Project Dodanduwa

November 2016

Authors:

M.C.W. Hamelink
J.A.W. Molendijk
A.J. van Renswoude
J.C.A.J. Vork
T. Wevers
V.T.M. van Zelst

MULTIDISCIPLINARY PROJECT

Project team	Study track	Student number
M.C.W. (Marc) Hamelink	Hydraulic Structures	4171357
J.A.W. (Jorrit) Molendijk	Geo-Engineering	4152522
A.J. (Arie-Jan) van Renswoude	Geo-Engineering	4172876
J.C.A.J. (Jeroen) Vork	Geo-Engineering	4079728
T. (Toon) Wevers	River, Ports & Waterways	4152077
V.T.M. (Vincent) van Zelst	Coastal Engineering	4169980

projectdodanduwa2016@gmail.com

Supervisors Delft University of Technology

Ir. H.J. Verhagen	Associate Professor in Hydraulic Engineering
Dr. Ir. D.J.M. Ngan-Tillard	Assistant Professor in Geo-Engineering

Local supervisor

Eng. C. Fernando	General Manager EML Consultants

Committee of recommendation

Prof. Ir. Ch. A.M. Luyben	Rector Magnificus Delft University of Technology
Prof. Dr. Ir. B.M. Geerken	Dean Faculty of Civil Engineering & Geosciences Delft University of Technology
Prof. Dr. Ir. S.N. Jonkman	KIVI Chairman Civil – and Hydraulic Engineering
Prof. Dr. Ir. M.J.F. Stive	Former Chairman of the Department Hydraulic Eng. Delft University of Technology
Prof. Dr. M.A. Hicks	Professor of Soil Mechanics Delft University of Technology
H.E. Ir. Doornewaard	Ambassador to Sri Lanka and the Maldives Embassy of The Netherlands

Faculty of Civil Engineering and Geosciences

Building 23

Stevingweg 1 / PO-box 5048

2628 CN Delft / 2600 GA Delft

1. ACKNOWLEDGEMENTS

Project Dodanduwa is a multidisciplinary project about the improvement of the fishery harbour of Dodanduwa. The project was conducted by a team of six students during a period of nine weeks. The students follow various Civil Engineering master programmes at the Delft University of Technology concerning two main fields: Hydraulic Engineering and Geo-engineering.

This project would not have been possible without the support and supervision of several parties. First we would like to thank our supervisors H.J. Verhagen, D.J.M. Ngan-Tillard and C. Fernando. Their feedback and input during the project was highly valued and appreciated. Without, we would not have reached the current result.

Moreover, we would like to thank EML Consultants and all employees for their generous hospitality and friendly welcome at the company. At EML Consultants we were provided with all facilities needed for the nine week period of work, which we are very grateful for.

We are proud of our partner DAMEN Shipyards B.V. and sponsor Van Oord and thank them for their support, which eased the financial aspects of Project Dodanduwa.

Furthermore, our gratitude goes to the following people, in random order: Jos Vork, Ad Wevers, Sarath Bandara, Dirk Fransen and Raju. Everyone supported our project in a different way, our end result greatly benefited from their support and advice. The Embassy of the Netherlands in Sri Lanka & Maldives was so kind to receive us twice and invite us to several events.

2. SUMMARY

Dodanduwa is a small fishery village in the south-west of Sri Lanka, about 20.000 inhabitants are dependent on the fishery industry. Currently the harbour facilities, that were constructed in 2009, are hard to reach due to the wave conditions and sedimentation at the harbour entrance. The Ministry of Fisheries and Aquatic Resources Development initiated a rehabilitation project in 2015. In this report the issues at the Dodanduwa harbour area are thoroughly investigated and conceptual solutions are presented.

The goal of this report is to provide several durable and feasible conceptual designs for the fishery harbour of Dodanduwa; which fulfil the needs of the community on the longer term, taking the socio-economic and environmental effects into account.

This goal is reached by answering five sub questions:

1. What does the coastal system look like?
2. What is causing navigability issues at the harbour?
3. What are the involved parties and what are their interests?
4. What is the impact of the improved harbour on the surrounding area?
5. Which requirements should be considered when designing the harbour improvement?

The report is divided into three parts: the preliminary study, the conceptual design and the evaluation.

The preliminary study starts with the geological site investigation, in which is concluded that the rock outcrops consist mainly out of gneisses aging back to the Precambrian. The main geological process identified is sediment transport which is shaping the coastline. A geohazard resulting from this process is underwater slope instability of loose sand deposits. Two more geohazards are identified: tsunami's and earthquakes; these geohazards are not taken into account in the design. Both hazards happen with a relatively low frequency, using these hazards in the design process is not feasible.

The climate of Sri Lanka is characterized by two monsoon- and two inter-monsoon periods. The tidal range is small in comparison to the wave height, the West coast can be classified as wave dominant. Swell waves dominate the wind waves most of the time, only during the SW monsoon the wind waves can dominate the swell waves. The overall dominating wind direction is WSW, the direction of the waves during the SW monsoon is SSW. During inter-monsoon periods, wind waves are negligible. Waves from the SW direction enter the bay of Dodanduwa directly, waves from the SSW and S direction might enter the bay either by refraction or diffraction.

The net sediment transport at the Dodanduwa region is directed northwards, the sandbar formed at the entrance of the harbour is mainly created due to sediment transported by diffracted waves. Set-up differences in the bay north of the harbour drive the current which transports the sediment towards the harbour entrance. Since the tidal range is low, the tidal influences on the sediment transport are low as well. Flood tide does transport fine sediments more upstream of the river.

In 2009 the first major adjustment has been made to the Dodanduwa fishery harbour, a breakwater was constructed at the northern bay. The breakwater was only partly constructed, resulting in a less safe situation in the harbour. The redevelopment plans of the harbour involve ensuring safe navigation inside the harbour area and enough anchorage capacity for the coming 10 years.

The governmental organisations (CFHC and PI&MU), are believed to be the most influential in the project. Those parties can both be seen as the client and are proactively involved. Other governmental stakeholders like the CC&CRMD and the ID&ASD have to decide whether the design meets the regulations. The actual users and people affected most by the new harbour design only have a small influence in the project. They do have representatives attending meetings in which they share their interests, but they are not in the position to stop the project. The opinion of fishermen is of great importance for the CFHC projects.

Stakeholder meetings provide input for the harbour design. The most important requirements are a safe and accessible harbour area. Moreover, the expected shift to multiday vessels in the Dodanduwa fishery community must be taken into account.

The redesign of the Dodanduwa fishery harbour will have a socio-economic- and environmental impact. During the construction phase of the project the impact on the environment is moderate, some nuisance to the residents is expected. After construction the new harbour is expected to cause a transition in traffic, changes in erosion and accretion patterns and an increase in fishing activities. The neighbouring harbours close to Dodanduwa will be affected by the new harbour, some of the fishermen living close to Dodanduwa will shift their fishing activities from neighbouring harbours towards Dodanduwa.

To solve the issues in the Dodanduwa harbour on the long term, five different conceptual designs are presented. In the first conceptual design the bay north of the harbour facilities is used for anchorage. The river mouths in this bay, creating access to the harbour facilities through the river. The second concept is similar to the first concept, but the river is diverted to the south and the harbour facilities are accessed at the south of the northern bay. After these two concepts the third concept uses the bay south of the harbour facilities for anchorage. Since this bay is smaller, close to the harbour facilities the river is widened to create additional anchorage. In the fourth concept both bays are connected and used for anchorage. The river mouth is adjusted to ensure access to the harbour facilities from both bays. The last concept presented is the fifth, in this concept only minor adjustments are suggested. A sediment trap is presented to prevent sedimentation, a buoy is installed to warn fisherman for the wave conditions.

To see the differences of the concepts, they have been evaluated. Concept 1 did well on durability and socio-economic criteria and has a good overall score. Concept 2 got high ratings for the safety and socio-economic criteria, but performed less on the extent of works. Concept 3 scored below average, with below average scores for durability and the extent of works. Concept 4 scored good in safety of the entrance and durability, but the extent of works of this concept are very large. Concept 5, where only minor adjustments are suggested, scored less on the boundary conditions which include the draught, capacity and accessibility. This is the result of not taking in account the multiday vessels, but scoring the concept on the same criteria. The extent of works of concept 5 is very limited, just like the socio-economic impact.

At the end of the design phase some recommendations are given. Six general recommendations are presented, closing with separate recommendations for the design of the hydraulic structures. Before continuing with the final design the expected future fleet should be investigated more thoroughly. Large uncertainties in the expected future fleet may lead to a non-functional harbour improvement. Another uncertainty is the effect of an all year open river mouth; with stakeholder interviews and research on salt water intrusion this should be cleared out. For a better prediction of the impact of the hydraulic structures on the coastline, performing wave and sediment modelling provides more insight. The extension of the current breakwater must be further investigated, the safety level of the current breakwater must be determined. The transition between the current breakwater and the new breakwater may not lead to a weak spot. The current preliminary designs are based on several assumptions due to data limitations. For the final designs it is strongly recommended that all investigations, such as bathymetry measurements and soil investigations, are performed properly. The financial feasibility of the project is uncertain, multiple fishery harbours in the region are already constructed. A study to determine total life cycle costs must be performed to determine the financial feasibility. Separate recommendations are made to improve and optimise the designs of the hydraulic structures. With gaining the needed data mentioned before most designs can be optimised, the breakwater design requires a quarry investigation.

3. DISCLAIMER

This report is written for a multidisciplinary project executed by MSc. students from Delft University of Technology. It is an informative report about the problems occurring in the Dodanduwa harbour area. Conceptual designs to counter these problems are presented, while keeping in mind what their impact on a technical, but also the socio-economical scale might be. Note that this report is written by a neutral party, therefore no recommendations or ‘best solution to the problem’ will be given. Project Dodanduwa was supervised by the Delft University of Technology, supported by Damen and van Oord and is executed in cooperation with EML Consultants. However, the statements in the report do not necessarily represent the views of these companies.

4. TABLE OF CONTENTS

5. LIST OF FIGURES AND TABLES

5.1 FIGURES

5.2 TABLES

6. LIST OF ABBREVIATIONS

Abbreviation	Meaning
CC&CRMD	*Coast Conservation and Coastal Resources Management Department*
CFHC	*Ceylon Fishery Harbours Corporation*
DWL	*Design Water Level*
FR BW	*Fully Reshaping Breakwater*
FRP	*Fibre-reinforced plastic*
GDP	*Gross Domestic Product*
GSMB	*Geological Survey and Mining Bureau*
ID&ASD	*Irrigation Department and Agrarian Services Department*
IDAY	*Single day boat*
IMUL	*Off-shore multiday boat*
MFARD	*Ministry of Fisheries and Aquatic Resource Development*
MHWS	*Mean High Water Spring*
MLWS	*Mean Low Water Spring*
MSL	*Mean Sea Level*
MTRB	*Mechanised traditional craft*
NARA	*National Aquatic Resources Research and Development Agency*
NBSB	*Beach Sein Craft (non-mechanised)*
NTRB	*Non-mechanised Traditional Craft*
OFRP	*FRP boat with outboard engine*
PI&MU	*Project Implementation and Monitoring Unit*
PIANC	*The World Association for Waterborne Transport Infrastructure*
PR BW	*Partly Reshaping Breakwater*
PVC	*Polyvinylchloride*
SLS	*Serviceability Limit State*
SPT	*Soil Penetration Test*
SWL	*Still Water Level*
TOR	*Terms Of Reference*
UCS	*Unconfined Compressive Strength*
ULS	*Ultimate Limit State*
UN	*United Nations*

7. Glossary

Accretion	Process in which sediment is added to the coast
Anchorage	Place where boats lower their anchor
Aqua culture	Cultivating fresh- and saltwater species under controlled conditions.
Bathymetry	The measurement of depths of water in oceans, seas and lakes; also information derived from such measurements.
Berm breakwater	Rubble mound structure with horizontal berm of armour stones at about sea side water level, which is often allowed to be (re)shaped by the waves.
Breaker zone	Zone within which the waves approaching the coastline commence depth induced breaking.
Breakwater	A structure protecting a shore area, harbour, anchorage or basin from waves.
Cambrian Period	The first geological time period, lasted for about 53 million years (541-485.4 million years ago).
Coastal fishing	Fishing in waters with a depth from 10 to 200 m. (fishing on the continental shelf)
Deep sea fishing	Fishing beyond the continental shelf.
Diffracted wave	Bending of a wave when encountering an obstacle.
Design storm	Sea walls will often be designed to withstand wave attack by the extreme design storm. The severity of the storm is chosen in view of the acceptable level of risk of damage or failure.
Discharge	Volume of water flowing through a channel in a given timespan.
Erosion	Process in which sediment is taken from the coast.
Folds	Bending or curvature of an originally flat and planar surface as a result of permanent deformation.
Faults	Fractures that have displacements of the rock along them
Gabion	A steel cage filled with rocks, concrete or soil.
Geohazards	A geological state that may lead to widespread damage or risk.
Harbour basin	The part of the river from the ocean to the harbour facilities.
Harbour bay	The specific area located between the breakwater and the coast.
Inter-monsoon	Period between two monsoons
Jetty	A pier or structure of stones, piles, or the like, projecting into the sea or other water body to protect a harbour, deflect the current etc.
Landing site	Place where fisherman lay their fishing boats at rest on the beach.
Monsoon	The seasonal changes in atmospheric circulation and precipitation associated with the assymetric heating of land and sea.
Mooring	A permanent structure to which a vessel can be secured.
Navigability	A body of water is navigable if it is deep, wide and slow enough for a vessel to pass or walk.
Overtopping	Water passing over the top of a structure.
Precambrian age	The time the earth began to form, 4.6 billion year ago untill the Cambrian period 451 million years ago.
Return period	The average time between two extreme events.
Revetment	A facing of stone, concrete, etc., built to protect a scarp, embankment, or shore structure against erosion by wave action or currents.
Sandy littoral zone	The area offshore in which sediment transport due to waves takes place.
Sediments	Mineral or organic matter deposited by water, air or ice.
SwanOne	Computer tool used for transformation of offshore waves to onshore waves.
Tombolo	Deposited landform in which an island is attached to the mainland by a narrow piece of land.
Trade wind	A wind blowing steadily towards the equator from the north-east in the Northern hemisphere, or the south-east in the Southern hemisphere
Trailing edge coast	A coastal area that is not on the side of plate subduction or collision.
Wave set-up	Waves travelling towards the coast increase the water level at the shore. Higher waves lead to a high set-up, while lower waves lead to a lower set-up.

8. INTRODUCTION

8.1 BACKGROUND

Dodanduwa is a village in the south-west of Sri Lanka, located approximately 80 kilometres south of the capital Colombo and 17 km north-west of the city Galle, see Figure 8-1. The focus of this report is on the fishery harbour of Dodanduwa. The fishing industry is one of the most important industries in Sri Lanka. Although it only covers 1.8 % of the GDP, over one million people are financially dependent on this industry (MFAR M. o., 2016). It can thus be said that this sector plays a key role in the economic and the social life of Sri Lanka. The fishery sector can be subdivided into three parts, namely coastal; offshore and deep sea; and inland and aquaculture. In total there are 25 districts of which the Galle-District in the south-west of Sri Lanka, is one of the biggest. The Galle-district covers with 51.550 Mt, 11.2% of the national fish catch (MFAR, 2016).

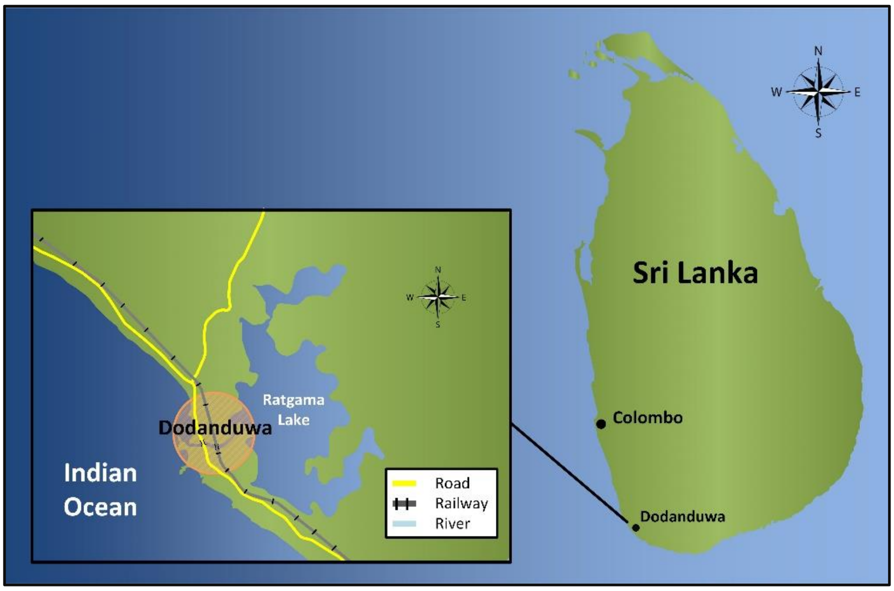


Figure 8-1: Sri Lanka, with a close-up from the fishery village Dodanduwa, located in the south west of the country

Within this Galle District there are nine fishery divisions of which Dodanduwa is the biggest. Dodanduwa is also one of the four divisions with a harbour facility. In the village of Dodanduwa, about 19.360 inhabitants are involved in fishery activities. Through this village, a river flows from the Ratgama Lake towards the Indian Ocean, shown in Figure 8-2.

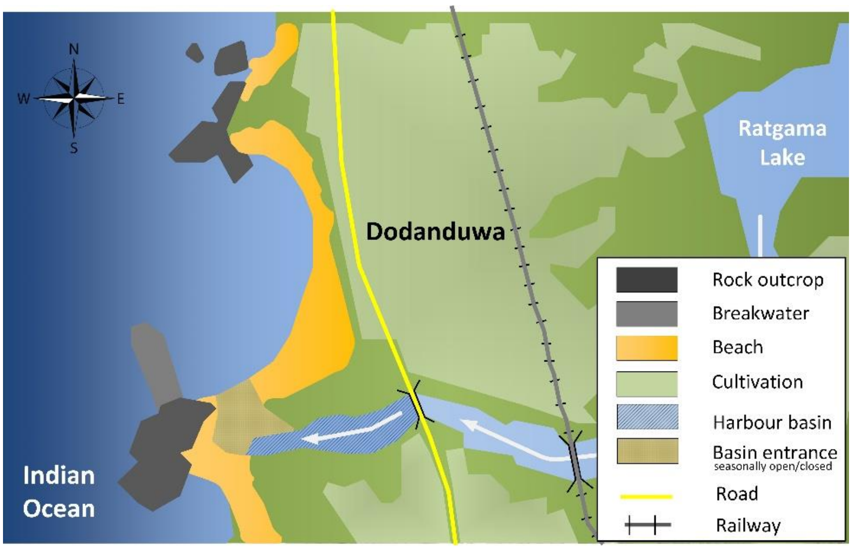


Figure 8-2: The surroundings of the Dodanduwa harbour area

8.2 PROBLEM DEFINITION

The harbour area of Dodanduwa is shown in Figure 8-3 , all key features of the environment of the onshore harbour facilities are described.

Figure 8-3: The Dodanduwa harbour area, all key features are marked

The onshore harbour facilities are best reached through the lagoon area, but most months of the year this area cannot be reached from the ocean. During the dry seasons the river flow through the lagoon, originating from the lake, is not strong enough to flush incoming sediment out. Due to this sedimentation of the harbour basin mouth, fully loaded vessels must be landed and unloaded on the beach, which is a labour-intensive job.

Moreover, large waves hit the current landing sites, creating a hard time for the fishermen to leave and enter the bay. In 2009 a project was started to improve the wave conditions in the harbour by constructing a breakwater. Since this breakwater was only partially constructed, conditions did not improve but got even worse. Sedimentation got more severe and navigation around the harbour entrance became more difficult.

Furthermore, the capacity of the harbour is currently not satisfactory. Next to the many vallams which operate in the harbour, more multiday vessels should be attracted. A harbour improvement might do so.

8.3 GOAL

The goal of the project is as follows:

"Providing several durable and feasible conceptual designs for the fishery harbour of Dodanduwa; which fulfil the needs of the community on the longer term, taking the socio-economic and environmental effects into account."

To reach this goal and make such conceptual designs, the following five sub questions should be answered first:

1. What does the coastal system look like?
2. What is causing navigability issues at the harbour?
3. What are the involved parties and what their interests?
4. What is the impact of the improved harbour on the surrounding area?
5. Which requirements should be considered when designing the harbour improvement?

8.4 PROCEDURE

To reach the stated goal and answer all sub questions the project team has made a project strategy, including planning and global task division. These can be found in an additional document: 'Strategy, planning and task division'.

The planning consists of four main phases: data acquisition, problem analysis, conceptual design and evaluation. The first two phases take about two to three weeks: stakeholder interviews and meetings, literature review and a site visit are essential. Thereafter the conceptual design phase follows with brainstorm sessions on possible solutions, this also provides insight in the missing knowledge. Further investigations for valuable data are planned based on the identified missing knowledge. Following up the brainstorm sessions, the chosen conceptual solutions will be worked out. Closing the project, an evaluation will take place and the report will be finished. Concluding, the designs will be presented to the main parties.

8.5 READING GUIDE

The preliminary study starts with chapter 9, discussing the geology both for Sri Lanka and the Dodanduwa region. The geology is followed by an analysis of the coastal system of Sri Lanka, discussing climate and wave climate. In chapter 11 the sediment transport along the south-west coast of Sri Lanka, Dodanduwa specifically, is discussed. The chapter about the current state elaborates on a timeline of the harbour development, the facilities and boats present, closing with the project proposed by the Ministry of Fisheries. The impact on the environment, Dodanduwa's community and neighbouring harbours are discussed in chapter 13. The preliminary study ends with an analysis of the stakeholders, showed in a power-interest diagram. All findings of the preliminary study are summarized in chapter 15, additional information is found in appendices A to F.

The conceptual design start with the design approach and constraints and requirements in chapter 16. All concepts are discussed afterwards in chapter 17, more details about the designs can be found in appendices G to N.

In the final part of the report, an evaluation of the conceptual designs is presented and recommendations for future investigation are done. Appendix O provides a more detailed evaluation. Appendices P to R provide information about the interviews and the site investigations.

PRELIMINARY STUDY

The preliminary study is the basis of this report, the study will slowly zoom in on the project area. After this study the five sub questions are answered, or in other words: the extent of the problem and the underlying causes must be clear. First the geology will be investigated, then the coastal aspects like wind, waves, tide, sediment and the effects they have on the site will be examined. An overview of the current state of the Dodanduwa harbour is given. For practical reasons, the availability of construction materials will also be dealt with. Finally, environmental impacts due to a possible interferences and the stakeholders will be mapped.

9. GEOLOGY OF SRI LANKA

In order to see what limitations or challenges can be expected when making a design for a civil technical construction, it is useful to gain knowledge about the geology of the site (Waltham, 2009). In this section the total geological history approach is used as a preliminary desk study. After the desk study a more detailed research of the geology has been performed by the use of a geological map. The chapter ends with the indication of possible geohazards and a conclusion.

9.1 TOTAL GEOLOGICAL HISTORY APPROACH

The geology of Sri Lanka is analysed using the total geological history approach from P. Fookes (Fookes, 2000). With this approach, a desk study is performed from which the knowledge of the subsurface conditions of the site can be improved in an early stage. Information gathered during the desk study will give insight used for identification of the following conditions:

- Tectonic models
- Geological models
- Geomorphological models

Tectonic model

An important chapter in Sri Lanka's geological history starts in the Precambrian age. Most of Sri Lankan rock formations are formed in a Precambrian sedimentary basin (Dissanayake C.B. & Munasinghe, 1984). Due to intraplate tectonics, the current rocks are metamorphic and heavily faulted and folded. These features strongly correlate with the intraplate setting: cratons (Fookes, 2000). The intraplate tectonics are confirmed by checking the position of Sri Lanka on the tectonic map, where it is located at the middle of the Indian Plate (Appendix A.1, Figure A-1). Therefore, this tectonic model is adopted as a basis for the geological history of Sri Lanka. Observations from a regional geological map and a brief walk over survey both confirm the presence of gneiss, faults, folds, intrusions and unconformities. In Appendix A.1, Figure A-2 the characteristics of the tectonic model are visualized. The more specific local geology will be discussed later in this report.

Geological models

The next step includes deriving relevant geological models for the site. Based upon the chosen tectonic model, the article Chemical Geology from (Dissanayake C.B. & Munasinghe, 1984) and a local geological map the following relevant geological models are selected:

- Metamorphic - Gneisses and Migmatites
- Structural - Multiple fold and shears

The features from both models are shown in Appendix A.1, Figure A-3.

Geomorphological model

Finally, the geomorphological model for the site location is determined. Since Dodanduwa is located at the coast, coastal features are dominating the geomorphological behaviour. Relevant aspects of the system are the ebb and flood tidal deltas, backbeach dunes with lagoon deposits, longshore drift, rock features and tombolos (Appendix A.1, Figure A-4). Both destructive and constructive behaviour at the coast was seen on satellite data and field observations.

9.2 GEOLOGY OF SOUTH-WEST SRI LANKA

In this section the geology of the South-West region of Sri Lanka is analysed. For the analysis a geological map of the Alutgama-Galle region is used, in which the project site is located. First the formation will be mentioned, after which the geography and geology of the region will be discussed. Finally, the geology of the area surrounding Dodanduwa and the potential geohazards will be analysed.

Formation

The biggest part of Sri Lanka is lying on late Proterozoic high-grade rocks; some Phanerozoic sediments can be found in the coastal region. As can be seen in Figure 9-1 there are three major lithotectonic bodies: The Vijayan Complex in the East, Wanni Complex in the North-West and in between the Highland Complex. The area of interest is located within this Highland Complex, previously also known as the Highland Series or Central Granulite Belt. In Figure 9-1 this area is indicated with the black box.

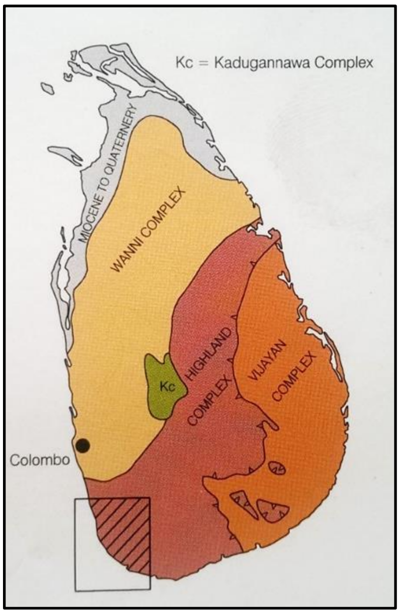


Figure 9-1: Lithotectonic Subdivisions of Sri Lanka. (GSMB, 2000)

Highland Complex

The Highland Complex is characterised by a sequence of metasediments and granulitic orthogneisses. The metasediments are interlayered with granitoid rocks and subordinate metabasic intrusive. The sediments are dated from 2000 million years ago, during the Precambrian eon. The rocks in the Highland Complex have been formed due to metamorphism 665-550 million year ago, under a medium pressure but very high temperature. These metasedimentary rocks are interlayered with more massive charnockites, probably of both sedimentary and igneous origin (Japan International Cooperation Agency (JICA), 2003).

Geography

The Alutgama-Galle region is a region where a tropical, wet and humid climate is present. Annual rainfall exceeds 3700 mm, of which the biggest part falls during the SW-monsoon (May to September) and the NE-monsoon (December to February). The monsoon winds and heavy rains regularly cause flooding of the lowlands and intense seasonal coastal erosion. The temperature is quite stable and varies from 27 to 30°C.

The land close to the coastline is characterized by lakes and lagoons that are seasonally connected to the ocean by small rivers. The vegetation around the lakes and lagoons is mainly of the mangrove type. Slightly further into the hinterland the vegetation alternates between tropical hard woods, scrub and grassland; which slowly transforms into an evergreen rain forest (GSMB, 2000).

Geology

The geology of the Highland complex is composed of inter-banded metamorphic rocks. In the Alutgama-Galle area the most abundant rock types are charnockite, charnockitic gneisses and garnetiferous quartzo-feldspathic rock. These metamorphic rocks are highly deformed due to tectonic movement as can be seen in Figure 9-2.

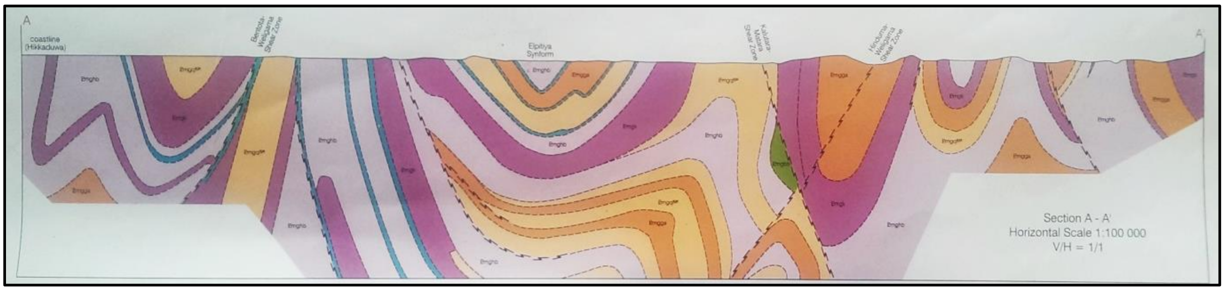


Figure 9-2: Section through the Alutgama-Galle region showing the highly folded rock (GSMB, 2000)

After the layers were folded, the top layer was eroded away leading to a relative flat profile near the coast. At the coast, some Quaternary deposits are present (Katupotha, 1994). These predominantly sandy deposits originate mainly from oceanic sediments (GSMB, 2000).

Geology of Dodanduwa

The Dodanduwa harbour area is characterized by unconsolidated brown and grey coastal sand; grey and white sand, underlain by layers of respectively garnet-bearing quartzofeldspathic rock and charnoclitic gneisses (Figure 9-3). At some points, outcrops of these underlying layers or coastal rock outcrops can be found (GSMB, 2000) (Pathirana, 1980). Gneisses are generally known as strong rocks, UCS varying between 50-200 MPa (Waltham, 2009) which tends to be more in the higher range. The tables for qualifying the rock strength can be found in Appendix A.2, Figure A-5 and Figure A-6. Known causes of complexity in gneiss are (Rosenbaum, 2016):

- Weak and strong materials in close association
- Heterogeneity
- Dominance of fabric
- Faulting
- Stress relief
- Groundwater flow through fissures

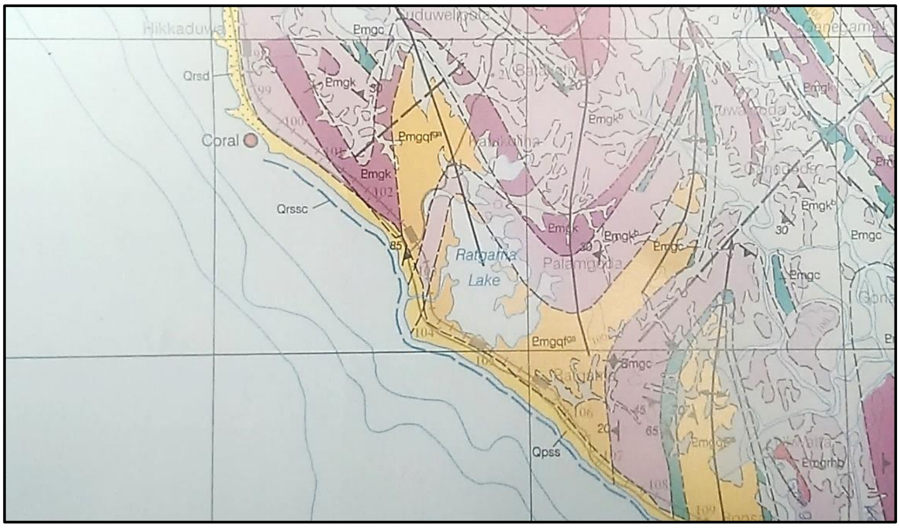


Figure 9-3: Outcrops near Dodanduwa (on the left of Ratgama Lake) (GSMB, 2000)

Since the harbour is located at the coast it is important to implement the coastal processes in the design. The geological process which can be expected to occur in this area is sediment transport (Waltham, 2009). The subject of sediment transport will be elaborated in chapter 11.

9.3 GEOHAZARDS

The last part of the preliminary desk study is the research for potential geohazards the project site is potentially exposed to. Two potential hazards which are discussed are earthquakes and tsunamis.

Earthquakes

Sri Lanka is located at the Indo-Australian plate, at a considerable distance of 1400 km from the plate boundaries (Figure 10-1). Based upon this information one might conclude that Sri Lanka is not exposed to earthquakes. However, studies provide evidence of a new plate boundary forming about 500-700 km from the southwest coast of Sri Lanka, resulting in earthquakes in the Indian Ocean. The new plate boundary will increase the possibility of an earthquake hitting Sri Lanka (Dissanayake, 2005). Records of earthquakes near Sri Lanka show some earthquakes have occurred in the region around Sri Lanka, but all at a fair distance. When considering the combination of frequency, distance and altitude of the earthquakes, it is concluded that no extra measures need to be taken into account in the new harbour design. Tsunamis generated by earthquakes in the Indian ocean might have to be taken into account, based on their frequency but that will be discussed in the next paragraph.

Tsunamis

Sri Lanka was hit by the Indian Ocean tsunami in December 2004, culminating in the loss of over 30.000 lives and a complete destruction of structures along the south-east coast line. The tsunami was caused when the Indian plate was sub ducted by the Burma plate, resulting in earthquakes and a series of tsunamis (Wijetunge J. , 2014).

After the tsunami of December 2004 the presence of tsunamis in Sri Lanka is unquestionable. However, the frequency of tsunamis hitting Sri Lanka is very low. Before the tsunami of 2014 the previous tsunami dated from 1883, as a result of the Krakatoa explosion. Previously mentioned earthquakes in the Indian Ocean do not generate tsunamis in such magnitude that they were noticed at the coast in Sri Lanka and are thus left out of consideration. Since the frequency of tsunamis occurring is extremely low it is not feasible to adapt the harbour design to the possible tsunami threat.

9.4 CONCLUSION

The identified total geological history models all indicate the complexity of the subsurface. During the investigation of the subsurface extra attention is needed when mapping the faults, folds and intrusions in the project area. These factors can have a large influence on the bearing capacity of foundations placed, the subsurface and the usability of armour stone from the region. The coastal features model shows the importance of the tides and waves in the project area. Conclusions from this analysis are stated in chapter 10 Coastal system, paragraph 10.3

From both desk study and field work (Appendix R), it was concluded that the rock head and outcrops in Dodanduwa consist mainly out of gneiss, with sometimes some garnet bearing layers in between. Gneisses are known as a strong to very strong rock, UCS varying between 50-200 MPa. Known causes of complexity in gneiss are (Rosenbaum, 2016); Weak and strong materials in close association/heterogeneity, dominance of fabric**,** faulting**, s**tress relief and groundwater flow through fissures.

Closer to the ocean, more recent deposits of sands can be found. Underwater slope stability from unconsolidated sands must be taken into account when construction in this area takes place. The expected geological process in this area is sediment transport.

Other potential geohazards identified were earthquakes and tsunamis. The frequency of those geohazards proved to be relatively low, designing a construction which is able to withstand these threats is thus not feasible

10. COASTAL SYSTEM

The present coast is a result of different processes which act on different timescales. On a long scale, coastal features like steep and narrow shelfs are formed by con- and diverging plates; on a smaller timescale waves and tides are shaping the coast. Besides that, typical features related to geological eras like cliffy and rocky coasts formed by glacial activity during the Pleistocene or coastal zones due to sea level changes during the Holocene, can be found.

10.1 LARGE SCALE GEOGRAPHY OF THE SRI LANKAN COAST

Sri Lanka is located at the Northern hemisphere, (7.9 ° N, 80.8 ° E) and located on the Indo-Australian plate. The coast of the island is characterized as a trailing edge coast of the Amero type, due to the availability of sediment. Sri Lanka has a very narrow continental shelf which is accompanied by a steep slope. On a narrow shelf large storm surges cannot develop. On the other hand, the wave height of breaking waves is higher and tend to break close to the shore due to the steep slope and absence of sandbars.

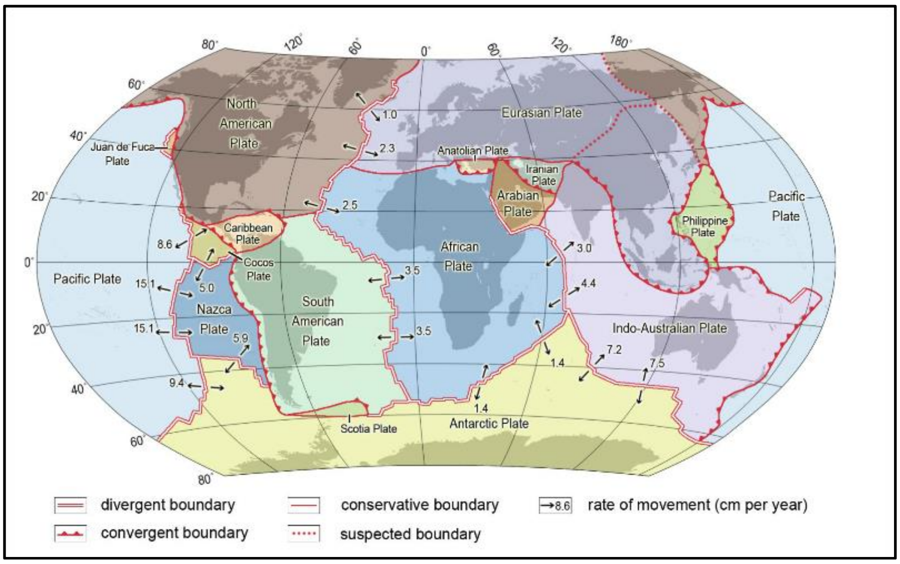


Figure 10-1: Location of Sri Lanka on the Indo- Australian plate (Stive, January 2015)

10.2 CLIMATE

Sri Lanka is situated just above the equator and for this reason the climate is characterized as tropical. Winds are generated due to uneven distribution of heat over the earth. Around the equator the solar radiation is high and warms up the air. The warmed air is directed to colder areas. This leads to the generation of the NE – and SE trade winds. These winds are moderate but persistent throughout the year. In South-Asia trade winds are overruled most of the time by seasonal winds. A wind is the result of pressure imbalances. An imbalance exists if a landmass is significantly warmer or cooler than the ocean. The difference in temperature is created due to the different heating capacities of the continent and the ocean. The continent heats and cools more quickly than the ocean. In South – Asia the ocean heats up during the summer, but the continent heats up more quickly. This results in a low pressure above land and a high pressure above the ocean. This causes a wind from the ocean towards the land. In the winter the wind direction reverses. This seasonally reversing wind is called the monsoon. The summer monsoon coincides with a rainy period, because the ocean to land wind brings moist air to the land. The pressure areas and the resulting winds are shown in Appendix B.2.

Seasons

In Sri Lanka four seasons can be distinguished. (Precipitation distribution can be found in Appendix B.1)

1) First inter-monsoon season March–April

A warm period accompanied by heavy thunderstorms at the end of the day. Especially SW Sri Lanka is receiving a lot of rainfall, on average order of 300-750 mm during this two-month period.

2) Summer monsoon/ South-West monsoon May–September

This period is classified as a warm and humid period. Rains occur during night and day. The SW-coast receives between 1000 – 1600 mm during this five-month period.

3) Second inter-monsoon season October–November

During this inter-monsoon period heavy thunderstorms occur. The whole country experiences heavy, quite even distributed rain fall. The weather is influenced by depressions and cyclones in the Bay of Bengal. The SW-coast receives rainfall in order 750 – 1200 mm during a period of only two-months.

4) Winter-monsoon / North-East December–February

Dry and cold winds result in a relatively cool and dry period in the southwestern region. This region receives rainfall in order 300 – 600 mm, the maximum rainfall is experienced in the East.

Global wave environment

The trade winds lead to moderate waves in the subtropics. Those smaller waves can be observed during the inter-monsoon periods and in absent of swell waves. Larger waves can be observed during the monsoon periods and due to swell waves. Those swell waves are generated in the Indian Ocean by the so called westerlies winds. Those waves are coming mainly from the South and South-West and could have a high wave height due to the long fetch. The waves generated during the monsoon are quite constant in direction and height. This leads to a narrow sandy littoral zone. During the summer monsoon the dominant wind direction is South-West and the South and the West coast of Sri Lanka are experiencing waves from the South-West. During the winter-monsoon wind is mainly coming from the North-East, during this period the North- and East coast are hit by waves from the NE.

Global tidal environment

Tidal waves are characterized by their magnitude and their duration, diurnal or semi-diurnal. The mean spring tidal range, MHWS-MLWS, in Sri Lanka is approximately 1 m. As shown in Figure 10-2, the tide has a mixed character at the West coast and semi-diurnal at the East coast.

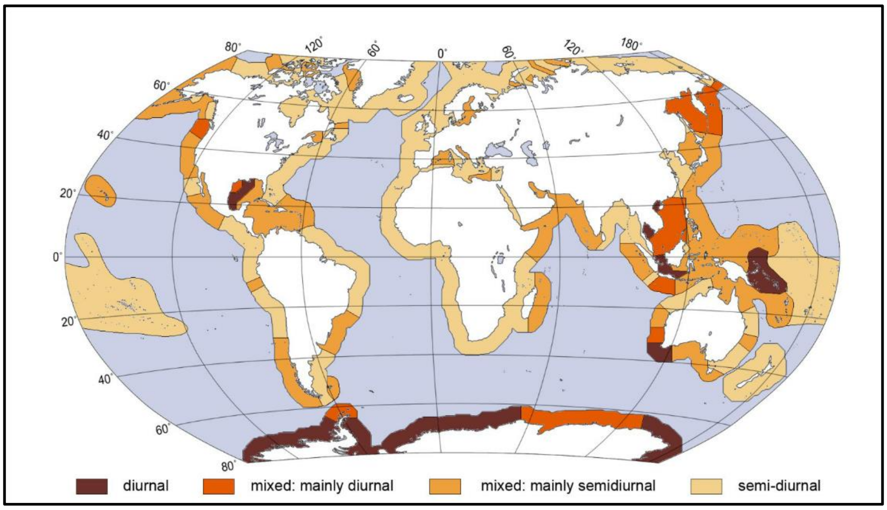


Figure 10-2: Tidal environments of the world (Stive, January 2015)

Coast classification

The tidal range is small in comparison to the wave height. Thus the influence of the tide is overruled by the waves. In Sri Lanka there are dozens of rivers with a total length in the order of 100 kilometres with their source in the hill country, an elevated area in the middle of Sri Lanka. The channels are quite straight close to the source, but are meandering when entering lowland. Most rivers have no or only a few bifurcations. When in the lowland, the discharge and/or sediment supply of the rivers is too low to counter the wave action and create a delta. Beside the larger rivers, a lot of small channels can be observed along the coast. Those channels serve often as link between the ocean and small lagoons. Those small rivers are not seldom seasonal rivers. In conclusion: the Sri Lankan coast is clearly wave dominated.

Coastal features and structures

Along the coast, concaved shaped beaches are formed between rocky outcrops. Those rocky outcrops could stop the littoral drift, depending on the length in which they extend into the ocean and the width of the breaker zone. Large bays are located at the south coast; examples are the bays close to Matara and Galle.

Along the West coast several human interferences can be observed. Examples of some projects: Detached breakwaters are built to counter the erosion problems in area North of Negombo. The railway from Colombo to Galle is built very close to the shore. Along several parts a revetment is built to protect the railway against erosion induced problems. Furthermore, a dozen small harbours can be found along the West coast.

10.3 Hydraulic conditions

In the next sections the hydraulic conditions will be discussed. This includes mean sea level, tides, wind, waves (seasonal and extreme), sea level rise and storm surge. The focus for the hydraulic conditions is on the harbour of Dodanduwa. More detailed information about the Hydraulic conditions can be found in Appendix B.3.

Mean Sea level

There is a seasonal change of mean sea level in Sri Lanka. This is due to the changing air pressure during the monsoons and freshwater input. The air pressure is at a maximum during the SW monsoon and at a minimum during the NE monsoon. In the 'Mean Sea Level' section in Appendix B.3 a figure is shown with the MSL change at the West and East coast. Table 10-1 shows the MSL changes during the monsoons at the West coast.

Table 10-1: Seasonal MSL Sri Lanka (WIJERATNE, 2007)

Characteristic	Determined level [m]
MSL SW Monsoon	+0.2
MSL NE Monsoon	+0.5

Tides

The tidal character in Dodanduwa is semidiurnal. The calculations of the tidal character of Dodanduwa can be found in the 'Tides' section in Appendix B.3. In Table 10-2 the tidal elevation is shown. The tidal range is 0.6 meter. During perigean spring tide the tidal range is 1.0 meter.

Table 10-2: Tides (J.I.C.A, 2003)

Characteristic	Determined level relative to MSL [m]
Highest Astronomical Tide (HAT)	+0.9
Mean High Water Spring (MHWS)	+0.7
Mean High Water Neaps (MHWN)	+0.4
Mean Low Water Neaps (MLWN)	+0.3
Mean Low Water Springs (MLWS)	+0.1
Lowest Astronomical Tide (LAT)	-0.1

Wind

Wind climate is predominated by the monsoons. During the NE-monsoon the dominant wind direction is NE. During the SW-monsoon the dominant wind direction is WSW. For the design of structures wind velocities of cyclones need to be taken into account. There are three zones in Sri Lanka determined with different dominating wind velocities per zone. Zone 3 applies for the SW coast of Sri Lanka.

Table 10-3: Wind velocities per zone (Wijerantne & Jayasinghe)

Zone	Description	Classification	Gust wind velocity [m/s]	Mean wind velocity [m/s]
3	South and West (Including Colombo)	I	33	18.3

More information about wind climate can be found in the 'Wind' section in Appendix B.3.

Waves

Seasonal waves

Sri Lanka's coast is exposed to swell -and wind waves. The swell waves dominate the wind waves most time of the year, however there are periods during the SW monsoon where the wind waves dominate the swell waves. The dominant wind waves direction is W and WSW. Swell waves come mainly from the SSW and S direction. During the SW monsoon the swell waves are almost twice as high than during the other months. The direction of the wind waves is correlated with the wind direction of the monsoon period. This is W and WSW direction during the SW monsoon and NE direction during the NE monsoon. During the inter-monsoon period wind waves are negligible. The waves from SW direction will enter the bay of Dodanduwa directly. Waves from SSW and S direction might enter the bay by either refraction or diffraction, depending on the local bathymetry. Table 10-4 shows the significant wave height, peak period of swell and wind waves and dominant directions. Table 10-5 shows the significant wave height, peak period and direction of the dominant waves.

Table 10-4: Yearly significant wave height, peak period of swell and wind waves

Characteristic		**Value**
Swell waves	Hs [m]	2.19
	Tp [s]	13.1
	Dominant direction	SSW - S
Wind waves	Hs [m]	1.25
	Tp [s]	4.1
	Dominant direction	WSW - W

Table 10-5: Overall significant wave height, peak period and dominant wave direction per month

Month	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**
Hs [m]	1.24-1.44	1.23-1.43	1.38-1.58	1.53-1.73	2.38-2.58	2.62-2.82
Tp (s)	12-13	12-13	13-14	13-14	13-14	13-14
Direction	SSW	SSE	S	S	SSW	SW
Month	**Jul**	**Aug**	**Sept**	**Oct**	**Nov**	**Dec**
Hs [m]	2.62-2.82	2.53-2.73	2.40-2.60	2.13-2.33	1.65-1.85	1.35-1.55
Tp (s)	13-14	13-14	13-14	12-13	13-14	13-14
Direction	SW	SW	SSW	SSW	SSW	S

Extreme waves

Return levels for extreme wave heights have a maximum during the SW monsoon. The design wave height is the overall extreme wave height per chosen return period. The overall wave height is the combination of wind wave height and swell wave height. In section 'Waves' in Appendix B.3, both wind- and swell wave heights are given.

Table 10-6: Extreme overall wave heights (Thevasiyani & Perera, 2014)

Probability	**Return Period**	**Overall extreme wave height**
	[years]	[m]
0.200	5	4.38
0.100	10	4.72
0.066	15	4.93
0.050	20	5.09
0.020	50	5.62
0.010	100	6.06

Sea level rise

Global warming is causing the sea level to rise. This has an impact on the coast of Sri Lanka; the coastal morphology will change and the risk on floods will increase.

The IPCC[1] is the international body for assessing the science related to climate change. In their report from 2007 they predict an increase of water level of 0.59 meter in 2100 (Disaster Management Center). Use of new information from IPCC report 2014 is recommended.

[1] Intergovernmental Panel on Climate Change

Storm surges

In the last 135 year two cyclonic storms have caused a storm surge on the SW coast of Sri Lanka. Storm paths have been analysed and placed in a model (Wijetunge, 2013). Storm surge levels are extracted from this model. More information about storm surge levels can be found in the 'Storm Surges' section in Appendix B.3.

Table 10-7: Return period and level of storm surges.

Return period [years]	Return level [m]]
50	0.50
100	1

10.4 Conclusion

The climate of Sri Lanka is characterized by two monsoon and two inter-monsoon periods. The tidal range is small in comparison to the wave height. Among the West coast several small rivers can be found, most of them seasonally closed off from the ocean. The sediment transport of those rivers is too small to build a delta. The West coast can thus be classified as wave dominant, characterized by high waves and absence of large tidal differences and sediment supply by rivers.

Sri Lanka's coast is exposed to swell- and wind waves. The swell waves dominate the wind waves most time of the year, however there are periods during the SW monsoon where the wind waves dominate the swell waves. The overall dominating wind direction is WSW. Swell waves come mainly from the SSW and S direction. During the SW monsoon the swell waves are twice as high as during the other months. The direction of the wind waves is correlated with the wind direction of the monsoon period. This is WSW direction during the SW monsoon and NE direction during the NE monsoon. During the inter-monsoon period wind waves are negligible. The waves from SW direction will enter the bay of Dodanduwa directly. Waves from SSW and S direction might enter the bay by either refraction or diffraction, depending on the local bathymetry.

11. SEDIMENT TRANSPORT

The island of Sri Lanka has a total coastline length of approximately 1600 km along which big amounts of sediments are transported. If such flows are interrupted, the interruption will cause coastline changes in that area. In this chapter, the sediment transport along the West coast and in particular around the Dodanduwa area are elaborated.

11.1 THE NET LONGSHORE TRANSPORT

The yearly net longshore sediment transport at the West coast is in northern direction (Wijayawardane, Ansaf, Ratnasooriya, & Samarawickrama, 2010). One could also conclude this by analysing the West coast of Sri Lanka looking at old interventions along the coast and the data given in section 10.3. This is caused by dominant swell waves arriving from the SSW direction, which are also significant in size compared to the sea waves.

11.2 THE GROSS LONGSHORE TRANSPORT

The monsoons periods influence the gross longshore sediment transport. During the year the wind direction is not persistent, resulting in different wave directions. In general, the swell dominates the transport to the North. Table 11-1 shows the dominant transport per monsoon.

Table 11-1: Dominant sediment transport per monsoon

Location	SW monsoon	1st Inter-monsoon	NE monsoon	2nd Inter-monsoon
Dodanduwa - Hikkaduwa	North	North	North	North
Dodanduwa - Galle	*	North	North	North

*The dominant transport in the SW monsoon is hard to determine due to the coastline orientation

During the SW monsoon and the 2nd inter-monsoon, wind waves hit the coast from the West. These waves cause the sediment transport to shift to the South at times, for the other monsoon periods, almost no waves arrive from the West. Waves from a storm event, which are not in the data given in section 10.3, are assumed to be higher than the swell waves. Where sediment transport is linked to wave height, these waves have a significant influence on the sediment transport to the south. Furthermore, there are days in which the swell is lower than wind waves. This also causes sediment transport to the south.

A longshore sediment transport study is performed by the university of Moratuwa (Wijayawardane, Ansaf, Ratnasooriya, & Samarawickrama, 2010). Using the numerical model MIKE21 NSW, they estimated the transport at the southwest coast. The total sediment transport has been calculated using the numerical expression from Kamphuis. More simplifications have been, i.e. just 1 grain size has been used. The stretch between Hikkaduwa and Gintota is of importance when focusing on Dodanduwa, see Figure 11-1. The study gives an indication of the order of magnitude of the transport and direction in the different periods. Results should be interpreted with care.

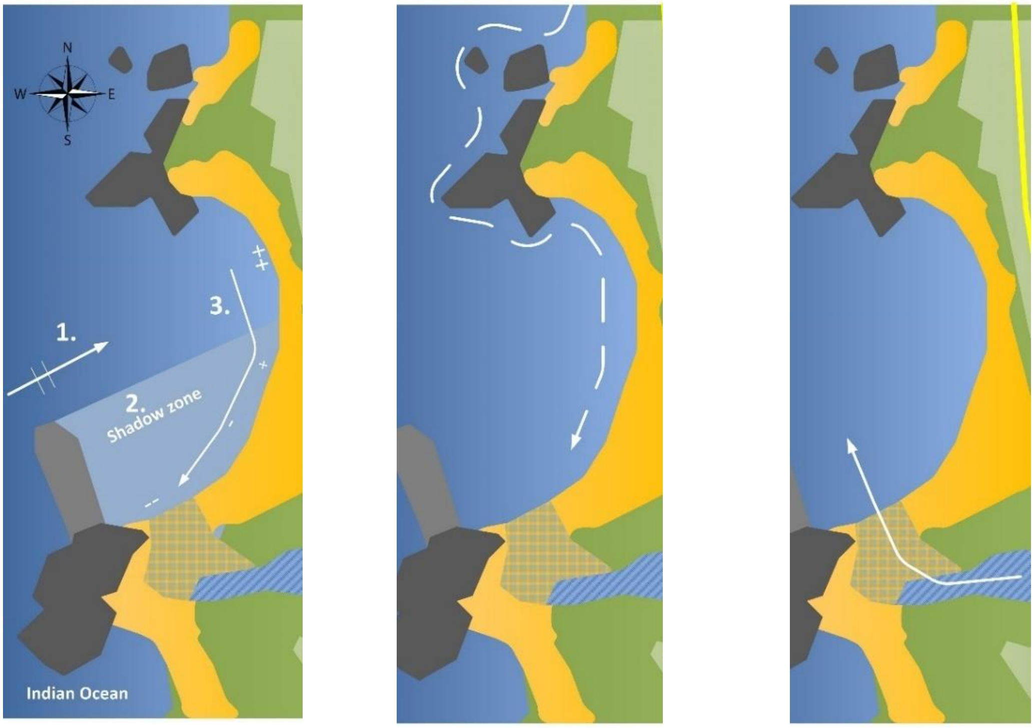
Alongshore sediment transport rate [x10^5 m^3/month]					
Postive values corresponds with transport to the North					
Location		**SW^a**	**IM^b**	**NC^c**	**IM^d**
Dodanduwa	Sea	-0.03	0.03	0.01	0.20
Hikkaduwa	Swell	1.25	0.70	0.23	0.35
	Total	*1.22*	*0.73*	*0.24*	*0.55*
Gintota	Sea	-0.44	-0.03	-0.04	0.20
Dodanduwa	Swell	1.28	0.83	0.23	0.37
	Total	*0.84*	*0.80*	*0.19*	*0.57*
aSouthwest Monsoon, bFirst Inter-monsoon, cNortheast Monsoon, dSecond Inter-monsoon					

Figure 11-1: Sediment transport rates at West coast

More on the sediment transport per monsoon can be found in Appendix C.

11.3 SEDIMENT TRANSPORT MECHANISMS

Different mechanisms are transporting sediment close to the harbour of Dodanduwa. The sum of these mechanisms is responsible for the net sediment transport rate. The mechanisms are treated briefly below:



Current caused by set-up differences | Longshore transport in Southern direction | River discharge

Figure 11-2: Sediment transport mechanisms

Current induced by set-up differences

The breakwater acts as a barrier for waves coming from the West and South (1), as shown in Figure 11-2. Due to diffraction, the wave height behind the breakwater is considerably lower (2). Alongshore wave heights differences result in wave force variations, which result in different wave set-ups. Set-up differences drive a current (3). These currents transport sediment towards the entrance of the basin.

Longshore sediment transport

Waves reaching the coast at an angle result in a longshore current which transports sediment. This current acts in the breaker zone. The width of the breaker zone is determined by the wave height distribution. Swell waves have a narrower spectrum, wind waves have a broader spectrum, leading to respectively a narrower and a wider zone. The distance from the shore to the breaker zone depends on the wave height and bathymetry. A steep slope will induce breaking relatively close to the shore and high waves break more off shore. Thus, the location of the breaker zone at the harbour differs during the year, due to the seasonal character of the waves.

Structures can block the sediment transported by the longshore current, a littoral barrier. Depending of the location of the breaker zone the northern- and southern rock outcrop could act as barrier. In this case sediment is trapped and will not leave the bay.

In general, the net sediment transport at the West coast of Sri Lanka is directed towards the North as mentioned above. Although the presence of the southern rock outcrop prevents sediment transport in this direction. Sediment by passes the northern outcrop, which is more shallow. There is no sediment bypassing the southernmost rock outcrop. (C. Fernando, Meeting, 2016).

River discharge

During the rainy season the discharge of the river increases and flushes away sediment which had accumulated in the basin entrance. Most of the sediment is assumed to be deposited in the harbour basin, and brought back to shore by mild wave conditions entering the bay. The intensity and duration of the rainy season differs from year to year, but in general the river is connected to the ocean for approximately five months a year.

11.4 Historical analysis of the Dodanduwa harbour

Research (Appendix C.1) has been carried out on the movement of sediment in and around the harbour area. The period of consideration was from March 2003 until January 2016. In this timespan several adjustments to the harbour area were made, all having their own effects on the sediment transport.

It was again concluded that the net sediment transport at the West coast is directed to the North. When focused on the harbour, local geometry became very important. The northern- and southern rock outcrops and the breakwater influence sediment transport. Due to the construction of the partly completed breakwater, the sediment transport has been disturbed recently. The southern rock outcrop near the harbour blocks longshore sediment transport while sediment bypasses the northern rock outcrop. A sum of the different mechanisms lead to the present state. A sandbar is formed at the entrance of the harbour basin by a mix of diffracted waves and tidal influence. During the rainy season the sediment is flushed out by a high river discharge connecting the basin seasonally to the ocean.

11.5 Conclusion

Net sediment transport at the West coast is directed to the North, however storms cause a shift in transport to the South. Inside the harbour bay, set-up differences and the tide account for the sediment transport. The input of sediment in the harbour bay has two origins. The sediment that by passes the northern outcrop and is trapped in the harbour bay due to the breakwater. Secondly, the sediment that is flushed from the river mouth during wet season and deposited in the deeper parts of the bay. For a possible design, sediment bypassing the northern outcrop and sediment flushed out of the river need to be taken into consideration.

The sum of different mechanisms lead to the present state. During the dry season, a sandbar is formed at the entrance of the harbour basin. During the rainy season the sediment if flushed seasonally by a high river discharge. Thus the basin is seasonally connected to the ocean.

12. Current State

In this chapter the current physical state of Dodanduwa fishery harbour will be described. First a timeline is shown were all major events over the past decade are summarized. After the timeline the factional data will be presented, closing with the goals of the initiated project by the Ministry of Fisheries.

12.1 Timeline

The development of the Dodanduwa harbour area has quite a history. The construction of a fishery harbour in Dodanduwa was initiated in 2002 (The Island, 2006). The first realized foundation works were washed away in the 2004 tsunami. In 2008 the United Nations Office for Project Services (UNOPS) initiated a tender for the rehabilitation of the harbour, funded by the Greek Hellenic Aid Scheme. A news article from 2009 states that the 1.7 million US dollar harbour was completed that year. A 100-meter breakwater and onshore facilities were constructed, helping the Dodanduwa fishery community (Daily News, 2009).

However, the newly constructed breakwater worsened the situation instead of improving it. The navigation inside the harbour area became less safe and the sedimentation increased. The situation escalated when a fisherman died due to the capsizing of his boat (ColomboPage News Desk, 2014). The fishermen protested and blocked a road with a boat. According to the fishermen the poor construction of the breakwater was the cause of the death. The Ministry took action with a new tender at the end of 2014. EML Consultants won the feasibility study and detailed design in 2016.

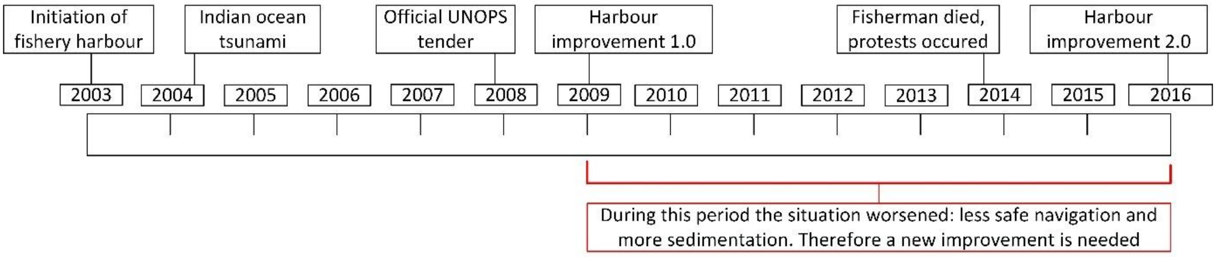


Figure 12-1: Timeline showing milestones during the Dodanduwa harbour development

12.2 Dodanduwa harbour and fishery industry

EML consultants made an inception report, delivered to the client June 2016, including the physical state of the Dodanduwa harbour. Summarized below are the main dimensions and quantities:

Table 12-1: Dimensions and quantities of all relevant facilities in the Dodanduwa harbour

Object	**Quantity**
Land area	1.10 Ha
Basin area	1.40 Ha
Breakwater length	273.0 m *
Dredging depth	3.0 m
Berthing capacity (No. of vessels 3.5-5 tonnage)	50
Market area (auction hall)	186 m^2
Net mending area	95 m^2
Water storage	14,000 l
Kerosene storage	36,000 l

*- only partly done, about 100 meters realized

Stakeholder interviews by EML Consultants concluded that the budget should be allocated to improve the facilities in the harbour basin and the connection to the ocean. The facilities that are available onshore are satisfactory according to the fishery organisations (EML Consultants, 2016).

In the table below an overview of the registered fishing boats is provided, these numbers were provided by the Galle District Office, MFARD.

Table 12-2: An overview of the fishery sector in Dodanduwa: number of people involved and boats used

Marine fishery data	Amount
Number of fisherman	4400
Number of fisher families	6825
Population of the fishery sector	19360
Number of fishery organisations	6 (3 relevant)
Off-shore multi-day boats (IMUL)	36
Single day boats (IDAY)	336 total
OFRP	33
MTRB	214
NTRB	81
NBSB	8

12.3 Proposed project

The Ministry of Fishery wants to improve the harbour and the surrounding area. The Project Implementing and Monitoring Unit (PI&MU) of the Ministry of Fisheries formulated the following goals (EML Consultants, 2016):

- Safe navigation for different types of fishing vessels operating from the harbour locations
- Adequate harbour basin with sufficient depths
- Improved landing facilities
- Access channels to the harbour with sufficient depth and facilitating safe navigation
- Elimination or reduction of sand accretion at the lagoon mouth and ensuring access to the lagoon basin; considering both the current fleet and the expected future fleet over a 10-year period

Next to the formulated goals the Ministry formulated a list of objects and works that, minimally, are expected to be included in the detailed design:

- Rubble mound structures for Northern Breakwater
- Expansion of harbour basin with the width of the entrance being 70m MSL and a minimum depth of -3.0m MSL
- Quay wall for lagoon bank with sufficient depth for small fishing craft
- Navigation lights at the harbour entrance and for access channel
- Entrance access channels with a depth of up to -3.0m MSL contour line towards the sea

The current project performed by EML consultants is still in an early stage. In this stage decisions have to be made which have great influence on functionality and costs of the improved harbour as well on the environmental impact.

12.4 Conclusion

Physical boundaries are formed by the current breakwater, rock outcrops and harbour basin area. The demanded minimal dredging depth of 3.0 meters states another physical requirement. Based on a process meeting with C. Fernando from EML, the expected future fleet consists of 100 vallams and 80 to 100 multiday vessels (Appendix P.3). Not all registered single day boats are operated from out Dodanduwa, or even used at all.

The Ministry of Fisheries has already initiated an improvement of the harbour area with a list of goals and minimally required structures. For this report to be valuable these requirements should be taken into account if proven relevant after the preliminary study.

The preliminary material availability investigation shows no urgent limitations. Concrete, steel, wood and armour stone are available in the region. Some minor limitations or details to be dealt will be mentioned. For more details, see Appendix D. A more detailed quarry study is required when armour stone is demanded in the project. The national steel industry will probably reach capacity in the near future, though issues are not expected concerning the probable size of the structures needed. Sri Lanka imports most of its cement, this is likely to be more expensive and causes a possibly longer delivery time.

The local infrastructure is likely to be sufficient for the project, the Galle-Colombo road and Galle harbour provide enough connections to the region, both locally and internationally.

13. IMPACT ANALYSIS

In this chapter the impact of the harbour development is described. The analysis is performed on both environmental- and social impacts, after which the impact on neighbouring harbours will be discussed.

13.1 ENVIRONMENTAL- AND SOCIAL IMPACT

In this section the environmental and social impact of the new harbour design will be discussed. The environmental impact is divided into two parts, the first part elaborating on the impact of the new design after construction has finished, the second part focussing on the environmental impact during construction.

Environmental impact in operational phase

In this subsection the environmental impact of the new harbour design during the operational phase is discussed.

Coastal erosion and accretion

The Dodanduwa fishery harbour is located near the coast, just upstream of the mouth that connects the Ratgama lake to the Indian Ocean. A new harbour design can cause the sediment transport surrounding the harbour area to change. Thus influencing the erosion/accretion pattern around the harbour and bay.

The impact of these changes on the environment and surroundings must be studied. This can be done with scale or numerical models. Severe erosion/accretion must be mitigated but preferably avoided.

Transition of traffic

Changing the harbour facilities will cause a change in traffic patterns. In the current situation fishermen from Dodanduwa are also working in other cities with harbours, i.e. Galle. These harbours facilitate multiday fishing boats. Designing a harbour in Dodanduwa which enables bigger ships to anchor, possibly attract local fishermen to work in Dodanduwa instead of Galle. This new situation might lead to a small decrease in work related traffic between Dodanduwa and the neighbouring cities.

Improving the harbour might cause an increase in efficiency and productivity, leading to an increase in fish leaving the harbour. In the current situation the disposal of fish takes mostly place on the local market, while the other part is moved to the market in other cities. Hence, the traffic of fish in the direction of other cities might increase.

Increase in fishing activities

When the harbour is improved to a certain extend it is possible that more fishermen will use the Dodanduwa harbour as anchorage. Fishermen from nearby cities may prefer the Dodanduwa fishery harbour above their own city harbour causing the fishing community to grow. The increase in fishing activity around the harbour can, in the long run, result in overfishing of that part of the ocean. Following the 'Fishing down the food chain' principle, this first leads to an increase in catches; then a phase transition will start in which the catch is stagnating and in the end it starts declining (D. Pauly, 1998). Fishermen will have to move out further away from the harbour to find fish; Costing more fuel and time, thus losing profit. Besides that is the fishing industry if vital importance for namely the people who live in coastal areas, a sustainable way of fishing is thus desirable.

Saltwater intrusion

The improvements to the harbour as described in the inception report include the permanent opening of the Ratgama Lake estuary towards the harbour basin. Because the salinity is a complicated issue, its possible effects, stakeholders and eventual measures regarding this permanent connection between ocean and the fresh water reserve, can be comprehensively found in Appendix E.

Environmental impact during construction

In this subsection the environmental impact of the new harbour design during the construction is discussed.

Constructing near the entrance of the harbour basin

The new harbour design might include an extension of the current breakwater. Extending the breakwater requires building equipment and a supply of materials near the entrance of the harbour basin. During the construction the harbour entrance possibly needs to be closed due to safety for the fishery boats. Closing the basin during this phase might disable the fisherman to use the harbour. As a result the fisherman needs temporary facilities or mooring places in order to continue their work or possibly need to be compensated for the days they cannot use the harbour.

Dredging activities
During the construction phase the harbour basin and the estuary at the mouth of the lake need to be dredged. The dredging activities, including the disposal of dredged material and the movement of the dredger itself have different impact on the environment. The transport of dredged material will limit the navigation and boat movements inside the harbour. The location at which the dredger will be active is unusable for berthing.

The disposal of dredged material is a main environmental issue. The disposal and location of disposal needs to be approved by the government.

Noise
Activities related to constructional work might exceed the maximum allowable noise limit for public areas. Since the rural location of the Dodanduwa harbour, the nuisance is believed to be limited. Residents living close to the harbour might need to be protected from the noise during construction, depending if it exceeds the prescribed limit.

Construction related traffic
Materials and equipment needed for the harbour have to be transported to the project site. The transport of heavy equipment and materials can cause nuisance when transported over the road. An option to lower this nuisance is, when possible, to use transport over sea. Another option is transportation of the materials during the night.

Air pollution
The heavy equipment used for the construction will cause an increase in emission of air pollutants. The impact of these emissions is believed to low, since there are only a few residents living nearby and the wind from the sea will disperse the air pollutants through the air, lowering the concentration considerably.

Harbour operations
As mentioned before, the harbour is during construction not or only partly serviceable for the fishermen. By staging the activities it might be possible to keep parts of the harbour open while others are being in construction. In case parts of the harbour remains open for fishermen the safety of the fishermen should be guaranteed all the time.

Even if the harbour if partly open for service, the construction will affect the harbour operations. For the fishermen of Dodanduwa this could mean they have to be moved towards a different harbour or temporary facilities have to be realised in order to ensure that the fishermen can continue with their work. In case none of the option is sufficient for the fishermen to continue their job, compensation needs to be offered.

Employment
During construction the development of the harbour will create labour for both skilled and unskilled labourers. The people living close to the project location might be suited for these jobs. Therefore, during the construction the project will temporary increase the employment opportunities in the surrounding area.

13.2 IMPACT ON NEIGHBOURING HARBOURS

The new design of the fishery harbour of Dodanduwa might influence the surrounding harbours or landing sites. Improving the conditions of the harbour is likely to attract more fishermen, the exact number of extra fishermen using the new harbour remains unknown.

There are nine fishery harbours distinguished in the Galle region, positioned at the following locations. The harbours of Hikkaduwa (North) and Galle (South) are located near Dodanduwa. The harbours in the Galle region, the distance towards Dodanduwa and the presence of anchor facilities for multiday vessels are listed in Table 13-1: Harbours of the Galle District.

Table 13-1: Harbours of the Galle District.

Harbour	Distance from Dodanduwa [km]	Anchorage facilities multiday vessels
Bentara	35	No
Balapitiya	24	No
Ambalangoda	18	Yes
Peraliya	9	No
Hikkaduwa	7	Yes
Dodanduwa	0	No*
Galle	16	Yes
Unawatuna	21	No
Kataluwa	30	No
***** Becomes possible after re-design of the harbour.		

Multiday vessels

The Hikkaduwa- and Galle harbours are the closest to the project site and both facilitate anchorage for multiday vessels. Fishermen prefer an anchorage close to home to limit the travel time towards work, see Appendix Q.2. Since the current harbour of Dodanduwa has no anchorage for multiday vessels, the fishermen working on these boats are forced to anchor in Galle or Hikkaduwa. A new harbour design will facilitate anchorage for these vessels, which is likely to attract multiday vessels owned by fishermen living close to Dodanduwa. The Hikkaduwa- and Galle harbour activities might therefore decrease. The other harbours are believed not to be affected by the new harbour design, considering the distance between those harbours in regard to the project site.

During the SE monsoon navigation in the Hikkaduwa harbour becomes challenging (Interview CFHC, Appendix Q.2). In the future the Dodanduwa harbour could be used for temporary anchorage when the Hikkaduwa harbour becomes to challenging to navigate in. During this period the new harbour is likely to be used for anchorage more often.

Vallams

Next to the multiday vessels, vallams are used by the fishermen. In most harbours the vallams use the beach as landing site, which will also be the case in the new harbour design. The fishermen using the vallams are most likely to choose a landing area close by home, decreasing the travel distance, see Appendix Q.2. Since the vallams don not require special anchorage the increase in vallams after redesigning the Dodanduwa harbour is believed to be small. Currently the bays located in the north and south are used for landing vallams, in case one of the bays will be redesigned it is likely that the vallams currently landing in this area will use the new harbour basin, therefore the concentration of boats landing in the harbour basin might increase, but the total amount of vallams is believed the remain the same.

13.3 Conclusion

The environmental impact is considered for two phases, during construction and after completion of the project. During construction the impact on the direct environment is believed to be moderate, since the small amount of residents near the construction site. The transport of materials could have a negative impact, transport by sea might avoid this.

After construction the environmental impact includes saltwater intrusion, coastal erosion and accretion, transition of traffic and an increase in fishing activities. The impact of these are difficult to predict, but possible effects have been elaborated. The saltwater intrusion might occur when the new design ensures an open connection between the Indian ocean and the Ratgama lake all-year. The effects of this might be severe, extra attention to this phenomena is therefore prescribed.

Besides environmental impacts, it is believed that some of the multiday vessels that currently anchor in Galle or Hikkaduwa, will move to the new Dodanduwa harbour. Furthermore during bad weather conditions, multiday vessels from Hikkaduwa are expected to come to Dodanduwa due to the safer anchoring conditions. The change in number of vallams is expected to be minimal.

14. STAKEHOLDER ANALYSIS

In this paragraph the stakeholders for this project will discussed. An explanation of the stakeholders, their interests and their involvement can be found in Appendix F. This stakeholder analysis has partially been made with the use of the inception report made by EML consultants.

14.1 INTEREST VS. POWER

As can be seen in Figure 14-1 is the PI&MU the most powerful party, which makes sense since this is the initiator of the project. Other parties which are powerful are the CC&CRMD, CFHC, ID&ASD and EML. Except for EML, the organizations are parts of different ministries. The new design needs to be approved by these parties before the construction can start. EML is considered as being powerful as well, since this party is the consultant of this project and therefore works together with the PI & MU.

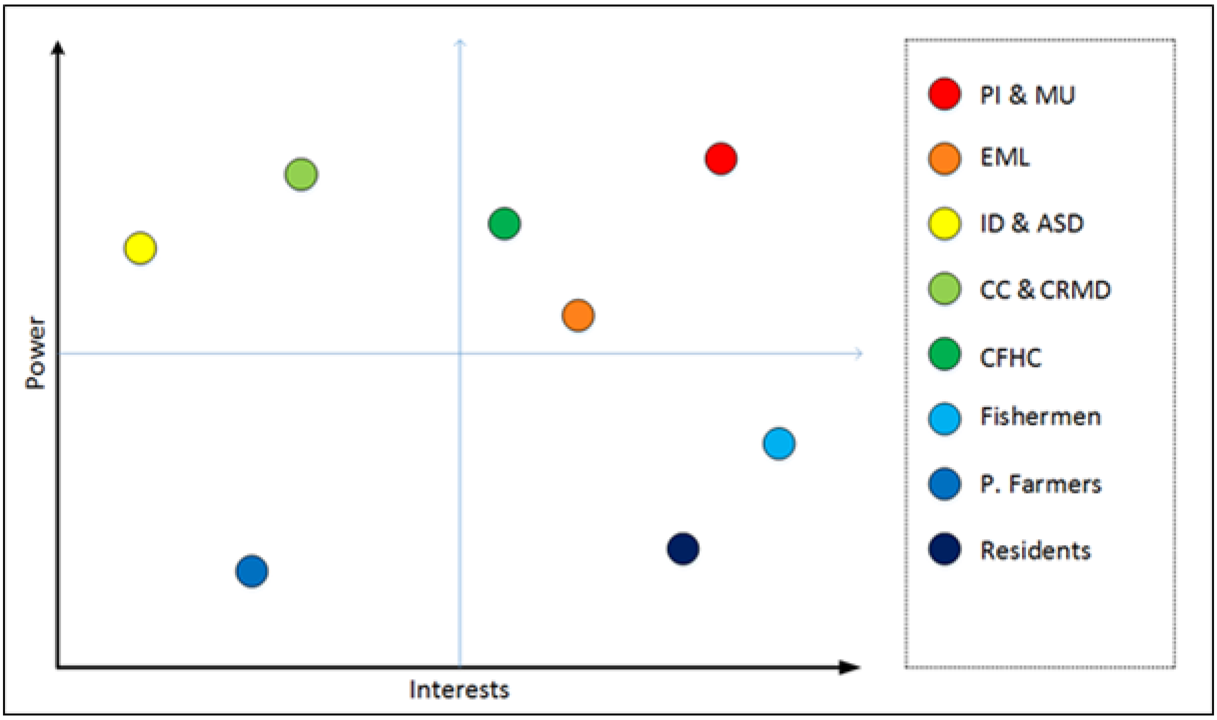


Figure 14-1: Interest vs power per stakeholder

The local residents and fisherman are the groups with the most interest but with moderate power, because they will use the harbour or are affected by it on a daily basis. The power of this group is considered to be moderate, since they made their interests clear but they don't have any power in the course of the project or the approval of the designs.

The paddy farmers are placed in the low power and low interest section, because it is not sure how the new design will affects this group.

The structure in which the different stakeholders are connected to one another is visualized in Figure 14-2. The black lines visualize a direct link between the different stakeholders. The stakeholder meetings which are planned every few weeks makes it possible for more stakeholders to meet and discuss their concerns and opinions.

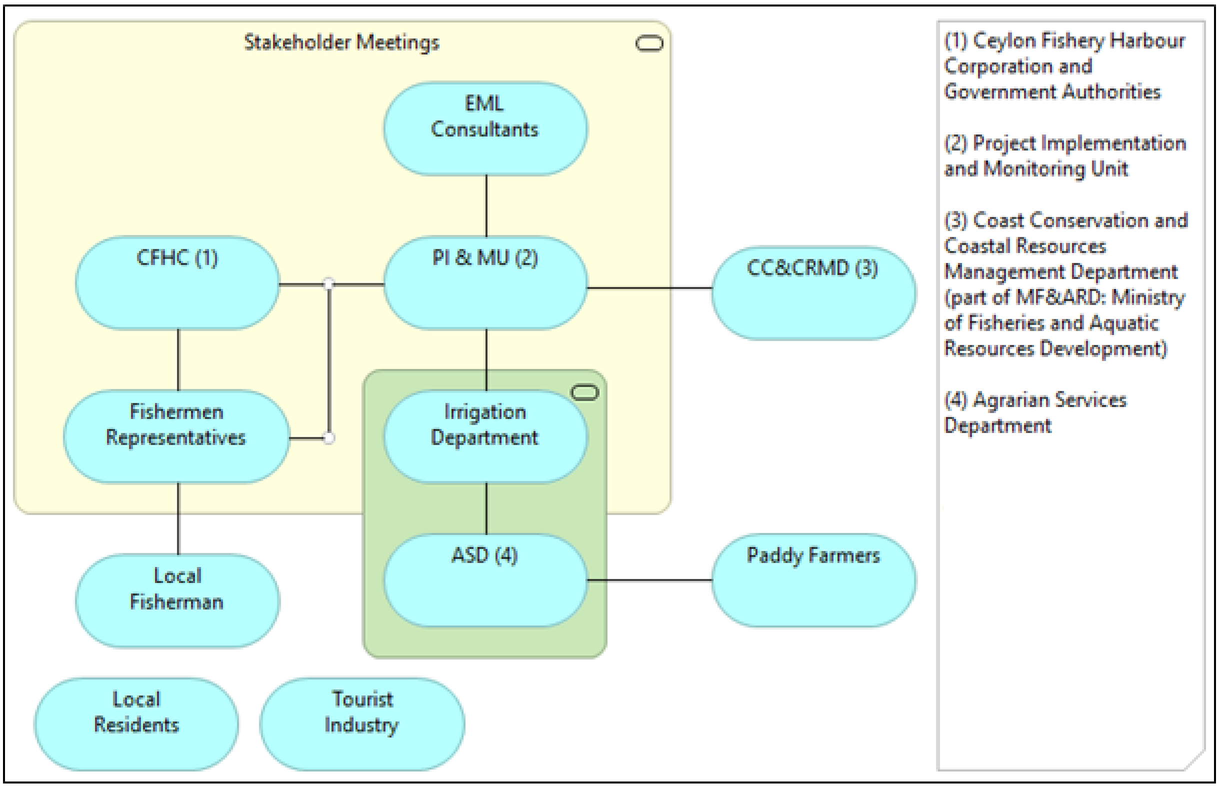


Figure 14-2: Schematized diagram of connections between stakeholders

For some of the stakeholders it is not clear how they are connected to the project. They are believed to influence or be influenced by the project, but it remains unclear how this is related to other stakeholders.

The Irrigation Department (ID) and the Agrarian Service Department (ASD) are more or less connected to each other since they both have interests in the local agriculture. Both are believed to not directly be affected by the project, but their consent with the project is mandatory. The interests of the paddy farmers are represented by the ASD and the ID.

14.2 Conclusion

The interests and influences of the different stakeholders involved in the project have been analysed. The governmental organisations are believed to have the greatest power in the project, CFHC and PI&MU. Those parties can both be seen as the client and are proactively involved. Other governmental stakeholders like the CC&CRMD and the ID&ASD have to decide whether the design meets the regulations, which gives them power as well. Since EML consultants have the lead in the design and are the party with the specific knowledge, their advice is powerful.

The actual users and people affected most by the new harbour design have only a small influence in the project. They do have representatives attending the meetings in which they share their interests, but they do not hold the official power to stop the project and don't have the technical knowledge to contribute much to the design. But, as can be seen in Appendix Q.2, the opinion of fishermen communities are of great importance for the CFHC projects.

During the project multiple stakeholder meetings are planned by the project consultant. In these meetings the different stakeholders can share information and concerns they have regarding the project

15. CONCLUSION PRELIMINARY STUDY

In this chapter, a final conclusion on the preliminary study is presented. Except for the last question 'Which requirements should be considered when designing a harbour improvement?', all sub questions were answered. This fifth question will be answered in section 16.3. A visualized overview of the preliminary study can be found in Figure 15-1.

The rocky outcrops near the harbour of Dodanduwa consist mainly out of gneiss. Closer to the ocean, more recent deposits of sand can be found, a less troublesome subsoil thus. The expected geological process in this area is sediment transport, which shapes the coastline. Identified geohazards are underwater slope stability of loose sand deposits, tsunami's and earthquakes.

Sri Lanka's coast is exposed to swell – and wind waves. The swell waves are dominant most time of the year and are coming from SW and SSW. Overall the dominant wind direction is WSW. High swell waves make it hard for the fishermen to enter or leave the harbour. Anchorage facilities are not present nowadays but would make life a lot easier for the fishermen.

Sediment is transported by different mechanisms. Net sediment transport is directed northwards. The sandbar formed at the entrance of the basin, is mainly created due to diffracted waves. The resulting set-up differences drive a current which transports the sediment towards the mouth. The tidal range along the coast of Sri Lanka is low. Although the flood tide transports the sediment more upstream in the basin.

Physical boundaries are formed by the current breakwater, the rock outcrops and the present basin area which is not accessible throughout the year at the moment. The Ministry of Fisheries has already set a list of goals and minimally required structures. Study shows no urgent material limitations, besides the local infrastructure is likely to be sufficient for the project.

During construction the present basin will probably be closed off to make the existing breakwater accessible for equipment. Besides that will dredging works hinder navigability in the bay. After-construction impacts includes saltwater intrusion, coastal erosion and accretion, transition of traffic and an increase in fishing activities. Especially salt water intrusion and coastal erosion should get extra attention. Probably multiday vessels which currently anchor at other harbour will move to the new Dodanduwa harbour. The change in vallams is expected to be minimal on the short term.

The most important stakeholders are the PI&MU and the CFHC, these stakeholders should be managed closely. From the perspective of this report EML is also part of this group. The regulatory organisations ID&ASD and the CC&CRMD should be satisfied by meeting the standards and norms for coast protection and irrigation. Fishermen and residents must be properly informed about the status of the project.

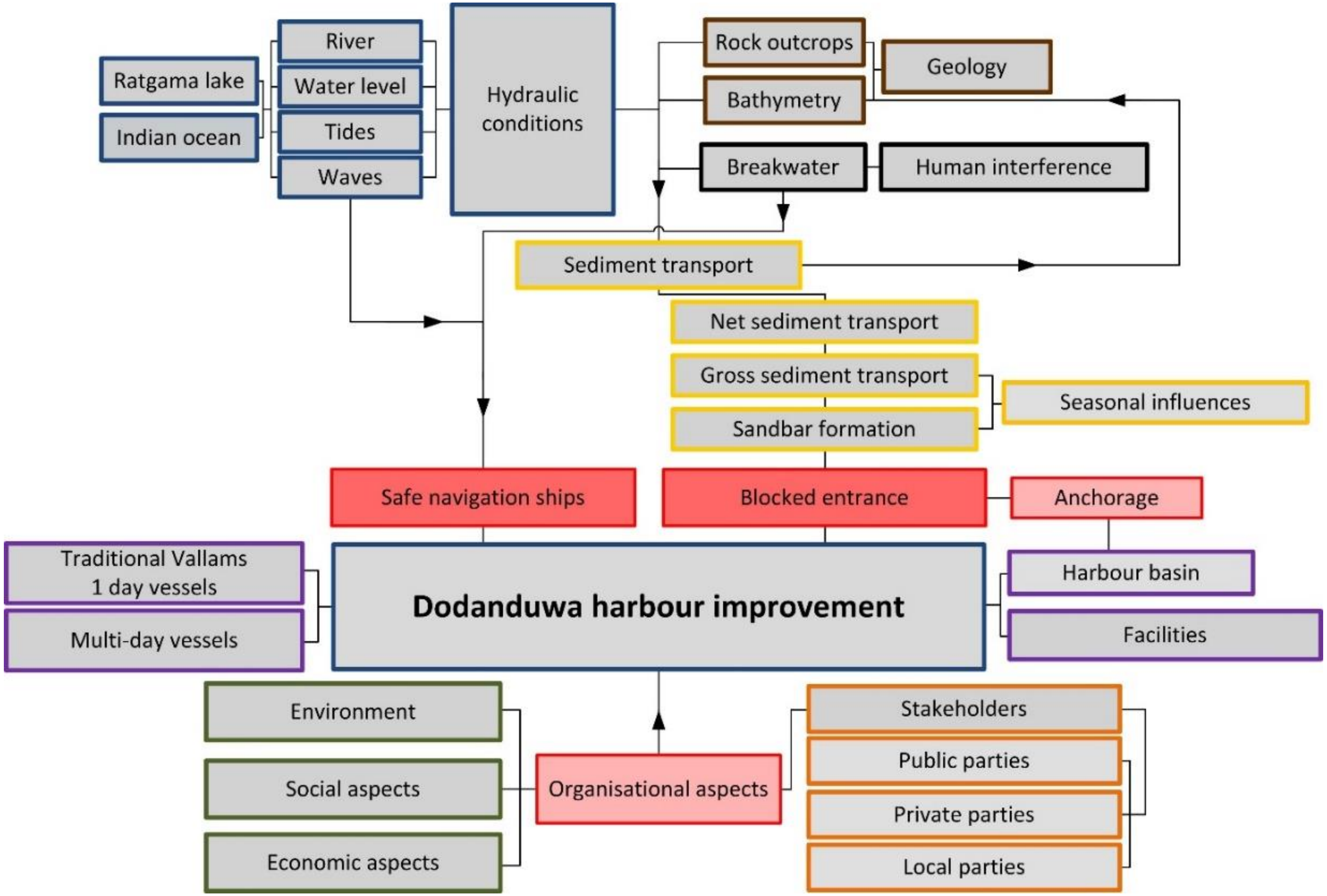


Figure 15-1: Overview scheme of preliminary study

CONCEPTUAL DESIGN

16. DESIGN APPROACH

In the design phase information of the preliminary study is used to develop several conceptual solutions for the identified problems. Since the project is still in the conceptual design phase, a broad range of concepts will be presented. These concepts will be visualized, elaborated and summarized in this chapter.

As mentioned in the preliminary study, the feasibility study of EML focusses on improvement of the current harbour area. EML makes a distinction between lagoon development and sea front development. There is a clear list of demands from the Ministry of Fisheries, and stakeholder meetings gave insights about the wishes of the fishermen. In this report the issue is approached from a functional point of view. Two main functions of the harbour that must be facilitated in the conceptual design phase are:

- Sufficient and all-year accessible anchorage capacity
- Providing safe navigation to and inside the basin

After summarizing the current issues for the two main functions a list of all constraints, requirements and wishes that have to be taken into account while providing for the two main functions is given. In chapter 17 then, the conceptual designs are presented.

16.1 ANCHORAGE CAPACITY

The current basin in the lagoon can be used to create anchorage, this is suggested by EML consultants. Project Dodanduwa is also including other possible basin locations. These other locations are chosen in the surroundings of the current onshore harbour facilities. Since the project goal is helping the Dodanduwa fishery community a whole new location is not considered. Three locations are considered, each with their own challenges. These different locations and challenges are shown Figure 16-1.

Another unresolved issue is known: keeping the current basin accessible all year possibly results in more salt water intrusion in the river and lake. Separating the basin and the river will not create this problem. Moreover, the capacity of the current basin is known to be insufficient for the number of registered and active fishermen. The capacity of the other locations that are considered are still to be researched.

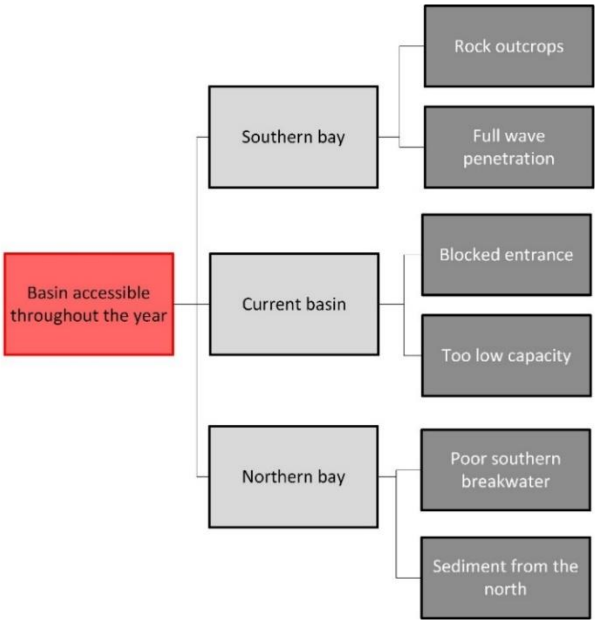


Figure 16-1: An overview of the three considered options and their challenges

16.2 Safe navigation

Currently, high waves make safe navigation in the harbour bay impossible for most of the time. The partially built breakwater only blocks part of the waves. Fishermen have to go through the breaker zone navigating against shoaling waves. This makes navigation out of the harbor bay difficult and unsafe. Besides the partially constructed breakwater, the lack of lighting results in poor visibility at night. Furthermore, fishermen stated that the partially built breakwater makes it more difficult for them to see the incoming waves. It is therefore harder to leave the harbour at the right time.

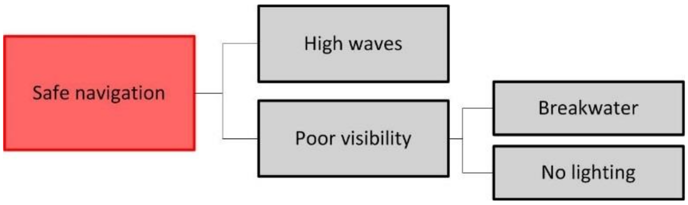


Figure 16-2: An overview of the navigation issues and the causes.

16.3 Constraints and Requirements

In this paragraph an overview is shown of all constraints, requirements and wishes derived from the preliminary study. Most of the requirements are stakeholder related. This chapter start with a list of the constraints, imposed by the location itself, after which the requirements are discussed, followed by desires and made assumptions.

Constraints

The project location on its own is imposing limitations to the possibilities of the new harbour design. The identified constraints of the project location are listed below:

- Location of the current onshore harbour facilities is fixed.
- The use of the river as a basin is constrained by the A2 road bridge.
- Rock outcrops form hard boundaries along the coastline. Removal is possible but at high cost.
- Reclamation of land near the harbour basin is possible, but a buy-out of the current owners is needed.
- The current breakwater can be extended but removal is not inconvenient.

Requirements

From the Ministry of Fisheries and Aquatic Resources Development and EML Consultants the following requirements are obtained:

- Safe navigation for the different types of fishing vessels operating in the harbour
- Lights, at least operational during the night or during times of poor visibility
- Jetties or quay wall along the harbour basin for refuelling and unloading of fish.
- Waves with a maximum height of 0.3 meter at the (un)loading facilities in order to ensure safe operation; maximum exceedance probability once per year.
- A channel with sufficient depth and width for 2 multiday fishing vessels.
- Depth of at least 3.0 meters in the harbour basin and navigation channels.
- Accretion of sand near the entrance of the basin and in the access channel is allowed. But dredging frequency may not exceed the once in 5 year frequency policy of the CFHC.
- Anchorage facilities for 80-100 multiday fishing vessels (based on expected 10-year growth)
- Enough landing space, ± 100 vallams.

Requirements were obtained from an inception report (EML Consultants, 2016), meetings with C. Fernando (Appendix 0) and an interview with the general manager from the CFHC (Appendix Q.2).

Desires

The desires are all covered in the requirements listed above. Main desires were improved landing facilities and a calmer bay to make navigation safer and landing easier.

Assumptions

- The amount of vallams in the harbour will remain the same, even with an increase in multiday vessels (Interview CFHC, Appendix Q.2)
- Vessels will be moored side by side in order to provide more berthing places on jetties. (Appendix G.1)
- Vallams have average dimensions of:
    - Length: 10.0 meter
    - Width: 3.5 meter
    - Draught: 0.4 meter

  (Google Maps, 2016)
- Multiday vessels have average dimensions of:
    - Length: 12.0 meter
    - Width: 4.0 meter
    - Draught: 1.6 meter

  (Google Maps, 2016); (NMDF-40 Multi Day Fishing Boat)
- There are no geohazards for which have to be designed. (Paragraph 9.4)
- Subsoil conditions are in general good. (Paragraph 9.4)
- Dominant wave direction is WSW (Paragraph 10.4)
- Sediments come from the North and from the Ratgama river. (Paragraph 11.5)
- No direct limitations in building material availability are present. (Paragraph 13.3)

After gathering the requirements, limitations and stating the main functions which the harbour should facilitate, the concepts can be developed which is done in the next chapter.

17. CONCEPTS

Based on the constraints and requirements presented in the previous chapter and brainstorm sessions on separate solutions for the different issues in the harbour. Five concepts for the harbour design were derived. Four of the concepts satisfy the requirements and main functions, the fifth partly neglects this and solves the main issues encountered by fishermen.

In the next paragraphs, the ideas behind each concept are explained after which the concepts are presented in sections 17.1 to 17.5. Additional information about the designs can be found in Appendix G, for example process analysis or harbour lighting.

Concept 1: 'Optimized Current Situation'
The first concept is basically a completion of the former plans for the Dodanduwa harbour. The northern bay and the basin are used as harbour area. Both are connected via the Ratgama river.

Concept 2: 'Northern Solution'
The idea behind the second concept was to take the river, which makes the basin seasonally inaccessible, out of the problem area. The river is diverted in a way where it does not longer influences the harbour area and the processes going on inside it, and the basin is directly connected to the northern bay.

Concept 3: 'Southern Solution'
In the 'Southern Solution', the option to use the southern bay as a harbour area instead of the maybe convenient northern bay. This is done in combination with the basin to have enough anchorage capacity. The river is diverted to make a connection between the basin and bay.

Concept 4: 'Eyes on the Future'
This concept is based on a huge growth of the fishing activities in the Dodanduwa harbour. To arrange enough anchorage for a big amount of multiday vessels, both northern and southern bays are used for anchorage, and the basin for the unloading facilities. The river connects both bays with each other's and the basin.

Concept 5: 'Null-option'
An interview with a fisherman is the basis for the fifth concept. He said that if they only could see the waves coming, they could anticipate on them and sail out easily. A tall buoy behind the breakwater makes anticipation to the waves possible and makes the conditions much safer. Furthermore, a sediment trap should keep the basin accessible all-year long.

17.1 Concept 1: 'Optimized Current Situation'

The first conceptual design is focussed on the improvement of the current harbour layout. The northern harbour bay facilitates the anchorage of the multiday vessels while the old basin is adjusted and used for unloading and refuelling of the boats. This concept is visualized in Figure 17-1.

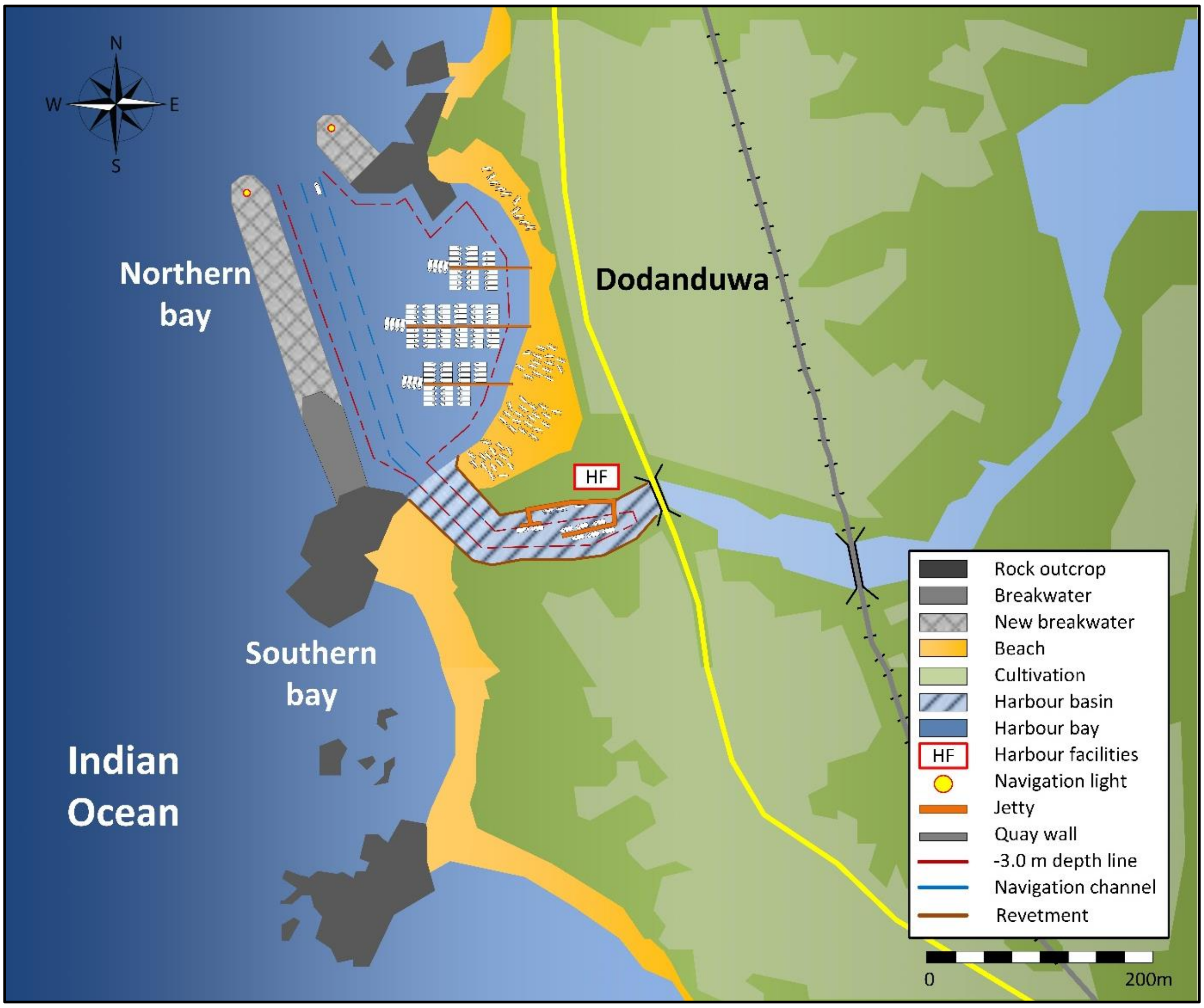


Figure 17-1: Lay-out of the 'Optimized Current Situation' conceptual design.

Fish unloading and refuelling structure

Fishing boats will enter the harbour at the northern part of the bay. The fishermen can unload their fish and refuel their boats at the harbour basin. The jetty located at the basin is constructed for refuelling and unloading of fish catch, materials and equipment of the boats. Six multi-day fishing vessels can simultaneously unload their fish, vallams can use the part of the jetty at the embankment as vallams do not require a large draught. After unloading, all boats can use the western side of the jetty for refuelling water, ice and gasoline. A different jetty for refuelling is chosen for safety and hygienic reasons.

In this concept a separate jetty is chosen for unloading and refuelling. There is enough space in the basin to construct a jetty where multiday vessels can manoeuvre freely. A jetty is a structure which is easy to construct and requires almost no groundwork. Another option is to construct a quay wall. A quay wall requires more groundwork and skilled labour to place the gabions. Furthermore, the quay wall needs finger jetties for the multi-day fishing vessels to moor.

More information about jetty design can be found in Appendix K.

Anchorage

After unloading and refuelling boats can anchor at the jetties in the northern bay, vallams will still land on the beach. Three jetties are constructed to meet the capacity requirement. The theoretical capacity of these three jetties is 110 vessels. In practice there will be less space due slightly less structured mooring of the vessels. This results in space for about 90 vessels at the jetties and approximately 100 vallams can land on the beach. Jetties are chosen for the anchorage because this will give a structured anchorage. Also there is no need for small boats to come to shore. It was also possible to have anchorage in the middle of the bay, but this would result in a chaotic situation.

Breakwater

The current breakwater in the northern bay is extended. This extension prevents waves entering the bay from the most dominant directions; thus, decreasing the amount of waves entering the bay.

A second breakwater is placed at the northern outcrop. The orientation of this breakwater is towards the north-east. This will trap the longshore sediment coming from the north, outside the harbour bay. See Figure 17-1 for the orientation and location of the breakwaters. This breakwater design creates a safe transition from bay to the sea.

More information about the breakwater can be found in Appendix N

Future sedimentation

As said in the breakwater section, the breakwater at the northern rock outcrop will block the longshore sediment transport from the north. The southern breakwater is extended to the same height as the northern rock outcrop. These breakwaters will prevent most of the sedimentation.

More information about future sedimentation can be found in Appendix I.1

Revetment

Ships entering and leaving the harbour basin will induce currents and waves. To prevent erosion and slope instability on the embankments a revetments will be constructed.

More information about the revetment can be found in Appendix L.

Work estimation

The estimated amount of structures and groundworks are presented in Table 17-1.

Table 17-1: Crude estimation of the extent of the works

Structure/works	Amount	Dimension
Breakwater	350	[m]
Dredging	1.0	[hectare]
Revetment	350	[m]
Jetties	430	[m]

17.2 CONCEPT 2: 'NORTHERN SOLUTION'

In the second conceptual design, the river is diverted to the south as was the case before the construction of the current breakwater. This separates harbour and river; seasonal closure of the river does not interfere with the navigability of the harbour anymore. The harbour basin will be extended to ensure a loading and unloading area. The current breakwater is extended to protect the harbour from waves and to ensure safe navigability in and out of the harbour. A small breakwater is constructed at the northern outcrop to block sediment coming from the north, thus decreasing the sedimentation in the harbour bay. This concept is visualized in Figure 17-2.

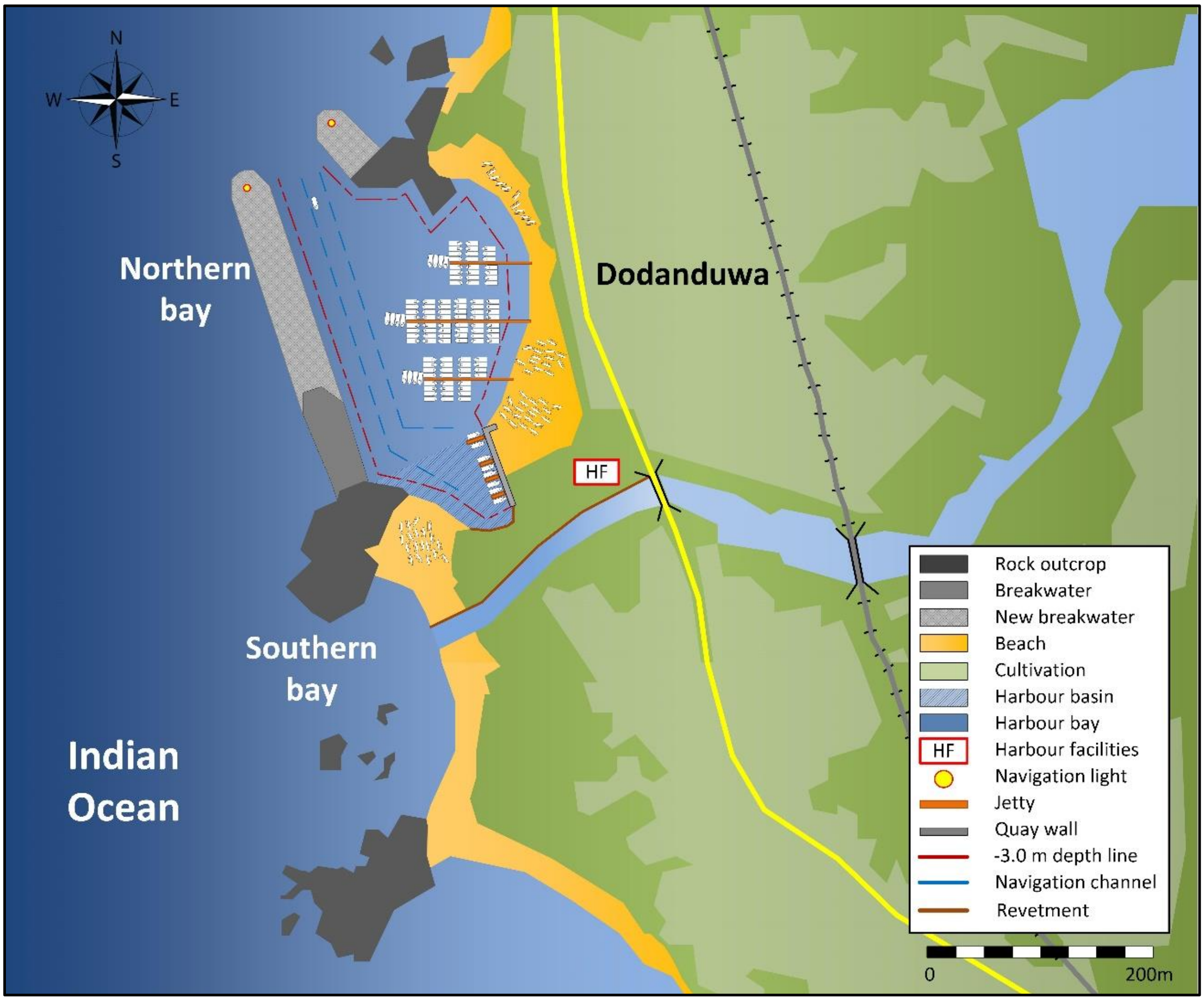


Figure 17-2: Lay-out of the 'Northern Solution' conceptual design.

Fish unloading and refuelling structure

Fishing boats will enter the harbour from the northern side of the bay. In the harbour basin, fishermen can unload their boats and refuel. A quay wall with finger jetties is constructed for the unloading and refuelling of the boats. A quay is chosen due to the limited space. When constructing jetties, a slope is needed just like in conceptual design 1. This would increase the amount of space needed, which is scarce, and require more dredge works. Secondly, there needs to be enough land between the harbour basin and diverted river. As this part will be used as pathway and used as buffer to prevent breaching. However, breaching is not likely due to the construction of a revetment to prevent erosion of the river bank.

Multiday vessels and vallams can moor at the finger jetties to unload their fish. Three jetties will facilitate unloading spots for six vessels. The finger jetty on the northern side of the quay wall will facilitate refuelling for all boats. Again, unloading and refuelling is separated for safety and hygienic reasons.

More information about quay wall design can be found in Appendix J

Anchorage

After unloading and refuelling boats can anchor at the jetties in the northern bay, Vallams will still land on the beach or at the jetty in the shallow parts (-3.0 to 0.0 m). Three jetties are constructed to meet the capacity requirement. The theoretical capacity of these three jetties is 110 vessels. In practice there will be space for 90 vessels at the jetties, due slightly less structured mooring of the vessels. There is space for approximately 100 vallams to land on the beach. Jetties are preferred above an anchorage in the middle of the bay. Jetties give a more structured mooring and more efficient use of space than an anchorage.

More information about jetty design can be found in Appendix K.

Breakwater

The current breakwater in the northern bay is extended. This extension prevents waves entering the bay from the most directions; thus, decreasing the amount waves entering the bay.

A second breakwater is built at the northern outcrop. The function of this breakwater is to trap the sediment north of the bay, thus stopping the bypassing of sediment around the northern outcrop. See Figure 17-2 for the orientation and location of the breakwaters. This breakwater design creates a safe transition from bay to ocean.

More information about the breakwater can be found in Appendix N

Future sedimentation

A new coastal cell is created due to the construction of the northern breakwater and extension of the current breakwater. Sediment from the north will be trapped north of the bay. Furthermore, the forcing that would transport sediment in the bay has been reduced significantly. Maintenance dredging is still needed, but in not expected to exceed the once in five years policy of the CHFC.

By diverting the river to the south it is assumed that the seasonal variability of the river will continue.

More information about future sedimentation can be found in Appendix I.2

Revetment

By diverting the river to the south, it must be assured that the river does not erode in the outer bend. This can lead to erosion or even a breach. The outer bend of the river must therefore be prevented from erosion. This is done by placing a revetment.

The increase in vessels also lead to attack of the banks in the harbour basin. To decrease erosion and ensure safety against breaching a revetment is placed there as well.

More information about the revetment can be found in Appendix L.

River diversion

In this concept the lagoon river does not discharge water in the harbour bay. The river is diverted towards the southern bay. The river mouth will be open and close seasonally. Benefit of the diversion is that there is no change in salinity upstream.

More information about the river diversion can be found in Appendix H.

Work estimation

The estimated amount of structures and groundworks are presented in Table 17-2.

Table 17-2: Crude estimation of the extent of works

Structure/works	Amount	Dimension
Breakwater	350	[m]
Dredging	2.0	[hectare]
Claimed land	1.5	[hectare]
Land reclamation	1.0	[hectare]
Jetties	330	[m]
Revetment	300	[m]
Quay wall	70	[m]

17.3 Concept 3: 'Southern Solution'

The third conceptual design makes use of the southern bay as harbour basin, together with an expanded lagoon basin. The river which connects the Ratgama Lake with the Indian Ocean is diverted towards the southern bay. Multiday vessels can anchor in the harbour bay and the harbour basin. This concept is visualized in Figure 17-3

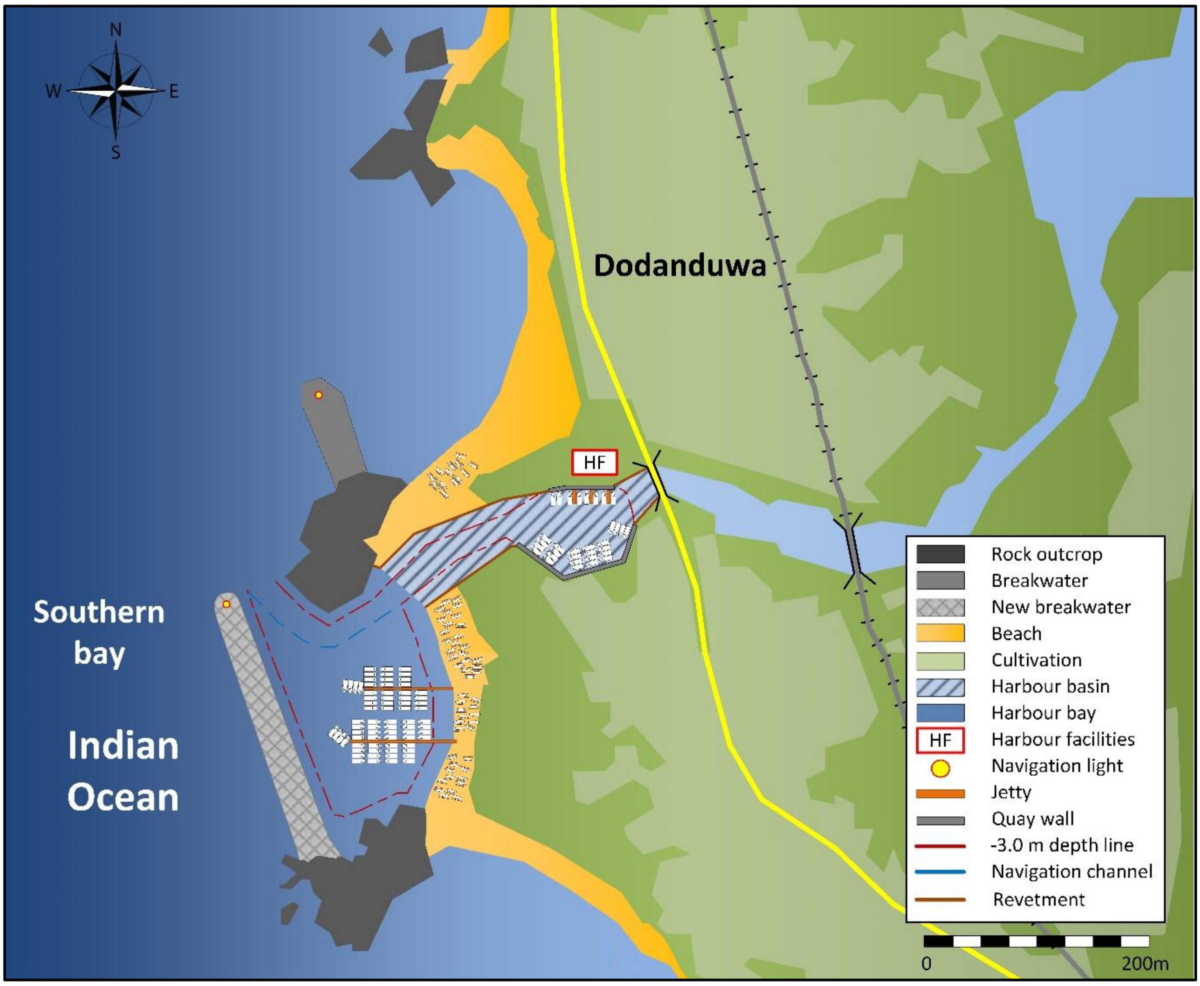


Figure 17-3: Lay-out of the 'Southern Solution' conceptual design.

Fish unloading and refuelling structures

Fishing boats will enter the harbour from the north side of the southern bay. The breakwater provides a safe entrance into the bay. The harbour facility is located near the harbour basin, which is accessible from the river. The river is diverted towards the South. A quay wall with finger jetties at the harbour facility will provide berthing for multiday vessels. Vessels can unload their catch at the finger jetties.

A quay wall design is chosen because of the limited space in the harbour basin. Another quay wall at the southern bank of the basin is needed to provide enough capacity for anchorage. A jetty design would be possible if the southern bank of the harbour basin would be excavated further. This would create more space for a jetty on the northern bank and a quay wall for anchorage on the southern bank. Excavating the southern bank further would result in expropriation of land owners and extra ground work. Therefore a quay wall design is chosen.

The quay wall with finger jetties will provide six anchorage places for vessels to unload their fish. On the west side of the quay wall, vessels can moor perpendicular to the quay for refuelling purposes. Again, this part is separated from the unloading area for safety and hygienic reasons.

More information about quay wall design can be found in Appendix J

Anchorage

Multiday vessels can moor at two location in this concept. Two locations are needed to satisfy the anchorage place requirement given by EML and CFHC. The first place where multiday vessels can moor is in the harbour bay, where two jetties with a theoretical capacity of 72 boats are placed. Other boats can moor at the southern bank of the harbour basin; here a quay wall provides anchorage for another 24 multiday vessels. In total about 96 multiday vessels can anchor in this concept.

Vallams must land on the beach; if the beach in the southern bay is stocked, vallams can land on the beach in the northern bay. This beach is still partially protected by the old breakwater.

More information about jetty design can be found in Appendix K.

Breakwater

To ensure calm water conditions inside the southern bay, a breakwater is constructed. It will be constructed from the southern rock outcrop towards northeast direction. This to prevent waves from the dominant wave direction entering the harbour basin, thus ensuring safe navigation in the harbour bay.

More information about the breakwater can be found in Appendix N.

Future sedimentation

Sediment does not bypass the southern and middle outcrops. Sediment in the northern bay will be trapped there as the old breakwater will not be removed and acts a trap. Sedimentation of the river mouth will not be an issue in this conceptual design. It is expected that the only source of sedimentation of the river mouth, are the tide and the little wave action still present in the harbour bay.

More information about future sedimentation can be found in Appendix I.3

Revetment

To prevent riverbank erosion from waves and currents generated by the multiday vessels, which have a much stronger engine than the vallams, revetments are placed alongside these banks. The revetment will prevent the river banks from eroding, thus maintaining the required draught. Secondly, a revetment makes sure the river is fixed and will not change over time.

More information about the revetment can be found in Appendix L.

River diversion

In this conceptual design, vessels can enter the river from a calm bay. Sedimentation of the river mouth is prevented, as hardly any sediment it brought into the coastal cell. This ensures access to the harbour facilities all-year long.

More information about the river diversion can be found in Appendix H

Work estimation

The estimated amount of structures and groundworks are presented in Table 17-3.

Table 17-3: Crude estimation of the extent of works

Structure/works	Amount	Dimension
Breakwater	350	[m]
Dredging	2.0	[hectare]
Claimed land	0.5	[hectare]
Land reclamation	0.7	[hectare]
Jetties	250	[m]
Revetment	350	[m]
Quay wall	160	[m]
Rock removal	200	[m3]

17.4 Concept 4: 'Eyes on the Future'

This conceptual design focus on a fast-growing fleet. Therefore, the northern- and southern bays are used as harbour basins in order to create enough space for this fleet. The harbour bays are protected from waves and sediment by two north-northwest pointing breakwaters while keeping both bays accessible. Besides that, the barrier between the harbour basin, northern- and southern bay is breached. The river discharges into both bays and harbour facilities are accessible from both bays. This concept is visualized in Figure 17-4.

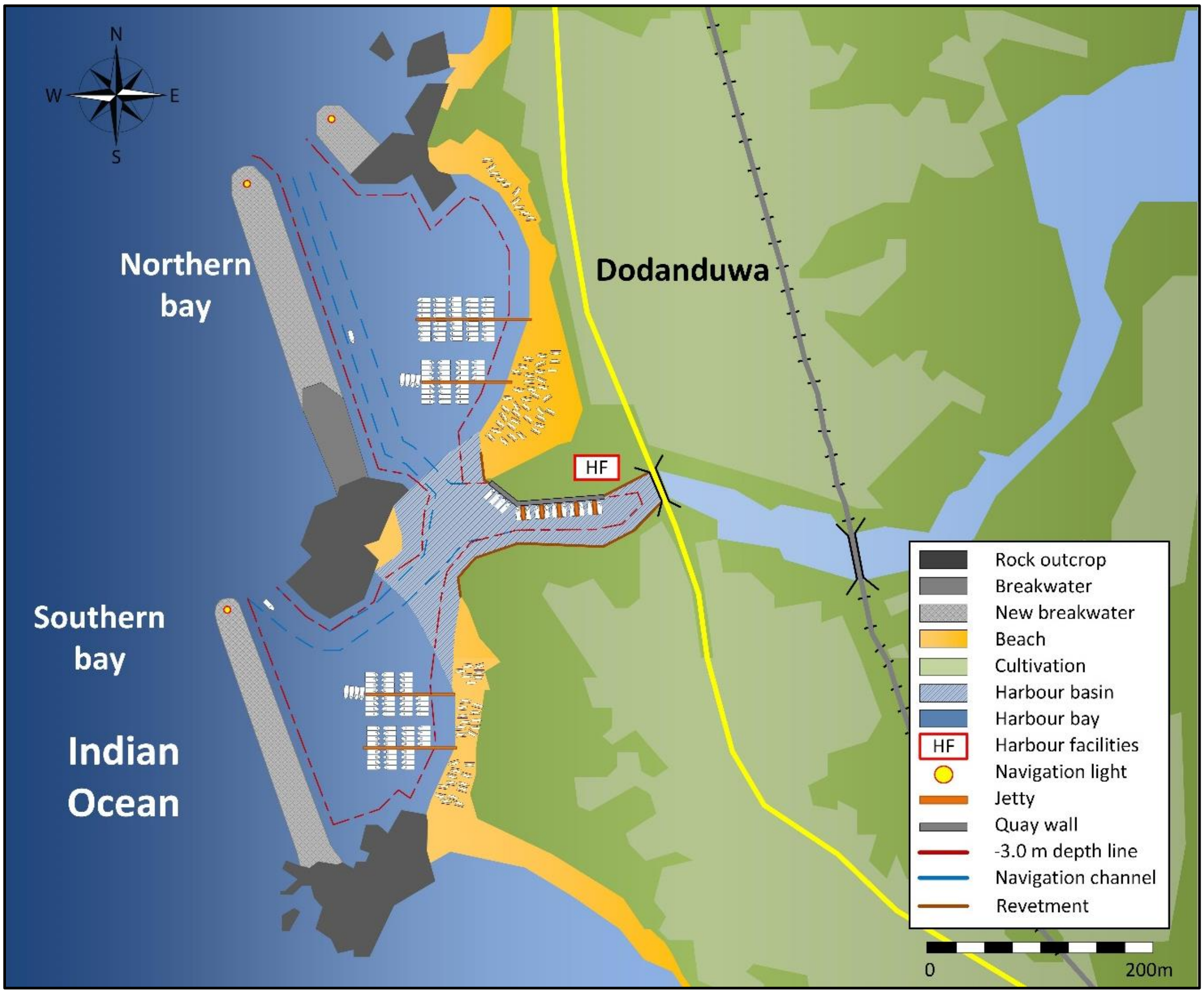


Figure 17-4: Lay-out of the 'Eyes on the Future' conceptual design.

Fish unloading and refuelling structures

The harbour basin is accessible from both the northern- and southern bay. The river mouth is dredged to make navigation between the bays and the basin possible. A quay wall with finger jetties is constructed at the harbour facility. This to facilitate unloading and refuelling. Vessels can moor at the finger jetties for unloading. There are five finger jetties to make unloading possible for ten vessels which is more than enough based on the process analysis, appendix G.1. On the western side of the quay vessels can moor for refuelling. Boats must moor perpendicular to the quay wall, in this way more boats can refuel at a time, with a maximum of four boats.

A quay wall design is chosen because of the space needed. The quay wall allows place for ten vessels at the same time. The space required for a jetty that can process ten vessels would extent towards the bridge, where no space is available. Furthermore the river must be widened for a jetty construction, again close to the bridge. Where this might cause instability of the bridge foundation.

If the capacity for unloading and refuelling is not sufficient, another quay wall can be constructed at the south side of the harbour basin. However, land must be reclaimed to create the space needed for this. Land owners must be compensated for this.

More information about quay wall design can be found in Appendix J

Anchorage

Boats can enter the harbour bay via a northern- and southern entrance. This to prevent situations where only one side of the harbour bay will be used, investigation of other harbours have proven this, i.e. Dikkowita. Otherwise one bay can be overcrowded with boats while the other, closed off bay, is empty. Still an option is to close of one of the bays, this option will be discussed in the next breakwater indention. This concept is designed on a fast-growing fleet. With the eyes on the future more multiday vessels need to anchor. Therefore two jetties are placed in the northern bay and two jetties in the southern bay. The jetties have in total a theoretical capacity of 132 multiday vessels. In practice this will be around 110 multiday vessels, due slightly less structured mooring of the vessels.

If the anchorage capacity is not sufficient, it is possible to build another jetty in the north side of the northern bay. Also a breakwater with quay wall can be constructed to facilitate anchorage.

More information about jetty design can be found in Appendix K.

Breakwater

In this design, the existing breakwater is extended to about the same height as the northern rock outcrop forming the first main breakwater. At the southern rock outcrop, a second breakwater is constructed with the same direction as the first one. These two main breakwaters prevent waves from the dominant direction from entering the bays and create a safe transition from ocean to bay and the other way around. A small breakwater is installed at the northern outcrop, this breakwater is oriented in the north-northwest direction. That way, it acts like a sediment trap for sediment coming from the north.

Another option for the breakwater is to close off the northern bay completely. This has as advantage that the breakwater length and depth will be reduced. This will result in less stones required for the breakwater. On the leeside of the breakwater it is possible to construct extra anchorage facilities. Downside of this option is that many boats will navigate between the middle outcrop and the harbour basin entrance. This is a potential bottleneck. Furthermore is it possible that most fishermen want a place closer to the harbour entrance, preferring the southern bay over the northern one, making it very crowded. Lastly, the sediment that would bypass the northern outcrop now follows the breakwater, thus creating a new sedimentation problem in the southern bay. This sedimentation could again be prevented by constructing a small breakwater at the northern outcrop. The option is visualized in Figure 17-5.

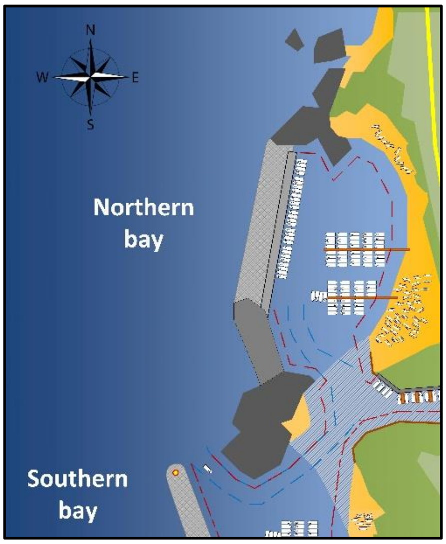


Figure 17-5: Option 'close-off northern bay'

The same accounts for the southern bay. This can also be closed off with the northern bay open as shown in Figure 17-4. However, this would not increase the capacity nor decrease the amount of stones needed for the breakwater significantly.

More information about breakwater design can be found in Appendix N.

Future sedimentation

The breakwater orientation in the northern bay is the same as in conceptual designs 2 and 3, the possible future sedimentation is already discussed there. No sediment bypasses the southern outcrop. The only sources of sediment are from the minor wave action in the harbour bays and tide.

More information about future sedimentation can be found in Appendix I.4

Revetment

A revetment is placed on the river banks to protect it against erosion from wave and ship action. The revetment will ensure the required draught of the river and keep the river bank in place.

More information about the revetment can be found in Appendix L.

River diversion.

In this option the lagoon river mouth is dredged to connect the northern and southern bay. Boats can enter the harbour basin from both bays. Furthermore the entrance is wider which should make navigation easier when a higher amount of vessels operate in the harbour.

More information about the river diversion can be found in Appendix G

Work estimation

The estimated amount of structures and groundworks are presented in Table 17-4.

Table 17-4: Crude estimation of the extent of the works

Structure/works	Amount	Dimension
Breakwater	520	[m]
Dredging	2.5	[ha]
Rock removal	200	[m^3]
Jetties	350	[m]
Quay wall	110	[m]
Revetment	250	[m]

17.5 Concept 5: 'Null-option'

In this section the fifth and most simplified conceptual design is presented. This concept is partly based on an interview with a fisherman which can be found in appendix Q.1 and solves most of their encountered problems while keeping the work to a minimum. A buoy is installed behind the breakwater and a sediment trap is placed next to the river mouth. Furthermore, the current river mouth will be enlarged and jetties are installed inside the basin. The concept is visualized in Figure 17-6.

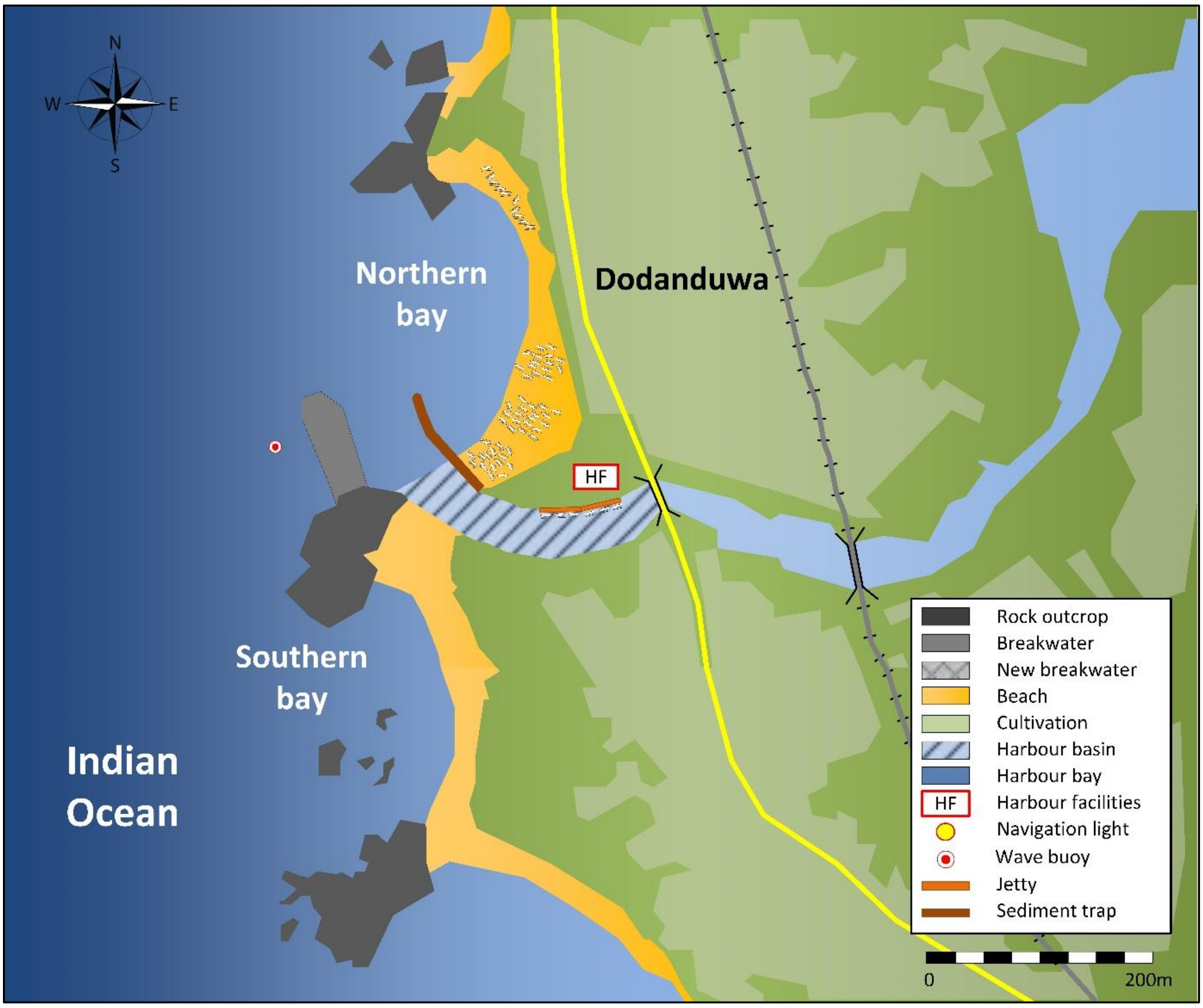


Figure 17-6: Lay-out of the 'Null-option' conceptual design.

Fish unloading and refuelling structure

This concept is close to the original situation, except for the harbour basin which will be open throughout the whole year due to the construction of the sediment trap on the east side of the lagoon mouth. A jetty will be constructed at the northern side of the basin for unloading and refuelling. This is only meant for vallams, multiday vessels cannot enter the bay and basin because of the lack of draught and anchorage facilities. In this concept five vallams can unload their fish at a time and one vallam can refuel. A jetty is chosen over a quay wall due to the smaller amount of structural work.

More information about jetty design can be found in Appendix K.

Anchorage

Vallams have to land on the beach after unloading, just as in the current situation. The beach will increase in size because of accretion on the northern side of the sediment trap. Boats can also land on the beach behind the middle outcrop.

Breakwater

The current breakwater is still in place, but a buoy is installed behind the breakwater (Figure 17-7). This buoy provides the outgoing fishermen with information about the incoming waves. The buoy follows the movement of the waves and with the top of the buoy visible from the bay, fishermen can determine when it is the best moment to sail out of the bay. Additionally a light is installed at the top of the buoy, therefore it is also visible at night.

Figure 17-7: Impression of outgoing fishermen with the old breakwater and the buoy in the background.

Future sedimentation

At the river mouth, a sediment trap is installed which blocks the sediment coming from the north. By trapping the sediment, the river mouth will stay open all year and sediment will accumulate in front of the trap. To prevent that sediment bypasses the trap, it is necessary to remove this sediment occasionally.

More information about future sedimentation can be found in Appendix I.5, and the design of the sediment trap in Appendix M

Work estimation

The estimated amount of structures and groundworks are presented in Table 17-5.

Table 17-5: Crude estimation of the extent of the works

Structure/works	Amount	Dimension
Sediment trap	100	[m]
Dredging	0.2	[hectare]
Jetty	80	[m]
Buoy	1	[-]

EVALUATION & RECOMMENDATIONS

18. EVALUATION

In the evaluation, the concepts are compared to one another. The following criteria are used: the boundary conditions, the durability, amount of work, and the socio-economic and environmental impact. Two different ways are used to grade the concept. The first one is if a concept satisfies a condition or not, mainly used for boundary conditions. The second scale expresses features of the concepts in a number from 1 to 5, where 1 is the lowest score and 5 the highest score. The used evaluation scale and its meaning can be found in Table 18-1.

Table 18-1: Scale used for evaluation the concepts.

	1	**2**	**3**	**4**	**5**
(A)	Very poor	Poor	Satisfactory	Good	Very good
(B)	Very large	Large	Moderate	Limited	Very limited
(C)	Very limited	Limited	Moderate	Large	Very large

18.1 EVALUATION MATRICES

In the following paragraphs, the concepts are rated on several criteria. Specific meaning of those criteria can be found in Appendix O.1 More information about the individual concept evaluation can be found in Appendices O.2 to O.6.

18.2 BOUNDARY CONDITIONS

As can be seen in Table 18-2, do all concepts satisfy the boundary conditions for draught, capacity and accessibility except for the fifth concept. The fifth concept does not suffice because it is judged on multiday vessel criteria while the design is focussed on the current vallam fleet, without the possible increase of multi day vessels. If this concept is only judged on its ability to handle vallams, it would suffice as well.

The safety (entrance) condition is best met in the fourth concept where two entrances are present, and least met in the fifth concept due to the partly constructed breakwater which does not ensure a calm bay. A buoy and lights are the only aids in navigation in and out of the bay. For the unloading facilities, the second concept scores best due to the fact that the basin is separated from the river and placed in the calm bay.

Table 18-2: Summary of the boundary condition evaluation.

Boundary conditions	**Scale type**	**Concept 1**	**Concept 2**	**Concept 3**	**Concept 4**	**Concept 5**
Draught	-	✓	✓	✓	✓	*x*
Capacity	-	✓	✓	✓	✓	*x*
Safety (Entrance)	(A)	4	4	4	5	3
Safety (Unloading facilities)	(A)	3	5	3	3	3
Accessibility	-	✓	✓	✓	✓	*x*

18.3 DURABILITY

The durability evaluation is split up in engineering durability and efficaciousness. The engineering durability is the same for all concepts due to the same building materials for all concepts. Slight differences in for example quay wall or jetty were not significant enough to make a concept ‘less’ or ‘more’ durable.

For the functional durability (efficaciousness of the concepts), concept four scored the best based on the huge amount of mooring places for both vallams and multiday vessels. The third concept has only nearly enough space for the expected 10-year growth of the fleet and has therefore a lower score. The fifth concept scores very poorly, again due to the multiday vessel criteria. The exact scores of the concepts can be found in Table 18-3.

Table 18-3: Summary of durability evaluation.

	Scale type	Concept 1	Concept 2	Concept 3	Concept 4	Concept 5
Engineering durability	(A)	4	4	4	4	4
Efficaciousness	(A)	4	4	3	5	1

18.4 Extent of works

The extent of work that has to be carried out to complete a concept is summarized in Table 18-4. It can be seen that concept five brings a very small amount of work, as it was designed for. Concept four obviously has the biggest workload, mainly due to the total breakwater length that has to be realized. Concepts two and three have a roughly large workload due to the diversion of the river which causes additional claimed land from home owners and land reclamation from the river. The first concept has an average workload and no further special work features that have to be carried out.

It should be kept in mind that the concepts are evaluated on the extent of work, not on the expected costs that are accompanied by it. Some jobs might be bigger in a concept, but much cheaper to carry out than a 'smaller' job in another concept.

Table 18-4: Summary of the work extent evaluation.

Structure/works	**Concept 1**	**Concept 2**	**Concept 3**	**Concept 4**	**Concept 5**	**Dimension**
Breakwater	350	350	350	520	–	$[m]$
Dredging	1.0	2.0	2.0	2.5	0.2	$[ha]$
Rock removal	–	–	200	200	–	$[m^3]$
Claimed land	–	1.5	0.5	–	–	$[ha]$
Land reclamation	–	1.0	0.7	–	–	$[ha]$
Jetties	430	330	250	350	80	$[m]$
Quay wall	–	70	160	110	–	$[m]$
Revetment	350	300	350	250	–	$[m]$
Sediment trap	–	–	–	–	100	$[m]$
Buoy	–	–	–	–	1	$[-]$
Scale type (B)						
Overall score:	3	2	2	1	5	$[-]$

18.5 Impact

The final evaluated aspects are the socio-economic and environmental impacts. Socio-economic impacts of the different concepts do not differ that much, except for the fifth concept in which basically everything stays the same as in the current situation. No extensive harbour area or bigger fleet are realized, keeping the socio-economic situation the same. The same yields for the fifth concept and the environmental impact. The reason that concept five has a smaller environmental effect is because the river is taken out of the harbour area and left over to the seasonal processes, limiting the salt water intrusion.

Table 18-5: Summary of the impact evaluation.

	Scale type	Concept 1	Concept 2	Concept 3	Concept 4	Concept 5
Socio/economic						
Positive effects	(C)	4	4	4	4	1
Negative effects	(B)	3	4	3	3	5
Environmental*	(B)	3	4	3	3	5

* negative effect

19. RECOMMENDATIONS

The goal of this project is to provide several conceptual designs. Before one of these concepts can be implemented further research should be executed.

19.1 EXPECTED FUTURE FLEET

The expected future fleet was given by C. Fernando (Appendix P.4). At this moment around 28 multiday vessels are in possession of the Dodanduwa fishermen and are expected to use the Dodanduwa harbour after the reconstruction. It is expected that this fleet will grow once the harbour is improved, but at the same time it is possible that this will not be the case (Appendix Q.2). For instance, the suspended import ban by the EU could result in a shift to deep-sea fishing. The future fleet is a determining factor for the capacity needed. Furthermore, one should check if the onshore facilities match the future capacity.

19.2 IMPACT OPEN RIVER MOUTH

If the river is open all year around, saltwater might intrude the Ratgama Lake as briefly discussed in the report. No data was available and it was not allowed to interview the involved parties. For this reason saltwater intrusion is out of the scope of this report. However, it is believed that the salinity in the lake is a potential hazard which can affect the area inside and surrounding the lake. Inquiries regarding this phenomenon will help deciding if measures are needed. Diverting the river mouth is a possible solution, the impact of this measure must be investigated further.

19.3 WAVE DATA AND SEDIMENT TRANSPORT

Once a concept is chosen, the impact of this design should be modelled. Wave modelling should show what the expected wave height will be inside the bay. In addition a model should be set up to model sediment transport in this area and model the impact of the proposed design on the nearby coastlines. Some erosion is expected north of the harbour. Another aspect to be investigated is the siltation rate inside the bay, not neglecting the possible sediment supply by the river.

19.4 EXTENDING THE CURRENT BREAKWATER AND ROCK OUTCROPS

In concepts 1 to 4, the current breakwater is extended. The old breakwater does not satisfy the requirement level of the new proposed breakwater. First the present state of the existing breakwater should be investigated. Next should be decided how to improve the breakwater and how the transition between the new and old breakwater is made. The same applies for extending the rock outcrops.

19.5 DATA LIMITATIONS

The calculations made are based on several assumptions, for instance assumed bathymetry. The presented structural designs should be considered taking the assumptions into account. The technical designs of the structures are therefore not suitable for direct implementation. Additional data and in-depth calculations are required. A probabilistic calculation is preferable.

19.6 COSTS

In the study performed by the project team costs are not included. So far as the project team is aware, there is no budget secured for the project. The revenues, initial costs, operational costs and maintenance costs are unclear. A former student group from the Delft University of Technology conducted a study about the financial problems of the Dikkowita fishery harbour. Besides the Hikkkaduwa and Galle harbour are quite close by to Dodanduwa. This all brings concerns to the feasibility of the Dodanduwa fishery harbour. A study should be conducted to prove financial feasibility of the project.

19.7 Hydraulic structures

Quay wall

Other designs for a quay wall have to be evaluated. Most likely options are: concrete block walls, L- or T-shaped profiles, sheet piles, quay walls and jetty like structures with a small retaining wall and slope.

There are certain technicalities for the gabion quay wall which needs to be further investigated. These are the foundation of the gabion wall. Also the size of the gabions itself needs to be investigated. This to find an optimum for transport and lift capacity. The concrete slab on top of the quay wall needs to be dimensioned.

Jetty

It is common to anchor a multiday vessel and reach the shore by smaller boat. In this case no jetties are needed, although the project team thinks this is not a very convenient method. An option for the jetty design is the construction of a concrete jetty. Another option which can be evaluated is a floating jetty structure.

Sediment trap

The calculations done for the sediment trap are uncertain. The sheltering effect of the current breakwater are not accounted for, resulting in a big wave height hitting the structure. Models must be determine the waves hitting the structure, this also applies for the current. The same accounts for the extent of the sediment trap in the bay.

Secondly, other options for the sediment trap design can be looked at. As mentioned in Appendix M, gabions can be used. Sheet pile walls or a rubble mount structure could also be an option. The latter two are depending on the soil profile and hydraulic conditions respectively.

Revetment

It should be checked if a revetment can increase the stability and strength of the slope. This could increase the slope, thus reducing the space needed by the river. Now the slope of 1:5 is assumed in all revetment cases.

Breakwater

Calculations show that due to the high design wave height a conventional breakwater is not preferred. A partly or fully reshaping berm breakwater seems to have the smallest cross-sectional area. Increasing the damage level and increasing the maximum allowable overtopping rate will make a conventional breakwater design more attractive.

It is recommended to prevent anchorage directly behind the breakwater, in this way the maximum overtopping discharge can probably be increased. Furthermore the available quarries should be investigated in an early state, because it is economical to match the breakwater design with the quarry output.

20. REFERENCES

A. de Vos, C. B. (2013). *Surface circulation and upwelling patterns around Sri Lanka.* Copernicus Publications.

Alataristarion. (2015, April 30). *A map of the Indian Plate.* Retrieved from commons.wikimedia.org: https://commons.wikimedia.org/wiki/File:IndianPlate.png

Amarasekera, H. (2008, January 29). *RECENT TRENDS IN THE FORESTRY SECTOR OF SRI LANKA*. Retrieved from environmentlanka.com: http://environmentlanka.com/blog/2008/recent-trends-in-the-forestry-sector-of-sri-lanka/

Amarasinghe, O. (2014, 01 20). *Economics of fishing by multi-day crafts in Sri Lanka.* Retrieved from Freedom to nation SL: http://freedomtonationsl.blogspot.com/2014/01/multiday-boat-study.html

Argoss, B. (2016). *waveclimate.com*.

Bandara, M. S. (2016, October 12). General Manager, CFHC. (P. Dodanduwa, Interviewer)

C. Fernando, G. m. (2016). (P. t. Dodanduwa, Interviewer)

C. Fernando, G. m. (2016). Meeting. (P. t. Dodanduwa, Interviewer)

Center, D. M. (1934). Hydrographic chart of the Indian Ocean, Sri Lanka and the Northern portion of the Maladives. Washington, USA. Retrieved from http://www.oceangrafix.com/chart/zoom?chart=63010

Chathurika, K., & Dissanayake, D. (2016). Initial study on catch, species composition and reproductive biology of fishes off the south-west coast of Sri Lanka, targeted by ring nets while utilizing natural floating objects. *Journal of Applied Ichthyology*, 464-470.

Chris R.I. Clayton, R. I. (2014). *Earth Pressure and Earth-Retaining Structures, Third Edition.* Boca Raton: CRC Press, Taylor & Francis Group.

CIRIA; CUR; CETMEF. (2007). *The Rock manual, the use of rock in hydraulic engineering.* London: C683, CIRIA, London.

Coastal Resources Management project. (2005). *Wave Climate.*

ColomboPage News Desk. (2014, June 28). *Sri Lankan fisher community takes to streets to protest death of colleague*. Retrieved from ColomboPage: http://www.colombopage.com/archive_14A/Jun28_1403898087CH.php

D. Pauly. (1998). Fishing Down Marine Food Webs. In *Science, New Series, Vol 279, No. 5352* (pp. 860-863). American Association for the Advancement of Science.

Daily News. (2009, August 14). *Quick look*. Retrieved from DailyNews.lk: http://archives.dailynews.lk/2001/pix/PrintPage.asp?REF=/2009/08/14/news60.asp

Department of Meteorology Sri Lanka. (2016, September 14). *Climate in Sri Lanka*. Retrieved from meteo.gov: http://www.meteo.gov.lk/index.php?option=com_content&view=article&id=13&Itemid=132&lang=en#4-northeast-monsoon-season-december-february

Disaster Management Center. (n.d.). *Sea level rise.* Sri Lanka: Ministery of disaster management.

Dissanayake C.B. & Munasinghe, T. (1984). Reconstruction of the Precambrian sedimentary basin in the granulite belt of Sri Lanka. *Chemical Geology*, 221-247.

Dissanayake, C. (2005). A new plate boundary near Sri Lanka: Implications for future geohazards. *J.Natn.Sci.Foundation Sri Lanka 2005*, 5-8.

Dominic Reeve, A. C. (2004). *Coastal Engineering: Processes, Theory and Design Practice.* New York: Spon Press.

EML Consultants. (2016). *Feasibility Study on Upgrading of Proposed Infrastructure Facilities - Dodanduwa Inception Report.* Colombo: EML Consultants.

Fookes, P. B. (2000). Total geological history: a model approach to the anticipation,observation and understanding of site conditions. *Proceedings GeoEng 2000* (pp. 370-460). Melbourne: Technomatic.

Francis K. Davis, H. N. (1968). The variation of gust factors with mean wind speed and with height. *Journal of applied meteorology*, 7.

GSMB. (2000). *Sri Lanka Geology, Sheet 19.* Geological Survey and Mines Bureau of Sri Lanka.

Henk Jan Verhagen, K. d. (2012). *Breakwaters and closure dams.* Delft: VSSD.

Holthuijsen, L. H. (2007). *Waves in oceanic and coastal waters.* New York, United states of America: Cambridge University Press.

Holy, S. (2014, Octobre 19). *Understanding Sri Lanka's cement industry from a customer's perspective*. Retrieved from Sunday Times: http://www.sundaytimes.lk/141019/business-times/understanding-sri-lankas-cement-industry-from-a-customers-perspective-122531.html

J. van der Meer, S. S. (2016). *Design and construction of berm breakwaters.*

J.I.C.A. (2003). *THE STUDY ON STORM WATER DRAINAGE PLAN FOR THE COLOMBO METROPOLITAN REGION IN THE DEMOCRATIC SOCIALIST REPUBLIC OF SRI LANKA.* MINISTRY OF HOUSING & PLANTATION INFRASTRUCTURE SRI LANKA LAND RECLAMATION & DEVELOPMENT CORPORATION.

Japan International Cooperation Agency (JICA). (2003). *The study on comprehensive groundwater resources development for Hambantota and Monaragala districts in the democratic socialist republic of Sri Lanka.* Pacific Consultants International .

Katupotha, J. (1994). Quaternary Research in Sri Lanka. *Journal of the Geological Society of Sri Lanka, Vol. 5*, 141-152.

Maertens, J. (2009). Stability of underwater slopes realized by means of a suction dredger. *ICSMGE* (pp. -). Amsterdam: IOS Press.

Manual Hydraulic Structures. (n.d.). Delft: TU Delft.

MFAR. (2016). *Fisheries Statistics - 2015 (Annual Report).* Colombo: Ministry of Fisheries and Aquatic Resources.

MFAR, M. o. (2016). *Fisheries in Sri Lanka*. Retrieved September 14, 2016, from Ministry of Fisheries and Aquatic Resources: http://www.fisheries.gov.lk

Mushtaq, M. (2014, March 03). *Sri Lanka relaxes opposition to foreign steel production investments*. Retrieved from Metalbulletin: https://www.metalbulletin.com/Article/3315291/Sri-Lanka-relaxes-opposition-to-foreign-steel-production-investments.html

Nielsen, D., & Brock, M. (2003). Effects of increasing salinity on freshwater ecosystems in Australia. *Australian Journal of Botany*, 655-665.

NMDF-40 Multi Day Fishing Boat. (n.d.). Neil Marine.

Normcommissie 351 006 Geotechniek. (2012, April). EN - 1997 - 1+C1. Delft, The Netherlands: NEN.

Pathirana, H. (1980). Geology of Sri Lanka in relation to Plate Tectonics. *L. Natn. Sci. Coun. Sri Lanka*, 75-85.

Quick Reference. (n.d.). Delft: TU Delft.

Rosenbaum, M. (2016, September 14). *Engineering Geology.* Retrieved from Blackboard TU Delft: -

RWS/DHL. (1988). *Aantasting van Dwarsprofielen in Vaarwegen.*

Sigurdarson, J. v. (2014). *Geometrical design of berm breakwaters.*

Stive, J. B. (January 2015). *Coastal Dynamics I lecture notes.* Delft: Delft Academic press.

Te Chow, v. (1959). *Open channel Hydraulics.* New York: McGraw-Hill.

The Island. (2006, October 19). *Construction of Dodanduwa fishing Harbour at a standstill*. Retrieved from The Island - online edition: http://www.island.lk/2006/10/19/news27.html

Thevasiyani, T., & Perera, K. (2014). *Statistical analysis of extreme ocean waves in Galle, Sri Lanka.* Peradeniya: University of Peradeniya, Sri Lanka.

Thevasiyani, T., & Perera, K. (2014). *Statistical analysis of extreme ocean waves in Galle, Sri Lanka.* Peradeniya: University of Peradeniya, Sri Lanka.

Timberstructures Dictate. (n.d.). Delft: TU Delft.

United Nations Food and Agricultural Organization. (2003, - -). *General situation of world fish stocks.* Retrieved from FAO.org: http://www.fao.org/newsroom/common/ecg/1000505/en/stocks.pdf

Van der Meer, J. A. (2016). *EurOtop,2016. Manual on wave overtopping of sea defences and related structures. An overtopping manual largely based on European research, but for worldwide application.* Retrieved from www.overtopping-manual.com

Vellinga, P. (1986). *Beach and Dune Erosion during Storm Surges.* Delft University of Technology.

Waltham, T. (2009). *Foundations of Engineering Geology.* Abingdon: Taylor & Francis.

WDBG. (2011, July 5). *Utilize Cost Management Throughout the Planning, Design, and Development Process*. Retrieved from WDBG.org: https://www.wbdg.org/design/utilize_management.php

Wijayawardane, I., Ansaf, K., Ratnasooriya, A., & Samarawickrama, S. (2010). *Coastal erosion: Investigations in the southwest coast of Sri Lanka.* Kandy: ICSBE.

Wijdeven, B. (2015, May 22). Fishery ports and inland ports. Delft, Zuid-Holland, Nederland.

Wijerantne, M., & Jayasinghe, M. (n.d.). Wind Loads for high-rise buildings constructed in Sri Lanka. Sri Lanka.

WIJERATNE, E. (2007). *Levelling of Sri Lanka Tide Gauges for improved hydrodynamic modelling of the Palk Strait.* Crow Island, Mattakkuliya, Colombo15, Sri Lanka: Oceanography Division, National Aquatic Resources Research and Development Agency.

Wijetunge. (2013). *Cyclones and Storm Surges Pose a More Frequent Threat than Tsunami.* Retrieved from The Institution of Engineers Sri Lanka: http://www.iesl.lk/page-1734626

Wijetunge, J. (2014). A deterministic analysis of tsunami hazard and risk for the southwest coast of Sri Lanka. *Continental Shelft Researh*, 23-45.

Yell, D., & Riddell, J. (1995). *ICE - design and practice guides - Dredging.* London: Thomas Telford Publications.

Young, M., Hayman-Joyce, J., & Hyeon Kim, S. (2012). Use of single layer concrete armour units as toe reinforcement. 1-12.

APPENDICES

A. GEOLOGY

A.1 TOTAL GEOLOGICAL HISTORY APPROACH

The tectonic map of the Indian plate and the models chosen from the Total Geological History approach (Fookes, 2000) are shown below.

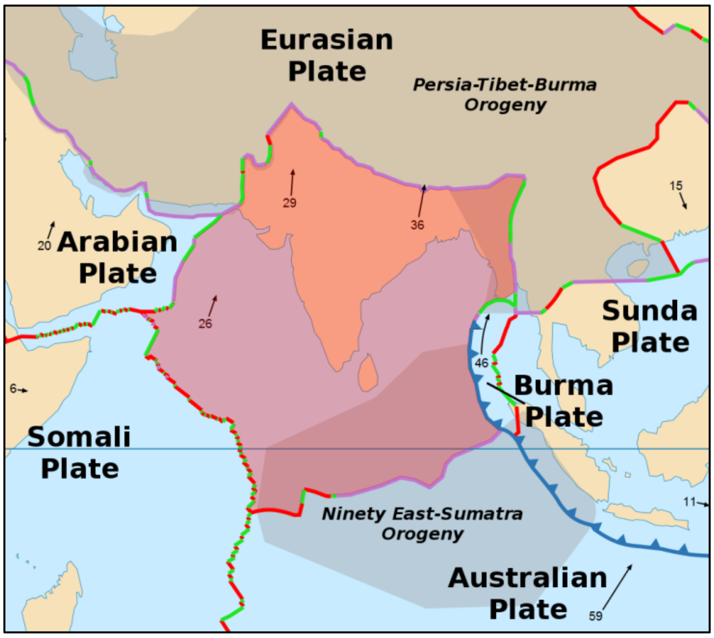


Figure A-1: Position of Sri Lanka on the Indian Plate (Alataristarion, 2015)

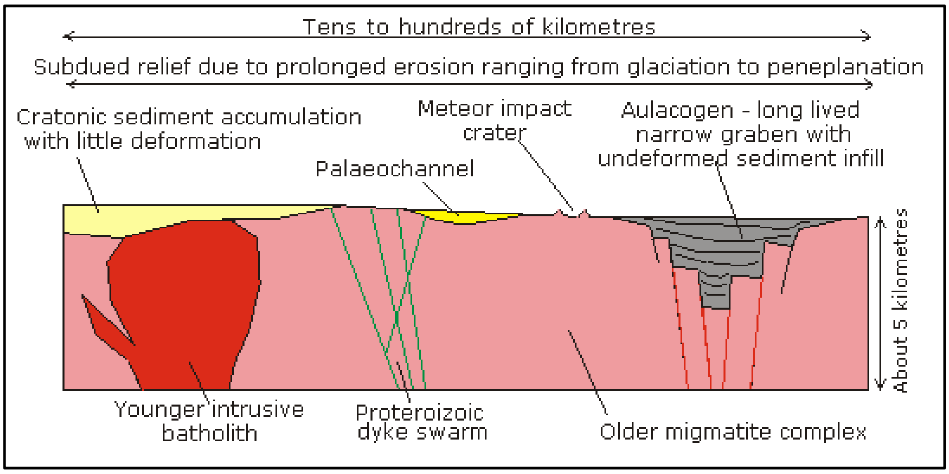


Figure A-2: Schematised visualisations of the tectonic model: intraplate setting cratons (Fookes, 2000)

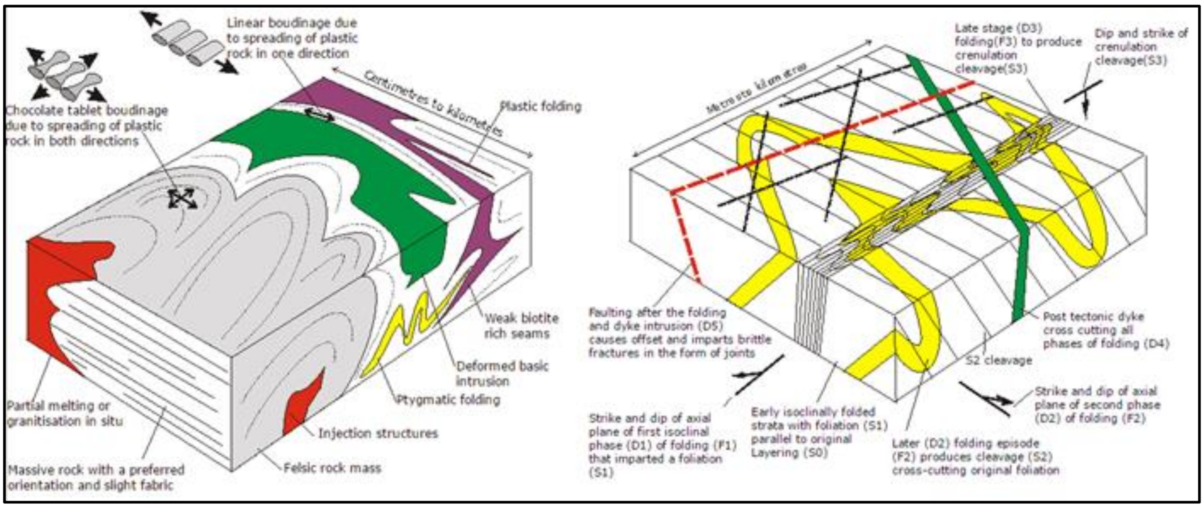


Figure A-3: Schematised visualisations of the geological model: metamorphic rocks (l) and multiple folds and shear (r) (Fookes, 2000)

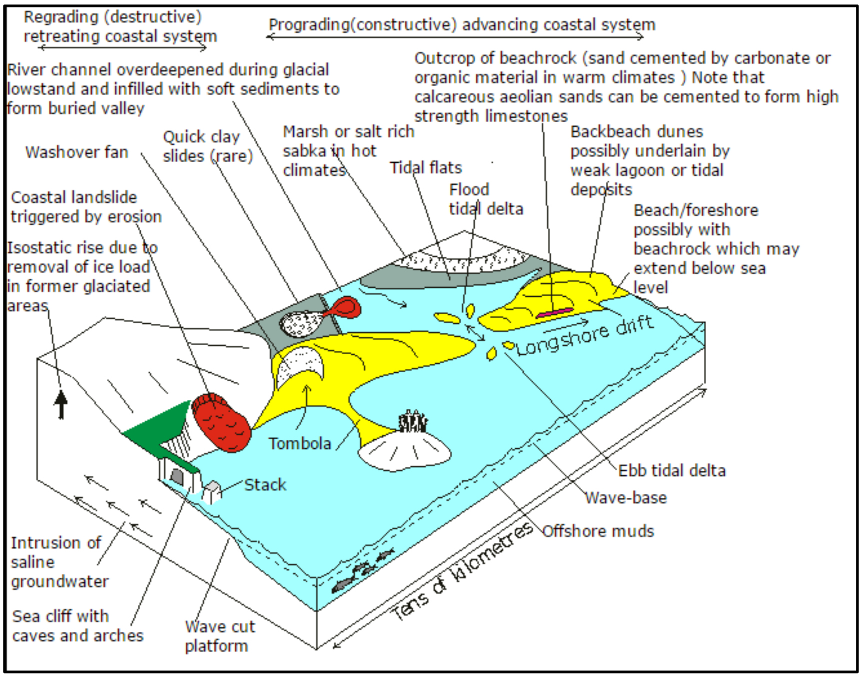


Figure A-4: Schematised visualisations of the geomorphological model: coastal features (Fookes, 2000)

A.2 GEOLOGY OF SOUTH-WEST SRI LANKA

Strength Properties of Rocks

rock type	density dry t/m³	porosity %	dry UCS range MPa	dry UCS mean MPa	UCS saturated MPa	modulus of elasticity GPa	tensile strength MPa	shear strength MPa	friction angle φ°
Granite	2·7	1	50–350	200		75	15	35	55
Basalt	2·9	2	100–350	250		90	15	40	50
Greywacke	2·6	3	100–200	180	160	60	15	30	45
Sandstone – Carboniferous	2·2	12	30–100	70	50	30	5	15	45
Sandstone – Triassic	1·9	25	5–40	20	10	4	1	4	40
Limestone – Carboniferous	2·6	3	50–150	100	90	60	10	30	35
Limestone – Jurassic	2·3	15	15–70	25	15	15	2	5	35
Chalk	1·8	30	5–30	15	5	6	0·3	3	25
Mudstone – Carboniferous	2·3	10	10–50	40	20	10	1		30
Shale – Carboniferous	2·3	15	5–30	20	5	2	0·5		25
Clay – Cretaceous	1·8	30	1–4	2		0·2	0·2	0.7	20
Coal	1·4	10	2–100	30		10	2		
Gypsum	2·2	5	20–30	25		20	1		30
Salt	2·1	5	5–20	12		5			
Hornfels	2·7	1	200–350	250		80			40
Marble	2·6	1	60–200	100		60	10	32	35
Gneiss	2·7	1	50–200	150		45	10	30	30
Schist	2·7	3	20–100	60		20	2		25
Slate	2·7	1	20–250	90		30	10		25

Figure A-5: Rock strength properties of rocks. From Foundations of Engineering Geology, p. 52, by T. Waltham, 2009, Abingdon: Taylor & Francis.

Strength Recognition and Description

Rock/soil description	UCS (MPa)	Field properties
Very strong rock	>100	firm hammering to break
Strong rock	50–100	break by hammer in hand
Moderately strong rock	12·5–50	dent with hammer pick
Moderately weak rock	5·0–12·5	cannot cut by hand
Weak rock	1·5–5·0	crumbles under pick blows
Very weak rock	0·6–1·5	break by hand
Very stiff soil	0·3–0·6	indent by fingernail
Stiff soil	0·15–0·3	cannot mould in fingers
Firm soil	0·08–0·15	mould by fingers
Soft soil	0·04–0·08	mould easily in fingers
Very soft soil	<0·04	exudes between fingers

Figure A-6: Rock recognition and description. From Foundations of Engineering Geology, p. 53, by T. Waltham, 2009, Abingdon: Taylor & Francis.

B. Coastal system

B.1 Global wind patterns

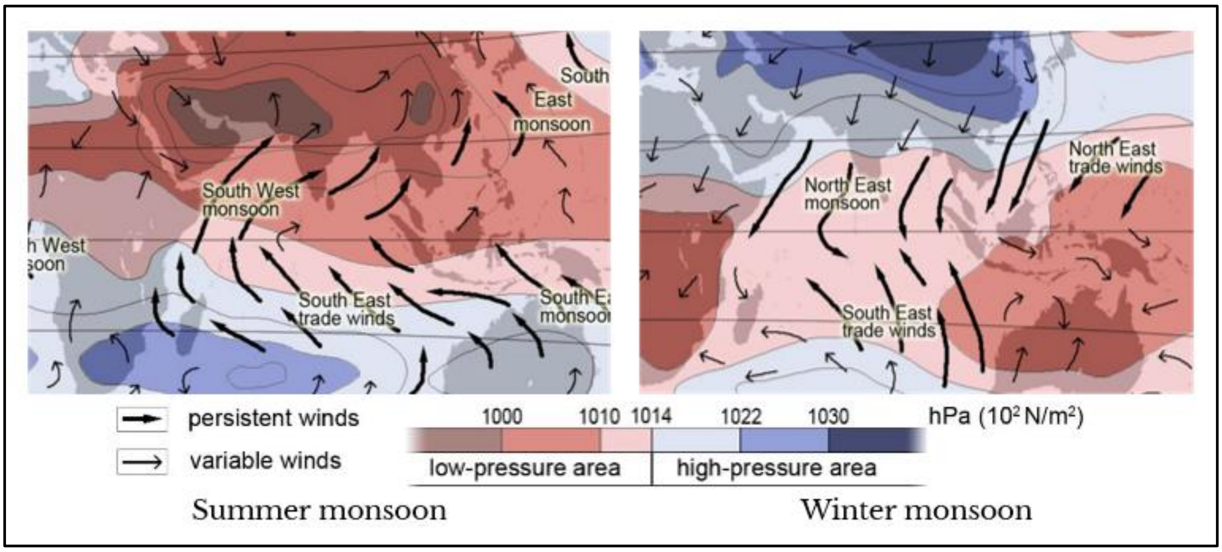
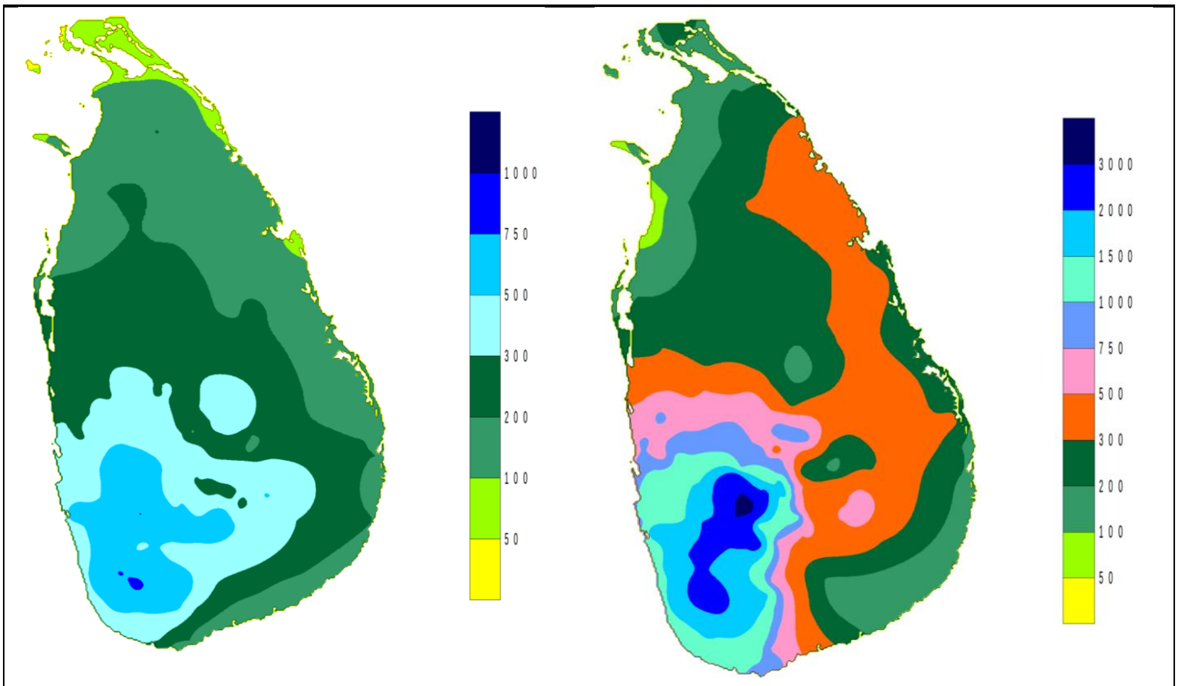


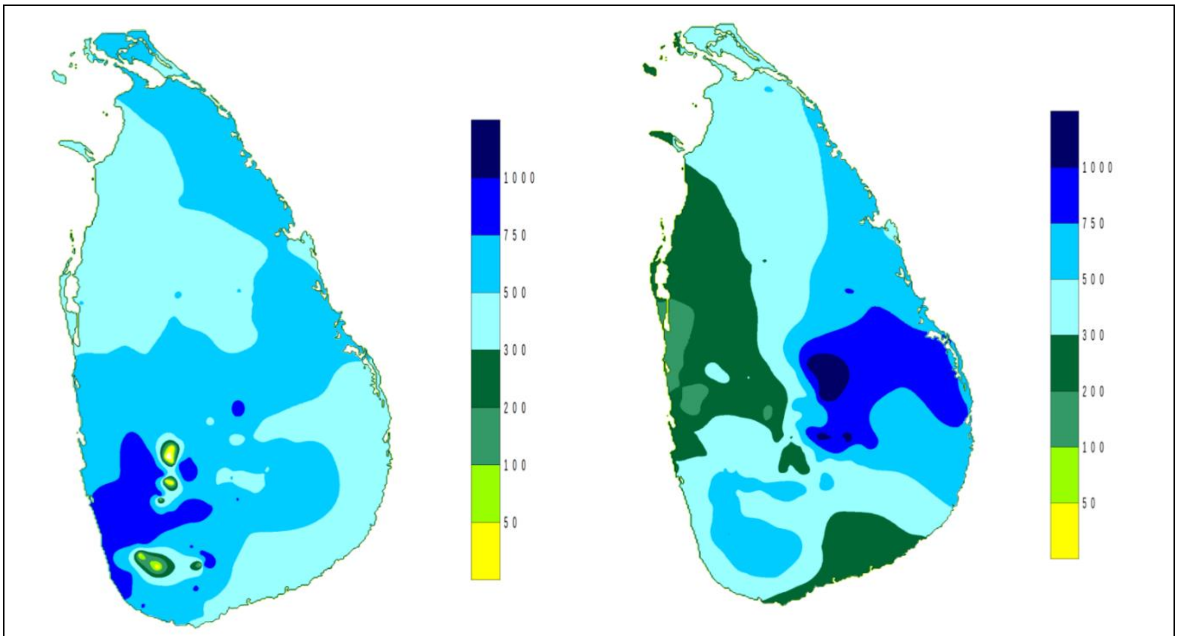
Figure B-1: Global wind patterns (Stive, January 2015)

B.2 Rainfall during the seasons



First intermonsoon period [March – April]

Summer monsoon [May – September]



Second intermonsoon [October – November] Winter-monsoon [December – February]

Figure B-2: Rainfall in the different seasons (Department of Meteorology Sri Lanka, 2016)

B.3 HYDRAULIC CONDITIONS

Mean sea level

In Figure B-3 the seasonal changes in mean sea level is shown. At the West coast the seasonal changes are bigger than at the East coast.

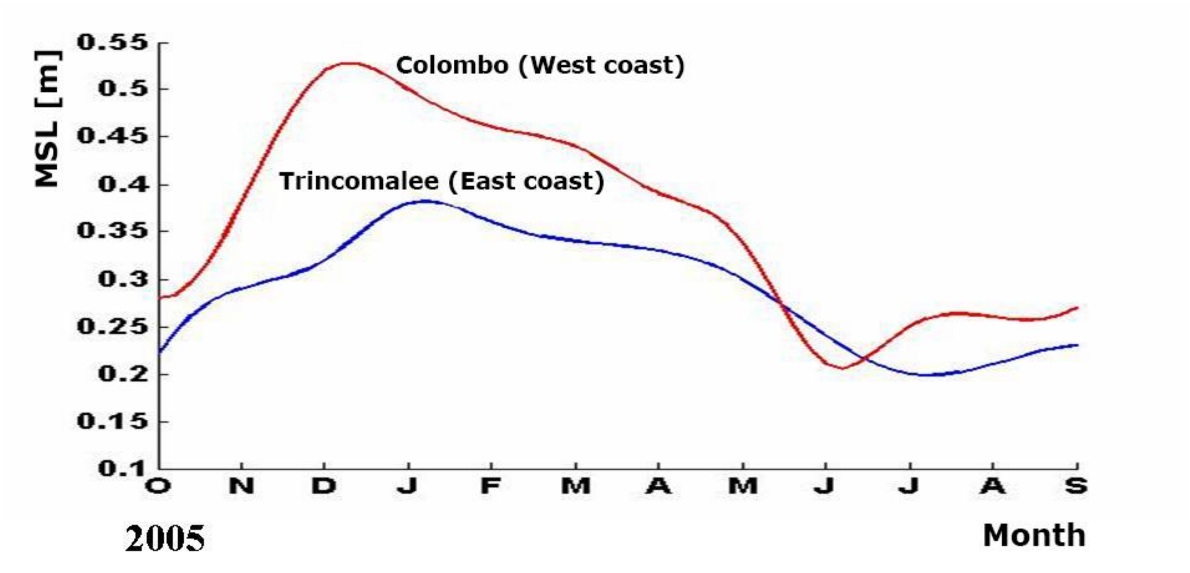


Figure B-3: Seasonal MSL changes west and east coast (WIJERATNE, 2007)

Tides

In this chapter the tide around the coast of Sri Lanka will be discussed. This includes the tidal character and tidal range.

There is no tidal information available for Dodanduwa. Therefore, information of stations nearby is used. Data is retrieved from the tide stations from Colombo and Galle.

Table B-1: Tidal components (A. de Vos, 2013)

	M2	**S2**	**K1**	**O1**
	a[m]	a(m)	a(m)	a(m)
Galle	0.16	0.11	0.05	0.01
Colombo	0.18	0.12	0.07	0.03

To check the tidal character we need to calculate the form factor F (Stive, January 2015). This can be done with the following formula:

$$F = \frac{K_1 + O_1}{M_2 + S_2} \tag{B-1}$$

Table B-2: Tidal character

	F	**Category**
Galle	0.22	Semidiurnal
Colombo	0.33	Mixed, mainly semidiurnal

From this data we can conclude that the tidal character at Dodanduwa is mainly semidiurnal. This is because the tidal character is semidiurnal in Galle. Which is near Dodanduwa.

Wind

This paragraph is divided into two parts. In the first part wind data will be reviewed from waveclimate.com by BMT Argoss. The data from BMT Argoss doesn't include storm data like cyclones. The data from BMT Argoss shows the dominant wind direction and the mean wind velocity throughout the year. In the second part winds are analysed for the design of structures. Wind load due to storms is included in this data.

Winds in Sri Lanka are predominantly generated by the monsoons. Due to the asymmetric heating of land and sea atmospheric circulations are generated. This causes trade winds which have a NE direction during the NE-monsoon and WSW direction during the SW-monsoon. The wind direction changes during the inter-monsoon periods.

Table B-3: Wind direction and mean velocity per month

	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**
Direction	NE	NE	E-W	WSW	WSW	W
Mean velocity [m/s]	3-4	2-3	2-3	3-4	5-6	6-7
	Jul	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
Direction	W	WSW	WSW	WSW	NW	NNE
Mean velocity [m/s]	6-7	6	5-6	4-5	3-4	3-4

Design wind velocity

In the past 135 years, eleven cyclonic storms and five severe cyclonic storms have crossed Sri Lanka. Most of them generated in the Bay of Bengal, but some in Arabian sea. These cyclones happen nearly all during the NE-monsoon. A design code is formed to take wind forces from cyclones into account.

A clear distinction is made between three areas in Sri Lanka. Zone 1 has the most severe wind load from cyclones. This is because most of the cyclones hit the East coast of Sri Lanka. After landfall, the cyclone will lose a part of its energy. Therefore zone 1 and 2 has a lower wind load. Still there is a chance a cyclone will hit Sri Lanka on the West coast.

In Figure B-4 is shown where the zones are per classification. In Table B-4 is shown what the gust and mean wind velocities are per zone. These gust wind velocities are given in the design code. The mean wind velocity is calculated by dividing the gust wind velocity with a factor of 1.8 (Francis K. Davis, 1968).

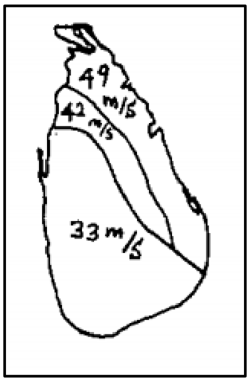


Figure B-4: Gust wind velocities per zone

Table B-4: Wind velocities per zone (Wijerantne & Jayasinghe)

Zone	Description	Classification	Gust wind velocity [m/s]	Mean wind velocity [m/s]
1	50 km from east coast	III	49	27.2
2	Inland strip	II	42	23.3
3	South and West (Including Colombo)	I	33	18.3

Waves

Seasonal waves

The data used for seasonal waves is obtained from waveclimate.com by BMT Argoss and Wisuki. This data is all offshore wave data. The data from BTM Argoss is retrieved for the south-west of Sri Lanka. The data from Argoss BMT does not include storms. These waves are not represented in the data given in this section. This is treated in a separate chapter.

There are two kinds of waves arriving on the coast of Sri Lanka. Those are swell waves and wind waves. Swell waves arriving at the coast are formed during storms on the Indian Ocean. Swell waves dominate the wave climate on the south-west coast. Wind waves are the results of the monsoon winds and the trade winds. There are periods during the SW monsoon that wind waves dominate the swell waves.

Significant wave height is calculated by taking the average of 1/3 of the highest waves heights. The monthly wave height is divided in bins of 0.2 meter. Therefore, the results are presented in sections of 0.2 meter. The yearly wave height is calculated by using probability of exceedance tables. Where only one value is presented. Therefore, the results are presented as single value number.

Peak period is presented by making use of tables from wave statistics. Hereby is checked the distribution of wave height versus peak wave period.

Swell waves

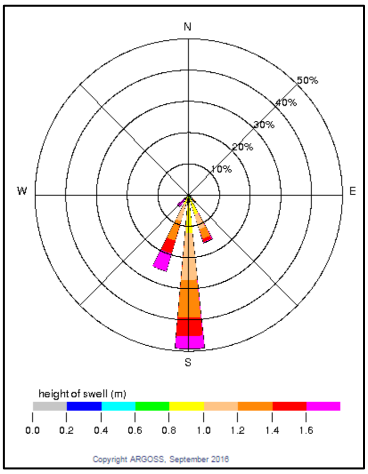


Figure B-5: Swell wave height and direction Oct - Mar

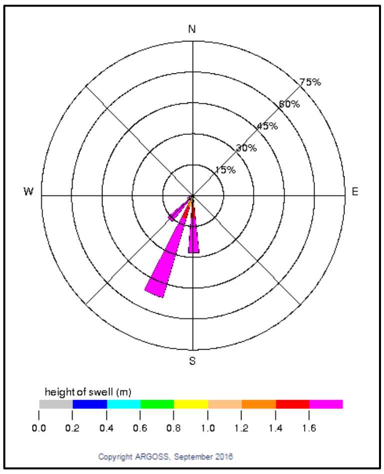


Figure B-6: Swell wave height and direction Apr – Sept (SW

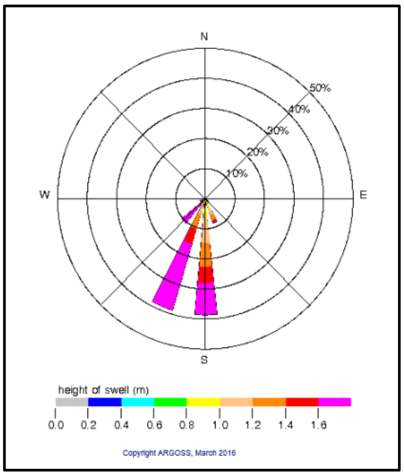


Figure B-7: Yearly swell wave height and direction

Swell waves approach the coast of Sri Lanka mainly from the SSW and S direction. This is shown in Figure B-7. The wave height of the swell waves differs over the year. During the months April to September the swell waves increase in height. The swell during this period is almost twice as high than during the other months of the year. High swell waves have been recorded with a height of over 4 meters (Thevasiyani & Perera, 2014)

Wind waves

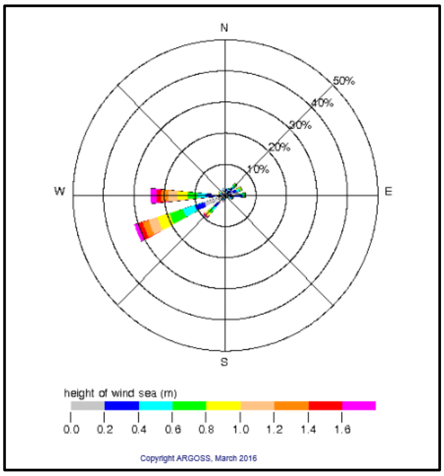


Figure B-8: Direction and wave height of wind waves

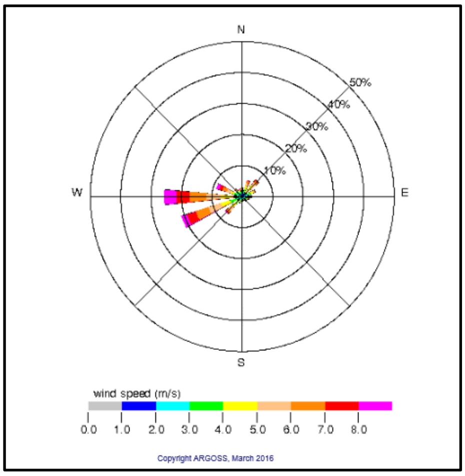


Figure B-9: Yearly wind direction and speed

The direction of the wind waves is correlated with the wind direction of the monsoon period, as is shown in Figure B-8 and Figure B-9. During the SW monsoon waves mainly come from W and WSW direction. During the NE monsoon waves originate on the north-east side of Sri Lanka. Those waves will not reach the south-west coast. Waves in Figure B-8 from NEE and E direction are measured offshore.

Dominant waves

Both swell and wind waves arrive at the coast. Swell waves arrive mainly from the SSW direction and wind waves from the WSW direction. The swell waves dominate the wind waves. The dominant wave direction is therefore SSW. As is shown in Figure B-10. However, it is possible during the SW monsoon that the wind waves dominate the swell waves for a short period. This will result in waves coming from the West.

The bay of Dodanduwa is positioned towards the West as shown in Figure B-11. The waves from SW direction will enter the bay directly. Waves from SSW and S direction will not enter the bay of Dodanduwa directly. These waves might enter the bay by either refraction or diffraction, depending on the local bathymetry.

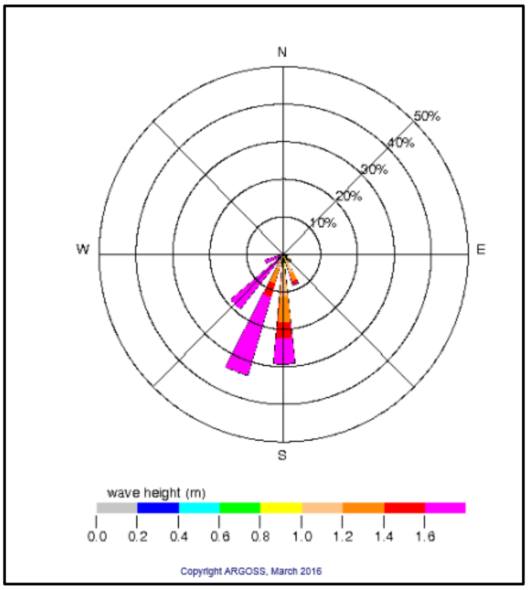


Figure B-10: Direction and wave height of dominant waves

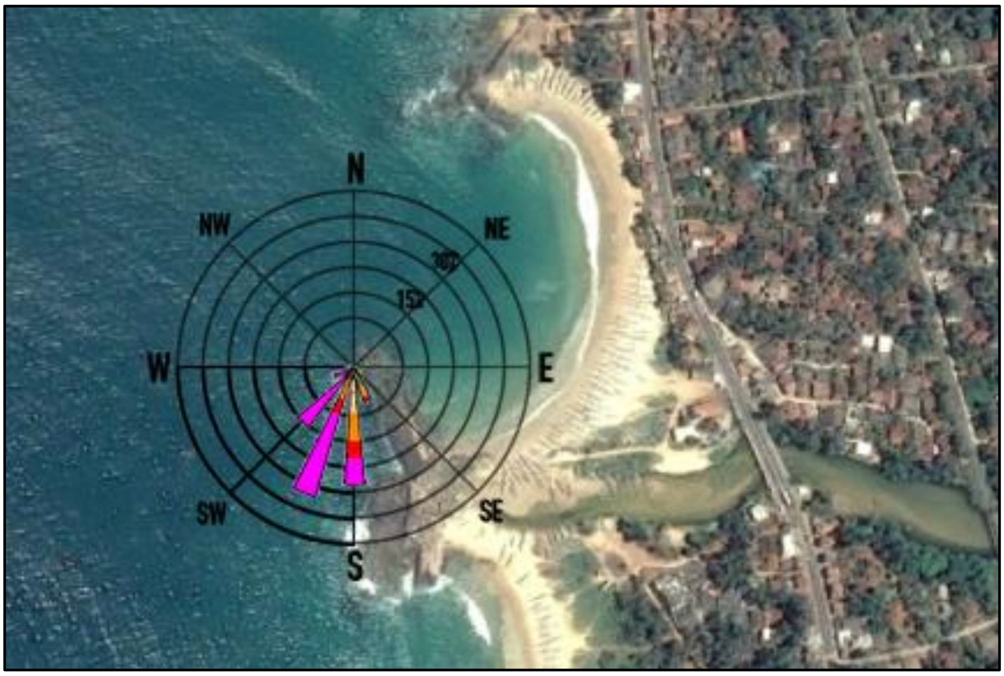


Figure B-11: Yearly averaged wave directions at the project location

Extreme waves

The extreme waves statistics of the south west coast have been researched (Thevasiyani & Perera, 2014). In this study wave data is used which is collected by the CCF-GTZ. They have used a directional and roll buoy WAVEC. Which was located 8 km offshore from Galle, at 70-meter depth. For the calculation of the extreme waves the Peak Over Threshold method is used with a specific threshold selected using the Generalized Pareto Distribution. In Table B-5 the extreme wave heights are given for the SW and NE monsoon. Wind waves are higher than the swell waves. This is because swell waves are not affected by the wind after the generation of swell waves. The overall wave height is calculated as follows:

$$H_{overall} = \sqrt{H_{wind}^2 + H_{swell}^2} \qquad \text{B-2}$$

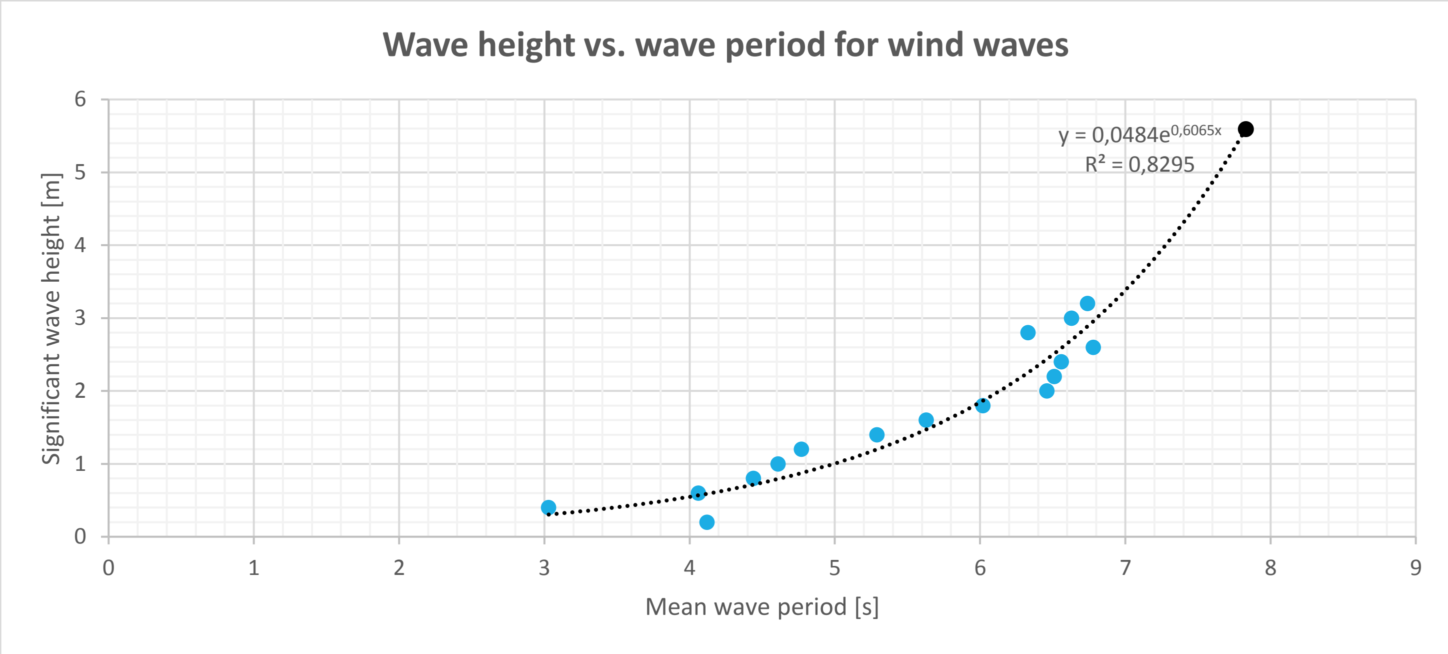
Periods corresponding to the extreme wave heights are presented in the study of the University of Peradeniya. The project team estimated the corresponding wave periods by extrapolating data gained by the Coastal Conservation and Coastal Resource Department. The SW monsoon period is by far the dominant period. Therefore, only values representative for the SW monsoon are presented.

Table B-5: Extreme wind and swell wave height (Thevasiyani & Perera, 2014)

Probability	**Return Period**	**SW Monsoon**		**NE Monsoon**	
	[years]	Wind wave height [m]	Swell wave height [m]	Wind wave height [m]	Swell wave height [m]
0.200	5	4.12	2.82	2.60	2.11
0.100	10	4.44	2.90	2.82	2.21
0.066	15	4.63	2.92	2.95	2.26
0.050	20	4.77	2.95	3.04	2.30
0.020	50	5.22	3.00	3.32	2.41
0.010	100	5.58	3.03	3.53	2.49

Table B-6: Wave height vs. wave period for wind waves (Coastal Resources Management project, 2005)

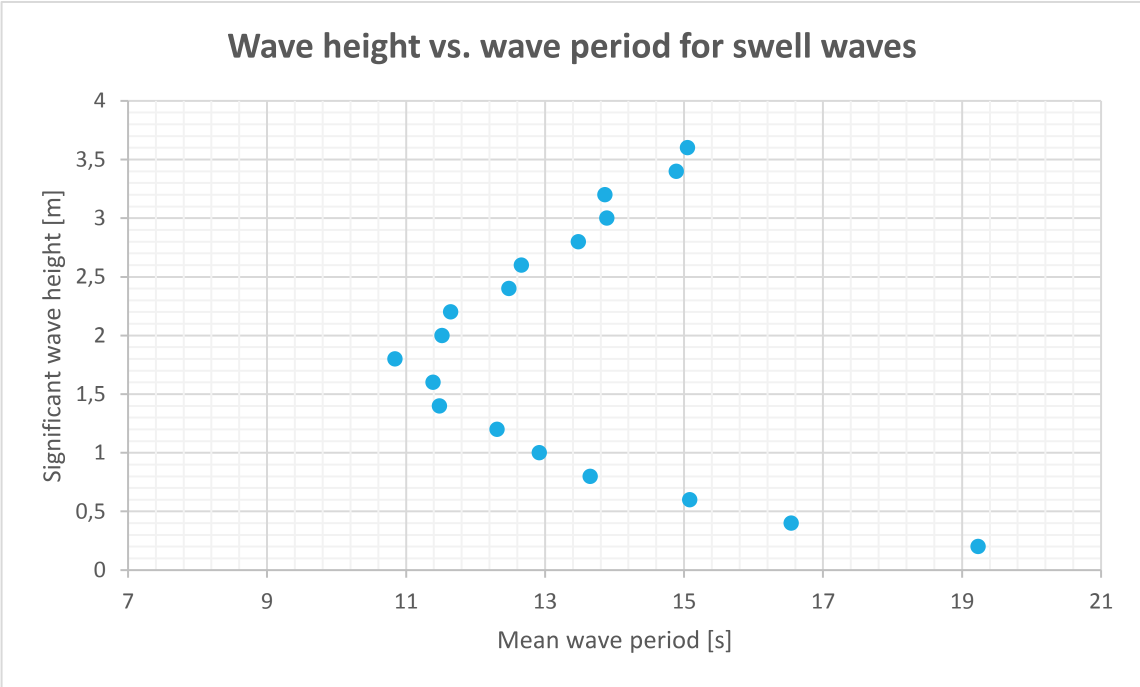
H_s [m]	T_m [s]
0.2	4.12
0.4	3.03
0.6	4.06
0.8	4.44
1	4.61
1.2	4.77
1.4	5.29
1.6	5.63
1.8	6.02
2	6.46
2.2	6.51
2.4	6.56
2.6	6.78
2.8	6.33
3	6.63



Probability	Return period	H_{ss} [m]	Confidence interval		$H_{mo,storm}$ [m]	T_m [s]	T_p [s]
0.2	5	4.12	3.940	4.399	4.45	7.30	8.75
0.1	10	4.44	4.190	4.801	4.80	7.45	8.93
0.066	15	4.63	4.335	5.027	5.01	7.55	9.05
0.05	20	4.77	4.437	5.190	5.16	7.60	9.11
0.02	50	5.22	4.757	5.726	5.64	7.75	9.29
0.01	100	5.58	5.009	6.149	6.03	7.80	9.35

= Estimation

H_s [m]	T_m [s]
0.2	19.23
0.4	16.54
0.6	15.08
0.8	13.65
1	12.92
1.2	12.31
1.4	11.48
1.6	11.39
1.8	10.84
2	11.52
2.2	11.64
2.4	12.48
2.6	12.66
2.8	13.48
3	13.89
3.2	13.86
3.4	14.89
3.6	15.05

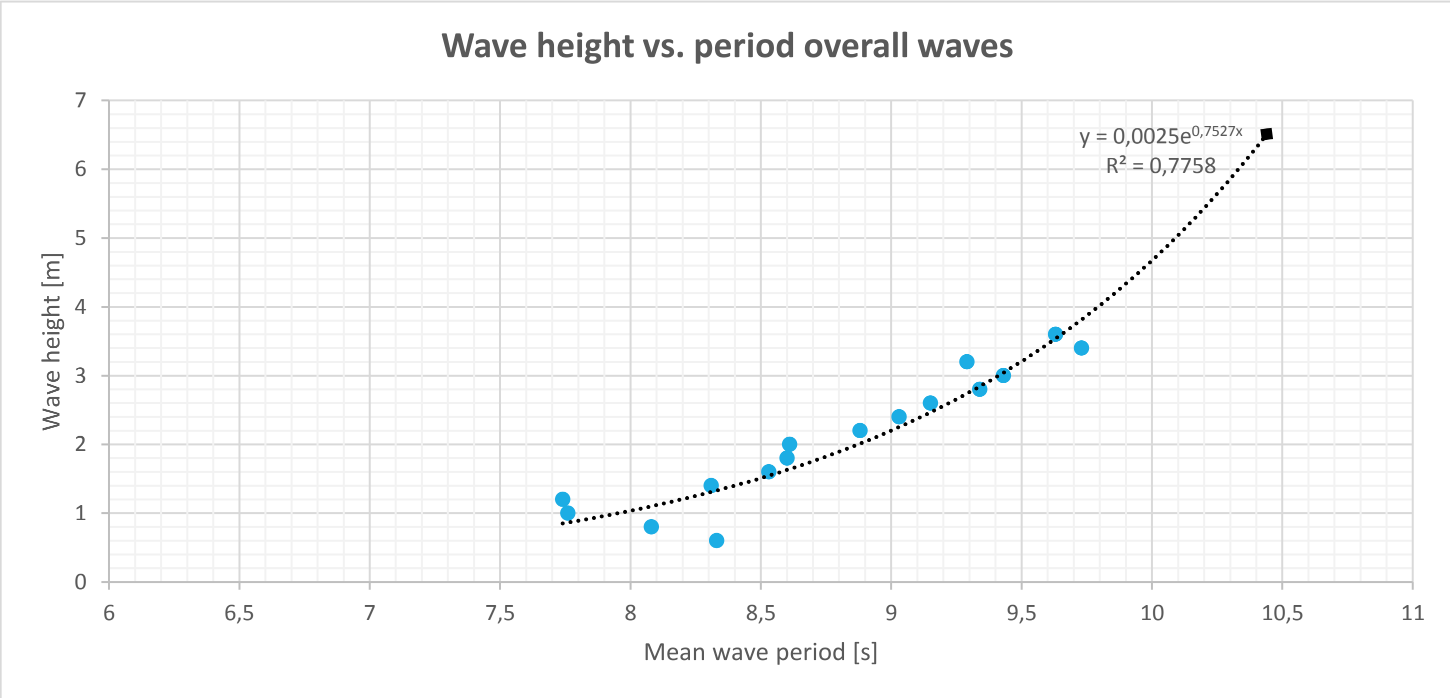


Probability	Return period	H_{ss} [m]	Confidence interval		T_m [s]	T_p [s]
0.2	5	2.82	2.770	2.890	13.5	15.0
0.1	10	2.9	2.833	2.982	13.6	15.1
0.066	15	2.92	2.862	3.033	13.7	15.2
0.05	20	2.95	2.881	3.067	13.8	15.3
0.02	50	3	2.929	3.164	13.9	15.4
0.01	100	3.03	2.956	3.228	13.9	15.4

= Estimation

Table B-8: Wave height vs. period overall waves (Coastal Resources Management project, 2005)

H_s [m]	T_m [s]
0.2	16.3
0.4	11.56
0.6	8.33
0.8	8.08
1	7.76
1.2	7.74
1.4	8.31
1.6	8.53
1.8	8.6
2	8.61
2.2	8.88
2.4	9.03
2.6	9.15
2.8	9.34
3	9.43
3.2	9.29
3.4	9.73
3.6	9.63



Probability	Return period	H_{ss} [m]	Confidence interval		$H_{mo,storm}$ [m]	T_m [s]	T_p [s]
0.2	5	4.83	4.165	4.729	5.22	10.05	12.04
0.1	10	4.72	4.414	5.149	5.10	10.02	12.01
0.066	15	4.93	4.560	5.409	5.33	10.1	12.10
0.05	20	5.09	4.664	5.602	5.50	10.15	12.16
0.02	50	5.62	4.998	6.257	6.08	10.25	12.28
0.01	100	6.06	5.316	6.800	6.55	10.35	12.40

(shaded) = estimation

Storm surges

Cyclonic storms are common in Sri Lanka. As read in Design wind velocity, sixteen cyclonic storms have happened during the last 135 year.

An abnormal rise in sea level caused by wind set-up and a low-pressure system is called a cyclone induced storm surge. A storm surge in combination with high waves can cause a lot of damage to coastal structures. Storm surges can also cause flooding in coastal areas.

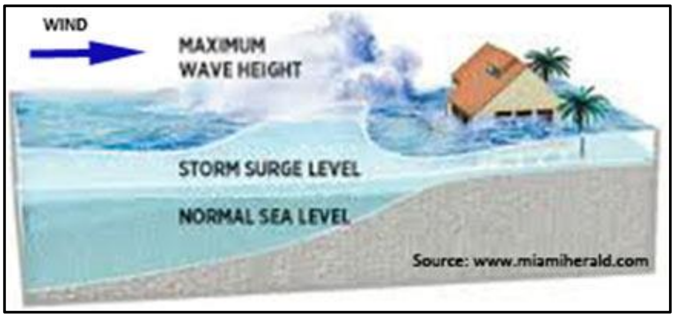


Figure B-12: Storm surge causing flooding

From the sixteen cyclonic storms, which have crossed Sri Lanka, fourteen have hit the North and East coast of Sri Lanka. The other two storms have hit Sri Lanka in the West and in the South coast of Sri Lanka. Those two storms have caused a storm surge on the South-West coast of Sri Lanka. From this we can conclude that storm surges are not common on the south-west coast in Sri Lanka. But because the consequences are large, they have to be taken into account.

A model to predict storm surge levels is made by the University of Peradeniya (Wijetunge, 2013). These simulations were made to model possible paths of a cyclone. The peak surge heights were taken from these simulations. In Figure B-13 the storm surge level for a 100-year return period are shown.

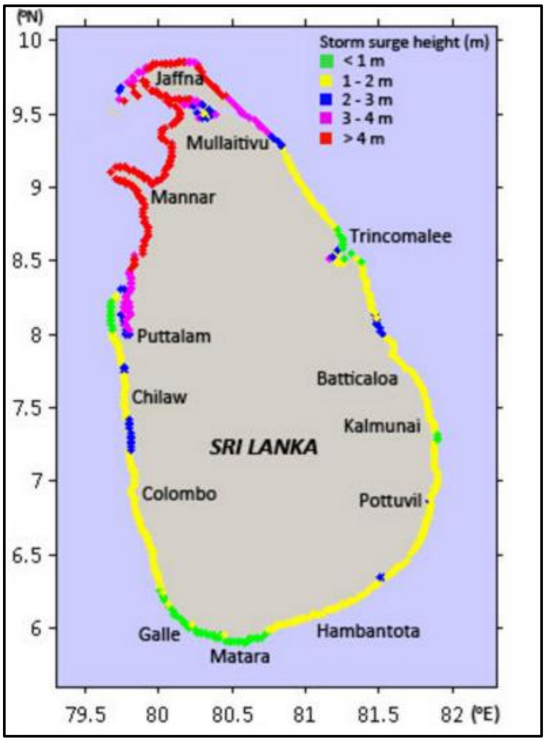


Figure B-13: Storm surge heights Sri Lanka, 100-year interval

Storm surge levels are depending on wind speeds, pressure drop, angle of incidence and bottom profile sea shore.

Estimated is that the storm surge level in Dodanduwa will be 1 m with a return period of 100 years. All results are shown in Table B-9.

Table B-9 Storm surge heights, 50 and 100-year return period.

Return period [years]	Return level [m]]
50	0.50
100	1

Boundary conditions wave modelling Swan One

Waves forces act as loads on hydraulic structures around the Dodanduwa harbour. Earlier presented wave characteristics are valid for deep water conditions. Waves transform when entering shallow water. For this reason, it is important to model the transformation of the waves from offshore to nearshore. The nearshore wave characteristics will be used as boundary conditions in the design phase

Swan One is used to model wave transformation from offshore to nearshore. The model simulates a John Swap spectrum, gamma 3.0 and tail f^{-5} based on input of the user. For this simulation the user should provide H_{m0} and T_p.

The significant wave height H_s differs slightly from H_{m0}. The Swan One model uses H_{m0} to set-up a John Swap spectrum, because H_{m0} is not known from the data a calculation is made. Longuet Higgins suggests $H_s = 0.925\ H_{m0}$ (Holthuijsen, 2007).

The peak period, T_p, is the period of the most energetic wave. This period determines the peak of the John Swap spectrum. From the data only the mean wave period known. The peak period is estimated by: $T_p = 1.20\ T_{m01}$ (Dominic Reeve, 2004). This estimation is quite accurate for wind waves. Dominant wind waves are mainly coming from the West and South-West-West, 259 degrees.

The spectrum of swell waves is more narrow, in this case the difference between the peak period and the mean period is smaller. When modelling swell waves of the overall ocean state most preferably an accurate spectrum file should be load in Swan One. The required spectrum file is not available, thus the generated John Swap spectrum is used to get a first estimate. For swell the following relation is used: $T_p = 1.11\ T_{m01}$ as estimate. Swell waves are from South-South-West and South direction, 191 degrees.

Most powerful winds are coming from the range 247.5 – 292.5 degrees during the SW monsoon period. Sri Lanka is divided in three wind zones. The South and West coast is classified as wind zone I. This corresponds to a three second gust velocity of 33 m/s. Using a gust factor of 1.8 results in a mean wind speed of 18.3 m/s.

Nearshore wave conditions

In total runs are made for different return periods and runs with -and without swell waves.

Bathymetry

Local bathymetry is estimated using chart from Defence Mapping Agency Hydrographic Center, because the bathymetry survey is not yet executed (Center, 1934). The bottom slope inside the harbour bay estimated based on words of the local supervisor. According Mr. Fernando the water depth near the existing breakwater, which is situated 150 m from the shoreline, is about 7 meters. During the multidisciplinary project bathymetry survey was carried out, but the results were not available yet to implement in this report.

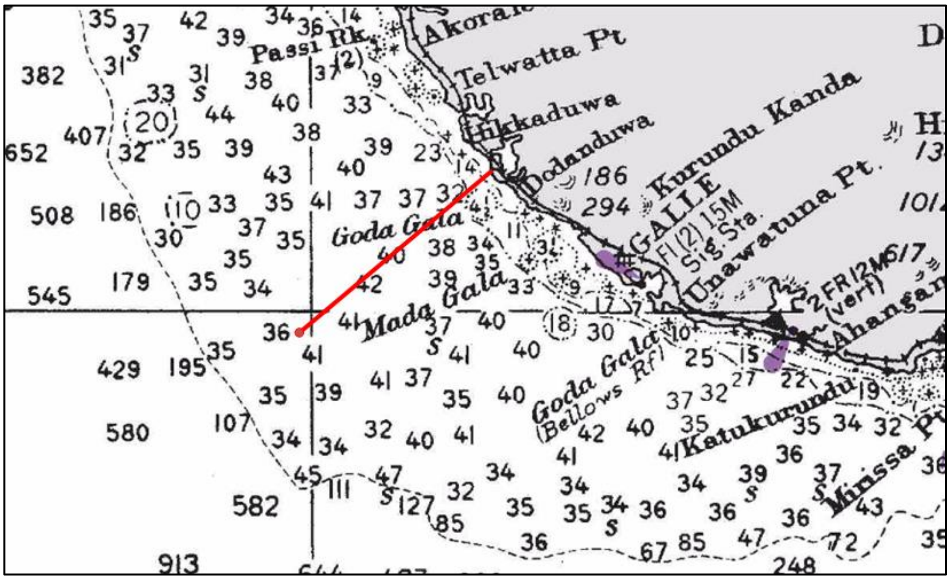
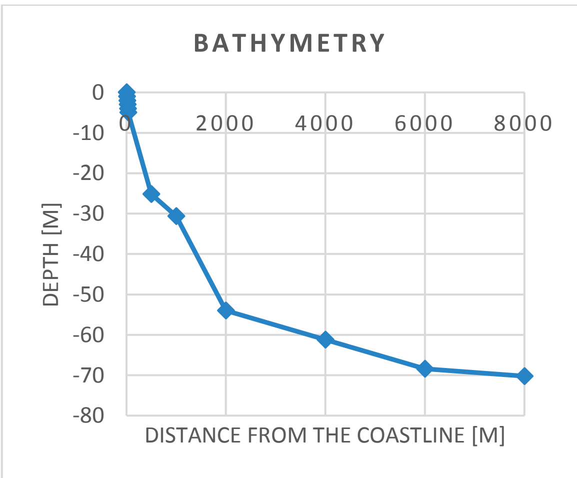


Figure B-14: Hydrographic chart South-West coast of Sri Lanka, depths are in Fathoms (Center, 1934)

As input Swan One needs the depth profile perpendicular to the coastline. As can be seen in the chart, a shoal is located near Dodanduwa. Especially waves from the South will be affected by this shoal before reaching the Dodanduwa harbour. Waves from the West are less affected.



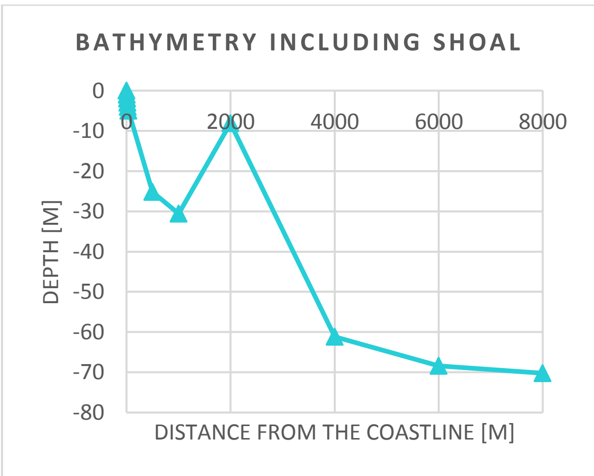


Figure B-15: Bathymetry

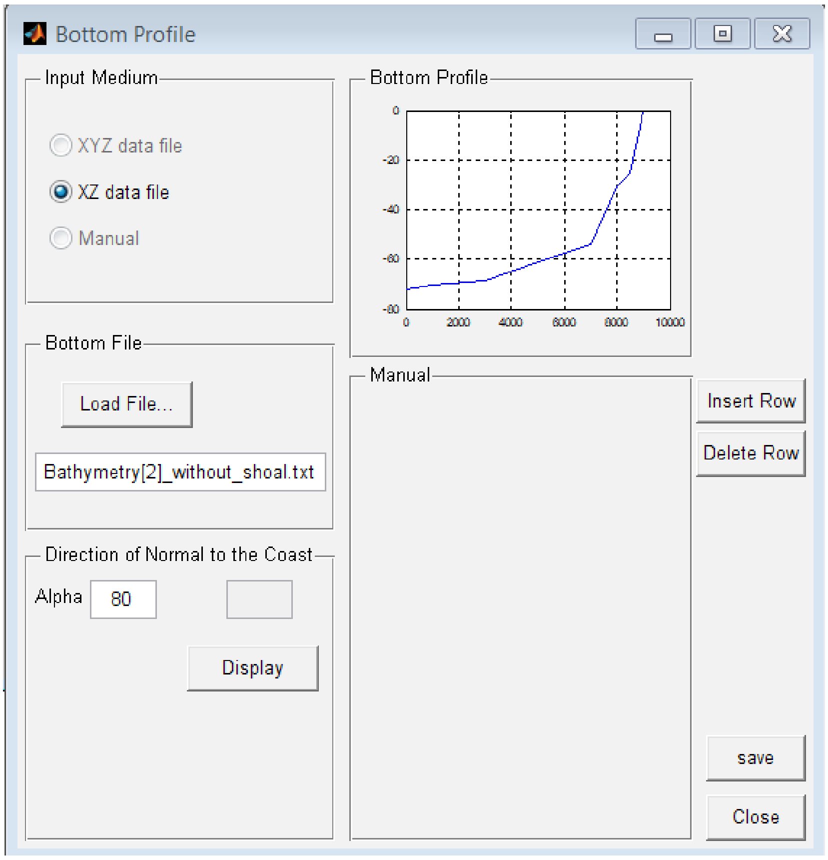


Figure B-16: Input bottom profile situation without shoal

Boundary conditions

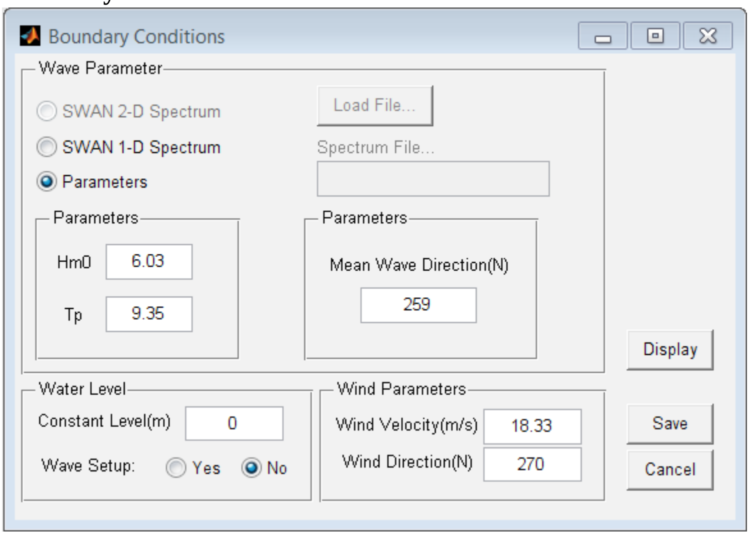


Figure B-17: Example input boundary conditions for 1/100 years sea waves

Output locations

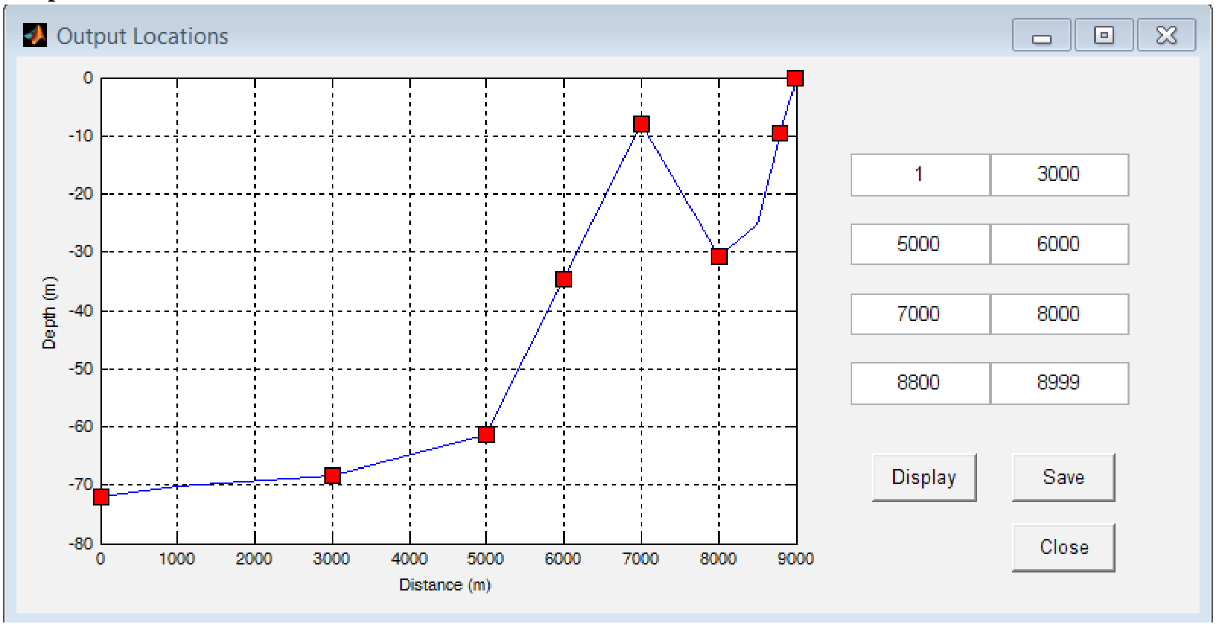


Figure B-18: Output locations runs including the shoal

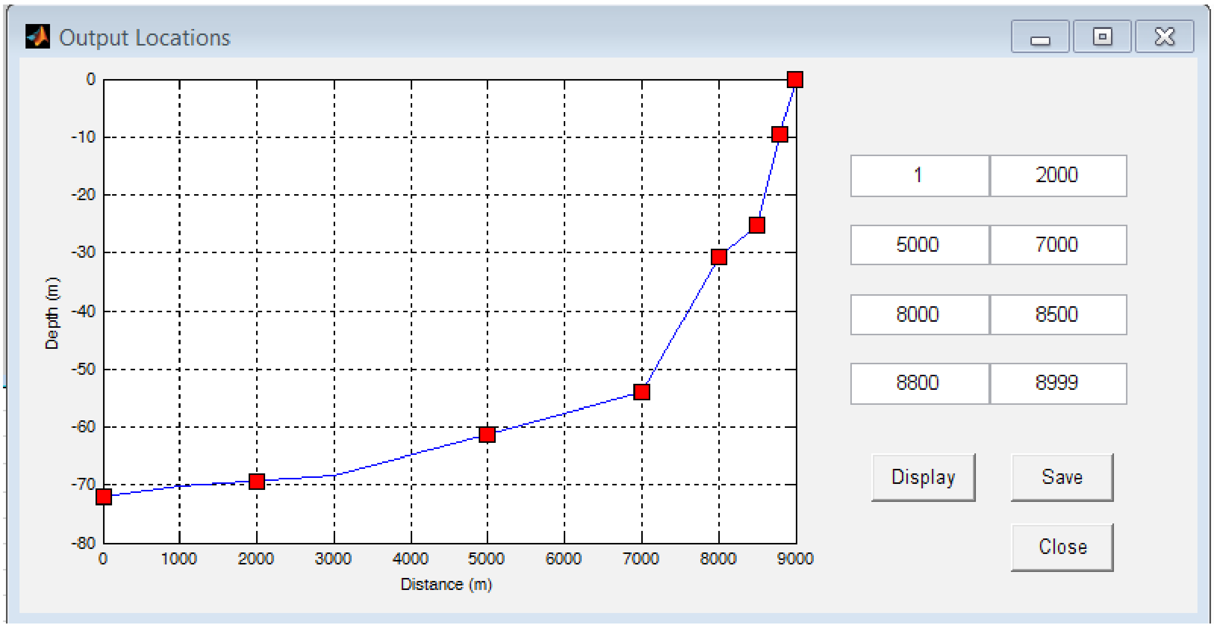


Figure B-19: Output locations runs excluding the shoal

Results
Results are presented at a nearshore depth of 9.6 meters, which corresponds to the proposed location of the new breakwater.

	Overall waves								
	Offshore (9000m, WL 72m)			Nearshore (200m, WL 9.6m)					
Return period				With shoal			Without shoal		
1/20	Hs	5.50	[m]	4.3	[m]		5.3	[m]	
	Tp	12.16	[s]	12.4	[s]		12.4	[s]	
	Tm-1	10.97	[s-1]	9.5	[s-1]		10.6	[s-1]	
	Tm01	10.08	[s]	7.9	[s]		9.3	[s]	
1/50	Hs	6.08	[m]	4.5	[m]		5.7	[m]	
	Tp	12.28	[s]	12.4	[s]		12.4	[s]	
	Tm-1	11.09	[s-1]	9.6	[s-1]		10.5	[s-1]	
	Tm01	10.23	[s]	8.1	[s]		9.1	[s]	
1/100	Hs	6.55	[m]	4.5	[m]		6.0	[m]	
	Tp	12.40	[s]	12.4	[s]		12.4	[s]	
	Tm-1	11.19	[s-1]	9.7	[s-1]		10.9	[s-1]	
	Tm01	10.33	[s]	8.2	[s]		9.8	[s]	

	Sea waves								
	Offshore (9000m, WL 72m)			Nearshore (200m, WL 9.6m)					
Return period				With shoal			Without shoal		
1/20	Hs	5.16	[m]	4.1	[m]		4.7	[m]	
	Tp	9.11	[s]	9.0	[s]		9.0	[s]	
	Tm-1	8.22	[s-1]	7.9	[s-1]		8.3	[s-1]	
	Tm01	7.58	[s]	7.0	[s]		7.6	[s]	
1/50	Hs	5.64	[m]	4.2	[m]		5.0	[m]	
	Tp	9.29	[s]	9.0	[s]		9.0	[s]	
	Tm-1	8.38	[s-1]	8.0	[s-1]		8.5	[s-1]	
	Tm01	7.73	[s]	7.1	[s]		7.8	[s]	
1/100	Hs	6.03	[m]	4.3	[m]		5.2	[m]	
	Tp	9.35	[s]	9.0	[s]		9.0	[s]	
	Tm-1	8.43	[s-1]	8.0	[s-1]		8.6	[s-1]	
	Tm01	7.78	[s]	7.1	[s]		7.9	[s]	

The difference between the situation with and without the shoal is significant. Swell waves will especially affected by the shoal. In this stage the data of the situation without shoal will be used as input for the designs.

C. Sediment Transport

C.1 Historical analysis of the Dodanduwa harbour

In this Appendix paragraph, the coastal changes of the harbour area of Dodanduwa will be researched. The period which is taken into consideration is from March 2003 until January 2016. During this period people started interfering with the coastal cell. The first part of this paragraph will be focussing on the Dodanduwa harbour, after which the surrounding area will be analysed.

The Dodanduwa harbour

The Dodanduwa harbour basin is located just upstream of the river mouth and is connected with the Indian Ocean and the Ratgama lake. Figure C-1 shows the situation in March 2003. The river mouth enters the Indian Ocean southwards, just eastwards of a rocky outcrop. The harbour basin is still accessible for boats, but a sand bank is beginning to form in the river mouth and sedimentation can clearly be seen in the river just upstream of the mouth, limiting the accessibility of the harbour.

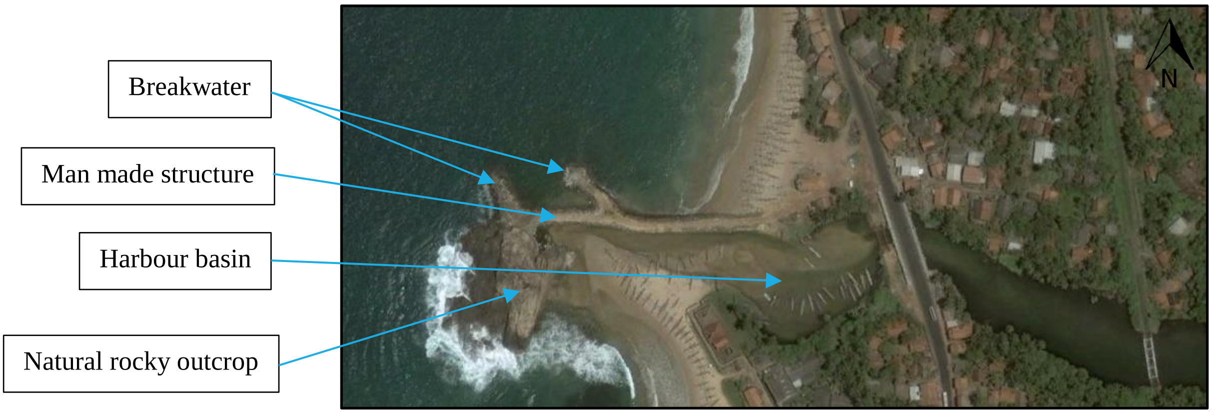


Figure C-1: Harbour of Dodanduwa in 2003

The formation of the sand bar is a natural process. Behind the rocky outcrop a shadow zone is formed in the Dodanduwa basin, which leads to a difference in wave set-up and sediment is transported towards the shadow zone. In the small bay south of this rocky outcrop, wave action creates a longshore current north. Sand is deposited where the currents due to the set-up difference meet and forming a tombolo/salient. In this case the sand deposition happens in front of the river mouth. The sand is further transported by the tide into the river channels and harbour basin if the ebb tide and river discharge is not strong enough to counter. This process eventually creates a big sand bar, closing off the river from the ocean and making navigation into the harbour impossible.

The manmade dam structure acts as the northern river bank. This dam is diverging the river mouth to the south. Two small breakwaters are attached to this dam, whose function remains unknown.

The situation in January 2005 is visualized in Figure C-2 on the right side. The first thing to notice is disappearance of the two breakwaters. The reason behind the removal of the breakwaters is unknown.

The harbour basin has decreased in size considerable in these two years and land has been reclaimed. Whether this is natural sedimentation or human interference is unknown. But it is unlikely that this is natural sedimentation. This part of the basin still has the largest depth when looking at the colour of the water and the placements of the boats (see Figure C-1). Natural sedimentation takes place in the main channel and not the reclaimed part. Human interference more likely to have caused this part of the basin to be reclaimed.

Figure C-2: Harbour of Dodanduwa in 2003(L) & 2005(R)

Figure C-3 shows the situation in August 2009 on the right. As can be seen in the figure a breakwater has been constructed starting from the rocky outcrop towards the sea. This breakwater has been constructed to provide safe entry and berthing for fishing boats. However, the breakwater is only partially built. Only 100m of the original 273m has been realized.

The river mouth is completely blocked by a sand bar and the harbour basin is not accessible anymore for boats. The closure of the river mouth is a natural process and is not only caused by the breakwater, but the construction of the breakwater worsened the sedimentation at the river mouth. As a result, boats are forced to land on the beach of Dodanduwa and surrounding areas.

As stated before the breakwater causes the bay to fill up with sediment. In the new situation again a shadow zone is formed and a difference in wave set-up leads to infilling of the bay. Furthermore, sediment which is transported in southern direction is trapped by the breakwater. This increases sedimentation of the bay as the breakwater traps sediment and blocks any sediment out of the coastal cell. The difference in sedimentation can clearly be seen when comparing the photos of 2005 and 2009 in Figure C-3.

Figure C-3: Harbour of Dodanduwa in 2005(L) & 2009(R)

Figure C-4 shows the situation in October 2012. The available data does not show new developments until 2012 when the dam that served as the north river bank has been demolished. It is unknown if nature or human interference is the cause. However this has caused the river to deflect to the north and entering the Dodanduwa bay. It is also unknown when the dam has breached. During October and November the river discharge is high enough to keep the river mouth open and to keep sufficient navigation depth. Also further upstream no sedimentation is visible. Somewhere between 2009 and 2012 the breakwater has suffered some severe damage. See the blue arrow in Figure C-4.

Figure C-4: Harbour of Dodanduwa in 2009(L) & 2012(R)

Figure C-5 shows the situation in March 2014. The river mouth is again closed off by a sand bar and upstream of the river accretion can be observed.

Figure C-5: Harbour of Dodanduwa in 2012(L) & 2014(R)

Since 2014 no large changes are observed.

The surrounding area

In this paragraph the surrounding area of the Dodanduwa harbour will be analysed for significant changes that have happened in the period of 2003 until 2013.

Situation before the construction of the breakwater

Between 2003 and 2009 there were no significant changes to the coastal system besides the seasonal fluctuation. The system is in a natural equilibrium.

Situation after the construction of the breakwater

After the construction of the breakwater in 2009 little has changed north and south of this breakwater. The bathymetry around Sri Lanka is characterized by a very steep slope, causing the waves to shoal and break close to the shore. The breakwater is therefore assumed to not interfere with the longshore current.

When looking at the Dodanduwa bay, the breakwater has caused a shadow zone as explained earlier. This leads to a set-up difference and causes a current to flow from the north towards the river mouth. This causes the beach to erode. The beach in front of the houses next to the road eroded rapidly. A revetment is placed in front of the houses to prevent further erosion, this can be seen in Figure C-6 & Figure C-7.

Figure C-6: Location of erosion

Figure C-7: The revetment in the harbour bay to prevent erosion

Further to the South it seems that a small stretch of beach is eroding. This started to occur after the construction of the breakwater. See Figure C-8.

Figure C-8: Clockwise: 2005, 2011, 2014 and 2012

C.2 SEDIMENT TRANSPORT PER MONSOON

In this section the direction of the sediment transport will be elaborated per monsoon. This section includes in particular the stretch North of Dodanduwa. The sediment transport between Dodanduwa and Galle will be discussed shortly. The coast North of Dodanduwa has a mean orientation around SW, the coast between Dodanduwa and Galle is shifted more towards SSW.

These results have been found with data from Argoss BMT and printed maps of the area. It should be noted that the data from Argoss does not include storms. These storms are typical during the South-West monsoon and come from the westerly direction.

Dodanduwa – Hikkaduwa

First the sediment transport between Dodanduwa and Hikkaduwa will be discussed. This part is most important for sediment that enters the Harbour bay.

South-West monsoon

During the South West monsoon, wind is coming from the West and has influence on the waves that hit the coast. The Direction of the dominant waves has changed more to the SW and this causes sediment transport to the North as well as to the South. The sediment that is transported to the South will gets trapped in the harbour bay. (Figure C-9)

One could still say that during this time of year the net transport is to the North.

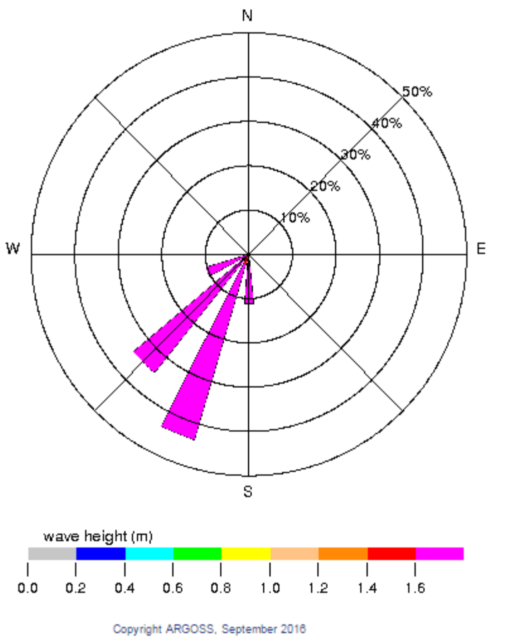


Figure C-9: Wave directions SW monsoon

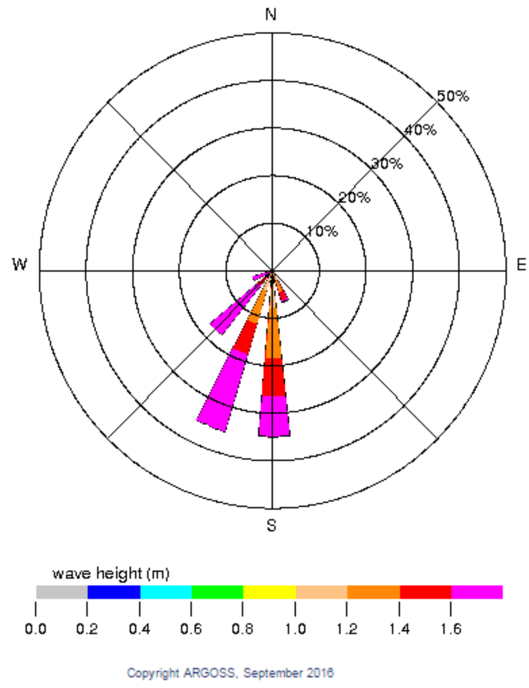


Figure C-10: Wave directions 2nd inter-monsoon

2nd inter-monsoon (October – November)

During this inter period, the wind calm down and the dominant wave direction is from the South-SSW again. Causing a northwards sediment transport. Occasionally waves are still from the SW and SWW, causing transport to the South. (Figure C-10)

During this period, the net transport is again towards the North.

North East monsoon (December – February)

During the North East monsoon, the dominant waves are coming from the South, thus leading to a transport to the North. Hardly any waves come from the SW and the sediment transport towards the South can be neglected. (Figure C-11)

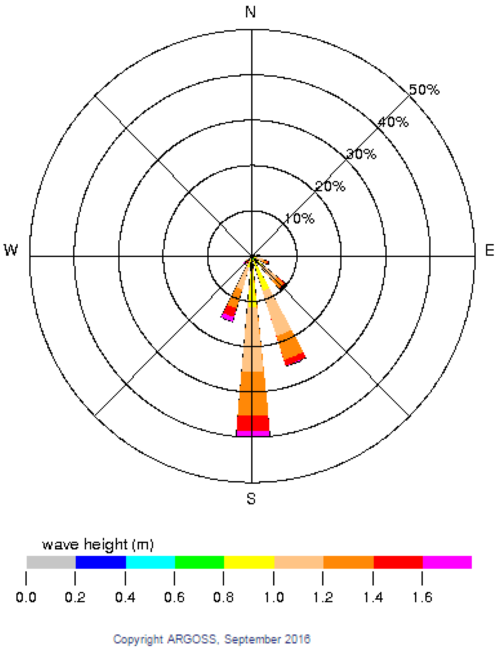


Figure C-11: Wave directions NE monsoon

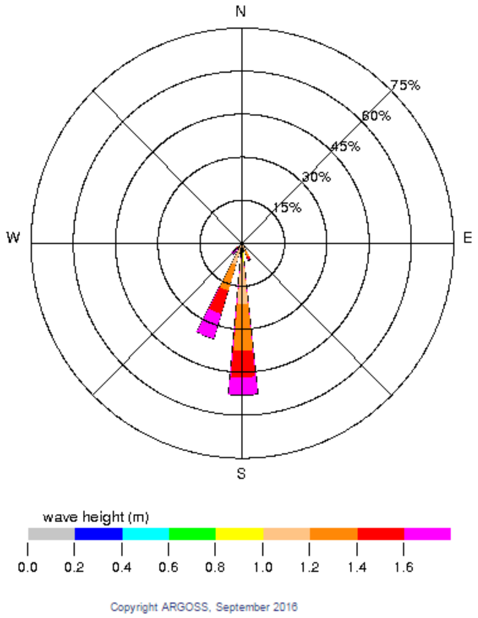


Figure C-12: Wave directions 1st inter-monsoon

1st inter-monsoon (March – April)
During the first inter-monsoon, Waves predominantly come from the South and SSW. These waves cause a northwards transport of the sediment. (Figure C-12)

Dodanduwa – Galle

The sediment transport between Dodanduwa and Galle will shortly be discussed in this section. This area is of less importance for the harbour of Dodanduwa. The coast is orientated between the Southwest and South-south-west.

During the South-West monsoon, the transport is hard to determine. As the coastline is between the two dominant wave directions. It can be stated that there is significant more transport to the south than between Dodanduwa and Hikkaduwa. Waves from storms also cause southern transport.

During the other periods the Southwest waves have calmed down and are predominantly South-south-west again. These waves give a sediment transport to the North.

Conclusion

The sediment transport North of Dodanduwa is towards the North predominantly. South transport occurs during the SW monsoon and 2nd inter-monsoon. Storms that occur during these monsoons also cause transport to the South.

The sediment transport South of Dodanduwa is predominantly towards the North. However, during the SW monsoon there is a significant sediment transport to the south caused by the waves from the SW direction and wind waves from storms.

D. AVAILABILITY OF CONSTRUCTION MATERIALS

Solutions presented in this report must be feasible to be constructed in Dodanduwa. Therefore, the availability of- and the transport options for the construction materials are briefly summarized.

Materials and industry

This first section elaborates on the main building materials, namely steel, concrete, sand, rocks and wood.

Steel

Ceylon Steel Corporation is the largest steel producer in Sri Lanka (Mushtaq, 2014). This corporation has a department situated in Galle. Government policies for the steel industry are not consistent, foreign investments are limited because of a strong local industry. However, the construction boom in the hotel industry might overgrow the capacity of steel production in Sri Lanka. Therefore, the government announced to make sure the policy is flexible and will be treated for every case.

Concrete

The Sri Lankan concrete industry is strongly dependent on the import of cement from the Southeast Asian region (Holy, 2014). Aggregates and water are available locally, aggregates will be discussed separately.

Sand

Sand is mined locally in the coastal and river areas; the largest source is the Kelani River near Colombo. This is due to the construction boom in de capital, in the Dodanduwa region other sources are available. Since many sand sources are overused, the current sand mining industry is not sustainable and causes a lot of erosional problems.

Rocks

Coastal structures like revetments and breakwaters consist of different types of rocks. The availability of the right amount of bigger sized rocks is limited. Sri Lankan quarries normally produce road metal and concrete aggregates, very few are equipped with the heavy drill equipment required for rock armour production. In case the quarry is located at a considerable distance from the project location or the roads are not sufficient for heavy transport, the material might be transported using sea transport (Young, Hayman-Joyce, & Hyeon Kim, 2012).

The supply of armour stones of similar harbour development projects has been researched, in order to map the sources of the rocks. In the table below a summary is given of the different projects and the origin of the rocks used in the project.

Table D-1: Inquiry of the origin of rocks used in similar harbour development projects

Project/Harbour	Origin of rocks	Comments
Hambantota Harbour	19# Mountain quarry, 2.5 km distance of project site.	Research has been conducted on two rock outcrops near an existing quarry. It was deemed possible to get the right amount and size of rocks.
Hikkaduwa Harbour	Granite rock in Wellawatta, 0.8 km from project area.	The rock was already used for quarrying, but used for extracting materials for road and building construction.
Port of Colombo	11 different quarries, 9 in the Colombo district and 2 in Gampaha district.	The large quantity of rocks needed could only be supplied combining 11 quarries. The transport of rocks to the site was mostly done using sea transport.

As can be seen in Table D-1, the origin of the rocks used in the projects is always located near the project site. A possible explanation is the high transport costs of the large quantities of rocks.

Wood

Wood is a common building material in Sri Lanka. Wood construction in Sri Lanka dates back to the pre- and post-historic periods but nowadays wood is mainly used for furniture and for finishing works on houses (roofs, railings and decorations).

As Sri Lanka is self-sufficient for wood. This does have effect on the forest cover as demands has been growing ever since. From 1956 it has been declining from 56% to 22% at present (Amarasekera, 2008).

The availability of wood is not a problem in Sri Lanka, but the resistance of the wood against the fresh and salt water must be guaranteed to prevent rotting. If the right type of wood can't be found in Sri Lanka it will have to be imported from nearby countries like India.

Local infrastructure

In this section the local infrastructure will be discussed. The local infrastructure consists mainly of roads, railways and waterways.

Roads

The Galle main road (highway) and the expressway between Colombo and Galle are the main roads near the project site.

The Galle main rd.

The Galle main rd. is a one lane road from Colombo to Galle. The length is around 120 kilometres and is located next to the ocean. Multiple cities are located along this road, i.e. Dodanduwa. This road is still widely used for transport between the different cities and Colombo, even though a new expressway has been constructed.

The road consists of one lane towards the North and one to the South. The road is mainly used by busses, bigger trucks and tuk-tuks. The traffic intensity is high. The use of this road is almost inevitable. Thus transport of construction material will always be partly over this road. The only alternative is transport by sea, which will be discussed later.

The expressway

The expressway has been built to decrease the traffic intensity on the Galle main road. The road is between Colombo and Galle located more inland and has a length of 133km. This road decreased the traveling time between Colombo and Galle from 3 to 1 hour. The road itself is a toll road, which might explain why people still prefer the old road over the expressway.

This road can be used to get faster nearby the location, however the use of the Galle main road is probably inevitable as mentioned before.

Railway

The railway between Colombo and Galle is located near the coast line and is one of the busiest rail services in the country. It is a two track railway between Colombo and Panadura, after which it becomes a single track.

The railway is located along the Galle main rd. and due to the time schedule the transport of construction materials via rail will be hard.

Water transport

The two closest ports to the project location in Dodanduwa are the Hikkaduwa- and Galle harbour.

The port of Galle is the most active port in the region. The Port is mostly used for yachts, but fishermen also use the harbour with boats bigger than the traditional vessels.

The port of Hikkaduwa is used for fishing and yachts. The port of Hikkaduwa does not have the space nor the capacity to store construction materials, while the port is located close to Dodanduwa. The port of Galle does have some space for the storage of construction materials but the distance covered by sea will likely be too inefficient. A solution might be to bring in construction materials by boats and transport them to the final location by truck.

E. SALINITY

E.1 SALINITY

In this chapter the possible saltwater intrusion of the water of the Ratgama Lake will be discussed. At first the current state of the salinity will be mentioned, after which the possible effects of changing the current situation will be discussed. The data available together with the involved stakeholders will be mentioned as well, after which the proposed measures are listed.

Current state

At the current situation the river mouth, which connects the lake with the ocean, is being closed naturally with the build-up of sand during the dry seasons. During this period the barrier prevents the salt water from entering the river and the lake. The wet season, starting in October/November until March, induces a water flow from the lake towards the ocean, breaching the natural build-up of sand and connecting the river and the ocean. During this period the flow of fresh water towards the ocean prevents salt water from entering the lake. Hence, the saltwater intrusion towards the lake is considered to be minimal (EML Consultants, 2016).

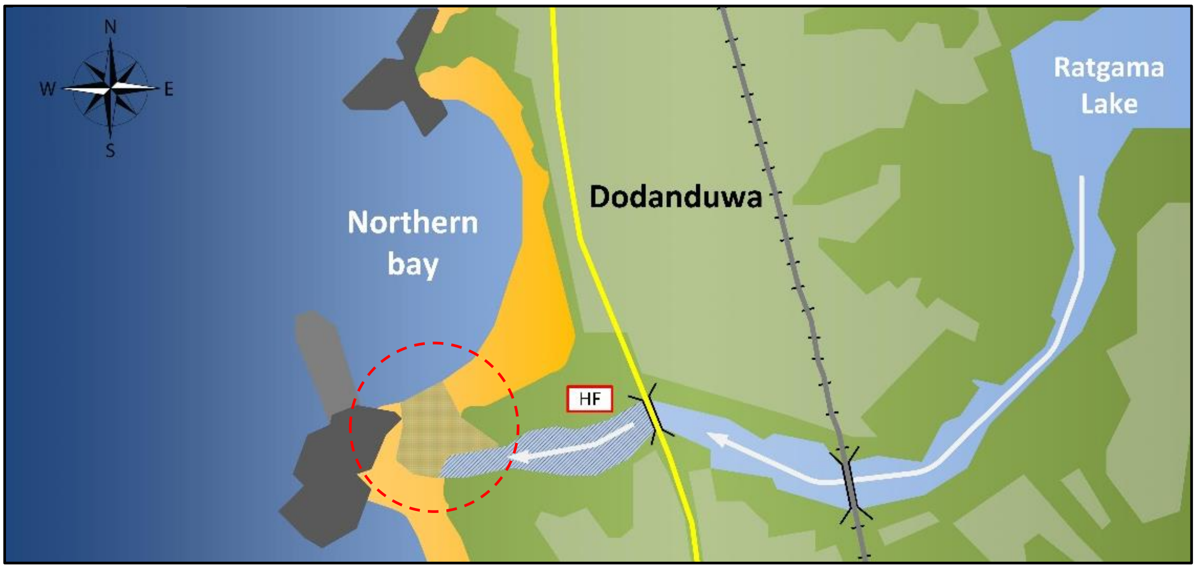


Figure E-1: Current layout of the river connection between the Ratgama Lake and the Indian Ocean. The natural barrier is indicated with the red circle.

Removing the natural barrier

In at least three of the proposed conceptual designs the natural barrier between the river and the ocean will disappear. In these concepts the sediments which causes the built-up of the natural sand barrier is being trapped or prevented from reaching the river mouth. By doing so, the river connection between the ocean and the lake will be permanent. During the wet season the river connection will not cause any severe saltwater intrusion, as the current from the lake prevents salt from entering the river.

The removal of the barrier will affect the saltwater intrusion of the river and the lake during the dry season. In the current state the barrier built-up during the dry season minimizes the salt water intrusion, after removal of this barrier the salt water is free to enter the river and the lake. The changing salinity of the water in the lake is therefore a potential risk, especially during the dryer season.

Data available

The potential saltwater intrusion of the water inside the Ratgama Lake has been mentioned in the Inception report made by EML consultants. However, next to presenting the possibility of the threat, no data is made available about the severity of the salt water intrusion. The potential saltwater intrusion will therefore only be described qualitatively, an in-depth detailed description is beyond the scope of this report. Advised is to conduct extra research regarding the possible saltwater intrusion, in order to gain insight in the likelihood and magnitude of this potential hazard.

Stakeholders

The saltwater intrusion of the water in the lagoon affects several stakeholders. The main stakeholders are the Irrigation Department (ID), the Agrarian Services Department (ASD) and the paddy farmers cultivating the area surrounding the Ratgama Lake.

The Ratgama Lake is being used by the paddy farmers for depositing the water used for drainage. Since the saltwater intrusion might affect the groundwater the ASD and ID are conservative against any changes concerning the current system. Before the PI&MU can implement the new design, the ASD and ID need to give their approval.

A meeting with the ID was proposed by the project team, but on advice of Mr. Fernando not held due to the sensitivity of the topic.

Environmental considerations

Saltwater intrusion in the Ratgama Lake can have considerable influence on the local biodiversity. The land surrounding the lake is being used for agriculture by paddy farmers, they are using the lake for the disposal of their irrigation water. The salt water intrusion might affect the groundwater as well, resulting in possible negative effects on the agriculture.

Next to the effects on the paddy farmers, the saltwater intrusion possible affects the local ecosystem of the Ratgama Lake area. There is a natural variation in the ability of plant and animal species and ecosystems to tolerate salinity. Effects of an increase in salt water content could vary from minor effects, for weaker plant structures, until the extinction of an animal or plant species in the lake area (Nielsen & Brock, 2003). These effect strongly depend on the severity of the saltwater intrusion. Since no data is available regarding the local ecosystems and the severity of the intrusion, it is strongly advised to conduct extra research on the local ecosystems and the severity of the saltwater intrusion.

Proposed measures

If the approved design includes the permanent opening of the river mouth and research proves the potential saltwater intrusion, some measures must be taken to limit the effects of this phenomenon. First of all, extra research must be conducted so the effects of the intrusion are mapped. Second, a construction must be built as a measure against the intrusion.

Gathering extra data

In order to map the possibility of the salinity, extra data is required. By gathering data of the (geo-)hydrological conditions and the total discharges of the river during the year it might be possible to estimate the severity of the intrusion. In addition to the technical data, the sensitivity of the flora and fauna regarding the possible salinity could be researched.

Sluices

One of the options to prevent the saltwater intrusion in the Ratgama Lake is to construct sluices inside the river. The sluices will operate in the same manner as the natural barrier. During the wet season when the flow of water towards the ocean is preventing the salt water from entering the lake the sluices can be opened. During the dry season the sluices can prevent the inflow of salt water towards the lake by closing them.

The downside of using sluices are the construction costs. The PI & MU did not take the construction of sluices into the project budget, while the IR&ASD are not going to pay for it as well.

Saltwater barrier

The construction of a saltwater barrier in the river could be an effective method for limiting the salt water intrusion. By building a barrier in the river the heavier salt water is not able enter the river. This solution is based upon the difference in density between salt- and fresh water. Differences in temperature of these waters influence this behaviour, for the applicability of this method further hydrological investigation is needed.

The saltwater barrier is not a guaranteed applicable option, also the effects remains rather unknown.

Dam
The construction of a dam would form a more permanent solution compared to the sluices but, if constructed correctly, works the same. The advantage of a dam is that it doesn't need to be operated during the change of season. A disadvantage of a dam is that the height is not adjustable so it might not be effective for many years.

Conclusion

In the current lay-out of the river connection between the Ratgama Lake and the Indian Ocean saltwater intrusion is not an issue. A natural seasonal blockade is limiting the saltwater intrusion over the year. By redesigning the river entrance the built-up of this blockade might be prevented, resulting in potential salinity issues. The severity of these issues remains unknown, extra research is required for mapping the effects of the saltwater intrusion.

Based upon the information available some prevention measures have been proposed; installing sluices in the river, the construction of a salt water barrier and building a dam. Detailed description of the proposed measures is due to the lack of information not possible and is beyond of the scope of this project.

F. STAKEHOLDERS AND INTERESTS

Governmental parties	
Ministry of Fisheries and Aquatic Resources Development[2] (MF&ARD) and Project Implementation and Monitoring Unit (PI&MU)	The Ministry wants to improve the harbour to enhance the operational effectiveness and efficiency; and is therefore the client. The project is run by the PI & MU, which is part of Ministry and formally can be seen as the client where the other stakeholders deal with. The goals and proposed project of the client were discussed in the paragraph 12.3.
Coast Conservation and Coastal Resources Management Department (CC&CRMD)	All coastal projects must be approved by the CC&CRMD. Since the project is located in the coastal zone, the CC&CRMD has to give prior approval of the design before the construction can take place. The CC&CRMD aims to protect the coastal zone against negative alternations
Irrigation Department (ID)	The area around the Ratgama lagoon is used by paddy farmers. The water itself is supplied by the rain; excessive water is transported by irrigation channels that mouth in the Ratgama lagoon. This lagoon is falling under the jurisdiction of the ID. Hence, any development in the project area shall have the approval of the ID particularly when developments interfere with the lagoon flows. When the lagoon is accessible all year, salt intrusion might cause salinity of fresh water inside the lagoon. The ID wants to ensure that the water in the lagoon and groundwater is not affected by the new harbour design. The ID discusses all matters concerning the lagoon with the dept. of Agrarian Services.
Agrarian Services Department (ASD)	Surrounding the Ratgama lagoon there are multiple hectares of agricultural land which use the lagoon for discharging irrigation drainage water. The agricultural land is under governance of the ASD. The interest of the ASD is the same as of the ID, they want to ensure the current situation of the agrarian activities will not be affected by the new harbour design.
Ceylon Fishery Harbour Corporation & government authorities (CFHC)[3]	The CFHC is an institution responsible for the management of fishery harbours and anchorages. The focus of the CFHC is to ensure the safety and functionality of the Dodanduwa harbour. The interest of the CFHC is to deliver the fishing community sound infrastructure and facilities. The CFHC has to approve all the changes made to the design of the harbour.
Companies	
EML Consultant (Pvt) Ltd	The inception report in which the project and work is described is written by EML consultants. EML has been awarded by the PI & MU with the contract for the feasibility study and detailed design for the upgrading of the proposed upgrade of the Dodanduwa fishery harbour. The interest of EML is making profit and delivering a high quality design, matching their reputation as consultancy firm.
Local parties	
The local fishermen	The local fishermen are the main users of the Dodanduwa harbour. The interests of the fishermen are: having a safe and easy access from sea to harbour and vice versa and enough anchorage places for their boats.
Local residents	There are about 6800 fishermen families depending on the harbour. Any changes or adjustments to the harbour area may not affect the residents of Dodanduwa in any way.
The paddy farmers	The area surrounding the Ratgama lake is used by paddy farmers. These farmers use the lagoon for mouthing their irrigation channels. As mentioned earlier, the changes to the Dodanduwa harbour might cause the lagoon to be connected to the sea all year round. The salt water intrusion might affect the

[2] The Project Implementation and Monitoring Unit will oversee the project

[3] Also includes government authorities concerning the fisheries infrastructure development and operation

water in the lagoon. In this case the groundwater might get affected as well, having a negative influence on the crops grown by the farmers. The interest of the paddy farmers is to have no negative effect of the new harbour design.

Other parties

The tourist industry

Tourism is an important part of Sri Lankan economy. Dodanduwa is located near the touristic city of Galle. If the project results in facilitates that might attract tourist the industry needs to be informed about the new situation. Their interest is to profit from the project.

G. CONCEPTUAL DESIGN

G.1 PROCESS ANALYSIS

All processes taking place in the harbour will be analysed and visualised in this chapter. The goal of this process analysis is gaining extra insight on the requirements for the harbour structures. What is the required width of the channels? What type of anchorage, and where should it be placed? How large does the fish unloading facility have to be? These questions will be answered with the process analysis.

In Figure G-1, the main chain of processes in the harbour is shown. Fishermen will enter the harbour, returning with fish. After entering the harbour the fishermen need to unload the fish and possibly refuel, anchorage must be provided afterwards. Before leaving the harbour again maintenance might be required. The process can be seen as a cycle that can be repeated several times a week (vallams) or about once a week (multi-day vessels).

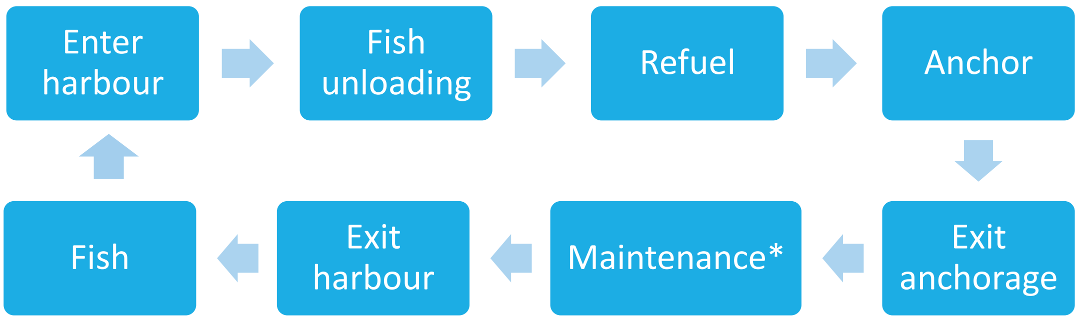


Figure G-1: Process cycle for fishing activities.

The different processes can be compiled in three groups: Navigation, Preparation and Anchoring. The possible facilities for these processes will be discussed in the following paragraphs.

Navigation

The fishing boats must be able to navigate in and out of the harbour. Inside the harbour, routes are required to reach all facilities. The channels towards the fishing preparation facilities should be appropriate for two vessels passing each other. With a ship width of 4.6 meters plus a safety margin this means a minimal width of about 11 meters for the navigation channel. At places close to the beach, slopes towards a minimum channel depth of 3 meters are chosen to have an angle of approximately 11.3° (1:5). According to (Yell & Riddell, 1995)* and (Maertens, 2009) underwater slopes of 1:5 in semi-active water conditions with a coarse sand profile are generally stable. Slopes at places where outcrops are present can be 1:3. To guarantee stability in more rough water conditions (possibly stronger river current, return current) revetments will be needed to assure slope stability and prevent erosion. Beach slopes are chosen to be 1:6 because vallams need to land on the beach. This angle will provide a stable slope.

The total width of the main channel is therefore 41 meters. The entrance of the harbour will be wider: the water depth at the harbour entrance is assumed to be 7 meters (Appendix P.1). The breakwaters inner slopes are conservatively estimated to be 1:3, resulting in the total width of the harbour entrance being 53 meters. An overview of the different kind of slopes and distance from the shore to the navigation channel is shown in Table G-1. In the concepts this is presented as a red striped line. Crossing a such 'line' during navigation will not directly cause problems due to the unloaded ship draught of 1.6 meters and the sediment buffer in the 3 meter depth requirement.

Table G-1: Overview slopes and distance shore to navigation channel

	Slope	**Distance to 3.0 m draught [m]**
Breakwater	1:3	9.0
Revetment	1:5	15.0
Beach	1:6	18.0

The navigation channels between the jetties are normally one way, this is because waiting times are short and space is limited. Vessels can pass each other in the main channel. The navigation between the jetties is treated separately for each area in the next paragraphs. For an impression of the navigation channel, see Figure G-2.

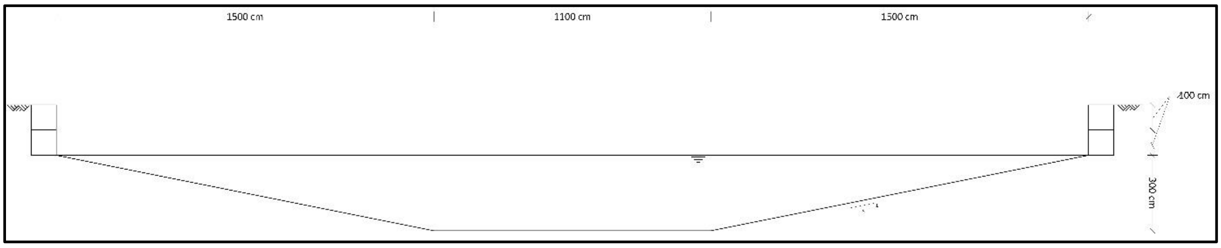


Figure G-2: Cross-section of the river

Northern bay
The maximum amount of jetties placed in the northern bay is three, the case in concepts 1 and 2. Form jetty, heart to heart, about 50 meters are needed. Total width of 8 vessels is 36.8 m, some extra space between the boats and jetty width of 2 meters, add up to roughly 40 meters. The at least residual 10 meters can be used for navigating and manoeuvring into mooring position.

Southern bay
The smaller southern bay has only enough space for two jetties. at the narrowest part these jetties are 44 meters apart. The narrowest part is in between the end of the T-shaped jetties, where the boats will not anchor in the navigation channel. The maximum amount of vessels that can anchor at the base of the T-shaped jetties is about 5 boats per jetty, leaving room for a 15 meter wide navigation channel.

Harbour basin
Only in the third concept the harbour basin is used for anchorage, a quay wall is there used for the anchorage of up to 24 multiday vessels.

Preparation

After a day or days of fishing, the fishermen will come to the harbour to unload their fish. In all concepts this will be done at the harbour facility in the harbour basin. Two options are possible to facilitate unloading of fish.

The first option is to construct a jetty. An alongshore jetty will facilitate unloading for the vallams. Finger or T-jetties can facilitate unloading for multiday vessels. Those jetty types are necessary to maintain enough draught for the multiday vessels.

The second option is to create a quay wall. The depth at the quay wall can be dredged to required draught. Ten-meter-long finger jetties will provide unloading at the quay wall. The jetties are attached to the quay wall. Multiday vessels are moored double-sided at the finger jetties during unloading. This to provide perpendicular unloading of the boats to increase unloading speed and decrease unloading time (Wijdeven, 2015). Also, a quay wall with finger jetties will reduce the required length of the quay wall. This is shown in Figure G-3.

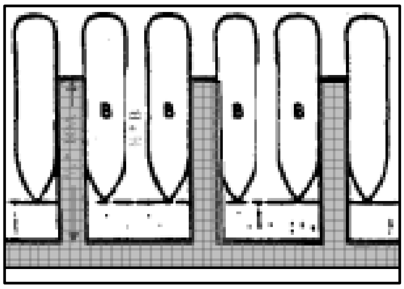


Figure G-3: Double-sided, perpendicular mooring

Becised unloading at the jetty/quay, these structure must also include a refuelling point. It is important to isolate unloading and refuelling from the unloading. This is done for safety and hygienic reasons.

Maintenance and repair of vallams can be done on the beach. The multiday vessels need to go to another harbour for maintenance and repair. Neighbouring harbours like Galle and Hikkaduwa have facilities for repair and maintenance.

Capacity fish unloading facilities
Close to the harbour facilities, fish unloading facilities are constructed. Several options for such structure were mentioned before; the needed capacity is defined in this paragraph. Based on two researches (Amarasinghe, 2014) (Chathurika & Dissanayake, 2016) on multi-day vessels and their fish catch, an average catch of 600 to 5000 kg per boat trip was found. In Figure G-4 the trip duration and average catch are shown. For the design of the quay wall facilities and unloading time of two hours is assumed, based on two expert stakeholders, (CFHC, S. Bandara and EML, C. Fernando). The unloading is done manually, sorting the fish on the deck of the ship. Fish is either directly sold or sold in plastic crates to truck drivers. The plastic crates contain about 35 kg. fish each.

Table 6. Duration of multiday fishing trips

Harbour	Duration of fishing trips by type of multiday craft (days)		
	34-35 ft	36-38 ft	40 ft and above
Negombo	-	16	25
Beruwala	7	18	30
Galle	10	16	16
Kudawella	9	17	19
Trincomalee	-	20	29
All harbours	**9**	**17**	**24**

Table 10. Avrege fish landings by multiday crafts

Harbour	Average fish landings by type of multiday craft (kg per trip)		
	34-35 ft	36-38 ft	40 ft and above
Negombo	763.0	1570.0	2856.0
Beruwala	2700.0	4185.0	5072.0
Galle	2006.0	1824.0	1859.0
Kudawella	2696.0	3745.0	5233.0
Trincomalee	855.0	1445.0	2291.0
All harbours	**1804.0**	**2553.8**	**3462.2**

Figure G-4: Two tables from (Amarasinghe, 2014), showing de duration and catch per fishing trip of different sized boats.

The harbour is designed for a future fleet of 100 multi-day vessels, which have a maximum length of 11.9 meters. Currently Dodanduwa is well-known for its daily caught fresh fish. From research (Amarasinghe, 2014) it is seen that the amount of high quality fish from a catch, decreases if the boat makes a trip longer than one week. It is assumed that fishermen will keep this standard and most likely will purchase multi-day vessels from the smaller category.

To determine the amount of fishing vessels unloading each day, it is assumed that the trips have a duration of one week. Spreading the 100 vessels over the week results in approximately 15 vessels unloading per day. Assumed is that most fishermen will return after a last night or day of fishing. The 15 vessels will most likely arrive in a period of 5 hours: in the morning (05:00-10:00) or evening (19:00-24:00). Therefore it is estimated that the facilities should be able to process 3 boats per hour. With every boat taking approximately 2 hours to unload, a maximum of 6 boats should be facilitated at the fish unloading area. This estimation is done quite conservatively: choosing a 100 multi-day vessels, long processing time, sort duration of fishing trips and peak hours in morning and evening.

Anchorage

There are three options for the anchorage of multiday vessels. Common in Sri Lanka is to anchor the vessel in the middle of the harbour. This is done by putting down the anchor in the bay or attach your boat to another already anchored boat. This is shown in Figure G-5. No structure is required to facilitate berthing. Fishermen use a small boat to get on land. The second option is to construct jetties to facilitate anchorage. After 18 meter of the jetty from shore the depth is sufficient (-3.0 m) to moor multiday vessels. These boats can anchor directly at the jetty or attach their boat to a moored boat at the jetty. The final option is to construct a quay wall as anchorage place. In the harbour bay this is possible at the leeside of the breakwater. In the harbour basin, a quay wall for mooring can be constructed in the extended part.

Figure G-5: Putting down the anchor in the middle of the harbour

Every concept should meet the required capacity of vessels. In this paragraph is described how the vessels should be moored per option.

Option 1 (Figure G-6, *left*)
In option one and two the boats are just anchored in the harbour bay. Without any structure to moor at. The boats are attached to each other and anchored to the bottom of the bay. Space between the boats is estimated to be 0.60 meter. This give a width of 4.60 meter per boat. Twelve meters of space should be taken between the anchored boats to make navigation possible. For this option it is possible to place 30 boats per 50x50 meters in an ideal situation. But in practice this will be lower, approximately 25 boats (Bandara, 2016).

Option 2 (Figure G-6, *middle*)
Option two is the construction of a jetty. The length of the jetty depends on the available space in the concept and the number of vessels which need to be anchored. Boats will be moored side by side like as in option 1. But now the first boat will be moored to the jetty. It is now possible for the fishermen to reach shore by walking over the jetty. Fishermen who attach their boat to another boat must walk over the other boats to reach the jetty. There is a maximum of four boats next to each other. This to make sure it is still possible to navigate between the boats. The space for navigation between the boats on the jetties is twelve meter. In this option it is possible to place 8 boats per 14 meter of length of jetty. 29 boats in an area of 50x50 meter. But in practice this will be lower, approximately 25 boats.

Option 3 (Figure G-6, *right*)
Option three includes a quay wall for anchorage. The length of the quay depends on the space available in the concept and the number of vessels which need to be anchored. Boats will be moored side by side. Where one boat will be moored to the quay and other boats against the moored boat. This with a maximum of four boats in line. Per 14 meter length of quay it is possible to moor 4 boats. In this option the total number of boats which can be moored depends on the length of the shore and not the area of water.

It is possible in the concepts to combine options. For a better configuration of berthing places. In practice boats will moor where there is space. This will give a density of 25 boats per 50x50 meter (Bandara, 2016).

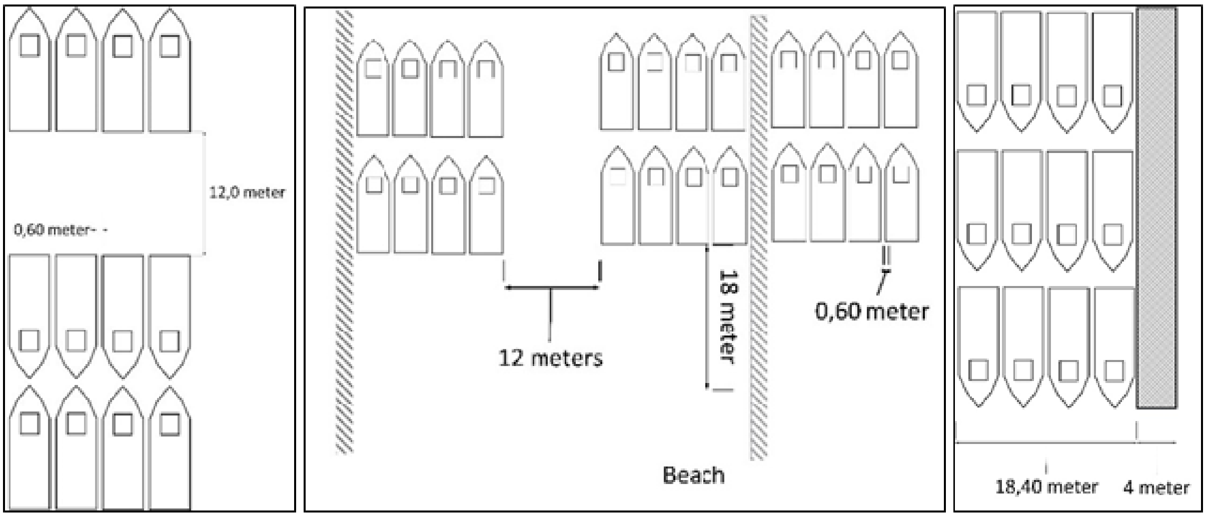


Figure G-6: Distribution of the vessels for options 1(left), 2(middle) and 3(right)

G.2 Harbour Lighting

For every conceptual design lighting is considered to be essential: a significant part of the fishermen catches fish at night or arrives at the harbour late at night. Night fishing is done because fish are less responsive to the nets that are thrown into the water during the night. There are two main functions of the harbour lighting: first is safety at the harbour entrance, second the navigability inside the harbour basin.

For every concept navigation lights at the harbour entrance are needed to assure a safe entrance. Concept 4 will need more navigation lights, since this concept has two harbour entrances. The ocean might still be rough around the harbour entrance, a navigation light will provide guidance for the fishermen returning in the dark.

The second function of the navigation lights is guidance in the harbour itself and around the jetty facilities. Some general street lighting along the jetties should be sufficient for safe navigation in the harbour bay at night. The lighting of the Dikkowita fishery harbour is shown as example in Figure G-7.

Figure G-7: The Dikkowita fishery harbour, largest of its kind (BAM International)

For every jetty in the harbour bays two lights are suggested: one at the start and one at the end of the jetty, both aimed at lighting the jetty structure, similar to the lightning in the Dikkowita fishery harbour. Around the onshore harbour facilities lights are suggest along the riverside, spaced at approximately 30 meters.

To safe costs on the long term and increase the sustainability of the solution sun powered lighting might be favoured. The cost of electricity is relatively high in Sri Lanka, therefore an investment in sustainable resources has a lower return period.

H. RIVER DIVERSION

In the current situation, the river connecting the Ratgama Lake and the Indian Ocean mouths in the northern bay, as can be seen in Figure H-1. The current in the river is depending on the season, during the dry season the river current is to be negligible, only the tides cause some current in the river. In the wet seasons the rainfall accumulating in the Ratgama Lake will cause a current in the river towards the ocean.

In three of the conceptual designs, the position of the river is adjusted. In the next sections the diversion of the river for the different concepts will be discussed.

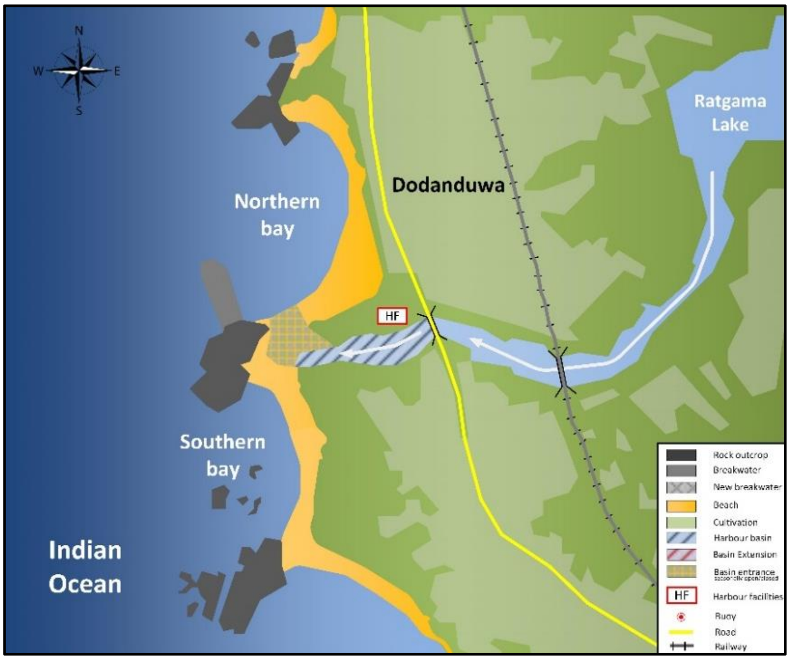


Figure H-1: Current lay out of the river

Diversion of river towards the southern bay

The river as in the current situation mouths in the northern bay. In two of the conceptual designs the river is diverted towards the southern bay, namely conceptual design 2 and 3.

Moving the river

In conceptual design 2, the northern bay area is expanded towards the South. The river is diverted towards the South, mouthing in the southern bay. The diversion starts in front of the motorway bridge, and goes all the way towards the new connection with the Indian ocean, as visualized in Figure H-2.

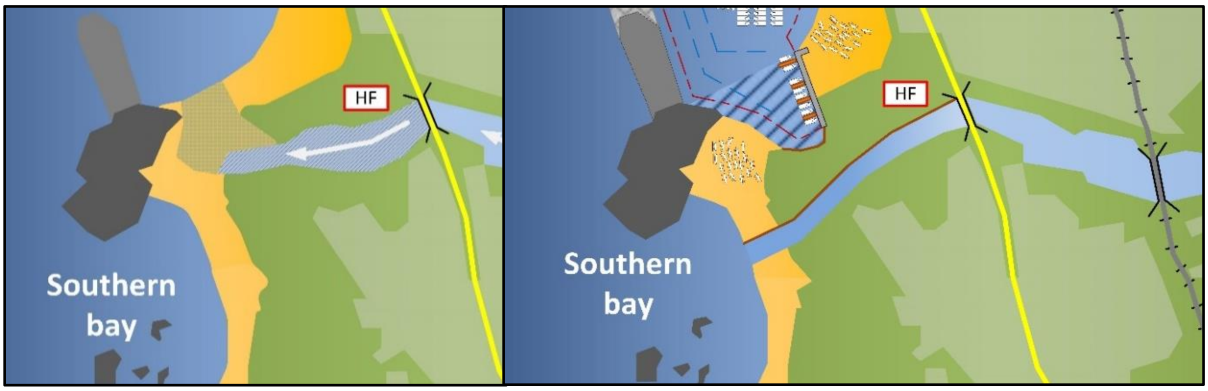


Figure H-2: Current position of the river (l) and the redesigned channel (r)

The transition of the river towards the new designed trajectory would require a large amount of groundwork. The new channel is designed to have a width of 30 meter and a length of 230 meter. The estimated depth of the channel is 1.0 to 2.0 meter. The border between the river and the extended northern basin is relative small, an additional structure in the form of a revetment is placed to prevent this border from breaching due to the wet seasonal discharge from the Ratgama Lake. The position of this structure is indicated with the brown line in Figure H-2. Only the outer bend is protected, bend flow will cause more erosion at this border.

The before mentioned structure is placed on the riverbank. A geotextile will serve as filter layer on which a layer rocks is placed. It can also be built up with multiple layers instead of a geotextile. The calculation for the revetment can be found in Appendix L together with a visualisation.

Since the harbour basin is moved towards the northern bay, the river does not necessarily has to be accessible for fishery boats. Maintaining a minimum draught of the river is therefore not mandatory. This new location of the river mouth is possibly subjected to seasonal erosion and accretion of sediment; the accreting sediment might close the river mouth during the dry season. In case the river mouth is closed off by sand, during the wet season the water level in the lake will rise and the river mouth will be flushed open again.

Diversion of the river mouth
The river in the third conceptual design is only relocated at the river mouth. At the current location the river bends towards the North into the northern harbour bay. In the conceptual design the river mouth is closed off and a new mouth is created in the extension of the current river trajectory, as can be observed in Figure H-3. The new river channel is protected from currents and ships by a revetment. This revetment creates a fixed slope and guarantees the minimum draught in the navigation channel. For the calculation of the revetment, see again Appendix L.

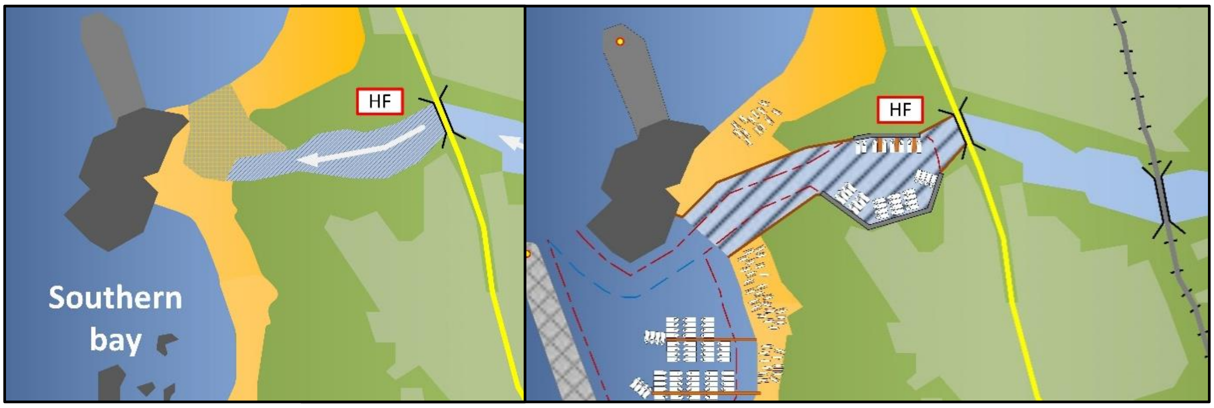


Figure H-3: Current position of the river (l) and the redesigned river mouth (r)

The groundwork of this concept is convenient, the soil dredged for opening the river mouth towards the southern bay can be used for closing of the current river mouth in the North. The new entrance of the river at the southern bay will be used for accessing the harbour basin. Therefore, the design draught of the river including the river mouth must be maintained at 3.0 meter. Currently the river is shallow and dredging must be done to achieve this minimum draught.

The river mouth is located at the southern bay, which is protected against incoming waves by breakwaters, therefore sedimentation is expected to be less of an issue. The new alignment of the river will decrease the seasonal sedimentation of the river entrance. No sediment will be supplied from the North. The amount of sediment supplied by the river and tidal influences are still unknown. To ensure the minimal draught for the multi day vessels, maintenance dredging is necessary once every few years.

Connecting the bays
In conceptual design 4 the bay at the northern and the southern side of the river mouth are extended towards the land. In the new design the river is therefore being shortened by approximately 50m. The groundwork for this concept is mostly consisting of removal of soil in order to reclaim the land near the current river mouth. The layout of the shortened river is presented in Figure H-4.

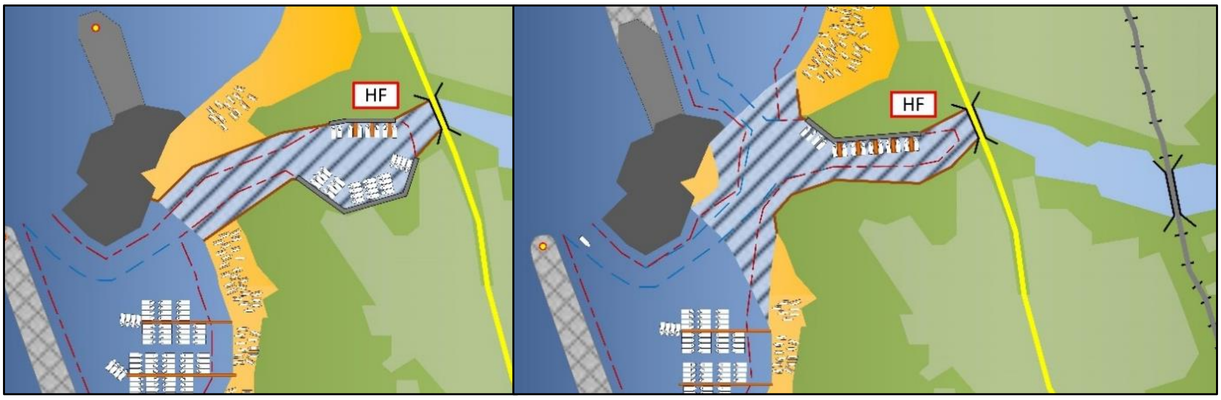


Figure H-4: Current position of the river (l) and the shortened design of the river (r)

In this conceptual design the current harbour basin will be kept in place. The new design is believed to cause some sedimentation problems. To which extend the river or tide influences this is still unknown. In order to ensure the draught of 3.0 meter, dredging of the basin and the basin entrances every few years is inevitable.

Again, the riverbanks are protected against erosion. This revetment is the same as for the diversion of the river mouth.

I. FUTURE SEDIMENT

In this section the impact on the coast line due to the different concepts will be discussed. What is discussed in the following sub-sections is based on engineering judgement and must be verified with research and or modelling.

I.1 CONCEPT 1: 'OPTIMIZED CURRENT SITUATION'

By extending the current breakwater and strengthening the northern outcrop, a new coastal cell is created. Due to the northern breakwater, sediment that would come from the north is trapped. This will accrete and widen the beach just north of the rock outcrop. From a process meeting with C. Fernando (Appendix P.1), no sediment bypasses the most southern rock outcrop and breakwater. The only source of sediment could be from the river or brought in by the tides.

The river must be dredged to a depth of 3.0 m to ensure the draught, for the longer term. From a bathymetry measurement which can be found in Appendix R.3 it is found that the river only has a maximum depth of 2.0 m. The transition from 2.0 m to 3.0 m will affect the river flow, this will cause sedimentation due to the sudden increase in water depth. However, the sediment supplied by the river is assumed to be small (CHANNA BRON).

The tidal influence on the current situation is unknown. With $h_{Hw} > h_{Lw}$ in the new situation the system will be a flood dominated system, see Figure G-2, thus importing sediment. The influence of the new channel might be small to negligible. The dimensions of the channel $(lxbxd)$ might be too small to significantly alter the tidal flow. A second possibility is that the system behaves as a rigid column. If this is the case the sediment input is negligible.

The new orientation of the breakwaters effects the coast line just north. The new breakwater creates a shadow zone behind it. Depending on the wave orientation, the shadow zone creates differences in wave set-up. These differences will locally effect the beach and cause sediment transport to the South. The sand will eventually accumulate in front of the northern breakwater/outcrop. See Figure I-1.

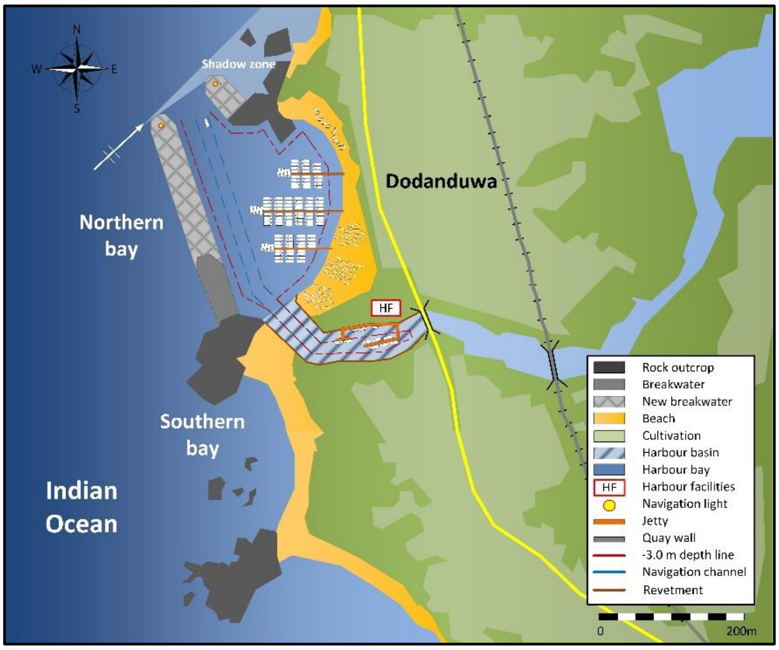


Figure I-1: Overview concept 1

I.2 CONCEPT 2: 'NORTHERN SOLUTION'

By diverting the river to the South one creates a big basin that is used as harbour. The breakwater orientation is the same as mentioned in concept 1, I.1. The possible coastal changes due to the breakwaters are already discussed in this section.

By diverting the river to the south, it is assumed the natural variability of the river continues. See the red circle in

In this concept there is only tidal influence on the sedimentation in the harbour. The harbour basin can be schematized as a short basin with a closed end. Possibly the system behaves as a rigid column and, as discussed in I.1, the sediment import is negligible.

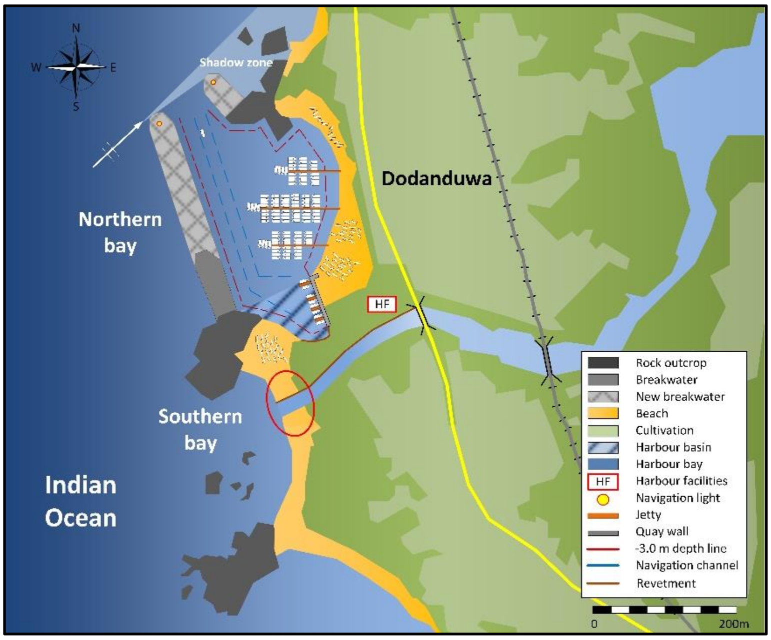


Figure I-2: Overview concept 2

I.3 CONCEPT 3: 'SOUTHERN SOLUTION'

The coastal changes due to the construction of the new southern breakwater are assumed to be minimal. As no sediment is transported past the most southern outcrop and the sediment from the North is already trapped in the northern bay by the current breakwater.

For the tidal and river influence, the same accounts for this concept as discussed in I.1 and I.2.

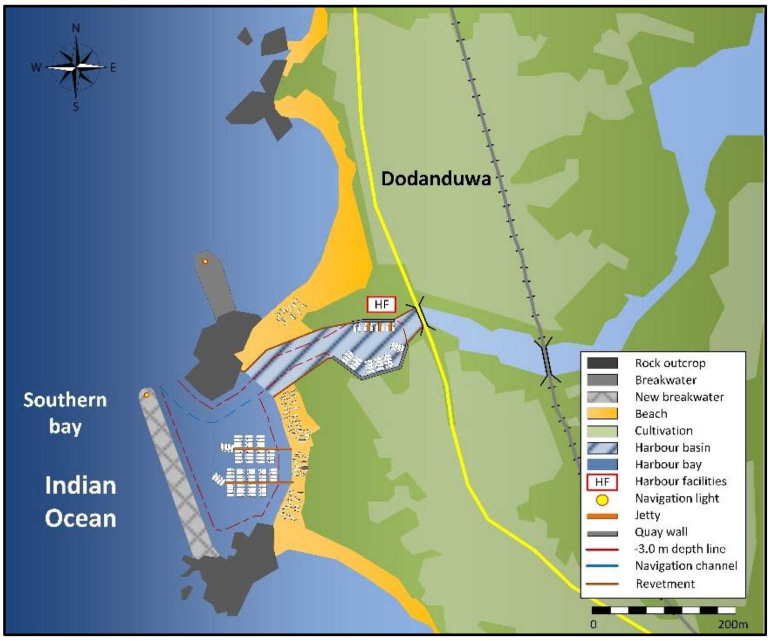


Figure I-3: Overview concept 3

I.4 CONCEPT 4: 'EYES ON THE FUTURE'

Again, the possible impacts on the coastal system due to the breakwaters are discussed in I.1 and I.2

The river could deposit sediment up until the outcrop. The tidal wave, which travels from north to south, might pick up the sediment and transport is towards the southern outcrop where it is trapped and deposited. This could decrease the draught in this area significantly. See the red circle in Figure I-4.

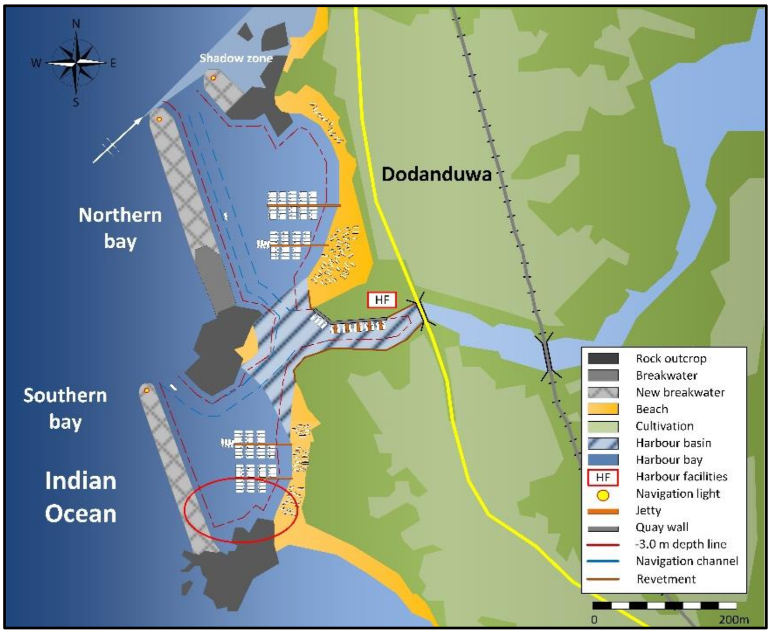


Figure I-4: Overview concept 4

I.5 CONCEPT 5: 'NULL-OPTION'

In this concept the beach in front of the sediment trap will accrete as sediment is stopped here. Possibly there will still be sedimentation at the river mouth due to diffracted waves and tidal influence. However it will be significantly less than before the construction of the sediment trap.

I.6 SUGGESTIONS

It is suggested to do modelling of the different concepts to see the effect of the river and tide. If modelled right, the different influences of the river and tide can be seen and conclusions about the working of the concepts can be drawn.

Secondly, it must be investigated for the third concept if the natural variability of the river continues. Concept 3 assumes the presence of the natural variability, therefore no action against salt intrusion are considered.

J. QUAY WALL DESIGN

The quay wall of the Dodanduwa harbour functions as a ground retaining structure facilitating the fish unloading and refuel facilities. In this paragraph the preliminary design of a gabion quay wall is elaborated. The gabion quay wall is a gravity based soil retaining structure. Alternatives to gravity based structures are various types of sheet piles or pile-based structures. The exact soil profile at the location of the quay wall is uncertain, it is known that the subsurface consists of rock overlain by sand. The combination of an unknown thickness of the sand layer and the rock head beneath favours a gravity based structure. A gravity based structure requires a solid foundation, sheet piles require sufficient embedment. A gabion quay wall is preferred over for example concrete alternatives due to the abundance of rock in the Dodanduwa area. The breakwater structures require a considerable amount of quarry rock; smaller stone sizes can be used for the gabion quay wall. Locally lots of gabion embankment structures are encountered, proving the feasibility of this solution. In general, a concrete quay wall will be more expensive than a gabion quay wall, which favours the gabion wall as well.

J.1 DESIGN CALCULATIONS & ASSUMPTIONS

Since no soil investigation is done, the soil parameters are conservatively estimated from EN-1997: table 2b (Normcommissie 351 006 Geotechniek, 2012). An overview of the estimated parameters for the material properties are shown in the table below. The proposed geometry is shown in Table J-2 and Figure J-1.

Table J-1: Estimated material properties

Material properties, X_k		
Critical friction angle	30	degrees
Sand, 'dry' density	16	kN/m^3
Sand, saturated density	20	kN/m^3
Salt water	10.25	kN/m^3
Gabion density	16	kN/m^3
Gabion porosity	0.3	-
Coeff. active earth pres.	0.33	-
Coeff. passive earth pres.	3.00	-
Coeff. wall friction	0.58	-
N gamma	20.09	-
i gamma	0.27	-

Table J-2: Proposed geometry

Geometry		
Harbour, water depth	-3.0 [LLWL]	m
Groundwater level	-2.25 [BGS]	m
Height quay wall	6.25	m
Width quay wall	5.0	m
B, out of plane	1.0	m
Embedment depth	1.0	m
Height dry sand	2.25	m
Height sat. sand	4.0	m

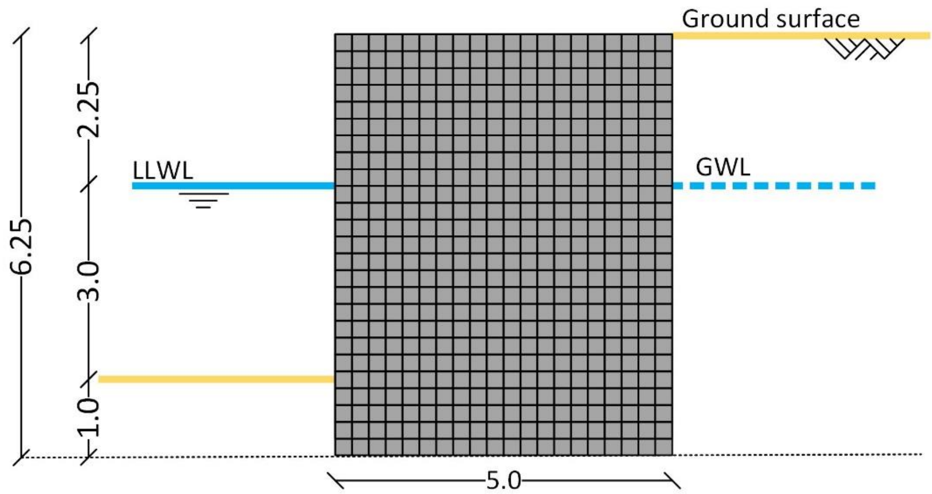


Figure J-1: The geometry of the gabion quay wall (dimensions in m)

Next to the assumed parameters it is assumed that the gabion quay will function as one structure. For this preliminary design no failures of the structure itself are taken into account. This is a valid assumption if the installation of the gabion quay is done properly and all elements are of a high-quality standard. The geometry is determined through iteration.

Four soil failure mechanisms are discussed: sliding, bearing failure, overturning and slip failure. The slip failure will be discussed qualitatively; software limitations prevent a relevant quantitative analysis in this report. The mooring forces are considered in the jetty design, Appendix K. Concept 3 requires additional structures to coop with the mooring forces of the anchored multi-day vessels in the harbour basin.

Safety factors

To obtain design values, partial safety factors are used per EN-1997 Appendix A. These factors are shown in the tables below, copied from EN-1997 (Normcommissie 351 006 Geotechniek, 2012). NEN Table A.13 shows the factors used for the resistance in the calculations for 'Sliding' (Horizontaal afglijden) and 'Bearing capacity' (Draagkracht). For the 'Overturning' calculations, partial factors for the favourable (Gunstig) and unfavourable (Ongunstig) permanent (Blijvend) loads are applied, shown in NEN table A.1 presented below.

Tabel A.1 — Partiële factoren voor belastingen (γ_F)

Belasting	Symbool	Waarde
Blijvend		
Ongunstig [a]	$\gamma_{G;dst}$	1,1
Gunstig [b]	$\gamma_{G;stb}$	0,9
Veranderlijk		
Ongunstig [a]	$\gamma_{Q;dst}$	1,5
Gunstig [b]	$\gamma_{Q;stb}$	0

[a] Aandrijvend.
[b] Weerstandbiedend.

Tabel A.13 — Partiële factoren (γ_R) voor de weerstand van grondkerende constructies

Weerstand	Symbool	Verzameling		
		R1	R2	R3
Draagkracht	$\gamma_{R;v}$	1,0	1,4	1,0
Horizontaal glijden	$\gamma_{R;h}$	1,0	1,1	1,0
Grondweerstand	$\gamma_{R;e}$	1,0	1,4	1,0

Sliding

Sliding occurs when the net horizontal pressure is higher than the friction below the structure. The relevant pressures and forces acting on the gabion quay wall are shown in Figure J-2.

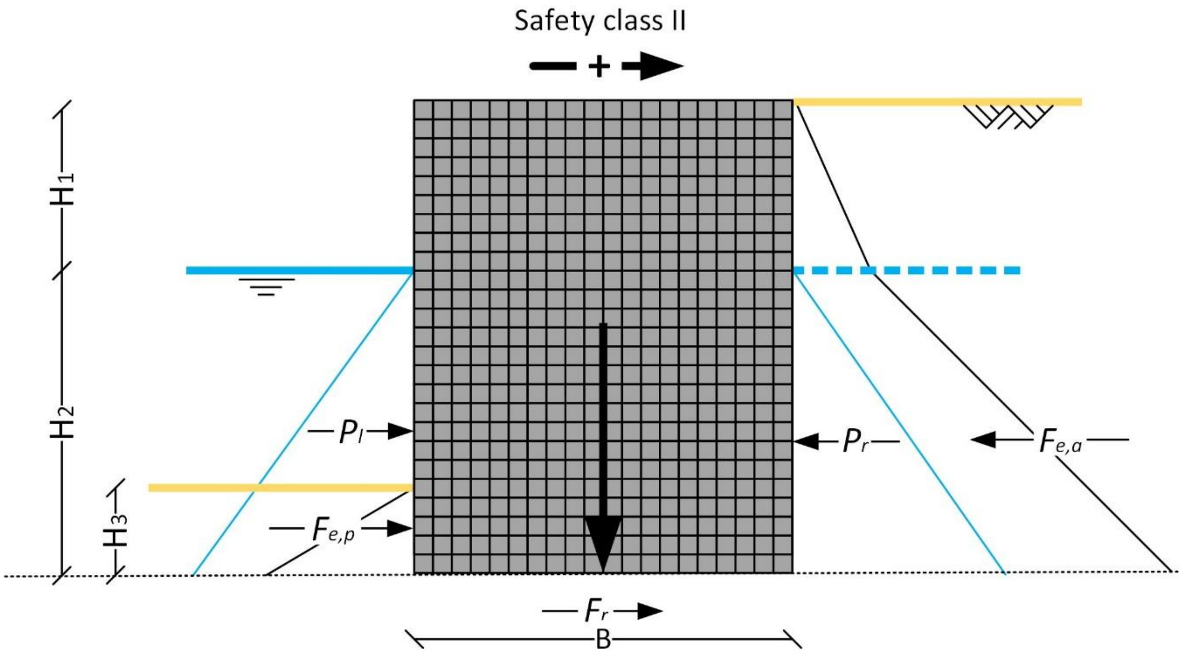


Figure J-2: All relevant pressures, forces and dimensions for the failure mechanism 'Sliding'

Loads

Horizontally the gabion quay wall is loaded by the water pressure [Pr,P_l]and active soil pressure [$F_{e,a}$]. Due to the high permeability of sandy subsurface and the gabion quay wall, the water pressures on the left- and right-hand side are taken equally. For the final design calculations a water level difference must be taken into account. The active soil pressure is only present on the right-hand side. The passive soil pressure [$F_{e,p}$] is only present on the left-hand side. All calculations are shown below.

$$F_{e,a} = K_a \cdot (0.5 \cdot L \cdot H_1^2 \cdot \gamma_{sd} + H_1 \cdot \gamma_{s,d} \cdot L \cdot H_2 + 0.5 \cdot L \cdot H_2^2 \cdot \left(\gamma_{s,w} - \gamma_w\right) = -96\ kN \quad \text{J-1}$$

$$P_r = 0.5 \cdot L \cdot H_2^2 \cdot \gamma_w = -82\ kN \quad \text{J-2}$$

$$F_{e,p} = K_p \cdot 0.5 \cdot L \cdot H_3^2 \cdot \left(\gamma_{s,w} - \gamma_w\right) = 15\ kN \quad \text{J-3}$$

$$P_l = 0.5 \cdot L \cdot H_2^2 \cdot \gamma_w = 82\ kN \quad \text{J-4}$$

Resistance

The friction between the base of the quay wall and the soil beneath results in a resisting force [F_r]. The friction between the sand layer and the gabion quay wall is approximated with the critical friction angle of the sand.

$$F_r = \delta \cdot \left(H \cdot B \cdot L \cdot \gamma_{gab} - H_2 \cdot B \cdot L \cdot \gamma_w \cdot (1-n)\right) = 206\ kN \quad \text{J-5}$$

Factor of safety

To determine the factor of safety unfavourable loads are multiplied with 1.1 and the resistance against sliding is divided by 1.1. This results in a factor of safety (FoS) of 187 / 89 = 2.1.

Bearing failure

Bearing failure occurs when the loads on the soil surrounding the structure fails. The bearing capacity of the soil can be estimated through Brinch-Hansen. The relevant stress and forces are shown in Figure J-3.

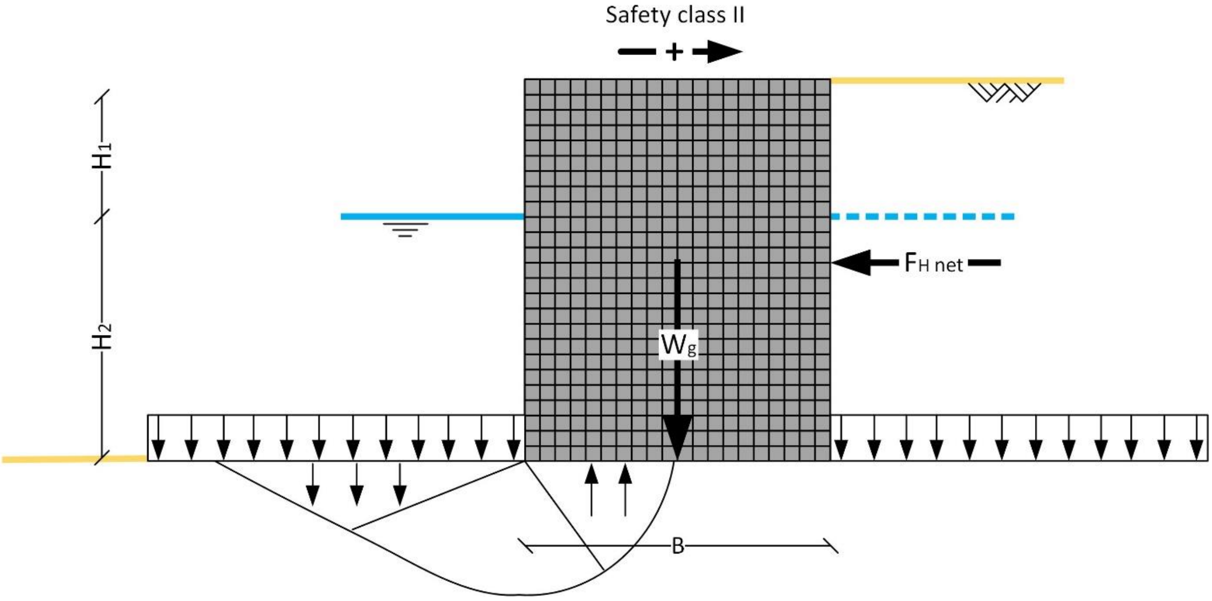


Figure J-3: All relevant pressures, forces and dimensions for the failure mechanism 'Bearing failure'

Loads

The soil is loaded by the self-weight of the gabion structure [W_g]. For the estimation of the vertical force a density of 16 kN/m^3 is assumed, typical for gabions filled with strong to very strong rock like basalt of granite (Chris R.I. Clayton, 2014). The porosity of gabions is relatively high; namely 30 percent. During the lifetime of the gabion it is expected that the porosity will drop and the pores are filled by sand. Conservatively the porosity of 30 percent is used to determine the buoyancy forces on the quay wall.

$$W_g = H \cdot B \cdot L \cdot \gamma_{gab} - H_2 \cdot B \cdot L \cdot \gamma_w \cdot (1 - n) = 357\ kN \quad \text{J-6}$$

Resistance

The sandy soil is assumed cohesion less, a realistic and risk averse assumption. Both the embedment of one meter and the self-weight of the soil are taken into account, both contributing to the bearing capacity [F_{bc}]. The distributed load on the right side of the quay wall, being the self-weight of the soil column, is not taken into account since it is not present at the left side.

$$F_{bc} = \left(i_q \cdot q \cdot N_q + i_\gamma \cdot 0.5 \cdot B \cdot \left(\gamma_{s,w} - \gamma_w\right) \cdot N_\gamma\right) \cdot B \cdot L\ = 884\ kN \quad \text{J-7}$$

Factor of safety

To determine the factor of safety unfavourable loads are multiplied with 1.1 and the resistance against bearing failure is divided by 1.4. This results in a factor of safety (FoS) of 631 / 393 = 1.6.

Slip failure

Slip failure occurs when local failure eventually results in a failure plane through the weakest part of the soil. To evaluate a possible slip failure, one has to check numerous slip planes against the factor of safety. Both Fellenius and Bishop provide methods for hand calculations in case of slope slip failures.

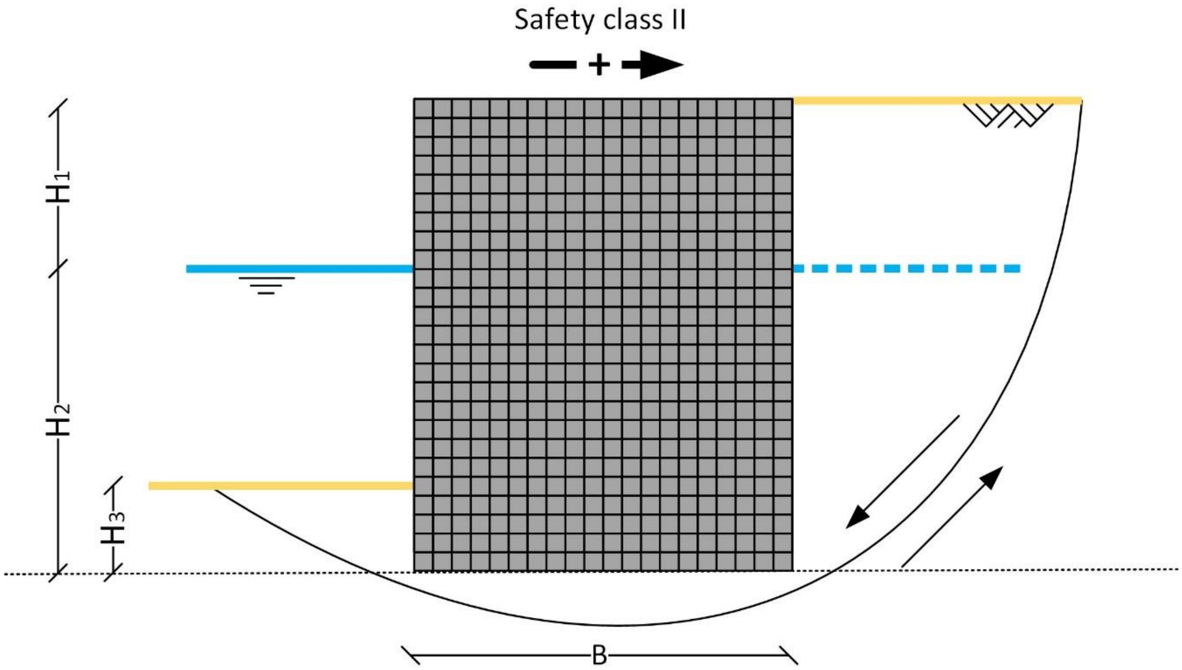


Figure J-4: A possible circular failure plane, showing the driving and resisting forces along the plane

These methods done by hand once are not deemed useful in the preliminary design stage for stability of the quay wall. With several software packages the slip failure can be checked for lots of different slip plane, resulting in a more trustworthy result on the overall stability of the structure. This software was not available to the project team.

Moreover, due to the nature of the subsurface slip failure is not expected. Slip failures are associated with more softer soils like peat, weak clay or loosely packed underwater sand slopes. In this case the sand is expected to be at least medium packed and of medium grain size. The possibly close by rock head would also prevent the development of a slip failure. At last the embedment of the structure is preventing more shallow failure planes to develop as well.

Nevertheless, it is advised to determine the safety against slip failure after more extensive site investigation, if soil conditions do provide caution.

Overturning

When a structure has a small width and endures large horizontal loads, overturning might occur. The relevant moments around the centre of the base are shown in Figure J-5.

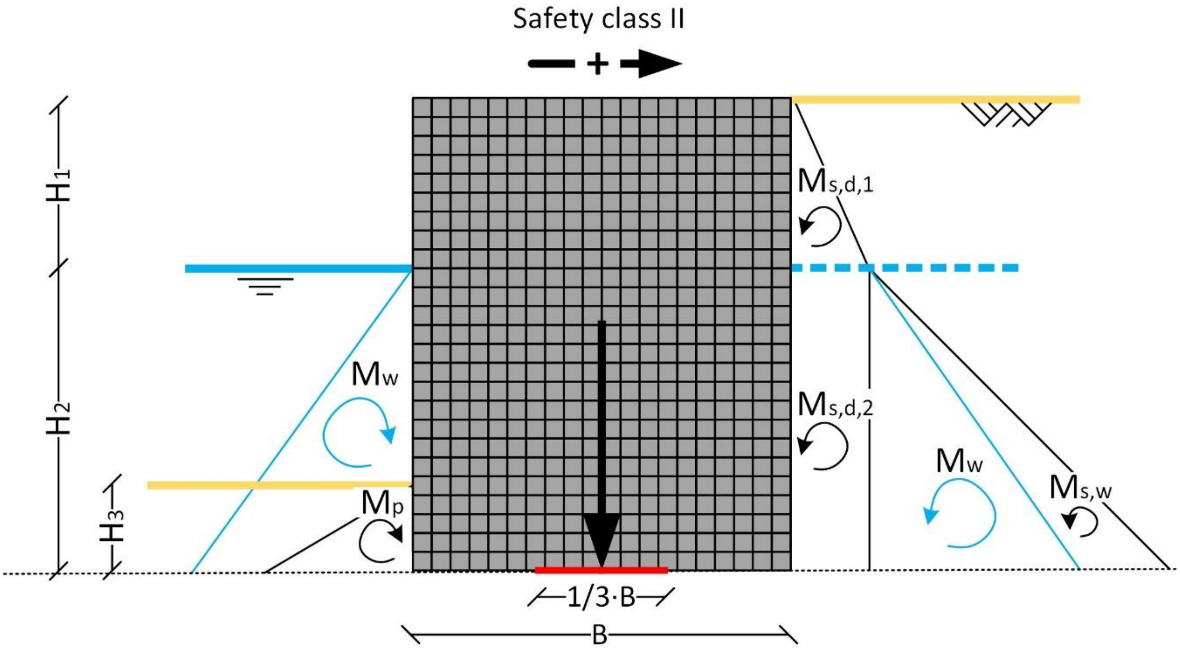


Figure J-5: All relevant pressures, forces and dimensions for the failure mechanism 'Overturning'

Loads

The stress acting on the gabion wall result in four moments that are evaluated. The moments due to the water pressure are left out since these do not result in a net moment. All these moments are design values, using the partial factor 1.1 for unfavourable loads and 0.9 for favourable loads. The contribution of the active earth pressure from the dry sand is split in two moments [$M_{s,d,1}$,$M_{s,d,2}$]. Acting on the right-hand side as well is the moment from the active earth pressure, resulting from the effective stress of the wet sand [$M_{s,w}$]. On the left-hand side only the passive earth pressure is determined [M_p].

$$M_{s,d,1} = \left(K_a \cdot 0.5 \cdot L \cdot H_1^2 \cdot \gamma_{s,d}\right) \cdot \left(H_2 + \frac{H_1}{3}\right) = 80\ kNm \quad \text{J-8}$$

$$M_{s,d,2} = \left(K_a \cdot L \cdot H_1 \cdot \gamma_{s,d}\right) \cdot \left(\frac{H_2}{2}\right) = 118\ kNm \quad \text{J-9}$$

$$M_{s,w} = K_a \cdot 0.5 \cdot L \cdot H_2^2 \cdot \left(\gamma_{s,w} - \gamma_w\right) \cdot \left(\frac{H_2}{3}\right) = 38\ kNm \quad \text{J-10}$$

$$M_p = -(K_p \cdot 0.5 \cdot L \cdot H_3^2 \cdot \left(\gamma_{s,w} - \gamma_w\right)) \cdot \left(\frac{H_3}{3}\right) = -4\ kN \quad \text{J-11}$$

Resistance

The self-weight of the gabion quay resists against overturning: an increasing weight increases the resistance. The self-weight of the gabion is determined in equation J-6. Since the quay walls self-weight is a favourable load in this scenario, a partial factor of 0.9 is applied.

Safety

After the sum of moments and sum of vertical forces are determined, the eccentricity of the resulting force is calculated. If this eccentricity is within 1/6 of the width of the object, the structure is safe.

$$e = \frac{\Sigma M}{\Sigma V} = 0.72\ m \qquad \frac{1}{6} \cdot B = 0.83\ m \quad \text{J-12}$$

In this case safety is ensured, the determined eccentricity is within 1/6 · B. The margin is low, a reduction of the width by 0.5 meter results in failure.

J.2 QUAY WALL, FINISHING STRUCTURES

To complete the quay wall a concrete slab is installed at the top of the jetty, to accommodate easy use of the structure. The concrete slab will also cover part of the front, providing protection from possible impact. Car tyres are used to reduce the impact in case of a collision, in addition to the concrete slab. On the embankment side of the quay wall a geotextile will be placed to prevent erosion.

The quay wall is built of gabions with a height of 0.5 meters, varying in width from 0.75 to 1.25 meters. The various sizes are chosen to avoid easy failure paths. Moreover, smaller gabion baskets are easier to lift and install during construction. Gabions are connected to each other with either spirals (Figure J-6) or C rings (Figure J-7). If necessary, more stiffness can be acquired by using stiffeners (Figure J-8). This to make sure the quay wall acts as a solid structure. The preliminary design of the quay wall is shown in Figure J-9.

Figure J-6: Spiral connection

Figure J-7: C ring connection

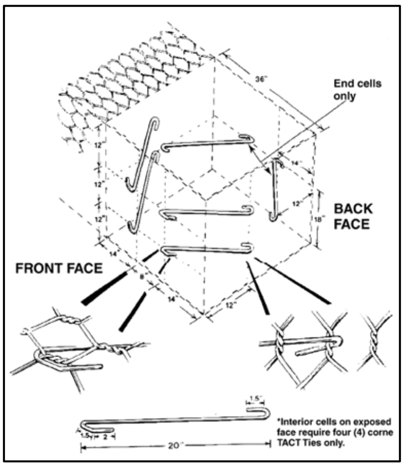


Figure J-8: Stiffeners for gabions

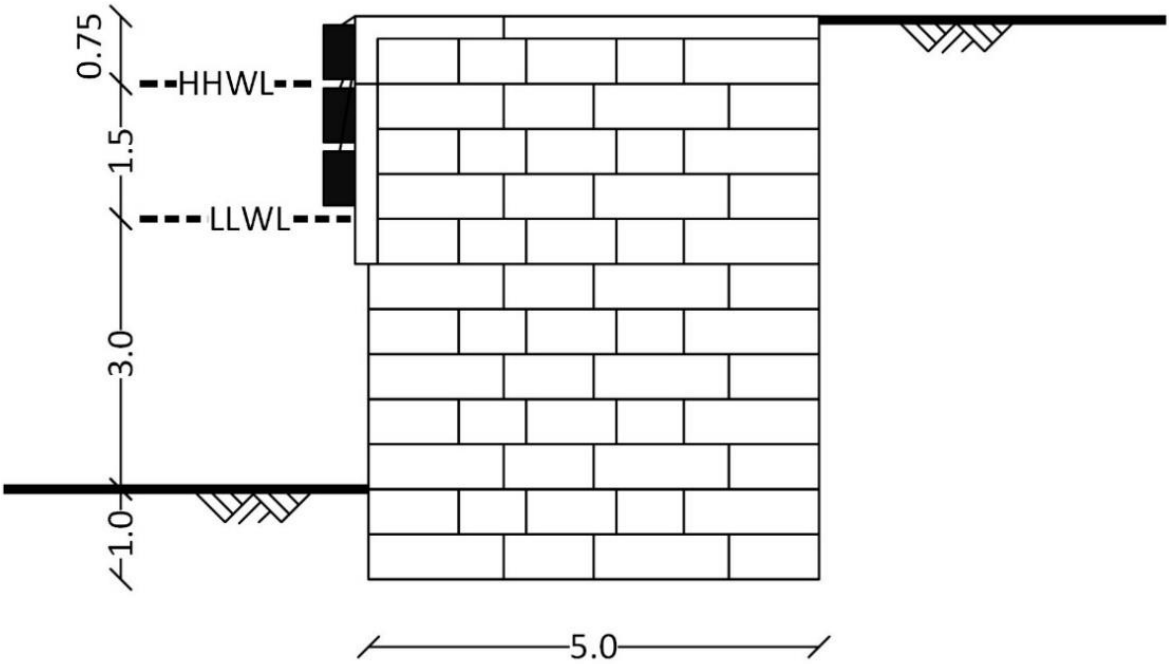


Figure J-9: Quay wall design in more detail; showing the gabion structure, concrete slab and car tyres.

As optimisation a stair-like structure for the gabion can be reviewed, ranging in width from approximately 1.0 meter at the top to 5.0 meter at the bottom. This optimisation is shown in Figure J-10. For this structure the governing cross-section is at the same depth as in the design calculations above. Cross-sections at shallower depths are more safe since the connection between the gabion-layers is stronger than the friction between the gabions and sand. Safety against overturning and bearing capacity is higher as well, gabions below the cross-section provide high resistance. There is a difference in the load on the foundation of five meters, the sand has a higher density than the gabions. This favours the safety of the structure for all failure mechanisms. The safety against sliding and overturning both benefit from a higher vertical load. The bearing capacity benefits as well due to a significant decrease in inclination factors. Moreover, material can be saved significantly by using the stair-like structure. It is recommended to consider this in the final design.

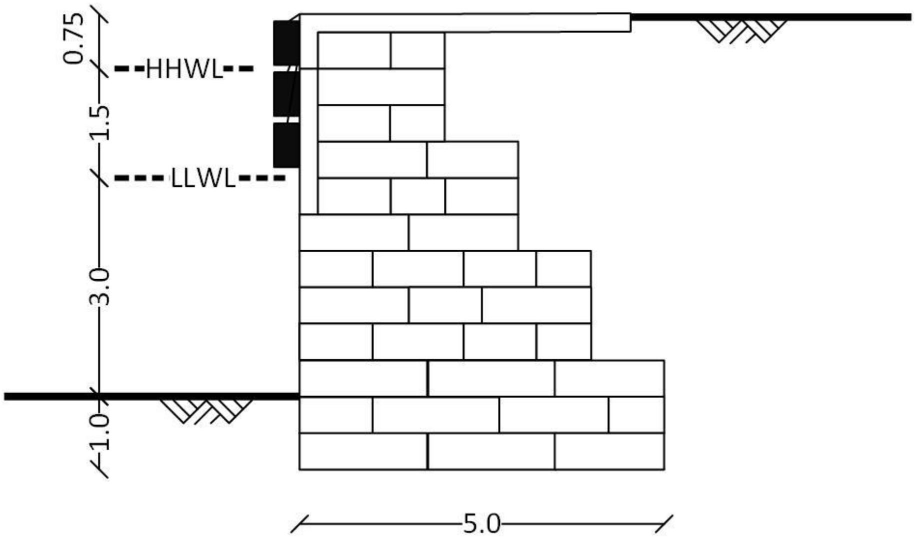


Figure J-10: Quay wall, stair-like structure

K. JETTY DESIGN

In this part of the report a rough design for the jetties, to be placed in the harbour area, is presented. Several safety and contractual factors will be used in the calculations based on material or environmental conditions.

K.1 LOADS AND LOAD FACTORS

The jetties will be placed in a wet environment which corresponds with climate class 3 (Quick Reference), the maximum loading time is expected to be short, supplies and cargo will be carried over the structure but not placed on it for a long term period. The modification factor k_{mod} used for calculating the design strength of the material has a value of 0.70, based on the climate class and loading time.

The low chance on loss of human life puts the jetty in consequence class CC1, which corresponds with a factor $K_{FL} = 0.9$. The applicable permanent- and variable load factors have to multiplied by this factor.

The permanent load due to the self-weight will be calculated in the section of each structural element, which is dependent on its dimensions. The distributed variable load p_k is assumed to be $1.75\ kN/m^2$.

Factors Serviceability Limit State (SLS)

The formula to calculate the total load in the Serviceability Limit State (SLS) is as follows:

$$G_k + \gamma \cdot Q_k \qquad \text{K-1}$$

In which G_k the total permanent load is, Q_k the characteristic value of the leading variable load and γ the variable load factor of 1.5 which still has to be multiplied by K_{FL}. The exact SLS load will be calculated in the structural element sections.

Factors Ultimate Limit State (ULS)

The formula to calculate the total load in the Ultimate Limit State (ULS) is as follows:

$$\gamma_G \cdot G_k + \gamma_Q \cdot Q_k \qquad \text{K-2}$$

In which γ_k the total permanent load factor of 1.2 is and γ_k the variable load factor of 1.5. Both factors still have to be multiplied by K_{FL}. The exact ULS load will be calculated in the next sections.

K.2 MATERIALS

The material used for the jetty construction is of a local available sewn tropical hardwood type; for the piles of the jetty, also concrete is taken into consideration. Average material properties for the used timber can be found in Table K-1.

Table K-1: Average material properties for tropical hardwood. (Timberstructures Dictate)

Property		Symbol	Value	Unit
Flexural strength		$f_{m,k}$	50	N/mm^2
Shear strength		$f_{v,k}$	4	N/mm^2
Density		ρ	1050	kg/m^3
Young's Modulus	SLS	E_{SLS}	14000	N/mm^2
	ULS	E_{ULS}	11800	N/mm^2
Factor for sawn wood		γ_M	1.3	$-$

Design strength

The design strength of a timber material is beside its strength also dependent on the height of an element by the factor k_h, which has to be between 1.0 and 1.3 and corrects for defects which influence the tensile strength. The total formula to calculate this given in equation K-3, with h the height of an element.

$$f_d = k_{mod} \cdot k_h \cdot \frac{f_k}{\gamma_M}$$ K-3

With $k_h = \left(\frac{150}{h}\right)^{0.2}$ K-4

This k_h factor is not used to determine the strength of elements in which no tensile stresses appear.

K.3 JETTY BOARD

The first element of the jetty construction is the covering. The covering is used to walk on, but also to transport the caught fish and supplies over.

Dimensions and strength

The chosen dimensions of a jetty board plank are 2000x200x35 mm ($lxbxh$). The span of the plank (l_s) is 1800mm, see Figure K-1. which can be schematized as a beam on 2 supports.

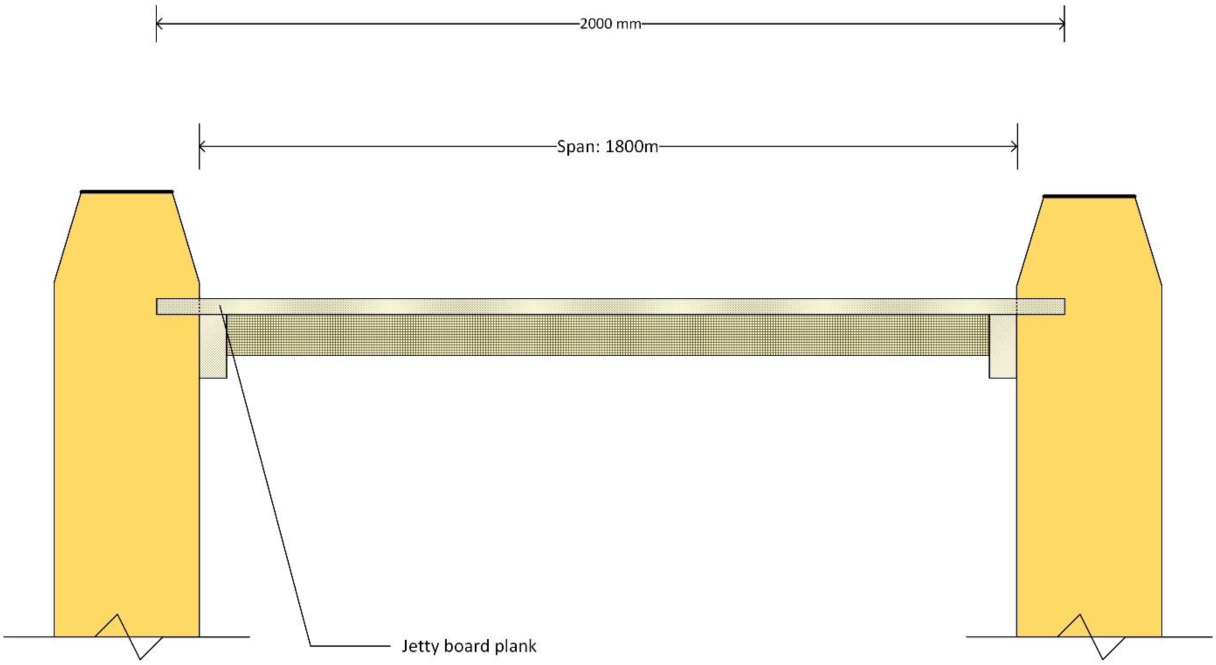


Figure K-1: Schematic front view jetty

The factor for the height effect for these dimensions is higher than the maximal applicable value of 1.3 and therefore chosen to be 1.3. The tensile and shear strength of the boarding planks now yield:

$$f_{m,d} = k_{mod} \cdot k_h \cdot \frac{f_{m,k}}{\gamma_M} = 0.7 \cdot 1.3 \cdot \frac{50}{1.3} = \mathbf{35\ N/mm^2}$$ K-5

$$f_{v,d} = k_{mod} \cdot \frac{f_{v,k}}{\gamma_M} = 0.7 \cdot \frac{4}{1.3} = \mathbf{2.15\ N/mm^2}$$ K-6

The maximum allowed deflection of a beam is defined as its length divided by 250 (Quick Reference). With a length of 1800 mm, this gives a maximum deflection of:

$$w_{max} = \frac{1800}{250} = \mathbf{7.2\ mm}$$ K-7

Furthermore, the moments of inertia`(I_{zz}) and resistance (Wy) are:

$$I_{zz} = \frac{1}{12} wh^3 = \frac{1}{12} \cdot 200 \cdot 35^3 = 7.1e6\ mm^4$$ K-8

$$W_y = \frac{1}{6} wh^2 = \frac{1}{6} \cdot 200 \cdot 35^2 = 4.1e4\ mm^3$$ K-9

Load

Based on the (Quick Reference), a distributed variable load was assumed to be $1.75\ kN/m^2$. Expressed in meters plank this becomes

$$q_k = 1.75 \cdot 0.2 = 0.35\ kN/m. \qquad K\text{-}10$$

The permanent load per meter due to the self-weight of the structure is

$$G_k = b \cdot h \cdot \rho_{hardwood} \cdot g = 0.2 \cdot 0.035 \cdot \frac{1050}{1000} \cdot 9.81 = 0.072\ kN/m. \qquad K\text{-}11$$

Note that these loads are characteristic loads which need to be transferred to design loads for both limit states.

SLS

The distributed loads that have to be taken by the boarding in the serviceability limit state was stated in equation K-1. Together with the load due to self-weight, distributed loads for momentum and shear force are found with values of:

$$q_d = G_k + K_{FL} \cdot \gamma \cdot q_k = 0.072 + 0.9 \cdot 1.5 \cdot 0.35 = 0.54\ kN/m \qquad K\text{-}12$$

From this distributed design load, the design momentum, the shear force and their corresponding stresses in the material can be found in equations K-13 to K-16. Note that for the maximum design momentum, the span length is used instead of the total length. The ends of the plank will be neglected in this simplified calculation which is possible because they decrease the maximum momentum and deflection. Finally, the maximum deflection is given in equation K-17.

$$M_d = \frac{1}{8} \cdot q_d \cdot l_s^2 = \frac{1}{8} \cdot 0.54 \cdot 1.8^2 = 0.22\ kNm \qquad K\text{-}13$$

$$\sigma_{m,d} = \frac{M_d}{W_y} = \frac{0.22e6}{4.1e4} = \mathbf{5.4\ N/mm^2} \qquad K\text{-}14$$

$$V_d = \frac{q_d \cdot l}{2} = \frac{0.54 \cdot 2}{2} = 0.54\ kN \qquad K\text{-}15$$

$$\sigma_{v,d} = \frac{3V_d}{2wh} = \frac{3 \cdot 0.54e3}{2 \cdot 200 \cdot 35} = \mathbf{0.12\ N/mm^2} \qquad K\text{-}16$$

$$w = \frac{5}{384} \frac{q_d \cdot l_s^4}{E_{SLS} I_{zz}} = \frac{5}{384} \frac{0.54 \cdot 1800^4}{14000 \cdot 7.1e6} = \mathbf{7.4\ mm} \qquad K\text{-}17$$

Check

The maximum stresses in boarding must be lower than the maximum stresses that can be taken by the material. A boarding plank should satisfy the following requirements in order to be safe:

1. $\sigma_{m,d} = 5.4\ N/mm^2 \le f_{m,d} = 35\ N/mm^2$ K-18
2. $\sigma_{v,d} = 0.12\ N/mm^2 \le f_{m,d} = 2.15\ N/mm^2$ K-19
3. $w = 7.4\ mm \le w_{max} = 7.2\ mm$ K-20

It can be seen that the first two requirements are met but the third one is not. The deflection is 0.2 mm too big. It is assumed that this will not be a problem due to the neglection of the small overhang as can be seen in Figure K-1: Schematic front view jetty and discussed at the start of this paragraph.

ULS

For the ultimate limit state, the total distributed load for momentum and shear force, following equation K-2, yield:

$$q_{m,d} = K_{FL} \cdot \gamma_G \cdot G_k + K_{FL} \cdot \gamma_Q \cdot q_k = 0.9 \cdot 1.2 \cdot 0.072 + 0.9 \cdot 1.5 \cdot 0.35 = 0.55\ kN/m \qquad K\text{-}21$$

From this load, again the design momentum, the shear force, their corresponding stresses in the material and the maximum deflection are determined in equations K-22 to K-26.

$$M_d = \frac{1}{8} \cdot q_d \cdot l_s^2 = \frac{1}{8} \cdot 0.55 \cdot 1.8^2 = 0.28\ kNm \qquad K\text{-}22$$

$$\sigma_{m,d} = \frac{M_d}{W_y} = \frac{0.28e6}{4.1e4} = \mathbf{6.7\ N/mm^2} \qquad K\text{-}23$$

$$V_d = \frac{q_d \cdot l}{2} = \frac{0.55 \cdot 2}{2} = 0.55\ kN \qquad K\text{-}24$$

$$\sigma_{v,d} = \frac{3V_d}{2wh} = \frac{3 \cdot 0.54e3}{2 \cdot 200 \cdot 35} = \mathbf{0.12\ N/mm^2} \qquad K\text{-}25$$

$w = \frac{5}{384}\frac{q_d \cdot l_s^4}{E_{ULS} I_{zz}} = \frac{5}{384}\frac{0.55 \cdot 1800^4}{11800 \cdot 7.1e6} = \mathbf{8.9}\ \boldsymbol{mm}$ *K-26*

Check

The boarding planks should again satisfy the following requirements in order to be safe:

1. $\sigma_{m,d} = 6.7\ N/mm^2 \leq f_{m,d} = 35\ N/mm^2$ *K-27*
2. $\sigma_{v,d} = 0.12\ N/mm^2\ \leq f_{m,d} = 2.15\ N/mm^2$ *K-28*
3. $w = 8.9\ mm \leq w_{max} = 7.2\ mm$ *K-29*

The deflection is the only requirement that is not met, but the margin is bigger here. In the schematization of a boarding plank, the connections are assumed to be perfect hinges, which is in reality not the case. The connection will partially prevent the deflection of the plank. A problem due to this deflection is therefore not expected.

Another option to get the maximum deflection in the range is to use planks with a thickness of 40 mm instead of 35 mm. the SLS and ULS deflections are then respectively **5.1** and **6.1** mm.

K.4 JETTY BEAM

The second jetty element that is designed is the beam. The boarding is placed on 2 beams, which are connected to the piles. In Figure K-2 the construction is visualized.

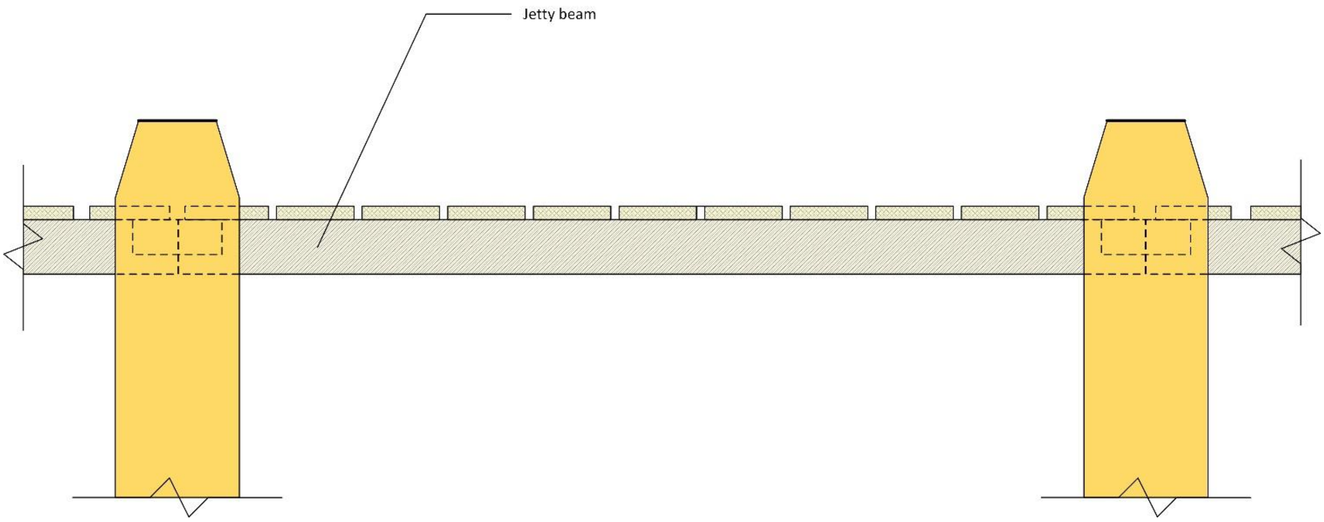


Figure K-2: Schematic side view jetty

Dimensions and strength

The chosen dimensions of the beam are 2500x60x140 mm ($lxbxh$) and is schematized as a beam on 2 supports.

The factor for the height effect based on these dimensions is higher and lies in between 1.0 and 1.3, and thus taken into account. The tensile and shear strength of the beams are:

$f_{m,d} = k_{mod} \cdot k_h \cdot \frac{f_{m,k}}{\gamma_M} = 0.7 \cdot \left(\frac{150}{140}\right)^{0.2} \cdot \frac{50}{1.3} = \mathbf{27.3}\ \boldsymbol{N/mm^2}$ *K-30*

$f_{v,d} = k_{mod} \cdot \frac{f_{v,k}}{\gamma_M} = 0.7 \cdot \frac{4}{1.3} = \mathbf{2.15}\ \boldsymbol{N/mm^2}$ *K-31*

The maximum allowed deflection of a beam is:

$w_{max} = \frac{2500}{250} = \mathbf{10.0}\ \boldsymbol{mm}$ *K-32*

Moments of inertia`(I_{zz}) and resistance (Wy) are:

$I_{zz} = \frac{1}{12} wh^3 = \frac{1}{12} \cdot 60 \cdot 140^3 = 13.7e6\ mm^4$ *K-33*

$W_y = \frac{1}{6} wh^2 = \frac{1}{6} \cdot 60 \cdot 140^2 = 19.6e4\ mm^3$ *K-34*

Load

The load consists of the self-weight of the beam and the load by the boarding. The loads received from the boarding are dealt with in the limit state sections. The characteristic permanent load due to self-weight of the beam is:

$G_k = b \cdot h \cdot \rho_{hardwood} \cdot g = 0.06 \cdot 0.14 \cdot \frac{1050}{1000} \cdot 9.81 = 0.087 kN/m.$ *K-35*

SLS

The loads that have to be taken by the beam in the serviceability limit state are a combination of equation K-1 and the total load due to the boarding, which is already in the SLS form. The distributed load for momentum and shear force is given in equation K-36.

$q_d = G_k + \frac{q_{d,boarding} \cdot l_{boarding}}{2 \cdot w_{boarding}} = 0.087 + \frac{0.54 \cdot 2}{2 \cdot 0.2} = 2.81\ kN/m$ *K-36*

The design momentum, shear force, corresponding stresses and the maximum deflection can be found in equations K-37to K-41.

$M_d = \frac{1}{8} \cdot q_d \cdot l^2 = \frac{1}{8} \cdot 2.81 \cdot 2.5^2 = 2.19\ kNm$ *K-37*

$\sigma_{m,d} = \frac{M_d}{W_y} = \frac{2.19e6}{19.6e4} = \mathbf{11.2\ N/mm^2}$ *K-38*

$V_d = \frac{q_d \cdot l}{2} = \frac{2.19 \cdot 2.5}{2} = 3.51\ kN$ *K-39*

$\sigma_{v,d} = \frac{3V_d}{2wh} = \frac{3 \cdot 3.51e3}{2 \cdot 60 \cdot 140} = \mathbf{0.63\ N/mm^2}$ *K-40*

$w = \frac{5}{384} \frac{q_{m,d} \cdot l_s^4}{E_{SLS} I_{zz}} = \frac{5}{384} \frac{2.81 \cdot 2500^4}{14000 \cdot 7.1e6} = \mathbf{7.4\ mm}$ *K-41*

Check

The maximum stresses in boarding must be lower than the maximum stresses that can be taken by the material. A boarding plank should satisfy the following requirements in order to be safe:

1. $\sigma_{m,d} = 11.2\ N/mm^2 \leq f_{m,d} = 35\ N/mm^2$ *K-42*
2. $\sigma_{v,d} = 0.63\ N/mm^2\ \leq f_{m,d} = 2.15\ N/mm^2$ *K-43*
3. $w = 7.4\ mm \leq w_{max} = 10.0\ mm$ *K-44*

All 3 requirements are met, therefore SLS design is sufficient.

ULS

A combination of equation K-2 and the load from the boarding give the load that has to be endured in the ultimate limit state:

$q_d = K_{FL} \cdot \gamma_G \cdot G_k + \frac{q_{d,boarding} \cdot l_{boarding}}{2 \cdot w_{boarding}} = 0.9 \cdot 1.2 \cdot 0.087 + \frac{0.55 \cdot 2}{2 \cdot 0.2} = 2.85\ kN/m$ *K-45*

From these loads, again the design momentum, the shear force, their corresponding stresses in the material and the maximum deflection are determined in equations K-46 to K-50.

$M_d = \frac{1}{8} \cdot q_d \cdot l_s^2 = \frac{1}{8} \cdot 2.85 \cdot 1.8^2 = 2.22\ kNm$ *K-46*

$\sigma_{m,d} = \frac{M_d}{W_y} = \frac{0.28e6}{4.1e4} = \mathbf{11.3\ N/mm^2}$ *K-47*

$V_d = \frac{q_d \cdot l}{2} = \frac{2.85 \cdot 2}{2} = 3.557\ kN$ *K-48*

$\sigma_{v,d} = \frac{3V_d}{2wh} = \frac{3 \cdot 3.557e3}{2 \cdot 60 \cdot 140} = \mathbf{0.63\ N/mm^2}$ *K-49*

$w = \frac{5}{384} \frac{q_d \cdot l_s^4}{E_{ULS} I_{zz}} = \frac{5}{384} \frac{2.85 \cdot 2500^4}{11800 \cdot 7.1e6} = \mathbf{8.9\ mm}$ *K-50*

Check
The boarding planks should again satisfy the following requirements in order to be safe:

1. $\sigma_{m,d} = 11.3\ N/mm^2 \leq f_{m,d} = 35\ N/mm^2$ *K-51*
2. $\sigma_{m,d} = 0.63\ N/mm^2\ \leq f_{m,d} = 2.15\ N/mm^2$ *K-52*
3. $w = 8.9\ mm \leq w_{max} = 10.0\ mm$ *K-53*

Again all requirements for the beam are met, proving the design to be good.

K.5 MOORING PILE

Mooring forces can be high, depending on the speed and control that a captain has over its vessel. It is therefore assumed that the vertical forces due to anchored- or mooring vessels are significantly higher than the forces like self-weight and the variable loads. For the design of the mooring piles, only those vertical mooring forces are taken in consideration. The piles are designed for a water depth of 3 meters. Later on, the dimensions needed for bigger depths are given.

For the mooring piles, two types of materials are considered. The first one is the same timber as used for the rest of the jetty, the other one is concrete. Steel is a less used building material in Sri Lanka and therefore left out of consideration.

Loads

As mentioned before, the vertical mooring force is assumed to be of most significance while designing the mooring piles. A rough calculation to get the order of magnitude of such a mooring force is presented below, but first the technical details of the multiday vessels are given.

Table K-2: Multiday vessel specifications (NMDF-40 Multi Day Fishing Boat)

	Symbol	Value	Unit	
Length	l	40	ft	
		12.3	m	
Width	w	14	ft	
		4.25	m	
Draft	d	1.21	m	Empty vessel
	d_l	1.60	m	Loaded vessel
Mass	m_s	28	$tons$	Loaded (assumption)
		12	$tons$	Empty vessel
Volumetric water displacement	V_w	27.3	$m\text{^}3$	
Travel speed	v_max	12	NM/h	Maximum travel speed of vessel
		22.2	km/h	
Mooring velocity of the ship	v_s	0.20	m/s	(Manual Hydraulic Structures)

To get an idea of the mooring force on a pile, the kinetic energy that the mooring vessel transforms is determined with the following formula:

$$E_{kin} = \frac{1}{2} m_s v_s^2 C_H C_E C_S C_C \qquad \textit{K-54}$$

With C_H the hydraulic coefficient defined as:

$$C_H = 1 + \frac{2d}{w} = 1 + \frac{2 \cdot 1.6}{4.25} = 1.75 \qquad \textit{K-55}$$

C_E is determined as follows:

$$C_E = \frac{k^2 + r^2 \cos^2(\gamma)}{k^2 + r^2} \qquad \textit{K-56}$$

$$r = \sqrt{\left(\left(\frac{l}{2} - l_{bow}\right)^2 + \frac{w^2}{4}\right)} = \sqrt{\left(\left(\frac{12.3}{2} - \frac{12.3}{3}\right)^2 + \frac{4.25^2}{4}\right)} = 2.95\ m \tag{K-57}$$

With r is defined as the radius between gravitational centre and the berthing point; in which l_{bow} is estimated to be one third of the hull length. γ is assumed to be 70° and the radius of gyration k, is given in equation K-58 with the block coefficient C_b of 0.6 for slender ships.

$$k = (0.19C_b + 0.11)l = (0.19 \cdot 0.6 + 0.11) \cdot 12.3 = 2.76\ m \tag{K-58}$$

Equation K-56 now yields:

$$C_E = \frac{2.76^2 + 2.95^2 \cos^2(70)}{2.76^2 + 2.95^2} = 0.53 \tag{K-59}$$

The softness- and configuration coefficients C_S and C_C are both chosen to be 1 for respectively a negligible deformation of the ship's hull and safety reasons. The mooring energy becomes then:

$$E_{kin} = \frac{1}{2} \cdot 28000 \cdot 0.2^2 \cdot 1.75 \cdot 0.53 \cdot 1 \cdot 1 = 518\ J \tag{K-60}$$

With the mooring post schematized as a mass-spring system, the kinetic energy is written as a force in equation K-61.

$$F = k \cdot u = k\sqrt{\frac{2 \cdot E_{kin}}{k}} = \sqrt{2k \cdot E_{kin}} \tag{K-61}$$

$$k = \frac{3EI}{l^3} \tag{K-62}$$

This force is thus dependent on the stiffness of the pile, which is different for the concrete and timber pile. Further elaboration of the force will be provided in the section considering the different building materials and dimensions.

Timber mooring pile

The first considered mooring pile system is a construction made of timber with the same material properties as the timber used for the boarding and beams.

Dimensions and strength

The chosen dimensions of the piles are 240x240 mm (width, height). One pair of piles is schematized as two 4.25 meter long cantilevered beams next to each other's, connected by a pendulum rod which divides the mooring force over the two beams. The mooring force acts orthogonally to the direction of the beam. The dimensions are big enough to leave the factor for the height effect out of consideration. The tensile and shear strength of the piles are respectively:

$$f_{m,d} = k_{mod} \cdot \frac{f_{m,k}}{\gamma_M} = 0.7 \cdot \frac{50}{1.3} = \mathbf{26.9\ N/mm^2} \tag{K-63}$$

$$f_{v,d} = k_{mod} \cdot \frac{f_{v,k}}{\gamma_M} = 0.7 \cdot \frac{4}{1.3} = \mathbf{2.15\ N/mm^2} \tag{K-64}$$

Moments of inertia`(I_{zz}) and resistance (Wy) per pile are:

$$I_{zz} = \frac{1}{12}wh^3 = \frac{1}{12} \cdot 240 \cdot 240^3 = 2.76e8\ mm^4 \tag{K-65}$$

$$W_y = \frac{1}{6}wh^2 = \frac{1}{6} \cdot 240 \cdot 240^2 = 2.30e6\ mm^3 \tag{K-66}$$

Load

The mooring force as determined in equation K-61 showed that it was dependent on the stiffness of the structure. With the dimensions known, the stiffness and the force related to it can be determined:

$$k = \frac{3EI}{l^3} = \frac{3 \cdot 14000 \cdot 2.76e8}{4250^3} = 151\ N/mm = 15.1e4\ N/m \tag{K-67}$$

This k inserted in K-61 yields a mooring force of:

$$F = \sqrt{2k \cdot E_{kin}} = \sqrt{2 \cdot 15.1e4 \cdot 518} = 12.5\ kN \tag{K-68}$$

Because this force is a rough estimation of the mooring force on the timber pile, the pile will only be checked in ULS with a safety factor of 1.5. The force becomes then $F = 18.8\ kN$ which still needs to be divided over the two piles, resulting in a design force of $F_d = 9.4\ kN$ per pile. Momentum (M), shear force (F), corresponding stresses (σ) and the deflection (w) are given in equations K-69 to K-73.

$$M_d = F_d l = 9.4 \cdot 4.25 = 39.9\ kNm \tag{K-69}$$

$$\sigma_{m,d} = \frac{M_d}{W_y} = \frac{39.9e6}{2.30e6} = \mathbf{17.3\ N/mm^2} \tag{K-70}$$

$$V_d = F_d = 9.4\ kN \tag{K-71}$$

$$\sigma_{v,d} = \frac{3V_d}{2wh} = \frac{3 \cdot 9.4e3}{2 \cdot 240 \cdot 240} = \mathbf{0.24\ N/mm^2} \tag{K-72}$$

$$w = \frac{1}{3}\frac{Fl^3}{E_{SLS}I_{zz}} = \frac{1}{3}\frac{9.4e3 \cdot 4250^3}{14000 \cdot 2.76e8} = \mathbf{62.1\ mm} \tag{K-73}$$

Check
The mooring piles should satisfy the following requirements in order to be safe:

1. $\sigma_{m,d} = 17.3\ N/mm^2 \leq f_{m,d} = 35\ N/mm^2$ *K-74*
2. $\sigma_{m,d} = 0.24\ N/mm^2\ \leq f_{m,d} = 2.15\ N/mm^2$ *K-75*

The stresses due to the mooring force are within the material strength boundaries, the dimensions are thus sufficient. The requirement for the deflection of the pile is not strictly taken into account. The deflection was determined to be about 6.2 cm in K-73. This deflection occurs when a vessel hits the pile during mooring, it happens thus for a very short time. After the mooring impact, it is assumed that the moored vessels put a significantly smaller force on the structure. Besides that, extra measures like fenders can be used to decrease this deflection. It is thus assumed that the deflection will not cause any significant problems.

Since the depth of the harbour basin is varying, the dimensions of the pile will be varying as well. In Table K-3 the dimensions of the piles for bigger depths are presented.

Table K-3: Timber pile dimensions for different depths.

Depth [m]	Pile length [m]*	Dimensions [mm]	Deflection [mm]	Mooring force [kN]
1	2.25	$220x220$	28.5	41.0
2	3.25	$200x200$	59.8	33.9
3	4.25	$240x240$	62.1	18.8
4	5.25	$280x280$	62.6	18.6
5	6.25	$320x320$	62.3	18.7
6	7.25	$360x360$	61.5	18.9
7	8.25	$390x390$	63.6	18.3
8	9.25	$430x430$	62.1	18.8
9	10.25	$460x460$	63.3	18.4
10	11.25	$490x490$	64.1	18.2
11	12.25	$520x520$	64.7	18.0
12	13.25	$560x560$	62.8	18.6

* Length of the pile between the foundation and the point of action of the force (water depth + 1.25).

The reason that the first pile has bigger dimensions is that first, the stresses are governing in the design and rather quickly the deformation takes over. The maximum deflection is tried to kept below or around the 65 mm. It is assumed that no multiday vessels moor at places with a water depth less than 3 meters. Therefore the mooring forces at depths of 1 and 2 meters will be neglected during the design of the foundation.

Concrete reinforced mooring pile

Concrete is a highly used building material in Sri Lanka and therefore taken in consideration as second option for the mooring piles. Again, piles for a depth of 3 meters are determined after which more depths are presented in Table K-6.

Dimensions and strength
The concrete strength for the pile is chosen to be C35/45. In Table K-4, the characteristics of this concrete class are presented. Dimensions of the pile are chosen to be 200x200mm.

Table K-4: Concrete C45/55 characteristics. (Manual Hydraulic Structures)

Property	**Symbol**	**Value**	**Unit**
Compressive cylinder strength (28 days)	f_{ck}	35	N/mm^2
Compressive cube strength (28 days)	$f_{c\ cub\ k}$	45	N/mm^2
Mean value $\boldsymbol{f_{ck}}$	f_{cm}	43	N/mm^2
Mean value axial tensile strength	f_{ctm}	3.21	N/mm^2
Characteristic axial tensile strength (5% fractile)	$f_{ctk,0.05}$	2.25	N/mm^2
Characteristic axial tensile strength (95% fractile)	$f_{ctk,0.95}$	4.17	N/mm^2
Design strength	f_{cd}	23.33	N/mm^2
Secant modulus of elasticity	E_{cm}	34.1	GPa

(Note: $1\ MPa = 1\ N/mm^2$)

Moments of inertia`(I_{zz}) and resistance (Wy) per pile are:

$$I_{zz} = \frac{1}{12}wh^3 = \frac{1}{12} \cdot 200 \cdot 200^3 = 1.33e8\ mm^4 \qquad \text{K-76}$$

$$W_y = \frac{1}{6}wh^2 = \frac{1}{6} \cdot 200 \cdot 200^2 = 1.33e6\ mm^3 \qquad \text{K-77}$$

Because the tensile strength of concrete is poor, the piles are reinforced with B500B steel. The minimum concrete cover is due to the seawater assumed to be 40 mm; and the reinforcement area A_s needs to be in-between the 0.21 and 2.49% (Manual Hydraulic Structures). Reinforcement bars with a diameter of 12 mm are chosen with stirrups of 8 mm. Values for the reinforcement are presented in Table K-5.

Table K-5: Reinforcement steel B500B characteristics + dimensions reinforcement. (Manual Hydraulic Structures)

Property	**Symbol**	**Value**	**Unit**
Yield strength	f_{yk}	500	N/mm^2
Design yield strength	f_{yd}	435	N/mm^2
Strain at maximum load	ϵ_{uk}	2.75	N/mm^2
Ø stirrups	$\emptyset_{stir}$	8	mm
Ø reinforcement	$\emptyset_{reinf}$	12	mm
Cross-sectional area of the reinforcement	A_s	452.4	mm^2

The maximum moment that can be taken by a mooring pile is defined as:

$$M_u = A_s \cdot f_{yd} \cdot d \cdot (1 - 0.52 \cdot \rho \cdot k) \qquad \text{K-78}$$

In which d (arm), ρ (reinforcement ratio) and k (concrete-steel strength ratio) are defined in K-79 to K-81.

$$d = h - \left(c + \emptyset_{stir} + \frac{1}{2}\emptyset_{reinf}\right) = 200 - (40 + 8 + 6) = 146\ mm \qquad \text{K-79}$$

$$\rho = \frac{A_s}{w \cdot h} = \frac{452.4}{200 \cdot 200} = 0.011 \qquad \text{K-80}$$

$$k = \frac{f_{yd}}{f_{cd}} = \frac{435}{23.33} = 18.6 \qquad \text{K-81}$$

Resulting in an ultimate absorbable moment of:

$$M_u = 452.4 \cdot 435 \cdot 146 \cdot (1 - 0.52 \cdot 0.011 \cdot 18.6) = 25.6\ kNm \qquad \text{K-82}$$

The maximum shear force that can be taken per pile is determined in K-84 with $C_{RD,c} = 0.12$, $k = 1 + \sqrt{\frac{200}{c}} = 3.24$, reinforcement ratio $\rho_1 = \frac{A_s}{W \cdot c} = 0.057$, $k_1 = 0.15$, $\sigma_{cp} = 0.2 \cdot f_{cd} = 4.67$ and $v_{min} = 0.035 \cdot k^{\frac{3}{2}} \cdot f_{ck}^{\frac{1}{2}} = 1.21$. The minimum shear resistance is:

$$V_{RD,c,min} = \left(v_{min} + k_1 \cdot \sigma_{cp}\right) \cdot w \cdot c = (1.21 + 0.15 \cdot 4.67) \cdot 200 \cdot 40 = 15.2kN \qquad \text{K-83}$$

$$V_{RD,c} = \left[C_{RD,c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{\frac{1}{3}} + k_1 \cdot \sigma_{cp}\right] \cdot w \cdot c = \left[0.12 \cdot 3.24 \cdot (100 \cdot 0.057 \cdot 35)^{\frac{1}{3}} + 0.15 \cdot 4.67\right] \cdot 200 \cdot 40 = 23.7\ kN \qquad \text{K-84}$$

The shear resistance is bigger than the minimum shear resistance and thus can be taken to be $23.7\ kN$.

The maximum deflection that is allowed is:

$$w_{max} = \frac{l}{250} = \frac{4250}{250} = \mathbf{17\ mm} \qquad \text{K-85}$$

Loads

The force on the pile is again dependent on the stiffness of the pile and yields as determined in K-61:

$$F = \sqrt{2k \cdot E_{kin}} = \sqrt{2 \cdot 17.7e4 \cdot 518} = 3.5\ kN \qquad \text{K-86}$$

With $k = \frac{3EI}{l^3} = \frac{3 \cdot 34.1e3 \cdot 1.33e8}{4250^3} = 177.1 \frac{N}{mm} = 17.7e4 N/m$ K-87

A factor of safety of 1.5 for ULS taken into account gives a design force of $\boldsymbol{F_d = 5.2\ kN}$ and a maximum momentum of $\boldsymbol{M_d = 22.2\ kNm}$.

The maximum deflection of a pile is, assumed that F_d is equally distributed over the 2 piles in the same way as for the timber piles:

$$w = \frac{1}{3}\frac{\frac{1}{2}F_d l^3}{E_{SLS} I_{zz}} = \frac{1}{3}\frac{\frac{1}{2}5.2e3 \cdot 4250^3}{34.1e3 \cdot 1.33e8} = \mathbf{14.7\ mm} \qquad \text{K-88}$$

Check

The mooring piles should again satisfy the following requirements in order to be safe:

1. $M_d = 22.2\ kNm \leq M_u = 25.6\ kNm$ K-89
2. $F_{d,pile} = 2.6\ kN \leq V_{RD,c} = 23.7\ kN$ K-90
3. $w = 14.7\ mm \leq w_{max} = 17\ mm$ K-91

All requirements are met, the design for a depth 3 meters is sufficient. The reason that the deflection criteria is used for the concrete piles is that a bigger deflection will cause cracks in the pile in such order that the reinforcement will be slowly oxidized by the seawater. In Table K-6, dimensions for depths other than 3 meters can be found.

Table K-6: Concrete pile dimensions for different depths.

Depth [m]	Pile length* [m]	Dimensions [mm]	A_s [mm]	Deflection[mm]	Mooring force [kN]
1	2.25	200x200	615.8	5.7	13.5
2	3.25	200x200	452.4	9.8	7.8
3	4.25	200x200	452.4	14.7	5.2
4	5.25	200x200	452.4	20.2	3.8
5	6.25	220x220	452.4	21.7	3.5
6	7.25	220x220	452.4	27.1	2.8
7	8.25	220x220	452.4	32.8	2.3
8	9.25	240x240	452.4	32.8	2.3
9	10.25	240x240	452.4	38.2	2.0
10	11.25	240x240	452.4	43.9	1.7
11	12.25	250x250	452.4	46.0	1.7
12	13.25	250x250	452.4	51.8	1.5

* length of the pile between the foundation and the point of action of the force (water depth + 1.25).

It is assumed that no mooring force is applied on piles placed in depths smaller than 3 meters, their dimensions can therefore also be carried out as the 3 meters depth one. Furthermore, because the concrete piles can be made prefab, piles with dimensions of 250x250 and an A_s of 452.4 mm^2 were checked and proved to be applicable to all mentioned depths in Table K-6. The maximum mooring force in that case will be 12.2 kN.

K.6 Foundation

The current jetty piles will be hammered in the subsoil. No soil data is available yet and the thickness of this sand layer might be too short to guarantee the stability of the jetty piles. If so, a shallow pile foundation must be needed to guarantee the stability of the jetty.

The bearing capacity of the designed structure has been calculated by using the formula of Brinch Hansen. The maximum pressure the soil underneath the footing of the structure can withstand can be assessed with help of this formula. By comparing the resistance of the soil with the forces acting on the soil, the stability can be checked.

$$p = c * N_c + q * N_q + \frac{\gamma}{2} * B * N_\gamma \qquad \text{K-92}$$

With

$$N_c = \left(N_q - 1\right) \cdot \cot(\phi) \text{ [-]} \qquad \text{K-93}$$

$$N_q = \left(\frac{1+\sin(\phi)}{1-\sin(\phi)}\right) \exp(\pi \cdot \tan(\phi)) \text{ [-]} \qquad \text{K-94}$$

$$N_\gamma = 2\left(N_q - 1\right) \cdot \tan(\phi) \text{ [-]} \qquad \text{K-95}$$

In which p the maximum resistance of the soil in kPa, c the cohesion in kPa, q the pressure of the soil next to the footing in kN/m, γ the density of the soil in kN/m^3, ϕ the friction angle in ° and B the width of the footing in m.

The friction angle is assumed to be 30°. This is chosen as a conservative value as the soil properties are unknown. The height of the concrete foundation block is assumed to be 0.6 m and buried below 0.5 m of soil.

The results for the wooden jetty piles are listed in Table K-7 and the results for concrete jetty piles in Table K-8.

Table K-7: Width shallow pile foundation wooden jetty piles

Depth [m]	Pile length [m]	Width piles [m]	Mooring force [kN]	Width footing [m]
2	3.25	0.20	34	6.0
3	4.25	0.24	20	5.0
4	5.25	0.28	20	5.25
5	6.25	0.32	20	5.75
6	7.25	0.36	20	6.0
7	8.25	0.39	20	6.25
8	9.25	0.42	20	6.75
9	10.25	0.46	20	7.0
10	11.25	0.49	20	7.25
11	12.25	0.52	20	7.5
12	13.25	0.52	20	7.75

In the calculation for the width of the foundation the own weight of the wood is left out. The weight of the wood is approximately the same as salt water and therefor left out. The wood above sea water has also been neglected as it is of minor influence.

Table K-8: Width shallow pile foundation concrete jetty piles

Depth [m]	Pile length [m]	Width piles [m]	Mooring force [kN]	Width footing [m]
2	3.25	0.2	8.0	2.5
3	4.25	0.2	5.2	2.2
4	5.25	0.2	4	2.2
5	6.25	0.2	4	2.2
6	7.25	0.2	4	2.25
7	8.25	0.2	4	2.25
8	9.25	0.2	4	2.3
9	10.25	0.2	4	2.3
10	11.25	0.2	4	2.3
11	12.25	0.2	4	2.3
12	13.25	0.2	4	2.3

Table K-8 shows that the width does not differ that much. The foundation with concrete piles is more stable than with wooden piles. This is due to the increased vertical force on the soil which increases the strength. The width shown in Table K-8 is the minimal width to guarantee the 1.80 meter distance between the piles.

In the excel sheet below the check has been performed for only the vertical forces acting on the footing and an additional calculation in which the influence of the horizontal forces are taken into account. This excel sheet shows an example for the calculation for a concrete jetty pile.

Bearing shallow pile foundation

Concrete

Use of the Brinch Hansen formula

Material	Density	Unit
Concrete	24	[kN/m3]
Sand	19	[kN/m3]
Water (salt)	10,25	[kN/m3]

Width footing	
2,2	[m]
Width Poles	
0,2	[m]

Lenght poles	4,25	[m]

Dimensions sediment trap

		Width		height		Vertical load		Effective vertical load	
Concrete	box 1	2,2	[m]	0,6	[m]	31,68	[kN/m]	18,15	[kN/m]
	Piles	0,2	[m]	4,75	[m]	45,6	[kN/m]	26,125	[kN/m]
Soil	box 1	1,8	[m]	0,5	[m]	34,2	[kN/m]	7,875	[kN/m]
	box 2	0	[m]	0,5	[-]	0	[kN/m]	0	[kN/m]

Total forces	box 1	26,025	[kN/m]
	box 2	26,125	[kN/m]

Vertical forces water

Force mouring boats			Total force		Bending moment	
force	6	[kN]	6	[kN]		
				[kN]	32,1	[kNm]
					32,1	[kNm]

Check Vertical Equilibrium (Brinch Hansen)

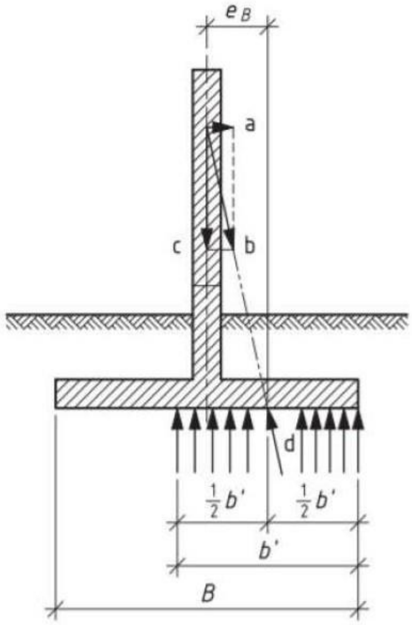
p = c * Nc + q*Nq + y/2 * B*Ny

Cohesion	c	0	[kN/m]
Density soil	y	19	[kN/m]
Pressure	p	52,15	[kN/m]
Friction	phi	30	[degree]
Soil pressure	q	1,8	[kN/m]
Width	B	3	[m]

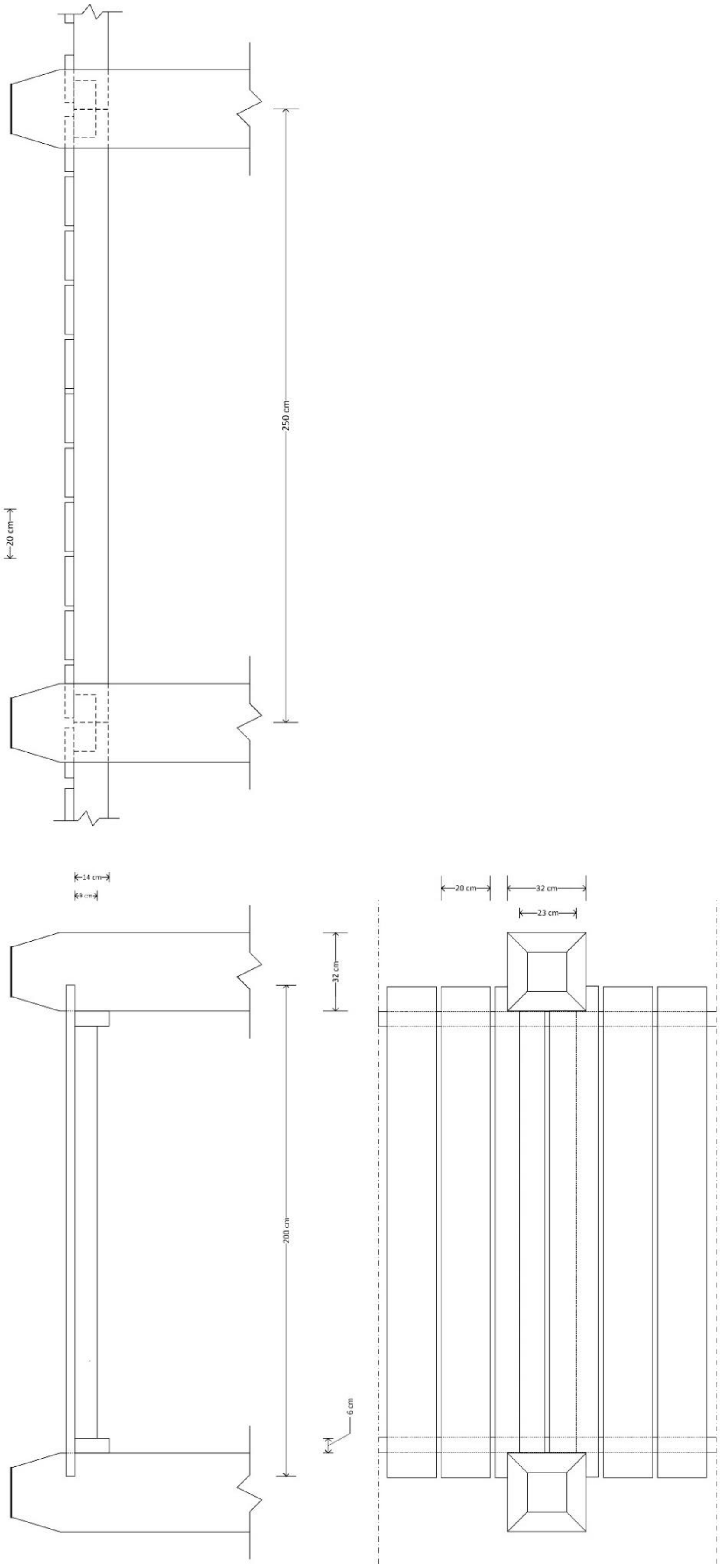
Coefficient	Nq	13,19914636
	Ny	14,08636087
	Nc	21,1295413

Check		
p_res =	213,9243	[kN]
p_load =	52,15	[kN]

Horizontal component			
Horizontal force left	F_h,left	6	[kN]
Horizontal Force right	F_h,right		[kN]
Net horizontal force	F_h	6	[kN]
Vertical net force	F_v	52,15	[kN]
Design force	F_d	52,49402347	[kN]
Angle of design force	Alpha_2	6,563178297	[degree]
Distance excentric load	e_B	0,615532119	[m]
Effective width	b'	0,968935762	[m]
Design Force	F_d	52,49402347	[kN]
Resistance soil	R_d	83,47187073	[kN]



Technical drawing



L. Revetment Design

In this section, a general design for the revetment is presented.

L.1 Bank protection

To prevent the river bank from eroding or changing orientation (concept 2) it must be protected against erosion. This will be elaborated in this section. Two cases will be discussed, one where the river is diverted (concept 2) and the general bank protection in the harbour basin.

In Table L-1 the general values used in the preliminary calculations can be found.

Table L-1: Hydraulic conditions

	Value	**Unit**
Lagoon flow	$1.0*$	$[m/s]$
Bank slope	$1:5$	$[-]$
Tidal currents	0.5	$[m/s]$
D_n50 ground	0.1	$[mm]$

* This is a conservative estimation for the lagoon flow

Table L-2: Ship and channel dimension

	Value	**Unit**
Beam	12	$[m]$
Draft	1.6	$[m]$
Channel width	41	$[m]$
Channel depth	3	$[m]$

In the calculation below no waves from the sea are considered. It is assumed these waves are low when traveling up to the river directly (concept 2), for the other concepts the breakwater will prevent waves from entering the bay/river.

Diverting the river

The currents of importance are the river current and tidal current. The effect of the tidal wave and currents in the diverted river must be investigated. But in this preliminary calculation it has been taken in account. Wave impact has been left out of this calculation.

The river current is 1.0 m/s, but needs to be corrected for bend-effect. This is $\pm 40\%$, which leads to a river current of 1.4 m/s. Together with the ebb-flow this becomes 1.9 m/s.

For the calculation of the required stone size, formula (L-1) and (L-2) are used iteratively. Starting with a guess for C. The results are found in Table L-3.

$$D_{n50} = \frac{u_c^2}{\Psi * \Delta * C^2} \quad \text{(L-1)}$$

$$C = 18 * \log\left(12 * \left(\frac{h}{K_r}\right)\right) \quad \text{(L-2)}$$

Table L-3: Calculation of D_{n50}

Variables	Value	Unit		Iteration		Value	unit
h	2	[m]		2^{nd}	D_{n50}	0.035	[m]
u	1.90	[m/s]			C	45.57	[$m^{0.5}$/s]
Δ	1.65	[kg/m^3]		3^{rd}	D_{n50}	0.034	[m]
Ψ	0.03	[-]			C	45.20	[$m^{0.5}$/s]
Ks	± 0.95	[-]		4^{th}	D_{n50}	0.034	[m]
C*	50	[$m^{0.5}$/s			C	45.07	[$m^{0.5}$/s]
D_{n50}**	0.031	[m]		11^{th}	D_{n50}	0.035	[m]
K_r**	0.062	[m]			C	45.01	[$m^{0.5}$/s]

* First estimate

** After 1^{st} iteration

This results in a D_{n50} of 3.5 cm, thus a revetment is needed. From EN13383 this leads to stone class 45/125 mm to prevent erosion of the top layer.

Using the filter rules for a closed filter and assuming sand of 0.3 mm as base, in total 1 filter layer is needed between the sand and required stones.

$$Stability\ \frac{D_{15F}}{D_{85B}} < 5 \tag{L-3}$$

$$Permeability\ \frac{D_{15F}}{D_{15b}} > 5 \tag{L-4}$$

$$Internal\ stability\ \frac{D_{85}}{D_{15}} < 13 - 14 \tag{L-5}$$

Table L-4: 1st filter layer

Values of coarse sand:			
	D_{n50}	0.001	[m]
	D_{n15}	0.00075	[m]
	D_{n85}	0.0015	[m]
$\boldsymbol{D_{n15f}} >$		0.001	[m]
$\boldsymbol{D_{n15f}} <$		0.002	[m]
First filter layer:			
D_{n85}/D_{n15} = 13-14	D_{n15f}	0.005	[m]
	D_{n85f}	0.07	[m]
1st filter layer properties			
	D_{n50}	0.0375	[m]
	D_{n15f}	0.005	[m]
	D_{n85f}	0.07	[m]

Using the standard grading of 45/125 and the values found in Table L-4, they satisfy the filter rules.

Another solution might be to use a geotextile on top of which somewhat larger stones are placed. A geotextile can be easily installed along a river bank and with this only 1 stone size is required.

Bank protection basin channel

Due to the increase in shipping due to the arrival of multi-day fishing vessels, it must be checked whether the banks of the river need to be protected against the waves and return current.

First the primary and secondary waves will be discussed, after which the return current will be discussed. The calculation has been done for just 1 boat in the channel. This all is done with the Method of Schijf.

Primary and secondary waves
First the A_s/A_c is determined, giving 0.246. From the left graph in Figure L-1 we get a limit speed of

$$v_l = 2{,}1\,\frac{m}{s} \tag{L-6}$$

And a design speed of

$$v_d = v_l * 0.9 = 1{,}95\,\frac{m}{s} \tag{L-7}$$

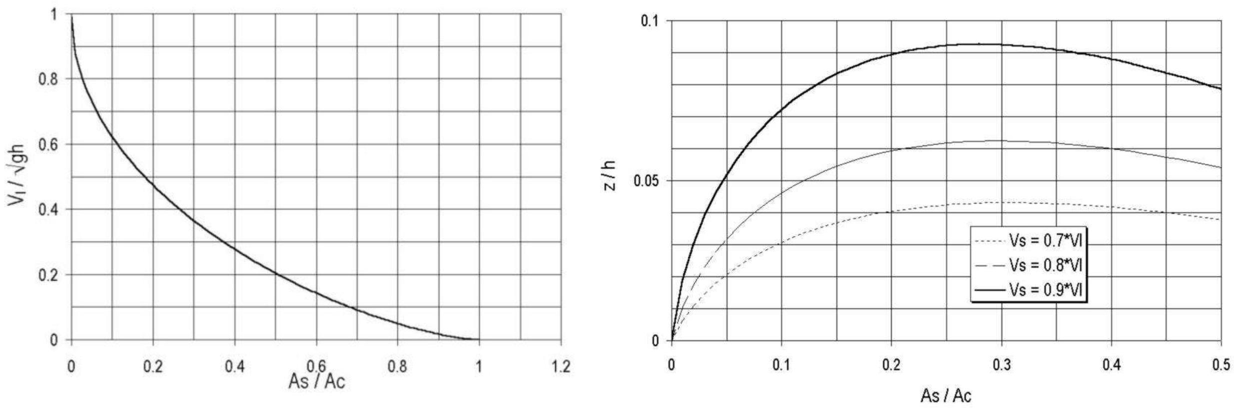


Figure L-1: Limit speed and primary wave

Then from the right graph in Figure L-1 we get

$$z = 0.27\ m$$

Taking in account the eccentricity of the boat,

$$z_{ecc} = \left(1 + \frac{2y}{b}\right) * z = 0.31\ m \tag{L-8}$$

Where y is the distance from the centre of the channel to the centre of the vessel. In this case $3.5\ m$. The total primary wave will be:

$$z_{max} = 1.5 * \ z_{ecc} = 0.48\ m \tag{L-9}$$

The secondary wave is calculated with

$$H = 1.2 * h * \frac{s}{h}^{-0.33} * Fr^4 \tag{L-10}$$

Which results in a secondary wave of $0.055\ m$.

Return current
The return current can be found by using the As/Ac value found earlier and multiplying it with: $\sqrt{gh}$, this leads to

$$u_r = 0.15 * \sqrt{9.81 * 3.0} = 1.33\,\frac{m}{s} \tag{L-11}$$

Taking eccentricity in account with the same y used in *(L-8)*

$$u_{r-ecc} = 1.33 * \left(1 + \frac{y}{b}\right) = 1.50\,\frac{m}{s} \tag{L-12}$$

Protection against waves and current
To calculate the protection against erosion, first the possible erosion from waves and return current have been observed. (Vellinga, 1986) and (Te Chow, 1959) give a good insight in the possible erosion. Due to the little space that is available and the possible decrease of channel depth. It is chosen to protect the banks.

For the protection against the primary wave the formula from (RWS/DHL, 1988) is used based on experimental data.

$$D_{n50} = \frac{z_{max}}{1.8 * \cot(\alpha)^{0.33} * \Delta} \tag{L-13}$$

Leading to a D_{n50} of 0.095 m.

For the stability of the return current an Izbash-type of relation is used. This is done because the return flow is first accelerating and then decelerating.

$$D_{n50} = 0.47 \frac{\left(u_r(1+3r)\right)^2}{2 * g * K_s * \Delta} \tag{L-14}$$

0.2 for r is chosen and K_s amounts to ± 0.95. With this a D_{n50} of $0.093\ m$ is needed. For this case the secondary wave has not been calculated as the primary wave will dominate.

Combination of return current and lagoon flow
In concept 1 the river enters the Northern bay. In this case the return current and the lagoon flow enhance each other.

For simplicity, the lagoon flow, return current and tidal current are added. Resulting in $u^{\S} = 3.0\ m/s$. Using the same approach as in the diverted river section, this leads to:

$D_{n50} = 17.5\ cm$

Thus 10-60 kg rocks are needed to protect the bottom according to EN13383.

Instead of constructing filter layers a geotextile can be placed as filter. 10-60 kg stones can be directly placed on the geotextile and reduces the amount of work and transport needed.

§ In this case, bend flow is not accounted for.

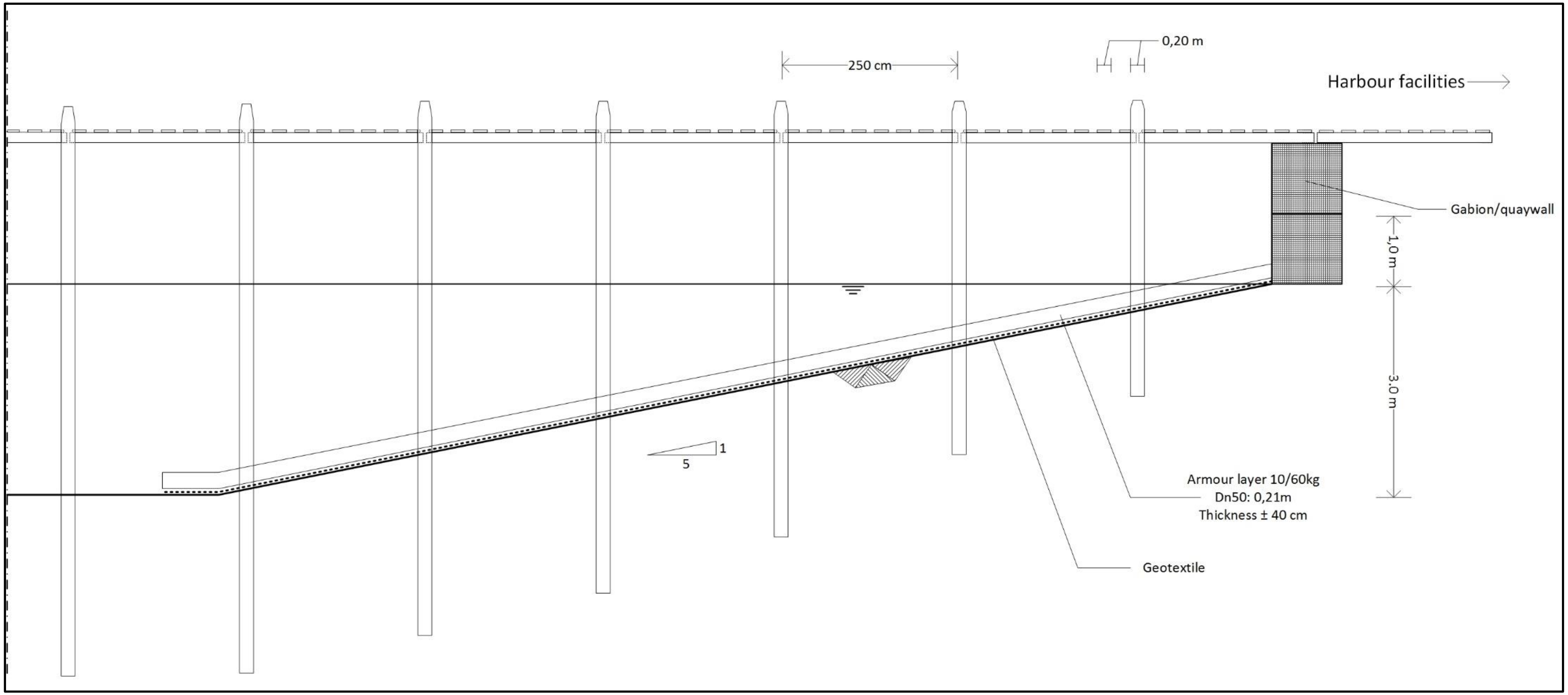


Figure L-2: example bank protection; geotextile

M. Sediment Trap Design

In this section two options for the sediment trap will be discussed. The focus will be on a design that traps sediment, preventing it from entering and blocking the basin. First the structural requirements are given after which possible solutions are discussed. The construction of a sediment trap has been used in the null-option (concept 5), therefore it should be easy to construct.

M.1 Structural requirements

The sediment trap must block the sediment coming from the north and may not hinder navigation into the harbour basin. Damage to the structure it not a problem as long it can be easily repaired. Because of the sheltered location of the trap, the hydraulic conditions are significantly less than elsewhere in the bay. However, for preliminary calculations the wave height from Table M-2 is used. This wave height is derived with SwanOne, any sheltering effects have not been accounted for.

Table M-1: Dimensions of the sediment trap

Dimensions	Value	Unit
Length	50	[m]
Height	+ 1.5 MSL	[m]
Max depth	- 5.7 MSL	[m]

Table M-2: Hydraulic conditions

	Value	Unit
Wave height ($\frac{1}{20}$ $\boldsymbol{year\ storm}$)	3.80	[m]
H_{rms}	2.68	[m]

M.2 Groyne made of wood

Wood has been proven successful for blocking sediment, this is based previous projects and many coastal structures around the world. Wood is a local product and easily available. Wood in combination with other materials should provide a good solution for trapping the sediment. First a general concept will be discussed after some preliminary calculation are done.

General concept

A possible groyne could consist of wooden piles (possible bamboo) casted into a concrete. Between the wooden piles a plate is fixed, this will ensure that sediments will not pass the trap.

The concrete blocks will be placed below the sea bottom to increase stability. The structure might guide the tidal current offshore, special attention must be payed to prevent severe erosion surrounding the concrete. This might cause instability of the unit and eventually failure. The concrete blocks will be placed in two rows and extending approximately 40 m in the bay.

Due to insufficient soil data of the area, a shallow concrete foundation is chosen, above timber piles or sheet piles, as the base of the construction. This was also discussed in K.6

Durability

Wood is a relative strong and natural product, and as said earlier, easily available. Using wood in marine environments will cause mechanical and natural degradation. Normally this will lead to a lifetime or around 20 to 25 years, while concrete units have a lifetime of approximately 50 years.

The sole purpose of the trap is to keep the sediment away from the river mouth and not to defend any land behind the structure. Damage to the construction is allowed as long as it is repairable. If failure of the concrete units occurs, the trap will not stop sediment anymore.

Calculations

To check whether a bed protection is needed, the currents need to be known. For now, only the orbital movement and tidal current are considered. For this preliminary calculation assumptions are done and the currents are calculated with basic formulae. Any shoaling/diffraction effects are not taken in account for the orbital velocities and undertow is left out for that reason. Measurements and modelling must be done to get the currents and other variables.

The maximum orbital velocity at the bottom is 2.38 m/s. The tidal current is between 0.1-0.5 m/s (C. Fernando, Meeting, 2016) For design purpose the total velocity is given as 2.88 m/s.

The required stone size can be found in Table M-3, the same method is used as in Appendix L for the calculation of the revetment.

Table M-3: Calculation required stone size

Variables	Value	Unit	Iteration		Value	unit
h	5.65	[m]	2nd	D_{n50}	0,095	[m]
u	2.88	[m/s]		C	45.93	[$m^{0.5}$/s]
Δ	1.65	[kg/m^3]	3rd	D_{n50}	0,099	[m]
Ψ	0.03	[-]		C	45.60	[$m^{0.5}$/s]
Ks	0.8	[-]	11th	D_{n50}	0.10	[m]
C*	50	[$m^{0.5}$/s		C	45.48	[$m^{0.5}$/s]
D_{n50}**	0.0837	[m]	11th	D_{n50}	0.101	[m]
K_r**	0.1675	[m]		C	45.43	[$m^{0.5}$/s]

* First estimate

** after first iteration and k_r=2* D_{n50}

According to EN13383, the required stone class is 90/250mm.

Resulting in a final D_{n50} of 0.10 m, thus stones of 90/250mm will do according to EN13383. These stones cannot be placed directly on sand, thus a filter layer is needed to prevent erosion of the sea bottom.

Table M-4: Required filter layer

Values of coarse sand:			
	D_{n50}	0,001	[m]
	D_{n15}	0,00075	[m]
	D_{n85}	0,0015	[m]
$\boldsymbol{D_{n15f}} >$		0,00375	[m]
$\boldsymbol{D_{n15f}} <$		0,0075	[m]
First filter layer:			
$\boldsymbol{D_{n85}/D_{n15}}$ **= 13-14**	D_{n15f}	0,005	[m]
	D_{n85f}	0,07	[m]
1st filter layer properties			
	D_{n50}	0,0375	[m]
	D_{n15f}	0,005	[m]
	D_{n85f}	0,07	[m]

The required filter layer is the same as discussed in the Revetment design in Appendix L.

From the filter rules, only one filter layer is needed. On top of that the armour layer is placed to prevent erosion near the structure. It should be investigated whether an open filter is more economical. The use of an open filter only requires one stone size and is easier to place as only one stone size will be dumped. Research must be done for the exact hydraulic conditions for construction of an open filter.

Lastly, the trap must withstand wave impacts. Due to the orientation of the trap, waves will hit the construction. As the wave height of 3.80 m is bigger than the structure above MSL, ± 1.5 m. Again note that this wave height is from SwanOne that did not take the breakwater in account. Therefore the wave height given in Table M-2 will never hit the structure. The wave will not hit the structure but 'passes' over it. This increases the water level on 1 side. The bearing capacity of the sand must be sufficient to withstand the extra force of the water level rise.

The bearing capacity of the designed structure has been calculated by using the formula of Brinch Hansen. The maximum pressure the soil underneath the footing of the structure can withstand can be assessed with help of this formula. By comparing the resistance of the soil with the forces acting on the soil, the stability can be checked.

$$p = c * N_c + q * N_q + \frac{\gamma}{2} * B * N_\gamma \quad \text{M-1}$$

$$\text{With: } N_c = \left(N_q - 1\right) \cdot \cot(\phi) \text{ [-]} \quad \text{M-2}$$

$$N_q = \left(\frac{1+\sin(\phi)}{1-\sin(\phi)}\right) \exp(\pi \cdot \tan(\phi)) \text{ [-]} \quad \text{M-3}$$

$$N_\gamma = 2\left(N_q - 1\right) \cdot \tan(\phi) \text{ [-]} \quad \text{M-4}$$

In which p the maximum resistance of the soil in kPa, c the cohesion in kPa, q the pressure of the soil next to the footing in kN/m, γ the density of the soil in kN/m^3, ϕ the friction angle in ° and B the width of the footing in m.

In the excel sheet below the check has been performed for only the vertical forces acting on the footing and an additional calculation in which the influence of the horizontal forces are taken into account.

Bearing capacity sediment trap

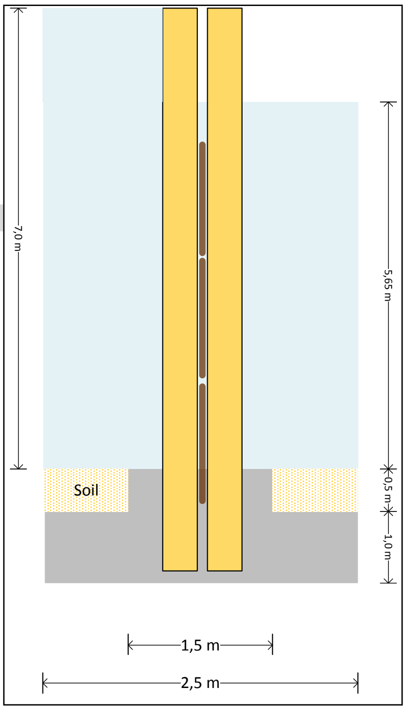
Use of the Brinch Hansen formula

Material	Density	Unit	Width footing
Concrete	24	[kN/m]	2.5 [m]
Sand	19	[kN/m]	
Water (salt)	10.25	[kN/m]	
Wood	10.5	[kN/m]	(effective weight due to wood is neglected)

Dimensions sediment trap

			Vertical load	Effective vertical load
Concrete	box 1	1 [m]	24 [kN]	14 [kN]
	box 2	1.5 [m]	36 [kN]	21 [kN]
	box 3	1 [m]	24 [kN]	14 [kN]
Soil	box 1	0.5 [m]	9.5 [kN]	4.5 [kN]
	box 2	0 [m]	0 [kN]	0 [kN]
	box 3	0.5 [m]	9.5 [kN]	4.5 [kN]

Total forces	box 1	18.5 [kN]
	box 2	21 [kN]
	box 3	18.5 [kN]



Vertical forces water (no flow)

Height water column			Total force
Left	7	m	35.875 [kN]
Right	5.65	m	28.95625 [kN]
Average pressure	20		

Check Vertical Equilibrium (Brinch Hansen)

p = c * Nc + q*Nq + y/2 * B*Ny

Check		
p_res =	318.0434	[kN]
p_load =	58	[kN]

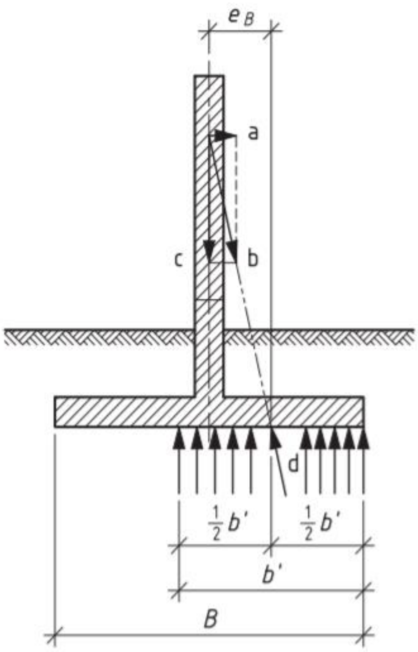
Cohesion	c	0	[kN/m]	
Density soil	y	19	[kN/m]	
Pressure	p	58	[kN/m]	
Friction	phi	27	[degree]	
Soil pressure	q	13.5	[kN/m]	(1,5m thick sand later next to structure toe)
Width	B	2.5	[m]	

Coefficient	Nq	13.199146
	Ny	12.431551
	Nc	23.942173

Horizontal component

Horizontal force left	F_h,left	35.875	[kN]
Horizontal Force right	F_h,right	28.95625	[kN]
Net horizontal force	F_h	6.91875	[kN]
Vertical net force	F_v	58	[kN]
Design force	F_d	58.411207	[kN]
Angle of design force	Alpha_2	6.8025993	[degree]
Distance excentric loac	e_B	0.2783405	[m]
Effective width	b'	1.943319	[m]

Design Force	F_d	58.411207	[kN]
Resistance soil	R_d	286.90159	[kN]



Construction

The concrete units with the poles can be made on site or elsewhere. The units that will be placed close to the waterline can be made elsewhere and transported to location. The units that are required for the deeper part are preferably made on site. Due to their size, transportation will be difficult.

To place the units, a trench must be dredged. This can be done with a backhoe on a pontoon. The first units can be placed from land, while the others must be placed from sea. A crane must place the latter ones.

The units will be placed in two rows, extending approximately 40 meters into the bay. Once a couple of units are placed, construction of the filter layer can start. Preferably an open filter because of the gaps between the units and rows. This will fill up the total construction and only requires dumping one stone class.

Other constructional aspects

At the deepest point, the piles must be around 7 to 8 meters long. Whether wood with this length and strength is available must be checked. Else two poles must be fixed together. The connection must be strong enough to withstand the forces.

The first few concrete units require wood that has less strength. Smaller wood and less thick bundles can be used for the first few concrete units.

The effect of smaller waves hitting the structure has not been considered. Only the last few units might have significant wave impact. But due to their length it is assumed that they are strong enough.

Basic layout
The basic lay-out is given for the deepest point, ±5.65 m below MSL. See Figure M-1 for the general lay-out. At this point, the pole will be around 1.25 m above MSL to ensure it is always visible for the fishermen. An open filter is assumed in this figure.

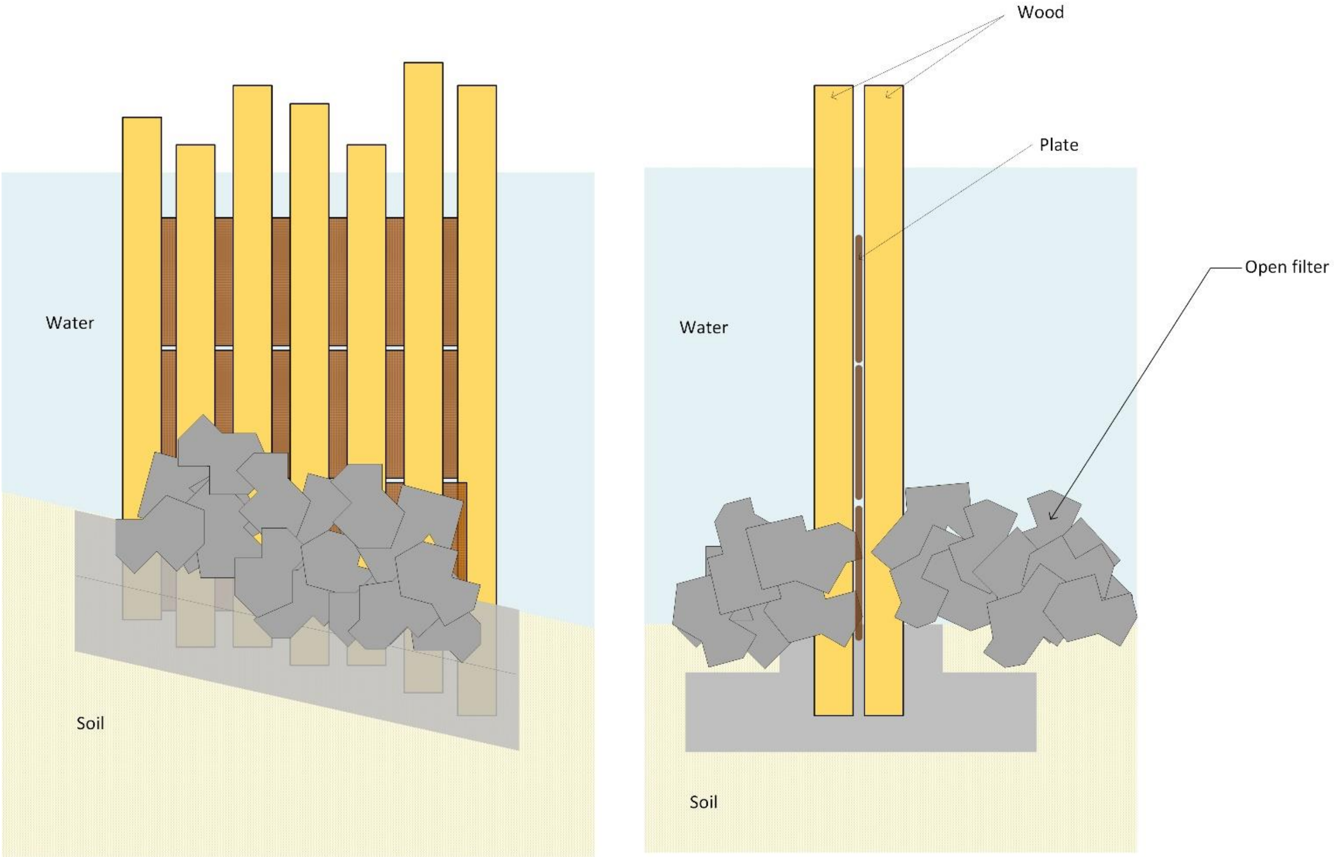


Figure M-1: General lay-out sediment trap (not to scale)

In Figure M-2 a 3D impression can be found.

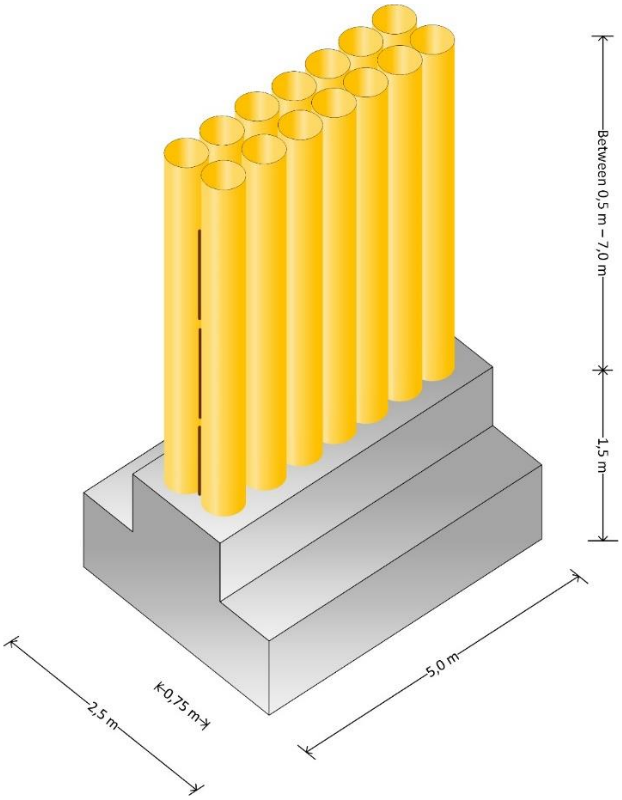


Figure M-2: 3D impression

Other designs
The possible sediment trapped discussed above can also be made with no plates in between. If this would be the case, the wood must be tightly packed together to ensure no sediment will pass through.

Instead of concrete units, one can also think of gabions as a sediment trap. The gabions can be made of bamboo and filled with rock. The gabions are flexible and can be made elsewhere and easily transported to the location. The length of a gabion would be around 5.0 m and a height of 0.3 m.

The gabions will be dumped on top of each other, forming a barrier. On top of which stones will be dumped. These stones will also serve as an open filter.

Lastly it must be checked what the lifetime of this bamboo is in a marine environment, and checked whether steel or concrete might better. However, those materials decreases the flexibility of this solution.

A small breakwater can also be used as sediment trap. This concept is not worked out. The wave height in the sheltered zone must be known in order to get the required armour stones. The SwanOne model calculates the wave height without the effects of the current breakwater.

N. Breakwater Design

Concepts 1 to 4 include a breakwater. Breakwater design depends among others on the loads and the available material. The yield curve of the quarry and the distance from the quarry to the project location are important factors. The infrastructure can be a limiting factor, due to the presence of the small bridges which cannot stand heavy loads. The local supervisor of project Dodanduwa stated that the use of natural rocks is highly preferred in this project. The breakwater length differs from 200 to 250 m depending on the concept. To get insight several designs are made which are discussed in this chapter.

N.1 Functions

The breakwater should limit wave intrusion in the basin, so the fishermen can moore their vessels safely. Furthermore, the breakwater should provide protection against breaking waves, when heading for open water.

N.2 Breakwater design

Use of natural rock is common practice for breakwater design in Sri Lanka, use of concrete armour units is rare. Furthermore it is common that a quarry is leased from the government to acquire the material needed. In this case it is economical to use all the material and not only the higher small percentage of heavy stones. A (partially) reshaping berm breakwater is in this case economically more attractive. Information about the yields curves of nearby quarries is not available, for this reason different designs are made based on different rock classes.

In the absence of accurate bathymetry quarry's yield curves making a very detailed design is pointless. In this phase several designs are made based on several assumptions and guidelines given by Mr. Fernando.

Only for one location a cross section is presented this section applies to the deepest part of the breakwater, with the highest loading. In this phase the design of the breakwater head is neglected. Furthermore making different cross sectional designs along the breakwater in a later stage gives opportunity to the engineer to make effective use of the quarry output.

The use of natural rock is common in Sri Lanka for this reason. Three different types of breakwaters are discussed:

- Conventional rubble mound breakwater
- Partly reshaping berm breakwater
- Fully reshaping berm breakwater

This approach is chosen, because of the absence of quarry data.

N.3 Breaker zone

It is important that the breakwater stretches further than the breaker zone, to limit siltation of the channel. The southern bay is a closed cell. No sediment is by passing the southern rock outcrop. In present state sediment is by passing the northern rock outcrop. The future breakwater, which will be attached to the existing northern outcrop should be long enough to act as sediment trap.

The breaker zone is estimated based on yearly wave data from Agross and on the bathymetry which is assumed based on a process meeting with C. Frenando (Appendix P.1).

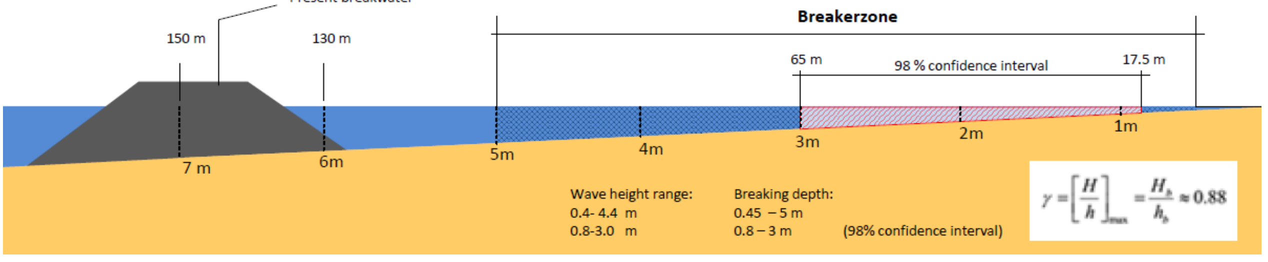


Figure N-1: Location of the breaker zone

N.4 STATEMENTS AND REQUIREMENTS

Preconditions

- Previous built breakwater with a length of 100 m at maximum depth of 7 m (C. Fernando, 2016).
- Rock outcrops present in the Northern and Southern bay.

Requirements

- An exceedance of H_s = 30 cm is accepted once a year.
- An exceedance of H_s = 60 cm is accepted for 1/50 year design storm.
- Damage to the breakwater is only accepted for a storm condition with a probability of 1/50 or less. (C. Fernando, 2016)

Assumptions

- Wave height in the middle of the harbour basin is equal to 0.9 times the wave at the lee side of the breakwater.
- Design water level and wave height is fully dependent.
- Ocean bottom consists of course sand, d_{n50} = 1 mm.
- Sea level rise is 50 cm/century.
- Overtopping discharge limited to 10 l/s/m during 1/50 year design condition with respect to possible anchorage of vessels behind the breakwater.
- Water depth at the proposed location is maximum 9.6 meters

N.5 BOUNDARY CONDITIONS

The DWL is a summation of different components. Subsidence is not included, because the ocean bottom consists of sand and rock. The joint probability of the different components and the probability of the corresponding wave height is neglected. A conservative approach. The transformation of deep water wave heights to nearshore wave heights is performed with SwanOne. Further details about the hydraulic conditions can be found in chapter 10.3.

Table N-1: Hydraulic boundary conditions for breakwater design

Return period		**Offshore**		**Nearshore**		**Design water level**				
1	Hs	2.89	[m]	2.85	[m]	SLR	25	cm	2.2	%
	Tp	13.89	[s]	13.89	[s]	Tide	70	cm	6.1	%
	Tm01	11.5	[s]	9.57	[s]	Seasonal var.	50	cm	4.3	%
						Wind setup&stormsurge	50	cm	4.3	%
1/50	Hs	6.08	[m]	5.7	[m]	Subtotal	195	cm	16.9	%
	Tp	12.4	[s]	12.4	[s]					
	Tm01	10.23	[s]	9.1	[s]	SWL	960	cm	83.1	%
						DWL	1155	cm	100.0	%
1/100	Hs	6.55	[m]	6	[m]					
	Tp	12.4	[s]	12.4	[s]					
	Tm01	10.33	[s]	9.8	[s]					

N.6 CONVENTIONAL RUBBLE MOUND BREAKWATER

A traditional multi layered breakwater consists of an armour layer, a sublayer(s), a core, a toe and often a set of filter layers. The West coast of Sri Lanka is often exposed to high waves. In this case a gentle slope and a high freeboard should prevent exceeding the maximum overtopping discharge. For stability reasons high waves result in heavy armour. Accepting some damage can reduce the initial costs significantly. The client should decide which risk is accepted. A no damage criterion during a 1/50 year storm event, given by Mr. Fernando, is normative for the calculations made. The impact of several factors is analysed to provide information which can be used during decision making in next phases, no complete conventional breakwater design is made.

The factors investigated are:

- Slope angle of the upper slope
- Accepted damage level
- Accepted maximum overtopping discharge

Required armour size for different slopes, overtopping rates and damage levels

Based on the wave corresponding to the 1/50 year design storm calculations are made for the required armour stone size according to the Vd. Meer formulae (CIRIA; CUR; CETMEF, 2007):

For *plunging waves* ($\xi_m < \xi_{cr}$):

$$\frac{H_s}{\Delta D_{n50}} = c_{pl} \cdot P^{0.18} \cdot \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \cdot \xi_m^{-0.5} \qquad \text{N-1}$$

For *surging waves* ($\xi_m \geq \xi_{cr}$):

$$\frac{H_s}{\Delta D_{n50}} = c_s \cdot P^{-0.13} \cdot \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \cdot \sqrt{cot\alpha} \cdot \xi_m^{P} \qquad \text{N-2}$$

Where:

N	=	number of incident waves at the toe	[-]
H_s	=	significant wave height	[m]
ξ_m	=	surf similarity parameter using the mean wave period T_m (s)	[-]

$$\xi_m = \frac{tan\alpha}{\sqrt{\frac{2\pi \cdot H_s}{g \cdot T_m{}^2}}} \qquad \text{N-3}$$

ξ_{cr}	=	critical value of the similarity parameter	[-]

$$\xi_{cr} = \left[\frac{c_{pl}}{c_s} P^{0.31} \sqrt{tan\alpha}\right]^{\frac{1}{P+0.5}} \qquad \text{N-4}$$

α	=	slope angle	[°]
Δ	=	relative buoyant density	[-]
P	=	notional permeability of the structure	[-]
c_{pl}	=	6.2 (with standard deviation of 0.4)	[-]
c_s	=	1.0 (with standard deviation of 0.08)	[-]

The damage level S_d is formulated as (CIRIA; CUR; CETMEF, 2007):

$$S = \frac{A_e}{D_{n50}{}^2} \qquad \text{N-5}$$

Where	S	= Damage level	[-]
	A_e	= Erosion area around still water level	[m^2]
	D_{n50}	= Nominal stone size	[m]

The magnitude of damage; initial damage, intermediate damage and the corresponding damage level, S_d, differs for various slopes.

Table N-2: Damage levels for various slope angles (CIRIA; CUR; CETMEF, 2007)

Slope	Initial damage	Intermediate damage	Failure
$\mathbf{1:1.5}$	2	$3-5$	8
$\mathbf{1:2}$	2	$4-6$	8
$\mathbf{1:3}$	2	$6-9$	12
$\mathbf{1:4 \ - \ 1:6}$	3	$8-12$	17

An overview is made of the required rock size for different slopes and damage levels. The calculation is made using the program Breakwat. The results are shown below in which:

(green)	=	Inital damage
(yellow)	=	Intermediate damage

Table N-3: Required armour size for different damage levels in case of sea ward slope of 1:2

		M50 [kg]	D_{n50} [m]	Reduction
Seaward slope cot(α) [-]	**2**			
Damage level (S) [-]	2	22696	2.05	-
	3	17795	1.89	22%
	4	14973	1.78	34%
	5	13097	1.70	42%
	6	11740	1.64	48%

Table N-4: Required armour size for different damage levels in case of sea ward slope of 1:2.5

		M50 [kg]	D_{n50} [m]	Reduction
Seaward slope cot(α) [-]	**2.5**			
Damage level (S) [-]	2	16240	1.83	-
	3	12733	1.69	22%
	4	10714	1.59	34%
	5	9372	1.52	42%
	6	8400	1.47	48%
	7	7658	1.42	53%

Table N-5: Required armour size for different damage levels in case of sea ward slope of 1:3

		M50 [kg]	D_{n50} [m]	Reduction
Seaward slope cot(α) [-]	**3**			
Damage level (S) [-]	2	12354	1.67	-
	3	9686	1.54	22%
	4	8151	1.45	34%
	5	7129	1.39	42%
	6	6390	1.34	48%
	7	5826	1.30	53%
	8	5377	1.27	56%
	9	5010	1.24	59%

Table N-6: Required armour size for different damage levels in case of sea ward slope of 1:4

		M50 [kg]	D_{n50} [m]	Reduction
Seaward slope cot(α) [-]	**4**			
Damage level (S) [-]	2	8024	1.45	-
	3	6291	1.33	22%
	4	5294	1.26	34%
	5	4631	1.20	42%
	6	4150	1.16	48%
	7	3784	1.13	53%
	8	3493	1.10	56%
	9	3254	1.07	59%
	10	3055	1.05	62%
	11	2885	1.03	64%
	12	2739	1.01	66%

Table N-7: Required armour size for different damage levels in case of sea ward slope of 1:5

		M50 [kg]	D_{n50} [m]	Reduction
Seaward slope cot(α) [-]	**5**			
Damage level (S) [-]	2	5835	1.30	-
	3	4575	1.20	22%
	4	3849	1.13	34%
	5	3367	1.08	42%
	6	3018	1.04	48%
	7	2752	1.01	53%
	8	2540	0.99	56%
	9	2366	0.96	59%
	10	2221	0.94	62%
	11	2098	0.93	64%
	12	1991	0.91	66%

Table N-8: Required armour size for different damage levels in case of sea ward slope of 1:6

		M50 [kg]	D_{n50} [m]	Reduction
Seaward slope cot(α) [-]	**6**			
Damage level (S) [-]	2	4439	1.19	-
	3	3480	1.10	22%
	4	2928	1.03	34%
	5	2561	0.99	42%
	6	2296	0.95	48%
	7	2093	0.92	53%
	8	1932	0.90	56%
	9	1800	0.88	59%
	10	1690	0.86	62%
	11	1596	0.84	64%
	12	1515	0.83	66%

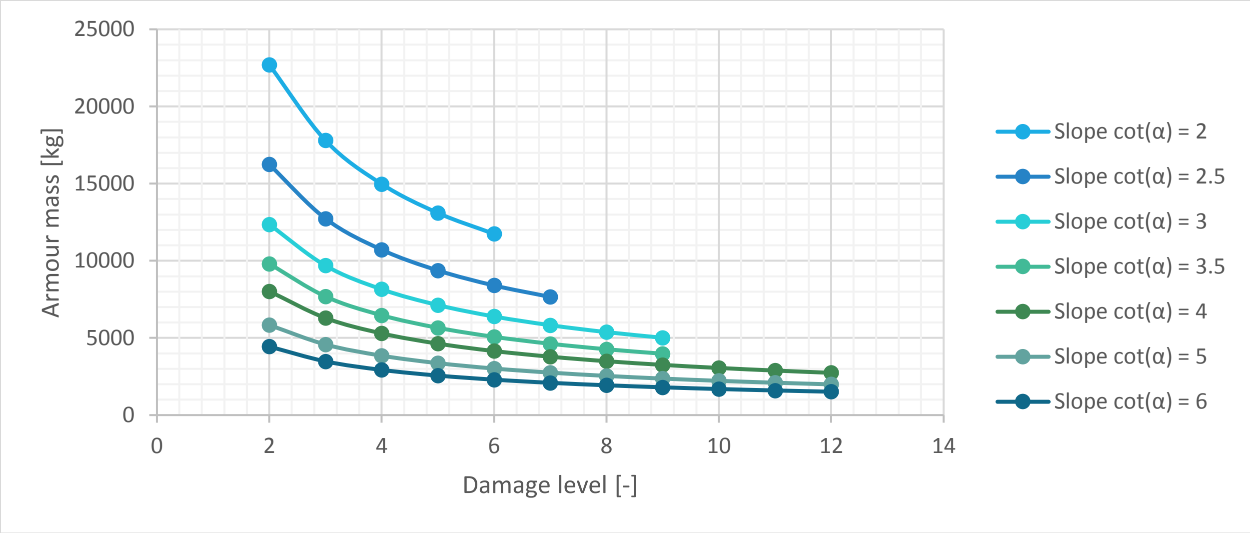


Figure N-2: Required armour mass for different damage levels and seaward slopes, 1/50 years condition, Hs = 5.7 m, Tm = 9.1 s

From Figure N-2 can be concluded that using a 1:2 slope requires a very large armour size. The required armour size decrease significantly for gentler slopes. One should be aware that the consequences of the damage level differ for different slopes, as shown in Table N-2. Accepting some intermediate damage can clearly reduce the initial cost for the armour layer. Only in this case the design should preferably include a maintenance road on top of the breakwater to provide easy access.

The same analysis is performed for the 1/100 years wave condition.

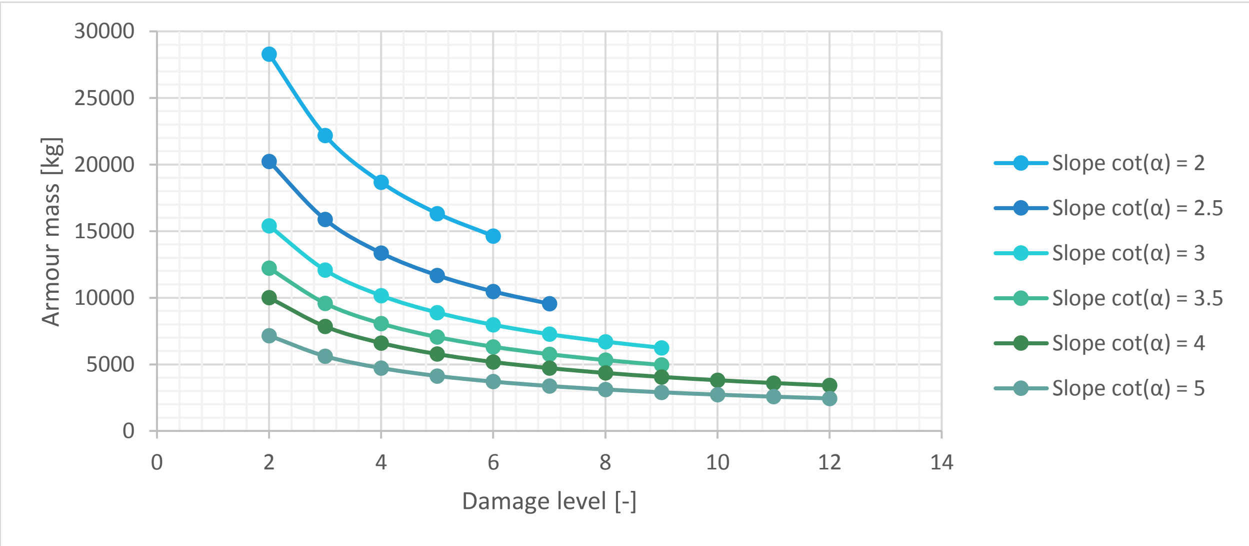


Figure N-3: Required armour mass for different damage levels and seaward slopes, 1/100 years condition, Hs = 6 m, Tm = 9.8 s

The shape of the curves remains the same, although a large increase of required armour size can be observed. This shows the sensitivity of the required armour size for the hydraulic conditions.

Crest height

No activities are planned on top of the breakwater. Although in the future anchorage might be created at the lee side of the breakwater (C. Fernando, 2016).

From the previous figures can be concluded that a gentler slopes leads to a more economic design. The maximum overtopping discharge is set as 10 l/s/m, due to the possible anchorage of vessels in the future (Van der Meer, 2016). The overtopping discharge is a determining factor for the required crest height.

In case of a steeper slope a higher crest height is needed to satisfy the overtopping criterion. A steeper slope results in a smaller cross sectional section, but this section will increase if a higher crest level is needed. An analysis is made to get insight in the influence of the overtopping rate on the general geometry of the breakwater. This analysis is performed for different seaward slopes and for different overtopping discharges. The latest is chosen, because the limit of 10 l/s/m is quite strict. This limit could possibly be increased if vessels are moored more towards the shore and not in lee of the breakwater. This is the case in all the harbour concepts made by the project team. The exact effect of a larger overtopping discharge on for instance the wave height inside the basin should be investigated in a later stage. The following scenarios are considered:

Overtopping discharge:

- Q_{max} = 10 l/s/m
- Q_{max} = 25 l/s/m
- Q_{max} = 50 l/s/m

Seaward slope:

- 1:2
- 1:2.5
- 1:3
- 1:3.5
- 1:4
- 1:5
- 1:6

During the calculations the crest width is estimated by: 3 D_{n50} of the armour layer.

The overtopping calculations are executed with Breakwat, based on the TAW-formula. A roughness reduction factor for the seaward slope of 0.55 is used (CIRIA; CUR; CETMEF, 2007). The 1/50 years wave conditions are used as input.

For Q_{max} = 10 l/s/m

Table N-9: Influence of maximum overtopping discharge on geometrical parameters, case Qmax = 10 l/s/m

Front slope	$[-]$	**2.0**	**2.5**	**3.0**	**3.5**	**4.0**	**5.0**	**6.0**
Freeboard	$[m]$	10.1	9.6	9.2	7.8	6.7	5.3	4.4
Crest height	$[m]$	21.7	21.2	20.8	19.4	18.3	16.9	15.9
Crest width	$[m]$	6.1	5.5	5.0	4.6	4.3	3.9	3.6
Rear slope	$[-]$	2.0	2.0	2.0	2.0	2.0	2.0	2.0
Width	$[m]$	93	101	109	111	114	122	131
Area	$[m2]$	1070	1123	1180	1119	1078	1059	1068

For Q_{max} = 25 l/s/m

Table N-10: Influence of maximum overtopping discharge on geometrical parameters, case Qmax = 25 l/s/m

Front slope	$[-]$	**2.0**	**2.5**	**3.0**	**3.5**	**4.0**	**5.0**	**6.0**
Freeboard	$[m]$	8.7	8.3	7.9	6.7	5.8	4.6	3.8
Crest height	$[m]$	20.3	19.9	19.5	18.3	17.4	16.2	15.3
Crest width	$[m]$	6.1	5.5	5.0	4.6	4.3	3.9	3.6
Rear slope	$[-]$	2.0	2.0	2.0	2.0	2.0	2.0	2.0
Width	$[m]$	87	95	102	105	108	117	126
Area	$[m2]$	944	996	1043	1001	978	976	991

For Q_{max} = 50 l/s/m

Table N-11: Influence of maximum overtopping discharge on geometrical parameters, case Qmax = 50 l/s/m

Front slope	$[-]$	**2.0**	**2.5**	**3.0**	**3.5**	**4.0**	**5.0**	**6.0**
Freeboard	$[m]$	7.7	7.3	7.0	5.9	5.1	4.0	3.3
Crest height	$[m]$	19.3	18.9	18.6	17.5	16.7	15.6	14.8
Crest width	$[m]$	6.1	5.5	5.0	4.6	4.3	3.9	3.6
Rear slope	$[-]$	2.0	2.0	2.0	2.0	2.0	2.0	2.0
Width	$[m]$	83	90	98	101	104	113	122
Area	$[m2]$	859	903	953	918	904	907	929

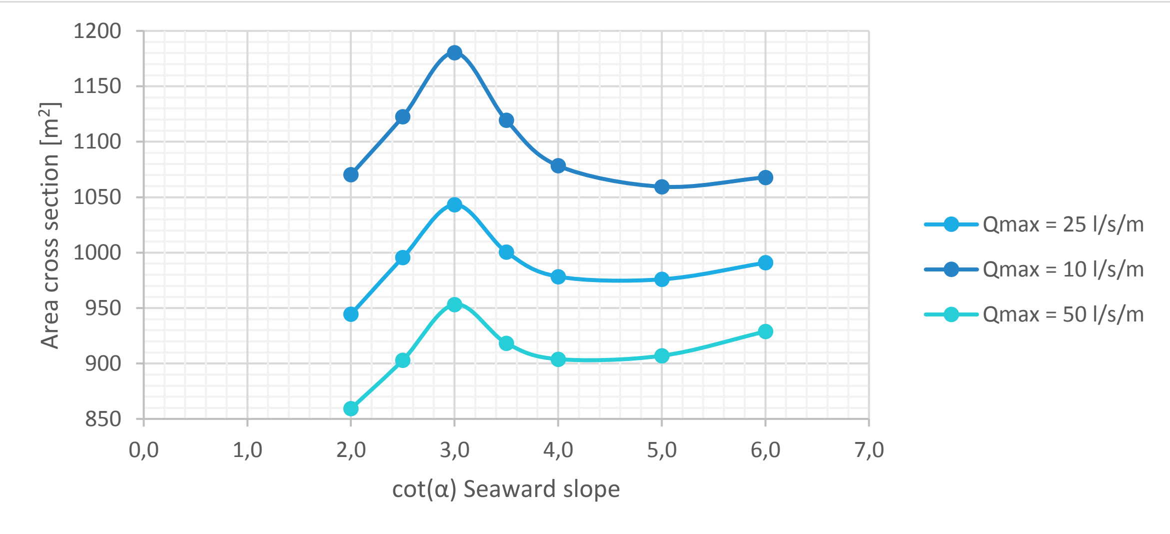


Figure N-4: Cross sectional volume for different seaward slopes and overtopping discharges

Use of less material leads to a decrease of costs. On average a slope of 1:2 needs the least material, but previous is shown that this slope requires very heavy armour stones. When assuming a 1:2 slope not to be feasible, a 1:4 or 1:5 slopes are logical choices depending on the maximum overtopping discharge.

The choice for a gentler slope, 1:4-1:5, results in a large footprint of the structure. For these slopes the footprint is between approximately between 104 and 122 meter, depending on the chosen maximum overtopping discharge.

N.7 Berm breakwaters

A design containing stable units requires heavy armour. It could be that this armour stone is very expensive to obtain, because only a small fraction of quarry output can be used. Or perhaps the heavy units cannot be obtained due to geological limitations. Using quarry stones from quarries outside Sri Lanka will result in very high logistic costs.

Using smaller armour and allowing the waves to reshape the breakwater profile can reduce the costs significantly. Especially when the quarry location is close to the project side. Extra material is needed to assure the function of the breakwater. This material is placed as a berm at the seaward side. This kind of breakwater becomes even more attractive if the costs related to the extra material are not too high. Several types of berm breakwaters exist.

The following types are considered in this report:

Type	Stability number (CIRIA; CUR; CETMEF, 2007)
Partially reshaping berm breakwater (PR BW)	1.5 – 2.7
Fully reshaping berm breakwater (FR BW)	> 2.7

These types have different stability criteria. In case of the statically stable berm breakwater the profile is allowed to reshape into a new profile which is stable and where the individual stones are also stable. In the dynamic case the profile is allowed to reshape into a stable profile, but the individual stones may move up and down the slope (Henk Jan Verhagen, 2012).

The designs are made based on the hydraulic boundary conditions and geometrical design rules (Sigurdarson, 2014).

Boundary conditions
The boundary conditions of the reshaping breakwaters are presented in Table N-12.

Table N-12: Boundary condtions berm breakwater design

Boundary conditions			PR BW	FR BW
Wave height	Hs	[m]	5.7	5.7
Peak period	Tp	[s]	12.4	12.4
Mean period	Tm	[s]	9.1	9.1
Wave direction	β	[deg]	79	79
Gravitational acc.	g	[m/s^2]	9.8	9.8
Relative density	Δ	[-]	1.59	1.59
rho water	ρ_w	[kg/m3]	1025	1025
rho rock	ρ_r	[kg/m3]	2650	2650
Stability nr.	Ho	[-]	2.48	2.97
	To	[-]	23.66	25.89
Design water level	DWL	[m]	11.55	11.55
DW wave length		[m]	129	129
SW wave length		[m]	97	97

Slopes
The initial slopes of the breakwater can be quite steep, because the profile is allowed to reshape. By downward slope at the seaward side is steeper, initiating reshaping.

Table N-13: Slopes reshaping berm breakwaters

Seaside		Harbour side
Upper slope	1:1.5	1:1.5
Down slope	1:1.2 for PR BW 1:35 for FR BW	

Recession and resiliency
The recession length is often expressed by the dimensional number Rec/D_{n50}. By curve fitting a formula is derived, which can be used to estimate the recession length based on the stability number.

$$\frac{Rec}{D_{n50}} = 1.6 \cdot \left(\frac{H_s}{\Delta D_{n50}} - 1.0\right)^{2.5} \text{ (Sigurdarson, 2014)} \quad \text{N-6}$$

Resiliency refers to the capability to cope with extremes, expressed in a percentage P.

Values for P:	Partly reshaping BW	Fully reshaping BW
	▪ P = 20-40%	▪ P < 70 %

Table N-14: Recession and resiliency

Recession and resiliency			PR BW	FR BW
Recession length	Rec	[m]	6.18	10.55
Relative recession	Rec/Dn50	[-]	4.26	8.72
Resiliency	P%	[%]	35	50

Berm

Two design options can be distinguished. A berm at crest level or just above water level. The height of the berm relative to the DWL is estimated by ratio d_B / H_s which should be between 0.5 -0.6. A value higher than 0.6 will lead to a safer design.

The berm width is dimensioned as the sum of the recession length plus a safety margin.

$$B = \frac{Rec}{P\%/100} \qquad B_{min} = Rec + 1\, D_{n50}$$

Table N-15: Geometerical berm parameters

Berm dimensions			PR BW	FR BW
Berm level	d_B	[m]	3.0	3.5
	d_B/H_s	[-]	0.53	0.61
Berm width	B_b	[m]	17.65	21.11
	B_{min}	[m]	7.63	11.76

Crest height and width

Crest height	h_c =DWL + freeboard
Freeboard	h_f = [1.0-1.2] x $H_{s,design}$
Crest width	$B_{crest} = 3\, k_t\, D_{n50}$ with $k_t = 0.87$

Table N-16: Berm breakwater crest dimensions

Crest dimensions			PR BW	FR BW
Free board		[m]	5.7	6.3
	[1.0-1.2]	[-]	1.0	1.1
Crest height	h_c	[m]	17.3	17.8
Crest width	B_c	[m]	3.78	3.16

Toe

Average toe depth: $2.0 < h_t / H_s < 2.5$ with h_t = toe depth

In this case considering the water depth a toe should be built a lower depth. This results in a safer design, but requires a heavier toe. The minimum required stone size is calculated according to (CIRIA; CUR; CETMEF, 2007):

$$\frac{H_s}{\Delta D_{n50}} = \left\{2 + 6.2\left(\frac{h_t}{h}\right)^{2.7}\right\} N_{od}^{0.15} \qquad \text{N-7}$$

Toe dimensions			PR BW	FR BW
Toe depth	h_t	[m]	8.55	7.98
	[2.0-2.5]	[-]	1.5	1.4
Damage level	N_{od}	[m]	2	2
Stability number toe		[-]	5.27	4.75
Lowest level toe		[m]	9.0	9.0
Thickness toe	t_{toe}	[m]	1.5	1.5
	[2-3]	[-]	2.0	2.0
Width toe	B_{toe}	[m]	10	15.0
Minimum nominal rock size	$D_{n50,t}$	[m]	0.7	0.8

Bottom protection
In front of the breakwater the bottom may be affected by erosion due to changing currents. A bottom protection should be placed. The required length should be determined in a later stage. A first approximation is ¼ wave length, thus approx. 30 m.

Wave overtopping – and transmission
Van der Meer and Sigurdarson studied overtopping of berm breakwaters for a large data set. The result is an EurOtop-like formula with an influence factor for the berm:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2 \cdot exp\left(-2.6\,\frac{R_c}{H_{m0} \cdot \gamma_{BB} \cdot \gamma_\beta}\right) \qquad \text{N-8}$$

With:

$$\gamma_{BB} = 0.68 - 4\, s_{op} - 0.05\,\frac{B}{H_s} \qquad \text{For PR BW}$$

$$\gamma_{BB} = 0.70 - 9.0\, s_{op} \qquad \text{For FR BW}$$

Table N-17: Overtopping discharge for designed berm breakwaters

Overtopping			PR BW	FR BW
Berm factor	ybb	[-]	0.41	0.47
Oblique wave attack	yb	[-]	0.76	0.76
Fictitious wave steepness	sop	[-]	0.03	0.03
Overtopping	q	l/s/m	2.05	2.84

Results from physical model tests carried out in the past are used to estimate the transmission coefficient. The majority of the data points show wave transmission coefficients below 0.1 (J. van der Meer, 2016). This implies that with a yearly wave height of 2.85 meter the transmission requirement is satisfied.

Rock classes
A traditional berm breakwater exists of two classes: a class for the armour layer and a class for the core. Splitting the armour layer in two different classes will probably lead to more effective use of material (Sigurdarson, 2014). The partly reshaping breakwater is designed according this design philosophy. The required armour size is calculated based on the stability number of 2.48 for the PR BW and 2.97 for the FR BW. The fully reshaping breakwater has only one armour layer, of 3-6 ton.

The used rock classes are in line with European EN 13383 standard.

Table N-18: Rock classes partly reshaping berm breakwater

	1st Armour layer	6-10t					
W15	7500	[kg]	Dn15	1.41	[m]		
W50	8125	[kg]	Dn50	1.45	[m]		
W85	10500	[kg]	Dn85	1.58	[m]		
	2nd Armour layer	**3-6t**					
W15	3600	[kg]	Dn15	1.11	[m]		
W50	4700	[kg]	Dn50	1.21	[m]		
W85	5250	[kg]	Dn85	1.26	[m]		
	Core	**Quarry run**		**10 mm - 1000 kg**			
W15	2.65	[kg]	Dn15	0.10	[m]		
W50	426	[kg]	Dn50	0.54	[m]		
W85	850	[kg]	Dn85	0.68	[m]		
	D15	<	0.222	m	permeability		
	D85/D15	=	6.85	<	12.0		
	D85	>	0.222	m	stability		
	188	< W50 <	470	kg			
	Toe rocks	**1-3t**					
W15	1500	[kg]	Dn15	0.83	[m]		
W50	1870	[kg]	Dn50	0.89	[m]		
W85	2200	[kg]	Dn85	0.94	[m]		
	Toe filterlayer 1	**45/180 mm**		thickness = 0.5 m			
W15	0.24	[kg]	Dn15	0.05	[m]	45	mm
W50	3.53	[kg]	Dn50	0.11	[m]	110	mm
W85	14.20	[kg]	Dn85	0.18	[m]	175	mm
	D15	<	0.165	m	permeability		
	D85/D15	=	3.9	<	12		
	D85	>	0.165	m	stability		

	Toe filterlayer 2	Fine gravel		**thickness = 0.5 m**			
W15	0.0001696	[kg]	Dn15	0.0040	[m]	4.00	mm
W50	0.0007404	[kg]	Dn50	0.0065	[m]	6.54	mm
W85	0.0019805	[kg]	Dn85	0.0091	[m]	9.08	mm
	D15	<	0.008	m	permeability		
	D85/D15	=	2.3	<	12		
	D85	>	0.0083	m	stability		
	Toe filterlayer 3 (not needed)			**Ocean bottom**			
W15	0.0000011	[kg]	Dn15	0.0008	[m]	0.75	mm
W50	0.0000027	[kg]	Dn50	0.0010	[m]	1.00	mm
W85	0.0000089	[kg]	Dn85	0.0015	[m]	1.50	mm
	D15	<	0.00080	m	permeability		
	D85/D15	=	2.0	<	12.0		
	D85	>	0.00080	m	stability		

Cross section

Horizontal armour width

Sufficient rocks should be placed to prevent the core to be exposed. This criterion is addressed by the horizontal armour width. Defined as the horizontal distance from the seaward slope of the armour to the transition to the core. The distance is obtained by multiplying the design wave height with a parameter.

For partly reshaping berm breakwaters this factor lies between: 4-5
For fully reshaping berm breakwaters this factor lies between: 5-6

Table N-19: Horizontal armour width berm breakwaters

Armour width			PR BW	FR BW
Hor. armour width	A_h	[m]	23.37	31.35
	[4-6]	[-]	4.1	5.5

Transition from rock class I to class II

If several sorted rock classes are used it is economical at a distance below LAT to make a transition between the two classes, because of absence of wave motion at this level.

Class I stones should reach 1.45 $\Delta D_{n50,class\ I}$ or 1.85 $\Delta D_{n50,class\ II}$

Based on earlier designs a limit of 0.4 $H_{sdesign}$ is used (Sigurdarson, 2014).

Table N-20: Transition length between rock classes I and II

Transition length between sorted rock classes			PR BW	FR BW
Distance to class II	$h_{I\text{-}II}$	[m]	3.3	2.8
		[m]	3.6	-
	$h_{I\text{-}II,\ max}$	[m]	2.3	2.3

Table N-21: Cross sectional area berm breakwaters

Cross sectional area			PR BW	FR BW
Area	$A_{cross\ section}$	[m^2]	710	780

The required cross sectional area is considerable smaller in comparison with the conventional breakwaters.

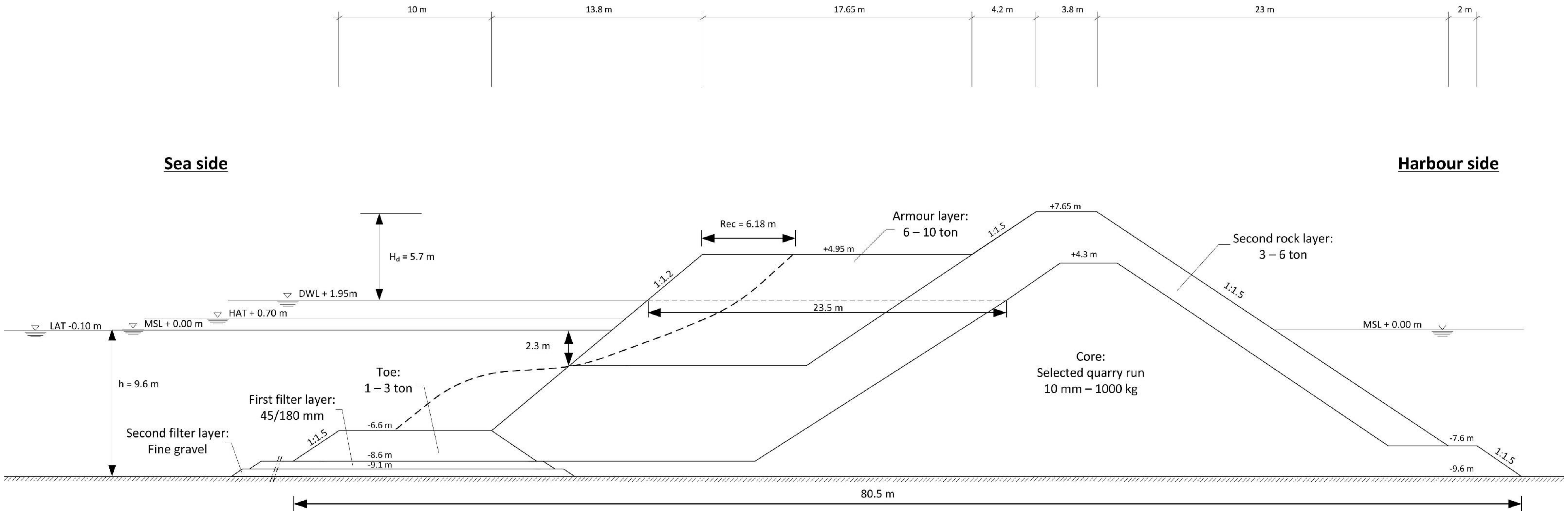


Figure N-5: Typical cross section partly reshaping berm breakwater

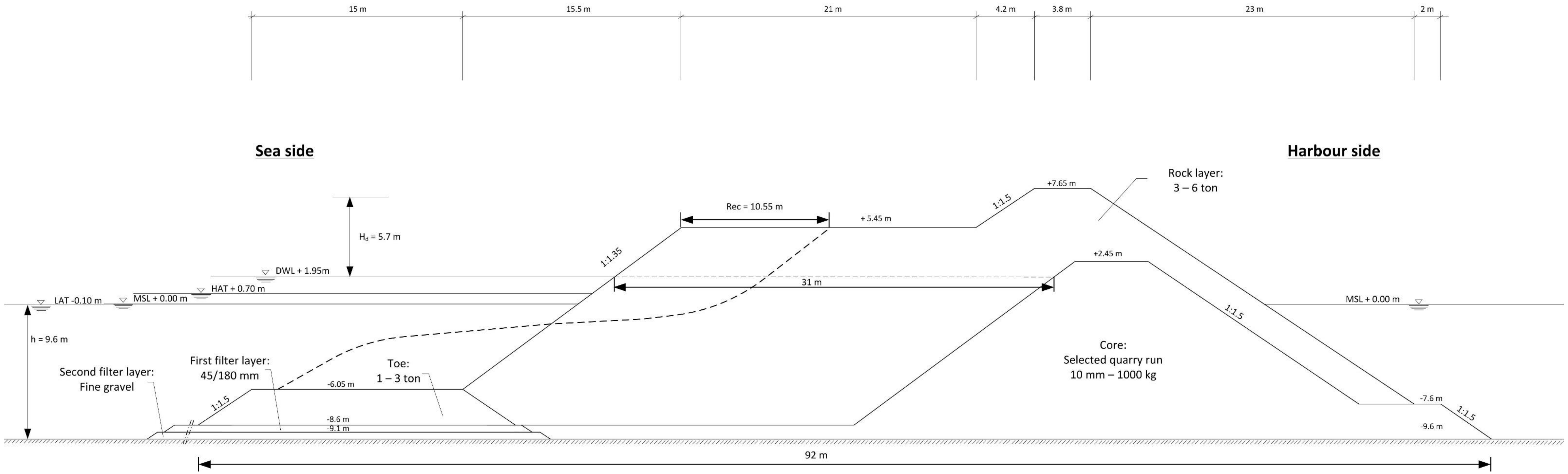


Figure N-6: Typical cross section fully reshaping berm breakwater

N.8 CONCLUSION

Conventional rubble mound breakwater
A first optimization analysis is performed based on different slopes and different overtopping rates. In conclusion, a relative steep slope of 1:2 results in the smallest cross sectional volume considering overtopping discharges of 25 – 50 l/s/m. Although this slope requires very heavy armour in the order of 20 ton. Slopes of 1:4 – 1:5 seem to result in the most economical designs: the increase in cross sectional volume is small and armour layers of 6 and 8 ton are needed respectively.

Overtopping rates
The crest height and the slope are the two factors which determine mainly the cross- sectional volume of the breakwater. Higher acceptable overtopping rates result in a smaller freeboard, thus in a considerable smaller volume. Besides, a gentler slope requires also less freeboard. For this reason, in the concepts made by the project team no anchorage or other activities are located directly behind the breakwater. An overtopping discharge of 10 l/s/m is assumed due to possible anchorage behind the breakwater in the future, as mentioned by Mr. Fernando. Increasing the maximum amount of wave overtopping during the 1/50 years conditions are in favour of a traditional breakwater design, because the high freeboard is directly linked to this property. Decreasing this requirement to 50 l/s/m results in a decrease of the cross-sectional area by approximately 15%.

Damage level
It seems accepting intermediate damage during the 1/50 years design storm results in a decrease between 34-66 % in rock size depending on the exact damage level and the chosen slope. Question remains if this reduction balances the maintenance costs. Furthermore, a maintenance road should be included in the design in this case.

Berm breakwaters
Using smaller armour and allowing the waves to reshape the breakwater profile can reduce the costs significantly. Especially when the quarry location is close to the project side. Extra material is placed as a berm at the seaward side. Too types of berm breakwaters are considered:

- Statically stable reshaping berm breakwater
- Dynamically stable reshaping berm breakwater

In conclusion, the material required for one of the berm breakwaters is approximately 30% less than for a traditional multi layered breakwater. Exact figure depends on the chosen slope, overtopping rate and accepted damage level.

Table N-22: Geometerical properties breakwater types

		Multi layered BW	Partly reshaping berm BW	Fully reshaping berm BW
Front slope	[-]	1:5	1:1.2 - 1:1.5	1:1.35 - 1:1.5
Freeboard	[m]	5.3	7.7	7.7
Crest height	[m]	16.9	17.3	17.3
Crest width	[m]	4	4	4
Rear slope	[-]	1:2	1:1.5	1:1.5
Width	[m]	122	81	92
Cross section	[m^2]	1059	710	780

O. EVALUATION

O.1 EVALUATION

In this section the evaluation is discussed in more detail. The different criteria are explained and per concept discussed and compared. First the boundary conditions will be explained, after which the social and economic criteria are discussed. The used scales can be found in Table O-1Table O-1: Scale used for evaluation the concepts.

Table O-1: Scale used for evaluation the concepts.

	1	**2**	**3**	**4**	**5**
(A)	Very poor	Poor	Satisfactory	Good	Very good
(B)	Very large	Large	Moderate	Limited	Very limited
(C)	Very limited	Limited	Moderate	Large	Very large

Boundary conditions

Draught

A draught of 3.0 m must be present after construction. This ensures the MDOV's to safely navigate in the harbour while keeping accumulation space for future sediment. If this draught is not yet present, it must be dredged to the required depth.

Capacity

The improved harbour must be able to berth and handle 80 to 100 multiday vessels. This is the anticipated future of the Dodanduwa harbour. The vallams do not require any berthing facilities, the beach to land on is sufficient.

Safety

The current breakwater does not provide a safe entry to the harbour bay. An improvement must ensure safe navigation into or out of the harbour bay.

Accessibility

The accessibility of the harbour is of importance for the loading and unloading at the harbour facilities. This requires the river mouth to be open all year. Sedimentation of the river mouth is allowed if it stays within the buffer zone of the 3.0m depth, closure should be avoided.

Durability

Engineering durability

The engineering durability is defined as the expected lifetime of structures, and the ease at which broken structural components can be replaced if the lifetime is not sufficient.

Efficaciousness

The functional durability is more focussed on if the design is 'future resistance'. To what extent is a harbour design able to handle a growing fleet, accumulating sediment in parts of the harbour and the needed dredging frequency.

Extent of works

The re-designing of the harbour area involves different components to be constructed or removed. In the extent of work, all work that has to be carried out in the form of structures or groundwork, is evaluated. Note that this is not expressed in money due to the unknown costs of each work. All parts together are graded.

Socio-economic impact

The socio-economic effects are distinguished in positive and negative effects caused by the changes to the harbour area. Possible effects are an increase in fishermen, stimulating the local economy; generation of new jobs, increase in fish production; but also increase in traffic; land dispossession and possible salt intrusion.

The evaluation of the socio-economic impacts handles both positive and negative effects.

Environmental impact

Environmental impacts are the effects of the new harbour area on the surrounding nature, focussed on effects on coastline, fish population and salinity. All effects are negative.

O.2 Evaluation concept 1

Introduction

The first conceptual design features the use of the northern bay as the new harbour bay, in combination with an improvement of the old harbour basin. The conceptual designs will be evaluated by testing the design on the boundary conditions and other evaluation criteria.

Boundary conditions

In this section the boundary conditions imposed on the project are briefly discussed.

Draught

The minimally required depth of the harbour is 3.0 meters, which is induced by the PI&MU. Concept 1 meets this requirement.

(✓), satisfies the condition.

Anchorage

In conceptual design 1 the number of multiday vessels which can berth in the harbour is 110. Moreover, the northern harbour basin provides landing places for 100 vallams. Based upon the requirements from the PI & MU and EML consultants the number of anchorages provided by the new design meets the demand.

(✓), satisfies the condition.

Safety

The safety of the harbour entrance is ensured by the orientation of the new breakwaters. Facing the north-west, these breakwaters provide an entrance without severely shoaling waves. Furthermore, beacons on the breakwater head provide clear sight at the harbour entrance. Next to the improved wave conditions at the harbour entrance, the harbour bay is protected from waves as well. These calm conditions at the entrance and in the bay provide for safe navigation.

In this conceptual design, the river mouths in the harbour bay. The harbour facilities are reached by navigating up the river. The river current, during wet season, might cause hindrance at the fish unloading and refuelling facilities.

Entrance:	A: Good (4)
Unloading facilities:	A: Satisfactory (3)

Accessibility

Ideally the harbour facilities can be reached through-out the year, only extreme weather might cause inconvenience and down-time of the harbour. This concept ensures limited sedimentation, sediment input from the north is blocked and the mechanism that causes sediment transport inside the bay is reduced. The demanded draught of 3.0 m, mentioned before, is needed to provide a buffer for possible sedimentation. Hence, dredging of the harbour every few years is inevitable.

(✓), satisfies the condition.

Durability

The durability of the first conceptual design is divided by the following topics:

Engineering durability

- Jetties — The jetties are constructed with either hardwood or concrete. The expected lifetime of impregnated wood in sea water is 25 years. For concrete the expected lifetime in sea water is 50 years.
- Revetments — Due to the waves produces by the boats and the discharge from the river the banks bordering the river might erode. By designing the river bank with a revetment, the erosion is prevented. Lowering the maintenance of the river banks. With proper installation the life time of the revetment is expected to be 50 years.
- Breakwaters — The expected lifetime of the breakwater is 50 years. Most likely failure takes place during a 1/250 year storm.

Efficaciousness

- Capacity — The harbour facilitates anchorage for ±110 multiday vessels, expected is that over a period of 10 years the number of vessels using the harbour is increasing to a total of ±80-100. The harbour is therefore expected to fulfil the requirements for at least the coming 10 years. However, great caution is needed for these predictions.
- Accessibility — With the additional draught it is expected that dredging frequency will not exceed CFHC regulation, namely every five years.

Engineering durability: A: Good (4)

Efficaciousness: A: Good (4)

Extent of works

The construction of the harbour can roughly be divided into several different structures/works, as presented in the list below. The works are not presented in a particular order.

- Extension of the old breakwater
- Construction of the new breakwater at the northern outcrop
- Dredging/ground works
- Construction of the jetties
- Construction of the revetments

Table O-2: Crude estimation of the extent of the works

Structure/works	Amount	Dimension
Breakwater	350	[m]
Dredging	1.0	[hectare]
Revetment	350	[m]
Jetties	430	[m]

Total extent of work: B:Moderate (3)

Socio-economic impact

After construction the new harbour design will change the economic activities in and around the Dodanduwa fishery harbour. The new harbour provides anchorage for 110 multiday vessels, attracting fishermen owning such a vessel to anchor in the Dodanduwa fishery harbour.

Positive effects

- Increase in fishermen — The increase in capacity of the Dodanduwa harbour is expected to increase the amount of fishermen using the harbour. Fishermen might move their vessels towards Dodanduwa.
- Generating jobs — The current facilities in the new harbour need to be employed, due to the expected increase in activities more employees might be needed.
- Production increase fish — By increasing the fishing activities in the harbour, the weekly fish catch is growing as well. The multiday vessels fish in different waters compared to the vallams, resulting in a bigger fish catch.
- Local inhabitants — Around the harbour facilities, local shops might benefit from increased activities

Negative effects

- Traffic increase — The increased fishing activities result in more regional traffic. Since Dodanduwa is only reached through the one lane Galle-Colombo road, heavier traffic might cause inconvenience.
- Paddy farmers — The permanent connection between the lake and the ocean might cause reduction of the crops harvested, due to the change in salinity of the groundwater.
- Local inhabitants — If expansions of the harbour facilities is needed, they might have to relocate their business. Local landowners have to be compensated as a result of the slight adjustment of the harbour basin.

Impact positive effects: C: Large (4)

Impact negative effects: B: Moderate (3)

Environmental impact

The improved harbour causes changes to its surrounding with respect to the environment. The breakwater effects the coastline; the increase of fishing activities decreases the local fish population. Lastly, the permanent connection of the sea with the lake might cause salt water intrusion.

Coastline changes
The extension of the breakwater possibly results in erosion at the beach north of the Dodanduwa harbour area. Within the harbour bay the coastline is expected to stabilise. There are no effects to the south of the harbour.

Fish population
More fishing activities in the Dodanduwa area results in a decrease in the local fish population. The fish population in this area is already fully exploited (United Nations Food and Agricultural Organization, 2003). Near shore populations might be restored due to the increase in deep sea fishing from the multi-day fishing vessels, assumed a shift in use of vallams to multiday vessels.

Saltwater intrusion
In the first conceptual design the sediment is partly prevented from entering the harbour area. The natural blockade of the river entrance will therefore not occur resulting in possible saltwater intrusion to the Ratgama Lake.

Impact negative scenario: B: Moderate (3)

O.3 EVALUATION CONCEPT 2

Introduction

The second conceptual design is using the northern bay for the anchorage of the fishery boats. The northern bay is using the old river mouth as harbour basin, the river is diverted towards the south and mouths in the southern bay. The conceptual designs will be evaluated by testing the design on the boundary conditions and other evaluation criteria.

Boundary conditions

In this section the boundary conditions imposed on the project are briefly discussed.

Draught
The minimally required depth of the harbour is 3.0 meters, which is induced by the PI&MU. Conceptual design 2 meets this requirement.

(✓), satisfies the condition.

Capacity
In conceptual design 2 the number of MDOV's which can anchor in the harbour is 110. Moreover, the northern harbour basin provides landing places for 100 vallams. Based upon the requirements from the PI & MU and EML consultants the number of anchorages provided by the new design meets the demand.

(✓), satisfies the condition.

Safety
The safety of the harbour entrance is ensured by the orientation of the new breakwaters. Facing the north-west, these breakwaters provide an entrance without severely shoaling waves. Furthermore, beacons on the breakwater head provide clear sight at the harbour entrance. Next to the improved wave conditions at the harbour entrance, the harbour bay is protected from waves as well. These calm conditions in the entrance and the bay provide for safe navigation.

In this conceptual design, the river mouths in the southern harbour bay. The harbour facilities at the basin can be reached from the northern harbour bay. Since the river is not mouthing in the harbour basin the river is not expected to cause any hindrance at the harbour facilities.

Navigation lights located at the breakwaters will provide guidance for fishermen leaving or approaching the harbour at night or times of less visibility.

Entrance: A: Good (4)

Unloading facilities: A: Very Good (5)

Accessibility
Ideally the harbour facilities can be reached through-out the year, only extreme weather might cause inconvenience and down-time of the harbour. This concept ensures limited sedimentation, sediment input from the north is blocked. The demanded draught of 3.0 m as mentioned before, is needed to provide a buffer for possible sedimentation. Hence, dredging of the harbour every few years is inevitable.

The jetties and beach inside the harbour bay are all-year accessible for the MDOV's and vallams, blockage of the harbour and basin entrance is not expected to occur since the sediments transport is blocked by the breakwaters.

Durability
The durability of the second conceptual design is divided in two categories, namely engineering durability and efficaciousness.

Engineering durability

- Jetties — The jetties are constructed with either hardwood or concrete. The expected lifetime of impregnated wood in sea water is 25 years. For concrete the expected lifetime in sea water is 50 years.
- Revetments — Due to the waves produces by the boats and the discharge from the river the banks bordering the river might erode. By designing the river bank with a revetment, the erosion is prevented. Lowering the maintenance of the river banks. With proper installation the life time of the revetment is expected to be 50 years.
- Breakwaters — The expected lifetime of the breakwater is 50 years. Most likely failure takes place during a 1/250 year storm.
- Quay wall — The quay wall designed at the harbour basin is constructed with gabions filled with stones. The construction is expected to last at least 50 years, with a minimum of maintenance needed.

Efficaciousness

- Capacity — The harbour facilitates anchorage for ±100 multiday vessels, expected is that over a period of 10 years the number of vessels using the harbour is increasing to a total of ±80-100. The harbour is therefore expected to fulfil the requirements for at least the coming 10 years. However, great caution is needed for these predictions.
- Accessibility — With the additional draught it is expected that dredging frequency will not exceed CFHC regulation, namely every five years.

Engineering durability: A: Good (4)

Efficaciousness: A: Good (4)

Extent of works
The construction of the harbour can roughly be divided into several different structures/works, as presented in the list below. The works are not presented in a particular order.

- Extension of the old breakwater
- Dredging of new harbour basin
- Claimed land and dredging at new river trajectory
- Land reclamation at old river trajectory
- Construction of the new breakwater at the northern outcrop
- Construction of the jetties
- Construction of the quay wall
- Construction of the revetments

Table O-3: Crude estimation of the extent of works

Structure/works	Amount	Dimension
Breakwater	350	[m]
Dredging	2.0	[hectare]
Claimed land	1.5	[hectare]
Land reclamation	1.0	[hectare]
Jetties	330	[m]
Revetment	300	[m]
Quay wall	70	[m]

Total extent of work: B: Large (2)

Social economic impact

After construction the new harbour design will change the (work) activities in and around the Dodanduwa fishery harbour. The new harbour provides anchorage of for 100 multiday vessels, attracting fishermen owning such a vessel to anchor in the Dodanduwa fishery harbour.

Positive effects

- Increase in fishermen — The increase in capacity of the Dodanduwa harbour is expected to increase the amount of fishermen using the harbour. By facilitating anchorage for multiday vessels fishermen using these vessels will use the harbour as well. Fishermen might move towards Dodanduwa, resulting in an increase in the population of the village.
- Production increase fish — By increasing the fishing activities in the harbour the fish production is growing as well. The multiday vessels fish in different waters compared to the vallams, resulting in a wider assortment of fish.
- Generating jobs — The facilities in the new harbour needs to be employed, resulting in extra jobs available.

Negative effects

- Property owners — The new river trajectory might cross private owned land, the owners of these properties need to be compensated for using their property.
- Traffic increase — The progressive amount of fishermen will result in an increase in traffic as well. The local infrastructure might need to be improved when the traffic becomes too dense.

Impact positive scenario: C: Large (4)

Impact negative scenario: B: Limited (4)

Environmental impact

The new harbour design will have an impact on the environment. In the assessment the coastal changes, fish diversity and population and the saltwater intrusion are considered.

Coastal changes

The extension of the breakwater possibly results in erosion at the beach north of the Dodanduwa harbour area. Within the harbour bay the coastline is expected to stabilise. The new location of the river mouth is not expected to cause severe changes to the coastline.

Fish population

More fishing activities in the Dodanduwa area results in a decrease in the local fish population. The fish population in this area is already fully exploited (United Nations Food and Agricultural Organization, 2003). Near shore populations might be restored due to the increase in deep sea fishing from the multi-day fishing vessels.

Saltwater intrusion

In the second conceptual design the river is diverted towards the south, mouthing in the southern bay. Equal to earlier situations when the river naturally mouthed in the southern bay, natural processes take over which limit the salt water intrusion

Impact negative scenario: C: Limited (4)

O.4 EVALUATION CONCEPT 3

Introduction

The third conceptual design uses the southern bay as harbour basin, together with the expanded basin at the lagoon. The river connecting the Ratgama Lake with the Indian ocean is diverted towards the southern basin. The conceptual design will be evaluated by testing the design on the boundary conditions and other evaluation criteria.

Boundary conditions

In this section the boundary conditions imposed on the project are briefly discussed.

Draught
The minimally required depth of the harbour is 3.0 meters, which is induced by the PI&MU. Conceptual design 3 meets this requirement.

(✓), satisfies the condition.

Capacity
In conceptual design 3 the number of MDOV's which can anchor in the harbour is 72, in the basin extra anchorage is facilitated for 24 MDOV's. Moreover, the southern harbour basin provides landing places for 100 vallams. Based upon the requirements from the PI & MU and EML consultants the number of anchorages provided by the new design meets the demand.

(✓), satisfies the condition.

Safety
The safety of the harbour entrance is ensured by the orientation of the new breakwater, with the entrance facing the north-west direction. The breakwater provides an entrance without severely shoaling waves. The beacons on the breakwater head provide clear sight at the harbour entrance. Next to the improved wave conditions at the harbour entrance, the harbour bay in protected from the waves as well. The calm conditions in the entrance and the bay provide for safe navigation.

The river mouth of the river connecting the Ratgama Lake and the Indian ocean is diverted towards the southern bay. Navigation towards the harbour basin is done via this river. The river mouthing in the southern bay is expected to cause some hindrance during the wet season, as the discharge of the lake enters the basin.

Entrance: A: Good (4)

Unloading facilities: A: Satisfactory (3)

Accessibility
The harbour basin is designed to be accessible all-year, only extreme weather might cause inconvenience and down-time of the harbour. This concept ensures only limited sedimentation transport will take place in the harbour. The sediment transported along the coast will be blocked by the old breakwater. The mechanisms causing sediment transport inside the harbour are reduced. The required draught of 3.0 meter is ensures a buffer for possible sedimentation. Since some sediments will still be able to enter the harbour, dredging every few years is inevitable.

(✓), satisfies the condition.

Durability

The durability of the second conceptual design is divided in two categories, namely engineering durability and efficaciousness.

Engineering durability

- Jetties — The jetties are constructed with either hardwood or concrete. The expected lifetime of impregnated wood in sea water is 25 years. For concrete the expected lifetime in sea water is 50 years.
- Revetments — Due to the waves produces by the boats and the discharge from the river the banks bordering the river might erode. By designing the river bank with a revetment, the erosion is prevented. Lowering the maintenance of the river banks. With proper installation the life time of the revetment is expected to be 50 years.

- Breakwaters — The expected lifetime of the breakwater is 50 years. Most likely failure takes place during a 1/250 year storm.
- Quay wall — The quay wall designed at the harbour basin is constructed with gabions filled with stones. The construction is expected to last at least 50 years, with a minimum of maintenance needed.

Efficaciousness

- Capacity — The harbour facilitates anchorage for ±96 multiday vessels, expected is that over a period of 10 years the number of vessels using the harbour is increasing to a total of ±80-100. The harbour is therefore expected to fulfil the requirements for at least the coming 10 years. However, great caution is needed for these predictions.
- Accessibility — With the additional draught it is expected that dredging frequency will not exceed CFHC regulation, namely every five years.

Engineering durability: A: Good (4)

Efficaciousness: A: Satisfactory (3)

Extent of works

The construction of the harbour can roughly be divided into several different stages, as presented in the list below. The order in which the stages are presented is not necessarily the order in which the construction works will be conducted.

- Construction of the new breakwater at the southern outcrop
- Reclamation of land at the new basin
- Dredging of the new harbour basin
- Claimed land and dredging at new river trajectory
- Land reclamation at old river trajectory
- Construction of the jetties in bay and basin
- Construction of the quay wall at the basin
- Construction of the revetments
- Removal of rock outcrops in the southern bay

Table O-4: Crude estimation of the extent of works

Structure/works	Amount	Dimension
Breakwater	350	[m]
Dredging	2.0	[hectare]
Claimed land	0.5	[hectare]
Land reclamation	0.7	[hectare]
Jetties	250	[m]
Revetment	350	[m]
Quay wall	160	[m]
Rock removal	200	[m3]

Total extent of work: B: Large (2)

Socio-economic impact

After construction the new harbour design will change the activities in and around the Dodanduwa fishery harbour. The new harbour provides anchorage of for 96 multiday vessels, attracting fishermen owning such a vessel to anchor in the Dodanduwa fishery harbour.

Positive effects

- Increase in fishermen — The increase in capacity of the Dodanduwa harbour is expected to increase the amount of fishermen using the harbour. By facilitating anchorage for multiday vessels fishermen using these vessels will use the harbour as well. Fishermen might move towards Dodanduwa, resulting in an increase in the population of the village.
- Generating jobs — The facilities in the new harbour needs to be employed, resulting in extra jobs available.

▪	Production increase fish	By increasing the fishing activities in the harbour the fish production is growing as well. The multiday vessels fish in different waters compared to the vallams, resulting in a wider assortment of fish.
▪	Local inhabitants	Around the harbour facilities, local shops might benefit from increased activities

Negative effects

▪	Property owners	The expanded harbour basin might covers private owned land, the owners of these properties need to be compensated for using their property.
▪	Traffic increase	The progressive amount of fishermen will result in an increase in traffic as well. The local infrastructure might need to be improved when the traffic becomes too dense.
▪	Paddy farmers	The permanent connection between the lake and the ocean might cause salinity changes in the groundwater, reduction of the crops harvested.

Impact positive effects: C: Large (4)

Impact negative effects: B: Moderate (3)

Environmental impact

The new harbour design will have an impact on the environment. In the assessment the coastal changes, fish diversity and population and the saltwater intrusion are considered.

Coastal changes
With the breakwater layout as presented in concept 3, sedimentation of the northern bay will continue, and will not be flushed out due to the diverted river mouth. Erosion to the coast north of the area of interest is expected; no effects to the southern coast are expected.

Fish population
More fishing activities in the Dodanduwa area results in a decrease in the local fish population which is already fully exploited (United Nations Food and Agricultural Organization, 2003).

Saltwater intrusion
The river is diverted to the south and will be open all-year long. Due to this opening, salinity upstream might increase.

Impact negative scenario: B: Moderate (3)

O.5 Evaluation concept 4

Introduction

The fourth conceptual design features the use of both the northern and southern bay as new harbour bays. This conceptual design will be evaluated by testing the design on the boundary conditions and other evaluation criteria.

Boundary conditions

In this section the boundary conditions imposed on the project are briefly discussed.

Draught
The minimally required depth of the harbour is 3.0 meters, which is induced by the PI&MU. Concept 4 meets this requirement.

(✓), satisfies the condition.

Anchorage
The amount of multiday vessels that can moor in this concept is theoretically up to 132. Beside those places are both beaches available for 140 vallams to land on. This concept amply meets the requirements from the PI & MU and EML Consultants.

(✓), satisfies the condition.

Safety
The orientation of the breakwaters ensure safe navigation in the harbour itself by blocking waves coming from the dominant direction. Lights on the breakwater also provide safe navigation around the breakwater during night.

Harbour facilities can be reached when navigating into the river. A high river discharge during monsoon periods can cause hindrance when mooring to the quay wall. Two entrances are available for entering or leaving the harbour. In case one of the entrances experiences trouble, the other one can still be used.

Due to the size of the harbour and the huge amount of available mooring places, waterborne traffic is less dense, causing a safer navigability. Only, when a lot of ships return and have to unload, the river area might get crowded which can again result in collisions. Furthermore can high river discharges impede mooring and unloading at the quay wall.

Entrance: A: Very Good (5)

Unloading facilities: A: Satisfactory (3)

Accessibility
The harbour is accessible all year long, except for some extreme weather conditions. Sedimentation of the harbour is limited by the breakwater layout, and if some parts of the harbour siltate, are there still the abundantly available mooring places to keep the harbour operational. Sedimentation is not completely blocked, so dredging every few years is inevitable.

(✓), satisfies the condition.

Durability
The durability of the first conceptual design is divided by the following topics:

Engineering durability

- Jetties: The jetties are constructed with either hardwood or concrete. The expected lifetime of impregnated wood in sea water is 25 years. For concrete the expected lifetime in sea water is 50 years.
- Revetments: Due to the waves produces by the boats and the discharge from the river, the river banks might erode. This erosion is prevented by a revetment. With proper installation the revetment is expected to last at least 50 years.
- Breakwaters: The expected lifetime of the breakwater is 50 years. Most likely failure takes place during a 1/250 year storm.
- Quay wall: The quay wall designed at the harbour basin is constructed with gabions filled with stones. The construction is expected to last at least 50 years, with a minimum of maintenance needed.

Efficaciousness

- Capacity: The harbour facilitates anchorage for ±132 multiday vessels, expected is that over a period of 10 years the number of vessels using the harbour is increasing to a total of ±80-100. The harbour is therefore expected to easily fulfil the requirements for at least the coming 10 years.
- Accessibility: With the additional draught it is expected that dredging frequency will not exceed CFHC regulation, namely every five years.

Engineering durability: A: Good (4)

Efficaciousness: A: Very Good (5)

Extent of works

The construction of the harbour can roughly be divided into several different structures/works, as presented in the list below. The works are not presented in a particular order.

- Extension of the old breakwater
- Construction of the new breakwater at the northern outcrop
- Construction of the new breakwater at the southern outcrop
- Dredging/ground works
- Removing rocks in southern bay
- Construction of the jetties
- Construction of quay wall
- Construction of the revetments

Table O-5: Crude estimation of the extent of the works

Structure/works	Amount	Dimension
Breakwater	520	[m]
Dredging	2.5	[ha]
Rock removal	200	[m^3]
Jetties	350	[m]
Quay wall	110	[m]
Revetment	250	[m]

Total extent of work: B: Very Large (1)

Socio-economic impact

After construction the new harbour design will change the economic activities in and around the Dodanduwa fishery harbour. The new harbour provides anchorage for 132 multiday vessels, attracting fishermen owning such a vessel to anchor in the Dodanduwa fishery harbour.

Positive effects

- Increase in fishermen — The increase in capacity of the Dodanduwa harbour is expected to increase the amount of fishermen. Fishermen might lay their vessels in Dodanduwa and other citizens might change their profession to the fishing industry due to the availability of a satisfying harbour.
- Generating jobs — The current facilities in the new harbour need to be employed, due to the expected increase in activities more employees might be needed.
- Production increase fish — Weekly fish catch is growing as a result of the increased fishing activities. It is possible that part of the catch is to be canned and transported to other parts of Sri Lanka or even abroad.

Negative effects

- Paddy farmers — The permanent connection between the lake and the ocean might cause reduction in crops harvested due to the salinity increase of the groundwater.
- Local inhabitants — Around the harbour facilities, local shops might benefit from increased activities. However, if expansions of the harbour facilities is needed, they might have to relocate their business.
- Traffic increase — The increased fishing activities result in more regional traffic. Since there is only the Galle-Colombo road, heavier traffic might cause inconveniences.

Impact positive scenario: C: Large (4)

Impact negative scenario: B: Satisfactory (3)

Environmental impact

The improved harbour causes changes to its surrounding with respect to the environment. The breakwater effects the coastline; the increase of fishing activities decreases the local fish population. Lastly, the permanent connection of the sea with the lake might cause salt water intrusion.

Coastal changes
The extension of the breakwater possibly results in erosion at the beach north of the Dodanduwa harbour area. South of the harbour, no erosion or sedimentation effects are expected. Within the harbour bay the coastline is expected to stabilise.

Fish population
An increase in fishing activities in the Dodanduwa area will result in a decrease in the availability of fish and fish of the proper size. As already discussed before, is the fish population in the area already fully exploited.

Saltwater intrusion
The permanent connection between the river and the northern- and southern bay can result in intrusion of salt water upstream which might result in a lower crop yield.

Impact negative scenario: B: Large (2)

O.6 EVALUATION CONCEPT 5

Introduction
The final conceptual design is basically a solution in which only the basic needs of the fishermen are provided against a minimal amount of work. The evaluation of this fifth concept will be done by testing the design on the boundary conditions and other evaluation criteria.

Boundary conditions
In this section the boundary conditions imposed on the project are briefly discussed.

Draught
The minimally required depth of the harbour is 3.0 meters is not met in this concept, but the focus of this concept is on the current fleet without multiday vessels.

(X), does not satisfy the condition

Anchorage
The concept is focussed on the current vallam fleet; no places for multiday vessels are realized. The amount of vallams that can be placed on the shores of the northern and southern bays is about 140.

(X), satisfies the condition for vallams

Safety
Safety in this concept is realized in another way than the other concepts. Based on an interview with fishermen (See Appendix Q.1), it became clear that just seeing incoming waves was sufficient to navigate safely out of the bay. A buoy is placed behind the breakwater. Outgoing fishermen can see the buoy going up and down and time their navigation out of the harbour on it. A light on top of the buoy make this concept also useable during night.

Furthermore, a jetty where fish can be unloaded is installed along the river bank in the basin. High river discharge during rainy season might temporarily make mooring and unloading more difficult.

Entrance: Satisfactory (3)

Unloading facilities: Satisfactory (3)

Accessibility
The accessibility of the bay does not change from the current situation. During rough weather, vessels still have to find a calmer bay to unload their catch. For times when the bay is navigable, a sediment trap keeps the Ratgama basin open in order to keep the harbour facilities reachable. Dredging of the harbour every few year is necessary to ensure the effect of the sediment trap.

(X), partly satisfies the condition

Durability
The durability of the first conceptual design is divided by the following topics:

Engineering durability

- Jetties: The jetties are constructed with either hardwood or concrete. The expected liftetime of impregnated wood in sea water is 25 years. For concrete the expected lifetime in sea water is 50 years.
- Sediment trap: The materials used for the sediment trap are expected to last for 25-50 years, dependent on their choice. The weakest parts of the trap can easily be replaced.
- Breakwaters: The expected lifetime of the current breakwater is 50 years, not additional breakwater meters are placed. Most likely failure takes place during a 1/250 year storm.

Efficaciousness

- Capacity: The harbour facilitates landing places for ±140 vallams. This concept does not account for the coming of multiday vessels.
- Accessibility: No dredging of the harbour is needed in this concept, except for the part in front of the sediment trap. It is expected that this dredging frequency will not exceed CFHC regulation, namely every five years.

Engineering durability: A: Good (4)

Efficaciousness: A: Very Poor (1)

Extent of works
The construction of the harbour can roughly be divided into several different structures/works, as presented in the list below. The works are not presented in a particular order.

- Construction of the sediment trap
- Dredging/ground works
- Construction of the jetty
- Placement of the buoy

Table O-6: Crude estimation of the extent of the works

Structure/works	Amount	Dimension
Sediment trap	100	[m]
Dredging	0.2	[hectare]
Jetty	80	[m]
Buoy	1	[-]

Total extent of work: C: Very Limited (5)

Social economic impact after construction
After construction the new harbour design will not really change the economic activities in and around the Dodanduwa fishery harbour. The harbour still provides anchorage for the current fleet, no multiday vessels are attracted. Only crop yield for paddy farmers might drop due to the permanent connection to salt water.

Impact positive scenario: B: Very limited (1)

Impact negative scenario: C: Very limited (5)

Environmental impact
Except for probable salt water intrusion, no environmental impacts are expected

Saltwater intrusion

In the current situation, the Ratgama lake is seasonally closed off from the ocean, permanent connection to it due to the effects of the sediment trap, might cause salinity upstream.

Impact negative scenario: C: Very Limited (5)

P. MEETINGS

P.1 FIRST PROCESS MEETING

Present: Team Project Dodanduwa
Mr. C. Fernando
Date: 09-09-2016

Questions

In which phase is the project at the moment?

- After the first report made by EML not much happened, had to wait for the consent of the ministry and they just received it.
- The survey of the area will consist of 2 parts, the first part takes place in the water. This survey can only be conducted when the sea is quiet, at the moment it is not possible. At the end of September the sea will be calm and the survey will be possible. The second part of the survey is on land and consists of boreholes (and possible SPT's?).
- The harbour needs to have facilities for (bigger) mechanised boats. This can be a jetty facility around the bedrock and breakwaters to keep the lagoon open all year.
- The monsoon starts in October and ends in November. During this monsoon the sand wall will be breached. During the monsoon most of the fishermen won't go fishing because of the rough sea. These fishermen will have different jobs in this period, i.e. drivers and in labour.

Why do they want the harbour in the lagoon?

- The aim is to make facilities for bigger boats (10-15m) called 'mantitables'. These boats have their engine on the inside. These boats are locally produced and have the advantage that they can be out at sea for 1 to 5 weeks instead of the 1 day trips one can make with a vallam. When the breakwater is finished these boats will come to the Dodanduwa harbour.
- The draft inside the lagoon might be a problem.
- The slope of the beach in the lagoon is 1:5 – 1:7 and goes to a depth of 7m.

How many boats will there be in the new harbour?

- There will be roughly 250 traditional boats and 20 big boats after the harbour is operational.

Solution Mr. Fernando was thinking about:

- Extension of the breakwater as it is right now, option 1 is to extend it towards the sea, so the breakwater will have an curve in the middle. The downside of this design are the higher construction costs, since the breakwater will be constructed in deeper water. However, this design makes the entrance of the harbour much safer for the fishermen to navigate in. A part of the new breakwater can be designed as an reef structure. An optimisation in this design is needed.
- The second option Mr. Fernando proposed is to extend the current breakwater with the same construction as the current breakwater.
- The soil in the harbour is mainly sand, there isn't much mud so there will be no site investigation, only at the quay walls.

Why did the current breakwater fail?

- The failure of the current breakwater might be due to one or more of the following:
    - Construction not been done properly
    - Undersized armour stones
- The contractor will repair the breakwater, the contract has already been rewarded.

Is it known which quarry will supply the armour stones for the breakwater?

- Survey still needs to be concluded, but after the armour stones have been dimensioned in the design.

What about the salt intrusion of the water inside the lagoon?

- When the lagoon is connected to the sea all year, salt water will intrude inside the lagoon. The ministry however is not funding any research regarding this intrusion, but EML has to find a solution.
- A possible solution is to construct gates in the river which close during flood tide. During rainy season these gates will be open. This is however not included in the scope so there is no money available for this solution.
- The paddy farms around the Ratgama lagoon are not using the water of the lagoon for irrigation, they use rainwater. The salt intrusion of the lagoon will have influence of the groundwater which can influence the drinking water.
- Every month there is a meeting in which several stakeholders discuss the development plan.

The design of sand traps

- In the design a choice have to be made whether there will be sand traps or a quay wall, there is no space for both in the design. Extending the harbour towards the land is no option, but maybe for anchor places?
- The harbour needs to be cleaned every few years, once in the 3 -5 years is acceptable, more frequent cleaning is not acceptable.

The fishermen want a harbour close to home. The fishermen who work at the bigger boats have to go to Galle in the current situation. They would prefer to work at the Dodanduwa harbour but there are no facilities for the bigger boats.

During the district meetings they do not want counselling, they just want to have designs.

What data is available for us?

- EML has wave condition data which will be given to the project group soon.
- NARA has tide data, no wave data. This data is not very useful since the tidal difference in water level is not that big, maximum is 0.7 m.
- Boreholes from TOR (Term Of Reference) are not available yet.
- Mr. Fernando can arrange a meeting with the project management/direction of the client.

Request from Mr. Fernando:

- It would be nice to have some (2D) models of how the waves and erosion might take place in the new harbour. Fernando will supply all the data we need to build such a model. The project group did not accept to build a model, they might if it is relevant for their own project and if there is time do built one.
- The software used for modelling is MIKE21. It is a free modelling program.

P.2 SECOND PROCESS MEETING

Present:	Team Project Dodanduwa
	Mr. C. Fernando
Date:	20-09-2016

Questions

Who made the present breakwater? Contractor/ Client?

- The client of the current breakwater was the Ceylon Fishery Harbour Corporation. The breakwater is built by a private contractor and funded from outside by the UN project services.
- During construction the shoreline was situated more inland, near the current concrete pavement. The water depth around the breakwater is 7 meter.

Sedimentation of the river mouth could be expected upfront. Why did the Coast Conservation department approved the project?

- The Coast Conservation department is not checking the structural specifications of for instance a breakwater design. The task of the department is to prevent severe erosion. They will not give approval to project which will result in severe erosion. According to Mr. Fernando some erosion occurred north from the breakwater.
- During the NE monsoon (December – March) sediment is directed southwards. Sand is not by passing the southern rock outcrop.

What mechanisms are mainly responsible for the sedimentation of the mouth?

- The sandbar at the entrance of the harbour basin is mainly formed by:
  Diffracted waves, which also stir up the sediment
  Flood tide, which transports the sediment to the mouth and further upstream.
  During dry season there is no opposing force from the river. At the end of October the river is strong enough, and the sea becomes calm.
- During the rainy season the river discharge and the ebb tide flush away the sediment.

How is the flow and sediment transport around the rocky outcrops?

- Sediment is bypassing the northern rocky outcrop. The southern rocky outcrop is a littoral barrier; no sand is bypassing this out crop. Plans are made to reinforce the northern outcrop to prevent bypassing. This will probably limit sedimentation in the bay and stabilize the North beach.
- During the NE monsoon transport mainly to the South, but at some places to the North depending on the coastline orientation. Annual sediment transport at Dodanduwa is almost zero, but gross transport should not be forgotten. Sand is transported into the bay during the NE monsoon. Maybe convienent to model the transport rates with Unibest. Transport is approx. 30-40 thousand m^3/year.

Bathymetry survey

- Recently bathymetry study for the river section is performed. Bathymetry of the bay is planned in the end of September, begin October, during more calm wave conditions. In the meanwhile, the project team can use charts, which Mr. Fernando will provide.

Nourishments

- Around 2002 several stretches are artificial nourished. Among others area close to the wetland in Galle and the area north of Hikkaduwa.

Any data from the region or site investigation plan for the current site available?

- Taking samples from boreholes is scheduled for coming month. No data available yet. The survey is executed preferably during the dry season, because this is cheaper. Research will be performed focussed on constructing a jetty or quay wall. It is not common to preform geotechnical study when building a breakwater.

Which is quarry used to produce stones for the present breakwater? Are there any quarries situated in the region which could produce armour stone?

- Material for the current breakwater is produced in quarry which is 40 km away from the project location. There are several quarries in the region, but is not clear what kind of quality/sizes they are capable to produce. A study is planned, because last study is performed ten years ago. A map of registered quarries can be obtained from the GSMB, the same place where the project team obtained the geology map earlier.

Waves

- The wave data provided by Mr. Fernando is based on buoys near Galle. Several organizations deployed buoys in this region at different depths. Buoys at a depth of 70 m will mostly track swell waves with a long period. Most preferably using data from a buoy located at 15-20 m depth line, where the waves are not yet affected by shoaling.

Interview with stakeholders

The project team hoping to be able to interview employees of the following organizations:

- Ceylon Fishery Harbour Corporation (CFHC)
- Coast Conservation and Coastal Resource Management Department (CC&CRMD)
- Project implementation and monitoring unit (PI&MU)
- Fishery organisations?
- Geo engineer from project team (Prof. S. Thilakasiri).

Final words of Mr. Fernando

Would be convenient for both parties to carry out some modelling. Unibest can be used to get some insight in the sediment transport. A more complex model can be used to set up a harbour disturbance model, for instance with use of Delft3D. Convenient to get insight in wave penetration and possible resonance problems. The project team will first think on possible conceptual designs.

The East coast has almost a straight coastline. Around 1996 a harbour is built at the East coast. The project was financed using Danish funds. The Danisch provided a grant a loan. The breakwater was constructed at a depth of 8 m. In those years the Coastal Conservation department was not well equipped to study effects upfront. Since construction the East coast is threatened by severe erosion. The Sri Lanka institute proposed in the past to build offshore breakwaters to counter the problems. DHV and EML were involved by a project to counter the erosion.

P.3 THIRD PROCESS MEETING

Present: Team Project Dodanduwa
Mr. C. Fernando
Date: 04-10-2016

Questions

What is the capacity that the harbour will be designed for?

- All vessels are expected to unload their fish at jetties close to the onshore harbour facilities. Afterwards the vallams will be landed on the beach, there is nog need for structures for the vallams to moor. At the moment about 28 multi day fishing vessels are in the possession of Dodanduwa fishermen and they are expected to anchor in the harbour when the harbour is finished. The harbour will be designed for a future fleet of multi day fishing vessels of 80-100.

To what extent will the tidal waves transport sediment towards the river mouth once the breakwaters are constructed?

- The tidal waves in this area move at a very slow speed of 0.1-0.5 m/s. Moreover, the tidal wave height is limited to 0.6/0.7 meters. Waves cause most of the sediment transport in this area. The tidal waves are not expected to be a major part of the problem.

Is it possible to meet with the several stakeholders: PI&MU, CC&CMRD, CFHC and ID/ASD?

- Fernando: yes, I will send you the contact details of the PI&MU and CC&CMRD via e-mail. The ID/ASD cannot be contacted due to business concerning the client (PI&MU). Mail address of general manager of the CFHC was received.

Any soil data of bathymetry available yet?

- Bathymetry will most likely arrive this week (03/10-07/10), hardcopy. Soil data will arrive later, not yet clear when.

P.4 FOURTH PROCESS MEETING

Present: Team Project Dodanduwa
Mr. C. Fernando
Date: 05-10-2016

Questions

For the ultimate limit state, which return period should be considered?

- 50 years

For ULS what is acceptable damage return period?

- We take: 5% damage, vd Meer. Better to do some comparison between 2-5% To see changes

Service limit state, what kind of waves are allowed in the basin? With which return period is exceedance of this wave height accepted?

- 0.3 meters normally used, from PIANC publications. Anytime, the limit.

Any facilities on top of the breakwater?

- If they ask for a quay wall, 2 possibilities. Harbour side on the main breakwater or on the shoreface. So on some time they can ask this, so get overtopping as minimum.
  Could a sea wall after breakwater just to cover the jetty/quaywall.
  Mostly, unload fish and refuel at facilities. Then moore somewhere else. So mooring behind breakwater is possible.
- Multi day vessels do need a quay at facilities.
- Fingerjetties widely used by fishermen.

Preference, armour stone or concrete armour? How common is it to use concrete armour units?

- moderate conditions → rock armour
- if deeper and heavier → concrete armour. Is possible here but placing techniques is more difficult. (not economical for shallow waters) EML has experience with rock armour so preferred.

Preference, construction by waterborne equipment or construction via land?

- By land.

What wood do they use?

- Concrete, prefab piles.
- Wooden piles → Normally resisting the water (?). Certain varieties that resist the water easily available.

What is the status of the bathymetry survey

- As soon as the PI&MU has the bathymetry share it with you
- He has the bathymetry of the lagoon area, will pass it on to us.

Where could we find accurate tidal information, cause we are finding some different values?

- Find it at: British admiralty time table. Both Colombo and Galle are given.

Final notes of Mr. Fernando

3-meter depth is done because:

- A 3m depth gives enough space for accumulation for the sediment. This will lead to less frequent dredging.

Quay wall:

- Gabions often used. This is a cheap solution.

Q. INTERVIEWS

Q.1 FIRST SITE VISIT: FISHERMEN

Interview with: Fishermen ‘Chameer’
Date: 06-09-2016

General Questions

- No general questions were asked

Specific Questions

- **How do you experience fishing in the Dodanduwa area?**
  Fishing is done to provide livelihood. Locals mostly catch the smaller tuna, up to about 80cm. The bigger ones are caught by bigger ships. Sometimes there is nearly no tuna to catch. Bigger ships probably catch them all or scare them away.

  He also explained that the colour of the ocean is very important for fishing during the day. A greenish ocean results in a way bigger catch than a blue ocean. This because in a blue ocean, the fish can see the net and avoid it.

- **What do you think of the Dodanduwa harbour itself, is it a good/safe harbour?**
  During bad weather conditions the harbour is not really accessible for fishermen still on the ocean (especially the less experienced ones). They are forced to return to a calm bay near Galle to land there and transport the fish (mostly Tuna) via road to Dodanduwa.

  Also it is bad weather conditions like storm, high waves etc., it is too dangerous to sail out. This means a lost day of income for them.

  In the old situation without the breakwater, it was safer. They could see the waves coming and anticipate on them manoeuvring out of the bay.

- **What can, in your opinion, be done to make the harbour safer?**
  Make a higher breakwater and make the top flat. Standing on the breakwater to help an outgoing ship is quite dangerous. The breakwater is not high enough and waves topple over it, resulting in a slippery layer of algae on the armour stones.

  Remove the breakwater so outgoing ships can see the waves coming and anticipate on them. The breakwater now blocks sight on the ocean

- **Do you have any other comments**
  An all-year accessible harbour would be nice
  We received his mail address in case we needed more information or had any other questions. (*No further contact has been made due to imposed stakeholder contact restrictions by Mr. Fernando.*)

Note: More fisherman were approached during the site visit, but did not understand the questions or could not answer them in English. Answers like “*yes yes, dangerous, very dangerous*.” and “*Hello, my name is Charmander*” were received and thus left out of further consideration.

Q.2 STAKEHOLDER MEETING: CEYLON FISHERY HARBOURS CORPORATION

Interview with:	Mr.Sarath Bandara General manager of Ceylon Fishery Harbours Corporation (CFHC)
Date:	12-10-2016
Time:	15:30 – 16:30 h

Introduction project Dodanduwa

Mr. Bandara started with an introduction of the Dodanduwa Fishery Harbour, explaining the current situation with the partly finished breakwater and the problems the fishermen have with the navigation in the harbour bay. Next to the problems of navigation, the river mouth is discussed. The river which connects the Ratgama Lake with the Indian Ocean was first located at the Southern side of the rock outcrop, which got closed by sediment during the dry season. The farmers located next to the lake want the passage to be open all-year long, preventing the water level in the lake to rise, which could lead to flooding.

General Questions

- **What is your role in the organisation?**
  Mr. Bandara is the general manager of CFHC

- **With which other parties involved in the project do you have direct contact?**
  Ministry PI & MU, client is CFHC (end user) they have a direct link with the Ministry of Fishery (they are part of the Ministry). The CFHC will do the maintenance and exploitation of the harbour.

- **Why is there a demand from the fishermen to have a harbour close by their homes?**
  So many harbours are close by, everyone wants a harbour close by home. The people mostly don't have vehicles, they can't travel a long distance to work. A harbour close by is very convenient.

- **Who initiated the project?**
  Fisherman themselves asked to change the harbour themselves by the CFHC, after a series of accidents. New harbour built for safe navigation. Several fatal incidents due to unsafe conditions near the harbour.

- **What is your long term vision on the development of the fishery industry?**
  Long term vision is to design construction and maintenance of the fishery harbours. Small boats will be reduced, fishing close by shore is currently exploited, the amount of fish is decreasing. Fishermen will be encouraged to fish further away from the shore.
  Dodanduwa in 10 years' time from now, vallams will remain. They don't have to go far away, they fish in different waters where multiday doesn't go to. Local fishermen don't want to give up this way of fishing, it is a tradition. Mechanised vallams are most used in Dodanduwa.

Specific Questions

- **How many people work in the Dodanduwa Fishery Harbour?**
  25 people are on the pay list of the CFHC in Dodanduwa. Mr. Bandara will present the actual figures.

- **Do you expect the current onshore facilities to be future proof?**
  The new harbour design should include definitely improvement of the current facilities in the harbour of Dodanduwa. The new harbour design is currently made by EML consultants.

- **Why wasn't the current breakwater finished?**
  The partly built breakwater dating from 2010 was not finished due to financial problems with the contractor. The CFHC ordered the contractor to finish the breakwater this year. The construction is not yet started due to bad weather conditions. Construction will be based on the design made in 2001. This design is in possession of Mr. Bandara, he will send it to us later on.

- **What are the expectations regarding the increase in multiday vessels, after redesigning the harbour?**
  At the moment the multiday vessels anchor in the Hikkaduwa harbour, but during SE monsoon navigation through the entrance channel in Hikkaduwa is hard. In the future the Dodanduwa harbour could be used to anchor these vessels during the SE monsoon. Mr. Bandara mentioned that a new harbour for multiday vessels will be built north of Hikkaduwa. The new Dodanduwa design should include vallams and multiday vessels, since the minimum depth has to be ensured confirm the norm. Design guideline: accommodate 40 boats of 34 feet in 1 ha. In practise this number is 100 vessels per ha.

- **What is the dredging policy for the fishery harbours?**
  Depends on the sedimention rate, for some harbours it is once every year, in other harbours once every five years. For the new harbour the sedimentation is to be kept to a minimum. In short, there is no real policy, the dredging of the harbours will be done when needed.

- **You mentioned the that it is a wish of the farmers to keep the river open all year long, but what about possible salt water related problems?**
  Salt water intrusion is problem for farmers. Cultivation is problem for farmers. No research had been done before, this is not the concern of the fishery industry (and therefore the CFHC?).

- **When will the harbour of Dodanduwa being improved?**
  At the moment the contract for extending the breakwater is given to the contractor, according to the old design. EML consultants, are executing a feasibility study. Financing is probably the most difficult part. Maybe the Asian development bank can provide a loan or grant.

- **What other harbour project are currently in progress?**
  At the moment PI & MU has five projects running, one in the Kudawella harbour (jetties), new jetty construction in Galle and studies in Northern area selected three places for potential areas for a new harbours. There is no operational harbour in the North at the moment, and no money is available. A new harbour is planned near Chilaw, located 50 miles to the north of Colombo. At the moment only a lagoon is present. The entrance of the lagoon struggles with sedimentation problems. Vessels should wait for high tide to enter the lagoon. Lanka Hydraulic Institute has a solution, but Mr. Bandara has its doubts. Design is going on, comments have been made by the CFHC.

- **What will bring the future to the fishery industry in Sri Lanka?**
  Most activities will stay the same. Although, the Ministry has project to extent the catchment area of Sri Lanka to deeper water. In the past the European Union had a ban on Sri Lankan fish, this ban is very recently (two months ago) lifted. Fish can now be exported, which can boost the fishery industry. The number of fishermen might increase due to an increasing demand for fish. “Deep sea” has no fixed definition, 200km around the country is the border of the national waters. You can fish in international waters, not in a territory of another country. There is a need for bigger boats. At the moment Point Pedro is designed for deep fishing. Water depth is 9m in point Pedro, the boats have a length of 60 feet.

R. FIELD WORK

On the 7th of October a second site visit to Dodanduwa was planned. During this visit the results of the desk study would be verified and additional data needed for the problem analysis was gathered. The fieldwork included research for the following topics:

- Geological analysis: Rock(mass) classification of the different rock outcrops
- Built up of breakwater, inclusive stone sizes
- Bathymetry of the river
- Different sand samples

The bathymetry of the bay could only be measured at the location of the bridge, due to safety reasons it was not possible to gather data of the bathymetry inside the harbour.

R.1 ANALYSIS GEOLOGY

During the second fieldwork at the project location in Dodanduwa the geology at the site has been researched. In the preliminary design assumptions regarding the geology have been made, with the fieldwork these assumptions have been tested. In three areas the rock outcrops have been tested, namely at the northern-, middle- and southern rock outcrops. The method of rock testing and classification of these outcrops is described in the first section, after which the outcome of each test area is briefly described.

Rock classification and testing

The rock outcrops in the different areas have been investigated by using the following tools:

- Geological hammer
- Strength recognition and description table (Waltham, 2009)
- Strength properties of Rocks (Waltham, 2009)
- Lecture notes Engineering Geology (AES1630), Metamorphic rocks
- Desk study as performed during the preliminary design

By testing the rock strength with the geological hammer in combination with the knowledge obtained from the lecture notes of Engineering Geology and the desk study, the rock have been classified. Next to the rock type, the foliation, folds, faults and weathering have been investigated.

General features

The three rock outcrops researched have features that correspond to all outcrops. These general features have been summarized in Table R-1.

Table R-1: Summary general features rock outcrops Dodanduwa

General features rock outcrops around Dodanduwa	
Rocks encountered	
- (Charnockite) Gneiss	The rock mass at all the different outcrops mainly consists of charnockite gneiss, based upon the rock recognition, rock testing and the information obtained from the desk study.
- (Mica) Gneiss	Dark coloured intrusions have been found at every outcrop, this dark rock is expected to be mica gneiss, based upon testing with a geologic hammer.
Foliation	The main part of the rock mass consists the metamorphic rock gneiss, in which the foliation of the different deposits is clearly visible.
Folds	No general trend in folding has been identified.
Faults	Some normal faults have been observed in the different rock outcrops, splitting the different rock masses into several parts. The strike direction and dip of the faults differ for each fault, no general orientation and direction have been found.
Weathering	At some locations the rock mass is weathered severely, at other locations no weathering has been observed. The weathered parts of the rock is only found at the first few millimetre of the rock, after removal of the thin weathered layer fresh rock was found. Believed is that the origin of the weathering is due to wetting and drying cycles of the top layers of the rock, since the weathering is mostly found at rocks located close to the sea water.

Now that the general features of the different outcrops have been identified, the specific features of the different rock outcrops will be mentioned separately.

Northern outcrop

Researching of the geology of northern outcrop results in location specific features. The features of the northern outcrop which differ from the general features are summarized in Table R-2.

Table R-2: Summary of features specific for the northern outcrop

Summary northern outcrop fieldwork	
Rocks encountered	
-	-
Foliation	-
Folds	The presence of folds in the northern outcrop has not been confirmed during the fieldwork.
Faults	-
Weathering	At some locations of the rock outcrop brown/red coloured weathering has been observed. The classification of the weathered rock is grade II, slightly weathered rock with increased fractures and mineral staining.

Southern outcrop

The second research of the fieldwork was conducted at the rock outcrop located at the south of the current harbour area. During the fieldwork the rock outcrop has been analysed and tested. The results of the fieldwork are summarized in Table R-3.

Table R-3: Summary of features specific for the southern outcrop

Summary southern outcrop fieldwork	
Rocks encountered	
-	-
Foliation	-
Folds	Some overturned folds have been identified at different places in the rock mass.
Faults	-
Weathering	The degree of weathering at the southern outcrop is severe. After researching the weathering profile it seemed that the weathering was only occurring at the surface of the rocks. After removal of the weathered rock layer of approximately 5mm, fresh rock emerged. The brown/red colour of the weathered rock indicates the presence of chemical weathering by oxidation. The classification of the weathered rock is grade II, slightly weathered rock with increased fractures and mineral staining.

Western outcrop

The third research of the fieldwork was conducted at the rock outcrop located at the west of the current harbour area. During the fieldwork the rock outcrop has been analysed and tested. The results of the fieldwork are summarized in Table R-4.

Table R-4: Summary of features specific for the western outcrop

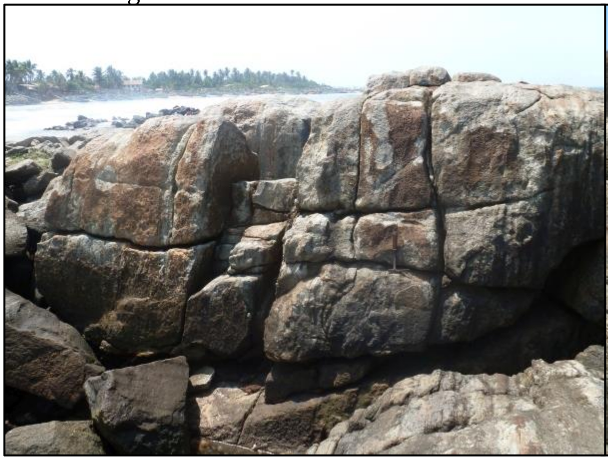
Summary western outcrop fieldwork	
Rocks encountered	
Diorite	Some black coloured, probably igneous, rocks have been found at the intrusions. Based upon strength testing with the geological hammer this rock is believed to be diorite, containing hornblende.
Foliation	-
Folds	-
Faults	The amount of faults in the western outcrop is large, multiple faults were observed. The direction of the faults was varying, some faults intersected one another.
Weathering	The rock mass is only slightly weathered, some minor colour changes at the weathered rock have been observed.

Pictures northern outcrop

Rock strength testing


Intrusion and layering of rocks


Weathering of rocks and faults

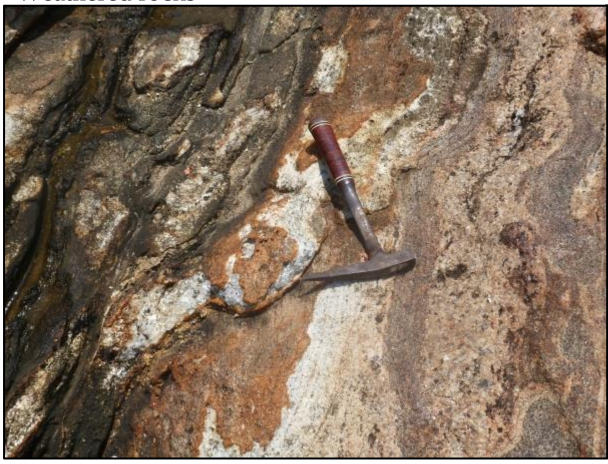



Pictures southern outcrop

Intrusion, foliation and layered metamorphic rock


Faults and weathering rocks

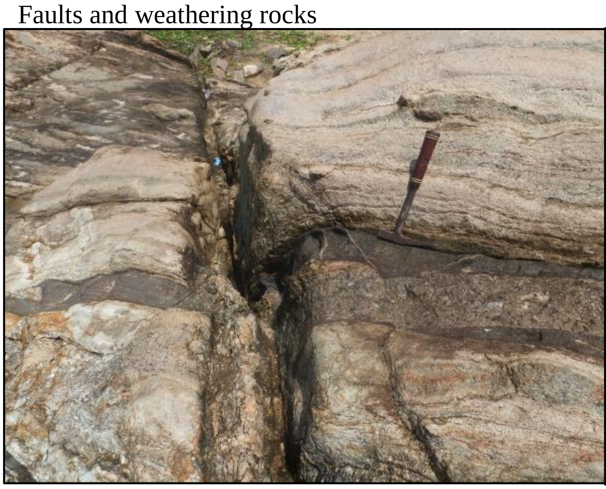

Weathered rocks




Pictures western outcrop

Intrusions in the rock mass


Faults and weathering rocks




Intrusives in rock mass


R.2 BREAKWATER BUILT UP

The second part of the fieldwork was focussed on the current breakwater and how it has been built. Different aspects of the breakwater have been studied, including the sizes of the armour stones and checking the different layers of the breakwater.

Due to safety reason only the beginning of the breakwater could be investigated. By means of visual inspected no significant layer built up could be observed. Both on top of and in between the armour stones smaller rocks have been perceived, as can be seen in Figure R-1. It remains unclear whether the breakwater was designed with this built up or a storm caused this configuration of stones.

Figure R-1: Small stones in between big stones

The armour stones of the breakwater varied around $1.10 - 1.30\ m$, while bigger stones were observed from $1.60\ to\ 2.10\ m$, as can be seen in Figure R-2, Figure R-3 and Figure R-4.

Figure R-2: Armour stone on the breakwater

Figure R-3: Armour stone on the breakwater

Figure R-4: Armour stone on the breakwater

Northern rock outcrop

The armour stones positioned at the northern rock outcrop have been researched. The stones investigated had the same size and composition as the armour stones located at the breakwater. The function of these placed rocks is unclear, expected is that the thought was that the rocks are strengthening the northern outcrop. In Figure R-5 the armour stones on the northern outcrop are presented, in Figure R-6 the armour units which are extending in the harbour bay is given.

Figure R-5: Armour stones on Northern rock outcrop

Figure R-6: Armour units extending in the sea

The size of the armour units on the northern outcrop are about the same as the units researched on the breakwater. No particular built up of the strengthening was observed.

Figure R-7: Armour size Northern outcrop

Figure R-8: Armour size Northern outcrop

Figure R-9: Armour stones Northern outcrop

R.3 BATHYMETRY OF THE RIVER

The new design of the harbour is required to have a guaranteed minimum depth of 3.0 meter. In order to design the dredging policy information regarding the bathymetry is needed. Gathering data in the harbour basin and the river is rather perilous, therefore the depth of the river is only measured at the motorway bridge crossing the river upstream of the harbour basin. The bathymetry of the river is measured on both sides of the bridge.

The measurements have been performed by using a PVC tube with a length of 4.0 meter. Every 0.25 meters a piece of tape has been attached to the tube, in order to measure the depth of the river. The tube with the markings is presented in Figure R-10. The depth of the river is measured in whole and half intervals between the tape markings. This method of measuring the depth includes a large error, but the method used is not exact to start with since the river depth is varying over the year. The rod itself has a little curvature due to the transportation of the rod. However, the measurements give a rough insight in the river depth. The depth of the river has been measured every 0.8 meter.

Figure R-10: Making of the measuring tube

Figure R-11: Measuring the bathymetry

The results of the measurements of the river are presented in Figure R-12.

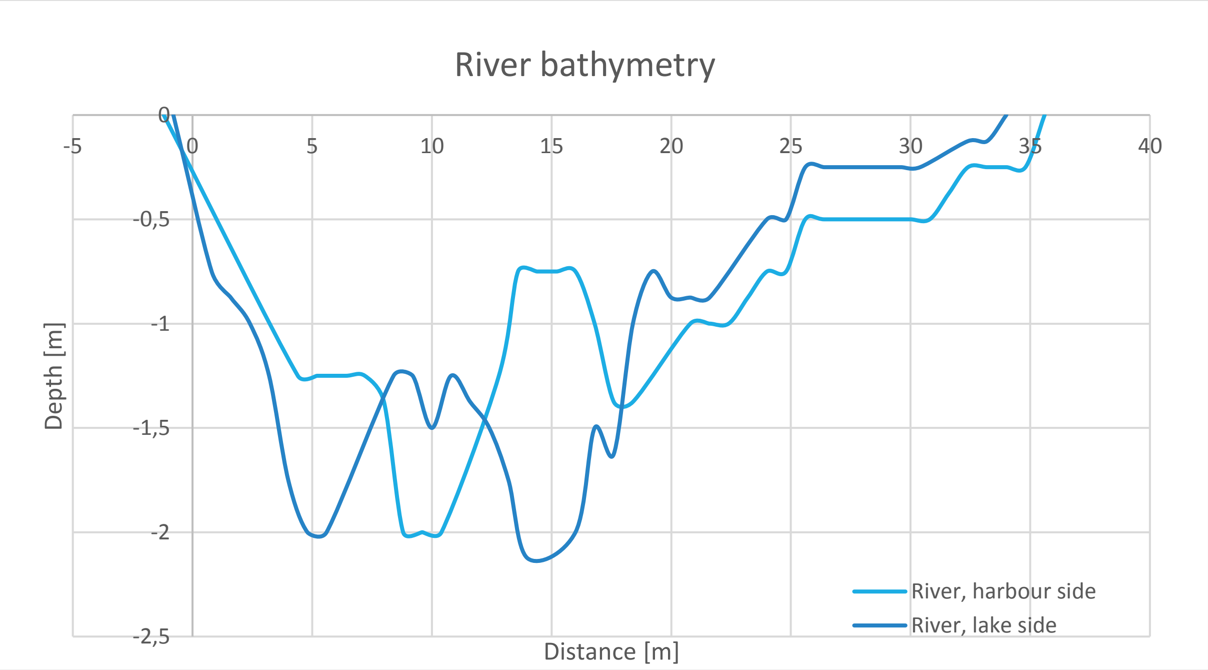


Figure R-12: Bathymetry of the river on both sides of the river

When looking at the bathymetry of the river it can be observed that the left river bank is steeper than the right river bank. At the left river bank, structural elements hindered measurements. Between 5 and 17 meter the deeper part of the river is located. After the deeper part the depth of the river is rapidly decreasing. When comparing the depths of both sides of the river it can be noticed that the deep parts on either side are opposite to each other, the reason for that is unknown.

The last 11.0 meter the river depth is rather unnatural and very shallow. This shallow area is located at the northern border of the river, as indicated with the yellow area in Figure R-13. The red area shows the natural course of the river, while the yellow part of the river seems not natural.

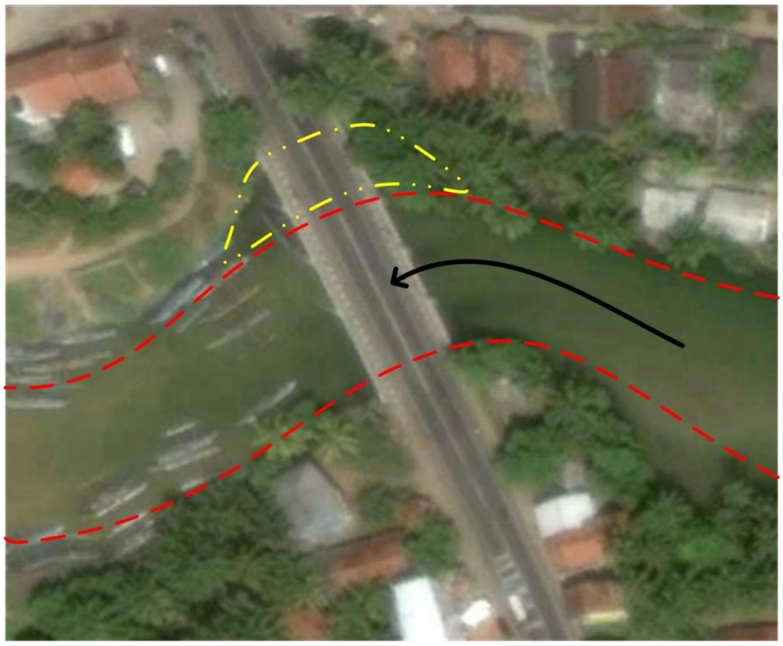


Figure R-13: Natural course of the river

R.4 SEDIMENT

During the fieldwork in Dodanduwa three sediment samples were collected. These three samples were taken at different locations on the beach and in the lagoon. This was done to check if the grain size assumption were correct.

The correct way to measure grain sizes is to sieve the sediment samples. This was unfortunately not possible at location. To get an idea of grain size a simple and available tool was used: a tapeline. The sand was dried before it was measured. It must be said that all measurements made with the tapeline are rough estimates.

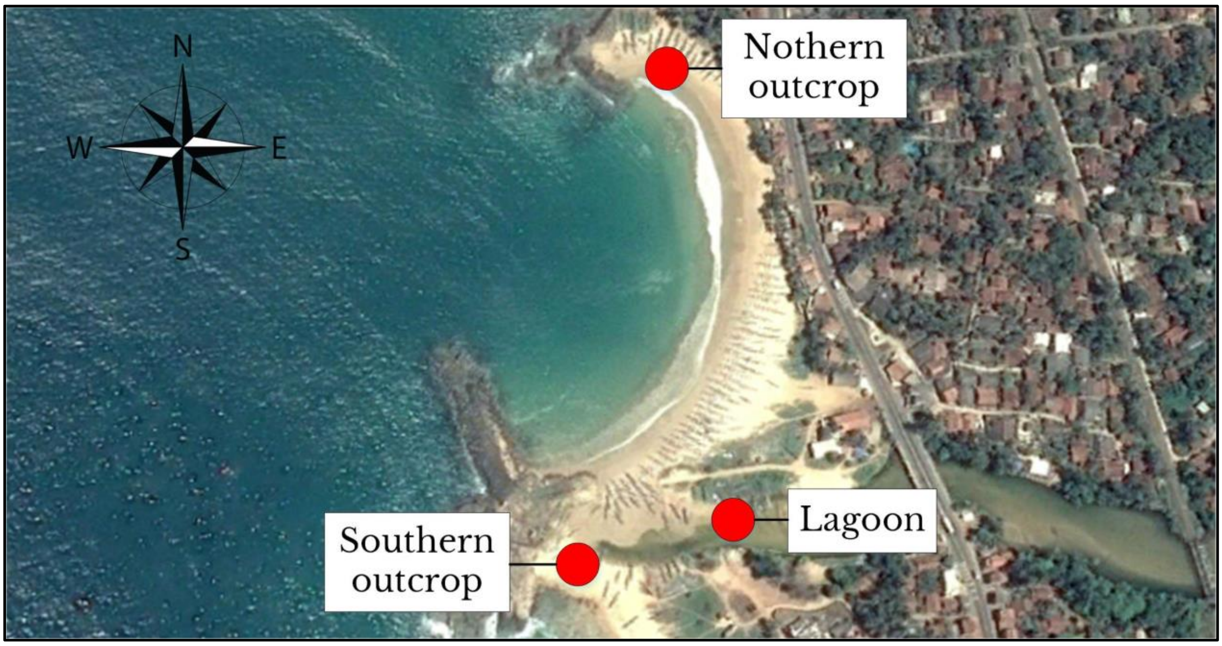


Figure R-14: Locations sediment samples: northern outcrop, behind southern breakwater and lagoon

Grain sizes

Millimeters (mm)		Micrometers (µm)	Phi (φ)	Wentworth size class		Rock type
	2.00		-1.0			
				Very coarse sand	Sand	Sandstone
	1.00		0.0			
				Coarse sand		
1/2	0.50	500	1.0			
				Medium sand		
1/4	0.25	250	2.0			
				Fine sand		
1/8	0.125	125	3.0			
				Very fine sand		
1/16	0.0625	63	4.0			

Udden-Wentworth grain-size classification. (source: Wentworth, 1922)

Figure R-15: Grain size sand. (Original table size adjusted)

There are five classifications for sand grain size. From very fine sand to very coarse sand. For the calculations during the design stage it was assumed that the sand in the Dodanduwa harbour is coarse. The sediment grain size found in Dodanduwa will be discussed in the next chapters.

Northern outcrop

The first sediment sample is taken on the beach behind the northern outcrop. In Figure R-16 the sediment samples of the northern outcrop are shown.

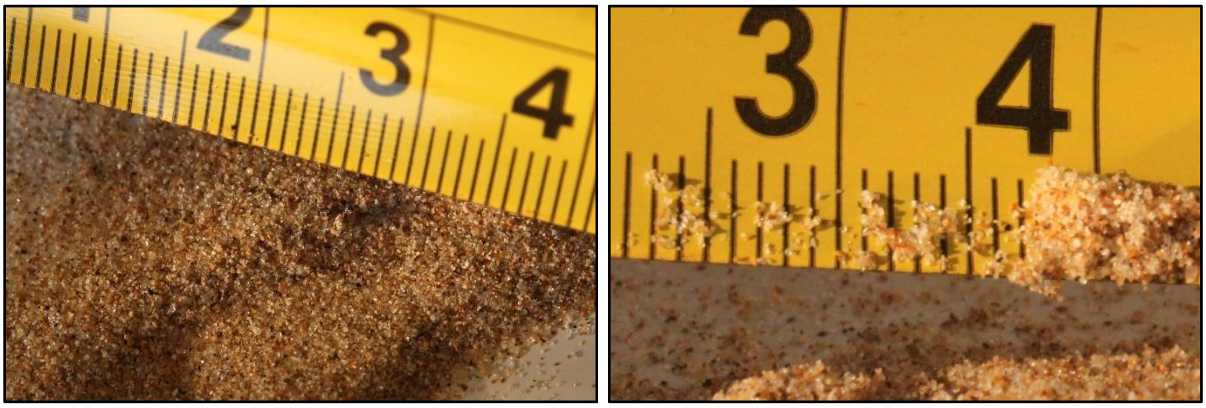


Figure R-16: Sediment sample northern outcrop

As can be seen in the second picture, the sediment found here is not very coarse sediment. The average sediment size is around a quarter to half a millimetre. This would be classified as fine to medium sand.

Southern breakwater

The second sediment sample was taken behind the southern outcrop. This sand is coarser than the sediment found at the northern outcrop. Measurements give an average grain size of half a millimetre to one millimetre. This is classified as coarse sand. Still in these samples a lot of fine material was be found.

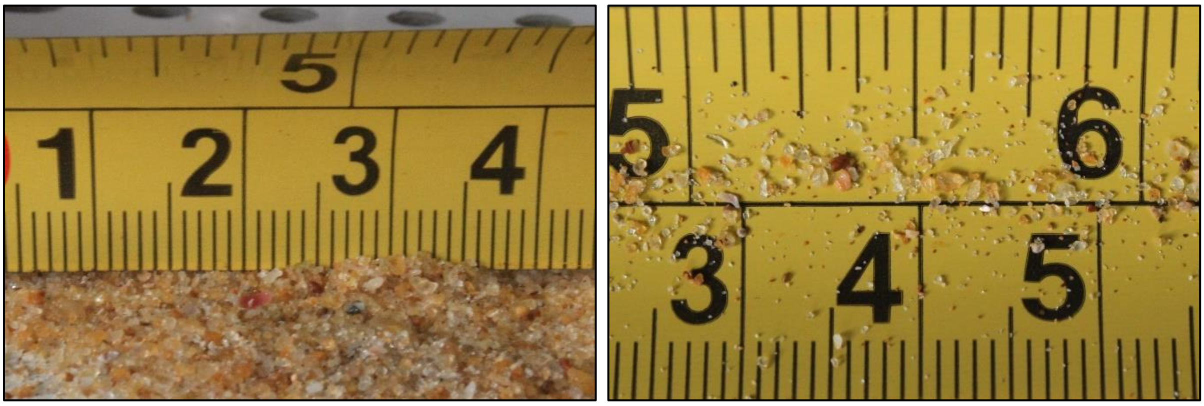


Figure R-17: Sediment sample southern breakwater

Lagoon

The third sample was taken from the mouth of the lagoon. During the time this sample was taken the lagoon mouth was closed off. Therefore, it was possible to take a sample. The sediment found here is finer than the sediment found at the other places. Measurements give an average grain size of one eight of a millimetre. This sediment is classified as very fine to fine sand.

Figure R-18: Sediment sample lagoon mouth

Conclusion

From the fieldwork it can be concluded that there is a large diversity in sediment grain size around the harbour of Dodanduwa. Therefore it is recommended to research more locations for sediment grain size and to sieve the samples for analysis.

The assumption that there is coarse sand all around the harbour area of Dodanduwa is not accurate. This assumption is only valid behind the southern outcrop. However, the samples were taken at the beach. The sea bed or the river bank can consist of finer or coarser material. Therefore, it cannot be said that the assumption of coarse sand is completely invalid.

R.5 PRESENT HARBOUR PHOTOS

Taken from the Northern outcrop

In Figure R-19 can be seen that all the Vallams land on the beach. Still rough waves penetrate the bay. On the right the current breakwater is visible. A lot of armour stone is displaced. Figure R-20 shows a close up of the breakwater.

Figure R-19: Panorama photo bay. On the left Vallams, on the right the current breakwater

Figure R-20: The current breakwater

Photos taken from the current breakwater and outcrop

In Figure R-21 it can be seen that the shoal in front on the lagoon mouth is large. A lot of sediment has been accreted behind the outcrop and breakwater. Most of the boats are not landed on the beach in front of the harbour facilities. On the right side of the picture the southern bay can seen.

Figure R-21: Panorama photo overview harbour bay

Figure R-22: Measuring stone size current breakwater

Figure R-23: Breakwater, displacement of armour stone

Figure R-24: Southern outcrop, on the left some vallams on the beach

Photos taken of the lagoon

Figure R-25: Lagoon mouth, picture taken from road bridge

Figure R-26: Lagoon mouth; picture taken from shoal

Figure R-27: Panorama picture a small part of the lagoon

DODANDUWA

TU Delft

www.project-dodanduwa.com
projectdodanduwa2016@gmail.com