Design tool for an offshore harbour MSc Thesis

Ala-Eddine Kassimi



JDelft

Design tool for an offshore harbour

by

Ala-Eddine Kassimi

to obtain the degree of Master of Science at the Delft University of Technology, to be defended publicly on Monday August 29, 2016 at 13:30.

Student number:	4166280	
Project duration:	November 9, 2015 – August 29, 2016	
Thesis committee:	Prof. ir. T. Vellinga,	TU Delft, supervisor
	Drs. ir. J. Verlaan,	TU Delft
	Ir. B. Wijdeven,	TU Delft
	Ir. P. B. Beimers,	Royal HaskoningDHV
	Ir. A. van der Hout,	Deltares

An electronic version of this thesis is available at http://repository.tudelft.nl/.



Summary

Harbours were traditionally constructed in sheltered locations, either naturally sheltered or artificially sheltered by means of a breakwater. Due to the increased size of ships and the desire to avoid extensive dredging, harbours are nowadays more often constructed in deeper water at relatively exposed locations. A detached breakwater is usually constructed to provide shelter for the ships at the berth. Several design choices have to be made in the design of these offshore harbours. One of these choices concerns the offshore distance of the berth, denoted in this study as x_{ship} . A large offshore distance is advantageous in the sense that high dredging costs are avoided. On the other hand, the jetty and breakwater dimensions increase which means that the costs for these structures will also increase. Moreover, the short wave conditions far offshore are generally more rough, which may be a problem considering the limited degree of sheltering. Close the coast the jetty and breakwater costs are lower. However, extensive dredging is necessary as the water depth is usually not sufficient. Moreover, the effect of reflected infragravity waves is larger near the coast. The wave period of these infragravity waves is generally close to the eigenperiod of the moored vessel, which means that resonance may occur, leading to excessive vessel motions which can cause downtime. The length of the breakwater, L_{BW} , is also an important parameter: the longer the breakwater, the more expensive it becomes, but also the larger the degree of sheltering.

The port engineer thus has to find a balance between capital and operational costs on one hand and sufficient uptime on the other hand. In this study, a design tool is developed that may help the port engineer in finding this balance. The main goal of the design tool is to recommend an optimum offshore harbour lay-out (i.e. offshore distance of the berth and breakwater length) based on the CAPEX, OPEX and the downtime caused by wave action. The design tool is meant to be applied for offshore dry bulk harbours with one berth along the West-African coast. The offshore harbour considered consists of three elements: a detached breakwater, an exposed jetty and dredging works. Two breakwater types are assessed: a caisson breakwater and a rubble mound breakwater with concrete armour units. Several relevant parameters and processes have been identified that have to be taken into account for the design of an offshore harbour. PIANC guidelines can be used to determine the dimensions of the turning circle and approach channel. The breakwaters are dimensioned based on design wave conditions. Downtime at the berth and tugboat unavailability are determined based on operational wave conditions. The wave height at the berth and during the tugging journey is determined using linear wave theory combined with several empirical formulas. This combination enables the design tool to account for shoaling, refraction, wave breaking, diffraction, transmission and infragravity waves.

The design tool consists of four modules. The cost module determines the CAPEX and OPEX for the jetty and dredging works as a function of the offshore distance of the ship. The breakwater module designs the breakwaters and determines the CAPEX and OPEX of the breakwaters at four different lengths (1, 2, 3 and 4 times the length of the design vessel). The wave calculation module determines the short and long wave height at the berth and the significant wave height during the tugging journey for each harbour lay-out. Based on these wave heights it is determined whether operational and/or navigational downtime occurs. The waiting cost module translates the downtime figures into waiting costs. A total of nine total cost functions of x_{ship} are obtained: one for the case without a breakwater, four for the a harbour with a rubble mound breakwater

and four for a harbour with a caisson breakwater. The total costs are equal to the sum of the CAPEX, OPEX and waiting costs. The design tool recommends the combination of x_{ship} , L_{BW} and breakwater type with the lowest total costs as an optimum harbour lay-out.

The design tool has been verified and evaluated. The result of the verification was that the reflection of infragravity waves off the coast is generally underestimated, which means that the long wave height at the berth is lower than obtained using wave modelling software. Moreover, neglecting short wave reflection off the breakwater may lead to an underestimation of the short wave height for caisson breakwaters. The reliability of the downtime prediction largely depends on the value of C_x chosen in Mol's empirical formula. Furthermore, a sensitivity analysis has been performed in which the effect of uncertainties in several input parameters on the output of the design tool has been examined. Deviations in the input parameters considered may affect the feasibility and the attractiveness of certain harbour lay-outs.

Contents

Su	mma	ary	iii
Lis	st of	Figures	ix
Lis	st of	' Tables x	iii
Lis	st of	Symbols	xv
Ac	knov	wledgements	vii
1	Intr	roduction	1
	1.1	Background and problem statements	1
		1.1.1 Background	1
		1.1.2 Problem description	1
	1.2	Objectives and research questions	3
		1.2.1 Objectives	3
		1.2.2 Research questions \ldots	3
	1.3	Scope	3
	1.4	Methodology	4
		1.4.1 Phase 1: Literature study	4
		1.4.2 Phase 2: Development of the design tool	4
		1.4.3 Phase 3: Verification and evaluation	5
2	Ger	neral overview of the design tool	7
	2.1	Background information	7
		2.1.1 Motivation for offshore harbours	7
		2.1.2 Possibilities and challenges	8
	2.2	Harbour lay-out	9
		2.2.1 Boundary conditions	9
		2.2.2 Demands and requirements	11
		2.2.3 Chosen lay-out	12
	2.3	Goals and demands	16
		2.3.1 Primary and secondary goals	16
	2.4	2.3.2 Demands	16
	2.4	Conclusions	17
3	Rel	evant parameters and processes	19
	3.1	Parameters influencing the design of offshore harbours	19
		3.1.1 Design of dredged areas	19
		3.1.2 Breakwater design	20
	3.2	Parameters influencing the downtime	22
		3.2.1 Behaviour at the berth	22

		3.2.2 Tugboat availability $\ldots \ldots 25$
	3.3	Relevant wave processes
		3.3.1 Incorporation of wave processes
		3.3.2 Nearshore wave propagation
		3.3.3 Infragravity waves
		3.3.4 Wave penetration
	3.4	Conclusions
1	Stri	cture of the design tool 37
4	A 1	Introduction 37
	4.1	$4.11 \text{Software} \qquad \qquad 37$
		$4.1.1 \text{Software} \qquad \qquad$
		4.1.2 Giobal Structure
	19	Cost modulo
	4.2	$421 \text{Input parameters} \qquad 40$
		$4.2.1 \text{input parameters} \dots \dots$
		$4.2.2 \text{Jetty costs} \dots \dots \dots \dots \dots \dots \dots \dots \dots $
	19	4.2.3 Dreaging costs
	4.0	$\begin{array}{cccccccccccccccccccccccccccccccccccc$
		4.3.1 Design wave neight
		4.3.2 Rubble mound breakwater
	4 4	4.3.3 Calsson breakwater
	4.4	wave calculation module $\dots \dots \dots$
		$4.4.1 \text{input parameters} \dots \dots$
		4.4.2 Undisturbed wave propagation
		$4.4.3 \text{Breakwater effect} \qquad 64$
	4 5	4.4.4 Downtime prediction
	4.5	Waiting cost module
		4.5.1 Input parameters
	1.0	4.5.2 Total cost function
	4.6	Example run
		4.6.1 General input parameters
		$4.6.2 \text{Cost parameters} \dots \dots \dots \dots \dots \dots \dots \dots \dots $
		$4.6.3 \text{Wave calculation} \dots \dots \dots \dots \dots \dots \dots \dots \dots $
		$4.6.4 \text{Results} \dots \dots \dots \dots \dots \dots \dots \dots \dots $
	4.7	Conclusions
5	Ver	fication of the design tool 81
	5.1	Introduction
	5.2	Verification 1: undisturbed wave propagation
		5.2.1 Methodology $\ldots \ldots \ldots$
		5.2.2 Input parameters
		5.2.3 Results and discussion $\ldots \ldots \ldots$
	5.3	Verification 2: wave penetration
		5.3.1 Project information
		5.3.2 Input parameters and methodology
		5.3.3 Results and discussion $\ldots \ldots $
	5.4	Verification 3: downtime prediction
		5.4.1 Project information
		5.4.2 Input parameters and methodology
		5.4.3 Results and discussion
	5.5	Conclusions

6	Sen	sitivity analysis	105
	6.1	Introduction	105
		6.1.1 Motivation for the sensitivity analysis	105
		6.1.2 Methodology and input parameters	105
		6.1.3 Analysis 0: Reference case	107
	6.2	Analysis 1: Waiting costs	107
	6.3	Analysis 2: Maximum allowable wave height at berth	110
	6.4	Analysis 3: Maximum allowable wave height for tugboats	112
	6.5	Analysis 4: Maximum allowable overtopping volume	113
	6.6	Analysis 5: Step size	115
	6.7	Conclusions	118
	0.1		110
7	Con	nclusions and recommendations	119
	7.1	Introduction	119
	7.2	Conclusions	120
	7.3	Reflection	121
	7.4	Recommendations	122
Α	Env	rironmental analysis of West-Africa	125
	A.1	Wave conditions	125
		A.1.1 Operational wave conditions	125
		A.1.2 Design wave conditions	127
	A.2	Wind conditions	129
		A.2.1 General	129
		A.2.2 Governing wind conditions	130
		A.2.3 Limitations on wind speed	130
	A.3	Currents	131
		A.3.1 General	131
		A.3.2 The Guinea Current	131
	A.4	Tides	132
	A.5	Bathymetry	132
Б	Б		100
в	Res	ults verification	139
	B.1	Verification 1: undisturbed wave propagation	139
	B.2	Verification 3: downtime prediction	139
\mathbf{C}	Mo	delling of moored vessel responses	145
	C.1	Modelling of nearshore wave propagation	145
	0.1	C11 Model classes	145
		C12 Phase averaging models	146
		C 13 Phase resolving models	1/8
	C_{2}	Modelling of wave body interaction	159
	C_{2}	Dynamic mooring analysis	152
	0.3	C 2.1 Transford frequencies	150
		C.3.1 Types of moorning analysis	153
		$\mathbf{U.3.2} \mathbf{AINYSIM} \dots \dots \dots \dots \dots \dots \dots \dots \dots $	154
D	Scre	eenshots of the design tool	157
Bi	bliog	graphy	161

List of Figures

1.1	Example of an offshore harbour consisting of a detached breakwater and an exposed jetty. [11]	2
2.1	World map with locations where an open container port could be feasible for a Super Past Parameter was a location with MagnMaster TM units [15]	0
0.0	Super-Post Panamax vessel, when mooring with MooriMaster ^{-1,2} units [15]	9
2.2	The geographical area for which the environmental conditions will be assessed. [13]	10
2.3	Determination of the average orientation of the west-African coast. [13]	11
2.4	Harbour lay-out with a mooring arrangement perpendicular to the coast.	13
2.0 9.6	Comparison of multile mound and esisten breakwater costs for increasing water	14
2.0	depth. [6]	15
		10
3.1	Vessel motion definitions. [11]	22
3.2	An illustration of the relation between cargo handling efficiency and ship motions	
	$[29]. \ldots \ldots$	23
3.3	Tug curves. $[35]$	26
3.4	Effects of a variation in s on the directional spreading of wave energy. The x-	
	axis shows the wave direction in degrees and the y-axis shows the distribution of y -axis shows the distribution of y	าก
25	Diffusction diagram of a comi_infinite breakwater for rendem can waves of normal	32
5.5	incidence [12]	24
36	Transmission coefficients for long waves as found by Hossain (2001) [17]	35
0.0	Transmission coefficients for long waves as found by mossain (2001). [11]	00
4.1	Schematized structure of the design tool	38
4.2	Bathymetry for a $x_{min} = 0m$, $x_{max} = 6,000m$ and $s_{bottom} = 0.01.$	43
4.3	Definition of the directions of several parameters	44
4.4	Some allowances of the approach channel width as prescribed by PIANC. The	
	numbers indicate the allowances as a fraction of the vessel width W_s . [30]	45
4.5	Wave length calculation table, as used in the design tool	47
4.6	Cross-section of the offshore harbour	48
4.7	CAPEX of the jetty as a function of x_{ship} , as calculated by the design tool. For	
	the input values used, reference is made to section 4.6	51
4.8	3D sketch of the approach channel with relevant dimensions.	52
4.9	3D sketch of the turning circle with relevant dimensions	53
4.10	Total CAPEX for dredging works as a function of x_{ship} , determined by the design	
	tool. For the input values used, reference is made to section 4.6.	53
4.11	The significant wave height as a function of the offshore distance under design	
	conditions, as determined by the design tool. $H_{m0,0} = 10 \text{ m}, T_p = 36 \text{ s}, \phi_0 = 210^\circ,$	
1 10	$s_{bottom} = 1/100$	55
4.12	Sketch of the cross-section of the breakwater.	56
4.13	10p view of the offshore harbour.	59 60
4.14	Uross-section of the calsson preakwater	-00

$\begin{array}{c} 4.15 \\ 4.16 \\ 4.17 \end{array}$	Cross-section of the caisson breakwater designed by Carstens. [6] Cross-section of the caisson breakwater designed by Tutuarima et al. [36] Total and infragravity significant wave height as a function of the offshore distance, using: $H_{m0,0} = 1.25$ m, $T_p = 12$ s, a bed slope of 1/100 and an obliqueness	61 61
	at the offshore boundary of 25° . The calculation starts at an offshore distance of	
1 10	$x_{max} = 6,000 \text{ m.}$	64
4.10	values of \mathbf{X}_d as obtained from Goda (2000) and inserted in the design tool, which can be used for the diffraction of short waves	66
4.19	Values of K_d as obtained from Goda (2000) and inserted in the design tool, which	00
	can be used for the diffraction of long waves.	66
4.20	Determination of the total number of ships that have to wait at the anchorage	
	during a certain wave condition.	71
4.21	Total CAPEX and OPEX as a function of the offshore distance for a harbour	
4.00	with a caisson breakwater of $2L_s$	73
4.22 4.23	Cost functions for the offshore harbour without the incorporation of downtime Wave conditions that occur 90% along the West-African coast. The thick black lines indicate the sections based on which the wave conditions that were used as	75
	input for the design tool were chosen.	76
4.24	Total cost functions for the reference case	78
E 1	Example of a time gaming of the surface elevation of altrinod from SWACH Input	
0.1	Example of a time series of the surface elevation as obtained from SWASH. Input parameters: $H_{-0} = 1.25m$, $T_{-1} = 12s$ hed slope $1/100$ water depth $50m$	83
5.2	Example of a variance density spectrum obtained from the MATLAB post-processing	
0.1	Input parameters: $H_{m0} = 1.25m$, $T_p = 12s$, bed slope 1/100, water depth 50m.	84
5.3	Nearshore wave propagation as determined by the design tool and SWASH, for	
	wave condition 1 on a slope of $1/100$	88
5.4	Nearshore wave propagation as determined by the design tool and SWASH, for wave condition 1 on a slope of $1/300$	88
5.5	Nearshore wave propagation as determined by the design tool and SWASH, for wave condition $1 \text{ on a slope of } 1/20$	00
56	Preliminary general lay-out of the offshore phosphates export harbour [34]	09 00
5.0 5.7	Extent of the MIKE21 BW model and wave boundary locations.	91
5.8	Caisson breakwater used for alternative 1. ZH is the Lowest Astronomical Tide	-
	and is 1.49 m below Mean Sea Level. [34]	92
5.9	Rubble mound breakwater used for alternative 2. ZH is the Lowest Astronomical	
	Tide and is 1.49 m below Mean Sea Level. [34]	93
5.10	Dimensions of the breakwater and positions of the berth behind the breakwater.	
F 11	[34]	93
0.11	height red indicates high values. The left image shows the total significant wave	
	height, the right image shows the infragravity wave height [34]	94
5.12	Bathymetric map of the coastal zone	95
5.13	Wave reflection coefficient as a function of steepness of the slope and the incident	
	wave period for plain, smooth and straight slopes.	96
5.14	Lay-out of the offshore OKLNG harbour. [37]	98
5.15	Load/extension characteristics for 44 mm steelite xtra superlines. $[37]$	99
5.16	Bathymetry used for the SURFBEAT model.	102
5.17	Mooring arrangement of a 145,000 m^3 LNG vessel at berth 2	103
6.1	Total cost function for design tool run 1a (hourly waiting costs 1,000 USD)	108
6.2	Total cost function for design tool run 1b (hourly waiting costs 5,000 USD)	109
6.3	Total cost function for design tool run 1c (hourly waiting costs 50,000 USD)	109

6.4	Total cost function for design tool run 2a (maximum H_{m0} 0.8 m)	111
6.5	Total cost function for design tool run 2b (maximum H_{m0} 1.5 m)	111
6.6	Total cost function for design tool run 3a (maximum H_{m0} for tugboats 1.9 m).	113
6.7	Total cost function for design tool run 3b (maximum H_{m0} for tugboats 2.2 m).	114
6.8	Total cost function for design tool run 4a (maximum overtopping volume 0.2	
	$m^3/s/m$).	115
6.9	Total cost function for design tool run 4b (maximum overtopping volume 0.5	
0.0	$m^3/s/m)$.	116
6 10	Total cost function for design tool run 5a ($r_{stern} = 80 \text{ m}$)	117
6 11	Total cost function for design tool run 5b $(x_{step} = 00 \text{ m})$.	117
0.11	$10tar \cos tunction for design toor tun ob (x_{step} = 400 \text{ m}) $	111
A.1	Average H_{m0} , T_n -table for all four West-African locations.	126
A.2	Yearly wave roses for the four West-African locations. Top left: Accra: top left:	
	Abidian: bottom left: Lomé: bottom right: Lagos	127
A 3	Scatter plot of the observed NOAA data for Accra with lines of equal wave steep-	
11.0	nesses	120
Λ /	Direction of the Quines Current during April to June (left) and July to August	120
A.4	(right) [14]	129
۸ F	$(11g_{110}) \cdot (14) \cdot \cdots \cdot $	102
A.5	Batnymetry of west-Africa.	133
A.6	Extrapolation of wave data using three probability distributions for the location	104
	of Abidjan.	134
A.7	Yearly wave roses for four West-African locations. Top left: Accra, top left:	
	Abidjan; bottom left: Lomé; bottom right: Lagos	135
A.8	Yearly tidal elevations for four West-African locations. Top left: Accra, top left:	
	Abidjan; bottom left: Lomé; bottom right: Lagos.	136
A.9	Coastal bathymetries for four West-African locations. Top left: Accra, top left:	
	Abidjan; bottom left: Lomé; bottom right: Lagos	137
-		
B.1	Results SWASH verification average slope	140
B.2	Results SWASH verification mild slope.	141
B.3	Results SWASH verification steep slope	142
B.4	Results downtime prediction verification at -9 m + CD, applying $C_x = 1.7.$	143
B.5	Results downtime prediction verification at -11 m + CD, applying $C_x = 1.7$	143
B.6	Results downtime prediction verification at -9 m + CD for ship 1, applying $C_x =$	
	1.0 to 1.6	143
B.7	Results downtime prediction verification at -9 m + CD for ship 2, applying $C_x =$	
	1.0 to 1.6.	143
B.8	Results downtime prediction verification at $-11 \text{ m} + \text{CD}$ for ship 1, applying	
	$C_{\pi} = 1.0$ to 1.6.	144
B 9	Results downtime prediction verification at -11 m + CD for ship 2 applying	
D.0	$C_{\rm r} = 10$ to 16	144
	$C_x = 1.0$ to 1.0 .	
C.1	Polynomial expansion of the vertical velocity profile in Boussinesq-type models. [9	150
C.2	Strip theory representation by cross-sections. [19]	152
D.1	Screenshot of the 'Front' sheet of the cost module	157
D.2	Intermediate results of the wave calculation module. In the graphs displayed,	
	the short and long wave height behind the two breakwater types are plotted	
	as a function of x_{ship} , for wave condition 1. The different lines show different	
	breakwater lengths.	158
D.3	Part of the input sheet of the cost module. The input parameters are divided into	
	categories in order to maintain the overview.	159

D.4 Calculation sheet in the wave calculation module. In this calculation sheet the transmission coefficient is determined for a range of x_{ship} , for wave condition 1. . 160

List of Tables

$2.1 \\ 2.2$	Extreme water levels over the year 2014 for four West-African locations Average bed slope along four West-African locations	$\begin{array}{c} 11 \\ 12 \end{array}$
3.1 3.2	Minimally required water depth for inner channels. [30]	20 20
3.4	the deep water wave steepness calculated using the mean wave period. [43] \ldots Recommended motion criteria for bulk carriers. [35]	21 24
3.5	Maximum allowable significant wave heights for different dead weight tonnages and wave directions. [35]	25
3.6	Relative importance of various physical mechanisms in different domains: negligible; -: small significance; +: significant; ++: large significance. [46]	28
3.7	Coefficients for approximate estimation of wave heights within the surf zone. [12]	30
4.1	Structure of each module.	40
4.2	Unit costs for dredging works. [2]	41
4.3	Unit costs for conveyor belts. [2]	41
4.4	Unit costs for the quay. $[2]$	41
4.5	Trestle unit costs at several water depths. $[2]$	42
4.6	Unit costs for materials used in breakwaters. $*$ means that in the resource the unit cost was given in tonnes. The price was converted to a unit cost per m ³	
	using a rock density of 2,700 kg/m ³ [36]	58
4.7	Comparison of the dimensions of the three caisson breakwater designs	61
4.8	Input data for the CAPEX and OPEX calculation.	73
4.9	Wave conditions used as input for the reference case	77
5.1	Wave conditions and bed slopes that will be used for the SWASH verification	85
5.2	Selected wave conditions for the wave peentration study. [34]	91
5.3	Results of the MIKE21 BW calculation. The wave heights displayed are the	00
54	Values for the wave height behind the enissen breakwater, obtained from the	92
0.4	values for the wave height behind the calsson breakwater, obtained from the	07
5.5	Values for the wave height behind the rubble mound breakwater, obtained from	91
	the design tool	97
5.6	Vessel particulars. [37]	99
5.7	Limiting H_{m0} for the vessels at berth 2. The wave direction at the offshore boundary is $195^{\circ}N$. [37]	100
5.8	Limiting H_{m0} for the vessels at berth 2 in case a longer trestle is applied (harbour located at -11 m + CD). The wave direction at the offshore boundary is $195^{\circ}N$.	
	[37]	100
5.9	Maximum allowable values of the infragravity wave height at the berth in cm	103

6.1	Baseline values for the sensitivity analysis	107
6.2	Design tool runs for analysis 1	107
6.3	Design tool runs for analysis 2	110
6.4	Design tool runs for analysis 3	112
6.5	Design tool runs for analysis 4	113
6.6	Design tool runs for analysis 5	116
A.1	Results of the regression analysis for the NOAA West-African wave data	128
A.2	Number of storms with a certain storm duration for the four West-African loca-	
	tions considered.	130
A.3	Seasonal variations in the surface current velocity for the Guinea Current. [26] .	132

List of Symbols

Symbol Meaning

A_t	Cross-sectional surface area of the breakwater in $[m^2]$
с	Wave celerity in [m/s]
D_{n50}	Nominal diameter of the concrete armour unit in [m]
D_s	Draught of the design vessel in [m]
f	Frequency of occurrence of the design wave conditions
g	Gravitational acceleration in [m/s]
h	Water depth in [m]
h_{gd}	Minimum required water depth for the design vessel in [m]
$H_{l,m0}$	Low frequency significant wave height in [m]
H_{m0}	Significant wave height in [m]
H_{max}	Maximum wave height in [m]
k	Wave number in $[m^{-1}]$
K_D	Stability number of concrete armour unit
K_{diff}	Diffraction coefficient
k_p	Stiffness of the mooring system parallel to the quay
K_S	Shoaling coefficient
K_t	Transmission coefficient
k_t	Layer coefficient
K_r	Refraction coefficient
L	Wave length in [m]
L_{BW}	Length of the breakwater in [m]
L_{quay}	Length of the quay in [m]
L_s	Length Overall of the design vessel in [m]
M	Mass of the design vessel in [kg]
N_{od}	Damage level of concrete armour units
p	Failure probability of the breakwater
q_{berth}	Percentage of the time that a ship is at the berth
q_{over}	Maximum allowable overtopping volume in $[m^3/s/m]$
q_{OPEX}	Yearly OPEX as a percentage of the total CAPEX
r	Yearly discount rate
s_{bottom}	Bed slope
s_{BW}	Breakwater slope
s_{dredge}	Slope of dredged soil
s_{0m}	Deep water wave steepness determined using the mean wave period

Symbol	Meaning
T	Wave period in [s]
t	Duration of the calculation period for OPEX and waiting costs in [y]
T_m	Mean wave period in [s]
T_p	Peak wave period in [s]
t_{wave}	Duration of a wave condition in [d]
v_{min}	Vessel speed during the tugging journey in [m/s]
W_s	Breadth of the design vessel in [m]
WC	Waiting costs in [USD]
x	Offshore distance in [m], measured from the zero depth contour
x_{dredge}	Offshore distance where the depth is equal to the minimum required depth in [m]
x_{max}	Offshore boundary design tool calculation in [m]
x_{min}	Onshore boundary design tool calculation in [m]
x_{ship}	Offshore distance of the ship, measured from the centre of the ship in [m]
x_{step}	Step size design tool calculation in [m]
x_{surf}	Offshore boundary surf zone in [m]
z	Bottom level relative to MSL in [m]
$\alpha_{channel}$	Deviation approach channel from the shore normal in $[^{\circ}]$
Δ	Relative density
ζ	Wave amplitude in [m]
ϕ	Wave direction in $[^{\circ} N]$
$\phi_{channel}$	Approach channel direction in $[^{\circ} N]$

Acknowledgements

All praise is due to Allah, may He be praised and exalted, the All-Knowing, He who made the quest for knowledge an act of worship.

I would like to express my sincere gratitude to my supervisor professor Tiedo Vellinga for the continuous support and guidance during my MSc thesis. Besides my supervisor, I would also like to thank the rest of my thesis committee for their insightful comments and encouragement: Jules Verlaan, Bas Wijdeven, Benno Beimers and Arne van der Hout. Furthermore, I would like to thank the staff of Royal HaskoningDHV for sharing their experience and enabling me to do my graduation project at their office. Last but not the least, I would like to thank my parents, family and friends for supporting me spiritually throughout the writing of this thesis.

Chapter 1

Introduction

1.1 Background and problem statements

1.1.1 Background

Marine terminals were traditionally constructed in locations that are sheltered or protected from large marine loadings such as high waves. These locations can be naturally sheltered or artificially sheltered by means of a breakwater. Nowadays these terminals are more often constructed in relatively exposed locations. This is due to an increased demand for the development of large single use industrial terminals, especially for the export of dry and liquid bulk. These terminals are usually constructed in remote locations for safety considerations. Another consideration that can lead to the construction of such a terminal is that a certain location is economically attractive due to its proximity to a powerplant or railway connection. The berths for such offshore harbours are located in relatively deep water, with approach trestles guiding to the moored ship [24].

The absence of a shelter or existing infrastructure in the nearby surroundings means that hydraulic forces on the moored vessel may be large, which can lead to excessive (horizontal) motions of the vessel. It would not be cost effective to construct a new protective breakwater for the whole facility. Therefore usually a detached breakwater is constructed at some distance offshore in order to provide some shelter for the moored vessel [24]. A typical example of such a design is displayed in figure 1.1.

Nevertheless, vessel motions can be large if the frequency of the waves near the berth approaches the natural frequency of the moored vessel. This is usually the case for bound or free long waves with a wave period in the range of 25 to 300 s. Damping at these frequencies is small, which may lead to excessive vessel motions that can cause large mooring forces. This creates unsafe situations at the berth and may even lead to a temporary cease of the (un)loading process, which results in downtime [42]. Downtime results in a reduction of the throughput of a harbour, and leads to an increase in vessel waiting costs. In order to reduce downtime due to large vessel motions and mooring forces, two measures can be taken: use a mooring system that dampens the vessel motions (e.g. ShoreTension \mathbb{R}) or change the lay-out of the port (e.g. extend the breakwater length) [42]. In this report the emphasis will be on the second option: the lay-out of the port.

1.1.2 Problem description

The offshore harbour design described above consists of a number of elements (breakwater, jetty, dredging). Each element can be considered a cost head, of which the capital and operational expenditures depend on the offshore distance of the berth location. When the berth is located close to the shore, the costs for the construction of the jetty will be relatively low as the length of the access trestle does not have to be that large. Furthermore, the required dimensions of



Figure 1.1: Example of an offshore harbour consisting of a detached breakwater and an exposed jetty. [11]

the breakwater are also lower near the coast, which means that the costs for the breakwater may also be relatively low. However, costs for dredging will be large as the depth near the coast is not sufficient to safely manoeuvre and berth a vessel. When the berth is located further offshore, dredging costs will be low as the depth offshore is usually sufficient for the vessel to safely moor. However, the access trestle will have to be longer and due to more severe (short) wave conditions and larger water depths a larger breakwater will be required, which will lead to high costs for these structures.

Capital and operational costs are not the only factors that have to be taken into account when designing an offshore harbour. The harbour must also provide a workable situation in which safe berthing, deberthing, loading and unloading of vessels can take place. This implies that the downtime over a certain period must not be too large in order to reach a certain minimally required throughput. One can imagine that a solution with relatively low (capital) costs is to construct a very long jetty without a breakwater to a depth where no dredging is necessary. However, the (short) wave climate further offshore is generally more rough (depending on the actual location considered), and the absence of a sheltering structure may result in difficulties for the loading and unloading process. Another option is to construct a well protected harbour with large breakwaters close to the shore. This is a very expensive solution, but on the other hand, downtime will be very rare as the wave conditions are very mild. It can be said that besides construction and maintenance costs, downtime is also an important factor in determining the feasibility of a certain harbour design. Amongst the factors that influence the downtime is the prevailing wave climate, which depends mainly on the geographic location. Further away from the coast the short waves will be larger which can result in downtime. However, short waves are not the only type of waves that can cause downtime. Long waves (also called "low frequency wave motions" or "infragravity waves"), in this study defined as waves with a wave period of larger than 25 s, may also significantly affect the ship. Closer to the coast the influence of (reflected) long waves will be larger, which can cause excessive vessel motions and large line tensions due to resonance, resulting in downtime. Besides the distance offshore, the length of the breakwater also influences the downtime: the larger the breakwater length, the less (short) wave energy will spread behind the breakwater, creating more workable situations.

Currently there is no design tool that is able to predict or recommend an optimum offshore harbour lay-out. Optimum in the sense that the profits of the harbour over the period during which it will be used (the lifetime of the harbour) are maximal. In order to reach optimum harbour profits the difference between the revenues and the costs should be at a positive maximum. The revenues are related directly to the throughput of the harbour, which is influenced by the downtime. A minimum downtime results in a longer period during which harbour operations can be performed, and thus larger revenues. A smaller downtime also results in lower waiting costs for the vessels approaching the harbour. The costs of a harbour can be divided into capital expenditures (CAPEX) and operational expenditures (OPEX). As mentioned earlier this section, both the downtime and the costs are influenced by the lay-out of the harbour, especially by the offshore distance of the berth and the breakwater length.

1.2 Objectives and research questions

1.2.1 Objectives

The primary objective of this study is to develop a design tool that is able to recommend an optimum offshore harbour lay-out (i.e. offshore distance of the berth and breakwater length). As discussed in the previous section, an optimum harbour design is the design that has a large uptime (relatively low downtime) and limited expenditures during the lifetime of the harbour.

1.2.2 Research questions

Based on the primary objective the following research questions can be formulated:

- 1. What requirements does the design tool has to fulfill with regard to functionality and usability?
- 2. Which parameters and processes are relevant for the design of an offshore harbour and how can these be incorporated in the design tool?
- 3. How do the offshore distance of the berth and the breakwater length influence the capital and operational expenditures of an offshore harbour?
- 4. How do the offshore distance of the berth and the breakwater length influence the downtime of a vessel moored at an offshore harbour?
- 5. Is the calculation of the nearshore wave propagation and wave penetration into the port as determined by the design tool comparable to the results obtained using wave modelling software?
- 6. Does the design tool give a correct prediction on whether downtime occurs under a certain wave condition?
- 7. How sensitive is the design tool when it comes to uncertainties in the model input and formulations?

1.3 Scope

The offshore harbour design investigated for this study consists of three main parts: an exposed jetty, a detached breakwater and dredging works. In order to reduce the complexity of the design tool that has to be developed, a number of simplyfing assumptions and decisions are made:

• A constant bed slope and an alongshore uniform coast with a straight coastline are assumed. The soil is completely homogeneous and consists of one soil type with constant soil properties.

- Only downtime due to wave action (short and long waves) is taken into account. It is assumed that wind, currents and other factors are of minor importance and are not able to cause downtime. This means that the design tool can only be used at locations where wind speeds and current velocities are low. An example of such a region is the West-African coast.
- The offshore harbour considered is a dry bulk harbour. This choice has been made because dry bulk vessels do not have very strict limitations for vessel motions, which means that an offshore harbour might be more feasible for dry bulk vessels compared to other cargo types (e.g. container vessels [1]).
- The harbour consists of only one berth.

It is obvious that these assumptions reduce the applicability of the design tool, as it will not be possible to apply the design tool for instance at locations with complex bottom geometries. However, the design tool is mainly aimed to be used in an early design stage, such as (pre-)feasibility studies. In such stages there is usually no need for very detailed output and design choices are usually made based on indicative figures and simple calculations.

1.4 Methodology

The development of the design tool will take place along three phases that will be discussed in this section. The design tool itself roughly consists of two parts: the first part calculates the costs based on the offshore distance of the berth and the breakwater length, whereas the second part calculates the downtime of the harbour based on wave action inside and nearby the harbour.

1.4.1 Phase 1: Literature study

In the first phase an assessment is made of all parameters and processes that have to be taken into account for the design of an offshore harbour. This phase consists of the following steps:

- An inventarisation of previous studies performed on the topic of offshore harbours and a formulation of all requirements and demands the design tool has to fulfill. This will answer the first research question. This can be found in chapter 2 of this report.
- A literature review on the design practice and operationality (uptime and downtime) of offshore harbours. This can be found in chapter 3 of this report.
- A literature review on the relevant wave processes that may influence the design of offshore harbours. This can also be found in chapter 3 of this report.

Based on the latter two assessments the second research question can be answered.

1.4.2 Phase 2: Development of the design tool

In the second phase of this research the actual design tool has to be developed, based on the information gathered in the first phase. The development of the design tool can be found in chapter 4 of this report. The following steps will be taken for this purpose:

- The design tool has to have a cost estimation function that is able to express the capital and operational expenditures as a function of the offshore distance of the berth and the breakwater length. This will lead to the answer to the third research question.
- The design tool has to have a downtime estimation function that is able to give an estimate for the downtime based on the wave conditions at the berth and translate the downtime to a monetary unit. This will answer the fourth research question.

1.4.3 Phase 3: Verification and evaluation

The final phase of this research is verification and evaluation of the design tool. This is done based on the following steps:

- The first verification of the design tool is done by comparing the nearshore wave propagation and wave penetration into the port for a number of wave conditions as determined by the design tool with the results of a calculation made using a wave modelling program. This answers the fifth research question.
- The second verification is performed by comparing the downtime prediction for a certain set of wave conditions and a certain harbour lay-out as given by the design tool with the result of a Dynamic Mooring Analysis performed for the same harbour and wave conditions. This leads to the answer to the sixth research question.

Both verifications can be found in chapter 5.

• An evaluation of the design tool is made by performing a sensitivity analysis. This analysis is done in chapter 6 and will answer the seventh and final research question.

Based on the verifications and evaluation, conclusions can be drawn on the performance of the model and recommendations can be given for further improvements and adaptations. An overview of these conclusions and recommendations is given in chapter 7. The methodology for this research is summarized in the figure below.



Chapter 2

General overview of the design tool

In this chapter a general overview of the design tool is given. First some background information about offshore harbours is presented. This background information is necessary in order to understand why the design tool is being developed. The second section describes the lay-out of the offshore harbour that will be investigated during this study. The design tool will be made for this type of harbour lay-out. The third section discusses the main objectives and requirements of the design tool. The design tool is elaborated in more detail in chapter 4.

2.1 Background information

Various studies have been performed to assess the feasibility and possibilities of the offshore harbour concept for various types of cargo. In this section a summary of some of these studies is given.

2.1.1 Motivation for offshore harbours

A study performed by Burdall and Williamson (1991) encouraged the idea of an offshore harbour. During their study, Burdall and Williamson tried to address the question "Is it possible to have an efficient and environmentally friendly port?" A 'green port' was defined by them as a port that satisfies both the environmental acceptability and commercial port requirements. They stated that both capital and maintenance dredging form an integral part of many port operations giving consequential environmental problems associated with the disposal of the dredged spoil. They considered this to be the single most important environmental issue for many ports. The study predicts that in the future larger, more fuel efficient vessels will require deeper water but will be less susceptible to wave action, which will lead to the development of offshore harbours [5]. The offshore harbour concept was elaborated further by Bruun (1992). He stated that there is a growing demand for berths in deep water due to increased vessel sizes. This leads to the construction of larger breakwaters in 20 to 30 m depth. The total height of the breakwater may therefore be as much as 40 m, leading to a total breakwater cost of 50 to 150 million USD. Bruun mentioned that in these cases it might be difficult to justify the cost of such breakwaters and it might be necessary to find less expensive alternatives. An alternative may be to avoid the use of the breakwater altogether and construct an unprotected berth, and thus an unprotected pier. The consequence is that vessels are more exposed to wave and current loads, which are usually larger in deeper water. Bruun mentioned that in this case the moored vessel loses its protection, i.e. the hindrance to or decreasing of movements of the vessel, which may lead to an exceedance of functional limits for vessel movements. A solution for this problem may be to consider only relative movements, and eventually design an unloading/loading system where the relative movement of the vessel and the handling or lifting equipment is equal to zero. A more acceptable idea is to tie the vessel down to "zero" or "elastic movements" in fenders and moorings only [4].

Besides the environmental considerations mentioned by Burdall (1991), Bruun also mentioned safety considerations as a reason to consider offshore harbour development. LNG- and LPG-vessels require terminals in remote areas for reasons of safety. Dry bulk terminals can result in several types of nuisance inside an existing port. Furthermore, the increasing size of the vessels influences depth requirements. [4].

2.1.2 Possibilities and challenges

De Jong et al. (2014) built further on the conclusions made by Burdall and Bruun and performed a study on possibilies and challenges for so-called 'open water ports', i.e. harbours located in deep water with a relatively large degree of exposure. De Jong et al. proposed a design for an open nearshore container terminal, consisting of a detached reclamation behind which vessels are moored. A number of advantages were mentioned for this design [10]:

- Seiching in basins may be reduced due to the less confined geometry.
- Fewer complications are foreseen in terms of future port extension compared to the traditional port located inland. More directions of freedom for port expansion are available offshore, resulting in less conflicting functions.
- Blocking of longshore sediment transport is avoided, which means accretion and erosion issues are minor compared to a traditional port design. Salient formation might occur however due to the influence the port infrastructure will have on local wave conditions.
- Less or possibly no dredging activities of the port basins, which will not only be advantageous during the construction phase but also for the following operational phase of the port basins.

Hadijah (2013) performed a study on the technical and financial feasibility of open container ports. For a large number of locations worldwide with a water depth between 10 and 20 m, the technical feasibility was investigated based on the maximum surge motions for vessels at berth and limitations for tugboats. Both factors play an important role for the downtime of a ship. The result of the technical feasibility analysis is displayed in figure 2.1. For these locations an offshore container port is feasible using MoorMasterTM units [15]. It must be mentioned however that container vessels have the most strict limitations for vessel movements. Bulk vessels usually have less strigent criteria which means that offshore harbours for bulk cargo might be feasible at more locations, and maybe with more traditional mooring systems.

Bakermans (2014) also investigated possibilities for offshore and exposed container ports. He performed case studies for two regions that were recommended by Hadijah: Singapore and West-Africa. There were two main reasons to consider the construction of an offshore harbour along the West-African coast [1]:

- The region has a very dynamic morphology. An offshore harbour would not disrupt morphological processes which means erosion of the coastal zone is avoided. Furthermore, costs for the dredging of the approach channel and the port basin are avoided.
- By constructing the harbour further off the coast, the negative impact of low frequency wave motions is avoided.

An open container port along the West-African coast seemed both technically and financially feasible, which means it might be interesting to further investigate this concept for other cargo types in this region.



Figure 2.1: World map with locations where an open container port could be feasible for a Super-Post Panamax vessel, when mooring with MoorMasterTM units [15].

2.2 Harbour lay-out

In this section the harbour lay-out that will be used as a starting point for the design tool is described. First the boundary conditions, requirements and demands that have to be taken into account for the harbour design are presented. Based on these conditions and demands, the lay-out for the offshore harbour considered by the design tool can be determined.

2.2.1 Boundary conditions

First the boundary conditions for the offshore harbour design are described. In the previous section it was mentioned that Hadijah (2013) found that West-Africa is an interesting location for offshore container harbour development. Hadijah (2013) found that an open container port in West-Africa might be feasible if MoorMasterTM is used. Container ships have the most strict criteria for vessel motions, which means that it might be interesting to investigate offshore harbours for other cargo types and more traditional mooring systems. Dry bulk vessels usually have less stringent criteria, which might mean that an offshore dry bulk harbour might be feasible using more traditional mooring arrangements. Furthermore, Hadijah did not consider a harbour with a detached breakwater, which provides even more protection for moored vessels. The dynamic coastal morphology was mentioned by Bakermans (2014) as an important reason to consider offshore harbour development in West-Africa. Therefore the harbour that will be studied for this research is a harbour for dry bulk cargo in West-Africa.

For this report, the West-African coast is defined as the South-Atlantic coast starting from Liberia westwards to Nigeria eastwards. This area comprises the Gulf of Guinea and its near surroundings of the South-Atlantic ocean. This area is located in the region of the Equator, between a latitude of 7° North and 3° South and between a longitude of 11° East and 5° West. This region will be referred to during the remaining of this report as 'the West-African coast', 'the region of interest' or simply 'West-Africa'. A map of this region is displayed in figure 2.2 below.

Model simplifications

The coastal zone where the offshore harbour will be constructed is schematized by means of a number of simplifying assumptions. An alongshore uniform coast is assumed with a straight coastline and parallel depth contours. This implies that a constant bed slope s_{bottom} is assumed throughout the coastal region. This means that the water depth z is a linear function of the



Figure 2.2: The geographical area for which the environmental conditions will be assessed. [13]

offshore distance x, given by: $z = s_{bottom} \cdot x$ in which s_{bottom} is the bottom slope. The soil of the coastal zone is assumed to be completely homogeneous, with uniform soil properties throughout the region. These model assumptions may seem a bit oversimplifying and it may be practically impossible to find a location on the globe where such conditions may apply. This may impose serious restrictions on the applicability of the design tool. Complex bottom geometries or regions with large alongshore differences in the coastal profile might not be suitable to be analysed using the design tool. However, these assumptions are necessary in order to reduce the complexity of the model formulations and enable a quick, easy assessment of offshore harbour feasibility. Furthermore, the design tool is meant to be used in an early design stage, such as (pre-)feasibility studies. Design choices in such early stages are usually made based on inidicative figures and rules of thumb, and there is no need yet to obtain very detailed output. Moreover, the main purpose of the design tool is to recommend a value for the offshore position of the ship x_{ship} and the length of the breakwater L_{BW} that might be interesting to investigate for a certain coastal zone. The goal is not to produce a very detailed offshore harbour design, but rather to compare various alternatives with different values for x_{ship} and L_{BW} , based on CAPEX, OPEX and downtime. A simplified initial harbour design might be sufficient to make this comparison.

Wave conditions

As was already mentioned in the introduction, only downtime caused by waves will be considered by the design tool. Therefore a good understanding of the offshore wave climate is necessary. Appendix A.1 presents wave data for West-Africa in the form of a H_{m0}, T_p -table and wave roses. It can be seen that under operational wave conditions, the significant wave height is generally of moderate height, with wave heights between 1.0 and 1.25 m having the largest frequency of occurrence. It can also be seen that a H_{m0} of 2.5 m is exceeded less than 1% of the time. Furthermore, swell waves with a T_p between 8 and 16 s make up nearly 95% of the wave climate. Moreover, waves coming from the South-West make up the dominant wave direction for all four locations.

For the design of a protective structure such as a breakwater wave data that apply to design conditions have to be used. These conditions are much more severe and have a much smaller probability of occurrence. Appendix A.1 also shows the determination of the design wave conditions. It was found that for breakwater design a H_{m0} of 10 m and a T_p of 36 s can be used.

Other boundary conditions

In appendix A an analysis of the wind and current conditions in West-Africa are presented. It can be seen that the wave and current velocity are generally low and are not expected to cause any problems for moored vessels. This is in accordance with the assumption that factors other than wave action are not likely to cause downtime. Furthermore, tide data for a period of one year (1 January 2014 to 1 January 2015) was gathered. These are also presented in appendix A. The maximum and minimum tidal elevations are displayed in table 2.1. Tidal elevations are relevant for the determination of the minimally required depth and the freeboard of the breakwater. It can be seen that the high and low water are generally less than 1 m above and below MSL.

Location	Maximum water	Minimum water	Difference (m)
	level $(m + MSL)$	level $(m + MSL)$	
Abidjan	0.7530	-0.7839	1.5369
Accra	0.9062	-0.9456	1.8518
Lomé	0.9215	-0.9673	1.8888
Lagos	0.9350	-0.9839	1.9189

Table 2.1: Extreme water levels over the year 2014 for four West-African locations.

Finally, the bed slope at the four West-African locations was also assessed. Coastal profiles for these locations are displayed in appendix A. The average bed slope is less uniform along the West-African coast, as can be seen in table 2.2. The coastal orientation also shows large variations. The average coastal orientation along the West-African coast is West-East, and the coastal normal makes an average clockwise angle of 186.25° with the North. However, the deviation of this average reaches nearly 70% for certain areas along the Nigerian coast. Both the bed slope and the coastal orientation influence the way in which the waves propagate towards the coast. Processes such as shoaling, refraction and wave breaking are largely influenced by the bed slope and the deviation of the wave direction from the coastal normal, and as such the wave height at a certain location is also influenced by these parameters. The local wave height at a certain offshore distance on its turn will influence the design and the feasibility of the offshore harbour at that location.



Figure 2.3: Determination of the average orientation of the West-African coast. [13]

2.2.2 Demands and requirements

Based on the boundary conditions a number of demands and requirements that the offshore harbour has to fulfill can be formulated. These demands are presented in the following.

1. The harbour should provide a berth where a dry bulk vessel can be loaded with cargo. In

Location	Bed slope (%)
Abidjan (Ivory Coast)	2.25
Accra (Ghana)	0.34
Lomé (Togo)	0.22
Lagos (Nigeria)	0.26

Table 2.2: Average bed slope along four West-African locations.

order to be able to safely load the vessel, acceptable limits for vessel motions, mooring line and fender forces should not be exceeded.

- 2. The harbour should be located outside the surf zone to avoid dredging in the morphologically active area, in order to not disturb coastal morphological processes and to avoid the need for frequent maintenance dredging. The trestle that is constructed to connect the berth with the coastline should not form an obstruction for alongshore sediment transport.
- 3. The local water depth at all locations in the harbour should be sufficient for a safe manoeuvring and berthing of the design vessel.
- 4. Sufficient space must be present for the design vessel to change its direction inside the harbour (a turning circle).
- 5. An approach channel should be present along which the vessel can reach the turning circle. The approach channel should be a straight channel in line with the dominant wave direction. Bends in the approach channel should be avoided [21]. The width of this approach channel should be sufficient to safely manoeuvre the vessel, taking into account cross-winds and currents.
- 6. Under operational conditions, the ship has to be able to enter and leave the harbour by means of tugboat assistance. Due to the limited length of the breakwater, the tugboat has to be a able to sail into unsheltered waters. The wave conditions during the tugging journey should be such that the tugging efficiency is preserved.
- 7. In case a breakwater is constructed, it should be dimensioned such that it does not fail under design conditions. Under operational conditions, wave transmission into the harbour should be limited to an acceptable value.
- 8. The position and orientation of the vessel behind the breakwater should be such that the wave action on the moored vessel is at a minimum.

2.2.3 Chosen lay-out

Based on the boundary conditions and requirements formulated in this section, a standard layout for the offshore harbour can be designed. Figure 2.4 and 2.5 show the lay-outs that are considered by the design tool. In the remainder of this section, the design choices made are motivated.

Exposed jetty

An exposed jetty perpendicular to the coastline is used to connect the quay to the coast. The open piled jetty type is chosen because it is largely pervious for waves, currents and sediment, causing a minimum blockage of alongshore sediment transport resulting in a minimum impact on coastal morphological processes [24] (requirement 2). Loading of the ship will take place by means of a conveyor belt, which transports bulk goods from the facilities at the coast towards the ship. The length of the jetty depends on the offshore distance of the centre of the ship,



Figure 2.4: Harbour lay-out with a mooring arrangement perpendicular to the coast.

denoted as x_{ship} . The vessel is moored to the quay by means of mooring lines.

Detached breakwater

In addition to the jetty, a breakwater can also be present in the harbour design in order to provide shelter for the moored vessel. The breakwater is constructed detached from the coast, parallel to the coastline. In section 2.2.1 it was found that the waves that reach the West-African coast predominantly come from the South, whereas the coastline generally has a West-East orientation. From this it can be concluded that waves will generally approach the coast perpendicularly. However, large variation can be found in the orientation of the coast, which means that the waves might approach the coast in a different direction at certain locations. On the other hand, waves do turn more to the shore normal as they approach the coast due to refraction effects. Therefore it is expected that a breakwater parallel to the coast is the most effective in protecting the ship from wave action.

A rubble mound breakwater with concrete armour elements and a caisson breakwater are considered. The choice to consider two breakwater types mainly comes from the fact that each breakwater type is financially attractive at a certain range of water depths. Rubble mound breakwaters are more attractive at small water depths close to the coast because the material costs for quarry stone (which makes up the core of the breakwater) are generally low. The construction of a prefab concrete caisson is a financially unattractive alternative at low water depths. Due to the sloping shape of the rubble mound breakwater, the amount of stones and prefab elements increases rapidly with an increasing water depth. Furthermore, due to the required exact placement of concrete armour elements, the construction at large depths becomes very difficult. It is expected that at some value for the water depth the required stone volume becomes so large, that a prefab caisson becomes attractive again. This comparison was made by Carstens (2001) and can be seen in figure 2.6. For the case considered by Carstens, a break-even point can be found at a depth of 18 m [6]. Tutuarima et al. (1998) also compared the costs of several breakwater types, and found that starting from a water depth of 8 m, caisson breakwa-



Figure 2.5: Harbour lay-out with a mooring arrangement parallel to the coast.

ters are more cost-effective [36]. It can thus be expected that for the depths at which offshore harbour development is interesting, both breakwater types might provide a feasible solution depending on the water depth considered.

Both breakwater types have their own characteristics and influence on the wave field, which will be explained in more detail in the next chapter. The armour for the rubble mound breakwater consists of concrete elements. Armour rock is not feasible for offshore harbours, because the average stone diameter would become some large that construction of the breakwater would be practically impossible. Furthermore, concrete elements can be placed on a steeper slope, which means that less stone volumes are required. The breakwater is placed on the original bed, which means that no dredging is required for the breakwater. Furthermore, the required volumes for the breakwater are also smaller in this case.

Several breakwater lengths (L_{BW}) can be applied. During this study, five breakwater lengths are examined: 0, 1, 2, 3 and 4 times the length of the design vessel, L_s . The length of the breakwater has a large influence on the wave conditions in the harbour basin and therefore it has been chosen to investigate the effect this parameter as well. Generally the significant wave height behind the breakwater decreases with increasing breakwater length because the effect of diffraction becomes less for an increasing ratio between the breakwater length L_{BW} and the wave length of the incoming wave L [3]. Furthermore, the option without a breakwater is also investigated. The ship will always be located behind the centreline of the breakwater, because the wave height of the diffracted waves will be the smallest behind the centre of the breakwater (in the case of normal wave incidence).

The choice for the mooring arrangement of the vessel is difficult. A mooring arrangement perpendicular or parallel to the coast both have their advantages and disadvantages. If a parallel arrangement is chosen, the vessel can be placed closer to the breakwater, which means that the vessel is more sheltered. Furthermore, the vessel is then placed in line with the main current direction. This last advantage might not be that important because the current velocities are generally low and not governing for the mooring behaviour of the vessel. However, if no break-



Figure 2.6: Comparison of rubble mound and caisson breakwater costs for increasing water depth. [6]

water is used, a parallel mooring arrangement will lead to beam waves, which are generally more disadvantageous for the mooring behaviour of the ship than head-on or stern-on waves. A perpendicular arrangement has the main disadvantage that the ship is generally further off the breakwater, which means that it is less sheltered. On the other hand the ship is in line with the dominant wave and wind direction if no breakwater is used. Both arrangements have their justifications and thus the design tool has to be able to consider both a parallel and a perpendicular arrangement.

Dredging works

In case the natural water depth is not sufficient, dredging works will be necessary. In this research, x_{dredge} is defined as the offshore distance beyond which the natural depth is larger than the minimally required depth for the ship to safely manoeuvre (h_{qd}) . One value for h_{qd} is applied in the entire dredged area. The amount of soil that has to be dredged depends amongst others on the offshore position of the ship. A turning circle has to be dredged in which the vessel that enters or leaves the harbour can change its direction (requirement 4). Because only one ship has to be able to moor at the harbour (requirement 1), the berth is located inside the turning circle. This means that no special berthing pockets or additional harbour basins will have to be dredged. If the turning circle is located completely more onshore than x_{dredge} a dredged approach channel will be necessary to connect the turning circle with the deeper areas. The approach channel is completely straight with no bends (requirement 5). In the next chapter several rules of thumb and guidelines for the design of these dredged areas. It must be noted that both for the turning circle and the approach channel, side slopes must be present. Furthermore, the position of the breakwater in relation to the dredged areas also deserves special attention. The breakwater must not be located too close to the slopes of the dredged areas, as this may cause slope instability, resulting in structural failure.

2.3 Goals and demands

Bruun (1992) mentioned an important dilemma that a port engineer has to face when designing an offshore harbour: does he or she choose for more protection for the moored vessel, which means that a (often) large and expensive breakwater has to be constructed, or does one choose to leave the breakwater away, resulting in a large exposure of the vessel to wave action and maybe even in an exceedance of motion limits [4]. The design tool is meant to provide a solution for this dilemma. In this section the goals of the design tool are explained in more detail. Furthermore, several functionality and usability requirements the design tool has to fulfill in order to reach the mentioned goals are presented.

2.3.1 Primary and secondary goals

The design tool that will be developed is meant to optimize the lay-out of the harbour described above. The primary goal of the design tool can thus be formulated as follows:

Recommend an offshore distance of the ship x_{ship} , a breakwater type (rubble mound or caisson) and a breakwater length L_{BW} for which the total costs of the offshore dry bulk harbour in West-Africa over a certain calculation period are minimal.

This primary goal of the design tool can be divided into a number of secondary goals, which are listed below:

- 1. Design (elements of) the offshore harbour for a number of offshore distances x_{ship} and breakwater lengths L_{BW} .
- 2. Determine the costs (CAPEX and OPEX) of the offshore harbour for a number of offshore distances and breakwater lengths, over a certain calculation period.
- 3. Determine the short and long wave height at the berth for a number of offshore distances and breakwater lengths, for several wave conditions.
- 4. Determine the total waiting costs due to downtime caused by (short and/or long) wave action over a certain calculation period, for various x_{ship} and L_{BW} .
- 5. Sum up the CAPEX, OPEX and waiting costs and determine at which x_{ship} and L_{BW} the total costs are minimal, and what breakwater type is associated with these minimum costs.

2.3.2 Demands

In order to give clear and understandable results that can be used for harbour development projects, a number of demands must be fulfilled by the design tool. These demands can be divided into functionality demands that are related to the degree in which the primary and secondary goals are achieved, and usability demands that are related to the user-friendliness of the design tool.

Functionality demands

1. Based on the dimensions of the design vessel and the environmental conditions, the design tool must determine the required dredging depth and the horizontal dimensions of the turning circle and the approach channel for each x_{ship} . Based on these dimensions the total dredging CAPEX and OPEX have to be determined.
- 2. Based on the position and dimensions of the vessel, the dimensions of the jetty have to be determined. Using these dimensions, the CAPEX and OPEX for the jetty have to be calculated.
- 3. The rubble mound and the caisson breakwater have to be dimensioned based on the wave conditions under design conditions. Based on the breakwater dimensions, the CAPEX and OPEX for both breakwaters have to be determined for several L_{BW} .
- 4. The offshore wave climate under operational conditions has to be translated to a wave climate inside and nearby the harbour.
- 5. Based on the wave conditions inside and nearby the harbour, it has to be determined for each x_{ship} , L_{BW} and breakwater type whether the vessels are able to safely enter, moor and leave the harbour or not. In the latter case, downtime will occur, either due to excessive vessel motions (at the berth) or due to tugboat unavailability.
- 6. The downtime has to be expressed in a monetary unit (the waiting costs). The total costs of an offshore harbour for each x_{ship} , L_{BW} and breakwater type are determined by summing the CAPEX, OPEX and waiting costs up.
- 7. The optimum harbour lay-out $(x_{ship}, L_{BW}$ and breakwater type) have to be determined based on the total cost function.

Usability demands

- 1. In order to prevent confusion, one input screen should be available in which the user is able to insert all input values related to a certain offshore harbour project. The user must be able to easily change these values. It must also be clear in which units the parameters have to be inserted. Directions should also be clearly mentioned (e.g. 'coming from', 'going to', 'clockwise relative to North').
- 2. The calculation procedure should be briefly explained somewhere in the design tool, including mathematical expressions used to obtain certain output.
- 3. The most important output (cost functions for all harbour lay-outs optimum values for x_{ship} , L_{BW} and breakwater type) also has to be present on one screen. It must also become clear from a glance where input and output values are stored.
- 4. Intermediate results should be presented on separate sheets, in both tabulated and graphic form. Results of one group of calculations (e.g. CAPEX calculation, wave calculation) should be grouped so the user can easily maintain the overview. The intermediate results should be easily accessable.

Based on these demands a design tool will be developed for the harbour design described in section 2.2. The structure of the design tool is explained in chapter 4.

2.4 Conclusions

The offshore harbour concept was initially introduced as a solution to avoid extensive dredging, which is becoming more and more a problem due to increasing vessel sizes. However, this solution also poses a new problem: extensive breakwaters are required to provide shelter for the vessels in deeper water. Some researchers questioned whether a breakwater is really necessary and whether the harbour cannot be constructed with less shelter. The design tool is meant to help in answering this question, and considers a harbour consisting of three main elements: an exposed jetty, a detached breakwater and dredging works. The harbour design should fulfill criteria with regard to navigational, operational and structural safety. The total costs of the harbour (CAPEX and OPEX) and the total waiting costs caused by downtime due to short and long waves are calculated by the design tool as a function of the offshore distance of the ship x_{ship} and the length of the breakwater L_{BW} . Based on this total cost function an optimum solution can be recommended.

Chapter 3

Relevant parameters and processes

A lot of different factors may influence the design of an offshore harbour. The relevance of these factors for the design tool varies from one factor to another. Some factors are expected to have a large-scale impact on the optimum lay-out for a certain harbour project, whereas others only influence the design in very detailed aspects and can therefore be neglected. In this chapter an overview is presented of all processes and parameters that are considered relevant for the design tool. A distinction is made between factors that influence the dimensioning of the harbour and factors that influence the operationality (uptime and downtime) of a harbour. Special attention is given to wave processes as the focus of this study is on downtime caused by wave action. By means of guidelines, theories and empirical formulas the influence of several parameters and processes may be expressed in terms of mathematics. Using these expressions, the relevant parameters and processes may be incorporated in the design tool.

3.1 Parameters influencing the design of offshore harbours

In the first section of this chapter, the parameters that influence the dimensioning of the offshore harbour are presented. These parameters have to be identified because the dimensions of the various elements in the offshore harbour will influence the CAPEX and OPEX of the harbour. Furthermore, the breakwater influences the wave conditions at the berth, which has an effect on the downtime.

The main focus will be on the design of the dredged areas and the rubble mound breakwater. This is because simple guidelines and empirical formulas exist that can be used for a preliminary design of these elements. These simple formulas can be easily incorporated in the design tool. The design of a jetty and caisson breakwater is not so straightforward, because the number of variables in the design process is enormous, and difficult processes have to be taken into account. An extensive analysis of pressures and forces is required in order to find the required dimensions [24][12]. Performing this analysis for a large range of offshore distances is an extremely difficult task that may not be wanted for a relatively simple design tool that in essence is not meant to be used for the purpose of structural design. An exact design of the jetty and caisson breakwater is therefore outside the scope of this study. Instead the dimensions are estimated using simple rules of thumb.

3.1.1 Design of dredged areas

For a conceptual design in an early stage, PIANC guidelines can be used to determine the dimensions of the dredged areas. These guidelines are relatively easy in use and account for environmental parameters that may influence the manoeuvring behaviour of the vessel, which makes them suitable for use in the design tool.

First a look is taken at the required depth h_{gd} . It is assumed that the required depth is the same for all dredged areas. In practice the keel clearance in areas where the vessel does not have to manoeuvre (e.g. harbour basins where the vessel is at the berth) is taken to be a bit less compared to areas where the ship has to move. For the sake of simplicity however the depth is assumed to be constant throughout the harbour. PIANC recommends the following guidelines to determine the minimally required water depth for inner and outer approach channels [30]:

Vessel speed	Required depth	Channel bottom type	Add
$\leq 10 \text{ kts}$	$1.10 D_s$	Mud	0
10 - 15 kts	$1.12 D_s$	Sand/clay	0.4 m
$\geq 15 \text{ kts}$	$1.15 D_s$	Rock/coral	$0.5 \mathrm{m}$

Table 3.1: Minimally required water depth for inner channels. [30]

Wave conditions	Required depth	Channel bottom type	Add
Low swell $(H_{m0} < 1m)$	1.15 to 1.2 D_s	Mud	0
Moderate swell $(1m < H_{m0} < 2m)$	1.2 to 1.3 D_s	Sand/clay	0.5 m
Heavy swell $(H_{m0} > 2m)$	1.3 to 1.4 D_s	Rock/coral	1.0 m

Table 3.2: Minimally required water depth for outer channels. [30]

The width of the approach channel depends on the amount of traffic (one-lane of twolane), the basic width W_{BM} (which on its turn depends on the vessel width and the available depth), bank clearances W_{BR} and W_{BG} (if applied) and additional widths ΣW_i , taking into account prevailing cross-winds and –currents, wave height, navigational aids and hazardous cargo [21]. PIANC recommends the following formula to determine the required width of a one-way approach channel [30]:

$$W_{channel} = W_{BM} + \Sigma W_i + W_{BR} + W_{BG} \tag{3.1}$$

Using these guidelines, the harbour can be designed such that requirement 3 and 5 (section 2.2.2) are fulfilled. Besides the approach channel, a turning circle for the vessel is also necessary (requirement 4). The diameter of the turning circle is usually taken to be twice the length of the design vessel. In cases with strong wind, currents or no tugs available, a turning circle with a diameter of three times the vessel length is taken [30]. For the design tool a value of $3L_s$ is chosen. This is because the turning circle is located in relatively exposed conditions, which leads to increased wave action compared to the case when the turning circle is sheltered. Furthermore, the berth is also located inside the turning circle, which restricts the available space in which the vessel can turn.

3.1.2 Breakwater design

Several design formulas exist that can be used to determine the dimensions of a rubble mound breakwater. Dimensions of the breakwater is important for two reasons. First of all, the dimensions can be used to estimate the CAPEX and OPEX of the breakwater. Secondly, the dimensions of the breakwater influence the degree to which wave penetration into the harbour occurs.

It was mentioned in the previous chapter that a rubble mound breakwater with concrete armour elements will be considered. No quarry stone armour elements are used because the required diameter to resist the design wave conditions at the offshore distances considered is so large (order 5 m) that the construction would be practically impossible. This is because rock armour elements derive their stability mainly from their weight, whereas for concrete elements interlocking between the elements leads to a larger stability against wave action. Furthermore, the use of concrete armour elements is advantageous because they can be placed on a steeper slope, which means that the total volume of the breakwater becomes smaller. In the following the design of several breakwater dimensions is treated. By applying these dimensions, requirement 7 (section 2.2.2) can be fulfilled. The design formulas presented will be incorporated in the design tool.

Armour size D_{n50}

For concrete blocks on a sloping surface, the stability is usually expressed in terms of the parameter $K_D = \frac{H_{m0}}{\Delta D_{n50}}$. The acceptable value of the stability parameter depends on the amount of damage one permits. This is expressed in the damage level N_{od} . The larger the value for N_{od} , the more damage is allowed. For the design tool it is assumed that N_{od} is always equal to 0. This corresponds to the start of damage [43].

A distinction can be made between single-layer elements that can be places on steep slopes $(1V:1\frac{1}{3}H)$ and double-layer elements that are placed in double layers on milder slopes (1V:1.5H). The expression for the stability number for several concrete elements can be found in table 3.3 [43]. These concrete elements have been chosen because their stability formulas are relatively simple expressions that are suitable for a quick preliminary design. These formulas will also be incorporated in the design tool.

Block type	Block name	K_D	Roughness coefficient γ_f	Layer coefficient k_t
Single laver	Accropode	3.7	0.46	1.29
	Xbloc	3.5	0.45	1.40
	Core-Loc	3.7	0.44	1.52
Double layer	Cube	$s_{om}^{-0.1} - 0.5$	0.47	1.10
	Tetrapod	$0.85s_{om}^{-0.2} - 0.5$	0.38	1.02

Table 3.3: Characteristics of several concrete armour elements. The variable s_{om} stands for the deep water wave steepness calculated using the mean wave period. [43]

Crest height and width

The required crest height of the breakwater depends on the amount of transmission and overtopping is allowed into the harbour basin. The breakwater crest should be high enough to prevent large overtopping over the breakwater under design conditions and large wave transmission into the harbour basin under operational conditions. Furthermore, overtopping also influences the stability of the inner breakwater slope: the overtopping volume should be limited to prevent failure of the inner slope. Both overtopping and transmission are taken into account in the design tool. For rubble mound breakwaters, the required crest height is determined based on an overtopping criterion: the breakwater crest must be at such a level that under design conditions, a given overtopping volume is not exceeded. This can be determined using the following expression [43]:

$$\frac{q_{over}}{\sqrt{gH_{m0}^3}} = 0.2 \exp\left(-2.3 \frac{R_C}{H_{m0} \gamma_f \gamma_\beta}\right)$$
(3.2)

In this equation γ_f stands for the roughness coefficient, which can be found in table 3.3 for several concrete armour elements. γ_{β} is a reduction factor for wave obliqueness, given by: $\gamma_{\beta} = 1 - 0.0022\beta$ (β being the incident wave obliqueness) and R_C stands for the required freeboard of the crest above MSL [43]. Overtopping plays a much smaller role for caisson breakwaters, due to the large horizontal surface area and the absence of an inner slope. Goda (2000) states that a freeboard of 0.6 times the design wave height usually is enough to achieve an acceptable amount of overtopping and transmission [12]. The total required height of the breakwater is equal to the sum of the local water depth, the maximum tidal elevation (table 2.1) and the required freeboard for overtopping.

The crest width is less straight forward. For loose armour units, usually a crest width of

approximately three armour units is applied. The layer coefficient k_t has to be applied in order to take into account unaccuracies in concrete unit placement. This means that the crest width can be approximated using: $W_{crest} = 3k_t D_{n50}$. However, other structures (such as crown walls) may also be applied to provide extra sheltering for the moored vessel [43].

3.2 Parameters influencing the downtime

In this section the factors that influence the operationality of an offshore harbour are assessed. It is not only important that the harbour constructions are structurally safe, but also that the lay-out provides a workable situation in which the vessel can enter the harbour, load or unload and leave the harbour without complications. An important concept in this matter is downtime, which is the period during which the harbour is unavailable. Thoresen (2010) recommends a yearly overall downtime percentage of less than 5 to 10% due to the extra waiting costs that are related to the downtime [35]. Downtime can be divided into two types. Operational downtime means that loading and unloading operations at the berth are impossible. An important cause for operational downtime is excessive vessel motions. Operational downtime is mainly related to requirement 1 (section 2.2.2). The behaviour of moored vessels is assessed in the first part of this section. Navigational downtime means that the vessel is not able to call at a harbour or berth safely from the open sea or ocean. Tugboat availability (requirement 6, section 2.2.2) is an important criterion for navigational availability and is therefore treated in the second part of this section.

3.2.1 Behaviour at the berth

General

When considering the motions of a free floating vessel, it is important to realise that it essentially behaves as a mass-spring-dashpot system with six degrees of motional freedom. Three of them are translational and the other three are rotational. The different motions a vessel can make are displayed in figure 3.1. External forces set the vessel's body (the mass) in motion which results in time varying forces in the mooring system. The response of the vessel and the mooring system depends on the dynamics of the external forces and the response characteristics of the vessel [41].



Figure 3.1: Vessel motion definitions. [11]

Wave action

Various forces may excite vessel motions. Wave forces have a very dynamic nature and may result

in unacceptable vessel motions and significant mooring forces. The following wave conditions can result in excessive vessel motions [11]:

- Wave periods of larger than approximately 4 s and wave heights larger than approximately 1.2 m.
- Long waves at resonant periods (slowly varying drift forces), even of low height.
- The beam of the vessel is oriented towards the sea.

One of the factors on which the vessel response depends is the wave period [11]. In order to excite a dynamic response a critical wave period needs to be exceeded. Waves with a period between 5 and 300 s generally cause the vessel to respond dynamically. High frequency load variations (period less than about 5 s) generally do not cause a vessel to respond dynamically. Furthermore, very slowly varying forces such as tide excite a static response of the vessel [41]. Furthermore, the ratio between the wave length and the vessel's breadth or length also influences the response of the vessel. For a vessel moored in a head- or stern-to-sea condition, the vessel nearly moves in concert with the wave profile for heave and pitch motions [11]. Due to the high mass of the ship and soft springs (mooring lines), the natural period for horizontal ship motions is usually long. Long waves with a period between 25 and 300 s can cause large problems to moored vessels, because the natural period of the dynamic response of a moored ship usually is close to the period of long waves. Furthermore, damping at these frequencies is small for horizontal motions. This means that even small amplitude long waves can cause large amplitudes in ship motions due to resonance. The large ship motions can cause a number of problems, such as a slowing down or interruption of the (un)loading process. Furthermore, large ship motions yield large line tensions which may cause the ship to break loose, leading to damage to the port infrastructure or other moored vessels [42].

Hence, it becomes clear that the wave period is an important parameter when it comes to vessel response. The effect of infragravity waves with periods close to the eigenperiod of the moored vessel is much more disastrous than the effect of waves with shorter periods (up to a T_p of 25 s). Therefore a clear distinction must be made in the design tool between short and long waves.

Guidelines and limitations

In figure 3.2, cargo handling efficiency is plotted against the ship motion amplitude [29].





Ship motions with an amplitude between A and B do not affect cargo handling operations which means the efficiency is at 100%. Between B and C the efficiency decreases due to larger vessel motions. Handling operations are impossible when level C is exceeded. It is no longer safe to handle the vessel. Between C and D the vessel can still stay at the berth, but once the ship motions exceed level D, the ship has to leave the berth because the safe mooring limits are exceeded. Of course one has to take into account that this is just a schematization, and that other parameters, such as cargo handling equipment, operational conditions, light conditions and skill of crane operators also affect cargo handling efficiency [29].

It is difficult to formulate a set of motion criteria above which motions are deemed unacceptable. This is because the amount of movement considered acceptable depends on a wide and complex set of factors. Thoresen (2010) recommended motion criteria for safe working conditions for the handling of different types of ships. Table 3.4 presents recommended motion criteria for bulk carriers [35]. It must be noted that not all vessel movements are equally important. Motions in the horizontal plane (surge, sway and yaw) are the most dangerous as they could break the ship loose from the berth. From a safety point of view these movements should therefore be minimised [35].

Cargo handling	Surge	Sway	Heave	Yaw	Pitch	Roll
$\mathbf{equipment}$	(m)	(m)	(m)	(°)	(°)	(°)
Cranes	2.0	1.0	1.0	2	2	6
Elevator / bucket-	1.0	0.5	1.0	2	2	2
wheel						
Conveyor belt	5.0	2.5	-	3	-	-

Table 3.4: Recommended motion criteria for bulk carriers. [35]

Surge motions are dominated by low frequency behaviour [45]. Mol et al. (1986) developed a semi-empirical formula to estimate the significant surge motion x_{surge} of a ship based on the significant wave height of the low frequency waves $H_{l,m0}$ [25]. In order to come to this formula the moored vessel is schematized as a linear damped mass-spring system with one degree of freedom. The dynamic motion of the ship is approximated by the static motion multiplied by a coefficient that represents the dynamic influence. The semi-empirical formula reads as follows:

$$x_{surge} = C_x \cdot H_{l,m0} \cdot \sqrt{\frac{g \cdot M}{h \cdot k_p}}$$
(3.3)

In this formula M is the mass of the vessel and k_p is the stiffness of the mooring system parallel to the quay. C_x is a dimensionless coefficient with values ranging between 1 and 3, with an average of 1.7. Furthermore, g is the gravitational acceleration and h is the local water depth. The formula is based on model tests executed for ships with a dead weight tonnage ranging from 5,000 to 70,000. Added mass, damping and the angle of the wave are not included in this formula, which makes it a large simplification. If the low frequency wave height at the berth is known, a prediction for whether downtime occurs or not can be made by calculating the surge motion using Mol's formula. By comparing the calculated surge motion with the maximum allowable surge motion recommended by Thoresen (2010), one may estimate whether or not downtime occurs. This is a simple yet inaccurate method that can be incorporated in the design tool.

The other degrees of motion also have to be considered. Sway and heave depend on both highand low-frequency waves, but are much more difficult to estimate [45]. Due to the short wave dependency of sway and heave motions, one might use criteria for maximum allowable significant wave heights at the berth to obtain insight in whether or not the sway and heave motions are acceptable. Table 3.5 gives values for maximum significant wave heights for a dry bulk carrier at different wave directions and loading conditions, as recommended by Thoresen (2010) [35]. As an initial estimate, one might assume that the maximum allowable sway and heave motions are exceeded when the significant wave height at the berth exceeds the maximum allowable H_{m0} as recommended by Thoresen (2010). In this way, downtime due to excessive sway and heave motions can also be incorporated in the design tool.

Due to the large amount of physical factors that affect ship movements at the berth it is quite difficult to make a simple estimate on which wave heights are considered acceptable at the berth. A larger wave height is more acceptable for head-on or stern-on waves than for beam waves. It must be noted that the wave period also has a very large influence on whether a wave with a certain wave height can cause problems or not. Locally wind-generated waves will generally have little effect on a moored vessel. Long waves cause problems at much lower wave heights. The limitations shown in the table below apply for residual deep-water waves with periods in the range of 7 to 12 s [35].

Dead Weight Tonnage	Limiting wave height H_{m0} in meters		
	0° (head-on or stern-on)	$45^{\circ} - 90^{\circ}$	
< 30,000 DWT	0.80		
30,000 - 100,000 DWT (loading)	1.50	1.00	
30,000 - 100,000 DWT (unloading)	1.00	0.80 - 1.00	

Table 3.5: Maximum allowable significant wave heights for different dead weight tonnages and wave directions. [35]

3.2.2 Tugboat availability

Besides downtime due to excessive vessel motions at the berth, downtime can also occur due to tugboat unavailability. It is necessary that causes for navigational downtime are also taken into account. If a harbour provides safe loading conditions at the berth, but the vessels are not able to berth and unberth in the first place, the harbour is practically unfeasible. Whether a vessel can enter the harbour or not depends on tugboat availability.

General

When a ship approaches a harbour, it starts to decelerate. When the speed of its engine is less than about 3 to 4 knots, the ship will start to lose steerage. From this point tugboats will be necessary to provide the necessary assistance during the berthing and unberthing operations to counteract wave forces (and other forces, if present). Tugboats guarantee a simpler and easier berthing and unberthing procedure and represent an important safety factor if engine or steering troubles are encountered by the ship. From a safety point of view, tugboat assistance should always be used during berthing and unberthing operations [35].

The efficiency of the tugboats depends on a number of factors. Wave conditions have a large influence on the tugboat efficiency. An increasing wave height causes a decrease in tugboat efficiency due to three reasons:

- A larger wave height makes the tug unable to deliver a constant level and angle of thrust due to tug motions and possible line snatching.
- The wave-induced ship and tug motions are generally different, causing the tug to move relative to the ship.
- A proportion of the tug's power is required to keep the tug under control or to keep up with the lateral speed of the vessel.

In figure 3.3, the tugboat efficiency is plotted as a function of the wave height, for several tug types. In general, it is accepted that tugboats with conventional static winches can operate in waves of up to approximately 1.5 to 2 m. Beyond this limit, the effectiveness of the tugs becomes more uncertain, and the use of dynamic winches which act to reduce snatching may have to be

considered [35].



Figure 3.3: Tug curves. [35]

Berthing and deberthing process

The berthing process already starts several kilometers off the harbour when the vessel decelerates until it reaches the minimum speed required for manoeuvring. When the minimum speed is reached, the tugboats are tied up. The distance during the process of entering the harbour can be expressed using the following formula [21]:

$$L = L_1 + L_2 + L_3 \tag{3.4}$$

In which:

- L_1 is the distance travelled during deceleration of the vessel. This is usually approximated by $L_1 = (v_s - 2)\frac{3}{4}L_s$, v_s being the sailing speed of the vessel and L_s the length of the vessel.
- L_2 is the distance travelled during the time required to tie up the tugboats. In average this time is about 10 min and the sailing speed is approximately 2 m/s, which means that the total length of this part is 1.2 km.
- L_3 is the final stopping distance, usually approximated with 1.5 times the vessel length.

Based on these distances, it can be determined at what offshore distances the tugboats need to sail for a certain harbour lay-out. Tugboats are required for L_2 and L_3 . For the offshore harbour design considered during this study, the distance the tugboats have to sail will be largely (if not completely) located in unsheltered waters. This choice has been made because constructing a

protective breakwater for the tugging journey will lead to a very expensive solution. To protect the distance L_2 only, a breakwater of at least 1,200 m length is required, constructed at depths that may be as large as 20 m. This leads to incredibly large required material volumes which may be hard to justify. Instead, the breakwater that will be constructed for the harbour only serves to protect the berth. This means that tugboat availability depends on the wave height in unsheltered waters.

Based on a minimum required tugboat efficiency, a maximum allowable value for the wave height along the travelling distance of the tugboat can be defined. For example: a minimum tugboat efficiency of 30% for static winches and direct pull corresponds with a maximum allowable H_{m0} of 2 m (for $T_p > 14$ s, figure 3.3). If the wave height along the tugging distance exceeds this value, tugboats are deemed unavailable. By checking whether the wave height along the tugging journey exceeds this maximum H_{m0} or not, it can be determined whether downtime due to tugboat unavailability occurs or not. In this way, the effect of navigational downtime can be taken into account in the design tool.

3.3 Relevant wave processes

From the previous two sections it became clear that the wave conditions at a certain location have a huge influence on the design and operationality of offshore harbours. Therefore a good representation of wave propagation, transformation and penetration into the harbour is an important issue. In this section wave processes relevant for the design tool are described. But first, a method to incorporate the wave processes in the design tool has to be found.

3.3.1 Incorporation of wave processes

Wave processes

Which wave processes are of importance depends largely on which physical domain the waves are located in. Four physical domains can be distinguished:

- Deep oceans: where bottom influences can be neglected.
- Shelf seas: the area between deep oceans and the shoaling zone.
- Shoaling zone: in this area shoaling becomes important.
- Harbours: areas in which interaction between the waves and any structure (islands, reefs, breakwaters, oil platforms, etc.) has to be taken into account.

In the table below an overview is given of the relative importance of various physical processes regarding the development and propagation of waves in the above mentioned physical domains [46].

From table 3.6 it can be concluded that shoaling, depth refraction and diffraction are the most important processes for the design of harbours. These and more processes are treated in this section, but first a method to describe these processes in the design tool has to be found.

Choice for an incorporation method

Several methods to incorporate the wave processes in the design tool exist. Wave modelling software can be used to solve complex differential equations related to the behaviour of waves in coastal waters. Appendix C presents several types of wave models that can be used to accurately describe the relevant wave processes. The large accuracy of course comes with a cost: usually a large computational effort is required before results are obtained, which might make these model types not suitable for a quick assessment tool.

On the other hand, more simple methods to describe the relevant wave processes also exist.

Physical process	Deep oceans	Shelf seas	Shoaling zone	Harbours
Diffraction			-	++
Depth refraction / shoaling		+	++	+
Current refraction		-	+	
Quadruplet interactions	++	++	-	
Triad interactions		-	+	-
Atmospheric input	++	++	-	
White-capping	++	++	-	
Depth breaking		-	++	
Bottom friction		++	+	

Table 3.6: Relative importance of various physical mechanisms in different domains. - -: negligible; -: small significance; +: significant; ++: large significance. [46]

Random ocean waves are often described using linear wave theory. This theory consists of two fundamental equations: the mass and the momentum balance. Furthermore, some simple boundary conditions are used to describe certain kinematic and dynamic aspects of the waves. Linearization of the equations and boundary conditions involved results in a solution consisting of freely propagating, harmonic waves, also called Airy waves. The linearity implies that the waves do not affect one another while they travel together across the water surface [16].

A number of simplifying assumptions (e.g. no dissipation, alongshore uniform, parallel depth contours) make it possible to describe nearshore wave processes such as shoaling and refraction with relatively simple mathematical expressions. The simplified representation of the coastal zone as described in section 2.2 might make these expressions applicable for the design tool. Other processes however cannot be described by linear wave theory only. Wave breaking for example takes place in an area in which an important assumption of linear wave theory (wave amplitude negligible compared to water depth) is no longer valid. The occurrence of infragravity waves is the result of nonlinear wave processes that are also out of the scope of linear wave theory. Moreover, linear wave theory is also unsufficient to model the interaction between the propagating waves and the breakwater via transmission and diffraction. The inability of linear wave theory to describe these processes makes the theory on its own unsuitable to be used to model wave processes. However, empirical formulas and rules of thumb (as will be presented in the remainder of this section) exist that can be used to describe these processes. An approach in which linear theory is combined with empirical formulas to estimate the effect of wave breaking, infragravity waves, transmission and diffraction can also be adapted. In this way the processes that are out of the scope of linear wave theory can also be incorporated.

Different levels of accuracy also influence the choice of a method to incorporate wave processes. A numerical wave model has a relatively large accuracy when it comes to modelling the waves. The output of these models however must be analysed and post-processed with methods of a similar degree of accuracy. It would not be justifiable for example to accurately model the waves using a computationally expensive wave model, and post-process it using rules of thumb or simple guidelines, because the end result will still be inaccurate. For example: consider the case in which the wave propagation is calculated to determine the vessel response at the berth. If a wave modelling program is used, the results will probably have to be inserted in a panel or strip method program to calculate the wave forces, followed by a mooring analysis program to determine the vessel motions. This is a computationally expensive method that requires a lot of time. The main aim of the design tool was to avoid the use of extensive modelling software and to come up with a quick, simple and user-friendly method that can be used for a rapid assessment. The use of linear wave theory in combination with rules of thumb and guidelines to estimate nonlinear phenomena is a quick and simple method of which the subsequent chains in the calculation process all have a similar degree of inaccuracy.

In conclusion, the choice has been made to use linear wave theory in combination with empirical formulas and guidelines for those calculations for which linear theory does not provide a solution. The motivation for this choice is:

- It is a quick and simple method that lends itself for a quick assessment.
- The simplified coastal zone considered by the design tool fulfills the assumptions for which the expressions of linear wave theory are applicable.
- The several chains in the calculation process have a similar degree of inaccuracy.

In the remainder of this chapter, the expressions used to incorporate the relevant wave processes in the design tool are described.

3.3.2 Nearshore wave propagation

In the following several processes related to the undistured nearshore propagation of waves are described. For all cases an alongshore uniform coast with parallel depth contours is assumed. The mathematical expressions are incorporated in the design tool.

Dispersion

The dispersion relation is given by [3]:

$$\omega = \sqrt{gk \tanh kh} \tag{3.5}$$

The phase velocity (or wave celerity) c of the wave is defined as the rate at which any phase of the wave propagates in space. Based on the dispersion relation the phase velocity can be expressed as follows [3]:

$$c = \frac{\omega}{k} = \frac{gT}{2\pi} \tanh kh = c_0 \tanh kh \tag{3.6}$$

For deep water or short waves $(h/L > 0.5 \text{ so } kh \gg 1) \tanh kh$ approaches 1, which means that the phase velocity will be equal to the deep water phase velocity $c_0 = \sqrt{gL_0/2\pi} \approx 1.56T$. It can be seen that in deep water the phase velocity depends on the wave period. The waves are therefore referred to as dispersive waves. Because longer waves in deep water travel faster than short waves, the different harmonic components of a wind wave field are seperated from each other (frequency dispersion)[3].

For shallow water or long waves (h/L < 1/20) tanh kh approaches kh which means that the dispersion relation reduces to $c = \sqrt{gh}$. The phase velocity is only dependent on the water depth and is independent of the wave period. The waves are called non-dispersive [3]. Dispersion is not so much a wave transformation process in itself, but forms a theoretical basis to understand the other nearshore wave processes according to linear wave theory.

Shoaling

Shoaling refers to the change in wave height that occurs due to depth differences along a wave ray. In shallower water the wave decelerates, leading to a compaction of wave energy, resulting in an increase in wave height. The shoaling factor K_s is defined as the ratio between the wave height at a location nearshore and the deep water wave height. Using the energy flux balance, the following expression for the shoaling factor can be found:

$$K_s = \frac{H}{H_0} = \sqrt{\frac{n_0 c_0}{nc}} \tag{3.7}$$

The subscript 0 stands for the values at deep water. The parameter n stands for the ratio between the group velocity and the individual phase velocity of the waves, and is defined as:

$$n = 0.5(1 + \frac{2kh}{\sinh(2kh)}) \tag{3.8}$$

This parameter approaches 1 for shallow water and 0.5 for deep water.

Refraction

Refraction is related to depth changes along the wave *crest*, i.e. perpendicular to the direction of wave propagation. Depth changes along the crest cause a change in wave direction. The parts of the wave ray in deeper waves travel faster, which means that the wave wil turn towards the shore normal. The wave angle at a certain location can be found using Snell's law:

$$\frac{\sin\phi_0}{c_0} = \frac{\sin\phi}{c} \tag{3.9}$$

Based on this energy flux balance a refraction coefficient K_r can be defined, that expresses the effect of refraction on the wave height nearshore.

$$K_r = \sqrt{\frac{b_0}{b}} = \sqrt{\frac{\cos\phi_0}{\cos\phi}} \tag{3.10}$$

It is expected that the effect of refraction will be relatively small because the incident waves usually already have a small obliqueness.

Wave breaking

Wave breaking can be induced due to a too large steepness and due to a too small depth. Miche (1944) expressed the limiting wave steepness based on Stokes wave theory [3]:

$$\left[\frac{H}{L}\right]_{max} = 0.142 \tanh kh \tag{3.11}$$

In deep water this reduces to: $[H/L]_{max} = 0.142$. This causes steepness-induced breaking, also referred to as white-capping. For shallow water it is found based on the Miche breaking criterion that wave breaking starts if the breaker index γ , which is the ratio between the incoming wave height and the water depth, becomes larger than 0.88. This is called depth-induced breaking [3].

In the surf zone, which is defined as the region in front of the coast where depth-induced breaking occurs, the wave amplitudes are no longer negligible in comparison to the water depth. The turbulence that occurs due to wave breaking also means that the water particles are no longer irrotational. Therefore linear wave theory can no longer be used to give a reliable estimation of the wave height.

Empirical formulas to estimate the wave height in the surf zone provide a good alternative. Goda (2000) developed the following expressions to estimate the significant wave height [12]:

$$H_{1/3} = \begin{cases} K_s K_r H_0 & \text{for } h/L_0 \ge 0.2\\ \min(\beta_0 K_r H_0 + \beta_1 h), \beta_{max} K_r H_0, K_s K_r H_0) & \text{for } h/L_0 < 0.2 \end{cases}$$
(3.12)

The coefficients used in these expressions are defined as follows:

β_0	$0.028(K_rH_0/L_0)^{-0.38}\exp\left(20s_{bottom}^{1.5}\right)$
β_1	$0.52 \exp\left(4.2 s_{bottom} ight)$
β_{max}	$max\{0.92; 0.32(K_rH_0/L_0)^{-0.29} \times \exp(2.4s_{bottom})\}$

Table 3.7: Coefficients for approximate estimation of wave heights within the surf zone. [12]

Wave breaking is usually limited to smaller water depths close to the coast (the surf zone). Since one of the requirements of the offshore harbour design is that harbour development in the surf zone has to be avoided, it can be expected that the effect of wave breaking on the result of the design tool will be small. Nevertheless, it has to be taken into account because the wave height under design conditions (return period 1/5000 years) is much larger than the wave heights under operational conditions, which means that these waves may already break at large depths outside the surf zone. Hence, it can be stated that for the design of the breakwater wave breaking may play a role, and therefore this process is taken into account.

3.3.3 Infragravity waves

General

In this section the phenomenon of low frequency wave motions is described. In this study low frequency wave motions (infragravity waves or simply 'long waves') are defined as waves with a wave period larger than approximately 25 s. The incorporation of infragravity waves is very important, because it was found in section 3.2.1 that infragravity waves cause large problems for moored vessels, because a dynamic response via resonance is excited. There are three sources for low frequency wave energy [42]:

- Bound long waves (wave set-down). Bound long waves are associated with swell wave groups travelling towards the shore. These waves travel with the same group velocity as the short waves: their troughs coincide with a group of high amplitude short waves and their crests with low amplitude short waves [44]. Occurrence of these bound long waves is related to the occurrence of short waves. Physical characteristics (wave height, period, etc.) are related to short wave properties such as directional spreading, spectrum shape, wave height etc. Therefore, the bound long wave climate can be estimated from the short wave climate, if the relation with the short waves is known [44].
- Free long waves. These waves occur when the bound long wave is set free after the swell waves have broken in shallow waters or on the shore (due to limited depth) [42]. Release of long waves may also occur at diffraction points such as breakwater heads and bathymetric changes [41]. The long wave is released from the group, reflects off the coast and returns to the sea. The reflection of the long wave leads to a partial standing wave pattern known as surfbeat [7]. Due to the long wave period free long waves can reflect even at flat sandy beaches. If the angle to the shore-normal is small, the reflected free long wave returns back into the sea as a leaky wave. However, if the angle is larger than a certain critical value, the long wave is trapped in the coastal area forming a trapped edge-wave [41]. Free long waves can also occur due to storms or other physical phenomena at remote locations. Frequency of occurrence and related physical properties can be considered to be stochastic quantities [44].
- Seiching or harbour oscillations. This is a standing wave related to the natural period of the harbour basin [42]. As the design considered during this study consists of an exposed berth (i.e. without a basin), the phenomenon of seiching will not be further investigated during this research [11].

The occurrence of bound long waves is heavily influenced by the directional spreading of the short wave field. The more unidirectional the wave field is, the more interaction (via resonance) will occur between the primary waves, which will result in a higher bound wave energy. The wave directional spreading coefficient is usually expressed as a standard deviation in the directional spreading (in radians or degrees) or as an exponent s in a \cos^{2s} function, that expresses the spreading of wave energy over the various directions. The effect of s on the directional spreading of wave energy is depicted in the figure below. The larger the exponent s, the less directional spreading there will be and the higher the bound long wave energy.

Estimation of the infragravity wave height

Since linear wave theory does not take into account second order wave effects, a different approach to determine the low frequency wave height has to be found. Different empirical formulas



Figure 3.4: Effects of a variation in s on the directional spreading of wave energy. The x-axis shows the wave direction in degrees and the y-axis shows the distribution of energy. The integral under the curves is unity [31].

exist to estimate $H_{l,m0}$.

Vis (1985) developed an empirical formula to estimate the bound long wave height $H_{bl,m0}$. The formula reads as follows [44]:

$$H_{bl,m0} = \begin{cases} \frac{0.22}{T_p^2} H_{m0}^2, & \text{for deep water}(\frac{h}{gT_p^2} > 0.09).\\ \frac{0.08}{h^2} T_p^2 H_{m0}^2, & \text{for shallow water}(\frac{h}{gT_p^2} < 0.04). \end{cases}$$
(3.13)

This formula has two main drawbacks. The first drawback is that it only estimates the bound long wave. The free long wave that is reflected off the coast is not taken into account in this formula. Especially near the coast the influence of the reflected long waves becomes very large and thus has to be taken into account. Secondly, the formula is only valid in deep and shallow water. Vis (1985) does not present a method that can be used in intermediate water depths, which form an important part of the coastal zone investigated. Therefore this formula is deemed unsuitable for the design tool.

Kamphuis (2001) proposed the following formula to estimate the significant infragravity wave height [20]:

$$\frac{H_{l,m0}}{H_{b,m0}} = 0.11 \left(\frac{H_{b,m0}}{gT_p^2}\right)^{-0.24} \tag{3.14}$$

In this formula $H_{b,m0}$ stands for the breaking significant wave height, which can be estimated using Miche's formula. This formula was originally developed to determine the infragravity wave height in front of a reflecting structure rather than to determine the development of the long wave height at a certain location offshore. Moreover, Kamphuis' formula was derived from experimental results in which the reflection coefficient for long waves was equal to 22 %. This coefficient may vary for different infragravity wave periods and bottom steepnesses. Therefore Kamphuis' formula is not considered suitable for the purpose of generic wave calculations.

Goda (2000) derived an empirical formula to estimate the order of magnitude of the surf-beat amplitude ζ in shallow waters and in the surf zone [12]:

$$\frac{\zeta_{rms}}{K_r H_{m0,0}} = 0.01 \cdot \left(\frac{K_r H_{m0,0}}{L_0} \cdot \left(1 + \frac{h}{K_r H_{m0,0}}\right)\right)^{-1/2}$$
(3.15)

In which ζ_{rms} is the root-mean-square amplitude of the surfbeat profile. The root-mean-square wave height $H_{l,rms}$. can be found by doubling the value of ζ_{rms} . The significant low frequency wave height can be found by multiplying this $H_{l,rms}$ with a factor of $\sqrt{2}$ [3]. This results in:

$$H_{l,m0} = 0.02\sqrt{2}K_r H_{m0,0} \left(\frac{K_r H_{m0,0}}{L_0} \left(1 + \frac{h}{K_r H_{m0}}\right)\right)^{-1/2}$$
(3.16)

Goda's formula was originally developed to estimate the surfbeat amplitude, which is equal to the sum of the amplitude of the incoming and reflected infragravity wave. This means that the effect of infragravity wave reflection is taken into account by this formula. Furthermore, Goda's formula is not limited to a certain reflection coefficient. Therefore Goda's formula is considered to be a suitable method to incorporate infragravity waves in the design tool.

3.3.4 Wave penetration

The processes described above all concern undisturbed wave propagation. In the remainder of this section a look is taken at processes that are related to the interaction between the incident waves and the breakwater. The breakwater does not completely eliminate all wave action: diffraction and transmission of waves ensure that wave energy still reaches the moored vessel. Linear wave theory is not applicable to study wave penetration into harbours. Other methods and empirical formulas however exist to describe the relevant processes.

Diffraction

An obstruction to wave propagation (e.g. an offshore island, a breakwater, etc.) or abrupt changes in bottom contours may lead to a large (initial) variation of wave energy along a wave crest. This will lead to transfer of energy along the wave crest. This phenomenon is referred to as diffraction. Diffraction effects behind a breakwater depend strongly on the ratio between the length of the breakwater L_{BW} and the wavelength L. If $L/L_{BW} \ll 1$, diffraction will occur around each breakwater head, but wave energy will not spread in the entire zone behind the breakwater. However, if $L/L_{BW} \gg 1$, wave energy will spread everywhere behind the breakwater [3]. Based on this principle, diffraction diagrams were developed in order to estimate the wave height behind a structure. An example of such a diagram from random sea waves approaching a semi-infinite breakwater is displayed in figure 3.5. The solid lines can be used to estimate the diffraction coefficient K_{diff} , which is the ratio between the diffracted and the incoming wave height.

Transmission

Transmission of wave energy through a rubble mound breakwater can occur *over* and *through* the breakwater. Firstly, overtopping water can lead to wave action in the lee of the structure. Secondly, a permeable core in combination with a long wave period may cause wave transmission [43]. Long waves travel more easily through the breakwater [45].

A large number of transmission formulas were developed, that can be used to estimate the wave height behind the breakwater based on a transmission coefficient K_t , which is defined as the ratio between the transmitted and incident wave height. For cases with small waves (low values of H_{m0}/D_{n50} and relatively large positive freeboards ($R_c/H_{m0} > 1$) the following empirical formula was derived by Ahrens (1987) [7]:

$$K_t = \frac{1.0}{1.0 + X^{0.592}} \tag{3.17}$$

With X defined as follows:

$$X = \frac{H_{m0}A_t}{L_p(D_{n50})^2} \tag{3.18}$$

With A_t the cross-sectional area of the breakwater in m^2 . It is expected that the freeboard of the breakwater applied for the offshore harbour will be large compared to the wave height under normal circumstances, because for West-Africa the wave height under design conditions



Figure 3.5: Diffraction diagram of a semi=infinite breakwater for random sea waves of normal incidence. [12]

(10 m) is far above the wave height under operational conditions (usually not larger than 2 m). This means that this formula can be applied to the offshore harbour considered and should be incorporated in the design tool.

Transmission of infragravity waves was investigated by Hossain (2001) [17]. He compared theoretical results on the transmission of long waves through a composite breakwater with a rubble base with results from laboratory experiments. He found that a very large part of the long wave energy was transmitted through the breakwater, and that both in the theoretical and the experimental results a transmission coefficient of approximately 0.8 was found. The results of his comparison can be seen in figure 3.6 [17].

3.4 Conclusions

The relevant parameters and processes are divided into three categories. The first category consists of parameters that influence the design of the offshore harbour. The dimensions of the dredged areas mainly depend on the dimensions of the design vessel and the environmental conditions at the location of interest. The dimensions of the rubble mound breakwater are determined by criteria for maximum overtopping volumes and concrete armour element stability. PIANC guidelines and empirical formulas were proposed as a method to incorporate these parameters in the design tool.

The second category consists of parameters that influence the downtime. Two types of downtime are considered: downtime at the berth due to excessive vessel motions and downtime due to tugboat unavailability. The first type of downtime depends largely on the wave conditions at the berth. Especially low frequency waves are important as these may cause excessive vessel motions due to resonance. Thoresen (2010) presents limits for the wave height at the berth and vessel motions. These limits may be used in the design tool to obtain rapid insight in the occurrence of downtime. Downtime due to tugboat unavailability may be incorporated in the design tool by assessing the wave height along the tugging journey. If a certain limit for the



Figure 3.6: Transmission coefficients for long waves as found by Hossain (2001). [17]

wave height, based on the minimally required tugging efficiency, is exceeded, the navigational uptime cannot be ensured anymore.

From the previous two categories of parameters it has become clear that waves play a huge role in the design and behaviour of offshore harbours. As waves undergo a lot of transformations during nearshore propagation, an accurate representation of the processes related to these processes is required in the design tool. Mathematical expressions based on linear wave theory can be used to assess the effect of shoaling and refraction. Goda's empirical formulas and diagrams can be used to take into account wave breaking and diffraction and to estimate the low frequency wave height. Moreover, Ahrend and Hossain developed methods to estimate the amount of transmission for short and long waves. By combining linear wave theory with the mentioned empirical formulas, a quick and rapid assessment method is obtained that can be used to incorporate the relevant wave processes occuring in the simplified coastal area.

Chapter 4

Structure of the design tool

In the chapter 2 the goals and requirements of the design tool were formulated. In chapter 3 several aspects, processes and parameters that influence the design and feasibility of offshore harbour design were listed. Based on these analyses the design tool can actually be developed. This chapter discusses the structure of the design tool. The first section is an introduction in which a general overview of the design tool is given. In the subsequent sections the various modules of which the design tool consists are elaborated in more detail. Furthermore, the assumptions and formulas used in the design tool are discussed, including the motivation for these choices. In section 4.6 the results of an example run are presented, including the parameters used to obtain these results.

4.1 Introduction

4.1.1 Software

Various types of software could be used for the construction of the design tool. Two software programs were presented as possible candidates for the design tool:

- Microsoft Excel had the main advantage that it is easy in use (input data and formulas can be inserted easily). Furthermore, rapid insight can be obtained in the effect of one parameter on the rest of the calculation: all other calculation results can be changed immediately by just changing the value of one parameter. Moreover, Excel does not require any (expensive) licences. The disadvantage of Excel is its limited computational strength: as soon as a spreadsheet becomes too large, the Excel files become corrupt and can no longer be used.
- MATLAB has the advantage that it has a large computational strength. Large amounts of data can be analysed with MATLAB in a relatively short period of time. However, MATLAB is more difficult to use than Excel as the user must be familiar with its programming language. Furthermore, MATLAB requires a licence which may not be available for everyone.

In order to make the design tool easily accessible for the staff of Royal HaskoningDHV, it has been chosen to develop the design tool in Excel. Due to the license required for MATLAB and the complex programming language, it has been concluded that MATLAB is less user-friendly compared to Excel and should therefore not be used. Moreover, MATLAB is more suited to perform complex analyses on data obtained from wave modelling software. Because none of these modelling programs are used in the design tool and only simple formulations will be applied (section 3.3.1), Excel is deemed to be sufficient for the level of accuracy required.

4.1.2 Global structure

In figure 4.1 the global structure of the design tool is presented. The design tool consists of various modules, that are indicated with different colours in the scheme of figure 4.1. In this subsection this structure is explained in more detail.



Figure 4.1: Schematized structure of the design tool.

Cost module

The first module of the design tool is the so called 'Cost module'. This tool forms the basis of the design tool, mainly because all input parameters, including computational grid parameters, are inserted in this tool. Furthermore, the final results of the design tool are also displayed in this file. The main function of the cost tool is to calculate the costs of the offshore harbour as a function of x_{ship} . In this module the CAPEX and OPEX of the jetty and the dredging works are calculated (functionality requirement 1 and 2). By importing the breakwater CAPEX and OPEX (for two breakwater types at four lengths) and the waiting costs from the other modules, the total costs as a function of x_{ship} can be determined for all nine harbour types (no breakwater, four rubble mound breakwater lengths and four caisson breakwater lengths). Section 4.2 discusses the cost tool in more detail.

Breakwater module

The second module calculates the dimensions and costs (CAPEX and OPEX) of the rubble

mound and caisson breakwater for four different lengths. As was mentioned in section 3.1, the dimensions of the breakwater depend on the significant wave height under design conditions. With the formulas mentioned in chapter 3.3 the development of the significant wave height under design wave conditions from the offshore boundary of the computational domain to the coastline can be determined. Based on the design wave height at a certain location, the dimensions of the caisson and rubble mound breakwater can be estimated using the design formulas mentioned in sections 3.1. Using the calculated dimensions, the CAPEX and OPEX can be determined for both breakwater types at four different lengths (1, 2, 3 and 4 times the length of the design vessel L_s). In this way functionality requirement 3 (section 2.3.2) is fulfilled. Section 4.3 gives more information about the breakwater module.

Wave calculation module

The design tool gives the user the ability to define a number of governing wave conditions. With the formulas of chapter 3 the significant wave height H_{m0} can be determined as a function of the offshore distance for the case without a harbour. Furthermore, the infragravity wave height $H_{l,m0}$ can also be calculated using equation 3.16. After the wave heights have been calculated for the undisturbed case, the effect of the rubble mound and caisson breakwater is incorporated by determining diffraction and transmission coefficients. The total short and long wave height at the berth is equal to the sum of the transmitted and diffracted short and long waves respectively (functionality requirement 4). Based on criteria for maximum short wave height (as presented in table 3.5) and maximum long wave height, it can be determined whether downtime at the berth will occur under a certain wave condition or not. Furthermore, based on the tugging distance and the maximum allowable wave height for tugboats, it can be determined whether downtime due to tugboat unavailability occurs. Functionality requirement 5 (section 2.3.2) can be fulfilled by the latter two operations. Section 4.4 elaborates more thoroughly on the wave condition tool.

Waiting cost module

In the final module the results of the wave calculation are expressed in waiting costs (functionality requirement 6). The total waiting costs are equal to the costs due to operational downtime plus the costs due to navigational downtime. The first category of waiting costs can be determined using the total amount of time that a ship is at the berth and cannot be loaded or unloaded. The second category of waiting costs can be determined by determining the total amount of time that ships have to wait at an anchorage in order to enter the harbour. The total waiting costs are exported to the cost module, where based on the total cost functions a recommendation is given for the optimum harbour lay-out (functionality requirement 7).

4.1.3 Structure of each module

Each module has globally the same structure. The module starts with a 'Front' sheet on which the name of the module is displayed, as well as a short description of what the task of the module within the design tool is. The next sheet is a 'Summary' sheet, on which the most important output of the module calculations are displayed. Only the cost module starts with the 'Front sheet', followed by the 'Final results' sheet which displays the total cost functions for all nine harbour lay-out types. This is in accordance with usability demands 3 and 4 which stated that the most important output should be stored on one single sheet, and intermediate results should be displayed on seperate sheets. In the following sheet intermediate cost results are displayed in a 'Summary sheet'.

After the 'Summary' sheet, an 'Input' sheet is available in which the user can see the input values that are used for the calculations in a certain module. The cost module is the only module in which the input values are allowed to be changed. The three other modules all import their input values from the cost module, which means that no input values have to be inserted in sheets other than the cost module input sheet. This is in accordance with usability demand 1 in section 2.3 which stated that the user should only fill in the input data in one sheet. This means that all input and final results can be found in one Excel file. Intermediate calculations are done in the other Excel files. Using references, the output of one Excel file can be used as input for another Excel file.

The input sheet in all modules is followed by calculation sheets in which various parameters are calculated as a function of x_{ship} . An overview of the structure of the modules is displayed in table 4.1. Screenshots of examples of sheets are displayed in appendix D.

Sheet name	Remarks
Front sheet	
Final results	only in cost module!
Summary	
	only the cost module values are
Input sheet	allowed to be changed; the other
	modules import their input values
Calculation sheets	

Table 4.1: Structure of each module.

4.2 Cost module

4.2.1 Input parameters

The first step the user has to take is inserting a number of input parameters that can be used for the calculation. It must be noted that these parameters can be changed freely by the user in order to investigate the effect of a certain parameter on the model output. Only parameters that are used directly in the cost module are treated here. Parameters that are inserted in the cost module, but are used in other modules are treated at the relevant modules. Example values are given in this section, but the user is of course free to adapt them the way he or she likes.

Grid parameters

A computational grid in the cross-shore direction (along the shore normal) is defined by the user. The users inserts the value of the onshore boundary in m x_{min} (with 0 m being equal to depth contour of zero water depth) and the offshore boundary of the grid x_{max} . It is advised to locate x_{max} at the location where offshore wave data is available, because x_{max} will also be the offshore boundary of the wave calculation. Furthermore, a step size x_{step} is defined by the user. The step size must be defined such that the number of grid elements does not exceed the maximum allowable amount of grid elements (calculated as $n_{elements} = \frac{x_{max} - x_{min}}{x_{step}} - 1$), which is equal to 500. This limitation has been chosen in order to prevent the Excel spreadsheets from becoming too large, resulting in corrupted Excel files that can no longer be used. If the maximum number of elements is exceeded, an error message is displayed in the input sheet.

Cost parameters

Unit costs for dredging and the jetty have to be defined. In the following it is explained how the unit costs have to be defined. The form of the unit costs is mainly based on a CAPEX estimate for an offshore dry bulk harbour in Liberia. Furthermore, the estimations for the unit costs for the jetty and dredging are also obtained from the project [2].

• The total dredging costs depend on the amount of soil that has to be dredged. Larger dredging volumes logically result in larger dredging costs. In the Liberia cost estimate, the

CAPEX is determined using a unit cost for the dredging of 1 m^3 of soil. By multiplying the unit cost with the total dredged soil volume, the CAPEX of the dredging works can be determined. Examples of dredging unit costs are displayed in table 4.2 [2].

Soil type	Unit cost [USD / m^3]
Sand / soft material	10
Weathered rock	15

Table 4.2: Unit costs for dredging works. [2]

• The jetty consists of three elements: the conveyor belt, the trestle and the berth. In the Liberia cost estimate, the unit cost for the conveyor belt is simply an average price per m. This unit cost depends mainly on the size of the design vessel. Furthermore, the degree of exposure also seemed to influence the unit cost for the conveyor belt. In the design tool, simply one value for the unit cost for the conveyor belt has to be defined. Examples as obtained from the Liberia cost estimate are displayed in table 4.3 [2]. The unit costs for the quay and the trestle have to be estimated in a different manner. Tables 4.4 and 4.5 display values for the quay and trestle unit costs at different water depths, as obtained from the Liberia cost estimate. It can be seen that the unit cost increases linearly with the water depth. This is a logical assumption: the deeper the water becomes, the longer the piles have to become and the more material required. Furthermore, at larger distances off the coast the wave conditions might be more rough, which means that the structural strength of the jetty has to increase. The unit cost seems to increase linearly with depth, which means that the following unit cost function can be defined:

$$uc_x = uc_0 + uc_h \cdot h \tag{4.1}$$

With uc_x the unit cost in USD/m, uc_0 the unit cost at zero water depth, h the average water depth along the trestle or berth in m and uc_h the rate with which the unit cost increase for an increasing water depth in USD/ m^2 . Using the values obtained from the Liberia cost estimate, it can be seen that for the quay uc_0 is equal to 40,000 USD and uc_h is equal to 1,333 USD. For the trestle uc_0 is equal to 10,000 USD and uc_h is equal to 5,000 USD.

Ship size	Unit cost conveyor belt [USD/m]
Exposed VLOC	2407
Sheltered VLOC	2037
Capesize sheltered	1944
Panamax sheltered	1905

Table 4.3: Unit costs for conveyor belts. [2]

Water depth along the quay [m]	Unit cost [USD/m]
5	46,667
10	53,333
15	60,000
20	66,667
25	73,333

Table 4.4: Unit costs for the quay. [2].

Water depth along the trestle [m]	Unit cost [USD/m]
5 to 10	15,000
10 to 15	20,000
15 to 20	25,000

Table 4.5: Trestle unit costs at several water depths. [2]

The costs of the breakwaters are calculated in the breakwater design tool, which is treated in section 4.3.

The unit costs are used to calculate the CAPEX for each cost element. The OPEX also has to be determined. For preliminary cost estimates, the OPEX is usually assumed to be equal to a fixed fraction q_{OPEX} of the total CAPEX: $OPEX_{yearly} = CAPEX \cdot q_{OPEX}$. OPEX percentages for the jetty, breakwaters and dredging can be inserted. This percentage can be used to express the amount of maintenance required for a certain structure or element. For example: if the harbour is located in an area that is very sensitive for sedimentation, maintenance dredging works will have to be performed relatively often, leading to an increase in maintenance dredging costs. This effect can be taken into account by using a relatively high OPEX percentage (for example: the yearly OPEX for dredging amounts 5% of the total CAPEX).

Because CAPEX is an expenditure that is made only *once* (i.e. at the start of harbour development), whereas OPEX is a cash flow that takes place over a certain *period* (throughout the lifetime of the harbour), the effect of the decreasing time value of money has to be taken into account. This is done by determining the total net present OPEX over a certain calculation period. This period can be defined by the user and is for example the expected payback period for the harbour project. In order to determine the net present OPEX, the rate with which the value of money decreases (i.e. the yearly discount rate r) has to be known. The total OPEX for a certain element can then be calculated by using the following formula:

$$OPEX_{tot} = \frac{OPEX_{yearly} \cdot t}{(1 + (r/100))^t}$$

$$\tag{4.2}$$

With t the calculation time in years (usually taken to be equal to 20 years, but can be freely defined by the user) and r the discount rate in %. The total costs for each element are determined by simply summing up the CAPEX and OPEX.

Other input parameters

The following parameters still have to be inserted by the user:

• The bed slope s_{bottom} , given as the ratio of the vertical over the horizontal slope distance. A constant bed slope (parallel depth contours) is assumed throughout the computational domain (figure 4.2), which means that the water depth z at each offshore distance x can be calculated using:

$$z = s_{bottom} \cdot x \tag{4.3}$$

- The average orientation of the coast and the orientation of the approach channel $\phi_{channel}$, given as the angle in degrees measured clockwise relative to the North (figure 4.3).
- Wind and current speeds and directions in m/s and degrees measured clockwise relative to the North respectively. These are only used for the design of dredged areas and are not further used for any downtime calculations. The way in which directions are defined is displayed in figure 4.3. Moreover, the maximum and minimum tidal elevation in m + MSL also have to be inserted, as they will be used for the design of the breakwater and the dredged areas.

Bathymetry



Figure 4.2: Bathymetry for a $x_{min} = 0m$, $x_{max} = 6,000m$ and $s_{bottom} = 0.01$.

- Governing wave statistics, i.e. a governing H_{m0} , T_p and ϕ based on wave data for an entire year. The governing wave parameters are used for two calculations. First, these conditions are used in the design of dredged areas, as the dredging width depends on the prevailing wave conditions. Secondly, these conditions are used to determine the width of the surf zone. Based on this surf zone width, it is determined what the onshore boundary of offshore harbour development has to be. It was mentioned in chapter 2 that offshore harbours are especially advantageous if they are constructed outside the surf zone, as this will lead to a minimum disturbance of the coastal morphological processes.
- Soil type and dredged soil underwater slope s_{dredge} . The soil type is necessary to determine the required keel clearance under the ship. A choice can be made between mud, sand / clay and rock / coral. The underwater slope s_{dredge} is required to determine dredged soil volumes.
- Vessel dimensions (length, breadth, draft) and mass. These are used for several calculations throughout the design tool. The mooring arrangement of the ship (parallel or perpendicular to the coast) can also be chosen. Furthermore, the user can choose the quality of manoeuvring characteristics (poor / moderate / good), sailing speed when entering the approach channel (5 - 8 (low) kts, 8 - 12 kts (moderate), > 12 kts (fast)) and the quality of navigation aids (moderate / good / excellent). These final three parameters are also used to determine the required dimensions of dredged areas.

Calculation of required dimensions dredged areas

The dredging depth is determined using the PIANC guidelines. Because the approach channel is an outer channel (it is located completely in unsheltered waters) table 3.2 is used to determine the required approach channel water depth. Based on the inserted value of the governing significant wave height a ratio between the water depth and the draft of the design vessel can be calculated. An average value for this ratio is chosen (e.g. if the required depth is between 1.3 and 1.4 D_s , a ratio of 1.35 D_s is chosen). An extra allowance is added depending on the soil type chosen by the user. Furthermore, the dredging depth has determined with respect to low tide (e.g. LLW or LAT), in order to ensure harbour accessability under all tidal levels. It was mentioned in section 3.2 that the drop in water level during low tide is generally smaller than 1 m. For a design vessel draft of 12 m, a moderate swell climate with a $H_{m0} = 2m$ and a bottom made up of sand, a required water depth of $h_{gd} = 12 \cdot 1.25 + 0.5 + 1 = 16.5m$ is found.

The determination of the required channel width requires much more effort due to the large number of allowances that have to be taken into account. Figure 4.4 shows some (not all) allowances that are prescribed by PIANC for outer channels [30].



Figure 4.3: Definition of the directions of several parameters.

The dredging width is determined using equation 3.1 [30]:

- The basic width W_{BM} depends on the manoeuvring characteristics: poor manoeuvring characteristics require a basic width of $1.8W_s$, good characteristics require $1.4W_s$ and excellent characteristics require $1.3W_s$.
- An allowance for vessel speed is only required if the vessel sails fast. In that case $0.1W_s$ should be added to the dredging width.
- An allowance for the cross-wind is also necessary. The cross-wind speed is determined by multiplying the inserted wind speed with the sine of the difference between the approach channel direction and the governing wind direction. Depending on the value of the cross-wind speed and the vessel sailing speed, the value of the allowance (as a fraction of the vessel width) is read out of the top table of figure 4.4. In this table a strong cross-wind speed is larger than 33 kts (16.5 m/s), a moderate wind speed is between 33 and 15 kts and a mild wind speed is smaller than 15 kts (7.5 m/s).
- An allowance for the cross-current is determined based on the middle table of figure 4.4. To this end, the cross-current speed first has to be determined by multiplying the inserted current speed with the sine of the differential angle between the governing current direction and the approach channel. A cross-current is negigible if the speed is below 0.1 m/s, low between 0.1 and 0.25 m/s, moderate between 0.25 and 0.75 m/s and strong if the speed is larger than 0.75 m/s.
- An allowance for longitudinal currents is determined using the bottom table in figure 4.4. The longitudinal current speed is determined by multiplying the inserted current speed with the cosine of the differential angle between the approach channel direction and governing current direction. A longitudinal current is considered strong if its speed is larger than 1.5 m/s, low if the speed is below 0.75 m/s and moderate if it is in between these values.

Cross-wind allowances					
	vessel speed				
P.	fast	moderate slow			
a mild		0.1	0.2	0.3	
g moderate		0.3	0.4	0.6	
strong		0.5	0.7	1.1	

Cross-current allowances					
	vessel speed				
-		fast	moderate slow		
ee	negligible		0	0	0
tsp	low		0.2	0.25	0.3
ren	moderate		0.5	0.7	1
5	strong		1	1.2	1.6

Longitudinal current allowances					
p		vessel speed			
0ee		fast		w	
it s	low		0	0	0
Ter	moderate		0	0.1	0.2
CUL	strong		0.1	0.2	0.4

Figure 4.4: Some allowances of the approach channel width as prescribed by PIANC. The numbers indicate the allowances as a fraction of the vessel width W_s . [30]

- An allowance for beam waves is determined based on the inserted governing wave direction. If the angle between the governing wave direction and the approach channel is smaller than 20° no allowance is required. An angle between 20° and 40° required an allowance of $0.5W_s$ and an angle larger than 40° required an angle of W_s .
- An extra allowance may be required depending on the quality of navigational aids. If the aids are excellent no extra allowance is required; good aids require an allowance of $0.2W_s$ and moderate aids require an allowance of $0.4W_s$.
- An allowance for the bottom surface is determined based on the soil type: a soil consisting of rock / coral requires an allowance of $0.2W_s$, other soil types require an allowance of $0.1W_s$.
- An allowance for the water depth is required: a depth smaller than $1.25D_s$ results in an allowance of 0.2 W_s , an depth between 1.25 and $1.5D_s$ results in an allowance of $0.1W_s$ and a water depth larger than $1.5D_s$ results in no extra allowance.
- The bank clearances W_{BR} and W_{BG} depend on the vessel speed and the type of clearance. In this harbour design, the channel banks are always sloping, usually with a slope steeper than 0.10 (the exact value has to be inserted by the user). In the case of sloping banks, the bank clearance is equal to $0.7W_s$ for large sailing speeds, $0.5W_s$ for moderate sailing speeds and $0.3W_s$ for low sailing speeds.

The total width of the approach channel is equal to the sum of the basic width, bank clearances and all required allowances. PIANC recommends a turning circle diameter of 2 to 3 times the

vessel length. Because the turning circle is located in relatively exposed water, the vessel might need more space in order to safely change its direction. Furthermore, the berth is also located in the turning circle, which means extra space is required. This is why the diameter D_{tc} is set equal to $3L_s$.

Furthermore, the quay length, even though not a dredged section, also deserves attention. Because the berth only has to accomodate one vessel, the length of the quay is set equal to L_s plus an extra quay clearance fore and aft. It is assumed that 10% clearance fore and aft is sufficient, which means that the quay length becomes equal to: $L_{quay} = 1.2L_s$.

Surf zone width

It has been mentioned in chapter 2 that one of the requirements of the harbour is that it should be located outside the surf zone. In this way the morphological processes near the coast are not disturbed and required maintenance dredging volumes are expected to be lower. This is why it is necessary to determine at which distance off the coast the surf zone starts. An important concept in this matter is the depth of closure, which is defined as the most landward depth seaward of which no significant net sediment transport between the nearshore and the offshore occurs. No significant change in bottom elevation occurs for depths larger than the depth of closure [32]. In order to minimize the effect of the harbour on coastal morphological processes, it is thus important to construct the harbour in depths larger than the depth of closure.

The depth of closure $h_{closure}$ can be estimated using an empirical formula developed by Birkemeier (1985), which is given by [32]:

$$h_{closure} = 1.75H_{m0} - 57.9(\frac{H_{m0}^2}{gT^2})$$
(4.4)

It can be seen that the depth of closure depends on the wave conditions. In the design tool, the operational wave conditions as inserted by the user are used for the determination of the closure depth. For example, a H_{m0} of 2.5 m can be inserted because waves larger than 2.5 m occur less than 1% of the time, as was found in appendix A.1. It is advised to use wave conditions with a small frequency of occurrence in order to make sure that the offshore harbour is always located outside the surf zone. However, the wave height as inserted by the user is the wave height at the offshore boundary. The wave height does not remain constant, but changes due to the processes of nearshore wave propagation as discussed in chapter 3. Therefore the wave height as a function of the offshore distance has to be found. This can be done using the expressions of linear wave theory as presented in chapter 3. The following steps are taken:

• First the wave length has to be calculated. The wave length is equal to the phase velocity multiplied by the peak period: $L = cT_p$. The peak period is inserted by the user. By substituting the expression for the phase velocity as given by equation 3.6, the following equation is obtained:

$$L = \frac{gT^2}{2\pi} \tanh(\frac{2\pi h}{L}) \tag{4.5}$$

This is an implicit equation that cannot be solved analytically. Luckily, Excel comes with a numerical Solver that is able to solve implicit equations by means of iteration. By using Excel's programming language (macro's) a program can be written in Visual Basic Editor that solves this equation upon user's request. However, the Excel Solver takes a relatively long time to solve implicit equations. Solving the equation for all defined grid points (which can be as much as 500 points), might take hours, and that is why equation 4.5 is only solved for a limited (25) amount of grid points. These values are displayed in a table, as is displayed in figure 4.5. The right column contains the offshore distances for which the wave lengths are calculated. The second column contains the values for the wave length in m. The last column displays the solution of the following calculation: $L - \frac{gT^2}{2\pi} \tanh(\frac{2\pi h}{L}) = 0$. The Solver solves this equation by means of iteration and displays the solution (i.e. the wave length) in the third column. If the iteration is completed, the value in the fourth column becomes equal to zero. A nonzero value indicates that the wave lengths have to be calculated again. By clicking on the "Calculate L" button, the macro is activated and the wave length calculation can start. The wave length for all other points (which are not present in the table) are determined by means of linear interpolation between the calculated wave lengths.

Bathymetry and wave lengths				
Offshore distance [m]		Depth [m]	Wave length [m]	f(L)
	0	0	0	0.00
Calculate L	120	1.2	68.48259406	0.00
	240	2.4	96.65363027	0.00
	360	3.6	118.1366523	0.00
	480	4.8	136.1359365	0.00
	600	6	151.8953361	0.00
	1200	12	212.6234557	0.00
	1800	18	257.7248216	0.00
	2400	24	294.4917938	0.00
	3000	30	325.7800607	0.00
	3600	36	353.070044	0.00
	4200	42	377.2509816	0.00
	4800	48	398.9090816	0.00
	5400	54	418.4566799	0.00
	6000	60	436.1981455	0.00
	6600	66	452.3667452	0.00
	7200	72	467.1467619	0.00
	7800	78	480.6874866	0.00
	8400	84	493.1124754	0.00
	9000	90	504.5258864	0.00
	9600	96	515.0169571	0.00
	10200	102	524.6632355	0.00
	10800	108	533.532965	0.00
	11400	114	541.6868816	0.00
	12000	120	549.179572	0.00

Figure 4.5: Wave length calculation table, as used in the design tool.

- The wave direction relative to the shore normal ϕ at x_{max} is determined by taking the absolute difference between the wave direction and the coastal orientation.
- For each grid point, the wave celerity is determined using $c = L/T_p$. The water depth is calculated using equation 4.3 and the wave number is calculated using $k = 2\pi/L$. With these values, the parameter n can be determined at each grid point using equation 3.8. Moreover, the deviation of the wave direction from the shore normal can be determined using Snell's law (3.9), by inserting for ϕ_0 and c_0 the values obtained at the offshore boundary x_{max} .
- The wave height without wave breaking is calculated for each offshore distance by multiplying the design wave height at the offshore boundary $H_{m0,0}$ with the shoaling and refraction coefficients K_s and K_r . These coefficients are determined for every x using equations 3.7 and 3.10.

• For each offshore distance the closure depth $h_{closure}$ is determined based on the local wave height as determined in the previous step. It is verified whether the closure depth as found using equation 4.4 is larger or smaller than the actual water depth at that location. The first offshore point (as seen from the coastline) where the water depth is smaller than the closure depth is considered to be the offshore boundary of the surf zone, denoted as x_{surf} .

For example: an operational wave condition of $H_{m0} = 2.5$ m, $T_p = 12$ s and $\phi = 210^{\circ}$, and a bed slope of 1/100, the surf zone starts at an offshore distance of 680 m. The design tool has to make sure that all harbour alternatives developed are more offshore than x_{surf} .

Harbour cross-section

Two mooring arrangements can be chosen: an arrangement parallel to the coast and an arrangement perpendicular to the coast. In chapter 2, several advantages and disadvantages of both arrangements were mentioned. Furthermore, in figure 2.6 and 2.7 the lay-out of the harbour is displayed. It is tried to locate the ship as close to the breakwater as possible, in order to improve the amount of breakwater sheltering at the berth. In figure 4.6 a schematization of the cross-section of the offshore harbour (without approach channel and breakwater and not to scale) is displayed. The water level is at MSL.



Figure 4.6: Cross-section of the offshore harbour.

Several parameters are indicated in this figure, which are defined as follows:

- s_{bottom} is the slope of the original bottom without any human intervention, whereas s_{dredge} is the slope of the side banks of dredged areas (turning circle and approach channel).
- x_{surf} is the distance between the coastline and the start of the surf zone.
- x_{ship} is the distance between the centre of the design vessel moored at the berth and the coastline where the water depth is equal to zero.
- x_{dredge} is the offshore distance beyond which the natural depth is larger than the minimally required depth h_{qd} for the vessel.
- D_{tc} is the diameter of the turning circle. x_{centre} is the centre of the turning circle. The harbour is designed such that the approach channel reaches the turning circle at an offshore distance of x_{centre} . In this way the approach channel is located further off the breakwater which reduces the probability of intersection between the breakwater and the approach channel (section 4.3.2)

- x_{int} and $x_{int,2}$ are the intersection points of the side banks of the turning circle with the original bottom profile. These are the most onshore and offshore points of the turning circle.
- x_{tc} and $x_{tc,2}$ are the intersection points of the side banks of the turning circle with the dredged bottom of the harbour where the water depth is equal to the minimally required depth h_{qd} .
- x_{safe} is the minimum distance required between the centre of the ship and the edge of the turning circle. If the ship is placed perpendicular to the coast, x_{safe} is equal to $0.6L_s$, which is the sum of half the ship length (as this distance is measured from the centre of the ship) plus an allowance of $0.1L_s$. If the ship is placed parallel to the coast, x_{safe} is equal to $1.5W_s$, which is the sum of half the vessel width plus an allowance equal to W_s . The allowances are necessary in order to prevent that the ship hits the sloping edge of the breakwater.

If x_{ship} is known, then all other cross-sectional parameters can be determined based on the input parameters. It can easily be seen that the inner edges of the turning circle can be determined using:

$$x_{tc,2} = x_{ship} + x_{safe} \tag{4.6}$$

$$x_{tc} = x_{tc,2} - D_{tc} \tag{4.7}$$

$$x_{centre} = \frac{1}{2}(x_{tc} + x_{tc,2})$$
(4.8)

All slopes can be expressed as vertical level z as a function of the offshore distance x. The original bottom profile is defined by equation 4.3. The onshore dredged slope is defined by: $z = s_{dredge}x + a_1$ whereas the offshore dredged slope is defined by: $z = -s_{dredge} + a_2$. The coefficients a_1 and a_2 can be determined by solving the first equation for: $z(x_{tc}) = h_{gd}$ and $z(x_{tc,2}) = h_{gd}$. The following equations are then obtained:

$$x_{int} = \frac{s_{dredge} x_{tc} - h_{gd}}{s_{dredge} - s_{bottom}}$$
(4.9)

$$x_{int,2} = \frac{h_{gd} + s_{dredge} \cdot x_{tc,2}}{s_{dredge} + s_{bottom}}$$
(4.10)

Boundaries of harbour development calculation

Offshore harbour development is not investigated throughout the entire computational domain, because large areas of this domain might be not interesting for harbour development from an economic, technical or environmental perspective. It was already mentioned that offshore harbour development has to be sought outside the surf zone, i.e. beyond x_{surf} , in order to prevent disturbance of coastal morphological processes. On the other hand, placing the ship far offshore, way beyond x_{dredge} , which is the offshore distance beyond which the natural water depth is larger than the minimally required water depth, would lead to a serious increase of costs without actual additional benefits compared to the case when the ship is positioned closer to the coast, but at a distance further offshore than x_{dredge} . That is why two boundaries for offshore harbour development are determined based on the dredging distances.

• $x_{ship,min}$ is the most onshore boundary of the centre of the ship and is defined as the x_{ship} where the onshore side slope of the turning circle hits offshore boundary of the surf zone, or in other words: where $x_{int} = x_{surf}$. By setting equation 4.9 equal to x_{surf} and solving for x_{ship} , the following onshore boundary can be obtained:

$$x_{ship,min} = \frac{x_{surf}(s_{dredge} - s_{bottom}) + h_{gd}}{s_{dredge}} + D_{tc} - x_{safe}$$
(4.11)

• $x_{ship,max}$ is the most offshore boundary of the centre of the ship and is defined as the x_{ship} where the onshore side slope of the turning circle hits the depth contour where the natural depth is equal to the required depth, or: $x_{int} = x_{dredge}$. Beyond this boundary no dredging is required any more and the natural depth is sufficient for the entire harbour to be accomodated without any additional dredging works. Setting equation 4.9 equal to x_{dredge} and solving for x_{ship} results in:

$$x_{ship,max} = \frac{x_{dredge}(s_{dredge} - s_{bottom}) + h_{gd}}{s_{dredge}} + D_{tc} - x_{safe}$$
(4.12)

The CAPEX, OPEX and waiting costs are only calculated for values of x_{ship} between $x_{ship,min}$ and $x_{ship,max}$.

4.2.2 Jetty costs

The CAPEX for the jetty are determined by summing up the CAPEX of the conveyor belt, berth and trestle. The OPEX is determined using equation 4.2. Estimates for unit cost rates have been treated in section 4.2.1 and are displayed in tables 4.3 to 4.5.

Conveyor belt costs

The CAPEX for the conveyor belt is determined by multiplying the length of the conveyor belt with the unit cost. The length of the conveyor belt is (at least) equal to the offshore distance of the ship x_{ship} . Therefore the CAPEX calculation becomes:

$$CAPEX_{conveyor} = uc_{conveyor} \cdot x_{ship} \tag{4.13}$$

Berth costs

The costs for the berth can be found by multiplying the length of the berth with the unit cost. In section 4.2.1 it was explained that the unit cost depends on the local water depth, which for the berth is equal to h_{qd} . The cost function therefore becomes:

$$CAPEX_{berth} = L_{quay} \cdot (uc_0 + uc_h \cdot h_{qd}) \tag{4.14}$$

The berth cost is constant for each offshore distance, as none of the parameters in the equation above depend on the offshore distance.

Trestle costs

The CAPEX of the trestle can be calculated by multiplying the unit cost with the length of the trestle. The trestle length is equal to the distance between the shoreline x = 0 and the onshore boundary of the quay. In the perpendicular arrangement, the length of the trestle is equal to $x_{trestle} = x_{ship} - 0.5L_{quay}$. The trestle length is slightly longer for the parallel arrangement. Assuming that the distance between the beam of the ship and the quay is equal to $0.1W_{vessel}$, the trestle length becomes: $x_{trestle} = x_{ship} + 0.6W_s$.

The unit cost of the trestle depends on the water depth in which the trestle is located, as was treated in section 4.2.1. In this case this water depth does depend on the offshore distance of the ship, and is equal to the weighted average \bar{h} of three water depths:

- The water depth in the part of the trestle that lies in non-dredged soil. This part is located between x = 0 and x_{int} . The average depth in this part is equal to $\frac{1}{2}x_{int}s_{bottom}$.
- The water depth in the part of the trestle that lies in the side slope of the turning circle. This part is located between x_{int} and x_{tc} and has an average depth of $\frac{1}{2}(h_{qd} + x_{int}s_{bottom})$.

• The water depth in the part of the trestle that is located completely within the non-sloping part of the turning circle. This part is located between x_{tc} and $x_{trestle}$ and has an average depth of h_{qd} .

The trestle cost in USD as a function of x_{ship} then becomes:

$$CAPEX_{trestle} = x_{trestle}(uc_0 + uc_h \cdot h) \tag{4.15}$$

The design tool plots the total jetty cost as a function of x_{ship} . An example of such a plot is displayed in figure 4.7. Note that the berth costs are not included in the plot because this is a constant value that does not depend on x_{ship} . It is included in the calculation of the total CAPEX of the jetty.



Figure 4.7: CAPEX of the jetty as a function of x_{ship} , as calculated by the design tool. For the input values used, reference is made to section 4.6.

4.2.3 Dredging costs

Dredging costs are determined by multiplying the total volume of soil that has to be dredged with the cost to dredge 1 m^3 of soil. The total dredged volume V_{dredge} is equal to the sum of the volume of the approach channel $V_{approach}$ and the turning circle V_{tc} :

$$CAPEX_{dredge} = uc_{dredge} \cdot (V_{approach} + V_{tc}) \tag{4.16}$$

Approach channel

The approach channel consists of two main parts: the main channel and the sloping edges. The channel starts at x_{dredge} and hits the turning circle at x_{centre} . The channel has an angle of $\phi_{channel}$ relative to the shore normal. This means that the total channel length is equal to $(x_{dredge} - x_{centre}) \cdot \cos \phi_{channel}$. The dredging depth at the offshore boundary x_{dredge} is equal to zero whereas it has a value of $(h_{gd} - z_{centre})$ at the onshore side of the channel. A simplified sketch of the volumetric body that has to be dredged is displayed in figure 4.8. The main channel has the shape of a prism, of which the volume is equal to:

$$V_{approach,1} = \frac{1}{2} W \cdot (\cos \phi_{channel}) (x_{dredge} - x_{centre}) (h_{gd} - z_{tc,2})$$
(4.17)

The sloping sides have the shape of a pyramid of which the total volume is equal to:



Figure 4.8: 3D sketch of the approach channel with relevant dimensions.

Turning circle

The turning circle has the shape of a truncated cone. At the bottom plane of the turning circle has a circular shape with a diameter of $D_{tc} = 3L_{ship}$. The top plane has an elliptic shape of which the axes of approximately the same length, which means that it can be approximated by a circle with a diameter of $x_{int,2} - x_{int}$. The top plane is actually inclined with a slope equal to s_{bottom} . This inclination is neglected for the volume calculation, and the turning circle has a height equal to $(h_{gd} - z_{centre})$, of which the volume can be determined as follows:

$$A_{top} = \frac{1}{4}\pi (x_{int,2} - x_{int})^2 \tag{4.19}$$

$$A_{bottom} = \frac{1}{4}\pi (3L_{ship})^2 = \frac{9}{4}\pi L_{ship}^2$$
(4.20)

$$V_{tc} = \frac{(h_{gd} - z_{centre})(A_{top} + A_{bottom} + \sqrt{A_{top}A_{bottom}})}{3}$$
(4.21)

Total amount of dredged soil

As mentioned before, the total amount of dredged soil is equal to the sum of the volume of the approach channel and the volume of the turning circle. This is a simplification as in the contact area of the turning circle and approach channel some overlapping occurs between the two dredged sections, which means that the calculated volume is slightly too much. However, one must not forget that the formulas given above are just approximations, and that the actual dredged areas do not have perfect geometrical shapes as has been described above. Furthermore, usually some overdepth is applied during dredging to take into account siltation effects. The CAPEX of the dredging works as calculated by the design tool is displayed in figure 4.10.


Figure 4.9: 3D sketch of the turning circle with relevant dimensions.

CAPEX Dredging



Figure 4.10: Total CAPEX for dredging works as a function of x_{ship} , determined by the design tool. For the input values used, reference is made to section 4.6.

4.3 Breakwater module

The design and cost estimate of the rubble mound and the caisson breakwater is done in a separate module that is described in this section. This is done because the design of the breakwaters depends on the design wave height. The determination of this wave height for each offshore distance requires a calculation procedure on it own. First the design wave height for the breakwaters is determined. Based on the design wave height, the required cross-sectional dimensions of the rubble mound and caisson breakwater are determined. With these dimensions the CAPEX and OPEX of the breakwaters can be found. Furthermore, several distances required for the wave calculation module are determined.

4.3.1 Design wave height

The design of the caisson and rubble mound breakwater largely depends on the design wave conditions. This was clear from the design formulas that were presented in chapter 3. Both breakwater types should not fail during design conditions. In appendix A.1 it was determined that a breakwater lifetime of 50 years and a failure probability of 1% correspond with a return period of 5,000 years for the design wave conditions. The values for the breakwater lifetime and failure probability were only used to get an idea about the order of magnitude in which the return period for the design wave height has to be sought. These variables may vary from one project to another.

Several wave transformation processes were described in chapter 3. From these processes it becomes clear that the wave height is not constant in the entire coastal zone, but varies with the offshore distance x. In order to be able to make an accurate estimate for the dimensions of the breakwater required at a certain location, the design wave height at that location must also be known. Therefore the design wave height has to be determined as a function of the offshore distance, taking into account the processes related to nearshore transformation of ocean waves. To determine the design wave height as a function of the offshore distance, the following input parameters are required:

- The significant wave height $H_{m0,0}$, peak period T_p and wave direction ϕ_0 at the offshore boundary under design conditions. In appendix A.1 an attempt has been made to estimate the 1/5,000 years wave height and period based on the available NOAA data. It has been found that the wave height is approximately 10 m, whereas T_p is equal to 36 s.
- Furthermore, the grid parameters, the coastal orientation and the average bed slope are also used during this calculation. The coastal orientation and bed slope are especially important to model refraction and shoaling. The definition of these parameters has already been treated in the previous section.

The determination of the design wave height as a function of x for the breakwater module is largely similar to the method with which the surf zone width was determined. The calculation is initiated with the determination of the wave length L for a number of offshore distances by solving equation 4.5. By subsequently calculating the phase velocity c, the parameter n and the wave direction ϕ , the shoaling and refraction coefficients K_r and K_s can be determined for each location. By multiplying these coefficients with the design wave height at the offshore boundary, the design wave height at each x can be found without the incorporation of wave breaking. However, because the waves under design conditions are larger than the waves under operational conditions, wave breaking might occur at a location outside the conventional surf zone. Therefore wave breaking has to be taken into account. This is done using the empirical formulas of Goda for the estimation of the wave height in the surf zone. For each offshore distance, the parameter β_0 , β_1 and β_{max} as displayed in table 3.7 are determined. These parameters are substituted in equation 3.12, after which the design wave height *with* the incorporation of wave breaking is found. In figure 4.11, an example of the design wave height as a function of the offshore distance x is displayed. Based on this function, the breakwaters can be designed.



Figure 4.11: The significant wave height as a function of the offshore distance under design conditions, as determined by the design tool. $H_{m0,0} = 10$ m, $T_p = 36$ s, $\phi_0 = 210^\circ$, $s_{bottom} = 1/100$.

4.3.2 Rubble mound breakwater

General

The first breakwater type that is considered is the rubble mound breakwater. A cross-section of the breakwater as modelled by the design tool is displayed in figure 4.12. It can be seen that the breakwater cross-section essentially has a trapezoidal shape with no berm, with the same value for the slope on both sides of the breakwater. It has been chosen to apply the same slope for the outer and inner slope because the wave conditions on the inner slope under design conditions might still be quite rough due to the exposed location of the breakwater. Moreover, differences in bottom level over the breakwater foot are neglected. This is done because the slope along the West-African coast is generally mild, which means that the drop in bottom level over the breakwater foot will generally be low.

Several cross-sectional parameters can be found in figure 4.12. The definition of these parameters is given in the following:

- x_{on} : the onshore boundary of the breakwater. The water depth at this location is equal to z_{on} .
- x_{off} : the offshore boundary of the breakwater. The water depth at this location is equal to z_{off} .
- x_{wave} : the offshore point where the incoming waves hit the breakwater.
- x_{diff} : the offshore point where it is assumed that the diffracted waves leave the breakwater.
- W_{crest} : the width of the breakwater crest.
- H_{BW} : total height of the breakwater.
- R_C : the freeboard of the breakwater crest, defined as the distance between MSL and the crest of the breakwater.



Figure 4.12: Sketch of the cross-section of the breakwater.

- W_{BW} : the width of the breakwater at the bottom.
- s_{BW} : the slope of the breakwater.
- t_{armour} : the thickness of the armour layer.

Dimensioning

For the dimensioning of the rubble mound breakwater, the user inserts the following parameters:

- The concrete armour unit applied. The user is able to choice one of the five concrete armour units that are displayed in table 3.3. Based on the armour unit, the number of elements in the armour layer can be determined by the design tool. Furthermore, the slopes of the breakwater can also be determined (1:1.5 for double-layer elements, $1:1\frac{1}{3}$ for single-layer elements). Moreover, the roughness coefficient γ_f and the layer coefficient k_t are automatically determined by the design tool based on the concrete armour unit. Furthermore, the stability parameter K_D required is also determined.
- The density of concrete ρ_c and sea water ρ_w . Based on these densities, the relative density $\Delta = \frac{\rho_c \rho_w}{\rho_w}$ of concrete armour units can be determined.
- The maximum acceptable overtopping volume q_{over} . This volume is used to determine the required freeboard. It is difficult to define a maximum allowable overtopping volume for the rubble mound breakwater. Maximum allowable overtopping rates exist, but are usually only applicable to dikes and revetments. For breakwaters the overtopping criterion is usually combined with the transmission criterion and the total amount of wave transmission into the harbour under operational conditions is used as a guideline to determine the breakwater dimensions. Schiereck (2012) states that for overtopping over dikes with high quality slopes a maximum overtopping rates can be used [33]. The Eurotop manual states that for breakwaters larger overtopping rates can be used [33]. The Eurotop manual states that at a value of 50 1/s/m, significant damage to larger yachts in a harbour may occur [43]. Because no other guidelines specific for detached breakwaters could be found, initially the Eurotop criterion may be adapted as a maximum allowable overtopping rates is examined in section 6.5. Moreover, the maximum tidal elevation is also used as input parameter to determine the required breakwater height.

- As mentioned in section 3.1.2, there are several ways to determine the crest width of the breakwater. A guideline of three armour elements can be applied. However, sometimes other requirements for the breakwater crest (related to the construction or maintenance of the breakwater) or other structural elements (crown wall, concrete mass) can be applied for the crest. Because the breakwater crest generally becomes wider with increasing height, it has been chosen to relate the crest width W_{crest} to the freeboard R_C of the breakwater, relative to MSL. The ratio of crest width over freeboard required can be inserted by the user. For example: for a detached breakwater crest is 21 m wide (much higher than the guideline of three armour elements) and the freeboard relative to MSL is equal to 7.5, resulting in a ratio of 2.8 [34].
- Geotechnical stability of the breakwater has not been taken into account so far. This is an important criterion however. If large amounts of concrete and core volumes are dumped near a sloping area (slopes of the breakwater), failure due to slope instability may occur. Therefore a certain safety margin is required to prevent slope instability. Usually the required margin is determined by performing a slope stability calculation using Bishop's method. This is a complicated iterative procedure which is why it has been chosen to estimate the required safety margin around the breakwater based on the width of the breakwater foot. It is assumed that a safety margin of approximately $0.25W_{BW}$ is enough to prevent slope instability. The ratio between the safety margin width and the breakwater width is a input variable that may be adapted by the user.

The dimensioning of the breakwater takes place as follows:

• The required stone size is determined using the stability parameter K_D required for the concrete armour unit chosen:

$$\frac{H_{m0}}{\Delta D_{n50}} = K_D \tag{4.22}$$

The values for the stability parameter were displayed in table 3.3. It must be mentioned that for a certain x_{ship} , the wave height that has to be used to determine the breakwater for the offshore harbour at that location has to be taken at x_{wave} , which is the point where the waves reach the breakwater. However, the value of x_{wave} is not known beforehand. That is why initially, x_{wave} is estimated to be equal to $x_{ship} + 300m$.

For some Cube and Tetrapod armour units, the stability parameter depends on the deep water wave steepness, defined as:

$$s_{0m} = \frac{2\pi H_{m0}}{gT_m^2} \tag{4.23}$$

In this formula T_m is equal to the mean wave period. A swell wave climate as present in West-Africa is usually described using a JONSWAP spectrum [12]. For these spectra, the mean period is equal to approximately 0.79 to 0.87 times the peak period. For the design tool an average value of 0.83 is used. Based on equation 4.23, the wave steepness for each x_{ship} can be determined, based on which the required concrete armour size can be calculated.

The armour layer thickness is calculated using $t_{armour} = nk_t D_{n50}$. In this formula n is the number of elements (1 or 2 depending on the concrete unit chosen) and k_t is the layer coefficient, as displayed in table 3.3. This parameter is used to express the inaccuracies in concrete unit placement.

• The breakwater height H_{BW} is equal to the sum of the local water depth, the maximum tidal elevation and the freeboard R_C . The freeboard is determined using the overtopping

criterion (equation 3.2). Based on the freeboard, the crest width W_{crest} is determined. The bottom width is determined using the breakwater slope:

$$W_{BW} = W_{crest} + \frac{2H_{BW}}{s_{BW}} \tag{4.24}$$

Volumes and costs

Now that all dimensions have been found, the required volumes can be determined:

- Total volume: $V_{total} = (\frac{1}{2}(W_{crest} + W_{BW})H_{BW})L_{BW}$
- Core volume: $V_{total} = (\frac{1}{2}((W_{crest} 2t_{armour}) + (W_{BW} 2t_{armour})(H_{BW} t_{armour}))L_{BW}$
- Armour volume: $V_{armour} = V_{total} V_{core}$

The total CAPEX of the breakwater is determined by multiplying the required volumes for armour and core material with the respective unit costs:

$$CAPEX_{rubble} = uc_{core}V_{core} + uc_{armour}V_{armour}$$

$$(4.25)$$

A cost comparison of several breakwater types carried out by Tuaturima et al. (1998) gives several unit costs for breakwater materials [36]. It must be noted that these costs were given in NLG (Dutch guilders) and are quite out-dated. In order to obtain insight in present-day unit costs for breakwater materials, the obtained unit costs are converted to USD (1 NLG = 0.667 USD) and multiplied by $(1 + r)^{18}$, r standing for the average inflation rate and 18 being the number of years between the publication of the report and the moment in which this report is written. The average inflation rate over the last 18 years was 1.92 % [18]. The results of this calculation are displayed in table 4.6.

Material name	Unit cost $[NLG/m^3]$, 1998	Unit cost $[USD/m^3]$, 2016
Rock armour and filter layers [*]	40.5	38.0
Cubes	300	271
Accropods	325	294
Tetrapods	400	362
Concrete caisson (all in)	500	452
Quarry run*	40.5	38.0

Table 4.6: Unit costs for materials used in breakwaters. * means that in the resource the unit cost was given in tonnes. The price was converted to a unit cost per m^3 using a rock density of 2,700 kg/m³ [36].

Boundaries breakwater

The boundaries of the breakwater are important input parameters for the wave calculation. Based on the boundaries, the amount of transmission and diffraction behind the breakwater can be determined.

- The onshore boundary of the breakwater x_{on} is equal to $x_{int,2}$ plus the required safety margin.
- The offshore boundary is equal to $x_{on} + W_{BW}$.
- The diffraction point can be determined using the breakwater slope and the local water depth: $x_{diff} = x_{on} + \frac{z_{on}}{s_{BW}}$.
- The point where the waves hit the breakwater can be determined in a similar manner: $x_{wave} = x_{off} - \frac{z_{off}}{s_{BW}}$.

Intersection breakwater and approach channel

If the deviation of the approach channel from the shore normal is small and the breakwater length is large, it might happen that the breakwater intersects with the approach channel, or that dredging works have to take place in the assumed safety zone to prevent slope instability around the breakwater. In order to determine whether intersection takes place or not, a simple calculation has to be performed, based on the dimensions given in figure 4.13.



Figure 4.13: Top view of the offshore harbour.

The red line indicates the safety margin that is necessary in order to prevent slope instability around the breakwater (= $0.25W_{BW}$). The red line is not allowed to intersect with the approach channel, that has a deviation from the shore normal equal to the angle $\alpha_{channel}$. When the approach channel reaches the onshore end of the breakwater, it has travelled a distance of $\tan \alpha_{channel} \cdot (x_{on} - x_{centre})$ in alongshore direction. Furthermore, the ship is positioned behind the centreline of the breakwater and at a distance of $2L_{ship}$ from the point where the approach channel meets the turning circle (= x_{centre}). Intersection takes place if:

$$\tan \alpha_{channel}(x_{on} - x_{centre}) < 0.25W_{bottom} + 0.5L_{BW} \tag{4.26}$$

If intersection does take place at a certain x_{ship} , for any of the breakwater lengths (1 to 4 L_{BW}), the model gives a warning.

4.3.3 Caisson breakwater

General

A cross-section of the caisson breakwater designed by the design tool is displayed in figure 4.14.

The design is basically a concrete rectangular shaped caisson filled with sand, placed above a rubble mound foundation. The foundation is necessary to spread the vertical load due to the weight of the caisson and the wave force over a wide area of the seabed [12]. The bottom under the caisson is again assumed to be horizontal.



Figure 4.14: Cross-section of the caisson breakwater.

The dimensions are defined as follows:

- $H_{caisson}$ is the total height of the caisson. $W_{caisson}$ is the width of the caisson.
- h_{sill} is the height of the sill that forms the foundation of the caisson. W_{found} is the width of the rubble mound foundation layer at the contact surface with the concrete caisson.
- x_{diff} is the point where diffracted waves leave the breakwater whereas x_{wave} is the point where incoming waves hit the breakwater.

Dimensioning

The dimensioning of the caisson breakwater is more complicated than for the rubble mound breakwater. The dimensions are usually determined based on stability calculations, in which the resistance against sliding, overturning and uplift is determined based on the acting (wave) forces and pressures [12]. Such an extensive calculation would need a design tool for its own and therefore in this design tool only simple rules of thumb are used to determine the required dimensions. These rules of thumb are mainly derived by comparing three caisson breakwater designs:

- The caisson breakwater for a port in Hazira (India), as designed by Carstens (2001) [6]. See figure 4.15.
- The caisson breakwater as designed by Tutuarima et al. for a cost comparison of various breakwater types [36]. See figure 4.16.
- The caisson breakwater designed for a offshore phosphates harbour in a certain location in Africa [34]. See figure 5.8.

The dimensions and breakwater boundaries are determined as follows:



Figure 4.15: Cross-section of the caisson breakwater designed by Carstens. [6]



Figure 4.16: Cross-section of the caisson breakwater designed by Tutuarima et al. [36].

- The total height of the breakwater $H_{caisson}$ is equal to the sum of the local water depth, the maximum tidal elevation and the required freeboard. Goda (2000) states that a freeboard of 0.6 times the design wave height leads to a tolerable amount of overtopping [12]. Again the design wave height is taken at 300 m offshore of x_{ship} as an initial estimate for x_{wave} .
- The width of the breakwater $W_{caisson}$ is more difficult to estimate. However, it can be seen that a caisson breakwater usually has a rectangular shape of which the width is not much larger than the height. An increase in height locally leads to an increase in width in order to maintain stability against overturning. Therefore it is sought to define the width of the caisson as a ratio of the caisson height. For the three breakwater designs mentioned above, the following ratios are found:

Design	$H_{caisson}$ [m]	$B_{caisson}$ [m]	$\left(\frac{B}{H}\right)_{caisson}$	h_{sill} [m]
Carstens (2001)	19	24.5	1.29	2
Tutuarima et al. (1998)	17.3	22.4	1.29	-
African phosphates harbour	25	25	1	2

Table 4.7: Comparison of the dimensions of the three caisson breakwater designs.

It can be seen that the width is always slightly higher than the height, with a ratio of the

width over the height usually is between 1 and 1.3. The user is able to insert a ratio he or she finds realistic.

- The sill height is also a parameter that can be inserted by the user. Goda (2000) states that the height of the rubble mound foundation should be as low as possible to prevent the generation of large wave pressures. Nevertheless, the height of the sill must not become too small to maintain the vertical load spreading function of the caisson. Goda (2000) recommends a minimum sill height of 1.5 m [12]. From table 4.7 it can be seen that a sill height of 2 m is very common.
- The width of the rubble mound foundation is not given for any of the breakwater designs mentioned. Goda (2000) states that this width is especially important to provide protection against scouring of the seabed. However, a too large berm width may cause impulsive breaking wave pressure. A minimum berm width of 5 m is recommended under normal conditions and 10 m in areas attacked by large storm waves. In the design tool, the user is able to define W_{found} as a ratio of the width of the breakwater, with a minimum value of 1 (no berms, only side slopes). Applying Goda's recommendations to the breakwater widths found above, this would lead to ratios of approximately 1.2 to 1.4. The side slopes of the rubble mound foundation have a value of s_{dredae} .
- x_{diff} is determined by adding the determined berm width to $x_{int,2}$. x_{wave} can be found by adding the breakwater width to x_{diff} .

Volumes and costs

The CAPEX of the breakwater can be found by multiplying the volumes of the caisson and the rubble mound foundation with the respective unit costs. The volumes can be found using the following formulas:

$$V_{caisson} = H_{caisson} B_{caisson} L_{BW} \tag{4.27}$$

$$V_{found} = h_{sill} \left(B_{found} + \frac{h_{sill}}{s_{dredge}} \right)$$
(4.28)

Unit costs for the caisson and the rubble mound foundation (expected to be in the same order as rock armour and quarry run) can be found in table 4.6. The cost function for the rubble mound becomes:

$$CAPEX_{caissonBW} = V_{caisson}uc_{caisson} + V_{found}uc_{found}$$

$$(4.29)$$

4.4 Wave calculation module

The goal of the wave calculation module is to determine whether or not downtime occurs at the berth and due to tugboat unavailability. This is done based on the wave heights at the berth $(H_{m0} \text{ and } H_{l,m0})$ and based on the wave height during the towing journey. These wave heights are based on the wave conditions at the offshore boundary of the grid. The user is able to define a number of wave conditions in the cost module. For the time being, this number of wave conditions is set equal to eight. For each wave condition, a separate Excel spreadsheet is necessary in which the required calculations are performed. These calculations are of course identical for each wave condition. More wave conditions can be defined by adding spreadsheets. However, this also means that in the other spreadsheets that import data from the wave calculation modules, columns and rows have to be added where necessary.

4.4.1 Input parameters

Two types of input parameters have to be inserted: general parameters and wave condition specific parameters.

Wave condition specific parameters

For each wave condition, the following parameters have to be inserted.

- The significant wave height $H_{m0,0}$ at the offshore boundary x_{max} in m.
- The peak wave period T_p .
- The wave direction ϕ_0 at the offshore boundary x_{max} , measured clockwise relative to the North (= 0°).
- The frequency of occurrence of the wave condition in %.
- The percentage of the time that a ship is expected to be present at the berth during the wave condition.

General parameters

The following parameters are the same for all wave conditions:

- The maximum allowable significant wave height at the berth. This parameter is required to determine whether downtime at the berth occurs due to excessive sway and heave motions. For the design tool, the values recommended by Thoresen (2010) are used (table 3.5).
- The maximum allowable surge motion. This parameter is required to determine whether downtime due to infragravity waves occurs at the berth or not. The values recommended by Thoresen (2010) can be used for this purpose (table 3.4). The determination is done using Mol's expression (equation 3.3), which means that values for the vessel's mass, mooring system stiffness an C_x are also necessary.
- The maximum allowable wave height for tugboats. This depends on the required efficiency of the tugboats. It has been mentioned in section 3.2.2 that for wave heights higher than 2 m, tugboat unavailability can be expected.
- The minimum vessel speed during the tying up of the tugboats and the duration of the tying up. This is required to determine the distance along which the tugboats have to travel.

4.4.2 Undisturbed wave propagation

The first step in the wave calculation module is to determine the total significant wave height H_{m0} and the infragravity wave height $H_{l,m0}$ as a function of the offshore distance x. For this initial calculation, the effect of the breakwater is neglected, and waves propagate freely from the offshore boundary x_{max} towards x_{min} . A similar procedure as for the surf zone width calculation and the breakwater design wave height calculation is applied. The calculation starts with the determination of the wave length L for a number of offshore locations, followed by the calculation of the phase velocity c, the parameter n and the wave direction ϕ . Based on these parameters the shoaling and refraction coefficients can be determined. This is all done based on linear wave theory as was demonstrated in the previous sections. The effect of wave breaking is incorporated using Goda's empirical formulas (equation 3.12). The result is the total significant wave height as a function of the offshore distance.

The wave calculation module goes a step further and also estimates the infragravity wave height. This is done using equation 3.16. This equation relates the infragravity wave height to the total significant wave height at a certain location. The formula was originally developed to estimate the surf beat amplitude, which is the amplitude of the standing wave pattern that occurs due to the interaction of the incoming and reflected wave height. Therefore it is assumed that the reflected long wave is already accounted for in this formula.

In figure 4.17, the result of the undisturbed wave calculation for a $H_{m0,0}$ of 1.25 m, a T_p of 12 s and a ϕ_0 of 210° is displayed. The calculation was performed using a bed slope of 1/100 and a coastal orientation of 185°.



Figure 4.17: Total and infragravity significant wave height as a function of the offshore distance, using: $H_{m0,0} = 1.25$ m, $T_p = 12$ s, a bed slope of 1/100 and an obliqueness at the offshore boundary of 25°. The calculation starts at an offshore distance of $x_{max} = 6,000$ m.

4.4.3 Breakwater effect

The effect of the breakwater is incorporated by means of two processes: transmission and diffraction. This is done only for the offshore distances between $x_{ship,min}$ and $x_{ship,max}$.

Transmission

Transmission is only accounted for at the rubble mound breakwater. This is because the rubble mound breakwater is a porous structure, whereas the caisson breakwater is an imporous concrete block, which means that transmission *through* the breakwater is not possible. Transmission *over* the breakwater due to overtopping is also very unlikely to occur, because the crest height is generally much higher than the wave height under operational conditions, which means that overtopping will be very rare.

A distinction is made between transmission for short and long waves. For short waves the empirical formula developed by Ahrens is used (equations 3.17 and 3.18). It is assumed that the short wave height is equal to the total significant wave height. This is because low frequency wave energy is usually much lower than high frequency wave energy, and the total contribution of the infragravity waves to the significant wave height is usually very small compared to the contribution of short waves. Especially swell waves are assumed to have to most energy along the West-African coast, and are expected to have significant wave heights not much smaller than the total significant wave height.

For long waves a fixed transmission coefficient of 0.8 is applied. This coefficient was based on Hossain's study on infragravity wave transmission through porous structures (section 3.3.4). For both transmission types the wave height at x_{wave} of the rubble mound breakwater is used. Because the breakwater length is not present in the transmission formulas it is assumed that transmission is the same for all four breakwater lengths.

Diffraction

Diffraction coefficients are determined using Goda's diffraction diagram for a semi-infinite breakwater for random swell waves of normal incidence (figure 3.5). The values of K_{diff} are inserted in a table in the wave condition calculation file (as displayed in figures 4.18 and 4.19). The relevant diffraction coefficient is determined as follows:

- The x-distance in the diagram is equal to the alongshore distance between the ship and the left (or right) end of the breakwater (i.e. half the breakwater length as the ship is positioned behind the centreline of the breakwater) multiplied (or divided) by the cosinus of the angle with which the waves approach the breakwater. The obliqueness of the waves has to be taken into account as Goda's diagram is developed for normal wave incidence.
- The y-distance is equal to the onshore end of the breakwater x_{diff} as calculated in the breakwater design tool minus x_{ship} . The value of x_{diff} is different for caisson and rubble mound breakwaters and therefore each breakwater type is treated separately.
- Both distances are divided by the corresponding wave length, resulting in a value for x/L and y/L for each breakwater end.
- Based on these values the diffraction coefficient is obtained from the tabulated diffraction diagram. Linear interpolation is applied when necessary.

The occurring pattern of diffracted waves is assumed to be equal to the superposition of two semi-infinite breakwaters: each semi-infinite breakwater corresponds to one breakwater end. The total diffraction coefficient is therefore equal to the root of the sum of the squared diffracted coefficients from the left and right end of the breakwater.

Diffraction of long waves is calculated in the same manner. Diffracted long waves form a pattern which resembles Goda's diagram between the points (0,0), (2,0), (0,2) and (2,2): the wave height decreases along nearly parallel lines. Long waves have a wave length that is much longer than the wave length of short waves. For example: if a long wave period of 100 s and a water depth of 10 m are assumed, then the wave length is equal to: $L = cT = \sqrt{ghT} = 990m$. Note that the shallow water wave length was used in this calculation because h/L < 1/20. Even if a breakwater of a km length is used, then still both x/L and y/L would be located in the bottom left corner of the bottom middle box of figure 3.5, which resembles the parallel decreasing wave pattern. Therefore a smaller tabulated version of Goda's diagram is used which focuses on this bottom middle box, as can be seen in figure 4.19.

In order to be able to use this table, one needs a wave length for the long waves. The wave length depends on the infragravity wave period. In the input sheet of the cost module a representative infragravity wave period can be inserted by the user. Based on this wave period the wave length which will be used for the calculation of the K_{diff} for long waves can be determined in the same way as has been done above. For this report, a standard infragravity wave period of 100 s is assumed.

		Diffraction diagram										
			Semi-infinite breakwater, directional random waves of normal incidence									
			s_max = 75									
							x/L					
		0	1	2	3	4	5	6	7	8	9	10
y/L	0	0.7	0.22	0.18	0.15	0.12	0.1	0.06	0.04	0.02	0.01	0
	1	0.7	0.3	0.19	0.16	0.12	0.1	0.07	0.05	0.04	0.03	0.02
	2	0.7	0.33	0.2	0.17	0.13	0.11	0.09	0.08	0.05	0.04	0.04
	3	0.69	0.4	0.25	0.19	0.15	0.12	0.1	0.09	0.07	0.05	0.05
	4	0.68	0.43	0.29	0.2	0.17	0.14	0.12	0.1	0.08	0.07	0.06
	5	0.67	0.48	0.3	0.24	0.19	0.15	0.13	0.11	0.1	0.09	0.08
	6	0.67	0.5	0.35	0.27	0.2	0.18	0.15	0.12	0.11	0.09	0.09
	7	0.68	0.53	0.39	0.29	0.23	0.19	0.17	0.14	0.12	0.11	0.1
	8	0.68	0.57	0.4	0.3	0.25	0.2	0.18	0.16	0.14	0.12	0.11
	9	0.68	0.58	0.43	0.33	0.28	0.22	0.19	0.18	0.16	0.14	0.12
	10	0.69	0.6	0.46	0.38	0.29	0.25	0.21	0.19	0.17	0.15	0.13
	11	0.69	0.6	0.49	0.4	0.3	0.26	0.23	0.2	0.18	0.16	0.14
	12	0.69	0.6	0.5	0.4	0.32	0.29	0.25	0.21	0.19	0.18	0.16
	13	0.69	0.6	0.53	0.42	0.35	0.3	0.27	0.22	0.2	0.19	0.17
	14	0.69	0.6	0.54	0.46	0.38	0.31	0.28	0.25	0.21	0.19	0.18
	15	0.69	0.61	0.55	0.48	0.39	0.33	0.29	0.26	0.22	0.2	0.18
	16	0.69	0.62	0.56	0.49	0.4	0.36	0.3	0.28	0.24	0.2	0.19
	17	0.69	0.62	0.56	0.5	0.42	0.37	0.32	0.29	0.26	0.22	0.19
	18	0.69	0.63	0.57	0.5	0.43	0.39	0.33	0.3	0.27	0.24	0.2
	19	0.69	0.63	0.58	0.5	0.44	0.4	0.35	0.31	0.28	0.25	0.22
	20	0.69	0.63	0.58	0.52	0.46	0.42	0.36	0.32	0.29	0.26	0.23

Figure 4.18: Values of K_d as obtained from Goda (2000) and inserted in the design tool, which can be used for the diffraction of short waves.

		x/L								
		0	0.5	1	1.5	2				
y/L	0	0.7	0.4	0.2	0.19	0.18				
	0.5	0.7	0.45	0.25	0.2	0.18				
	1	0.7	0.5	0.3	0.2	0.19				
	1.5	0.7	0.5	0.32	0.23	0.19				
	2	0.7	0.52	0.4	0.3	0.2				

Figure 4.19: Values of K_d as obtained from Goda (2000) and inserted in the design tool, which can be used for the diffraction of long waves.

4.4.4 Downtime prediction

The final step of the wave condition tool is to determine whether downtime occurs or not. A distinction is made between downtime at the berth and downtime due to tugboat unavailability.

Downtime at the berth

In the design tool, downtime at the berth is assumed to occur when the sway, surge and/or heave motions exceed the acceptable limits. Whether the limits are exceeded or not depends on the wave heights at the berth. This wave height is equal to the sum of the transmitted wave (if present) and the diffracted waves from both sides of the breakwater:

$$H_{m0,berth} = \sqrt{H_{m0,diff,1}^2 + H_{m0,diff,2}^2 + H_{m0,trans}^2}$$
(4.30)

In which the subscript diff stands for the diffracted wave from the left and right end of the breakwater (1 and 2) and *trans* stands for the transmitted wave height. This formula is applied for both short and long waves. It is assumed that criteria for sway and heave are exceeded if the short wave height at the berth is larger than the maximum allowable wave height as recommended by Thoresen (2010) (table 3.5): $H_{m0,berth} > H_{m0,max}$. The surge motion criterion is exceeded when the maximum allowable infragravity wave height at the berth is exceeded: $H_{l,m0,berth} > H_{l,m0,max}$. In this way, the three translational modes (sway, heave and surge) have been incorporated in the downtime prediction. The maximum acceptable infragravity wave height is determined using Mol's empirical formula, rewritten as:

$$H_{l,m0,max} = \frac{x_{surge,max}}{C_x} \sqrt{\frac{h \cdot k_p}{g \cdot M}}$$
(4.31)

The following parameters are required for this formula:

- The maximum surge motion $x_{surge,max}$ can be determined using the value recommended by Thoresen (2010), see table 3.4.
- According to Mol, C_x has a value between 1 and 3, with an average of 1.7. The user is free to pick a value. More information on the use of this parameter in the design tool can be found in section 5.4.
- The water depth h is equal to the water depth h_{gd} in the harbour, as determined in the cost module (section 4.2).
- The stiffness of the mooring system parallel to the quay k_p is difficult to estimate. In the design tool, it is assumed that this stiffness is equal to the stiffness of a spring line, as spring lines are meant to limit surge motions. The horizontal angle between the spring line and the quay is neglected. Furthermore, it is assumed that the spring line is linearly elastic, which means that the stiffness in [N/m] can be expressed as:

$$k_t = \frac{EA}{l} = \frac{\frac{1}{4}\pi D^2 E}{l}$$
(4.32)

E being the Young's modulus of the spring line in $[N/m^2]$ (material constant), A the cross sectional area, D the diameter and l the length of the spring line. It must be noted that mooring lines are usually not linearly elastic, which means that this approach is a huge simplification.

• The mass of the vessel is according to Archimedes' principle equal to the displaced water mass. This means that the user has to give the displacement in kg.

The criteria are checked for each x_{ship} , L_{BW} and breakwater type. If one of the wave height criteria is exceeded, operational downtime is expected to occur for that combination of x_{ship} , L_{BW} and breakwater type.

Downtime due to tugboat unavailability

The second cause for downtime is tugboat unavailability. Tugging takes place in unsheltered waters. If the wave height in these waters exceeded a certain limit (given by the user), the tugboats cannot operate and the berthing process cannot start, which means that the approaching vessel has to wait at an anchorage until the wave conditions are milder again.

To determine whether navigational downtime occurs, first it has to be determined how far the tugboats have to sail. Equation 3.4 divides the berthing process into three phases: a deceleration phase L_1 , a tying up phase L_2 and a final stopping phase L_3 . The tugboats are involved in the second and the third phase, which means that they have to travel a distance $L_2 + L_3$ along the approach channel. Taking into account the direction of the approach channel relative to the shore normal $\alpha_{channel}$ and starting from x_{centre} , which is the starting point of the approach channel, this means that the maximum offshore distance where tugboats have to operate $x_{tug,max}$ is equal to:

$$x_{tuq,max} = x_{centre} + (L_2 + L_3) \tan \alpha_{channel} = x_{centre} + (v_{min}t_{tie} + 1.5L_s) \tan \alpha_{channel}$$
(4.33)

In which v_{min} is the speed of the vessel during the tying up of the tugboats (usually 3 to 4 kn) and t_{tie} is the time required to tie up the tugboats (usually 10 min). These parameters are inserted by the user.

Tugboat unavailability occurs if the maximum allowable wave height for tugboats is exceeded somewhere between x_{centre} and $x_{tug,max}$. The wave heights in this region are obtained from the undisturbed wave propagation calculation, as this distance is travelled in unsheltered waters.

4.5 Waiting cost module

In the final module of the design tool the results of the wave calculation of all eight wave conditions are combined and the operational and navigational downtime is expressed in terms of money: the waiting costs. By summing the total waiting costs, the CAPEX and the OPEX up, the total costs for a certain harbour design are obtained.

4.5.1 Input parameters

For the determination of the waiting costs, the following input parameters (in addition to the parameters mentioned at the previous modules) are required:

- The hourly waiting costs in [USD/h]. These are the costs made during downtime, when a ship is not able to enter the harbour, load, unload or leave the harbour due to disadvantageous wave conditions. The waiting costs are very difficult to estimate, as a large number of factors influence the waiting costs (e.g. number and salaries of personnel, amount and value of cargo that has to be loaded, etc.). Experienced senior port engineers at Royal HaskoningDHV estimate that the hourly waiting costs are approximately in the order of 10,000 USD. The large uncertainty and difficulty to estimate this value is one of the reasons why a sensitivity analysis is performed on this variable, which is done is section 6.3.
- The yearly number of vessels that are expected to call at the harbour. This parameter is required to determine the effect of navigational downtime. It is assumed that the interarrival time of the vessels is constant.

• The average service time $t_{service}$ of the vessel. This is the total time a vessel has to spend at the harbour. It is the sum of the berthing, loading / unloading and deberthing time. Based on this service time it can be determined what the maximum number of ships is that can be handled during a wave condition s, simply by dividing the duration of the wave condition t_{wave} over the service time:

$$s = \frac{t_{wave}}{t_{service}} \tag{4.34}$$

4.5.2 Total cost function

In the remainder of this section it is described how an optimum harbour lay-out is recommended based on the total costs.

CAPEX and **OPEX**

The determination of the CAPEX and OPEX of jetty, dredging works, rubble mound and caisson breakwater has been treated in section 4.2 and 4.3. By summing the CAPEX and OPEX functions for all harbour elements up, a total cost function without the incorporation of downtime is obtained. The result is a total of nine cost functions: one for the harbour without a breakwater (i.e. only a jetty and dredging works), four for a harbour with a rubble mound breakwater (with lengths of 1, 2, 3 and 4 L_s) and four for a harbour with a caisson breakwater. In figure 4.22, an example of such a result is displayed. An analysis of these results is given in section 4.6.

Operational downtime

Operational downtime occurs when the maximum allowable short or long wave height at the berth is exceeded. It is assumed that during the entire wave condition loading and unloading operations are impossible. The waiting costs due to operational downtime can be determined by multiplying the hourly waiting costs with the number of hours during which operational downtime occurs. The number of hours during which downtime occurs is equal to the time that there actually is a ship at the berth. This percentage is inserted for each wave condition by the user. This results in the following expression for operational waiting costs WC_{op} for a certain wave condition:

$$WC_{op} = t_{wave} q_{berth} WC_h \tag{4.35}$$

In this equation, t_{wave} is equal to the total duration of the wave condition (determined by the frequency of occurrence as inserted by the user), q_{berth} is the expected percentage of the time that there is a ship at the berth (also inserted by the user for each wave condition) and WC_h are the hourly waiting costs in [USD/h]. The yearly operational waiting costs are the sum of the operational waiting costs for all wave conditions during which downtime at the berth occurs. Moreover, a harbour design is considered unfeasible if operational downtime occurs under all wave conditions inserted. If a harbour is considered unfeasible, the total costs of this harbour lay-out will not be determined.

Navigational downtime

The determination of waiting costs due to navigational downtime is more complicated compared to operational downtime. That is because downtime at the berth only affects one ship (the ship at the berth), whereas navigational downtime affects all vessels calling at the harbour during a certain wave condition. The vessels are not able to enter the harbour and therefore they have to wait at the anchorage, which results in downtime. The number of vessels that arrive during a certain wave condition is equal to the yearly number of ships multiplied with the frequency of occurrence of a wave condition. In order to determine the navigational waiting costs, it has to be determined how many ships have to wait because they cannot enter the harbour. This is done using the scheme in figure 4.20. For each wave condition, the number of ships that arrive n and the number of ships that can be handled s have to be determined using the input. Based on whether downtime at the berth or downtime due to tugboat unavailability occurs, it is determined how many ships cannot enter the harbour during a certain wave condition and have to wait at the anchorage. If tugboats are unavailable, no ships can enter. If tugboats are available but downtime at the berth occurs, one ship can enter the harbour, but it cannot load or unload which means that it has to wait. During the waiting of the ship at the berth, other ships are not able to enter the harbour and have to wait at the anchorage. If the berth is still occupied by a ship because it had to wait during the previous wave condition, no ships are able to enter the harbour if downtime at the berth occurs again during the wave condition considered. If however no downtime at the berth and no downtime due to tugboat unavailability occurs, vessels are able to enter the harbour, load / unload and leave the harbour without having to wait. This means that the number of vessels at the anchorage will decrease. How many ships can be handled during a wave condition with uptime s depends on the service time of a vessel $t_{service}$.

If downtime occurs for two subsequent wave conditions, the number of ships that have to wait at the anchorage adds up and increases. However, a limited number of ships can wait at the anchorage. The maximum allowable number of ships at the anchorage is inserted by the user. If based on the wave conditions defined this number is exceeded for a certain harbour lay-out, this lay-out is considered to be unfeasible and it is not taken into account for the total waiting cost calculation. If a harbour design is feasible however, the waiting costs due to navigational unavailability for a certain wave condition are determined by multiplying the hourly waiting costs with the time during which the ships have to wait at the anchorage and the number of ships that have to wait n_{anch} . The waiting time differs for each vessel as the vessels do not arrive at the harbour at once: some vessels arrive at the beginning of a wave condition and have to wait during the entire period that condition is present, whereas others arrive at the end. Therefore an average waiting time is used, which is estimated to be equal to half the duration of the wave condition:

$$WC_{nav} = 0.5t_{wave} n_{anch} WC_h \tag{4.36}$$

If for two subsequent wave conditions navigational downtime occurs, t_{wave} is set equal to the sum of the duration of both wave conditions. The yearly navigational waiting costs are equal to the sum of the waiting costs for all wave conditions.

Total costs

The total yearly waiting costs WC_{yearly} are equal to the sum of the operational and navigational waiting costs. The CAPEX and OPEX were calculated over a certain calculation period (defined by the user). The same calculation period is used to determine the total waiting costs for the harbour. Because the waiting costs occur each year and take place over a certain period, the changing value of money has to be taken into account. This is done in the same way as for the OPEX:

$$WC_{tot} = \frac{WC_{yearly}t}{(1 + (r/100))^t}$$
(4.37)

The total costs for a harbour lay-out are determined by summing op the total CAPEX, OPEX and waiting costs over the entire calculation period. This is done only for the harbour lay-outs that are considered feasible. An example of such a total cost function is displayed in figure 4.24. The optimum harbour lay-out that is recommended by the design tool is the combination of x_{ship} , L_{BW} and breakwater type that has the lowest total costs with the incorporation of downtime and is still deemed feasible. For the example displayed in figure 4.24, the optimum lay-out can be found at $x_{ship} = 2,100$ m and a caisson breakwater of $2L_s$ length. An more elaborate analysis of the results and the input parameters used to achieve these results can be found in the following section.



Figure 4.20: Determination of the total number of ships that have to wait at the anchorage during a certain wave condition.

4.6 Example run

In order to clarify the use of the design tool, an example run is treated in this section. For the example run a hypothetical offshore harbour in West-Africa is considered. First the parameters that are used as input are described, including a motivation for why these parameters were chosen at these values. After that, the results of the example run are displayed and analysed. The example run is also used as a reference case for the sensitivity analysis in chapter 6.

4.6.1 General input parameters

An optimum lay-out is sought for a hypothetical offshore dry bulk harbour project along the West-African coast. The harbour is only used to load vessels with a certain type of bulk cargo. It is assumed that offshore wave data is present at an measuring post at a distance of 6,000 m off the coast, which means that $x_{max} = 6,000$ m. It was found in section 2.2 that the bed slope along the West-African coast shows large variations, ranging from 1/44.4 at the Ivory Coast to 1/454.5 in Togo. An average valued bed slope of 1/100 is chosen. Initially a step size of $x_{step} = 20$ m is chosen. With this step size the maximum possible number of grid elements of 500 is not exceeded, and results at a large range of offshore distances can be achieved.

It has been tried to simulate a typical West-African coast, based on the data found in appendix A.1. A sandy coast is assumed with a seeward coastal normal making a clockwise angle of 185°

with the North. In appendix A it was found that the wind speed generally does not exceed 9 m/s and mainly comes from the South-West. A wind direction of $225^{\circ}N$ (coming from) is chosen. Furthermore, it was found that the current speed is generally below 0.30 m/s and is mainly West-East directed parallel to the coast, resulting in a direction of $95^{\circ}N$ (going to). The underwater slope of dredged soil is assumed to be equal to 1/3. This is milder than the natural angle of repose for wet sand (45° , corresponding to a slope of 1/1). The orientation channel is chosen in the most occurring wave direction for West-Africa which is equal to $210^{\circ}N$ as was found in appendix A.1. It is expected that waves have the largest potential at causing problems for approaching or leaving vessels since the values for the wind and current speed are relatively low.

The harbour consists of one berth where one ship is able to moor parallel to the coast. This arrangement has been chosen in order to place the ship as close to the breakwater as possible. It is expected that relatively large bulk carriers have to be able to enter the harbour. The dimensions of the design vessel are equal to the dimensions of ship 2 as displayed in table 5.6. Furthermore, the same data with regard to the mooring line length, diameter and stiffness are applied (based on figure 5.14). It is assumed that the manoeuvring characteristics, aids to navigation and the sailing speed in the approach channel are all moderate. A required depth for the approach channel of 17.7 m is found using the PIANC guidelines, whereas the dredging width of the approach channel becomes equal to 175 m.

The rubble mound breakwater is constructed using XBloc armour elements. For the design wave conditions (see below) these armour elements result in the smallest required diameter. Furthermore, because of the steeper breakwater slope $(1:1\frac{1}{3})$ and because only one armour layer is required, the total volume of the breakwater also becomes smaller. An initial estimate for the acceptable overtopping volume for the rubble mound breakwater is 0.05 m³/s. The motivation for this value has been given in section 4.3. It is assumed that a safety zone around the rubble mound breakwater of 0.25 times the bottom width is sufficient for geotechnical stability. The crest width is chosen to be equal to 2 times the freeboard. As for the caisson breakwater is equal to 2 m. Furthermore, it is assumed that the total foundation width is equal to 1.3 times the width of the caisson.

It is assumed that 100 vessels per year will call at the harbour, and that the total service time for one vessel (berthing, loading and deberthing) is equal to 25 hours. For the Liberia dry bulk harbour project, it was found that for a design vessel with similar dimensions, a larger number of calls per year (144) and a short service time (21 hours), one berth was sufficient. Moreover, a maximum of three ships is allowed to wait at the anchorage.

4.6.2 Cost parameters

In section 4.2 several examples for unit cost rates for the jetty, dredging and breakwaters were given, including the resources where these were found. The OPEX percentages were obtained from a cost estimate for a harbour project in Portugal [8]. The unit cost rates and the OPEX percentages applied for the example run are displayed in table 4.8. The OPEX percentage for the caisson breakwater was assumed to be half of the percentage for the rubble mound breakwater, because a monolithic structure like a caisson generally requires less maintenance compared to a rubble mound breakwater that consists of loose elements. The period of the OPEX calculation is equal to 20 years, with an average interest rate of 3 %.

In figure 4.21 the total cost function (without downtime) of a harbour with a caisson breakwater with a length of $2L_s$ is displayed. The total cost function is the sum of three cost functions: the dredging costs, the jetty costs and the breakwater costs. An increase in x_{ship} leads to an increase in jetty costs, since the trestle has to become longer. Furthermore, the unit costs of the trestle also increases because the average water depth increases with an increasing trestle length.

Cost elem	ents	Unit cost	OPEX $\%$ of CAPEX	
	Borth	uc_0	40,000 USD/m	
	Dertii	uc_h	$1,333 \text{ USD/m}^2$	
Jetty	Trostlo	uc_0	10,000 USD/m	0.8
	ffestie	uc_h	10 USD/m^2	
	Conveyor belt	uc	2,400 USD/m	
Pubble mound breekwater	Armour elements		300 USD/m^3	ე
Rubble mound breakwater	Core elements		40 USD/m^3	2
Caisson breakwater	Caisson		350 USD/m^3	1
Caisson breakwater	Foundation		40 USD/m^3	1
Dredging	·		10 USD/m^3	0.3

Table 4.8: Input data for the CAPEX and OPEX calculation.

The breakwater costs also increase, since the water depth in which the breakwater has to be constructed increases. From formulas in section 4.3.3 it can be seen that an increasing water depth causes an increase in breakwater height, which leads to a quadratic increase in required volumes (since the breakwater width also increases), resulting in a more rapid increase in breakwater costs. The dredging costs however decrease with increasing x_{ship} because the available depth is larger, resulting in a smaller dredging depth and lower dredging volumes. Furthermore, the approach channel can also be shorter as the harbour is located closer to x_{dredge} . An optimum for this harbour design can already be found at an x_{ship} of 1,980 m.



Harbour costs with Caisson BW (2Ls)

Figure 4.21: Total CAPEX and OPEX as a function of the offshore distance for a harbour with a caisson breakwater of $2L_s$.

However, harbour designs with other breakwater types and lengths are also possible. Using the same parameters, the nine cost functions as displayed in figure 4.22 are obtained. It can be seen that the solution with the lowest costs is a harbour without a breakwater and a long jetty, to a depth where no dredging is required anymore. This is without the incorporation of downtime however. By taking into account the effect of operational and navigational downtime, it can be determined whether such harbour designs are feasible or not. Moreover, it can be seen that the longer the breakwater, the more onshore the optimum location for the vessel becomes. This is because the breakwater costs become dominant at large breakwater lengths and outweigh the relatively high dredging costs more onshore, causing an onshore shift of the x_{ship} with the lowest costs. In order to incorporate the downtime, the waiting costs are initially estimated to be equal to 10,000 USD.

Furthermore, some transition points can be observed in figure 4.22. These transition points are especially obvious for the options with a rubble mound breakwater. The causes for these transition points are listed below.

- The left transition point (at approximately 1,640 m) is caused by a change in the slope of the rubble mound breakwater cost function. The costs of the rubble mound breakwater depend on the required armour and core volumes, which again depend on the dimensions of the breakwater. As was mentioned in section 4.3, the dimensions of the breakwater depend highly on the design wave conditions. At the transition point, wave breaking takes place under design wave conditions, which causes a slope change between the shoaling zone (where the wave height increases) and the wave breaking zone (where the wave height decreases). Reference is made to figure 4.11 where this transition is clearly displayed. The change in the slope of the wave height function affects the dimensions of the breakwater, which will ultimately affect the total cost function of the harbour. The larger the breakwater costs (compared to jetty and dredging costs), the larger the effect of the transition on the total cost function of the harbour becomes.
- The right transition point (at approximately 1,980 m) is caused by a change in slope of the dredging function. At this point, the average dredging depth for the turning circle becomes zero, which means that the dredging costs will also become zero. In reality some dredging will still be necessary at the edges of the turning circle. However, in the design tool the amount of soil that has to be dredged for the turning circle is calculated based on the depth in the centre of the turning circle z_{centre} (equation 4.21), which at that transition point becomes equal to the minimally required depth h_{gd} , resulting in zero dredging volumes. This causes a change in slope in the dredging cost of the harbour all equal to the sum of the jetty and breakwater costs only, since the dredging costs are equal to zero.

4.6.3 Wave calculation

In appendix A.1 and section 2.2.1 the wave climate along the coast of West-Africa was described. Based on this wave climate, the following input data for the wave calculation are used:

- Waves with a H_{m0} larger than 2.5 m occur less than 1% of the time. The most occuring wave period is 12 s and the most occuring wave direction is 210°. This wave condition is chosen to be the operational wave condition. Based on this wave condition, it was determined that the surf zone approximately starts at 680 m, from which follows that the most onshore distance of the ship for which harbour development is investigated is equal to $x_{ship,min}$ 1,440 m. The largest offshore distance $x_{ship,max}$ is equal to 2,480 m. Beyond this distance no more dredging is required.
- Extreme wave conditions used for the breakwater dimensioning are: $H_{m0} = 10 \text{ m}, T_p = 36 \text{ s and } \phi_0 = 210^{\circ} \text{N}.$

Furthermore, eight dominant wave conditions are defined based on figure 4.23. The dominant wave conditions are chosen as follows. The area of the wave climate table (figure A.1) which represents 90% of the incoming waves is divided into eight sections. For each section the most



Total costs (without downtime)

Figure 4.22: Cost functions for the offshore harbour without the incorporation of downtime.

bottom-right condition is chosen to be representative for that section, since this is the most extreme condition (combination of highest H_{m0} and largest T_p). If uptime is ensured at this wave condition, it can be said that there will be uptime for all other wave conditions in that section as well. If for none of the eight dominant wave conditions downtime occurs, an uptime of at least 90% is guaranteed by the design tool. In Thoresen (2010) it was mentioned that harbours usually strive for a minimum uptime percentage of 90 – 95%. The eight wave conditions are displayed in table 4.9 below. For the wave directions, typical values were obtained from the wave roses in figure A.2. The frequencies of occurrence were determined by summing up the frequencies for all wave conditions in one section, determining what percentage this frequency is of 90% and rounding it off. For all wave conditions, it is assumed that there is a ship at the berth for 50% of the time. Because 100 ships arrive and the inter-arrival time of the ships is constant, the frequency of occurrence is also equal to the number of ships that arrive during a certain wave condition.

The wave conditions are used as input for the wave calculation. The following limiting values were taken as input:

- Thoresen (2010) states that for waves approaching a bulk carrier with a DWT between 30,000 and 100,000 at beam during loading operations, a maximum H_{m0} of 1 m is allowed [35].
- Thoresen (2010) states that the maximum allowable surge motion for dry bulk carriers being handled using a conveyor belt is equal to 5 m [35]. The infragravity wave length is set equal to the standard value of 100 m, and 80 % long wave transmission through rubble

1.7	1.	1.2		0.7	0.	0.2		Hm0		
2	5 1.75	5 1.5	1 1.25	5	5 0.75	5 0.5	0 0.25	(m)		
0	0	0	0	0.00075	0	0	0		4	ω
0	0.0015	0.01225	0.04175	0.00775	0	0	0		5	4
0.02	0.03225	0.0505	0.09225	0.05575	0.0025	0	0		6	5
0.0365	0.10775	0.14625	0.2365	0.267	0.01125	0	0		7	6
0.0365	0.0825	0.094	0.21475	0.19925	0.02625	0	0		8	7
0.02175	0.1015	0.3995	1.05925	0.9975	0.116	0	0		9	8
0.13475	0.77	2.49825	3.4975	2.31025	0.28025	0.00175	0		10	9
0.6485	2.32	3.76625	4.4105	2.5795	0.477	0.00175	0		11	10
1.14	2.7405	4.41825	5.80025	3.07775	0.3325	0	0		12	11
1.4105	2.98925	4.62425	5.11625	2.2435	0.1785	0	0		13	12
1.71175	2.998	4.13175	4.20225	1.58525	0.14775	0	0		14	13
1.3185	2.13125	2.74625	2.74475	1.0175	0.0685	0	0		15	14
0.5555	0.823	1.09375	0.9095	0.32	0.034	0	0		16	15

₽

s

conditions that were used as input for the design tool were chosen. Figure 4.23: Wave conditions that occur 90% along the West-African coast. The thick black lines indicate the sections based on which the wave

Wave condition	H_{m0} [m]	T_p [s]	$\phi_0 \ [^\circ N]$	Frequency
Wave condition 1	1.25	12	210	30
Wave condition 2	1.25	14	200	10
Wave condition 3	1.25	16	210	10
Wave condition 4	2	12	190	3
Wave condition 5	1.75	12	200	20
Wave condition 6	1.75	16	210	5
Wave condition 7	1.5	12	190	20
Wave condition 8	2	16	180	2

Table 4.9: Wave conditions used as input for the reference case.

mound breakwaters is assumed. Moreover, the coefficient of Mol is assumed to be equal to the average value of 1.7. Whether this is a reliable value is examined in section 5.4.

• In the tug curves of figure 3.3, it can be seen that above a wave height of 2 m, much of the tug effectiveness is lost. Especially pushing tugs have practically no bollard pull beyond this wave height. Therefore the maximum allowable wave height for tugboats is set equal to 2 m. The vessel speed during the tying up of the tugboats is set to 2 m/s, and it is assumed that the tying up takes 10 minutes [21].

Other criteria for the maximum wave height at the berth and for tugboats also exist. These will be evaluated in section 6.3 and 6.4.

4.6.4 Results

The total cost function for the example run are displayed in figure 4.24. The minimum costs are achieved at an x_{ship} of 2,100 m, using a caisson breakwater with a breakwater length of 2 times the design vessel length. The costs for this harbour design are equal to 234 million USD. With this harbour design, no downtime at the berth occurs, whereas the ships are not able to enter the harbour only 2% of the time (during wave condition 8). Other observations that can be made:

- Based on the defined criteria the harbour lay-out has to fulfill, the design tool has determined that a harbour lay-out without breakwater or with a breakwater with a length of L_s is not feasible for all x_{ship} . Using a breakwater length of L_s apparently means that the diffracted wave is still too large under most wave conditions, which means that downtime due to excessive vessel motions will occur. An increased breakwater length leads to smaller diffracted waves, thus reducing the probability of downtime.
- The remaining breakwater types and lengths are only feasible starting from an offshore distance of 1,480 m. The wave conditions closer to the coast are generally disadvantageous, mainly due to the effect of shoaling which causes an increase in the significant wave height, which leads to too large wave heights at the berth and during the tugging journey. Moreover, the infragravity wave height also increases with decreasing x_{ship} , which means that downtime due to surge motions is more likely to occur. Especially the tugboat criterion seemed dominant in the example case. Further onshore from $x_{ship} = 1,480$ m, the maximum allowable wave height for tugboats is exceeded for a too long period of time (wave condition 4, 6 and 7), which means that too many ships will have to wait at the anchorage.
- All cost functions show little variation over the offshore distance x_{ship} . Furthermore, all cost functions show a small drop at a x_{ship} of 2,100 m. For all harbour lay-outs, downtime due to tugboat unavailability occurs only during wave condition 8 beyond $x_{ship} = 2,100$

m. More onshore of this distance, downtime occurs for more wave conditions, which leads to an increase in waiting costs. Since the downtime criterion is quite abrupt (downtime occurs immediately when the maximum allowable H_{m0} is exceeded), a sudden drop in the waiting costs can be seen. The magnitude of this drop depends largely on the value of the waiting costs inserted, as will be explained more thoroughly in section 6.2.

• It can be seen that with the current value for the acceptable overtopping discharge, a rubble mound breakwater is always more expensive than a caisson breakwater of the same length.



Total costs harbour (with downtime)

Figure 4.24: Total cost functions for the reference case.

4.7 Conclusions

In this chapter the structure of the design tool was described. The design tool is constructed in Excel as it is a user-friendly program that does not require any licences. The design tool consists of four modules:

- The cost module is the most important module as all input parameters and final results are stored here. The cost module determines the jetty and dredging costs. The jetty costs are found to increase with the offshore distance due to the larger dimensions required, whereas the dredging costs decrease due to the increase depth. Furthermore, it determines the boundaries between which offshore harbour development will be investigated.
- The breakwater module designs the rubble mound and caisson breakwater based on the design wave conditions. The dimensions are used to determine the breakwater costs and are used as input for the wave calculation module.
- The wave calculation module calculates the wave heights at the berth and during the tugboat journey for a series of offshore wave conditions, using linear wave theory combined

with several empirical formulas. Based on the wave heights it is determined whether operational and navigational downtime occurs.

• The waiting cost module determines the waiting costs based on the downtime figures obtained from the wave calculation module.

The CAPEX, OPEX and waiting costs are summed up for all nine harbour alternatives. Based on the alternative with the minimum costs after incorporation of downtime that is still feasible, a recommendation for x_{ship} , L_{BW} and breakwater type are given.

An example run showed that the total costs for the jetty and breakwater generally increase with increasing x_{ship} , due to the larger dimensions required at large distances off the coast (and large water depths). The dredging costs generally decrease since the dredging depth decreases with an increasing available water depth. Moreover, the length of the approach channel can also be smaller if the distance off the coast increases. An increasing L_{BW} logically leads to an increase in total costs. Furthermore, an onshore shift in the optimum solution (without downtime) can be seen because at larger x_{ship} the breakwater costs become dominant for larger L_{BW} .

For the downtime calculation it was observed that the effect of shoaling has a large influence on the value of x_{ship} that has to be chosen. For small offshore distances, the effect of shoaling becomes so large that the wave height exceeds acceptable values at the berth and along the tugging journey. Furthermore, the low frequency wave height is also higher close to the coast, which results in excessive surge motions. The length of the breakwater mainly influences the downtime via diffraction: a larger L_{BW} results in a smaller diffracted wave height and more sheltering at the berth, reducing the probability of downtime.

Chapter 5

Verification of the design tool

In the previous chapter the structure and model formulations of the design tool were described. In this chapter it is examined whether these formulations actually lead to a reliable result that can be compared to results obtained from sophisticated modelling software, such as wave modelling programs and DMA software. Three verifications are performed. The first section of this chapter motivates the choice for these three verifications. The following sections describe the methodology and results of these verifications. Conclusions are stated in the last section of this chapter.

5.1 Introduction

The design tool introduces a large number of simplifying assumptions and approximations in order to reduce model complexity. However, these simplifications may reduce the reliability and accuracy of the calculations performed, and therefore it is necessary to check whether the design tool still gives valid and realistic results that may be used for port planning purposes. Therefore three different verifications are performed.

- The first verification is performed on the undisturbed wave calculation, which was described in section 4.4.2. During this calculation, the development of the significant wave height H_{m0} from the offshore boundary to the coastline is determined using a combination of linear wave theory and Goda's empirical formulas for wave height estimation in the surf zone. Furthermore, the development of the infragravity wave height as a function of the offshore distance is also determined using Goda (2000). During this calculation several processes are taken into account (e.g. shoaling, refraction), whereas other processes are neglected (e.g. bottom friction). The result of this calculation is compared to a 1D SWASH calculation performed using the exact same wave condition parameters. This comparison is made for various wave conditions and bottom steepnesses that are characteristic for the West-African coast, to check whether the design tool performs better or worse depending on the inserted parameters. This verification is necessary because it serves as an input for the following wave calculation and eventually determines whether downtime occurs under a certain condition or not.
- The second verification is performed on the wave penetration calculation, as described in section 4.4.3. The wave height that reaches the ship is not equal to the undisturbed wave height. The sheltering effect of the breakwater has to be taken into account. By taking into account transmission and diffraction coefficients, the design tool attemps to estimate the short and long wave height behind the breakwater, at the ship. Again, this is a simplified approach that does not take into account a number of phenomena such as depth-induced effects on the wave height, reflections off the breakwater, etc. The verification is performed by comparing, for a certain offshore harbour project for which a wave penetration study

has been performed using a wave modelling program, the calculated wave height behind the breakwater with the wave height obtained from the design tool when the same project parameters are inserted. This verification is performed for various breakwater types and wave conditions. It serves as an input for the actual downtime determination, which is done based on the short and long wave height.

• The third and last verification is performed on the downtime determination. Based on the short and long wave height at the berth, the design tool determines whether downtime occurs or not. This is done based on Thoresen's (2010) guidelines for maximum allowable short wave height at the berth and surge motions. As has been mentioned in 3.2.1, a lot of factors that influence the mooring behaviour of the ship (added mass, damping, nonlinear stiffness of the mooring lines, etc.) are not taken into account during this procedure. The downtime determination is therefore compared to the results of an actual DMA performed for a certain offshore port project, in which for a number of wave conditions it has been determined whether or not downtime occurs. By inserting the same project parameters and wave conditions in the design tool, a comparison can be made and conclusions can be drawn on the reliability of the downtime determination.

It can be seen that all verifications are related to the wave calculation module. This is because the wave calculation plays a crucial role for the final result of the design tool. Based on the wave heights outside and inside the harbour, the design tool draws conclusions on whether a certain harbour design is feasible or not. Even if a harbour design might be attractive due to low capital and operational expenditures, disadvantageous wave conditions at the harbour may again make a certain harbour design completely unprofitable due to operational difficulties. Thus, an accurate description of short and long wave action by the design tool is highly necessary to obtain reliable results. Therefore it was chosen to focus on the wave calculation module rather than the cost module.

Furthermore, it has also been chosen to not verify the final result of the design tool (i.e. the recommendation for offshore distance of the berth, breakwater length and type). This is because for the final choice of these parameters, other factors (besides CAPEX, OPEX and downtime) may also play a role. One can think of environmental considerations or factors related to the actual construction of the harbour (e.g. staged development, constructional risks). Furthermore, by comparing the final result of the design tool to the actual solution chosen for a certain offshore harbour project, one automatically assumes that that solution is *the* optimum solution, which does not necessarily have to be the case.

5.2 Verification 1: undisturbed wave propagation

In this section the first verification is discussed. First the methodology and the motivation for this methodology are presented. After that the data that will be used as input for the verification will be discussed, followed by results and conclusions.

5.2.1 Methodology

It has been chosen to use the non-hydrostatic multi-layer wave model SWASH for the first verification. This choice has been made because SWASH is able to accurately model both short and long wave propagation, taking into account complex nonlinear processes, resulting in a large accuracy. Furthermore, SWASH is not limited to a certain bottom steepness or water depth, which is the case for Boussinesq and mild slope models. This means that SWASH can be used to model the waves for a large range of wave conditions and bottom steepnesses. More information on SWASH and other wave modelling programs can be found in appendix C.1.

The undisturbed wave propagation as calculated by the design tool takes place on an alongshore

uniform coastal area with a constant bed slope. Because there are no alongshore differences, a 1D SWASH calculation is considered to be sufficient for this verification. This has one disadvantage: a 1D calculation means that obliquely incident waves cannot be modelled, which means that the effect of refraction cannot be examined.

SWASH is used to produce, based on the input parameters that will be discussed below, a time series of the surface elevation over a certain calculation period, at a fixed number of output locations of various offshore distances. An example of such a time series is displayed in figure 5.1.



Figure 5.1: Example of a time series of the surface elevation as obtained from SWASH. Input parameters: $H_{m0} = 1.25m$, $T_p = 12s$, bed slope 1/100, water depth 50m.

These time series are post-processed using a MATLAB script that is able to create a variance density spectrum E(f) of the time series. An example of such a spectrum is displayed in figure 5.2. Using the variance density spectrum, the total significant wave height H_{m0} can be found using the zero-order moment of the variance density spectrum m_0 : $H_{m0} = 4\sqrt{m_0}$. The zero-order moment is found by integrating over the variance density spectrum:

$$m_0 = \int_0^\infty E(f) df \tag{5.1}$$

The infragravity wave height $H_{l,m0}$ is found in a similar manner, only in this case only the zero-order moment in the low-frequency range is used. The low-frequency range is defined as all frequencies smaller than $f = \frac{1}{25}$ Hz (i.e. $T_p > 25$ s). This results in the following expression for the infragravity wave height:

$$H_{l,m0} = 4\sqrt{m_{l,0}} = 4\sqrt{\int_0^{\frac{1}{25}} E(f)df}$$
(5.2)

By performing this calculation for a series of offshore distances, the significant wave height and infragravity waves height are obtained as a function of the offshore distance. This function is compared with the function obtained from the design tool (figure 4.17). For each output point of the SWASH calculation, the relative difference with the values obtained from the design tool is determined. For the significant wave height, a relative difference of 15% is deemed acceptable. For infragravity waves however, the maximum acceptable relative difference is set equal to 30%. This is because infragravity waves are usually much smaller in height, which means that an absolute difference of a few centimeters might already result in a large relative difference. Based on the observed differences, conclusions can be drawn on the reliability of the wave calculation.

5.2.2 Input parameters

Wave conditions and bed slope

Because the design tool is mainly aimed to be used for harbour projects along the West-African



Figure 5.2: Example of a variance density spectrum obtained from the MATLAB postprocessing. Input parameters: $H_{m0} = 1.25m$, $T_p = 12s$, bed slope 1/100, water depth 50m.

coast, it has been chosen to verify the design tool using wave conditions characteristic to the West-African coast. In figure A.1 the H_{m0} , T_p -diagram for the West-African coast was presented. It was found that this coast is characterised by waves of low to moderate heights (95% of the waves under 2 m height) and long periods (94% of the waves have peak periods between 8 and 16 s). This is characteristic for swell wave climates. The wave condition that occurs the most has a H_{m0} between 1 and 1.25 m and a peak period of 11 to 12 s. Therefore the wave condition with $H_{m0} = 1.25$ m and $T_p = 12$ s is taken as the base wave condition.

In addition to the base wave condition, other wave conditions might also be interesting to verify. For example, the ability of the design tool to predict the nearshore wave propagation might vary from one wave height to another. Therefore it has been chosen to verify the design tool also at a larger wave height: $H_{m0} = 2.5m$ and $T_p = 12s$. Waves larger than 2.5 m occur less than 1% of the time and are therefore less relevant for the West-African coast. Furthermore, the influence of the peak period on the design tool reliability is also interesting to verify: how accurate is the design tool in predicting the total and infragravity significant wave height for waves with shorter and longer periods? To answer this question, two other wave conditions are verified: $H_{m0} = 1.25m$, $T_p = 8s$ and $H_{m0} = 1.25m$, $T_p = 16s$. Finally, the design tool also calculates the nearshore wave propagation under design conditions, in order to determine the wave height that has to be taken into account for the design of the breakwaters. The long-term wave analysis of appendix A.1 has led to the conclusion that a H_{m0} of approximately 10 m and a T_p of 36 s has a return period of 5000 years, which is equivalent to a failure percentage of 1% during a breakwater lifetime of 50 years. This leads to a total of five wave conditions that will be used for the verification of the design tool. It must be noted that for the last wave condition, only the total significant wave height is relevant for the verification, because the infragravity wave height under extreme conditions is not used for breakwater design.

Moreover, the bed slope also has a significant influence on the behaviour of waves. The bed slope especially influences the process of shoaling, wave breaking and wave reflection, as was mentioned in section 3.3.2. Different bed slopes therefore result in a different behaviour of waves in the coastal zone. It was mentioned in section 2.2.1 that the West-African coast does show some variation in bottom steepness, which means that the design tool has to be able to give accurate results at several steepnesses. Therefore all five wave conditions are verified on three different steepnesses: 1/30 (relatively steep), 1/100 and 1/300 (relatively mild). This results in a total number of 15 SWASH calculations, which are displayed in the table below.

Wave condition	Steep slope $(1/30)$	Average slope $(1/100)$	Mild slope $(1/300)$
Wave condition 1		$H_{m0} = 1.25m; T_p = 12s$	
Wave condition 2		$H_{m0} = 2.5m; T_p = 12s$	
Wave condition 3		$H_{m0} = 1.25m T_p = 16s$	
Wave condition 4		$H_{m0} = 1.25m; T_p = 8s$	
Wave condition 5		$H_{m0} = 10m; T_p = 36s$	

Table 5.1: Wave conditions and bed slopes that will be used for the SWASH verification.

Other input parameters

Besides the wave parameters, the following model settings were applied:

- A grid size of 1 m was applied for all model runs. By using a sufficiently small grid size, the occurrence of numerical dissipation is avoided. Two vertical calculation layers are usually sufficient to accurately model the waves.
- Considering the distance of the offshore boundary, it has been tried to find a balance between computational efficiency and relevance for verification. Computational efficiency in the sense that no errors occur and that the simulation time is limited. Relevance for verification in the sense that only water depths that are interesting for offshore harbour development are investigated. All simulations for the average slope are carried out to a depth of 60 m (offshore boundary x_{max} located at 6,000 m). The mild slope simulations are carried out to a depth of 20 m ($x_{max} = 6,000m$), except for wave condition 5 which is extended to a depth of 60 m ($x_{max} = 18,000m$). The steep slope simulations are carried out to a depth of 20 m ($x_{max} = 3,000m$), except for wave condition 5 which is extended to a depth of 100 m ($x_{max} = 3,000m$).
- Goda (2000) states that for engineering applications a swell spectrum may be approximated using a JONSWAP spectrum with a peak enhancement factor γ between 3 and 10. He gives as an example the Pacific coast of Costa Rica, on which swell waves that have travelled over a distance of more than 9000 km across the Pacific propagate. A γ of 8 to 9 seemed as a suitable value to model these waves [12]. Because the swell waves that reach the West-African coast also originate from storms that occur at large distances off the coast, in the Southern Atlantic Ocean, it is assumed that similar values for this γ can be applied for West-Africa. Therefore γ is set equal to a value of 8.
- Olagnon (2014) states that swell waves along the West-African coast have an angular standard deviation of 18° [28]. This value is therefore chosen for the directional spreading.
- The SWASH manual states that for a suitable simulation 500 to 1000 waves are required. Furthermore, one has to take into account that the first 10 to 15 % of the simulation time is lost to spin up. Three different wave periods are used for wave modelling: 12 s, 16 s and 36 s. For the simulations with $T_p = 12s$, a simulation period of 2 hours is sufficient. For simulation with $T_p = 16s$, a period of 3 hours is used. Finally, for an extreme peak period

of $T_p = 36s$, a simulation period of 6.5 hours is used. This is deemed sufficient based on the following calculation: 6.5 hours = 23,400 s. Assume 15% is lost to spin up, then 19,890 s remain. In this simulation period one can fit 552.5 waves with a period of 36 s, which lies in the acceptable range of 500 to 1000 waves required for a suitable simulation. The first 15% of the simulation time is not taken into account during post-processing. The cycle time is set equal to the remaining 85% of the simulation time. In that way, all simulated waves appear at least once during the period that is used for the post-processing of the surface elevation time series. The initial conditions are all set equal to zero.

• Default values for wave breaking and bottom friction are used. This results in a Manning formula for bottom friction with a friction coefficient of $0.019m^{1/3}/s$. Furthermore, SWASH uses a standard formula to model wave breaking, in the form of:

$$\frac{\partial \zeta}{\partial t} > \alpha \sqrt{gh} \tag{5.3}$$

$$\frac{\partial \zeta}{\partial t} > \beta \sqrt{gh} \tag{5.4}$$

With $\alpha > \beta$. The first inequality means that if the vertical velocity of the water surface exceeds a certain fraction of the phase velocity, wave breaking is initiated. The second inequality represents persistence of wave breaking: if wave breaking has been initiated, it will not stop until the left inequality is no longer valid. SWASH uses default values of $\alpha = 0.6$ and $\beta = 0.3$.

• When simulating the extreme wave conditions (wave condition 5), errors may occur close to the coastline because the large waves may cause the water level to fall too far below the bottom level. This error only occured at the simulations with a steep slope. Therefore a sponge layer is used for these simulations at the onshore boundary. A sponge layer absorbs wave energy at an open boundary. The main aim of the wave simulation is to verify the reliability of the design tool in determining the wave height *in areas that are interesting for offshore harbour development*. This is generally at some distance off the coast (outside the surf zone), which means that the accuracy in determining the wave height close to the coast is not relevant, which makes the use of a sponge layer justifiable. The sponge layer usually has a width of 3 to 5 times the typical wave length. For the steep slope simulation, a sponge layer with a width of 1,000 m is used.

All other model parameters are set to default values.

As for the design tool, only the following input parameters are relevant: H_{m0} , T_p , x_{max} $(x_{min} = 0)$ and the bed slope. Furthermore, the angle between the incident wave at the offshore boundary and the shore normal is set equal to zero. All other input parameters have no influence whatsoever on the result of this verification.

5.2.3 Results and discussion

The input parameters as presented above are inserted in both SWASH and the design tool. The results are compared at a fixed number of output locations and the wave heights are plotted as a function of the offshore distance. Furthermore, the relative difference between the SWASH and design tool results are computed using:

$$error = \frac{H_{m0,SWASH} - H_{m0,dt}}{H_{m0,dt}} \cdot 100\%$$
(5.5)

In which the subscript SWASH stands for the value of the wave height obtained from the SWASH calculation, whereas the subscribt dt stands for the value obtained from the design tool. This is done for both the total significant wave height H_{m0} and the significant infragravity

wave height $H_{l,m0}$. Tabulated results of this computation are displayed in appendix B.1. A red cell indicates that the maximum allowable value for the relative difference (15% and 30% for total and infragravity wave height respectively) is exceeded. A positive value indicates that the design tool underestimates the wave height, whereas a negative value indicates an overestimation. In figure 5.3 to 5.5, graphical results for wave condition 1 are displayed for the three different bed slopes.

The following observations can be made:

- The relative error for the total significant wave height is much smaller than for the infragravity wave height. In nearly all simulations, the number of output locations where the acceptable relative error is exceeded is larger for the infragravity waves. This is probably because the infragravity wave height estimation in the design tool is completely based on an empirical formula, whereas for the total significant wave height, a combination of empirical formulas and mathematically derived expressions is used. In most cases H_{m0} is slightly overestimated, whereas the infragravity wave height is generally underestimated.
- The error tends to become larger for both the total and the low frequency wave height as the waves approach the coast. Especially the infragravity wave height close to the shore is seriously underestimated by the design tool. Apparently the empirical formula used underestimates the amount of infragravity wave energy that reflects off the coast. The total significant wave height also shows larger errors near the coast, because in the design tool wave breaking takes place closer to the shore, which means that the wave are allowed to shoal to larger wave heights. However, the errors made close to the shore are less relevant for the design tool, as the design tool always makes sure that the harbour is designed outside the surf zone.
- The infragravity wave height is largely underestimated for large wave heights (wave condition 2). At moderate wave heights, the error is generally within acceptable limits (with exception of the surf zone as mentioned at the previous bullet). The error also tends to be larger for short wave periods (wave condition 4). Peak periods from 12 to 16 s give better results. Long swell periods and moderate wave heights dominate along the West-African coast.
- The wave height estimation under extreme conditions (wave condition 5) is generally within acceptable limits. The maximum errors can again be found in the surf zone, which is, as mentioned before, not interesting for offshore harbour development.
- The average slope has the smallest overall error. The steep slope produces comparable results when it comes to the total H_{m0} , and has a slightly larger error when it comes to the infragravity waves. The mild slope has the largest errors, especially when it comes to infragravity waves. West-Africa is dominated by coasts with low steepnesses.

Generally it can be said that for the offshore distances of interest the design tool gives reliable results when it comes to the total significant wave height H_{m0} . The reliability of the estimate for the undisturbed infragravity wave height depends much on the case considered. For large wave heights (wave condition 2) the error becomes unacceptably large, which means the design tool is not accurate enough. Large wave heights (order 2.5 m) do not occur that much on the West-African coast, which means that this drawback does not have serious consequences on the reliability of the design tool. The reliability for moderate wave heights (wave condition 1, 2 and 4) depends much on the bed slope and wave period considered. For the mild West-African slopes (slope 3, 1/300) it can generally be said that the design tool gives much better (acceptable) results for long peak periods (wave condition 3) and unacceptable results for shorter values of T_p (wave condition 1 and 4). A similar trend can be seen at the other two (steeper) values for the bed slope, and the accuracy is even larger in the latter two cases. However, it was found that

steep slopes are rare along the West-African coast, which means that these results might not be that relevant for West-Africa.

One has to take into account that the comparison was made based on an alongshore uniform coast with a constant bed slope. In reality such coastal zones are very rare, and usually alongshore differences in bed profile can be found. This will result in 2D and 3D effects that cannot be taken into account by the design tool, which will probably increase the error. Furthermore, in the design tool one value for the bed slope can be inserted, whereas in reality the bed slope varies in both alongshore and cross-shore direction. This will also result in a larger error. The next verification will be performed on a coast with a less uniform and more irregular alongshore and cross-shore profile.



Figure 5.3: Nearshore wave propagation as determined by the design tool and SWASH, for wave condition 1 on a slope of 1/100.

Mild slope, wave condition 1



Figure 5.4: Nearshore wave propagation as determined by the design tool and SWASH, for wave condition 1 on a slope of 1/300.
Steep slope, wave condition 1



Figure 5.5: Nearshore wave propagation as determined by the design tool and SWASH, for wave condition 1 on a slope of 1/30.

5.3 Verification 2: wave penetration

In the previous section the wave calculation without the presence of a breakwater was verified. The next step in the design tool is the incorporation of the breakwater and determining the wave height at the berth, in the area behind the breakwater. This calculation is verified in this section by comparing the total and low frequency wave height at the berth as determined by the design tool with the result of an actual wave penetration study, performed for an actual harbour project using wave modelling software. First some general data about the project that will be used for the verification is given, including the method that was originally used to determine the wave penetration. The wave penetration study has been performed by Royal HaskoningDHV, and the information and original results presented have been obtained from the wave penetration report for this harbour [34]. After that, the verification method will be explained and the parameters that will be used as input for the design tool are presented, followed by the results of the verification.

5.3.1 Project information

General

An offshore harbour for the export of phosphates is being constructed at some location along the African coast. The harbour is located **outside** the area defined as 'the West-African coast' earlier in this study. The berth facility of the harbour is connected to the coast by means of a 3 km long trestle. The berth facility is protected from wave action by means of an L-shaped breakwater. Three vessels are able to moor in the harbour, and an approach channel and (parts of) a turning circle are being dredged to a depth of -19.6 m + MSL in order to make sure that the vessels can safely reach the berths. The preliminary general lay-out of the port, including the bathymetry and dimensions, can be seen in the figure 5.6 [34]. The lay-out of the harbour generally resembles the lay-out of the harbour used for the design tool, which makes it therefore suitable to be used for the verification.

A wave penetration study is performed in order to determine operational wave conditions in the port area and additionally to identify the presence of long waves inside the port area that may impact ship operations. Previous to the wave penetration study, a nearshore wave climate had been established using SWAN. SWAN is not suitable to model the sheltering effect of the



Figure 5.6: Preliminary general lay-out of the offshore phosphates export harbour. [34]

breakwater, nor can it accurately model wave reflection from structures. Furthermore, infragravity waves cannot be modelled using SWAN. Therefore a MIKE21 Boussinesq wave model is used to perform the wave penetration study. MIKE21 BW is able to handle nonlinearities such as infragravity waves and nonlinear wave-wave interactions. Furthermore, wave transmission and reflection at the boundaries of the harbour (breakwaters, quays and revetments) can be included. More information on SWAN and MIKE21 BW can be found in appendix C.1. The results of the wave penetration study will eventually be used as input for a DMA. The calculated pressures and flow velocities in the area around the moored vessels can be exported to a HAR-BERTH model, which determines the wave forces acting on the ship [34]. For more information about DMAs, reference is made to appendix C.3.

MIKE21 BW model

The first step in the MIKE21 BW calculation was to determine the governing wave conditions. Offshore wave data for a period of 11 years was obtained from BMT ARGOSS, at a large distance offshore. By using SWAN the offshore conditions were transformed to nearshore conditions, at location 1038 and 1012 as displayed in figure 5.7 below. These are the boundaries of the MIKE21 BW model. Based on the obtained data, wave roses and H_{m0} , T_p -tables can be made to obtain insight in the prevailing wave climate. Because the harbour is not located along the West-African coast, some differences exist between the wave climate at the harbour and the wave climate described in 2.2.1:

- The waves predominantly come from the North, whereas the coast has a North-South orientation. This means that offshore, the direction of wave propagation is nearly parallel to the coast. This is also one of the reasons why an L-shaped breakwater is being constructed.
- The waves are generally a bit larger in height $(H_{m0} = 2.75m \text{ exceeded } 1\% \text{ of the time})$ and shorter in period $(T_p = 7 - 8s \text{ most occurring peak period}).$

Now that the wave climate is known, a number of wave conditions for the wave penetration study is selected. This was done based on a rule of thumb: wave conditions are likely to cause downtime if the following inequality applies [34]:

$$H_{m0} > \frac{22}{T_p} \tag{5.6}$$

Based on this rule of thumb it was found that wave conditions with peak periods between 10 and 14 s are most likely to cause downtime, with $T_p = 12s$ being the most dominant period. The following wave conditions were selected for the penetration study:

Wave condition	H_{m0} (m)	T_p (s)	Direction ($^{\circ}N$)
Wave condition 1	1.75	12	340
Wave condition 2	1.75	12	315
Wave condition 3	1.75	12	285
Wave condition 4	1.75	8	355

Table 5.2: Selected wave conditions for the wave peentration study. [34]



Figure 5.7: Extent of the MIKE21 BW model and wave boundary locations.

Furthermore, several breakwater types were examined during the wave penetration study. Two of these alternatives will be used for the verification of the design tool. Alternative 1 is a caisson breakwater and alternative 2 is a rubble mound breakwater. Both breakwater types have the same dimensions and have standalone berthing facilities at the leeside of the quay. The vessels are moored parallel to the coast. Cross-sections of the breakwaters are displayed in figure 5.8 and 5.9. The dimensions of the breakwater and the positions of the berths are displayed in figure 5.10. It must be noted that in the MIKE21 BW model, zero wave transmission is assumed for both breakwater types [34].

Running the model leads to graphical results as displayed in figure 5.11. The average of the wave height at the three berths for all four wave conditions and two breakwater alternatives is displayed in table 5.3. These values will be used to verify the design tool. The total significant wave height behind the breakwater decreases to a value of 0.40 - 0.60 m for wave condition 1, 2 and 3, whereas wave condition 4 results in wave heights of 0.20 - 0.30 m for the same offshore wave height (1.75 m). The 12 s swells cause relatively large infragravity wave heights in the harbour, in the order of 0.20 to 0.25 m. This is equal to 12.5% to 25% of the total wave energy. The 8 s swell causes less low frequency wave energy to reach the vessels (0.05 to 0.10 m, approximately 5 to 10 % of the wave energy). Furthermore, the wave direction does not seem to have a large effect on the low-frequency wave height.



CAISSON SECTION (25m WIDE)

Figure 5.8: Caisson breakwater used for alternative 1. ZH is the Lowest Astronomical Tide and is 1.49 m below Mean Sea Level. [34]

	Alternative 1 (caisson)		Alternative 2 (rubble mound	
Wave condition	H_{m0} (m)	$H_{l,m0}$ (m)	H_{m0} (m)	$H_{l,m0}$ (m)
Wave condition 1	0.44	0.18	0.35	0.17
Wave condition 2	0.59	0.22	0.42	0.20
Wave condition 3	0.50	0.24	0.35	0.19
Wave condition 4	0.26	0.07	0.17	0.07

Table 5.3: Results of the MIKE21 BW calculation. The wave heights displayed are the averages of the wave heights at the three berth.

5.3.2 Input parameters and methodology

The methodology for the second verification is quite simple: the project parameters for the offshore phosphates harbour are inserted in the design tool and the resulting significant and infragravity wave height behind the breakwater are compared to the values displayed in table 5.3. The comparison is made based on the relative error, which is defined in the same way as in verification 1:

$$error = \frac{H_{m0,MIKE21BW} - H_{m0,dt}}{H_{m0,dt}} \cdot 100\%$$
(5.7)

In which the subscript MIKE21BW stands for the wave height found from the MIKE21 BW calculation. Because both H_{m0} and $H_{l,m0}$ are in the order of decimeters, the maximum allowable error for both wave types is set equal to 20%.

The wave conditions in table 5.2 are used as input for the verification. It must be noted that the design tool only determines the wave height at the location of the berth behind the breakwater. As mentioned in chapter 2, in the offshore harbour as designed by the design tool only one ship is able to moor. The wave heights in 5.3 however are the average of the three wave heights at the three berths in the harbour.

The following parameters are used as input for the design tool.



Figure 5.9: Rubble mound breakwater used for alternative 2. ZH is the Lowest Astronomical Tide and is 1.49 m below Mean Sea Level. [34]



Figure 5.10: Dimensions of the breakwater and positions of the berth behind the breakwater. [34]

- The model area used for the MIKE 21BW calculation has a size of 11,150 m × 9,000 m. Because the design tool performs a 1D calculation, the offshore boundary of the computational domain is set equal to $x_{max} = 10,000m$. The onshore boundary is located at the coast $(x_{min} = 0m)$. A step size of $x_{step} = 100m$ is used.
- Based on the bathymetric map in figure 5.12, it can be determined that the shore normal makes a clockwise angle of 292° with the North. By drawing the line normal to the coastline at the location where the trestle reaches the coast to the offshore boundary of the computational area, the average bed slope can be determined by dividing the total vertical distance along this line over the total horizontal line. This results in a relatively mild bed slope of 1/206.
- The choice for x_{step} was made as follows. From figure 5.6 it becomes clear that the offshore distance of two of the three berths is equal to 3,210 m. The third berth is located slightly more onshore. Using the bed slope as determined in the previous bullet point, this would mean that the berths are approximately located between the 15 and 16 m depth contour. However, from figure 5.6 it becomes obvious that the berths are located between the 17



Figure 5.11: Result of the MIKE21 BW calculation. Blue indicates low values for the wave height, red indicates high values. The left image shows the total significant wave height, the right image shows the infragravity wave height. [34]

and 18 m depth contour. The depth plays an important role in the wave calculation, as processes such as shoaling and refraction are largely influenced by the water depth. It is attempted to find a balance between the offshore distance and the prevailing depth that has to be applied. Therefore a slightly larger offshore distance of the berth is applied: $x_{step} = 3,500m$. This distance corresponds with a water depth of 17 m.

- The design tool is only able to model straight breakwaters. The breakwater length is set equal to 890 m (the sum of the two sections of the breakwater as displayed in figure 5.10). Instead of letting the design tool determine the dimensions of the breakwater, the dimensions are inserted manually where possible. For the rubble mound breakwater, this results in a crest width of 21 m, a total height of 28 m, and a double tetrapod layer in the breakwater armour with a slope of 1:1.5. For the caisson breakwater, this results in a width and height of 25 m and a sill height of 2 m. The crown wall on top of the caisson breakwater is not taken into account in the calculation, as the design tool is only able to model caissons with a completely square cross-section. Because the ships are located directly behind the breakwater, no safety margins are taken into account.
- The transmission coefficients for both short and long waves are set equal to zero. Waves are able to penetrate into the harbour only by means of diffraction. This means that the distance between the ship and the breakwater is an important parameter, because the diffraction coefficient is a function of the ratio of the distance of the ship behind the breakwater divided by the wave length, according to the Goda's diffraction diagram that is used in the design tool. The orientation of the ship is set parallel to the coast. Furthermore, based on figure 5.10, it is estimated that the *average* distance between the ship and the foot of the breakwater is equal to 90 m (= 270/3). Due to the sloping surface of the rubble mound breakwater, this will lead to a slightly larger distance for the rubble mound breakwater than for the caisson breakwater.

All other input parameters (e.g. unit cost rates) have no influence whatsoever on the result of this verification and are therefore not relevant for this calculation.

5.3.3 Results and discussion

The results of the second verification are displayed in table 5.4 and 5.5. The relative difference between the MIKE21 BW and the design tool value of the wave height is determined using equation 5.7. The following observations can be made:

• The design tool gives a much smaller difference in wave height between alternative 1 and 2 compared to the MIKE21 BW calculation. This is because the reflection of waves off the



Figure 5.12: Bathymetric map of the coastal zone.

breakwater is also taken into account in the MIKE21 BW calculation, whereas this is not done in the design tool. The reflection coefficient depends on the period of the incident waves and the steepness of the reflecting structure, as can be seen in figure 5.13. The caisson breakwater has a vertical surface which is fully reflective (the red line), resulting in a reflection coefficient of 1. The rubble mound breakwater however has a sloping surface which results in a reflection coefficient of smaller than 1. Due to the angular shape of the breakwater and wave reflection, a standing wave pattern may develop in the harbour. The resulting wave height will be larger for the caisson breakwater since it is fully reflective. In the design tool, the only difference in wave height (if any) is caused by the difference in the ship's position relative to the breakwater. When a caisson breakwater is used, the ship can be located closer to the breakwater, which should result in slightly smaller waves. From the results in table 5.4 and 5.5 it can be seen that this difference is negligible.

- The error for the significant wave height estimate is generally under the maximum allowable error for the rubble mound breakwater. For the caisson breakwater however, the error is much larger, mainly due to the reflection effects that were described in the previous bullet point.
- The low frequency wave height is largely underestimated by the design tool. This can have two causes. The first cause is that the infragravity wave height at the seeward side of the breakwater (i.e. that diffracts into the harbour) is underestimated. It was found in the first verification that for mild slopes and relatively high wave heights (as is the case for this harbour), the infragravity wave height is underestimated. The second cause may lie in the way infragravity wave penetration into the harbour is modelled by the design tool. The infragravity wave height at a certain offshore location is the sum of two waves: an incoming wave and a wave that has reflected off the coast. As one approaches the coast, the reflected long wave becomes larger in height. When a detached breakwater is constructed at a certain offshore location, only the *incoming* wave is blocked by the breakwater, and makes its way to the berth by means of transmission (neglected here) and



Figure 5.13: Wave reflection coefficient as a function of steepness of the slope and the incident wave period for plain, smooth and straight slopes.

diffraction. The *reflected* wave however is not influenced by the breakwater, because it can freely propagate from the coast towards the berth. Thus, in order to correctly model the long wave penetration, transmission and diffraction coefficients only need to be applied to the incoming wave. The distinction in incoming and reflected waves is not made in the design tool, and transmission and diffraction coefficients are applied to the *total* low frequency wave height, including the reflected wave.

One has to remember that differences between the harbour lay-out that was modelled in MIKE21 BW and the lay-out that is used in the design tool also may cause differences in output. Some of these differences:

- The breakwater in the design tool is a straight breakwater parallel to the coastline, whereas in reality it has an angular shape. Furthermore, the long section of the breakwater is not exactly parallel to the coast. The difference in breakwater shape and orientation may significantly diffraction of waves, especially diffraction from the eastern end of the breakwater (at the end of the short section). Furthermore, the angular shape may increase the effect of wave transmission for the caisson breakwater.
- The bed slope is not constant, but the bottom is slightly steeper near the coast. This may influence the degree in which waves shoal and break. Moreover, alongshore differences may influence refraction of incident waves.
- The wave heights that are compared to each other also differ in the way they are determined. The design tool takes the wave height at some point behind the centreline of the breakwater (which is assumed straight). In the wave penetration study however, the wave height is defined as the average of the wave heights at the three berths.

		Alternative 1 (caisson)			
Wave condition	H_{m0} (m)	error (%)	$H_{l,m0}$ (m)	error (%)	
Wave condition 1	0.36	22.2	0.13	38.5	
Wave condition 2	0.40	47.5	0.14	57.1	
Wave condition 3	0.40	25	0.14	71.4	
Wave condition 4	0.21	23.8	0.07	0	

Table 5.4: Values for the wave height behind the caisson breakwater, obtained from the design tool.

	Al	Alternative 2 (rubble mound)			
Wave condition	H_{m0} (m)	error (%)	$H_{l,m0}$ (m)	error (%)	
Wave condition 1	0.36	-2.8	0.13	30.8	
Wave condition 2	0.40	5.0	0.14	42.9	
Wave condition 3	0.41	-14.6	0.14	35.7	
Wave condition 4	0.21	23.8	0.07	0	

Table 5.5: Values for the wave height behind the rubble mound breakwater, obtained from the design tool.

5.4 Verification 3: downtime prediction

After the wave penetration calculation of the design tool has been verified, the final verification is performed: the downtime prediction. Based on the wave height at the berth, the design tool predicts whether or not downtime occurs at a certain x_{ship} and L_{BW} , given the offshore wave conditions. In this section it is verified whether this estimate is reliable. This is done by comparing the result of the downtime prediction with the result of an actual DMA, performed for an actual harbour project using mooring analysis software. Just like in section 5.3, first some general information on the port project and the DMA that has been performed is given. After that, the methodology and parameters that are used as input for the verification are presented and motivated, followed by a discussion of the results. Again, the project information and DMA results were obtained from the DMA report as issued by Royal HaskoningDHV [37].

5.4.1 Project information

General

An offshore LNG terminal is at some location along the West-African coast (as has been defined earlier this study). The lay-out of the harbour is displayed in figure 5.14. Again there is a long exposed trestle that connects the berthing facility to the coastline. The berthing facility provides space for three ships to berth: an LPG vessel at berth 1 and an LNG vessel at berths 2 and 3. The detached breakwater has a total length of 1,900 m and consists of two sections of 1,250 and 650 m respectively. All berths are located relatively close behind the breakwater. Moreover, an approach channel is dredged to a depth of -15.5 m CD. The harbour basin is dredged to a depth of -14 m CD [37]. Even though the harbour is a liquid bulk harbour, its shape still resembles the shape of the harbour for which the design tool is developed. Furthermore, the type of cargo does not influence the downtime prediction, in the sense that none of the formulas or parameters involved in the calculation are influenced by the type of cargo. The type of cargo is more likely to influence the CAPEX and OPEX calculation as different harbour facilities may be required for an LNG terminal. The only parameters that have to be adapted are the maximum allowable H_{m0} and surge motion, as different criteria apply for liquid bulk vessels. Therefore this harbour project is considered to be suitable to verify the design tool with.



Figure 5.14: Lay-out of the offshore OKLNG harbour. [37]

A dynamic mooring analysis is performed to determine the limiting environmental conditions at which a vessel can still stay at berth. Once these limiting conditions have been established, the downtime of the berth can be determined. Furthermore, the maximum vessel motions that are obtained at these limiting conditions will be used to design the required loading arms at the berth. The focus of the DMA was on the effect of long waves on the mooring line forces and motions of the moored vessels, because it was expected that the most important loads on the vessels behind the breakwater are due to long waves. For the DMA a stepwise approach is adopted in which the strength of several numerical models is utilised. The first model in the chain is SWAN, which is used to transform the offshore wave climate at deep water to a nearshore wave climate. The next step is to determine the infragravity wave height at the boundary by means of Delft3D-Surfbeat. The wave forces on the moored ship are determined using lf-strip, which models the diffraction of the waves around the ship hull. Finally, ship motions are simulated using TERMSIM, resulting in the mooring line loads [37]. More information on the functions and possibilities of the mentioned modelling software can be found in appendix C.

Dynamic Mooring Analysis

The offshore wave climate at the location of the harbour are very similar to the typical climate described in 2.2.1. The dominant wave direction is the South-West, and waves are usually not higher than approximately 2.4 m and mainly have periods between 8 and 16 s, with 10-12 s being the most dominant range of peak periods. Of the three berths located in the harbour, only berth 2 is of interest for the verification, because it is located approximately behind the centreline of the breakwater. At berth 2 two ships have to be able to moor, of which the relevant information is displayed in table 5.6. For both ships a DMA is performed in ballasted conditions. Both ships are moored using 44 mm diameter steelite xtra superlines, of which the load/extension characteristics are displayed in figure 5.15 [37]. For the DMA a worked steelite

xtra line is taken, which has a linear relationship between load and extension. The verification is performed for both ships in order to investigate whether the design tool performs differently on different ship dimensions. However, it must be noted that the difference in ship dimensions are not that large, which makes it logically to expect that differences in performance will be small.



Figure 5.15: Load/extension characteristics for 44 mm steelite xtra superlines. [37]

	Ship 1: LNGC4	Ship 2: LNGC5a
Capacity $[m^3]$	145,000	217,000
Displacement (ballasted) [t]	105,000	117,000
LOA [m]	289.5	315
Beam [m]	49	50
Draft (ballasted) [m]	9.4	10

Table 5.6: Vessel particulars. [37]

Available short and long wave measurements have been used as input and calibration data. A long wave transmission coefficient of 30% is used for the calculation. This coefficient was chosen based on physical model tests at HR Wallingford. The long wave forces on the vessel were computed based on this transmission coefficient [37].

By systematically varying the environmental conditions in the DMA, a set of limiting environmental conditions has been determined. At these offshore wave conditions, the infragravity wave height in the harbour becomes so high that the forces in the mooring lines reach 55% of the Minimum Breaking Load (MBL) of the lines. The limiting conditions were displayed as a set of maximum allowable wave heights at a certain range of wave periods. During the DMA, all energy was assumed to be concentrated in the swell peak of the energy density spectrum. This has been done mainly because low-frequency wave energy emerging from the sea spectrum alone is hardly present, and the nonlinear effects occurring from the swell waves are much larger, which means that swell waves lead to much larger infragravity waves. This means that the limiting wave heights as established by the DMA only concern the swell wave height. In order to find the limiting total significant wave height, about 0.20 m has to be added to the swell wave height. The limiting total significant wave heights for several wave periods can be found in table 5.7 [37].

Peak period T_p	8 - 10 s	10 - 12 s	12 - 14 s	14 - 16 s
Ship 1	2.6 m	2.6 m	2.2 m	2.0 m
Ship 2	2.6 m	2.6 m	2.2 m	2.0 m

Table 5.7: Limiting H_{m0} for the vessels at berth 2. The wave direction at the offshore boundary is $195^{\circ}N$. [37]

After the first DMA, a sensitivity analysis was performed to investigate the effect of several parameters on the mooring behaviour of the vessel. One of the parameters varied was the length of the trestle. In the initial situation, the harbour is located at the -9 m + CD depth contour. It was investigated whether the mooring behaviour would improve if the berthing facility was located at the -11 m + CD depth contour (i.e. further off the coast, longer trestle). The long wave heights were significantly lower at this depth contour, which had a positive effect on the mooring line loads. The limiting wave conditions in the new solution are displayed in table 5.8. It can be seen that only for waves with $T_p = 14 - 16s$ the mooring behaviour has improved [37]. Because the main purpose of the design tool is to investigate differences between several offshore locations of the berth and optimize this location, it might be interesting to verify the DMA results at a larger offshore distance as well.

Peak period T_p	8 - 10 s	10 - 12 s	12 - 14 s	14 - 16 s
Ship 1	2.6 m	2.6 m	2.2 m	2.2 m
Ship 2	2.6 m	2.6 m	2.2 m	2.2 m

Table 5.8: Limiting H_{m0} for the vessels at berth 2 in case a longer trestle is applied (harbour located at -11 m + CD). The wave direction at the offshore boundary is 195°N. [37]

5.4.2 Input parameters and methodology

The final verification is performed as follows: the relevant project parameters (described below) are inserted in the design tool. For a set of wave periods ranging from 8 to 16 s, the offshore significant wave height is increased with steps of 0.10 m until downtime occurs at the x_{ship} where the harbour is located. These wave periods are chosen because almost 95% of the waves along the West-African coast have peak periods in this peak period range, making these periods the most relevant to verify. The first H_{m0} at which downtime occurs is denoted as the limiting H_{m0} for a certain wave period. The verification is done only for berth 2, for the two types of ships described earlier this section, and at two different values for x_{ship} : at the -9 m + CD and at the -11 m + CD depth contour. It must be noted that the design tool determines whether or not downtime occurs based on the maximum allowable H_{m0} at the berth and the maximum allowable surge motion. The DMA determines the limiting wave conditions based on the occurring mooring lines forces, which are not allowed to exceed 55% of the MBL. Nevertheless, the DMA report states that the surge motions are dominant in determining the results of the mooring analysis. The following input parameters are used:

• The onshore boundary of the design tool calculation x_{min} is again set at the coastline $(x_{min} = 0m)$. The bathymetry used for the SURFBEAT model is displayed in figure 5.16. Based on this figure the bed slope and the offshore boundary of the wave calculation are determined. A line perpendicular to the coastline (where the water depth is equal to zero) is drawn from the onshore to the offshore boundary through berth 2. The distance along this line is measured and is approximately equal to 9,850 m, which means that

 $x_{max} = 9,850m$. The water depth at the offshore boundary is equal to 15.5 m, which means that the average bed slope is equal to 0.157% = 1/637. A step size x_{step} of 50 m is chosen. Using the bed slope, the offshore position of the berth can be found. The first verification is performed for the berth at the -9 m + CD depth contour, which corresponds to an offshore distance x_{ship} of 5750 m. The second verification, for -11 m + CD, is performed at $x_{ship} = 7000$ m. Furthermore, the coastal normal makes a clockwise angle of 202° with the North.

- Dimensions of the ships as displayed in table 5.6 are used. The ships are moored parallel to the coast. In order to reduce the distance between the ship and the breakwater, the safety area around the breakwater is set equal to 0 m. The water depth at the berth is equal to 14 m. The rubble mound breakwater is assumed to be completely straight and has a total length of 1,900 m, and has a long wave transmission coefficient of 0.30.
- According to Thoresen (2010), the maximum allowable surge motion for gas tankers is equal to 2 m [35].
- According to the DMA report, the minimum breaking load of 44 mm steelite xtra superlines is equal to 1314 kN. From figure 5.15 it can be seen that this corresponds to an extension of 2.1%. Using linear elastic theory, the following value for the mooring line stiffness (Young's modulus E) can be found:

$$E = \frac{N}{A\epsilon} = \frac{1314 \cdot 10^3}{\frac{1}{4}\pi \cdot 0.044^2 \cdot 2.1 \cdot 10^{-2}} = 4.12 \cdot 10^{10} N/m^2$$
(5.8)

Based on the applied mooring arrangement (figure 5.17), it is estimated that for both ships the length of a spring line is equal to 45 m. The length and the Young's modulus of the spring line are necessary to determine the stiffness of a spring line, which is used as input parameter to determine the maximum allowable long wave height using Mol's rule of thumb (equation 3.3). The spring line stiffness is equal to:

$$k_p = \frac{EA}{l} = 443,128.9N/m \tag{5.9}$$

Two concluding remarks on the methodology are made:

• The focus of the DMA was on the effect of infragravity waves on the mooring behaviour of the vessel at the berth. This implies that one has already assumed that short waves do not significantly impact the mooring behaviour and are not able to cause downtime. This assumption was checked using the design tool. The transmission of short waves is influenced by the dimensions of the breakwater (see Ahrend's expression for the transmission coefficient in equation 3.17 and 3.18). As an initial estimate, the breakwater dimensions (surface area and stone size) were determined using the design tool because no data about the breakwater is given in the DMA report. A maximum overtopping volume of 0.05 $m^3/s/m$ was assumed in the estimate of the breakwater dimensions. Furthermore, an armour consisting of Cube elements seemed to be the most critical with regard to transmission (highest transmission coefficients, mainly due to the large diameter required). Indeed it was found that with a breakwater length of $L_{BW} = 1,900m$, the values for the diffraction coefficient for short waves are so small that the sum of the diffracted and transmitted total significant wave height never exceeds the maximum allowable value for H_{m0} , even if a relatively strict criterion of $H_{m0,max} = 1.5m$ is applied (This criterion corresponds to tankers larger than 30,000 DWT and waves approaching the vessel head-on [35].). Therefore it can be said that short wave action at the berth can be neglected, and only long wave action has to be taken into account. In order to determine long wave



Figure 5.16: Bathymetry used for the SURFBEAT model.

penetration into the harbour, only the length of the breakwater $(L_{BW} = 1,900 \text{ m})$ and the transmission coefficient (0.30) are relevant (according to the model formulations of the design tool).

• An important factor of uncertainty is the Mol coefficient C_x . This is an important coefficient as it determines the maximum allowable infragravity wave height at the berth: the larger C_x , the lower the infragravity wave height is allowed to be. Mol states that this coefficient ranges approximately between 1 and 3, with an average of 1.7 [25]. However, he does not give a recommendation for when to use which value. Therefore initially a value of $C_x = 1.7$ is assumed. Using Mol's empirical formula, this results in a maximum allowable low frequency wave height of 9.1 cm for ship 1 and 8.6 cm for ship 2. This seems quite conservative, keeping in mind that infragravity waves usually start to cause problems between a $H_{l,m0}$ of 10 to 15 cm. Therefore the value of C_x is varied in order to find out which value of C_x gives the most reliable results.

All other input parameters of the design tool have no influence whatsoever on the downtime prediction and are therefore not relevant.

5.4.3 Results and discussion

The first set of results is obtained using the average value of C_x of 1.7. Again the analysis is made based on the relative error, defined as:

$$error = \frac{H_{m0,DMA} - H_{m0,dt}}{H_{m0,dt}} \cdot 100\%$$
(5.10)

In which $H_{m0,DMA}$ is the value for the limiting offshore wave height at a certain peak period as determined by the DMA. Errors smaller than 15% are deemed acceptable. The results for $C_x = 1.7$ are displayed in the tables B.4 and B.5 in appendix B.2.

It can be seen that for C_x the design tool largely underestimates the limiting wave condition and predicts downtime at much smaller offshore values of H_{m0} . Especially for larger wave



Figure 5.17: Mooring arrangement of a 145,000 m^3 LNG vessel at berth 2.

periods, the design tool predicts downtime at values of H_{m0} equal to nearly half the DMA values. Therefore smaller values of C_x (i.e. less conservative criteria for maximum allowable infragravity wave height) are also examined. The values and corresponding limiting values for $H_{l,m0}$ are displayed in table 5.9.

C_x	1.0	1.1	1.2	1.3	1.4	1.5	1.6
Ship 1	15.5	14.1	12.9	11.9	11.1	10.3	9.7
Ship 2	14.7	13.4	12.3	11.3	10.5	9.8	9.2

Table 5.9: Maximum allowable values of the infragravity wave height at the berth in cm.

The verification is done again using the new values for C_x . The results are displayed in tables B.6 to B.9. Based on the obtained values, the following conclusions can be drawn:

- The larger the peak period, the smaller the value for C_x has to be in order to obtain results similar to the DMA output. It can generally be said a value of $C_x = 1.0$ gives results that are comparable to the DMA for wave periods between 11 and 14 s. Limiting wave heights for wave heights larger than 14 s are generally underestimated by all values of C_x . For wave periods below 11 s, larger values of C_x have to be used. In the previous section however it was found that the low frequency wave height at the berth is generally underestimated, which means that if the downtime criterion based on Mol's formula would be applied on the actual (larger) infragravity wave height at the berth, the limiting value for H_{m0} would become ever smaller, resulting in an underestimation of the limiting offshore wave conditions.
- The difference between the limiting wave conditions for the two ships are extremely small for both the design tool and the real DMA. This is mainly due to the fact that both ships have similar dimensions and that the difference in displaced volumes of water is not that large, which means that the difference in the infragravity wave height criteria for both ships are also not that large.
- Similar results are obtained for both offshore distances verified (at -9 m and -11 m + CD). The limiting wave height values for the longer trestle are slightly higher because the low frequency wave height further off the coast is generally lower.

Of course one has to keep in mind that the error made in the downtime prediction not only depends on the value of C_x (and thus the criterion for the maximum allowable $H_{l,m0}$), but more importantly depends on the wave height at the berth, which on its turn depends on the incoming offshore wave height. Errors in the previous two calculation steps (undisturbed wave

propagation and wave penetration) will pile up in the last calculation step. Furthermore, one has to take into account that the offshore harbour designed by the design tool is different from the actual LNG harbour in many aspects. The angular shape of the breakwater, the location of the ship behind the breakwater, the curved shape of the coastline, the position of the ship relative to the coastline (not exactly parallel), variations in bed slope and alongshore differences in the coastal profile are all factors that may contribute to the final error.

5.5 Conclusions

The design tool was verified in three different ways. The results of these verifications were presented and discussed in this chapter.

In the first verification the undisturbed wave propagation as calculated by the design tool was compared to a SWASH calculation for five different offshore wave conditions and three different bed slopes. It was found that for depths which are interesting for the development of offshore harbours, the results for the total wave propagation as calculated by the design tool are generally comparable to the results of a 1D SWASH calculation. The reliability of the infragravity wave calculation depends largely on the case considered: large H_{m0} , mild bed slopes and short T_p generally lead to unacceptable results, whereas moderate H_{m0} , average bed slopes and long T_p lead to reliable results. The error in both the total and the infragravity wave height increases close to the coastline.

The wave penetration calculation was verified in the third section by comparing the total and the low frequency significant wave height at the berth behind the breakwater as determined by the design tool with the wave heights found using a MIKE21 BW model for a certain offshore harbour project. The low frequency wave height was largely underestimated by the design tool, mainly because the tool does not distinguish between incoming and reflected long waves. Furthermore, the total significant wave height was underestimated for caisson breakwater because wave reflection off the structure is not taken into account by the design tool.

The downtime prediction was verified in the fourth section, by comparing the limiting wave conditions for a certain offshore harbour project as found by the design tool with the limiting wave conditions found by performing a DMA. It was found that the parameter C_x has a large influence on the results, and that in this specific case the design tool produces results that are comparable with the DMA when a value of $C_x = 1.0$ is used for peak periods between 11 and 14 s. For the second and third verification one has to bear in mind that differences in the actual harbour lay-out and the harbour lay-out as designed by the design tool may also cause errors.

Chapter 6

Sensitivity analysis

A large number of parameters is used as input for the design tool. Some input parameters influence the output of the design tool more than others. Uncertainties in these influential input parameters may have a large impact on the final result of the design tool, depending on which parameter is being considered and what model formulations are applied to that parameter. It is therefore necessary to realize what effect uncertainties in these model formulations may have on the result of the design tool. In order to acquire a good understanding of these effects, a sensitivity analysis is performed. The first section of this chapter describes the methodology for this analysis and the parameters that are chosen for the sensitivity analysis. The following sections discuss the results of the sensitivity analysis, performed for various parameters. The final section contains a summary of the most important conclusions of this chapter.

6.1 Introduction

6.1.1 Motivation for the sensitivity analysis

In this chapter the sensitivity of the output of the design tool to uncertainties in the model input is investigated. The design tool is in essence a deterministic model, in the sense that for each input parameter one deterministic value is inserted. No standard deviations or probability distributions are used to describe the input parameters. However, defining one fixed value for a parameter which in reality may show some variation might be too much of a simplistic approach. For example: the maximum allowable significant wave height at the berth might vary depending on the exact wave period, the exact angle in which the waves approach the vessel, the mooring configuration, the size of the ship and other factors. The impact that these uncertainties may have depends mainly on the model formulations of the design tool and how the input parameters are processed in order to obtain output. Because the design tool is meant to be used in an early design stage, relatively simple model formulations are applied, in which complex processes (such as the mooring behaviour of a vessel) are described using simplified rules of thumb. The small number of input parameters required for these simple model formulations means that a deviation in the input value may have a relatively large impact on the output of the design tool. Based on the value of some input parameters, complete harbour designs can be considered unfeasible. The large effect that certain input parameters may have on the output justifies the need for a sensitivity analysis, that results in a qualitative and quantitative description of these effects.

6.1.2 Methodology and input parameters

The sensitivity analysis is performed using the one-factor-at-a-time method. Initially a reference case is defined in which baseline values for the input parameters are inserted. For a certain parameter the input values are changed, while all other input parameters are kept at their baseline values. The effect of the change in this input parameter on the output of the design tool

is assessed. In that way, the sensitivity of the design tool to that parameter can be investigated. It has been chosen to perform the sensitivity analysis only to the final output of the design tool, i.e. the total cost function (CAPEX, OPEX and waiting costs) for all nine harbour designs, plus the recommendation for the optimum harbour lay-out. Intermediate results may be used to clarify certain observations, but are not the main aim of this analysis because the design tool is essentially aimed at comparing several lay-out alternatives and recommending an optimum offshore harbour lay-out.

The following parameters have been chosen for the sensitivity analysis:

- The hourly waiting costs. This parameter has been chosen because it can be considered a weight that is given to the downtime criterion. The final recommendation for the harbour lay-out is given based on the combination of two criteria: the CAPEX and OPEX on one hand and the downtime caused by wave action on the other. The larger the waiting costs, the more weight is given to the downtime criterion, which means that the choice for a harbour will be determined much more based on the estimated downtime, and less based on the calculated capital and operational expenditures. It can thus be expected that the waiting costs will have a significant impact on the final result. The waiting costs however are difficult to estimate on forehand, which means that a large uncertainty is present in this input parameter.
- The maximum allowable wave height at the berth. From Thoresen (2010) it becomes clear that this wave height depends on a large number of factors, such as the direction in which the waves approach the ship (head-on or at beam), the wave period, the size of the vessel, whether a vessel is being loaded or unloaded, etc. Furthermore, the effect of the wave height on a moored vessel is not as abrupt as has been modelled in the design tool, as loading or unloading operations gradually decrease in efficiency with increasing wave height rather than being ceased when a certain wave height is exceeded. It can thus be seen that large uncertainties are accompanied with the use of this input parameter. The maximum allowable H_{m0} at the berth however has a large impact on the result, as based on this criterion it is determined whether or not downtime occurs, influencing the feasibility and attractiveness of a certain harbour design.
- The maximum allowable wave height for tugboats. Tugging is a complex process and the performance of a tugboat depends on a large number of factors. The maximum allowable wave height is influenced by the wave period, whether the tugboat pushes or pulls, whether the tugboat pulls directly or only provides indirect assistance, and whether a static or dynamic winch is applied. Furthermore, the tugboat efficiency also decreases gradually with increasing wave height, as does not directly drop to zero when a certain wave height is exceeded. It is therefore difficult to assume one value above which vessels are not able to enter the harbour. The tugboat criterion also has a large influence on the final result of the design tool, because based on this criterion it is determined whether or not a certain harbour design is feasible. Therefore this parameter is also considered to be suited for a sensitivity analysis.
- The maximum discharge of water that is allowed to overtop over the rubble mound breakwater. In the design tool this parameter is used to determine the dimensions of the rubble mound breakwater. A larger allowable discharge generally results in a lower required freeboard, which influences the vertical dimension of the breakwater. As the total costs of the breakwater are proportional to the breakwater height squared, it can be seen that a lower overtopping volume will lead to a decrease in required stone and armour volumes, causing a decrease in rubble mound breakwater costs. This might influence the final result of the design tool, as the use of a rubble mound breakwater may become more attractive compared to a caisson breakwater. Thus, the overopping volume does not influence the

feasibility of a harbour design as much as it influences the attractiveness of a design with a rubble mound breakwater compared to a design using a caisson breakwater. However, the accepted overtopping volume is difficult to estimate as it depends on a large number of factors, such as demands for harbour tranquility, leeward slope stability, potential use of the breakwater crest, etc.

• The step size used in the design tool. This parameter mainly influences the userfriendliness of the design tool, and does not so much influence the feasibility of attractiveness of the harbour designs involved. A small step size results in a large quantity of data that has to be analysed and a large number of harbour designs that have to be compared to each other. However, a port engineer might want to enlargen the step size in order to reduce the amount of data that has to be analysed and compare a limited number of offshore distances, resulting in a more rapid assessment. The main question in this matter is how the larger step size influences the model output, and whether financially more attractive alternatives for the harbour lay-out are not omitted when a coarse grid is applied.

6.1.3 Analysis 0: Reference case

The example run described in section 4.6 will be considered the reference case for this sensitivity analysis. The parameters used in the example run are the baseline values. By changing the values of the parameters mentioned above, the sensitivity of the model output to deviations in the model input can be observed and examined. The baseline values for the sensitivity analysis are displayed in table 6.1. In chapter 4 a motivation for these values has been presented.

Parameter	Baseline value
Hourly waiting costs	10,000 USD
Maximum H_{m0} at berth	1.0 m
Maximum H_{m0} for tugboats	2.0 m
Maximum overtopping volume	$0.05 \text{ m}^3/\text{s/m}$
Step size	20 m

Table 6.1: Baseline values for the sensitivity analysis.

6.2 Analysis 1: Waiting costs

In the first analysis the effect of the hourly waiting costs is investigated. As mentioned before, it is difficult to estimate the waiting costs due to the large number of parameters that influence these costs. Therefore the focus is not so much on the actual value of the waiting costs, but more on the effect of an increase or decrease of the costs on the model output. Three hypothetical values for the waiting cost are chosen, as displayed in table 6.2.

Case	Name	Hourly waiting costs [USD]
0	Reference	10,000
1a	Low weight	1,000
1b	Medium weight	5,000
1c	High weight	50,000

Table 6.2: Design tool runs for analysis 1.

The results are displayed in the figure 6.1 to 6.3. For all design tool runs, the minimum costs can be found using a caisson breakwater with a length of $2L_s$. The total cost function without the incorporation of downtime (only CAPEX and OPEX) is displayed in figure 4.21.

The minimum of this function can be found at an x_{ship} of 1,980 m. The following remarks can be made:

- For run 1a, the minimum costs are equal to 204 million USD and can be found at an x_{ship} of 1,860 m, using a caisson breakwater of $2L_s$. The optimum is reached at the same value for x_{ship} as for the cost function without downtime. For this harbour lay-out, ships are not able to enter the harbour during two wave conditions (5% of the time), whereas in the reference case only wave condition 8 (2%) caused tugboat unavailability. Due to the lower hourly waiting costs, the downtime percentage is less crucial in determining the final result, and the influence of the CAPEX and OPEX computation becomes larger.
- For run 1b and 1c, the minimum costs are equal to 224 million USD and 314 million USD respectively, and can in both cases be found at an x_{ship} of 2,100 m. It can be seen that the downtime criterion becomes dominant again, and that x_{ship} with a larger downtime figures (either at the berth or at the entrance) become less attractive. This is especially the case for run 1c, where a large drop in total costs can be seen at 2,100 m. The downtime due to tugboat unavailability decreases here with 3%, because vessels are able to enter the harbour during wave condition 4. A large difference in costs is created and a yearly downtime of 3% less results in a drop in costs of almost 200 million USD over the entire calculation period.

From this analysis it becomes clear that the hourly wave costs mainly influence the relative attractiveness of several harbour lay-outs and the way in which several alternatives are compared to each other.





Figure 6.1: Total cost function for design tool run 1a (hourly waiting costs 1,000 USD).



Figure 6.2: Total cost function for design tool run 1b (hourly waiting costs 5,000 USD).



Figure 6.3: Total cost function for design tool run 1c (hourly waiting costs 50,000 USD).

6.3 Analysis 2: Maximum allowable wave height at berth

The second analysis is aimed to assess the effect of the maximum allowable wave height at the berth. Thoresen (2010) recommends various values as maximum acceptable H_{m0} at the berth. For bulk cargo with a dead weight tonnage of 30,000 to 100,000, the limiting wave height at the berth is between 0.80 and 1.50 m, depending on the wave direction (head-on / stern-on or at beam) and whether loading or unloading takes place [35]. This is quite a large range, and the criterion is used quite strictly in the sense that downtime is assumed as soon as the criterion is exceeded, without a gradual decrease in productivity (as displayed in figure 3.2). In table 6.3 the values used for the second sensitivity analysis are displayed. It can be seen that these are the boundaries as given by Thoresen (2010).

Case	Name	Maximum H_{m0} at berth [m]
0	Reference	1.0
2a	Lower boundary	0.8
2b	Upper boundary	1.5

Table 6.3: Design tool runs for analysis 2.

The cost functions are displayed in figures 6.4 and 6.5. The following observations can be made.

- For run 2a, the optimum solution remains the same as for the reference case. Apparently the caisson breakwater protects the berth so well, that even a more strict criterion for the maximum allowable H_{m0} is not exceeded. The designs with a rubble mound breakwater however are significantly influenced. For the option with a rubble mound breakwater with a length of $2L_s$, downtime occurs at the berth for wave condition 6. This means that a vessel can only enter the harbour, but cannot be loaded, which means that the vessel has to wait at the berth. In the meanwhile, four other vessels also arrive at the harbour, and because the berth is already taken, these vessels have to wait at the anchorage. The maximum amount of vessels that is allowed to wait at the anchorage is exceeded for all x_{ship} which means that all lay-outs with a rubble mound breakwater of $2L_s$ become unfeasible. Moreover, when a rubble mound breakwater with a length of 3 or $4L_s$ is used, the harbour remains feasible, but an small increase in total costs can be observed for some x_{ship} because downtime at wave condition 4 and 8 occurs for more values of x_{ship} due to the more strict criterion. The difference in the performance of the rubble mound and caisson breakwater is caused by the fact that wave transmission does not occur for a caisson breakwater, which means that the wave height at the berth will be lower.
- For run 2b, the optimum harbour lay-out does change. Due to the less strict wave height criterion, lay-outs with a breakwater length of L_s become feasible. In the other two runs, these lay-outs were not feasible because downtime occured at too many wave conditions (wave condition 6 and 8), which prevented ships from entering the harbour for an unacceptably long time, causing an unacceptable amount of vessels that have to wait at the anchorage at a certain period of time. In the new situation, the minimum costs are equal to 154 million USD and can be found at an x_{ship} of 2100 m, using a caisson breakwater of only one times the length of the design vessel. With the new criterion, no downtime at the berth occurs, and the only downtime due to tugboat unavailability occurs for wave condition 8.

It can be concluded that the maximum allowable wave height criterion for the berth has a large influence on the feasibility of several harbour lay-outs. If a less strict criterion is applied, more (and more important: less expensive!) solutions for the harbour lay-out become feasible.



Figure 6.4: Total cost function for design tool run 2a (maximum H_{m0} 0.8 m).



Figure 6.5: Total cost function for design tool run 2b (maximum H_{m0} 1.5 m).

6.4 Analysis 3: Maximum allowable wave height for tugboats

In the third analysis, the effect of the maximum allowable wave height for tugboats on the final results of the design tool is assessed. As was displayed in figure 3.3, tugboat effectiveness decreases with increasing wave height. The rate in which this occurs depends largely on the type of tugboat and the wave period. Tugboats with static winches can operate in wave heights from 1.5 to 2.0 m, with reduced effectiveness however. Above the limit of 2.0 m the static winches will become largely ineffective, and dynamic winches may provide an alternative in these cases [35]. Hence, the maximum allowable wave height will generally not be much larger or smaller than 2.0 m. In table 6.4 the values for the third sensitivity analysis are presented.

Case	Name	Maximum H_{m0} for tugboats [m]
0	Reference	2.0
3a	Lower boundary	1.9
3b	Upper boundary	2.2

Table 6.4: Design tool runs for analysis 3.

The cost functions for run 3a and 3b are displayed in figure 6.6 and 6.7. The following observations can be made:

- For run 3a, a large range of offshore distances becomes unfeasible because the wave height in this region is too high for tugboats to operate, which would lead to large queues at the anchorage. Only at relatively large values of x_{ship} , where the increase in wave height due to shoaling has not occured yet and the wave height is below the maximum acceptable H_{m0} , offshore harbour development is considered feasible. The solution with the lowest costs can be found at an x_{ship} of 2,040 m. A caisson breakwater with a length of $2L_s$ is applied for this solution. The costs are equal to 270 million USD. More onshore of $x_{ship} = 2,040$ m, downtime due to tugboat unavailability occurs at wave condition 6, which means that a total of five ships have to wait at the anchorage, which is more than the acceptable amount of 3 ships. Starting from $x_{ship} = 2,040$ m, no more downtime occurs for this wave condition, which means that the maximum allowable amount of ships waiting at the anchorage is no longer exceeded and the harbour becomes feasible again.
- The opposite occurs for run 3b. The less strict restriction to the wave height means that no downtime occurs for all wave conditions except for wave condition 4 and 8. This means that the maximum number of ships waiting at the anchorage is not exceeded and thus all offshore distances become feasible. The wave height still increases due to shoaling, but never exceeds the maximum value for tugboats for a wave condition with a relatively long duration. The optimum solution here is found at 1,860 m and a caisson breakwater with a length of $2L_s$ and has a total cost equal to 217 million USD.

Based on these results it can be said that the tugboat criterion has a large impact on the feasibility of a harbour lay-out. A decrease in the criterion of only 0.10 m was enough to consider nearly 600 m of cross-shore distance not feasible for the development of offshore harbours. This large impact is mainly due to the abruptness of the criterion: no gradual degree in efficiency as displayed in figure 3.3 is taken into account, but the efficiency drops to zero as soon as a certain wave height is exceeded.

Furthermore it was seen in analysis 2 and 3 that wave condition 6 can be considered a critical wave condition. If downtime occurs for this wave condition, either due to tugboat unavailability of excessive vessel motions, the harbour is no longer considered feasible. A total of 5 ships have to be able to enter, load and leave the harbour during this wave condition. Downtime means that 5 ships have to wait, which is larger than the maximum acceptable amount of vessels at

the anchorage (3). From this it also becomes clear that the maximum number of ships that are able to wait at the anchorage plays a large role in the feasibility of a harbour lay-out. A large acceptable number of ships at the anchorage may make more harbour designs feasible, even at larger percentages of downtime.



Total costs harbour (with downtime)

Figure 6.6: Total cost function for design tool run 3a (maximum H_{m0} for tugboats 1.9 m).

6.5 Analysis 4: Maximum allowable overtopping volume

The fourth analysis is performed on the maximum overtopping volume that is allowed for the rubble mound breakwater. This parameter only affects the harbour lay-out in which a rubble mound breakwater is used. As mentioned before, it is difficult to find wave overtopping criteria for offshore breakwaters since the maximum amount of overtopping usually is not expressed in a discharge per running meter, but in a maximum amount of wave transmission behind the breakwater. For other coastal structures however, overtopping criteria do exist. In section 4.3, it was noted that for harbours an overtopping volume of $0.05 \text{ m}^3/\text{s/m}$ corresponds to damage to larger yachts. This value is used as a starting point for the sensitivity analysis. Schiereck (2012) states that for breakwater much higher values can be used [33]. Therefore, the overtopping volume is increased and the effect of this increase on the total harbour costs is assessed. Based on the results, an estimate for a more realistic criterion for overtopping volumes can be formulated. In table 6.5 the values for the fourth sensitivity analysis are presented.

Case	Name	Maximum overtopping volume [m ³ /s/m]
0	Reference	0.05
4a	Lower boundary	0.2
4b	Upper boundary	0.5

Table 6.5: Design tool runs for analysis 4.

In the reference case an extremely large freeboard (order 11 - 13 m) is required to prevent overtopping under design conditions ($H_{m0} = 10$ m). This explains why in the reference case the



Figure 6.7: Total cost function for design tool run 3b (maximum H_{m0} for tugboats 2.2 m).

rubble mound breakwater costs are so high compared to the caisson breakwater costs. The results of the other two model runs are displayed in figures 6.8 and 6.9. The following observations can be made.

- For run 4a, the solution with the lowest costs remains the same. However, the total cost functions for the rubble mound breakwater have significantly changed. The costs are now much lower because a smaller freeboard relative to MSL is required. This means that less stone volumes are required and the breakwater becomes less expensive.
- For run 4b, the breakwater dimensions have decreased so much that the rubble mound breakwater costs are now of the same order of magnitude as the caisson breakwater costs. The optimum solution also has changed: a rubble mound breakwater with a length of $2L_s$ is now part of the recommended lay-out, which is located at an x_{ship} of 2,100 m and has a total cost of 231 million USD. The freeboard of the breakwater relative to MSL is now in the range of 7 8 m, which is comparable to the breakwater used in the offshore harbour design of verification 2 (section 5.3).

Based on this analysis it can be concluded that for detached rubble mound breakwaters overtopping volumes in the order of several hundreds of liters per second per running meter of the breakwater result in acceptable dimensions with costs in the same order as caisson breakwater costs.

Moreover, the overtopping volume also affects the downtime calculation via wave transmission. If more wave overtopping is allowed, this logically results in more wave transmission into the harbour. This may also affect the result of the downtime calculation and therefore the effect of the overtopping volume of the wave transmission is also investigated. Wave transmission during operational conditions for the harbour designs considered in this analysis will mainly take place *through* the breakwater and not so much *over* the breakwater, because the freeboard is much larger than the maximum wave heights that occur under operational conditions (7 - 8 m freeboard compared to H_{m0} of 2.5 m). For wave transmission through the breakwater, the cross-sectional area of the breakwater is an important parameter, as can be seen in Ahrend's

formula for wave transmission (equations 3.17 and 3.18). The larger the cross-sectional area, the larger the distance the wave has to travel through the breakwater and the more wave energy dissipation occurs, resulting in less transmission. An increasing overtopping volume results in smaller breakwater dimensions, which will logically lead to more transmission. Comparing the reference case with case 4b, the wave transmission for a wave height of $H_{m0} = 2$ m and a wave period of 16 s (wave condition 8) increases from approximately 27% in the reference case to 32% in case 4b. This means that slightly higher wave heights can be expected at the berth. However, it can be seen from the results of run 4a and 4b that this effect is negligible compared to the large decrease in CAPEX and OPEX on the final results of the design tool caused by the larger overtopping volumes.



Total costs harbour (with downtime)

Figure 6.8: Total cost function for design tool run 4a (maximum overtopping volume 0.2 $\rm m^3/s/m).$

6.6 Analysis 5: Step size

The final sensitivity analysis is performed on the step size x_{step} . As mentioned in chapter 4, the step size always has to be chosen such that the total offshore grid (from x_{min} to x_{max}) has less than 500 elements. For the harbour design calculations however, only values of x_{ship} between $x_{ship,max}$ and $x_{ship,min}$ are taken into account. Distances that are considered not interesting for offshore harbour development are automatically neglected by the design tool.

In the reference case a relatively small value for the step size was chosen (20 m), which resulted in a large set of output data for a large number of x_{ship} (53). However, a port engineer might not be interested in such a large set of output data and might want to compare a limited number of output locations. In order to do this, he or she might choose to use a larger step size. The effect of a larger step size on the final output is investigated in this section.

The model runs for this analysis are displayed in table 6.6. In the reference case the optimum solution was located at $x_{ship} = 2,100$ m. To prevent similar results for other step sizes, it has to be ensured that the step sizes in the other runs are not divisors of 2,100 m.



Total costs harbour (with downtime)

Figure 6.9: Total cost function for design tool run 4b (maximum overtopping volume 0.5 $\rm m^3/s/m).$

Case	Name	Step size [m]
0	Reference	20
5a	Medium step size	80
5b	Large step size	400

Table 6.6: Design tool runs for analysis 5.

The results are displayed in figure 6.10 and 6.11. For all runs a caisson breakwater of $2L_s$ proved to be the most advantageous solution. The following remarks can be made:

- For run 5a, only 15 offshore locations are considered. The minimum costs are found at an x_{ship} of 2,160 m, and are equal to 244 million USD. For a x_{ship} of four times larger, the costs have increased with 10 million USD (4.27%), whereas the offshore position of the ship has increased with 60 m.
- For run 5b, only 4 offshore locations are considered. This time the minimum costs are found at an x_{ship} of 1,600 m. The minimum costs for this run are equal to 265 million USD: an increase of 31 million USD (13.25%) compared to the original value. Furthermore, the offshore position of the berth is now half a kilometer closer to the coastline.

It can thus be concluded that for larger values of x_{step} , it is less likely that the real optimum location will be found, and more expensive solutions will be recommended by the design tool as optimum harbour lay-outs. Hence, the user has to choose between a limited number of output locations on the one hand and a larger deviation from the real optimum location on the other hand.



Figure 6.10: Total cost function for design tool run 5a ($x_{step} = 80$ m).



Figure 6.11: Total cost function for design tool run 5b ($x_{step} = 400$ m).

6.7 Conclusions

In the final section of this chapter the conclusions of the sensitivity analysis are summarized. The influence of several input parameters on the design tool input has been assessed by varying the value of these parameters one by one and observing differences in model output.

The maximum wave height at the berth and maximum allowable wave height for tugboats both have similar effects on the design tool output. Both influence the output of the design tool via the *feasibility* of harbour lay-outs. Less strict criteria make more (and less expensive) harbour lay-out types feasible, which may cause a shift in the recommended solution.

The waiting costs and overtopping criterion influence the output of the design tool via the *attractiveness* of harbour lay-outs. The waiting costs change the proportion in which the CAPEX and OPEX on one hand and the downtime on the other hand influence the design tool output: higher waiting costs give a larger weight to the downtime criteria, which makes lay-outs with little downtime much more attractive compared to lay-outs with moderate downtime figures. The overtopping criterion influences the choice between a rubble mound and caisson breakwater. The more overtopping is allowed, the smaller the dimensions of the breakwater can be and the more attractive a rubble mound breakwater becomes.

Finally, the step size mainly influences the *accuracy* of the design tool. A larger step size causes larger deviations from the real optimum solution, which may result in higher minimum costs.

Chapter 7

Conclusions and recommendations

In the final chapter of this report the research questions are answered and the most important conclusions of this study are presented. Moreover, recommendations for further study are given.

7.1 Introduction

In this study an attempt has been made to develop a design tool for offshore harbours. An offshore harbour generally consists of three elements: an exposed jetty, a detached breakwater and dredging works. The feasibility and overall costs of the harbour depend on a variety of parameters. During this study the focus was on two of these parameters: the offshore distance of the berth x_{ship} and the length of the breakwater L_{BW} . A larger offshore distance results in lower dredging costs, but higher breakwater and jetty costs. Furthermore, the short wave climate is generally more rough, and due to the exposed nature of the harbour this may lead to downtime. Close to the shore the jetty and breakwater costs decrease, however, the dredging costs increase. Moreover, the effect of reflected long waves, that are able to cause excessive vessel motions due to resonance, increases close to the shore. As for the length of the breakwater: a larger length logically results in more costs. However, more shelter is provided for the vessel which means downtime is less likely to occur. Overall, it can be expected that an optimum value for x_{ship} and L_{BW} exists at which a balance can be found between capital and operational costs on one side and sufficient uptime on the other side. The goal of the design tool is to find this optimum. The primary objective of this study was defined as follows:

To develop a design tool that is able to recommend an optimum offshore harbour lay-out (i.e. offshore distance of the berth and breakwater length). An optimum harbour design is the design that has a large uptime (relatively low downtime) and limited expenditures during the lifetime of the harbour.

A scope for this study has been defined in chapter 1. Only dry bulk harbours with one berth are considered. Moreover, downtime is assumed to be caused only due to wave action. A simplified alongshore uniform coastal profile with a homogeneous soil and parallel depth contours is assumed.

In order to achieve this objective, several steps were undertaken. A literature study was performed to identify relevant parameters and processes and to formulate a set of criteria the design tool has to fulfill. Furthermore, a method to incorporate the relevant parameters and processes in the design tool had to be found. After that, the actual development of the design tool could start. Using empirical formulas, rules of thumb and simple mathematical formulations, the design tool was constructed such that for a given set of input parameters, an optimum configuration for x_{ship} , L_{BW} and breakwater type can be recommended. By verifying and evaluating the design tool, conclusions on the reliability and accuracy of the model formulations could be drawn.

7.2 Conclusions

In this section the conclusions of this study are presented based on the answers of the research questions.

- The design tool has to fulfill several requirements in order to be applicable for offshore harbours projects. An offshore harbour has to be designed for a range of offshore distances x_{ship} and breakwater lengths L_{BW} , based on a set of input criteria given by the user. This harbour design should fulfill criteria with regard to navigational, operational and structural safety. For each offshore harbour design (one without breakwater, four with a caisson and four with a rubble mound breakwater) the CAPEX and OPEX have to be determined as a function of x_{ship} . Moreover, the waiting costs due to downtime have to be determined based on the short and long wave height at the harbour. The optimum harbour lay-out is defined as the combination of x_{ship} , L_{BW} and breakwater type for which the sum of the CAPEX, OPEX and waiting costs due to downtime are at a minimum. These requirements form the answer for the first research question.
- Several parameters influence the design of an offshore harbour. The dimensions of dredged areas mainly depend on environmental conditions and ship dimensions. PIANC guidelines can be used to incorporate these parameters in the design tool. The dimensions of an exposed breakwater depend mainly on the design wave conditions the breakwater has to resist. Design formulas for required stone size and freeboard can be used to incorporate this dependency in the design tool. Operational downtime depends on the wave conditions (height, period and direction) at the berth. Especially low frequency waves are able to cause downtime due to resonance. Thoresen (2010) formulated maximum allowable motions at which loading and unloading operations are still possible. Combining these with Mol's empirical formula for the estimation of the surge motion can be a method to determine whether downtime at the berth occurs or not. Downtime due to excessive sway and heave motions can be predicted based on the maximum allowable values of H_{m0} at the berth as recommended by Thoresen (2010). Tugboat unavailability is also a cause for downtime and can be predicted by considering the wave height during the tugging journey. It can be seen that the wave conditions play a large role in the design and feasibility of an offshore harbour. By combining linear wave theory with several empirical formulas, the wave transformation processes (shoaling, refraction, wave breaking), wavebreakwater interaction (transmission, diffraction) and occurrence of infragravity waves can be incorporated in the design tool. This overview of parameters and processes forms the answer for the second research question.
- The capital and operational costs have been expressed as a function of the offshore distance of the ship x_{ship} and the breakwater length L_{BW} in order to answer the third research question. Several cost functions have been formulated, which summed up the jetty, breakwater and dredging costs. An increase in x_{ship} leads to an increase in jetty costs since the jetty has to become larger and has to be constructed in deeper water, leading to increased unit costs. An increase in x_{ship} also means larger breakwater costs, mainly due to the increase in depth, with leads to a quadratic increase in cross-sectional area, resulting in higher material volumes required. The dredging costs decrease with increasing x_{ship} since the available water depth increases, which means that the dredging depth becomes smaller. Moreover, the more offshore the harbour is located, the shorter the approach

channel has to be. The length of the breakwater only influences the breakwater costs: a longer breakwater means more material volumes required, logically resulting in higher costs. For shorter breakwater lengths, the optimum solution (without the incorporation of downtime) is located more offshore since the dredging costs will dominate near the shore. For longer breakwaters however, the breakwater costs will dominate for large x_{ship} , which means that the optimum will move onshore.

- The wave calculation tool as developed for the design tool can be used to answer research question number 4. The offshore distance of the berth mainly influences the amount of shoaling that has occured. Shoaling effects generally become larger close to the coast (outside the surf zone), which means that an increasing x_{ship} will generally lead to a smaller incoming wave height and thus less downtime, either at the berth or due to tugboat unavailability. Furthermore, the low frequency wave height is generally also larger close to the coast, which means that a larger x_{ship} will result in less downtime due to excessive surge motions. The breakwater length influences the amount of sheltering at the berth. A larger length results in a smaller diffracted wave, which means that the total wave height at the berth will be lower, reducing the probability of downtime.
- The verifications for the nearshore wave propagation and wave penetration calculation lead to the answer to the fifth research question. For the undisturbed wave propagation, it was found that the low frequency wave height is generally underestimated for large wave heights, short peak periods and mild slopes, whereas the total significant wave height is generally correctly estimated for the offshore distances of interest. The wave penetration calculation has the main drawback that wave reflection off the breakwater is not taken into account. This may especially be an issue for caisson breakwaters since these are highly reflective. Furthermore, the low frequency wave height at the berth is underestimated because the design tool assumes that the *total* infragravity wave is blocked by the breakwater, whereas only the incoming wave is blocked and the reflected wave is still able to reach the ship.
- The reliability of the downtime prediction (research question 6) depends largely on the value of C_x applied. For the case study considered, values of approximately 1.0 seemed to result in a correct prediction for peak periods between 11 and 14 s.
- The sensitivity of the design tool for several important input parameters has also been investigated, in order to answer the final research question. The waiting costs are an important factor when it comes to the weight given to the criterion 'downtime'. Higher waiting costs make better protected harbours more attractive. The overtopping criterion influences the attractiveness of rubble mound breakwaters compared to caisson breakwaters. The maximum allowable wave height at the berth and for tugboats influence the feasibility of the offshore harbour: more strict criteria result in less feasible alternatives. The step size of the calculation influences the accuracy of the design tool: larger step sizes cause larger deviations from the actual optimum.

In general, it can be said that the primary objective of this study has been succesfully achieved, since a functional and usable design tool has been developed that can be used for port planning purposes.

7.3 Reflection

The design tool has been tested and evaluated for this report based on a total of eight wave conditions. The main focus of this study was on finding suitable formulations and combine these formulations into a simple, user-friendly tool that can be used for quick assessments. Therefore initially a limited number of wave conditions has been used to set up the tool, since the formulations that will be chosen will be the same for each wave conditions. If the formulations are found to give accurate solutions, the number of wave conditions applied can be expanded quite easily by simply adding Excel-sheets, rows and columns to the design tool. Furthermore, by using a limited number of wave conditions, the effect of downtime on the results of the tool can be investigated better, since downtime under only one wave condition may already have serious consequences on the result of the tool. This has been elaborated more thoroughly in the sensitivity analysis.

However, one might argue that a wave climate consisting of only eight wave conditions is a very inaccurate and rough schematization that may not be realistic at all. In section 4.6.3 the selection of wave conditions for the example run and the sensitivity analysis has been explained. The H_{m0}, T_p -table has been divided into eight sections, and the most disadvantageous wave condition (combination of highest H_{m0} and longest T_p) was taken to be representative for the entire section. If uptime occurs for this wave condition, this means that uptime will occur for all other wave conditions in that section. However, if downtime occurs for this wave conditions in that section, since the other wave conditions are less heavy. For example: if downtime occurs during wave condition 1 ($H_{m0} = 1.25$ m; $T_p = 12$ s), the design tool assumes that downtime will also occur during a lighter wave condition (e.g. $H_{m0} = 0.75$ m; $T_p = 7$ s) that is also in that section. This of course does not necessarily have to be the case.

Moreover, the design tool assumes that the wave condition will persist over its entire period of occurrence. For example: if wave condition 1 has a frequency of occurrence of 30%, the design tool assumes that over a persistent period of $0.30 \times 12 = 3.6$ months, only waves with that H_{m0} and T_p will occur, and no other (smaller) waves will be present. This is also a quite rough and conservative assumption: in reality a (local) storm of a few hours might cause high waves for a certain period of time, but when the storm ends, smaller waves will occur again. Especially the persistent nature of the wave conditions inserted in the design tool may have large consequences, since downtime due to one wave condition might mean that ships are not able to enter the harbour for a long period of time, causing the harbour design tool lead to a very rough and conservative approach which might not give a clear image on the effect of downtime on a certain harbour design. In reality, much more wave conditions have to be used in the design tool in order to come to an accurate prediction of the downtime.

Another quite conservative approach adapted in the design tool is the way in which downtime is predicted. As soon as the maximum allowable H_{m0} or $H_{l,m0}$ at the berth or the maximum allowable H_{m0} during the tugging journey is exceeded, downtime is assumed to occur, either due to excessive sway, heave or surge motions or due to tugboat unavailability. This criterion is quite abrupt, and leads to notable drops and lifts in the total cost function, which was especially noticeable in figure 6.3 where a large value for the waiting costs was assumed. In reality however such drops do not occur, since downtime does not occur immediately after a value for the wave height is exceeded. It was mentioned in section 3.2 that in reality, the efficiency of handling operations and tugging operations gradually decreases with increasing wave height. Therefore, in reality a more gradual decrease (or increase) in the waiting costs (and total costs) will be perceivable. The abruptness of the downtime criteria is also an extremely conservative approach which might be sufficient for a first estimate, but will have to be adapted if more accurate cost estimates are required.

7.4 Recommendations

Finally, some recommendations for further study are given. The design tool is of course in an early stage of development and possibilities for further expansion and improvements exist. It

was found in chapter 5 that the infragravity wave height is generally underestimated. This is mainly because the influence of long waves that are reflected off the coast is underestimated. Incorporating a method to accurately determine the height of the reflected long wave may improve the reliability of the design tool. Moreover, by separating the incoming and reflected long wave height, a better estimate of the long wave height at the berth can be obtained.

Furthermore, it is also recommended to investigate possibilities to expand the applicability of the model. This can be done by investigating for example how the design tool can deal with coastlines with varying slopes or alongshore non-uniformities. Another recommendation is to add variables to the harbour design. In the present design tool, only one simple harbour design with one berth can be investigated. It might be an idea to expand the design tool so it can deal with harbours with multiple berths, harbours meant for other types of cargo than dry bulk and/or different breakwater shapes or orientations. In this way the design tool can be used for more realistic harbour designs. The downtime prediction may also be improved, for example by also taking into account mooring line and fender forces instead of vessel movements only. In this way, the downtime prediction might be more in line with actual DMAs. As was mentioned in the reflection, a gradual increase in waiting costs due to a gradual decrease in cargo handling efficiency or tugging efficiency might also form a large improvement for the design tool.
Appendix A

Environmental analysis of West-Africa

In this chapter the results of an environmental analysis of the West-African coast are presented. The purpose of this analysis is to determine the boundary conditions that have to be taken into account for the design of the offshore harbour. The boundaries of the West-African coast were presented in section 2.2. Generally data at four points along the West-African coast are obtained:

- Abidjan, the Ivory Coast at $(5^{\circ}N, 4^{\circ}W)$.
- Accra, Ghana at $(5^{\circ}N, 1.5^{\circ}W)$.
- Lomé, Togo at $(6^{\circ}N, 1.5^{\circ}E)$.
- Lagos, Nigeria at $(6^{\circ}N, 3.5^{\circ}E)$

The first section of this appendix treats the wave height, period and direction under operational and design conditions. Sections A.2 and A.3 present wind and current data. Based on these data it can be concluded that wind and current action along the West-African coast are negligible when it comes to determining downtime. Tidal elevations and bathymetry are assessed in the final two sections.

A.1 Wave conditions

A.1.1 Operational wave conditions

The wave climate in West-Africa can be classified as a west coast swell climate. The waves that arrive at the coast originate from storm waves generated by westerlies in the Southern storm wave belt. The waves arrive predominantly from the southwest and have a typical persistent nature with relatively long periods. The waves are uniform in direction, size and shape and usually have moderate wave heights. There is not a large variation in wave heights around the mean. There is a certain seasonal variation, with waves being larger during the summer months [3].

The average H_{m0}, T_p -table for the four locations is displayed in figure A.1. Wave roses for the four locations can be found in figure A.2.

	3		2	2	2		E		E				0		HmO			
Tota	5	ω	3	5	23	2	3	ن ہ	5		3	5	5	0	(m)			
-	3.5	3.25	ω	2.75	2.5	2.25	2	1.75	5	1.25	_	0.75	<mark>0</mark>	0.25				1_
0.00075	0	0	0	0	0	0	0	0	0	0	0.00075	0	0	0		4	ω	þ
0.0632								0.001	0.0122	0.0417	0.0077							(s)
5 0.	0	0	°	°	0.0	0	0	5 0.0	0	5 0.0	5 0.0	0		0		5	4	
2575	•	•	•	•	0075	0035	0.02	3225	0505	9225	5575	0025	•	•		6	л	
0.835	0	0.00075	0	0.00175	0.01025	0.017	0.0365	0.10775	0.14625	0.2365	0.267	0.01125	0	0		7	6	-
0.68675	0	0	0.00075	0.0025	0.00425	0.026	0.0365	0.0825	0.094	0.21475	0.19925	0.02625	0	0		8	7	
2.695							0.0217	0.101	0.399	1.0592	0.997	0.11						
5 9.5	0	0	0	0	0	0	0	5	5 2.4	5	5 2.3	6	0.0	Î		9	00	
3175 1	0	0	0	0	.0025	.0365	13475	0.77	19825	.4975	31025	28025	00175	•		10	9	
4.32325	0	0	0	0	0.011	0.10875	0.6485	2.32	3.76625	4.4105	2.5795	0.477	0.00175	0		11	10	
18.10075	0	0	0	0.00775	0.15575	0.428	1.14	2.7405	4.41825	5.80025	3.07775	0.3325	0	0		12	11	
17.357			0.0017	0.036	0.20	0.54	1.410	2.9892	4.6242	5.1162	2.243	0.178				1	H	
5 15.93	Ĭ	Ĭ	5 0.016	0.08	0.0	0.80	5 1.71:	5 2.9	5 4.13:	4.203	5 1.58	0.14	Ĭ	Ĭ		Ĩ	2	
295	•	•	525	175	248	575 (175	866	175	225	525	775	•	•		14	13	
11.2755	0	0.00675	0.04875	0.1255	0.3585	0.70925	1.3185	2.13125	2.74625	2.74475	1.0175	0.0685	0	0		15	14	
4.36225	0	0.005	0.04	0.08875	0.19525	0.2975	0.5555	0.823	1.09375	0.9095	0.32	0.034	0	0		16	15	
2.52025	0.00075	0.0155	0.00775	0.03825	0.1035	0.15825	0.21325	0.4645	0.65275	0.6045	0.2395	0.02175				17	16	
1.53		0.01	0.0	0.0	0.065	0.104	0.148	0.285	0.35	0.3	0.141	0.016		Ĩ				
45	0	65 O.U	06 0.0	25 0.0	25	25 0	75 0.0	25 0.0	75 0.0	68 0	75 0	25	0	•		18	17	
0.199	0	00175	00425	00775	0.005	.0155	02775	03125	05475	.0285	.0225	0	•	•		19	18	
0.28675	0	•	•	0.00075	0.004	0.0025	0.01025	0.053	0.0965	0.06525	0.04525	0.00925	0	•		20	19	
0.0175	0	0	0	0	0	0.00225	0.00075	0.00325	0.00575	0.00325	0.0015	0.00075	0	0		21	20	
0.011									0.003	0.00								
75	0	0	0	0	•	0	0	0 15	25 25	85	0 15	•	•	0		22 cla	21 AII	
100	1.00075	1.04625	0.1255	1.41625	1.373	3.264	7.435	5.93475	1.15175	9.4035	5.11225	1.7225	0.0035	0		isses		

Figure A.1: Average H_{m0}, T_p -table for all four West-African locations.



Figure A.2: Yearly wave roses for the four West-African locations. Top left: Accra; top left: Abidjan; bottom left: Lomé; bottom right: Lagos.

A.1.2 Design wave conditions

In section 2.2.2 it was mentioned that one of the requirements the offshore harbour has to fulfill is that the breakwater should not fail under design wave conditions. In order to design the breakwater such that it can resist these conditions, first the design conditions have to be identified. Design conditions are conditions that have a very low probability of occurrence. This small probability is of course necessary to prevent the breakwater from failing under everyday wave conditions, and is usually related to the economic lifetime of a structure and must be sufficiently low to prevent the occurrence of serious damage during the lifetime. The probability of occurrence of design conditions can be determined using the following formula [43]:

$$f = -\frac{1}{T_L}\ln(1-p)$$
 (A.1)

In which f is the average frequency with which design conditions occur per year, T_L is the lifetime of the structure and p is the probability that design conditions occur during the lifetime of the structure, i.e. the probability the structure fails during its lifetime. Taking the economic lifetime of a breakwater to be equal to 50 years and an accepted failure probability of 1%, this results in an occurrence probability of approximately 0.0002, i.e. once every 5,000 years.

Wave records that go back 5,000 years are of course not available. However, an estimate for the wave conditions that occur once every 5,000 years can be made by extrapolating available wave data using a probability distribution. Hence, the first step is to choose a suitable probability distribution. Three different distributions are investigated:

- An exponential distribution, given by: $Q = A \cdot \ln H_{m0} + B$. This distribution is examined because long-term wave data is usually logarithmically distributed.
- A Gumbel distribution, given by: $Q = exp(-exp(-\frac{H_{m0}-\gamma}{\beta}))$ or $G = AH_{m0} + B$, in which G is the so called reduced Gumbel variate $G = -ln(ln(\frac{1}{Q}))$.
- A Weibull distribution, given by: $Q = exp(-(\frac{H_{m0}-\gamma}{\beta})^{\alpha})$, or $W = AH_{m0} B$ in which W is the reduced Weibull variate $W = -ln(Q)^{\frac{1}{\alpha}}$.

Gumbel and Weibull are extreme value distributions, which are developed specially to deal with extreme deviations from the median of probability distributions. This makes these distributions extra suitable to determine extreme wave conditions. In all formulas displayed above, Q is the exceedance probability of a certain H_{m0} in [1/years]. Furthermore, it can be seen that all distributions have several unknown coefficients (A, B and α).

Based on the NOAA wave data obtained for the three West-African locations for 10 years (1-10-2005 to 31-7-2014), the exceedance probability of a certain wave height can be determined by summing up the number of observations that exceed this H_{m0} ($n(H > H_{m0})$) and dividing over the total number of observations (n_{tot}):

$$Q(H_{m0}) = \frac{n(H > H_{m0})}{n_{tot}}$$
(A.2)

By doing this for a number of values of H_{m0} , the exceedance probability Q can be plotted as a function of H_{m0} . Using Excel, a trend line can be plotted for this function. The unknown coefficient $(A, B \text{ and } \alpha)$ can be determined using this trend line: A is simply the slope and Bthe intersection with the vertical axis. Based on the value of the coefficient of determination (R^2) , it can be determined how well the chosen distribution fits the obtained wave data. For the Gumbel and Weibull distributions, an additional step is required: the reduced Gumbel and Weibull variates G and W have to be determined using the obtained values for Q. By plotting the reduced variates as a function of H_{m0} and adding the trendline, it can be determined whether the distributions fit the NOAA data. The results of this calculation are displayed in the table below.

Distribution	Parameter	Abidjan	Accra	Lagos	Lomé
	А	0.8531	0.8919	0.7937	0.9961
Exponential	В	-1.7859	-1.9603	-1.6383	-1.6469
	R^2	0.9587	0.9420	0.9626	0.9692
	А	0.9899	1.0351	0.9261	1.1376
Gumbel	В	-2.8503	-3.0758	-2.6751	-2.5453
	R^2	0.9927	0.9836	0.9945	0.9942
	А	0.3510	0.3377	0.3404	0.4557
Woibull	В	-0.3214	-0.3097	-0.3042	-0.3464
Weibuli	α	1.75	1.85	1.74	1.62
	R^2	0.9992	0.9992	0.9987	0.9995

Table A.1: Results of the regression analysis for the NOAA West-African wave data.

Furthermore, examples of wave height exceedance graphs made with the exponential, Gumbel and Weibull distribution for the Ivorian city of Abidjan are shown in figure A.6. It can be concluded that a Weibull distribution fits the obtained data best. As there are small differences between the four considered locations with regard to the parameters related to the Weibull distribution, an estimate can be made for a probability distribution that fits the entire West-African coast. This estimate is based on the weighted average of the parameters of the four Weibull distributions found and reads as follows:

$$(-\ln Q)^{1/1.74} = 0.3712H_{m0} - 0.3204 \tag{A.3}$$

Based on this equation the significant wave height H_{m0} that has an exceedance probability of Q in [1/year] can be found. Using an exceedance probability of 1/5000, this leads to a design wave height of $H_{m0} = 10.1$ m.

The corresponding wave period can be found by assessing the correlation of H_{m0} and T_p . In figure A.3, a scatter plot of all obtained H_{m0} and T_p data points is shown. Lines along which the deep water wave steepness s_0 is the same are also plotted. It can be seen that a wave steepness of 0.005 corresponds with the most data point. The deep water wave steepness is defined as follows:

$$s_0 = \frac{2\pi H_{m0}}{gT_p^2}$$
 (A.4)

An estimate for the wave period that corresponds with the design wave height can be made by using this wave steepness. Using a H_{m0} of 10.1 m, a T_p of 36 s can be found.



Figure A.3: Scatter plot of the observed NOAA data for Accra with lines of equal wave steepnesses.

A.2 Wind conditions

A.2.1 General

Wind can affect a moored vessel through static and dynamic action. Static action means that the wind is constant or varies slowly in intensity, whereas dynamic action considers the effect of wind gusts, intensity blusters and changing direction. The effect of winds increases with larger wind velocities and for ships with a large superstructure or deck load, such as tankers in ballast and container ships. However, the effect of wind is not always harmful. If the wind blows towards the quay and pushes the ship towards the fenders, the wind may act as a sort of pre-tension of the mooring lines, causing an increase in fender friction [29].

During a certain part of the year the region of interest is located in the Intertropical Convergence Zone (ITCZ), which is a band of low air pressure where the northeast and southeast trade winds meet. The wind climate in the ITCZ is generally calm, with very low wind speeds. These quiet periods of wind are referred to as doldrums or equatorial calms. The location of the ITCZ varies over the year due to the varying position of the Sun. The seasonal shifts of the ITCZ result in great seasonal temperature and humidity differences between the Sahara and the equatorial Atlantic Ocean, which causes the West African monsoon, which mainly affects regions between latitudes of 9° and 20° . The influence of the monsoon declines outside this region [3].

A.2.2 Governing wind conditions

In order to get a more accurate view for four locations along the West-African coast are analysed: Abidjan, Accra, Lomé and Lagos. Based on wind speed and velocity data obtained from NOAA for a period of 10 years (October 2005 - July 2014) wind roses for the four locations were made. The results are displayed in figure A.7. It can be seen that wind from the South-West (between 210° and 240° N) is dominant. Furthermore, wind speeds above 9 m/s are extremely rare.

Furthermore, the storm duration is also an important parameter that influences the design of an offshore harbour. The average storm duration can be estimated using the NOAA data, which shows the wind speed for every three hours measured over a period of approximately 10 years, assuming the wind speed remains constant during the measurement interval. A storm is defined here as a period in which the wind speed exceeds Beaufort scale 5, i.e. a wind speed of 10.8 m/s or more. The determined storm durations are displayed in the table below.

Storm duration	3 - 6 hours	6 - 9 hours	9 - 12 hours	Average storm duration
Abidjan	7	5	1	6.11
Accra	7	5	1	6.11
Lagos	6	5	1	6.25
Lomé	6	5	1	6.25

Table A.2: Number of storms with a certain storm duration for the four West-African locations considered.

Hence, it can be concluded that the average storm duration is approximately equal to 6 hours.

A.2.3 Limitations on wind speed

Different limitations for wind speed exist for different types of ships. Generally it can be said that a ratio of wind speed over vessel speed of about 6 - 7 can cause great difficulty for the manoeuvrability of ships with very high superstructures, such as oil tankers in ballast, containerships and car carriers. Lightly loaded ships or ships with high windage areas experience serious difficulties if such a ratio is present, and fully loaded ships are hard to control for ratios of about 10. Some examples of operational wind limitations are given in the following [35].

- \bullet In order to ensure operational safety for cruise ships, motions should be minimized in winds up to 18 m/s.
- Large ferries can berth up to wind speeds between 15 and 30 m/s and unberth at wind speeds between 12 and 35 m/s, depending on the wind direction.

- Cargo and container ships will not be able to operate in a wind stronger than about 20 m/s, as it is impossible to handle cranes at such a wind speed. Other types of loading and unloading however remain possible, provided that the movements of the ship itself remain acceptable.
- An oil or gas tanker with a projected wind area greater than 5000 m^2 will generally not be able to safely berth at a wind speed of 10 m/s, provided that the significant wave height is less than 0.6 m. Lower wind limits apply at exposed berths.
- For modern tugboats a wind speed of about 12 to 15 m/s in combination with a significant wave height of 1.0 1.3 m is taken as a guideline limit for safe operation.

From the wind roses it can be observed that the wind speed generally not exceeds 9 m/s, which is why it is expected that wind will not be able to cause downtime.

A.3 Currents

A.3.1 General

Currents are usually negligible for ships moored in a harbour basin. This is different for ships moored or navigating in rivers or estuaries, or for ships moored at an open jetty. Currents can cause lateral oscillations due to 'flutter', which means that the arm of the moment exerted by the total of external forces (mooring and fender forces, currents) relative to the centre of gravity reaches a value close to the radius of gyration. The underkeel clearance plays a large role in the effect of currents. A small underkeel clearance can increase the effect of currents up to six times compared to deep water [29].

Ocean currents can have different driving mechanisms. The first type of currents is related to density differences and is called thermohaline currents. The density of water varies with salinity, temperature and pressure. At any given depth, density differences are only due to temperature and salinity differences. These density differences cause differences in water level. High density areas have a lower water level than low density areas, which means that water tends to flow from areas with low density (higher water) to areas with high density (lower water). Coriolis causes a deflection of the thermohaline currents. The second type of currents is wind driven currents. Wind stress causes the surface layer of water to move. Coriolis balances the wind stress in the top 10 to 200 m (the Ekman layer) and causes a deflection of 90° of the current with respect to the wind direction. Finally, tides and alongshore variations in radiation stress also cause ocean currents [27].

A.3.2 The Guinea Current

The most important current in the region of interest of this study is the Guinea Current, which flows east in the region between $(5^{\circ}N, 10^{\circ}W)$ and $(2^{\circ}S, 10^{\circ}E)$ and carries warm water to the coast, with velocities in the range of 1 - 3 kn (0.5 - 1.5 m/s). It has two sources: the North Equatorial Countercurrent and the Canary Current, and is mainly wind-driven and relatively shallow. The seasonal instability of these two currents affects the seasonal variability of the Guinea Current. The Guinea Current experiences a minimum during the winter months (November through February) and a maximum during the summer (May through September). It can obtain velocities of 1 m/s near 5° West, in the Gulf of Guinea. In figure A.4 the direction of the Guinea Current during the summer months (April to September) is shown. The direction remains from West to East throughout the year [14].

The variability of the surface current velocity in the Tropical Atlantic was investigated by Muñoz (2003). Table A.2 presents the maximum values for the Guinea Current throughout the year as determined by Muñoz [26].



Figure A.4: Direction of the Guinea Current during April to June (left) and July to August (right). [14]

Season	Region of maximum values	Magnitude
Winter	From $(15^{\circ}W, 5^{\circ}N)$ to $(5^{\circ}E, 0^{\circ}N)$	$\sim 30 \text{ cm/s}$
Spring	From $10^{\circ}W$ to the east along the coast $(4^{\circ}N)$	25 cm/s
Summer	From $7^{\circ}W$ to $4^{\circ}E$ and from the coast to $2^{\circ}S$	> 30 cm/s
Fall	From $12^{\circ}W$ to the east and from coast to equator	Very weak

Table A.3: Seasonal variations in the surface current velocity for the Guinea Current. [26]

A.4 Tides

The astronomical tide does not exert a force on a moored vessel. Nevertheless, it may cause an alongshore current, which can exert a force in an open jetty situation. Furthermore, it causes a rise and fall of the vessel that influences the design of the harbour, especially the berth. The vertical position of fenders should be taken into account to allow for the maximum tidal range expected at the site [29]. West-Africa experiences a semi-diurnal tide, which means that both high and low water are reached two times a day. In figure A.8 the water levels over the year 2014 (from 1 January 2014 to 1 January 2015, with intervals of 10 minutes) are shown for four West-African locations, relative to Mean Sea Level (MSL).

A.5 Bathymetry

In figure A.5 a bathymetric map is displayed for the region of West-Africa. It can that the bed slope varies over the region, with a relatively steep slope along the coast of Ghana and more mild slopes along the Ivory Coast and Nigeria. The map however shows a very large region with offshore distances in the order of hundreds of kilometers and depths in the order of kilometers. Only the coastal zone with depths up to 40 or 50 m is of interest for harbour development. In figure A.9 the bathymetry for four West-African locations is shown up to a depth of 45 m. The average bed slope is calculated based on this figure and the results are displayed in table 2.2. It must be noted that these figures show the height of the bed along the meridians, so along straight lines from North to South. The bed levels are given relative to Mean Sea Level.

It can be seen from the values in table 2.2 that the bed slope is generally in the order of 1:200 to 1:500. Abidjan is an exception on this rule with a quite steep bed slope of 1:44 in the near coastal region. From this it can be concluded that the bed slope is not uniform along the West-African coast.



Figure A.5: Bathymetry of West-Africa.



Gumbel Abidjan



Weibull Abidjan



Figure A.6: Extrapolation of wave data using three probability distributions for the location of Abidjan.



Figure A.7: Yearly wave roses for four West-African locations. Top left: Accra, top left: Abidjan; bottom left: Lomé; bottom right: Lagos.



Figure A.8: Yearly tidal elevations for four West-African locations. Top left: Accra, top left: Abidjan; bottom left: Lomé; bottom right: Lagos.



Figure A.9: Coastal bathymetries for four West-African locations. Top left: Accra, top left: Abidjan; bottom left: Lomé; bottom right: Lagos.

Appendix B

Results verification

In this appendix the results of the verifications applied in section 5.2 and 5.4 are presented.

B.1 Verification 1: undisturbed wave propagation

The results of the SWASH verification of the undisturbed wave propagation on the three bed slopes are presented in figure B.1 to B.3. The results are presented as procentual differences between the SWASH and design tool values of the wave height, determined using equation 5.5. Red values indicate that the maximum allowable error is exceeded.

B.2 Verification 3: downtime prediction

The results of the verification of the downtime prediction are presented in figures B.4 to B.9. Figure B.4 and B.5 contain the relative errors for the harbour at -9 and -11 m + CD, for both ship types and using a C_x of 1.7. Figures B.6 to B.9 contain the relative errors obtained applying other values of C_x , ranging from 1.0 to 1.6. Red values indicate that the maximum allowable relative error is exceeded.

				Average sig	ope, results SWASE	i verification			
	Wave cond	ition 1	Wave con	dition 2	Wave co	ndition 3	Wave co	ndition 4	Wave condition 5
Offshore distance	Deviation Hm0 [%] Dev	viation infra [%]	Deviation Hm0 [%] 1	Deviation infra [%]	Deviation Hm0 [%]	Deviation infra [%]	Deviation Hm0 [%]	Deviation infra [%]	Deviation Hm0 [%]
5750	0	12.17447353	0	18.14122309	0	32.24743632	0	11.29261081	0
5500	0.401742973	11.45457718	0.43182728	15.99527577	-0.093840663	29.98273361	0.407469625	9.212854367	-0.048618893
5250	1.032736618	10.04933127	0.651682563	15.52179354	0.105863281	28.22675996	0.874301643	8.713639495	-0.153646018
5000	1.384024223	9.700384647	0.994557684	16.16458401	0.260529179	26.95837824	1.766787565	7.552905464	-0.81858701
4750	1.873054915	8.579945773	1.510362432	15.89547493	-0.006723303	25.72220493	2.462830272	5.544077944	-0.908681715
4500	2.378616347	7.935397096	1.586817733	14.11829784	0.040757913	23.42004441	3.388503534	5.154006418	-1.01387159
4250	2.665899679	6.887616143	1.856752714	11.81855569	-0.204828549	21.37199738	4.249125544	4.956633473	-1.41488305
4000	3.148846533	5.011139102	2.072346895	13.66975888	-0.106420962	19.63061465	5.228459296	3.692930826	-1.398202531
3750	3.190529134	5.31223643	2.09891495	12.61304607	-0.278565817	14.74891557	6.258360361	2.711968128	-1.369689489
3500	3.362786259	1.609137973	2.036591645	12.81506848	-0.311507305	12.41844539	7.270976274	-0.036891428	-2.104625625
3250	3.428395245	-0.260697191	1.981677108	6.394127084	-0.691737164	11.06454778	8.280571319	-0.538491596	-2.411344045
3000	3.522556208	0.033891188	1.900579364	8.530056739	-0.724529008	9.308960796	9.126567745	-0.211783517	-2.890963375
2750	3.351096971	-5.086189764	1.744398361	6.941114476	-0.966767123	4.697833815	10.04916349	-4.021830496	-2.800668941
2500	3.057656366	-1.996435235	1.3/0899677	7/7266677	E012372225	2.748036589	10.63843805	-4.39/139989	-3.93/141111
0000	2 711020127	- 10.02000 + 2.0	0 500014407	0 974105700	-1 70000700	-0.020720020	11 20000210	-0.4000r0404	
1750	2.153071233	-7.552107733	0.228921461	8.594922376	-2.245080133	-3.619973121	11.3710043	-6.185703974	0.964573793
1500	1.546078007	-15.51875551	-0.466237279	22.21190608	-2.69257306	-1.051312344	11.05015409	-7.810605329	4.589537298
1250	0.893009325	-7.471129606	-1.21311098	32.24576732	-3.281614102	-2.496463761	10.29590904	-8.753203288	5.610908219
1000	-0.006582375	-6.977729837	-1.708426719	47.63106726	-3.494380427	1.417776743	9.423827138	-12.3459094	5.666185663
006	0.068461374	0.872267482	-2.119836793	67.01441583	-3.825798145	8.425791436	9.235185425	-3.541232916	5.827021981
008	-0.494614054	12.40561745	-3.082538445	66.82450084	-4.303888256	14.00370389	8.521286851	-3.366492253	5.869979517
200	-1.1689345	11.56901561	-4.282602063	75,10063715	-5.027043277	14.35560686	7.911103507	5.784898788	6.640676166
500	577LDD69CL-	22.62785663	-6.380807643	/1.5413/3/9	-5.826558976	7990017781	7.47643565	16.36463021	10.4391/401
006	-3.79787215 B1778767.5-	24. I912374b	-9.24871391	75,12422888	-8.391/5/643	18.83409722	29676701816	26.09911213	12. 16/11/645
400	-7.767332691	34,18915152	-6.972921938	94.82921622	-14.21864712	21.95470844	2.732821/6	41,68594532	13.1125118
000	-18.80000F	25.90377151	-4.892102872	298667 EDL	-21.98760984	23.91893108	-4.404862919	52.528U1/78	16.49400272
200	GID8288871-	43.667609	-5.413868771	133.0927.02	- lb. / 2/82633	46.0940831	-23.66630613	43.22460954	28.13609644
	-12.33313836	75.03824287	b./3/2/23b/	177.6133360	-4.002307242	/7.28004371	76069074781-	88.88470040	70.85283017
50	2.155393565	97,15076333	36,43766968	213.2653314	12.8290177	83,7037785	-7.874257696	117.7394876	115.6556322
Average deviation	-0.619901778	12.14179719	0.655706217	46.03953636	-3.000249422	19.38451339	4.074788978	12.94450014	8.961403944
Total deviation	2.014349588	22.62758677							

Figure B.1: Results SWASH verification average slope.

Wave co	indition 5
Offshore distance	Deviation Hm0 [%]
18000	0
16000	-2.625283117
14000	-3.420241081
12000	-4.667023905
10000	-5.709687208
8000	-7.675610486
6000	-6.208191528
4000	2.645048874
2000	2.227665288
500	4.816325743
Average deviation	-2.061699742

			Mild	slope, results (SWASH verificatio	n		
	Wave condition 1		Wave cond	dition 2	Wave cond	lition 3	Wave cor	ndition 4
Offshore distance	Deviation Hm0 [%] Deviatior	n infra [%]	Deviation Hm0 [%] D	eviation infra [%]	Deviation Hm0 [%] D	eviation infra [%]	Deviation HmO [%]	Deviation infra [%]
5750	0 -17.	.40270023	0	42.74334002	0	-11.34997177	0	-35.84504189
5500	-0.690541074 -18.	.07641599	-0.61445796	40.86563364	-0.464405958	-15.90798195	-0.773870109	-35.72159527
5250	-1.097587416 -18.	27860804	-1.057181375	35.39143884	-0.806562288	-17.12354956	-1.169281197	-33.52660442
5000	-1.449833644 -17.	.86788021	-1.663227043	36.21556069	-0.988509822	-15.28766743	-1.879272728	-33.52778597
4750	-1.972728185 -19.	94883317	-2.122287081	34.90911964	-1.291616025	-16.17303212	-2.524213691	-33.00915825
4500	-2.452043941 -20.	.00300928	-2.677320959	34.55671455	-1.624492568	-17.54815377	-3.064437454	-32.00154332
4250	-2.890409168 -20.	.16490472	-3.209479576	34.48615974	-1.968377693	-16.19225885	-3.756180072	-33.00457156
4000	-3.564611759 -20.	.09355137	-3.772182816	42.24008882	-2.505322148	-15.90597769	-4.3346859	-32.67818866
3750	-3.88927748 -18	8.2970311	-4.249037845	43.70752945	-2.752621024	-12.32481808	-5.271758575	-32.98033774
3500	-4.529737198 -12.	.93269966	-5.120435055	42.19985444	-3.140547394	-7.266366415	-6.184350821	-30.67548893
3250	-5.132683277 -10.	.01277818	-5.585267842	52.64224376	-3.770140031	-1.497083628	-7.002276665	-28.50684409
3000	-5.852912233 -3.3	310878827	-6.245618405	53.37778166	-4.30975919	3.797002086	-7.972463866	-24.3957464
2750	-6.490946604 -3.	91504719	-6.685929591	57.12577554	-4.877270526	5.828865623	-8.888193608	-18.63936553
2500	-7.034059785 3.	.98815294	-7.019670934	66.67282374	-5.341403931	11.67705689	-9.635579978	-12.80317904
2250	-7.544070554 12.	.08918116	-7.513909872	71.576551	-5.819116562	16.47466066	-10.55548498	-4.238058434
2000	-8.176536149 16.	70011377	-7.467513727	78.51374282	-6.343793207	20.48224205	-11.27468101	8.833245199
1750	-8.637341319 26.	21938261	-4.600195266	95.04198678	-6.896434817	21.82249973	-12.17094084	17.74766608
1500	-9.175675003	5.8999306	-4.705817322	100.5937324	-7.436671481	25.91194247	-13.24102871	31.7445993
1250	-9.802719605 44.	.78851396	2.564220553	91.75971278	-8.048149163	29.37924736	-14.08814289	46.64300683
1000	-12.20479315 41.	.67952838	4.389878508	94.91385032	-11.91684927	23.60835054	-12.90311976	61.90446365
006	-15.70412486 33.	.38275662	3.226382627	95.67499065	-17.20990319	12.02740304	-12.6894225	64.02691288
800	-14.63371459 29.	60589292	2.658196107	103.807469	-15.7952507	10.83635384	-13.81698088	63.58444113
700	-13.3553169 32.	.39075378	2.456460836	110.5510245	-16.65793496	12.19823747	-18.18466558	51.9476492
600	-15.26003954 38.	.46389391	2.452461904	121.758583	-18.88720426	14.88289782	-17.00732317	58.05741159
200	-18.48070111 43.	79275312	2.835552883	126.7343544	-21.94495199	16.09508141	-18.86833181	63.37480948
400	-21.90689753 48.	.28998806	4.728222915	149.9641419	-24.64681318	17.55300862	-23.4551088	73.46050172
300	-23.64623514 65.	.28378802	12.97199448	185.6392362	-25.86153959	23.9576865	-27.52777914	91.41710692
200	-20.2967285 73.	.80251949	28.78786532	210.3093789	-21.78182347	33.01108158	-26.44784806	114.1512056
100	0.361202259 91.	.80025946	67.93304369	251.6357486	-3.957296622	44.21661178	-11.40537855	130.7952015
Average deviation	-8.46589874 15.	.09907141	2.092922316	86.40029544	-8.518784864	6.799426489	-10.55492418	15.72878316
Total deviation	-5.501677043 31.	.00689412						

Figure B.2: Results SWASH verification mild slope.

			Stee	p slope, result	s SWASH verificat	ion		
	Wave conc	dition 1	Wave con	dition 2	Wave conc	lition 3	Wave con	dition 4
Offshore distance	Deviation Hm0 [%]	Deviation infra [%]	Deviation Hm0 [%]	Deviation infra [%]	Deviation Hm0 [%] D	eviation infra [%]	Deviation Hm0 [%]	Deviation infra [%]
575	0	27.57946071	0	59.96892535	0	17.36832674	0	45.8958658
550	0.235010082	27.02188302	-0.29092745	56.34089747	-0.22096033	15.01317857	0.901197157	43.9235221
525	-0.088010191	26.88079393	-0.265227771	51.65528472	-0.854960182	13.08889167	0.48841131	43.6702157
500	-0.531239661	24.44503641	-0.457083887	46.08012451	-1.085350622	10.35020505	0.444647084	44.8437424
475	-0.475501822	20.56885421	-0.907098699	43.02662761	-0.735339163	7.481279443	-0.33230916	44.4177488
450	-0.650645079	16.86139012	-0.887344947	42.25606106	-1.014621886	5.992183037	-0.066350734	42.3598315
425	-1.193717808	14.6142884	-1.421928807	45.94639803	-1.561201063	5.515069635	-0.804481844	38.7499687
400	-1.13486312	14.9002055	-0.982392738	49.09907251	-0.962073597	5.370776828	-0.39239655	34.5183089
375	-1.25695976	16.34151974	-1.614301672	50.19475955	-1.677059518	6.799864516	-1.142911663	34.8808988
350	-1.661934223	15.43524424	-1.93464515	49.37242273	-1.528487848	4.504508217	-0.795623083	36.5264076
325	-1.924435214	12.67286974	-1.858844297	47.75874451	-1.643486911	0.080320023	-1.638479845	36.6522438
300	-1.713105551	9.57188476	-2.672684458	46.01515561	-2.260720121	-3.167741808	-1.878299916	32.9480139
275	-2.60585214	8.585313626	-3.139719257	41.13262853	-1.992444654	-5.156945793	-2.352594561	31.6252956
250	-2.240366914	5.98506654	-3.237724752	31.06388302	-2.905152424	-7.661631157	-2.443849314	29.8935450
225	-2.965859586	-2.798236912	-4.773815369	26.45966549	-3.168409177	-12.41343844	-3.407124693	22.8055689
200	-3.546807579	-10.17606042	-4.954409716	39.19182249	-2.769935722	-17.71319823	-3.276622195	14.1701853
175	-4.179021759	-6.223187858	-8.179339928	54.24487733	-4.233810947	-17.43961334	-4.858985649	13.5344622
150	-4.84240184	5.444171416	-11.95114508	56.92306906	-4.685693488	-9.413463859	-4.289192248	22.9377518
125	-7.470611515	9.38073336	-10.09408229	58.8760161	-7.379920659	-4.418173918	-7.337419702	28.1159918
100	-11.19590491	1.865029621	-0.291162614	69.08932488	-13.09926297	-9.293405786	-9.310786384	28.636714
90	-14.60040821	-1.405836233	2.194503409	77.15361184	-16.60414159	-13.19826042	-11.5406219	28.7953599
08	-18.85184862	-2.992411736	4.817643148	88.22838178	-20.83472966	-15.59043629	-14.1571821	27.4809451
70	-15.55098264	6.02399745	8.66881859	100.1321872	-16.14000597	-8.664853117	-17.56004243	27.0106690
60	-9.549131607	18.99219069	10.8828261	105.453324	-12.45169217	-3.244673187	-15.37093738	34.2455632
50	-4.113361223	27.16932999	13.6555903	108.7719339	-9.688925582	-1.019352275	-8.741932965	50.505072
40	0.32773952	27.8187274	18.75409129	119.1295825	-4.891591207	0.644141164	-1.270487727	63.382224
30	7.916315949	32.50749405	33.54154762	165.1164359	3.795505735	12.58675597	7.325560299	71.7754283
Average deviation	-3.846811312	12.85443525	1.207449688	64.02523028	-4.836832286	-0.874062473	-3.84477097	36.0852424
Total deviation	-2.83024122	22.2312937						

Figure B.3:
Results
SWASH
verification
steep
slope.

Average deviation	1125	1250	1375	1500	1625	1750	1875	2000	2125	2250	2375	2500	2625	2750	2875	3000	Offshore distance	Wave co
-0.934377042	-2.748461291	-1.840318198	-1.913101504	-1.564056525	-1.422283847	-1.213924024	-1.090284518	-1.012708639	-0.83600298	-0.6046065	-0.408143851	-0.246539031	-0.129179	0.019644591	0.059932647	0	Deviation Hm0 [%]	ndition 5

		C	_x = 1.7;	: -9 m + 0	D depth	contou	r			
Pe	eak period [s]	8	9	10	11	12	13	14	15	16
Chin 4	Value design tool	2.3	1.9	1.7	1.5	1.4	1.2	1.2	1.1	1
Ship I	Error	13.04348	36.84211	52.94118	73.33333	71.42857	83.33333	75	81.81818	90
chin a	Value design tool	2.3	1.9	1.7	1.5	1.4	1.2	1.2	1.1	1
Ship 2	Error	13.04348	36.84211	52.94118	73.33333	71.42857	83.33333	75	81.81818	90

Figure B.4: Results downtime prediction verification at -9 m + CD, applying $C_x = 1.7$.

		C_	_x = 1.7;	-11 m +	CD dept	h contou	ır			
Pe	eak period [s]	8	9	10	11	12	13	14	15	16
Chin 4	Value design tool	2.3	1.9	1.7	1.5	1.4	1.3	1.2	1.1	1
Ship I	Error	13.04348	36.84211	52.94118	73.33333	71.42857	69.23077	83.33333	81.81818	90
Chin 3	Value design tool	2.3	2	1.7	1.6	1.4	1.3	1.2	1.1	1.1
Ship 2	Error	13.04348	30	52.94118	62.5	71.42857	69.23077	83.33333	81.81818	72.72727

Figure B.5: Results downtime prediction verification at -11 m + CD, applying $C_x = 1.7$.

			Ship	1, -9 m -	+ CD			
Сх	DMA	1	1.1	1.2	1.3	1.4	1.5	1.6
8	2.6	39.53488	33.33333	25.71429	18.75	13.33333	3.703704	-4
9	2.6	31.57895	23.52941	13.33333	0	-13.0435	-18.1818	-30
10	2.6	16.12903	3.703704	-8.33333	-18.1818	-23.8095	-36.8421	-44.4444
11	2.6	0	-8.33333	-18.1818	-30	-44.4444	-52.9412	-62.5
12	2.4	0	-14.2857	-20	-33.3333	-50	-50	-60
13	2.2	0	-10	-22.2222	-37.5	-46.6667	-57.1429	-69.2308
14	2	0	-11.1111	-25	-33.3333	-42.8571	-53.8462	-66.6667
15	2	-5.26316	-11.1111	-33.3333	-42.8571	-53.8462	-66.6667	-81.8182
16	1.9	-11.7647	-18.75	-35.7143	-46.1538	-58.3333	-72.7273	-72.7273

Figure B.6: Results down time prediction verification at -9 m + CD for ship 1, applying C_x = 1.0 to 1.6.

			Ship	2, -9 m -	+ CD			
Сх	DMA	1	1.1	1.2	1.3	1.4	1.5	1.6
8	2.6	39.53488	33.33333	25.71429	18.75	10.34483	3.703704	-13.0435
9	2.6	31.57895	23.52941	13.33333	0	-23.8095	-18.1818	-36.8421
10	2.6	13.33333	0	-8.33333	-18.1818	-30	-36.8421	-52.9412
11	2.6	0	-8.33333	-18.1818	-30	-44.4444	-52.9412	-73.3333
12	2.4	0	-14.2857	-20	-33.3333	-41.1765	-60	-71.4286
13	2.2	0	-10	-22.2222	-37.5	-46.6667	-57.1429	-83.3333
14	2	-5	-16.6667	-23.5294	-40	-61.5385	-75	-75
15	2	-11.1111	-17.6471	-33.3333	-42.8571	-53.8462	-66.6667	-100
16	1.9	-11.7647	-18.75	-35.7143	-46.1538	-72.7273	-72.7273	-90

Figure B.7: Results down time prediction verification at -9 m + CD for ship 2, applying $C_x=1.0$ to 1.6.

			Ship	1, -11 m	+ CD			
Сх	DMA	1	1.1	1.2	1.3	1.4	1.5	1.6
8	2.6	39.53488	33.33333	25.71429	18.75	13.33333	3.703704	-4
9	2.6	31.57895	23.52941	13.33333	0	-8.33333	-18.1818	-23.8095
10	2.6	16.12903	3.703704	-4	-13.0435	-23.8095	-30	-44.4444
11	2.6	3.703704	-8.33333	-18.1818	-30	-36.8421	-44.4444	-62.5
12	2.4	4	-9.09091	-20	-26.3158	-41.1765	-50	-60
13	2.2	0	-10	-15.7895	-29.4118	-37.5	-46.6667	-57.1429
14	2	-4.7619	-15.7895	-29.4118	-37.5	-46.6667	-57.1429	-69.2308
15	2	-15.7895	-29.4118	-37.5	-46.6667	-57.1429	-69.2308	-83.3333
16	1.9	-16.6667	-31.25	-50	-50	-61.5385	-75	-90.9091

Figure B.8: Results downtime prediction verification at -11 m + CD for ship 1, applying $C_x = 1.0$ to 1.6.

			Ship	2, -11 m	+ CD			
Сх	DMA	1	1.1	1.2	1.3	1.4	1.5	1.6
8	2.6	40.90909	33.33333	27.77778	21.21212	13.33333	7.142857	-13.0435
9	2.6	31.57895	23.52941	13.33333	3.703704	-8.33333	-18.1818	-30
10	2.6	16.12903	7.142857	-4	-13.0435	-23.8095	-36.8421	-52.9412
11	2.6	3.703704	-4	-13.0435	-23.8095	-36.8421	-44.4444	-62.5
12	2.4	4	-4.34783	-14.2857	-26.3158	-41.1765	-50	-71.4286
13	2.2	4.347826	-4.7619	-15.7895	-29.4118	-37.5	-46.6667	-69.2308
14	2	-4.7619	-15.7895	-22.2222	-37.5	-46.6667	-57.1429	-83.3333
15	2	-10	-22.2222	-37.5	-46.6667	-57.1429	-69.2308	-100
16	1.9	-10.5263	-23.5294	-40	-50	-61.5385	-90.9091	-90.9091

Figure B.9: Results down time prediction verification at -11 m + CD for ship 2, applying C_x = 1.0 to 1.6.

Appendix C

Modelling of moored vessel responses

Determining the downtime of a ship based on the offshore wave conditions is a long process that consists of a number of steps. These steps are connected to each other in the sense that the output of a previous step is used as input for the following step. Generally the calculation process consists of the following phases:

- 1. Translate the offshore wave conditions to nearshore wave conditions. As the depth decreases towards the shore, the waves start to 'feel' the bottom which leads to a change in wave conditions. Processes that play a role during onshore wave propagation were discussed in section 3.3. Especially important in this phase is the ability of a calculation method or model to account for the occurrence of (bound) long waves and inhomogeneities in the coastal profile (e.g. alongshore variations in depth due to dredged channels, the presence of breakwaters and other hydraulic structures).
- 2. Translate the nearshore wave conditions to wave forces on the ship (wave-body interaction). This can be done using two types of methods: strip theory methods and panel methods. Section C.2 elaborates on strip theory methods since the DMA in section 5.4 uses lf-strip to determine the wave forces on the ship.
- 3. Perform a dynamic analysis during which the equations of motion for the moored vessel are solved. The result of this analysis consists of the vessel motions, line and fender forces. Based on criteria for maximum allowable vessel motions and mooring forces, the time during which (un)loading is impossible can be determined. Section C.3 treats ANYSIM as an example for DMA software, since the DMA in section 5.4 is performed using ANYSIM.

C.1 Modelling of nearshore wave propagation

In order to design an offshore harbour it is important to have accurate information about the prevailing wave conditions in a certain region. For this purpose wave modelling can provide a solution. In this appendix a number of models that can be used for this purpose are investigated. First some general information about wave model classes is given. The following two sections discuss two general types of models: phase averaging models and phase resolving models. Advantages and disadvantages of each model type are given.

C.1.1 Model classes

Various models exist to model and predict wind wave physics. A model that is applicable to all situations would be very expensive, which is why a variety of models have been proposed that can be used in specific situations. The relative importance of various physical processes plays an important role in the choice of such a model. Wave models can be divided into two general classes [46]:

- Phase resolving models, which predict both the amplitude and phase of individual waves.
- Phase averaging models, which predict average quantities such as the spectrum or its integral properties (H_{m0} , f_p , etc.). Modern phase averaging models are also able to predict the spatial and temporal evolution of the directional spectrum. Phase averaging models solve a single basic equation, usually of the form:

$$\frac{\partial F}{\partial t} + \mathbf{C}_{\mathbf{g}} \times \nabla F = S_{tot} \tag{C.1}$$

In which $F = F(f, \theta; x, y, t)$ is the directional spectrum, C_g is the group velocity vector and S_{tot} is the source term, which is typically represented as the summation of the atmospheric input, quadruplet nonlinear interactions between spectral components and dissipation by white-cap breaking [46].

If phase averaged properties vary rapidly (in the order of a few wavelengths), then a phase resolving model will be necessary. If wave properties vary slowly (on a scale of many wavelengths) then phase averaging models are sufficient. Phase averaging models can be very efficient as they require limited computational time and can be applied to large modelling areas. Battjes (1994) concluded that phase resolving models should only be used when they are strictly needed, as they are computationally much more demanding. Of the processes considered in table 3.6, only diffraction and triad interactions require phase resolving models, which means that phase resolving models are more useful to consider wave-structure interactions and the nearshore zone [46]. Generally a phase averaging model is used to transform the offshore wave climate to a nearshore wave climate, as it is able to model wave propagation over a large area with relatively small effort. When it comes to wave modelling in and around a harbour, usually a phase resolving model is used. The computational area can be smaller, which means that accurate results can be found without long computational time.

C.1.2 Phase averaging models

Spectral models

An example of a phase averaging model class is spectral models. Models of this class compute the wave energy distribution over large areas. An equation describing the transport of spectral wave energy in physical space (x and y) and in spectral space (frequencies and directions) is solved. Spectral models are based on a wave energy balance. When the influence of currents is taken into account, a wave action balance has to be used. Phase-averaged quantities, such as the 2D wave energy density spectra, are calculated which can be used to derive statistical quantities, such as T_p and H_{m0} . Phase information is not included, which means that time information describing wave components or wave crests and troughs is not resolved [9].

Wave propagation is calculated according to linear theory. Spectral models also calculate influences on and changes in wave energy. Spatial propagation of wave energy across the computed area, atmospherical input, white-capping and depth-induced breaking can be included. The generation of low-frequency wave motions and diffraction however can not be included. Spectral models are mainly used to hindcast the wave field in entire ocean basins and to transform an offshore wave field to a nearshore area in open coastal areas [9].

The advantages of spectral models are [9]

• Large modelling areas can be handled in relatively short computational times, which makes spectral models computationally efficient.

• Wave information that is suitable for many wave studies can be produced, e.g. directional wave spectra, wave parameters such as H_{m0} , T_p and main directions.

The disadvantages of spectral models are [9]:

- Individual waves are not computed, which means that phase resolving time/space information is not included.
- Only resonant second-order nonlinear interactions are included in the model, which means that both bound and free long waves are not included.
- Due to the absence of phase information the evolution of the energy levels of free low frequency waves can only be estimated, with the same limitations as for primary waves.
- Certain limitations apply to the steepness of the bottom slope.

Three generations of spectral models exist. First generation spectral models only calculate the evolution of wave parameters, such as the evolution of H_{m0} and T_p . This results in an extremely fast calculation which is comparable to a rule of thumb estimate. The present-day applicability is limited, but first generation models can still be used for a quick estimate. Second generation spectral models consider a parameterised wave height distribution and apply a specific prescribed spectral shape. Third generation spectral model discretise both the direction and the frequency space, which means no a-priori spectral shape is prescribed. This means that wave-wave interactions can be included [9].

An example of a third generation spectral model is SWAN (acronym for Simulating WAves Nearshore). SWAN can be used to obtain realistic estimates of wave parameters in coastal areas from given wind, bottom and current conditions. The model is based on the wave action balance. A time dependent and a steady state mode are available. Wave propagation processes represented in SWAN include: propagation through geographic space, bottom- and current-induced shoaling and refraction, blocking and reflections by opposing currents. Diffraction can only be approximated, which means that the reliability of the results depends on the complexity of the modelled area. Transmission through, blockage by or reflection against obstacles can be defined separately. Wave generation, wave-wave interactions (quadruplets and triads) and dissipation (depth-induced, bottom friction and white-capping) are also included in SWAN. Furthermore, wave-induced set-up of the mean sea surface can be determined [9].

Shallow-water model with forcing on primary wave group scale

Models of this class compute the effect of wind waves on low frequency wave motions in large coastal areas. A wave driver model which provides forcing on the primary wave group scale is used for the calculation of the generation terms for low frequency waves. This model is combined with a shallow-water model that is used to calculate the generation and propagation of low frequency waves. Low frequency waves are essentially shallow-water waves, which means that the propagation and generation of this type of waves can be described by the shallow water equations. Both bound and free long waves can be solved in the time domain, meaning that phase information is available for that type of waves. Individual primary wind waves however are not resolved, as the wind wave energy is calculated in bulk. This means that no phase information is available for primary waves [9].

The advantages of this model type are a relatively small calculation time and an efficient simulation of low frequency wave motions, especially in the surf zone [9]. However, the modelling of wind waves is not very detailed in the sense that:

• All primary wave energy propagates with the same group speed and along pre-calculated wave rays.

- Diffraction is not included, as well as frequency dependent refraction, dispersion and shoaling for primary waves.
- Empirical models are used for energy dissipation due to wave breaking.

An example of this model type is SURFBEAT. This model is part of the Delft 3D modelling package of Deltares. SURFBEAT uses the SWAN model to derive the wave rays. Moreover, it uses an additional wave driver model to calculate spatial and temporal changes in the primary wave energy (on the scale of wave groups). With these changes in primary wave energy changes in radiation stresses can be calculated, which can be used as the forcing term for the calculation of low frequency waves [9].

C.1.3 Phase resolving models

Different types of phase resolving models exist. These can be divided based on the following characteristic values [46]:

- The nonlinearity, which is measured by ak or a/h.
- The relative depth parameter *kh*.
- The bottom slope s_{bottom} .

A number of model types are discussed in this section, that can be distinguished based on the assumptions made about the above three characteristic values.

Free surface potential flow models

Free-surface potential flow models compute the wave dynamics in small areas of arbitrary shape using a 3D velocity potential formulation. Models of this class impose no restrictions on water depth and can be used for various purposes, e.g. ship-wave interaction, landslide problems and harbour resonance problems. These models require a relatively large computational effort and are therefore rarely used in coastal engineering. Many other model types consider simplified forms of free surface potential flow models. A linear potential flow model can be used to allow an independent and very efficient modelling of wave components in the frequency domain. Nonlinear wave propagation requires a nonlinear potential flow model, which is a more complex approach that requires even longer computational times. Nonlinear wave modelling with this type of models is in principle restricted to non-breaking waves, as a flow dissipation mechanism is absent due to the irrotational flow assumption. However, ways to include the effect of wave breaking exist [9].

The absence of restrictions on bathymetry (e.g. no criterion for the maximum bed slope) and the possibility to incorporate a ship are the main advantages of this type of model. The main disadvantages are the fact that only small areas can be included and the relatively long computation times [9].

Mild slope models

Mild slope models efficiently compute low-amplitude waves in areas of moderate size, which can have a complex shape. Mild slope models are based on linear potential flow models for independent wave components. A 2D potential flow problem is derived from the 3D velocity potential via an assumed vertical velocity profile, which is equal to the vertical structure of flow under linear Airy waves. As the vertical velocity profile is known, the dependence in vertical direction can be eliminated which reduces the 3D potential flow problem to a 2D problem. In order to apply this simplification, gradual changes in bottom have to be assumed, which means: $|\nabla h|/kh \ll 1$. Wave spectra can be approximated with the mild slope equation even though it only describes the propagation of harmonic waves. This can be done by superposition of wave components with different frequencies and directions [9].

The computational approach of mild slope models results in a very efficient computation. Dispersion, shoaling, refraction and diffraction all can be included. Reflection and transmission can be included in the form of special boundary conditions. Nonlinear dissipation effects such as bottom friction, wave breaking and entrance losses can be included by means of parameterised formulations[9] [46].

The mild slope equation as developed by Berkhoff reads [46]:

$$\nabla \times (CC_g \nabla \phi) + k^2 CC_q \phi = 0 \tag{C.2}$$

Mild slope equations do not pose any restrictions for the kh-value of waves, which means that short waves can be modelled in deep water without loss of accuracy. However, due to the assumed vertical structure of the flow, other limitations are posed which are equivalent to the limitations of linear wave theory. The nonlinearity cannot be too large $(a/h \ll 1)$ [9].

The mild slope equation described above is an elliptical partial differential equation that requires specification of boundary condition around the full computational domain. If reflection of waves propagating in the dominant direction can be neglected, a parabolic approximation can be made for the mild slope equation [9] [46]:

$$\frac{\partial \phi}{\partial x} = \left[ik - \frac{1}{2kCC_q}\frac{\partial}{\partial x}\left(kCC_q\right)\right]\phi + \frac{i}{2kCC_q}\frac{\partial}{\partial y}\left(CC_q\frac{\partial \phi}{\partial y}\right) \tag{C.3}$$

The x-axis is chosen in the principal direction of wave propagation. Due to the assumption of a principal wave direction, parabolic models are less versatile than elliptic mild slope models. However, parabolic models can be solved relatively fast as waves travelling in the other direction are neglected. Numerous extensions of this basic form have been developed, that can include the effect of currents, curvilinear coordinates, wind input, nonlinear dispersion equation, dissipation due to wave breaking and bottom friction etc. In this way the range of application of mild slope equation models can be extended [9] [46].

Atmospheric input is usually not included in mild slope models. Mild slope models were originally developed for wave penetration in harbours. They can however also be used in other coastal areas, such as open coastal regions, coastal inlets, bays and around islands [46].

The main advantages of mild slope models are that they are suitable for complex geometries and that the computations are very fast [9]. Disadvantages are:

- Because mild slope models are linear, nonlinear wave-wave interaction (set-down) is not included. The application is restricted to small wave heights.
- Bed slopes need to be sufficiently small.
- The application is limited to small areas of areas where atmospheric input is not dominant as wave generation by wind is not included.

An example of a mild slope model is PHAROS (Program for HARbour OScillations). PHAROS calculates the numerical solution of the elliptic mild slope equation. Depth- and current-induced refraction and diffraction are both included. Dissipation by bottom shear stress can be modelled based on a quadratic friction law. Dissipation by wave breaking is calculated using the formulation of Battjes and Janssen. By applying several types of boundary conditions, partial reflection and combined reflection and transmission by structures can be described. An incoming wave signal can be prescribed at a model boundary, while a reflected wave can still pass that boundary and move out of the domain. By combining the output of multiple computations the effects of directional spreading (short-crested waves) and multiple wave periods (a spectrum) can be taken into account. Long-wave resonance and seiching can be studied by carrying out computations for a large number of long wave periods [9].

Boussinesq-type models

Boussinesq-type models compute the wave dynamics by approximating the full 3D free-surface flow equations through a polynomial expansion of the horizontal velocity in the vertical direction, assuming irrotational flow in the vertical plane. Under the assumption of zero horizontal vorticity, the polynomial expansion can be expressed in terms of surface elevation and a single horizontal velocity variable, resulting in a reduction of the 3D model equations to 2D equations. An simple example of a polynomial expansion of the vertical velocity profile in Boussinesq-type models can be found in the figure below [9].



Figure C.1: Polynomial expansion of the vertical velocity profile in Boussinesq-type models. [9]

Models of this class are applied in areas of moderate size and of limited depth. The 2D form of the equations as developed by Peregine (1967), consisting of the depth integrated continuity equation and the horizontal momentum equation, reads as follows [46]:

$$\frac{\partial \eta}{\partial t} + \nabla \times \left[(h + \eta) \, \bar{u} \right] = 0 \tag{C.4}$$

$$\frac{\partial \bar{u}}{\partial t} + \bar{u} \times \nabla \bar{u} + g \nabla \eta = \frac{h}{2} \nabla \left[\nabla \times \left(h \frac{\partial \bar{u}}{\partial t} \right) \right] - \frac{h^2}{6} \nabla \left[\nabla \times \frac{\partial \bar{u}}{\partial t} \right]$$
(C.5)

Boussinesq-type models can be used for shallow-water conditions with relatively small values of kd, usually kh < O(3). This means that long period waves in shallow water can be accurately described by this model class. Moreover, restrictions on the wave steepness and bottom steepness also apply. More advanced models have less stringent restrictions due to the inclusion of many higher-order terms. Due to the large number of possible assumptions and approximation techniques, many different Boussinesq-type wave models have been developed in the last few decades. The Boussinesq-type equations include the effects of diffraction, refraction, reflection and wave-current interaction. Boussinesq-type models are particularly successful in predicting the evolution of higher harmonics during shoaling [9].

The main advantages of Boussinesq-type models are [9]

- Nonlinear wave-wave interaction is included automatically via the described physics.
- Primary and low frequency waves are described simultaneously, including (relative) phases.
- Rotational effects such as wave breaking can be included by suitable modelling.

The main disadvantage of this model class is that a relatively large computational effort is required. If one wants to reduce the computational effort the computation has to remain limited to relatively small areas. Furthermore, the application of this model type is limited to moderate water depths and wave generation by wind cannot be included [9].

An example of a Boussinesq-type wave model is MIKE21 BW. MIKE21 BW is based on the numerical solution of the enhanced Boussinesq equations formulated by Madsen and Sørensen (1992). MIKE21 BW is applicable for kh < O(3), which means that the maximum depth to wave length ratio is equal to h/L < 0.5. The effects of refraction, diffraction, wave breaking (based on a roller formulation), bottom friction, wave run-up and run-down (moving shoreline), partial transmission and reflection and nonlinear wave-wave interaction of short-crested and long-crested waves propagating over complex bathymetries can all be included. Moreover, phenomena such as wave grouping, generation of bound long waves and surf beats can be modelled. Both reflecting and non-reflecting open boundaries can be modelled. Furthermore, internal wave generation can be included which means that it is possible to generate unidirectional and directional waves. MIKE21 BW can be used to calculate and analyse short- and long-period waves in harbours and coastal areas [9].

Multi-layer models

Multi-layer models compute the wave dynamics by solving the full 3D free-surface flow equations for a number of discrete horizontal coupled layers, under the assumption that the position of the free surface can be described by a height function which represents this position with a single value. Each layer describes the vertical velocity-distribution. A non-hydrostatic correction to the shallow-water model is added (which is why these models are often called non-hydrostatic wave models) and the numerical scheme is applied for each layer. The full 3D free-surface flow equations are solved, which means no irrotational flow has to be assumed. However, wave breaking is still excluded because the free surface is represented by a height function, which described the position of the water surface by a single value. This means that complicated free-surface behaviour such as plunging breakers or overturning waves cannot be described. A dissipation mechanism can be added to describe wave breaking, which can be more elaborate than in a Boussinesq-type model due to the presence of multiple layers [9].

The main advantages of multi-layer models are that all nonlinear flow processes can be modelled and the accuracy can be increased by simply increasing the number of layers. All elements relevant for describing wave dynamics can be included. The disadvantage is that a large computational effort is required. If one wants to reduce the computational effort, the computation should be limited to small areas or moderate water depths [9].

An example of a non-hydrostatic multi-layer model is SWASH, which is an acronym of Simulating WAves till SHore. SWASH uses as its starting point the nonlinear shallow water equations (NLSW equations) to compute the surface elevation and currents. These equations include a non-hydrostatic pressure term, which enables the modelling of frequency dispersion. Various wave processes can be included, such as shoaling, refraction, diffraction and dispersion. Furthermore, wave breaking can be included by treating them as discontinuities in the flow where mass and momentum conservation still apply. The amount of energy dissipated by turbulence in the broken wave can be accounted for through an analogy with hydraulic jumps. Partial wave reflection and transmission, wave run-up and run-down and wave induced currents can also be included. Moreover, SWASH is also able to model wave damping induced by aquatic vegetation. With just two equidistant layers, accurate results (1% error) can be obtained up till $kh \approx 3$ [47].

Full free-surface Navier-Stokes models

Full free-surface Navier-Stokes models, also known as fully nonlinear viscous models are only suitable for relatively small scale phenomena. Therefore they are not suitable for practical applications in coastal engineering. However, these models can be used to consider detailed interaction between ship and waves. For example, the impact of green water can be investigated [9].

C.2 Modelling of wave-body interaction

Calculation of wave forces on moored vessels based on the wave conditions near the berth can be done in various ways. In this section strip theory methods will be discussed. The theoretical background of this model is explained and lf-strip is given an example of a modelling program. Strip theory is a slender body theory that is based on the potential flow theory. The ship is considered to be a rigid body, floating in the surface of an ideal fluid. The strip theory is based on linearity. The moored ship is divided into a finite number (usually 20 to 30) of crosssectional slices or strips. The shape of each slice closely resembles the segment of the ship that it represents. Each cross-sectional strip is treated as if it were part of an infinitely long floating cilinder, as displayed in the figure below. Hence, the fluid flow is considered to be entirely underneath the body. The flow past the ends of the ship is neglected. This means that all the waves that are produced by the oscillating ship (hydromechanic loads) and the diffracted waves (wave loads) are assumed to travel perpendicular to the middle line (parallel to the (y,z)plane of the ship). The fore and aft side of the body are not supposed to produce any waves in the x-direction. This assumption reduces the application to slender and long bodies only. Experiments have shown that strip theory can be applied successfully for floating bodies with a length to breadth ratio larger than three: $L_s/W_s \geq 3$. Furthermore, only wave lengths that are short compared with the ship length can be considered [19] [39].



Figure C.2: Strip theory representation by cross-sections. [19]

The forces on the floating body are determined using the results from two-dimensional potential theory. The two-dimensional hydromechanic coefficients and exciting wave loads are calculated for each cross-sectional strip. This can be done based on the theory of Ursell, who has derived an analytical solution for solving the problem of calculating the hydrodynamic coefficients of an oscillating circular cylinder in the surface of a fluid. To obtain the three-dimensional values, these 2D values have to be integrated over the ship length numerically. Finally, the equations of motion for the ship are solved to obtain the vessel motions. These calculations are performed in the frequency domain. Strip theory can be applied for a ship with zero velocity and with a nonzero forward velocity. In the first case, interactions between the cross-sectional strips can be ignored [19] [39].

Because strip theory is a slender body theory, the results of this method are less accurate for ships with low length to breadth ratios. Furthermore, problems can occur when calculating roll motions at resonance frequencies because viscous effects are neglected. Inaccuracies can also be expected when parts of the ship go out of or into the water or when green water is shipped, as the effect of the above water part of the ship is fully neglected in the strip theory. Due to linearity the vessel motions have to stay small relative to the cross-sectional dimensions of the ship. Although far more sophisticated fully 3D methods have been developed, strip theory is still used successfully, especially in early ship design [19] [39].

An example of a calculation method that computes the wave forces on the ship based on strip

theory is the SURFBEAT / LF-STRIP approach. This can be used if short wave effects can be neglected and only the response due to long waves has to be treated. Only the low-frequency near-resonant response of the ship (surge, sway and yaw) is calculated, taking into account long wave propagation and nonlinearities associated with the propagation of wave groups over an uneven bottom. The response in other modes is also calculated, but is of minor importance. The long waves are calculated in the time-domain using Delft3D/SURFBEAT, that uses the classical shallow water equations and considers the directionally spread short-wave energy associated with wave groups. The calculated surface elevations and particle motions at the locations of a number of cross-sections of the ship's hull are used as input for the program LF-STRIP. This is a Matlab function (lfstrip) that calculates the wave forces on the ship in the six modes. This can be used as input for a moored ship simulation to determine the vessel motions and mooring forces. Because no phase information is required for short waves, the grid can be based on long wave motions only, resulting in a coarser grid and a faster and cheaper calculation method [40] [38].

C.3 Dynamic mooring analysis

Before discussing the modelling software that is available to perform mooring analyses, background information on mooring analysis methods is given in this section. Mooring analysis is performed to ensure that the general arrangement of mooring points and fenders is adequate for the intended sizes and types of vessels in the most extreme mooring environment at the location. This has to be done for two reasons. First of all, sufficient mooring points and fenders have to be provided and it has to be ensured that these have sufficient strengths and are located at suitable locations for all vessel sizes. Secondly, it must be ensured that vessel motions are not excessive. Especially at manifolds, gangways and ramps this has to be ensured. During a mooring analysis the forces and moments applied to the vessel by the environment and other factors are balanced against the reaction forces exerted by mooring lines and fenders [11]. Generally two types of mooring analyses exist: static and dynamic mooring analysis.

C.3.1 Types of mooring analysis

Static mooring analysis

Static mooring analysis can be applied to most cases of wind and current loading where vessel motions are not important. In this case, two force equilibriums and one momentum equilibrium have to be solved (only considering forces in the horizontal plane) [11]:

$$\Sigma F_x = 0; \Sigma F_y = 0; \Sigma M_{xy} = 0 \tag{C.6}$$

Both horizontal and vertical mooring line angles have to be taken into account in order to come to a realistic analysis. Furthermore, stiffness differences between the various mooring lines and the stiffness behaviour of the lines (linear / nonlinear) also have to be considered.

Dynamic mooring analysis

Dynamic mooring analysis (DMA) is performed when vessel motions may significantly contribute to mooring line loads. This is mostly the case when wave action is present. DMA is usually performed when the following thresholds are exceeded [11]:

- Large moored vessels directly exposed to ocean waves with significant wave heights larger than 1.0 m and peak wave periods exceeding 6 s.
- Small moored vessels exposed to ocean and/or local waves with significant wave heights larger than 0.5 m and peak wave periods exceeding 3 s.

• Moored vessels exposed to harbour seiche and long period waves, especially those at jetties or piers.

If the excitation from environmental conditions is large enough, vessels moored to any type of mooring will experience dynamic loads and vessel motions that cannot be calculated by static mooring analysis. A dynamic analysis is then required to obtain accurate and safe results on the dynamic loads and motions [11]. Dynamic motions of a moored vessel are governed by Newton's second law, which can be summarized as follows [11]:

$$(m+a)\ddot{x} + b\dot{x} + cx = F(t) \tag{C.7}$$

This equation is quite difficult to solve due to a number of complications [11]:

- As mentioned in section 3.2.1, a ship can respond in six modes of motion. Each mode requires six separate equations. These motions can also be coupled, which implies that one mode will produce motion in another mode, which requires a simultaneous solution of all equations.
- The vessel accelerates water as it moves dynamically, which means that the ship behaves as if it has an additional mass *a*. This added mass is different for each mode of motion and varies according to the frequency of ship oscillation. A lower frequency motion (i.e. a longer period) causes more water to be accelerated. Furthermore, a decreasing under-keel clearance also increases added mass values.
- Damping includes the forces that resist dynamic amplification. High-frequency motions are mainly damped by waves that are created by the vessel as it moves through the water. Low-frequency motions are mainly damped by the drag force on a ship's hull.
- Mooring elements such as lines, fenders and chains, tend to have a nonlinear behaviour as the force-deflection curve of these elements has a hyperbolic shape rather than a straight line.

Overall, when performing a DMA one usually uses two models. The first one is a hydrodynamic model that computes vessel added mass coefficients, damping coefficients and wave forces. Most hydrodynamic models employ finite element or panel methods in order to calculate the hydrodynamic coefficients of added mass, radiation damping and first- and second-order wave loading. The second model is a mooring dynamics model that computes vessel motions and mooring line/fender forces by solving the equations of motion given above. This means that a time domain version of the equations of motion is integrated through time. Arbitrarily varying waves, winds and currents can be simulated in the time domain. Frequency domain analysis is also possible but not preferred, as it cannot simulate the complicated motions of single point mooring systems, where the applied forces depend on the vessel heading. Furthermore, the non-linear behaviour of mooring lines and second-order drift forces cannot be simulated [11].

C.3.2 ANYSIM

ANYSIM is presented in this appendix as an example of DMA software. ANYSIM is mainly aimed at simulating complex systems in the time domain. Nowadays three or more vessels can commonly operate simultaneously, leading to interaction between the various structures and ships in a harbour by means of mechanical links (lines, fenders, etc.), hydromechanical coupling (diffraction and radiation) and shielding effects (wind and currents). This results in a complex system with basically nonlinear characteristics. ANYSIM performs a N body simulation for a system with several floating and dry bodies and solves the equations of motion for a 6N degrees of freedom system. ANYSIM can perform both a deterministic and stochastical approach. ANYSIM is able to simulate the coupled motion behaviour of multiple floating bodies. Effects of mooring systems and hydrodynamic interactions can be included. Mooring systems in both deep and shallow water can be simulated [22] [23].

Different types of simulations can be performed. Ships moored to a jetty, quay or gravity based structure can be simulated. Furthermore, ships under dynamic positioning control can also be modelled. Dynamic positioning control means that the position and heading of a vessel are automatically maintained by using the propellers and thrusters of the vessel. A side-by-side simulation involving two or more vessels is also possible. Moreover, ANYSIM is able to simulate free hanging loads hanging at the cranes of a crane vessel without hoisting action [22] [23].

The equations of motion on which ANYSIM is based cope with the frequency dependency of the added mass and damping coefficients of all bodies. All N bodies modelled in ANYSIM are (optionally) subject to wave induced forces, wind loads and hydrodynamic reaction forces. First order wave forces and slowly varying wave drift forces can be included, as well as current and wind loading. Interaction forces between rigid bodies can be linear or nonlinear. Additional arbitrary forces can also be added. Arbitrary hull shapes are allowed and all transformations done in ANYSIM are still valid for large rotation angles [22] [23].

Appendix D

Screenshots of the design tool

In the final appendix, several screenshots of the design tool are presented. Four screenshots can be found in this appendix:

- The first screenshot shows the 'Front' sheet of the cost module.
- The second screenshot shows some intermediate results of the wave calculation module. The intermediate results presented are the short and long wave heights at the berth as calculated for wave condition 1 (table 4.9).
- The third screenshot shows a part of the input screen in the cost module. In this screen, the user is able to insert his or her input parameters.
- The fourth screenshot shows the intermediate results for a transmission calculation as carried out for wave condition 1.



Figure D.1: Screenshot of the 'Front' sheet of the cost module.



types are plotted as a function of x_{ship} , for wave condition 1. The different lines show different breakwater lengths. Figure D.2: Intermediate results of the wave calculation module. In the graphs displayed, the short and long wave height behind the two breakwater

Grid parameters			Vessel and mooring pro	erties	
Onshore boundary	0	F	Length Over All	315 m	
Offshore boundary	8000	F	Width	е 20 3	
Step size	2	F	Draught	12 E	
Number of elements (max. 500)	239	с К	Displacement (mass)	148,700,000.00 kg	
			Mooring arrangement ship	perpendicular to coast	
Environmental parame	ters		Length spring line	45 m	
Bed slope	0.0100000		Diameter spring line	0.044 m	
Soil type	sand / clay		Stiffness spring line	4.12E+10 N/m ²	
Underwater slope dredged soil	0.333333		Orientation approach channel (outbound)	190 deg	
Highest Astronomical Tide	-	n+MSL	Vessel speed during tugging	2 mls	
Lowest Astronomical Tide	7	n+MSL	Time required to tie up tugboats	600 s	
Wind speed	0	nis	Manoeuvring characteristics	moderate	
Wind direction (coming from)	225	deg	Sailing speed	8 - 12 kts moderate	ø
Gravitational acceleration	9.81 r	Tris"	Aids to Navigation	moderate	
Current speed	0.3	nis			I
Current direction (going to)	8	deg	OPEX calculation		
Crientation coastal normal	185		DPEX % of CAPEX: jaits	18 %	
			OPEX % of CAPEX; caisson BW	×	
Unit costs			OPEX % of CAPEX: rubble mound BW	2 %	
Cost per m² dredging	20	JSD/m"	OPEX % of CAPEX: dredging	0.3 %	
Unit cost conveyor belt	2400 (JSD/m	Period OPEX calculation	20 years	
Trestle cost at zero depth	10000	JSD/m	Interest rate	3 %	
Trestle cost per m depth	8	JSD/m°			
Berth cost at zero depth	40000	JSD/m	Input breakwater des	ign	
Berth cost per m depth	1333 (JSD/m,			
Cost core material	2	JSD/m°	Rubble mound breakwater p	arameters	
Cost armour material	300	JSD/m²	Density water	1025 kg/m²	
Cost per m' caisson	350 (JSD/m²	Density rock	2300 kg/m²	
Cost caisson foundation	80 (JSD/m²	Armour type	(Bloc	
			Safety margin around rubble mound breakwater	0.25 × Width B	≳
			Ratio crest width / freeboard	2	
			Maximum overtopping discharge (rubble mound)	0.05 m [*] ls/m	

Figure D.3: Part of the input sheet of the cost module. The input parameters are divided into categories in order to maintain the overview.

0.30451391	4.03534432	2744.289665	156.9471231	1.219062041	2000
0.305293126	4.010360343	2725.953166	156.4038136	1.219791178	1980
0.30607523	3.985474899	2707.714872	155.860504	1.220522399	1960
0.306876496	3.960176357	2690.251149	155.3171945	1.221255766	1940
0.307581352	3.938084743	2672.889865	154.6981381	1.222354037	1920
0.308287995	3.916088825	2655.631115	154.0790817	1.223457013	1900
0.3089964	3.894189181	2638.475001	153.4600253	1.224564767	1880
0.30970654	3.8723864	2621.421628	152.8409689	1.225677373	1860
0.310418389	3.850681081	2604.471107	152.2219125	1.226794908	1840
0.311131919	3.829073834	2587.623554	151.6028562	1.227917456	1820
0.311847099	3.807565283	2570.879091	150.9837998	1.229045101	1800
0.312378364	3.791683457	2547.21904	150.3647434	1.230177933	1780
0.312116568	3.799499552	2493.872221	149.745687	1.231316046	1760
0.311857681	3.807248163	2441.132225	149.1266306	1.232459537	1740
0.311601804	3.814925667	2388.998196	148.5075742	1.23360851	1720
0.311349043	3.822528303	2337.469264	147.8885178	1.234763071	1700
0.311099506	3.830052159	2286.544546	147.2694614	1.235923333	1680
0.310853305	3.837493169	2236.22315	146.650405	1.237089413	1660
0.310608257	3.844916939	2186.53786	146.0313486	1.238261433	1640
0.310237666	3.856177445	2137.452452	145.3194034	1.239910794	1620
0.30986987	3.867393021	2088.966028	144.6074582	1.241570475	1600
0.309504989	3.878559202	2041.077677	143.895513	1.243240634	1580
0.309143151	3.889671313	1993.786475	143.1835678	1.244921434	1560
0.308784488	3.900724454	1947.091483	142.4716226	1.246613042	1540
0.308429138	3.911713495	1900.991749	141.7596774	1.248315636	1520
0.308077246	3.922633061	1855.486308	141.0477322	1.2500294	1500
0.307728962	3.933477516	1810.574177	140.335787	1.251754523	1480
Transmission coefficient rubble mound, short [-]		Surface cross-section breakwater [m²] 🔽)	Wave length [m] 💌	Wave height [m] 💌 V	Offshore distance ship [m] 🕂
		Transmission calculation			

 x_{ship} , for wave condition 1. Figure D.4: Calculation sheet in the wave calculation module. In this calculation sheet the transmission coefficient is determined for a range of
Bibliography

- BAKERMANS, B. Open ports for container vessels. Master's thesis, Delft University of Technology, 2014.
- [2] Bos, C. Option analysis and selection for a dry bulk harbour in liberia (confidential). Tech. rep., Royal HaskoningDHV, 2013.
- [3] BOSBOOM, J., AND STIVE, M. Coastal Dynamics I, lecture notes CIE4305. Delft Academic Press, 2015.
- [4] BRUUN, P. Marine terminal technology. In Handbook of coastal and ocean engineering: Harbors, navigational channels, estuaries, environmental effects (1992), J. Herbich, Ed., vol. 3, Gulf Publishing Company.
- [5] BURDALL, A., AND WILLIAMSON, H. A green port: an engineer's view. In Ports into the next century: Proceedings of the Conference Uk Ports 2000 (December 1991), Institution of Civil Engineers, Thomas Telford Ltd.
- [6] CARSTENS, P. Casestudie naar caisson golfbreker voor lng terminal in hazira, india. Master's thesis, Delft University of Technology, 2001.
- [7] CIRIA, CUR, AND CETMEF. The Rock Manual: The use of rock in hydraulic engineering, 2nd ed. C683, CIRIA, 2007.
- [8] CROSS, J., AND BEIMERS, B. Stage 1 cost estimate report. Tech. rep., Royal HaskoningDHV, 2015.
- [9] DE JONG, M., AND BORSBOOM, M. shallow water initiative (hawai), wp1: Nearshore wave modelling - task a and b: Overview of numerical wave models in coastal engineering and their relevance to shallow water lng activities. Tech. rep., Delft Hydraulics, January 2007.
- [10] DE JONG, M., WEILER, O., AND SCHOUTEN, J. Open water ports: Possibilities and challenges for container terminals. In *Topics in Safety, Risk, Reliability and Quality* (2014), vol. 24, Deltares, Springer Netherlands.
- [11] GAYTHWAITE, J. Mooring of ships to piers and wharves. American Society of Civil Engineers, 2014.
- [12] GODA, Y. Random seas and design of maritime structures. World Scientific Publishing, 2000.
- [13] GOOGLE. Google maps. https://maps.google.com, 2016. [Online; accessed 14 January 2016].
- [14] GYORY, J., BISCHOF, B., MARIANO, A., AND RYAN, E. Ocean surface currents. http: //oceancurrents.rsmas.miami.edu/atlantic/guinea.html, 2005. [Online; accessed 10 January 2016].

- [15] HADIJAH, S. Assessing the feasibility of open container ports. Master's thesis, UNESCO-IHE, 2013.
- [16] HOLTHUIJSEN, L. Waves in Oceanic and Coastal Waters. Cambridge University Press, 2010.
- [17] HOSSAIN, A., KIOKA, W., AND T., K. Transmission of long waves induced by short-wave groups through a composite breakwater. Tech. rep., Nagoya Institute of Technology, 2001.
- [18] INFLATIE.EU. Historische inflatie nederland. http://nl.inflation.eu/ inflatiecijfers/nederland/historische-inflatie/cpi-inflatie-nederland.aspx, 2016. [Online; accessed 1 June 2016].
- [19] JOURNÉE, J., AND ADEGEEST, L. Theoretical Manual of Strip Theory Program "SEAWAY for Windows". Delft University of Technology and AMARCON, September 2003.
- [20] KAMPHUIS, J. Designing for low frequency waves. In Coastal Engineering 2000 Proceedings of the 27th International Conference on Coastal Engineering, ICCE 2000 (2001), vol. 276, American Society of Civil Engineers.
- [21] LIGTERINGEN, H., AND VELSINK, H. Ports and Terminals. Delft Academic Press, 2012.
- [22] MARIN. Time Domain Analysis of Multi Body Dynamics for Offshore Operations: aNySIM. MARIN, 2006.
- [23] MARIN. General. https://wiki.marin.nl/images/1/16/-GENERAL-.pdf, November 2011. [Online; accessed 4 January 2016].
- [24] MCCONNELL, K., ALLSOP, W., AND CRUICKSHANK, I. Piers, jetties and related structures exposed to waves: Guidelines for hydraulic loadings. Thomas Telford Publishing, 2004.
- [25] MOL, A., LIGERINGEN, H., AND WAANDERS, A. Motions of moored vessels: a statistical approach. In *First Australian Port, Harbour and Offshore Engineering Conference* (1986), The Institution of Engineers, Australia.
- [26] MUÑOZ, E. Surface current variability in the tropical atlantic. Tech. rep., University of Maryland at College Park, 2003.
- [27] NATHANIEL BOWDITCH, L. The American Practical Navigator: an epitome of navigation. National imagery and mapping agency, 2002.
- [28] OLAGNON, M., AGBÉKO KPOGO-NUWOKLO, K., AND GUÉDÉ, Z. Statistical processing of west africa wave directional spectra time-series into a climatology of swell events. Tech. rep., Archimer, 2014.
- [29] PIANC. Criteria for Movements of Moored Ships in Harbours. Working Group 24, 1999.
- [30] PIANC. Harbour Approach Channels Design Guidelines. Working Group 49, 2014.
- [31] ROELVINK, D., VAN DONGEREN, A., MCCALL, R., HOONHOUT, B., VAN ROOIJEN, A., VAN GEER, P., DE VET, L., NEDERHOFF, K., AND QUATAERT, E. XBeach Technical Reference: Kingsday Release. Deltares, UNESCO-IHE Institute of Water Education and Delft University of Technology, April 2015.
- [32] ROZYNSKI, G. Closure depth. http://www.marbef.org/wiki/Closure_depth, 2015. [Online; accessed 28 June 2016].

- [33] SCHIERECK, G. J. Introduction to Bed, bank and shore protection. Delft Academic Press, 2012.
- [34] SCHOLL, O. Etudes techniques pour la construction d'un nouveau port dans la région de (confidential). Tech. rep., Royal HaskoningDHV, 2016.
- [35] THORESEN, C. Port designer's handbook, 2 ed. Thomas Telford Limited, 2010.
- [36] TUTUARIMA, W., AND D'ANGREMOND, K. Cost comparison of breakwater types. Tech. rep., Delft University of Technology, 1998.
- [37] VAN DER LEM, C. Basic design report dynamic mooring analysis. Tech. rep., Royal HaskoningDHV, 2007.
- [38] VAN DER LEM, C. Wave forces on moored ships. Haskoning Nederland B.V., April 2007.
- [39] VAN DER MOLEN, W. Behaviour of Moored Ships in Harbours. PhD thesis, Delft University of Technology, 2006.
- [40] VAN DER MOLEN, W. Low-frequency wave force program SURFBEAT / LF-STRIP. Delft University of Technology, Faculty of Civil Engineering and Geosciences, Section of Hydraulic Engineering, 2013.
- [41] VAN DEYZEN, A. Port development at a location in west-africa. Tech. rep., HaskoningDHV Nederland B.V., 2015.
- [42] VAN DEYZEN, A., BEIMERS, P., VAN DER LEM, J., MESSITER, D., AND DE BONT, J. To improve the efficiency of ports exposed to swell. In Australasian Ports and Coasts Conference 2015 (2015), Royal HaskoningDHV.
- [43] VERHAGEN, H., D'ANGREMOND, J., AND VAN ROODE, F. Breakwaters and closure dams. Delft Academic Press, 2012.
- [44] VIS, F., MOL, A., RITA, M., AND DEELEN, C. Long waves and harbour design. In International Conference on Numerical Modelling of Ports and Harbours (1985), J. Pounsford, Ed., Delft Hydraulics Laboratory and PRC Engineering Inc. and Laboratorio Nacional d'Engenharia Civil, BHRA.
- [45] WUISMAN, W., AND VAN DER MOLEN, W. Hydrodynamics of a moored lng carrier behind a detached breakwater. Tech. rep., Delft University of Technology, 2005.
- [46] YOUNG, I. Wind generated ocean waves. Elsevier, 1999.
- [47] ZIJLEMA, M., STELLING, G., AND SMIT, P. Simulating nearshore wave transformation with non-hydrostatic wave-flow modelling. Tech. rep., Environmental Fluid Mechanics Section, Faculty of Civil Engineering and Geosciences, Delft University of Technology, 2011.