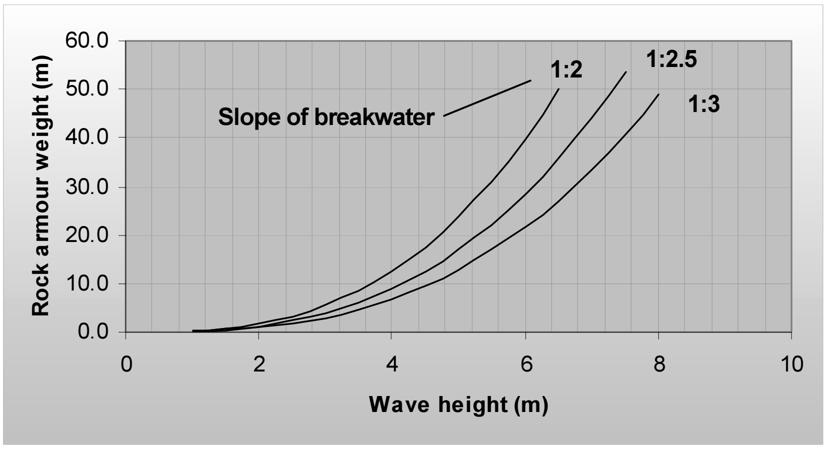

Figure 9-12 rock primary armour ratio between wave height and armour weight, wave steepness 3%

- **III** Translations of uncertainties in wave climate into costs

In Sections 9.2.2 the relations between changes in the design conditions and how they influence the costs were presented in four flowcharts. Following it was shown how changes in the wave conditions influence the design. The results were presented in several graphs. These variations in design conditions will now be translated into costs, Figure 9-13.

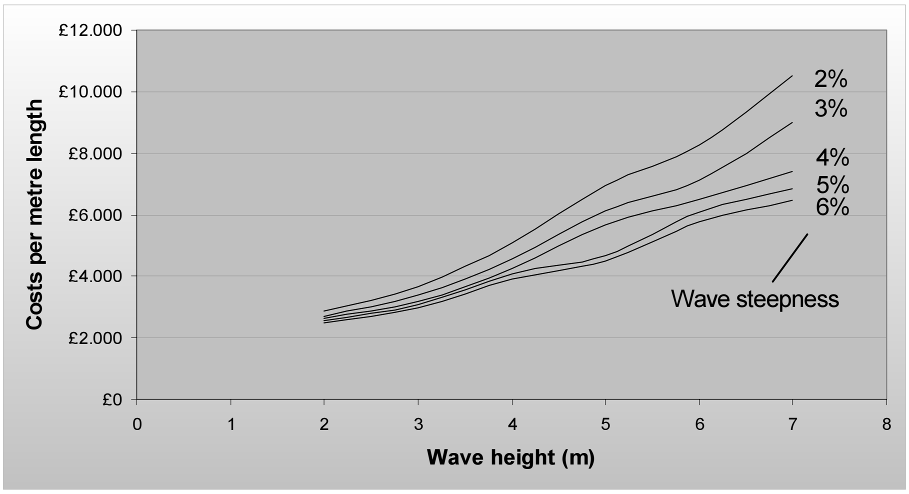

Figure 9-13 cost development for increasing wave height with Rock primary armour III

The lines in Figure 9-13 for rock primary armour have a much more irregular development with increasing wave heights than the lines in Figure 9-7. This can explained by the fact that the costs for primary armour and core material do not play a role as dominant as in the designs with Core-Loc.

The share of rock primary armour and core material have been calculated for several wave heights and wave periods. This been done for a water depth of 10m, and a water level of 3.2 m + CD. A slope of 1:3 was adopted for the design

Wave height (m)	H = 2			H = 4			H = 6		
Wave steepness (%)	2	4	6	2	4	6	2	4	6
Rock primary armour (%)	10	9	8	12	11	11	13	12	12
Core material (%)	73	73	73	55	53	53	48	45	44

Table 9-4 Share of rock primary armour

It was expected that the angle of the seaward slope of the breakwater would play a substantial role in the total costs. From Figure 9-14 it becomes clear that this is not the case.

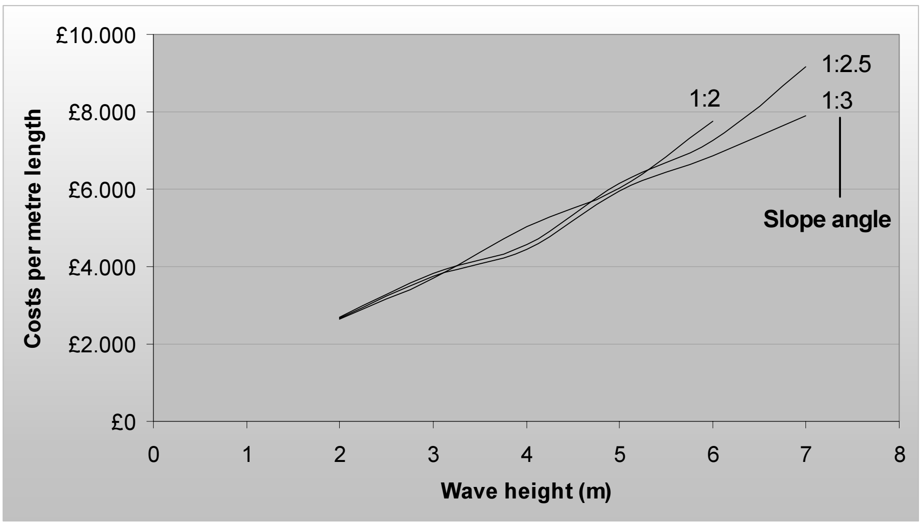

Figure 9-14 cost development for increasing wave height with Rock primary armour

9.2.4 Comments on design methods concerning the wave climate

Comments on design wave height calculation

The inshore wave climate has in both design stages been determined on basis of the same offshore wave data. There is a remarkable difference in the inshore wave climate. This difference can be explained by the fact that the SWAN-analysis does take account of the bathymetric profile and the shape of the land.

When looking at the results one could doubt the applicability of the used methods to calculate the inshore wave height. To verify the reliability of the calculations, the results in this study have been compared with previous studies on Duqm.

	Year	Design wave Height (m)	Comments *
Halcrow	1992	8.4	** In-house computer model
Gibb	1999	7.2	
Posford Haskoning	2002	7.0	
Posford Haskoning	2002	4.2	** Swan analysis *** Design on 5.8m cyclone attack
This study	2002	6.7	
* It is important to be aware of the fact that in all studies it concerned preliminary designs			

Table 9-5 Inshore design wave heights at Duqm in different studies

When taking a closer look at these figures the Halcrow estimate seems very conservative. Their estimate is based on an in-house computer model. The first Posford, the Gibb and the estimate in this study were all hand calculations. They are all three more or less in the same range (approximately 7m).

Without having any further information the results of the SWAN analysis seem very optimistic. As stated before the SWAN analysis is capable of taking account of the shape of the land and the bathymetric profile. The results demonstrate that the relatively shallow (<50m deep) 60-km wide shelf between Duqm and deep water has a significant influence on the wave height. Probably the bottom friction absorbs a lot of the wave energy.

Comments on shoulder level calculations

The height of the crest level of a breakwater depends primarily on the quantity of water allowed to pass over the crest. The quantity of water passing the crest, frequently referred to as the overtopping rate, depends on the incoming wave height, the wave period, the angle of wave attack, slope of the breakwater, width of the crest and the roughness of the slope. For this study different, widely accepted methods were reviewed, see Appendix XIX.

For a 7m-wave height, a wave period of 9 seconds and an overtopping criterion of 10 l/s/m the results are shown in, Table 9-6. For this study the method recommended by HR Wallingford was adopted.

	Recommended Shoulder level (m)	Comments
HR Wallingford	13.9	* used by Posford
Owen	15.2	* from CUR manual
Bradbury	6.9	

Table 9-6 comparison of different overtopping calculation methods

The quantity of overtopping is largely influenced by the nature of the outer slope and the shape and nature of the crest. There is no generally applicable formula for overtopping. Per specific problem one should select the most promising approach.

9.2.5 Sensitivity of the uncertainty in the geotechnical data on the breakwater

The uncertainties or changes in the soil characteristics only influenced the lee breakwater design. When analysing the cost estimates of the lee breakwater it is difficult to point out clearly what the effects of the uncertainties are. Therefore new designs, based on the Duqm conditions have been made. Following designs have been examined:

1. a design with ideal conditions, a standard toe structure can be constructed on the sea bottom;

2. 3 designs with unfavourable soil conditions, respectively 2, 4 and 6 metre of bad materials will have to be replaced under the seaward toe. A standard toe structure will be applied.

Cost estimates per metre length for these designs have been made, see Table 9-7. These designs were based on a 3m, a 4m and a 5m design wave height with a steepness of 3%, a water depth of CD – 10m and a water level of + 3.2m.

		Wave height (m)		
		3	**4**	**5**
Situation 1	Armour standard toe	3,400	4,600	6,100
Situation 2a	2m of bad material	3,900	5,100	6,800
Situation 2b	4m of bad material	4,500	5,900	7,600
Situation 3c	6m of bad material	5,200	6,700	8,500

Table 9-7 costs per metre length for different soil conditions

9.3 Comparison quay wall designs and costs

9.3.1 Comparison of the two design stages

The quay wall design is dominated by the soil conditions. In the preliminary design phase the subsoil under the breakwater was assumed to consist mainly of bedrock, covered with marine deposits. The marine deposits of recent years were considered to be too soft to support a quay wall and will have to be removed.

In the detailed design phase new information from an old site investigation was available. From this investigation it appeared that the materials under most of the site were weaker than expected.

In the preliminary design phase two structure types were adopted for further design, a block-work wall and a caisson wall. In the detailed design the block-work wall was preferred, see Section 7.3.3. In Figure 9-15 the costs for the different quay walls as estimated in the preliminary design phase are presented.

		Preliminary design phase		Detailed design phase
Structure type		Caisson wall	Block-work wall	Block-work wall
		lower bound design		
Quay wall depth	**5m**	1.7	1.9	2.0
	8m	5.1	5.7	5.4
	10m	6.5	6.8	6.4
	Total	**13.3**	**14.4**	**13.8**
		original design		
Quay wall depth	**5m**	1.8	2.0	2.1
	8m	5.2	6.0	5.4
	10m	6.7	7.2	6.4
	Total	**13.7**	**15.2**	**13.9**
		upper bound design		
Quay wall depth	**5m**	2.0	2.1	2.8
	8m	5.7	6.4	6.0
	10m	7.0	7.5	6.5
	Total	**14.7**	**16.0**	**15.3**

Figure 9-15 summary of quay wall costs, (in million £)

		- 5m	- 8m	- 10m
Preliminary design				
Caisson	Lower	£ 5.800	£ 8,600	£ 10,800
	Original	£ 6,200	£ 8,700	£ 11,200
	Upper	£ 6,500	£ 9,500	£ 11,600
Block work	Lower	£ 6,200	£ 9,500	£ 11,300
	Original	£ 6,700	£ 10,100	£ 11,900
	Upper	£ 7,100	£ 10,600	£ 12,100
Detailed design				
Block work	Lower	£ 6,700	£ 9,100	£ 10,700
	Original	£ 7,100	£ 9,100	£ 10,700
	Upper	£ 9,200	£ 10,000	£ 10,800

Figure 9-16 summary of quay wall costs per metre length

When taking a close look at the estimated quay wall costs there are three eye-catching phenomena:

1. When we look at the – 5m quay we see an increase in the costs, when comparing the preliminary and the detailed design phase. Controversially, the costs for the – 10m quay wall decrease.

2. In the detailed design phase there is almost no difference between the lower and upper bound designs for the – 8m and – 10m quay walls.

3. The caisson cost estimates are lower, nevertheless the block-work quay wall structure has been adopted for further design.

1. This phenomenon finds its origin in the changed soil conditions and in the more accurate method adopted for the design. First the increased costs for the – 5m quay wall. From the new information available in the detailed design phase it appeared that the layers of weak materials, overlying the layers strong enough to bear the quay wall, were thicker than assumed in the preliminary design phase. These layers will therefore have to be removed. The difference in the – 5m quay wall designs finds its origin in the variation of the thickness of these layers of bad material.

In the detailed design phase a more accurate design method was used to determine the size of the blocks. This resulted in a reduced length of some of the blocks. This explains the reduced costs for the – 10m quay wall.

2. The difference in design conditions for the detailed quay wall was the uncertainty in thickness of the weak materials. The variation in thickness influenced the – 5m design to a large extent, explaining the wide margin in costs between the different scenarios. Under the –8m and the –10m same soil conditions are to be expected. The variation in the thickness of the layers of bad material hardly influence these designs. This because the –8m and – 10m quay wall are reaching onto the layers of strong material.

3. In Section 7.3 this has already been discussed. The main reason to select the block-work quay wall structure for further design was because of the way of construction. When caissons would have been used the construction of the port would take considerably more time.

9.4 Comparison dredging and reclamation costs

9.4.1 Comparison of the two design stages

A simplified method was used to determine the costs of dredging works and land reclamation in this study. The total costs for dredging works and land reclamation where obtained by multiplying the estimated volumes with a certain rate. In this rate were all costs, for example mobilisation of equipment, incorporated. The rates applied depend on the soil characteristics. In Table 9-8 estimated volumes and costs are summarised.

		Dredged material suitable for land reclamation		Dredged material *un*suitable for land reclamation	
		Preliminary design phase	Detailed design phase	Preliminary design phase	Detailed Design phase
Volume to be dredged (m^3)		8.8 million	8.7 million	8.8 million	8.7 million
Volume to be reclaimed (m^3)		3.7 million	3.8 million	3.7 million	3.8 million
Costs (million £)	Lower bound	18.8	18.6	24.4	24.5
	Original	32.0	22.9	37.6	28.7
	Upper bound	45.1	31.6	50.7	37.4

Table 9-8 summarisation of volumes and costs of dredging works and land reclamation

When analysing the presented results there are three points that attract attention:

1. the estimated volumes hardly differ between the two design phases;

2. the decrease in costs between the preliminary design and the detailed design;

3. whether or not the dredged materials are suitable for land reclamation the absolute margins between the different scenarios are nearly the same .

1. For both design phases same survey data on the bathymetry of the site were used, the survey was undertaken by Nortech Surveys Ltd in 1989. Compared to the preliminary design phase a more accurate method was used to calculate the volumes to be dredged and reclaimed in the detailed design phase, see Section 7.4.

Despite the use of a more accurate method to estimate the volumes to be dredged and reclaimed these volumes hardly differ. The lack of detail in the first method does not seem to influence the outcome volumes to a large extent. As assumed in Section 4.4 it is likely that overestimations counterbalance underestimations.

2. The decrease in costs can simply be explained by the use of lower rates in the detailed design phase. In the preliminary design stage the subsoil under the marine deposits was assumed to consist of rock. From the Fugro boreholes it appeared that weaker materials are to be found, see Section 6.3. The weaker materials will have a lower resistance to dredging, this expresses itself in a lower rate for the dredging costs.

3. The explanation for the phenomenon that the absolute margins do not change is that the rate for disposal of dredged materials and the rate for reclamation material from a borrow area are not influenced by changes in assumptions concerning the soil characteristics in the vicinity of the construction site.

9.5 Comments on the cost estimating technique

In this research the unit rate estimating technique has been The unit rate estimating technique relies on historical data of various kinds. The associated risks of using historical data in estimating are important to recognise. Major drawbacks on the cost estimations in this research were:

- The small sample of comparable projects suitable to obtain data from
- Different currencies used in previous projects
- Different currencies had different inflation rates
- Different projects have different definitions of what costs are included

Because of these drawbacks the level of detail presented in the detailed design phase cost estimates has not been refined compared to the preliminary design phase. A refinement could introduce a real danger that precision and detail generated could give a misplaced level of confidence in the figures.

10 Conclusions and recommendations

10.1 Introduction

The initial view stated in the introduction and problem description at the starting point of this study, was the importance of the need to make the first estimate realistic, despite the lack of accurate data and the presence of unquantifiable factors.

Controversially, to realise this objective there is a fundamental need for relevant data. The main problem areas relate to difficulties in obtaining of and access to data, and with the methodology for the manipulation and interpretation of these data. To control part of these problem areas there is the need to know what the sensitivity of outcome construction costs due to variation in the site investigation data is.

10.2 Conclusions regarding the research

In port projects like Duqm the geotechnical and wave data are of uttermost importance. On basis of the data available it has been indicated what design margins could realistically be expected for the breakwater design, the quay wall design and the dredging works and land reclamation.

With the available information the sensitivity on costs of variations in the wave climate have been analysed. The results of this analysis are valid for Duqm but they provide a good view for comparable projects. Contrary, the limited geotechnical information available made it only possible to give an indication of the margins to be expected at Duqm. Unfortunately, it is therefore not possible to give a detailed analysis based on the Duqm geotechnical conditions that could be applicable in comparable circumstances.

10.3 Conclusions regarding the site investigation data

Wave climate

The inshore wave climate was established on basis of the same offshore wave data. However the interpretation and manipulation of these data was different in both design stages, causing remarkable differences.

Preliminary design phase:

- hand calculations were used to determine inshore wave climate
- waves increase when approaching the shore
- wave heights ranging from 5.3m up to 8.0m
- no cyclone attack

Detailed design phase:

- determination of inshore wave climate with the aid of SWAN
- waves decrease when approaching the shore

- wave heights ranging from 3.8m up to 4.6m
- cyclone attack taken into account

Cyclones

Based on a study by Baird and Associates (2001) it was stated that in the preliminary design phase the occurrence of cyclones was not decisive for the design wave height. In contrast, the occurrence of cyclones was taken into account in the detailed design phase on basis of an earlier study by Halcrow (1992).

The height of incoming waves during a cyclone attack could reach up to 15m. With the aid of SWAN, the design wave height was estimated at 5.8m in the detailed design phase. When the height of the waves during a cyclone attack would have been calculated in the preliminary design phase, they would reach up to 12m. The depth in front of the main breakwater is the limiting factor for these waves.

Geotechnical conditions

In the preliminary design phase the proposed site was expected to be underlain with hard rock, on basis of the Geological Map for Duqm. From the Fugro boreholes it appeared that weaker materials are to be found underneath much of the site. However safe conclusions can not be drawn.

The site investigation covers only part of the proposed port site, it is therefore not necessarily fully representative of the site as a whole and the small shallow bay in particular.

10.4 Conclusions regarding the designs

Breakwater

The use of rock as primary armour unit is the cheapest solution. Because of geological limitations of local quarries it is not possible to obtain blocks of quarry stone with sufficient weight to be stable under all conditions. When rock cannot be used as primary armour, Core-Loc units are preferred as primary armour.

Preliminary main breakwater design:

- Core-Loc primary armour units range from 6.3 ton up to 21.4 ton
- Shoulder level range from 11.4m up to 16.0m
- Costs range from £16.4 million up to £28.2 million

Detailed main breakwater design:

- Core-Loc primary armour units range from 6.0 ton up to 11.1 ton
- Shoulder level range from 8.5m up to 10.5m
- Costs range from £13.3 million up to £17.2 million

Preliminary lee breakwater design:

- Core-Loc as well as rock as primary armour units

- Costs range from £4.0 million up to £8.5 million

Detailed lee breakwater design:

- Only rock required as primary armour units
- Costs range from £3.0 million up to £4.1 million

Quay wall

In the preliminary design phase steel quay wall structures were precluded because of ALWC (accelerated low water corrosion). A caisson quay wall and a block-work quay wall solution seemed equal suitable. For both quay wall structures designs were presented (for a 5m, an 8m and a 10m deep quay wall). Boundaries in the preliminary design phase were set on the backfill material. The costs for the caisson quay wall were slightly lower.

- Range of total costs caisson quay walls: £13.3 million up to £14.7 million.
- Range of total costs block-work walls: £14.4 million up to £16.0 million.

Despite the higher cost estimation in the preliminary design phase the block-work quay wall structure was adopted for further design. The buildability and reliability of a block-work wall outweigh the higher construction costs.

- Range of total costs detailed block-work wall designs: £13.8 million up to £15.3 million.

Dredging works and land reclamation

The method used in the preliminary design to calculate the volumes is very coarse; the estimates of the different volumes are therefore likely to be subject to some variation. However it is likely that an overestimation may counterbalance an underestimation. A more accurate calculation method in the detailed design phase showed comparable volumes to be dredged or reclaimed.

- Volumes to be dredged: preliminary design 8.8 m^3, detailed design 8.7 m^3.
- Volumes to be reclaimed: preliminary design 3.7 m^3, detailed design 3.8 m^3.

With changing soil conditions the costs margins on the costs changed considerably:

- Preliminary design: £18.8 million up to £50.7 million.
- Detailed design: £18.6 million up to £37.4 million.

10.5 Conclusions regarding the sensitivity to variations in the data

- Weight of rock armour units is calculated with the Van der Meer formula, wave height as well as wave period have a significant influence.
- Weight of Core-Loc armour units is calculated with the Hudson formula, the wave height is decisive; the wave period has no influence.
- The rock armour – wave height ratio has an exponential character. The wave period also plays an important role.

- The Core-Loc – wave height ratio has an exponential character. For example: 20% increase of an 8m wave results in a 100% increase in weight of the required Core-Loc armour units.

- The wave height – shoulder level ratio has an exponential character. Also the wave period plays an important role. With same overtopping criteria and wave height a 2% steep wave can require a shoulder level almost twice as high as a wave with a steepness of 6%.

10.6 Conclusions regarding the cost-structure

- Costs for Core-Loc breakwaters are dominated, approximately 90%, by costs for primary armour and core material.

- In breakwaters that are protected with rock primary armour, costs for under-layer become also an important factor.

- If rock can be used as primary armour instead of Core-Loc this gives a substantial reduction of the costs.

10.7 Conclusions regarding the design methods

Although it was not the objective of this study the influence of preferred design methods could not be neglected.

- Widely accepted crest height calculation methods (Bradbury, Owen and HR Wallingford give significant recommended crest heights. The difference between the extremes is more than 100%.

- The SWAN-analysis results in a 40% decrease in wave height, compared to the hand calculations.

10.8 Recommendations

Based on the experiences during the research and the results of the research, following is recommended:

Design of port components concerning Duqm:

- For future expansions the use of a caisson quay wall structure could be a promising solution.

- A further study on the effects of accelerated low water corrosion at Duqm. Especially when from a new site investigation it would be concluded that there is no rock to be found under the proposed site.

- When new data are available on the geotechnical conditions the detailed design should be reviewed.

Design of port components in general:

- There are no generally applicable formula in breakwater design. For each specific problem it is advised to study the available literature and select the most promising approach.

- At the beginning of the design cycle it should be identified what the specific influence is of certain criteria on design and costs.

Further Research:

- The research on sensitivity of variations in the wave climate should be extended. For example the overtopping criteria could be varied.

- A no damage criterion was adopted for the design; when some damage would be accepted during the lifetime this could reduce the construction costs. The costs for maintenance would increase. A study should be carried out if total costs could be reduced with the acceptance of small damage.

11 Port of Duqm: present state of affairs

11.1 Developments during the study

The developments of the project till the beginning of April 2003 will be briefly discussed in this section.

The project officially started on the 5th of June 2002. End of June 2002 a tender document for a site investigation was prepared, because of the monsoon the tender was postponed. At the start of this study in October 2002 a number of companies were invited to tender. It was expected that this site investigation project would be awarded within days and site investigation works could commence within the matter of weeks. However works regarding the site investigation have yet still not been awarded by the Omani Government. With the Khareef (south-west monsoon) starting early June it is not to be expected that the site investigation, which will take approximately 8 weeks, will start before the end of this summer.

The Master Plan presented by Posford Haskoning in their Draft Final Report was adopted for this study. After submission of the Draft Final Report and consultation with the client the original port layout has been refined. The footprint of the port has not changed and the breakwaters are still in their original position. After reconsideration of some design criteria the main breakwater has been subject to changes, resulting in a reduced shoulder level.

The arrangement of the operational components within the port has also changed. Primary reason was to create some distance between the shipyard operations and the fish facilities. On suggestion of one potential shipyard operator the layout has been modified to allow future expansion of the shipyard up to 400,000 DWT ships. Space has been reserved for a 300,000 DWT and a 400,000 DWT graving dock, alongside the two floating docks. The new Master Plan layout and a layout including the two graving docks can be found in Appendix XX.

11.2 Port of Duqm?

Will there be a port at Duqm? The original and official reason for this project was to stimulate development of the region and bring employment opportunities to the area. Therefore a decision to go to the next design phase with this project should depend on financial, socio-economic and environmental factors. Most important facts are summarised below.

Financial

The costs involved with this project would approximately be £ 175 million for the port infrastructure and another £ 175 million for land infrastructure, an industrial area and a residential area. These costs are expected to be shared between the public sector (Government) and the private sector (investors). Major part (75%) will have to be publicly funded. Besides, the port and shipyard would require £ 2 million annual subsidies. Some investor interest in the shipyard has already been identified.

Socio-economic

The port would probably employ about 1,200 people. It can be expected that the project will attract considerable development to the region, which will provide a further 1,350 jobs. There will be a demand for new houses, shops, hotels, schools etc.

Environmental

The coastline near Duqm is notable for its ecological quality. The construction and operation of the port and dry-dock complex will have adverse environmental implications. The coastal zone near Duqm contains highly significant and sensitive habitats for endangered species, such as: coral reefs, mangroves and turtle nestling.

The most serious environmental considerations are considered to be:

- Endangered cetacean species are known to breed offshore of Duqm.
- Sediment plumes arising from dredging operations (duration approximately one year).
- Contamination and introduction of alien species, principally arising from discharge of ballast water.
- Contamination by airborne and waterborne spread of chemicals in paints arising from grit blasting operations in the shipyard.

Conclusion

The port and dry-dock complex are not viable from a financial or ecological point of view. Investments can be justified from an socio-economic point of view. The vital decision to invest £500,000 in a site investigation has been made but has yet not been approved by some authorities. However the decision to abandon the plan for a port and dry-dock complex at Duqm has also not been made.

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