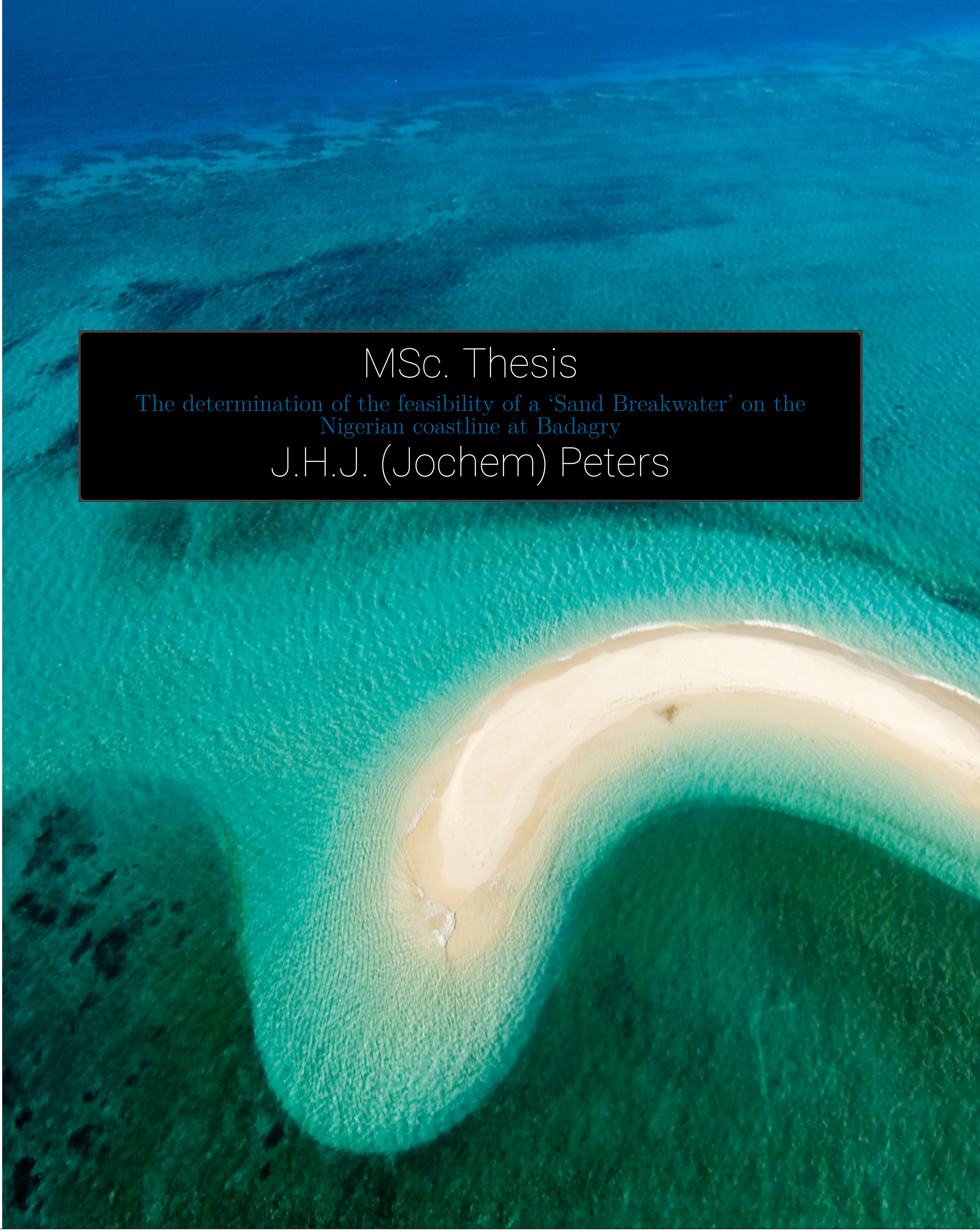

MSc. Thesis

The determination of the feasibility of a 'Sand Breakwater' on the Nigerian coastline at Badagry

J.H.J. (Jochem) Peters

TU Delft Delft University of Technology

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Royal HaskoningDHV
Enhancing Society Together

MSc. Thesis

The determination of the feasibility of a 'Sand Breakwater' on the Nigerian coastline at Badagry

by

J.H.J. (Jochem) Peters

to obtain the degree of Master of Science
at the Delft University of Technology,

to be defended publicly on Wednesday March 14th, 2018 at 11:00 AM.

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

Cover image: Sand bar at Great Barrier Reef (National-Geographic, 2017)

"The palest ink is stronger than the best memory."

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Abstract

The coastal waters of Nigeria are a difficult environment to build structures to control the local morphology and hydrodynamic conditions to ones desire. This difficulty is a consequence of the persistent high-energy swell wave climate and Longshore Sediment Transport (LST) in the order of one million cubic metres per year. Besides these tough conditions, the costs of conventional coastal mitigation solutions are high because of the lack of suitable material (rock) close by. These characteristics of the Nigerian coast led to the interest, whether it is possible to design and construct a 'Sand Breakwater' for a planned port on the Nigerian coastline at Badagry. A 'Sand Breakwater' is the complete construction which functions as the protection of the port and creates shelter from incoming waves for ships inside, consisting out of components (partly) made out of sand and hard structures. Especially the very uni-directional and consistent character of the wave climate creates an opportunity to embrace natural processes in the design. In addition, much sand needs to be dredged for the creation of the approach channel along with the port's basin. This volume of sand might possibly be re-used for a sand breakwater.

This report presents the feasibility of a sand breakwater on the Nigerian coastline at Badagry. This feasibility is determined both morphologically and economically. Long term coastline development was modelled with the one-line coastline evolution model LITPACK. Results show that the persistent uni-directional character of the wave climate prevailing at the coast of Badagry forces an equilibrium coastline orientation of 277.5° with respect to True North (TN). Because of this, all other orientations of coast rather than the equilibrium orientation are vulnerable and will develop to the equilibrium orientation over time. The implementation of this equilibrium orientation in the design along with some hard structures led to multiple long term morphological stable conceptual variants for a sand breakwater. The coastline impacts of these variants west of the sand breakwater were examined on large scale. In 38 years after construction a maximum of 20 meters coastline retreat occurs, which is considered acceptable. After this period, only shoreline accretion takes place.

To establish complete conceptual designs, the cross-shore profiles are first determined. The submerged part of the profile up to the top of the intertidal profile is constructed by integration of measured data in the design. The emerged part of the profile concerns the cross-section of a dune and was the critical part for the designing objective. To determine these profiles, first the design crest height is determined by applying maxmimum acceptable overtopping volumes (Tilmans, 1983). Knowing the design crest height, the dune width is determined by modelling storm impact. This width is established by examining storm impact and determine what minimal required width needs to be present for the sand breakwater to live up to its design criteria.

Storm impact for the found conceptual variants is examined with the numerical software XBeach to determine cross-shore sediment transport during storm impact. The impact of storms with shorter return periods results to be relatively high in comparison to the storms with longer return periods. This is taken into account by adapting the design conditions. The design storm conditions are based on the impact of a storms with a 1/100 year return period along with an expansion for a consecutive storm with a return period of 1/1 year. With this storm impact known, the required crest width is determined and with that the minimal required cross-shore profiles for the multiple variants are established. Consequently, besides long term morphologically stable, the conceptual designs are capable to design storm impact, confirming the morphological feasibility.

In order to define the economical feasibility of a sand breakwater, a comparison between the different conceptual variants and a conventional design is made. This comparison is conducted via a rough cost comparison along with examining future prospects. In order to be able to make this cost comparison, rough volume estimations are done for all conceptual variants and conventional design. These volume estimations combined with unit prices lead to a rough cost comparison. From this rough cost comparison appears that a sand breakwater showed to be in the same price range as a conventional design.

Three future prospects of a sand breakwater are determined. First of all the by-pass of sand is compared for the conceptual designs to the conventional design. The characteristic LST is problematic due to the occurrence of sand by-passing causing sedimentation in the approach channel and port. One of the conceptual variants shows that a sand breakwater is proven to be able to provide a period without by-passing of sand in the same order of magnitude as a conventional design.

Another future prospect is the required maintenance necessary to execute for the sand breakwater. The XBeach results of lower storm conditions show that maintenance costs are high. However, these results are assumed to be heavily overestimated. Results with more specific 'Nigerian conditions' create reason to believe that maintenance for lower storm conditions will be much lower and in acceptable range.

The last future prospect concerns the accreted land which arises due to the blockage of LST. A sand breakwater creates the possibility of acquiring land for new port space which is not possible with a conventional breakwater. In addition, the value of a Building with Nature component is present in this project and could enhance its economical interest.

Not only the morphological but also the economical feasibility of a sand breakwater is confirmed. The 'Sand Breakwater' succeeded to convert the drivers of the problem to its solution, leading to an innovative and in all likeliness even more cost-effective solution compared to the traditional approach.

Contents

Nomenclature

Acronyms

RHDHV	Royal HaskoningDHV
ASL	Above Sea Level
BwN	Building with Nature
ITCZ	Inter-Tropical Convergence Zone
SLR	Sea Level Rise
LST	Longshore Sediment Transport
LNG	Liquefied Natural Gas
AD	Active Depth
BH	Beach Height
HAT	Highest Astronomical Tide
MHWS	Mean High Water Springs
MSL	Mean Sea Level
MLWS	Mean Low Water Springs
MSTR	Mean Spring Tidal Range
LAT	Lowest Astronomical Tide
SWL	Still Water Level
RTR	Relative Tidal Range
CD	Chart Datum
URP	Ultimate Required Profile

Glossary

Accropode	Concrete armour elements which can be placed in a single armour layer on a breakwater with a steep slope
Bed load transport	Sediment which is transported along the bed level
Breaker zone	Subset of surf zone. Zone in which wave approaching the shore start breaking
Breakwater	Human made coastal structures for coastal mitigation measures or protection of ports
Diffraction	Changing of waves from direction towards areas containing lower amplitudes due to amplitude changes along the wave crest (Holthuijsen, 2007)
Foreshore	Part of the cross-shore profile between high- and low- water
Groyne	Human made coastal structure constructed for coastal mitigation measures constructed in and around the breakerzone
Littoral transport/drift	See Longshore Sediment Transport
Longshore Sediment Transport	Transport of sediment along the coast parallel to the shoreline caused by oblique incoming waves
Overwash	Flow of water over dune
Refraction	Bending of wave direction due to changes in water depth underneath (Bosboom and Stive, 2012)
Sand breakwater	Construction which functions as protection of the port and provides shelter for incoming waves with components (partly) made out of sand and hard structures
Shadow zone	Sheltered area behind coastal structure where no direct wave impact takes place and wave heights decrease due to diffraction.
Shoaling	Increase in wave height of approaching waves due to decrease in water depth

Sinks	Areas or locations which extract sediment from the system
Sources	Areas or locations which supply sediment to the system
Spreading coefficient	Coefficient which expresses the state of the sea
Squall	Short heavy storms which are caused by the growth of wave-like perturbations along the surface of discontinuity between the monsoon southwesterlies and the dry northeasterlies in West Africa (Adedoyin, 1989)
Storm surge	Rise in sea water level caused solely by a storm
Surf zone	Area of breaking waves
Suspended load transport	Sediment which is transported in suspension
Swell	Waves originating in distant storms which have travelled for large distances and have been subjected to frequency and directional dispersion of which uniformity in direction, period and height is developed over travelled time and distance
Tombolo	The accretion of sand behind a detached breakwater which connects to the original coastline
Wave train	Groups of swell waves
Wind set-up	The vertical rise in the still water level in front of a structure caused by wind stressed on the surface or the water (Bosboom and Stive, 2012)
Windsea	Waves mainly caused and affected by the input of wind.

List of Symbols

Latin Symbols

Symbol	Units	Description
y_c	m	Coastline position
Q	m^3year^{-1}	Longshore Sediment Transport
x	m	Alongshore position
Q_{Source}	$m^3/year^{-1}$	Supply or extraction of sediment from sources or sinks, including eventual dune erosion
h_{act}	m	Active height of the cross-shore profile
H_b	m	Wave height just before breaking
z_a	m	Static rise in water level
p_a	hPa	Atmospheric pressure at sea level
dy	m	Cross-shore change in coastline position
dQ	m^3year^{-1}	Change in Longshore Sediment Transport
dx	m	longshore discretisation step
H_{inc}	m	Wave height of incoming wave
q_t	m^2s^{-1}	Total sediment transport
q_b	m^2s^{-1}	Bed load transport
q_s	m^2s^{-1}	Sediment transport in suspension
u_f	ms^{-1}	Friction velocity
g	ms^{-2}	Gravitational acceleration
d	mm	Grain size d_{50}
u_{f_c}	ms^{-1}	Critical friction velocity
q	m^3s^{-1}	Overtopping volume
H_{m0}	m	Significant wave height
R_c	m	Freeboard of dune w.r.t. Still Water Level
$s_{m-1,0}$	-	Steepness parameter
$T_{m-1,0}$	s	Energy wave period
T_p	s	Peak wave period
d_{50}	mm	Median grain size
$A0$	m^3	Reduction volume
$R0$	m	Location of reduction line
k	-	Number of occurencies of storms per year

Greek Symbols

Symbol	Units	Description
ψ'	-	Stability parameter
Δ	-	Density difference
$\psi_{c,0}$	-	Critical stability parameter
θ	°	Angle of flow to the slope
β	°	Angle of slope
μ_s	-	Static friction coefficient
Φ_b	-	dimensionless bed load transport
α	°	Angle of slope
$\zeta_{m-1,0}$	-	Breaker parameter
γ_b	-	Factor for presence berm
γ	-	Factor for roughness of slope
γ_β	-	Factor for wave angle direction
γ_v	-	Factor for presence vertical wall
λ	-	Expected value of the number of occurrences per year

Conventions and Definitions

Interpretation of wind roses and tables

Wind- and wave roses provide a quick way of summarizing the directional wind- and wave condition statistics. The number in the centre of the rose represents the percentage of the time that calm conditions occur. The length of an arm of a rose represents the percentage of the time that winds or waves come from the direction sector that the arm points in (see the bar scale in Figure 1 presenting roses which indicates the percentage represented by unit length).

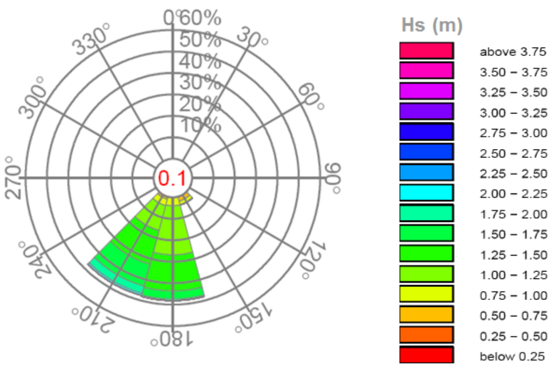


Figure 1: Example of a (30 degree sector) wave height rose (RoyalHaskoningDHV, 2014a)

In Table 5 a clarification of wind speeds related to the Beaufort scale is given. This gives insight in values given further on in this research.

Table 5: Beaufort scale in relation to wind speed [m/s] (RoyalHaskoningDHV, 2014a)

Beaufort scale	**Description**	**[m/s]**	**Sea state**
0	Calm	<0.3	Flat
1	Light air	0.3 - 1.5	Ripples without crests
2	Light breeze	1.5 - 3.3	Small wavelets. Crests of glassy appearance, not breaking
3	Gentle breeze	3.3 - 5.5	Large wavelets. Crests begin to break; scattered whitecaps
4	Moderate breeze	5.5 - 8.0	Small waves with breaking crests. Fairly frequent white horses.
5	Fresh breeze	8.0 - 11	Moderate waves of some length. Many white horses. Small amounts of spray.
6	Strong breeze	11 - 14	Long waves begin to form. White foam crests are very frequent. Some airborne spray is present.
7	High wind, Moderate gale	14 - 17	Sea heaps up. Some foam from breaking waves is blown into streaks along wind direction. Moderate amounts of airborne spray.
8	Gale, Fresh gale	17 - 20	Moderately high waves with breaking crests forming spindrift. Well-marked streaks of foam are blown along wind direction. Considerable airborne spray.
9	Strong gale	21 - 24	High waves whose crests sometimes roll over. Dense foam is blown along wind direction. Large amounts of airborne spray may begin to reduce visibility.
10	Storm, Whole gale	25 - 29	Very high waves with overhanging crests. Large patches of foam from wave crests give the sea a white appearance. Considerable tumbling of waves with heavy impact. Large amounts of airborne spray reduce visibility.
11	Violent storm	29 - 32	Exceptionally high waves. Very large patches of foam, driven before the wind, cover much of the sea surface. Very large amounts of airborne spray severely reduce visibility.
12	Hurricane	≥ 33	Huge waves. Sea is completely white with foam and spray. Air is filled with driving spray, greatly reducing visibility.

Interpretation of water levels

A relation between Mean Sea Level (MSL) and the presented modelling results needs to be clarified. The local datum is the Badagry Chart Datum (CD). Bathymetric data from different sources has been adapted to match the projection and the model datum, see Figure 2. MSL is used as standard datum in this study.

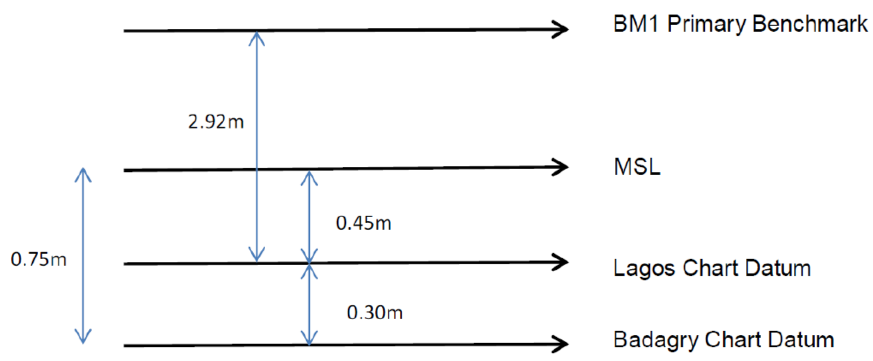


Figure 2: Local vertical datum and MSL relation

1 Introduction

1.1 Background

Nigeria's coastal area has been subjected to an increase in economic activities, primarily driven by the seaport and oil exploitation. These activities led to more extensive use of the coastline and have resulted in accelerated shoreline erosion estimated at 20-30 meters per annum at certain locations (Orupabo, 2008). The increase of economic activities was abruptly stopped by the economic crisis of 2008. Yet, rising prosperity level, growing middle class and overall growth of population has placed considerable pressure on the country's economical transportation infrastructure. Even despite ongoing improvements and upgrades to the country's ports, by all accounts, demand for port handling capacity will likely outstrip supply in country soon. This shortage in capacity will be particularly present in the Lagos area, where in 2014 nearly 85% of the country's non-oil imports and exports passed (RoyalHaskoningDHV, 2014a). Since the economy is recently growing, interest is taken in increasing the import and export capacity.

Net sediment loss is a phenomenon which has been the norm along the western end of the Nigerian coastline because of the poor understanding of coastal processes. A coastline undergoing such socio-economic stress needs a sufficient management in such a way that it will sustain itself via sediment redistribution within the coastal environment. The Nigerian coastline is known for its large Longshore Sediment Transport (LST). LST refers to the cumulative movement of beach and nearshore sand parallel to the shore by the combined action of tides, wind and waves and the shore-parallel currents produced by them (Seymour, 2005). The order of magnitude of the LST is one million cubic meters annually from West to East (Tilmans, 1993). In a morphological active environment such as the coastal waters of Nigeria, it can be difficult to build coastal structures to control the local morphology and hydrodynamic conditions to ones desire. This difficulty is a consequence of the harsh wave climate and large LST. Besides these tough conditions, the costs of traditional coastal mitigation solutions are high because of a lack of suitable material close by. Hence, challenges arise when constructing coastal structures and harbours.

Currently a client of Royal HaskoningDHV(RHDHV) desires to construct a container terminal. RHDHV had been appointed by the client to undertake a feasibility study for the proposed new port development. In search of this feasibility study a solution for the morphological challenge at the Nigerian coastline in Badagry is required. In Figure 1.1 an overview of the old proposed design of the future port layout is shown. This terminal, and its approach channel, need to be protected by a breakwater or a comparable construction that can sustain a sheltered area for port activities and shipping/navigation. The wave climate to which the port needs to be protected concerns a rather constant and uni-directional which is characteristic for the Nigerian coast. Persistent high-energy swell waves, reaching Badagry with the mean direction of 197° offshore, 189° nearshore both with respect to True North (TN).

Figure 1.1: Overview of old proposed design in Google Earth, (Google Earth, 2017)

1.2 Topic Introduction

The characteristics of the Nigerian coast led to the interest of RHDHV and a curiosity in general, whether it is possible to design and construct a 'Sand Breakwater' or hybrid breakwater.

Definition of a 'Sand Breakwater':

"The complete construction which functions as the protection of the port and creates shelter from incoming waves for ships inside, consisting out of components (partly) made out of sand and hard structures."

Taken into account both the high cost of acquiring rocks for the construction of a conventional type of breakwater and the challenging morphological and hydrodynamical environment, such a sand breakwater could lead to a possibly cheaper solution in the case of Badagry. Especially the very uni-directional and consistent character of the wave climate creates an opportunity to embrace natural processes in the design.

Little research has been done to this topic and consequently, has only been criticised so far due to the nature of the wave climate. Nonetheless, the last few years many new solutions with respect to coastal protection and mitigation have been developed which are not similar to traditional measures as well. Especially in the context of Building with Nature much has been established recently concerning coastal protection. Building with Nature (BwN) in essence is making use of the dynamics of the natural environment and provide opportunities for natural processes rather than simply disturbing the processes and minimising or mitigating the environmental impact (H. de Vriend, 2014). BwN in the context of this study concerns the use of sand instead of rock and creating a design which embraces the natural processes in a way that the coastal protection actually benefits from it. BwN does not only lead to more sustainable solutions in general but in the case of Badagry it is aimed to lead to a more cost-effective solution.

1.3 Research outline

This report contains the proposal to determine the feasibility of a 'Sand Breakwater' on the Nigerian coastline at Badagry. The following chapter, Chapter 2, contains the problem definition and the subsequent research (sub-)questions through which a clear scope of this study is established. An accompanying methodology for the research is included in this section. Chapter 3 concerns an analysis of the system of the area of interest with respect to key information and governing processes/influences. This analysis established a sufficient data pack which is used in the models and design. In Chapter 4, the one-line coastline evolution model "LITPACK", its set-up and calibration are presented. A thorough elaboration of the set-up of the one-line coastline evolution model is presented in Appendix F. The results of the one-line coastline evolution model are discussed in the following chapter, Chapter 5. This chapter focusses on finding long term morphologically stable conceptual variants by focussing on alongshore coastline development. The following chapter, Chapter 6, is mainly driven by the cross-shore transport analysis. Storm impact is examined for the found conceptual variants. The main objective in this chapter is to define cross-shore profiles for the sand breakwater which are sufficient for its design storm conditions. The governing storm conditions are modelled with a different morphological model than the coastline model in the previous chapter called "XBeach". Conclusively the different cross-shore profiles for the variants along with their coastal development are integrated into design in Chapter 7. In order to make a comparison between the different conceptual variants and conventional design a rough cost comparison along with future prospects are presented. These aspects together form the keystone in the main research objective. The penultimate chapter is Chapter 8 in which multiple liability aspects of various results are discussed. Finally in Chapter 9 the answer on the main question is presented along with the remaining gaps of knowledge and recommendations for further research.

2

Problem Definition and Research Objective

2.1 Problem Definition

During the previous study by RHDHV (RoyalHaskoningDHV, 2014c) the effects of the economical crisis were still present. These effects led to a financial obstruction and delayed the construction of a future port at Badagry. Now, a few years later, the question has risen whether it is possible to construct this port in a cheaper way. Especially the breakwater(s) of the port are put to question driven by this intent. Previous studies performed by RHDHV have led to a final design for a conventional breakwater constructed with rock and concrete elements. This design concerns a relatively expensive breakwater due to the large cost of rock (placement) in Nigeria. The desire to construct the breakwater in a cheaper manner actually rooted in the fact that large quantities of sand are available. Much sand needs to be dredged for the creation of the approach channel along with the port's basin. This volume of sand might possibly be re-used for a sand breakwater. In addition to the large amounts of sand available, a rather large Longshore Sediment Transport (LST) is caused at the coast of Badagry due to a persistent, high-energy and uni-directional swell wave climate. This characteristic LST might be used to the benefit. This LST needs to be carefully taken into account and possibly blocked. Otherwise, the LST will cause sedimentation in the approach channel and in the port itself and maintenance dredging will be required.

Conclusively, the problem is stated in twofold:

- The material costs for a conventional breakwater are significantly high.
- A rather harsh wave climate and huge Longshore Sediment Transport is present.

Due to the lack of research to possible solutions and their morphological development it is impossible to determine whether a 'Sand Breakwater' is actually feasible in this environment. Lack of insight in morphological processes requires modelling research.

2.2 Research Objective

Main research objective:

"The determination of the feasibility of a 'Sand Breakwater' on the Nigerian coastline at Badagry"

In Chapter 1 a definition of a sand breakwater is given. Feasibility is a rather generic term and requires a more accurate description as well. In this study the feasibility of a sand breakwater will be answered through two components. The first one being the feasibility in a morphological sense referring to the morphological stability of a sand breakwater. The second one being the feasibility in an economical sense. This last feasibility is determined by comparing conceptual variants to the conventional breakwater design. A sand breakwater is found feasible when it satisfies both requirements.

The main research objective is split up in multiple sub-questions through which the main objective is aimed to be accomplished. Two types of sub-questions exist: sub-questions related to the morphological part and sub-questions related to the economical part.

Morphological sub-questions:

- What is the equilibrium coastline orientation for the prevailing wave climate?
- Which alongshore parts of a 'Sand Breakwater' are most critical and vulnerable to erosion and how to cope with these vulnerabilities?
- What is the amount of sand by-passing the breakwater compared to the conventional design?
- What is the impact of the Sand Breakwater on the coastline west of the port?
- What kind of cross-shore profiles are stable under storm conditions?

Economical sub-questions:

- What is the estimated cost in comparison to a traditional breakwater?
- What is the future prospect of a sand breakwater in comparison to a conventional breakwater?

2.3 Methodology

To accomplish the main objective and determine whether a sand breakwater is feasible, first the sub-questions require investigation. The answer to these sub-questions together lead to an answer sufficiently reliable to accomplish the main research objective.

The process for the design of a sand breakwater requires an iterative approach. The approach can be roughly schematised in three phases:

1. Determine the morphological feasibility
   (a) Finding a long term morphological stable solution
   (b) Finding a sufficient solution with respect to storm impact
2. Determine the economical feasibility
3. Conclude on the feasibility of the sand breakwater

A rough outline of the research phases is shown below:

Phase 1a

Lack of insight in morphological processes requires modelling research. First of all is determined what kind of model(s) are used. Subsequently the model is set-up and calibrated after which insight is gained in the amount of LST along the coast of Nigeria and sub-questions:

- What is the equilibrium coastline orientation with respect to the prevailing wave climate?
- Which parts alongshore of a 'Sand Breakwater' are most critical and vulnerable to erosion and how to cope with these vulnerabilities?

One of the boundary conditions is to create conceptual designs which have zero LST on the tip of the breakwater. This way no by-pass of sand takes places and less sedimentation occurs in the approach channel. For a uni-directional wave climate the LST capacity is zero when the the orientation of the beach is perpendicular to the incoming waves (equilibrium coastline orientation). The starting scenario for modelling is to find this orientation and implement it in design. From there, coastline mitigation measures are taken until a stable solution is found. The starting scenario is a scenario containing a sand breakwater completely made out of sand. With conclusions from the previous step different designs for conceptual forms are proposed to integrate and cooperate with the vulnerable parts of the sand breakwater. Through modelling runs, adaptations are made until the conceptual variants are found to be morphologically stable. With 'morphological stablility' is meant that the coastal development of the conceptual variant which occurs is acceptable for both erosion and/or accretion and the sand breakwater still fulfils its function. When these morphologically stable conceptual variants are found they are modelled to accomplish the following goals:

- Retrieve the amount of bypassing of sand which is expected to settle in the approach channel
- Gain insight concerning the impact of the Sand Breakwater on the coastline west of the port

Phase 1b

The previous part beholds finding long term morphologically stable conceptual designs by focussing on alongshore coastline orientation. The following part is mainly driven by cross-shore processes. The sand breakwater needs to be designed in a way that it is sufficient for certain storm conditions. Through this approach, required cross-sections are established for the sand breakwater.

- Test stability of the design with a model for storm impact and establish what kind of cross-shore profiles are stable under storm conditions

Phase 2

By finishing the previous step the morphological feasibility is determined. The conceptual variants are finalised by combining the design steps with respect to alongshore coastline development and storm impact. The third part concerns the economical feasibility determination which is examined by determining:

- The rough estimated costs in comparison to a conventional breakwater
- The future prospect of a sand breakwater in comparison to a conventional breakwater

Phase 3

The final step is combining both feasibilities and conclude whether a 'Sand Breakwater' is feasible on the Nigerian coastline at Badagry.

2.4 Modelling methodology

Shoreline adaptation is the result of spatial gradients in net sediment transport rates. Coastal change occurs where spatial sediment transport gradients and/or sediment sinks or sources occur (Bosboom and Stive, 2012). It is necessary to model these net sediment transport rates to model to gain a comprehension of the local morphology. In this section a clarification is given why using modelling software creates benefit for this research. First off in Subsection 2.4.1 several options of different modelling methods are presented concluded by a choice of method. Since in phase 1a long term coastline development is examined and in phase 1b storm impact, two different modelling approaches are used. In Subsection 2.4.2 the chosen modelling methods are elaborated on followed by the choice of specific software in Subsubsection 2.4.3.

2.4.1 Different modelling methods

The main goal in phase 1a is to examine the long term coastline evolution. The solution for a sand breakwater is required to be long term morphological stable. Long term coastline evolution has been analysed all over the world using various methods throughout the years (Dean and Dalrymple, 2002):

- Measurements and analysis of historical shoreline positions
- Numerical coastal morphodynamic models
- Physical models
- One-line models based on the conservation of sand volume equation

Using a statistical analysis of data would be preferable for its accuracy, however it is impossible to use when predicting effects of human intervention or scenario's/events which have not occurred yet.

Numerical coastal morphodynamic models like Delft3D are extensive in use of computational resources due to its required computation time for the kind of spatial scale. In addition, numerical models are often especially preferable when complex systems are present. A system is assumed to be complex whenever it consists of (tidal) currents, inlets, and waves. As hardly any tidal influence is present in the area of interest, no inlet is present and the impact of large oceanic currents is found to be negligible, no complex system is present. The impact of the large oceanic currents and tide is presented later in this research in Subsections 3.3.3 and 3.3.6.

Physical models are suitable to local analyses but are rather costly. Secondly physical models (partly) made out of sand are rather difficult to create due to the scaling effect of sand.

For this research most of the modelling is done using software which focusses on one-line coastline change. This software is preferred above other modelling software as it is able to provide a good insight in the coastline development under influence of principally alongshore morphological processes and is specifically made for this purpose. The very uni-directional nature of the wave climate and minimal influences from tide or currents at the uniform coast of Badagry makes the one-line coastline evolution models particularly a good fit. **Phase 1a** is executed by using these kind of models, see Section 2.3.

2.4.2 One-line theory

One-line coastline evolution models are based on a couple assumptions with the most important one being the constant shape of beach profile.

The shape of the cross-section remains the same while the shore is retreating or advancing.

Other important assumptions in one-line models are (Thomas and Frey, 2013):

- The shoreward and seaward vertical limits of the profile are constant
- Sand is transported alongshore by the action of breaking waves and longshore currents
- The detailed structure of the nearshore circulation is ignored
- A long-term trend in shoreline evolution is present
- An adequate supply of sand is present (i.e., an infinite supply of sand is assumed)

Analytical models can only try to approach the reality. One should be aware of the limitations of models like those. Pelnard-Considère founded the essence of the one-line theory in 1956. From that moment on, various models have been invented of which three have become the notion of late. These three models are discussed in Subsection 2.4.3. Further elaboration of the one-line theory can be found in Larson and Kraus (1997).

Cross-shore transport is either neglected or included as a sink or source term in the sand conservation equation. The principle of mass conservation applies to the system at all times. Following the previous stated assumptions, mass conservation of sand along an infinitely small length of the shoreline can be formulated with negligible contribution from source and sink. The one-line theory model is based on the continuity equation for sediment volumes as expressed in Equation 2.1.

$$\frac{\delta y_c}{\delta t} = -\frac{1}{h_{act}}\frac{\delta Q}{\delta x} + \frac{Q_{source}}{h_{act}\Delta x} \tag{2.1}$$

In which:

y_c	= Coastline position (m)
t	= Time (year)
Q	= LST (m^3/year)
x	= Alongshore position (m)
Q_{source}	= Supply or extraction of sediment from sources or sinks, including eventual dune erosion (m^3/year)
h_{act}	= Active height of the cross-shore profile (m)

Long term cross-shore modelling is not the emphasis in this research. As consequence of the incoming oblique waves on the Nigerian coastline strong longshore currents are generated. Consequently, the LST component is dominant over cross-shore transport. The cross-shore transport is, however, important in **Phase 1b**, see Section 2.3, when examining storm conditions. These conditions are modelled with the numerical software called "XBeach" here moreover in Chapter 6. Due to the smaller computation times and spatial scale, numerical coastal morphodynamic models are applicable. For the 1D modelling of storm impact XBeach is one of the most common tools and especially made for this application. This modelling software is used to determine storm impact in Chapter 6.

2.4.3 Choice of coastal evolution model

With respect to one-line coastline evolution models there are three models currently used worldwide. The first one being LITPACK or LITtoral Processes And Coastline Kinetics (DHI-GROUP, 2017a). The second one being GENESIS (GENEralized model for SImulating Shoreline change) and lastly UNIBEST (UNIform BEach Sediment Transport). Although the software packages aim to accomplish the same results certain differences do exist between the different software packages.

The models LITPACK and UNIBEST act with direct integration of the wave-current interaction. GENESIS is not able to do this directly and does this by empirical coefficients. Next to this LITPACK and UNIBEST apply a model of shoaling and refraction to transform waves to breaking; the approach in Battjes and Janssen (1978) is used to calculate wave dissipation across the surf zone. GENESIS uses a more simplistic empirical approach (Thomas and Frey, 2013).

One of the largest differences between the models is the method which they use to calculate longshore transport. GENESIS use a simple routine for this where breaking wave height and angle are first calculated, providing the forcing for the transport equation. This information then can be filled in the CERC-formula (CERC, 1984). In comparison to this empirical manner of calculation LITPACK as well as UNIBEST use a fully deterministic method. The benefit of this approach over the kind of empirical rigorous approach is primarily that the shape of the beach profile is implicitly included in analysis (Thomas and Frey, 2013). This approach would lead to more realistic modelling of transport over bars.

In Table 2.2 all the previous points of discussion along with a couple other factors are presented. Based on this table a conclusion on the choice of models has been made.

Table 2.2: Overview of advantages and disadvantages

Items	Models GENESIS	UNIBEST	LITPACK
Transformation breaking waves	-	+	+
Deterministic transport formula method	-	+	+
Variable depth of closure	-	-	+
Variable cross-shore profile	-	-	+
Variable grain size	-	-	+
Breakwaters	+	+	+
Diffracting groyne	+	+	+
Jetties	+	-	+
Tombolos	+	-	-
Trench	-	-	+
Internal wave model	+	+	+
Coupled spectral wave model	+	+	-
Wave-current interaction	-	+	+
Inlet integration	+	-	-
Previous study RHDHV	-	-	+

The selling points of GENESIS (Multiple sorts of coastal structures and inclusion of inlets in the model) are not governing in this research as the local system does not contain these complexities. Next to this fact, taken into account that UNIBEST and LITPACK work with a more detailed fully deterministic method, it can be said that GENESIS would not be a good choice for a model in this research. UNIBEST and LITPACK are strongly comparable and both suitable for this research. The application of a variable grain size is important in this research as well seen the differentiation of the grain size across-shore. In earlier studies (RoyalHaskoningDHV, 2014c) LITPACK has been used to study the morphology with the traditional design of the breakwaters. Although this research uses a entirely new set-up it is interesting to use the same software package. This way comparisons with respect to traditional design of the breakwaters are based on outputs of the same modelling software. To conclude, the model LITPACK has been chosen to use for this research.

3

System and processes

3.1 Introduction

In this chapter the local system along with its large scale forcings and processes are presented. Firstly, is zoomed in from a large to local system level, see Section (3.2). This is done with the goal to create a sufficient comprehension of the area of interest. In this section the general characteristics of the area are presented, which is followed by Section 3.3. In order to be able to analyse the area and find conceptual variants for the sand breakwater key hydraulic conditions are required. All major forcings which influence the hydraulical and morphological system at Badagry are discussed. The major forcings lead to hydraulic conditions. These hydraulic conditions form input for both the one-line coastline model LITPACK and cross-shore transport model XBeach.

3.2 Geology and area

3.2.1 Coastal area of West Africa

The area of interest is located on the West African coast. The complete coastline of the countries Ivory Coast, Ghana, Togo, Benin along with the western part of the Nigerian coastline belong to one and the same coastal system, see Figure 3.1. This 1,300 kilometre long coastline, can be classified as an "Afro-trailing-edge" coast. Afro-trailing edge coasts are defined by their sediment supply which, as stated before, is also characteristic for the Nigerian coast (Bosboom and Stive, 2012).

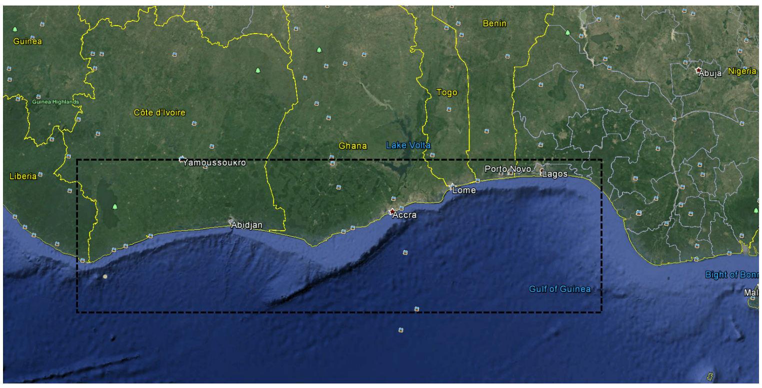


Figure 3.1: Coastal system on the West African coast (Google Earth, 2017)

Locations where man-made structures or interventions are affecting the littoral transport show large effects on the coastline development. A summary of these effects can be found in Appendix A. This appendix gives insight in how parts of the West African coastline react to human interferences. Multiple cases of construction of ports and their protection are presented comparable to the initial situation of this study.

3.2.2 Coastline of Nigeria

3.2.2.1 Barrier-lagoon system

The shoreline of Nigeria contains diverse sediment- and geomorphology characteristics. Roughly it can be divided into five sectors based on these small-scale geomorphic difference along with numerous physical characteristics like beach sediment grain size and coastal processes (W. J. Sexton, 1994).

1. Barrier-lagoon coast
2. Transgressive mud coast
3. Western and eastern delta flanks
4. Arcuate Delta
5. Strand coast

The division of shoreline segments is shown in Figure 3.2. This system consists out of interconnected lagoons, mangrove marshes, creeks, rivers, lakes and barrier islands up to a few metres above sea level (Ihenyen, 1997). These barrier islands are interconnected over time and form a relatively straight coastline. The coast has a straight sandy shoreline which is presently depositional except for localized erosion zones (Lagos, bar beach and other places where human intervention has taken place, see Appendix A).

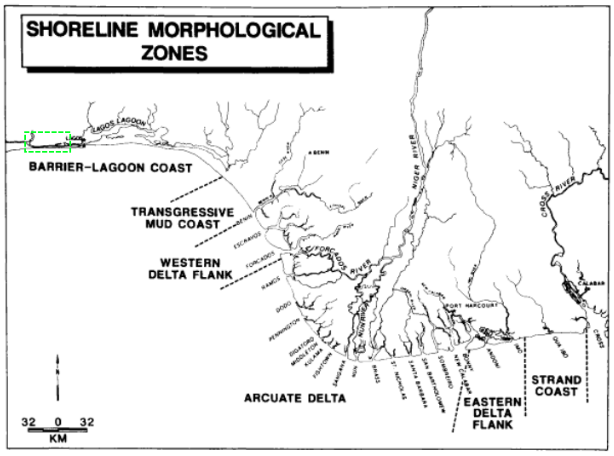


Figure 3.2: Division of five distinct morphological zones on the shoreline of Nigeria, with the study area highlighted in green (W. J. Sexton, 1994)

3.2.2.2 Offshore Canyons

Offshore of the Nigerian coast there are three canyons present. These canyons are vast and deep disruptions of the continental shelf and slope. It concerns the "Avon", the "Mahin" and the "Calabar" canyon. Bentum et al. (2011) concluded that for the analysis of the Lagos coastal behaviour the canyons do not have a relevant role. As Badagry lies even further away off these canyons, these are not taken into account in this study as a 'sink' of sediment.

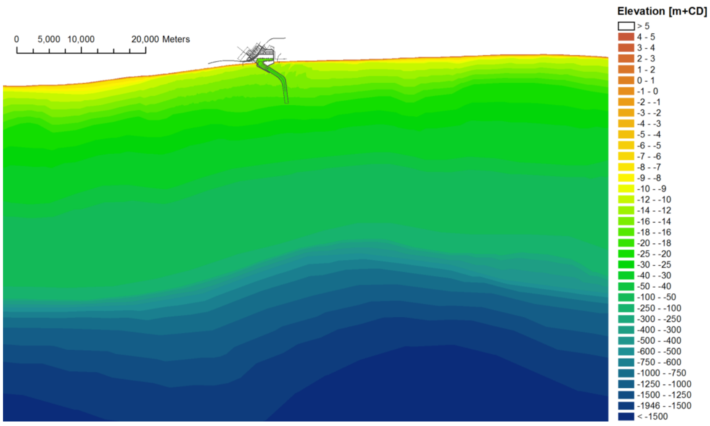
3.2.3 Coast of Badagry

The coast of Badagry is located within the coastal system earlier pointed out as the Barrier-lagoon complex. Close to the city of Badagry is the area of interest of this study. RHDHV has conducted feasibility studies and made designs for the construction of the Badagry Greenfield Port & Free zone. The planned location as it was designed a couple of years ago is shown in Figure 3.3.

Figure 3.3: Location of Badagry respectively to coastline (Google Earth 2017)

The barrier-lagoon system of Badagry comes with certain morphological characteristics. The beaches along this section contain steep slopes (1:20-1:30). This is typical for swell dominated sandy coasts (Bosboom and Stive, 2012). The reflective and micro-tidal character of these beaches created by the governing wave climate led to the presence of medium-grained, well-sorted, strongly coarse-skewed sand (over 400 µm) (W. J. Sexton, 1994). The LST is predominantly directed towards the east, and net LST values exceeding one million cubic meters per year have been determined (NEDECO, 1961; Tilmans, 1993).

The bathymetry used in this research is created from C-map Oceanview data combined with DEEP Bathymetric Survey data, see Subsection 3.3.2. Figure 3.4(a) shows the resulting bathymetry. The bathymetry is quite uniform alongshore, has a very gentle slope offshore and has a steep slope close to the shore as is confirmed by literature, see Subsection 3.2.1. The bathymetry data is important as it is necessary for the retrieval of required cross-shore profiles for both LITPACK and XBeach. In addition, it is a keystone in the design phase of the sand breakwater.



(a) Large scale bathymetry (RoyalHaskoningDHV, 2014a)

(b) Small scale bathymetry (RoyalHaskoningDHV, 2014a)

Figure 3.4: Local bathymetry

3.2.3.1 Soil characteristics

The deposits which have created the Nigerian coastline as it is comprise out of sands nearshore and silts in moderate depths and deep water (CEDA, 1997). Soil data containing soil properties is acquired on 90 locations in front of the coast and on the beach at Badagry. The data contains median grain size like D_{50}. This data is used to determine required sediment characteristics in the area of interest. The soil properties are acquired through grab samples and therefore are merely an indication of what kind of soil is superficially present. The spatially distributed present grain sizes are shown in Figure 3.5. Clearly the grain sizes have significant spatial variability. The cross-shore variability is much larger than the longshore variability. In addition, the alongshore uniformity of the coast is visible Figure 3.5.

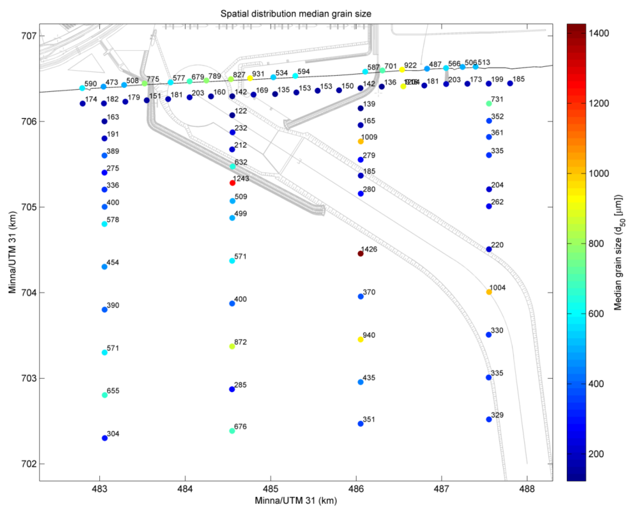


Figure 3.5: Distribution in space of median grain diameter (in grey the future port construction) (RoyalHaskoningDHV, 2014c)

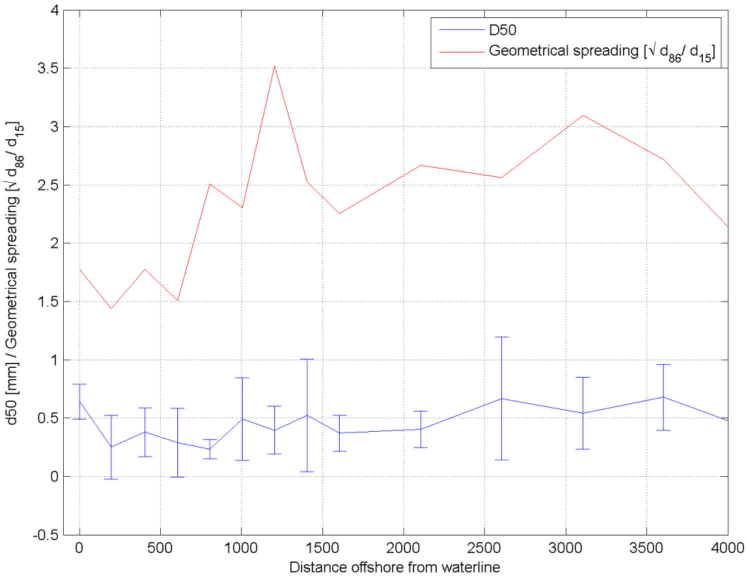


Figure 3.6: Cross-shore distribution of the median grain diameter (including corresponding standard deviation) and geometrical spreading (RoyalHaskoningDHV, 2014c)

Figure 3.6 shows the cross-shore distribution of the median grain diameter. The red line shows the average geometrical spreading defined as $(\sqrt{\frac{D_{86}}{D_{16}}})$. On the beach the median grain size varies roughly between 500 and 800 µm, which is classified as coarse sand, whereas along the five to nine meter depth contour in and just outside the surfzone the D_{50} roughly varies between 150 µm and 250 µm which is classified as fine sand. The average median grain diameter D_{50} on the beach is 645 µm. This sand concerns coarse sand which supports the beach state which is expected on this part of the Nigerian coast. These soil characteristics are taken into account for the models and further discussed in Subsection F.2.1 and Subsection 6.3.2.3

3.2.3.2 Land subsidence

In addition to the superficial soil characteristics it is necessary to examine the subsoil of the area of interest with respect to land subsidence. This subsidence needs to be taken into account for the design of conceptual variants. Badagry is located on the youngest deposit present at the Nigerian coastline. Consequently, high rates of subsidence occur. Natural compaction and natural de-watering of the soil are the cause of this. Lagos state appears to be subsiding at a rate of 7 mm/yr (Mahmud et al., 2016). In the areas around Lagos this subsidence rate is actually higher. Cities like Lekki, Badagry, Ikorodu and Epe have a much higher subsidence than in Lagos city although the water extraction is lower. This value was found, however, in studies to heavy structures constructed mainly in or on sand with high compaction rate. To be on the safe side of this figure, it is assumed that an advised subsidence rate for Badagry amounts to 1.5 mm/year. For a 50 year lifetime of the port of Badagry that would mean a 75 mm land subsidence. Land subsidence is further used in the design of the sand breakwater in Chapters 6 and 7.

3.2.3.3 Stability of coastline

Despite the presence of the large LST, the Nigerian coastline is more or less in equilibrium. The latter accounts for areas untouched by human interaction. Some areas are influenced by illegal sand mining but these areas are located out of our scope and is assumed not to be of influence morphologically. In addition occasionally breaching of the beach and the formation of a spit is not unfamiliar, but other than that the coastline changes are limited. It is necessary to confirm this coastline stability as a boundary condition for the research and conceptual variants.

Data sources have been obtained by RHDHV to research the equilibrium of the coastline of Badagry. The coastline positions throughout the years around Badagry are presented in Appendix G in Figure G.1. In

Figure 3.7 the change of coastline is graphically displayed. These coastline positions are derived from satellite images with a resolution of 0.6 to 1.0 meter and therefore taken to be sufficiently accurate.

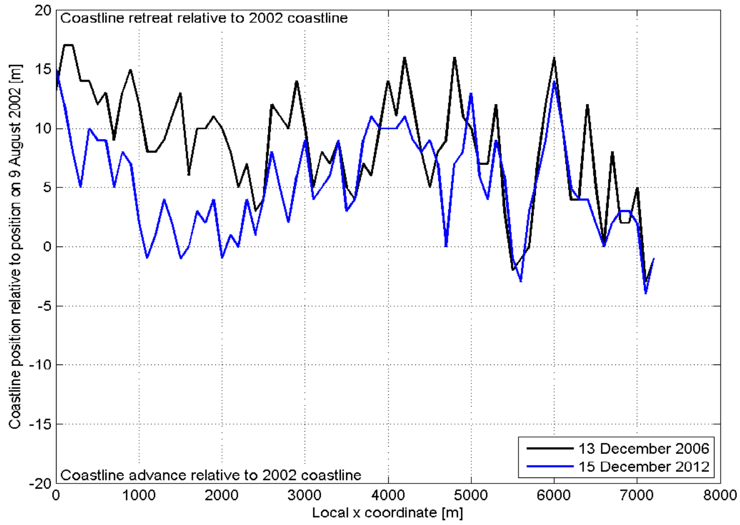


Figure 3.7: Coastline position on December 13 2006 and December 15 2012 relative to the coastline position on August 9 2002. Positive values mean a coastline retreat relative to the 2012 coastline position (RoyalHaskoningDHV, 2014c)

Figure 3.7 illustrates that the coastline in 2006 has retreated roughly 5 to 15 meter relative to the coastline position of 2002. In 2012 the coastline between x = 0 and 2500 meter has partly restored, whereas further east the coastline has stabilized. Within a period of 10 years the observed change in coastline position lies mainly between 0 to 10 meter for this considered stretch of eight kilometres of coastline around the planned port location (RoyalHaskoningDHV, 2014c). This amount of coastline development is taken to be negligible. Conclusively is noted that although a large LST is present, this part of the coast is indeed in equilibrium.

3.2.3.4 Sea level rise

The rising sea level has been a growing problem of late for the coast of Nigeria (Allersma and Tilmans, 1993; Ibe, 1988). As the West African coast is generally low-lying and consists out of vulnerable coastal zones like marshes, lagoons, swamps, estuaries, barrier beaches and others, the Sea Level Rise (SLR) has a significant negative effect on the coast. A change of the mean level of the sea does not change the quantity of sediments forming the coastal system; rather, the coastal system reacts with a redistribution of the sediments. If no sufficient supply of sediments is available, the effect is a regression of the coast line (Allersma and Tilmans, 1993). Due to the set-up of a new water level a new equilibrium situation for the cross-shore beach profile arises. Conclusively this leads to coastal regression shown in Figure 3.8. At Badagry the coastline is, however, currently in equilibrium, meaning that the regression due to SLR is minimal.

Although the coastline is in equilibrium, its important to determine the SLR with respect to design levels for the sand breakwater. The average rate of SLR is estimated to be 1.2 mm/year during the past century. The SLR has been estimated by IPCC-SRES (Intergovernmental Panel on Climate Change - Special Report on Emissions Scenarios) ((IPCC-SRES, 2000)(Odunuga et al., 2014)) at a low of 0.18 meter and a high of 0.59 meter around the year 2100. The difference between the rate of SLR last century and these expected values exists due to the fact that SLR increased non-linear during the past century. Therefore, a moderately conservative rate of SLR is taken to be **5 mm/year** (RoyalHaskoningDHV, 2014a).

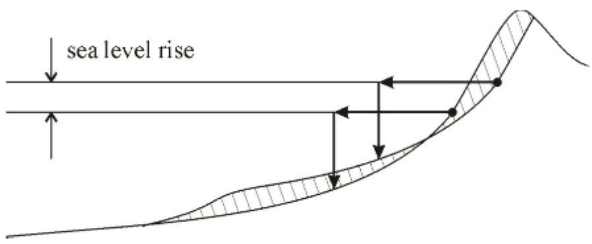


Figure 3.8: Bruun effect: the profile shape remains the same (The length of the vertical and horizontal lines respectively is constant), but the profile moves up and landward as a result of SLR. The volume of sediment eroded from the upper profile s equal to the deposited volume in deeper water (Bruun, 1954)

With the assumed life time of the port (assumed to be the same as RHDHV stated in their studies (RoyalHaskoningDHV, 2014c)) of 50 years, the SLR during its life time is 0.25 meter. This SLR is integrated in the design level later on in Chapter 7.

3.3 Hydraulic conditions

3.3.1 Introduction

In this section all major forcings which influence the hydraulical and morphological system at Badagry are elaborated. Hydraulic conditions are required as input for the modelling and follow from analysing acquired data. The first Subsection concerns an overview of available data. In the following subsection, Subsection 3.3.3 the effect of currents is discussed followed by Subsections 3.3.4 and 3.3.5 in which respectively wind and wave conditions presented. Subsection 3.3.6 examines the tidal influence on the system of Badagry. Finally Subsection 3.3.7 takes up the extreme wave conditions which follow from the previous sections. These extreme wave conditions are required for modelling storm impact with XBeach in Chapter 6.

3.3.2 Overview of data

Regarding the area of interest much hydraulic data has been acquired by RHDHV. This data was beneficial for an accurate set-up and calibration for the morphological modelling of the coastline development and modelling storm impact in respectively Chapter 4 and Chapter 6. Wind, wave and bathymetry data is acquired. The wind and wave conditions are of importance to create a valid wave climate input for both the models. In Table 3.1 a rough schematisation of the gathered data is shown.

Table 3.1: Overview of available data (after Table 3-1, (RoyalHaskoningDHV, 2014a))

Name	Source	Period	Parameters
C-map	OceanView 2001	-	Bathymetry
DEEP Bathymetric Survey	Bathymetric Iso Lines and points	2012	Bathymetry
Argoss offshore position	NOAA global wave model 5°N 3.75°E	Jan 1997 - Dec 2013	Waves & Wind
Argoss offshore position	NOAA global wave model 5°N 3.75°E	April & May 2014 (calibration purpose)	Waves & Wind
DEEP ADCP survey	ADCP recordings/Inland	April to July 2014	Waves & Wind

The Argoss/NOAA data contains wind- and wave data derived from the 3rd generation hind cast wave model WaveWatch III (Wavewatch III, 2005). The model uses ice- and wind-data provided by the National Center of Environmental Prediction (NCEP). The global model provides data along the coasts of the larger oceans and is the source for boundary data for regional models around the world. The global model provides 3-hourly time series of wave spectra covering a period of 17 years (1997-2013). A 3-hourly time series has been assumed to be sufficiently accurate taken into account the stationary wave climate at the Nigerian Coast. Validation of the wave model has been conducted by Argoss with observed altimeter wave heights. The model is validated for an extensive data set of satellite and buoy data, (NOAA 2004, Validation of the Global Database).

The Argoss/NOAA hind cast model data (49672 records, three hourly from the period Jan 1997 – Dec 2013) were extracted at a location offshore of Nigeria, at 5° N, 3.75° East about 157 kilometre offshore of the Nigerian coast, see Figure 3.9. This location is representative for the offshore boundaries of the outer grid of the wave model for creating the nearshore conditions. Local water depth at the offshore position exceeds 1000 m.

The Argoss/NOAA data set contains the following parameters:

- Significant wave height (H_{m0})
- Wind speed (U_{10})
- Peak wave period (T_p)
- Wave and wind directions

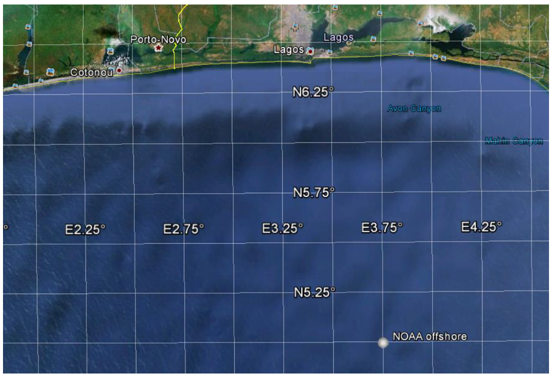


Figure 3.9: NOAA/BMT Argoss location (Google Earth, 2014)

RHDHV also acquired ADCP wave data in April to July 2014. An ADCP (Acoustic Doppler Current Profiler) is a hydroacoustic current meter placed on the bottom of the sea which functions like sonar. It is used to measure water current velocities over a depth (RoyalHaskoningDHV, 2014a). To compare these measurements with model predictions, additional hind cast data was purchased from BMT Argoss containing wave and wind data in April & May 2014. This data has been calibrated against satellite data of the same period, providing the boundary conditions for the calibration of the model against measured wave conditions. For further details of this calibration, see (RoyalHaskoningDHV, 2014a). Wave modelling has been done in previous studies (RoyalHaskoningDHV, 2014a) for the translation from offshore to nearshore using the spectral wave model SWAN. Wave periods and the offshore wind data predicted by the SWAN model is in agreement with the ADCP measurements (RoyalHaskoningDHV, 2014a).

3.3.3 Currents

Two types of currents at the area of interest are studied. Outside the surf zone, currents are dominated by large-scale ocean currents. Nearshore, in the breaker zone, the influence of ocean currents is hardly present and therefore negligible. Wave-driven currents are also present in the breakerzone and are the driving force of the littoral transport along the coast.

The effect of wave-driven currents is taken into account by the modelling program LITPACK itself. Non wave-driven currents are not included automatically. The only current which needs to be assessed at the project location is the Guinea Current, see Appendix B Subsection B.4.

In Appendix B a quick assessment of impact of currents is presented. Conclusively can to be stated that the impact of large-scale oceanic currents is assumed to be negligible. Average values as 0.0261 and 0.0162 m/s are negligible to the present wave-driven currents. These values led to the assumption that the effect of non-wave driven currents can be neglected for this research and therefore not is used as input in the model.

3.3.4 Wind conditions

The major air masses in the lower troposphere of West Africa regard the southwesterlies and the northeasterlies. Nigeria lies on the border of these airmasses. The Inter-Tropical Convergence Zone (ITCZ) is the area where these winds meet. Wind speeds occur between 0.5-2.5 m/s in the night and 2.0-9.0 m/s during the day in Nigeria. An important characteristic for the region is that storms are rare (Allersma and Tilmans, 1993).

From the wind data retrieved from Argoss of the period of 1997-2013 the offshore wind speed is presented in Figure 3.10. In the wind-rose the number 2.1 indicates a 'Light' to 'Gentle breeze', see Chapter Conventions and Definitions.

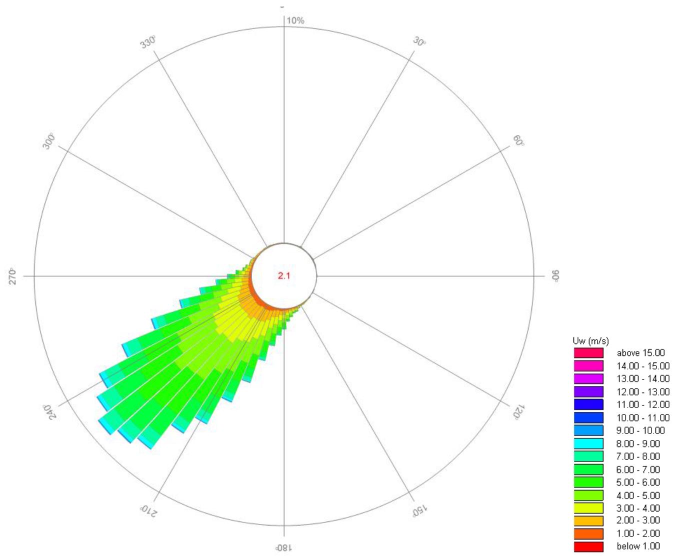


Figure 3.10: Offshore wind speed all year (1997-2013), U_{10} [m/s](RoyalHaskoningDHV, 2014a)

The governing wind direction prevails from the south-west, see Figure 3.10. Figure 3.10 is retrieved with Figure 3.11 in which all the occurring wind speeds are presented in bins. The wind speeds do not exceed a speed of 15 m/s.

	Uw (m/s)	Dir (deg, N) -15.00 15.00	15.00 45.00	45.00 75.00	75.00 105.00	105.00 135.00	135.00 165.00	165.00 195.00	195.00 225.00	225.00 255.00	255.00 285.00	285.00 315.00	315.00 345.00	All classes
0.00	1.00	0.161	0.067	0.067	0.069	0.101	0.089	0.222	0.355	0.410	0.270	0.212	0.107	2.128
1.00	2.00	0.091	0.063	0.061	0.042	0.054	0.184	0.492	1.305	1.344	0.757	0.262	0.167	4.822
2.00	3.00	0.099	0.071	0.056	0.032	0.048	0.178	1.035	3.456	3.587	1.372	0.303	0.105	10.341
3.00	4.00	0.042	0.026	0.044	0.018	0.012	0.149	1.348	6.442	7.313	1.681	0.186	0.069	17.330
4.00	5.00	0.020	0.048	0.050	0.006	0.022	0.111	1.491	9.002	10.321	1.572	0.107	0.026	22.777
5.00	6.00	0.004	0.024	0.030	0.004	0.012	0.075	1.120	7.937	9.736	1.237	0.042	0.006	20.227
6.00	7.00	0.006	0.014	0.018	0.002	0.002	0.018	0.531	4.999	6.906	0.779	0.010		13.285
7.00	8.00	0.002	0.008	0.016		0.002	0.002	0.153	2.021	3.446	0.476	0.002		6.129
8.00	9.00		0.004	0.012			0.010	0.059	0.787	1.206	0.216			2.294
9.00	10.00		0.002	0.012			0.002	0.008	0.186	0.270	0.050			0.531
10.00	11.00			0.002		0.002		0.002	0.044	0.052	0.006			0.109
11.00	12.00								0.012	0.004	0.002			0.018
12.00	13.00							0.002	0.004	0.002				0.008
13.00	14.00													
14.00	15.00								0.002					0.002
	Total	0.426	0.327	0.369	0.173	0.256	0.817	6.462	36.552	44.597	8.417	1.124	0.480	100.000

Figure 3.11: Offshore wind speed all year (1997-2013), U_{10} [m/s] Joint probability of occurrence (RoyalHaskoningDHV, 2014a)

Besides the maximum wind speed another important aspect is the fact that over 80% of all wind events is located in the directional bins of 195° - 255°. It can clearly be stated that the distribution all year is in a narrow bandwidth. In July / August the highest wind speeds are measured. Accurate monthly wind conditions are given in Figure C.11 in Appendix C. The wind conditions are assumed to be similar for the offshore and nearshore positions. In reality, there may be differences due to the presence of land, variety in air pressure, etc. For this research it is assumed to be more or less the same.

To conclude, the effect of wind is only taken into account via the integration of wind set-up and atmospheric pressure in the design water level in Chapter 6. The exclusion of wind and the explanation for its insignificance for the morphological modelling is discussed in Appendix F.4.1.

3.3.4.1 Wind set-up

The wind data was used to determine extreme values for determining the wind set-up is found in Appendix E. Wind set-up is the vertical rise in the still water level (SWL) in front of a structure caused by wind stressed on the surface or the water (Bosboom and Stive, 2012). A storm surge of 0.06 meter for a storm with a 1/100 year return period is found. Although this seems relatively small, the impact of storms is low which is discussed in Subsection 3.3.4.2 . This storm surge is determined for the conventional breakwater and therefore combined with a larger depth than would be present at a sand breakwater. It is assumed that the same storm surge would occur for the sand breakwater.

RHDHV found in a preliminary simulation that the limiting effect of the larger depth is dominant, which actually confirms the low storm surge. Nonetheless, to be on the safe side, a 1/100 year storm surge of 0.15 meter was advised by RHDHV. This extreme value is used in Chapter 6 in the process of determining the design crest height.

3.3.4.2 Atmospheric pressure

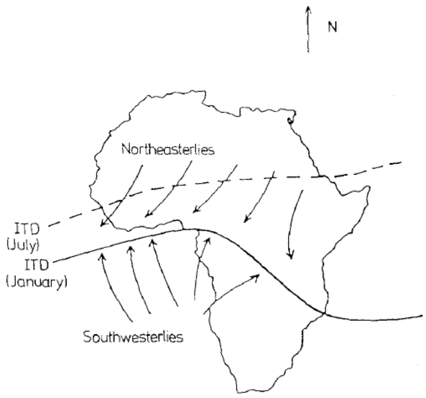
Atmospheric pressure variations also result in water level variations: a local decrease of the atmospheric pressure results in an increase of the water level. This is only possible provided that at another place the atmospheric presure is higher than the mean value. In the Rock Manual (et al., 2007) the relationship between the static rise in water level z_a(m) and the corresponding atmospheric pressure is presented for open water domains in Equation 3.1 (et al., 2007).

$$z_a = 0.01(1013 - p_a) \tag{3.1}$$

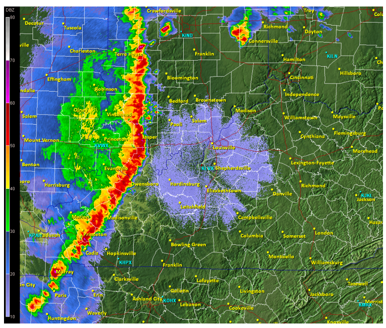
In which:
z_a = static rise in water level (m)
p_a = atmospheric pressure at sea level (hPa)

The atmospheric pressure variation in Nigeria however is known to be very limited, roughly varying between 1006 and 1016 hPa. This variation of 10 hPa corresponds to a water level variation of 0.1 meter.

Although storms are rare, the West African coastline is subjected to 'squalls'. Squalls are initiated through the growth of wave-like perturbations along the surface of discontinuity between the monsoon southwesterlies and the dry northeasterlies in West Africa (Adedoyin, 1989). The discontinuities between these winds cause a sudden increase in wind speeds over 8 m/s lasting for more than one minute, that is most of the time associated with active weather, see Figure 3.12(a).



(a) Air masses in the lower troposphere of West Africa showing the dry northeasterlies and the monsoon southwesterlies (Adedoyin, 1989)



(b) Example of a squall line above the USA (NSSL, 2017)

Figure 3.12: Squall lines in Western Africa

Squalls can form large 'squall lines' of lengths over 100 kilometres , see Figure 3.12(b). These lines are lines of active thunderstorms which leads to characteristics like strong updrafts and downdrafts, precipitation, lightning, thunder. Squall lines are seasonal as they are associated with moving winter cold fronts and summertime troughs of airmass discontinuity. Because of the tidings of the monsoons, the squall occurrences have two peaks at the coast of Nigeria throughout the year and exist in April-May (four squalls on average per month) and a secondary peak in October with an average of two squalls per month (Saha, 2010). During squalls considerably larger wind speeds can occur, but the time and length scale at which squalls occur are considered small.

Pressure variations during squall events are likely to be larger than the normal atmospheric pressure, however, still not significantly large as the duration of squalls is relatively short in the order of minutes. It is assumed that the water level variation during squalls can increase up to 0.2 meter (RoyalHaskoningDHV, 2014c). This extreme value is used in Chapter 6 in the process of determining the design crest height.

3.3.5 Wave Conditions

The West African coast borders on the South Atlantic Ocean. The wave climate prevailing on the coast can be distributed into windsea waves and swell waves. According to (Bosboom and Stive, 2012) the coast can be classified as a "West coast swell environment". The origins of the swell waves lie in distant storms. These storms are generated by the Westerlies. These winds are located in the Earth's Southern storm wave belt which prevailing year-round. Therefore, the swell waves are also year-round occurring. Through frequency and directional dispersion, uniformity in direction, period and height is developed over time. The larger the distance to the origin of the swell wave the larger the uniformity of the wave field is. The globally subjected area of the swell is shown in Figure 3.13. The high-energetic swell waves at the West African coast are originating mostly from a south - south-west direction.

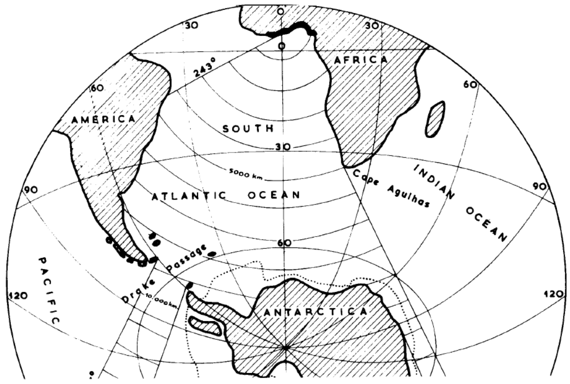


Figure 3.13: The global setting of the subject area (Allersma and Tilmans, 1993)

Throughout the year a consistent wave climate is present. The only known seasonal characteristic of the wave climate is the presence of non-linear waves in the winter. The phenomenon of non-linear waves is chosen not to examine and outside the scope of this research. It is recommended for further detailed research to take into account.

3.3.5.1 Distinction wave fields

In Figure 3.14 the distinction between types of waves has been conducted based on wave steepness. This distinction gives a quick insight in what type of waves are governing in the wave climate. The distinction is made at a wave-steepness of s = 0.014, indicated in yellow. The formula for wave-steepness is given in Equation 3.2 (Holthuijsen, 2007).

$$S_0 = \frac{2\pi H_s}{g {T_p}^2} \tag{3.2}$$

The coast of Badagry is dominated by swell waves as clearly is shown by the distinction shown in Figure 3.14. The highest waves as well as the largest segment of the waves occurs in the swell domain. Lastly, Figure 3.14 displays large swell wave periods. Waves with large wave periods are characteristic for the Nigerian coast. These waves have often travelled large distances and carry much energy.

The distinction in wave fields by steepness is not the distinction in swell and windsea as it is used for this research. The distinction in waves has been conducted by ARGOSS with a method designed by Hasselmann et al. (1996). This method uses an algorithm for the retrieval of ocean wave spectra from synthetic aperture radar image spectra.

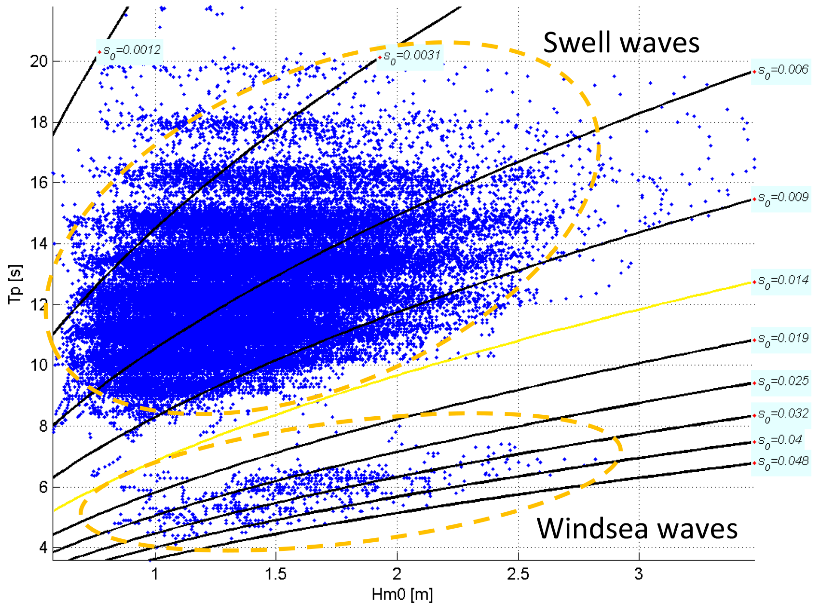


Figure 3.14: Scatter plot of wave height (H_m0) against wave period (T_p) with different wave steepness lines from ARGOSS data

3.3.5.2 Nearshore Conditions

In Appendix C a detailed description is given of the offshore analysis of the wave climate. From the offshore conditions a transformation was conducted to the nearshore wave climate conditions with SWAN. The results of this transformation are shown in Appendix D. In Section D.1 the nearshore characteristics of the windsea are discussed followed by the results of the nearshore swell characteristics in Section D.2. In Table 3.2 the average nearshore characteristics of the windsea waves and swell are presented. The wave climate as it has been implemented in the LITPACK model is elaborated in Appendix F in Subsection F.4.

Table 3.2: Summary of average characteristics of wave climate nearshore

	H_s (m)	Tp (s)	Dir (°)
Average Windsea	0.91	4.89	208.5
Majority Windsea	0.5-1.25	3-6	195-225
High segment Windsea	1.25-2.50	6-9	-
Average Swell	1.42	12.68	189.1
Majority Swell	1-1.75	10-14	185-195
High segment Swell	2.25-3.75	18-22	-

3.3.6 Tide

The prevailing tide on the Nigerian coast is governed by semi-diurnal components (Bosboom and Stive, 2012), which is found to be micro-tidal at Badagry. For this section of coast a tide between 0 and 2 meter with respect to Lowest Astronomical Tide (LAT) is present with an average tide around 1 meter. Along the entire coastal stretch of Nigeria the tidal wave occurs nearly simultaneously. This tidal wave is displayed in Figure 3.15 in which the yellow dashed line marks the situation for Badagry. Due to the simultaneous behaviour of the tidal wave, coastal longshore currents caused by these tides are weak. Only near inlets and estuaries, tidal streams are of significantly more influence on the Nigerian coast (Allersma and Tilmans, 1993).

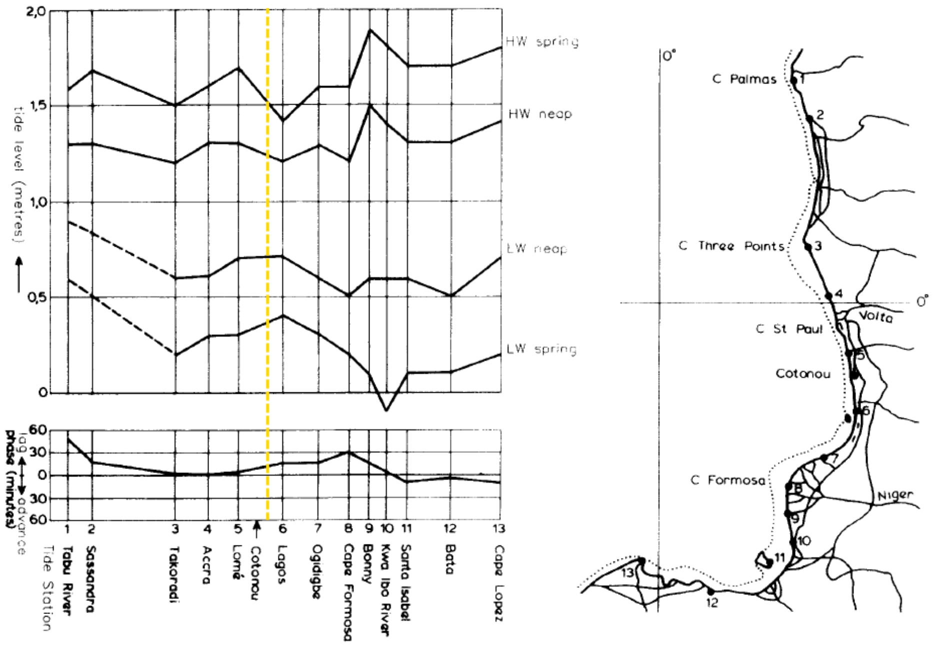


Figure 3.15: Propagation of tide waves at the Nigerian coast (Allersma and Tilmans, 1993)

No long term water level data of the exact location of Badagry is currently available. Data of a rather similar location from the International Hydrographic Organisation database has been used to examine the tide. In this database nine tidal constituents are included of which the closest location is Lomé. Another location is more nearby, however, this regards a location in open sea and therefore not comparable. The city Lomé in Togo is located approximately 180 kilometres from Badagry. Although this is a large distance, relatively to the scale of the tidal wave itself it is not. In addition, the tidal wave occurs nearly simultaneously along the coast. Conclusively, both beaches have similar characteristics. Due to this equality the assumption is made that the tidal constituents which are governing in Lomé are representative for Badagry (RoyalHaskoningDHV, 2014b).

A comparison of the astronomical tide at Lomé and measured water levels in Badagry in April 2014 has been made by RHDHV. This comparison supports the similarity of the tide, see Figure 3.16. Lomé's high waters seem to be higher than the Badagry's high waters. To define the Badagry tidal levels as the tidal levels in Lomé is therefore a conservative approach (RoyalHaskoningDHV, 2014b).

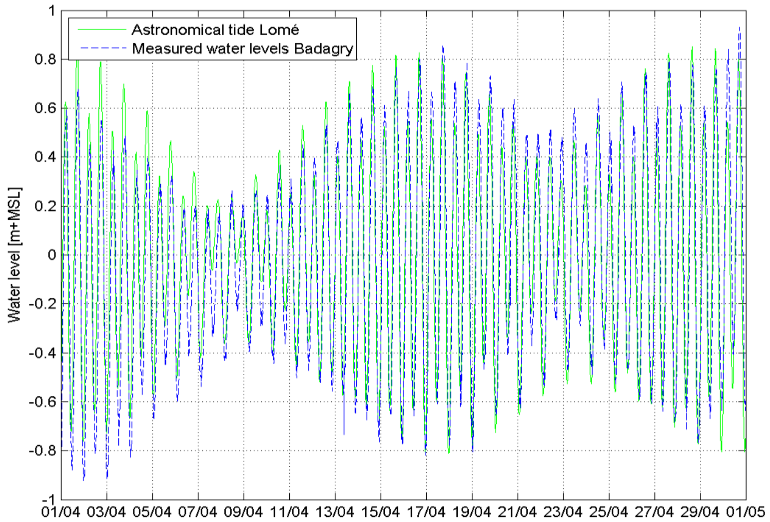


Figure 3.16: Astronomical tide in Lomé and measured water levels in Badagry in April 2014 (RoyalHaskoningDHV, 2014c)

Earlier studies used tidal constituents from Lomé to construct a 19 years long water level time series (RoyalHaskoningDHV, 2014a). The time series has been constructed for this period such that it is larger than the 18.6 year variation of the tidal components. This water level time series is consequently used to determine characteristics tidal levels like MSL and others. The resulting tidal levels are summarized in Table 3.3 (RoyalHaskoningDHV, 2014b). The datum MSL is mainly used in this study.

Table 3.3: Characteristic tidal levels in Lomé, Togo, assumed to be representative of tidal levels at Badagry, Nigeria ((RoyalHaskoningDHV, 2014b), (RoyalHaskoningDHV, 2014c))

Characteristics tidal level	**Relative to MSL [m]**	Relative to LAT [m]	Level relative to CD [m]
HAT	0.94	1.93	1.69
MHWS	0.78	1.77	1.53
MHHW	0.63	1.62	1.38
MHW	0.53	1.52	1.28
MSL	0	0.99	0.75
MLW	-0.53	0.46	0.22
MLLW	-0.65	0.34	0.10
MLWS	-0.84	0.15	-0.09
LAT	-0.99	0	-0.24

The Relative tidal range (RTR), see Equation 3.3, is a useful parameter to depict wave and tidal influence. For RTR < 3 we find wave-dominated beaches (Bosboom and Stive, 2012).

$$RTR = \frac{MSTR}{H_b} \tag{3.3}$$

In which:
MSTR = Mean Spring Tidal Range [m]
H_b = Wave height just before breaking [m]

The H_b at Badagry is close to 1.4 meter concluded from results of the littoral drift transport module conducted in this study with LITPACK, see Figure 3.17. In Section 4.3 this module and set-up is discussed.

$$RTR = \frac{2.0}{1.4} = 1.43 \tag{3.4}$$

As the mean spring tidal range is approximately 2 meters the RTR is be found to be 1.43. This value of RTR clearly shows that the coast of Nigeria is wave-dominated. Because of this wave-dominance the tide is averaged and not taken into account for the long term coastline evolution, see Appendix F.3. The tide ís taken into account as a timeseries for short term storm modelling in Chapter 6.

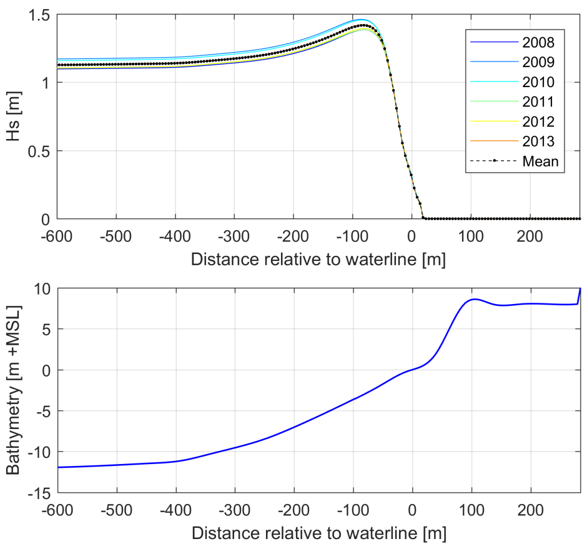


Figure 3.17: Results of the Littoral drift module at Badagry

3.3.7 Extreme wave conditions

In order to model storm conditions extreme values are required. In previous studies (RoyalHaskoningDHV, 2014a) the extreme nearshore wave conditions have been determined for multiple locations around the port of Badagry. Three locations from various are established to be governing for the position of the left breakwater, see Figure 3.18 for the locations. Swell and windsea waves have been treated as separate phenomena. These conditions have been determined for two water levels 0 m + MSL and 1 m + MSL due to the 1/100 year water level.

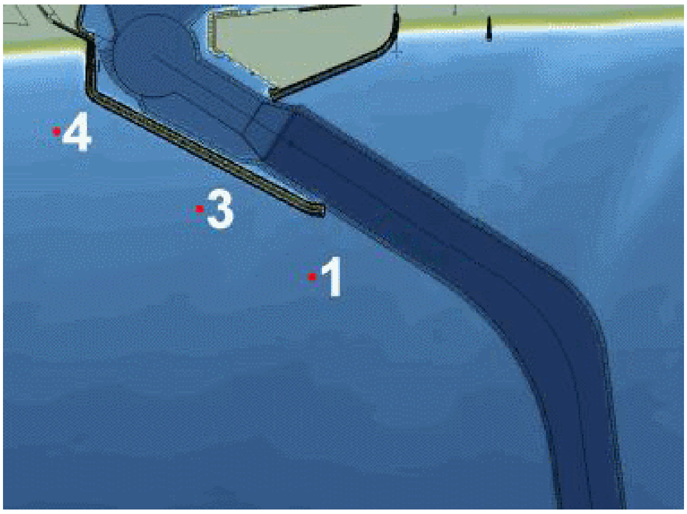


Figure 3.18: Specific locations for extreme values (RoyalHaskoningDHV, 2014a)

3.3.7.1 Results nearshore swell

Nearshore extreme values have been determined for swell conditions, see Table 3.5. The table shows the significant wave height (Hm0) and wave peak period (Tp) as a function of return period (Qs) for the nearshore locations. Because the waves all come from around 180° - 210°, no directional analyses has been applied. For the 1/100 year swell event the governing figures are displayed in red. The 1/100 year swell conditions was the actual design storm condition for the conventional breakwater.

In Subsection 6.2 a 1/100 year water level is determined which amounts to 1.29 m + MSL. However, from the Table 3.5 can be concluded that the higher modelled water levels do not lead to the higher extreme values. On the contrary, if higher water levels would be modelled the maximum wave height probably would not be higher because of a smaller shoaling effect.

Table 3.5: Nearshore extreme swell wave conditions (RoyalHaskoningDHV, 2014a)

Location	Return period	0 m + MSL		1 m + MSL	
		H_{m0}	T_p	H_{m0}	T_p
1	1/1	2.71	16.01	2.70	15.98
	1/10	3.30	17.66	3.27	17.58
	1/100	**3.87**	**19.12**	3.81	18.98
	1/1000	4.42	20.44	4.33	20.22
3	1/1	2.80	16.04	2.77	15.97
	1/10	3.4	17.67	3.37	17.61
	1/100	**3.96**	**19.30**	3.94	19.03
	1/1000	4.48	20.29	4.48	20.30
4	1/1	2.80	15.95	2.77	16.00
	1/10	3.37	17.50	3.35	17.60
	1/100	3.89	18.79	**3.90**	**18.98**
	1/1000	4.36	19.89	4.41	20.19

3.3.7.2 Results nearshore windsea

The results for the windsea conditions for a 1/100 event are displayed in Table 3.6. Other sectors are not relevant due to the absence of data in the sector, or insufficient data to determine correct extreme values. The maximum nearshore windsea wave height of 2.9 meter is found at point three with a 7.76 seconds wave period.

Table 3.6: Nearshore 1/100 windsea design conditions. Note that some sectors are not relevant for all points (RoyalHaskoningDHV, 2014a)

Location	165° - 195° sector		195° - 225° sector		225° - 255° sector	
	H_{m0} [m]	T_p [s]	H_{m0} [m]	T_p [s]	H_{m0} [m]	T_p [s]
1	2.77	7.47	2.86	7.66	1.48	6.27
3	2.81	7.57	2.9	7.76	1.45	6.18
4	2.71	7.51	2.77	7.68	1.39	5.98

3.3.7.3 Resulting storm conditions

The 1/100 design conditions for swell are summarised in Table 3.7. These conditions are the maximum of the simulations for water levels both 0 m and 1.00 m + MSL. Conclusively, it can be said that swell conditions are the most relevant. At point three, the maximum wave height of 3.96 meter and corresponding peak wave period 19.30 seconds are found. This is an unexpected result as location four is located closer to the coast in shallower water. The maximum extreme condition would be expected to exist at location four. Assumed is that the extreme values of location four are are subjected to bathymetry irregularities. Therefore is chosen to use the extreme values of location three further on in the research. These values will be used for required storm conditions and expanded with other requirements in Chapter 6.

Table 3.7: Nearshore 1/100 swell design conditions from 180-210∘ (RoyalHaskoningDHV, 2014a)

Location point	180° - 210° sector	
	H_{m0} [m]	T_p [s]
1	3.87	19.12
3	3.96	19.30
4	3.90	18.98

4

One-line Coastline evolution model

4.1 Introduction

In this chapter, the model used for the modelling of coastline evolution, LITPACK, is discussed. In Section 4.2 the model LITPACK along with its used components is introduced. Hereafter in Section 4.3 the results of the littoral drift module are presented. Along with these results, the confirmation of the littoral drift along the coast of Badagry is elaborated. This elaboration is used for model validation as well, necessary for accurate modelling. The complete configuration of the coastline evolution module and applied data in the software are presented in Appendix F. For a successful modelling execution the set-up was improved by means of a calibration. The calibration method which has been used in this research is presented in section 4.4. The results of the modelling of LITPACK are analysed in Chapter 5.

4.2 Model introduction

The model LITPACK (**LIT**toral **P**rocesses **A**nd **C**oastline **K**inetics) is part of the larger software package MIKE Zero which has been made by the international software development and engineering consultant DHI-Group. LITPACK or nowdays "Littoral Processes FM", concerns the modelling software focusses on one-line shoreline change. The Littoral Processes FM module is an integrated modelling system that models transport in points and along quasi-stationary coastlines. The model contains several modules which form a complex model that can be used for studies regarding non-cohesive sediment transport (DHI-GROUP, 2017a). The following modules are used in this research:

- Littoral drift
- Table generation
- Coastline evolution

4.2.1 Littoral Drift

The littoral drift module calculates the non-cohesive sediment transport in one or multiple cross-sections. Assumed is that each cross-shore profile represents uniform conditions along a straight coast. LITPACK determines and calculates how waves propagates to the coast and in term result in wave-driven currents. With this information it computes the non-cohesive LST. The total sediment load consists out of two elements, firstly the bed load and secondly the suspended load.

The sediment transport is calculated in LITPACK by adding the bed load transport to the sediment transport in suspension. The bed load transport is calculated using the model of Engelund and Fredsøe (1976) where the bed load transport is calculated from the instantaneous "Shields parameter". An elobaroration is presented in Appendix H.1. The vertical variation of the suspended sediment concentration is calculated from the vertical diffusion equation for suspended sediment, according to Fredsøe et al. (1985). Taking this variation into account, the suspended sediment transport is then calculated as the product of the instantaneous flow velocities and the instantaneous sediment concentration (DHI-GROUP, 2017b).

The 'intra-wave period' sediment transport model "STPQ3D" forms the basic sediment transport description for combined wave and current action in all the LITPACK modules (DHI-GROUP, 2017a). The main output from this model type is the bed load and suspended load in the two directions (DHI-GROUP, 2017a). The hydrodynamic model includes a description of propagation, shoaling and breaking of waves, calculation of the driving forces due to radiation stress gradients, momentum balance for the cross-shore and longshore direction giving the wave setup and the longshore current velocities (DHI-GROUP, 2017a).

Taking the variations on the hydrodynamic climate into account, the net/gross littoral climate is determined. The main result from the module is the sediment transport across the profile as well as the integrated littoral drift rates over the simulation period.

4.2.2 Transport Table Generation

This module generates a littoral drift transport table. This table is used in the coastline evolution module to decrease computation time. Instead of calculating instant littoral drifts in every location in the coastline evolution module, it uses the transport table which exists out of pre-generated littoral drifts. The table consist out of numerous littoral transport rates for all possible hydrodynamic conditions.

4.2.3 Coastline evolution model

This module is based on the theory of the one-line models, see Subsection 2.4.2. The module computes coastline change with respect to a straight baseline. As stated before, the cross-section along the coastline is assumed to remain constant during coastline retreat or advancement. That means that coastal morphology is only described by cross-shore position at a given long-shore position.

The sediment transport rates are extracted from transport tables and placed over the desired profiles along a coastline. On this coastline multiple nourishments (sources) or extractions of sand (sinks) or structures can be placed which influence this coastline evolution. Four types of structures can be taken into account: groynes, jetties, off-shore breakwaters and revetments.

The main results are computed by calculating all 1D grid cells separately at sequential time-steps. In every grid cell at each single time step, the longshore transport is calculated. The results in each cell together with inputs as sources and sinks lead to the total coastline change in that specific cell. All those cells together can give insight in the coastal development. The shoreline is then compared to the previous time step to determine the change in coastline position. The main result is the new computed position of the coastline which is read as output in the distance from the baseline to the coastline, the local littoral drift rates and accumulated transport along the coastline (DHI-GROUP, 2017a).

4.3 Littoral Drift Module

The Littoral drift module gains insight in the littoral transport along one or multiple cross-shore profiles. As it is assumed that the coast of Badagry is uniform, only one cross-shore profile is used. To generate results with the Littoral Drift Module most of the input is comparable to the input and set-up of the Coastline Evolution model which can be found in Appendix F. The content of this profile is presented in Subsection F.2. On this cross-shore profile multiple wave climates have been applied as the wave climate changes gradually alongshore through the modelled area. The clarification of distribution of the wave climates along the coast of Badagry, along with the choice of data in this wave climate, is presented in Subsection F.4.1. In Figures 4.1 is the distribution of wave time series over the stretch of coast displayed.

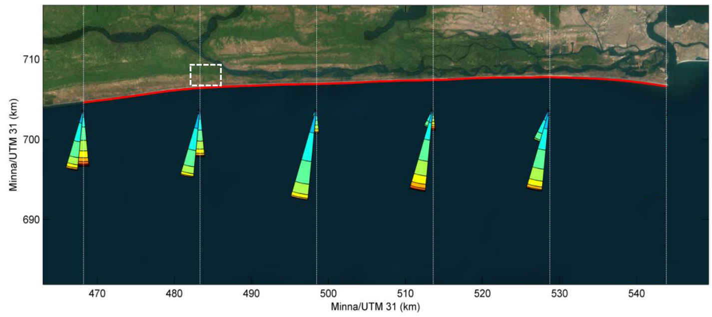


Figure 4.1: Distribution of wave time series along stretch of coast

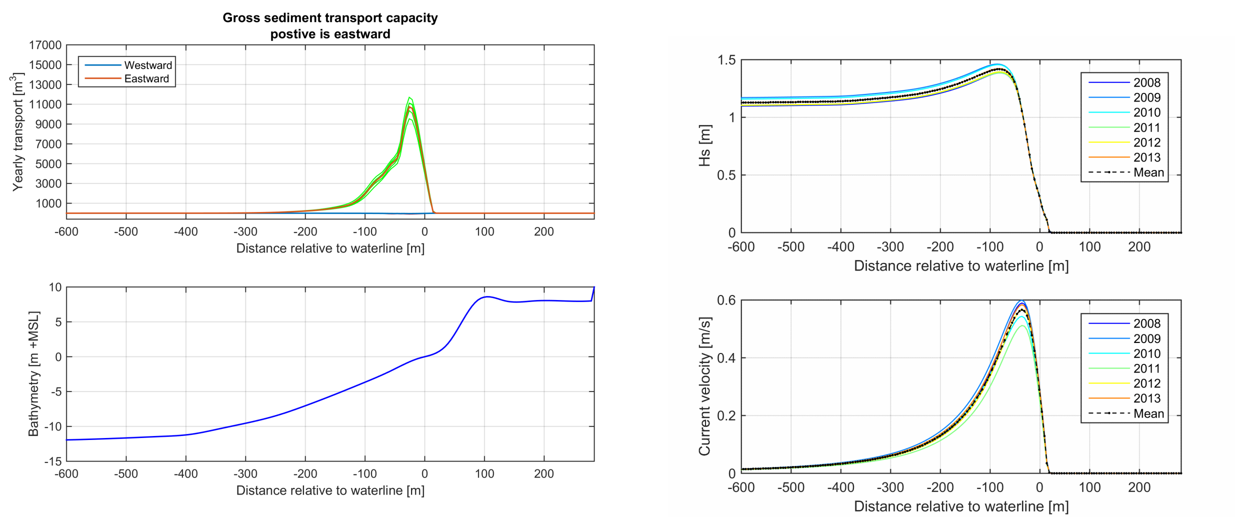
The littoral drift is determined along a stretch of coast with multiple wave climate inputs. Because of this, a transient is present in littoral drift along the coast of Badagry to Lagos. The littoral transport along the coast of Badagry as it has been determined with the LITPACK module is presented in Table 4.1. It considers five points along the coast where different wave time series have been developed. The distance along the coast is mapped from east to west. A big difference in LST is shown in between 30.27 kilometre and 45.41 kilometre. This is probably a consequence of a decrease in wave direction which causes the approaching waves to be more perpendicular to the coastline creating a smaller LST. Also the significant wave height and period is smaller with 45.41 kilometre compared to 30.27 kilometre. Further down the coast at 60.54 kilometre the LST decreases due to again a decreasing wave direction. At the last location the LST is computed to increase. This is a result of little change in the peak period and wave direction but a relatively big increase in significant wave height. Evidently there are many more parameters involved in the computation of the LST and next to this the relations of the parameters are not always linear. Changes in these parameters, however, show rough impacts on the LST.

Table 4.1: LST distribution along the coast

Distance from start of coastline [Km]	15.14	30.27	45.41	60.54	75.67
Average annual net LST [m^3/year]	**1,277,000**	**1,279,000**	**897,858**	**691,000**	**703,000**
Average Significant Wave Height (m)	1.49	1.68	1.48	1.51	1.64
Average Peak Wave Period (s)	13.32	13.43	13.16	13.24	13.39
Average Wave Direction w.r.t. TN (°)	194.06	191.25	189.68	186.88	185.68

The created wave time series closest to the area of interest is evidently the most important one. The characteristics of the generated littoral drift close to Badagry are displayed in Figure 4.3(a) and Figure 4.3(b). Almost all transport takes place in the first 100 meter in front of the shoreline, see Figure 4.3(a). This occurrence is in coherence with aerial photos confirming the width of the surfzone being indeed 100 meter, see Figure 4.2.

Figure 4.2: Width of surfzone around Badagry (Google Earth, 2017)



(a) Annual LST over Bathymetry

(b) Significant Wave Height and Current velocity over Bathymetry

Figure 4.3: Characteristics of Littoral Drift close to Badagry

In Figure 4.4 and Table 4.2 the littoral drifts versus coastal orientation are presented for parts of the Western African coast. The location of the area of interest lies in between Lomé harbour and Lagos Harbour. The littoral drifts from Table 4.1 are in the same order of magnitude as literature states (Allersma and Tilmans, 1993). Other reference projects confirm these LST rates, see Appendix A.

With respect to Table 4.2 it needs to be noted that the wave direction is bigger than the wave direction in this research. The explanation for this difference lies with the fact that in Table 4.2 the direction of both windsea waves and swell waves together is presented instead of the isolated swell waves only.

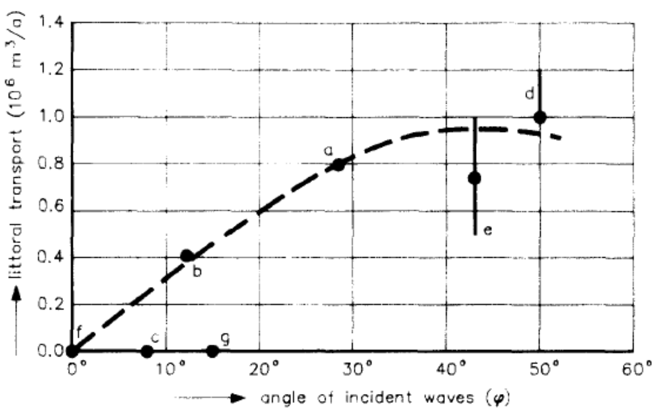


Figure 4.4: Littoral drift versus coastal orientation: (a) western Canal de Vridi; (b) eastern Canal de Vridi; (c) eastern Ivory Coast; (d) Lomé Harbour; (e) Lagos Harbour; (f) western Niger Delta; and (g) top of Niger Delta(Allersma and Tilmans, 1993)

Table 4.2: Observed Littoral Transports and Directions of Coasts and waves (Allersma and Tilmans, 1993)

Location	LST (million m^3 per year)	Normal to coast (°E)	Wave direction (°E)	Angle of wave approach, ϕ (°)
Western Canal de Vridi	0.8	172	200	28
Eastern Canal de Vridi	0.4	188	200	12
Eastern Ivory Coast	0	192	200	8
Lomé Harbour	1.0-1.2	160	210	50
Lagos Harbour	0.5-1.0	177	220	43
Western Niger Delta	0	220	220	0
Top of Niger Delta	0	205	220	15

4.4 Calibration

All along the coastline of West-Africa and with that Nigeria, numerous human interferences are found. Coastal impact caused by men close to the area of interest is found east of Badagry. The impact concerns an oil tanker which was grounded on the beach in June 1997. This is a phenomenon which is very common on the Nigerian coast. The grounding of a ship of this size can have a serious impact on the coast as shown in Figure 4.5. This grounding of the ship causes serious coastal development within short amounts of time. Using satellite images and aerial imagery obtained from Google Earth at this precise location the rough coastal development can be estimated with the accuracy of meters. This coastal development is reproduced with LITPACK for calibration purposes. Via this way a real scenario of coastal development can be used to calibrate the model in order to model as accurately.

Figure 4.5: Aerial photograph of stranded tanker at 08-09-2001 (Google Earth, 2017)

4.4.1 Method of calibration

The theory behind the coastline evolution model lies with the gradient of the LST. The Equation 4.1 which is a continuity equation describes this relation. The equation is required to to analyse for a sufficient understanding in calibration.

$$\frac{dy}{dt} = -\frac{1}{h_{act}} * \frac{dQ}{dx} + \frac{Q_{source}}{h_{act}\Delta x} \tag{4.1}$$

In which:

dy	= Cross-shore change in coastline position [m]
t	= Time [years]
h_{act}	= Active height which consists out of Active Depth (AD) and Beach Height (BH) [m]
$\frac{dQ}{dx}$	= Rate of change in LST [m^3/year/m]
dx	= longshore discretisation step (m)
Q_{source}	= Supply or extraction of sediment from sources or sinks, including eventual dune erosion (m^3/year)

The initial output results of the LITPACK model do not match the exact results as the aerial photos and data show. The models results were far off the realistic results. To improve these results the model needs to be calibrated. Three parameters were chosen to be shifted and adapted with which the model is calibrated in this research.

- Active Depth
- Beach Height
- Dunes

4.4.1.1 Active profile

The Active Depth (AD), indicates how much of the profile below mean water level is subject to erosion or accretion. The Beach Height(BH), indicates how much of the beach above mean water level is subject to erosion or accretion. These two together form the active profile, see Figure 4.6.

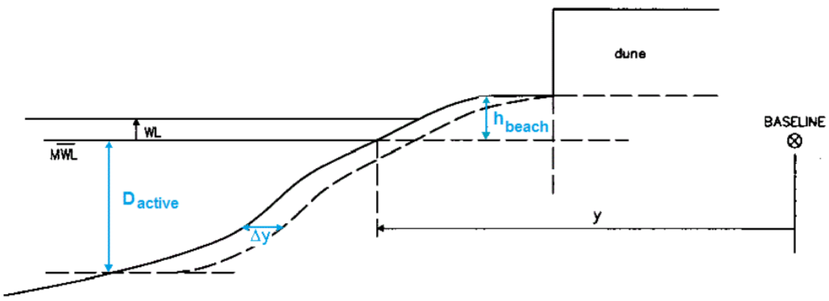


Figure 4.6: Definition of active profile height and initial movement of coastline (DHI-GROUP, 2017a)

4.4.1.2 Dunes

In LITPACK dunes can be added to the model. Dunes are implemented over a length "y" which indicates the distance from the baseline, see Figure 4.7. The implementation in the model is through a surplus of dQ when the location of the dune is reached by the erosion of the coast. The dune is uniformly present cross-shore and therefore the surplus of dQ is constant over "y".

Dunes generally not present at the area of interest, however, for calibration reasons they are researched and implemented. They are implemented in such a way that the model is artificially adapted to compute results which are more realistic. This is necessary as the model overestimates the erosion caused on the coast. It is purely an artificial way of increasing locally the dQ as it concerns a sudden source of sand whenever the dune line has been reached by erosion.

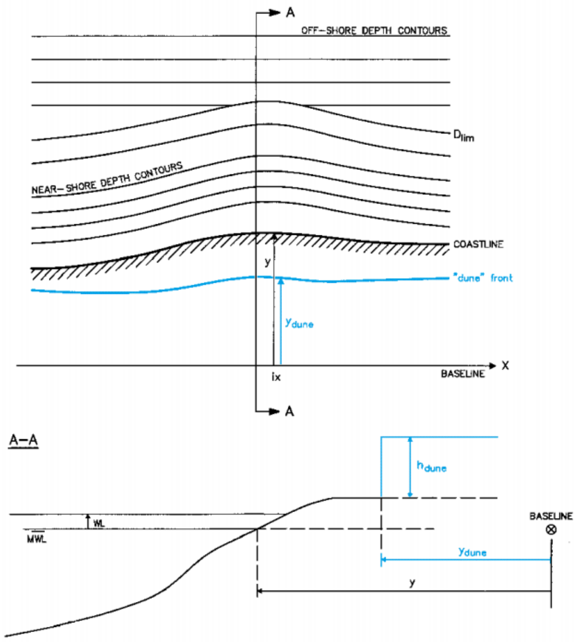


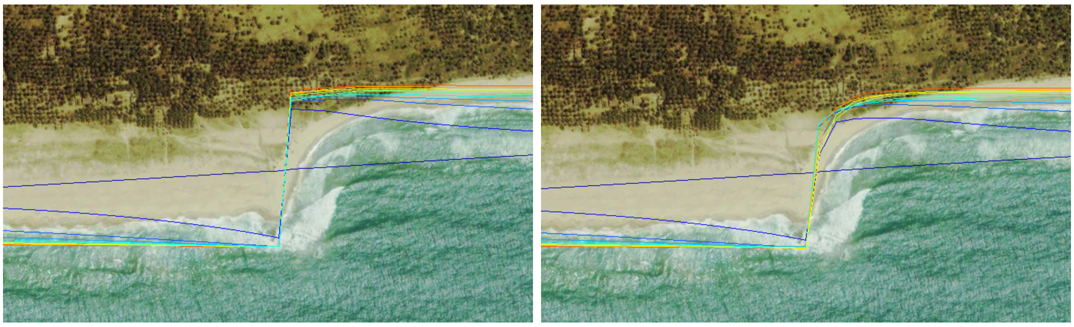
Figure 4.7: Definition of dune position and dune height (DHI-GROUP, 2017a)

4.4.1.3 Coastal structure

To model the stranded tanker on the coast, a hard coastal structure needed to be implemented in the LITPACK model. Several options were possible for this implementation. Either a jetty, an offshore breakwater or a groyne could be used to schematise this event. A groyne appeared to be the most accurate. The reason that an offshore breakwater was not used is it is numerically not capable of forming a closed tombolo behind an offshore breakwater (DHI-GROUP, 2017b). A tombolo is the accretion of sand behind a detached breakwater which connects to the original coastline. In case this phenomenon is not able to occur, the modelling software will not be able to model the accreted sand behind the offshore breakwater and cut off the LST behind it.

With a jetty, diffraction behind the structure is included in the model. Although this phenomenon is included, a jetty is a long and thin structures which continues far beyond the breakerzone. In this case, the tanker is located in the breaker zone less diffraction of waves is needed to model and therefore the advantage is negligible.

In Figure 4.8 the use of a jetty and the use of a groyne for modelling the coastal evolution are shown. The lines shown in the pictures regard the coastal evolution throughout the years. The use of a jetty leads to a straight start whereas the groyne displays the curve in the erosion pocket more accurately. This is consequence of the fact that a jetty blocks the complete littoral transport as it stretches out farther than the breaker zone and also the littoral zone.



(a) Coastal development modelled with Jetty period 1997 - 2007, current background image 04/07/2006 Google Earth

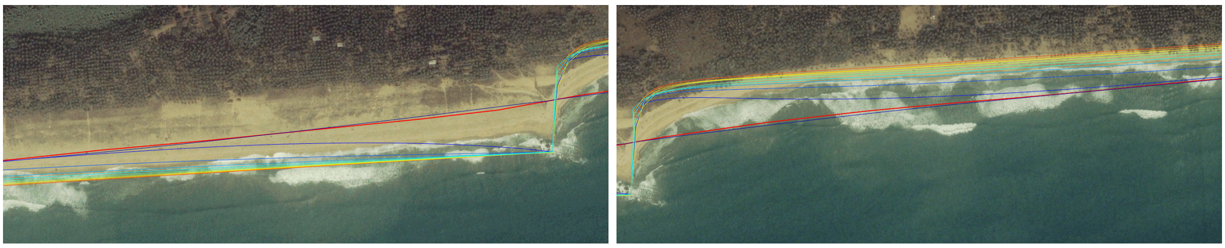
(b) Coastal development modelled with Groyne period 1997 - 2007, current background image 04/07/2006 Google Earth

Figure 4.8: Difference in modelled coastal development by using different coastal structures. Pink: Undisrupted coastline. Blue: 1997. Red: 2007.

4.4.2 Adjustments to model

To calibrate the model using the impact of the tanker, it is assumed that the cross-shore profile at the location of the tanker is equal to profile at Badagry. Physically realistic values extracted from measurements at location are an AD of 10 meter and a BH of 3 meter. The AD of 10 meters is realistic since the closure depth is located around 10-11 meters depth. As stated before, dunes are not present in reality so for the physically realistic set-up no dunes are taken into account. The calibration is started using these values resulting in Figures 4.9. The lines shown in the Figures 4.9 regard the coastal evolution throughout the years.

(a) Close-up at 10-12-2006 (Google Earth, 2017)



(b) Accretionside at 10-12-2006 (Google Earth, 2017)

(c) Erosionside at 10-12-2006 (Google Earth, 2017)

Figure 4.9: Results of coastal evolution using physically realistic values. Pink line is undisrupted coastline. Blue line is modelled coastline in 1997. Red line is modelled coastline in 2007.

A general remark about the LST concerns that at the time of the stranding of the tanker its location was still somewhat offshore. Although the tanker stranded the first year, in reality there has still been sand passing by because there was no tombolo formed yet. In the modelling with LITPACK this was not possible to take into account. A total blockage of LST has been modelled since the start. Therefore the accretion side in reality is much lower than the modelled accretion shows in Figure 4.9(b). The erosion on the other side is less than it should be in the first amount of years as shown in Figure 4.9(c). This phenomenon is especially present in the first couple of years. After the formation of the tombolo the modelling results begin to start look more like the realistic coastline development.

It is desirable to calibrate the model in a way in which its accuracy is assumed to be sufficient. The **maximum acceptable error** for modelled deviation was assumed to be **10 meters**.

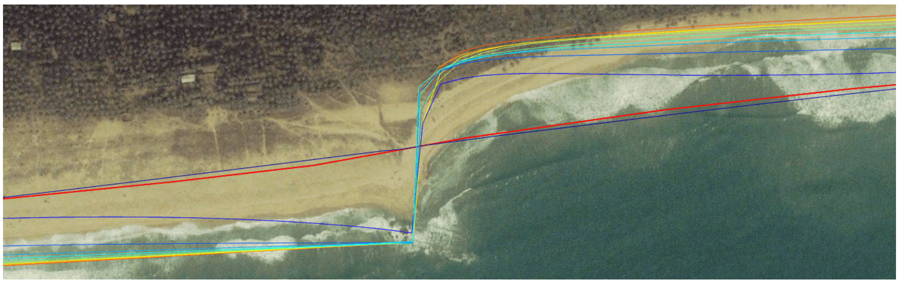
The accretion side is modelled actually quite realistically if the 'tombolo' disruptions at the start are neglected. By adapting the AD and BH to respectively 12 and 5 meters the deviation in accretion below 10 meters.

The erosion side gives the largest error in the results. The erosion pocket as shown in Figure 4.9(c) is overestimated in the year 2007. This overestimation of erosion is desired to compensate by calibration. The calibration is done by adding an artificial dune in the model. Multiple runs with different values for dune specifications lead to a set-up which matches the aerial photographs sufficiently.

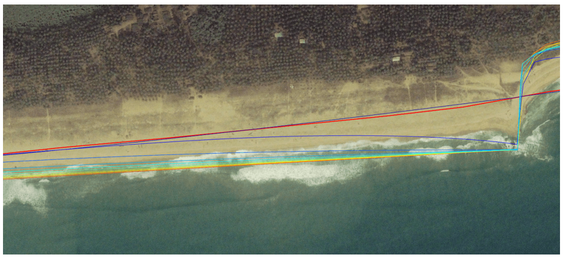
Since the total erosion pocket is 85 meters big a similarity of $\frac{85}{95}$=89% exists taken into account that the acceptable error is 10 meters. The model performs in this case **89%** of 100%. The set-up where this maximum acceptable error exists contains an artificial dune with an height of 10 meters at 50 meters behind the coastline. An artifical supplement dQ of 10 m/per running meter alongshore per meter erosion is added like this in the continuity equation 4.1. These dune parameters are less realistic values but they do make the model fit more accurate. In Figure 4.10(b) and 4.10(c) clearly can be seen that the modelled results match the coastline development more accurately than in Figure 4.9. For this reason these values are used during this study.

Table 4.4: Final calibrated values

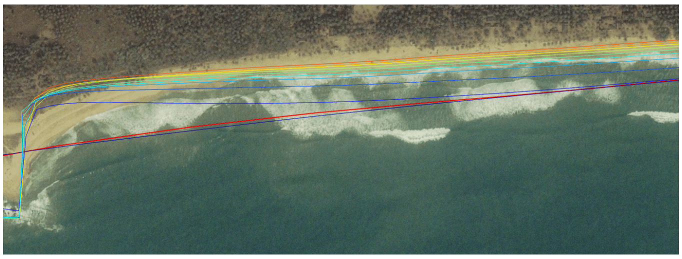
Parameter	Calibrated value
Beach height [m]	**5**
Active depth [m]	**12**
Dune height [m]	**10**
Dune position w.r.t. coastline [m]	**50**



(a) Close-up at 10-12-2006(Google Earth, 2017)



(b) Accretionside at 10-12-2006(Google Earth, 2017)



(c) Erosionside at 10-12-2006(Google Earth, 2017)

Figure 4.10: Results of coastal evolution using calibrated values

5

Analysis of coastline development

5.1 Introduction

In this chapter **phase 1a** of the study is presented, see Subsection 2.3. This phase concerns analysing of the long term coastline development in order to find conceptual variants for the sand breakwater. The objective in this chapter is to establish an alongshore set-up of the sand breakwater (partly) made out of sand which is morphologically stable. As stated in Chapter 2, the process of finding conceptual variants for a sand breakwater is iterative. The set-up and calibration of the model from respectively Appendix F and previous chapter have been used in this process. In Section 5.2 the process is started by establishing the equilibrium orientation of the coastline. This orientation has been implemented in set-ups and coastline mitigation has been exerted for the purpose to engender conceptual variants. During this iterative process three variants arose in which the coastline development is in control. These variants are presented in Section 5.3 in which also the coastal development of these variants is discussed. These variants are further integrated in the design in Chapter 6 and 7. Following, the section on the large scale coastline impact by the conceptual variants of the sand breakwater. Finally, in Section 5.5 the sensitivity of these variants is discussed by presenting the sensitivity of multiple parameters. These sensitivities are important for the further design of the sand breakwater and taken into account in Chapter 7.

5.2 Coastline Equilibrium

As stated in Chapter 2 it is desirable to establish conceptual variants in which no LST occurs at the tip of the breakwater. Since the wave climate is significantly uni-directional almost no LST will occur at the tip of the breakwater when the equilibrium orientation is implemented in the design of a sand breakwater. Therefore, it is necessary to establish the equilibrium orientation of the coastline. The starting point for the morphological analysis is analysing the behaviour of the shape of the conventional breakwater design as coastline. In Figure 5.1 the modelling result is shown. The white line is the original modelled coastline. A period of 30 years has been modelled with the blue line being the start year and the red line being the 30th modelled year. In Appendix F the overall scope of the model is displayed.

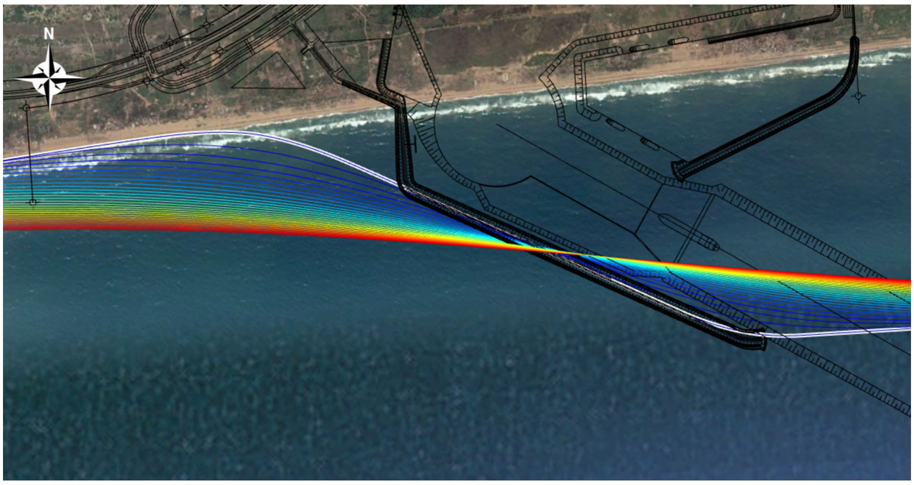


Figure 5.1: Overview of coastline development of initial set-up for a period of 30 years

In Figure 5.1 the coastline rapidly adapts itself and develops due to the rather consistent forcing of the governing wave climate. Since, in Figure 5.1 in the west the red line is located in front of the original coastline, accretion occurs. About halfway the conventional breakwater design (in black) this accretion effect has faded and the red line crosses the original modelled coastline. From here eastwards the red line is displayed behind the original modelled coastline. Thus erosion has taken place in the east. The result shows that a coastline without any hard structures is rather vulnerable.

The morphological behaviour of this sand nourishment shows that orientation of the coast is forced to a certain orientation. The orientation imposed by the wave climate beholds the 'equilibrium orientation'. After the modelled period of 30 years the orientation can be assumed to be constant over the time. The equilibrium coastline orientation angle with respect to the True North (TN) as found in the figures amounts to 278.6°. When taking into account that the average swell direction concerns 189° this is a logic response of the coastline to its forcing. After later runs with LITPACK a new more detailed equilibrium orientation is determined. This orientation is presented at Subsection 5.3.2 and amounts to 277.5°. In the design of the conceptual variants this orientation is taken into account instead of the other orientation. The answer to the first sub-question is achieved:

The equilibrium coastline orientation for the prevailing wave climate is 277.5° w.r.t. TN.

5.3 Multiple Variants

In the previous section it is clearly visible that a coastline completely made out of sand is vulnerable. It will flatten out within a short period to the equilibrium orientation. A hybrid version of the sand breakwater, partly consisting out of sand and partly hard structure, is consequently proposed. For an optimal hybrid design, in which as much sand as possible is used, it is essential to implement the equilibrium coastline orientation.

First of all, the equilibrium orientation of the coastline leads to little coastline development due to low amounts of LST. Consequently it would establish a rather more consistent coastline. This more consistent and stable coastline is desirable for it requires a smaller cross-section for the sand breakwater design than a design with a different orientation. Evidently a more rank design is cheaper due to less required material, and hence more feasible, to construct.

Secondly, this small LST capacity actually leads to a huge decrease in siltation in the approach channel due to almost no LST entering the trench. The little siltation which still might occur originates possibly in the navigation of ships.

From the perspective of using this coastline orientation in the advantage, an iterative process of creating new set-ups was started. These set-ups led to three conceptual variants worth investigating for their profitability versus the conventional breakwater design. These three variants are investigated as the emphasis is on different unique components. This makes them interesting to interesting to investigate to feasibility but also to compare to each other. Until this phase the analysis was executed with a modelling period of 30 years. Given the fact that the assumed economical design lifetime of the port is 50 years, the modelled period is also set to 50 years. Modelling the complete design lifetime of the port enables us to draw conclusions on what approximately would be an optimal form for the variants.

5.3.1 Variant 1: Sand Nourishment

The broad idea of variant one is the **maximum use of sand** within the sand breakwater and **as little hard structures** as possible. Which in essence embodies the idea of a sand breakwater as much as possible.

The location of the tip of the breakwater along with the orientation of the approach channel is assumed to be a boundary condition. In Figure 5.1 it is clearly visible that the whole modelled coastline is redistributed due to no coastal mitigation. The coastline would erode into the port and approach channel on the east side if the tip of the breakwater is not protected sufficiently.

The tip of the breakwater is vulnerable to erosion if it is unprotected.

5.3.1.1 Tip of the breakwater

The tip of the sand breakwater was, therefore, necessary and inevitable to construct as a hard structure. By modelling the tip of the breakwater as a hard structure and implementing an equilibrium orientation of coast next to the breakwater, variant 'Sand Nourishment' was found. This variant was named: "Sand Nourishment" since the only hard structure in this hybrid design concerns the tip of the sand breakwater, see Figure 5.2.

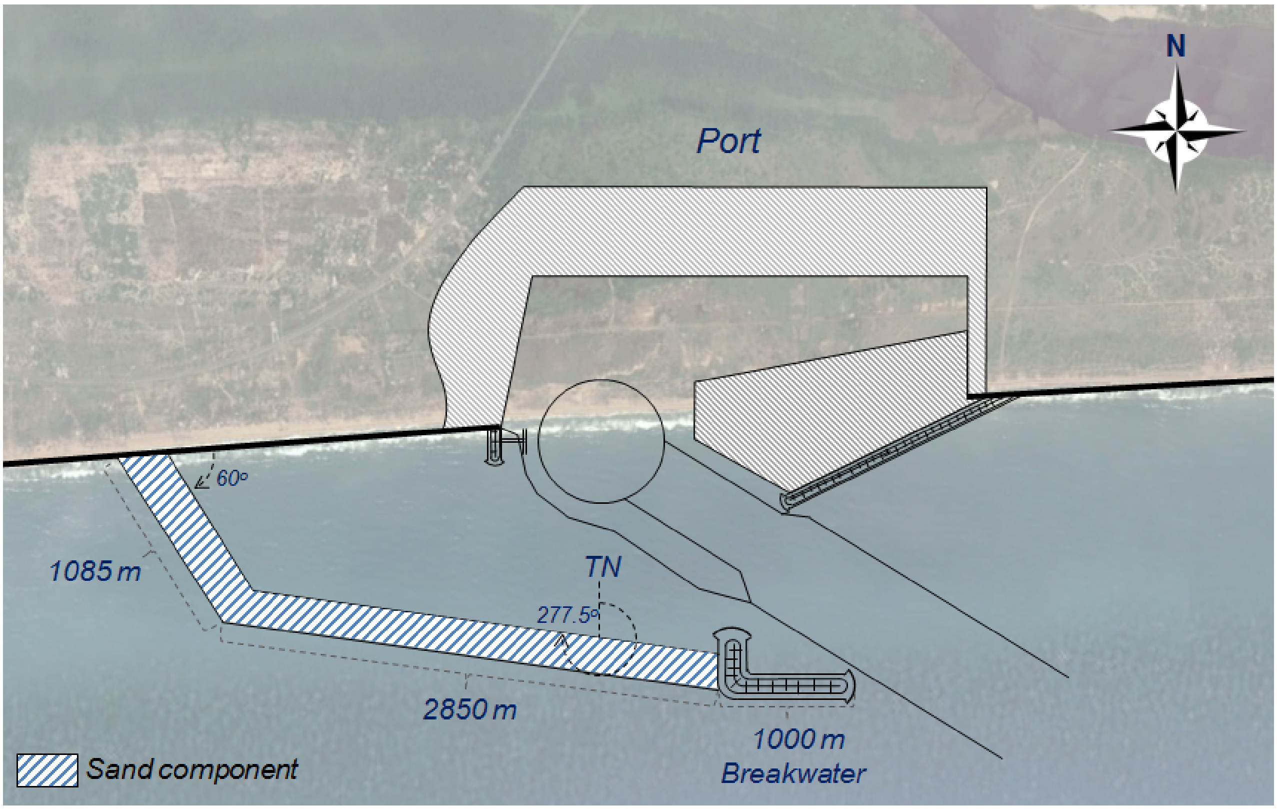


Figure 5.2: Schematisation of variant 'Sand Nourishment', not to scale

The length of the tip is based on the cross-sectional space available for a sand breakwater. This construction requires a certain cross-sectional width. The location to where this tip needs to be constructed depends on the amount of coastline development during the lifetime of the port.

The length of the tip of the breakwater is required to be 800 meter horizontally and 200 meter along the emerged part of the sand dune. This 200 meter protects the side of the emerged part of the sand component and also functions as a groyne for 50 meters offshore. With these properties, sufficient room is present for a dune cross-section behind the coastal development. The tip of the breakwater is assumed to be constructed similarly as it was conventionally designed as hard structure.

5.3.1.2 Sand Component

This variant furthermore contains a stretch of coast with equilibrium orientation which borders to the breakwater on the east and continues on the west under angle of 60° with the original coastline. Next to this 60° angle coastline space is available for the accretion of sand. This is important as a constant LST is present from the west approaching the sand breakwater. By accreting on this location, no sand by-pass is present around the sand breakwater for a certain amount of time.

Since, the coastline constructed on the left of the hard tip is unprotected, this coastline will redistribute its sand due to the wave climate. A **duality** exists in the optimum of the design of this variant. If the stretch of sand created under equilibrium orientation is longer, the coastline development throughout the years decreases. However, if the stretch of sand created under equilibrium orientation is longer, the space which is available for the accretion of sand on the west of the sand breakwater is smaller.

As a boundary condition it is assumed that:

No sand by-pass should occur in the economical design lifetime of the port.

This design lifetime is 50 years and taken this fact into account the variants optimum is found through iterative modelling, see Figure 5.3. This set-up has been modelled for the period of 50 years and is shown in Figure 5.3. The white line is the initial modelled coastline.

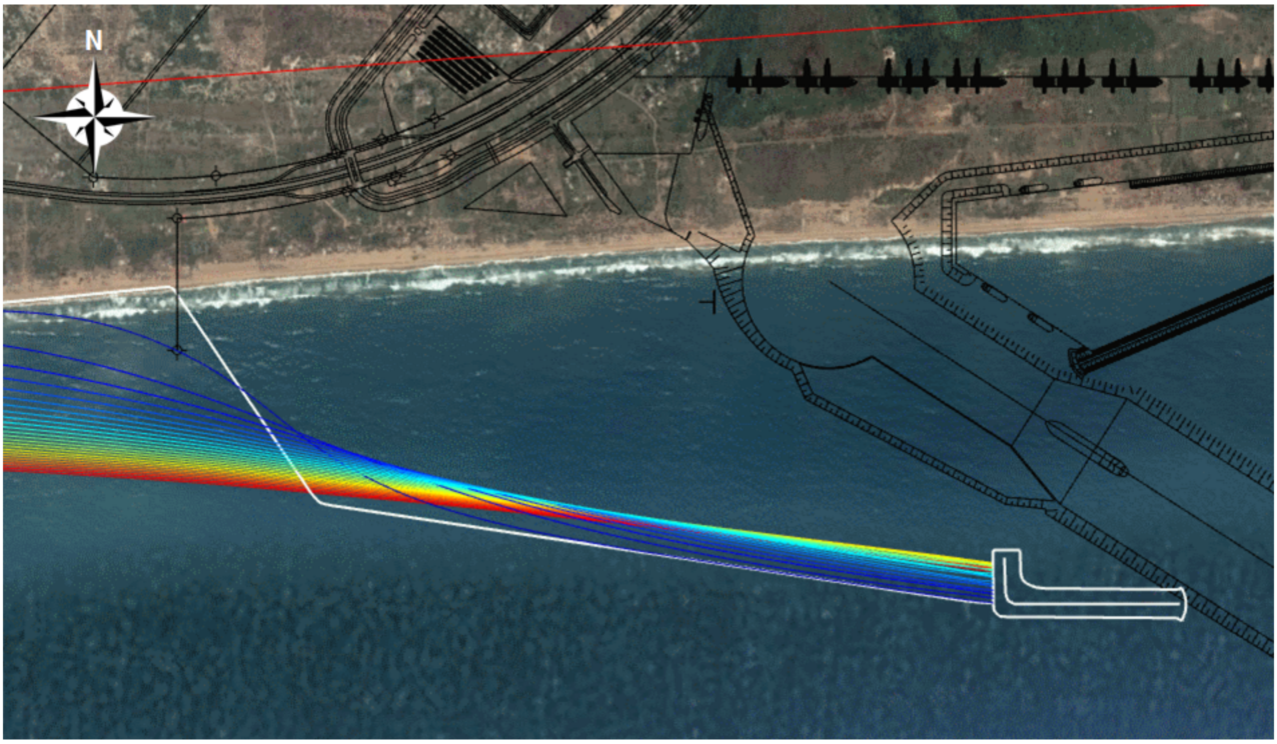


Figure 5.3: Overview Coastal development for a modelled period of 50 years for variant 'Sand Nourishment'

From Figure 5.3 it can be concluded that on the east side of the coastline over time not only erosion occurs, but from 2041 on accretion starts to occur. The accretion is, however, not yet trespassing the tip of the breakwater within the modelled 50 years.

After an amount of years the accretion on the west increased to certain quantities that the original coastline has moved up all the way until the new coastal stretch orientation. Since the orientation of the coastal stretch is starting to change due to more accretion, it loses its equilibrium orientation. Consequently, the transport capacity is starting to increase again eastwards. This transport capacity is initially small after 50 years, see Figure 5.4.

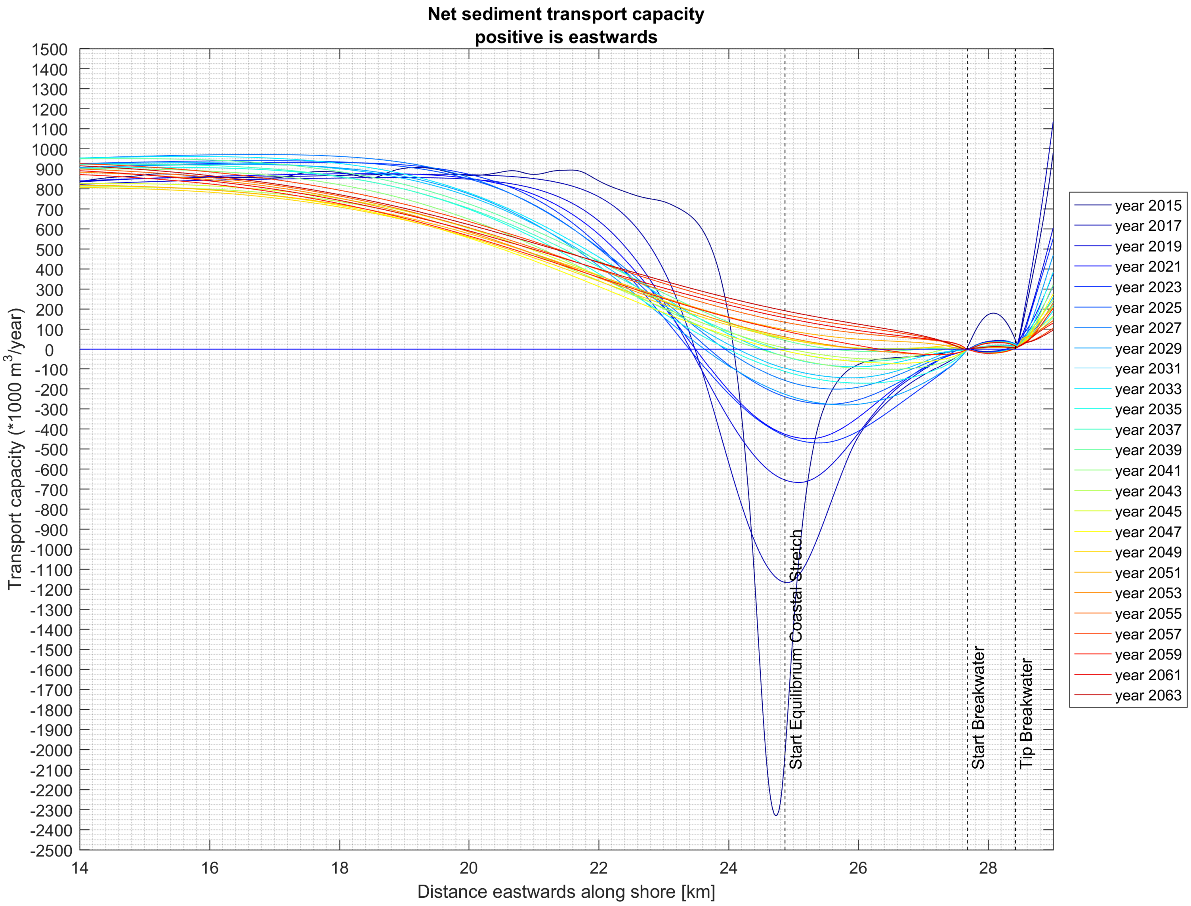


Figure 5.4: Overview Net LST capacities along coastline for a modelled period of 50 years for variant 'Sand Nourishment'

The coastline in the modelled period does not develop to the tip of the breakwater on the east side, see Figure 5.5. This means that no sand has passed by the tip and little siltation in the channel has occurred.

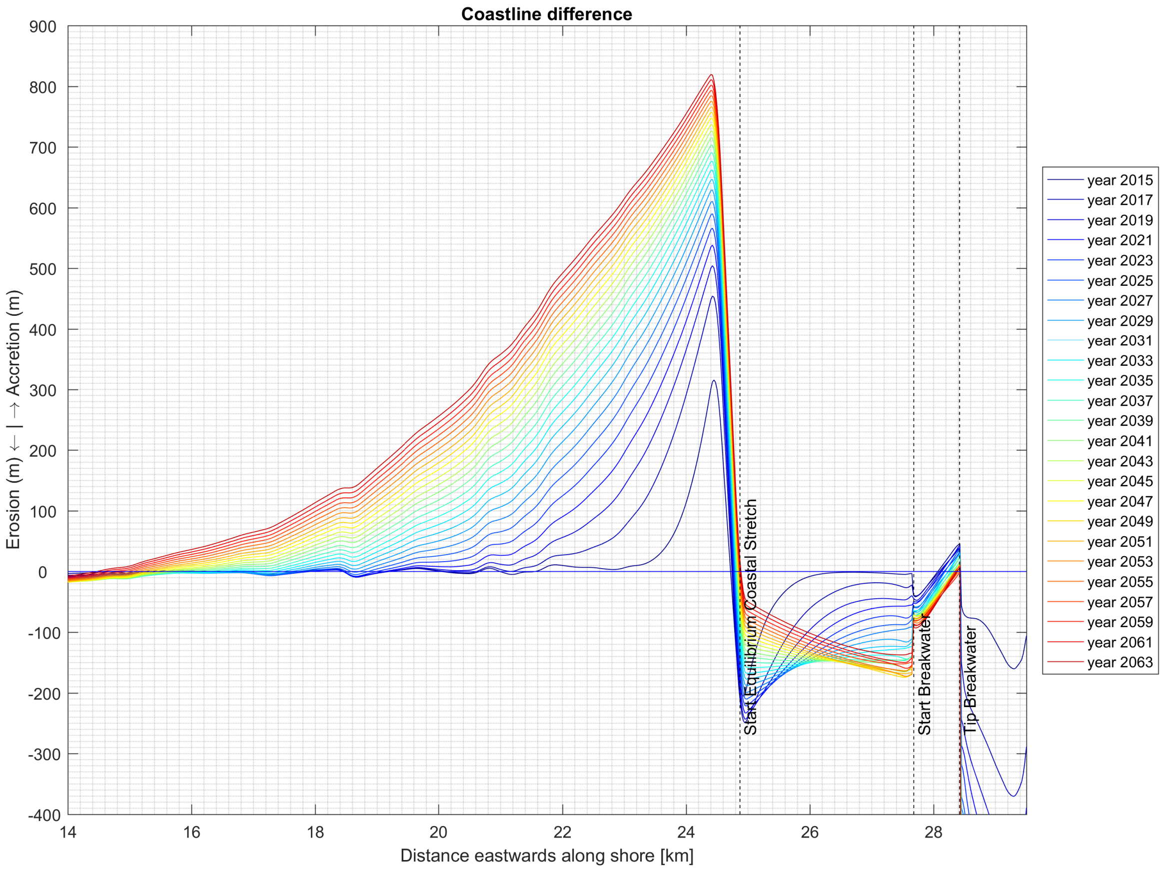


Figure 5.5: Overview Coastal development for a modelled period of 50 years for variant 'Sand Nourishment'

The optimisation of the variant is done by first of all providing enough space for 50 years of accretion. Secondly, no LST is present on the end of the breakwater on the east. Thirdly, this set-up provides enough space between the coastline and the port for a minimum cross-section of a dune. This minimum cross-section is further elaborated in Chapter 6.

With the optimum found for this conceptual variant it is important to determine the coastline development which occurs over the constructed coastline. This insight is required to gain as the design of the sand breakwater should be able to withstand this coastline development throughout the years. In Figure 5.5 the coastal development throughout the years is shown. In Figure 5.6 a smaller scope is displayed which shows the coastline retreat sufficiently. On the east side of the equilibrium coastal stretch the maximum coastline retreat is less than 185 meter. On the west side, at the start of the equilibrium coastal stretch, the coast erodes less than 250 meter. On the west, therefore, a surplus of 250 meter is required for extra cross-sectional width in the design. On the east this extra width amounts to 185 meter.

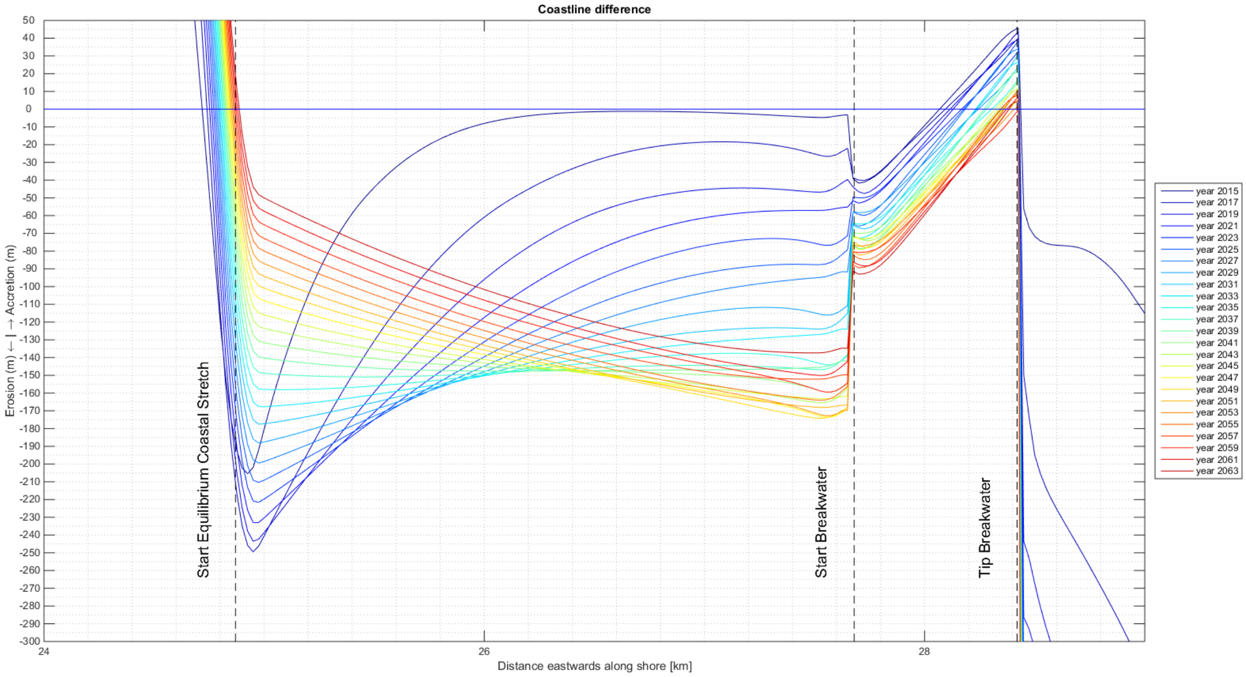


Figure 5.6: Coastal development zoomed in for equilibrium coastal stretch with new orientation variant 'Sand Nourishment'

This variant is from a morphological perspective very active since the tip is the only hard structure in this set-up. As a result, much sand is required for this variant to live up to the requirements. The consequence in volume of sand and economic cost is presented in Chapter 7 and this variant is further integrated in the design in Subsection 7.2.1.

5.3.2 Variant 2: Groyne

Besides the tip of the sand breakwater being a critical part, every part of the sand breakwater without equilibrium orientation is vulnerable to coastal development.

Parts of the sand breakwater constructed not in equilibrium orientation are vulnerable to erosion if they are unprotected.

Therefore, this orientation needs to be preserved. In variant 'Sand Nourishment' the benefit of the absence of LST on the equilibrium orientation of the coastline loses its strength, as soon as the sand is redistributed westwards. The sand present on the stretch is redistributed quickly after construction. The coastline development due to this LST is considerable large and consequently, also large next to the hard tip of the sand breakwater on the east side. When this redistribution of sand from the equilibrium coastal stretch to the west is prevented, the erosion next to the tip would be prevented as well or decreased at least. As stated in the previous subsection, the location until where a possible sand breakwater can be constructed, depends on the amount of coastline development during the lifetime of the port. Due to this prevention of coastal erosion on the east, less space is required for a cross-sectional width for a design and therefore the length of the tip of the breakwater as hard structure can be decreased. This is evidently desirable as this would be more cost-effective taken into account the high price of materials.

Variant 'Groyne' was invented via preventing the redistribution of sand from variant 'Sand Nourishment' by blocking the westwards LST with a groyne.

This groyne is designed to be constructed from the original coastline all the way to the tip of the coastal stretch which is placed with equilibrium orientation. In this way a 'Pocket Beach' is created which is protected on the west side by a groyne and on the east side by the tip of the breakwater, see Figure 5.7.

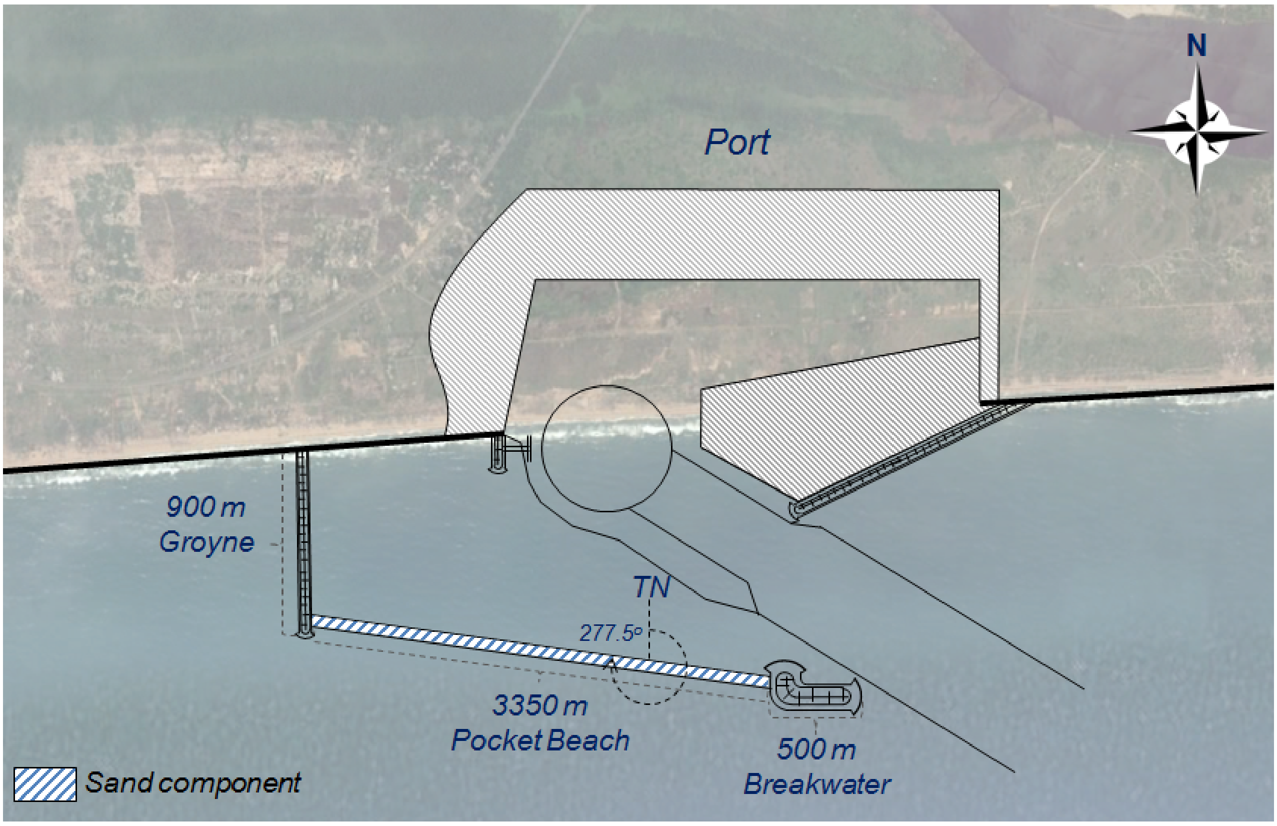


Figure 5.7: Schematisation of variant 'Groyne'. not to scale

The trade-off in this variant is the length of the groyne and the length of the coastal stretch versus the amount of years no sand by-pass is occurring. As a boundary condition it is assumed, similarly as in the previous variant, that no sand by-pass should occur in the economical design lifetime of the port. Through an iterative process the optimal length of a groyne was established at 900 meters, providing enough space on the west side for accretion for 50 years. The optimised set-up is modelled for 50 years and shown in Figure 5.8. The groyne is displayed on the left in yellow. The tip of the breakwater in this variant was able as to decrease in length as the coastal erosion was less. Due to this decrease it is required to be 400 meter horizontally and only 100 meter along the cross-section of the emerged part of the dune next to it.

Figure 5.8: Overview Coastal development for a modelled period of 50 years for variant 'Groyne'

The results of LITPACK show little LST on the equilibrium coastal stretch. As the groyne prevent sand redistribution, the pocket beach remains morphologically stable during the modelled 50 years. In Figure 5.9 is clearly graphically visible that zero transport capacity is present along the coastal stretch. Consequently, as little siltation as possible occurs in the approach channel.

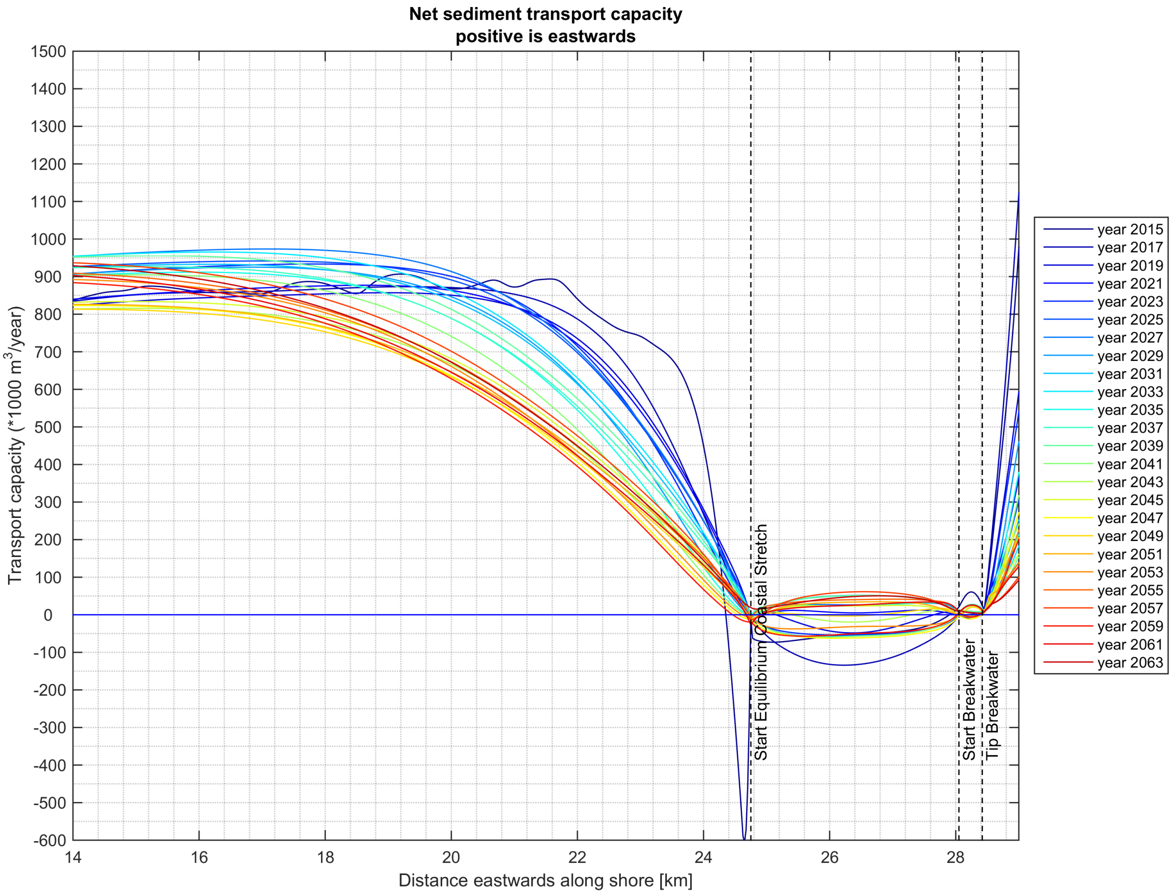
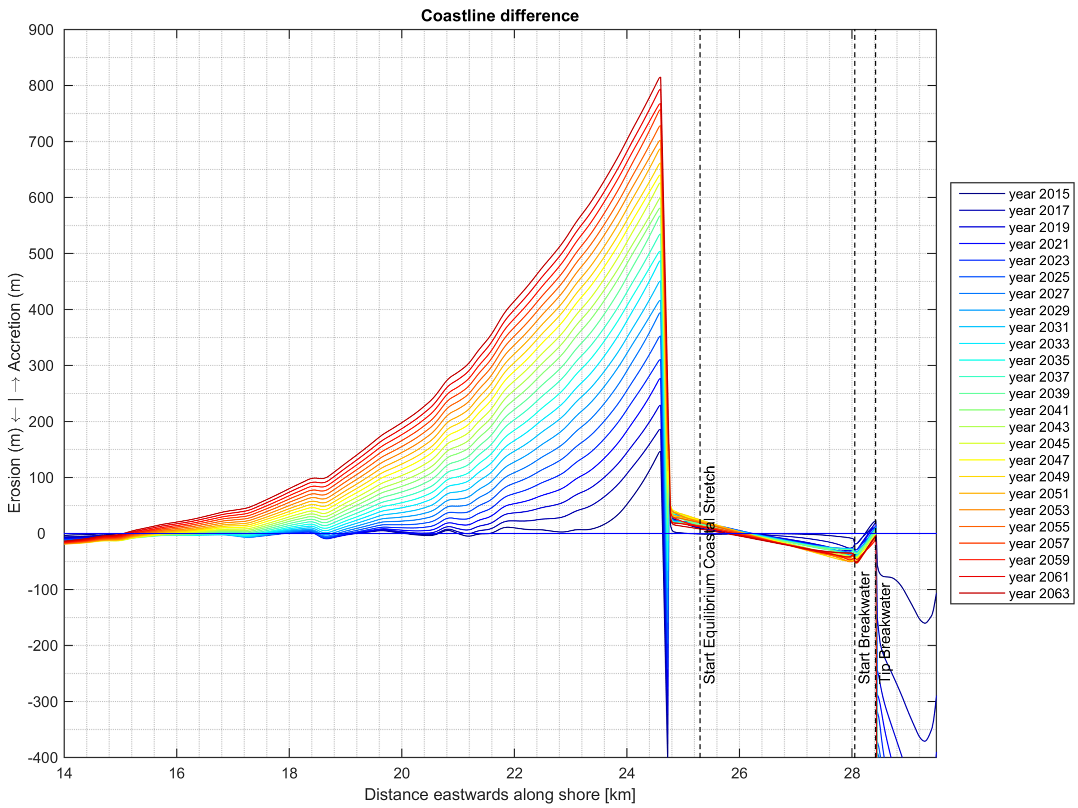


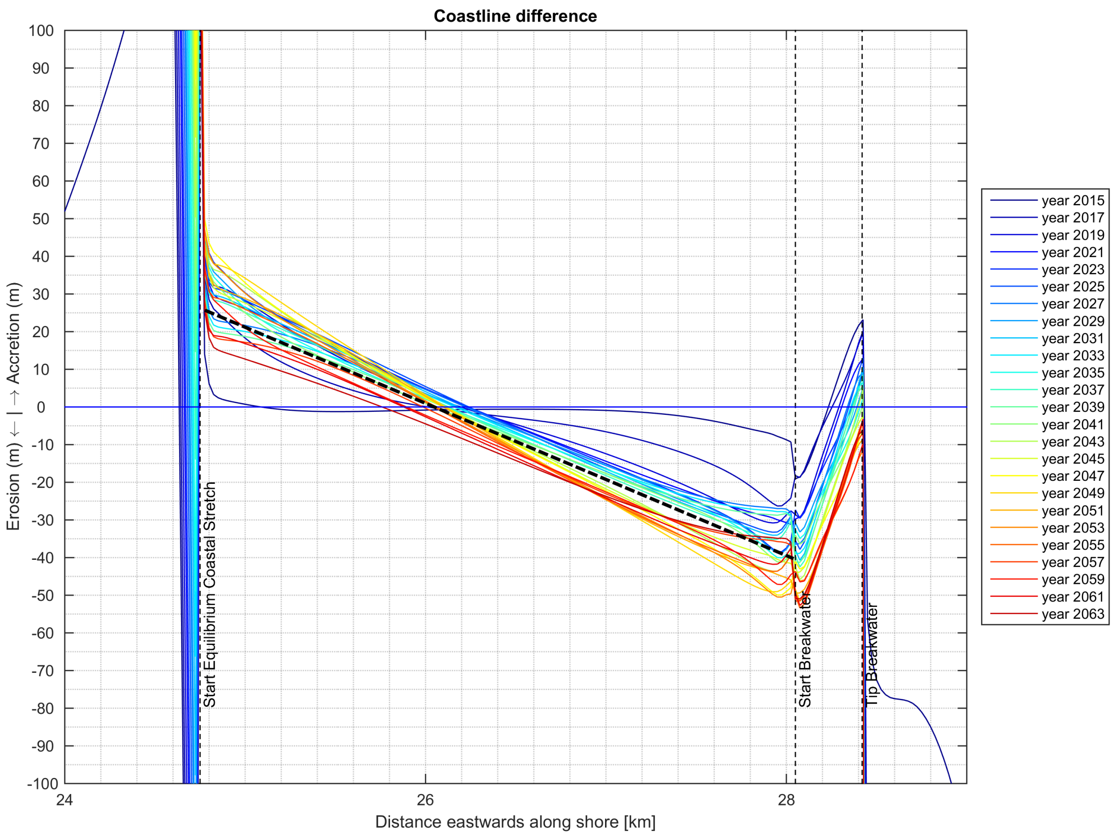
Figure 5.9: Overview Net LST capacities along coastline for a modelled period of 50 years for variant 'Groyne'

In Figure 5.10(a) the coastline development along the coast is shown for variant 'Groyne'. On the west side of the start of the equilibrium coastal stretch, much accretion is shown. This accretion is coherent with Figure 5.8. To the right of this start of the equilibrium coastline stretch to the start of the breakwater you see little coastline development. In Figure 5.10(b) the stretch of coast of the pocket beach is enlarged.

Although little coastline development is present, a small difference in coastline equilibrium orientation appears to be engendered. The equilibrium orientation of 278.6 ° as was defined before is slightly different. The black dashed line represents the new equilibrium orientation which is directed 277.5 ° with respect to TN. After two years this equilibrium orientation is present and throughout the years it varies around this orientation. This variation concerns a bandwidth of only 25 meters on both sides of the pocket beach. To be on the conservative side a margin of 30 meters of the complete stretch of sand breakwater is taken into account. This margin is implemented in the final variant design such that it is able to withstand this coastal development, see Subsection 7.2.2.



(a) Overview Coastal development for a modelled period of 50 years for variant 'Groyne'



(b) Coastal development zoomed in for equilibrium coastal stretch with new equilibrium orientation

5.3.3 Variant 3: Lagoon

Lastly, a third variant arose out of the previous variants. The groyne on the western part of the pocket beach actually blocks all the sand while there is much space behind the pocket beach for accretion. In addition, there is no benefit of the accretion of sand on the west side so far. In this variant extra space for accretion is provided by a rather different connection to the original coastline. This connection is designed to be constructed behind the pocket beach and is made from sand. This connection to the original coastline is certainly required as it protects the port from siltation due to secondary currents and the attack of lower diffracted waves. By using sand instead of rocks, the accreted area around the connection can be used in the future for port expansion. This gives added value to the design.

Variant 'Lagoon' was invented via replacing the groyne from variant 'Groyne' by a lower sand dune with which more space for accretion is provided and can be used to the benefit.

The pocket beach does need to be protected on the west side and therefore, contains a small groyne which protects the coastal stretch from eroding. It is proven in previous Subsection 7.2.2 that given the angle of equilibrium orientation almost no LST will take place along the pocket beach. The same tip of the sand breakwater from the last variant is implemented in this design, see Figure 5.10.

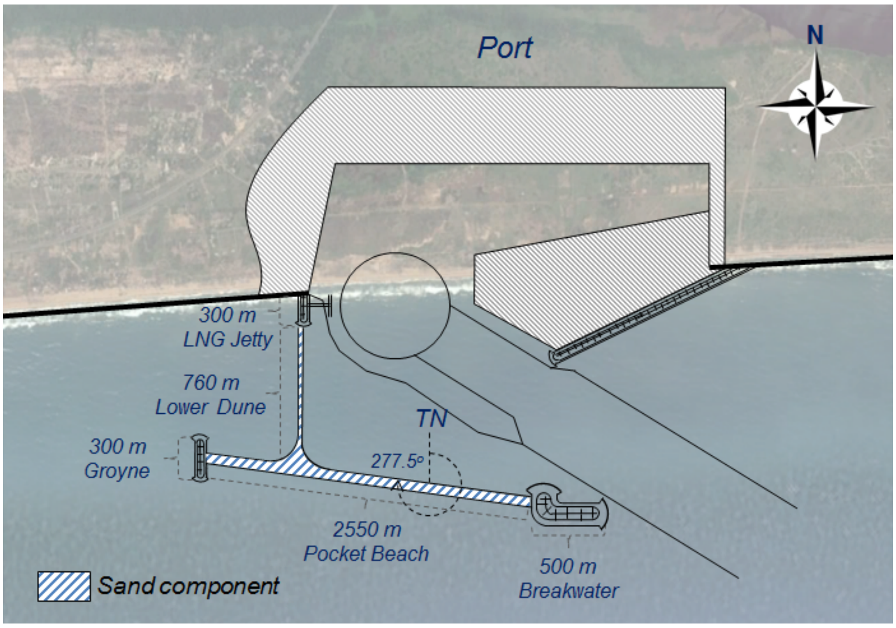


Figure 5.10: Schematisation of variant 'Lagoon', not to scale

On the back of the coastal stretch a small lower dune protects the port and coast. The dune connects the pocket beach to a small breakwater on which a LNG jetty is designed. This LNG jetty is present in all three variants and is assumed to be a boundary condition. This jetty might be able to construct in a different way but this is outside the scope of this study. The 300 meter long breakwater is assumed to be sufficient for the LNG jetty to be constructed on and continues in the lower dune southwards.

Due to the large storage capacity of sediment the variant is called "Lagoon". Next to this advantage, a second benefit exists. In the variant 'Groyne' the length of the groyne was minimised so it would provide just enough space for the accretion throughout the 50 years as determined by the boundary condition. Since, in this variant the minimal length of the groyne is not dictating the length of the pocket beach and therefore this stretch of coast can be reduced.

This variant is not possible to model in LITPACK due to two positions of coastlines. Instead of LITPACK a different approach is followed to optimise the variant's design. The factor which is dictating the required length is the shelter capacity of the port in all swell events. The complete port needs to be protected from all swell events. That means that every wave under an angle of 177.50° to 202.50° degrees need to be captured by the sand breakwater. If this bandwidth is protected 97.50% of the nearshore swell events is captured. This percentage is extracted from the data series of swell events of 2008-2013. In Figure 5.11 the wave rose is presented for nearshore swell waves in which the outer bandwidth directions are designated as well.

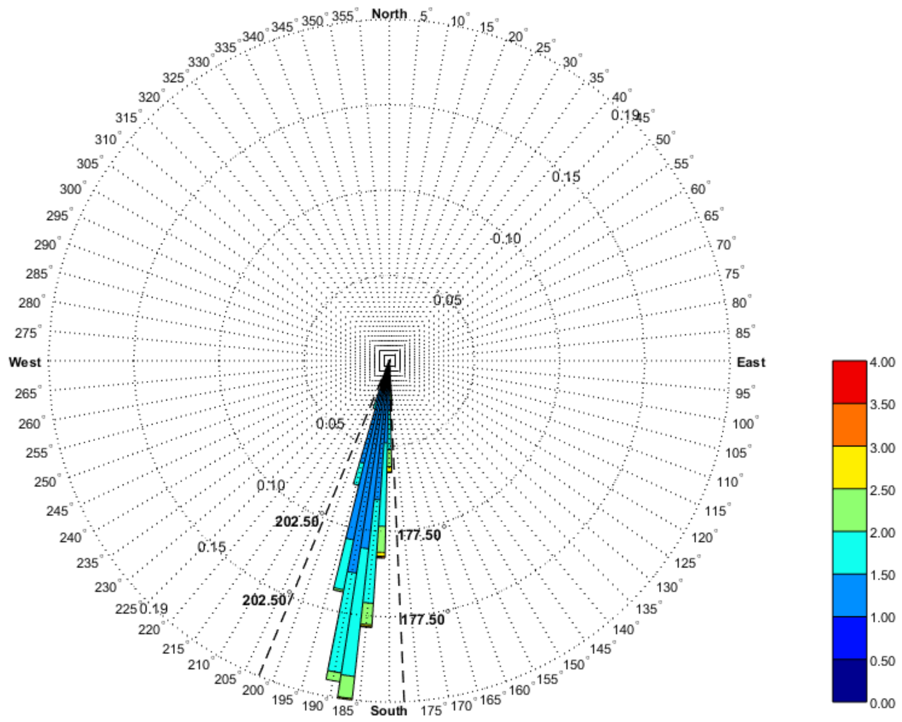


Figure 5.11: Wave rose for nearshore swell for period 2008 - 2013

This bandwidth of events determines the length of the required coastal stretch. Next to this bandwidth the phenomenon diffraction is also of influence on the length of the stretch. The phenomenon diffraction is the changing of waves from direction towards areas containing lower amplitudes due to amplitude changes along the wave crest (Holthuijsen, 2007). Diffraction is particularly strong along the geometric shadow line of obstacles such as islands, headlands and breakwaters. The phenomenon of diffraction is displayed in Figure 5.12.

For the breakwater case shown in Figure 5.13 the 'Sommerfeld solution' has been used in literature to determine wave heights in the shadowzone behind the breakwater. Sommerfeld has determined this solution for a very specific scenario regarding a uni-directional wave climate which therefore is well applicable in the scenario of Badagry.

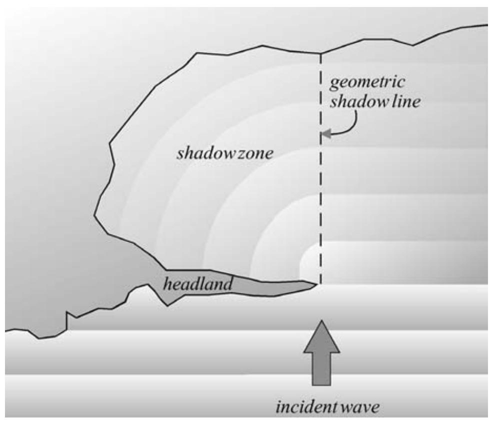


Figure 5.12: Diffraction around a headland with a circular wave pattern in the shadow zone (constant depth and no reflections) (Holthuijsen, 2007)

The scenario which Sommerfeld used, is quite common. In this study however, the sand breakwater is first of all not positioned perpendicular to the incoming waves. Secondly, the maximum wave direction is not originating in the south. In this study, as stated above, a maximum direction of 202.5° with respect to TN is used for the occurrence of swell events. As seen in the right part of Figure 5.13 the rate of wave height decrease is decreasing rapidly after $0.2 = H/H_{inc}$. For $0.1 = H/H_{inc}$ relatively much stretch of coast would be required to construct to suffice the amount of shadowzone. It is assumed that the graph of Sommerfeld still is applicable under an angle. Assumed is that a $0.2 = H/H_{inc}$ is desirable. This is the lowest rate which is practically feasible. With the 1/100 design storm conditions of a wave height of 3.95 meter and a period of 19.3 seconds, a wave length of 212.5 meter on a depth of 13 m - MSL is present.

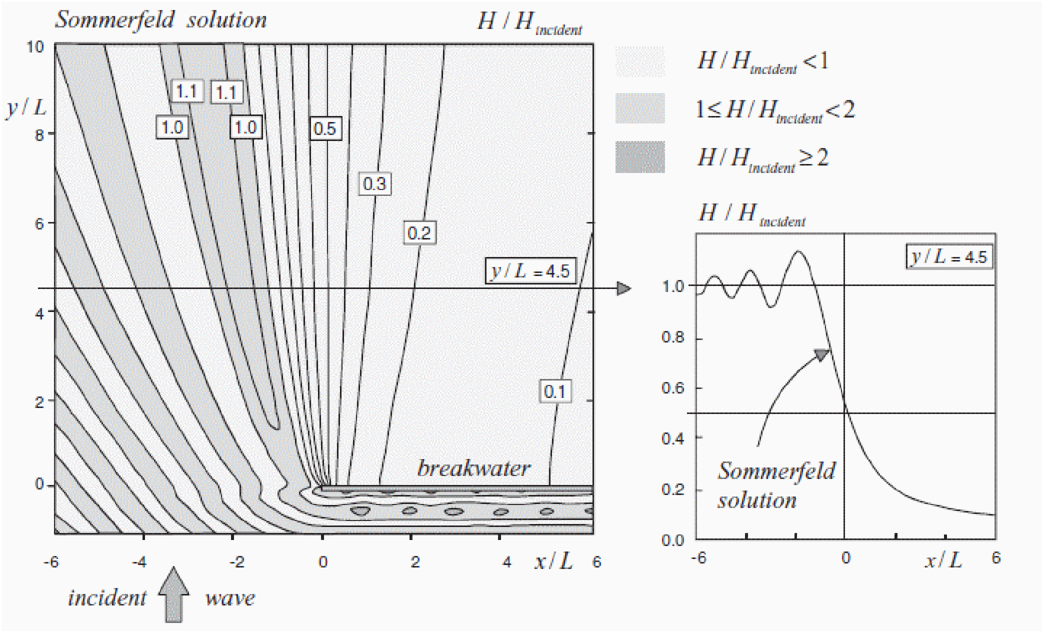


Figure 5.13: Diffraction (normalised wave height) of a normal incident, harmonic wave around a semi-infinite, straight breakwater inwater with constant depth (Sommerfeld solution; "L" being the wave length and "H" being the wave height)(Holthuijsen, 2007)

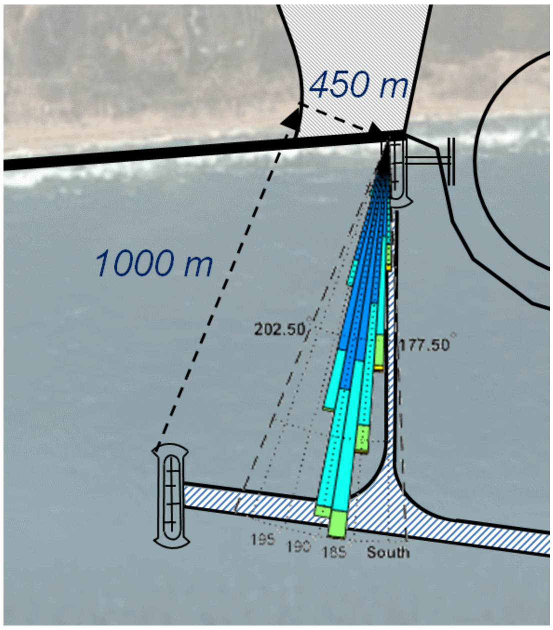


Figure 5.14: Schematisation of required length for pocket beach due to diffraction criteria

In the direction of the maximum swell event of 202.5° with respect to TN a length of about a 1000 meters is present between the port and lower dune. Seen Figure 5.13 1000/L = 4.7. To accomplish the H/H_{inc} = 0.2 a certain width of shadowzone behind the breakwater is required. This width is displayed in Figure 5.13 as x/L. Having said that, for H/H_{inc} = 0.2 being true a horizontal width of 425 meter is required to create the necessary amount of shadowzone. To be on the safe side a width of 450 meter is applied in the design, see Figure 5.14. Following from these boundary conditions a required length of 2550 meter is determined for the pocket beach.

Although it is assumed that the case of Sommerfeld is applicable, other phenomena could influence the principle. These phenomena concern the wave set-up on the coast nearby causing secondary currents and refraction of the waves due to decreasing depth behind the pocket beach. The refraction would cause the wave height to decrease further towards the lower dune hence the required dune height for the lower dune, which is presented in Subsection 6.2.1.2, is conservative. In Chapter 6 Subsection 6.2.1.2 the height of the lower dune is computed following from the wave height occurring in the shadowzone of the breakwater. It must be stated that the H/H_{inc}=0.2 is used as boundary condition for the lower dune in the most northern position, closest to the port. Moving further away from the port deeper into the shadow zone the present wave height is actually lower than 0.2 H_{inc}.

Conclusively is stated that the goal behind this variant is achieved. First of all a reduction in length of 250 meter for the pocket beach but, more important, extra space is present for sand accretion behind the pocket beach. Thirdly, the accreted area can be put to use in the future, since, the lower dune is constructed out of sand. LST which occurs from the west of the pocket beach will be trapped in the 'lagoon'. Although zero transport capacity for sediment is present behind the pocket beach in the 'lagoon', still sediment transport will take place. Accretion of sand will still occur due to currents caused by differences in wave set-up. In chapter 7 the economic surplus of this extra space is determined.

Equal to the previous variant, an extra design width due to the small coastal development along the pocket beach is required. This width is 30 meters over the complete length of the pocket beach. This figure is further integrated in design in Subsection 7.2.3.

5.4 Large scale Impact on coastline

The impact of the variants on the coastline on large scale is examined for the modelled period of 50 years. The impact is split up into two parts:

- The coastline west of the port
- The coastline east of the port

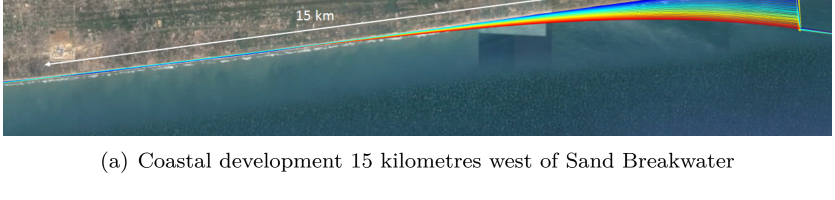
5.4.1 West

The effect of the variants 'Sand Nourishment' and 'Groyne' are similar as the same space for accretion is available. Even though variant 'Lagoon' is not modelled, it is assumed that the effect on the coast west of the sand breakwater will be the same, however, it will take longer for the effect to arise. This assumption is made because it takes longer for the same effect to occur on a coastline. with a larger space available for accretion.

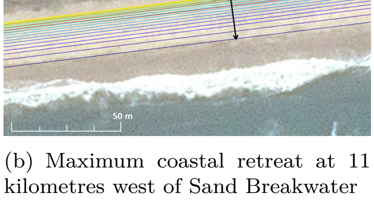
The impact of the variants on the coastline to the west from the project is small. In the first 38 modelled years the coastline towards the west shows to be in equilibrium. The only erosion which appears to take place occurs 15 kilometres upstream from the sand breakwater and concerns a maximum of 20 meter of coastal regression, see Figures 5.15(a) and 5.15(b). Beyond 15 kilometres from the Sand Breakwater solemnly accretion will occur. This coastline regression decreases after 38 years due to the increasing accretion at the sand breakwater. The original coastline starts to develop towards the new coastal stretch under equilibrium orientation due to accretion at the sand breakwater. Consequently, the effect of accretion will start to propagate away from the Sand Breakwater towards the west. This movement causes the coastline west of the Sand Breakwater to accrete, see Figure 5.16. The yellow arrow visualise the accretion at the sand breakwater which cause the movement in accretion along the coastline towards the west indicated as a white arrow.

The maximum coastline regression of 20 meter is assumed to be within acceptable ranges. The actual accretion of the coast is assumed not to be a problem as land will only be gained instead of lost to SLR which is more common along the Nigerian coast. Having said that, the answer is achieved for the sub-question.

The impact of the Sand Breakwater on the coastline west of the port is within acceptable boundaries and concerns a maximum 20 meter of coastline regression.



(a) Coastal development 15 kilometres west of Sand Breakwater



(b) Maximum coastal retreat at 11 kilometres west of Sand Breakwater

Figure 5.15: Coastal development west of Sand Breakwater

Figure 5.16: Increasing area of accretion west of Sand Breakwater

5.4.2 East

Since the complete littoral transport is blocked after construction, a demand for sediment occurs on the east which leads to an erosion pocket east of the port. Hence, coastal mitigation measures are required. The impact on the coast east of the port is similar for the sand breakwater and conventional breakwater design. Consequently the impact of the construction of a sand breakwater on the coastal development to the east is excluded in the scope of this research.

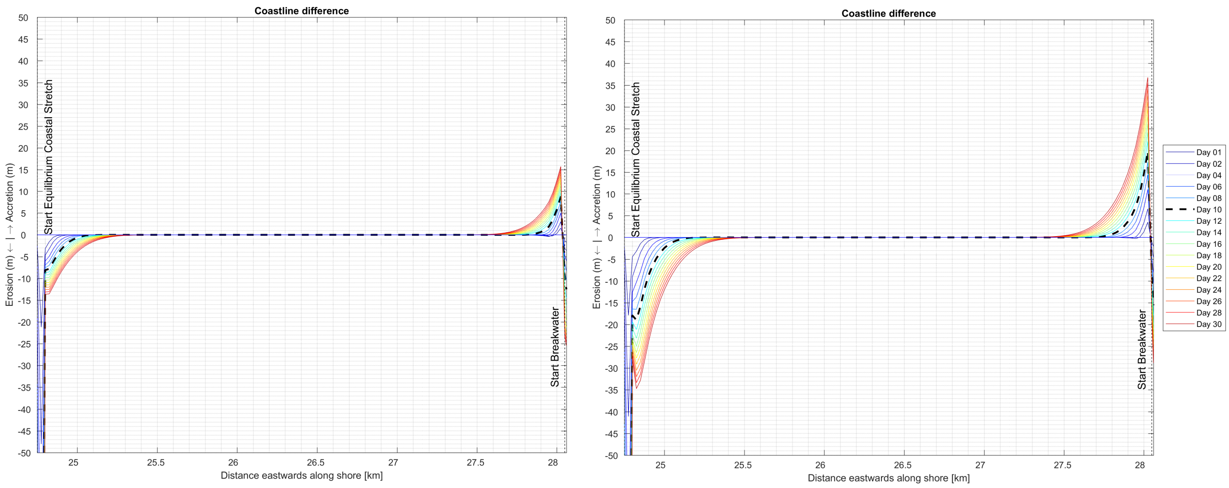
5.5 Sensitivity analysis

As stated before, the modelling software used in this research is only approaching reality. It is not yet developed such that it can compute exactly what it is used for nowadays. For this reason it is inevitable to research the sensitivities of certain parameters in the model. This provides a better understanding of the results of the model. First of all the sensitivity of the wave angle will be discussed. After which the grain size is examined and finally the "Spreading factor" is discussed. The sensitivity to the "critical Shields parameter" is also determined but was found of negligible influence, moreover in Appendix H.

5.5.1 Wave angle

So far long term modelling has been executed and only yearly averaged results have been looked at. As the wave climate is rather constant and uni-directional the band width of the wave climate is small. Hence, no large bruto coastline development is expected. Although the net coastline development is thought to be in the same order as the bruto coastline development in one year, it is the bruto coastline development to which the Sand Breakwater is required to be resistant. The netto coastline development neglects the scales of events like groups of swell waves coming from certain directions for short amount of times relatively to the previously modelled periods of years. Consequently, it is important to model a shorter period to determine the sensitivity for waves from different directions.

The wave climate which is modelled for this sensitivity analysis is the average swell wave climate nearshore. A significant wave height of 1.42 meter and a peak wave period of 12.68 seconds has been applied. With these characteristics coastal development was modelled for a period of a month for waves from four different directions. Two on each of the outer edges of the swell event bandwidth (177.5° and 202.5°) and two bins located five degrees off the average wave direction (184° and 194°). In Figure 5.17(a) the coastal development of the pocket beach is displayed for a period of a month with waves originating from 194° with respect to TN. The coastline displacement amounts to 15 meter erosion and accretion respectively west and east gradually over a length of half a kilometre. This coastal evolution in comparison to the modelled evolution for waves originating from 202.5° is yet small, see Figure 5.17(b). The coastline displacement for this direction amounts to about 35 meter after a month decreasing over a length of 750 meter on both sides along the pocket beach.



(a) Coastal development wave angle 194.0 degrees

(b) Coastal development wave angle 202.5 degrees

Figure 5.17: Coastal development for multiple wave angles

The persistency of waves at Badagry is determined in order to establish a required margin for the sensitivity of the direction of waves. This computation is conducted on a dataset of the waveclimate from 1997 to 2013. The persistency for waves is determined in five bins with each five degrees of directional width, see Table 5.1. Two on each of the outer edges of the swell event bandwidth and two bins located five degrees off the average wave direction. The maximum duration of waves from within the directional bin on the outer edge (175° to 180°) is determined as six days. This concerns a period of six days in the period of 1997 to 2013. This period is not likely to occur often in the lifetime. To establish a conservative period it is assumed that a period of **ten days** will be used for the design margin for wave angle sensitivity.

Table 5.1: Persistency wave direction over five bins

	Directional bins				
	177.5°	184.0°	189.0°	194.0°	202.5°
Maximum duration [hours]	144	174	222	195	78
Maximum duration [days]	6.00	7.25	9.25	8.13	3.25

Taken this duration of 10 days into account for waves approaching from the outer edges of the swell events bandwidth 202.5° and 177.5°, a coastal development of 20 meter occurs both accretion and erosion depending on the direction. Hence, a margin for the berm width of the sand breakwater of **20 meter** will be taken into account and further integrated in Chapter 7 for conceptual variants 'Groyne' and 'Lagoon'. These two variants contain a pocket beach which is thinly designed and needs a margin for this parameter. Variant 'Sand Nourishment' is assumed not to need this margin since it already has such large margins for coastal development.

5.5.2 Grain size

For determining the sensitivity of the grain size it is chosen to compare the results of grain sizes varying from 200 µm to 400 µm. The default runs are executed with the grain sizes which are measured along the actual profiles. On average this comes down to a little bit over **300 µm**. In Figures 5.18 and 5.19 the LST capacity along the coast of the area of interest is shown for two different years.

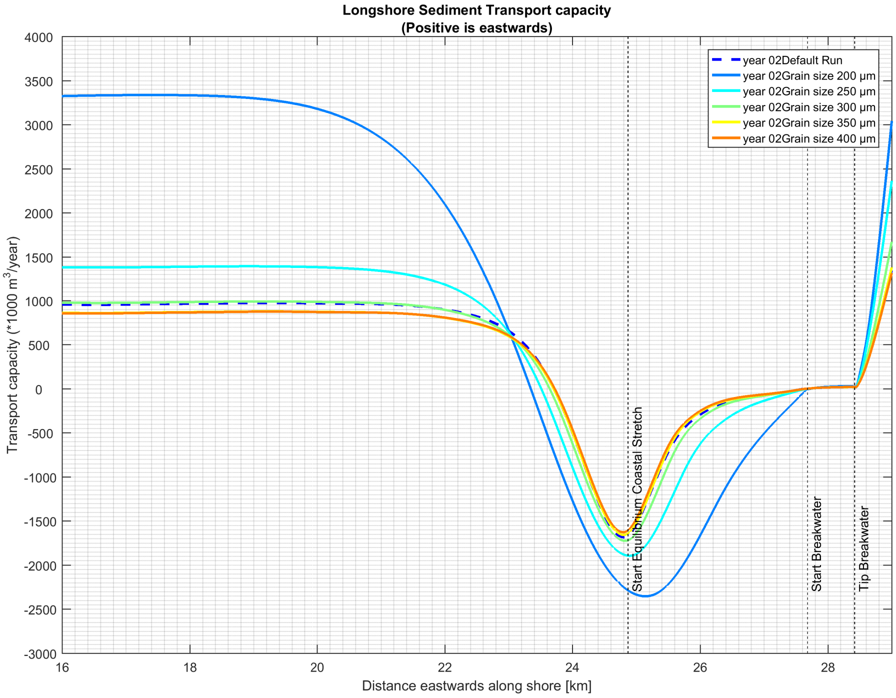


Figure 5.18: LST Capacity along the coast of the area of interest for different grain sizes in year 2

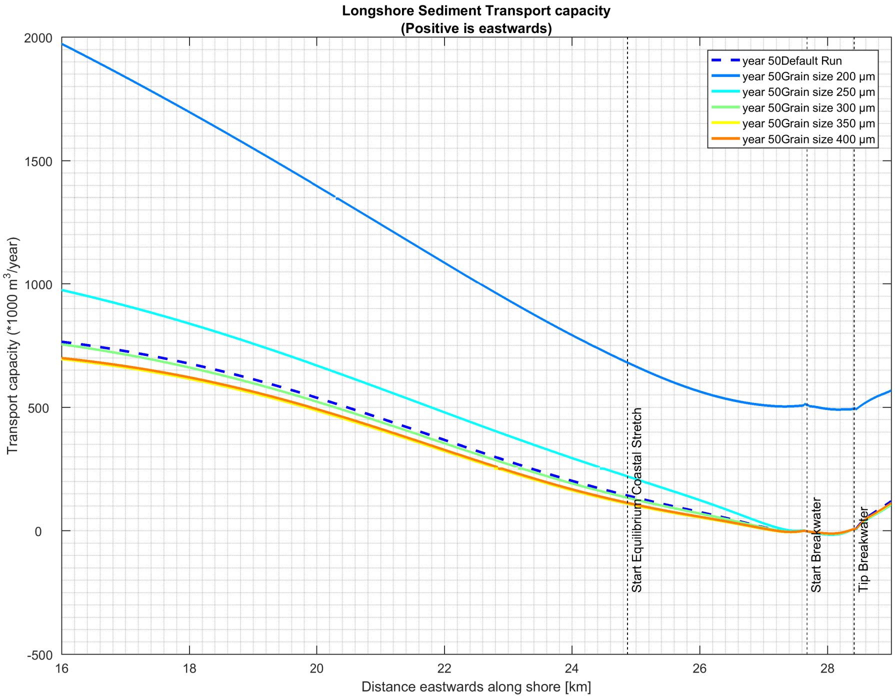


Figure 5.19: LST Capacity along the coast of the area of interest for different grain sizes in year 50

The modelled LST is significantly higher for a lower grain size than for a higher grain size which is the expected outcome. The LST as its known in front of the Nigerian coast at Badagry is about 0.8 - 1 million cubic meters per year as stated in Chapter 3. This number is coherent with the grain sizes of 300 μm as shown in Figure 5.18.

Larger grain size First of all the sensitivity for a grain size higher than the default grain size is estimated. For a grain size of 400 µm compared to the default amounts to about 50.000 $m^3/year$, see Figure 5.19. In amount of coastal development this is close to 10 to 20 meters over a period of 50 years and assumed **negligible**, see Figure 5.20.

Smaller grain size Secondly, the sensitivity for grain sizes lower than the default grain size is determined. It is known that the swash zone contains overall sediment with a smaller grain size than the rest of the profile. The impact of smaller grain sizes on both modelled accretion and erosion in the variants is discussed.

The Figures 5.18 and 5.19 do show that with a smaller D_{50} there would be significantly more erosion along the coast and with that along the Sand Breakwater. However the Sand Breakwater will not be constructed with sand which is available in the swash zone. The sand breakwater is assumed to be constructed with the dredged sand from the basin of the port and the approach channel. This is significantly larger sediment than sediment from the swash zone. From the grabsamples which are acquired, the sediment on the beach concerns an average grain size of 650 µm. The approach channel has varying sizes however below the layer of sand with small grain sizes larger grain sizes seem to exist. Therefore it is safe to say that this construction will never be constructed with sand of 200 µm or 250 µm and for that reason it is a very conservative design and the erosion which is taken into account for a grain size of 300 µm is sufficient. If more detailed information about the subsoil is known it could be interesting to try to create a design with a higher grain size. Conclusively, can be said that for lower grain sizes the modelled erosion is **conservative**.

It is different when modelled accretion is tried to examine on sensitivity. The swash zone is the zone in which the most littoral transport takes place. With a smaller grain size the accretion which takes place with the different variants for design could be higher than computed now. In that case, as is shown in Figure 5.19, for a grain size of 200 µm the LST will not be zero at the tip of the breakwater and significantly more siltation in the approach channel will take place. For a grain size of 250 µm the LST capacity will still be zero at the tip of the breakwater after 50 years. In Figure 5.20 the accretion is visibly still smaller than the tip of the breakwater for grain size of 250 µm. For a grain size of 200 µm the accretion will have past the tip of the breakwater. These graphs are determined for variants 'Sand Nourishment' and 'Groyne' which both rely on a storage space for accretion of 50 years. Variant 'Lagoon' evidently has more storage space. So when smaller grain sizes would be present, the accretion will pass the tip of the breakwater later than in the first two variants.

Having said this, it must be stated that the model LITPACK is calibrated using the value of 300 µm. The known LST from literature amounts in the order of 800.000 - 1 million cubic meters per year (Tilmans, 1993). It is not possible to say that an LST of 2 million cubic meters per year would occur when smaller grain size would be implemented as the model needs to be recalibrated. The sensitivity is difficult to define as the LITPACK model also was calibrated using the default value. Shifting the grain size would mean recalibrating. Although the relation is certainly not linear a smaller grain size is examined to its impact on LST.

- A lower grain size (250 µm) leads to an increase in LST (+450.000 m^3/year which is about 50%)
- A larger LST leads to a shorter period without the by-pass of sand around the tip the sand breakwater (Variant 'Sand Nourishment' and 'Groyne' approximately 34 years, 'Lagoon' is not known

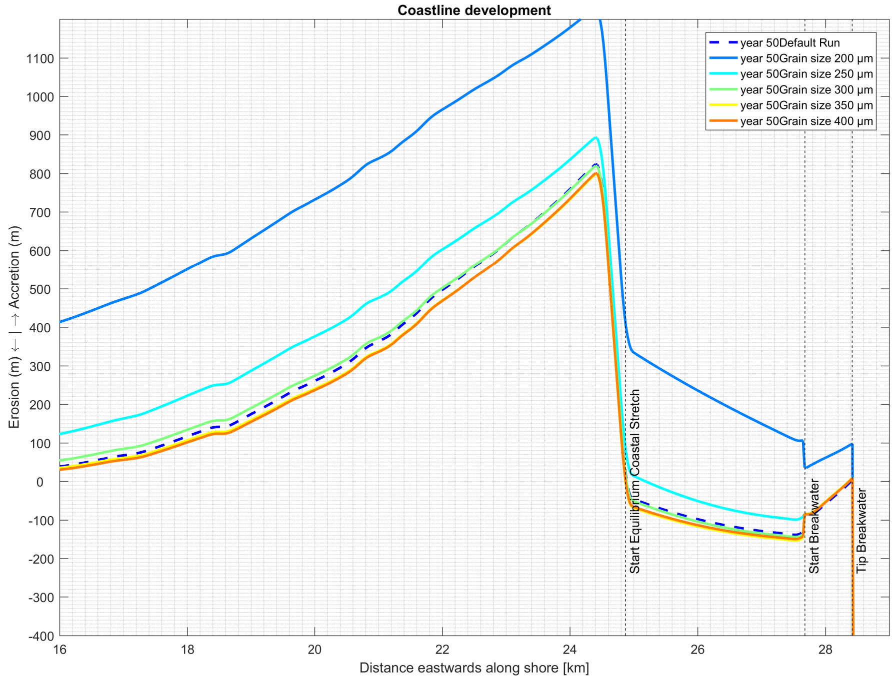
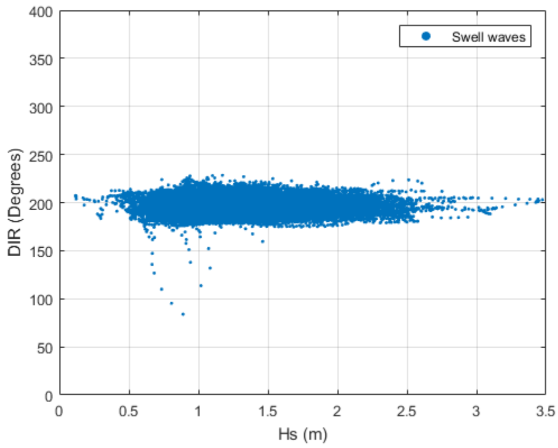


Figure 5.20: Coastal development along stretch of coast of the area of interest for different grain sizes after 50 years

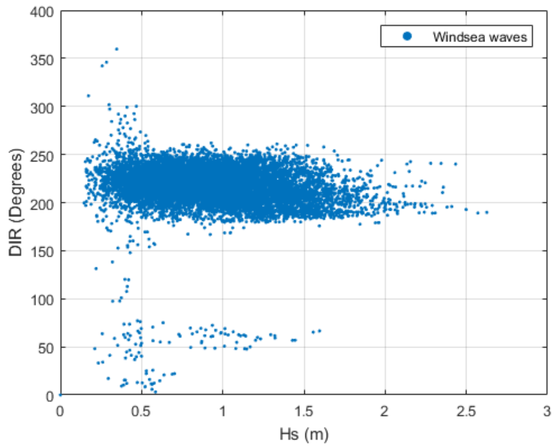
5.5.3 Spreading factor

It is relevant to include the effect of the directional spreading of the waves in the simulation of wave set-up and littoral current. Directional spreading of the wave climate is taken into account by LITPACK through the so called "Spreading factor" or reduction factor. The spreading factor represents the decrease in radiation stresses due to the directional spreading of the approaching waves. A maximum value of 0.8 represents a unidirectional sea, 0.35 represents a complex directional sea while 0.5 is a typical value.

The effect of this parameter is rather opaque. The manual of LITPACK states that a reduction of e.g. 50% in the driving shear radiation stress will give a reduction in the littoral current velocity of about 30% (DHI-GROUP, 2017b). To find the effect of the Spreading factor on the LST multiple model runs have been executed. The wave climate is as we know highly swell dominated but also very uni-directional. This is supported by the Figures 5.21(a) and 5.21(b) in which clearly is visible that the sea state for both types of waves is clearly unidirection. Also taken into account the fact that the events of windsea waves are a lot less than the swell events. Because of the uni-directional state of the sea, the spreading factor is varied from 0.6 to 0.8 to see what the relation is to change in LST and if it has a big impact.



(a) Joint distribution of wave height and direction of swell waves



(b) Joint distribution of wave height and direction of windsea waves

Figure 5.21: Joint distribution of wave height and direction of different sea states waves

In Table 5.2 the results of these runs are shown. The impact of the spreading factor is of significant impact. For example for wave time series 60.54, which is nearby the location of Badagry, the 0.7 spreading factor is 124% compared to 0.6. The 0.8 value is 148% compared to 0.6. This is of significant impact as it concerns a 50% increase of LST. This obviously supports the statement that modelling in sediment transport is still very rough as the choice in spreading factor is in the end subjective and rough. The sea state is highly uni-directional and therefore, the value of 0.7 is chosen for the spreading factor during the modelling with LITPACK.

Table 5.2: Yearly averaged net longshore sediment transport under different spreading/reduction factors along multiple wave time series

Wave time series / Reduction factor	75.67	60.54	45.41	30.27	15.41
0.6 (Averaged LST/year)	622,700	614,900	799,400	1,123,100	1,127,800
0.7 (Averaged LST/year)	769,000	761,200	990,400	1,397,800	1,401,200
0.8 (Averaged LST/year)	920,800	911,100	1,187,800	1,687,500	1,688,200

5.5.4 Conclusions

Multiple parameters examined on sensitivity in the model show that the LST and with that the coastal development can vary significantly when these parameters are shifted.

First of all, with a parameter like the Shields parameter (Appendix H) a clear conclusion can be drawn. The Shields parameter is not of significant influence. As stated previously, the largest difference in coastal development after 50 years is 10 meter for the minimum Shields value of 0.030 and the default value of 0.045. Assumed is that this impact can be neglected as a lot of margins are already taken into account and the value of 0.045 is an advised default value supported by literature (Schiereck and Verhagen, 2012). The reason why the impact of different Shields parameters is low is most likely because the velocities occurring at the bed are possibly much higher than the critical friction critical velocity.

Secondly, the wave angle was researched and shows a significant impact. Since the wave climate is rather uni-directional, the net impact of the wave angle averaged out over a year is only small. The bruto impact of short periods, however, is significant. The maximum duration of waves from within the directional bin on the outer edge (175° to 180°) is determined as six days. This concerns a period of six days in the period of 1997 to 2013. This period is not likely to occur often in the lifetime. To conclude, a duration of 10 days is taken into account for waves approaching from the outer edges of the swell events, coastal development of 20 meter occurs. Due to this, a margin of 20 meters for the berm width of the sand breakwater is taken into account and further integrated in Chapter 7.

Thirdly, the grain size was adjusted in order to establish sensitivity of the results. A larger grain size than the used default value of 300 µm, shows little difference in result. When the grain size is smaller than the assumed grain size present at Badagry, it would lead to a significantly higher LST and coastal development. The modelled erosion is conservatively large for the default value of 300 µm, taken into mind that the construction will be constructed with a relatively large grain size. When considering the modelled accretion a lower grain size can have a significant impact.

- A lower grain size (250 µm) leads to an increase in LST (+450.000 m^3/year which is about 50%)
- A larger LST leads to a shorter period without the by-pass of sand around the tip the sand breakwater (Variant 'Sand Nourishment' and 'Groyne' approximately 34 years, 'Lagoon' is not known

Lastly, the spreading coefficient is a parameter with a large impact on the LST and in turn on the coastal development. The deviation of the factor from 0.7 to either 0.6 or 0.8 amounts to 20% of the the total LST.

- A higher spreading factor (0.8) leads to an average increase in LST (+20%)
- A larger LST leads to a shorter period without the by-pass of sand around the tip the sand breakwater (Variant 'Sand Nourishment' and 'Groyne' approximately 42 years, 'Lagoon' is not known

The factor is difficult to estimate as it is highly empirical to estimate the complexity of the state of the sea to an exact number. Taken the uni-directional character of the sea into account, assumed is that the 0.7 is representable for the sea state throughout the year for Badagry.

Both grain size and spreading factor are parameters which are included while calibrating. That means that the calibration is also based on their input. If other grain sizes or spreading factors would be used the model needs to be recalibrated. To conclude, no margin with respect to its sensitivity is taken into account.

6

Analysis of cross-shore development

6.1 Introduction

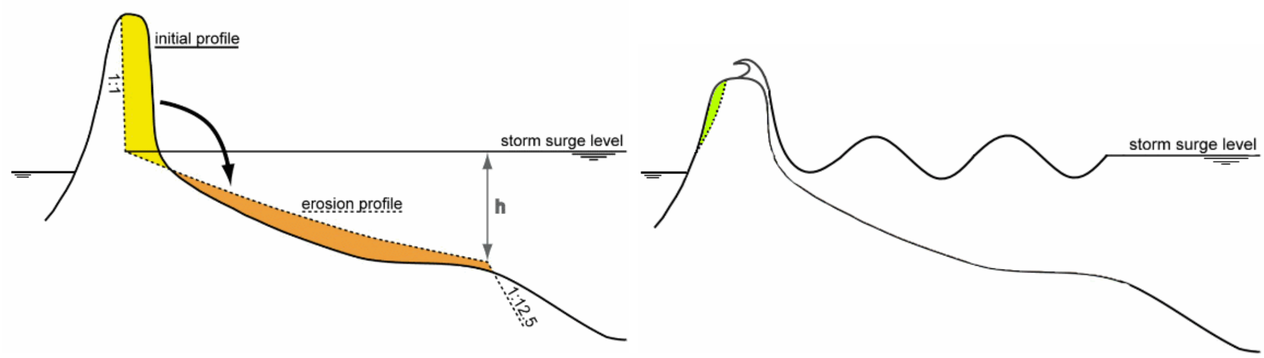
The previous chapter focussed on finding long term morphologically stable, conceptual designs by focussing on alongshore coastline development. This chapter, however, is mainly driven by the cross-shore transport analysis. The sand breakwater needs to be designed in a way that it on the one hand is sufficient for certain design storm conditions. In addition, it is desired to construct the sand breakwater with a minimal volume sand volume as possible for the sand breakwater being an economical attractive solution. An optimisation of these aspects leads to a sufficient cost-effective cross-shore profile.

This main objective in this chapter is to define cross-shore profiles for the sand breakwater which are sufficient for its design storm conditions.

The cross-shore profile of the sand breakwater is divided roughly in two parts: a submerged part and an emerged part. Having said that, an assumption is:

The submerged part of the profile up to the top of the intertidal profile is integrated in the design from acquired profile data from Badagry.

As this specific profile is stable under the harsh wave climate throughout the year, this assumption seems valid. Other profiles will transform to this profile due the hydraulic forcing from the wave climate. The emerged part of the profile concerns the cross-section of a dune and is the critical part for the design objective.



(a) Schematisation of avalanching after (Bosboom and Stive, 2012)

(b) Schematisation of overtopping of waves and erosion on top and backside

Figure 6.1: Different failure mechanisms

To live up to its criteria, a dune needs to be well-designed and requires a certain minimal cross-section. If this cross-section is not present the function of the dune and with that the sand breakwater is not possible to fulfil. As a dune is practically unprotected by any coverage, except for maybe some dune grass, the profile is vulnerable to certain failure mechanisms. In the case of the sand breakwater, it is assumed that the leading failure mechanisms are similar to the ones of a dune: avalanching and the overtopping of waves which can lead to overwash during a storm, see Figure 6.1. If the dune is too low, much overtopping will take place. Consequently, the dune erodes on the top and backside due to overwash which can lead to failure of the dune. If the dune is constructed too small in width the avalanching process might erode the sand breakwater in a storm up to a critical level.

The cross-shore profiles are found by modelling the transport of sand caused by storm conditions at the coast of Badagry for certain profiles and orientations for the sand breakwater. Via this approach, **phase 1b** of the study is conducted, see Subsection 2.3.

In Section 6.2 regulations and guidelines for the design of dunes are examined and a required crest height for the dune is established. Taking this crest height, an optimal dune width is found for storm conditions by using the modelling software XBeach. In Section 6.3 the modelling software XBeach is discussed along with its set-up. The results and sensitivity analysis of the model are presented along with the final required profiles which are required for different parts of the variants.

6.2 Determination of minimal required profile

6.2.1 Alternative approach

In general, for dune design at the Dutch coast certain regulations are applicable and standard for dune design (T. Vijverberg, 2014). In Appendix I this approach is elaborated on. This approach is part of a design method developed for storms occurring at the Dutch coast. In addition, it is created for a dune with a hinterland which is required to be protected. This coast along with its storm occurrence differs heavily to the Nigerian coast and its forcing. The long wave period of the swell waves at the Nigerian coast in combination with different characteristics for the type of beach (larger grain size and steeper slopes) lead to uncertainties using the approach stated above. Concluded is that an alternative approach is used for determining a design crest height for dunes.

The approach used in this research approach is based on a study done by Tilmans (1983). This study is conducted in order to establish a regulation for the minimum height of the dune and the width and height of a required profile. In this study overwash and overtopping volumes of waves are measured during physical modelling.

The common approach of defining wave run-up for dikes is, the wave run-up height above SWL, $z_{2\%}$, which is exceeded by 2% of the waves. Taken into account that dunes are much steeper and do not have a slope as fixed as a dike but deform, run-up of individual waves is less present. Large volumes of water overtopping and crashing down the dune front is more common. Tilmans (1983) states that it is therefore unrealistic to define wave overtopping based on an amount of individual waves running up the dune. A phenomenon called "Ruitereffect" embodies the grouping of waves which cause a big volume of water to attack the dunefront instead of separate wave run-up. Taken into account the uncertainties in the definition of a run-up height, based on the amount of exceedances, a run-up height based on a period of time of the exceedances is preferred. With this, physical relation between wave run-up and wave overwash is taken into account in a more accurate way (Tilmans, 1983). Therefore, the following design criteria from the study is handled:

During the peak of the storm 2% of the time overtopping may take place.

Conclusively it is stated by Tilmans (1983) that the scenario above led to an advised dune height above SWL of 4.0 meter. With this advised dune height, an advised overtopping volume of 18 L/s per running meter averaged over the time of the storm is measured. Averaged over the period of overtopping this concerns a volume of 0.9 m^3/s per running meter of overtopping water. This regulation is close to the dune height found in the previous section with the approach of defining an URP (4.0 meter and 4.6 meter). However this approach is also based on modelling different storm conditions than in Nigeria.

Instead of using the advised dune height of 4.0 meters above SWL, the coherent overtopping volume is handled. Tilmans (1983) does not state that 18 L/s per running meter is a regulation or safe volume for an unprotected inner slope. However, the study states that the wave overtopping volume coherent for a crest height where during the peak of the storm 2% of the time overtopping takes place amounts to 18 L/s per running meter. Due to this fact, is assumed that in this research also its coherent overtopping volume is acceptable.

As stated in Subsection 3.3.7 for the lifetime of the port is assumed that the design storm condition concerns a storm with a 1/100 return period. This extreme event was set for the conventional breakwater and is also the starting point for the sand breakwater. However, that does not mean that it cannot take certain damage in this storm. The amount of damage which is acceptable for the sand breakwater to undergo during the storm is determined to be the damage caused by the overtopping volume of 18 L/s per running meter.

6.2.1.1 Determination main crest height

This overtopping volume creates the opportunity to define the crest height for the case of the Sand Breakwater by reversely using the general overtopping formula. This formula, see Equation 6.1(J.W. van der Meer, 2016), is used for computing overtopping volumes for dikes and embankments. Assumed is that is also able to apply for overtopping with dunes.

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = \frac{0.023}{\sqrt{tan\alpha}} \cdot \gamma_b \cdot \xi_{m-1,0} \cdot exp[-(2.7\frac{R_c}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v})^{1.3}] \tag{6.1}$$

In which:

q	= Overtopping volume [m^3/s per m]
H_{m0}	= Significant wave height [m]
α	= angle of slope [°]
R_c	= freeboard of dune w.r.t. Still water level (SWL) [m]
$\xi_{m-1,0}$	= Breaker parameter [-]
γ_b	= Factor for present berm, no berm = 1.0 [-]
γ_f	= Factor for roughness = 1.0 [-]
γ_β	= Factor for wave angle direction = 1.0[-]
γ_v	= Factor for vertical wall, no wall = 1.0 [-]

The slope is determined to be tanα = 1:8 in the design of which the sensitivity is elaborated on in Appendix J.3. The significant wave height is 3.95 meter retrieved from the 1/100 design storm conditions. The breaker parameter $\xi_{m-1,0}$ is a parameter which indicates the wave action on a slope, see Equation 6.2.

$$\xi_{m-1,0} = tan\alpha / \sqrt{s_{m-1,0}} \tag{6.2}$$

In which:

$s_{m-1,0}$	= Steepness parameter [-]

The steepness parameter $s_{m-1,0}$ is calculated through Equation 6.3 (et al., 2007):

$$s_{m-1,0} = \frac{2\pi}{g} \frac{H_{m0}}{T_{m-1,0}^2} \tag{6.3}$$

In which:

$T_{m-1,0}$	= The energy wave period [s]

The energy wave period can easily be extracted from the Equation 6.4 (et al., 2007).

$$T_p = 1.107 \cdot T_{m-1,0} \tag{6.4}$$

For the peak wave period being 19.30 seconds for the governing storm condition of 1/100 years, the $T_{m-1,0}$ becomes 17.43 seconds. With this in mind the steepness $s_{m-1,0}$ is computed and amounts to 0.008323. The breaker parameter $\xi_{m-1,0}$ on its turn becomes 1.37.

Remaining parameters in the overtopping formula concern factors for specific cases. In this dune design is no berm of vertical wall present so these factors are taken to be 1. The most conservative approach for the factor for the angle from which waves approach the dune is to be set on 1 as well. The slope is assumed to be smooth and therefore also set on 1 which might be conservative.

Taken all these parameters into account and using the overtopping volume of 18 L/s per running meter the result is R_c = **6.70 m**

Design Still Water Level The crest height is designed with respect to the design: level Still Water Level (SWL). In this study is used the 1/100 year SWL which consists of:

- HAT (Highest Astronomical Tide) being 1.93 meter above LAT (Lowest Astronomical Tide)
- Advised height of the 1/100 year storm surge = 0.15 meter
- Water level variation due to squall passage = 0.20 meter

All together the estimated 1/100 year SWL is 2.28 meter above LAT. This level is equal to **1.29 meter above MSL**. The 1/100 year SWL is determined at a location in front of the conventional breakwater. The 1/100 year water level is assumed to be the same for the sand breakwater. An overview of local datum with respect to chart datum is shown in Chapter Conventions and Definitions Figure 2.

Design crest height The required crest height is found by adding the R_c to the SWL. During the lifetime of the port however land subsidence occurs and the SWL is subjected to sea level rise (SLR).

The total design crest height with respect to MSL is 8.32 meter according to Table 6.4. The design crest height for the part of the sand breakwater in which a dune is integrated is therefore set on **8.50 m + MSL**. A sensitivity analysis in XBeach in which the height has been varied, is executed and discussed in Appendix J.2.

Table 6.4: Determination of crest height

Components	[m]
Freeboard	6.70
Sea level rise	0.25
Land subsidence	0.075
Still water level difference to MSL	1.29
Design crest height	**8.32**

Conservative approach A statement needs to be made about the applicability of the overtopping formula for dunes. This formula is normally not used for computing overtopping for dunes since dunes deform due to storm impact. Therefore, the wave run-up changes. Recommended is to investigate in further research whether this approach is applicable or what approach would be better to use. However, since the crest height found through the approach above is significantly higher than the crest height found through the approach of URP it is assumed that it is a conservative approach and can be used in the research.

Secondly, this design crest height is computed for a rather small dune profile. In this overtopping computation the effect of a large berm width absorbing a large volume of water is therefore not taken into account sufficiently. The dissipation of the overwash in the berm of the dune decreases its requirement of the protection against overwash. Having said that, the dune height of 8.5 meter is very conservative. From a practical perspective it is possibly not required to be 8.5 meter over the complete berm width along the sand breakwater. It could possibly also be constructed with a lower height of for example 5 meter on the sea side with a gentle slope going from 5 meter to 8.5 meter over the berm width. This way, volumes of water which possibly cause overwash or run-up are still forced back by the gentle slope but the berm width is used to dissipate the overwash. Instead of preventing overwash at the sea front of the berm, the overwash is partly absorbed and less volume of sand is required to used. Consequently, less storm reduction volume due to the smaller berm height. However, this is only possible whenever the dune is wide enough to provide the width for a gentle slope to 8.5 meter. This discussion does not affect the outcome of the feasibility of a sand breakwater due to its insignificance. Conclusively can be stated that for the design further on 8.5 meter has been applied over the total berm width, which is considered a conservative approach.

6.2.1.2 Determination crest height lower dune

The lower dune on the back of the pocket beach in variant 'Lagoon' as described in Chapter 5 requires a different crest height. In Subsection 5.3.3 the governing wave height for the lower dune behind the pocket beach has been determined. As a consequence from the diffracted wave height of H/H_{inc}=0.2 a wave height of about 0.80 meter is considered for the incident wave height. Using the same overtopping formulas and limiting volumes as in Section 6.2 a required crest height for the lower dune can be determined. For a wave height of 0.80 meter the required free board of the dune is 2.2 meter. Assumed is that the same margins are taken into account for the design height as are used for the normal dune crest.

In Table 6.5 the determination of the crest height for the lower dune is shown. It is found that a design crest height with margin of 4.0 m + MSL needs to be integrated in the design. A side note needs to be made about the fact that it could be argued that the SLR and land subsidence are of less importance at the lower dune. This lower importance roots in the fact that sand will accrete on the west side of the lower dune and consequently the dune certainly does not require a 50 year design criteria.

Table 6.5: Determination of crest height

Components	[m]
Freeboard	2.20
Sea level rise	0.25
Land subsidence	0.075
Still water level difference to MSL	1.29
Design crest height	**3.82**

6.3 XBeach

6.3.1 Model introduction

XBeach is an open-source numerical model developed to simulate hydrodynamic and morphodynamic processes and impacts on sandy coasts with a domain size of kilometres and on the time scale of storms (PI et al., 2015). For the purpose of this research the model XBeach has only been applied for one dimensional modelling (1-DH). From the impact of the storms a profile can be determined with a sufficient dune cross-section for certain storm conditions.

The usage of XBeach concerned actually two motivations. First of all, it is used for modelling storm conditions and determining a required **dune width** from these results. Secondly, it can be used to confirm the measured cross-shore profile from the waterline offshore. Long term modelling was executed with XBeach in a way that the profile was modelled on normal, year round, wave conditions. However, the results of the long term modelling of 'normal conditions' turned out unrealistic. The profile of the results forced by the normal conditions in XBeach was far off the present profile which is forced by the wave climate. Long term modelling is not the main purpose of XBeach as it is especially designed for storms and short events but it should be able to resolve the waves to basic level. Since, pre- and post storm morphological data and information are not available, a calibration is not possible. Hence, the modelling programm XBeach is further on discussed with respect to storm impact. With respect to the possible over-estimation of storm reduction in Chapter 8 the liability of the XBeach results determined with default settings are discussed.

6.3.2 Set-up

6.3.2.1 Extreme events

In this study the impact of storms is assumed to be the extreme event for which a sand breakwater needs to be designed. The extreme values used in the design for the conventional breakwater concern extreme swell events, see Subsection 3.3.7. Storms are events in which wind is the driving force causing the wave height to rise and in the middle of the storm, when the wind speed is at his maximum, to lead to high wave heights. Storms are actually quite rare on the Nigerian coast and not governing for morphological impact. These swell events of which the extreme values are computed are rather found in wave trains which are groups of swell waves. This type of event is due to complexity, however, not taken into account in this study and is recommended for further research. In Chapter 8 is elaborated on this different extreme event.

6.3.2.2 Storm conditions

Storms in XBeach are set-up with a JONSWAP file which is a summary of all the wave conditions for each timestep during the storm and modelled period containing the wave height, period, spreading factor, the main wave direction and duration of storm.

In Subsection 3.3.7 the storm conditions for a 1/100 year storm were presented. From Table 3.5 are the 1/1 and 1/10 year storm conditions retrieved. In Table 6.6 these conditions are summarised with the design storm conditions displayed in red.

Table 6.6: Summary of Storm conditions

Return period (years)	H_s (m)	Tp (s)	Dir (°)
1/1	2.80	16.0	189
1/10	3.40	17.9	189
1/100	3.95	19.3	189

A typical storm was created for Badagry for all three conditions: A 1/1 year storm, a 1/10 year storm and a 1/100 year storm. These storm conditions have been set-up by searching in the historical wave data to what the typical storm looks like. The main direction of the waves was established on the average swell direction of 189° with respect to TN as the extreme value found for these storm conditions belonged to the swell waves coming from this direction. This angle is varied for sensitivity purposes in Subsection 6.4.4

Duration A typical storm of 44 hours was modelled with XBeach. The extreme value was set to prevail atleast nine hours of the complete 44 hours of storm. The surrounding values were extrapolated.

Water level variation For the water level input a tidal data series from Lagos has been used for the same period of data as the storm data was found. This data series has been amplified with a storm surge found in Subsection 3.3.4.1 of 0.15 meter. This storm surge starts at 0 meter and increases to 0.15 meter halfway the storm after which it decreases to 0 meter on the end of the storm. In Figure 6.2 the components are shown throughout the storm period. Wave set-up has been taken into account by the modelling software itself

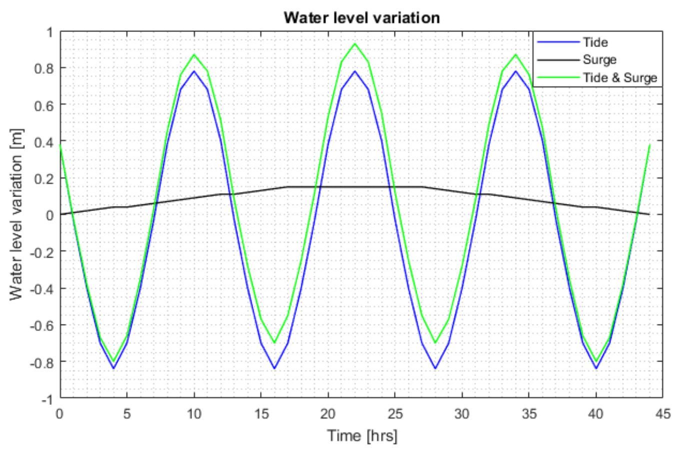


Figure 6.2: Water level variation for modelled storm period

and therefore does not need to be included in the input.

6.3.2.3 Profile

For the bed level profile in XBeach the same profile is used as for LITPACK. This profile is orientated normal to the original coastline with an orientation of 176 degrees with respect to TN. This way the profile is orientated perpendicular to the coastline. As stated in the introduction, the part of the cross-shore profile which is offshore and foreshore up to the top of the intertidal zone is used from a measured dataset containing bathymatry information of the coast Badagry. The measured dataset is integrated to a height of + 1 m MSL. Offshore, the profile has an average gentle slope of (1:300-350) to a depth of around 11.30 m - MSL. From here onwards a steeper slope of 1:50 to 9.00 m - MSL is present. The slope turns even more steep from -9.00 MSL to the beginning of the intertidal zone -1 m MSL. The intertidal zone from -1 m MSL to +1 m MSL has a steep slope varying from 1:30 to 1:20 as known for beaches in the area of Badagry. Above the intertidal zone 1.00 m + MSL the slope of the dune which is applied in the modelled profile amounts to 1:8 until a crest height of 8.5 m + MSL as found in Subsection 6.2.

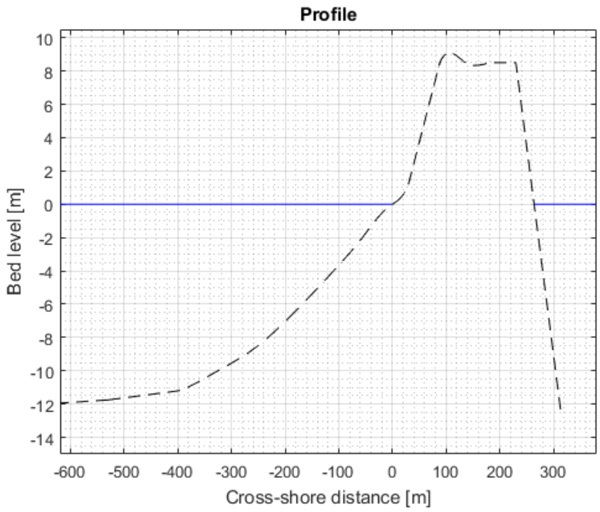


Figure 6.3: Overview of start profile

The dune of the sand breakwater is designed to be constructed with a slope of 1:8 on the sea side. This slope of 1:8 is chosen as it is a slope which is practical and feasible to construct. Although this slope is used to construct the dune, a dune's known reduction slope is 1:1. This is the avalanching slope of sand at a dune. For this reason the reduction with the first (storm) impact on the sand breakwater leads to a different front slope of the dune than as it was constructed, leading to a 1:1 slope on sea side. As long as the dune's berm has a sufficient width that is not something of importance.

Grain size As stated before, the beaches of Badagry consist out of rather coarse sand, from which is assumed that the d_{50} amounts to 300 µm. The swash zone might contain a bit less coarse material however the dune profile itself will be constructed with more coarser material and for that this grain size is more conservative. The d_{90} is set on the coherent 700 µm. As XBeach is not able to take into account spatial variation of grain size only one value for the grain size can be taken into account. The default value for implemented for grain size d_{50} is 300 µm. Sensitivity of this parameter is further discussed at Subsection 6.4.4.2.

6.4 Results

6.4.1 Introduction

As stated in Subsection 6.3.1, the goal of using XBeach is to find the impact of storms on the profile. Followed by concluding on the impact of storms what kind of minimal required crest width is required for certain design conditions for the sand breakwater. The minimal required dune width is determined on a number of parameters.

During a storm the dune cross-section will decrease with a volume of sand which is called the "Reduction volume" or "A0". Coherent with reduction the "R0", the "Reduction line", is designated on top of the dune. This is the location to where the avalanching of the dune is modelled to take place.

In this section three storms with different conditions are examined on their impact in Subsection 6.4.2. Following from the impact of the multiple storms an analysis of consecutive storms is done in Subsection 6.4.3. This is followed by a sensitivity study to various parameters and assumptions in the XBeach model in Subsection 6.4.4. Lastly, the results from the previous subsections are combined in Subsection 6.4.5. In this subsection the profiles are determined for all conceptual variants.

6.4.2 Multiple storm conditions

For three storms with different return periods the storm impact has been modelled on the profile. As stated in the previous section, the profile is orientated on the original coastline. In Section 6.4.5 the results of the orientation of the profiles of interest for the design of the multiple variants are presented. The results of the model runs for the original coastline are shown in Figure 6.4 and Table 6.7. The reduction of the dune profile by the storms with longer return periods are evidently larger than storm with shorter return periods (Damage 1/100 > 1/10 > 1/1). Although the damage is larger, it is not significantly larger. The impact of the three storms are relatively close to each other.

Designing the Sand Breakwater rank and repair it when it fails within lifetime is possible. However, when the structure fails, the breaching is expcted to cause rapid destruction taken into the persistent wave climate. The costs of repairing such an impact are significantly high as the port will have much downtime. In Chapter 7 the storm conditions are discussed from economic perspective with respect to maintenance.

Table 6.7: Result of multiple storm conditions on start profile

	Return period [years]		
Items	**1/1**	**1/10**	**1/100**
A0 = Reduction volume [m^3/m]	255.76	293.06	330.48
R0 = Location reduction line [m]	95.19	101.19	107.19

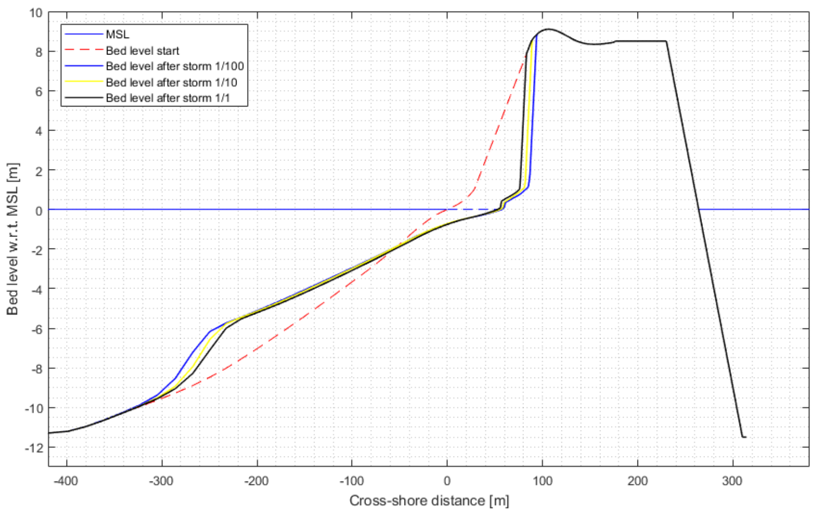


Figure 6.4: Results of storms with different return periods on start profile

Design condition For the conventional breakwater design an extreme value of a 1/100 year swell wave has been used as design criteria. Normally, when constructing a hard structure, a 1/100 year storm is of significantly larger impact than the other two storms. In this case, a 1/10 year storm or even a 1/1 year storm is also of relatively big impact. This is due to the fact that morphological design conditions are not able to compare to design conditions for hard structures like a conventional breakwater. Therefore, is assumed that the main design condition of the 1/100 year storm condition should be expanded with extra safety margins more related to sediment transport.

6.4.3 Consecutive storms

An important note about the modelled impact of the storms is that a storm with a 1/1 year return period is relatively significant to the impact of a storm with a 1/100 year return period. If this damage would occur and the sand breakwater would not be repaired in time before another storm occurs this would lead consequently to yet again significantly large damage. Therefore the impact of a 1/1 year storm is interesting to model on a different profile then the start profile. The impact of two consecutive 1/1 year storms on the start profile is therefore chosen to examine.

The probability of occurrence of a certain storm is not given by the return period of the storm alone. The probability is actually given by the "Poisson Distribution". This distribution predicts the likelihood that a given event will occur a certain number of times during a specific period of time, and is valid for all processes where the duration of the event itself is much shorter than the period of time that is being analysed (Verhagen et al., 2009). In this study we examine storms of 44 hours in the time span of years so it is a valid distribution.

The probability that a 1/1 year storm occurs k times in a year is given by Equation 6.5.

$$Pr(k) = \frac{\lambda^k \cdot e^{-\lambda}}{k!} \tag{6.5}$$

λ is the expected value of the number of occurrences per year. As it concerns a storm with a return period of 1 year λ is 1. The scenario that this breakwater is hit by a 1/1 year storm twice or maybe thrice before repair works are executed is present. The probability for three storms of return period 1/1 in one year amounts to 6%. That is actually a rather low probability. Although this chance might be low, the chance that two storms would occur in one year amounts to 18% which is a significant probability. It is unknown how much time is available for repair works in between two consecutive storms. Nonetheless, as it concerns Nigeria, maintenance could delay and it is assumed that this probability is important to take in mind.

In Figure 6.5 the result is shown of two consecutive 1/1 year storms. This run has been executed for the average swell angle of 189° with the profile orientated on the original coastline.

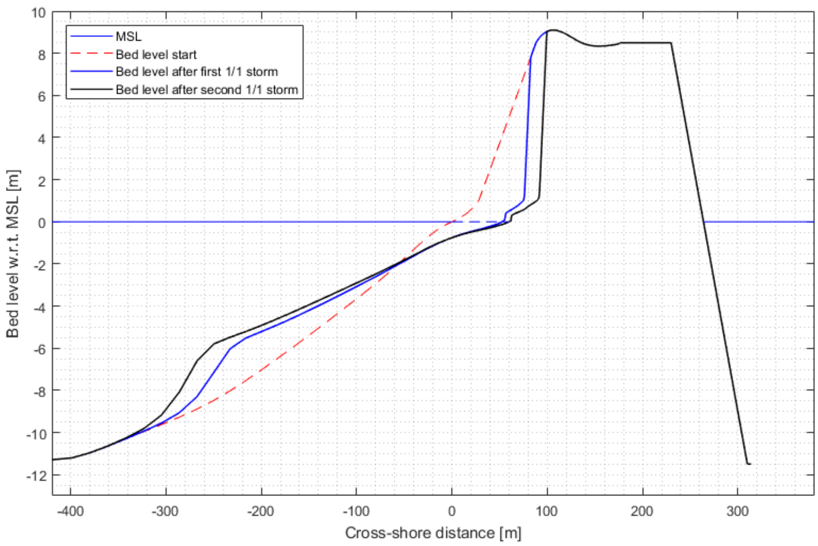


Figure 6.5: Results of two consecutive storms with a 1/1 year return period on start profile

In the Figure 6.5 is clearly visible that the impact of a second 1/1 year storm is significantly lower than the first 1/1 year storm. This is a consequence of the adapted profile which is caused by the storm deposition of the first storm. The question is, whether this sand which is deposited from the first storm actually will stay in place before the second consecutive storm impacts the coast which is discussed in Subsection 7.3.3.2.

Table 6.8: Result of two consecutive 1/1 storms on start profile

Items	First storm	Second storm
A0 = Reduction volume $[m^3/m]$	253.92	132.24
R0 = location reduction line [m]	99.80	82.65

The modelling has been executed for the impact of a 1/1 year storm on the coast. This impact could be in reality somewhat different as the deposited volume of sand might already be transported alongshore, onshore or offshore which leads to different results. These outputs are shown in Figure 6.6. The impact of the storm is obviously higher as the shoaling of the waves is occurring later on the cross-shore profile.

For reasons with respect to conservative design it is desired to take this consecutive storm in account. Therefore is decided that the Sand Breakwater should have the following design condition:

A Sand Breakwater should not fail for a design storm with a return period of 1/100 and should have extra safety margins with respect to occurrence of consecutive storms with a return period of 1/1.

The difference in location of reduction lines of both storms amounts to about 17 meter with the deposited slope intact for the second storm, see Table 6.8. The difference when the slope is cleared already when the second storm impacts the coast amounts to 21 meter. A margin of 19 meter is advised for this criteria and integrated in the profiles in Subsection 6.4.5.

Table 6.9: Result of two consecutive 1/1 storms on start profile

Items	First storm	Second storm
A0 = Reduction volume $[m^3/m]$	253.92	203.03
R0 = location reduction line [m]	82.20	103.19

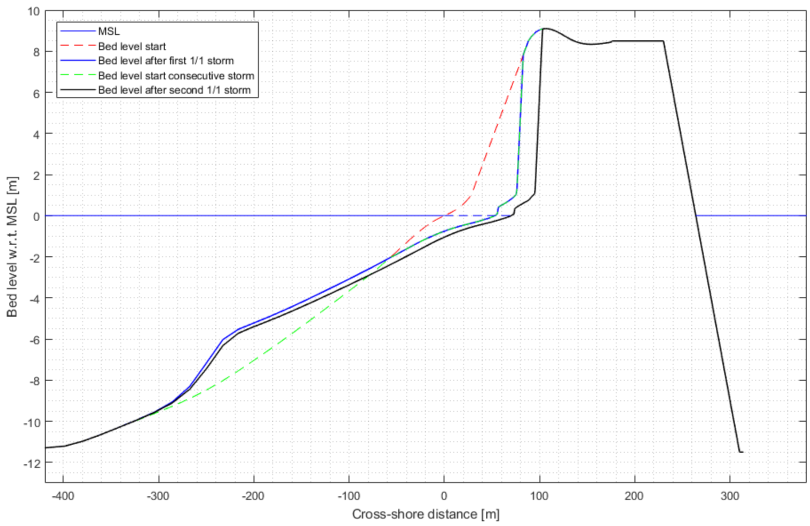


Figure 6.6: Results of two consecutive storms with a 1/1 year return period on start profile without deposited volume for second storm

A note needs to be made about the storm reduction profile. The impact of a consecutive storm with the deposited volume on the foreshore is significant lower. This profile is apparently better able to withstand storms. The reason that this profile is not used and integrated in the designs is that from measurements is known that this profile is not present in measured data at the coast of Badagry. This deposited volume of sand will not stay there due to the present wave climate. The known measured profile is the profile which is present almost all year round on the coast of Badagry. For this reason the measured profile is used in the design of the sand breakwater and not a storm reduced profile even though this would suit better to the lowering of storm impact.

6.4.4 Sensitivity analysis

In this section the sensitivity of certain parameters in XBeach are presented. This sensitivity analysis provides the accuracy of the designs for the different variants. From these Subsections follow certain impacts on the design which might lead to extra margin in dune cross-section. First of all the sensitivity of the main wave angle in a storm will be discussed. After which the grain size is examined and finally the "Spreading factor" is discussed. The sensitivity of the crest height, angle of slope and friction formula are also examined. The impact was found of negligible influence, moreover in Appendix J.

6.4.4.1 Main wave angle

The main angle of the storm is in all runs set to 189° with respect to TN as that concerns the average swell direction. It is important to know what the sensitivity of this angle is on the storm reduction. The range of angles which have been examined is 177.50° to 202.50° this range contains 97.5% of all swell events as was stated before in Subsection 5.3.3. As presented in Section 3.3 these swell events are the extreme events with impact and are uni-directional. Taken this into mind it can be said that this range suffices to examine the impact of change in wave angle. The main angle of swell events is 189° so the maximum deviation in the bandwidth of swell events is 202.5-189=13.5°.

In Figure 6.7 the results of four different main angles of storm conditions are displayed. In Figure 6.7 and also Table 6.10 can be concluded that when the angle of the storm increases wrt the main angle, the storm reduction increases as well. When the storm reduction increases, the reduction line of the dune increase in distance from the beginning of the profile. This increase was expected as the transport of sand is known to be larger when the angle of the incoming waves increase with respect to the coastline. The angle of 176° concerns the direction which is perpendicular to the original coastline. The least storm reduction is occurring with this angle obviously. The direction of 189.5° concerns a difference in main wave angle of 13.5° which is the maximum deviation in bandwidth of storm direction. The difference in storm impact is 14% between the direction of 176° and 189° and amounts to a difference of five meter in location of the reduction line in this run. Conclusively is stated that this difference is taken into account as a margin for main wave angle sensitivity.

A margin for wave angle sensitivity of five meters is advised and integrated in the cross-sections in Section 6.4.5.

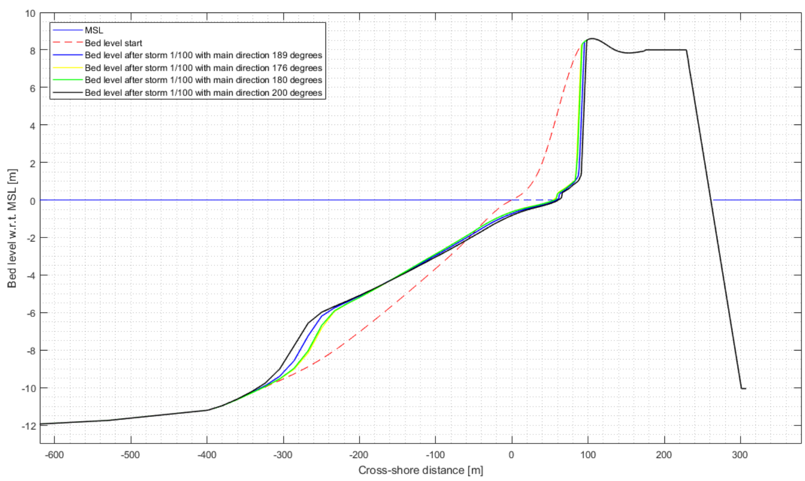


Figure 6.7: Overview of results of 1/100 storm conditions on start profile with different main angles for the storm conditions

Table 6.10: Result of 1/100 storm from multiple directions

	Different direction w.r.t. TN			
Items	**176.0°**	**180.0°**	**189.0°**	**200.0°**
A0 = Reduction volume $[m^3/m]$	286.42	293.18	326.56	363.31
R0 = location reduction line [m]	104.19	105.19	109.19	113.69

6.4.4.2 Grain size

Another parameter is examined for its sensitivity is the grain size. At Badagry is rather coarse sand present on the beach and behind the swash zone(650 µm) but in the swash zone this grain size can be a significantly lower (150-200 µm). Assumed, however, is that the dune itself will be constructed of coarser sand than the sand found in the swash zone. As default value for d_{50} is 300 µm assumed. In this subsection the impact of varying values around that figure are examined. In Figure 6.8 the results of different grain sizes of d_{50} are displayed.

Table 6.11: Result of 1/1 storm on start profile with multiple grain sizes

	Different grain sizes D_{50}			
Items	**250 µm**	**300 µm**	**350 µm**	**400 µm**
A0 = Reduction volume $[m^3/m]$	291.79	268.92	246.83	234.04
R0 = location reduction line [m]	104.08	101.08	97.08	90.08

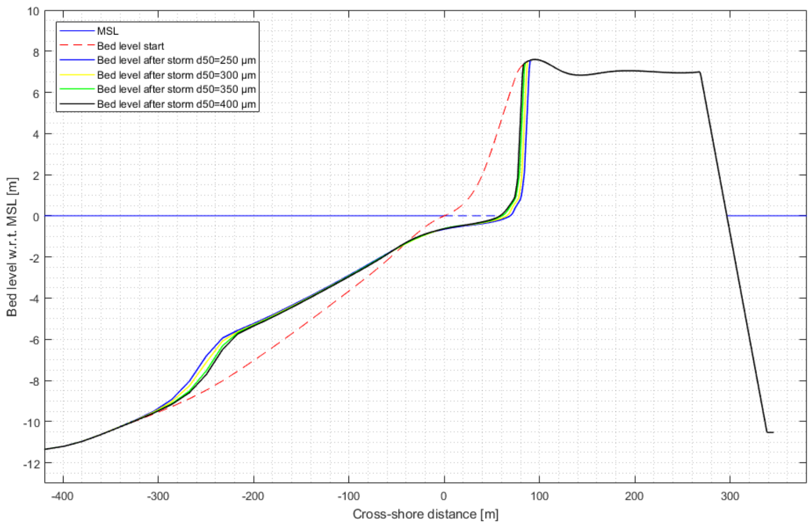


Figure 6.8: Overview of results of 1/100 storm conditions on start profile with different grain sizes

From the Table 6.11 and Figure 6.8 can be concluded that the different grain sizes have an impact on the volume of storm reduction and with that the location of the reduction line. The impact of different grain size is evidently larger with smaller grain sizes as the relation between sediment transport and grain size is not linear. Taken into account the fact that the material on the beach is quite coarse and the material which will be dredged from the port and the approach channel as well, the assumption of default value being 300 µm is seen as conservative. Lastly the difference between 300 µm and 250 µm however is only three meter in location of reduction line.

A margin for grain size sensitivity of three meters is advised and integrated in the cross-sections in Section 6.4.5.

6.4.4.3 Spreading coefficient

Just as in LITPACK a spreading coefficient is applied in XBeach. This spreading coefficient is very empirical and hard to define for a specific case. As discussed in chapter 4 Badagry is swell dominated and has a uni-directional character. Two set-ups with different coefficients are compared in this subsection. In XBeach a default value for "S", being the spreading coefficient, is 10. This regards normal sea state. Swell dominated sea states with a more unidirectional character are often described by a value of 20. This is a conservative number as in reality during an extreme event or storm the waves are described by a value of S over 20.

Table 6.12: Result of 1/100 storm condition on start profile with different spreading coefficients

Items	Spreading Coefficients S = 10	S = 20
A0 = Reduction volume $[m^3/m]$	355.59	379.84
R0 = location reduction line [m]	116.08	120.08

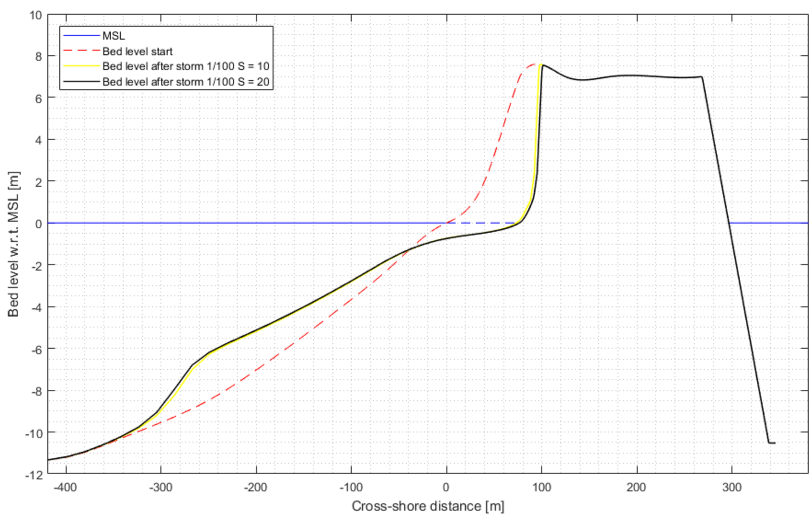


Figure 6.9: Overview of results of 1/100 storm conditions on start profile with different spreading coefficients

Figure 6.9 and Table 6.12 show that the result is that more sediment is transported by storm reduction when S has a higher value. Although the sediment transport is higher it is not significantly higher. Only four meter is the difference in location of reduction line. We assume that S being 20 for a swell dominated wave climate is a realistic set-up and needs no further margin for the dune cross-section.

No margin for the spreading factor is integrated in the cross-sections

6.4.4.4 Conclusion

Multiple parameters examined on sensitivity in the model show that the storm impact can vary significantly when these parameters are shifted.

First of all, the parameter like the main wave angle was examined. The difference in storm impact due to a storm coming from average swell direction or the outer edge of the swell event bandwidth concerns 14%. This translates into a difference of five meter in location of the reduction line on the crest. Taken this difference into mind an extra crest width of five meter is advised for the sensitivity with respect to the main angle.

Secondly, the grain size has been varied in XBeach to examine the sensitivity of grain size in storm impact. The modelling with XBeach has been executed with a grain size of 300 µm. The difference between 300 µm and 250 µm concerns only three meter in location of reduction line. Conclusively can be said that a margin of three meter for the dune's cross-section needs to be taken with respect to grain size.

Thirdly, the spreading coefficient was examined on sensitivity to storm impact. The result is more sediment transport by storm reduction when the spreading coefficient "S" has a higher value. Although the sediment transport is higher it is not significantly higher. Only four meters is the difference in location of reduction line. Since, the modelling has been executed with a setting which is realistic for a swell dominated wave climate, no extra margin is applied in further design for this parameter.

Lastly, also other parameters which have been examined on their sensitivity, are the crest height, angle of slope and friction formula. The impact of these parameters is negligible and not taken into account. These sensitivities and can be found in Appendix J.

6.4.5 Variant Profiles

In the three conceptual variants from Chapter 5 three different orientations of profiles exist. These profiles are modelled and examined in this subsection instead of analysing the profile orientation on the normal coastline. The final required profile cross-section determined for these orientations are in this subsection **only presented with respect to storm conditions**. The extra margins for these profiles with respect to coastal development of the previous chapter are further integrated in final conceptual design in Chapter 7.

In variant "Sand Nourishment" there are two stretches of coast which have two different orientations. One is orientated with an angle of 60° with respect to the original coastline of Badagry. Perpendicular to this coastline is the first profile which requires modelling, see Subsection 6.4.5.1. This profile is shown in red in Figure 6.10.

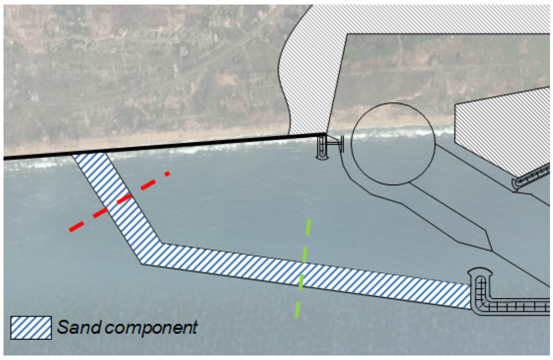


Figure 6.10: Overview of different profiles in variant 'Sand Nourishment'

The second profile concerns the profile of a coastal stretch constructed in equilibrium orientation. This profile is shown in green in Figure 6.10. This orientation of the coastline was found in Chapter 5, see Subsection 6.4.5.2. This profile is also present in variant 'Groyne' and variant 'Lagoon'.

In variant 'Lagoon' is yet another profile present. The profile of the lower dune behind the pocket beach. The location of the profile is shown in Figure 6.11 in orange and presented in Subsection 6.4.5.3. In Table 6.13 the results of the XBeach modelling for the red and green profile orientations are shown.

Table 6.13: Result of 1/100 storm condition on start profile with different orientations

Items	Orientation 236.3°	Orientation 187.5°
A0 = Reduction volume [m^3/m]	406.57	290.24
R0 = location reduction line [m]	99.69	90.19

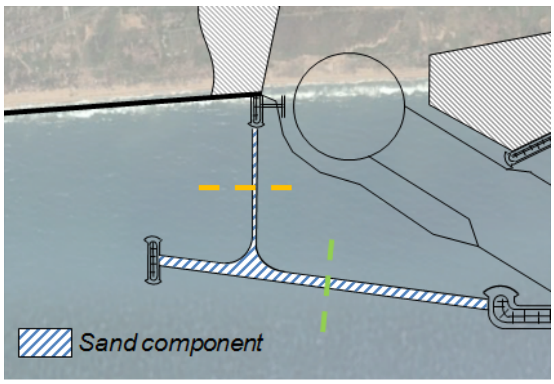


Figure 6.11: Overview of different profiles in variant 'Lagoon'

6.4.5.1 Profile 1

Profile one concerns the red profile and is located on the stretch of coast with an orientation of 326.3° w.r.t. TN. The profile which coheres with this stretch is orientated with an angle of 236.3° to TN. The main angle of the storm direction should not be mistaken and is still 189° w.r.t. TN.

Table 6.13 shows that the reduction line R0 is located at about 100 meter distance from the water line of the profile. In Subsection 6.4.4 an extra margin of five meter was advised due to different storm directions. In Subsection 6.4.4.2 another margin of three meter was advised for the reduction line. From the occurrence of consecutive 1/1 storms a margin of 19 meter was computed. The end of the crest is all together advised to be located at 126 meter from the water line. Taken into account that the dune crest begins at a distance of 88 meter the crest width becomes 38 meter. This profile is displayed in Figure 6.12.

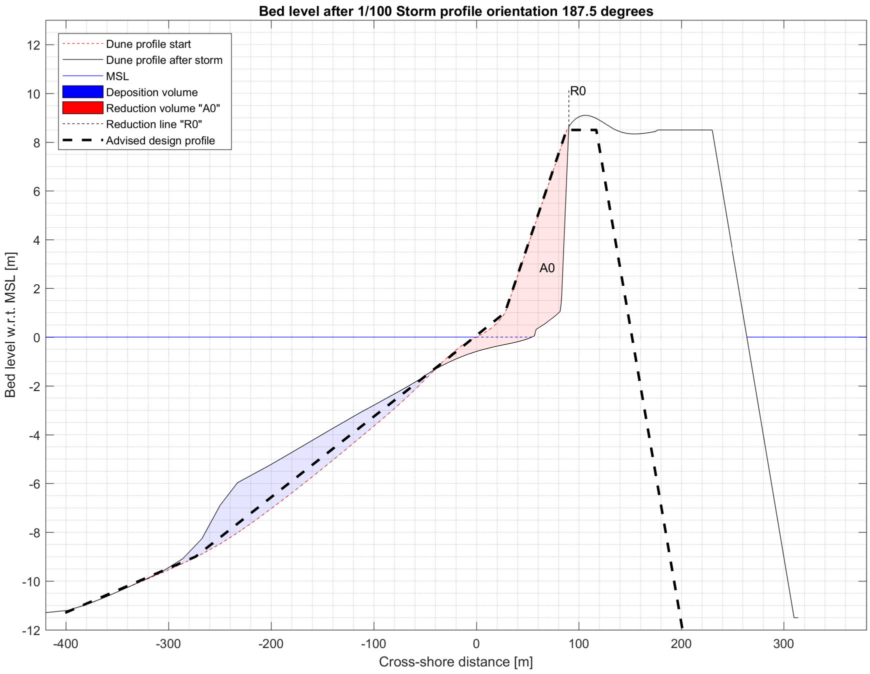


Figure 6.12: Overview of profile orientation two modelled in XBeach

6.4.5.2 Profile 2

In Subsection 7.2.2 the equilibrium angle of the coastline was redefined on 277.5°. The profile of this stretch of coast is in that case located under an angle of 187.5°. Table 6.13 shows that the reduction line R0 is located at 90 meter distance from the water line. This distance summed up with the extra margin of five meter, three meter and 19 meter due to different margins directions amounts to 117 meter for the location of the end of the crest. Taken into account that the crest begins at a distance of 88 meter the crest width becomes 29 meter. This profile is displayed in Figure 6.12.

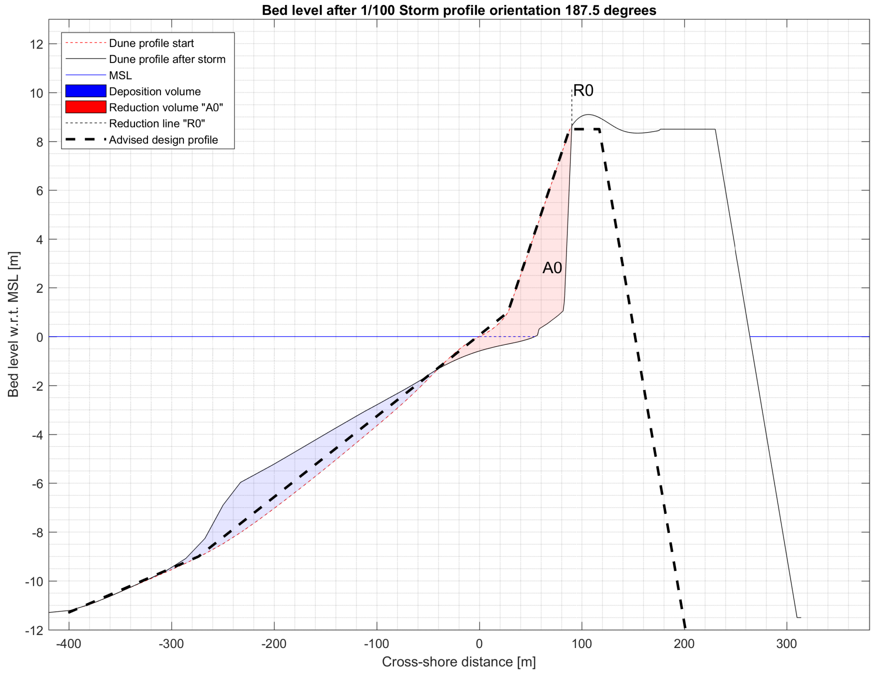


Figure 6.13: Overview of profile orientation two modelled in XBeach

6.4.5.3 Profile 3

In variant 'Lagoon' "Lagoon" a third profile is present. This profile concerns the lower dune. This lower dune as described in Subsection 6.2 contains a required height of 4.0 m + MSL. The required dune width is taken to be the same as that of profile two, 29 meter.

6.4.6 Conclusion

They key objective in this chapter was to achieve the answer on the sub-question:

What kind of cross-shore profiles are sufficient for its storm conditions?

First of all, a design crest height was determined for the minimal required storm profile based on the overtopping of waves. This design crest height was determined for a 1/100 year storm design condition.

With this parameter known, various modelling was executed to determine the stability of the sand breakwater for several storms conditions. The impact of storms with shorter return periods appeared significant for which the design criteria of a design storm of 1/100 was expanded with a margin for consecutive storms with a return period of 1/1.

The sensitivity was afterwards determined for certain parameters which were adopted as margins in the final required cross-shore profiles.

This all together led to three different minimal required profiles which are required in the different conceptual variants. These profiles are proven to be able to withstand the 1/100 year storm condition expanded with the margin for consecutive storms with a return period of 1/1. Therefore, the cross-shore profiles are proven to be stable for storm impact and the objective is achieved. The minimal required profiles for the conceptual variants are integrated in Chapter 7 for the coherent cross-sections of which the final conceptual designs exist.

7

Design Sand Breakwater

7.1 Introduction

The answer on the main research question whether it is feasible to construct such a sand breakwater is, as stated before in Chapter 2, intertwined with the economical feasibility. In the process of establishing an answer on the main research question it is important to compare the variants to each other but mostly to the conventional breakwater. This chapter forms **phase 2** of the research, see Section 2.3. As stated in Chapter 2 only the main breakwater is posed in the scope of this study. All other elements such as the port are ignored in the volume estimations and cost comparison.

In Section 7.2 the final designs of all conceptual variants are presented. To be able to make a rough cost comparison, volume estimations of the main materials are made for all variants. A conventional design is used as a raference case. All these volume estimations are presented in Appendix L. In this appendix thorough volume estimations are presented for the variants and a conventional breakwater. In addition, the process the volume estimations and the used assumptions are presented. In Appendix K, detailed graphical information of the cross-sections is given.

From these volume calculations a cost comparison is examined based on unit prices in Section 7.3. A more detailed cost overview is presented in Appendix L in Section L.2.

The rough cost comparison in this chapter in Subsection 7.3.2 along with the future prospect in terms of maintenance and loss of sediment, see subsection 7.3.3, form the keystones in the conclusion concerning the main research question. This conclusion is presented in Chapter 9.

7.2 Final design of the conceptual variants

7.2.1 Variant Sand Nourishment

In the variant 'Sand Nourishment' the baseline was the maximised usage of sand instead of hard structure. The hard structures present in this variant are the short LNG breakwater and the hard tip of the sand breakwater.

7.2.1.1 Sand component

To estimate the required volume of sand for variant 'Sand Nourishment' the bathymetry line of the sand breakwater has been examined along its axis over the water line on the dune front. Along this bathymetry line of the sand breakwater, multiple cross-sections have been examined to gain a relative accurate estimation of the quantity of sand required to construct the sand breakwater. To find accurate volumes, the cross-sections are examined at the transition between two approximately linear sections in the bathymetry.

In Figure 7.1 the locations of the cross-sections are displayed in a scaled overview of the variant. In yellow is the emerged area of the sand breakwater indicated and in blue the submerged area. It needs to be noted that on the east side of the cross-section E a toe of sand has been estimated. Further research is required on whether the toe might scour into the approach channel around the tip of the breakwater.

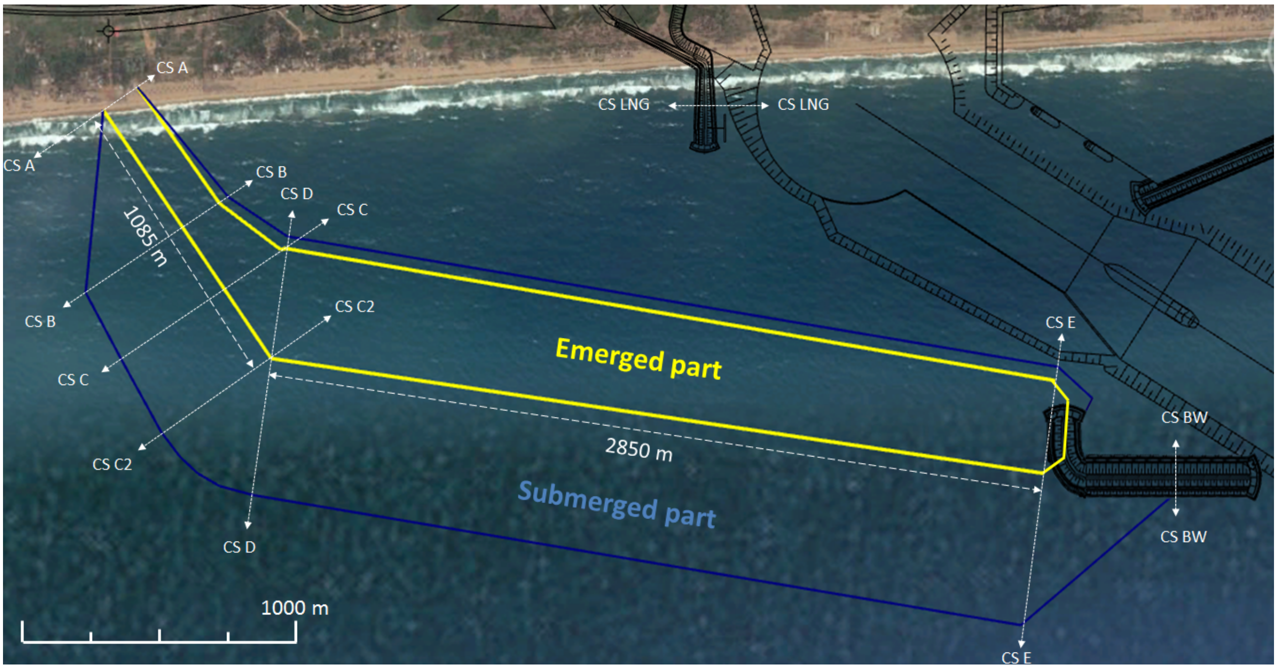


Figure 7.1: Overview of variant 'Sand Nourishment' with designated cross-sections along the sand breakwater. The emerged area of the sand breakwater is indicated with yellow and the submerged part in blue.

The cross-sections differ on two aspects. First of all, the cross-sectional profiles differ in local bathymetry obviously due to different locations. Secondly, the cross-sections depend on the required berm width which on its turn consist out of two components:

- Required berm width due to storm impact, see Chapter 6
- Required berm width due to coastline development, see Chapter 5

The cross-sections are shown in Appendix K. The coherent berm widths are shown in Table L.1.

Table 7.1: Required cross-sectional dune area variant 'Sand Nourishment'

	Required Berm width [m]		
Cross-sections	Storm reduction	Coastline development	Total
A	38	0	**38**
B	38	0	**38**
C	38	90	**128**
C2	38	90	**128**
D	29	250	**279**
E	29	185	**214**

Cross-sectional required volumes per running meter have been used to interpolate the volume of the stretches of sand breakwater in between these cross-sections. Moreover this volume estimation can be found in Appendix L. The total volume of sand required for the construction of variant 'Sand Nourishment' is estimated on 34.3 million cubic meters. This amount concerns the rough estimated construction volume purely required for the profile as it is designed. In practice a lot of sediment is expected to be lost during construction due to wave interaction, profiling and possible settling.

The expected loss of sediment due to wave interaction, profiling and possible settling is asumed to be a quarter of the original required construction volume.

This loss amounts to 8.6 million cubic meters. Taken this extra margin of required volume of sand into account the total required volume of sand for variant 'Sand Nourishment' comes down to **42.9 million cubic meters**. Although this is a large volume of sand, based on basic estimations atleast 50-60 million cubic meters should be available due to the dredging activities for the port basin and approach channel. The question remains whether all dredged sand can be re-used in the sand breakwater. In most of the dredging locations the grain size of the sand is large, on average larger than 300 µm and therefore assumed usable for this sand breakwater.

7.2.1.2 Breakwater tip

For the construction of the hard tip of the sand breakwater the conventional design is used. The tip of the sand breakwater is assumed to be constructed first, the tip of the sand breakwater is constructed, followed by the sand component. In Figure 7.4 in which the emerged part of the dune is shown to go across the curved part of the breakwater. The tip of the breakwater consists of two parts in a "L"shape. The short side is 200 meters long and should protect the sand breakwater, along cross-section E, against coastal development and little LST during the modelled period of 50 years, see Chapter 5. The Long part, between the breakwater and the approach channel is 800 meters.

The volume estimation for the tip of the breakwater is done by calculating required volumes of rock per running meter which are extracted from the conventional design shown in Figure L.4. These volumes of rock per running meter together with the total volume is presented in Appendix L.

All types of rock can be extracted from quarries within Nigeria. This has been done similarly for earlier projects like Eko Atlantic, see Appendix A, and shown in previous (RoyalHaskoning, 2011a). In this conventional design, concrete Accropodes are used. These are armour elements which can be placed in a single layer on the breakwater with a steep slope 3V:3H. The placement of these elements however is rather time consuming as certain criteria exist for the elements like enough interlocking and 'interaction surface' with other elements. This is taken into account for defining the unit prices in Subsection 7.3.2.

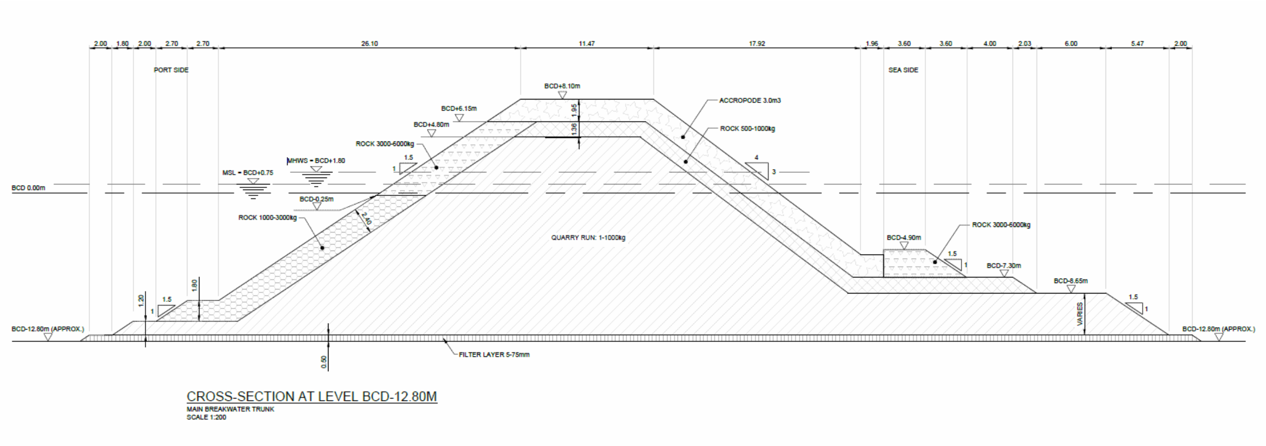


Figure 7.2: Cross-sectional overview of Breakwater (CS BW)

7.2.1.3 Breakwater LNG jetty

The length of breakwater required for the LNG Jetty is an assumed boundary condition in this study. This structure is required in all variants and thus similar in all variants.

For the estimation of the materials required for the construction of this hard structure another cross-section (CS LNG) of the conventional design is used, see Figure 7.3. This cross-sectional design of the conventional breakwater is at a depth of 9.25 m - MSL. The LNG jetty is assumed to have a length of 300 meters and with this length it reaches a depth of about 9.25 meter. From the cross-sectional design in Figure 7.3 the required volumes of material per running meter are estimated. These volumes of rock per running meter, together with the grand total for this hard structure, are presented in Appendix L.

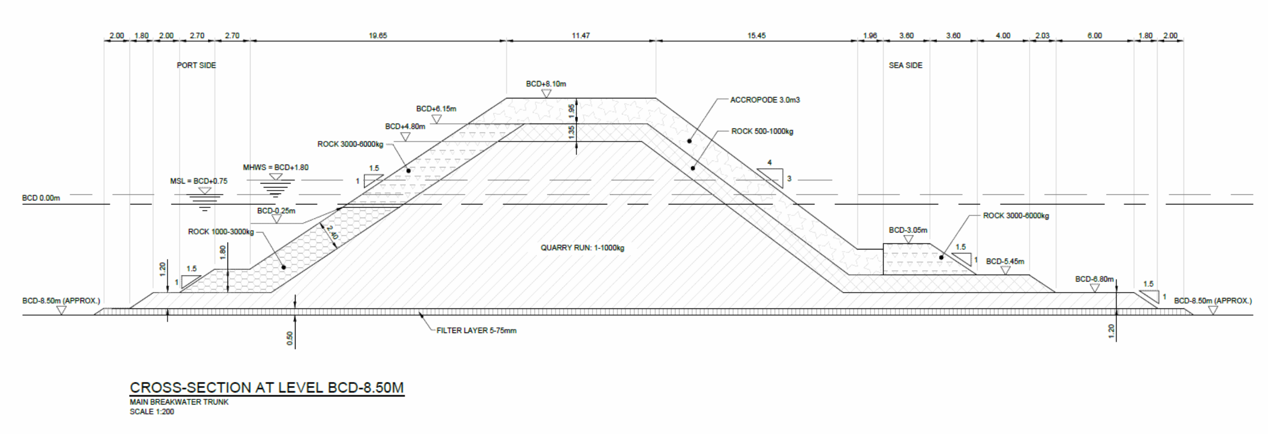


Figure 7.3: Cross-sectional overview of smaller breakwater used for placement of the LNG jetty (CS LNG)

7.2.2 Variant Groyne

In the variant 'Groyne' the groyne was implemented to preserve the orientation of the coastline constructed under equilibrium orientation by preventing LST westwards. The hard structures presented in this variant are the short LNG breakwater, which is equal to the last variant, the tip of the sand breakwater and the groyne on the west side.

7.2.2.1 Sand Component

In this variant only the coastal stretch with equilibrium orientation is present and therefore only has two cross-sections on both sides of the stretch, cross-sections A and B. In Figure 7.4 the cross-sections are displayed in an overview of the variant.

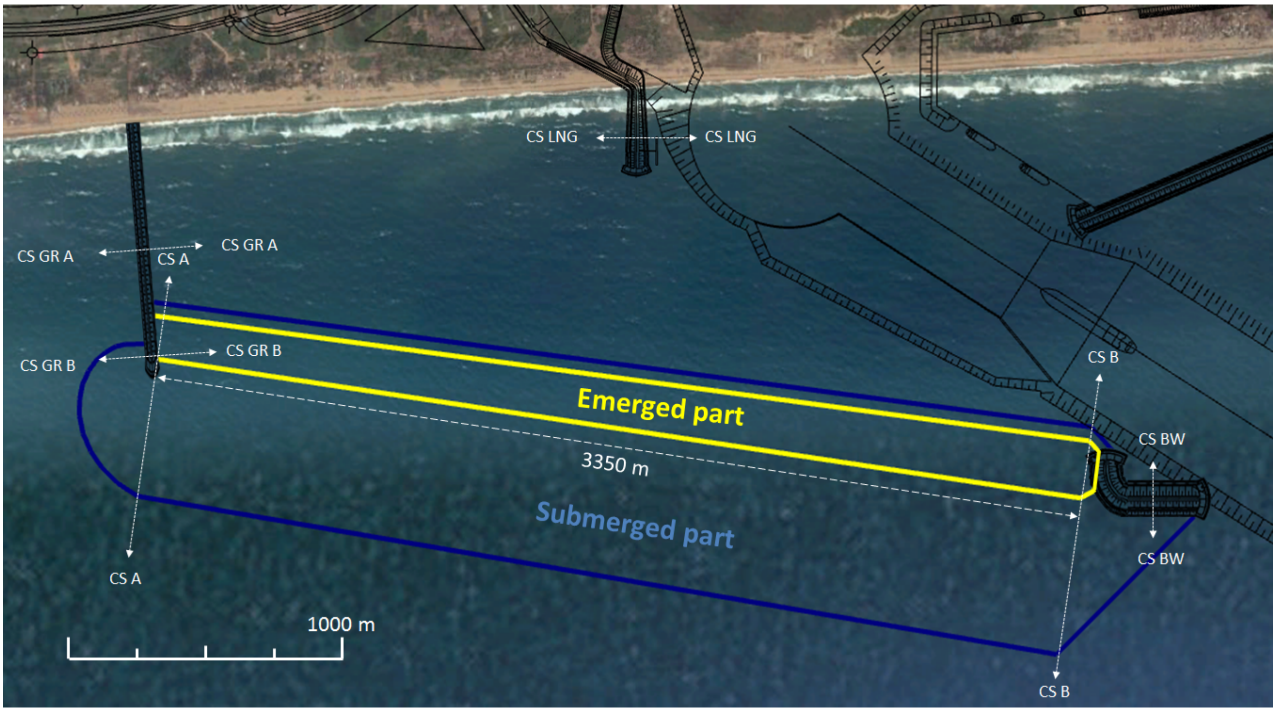


Figure 7.4: Overview of variant 'Groyne' with designated cross-sections along the sand breakwater. The emerged area of the dune indicated with yellow and the submerged part of the dune indicated in blue.

Since, the berm widths for variant 'Groyne' and variant 'Lagoon' are ranker, sensitivity to wave angle is important to take into account, see Subsection 5.5.1. Due to this, the required berm widths for these cross-sections consist out of three components:

- Required berm width due to storm impact, see Chapter 6
- Required berm width due to coastline development, see Chapter 5
- Required berm width due to sensitivity in wave angle, see Subsection 5.5.1

These requirements together lead to the total required bermwidth, see Table 7.2.

Table 7.2: Required cross-sectional dune area variant 'Groyne'

	Required Berm width [m]			
Cross-sections	Storm reduction	Coastline development	Sensitivity wave angle	Total
A	29	30	20	**79**
B	29	30	20	**79**

The cross-sectional surfaces from Appendix L Subsection L.1.3.1 have been used to interpolate the volume of sand used in the stretch. From these figures a total amount of required sand for this variant is estimated on 21.4 million cubic meters. To this rough estimated volume a quarter is added as surplus (5.4 million cubic meters) leading to a grand total of required volume of sand for variant 'Groyne' of **26.8 million cubic meters**.

7.2.2.2 Breakwater tip

In variant 'Groyne' the horizontal stretch of the breakwater, see Figure 7.1, is 400 meter and the stretch curved along the dunes cross-section B amounts to 100 meter. Hence assumed is that the total breakwater length amounts 500 meter. In Appendix L the volume estimations for the hard tip of the sand breakwater are presented.

7.2.2.3 Groyne

The function of a groyne is in this variant different than that of the hard tip of the sand breakwater. Instead of having a berm height of 7.35 m + MSL the groyne only needs to be able to protect the dune from alongshore sediment transport on the west side of the in equilibrium orientated coastline. In addition, it needs to stop the LST along the original coastline. Possibly due to secondary currents, e.g. currents caused by set-up differences, sediment might be transported into the port. This 'secondary LST' could occur even though there is no wave action behind the sand breakwater. Due to this different function of the hard structure, the groyne is assumed to be constructed much cheaper. No detailed design for the groyne is executed. It is assumed that the groyne requires half of the materials which are used in the conventional breakwater design. The cross-sectional information and approach presented in Appendix L were used to find the volumes for the groyne.

7.2.3 Variant Lagoon

In the variant 'Lagoon' three hard structures can be distinguished. The smaller groyne on the west side of the breakwater, the tip of the breakwater and the LNG jetty breakwater. The tip of the breakwater is equal to the tip of the variant 'Groyne'. The LNG jetty is equal to previous variants.

7.2.3.1 Sand Component

In Figure 7.5 the design of variant 'Lagoon' is shown. In this overview all the cross-sections along the sand breakwater are displayed to scale.

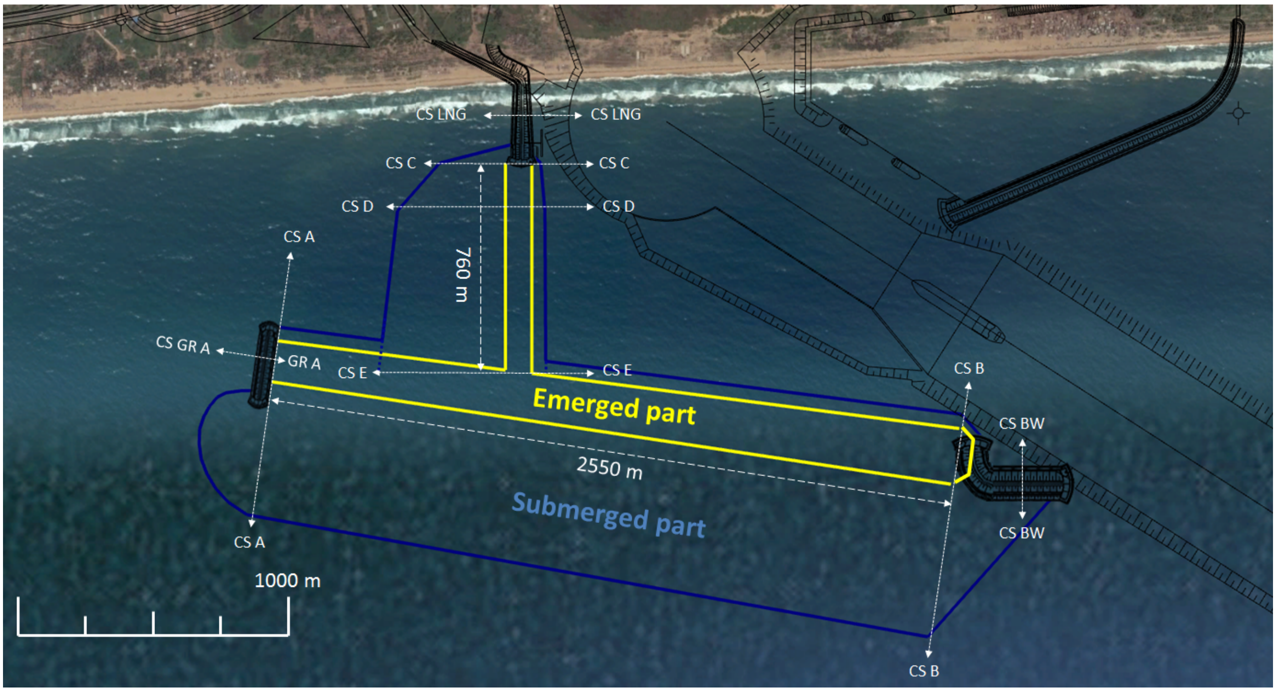


Figure 7.5: Overview of variant 'Lagoon' with designated cross-sections along the sand breakwater. The emerged area of the dune indicated with yellow and the submerged part of the dune indicated in blue.

Berm widths of the sand component are equal to variant 'Groyne' and can be found in Table 7.2. The emerged part of the pocket beach is equal to the variant 'Groyne' except it is short. Also the lower dune requires a certain volume of sand. This is computed from three cross-sections C, D and E in Figure 7.5 at - 8.0 m, -11.3 m and -13.0 m MSL respectively. The sand volume required for this variant is 21.6 million cubic meters. On top of this the surplus of a quarter is added leading to a grand total of 27.0 million cubic meters.

7.2.3.2 Groyne

The length of the groyne in variant 'Lagoon' is smaller than the groyne in variant 'Groyne'. It is assumed that the groyne has a length of only 300 meters. This length covers the complete berm and submerged slope on the back of the dune. In addition, it stretched out on the front of the dune for a length of 100 meters. Due to this, it is able to block the little LST occuring on the pocket beach. This is evidently covering the zone in which LST largely takes places as this is confirmed in the width of the breaker zone in Section 4.3. In Appendix L volumes are presented for the groyne in this variant.

7.2.4 Conventional design

The conventional design of the breakwater, shown in Figure 7.6, has been estimated using the two cross-sections which have been used previously for the breakwater tip and the LNG jetty, respectively Figures L.4 and 7.3. Using the cross-sectional volumes of rock per running meter the estimated volumes of each material has been determined for the complete breakwater length, see Appendix L.

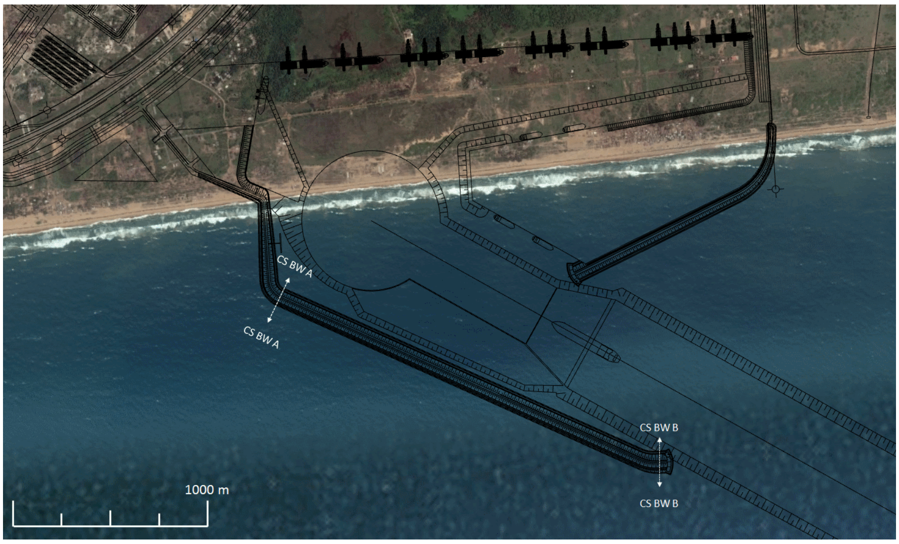


Figure 7.6: Overview of the conventional breakwater design with designated cross-sections along the breakwater.

7.3 Economical cost comparison

In this section the economical cost comparison is presented for all variants and the conventional breakwater design. To accomplish a valid comparison, unit prices are combined with the total estimated volumes for each components, see Appendix L. The results are put into perspective by cost estimations.

7.3.1 Unit prices

7.3.1.1 Rock and Concrete elements

For the economical cost comparison, assumptions for unit prices are made. These unit prices are established based on three criteria:

1. The production costs which refer to either the costs of the quarry/factory
2. Transportation costs of the rocks/elements
3. The placement costs of the specific rocks/elements on site

From a study to conventional rubblemound breakwaters (M. Hauer, 1998) multiple unit prices have been found. Benchmarks within RHDHV have confirmed these numbers. Placement cost is found to be in the range $3 per ton for the placement of core material to $10 per ton for the placement of stones over five tons. Placement costs are subjected to the method of the placement itself. This is especially of impact when regarding larger stone sizes. Whether the stones are placed, through rolling or floating equipment and submerged or emerged impacts the unit price. To be on the conservative side the unit price of floating submerged placement is taken into account. Very large rocks and concrete elements are designed to implement. These rocks and concrete elements require accurate placements and therefore are extra vulnerable to a harsh wave climate. Three times the normal placement cost for material(M. Hauer, 1998) is consequently used in this cost comparison.

Table 7.3: Unit prices

	Cost component				
Material	Placement	Production	Transportation	Total	Unit
Heavy grading	30.00	7.50	20.00	**57.50**	[$/ton]
Secondary armour	22.50	7.50	20.00	**50.00**	[$/ton]
Quarry run and coarse grading	9.00	7.50	12.50	**29.00**	[$/ton]
Granular filter	22.50	7.50	19.88	**49.88**	[$/$m^3$]
Concrete	250.00	200.00	50.00	**500.00**	[$/$m^3$]

In the study (M. Hauer, 1998) an analysis of costs for the opening of a quarry, equipment, blasting, sorting, loading and overhead costs was included. With this analysis the total average production costs were approximated at $ 7,5 per ton irrespective of the volume and size of the different rock classes.

For the transportation of the rocks and accropodes a determination based on weight and distance is executed. In case of the lighter class of rocks (lighter than 700 kilogramme) the transport costs were estimated at $0.125 per ton per kilometre. For this estimation, fuel, equipment, and unloading at the project site is taken into account. For rock sizes heavier than 700 kilogramme the transport costs were estimated at $0.20 per ton per kilometre. The accropodes are more expensive and estimated at a $0.50 per m^3 per kilometre. The accropodes and heavy rock classes require heavy equipment and the transportations are more precise and time consuming, which explains the high costs.

7.3.1.2 Sand suppletion

The unit price with respect to the construction of the sand component of the sand breakwater is determined in a different way than the other unit prices. It is based on assumptions with respect to logistics and method of placement.

The sand suppletion could be executed in multiple ways. It is assumed that the method of pumping sand through pipes is used. As the sand breakwater is a continuing stretch of sand for all variants, it could benefit from a method like pumping sand to the desired location through pipes. The further the sand breakwater is constructed the further the pipes can be placed to transport the sand. During construction multiple extra pumping stations could be used to keep the desired pumping capacity.

Due to the dredging activities in the port and the approach channel, dredging vessels already need to be present. Consequently, it is assumed that little extra mobilisation required for the sand suppletion part. The placement of required material like pipes, extra power pumping stations, machinery to profile, etc. is inevitable. This is yet, relatively low with respect to cost of mobilisation of a dredging vessel in any case. Also, the removal of sand and transporting it to an offshore location will be required in less quantities for the construction of the complete port as much sand can be re-used in all variants. This re-using of sand decreases the cost of sand transportation which could take up much time as the sand should not be dumped close to the port. If it would be dumped close to the port it could be transported back towards the coast after dumping and cause a need for dredging.

Conclusively, the bulk price of pumping, placing and shaping of the profile is estimated to be $ **4.50 /** m^3. This number is based on benchmarks from other projects within RHDHV taken into account the reduced rate due to shared mobilisation costs and less transport activity offshore.

7.3.2 Cost Comparison

For the cost comparison, the material volumes and the corresponding unit prices are separated in several material classes. This division led to the class "heavy grading" concerning rocks over 1000 kg, "Secondary armour" is assumed to be the 500-1000 kg class and the other categories are designated to their own class.

All the variants differ in cost for the reason that the variants are unique and have each their center of gravity on different components. These components have been estimated roughly in the previous subsections. The total overview of cost in comparison to each other is presented in Table 7.4. The variants have been estimated in costs for each variant concerning each component. A more detailed estimation can be found in Section L.2.

Table 7.4: Cost perspective of all variants and coventional design

	Construction cost			
Component	**'Sand Nourishment'**	**'Groyne'**	**'Lagoon'**	**Conventional design**
Sand suppletion (coastal stretch)	$ 192,870,000	$ 120,607,000	$ 105,628,000	-
Sand suppletion (lower dune)	-	-	$ 15,980,000	-
Groyne	-	$ 13,605,000	$ 9,325,000	-
Breakwater (tip)	$ 55,495,000	$ 27,747,000	$ 27,747,000	$ 164,339,000
Breakwater LNG jetty	$ 4,058,000	$ 4,058,000	$ 4,058,000	-
Total cost	$ 252,422,000	$ 166,025,000	$ 162,737,000	$ 164,339,000

Emphasized is that all unit prices are subject to assumptions and benchmarks. In reality these unit prices could turn out quite different due to logistics and local circumstances. The total cost of variants can heaviliy fluctuate due to minor variations in unit prices. Although this cost comparison is based on a lot of assumptions there are multiple clear findings in the cost comparison.

First of all, the variant 'Sand Nourishment', based on the idea that as much sand as possible should be used, is much more expensive than the other variants and also the conventional design. On the one hand, this is a consequence of the large volume of sand required for this variant due to the large amount of coastline development in the modelled 50 years. Consequently, that specific coastline development led to the large required berm width. On the other hand, it was a consequence of the berm width due its requirement for a length of 800 meters for the long side of the L shaped hard tip. This was a consequence of the fact that it required enough space to not conflict the required port basin depth on the back of the dune. This all together shows that the stretch of coast under equilibrium orientation definitely needs to be more morphologically stable and protected against sand redistribution.

Secondly, the rough cost comparison shows that variant 'Groyne' and variant 'Lagoon' are in the same price range. Due to the shorter required length of coastal stretch in variant 'Lagoon' a lower volume of sand is required. However, the lower dune behind the pocket beach nullifies the saved volume by the shorter length. A notion of larger importance, is that the lower dune has been designed to a uniform height over its complete length. This height was determined at the closest point to the original coastline. However, in the shadowzone of the coastal stretch the height of the lower dune could decrease. Especially, since all the accretion will occur in the 'Lagoon'. This would require more precise diffraction and refraction studies to the governing waves approaching this lower dune.

Although the groyne in variant 'Lagoon' is obviously cheaper as it only concerns a third of the length of the groyne in variant 'Groyne', it is not a difference in the same order of magnitude. This is a consequence of the fact that the groyne in variant 'Lagoon' is situated in a deeper part.

Thirdly both variants are in the same price range as the conventional design as well, moreover variant 'Lagoon' is even cheaper than the conventional design.

Due to the sensitivity of the unit price and the relatively small difference in estimated total costs, the variant sand breakwater is considered equally feasible as a conventional breakwater with respect to costs.

7.3.3 Future prospect

In this section the future prospects, for all the variants and the conventional breakwater are discussed. Construction costs have been discussed in the previous chapter. The costs regarding maintenance of the (sand) breakwater and possible opportunities in the future are important to examine for a final conclusion.

7.3.3.1 By-pass of sediment

One of the sub-questions was the determination of the amount of sand by-pass in all conceptual variants. All the variants, however, were created on the boundary condition that no by-pass of sediment occurs during its design life time of 50 years.

Variant 'Sand Nourishment' and 'Groyne' are designed such, that after 50 years the LST capacity on the coastline under equilibrium orientation starts to increase and the by-pass of sand restarts. This will gradually increase when the original coastline has moved beyond the sand breakwater due to accretion.

The storage capacity of variant 'Lagoon' is thought to be large enough to accommodate accretion for well-over the required 50 years. The lagoon which is artificially created behind the pocket beach and next to the lower dune can be filled up with sediment as well. In addition, the pocket beach is shorter than in variant 'Groyne' so even more space is available with respect to variant 'Groyne'.

In Appendix A multiple projects are presented in which a port was constructed on the West-African coast. The port at for example Lomé in Togo was constructed after which west of the port the original coastline started to move offshore due to accretion. In Appendix A in Figure A.3 this development is shown. The original coastline migrated seaward until the tip of the breakwater. After many years the accreted land enables the acquiring of a new part of the port. With a sand breakwater at Badagry a similar opportunity could be established.

Variant 'Lagoon' actually provides this possibility as well. If desirable, the accretion behind the pocket beach and next to the lower dune can be replaced with machinery to the east of the lower dune. This way the maximum amount of available space for accretion is used. In that case, an opportunity just like in Lomé is exploitable in the future. In the current designs the lower dune is a straight line down to the pocket beach as shown in Figure 7.5. If this future prospect is indeed desirable, the lower dune could also be constructed under an angle directed towards the tip of the breakwater in the shape of the conventional breakwater, see Figure 7.7. This way the natural accretion will possibly take place in an increased rate. The accretion behind the pocket beach does rely on the amount of sediment transport that occurs with secondary currents (caused by set-up differences). Further research should be executed to these secondary currents and their ability to cause the sediment transport into the corner of the storage area.

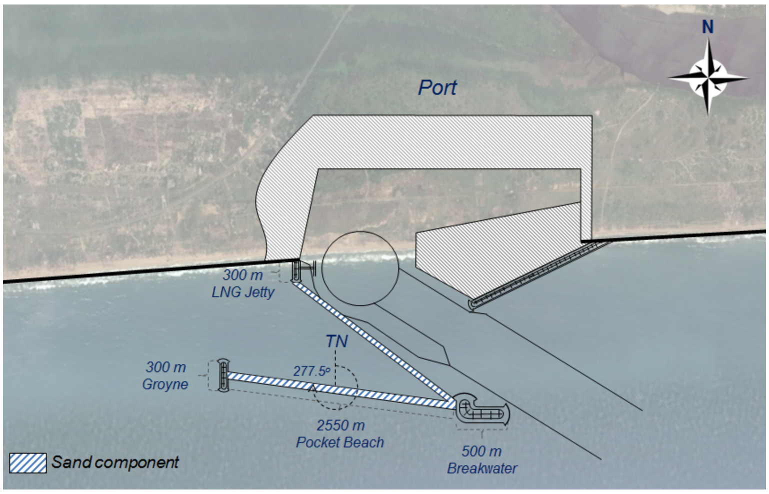


Figure 7.7: Overview of alternative variant 'Lagoon', not to scale

However, to put this future prospect more into perspective. As it takes 50 years before accretion has filled up the space in variant 'Sand Nourishment' and 'Groyne', it will take more time for variant 'Lagoon' to fill up and even more time for the conventional breakwater. In terms of available space for accretion, the conventional breakwater design offers the best solution. The conventional breakwater is yet making it impossible to use the accreted land in the future similarly to Lomé. The existence of the breakwater leads to difficulty with acquiring extra port basin area as it is impossible without having to (partly) remove the conventional breakwater.

7.3.3.2 Maintenance

From a maintenance point of view with respect to the conventional breakwater, only corrective maintenance, after a design storm, can be labelled as large cost drivers. For most of the circumstances and events below that level of impact the maintenance will be relatively low.

The amount of maintenance which is required for a sand breakwater, is more difficult to define. Morphological changes happen on the sand breakwater well below the design event of a conventional breakwater, see Chapter 6. The impact of a 1/1 year storm is elaborated on in Chapter 6 Subsection 6.4.2. Obviously, storms with shorter return periods occur more often and therefore maintenance is required more often than for a conventional breakwater. A margin for the berm width has already been taken into account for consecutive storms in the conceptual designs so the breakwater will survive the storms but it does require corrective maintenance.

Expected maintenance costs based on this 1/1 year storm are enormous when taken into account the storm reduction volume of the profile on the orginal coastline. The storm reduction in that case amounts to 250 m^3/m. The storm reduction for a 1/1 year storm on the profile on the equilibrium coastline amounts to about 200 m^3/m. This is similar to a reduction of approximately 20 meters of berm width, see Table 6.9. To place that in perspective it means that the berm width is reduced with 25%, see Table 7.2. The repairworks for the variant 'Groyne' would require a volume of 0.67 million m^3 of sand and for variant 'Lagoon' 0.51 million m^3 of sand. Even if all the sand is on the foreshore and still re-usable in maintenance this is rather costly. In Chapter 6 Subsection 6.4.2 is also stated that the effect of storms with shorter return periods and the effect of normal conditions is expected to be heavily overestimated by XBeach, given the present beach bathymetry at Badagry. Due to this fact, it is expected that less than 200 m^3/m storm reduction will take place for a 1/1 year storm and should certainly not be expected as the amount required for maintenance. In Chapter 8 the reliability of the XBeach results are discussed. The impact that further research might have on the topic of storm impact is discussed as well. Conclusively, can be stated that with respect to the conventional design, it is expected that more maintenance works need to be executed with a sand breakwater but further research is required for defining this quantity.

Loss of sediment If storms occur and cross-shore redistribution of sand takes place alongshore sediment transport could cause a loss of sediment. The deposited volume of sand on the foreshore in the cross-shore profile could be transported with LST. This would make repair works after a storm more expensive. This phenomenon only occurs when there is a transport capacity either eastwards or westwards.

Profile one is orientated in the main direction of swell events. As presented in Chapter 5 there is little LST along the coastline which is constructed under equilibrium orientation and for that reason the loss of sediment through alongshore currents will be very small as long as this orientation is present. In variants 'Groyne' and 'Lagoon' this is the case during the design lifetime. Variant 'Sand Nourishment' is slightly different due to a developing coastline. Along the stretch of coast placed under 60 degrees w.r.t. the original coastline profile one is present. This profile has a different orientation. LST does occur along variant 'Sand Nourishment' and in the case of a storm the deposited volume could erode due to LST westwards. Although erosion might occur, the fact is that the orientation of the coastline in variant 'Sand Nourishment' tends to turn to equilibrium orientation. The more it turns to this orientation the less LST capacity is present. Therefore, the scenario of an eroding storm deposited volume is only possible in the first couple of years. After 10 years the LST capacity is less then 300.000 m^3/year, see Figure 5.4. This LST is over a factor of three lower than the present LST capacity along the coast of Badagry. Conclusively can be said that the chance of loss of sediment is a disadvantage of only the variant 'Sand Nourishment' and is only likely to be of large impact in the first years after construction.

7.3.4 Building with Nature

Since a sand breakwater integrates Building with Nature values in its design, this could play a keyrole in the economical considerations. It is impossible to quantify this aspect but it certainly is an important aspect in the comparison to a conventional breakwater. As stated in Chapter 1, the idea of Building with Nature is to make use of the dynamics of the natural environment and provide opportunities for natural processes. When (re-)using a natural component like sand to the benefit of coastal protection works and integrating the dynamics of the natural environment this philosophy is perfectly accomplished. The dynamics of the natural environment are integrated by using the equilibrium orientation of the coast and the accretion of sand in the 'Lagoon' which later on can be used to acquire land.

8 Discussion

8.1 Introduction

In this chapter the liability of multiple aspects and the most important assumptions on which the study is based are discussed. Aspects of liability of both two models are elaborated on. The discussion starts about the calibration of the model LITPACK, see Section 8.2. In addition to this calibration also two sensitivities of LITPACK are discussed, see Section 8.3. This section is followed by Section 8.4 in which the limited applicability of XBeach is discussed. Lastly, a discussion about modelling method of extreme events is presented, see Section 8.5.

8.2 Realistical calibration LITPACK

In the process of setting-up LITPACK and calibrating the model, three parameters were used to gain a sufficiently calibrated model.

- Beach height
- Active depth
- Dune height

Since, the model LITPACK was overestimating the modelled erosion in a large quantities it was necessary to compensate this effect. This was conducted by adding an artificial dune line, 50 meter behind the coastline with a dune of 10 meter. In reality there is no dune present at the coast of Badagry. Also, the active depth and beach height which form together the active profile of the complete profile were calibrated. The active profile indicates what part of the profile is subjected to erosion or accretion. These parameters were set at respectively 12 and 5 meter which are somewhat higher than the realistic values, however, not significantly.

In addition, the calibration of the model is executed using aerial pictures of a stranded tanker. In absence of accurate data with respect to coastal evolution close to the area of interest this stranded tanker was the only way the model could be calibrated. This stranded tanker functioned in the first years as an offshore breakwater, still letting sand pass by between the tanker and the coastline. The calibration and modelling of the tanker however was done by using a groyne which is impermeable from the start. It can be found questionable to what extent the effect of this start-up has been integrated well enough in the calibration.

Due to these two aspects it can be questioned to what extent this calibration is found realistic even though the rough results are found to be similar to the reality. To optimise this calibration, a sensitivity analysis could be done for the three parameters through which more insight is gained in the impact of these parameters. Next to this, a situation in which no 'start-up effects' like a tombolo are present, for example calibrate it on one of the recently placed groyne field close to Lagos. This way satellite imagery will give a better insight in the coastal development due to a coastal structure and is a more accurate calibration possible.

8.3 Sensitivity LITPACK

8.3.1 Spreading factor

In the process of setting up the LITPACK model the spreading factor for the wave climate was determined. An analysis of the sensitivity is presented in Subsection 5.5.3. As shown in this Subsection the spreading factor is a factor which is highly empirically determined. The calibration of LITPACK with respect to littoral transport is subjected to this parameter. As stated before its effect is significant. The difference in LST with factor 0.6 or 0.8 instead of 0.7 is determined to be 20%. In the analysis to the spreading factor has been assumed that 0.7 is taken to be well representing the sea state of Nigeria taken in mind that this sea is very uni-directional. However, it is not possible to guarantee that 0.7 is the most accurate fit for the sea state. It could be the fact that for example 0.8 an even better fit for a sea state. Further research to the spreading factor should be executed for a better comprehension in the search of a more accurate fit.

8.3.2 Grain size

In addition the sensitivity was determined of the grain size. Even though much data was available for this research, not all data was very reliable. The soil data used in this research concerned grab samples of the top layer of the soil at the coast of Badagry. These superficial soil samples do not gain an accurate comprehension of the subsoil. Only four rows of grab samples are acquired along the coast of Badagry located partly at the location of the sand breakwater.

The profile which has been used in LITPACK was integrated with extrapolated values of location specific grain diameters. This integration is found important to account for the fact that in the swash zone, close to the shoreline, significantly smaller grain sizes are present than in the other regions. As described in the sensitivity analysis to grain size differentiation, Subsection 5.5.2, accretion and erosion of the coastline are impacted significantly by the grain size. The **modelled erosion** is assumed to be **conservative** as the grain size with which the sand breakwater is assumed to be created is assumed to be at least 300 µm. However, the deviation of grain size in the profile used in LITPACK in the swash zone, could be significant and with that the occurring LST could increase. This could on its turn lead to **higher accretion rates**.

To conclude, it is questionable to what extent the location specific grain size is adopted in the LITPACK model sufficiently. Further research to subsoil and acquiring of data of grain sizes is required. Through this approach a more accurate integration of grain sizes in the profile can be applied and with this a better realisation of the modelling of LST. Consequently, its coastal development is modelled more realistically.

8.4 Limited applicability XBeach

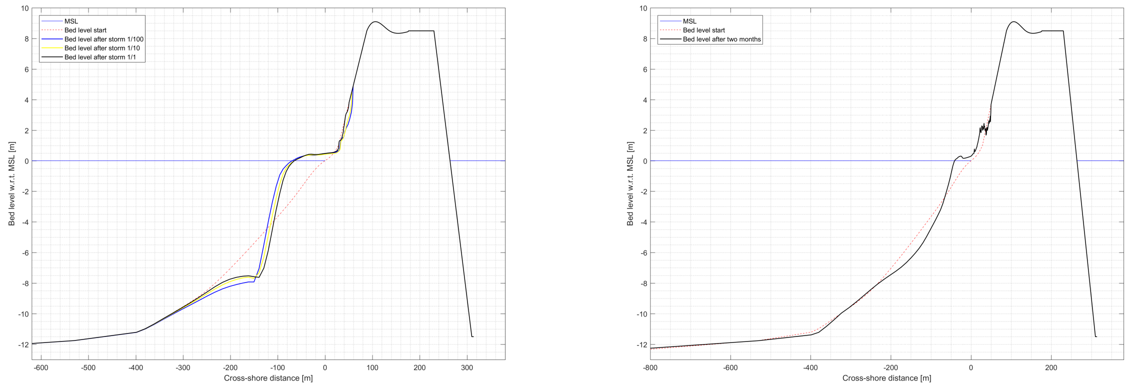
In the introduction, Subsection 6.3.1, was stated that the modelling of normal conditions was unsuccessful with XBeach. The profile which is forced by XBeach with 'normal' or average conditions is far off the measured profile as it currently is present at Badagry. The forcing keeps eroding the profile throughout time. No beach restoration by the waves itself seems to appear. In addition, the quantities of storm reductions with a 1/1 year storm and a 1/10 year are significantly large and exceed more than 80% of the 1/100 year storm, see Section 6.4.

Throughout this research XBeach has been used with the default settings. XBeach is however a modelling tool designed and calibrated for the Dutch and North American coasts. In comparison to these coasts with a gentle slope and small grain size, the coast at Badagry has a very steep slope with a large grain size. Formulations within XBeach are based on the effects of wave interaction on gentle slopes and small grain sizes subjected to wave climates known to be prevailing at the Northern American coast and the Dutch coast.

This all together, leads to the assumption that the storm reduction is heavily overestimated. This assumption is backed up by the lack of modelled transport directed onshore by the waves which is certainly present at Badagry. This philosophy is supported by new runs which have been executed with adapted settings to approach specified settings for the Nigerian coast. In these setting the effect of the explicit wave run-up of short waves and swash physics is integrated differently for Badagry. The following changes have been made:

- The transport caused by wave asymmetry is increased and in compensation for the increased transport in general, the transport by wave skewness is decreased.
- A different transport formula is inserted which is more sensitive to the grain size diameter with which the transport is lower for larger grain sizes.

These changes caused the large storm reduction which XBeach showed before to change. The effect of these 'Nigerian specific settings' are shown in Figure 8.1(a) for all three storm conditions. The results are gained for the same profile and scenario as previous runs. The onshore transport is much more than before by using these changed settings. There needs to be emphasized that these adapted settings are not known to be the correct settings but they do show a large difference in profile change due to the added onshore directed transport. This phenomenon is key in the discussion that the storm reduction volumes of the default settings are overestimated.



(a) Storm conditions) (RoyalHaskoningDHV, 2014c)

(b) 'Normal' conditions

Figure 8.1: XBeach results of different conditions with 'Nigerian specific settings'

The effect of these new Nigeria specified settings for normal conditions is shown in Figure 8.1(b). Although these settings show the normal conditions to be more or less around the measured profile, **it is obviously not possible to state that these settings are the correct settings**. A thorough calibration for the Nigerian coast would be required to execute in order to be sure that the results of the XBeach model are accurate. This calibration would require much profile data of the coast before and after storms/events.

This acquiring of new data is a rather expensive activity but on the other hand if for example the results shown in Figure 8.1(a) are examined to their result in economical benefit, the profit is significantly. The storm reduction of the storms does not even reach the top of the dune. If it would be possible to reduce the width of the berm of the sand breakwater this would mean a reduction in required volume of sand. A reduction of 25 meters would lead to a quantity of 2.20 and 1.95 million cubic meters for respectively variant 'Groyne' and 'Lagoon' can be saved. With that the cost of the breakwater would be reduced with $ 10 million and $ 9 million for respectively variant 'Groyne' and 'Lagoon'. In addition, the costs of the groyne and breakwater tip can be reduced due to the smaller berm width. Finally the aspect of maintenance would, taken the results of the new settings into account be much more feasible as the storm impact of a 1/1 year storm is significantly lower. This taken into consideration leads to the sand breakwater turning out to be a more economically profitable option.

The most important assumption is, seen the results and the output of XBeach that the construction as it is designed in the previous chapters, based on the results of these storm outputs, it is designed **conservatively**.

8.5 Modelling method XBeach

In this study the design criteria for the sand breakwater concerning overtopping regarded the 1/100 year storm condition. For the determination of the berm width the design criteria was determined to be that the sand breakwater was required to resist the storm reduction of a 1/100 year storm expanded with the impact of consecutive 1/1 year storms. The storms in this study were created according to normal storm definitions as was executed in earlier studies by RHDHV and described at Subsection 6.3.2. The extreme values used in the determination of the design storms are actually not storm related extreme values but concern extreme swell events, see 3.3.7. Storms are events in which wind is the driving force causing the wave height to rise and in the middle of the storm, when the wind speed is at his maximum, to lead to high wave heights. Storms are actually quite rare on the Nigerian coast and not governing for morphological impact. These swell events of which the extreme values are found are rather found in wave trains which are groups of swell waves. The occurrence of wave trains is visible in Figure 8.2 in which the wave train is shown in red for its energy density.

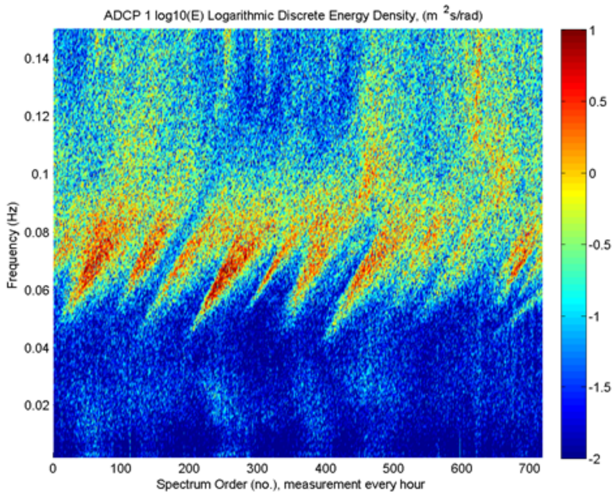
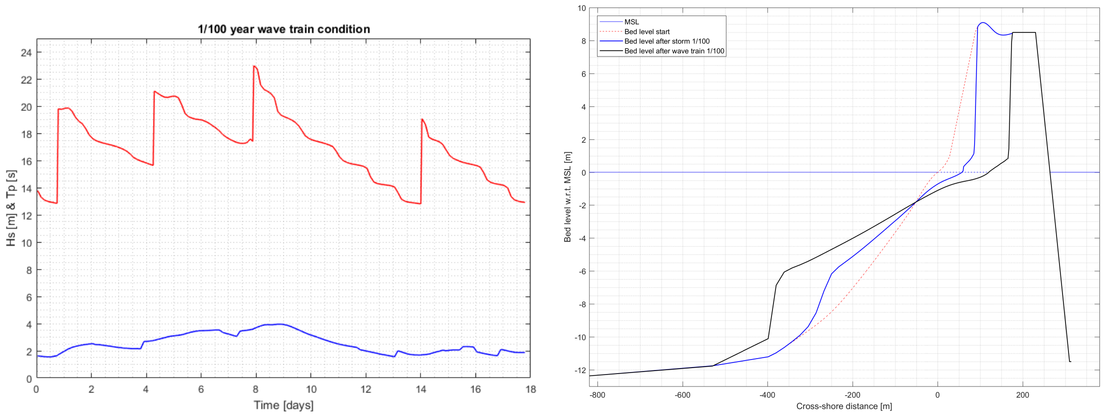


Figure 8.2: Occurrence of wave trains shown in a wave spectra measured at ADCP1 location, (RoyalHaskoningDHV, 2014a)

One could argue how to determine in a case a design conditions or criteria which suits the wave climate and occurring extreme events in a better way. Storms are described in XBeach by a JONSWAP spectrum. Yet a JONSWAP spectrum is based on the assumption that it concerns a developing sea state, which is typically for storms. At the Nigerian coasts this is not the sea state which prevails and waves are already fully developed after the long distance that they travelled over. The input which actually would match the wave forcing at Nigeria regards not a storm which builds up from normal conditions and prevails for 44 hours. It should regard an occurrence of a wave train. Possibly the occurrence of a wavetrain extrapolated to the 1/100 year swell event would be a better fit. In Figure 8.3(a) the wavetrain with a maximum wave height of 3.95 meter is shown. The wave train is easily recognised by the grouping of the wave periods which cause the sawtooth shape in the figure. This set-up for a input for XBeach could be a sufficient design criteria for the Nigerian coast. Instead of a JONSWAP spectrum, the input in XBeach should be computed with a Gaussian swell spectrum. In Appendix C Figure C.10 the gaussian fit for swell has been shown. The morphological effect of a wave train on the coast of Nigeria are significantly different than the morphological effect of storms, see Figure 8.3(b). For this reason, further research is recommended in order to find the correct forcing for extreme events for a sand breakwater on the Nigerian coast.



(a) Possible set-up of wave train 1/100 year condition (RoyalHaskoningDHV, 2014c)

(b) Comparison of wave train 1/100 year condition with default XBeach setttings and storm conditions 1/100 year with default XBeach setttings

Figure 8.3: Graphical presentation of wave trains and impact on dune

9

Conclusions and Recommendations

9.1 Conclusions

9.1.1 Introduction

In this master thesis the main research objective is:

"The determination of the feasibility of a 'Sand Breakwater' on the Nigerian coastline at Badagry"

The feasibility of a sand breakwater was researched through two components which in the conclusions are presented similarly. A sand breakwater is found to be feasible when it satisfies both components. In this Chapter phase 3 of the research is presented, see Section 2.3.

9.1.2 Morphological feasibility

The first component being the feasibility in a morphological sense with which is meant the morphological stability of a sand breakwater. The morphological feasibility is be presented through concluding on the five sub-questions.

What is the equilibrium coastline orientation for the prevailing wave climate? The uni-directional character of the wave climate prevailing at the coast of Badagry forces an equilibrium orientation of 277.5° w.r.t. TN, see Section 5.2.

Which parts alongshore of a 'Sand Breakwater' are most critical and vulnerable to erosion and how to cope with these vulnerabilities? Due to the harsh uni-directional wave climate all other orientations of coast than the equilibrium orientation are vulnerable and will develop to the equilibrium orientation over time. In all three conceptual variants other solutions to this vulnerability are applied. However, in all variants, a hard structured tip is implemented. This was necessary to prevent the sand breakwater from eroding into the port and approach channel.

In addition, the equilibrium orientation of coastline is found to be morphologically stable whenever preserved. This orientation is integrated in the conceptual variants led to results of LITPACK which support the morphological stability of the sand breakwater with respect to long term coastline evolution for all variants. The equilibrium orientation is in variant ´Sand Nourishment´ applied but is designed such that when it loses this orientation it can withstand the occurring coastal development. Variant ´Groyne´ and ´Lagoon´ integrate the orientation and rely on the fact that this orientation is preserved. Therefore these variants are less vulnerable to erosion due to LST.

What is the amount of sand by-passing the sand breakwater compared to the conventional design? Constructing a conventional breakwater is the best option when looking solely at the period in which no sand by-pass would occur. This is the fact due to the maximised available space for accretion.

Variant 'Sand Nourishment' and 'Groyne' are designed such that they provide space for accretion for the economical design life time of the port which is 50 years. After this period accretion will start to occur in that amount that the shape of the coastline orientation passes the equilibrium orientation and LST occur eastwards. Due to this LST, sand by-pass will occur.

Variant 'Lagoon' offers space for accretion for 50 years but as well after that period for a large amount of years. The 'lagoon' part in this variant can be filled up with sediment. When the lower dune in this variant would be constructed differently as shown in Figure 7.7 this aspect would be maximised. In that case the variant offers a period without sand by-passing which is in the same order as the conventional design, see Section 7.3.3.1.

What is the impact of the Sand Breakwater on the coastline west of the port? The coastline development west of the port concerns a maximum 20 meter of coastline regression over 15 kilometres West of the port. This small coastline regression will 38 years after construction change to accretion, see Section 5.4. Beyond the 15 kilometres no negative impact on the coast is found. Due to these findings, the coastline development due to the construction of the Sand Breakwater west of the port is small and considered within acceptable boundaries.

The impact on the coast east of the port was not in the scope of this study, see Section 5.4.

What is the stability of the design under storm conditions? The modelling results of XBeach led to a design berm width for the sand breakwater. These results combined with the design crest height led to an minimal required profiles. All variants are integrated with these profile. These profiles are found to be satisfying the design criteria. The design storm conditions were based on the impact of a storms with a 1/100 year return period along with an expansion for a consecutive storm with a return period of 1/1 year.

The resulting profile of XBeach which is forced by the 'normal conditions' is not representable for the present profile existing at coast of Badagry. Also the impact of lower storm conditions are not showing representable results. This design is concluded to be conservative for the fact that the storm reduction is taken to be overestimated.

9.1.3 Economical feasibility

The second component is the feasibility in an economical sense. This feasibility was determined by comparing the conceptual variants to the conventional breakwater design.

What is the estimated cost in comparison to a traditional breakwater? First of all, the question whether a sand breakwater created with almost only sand is feasible (variant 'Sand Nourishment'), is proven to be not the case. The enormous costs of the placement of the sand leads to this option being too expensive. In addition, loss of sediment is possible to occur during storms due to LST.

Secondly, the estimated construction costs of a sand breakwater in a more hybrid way as for variant 'Groyne' and 'Lagoon' are in the same range as the construction costs of a conventional breakwater. Variant 'Lagoon' even showed to be a more profitable solution than the conventional breakwater.

What is the future prospect of a sand breakwater in comparison to a conventional breakwater? The XBeach results of lower storm conditions showed that maintenance costs are high. However, these results are assumed to be heavily overestimated. Results with more specific Nigerian conditions create reason to believe that maintenance for lower storm conditions will be much lower and in acceptable range.

Since the design lifetime of the conventional breakwater was set to be 50 years. It was assumed to be the same lifetime with respect to the sand breakwater. Coastal development is mitigated in a way that all variants can suffice 50 years of accretion but variant 'Lagoon' for a much longer period. An analysis of a sensitivity to grain size showed that significantly more accretion occurs with lower grain sizes, see Subsection 5.5.2. In addition the sensitivity to spreading factor is large. Due to these sensitivities, accretion could occur in a higher rate at the sand breakwater then modelled.

Obviously when the port will be constructed, it will be used for more than 50 years but this period of 50 years is only a matter of economical worth which is designated to the breakwater. Variant 'Lagoon' has by far the most future prospect taken into account the storage space for accretion of sand. In addition, the future prospect of possible land acquiring for new port space is a surplus to variant 'Lagoon' and makes it certainly the most optimal variant.

The value of a Building with Nature component is present in this project and could enhance its economical interest.

9.1.4 Final conclusion

The single most important conclusion to be drawn from the previous conclusions, is that the answer on the question: "Is a sand breakwater feasible?" is proven to be, "Yes.". Variant 'Lagoon' is morphological stable long term and short term with respect to storms.

This variant is in the same cost range as the conventional breakwater and shows good future prospect. New settings of XBeach are even showing a promising result which give reason to believe that a sand breakwater can be more profitable than the conventional breakwater.

9.2 Recommendations

During this research multiple factors have been pointed out which are key when moving forward with subsequent research to this opportunity. This section suggests recommendations to remove uncertainties and increase the affirmation of the construction of a sand breakwater. The main key recommendations are presented below.

Key recommendations

1. At the moment, as stated in the discussion, cross-shore transport is modelled with default settings of XBeach which do not represent the specifics of the coastal system of Nigeria. Further research is required to gain insight in specific settings for this region. A calibration of the model by using pre- and post event(storm/wavetrain) profile data of the coast of Badagry will lead to an improved set-up. With this improved set-up also 'normal conditions' should be able to model on the coast of Badagry. This way measured profile data could be confirmed and in that case event(storm/wavetrain) results would be more relying.

2. Further research to the type of forcing for the design event(s) in the model is also recommended. As discussed in the Chapter 8 possibly the phenomenon of wave trains fits the wave climate of Nigeria in a more suitable way in perspective of extreme events.

3. The complete transition between hard structure and sand component of the sand breakwater is a subject which is not looked into in this study. As the weakest link is the governing in a chain, the connection of those two components is of extreme importance.

Lesser recommendations In addition to the main recommendations other smaller recommendations are presented below. These recommendations would increase the accuracy of the research and with that the determination of the feasibility of the sand breakwater. However, these recommendations are not fundamental for the main objective.

- In the scope of this research the effect of non-linear waves is not taken into account. In reality non-linear waves are present in the winter in Nigeria. The morphological effect of these waves is recommended to research.
- For determination of the design crest height for the sand breakwater an approach has been used which is not standard in dune design. Further research is recommended such that the establishing of a design crest height is done more accurately.
- A fundamental part of the feasibility of a sand breakwater is also execution. This part is neglected in this study but of vital importance. The practical side of the concept is highly intertwined with the theoretical part as the location of Badagry is a challenging environment to construct such a design. Due to this practical challenge it is also necessary to gain insight in the behaviour of different execution phases of the sand breakwater. The baseline would be to determine the most vulnerable aspects of the construction process of a sand breakwater are and how to cope with these vulnerabilities.
- The execution of the sand breakwater has much cohesion with unit prices. In this study all unit prices are based on assumptions. In reality in Badagry these prices could differ easily due to for example heavier conditions in terms of placement of the material. Detailed research is required for solid numbers for unit prices.
- The estimation of available sand for the construction of a sand breakwater amounted to 50-60 million cubic meters of sand. Firstly, this is a rough estimation. Secondly, due to lack of soil data about the subsoil it is hard to define whether the total quantity of sand concerns usable sand with a large enough grain size. Therefore, further research should be executed to make sure there is enough usable sand. Lastly, further research is required to determine that the soil is not contaminated.
- In this study only rough required volume estimations are done especially with respect to the hard components of the sand breakwater. Future research to detailed design should give a good insight in the exact required volumes.
- The estimated extra required quantity of sand due to sediment loss, profiling of the sand breakwater and settling, taken to be a fourth of the total, is a large assumption. Detailed research should be executed to the execution part of the construction of the sand breakwater to determine the loss of sediment.
- The design presented for the different variants was roughly made, based on various cross-sections. The outer edges of the sand breakwater however, require more research. Sediment transport could possibly occur at the toe of the sand breakwater around the transition of the sand breakwater and breakwater tip. Further research to this toe is required.
- The impact of current contradiction around the tip(s) of the sand breakwater is not defined accurately. More research will give a better insight in the required length of the protection of the coastal stretch constructed under equilibrium orientation. Next to this current contradiction also the impact of the Guinea Current, see Appendix B, needs further looking into as it can be of significant difference with respect to its impact currently on the coast of Badagry.
- Other impacts of currents which are necessary to determine are the secondary currents in the lagoon of variant 'Lagoon'. These currents are assumed to be able to transport sediment into the lagoon. With this sediment transport new land can be acquired but to what extent these currents have definite impact is unclear and needs further attention.
- This study only researches the morphological effect of waves and currents on the sand breakwater. The aeolian transport caused by for example squalls could be of significant effect when the sand breakwater is not (yet) protected with vegetation. It is quite likely that is necessary to place vegetation on the sand breakwater to protect the sand breakwater against this phenomenon.

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Appendices

A
Reference projects

Reference projects with respect to the subject sand breakwater are not known to be present. However, there has been done a lot of research to the coast of Nigeria and its developments as a consequence on interferences by humankind. These interferences are usefull for a complete understanding of the Nigerian coastline and its processes. By having a good understanding of the coastal development, correct assumptions and estimations can be made in the study to the reaction of the coast on new breakwater designs. Bentum et al. (2011) has extensively researched the historical coastal development of the Lagos coastline which covers the history of the coastal development of the Badagry coastline as well. Only the major coastal developments are looked into in this study.

Major projects or developments are discussed considering the coastline starting at the Volta river located in Ghana(260 km west of Badagry), until the city of Lagos located in Nigeria (50 km east of Badagry). Bentum et al. (2011) found LST volumes with UNIBEST-CL matching the literature quite well (500,000 m^3 - 1,000,000 m^3) concluding an LST volume of 600,000 m^3 was used for her research. In Royal HaskoningDHV reports often 700,000 m^3 is used.

A.1 Ghana - Volta

Located in Ghana lies Lake Volta, see Figure A.1. It is about 250 km distanced to the east of Badagry. Lake Volta ends up in the ocean in the Gulf of Guinea through a delta called the "Volta Delta". This delta is a big source for the LST along the West African coast for the coasts until even West Nigeria (Anthony and Blivi, 1999). Within this delta is the "Keta Lagoon" located. As the delta is a source of sediment the coastline of this delta has a transient in characteristics. The west of the delta has a rocky coastline as in the east beaches are found.

The volta river roots in lake Volta and ends up in the Keta lagoon. In this river the Akosombo dam was built in 1965. This dam has as main purpose the production of hydropower. It is located somewhat 60 km upstream of the river mouth (Anthony and Blivi, 1999). When this dam was built a decrease in sediment supply to the coast occurred. Allersma and Tilmans (1993) estimate that the Akosombo Dam led to a reduction of the sediment amount supplied to the coast by the Volta River to sixty per cent of its original amount. This reduction in sediment supply was caused by two phenomena. First of all the settlement of sediment in the water reservoir upstream of the dam. Secondly the transport capacity of the river was reduced as a consequence of a regulated discharge (P. R. Jakobsen, 1989). This reduction of sediment supply to the coast led to a big coastline retreat by severe erosion at the Keta village next to the coast. This coastline retreat was at some places measured to lengths of almost one km (Anthony and Blivi, 1999).

The question whether the sediment reduction of the volta river is definining the coastal system at Badagry is highly disputed. The sediment supply of the river is stated to be a source of the coastal system of West Africa however P. R. Jakobsen (1989) states that the impact of the construction of the dam is counteracted by the erosion at the Keta village south east of the Volta Delta. This erosion could counteract the sediment supply completely possibly for the littoral drift.

Figure A.1: Volta Delta (Google Earth, 2017)

A.2 Togo - Lomé

In 1967 the Lomé Harbour in Togo was inaugurated. The city lies about 200 km west of Badagry in the country Togo. The breakwater which was constructed to protect the harbour from incoming waves, is placed almost 1.5 km into the ocean. This obviously has led to a huge coastal impact visible over multiple kilometres. When the breakwater was constructed it (partly) blocked the littoral drift and meanwhile for over one kilometre of shoreline on the west side updrift of this breakwater has accreted, see Figure A.2. Because of this huge accretion on the west side, there must have been an absence of sediment on the east side. This is clearly noticable as Figure A.3 and A.4 show clearly erosion of the shoreline on the east of the harbour.

Figure A.2: Port of Lomé in 2006 (Google Earth, 2017)

Figure A.3: Port of Lomé in 2017 (Google Earth, 2017)

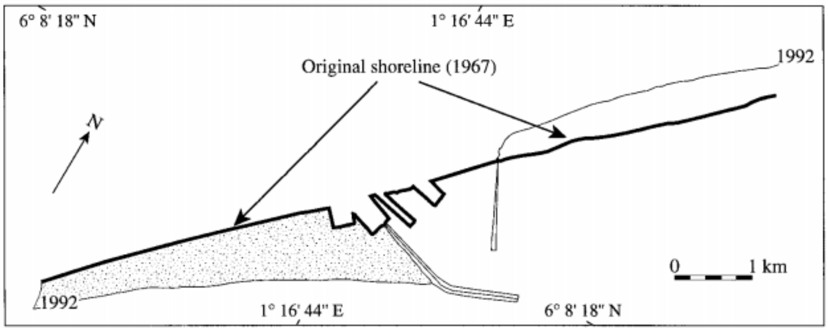


Figure A.4: Coastal development of Lomé (Anthony and Blivi, 1999)

To protect near by villages as Ane´ho and a nearby phosphate export facility measures have been taken. To cope with the erosion, an expensive groyne field has been placed along the coast however it is merely a matter of delay of execution as the coast of West Benin as well is facing the consequences for this project, see Figure A.5. On this coast exacerbated erosion rates are present according to Anthony and Blivi (1999). Impact of the harbour of Lomé is not reaching the scope of Badagry. The coastal development did not only have negative impact for the local surrounding. The harbour adapted throughout the years to the changing coastline as an extra harbour basin was created in the land which accreted from 1967 to 2013. In 2011 the accretion had reached such quantities that the tip of the breakwater was reached and the volume of sand that bypassed had become too large. As a result the port authority had created an extra T-groyne on the big breakwater as is shown in Figure A.3.

Figure A.5: Groyne field in east Togo east of Lomé (Google Earth, 2017)

A.3 Benin - Cotonou

Further up the coast in Benin is a city called Cotonou located. The city is distanced 50 km west of Badagry. Next to the fact that this place is subjected to heavy erosion and a withdrawing coastline because of the big incoming waves, also the coastal development due to the construction of the deep see harbour of Cotonou is significant in this area. The coastline to the east of Cotonou has moved back 400 metres (yards) in 40 years which amounts to an annual retreat of 10 metres (AFP, 2008). The retreat of the coastline can clearly be seen in the Figures A.6 and Figures A.7. A SLR of more than 50 centimetres at the end of the century is expected. According to the French firm SOGREAH-Laboratoire DEFT the coastline will lie 950 metres inland compared to the situation in 1963 when the harbour was just constructed.

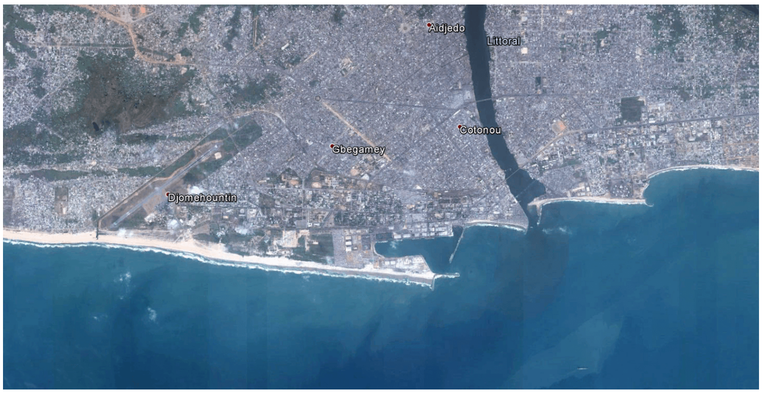


Figure A.6: Harbour of Cotonou in 2002(Google Earth, 2017)

Figure A.7: Harbour of Cotonou in 2017(Google Earth, 2017)

The coastal development due to the construction of the harbour in the 1960's has led to accretion of sediment west of the harbour on the updrift side and erosion east of the harbour. The harbour's western groyne was trapped with sediment almost all the way to the tip of the groyne. Because of this amount of accreted sediment, a larger quantity of sediment bypassed the groyne. In 2011 an extension of this groyne has been executed as shown in Figure A.7. Already in the 1960's, two additional groynes were constructed downstream the harbour breakwaters to protect the area east of the harbour against erosion. Maximum erosion rates of 10 – 15 m/year have been measured (Bentum et al., 2011). Roughly 600 m has eroded downstream of the second breakwater between 1962 and 1993 according toTilmans (1993) Also according to Tilmans (1993) the volume of LST at Cotonou is roughly 1.25 m^3/year and the mean angle of wave incidence relative to the orientation of the coastline at Cotonou is approximately 22°.

The question whether Cotonou is of influence on our scope locally at Badagry is already answered by Bentum et al. (2011). In her thesis research showed that the area of effect of this harbour does not have an effect noticeable in Lagos in 2020. In 2020, the effect of the Cotonou harbour only extends to 22 km downstream from Cotonou. Assumed is that the effect of the harbour of Cotonou does not need to be taken into account as Badagry is located over 50 km downstream,

A.4 Nigeria - Lagos

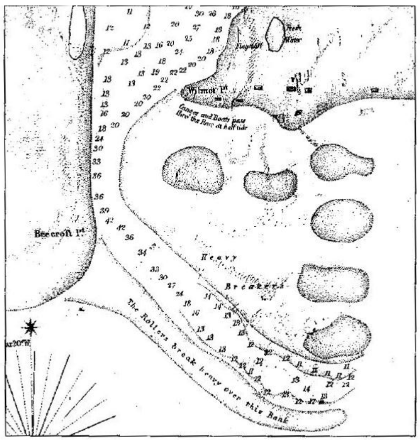
A.4.1 Sand mining

The coastline at Badagry is not densely populated or used for industry at the moment. The area in between Lagos and Badagry however is used a lot for sandmining activities. This sandmining is often illegal but this does not withhold many inhabitants to make a living off this service. This sandmining occurs in the Lagos lagoon but also in the large pockets close to the ocean which is shown in Figure A.8. The inland creek runs at some locations very close behind the beach line, at some places no more than a few hundred meters. At places where the beach is narrow, sand mining in the creek results in deepening of the creek. This causes the banks of the creek to collapse and with this the distance to the beach is reduced. This process of coastline retreat is relatively slow though.

Figure A.8: Sandmining activities close to Badagry (Google Earth, 2017)

The moles at the entrance of the Lagos harbour have been constructed in the early nineteen-hundreds. In Figure A.9(a) a map is shown which displays the planned locations of the moles. These locations show resemblance to the depth chart of the area which is shown in Figure A.9(b). The 'West Mole' lies precisely above the bar in front of the approach channel. The 'East mole' was designed to be constructed on the other side of the 'approach channel' on the shallow parts of the ebb-tidal delta as is clearly seen in Figure A.9(b).

(a) British plan for locations of the three Lagos Harbour Moles (Coode and Son, 1898)



(b) Shallow area near Lagos Harbour entrance in 1858 (RoyalHaskoning, 2001)

Figure A.9: The Lagos Moles

Of all three moles, the first one to be constructed was the 'East Mole' which started in 1908. When this mole was constructed it directly started to catch sediment. This happened because the eastward alongshore sediment transport was (partly) blocked. This accretion of sediment led to the formation of one large spit out of several shallow shoals of the ebb-tidal delta.

After the construction of the 'West Mole' sediment which came from the west with the alongshore transport got trapped updrift of the West Mole. The width of the beaches updrift of this mole increased significantly. This sediment accumulation led to the ebb-tidal delta functioning as source of sediment for the alongshore current. Therefore it slowly started to erode.

As last of the three moles, the 'Training Mole' was constructed, this mole resulted in an accretion of sediment on the inside of the channel into the Lagos Lagoon at Tarkwa Bay. This Bay is 500 m wide and accreted for around 0.5 km. This accretion probably took place because of the result of sediment transport directed into the lagoon by the flood tides (RoyalHaskoning, 2001).

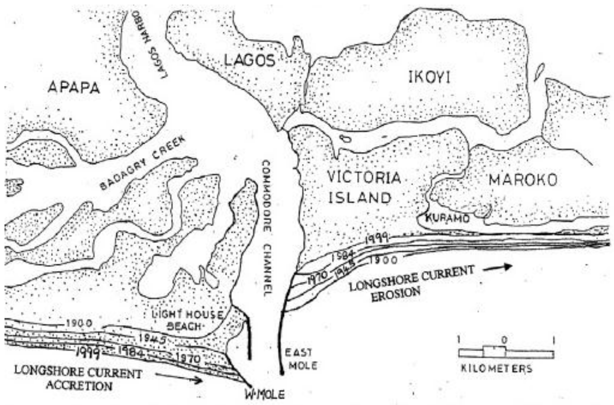


Figure A.10: Accretion and erosion of the Lagos coast, between 1900 and 1999 (RoyalHaskoning, 2011a)

The ebb-tidal delta which was eroding away slowly was not the only consequence to the sediment transport locally. The alongshore current in front of "Bar Beach" to the east of the East Mole had a certain transport capacity and caused littoral drift, but barely contained any sediment. This caused erosion at Bar Beach. Next to this effect also local eddies behind the East mole played their part in the in increased erosion. The eddies led to sediment suspension by an increase in turbulence.

A.4.1.1 Morphology Updrift of West Mole

Between 1910 and 1960 The following rates of accretion of the Lighthouse Beach, between 1910 and 1960, were retrieved by Bentum et al. (2011):

- 440 m directly west of the West Mole
- 340 m at 2 km upstream the West Mole
- 280 m at 5 km upstream the West Mole

The amount over 50 years accreted volume came down to an accreted volume of $20 * 10^6 m^3$. This translated to annual accretion volumes leads to approximately 400,000 $m^3/year$. According to Bentum et al. (2011) is the total sediment alongshore transport about 600,000 $m^3/year$. The Lagos Moles can be schematised as a sink in the LST of 66%.

Between 1960 and 2017

- 280 m directly west of the West Mole
- 280 m at 2 km upstream the West Mole
- 240 m at 5 km upstream the West Mole

If this amount is summed up to the amount of accretion between 1910 and 1960 the aggregate amounts according to Bentum et al. (2011) to:

- 720 m directly west of the West Mole
- 620 m at 2 km upstream the West Mole
- 520 m at 5 km upstream the West Mole

These shoreline expansions have led to an accretion of $10.5 * 10^6 m^3$ in the time span of 1960 to 2010. This is translated annually a volume of 210,000 m^3/year. This accretion can be schematised as a sink of 35%.

Conclusive regarding the morphological impact that the Lagos Moles does not impact the area of interest around Badagry. This is assumed for the fact that Bentum et al. (2011) has discovered an area of influence of 30 km downstream as well as upstream with the Lagos moles using the Single Line Theory. She notes that because of bypassing the area of influences is actually even less upstream however it does not really matter for the fact that Badagry is located over 50 km away from Lagos. However there is one side note which needs to be made. This sidenote regards the fact that Bentum et al. (2011) uses the Pelnard-Considère model underlying the single line theory. This model however is based on the fact that no sand is bypassing the groyne untill accretion has reached the tip of the groyne (Razak, 2015). She noticed this as well as she found that the coast updrift should have accreted 1,200 m approximately 110 after construction of the moles. This was however not the case but it only expanded with around 720 m.

A.4.1.2 Morphology Downdrift of East Mole

The values of retreat of the Bar Beach, found by Bentum et al. (2011) are:

- 690 m directly east of the East Mole
- 370 m at 2 km downstream the East Mole
- 260 m at 5 km downstream the East Mole

These shoreline stagnations have led to a total volume of sediment erosion of $22.3 * 10^6 m^3$. Which leads to a yearly erosion volume of 450,000 m^3/year. As again the total annual sediment transport is known in the alongshore littoral drift (600,000 m^3), there can be said that the erosion of the Bar Beach functions as a source term of 75% in the sediment transport balance.

Between 1960 and 2017 The regression of the shoreline led to a total eroded volume of $10.4 * 10^6 m^3$ in the period of 1960 and 2010. When this is translated to an annual number it comes down to 200,000 m^3/year, which is also schematised as 35 per cent of the total alongshore sediment transport. Though there must be taken into account that this number not only exists out of erosion purely due to the moles. In this period of time several nourishment have been executed at Bar Beach which makes the 35 per cent rather a netto erosion number than a total erosion number. An annual volume of roughly 580,000 m^3/year is supplied to the Bar Beach (Bentum et al., 2011).

A.4.2 Eko Atlantic

One of the largest man-made constructions of late is the Eko Atlantic Cite project at Lagos in front of the Bar Beach. This project has as main goal the reclamation of land (9 km^2) for housing and development purposes next to the fact that it aims to create a more natural shape of the coastline in such a way that the erosion will be reduced to the east of the Lagos Moles. This way the erosion at Bar Beach will have a permanent mitigation solution. This project is known for its 6.5 km long revetment which protects the land reclamation on the ocean side and an average width of approximately 1.3 km (Oniru, 2011). In Figure A.11 the lay-out of the design is shown. Currently phase 3 of the project is almost completed.

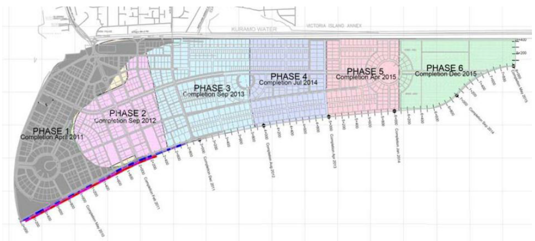


Figure A.11: Layout of Eko Atlantic, the different colours show the different construction phases (RoyalHaskoning, 2011b)

Bentum et al. (2011) has researched multiple kinds of bypassing scenarios in the Lagos Lagoon which were modelled with UNIBEST. Concluded from this case study is that for all scenarios, the Eko Atlantic project reduces erosion directly next to the East Mole completely however this volume or erosion which would take place without the project is now only delayed until further up the coast next to the project.

The total absolute volume of erosion between 2010 and 2020 is not altered by the presence or absence of Eko Atlantic City. Although the total erosion volume is not altered by the presence of Eko Atlantic City and further erosion of the Bar Beach is prevented, there occurs a shift in erosion location due to the project. The translation of erosion induces erosion of the coast downstream Eko Atlantic City (Bentum et al., 2011). One remarkable aspect as well is that because of the project, the coastline subjected to erosion has been reduced to 8-10 km instead of the 16 km without the project. Thus, the high erosion rates directly downstream of Eko Atlantic are, most probably, occurring very locally (Bentum et al., 2011).

B

Current impact assesment

B.1 Introduction

In this appendix an assessment of the impact of currents in the area of interest is presented together with its consequences for the modelling set-up.

B.2 Morphological impact

Previous studies regarding non-wave driven currents (RoyalHaskoningDHV, 2014c) have been executed with as goal to predict the impact on navigability and the siltation of the approach channel.

Nearshore, within the surfzone, the effect of the large scale ocean currents is found negligible (RoyalHaskoningDHV, 2014c). Large scale ocean currents are assumed not to influence the sediment balance nearshore. Consequently, there was no reason to include an external influence of current in the earlier morphological studies for the traditional proposed design of the breakwater.

In contradiction to earlier studies, the area in which sand is modelled extends to the tip of the breakwater in this research. Consequently, the area which is vulnerable to sediment transport, is stretching out far beyond the breakerzone offshore. This leads to the fact that currents might do play a role. This concerns non wave-driven currents like the effects of the Guinea current but also wave-driven currents.

In order to assess the navigability of the future Badagry port approach channel, a flow modelling study has been carried out a couple of years ago (RoyalHaskoningDHV, 2014c).

A metocean data collection campaign, conducted by DEEP (RoyalHaskoningDHV, 2014c), started in April 2014. The survey campaign consists of two instrument frames (Acoustic Doppler Current Profiler, ADCP) on the sea bed measuring current, wave, water level and turbidity data. ADCP's and other measurement tools have been used to validate the wave model. The nearshore frame, known as "Station 1", is located at a depth of approximately 15 m and an offshore frame, referred to as "Station 2", is situated around the 17 m depth contour, see Figure B.1.

For the flow model set-up the same model domain, grid and bathymetry have been used as for the SWAN computations. The SWAN analysis of the waveclimate offshore is presented in Appendix C and the transformation to nearshore in Appendix D. The model domain and bathymetry which have been used during this study are shown in Figure B.1.

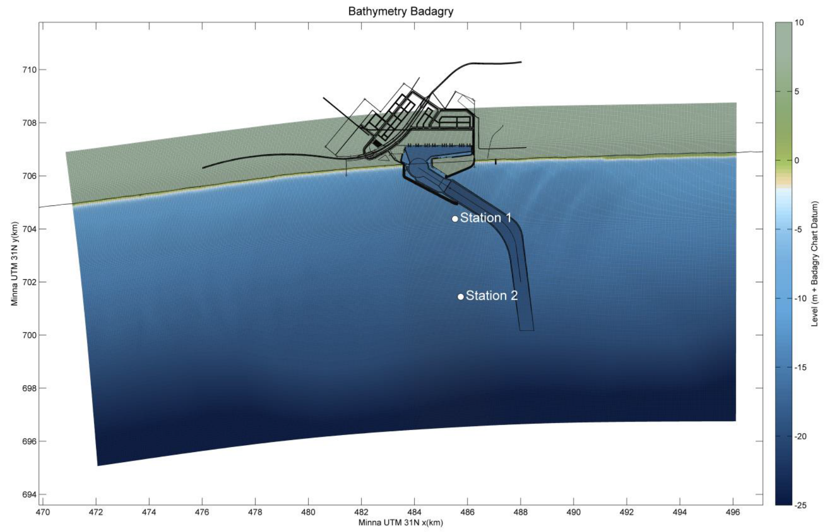


Figure B.1: Model domain and bathymetry of the Badagry flow model, including location metocean stations (RoyalHaskoningDHV, 2014c)

B.3 Wave-driven currents

Wave-driven currents have impact on sediment transport especially in the surf zone. As stated before the breakwater is modelled as a coastline and has more or less the same slope throughout the profiles. Normally wave-driven currents would be less present at the tip of the breakwater because of the large length of the breakwater compared to the width of the surf zone. Due to the uniformity of the final beach profile, there is also a breaker zone established along the breakwater and with that wave-driven currents have influence on sediment transport. Wave-driven currents are taken into account by LITPACK automatically and are included.

B.4 Non wave-driven currents

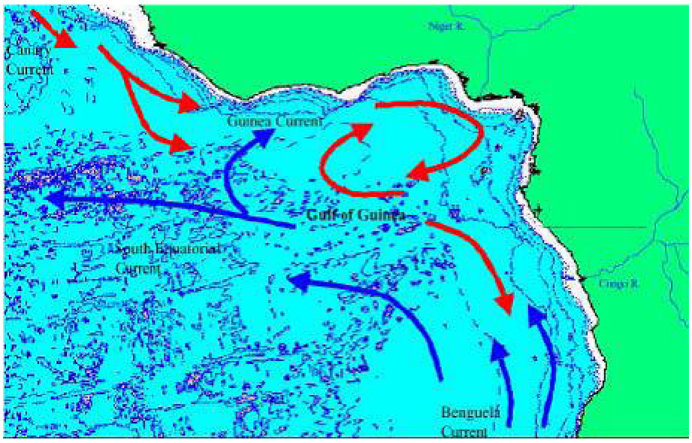


Figure B.2: Schematic overview of large ocean scale currents off the West African coast.(Awosika and Folorunsho, 2010)

In the oceanic waters in front of the West African coast the "Guinea Current" flows from west to east. This is a continuation of the Equatorial Counter Current which is located in the middle part of the Atlantic Ocean, see Figure B.2. Its velocity offshore varies between 1 m/s (max. 1.5 m/s) in the summer, and 0.5 m/s (max. 0.75 m/s) in the winter (Allersma and Tilmans, 1993). The current is enhanced by the monsoon and can be modified by the 'Harmattan'. The Harmattan is a hot, dry wind that blows from the northeast or east in the western Sahara and is strongest in late fall and winter (late November to mid-March) (Britannica, 2017). In addition the current is quite superficial and barely near the coast (Bosboom and Stive, 2012). Upwelling of cold (22-25 °C) water occurs especially off the coast of Ghana in the summer and during the Harmattan. In other periods of the year, the temperature of the sea varies between 27 and 29 °C. The annual variation in mean sea level (MSL) is negligible in the middle part of the coast. This current is convectively-unstable and becomes weaker in the east. Below the Guinea Current flows the Guinea Under Current, which flows from east to west as a return branch of the Equatorial Under Current (Binet and Marchal, 1993). Since this current can either flow eastward or westward both current directions are assessed.

A time series of the depth-averaged current velocities measured in station one in April 2014 is shown in Figure B.3. It illustrates that depth-averaged current velocities up to 0.34 m/s in east direction have been measured.

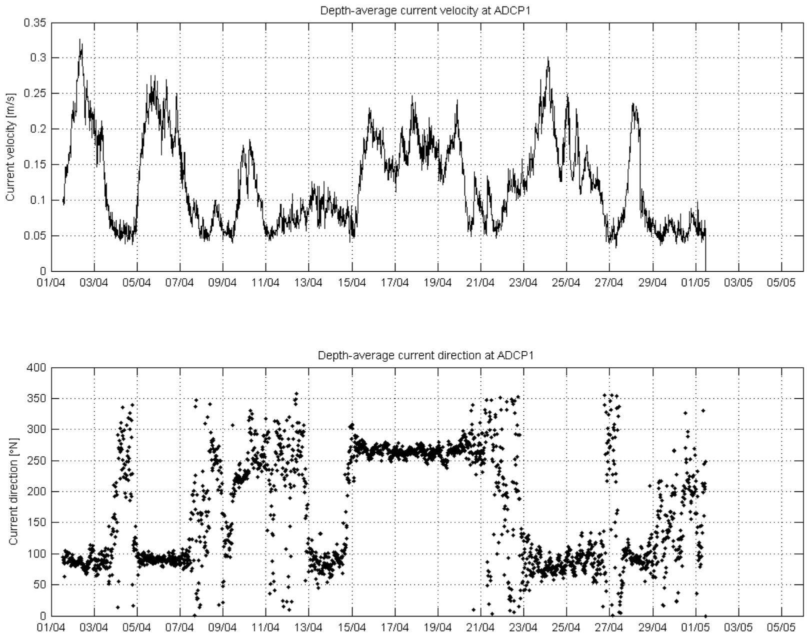


Figure B.3: Time series of depth-averaged current velocity (top) and associated direction (bottom) in station one in April 2014 (RoyalHaskoningDHV, 2014c)

In the period afterwards data has been gathered from May to July which is shown in Figure B.4. In this period depth-averaged current velocities up to 0.43 m/s in east direction have been measured.

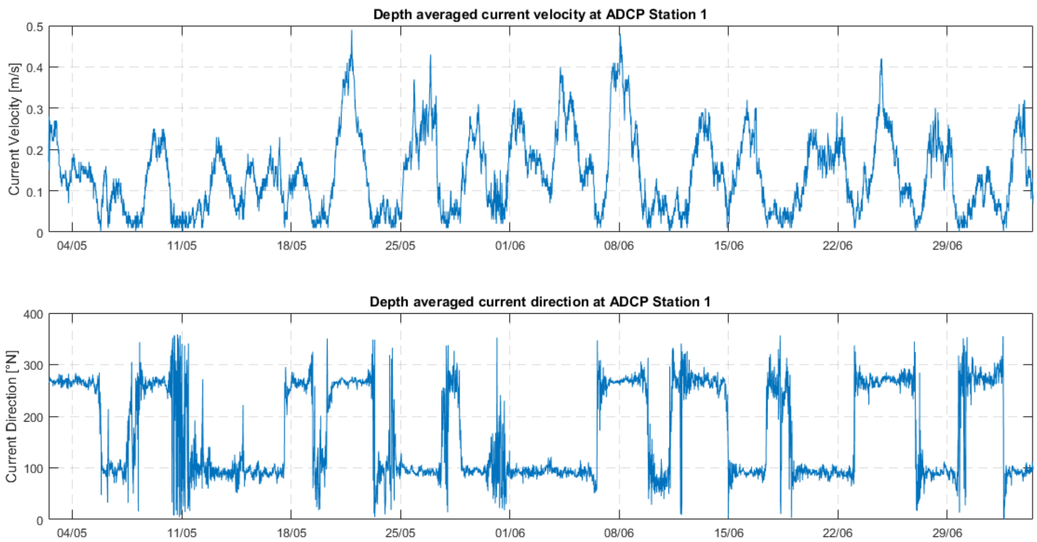


Figure B.4: Time series of depth-averaged current velocity (top) and associated direction (bottom) in station one in May to July

When this current is also direction-averaged, a current of 0.0261 m/s existed in April and in the period from

May to July 2014 it was only 0.0162 m/s. Next to these findings, a remark needs to be made about the period of direction reversal. The periods taken by the current to reverse are clearly larger than the period of the governing semi-diurnal tide as it concerns multiple days on average. Therefore can be assumed that the tide is not the driving force of this current which was already assumed.

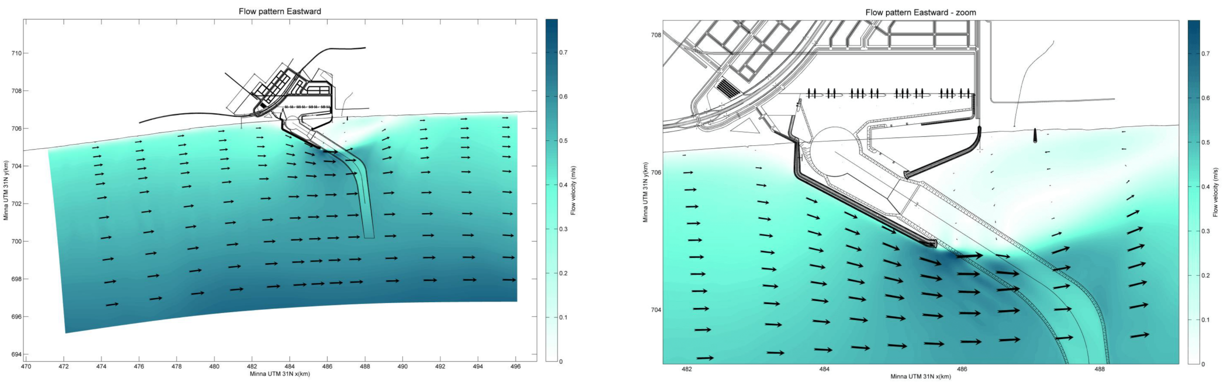
Conclusively needs to be stated that the impact of large-scale oceanic currents is assumed to be negligible. Average values as 0.0261 and 0.0162 m/s are negligible to the present wave-driven currents. This led to the assumption that the effect of non-wave driven currents can be neglected for this research and therefore not is used as input in the model.

B.5 Current contraction

The construction of the traditionally designed breakwater can lead to contraction and acceleration of currents. The traditional design is not very sensitive for these currents as it much better protected against erosion than a sand breakwater.

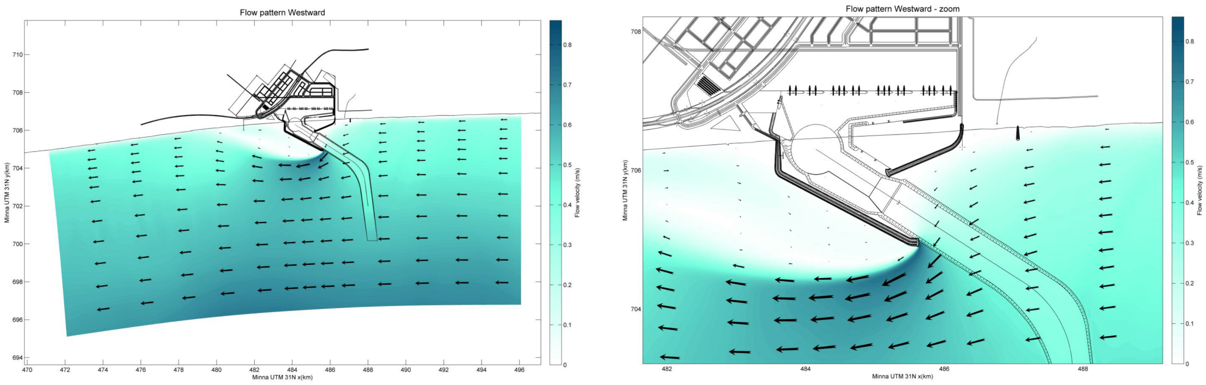
In earlier studies RHDHV assessed the current contraction and acceleration around the traditional breakwater. This modelling execution has been done with a coupled flow and wave model therefore the total current is modelled and not only a wave-driven or a large-scale oceanic current. They setted the boundary conditions for a depth-averaged current velocity of 0.5 m/s in station 2. This value is assumed to be representative for only a few times per year. This is roughly in agreement with values found in literature.

The results of the previous study of RHDHV are shown in Figure B.5 and B.6 for respectively eastward and westward direction. In these the flow of patterns of a current with a magnitude of 0.5 m/s in station two is shown. In these figures not all current vectors are visible.



(a) Large scale (b) Close-up

Figure B.5: Flow pattern corresponding to an east going ocean current with a magnitude of 0.5 m/s in station 2. (RoyalHaskoningDHV, 2014c)



(a) Large scale (b) Close-up

Figure B.6: Flow pattern corresponding to a west going ocean current with a magnitude of 0.5 m/s in station 2. (RoyalHaskoningDHV, 2014c)

In the same figures is shown that contraction of the current increases the magnitude by almost 50% around the tip of the breakwater. It is a significantly higher flow when compared to only the wave-driven part. This part is shown in Figure B.7.

In the approach channel the depth significantly increases spatially with respect to surrounding depths. This increase in depth leads to decrease of the current velocity in the approach channel. The maximum decrease reaches 20% (RoyalHaskoningDHV, 2014c).

Conclusively, needs to be stated that the effect is assumed to be the same as in previous studies. This is because of the fact that the tip of the breakwater is designed to be the same as the traditional design, see Chapter 7. This current contradiction is not taken into account for the modelling LITPACK.

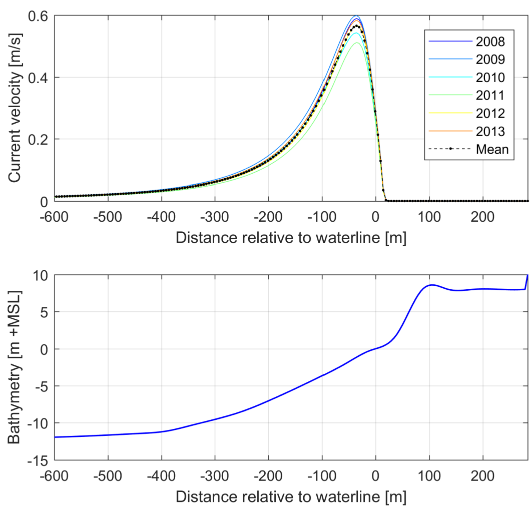


Figure B.7: Wave-drive current velocity over cross-section at Badagry, modelled with LITPACK for 2008-2013

C

Offshore Conditions data

C.1 Offshore Windsea Waves

The windsea events from the Argoss data have been analysed to determine suitable combinations for the different SWAN calculations for windsea. The various steepness lines in Figure 3.14 are used to define the correct steepness range. Suitable values for significant wave height are shown in Figure C.2, in which for each 30° sector the occurrence in percentages is shown for all data. Although further data processing distributions are executed in bins of 5°. The largest significant wave height is 2.75 m, most of the windsea waves however have wave heights in the range 0.5 - 1.5 m. The average significant windsea waveheight amounts to 0.94 m. In line with the wind records most wind generated waves come from south to southwest (165 - 255°) with an average of 218.14°.

The data used for the windsea study has been extracted and is shown in Figure C.1. The average values together with the steepness line of 0.014 are displayed as well. You can see clearly that the extraction is quite clean as there are almost no events above the steepness line.

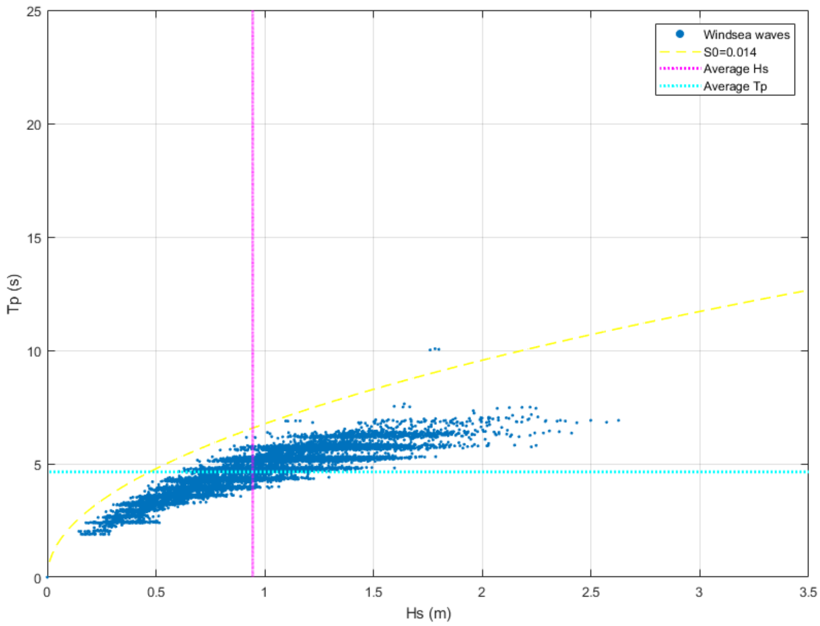


Figure C.1: Extraction windsea dataset together with characteristics

Hm0 (m)		Dir (deg. N) -15.00 15.00	15.00 45.00	45.00 75.00	75.00 105.00	105.00 135.00	135.00 165.00	165.00 195.00	195.00 225.00	225.00 255.00	255.00 285.00	285.00 315.00	315.00 345.00	All classes
0.00	0.25		0.008	0.008		0.008		0.032	0.237	0.309	0.032	0.008		0.641
0.25	0.50	0.047	0.103	0.142	0.040	0.032	0.071	0.340	4.932	3.388	0.253	0.071	0.008	9.428
0.50	0.75	0.071	0.055	0.047	0.008		0.032	0.776	11.495	7.544	0.063			20.092
0.75	1.00			0.135				0.863	16.411	11.439	0.111			28.958
1.00	1.25			0.142				1.892	13.513	7.687	0.024			23.258
1.25	1.50			0.040				1.845	6.484	2.668	0.071			11.107
1.50	1.75			0.016				1.092	2.802	0.736	0.016			4.663
1.75	2.00							0.356	0.879	0.111				1.346
2.00	2.25							0.079	0.285	0.032				0.396
2.25	2.50							0.024	0.047	0.024				0.095
2.50	2.75							0.016						0.016
	Total	0.119	0.166	0.530	0.047	0.040	0.103	7.315	57.085	33.938	0.570	0.079	0.008	100.000

Figure C.2: Offshore significant wave height for windsea waves (all year); Joint probability of occurrence

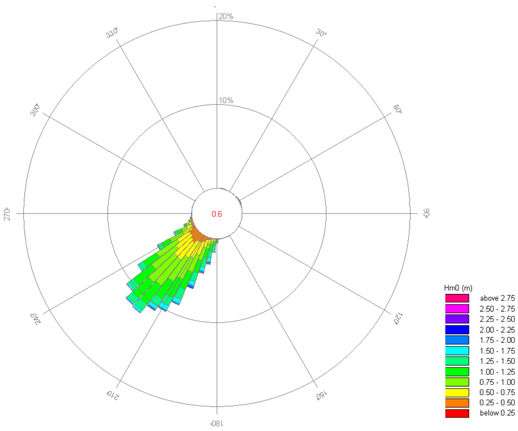


Figure C.3: Significant wave height windsea waves, all year, 5° sectors in m (RoyalHaskoningDHV, 2014a)

Peak wave periods for windsea are shown in Figure C.4. The largest peak wave periods are between 10 and 11 s. The majority of peak periods are between 3 and 6 s and the average is found to be 4.64 s.

Tp (s)		Dir (deg. N) -15.00 15.00	15.00 45.00	45.00 75.00	75.00 105.00	105.00 135.00	135.00 165.00	165.00 195.00	195.00 225.00	225.00 255.00	255.00 285.00	285.00 315.00	315.00 345.00	All classes
0.00	1.00													
1.00	2.00								0.135	0.119				0.253
2.00	3.00	0.024	0.040	0.055	0.008	0.016	0.016	0.214	2.557	1.916	0.198	0.032	0.008	5.082
3.00	4.00	0.095	0.127	0.198	0.040	0.024	0.087	0.530	10.877	9.476	0.230	0.047		21.731
4.00	5.00			0.214				1.069	18.287	15.595	0.103			35.268
5.00	6.00			0.063				3.325	21.168	6.650	0.040			31.246
6.00	7.00							2.137	3.895	0.182				6.214
7.00	8.00							0.040	0.142					0.182
8.00	9.00													
9.00	10.00													
10.00	11.00								0.024					0.024
	Total	0.119	0.166	0.530	0.047	0.040	0.103	7.315	57.085	33.938	0.570	0.079	0.008	100.000

Figure C.4: Offshore wave period for windsea waves (all year); Joint probability of occurrence

The relevant combinations for the SWAN simulations for windsea, based on the analysis of steepness, wave height, wave period and the wind analysis in Section 3.3.4 are shown in Table C.1.

Table C.1: Relevant combinations for SWAN simulation for Windsea Waves

Item	Amount of variables							
Wind speed [m/s]	3	0	7.5	15	-	-	-	-
Steepness [-]	5	0.014	0.02	0.025	0.04	0.06	-	-
Wave height [m]	5	0.5	1.0	1.5	2.0	2.75	-	-
Wave direction [°]	7	90	120	150	180	210	240	270
Water level [m +CD]	2	0.75	1.75	-	-	-	-	-

A total of 1050 simulations have been done to simulate all possible windsea conditions. The spectral shape of the windsea data resembles a JONSWAP (or Pierson-Moskovitz) spectrum. In order to correctly apply these theoretical spectra on the windsea data, a JONSWAP spectrum has been fitted on the data (for each month) to match the frequency distribution. The JONSWAP spectrum is defined as shown in Equation C.1.

$$S(f) = Af^{-5}exp(-Bf^{-4})\gamma^{\alpha} \tag{C.1}$$

In which:

A	$= \frac{5H_{m0}^2}{16T_p^4}$
B	$= \frac{5}{4T_p^4}$
α	$= exp[-\frac{1}{2}(\frac{f-f_p}{\sigma f_p})^2]$
σ	$= f(x) = \begin{cases} \sigma_a, & f \leq f_p \\ \sigma_b, & f > f_p \end{cases}$
F_p	= peak frequency $[\frac{1}{s}]$
γ	= shape parameter

A value of 3.3 for the shape parameter corresponds to a standard (North Sea) JONSWAP spectrum. Default values of the standard JONSWAP spectrum for σ_a and σ_b are 0.07 and 0.09 respectively (RoyalHaskoningDHV, 2014a).

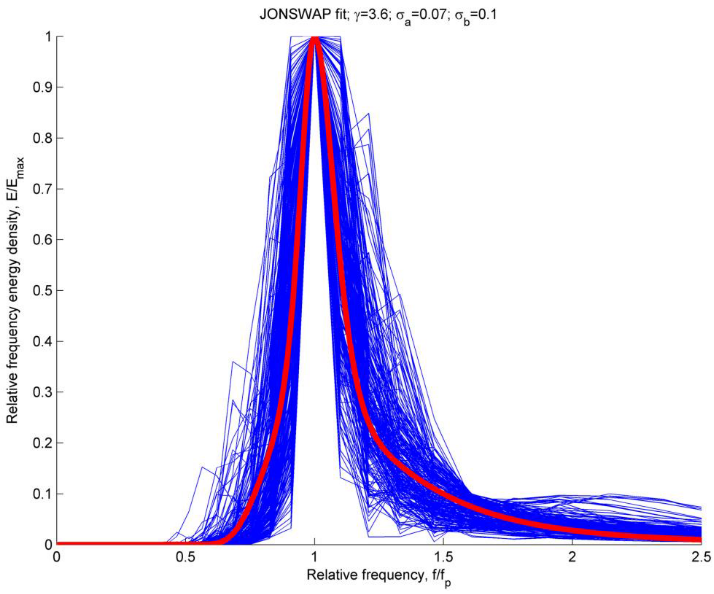


Figure C.5: JONSWAP fit through windsea wave spectra for the period 1997-2013 (RoyalHaskoningDHV, 2014a)

Figure C.5 shows a fitted JONSWAP spectrum for all windsea data from 1997 to 2013. The best fit gives the following estimate: $\gamma = 3.6$, $\sigma_a = 0.07$ and $\sigma_b = 0.1$. These values have been used in the SWAN model for windsea waves.

C.2 Offshore Swell Waves

The long swells are subject to considerable shoaling. Due to this phenomenon they become governing in shallow water nearshore, where they form high breakers.

The Figure C.7 shows the significant wave height H_{m0} in 30° sectors for swell waves in the period 1997 - 2013.

Although further data processing distributions are executed in bins of 5°. The maximum wave height does not exceed 3.5 m. The average wave height amounts to 1.27 m. This maximum wave height is much higher than the maximum wave height of the windsea waves. All swell events come from the south to southwest. The range in the SWAN model has been limited from 120 to 240°.

The data used for the swell study has been extracted and is shown in Figure C.6. A remark needs to be made about the events displayed in the Figure which are below the steepness of 0.014. ARGOSS uses a method to distinct the two types of waves in which the root of the two separate spectra squared and summed is the total sea spectrum. Not only the steepness is taken into account to distinct the states and therefore some events below the steepness line are also determined to be swell waves.

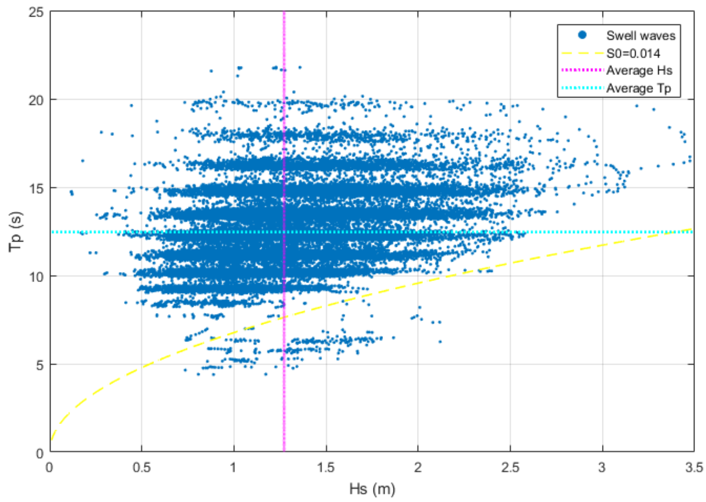


Figure C.6: Extraction windsea dataset together with characteristics

	Dir (deg, N)	-15.00	15.00	45.00	75.00	105.00	135.00	165.00	195.00	225.00	255.00	285.00	315.00	All
	Hm0 (m)	15.00	45.00	75.00	105.00	135.00	165.00	195.00	225.00	255.00	285.00	315.00	345.00	classes
0.00	0.25								0.032					0.032
0.25	0.50							0.040	0.180					0.220
0.50	0.75					0.004	0.010	1.957	1.737					3.709
0.75	1.00				0.002		0.008	9.197	10.737	0.004				19.949
1.00	1.25					0.004	0.004	11.591	17.891	0.008				29.497
1.25	1.50						0.002	8.507	14.115	0.002				22.626
1.50	1.75							5.295	8.505					13.800
1.75	2.00							2.645	3.457					6.102
2.00	2.25							1.154	1.316					2.470
2.25	2.50							0.587	0.571					1.158
2.50	2.75							0.149	0.137					0.287
2.75	3.00							0.032	0.044					0.077
3.00	3.25							0.042	0.008					0.050
3.25	3.50								0.022					0.022
	Total	0.000	0.000	0.000	0.002	0.008	0.024	41.199	58.753	0.014	0.000	0.000	0.000	100.000

Figure C.7: Offshore significant wave height for swell waves (all year); Joint probability of occurrence

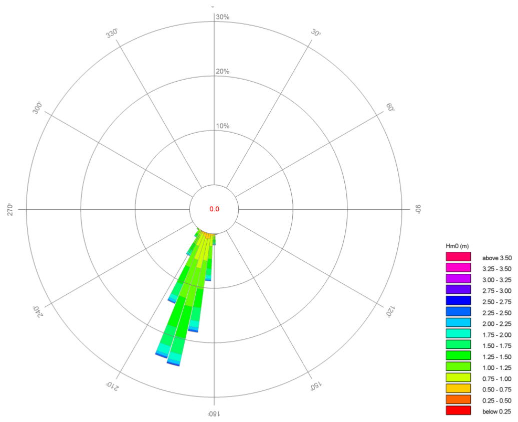


Figure C.8: Significant wave height swell waves, all year, 5° sectors in m

Offshore wave periods vary from 5 s to 22 s, as shown in the table below. The majority of events occur in the 10 to 15 s range in the 192 to 208° sectors with an average of 196.67°.

	Dir (deg. N)	-15.00	15.00	45.00	75.00	105.00	135.00	165.00	195.00	225.00	255.00	285.00	315.00	All
	Tp (s)	15.00	45.00	75.00	105.00	135.00	165.00	195.00	225.00	255.00	285.00	315.00	345.00	classes
0.00	1.00													
1.00	2.00													
2.00	3.00													
3.00	4.00													
4.00	5.00				0.002	0.002			0.032	0.002				0.038
5.00	6.00							0.024	0.182	0.002				0.208
6.00	7.00							0.145	0.111					0.256
7.00	8.00							0.038	0.048					0.087
8.00	9.00						0.002	0.581	0.157					0.741
9.00	10.00					0.002	0.008	3.890	1.310	0.002				5.212
10.00	11.00							7.902	5.121	0.002				13.025
11.00	12.00					0.002	0.002	7.906	10.519	0.004				18.433
12.00	13.00					0.002	0.002	7.946	15.859	0.002				23.811
13.00	14.00						0.004	6.439	13.360					19.803
14.00	15.00						0.004	3.513	6.728					10.245
15.00	16.00						0.002	0.755	1.332					2.089
16.00	17.00							1.376	2.767					4.143
17.00	18.00							0.373	0.722					1.096
18.00	19.00							0.163	0.274					0.438
19.00	20.00							0.129	0.204					0.333
20.00	21.00							0.004	0.010					0.014
21.00	22.00							0.012	0.016					0.028
	Total	0.000	0.000	0.000	0.002	0.008	0.024	41.199	58.753	0.014	0.000	0.000	0.000	100.000

Figure C.9: Offshore peak wave period for swell waves (all year); Joint probability of occurrence

The relevant combinations for the SWAN simulations for swell, based on the analysis of wave height and wave period are displayed in Table C.3.

Table C.3: Relevant combinations for SWAN simulation for Swell Waves

Item	Amount of variables								
Wind speed [m/s]	3	0	7.5	15	-	-	-	-	-
Steepness	6	510^{-4}	1.510^{-3}	0.003	0.005	0.01	0.015	-	-
Wave height [m]	7	0.5	1.0	1.5	2.0	2.5	3.0	3.5	-
Wave direction [°]	8	120	150	170	180	190	200	210	240
Water level [m +CD]	2	0.75	1.75	-	-	-	-	-	-

A total of 2016 simulations have been done to simulate all relevant swell conditions that occur in the ARGOSS data. For both swell and windsea, water levels of 0.75 and 1.75 m +CD are considered. The 0.75 m +CD level which equals MSL, the 1.75 accounts for tidal range and wind set-up. Set-up due to wind in Nigeria is relatively small and is presented at Subsubsection 3.3.4.1.

In a similar manner as for the windsea waves, the shape of the swell waves frequency density spectrum is estimated. The swell waves spectrum in general does not match a JONSWAP spectrum well but tends to fit a Gaussian spectrum way more accurately. A Gaussian fit has been found on the standardised frequency. Figure C.10 shows the result for the period 1997 - 2013.

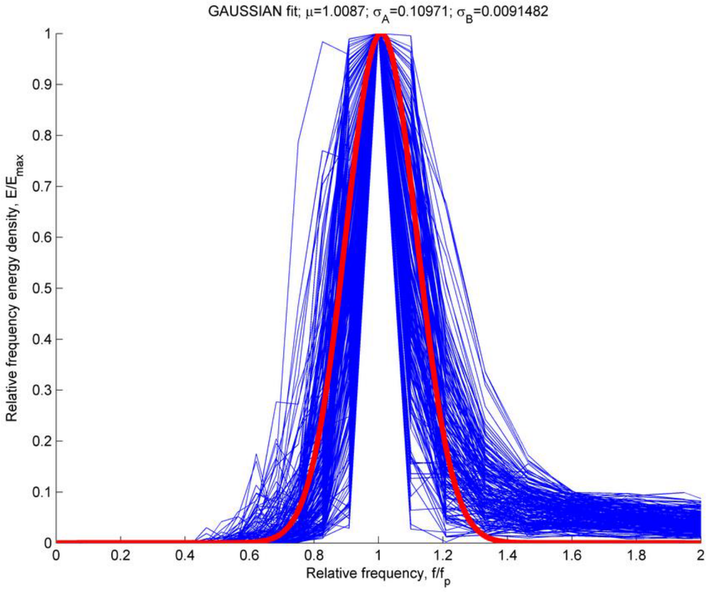


Figure C.10: Gaussian fit for the period 1997-2013 (RoyalHaskoningDHV, 2014a)

The best Gaussian around-the-year fit for the swell waves is estimated through Equation C.2.

$$S(f) = \frac{1}{\sigma\sqrt{2\pi}} exp[-\frac{1}{2}(\frac{f-mu}{\sigma})^2] \tag{C.2}$$

The best estimate for the standard deviation is: $\sigma = 0.009$. This Gaussian spectrum has been used in SWAN for the swell waves simulations.

C.3 Results year round

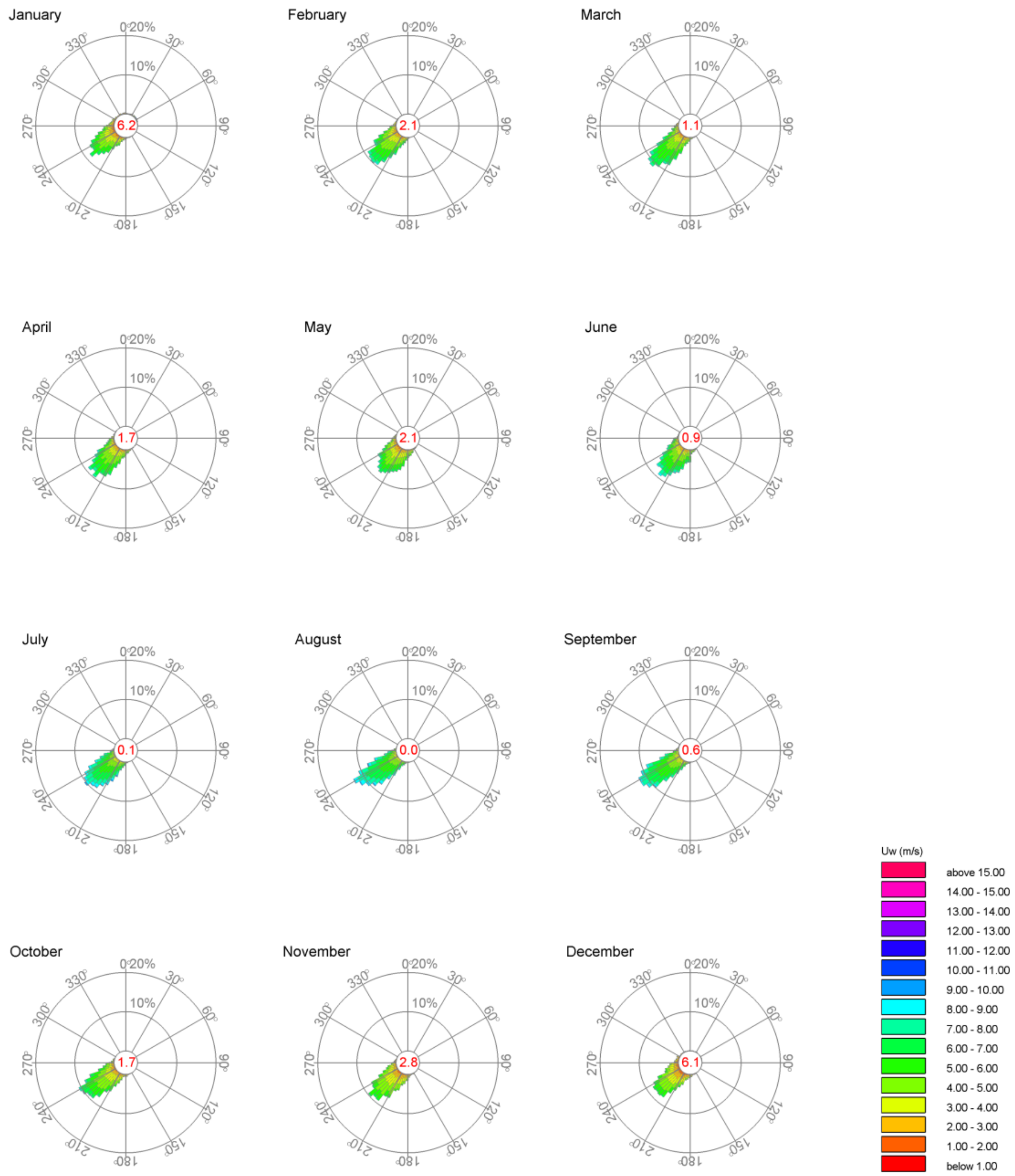


Figure C.11: Year-round wind speed U_{10}[m/s] (RoyalHaskoningDHV, 2014a)

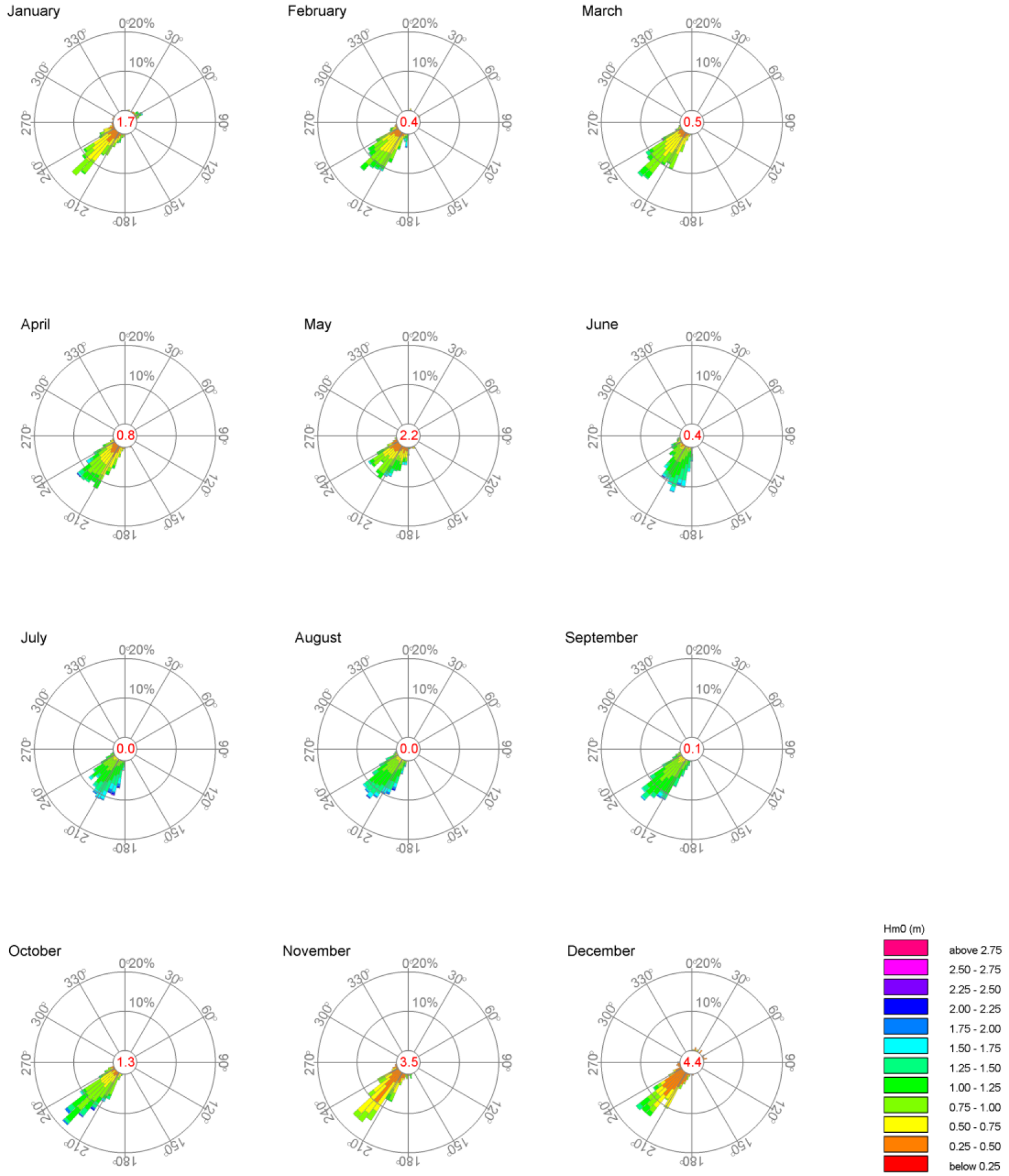


Figure C.12: Wave height offshore windsea waves (RoyalHaskoningDHV, 2014a)

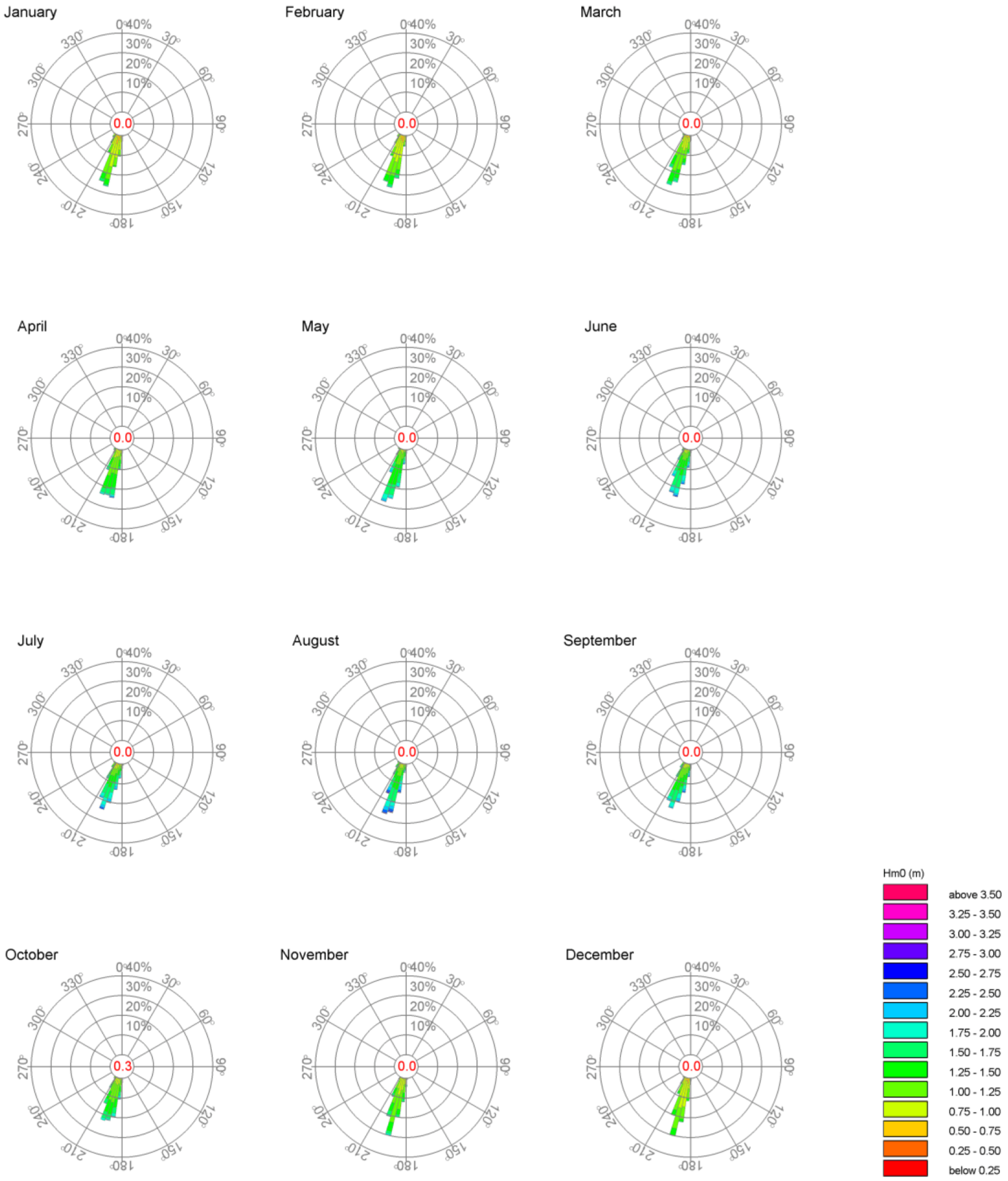


Figure C.13: Wave height offshore swell waves (RoyalHaskoningDHV, 2014a)

D

Nearshore Conditions data

D.1 Nearshore Windsea

Wind is an important factor for windsea waves as it is the driving force for these kind of waves. In the case of Swell, wind is of less importance as it is not its driving force nor of any influence. As stated before, the wind conditions are assumed to be similar for the offshore and nearshore positions. The nearshore characteristic results are shown for point 1 (for location see Figure D.4). Point 1 has been proved governing for normal conditions at Badagry from earlier studies (RoyalHaskoningDHV, 2014a). Therefore data from this point will be applied in this research.

		Dir (deg, N) -15.00	15.00	45.00	75.00	105.00	135.00	165.00	195.00	225.00	255.00	285.00	315.00	All
	Hm0 (m)	15.00	45.00	75.00	105.00	135.00	165.00	195.00	225.00	255.00	285.00	315.00	345.00	classes
0.00	0.25													
0.25	0.50					0.174	0.008		0.055	0.214				0.451
0.50	0.75					0.459	0.087	0.879	17.741	4.117				23.282
0.75	1.00					0.174	0.008	2.929	41.680	3.016				47.807
1.00	1.25					0.063		5.272	14.606	0.380				20.321
1.25	1.50					0.008		2.802	3.380	0.016				6.206
1.50	1.75							0.784	0.752					1.536
1.75	2.00							0.206	0.103					0.309
2.00	2.25							0.040	0.008					0.047
2.25	2.50							0.032	0.008					0.040
	Total					0.879	0.103	12.943	78.333	7.742				100.000

Figure D.1: Significant wave height for windsea waves (all year); Joint probability of occurrence, Point 1

		Dir (deg, N) -15.00	15.00	45.00	75.00	105.00	135.00	165.00	195.00	225.00	255.00	285.00	315.00	All
	Tp (s)	15.00	45.00	75.00	105.00	135.00	165.00	195.00	225.00	255.00	285.00	315.00	345.00	classes
0.00	1.00													
1.00	2.00													
2.00	3.00													
3.00	4.00					0.507	0.087	0.602	10.228	2.351				13.775
4.00	5.00					0.277	0.016	1.734	35.560	4.734				42.321
5.00	6.00					0.095		6.072	28.689	0.649				35.505
6.00	7.00							4.314	3.792	0.008				8.114
7.00	8.00							0.222	0.040					0.261
8.00	9.00								0.024					0.024
	Total					0.879	0.103	12.943	78.333	7.742				100.000

Figure D.2: Peak wave period for windsea waves (all year); Joint probability of occurrence , Point 1

From the Figures D.1 and D.2 can be concluded that the significant wave height for windsea waves nearshore does not exceed 2.50 m. The average height of a windsea wave amounts 0.91 m. Peak wave periods are typically between 3 and 7 s with an average of 4.89 s. These approach the coast from a direction of 208.46°.

Figure D.3 shows a typical windsea wave event for wave steepness 0.1, or $T_p = 4.3$ s.

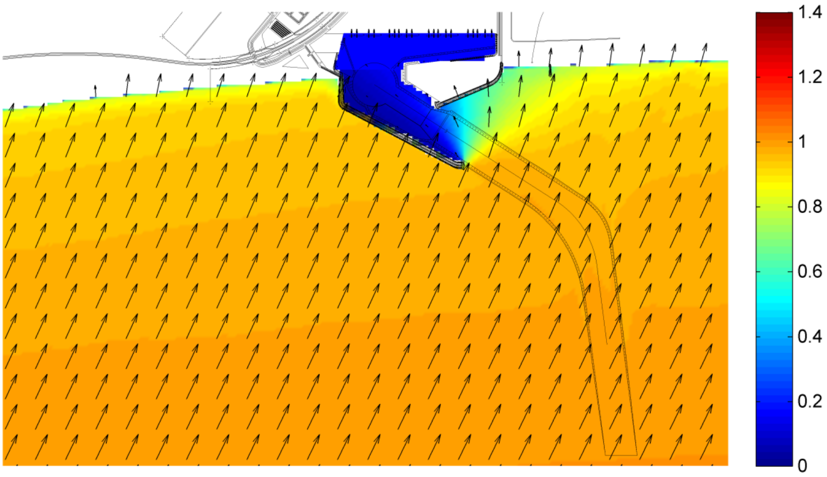


Figure D.3: Significant wave height SWAN result; $u_w = 7.05$ m/s, $s_0 = 0.025$, $H_{m0} = 1.0$ m, Dir = 210°, Water Level = MSL (RoyalHaskoningDHV, 2014a)

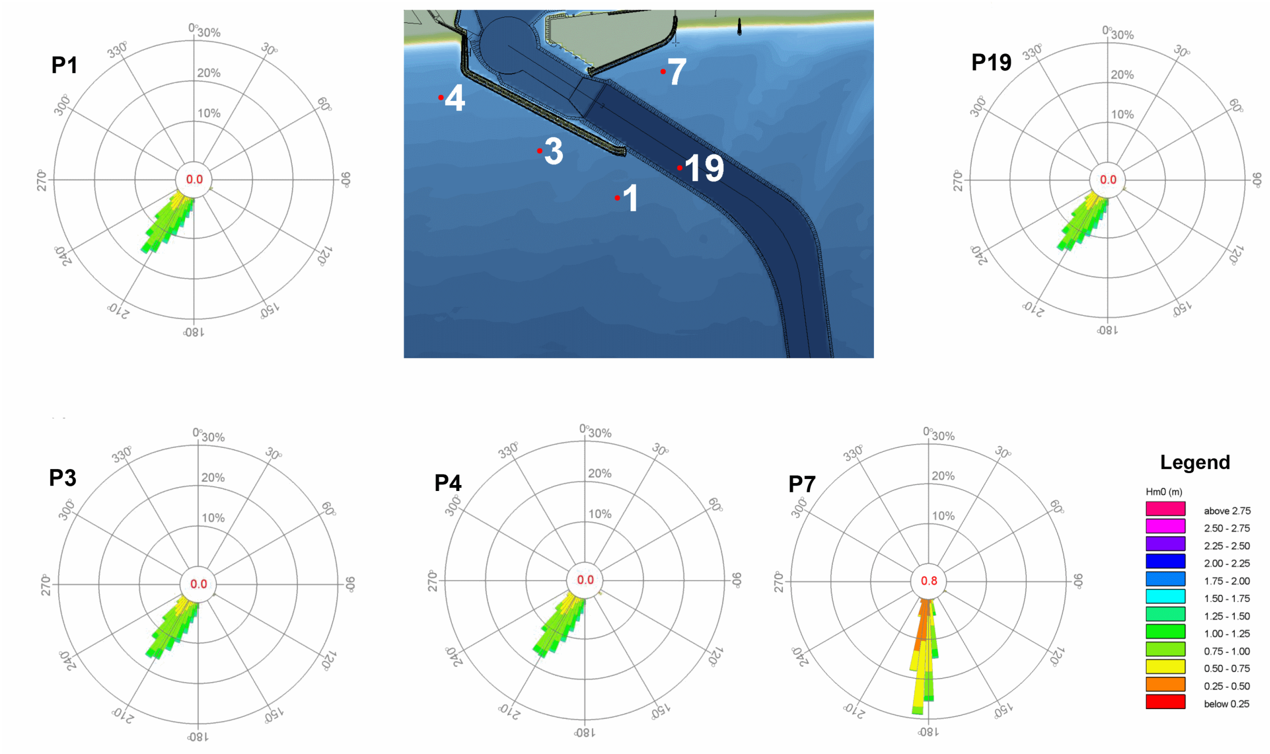


Figure D.4: Nearshore windsea waves at Project location (RoyalHaskoningDHV, 2014a)

D.2 Nearshore Swell

The SWAN output for point 1 (for location see Figure D.8), is shown in the Figures D.5 and D.6 for the all year swell waves. Point 1 has been proved governing for normal conditions at Badagry from earlier studies (RoyalHaskoningDHV, 2014a). Therefore data from this point will be applied in this research.

More than 50% of swell waves nearshore come from the 182.5 – 192.5° sector. The highest waves are up to 3.75 m high (H_{m0}), but the majority does not exceed 1.75 m. The average significant wave height of a swell wave concerns 1.42 m. The average direction of a swell wave nearshore amounts to 189.13°.

Hm0 (m)		Dir (deg, N) 162.50 167.50	167.50 172.50	172.50 177.50	177.50 182.50	182.50 187.50	187.50 192.50	192.50 197.50	197.50 202.50	202.50 207.50	207.50 212.50	212.50 217.50	217.50 222.50	All classes
0.00	0.25													
0.25	0.50					0.002								0.002
0.50	0.75	0.002	0.004	0.008	0.067	0.176	0.204	0.071	0.008	0.004				0.545
0.75	1.00	0.006	0.010	0.073	0.628	1.796	2.615	1.344	0.321	0.149	0.040	0.006	0.006	7.004
1.00	1.25	0.004	0.002	0.107	1.598	5.357	8.705	6.237	2.520	0.648	0.093	0.022	0.006	25.308
1.25	1.50	0.002		0.085	2.107	7.698	12.071	7.513	2.206	0.315	0.042	0.004		32.042
1.50	1.75		0.002	0.157	2.363	6.760	8.291	2.668	0.416	0.087	0.006			20.750
1.75	2.00			0.180	1.705	4.215	2.787	0.456	0.081	0.020	0.006			9.450
2.00	2.25			0.161	0.981	1.336	0.630	0.052	0.008	0.002	0.004			3.174
2.25	2.50			0.105	0.450	0.448	0.093	0.006	0.002		0.002			1.106
2.50	2.75			0.081	0.159	0.141	0.014	0.002						0.398
2.75	3.00			0.018	0.063	0.038								0.119
3.00	3.25			0.008	0.057	0.012								0.077
3.25	3.50				0.004	0.008								0.012
3.50	3.75				0.008	0.006								0.014
	Total	0.014	0.018	0.983	10.188	27.994	35.410	18.349	5.561	1.225	0.194	0.032	0.012	100.000

Figure D.5: Significant wave height for swell waves (all year); Joint probability of occurrence, Point 1

Tp (s)		Dir (deg, N) 162.50 167.50	167.50 172.50	172.50 177.50	177.50 182.50	182.50 187.50	187.50 192.50	192.50 197.50	197.50 202.50	202.50 207.50	207.50 212.50	212.50 217.50	217.50 222.50	All classes
0.00	1.00													
1.00	2.00													
2.00	3.00													
3.00	4.00													
4.00	5.00									0.006	0.002			0.008
5.00	6.00					0.008	0.008	0.006	0.002	0.026	0.002			0.057
6.00	7.00				0.036	0.032	0.036	0.016	0.044	0.042	0.044	0.018	0.010	0.280
7.00	8.00		0.004	0.008	0.038	0.123	0.167	0.161	0.081	0.028	0.018	0.012	0.002	0.644
8.00	9.00	0.006		0.010	0.141	0.450	0.509	0.361	0.159	0.065	0.022			1.725
9.00	10.00		0.004	0.024	0.529	1.437	1.628	1.088	0.515	0.214	0.042			5.483
10.00	11.00			0.083	1.342	2.938	3.727	2.797	1.275	0.357	0.057	0.002		12.581
11.00	12.00	0.002	0.004	0.172	1.546	3.723	5.896	4.068	1.768	0.327	0.006			17.511
12.00	13.00			0.139	1.320	4.905	8.937	5.755	1.293	0.129				22.483
13.00	14.00	0.004	0.004	0.087	1.189	4.901	7.317	2.813	0.333	0.026				16.676
14.00	15.00			0.117	0.807	2.587	3.085	0.609	0.054	0.004				7.264
15.00	16.00	0.002	0.002	0.113	1.358	3.719	3.029	0.509	0.026					8.760
16.00	17.00			0.067	0.724	1.689	0.771	0.101	0.004					3.356
17.00	18.00			0.071	0.432	0.743	0.184	0.018	0.002					1.449
18.00	19.00			0.036	0.482	0.589	0.091	0.032	0.002					1.233
19.00	20.00			0.044	0.167	0.125	0.014	0.006						0.357
20.00	21.00			0.012	0.063	0.018	0.008							0.101
21.00	22.00				0.014	0.006								0.020
22.00	23.00						0.002	0.008	0.002					0.012
	Total	0.014	0.018	0.983	10.188	27.994	35.410	18.349	5.561	1.225	0.194	0.032	0.012	100.000

Figure D.6: Peak wave period for swell waves (all year); Joint probability of occurrence , Point 1

The peak wave periods for the nearshore swell waves, for point 1, are shown in Figure D.6. From the figure, it can be concluded that the majority of nearshore swell wave periods is in the range of 9 - 15 s. Maximum wave periods are between 22 and 23 s. The average peak period for a swell wave nearshore regards 12.68 s. Figure D.7 shows a typical SWAN output field for significant wave height for a 3.5 m swell wave, $T_p = 20$ s from 205° (offshore). Nearshore, the wave height increases due to refraction and shoaling. This effect is largely determined by local bathymetry.

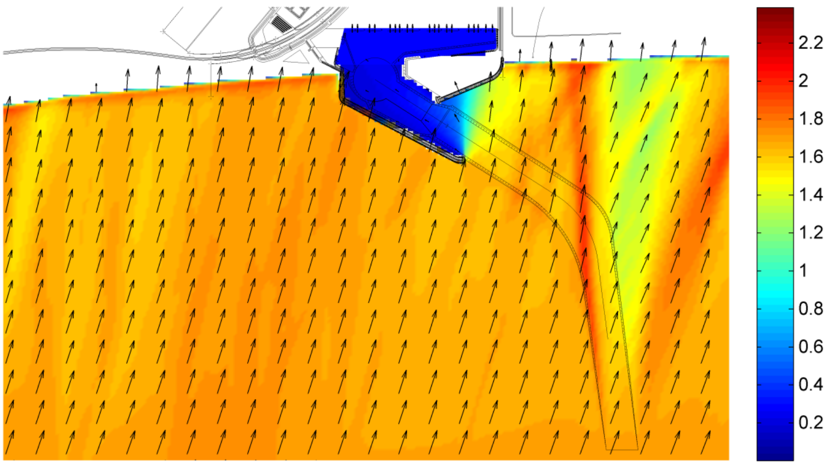


Figure D.7: Significant wave height SWAN result; $u_w = 7.05$ m/s, $s_0 = 0.005$, $H_{m0} = 1.5$ m, Dir =210°, Water Level = MSL (RoyalHaskoningDHV, 2014a)

Waves refract slightly towards the coast. Because of this, wave energy decreases so wave heights are slightly smaller than offshore. Offshore wave periods change little.

In the Figures D.8 the nearshore wave roses for swell waves are plotted for the future harbour layout. These roses are assumed to be the same for the purpose of this research.

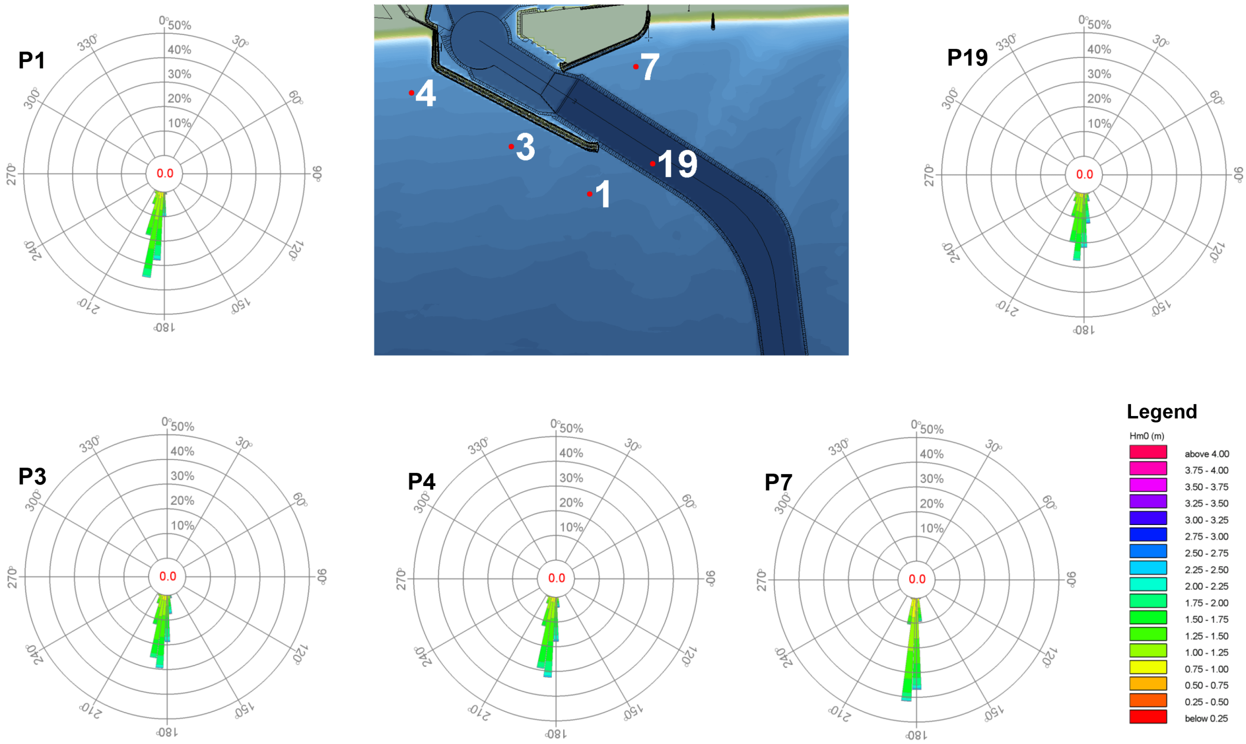


Figure D.8: Nearshore swell waves at Project location (RoyalHaskoningDHV, 2014a)

E

Wind Set-up

E.1 Introduction

In earlier studies RHDHV has determined the surge due to wind set-up which is present at Badagry numerically. The latter has been done by using a hydrodynamic flow model. This has been compared to an analytical approach.

E.2 Analytical approach

In the Rock Manual (et al., 2007) a formula is presented to visualize the set-up of water due to wind. This formula is shown in Equation E.1.

$$\frac{\delta\eta}{\delta x} = \frac{1}{\rho_w g d}\tau_w \tag{E.1}$$

In which:

η	= wind-induced set-up [m]
d	$= h + \eta =$ actual water depth, including the induced wind set-up [m]
ρ_w	= density of water [kg/m^3]
τ_w	= wind shear stress acting on the water surface in the direction normal to the coast [N/m^2]

It is assumed that the wind stress at the water surface is balanced by a water level gradient only and that the wind does not force a current. Therefore Equation E.1 becomes E.2.

$$\frac{\delta\eta}{\delta x} = \frac{ku^2 cos(\phi)}{gh} \tag{E.2}$$

In which the left-hand-side represent the water level gradient. The right-hand-side contains the following variables:

In which:

u	= wind speed [m/s]
ϕ	= wind direction relative to shore normal [°]

g = gravitational acceleration [m/s^2]
h = water depth [m]

The factor k is defined as:

$$k = c_w * \frac{\rho_{air}}{\rho_{water}} \tag{E.3}$$

In which:
c_w = friction factor varying between 0.8 x 10^{-3} and 3.0 x 10^{-3} (et al., 2007)

Taken into account that the densities of water and air are 1030 and 1.21 kg/m^3 respectively Kamphuis (2010) suggests that k should be $3.2*10^{-6}$ yielding a value for c_w of $2.74*10^{-3}$. Using a wind speed of 25 m/s and ϕ being 0°, the wind set-up can be determined if its plotted over the depths of the cross-shore bathymetry of Badagry. The cross-shore profile of the wind set-up is depicted in Figure E.1 as an example (RoyalHaskoningDHV, 2014c). The return period associated to u being 25 m/s is larger than a 1000 years.

Figure E.1 shows that the wind set-up is determined at 0.13 m. This is quite small for a windspeed of 25 m/s. For a beach that steep however it can be explained by the fact that the wind set-up only develops in shallow water.

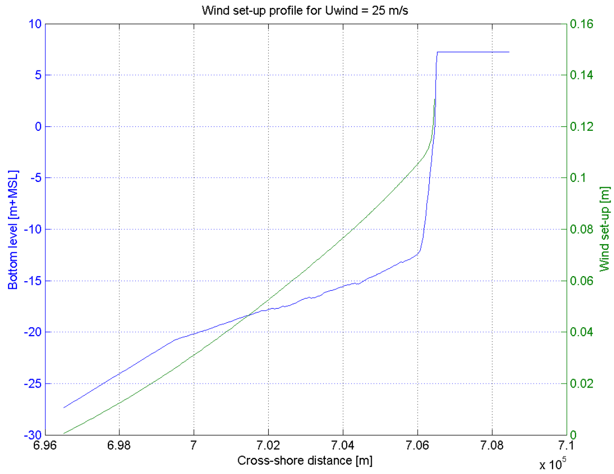


Figure E.1: Cross-shore profile of analytical wind set-up for a wind speed of 25 m/s (RoyalHaskoningDHV, 2014c)

Due to the steep beach and nearshore zone, the shallow area with a large water level gradient is for the Badagry coast relatively small, only about 100 m wide. Despite the large water level gradient in that zone, the actual set-up, which is the product of the gradient and the width of the zone, is limited (RoyalHaskoningDHV, 2014c).

It needs to be stated that however this is an underestimation of the wind set-up. This due to the fact that the wind set-up is not zero at the offshore end. This means that the coastal cross-shore profile was too small in the analytical approach and underestimates the set-up nearshore. Although it is an underestimation, earlier has been assumed that the analytical approach presumes a balance between the wind and the water level gradient only. In reality however the wind forcing will also result in a current. this current then again results in a lower wind set-up. That makes the whole approach rather conservative. The wind set-up is determined for a number of current velocityies and shown along with the numerical results, see Figure E.2.

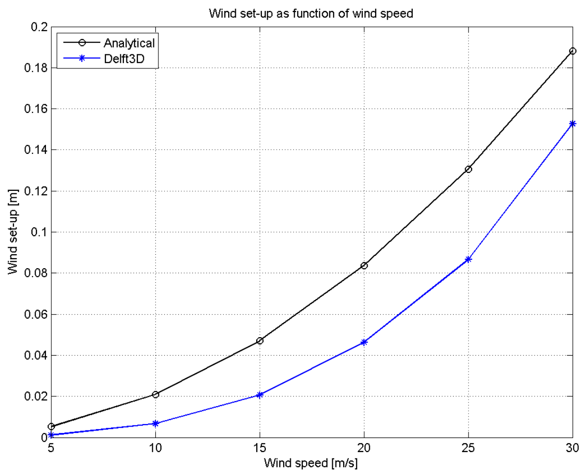


Figure E.2: Nearshore wind set-up as a function of the wind speed, calculated with both an analytical approach and a numerical flow model (RoyalHaskoningDHV, 2014c)

E.3 Numerical flow model

RHDHV has used a flow model to calculate the
wind set-up by means of imposing a uniform and constant wind field. Like in the analytical approach this is done for a number of wind speeds. From the resulting water level field the set-up can be deduced.

Like in the analytical approach, the simulated set-up levels are limited, even smaller than the ones obtained with the analytical approach. Given the numerous differences between the two methodologies, the small absolute differences are remarkable.

E.4 Extreme wind speeds

In FigureE.2 the storm surge as a function of the wind speed has been presented. In this section extreme wind speeds are established resulting in extreme storm surge levels. The basis for the extreme wind conditions is an ARGOSS hindcast wind dataset of 17 years in location 5°N and 3.75°E (RoyalHaskoningDHV, 2014a).

From this time series 140 storm events with a minimum wind speed of 8.95 m/s have been selected. Various distributions (Weibull / Gumbel / GPD / GEV) have been applied to these events and it appears that the GEV (Generalized Extreme Value) distribution with the Probability Weighted Moments yields the best fit, see Figure reffig:surge. In Table E.4 the extreme wind speeds for a range of return periods are summarized.

Because the wind conditions off the Nigerian coast are generally mild, also the extreme wind speeds are not large. It has to be emphasized although the hindcast datasets as the one used for this analysis generally do not contain small-scale features like squalls. During squalls, see Subsection 3.3.4.2, considerably larger wind speeds can occur, but the time and length scale at which squalls occur are considered to be too small to generate a well-developed surge.

Table E.4: Extreme wind speeds as function of return period

Return period [yrs]	Wind speed [m/s]
1	10.4
5	11.8
10	12.6
50	15.0
100	16.3
500	20.5
1000	22.8

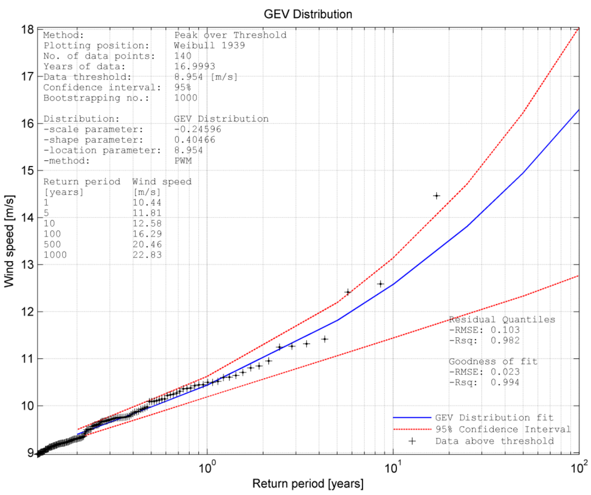


Figure E.3: GEV fit through 140 storm events (RoyalHaskoningDHV, 2014c)

Combining the 1/100 year wind speed of 16.3 m/s with Figure E.2 results in a 1:100 year storm surge of only 0.06 m, taking the largest of the two surges presented in that figure.

F

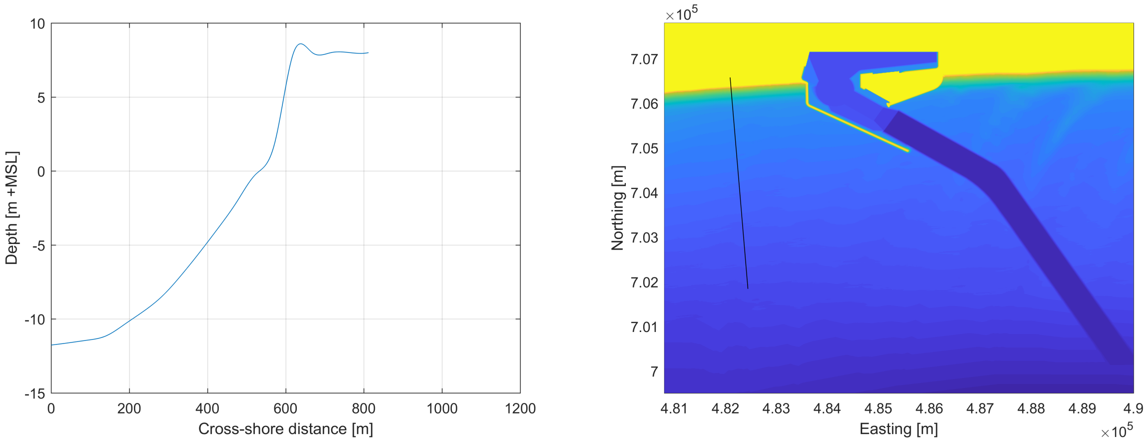
Set-up of Coastline Evolution Model Module

F.1 Introduction

In this appendix the set-up of the Coastline evolution model module is presented. Although the set-up was subjected to a lot changesduring the modelling the main set-up along with its assumptions is presented in this appendix.

F.2 Profile

The cross-shore profile along the stretch of coast in the area of interest has a highly uniform character. For this reason there has been assumed that for the complete coastline one cross-shore profile is possible to use. From bathymetry data a profile is extracted on which is elaborated in Subsection 3.2.3. The first part (ca 800 m) of this profile is shown in Figure F.1(a). This profile has been extracted out of a depth chart at the project area. In this depth chart is the preliminary traditional design included as shown in Figure F.1(b).



(a) Retrieved cross-shore uniform profile

(b) Overview of profile at project area

Figure F.1: Orientation of Profile 1

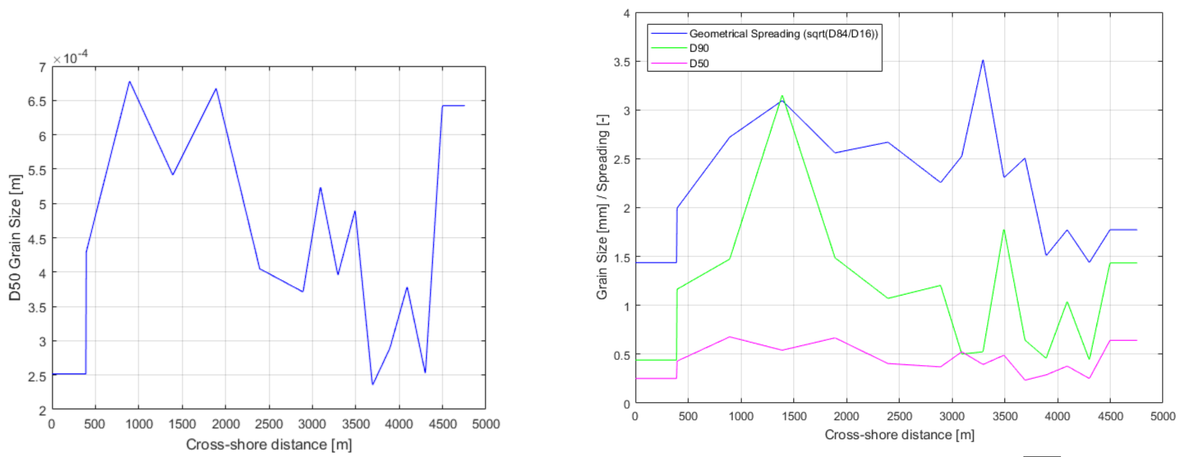
This extracted depth profile has been combined with the grab sample chart. Location specific grain sizes are implemented in the profile in this way. An elaboration on the grain sizes and the combining of this is presented at Subsection F.2.1.

Multiple set-ups with different numbers of profiles with location specific bathymetry data have been implemented in the model. In the end it is chosen to only use one profile for modelling coastline evolution. This is done for the sake of an assumption from Subsubsection 2.4.2. According to this assumption the profile shape stays the same during coastline regression or advancement. It is assumed to be more realistic to model the main profile uniformly along the coast. Eventually the coast will adapt itself to the new situation created by the sand breakwater and start to develop the very same profile as is currently at the coast. That is because this is the profile which endures the wave climate. Through this approach the uniform character of the coast is also withheld at the final situation including a sand breakwater. The spatial variation of grain size throughout the area does differ however this differs along the profile the most and not along the coast. Therefore a profile perpendicular to the coast characterizes a good uniform profile for the coast at the area of interest.

F.2.1 Grain size

The dataset of RHDHV contains over 90 grabsamples in front of the coast at Badagry. With these grab samples a spatial distribution of the grain sizes has been made which is shown in Subsection 3.2.3.1. This spatial distribution is used to combine a depth profile with location specified grain sizes. These grain sizes not only give insight in median grain diameters like D_{50} and D_{90} along the cross-shore profiles used in modelling but they are on their turn used to determine other parameters like fall velocities and roughness coefficients, see Subsections F.2.2 and F.2.3.

In Figure F.2(b) the median grain size distributions along the profile is presented along with its geometrical distribution. In Chapter 5 an elaboration on the sensitivity of the grain size is presented.



(a) Grain size enlarged along Profile

(b) Grain size and Geometrical Spreading $\sqrt{\frac{D_{84}}{D_{16}}}$ along Profile

Figure F.2: Profile characteristics. Left: Offshore Right: Coast

F.2.2 Fall velocity

The fall velocity of sediment which is integrated into the profile used for modelling is determined with a formula comparable to Soulsby's for natural sands (Davis and Dalrymple, 2011). Equation F.1 is used to determine the fall velocity.

$$w_s = 10 * \frac{\nu_{kin}}{D_{50}} * ((1 + \frac{0.01 * (s-1) * 9.81 * D_{50}^3}{\nu_{kin}^2})^{0.5} - 1) \tag{F.1}$$

In which:

T = Temperature 29°

ρ_s = Density sediment = 2650 kg/m^3

ρ_w = Density water = 998.2 kg/m^3

s = $\frac{\rho_s}{\rho_w}$

ν_{kin} = $(-2.8 * 10^{-6} * T^3 + 0.0006 * T^2 - 0.047 * T + 1.752) * 10^{-6}$ (EngineeringToolbox, 2017)

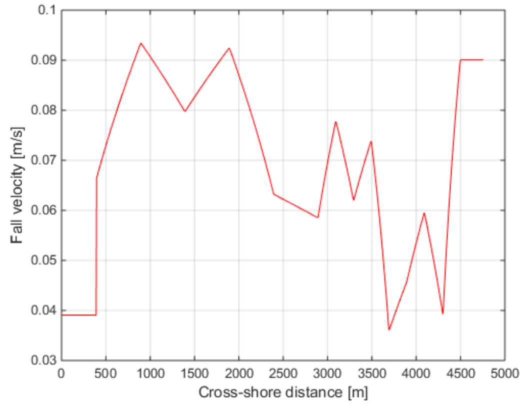


Figure F.3: Fall velocity along the profile. Coast on the right, offshore on the left

This formula is applied over the cross-shore profile with known grain sizes. This leads to fall velocity as displayed in Figure F.3. From this profile can clearly be concluded that due to the larger grain size on the beach the fall velocity is also quite high. It immediately drops to lower values for the finer sediment available in the breaker zone. After the zone in which finer sediment can be found the fall velocity increases again.

F.2.3 Bed Resistance

The roughness coefficient, or Nikuradse roughness has been determined by a simple rule of thumb, proposed by Van Rijn, 1986, Schiereck and Verhagen (2012), see Equation F.2. This can also depicted as $4-5D_{n50}$. In Figure F.4 the roughness throughout the profile is displayed.

$$k_r = 3 * D_{90} \tag{F.2}$$

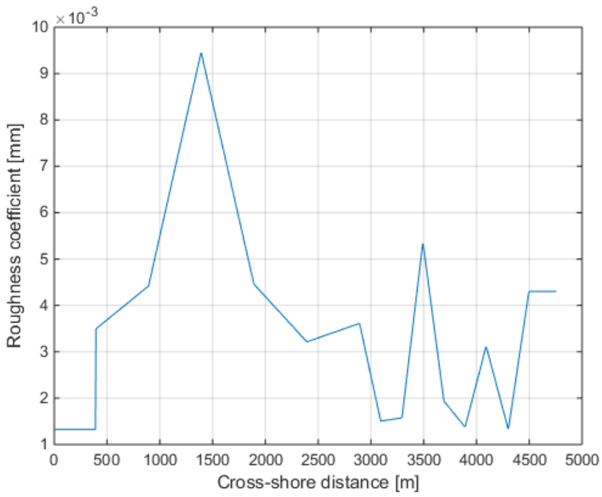


Figure F.4: Nikuradse Roughness coefficient along profile

F.3 Water Level Conditions

As stated in Subsection 3.3.6 the tidal influence on the water in this area is low. For modelling normal conditions for the long term coastline evolution using LITPACK the average water level is assumed to be 0 meter MSL which equals 0.75 meter Badagry CD and constant throughout time.

F.4 Wave Climate

At Subsection 3.3.5 has been elaborated on the type of wave climate which is present at Nigeria. As the wave climate is not constant but slowly varying along the coast, multiple wave climates have been used. In the calibration phase a large piece of coast has been examined. It concerns the coast from a bit west of Badagry to Lagos. This coastal stretch along with its wave climates is shown in Figure 4.1. Five wave climates have been created for this coastal stretch. These are created with five time series. These time series have been created with the modelling software SWAN (Booij, 2005).

F.4.1 Data selection

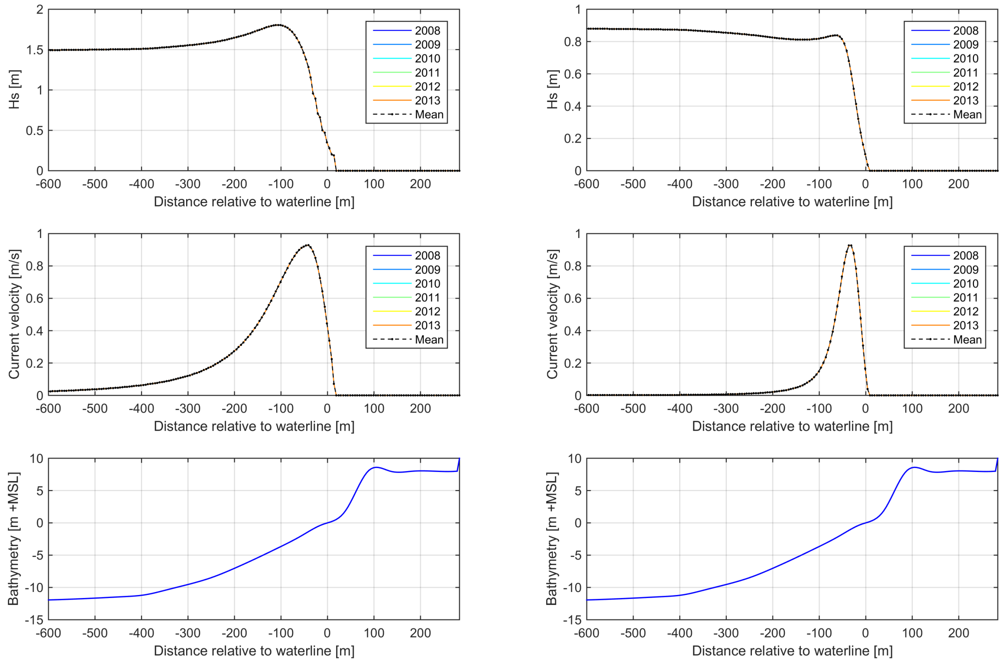
In Subsection 3.3.5 the distinction between the windsea waves and the swell waves from the dataset was presented. This distinction made it possible to use wave fields separately in LITPACK as they are both complete different datasets after distinction. In this research there has been chosen to solemnly use the swell waves dataset. The reason underlying is that this type of waves is governing at the location of interest. The windsea waves are actually of such a small impact it can be neglected.

To support this selection in data a sensitivity analysis to the impact of both the wave fields in LST is needed. Therefore a typical windsea wave versus a typical swell wave has been modelled with the Littoral Drift module of LITPACK. This module shows roughly the relation between both waves on the cross-shore profile. This Littoral drift has been set-up with a constant wave climate which is shown in Table F.2. Other settings of the Littoral drift set-up are comparable to the one discussed at Section 4.3.

Table F.2: Characteristics of windsea and swell waves used in LITPACK

Parameters	H_s(m)	T_p (s)	Dir. (°)	Reduction factor
Windsea wave	0.91	4.89	208.46	0.5
Swell wave	1.42	12.68	189.13	0.7

In Figure F.5 the results of the littoral drift in wave height and current velocity are shown for the different types of sea. There needs to be stated that due to the larger wave height of the swell sea the cross-width of the littoral current is bigger than the one of the windsea. This is due to the phenomenon shoaling.

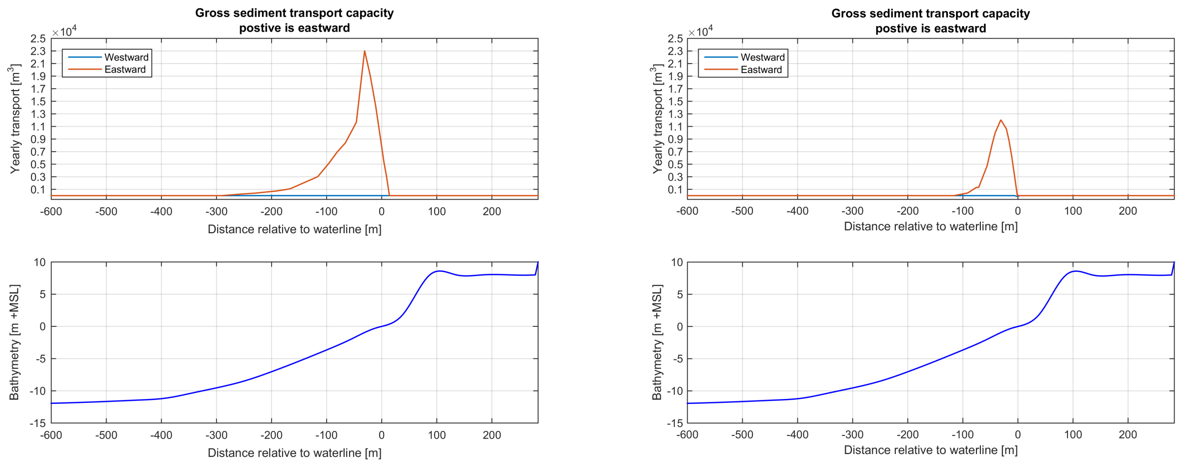


(a) Significant waveheight and Littoral current cross-shore due to Swell

(b) Significant waveheight and Littoral current cross-shore due to Windsea

Figure F.5: Characteristics for different types of sea

In Figure F.6 the bruto littoral drifts for the different sea states are shown. The larger LST peak is clearly displayed along with its broader width.



(a) Bruto Littoral Drift due to Swell cross-shore

(b) Bruto Littoral Drift due to Windsea cross-shore

Figure F.6: Bruto Littoral Drifts for different types of sea

The output of LITPACK led to results in LST which for these two typical waves are found in Table F.3. The amount of the total LST which is generated by windsea waves is determined on 8.99%. This is assumed as sufficient to support the statement that swell waves are the waves which need to be taken into account for the coastline evolution modelling in LITPACK.

Table F.3: Results of comparison LST in LITPACK

	Typical Swell wave	Typical Windsea wave
Averaged Net LST/year	1,462,199 m^3	521,724 m^3
Events	49,657	12,516
Relation	**11.12**	**1**

F.4.2 Period of data

The wind and wave data set which was used in earlier studies (RoyalHaskoningDHV, 2014c) and used for this research concerns the Argoss data set from 1997 to 2013. In this research the period of 2008 - 2013 has been extracted from this data set and used to create wave climates. Table F.4 and F.5 the characteristics of the swell data set are shown. These characteristics are shown throughout the years next to the averages. In general the numbers do not really differ much throughout the years. There is only a small increase in significant wave height and peak period in the period of 2008 to 2013. The data period of 2008 to 2013 has been used in this research and extrapolated to larger time frames for further modelling.

Table F.4: Characteristics of first part of the Argoss data set

year	Average	1997	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007
H(s)	1.27	1.23	1.15	1.22	1.14	1.20	1.25	1.20	1.24	1.22	1.28	1.36
T(p)	12.46	12.44	12.00	12.46	11.86	12.22	12.18	12.08	12.11	12.10	12.27	12.64
H(s)dir	196.68	196.45	196.39	198.28	195.71	195.60	196.68	198.89	197.23	196.16	196.24	196.66
U(w)	4.66	5.04	4.50	4.66	4.86	5.00	4.93	4.85	4.67	5.07	5.07	4.79
Winddir	225.71	224.76	230.17	231.22	230.04	224.01	224.27	232.68	229.63	222.09	220.87	223.23

Table F.5: Characteristics of second part of Argoss data set

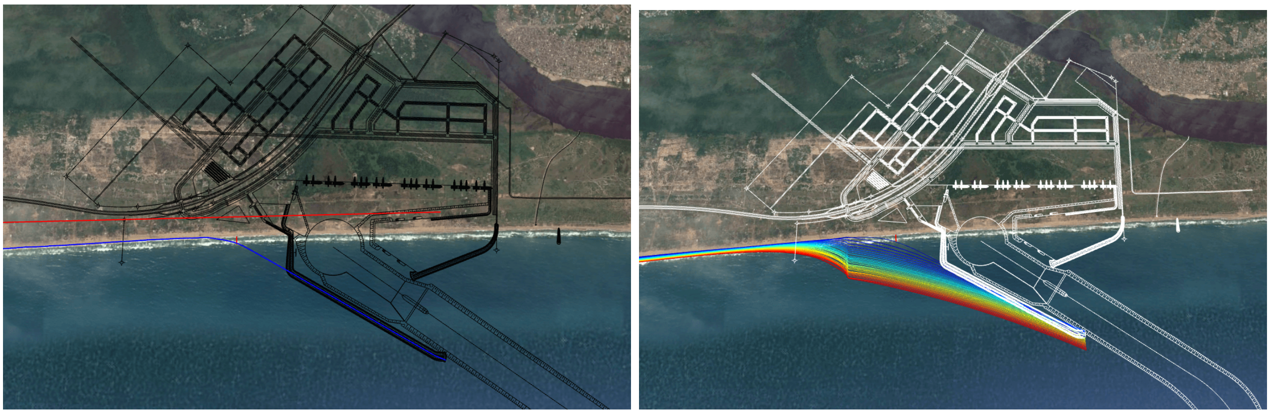
year	Average	2008	2009	2010	2011	2012	2013
H(s)	1.27	1.31	1.42	1.40	1.32	1.32	1.35
T(p)	12.46	12.57	13.02	13.15	12.73	12.88	13.07
H(s)dir	196.68	196.46	197.10	196.00	195.33	197.02	197.30
U(w)	4.66	4.52	4.49	4.24	4.05	4.25	4.31
Winddir	225.71	222.96	225.18	217.07	223.48	229.28	226.07

F.5 Coastline

The modelled coastline in this research was subjected to changes throughout the process. Changes were made mostly due to the fact that negative effects of the boundaries of the model were noticed. These negative effects are desirable to exclude from the area of interest. The boundaries of the model therefore needed to be replaced far away of the area of interest in such a way that they did not influence the result anymore.

The starting point for modelling is to model the complete traditional breakwater as a coastline, see Figure F.7(a). In black in the figure the future port of Badagry is shown and in blue the modelled coastline. In red the baseline of the model is shown.

With this set-up of modelling it was aimed to determine the scales of rates of accretion and to examine the vulnerable parts of a sand breakwater. Initially it was thought to stop the model at the end of the breakwater as showed in Figure F.7(a). In Figure F.7(b) the development of the coastline over a period of 30 years is shown with the red line being the coastal evolution of 30 years and the blue line being the start year. In white the future port is presented. This set-up however turned out quickly not to be realistic.



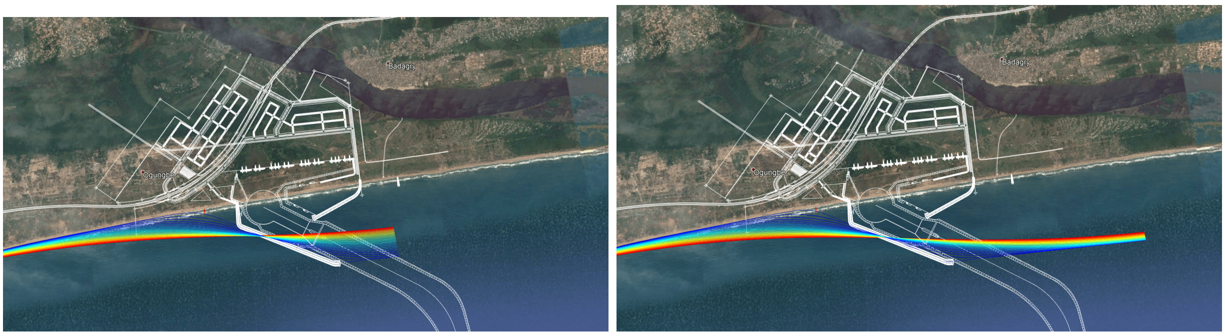
(a) Overview of modelled coastline in blue and baseline in red (b) Overview of output with modelled coastline

Figure F.7: Modelling of initial set-up of coastline

Firstly because the boundary of the model is located at the end of the breakwater there is almost no erosion visible. which is certainly not true for all the accreted sand in front of the modelled coastline. The model engine uses the fixed shoreline gradient at the boundaries to calculate the transport rates. In general model set-ups should cover a larger domain than just the focus area as the results closest to the boundary is affected by restrictions to the coastline.

Secondly, there is a disruption in the beginning of the breakwater. This disruption turned out to be a too large difference in profiles used for the model but as stated before finally one profile was used.

To enhance the coastline model set-up the boundaries were placed further outside and the coastline was modelled to continue straight parallel to the original coastline after the end of the traditionally designed breakwater as shown in Figure F.8(a). Through this approach the irregularities of the boundaries have been tried to extract from the area of interest. This took a few iteration steps as displayed in Figure F.8(b) before placing it far enough.



(a) Small extension of modelled coastline (b) Larger extension of modelled coastline

Figure F.8

There is assumed that if the point where accretion switches to erosion is not affected by the boundaries of the model they are outside the area of influence on the area of interest. This is accomplished in Figure F.9(a) with its output showed in F.9(c).

(a) Stretch of coastline used for modelling

(b) Result total scope

(c) Result zoomed in

Figure F.9: Final total model used for modelling

As already stated the change of coastline does not only change the output and result but also needs a change in wave climate set-up as there has been used a wave climate consisting out of multiple wave time series. This was discussed in Subsection F.4.

G

Overview Equilibrium Coastline at Badagry

Figure G.1: Coastline at Project area at Badagry in equilibrium throughout the years (RoyalHaskoningDHV, 2014c)

H

Extra sensitivity analysis of LITPACK

H.1 Shields parameter

Sediment transport can be approached in different ways. The best known formula for uniform flow is the one by Shields. The sensitivity of this subject is also interesting to determine as it concerns an empirical input which can be changed in LITPACK.

The total sediment transport in LITPACK is calculated as:

$$q_t = q_b + q_s \tag{H.1}$$

In which:
q_t = Total sediment transport $[m^2/s]$
q_b = Bed load transport $[m^2/s]$
q_s = Sediment transport in suspension $[m^2/s]$

In LITPACK the transport is calculated with the STPQ3D model. In this model the transport formula of Engelund and Fredsoe (1976) is used, where the bed load transport is calculated from the instantaneous Shields parameter (DHI-GROUP, 2017b).

According to the 'Shields approach' sediment transport starts to take place once the force which is exerted by the water on the bed is passing a critical friction force. After this critical value the bed starts to erode. The critical Shields parameter ψ_c is a stability parameter which is defined using a critical value of the friction velocity (Schiereck and Verhagen, 2012). Having said this, as sidenote needs to be stated. The Shields criterion is not a physical constant which means that transport below the criterion is not completely absent and that transport only occurrs above the criterion. The shields parameter has a mean and a standard deviation. The formula for stability parameter is shown in Equation H.2

$$\psi' = \frac{{u_f}^2}{\Delta g d} \tag{H.2}$$

In which:
u_f = the friction velocity [m/s]
Δ = Density difference [-]
g = Gravitational acceleration $[m/s^2]$
d = Grain size d_{50} [mm]

The critical stability parameter which is able to insert in LITPACK is shown in Equation H.3.

$$\psi_{c,0} = \frac{{u_{f_c}}^2}{\Delta g d} \tag{H.3}$$

In which:
u_{f_c} = the critical friction velocity [m/s]

This equation of Shields holds for a plane bed. The Shields parameter is adapted for a sloping bed with the Equation H.4 (DHI-GROUP, 2017b).

$$\psi_c = \psi_{c,0} * \left(\frac{-cos\theta sin\beta + \sqrt{\mu_s^2 cos^2\beta - sin^2\theta sin^2\beta}}{\mu_s}\right) \tag{H.4}$$

In which:
θ = angle of flow to the slope [°]
β = angle of slope [°]
μ_s = static friction coefficient ($\mu_s = tan\phi_s$, ϕ_s = angle of repose) [-]

The Shields parameter is then implemented in the dimensional bed load transport Φ_b, see Equation H.5 (DHI-GROUP, 2017b).

$$\Phi_b = 5 * p * (\sqrt{\psi'} - 0.7\sqrt{\psi_c}) \tag{H.5}$$

In which:
p = $(1 + (\frac{\frac{\pi}{6}\beta}{\psi' - \psi_c})^4)^{-0.25}$

This Φ_b on its turn is then implemented to find time-averaged values and inserted in two formulas for both bed load in the mean current direction and one for the normal to the mean current direction.

To explain the effect of the Shields parameter however we generalize the formula to Equation H.6.

$$q_b = \Phi_b * \sqrt{\Delta g d_{50}^3} \tag{H.6}$$

The Shields parameter is changed for finding out the sensitivity in the results. The $\psi_{c,0}$ is varied from a range of 0.030 to 0.055. The default value of the parameter is 0.045 and is used in all the other model runs.

In Figure H.1 the LST capacity of a zoomed in area of the total modelled area of interest is shown in year 50. Along three vertical lines in the graph the positions of different parts of the structures are shown.

Two major conclusions are drawn from the results. Firstly an important aspect of this figure is that it shows that the difference in LST for the different Shields parameters is not significant. Secondly, in the top left of the Figure it shows a minor difference which slightly fades due to the percentage of bed load transport going down. When this percentage goes down the differing of Shields parameter is of even less significance as it quite constant for the suspended load. The LST finally approaches zero as well due to the current going to zero because of the equilibrium angle of the coastline.

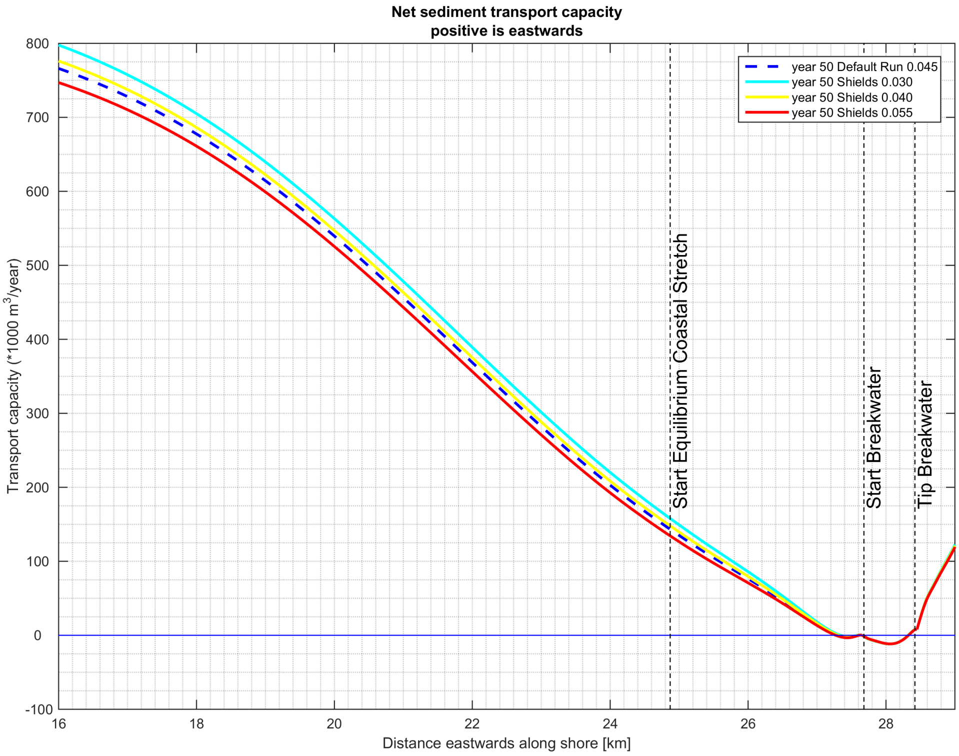
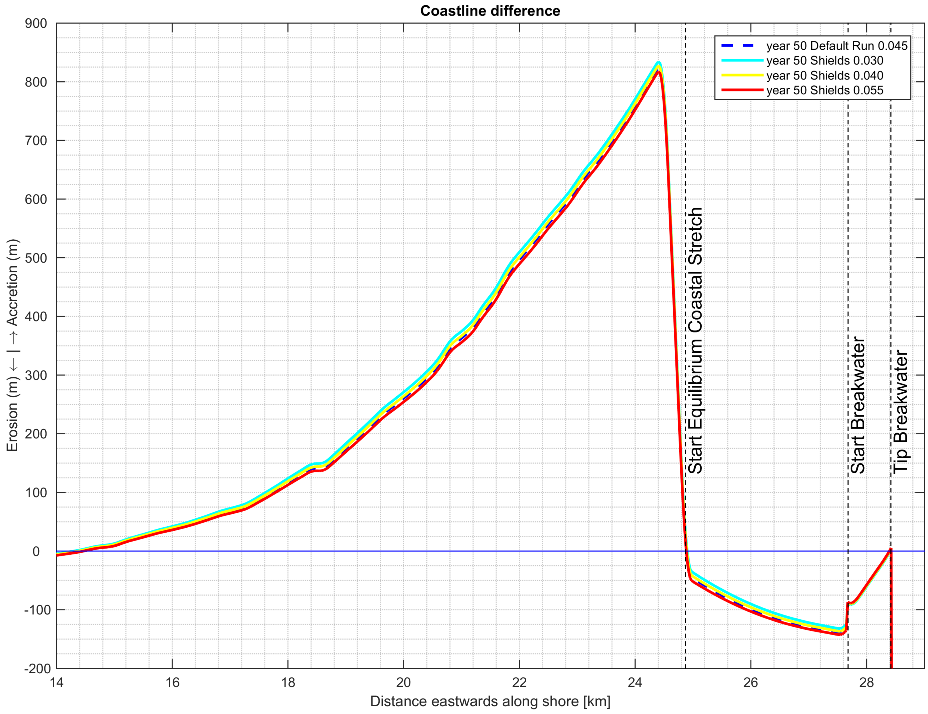
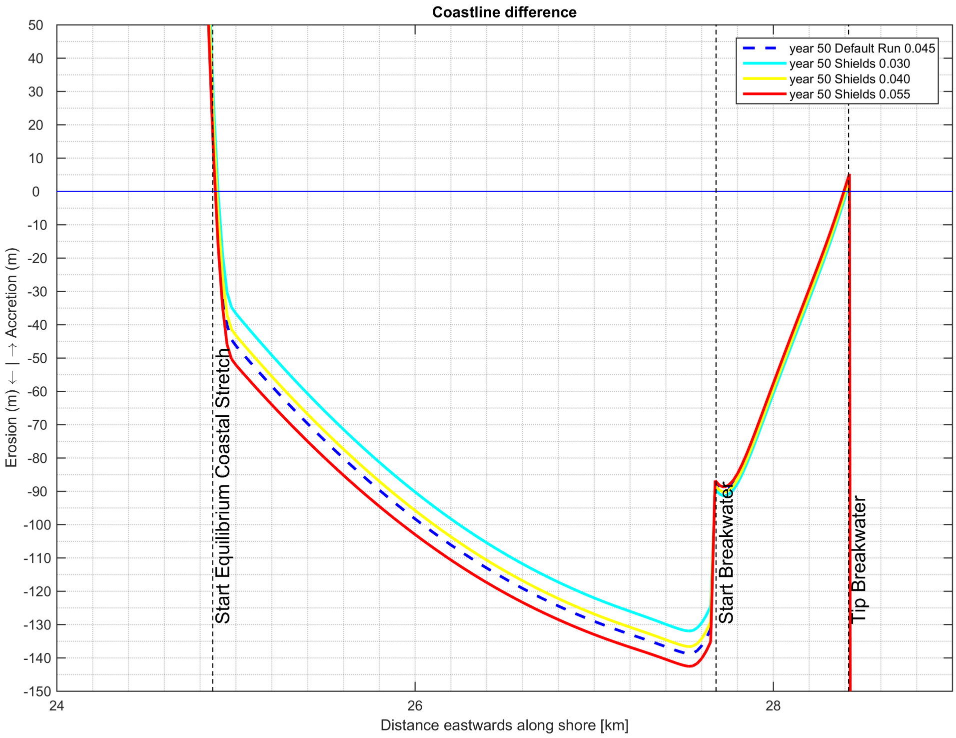


Figure H.1: LST capacity along the coastline after 50 years of modelling for different values for the Shields Criterion

The LST capacity is something which changes throughout the years due to the changing of coastal orientation along the stretches. However also the coastal development is not under big influence of the changing Shields parameter as is clearly visible from Figures H.2(a) and H.2(b). The largest difference in coastal development is around the 10 meters in difference for an ψ_c of 0.030 and the default value of 0.045.



(a) Large scale



(b) Small scale

Figure H.2: Coastline development along the area of interest after 50 years for different values for the Shields Criterion

I Dutch regulation dune design

I.1 Ultimate Required Profile

In general, for dune design at the Dutch coast certain regulations are applicable for dune design. One of these regulations beholds the definition of the Ultimate Required Profile (URP) (T. Vijverberg, 2014). This is a profile which needs to be present forms the final protection in case storm impact reduces the dune, see Figure I.1.

The determination of the URP is based on a trapezoidal shaped sand body . The crest height of the URP (h_{gp}) is based on insights regarding wave run-up. The h_{gp} can be computed using deep water parameters like wave height H_{m0} and wave period T_p in Equation I.1:

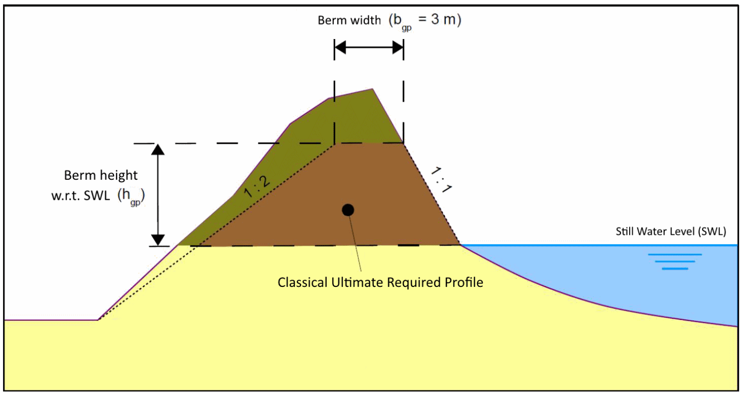


Figure I.1: Ultimate required profile, after T. Vijverberg (2014)

$$h_{gp} = 0.12 * T_p * \sqrt{H_{m0}} \tag{I.1}$$

In which:

h_{gp}	= Crest height of Ultimate Required Profile above Still Water Level [m]
T_p	= Peak wave period [s]
H_{m0}	= Significant wave height [m]

As stated in Subsection 3.3.7 the storm conditions of a 1/100 year storm are taken into account. Using a H_{m0} of 3.95 meter and a T_p of 19.30 seconds the h_{gp} becomes **4.60 meter**.

This approach is part of a larger design method developed for storms occurring at the Dutch coast. In addition, it is created for a dune with a hinterland which is required to be protected. This coast along with its storm occurrence differs heavily to the Nigerian coast and its forcing. The long wave period of the swell waves at the Nigerian coast in combination with different characteristics for the type of beach (larger grain size and steeper slopes) lead to uncertainties using the approach stated above. Concluded is that an alternative approach is used for determining a design crest height for dunes. This approach relies on the overtopping volume of waves, see Section 6.2.

J

Extra sensitivity analysis of XBeach

J.1 Introduction

In this appendix the sensitivity analysis for three parameters in XBeach are presented. First of all the crest height is examined on sensitivity with storm impact. Tbis is followed by the angle of the slope and finally two different friction formulas are compared with respect to difference in storm impact.

J.2 Crest height

The crest height was determined with by assessing overtopping volumes. To support the choice in crest height the sensitivity in storm reduction to different crest heights was researched. A range of crest heights from 5 m + MSL to 10 m + MSL has been examined. It turned out that the difference in crest height is not of relatively big impact. The impact does increase when lowering the crest height to below 5 meter however since the crest height is required to be 8.5 meter for the overtopping criteria it is outside the scope of interest. This is clearly visible from Table J.1. In Figure J.1 the results of different crest heights with the storm condition 1/100 are displayed.

On beforehand one would expect that a larger crest height would result in more storm reduction due to avalanching of more material. However, the impact of the storms is also affected by the volume of sand which is deposited on the foreshore. So if more material is reduced from the berm with a larger height in the beginning of the storm, this also affects the rest of the storm. This results in the location of the reduction line moving towards the sea as is shown in Figure J.1. The blue line being the 5 meter crest height is located the furthest into the berm. The larger the crest height becomes, the closer the reduction line is located to the beginning of the berm.

Table J.1: Result of 1/100 storm on multiple profiles with varying slopes

	Different crest heights					
Items	**5 m**	**6 m**	**7 m**	**8 m**	**9 m**	**10 m**
A0 = Reduction volume [m^3/m]	286.74	290.69	289.76	290.05	292.7	290.21
R0 = location reduction line [m]	93.69	90.69	89.69	90.19	90.19	90.19

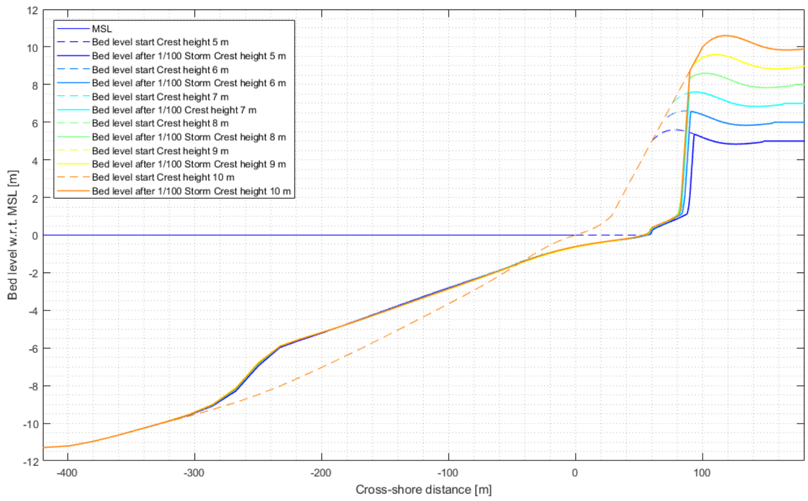


Figure J.1: Overview of results of multiple crest heights profiles with 1/100 storm conditions

The impact of increasing the height of the dune seems to be negligible above a crest height of about six meter. As shown in the Figure J.1 and Table J.1 the difference in reduction line is constant beyond this crest height. Conclusively can be stated that the impact of a different crest height on the storm reduction is negligible beyond six meter which is below the design height.

J.3 Angle of slope

The dune's slope was assumed to be in relation of 1:8 nonetheless it is interesting to find the impact of differing this relation. To find this impact different slopes within the profile have been examined. The default start profile has a relative steep beach profile up to a height of about 1 m + MSL. From this level upwards a dune profile is integrated with a slope of about 1:8. This slope is varied in XBeach from a range of 1:6 to 1:10 to check sensitivities for the reduction in cross-section for the 1/100 storm. Each slope has a coherent required estimate for the crest height as its criteria for overtopping is depended on the slope as well. These estimates were roughly estimated.

The results show that, when compared in numbers shown in Table J.2, the differences are negligible. The reduction line obviously increases with an increasing crest height but the reduction volumes of different slopes with different crest height are more or less the same. This insight supports the statement that there is no preference in slope with respect to the lowest reduction volume of sand. Conclusively is noted that the slope of 1:8 is a practical workable slope and is chosen to integrated for the design for initial construction. When the dune starts to change due to avalanching of sand and turns to a more steeper slope this is not of importance as long as the berm width is sufficiently large.

Table J.2: Result of 1/100 storm on multiple profiles with varying slopes

	Different slope angles				
Items	**1:6**	**1:7**	**1:8**	**1:9**	**1:10**
A0 = Reduction volume $[m^3/m]$	295.38	293.67	291.10	287.64	280.20
R0 = location reduction line [m]	95.19	98.69	102.44	106.69	110.19

J.4 Friction

In XBeach different set-ups are possible to use for the computations with respect to bed friction. While testing sensitivity to input parameters also the bed friction input was examined. The difference between two input formulations are examined. The first one being Manning and the second one Chézy. For both of the bed friction formulations the default values are used in XBeach while comparing their results.

In Figure J.2 and Table J.3 the results for the different friction formulations are shown. Using the default values for both the formulations leads to almost no difference in storm reduction of the profiles. From this is assumed that changing the friction formulations is not of big influence on the results given that default values are used. The default value formulation, which is Chézy, is therefore taken to be sufficient to use.

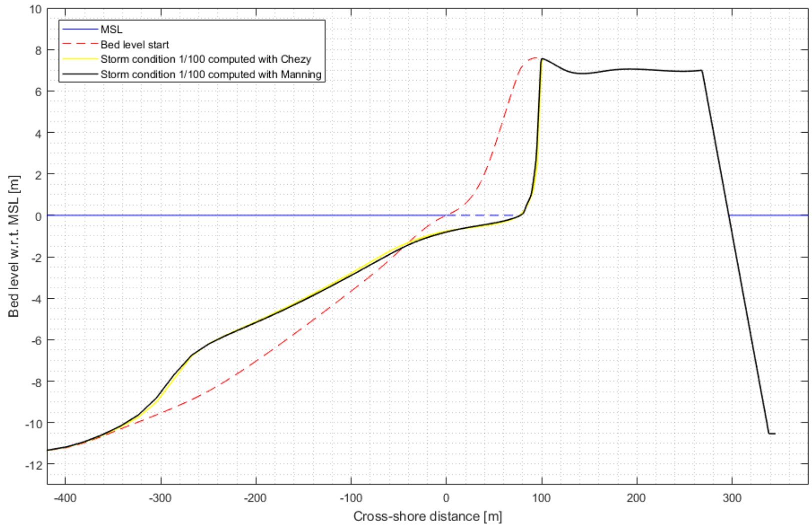


Figure J.2: Overview of results of 1/100 storm conditions on start profile with different friction set-ups

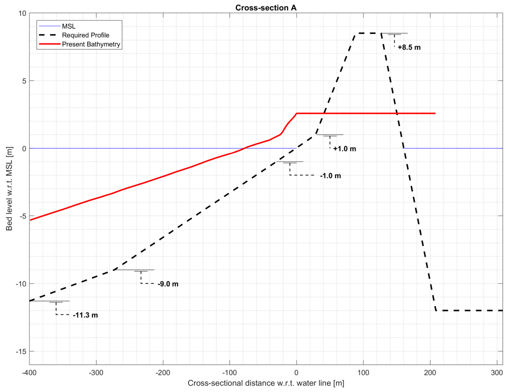
Table J.3: Result of 1/100 storm condition on start profile with different friction set-ups

	Friction	
Items	Chézy	Manning
A0 = Reduction volume [m^3/m]	381.40	376.25
R0 = location reduction line [m]	120.08	119.08

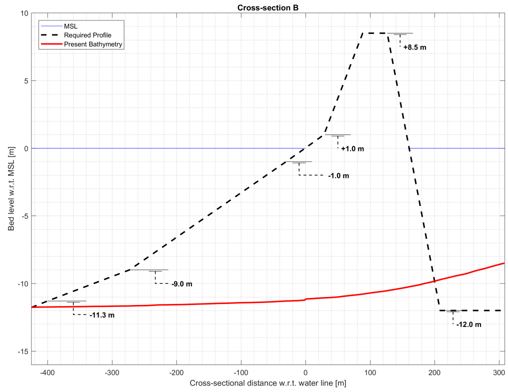
K

Detailed cross-sectional overview

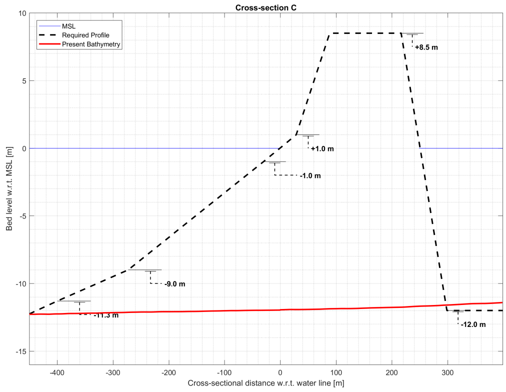
K.1 Detailed cross-sections variant 'Sand Nourishment'



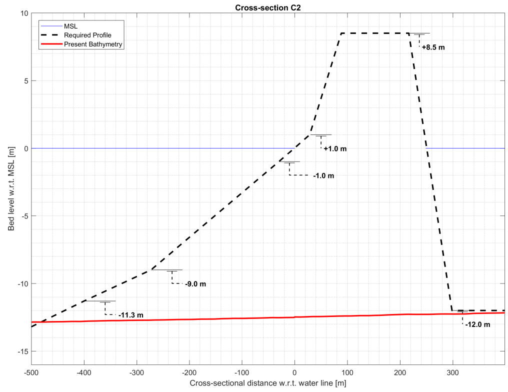
(a) Overview cross-section A variant 'Sand Nourishment'



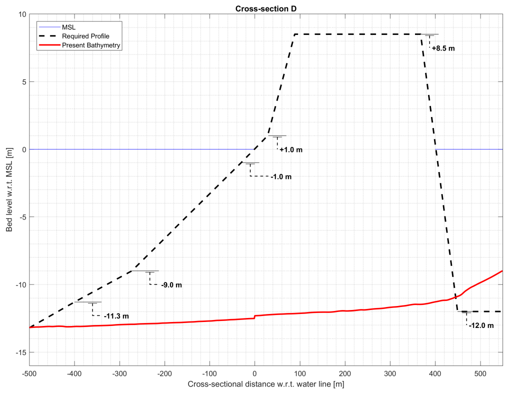
(b) Overview cross-section B variant 'Sand Nourishment'



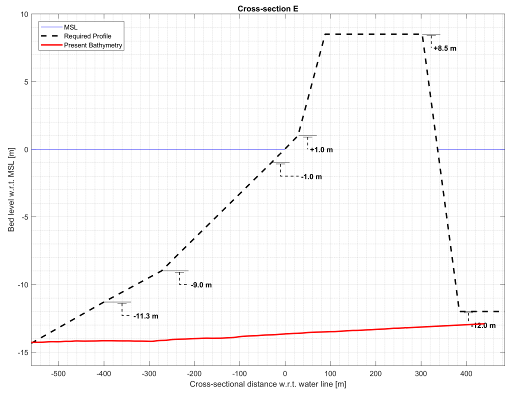
(c) Overview cross-section C variant 'Sand Nourishment'



(d) Overview cross-section C2 variant 'Sand Nourishment'



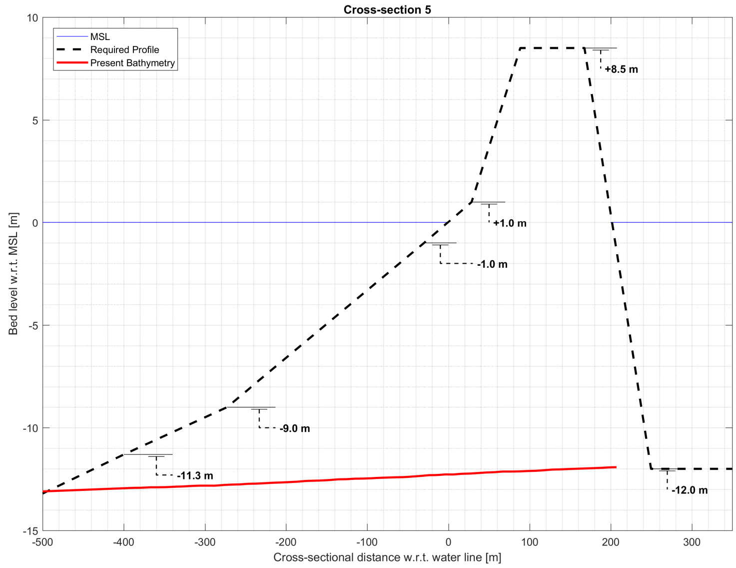
(e) Overview cross-section D variant 'Sand Nourishment'



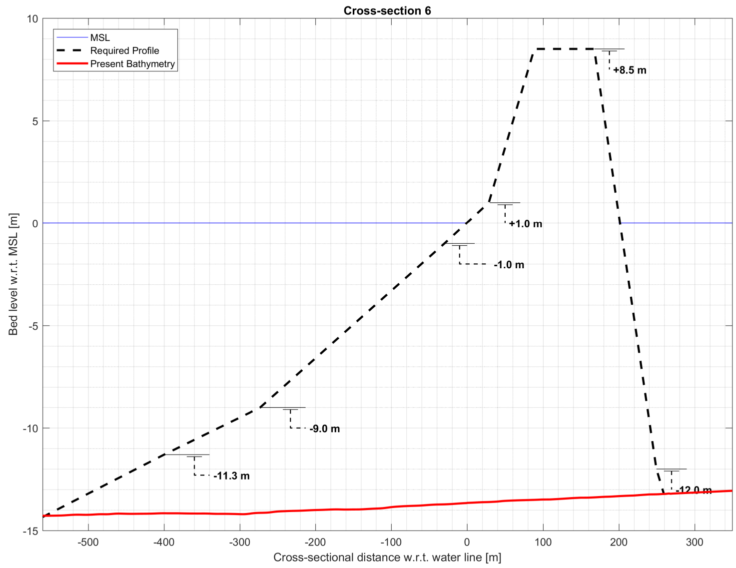
(f) Overview cross-section E variant 'Sand Nourishment'

Figure K.1: Overview cross-sections variant 'Sand Nourishment'

K.2 Detailed cross-sections variant 'Groyne'



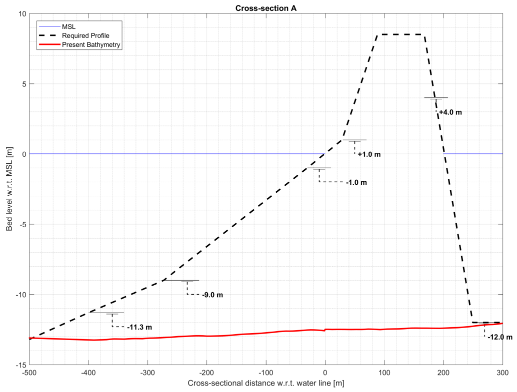
(a) Overview cross-section A variant 'Groyne'



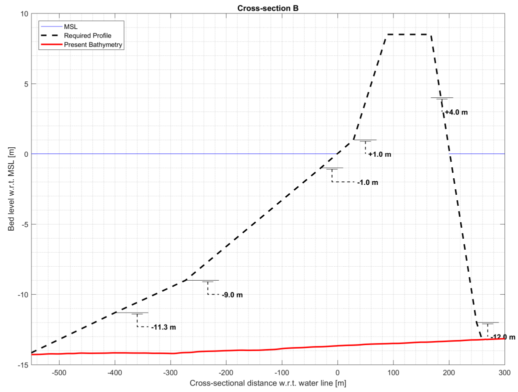
(b) Overview cross-section B variant 'Groyne'

Figure K.2: Overview cross-sections variant 'Groyne'

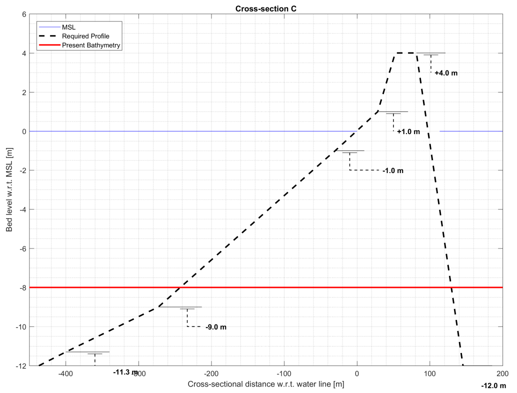
K.3 Detailed cross-sections variant 'Lagoon'



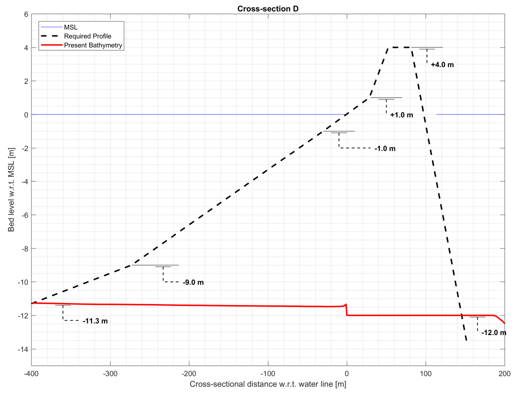
(a) Overview cross-section A variant 'Lagoon'



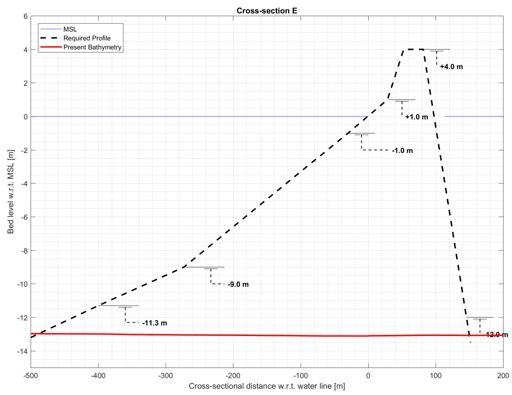
(b) Overview cross-section B variant 'Lagoon'



(c) Overview cross-section C variant 'Lagoon'



(d) Overview cross-section D variant 'Lagoon'



(e) Overview cross-section E variant 'Lagoon'

Figure K.3: Overview cross-sections variant 'Lagoon'

L

Volume and Cost estimations

L.1 Volume Estimations

L.1.1 Boundary conditions port design

In the initial port design and layout (see Figure L.1, the following dimensions apply:

Outer channel : -20.0 meter CD / 500 meter width
Inner channel : -17.0 meter CD / 350 meter width
Channel slopes : 1/4

Basin depths : -12 meter CD at the offshore supply base / -13.5 meter CD at the container terminal and general purpose terminal / -16.5 meter CD in front of the refined products jetty. The value for the offshore supply is governing for the Sand Breakwater. As this is located on the border of the sand breakwater.

Figure L.1: Local scale bathymetry 1:25000 in meter + CD (RoyalHaskoningDHV, 2014a)

L.1.2 Variant Sand Nourishment

In the variant 'Sand Nourishment' the broad idea was the maximised usage of sand instead of hard structure. The hard structures present in this variant are the short LNG breakwater and the tip of the sand breakwater which is a part of a conventional breakwater.

L.1.2.1 Estimation of Sand volume

On beforehand must be stated that it is assumed that there is no requirement of removal of sand from the ocean bed before placing the suppletions. To estimate a required volume of sand for variant 'Sand Nourishment' the sand breakwater is examined along a certain line which is the bathymetry along the water line of the sand breakwater.

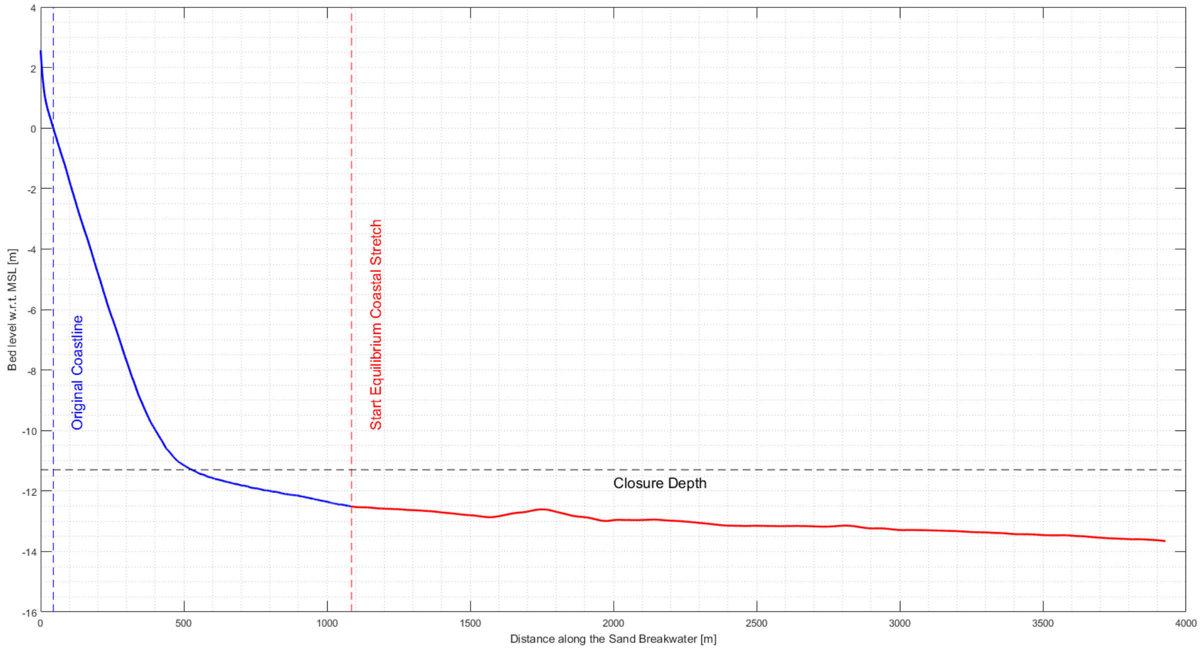


Figure L.2: Overview of bathymetry along the axis of the designed Sand Breakwater in variant 'Sand Nourishment', in Blue the stretch of coast orientated 60° to the original coastline and in Red the stretch under equilibrium orientation

Along the bathymetry line of the sand breakwater, multiple cross-sections are examined to gain a relative accurate estimation of the quantity of sand required to construct the sand breakwater. To find accurate volumes, the cross-sections are examined at the the locations of which the bathymetry in between is more or less linear.

In Figure 7.1 the locations of the cross-sections are displayed in a scaled overview of the variant. In yellow is the emerged area of the sand breakwater indicated and in blue the submerged area. There needs to be noted that on the east side of the cross-section E a toe of sand has been estimated but this toe of sand requires further research whether this toe might not scour into the approach channel around the breakwater tip.

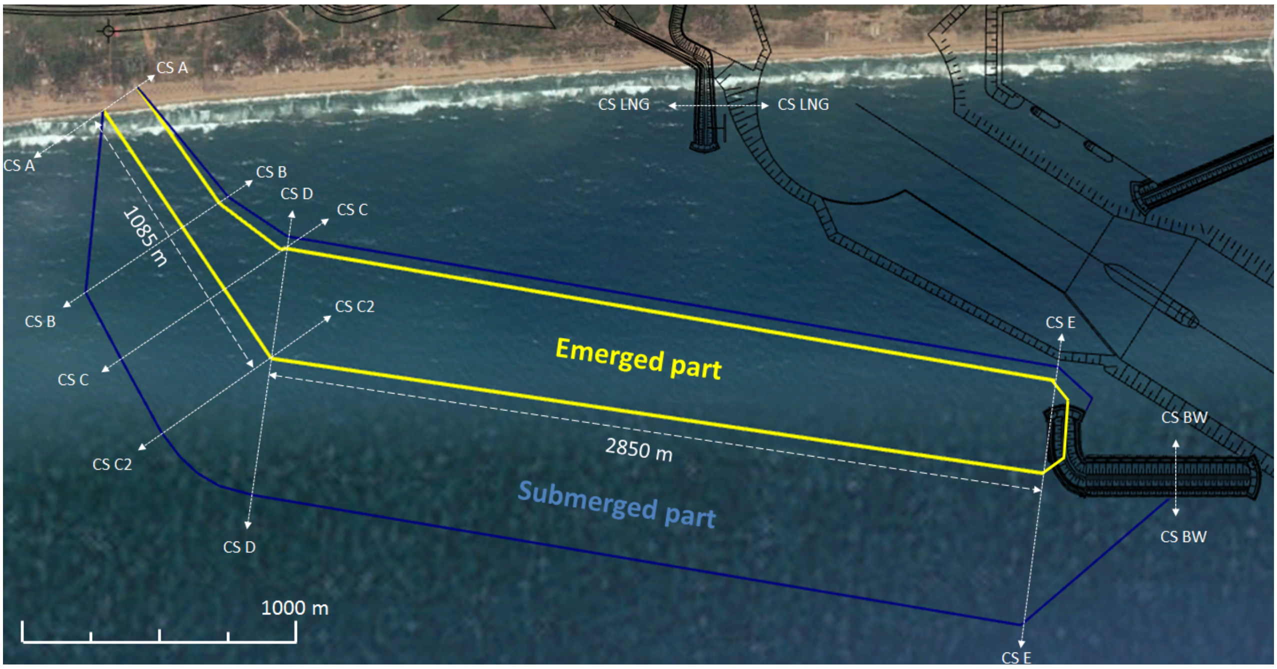


Figure L.3: Overview of variant 'Sand Nourishment' with designated cross-sections along the sand breakwater. The emerged area of the dune indicated with yellow and the submerged part of the dune indicated in blue.

Cross-section A is setted at the waterline of the original coastline. As can be seen in Figure L.2 the bathymetry along the sand breakwater is about linear to a depth of 9 meter. As is known from the profiles measured at Badagry the slope changes slightly to a depth of 11.3 meter. Although it slightly changes it is assumed here that a linear change in volume is present from 0 to 11.3 m. Consequently cross-section B is examined at a depth of 11.3 meter which is at a distance of about 530 meters from the beginning of the sand breakwater. After about 780 meters along the sand breakwater a third cross-section, C, is examined. From this length onwards the part of the sand breakwater constructed under an angle of 60° is crossed by the coastline constructed under equilibrium position due to the large extra width which was a consequence of the large coastal development throughout the years.

The last cross-section on the first piece of coastline, cross-section "C2", is examined after 1085 meter along the sand breakwater at the knick of the two coastlines. It is named "C2" as it concerns the exact same required dune cross-section as C. Only the bathymetry from the cross-section differ.

The sand volume required in the curve in the sand breakwater is difficult to estimate. It is assumed that the volume is calculated conservatively when taken that both coastlines are examined along their complete length with cross-sections. Hence overlapping on the inside of the sand breakwater and missing volume on the outside of the sand breakwater. It is assumed to be a conservative situation as the volume of sand required for the sand dune above water is much more than required under water.

For the coastline constructed under equilibrium orientation there are only two cross-sections, cross-sections D and E, examined which are located on both far ends of the coastal stretch of 2850 meter as shown in Figure L.3. For detailed cross-sectional information, see Appendix K in which all cross-sections are added.

These cross-sections firstly differ in berm widths due to the total of the required profile which was determined in Chapter 6 and the location of the cross-section with respect to the extra margin due to coastline development which was determined in Chapter 5. The cross-sections are shown in Appendix K. The coherent berm widths are shown in Table L.1.

Table L.1: Required cross-sectional dune area variant 'Sand Nourishment'

	Required Berm width [m]		
Cross-sections	Storm reduction	Coastline development	Total
A	38	0	**38**
B	38	0	**38**
C	38	90	**128**
C2	38	90	**128**
D	29	250	**279**
E	29	185	**214**

Secondly the profile of all the cross-sections differs on present bathymetry obviously due to different locations. The required cross-sectional surfaces are calculated for roughly three areas for all the cross-sections as these areas are able to linearly calculate for all stretches, see Table L.2.

Table L.2: Required cross-sectional dune area variant 'Sand Nourishment'

	Required cross-sectional surface		
Cross-sections	Above + 1 m MSL $[m^2]$	Between - 9 m and + 1 m MSL $[m^2]$	Below - 9 m MSL $[m^2]$
A	436	-	-
B	812	2850	1144
C8	1492	3787	1833
C2	1492	3787	2255
D	2624	5289	2633
E	2138	4648	3619

These cross-sectional surfaces have been used to interpolate the volume of the stretches of sand breakwater in between these cross-sections. The estimated volumes of sand required for the specific parts of the cross-sections are shown in Table L.3. From these figures a total amount of required sand for the construction of this variant is estimated. This total volume of sand required for variant 'Sand Nourishment' is estimated on 34.3 million cubic meters.

Table L.3: Volume estimation sand variant 'Sand Nourishment'

	Different stretches			
Volume sand component	A - B (530 m)	B - C (250 m)	C - C2 (305 m)	D - E (2850 m)
Above +1 m MSL $[m^3]$	$2.73 \cdot 10^5$	$3.03 \cdot 10^5$	$4.06 \cdot 10^5$	$67.67 \cdot 10^5$
Between +1 m and -9 m MSL $[m^3]$	$5.20 \cdot 10^4$	$8.72 \cdot 10^5$	$1.17 \cdot 10^6$	$1.41 \cdot 10^7$
Between -9 m MSL and bathymetry $[m^3]$	$2.06 \cdot 10^5$	$3.73 \cdot 10^5$	$6.18 \cdot 10^5$	$8.61 \cdot 10^6$
Total volume $[m^3]$	$9.96 \cdot 10^5$	$1.55 \cdot 10^6$	$2.25 \cdot 10^6$	$2.95 \cdot 10^7$

The amount of 34.3 million cubic meters concerns the rough estimated construction volume purely required for the profile as it is designed to be constructed. In practice a lot of sediment is expected to be lost during construction due to wave interaction, profiling and possible settling, etc.

The expected loss of sediment due to wave interaction, profiling and possible settling is estimated on a quarter of the original required construction volume.

This loss amounts to 8.6 million cubic meters. Taken this extra margin of required volume of sand into account the total required volume of sand for variant 'Sand Nourishment' comes down to **42.9 million cubic meters**. Although this is a large volume of sand, based on basic estimations atleast 50-60 million cubic meters should be available due to the dredging activities for the port basin and approach channel. The question remains whether all dredged sand can be re-used in the sand breakwater. These locations concern locations with mainly large grain sizes on average larger than 300 µm and therefore assumed usable for this sand breakwater.

L.1.2.2 Estimation Breakwater tip

The determination of the required volume of material for the tip of the sand breakwater is done taken into account the designs of the conventional breakwater. The tip of the sand breakwater can actually be seen as a breakwater on its own and is assumed to be constructed firstly after which the sand part can be constructed.

In Figure7.1 the breakwater is displayed in which the emerged part of the dune is shown to go accross the curved part of the breakwater. The tip of the breakwater exists out of 200 meter of breakwater following the line of the cross-section E for it needs to protect the sand breakwater along its complete coastal development during the modelled period of 50 years. The straight part of the breakwater concerns 800 meter until the approach channel.

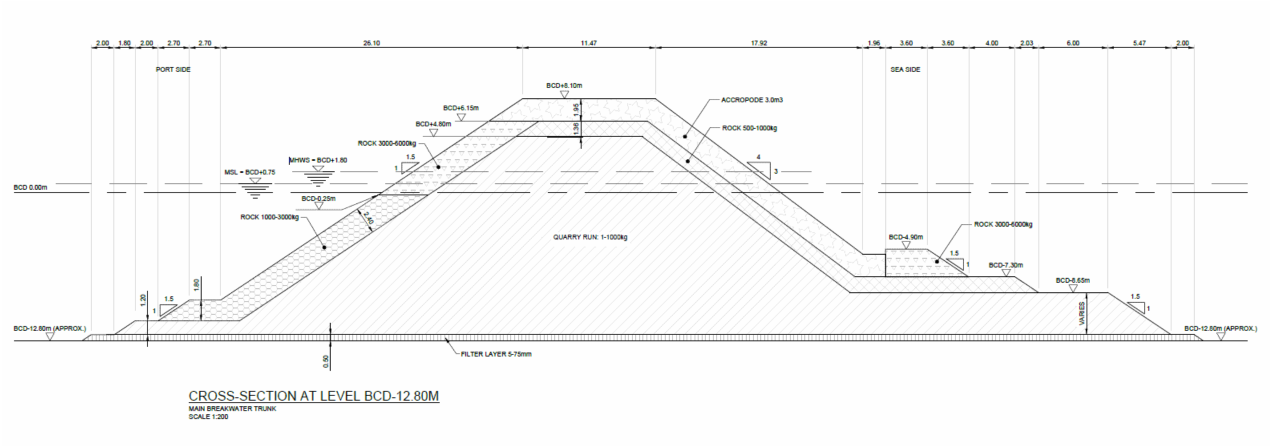


Figure L.4: Cross-sectional overview of Breakwater

Table L.4: Total required material in the tip of the breakwater variant 'Sand Nourishment'

Material	Grading	Volume $[m^3]$	Porosity [-]	Weight [Ton]
Filter later	5-75 mm	$4.77 \cdot 10^4$	0.40	$7.58 \cdot 10^4$
Quarry Run	1-1000 kg	$7.01 \cdot 10^5$	0.45	$1.02 \cdot 10^6$
Rock	500-1000 kg	$6.67 \cdot 10^4$	0.50	$8.84 \cdot 10^4$
Rock	1000-3000 kg	$5.32 \cdot 10^4$	0.50	$7.05 \cdot 10^4$
Rock	3000-6000 kg	$4.07 \cdot 10^4$	0.55	$4.85 \cdot 10^4$
Accropode	3 m^3	$6.60 \cdot 10^4$	0.60	$2.64 \cdot 10^4 m^3$

The volume estimation for the tip of the breakwater is done by calculating required volumes of stone per running meter which are extracted from the conventional design shown in Figure L.5. These volumes of rock per running meter are presented in Table L.5.

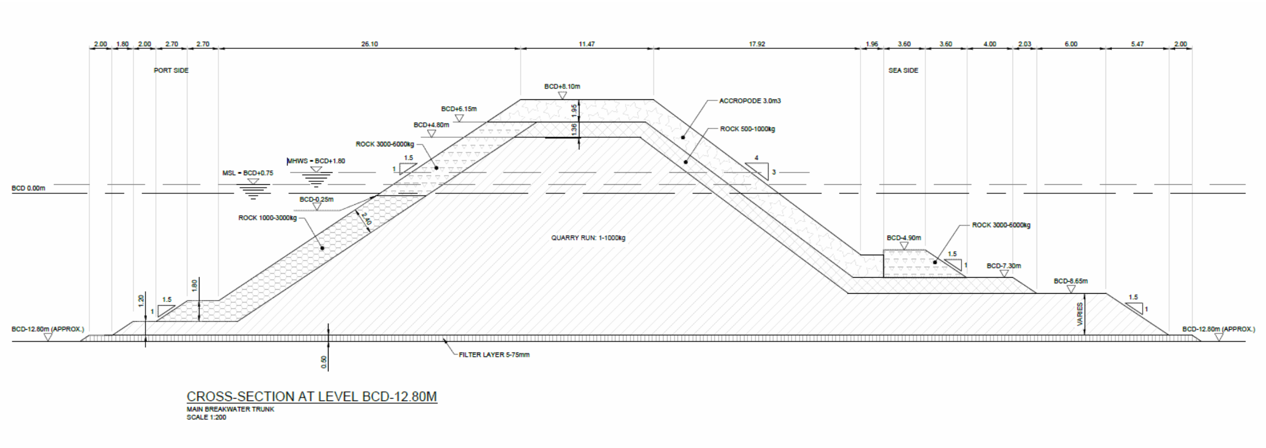


Figure L.5: Cross-sectional overview of Breakwater

Table L.5: Required material in a cross-section of the tip breakwater variant 'Sand Nourishment'

Material	Grading	Volume $[m^3/m]$
Filter later	5-75 mm	47.68
Quarry Run	1-1000 kg	701.04
Rock	500-1000 kg	66.72
Rock	1000-3000 kg	53.20
Rock	3000-6000 kg	40.65
Accropode	3 m^3	66.05

As stated before the horizontal stretch of the breakwater regards 800 meter and the stretch curved along the dunes cross-section E amounts to 200 meter. Hence assumed is that the total breakwater length amounts 1000 meter in variant 'Sand Nourishment'. A cumulative volume estimation for the total length is presented in Table L.6. For the complete length of 1000 meter the cross-sectional volumes of Table L.5 are considered. In reality the head requires more material however as this is done in all variants and the estimation of conventional design as well it is in balance further on in the cost comparison. The depth is roughly the same along the total length of the breakwater tip which supports the use of the cross-sectional design of Figure L.5. Through the porosity the weight of the required stones is calculated. A density of 2650 kg/m^3 is taken into account on average for all material originating from the quarry.

Table L.6: Total required material in the tip of the breakwater variant 'Sand Nourishment'

Material	Grading	Volume $[m^3]$	Porosity [-]	Weight [Ton]
Filter later	5-75 mm	$4.77 \cdot 10^4$	0.40	$7.58 \cdot 10^4$
Quarry Run	1-1000 kg	$7.01 \cdot 10^5$	0.45	$1.02 \cdot 10^6$
Rock	500-1000 kg	$6.67 \cdot 10^4$	0.50	$8.84 \cdot 10^4$
Rock	1000-3000 kg	$5.32 \cdot 10^4$	0.50	$7.05 \cdot 10^4$
Rock	3000-6000 kg	$4.07 \cdot 10^4$	0.55	$4.85 \cdot 10^4$
Accropode	3 m^3	$6.60 \cdot 10^4$	0.60	$2.64 \cdot 10^4 m^3$

All types of rock are able to extract from quarries within Nigeria. With earlier projects like Eko Atlantic, see Appendix A, and studies this has been done (RoyalHaskoning, 2011a). The concrete elements used in this conventional design are Accropodes. These are armour elements which can be placed in a single layer on the breakwater with a steep slope 3V:3H. The placement of these elements however is rather time consuming as certain criteria exist for the elements like enough interlocking and 'interaction surface' with other elements.

L.1.2.3 Estimation Breakwater LNG Jetty

The stretch of breakwater required for the LNG Jetty is assumed as a boundary condition in this study. This part of the port is required in all variants and assumed to be equal in all variants.

For the estimation of the materials required for the construction of this hard structure another cross-section of the conventional design is used. This concerns the cross-sectional design of the conventional breakwater at a depth of 9.25 meters MSL. The LNG jetty is assumed to have a length of 300 meters and with this length it reaches a depth of about 9.25 meters as well.

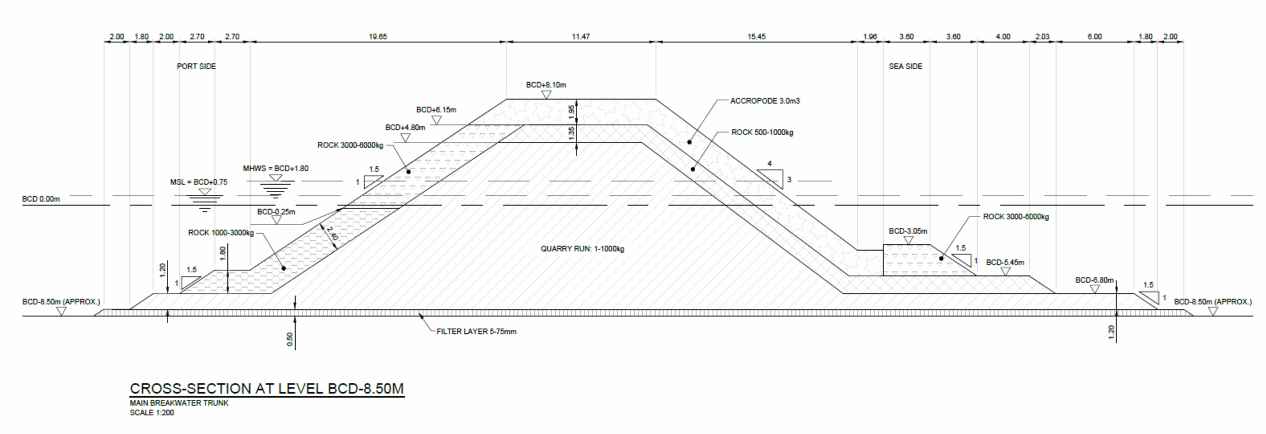


Figure L.6: Cross-sectional overview of smaller breakwater used for placement of the LNG jetty

From the cross-sectional design in Figure L.6 the required volumes of material per running meter are estimated. These volumes are shown in Table L.7.

Table L.7: Required material in a cross-section of the LNG breakwater variant 'Sand Nourishment'

Material	Grading	Volume $[m^3/m]$
Filter later	5-75 mm	41.38
Quarry Run	1-1000 kg	408.76
Rock	500-1000 kg	62.52
Rock	1000-3000 kg	42.38
Rock	3000-6000 kg	39.57
Accropode	3 m^3	60.03

In contradiction to the estimation of the required materials for the tip of the breakwater there is not a constant depth present at the LNG breakwater. This is due the fact that the cross-shore profile at 0 at the coast continues to about - 9 m. The Seen the fact that the required volume of materials decreases when moving to the coast due to a decreasing depth but also a decreasing width, a factor of $\frac{1}{3}$ is applied to the computation length of the LNG breakwater. This taken into mind the length used for volume computations concerns 100 m. With this length the total estimated material volumes are shown in Table L.8. These volumes are further used in Section 7.3.

Table L.8: Total required material in the tip of the breakwater variant 'Sand Nourishment'

Material	Grading	Volume $[m^3]$	Porosity [-]	Weight [Ton]
Filter later	5-75 mm	$4.14 \cdot 10^3$	0.40	$6.58 \cdot 10^3$
Quarry Run	1-1000 kg	$4.09 \cdot 10^4$	0.45	$5.96 \cdot 10^4$
Rock	500-1000 kg	$6.25 \cdot 10^3$	0.50	$8.28 \cdot 10^3$
Rock	1000-3000 kg	$4.24 \cdot 10^3$	0.50	$5.62 \cdot 10^3$
Rock	3000-6000 kg	$3.96 \cdot 10^3$	0.55	$4.72 \cdot 10^3$
Accropode	3 m^3	$6.00 \cdot 10^3$	0.60	$2.40 \cdot 10^3 m^3$

L.1.3 Variant Groyne

In the variant 'Groyne' the groyne was implemented to preserve the coastline constructed under equilibrium orientation by preventing LST westwards. The hard structures present in this variant are the short LNG breakwater, which is equal to the last variant, the tip of the sand breakwater and the groyne on the west side.

L.1.3.1 Estimation of Sand volume

In this variant only the coastal stretch with equilibrium orientation is present and therefore only has two cross-sections on both sides of the stretch, cross-sections A and B. In Figure L.7 the cross-sections are displayed in an overview of the variant. In yellow is the emerged area of the dune indicated and in blue is the submerged area of the sand breakwater displayed.

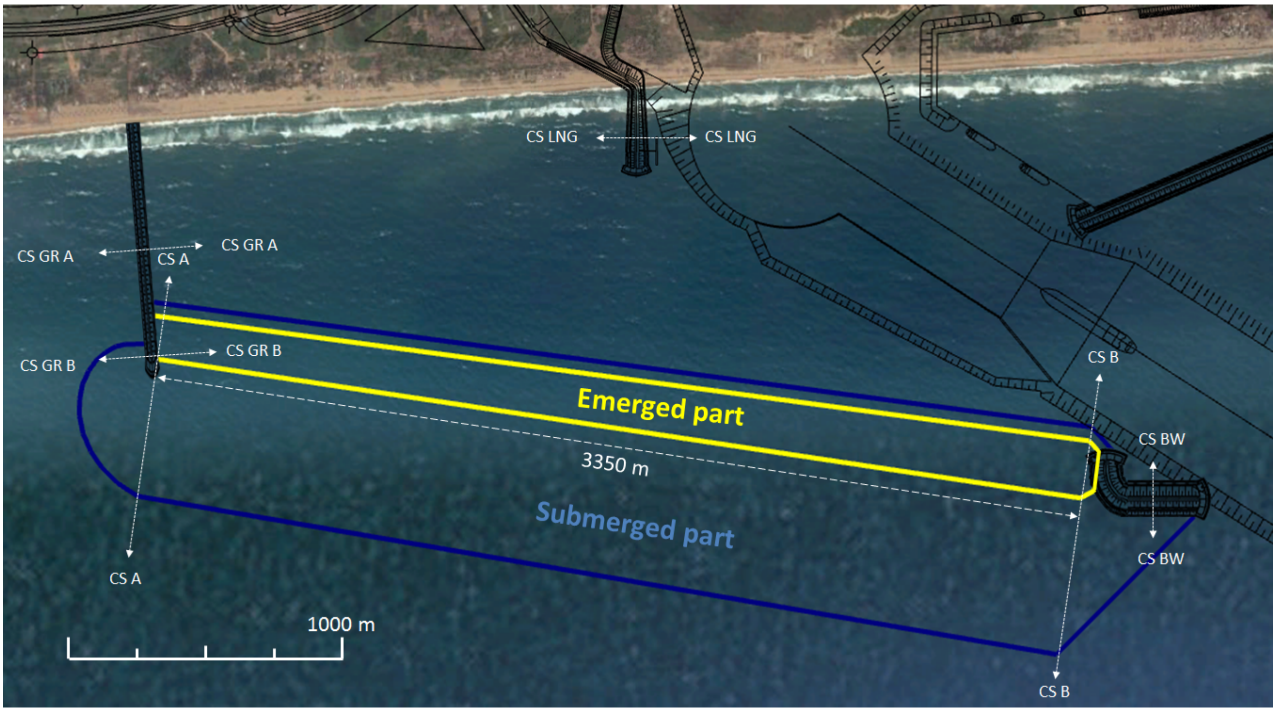


Figure L.7: Overview of variant 'Groyne' with designated cross-sections along the sand breakwater. The emerged area of the dune indicated with yellow and the submerged part of the dune indicated in blue.

The cross-sections differ only in berm width. The berm widths, see Table L.9, exist out of the storm reduction width and a coastline development width which are respectively determined in Chapter 6 and Chapter 5.

Table L.9: Required cross-sectional dune area variant 'Groyne'

	Required Berm width [m]			
Cross-sections	Storm reduction	Coastline development	Sensitivity wave angle	Total
A	29	30	20	**79**
B	29	30	20	**79**

Table L.10: Required cross-sectional dune area variant 'Groyne'

	Required cross-sectional surface [m^2]		
Cross-sections	Above + 1 m	Between - 9 m and + 1 m	Below - 9 m
A	750	2798	1886
B	1163	3348	3085

The cross-sectional surfaces from Table L.10 have been again used to interpolate the volume of sand used in the stretch. The estimated volumed of sand required for all of the parts of the cross-sections are shown in Table L.11. From these figures a total amount of required sand for this variant is estimated. This total volume of sand required for variant 'Sand Nourishment' is estimated on 21.4 million cubic meters. This is the rough estimated volume and like in the first variant a quarter of this volume is taken as a surplus for expected loss of sediment and possible settling. This quarter leads to the conservative number equal to 5.4 million cubic meters. Taken this extra margin of required volume of sand into account the total required volume of sand for variant 'Sand Nourishment' is **26.8 million cubic meters**.

Table L.11: Volume estimation sand variant 'Groyne'

Volume sand component	Stretch A - B (3350 m)
Above +1 m MSL [m^3]	$3.21 \cdot 10^6$
Between +1 m and -9 m MSL [m^3]	$1.03 \cdot 10^7$
Between -9 m MSL and bathymetry [m^3]	$7.93 \cdot 10^6$
Total volume [m^3]	$2.14 \cdot 10^7$

L.1.3.2 Estimation Breakwater tip

In variant 'Groyne' the horizontal stretch of the breakwater regards 400 meter and the stretch curved along the dunes cross-section B amounts to 100 meter. Hence assumed is that the total breakwater length amounts 500 meter in variant 'Sand Nourishment'. A cumulative volume estimation for the total length is presented in Table L.12. For the complete length of 500 meter the cross-sectional volumes of Table L.5 are considered. Just as in variant 'Sand Nourishment' the cross-sectional design of Figure L.5 is used. Consequently the same required cross-sectional surfaces are used as in variant 'Sand Nourishment', see Table L.5.

Total estimated volumes for the breakwater tip are computed using these cross-sectional surfaces. The estimated volumes for the breakwater tip of 500 meter are obviously half of what the breakwater tip in variant 'Sand Nourishment' was determined at taken into account that in that variant the length was 1000 meter.

Table L.12: Total required material in the tip of the breakwater variant 'Sand Nourishment'

Material	Grading	Volume [m^3]	Porosity [-]	Weight [Ton]
Filter later	5-75 mm	$2.38 \cdot 10^4$	0.40	$3.79 \cdot 10^4$
Quarry Run	1-1000 kg	$3.51 \cdot 10^5$	0.45	$5.11 \cdot 10^5$
Rock	500-1000 kg	$3.34 \cdot 10^4$	0.50	$4.42 \cdot 10^4$
Rock	1000-3000 kg	$2.66 \cdot 10^4$	0.50	$3.52 \cdot 10^4$
Rock	3000-6000 kg	$2.03 \cdot 10^4$	0.55	$2.42 \cdot 10^4$
Accropode	3 m^3	$3.30 \cdot 10^4$	0.60	$1.32 \cdot 10^4 m^3$

L.1.3.3 Estimation Groyne

The function of a groyne is somewhat different than that of the breakwatertip. Instead of having a berm height of + 7.35 m MSL the groyne only needs to be able to protect the dune from alongshore sediment transport on the west side of the in equilibrium orientated coastline. It also needs to stop the LST along the original coastline. Possibly due to secondary currents, e.g. currents caused by set-up differences, sediment might be transported into the port. This 'secondary LST' could occur even though there is no wave action behind the sand breakwater. Due to this different function of the hard structure, the groyne is assumed to be constructed a lot cheaper. No detailed design for the groyne is executed. Assumed is that the groyne requires half of the materials which are used in the conventional breakwater design.

In Figure L.7 two cross-sections for the groyne are shown. Cross-section Groyne A is located at the depth of about 9.25 meters w.r.t. MSL. The required material is therefore half of the required material in the cross-section of the conventional breakwater at 9.25 m MSL. These estimated volumes are displayed in table L.13.

Table L.13: Required material in cross-section A of the groyne variant 'Groyne'

Material	Grading	Volume $[m^3/m]$
Filter later	5-75 mm	20.69
Quarry Run	1-1000 kg	204.38
Rock	500-1000 kg	31.26
Rock	1000-3000 kg	21.19
Rock	3000-6000 kg	19.78
Accropode	3 m^3	30.01

The second cross-section of the groyne is located around the end of the groyne at which the second cross-sectional design of the conventional breakwater is valid w.r.t. depth. From this design again half of the required material in the cross-section is applied for the groyne shown in Table L.14.

Table L.14: Required material in cross-section B of the groyne variant 'Groyne'

Material	Grading	Volume $[m^3/m]$
Filter later	5-75 mm	23.84
Quarry Run	1-1000 kg	350.52
Rock	500-1000 kg	333.36
Rock	1000-3000 kg	26.60
Rock	3000-6000 kg	20.33
Accropode	3 m^3	33.02

The first stretch of the groyne from coast to cross-section groyne A amounts to a length of about 500 meter. However due to the fact that the depth is decreasing when moving to the coast and the required width is hence smaller at the coast is the computed length for this stretch assumed to be 200 meters. With this length the volumes for the first stretch are extrapolated using the cross-sectional information of Table reftab:volumeestimationvarianttwoLgroyneAapp. These volumes are shown in Table L.15.

Table L.15: Total required material stretch one

Material	Grading	Volume $[m^3]$	Porosity [-]	Weight [Ton]
Filter later	5-75 mm	$4.14 \cdot 10^3$	0.40	$6.58 \cdot 10^3$
Quarry Run	1-1000 kg	$4.09 \cdot 10^4$	0.45	$5.96 \cdot 10^4$
Rock	500-1000 kg	$6.26 \cdot 10^3$	0.50	$8.29 \cdot 10^3$
Rock	1000-3000 kg	$4.24 \cdot 10^3$	0.50	$5.62 \cdot 10^3$
Rock	3000-6000 kg	$3.96 \cdot 10^3$	0.55	$4.72 \cdot 10^3$
Accropode	3 m^3	$6.01 \cdot 10^3$	0.60	$2.40 \cdot 10^3 m^3$

Stretch two of the groyne is computed using both cross-section A and cross-section two and intrapolating the volume in between these cross-sections. These estimated volumes are shown in Table L.16 and further used in Section 7.3.

Table L.16: Total required material stretch two

Material	Grading	Volume $[m^3]$	Porosity [-]	Weight [Ton]
Filter later	5-75 mm	$8.20 \cdot 10^3$	0.40	$1.30 \cdot 10^4$
Quarry Run	1-1000 kg	$1.21 \cdot 10^5$	0.45	$1.76 \cdot 10^5$
Rock	500-1000 kg	$1.15 \cdot 10^4$	0.50	$1.52 \cdot 10^4$
Rock	1000-3000 kg	$9.15 \cdot 10^3$	0.50	$1.21 \cdot 10^4$
Rock	3000-6000 kg	$6.99 \cdot 10^3$	0.55	$8.34 \cdot 10^3$
Accropode	3 m^3	$1.14 \cdot 10^4$	0.60	$4.53 \cdot 10^3 m^3$

L.1.4 Variant Lagoon

The hard structures present in variant 'Lagoon' regard the smaller groyne on the west side of the pocket beach, the tip of the breakwater which is equal to the one in variant 'Groyne' and the LNG jetty breakwater. The last one is the same as in both other variants and is therefore not discussed in this subsection. The groyne on the west side, however, is estimated, along with a estimation of sand volume for the pocket beach and lower dune behind it.

L.1.4.1 Estimation of Sand volume

In Figure L.8 the overview of the design of variant 'Lagoon' is shown. In this overview on scale all the cross-sections along the sand breakwater are displayed.

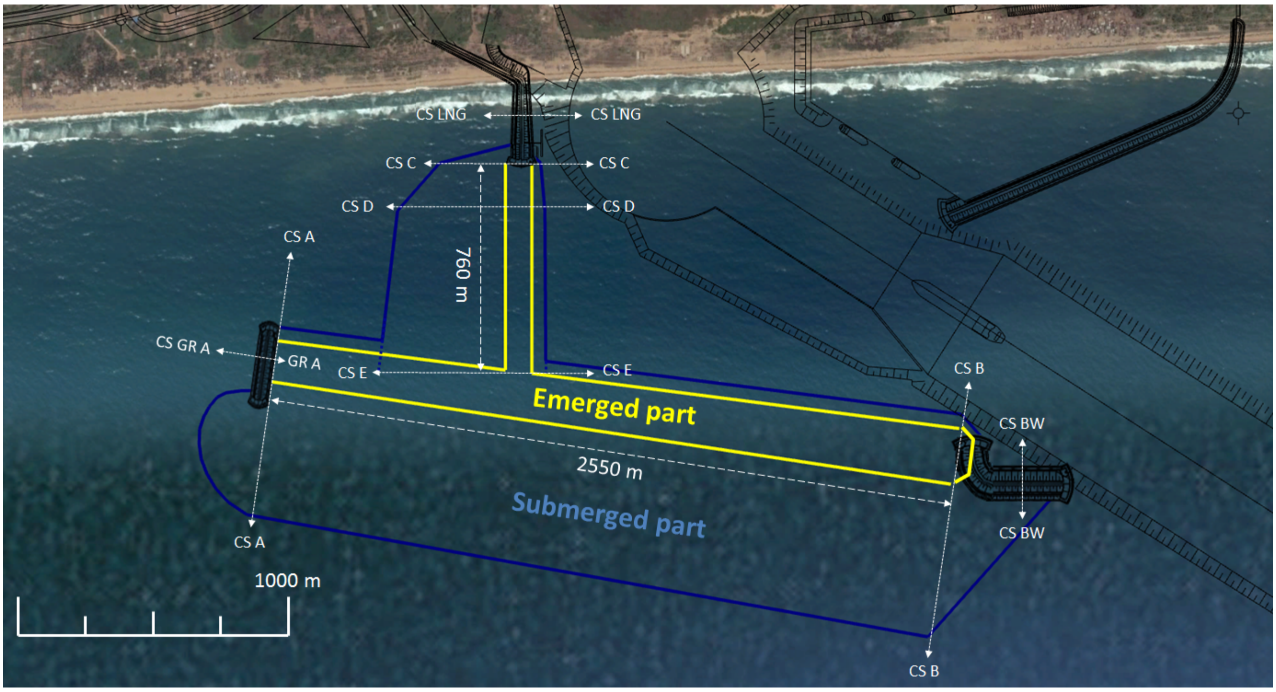


Figure L.8: Overview of variant 'Lagoon' with designated cross-sections along the sand breakwater. The emerged area of the dune indicated with yellow and the submerged part of the dune indicated in blue.

The first volume estimation concerns the pocket beach which is practically according the same system as in variant 'Groyne'. The berm widths are taken into account from Table L.9. From these widths onwards the cross-sectional volumes per meter of required materials have been computed which are shown in Table L.17.

Along with the pocket beach also the lower dune requires a certain volume of sand. This is computed from three cross-sections designated as C, D and E in Figure L.8 at respectively - 8.0 m, -11.3 m and -13.0 m MSL. These three cross-sections are also computed with respect to required volume of sand and displayed in Table L.17.

Table L.17: Required cross-sectional dune area variant 'Groyne'

	Required cross-sectional surface $[m^2]$		
Cross-sections	Above + 1 m	Between - 9 m and + 1 m	Below - 9 m
A	746	2794	1953
B	1163	3348	3085
C	212	1889	-
D	217	2343	1149
E	217	2343	2041

Three different stretches are computed with the previously cross-sections. The first stretch is the pocket beach itself. This pocket beach is 2550 meter and is sided by cross-section A and B. Stretch two is defined by cross-section C on the north side directly next to the LNG breakwater in which the dune is continuing and cross-section D on the south side after 175 meters. Stretch three is defined by cross-sections D and E on both sides of the length of 585 meter. The total volumes of the stretches are computed by interpolating the volume of sand required for the stretched in between the specific cross-sections. These volumes are shown in Table L.18.

Table L.18: Volume estimation sand variant 'Lagoon'

	Different stretches		
Volume sand component	A - B (2550 m)	C - D (175 m)	D - E (585 m)
Above +1 m MSL $[m^3]$	$2.43 \cdot 10^6$	$3.75 \cdot 10^4$	$1.27 \cdot 10^5$
Between +1 m and -9 m MSL $[m^3]$	$7.83 \cdot 10^6$	$3.70 \cdot 10^5$	$1.37 \cdot 10^6$
Between -9 m MSL and bathymetry $[m^3]$	$6.13 \cdot 10^6$	$5.03 \cdot 10^4$	$8.82 \cdot 10^5$
Subtotal volume $[m^3]$	$1.64 \cdot 10^7$	$4.58 \cdot 10^5$	$2.38 \cdot 10^6$
Total volume $[m^3]$		$21.62 \cdot 10^6$	

A total of 21.62 million cubic meters is required for the variant 'Lagoon' with respect to the sand volume. On top of this the surplus of a quarter is added leading to a grand total of 27.0 million cubic meters. These figures are used in the next section, Section 7.3.

L.1.4.2 Estimation Groyne

The groyne integrated in variant 'Lagoon' is smaller than the one in variant 'Groyne' with respect to its length. The groyne is assumed to have only a length of 300 meters. With this length it is covering the complete berm and slope under water on the back of the dune. On the front of the dune it is covering 100 meter from the waterline. This is evidently covering the zone in which LST largely takes places as this is confirmed in the width of the breaker zone in Section 4.3.

The cross-sectional information of the groyne is furthermore taken to be the same as the second cross-section of the groyne in variant 'Groyne' due to its larger depth, see Table L.14. From these surfaces a total volume estimation of all the required material for the groyne has been executed. The volumes are shown in Table L.19 and used in a rough cost comparison in Section 7.3.

Table L.19: Total required material stretch two

Material	Grading	Volume $[m^3]$	Porosity [-]	Weight [Ton]
Filter later	5-75 mm	$7.15 \cdot 10^3$	0.40	$1.14 \cdot 10^4$
Quarry Run	1-1000 kg	$1.05 \cdot 10^5$	0.45	$1.53 \cdot 10^5$
Rock	500-1000 kg	$1.00 \cdot 10^4$	0.50	$1.33 \cdot 10^4$
Rock	1000-3000 kg	$7.98 \cdot 10^3$	0.50	$1.06 \cdot 10^4$
Rock	3000-6000 kg	$6.10 \cdot 10^3$	0.55	$7.27 \cdot 10^3$
Accropode	3 m^3	$9.91 \cdot 10^3$	0.60	$5.95 \cdot 10^3 m^3$

L.1.5 Conventional design

The conventional design of the breakwater, shown in Figure L.9, has been estimated using the two cross-sections which have been used previously for the breakwater tip and the LNG jetty. The cross-sectional required volumes of rock per running meter are presented in Table L.20.

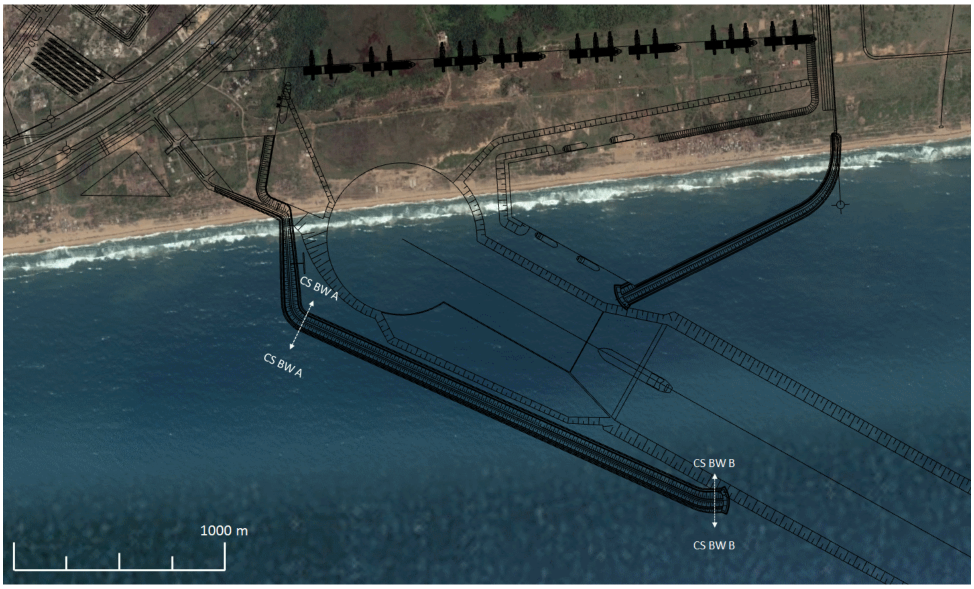


Figure L.9: Overview of the conventional breakwater design with designated cross-sections along the breakwater.

Table L.20: Required material in both cross-sections of the conventional breakwater

Material	Grading	Different stretches Stretch 1 (600 m) Volume $[m^3/m]$	Stretch 2 (2300 m) Volume $[m^3/m]$
Filter later	5-75 mm	47.68	41.38
Quarry Run	1-1000 kg	701.04	408.76
Rock	500-1000 kg	66.72	62.52
Rock	1000-3000 kg	53.20	42.38
Rock	3000-6000 kg	40.65	39.57
Accropode	3 m^3	66.05	60.03

Using these cross-sectional volumes of rock per running meter the estimated volumes of each material have been determined for two stretches of the breakwater. Assumed is that the breakwater can be determined on account of those two stretches. The first stretch starts at the coast and end at cross-section BW A. It concerns a stretch of 600 meters of breakwater with the required cross-sectional volumes per running meter from Table L.20. This cross-section has an average depth of about + 9.25 m MSL which decreases towards the coast. In addition the width of the breakwater decreases towards the coast. These two effects combined require a compensation factor for the volume calculation. A factor of 1/3 of the total length has been applied for the volume calculation. The volume estimation found for the stretch is shown in Table L.21. The second and final stretch of the breakwater contains a volume intrapolated between the two cross-sections discussed previously. This volume is intrapolated over a length of 2300 meter and shown in Table L.22. These estimations are further used for an economical cost comparison in the next section, Section L.2.

Table L.21: Total estimated required material for the first stretch of the conventional breakwater

Material	Grading	Volume $[m^3]$	Porosity [-]	Weight [Ton]
Filter later	5-75 mm	$8.28 \cdot 10^3$	0.40	$1.32 \cdot 10^4$
Quarry Run	1-1000 kg	$8.18 \cdot 10^4$	0.45	$1.19 \cdot 10^5$
Rock	500-1000 kg	$1.25 \cdot 10^4$	0.50	$1.66 \cdot 10^4$
Rock	1000-3000 kg	$8.48 \cdot 10^4$	0.50	$1.12 \cdot 10^4$
Rock	3000-6000 kg	$7.91 \cdot 10^3$	0.55	$9.44 \cdot 10^3$
Accropode	3 m^3	$1.20 \cdot 10^4$	0.60	$7.22 \cdot 10^4 m^3$

Table L.22: Total estimated required material for the second stretch of the conventional breakwater

Material	Grading	Volume $[m^3]$	Porosity [-]	Weight [Ton]
Filter later	5-75 mm	$1.10 \cdot 10^5$	0.40	$1.74 \cdot 10^5$
Quarry Run	1-1000 kg	$1.61 \cdot 10^6$	0.45	$2.35 \cdot 10^6$
Rock	500-1000 kg	$1.53 \cdot 10^5$	0.50	$2.03 \cdot 10^5$
Rock	1000-3000 kg	$1.22 \cdot 10^5$	0.50	$1.62 \cdot 10^5$
Rock	3000-6000 kg	$9.35 \cdot 10^4$	0.55	$1.11 \cdot 10^5$
Accropode	3 m^3	$1.52 \cdot 10^5$	0.60	$9.13 \cdot 10^4 m^3$

L.2 Cost Estimation

L.2.1 Variant 'Sand Nourishment'

The costs of variant 'Sand Nourishment' consist out of three components: the sand component, the tip of the breakwater and the breakwater for LNG jetty. Of which the sand component is the dune constructed with equilibrium orientation and the second stretch two the original coastline together.

L.2.1.1 Sand component

The sand component is divided into the four stretches which also have been used in the volume computations between the different cross-sections. The costs connected to these stretches are shown in Table L.23. Obviously the large quantity of sand and therefore cost is traced back in the coastal stretch constructed under equilibrium orientation due to that stretch being the longest and also located at larger depth. As displayed in the table, the surplus of a quarter of the initial construction volume is added to be on the conservative side.

Table L.23: Estimated cost of sand component of variant 'Sand Nourishment'

	Volumes			Cost	
Stretch	Construction $[m^3]$	Surplus $[m^3]$	Total $[m^3]$	Unit price	Total cost
1	$9.96 \cdot 10^5$	$2.49 \cdot 10^5$	$1.25 \cdot 10^6$	4.50 $[\$/m^3]$	$ 5,603,000
2	$1.55 \cdot 10^6$	$3.87 \cdot 10^5$	$1.93 \cdot 10^6$	4.50 $[\$/m^3]$	$ 8,703,000
3	$2.25 \cdot 10^6$	$5.62 \cdot 10^5$	$2.81 \cdot 10^6$	4.50 $[\$/m^3]$	$ 12,639,000
4	$2.95 \cdot 10^7$	$7.37 \cdot 10^6$	$3.69 \cdot 10^7$	4.50 $[\$/m^3]$	$ 165,925,000
Total					**$ 192,870,000**

L.2.1.2 Breakwater tip

The costs for the tip of the breakwater are shown in Table L.24 for the complete length of 1000 meters.

Table L.24: Estimated cost of the complete breakwater tip

Material	Unit price	Amount	Total cost
Heavy grading	57.50 [$/ton]	$1.19 \cdot 10^5$ [ton]	$ 6,840,000
Secondary armour	50.00 [$/ton]	$8.84 \cdot 10^4$ [ton]	$ 4,420,000
Quarry run	29.00 [$/ton]	$1.02 \cdot 10^6$ [ton]	$ 29,631,000
Granular filter	49.88 [$/$m^3$]	$2.86 \cdot 10^4$ [m^3]	$ 1,427,000
Concrete	500.00 [$/$m^3$]	$2.64 \cdot 10^4$ [m^3]	$ 13,176,000
Total			**$ 55,495,000**

L.2.1.3 LNG jetty

The costs for the breakwater for the LNG jetty are computed for the total length of 300 meters with the correction factor on it concerning the decreasing width and depth toward the coast, see Subsection L.1.2.3. These costs are shown in Table L.25.

Table L.25: Estimated cost of breakwater for LNG jetty

Material	Unit price	Amount	Total cost
Heavy grading	57.50 [$/ton]	$1.03 \cdot 10^4$ [ton]	$ 594,000
Secondary armour	50.00 [$/ton]	$8.28 \cdot 10^3$ [ton]	$ 414,000
Quarry run	29.00 [$/ton]	$5.96 \cdot 10^4$ [ton]	$ 1,728,000
Granular filter	49.88 [$/$m^3$]	$2.48 \cdot 10^3$ [m^3]	$ 124,000
Concrete	500.00 [$/$m^3$]	$3.61 \cdot 10^3$ [m^3]	$ 1,804,000
Total			**$ 4,664,000**

L.2.1.4 Cost summary

A cost summary of variant 'Sand Nourishment' is presented in Table L.26.

Table L.26: Cost overview of variant 'Sand Nourishment'

Component	Cost
Sand suppletion	$ 192,870,000
Breakwater tip	$ 55,495,000
Breakwater LNG jetty	$ 4,664,000
Total cost	**$ 253,029,000**

L.2.2 Variant 'Groyne'

The costs of variant 'Groyne' consist out of four components: the sand component, the tip of the breakwater, the breakwater for LNG jetty and the groyne. The sand component is only the pocket beach constructed with equilibrium coastline orientation.

L.2.2.1 Sand component

The sand component for variant 'Groyne' is presented in table L.27.

Table L.27: Estimated cost of sand component of variant 'Sand Nourishment'

	Volumes			Price	
Stretch	Construction [m^3]	Surplus [m^3]	Total [m^3]	Unit price	Total Price
1	$2.14 \cdot 10^7$	$5.36 \cdot 10^6$	$2.68 \cdot 10^7$	4.50 [$/$m^3$]	$ 120,615,000
Total					**$ 120,615,000**

L.2.2.2 Breakwater tip

The costs for the breakwater tip are not the same as in variant 'Sand Nourishment' as the length is only half the length of the previous variant. The costs are therefore also half the costs of the previous variant, see Table L.28.

Table L.28: Estimated cost of the complete breakwater tip

Material	Unit Price	Amount	Total Price
Heavy grading	57.50 [$/ton]	$5.59 \cdot 10^4$ [ton]	$ 3,420,000
Secondary armour	50.00 [$/ton]	$4.42 \cdot 10^4$ [ton]	$ 2,210,000
Quarry run	29.00 [$/ton]	$5.11 \cdot 10^5$ [ton]	$ 14,816,000
Granular filter	49.88 [$/$m^3$]	$1.43 \cdot 10^4$ [m^3]	$ 713,000
Concrete	500.00 [$/$m^3$]	$1.32 \cdot 10^4$ [m^3]	$ 6,588,000
Total			**$ 27,747,000**

L.2.2.3 Groyne

The costs of the new component, the groyne, are presented in Table L.29.

Table L.29: Estimated cost of groyne in variant 'Groyne'

Material	Unit Price	Amount	Total Price
Heavy grading	57.50 [$/ton]	$3.08 \cdot 10^4$ [ton]	$ 1,771,000
Secondary armour	50.00 [$/ton]	$2.35 \cdot 10^4$ [ton]	$ 1,175,000
Quarry run	29.00 [$/ton]	$2.35 \cdot 10^5$ [ton]	$ 6,825,000
Granular filter	49.88 [$/$m^3$]	$7.40 \cdot 10^3$ [m^3]	$ 369,000
Concrete	500.00 [$/$m^3$]	$6.93 \cdot 10^3$ [m^3]	$ 3,465,000
Total			**$ 13,605,000**

L.2.2.4 Cost summary

The total cost summary of the variant 'Groyne' is presented in Table L.30.

Table L.30: Cost overview of variant 'Groyne'

Component	Cost
Sand suppletion	$ 120,615,000
Groyne	$ 13,605,000
Breakwater tip	$ 27,747,000
Breakwater LNG jetty	$ 4,664,000
Total cost	**$ 166,025,000**

L.2.3 Variant 'Lagoon'

The costs of variant 'Lagoon' consist out of four components: the sand component, the tip of the breakwater, the breakwater for LNG jetty and the short groyne. Of which the sand component is the pocket beach constructed with equilibrium orientation along with the lower dune behind it.

L.2.3.1 Sand component

The sand component for variant 'Lagoon' is presented in table L.31.

Table L.31: Estimated cost of sand component of variant 'Lagoon'

	Volumes			Price	
Stretch	**Construction [m^3]**	**Surplus [m^3]**	**Total [m^3]**	**Unit price**	**Total Price**
1	$1.64 \cdot 10^7$	$4.69 \cdot 10^6$	$2.35 \cdot 10^7$	4.50 [$\$/m^3$]	$ 105,628,000
2	$4.58 \cdot 10^5$	$1.15 \cdot 10^5$	$5.73 \cdot 10^5$	4.50 [$\$/m^3$]	$ 2,577,000
3	$2.38 \cdot 10^6$	$5.96 \cdot 10^5$	$2.98 \cdot 10^6$	4.50 [$\$/m^3$]	$ 13,402,000
Total					**$ 121,607,000**

L.2.3.2 Groyne

Costs related to the short groyne of variant 'Lagoon' are presented in Table L.32.

Table L.32: Estimated cost of groyne variant 'Lagoon'

Material	Unit Price	Amount	Total Price
Heavy grading	57.50 [$/ton]	$1.78 \cdot 10^4$ [ton]	$ 1,026,000
Secondary armour	50.00 [$/ton]	$1.33 \cdot 10^4$ [ton]	$ 663,000
Quarry run	29.00 [$/ton]	$1.53 \cdot 10^5$ [ton]	$ 4,445,000
Granular filter	49.88 [$\$/m^3$]	$4.29 \cdot 10^3$ [m^3]	$ 214,000
Concrete	500.00 [$\$/m^3$]	$5.95 \cdot 10^3$ [m^3]	$ 2,977,000
Total			**$ 9,325,000**

L.2.3.3 Cost summary

The total cost summary of the variant 'Lagoon' is presented in Table L.33.

Table L.33: Cost overview of variant 'Lagoon'

Component	Cost
Sand suppletion coastal stretch	$ 105,628,000
Sand suppletion lower dune	$ 15,980,000
Groyne	$ 9,325,000
Breakwater tip	$ 27,747,000
Breakwater LNG jetty	$ 4,664,000
Total cost	**$ 162,737,000**

L.2.4 Conventional design

Costcomponents of the conventional design are shown below in Table L.34.

Table L.34: Estimated cost of complete conventional breakwater design

Material	Unit Price	Amount	Total Price
Heavy grading	57.50 [$/ton]	$3.06 \cdot 10^5$ [ton]	$ 17,605,000
Secondary armour	50.00 [$/ton]	$2.29 \cdot 10^5$ [ton]	$ 11,437,000
Quarry run	29.00 [$/ton]	$2.57 \cdot 10^6$ [ton]	$ 74,570,000
Granular filter	49.88 [$/$m^3$]	$1.90 \cdot 10^5$ [m^3]	$ 9,486,000
Concrete	500.00 [$/$m^3$]	$1.02 \cdot 10^5$ [m^3]	$ 51,240,000
Total			**$ 164,339,000**