Coastal protection in the Mekong Delta
Investigation of shore nourishment and mangroves as Building with Nature solutions
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Investigation of shore nourishment and mangroves as Building with Nature solutions

by

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Preface

About nine months ago I started my graduation project in order to complete the master Hydraulic Engineering at the Delft University of Technology. This research was carried out at Royal HaskoningDHV (RHDHV), as part of their project "Decision Support Tool for flood protection measures at the coastline of the southern Mekong Delta in Vietnam". A great aspect of this Building with Nature project was the opportunity to combine my two specialisations: Coastal Engineering and Dredging Engineering. I can dearly remember that at the first meeting I had with my daily supervisor Jasper Fiselier I said: "I don’t think I want to use a model to perform this research. Or perhaps a tool, only to verify some calculations." I couldn’t have been more off, and challenged myself. After a couple of months of research I was searching desperately for a model that could help me to gain insight in the wave transformation along the Mekong Delta its very gentle slopes in order to test the performance of my conceptual design. The numerical model SWASH gave some peculiar results and this became one of my real challenges. It was a puzzle I got enthusiastic about to unravel. It gave a great sense of accomplishment when I moved forward, but at the same time it took a lot of time and persistence when things were not going as expected. Although it may not have been the easiest journey at times, it was definitely one worthwhile. This journey taught me how to perform academic research, to be critical and objective, to challenge yourself, and to carry on, even when your numerical model is driving you nuts or you just get overwhelmed. I hope that my findings with respect to wave transformation on very gentle slopes can contribute to full understanding of the processes and to the improvement of numerical models and that this, in turn will contribute to the implementation of Building with Nature alternatives on mangrove coasts.

I would like to sincerely thank my daily supervisor at RHDHV, Jasper Fiselier, for his knowledge, and guidance during my graduation project. Thanks to our discussions I was never short on new ideas and topics to think about. I owe my gratitude to RHDHV, not only for having me as an intern, but to give me the unique opportunity and experience to visit my project location in Vietnam as well. Prof. dr. ir. Stefan Aarninkhof, chairman of my graduation committee, thank you for your guidance and enthusiasm in this research topic. I would like to thank prof. dr. ir. Marcel Stive for involving me in this Building with Nature project and for sharing his knowledge about coastal processes, prof. dr. ir. Han Winterwerp for helping me narrowing down my research question and goal, and keeping me focused on the aim of this research. Thank you dr. ir. Jaap van Thiel de Vries for keeping me enthusiastic and for your comments which helped me to link the model results to reality. Dr. ir. Marcel Zijlema and ir. Silke Tas thank you for your help with overcoming difficulties with SWASH. Finally, I would like to thank my family, friends and Wouter for their support. A special thanks to my parents, who have always supported me in everything I have done so far, and to my sister Petra, who read along my entire thesis duration and gave valuable advice.

Sandra Bakker
Delft, April 2017
Summary

The low lying Mekong Delta plain is very vulnerable to flooding and is densely populated, which causes pressure on coastal land use. The coastline suffers from severe erosion and mangrove degradation. Due to climate change the rate of sea level rise will increase and this will, in combination with land subsidence, lead to exposing the coastal zone to raised hydrodynamic forces and increased sediment demand. Furthermore, there is a decrease in sediment supply as a result of damming the Mekong River more upstream and extensive sand mining in the river and its delta. These two conditions, combined with mangrove degradation, are leading to severe erosion rates, up to 50 m/year, along the southern Vietnamese coast and a retreating shoreline.

Mangroves are, when in optimal condition, a natural coastal protection measure, as they serve as a sediment trap and attenuate incoming waves. This function of mangroves has been duly recognised by Vietnamese authorities. However, there is discussion regarding the way in which mangrove forests can be restored, rehabilitated and also enhanced. The ongoing discussion focusses on the required width of the mangrove forest in order to ensure a certain level of protection, and the current use of breakwaters. So far the option of a nourishment has not been considered.

Vietnam is in desperate need of a coastal protection strategy for the Mekong Delta. Due to relative sea level rise and climate change it is crucial to design adaptable and proactive coastal protection measures. Building with Nature measures, such as a nourishment, can reduce the loss of land and can contribute to mangrove rehabilitation. This research looks into the possibilities for natural and sustainable coastal defence systems including mangroves and active sediment management, which can be embedded in the Decision Support Tool (DST). The DST is developed by Royal HaskoningDHV and Deltares, and should give advice on coastal protection strategies, depending on the characteristics of a specific location along the Mekong Delta coast.

The following research question has been defined:

In what way can active sediment management be used to maintain and create a natural habitat for a self-sustaining mangrove fringe, as part of a Building with Nature solution for coastal protection in the Mekong Delta in Vietnam?

The main goal of this research is to determine quantitative knowledge rules for a self-sustaining mangrove fringe as part of the coastal profile, including the foreshore, to serve as a natural coastal protection measure.

Approach

The ongoing coastal processes leading to erosion and coastal protection problems in the project area were taken from literature, as were the requirements for a natural and healthy
habitat for a self-sustaining mangrove fringe. From this literature study boundary conditions were determined.

As a basic concept of a Building with Nature alternative for the DST, a combination of a mangrove fringe with its accompanying wave attenuating foreshore and an earth dyke is considered. The use of an earth dyke protected by mangroves is fairly common in the Mekong Delta. Earth dykes can be seen as a local cost-effective coastal protection measure that can be build with local materials, without the need of costly revetments. Furthermore, an earth dyke can easily be adapted to heavier hydraulic conditions in the future.

The Building with Nature alternative has to fulfil two requirements. First, in order to ensure the stability of the earth dyke, a maximum wave height of 0.5 m behind the mangroves, in front of the dyke, is allowed during heavy storm conditions. Secondly, for young mangroves to develop a maximal wave height of 0.5 m in front of the mangroves is assumed to be acceptable during normal conditions. Heavy storm conditions in this research consist of a storm with a return period of 100 years. Normal conditions in this research means a storm with a return period of one year, which is equal to annual storm conditions. The boundary conditions of the annual and heavy storms are expressed in offshore wave heights and periods, and a water level. An impression of the locations of the requirements and boundary conditions is given in Figure 1.

There are multiple possibilities to create a Building with Nature solution for the protection of the coastal area of the Mekong Delta. Because it was decided that the Building with Nature solution for this area will be developed in combination with a dyke without a revetment, the alternatives differ mainly in foreshore solutions. Foreshore solutions considered in this research are a mud nourishment without extra measures, a mud nourishment in combination with a permeable breakwater or Melaleuca fences or a sand-mud mixture nourishment without extra measures. The main focus was on the alternative 'a mud nourishment without extra measures', as up to now (mud) nourishments have not been considered as a possible coastal protection solution.

The requirements with respect to the maximum wave height in front of the mangrove fringe and earth dyke, and how to fulfil them using active sediment management, were translated into a conceptual profile of the foreshore: a convex-up bathymetric profile. This schematised convex-up profile was adjusted for two case study areas, Tam Giang Đông and An Minh, both suffering of severe erosion. The erosion rates are 40 m/year for Tam Giang Đông at the east coast and 20 m/year for An Minh at the west coast. In both case-study areas the present cross-shore profile is concave-up, inducing an erosion cliff. A direct effect is a foreshore with a too low bed level for mangrove restoration. The east and
west coast represent very different hydraulic conditions. The numerical model SWASH was used to translate the offshore boundary conditions into near shore conditions up to the shoreline and the earth dyke. Annual and heavy storm conditions were modelled for both case study areas, in order to test the performance of the convex-up profile with respect to the two requirements.

As there were no field measurements available to calibrate and validate the numerical model SWASH, the validity of the model has been assessed by comparing the outcomes to expected behaviour based on theory, literature, and expert judgement. During this research SWASH showed behaviour that is physically not possible. The wave height exceeded the water depth in shallow water on very gentle slopes. This behaviour was analysed by examining the variance density spectrum. It was decided to filter the wave spectrum and exclude the very low frequencies in order to improve the model results.

The model results led to proposed designs of the foreshores of the case study areas. The initial design of the convex-up profile was modified in order to reduce the nourishment volume. Possible dredging locations, dredging equipment and transport of the dredged material, and different nourishment techniques were considered. The proposed designs and design alternatives were compared with current coastal protection measures by a cost-effectiveness and cost-benefit analysis, in order to determine whether a Building with Nature strategy might be a viable alternative for the present coastal protection strategies. Furthermore, the possible use of nourishment as a coastal protection measure instead of, or in combination with hard measures, was proposed in a site specific manner for the entire southern Mekong Delta of Vietnam by comparing site-specific factors in combination with the outcome of the cost-benefit analysis.

Conclusions

Active sediment management can be used to create a convex-up profile, by means of a mud or sandy-mud nourishment, in order to create and maintain favourable hydrological and morphological conditions for young mangroves to develop, with the aim of establishing and preserving a self-sustaining mangrove fringe. A healthy self-sustaining mangrove fringe and its accompanying convex-up foreshore profile are able to attenuate waves and therefore can contribute to a Building with Nature solution for coastal protection in the Mekong Delta in Vietnam.

A convex-up profile improves the conditions for mangroves to develop, as the hydrodynamic forces in front of the mangrove fringe decrease significantly compared to the existing situation along the eroding concave-up profiles. The convex-up profile also creates a zone above mean sea level suitable for mangrove colonisation.

The minimal required mangrove width to attenuate waves is dependent on the density of the mangrove fringe and on the wave height requirement of the earth dyke. The minimal required mangrove width for the forest to be self-sustaining is dependent on the hydrology, morphology and ecology gradient. The minimal required mangrove width to fulfil the wave height requirement of the earth dyke comprises 700 m for the case study site of Tam Giang Đồng at the east coast and 350 m for An Minh at the west coast, where the hydraulic conditions are milder. These widths are sufficient for the mangrove forest to be self-sustaining.

The point at which full restoration of the convex-up profile can compete economically with a breakwater is very location specific and dependent on the foreshore profile and
erosion rate. The foreshore profile and erosion rate are coupled, as for higher erosion rates the foreshore becomes more concave-up and hence a larger nourishment volume is needed to restore the convex-up profile. For the east coast, in order to maintain the present situation, an erosion rate of 24 m/year or less was determined as the tipping point for which a nourishment can compete economically with a breakwater. For full restoration of the convex-up profile to be economically favourable the upper limit of the erosion rate is 20 m/year. For the west coast this point comprises an erosion rate of 17 m/year or less for maintaining the present situation and 10 m/year or less for full restoration of the convex-up profile.

A convex-up profile may add additional benefits to a coastal protection strategy compared to a breakwater, as it is able to continue to attenuate waves up to the shoreline. In addition, a shore nourishment adds sediment to the foreshore, hence not exacerbating an already negative local sediment balance. When the boundary conditions for a coastal protection measure are within a large bandwidth, it is not wise to implement expensive non-adaptive coastal protection measures with a long lifetime. Rather, an adaptive coastal protection strategy should be considered, such as a nourishment, that can withstand current heavy storm conditions and has the possibility to be adapted when necessary.

Building with Nature coastal protection alternatives containing a nourishment can compete with the current coastal protection strategies. The suitability of several design alternatives for different locations is based on the erosion rate, the status of the existing mangrove fringe, and the position relative to sandy-mud or mud dredging locations. The proposed design for the case study site of Tam Giang Đông, because of a high erosion rate of 40 m/year and high cliff in front of the existing mangrove fringe, is the alternative ‘mud or sandy-mud (with a steeper slope) nourishment in combination with a permeable breakwater’. Similar to the case study site at the east coast, due to the high erosion rate of 20 m/year and high cliff in front of the mangrove fringe, the alternative ‘mud nourishment in combination with a permeable breakwater’ is proposed as the most suited coastal protection measure. In other parts of the Mekong Delta area with lower erosion rates Building with Nature alternatives without permeable breakwaters are viable as well. An overview of possible coastal protection measures for the Mekong Delta is given in Figure 2.

Figure 2: Overview possible coastal protection measures Mekong Delta.
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List of Symbols and Abbreviations

Symbols

\( a \) amplitude [m]
\( a_0 \) tidal amplitude [m]
\( C_D \) drag coefficient [-]
\( c_b \) sediment concentration near the bed [kg/m\(^3\)]
\( d \) actual water depth, including surface elevation [m]
\( d_{50} \) median grain size diameter [m]
\( E \) sediment erosion rate [kg/m\(^2\)/s]
\( F \) fetch [m]
\( F \) form factor [-]
\( \hat{F} \) dimensionless fetch [-]
\( f_w \) friction factor [-]
\( f \) frequency [Hz]
\( g \) gravitational constant [m/s\(^2\)]
\( H \) wave height [m]
\( H_s \) significant wave height [m]
\( \hat{H} \) dimensionless wave height [-]
\( \hat{H}_\infty \) dimensionless wave height for fully developed sea state in deep water [-]
\( \hat{h} \) dimensionless water depth [-]
\( h \) water depth [m]
\( \Delta h \) set-up due to wind friction [m]
\( K_1 \) lunar diurnal component (amplitude) [m]
\( k_i \) imaginary wave number [-]
\( k_s \) Nikuradse roughness height [-]
\( L \) wave length [m]
\( L_r \) reference length of the flat [m]
### List of Symbols and Abbreviations

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
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<tr>
<td>$M_2$</td>
<td>lunar semi-diurnal component (amplitude)</td>
<td>[m]</td>
</tr>
<tr>
<td>$O_1$</td>
<td>lunar diurnal component (amplitude)</td>
<td>[m]</td>
</tr>
<tr>
<td>$r$</td>
<td>roughness height</td>
<td>[m]</td>
</tr>
<tr>
<td>$S_2$</td>
<td>solar semi-diurnal component (amplitude)</td>
<td>[m]</td>
</tr>
<tr>
<td>$S_d$</td>
<td>sediment deposition flux</td>
<td>[kg/m$^2$/s]</td>
</tr>
<tr>
<td>$S_e$</td>
<td>sediment erosion flux</td>
<td>[kg/m$^2$/s]</td>
</tr>
<tr>
<td>$T$</td>
<td>wave period</td>
<td>[s]</td>
</tr>
<tr>
<td>$T_p$</td>
<td>peak wave period</td>
<td>[s]</td>
</tr>
<tr>
<td>$T_r$</td>
<td>return period</td>
<td>[year]</td>
</tr>
<tr>
<td>$\hat{T}$</td>
<td>dimensionless wave period</td>
<td>[-]</td>
</tr>
<tr>
<td>$\hat{T}_\infty$</td>
<td>dimensionless wave period for fully developed sea state in deep water</td>
<td>[-]</td>
</tr>
<tr>
<td>$\hat{u}_0$</td>
<td>velocity amplitude</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$u_{wind}$</td>
<td>wind speed</td>
<td>[m/s]</td>
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<tr>
<td>$w_s$</td>
<td>fall velocity</td>
<td>[m/s]</td>
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<tr>
<td>$W$</td>
<td>width of the mangrove fringe</td>
<td>[m]</td>
</tr>
<tr>
<td>$x$</td>
<td>coordinate in x-direction (horizontal)</td>
<td>[m]</td>
</tr>
<tr>
<td>$Z$</td>
<td>bed level</td>
<td>[m]</td>
</tr>
<tr>
<td>$z$</td>
<td>coordinate in z-direction (vertical)</td>
<td>[m]</td>
</tr>
<tr>
<td>$\rho_a$</td>
<td>density of air</td>
<td>[kg/m$^3$]</td>
</tr>
<tr>
<td>$\rho_w$</td>
<td>density of water</td>
<td>[kg/m$^3$]</td>
</tr>
<tr>
<td>$\eta$</td>
<td>surface elevation</td>
<td>[m]</td>
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<tr>
<td>$\tau_w$</td>
<td>bed shear stress under waves</td>
<td>[N/m$^2$]</td>
</tr>
<tr>
<td>$\tau_b$</td>
<td>bed shear stress</td>
<td>[N/m$^2$]</td>
</tr>
<tr>
<td>$\tau_{cd}$</td>
<td>critical bed shear stress for deposition</td>
<td>[N/m$^2$]</td>
</tr>
<tr>
<td>$\tau_{ce}$</td>
<td>critical bed shear stress for erosion</td>
<td>[N/m$^2$]</td>
</tr>
<tr>
<td>$\xi_0$</td>
<td>particle excursion amplitude</td>
<td>[m]</td>
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### Abbreviations

- **ADCP**: Acoustic Doppler Current Profiler
- **CSD**: Cutter Suction Dredger
- **DST**: Decision Support Tool
- **GIZ**: Deutsche Gesellschaft für Internationale Zusammenarbeit
- **ITCZ**: Inter-Tropical Convergence Zone
<table>
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<th>Symbol</th>
<th>Abbreviation</th>
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<td>MSL</td>
<td>Mean Sea Level</td>
</tr>
<tr>
<td>NE</td>
<td>North-East</td>
</tr>
<tr>
<td>NOAA</td>
<td>National Oceanic and Atmospheric Administration</td>
</tr>
<tr>
<td>SIWRP</td>
<td>Southern Institute for Water Resources and Planning</td>
</tr>
<tr>
<td>SIWRR</td>
<td>Southern Institute of Water Resources Research</td>
</tr>
<tr>
<td>SW</td>
<td>South-West</td>
</tr>
<tr>
<td>SWASH</td>
<td>Simulating WAves till SHore</td>
</tr>
<tr>
<td>TSHD</td>
<td>Trailing Suction Hopper Dredger</td>
</tr>
<tr>
<td>WAM</td>
<td>WAve Model</td>
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<tr>
<td>WID</td>
<td>Water Injection Dredger</td>
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Chapter 1

Introduction

This thesis subject has been assigned by Royal HaskoningDHV as part of their project “Decision Support Tool for flood protection measures at the coastline of the southern Mekong Delta in Vietnam”. The project is being supported by two departments in the business line Water; Ecology, and Rivers and Coasts due to the emphasis of the subject on coastal protection measures with Building with Nature solutions. The project is executed by a consortium that involves Deltares and the local office of Royal HaskoningDHV in Vietnam. This chapter starts with some background information on the project area and the Decision Support Tool project. The next section presents the problem definition and objective of this research, followed by the last part of this chapter that contains the research approach and structure of this report.

1.1 Background information

1.1.1 Context

Mekong Delta Vietnam

The Mekong River, which has a length of 4800 km and a discharge of 470 km$^3$ per year, originates in the Tibetan Plateau at an elevation of approximately 5100 m and flows through six countries; China, Myanmar, Laos, Thailand, Cambodia and Vietnam (Lu & Siew, 2005; WWF, 2004). Figure 1.1 shows the Mekong River basin. The Mekong River has a sediment transport of about 160 million tons per year. Compared to other rivers in the same order of magnitude, such as the Yangtze or the Mississippi, the Mekong River has a smaller drainage area. When the sediment budget is compared it is noticed that this is twice the amount of the Mississippi and comparable to that of the Yangtze (Xue, Liu, DeMaster, Nguyen, & Ta, 2010). The Mekong Delta section of Vietnam is situated in the southern part of Vietnam and is the most downstream part of the lower Mekong basin. At this location the river discharges into the East Sea (Tuan, Hoanh, Miller, & Sinh, 2007). At present the Mekong River also discharges into the Gulf of Thailand, at the west coast of the delta, through a network of canals. Since 1705 several canals have been constructed for navigational purposes and national defence. These were constructed in line with the flood flow direction and situated near the borders of the inundated areas and therefore did not cause a transformation in the flood regime in the Mekong Delta (Q. Nguyen, 2000). More upstream of the lower Mekong basin, near Phnom Penh in Cambodia, the Mekong River branches into Bassac (Hau River) and Mekong (Tien River). These two branches drain the Mekong Delta situated in Vietnam (hereinafter named as the Mekong Delta), see Figure 1.2.
Chapter 1. Introduction

The Mekong Delta is also known as the Nine Dragon delta, named after its nine estuaries, and is described as a triangular surface of 40,500 km$^2$. Vietnam its total area is about 331,000 km$^2$, so the Mekong Delta represents 12 percent of its total area and it is the third largest river delta in the world (Coslett & Coslett, 2014; RoyalHaskoningDHV, Deltares, Rebel, & WUR, 2013). The delta plain is in size only exceeded by the Amazon delta and the deltas of the Ganges-Brahmaputra (Xue, Liu, DeMaster, et al., 2010). The delta region is of great importance for shrimp farming and agriculture. Nearly 50 percent of all rice produced in Vietnam is cultivated in this area, which is therefore after Ho Chi Min City and Hanoi the economically most successful region in the country (RoyalHaskoningDHV et al., 2013). The Mekong Delta is, like most deltas, densely populated and houses about 18 million people, nearly 22 percent of Vietnam’s total inhabitants (Tuan et al., 2007). In size and population the Mekong Delta is comparable to the Netherlands. Furthermore, the delta is very flat. The Mekong Delta consists of a vast floodplain with an elevation between 0 m and 4 m above Mean Sea Level (MSL) with an average of 0.8 m above MSL and is therefore extremely vulnerable to flooding and salinity intrusion (RoyalHaskoningDHV et al., 2013). The most important current coastal protection measures in Vietnam are dykes. They protect people and their houses, agricultural areas and infrastructure. Dykes can only provide the necessary protection when the dyke ring is closed. If not, the water will find its way into the hinterland which should be protected of flooding. Furthermore, they have to be well maintained and of sufficient construction quality to withstand extreme events. One of the problems in Vietnam is that the dyke
1.1. Background information

rings are not closed and/or the dykes are in poor condition. Besides this, parts of the coastal area are suffering from mangrove degradation. Mangroves are, when in optimal condition, a natural coastal protection as they serve as a sediment trap and attenuate incoming waves (Heiland, Schüttrumpf, & Schüttrumpf, 2013).

Figure 1.2: Map of the Mekong Delta in Vietnam with its 13 provinces and 9 branches (Kuenzer et al., 2013).

Motivation and relevance

The coastline of the Mekong Delta has a length of approximately 600 km and suffers from severe erosion and mangrove degradation. Due to land subsidence, climate change and consequent sea level rise the erosion has become a serious problem. The subsequent land losses are unwanted, especially because the area is densely populated and important for agricultural purposes, and therefore a coastal protection strategy is desirable. At present a coastal protection plan is in development for Vietnam, including the Mekong Delta. A Decision Support Tool (DST) will be created by Royal HaskoningDHV to support the ongoing planning process for flood protection measures at the coastline of the southern Mekong Delta. The DST encompasses various coastal types, forms of analysis and includes the scoping of different protection strategies. These protection strategies comprise of conventional dykes but will also emphasize on Building with Nature alternatives. For years the Deutsche Gesellschaft für Internationale Zusammenarbeit (GIZ) has worked on the planning and design of the coastal protection measures in the Mekong Delta. Up to now, coastal nourishments have not been considered as a possible solution. As part of the DST this research will investigate the possibilities of coastal nourishments and mangrove rehabilitation as natural coastal protection measures in the Mekong Delta. It depends on different parameters whether a solution with nourishments and/or mangroves is possible and favourable, but it is to a great extent determined by the rate of erosion. Besides being an appropriate solution when looking at the erosion-parameter and the feasibility to implement the measure in practice, the solution also requires a positive cost-benefit ratio.
Traditionally the main purpose of coastal protection measures is protection against flooding, focusing on minimizing the negative impact of the infrastructure projects in nature. Due to sea level rise and climate change it is crucial to design adaptable and proactive coastal protection measures, while working with natural processes. This sustainable concept is the Building with Nature approach. Building with Nature measures in combination with traditional, already proven solutions, can lead to cheaper and more aesthetically appealing solutions as well (De Vriend & Van Koningsveld, 2012). Most of the current implemented coastal protection measures are not suited for high erosion rates. For example, a sea dyke without a foreshore has to be outstandingly strong to withstand extreme events and hence will be expensive. When the coastal protection measure is a dyke in combination with a Building with Nature approach, such as foreshore nourishment and/or mangroves, construction costs of the dyke can be reduced due to the damping properties of the foreshore and mangroves (Verhagen & Loi, 2012). For situations with a more limited erosion rate, mangroves are also a viable protection measure.

Besides reducing the constructional costs of dykes, Building with Nature can reduce the loss of land due to dyke retreat. Furthermore, it is a more sustainable way of protecting the coast and it has positive effects on the existing ecosystem. Because of these reasons it is important to embed more natural and innovative solutions for coastal protection measures in the Decision Support Tool.

1.1.2 Project Decision Support Tool

At present the project “Integrated Coastal Management Programme” of the GIZ is carried out. They give technical support and develop an “Integrated Coastal Protection Plan”. Incentive for this project is the need for strategic advice on dykes, the level of protection and what kind of coastal defence is suitable for specific locations. For this purpose a DST is developed by Royal HaskoningDHV. In 2013 Royal HaskoningDHV completed the “Mekong Delta Plan” as part of a consortium. The aim of this plan is to contribute to realizing and maintaining a flourishing delta. This means creating a both economically viable and environmentally sustainable and climate proof delta (RoyalHaskoningDHV et al., 2013). The DST has a link to this plan and will emphasize on a strategy that includes Building with Nature measures. Furthermore, it is also based on a cost-benefit analysis taking uncertainties of climate change and socio-economic developments into account. The DST should give advice on a coastal defence strategy, depending on the characteristics of a specific location. The user first chooses the study area to elaborate on. The next step is to enter the climate change scenario; this can be adjusted while taking the next steps to compare the impact of different scenarios. After entering a scenario the user will get indicators that describe the physical system state and flood protection design indicators, describing design requirements for the flood protection. These are based on water levels, wave height, calculated water depths, level of subsidence and an indication of erosion or accretion. When this is established, a list of possible designs will be given for seaward, foreshore, embankment, and the landinward position. This could be for example a dyke, with its location and strength, in combination with mangroves. In the last step a set of indicators will be shown that describe estimated costs and benefits per solution. An example of a benefit could be: avoided damage in the protected situation compared to the original situation. The costs are divided in investments, and operation and maintenance expenditure (Royal HaskoningDHV, 2016).
1.2 Problem definition

The low lying Mekong Delta plain is very vulnerable to flooding during storm events and is densely populated, which causes pressure on coastal land use. In the last 50 years the hydrology and sediment supply in the Mekong Delta changed drastically and a tendency towards erosion instead of sedimentation of the shoreline is observed (Xue, Liu, & Ge, 2010). Due to climate change sea level rise rates will increase and this will, in combination with land subsidence, lead to a high relative sea level rise, hereby exposing the coastal zone to raised hydrodynamic forces and leading to increased sediment demand. Furthermore, there is a decrease in sediment supply as a result of damming the Mekong River more upstream and extensive sand mining upstream in the river and its delta. These two conditions, combined with mangrove degradation, are leading to severe erosion rates along the southern Vietnamese coast and a retreating shoreline.

Vietnam is in desperate need of a coastal protection strategy for the Mekong Delta. Because of the pressure on coastal land use, a natural or engineered shoreline retreat strategy is not favourable and should be avoided when alternative protection measures have potential. The Decision Support Tool is developed to compare different coastal protection measures to each other for specific locations along the Mekong Delta coast. This research will look into the possibilities for natural and sustainable coastal defence system combinations including mangroves and nourishments through active sediment management, which can be embedded in the DST.

1.3 Research question and goal

The following research question has been defined:

In what way can active sediment management be used to maintain and create a natural habitat for a self-sustaining mangrove fringe, as part of a Building with Nature solution for coastal protection in the Mekong Delta in Vietnam?

The main goal of this research is to determine quantitative knowledge rules for a self-sustaining mangrove fringe as part of the coastal profile, including the foreshore, to serve as a natural coastal protection measure.

This goal can be achieved by answering the following sub questions:

1. What are the requirements for a natural and healthy habitat of self-sustaining mangroves?
   - Determine the soil requirements, tidal requirements, drainage requirements, nutrients requirements, wave energy and related dimensions of the foreshore, mangrove belt and internal zonation.
   - Natural causes and engineering measures that influence mangroves in a negative or positive way.
   - Maximum wave climate in which they can survive as seedlings.
   - Types of artificial protection to protect seedlings and the time required for seedlings to grow mature and hence can withstand more wave energy.
   - What is the minimal sediment balance needed.

2. What are the requirements for mangroves to function as a coastal protection measure?
3. What are the thresholds for the independent boundary conditions of the design?
   - Determine optimal coastal profile and shoreline position.
   - Determine the current system conditions:
     - Determine the bathymetry of the current coastal profile and its shoreline position.
     - Sediment balance; localisation of sediment shortages.
     - Investigation of the available sediment at specific locations, mud/silt or sand.
     - List the dynamics e.g. wave field, tidal regime, wind conditions.

4. What are the possible techniques to enhance sedimentation by nourishing?

5. Are there site-specific factors that increase or diminish the possible use of nourishment as an alternative for hard measures such as a breakwater?

1.4 Research approach

This research consists of two main parts. The first part is a literature study to gain insight in the general processes of coastal erosion and to understand the ongoing coastal processes and problems in the project area. In addition, the literature study provides knowledge about possible Building with Nature alternatives for coastal protection measures, leading to a conceptual design of a Building with Nature strategy for the coastal protection of the Mekong Delta. The second part of this research is the implementation of this conceptual design in the numerical model Simulating WAves till SHore (SWASH), and to test its performance as coastal protection measure, including the fulfilment of the requirements in order to accommodate a self-sustaining mangrove fringe. Based on a performance analysis of the design and a cost-benefit analysis the optimal Building with Nature strategy can be determined.

1.5 Report structure

The report structure is in line with the research approach. Chapter 2 presents the theoretical background of coastal erosion and explains the hydrodynamics and sediment dynamics in the coastal area. In Chapter 3, the project area analysis, the case study sites of An Minh at the west coast and Tam Giang Đồng at the east coast are introduced and the theoretical concepts of Chapter 2 are applied in order to analyse both case study sites. Chapter 4 elaborates on the natural habitat of mangrove forests, their rehabilitation and what requirements a mangrove fringe has to fulfil in order to be able to serve as a coastal protection measure. In Chapter 5 the requirements of the conceptual design are listed, which leads to four possible alternatives. Chapter 6 is a summary of all boundary conditions, based on Chapter 3. In Chapter 7 the numerical model SWASH and its model set-up are introduced. Chapter 8 contains the model performance analysis of SWASH and the results of the numerical simulation of both case study areas. Furthermore, this chapter gives the proposed design and possible design alternatives. This chapter concludes with a cost-benefit analysis in order to compare the proposed design and design alternatives with existing coastal protection measures. Chapter 9 presents a discussion of this research, followed by the conclusions and recommendations in Chapter 10.
Chapter 2

Theoretical background coastal erosion

This chapter will introduce the main components influencing and leading towards coastal erosion. River deltas are dependent on sufficient sediment supply to maintain their shoreline position while balancing land subsidence and sea level rise. The position of the shoreline is determined by the process of erosion or accretion, which is dependent on hydrodynamics and sediment dynamics. The first section will explain the hydrodynamic processes occurring in the coastal zone, followed by the sediment dynamics in the second section. Most information in this chapter is based on Bosboom & Stive (2015), Holthuijsen (2007), Mehta (2014), Winterwerp & Van Kesteren (2004) and lectures of the Sediment Dynamic course CIE4308 at Delft University of Technology.

2.1 Hydrodynamics

2.1.1 Tides

Equilibrium theory

The Equilibrium theory of tides comprises the tide generating forces at which the ocean water responds instantly. Those tide generating forces are induced by the gravitational pull of the moon and the sun on the water in the ocean on earth. If there were no land masses present on earth, the global tidal wave would travel around without distortion. The presence of continents are causing distortion and are therefore responsible for global tidal variation in height and character. The tidal character is determined by the tidal constituents originating from influences of the moon, sun and the earth its rotation. Tidal constituents are indicated with a letter which presents the origin of the constituent and an index number presenting the number of daily cycles, for example M2, a semi-diurnal component influenced by the moon. The main lunar tide has a longer period than the main solar tide. This causes spring and neap tide. Spring tide occurs when the M2 and S2 component are in phase, for which the sun and moon have to be aligned. In case of neap tide, the two components are out of phase. Furthermore, the form factor F indicates a measure of the tidal character and is defined as the relative amplitude of the main diurnal components, K1 and O1, compared to the amplitude of the main semi-diurnal components mentioned above:

\[
F = \frac{K_1 + O_1}{M_2 + S_2} (2.1)
\]

The form factor distinguishes four categories, which are shown in Table 2.1.
Table 2.1: Tidal character expressed by form factor \(F\).

<table>
<thead>
<tr>
<th>Category</th>
<th>Value of form factor (F)</th>
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<tr>
<td>Semi-diurnal</td>
<td>0-0.25</td>
</tr>
<tr>
<td>Mixed, mainly semi-diurnal</td>
<td>0.25-1.5</td>
</tr>
<tr>
<td>Mixed, mainly diurnal</td>
<td>1.5-3</td>
</tr>
<tr>
<td>Diurnal</td>
<td>(&gt;3)</td>
</tr>
</tbody>
</table>

A semi-diurnal tide holds two low and two high waters a day. Due to the daily inequality the height of those two high and low waters a day are not equal. In principal the daily inequality is zero at the equator and increases with latitude, but there are exceptions caused by the presence of land masses. A semi-diurnal tide with a very large daily inequality shifts to one high and low water a day, and is called a diurnal tide. A diurnal tide has one high and low water a day. The mixed types are in between the semi-diurnal and diurnal tide (Bosboom & Stive, 2015).

Definitions
The vertical tide, or tide, is the vertical rise and fall of the water level. High tide indicates high water levels and low tide means low water levels. The falling period is the time it takes for the water level to get from highest level to the lowest water level, whereas rising period is the period of time to get from the lowest to the highest water level. During the rising and falling period the water moves in horizontal direction: the horizontal tide or tidal current. Ebb currents are directed against the propagation direction of the tidal wave. If the current velocity is in the same direction as the propagation direction, it is called a flood current. A falling or rising tide does not necessarily coincide with an ebb respectively flood current. This research refers to the vertical tide with the water level and the horizontal tide is referred to as the current. Slack water encompasses the event of tidal flow reversal and can be distinguished in high water slack and low water slack. High water slack appears at high water during flow reversal from flood to ebb, while low water slack occurs at low water when shifting from ebb to flood. The slack water period is the period in the tidal cycle in which current velocities are beneath a certain boundary limit (Bosboom & Stive, 2015).

Tidal propagation near shore
The tidal wave has a large wave length relative to its amplitude, categorizing it as a long wave. While propagating towards the coast, the tidal wave is influenced by local differences in water depth and by the presence of land masses. The propagation speed of a tidal wave is proportional to the square root of the water depth. As the tidal wave propagates into shallower water, the wave celerity decreases, resulting in a concentration of energy and hence an increase in tidal amplitude (Bosboom & Stive, 2015). Therefore, the coastal form of the Mekong Delta, with its wide and shallow foreshore, has a large effect on the tidal amplitude.

2.1.2 Waves
The definition of a wave in a time record is the profile of the surface elevation between two successive downward crossings of the mean of the surface elevations. Note that surface elevations can be negative, whereas waves can not (Holthuijsen, 2007).
2.1. Hydrodynamics

Propagation of a harmonic wave in ocean waters

In Figure 2.1 the parameters and corresponding equation of a propagating harmonic wave in ocean waters are defined. In this case a sine wave is shown. The propagation of a wave is best explained by following a point sticking to the crest of a wave. The forward speed of this point is by definition the forward speed of the wave, where the phase of the wave remains constant (Holthuijsen, 2007).

\[ \eta(x, t) = a \sin(\omega t - kx) \]

*Figure 2.1: Propagating harmonic wave and its parameters (Holthuijsen, 2007).*

Underneath the wave surface the fluid motion is coupled with the motion of the water surface. The fluid particles beneath the surface follow an orbital path. This orbital path changes with decreasing water depth, see Figure 2.2. In deep water they describe a circular path, as in shallow water they transform into an elliptical form. However, the horizontal displacement remains almost constant, whereas the vertical displacement diminishes towards the bed.

*Figure 2.2: Orbital motion of fluid particles in deep water, intermediate-depth and very shallow water (Holthuijsen, 2007).*

Wave transformation in coastal waters

Waves travelling from deep offshore waters towards the shallower coastal waters are transformed through various processes, such as shoaling, refraction, diffraction, bottom friction and wave-breaking. Shoaling is induced by decreasing water depth effecting the group velocity in longitudinal direction. When the water depth becomes less than approximately half the wave length the bottom will affect the wave. The dispersion relationship remains valid, hence the wave length will decrease because the depth decreases, causing a decreasing phase speed. The following wave is still at deeper water and thus moving at a higher speed. The next wave will catch up with the prior wave, resulting in a concentration of energy. This energy concentration increases the wave height and this whole process is known as shoaling (Bosboom & Stive, 2015). Refraction is the turning of waves due to depth variation and related variation in phase speed along the wave crest. A wave will always turn to waters with less depth and corresponding lower propagation speed (Holthuijsen,
Diffraction is the propagation of waves around obstacles inducing rapid variations in amplitude. The wave will turn to the area with a lower amplitude. Bottom friction and wave breaking are related. Bottom friction is affecting the waves when the water depth becomes less than in the order of half the wave length, and shoaling takes place. When shoaling is increasing the wave height, at a certain point the wave gets too steep, becomes unstable and starts breaking. This happens when the particle velocity exceeds the wave celerity. Depth induced breaking is dependent on the wave height - water depth ratio. At the breaking point the value of this index is between 0.78 and 0.88. Breaking waves generate surface rollers, which can be observed as the foam layer in the breaker zone, serving as a temporary storage of energy and momentum. After the breakpoint the wave energy is first converted into turbulent kinetic energy, thereafter completely dissipated by producing turbulence (Bosboom & Stive, 2015).

Wave dissipation by muddy seabeds

Multiple theories exist to explain the dissipation of waves due to interaction with muddy layers near the bed. They differ in the assumptions made regarding the rheology of the sediment. Rheology holds the combinations of elastic, plastic, viscous and porous media (Elgar & Raubenheimer, 2008). Mud rheology therefore covers the deformation and flow behaviour of mud (Mehta, 2014). Elgar (2008) observed waves propagating 1.8 km across a mudflat on the Louisiana continental shelf to estimate the frequency dependent and depth dependent wave dissipation rate function on muddy foreshores. In this research they found that the wave dissipation is dependent on water depth and therefore also on tidal level, as the dissipation is depth dependent with $h^{-3.4}$. In case of larger water depths, the dissipation is less, as is showed in Figure 2.3.

**Figure 2.3:** Wave dissipation rate, frequency and depth dependent (Elgar & Raubenheimer, 2008).
2.1. Hydrodynamics

Wave skewness and asymmetry

Waves propagating towards the shore are becoming more asymmetric until the breaking point. Shoaling induces an increasing wave height but also a gradual peaking of the wave crest and flattening of the wave trough. This is called skewness; asymmetry relative to the horizontal axis. Furthermore, relative steeping of the front of the wave until breaking results in a pitch-forward wave shape. This asymmetry relative to the vertical axis is just called asymmetry. These non-linear effects can not be described by the linear wave theory, but several theories have been developed, such as Stokes theory and Boussinesq equations, to take these processes into account. Skewness and asymmetry are important because they are essential in determining the amount of the wave-induced transport (Bosboom & Stive, 2015).

Wave boundary layer and bed shear stress

A wave boundary layer, in the order of 1-10 cm (if T < 10 s), is formed close to the bed. From the water surface down to this boundary layer most wave theories are applicable. This upper layer is not affected by the bed, whereas the boundary layer is. Within the boundary layer vorticity can be generated, which is not included in the linear wave theory or in most of other wave theories. The water in the wave boundary layer moves along the bed inducing a shear stress. The orbital velocity is therefore zero at the bed and increases towards the top of the boundary layer to the undisturbed velocity of the upper free streaming layer. Due to the thinness of the boundary layer, the velocity gradients perpendicular to the bed are large, inducing large stresses in the layer. This friction results in dissipation of wave energy and mobilisation of sediment (Bosboom & Stive, 2015). Bed shear stress can be induced by waves and currents. Because the wave boundary layer is limited in thickness and the current boundary layer most of the time is not, the bed shear stress due to waves is in general higher than those due to currents, for waves and currents with the same orbital or current velocity (Bosboom & Stive, 2015).

The occurring bed shear stress under waves \( \tau_w \) can be determined by using Equation 2.2.

\[
\tau_w = 0.5 * \rho * f_w * \hat{u}_0^2
\] (2.2)

Where:

- \( \rho \): density of water [kg/m\(^3\)]
- \( f_w \): friction factor [-]
- \( \hat{u}_0 \): velocity amplitude [m/s]

\[
f_w = \exp[-5.977 + 5.213(\hat{\xi}_0/r)^{-0.194}]
\] (2.3)

Where:

- \( \hat{\xi}_0 \): particle excursion amplitude [m]
- \( r \): roughness height [m]

\[
\hat{\xi}_0 = \frac{\hat{u}_0 * T}{2\pi}
\] (2.4)

Where:

- \( T \): wave period [s]
Wave induced set-up and currents

Waves do not only transfer energy, but momentum as well. The sum of wave momentum per unit surface area in propagation direction is found by integration over depth. By averaging over time the momentum \( q \) is defined, which is the net flux of mass between wave trough and wave crest associated with wave propagation. The mass flux is larger inside the surf zone than outside, due to surface rollers in breaking waves and the increasing character of the wave. When there is a coastline present, which can be seen as a closed boundary, a zero net mass transport throughout the vertical is present. If not, water would pile up unlimited against the coast. This net zero mass transport in the surf zone is induced by undertow, a velocity below mean trough level in offshore direction, also indicated as return currents in shoaling and breaking waves near the coast. The return current of non-breaking waves are substantially smaller than of breaking waves. Undertow is significant for offshore sediment transport, due to the rather high velocities in the lower and middle part of the water column in seaward direction. In the lower and middle part of the water column the sediment concentrations are quite high due to wave breaking. Therefore, during severe storms, undertow is responsible for intensive beach erosion (Bosboom & Stive, 2015).

Radiation stress

Radiation stress is the depth-averaged and wave-averaged flux of momentum induced by waves (Longuet-Higgins & Stewart, 1964). Variations in radiation stress between two locations act on the fluid as forces. These forces cause differences in water level and motion, such as set-down in the shoaling zone, set-up in the surf zone and forcing of a longshore current in case of waves approaching the shore at an angle. This longshore current is induced by a cross-shore gradient in radiation stress, while limited to the surfzone and is able to influence the sediment transport in that area. The Mekong Delta is known for its very gradual slopes, and therefore the cross-shore gradient will be limited. Wind induced set-up and currents, next to tidal induced currents, will be of bigger influence in the coastal area.

Wind induced set-up and currents

Wind blowing over the water surface induces a shear stress. The top layer of the water body will move in the same direction as the wind is blowing, creating a current in the upper part of the water column. If the wind blows in onshore direction, the coastline can again be seen as a closed boundary. In equilibrium situation this would indicate a zero onshore flow. To accomplish this a compensating flow in the lower water layers in offshore direction is generated. Furthermore, a water level set-up or set-down develops near shore to balance the shear stress created by the wind. The wind set-up is inversely proportional to the water depth. For shallow waters, as are present in the coastal zone of the Mekong Delta, the water can pile up to enormous heights. As for the wind induced currents, they are in particular present in the upper water layer. Their velocity decreases rapidly in vertical direction. Wind induced currents are therefore of limited influence on the sediment transport along the coast (Bosboom & Stive, 2015).
2.2 Sediment dynamics

In order to gain insight in the morphological processes occurring at the muddy Mekong Delta coast, this chapter describes the dynamics for fine sediment.

Fine sediment characteristics

Mud exists of a mixture of water, fine sand, silt, clay and organic materials (Bosboom & Stive, 2015). The cohesive behaviour of mud is determined by clay and organic materials in relation to chemical properties of water (J. Winterwerp & van Kesteren, 2004). The polymeric effects, in particular the non-ionic polymers, which are part of the organic composition, are responsible for binding the clay particles in flocs. These non-ionic polymers are able to bind the clay particles by bipolar interactions, hydrogen bonds and Van der Waal forces. Clay particles are negatively charged. In order to bind the clay particles, the repulsive force due to negative charge has to be neutralised. Cations present in the surrounding water, such as sodium ions in saline water, are able to neutralise the repulsive forces (J. Winterwerp & van Kesteren, 2004).

The combination of cohesive sediments and fluid, like water, can lead to flocculation. Flocculation is a dynamic process, dependent on factors such as space, time and organic composition. This indicates that flocs do not have to be in equilibrium state. The flocs that are formed in the process consist of open structures formed by numerous clay particles and significant water content in the order of 80 to 95 percent, increasing with floc size. Cohesive sediments have three different states of appearance; the liquid state, plastic state and solid state. The liquid limit is the transition from liquid to plastic state and represents the lowest water content at which the soil behaves mainly as liquid. The plastic limit is the transition between plastic and solid state and is defined as the limit at which the soil still can be rolled to threads with a diameter of 3 mm. If the soil is too dry, the threads will break and the soil is therefore classified as being in solid state (Verruijt & Broere, 2011). The high water content of mud mixtures is responsible for a much lower bearing capacity of mud in comparison with sand. In addition, mud mixtures are less permeable than sand, due to the small grain size. Large water content in combination with poor permeability can induce liquefaction. At this point the solid will lose its strength and stiffness and starts to behave as a fluid.

Sediment transport on mud coasts

“Sediment transport can be defined as the movement of sediment particles through a well-defined plane over a certain period of time” (Bosboom & Stive, 2015, p. 257).

The dynamic characteristics of the coast are the result of spatial gradients in net sediment rates. These spatial gradients in net sediment rates can be explained as follows: when the volume of incoming sediment equals the outgoing sediment, the coastal area is subject to sediment transport, although the net gradient in sediment transport is zero and the coastline remains stable. In case of a positive gradient in sediment transport, more sediment is going out than comes is, thus leading to erosion. Subsequently, in case of a negative gradient, the shoreline is subject to sedimentation. Gradients in longshore transport are the main cause of long-term changes of the coastline. Cross-shore transport is responsible for short-term variations, for example position and size of sand and mud bars (Bosboom & Stive, 2015).

Mangrove mud coasts in general, when in equilibrium, have a convex-up profile with
mildly sloping foreshores. Therefore, waves damp more by viscous dissipation than by wave-breaking. Waves stir up sediment from the bed. In case of non-breaking waves, the boundary layer is very thin and subsequently the area where turbulence occurs is as well. Because of this, mixing of sediment in the water column is not achieved and wave induced currents near the bed are able to transport the sediment in onshore direction. Note that this wave induced transport is much smaller than tidal current induced sediment transport. When entering the breaking zone, depth induced breaking will enhance vertical mixing. Furthermore, waves are able to weaken the soil by oscillatory movements liquefying the soil. The hydrodynamic energy needed to mobilise fine sediments from the bed is small relative to the energy needed to keep the fine sediments in suspension. Therefore, waves are dominant in the breaker zone when referring to erosion of the seabed. When the sediment has been stirred up by the waves, currents will mix and transport the sediment. Muddy coasts are known for their low wave action, otherwise fine sediments cannot accumulate. This indicates that the tidal induced currents will be larger than wave induced currents. Hence, the tidal induced currents are dominant in sediment transport at the Mekong Delta.

If fine sediments are in suspension, they are able to deposit on the bed due to settlement. Fine sediments are in general found on top of the bed, as larger particles tend to settle faster than smaller ones. This process causes sorting of sediment, called segregation. During slack water the fine sediment particles are able to settle, due to small current velocities. Next the waves will stir them up again. This process of stirring up and settling is continuous over the tidal cycle. Settling of cohesive sediment particles such as mud cannot be described by the classical settling law of Stokes. Stokes’s settling law is applicable to massive Euclidian particles, such as sand and silt. Multiple formulations for settling velocity of Euclidian particles are available in literature. Cohesive sediment particles are not massive, but are present in flocs, growing with increasing water content. As settling velocity increases with increasing particle size, it increases with floc size. Settling velocity first increases with suspended particle matter concentration by growing flocculation. At a certain point this turns into a decreasing settling velocity due to the phenomenon of hindered settling. Hindered settling occurs in high concentration mixtures when the settling velocity of a single particle is reduced due to interference with other particles. Consequently it takes particles more time to settle and this allows them to spread out over a larger distance (Dankers, 2006). The reduced settling velocity is a consequence of downward particle movements. The volume of grains going downward causes a similar volume flow going upward, and thereby slowing down the surrounding particles (Bosboom & Stive, 2015). Due to flocculation and hindered settling, the settling velocity is not constant.

To calculate the mass exchange of suspended sediment between the bed and the water column, the Partheniades-Krone formulation can be used. This formulation is based on the erosion and deposition flux. If the erosion flux exceeds the deposition flux, net sediment transport into the water column occurs. In case the deposition flux is bigger than the erosion flux, net sedimentation occurs.

\[
S_e = E \left( \frac{\tau_b}{\tau_{ce}} - 1 \right) \tag{2.5}
\]

Where:

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>(S_e)</td>
<td>sediment erosion flux</td>
<td>[kg/m²/s]</td>
</tr>
<tr>
<td>(E)</td>
<td>sediment erosion rate</td>
<td>[kg/m²/s]</td>
</tr>
<tr>
<td>(\tau_b)</td>
<td>bed shear stress</td>
<td>[N/m²]</td>
</tr>
<tr>
<td>(\tau_{ce})</td>
<td>critical bed shear stress for erosion</td>
<td>[N/m²]</td>
</tr>
</tbody>
</table>
2.2. Sediment dynamics

\[ S_d = c_b \times w_s \left( 1 - \frac{\tau_b}{\tau_{cd}} \right) \]  \hspace{1cm} (2.6)

Where:

- \( S_d \) sediment deposition flux \([\text{kg/m}^2/\text{s}]\)
- \( c_b \) sediment concentration near the bed \([\text{kg/m}^3]\)
- \( w_s \) fall velocity \([\text{m/s}]\)
- \( \tau_b \) bed shear stress \([\text{N/m}^2]\)
- \( \tau_{cd} \) critical bed shear stress for deposition \([\text{N/m}^2]\)

**Erosion of mud coasts**

In general, waves erode the muddy coast, whereas tidal currents bring sediment to the shore. However, large waves do not only erode the coast but are also responsible for stirring up sediment at the foreshore which is transported onshore direction during rising water. The sedimentation rate is the deposition rate minus the erosion rate (Mehta, 2014). Hence, the dynamic position of the shoreline is dependent on the sediment transport toward the coast by the tide and large waves, and the offshore transported sediment by large and small waves. In case of eroding mud shorelines and mudflats the concave-up profile enhances the eroding wave effects. The water depth increases, hence increasing the incoming wave height. Furthermore, sea level rise and especially land subsidence contribute to the erosion of the Mekong Delta coast as well.

When zooming in on the processes of erosion of fine sediments, the erosion depends on the properties of the eroding bed. Four erosion modes can be distinguished: surface erosion, mass erosion, fluid mud entrainment and generation of fluid mud by waves (Mehta, 2014). Surface erosion occurs when the bed scours off at the surface due to flow induced stresses, when flocs or small particles get detached. When the upper layer is eroded, the new exposed surface will swell due to negative pore water pressure gradients, which contributes to drainage. At first, water will flow into the bed and the sediment strength decreases. Secondly, surface erosion will lower the bed gradually. In case of mass erosion the undrained bed beneath the upper layer fails sporadically at a certain depth and lumps of material will erode. Entrainment of fluid mud holds the principle of aqueous particles will lift out of the fluid mud and mix with the water above it. Generation of fluid mud is a form of resuspension. Resuspension is defined as the erosion of sediment that recently has been deposited by currents or waves at the eroding bed (Mehta, 2014).
Chapter 3

Project area analysis

In this chapter the project area of the Mekong Delta and in specific two case study areas are analysed to understand the ongoing processes in the coastal system. First, the case study areas are introduced in section 3.1. Secondly, in section 3.2 the chosen lifetime and return period are given. In section 3.3 the climate conditions influencing the area of interest are treated, followed by the hydrodynamics in section 3.4, including tides, wave conditions and the water level. Finally, in section 3.5 the morphology is analysed including the bathymetry, sediment budget, sediment characteristics, and shoreline position.

3.1 Case study areas

The Mekong Delta plain is situated in the southern part of Vietnam. This research will zoom in on two case study areas. One situated at the west coast, adjacent to the Gulf of Thailand. The other is located at the east coast, facing the East Sea. The first case study site is part of the district of An Minh in Kiên Giang province at the west coast, including the communes of Vân Khánh, Vân Khánh Tây and Vân Khánh Đông, hereinafter indicated as the case study site of An Minh. This case study area comprises a coastline stretch of 13 km. The other case study site is the commune of Tam Giang Đông in the district of Năm Căn in Cà Mau province at the east coast, in this research indicated as the case study site of Tam Giang Đông. This coastline stretch has a length of 12.5 km. Locations of both case study areas are displayed in Figure 3.1. Both sites are currently subjected to coastal erosion, and in urgent need of coastal protection measures. The sites are influenced by different hydrological and morphological processes, which will be analysed in the following sections.

3.2 Lifetime and return period

The combination of the lifetime of a coastal protection structure and the return period determine the probability of failure. Currently it is common in Vietnam to choose a return period that is less than the lifetime or at best equal to the lifetime of the structure. This leads to extreme high probability of failure, as it is almost certain that the structure will have to endure heavier conditions than the level it is designed for. It is important to keep in mind that the chosen return period has to justify the value of the protected land. If the hinterland is of low economical value, it is not necessary to choose a return period of 10,000 years (as is common in the Netherlands), since this would increase the costs of the coastal protection measure extremely. Due to the increasing economical value of the hinterland in the Mekong Delta, it would be appropriate to consider a lifetime of 20, 50 or 100 years, with corresponding return periods of 10, 20, 50 and 100 years for a life time of 20 years. Return periods of 25, 50, 100 and 250 years for a lifetime of 50 years and
for a lifetime of 100 years, return periods of 50, 100, 200 and 500 years. These return periods are based on both the common practise in Vietnam as the one of the Netherlands, to introduce a lower probability of failure.

Figure 3.1: Case study areas: An Minh at the west coast and Tam Giang Đòng at the east coast (Google Earth).

3.3 Climate conditions

3.3.1 Monsoon

The climate of the Mekong Delta is humid and tropical and is controlled by two monsoon seasons: the South-West (SW) monsoon and the North-East (NE) monsoon. The SW monsoon is the summer monsoon and occurs between May and November; it is the rainy season with dominant wind directions from the South-West. The rainy season accounts for 80 percent of the annual rainfall. The NE winter monsoon, present from December to April, has prevailing winds from the North-East direction and is the dry season (Xue, Liu, & Ge, 2010).

The monsoon system is characterized by the seasonal reversal of the dominant wind directions and the ocean surface current system. This is caused in a global manner by the alternation of the atmospheric pressure system (Proske, 2010). Between May and November the air above Asia is warmer than the air above the adjacent sea and will rise, causing a low pressure zone above the continent. Wind from the sea will flow towards the land, carrying water content, hereby creating the wet SW monsoon. The average wind speeds vary between 1.8 to 4.5 m/s. During the NE monsoon the air above the continent is colder, shifting the Inter-Tropical Convergence Zone (ITCZ) southwards. Air above the sea will rise, flowing in northern direction. The low pressure zone is now present above the sea creating air transport above the continent in southern direction. Average wind speeds vary between 1.6 and 2.8 m/s during the dry NE monsoon (SIWRP, 2013). The wind and current directions are displayed in Figure 3.2.
3.3. Climate conditions

Figure 3.2: Characteristics and major components of the NE en SW monsoon season. (A) Southward shift of the ITCZ during the NE monsoon. (B) Northward shift of the ITCZ during the SW monsoon. (C) Ocean surface currents during the NE monsoon. (D) Ocean surface currents during the SW monsoon. Figure via (Proske, 2010).

3.3.2 Wind

Normal wind conditions

To investigate the wind speeds and directions at the case study locations three sources have been used: measured data in Cà Mau Airport and Can Tho Airport via Windfinder, modelled data of Meteoblue at the specific locations of both case study sites and measured data obtained by GIZ. The measured data of Windfinder is obtained since December 2013 at two monitoring stations in the Mekong Delta. Meteoblue has been archiving weather
model data since 2007 and from 2014 onwards they have started to do their calculations based on historical data starting from 1985. The data of Meteoblue has also been assessed on a monthly basis to get insight in the prevailing wind directions during the two different monsoon seasons. In Figure 3.3 the corresponding windroses of Windfinder are displayed. In Figure 3.4 and Figure 3.5 wave data of Meteoblue is showed. From these two sources it follows that during the SW monsoon the main wind directions are west to south-west and in the NE monsoon period north-east to south-east. During the SW monsoon, the east coast is on the lee side, while the west coast lies upwind. On the other hand, in the NE monsoon period, the east coast lies upwind, while the west coast is located at the lee side.

![Figure 3.3: Yearly wind direction distribution in percentage.](image)

(a) Cà Mau Airport  
(b) Can Tho Airport

![Figure 3.4: Wind distribution District of An Minh at the west coast.](image)

(a) Yearly wind distribution in m/s per direction expressed in hours.  
(b) Monthly wind distribution in m/s expressed in days.
3.3. Climate conditions

Furthermore, in 2013 two measurement campaigns were carried out to map the coastal area of Cà Mau Province by the German GIZ (Albers & Stolzenwald, 2014) in cooperation with the Vietnamese Southern Institute of Water Resources Research (SIWRR). One measurement campaign was carried out in March/April 2013 and the other in September/October 2013, to cover respectively the NE Monsoon (dry season) and the SW Monsoon (rainy season). Cà Mau Province is situated in the most southern tip of the Mekong Delta. Three sites were explored in the investigation area: Đầm Dơi District at the east coast, Ngọc Hiển District at the southern tip and U Minh District at the west coast. Their locations are shown in Figure 3.6. The three areas of investigation are all covering 23 km of coastline and are indicated with a red dotted line. These sample sites are chosen in such a way that it should be possible to collect data about bathymetry, water levels, sediment, currents, tidal regime and wave field and with this information understand all the occurring processes at the shoreline of the area of investigation (Albers & Stolzenwald, 2014). During the measurement campaigns the wind data at Côn Đảo Island was obtained. Côn Đảo Island is located 130 km east from Đầm Dơi District.

(a) Yearly wind distribution in m/s per direction expressed in hours.

(b) Monthly wind distribution in m/s expressed in days.

Figure 3.5: Wind distribution Commune Tam Giang Đông at the east coast.
In Appendix A the measured data can be found. For Đầm Dơi District, during the NE monsoon, the wind speed varies between 1.0 and 3.0 m/s, with a main direction from the east. During the SW monsoon the main wind speed was 3.0 m/s coming from south-western direction at Côn Đảo Island. However, observations in the field indicated wind coming from north-west direction. This is due to the fact that these wind conditions were influenced by a storm in the northern part of Vietnam. For Ngọc Hiển District, during the NE monsoon, the wind speed was low with maximum values of 2.0 m/s. The wind direction was shown to be variable, varying from east to north-west direction, but came in mainly from south south-west. During the SW monsoon the main direction was from the south-west with an average value of 2.5 m/s and a peak towards 5.0 m/s. In U Minh district, during the NE monsoon, wind speeds of 2.0 m/s from west-south-west were measured. The actual wind velocity in the field was higher and the direction north-west, indicated by higher measured wave heights coming from western direction. During the SW monsoon the wind velocities varied between 1.0 and 4.0 m/s coming from south-west direction.

**Storm conditions**

In Vietnam the Vietnamese code of practice for wind load on structures is used (Lien, Bich, & Thong, 2004). This code includes a wind database, comprising mean wind speed and wind pressure zones respective to different return periods. Of the in total 175 existing climate stations in Vietnam, data of four stations is available and representative for the project area: Rạch Giá in Kiên Giang Province, north of the case study area An Minh; Sóc Trăng in the province with the same name, north of the case study area of Tam Giang Đông; Cà Mau in Cà Mau Province, in between the case study areas of An Minh and Tam Giang Đông; and Côn Đảo Island for offshore wind data. The wind speed is given averaged over 10 minutes for different return periods, representing the maximum wind speed occurring once in the duration of the return period.
3.4 Hydrodynamics

In general the availability of hydrological data is minimal in Vietnam. This is mainly due to the lack of measurement campaigns, especially long term measurements, but also due to the fact that measured data is primarily owned by institutes and is therefore not always available for free or not available for third parties at all.

3.4.1 Tides

The tidal regime affects the mangrove belts and will influence the possibilities of nourishing. In order to determine to what extend the mangroves and their seedlings are exposed to the tide, more information is required. In case of investigation of the possibilities of nourishing, there are favourable time windows during the tidal cycle in which nourishing preferably should be performed. This paragraph will elaborate on tidal regimes and the local tides in the project area.

The tidal regime of the East Sea differs from the one of the Gulf of Thailand. The eastern coast is exposed to a semi-diurnal tide, including some daily inequality, and a tidal range varying between 2.5 and 3.5 m. The amplitude increases from north to south, starting at Vũng Tàu and reaching its maximum amplitude near Bạc Liêu, thereafter decreasing in southern direction until the tip of Cà Mau peninsula has been reached. The daily inequality increases in southern direction from Bạc Liêu onwards (Dung, Ngoc, Thanh, & Cam, 2013). The East Sea has two spring tide periods per month. A semi-diurnal tide holds two low and two high waters a day. Due to the daily inequality the height of those two high and low waters a day are not equal (Bosboom & Stive, 2015). The western coast along the Gulf of Thailand experiences a mixed tidal regime, consisting of semi-diurnal and diurnal characteristics. However, despite of the diurnal characteristics being predominant, at most places two high and low waters a day occur, but with significant daily inequality. In this case it means that the two high water levels are very different, while the two low water levels are almost equal. Furthermore, this type of tidal regime consists of long periods of low water levels. The Gulf of Thailand has a tidal range of 0.7 to 1.0 m and one spring tide period per month around full moon (Albers & Stolzenwald, 2014; Dung et al., 2013; Xue, Liu, DeMaster, et al., 2010). The southern tip of the Mekong Delta is located at the intersection of the two mentioned tidal regimes. At this point, where the two tidal systems meet, reduced tidal energy prevails (Xue, Liu, DeMaster, et al., 2010).

The tidal regime of the East Sea influences the one of the Gulf of Thailand (Albers & Stolzenwald, 2014). The SIWRR has measured the tidal movement at Gành Hào at the west coast and at Sông Đốc at the east coast, see Figure 3.7. Tidal forecasts for gauges at these locations are displayed in Figure 3.8 and Figure 3.9. These measurements show

<table>
<thead>
<tr>
<th>Climate station</th>
<th>Return period [year]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10</td>
</tr>
<tr>
<td>Rạch Giá</td>
<td>21.0</td>
</tr>
<tr>
<td>Sóc Trăng</td>
<td>18.6</td>
</tr>
<tr>
<td>Cà Mau</td>
<td>19.7</td>
</tr>
<tr>
<td>Côn Đảo Island</td>
<td>25.7</td>
</tr>
</tbody>
</table>

Table 3.1: Wind speed [m/s] averaged over 10 minutes for different return periods.
a tidal range in the order of 3.4 m at Gành Hào and approximately 0.7 m in Sông Đốc.

**Figure 3.7:** Indication of tidal gauges and ADCP measurement locations (Google Earth).

**Figure 3.8:** Tidal measurements for gauges at Gành Hào. Figure by SIWRR (Albers & Stolzenwald, 2014).
3.4. Hydrodynamics

Figure 3.9: Tidal measurements for gauges at Sông Đốc. Figure by SIWRR (Albers & Stolzenwald, 2014).

Tides at case study sites

The measurement campaign by GIZ in 2013, mentioned in Chapter 3.3.2, also provides tidal measurements. In Appendix B an elaborated version of the data analysis is presented. The location of the first case study site, the district of An Minh in Kiên Giang province at the west coast, is adjacent to U Minh district in the north and hence the tidal data of U Minh is representative for this site. At this part of the coast there is little tidal variation. The measurements, during the NE monsoon in March 2013, show a maximum tidal amplitude of 0.8 m. During the SW monsoon it is slightly higher with a maximum of 0.9 m.

The commune of Tam Giang Đông in the district of Năm Căn in Cà Mau province at the east coast is adjacent to Đầm Đôi, while lying between Ngọc Hiển and Đầm Đôi. Therefore some assumptions have to be made concerning its tidal regime, because the variation at this part of the coast is larger than at the west coast. In general the tidal amplitude decreases from Đầm Đôi towards Ngọc Hiển. Due to its adjacent location to Đầm Đôi the decrease in tidal amplitude will be in the order of centimetres. The maximum tidal amplitude measured during the NE monsoon in Đầm Đôi is 2.2 m, while during the SW monsoon this increases towards 2.5 m.

3.4.2 Wave conditions

The wave conditions at the Mekong Delta are determined by the monsoon winds. At the east coast the NE monsoon creates the highest waves, as for the west coast the SW monsoon is responsible for the dominating wave climate. In general the waves created by the monsoon winds are similar to swell waves; they are moderate and constant in height and direction. Storm conditions can cause much higher waves and high storm surge levels.

Offshore wave data

Offshore wave data is retrieved from three sources: offshore wind data of the climate station at Côn Đảo Island, measured offshore wave data of Bach Ho wave station and the European Centre for Medium range Weather Forecasts WAve Model (WAM). Côn Đảo Island is located 130 km east of Tam Giang Đông. Bach Ho wave station is located north of the Mekong Delta.
Chapter 3. Project area analysis

Côn Đảo Island  At Côn Đảo Island the earlier mentioned wind speeds are measured. Wind speeds can be translated into wave characteristics. First, the wind speeds averaged over 10 minutes for different return periods, representing the maximum wind speed occurring once in the duration of the return period, need to be translated into storm conditions. Generating a storm requires more than 10 minutes. An assumption is made of a representative design storm with a duration of six hours with wind speeds of 75 percent of the maximum averaged over 10 minutes. Next, the wind speeds are extrapolated using a logarithmic function to estimate the wind speeds for larger return periods.

\[ u_{\text{wind}} = 3.1645 \times \ln(T_r) + 12.4325 \]  

(3.1)

Where:

- \( u_{\text{wind}} \): wind speed [m/s]
- \( T_r \): return period [year]

<table>
<thead>
<tr>
<th>Return period [year]</th>
<th>1</th>
<th>10</th>
<th>20</th>
<th>50</th>
<th>100</th>
<th>200</th>
<th>250</th>
<th>500</th>
</tr>
</thead>
<tbody>
<tr>
<td>( u_{\text{wind}} ) [m/s]</td>
<td>12.4</td>
<td>19.7</td>
<td>21.9</td>
<td>24.8</td>
<td>27.0</td>
<td>29.2</td>
<td>29.9</td>
<td>32.1</td>
</tr>
</tbody>
</table>

Table 3.2: Wind speed for different return periods at Côn Đảo Island

The wind speeds corresponding to representative design storm conditions are translated into wave characteristics, such as wave height and period, using the approach of Young and Verhagen (1996), modified by Breugem and Holthuijssen for all sea states and water depths (Holthuijsen, 2007). The corresponding equations for the growth curves of the significant wave height and peak period are shown in equations 3.2 and 3.3.

\[ \tilde{H} = \tilde{H}_\infty \left[ \tanh(k_2 \ast \tilde{h}^m_3) \ast \tanh \left( \frac{k_1 \ast \tilde{F}^n_1}{\tanh(k_2 \ast \tilde{h}^m_3)} \right) \right]^p \]  

(3.2)

\[ \tilde{T} = \tilde{T}_\infty \left[ \tanh(k_4 \ast \tilde{h}^m_4) \ast \tanh \left( \frac{k_2 \ast \tilde{F}^n_2}{\tanh(k_4 \ast \tilde{h}^m_4)} \right) \right]^q \]  

(3.3)

Where:

- \( \tilde{H} \): dimensionless wave height [-]
- \( \tilde{H}_\infty \): dimensionless wave height for fully developed sea state in deep water [-]
- \( \tilde{T} \): dimensionless wave period [-]
- \( \tilde{T}_\infty \): dimensionless wave period for fully developed sea state in deep water [-]
- \( k_n \) and \( m_n \): tunable coefficients (determined from observations) [-]
- \( \tilde{h} \): dimensionless water depth [-]
- \( \tilde{F} \): dimensionless fetch [-]
- \( p \) and \( k \): added parameters by Young and Verhagen [-]

The coefficients that are used in Equations 3.2 and 3.3 are listed in Table 3.3.
### 3.4. Hydrodynamics

#### Table 3.3: Coefficients for the growth curve equations of the significant wave height and peak period (Holthuijsen, 2007).

<table>
<thead>
<tr>
<th>Coefficient</th>
<th>Value</th>
<th>Coefficient</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\tilde{H}_\infty$</td>
<td>0.24</td>
<td>$\tilde{T}_\infty$</td>
<td>7.69</td>
</tr>
<tr>
<td>$k_1$</td>
<td>$4.41 \times 10^{-4}$</td>
<td>$k_2$</td>
<td>$2.77 \times 10^{-7}$</td>
</tr>
<tr>
<td>$k_3$</td>
<td>0.343</td>
<td>$k_4$</td>
<td>0.10</td>
</tr>
<tr>
<td>$m_1$</td>
<td>0.79</td>
<td>$m_2$</td>
<td>1.45</td>
</tr>
<tr>
<td>$m_3$</td>
<td>1.14</td>
<td>$m_4$</td>
<td>2.01</td>
</tr>
<tr>
<td>$p$</td>
<td>0.572</td>
<td>$q$</td>
<td>0.187</td>
</tr>
</tbody>
</table>

The growth curves of the significant wave height and peak period are expressed in dimensionless parameters, indicated with a tilde. These dimensionless parameters are formed using Equations 3.4, 3.5, 3.6 and 3.7.

\[
\tilde{H} = \frac{g \ast H}{u_{wind}^2}
\]  
(3.4)

\[
\tilde{T} = \frac{g \ast T}{u_{wind}^2}
\]  
(3.5)

\[
\tilde{F} = \frac{g \ast F}{u_{wind}^2}
\]  
(3.6)

\[
\tilde{h} = \frac{g \ast h}{u_{wind}^2}
\]  
(3.7)

Where:

- $g$: gravitational constant [m/s²]
- $H$: wave height [m]
- $u_{wind}$: wind speed [m/s]
- $T$: wave period [s]
- $F$: fetch [m]
- $h$: water depth [m]

The outcome of the growth curve equations, dependent on the different wind speeds, are translated via the dimensionless parameter equations into wave height and wave period for different return periods, and can be found in Table 3.4. A fetch of 250 km is used, an assumption made in the MSc thesis of Tas, who conducted research at the same topic and project location (Tas, 2016). This assumption was based on the diameter of cyclones, and particularly the diameter in the centre of the cyclones, as the wind velocity within a cyclone differs significantly. An averaged offshore depth of 40 m is assumed, based on a bathymetric data viewer of the National Oceanic and Atmospheric Administration (NOAA) (National Oceanic and Atmospheric Administration, 2016).
Chapter 3. Project area analysis

<table>
<thead>
<tr>
<th>Return period [year]</th>
<th>Wind speed [m/s]</th>
<th>Wave height [m]</th>
<th>Wave period [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12.4</td>
<td>2.9</td>
<td>7.8</td>
</tr>
<tr>
<td>10</td>
<td>19.7</td>
<td>4.7</td>
<td>9.4</td>
</tr>
<tr>
<td>20</td>
<td>21.9</td>
<td>5.1</td>
<td>9.8</td>
</tr>
<tr>
<td>50</td>
<td>24.8</td>
<td>5.7</td>
<td>10.3</td>
</tr>
<tr>
<td>100</td>
<td>27.0</td>
<td>6.1</td>
<td>10.6</td>
</tr>
<tr>
<td>200</td>
<td>29.2</td>
<td>6.5</td>
<td>10.8</td>
</tr>
<tr>
<td>250</td>
<td>29.9</td>
<td>6.6</td>
<td>10.9</td>
</tr>
<tr>
<td>500</td>
<td>32.1</td>
<td>7.0</td>
<td>11.2</td>
</tr>
</tbody>
</table>

**Table 3.4:** Estimated significant wave height and peak period based on wind measurements at Côn Đảo Island.

**Bach Ho wave station** Bach Ho wave station is situated north of the Mekong Delta. The water depth at the wave station is 50 m and therefore the wave measurements are qualified as offshore waves, as they are not influenced by the sea bed. The data is presented by Hoang Van Huan and Nguyen Huu Nhan (2006). The measurements are conducted between 1986 and 2006, and for the waves with higher return periods data of a nearby offshore oil platform, Vietsovpetro, is used. The highest waves are coming from NE direction and their characteristics are shown in Table C.1 in Appendix C. The waves coming from the NE are normative for wave conditions at the east coast. The waves coming from SW are normative for wave conditions at the west coast.

The data is extrapolated for higher return periods. Two logarithmic functions are defined to extrapolate the wave height and the wave period, see Equations C.1 and C.2 in Appendix C. In Table 3.5 the results are displayed for the east coast.

<table>
<thead>
<tr>
<th>Return period [year]</th>
<th>Wave height [m]</th>
<th>Wave period [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.2</td>
<td>8.0</td>
</tr>
<tr>
<td>10</td>
<td>5.0</td>
<td>8.7</td>
</tr>
<tr>
<td>25</td>
<td>5.7</td>
<td>9.2</td>
</tr>
<tr>
<td>50</td>
<td>6.3</td>
<td>9.4</td>
</tr>
<tr>
<td>100</td>
<td>6.9</td>
<td>9.7</td>
</tr>
<tr>
<td>200</td>
<td>7.4</td>
<td>9.9</td>
</tr>
<tr>
<td>250</td>
<td>7.6</td>
<td>10.0</td>
</tr>
<tr>
<td>500</td>
<td>8.1</td>
<td>10.3</td>
</tr>
</tbody>
</table>

**Table 3.5:** Estimated wave height and wave period after extrapolation for different return periods measured at Bach Ho wave station for the east coast.

**WAM** The European Centre for Medium range Weather Forecasts uses and develops the WAVE Model (WAM). It is coupled to the atmospheric model, but it is also possible to run it in standalone mode. The WAM resolves 30 wave frequencies and 24 wave directions per node of the 1.0°x1.0° latitude/longitude grid. Two grid points are representative for the case study areas and provide data about wave height and period respective to different return periods. This data can be found in Appendix C. When comparing the data of WAM with the measurements at Côn Đảo Island and Bach Ho wave station it is immediately
visible that the data of WAM differs greatly from the other two data sources. From the WAM data it is not clear for what location the data is representative. The data does give a certain location, but this is at the shoreline, where it is simply not possible to have such big wave heights. For this reason the offshore wave characteristics will only be based on the measurements at Côn Đảo Island and Bach Ho wave station.

**Offshore wave characteristics** The offshore wave characteristics are based on the measurements at Côn Đảo Island and Bach Ho wave station. In Figure C.1 in Appendix C the offshore wave characteristics are displayed in graphs for the east coast. The fitted line indicates the design wave characteristics. In Table 3.6 the corresponding numbers are shown for the east coast. The same approach is used for the west coast, but with data of wave heights coming from SW direction. Table 3.7 shows the offshore wave characteristics for the west coast.

<table>
<thead>
<tr>
<th>Return period [year]</th>
<th>Wave height [m]</th>
<th>Wave period [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.2</td>
<td>8.0</td>
</tr>
<tr>
<td>10</td>
<td>4.5</td>
<td>9.1</td>
</tr>
<tr>
<td>25</td>
<td>5.5</td>
<td>9.5</td>
</tr>
<tr>
<td>50</td>
<td>6.0</td>
<td>9.8</td>
</tr>
<tr>
<td>100</td>
<td>6.5</td>
<td>10.1</td>
</tr>
<tr>
<td>200</td>
<td>6.9</td>
<td>10.4</td>
</tr>
<tr>
<td>250</td>
<td>7.1</td>
<td>10.5</td>
</tr>
<tr>
<td>500</td>
<td>7.6</td>
<td>10.8</td>
</tr>
</tbody>
</table>

**Table 3.6:** Design offshore wave height and wave period for the east coast.

<table>
<thead>
<tr>
<th>Return period [year]</th>
<th>Wave height [m]</th>
<th>Wave period [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.9</td>
<td>7.9</td>
</tr>
<tr>
<td>10</td>
<td>4.3</td>
<td>8.8</td>
</tr>
<tr>
<td>25</td>
<td>4.9</td>
<td>9.2</td>
</tr>
<tr>
<td>50</td>
<td>5.3</td>
<td>9.5</td>
</tr>
<tr>
<td>100</td>
<td>5.7</td>
<td>9.8</td>
</tr>
<tr>
<td>200</td>
<td>6.2</td>
<td>10.0</td>
</tr>
<tr>
<td>250</td>
<td>6.3</td>
<td>10.1</td>
</tr>
<tr>
<td>500</td>
<td>6.7</td>
<td>10.4</td>
</tr>
</tbody>
</table>

**Table 3.7:** Design offshore wave height and wave period for the west coast.

### 3.4.3 Currents

The currents along the east and west coast of the Mekong Delta are controlled by multiple processes interacting with each other. Both coastlines are subject to their own respective tidal regime and normal tidal currents. Moreover, currents induced by the monsoon winds are existent in the upper layer of the water mass (Albers & Stolzenwald, 2014). Due to the tidal phase differences between the tidal regimes of the East Sea and the Gulf of Thailand...
also a water level induced flow is created. If the East Sea or Gulf of Thailand experiences a high tide while the other sea experiences a low tide, this water level induced flow is present and reinforces or reduces the other two currents.

The resulting current is hence dependent on the monsoon period, tidal phase and topographical location of the area of interest. Multiple data sources have been assessed, such as the measurement campaign which can be found in Appendix A and a literature study. In Tables 3.8 and 3.9 the analysis is presented of the flow direction due to the different processes. In Table 3.10 the resulting flow direction according to the measurements is shown.

### Table 3.8: Flow direction tidal current

<table>
<thead>
<tr>
<th>Location</th>
<th>Tide</th>
<th>Flow direction current</th>
</tr>
</thead>
<tbody>
<tr>
<td>Đầm Dơi</td>
<td>Ebb</td>
<td>NE</td>
</tr>
<tr>
<td></td>
<td>Flood</td>
<td>SW</td>
</tr>
<tr>
<td>Ngọc Hiển</td>
<td>Ebb</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td>Flood</td>
<td>W</td>
</tr>
<tr>
<td>U Minh</td>
<td>Ebb</td>
<td>S</td>
</tr>
<tr>
<td></td>
<td>Flood</td>
<td>N</td>
</tr>
</tbody>
</table>

### Table 3.9: Flow direction wind induced current

<table>
<thead>
<tr>
<th>Location</th>
<th>Wind direction/monsoon season</th>
<th>Flow direction current</th>
</tr>
</thead>
<tbody>
<tr>
<td>Đầm Dơi</td>
<td>NE dry</td>
<td>SW</td>
</tr>
<tr>
<td></td>
<td>SW wet</td>
<td>NE</td>
</tr>
<tr>
<td>Ngọc Hiển</td>
<td>NE dry</td>
<td>W</td>
</tr>
<tr>
<td></td>
<td>SW wet</td>
<td>E</td>
</tr>
<tr>
<td>U Minh</td>
<td>NE dry</td>
<td>S</td>
</tr>
<tr>
<td></td>
<td>SW wet</td>
<td>N</td>
</tr>
</tbody>
</table>

### Table 3.10: Resulting current direction. Dominant directions are bold. Overall at the east coast, the SW flow direction is dominant. At the southern tip there is no dominant direction, instead accumulation is observed. At the west coast, overall, the southward currents are dominant.

<table>
<thead>
<tr>
<th>Location</th>
<th>Wind direction/monsoon season</th>
<th>Tide</th>
<th>Flow direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Đầm Dơi</td>
<td>NE dry</td>
<td>Ebb</td>
<td>NE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Flood</td>
<td>SW</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Ebb</td>
<td>NE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Flood</td>
<td>SW</td>
</tr>
<tr>
<td>Ngọc Hiển</td>
<td>NE dry</td>
<td>Ebb</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Flood</td>
<td>W</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Ebb</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Flood</td>
<td>W</td>
</tr>
<tr>
<td>U Minh</td>
<td>NE dry</td>
<td>Ebb</td>
<td>S</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Flood</td>
<td>N</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Ebb</td>
<td>S</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Flood</td>
<td>N</td>
</tr>
</tbody>
</table>
3.4. Hydrodynamics

The results of the measurement campaign for the flow direction seem to have contradictions. During the NE monsoon season, with wind coming from north-eastern direction, the flow towards the north is dominant. This would indicate that the wind has no significant influence on the current direction. Similarly, during the SW monsoon, with wind coming from the south-western direction blowing towards the north, the dominant flow direction is to the south. The Southern Institute for Water Resources and Planning (SIWRP) states that during both NE and SW monsoon, the main current direction is southwards. The current velocities during the SW monsoon season are lower than during the NE monsoon, and this could indicate that the wind does influence the current.

In addition, a group of students of Delft University of Technology (Stoop, Bouziotas, Hanssen, Dunnewolt, & Postema, 2015) mention in their research that the M2 tide in the Gulf of Thailand is in clockwise- instead of counterclockwise direction, which is custom in the Northern Hemisphere. This is in contradiction to the results of the measurements. The flood direction should be in southern direction following the clockwise theory, while the measurements indicate a northern direction. In literature Yanagi and Takao state that the propagation of the semi-diurnal tide in the Gulf in Thailand is indeed in clockwise direction (Yanagi & Takao, 1998). However, the diurnal tide propagates in counter-clockwise direction. The tidal gauge at the west coast shows a mixed tidal regime, consisting of semi-diurnal and diurnal characteristics. As mentioned before, the diurnal characteristics are predominant, however, at most places two high and low waters occur a day, but with a significant daily inequality. Therefore it is hard to conclude from theory whether the tidal propagation is in clockwise or counter-clockwise direction. Hence, this research will assume that the measurements are indicating the right current direction. It should be noted that due to the tidal phase differences between the tidal regimes of the East Sea and the Gulf of Thailand, the water level induced flow is also affecting the flow direction, mainly at the west coast. The measurement campaign at the east coast and west coast were not executed at the same time, hence the influence of the water level induced flow can not be analysed.

3.4.4 Water level

In this research both normal conditions and storm conditions are important, with respect to water levels. The normal conditions are used as boundary conditions to determine the optimal coastal profile and mangrove zonation. Storm conditions are used to indicate the boundary conditions for the entire coastal protection design. Design water levels are determined by tidal phase, storm surge and relative sea level rise (including absolute sea level rise and subsidence). Note that in the water level the wave height is not included.

The tidal range for An Minh and Tam Giang Động is 0.9 m and 2.5 m respectively. Storm surge comprises wind set-up, wave set-up, barometric effect and the effect of the shape of the coastline. Wind set-up is predominant over wave set-up in the coastal area of the Mekong Delta, due to the broad shallow shelf (Russell, 2013). In case of a narrow shelf, wave set-up would be predominant. Furthermore, wind set-up decreases with an increasing slope of the foreshore. Therefore, the very gradual slopes at the east coast induce higher water levels during storm conditions than those of the west coast, which are slightly steeper. Following Vietnamese design guidelines a sea level rise of 6 mm/year should be taken into account, as is mentioned in the MSc Thesis of Silke Tas (2016) on the same project location. Land subsidence can differ strongly locally. Therefore, Anthony et al. (2015) estimated, because of the lack of data, the land subsidence of the Mekong Delta in the order of 15 mm/year.
Water level under normal conditions

The water level under normal conditions is MSL plus half of the tidal amplitude. Storm surge will not be taken into account in this case. For An Minh this means a water level of MSL +0.45 m and for Tam Giang Định this gives a water level of MSL +1.25 m.

Tam Giang Định  The bathymetry at Tam Giang Định is discussed in section 3.5 and gives in combination with the water level an indication of the water depth during normal conditions. At the transition from mangroves to the sea a cliff has formed, from MSL up. This cliff has a height of approximately 1.0 m. Therefore, at spring tide the mangroves on top will be inundated. The mangroves that should be present between MSL and spring tide have been eroded, except for the upper 0.25-0.5 m. At a distance of 20 m in front of the cliff, at the narrow tidal flat (see Figure 3.13), the water depth is in the order of 1.25 m during spring tide under normal conditions. At a distance of 100 m in front of the cliff the water depth is approximately 3.25 m. According to the Technical Guideline for Seadike Design (MARD, 2012) the water depth at Tam Giang Định for a return period of one year varies between 1.71 m at the northern border and 1.53 m at the southern border. The difference with the approximated 1.25 m has to do with the fact that the bathymetry along the coast varies and is not precisely known to the author and the exact position of the modelled water height from the technical guideline is not given.

An Minh  In section 3.5 the bathymetry of An Minh is analysed. A cliff height of approximately 0.3 m is assumed above MSL. Directly in front of the cliff the water depth is in the order of 0.45 m at spring tide, assuming a narrow tidal flat at MSL in front of the cliff. This is based on the characteristics of the eroding profile of the case study site at the east coast. At a distance of 300 meter in offshore direction, with respect to the cliff, the water level is approximately at 3.0 m during spring tide, taking into account the bed step and the increased slope. Following the technical guideline the water depth is in the order of 0.65 m for a return period of one year.

Water level at storm conditions

Extreme design water levels are determined by a combination of high tides and storm surge. Since storm conditions will last longer than one tidal cycle, a tidal amplitude of 80 percent of the highest tidal amplitude is taken into account. Barometric effects increase the water level due to lower pressures when a storm is passing. Because there is little data available on the strength of cyclones in the Mekong Delta, the sum of the barometric- and wave set-up will be assumed to 0.5 m for the east coast (Tas, 2016), and 0.25 m for the west coast, as the west coast has a steeper bathymetric profile and lower water levels. The wind set-up is dependent on the bathymetry of the foreshore and the wind velocity corresponding to different return periods. In order to calculate the wind set-up the formula of Bretschneider (1966) is used, see Equation 3.8. The formula is applicable to an open sea with a horizontal bottom. To take the slope of the foreshore into account, the profile is divided into segments. Per segment the wind set-up is calculated and added to the still water depth of the next one, before calculating the set-up in that particular segment.

$$\Delta h = \sqrt{2 \cdot \kappa \cdot \frac{u^2}{g} \cdot F + h^2} - h$$  \hspace{1cm} (3.8)
3.4. Hydrodynamics

Where:

\[ \Delta h \] set-up do to wind friction [m]
\[ \kappa \] \( C_D + \frac{\rho_a}{\rho_w} \) [-]
\[ C_D \] drag coefficient \((0.8 \times 10^{-3} - 3.0 \times 10^{-3})\) [-]
\[ \rho_a \] density of air \((1.21)\) [kg/m\(^3\)]
\[ \rho_w \] density of water \((1030)\) [kg/m\(^3\)]
\[ F \] fetch [m]
\[ h \] still water depth [m]

**Tam Giang Đông** 80 percent of the tidal amplitude at the case study site of Tam Giang Đông is 2.0 m, half of the 80 percent tidal amplitude is therefore 1.0 m. The barometric effects and wave set-up comprise 0.5 m. The relative sea level rise will be taken into account with a value of 0.021 m/year. The simplified bathymetry is divided into eight segments with the starting point at an offshore location at a depth of 40 m. The water level set-up is calculated per segment. The simplified bathymetry divided into segments can be found in Figure D.1 in Appendix D.

**An Minh** 80 percent of the tidal amplitude at the case study site of An Minh is 0.72 m, half of the 80 percent the tidal amplitude is therefore 0.36 m. The barometric effects and wave set-up comprise 0.25 m. The relative sea level rise will be taken into account with a value of 0.021 m/year. The simplified bathymetry is divided in seven segments. These can be found in Figure D.2 in Appendix D.

The total water level set-up per return period, due to wind fraction, is calculated in a case study site specific manner. This is displayed in Table 3.11. In Table 3.12 the calculated water depths for different life times and corresponding return periods, as mentioned in section 3.2, are listed for both case study sites.

<table>
<thead>
<tr>
<th>Return period [year]</th>
<th>Wind speed [m/s]</th>
<th>Set-up Tam Giang Đông [m]</th>
<th>Set-up An Minh [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>19.7</td>
<td>0.4</td>
<td>0.2</td>
</tr>
<tr>
<td>20</td>
<td>21.9</td>
<td>0.5</td>
<td>0.3</td>
</tr>
<tr>
<td>25</td>
<td>22.4</td>
<td>0.5</td>
<td>0.3</td>
</tr>
<tr>
<td>50</td>
<td>24.8</td>
<td>0.6</td>
<td>0.3</td>
</tr>
<tr>
<td>100</td>
<td>27.0</td>
<td>0.7</td>
<td>0.3</td>
</tr>
<tr>
<td>200</td>
<td>29.2</td>
<td>0.8</td>
<td>0.4</td>
</tr>
<tr>
<td>250</td>
<td>29.9</td>
<td>0.8</td>
<td>0.4</td>
</tr>
<tr>
<td>500</td>
<td>32.1</td>
<td>0.9</td>
<td>0.5</td>
</tr>
</tbody>
</table>

*Table 3.11:* Estimated water level set-up due to wind friction for different return periods, case study site specific.
### Chapter 3. Project area analysis

#### 3.5 Morphology

3.5.1 Bathymetry

In general the coastal zone of the Mekong Delta is very shallow with a mildly sloping foreshore (1:1000 to 1:1500), as is typical for mangrove coasts. Unfortunately, little data is available on the bathymetry of the coastal area. There are bathymetric profiles available at three locations, measured during the measurement campaign by Albers and Stolzenwald in 2013. In Appendix E an analysis and the results of the bathymetric measurements can be found. The bathymetric profiles are representing the bathymetry at U Minh at the west coast, Ngọc Hiền at the southern tip and Đầm Dơi at the east coast. The three bathymetric profiles that are available for U Minh are comparable. Therefore the bathymetric profile for An Minh will be assumed to be similar to that of U Minh. The profile nearest to An Minh is used, at transect 9 in the area of investigation, and is displayed in Figure 3.10. As can be seen this profile has an average slope of 1:600 and ends at a distance of 300 m in front of the shoreline. During the field trip in September/October 2016 the author observed cliff forming at the shoreline. Therefore it is assumed that the slope of the remaining 300 m increases, leaving a bed step of 0.5 m at the shoreline at MSL and a small cliff height of 0.3 m above MSL.

![Bathymetric profile transect 9 U Minh](image)

**Figure 3.10:** Bathymetric profile transect 9 U Minh, representative for An Minh. Figure from (Albers & Stolzenwald, 2014).

---

### Table 3.12: Estimated water levels relative to MSL for different lifetimes and return periods, case study site specific.

<table>
<thead>
<tr>
<th>Lifetime [year]</th>
<th>Return period [year]</th>
<th>Water level Tam Giang Đôn [m]</th>
<th>Water level An Minh [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>10</td>
<td>2.3</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>2.4</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>2.5</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>2.6</td>
<td>1.3</td>
</tr>
<tr>
<td>50</td>
<td>25</td>
<td>3.1</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>3.2</td>
<td>2.1</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>3.3</td>
<td>2.1</td>
</tr>
<tr>
<td></td>
<td>250</td>
<td>3.4</td>
<td>2.2</td>
</tr>
<tr>
<td>100</td>
<td>50</td>
<td>4.2</td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>4.3</td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>4.4</td>
<td>3.1</td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>4.5</td>
<td>3.2</td>
</tr>
</tbody>
</table>
3.5. Morphology

Figure 3.11: Bathymetric profile transect 17 Đầm Dơi, representative for the northern part of Tam Giang Đông. Figure from (Albers & Stolzenwald, 2014).

For the other case study site at Tam Giang Đông it is more difficult to determine the bathymetry. Although Đầm Dơi is adjacent, the position of the shoreline and therefore its position relative to the incoming waves and longshore currents is different. Furthermore, the three bathymetric profiles of Đầm Dơi, with slopes varying between 1:1000 and 1:2000, are also showing dissimilarities with respect to each other. Overall, the coastal area at Đầm Dơi is characterized by a plateau with an almost constant depth in front of the shoreline, but the profile measured in the middle, shows a less constant depth, see Appendix E. The bathymetric profile at transect 17 of Đầm Dơi is measured at a distance of 3 km of the northern border of Tam Giang Đông. This location is pointed out in Figure 3.12. The profile itself is shown in Figure 3.11. At the mouth of Bo De tidal channel a near shore bathymetry profile is available, provided by Nguyen Cong Thanh in his doctoral dissertation (Thanh, 2012). This is located at the southern border of Tam Giang Đông. In Figures 3.12 and 3.13 the location of the bathymetric profile and the profile itself are displayed.

Figure 3.12: Location bathymetric profile Bo De. ST1-BD and ST2-BD are indicating measurement stations and location of the bathymetric profile, modified from (Thanh, 2012).
Chapter 3. Project area analysis

3.5.2 Shoreline position

The present Mekong Delta plain has been formed during the Holocene sea-level high stand, about 6000 to 3000 years ago. During this time the estuary prograded into a delta from Cambodia to the South China sea for at least 250 km, due to sediment provided by the Mekong River. This has not been a continuous progradation, but occurred at a decreasing rate. As a consequence of progradation, the seaward moving shoreline became more exposed to ocean waves. The progradation rate decreased from 18 m/year to 14 m/year at the now sandy beach-ridge dominated part of the delta. This decreased rate is due to longshore currents, initiated by monsoon generated waves, that causes southward sediment transport. On the other hand, due to westward longshore-transport of mud, there has been a progradation rate of 26 m/year at the southern tip of the delta, in the Ca Mau sector, in the past 3000 years. Whereas the delta was originally tide-dominated, during the last 3000 years in the Late Holocene, there has been a shift to an intermediate type; tide- and wave dominated delta (Anthony et al., 2015; Xue, Liu, DeMaster, et al., 2010).

The coastal area is characterized by two different coastal landform types. At the discharge area of the Mekong River in the East Sea and approximately 250 km further downstream (southwards) sandy beach-ridges with large inter-ridge cavities of sand and finer sediment can be found. More south-westwards the coastal area is mud-dominated, forming the remaining 350 km of the Mekong Delta shoreline. Nowadays the shoreline of the Mekong Delta is subjected to strong erosion. The muddy-area is affected intensively by the shoreline transgression in the order of 50 m/year. Almost 90 percent of the mud-dominated part of the East Coast is in retreat. In the period of 2003 until 2012 more than 50 percent of the entire Mekong Delta coastline suffered from erosion (Anthony et al., 2015).

Three sources are used to investigate the trend of the shoreline position of the Mekong Delta. Anthony et al. (2015) analysed the shoreline position at the east and west coast over time, using SPOT 5 satellite images with a resolution of 2.5 m, between 2003 and 2012. Besset et al. (2016) did the same for the period between 1973 and 2014 using Landsat images (resolution of 60 m for the older images and 30 m for the newer ones) and readjusted these images with higher resolution SPOT 5 satellite images. Sorgenfrei (2015) used historical maps from French archives as well as recent GIS material to estimate the
3.5. Morphology

coastline regression from 1904 until 2015.

Mekong Delta shoreline in general

In Figure 3.14 the dynamic position of the shoreline between 1904 and 2015 is displayed. The east coast is more prone to erosion than the west coast. In Appendix F multiple figures can be found, showing the coastal transgression and regression in hectares from 1965 until 2015. It is remarkable that these figures indicate a total accretion of 8.11 km$^2$ for Bạc Liêu, Sóc Trăng, Kiên Giang and Cà Mau. The first three provinces have been subjected to erosion, but in the end they gained more land than they have lost between 1965 and 2015. Cà Mau is the only province that has eroded more than accreted. The east coast of Cà Mau in particular is responsible for this high amount of erosion: approximately 21.11 km$^2$ is lost (Sorgenfrei, 2015). Furthermore, the data of Sorgenfrei(2015) indicates that the past 49 years about half the area is lost in comparison with the 61 years before (for all four provinces together). Between 1973 and 2014 Besset (2016) observed a gain of land of 3.75 km$^2$/year and a loss of 4.40 km$^2$/year for the shoreline of the Mekong Delta at the East Sea. Interestingly, Anthony (2015) observed a gain of only 0.37 km$^2$/year over the period 2003-2011 for the same area and a loss of land of 6.98 km$^2$/year. This indicates an increased erosion rate at the east coast. Between 2003 and 2012 the entire delta lost over 5 km$^2$ (Anthony et al., 2015).

![Figure 3.14: Shoreline position 1904-2015 (Sorgenfrei, 2015).](image)

Shoreline trend analysis Tam Giang Đông

In Appendix F, Figure F.8 zooms in on the shoreline changes between 1973 and 2014 for the east coast. The shoreline of the case study site Tam Giang Đông is subject to severe erosion with an estimated average rate of 40 m/year. For this area also averaged annual surface change rates are observed for the period of 1973-2015 and 2003-2011. Between 1973 and 2015 this was an average rate of -0.8 km$^2$/year (64 m/year), while between 2003 and 2011 this decreased to -0.3 km$^2$/year (24 m/year).
Shoreline trend analysis An Minh

During the field trip in October 2016 the study case area of An Minh was visited. At the border with the adjacent province of U Minh, severe erosion was observed. Every year for the last four to five years, according to the local inhabitants, houses had to be moved land inwards by 30 meter. At a distance of 4.0 km in northward direction of the province border, severe erosion was observed as well. The earth body, see Figure 3.15, which was supposed to be a dyke, was almost breached. According to the satellite images of Anthony (2015) the area of An Minh is subject to both erosion and accretion (2003-2011). The erosion rate at the border with U Minh province was less than 10 m/year between 2003 and 2011. In northern direction accretion with a maximum of 20 m/year is documented, followed by an averaged erosion rate of 20 m/year north of the case study site. Therefore there seems to be a trend of moving erosion and accretion along the coast. This phenomenon is supported by interviews with the local fisherman. They mentioned mudbanks moving in longshore and cross-shore direction along the coast. This could explain the moving pattern of accretion and erosion along the west coast of the Mekong Delta. Furthermore, according to the findings of Sorgenfrei (2015), the coastal area of An Minh was quite stable between 1965 and 2015. Area that was lost, was gained again in later years and vice versa. Over those 50 years in total only 0.26 km$^2$ was gained. Nevertheless, the coastal area of An Minh is in great need of coastal protection as the dyke is almost breached. The inhabitants of the coastal area of An Minh do not have time to wait for a period of accretion.

Figure 3.15: Dyke at eroded area in An Minh, picture taken by author during the field trip in October 2016.

3.5.3 Sediment budget

As there is no evident change in neither the river its discharge, nor the wave and wind conditions in the period between 2003 and 2012, other options to explain the geomorphic
3.5. Morphology

instability have been explored (Anthony et al., 2015). A significant decrease in sediment supply to the coast by the river was observed. This could be induced by dams further upstream in the Mekong River and commercial mining in the river and its delta. Due to sand mining, sediment traps are formed. During flood discharges the sediment is prevented from reaching the coastal region. Land subsidence is also contributing to the erosion problem. Because of sediment drainage and compaction it is normal for a delta plain to subside. Land subsidence can also be induced by human intervention such as extraction of fluids and gas and long-term drainage. During the field trip, multiple pumps were spotted where local people extract water for personal use. Furthermore, farmers use groundwater for irrigation and agricultural purposes. The land subsidence at the Mekong Delta is approximately in the order of 1 to 2 cm per year (RoyalHaskoningDHV et al., 2013). This year Darby (2016) suggests that shifting tropical cyclone activity contributes to the changing magnitude of sediment load in delta areas as well (Darby, Hackney, Leyland, Kummi, & Lauri, 2016).

Figure 3.16: Sediment transport and deposition Mekong Delta during normal flood year (2009), modified from Manh et al. (2014).

Reported estimated annual sediment loads for the Mekong River vary between 50 and 160 million tons over the last decade (Lu, Kummu, & Oeurng, 2014; Milliman & Farnsworth, 2011; Walling, 2008). In 2014 a quasi-2D hydrodynamic model was combined with a cohesive sediment transport model by researchers of the German Research Center for Geosciences and the Vietnamese SIWRR. This study is the first to quantify the sediment transport and deposition in the entire Mekong Delta (Manh et al., 2014). The model is calibrated using 13 stations that measure daily water levels and 10 stations that measure daily discharges. In addition, they used daily suspended sediment concentrations from two river stations and from 79 stations for six points in time. At 11 locations in the Vietnamese floodplain data of cumulative sedimentation masses was collected. The starting point of this research is the gauging station in Kratie, Cambodia. Three scenarios have been modelled. The first is an extremely low flood of 2010. The second one comprises normal flood conditions in 2009 and the last one an extreme high flooding in 2011.

The model shows that annual sediment loads reaching the coastal area, weirs, culverts
and sluice gates taken into account, vary from 48 to 60 percent of the sediment load at Kratie. The variation in sediment load reaching the coast is dependent on the flood scenario. The sediments lost between Kratie and the coastal area of the Vietnamese Mekong Delta are deposited at the Cambodia floodplains (19-23 percent) and for a smaller part at the floodplains of Vietnam (1-6 percent). A distribution of the sediment transport and deposition for the normal flood conditions is displayed in Figure 3.16. The abbreviations LXQ, THA and PoR are Vietnamese floodplains adjacent to the main river branches. In Table 3.13 the number for all three scenarios are showed. Furthermore, in Figure 3.17 the locations of the measuring stations and floodplains are indicated, including the ones at the coastline.

<table>
<thead>
<tr>
<th>Subsystem</th>
<th>Flood volume [%]</th>
<th>Sediment load [Mt]</th>
<th>Sediment load [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kratie</td>
<td>100  100  100</td>
<td>78.4  43.4  104.2</td>
<td>100  100  100</td>
</tr>
<tr>
<td>Cam floodplains</td>
<td>11   9    16</td>
<td>21.4  10.3  27.3</td>
<td>27   24   26</td>
</tr>
<tr>
<td>Overflow to VMD</td>
<td>6    4    9</td>
<td>3.5   1.5   7</td>
<td>4    4    7</td>
</tr>
<tr>
<td>Tonle Sap Lake</td>
<td>8    6    12</td>
<td>5.2   2.1   10.6</td>
<td>7    5    10</td>
</tr>
<tr>
<td>MD Vietnam</td>
<td>92   93   86</td>
<td>51.8  31    66.3</td>
<td>66   71   64</td>
</tr>
<tr>
<td>Tan Chau</td>
<td>67   70   60</td>
<td>41    24.7  50.3</td>
<td>53   57   48</td>
</tr>
<tr>
<td>Chau Doc</td>
<td>19   19   18</td>
<td>7.3   4.7   9</td>
<td>9    11   9</td>
</tr>
<tr>
<td>Vam Nao</td>
<td>26   28   23</td>
<td>14.7  9.3   18.7</td>
<td>19   21   18</td>
</tr>
<tr>
<td>Viet floodplains</td>
<td>21   17   24</td>
<td>10.9  5.6   17.6</td>
<td>14   13   17</td>
</tr>
<tr>
<td>Coast</td>
<td>-    -    -</td>
<td>42    25.9  50.5</td>
<td>55   60   48</td>
</tr>
</tbody>
</table>

Table 3.13: Flood volume and sediment load (total and relative) at key locations in the Mekong Delta for three flood scenarios.

![Figure 3.17: Sediment stations.](image-url)
3.5. Morphology

The results of the model show that the annual sediment load for the coastal area of the Mekong Delta ranges between 25.9 and 50.5 million ton. According to Darby (2016) the suspended sediment load at Kratie decreased with $33.0 \pm 7.1$ million ton in the period between 1981 and 2005. The current estimated sediment load for Kratie is $87.4 \pm 28.7$ million ton per year (Darby et al., 2016). This is in line with the model results.

3.5.4 Local sediment

In Figure 3.18 the sediment distribution in the Mekong Delta is displayed. It shows that the coastal area consists of very fine sediment particles. During the measurement campaign of GIZ, sediment samples were taken in Đầm Dơi, Ngọc Hiền and U Minh. Tam Giang Đông is situated between Đầm Dơi and Ngọc Hiền, therefore a median grain size diameter ($d_{50}$) is chosen dependent on both measurements. The sediment texture group in Tam Giang Đông is silt, very clayey, slightly sandy with $d_{50}=18 \mu$m. For An Minh, the same $d_{50}$ as measured in U Minh is chosen; $d_{50}=12 \mu$m. This sediment texture group is clayey silt, slightly sandy (Albers & Stolzenwald, 2014).

![Figure 3.18: Sediment distribution in the Mekong delta. Image obtained via Albers & Stolzenwald (2014), original source Nguyen (2009).](image)

Sediment volumes Tam Giang Đông

If we use the estimated average shoreline retreat rate of 40 m/year for Tam Giang Đông and an active zone up to a depth of 5 m, in combination with the high water level of MSL +1.25 m, and a shoreline length of 12.5 km; the volume of sediment that is eroded is $3,125,000 \text{ m}^3$/year. Assuming a relative sea level rise, including subsidence of 0.021 m/year and an average slope of the foreshore of 1:1000, 21 m/year of the erosion rate of 40 m/year is due to relative sea level rise.

Sediment volumes An Minh

For An Minh an estimated average shoreline retreat of 20 m/year is used to determine the annual lost sediment volume, of which 9 m/year is due to relative sea level rise, considering an average slope of the foreshore of 1:600. An active zone of 5 m, in combination with a high water level of MSL + 0.45 m is taken into account. The length of the shoreline is 13 km. The volume of the sediment that is eroded is $1,417,000 \text{ m}^3$/year.
Mangroves

Chapter 4

Mangroves

The Mekong Delta has potential to support a large mangrove forest, since the ecological conditions are promising. The southern coast of Vietnam is near the Indonesian and Malaysian islands, where many mangrove species find their origin (Marchand, 2008). Unfortunately, despite of seedlings carried by streams and the south-western winds towards the southern Vietnamese coast, the area suffers from severe mangrove degradation. In this chapter a study is performed to gain knowledge on the requirements for a healthy self-sustaining mangrove ecosystem in order to investigate the possibilities of implementing mangroves in the coastal protection strategy for the Mekong Delta in Vietnam.

Mangrove trees belong to diverse taxonomic groups and exist of numerous species. Mangrove forests can be found at low latitude, in tropical and some subtropical sheltered coastal areas, where they fulfil an important role in the coastal ecosystem (Bosboom & Stive, 2015). They provide an ecological and socio-economic function. Mangrove forests give shelter and nursery grounds to aquatic and terrestrial animals and filter seawater from nutrients and contaminants (Hong & San, 1993). Besides these functions, the mangrove fringe also has an important advantage as stabiliser of the coast. When waves enter the mangrove forest they lose energy mainly due to the network of roots and, to a lesser extent, due to trunks and branches (McIvor, Möller, Spencer, & Spalding, 2012; Verhagen & Loi, 2012). The second contribution of mangroves as coastal stabilisers is their ability to adapt to sea level rise. Mangroves are capable of changing the environment by slowing down currents and by the earlier mentioned wave attenuation. This leads to deposition of sediments and increase of soil volume (McIvor, Möller, Spencer, & Spalding, 2013).

4.1 Natural habitat

4.1.1 Location coastal area

Mangroves are tidal forests that are found between mean and high water level, at the transitional zone between marine and terrestrial environments, and are adapted to saline habitats. Mangrove belts exist of one of more of the approximately 70 different mangrove plant species which can grow in loose wet soil like mud. They are mainly located behind tidal flats, which provide shelter. On the contrary, shorelines which are exposed to wind and waves directly, mainly consist of coarse sediment and have sandy foreshores and beaches, which are not suitable conditions for mangroves to develop (Marchand, 2008). The habitat of a mangrove fringe is characterized by the continuously varying chemical, physical and biological characteristics due to tidal influences and coastal currents (Saenger, 2002). Furthermore, mangroves are able to survive in freshwater environments, as they are facultative halophytes, but under those conditions they are outcompeted by communities
of obligate fresh water species. In contrast of what seems likely, mangroves are not keen on saline water. In fact, salinity is a stress factor for mangroves (Marchand, 2008). The saline environment is important because mangroves grow slowly compared to faster growing fresh water species, the salinity suppresses or eliminates the growth of the fresh water species and hence the mangroves are able to flourish (Saenger, 2002).

4.1.2 Soil requirements and nutrients

Mangroves can survive at various waterlogged and anaerobic soils consisting of coarse sediments like sand, or finer sediments like silt and clay (Hong & San, 1993). However, mangroves prefer mud soils over coarse sediments as they can be found at low energy coasts. Mud exists of a mixture of water, fine sand, silt, clay and organic materials (Bosboom & Stive, 2015). In order to survive in their tropical, saline environment in oxygen-starved mud, mangroves have adapted throughout time in several ways (C. Field, 2007).

Mangroves need oxygen for respiration, the process in which nutrients are converted to energy in a cell. In waterlogged soil the pores are filled with water in stead of air. Even if the water is saturated with oxygen, its oxygen level is 10,000 times less of that of air filled pores (Hogarth, 1999). To adapt to the oxygen-starved mud mangroves developed aerial roots: stilt roots, pneumatophores and knee roots (Spalding, Kainuma, & Collings, 2010), to provide oxygen for the underground roots (Hogarth, 1999). Stilt roots diverge from the tree above ground and penetrate the soil at a certain distance from the trunk. If the aerial root reaches the soil, absorptive roots grow vertically and possibly a secondary aerial root will part. Pneumathores are vertical extensions of the underground roots breaching the soil upwards, varying from 30 cm to 3 m in height above the soil surface, depending on the size and specie of the mangrove tree, in order to provide oxygen for the underground roots (Hogarth, 1999). Knee roots are round knob kind of extensions that breach the soil similarly as the pneumathores (Spalding et al., 2010). The aerial root system of some mangrove species perform very efficiently, aerating the surrounding soil. However, other mangrove species make the surrounding soil even more hypoxic (Hogarth, 1999).

Mangroves are found at the transitional zone between the saline sea and fresh water terrestrial environment. The salt content of sea water is in the order of 35 g/l. However, the real difficulty to cope with for mangroves is the variation in salt content due to the tide, not the high salinity itself (Hogarth, 1999). To adapt to the saline environment mangroves have developed multiple mechanisms such as salt exclusion by roots, elimination of salt by secretion and tolerance of tissue with high salt content (Hogarth, 1999). The latter may hold to store salt in for example older leaves, which get shed when the concentration reaches its maximum. Furthermore, a number of mangrove species have glands on their leaves to excrete salt (C. Field, 2007).

Salt exclusion by roots induces higher concentrations of salt in the soil. Salinity causes a negative osmotic potential which means that water must be overcome this pressure before taken in. In case of major salt built-up in the soil, water uptake will be limited rigorously (Hogarth, 1999). To prevent this from happening, flushing of the environment is of utmost importance. Furthermore, the presence of macropores and animal burrows in intertidal soil also support the prevention of excessive salt built-up (Saenger, 2002).

Besides water, mangrove plants are in need of mineral macronutrients such as nitrogen and phosphorus, and micronutrients, for example iron, manganese and zinc (Hogarth, 2007). Nitrogen is obtained by assimilation of inorganic nitrate and phosphorus through phosphate. Inorganic nutrients are provided by the production of amonnia out of gaseous
nitrogen by bacteria (nitrogen fixation) followed by a process called nitrification and by the
release of mineral nutrients out of organic compounds, such as proteins, by decomposers.
Inorganic nutrients can be transported by rainfall, imported by animals, rivers and land
run-off, or as tide-borne soluble or particle-bound molecules (Hogarth, 1999). There is a
minimal number of cases analysed regarding mangrove nutrients. Rainfall is probably not
the direct source of nutrients for mangroves, but increases those gained and lost by rivers
and land run-off. Overall the analyzed cases tend to conclude a nutrient supply mostly of
terrestrial origin through land drainage instead of tidal contribution by sea water (Lugo,
Sell, & Snedaker, 1976). Mangrove habitats in general have a net nutrient gain instead of
a loss (Hogarth, 2007). Nutrient deficiency may limit mangrove growth. However, other
stress factors are dominant and hence nutrient supply is not very likely to become the
limiting factor for mangrove growth (Luttge, 2008).

4.1.3 Tidal and drainage requirements

Tidal and freshwater flows within the mangrove fringe are essential to create and preserve
a healthy self-sustaining mangrove ecosystem. If these two factors are (partially) blocked,
it can lead to increased salinity and anaerobic conditions and free sulfide availability, which
have a negative effect on the health of the mangroves. Furthermore, the periods of in-
undation are important. Too frequent flooding causes stress, whereas too long periods of
dryness will cause difficulties for the mangroves to survive as well (R. R. Lewis, 2005).
Hence mangroves can be found between mean sea level, or slightly above, and spring tide
level. Mean sea level is defined as the averaged water level between mean high water and
mean low water. According to Lewis (2005) flooding depth, duration and frequency have
a high impact on survival and growth of mangrove seedlings and mature trees. Therefore,
in addition, diurnal tides are less favourable for the development of mangroves compared
to semi-diurnal tides (Hong & San, 1993). Diurnal tides have, compared to semi-diurnal
tides, long periods of low water. Although the west coast experiences two high waters a
day, one of those two high waters is significantly lower than the other, which means that
the upper part of the mangrove fringe is inundated only once a day.

4.1.4 Internal zonation

Within a mangrove fringe an internal zonation can be distinguished in which different
species are located at certain vertical levels. In Table 4.1 the explanation of the inundation
classes can be found, used in the list of mangrove species occurring in the coastal area of
the Mekong Delta and their characteristics which are presented in Table 4.2. Furthermore,
in Tables 4.3 and 4.4 the mangrove zonation for both case study areas is listed.

<table>
<thead>
<tr>
<th>Inundation class</th>
<th>Flooded by</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>All high tides</td>
</tr>
<tr>
<td>2</td>
<td>Medium high tides</td>
</tr>
<tr>
<td>3</td>
<td>Normal high tides</td>
</tr>
<tr>
<td>4</td>
<td>Spring high tides</td>
</tr>
<tr>
<td>5</td>
<td>Abnormal (equinoctial) tides</td>
</tr>
</tbody>
</table>

Table 4.1: Explanation inundation classes (Hong & San, 1993).
Chapter 4. Mangroves

<table>
<thead>
<tr>
<th>Scientific name</th>
<th>Inundation class</th>
<th>Soil and position</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>AVICENNIACEAE</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><em>Avicennia alba</em></td>
<td>1-2</td>
<td>Deep mud, sea face</td>
</tr>
<tr>
<td><em>A. lanata</em></td>
<td>2-3</td>
<td>Sandy mud</td>
</tr>
<tr>
<td><em>A. marina</em></td>
<td>1-4</td>
<td>Deep sandy mud, sea face</td>
</tr>
<tr>
<td><strong>MYRSINACEAE</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><em>Aegeciras corniculatum</em></td>
<td>1-2-4</td>
<td>Wet, sandy mud, sea face</td>
</tr>
<tr>
<td><strong>RHIZOPHORACEAE</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><em>Brugiera gymnorrhiza</em></td>
<td>3-4</td>
<td>Loam, sandy mud, foot of limestone</td>
</tr>
<tr>
<td><em>B. cylindrica</em></td>
<td>3-4</td>
<td>Firm mud, not far from sea</td>
</tr>
<tr>
<td><em>Ceriops decandra</em></td>
<td>3-5</td>
<td>Firm mud, land fringe</td>
</tr>
<tr>
<td><em>Rhizophora apiculata</em></td>
<td>2-4</td>
<td>Deep soft mud, river banks</td>
</tr>
<tr>
<td><strong>SONNERATIACEAE</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><em>Sonneratia alba</em></td>
<td>1-3</td>
<td>Deep soft mud, sea face</td>
</tr>
</tbody>
</table>

Table 4.2: Mangrove species coastal area Mekong Delta (Hong & San, 1993).

<table>
<thead>
<tr>
<th>Inundation class</th>
<th>Elevation [m above MSL]</th>
<th>Inundation frequency [times per month]</th>
<th>Mangrove species</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>&lt; 0</td>
<td>56-62</td>
<td>none</td>
</tr>
<tr>
<td>2</td>
<td>0-0.9</td>
<td>45-59</td>
<td><em>Avicennia, Sonneratia</em></td>
</tr>
<tr>
<td>3</td>
<td>0.9-1.25</td>
<td>20-45</td>
<td><em>Rhizophora</em></td>
</tr>
<tr>
<td>4</td>
<td>1.25-1.5</td>
<td>2-20</td>
<td><em>Rhizophora</em></td>
</tr>
<tr>
<td>5</td>
<td>&gt; 1.5</td>
<td>&lt; 2</td>
<td><em>Ceriops</em></td>
</tr>
</tbody>
</table>

Table 4.3: Inundation classes and the related mangrove species zonation in Tam Giang Đông.

<table>
<thead>
<tr>
<th>Inundation class</th>
<th>Elevation [m above MSL]</th>
<th>Inundation frequency [times per month]</th>
<th>Mangrove species</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>&lt; 0</td>
<td>42-47</td>
<td>none</td>
</tr>
<tr>
<td>2</td>
<td>0-0.15</td>
<td>34-44</td>
<td><em>Avicennia, Sonneratia</em></td>
</tr>
<tr>
<td>3</td>
<td>0.15-0.30</td>
<td>15-30</td>
<td><em>Rhizophora</em></td>
</tr>
<tr>
<td>4</td>
<td>0.30-0.5</td>
<td>2-15</td>
<td><em>Rhizophora</em></td>
</tr>
<tr>
<td>5</td>
<td>&gt; 0.5</td>
<td>&lt; 2</td>
<td><em>Ceriops</em></td>
</tr>
</tbody>
</table>

Table 4.4: Inundation classes and the related mangrove species zonation in An Minh.

4.2 Rehabilitation mangrove forests

When a mangrove ecosystem is no longer healthy and self-sustaining, it is not capable to recover from damage (R. R. Lewis, 2005). At that point restoration or rehabilitation is needed to prevent the mangrove fringe from degrading. There is a distinction between restoration and rehabilitation. According to Lewis (1990) the definition of restoration holds any process that aims to return a system to prior system conditions, whether this
was pristine or not. Rehabilitation is a more general term and refers to any activity, including restoration and habitat creation, that aims to convert a degraded system into a stable alternative to enhance the overall ecology of the system (R. Lewis, 1990). Furthermore, the term ecological restoration is of importance as it is defined by the Society for Ecological Restoration as “The process of assisting the recovery of an ecosystem that has been degraded, damaged, or destroyed. It is an intentional activity that initiates or accelerates an ecological pathway – or trajectory through time - towards a reference state (Gann & Lamb, 2004, p.3).” Ecological restoration can be achieved by using existing and healthy ecosystems and their characteristics as a model that can be pursued.

4.2.1 Natural causes and engineering measures influencing mangroves

Although mangroves are favourable vegetation at the shoreline of the Mekong Delta, there are many places where mangrove belts have become scarce. There are multiple reasons to explain the mangrove degradation, as described by Linh (2012). During the Vietnam War (1962-1971) nearly 40 percent of the mangroves in the Mekong Delta were destroyed. Between 1980 and early 1990s the mangroves were used as a source for construction timber and charcoal, and forest land was converted into aquaculture-fishery farming systems (Linh, 2012). This led to the degradation of a highly diverse mangrove forest to a monoculture forest, which has a lower survival rate. At present, the shrimp farms are a major problem as the farmers cut the mangroves and leave behind the area in a polluted state (Verhagen & Loi, 2012). One of the main causes of mangrove degradation is disruption of hydrology. Reasons for alterations of hydrology are amongst others the nearby construction of dykes, levees and roads. However, also more natural causes attribute to mangrove degradation due to changes in hydrology, such as hypersalinity and disruption of fresh water flows (R. R. Lewis, 2005). Hypersalinity occurs due to variations in rainfall over the years. In case of freshwater shortage, the saline sea water will not be diluted and the soil becomes too saline for mangroves to grow and they may not survive (C. D. Field, 1998). Next to hydrology, morphology plays an important role in mangrove degradation and rehabilitation as well. As explained in Chapter 2.2, the Mekong Delta coast is supplied with sediment by the Mekong River and tides. Waves stir up the sediments and currents transport it in onshore or offshore direction. If these processes are balanced, the shoreline is in dynamic equilibrium. If not, the coast will erode or accrete, which has a direct effect on the mangrove fringes.

4.2.2 Principles

The survival and growth of mangroves seedlings, as well as mature trees is mainly dependent on soil saturation, and the depth, duration and frequency of flooding (R. R. Lewis, 2005). Mangrove forests are able to be self-sustaining and can self-repair after damage or undergo secondary succession in a period of 15-30 years if the normal tidal hydrology has not been disrupted and the availability of seeds or seedlings is sufficient (C. D. Field, 1998). Ecological succession is the transition in species composition of a community or ecosystem following a disturbance. A distinction can be made between primary and secondary succession. Primary succession holds the beginning of succession when there is no soil available and pioneer species have to build the foundation of the community. Secondary succession occurs after a disturbance at an area where soil is still present (Campbell & Reece, 2008).

In the past many mangrove restoration projects have failed. A vast contribution to these failures is ascribed to directly planting mangroves without determining the cause of the initial deterioration (R. Lewis & Streever, 2000) and to neglecting to work with the
potential of the ecosystem to recover naturally, without intervention (R. Lewis, 2004). It is important to gain knowledge on the history of the site and to determine what prior activities are responsible for the current conditions (Saenger, 2002). In case there is damage due to a disaster, observations of secondary succession, specifying recovery in time, can indicate whether the site needs support to accelerate the recovery or if it can recover fully by itself (R. Lewis, 2009). If mangroves are planted in the degraded area before stress factors have been determined and if needed, removed, the survival rate of the mangrove plants is low. For example, Sanyal (1998) reported an extremely low survival rate of 1.52 percent of 9,050 ha mangroves planted between 1989 and 1995 in West-Bangal, India. In Ha Tinh province in Vietnam a higher survival rate of 40 percent was observed (Erftemeijer & Lewis, 1999), however in the northern part of Vietnam the survival rates were significantly lower (Marchand, 2008). Due to the uncertainty of survival rates when planting mangroves, five steps for successful mangrove restoration have been presented by Lewis and Brown (2006, p. 2):

1. "Understand the ecology of individual mangrove species at the site; in particular the patterns of reproduction, propagule distribution, and successful seedling establishment.

2. Understand the normal hydrologic patterns that control the distribution and successful establishment and growth of targeted mangrove species.

3. Assess modifications of the original mangrove environment that currently prevent natural secondary succession.

4. Design the restoration program to restore appropriate hydrology and, if possible, utilise natural volunteer mangrove propagule recruitment for plant establishment.

5. Only utilise actual planting of propagules, collected seedlings, or cultivated seedlings after determining through step 1-4 that natural recruitment will not provide the quantity of successfully established seedlings, rate of stabilisation, or rate of growth of saplings established as objectives for the restoration project."

As is mentioned in the five steps above, reinstatement of the hydrological conditions is essential. Restoring the morphodynamic conditions, the fine sediment balance, is crucial, as this provides soil above mean sea level in order for mangroves to develop (J. C. Winterwerp, Erftemeijer, Suryadiputra, Van Eijk, & Zhang, 2013). This can be achieved by active sediment management. Furthermore, there is evidence at several locations in Vietnam, observed during former rehabilitation projects, of limited propagule abundance. Limited propagule abundance describes the situation in which propagules are limited in natural occurring availability due to different causes, such as hydrological blockages, which prevent natural waterborne transport, or lack of mangrove specimen in the adjacent area (R. R. Lewis, 2005). These circumstances will avert natural recruitment. It is therefore recommended to perform a thorough research if this concerns the case study areas as well.

### 4.2.3 Seedlings

In this research propagules with roots are called seedlings. In spite of the overwhelming availability of literature on mangroves, there has not been a lot of research done regarding the threshold of mangrove seedling establishment. Due to the research of Balke et al. (2011) about the pioneer species *Avicennia alba*, it is known that there are three thresholds that are important for the establishment of mangrove seedlings. First, propagules are in need of a inundation free period, after stranding on the mud flat, to be able to anchor into the mud. Secondly, after anchoring, the roots have to gain a minimal length
in order to withstand hydrodynamic forces by currents and waves. The required length of the roots is proportional to the force they have to resist. The third threshold is the larger minimal root length in order to withstand sediment removal around the seedling, caused by erosion or mixing of sediments. *A. alba* roots rapidly and is therefore, based on experimental studies by Balke (2011), able to withstand the forces described above by eight days post germination. A window of opportunity for successive mangrove rehabilitation could therefore be at neap tide during fruiting season (Balke et al., 2011).

The growth rate of the roots of mangrove seedlings can not be influenced. In case of the presence of hydrodynamic forces which are exceeding the forces the seedlings can withstand, measures to reduce the hydrodynamic forces should be considered. In this research a maximum wave height of 0.5 m is assumed as the hydrodynamic force the mangrove seedlings can withstand, without getting uprooted. This wave height is based on literature and observations of experts. From literature it is not clear what the maximal frequency of these maximum hydrodynamic forces is, that the seedlings still can endure.

### 4.2.4 Sediment balance and mangroves

Mud-coasts are highly dynamic and are subject to cycles of accretion and erosion, which effects the mangrove fringe (J. C. Winterwerp et al., 2013). In a healthy self-sustaining mangrove ecosystem the cycles of accretion and erosion are balanced, hence the coastline is in a dynamic equilibrium. The mangrove root system is able to enhance the sedimentation by trapping and accumulating sediment, increasing the soil volume and hence inducing elevation of the bed and a progradating shoreline. In order to have successful rehabilitation of the mangrove fringe in the Mekong Delta, it is therefore important to understand the sediment balance and to enhance sedimentation where necessary. Both case study sites are subject to severe erosion and this is highly indicative of a sediment imbalance. This imbalance leads to a less favourable habitat for mangrove species as their roots anchor at a depth of just 0.5 m into the soil. Just a few decimeters of vertical erosion can destabilise the mangrove trees, which eventually leads to mangrove death (Tomlinson, 1986). It is clear that measures need to be taken to restore the sediment balance at both case study areas.

### 4.2.5 Dimensions foreshore

The dimensions of the foreshore are related to tidal regime and currents, wave energy and topography of the coastline. If the ratio of tidal activity to wave activity is high at a mud-coast, the profile of the foreshore will be convex. If this ratio is low, it tends towards a concave profile. Furthermore, the topography of the shoreline is also of influence on the foreshore profile. A straight coastline will have a linear equilibrium profile, whereas an equilibrium embayed or lobate shoreline have respectively a convex or concave profile (Friedrichs & Aubrey, 1996). Mangroves prefer an equilibrium profile that is convex-up. Eroding mangrove coasts tend towards a concave-up profile. This will further increase the impact of the waves due to a larger depth in front of the mangrove fringe and therefore will increase the erosion rate. Once a cliff has formed at the edge of the mangrove fringe, it becomes almost impossible for the mangrove forest to withstand the erosion process.

For a straight coastline Friedrichs and Aubrey (1996) established Formula 4.1 for an equilibrium profile under tidal influence. They assumed that the bed shear stress induced by the tidal movements is uniformly distributed over the mudflat. The bed level has a linear-sinusoidal profile.
Chapter 4. Mangroves

\[ Z_b(x) = \begin{cases} 
  a_0(x/L_r - 1) & \text{for } x \leq L_r \\
  a_0 \sin(x/L_r - 1) & \text{for } x \geq L_r 
\end{cases} \quad (4.1) \]

Where:

- \( Z_b \): bed level \([m]\)
- \( x \): coordinate along mudflat \([m]\)
- \( a_0 \): tidal amplitude \([m]\)
- \( L_r \): reference length of the flat \([m]\)

The reference length of the flat is the length of the mudflat below mean sea level. This equilibrium solution shows that under mean sea level the profile is linear, whereas above mean sea level it has a convex-up shape. In case of an embayed shoreline, the profile gets more convex-up due to the rate between tidal velocity and the rate at which tidal elevation is changing. A more convex profile is needed to compensate for the decreasing tidal velocity due to the lateral divergence of the flow (Friedrichs, 2011). If the shoreline is of the lobate type, it is the other way around. Due to convergence the tidal velocity will not slow down and therefore a concave profile will be developed to compensate for the tidal velocity.

Lee and Mehta (1997) developed a formula predicting the shape of the foreshore as a parabolic-exponential cross-shore profile. Taking into account the effect of wind waves and swell waves. They assumed that the wave dissipation is uniformly distributed over the mudflat.

\[ \frac{Z_b(x)}{Z_{b,0}} = \left( \frac{x}{X_0} \right)^2 \exp \{-4k_i(x - X_0)\} \quad (4.2) \]

Where:

- \( Z_b(x) \): bed level \([m]\)
- \( Z_{b,0} \): bed level at offshore location \(X_0\) \([m]\)
- \( x \): coordinate along mudflat \([m]\)
- \( X_0 \): offshore location where the bed start to feel the waves \([m]\)
- \( k_i \): imaginary wave number \([-]\)

Multiple research groups have investigated the influence of tides, waves, and sediment supply on the crossshore profile of the foreshore of a muddy coast. 1-D combined hydrodynamic and morphodynamic models have been used to model the equilibrium profile while being influenced by tides and waves. Roberts et al. (2000) and Waeles et al. (2004) both simulated this, and the outcome supports the earlier mentioned theory that the equilibrium convex-up profile becomes more concave as the wave energy increases (Friedrichs, 2011).
4.3 Coastal protection requirements

A self-sustaining mangrove fringe can stabilise the shoreline position, as it traps sediment. Furthermore, waves entering the mangrove fringe will be attenuated while losing energy. Note that not only the mangrove fringe induces energy loss of the waves, but the foreshore is of importance as well. Mudflats with very gentle slopes add additional bottom friction, mainly due to higher sediment concentrations near the bottom, hence help damping the wave heights (Linh, van Thiel de Vries, & Stive, 2015). For mangroves to serve as a coastal protection measure, a certain width of the mangrove fringe is required. For the east coast of the Mekong Delta the coastline was stable with an averaged mangrove fringe width of 140 m (Linh et al., 2015). Due to the smaller tidal range at the west coast, a smaller width will be needed. This is the minimal width needed for the mangroves to be self-sustaining along the Mekong Delta coast. A width of 100 m is required to attenuate short waves. For wave attenuation of long waves a width of approximately 300-400 m is favourable, as this reduces the wave height to 10 percent of the seaward wave height (Linh et al., 2015). The required width may increase for severe storm conditions.

Figure 4.1: Conceptual diagram of the properties and response of muddy foreshore profiles to external forcings. Modified from (Friedrichs, 2011).
Chapter 5

Alternatives

This chapter presents the requirements of a Building with nature alternative for the DST, followed by the identification of several possible alternatives.

5.1 Requirements

In order to generate a Building with Nature alternative for the DST, a combination of a mangrove fringe with its accompanying wave attenuating foreshore and an earth dyke are considered. The use of an earth dyke protected by mangroves is fairly common in the Mekong Delta. Earth dykes can be seen as a local cost-effective coastal protection measure that can be build with local materials, without the need of costly revetments. Furthermore, an earth dyke can easily be adapted to heavier hydraulic conditions. To ensure the stability of the earth dyke, a maximum wave height of 0.5 m behind the mangroves is allowed during heavy storm conditions. Heavy storm conditions in this research consist of a storm with a return period of 100 years and a duration of 6 hours. In reality a severe storm will probably last longer than 6 hours. However, usually the duration of the peak of the storm is less than 6 hours. Therefore, a storm duration of 6 hours at peak conditions is a robust design assumption.

For seedlings and young mangroves to develop a maximum wave height of 0.5 m in front of the mangroves is assumed to be acceptable. The frequency at which seedlings can withstand this wave height, compared to young mangroves, is lower. In this research the requirement of a maximum wave height of 0.5 m during normal conditions is translated in a storm with a return period of one year, hereafter named as annual storm conditions. This means that seedlings can colonise the mudflat and develop almost continuous throughout the year, without getting uprooted.

![Figure 5.1](image)

**Figure 5.1:** Overview locations of the requirements for a Building with Nature alternative, including a foreshore, mangrove fringe and an earth dyke.
Optimal coastal profile and shoreline position

The optimal coastal profile is a convex-up profile, as this is related to a prograding profile and net shoreward sediment transport. The shoreline position depends on the required minimum width of the mangrove fringe. Factors contributing to this width are displayed in Figure 5.2.

**Figure 5.2:** Factors contributing to the minimum required mangrove width. The minimum width is the maximum of W1-W4.

### 5.2 Identification alternatives

Multiple alternatives are possible to generate a Building with Nature solution for the coastal protection of the Mekong Delta. As is mentioned in paragraph 5.1 the alternative holds a solution with a dyke without a revetment. This means that the alternatives differ in the foreshore solutions. The alternatives are listed below:

- **Alternative 1:** mud nourishment without extra measures
- **Alternative 2:** mud nourishment in combination with a permeable breakwater
- **Alternative 3:** mud nourishment in combination with Melaleuca fences
- **Alternative 4:** sand nourishment without extra measures

The main focus will be on the alternative with a mud nourishment without extra measures, as this alternative will be investigated quantitatively, because up to now it has not been considered as an option. In case this option will not meet all the requirements, a qualitative advice will be given regarding the other alternatives.
Chapter 6

Boundary conditions

The alternative ‘mud nourishment’ is simulated with a numerical model called SWASH. The numerical model itself will be explained in Chapter 7. The input parameters are based on the following boundary conditions, which are mainly obtained from Chapter 3.

6.1 Lifetime and return period

In Vietnam it is common to set the lifetime of an earth dyke without a revetment to a maximum of 20 years. To design a more sustainable solution for the coastal protection of the Mekong Delta a life time of 50 years is proposed in this research. The design should be able to withstand storm conditions with a return period of 100 years, also uncommon to Vietnamese practice, as is explained in Chapter 3.2. Practise shows that the maintenance costs of a dyke with a lifetime of 20 years are very high. When designing a dyke for a lifetime of 50 years and a return period of 100 years, the design will be more robust and will therefore need less maintenance to withstand the daily conditions, as it is designed for a maximum wave height of 0.5 m which will occur rarely.

6.2 Bathymetry

Figure 6.1 shows the present bathymetric profile of the foreshore at the case study site of Tam Giang Đông, situated at the east coast. This profile is derived from different profile measurements as presented in Chapter 3. Mangrove trees are found between MSL and spring tide level. Therefore, in the existing bathymetric profile with the cliff, mangroves can only be present on top of the cliff. In front of the cliff the mangroves are not able to develop or survive as they would be inundated constantly. Hence, natural recruitment and progradation of the mangrove fringe is impossible without taking measures. In Figure 6.2 the proposed convex-up profile is displayed, based on the formula of Friedrichs and Aubrey (1996), see Formula 4.1.

Figure 6.3 shows the present bathymetric profile of the foreshore at the case study site of An Minh, situated at the west coast. This profile is derived from Chapter 3. In Figure 6.4 the proposed convex-up profile is displayed, based on the formula of Friedrichs and Aubrey (1996) for the upper part, starting at x=2.0 km with a slope of 1:600. The part between x=0.0 km and x=2.0 km has a slope of 1:375, and is returning to the original bathymetric profile with a slope of 1:600 at the offshore stretch.
Figure 6.1: Bathymetric profile Tam Giang Đông. The dark blue line indicates MSL and the light blue lines indicate the tidal range.

Figure 6.2: Convex-up bathymetric profile Tam Giang Đông. The dark blue line indicates MSL and the light blue lines indicate the tidal range.

Figure 6.3: Bathymetric profile An Minh. The dark blue line indicates MSL and the light blue lines indicate the tidal range.

Figure 6.4: Convex-up bathymetric profile An Minh. The dark blue line indicates MSL and the light blue lines indicate the tidal range.
6.3 Wave height

For the case study area at the east coast, Tam Giang Đông, wave characteristics are determined. The offshore significant wave height during annual storm conditions with a return period of one year is 3.2 m, with a peak period of 8.0 s. During heavy storm conditions with a return period of 100 years, the significant wave height is 6.5 m and the peak period is 10.1 s. According to the measurements conducted at Bach Ho wave station (Huan & Nhan, 2006), the average wave height is 2.0 m with a wave period of 5.8 s and a return period of one month. For the case study site at the west coast, An Minh, the offshore significant wave height during annual storm conditions with a return period of one year is 2.9 m with a peak period of 7.9 s. During heavy storm conditions the offshore significant wave height is 5.7 m with a peak period of 9.8 s.

6.4 Water level

The water level during annual storm conditions was determined in paragraph 3.4.4. For Tam Giang Đông a water level of MSL +1.25 m was found and for An Minh a water level of MSL +0.45 m. These design water levels only occur during high tide. In paragraph 3.4.4 the water levels for different life times and return periods were determined. In this research the heavy storm conditions encompass a lifetime of 50 years and a return period of 100 years. This would indicate a water level of MSL +3.3 m for Tam Giang Đông and a water level of MSL +2.1 m for An Minh. These water levels are based on a worst case scenario of land subsidence and sea level rise. This implies that the profile will not adapt morphologically to the occurring conditions. In case of sufficient sediment supply, a coastal profile adapts to increasing water levels by growing in vertical direction. Due to the fact that it is not clear whether the Mekong Delta coast has a sufficient sediment supply, this is not assumed. However, there is evidence of a certain degree of sediment supply and hence it is assumed that there is some adaptation to sea level rise. The water levels during storm conditions will therefore be based on a lifetime of 50 years with a return period of 100 years, but taking into account the land subsidence and sea level rise for a period of 20 years. Hence, an adaptation rate of 40 percent is assumed, and this gives a water level of MSL +2.6 m for Tam Giang Đông and MSL +1.3 m for An Minh. The bed level in SWASH is set at a constant level beneath MSL, whereas the water depth is increased or decreased in order to simulate sea level rise, land subsidence and vertical adaptation, this is expressed in the water level boundary condition.

6.5 Vegetation

To take vegetation into account in SWASH, mangroves are divided into three different vertical layers: roots, stems and canopy. The different vertical layers are described by four parameters each: height, diameter, density and a drag coefficient (The SWASH team, 2016). The input parameters, except for the drag coefficient, will be based on literature by Narayan (2009), who investigated the characteristics of two mangrove species in India. According to Linh et al. (2015) the values of these characteristics are applicable to the mangrove species in the Mekong Delta, due to resemblance between Vietnamese and Indian mangrove species. Mangrove forests can differ in density due to multiple reasons, such as age of the forest, the degree of self-sustainability, forest management and even illegal cutting. An older and self-sustaining mangrove fringe will be much denser compared to one that has been restored recently. Therefore, Linh et al. (2015) varied the density of the mangrove plants to simulate different states of the mangrove fringe.
The drag coefficient has an effect on wave attenuation by the mangroves. Increasing the drag coefficient induces a higher wave attenuation. Burger (2005) researched the value and sensitivity of the drag coefficient in her Master thesis. She concluded that although it is an important parameter, it is the hardest one to determine. In case of measurements, this parameter can be calibrated. For the Mekong Delta mangrove fringe, these measurements are not available and therefore a value of 0.25 will be used and tested in the numerical modelling phase. Hence the same drag coefficient is applied to all three vertical layers in SWASH and does not vary for different density scenarios. This is similar to the value Tas (2016) used in her research at the same project location.

In SWASH the vegetation can only be varied horizontally. As is explained in Chapter 4 the mangrove fringe exists of a certain zonation, dependent on the inundation frequency. This can not be simulated in SWASH and only one species, *Sonneratia sp*, will be modelled. The corresponding values of the parameters can be found in Table 6.1.

<table>
<thead>
<tr>
<th>Density [m$^{-2}$]</th>
<th>Height [m]</th>
<th>Diameter [m]</th>
<th>Sparse</th>
<th>Average</th>
<th>Dense</th>
<th>Drag coefficient [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roots</td>
<td>0.5</td>
<td>0.02</td>
<td>25</td>
<td>50</td>
<td>100</td>
<td>0.25</td>
</tr>
<tr>
<td>Stem</td>
<td>6</td>
<td>0.3</td>
<td>0.5</td>
<td>0.7</td>
<td>1.7</td>
<td>0.25</td>
</tr>
<tr>
<td>Canopy</td>
<td>2</td>
<td>0.5</td>
<td>50</td>
<td>100</td>
<td>100</td>
<td>0.25</td>
</tr>
</tbody>
</table>

Table 6.1: Input parameters to model the mangrove vegetation in SWASH.

### 6.6 Sediment

The cohesive sediment module in SWASH requires a set of input parameters: the critical bed shear stresses for erosion and for sedimentation, the entrainment rate for erosion, and the fall velocity of the sediment particles. Furthermore, the Nikuradse bed roughness height is required. In this research, this last parameter is based on the relationship proposed by Madsen (1993), see equation 6.1, as was mentioned in Liao (2015). The median sediment grain diameter in Tam Giang Dông and An Minh are based on measurements executed by GIZ (Albers & Stolzenwald, 2014). For Tam Giang Dông a median sediment grain diameter of 18 µm is used and for An Minh of 12 µm. This results in a Nikuradse roughness height of 0.00028 m and 0.00018 m respectively.

$$\frac{k_s}{d_{50}} = 15$$  \hspace{1cm} (6.1)

Where:

- $k_s$: Nikuradse roughness height [m]
- $d_{50}$: median grain diameter [m]

The critical bed shear stresses for erosion ($\tau_{cr,e}$) and sedimentation ($\tau_{cr,s}$), the entrainment rate for erosion ($E$) and the fall velocity ($w_s$) are harder to determine, as no measurements are available to calibrate the model. This will be elaborated upon in Chapters 7 and 8. The values of the parameters were set at $\tau_{cr,e}=0.5$ N/m$^2$, $\tau_{cr,s}=1000$ N/m$^2$, $E=0.00001$ kg/m$^2$/s and $w_s=1$ mm/s after extensive variation.
Chapter 7

Numerical modelling

For both case study areas two scenarios, the present situation regarding the bathymetry of the foreshore, and the proposed convex-up profile after nourishing, will be modelled numerically. The goal is to gain insight in how the design of the profile influences wave height, sediment concentration and occurring bed shear stresses in cross-shore direction. This chapter will first list the model requirements followed by the introduction of the numerical model SWASH. The last section contains an overview of the model set-up.

7.1 Model requirements

The goal of this research is to design a Building with Nature coastal protection strategy, including the mangrove fringe and its foreshore, through active sediment management. This implies that, when looking into a solution with mud nourishment, the design of the foreshore profile should generate conditions that meet the requirements of a natural habitat for a self-sustaining mangrove fringe under normal conditions. Furthermore, when storm conditions are present, the mangroves need to attenuate the waves to a maximum wave height of 0.5 m in order to ensure the stability of the earth dyke without a revetment.

The numerical model should give insight in the hydrodynamics and morphodynamics as well. The latter implies that the model should take the very fine sediment into account for the wave transformation, for the bed shear stress and how the sediment concentration differs in cross-shore direction. The morphodynamic response of the bed is outside the scope of this research. The model should be able to model the wave transformation from offshore to near shore and up to the shoreline, taking into account different dissipation mechanisms such as bottom friction, vegetation, wave breaking and the density effect of suspended sediment mixture in the water. One of the most important model requirements is the ability to model wave transformation on very gently slopes and wave attenuation by mangroves.

7.2 Model description SWASH

SWASH, short for Simulating WAves till SHore, is a non-hydrostatic wave model, applicable to predict the transformation of dispersive surface waves from offshore to the shoreline. SWASH is based on the nonlinear shallow water equations and includes the non-hydrostatic pressure term. The latter is important near shore, as the non-hydrostatic pressure term has to be taken into account, since it is no longer negligible compared to the hydrostatic pressure term. At offshore depth, the hydrostatic pressure assumption is valid. Near shore, when the non-hydrostatic pressure term becomes important, SWASH
makes use of the full vertical momentum equation.

For this research the occurring bed shear stress is of importance. It is possible to run SWASH in a multiple layer mode, thereby dividing the vertical domain in layers. By doing this, a more precise indication of the orbital wave velocity at the bed can be gained. This velocity at the bed is needed to determine the bed shear stress. Furthermore, SWASH is able to model the physical processes taking place at the Mekong Delta coast, such as wave propagation, bottom friction, wave attenuation induced by vegetation, cohesive sediment transport, wave breaking, frequency dispersion, shoaling, refraction, diffraction, nonlinear wave-wave interactions, partial reflection and transmission, wave run-up and run-down, and depth limited wave growth by wind.

7.3 Model set-up

SWASH was used in 1-D mode in this research for all runs. 1-D mode means that only one cross section perpendicular to the coastline will be modelled. The computational grid for all runs contains a grid cell size of 1 m. This is a high resolution, contributing to the accuracy of the results. The decision of the grid cell size was based on extensive variation and analysing the hydrodynamic model results with common sense, as no measurements were available to calibrate the model. The bathymetry was defined on a regular grid. The bathymetric profile varied per case study location and scenario. The vertical domain consisted of two equidistant layers. Furthermore, a storm with a duration of 6 hours was assumed. Each run time comprised 6 hours and 20 minutes, Chapter 8.1.1 will elaborate on this choice, including a spin-up time of 20 min.

Next to the included spin-up time the initial conditions for the water level and velocity components were set to zero. At the western offshore boundary waves were imposed via a JONSWAP wave spectrum, established from a significant wave height and peak period. Due to nonlinearities the wave height drops at the boundary, and therefore it was multiplied by a factor to compensate for the drop in wave height. The boundary was set weakly reflective in order to avoid the reflection of long waves. The eastern boundary included a sponge layer of 250 m to absorb wave energy at the boundary, in order for the wave to leave the computational domain freely. This prevents wave reflection, which influences the outcome of the model.

Bottom friction was included through the Nikuradse roughness height, as the logarithmic wall law needs to be applied when including cohesive sediment transport. The value was set to 0.00028 m and 0.00018 m for Tam Giang Dông and An Minh respectively, based on the median grains size of sediment samples. In order to use the module for cohesive sediment transport, values of the critical bed shear stresses for erosion ($\tau_{cr,e}$) and sedimentation ($\tau_{cr,s}$), the entrainment rate for erosion (E) and the fall velocity ($w_s$) are required. The values were set at $\tau_{cr,e}$=0.5 N/m$^2$, $\tau_{cr,s}$=1000 N/m$^2$, E=0.00001 kg/m$^2$/s and $w_s$=1 mm/s. The value of the fall velocity was based on literature (Ferre et al., 2005), who measured for grains with a $d_{50}=30 \mu\text{m}$ a fall velocity of 0.8 mm/s. After variation in the model this value was increased to 1 mm/s. The other three parameters have been varied extensively in order to obtain reasonable values for the sediment concentration. Similar values for the critical erosion and sedimentation rate were used, as in the research regarding a fine sediment nourishment of Julianus (2016), which were based on expert judgement. The initial sediment concentration was distributed uniformly and was set to 20 mg/l. The viscosity command was activated, which is compulsory when activating the cohesive sediment transport module, to activate vertical turbulent mixing and was mod-
eled by the standard kappa-epsilon model. The background viscosity was set to 0.0001 m²/s as is recommended in the SWASH user manual. The settings for the module to use the cohesive sediment module were varied extensively in order to get reasonable outcomes, this will be explained in more detail in Chapter 8.1.

Mangrove vegetation was modelled in SWASH as explained in Chapter 6. The values of the parameters (height, diameter, density and drag coefficient) were based on the work of Narayan (2009), Linh et al. (2015) and Burger (2005). The location of the vegetation was defined in a regular grid. As the vertical domain existed of two layers, the resolution was to coarse to account for the correct dissipation due to wave breaking in the surfzone and hence the command wave breaking was activated. If the wave breaking command is not used, the dissipation in the surfzone will be underestimated.

Upwind discretisation was used for the momentum equations and explicit time integration is applied with a Courant number restricted between 0.1 and 0.5. This is recommended when high waves are present and nonlinearities occur (Tas, 2016).

**Computational grid Tam Giang Đông**

The computational grid for Tam Giang Đông encompasses a length of 30 km when annual storm conditions are modelled. The mangrove vegetation starts at a distance of 10 km from the offshore boundary. The offshore boundary is located at x=0 m. The offshore boundary starts at depth of z=-10.5 m for the original bathymetric profile and z=-10 m for the convex-up shaped profile. In case of heavy storm conditions, the computational grid is extended with 5 km to ensure the breaking point of the wave is included in the domain. Therefore, the offshore boundaries will start at a depth of z=-15.5 m and z=-15 m respectively for the original and convex-up shaped profile. Under these conditions the mangrove vegetation starts at a distance of 15 km in front of the offshore boundary, as the offshore boundary is placed 5 km further offshore.

**Computational grid An Minh**

The computational grid for An Minh when annual storm conditions are modelled has a length of 30 km, the mangrove vegetation starts at a distance of 4.8 km from the offshore boundary. The offshore boundary is located at x=0 m. The offshore boundary starts at depth of z=-10 m for both the original bathymetric profile and the convex-up shaped profile. In case of heavy storm conditions, the computational grid is extended with 5 km to ensure the breaking point of the wave is included in the domain. Therefore the offshore boundaries will start at x=0 at a depth of z=18.33 m. Under these conditions the mangrove vegetation starts at a x=9.8 km, as the offshore boundary is placed 5 km further offshore.
Chapter 8

Results

This chapter presents the results of the numerical modelling and the proposed design of the building with nature solution. The first section is an analysis of the model performance, followed by the second and third section that will elaborate on the model results in a case study site specific manner. The fourth section gives an overview and short conclusion of the model results. The last part of this chapter contains the design. This part encompasses the final design of the foreshore including its profile, width of the mangrove fringe, the volume of the nourishment and nourishing techniques. Furthermore, possible design alternatives will be compared.

8.1 Model performance analysis

As there are no measurements available to calibrate the model and validate the model results, this section will assess the validity of the model by comparing the outcomes to the expected behaviour based on theory, literature, expert judgement and common sense.

8.1.1 Grid size and run time

In order to choose the grid size that will improve the accurateness of the model, the grid size was varied for the convex-up profile at low wave conditions with \( H_s = 0.5 \) m and without the mangrove vegetation. When the grid size cell is 1.0 m SWASH simulates the behaviour best as is expected on very gentle slopes: the shoaling behaviour continues longer until the wave steepness at the point of breaking is reached. When the grid size cell is chosen too small, the model shows unstable results (wiggles) when higher wave heights are modelled. Therefore, in this research a grid size cell of 1.0 m is chosen. Furthermore, as a storm duration of 6 hours is assumed in this research, a run time of 6 hours is used. In Appendix G more information on grid size choice and run time can be found.

8.1.2 Sediment concentration

The parameters that directly influence the sediment concentration are the critical bed shear stresses for erosion \( (\tau_{cr,e}) \) and sedimentation \( (\tau_{cr,s}) \), the entrainment rate for erosion \( (E) \) and the fall velocity \( (w_s) \). Vinh (2016) performed numerical simulations of suspended sediment dynamics due to seasonal forcing in the Mekong coastal area using Delft3D. Vinh used the following values for the sediment parameters with the same median grain size as measured by GIZ in the case study areas: \( \tau_{cr,e} = 0.2 \) N/m\(^2\), \( \tau_{cr,s} = 1000 \) N/m\(^2\), \( E = 0.00002 \) kg/m\(^2\)/s and \( w_s = 0.325 \) mm/s. The sediment concentration shown in Figure 8.1 and Figure 8.2 represents the sediment concentration in the second vertical layer in SWASH, the layer near the bed. In Figure 8.1 the yellow line represents these parameter settings.
under annual storm conditions. In this research the annual storm condition consists of an offshore wave height of 3.2 m, which occurs on average once a year. When the coastal area is experiencing calm conditions, with a near shore wave height in the order of 0.25 m, the maximum sediment concentration will be approximately 0-3 g/l, according to the measurements of GIZ (Albers & Von Lieberman, 2011). Average sediment concentrations are in the order of 0-0.2 g/l, according to measurements conducted by Vinh (2016). As can be seen, the yellow line is in the order of 0-160 g/l. Even for the higher wave conditions this is not a reasonable and expected outcome. Figure 8.1 shows that with increasing fall velocity and the threshold at which erosion occurs (increasing the value for $\tau_{cr,e}$), the sediment concentration drops significantly.

Figure 8.1: Comparison of the effect of the variation in fall velocity and variation of the critical bed shear stress for erosion on the sediment concentration on a very gentle slope (1:1000) of a convex-up profile. The offshore wave height is 3.5 m with an offshore depth of MSL -10 m.

Xiang-Yu (2014) performed a study on sediment concentration distribution beneath breaking waves on a muddy coast. According to his experiments with a mud density that is similar to what is present at the Mekong Delta coast and a similar wave height - water depth ratio, the sediment concentration at the point where the wave breaks, or to be more complete and correct, where the wave is significantly dissipated by the bed, is in the order of 10-15 g/l. This concentration is depth averaged over the layer near the bed. The layer covers half of the entire water column, which is similar to the vertical domain of the SWASH model set-up in this research.

To adjust the model to get the expected outcome for the sediment concentration literature was consulted with respect to model parameters. The critical bed shear stress for erosion was set to $\tau_{cr,e}=0.5$ N/m$^2$, following Julianus (2016), based on expert judgement. The critical bed shear stress for sedimentation remained $\tau_{cr,s}=1000$ N/m$^2$. The erosion parameter was set to 0.00001 kg/m$^2$/s after variation. The value of the fall velocity was based on literature of Ferre (2005), who measured a fall velocity of 0.8 mm/s for grains with a $d_{50}=30$ μm.

In Figure 8.2 the fall velocity was varied to arrive at the expected outcome of the sediment concentration following Xiang-Yu (2014). The Figure shows the sediment concentration at the last time step, after 6 hours, and the mean sediment concentration during the 6 hours, for two different fall velocities. Based on the research of Xiang-Yu (2014) a fall velocity of 1.0 mm/s was chosen. This is supported by the research of Julianus (2016), where the same value was used.
8.1. Model performance analysis

Figure 8.2: Comparison of the effect of a final variation in fall velocity on the sediment concentration on a very gentle slope (1:1000) of a convex-up profile. The offshore wave height is 3.5 m with an offshore water depth of 11.25 m.

8.1.3 Very gentle slopes and shallow water

In Chapter 7 it was explained that the performance of the model on very gentle slopes is of utmost importance. Furthermore, wave transformation in very shallow water depths is important in this research as well. Figure 8.3 shows the wave transformation and water depth modelled in SWASH on a convex-up profile with a very gentle slope of 1:1000 that continues into a bed level with a constant height starting at x=12 km. From x=10 km in shoreward direction the wave height over depth ratio increases rapidly to the point of the horizontal bed level, where the wave height exceeds the water depth. This is physically not possible and further investigation is required.

Figure 8.3: Wave height and water depth along the convex-up profile without vegetation.

A method to analyse the model results is by examining the variance density spectrum; the wave spectrum. At the offshore boundary a JONSWAP spectrum is imposed with a certain shape. By looking at the transformation of the shape of the wave spectrum in onshore direction valuable information can be gained. Using a Matlab script it is possible to build the wave spectra out of time series of the surface elevation at different points in cross-shore direction. Figure 8.4 displays the wave spectrum according to SWASH at several points on the convex-up profile. The offshore boundary is located at x=0 km at a depth of 10 m relative to MSL.
Chapter 8. Results

Figure 8.4: Evolution of the wave spectrum from an offshore JONSWAP spectrum ($H_s=3.2$ m, $T_p = 8.0$ s, x=0 km and a water depth of $d=11.25$ m) to near shore with water depths of respectively 10, 6.5, 3.2 1.8 and 0.7 m.

The wave spectrum shows a very low frequency peak for $f \leq 0.01$ Hz. In nature waves with such low frequencies do not exist. The frequency peak is generated mathematically by the numerical model in order to compute the wave height. It is the natural frequency of the model, thus a free vibration of the model itself. Figure 8.4 shows that this very low frequency continues in shallow water depths and hardly loses energy. This is the reason why the wave height as shown in Figure 8.3 exceeds the water depth.

After filtering the wave spectrum for all frequencies lower than 0.01 Hz, the wave height was computed from the new filtered spectrum. This is displayed in Figure 8.5 by the red line, where the blue line represents the wave height transformation of the original SWASH computation. The red line meets the expected outcome of the wave height transformation on a very gentle slope in shallow water. The wave height no longer exceeds the water depth. It seems that the model performs well on very gentle slopes in deeper water and that the problem occurs in the shallow water zone. This is due to the very low frequencies generated by the model, as this has a significant effect in shallow water and is of less influence on the wave transformation in deeper water. In deeper water most of the energy of the wave is located in higher frequencies between 0.1 and 0.2 Hz. Further wave height calculations in this report will be computed with the filtered spectrum approach.

Figure 8.5: Wave height transformation along the convex-up profile without vegetation, comparing the wave height according to the wave spectrum of SWASH and the filtered spectrum.

8.1.4 Vegetation

In section 8.2 in Figure 8.7 four situations are compared for vegetation. The light blue line represents the wave transformation along the convex-up profile without mangrove vegetation. The other three situations differ in density of the mangrove vegetation. The
wave attenuation due to the mangrove fringe meets the expectations and observations as is documented in literature (Linh et al., 2015).

### 8.1.5 Conclusion model performance analysis

In Table 8.1 the most important parameters that have been varied during the model set-up are listed. An overview of their range of variation, their final value and where this value is based on, and their reliability is given. The sediment parameters were based on the wave height of the original wave spectrum. However, as the sediment concentration was validated with concentrations that occur under the point of wave breaking, this was not influenced by the low frequency peak. The very low frequencies are of influence in the shallow water zone and the point of wave breaking is present in the deeper water part.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Varied between</th>
<th>Final value</th>
<th>Based on</th>
<th>Reliability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grid size</td>
<td>0.71-2.5 m</td>
<td>1.0 m</td>
<td>SWASH manual and common sense</td>
<td>Reliable</td>
</tr>
<tr>
<td>Run time</td>
<td>1-8 h</td>
<td>6 h</td>
<td>Assumed storm duration and no difference in wave height and stretch between a run of 6 h and 8 h during storm conditions</td>
<td>Reliable</td>
</tr>
<tr>
<td>( \tau_{cr,e} )</td>
<td>0.2-0.5 N/m²</td>
<td>0.5 N/m²</td>
<td>Literature, expert judgement and sediment concentration outcome of the model based on measurements in a flume</td>
<td>Questionable</td>
</tr>
<tr>
<td>( \tau_{cr,s} )</td>
<td>no</td>
<td>1000 N/m²</td>
<td>Literature and expert judgement</td>
<td>Reliable</td>
</tr>
<tr>
<td>( E )</td>
<td>0.00001-0.00002 kg/m²/s</td>
<td>0.00001 kg/m²/s</td>
<td>Literature and expected sediment concentration outcome of the model based on measurements in a flume</td>
<td>Quite reliable</td>
</tr>
<tr>
<td>( w_s )</td>
<td>0.325-1.0 mm/s</td>
<td>1.0 mm/s</td>
<td>Measurements related to grain size and expected sediment concentration outcome of the model based on measurements in a flume</td>
<td>Questionable</td>
</tr>
</tbody>
</table>

Table 8.1: Reliability of the settings of parameters in SWASH.

The sediment concentration is very dependent on three parameters: the critical bed shear stress for erosion, the entrainment rate for erosion and the fall velocity. As there are no measurements available for these parameters in the Mekong Delta coastal area, nor are there measurements for the sediment concentration during annual storm and heavy storm conditions as they are defined in this research, the outcome of the model should be handled with caution. The performance of the model on very gentle slopes in shallow water depths is not reliable if the wave spectrum is not filtered. After filtering the spectrum for very low frequencies, the model shows reliable results for wave heights in situations with and without vegetation.
8.2 Model results Tam Giang Đông

This section presents the model results of the case study site at the east coast, Tam Giang Đông. First, the wave transformations for the convex-up profile and the original profile during annual storm conditions are given. Secondly, the influence of different vegetation density scenarios is investigated and whether the requirement of a maximum wave height of 0.5 m in front of the mangrove fringe is fulfilled during annual storm conditions. Next, the bed shear stress, sediment concentration and sediment sources and sinks for the convex-up profile and present situation are compared.

The second part of this section elaborates on the heavy storm conditions. Equally to the first part of this section, the wave transformation is given to determine the minimal width of the mangrove fringe. The last part of this section gives the wave transformation during average conditions, to gain more insight into the chances for young mangrove trees to develop with respect to their maximum wave height requirements.

8.2.1 Annual storm conditions

<table>
<thead>
<tr>
<th>Case study site</th>
<th>Parameter</th>
<th>Annual storm conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tam Giang Đông</td>
<td>Scenario</td>
<td>Present</td>
</tr>
<tr>
<td></td>
<td>Return period</td>
<td>1 year</td>
</tr>
<tr>
<td></td>
<td>Water level</td>
<td>MSL +1.25 m</td>
</tr>
<tr>
<td></td>
<td>Offshore wave height</td>
<td>3.2 m</td>
</tr>
<tr>
<td></td>
<td>Offshore wave period</td>
<td>8.0 s</td>
</tr>
</tbody>
</table>

Table 8.2: Overview of annual storm conditions Tam Giang Đông.

Wave transformation convex-up profile

In Figure 8.6 the convex-up profile of Tam Giang Đông is shown. The mangrove fringe starts at MSL (x=10 km and z=0 m) and covers the entire onshore stretch.

Figure 8.6: Convex-up bathymetric profile Tam Giang Đông. The dark blue line indicates MSL and the light blue lines indicate the tidal range.

In Figure 8.7 the wave transformation during annual storm conditions on the convex-up profile in Tam Giang Đông is displayed. The light blue line represents the wave transformation when no vegetation is present. The dark blue line displays the wave transformation when an average density of the mangrove fringe is assumed. It can be seen that during annual storm conditions it takes a mangrove fringe width of approximately 300 m to attenuate the waves to reduce the wave height to 0.5 m, the maximum wave height the dyke without a revetment can withstand. However, the minimum width of the mangrove fringe...
8.2. Model results Tam Giang Dông

will be based on heavy storm conditions.

During annual storm conditions the requirement to reduce the wave height in front of the mangroves to 0.5 m has not been fulfilled, as the wave height is 1.15 m. This means that during a storm with a return period of one year, the wave height in front of the mangroves is too high. Note that this is modelled while the water level is at high tide level constantly, which gives the highest wave height that can occur. Furthermore, a maximum hydrodynamic force is translated into a requirement of a maximum wave height in front of the mangrove fringe of 0.5 m during annual storm conditions. However, as explained in Chapter 4.2, the frequency at which mangroves can withstand a wave height of 0.5 m is not clear from literature. Annual storm conditions only lasts for several hours per year and therefore this requirement with respect to the maximum wave height in front of mangrove fringe could be quite strict.

Figure 8.7: Wave transformation along the convex-up profile of Tam Giang Dông, comparing no vegetation with sparse, average and dense vegetation. The mangrove vegetation starts at x=10 km.

Figure 8.6 displays the bathymetric profile of the present situation. When comparing the present situation with the situation with the proposed nourishment, it is evident that the convex-up profile has a positive effect on the wave transformation, see Figure 8.9. The wave height at the convex-up profile decreases faster and is significantly lower in front of the mangroves than at the existing profile. This leads to milder hydrodynamic forces acting on the mangrove fringe and less erosion of the shoreline, which contributes positively to the self-sustainability of the mangrove belt.

Figure 8.8: Bathymetric profile Tam Giang Dông in the present situation without a nourishment. The dark blue line indicates MSL and the light blue lines indicate the tidal range.
Figure 8.9: Wave transformation along the existing profile and convex-up profile, comparing the wave height in front of the mangrove fringe during annual storm conditions. The mangrove vegetation starts at $x=10$ km, as does the cliff of the existing profile.

Bed shear stress

To gain insight in the influence of different foreshore profiles on the erosion process, the bed shear stress is important. If the bed shear stress exceeds a certain threshold, sediment particles will erode from the bed. Figure 8.10 shows the bed shear stress obtained from time integration of the velocity at the bed to get the orbital velocity amplitude. In Chapter 2 in paragraph 2.1.2 it is explained how to calculate the bed shear stress when the orbital velocity amplitude is known.

The velocity at the bed is one of the output parameters of SWASH. Figure 8.10 displays the bed shear stress along the two profiles without vegetation. The blue and red line are obtained from the absolute values of the velocity at the bed. The green and black line represent the bed shear stress when negative and positive bed velocities are both taken into account, hereafter named as the net bed shear stress. The net bed shear stress is based on the mean of the negative and positive velocities near the bed. In offshore areas these are quite balanced and this is why the net bed shear stress is displayed as almost zero. A negative velocity is offshore directed, as a positive velocity is onshore directed, representing the direction of the sediment transport.

Figure 8.10: Bed shear stress along existing profile and convex-up profile, without vegetation during annual storm conditions. The cliff of the existing profile starts at $x=10$ km.

For stirring of the bed and hence bringing the sediment in the water column, it is not of importance which way the velocity vector is directed. Therefore, this research will base the magnitude of the bed shear stress on the absolute value of the velocity. The blue and red line represent the occurring bed shear stresses. Note that the bed shear stresses are based on the velocity output parameter of SWASH. The orbital velocity near the bed
8.2. Model results Tam Giang Đông

in SWASH is based on the wave height of the not filtered spectrum. Therefore, the bed shear stress in shallow water \((x=10 \text{ km and shorewards})\) is based on too large waves. Hence, only the bed shear stresses ranging from the offshore boundary until \(x=10 \text{ km}\) are reliable. The grey coloured part of Figure 8.10 represents the not reliable part of the figure, because of the non filtered spectrum. In addition, it is not clear whether bed shear stress can be simulated accurately by SWASH within the mangrove forest. This means that sediment concentration and transport within the mangrove forest would be based on unreliable results of the bed shear stress. At the point of wave breaking a peak in bed shear stress is visible. Furthermore, when comparing the bed shear stress in the current situation to the one after the nourishment, the bed shear stress has decreased strongly in onshore direction.

**Sediment concentration**

Figure 8.11 contains the sediment concentration along both profiles. The sediment concentration can be linked to the occurring bed shear stress. Note that the sediment concentration in shallow water is based on too high wave heights as well. Therefore, the grey part of the figure is not representative for the sediment concentration in real life situations. At \(x=10 \text{ km}\) a peak in sediment concentration is visible for the existing profile. This is the location where the wave collapses against the cliff. At this point strong erosion occurs. When the slope is mild and convex-up the sediment concentration is lower than in the existing situation, similar to the decrease in occurring bed shear stress.

![Figure 8.11: Sediment concentration along the existing profile and convex-up profile of Tam Giang Đông, without vegetation during annual storm conditions.](image)

**Sediment sources and sinks**

In case of cohesive sediment SWASH uses the Partheniades-Krone formulations to compute the mass exchange of suspended sediment between the bed and the water column. These formulations are based on the erosion and deposition fluxes of the sediment. If the deposition flux exceeds the erosion flux, sedimentation occurs. The Partheniades-Krone formulations can be found in Chapter 2 in Equations 2.5 and 2.6.

The upper part of Figure 8.12 shows the sediment erosion flux. The red line represents the gross erosion flux on the original profile, while the blue line does so for the convex-up profile. The convex-up profile has a lower gross erosion flux when approaching the shoreline. However, the bottom figure displays the net sedimentation flux, as in this figure the gross sediment erosion flux is extracted of the gross sediment deposition flux. A sediment source is a point in the coastal area from which sediment is supplied, such as an eroding cliff in the Mekong Delta or the Mekong River. A sediment sink is a
point where the sediment is lost. In case the net sediment flux is positive, sedimentation occurs. A negative net sediment flux indicates erosion. It is remarkable that according to the results displayed in Figure 8.12 for both profiles nowhere in cross-shore direction net erosion occurs, as the sediment flux remains positive. This is in contradiction with observed erosion in the coastal area of the case study site. Moreover, the erosion flux is at its maximum 0.04 g/m²/s for annual storm conditions, which is less than 1 kg/m² during a 6 hour annual storm. This is almost negligible. Despite the efforts made to determine accurate values for all of the parameters in SWASH, the results for the sediment sinks and sources are questionable.

![Sediment erosion flux Tam Giang Dong](image1)

![Sediment sinks and sources Tam Giang Dong](image2)

**Figure 8.12:** Sediment erosion flux Tam Giang Dong and sediment sinks and sources along the existing and convex-up profile of Tam Giang Dong during annual storm conditions.

### 8.2.2 Heavy storm conditions

<table>
<thead>
<tr>
<th>Case study site</th>
<th>Parameter</th>
<th>Heavy storm conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tam Giang Dong</td>
<td>Scenario</td>
<td>Future</td>
</tr>
<tr>
<td></td>
<td>Return period</td>
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</tr>
<tr>
<td></td>
<td>Water level</td>
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</tr>
<tr>
<td></td>
<td>Offshore wave height</td>
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</tr>
<tr>
<td></td>
<td>Offshore wave period</td>
<td>10.1 s</td>
</tr>
</tbody>
</table>

**Table 8.3:** Overview of heavy storm conditions Tam Giang Dong.

The water level for heavy storm conditions was set to MSL +2.6 m, consistent with the future scenario, taking into account storm surge, sea level rise, land subsidence and water level set-up due to wind friction.

**Wave transformation**

Wave transformation during heavy storm conditions is important to model in order to give insight in the minimal required width of the mangrove fringe. The minimum width of the mangrove fringe is based on the requirement of a maximum wave height of 0.5 m in
front of the dyke. The dyke is located behind the fringe in order to protect the hinterland. The maximum wave height of 0.5 m in front of the dyke without a revetment ensures its stability and limits the needed maintenance as well.

Figure 8.13 shows that the wave height in front of the mangrove belt is significantly lower for the convex-up profile than the existing profile. When looking at the two runs which are modelled with vegetation for both profiles, it stands out that they drop to the same height when reaching the vegetation at x=15 km. Nevertheless, when zooming in on the location it can be observed that the red line (existing profile no vegetation) already drops to a wave height of 2.0 m. This indicates that the profile itself introduces a drop in wave height, causing loss of wave energy due to collapsing of the wave against the cliff. The mangrove vegetation is therefore not responsible for the drop in wave height from 4.0 m to 2.0 m and only takes care of the decrease in wave height from 2.0 m until the wave is totally dissipated. The wave attenuation due to the cliff is not favourable since this causes severe erosion.

In combination with the convex-up profile it takes a mangrove fringe in the order of 700 m to reduce the wave height to the required 0.5 m during storm conditions with a return period of 100 years. This is consistent with observations during storm Rammasun. Storm Rammasun corresponded to a storm with a return period of 100 years. It took a mangrove fringe width of 650 m to reduce the wave height with 80 percent, in the present situation (Dung et al., 2013). The present situation does not take the relative sea level rise into account. If Rammasun would occur in the future scenario, with a higher water level, the minimum required mangrove fringe width would increase. However, due to the earth body dyke behind the mangroves, the wave has to be attenuated less, up to 0.5 m. Therefore, the computed minimum mangrove width of 700 m is a plausible outcome.

![Wave transformation during heavy storm conditions Tam Giang Đông](image)

**Figure 8.13:** Wave transformation during heavy storm conditions in the future along the existing profile and convex-up profile for Tam Giang Đông. The mangrove fringe starts at x=15 km in case vegetation is simulated.

### 8.2.3 Average conditions

<table>
<thead>
<tr>
<th>Case study site</th>
<th>Parameter</th>
<th>Average conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tam Giang Đông</td>
<td>Scenario</td>
<td>Future</td>
</tr>
<tr>
<td></td>
<td>Return period</td>
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</tr>
<tr>
<td></td>
<td>Water level</td>
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</tr>
<tr>
<td></td>
<td>Offshore wave height</td>
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</tr>
<tr>
<td></td>
<td>Offshore wave period</td>
<td>5.8 s</td>
</tr>
</tbody>
</table>

**Table 8.4:** Overview of average conditions Tam Giang Đông.
During average conditions the requirement of a maximum wave height of 0.5 m in front of the mangroves is not met, see Figure 8.14. At x=10 km at which the mangrove fringe starts, the wave height is 0.8 m. It should be noted that this is modelled while assuming the peak of the storm lasts for 6 hours during a constant high tide water level, at which the highest possible wave height occurs.

Figure 8.14: Wave transformation during average conditions in the present along the convex-up profile for Tam Giang Đông, with and without vegetation. Vegetation starts at x=10 km in case it is simulated.

8.3 Model results An Minh

This section presents the model results of the case study site at the west coast, An Minh. First the wave transformation during annual storm conditions on the convex-up profile is discussed for both situations: with and without vegetation. Next to this, the wave transformation on the convex-up profile is compared with the present situation including the original bathymetric profile, with vegetation. Furthermore, the influence of the convex-up profile on the bed shear stress and sediment concentration is discussed. The second part of this section elaborates on the heavy storm conditions in order to determine the minimum required mangrove width.

8.3.1 Annual storm conditions

<table>
<thead>
<tr>
<th>Case study site</th>
<th>Parameter</th>
<th>Annual storm conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>An Minh</td>
<td>Scenario</td>
<td>Present</td>
</tr>
<tr>
<td></td>
<td>Return period</td>
<td>1 year</td>
</tr>
<tr>
<td></td>
<td>Water level</td>
<td>MSL +0.45 m</td>
</tr>
<tr>
<td></td>
<td>Offshore wave height</td>
<td>2.9 m</td>
</tr>
<tr>
<td></td>
<td>Offshore wave period</td>
<td>7.9 s</td>
</tr>
</tbody>
</table>

Table 8.5: Overview of annual storm conditions An Minh.

Wave transformation convex-up profile

Figure 8.15 displays the convex-up profile of An Minh. The mangrove fringe starts at MSL (x=4.8 km and z=0 m) and extends to the entire onshore part of the bathymetric profile.
8.3. Model results An Minh

Figure 8.15: Convex-up bathymetric profile An Minh. The dark blue line indicates MSL and the light blue lines indicate the tidal range.

Figure 8.16: Wave transformation along the convex-up profile of An Minh during annual storm conditions, comparing no vegetation with average vegetation. The mangrove vegetation starts at x=4.8 km. At this point the wave height is approximately 0.4 m and hence the requirement of a maximum wave height of 0.5 m in front of the mangrove fringe is met.

Figure 8.17: Bathymetric profile An Minh in the present situation without a nourishment. The dark blue line indicates MSL and the light blue lines indicate the tidal range.

When comparing the present situation with the situation after nourishing, to create a convex-up profile, it is clear that similar to the case study site at the east coast, the convex-up profile has a positive effect on the wave transformation, see Figure 8.18. The wave height at the convex-up profile decreases faster and is significantly lower in front of
the mangroves than at the existing profile. Figure 8.17 shows the existing profile with a cliff.

**Figure 8.17:** Shows the existing profile with a cliff.

**Figure 8.18:** Wave transformation along the existing profile and convex-up profile, comparing the wave height in front of the mangrove fringe during annual storm conditions. The mangrove vegetation starts at x=4.8 km, the cliff of the existing profile starts at x=4.5 km, increasing in steepness up to x=4.8 km.

**Bed shear stress**

Figure 8.19 displays the bed shear stress in the present situation and after nourishing, at the case study site of An Minh, during annual storm conditions. Similarly to the bed shear stress of Tam Giang Dong it is based on the absolute values of the orbital velocity at the bed, which is based on the wave height of the unfiltered spectrum of SWASH. Therefore, the bed shear stress at shallow water (x=4.8 km and the stretch in onshore direction, coloured grey) is overestimated.

**Figure 8.19:** Bed shear stress along existing profile and convex-up profile of An Minh, without vegetation during annual storm conditions. The cliff of the existing profile starts at x=4.5-4.8 km.

The bed shear stress on the convex-up profile decreases from the wave breaking point onwards, in shoreward direction. Therefore, less sediment will be eroded at the bed, compared to the existing situation.

**Sediment concentration**

Figure 8.20 shows the sediment concentration along both profiles. Note that the sediment concentration in shallow water is based on too high wave heights, this part is coloured grey in the Figure and is not representative for the occurring sediment concentration in reality. At x=4.8 km a peak in sediment concentration is visible for the existing profile, at the location where the wave collapses against the cliff. This induces a big sediment erosion flux, and a lot of sediment is transported into the water column. The convex-up
profile leads to a lower sediment concentration in onshore direction, and, as the waves are lower, the bed shear stress decreases and therefore less sediment is transported into the water column.

![Sediment concentration existing profile and convex-up profile An Minh](image)

**Figure 8.20:** Sediment concentration along the existing profile and convex-up profile of An Minh, without vegetation during annual storm conditions.

### 8.3.2 Heavy storm conditions

<table>
<thead>
<tr>
<th>Case study site</th>
<th>Parameter</th>
<th>Heavy storm conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>An Minh</td>
<td>Scenario</td>
<td>Future</td>
</tr>
<tr>
<td></td>
<td>Return period</td>
<td>100 years</td>
</tr>
<tr>
<td></td>
<td>Water level</td>
<td>MSL +1.3 m</td>
</tr>
<tr>
<td></td>
<td>Offshore wave height</td>
<td>5.7 m</td>
</tr>
<tr>
<td></td>
<td>Offshore wave period</td>
<td>9.8 s</td>
</tr>
</tbody>
</table>

**Table 8.6:** Overview of heavy storm conditions An Minh.

The water level was set to MSL +1.3 m during heavy storm conditions, consistent with the future scenario, taking into account storm surge, sea level rise, land subsidence and water level set-up due to wind friction.

**Wave transformation**

Figure 8.21 shows the wave transformation on the convex-up profile during storm conditions. A minimal mangrove fringe width of 350 m is required to attenuate the waves to a wave height of 0.5 m, in order to meet the requirement of the maximum wave height in front of the dyke without a revetment.

![Wave transformation during heavy storm conditions on the convex-up profile An Minh](image)

**Figure 8.21:** Wave transformation during heavy storm conditions in the future along the convex-up profile for An Minh. The mangrove fringe starts at x=9.8 km.
8.4 Conclusion model results

The wave height compared to the water depth on the foreshore is in general high when calculated by SWASH, especially for a muddy coast. It seems that the model does not sufficiently account for wave attenuation due to the mud and the very gentle slope, when one compares the results with Chapter 2.1.2 and literature. Measurements documented in literature give a maximum wave height over depth ratio varying between 0.15 and 0.5 (Friedrichs, 2011). SWASH accounts for a maximum wave height over depth ratio at the upper limit of this ratio, or exceeds this ratio. This will be discussed extensively in Chapter 9.2.

The convex-up profile significantly decreases the wave height in front of the mangroves during annual storm and heavy storm conditions, compared to the existing bathymetric profile. The decrease in wave height is continuous and does not collapse at a cliff during storm conditions, as it does in the current situation. Furthermore, the convex-up profile decreases the bed shear stress on the upper part of the profile, which decreases the gross erosion of sediment. Due to a contradiction between the outcome of the sediment sinks and sources of the model and observed erosion, the effect on the net sediment flux can not be substantiated with the model results.

The requirement of a maximum wave height of 0.5 m in front of the mangroves during annual storm conditions in the present situation, with a convex-up profile, is not met in Tam Giang Dông as the wave height is 1.15 m. The wave height over depth ratio at this point is 0.64, which is too high for a muddy gentle slope, according to observations documented in literature. The requirement of a maximum wave height in front of a mangrove fringe of 0.5 m might be quite strict, as was explained in Chapter 8.2.1.

When average conditions are present during high tide level at Tam Giang Dông, the wave height in front of the mangroves at MSL could be still too high for young mangrove trees to develop. In this case the wave height over depth ratio is 0.49. When average conditions occur during lower tidal levels, the requirement will be met.

At the case study site on the west coast, An Minh, the maximum wave height of 0.5 m in front of the mangroves during annual storm conditions in the present situation on the convex-up profile is met. The requirement of a maximum wave height behind the mangroves, during heavy storm conditions in the future, requires a mangrove fringe width of 700 m in Tam Giang Dông and 350 m in An Minh. An overview of all the outcomes per scenario and case study site can be found in Table 8.7.

The model results can lead to several conclusions. First of all, the convex-up profile, which can be achieved by a nourishment, does improve the conditions for mangroves to develop. Furthermore, an alternative could be proposed, such as adding artificial protection (for example Melaleuca fences) for the case study site at the east coast. This will assure mild wave conditions in which young mangrove trees can develop. Next to this, the model is perhaps not performing optimal on very gentle mud slopes, especially in combination with shallow water, because it overestimates the wave height.
8.5. Design

8.5.1 Foreshore profile and mangrove fringe

Figures 8.22 and 8.23 display the designs of the foreshore and the wave transformation on the different foreshore designs for the case study site at the east coast. The designs represented by the brown and red line for the east coast, at Tam Giang Đông, deviate from the modelled foreshore profile in SWASH, which is the grey line, as discussed in the Model Results section in this chapter. This is in order to ensure the minimal mangrove fringe width of 700 m and to adapt the design as much as possible to the existing profile. The new designs have been modelled in SWASH, see Figure 8.23, and the option displayed by the brown line, nourishment 2, gives approximately the same wave height ($H_s=1.13 \text{ m}$) in front of the mangrove fringe as the former profile design, nourishment 1. Nourishment option 1 and 2 have the same effect on the wave transformation, however nourishment option 2 is adapted to the original profile and creates an extra 1000 m width for mangroves to develop. Nourishment option 3 reduces the nourishment volume significantly, as will be elaborated on in Chapter 8.5.2.

The mangrove fringe at the east coast could start at $x=9 \text{ km}$ at MSL for nourishment option 2 and 3, developing in landward direction. However, when the hydrodynamic forces on the fringe are still too high the mangrove fringe will start higher in the profile. There is a limit to this movement of the starting point of the mangrove fringe as the minimal mangrove width has to be met in combination with an appropriate internal zonation. This point is at $x=9.5 \text{ km}$ and encompasses a reduction in water depth of 0.5 m. According to the upper limit of the wave height over depth ratio of 0.5, this means a minimal reduction in wave height of 0.25 m. Based on the model results, the wave height in front of the mangrove fringe will be in the order of 0.8 m and still exceeds the requirement of 0.5 m. Extra measures will be proposed in Chapter 8.6.

<table>
<thead>
<tr>
<th>Case study site</th>
<th>Parameter</th>
<th>Annual storm conditions</th>
<th>Heavy storm conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tam Giang Đông</td>
<td>Scenario</td>
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<td>Future MSL +2.6 m</td>
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<td>Water level</td>
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<td>10.1 s</td>
</tr>
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<td></td>
<td>Offshore wave period</td>
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<td></td>
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<tr>
<td></td>
<td>H mangroves filtered model</td>
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<td>Minimum width mangroves</td>
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<tr>
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<td>Scenario</td>
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<td>Water level</td>
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<td>5.7 m</td>
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<td></td>
<td>Offshore wave height</td>
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<td>9.8 s</td>
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<td></td>
<td>Offshore wave period</td>
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<td>H mangroves model</td>
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<td>-</td>
</tr>
<tr>
<td></td>
<td>Minimum width mangroves</td>
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<td>350 m</td>
</tr>
</tbody>
</table>

Table 8.7: Overview of the scenarios and outcomes. $H$ is the wave height in front of the mangrove fringe, respectively for the outcome of the filtered spectrum.
Figure 8.22: Foreshore design Tam Giang Dông. The dark blue line represent MSL and the light blue lines represent the tidal range.

Figure 8.23: Wave transformation along different foreshore designs for Tam Giang Dông, during annual storm conditions and with mangrove vegetation.

Figure 8.24: Foreshore design An Minh. The dark blue line represents MSL and the light blue lines represent the tidal range.

The situation in An Minh is modelled in SWASH with the design displayed by the brown line in Figure 8.24 and this gives good results, as is explained in Chapter 8.3. The mangrove fringe at the west coast should be able to start at MSL, as the wave height during annual storm conditions at high tide level does not exceed 0.5 m. The minimal width of the mangrove fringe is 350 m for the west coast, and therefore the mangrove fringe continues for 80 m on the horizontal bed level at the upper part of the foreshore profile. To reduce the nourishment volume, nourishment option 2 is displayed in Figure 8.24 as well. The differences in bathymetric profile of nourishment option 1 and 2 for An Minh have the same effect on the wave transformation as nourishment options 2 and 3 do for the wave transformation in Tam Giang Dông. Breaking of the wave is more or less at the same depth, but due to the change in bathymetric profile for the nourishments...
with the reduced volume, this occurs closer to shore. The wave heights in front of the mangrove fringe are the same for nourishment option 1 and 2 for An Minh.

8.5.2 Nourishment

Volume in case of mud

A nourishment exists of an initial volume and a wear layer. The wear layer for the two case study sites is based on the present erosion rate as is determined in Chapter 3.5.4 and hence is the absolute minimum nourishment volume. The nourishment wear layer probably will erode at a higher rate than the eroding cliffs, due to the looser packed sediment after nourishing. In case this minimum volume is nourished, the shoreline would, in theory, become stable and would remain in its present situation. This means that the mangrove fringe probably will not recover and will not protect the hinterland sufficiently.

The maximum nourishment volume is the initial volume based on the convex-up profile plus the wear layer. For Tam Giang Dông this comprises an initial volume of 12,275 m$^3$/m for nourishment option 2 and a wear layer of 250 m$^3$/m/year. For an interval rate of 5 years between nourishing activities the total volume would be 13,525 m$^3$/m. Nourishment option 3 has an initial volume of 6,275 m$^3$/m. The total volume for nourishment option 3 comprises 7,525 m$^3$/m for 5 years, which means a volume reduction of 45 percent in comparison with nourishment option 2.

For An Minh the foreshore design for nourishment option 1 needs an initial volume of 7,375 m$^3$/m and a wear layer of 109 m$^3$/m/year. The total volume of the mud suppletion for 5 years for nourishment option 1 hence encompasses 7,920 m$^3$/m. Nourishment option 2 needs an initial volume of 1,875 m$^3$/m, and including a wear layer for 5 years this means a total volume of 2,420 m$^3$/m. The maximum slope of nourishment option 2 is 1:340, this is quite steep for a muddy profile. However, these kind of slopes can be found in the upper part of the foreshore profile at the west coast. This indicates that the muddy sediment at the west coast is able to form and maintain such a relative steep slope.

Volume in case of a sand-mud mixture

Nourishing with a sandy mixture in stead of pure mud can decrease the nourishment volume. First of all, one may assume that sandy mixtures erode at a lower rate than a pure mud mixture, as there is more energy needed to pick up sand particles than the finer mud particles. The difference in erosion rate between different sand-mud mixtures and pure mud is difficult to estimate. Available formulae allow for comparisons to be made between very fine sand and coarser sand, but not between sand, a sand-mud mixture and pure mud. In addition, different formulae show different sensitivities regarding the $d_{50}$ of a sediment mixture as well. To give a rough indication the simplified formula for long shore transport of van Rijn is used (Van Rijn, 2002), see Appendix H. An assumption is made that the sandy mixture near the east coast consists of very fine sand in the order of 100-200 µm. The formula shows for relatively small particle sizes a decrease in long shore transport of 50 percent when the $D_{50}$ doubles. However, the sediment mixture will not exist of pure sand, so the influence of the finer mud particles is of importance. According to laboratory and field observations of van Rijn (1993) the pick up rate of sand particles decreases by the presence of mud particles (Van Rijn, 2002). Although the simplified formula of van Rijn is not valid for the very fine mud particles with $d_{50}=18$ µm at the east coast, a rough estimate can be made by using this formula in combination with the decreased pick-up rate due to the mud. The erosion rate at the east coast would decrease from the current erosion rate of 250 m$^3$/m/year, to an erosion rate in the order of 50 m$^3$/m/year for a
mixture with a high sand content. Furthermore, sand nourishments tend to stay close to the coast, whereas the behaviour of mud nourishments is unclear. A third benefit of a sandy mixture is that a steeper equilibrium profile can be established. A steeper foreshore profile requires a much smaller nourishment volume to establish a convex-up profile.

Dredging location

For the case study site at the east coast, Tam Giang Đông, geological data show a sandy mixture that can be found at approximately 15 km in front of the coast, see Figure 8.25. This sediment mixture contains a sand content up to 75 percent. As can be seen in the same figure, at the west coast, only mud is available. The distance to a mud-sand mixture for the case study site at the west coast is too big to be cost-efficient. The reason to look at sediment mixtures that contain sand is the possibility to nourish with sand or sand-mud mixtures in stead of only mud. This can be of interest when choosing the dredging equipment and in choosing and optimizing the possible alternative.

![Figure 8.25: Possible dredging location for Tam Giang Đông at 15 km from the shoreline. Figures (upper) adapted from Nguyen (2009) and (lower) Unverricht (2013).](image)

Dredging equipment and transport of the dredged material

For this research four different types of dredging equipment have been considered and investigated.

- **Trailing Suction Hopper Dredger:** A TSHD can be used to dredge and transport sediment over larger distances.

- **Cutter Suction Dredger:** A CSD can be used to dredge sediment efficiently from the foreshore.

- **Dredging pump:** A dredging pump can be used to transport smaller volumes locally, but in a continuous mode.
Hydrodynamic dredging: hydrodynamic dredging can be used to remobilise sediments which subsequently can be transported by the local tidal current. This can be accomplished using a variety of dredging equipment and dredging techniques.

A TSHD is not very suitable for nourishing pure mud, as this mixture contains much water, and therefore it is difficult to make use of the hold of the ship in an efficient way. A mixture of mud and sand is possible to dredge and to transport with a TSHD, but it is not as efficient as using pure sand. Therefore, the TSHD is considered as an option for Tam Giang Dông, but not for An Minh at the west coast.

A CSD can have a draught of less than 2 m, which is far less than that of a TSHD, and therefore it is able to dredge closer to shore. A CSD can dredge and pump the dredged material directly into the desired location. The production capacities are higher than those of a TSHD of a similar size. A CSD is able to dredge pure mud and therefore is the appropriate dredging equipment for the west coast.

Instead of using a TSHD or CSD every 5 years, it is also possible to use a mobile dredging pump continuously. This option could consist of a floating platform with a diesel-generator and a pump with a capacity in the order of 100-200 m³/hour. However, this option would be mainly suitable for smaller and local dredging works, such as supplying mud to an area situated behind a breakwater to enhance sedimentation rates to aim for quicker mangrove colonisation. The pumping capacity is far less than that of a CSD and therefore the pumping distance should be small. A pumping platform is less mobile than a TSHD or CSD and hence the production capacity depends on the natural inflow of mud to the location where the platform is installed. The gross sediment transport along the coast is very large, so the use of a mud pit may attract sufficient sediment for further transport. In case this option is used in combination with a (permeable) breakwater, the ideal location would depend on the lay-out of the breakwater as well, since this will influence the sediment transport locally.

A fourth option is hydrodynamic dredging. Hydrodynamic dredging includes all dredging techniques which lead to suspension or re-suspension of the bed material, and uses natural processes, such as tidal currents, to transport the suspended material. This type of dredging requires full understanding of the local hydrodynamics. Dredging techniques classified as hydrodynamic dredging are for example water injection dredging (WID) and agitation dredging. WID uses a low pressure water jet to inject water in the sediment layer in order to suspend sediment and create a density current. Agitation dredging can be performed by two methods: stirring up the sediment from the bed using a powerful water jet or by dragging an instrument along the seabed, and lifting up the sediment using a pumping system (Sigwald, Ledoux, & Spencer, 2015). The second method, using a pumping system, can be executed by a CSD or TSHD. The CSD can pump up the sediment and discharge it directly back into the water column. The TSHD can be used by establishing a continuous overflow of sediment from the ships hold or by discharge of the sediment back into the water column using a pump.

In practice hydrodynamic dredging is mainly used for maintenance dredging operations in ports and channels and to a lesser extent for deepening navigation channels. In this capacity the sediment is stirred up from the bed, during high tidal level, at the shallower water section of the bottom slope, in order to use the falling period and its accompanying offshore directed tidal current to transport the sediment to deeper water, and benefit from gravitational forces. It is much more complicated to apply hydrodynamic dredging as an active sediment management method at the Mekong Delta coast in order to restore the
convex-up profile.

In this section the feasibility of hydrodynamic dredging at the case study site at the east coast will be investigated. First of all, the sediment must be stirred up during low tidal water, in order to be transported in onshore direction during rising tide. This means that when using a small vessel with a draught of 1.5 m and a free board of 0.5 m, the minimal offshore distance is 4.5 km with respect to the position of the shoreline at high tidal level, see Figure 8.26. At $t=0$ the sediment is stirred up from the bed and transported in onshore direction. It is assumed that the sediment particles are suspended over the entire height of the water column of 2.0 m. At $t=x$, the sediment has reached the maximum distance that can be reached due to the tidal current, based on the maximum volume of water that is moved. This implies that the sediment has been transported 2.5 km in onshore direction and settles approximately at an offshore distance of 2.0 km. In theory the sediment has an hour to settle, before the tidal current changes to offshore direction. With a settling velocity between 0.3 and 1.0 mm/s for fine sediments this implies that the sediment would be able to settle at a vertical distance between 1.08 and 3.6 m. This is sufficient for most suspended sediment particles to settle at the bed.

Secondly, in contrast to how hydrodynamic dredging is mainly used in practice, the gravitational force is directed opposite of the tidal current and therefore has a negative impact on the sediment transport in onshore direction. In order to (partially) compensate this opposing force, it is wise to execute the hydrodynamic dredging during offshore directed wind, as this induces a current in the lower part of the water column in onshore direction. Furthermore, wind in offshore direction will reduce the waves near shore, which is beneficial for the sediment to settle. This is applicable to all nourishment techniques in order to reduce loss of sediments during the execution of the nourishment. However, as can been seen in Figure 8.26, the sediment will not reach the upper part of the convex-profile and will settle below MSL. Therefore, a bed level above MSL, required for mangroves to be able to colonise the soil, will not be formed. In addition, when a cliff, in combination with deep water in front of this cliff, is present, the hydrodynamic forces of the waves remain high. Although the sediment may be able to settle closer to the shoreline, because a larger volume of water can be stored near shore, the hydrodynamic forces of the waves will diminish the effects of the hydrodynamic dredging, as more sediment could be stirred up again than is supplied by hydrodynamic dredging. As a consequence sediments gets

Figure 8.26: Schematisation of the processes of hydrodynamic dredging in order to restore the convex-up profile.
transported in offshore direction, totally nullifying the beneficial properties of the hydrodynamic dredging activities. At the west coast it is even harder to accomplish good results with hydrodynamic dredging, due to a smaller tidal range and steeper bed slope.

Hydrodynamic dredging will probably not lead to full restoration of the convex-up profile. However, this method could contribute to other nourishment techniques, for example the ones using a CSD or TSHD, as it can bring sediment from a more offshore location closer to shore. Although the sediment will not be brought all the way up to the shoreline, the costs of the total operation can be reduced because of reduction of the sediment volume nourished by more expensive nourishment techniques.

Because a mobile dredging pump will not suffice in nourishing the needed quantities at both project locations, and the success rate of hydrodynamic dredging is probably low, the following propositions are made with respect to dredging equipment.

A TSHD is proposed to use as dredging equipment for the case study site at the east coast. The TSHD has to dredge the sandy material at a distance of 15 km offshore and then return to a coupling location connected to a pipeline. Due to the use of a coupling location, the dredging scheme is vulnerable to weather conditions, since only during mild wave conditions coupling is possible. This coupling location is considered to be at a distance of 8 km offshore, with an effective draught of 10 m, as is displayed in the lower part of Figure 8.25. However, to get to this location, the TSHD has to make a detour to pass a higher mudbank at a depth of 8 m. In order to limit travelling distance, a smaller TSHD is proposed with a draught of 8 m. This would make it possible to move the coupling location to a distance of 6 km in front of the shoreline, decreasing the needed pumping capacity. A TSHD with a draught of 8 m would be a ship with a hold capacity of 6,000 m$^3$ (van der Schrieck, 2015).

A CSD in combination with a pipeline of 3 km, which consists of two floating flexible pipeline sections, is proposed to use as dredging equipment for the case study site at the west coast. As a CSD can pump material directly into the desired location, no coupling location is needed and the dredging scheme is less dependent on weather conditions. The distance between a dredging location and a nourishment site is preferably not more than 3 to 4 km, as otherwise production would be reduced significantly. In case a CSD is used with a draught less than 4 m, this should not be a problem at the west coast. However, it is not clear from literature and practise what the effect of dredging in the foreshore will be on the coastal system. The dredging location can negatively influence the sediment balance when it is filled by cross-shore sediment transport. In that case, the dredging location should be put further offshore. If the dredging location is filled with alongshore sediment transport, the influence on the sediment balance is minimal. The CSD may take up material with less than 20 percent sand in the order of 100,000 m$^3$/week.

Nourishment techniques

For the east coast at the case study area of Tam Giang Đông, in case a TSHD is used that dredges a sediment mixture with a minimum of 60 percent sand, the following nourishment techniques are possible:

- Creation of a sandy beach, as proposed in Chapter 8.5.1, by putting the sand directly in place using a land pipeline. The big advantage of this nourishment technique is that the erosion stops at once, since the sediment is put directly where it is needed to create the convex-up profile.
• Creation of a sand engine at the shoreline at a location where sediment transport in the right direction is ensured. This will create a shoreline with a high sand content, which will reduce ongoing erosion. This research did not look into the technicalities of this option, but reference conditions can be found at the mouths of the Mekong River.

• Creation of a sand engine in the foreshore in order to stimulate the formation of a sand bank to provide sufficient protection from the waves. This option has not been studied as well. However, according to experts from Soc Trang Province at the east coast, and local fishermen at the west coast, the movement of sand or mud banks are an important feature of the coastline, leading to alternating erosion and accretion events of the shoreline.

For the west coast at the case study site of An Minh a CSD that dredges a sediment mixture which contains less than 20 percent sand is proposed to use. For this case study area the following nourishment techniques are possible:

• Creation of a muddy beach, as proposed in Chapter 8.5.1, by putting the mud directly in place using a land pipeline. The plus of this method is stopping the erosion immediately by creating the convex-up profile at once.

• Creation of a mud engine at the shoreline or foreshore at the location where sediment transport in the right direction is ensured. However, as there is no experience with this method, it is unknown how the mud will behave and how it will be transported along the coast and in cross-shore direction. This research did not look into the technicalities of this option.

8.6 Design alternatives

One of the objectives of this research is to compare nourishment as an alternative for coastal protection. At present breakwaters and Melaleuca fences are build in the Mekong Delta coastal area. Therefore, the nourishment is seen as an alternative for those two measures to prevent erosion and maintain or develop the required minimal width of the mangrove fringe.

The alternatives identified in Chapter 5.2 are as followed:

• Alternative 1: mud nourishment without extra measures
• Alternative 2: mud nourishment in combination with a permeable breakwater
• Alternative 3: mud nourishment in combination with Melaleuca fences
• Alternative 4: sand nourishment without extra measures

This section will compare a nourishment strategy with all its alternatives to the present coastal protection measures, based on cost-effectiveness in erosion prevention and on a cost-benefit comparison. In addition, time-scales are discussed of the execution of the coastal protection measures and expected time to achieve the desired effect.

Cost-effectiveness in erosion prevention is comparing the costs of a breakwater or Melaleuca fence with the costs of a nourishment only for preventing further erosion. For both options, the benefits of erosion prevention in order to safeguard the existing mangrove vegetation, can be assumed to be the same. This is a simplified representation of reality,
since the area between a permeable breakwater and the shoreline may be colonized by mangroves in the future. Similar, the nourishment could lead to less erosion or additional accretion at other locations, and mangrove development as well. This basic comparison is between the costs of building and maintaining a permeable breakwater or Melaleuca fence and a nourishment. This cost comparison is location specific, as the costs of a permeable breakwater and Melaleuca fences depend on water depth and the costs of a nourishment depend on specific needs for the dredging cycle and nourishment volume. The technical lifetime of a permeable breakwater is assumed to be 20 to 25 years. Therefore the comparisons will be done within this time frame.

The cost-benefit comparison covers the costs of erosion prevention and benefits of establishing additional mangrove forest. Permeable breakwaters and Melaleuca fences enable further growth of the mangrove fringe, but in other ways than a nourishment. Therefore, the benefits of the mangrove fringe are important as well. In this research the minimum required width of the mangrove fringe is dependent on heavy storm conditions, a storm with a return period of 100 years. The requirement consists of a maximum wave height of 0.5 m behind the mangrove fringe in front of the dyke. One should note that the relatively large minimum width of the mangrove fringe is in part needed because it is assumed that the foreshore and onshore profile within the mangrove forest will not entirely keep up with sea level rise and land subsidence. Hence, as a consequence, the depth in front of the mangroves might be larger, as is the incoming wave into the mangrove forest. One may argue whether this will occur, and also whether there may be a difference between the situation with a permeable breakwater and a nourishment. For example, it is possible that active nourishment leads to better conditions for keeping up with relative sea level rise. Similarly, it is reasonable to assume that in the case of nourishment, when the foreshore will grow with relative sea level rise, it is not necessary to immediately establish a profile that can withstand heavy storm conditions 50 years from now. Because some costs are de-counted, postponement will lead to lower costs. It should be noted that in the future the foreshore in front of a breakwater may not follow relative sea level rise, or may even be lowered by erosion. This implies that the future breakwaters will be positioned at greater depth and have to withstand larger waves. This will probably increase the costs of a breakwater in the future.

8.6.1 Costs

Permeable breakwaters

Permeable breakwaters in the Mekong Delta exist of concrete piles. There are two variants, namely a double concrete pile breakwater and a single concrete pile breakwater. The double concrete pile breakwaters are filled with rocks between both rows of piles. GIZ did research into the costs of these permeable breakwaters, as it was not clear what the costs of the already existent permeable breakwaters were.

Double concrete pile breakwater  This type has been built in the past years as a first attempt of coastal protection along the east coast. The building costs are in the order of 3,500 US$m/m for the east coast and 2,000 US$m/m for the west coast. There is no hands-on experience with the technical lifetime and maintenance costs of this kind of breakwater. During a field visit in October 2016 some wear and tear of the structure was observed. Local experts assume that the technical lifetime is in the order of 25 years or less. In general the concrete used for the breakwaters is of bad quality, which decreases the technical lifetime significantly. A second aspect of importance is the structural robustness. At the east coast the piles of the structure reach to a depth of 5 m below the sea bed. The
upper part of the structure above the sea bed has a height of 3-3.5 m. In sandy substrates this is expected to be sufficient. However, these concrete piles are located in mud and there is no experience with the morphological development of the sea bed at both sides of the breakwater. At present, it seems some sedimentation takes place at both sides. Nevertheless, experts question if the hard structure leads to concentration of wave energy, leading to erosion of the sea bed on the seaward side. This kind of erosion has not been observed so far, but the breakwaters are only in place for several years now.

**Single concrete pile breakwater** This type is developed from the double concrete pile breakwater. The construction costs are significantly lower, in the order of 1,750 US/m for the east coast and 1,000 US/m for the west coast. An assumption is made that the lifetime of a single concrete pile breakwater will be half of that of the double concrete pile breakwater and hence will be in the order of 12.5 years. This means that the costs over a 25 year period of a single concrete breakwater are still less than of a double concrete pile breakwater, due to depreciation.

**Melaleuca fence**
An alternative to breakwaters is a Melaleuca wave breaking fence. This type of fence is used along the Mekong Delta to ensure a mild wave climate for mangroves to develop. It exists of Melaleuca poles, bamboo mats, fishing nets and stainless steel wire. It is a permeable/open structure. These fences are not able to withstand heavy storm conditions, as their structural robustness is not sufficient. The construction costs are in the order of 50 US/m for the west coast and 80 US/m for the east coast. However, the lifetime of Melaleuca fences is maximum 5 years and hence, after their initial installation, they have to be replaced at least four times during a 25 year period. In addition continuous maintenance is needed.

**Nourishment**
The costs of a nourishment depend on the equipment used, the distance to the dredging location and the volume of the nourishment. The cost figures are a rough indication, based on information provided by Royal HaskoningDHV, as not all factors could be taken into account or estimated exactly. Although in Chapter 8.5.2 a TSHD was proposed as dredging equipment to dredge the 60 percent sand mixture for the east coast, alternatives should be taken into account as well. Therefore an option to dredge mud with a CSD closer to shore at the east coast is listed as well.

<table>
<thead>
<tr>
<th>Case study area</th>
<th>TSHD</th>
<th>CSD</th>
<th>Mobile pump</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tam Giang Đồng</td>
<td>Single location 6 US/m³</td>
<td>2 US/m³</td>
<td>1 US/m³</td>
</tr>
<tr>
<td></td>
<td>Along coastline 8 US/m³</td>
<td></td>
<td></td>
</tr>
<tr>
<td>An Minh</td>
<td>Not relevant</td>
<td>2 US/m³</td>
<td>1 US/m³</td>
</tr>
</tbody>
</table>

Table 8.8: Nourishment costs per location and per m³. The TSHD dredges (>60 % sand) at 15 km offshore, the CSD dredges (mud/clay) at 3 km offshore and the pump dredges (mud/clay) within 200 m offshore.

### 8.6.2 Compounded costs and benefits

**Compounded costs**
In Vietnam a discount rate of 10 percent is used, therefore the compounded costs are determined for 25 years using a 10 percent discount rate.
8.6. Design alternatives

| Compounded nourishment costs at a discount rate of 10 % for Tam Giang Đồng |
|-----------------|-----------------|-----------------|-----------------|-----------------|
| Equipment       | CSD Mud volume 2 | CSD Mud volume 3 | CSD Sand 20% volume 3 | THSD Sand 60% volume 3 |
| Material        | 250             | 250             | 220             | 50             |
| Erosion [m³/m²] | 30,600          | 18,600          | 16,110          | 41,280¹       |
| Restore [US/m]  | 6,050           | 6,050           | 5,325           | 3,630²        |
| Maintain [US/m] | 30,600          | 18,600          | 16,110          | 41,280¹       |
|                 | 6,050           | 6,050           | 5,325           | 3,630²        |

Table 8.9: Compounded nourishment costs of Tam Giang Đồng for 25 years. Costs marked with a 1 are for a nourishment on a single location. Costs marked with a 2 are for a nourishment along the coastline.

| Compounded costs of alternatives at a discount rate of 10 % for Tam Giang Đồng |
|-----------------|-----------------|-----------------|-----------------|
| Equipment       | Breakwater Double concrete pile | Breakwater Single concrete pile | Fence Melaleuca |
| Material        |                 |                 |                 |
| Maintain [US/m] | 4,080           | 2,710           | 195             |

Table 8.10: Overview of the compounded costs alternatives Tam Giang Đồng for 25 years.

| Compounded costs at a discount rate of 10 % for An Minh |
|-----------------|-----------------|-----------------|-----------------|-----------------|
| Equipment       | CSD Mud volume 1 | CSD Mud volume 2 | Breakwater Double concrete pile | Breakwater Single concrete pile |
| Material        | 109             | 109             |                             |                              |
| Erosion [m³/m²] | 17,390          | 6,390           | 2,330           | 1,550           |
| Maintain [US/m] | 2,640           | 2,640           | 2,330           | 1,550           |

Table 8.11: Overview of the compounded costs of An Minh for 25 years.

When looking only at the compounded costs for maintaining the situation as it is at this moment, nourishing costs exceed the costs for permeable breakwaters. Melaleuca fences only work in combination with a nourishment, as they are not capable to withstand heavy storm conditions or in situations that do not require immediate action concerning erosion control and where the mangrove fringe has enough time to rehabilitate. The case study site at the east coast suffers from an erosion rate of 40 m/year (250 m³/m/year). A mud nourishment of only a wear layer (maintaining) already becomes cheaper than a solution with a double pile breakwater for locations with an erosion rate of 24 m/year or less at the east coast. At the west coast this turning point is at an erosion rate of 17 m/year or less.

For Tam Giang Đồng a mud nourishment for nourishment option 3 is cheaper than nourishing the same amount with the mud-sand mixture consisting of 60 percent sand. However, in case such a sand mixture is nourished, the slope of the nourishment can be steeper. Therefore, the nourishment costs are somewhere in between the THSD option for volume 3 and for a slope of 1:80. The most upper part of the profile has to remain at a 1:1000 slope to create the optimal convex-up profile and habitat for mangroves. This means that the costs of nourishing mud or sand are more or less similar, as for mud a bigger volume is needed, where the sand is more expensive per unit of volume because
it has to be transported over a larger distance. However, it should be noted that there are large uncertainties regarding the erosion resistance of the mud and of the sandy-mud mixture that would be nourished.

**Benefits mangroves**

According to information provided by Royal HaskoningDHV, the total benefits of mangroves, consisting of forestry and aquacultural products, is in the order of 5,500 US/ha/year, an average based on Vietnamese data is estimated on 0.55 US/m²/year. In Vietnam it is common to use a discount rate of 10 percent. Whereas, in comparison, in the Netherlands a discount rate of 3 percent is used. With a discount rate of 3 and 10 percent respectively, for a duration of 25 years, this means capitalised benefits of 9.7 US/m² and 5.0 US/m².

Additional benefits are related to reduced costs in coastal protection and due to a reduction in construction costs of dykes, since dykes can be build less high and/or do not need a costly revetment. The total reduction in coastal protection costs is location specific and dependent on the additional mangrove width. Assuming no dyke upgrade by a revetment is needed, in combination with a reduction in maintenance costs, the additional reduction could be in the order of 2,000 US/m. However, the same reduction in costs can be expected when a permeable breakwater is constructed, which maintains the minimal required mangrove width. Therefore, the difference in the extra benefits gained by mangroves is mainly dependent on how the alternatives add to the width of the mangrove forest and how it enables their restoration.

A permeable breakwater is located not too far offshore, so the maximum additional mangrove width would be in the order of 200 m. In case of a nourishment on a slope of 1:1000 with a high tide level of MSL +1.25 m, a mangrove width of 1250 m could be realised theoretically. For a slope of 1:600 and a high tide level of MSL +0.45 m, an additional 270 m of mangroves could be added. If these widths will be reached in practise, depends on several factors such as the hydrodynamic forces, whether the convex-up profile behaves in reality equal to the modelled scenarios and whether this profile is maintained naturally or by periodical nourishment.

**8.6.3 Time-scales design alternatives**

For the time scales of the design alternatives one can distinguish execution time, to build the alternative, from the time it takes the alternative to reach the desired wave attenuation and sedimentation, in order to create favourable conditions for mangrove colonisation to protect the hinterland from flooding.

**Execution**

In Appendix I the weekly production of the proposed CSD and TSHD are determined, based on vessel specifications, productivity, work cycle time, and service hours and downtime. The production of the selected CSD is approximately 118,500 m³/week. Despite the fact that the chosen TSHD has less operational hours a week compared to the CSD, the weekly production exceeds the one of the CSD, as it is in the order of 190,500 m³/week. This is due to the bigger size of the TSHD. The weekly production in combination with nourishment volumes, as established in Chapter 8.5.2, leads to the estimated execution times for different design alternatives, which are listed in Table 8.12.
8.6. Design alternatives

### Desired effect

When comparing a nourishment to restore the convex-up profile to the use of a permeable breakwater, there will be a substantial difference in the time needed to reach the desired effect. After execution of the nourishment this coastal protection measure immediately fully attenuates waves and stops erosion. Furthermore, the bed above MSL is created at once and this generates favourable conditions for mangroves to colonise. In case of building a breakwater waves will be attenuated as well, but perhaps to a lesser extent, especially near shore. Sedimentation can take place, but it will take a considerable amount of time, in the order of years, to create a mangrove habitat. Whether a mangrove habitat of a similar width as for the nourishment can be established, depends on the offshore distance of the breakwater. When a breakwater is constructed too far offshore the wave attenuating effect might be less, as the wave height can increase again after passing the breakwater. If the breakwater is constructed near shore, the mangrove width that can be established will be reduced. Therefore, in order to accomplish a similar mangrove width for the breakwater as for convex-up restoring nourishment, the breakwater needs to be relocated in time, which increases the costs.

The big difference in time-scales of a nourishment and a breakwater as coastal protection measure, lies not within the execution time, but in the time needed to reach the desired effect. Therefore, a Building with Nature coastal protection solution including a nourishment could be favourable for the local people as they can benefit from the effects of mangrove restoration on a smaller time-scale.

### Conclusion design alternatives

A mud nourishment can be an alternative for a breakwater if the erosion rate is not exceeding 24 m/year at the east coast and 17 m/year at the west coast, when looking at maintaining the present situation. In case of restoring the convex-up profile, mangroves can develop and this creates additional benefits. For the east coast this means a maximal benefit out of mangroves of 6,250 US/m, for an additional mangrove width of 1250 m. In case a breakwater is constructed and 200 m of mangroves are developed in front of the existing mangroves, the benefits out of mangroves are 1,000 US/m. This means that at the east coast a maximum additional benefit of 5,250 US/m can be added. For the west coast the benefits out of mangroves are 1,350 US/m, for the calculated additional mangrove fringe width of 270 m. A breakwater will be positioned closer to shore in this situation and therefore the additional benefits out of mangroves would drop to 850 US/m for this area. The point at which full restoration of the convex-up profile can compete economically with a breakwater is very location specific and depends on the foreshore profile and erosion rate. The tipping point at which full restoration of the convex-up profile by means of a mud nourishment is economically viable, taking the additional benefits out of mangroves.

<table>
<thead>
<tr>
<th>Case study area</th>
<th>Profile</th>
<th>CSD Mud [m/week]</th>
<th>TSHD Sand 60% [m/week]</th>
<th>Breakwater Double concrete pile [m/week]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tam Giang Đông</td>
<td>Maintain</td>
<td>95</td>
<td>760</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td>Restore</td>
<td>9-16</td>
<td>29</td>
<td>-</td>
</tr>
<tr>
<td>An Minh</td>
<td>Maintain</td>
<td>217</td>
<td>-</td>
<td>140</td>
</tr>
<tr>
<td></td>
<td>Restore</td>
<td>16-49</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 8.12: Execution time of the design alternatives.
of mangroves into account, is 20 m/year at the east coast and 10 m/year at the west coast.

A permeable breakwater can compete with or contribute to the performance of a convex-up profile if the dimensions are sufficient to attenuate the waves. Alternative 2, a mud nourishment in combination with a permeable breakwater is a good option at locations with high erosion rates. Due to the breakwater, the nourishment volume can be reduced. If the breakwater is build further offshore than is customary at the moment, in combination with applying a nourishment to speed up sedimentation, additional mangroves can develop and hence compensate partly for the extra investment that has to be made compared to only building a breakwater. Another benefit of alternative 2 is that the sediment balance is not negatively influenced by the breakwater, as this is compensated by the nourishment.

Another option is to create a sand/mudbank. Nourishment of the coast at specific sections may lead to a reduction in erosion in a down-current section as well. A sand bank that is periodically nourished may tend to extent with time, sheltering a larger part of the coast. In this sense restoring the wear layer will also lead to the gradual built-up of a sand bank, or in the case of a mud mixture, the original prograding profile. However, this option has not been investigated on economical feasibility, as too little is known about the technicalities.

Tam Giang Đông

A mud nourishment, more specifically option 2 as shown in Figure 8.22, with a CSD costs 30,600 US/m for 25 years, for the case study site at the east coast. Mud nourishment option 2 creates a maximum additional benefit of 5,250 US/m out of mangroves. Hence the additional benefits of the mangroves are not enough to compensate for the much higher construction costs of the nourishment, as the breakwater costs 4,080 US/m. The costs of a breakwater that is able to withstand heavy storm conditions in the future will be in the order of twice the costs of a breakwater as is determined in Chapter 8.6.2. The concrete piles need to be approximately 15 m, due to a water depth of 5.8 m in front of the cliff during heavy storm conditions in the future.

Taking not only economical aspects into account, nourishment option 3, see Figure 8.22, with mud or sand, could be an alternative for a breakwater although it is more expensive. The third mud nourishment described for this site, also by using a CSD, costs 18,600 US/m for 25 years. The additional maximum benefit out of mangroves comprises 5,250 US/m. In case of a sand nourishment for nourishment option 3, the same additional benefit out of mangroves is considered, as the upper part of the convex-up profile will have the same bathymetric shape as the mud nourishment. The minimal costs are 12,840 US/m and the maximum costs are 55,040 US/m, depending on the slope of the sandy foreshore profile. The possible environmental effects of dredging have not been considered in this research. For example, sandy sea beds may contain specific ecological value that need to be taken into account. Furthermore, it is not clear how the use of a sand-mud mixture may influence mangrove ecology in places where these are mainly established in mud. However, there are healthy mangroves near the river mouths that have developed on more sandy-mud mixtures.

A more realistic design for the case study site of Tam Giang Đông is alternative 2, a mud or sandy-mud (with a steeper slope) nourishment in combination with a permeable breakwater, due to the high erosion rate and high cliff in front of the existing mangrove fringe.
An Minh

A mud nourishment option 1, as displayed in Figure 8.24, with a CSD costs 17,390 US/m, for the case study site at the west coast. Mud nourishment option 2 costs 6,390 US/m. The additional benefits due to mangroves comprise 850 US/m. A permeable breakwater costs 2,330 US/m. From these figures it is easy to calculate that restoring the convex-up profile is more expensive than using a breakwater. Taking the costs of a breakwater into account that is able to withstand heavy storm conditions in the future, in the order of twice the costs of the compounded breakwater costs determined in Chapter 8.6.2, nourishment option 2 could be an alternative for a permeable breakwater. However, it is more realistic to implement a nourishment at a location with a lower erosion rate or a smaller cliff in front of the mangrove fringe. Hence, for the case study site of An Minh, alternative 2, a mud nourishment in combination with a permeable breakwater is proposed.
8.6.5 Mekong Delta in general

Figure 8.27 shows possible Building with Nature coastal protection measures for the Mekong Delta. The possible design alternatives for different locations are based on the erosion rate according to Anthony (2015), the status of the existing mangrove fringe, and the position relative to sandy-mud or mud dredging locations. An overview of the parameters leading to different design alternatives is given in Table 8.13.

![Figure 8.27: Overview possible coastal protection measures Mekong Delta.](image)

<table>
<thead>
<tr>
<th>Coast</th>
<th>Erosion rate [m/year]</th>
<th>Existing mangrove fringe</th>
<th>Nearby sediment</th>
<th>Design alternative</th>
</tr>
</thead>
<tbody>
<tr>
<td>East</td>
<td>&lt; 24</td>
<td>sufficient</td>
<td>sandy</td>
<td>sandy wear layer</td>
</tr>
<tr>
<td></td>
<td>≤ 24</td>
<td>sufficient</td>
<td>mud</td>
<td>mud wear layer</td>
</tr>
<tr>
<td></td>
<td>&lt; 20</td>
<td>not sufficient</td>
<td>sandy/mud</td>
<td>full sandy/mud</td>
</tr>
<tr>
<td></td>
<td>&gt; 24</td>
<td>not sufficient</td>
<td>sandy/mud</td>
<td>nourishment</td>
</tr>
<tr>
<td></td>
<td>no erosion</td>
<td>sufficient</td>
<td>sandy/mud</td>
<td>nourishment + breakwater</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>no intervention or monitoring</td>
</tr>
<tr>
<td>West</td>
<td>&lt; 17</td>
<td>sufficient</td>
<td>mud</td>
<td>muddy wear layer</td>
</tr>
<tr>
<td></td>
<td>≤ 17</td>
<td>sufficient</td>
<td>mud</td>
<td>mud pump</td>
</tr>
<tr>
<td></td>
<td>&lt; 10</td>
<td>not sufficient</td>
<td>mud</td>
<td>full mud</td>
</tr>
<tr>
<td></td>
<td>&gt; 17</td>
<td>not sufficient</td>
<td>mud</td>
<td>mud nourishment</td>
</tr>
<tr>
<td></td>
<td>no erosion</td>
<td>sufficient</td>
<td>mud</td>
<td>mud nourishment + breakwater</td>
</tr>
<tr>
<td></td>
<td>no erosion</td>
<td>not sufficient</td>
<td>mud</td>
<td>no intervention or monitoring</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Melaleuca fences</td>
</tr>
</tbody>
</table>

Table 8.13: Overview parameters leading to different design alternatives.
Chapter 9

Discussion

In this research, it was necessary to make several assumptions due to a lack of data. Furthermore, model limitations were encountered. These assumptions and limitations their contributions to the results and the objective of this research are discussed in this chapter. Section 9.1 elaborates on the input, including the project area analysis and the resulting boundary conditions, and the performance of the numerical model SWASH. In section 9.2 the reliability of the model results are discussed and in the final section the design and the design alternatives are discussed.

9.1 Input and numerical model SWASH

9.1.1 Convex-up profile

In this research a convex-up profile was proposed as being the optimal bathymetric profile for a gently sloping mangrove coast, as a convex-up profile is associated with a muddy prograding foreshore (Friedrichs, 2011). This is in line with observations at the Mekong Delta coast. At the southern tip of the Mekong Delta parts of the mangrove coast are not eroding and as expected, the corresponding measured bathymetric profile is convex-up, as can be seen in Figures E.7 and E.8 in Appendix E. The eroding coastal profiles of the case study areas are concave-up and show cliff formation in front of the mangrove fringe. However, the lower convex-up part of the convex-up profile has a steeper slope than the offshore located mud flat, at the southern tip of the Mekong Delta. This was also observed in Trang province in Thailand (Horstman, Dohmen-Janssen, & Hulscher, 2013). This steeper lower convex-up part progresses to a more gradual slope from MSL and becomes nearly horizontal at spring tide level, see Figure 9.1.

![Figure 9.1: Schematised convex-up parts of the convex-up profile.](image)

For this study measured profiles were taken as a starting point. For modelling purposes they have been schematised into a convex-up profile with a continuing gradual slope as well as a steeper lower convex-up part. According to SWASH, both profiles perform
equally well in attenuating waves in front of the mangrove fringe. However, there is an ongoing discussion between experts concerning the steepness of the slope of the upper part of the convex-up profile. It is still unclear whether this part has a steep or a milder slope. For a stable or prograding coast it would be reasonable to assume a milder slope of the upper convex part, because the zone in front of the mangroves and the mangroves between MSL and high spring tide level trap sediment in order to extend in seaward direction. This means that the upper part of the convex-up profile remains at the same level. A coast which is subjected to erosion or has been subjected to erosion, and is now prograding again, will have a steeper slope at the upper convex part.

In case of a nourishment, the construction of a mild convex-up slope for the upper part is proposed, as this attenuates the waves more efficient, which creates favourable conditions for young mangroves to develop. In time, the slope will adapt to nature, as the proposed convex-up profile is a simplified representation of reality. In reality a self-sustaining mangrove fringe and its accompanying foreshore are very complex and dynamic and are dependent on many variables, as was explained in Chapter 4.

9.1.2 Very gentle slopes and shallow water

SWASH was not developed whilst taking the very gentle slopes of the Mekong Delta into account. Quite the opposite is true, as it was particularly developed and validated for steeper sandy foreshores. However, SWASH solves the momentum and mass balance and therefore there is no obvious reason why SWASH would not perform well on very gentle slopes. Unfortunately, during the modelling SWASH showed behaviour that is physically not possible. In shoreward direction the wave height over depth ratio increased rapidly to the point of the horizontal bed level, where the wave height exceeded the water depth. The model areas where the wave height exceeded the water depth, was in very shallow water. This behaviour was analysed by examining the variance density spectrum. The wave spectrum showed a very low frequency peak for \( f \leq 0.01 \) Hz. In nature waves with such low frequencies do not exist. The frequency peak is generated mathematically by the numerical model in order to compute the wave height. It can be seen as a natural vibration at the natural frequency of the model which is not due to applied force. In this model waves with a very low frequency hardly lose energy while travelling from deep to shallow waters. This explains the overestimated and increasing wave heights on very gentle slopes, in combination with shallow water.

The wave spectrum of SWASH has been filtered to exclude the very low frequencies. The energy transfer between all other frequencies was not influenced by doing so. Figure 8.5 in Chapter 8.1 shows the wave height transformation at the convex-up profile for the wave spectrum of SWASH and for the filtered spectrum. The filtered spectrum displays a wave transformation as can be expected along a very gentle slope in shallow water. Therefore, the wave height calculations in this research are based on the filtered spectrum, although the legitimacy of the performed filtering can not be validated. To validate and justify this filtering, data is needed describing near shore wave heights during normal and storm conditions. Unfortunately, near shore wave height measurements in the area of interest during average, annual and heavy storm conditions do not exist.

Although the filtered spectrum showed promising results, it seemed that the wave height over depth ratio on the foreshore remained high in general, according to measurements documented in literature (Friedrichs, 2011). This is explained in Chapter 9.2. A possible reason for the raised wave height over water depth ratio could be that SWASH may not sufficiently account for viscous damping due to the mud on the foreshore.
9.1.3 Water level and bed level

The water level that was used in SWASH to model heavy storm conditions in the future was based on tidal phase, storm surge (wind set-up, wave set-up and barometric effect), relative sea level rise (including absolute sea level rise and land subsidence) and indirectly on sediment supply, as the latter determines the ability of the foreshore profile to keep up with relative sea level rise.

Although land subsidence can differ strongly locally, in this research, due to a lack of data, a relatively high estimated rate of land subsidence of 15 mm/year was used for the case study areas, following Anthony (2015). This assumption can lead to overestimation of the water depth in the future and therefore overestimation of the wave height in front of the mangrove fringe. In the most optimistic case of no land subsidence the water depth for heavy storm conditions reduces with 0.3 m. This has a direct effect on the outcome of the model results, and therefore on the proposed coastal protection measures.

The potential of a coast to adapt to sea level rise, land subsidence and change in sediment supply depends on lateral erosion (shoreline position) and vertical adaptation, see Table 9.1. In this research, lateral erosion estimations were based on a shoreline trend analysis. Vertical adaptation was harder to determine, as there were no bathymetric profiles available for the same location over time. It is not clear if the Mekong Delta coast has a sufficient sediment supply, hence full vertical adaptation can not be assumed. However, the mangroves that are present seem to adapt sufficiently, although no measurements are available, in the vertical direction, perpendicular to the coastline. This could be explained by the gross sediment transport still present in the Mekong Delta, which is supplied by the Mekong River and is, surprisingly, an effect of coastal erosion as well. Due to the horizontal tide this sediment is still able to reach the mangrove fringe, where it settles. The vertical adaptation of the foreshore, especially at the severe eroding zones, will be significantly less compared to the vertical adaptation of the mangrove fringe. Therefore, in this research a vertical adaptation to relative sea level rise of 40 percent was assumed for the entire coastal stretch including the foreshore and mangrove fringe. This could be an overestimation of the vertical adaptation of the foreshore or an underestimation of the vertical adaptation of the mangrove fringe. In case of an overestimation of the vertical adaptation of the foreshore, the wave height in front of the mangrove fringe is underestimated in the long term.

<table>
<thead>
<tr>
<th>Vertical adaptation</th>
<th>Foreshore</th>
<th>Fringe</th>
<th>Shoreline position</th>
</tr>
</thead>
<tbody>
<tr>
<td>Full adaptation</td>
<td>↑</td>
<td>↑</td>
<td>⇒</td>
</tr>
<tr>
<td>Partial adaptation</td>
<td>↓</td>
<td>↑</td>
<td>⇐</td>
</tr>
<tr>
<td>No adaptation</td>
<td>↓</td>
<td>↓</td>
<td>⇐</td>
</tr>
</tbody>
</table>

Table 9.1: Vertical adaptation of the foreshore and mangrove fringe.

9.1.4 Vegetation

To model vegetation in SWASH, mangroves are divided into three different vertical layers: roots, stems and canopy. All three vertical layers are described by four parameters, height, diameter, density and drag coefficient. All input parameters, except for the drag coefficient, were based on measurements of mangroves in India, which are resemblant to the mangrove species in the Mekong Delta. The drag coefficient affects the wave attenuation in the mangrove forest and is unfortunately the hardest to determine. Moreover, no measurements were available and therefore an assumption was made based on literature.

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Chapter 9. Discussion

This assumption may wrongfully change the minimal required mangrove width during heavy storm conditions, and therefore influence the design, which is amongst other variables dependent on the width of the mangrove fringe. Because the model results showed a minimal required mangrove width that is in line with observations made during storm Rammasun, the assumption made with respect to the drag coefficient is considered to be a reliable one.

9.1.5 Sediment module

SWASH is a hydrodynamic numerical model and is therefore especially suitable to model the wave transformation along the proposed convex-up profile. This gives insight on the effect of the cross-shore profile shape of a nourishment on the wave transformation. What is more, the Building with Nature protection measure including a convex-up shaped nourishment was also proposed to reduce the ongoing erosion processes at the shoreline. In order to do so, it was decided to compare the bed shear stress, via the orbital velocity amplitude, of the present situation and the convex-up profile. The convex-up profile showed promising results compared to the original bathymetric profile: the bed shear stress near shore decreased significantly for the convex-up profile. However, to say something solid about erosion and sedimentation processes, sediment sinks and sources need to be located.

SWASH has the option to include a sediment module for cohesive sediment. The sediment flux and therefore the sediment concentration in the water column is based on the Partheniades-Krone formulations. The sediment concentration is very sensitive to variation in sediment parameters. As there are no measurements available for the case study areas, it was hard to validate the model. However, based on measurements of flume experiments, literature and expert judgement, these sediment parameters were determined as accurate as possible. Although the sediment erosion flux decreased near shore for the convex-up profile and was smaller than the sediment erosion flux of the existing profile, the computed net sedimentation and erosion fluxes gave unexpected outcomes. No net erosion was present along the original bathymetric profile or the convex-up profile in cross-shore direction. This is not in line with observations of the present ongoing coastal erosion. Net erosion because of long-shore sediment transport can not be excluded, as only sediment transport in cross-shore direction was modelled. Therefore, the outcome of the investigation to locate sediment sinks and sources is not reliable.

9.2 Model results

9.2.1 Reliability results

The wave height compared to the water depth on the foreshore is in general high when calculated by SWASH. It seems that the model does not sufficiently account for wave attenuation due to the mud and the very gentle slope, see Figure 9.2, when one compares these results with Chapter 2.1.2 and literature. Measurements documented in literature give a maximum wave height over depth ratio varying between 0.15 and 0.5 on muddy slopes (Friedrichs, 2011). SWASH models a maximum wave height over depth ratio at the upper limit of this ratio, or exceeds this ratio.

\[ H_s = 0.260 \cdot d^{1.956} \]  \hspace{2cm} (9.1)

The most conservative algorithm in literature for wave attenuation on a gentle muddy slope is dependent on the wave height and water depth, see Equation 9.1 (Yang, Shi, Bouma, Ysebaert, & Luo, 2012).
Where:

\[ H_s \] significant wave height [m] \\
\[ d \] water depth [m]

In Figure 9.2 the blue lines show the wave height over water depth ratios along the convex-up profile for both case study sites, according to the wave height computed directly by SWASH. The water depth is obtained from a snapshot in time and therefore the wave crests and troughs are visible. The wiggles that are displayed in the blue lines are those waves, as the blue lines are computed by dividing the wave height by the water depth from a snapshot in time. As the still water depth is normally measured from the sea bed up to the mean level of the wave, the mean of the blue line is the representative wave height over depth ratio. The red lines represent the wave height over depth ratio computed out of the filtered spectrum. It is clear that for the shallow water part, the wave height over depth ratio has decreased significantly after filtering the spectrum. A more reliable and realistic wave height over depth ratio has been reached. Nevertheless, the wave height over depth ratios in front of the mangroves (\(x=10\) km and \(x=4.8\) km respectively) are still in the order of 0.65 for Tam Giang Đông and 0.4 for An Minh, during annual storm conditions.

Following the algorithm of Equation 9.1, with a water depth of 1.80 m in front of the mangrove fringe at Tam Giang Đông, the maximum significant wave height should be 0.82 m, instead of the computed 1.15 m (filtered spectrum). The new wave height over depth ratio is 0.46, compared to 0.65 for the filtered spectrum. With a water depth of 1.0 m in front of the mangrove fringe in An Minh, the maximum wave height should be 0.26 m, instead of the computed 0.4 m (filtered spectrum). This leads to a wave height over depth ratio of 0.26, compared to 0.4 for the filtered spectrum. The wave height could therefore still be overestimated by the model after filtering of the spectrum. This overestimation is calculated to be in the order of 30 percent.

Figure 9.2: Wave height over water depth ratio Tam Giang Đông and An Minh along the convex-up profile, during annual storm conditions in the present situation.
Tam Giang Đông

In this research a maximum wave height of 0.5 m was assumed as being the hydrodynamic force young mangroves can withstand, without getting uprooted. This assumption was based on literature, and on the experience and observations of experts. This was translated into a requirement of a maximum wave height of 0.5 m in front of the mangroves during normal conditions. This requirement has been tested in this research during the maximum of the normal conditions: annual storm conditions with a water level corresponding to high tidal level. The maximal frequency of this maximum hydrodynamic force the seedlings can withstand is not clear from literature and therefore the requirement of a maximum wave height of 0.5 m once a year may be quite strict.

The requirement of a maximum wave height of 0.5 m in front of the mangroves has not been fulfilled in Tam Giang Đông for the convex-up profile, as according to the model results the wave height comprises 1.15 m during annual storm conditions. The wave height over depth ratio at this point is 0.65, which is too high for a muddy gentle slope, according to observations documented in literature. Following the most conservative algorithm in literature for wave attenuation on a gentle muddy slope, see Equation 9.1 (Yang et al., 2012), the wave height over depth ratio in front of the mangrove fringe is 0.45. This would decrease the wave height to 0.8 m and hence the requirement is still not met. Field observations indicate that in case of a harsh wave climate, the mangrove fringe starts higher on the profile. If the mangrove fringe starts 0.5 m higher in the profile, the wave height in front of the mangrove fringe decreases to 0.45 m. This implicates that the requirement may be fulfilled. At the east coast this will not influence the mangrove zonation, as all inundation classes and their mangrove species will still be present. Following the algorithm and assuming that the requirement is quite strict, the mangrove fringe will most likely develop somewhere in between MSL and MSL +0.5 m.

In order to generate a Building with Nature alternative for the decision support tool in combination with an earth dyke without a revetment, a maximum wave height of 0.5 m behind the mangroves is allowed during heavy storm conditions. In order for the mangrove fringe to be able to attenuate the waves to this wave height, a minimum mangrove width is required. The minimal required mangrove width, following the model results, is 700 m at the case study site at the east coast. As explained in section 9.1, enough confidence has been gained in the model and its input parameters to accept this as a reliable outcome. An overview of all the outcomes per scenario can be found in Table 9.2.

An Minh

The requirement of a maximum wave height of 0.5 m in front of the mangrove fringe has been fulfilled in An Minh for the convex-up profile, as according to the model results the wave height comprises 0.4 m during annual storm conditions. Following the conservative algorithm this is reduced to 0.26 m. The minimal required mangrove width in order to attenuate the waves sufficiently with respect to the earth dyke without a revetment behind the mangrove fringe, is 350 m according to the model results, and this is accepted as a reliable outcome.

An overview of all the outcomes per scenario can be found in Table 9.2.
9.3. Design

9.3.1 Nourishment

As was explained in Chapter 8.5.2, a nourishment consists of an initial volume and a wear layer. The absolute minimum nourishment volume is the wear layer, because the shoreline...
would be stable and would remain in its present situation. The maximum nourishment volume includes the initial volume based on the proposed convex-up profile and the wear layer. Nourishment options are, among others, dependent on the available sediment mixtures. One may assume that pure mud will erode at a higher rate than coarser mud-sand mixtures. Furthermore, sediment mixtures with a larger volume of sand with a larger $d_{50}$ are capable of forming and maintaining steeper slopes. The steepness of the convex-up profile below MSL does not influence the wave height in front of the mangrove fringe, according to the model results. In case of only nourishing a wear layer, the steepness can be in the same order as the steepness of the eroding and therefore concave-up profile, for example a slope of 1:300, when the slope of the convex-up profile would be in the order of 1:600. This type of nourishing is not meant for creating a habitat for new mangroves to develop, but only to protect the mangrove fringe that is still present. The sediment erosion rate and steepness of the foreshore are of influence on the volume of the nourishment.

The exact contribution of a sand-mud mixture to the decrease in erosion rate of shorelines previously existing solely out of mud is complex to determine. Therefore, in this research a rough estimation based on the van Rijn (2002) formula has been made. An assumption was made that the sandy mixture near the east coast consists of very fine sand in the order of 100-200 $\mu$m. According to van Rijn, for relatively small particle sizes a decrease in long shore transport of 50 percent is valid in case the $d_{50}$ doubles. Furthermore, laboratory and field observations of van Rijn (1993) showed that the pick up rate of sand particles decreases by the presence of mud particles (Van Rijn, 2002). This has led to the used value for the erosion rate of 50 $m^3/m/year$ for a mixture with a high sand content, in stead of 250 $m^3/m/year$ for the fine mud particles with $d_{50}=18$ $\mu$m for the case study site at the east coast. An overestimation of the decrease in erosion rate will lead to quicker erosion of the nourishment. This could be solved by decreasing the interval time between nourishment actions or by increasing the volume of the wear layer. If the volume of the wear layer would be increased the first time the wear layer has to be replaced, the interval rate between nourishment action could be set at 5 years again. In case of underestimating the decrease in erosion rate, the time between two nourishment actions could be increased or the nourishment volume of the wear layer could be decreased after 5 years.

9.3.2 Design alternatives

In Chapter 5.2 four design alternatives were identified. Alternative 1, a mud nourishment without extra measures, has been modelled in SWASH. The performance of the other three alternatives was based on observations and measurements documented in literature. One of the objectives of this research is to investigate how active sediment management can be used to maintain and create a self-sustaining mangrove fringe, as part of a Building with Nature solution, in order to serve as a coastal protection measure. First of all, the design alternative needed to fulfil the requirements as stated in Chapter 5.1. However, the design alternative needed to be an alternative for present coastal protection measures as well. The latter has been determined by performing a cost-effectiveness and cost-benefit analysis. The cost-benefit analysis that was performed was limited and did not take all aspects into account. The costs are estimates, and benefits of coastal protection measures are complex to determine. Hence, only construction and maintenance costs and the benefits of mangroves were taken into account, at a discount rate of 10 percent, for a lifetime of 25 years.

The point at which full restoration of the convex-up profile can compete economically with a breakwater is very location specific and dependent on the foreshore profile and erosion rate. However, the breakwaters that have been built in the past years, will prob-
ably perform less than the convex-up profile, as the breakwaters become submerged due to the wind induced set-up and storm surge during storm conditions, because they are of insufficient height. The convex-up profile is designed for future conditions and might be able to adapt to relative sea level rise as well. In order for breakwaters to compete with a convex-up profile in the future, they should be able to attenuate waves during heavy storm conditions to a comparable wave height as the convex-up profile does. According to Figure 8.13, the breakwater at the west coast needs to attenuate the waves for 50 percent during heavy storm conditions to perform as well as the convex-up profile does.

Jordan (2015) investigated the wave attenuation of the existing breakwaters at the west coast of Vietnam. He performed field measurements and physical model measurements. According to the field measurements with a water depth of 1.5 m, a significant wave height of 0.77 m and a wave period of 5.9 s, the breakwater reduced the wave height by 80 percent. Very small waves in the order of 0.15 m were attenuated for 25 percent. The physical model showed the same correlation between wave height and wave attenuation. However, the physical model showed that the wave period is of importance as well. Waves with a small wave period (T=0-3 s) were attenuated more effective than waves with larger wave periods (T=4-6 s). Therefore, a mean wave attenuation of 50-60 percent is assumed in this research. In order for the permeable breakwater to attenuate the waves, the top of the structure needs to be of sufficient height. Due to relative sea level rise the height of the breakwaters needs to be increased, in order to be able to withstand storm conditions in the future. This increases the construction costs. Another disadvantage of a breakwater is that at the landward side of the breakwater, the water depth does not decrease as it does in case of the convex-up profile. The convex-up profile continues to attenuate the wave because of the steadily decreasing water depth. In case of the permeable breakwater the wave height could increase again, as the water depth is not reduced sufficiently or not reduced at all.

A breakwater prevents erosion and stimulates sedimentation locally, which may diminish the amount of sediment available for locations downstream. So far it is unclear whether the observed erosion is mainly due to a negative sediment balance or due to other factors. In case a breakwater is built along the Mekong Delta, you may expect erosion down-drift to occur, and hence you need to continue building them along the entire coastal stretch. Nourishment may effectively add sediment to the upper foreshore, hence not exacerbating an already negative local sediment balance.

When full restoration of the convex-up profile is economically not feasible because of extremely high erosion rates, a combination of a breakwater with a nourishment could be considered instead of building solely a breakwater, because this has a positive effect on the sediment balance and wave attenuation near shore. In addition, when the hydrodynamic and morphodynamic processes in the coastal area of the Mekong Delta are not understood in detail, because of a lack of data, and it is therefore complex to extend the consequences of these processes in time, the boundary conditions for a coastal protection measure are within a large bandwidth. Therefore, it is not wise to implement expensive non-adaptive coastal protection measures with a long lifetime. Rather, adaptive coastal protection measures should be considered, such as a nourishment, that can withstand current heavy storm conditions and have the possibility to be re-evaluated and adapted when necessary.
Chapter 10

Conclusions and recommendations

This chapter presents the conclusions and recommendations of this research. Section 10.1 answers the research question and sub questions as presented in Chapter 1.3. Section 10.2 is divided into two parts. In the first part recommendations focused on this research are given. The second part presents recommendations for further research on topics discussed in this report.

10.1 Conclusions

The main goal of this research was to determine quantitative knowledge rules for a self-sustaining mangrove fringe as part of the coastal profile, including the foreshore, to serve as a natural coastal protection measure. This was achieved by answering the research question: “In what way can active sediment management be used to maintain and create a natural habitat for a self-sustaining mangrove fringe, as part of a Building with Nature solution for coastal protection in the Mekong Delta in Vietnam?” In order to answer this research question, several sub questions, as formulated in Chapter 1.3, have been answered. In addition to answering the research question, some discoveries with respect to the numerical model SWASH have been made. First, a concise answer to the main research question is given. Thereafter, all the components of the research question are elaborated upon and answered in more detail.

Active sediment management can be used to create a convex-up profile, by means of a mud or sandy-mud nourishment, in order to create and maintain favourable hydrological and morphological conditions for young mangroves to develop, with the aim of establishing and preserving a self-sustaining mangrove fringe. A healthy self-sustaining mangrove fringe and its accompanying convex-up foreshore profile are able to attenuate waves and therefore can contribute to a Building with Nature solution for coastal protection in the Mekong Delta in Vietnam.

10.1.1 Self-sustaining mangrove fringe

Mangrove forests are able to be self-sustaining and can rehabilitate spontaneously after damage when the hydrology and morphology have not been disrupted and the availability of seedlings is sufficient. Flooding depth, duration and frequency have a high impact on the survival and growth of mangrove seedlings and mature trees. Within a self-sustaining mangrove fringe an internal zonation is present (ecology gradient), dependent on inundation classes. In a self-sustaining mangrove ecosystem, the cycles of accretion and erosion are balanced. A few decimeters of vertical erosion can destabilise mangrove trees, which eventually leads to mangrove death, while a healthy mangrove fringe can stabilise the
Chapter 10. Conclusions and recommendations

shoreline position, as it traps sediment. The optimal coastal profile of the foreshore for a self-sustaining mangrove fringe is a convex-up profile, as this has been shown to be related to a prograding profile and net sediment transport in shoreward direction. Waves entering the mangrove fringe will be attenuated while losing energy. The convex-up profile itself attenuates waves as well by viscous damping.

In this research the assumption has been made based on literature and observations of experts, that seedlings can withstand a maximum hydrodynamic force induced by a wave height of 0.5 m, without getting uprooted. The maximum frequency of these hydrodynamic forces seedlings still can withstand is not clear. The requirement is therefore strict, as it states a maximum wave height of 0.5 m in front of the mangrove fringe during annual storm conditions. This means that seedlings can develop almost continuous throughout the year without getting uprooted. For mangroves to serve as a coastal protection measure, a certain width is required in order to attenuate the waves to a wave height that is sufficiently low to function as or contribute to a coastal protection measure. In this research, in order to generate a Building with Nature alternative in combination with a dyke without a revetment, a maximum wave height of 0.5 m behind the mangrove fringe is allowed during storm conditions with a return period of 100 years, to ensure the stability of the dyke. The minimal required mangrove width to attenuate waves is dependent on the density of the mangrove fringe, and in this research it is dependent on the wave height requirement of a dyke without a revetment, and in a more general way, it is dependent on the wave height requirement of the coastal protection measure behind the mangrove fringe. The minimal required mangrove width to be self-sustaining is dependent on the hydrology, morphology and ecology gradient.

10.1.2 Active sediment management

Active sediment management can be used to create a convex-up profile, which can be achieved by a nourishment. A convex-up profile improves the conditions for mangroves to develop, as the hydrodynamic forces in front of the mangrove fringe decrease significantly compared to the existing situation along the eroding concave-up profiles. Furthermore, the convex-up profile decreases the bed shear stress in shore-onward direction compared to the existing situation, which reduces the sediment erosion at the bed. Active sediment management in the form of a nourishment can also be used in combination with other coastal protection measures, such as permeable breakwaters and Melaleuca fences.

Nourishment techniques are dependent on the bathymetric profile of the foreshore, available sediment mixtures, and dredging location of the preferred sediment. These factors influence the choice of dredging equipment and mode of transport of the dredged material. A TSHD can be used to dredge and transport sand and sand-mud mixtures over larger distances. However, it is not very suitable for nourishing pure mud, as this mixture contains large amounts of water, and therefore it is difficult to make use of the hold of the ship in an efficient way. Furthermore, due to the need of a coupling location connected to a pipeline, the dredging scheme is vulnerable to weather conditions. A CSD can be used to efficiently dredge sediment from the foreshore, at a maximum distance of 3-4 km in front of the shoreline. The CSD is able to nourish pure mud and can pump the dredged material directly into the desired location. In stead of using a TSHD or CSD, it is also possible to use a mobile dredging pump in a continuous matter. This would be mainly suitable for small and local dredging works, such as supplying mud to an area behind a breakwater to enhance sedimentation rates in order to aim for quicker mangrove colonisation. After exploring the possibilities of hydrodynamic dredging it can be concluded that this will probably not lead to re-establishment of the convex-up profile. However, this method
10.1. Conclusions

could contribute to the proposed nourishment techniques with a CSD en TSHD, as it can add sediment to the foreshore, but not all the way up to the shoreline, reducing the cost of the total operation because of reduction of the sediment volume nourished by the CSD or TSHD. What is more, nourishment techniques are distinguished by the location of the nourishment in the coastal profile: creation of a beach by putting the sediment directly in place at the shoreline, or creation of a mud/sand engine at the shoreline at a location where sediment transport in the right direction is ensured, creation of a sand/mud engine in the foreshore in order to stimulate formation of a mud/sand bank to provide sufficient protection of the shoreline from the waves. This research looked into the option of creating a beach by nourishing at the shoreline. The big advantage of using this method at this location is the immediate termination of erosion, as the sediment is put directly where it is needed in order to create a convex-up profile.

The volume of a nourishment is dependent on the erosion rate and steepness of the foreshore. One may assume that the very fine mud particles erode at a higher rate than coarser sand-mud mixtures. Moreover, sediment mixtures containing coarser particles are capable of forming and maintaining steeper slopes. It was found that the steepness of the convex-up profile below MSL does not influence the wave height in front of the mangrove fringe. Therefore, a sand-mud nourishment compared to a mud nourishment reduces the required nourishment volume significantly. A sand-mud nourishment is only viable if a sand-mud mixture is available close to the nourishment location. Also, one can distinguish between a minimum and maximum volume for a nourishment. The minimum nourishment volume is the wear layer, which would cause the shoreline to be stable and remain in its present situation. This type of nourishing is not meant for creating a habitat for mangroves to develop, but only to protect the mangrove fringe that is still present. The maximum nourishment volume includes both the volume needed to create the proposed convex-up profile and the volume of the wear layer. This option establishes soil above MSL, for example in front of a cliff, in order to create a habitat for mangroves to develop.

10.1.3 Building with Nature solutions for coastal protection in the Mekong Delta

Tam Giang Đông The modelled convex-up profile for the case study site at the east coast, Tam Giang Đông, fulfilled the requirement with respect to the maximum wave height in front of the mangrove fringe, during annual storm conditions. Furthermore, the requirement was met regarding a maximum wave height of 0.5 m behind the mangrove fringe, in front of the dyke, during heavy storm conditions. The first requirement was fulfilled according to the conservative algorithm presuming that the mangrove fringe starts to develop at MSL +0.5 m, instead of MSL. The minimum required mangrove width comprises 700 m in order to attenuate the waves sufficiently during heavy storm conditions. Following the basic cost-benefit analysis performed in Chapter 8.6.2 a full restoration of the convex-up profile with a mud or sand-mud nourishment with a high sand content of minimal 60 percent, can not compete with the current coastal protection measures, such as a breakwater, because of the extremely high erosion rate of 40 m/year. Therefore, a more realistic design for the case study site of Tam Giang Đông, because of this high erosion rate and high cliff in front of the existing mangrove fringe, is alternative 2, a mud or sandy-mud (with a steeper slope) nourishment in combination with a permeable breakwater. In case of performing a mud nourishment in combination with the construction of a permeable breakwater, a CSD is proposed to be used as dredging equipment. When a sandy-mud nourishment would executed, the use of a TSHD is recommended.
An Minh  The convex-up profile for the case study site at the west coast, An Minh, fulfilled all the requirements with respect to the maximum wave height in front and behind the mangrove fringe, according to SWASH and when applying the conservative algorithm. The mangrove fringe can develop landward from MSL onwards. The minimum required mangrove width comprises 350 m. Similar to the case study site at the east coast, due to the high erosion rate of 20 m/year and high cliff in front of the mangrove fringe, alternative 2, a mud nourishment in combination with a permeable breakwater is proposed as the most suited coastal protection measure. The nourishment part can be executed by a CSD.

Mekong Delta in general  The point at which full restoration of the convex-up profile can compete economically with a breakwater is very location specific and dependent on the foreshore profile and erosion rate. The foreshore profile and erosion rate are coupled, as for higher erosion rates the foreshore becomes more concave-up and hence a larger nourishment volume is needed to restore the convex-up profile. For the east coast, in order to maintain the present situation, an erosion rate of 24 m/year or less was determined as the tipping point for which a nourishment can compete economically with a breakwater. For full restoration of the convex-up profile to be economically favourable the upper limit of the erosion rate is 20 m/year. For the west coast this point comprises an erosion rate of 17 m/year or less for maintaining the present situation and 10 m/year or less for full restoration of the convex-up profile.

It was found that during storm conditions, the breakwaters that have been built the past years, probably will perform less than the convex-up profile, as the breakwater becomes submerged due to insufficient height. The wind set-up and storm surge in combination with relative sea level rise in the future may cause the waves to raise to a level above the extent of the current breakwaters. The convex-up profile is designed for future conditions. In order for breakwaters to compete with a convex-up profile in the future, they should be able to attenuate waves during heavy storm conditions to a comparable wave height as the convex-up profile does. Hence, the top of the structure needs to be of sufficient height, which increases the construction costs. In addition, at the landward side of the breakwater, the water depth does not decrease as it does in case of the convex-up profile. The convex-up profile continues to attenuate the wave all the way up to the shoreline, whereas a breakwater may not. In case of the permeable breakwater the wave height could even increase again after passing the structure, as the water depth is not reduced sufficiently or not reduced at all.

In addition, experts question if the hard structure leads to concentration of wave energy, leading to erosion of the sea bed on the sea side. This has a negative influence on the stability of the breakwater. Furthermore, a breakwater prevents erosion and stimulates sedimentation locally, which may diminish sediment availability at downstream locations. Nourishment may effectively add sediment to the upper foreshore, hence not exacerbating an already negative local sediment balance. Additionally, a convex-up profile may lead to the turning point at which conditions that induce coastal erosion, such as high waves, will be converted into conditions that will lead to a coast in dynamic equilibrium. Another benefit of nourishments can be observed when you compare the time-scales of the nourishment and a breakwater. The big difference lies not within the execution time, but in the time needed to reach the desired effect. In case of a nourishment, the execution time is the same as the time needed to create a mangrove habitat. Meanwhile, for a permeable breakwater, it could take years before a similar effect in establishment of a mangrove habitat is realised. Therefore, a Building with Nature coastal protection solution including a nourishment could be favourable for the local people as they can benefit from the effects.
of mangrove rehabilitation on a smaller time-scale.

From Chapter 8 and Chapter 9 it can be concluded that Building with Nature coastal protection alternatives can compete with the current coastal protection strategies. The suitability of several design alternatives for different locations was based on the erosion rate, the status of the existing mangrove fringe, and the position relative to sandy-mud or mud dredging locations. The possible Building with Nature coastal protection measures for the Mekong Delta are shown in Chapter 8.6.2 in Figure 8.27.

10.1.4 Performance SWASH on very gentle slopes in shallow water

As there were no measurements available to calibrate and validate the numerical model SWASH, the validity of the model has been assessed by comparing the outcomes to expected behaviour based on theory, literature, and expert judgement. SWASH solves the momentum and mass balance and therefore there was no indication why SWASH would not perform well on the very gentle slopes of the Mekong Delta. However, during this research SWASH showed behaviour that is physically not possible. The wave height exceeded the water depth in shallow water on the very gentle slope. This behaviour was analysed by examining the variance density spectrum. The wave spectrum showed a very low frequency peak for \( f \leq 0.01 \) Hz. In nature waves with such low frequencies do not exist. The frequency peak is generated mathematically by the numerical model in order to compute the wave height. In the model the waves with a very low frequency continued in shallow water depths and hardly lost energy. This explains the overestimated and increasing wave heights on a very gentle slope in combination with shallow water. It was decided to filter the wave spectrum and exclude the very low frequencies. Although the filtered spectrum showed promising results, it seemed that according to measurements documented in literature, the wave height over depth ratio on the foreshore remained high in general. This could be due to the fact that SWASH may not account sufficiently for viscous damping due to the mud on the very gentle foreshore.

10.2 Recommendations

There are several recommendations to improve this project, and thereby the DST, in order to give the best possible advice on coastal protection strategies. First of all, it is recommended to set-up a measurement campaign and collect data in order to validate and calibrate the numerical model SWASH. Secondly, it is advised to use a morphodynamic model, for example Delft3D, in order to locate sediment sinks and sources. This can optimise the shape of a nourishment, and may give insight in the tipping point when a bathymetric profile in combination with the occurring hydrodynamic forces will lead to net sedimentation in stead of net erosion. A last recommendation regarding this specific project is to improve the cost-benefit analysis. The cost-benefit analysis that was performed was limited, as the costs were estimates and the benefits of coastal protection measures are not straightforward and are complex to determine. A more detailed cost-benefit analysis would contribute to the knowledge with respect to the conditions when a nourishment becomes economically viable as a coastal protection strategy.

In addition to the project specific recommendations, there are recommendations for further research on a more general level for topics discussed in this report. The first recommendation is similar to the first project specific recommendation, as measurement campaigns and data collection of mangrove coasts are of utmost importance in order to protect these coastal areas. Furthermore, it is recommended to improve and validate the
numerical model SWASH for gentle slopes in shallow water, several research objectives are listed in Appendix G. The measurement campaign can contribute to this proposition. The third recommendation is to investigate how and to what extent a mangrove fringe and its foreshore keep up with relative sea level rise in the Mekong Delta. The potential of a coast to adapt the relative sea level rise influences the water level in front of the mangrove fringe and dyke and therefore has a large impact on the wave height in these two important areas. This has a direct effect on the proposed coastal protection measure. Finally, a recommendation is given to study the quantitative influence of adding coarser sediment particles (sand) to a mud mixture in order to decrease the erosion rate. This will contribute to more accurate estimates of the nourishment volumes when a Building with Nature solution is considered.
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Appendix A

Measurement campaign GIZ

In 2013 two measurement campaigns were carried out by the Deutsche Gesellschaft für Internationale Zusammenarbeit (Albers & Stolzenwald, 2014) in cooperation with the Southern Institute of Water Resources Research, to map the coastal area of Cà Mau Province. One measurement campaign was carried out in March/April 2013 and the other one in September/October 2013, to cover respectively the North-East Monsoon (dry season) and the South-West Monsoon (rainy season). Cà Mau Province is situated at the most southern tip of the Mekong Delta. Three sites were explored in the investigation area: Đầm Dơi District at the east coast, Ngọc Hiển District at the southern tip and U Minh District at the west coast. Their locations are shown in Figure 3.6. The three areas of investigation are all covering 23 km of coastline and are indicated by a red dotted line in Figure 3.6. The bathymetric measurement campaigns in the three areas of interest all consist of three sample sites. These sample sites were chosen in such a way that it should have been possible to collect data about bathymetry, water levels, sediment, currents, tidal regime and wave field, and to understand, with the help of the collected information, all the occurring processes at the shoreline of the area of investigation (Albers & Stolzenwald, 2014).

To collect data about current velocity and direction, wave height, wave period, and direction and water level, stationary measurements were conducted, using a bottom-mounted Acoustic Doppler Current Profiler (ADCP) Workhorse Sentinel for echo sounding from Teledyne RD instruments. The ADCP was connected to a metal frame and mounted on the sea-bed with the sensor looking upwards. The echo sounder requires a minimum water level of 1.0 m above the sensor, since otherwise the energy of the emitted signal lingers on while dubbing the real echoes. Furthermore, a minimal water depth beneath the ADCP is necessary to measure the real echo. These effects are called respectively ringing and blanking, and are of significant importance in shallow waters. In order to reduce the noise in the measured date, the settings of the ADCP have to be determined carefully (Albers & Stolzenwald, 2014). For the stationary measurements a minimum water depth of 2.0 m above the ADCP during low water was chosen as a boundary condition. This required a minimum distance of the site of measurements of 2.0 km perpendicular to the shoreline. During the first measurement campaign in March/April 2013, difficult weather conditions prevented installation of the ADCP at the at that time required distance of 2.0 km offshore. In order to compare the data of the two measurement campaigns, it was aimed to install the ADCP during the measurement campaign in September/October 2013 at the same location as they used during the first measurement campaign. During the second campaign, weather conditions influenced the stationary measurements again (Albers & Stolzenwald, 2014).
A.1 Đầm Dơi

The area of investigation in Đầm Dơi encloses its entire coastline starting from the river mouth at Gành Hào, part of Bạc Liêu Province, up to the southern border with Năm Căn District. The ADCP for the stationary measurements was located at N 8.915 °, E 105.389 °, both between March 19th and 23rd 2013 and during the second measurement campaign between September 24th and 27th 2013 (Albers & Stolzenwald, 2014). Both measurements were carried out close to neap tide, see Figures A.1 and A.2.

Figure A.1: Results of the measurements in Đầm Dơi in March 2013 (Albers & Stolzenwald, 2014).
A.2 Ngọc Hiển

The low lying area Ngọc Hiển District is located at the southern tip of Cà Mau Province. The ADCP for the stationary measurements was located at N 8.556°, E 104.793°, between March 24th and 28th 2013. The coordinates of the position of the ADCP between October 15th and 17th are unknown. During the first measurements its time frame, the nearest mandatory minimum water depth of 2.0 m at low water was present at a distance of 4.0 km offshore, instead of the desired 2.0 km offshore. Also, to protect the ADCP from theft, a vessel needed to stay at its location. However, at a distance of 4.0 km offshore, the waves were too high for the vessel and it was decided to mount the ADCP closer to shore. The minimum water depth of 2.0 m could not be guaranteed. At low water a water depth of less than 1.0 m above the ADCP occurred. Consequently the ADCP had only one depth cell, resulting in just one measured value and no depth averaging. It has to be noted that these measurements are more prone to errors (Albers & Stolzenwald, 2014). The second measurement campaign was originally planned between October 2nd and 6th. Due to high waves the measurements were stopped multiple times, resulting in changes of measurement positions. At the 4th of October a storm event stopped the measurement campaign entirely and it was postponed to October 15th until 17th. Although the weather conditions were good, the coordinates of the location of the ADCP were not documented. Despite the reliable data, because the minimum water level above the ADCP was sufficient, the measurements in the Ngọc Hiển District are hard to compare to each other due to the unknown location of the last measurement campaign (Albers & Stolzenwald, 2014).
The measurements carried out in March were close to spring tide, see Figure A.3, whereas the ones in October were close to neap tide, see Figure A.4.

Figure A.3: Results of the measurements in Ngoc Hiên in March 2013 (Albers & Stolzenwald, 2014).
Figure A.4: Results of the measurements in Ngọc Hiền in October 2013 (Albers & Stolzenwald, 2014).

A.3 U Minh

U Minh District is situated at the west coast of Cà Mau Province. Between March 30th and April 3rd 2013 the ADCP for the stationary measurement was located at N 9.346°, E 104.806°. The position of the ADCP between October 8th and 12th was further offshore. Although no difficult weather conditions were encountered during the first measurements,
the required water level of 2.0 m above the ADCP was not guaranteed. Because of sur-
passing the lower limit of the water level, the second survey has been performed further off-
shore. The coastal section was unchanged regarding to the first survey, hence the data of both measurement campaigns were still comparable. Further offshore a minimum water level of 2.2 m above the ADCP during low water was reached, which improved the data quality (Albers & Stolzenwald, 2014).

Both measurements in March and April 2013 as in October 2013 were conducted close to spring tide in the tidal cycle, see Figures A.5 and A.6.

Figure A.5: Results of the measurements in U Minh in March 2013 (Albers & Stolzenwald, 2014).
Figure A.6: Results of the measurements in U Minh in October 2013 (Albers & Stolzenwald, 2014).
Appendix B

Tides

In the following sections an overview of the outcomes of the tidal measurements per measurement location will be presented. In addition, there will be elaborated upon the effects of the weather conditions on the measured data.

B.1 Đầm Dơi

The ADCP measures the water level relative to its location, while the forecasts of the tidal gauges are expressed in water levels relative to the mean sea level. No difficult weather conditions occurred in the area of investigation during the time of the measurements, that could have influenced the obtained results.

In Figure B.1 the results of the first measurement campaign are displayed, which were conducted in the period close to neap tide. These results, in combination with the current velocity and current direction of Appendix A, show that slack water time, when the flow reversal is from high water to the lower low water and contrariwise, occurs at half tide level. This indicates that while the water is falling, the flood current is still present until half tide level has been reached. Likewise, during rising water, the ebb current is the current up to half tide level (Albers & Stolzenwald, 2014).

Figure B.1: Measurements in Đầm Dơi March 2013, water level above ADCP (Albers & Stolzenwald, 2014).

Figure B.2 shows the results of the second measurement campaign. This survey was carried out close to neap tide in the tidal cycle. Equally to the measurements in March 2013 the slack water during flow reversal from lower low water to high water occurs at half tide level. Nevertheless, when the water level is shifting from intermediate low water levels to high water, slack water time occurs closer to the extreme values of the water level. This means that the position of the slack water time is dependent on the tidal range. If the tidal range decreases towards neap tide, the position shifts from half tide level towards extreme water levels (Albers & Stolzenwald, 2014).
Appendix B: Tides

Figure B.2: Measurements in Đầm Dơi September 2013, water level above ADCP (Albers & Stolzenwald, 2014).

B.2 Ngọc Hiển

Ngọc Hiển is influenced by the tidal regime of the East Sea as well as by the one of the Gulf of Thailand. The tidal regime of the East Sea is predominant. Nevertheless, the influence of the Gulf of Thailand is the reason for the difference in the tidal amplitudes of Đầm Dơi and Ngọc Hiển, respectively 2.5 and 1.0 m. The tidal conditions result in a slack water time occurring around half tide level, in the order of three hours after reversal from high to low water or vice versa. Consequently, during the first three hours of rising water, the current is in ebb direction. Equally, in the first three hours of falling water, the current is in flood direction (Albers & Stolzenwald, 2014).

Figure B.3 presents the measurements in Ngọc Hiển in March 2013 in the period close to spring tide. As aforementioned, during low tide, the ADCP was exposed. This had consequences for the velocity measurements, which could not be recorded during that period of the tidal cycle. Figure B.3, in combination with the current velocity and current direction described in Appendix A, indicates that with limited difference in water level between high water and low water, the current direction does not change and remains in flood direction.

Figure B.3: Measurements in Ngọc Hiển March 2013, water level above ADCP. (Albers & Stolzenwald, 2014).

As can be seen in Figure B.4, only two tidal cycles were recorded during the measurements in October 2013. The measurements were carried out in the period close to neap tide. Unfortunately, the location of these measurements is not known and therefore the data is not comparable to the data of March 2013 (Albers & Stolzenwald, 2014). However, the data can be used to explore the conditions during the rainy South-West monsoon. The flood tide is in north-north-west direction, while the ebb tide is in eastern direction. If the change in water level is minimal between two high waters and the intermediate low water, the current stays in ebb direction.
Appendix B: Tides

B.3 U Minh

Figure B.5 depicts the water level measurements in March and April 2013, close to spring tide in the tidal cycle. When this data is combined with the current velocities, a flood dominant current is determined. The velocities during ebb tide are very low and the magnitude of the flood current velocities are approximately three times higher.

Figure B.6 shows the outcome of the survey performed during the rainy South-West monsoon in October 2013, which was carried out in the period close to spring tide. These results in combination with the current velocity and current direction in Appendix A, show an ebb tidal current in southern direction and a flood tidal current in northern direction. However, during the South-West monsoon and its strong winds, the current remains in ebb direction during the complete tidal cycle. This indicates that the southern directed current is dominant at the west coast of the Cà Mau peninsula ((Albers & Stolzenwald, 2014).

Figure B.4: Measurements in Ngoc Hiền September 2013, water level above ADCP (Albers & Stolzenwald, 2014).

Figure B.5: Measurements in U Minh March 2013, water level above ADCP (Albers & Stolzenwald, 2014).

Figure B.6: Measurements in U Minh October 2013, water level above ADCP (Albers & Stolzenwald, 2014).
Appendix C

Wave characteristics

C.1 Bach Ho wave station

<table>
<thead>
<tr>
<th>Return period [year]</th>
<th>Wave height [m]</th>
<th>Wave period [s]</th>
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<tr>
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<tr>
<td>100</td>
<td>7.2</td>
<td>9.7</td>
</tr>
</tbody>
</table>

Table C.1: Wave height and wave period for different return periods measured at Bach Ho wave station (Huan & Nhan, 2006) for the east coast.

Two logarithmic functions are defined to extrapolate for the wave height and the wave period for Bach Ho wave station:

\[ H = 0.7972 \ln(T_r) + 3.1818 \] \hspace{1cm} (C.1)

\[ T = 0.3583 \ln(T_r) + 8.0340 \] \hspace{1cm} (C.2)

Where:

- \( H \) wave height [m]
- \( T \) wave period [s]
- \( T_r \) return period [year]
Appendix C: Wave characteristics

C.2 WAM

The European Centre for Medium range Weather Forecasts (ECMWF) uses and develops the WAve Model WAM. It is coupled to the atmospheric model, but it is also possible to run it in standalone mode. The WAM resolves 30 wave frequencies and 24 wave directions per node of the 1.0°×1.0° latitude/longitude grid. Two grid points are representative for the case study areas and provide data about wave height and period respective to different return periods.

<table>
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<th>Return period [year]</th>
<th>Wave height [m]</th>
<th>Wave period [s]</th>
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</tr>
<tr>
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<td>6.3</td>
</tr>
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</tr>
<tr>
<td>500</td>
<td>6.2</td>
<td>6.5</td>
</tr>
</tbody>
</table>

Table C.2: Estimated wave height and wave period for An Minh by WAM.

<table>
<thead>
<tr>
<th>Return period [year]</th>
<th>Wave height [m]</th>
<th>Wave period [s]</th>
</tr>
</thead>
<tbody>
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</tr>
<tr>
<td>500</td>
<td>3.0</td>
<td>9.6</td>
</tr>
</tbody>
</table>

Table C.3: Estimated wave height and wave period for The commune of Tam Giang Đông by WAM.
Appendix C: Wave characteristics

C.3 Offshore wave characteristics

![Graphs showing offshore wave characteristics for the east coast.](image)

**Figure C.1:** Offshore wave characteristics for the east coast.
Appendix D

Water level set-up

Tam Giang Đông  As can be seen in Figure D.1 the bathymetry is divided into segments with the starting point at an offshore location at a depth of 40 m. The bathymetric profile shows a mudflat at a depth of 3.25 m beneath MSL from 300 m until 2500 m offshore. At the near shore part of this mudflat from 300 m to 120 m in front of the shoreline the slope steepens, resulting in a depth of 2.25 m beneath MSL at 120 m offshore. From that point a segment of 120 m width is computed for 1.0 m depth up to mean sea level. The offshore part from 2500 meter offshore at a depth of 3.25 m until the assumed 40 m depth has a slope of 1:1000, which gives the offshore starting point at approximately 40 km offshore. The first 5 segments computed from the offshore end from 40 km up to 2.5 km offshore have a width of 7.5 km. Per segment the water depth decreases with 7.5 m (set-up not included). The last three segments have a width of respectively 2200, 180 and 120 m. With water depths of 3.25, 2.25 and 1.0 m respectively. In Table 3.11 the results of the calculated water set-up due to wind friction per return period is displayed.

![Water level set-up due to wind friction](image)

**Figure D.1:** Water level set-up due to wind friction, calculated per segment of the simplified bathymetric profile of Tam Giang Đông.

An Minh  The segments to calculate the set-up due to wind friction for An Minh is showed in Figure D.2. At the offshore end at a depth of 40 m to a depth of 2.4 m a slope of 1:600 is used. This means that the offshore location is at a distance of 22.8 km in front
of the shoreline. The first 22.5 km is divided into 5 segments, each with a width of 4.5 km. Per segment the water depth decreases with 7.5 m (set-up not included). The last two segments are at a depth of 2.4 m and 0.5 m with a width of 280 m and 20 m respectively. This is according to the bathymetric profile of the case study location.

**Figure D.2:** Water level set-up due to wind friction, calculated per segment of the simplified bathymetric profile of An Minh.
Appendix E

Bathymetry

During the measurement campaigns by the Deutsche Gesellschaft für Internationale Zusammenarbeit (Albers & Stolzenwald, 2014) in cooperation with the Southern Institute of Water Resources Research in the coastal area of Cà Mau Province in 2013, the bathymetry was mapped as well. The bathymetric measurement campaign was carried out by using an Acoustic Doppler Current Profiler (ADCP) Workhorse Sentinel from Teledyne RD instruments for echo sounding to perform mobile measurements. The ADCP was mounted alongside a vessel that navigated at transects perpendicular to the shoreline with a minimum length of 2 km. Near the centre of the measurement area the transects had a spacing of 500 m, whereas the following five profiles had a spacing of 1000 meter and the last three a spacing of 2000 m. The difference in spacing was chosen to ensure that the centre survey is representative for the entire area of interest. Per area of investigation a total of 18 transects were surveyed in duplo; once during ebb tide and once during flood tide (Albers & Stolzenwald, 2014).

E.1 Đầm Dơi

The area of investigation in Đầm Dơi stretches the entire coastline from the river mouth at Gành Hào, part of Bạc Liêu Province, up to the southern border with Năm Căn District. The sediment distribution in this stretch is sand dominated in the northern part, while shifting to a sand-mud dominated sediment distribution in the middle and southern part of the investigated area. For the greater part this coastline suffers from severe erosion. There is no beach profile present and the mangroves are directly exposed to waves and the tidal regime. In this section of the Mekong Delta coastline there are hardly any man-made coastal protection measures and the hinterland is entirely dependent on the mangrove forest for protection against flooding. There are some small earth dykes situated in the mangrove forest but their main purpose is to form boundaries for aquacultural ponds. Furthermore, the mangroves are the sole protection against erosion. The main part of the coastline of Đầm Dơi consists of dead standing mangroves. This stretch of the coastline has a width of approximately 3-5 m and recently fallen and decaying mangroves are observed here. The soil on the shore consists mainly of silts with components of clay. The height of the shoreline at the erosion edge is in the range of 1 to 1.5 m. Moving in seaward direction, steps in the bathymetry of 0.5 m are observed. These indicate former shoreline positions (Albers & Stolzenwald, 2014).

Due to the draught of the survey vessel, the bathymetric profile measurements were limited by a minimal water depth of 1.5 to 2 m. For Đầm Dơi this means that the shoreline could not be approached within a distance of 120 m, due to site specific and tidal conditions in combination with the draught of the vessel. From the data of the surveys...
three bathymetric profiles were constructed. These profiles describe the bathymetry at transect 8 in the north of Đầm Đỗ, transect 11 in the middle of Đầm Đỗ and transect 17 in the south of Đầm Đỗ, see Figure E.1.

![Sample area 1: Đầm Đỗ](image)

**Figure E.1:** Position transects Đầm Đỗ (Albers & Stolzenwald, 2014).

The bathymetric profiles are displayed in Figure E.2, Figure E.3 and Figure E.4. From the figures it shows that the profiles are starting at an approximate distance of 120 m of the shoreline. The coastal area of Đầm Đỗ District is characterized by a plateau with an almost constant depth in front of the shoreline. With a red line the former shoreline in 2002 or 2000 is indicated, illustrating severe erosion over time in these areas. The mean erosion rate in this area is 22 m/year, but this is heavily exceeded especially in the middle of the investigated area at transect 11. Here, in the last decade, the shoreline has retreated over 630 m. This profile also shows less constant depth in front of the shoreline compared to those in the north and south (Albers & Stolzenwald, 2014).

![Figure E.2: Bathymetric profile transect 8 Đầm Đỗ](image)

**Figure E.2:** Bathymetric profile transect 8 Đầm Đỗ (Albers & Stolzenwald, 2014).
E.2 Ngọc Hiển

The low lying area Ngọc Hiển District is located at the southern tip of Cà Mau Province. Ngọc Hiển is an island formed by the sediment transported by the Mekong River. It is separated from the mainland by the Lớn River and has an average elevation above MSL of 1 m and is therefore extremely vulnerable to flooding. Due to its origin the soil consist of very fine particles. The sediment distribution is mainly mud-dominated, changing to a mud-sand dominated distribution at the eastern boundary of the investigated area. The coastline suffers from mangrove degradation caused by extensive chopping of mangrove forests to create space for shrimp ponds. Similar to Dăm Dơi District, there are no man-made coastal protection measures, except for a jetty at the tip of the peninsula to protect the shoreline of Mũi Cà Mau National Park from wave energy. This means that the mangrove forest is the only coastal protection measure against flooding and erosion. The soil on the shore consists mainly of silt and clay, while the seabed consists of sandy silt. There are a few locations at the shoreline where small sandy beaches are observed. The area of investigation can be divided into two different sites: the stretch from the southern point up to the west and the stretch from the southern point going to the east. They differ in bathymetry of the foreshore, which could be caused by the difference in sediment distribution. The western part is characterized by a very shallow foreshore containing sand mega ripples with a width of approximately 2 km in cross-shore direction. This stretch has a water depth in the order of 1 to 1.5 m during high tide. The foreshore of the part of the coast starting from the southern tip going in eastern direction is less shallow than the western part. This area is prone to erosion, whereas the other area hardly suffers from erosion and even accumulates sediment, leading to accretion at some locations. In general it can be said that the coastline of Ngọc Hiển is on average stable (Albers & Stolzenwald, 2014).
Due to the draught of the survey vessel, the bathymetric profile measurements were, as mentioned before, limited by a minimal depth of 1.5 to 2 m. For Ngọc Hiển this means that the shoreline could be approached to a distance of 30 m at the eastern part. However, at the middle and western section the vessel could not get any closer to the shoreline than 2.3 km respectively 1.8 km as a result of the very shallow foreshore. From the data of the surveys three bathymetric profiles were constructed. These profiles are located at transect 9 in the east of Ngọc Hiển, transect 10 in the middle of Ngọc Hiển and transect 17 in the west of Ngọc Hiển, see Figure E.5.

![Figure E.5: Position transects Ngọc Hiển (Albers & Stolzenwald, 2014).](image)

In Figure E.6 the bathymetric profile of the eastern part of Ngọc Hiển is shown, followed by Figure E.7 presenting the bathymetry of the middle part and in Figure E.8 the bathymetric profile of the western site is displayed. The bathymetry of the eastern study site is characterised by a very gradual slope up till a distance of 600 m from the shoreline. From this point towards the shore a much steeper gradient is observed, with a step height of 2 m. At this location gaps in the mangrove forests are observed. In the middle of the investigation area of Ngọc Hiển an extremely mild gradient is detected offshore up to the point of a bed level step of 2 m within a 500 m strip, beginning at a distance of approximately 2.8 km in front of the shoreline. At this point the shallow area with a width of 2.3 km starts. At the western section the offshore gradient is steeper, being in the range of 1:800. However, starting at 2.6 km offshore going shoreward up to 1.8 km offshore, a mega ripple is detected. This is the starting point of the shallow area continuing til the shoreline. This extremely shallow area functions as a natural coastal protection measure by attenuating waves through shallow water effects (Albers & Stolzenwald, 2014).
E.3 U Minh

U Minh District is situated at the west coast of Cà Mau Province. Its coastline is relatively straight. The area of investigation consists of 20 km coastline of U Minh District and a minor stretch of 3 km of Trần Văn Thời District, located south of U Minh District. The sediment distribution of the west coast of Cà Mau Province is mud dominated. A narrow mangrove belt is present at the shoreline, in front of a small earth dyke. The majority of the coastline is subject to mangrove degradation and at multiple locations the mangroves have disappeared entirely, hence exposing the earth dyke directly to the sea. The hinterland at these locations is protected only by the earth dyke. In the northern part of the area of investigation, where the mangroves are non-existent at present, different coastal protection measures have been built to prevent further erosion of the earth dyke. The soil in this area consists of silt and larger clay particles and is therefore muddy (Albers &
The bathymetry in the area of investigation is constant over the entire stretch. The gradient of the slope is constant in cross-shore direction and the bed level decreases with increasing distance to the coastline. Analysis of the shoreline position in time suggests a mean erosion rate of 10 m/year. In certain areas this rate is exceeded, for example near sluice gates. These weak spots are attacked by currents, which can lead to erosion of the nearby headlands (Albers & Stolzenwald, 2014).

Due to the draught of the survey vessel, the bathymetric profile measurements were, as mentioned before, limited to a minimal water depth of 1.5 to 2 m. For U Minh this means that the shoreline could be approached up to a distance of 300 m. From the data of the surveys, due to the similarity of the bathymetry profiles over the entire stretch, only two bathymetric profiles were composed. These profiles are located at transect 9 in the north of U Minh and transect 18 in the south of U Minh, see Figure E.9.

Figure E.9: Position transects U Minh (Albers & Stolzenwald, 2014).

In Figure E.10 and Figure E.11 the bathymetric profiles of the northern and southern part of U Minh District are displayed. The profiles are very similar and their slope is constant. Notable is the gradient of the slope. It is relatively high compared to other parts of the coast, and even more so taking into account the small sediment particles of the muddy environment.

Figure E.10: Bathymetric profile transect 9 U Minh (Albers & Stolzenwald, 2014).
Figure E.11: Bathymetric profile transect 18 U Minh (Albers & Stolzenwald, 2014).
Appendix F

Shoreline trend analysis

Figure F.1: Shoreline position 1904-2015 (Sorgenfrei, 2015).
Figure F.2: Shoreline position 1904-2015 Tam Giang Đồng (Sorgenfrei, 2015).

Figure F.3: Shoreline position 1965-2015 Tam Giang Đồng (Sorgenfrei, 2015).
Figure F.4: Total area change 1904-2014 Tam Giang Dong (Sorgenfrei, 2015).
Figure F.5: Shoreline position 1965-2015 An Minh (Sorgenfrei, 2015).
Appendix F: Shoreline trend analysis

Figure F.6: Shoreline position 2001-2015 An Minh (Sorgenfrei, 2015).
Figure F.7: Total area change 1965-2014 An Minh (Sorgenfrei, 2015).
Figure F.8: Shoreline position 1973-2014 Tam Giang Đỗng (Besset et al., 2016).
Figure F.9: Shoreline position 2003-2012 (Anthony et al., 2015).
Appendix G

SWASH

G.1 Set-up

G.1.1 Grid size and run time

In order to choose the grid size that will improve the accurateness of the model, the grid size was varied for the convex-up profile at low wave conditions with $H_s=0.5$ m and without the mangrove vegetation. In Figure G.1 the black line shows the wave transformation when the grid size cell is 1.0 m and simulates the expected behaviour best as on very gentle slopes it is expected that the shoaling behaviour continues longer until the wave steepness at the point breaking is reached. When the grid size cell is chosen too small, the model shows unstable results (wiggles) when higher wave heights are modelled. Therefore in this research a grid size cell of 1.0 m is chosen.

A storm duration of 6 hours is assumed in this research. To reduce the simulation time, first a run time of 2 hours was proposed. Due to the results shown in Figure G.2, it was expected that the waves could not reach the same point in the domain as waves during a storm of 6 hours. Therefore the run time was increased to 6 hours (spin-up time not included), displayed by the red line. The red line shows that the sponge layer at $x=20$ km is reached by a sudden drop in height and hence the domain was extended to a length of 30 km. Due to the nonlinearities present at the offshore boundary, the wave height is enhanced by a factor to account for the sudden drop in wave height. This factor is introduced in the run displayed by the yellow line.

![Comparing different sizes of grid cells](image)

**Figure G.1**: Comparison of the variation in the size of grid cells on the wave transformation on a very gentle slope (1:1000) with a convex-up shape, starting at a depth of MSL -10 m.
G.2 Model performance

During this research the numerical model SWASH showed questionable behaviour, which could not be explained by physical processes. This behaviour and the probable causes are elaborated upon in Chapter 8 and Chapter 9. A summary of these findings is given in this chapter. In addition, research objectives are proposed for further research.

G.2.1 Very gentle slopes and shallow water

Wave transformation According to the model results when modelling the wave transformation on very gentle slopes in combination with shallow water, the wave height over depth ratio increases rapidly in shoreward direction to the point of the nearly horizontal bed level, where the wave height exceeds the water depth. This is physically not possible. In order to investigate this behaviour, the variance density spectrum was examined. The wave spectrum showed a very low frequency peak for $f \leq 0.01$ Hz. In nature waves with such low frequencies do not exist. It is expected that the low frequency peak is generated mathematically by the numerical model in order to compute the wave height at the very gentle slope. It can be seen as the natural vibration of the model which is not due to applied force.

Wave height over depth ratio filtered spectrum After filtering the spectrum by excluding the frequencies $f \leq 0.01$ Hz, the wave height over depth ratio near shore remained high according to measurements documented in literature, which were performed on muddy gentle slopes. It seems that SWASH does not sufficiently account for viscous damping due to the muddy foreshore.

Bed shear stress In this research bed shear stress is computed with the orbital velocity amplitude, which is based on the velocity output parameter of SWASH near the bed. It is not clear whether bed shear stress can be simulated accurately within the mangrove forest. The velocity will decrease due to the drag coefficient of the vegetation module in SWASH. However, it is not known if this accurately simulates the effect of roots and stems on the bed shear stress, and therefore the sediment concentration and transport within the mangrove fringe.
Appendix G: SWASH

G.2.2 Sediment module

**Sediment concentration**  When using cohesive sediment, SWASH uses the Partheniades-Krone formulations to compute the mass exchange of suspended sediment between the bed and the water column, and hence the sediment concentration. The sediment concentration is heavily dependent on three parameters: the critical bed shear stress for erosion, the entrainment rate for erosion and the fall velocity of the sediment particles. Despite the efforts made to determine accurate values for these parameters in SWASH, the sediment concentration remains quite high, in the order of 10-12 g/l at the point of wave breaking, during annual storm conditions. There are no measurements available within the project area during annual storm and heavy storm conditions to validate the model.

**Sediment sinks and sources**  The Partheniades-Krone formulations are based on the erosion and deposition fluxes of the sediment. If the deposition flux exceeds the erosion flux, net sedimentation occurs. According to the model results nowhere in the proposed convex-up profile nor the present concave-up foreshore profile net erosion occurs, as the sediment flux remains positive. This is in contradiction with observed erosion in the case study areas.

G.2.3 Research objectives

The model results of the wave transformation on very gentle slopes in combination with shallow water and the model results with respect to the sediment module are not in line with what would be expected, what is reasonable, and/or is observed in reality. Therefore further research is required to determine what is causing this behaviour in SWASH. In this section possible research objectives are proposed. The proposed measurement campaigns should be carried out in the field, because modelling the very gentle slopes of mangrove foreshores in laboratory settings would require impossible long flumes.

In order to validate the wave transformation of SWASH on very gentle slopes in combination with shallow water, the set-up of measurement campaigns is of utmost importance. This is the first step in analysing the wave transformation on very gentle slopes in combination with shallow water, which could lead to improvements of the numerical model SWASH.

Furthermore, investigating what is causing the low frequency peak in the variance density spectrum is absolutely essential. Based on experience from handling the model it is expected that the frequency peak is generated mathematically by the numerical model in order to compute a wave height on the very gentle slopes. It can be seen as the natural vibration of the model which is not due to applied force. However, it is wise to investigate other options as well. For example, the problem could be caused by the offshore boundary condition. It is possible that the offshore boundary condition by itself is generating a low frequency peak, which is not visible to the naked eye. When it is determined what is causing the low frequency peak, it can be solved and thereby the suitability of the numerical model SWASH for modelling on very gentle slopes could be strongly improved.

The measurement campaign is also the key to solving the wave height over depth ratio uncertainties. The assumption that the computed wave height over water depth ratio according to SWASH is too high, is probably due to two processes. Firstly, the physical process determining the wave height with respect to the water depth may not work adequately on very gentle slopes. Secondly, viscous damping due to the mud on the gentle foreshore may not be taken into account sufficiently or taken into account at all. The
first process should be validated and improved if necessary, the second process should be validated and added to SWASH if needed.

With respect to the bed shear stress within the mangrove forest, it should be verified if and how SWASH accounts for mangrove roots and stems when the velocity near the bed is computed. These processes need to be validated with field measurements.

The cohesive sediment module in SWASH appears to perform insufficiently. The cause of the computed high sediment concentrations should be investigated, and a measurement campaign should be set-up in order to validate the sediment module of the numerical model. When the sediment module is validated and/or improved, the sediment sinks and sources should be computed anew in order to obtain a more probable result.

All of the proposed research objectives leading to improvements of SWASH, concerning the specific conditions found in the Vietnamese Mekong Delta and mangrove coasts in general, would improve accuracy, and thereby cost-efficiency, in designing adaptable coastal protection measurements specific to these areas.
Appendix H

Van Rijn formula

Equation H.1 displays the simplified equation for longshore sand transport according to van Rijn (2002).

\[ Q_{t,\text{mass}} = K_o \cdot K_{\text{swell}} \cdot K_{\text{grain}} \cdot K_{\text{slope}} \cdot H_{s,\text{br}}^{2.5} \cdot V_{\text{eff},L} \] (H.1)

Where:

- \( Q_{t,\text{mass}} \): longshore sand transport, dry mass [kg/s]
- \( K_o \): 42 [-]
- \( K_{\text{swell}} \): swell correction factor for swell waves [-]
- \( K_{\text{grain}} \): particle size correction factor [-]
- \( K_{\text{slope}} \): bed slope correction factor [-]
- \( H_{s,\text{br}} \): significant wave height at breakerline [m]
- \( V_{\text{eff},L} \): effective longshore velocity at mid surf zone [m/s]

The analysis of the particle size effect shows that for small particle sizes the longshore transport decreases by a factor 2 to 3 in case the particle size increases from 0.2 mm to 0.4 mm. The range for this strong decrease is tested for particle sizes from 0.15 to 10 mm. If the particle size exceeds 2 mm, the longshore transport is no longer influenced by the particle size.

![Figure H.1](image)

**Figure H.1:** Effect of particle size on longshore transport (Van Rijn, 2002). Offshore wave angle=30 degrees, offshore wave height=3 m at a offshore depth of 15 m.
Appendix I

Nourishment execution time

**TSHD**

**Hopper specifications**
- Draught empty: 3.5 m
- Draught full: 8.0 m
- Hopper volume: 6000 m³
- Sand: D=100%
- Silt: D=65%
- Distance from winning site: 9 km

**Productivity**
- Hopper contents 60% sand mixture: 5150 m³
- Loading production: 82 m³/min; 4900 m³/h
- Discharging production: 82 m³/min; 4900 m³/h
- Sailing speed full: 13 knots; 24 km/h
- Sailing speed empty: 14 knots; 26 km/h

**Work cycle time**
- Pumping time: 65 min
- Sailing full + turning time: 35 min
- Discharging time: 65 min
- Sailing empty + turning time: 30 min

Total cycle time: 195 min

**Execution**
- Service hours a week: 168 h
- Downtime:
- Mechanical 3%: -5
- Operational 5%: -8
- Tide and waves 20%: -34

Total operational hours a week: 121 h

- Number of dredging cycles: 37 cycles/week
- Weekly production: 190,550 m³/week
## Appendix I: Nourishment execution time

### Cutter specifications

<table>
<thead>
<tr>
<th>Specification</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Draught loaded</td>
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<tr>
<td>Distance from shoreline</td>
<td>3 km</td>
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<tr>
<td>Cutter power</td>
<td>700 kW</td>
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### Productivity

<table>
<thead>
<tr>
<th>Productivity</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter pipe</td>
<td>0.65 m</td>
</tr>
<tr>
<td>Discharge fine silts+water</td>
<td>3.0 m/s</td>
</tr>
<tr>
<td>Fine sediment concentration</td>
<td>30 %</td>
</tr>
<tr>
<td>Discharging production</td>
<td>1075 m³/h</td>
</tr>
<tr>
<td>Efficiency rate CSD</td>
<td>0.8</td>
</tr>
<tr>
<td>Net fine sediment discharge</td>
<td>860 m³/h</td>
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</table>

### Work cycle time

<table>
<thead>
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<th>Duration</th>
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</thead>
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<tr>
<td>Service hours a week</td>
<td>168 h</td>
</tr>
<tr>
<td>Downtime</td>
<td></td>
</tr>
<tr>
<td>Mechanical 3%</td>
<td>-5</td>
</tr>
<tr>
<td>Operational 5%</td>
<td>-8</td>
</tr>
<tr>
<td>Tide and waves 10%</td>
<td>-17</td>
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<tr>
<td>Total operational hours a week</td>
<td>138 h</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Production</th>
<th>Value</th>
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<tbody>
<tr>
<td>Number of dredging cycles</td>
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</tr>
<tr>
<td>Weekly production</td>
<td>118,680 m³/week</td>
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