MAIN REPORT

February 24, 2016

PROJECT yogya

Recommendations for the continued pursuit of a safe, durable and sustainable Tanjung Adikarto Port

Main Partners

DAMEN

DEME
Dredging, Environmental
& Marine Engineering

Van Oord
Marine ingenuity

Partner

Boskalis

Commissioner

BBWS
Serayu Opak

Academic Partners

TU Delft

Universitas
Gadjah Mada

DISCLAIMER

Seven disclaimers should be stated in order to guarantee the correct interpretation of the work presented in this report.

- Project Yogya was supervised by the TU Delft, powered by Boskalis, Damen, Deme and Van Oord, and commissioned by Balai P.W., however this report does not necessarily represent the views of these organizations nor its employees.
- The economic feasibility of the presented design, among other feasibility aspects, have not been assessed in this research. It is strongly recommended to assess the economic feasibility of the design as it is a key factor in the policy making. The current design draws on the assumption that a significant economic growth scenario develops upon completion of the harbour.
- It should be noted that the policy analysis presented are of less academic and neutral character than the technical analysis sections. Therefore Project Yogya stresses that this research was conducted with the upmost objectivity and that the views presented here are not necessarily representative of Balai P.W.'s views.
- Several important design criteria have been used based on the program of requirement from the Glagah Jetty Project. These design criteria, such as the vessel size and type and frequency of passage, have not been verified since this would involve economic considerations.
- The design presented in this report is the result of one iteration in the design cycle. Despite the fact that it can be implemented, it is expected to be suboptimal and more design optimisation can yield a more economical, more practical or otherwise improved design. Technical, political and economic optimization of the design is to be done in future iterations. In addition, it should be stressed that the design should be verified by means of wave intrusion and sediment transport models.
- The design approach is only illustrative for the integral engineering approach preferred by the TU Delft and could also be followed should a different economic scenario develop. In this case this may lead to a different design.
- All information gathered in this report has been reviewed by the investigators and although not specifically mentioned, supervisors and colleagues from Balai P.W. as well as TU Delft have provided additional information to verify statements and to provide background information to better comprehend situations and issues.

EXECUTIVE SUMMARY

INTRODUCTION

Tanjung Adikarto is a fishing port in Kulon Progo, a coastal region in the Special Province of Yogyakarta, Indonesia. Project Yogya was commissioned by Balai Besar Wilayah Sungai Serayu Opak with the objective to assess the issues that impede completion of the Tanjung Adikarto Port (Part I) and devise an adequate and comprehensive counter strategy (Part II). In 2012, the construction of two protective breakwaters known as the *Glagah Jetty Project* was halted for reasons disputed by the stakeholders involved and the economic impulse prospected to outweigh large investment costs was questioned. This two-legged report presents an analysis of the progress-impeding issues (Part I) and follows up with a recommended design proposal accompanied by an effective implementation policy (Part II).

DISCLAIMER

Key aspects in the consideration of pursuing further breakwater construction are economic, technical and implementation feasibility of the review design, which are heavily intertwined. Project Yogya is a review design and builds on the assumptions underlying the Glagah Jetty Project, most prominently the assumption that an economic growth scenario will develop upon completion of the harbour. Hence, aside from the inclusion of economical design considerations, no quantitative economic feasibility study was conducted in this report. It is strongly recommended to evaluate financially whether the economic benefits justify the investment costs in each iteration of the design cycle.

CURRENT ISSUES (PART I)

The issues that currently impede continuation of the Glagah Jetty Project were identified based on a literature review, stakeholder meetings and feedback sessions.

BREAKWATERS	▪	The existing breakwaters do not fulfil the objective of creating a safe harbour entrance;
	▪	The structural integrity of the breakwater is compromised which will result in the breakwater not meeting its design lifetime;
COASTLINE	▪	Erosion of the coast adjacent to the Eastern breakwater will soon threaten the functionality of the coastline and the nearby recreational facilities;
POLICY	▪	High-impact stakeholders have not reached consensus on the responsibility of breakwater lengthening and future maintenance;
	▪	There is an institution-wide lack of data and statistics gathering and sharing;
	▪	Maintenance dredging in the entrance channel is not safe nor durable and sustainable;
	▪	Severe issues have arisen from improper execution.

OBJECTIVE SPECIFICATION (PART II)

Cooperation with the commissioner, an in-depth stakeholder analysis and consultation with TU Delft engineers on the adaptation of standards allowed for further specification of the objective for Part II:

Design a safe harbour entrance including durable breakwaters, a sustainable coastline conserving the lagoon area and ensure its feasibility with an effective implementation governance policy.

RECOMMENDATIONS FOR THE BREAKWATERS

The reviewed breakwater design differs from the previous design on four main aspects. The comparison of the current situation and final design is given in Table 0-1.

TABLE 0-1 MAIN CHARACTERISTICS BREAKWATER DESIGN

	Eastern Breakwater		*Western Breakwater*	
	Current situation	**Final design**	**Current situation**	**Final design**
Length [m]	180	370*	215	272*
Height renovation [m]	6.0 + 2 m seawall	6.0 + 2 m seawall	6.0	6.0 + 2.5 m seawall
Height extension [m]	-	8.5	-	8.5
Armour layer outer [t]	9.0	18.0	9.0	18.0
Armour layer inner [t]	7.0	7.0	7.0	7.0
Armour layer head [t]	-	18.0	-	18.0

*Indication of length

- Breakwaters are elongated such that they cross the breaker zone.
- Breakwater heights are increased and the breakwater is equipped with stronger layers.
- Since the design is the result of one design cycle iteration, a framework is proposed to bridge construction limits and economically optimize the design.
- For implementation of the design a modulated building method was proposed.

RECOMMENDATIONS FOR THE COASTLINE

Considerations have been outlined for sustainable coastline preservation. For best preservation of the coastline, it is recommended to apply shoreface nourishments in conjunction with the planting of vetiver grass and potentially the construction of groynes.

POLICY RECOMMENDATIONS

Integral policy recommendations are given which enable the establishment of a design implementation policy that incorporates public responsibility, safety, durability and sustainability.

- Transfer the responsibility for construction in a Memorandum Of Understanding (MOU) from the Ministry of Maritime Affairs and Fisheries to the Ministry of Public Works.
- Decompose maintenance responsibilities on a detailed level using a Memorandum Of Detail (MOD).
- Establish a research consortium that serves as a knowledge sharing platform.
- Solve issues with Key Performance Indicators (KPIs) by stimulating the establishment of a Memorandum of Detail about dredging disposal locations.
- Acquire or lease an upgraded dredging vessel.
- For the executing party, develop more in-house knowledge and multi-year programs.

WINDOW OF OPPORTUNITY

A window of opportunity for successful completion of the Glagah Jetty Project is observed.

- Stakeholder motivation levels are unprecedented.
- The staff at Min. M.A.F. General Directory responsible for declining the Memorandum of Understanding (MOU) has been replaced.
    - Additionally, the Minister of Public Works considers a new (MOU), a realistic scenario under current conditions.
- The National Planning Agency has made available a budget for Glagah Jetty specifically.
    - This budget amounts to Rp223 Bln (€15 Mln).

FURTHER RESEARCH

It is strongly recommended to conduct further research on:

- The economic feasibility of the proposed design.
- More iterations of the design cycle in order to be able to economically optimize the design.

PROJECT TEAM

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PREFACE

Standing 6 meters above sea level on the east breakwater head, a flume of water loomed up above the 14-ton tetrapods and the rumbling sound of a hammering wave resonated profoundly with our new colleagues and us. Our first visit to Glagah beach left us speechless. According to Javanese myths, we were at the border of the domain of *Nyai Roro Kidul*, the Queen of the Southern Sea. Here within she would drag fishermen into the deep sea if they did not show her respect. Although we are not superstitious, we felt its power nonetheless.

Over the years, the Glagah Jetty Project -the design of two breakwaters at a steep coastal zone with a rough wave climate- caught the attention of many as it requires engineers to push the frontiers of science and construction workers to test the application of relevant insights. Considering our water-related backgrounds, we too had been asked to shed light on the matter. We have personally experienced the variety of dilemmas with this project, especially as we have had to adapt the standards fundamental to our knowledge. It was a challenge to make up for the long hours of intriguing discussions while steadily progressing with our findings given the two-month time period.

With this report, we aim to contribute to the realisation of a longstanding wish to stimulate an economic impulse for the Yogyakarta region. To this end, we first describe our observations about the initiation of this project from an outsiders perspective (Part I). This text is key for anyone to understand the essence of the project and lays the groundwork for further advancements. Second, a redesign of the breakwaters is presented based on guidelines set out by the TU Delft (Part II). Understanding the hydraulic-engineering models used in this part does require some knowledge of physics, mathematics and wave descriptions in particular.

The hostility of the Indian Ocean is met with the hospitability of the South Javanese people. The challenge we were entrusted with would have been impossible if not for the South Javanese deep-seated willingness to help. We felt particularly assisted in our challenge to organize a symposium that would bring together leaders from top governing bodies, professors and engineers that could prove to be of key importance in further development of the project. When untangling the complicated Indonesian regulatory environment and understanding our share of Kulunowun-like phenomena, the heartfelt support of our colleagues at BBWS Serayu Opak has been of paramount importance.

We were most delighted to see so many stakeholders at the first Glagah Jetty and Tanjung Adikarto symposium, driving momentum for near-term solutions to a new high. Among the participants was Dr. Arie Moerwanto, who overwhelmed us with a personal invitation from Minister of Public Works Basoeki Hadimoeljono to present the findings of Project Yogya to him in Jakarta. We were honoured with the occasion, and looking back at our time in Indonesia, we can say that these experiences absolutely outshined our university's previous projects and classes in terms of personal development.

We hope to have brought new life into discussions about the construction of the harbour entrance. Moreover, we hope that the provisional design presented in this report commits the stakeholders to a new realisation phase for a harbour. A harbour that, respecting the wishes of *Nyai Roro Kidul*, gives prosperity to the local people, provides access to safe fishery in the Indian Ocean, induces a trade platform for Yogyakarta and that preserves the precious beauty of the Javanese coast.

The Project Yogya team,
Rogier Burger, Jorrit Horst, Maarten Lanters, Laurens Leunge and Jeroen Werkhoven

ACKNOWLEDGMENTS

We are honoured with the support of Minister Basoeki Hadimoeljono and would like to thank him for showing his genuine interest in our project. It was an amazing experience to have been able to share our findings with the minister himself. We owe our deepest gratitude to Director General Arie Moerwanti, who has played a key role in bringing our recommendations to the national level.

The supporting environment of BBWS Serayu Opak has been of major importance; in particular we would like to thank our daily supervisor Hanungerah Purwadi, MT, our colleagues and friends Mas Shakti and Bu Kitty for their continuous and all-round assistance in every part of our work. On the academic level, we have been assisted greatly by Prof. Ir. Nur Yuwono Ph.D., Dip.HE. and we are very grateful for him sharing his profound knowledge of oceanic systems and breakwaters. In the making of this report we were supervised by Dr. Ir. S. De Vries and Ir. J. Bosboom for whom we have developed a sincere admiration.

We would like to express our gratitude to Vicky Ariyanti for providing us with a background of the case and for giving her insights in the matter. We are indebted to Anandro Armalando and Neil Andika for helping us in the preperations of this research. We would like to thank DEME, Damen, Van Oord and Boskalis for enabling us to realize this report.

We are thankful for having been warmly welcomed by the residents of Kampung Ambarukmo Satu, with whom we shared food, celebrated birthdays and national holidays and who introduced us to their beloved futsal.

GLOSSARY

TRANSLATIONS AND INTERPRETATIONS

For clarity purposes, the following translations from Bahasa Indonesia to English are provided.

Indonesian	English
(Glagah) Jetty	(Glagah)* Jetty (Worldwide) / (Glagah)* Breakwater (NL, Indonesia and USA)
Pemecah Gelombang	Breakwater
Pemecah	Breaker
Gelombang	Wave
Pelabuhan Perikanan (Tanjung Adikarta)	(Tanjung Adikarta) Port
Pelabuhan	Port
Perikanan	Fishing
Saluran masuk	Navigation Channel
Saluran	Channel
Masuk	Entry
Pengembangan daerah pantai selatan	South beach area development
Pengembangan	Development
Daerah	Area
Pantai	Beach
Selatan	South
Muara Sungai (Serang)	(Serang) River Estuary
Muara	Estuary
Sungai	River
Serang	Serang
Tanggul bronjong	Gabion Dikes
Tanggul	Dyke
Bronjong	Gabion
Kabupaten (Kulon Progo)	(Kulon Progo) Regency
Perbaikan	Repair
Desain	Design
Pengairan	Irrigation
Banjir	Flood
East	Timur
West	Barat
Tahap	Phase

* Glagah = Indonesian name for reed. The name of the beach around the breakwater is *Glagah*.

It is decided that the correct definitions used in this report should be as indicated in the formulation below:

Glagah Jetty Project is the name of the ongoing project of the *Serayu Opak Watershed Bureau* aimed towards constructing two sizable breakwaters that reduce the maintenance costs of the navigation channel which enables the development of the *Tanjung Adikarto Port*.

DEFINITIONS

An (adapted) definition of specific engineering terms is given.

Term	Definition
Abrasion	Process in which stone surfaces are damaged by being scrubbed by suspended sediment.
Absolute sea level rise	Sea level rise with respect to a certain reference level.
Accretion	Process in which sediment is added to the coast.
Bathymetry	The underwater topography with respect to the water level.
Breaker zone	Zone near the shore where the waves break due to decreasing depth.
Breakwater	Human-made structures for protection of a harbour or beach. In American English also referred to as *jetty*.
Bypass system	A pipe that is used to transport suspended sediment from one side of a structure to the other.
Caisson	Water-retaining structure.
Catchment area	The total area of rainfall which influences a certain river, also referred to as *river basin*.
Collision coast	Name of a coast on a leading edge where two plates collide. These coasts are mountainous. Also referred to as *leading edge coast*.
Continental plate	Tectonic plate that lies under land masses.
Discharge	The volume of water which flows through a river in a certain timeframe.
Ecosystem	Community of living organisms in conjunction with the non-living components.
El Niño	Natural phenomena which influences the air temperature around the equator. This difference in air temperature causes a rise of water level.
Ephemeral stream	A stream that flows for a certain period of time. This may be caused by rainfall of snowmelt.
Equilibrium	A state of rest or balance due to the equal action of opposing forces.
Erosion	The process in which sediment is removed from the coast.
Exponential method	Statistical method used for predictions by computing a trend line and ignoring irrelevant fluctuations.
Extrapolation	Estimating beyond the range of available data.
Feasibility	The quality of being possible and likely to be achieved. *(Oxford University Press, 2015)* Feasibility, being a broad concept, can be divided in a variety of aspects to be considered. Economic feasibility is one of these aspects and is commonly referred to as *(commercial) viability*. In this report, the term *economic feasibility* is used instead.
Floodplain	The stretches of land adjacent to a river which floods during high water.
Groyne	Structure built perpendicular to the coast or riverbank to control erosion.
Interlocking	Strengthening of breakwater protection units as a result of the integration of individual shapes.

Iteration	A step in a recurring process in which the results of an iteration are the starting point for the following iteration.
Jetty	See *breakwater*.
Land subsidence	The lowering of the land surface.
Leading edge coast	See *collision coast*.
Littoral zone	The physical zone between the extremes of high and low water.
Monsoon	Seasonal heavy rains carried by wind from the Indian Ocean.
Monte Carlo simulation	A problem solving technique used to approximate the probability of certain outcomes by running multiple trail runs.
Navigability	Indicator of capability to sail through a certain body of water with a vessel. Depends onf the width and depth of the body of water.
Navigation channel	A relatively deep channel which enables vessels to manoeuvre in.
Normal distribution	Bell-shaped symmetrical frequency distribution curve.
Nourishment	The process of dumping or pumping sand from a place to an eroding shoreline.
Overtopping	The passing of water over a structure.
Perturbation	A small change in a physical system.
Pluvial system	The system created in an period of abundant rainfall.
Quarry run	Stones and sand coming from a quarry.
Rainbowing	The process in which a dredging ship propels dredged sediment in a high arc to a particular location.
Regressive coast	A coast of which the shoreline is shifting seaward.
Relative sea level rise	Sea level rise with respect to the coast.
Revetment	Sloping shore structure built to protect an embankment or shore structure against erosion.
Shoreface nourishment	The supply of sand to the outer part of the coastal profile in order to strengthen the coastal profile.
Standard deviation	A measure used to quantify the amount of variation.
SwanOne	Computer tool which enables the transformation of offshore waves to onshore waves.
Swell waves	Waves generated by storms or winds far from the observed location.
Tetrapod	Four-legged concrete structure units that prevent coastal erosion or wave impact.
Transgressive coast	A coast of which the shoreline is shifting landward.
Wave spectrum	A mathematical representation of the distribution of wave energy.
Weibull distribution	A continuous probability distribution function. Here used to measure probabilities with respect to lifetimes.
Westerlies	Prevailing winds from the west towards the east generated by high pressure areas in the middle latitudes (30 to 60 degrees).

ABBREVIATIONS AND ACRONYMS

The following abbreviations have been used in this report.

Abbreviation / Acronym	Definition
Nat. Bappeda	National Bappeda (National Planning agency)
Min. Finance	Ministry of Finance
Min. P.W.	Ministry of Public Works and Public Housing
Balai P.W.	Balai Public Works (Serayu Opak Watershed Bureau)
Prov. P.W.	Provincial Public Works Department
Min. M.A.F.	Ministry of Maritime Affairs and Fisheries
Prov. M.A.F.	Provincial Department of Maritime Affairs and Fisheries
Prov. Bappeda	Provincial Bappeda (Provincial Planning Agency)
Prov. Irrig.	Provincial Irrigation Department
Reg. W.R.	Regency Department of Water Resources
BPPT	Balai Pengkajian Dinamika Pantai (Agency for the Assessment and Application of Technology)
UGM	University of Gadjah Mada
UPTD	Unit Pelaksana Tekmis Dinas (Public Harbour Master)
KPI	Key Performance Indicators
TU Delft	Delft University of Technology

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ABOUT PROJECT YOGYA

INTRODUCTION TO PROJECT YOGYA

Sri Paku Alam VIII, from 1988 to 1998 the ruling Sultan of the Special Province of Yogyakarta, Indonesia, envisioned the creation of a large-scale fishing harbour for the growing population of Yogyakarta. Based on infrastructure assessments, the harbour was chosen to be located at the Serang river mouth, at the time characterised by no more activity than that of local fishermen providing food supplies for their families.

At the time, the river mouth was subject to continuous change and shifting through natural intervention. Moreover, the intense wave climate and strongly seasonal precipitation character caused the mouth to be regularly blocked by sand from the Indian Ocean. In the rainy season, floods could occur around the river mouth when the peak discharges could not be accounted for.

In order to solve the identified problems and mitigate flood risks, the Indonesian Government, in consultation with the sultan, ordered the execution of a series of projects. First, embankments were built to fixate the Serang River. Second, development started for a harbour named Tanjung Adikarto Port. Last, the construction of two sizable breakwaters commenced in order to provide a safe harbour entrance, which is known as the *Glagah Jetty Project*.

However, issues arose during construction of the breakwater, particularly regarding the stability of the two breakwaters. Additionally, the construction of the breakwaters led to heavy erosion at the adjacent coast. The construction was put to a stop and the harbour, which was anticipated to be in operation already, remained unopened. To date, the construction is still not resumed for reasons disputed by the involved stakeholders and the economic impulse prospected to outweigh large investment costs is being questioned.

OBJECTIVE AND SCOPE OF PROJECT YOGYA

When participants of a study trip from Delft University of Technology visited the site in 2014, it was decided that the TU Delft would put forth a team of students entrusted with the task of providing insight in the progress-impeding issues from an engineering perspective. In 2015, this led to the initiation of Project Yogya. The project's location is Glagah Beach, in the regency of Kulon Progo, Yogyakarta, Indonesia, and is shown in FIGURE 0-1.

FIGURE 0-1 PROJECT LOCATION: GLAGAH BEACH, KULON PROGO, INDONESIA

OBJECTIVE

Project Yogya covered a period of 10 weeks. In consultation with commissioner Balai Besar Wilayah Sungai Serayu Opak (Hereinafter Balai P.W.) the general objective of Project Yogya was established and was formulated as:

Assess the issues that impede completion of the Tanjung Adikarto Port and devise an adequate and comprehensive counter strategy.

SCOPE

Given the limited time and resources available for the project, the investigators have initially had to carefully assess the research scope follows from the established objective.

In order to provide insight, the feasibility of a design according to TU Delft standards was to be verified. It was chosen to limit the scope of the integral design to engineering aspects and implementation governance and does not include any (quantitative) information relating to the economic considerations underlying the proposition that the breakwater construction should be resumed. Project Yogya is a review design and builds on the assumptions underlying the Glagah Jetty Project, most prominently the assumption that an economic growth scenario will develop upon completion of the harbour.

Generally, in the first stages of a design one has most influence on functionality, environment and costs, while influence on construction and technology is insignificant. As one is advancing to a detailed design and thereby determining the construction methods, materials and technologies to be applied, fewer changes can be made that influence functionality, environment and costs.

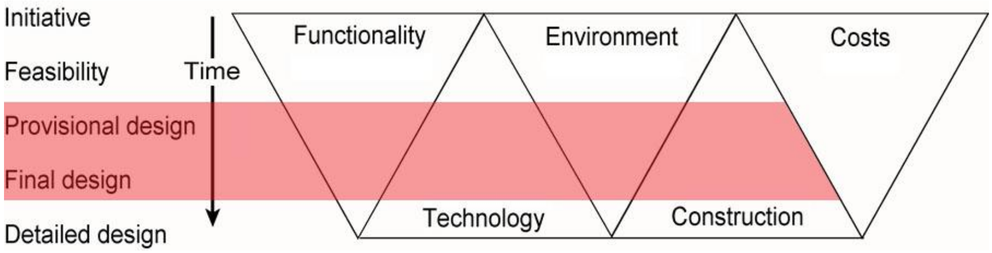

FIGURE 0-2 INFLUENCES TROUGHOUT THE DIFFERENT PHASES OF A DESIGN PROJECT (SCHIERECK & VERHAGEN, 2012)

The initiative for the construction of a harbour came from sultan Sri Paku Alam VIII. A feasibility study (Prov. M.A.F., 2012) was performed by the provincial department of the Ministry of Maritime Affairs and Fishery (Hereinafter Prov. M.A.F.). The choice of the location and the functional requirements for the harbour have neither been questioned nor re-evaluated. Functional requirements have been taken from the program of requirements for previous designs, most importantly the type and size of the vessels that enter the harbour and the frequency of harbour passage. These requirements serve as boundary conditions for the review design made by Project Yogya.

Project Yogya covered the stages of provisional design and final design. For time considerations, only one iteration of the design cycle was executed within these stages. The design process of Project Yogya is considered illustrative for the integral engineering approach preferred by Delft University of Technology (Hereinafter TU Delft). A detailed design was not made. The influence of Project Yogya is therefore limited be the bounds of provisional design and final design.

Although cost considerations generally have large influences on designs, they have only been made in assumptive manner. On the policy side, economic interests as well as the general characteristic effects of Indonesian policy, governance and culture have only been discussed marginally. In the framework presented in Figure 0-2 the influence area of Project Yogya has been marked in red.

PROJECT ORGANISATION

Project Yogya is a student-run, non-profit engineering initiative supervised by TU Delft, a renowned university that ranks 2nd worldwide in the field of Civil Engineering (QS rankings, 2015). The Project was commissioned by Balai P.W., a Ministry of Public Works watershed bureau concerned with the tasks of flood control, coastal protection, water management and river engineering covering the Serayu river basin to the Opak river basin. Project Yogya started 31 August 2015.

ACADEMIC VALUE AND INTERPRETATION

Project Yogya is technically a multidisciplinary project of the TU Delft. A multidisciplinary Project is an elective course within the master of Civil Engineering. The TU Delft promotes master students who wish to gain experience applying their academic knowledge to existing situations, especially in a challenging foreign environment. The findings from this project are highly relevant to the development of academic knowledge regarding breakwater construction and coastal dynamics at hydrodynamically challenging coasts such as in South Java.

READING GUIDE

This two-legged report presents an analysis of the progress-impeding issues (Part I) and follows up with a recommended design proposal accompanied by an effective implementation policy (Part II). The report then follows with a chapter that summarizes the conclusions and recommendations and ends with a chapter that gives a priority-ranked list of future research.

More specifically, in Part I an outline is given of the historical development of the Tanjung Adikarto Port (Ch. 2), current progress-impeding issues at the port area (Ch. 3) as well as future scenarios for the Port (Ch. 4). Based on conclusions drawn from these sections, the point of departure for Project Yogya's review design of the harbour entrance is determined in Part II. One iteration of the design cycle is executed up to the level of a final design for the breakwaters (Ch. 5) and concepts for coastline governance are given (Ch. 6). A policy proposal is made to provide a framework for continuation of the design process, implementation of the design, maintenance of the port entrance and overall process optimization (Ch.7).

References to meetings, visits and presentations where Project Yogya was involved, are made using brackets of the type *[Institution (optional activity number)]*. The choice has been made to refrain from ascribing certain insights and interpretations to the person involved in that meeting. A complete list of the stakeholder network that the investigators have engaged with is given, but it should be stated that none of these stakeholders can be held accountable for the information provided in this report.

Should one have any interest in linking a certain statement to a certain individual, one is advised to contact the investigators with the contact information provided in this report. In consultation with the stakeholder, an assessment will be made of the necessity of revealing stakeholder source information.

PART I: ASSESSMENT

ASSESS THE ISSUES THAT IMPEDE COMPLETION OF THE TANJUNG ADIKARTO PORT

[Chapter 1]	**HISTORY OF THE TANJUNG ADIKARTO PORT** *How has the Tanjung Adikarto Port project started and developed historically?*
[Chapter 2]	**CURRENT STATE OF TANJUNG ADIKARTO PORT** *What is the current state of the Tanjung Adikarto Port?*
[Chapter 3]	**FUTURE OF THE TANJUNG ADIKARTO PORT** *What are the prospects for the Tanjung Adikarto Port?*

The Tanjung Adikarto Port, which was anticipated to be in operation at te time of writing, is still unopened. Further construction of its breakwaters was put to a stop in 2012. To date, the construction has not been resumed for reasons disputed by the involved stakeholders and because of increased concerns regarding the significant economic impulse prospected to outweigh large investment costs.

In this section the issues that impede completion of the Tanjung Adikarto Port are assessed. Historical developments have been researched and how the construction came to a standstill. Additionally, a thorough analysis is presented on the current state of the port. Lastly, prospects for the future of the port are discussed. Based on the history, the current state and prospects for the future of the port, this section will conclude on the area of focus for future interventions.

1. HISTORY OF THE TANJUNG ADIKARTO PORT

In 2012 the construction of the breakwaters of the Tanjung Adikarto Port was put to a stop. The harbour, which was anticipated to be in operation already, remained unopened. To date, the construction is still not resumed for reasons disputed by the involved stakeholders and concerns regarding the significant economic impulse prospected to outweigh large investment costs.

In order to understand the present issues in the Tanjung Adikarto Port, the following question should be answered:

What have been historical developments in the realization of Tanjung Adikarto Port?

Having a complete timeline of the history will help understand the cause the present issues in the area and lay the groundwork for recommendations in the future.

1.1 PRE-HARBOUR CONSTRUCTION DEVELOPMENTS

Before large involvement of provincial authorities, local farmers were key in the mitigation of flood risks. If the farmers were late to act, the high discharges from the Serang River, in particularly in the beginning of the rainy season, would found their way to planes around the river estuary, flooding agricultural land. The Balai P.W. at the time decides to invest in river revetments and in 1993 its construction commences. In 1996, as part of the South Java Flood Control Sector Project, it is decided to further mitigate flood risks by fixating the harbour mouth. During this project, Prov. M.A.F. conducts a feasibility study for a fishing port in the Special Province of Yogyakarta as is elaborated upon in the next paragraph.

1.2 FEASIBILITY STUDY FOR HARBOUR CONSTRUCTION

According to the feasibility study that supported initiation of the Glagah Jetty Project, as conducted in 2001, the construction of a harbour at the South Java Coast in the Special Province of Yogyakarta would seize various opportunities. First and foremost, a harbour could bridge the gap between supply and demand within the province. As concluded in a 2012 study, about 43 000 tons of fish is imported into the province each year while utilization of the provincial coast fish resources remained relatively small. The generated fishing activities could have a substantial role in the development of the local economy: an increase of the income of fishermen and fish farms, an increase of employment and an enlargement of the quantity and diversity of trade commodities. It could thereby trigger the economic multiplier effect in the region. Also, the port could be of interest to the Indonesian Navy as it might accommodate safe stopover berths at the south Java Coast for the Indonesian fleets (Prov. M.A.F., 2012).

Glagah Beach was found to be the most strategic place within the province for the development of a harbour. The location is close to the main trade road across South Java: *Jalur Lintas Selatan*. It is close to major cities such as Wates (10 km), Purworejo (25 km), Yogyakarta (40 km), Kebumen (45 km) and Magelang (60 km). It is not far from the Adisucipto International Airport (55 km) and it is very close to the planned new international airport in the Kulon Progo district (3 km). Land for new facilities and expansion was found widely available around the harbour. Sediment input from the river was estimated low as a result of river works and related dredging activities (Prov. M.A.F., 2012).

It was concluded that the Tanjung Adikarta Port was an economically feasible concept. It should be noted that the hydraulic conditions have never been assessed in this research (Prov. M.A.F., 2012).

1.3 POST HARBOUR CONSTRUCTION DEVELOPMENTS

In 2003, a first design is made for a so-called *Glagah Por*t which can be seen in Figure 1-3. The designed breakwaters would serve as protection against river floods in the estuary, referred to by local authorities as *river bluff attacks*. As the construction of the port is advancing in the period of 2003 to 2005, the Serang River discharges still remain disruptive for the estuary and the river banks. In 2005 to 2007 the river embankments are further extended by means of gabion dikes as shown in Figure 1-4 and Figure 1-5. After Prov. M.A.F. formally transfers responsibility of the breakwater construction to Balai P.W., in 2007 a start is made with the construction of the western breakwater which is shown in Figure 1-7.

However, severe issues arose soon. Longshore sediment flows silted up the Serang River estuary and the high discharges of the rainy season in 2008 high discharges broke through the porous structure. The breakwater fails locally and the structure collapses under its own weight (Figure 1-1). In addition, the tip of the western breakwater collapses due to high waves from the Indian Ocean (Figure 1-2).

FIGURE 1-1 LOCAL FAILURE OF THE WESTERN BREAKWATER

FIGURE 1-2 COLLAPSE OF THE TIP OF THE BREAKWATER

In 2008, a review design is made and the construction of the eastern breakwater starts in 2008. When about half of the breakwater is built, construction stops as wave overtopping is considered too damaging. It was only in 2010 that further advancements were made in construction. A retaining wall on top of the eastern breakwater is created and inner breakwater and head protections are installed on the western breakwater. In 2011, a consecutive review design is made since UGM Research Center concludes that the breakwater angle should shift to open the navigation. Construction on the western and eastern breakwater is continued in 2011 and 2012, of which the result is shown in Figure 1-6. In 2012, the unfinished construction is brought to a halt as multiple parties disagree on further continuation. Therefore, merely repair works were executed in 2013. Figure 1-8 shows the breakwaters as is. A third review design is made in 2013, however construction based on this design is yet to be commenced.

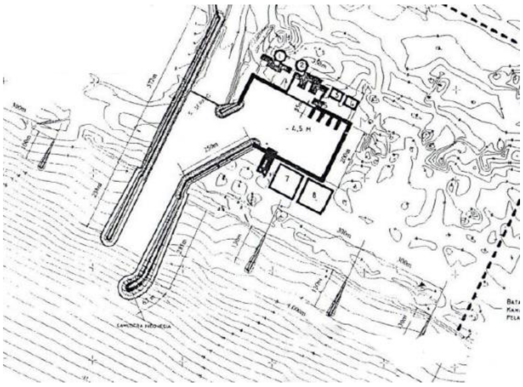

FIGURE 1-3 DESIGN DETAILS GLAGAH PORT (2003)

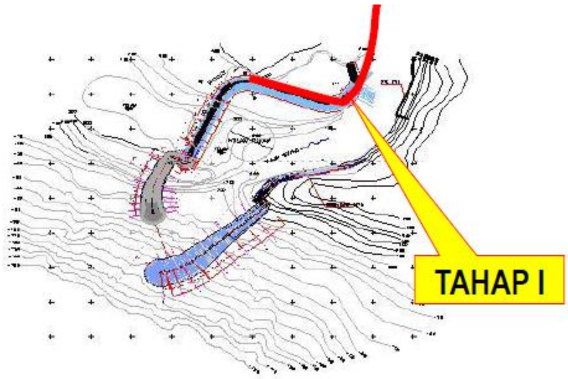

FIGURE 1-4 CONSTRUCTION OF WESTERN GABION DYKE PHASE I (2005-2006)

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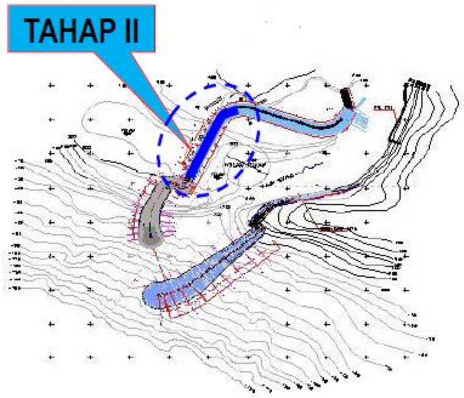

FIGURE 1-5 CONSTRUCTION EXTENSION OF WESTERN GABION DYKE PHASE II (2006-2007)

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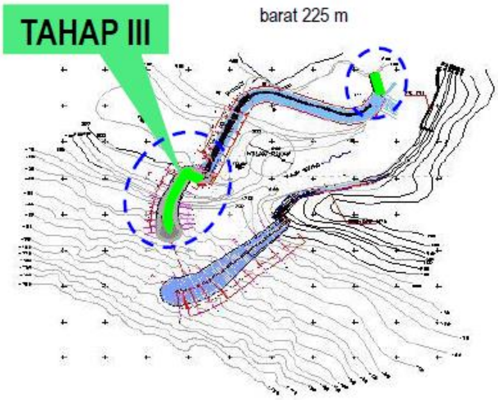


FIGURE 1-7 FIRST CONSTRUCTION OF THE WESTERN BREAKWATER (2007-2008)

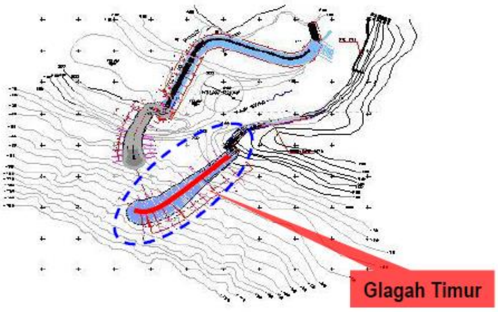


FIGURE 1-6 FIRST CONSTRUCTION OF THE EASTERN BREAKWATER (2008-2009)

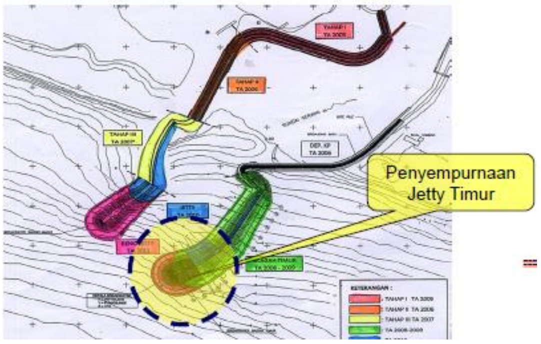


FIGURE 1-8 FURTHER CONSTRUCTION ON THE EASTERN BREAKWATER (2012)

1.4 TIMELINE GLAGAH JETTY PROJECT

As can be understood from the description above, the Glagah Jetty Project has a history full of planning, reviews and decomposed construction works. An overview of these activities is given in Table 1-1. An elaborate timeline is presented in appendix A.

TABLE 1-1 TIMELINE OF GLAGAH JETTY PROJECT

Year		Activity	Figure No. 2-#
	⚠	Initial Issue: floods	
1993	✎	**Detailed Design of the Serang River Estuary**	
1996	✎	**Java Flood Control Sector Project**	
2000	✎	**Development study on the South Beach Area**	
2001	✎	**Feasibility of a Fishing Port Development Plan Glagah**	
2003	✎	**Design Details Glagah Port** ▪ Construction starts.	1
2005	✎	**Design Details for Repair Works** ▪ The Gabion dikes are designed.	
2005	🏗	**Construction of Western Gabion Dyke (Phase I)** ▪ 440 m of Gabion Dyke constructed.	2
2006	🏗	**Construction extension of Western Gabion Dyke (Phase II)** ▪ Sheet pile insertions (625 m).	3
2007	🏗	**First Construction of the Western Breakwater (Phase III)** ▪ Construction of breakwater with 225 m length.	4
2008	⚠	The breakwater fails locally and the structure collapses under its own weight.	
2008	✎	**Review Design (1) of Glagah breakwaters**	
2008	🏗	**First Construction of the Eastern breakwater** ▪ Construction of eastern breakwater was not finished	5
2010	🏗	**Further Construction on the Glagah Jetty Project** ▪ Addition of a retaining wall on top of the eastern breakwater.	
2011	✎	**Review Design (2) of Glagah breakwaters**	
2011	🏗	**Further Construction on the western breakwater**	
2012	🏗	**Further Construction on the eastern breakwater** ▪ The design of Prof. Nur Yuwono is not fully executed.	6
2012	MOU	A Memorandum Of Understanding (MOU) proposal is sent by Min. P.W. to Min. M.A.F. The memorandum is not established.	
2012	⚠	**Construction works of the Glagah Jetty Project are postponed.**	
2013	🏗	**Critical Repair Works on the western breakwater**	
2013	✎	**Review Design Port, including Review Design (3) of Glagah breakwaters**	

2. CURRENT STATE OF THE TANJUNG ADIKARTO PORT

This chapter describes the current state of the Tanjung Adikarto port. Firstly, it presents an environmental analysis of the harbour area and describes the hydraulic conditions present. Secondly, a technical analysis of the identified issues is given: Floods, Breakwater integrity, Coastal instability and Navigability. Furthermore, a stakeholder analysis is described and its results are discussed. Finally, conclusions are drawn that form the basis for further chapters. The following question will be answered in this chapter:

What is the current state of the Tanjung Adikarto Port?

A thorough analysis of the current state of the port enables the investigators to link design choices in the past to current issues and thereby heavily contributes to the accomplishment of the goal of reaching an effective integral design for the future.

2.1 ENVIRONMENTAL ANALYSIS

This section will focus on creating an understanding of the project area. This will be achieved in the form of an environmental analysis in which the functions of the area around the project site will be given along with information about the infrastructure in this region. Next, a brief classification of the coast in the project area will be described.

PROJECT AREA

A framework needs to be defined which represents the area which is immediately affected by the project site. Within this framework we are interested in the functions of each of the areas. The defined framework, bounded by the red line, is shown in Figure 2-1.

FIGURE 2-1 PROJECT FRAMEWORK

The project area consists of the river basin, the navigation channel, two breakwaters and the surrounding beaches. The area in the framework has several functions of which a distinction is made in Figure 2-2. A description of the functions of the harbour area is given in the table below the figure.

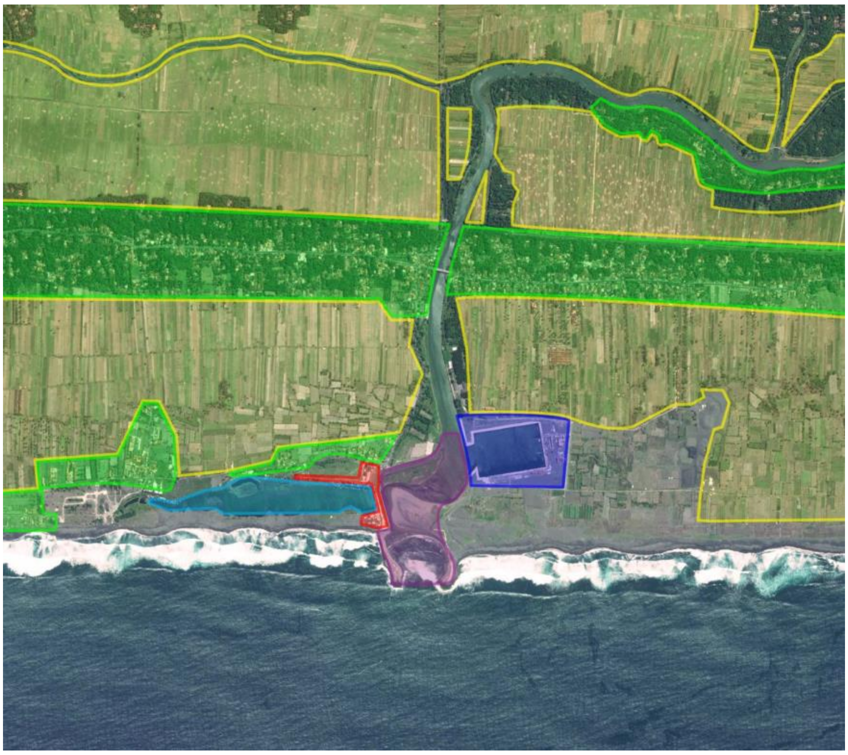


FIGURE 2-2 CURRENT AREA FUNCTIONS

TABLE 2-1 FUNCTION DESCRIPTION

Element	Description
Harbour basin: 🟦	The harbour basin is situated on the eastern river bank. The dimensions of the basin are 300 by 200 m. The basin is currently very shallow, but it will be dredged to a depth of 4.5 m below LWS. The quay wall of the basin is built of concrete sheet piles. On the quay wall, there are some storage buildings. Besides that there is enough space for other purposes like cranes.
Navigation channel: 🟪	The navigation channel connects the basin to the ocean. It is fixed by two breakwaters and revetments along the northwestern part of the waterway. At present time, large amounts of sediment regularly block the entrance.
Lagoon Area: 🟦 🟥	The old river lagoon (light blue) is currently used for fishery and recreation. Currently, little fish is found in the lagoon so fishery is still a minor part of the activities there. Recreation consists of boat rental and swimming as well as paddle boat rental. The dimensions of the lagoon are 1,000 by 120 m. The area around the lagoon (red) consists of small restaurants, food carts and several souvenir shops. This area is important since it makes the recreational lagoon more attractive to visit.
Resident area: 🟩	The green areas are residential areas. The area around the main road *Jalan Daendels is* the place where most residents' houses are situated.
Agriculture: 🟨	In the area around the harbour, a lot of agriculture activities take place. The most common activity is farming. On the western side of the breakwaters, there are a few fish farms.

The infrastructure around the harbour consists of many small roads that lead to the main road of the region. This *Jalan Daendels* main road is a two-lane asphalt road in good condition. The distance from this main road to the harbour basin is about 700 m. The road to the basin is significantly smaller than *Jalan Daendels*.

Transportation over the river is limited. Some fishermen from villages upstream use the river for fishing and for transportation of persons and goods. To this end, only small boats are currently being used.

CLASSIFICATION OF THE COAST

The project site is located at the convergent boundary between the Indian Ocean plate and the Eurasian continental plate. Hence, the coast can be characterized as a leading edge coast or in this case also an island arc collision coast. It has a sandy littoral zone of about 1 km. This zone is characterised by sand dunes, beach ridges and swales. Along the project area, the coast is wave-dominated. The wave field is dominated by swell waves, relatively low and long waves, generated by westerlies in the Southern storm wave belt. These high energetic waves are uniform in direction, shape and size with a typical significant wave height of 1.5 m with wave periods of about 10 s. Seasonal variations are small compared to storm waves. At this location the beaches consist of black volcanic sand with an approximate diameter of 0.2 - 0.3 µm.

2.2 HYDRAULIC CONDITIONS

In the following tables, the hydraulic conditions are given. A more detailed explanation of each condition is given in the Appendix B. These are the conditions used for further design.

TABLE 2-2 WATER LEVEL AND TIDE CHARACTERISTICS

Characteristic	Determined level
Highest High Water Level (HHWL)	+ 2.16 m
Mean Sea Level (MSL)	+ 1.08 m
Low Water Spring (LWS)	+ 0.00 m
Wind set-up	+ 0.60 m
Design water level	+ 2.76 m

TABLE 2-3 WAVES AND RETURN PERIOD

Return period [years]	Determined significant wave height [m]
1	3.03 m
5	4.13 m
10	4.53 m
30	5.16 m
50	5.44 m
100	5.80 m
120	5.89 m
150	6.07 m
250	6.37 m

TABLE 2-4 WAVES AND CHARACTERISTICS

Characteristic	Value
Significant wave period	15 s
Dominant wave direction	South, Southeast

TABLE 2-5 WATER LEVEL ADDITIONS

Characteristic	Value
Storm surge level and wind set-up	+ 0.60 m
Sea level rise	0.00 m, Investigation recommended

TABLE 2-6 DISCHARGE AND RETURN PERIOD OF THE SERANG RIVER

Return period [years]	Determined discharge [m^3/s]
2	125 m^3/s
5	189 m^3/s
10	228 m^3/s
30	299 m^3/s
50	332m^3/s
100	377 m^3/s
120	389 m^3/s
150	404 m^3/s
250	437 m^3/s

2.3 ANALYSIS OF TECHNICAL ISSUES

The issues related to environmental conditions and usability will be identified and discussed in this section.

FLOODS

Before construction of the breakwaters, the Serang river was unprotected and naturally carved its way through the lava sand into the Indian ocean. Its shape was continually subject to change through the conjunction of waves, sediment drift and river discharge. The river mouth could vary over a distance of 2 km.

In the dry season, a combination of a small tidal range and low river discharges often resulted in river flows which were insufficiently large for flushing sand deposits in the sea. Additionally, under prevailing longshore sediment transport and wave conditions a spit usually developed that blocked the creek of the river. When the wet season started, the river suddenly gained in power and high discharges approached the sand spit. If the outlet did not adapt in time, the hinterlands inundated. The area around the river mouth was most prone to floods.

To prevent the floods from occurring, villagers dug a guide channel for each instance when a high discharge was foreseen. When the timing was right, the river could then penetrate the sand spit and expand naturally by dissolving the surrounding sediment. However, when the villagers dug too early, the system naturally replaced the sediment and the area would inundate. Direct flood damage, when it occurred, was caused by unpredictable floods that people could not precisely forecast in a certain year. It consisted of damage to standing crops, residential areas, livestock, fishery infrastructure and communal buildings.

In order to reduce flood risks, plans were made to fixate the river mouth. In a series of works, river embankment was realized by the South Java Flood Control Sector together with drainage improvement in the flood plain covering 4 954 ha. The project reduced the flood area from 2 091 ha to 472 ha (Asian Development Bank, 2007). The project was followed by the construction of the breakwaters as can be seen in chapter 1. It was hoped that the river would be strong enough in the dry season that it would break through the spit automatically.

Since the construction of the breakwaters, floods around the river mouth are no longer observed [Balai P.W. 1] [Reg. W.R]. However, dredging works are still required. Floods that do persevere are caused directly by rainfall and river overbank spilling further upstream.

Furthermore, floods can enter the river basin when Progo River overflows which still has a non-fixated river mouth. Lack of maintenance of gates and other structures in the river systems, lack of drainage in the flood prone areas and abundant presence of weeds are believed to be the main contributing factors to flood occurrence. Weeds grow heavily as a result of fertilizer wash-out which is a consequence of current crop patterns. Right before the rainy season, farmers try to harvest, making use of fertilizers exuberantly [Reg. W.R].

WATER QUALITY

As part of the South Java Flood Control Sector Project, the Serang River was sampled for factors related to irrigation supply and public health. Water quality was reported excellent or adequate for irrigation, except for the Serang estuary as a result of salt intrusion. Regarding public health, water was found to be high in coliforms and low in iodine, the latter causing the onset of an iodine-dependent disease, scrua. As concluded from the project, water quality aspects important for fishery were found to be adequate. The conclusion was supported by the aquatic life that was found.

The South Java Flood Control Sector Project, initiated in 1996, led to the creation of retaining walls along the river site that put a stop to salt intrusion. The influence of salt intrusion was effectively diminished according to the district water manager of the Kulon Progo Regional Government [Reg. W.R.]. Salt intrusion is no longer an issue.

BREAKWATER INTEGRITY

The master plan of building a harbour meant breakwaters had to be built that provide a safe entrance for the fishing boats into the harbour area. They should also minimize the sediment intrusion coming from longshore direction such that the navigation channel remains navigable. The basis of the design for the Glagah Jetty Project is a breakwater with a tetrapod armour layer. Tetrapods are often used as an armour layer due to their high degree of interlocking, which makes them very stable.

ORIGINAL DESIGN

The original design, constructed from 2007 to 2009, was based on a significant wave height of 4.50 m. The design and its dimension are shown in Figure 2-3 and in Table 2-7 in the column *Design*. During construction, several issues arose. First, a decline in the height of the structure occurred. Second, the wave attacks made the tip of the western breakwater collapse. Third, as a consequence of building the western breakwater first, longshore sediment accreting at the eastern site of the western breakwater closed the river mouth. This caused the river to put pressure on the inside of the western breakwater and damaging it. The eastern breakwater lost significant amounts of core material during construction.

To solve these issues, several improvements were made to the design. This caused the final construction to differ from the original design in several aspects. The two main differences between design and construction are the height and the length of the breakwaters. Both breakwaters ended up with a core height of 4 m with the height of the pavement at 6 m above LWS. The improved design actually prescribes a height of 8 m above LWS. To solve this mismatch in design and/or construction, a prefab wall is constructed on top of the eastern breakwater up to +8 m LWS. For the western breakwater this wall is not yet constructed but planned in the future. Another difference between design and construction is that the breakwaters are not constructed up to the design length. This is because the construction was stopped due to lack of funds. The column *Construction* in Table 2-7 shows a complete list of what has been built.

TABLE 2-7 DESIGN AND CONSTRUCTION SPECIFICATIONS ORIGINAL DESIGN

		Design	Construction
Length western breakwater [m]		250 (-10.5 m LWS)	215
Length eastern breakwater [m]		300 (-12.0 m LWS)	180
Height western breakwater [+ m LWS]		8.0	6.0
Height eastern breakwater [+ m LWS]		8.0	6.0 (+2 m Seawall)
Armour layer outer [t]		9.0	9.0
Armour layer head [t]		11.5	11.5
Armour layer inner [t]		7.0	7.0
First under-layer [t]		1.0 - 1.5	Unclear
Core [t]		0.5 - 1.0	0.1 - 0.5
Toe berm	**Weight [t]**	3.5	3.5
	Length [m]	15	15
Filter layer [t]		0.5 - 1.0	0.1 - 0.5

*It is unclear whether this layer is still in place. It is estimated that most of the stones on the outer layer of the breakwaters are displaced or completely washed out when the tetrapods started sliding down.

Remark: Aside from these basic specifications, additional concrete blocks and tetrapods are placed on several locations where the breakwaters are damaged.

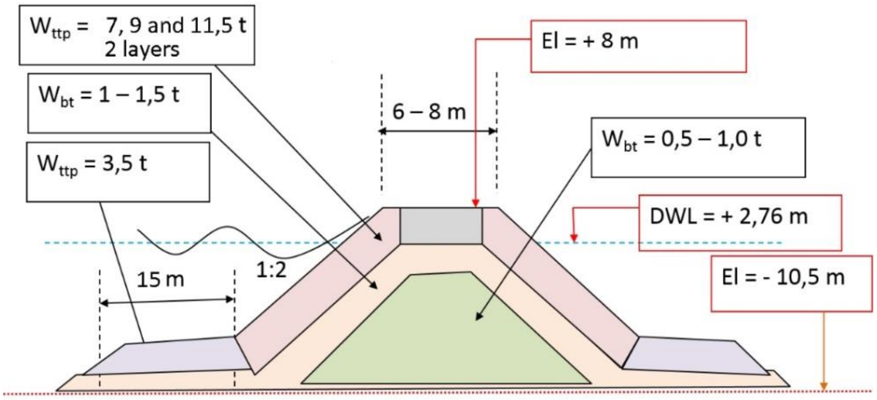


FIGURE 2-3 ORIGINAL BREAKWATER DESIGN (NUMBERS IN INDONESIAN DECIMAL STANDARDS)

These differences between design and construction occur due to several reasons. Firstly, the harsh wind and wave climate in the Indian Ocean makes construction challenging. Secondly, the lack of funds were key in the design adaptations during construction. Repairing a mistake in construction is too costly and lacks sufficient budget. The difference in weight of the core can be explained by the unreliability of the quarry run. Altogether, it is very challenging for the contractor to construct the breakwaters identical to the design or to solve unforeseen issues along the way.

STRUCTURAL INTEGRITY ASSESSMENT

The structural integrity can be affected by wrong assumptions in the design as well as differences between design and construction. The primer is investigated first.

DESIGN AND INTEGRITY

The improved design consists of two layers of tetrapods. As can be seen in Figure 2-4, the tetrapods are placed in straight rows. However, in Figure 2-5 it can be seen that most tetrapods have shifted and the interlocking effect is not in place any longer, making the armour less stable. The tetrapods are sliding down, which indicates that the toe should be longer and/or heavier.

Another reason for the shifting tetrapods may be an underestimation of the significant wave height. In the design, a design wave height of 5.0 m has been the point of departure. However, it is concluded that waves have at least a 1/30 year chance to exceed a height of 5.0 m (

Table 2-3).This is quite high, considering the breakwaters are designed to have a lifetime of 50 years. The probability of failure considering this 50-year construction lifetime would be at least 80%, which will be explained further in chapter 4.2. Failure can be defined as *the exceedance of the design wave height*. When the wave height exceeds the design wave height, the structure fails in its function to block waves and/or limit over wash to a certain amount.

Near the breakwaters, turbulent water streams have developed which are damaging the armour layer. This effect seems to have been neglected in the design. Also, the hangs of the tetrapods used for lifting have corroded strongly. This impedes replacement of tetrapods and should be investigated thoroughly.

FIGURE 2-4 PLACEMENT OF TETRAPODS

FIGURE 2-5 DISPLACEMENT OF TETRAPODS

CONSTRUCTION AND INTEGRITY

Mismatches in design and construction are clarified per element in Table 2-8, summarized in Table 2-10.

TABLE 2-8 DESIGN VERSUS CONSTRUCTION OF BREAKWATERS

Element	Clarification
Height	As a consequence of the limited height, significant overtopping occurs. To solve the mismatch in design and construction, a prefab wall has been constructed on top of the structure up to +8.0 m LWS. This wall seems to function sufficiently. The western breakwater still lacks a prefab wall.
Length	The mismatch in design and construction leads to heavy sedimentation in the harbour. The breakwaters do not cross the breaker zone and longshore transport is trapped in the harbour. The extra length of the western breakwater compared to the eastern breakwater only enhances the entrapment. Dredgers are in place to compensate for the sedimentation. However, the harbour is still full of sediment.
Core	The limited weight of the core causes the core elements to move significantly or even flush away. This process slowly degenerates the stability of the structure.
Armour layer	While the armour layer should consist of tetrapods only, displaced core elements and concrete cubes originating from the toe were found at the outer layer. The primer being a result of a collapse of the western breakwater, when core elements slid down the slope. Because reshaping the core would be too costly, it was decided to place the tetrapods on the upper part of the breakwater directly on top of the core. On the lower part of the breakwater, no tetrapods were placed because the extra core rocks that shifted down had already reached design height and width. The concrete cubes from the toe were placed for the purpose of stability.

REVIEW DESIGN

Prof. Nur Yuwono, an UGM professor in coastal engineering, proposes a review design which would be able to withstand the rough wave climate. The following tables show this design. The major changes are that this design uses a storm frequency of 1/100 years and a significant wave height of 5.8 m. Another large difference is the use of cube blocks of 1.8 tons as the first underlayer for the full length on both sides of the breakwaters and as filter layer below the toe berm. Table 2-9 gives a full comparison of the original design and the review design.

TABLE 2-9 SPECIFICATIONS OF THE REVIEW DESIGN COMPARED TO THE ORIGINAL DESIGN

		Original design	Review design
Storm frequency [years]		1/50	1/100
Probability of failure during lifetime [%]		63.2	39.3
Significant wave height [m]		5.0	5.8
Length western breakwater [m]		250 (-10.5 m LWS)	250 (-10.5 m LWS)
Length eastern breakwater [m]		300 (-12.0 m LWS)	300 (-12.0 m LWS)
Height western breakwater [+ m LWS]		8.0	8.0
Height eastern breakwater [+ m LWS]		8.0	8.0
Armour layer outer [t]		9.0	11.5 / 14.0
Armour layer head [t]		11.5	18.0
Armour layer inner [t]		7.0	9.0
First under-layer [t]		1.0 - 1.5	1.8 (Cube blocks)
Core [t]		0.5 - 1.0	0.5 - 1.0
Toe berm	**Weight [t]**	3.5	3.5 / 5.5 / 7.0
	Length [m]	15	15
Filter layer [t]		0.5 - 1.0	1.8 (Cube blocks)

SUMMARY

From the damage that occurred in the period after construction, it can be concluded that the breakwaters will most likely not last a full design lifetime. Too many tetrapods have moved already and the core is poorly protected due to the lack of a proper filter layer. It seems that the wave climate has been underestimated.

Another conclusion that can be drawn is that at this point the breakwaters do not support the creation of a safe harbour entrance, thereby not fulfilling their function. The waves still travel far into the harbour and are too high for fishing boats to sail through.

A design will have to be made which provides a durable solution which can be implemented. Since the review design is the most recent review design, the investigators will evaluate this design simultaneously. The original design will also be evaluated. By doing so, it can be made clear whether the instability issues are related to a fault in the design process.

A summary of the main issues along with their consequences and measures taken so far is given in Table 2-10. Figure 2-6 gives an overview of these, showing the location at which they occur.

TABLE 2-10 SUMMARY CONSEQUENCES AND MEASURES OF BREAKWATERS

Issue	Consequence	Measure
Insufficient height	Significant overtopping	Prefab wall 2 m on western breakwater
Insufficient length	Sedimentation	Dredging
Abrupt finish	Significant damage to breakwater, high probability of failure	-
Underestimated Significant wave height	Significant damage to breakwaters, high probability of failure	Increased in review design
Under dimensioned toe	Toe displacement and sliding down of tetrapods	-
Armour layer displacement	Reduced durability	Additional placement of concrete blocks
Core loss during construction	Increased duration of construction and total cost of project	-
Turbulent water	Abrasion of tetrapods	-
Corrosion reinforcements	Hampered replacement of tetrapods	-

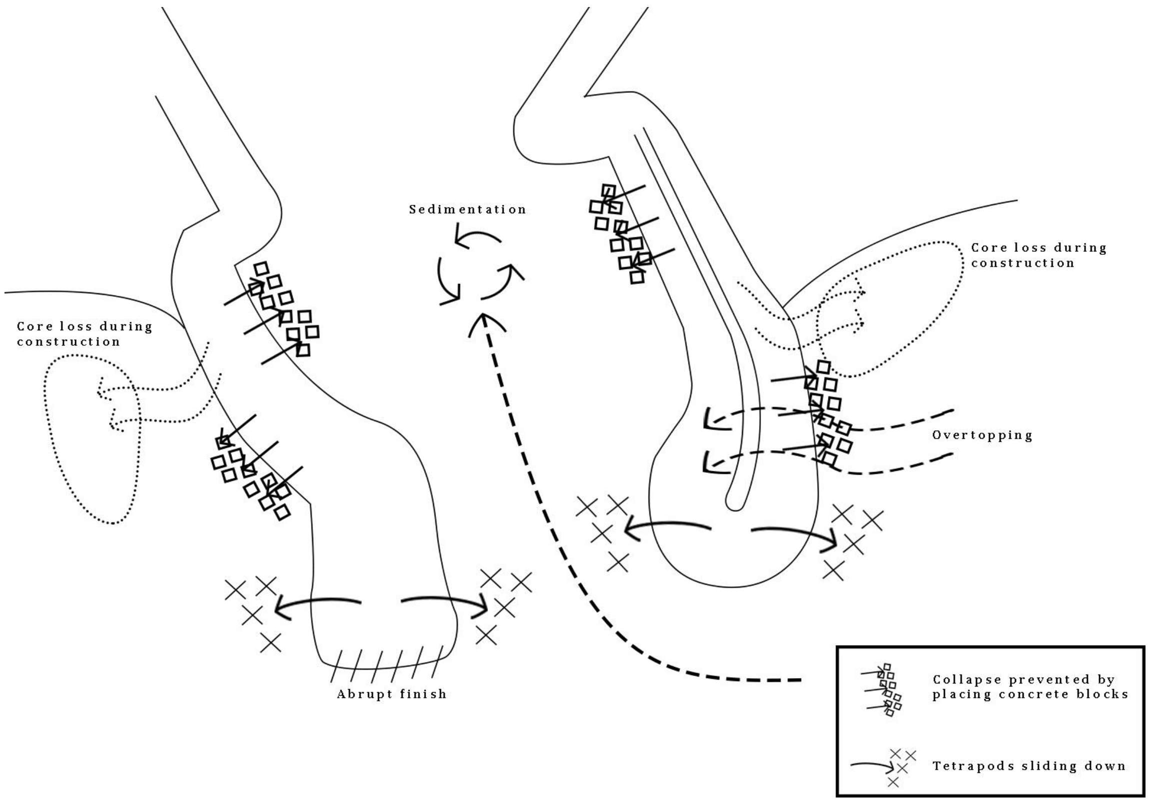


FIGURE 2-6 OVERVIEW OF FAILURES

COASTAL INSTABILITY

Since the orientation of the South-Javanese coastline barely varies near the project location, it can be concluded that the coast from Cilicap in the west up to Parangtritis beach in the east functions as one general coastal system. The boundaries of this system are formed by existence of two mature abutments at those locations. This defined coastal system itself can also be divided in two subsystems, separated by a small abutment at Gombong.

The sediment in the global system is trapped in the two subsystems, exposed to a net sediment transport to the west and supplied by 14 rivers. The sediment inside this system is transported due to wind, waves and tides, resulting in reshaping of the coast by changes in the sediment transport. Of the three processes, the shifting coastline is most influenced by the waves as they determine the largest rate of sediment transport along the coast.

The coastline responses to environmental changes can be divided into two different timescales, *short-term* and *long-term* development. At both time scales, natural processes as well human interaction cause environmental changes, resulting in a varying sediment distribution over time that endlessly shapes the coastal areas. *Short-term* development can be clearly divided into *nature-induced* and *human-induced* development. However, due to complexity *long-term* development is often a mix between natural processes and human interaction as human activity has an enormous impact at the environment and natural processes at present time. These two impacts are difficult to distinguish.

SHORT-TERM NATURE-INDUCED DEVELOPMENT

Short-term nature-induced development by natural processes is mainly driven by variations in wave height and direction as the longshore sediment transport is almost fully dependent on the wave climate. Although current sediment transport data is present, it is hard to quantify the process accurately. Nevertheless, multiple methods have been designed to determine the amounts of this transport. During the previous designs of the Glagah harbour the longshore sediment transport is estimated several times. In this report, the findings of *Pusat Studi Ilmu Teknik* (research department of UGM) have been compared with the study of Balai P.W. about the Glagah harbour.

PUSAT STUDI ILMU TEKNIK

The estimation of the longshore sediment transport by Prof. Nur Yuwono is based on wave statistics from the USNMCAW subject to modification by JICA in 1989. These wave statistics are shown in Table B-2 in appendix B.

In 2001, Prof. Nur Yuwono made his first estimation of the sediment transport at the project location. This first study, simply based on only two wave directions, also gave the first indication of the quantity of the sediment transport. The used sediment transport formula is not known. The outcomes are showed below:

S_{west} = 480 000 m^3/year
S_{east} = 405 000 m^3/year
S_{nett} = 75 000 m^3/year towards the west

A second review is performed by Prof. Nur Yuwono in 2012. Again, the wave data from Table B-2 in appendix B is used. The results of these approximations are calculated using the CERC formula. The results of this study are shown in Table 2-11.

TABLE 2-11 OUTCOMES FOR SEDIMENT TRANSPORT BY PROF. NUR YUWONO, 2012

Wave direction	Wave height [m]	Sediment transport [$10^3 m^3/y$]	Total sediment transport [$10^3 m^3/y$]	Direction
Southeast	0.5	5.2	575.0	Westwards
	1.5	184.6		
	2.5	298		
	3.5	87.2		
South	0.5	14	449.3	Westwards
	1.5	147.7		
	2.5	189.2		
	3.5	111.0		
		Total westwards transport	**1024.3**	**Westwards**
Southwest	0.5	5.1	731.0	Eastwards
	1.5	148.4		
	2.5	340.5		
	3.5	237.0		
		Total eastwards transport	**731.0**	**Eastwards**
		Net transport	**293.3**	**Westwards**

(BAB 5. Review perencanaan pemecah gelombang Glagah, 2013)

BBWS SERAYU OPAK

In 2013, Balai P.W. published a study on the Glagah harbour including a wave and sediment transport analysis (UGM, 2013). In this report, the coastline model *Genesis* (Generalized Model for Simulating Shoreline Change) is used to determine the rates of the sediment transport. The results of the simulation are based on a 10-year simulation time and are shown in Table 2-12.

TABLE 2-12 SIMULATION RESULTS SEDIMENT TRANSPORT

Direction	**Maximum sediment transport [$10^3 m^3/y$]**	**Average sediment transport [$10^3 m^3/y$]**
Westwards	1 028.0	906.4
Eastwards	477.0	347.9
Net	551.0	558.5

COMPARISON

The sediment transport directed westwards shows great similarity in both calculations, while the eastwards sediment transport shows large irregularities. The numbers indicate that the prediction by the CERC formula is overestimated to a small extent, assuming a direct relation between the wave height and direction and the sediment transport. According to the occurrence rate of the wind and wave data that are responsible for the western and eastern sediment transport, the ratio is about 2:1 respectively. When comparing the ratios estimated by Prof. Nur Yuwono and the Balai P.W. study, the ratio estimated by the Balai P.W. study shows the greatest similarities. Therefore it is decided to continue this report with the estimation of the study by the latter, as this seems the most plausible outcome based on the assumption mentioned above.

STORM EVENTS

Although regular short-term development of the coastline almost fully depends on variations in occurrence of the incident wave angle of the mean swell wave height, storm events also have large impact on the short-term development. Due to the unpredictability of the frequency and severity of the storms, it is hard to include these in calculations. As there is no data available about storm impact in the project area, the severity has to be determined by incidental observations.

FIGURE 2-7 STORM OBSERVATION (PROF. NUR YUWONO)

In Figure 2-7 the results of one of those observations is shown. After the occurrence of a severe storm, a well on the beach was almost bared for 4.5 m as in shown in the left picture. According to the observations, three weeks later only 2.5 m was bared as can be seen in the right picture. During the relative calm wave conditions after the storm, the beach recovered slowly until it found its stable position again. These kinds of observations show the enormous coastal dynamics of the exposed beaches of the south Javanese coasts.

SHORT-TERM HUMAN-INDUCED DEVELOPMENT

As explained in the section of natural-induced development, the direction of the wave-driven longshore sediment transport depends on the incident wave angle. Though this changes over the year, an annual net transport to the west dominates the shore. Directly after the construction of the breakwaters, the longshore sediment transport was blocked by the structure. Depending on the incident wave angle, accumulation takes place at the one side of the breakwaters. At the opposing side, the sediment transport rate restores itself by taking sediment from the coast, resulting in erosion of the coastline. This is referred to as short-term human-induced development.

Due to the imbalance of the transport directions, at the western side of the breakwater the accumulated sand of the eastern transport at the western side of the breakwater is not sufficient to compensate the ‘sediment hunger’ of the western transport. This results in a transgressive coastline at the western side of the breakwater. Adversely, on the eastern side large accumulation is extending the beach, resulting in a regressive coastline.

In Figure 2-8 these processes are shown by a comparison with situation of the coastline before and after the construction of the breakwaters. The current situation, captured by satellite images, is shown in the right picture. In the left picture an estimation of the original coastline is made by extending the orientation of the adjacent coastlines using the satellite images.

FIGURE 2-8 COASTLINE BEFORE AND AFTER CONSTRUCTION OF THE BREAKWATERS (ESTIMATION)

Note: The situation before the construction of the breakwaters in the left picture is purely an estimation based on averaging of the orientation of the adjacent coastlines. Regarding the experience of coastline changes after breakwater constructions, the estimation should approximate the actual situation in the past. However, one should not forget that the occurrence of small deviations with that situation is highly possible but that has no significant influence on this analysis.

In Figure 2-8 one can clearly observe erosion at the western side of the breakwaters. The coastal strip between the lagoon and the sea has become increasingly narrow. Notable is the shape of the erosion pattern. The shape is dissimilar to the accumulation pattern. This is the result of a return current in the shadow zone of the western breakwater, originating from set-up differences due to difference in wave heights in and outside the shadow zone. This return current is partly transporting the sediment in eastern direction, resulting in the observable shape it has nowadays.

These observations have been simulated by the *Genenis* model in the study of Balai P.W. In the simulations, the length of the west and east breakwaters are assumed to be 290 and 250 m respectively. It was chosen to make use of a prediction time of 10 years. The results of this simulation are shown in Figure 2-9.

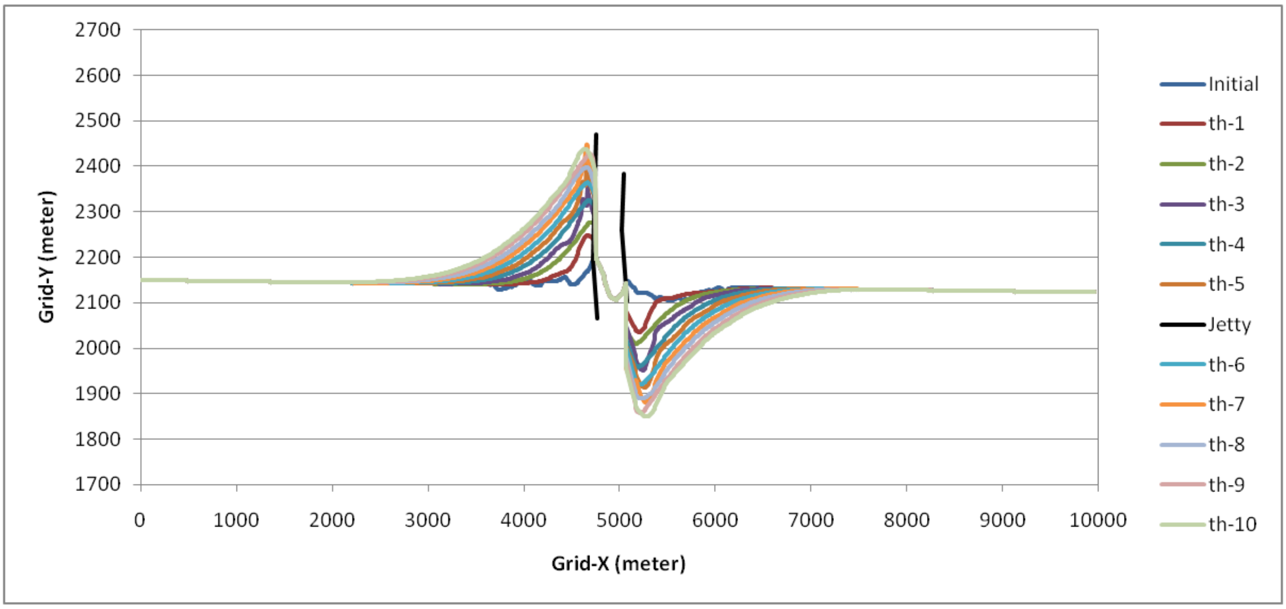


FIGURE 2-9 SIMULATION OF COASTLINE ADAPTATION DUE TO BREAKWATER PLACEMENT (UGM, PSIT, 2013)

The simulations show that during the first simulation year the coastline will be eroded by 87.38 m at the western side of the breakwaters and will accumulate 79.27 m at the eastern side of the breakwaters. After 10 years, the erosion zone will increase about 272.00 m and the accumulation zone will expand up to 279.48 m.

It is stated that it is safer to determine the accumulation zone to be 300 m with a deviation of 50 m due to the absence of calibration and large uncertainties in input data. Hence, after 10 years the accumulation of the coastline is able to reach 250m up to 350m. The erosion zone should be subjected to the same rate of deviation. However, in the model the return current induced by the shadow zone is being neglected. This return current is smoothening out the erosion zone, resulting in less erosion at one place. Therefore the erosion is assumed to be 275 m with a deviation of 50 m due to the uncertainties of the calculations. Therefore, after 10 years erosion of the coastline will be able to reach 225m up to 325 m.

LONG-TERM DEVELOPMENT

Long-term coastline development is influenced by two main phenomena. The first phenomenon is a rise or drop of the sea level that changes the accommodation space of the sea. The second phenomenon is a change in the volumes of sinks or sources in the coastal area that at their turn will influence the net wave-driven longshore sediment transport. The coastal profile will adapt to the new situation by regression or transgression of the coastline. In reality, most of the time, both phenomena occur due to complex interactions of environmental changes and human activities. Nowadays, the question rises on what scale these environmental changes are human-induced. Therefore, no distinction is made between natural-induced and human-induced development.

The current globally accepted sea-level rise supports transgression of the coastline in the system. Also, rumours about illegal mining activities in the river basins support the notion of transgression. Measurements at Kuwara beach performed in a field survey by Prof. Nur Yuwono, shown in Figure 2-10, hint on the very same process. In a period of 8 years, the coastline retreated for almost 30 to 35 m at the western part of the beach, which equals an average setback of about 4 m/year. Prof. Nur Yuwono expected that these variations in the position of the coastline are the result of disruption of the supply of sediment from the catchment area. It is wise to pay attention to these indications as it seems that the coastal profile is already adapting. Therefore, it is strongly recommended to start monitoring in order to get more insight into the short-term and long-term development of the coastal system.

FIGURE 2-10 COASTLINE RETREAT AT KAWURA BEACH (12 KM EAST OF THE GLAGAH JETTY PROJECT SITE)

NAVIGABILITY

Because of the high waves at the entrance between the breakwaters, the fishing boats can only exit the harbour area during low tide when the waves are less high. When returning, boats can enter the harbour area by riding the waves, so they can do this during both low and high tide.

At this point, the breakwaters do not help to create a safe harbour entrance and thereby their function is not fulfilled. The waves still travel far into the harbour and are too high for fishing boats to travel through.

The turning circle is designed to be 165 m. The turning circle cannot be recognized due to the large amount of sediment in the harbour entrance. Besides the turning circle, also the navigation channel and the harbour basin are currently not at a depth of the designed 4.5 m. This is due to sediment that comes from the ocean as the sediment from the river is blocked by structures upstream as is further described in appendix B.5. To improve the current situation, there are two companies dredging to reach the desired depth. The dredging machines have a pump capacity between 750 and 3000 m^3/s (Appendix E [Balai P.W. 1]).

TABLE 2-13 NAVIGATION CHANNEL DESIGN VERSUS CONSTRUCTION

Element	Design	Construction
Width navigation channel	80 m	<< 80 m
Diameter turning circle	165 m	-
General depth	4.5 m	<< 4.5 m

TABLE 2-14 SUMMARY CONSEQUENCES AND MEASURES OF THE NAVIGATION CHANNEL

Element	Consequence	Measure	Improvement necessary
Width navigation channel	Low navigability	Dredging	Yes
Diameter turning circle	Low navigability	Dredging	Yes
General depth	Low navigability	Dredging	Yes

2.4 STAKEHOLDER AND POLICY ANALYSIS

In this section a deep dive is taken into the stakeholders involved in the Glagah Jetty Project. Stakeholders are assessed based on their impact, their opportunities and their concerns. These characteristics yield a support factor, which can be interpreted as *the extent of support for successful completion of the Glagah Jetty Project*. For each stakeholder a corresponding conclusion is given about its position in the Glagah Jetty Project. A complete overview of the stakeholders is given in Appendix C.

Ministry of Public Works and Public Housing (Min. P.W.)

Menteri Pekerjaan Umum dan Perumahan Rakyat (PUPR)

Min. P.W.

Minister
Dr. Ir. M. Basoeki Hadimoeljono, M.Sc.

Impact	●●●●●	**Support**	−○●○○○+

Conclusion

Despite having constructed the current breakwaters, the Ministry is aware of the Tanjung Adikarto harbour development dilemma and is evaluating the proposition that expected prospects of economic activity in Kulon Progo justify large investments for the breakwater lengthening.

Ministry of Maritime Affairs and Fisheries

Menteri Kelautan dan Perikanan (KKP)

Min. M.A.F.

Minister
Minister Susi Pudjiastuti

Impact	●●●●●	**Support**	−○○○○●+

Conclusion

Min. M.A.F. is aware of the financial gains, the boost in public and project image and advances of technical know-how that come with development of the Tanjung Adikarto Port. Despite being highly in favour of the project, the Ministry currently acknowledges that the costs and profits are uncertain. It should be noted that the Min. M.A.F. position can be classified as unstable since some officials say that in the past an MOU has been declined on the Ministry's behalf for unclear reasons.

Provincial Government of the Special Region of Yogyakarta, Department of Maritime Affairs and Fisheries

Dinas Kelautan dan Perikanan (DKP), Daerah Istimewa Yogyakarta (DIY)

Prov. M.A.F.

Chair
Mr. Andung Prihadi Santosa

Impact	●●●●○	**Support**	−○○○○●+

Conclusion

Despite the realization that Prov. M.A.F. is responsible for development of harbour activities and its implications for construction and maintenance, the department does not pro-actively engage with these activities because it lacks experience and sufficient budgets.

Serayu Opak River Basin Bureau
Balai Besar Wilayah Sungai Serayu Opak (BBWS SO)

Balai P.W.

Chair
Ir. Tri Bayu Adji, MA

Impact	●●●●○	**Support**	−○●○○○+

Conclusion	As executing party, Balai P.W. takes execution orders from Min. P.W. and is therefore bounded by decisions on a national level. The Glagah Jetty Project is not Balai P.W.'s responsibility, but considering the project past, Balai P.W. is a natural partner for the responsible governmental bodies. Balai P.W. considers the policy surrounding the Glagah Jetty unstable, but promotes the completion of the Glagah Jetty Project as the Balai P.W.'s interests in economic development are aligned with those of the province.

Provincial Government of the Special Region of Yogyakarta,
Provincial Planning Agency
Bappeda, Daerah Istimewa Yogyakarta (DIY)

Prov. Bappeda

Chair
Mr. Tavip Agus Rayanto

Impact	●●●●○	**Support**	−○○●○○+

Conclusion	Prov. Bappeda can coordinate stakeholders and thereby succeed in its task to enable provincial departments to shift the Yogyakarta Province economy slightly towards maritime developments.

Government of Kulon Progo Regency,
Department of Maritime Affairs and Fishery,
Water Resources Benefit Section
Pemerintah Kabupaten Kulon Progo,
Departemen Kelautan dan Perikanan,
Sumber air Benefit Bagian

Reg. W.R.

Chair
R. Kuntarso

Impact	●●●●○	**Support**	−○○○○●+

Conclusion	The regency of Kulon Progo is at the verge of potentially entering a new era of economic progress, but relies on higher-level institutions for the investments in this future which causes an insecure future of the project.

Completed with information from Appendix C, the stakeholders can now be differentiated in groups that can be characterized by the need to be informed, leveraged, engaged or monitored. A stakeholder map can be derived from the findings.

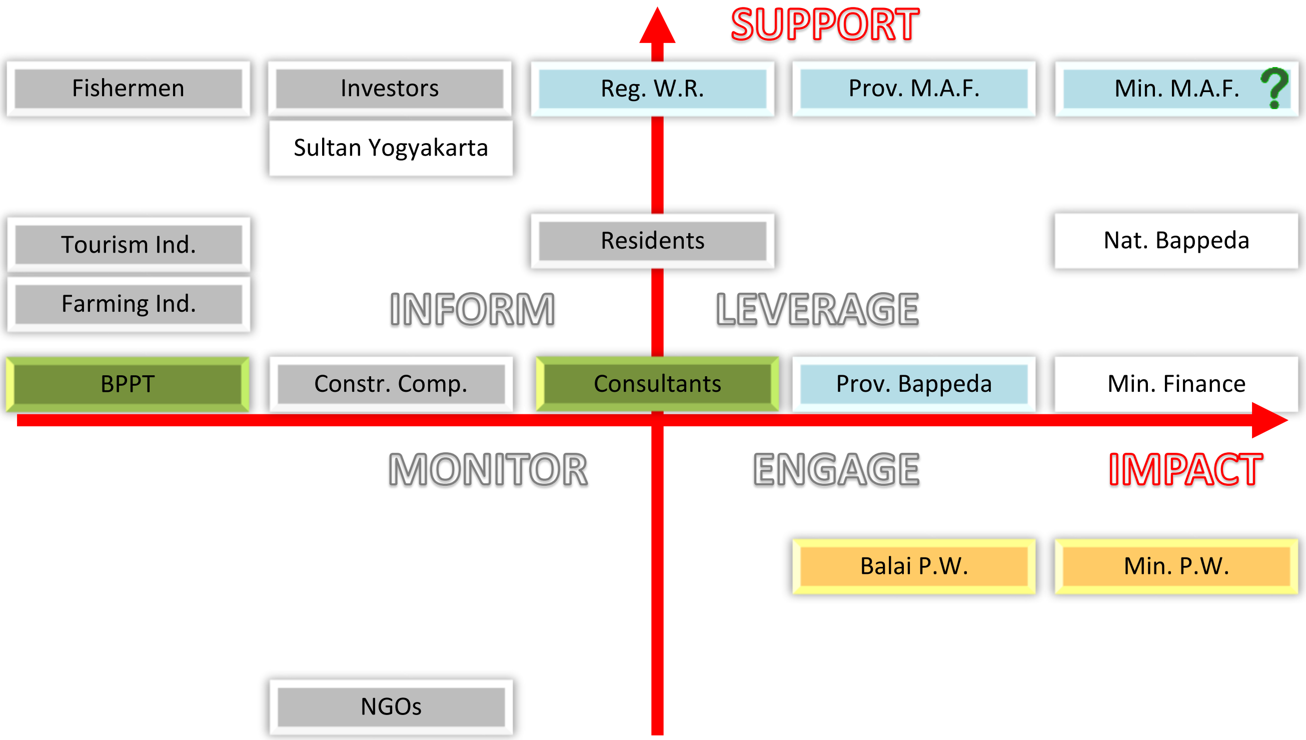


FIGURE 2-11 STAKEHOLDER MAP OF GLAGAH JETTY PROJECT.

The relationships between high-impact stakeholders have been investigated since this aspect uncovers potential problems.

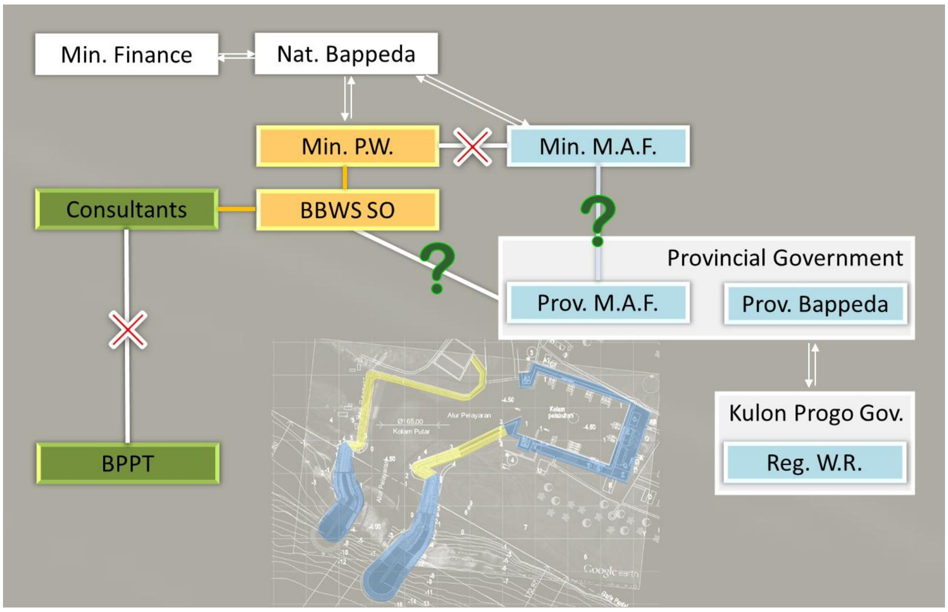


FIGURE 2-12 RELATIONSHIPS BETWEEN MAJOR STAKEHOLDERS

Figure 2-12 depicts the relationships between the stakeholders. Depicted in blue are stakeholders who carry the official responsibility for the area mapped in blue: the harbour basin and the breakwaters. Depicted in yellow are Serang River revetments who are responsibility of the Public Works institutions, also highlighted in yellow. The relationships between the Public Works institutions and Maritime Affairs and Fisheries institutions are unclear or unproductive at the time of writing. Balai P.W. hired a consultant and this relationship has been good. However, consultants and research institutions are lacking a professional, active working relationship. An elaborate overview of stakeholder relationships is given in Appendix C.

SUMMARY

Based on the stakeholder analysis, a series of conclusions can be drawn:

- Although the breakwaters have so far been constructed by the Balai P.W., the true responsibility lies with Min. M.A.F.
- Min. M.A.F. and Prov. M.A.F. are hesitant with continuing the Glagah Jetty Project but do acknowledge its necessity.
- The official legal responsibilities do not necessarily reflect a stakeholders commitment.
- In the current situation, aside from concerned local stakeholders, all major parties are in favour of pursuing further breakwater construction.
- When analysing the relationships of the stakeholders it can be concluded that a new cooperation framework should be established.

PROGRESS-IMPEDING ISSUES

Of particular interest to the investigators were the issues that impeded progress of the Glagah Jetty Project. By solving these issues, not only great advances can be made in the policy and governance framework of the stakeholders, direct effects will be significant for the construction of the breakwaters and their maintenance.

In this section the following issues will be discussed:

- Breakwater construction stop;
- Lack of adequate data collection and sharing;
- Lack of safe, durable and sustainable maintenance dredging;
- Issues with execution.

An elaborate explanation of the current issues is given in Appendix D.

Stakeholder meetings were executed to discuss these issues. The statements gathered in this section are based on 13 stakeholder meetings. The contact information of these stakeholders have been mostly forwarded to Project Yogya by Balai P.W. An approximation of the truth has been found by comparing statements from interviewees and by literature verification. In order to remove the language barrier, at all times a translator was present for translations from English to Bahasa Indonesia. It should be noted that the translator was employed by Balai P.W., but pledged to translate neutrally to the best of his or her ability. In case the translator wished to be involved in the discussion, permission was requested and granted when appropriate.

The 13 stakeholder meetings are listed below. The full stakeholder meetings are described in Appendix E and referred to in the texts below. For example, references are made in the form [UGM] when discussing results from stakeholder meeting 1 at UGM.

|1|	**Dialogue about Breakwater integrity and redesign by Prof. Nur Yuwono**	[UGM]
|2|	**Glagah Jetty Project Site Visit**	[Balai P.W. 1]
|3|	**Exploratory dialogue about floods, dredging and the involvement of Kulon Progo Regency**	[Reg. W.R.]
|4|	**Start-up meeting**	[Start-up]
|5|	**Exploratory Dialogue about harbour development**	[Prov. M.A.F. 1]
|6|	**Political structure dialogue about glagah jetty development and funding**	[Balai P.W. 2]

|7|	**Exploratory dialogue about Glagah Jetty research and lengthening**	[BPPT]
|8|	**Exploratory dialogue about harbour planning and feasibility**	[Prov. Bappeda]
|9|	**Elaborative dialogue about harbour development**	[Prov. M.A.F. 2]
|10|	**Elaborative dialogue about harbour development 2**	[Prov. M.A.F. 3]
|11|	**Internal Meeting about research results**	[Int. Pres.]
|12|	**Project Yogya Symposium**	[Symposium]
|13|	**Meeting with Minister of Public Works**	[Minister]

BREAKWATER CONSTRUCTION STOP

According to many parties [Balai P.W. 1] [Reg W.R.] [Prov. M.A.F. 2], the Glagah Jetty extension stop is not an engineering problem but it is a political problem. The reason for the construction stop of the eastern breakwater in 2012 has been disputed by the involved parties. The investigators have considered all scenarios that emerged from the stakeholder meetings. These scenarios have been evaluated as presented in Appendix D. Based on verification discussions and historical facts, it can be concluded that the following scenario is true:

“Since there is no MOU on a national level, legal responsibility for maintenance has not been discussed and constructing further would not pass the Balai P.W. audit. Due to the construction stop, the maintenance costs have risen unexpectedly high.”

LACK OF ADEQUATE DATA COLLECTION AND SHARING

A major issue in the design of the Glagah Jetty is the absence of accurate data and statistics for wave and wind climates. This is acknowledged by the current consultant [UGM]. At the time of writing, the following key information is absent or not shared:

- Front-end engineering guidelines for investment-maintenance ratios based on Indonesian standards for breakwater design;
- Location-specific near-shore wind and wave data and more recent offshore wave data;

It can clearly be stated that effective research collaboration and data and statistics gathering should be re-evaluated. Large research initiatives and results remain behind closed doors, despite being extremely valuable in some occasions. This led to the common belief among the stakeholders that Project Yogya’s destiny is *review upon review upon review*.

The severity of the lack of collaboration can be characterized by the fact that at least some involved workers at Balai P.W. were not aware of BPPT experiments on the Glagah Jetty Project.

LACK OF SAFE, DURABLE AND SUSTAINABLE MAINTENANCE DREDGING

Current dredging operations are aimed at:

|1| Enabling safe navigability in the navigation channel;
|2| Enabling safe navigability in the harbour basin;
|3| Removing accreted sediment bulks near the toe of the eastern breakwater.

The dredged materials are currently being disposed in onshore locations [Reg. W.R.] [Prov. M.A.F. 1]. A missing, fourth goal should be:

|4| Nourishing of coastal erosion zone west of the western breakwater.

Due to sub-optimally formulated KPIs, a dominant socialization responsibility and issues with the execution of dredging activities, it can be concluded that Prov. M.A.F. cannot guarantee a durable, sustainable and safe dredging policy, let alone its governance.

EXECUTION ISSUES

Major execution issues were revealed in the timeline analysis as well as the stakeholder meetings. Many issues have been acknowledged by both Balai P.W. and its consultants [E| UGM] [E| Balai P.W. 1]. The highest impact issues are highlighted.

Single-year programs, a lack of in-house knowledge among the executing governmental institutions and lenient execution supervision have resulted in major issues for the Glagah Jetty Project. Although the hiring of external consultants shows good progress at the ministries, the solution has not yet been found.

GENERAL ISSUES

The investigators have noted their general observations in order to raise awareness about cultural differences from an outsiders' point of view.

BUREAUCRACY

In any governmental institution worldwide strong bureaucracy will be observed. However, in Indonesia it is observed that extensive rule and law frameworks are impeding dynamic work behaviour. An example of excessive bureaucracy is observed at the organisation of the internal presentation for this research project where considerable time was spent on formalization of invitation documents at several desks.

The investigators realize that the impact of bureaucracy is justified as it is also an effective fraud prevention method. This trade-off could be re-evaluated on a regular interval.

PASSIVE APPROACH

The investigators observe a culture of passive approach. Passive approaches cause situations to sometimes escalate as employees do not look beyond their responsibilities. A vacuum of responsibility arises, and with no person acting upon the vacuum issues tend to increase in magnitude. An example of such a responsibility vacuum is the current issue that addresses the lack of clarity as to what governmental institution should lengthen the breakwaters.

Project Yogya promotes a pro-active culture in which people are pro-actively involved with their responsibilities. Pro-active behaviour can be characterized as upmost intent to anticipate issues and intervene in difficult situations.

KULONOWUN

The Javanese 'Kulonowun' is a cultural phenomenon that can be best described as 'over apologizing'. Kulonowun is both criticized and embraced on the work floor. The phenomenon creates a culture in which people show their regret for not being able to being able to satisfy one's expectations.

One the one hand, this culture is admirable as it polishes workers' interaction skills and yields a perception of comfort among colleagues. On the other hand, the culture of Kulonowun strongly promotes hierarchy. A worker that brings up a potentially sensitive topic to a higher ranked officer might be criticized. The high usage of Kulonowun implicates that workers are easily inclined to apologize for their ideas or thoughts. Hence, this could indicate a certain reticence among workers to speak up. Experiences from the (internal) stakeholder meetings endorse the hypothesis that this reticence can sometimes damage effectiveness.

Project Yogya stresses that incentivizing personnel to be more critical could facilitate a more open culture for discussion. However, the investigators acknowledge the hierarchical work ethics can be effective and admit that the Dutch style of open debates has its inefficiency downsides – let alone its effects on pleasant work atmospheres.

2.5 EVALUATION

A series of statements can be made relating to the four main issues discussed in the technical and stakeholder analysis.

Floods

After the construction of the breakwaters, floods around the river mouth are no longer observed. However, dredging works are still required.

Floods that do persevere in the river basin are directly caused by rainfall and river overbank spilling further upstream. Additionally, floods can enter the river basin when Progo river overflows since it still has a non-fixated river mouth.

Lack of maintenance to gates and other structures in the rivers in Kulon Progo, lack of drainage in the flood prone areas and abundant presence of weeds are believed to be the main contributing factors to flood occurrence in the river basin.

Breakwater integrity

The breakwaters will not last a full design lifetime. A sigfnificant amount of tetrapods has moved already and the core is poorly protected due to the lack of a proper filter layer. It seems that the wave climate has been underestimated.

At this point the breakwaters do not ensure a safe harbour entrance, thereby there not fulfilling their function. The waves still propagate far into the harbour and are too high for fishing boats to sail through.

There is a large disparity between the technical breakwater design and the actual execution outcome.

Navigability

The navigation channel, turning circle and the harbour basin are currently not at an adequate depth for the ships that the harbour was originally designed for.

Coastline stability

Western longshore transport is approximately two times as large as eastern longshore transport.

Both accretion at the eastside and erosion at the western side of the breakwaters are estimated as 250 - 350 m in 10 years. The sand wedge between the lagoon and the beach will erode in its entirety if no measurements are taken.

Observations indicate an overall regression of the shoreline between Cilicap and Parangtritis on the long-term. However the regression rate is unknown.

Stakeholders and policy

Many public and private companies, institutions and organisations hold a stake in the Glagah Jetty Project and support lengthening of the breakwaters. However, it is unclear what governmental or private organisation can be held responsible for continuing construction of the breakwaters. Moreover, the issues that impede further progress are related to stakeholder management, data acquisition, unverified assumptions, dredging approaches and issues related to execution. Additionally, cultural issues such as a passive approach and stringent bureaucracy can be seen as high-impact hurdles for advancement.

FURTHER RESEARCH

From the observations and analysis in this report it can be concluded floods around the river mouth do not occur ever since the construction of the breakwaters. Therefore this issue is neglected in further investigation. The issue of navigability is neglected as well. Currently the navigability of the harbour is disturbed due to the limited depth as the result of sedimentation in the harbour. It can be solved by adequate dredging. Nevertheless, this sedimentation is a result of coastal processes and longshore sediment transport. Ideally, one researches both preventive coastal governance measures and recursive dredging strategies. As longshore sediment transport turned out to be of high dimension, in the limited timespan for this project it is decided to lay the focus coastal governance rather than dredging.

3. FUTURE OF THE TANJUNG ADIKARTO PORT

This chapter discusses the opportunities for harbour construction at the Glagah beach, the future economic scenarios for the surrounding area and possible investments strategies regarding the Glagah Jetty Project. The main question to be answered in this chapter is:

What are the prospects for the Tanjung Adikarto Port?

At the end of this chapter the goal for Project Yogya's review design is established based on the history, current state and future prospects of the Tanjung Adikarto Port.

3.1 OPPORTUNITIES

As presented in the feasibility study the Tanjung Adikarto Port, the port can seize the following opportunities (Prov. M.A.F., 2012):

- *Fishing:* the harbour can fill the gap of import and export of fish in the region (43,000 tons per year).
- *Tourism and recreation:* the harbour could offer a residential area for tourists in the area. Several recreational facilities are already in place and a substantial new international airport is planned in the Kulon Progo district.
- *Stopover berth harbour:* the Indonesian Navy needs safe stopover berths at the South Java Coast.
- *Prestige:* the region as well as the Special Province of Yogyakarta have the opportunity to build a well-functioning harbour in harsh conditions, which would be a unique project in Indonesia. The Glagah Jetty Project is already considered national prestige.

3.2 ECONOMIC SCENARIOS

Despite positive location aspects with regards to potential, uncertainty in economic prospects is still significant. No statements can be made regarding the success of the harbour without evaluating these in an economic feasibility study. A broad spectrum of scenarios is considered realistic for the upcoming years.

In a very good scenario, the planned new international airport is realized and serves up to 10 million passengers per year. According to the airport plan, later expansions might accommodate up to 20 million passengers per year. With the airport in place, export of the fishing industry may expand and recreation around Tanjung Adikarto harbour may increase. Tourists could be drawn by Glagah Beach and its recreation facilities. Airport hotels at Glagah Beach would be developed and the lagoon would offer tourists a place for swimming since sea conditions are quite rough. Yachts may anchor in the harbour as the tourism industry flourishes. The local population would be able to find jobs at the harbour site and enjoy education for the required demand for labour. At the time of writing, land acquisition for the airport is 84% complete and the target completion date has been set to the 31st of December 2018. (CAPA centre for innovation, 2016)

However, if the airport does not develop and tourism stays away from Glagah Beach, the scene could be quite different. Recreational facilities would not be able to develop. And if local training programs are not adequate for intended fishing activities in the harbour then the local population may not benefit from the harbour activities at all. Disappointing harbour use could affect the profitability of the harbour.

When answering the question whether the Glagah Jetty Project should be resumed, it is absolutely crucial to perform a sound risk analysis and economic feasibility study to the harbour. Project Yogya builds on the assumptions underlying the Glagah Jetty Project that a favourable economic growth scenario will develop upon completion of the harbour.

3.3 THE INVESTMENT DILEMMA

Many policy makers have mentioned the dilemma regarding further investment in the Glagah Jetty Project. The dilemma was most adequately described by Dr. Ir. Arie Setiadi Moerwanto, MSc., at the first Glagah Jetty Project and Tanjung Adikarto Symposium [Symposium]. When asking oneself whether further investments are justified, two arguments can be made.

- *As soon as the harbour shows great potential, investments in the breakwaters are justified*
- *As soon as the breakwaters prove to protect the harbour entrance, investments in the harbour are justified*

This was named the *Chicken and Egg problem*. There is no incentive for any governmental institution to make an investment without its counterinvestment being certain. This problem is self-perpetuating and the solution opted for by Dr. Ir. Moerwanto is to find close cooperation within the government, provided that the harbour is economically feasible.

As for now, there are several possible courses of action for provincial and national-level policy makers. From Project Yogya's perspective, the investment dilemma consists of the following consideration:

- *Do nothing*
  One could decide to not change anything, accept the current situation and refrain from any further investments. In this case, erosion will continue, the potential harbour use is limited and capital destruction will most probably be accepted.
- *Execute current plans*
  The most recent review design was made in 2013 and is now under discussion and questions are raised regarding the justification of the investment. The main reason for that being the uncertainty in technical, political and economic feasibility of the proposal. This design will be discussed in this report.
- *Follow Project Yogya's recommendations*
  This report presents a review design on the 2013 design. It also brings forth policy recommendations for implementation of the design.

All three possible courses of actions need to be evaluated in a later stage. This evaluation is inextricably linked to an economic feasibility study.

PART II: STRATEGY

DEVISE AN ADEQUATE AND COMPREHENSIVE COUNTER STRATEGY FOR THE IDENTIFIED ISSUES THAT IMPEDE COMPLETION OF THE TANJUNG ADIKARTO PORT

The assessment of Part I showed that that the construction of Tanjung Adikarto Port was a long intermittent process of events, repairs and review designs. Evaluation on the current state of the harbour showed issues on three topics: breakwater integrity, coastline stability and stakeholders and policy. Future prospects show large uncertainty in economic scenarios. Based on history, current state and future prospects of the Tanjung Adikarto Port and the assumption that a favorable economic growth scenario will develop upon completion of the harbour, the following specification of the objective can be derived for an adequate and comprehensive counter strategy:

Design a safe harbour entrance including durable breakwaters, a sustainable coastline conserving the lagoon area and ensure its feasibility with an effective implementation governance policy.

Further chapters in this part will elaborate on this specified objective as decomposed below.

[Chapter 4] **GLAGAH JETTY REVIEW DESIGN**

What durable breakwater design can be recommended that ensures a safe harbour entrance?

[Chapter 5] **SUSTAINABLE COASTLINE GOVERNANCE**

What sustainable preservation measures can be recommended to prevent coastal erosion adjacent to the breakwaters?

[Chapter 6] **POLICY FRAMEWORK**

What governance and policy changes can be recommended for an effective integral approach to the Glagah Jetty and Tanjung Adikarto Projects?

4. GLAGAH JETTY REVIEW DESIGN

In this chapter a design of the breakwaters is proposed. In chapter 3 it turned out that the breakwaters do not facilitate a safe harbour entrance. Besides not fulfilling their function, it was determined that the breakwaters will not last the lifetime they are designed for. Too many tetrapods are displaced and the core is poorly protected due to the lack of a proper filter layer. It seems that the wave climate was heavily underestimated in the original design. In order to propose a design that is able to function properly, in Project Yogya's point of view the following question should be answered in this chapter:

What durable breakwater design can be recommended that ensures a safe harbour entrance?

To propose a design that will answer this question successfully, in this section a new design is made by going through the design process from an outsiders point of view.

4.1 PROGRAM OF REQUIREMENTS

In this section the specific requirements are summarized. The requirements are derived in the following subsections: Breakwater, Navigability, Floods, change of coastline and Stakeholders.

FUNCTIONAL REQUIREMENTS

TABLE 4-1 FUNCTIONAL REQUIREMENTS

Navigability	
Vessel volume	100 GRT
Vessel Weight	30 GWT
Vessel Length	55.0 m
Vessel Width	6.0 m
Vessel Depth	3.0 m
Navigation channel Depth	4.5 m
The amount of downtime has to be limited as much as possible	

Breakwater
The breakwaters should facilitate a safe entrance to the harbour basin.
The breakwaters should minimize dredging needs as a consequence of longshore transport
The breakwaters should secure the position of the river mouth.
The breakwaters should be designed for a lifetime of 100 years.

Coastal
The erosion at the lee side of the breakwaters should be reduced, preferably up to a level where there is a balance between dredging and nourishment.
The impact on longshore sediment transport by the breakwaters should be minimized.
No sand can leave the project area [Prov. M.A.F. 3].

Floods
Further development of the region should not influence the flood preventing functions of the structures.

BOUNDARY CONDITIONS

TABLE 4-2 PRINCIPLES

General
The current situation forms the basis for further construction.
It must be possible to construct the structure with Indonesian equipment and methods.
The design should take into account the risk of data being unreliable. High safety factors are preferred for the design.

ASSUMPTIONS

TABLE 4-3 ASSUMPTIONS

General	
Density of salt water	1 030 kg m^{-3}
Maximum fabricated tetrapod weight	18 t
Construction of the designed structure will be executed as planned.	

PREFERENCES

TABLE 4-4 PREFERENCES

General
Optimal ecologic integration is desired.
Building time should be as short as possible and a workaround needs to be found for single-year programs.
The construction should be as cheap as possible.
Any effects other functionalities in the region caused by construction should be limited.

Breakwaters
A design based on a wave return period of 1/250 years is desired for the breakwaters.
The design is desired to follow TU Delft standards.

4.2 YOGYA DRAFT DESIGN

In this section a design is made by going through the design process in one full iteration. Designing starts by choosing the right initial conditions and principles. In the case of a breakwater, that would be defining an allowable probability of failure for the design components. This determination is mostly a consideration between investment and maintenance and is calculated along economic standards for breakwater design. The probability of failure is directly related to the design wave height. Once an allowable probability of failure is defined, the design wave height can be determined and with that a draft design can be made.

The existing breakwaters form the point of departure for the draft design. Although design options should be kept open as far as possible, the breakwater type, characterized by a tetrapod armour layer and a concrete cap, as well as a 1:2 slope along the length of the existing breakwaters, are considered necessary conditions for a feasible redesign. In the breakwater design, the orientation, the height, the armour layer, the under layers, the core and the toe berm details are determined.

PROBABILITY OF FAILURE

The probability of failure needs to be defined to base the design on. This parameter will be used to find the significant wave height. A Poisson distribution is used to determine the probability of failure: (Verhagen & D'Angremond, 2012)

$$p = 1 - \exp(-f * T_L) \tag{5.1}$$

In which:

- p = probability of occurrence of an event one or more times in period t_L
- T_L = considered period (e.g. the lifetime of the breakwaters) in years
- f = average frequency of the event per year

The economic lifetime of a breakwater in the order of 50 years is commonly used. This leads to the failure probabilities for different storm frequencies depicted in Table 4-5.

The choice of the storm frequency not only depends on the probability but also on the consequences. Since the breakwaters create the entrance to the harbour they hold an important function for the Serang River basin. A damaged breakwater will be a large risk factor for the harbour area since it can compromise the functionally of the harbour.

TABLE 4-5 PROBABILITY OF FAILURE PER STORM FREQUENCY

Storm frequency [years]	Probability of failure [%]
1/20	91.8
1/30	81.1
1/50	63.2
1/80	46.5
1/100	39.3
1/150	28.3
1/200	22.1
1/250	18.1
1/300	15.4
1/400	11.8
1/500	9.5
1/1000	4.9

When designing a breakwater, a consideration should be made between investment and maintenance. A more heavily dimensioned and protected breakwater will require less maintenance but will initially cost more. To be able to predict the economic optimum, one should regard data of previous breakwaters. In the Netherlands, data from all designed breakwaters is collected and averaged. Figure 4-1 shows a graph of the storm frequency against the probability of failure during lifetime. The economic optimum lies generally between 5.0% and 20.0% accepted exceedance of design parameters represented by the horizontal green lines in the figure. Depending on the character of the project this optimum will be closer to the upper limit or lower limit. When investment costs are expected to be relatively much higher than maintenance cost, a higher exceedance probability is allowed.

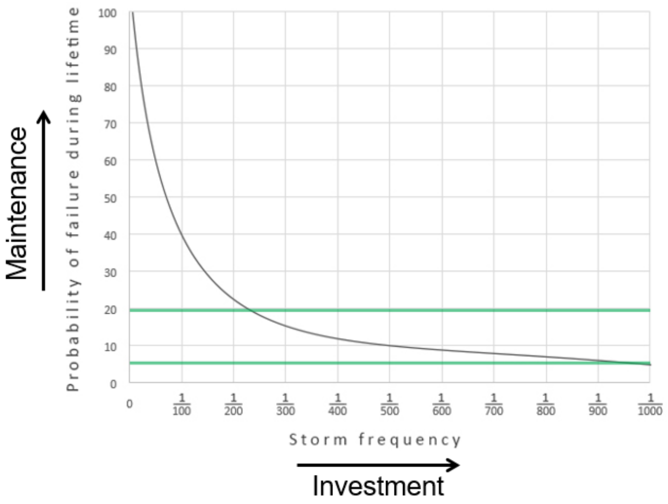


FIGURE 4-1 RELATION BETWEEN PROBABILITY OF FAILURE DURING LIFETIME AND STORM FREQUENCY

For the original design an exceedance probability of 63.2% was allowed, far above the Dutch bandwidth, and maintenance forms a major issue. As described before, damage to the breakwaters is significant and there currently is no repair policy.

In Prof. Nur Yuwono's review design, the balance between investment and maintenance has been revised to somewhat more investment and less maintenance. However the exceedance probability for this design is still outside the Dutch bandwidth.

Figure 4-2 shows the location of the chosen parameters for the original design and the review design.

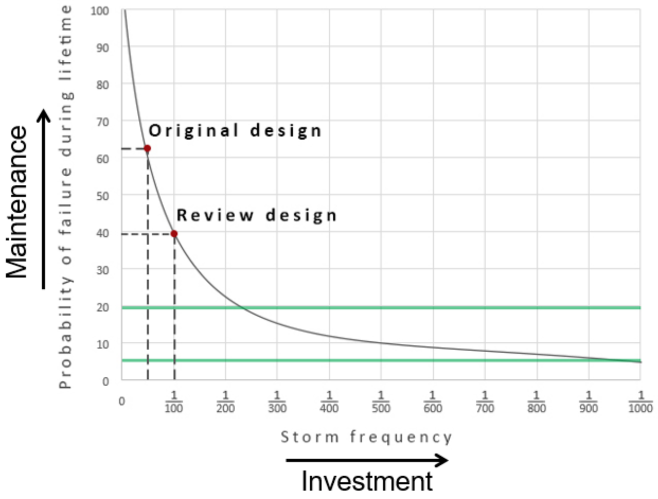


FIGURE 4-2 CHOSEN STORM FREQUENCY FOR THE ORIGINAL DESIGN AND THE REVIEW DESIGN

Because investment and maintenance cost in Indonesia may differ from those in the Netherlands, for example the cost of labour, the Indonesian bandwidth could also be different. Because the Indonesian bandwidth is yet to be established, in Project Yogya the Dutch bandwidth has been considered representative for the Indonesian bandwidth and thus the guiding principle for the design.

Additionally, there is uncertainty about the position of the design within the bandwidth. Considering the steep bathymetry, the investment costs could turn out to be relatively very high. However, up to now maintenance for the Glagah breakwaters showed to be the main issue. A logical decision for the allowed probability of failure for the initial design would therefore be to choose the middle of the bandwidth. In

Project Yogya, however, it was decided to let investment cost be leading for the design of this project. The underlying assumption is that maintenance costs can be reduced with a good governance framework.

In conclusion, Project Yogya considers the upper limit of the Dutch bandwidth as point of departure for the design. Hence, a storm frequency of 1/250 forms a good initial design condition as it corresponds with a probability of failure of 18.1% just below the upper limit of 20.0%. Figure 4-3 shows this position in the graph. A storm frequency of 1/250 yields a significant wave height of 6.38 m (chapter 2.2). This will be used for future calculations.

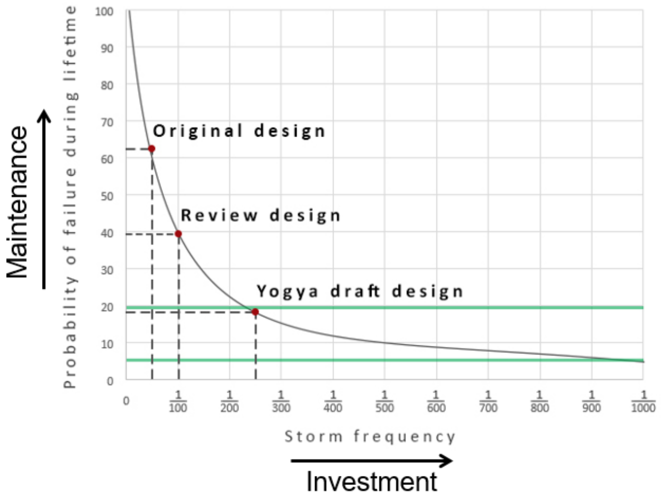


FIGURE 4-3 CHOSEN STORM FREQUENCY FOR YOGYA DRAFT DESIGN

The following table gives the design approach of each element of the breakwaters. The complete design steps can be found in Appendix F.

TABLE 4-6 DESIGN APPROACH

Element	**Determination approach**
Probability of failure	Poisson is used to determine the probability of failure in which the probability of occurrence of an event is based on the lifetime of the structure and the average frequency of the event per year.
Orientation of the breakwater	The width of the navigation channel is determined as a two-way channel exposed to open water. The determination of the required length of the breakwaters is based on sediment transport. The coastline is very dynamic and varies per season. The length is based on the amount of sediment that has to be blocked each season
Height	The required height is calculated based on the storm height of the water level, relative sea level rise, settlement and overtopping.
Armour layer	The required dimensions of the armour layer are calculated using the Van Der Meer formula for tetrapod armour units. A classical computation, a deterministic approach and a probabilistic approach are used in order to come to a conclusion.

First under-layer and core	The rock gradation of the first underlayer and core is determined using general design rules.
Toe berm	The toe berm consists of a filter layer with a double layer of tetrapods on top. The required tetrapods of the toe are determined according to the formula of Van der Meer, D'Angremond and Gerding which gives a relation between the unit weight of toe elements, toe level and damage.

Table 4-7 gives a summary of the draft design of the breakwaters.

TABLE 4-7 SUMMARY DRAFT DESIGN BREAKWATERS

Storm frequency [years]		*1/250*
Probability of failure during lifetime [%]		18.1 %
Length western breakwater [m]		305
Length eastern breakwater [m]		330
Height [m + LWS]		8.5
Armour layer outer [t]		20.0
Armour layer head [t]		25.0
Armour layer inner [t]		7.0
First under-layer [t]		1.0 - 2.0
Core [t]		0.5 - 1.0
Toe berm outer (0 – 8 m)	**Weight [t]**	20.0
	Length [m]	10
Toe berm outer (8 – 12 m)	**Weight [t]**	8.0
	Length [m]	7.5
Toe berm outer (12 m – end)	**Weight [t]**	3.0
	Length [m]	5

Figure 4-4 shows an overview of the location of the different types of tetrapods.

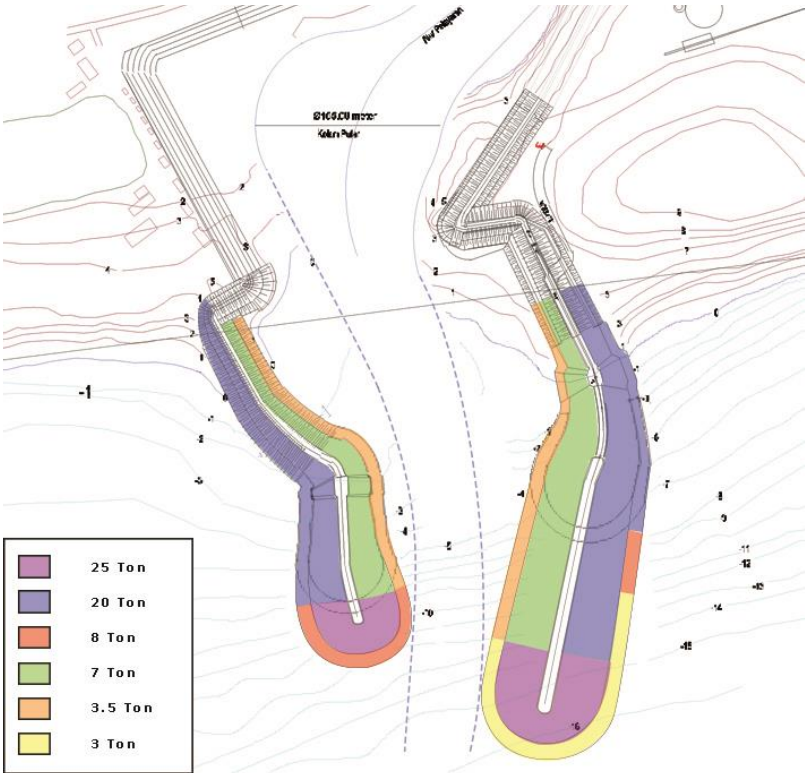


FIGURE 4-4 OVERVIEW OF THE LOCATION OF THE DIFFERENT KINDS OF TETRAPODS IN THE DRAFT DESIGN

REVIEW OF DESIGNS

This section reviews the original design and the review design. This is done by determining each part of these designs in the same way as was done for the draft design, but now by using the probability of failure and significant wave height belonging to these two design. This gives insight in the quality of these designs.

The full review can be found in the appendix G. What can be concluded from the reviews is that both the original plan and the proposal made by Prof. Nur Yuwono are well designed. The main difference lies in the chosen accepted probability of failure and the significant wave height belonging to this probability. This is also what causes the large difference between these designs and the draft design. The original design is based on a storm with a return period of 50 years (Pustek Kelautan - Universitas Gadjah Mada, 2003). This means that, with a lifetime of 50 years, the probability of failure in that period is more than 63%. This is a very high probability. The review design assumes a storm frequency of 1/100. This reduces the probability of failure to 39.3%. Still, this is also a high probability, which is not recommended.

4.3 **IMPLEMENTATION**

The building method for the breakwaters will consist of two main parts: renovation and extension. First the current state of the breakwaters needs to be improved. If the breakwaters are left unrepaired, the breakwaters will deteriorate more and more until they stop functioning properly or even completely collapse. After the improvements, the breakwaters can be extended. For this, several building methods are possible. In appendix H these different methods and the justification for the chosen methods are further explained. In the following paragraphs, the methods for renovation and extension that will be used are described further.

RENOVATION

The additional placement renovation method will be used. This type of renovation builds on top of the current state of the breakwaters. No tetrapods are removed. One layer of heavier tetrapods, which meet the draft design as close as possible, is placed on top of the existing outer layer and at the toe. Since most tetrapods in the current situation have shifted, this method can quickly fill up any gaps as well as create a decent double layer of tetrapod armour. The height of the breakwaters cannot be increased by increasing the thickness of the core. Instead, a sea wall is placed to reach to the required height.

EXTENSION

The *Modulated Building Method* will be used for the extension. This method divides the total extension into several modules. At the end of each module, a temporary head will be constructed. This means that the temporary end of the breakwaters will be protected with an armour layer. Since the breakwaters are build up in sections, there is less exposure of the layers beneath the armour layer to the incoming waves. The method also allows for construction in a slower pace, if required.

4.4 FINAL DESIGN

The final design combines the renovation method and the extension method. Table 4-8 and Table 4-9 give an overview of the final design. In Appendix I, the final design is explained in more detail giving the design value for each element in both the renovation part and the extension part of the breakwaters. An overview of the locations of the tetrapods is given in Figure 4-6. Figure I-1. In appendix I some technical drawings of the final design are shown.

LENGTH

A determination for the length of the breakwater is made by using the non-parallel accretion method. This method is used as the slope of the coast is very steep and continues to great depths. Several assumptions have been made in order to get insights in the lengths. Due to these rough assumptions the lengths of the breakwaters can be seen as an indication length instead of the length it has to be. To get a more detailed length calculation, a model is required as the coastal system around the breakwaters is very complex. The indication for the breakwaters is an extension of:

- 55 m with respect to the current situation for the western breakwater;
- 205 m with respect to the current situation for the eastern breakwater.

It has to be noted that these are just indications and further modelling is needed to get more insights. The total calculation can be read in Appendix F.1.2.

HEIGHT

The renovated section of the breakwaters will be +6.0 m LWS with a seawall to increase the height. The extension will increase in height from the renovated part at +6.0 m LWS up to +8.5 m LWS. The seawall will extend up the extension as well, remaining at a fixed height until the crest height has been reached. Figure 4-5 gives a longitudinal sketch of the western breakwater for clarification. Another option would have been to keep the extension at a crest height of +6.0 m LWS with a seawall. However, placing a seawall is an alternative method and sub-optimal solution with less certainty about the probability of failure. Therefore the 1/250 year design height is used for the extension.

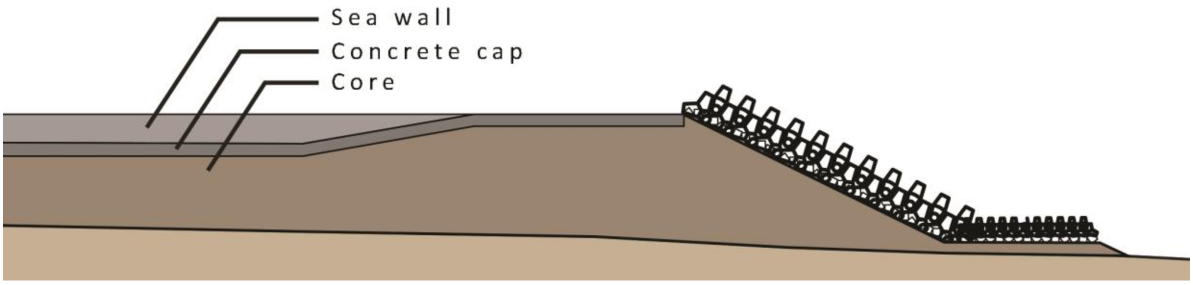


FIGURE 4-5 SKETCH OF POSITION SEAWALL

TETRAPOD WEIGHT

The maximum tetrapod size is assumed to be 18 tons. As explained by the construction supervisor, [E| Balai P.W. 1] the fabrication of 18 tons tetrapods is already considered difficult. Therefore, heavier tetrapods should not be used in the design. If the fabrication method is adjusted or the tetrapods are brought in from a different location, heavier tetrapods might be possible. If so, one should strive for the tetrapod weights determined for the 1/250 year design.

CORE AND FIRST UNDER-LAYER

These layers remain the same for the renovation part as they are not replaced. The extension will consist of the 1/250 year design values.

TABLE 4-8 CHARACTERISTICS OF THE EASTERN BREAKWATER

		Current situation	**Draft design 1/250 year**	**Final design**
Length [m]		180 m	370	370
			Renovation	
Height [m]		6.0 + 2 m seawall	8.5	6.0 + 2 m seawall
Armour layer outer [t]		9.0	20.0	18.0
Armour layer inner [t]		7.0	7.0	7.0
First under-layer [t]		Unidentifiable	1.0 - 2.0	Unidentifiable
Core [t]		0.1- 0.5	0.5 - 1.0	0.1 - 0.5
Toe berm outer (0 – 8 m)	**Weight [t]**	3.5	18.0	18.0
	Length [m]	15	10	10
Toe berm inner (0 – 3 m)	**Weight [t]**	3.5	3.5	3.5
	Length [m]	15	15	15
			Extension	
Height [m]		-	8.5	8.5
Armour layer outer [t]		-	20.0	18.0
Armour layer inner [t]		-	7.0	7.0
Armour layer head [t]		-	25.0	18.0
First under-layer [t]		-	1.0 - 2.0	1.0 - 2.0
Core [t]		-	0.5 - 1.0	0.5 - 1.0
Toe berm outer (8-12 m)	**Weight [t]**	-	8.0	8.0
	Length [m]	-	7.5	7.5
Toe berm outer (12 m – end)	**Weight [t]**	-	3.0	3.0
	Length [m]	-	5	5
Toe berm inner (3 – 12 m)	**Weight [t]**	-	8.0	8.0
	Length [m]	-	7.5	7.5
Toe berm inner (12 – end m)	**Weight [t]**	-	3.0	3.0
	Length [m]	-	5	5

TABLE 4-9 CHARACTERISTICS OF WESTERN BREAKWATER

		Current situation	Draft design 1/250 year	Final design
Length [m]		215	272	272
			Renovation	
Height [m]		6.0	8.5	6.0 + 2.5 m seawall
Armour layer outer [t]		9.0	20.0	18.0
Armour layer inner [t]		7.0	7.0	7.0
First under-layer [t]		Unidentifiable	1.0 - 2.0	Unidentifiable
Core [t]		0.1 - 0.5	0.5 - 1.0	0.1 - 0.5
Toe berm outer (0 – 5 m)	**Weight [t]**	3.5	18.0	18.0
	Length [m]	15	10	10
Toe berm inner (0 – 5 m)	**Weight [t]**	3.5	3.5	3.5
	Length [m]	15	15	15
			Extension	
Height [m]		-	8.5	8.5
Armour layer outer [t]		-	20.0	18.0
Armour layer inner [t]		-	7.0	7.0
Armour layer head [t]		-	25.0	18.0
First under-layer [t]		-	1.0 - 2.0	1.0 - 2.0
Core [t]		-	0.5 - 1.0	0.5 - 1.0
Toe berm outer (5 – 8 m)	**Weight [t]**	-	18.0	18.0
	Length [m]	-	10	10
Toe berm outer (8 m – end)	**Weight [t]**	-	8.0	8.0
	Length [m]	-	7.5	7.5
Toe berm inner (5 – end)	**Weight [t]**	-	8.0	8.0
	Length [m]	-	7.5	7.5

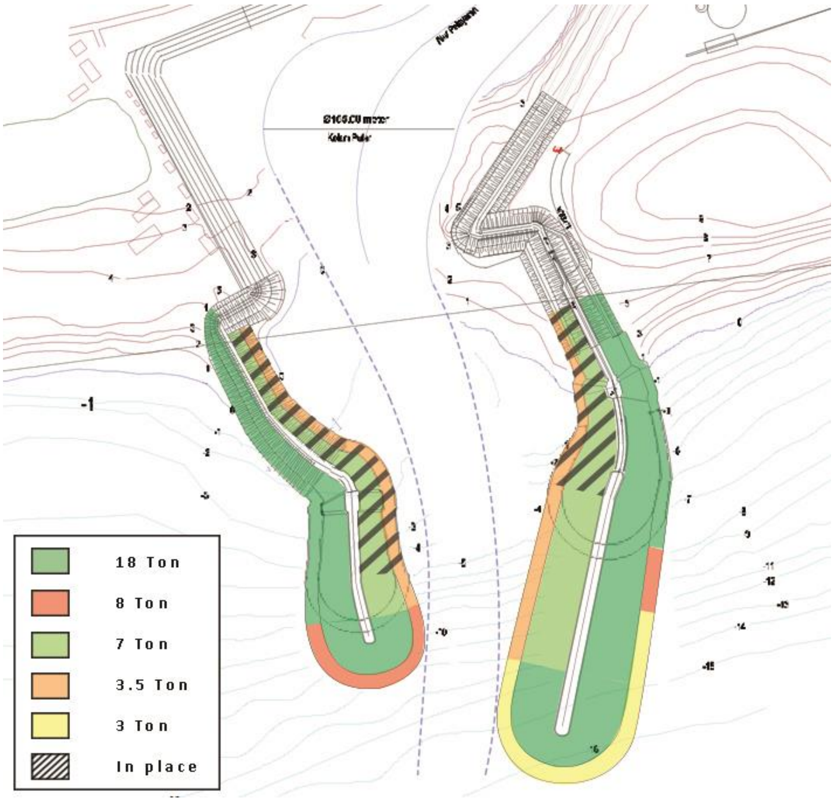


FIGURE 4-6 FINAL BREAKWATER DESIGN

4.5 CONSTRUCTION PLANNING

The construction of the breakwaters consist of two parts: a renovation of the existing breakwaters and an extension. Roughly speaking, the construction can be completed in the following order:

- Renovation of the toe
- Renovation of the armour layer
- Extension of the eastern breakwater
- Extension of the western breakwater
- Construction of seawall

Before starting the renovation of the toe one should prepare the construction area and the preparation of the storage areas at both sides of the river. It is recommended to first renovate the toe and the first underlayer of the breakwaters in order to have a solid base for equipment necessary for extension of the breakwaters. In the choice of what breakwater to renovate in which time period, one should take into account season-dependent wave incidence. In the dry season one should aim to focus on the western breakwater, since waves attacks are mostly on the eastern breakwater. Likewise one should focus on the eastern breakwater in the wet season.

After finishing the renovation of the current breakwaters the construction of the extension can start. In order to determine what breakwater to extend in which time period extension one should consider not only wave attack but also longshore sediment transport. In the dry season, wave attacks are on the eastern breakwater and therefore one should work on the western breakwater, however when it comes to longshore sediment transport one should do exactly the reverse, since more sediment will accumulate in the navigation channel as the western breakwater elongates. It is therefore, regarding sediments, preferred to extend the eastern breakwater instead of the western breakwater in the dry season. This decision depends therefore on economic consequences of dredging of the accumulated sediment and damage of wave attacks during construction.

The construction of the extension of the breakwaters can be divided in modules. A spare temporary head construction has to be available all the time at the storage area in order to finish a module within a limited time frame. Except the first module, all modules start with the (partly) removal of the temporary head. After this the breakwaters will be constructed as described in chapter 2.5. The time duration of a single module is not fixed. The decision to finish a module can be for instance based on the weather prospects, change of season or management reasons. In the extension it is recommended to start with the construction of the eastern breakwater to decrease the amount of sediment that will settle in the navigation channel. An example building schedule is given in appendix L.

BUILDING MATERIALS

An estimation of a number of building materials is shown in Table 4-10. This estimation is made by using averaged cross sections and average depths. This makes Table 4-10 more an indicator of the amount of material than exact values.

TABLE 4-10 BUILDING MATERIALS

	Renovation phase	
18 t tetrapods	1.900	Pieces
Seawall	215	m
	Construction	
0.5 - 1.0 t core material	185.000	m^3
1.0 - 2.0 t under layer material	45.000	m^3
18.0 t tetrapods	5.500	Pieces
8.0 t tetrapods	1.100	Pieces
7.0 t tetrapods	6.000	Pieces
3.0 t tetrapods	2.000	Pieces

4.6 EVALUATION

In the transformation of the draft design into a final design, the present actual design limits had to be taken into account. The impact of those limits on the final design had to be examined carefully in order to complete the design cycle. In Figure 4- Project Yogya's schematization of the design process is illustrated.

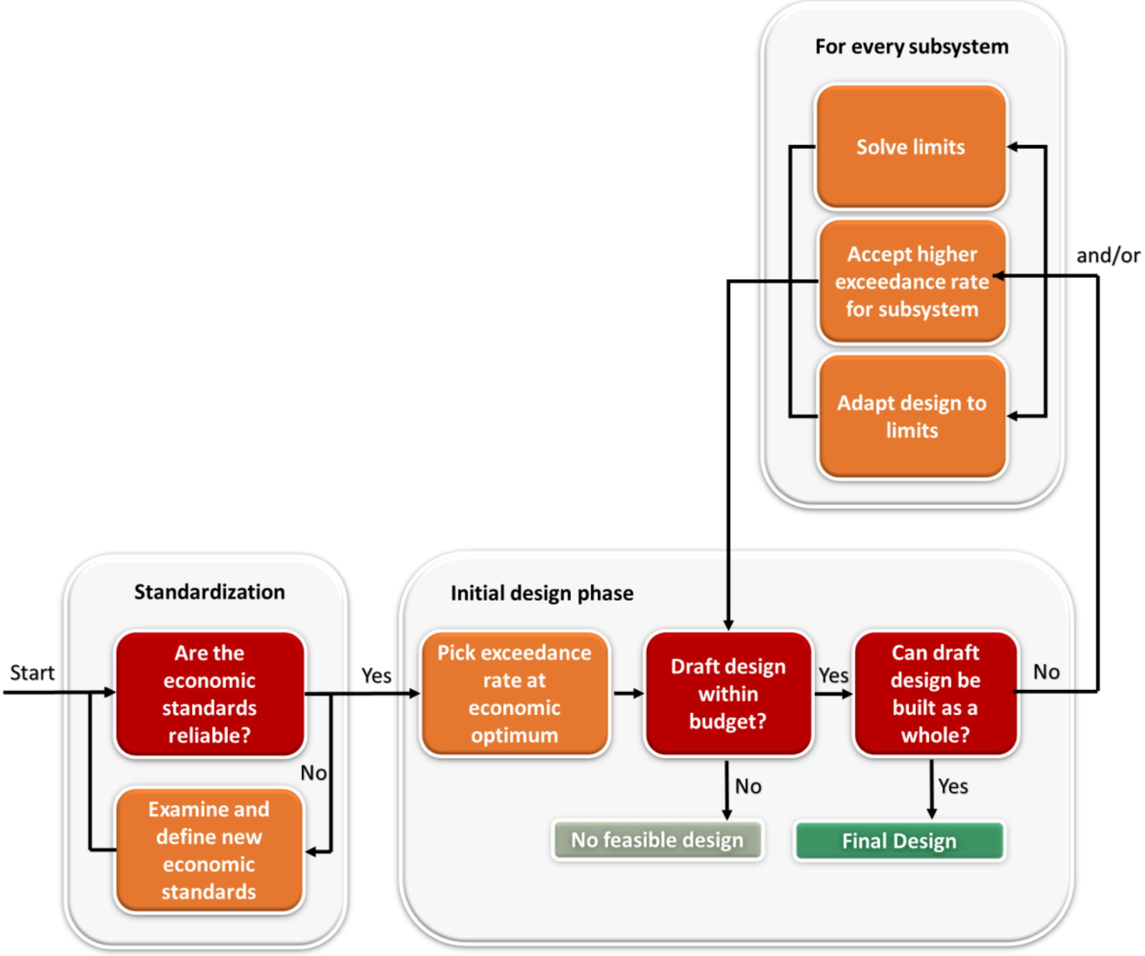


FIGURE 4-7 CONCEPTUAL DESIGN PROCESS

After finishing the draft design, the process of going from the draft design to the final design was characterized by adapting to present construction limits. Those forced limits consists of two types: boundaries due to fabrication and boundaries posed by the existence of the current breakwaters.

According to the investigators, there are three possible solution strategies for issues caused by the observed limits. The first is removing the limit, for example by means of innovative research. Another type of strategy is to alter the design such that with the existing limits the same allowable probability of failure can be obtained. The last strategy is to accept the additional risk of failure and to rely more on maintenance.

Project Yogya's translation to the final design adopted the last strategy. For example, elements that required tetrapods heavier than 18.0 tons in the draft design were adopted with 18.0 tons tetrapods in the final design. The risk of more damage is accepted, resulting in higher maintenance costs. However, one should consider that a limit as such may not only have a local effect, it may influence the probability of failure of other parts of the structure as well. When this is the case, other parts may be over-dimensioned. The limits can potentially be best characterized by the phrase: "a chain is only as strong as its weakest link".

Therefore, it is wise to divide the system into subsystems that act as one chain. For every subsystem a solution strategy can be chosen in order to optimize the design. Again, this new design should fit within the budget or a different solution strategy should be chosen.

FURTHER RESEARCH

Further research should be done on several topics which are given in Table 4-11. These topics will provide a better view on which solution strategy is the best option for each subsystem when going from the draft design towards the final design. In other words, these research topics will lead to a more accurate and valuable design process. Besides these topics, general improvements are proposed in order to optimize the whole design process.

TABLE 4-11 FURTHER RESEARCH TOPICS

Topic	Determination approach
Tetrapod fabrication	One should either look for new fabrication methods or find a new manufacturer. When heavier tetrapods become available this limit is stretched, making it possible to realize the final design closer to the draft design.
Sea wall	The necessary height of the structure can easily be achieved when placing a sea wall. However, a sea wall will have a different effect on the rest of the structure in comparison with an increase of the height of the entire structure.
Slope of the armour layer	The stability of armour units increases for less steep slopes. This is more complex for tetrapods, so further research should be conducted to find an optimal slope which leads to the most economical and feasible design.
Armour layer placement	The effectiveness of placing a new layer of tetrapods on top of a shifted layer of tetrapods of a different weight should be further researched.
Set standards	Using an Indonesian fixed standard, a future design will cost less and become more efficient. It is therefore recommended to document every design step and research that is made in this progress, in order to improve future construction of breakwaters.
Modelling	By using model-based reasearch, an economic consideration or optimization is more accurate. For such models detailed data is crucial and is thus necessary to establish a data collection program or such to support that level of detail.
Toe design	The design of the toe berm is based on the wave height above the toe. An alternative way to determine the toe berm design is to base the required dimensions of tetrapods of the toe on the weight of the armour units on the slope which it has to support. This could lead to very different results.
Tetrapod abrasion	The abrasion of the tetrapods will have an effect on the durability and quite possibly the strength of the tetrapods.

5. SUSTAINABLE COASTLINE GOVERNANCE

This chapter will focus on potential coastline governance strategies which are applicable to the preservation of the coastline adjacent to the Glagah breakwaters. In order to come up with potential strategies, in the synthesis several applicable coastal protection solutions are considered and outweighed.

With the selection of the most applicable solutions, coastline governance strategies are devised in which the solutions are implemented at the location. These strategies are then discussed in the following evaluation and the chapter finishes by mentioning topics for further research. The following question should be answered in this chapter:

What sustainable preservation measures can be recommended to prevent coastal erosion adjacent to the breakwaters?

This question should be answered by going through the process mentioned above.

5.1 COASTAL MANAGEMENT STRATEGIES

Preservation of the coastline is obtained if coastline stability is secured over time, which again is strongly dependent on proper coastal management. Therefore, in order to create a stable coastline, a well-defined coastal management plan is required. Management strategies can be divided into three categories: retreat, accommodate and protect.

Applying a retreat management strategy will result in unrestrained erosion of the coast west of the port. Since many structures, houses and economically strong agricultural areas are situated very close to the shoreline, a solution from this category is not considered as an option.

An accommodation strategy would mean to stop affecting sediment transport at all. As the current breakwaters already block part of that flow, a design that incorporates such a strategy implies changing the whole type of breakwater that is already applied. This is not considered as an option. The type of accommodation that may be included in the design is are principles from *building with nature.* Project Yogya aims to implement solutions that merge with and reflect the local ecosystems.

By deduction, for a stable coastal design for Glagah beach, one must seek a management strategy for the protection.

5.2 COASTAL PROTECTION SOLUTIONS

In this section multiple protection solutions to solve the coastal instability of the adjacent coast of the harbour will be discussed. A distinction between hard, soft and mechanical solutions is made and then the possible solutions are divided over these categories. Next, the solutions are weighted against each other. In Table 5-1 a brief overview of this comparison is showed, based on the research of Appendix J.

TABLE 5-1 COMPARISON COASTAL EROSION SOLUTIONS

Solutions	**Advantage**	**Disadvantage**
	Hard solutions	
Sea Wall	▪ Relatively easy to construct ▪ Directly effective after construction	▪ Scouring hole could lead to failure of sea wall ▪ Aesthetic value ▪ Shifts the erosion problem elsewhere
Revetments	▪ Relatively easy to construct ▪ Directly effective after construction	▪ Erosion could lead to cracks and failure ▪ Touristic function of the beach is lost completely ▪ Aesthetic value ▪ Shifts the erosion problem elsewhere
Groynes	▪ Directly effective after construction ▪ Trustworthy construction	▪ Aesthetic value ▪ Shifts the erosion problem elsewhere ▪ No storm defence
Breakwaters	▪ Directly effective after construction	▪ Shifts the erosion problem elsewhere ▪ Aesthetic value
	Soft solutions	
Mangrove	▪ *Green* solution ▪ Wave dampening ▪ Improves aquacultural potential	▪ Vulnerable in the beginning phase ▪ Large forest is required ▪ Long time to grow ▪ Shifts the erosion problem elsewhere
Grasses	▪ *Green* solution ▪ Relative fast solution ▪ Cost-effective solution	▪ Vulnerable in the starting phase ▪ Shifts the erosion problem elsewhere
Nourishment	▪ Solution for the sediment in the channel and basin ▪ Solves the local erosion problem	▪ Information about dredging is required ▪ No storm defence
	Mechanical solutions	
Bypass system	▪ Solves the local erosion problem	▪ A lot of maintenance required ▪ Relatively expensive ▪ No storm defence

From the comparison, several conclusions can be found. Firstly, it is concluded that the possibility of failure for the hard solutions like a sea wall or a revetment is assumed to be too large. Secondly, emerged breakwaters are too expensive compared to groynes due to the steep bathymetry and the fact that submerged breakwaters have proven not to always meet their design criteria or even to show reverse effects on erosion. Therefore, these structures will not be investigated further in this report. Finally, the by-pass-system is assumed to be too expensive and requires too much maintenance to function as a realistic solution in this project. Consequently, only groynes and the three soft solutions are considered as proper solutions in this report.

The fair comparison between applying groynes, a mangrove forest, grasses or nourishments is not possible as none of the solutions can be considered as a total solution on their own. In can be concluded that a total solution should consist of planting a mangrove forest or grasses with applying nourishments on a frequent base, possibly combined with groynes. In this project, grasses are chosen instead of a mangrove forest because mangrove trees take a long time to grow. Also, at the project location the available space is not sufficient enough for an effective mangrove forest.

5.3 IMPLEMENTATION OF COASTAL PROTECTION SOLUTIONS

Before variants of the solution can be designed, the type of grass and the options in nourishment should be chosen. Table 5-2 shows an overview of the choices made for the type of grass and nourishment. More detailed information about these choses can be found in Appendix J.

TABLE 5-2 GRASS TYPE AND NOURISHMENT TYPE

Element	**Preferred characteristics**	**Decision**
Grass type	▪ Vegetates in salt and brackish environment ▪ Long roots ▪ Native species	Vetiver grass
Nourishment type	▪ Cost effective ▪ Less disturbance of the beach	Shoreface nourishment

As already stated in the previous section, it can be concluded that a total solution should consist of planting vetiver grass in conjunction with applying shoreface nourishments, possibly combined with groynes. Based on these criteria, two main solutions have been designed.

- Option 1: 'Heal the Spot'
- Option 2: 'The Groyne Shift'

In the following paragraphs, these two options will be discussed further.

OPTION 1: 'HEAL THE SPOT'

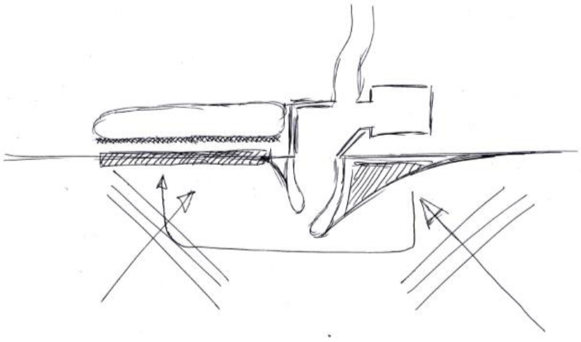


FIGURE 5-1 COASTAL EROSION OPTION: 'HEAL THE SPOT'

In the design of option 1 the proposal is made to apply vetiver grass as shore protection combined with placements of shoreface nourishments. The vetiver grass will be planted above the intertidal zone on the coast that separates the lagoon from the sea. The location is hatched in Figure 5-1 by the line of crosses directly below the lagoon. This zone will create a natural storm defence as the long roots of the grass will be able to keep the sand trapped in between. The beach itself will still be exposed to the sediment transport induced by storm events. Temporary shoreline setback by storm events is not a problem because during calm circumstances the cross-shore transport will recover the beach itself to its equilibrium state. However, the beach from the sea up to the grass zone should be wide enough to offer sufficient accommodation space for these variations.

In order to keep this the zone west of the breakwaters in balance, one has to place shoreface nourishments. The nourishments will be placed at the shoreface in front of the lagoon, which is showed in Figure 5-1 with the hatched block at the western side of the breakwaters. The volume of sediment of the nourishments will be distributed over the whole beach profile by the cross-shore transport. This volume should be sufficient enough to compensate the volume of the net longshore sediment transport. The required volume of sediment for this nourishments is available by dredging of the navigation channel and harbour basin and the accumulation zone east of the breakwaters, indicated by the hatched block at the eastern side of the breakwaters.

OPTION 2: 'THE GROYNE SHIFT'

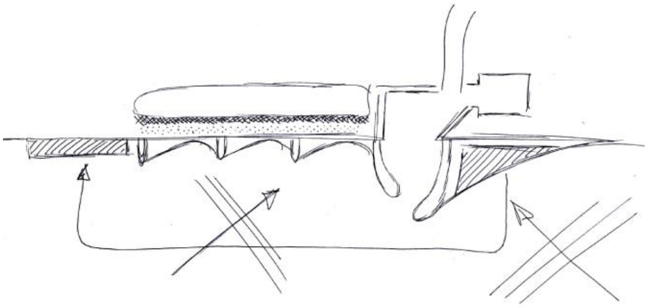


FIGURE 5-2 COASTAL EROSION SOLUTION 'THE GROYNE SHIFT'

The design of option 2 is based on the same mindset of option 1, except that a groyne field is added. The main reason for constructing an extra groyne field is increasing safety. Nourishments are designed to compensate the net longshore sediment transport, however the required volume is based on the maximum annual net longshore transport in order to compensate possible extreme annual circumstances. Still, it is possible that over a certain timeframe the longshore transport has increased, resulting in a decrease of the nourishment lifespan. Theoretically, it could happen that over certain periods of time the coast will be eroded again and in combination with a severe storm the already dubious width of the coast including the grass barrier might not be sufficient enough. By constructing a groyne field over the full length of the coast beneath the lagoon, the erosion spot will be fully shifted towards the west. This will ensure that the small coastal width will remain constant and is only exposed to the impact of storms. Assuming that the width is sufficient at the moment to ensure the required accommodation zone, the coast will restore itself from storm impact by the cross-shore transport. Besides that, due to the fixation of the beach by the groynes, it offers the opportunity to apply a revetment to upgrade the storm defence in case the grass barrier would not suffice. The groynes will remove the possibility for the varying longshore sediment transport to erode the coast in such a way that the required toe construction of, for example, a revetment would be destroyed, resulting in failure of the whole structure. Therefore, with the use of groynes, the safety of the lagoon can be guaranteed in any scenario.

At the new erosion spot west of the groyne field, shoreface nourishments have to be applied again in order to stabilise the coast at that location. The main advantage of this solution is the fact that the new erosion spot can be located at the most favourable location. This offers the opportunity to choose a location with less economic value, resulting in a decreased risk of economic losses.

5.4 **EVALUATION**

After testing the technical feasibility of option 1 and 2, the choice for the best solution should rely on a consideration of costs and benefits. This choice should be based on the ratio between costs of the project and the benefits that should be gained by the recreational potential of the lagoon and the western beach during the project life span.

The main difference between both options is the groyne field. These groynes require an initial construction budget and need to be maintained during their lifetime. This implies that option 2 will be more expensive than option 1. However, sometimes it is more economically beneficial to reduce the risk of damage than accepting the damage itself. If the recreational capacity of the lagoon and the beach will develop up to a certain level, that damage to this region will cost more than constructing a groyne field that will locally decrease the risk of damage, then it could be wise to construct the groynes and shift the risk of erosion. Unfortunately, the shift that will reduce the risk of damage to an enlarged lagoon and an enlarged beach, also has a disadvantage. Namely, one should also consider that shifting the erosion problem will not solve the erosion problem as downdrift areas will be exposed to this erosion. Locally, the risk of damage will increase, resulting in decreasing economic potential of that region. Therefore, in the consideration between the both options, one should calculate the variation in economic potential of the regions by applying a groyne field. Next, these associated benefits should be weighed against the total costs of a groyne field. Based on this outcomes, it can finally be stated whether or not an increase in safety is worth the additional costs of a groyne field. This cost-benefit analysis is out of the scope of Project Yogya.

As a final side note, besides this theoretical approach of the consideration, it should also be noticed that required maintenance is sometimes not carried out correctly or even not at all in Indonesia. Although groynes also demand maintenance, the impact on the coast, if no maintenance is applied is less than not placing nourishments. Therefore, the groynes will offer a more robust option in a worst case scenario. Hence, this should be included when considering these two options.

FURTHER RESEARCH

In paragraph 5.3 two coastal governance options are discussed based on a total solution consisting of planting grasses together with applying nourishments on a frequent basis, possibly combined with groynes. Combining these options, the two main solutions have been found. Nowadays it might be possible to introduce an optimization in the nourishment part resulting in a whole new concept. Nevertheless it has to be stated that an example of such an innovation, goes along with large uncertainties for now. Therefore one should be very careful in its implementation as the concept is explained in a very simplistic way including a lot of assumptions.

Frequent nourishments will disturb the beach and decrease its recreational value in a certain way. Therefore it is preferred to find a coastal governance solution with less or even no disturbances. Two options are available that comply with the preferences. First, one could decrease the frequency of the nourishments by placing larger volumes per time, thereby decreasing the disturbance of the beach. Second, if one should be able to create a sediment flow against the direction of the net longshore sediment transport, it will remove the disturbance of the beach in all. Fortunately, both required design characteristics are already present in a Dutch design, situated at the coast of Holland. An innovation that is commonly referred to as the *Dutch Sand Engine*. In Figure 5-3 the functioning of the design is showed.

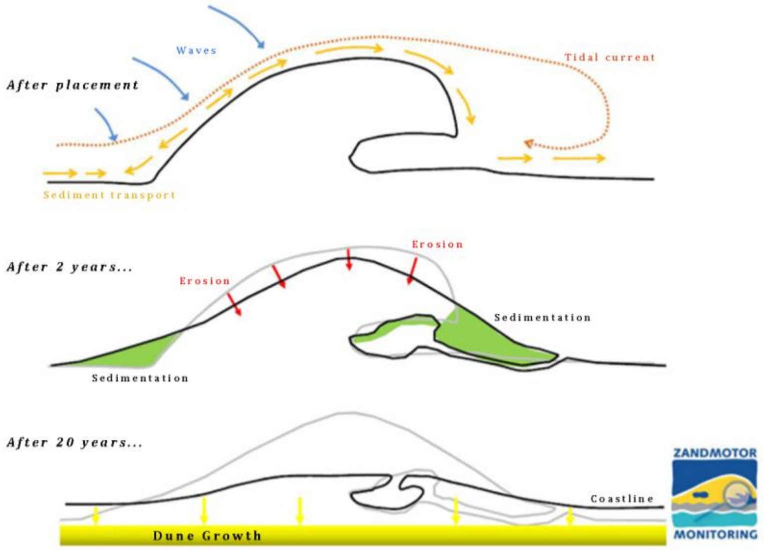


FIGURE 5-3 POSSIBLE CONSEQUENCES ON THE COASTLINE BY A WELL DESIGNED 'SAND ENGINE'

The idea behind the *Dutch Sand Engine* is to make use of the force of nature to distribute the sand over the beach in order to counteract the structural erosion of the Dutch coast. This implies that the nourishments normally applied over the years will now be deposited at one location directly at the start of the project, forming an enormous supply of sand for the longshore sediment transport. The wave driven sediment transport will distribute the sediment over the coast during the lifespan of the *Dutch Sand Engine,* resulting in a decrease of maintenance costs. Figure 5-3 shows that the typical shape of the design of the *Dutch Sand Engine* in conjunction with the local characteristics of the tide will create sediment transport in two directions. To the right, driven by the net wave direction, the main longshore sediment transport will occur. This sediment transport is intended to counteract the structural erosion. However, due to the shape and the tide, a secondary circulation pattern will occur resulting in a smaller sediment transport towards the left.

The main advantage of this solution is the reduction of the maintenance costs required to execute the nourishments since nature will distribute the sediment itself. Another advantage has an ecological basis. Due to the accompanying turbidity during nourishments, the marine life at the beach is destroyed. The ecological system demands at least two years to rehabilitate. Although a sand engine also requires nourishments for construction, the nourishment is applied once in a couple of years. Therefore, marine life has the possibility to develop itself in between the consecutive nourishments resulting in an increased ecological value of the project.

The Dutch design includes a decrease in the frequency of the nourishments by placing an enormous volume of sediment once. It also creates a sediment flow against the direction of the net longshore sediment transport direction due to the secondary circulation pattern. If possible, it may be an option to design a sand engine that is able to compensate the local erosion at Glagah beach by a sediment transport as a result of a secondary circulation pattern. As explained in 0, the indications of structural erosion of the South-Javanese coast could be solved by the main sediment transport. A certain solution could be able to solve two serious threats at once.

Again, it has to be stated that the *Dutch Sand Engine* is specifically designed for the Dutch coast in conjunction with the local characteristics of the tide. This resulted in the chosen typical shape and the initial sediment volume. It must be clear that without intensive research in advance it is not possible to propose such a solution at other coasts. The principle of the *Dutch Sand Engine* is nowadays the most technologically advanced solution for coastal governance and up to now it is still in the test phase and only tested at the Dutch coast. Nevertheless, this first test showed great results and in the future it may be an applicable option that can be applied at other coasts. Besides that, it has to be mentioned that the first tests are applied at a trailing edge coast. Now, the question rises if this principle could also be applied at leading edge coast like the South-Javanese coast.

As a side note the following considerations, the investigators stress that research is required to find an applicable and well-functioning sand engine at the South-Javanese coast. To execute a sand engine project, a large amount of sediment is demanded since only a small part of the required volume acts as compensation for the net longshore sediment transport. The other part is distributed along the down drift side, counteracting the indicated structural erosion of the South-Javanese coast. This implicates that applying a sand engine not only satisfies the project requirements, it also serves a larger goal, which should be considered in the project budgeting. For commencement, the initial required volume of sediment should be dredged or supplied from other locations. After the first lifespan, the next required volume might partly be dredged from the accumulation side of the breakwaters. This annual dredged volume can be stored somewhere and upon completion of the first lifespan, the total stored volume can be used to construct its second generation.

6. POLICY RECOMMENDATIONS

Following the assessment phase of Part I, it was concluded that the progress of the Glagah Jetty Project is heavily impeded by policy- and governance-related issues. As Project Yogya is aimed towards finding an integral design for a safe Tanjung Adikarto Harbour entrance, it was determined that this includes an analysis of how the design can be best implemented considering the current work methods of the governing bodies. This led the investigators to ask the sub-question:

What governance and policy changes can be recommended for an effective integral approach to the Glagah Jetty and Tanjung Adikarto Projects?

This chapter discusses a series of recommendations for the issues outlined previously (Ch. 3). In short, a solution strategy has been devised for the following issues:

- The reason for the construction stop is disputed by the involved parties.
- Data collection and sharing is inefficient due to collaboration issues and missing data.
- A lack of knowledge, single-year budgets and sub-optimal supervision leads to poor execution.
- The dredging policy is inadequate due to KPI constraints, socialization responsibility and execution issues.

In addition, this chapter outlines a few general observations from an outsiders perspective and concludes with opportunities that have been identified in the analysis process.

This section uses many references to the stakeholder meetings that are fully described in Appendix E.

6.1 SOLUTION FOR BREAKWATER CONSTRUCTION STOP

A clear distinction in responsibility has to be made between Min. P.W. and Min. M.A.F. that decomposes responsibility for the following sub-issues.

- **Sub-issue: completion of the breakwater construction;**

  A single Ministry should take full responsibility for the lengthening of the breakwaters.

Recommended Strategy:

  - Since the Min. P.W. has designed and built the breakwaters so far, it possesses the required planning capabilities and budget management to a higher extent than Min. M.A.F.
  - A Memorandum of Understanding (MOU) effectively transfers the responsibility and accountability on a national level, allowing for further agreements on lower-level governmental institutions.

- **Sub-issue: short term post-construction maintenance including initial adaptations and repair works versus long term maintenance of the breakwaters for durability;**
  A Memorandum of Detail (MOD) should be established in close partnership. A logical decomposition is provided below.

MOD

MOD decomposes responsibilities on a detailed level

Prov. M.A.F. Balai P.W.

Recommended Strategy:

- As Balai P.W. will undertake the actual construction of the breakwaters, new regulation should be established that addresses the maintenance responsibilities afterwards.
- Make Balai P.W. responsible for short term post-construction maintenance including the initial adaptations and repair works to the breakwaters.
- Allocate a budget for these maintenance and repair costs as part of a multi-year program.
- Long term maintenance is different from post-construction maintenance and its costs should be borne by the harbour master. As the completed breakwaters will enable harbour development, the anticipated tax revenue streams will enable cost bearing by Prov. M.A.F. for maintenance of the breakwaters as part of the safety of the harbour navigation channel.
- Instead of Prov. M.A.F., it is recommended to make the harbour master responsible for the dredging works. The UPTD will soon act as harbour master and upon economic viability the profit-oriented BUMD can guarantee maintenance of the breakwaters for the harbour users.
- This decomposition should be clearly communicated to Min. P.W. so it provides proof of feasibility and durability to Min. Finance and Nat. Bappeda.

- **Long-term goal for improvement:**
  Institutional strengthening of cooperation between all relevant governmental stakeholders.

Institutional strengthening of cooperation

- In the current situation, knowledge disparity between cooperating governmental institutions is sizeable.
- Knowledge sharing is key in the development of effective contracts that enable durable solutions.
- It is recommended to actively engage in knowledge-sharing sessions and report sharing. The current meetings at the Bappeda are a good start and follow-up meetings should be initiated.

6.2 SOLUTION FOR LACK OF KNOWLEDGE COLLECTION AND SHARING

Solving a problem for which no single entity can be held accountable or responsible is a great challenge. However, the investigators highly stress the necessity of a change in culture. A radical solution stimulates a more pro-active culture that enables solutions to the lack of data collection and sharing.

- **Sub-issue: collaboration issues, lack of front-end engineering guidelines and insufficient wave data**
  A consortium of researchers and representatives can serve as the neutral and open knowledge platform for the purpose of sharing data and statistics.

Establishment of a research consortium

Recommended Strategy:

- Budget allocation by national level for research institution, public works or general budget. The project can be considered important since it is national prestige.
- Organizational structure comparable to that of the *Water Council*: involvement of both private and public institutions. The research consortium should be established, presided over and facilitated by a neutral party. Suggestions include:
  - BPPT: A fully owned public company with the sole purpose of providing research insights for governmental institutions. According to stakeholder findings, BPPT is happy to establish and lead this research consortium [BPPT]. This is confirmed by other parties, for example Balai P.W. Reflecting on the stakeholder relationships, the BPPT would be an excellent host for this research consortium.
  - UGM: As lead consultant to Balai P.W., the Engineering Research Centre consultants have high authority and the ability to continue smoothly with more research involvement.
  - Balai P.W.: As construction planner, it is in Balai P.W.’s best interest to gather extensive information about the great challenges of the Glagah Jetty Project.
- Strengthened by private-public collaboration, the consortium has more power to engage in lobbying sessions for the acquisition or gathering of new, reliable data.
- BPPT’s role: Physical models and numerical models, verified by data from acquisition by the consortium.
- UGM’s role: Research for academic progress. The research consortium can be promoted to a great entity that provides student internships. Long-term results may include technolical advances that can be applied along the entire Southern Coast.
- Balai P.W.’s role: Expert representatives engage in active sessions about construction issues and design difficulties.
- Governmental institutions’ role on a national level: standardization research and data collection. Front-end engineering guidelines can directly be shared among the representatives in the research consortium.
- Short-term goal: acquisition or gathering of wind and wave data relevant to the design criteria for the new breakwater constructions.

- **Long-term goal for improvement:**
  The research centre would be greatly aided with national influence on the debate of institutionalization of data and statistics gathering

Institutionalization of data and statistics gathering

Recommended Strategy:

- Pro-active policy regarding data and statistics gathering. This could even be executed at already existing research institutions. Attempts for the institutionalization of data and statistics gathering have been made by the lead consultant [Int. Pres.].
- Initial focus on wave and wind data for the Glagah Jetty.

6.3 SOLUTION FOR LACK OF SAFE, DURABLE AND SUSTAINABLE MAINTENANCE DREDGING

A general solution is presented that should resolve issues related to the dredging works.

- **Sub-issues: KPI constraints, socialization responsibility and dredging execution problems.**
  Setup an MOD that enables revision of the Prov. M.A.F. dredging policy.

MOD describes erosion process and solution

Agreement between Prov. M.A.F. **& AUDITORS**

Recommended Strategy:

- Acquire independent, research-based dredging proposals from a to-be setup research consortium and formulate these in an MOD.
- An MOD can serve as a lawful protection against fraud and corruption suspicions from auditors and other governmental parties.
- Accompanied by effective public relation campaigns, this MOD can prove to be effective in explaining the local population about the necessity of nourishment of the coastal erosion zone instead of the sink holes.
- Based on the results from the research proposals and the details of the MOD, upscale the training programs for dredging operators and dredging execution engineers.
- Create a stringent dredging governance regulation and leverage its positive effects on reliability for potential investment in new dredging vessels or materials.

- **Long-term goal for improvement:**
  For future effectiveness and higher reliability, Prov. M.A.F. prefers to acquire or lease a single dredging vessel [Prov. M.A.F. 3]. The investigators highly support this policy intention.

Acquire or lease an upgraded dredging vessel

Fund with revenue streams from harbour master

- Await economic research that potentially justifies investments for the acquisition or lease of upgraded dredging vessels.

6.4 SOLUTION FOR EXECUTION ISSUES

This section presents a set of recommendations for the execution issues.

- **Sub-issues: Little knowledge of integral design, single-year budgets and lenient sub-optimal execution supervision**
  Acquire more in-house knowledge and deploy multi-year programs for large-scale projects. In addition, govern a culture of pro-active supervision at breakwater construction.

More inhouse knowledge &

Multi-year Programs

Recommended Strategy:

- More in-house knowledge about breakwaters at Balai P.W.
- Expert engineers represent Balai P.W. in meetings with research consortium.
- Highly dependent on knowledge from research consortium in general.
- Construct the Glagah Jetty as part of multi-year programs and higher budgets.
- Develop a culture best described as: “Plan the Work and Work the Plan”.
- Very stringent supervision on the construction company throughout the whole construction process.
- Implementation of the modulated building method as described in paragraph 4.3

6.5 GENERAL OBSERVATIONS

This section highlights a series of issues that have a strong basis in Javanese business culture. It should be noted that the investigators realize that solving cultural issues is farfetched. Additionally, it should be stressed that the investigators have considerable bias about these issues and are not educated in the field of cultural change. These issues should be read as interpretations from an outside point of view. There are no fixes for these findings and this section serves as a notion to raise awareness about these issues.

- **Bureaucracy**

The investigators realize that the impact of bureaucracy is justified as it is also an effective fraud prevention method. The trade-off could be re-evaluated on a regular interval.

- **Passive Approach**

Project Yogya promotes a pro-active culture in which people are pro-actively involved with their responsibilities. Pro-active behaviour can be characterized as upmost intent to anticipate issues and intervene in difficult situations.

- **Kulonowun**

The Javanese ‘Kulonowun’ is a cultural phenomenon that can be best described as ‘over apologizing’. Kulonowun is both criticized and embraced on the work floor. The phenomenon creates a culture in which people show their regret for not being able to being able to satisfy one’s expectations.

Project Yogya stresses that incentivizing personnel to be more critical could facilitate a more open culture for discussion. However, the investigators acknowledge the hierarchical work ethics can be effective and admit that the Dutch style of open debates has its inefficiency downsides – let alone its effects on pleasant work atmospheres.

6.6 WINDOW OF OPPORTUNITY

Despite having constructed the current breakwaters, Min. P.W. is aware of the Tanjung Adikarto harbour development paradox and is evaluating the proposition that expectated prospects of economic activity in Kulon Progo justify large investments for the breakwater lengthening. Provided that the harbour can be economically viable, it can be stated that a great window of opportunity presented itself at the time of writing:

- Stakeholder motivation levels are unprecedented [Int. Pres.] [Symposium] [Minister].
- The staff at Min. M.A.F. General Directory responsible for declining the MOU has been replaced [Prov. M.A.F. 2].
    - Additionally, the Minister of Public Works considers a new MOU a realistic scenario under current conditions. [Minister]
- Nat. Bappeda has made available a budget for Glagah Jetty specifically.
    - This budget amounts to 223 Billion Rupiah. [Prov. M.A.F. 2]

- The region as well as the Special Province of Yogyakarta has the opportunity to build a well-functioning harbour in harsh conditions, which would be a unique project in Indonesia. The Glagah Jetty Project is already considered national prestige.

6.7 EVALUATION

A more effective policy is of paramount importance in advancing with harbor development of the Tanjung Adikarto Port. Many stakeholders recognize the opportunities that come with closer cooperation in solving governance issues. Aside from identified cultural aspects of progress impediments, most important issues are the responsibility confusion and the lack of sufficient data. Detailed (wave) data can serve as the groundwork for better execution and maintenance solutions. Most importantly, cooperation in the field of data gathering and knowledge sharing can be highly beneficial for all involved stakeholders. Project Yogya stresses that a neutral research institution such as the BPPT or UGM can have an effective leadership role in this matter.

FURTHER RESEARCH

Further research should be done on several topics which are given in the table blow.

TABLE 6-1 FURTHER RESEARCH TOPICS

Topic	Determination approach
Breakwater Construction Stop	▪ What lobbying strategy that impede the establishment of the MOU and MOD? ▪ What are the costs of the breakwater maintenance? ▪ How can these costs be reduced using alternative execution methods or maintenance frameworks?
Lack of Data Collection and Sharing	▪ Discarded from this research for the fact that it is considered out of scope, more research should be conducted on climate change and its local implications. An important further research question is: ▪ How does climate change the future of the Glagah Jetty and Tanjung Adikarto Port? ▪ What is estimated to be the sea-level rise and what are the resulting consequences on significant wave height?
Lack of safe, durable and sustainable maintenance dredging	The current combined capacity of the dredging vessels is 200 m^3 per hour. It is assumed that this information is provided to Prov. M.A.F. by either Prof. Nur Yuwono or BPPT. Following these statements, it can be calculated that the 150 000 m^3 dredging activities can be completed in 31.35 days when maximum capacity is reached [Prov. M.A.F. 2]. This would not threaten soft launch deadlines for early 2016 and this legitimizes the question: ▪ How can the dredging plans be improved in order to optimize the dredging activities? ▪ Is the governance policy on current dredging activities sufficient? ▪ Stimulate research consortium or consultants to conducts studies on the long-term effects of coastal dynamics at Glagah Jetty.
Execution Issues	Aside from the new modulated building method, innovation in the field of execution, programming and planning can be stimulated. The investigators recommend further research into these fields: ▪ How can planning processes be improved? What lessons can be learned from other Public Works Balai that have been responsible for breakwater construction?
	The Swakelola contract provides a legal method to acquire one's service for critical situations and expires after the critical situation is over. This caused legal issues for Balai P.W. as the consultant had to be acquired through a web of legal structures afterwards. [Start Up] ▪ What are legal contracts aside from Swakelola contracts that can be used to acquire consultancy services for both critical and non-critical situations?

CONCLUSION AND RECOMMENDATIONS

This research presented in this report was conducted with the objective to assess the issues that impede completion of the Tanjung Adikarto Port (Part I) and devise an adequate and comprehensive counter strategy (Part II).

In Part I, it was concluded that the existing breakwaters do not fulfil the objective of creating a safe harbour entrance. The breakwaters are too short, causing waves to break at the harbour entrance. As a consequence, passage is limited to small vessels in a low tide window only. Additionally, the durability of the breakwaters is questioned as the structural integrity of the breakwater was compromised during the construction phase. The placement of the breakwaters has influenced and continues to influence the coastline. On the eastern side of the breakwaters, accretion occurs, whereas the western side of the breakwaters is affected by erosion of an estimated maximum of 350 m in 10 years. Hence, the sand wedge between the lagoon and the beach will erode in its entirety if no measurements are taken. Moreover, observations indicate an overall regression of the shoreline between Cilicap and Parangtritis on the long-term. Also, it was concluded that floods have not been observed around the harbour. Floods that do persevere in the river basin upstream are directly caused by rainfall and river overbank spilling further upstream or when Progo River overflows.

From the assessment phase, it was concluded that the construction stop of the breakwaters and the issues that impede further progress are largely a result of political issues. Firstly, there is no consensus among the high-impact stakeholders about the responsibility of breakwater lengthening and future maintenance. Secondly, there is an institution-wide lack of data and statistics gathering and sharing which resulted in unverified models and design assumptions. Thirdly, maintenance dredging in the navigation channel is neither safe nor durable and sustainable. Lastly, there have been severe issues with execution. Consequently, the political and technical challenges have resulted in uncertainty pertaining to the technical and economic feasibility of the harbour.

In Part II, the strategy for countering the progress-impeding issues was specified based on the findings. Progress can be made by creating an integral design for a safe harbour entrance including durable breakwaters, a sustainable coastline conserving the lagoon area and ensuring its feasibility with an effective implementation governance policy. Pertaining to the breakwaters, a design with a probability of exceedance of the design parameters of 1/250 year is proposed. Calculations showed that a minimum length of 200 m is required for safety of passage. Further elongation should be based on an economic optimum of dredging versus lengthening, but initially it is recommended to reach a length of 370 meter for the eastern breakwater and 272 meter for the western breakwater, both with a height of 8.5 meter and tetrapods of up to 18 tons. Project Yogya proposes two potential strategies for a sustainable coastline going by the names *Heal the spot* and *The groyne shift. In both strategies it is* recommended to apply shoreface nourishments in conjunction with the planting of vetiver grass. Despite seeming identical, the latter strategy proposes the additional construction of groynes in an effort to protect the lagoon better. The choice of strategy should rely on a cost-benefit analysis.

Fundamental policy solutions for resumption of the Glagah Jetty project include an MOU with two MOD agreements for clarity about responsibilities, the establishment of a research consortium, the acquisition of an upgraded dredging vessel and a proposition for more in-house knowledge and multi-year design and engineering programs. An economic feasibility study should be conducted to balance the large investment costs against the estimated benefits of a fully functioning Tanjung Adikarto Port. Should investment costs be justified, then the integral solution can be further specified in order to establish a Tanjung Adikarto Port which incorporates public responsibility, safety, durability and sustainability.

FURTHER RESEARCH

It is strongly recommended to conduct further research on a variety of topics. Some topics immediately improve the design when further researched. The importance of the research is represented by a value between 1 and 5 in which 1 is a topic of high importance.

GENERAL

Do the expected prospects of economic activity in Kulon Progo justify large investment costs for design and policy implementation presented in this study? This is by far the most important question in this project.	1

RESPONSIBILITIES

What lobbying strategy impedes the establishment of the MOU and MOD?	2

DATA

Stimulate research consortium or consultants to conducts studies on the hydraulic data.	1
More detailed wind and wave data is required in order to get a more accurate design.	1
What is estimated to be the sea-level rise and what are the resulting consequences on significant wave height?	5

DREDGING

How can the dredging plans be improved in order to optimize the dredging activities?	4
Is the governance policy on current dredging activities sufficient?	5

EXECUTION

How can planning processes be improved? What lessons can be learned from other Public Works Balai that have been responsible for breakwater construction?	2

BIBLIOGRAPHY

(2015). Retrieved from www.dezandmotor.nl: http://www.dezandmotor.nl

Aryanti, V. (2014). *The Sermo Dam Experience, Indonesia Community Development as Key Success for Dam Project Implementation and Operation.* Ministry of Public Works, Public Works, Housing and Mineral Resources Agency of Yogyakarta Special Region,Universitas Teknologi Yogyakarta.

Asian Development Bank. (2007). *Indonesia: South Java Flood Control Sector Project.* Completion report.

BAB 4. Surbey topografi, bathimetri dan pasang surut. (2013). Yogyakarta.

BAB 5. Review perencanaan pemecah gelombang Glagah. (2013). Yogyakarta.

BBWS Serayu Opak. (2008). *Usulan Multi Years (Konstruksi) Pembuata Pemecah Gelombang Glagah Bagian Timur Di Kabupaten Kulon Progo.* Yogyakarta.

BCEOM. (2005). *South Java Flood Control Sector Project.* Republic of Indonesia Ministry of Public Works Directorate General of Water Resources.

CAPA centre for innovation. (2016, January 5). *Kulon Progo (New Yogyakarta) Airport*. Retrieved from http://centreforaviation.com/profiles/newairports/kulon-progo-new-yogyakarta-airport

CIRIA. (2007). *The rock manual.* London.

dallasamplitudes. (2015, 10). Retrieved from dallasamplitudes.files.wordpress.com

Ecoshape. (2015). *Building with Nature Indonesia ~ Securing Eroding Delta Coastlines.*

EurOtop. (2007). *Wave Overtopping of Sea Defences and Related Structures; Assessment Manual.* Hamburg.

Food and Agriculture Organization of the United Nations. (2008). *Fishing harbour planning,.* Rome.

Hussain, I. (2004). *Pro-poor Intervention Strategies in Irrigated Agriculture in Asia: Issues, linkages, options and pro-poor interventions, Indonesia. Colombo, Sri Lanka: IWMI. (Country report Indonesia).*

Ir. Verhagen. (n.d.). H.J. (Henk Jan) Associate Professor in Hydraulic Engineering.

(2012). *Perbaikan konsep pengelolaan pelabuhan perikanan Tanjung Adikarto.* Yogyakarta.

PIANC. (1997). *Approach channels - A guide for design.*

Pilarczyk, K. W. (1998). *Dikes and Revetments: Design, Maintenance and Safety Assessment.*

Prov. M.A.F. (2012). *Perbaikan konsep pengelolaan pelabuhan perikanan Tanjung Adikarto.* Yogyakarta.

Pustek Kelautan - Universitas Gadjah Mada. (2003). *Laporan Akhir Pembuatan Detail Design Pelabuhan Glagah.* Yogyakarta.

Pustek Kelautan Universitas Gadjah Mada. (2003). *Laporan Akhir Pembuatan Detail Design Pelabuhan Glagah.* Yogyakarta.

Schiereck, G. J., & Verhagen, H. J. (2012). *Introduction to bed bank and shore protection.* Delft: VSSD.

Sihotang, W., Subardjo, P., & Saputro, S. (2014). *JURNAL OSEANOGRAFI, Volume 3.*

Sukarelawanto, E. (2015, Octobre 8). *Sepakbola*. Retrieved Septembre 22, 2015, from Bisnis.com: http://semarang.bisnis.com/read/20150810/1/81037/musim-kemarau

Sutikno. (1988). Geomorphological approach to surficial material evaluation in the serang river basin kulonprogo, yogyakarta, indonesia. *The Indonesian Journal of Geography, 18*(56), 29-42.

U.S. Navy, Naval Weather Service Command . (1976). *U.S. Navy Marine Climate Atlas of the World Vol 3, Indian Ocean.* Washington D.C.: U.S. Government Printing Office.

UGM, P. (2013). *Review Desain Pelabuhan Tanjung Adikarta, BBWS Serayu Opak.* Yogyakarta.

UGM, PSIT. (2013). *Pemodelan Gelombang Perairan Pelabuhan Adikarto Glagah.* Yogyakarta.

Van der Meer, J. (1999). *Design of concrete armour layers.*

Verhagen, H., & D'Angremond, K. (2012). *Breakwaters and Closure Dams.* Delft.

Vetiver-Indonesia. (2015, 10). Retrieved from http://www.vetiver-indonesia.com/projects.htm

vetiverlatrine. (2015, 10). *vetiverlatrine*. Retrieved from vetiverlatrine: http://vetiverlatrine.org/index.php

Wheeler. (1987). *USA Patent No. US4954012A.*

APPENDICES

APPENDIX A: TIMELINE

A timeline has been constructed based on the information from the stakeholders meetings. Using information provided by Balai P.W., the following timeline can be reconstructed. Icons used in this timeline are:

🏛 = Commissioning institution or organisation
💼 = Executing institution, organisation or company

Year	Activity	Details
⚠	**Initial Issue** ▪ During dry season, the estuary silted up due to sedimentation influx from longshore transport. A sandbank closes the estuary. ▪ During rainy season the high discharges from the Serang River find their way to planes around the estuary, flooding agricultural land. ▪ To prevent flooding, farmers with local knowledge could forecast high discharges and were able to dig a small channel through the sandbank with little means. ▪ The Balai P.W. at the time decides upon the realization of river revetments that would cost significantly more, but would these costs would outweigh the risk of high damages. [Balai P.W., Reg. W.R.]	
▪ **1993**	**Detailed Design of the Serang River Estuary** 🏛 Balai P.W. (former name) 💼 P.T. Puser Bumi Consultants	- No details available anymore -
▪ **1996** ▪ **2005**	**Java Flood Control Sector Project** 🏛 National level Ministries 💼 BCEOM & Associates **Components** **A** Flood control and protection **B** Institutional Strengthening of Water Resources Services **C** Flood Warning System **D** Monitoring River Characteristics	**Results** ▪ New 'Balai' organisational structure for management of River Basins: One River, One management. Balai P.W. Is part of this new organisational structure. ▪ Designs and construction works for the fixation of the Serang River mouth. ▪ Still no adequate flood warning system has been designed for the Serang River.
▪ **2000**	**Development study on the South Beach Area** 🏛 Prov. M.A.F. 💼 UGM Center for Marine Resources & Technology Studies ▪ This aim of this research is to find the most suitable location for port	**Results** ▪ It was concluded that in the Kulon Progo region the existing infrastructure was most adequate for harbour development. ▪ It should be noted that the hydraulic conditions

placement on the southern Java coast.

- Assessed criteria were: Intra-regional road infrastructure, availability of reliable electrical supply and economic growth potential.

have not been assessed in this research.

2001

Feasibility of a Fishing Port Development Plan Glagah

💰 Prov. M.A.F.
💼 UGM Center for Marine Resources & Technology Studies

- With the location found, a new study by the same research institute aims to assess the feasibility of the Tanjung Adikarta Port near Glagah Beach.

Results

- It is concluded that the Tanjung Adikarta Port is a viable concept.

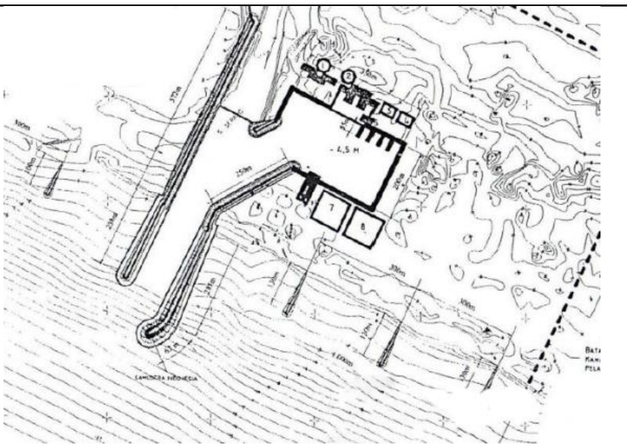
2003

Design Details Glagah Port

💰 Prov. M.A.F.
💼 UGM Center for Marine Resources & Technology Studies

Components

- Tanjung Adikarta Port preliminary design.
- Glagah breakwater design.



Results

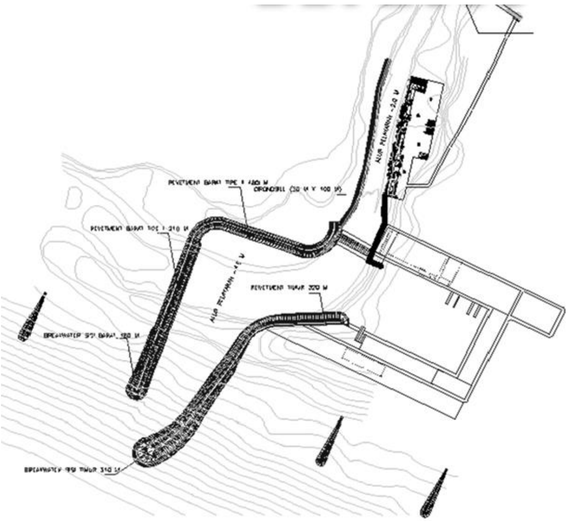
- Construction starts at Tanjung Adikarta Fishing Port.
- Protective Construction against river bluff attacks in the estuary from 2003-2005.

2005

2005 **Design Details for Repair Works**

Prov. Irrig.
P.T. Puser Bumi Consultants

- The Gabion dikes are designed after the Serang River discharges in rainy season proved to be disruptive for the estuary location and river banks.



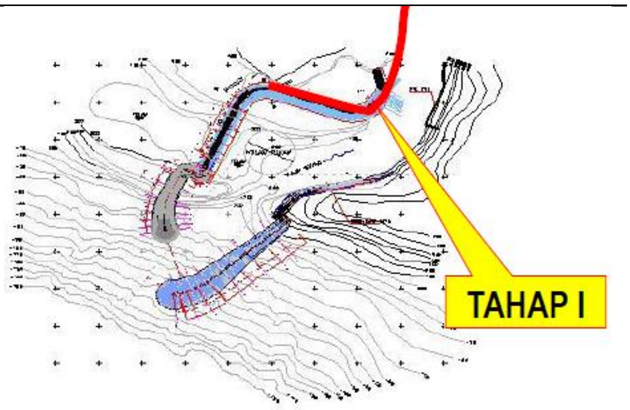
2005 **Construction of western Gabion Dyke (Phase I)**

Balai P.W.
PT PP (Persero) Tbk

Works

- 440 m of Gabion dike constructed as first element of Serang River estuary fixation and 'attack' mitigation strategies.

2006



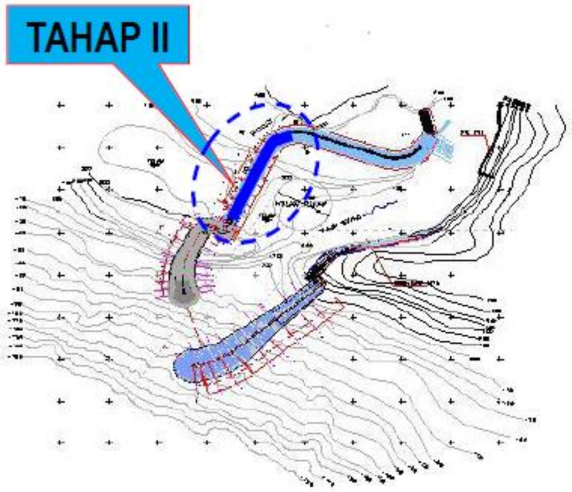
2006 **Construction extension of western Gabion Dyke (Phase II)**

Balai P.W.
PT PP (Persero) Tbk

Works

- Completion of the Gabion dike (phase I).
- Sheet pile insertions (625 m)
- Realization of tier-jack construction for further construction.

2007



2006 **Prov. M.A.F. requests construction of western breakwater**

By means of a formal letter to Balai P.W. This letter is required because the official responsibility of breakwater construction lies

Results:

- The formal letter is accepted by Balai P.W. and construction is divided into Jetty phase I (western breakwater) and later also for Jetty phase II (Eastern Breakwater). These phases are not to be confused with the internal Jetty project with Prov. M.A.F.

distinctions in phase I to III that describe the breakwater construction steps.

2006 – **2007**

Consummation Planning of the Breakwater Constructions

💰 Balai P.W.

💼 Balai P.W.

2007 – **2008**

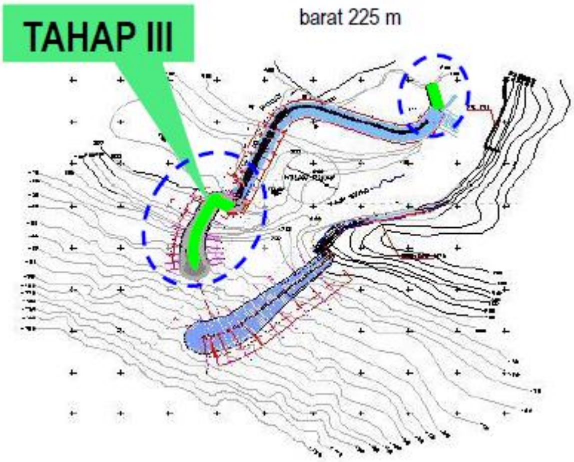
First Construction of the western breakwater (Phase III)

💰 Balai P.W.

💼 PT PP (Persero) Tbk

Works

- Completion of the Gabion Dikes (Phase II)
- Temporary installation of Tier-jack
- Construction of breakwater with 225m length.

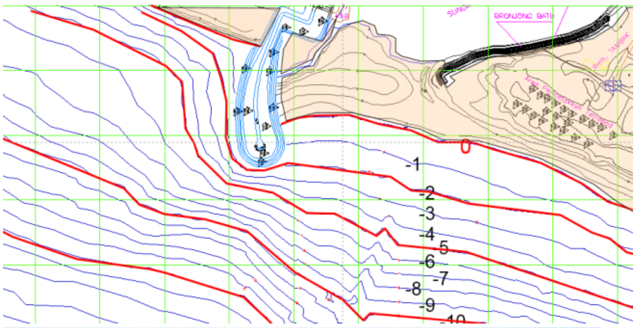


2008

Severe Issues Arose

⚠

- Longshore sediment flows silt up the Serang River estuary
- As the estuary remains closed in early rainy season, high discharges break through the porous structure.
- Flows carry large portions of grains of sand through the structure.
- The breakwater fails locally and the structure collapses under its own weight.



2008 **More Issues at the breakwater head**

- Wave height is underestimated in breakwater design.

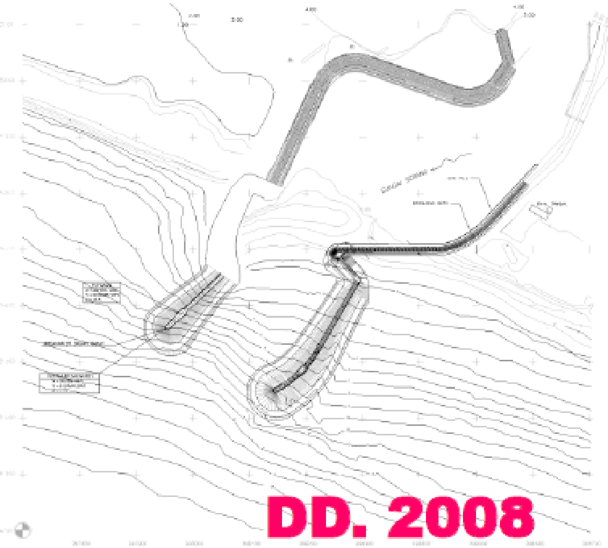
- Tip of the western breakwater collapses.

2008 **Review Design for Glagah Breakwaters**

Balai P.W.

C.V. Karsa Prawira (Assisted by Prof. Nur Yuwono)

- Pak Nur Yuwono is hired through a consultancy firm since this project is characterized as an emergency project.



2008 **First Construction of the Eastern breakwater**

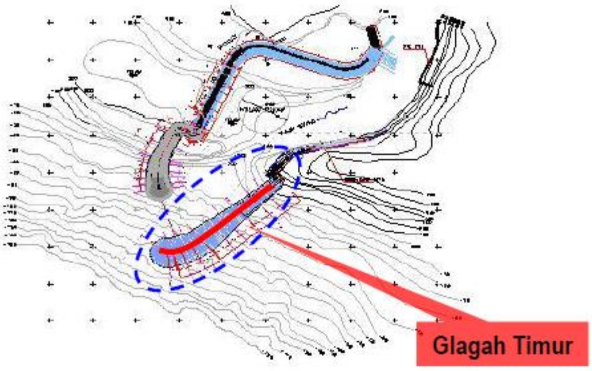
Balai P.W.

PT PP (Persero) Tbk

Works

- First multi-year Glagah Jetty Project at Balai P.W.
- Construction of eastern breakwater 307 m in length (220 m perpendicular to the shoreline).

2009



- Construction on the eastern breakwater was not finished, only about half of the breakwater was built due to wave overtopping.

2008 **Construction Complications and Issues**

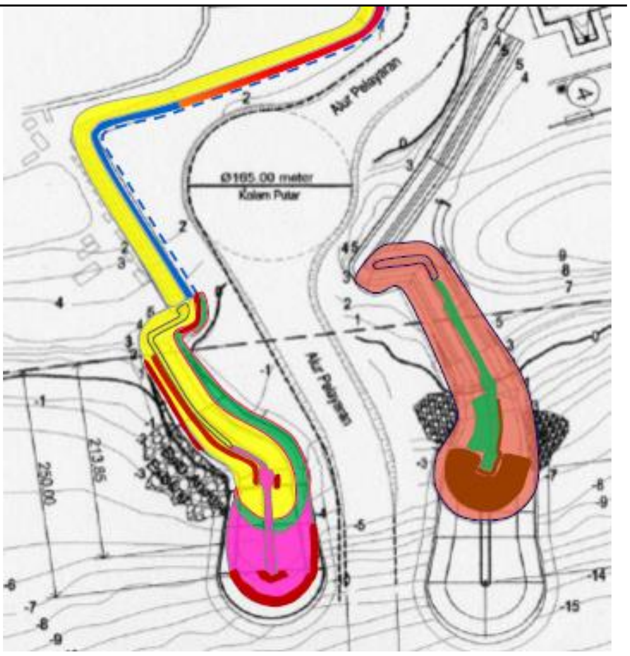
- Main Issue: storm caused core collapse of the breakwater.

2010 **Further Construction on the Glagah Jetty Project**

🏛 Balai P.W.
💼 PT PP (Persero) Tbk

Works

- Addition of a retaining wall on top of the eastern breakwater.
- Addition of western inner breakwater and head protection.

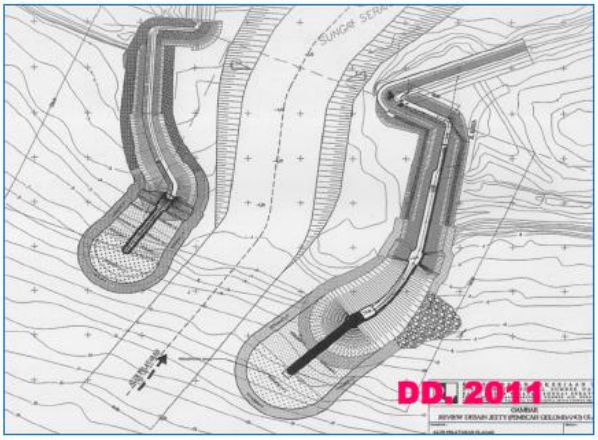


2010: Construction Elements highlighted in green

2011 **Breakwater Review Design**

🏛 Balai P.W.
💼 UGM Research Center for Engineering Science, led by Prof. Nur Yuwono

- Extension of western breakwater by 75 m.
- Prof. Nur Yuwono concludes that the breakwater angle should shift to open the navigation channel.



2011

Further Construction on the western breakwater

Balai P.W.

PT PP (Persero) Tbk

Works

- The western breakwater is lengthened and an access road is built that facilitates construction at the head.
- Construction works at the eastern breakwater are not executed (yet).

2010: Construction Elements highlighted in pink

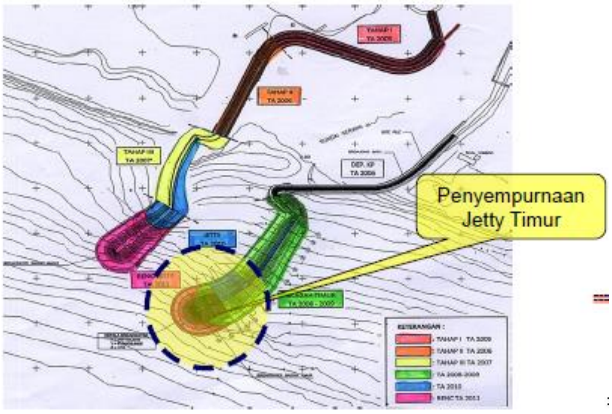
2012

Further Construction on the eastern breakwater

Balai P.W.

PT PP (Persero) Tbk

- The design of Prof. Nur Yuwono is not fully executed.



2012: Construction Elements highlighted with cricle

2012

Construction works of the Glagah Jetty Project are postponed. Further construction is subject to several conditions which multiple parties seem to disagree upon. Key word in this discussion is ‘Responsibility’, especially responsibility as to flood control versus navigation channel safety.

2012

MOU

A Memorandum Of Understanding (MOU) proposal has been sent by Min. P.W. to Min. M.A.F.

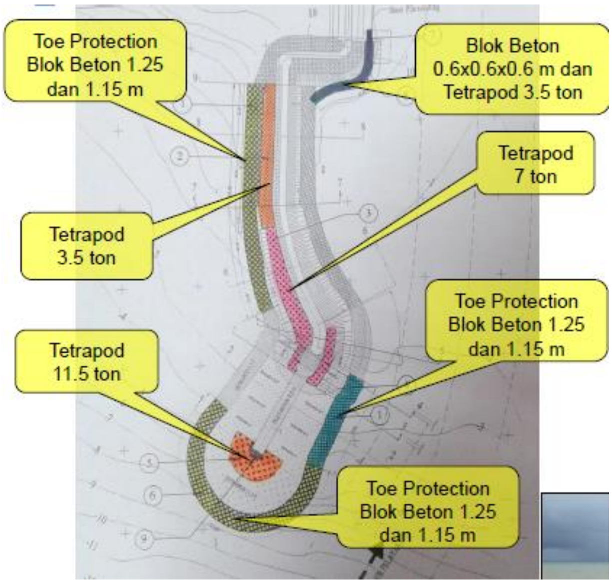
This proposal has been rejected by Min. M.A.F. for reasons unknown.

2013 **Critical Repair Works on the western breakwater**

Balai P.W.

PT PP (Persero) Tbk

- Critical Extension of western breakwater by a few meters.
- More key repair works.

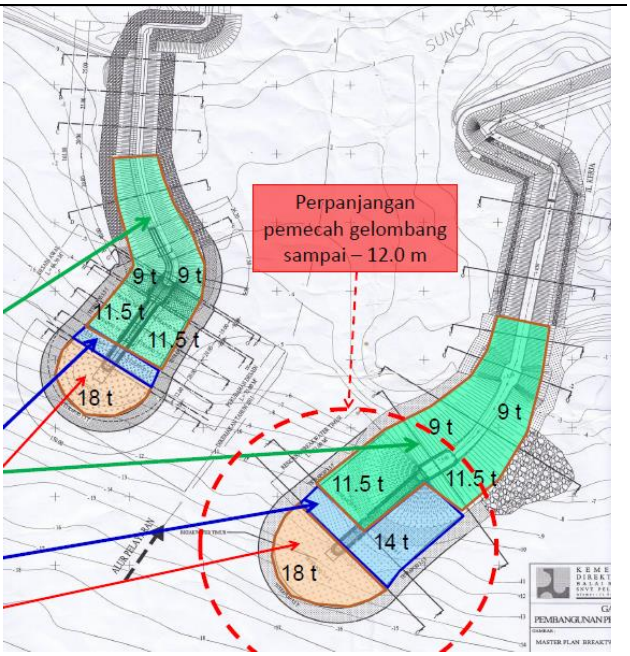


2013 **Review Design Tanjung Adikarta Port**

Balai P.W.

UGM Research Center for Engineering Science

- Analysis of Coastal Dynamics
- Review Design of Breakwater



2014 **Major Issue**

It remains unclear who should continue breakwater extensions and dredging works.

2015 **Tanjung Adikarta Port & Glagah Jetty Recommendations**

Project Yogya Partners

Project Yogya

Results:

- Technical analysis of breakwater design.
- Stakeholder mapping and issue identification.
- Recommendations for future improvement.

APPENDIX B: HYDRAULIC CONDITIONS

B.1 TIDE

At first, the local tide around the Glagah harbour is estimated. Measurements have already been performed by two institutions. The first institution is a consultant named CV. Karsa Prawire and did measurements to the tide in 2008. The second institution is University Gajah Mada which did measurements in 2011. These measurements are shown in the second and third column in Table B-1 respectively. In order to prevent underestimations in the design values, in this report the most extreme measurements are used in design calculations which are those of CV. Karsa Prawire. (BAB 4. Surbey topografi, bathimetri dan pasang surut, 2013). (BAB 5. Review perencanaan pemecah gelombang Glagah, 2013)

TABLE B-1 TIDES AT GLAGAH BEACH

Crucial elevation	**Tidal elevation [m]**		
	CV. Karsa Prawire (2008)	**PSIT UGM (2011)**	**Used in project**
HHWL	+ 2.16	+ 1.92	+ 2.16
MSL	+ 1.08	+ 0.96	+ 1.08
LLWL	+ 0.00	+ 0.00	+ 0.00

Note: Difference between measurements is tolerable considering the harsh sea conditions at Glagah Beach.

B.2 WAVES

As to waves, over time several studies to the south coast of Java have been carried out by multiple institutes. In a brief summary below these institutes are mentioned and the influences of their research on this report will be discussed.

As part of the South Java Flood Control Sector Project (SJFCSP), in 1999 wind and wave measurements were performed by UK Meteorological Office. These recordings were used to calibrate a SWAN model for the southern Javanese coast. At seventeen locations along the coast the wave development from offshore to nearshore was calculated by the SWAN model. Unfortunately, the input data of this model and the results at the locations are unavailable. Only a summary of the model is given in the SJFCSP report. In this report the average wave period is used, which is determined to be 15 sec, proven by a comparison between the model output and wave measurements. (Asian Development Bank, 2007)

Unfortunately, due to the loss of this data, a data source originating from the coast of Bali is used to perform wave calculations in this report. These measurements are performed by the U.S. Navy Marine and are published in the U.S. Navy Marine Climate Atlas of the World Vol 3, Indian Ocean (USNMCAW). However, this data source only contains already modified deep wave statistics by JICA (1989), which will limit the level of depth of the calculations in this chapter.

SIGNIFICANT WAVE HEIGHT

As already mentioned in the previous section, random wave statistics from USNMCAW are used in this chapter. These measurements are recorded in deep water conditions in front of the coast of Bali. Although the location of Bali is hundreds kilometres away from the project location, it can still be assumed that this deep water data is representative for Glagah beach considering the fact that there are no significant changes in the deep water conditions between both locations.

TABLE B-2 MODIFIED WAVE STATISTICS BY JICA 1989

Significant wave height (m)	Percentage of incident waves [%]								
	North	North-east	East	South-east	South	South-west	West	North-west	Total
0.0 – 1.0	0.50	2.50	4.29	4.67	3.30	2.54	0.60	0.50	18.90
1.0 – 2.0	0.30	0.80	7.86	9.89	20.27	7.79	4.64	1.43	52.98
2.0 – 3.0	0.00	0.00	3.66	4.48	7.54	5.07	2.46	0.97	24.18
> 3.0	0.00	0.00	0.00	0.56	1.89	1.50	0.00	0.00	3.95
Total	0.80	3.30	15.81	19.60	33.00	16.90	7.70	2.90	100.00

(U.S. Navy, Naval Weather Service Command , 1976)

From Table B-2 it can be concluded that the dominant wave directions are East, Southeast, South and Southwest. Comparing this results with observations of the design of the harbour and the surrounding area, from the shape of the breakwaters and the shape of the adjacent coastline it can be derived that the mature wave attack is from the South - Southeast.

The data provided by the USNMCAW is solely composed of offshore wave data hence only includes swell waves. One should notice that this data is not sufficient for calculating local wave conditions at Glagah beach since that consists of swell and wind waves. The hardest part of such a wave spectrum is the fact that both wave climates are fully independent of each other. The swell waves are generated far away from the southern Javanese coast and can therefore be seen as almost constant over the year in height and direction. On the contrary, the wind waves generated by the local wind climate are fully dependent of the wind direction. The wind has a significant seasonal change as the wind direction is almost changing 180°. Therefore wind data from JURNAL OSEANOGRAFI, Vol 3 is used to increase the accuracy of the calculations, which is shown in Table B-3.

TABLE B-3 PERCENTAGE OF AVERAGE WIND SPEED BY JURNAL OSEANOGRAFI, VOL 3

Wind speed (knots)	Percentage of average wind speed in the dominant directions [%]				
	West	Southwest	South	Southeast	East
0.0 – 6.0	6.00	3.00	4.80	10.70	0.42
6.0 – 10.0	7.40	2.60	2.72	18.00	4.60
10.0 – 14.0	1.40	0.15	0.40	12.50	9.40
14.0 – 16.0	0.50	0.00	0.10	3.10	3.20
16.0 – 19.0	0.00	0.00	0.00	2.70	2.20
> 19.0	0.00	0.00	0.00	0.67	0.50
Total	15.30	5.75	8.02	47.67	20.32

(Sihotang, Subardjo, & Saputro, 2014)

Comparing the average local wind data with the offshore wave climate, one can observe similarities between the averaged dominant directions. Over the year, the wind is often in the same direction as the incoming waves. This process will result in an increase of the wave height, as wind and swell waves will support each other. Due to the seasonal change in wind direction, it is important to include wind into the wave calculations as it has large influence on the direction of the longshore sediment transport. This topic is further discussed in chapter 0 .

In the wave calculations in the coming sections the significant wave height is determined. Before the calculations are executed, the local wind system will be investigated in order to understand its influence on the wave heights.

SHORTCOMINGS AND ASSUMPTIONS OF DATASETS

Due to the availability of only modified wind and wave data several shortcomings will occur compared to calculations based on the original measured data. In order to evaluate on deviations caused by these modified data sets shortcomings are discussed per data type.

WAVE DATA

- Wave bins are very large resulting in little data points for wave height extrapolation.
- The dataset does not contain storm wave height data. Therefore the method for random data had to be executed. Besides, the average storm duration had to be determined. The intervals were distributed over the exceedance rates per wave and wind directions. The following assumptions were done:

  Interval of the storm: $t_{Storm} = 12\ hours$
  Number of storm intervals in a year: $N_{Storm} = 730\ intervals$

- For the measured waves the wave periods are not measured. A wave period is chosen that originates from a different measurement by UK Meteorological Institute.
- There are no nearshore measurements to check model calculations. In order to discuss the model output, multiple reference cases are used.

WIND DATA

- Wind bins are very large and equal in size resulting in little and irregular data points for the wind speed extrapolation.
- The dataset contains merely averaged annual wind velocities. This results in an underestimation of extreme wind speeds.
- Graphs had to be transformed to tables by hand. This may have generated some small errors.

BATHYMETRY

- The bathymetry west and east of the harbour showed significant differences. Also the measurement location is not exactly known.
- Figures had to be transformed to tables by hand. This may have generated some small errors.

INFLUENCE OF WIND DATA INCLUDING VARYING WEST AND EAST BATHYMETRY

To reduce the amount of calculations a small test is done. In this test the influence of the wind on waves is tested as well as the influence of the western and eastern bathymetry on the waves. This test is performed using SwanOne. The model input is shown in the Table B-4 and Table B-5 and figure Figure B-1. The concept behind this test is to calculate the influence of wind by changing its direction and strength with respect to the wave direction. The independency of wave and wind systems is included by combining extreme and regular cases. Also a test is performed in order to determine the effect of a combined extreme case with extreme waves and wind speeds, however one should consider that the harbour should not be designed to withstand such an event.

First, a wind dominated climate with extreme wind speeds from three different directions is combined with regular waves from one direction. Second, a wave dominated climate with extreme waves from one direction is combined with regular wind speeds from three different directions. At last, extreme waves from one direction are combined with extreme wind speeds from three different directions. Next, these tests are repeated with the second bathymetry option, resulting in a total of 18 calculations.

TABLE B-4 WINDSPEEDS AND WAVEHEIGHTS PER DIRECTION FOR THE WESTERN BATHYMMETRY

Bathymetry West

Calculation	Direction	Wave heights Return period [years]	Input value [m]	Direction	Wind speeds Return period [years]	Input value [m/s]
1	**Southeast**	1/1	2.11	**East**	1/250	24.64
2		1/1	2.11	**Southeast**	1/250	24.64
3		1/1	2.11	**South**	1/250	24.64
4	**Southeast**	1/250	5.25	**East**	1/1	16.45
5		1/250	5.25	**Southeast**	1/1	16.45
6		1/250	5.25	**South**	1/1	16.45
7	**Southeast**	1/250	5.25	**East**	1/250	24.64
8		1/250	5.25	**Southeast**	1/250	24.64
9		1/250	5.25	**South**	1/250	24.64

TABLE B-5 WINDSPEEDS AND WAVEHEIGHT PER DIRECTION FOR THE EASTERN BATHYMETRY

Bathymetry East

Calculation	Direction	Wave heights Return period [years]	Input value [m]	Direction	Wind speeds Return period [years]	Input value [m/s]
10	**Southeast**	1/1	2.11	**East**	1/250	24.64
11		1/1	2.11	**Southeast**	1/250	24.64
12		1/1	2.11	**South**	1/250	24.64
13	**Southeast**	1/250	5.25	**East**	1/1	16.45
14		1/250	5.25	**Southeast**	1/1	16.45
15		1/250	5.25	**South**	1/1	16.45
16	**Southeast**	1/250	5.25	**East**	1/250	24.64
17		1/250	5.25	**Southeast**	1/250	24.64
18		1/250	5.25	**South**	1/250	24.64

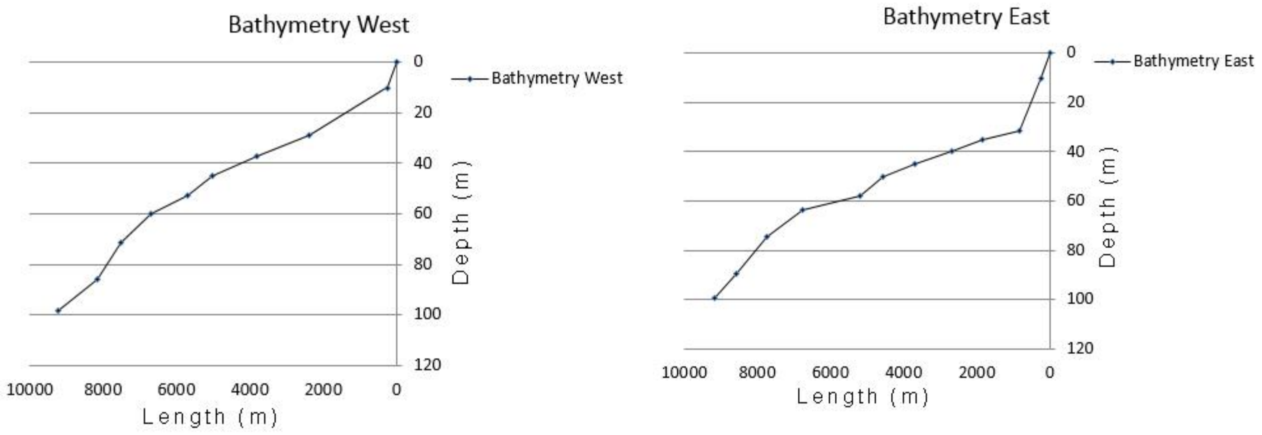


FIGURE B-1 WESTERN AND EASTERN BATHYMETRY

After the tests, the following conclusions were observed:

- Extreme wind events have reasonable influence on the wave height, however compared with extreme wave events it has no significant contribution to the wave attack on the harbour.
- The wind has most influence on the wave height if it blows in the wave direction. During wave dominant systems there is almost no influence by wind, no matter if the winds blow in the wave propagating direction or from under an angle.
- Wind has large influence on the mean absolute wave period. Offshore, the wave period decreases, close to the coast it increases again.
- No large difference between eastern and western bathymetry. Eastern bathymetry causes a small increase in wave height with respect to the western bathymetry.

SIGNIFICANT WAVE HEIGHT CALCULATIONS

Now all the possible errors and assumptions due to the modified datasets have been mapped and also the influence of the wind has been determined, the calculation can be performed. After testing the other two dominant wave directions with extreme wave conditions and varying regular wind directions, the following extreme cases were found:

- Case 1: Waves from the southeast combined with wind from the southeast.
- Case 2: Waves from the south combined with wind from the southeast.
- Case 3: Waves from the southwest combined with wind from the west.

The extreme case of waves from the east combined with wind from the east of southeast is neglected. This is because the wave and wind measurements are performed offshore. Here, the waves from the east will just propagate into the same direction while the depth of the ocean is too deep to disturb their path. Nearshore, the direction of Java will prevent waves from eastern direction. Therefore we assume no extreme waves will occur from the east near the coast.

In the next section first the extrapolation of the wave and wind data is explained in order to find the required design storm wave height. For this extrapolation the Exponential and the Weibull method are used. After that, in Table B-12 the SwanOne calculations of the extreme cases are summarized. Using the Exponential method, first the data has to be transformed to find the probability of exceedance of a storm (Q_s) which can then plotted against the significant storm wave height (H_{ss}) or storm wind speed (U_s). Using the Weibull method, first the data has to be transformed to find the reduced Weibull variable (W_s) which can then plotted against the significant storm wave height (H_{ss}) or storm wind speed (U_s).

The probability of exceedance of a storm and the reduced Weibull variate are determined as followed:

W_s	=	Reduced Weibull variate	=	$-\ln(Q_{ss})^{\frac{1}{\alpha}}$
Q_{ss}	=	Probability of exceedance of a storm	=	$Q * N_s$
Q	=	Probability of exceedance of a wave / wind speed	=	1 – P
N_s	=	Number of storms per year per wave and wind directions		
P	=	Probability of non-exceedance of a wave / wind speed		

For each case the parameters are calculated in the tables below after which the results are plotted. From the graphs the coefficients of the slope (A) and the intercept (B) are calculated. With the following formulas the design storm wave height and design storm wind speed for a certain storm frequency are calculated:

Exponential method: $H_{ss}\ or\ U_s = \gamma - \frac{\ln(Storm\ frequency)}{A}$

Weibull method: $H_{ss}\ or\ U_s = \gamma + \beta(-\ln(W_s))^{\frac{1}{\alpha}}$

With:

- γ = $-\frac{B}{A}$
- β = $\frac{1}{A}$

CASE 1: WAVES FROM THE SOUTHEAST COMBINED WITH WIND FROM THE SOUTHEAST

TABLE B-6 COMPUTATIONS OF WAVE DATA FROM SOUTHEASTERN DIRECTION

Significant wave height (m)	**P**	**Q**	**Qss (s/y)**	**ln(Qss)**	**Ws (α = 0.410)**
0.0 - 1.0	0.238265	0.761735	108.99	4.691247	-43.38
1.0 - 2.0	0.742857	0.257143	36.79	3.605280	-22.82
2.0 - 3.0	0.971429	0.028571	4.09	1.408056	-2.30
> 3.0	1.000000	0	0	0	0

- Exponential extrapolation

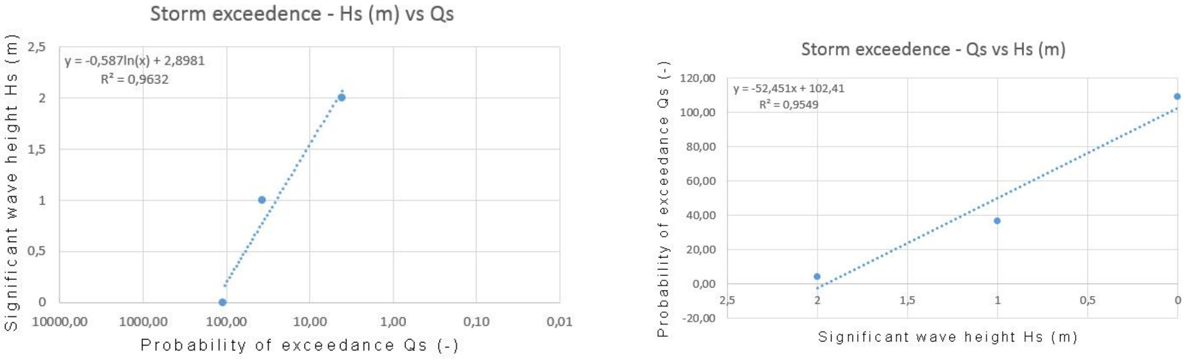


FIGURE B-2 EXTRAPOLATION OF SOUTHEASTERN WAVE STATISTICS (EXPONONTIAL METHOD)

A = 0.525
B = -1.024

H_s (1/1) = 1.95 m
H_s (1/50) = 9.41 m
H_s (1/100) = 10.73 m
H_s (1/150) = 11.51 m
H_s (1/250) = 12.48 m

- Weibull extrapolation

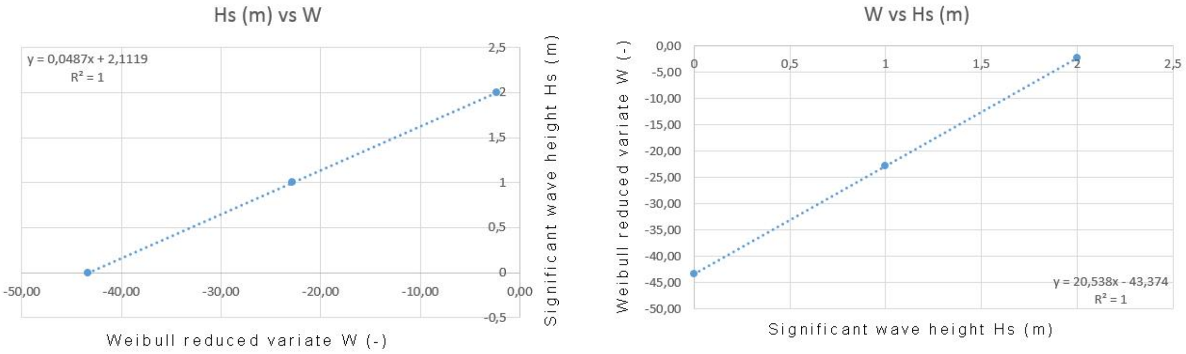


FIGURE B-3 EXTRAPOLATION OF SOUTHEASTERN WAVE STATISTICS (WEIBULL METHOD)

A = 20.538
B = -43.374
α = 0.410
β = 0.049
γ = 2.112

H_s (1/1) = 2.11 m
H_s (1/50) = 3.47 m
H_s (1/100) = 4.13 m
H_s (1/150) = 4.59 m
H_s (1/250) = 5.25 m

TABLE B-7 COMPUTATIONS OF WIND DATA FROM SOUTHEASTERN DIRECTION

Wind speed (knots)	**P**	**Q**	**Qss (s/y)**	**ln(Qss)**	**Ws (α = 0.339)**
0.0 - 6.0	0.224476	0.775524	269.86	5.597891	-160.90
6.0 - 10.0	0.602098	0.397902	138.46	4.930557	-110.65
10.0 - 14.0	0.864336	0.135664	47.21	3.854535	-53.52
14.0 - 16.0	0.929371	0.070629	24.58	3.201797	-30.96
16.0 - 19.0	0.986014	0.013986	4.87	1.582409	-3.87
> 19.0	1.000000	0	0	0	0

- Exponential extrapolation

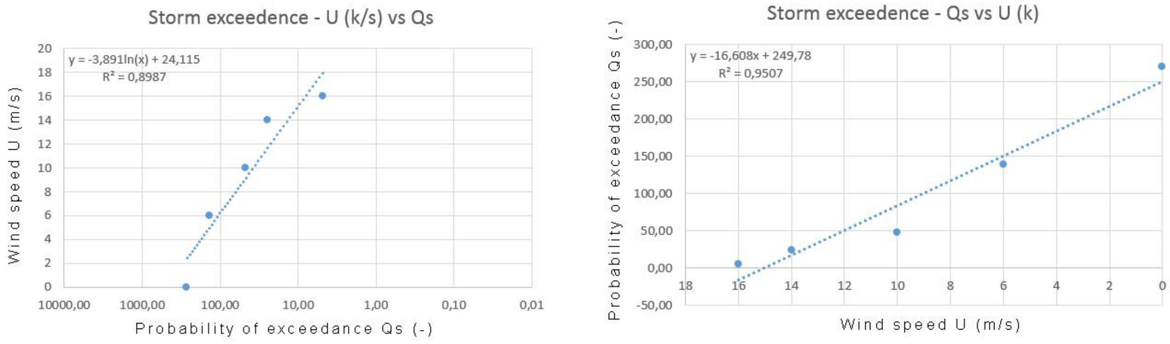


FIGURE B-4 EXTRAPOLATION OF WIND STATISTICS FROM SOUTHEASTERN DIRECTION (EXPONONTIAL METHOD)

A = 0.166
B = -2.498

U (1/1) = 15.04 m/s
U (1/50) = 27.16 m/s
U (1/100) = 29.30 m/s
U (1/150) = 30.56 m/s
U (1/250) = 32.14 m/s

- Weibull extrapolation

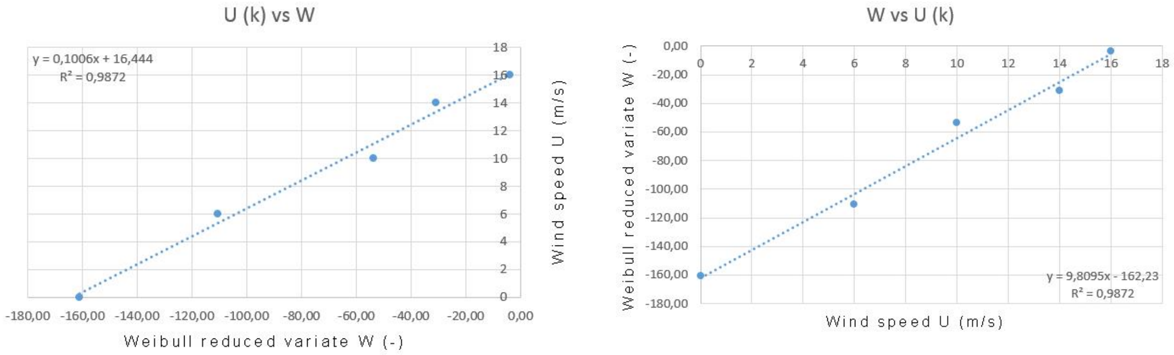


FIGURE B-5 EXTRAPOLATION OF SOUTHEASTERN WIND STATISTICS (WEIBULL METHOD)

A = 9.810
B = -162.230
α = 0.339
β = 0.102
γ = 16.538

U (1/1) = 16.45 M/S
U (1/50) = 19.47 m/s
U (1/100) = 21.28 m/s
U (1/150) = 22.62 m/s
U (1/250) = 26.64 m/s

CASE 2: WAVES FROM THE SOUTH COMBINED WITH WIND FROM THE SOUTHEAST

TABLE B-8 COMPUTATIONS OF WAVE DATA FROM SOUTHERN DIRECTION

Significant wave height (m)	**P**	**Q**	**Qss (s/y)**	**ln(Qss)**	**Ws (α = 0.500)**
0.0 – 1.0	0.100000	0.900000	216.81	4.691247	-348.46
1.0 – 2.0	0.714242	0.285758	68.84	3.605280	-191.54
2.0 – 3.0	0.942727	0.057273	13.80	1.408056	-34.03
> 3.0	1.000000	0	0	0	0

- Exponential extrapolation

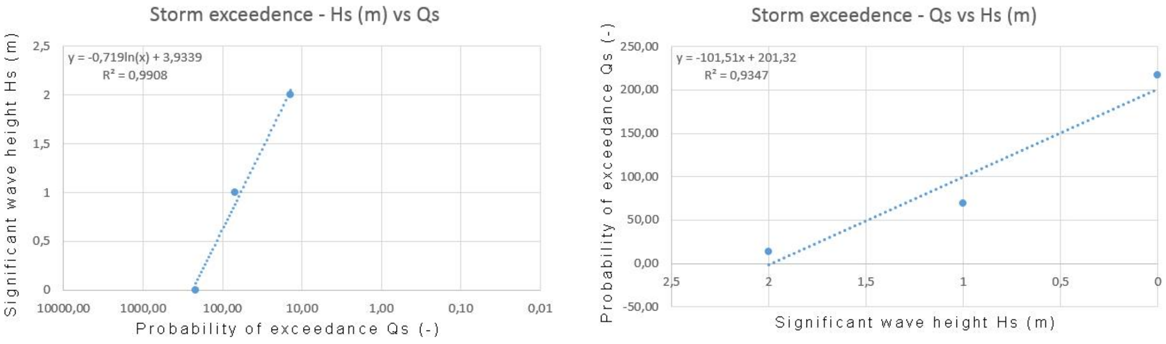


FIGURE B-6 EXTRAPOLATION OF WAVE STATISTICS FROM SOUTHERN DIRECTION (EXPONONTIAL METHOD)

A = 1.015
B = -2.013

H_s (1/1) = 1.98 m
H_s (1/50) = 5.84 m
H_s (1/100) = 6.52 m
H_s (1/150) = 6.92 m
H_s (1/250) = 7.42 m

- Weibull extrapolation

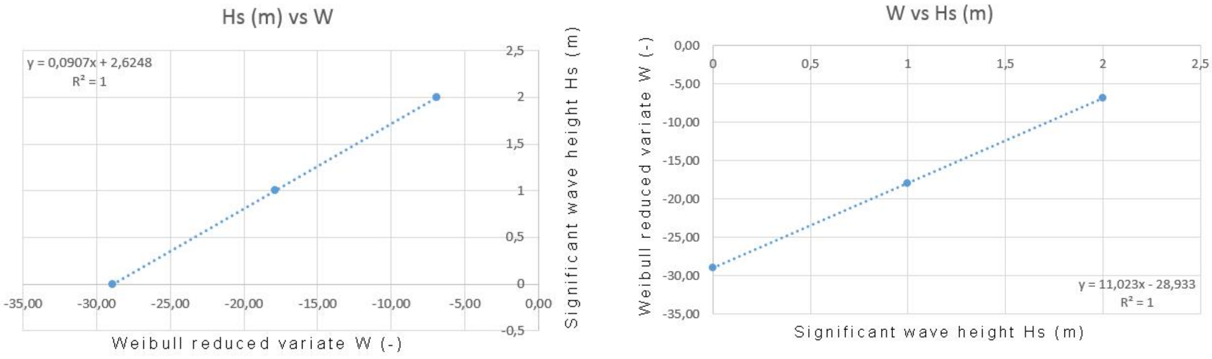


FIGURE B-7 EXTRAPOLATION OF WAVE STATISTICS FROM SOUTHERN DIRECTION (WEIBULL METHOD)

A	=	11.023	H_s (1/1)	=	2.62 m
B	=	-28.933	H_s (1/50)	=	4.01 m
α	=	0.500	H_s (1/100)	=	4.55 m
β	=	0.091	H_s (1/150)	=	4.90 m
γ	=	2.625	H_s (1/250)	=	5.39 m

TABLE B-9 COMPUTATIONS OF WIND DATA FROM SOUTHEASTERN DIRECTION

Wind speed (knots)	**P**	**Q**	**Qss (s/y)**	**ln(Qss)**	**Ws (α = 0.339)**
0.0 - 6.0	0.224476	0.775524	269.86	5.597891	-160.90
6.0 - 10.0	0.602098	0.397902	138.46	4.930557	-110.65
10.0 - 14.0	0.864336	0.135664	47.21	3.854535	-53.52
14.0 - 16.0	0.929371	0.070629	24.58	3.201797	-30.96
16.0 - 19.0	0.986014	0.013986	4.87	1.582409	-3.87
> 19.0	1.000000	0	0	0	0

- Exponential extrapolation

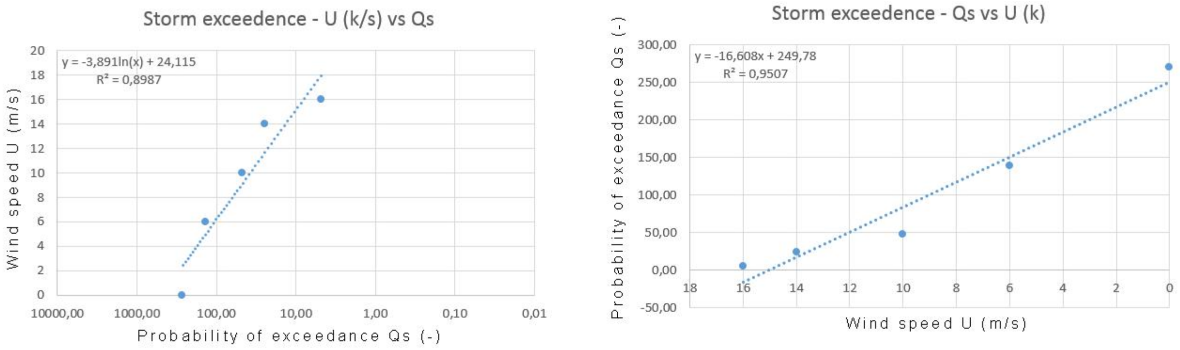


FIGURE B-8 EXTRAPOLATION OF WIND STATISTICS FROM SOUTHEASTERN DIRECTION (EXPONONTIAL METHOD)

A	=	0.166	U (1/1)	=	15.04 m/s
B	=	-2.498	U (1/50)	=	27.16 m/s
			U (1/100)	=	29.30 m/s
			U (1/150)	=	30.56 m/s
			U (1/250)	=	32.14 m/s

- Weibull extrapolation

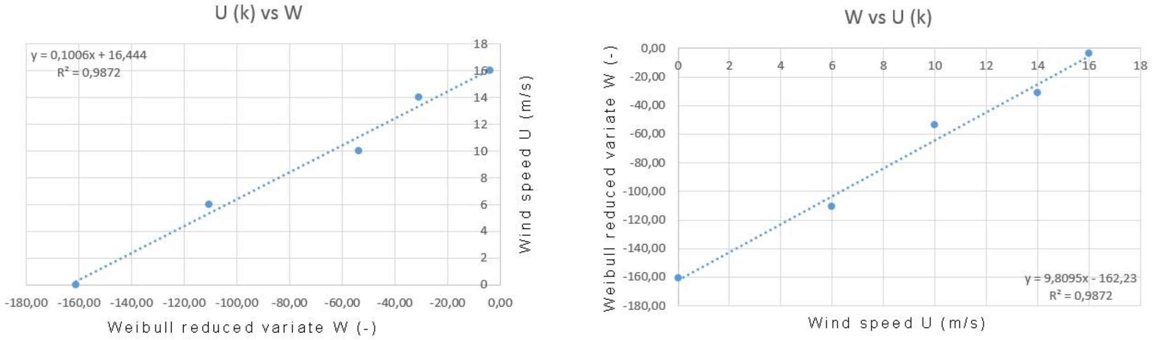


FIGURE B-9 EXTRAPOLATION OF WIND STATISTICS FROM SOUTHEASTERN DIRECTION (WEIBULL METHOD)

A = 9.810
B = -162.230
α = 0.339
β = 0.102
γ = 16.538

U (1/1) = 16.45 m/s
U (1/50) = 19.47 m/s
U (1/100) = 21.28 m/s
U (1/150) = 22.62 m/s
U (1/250) = 26.64 m/s

CASE 3: WAVES FROM THE SOUTHWEST COMBINED WITH WIND FROM THE WEST

TABLE B-10 COMPUTATIONS OF WAVE DATA FROM SOUTHWESTERN DIRECTION

Significant wave height (m)	**P**	**Q**	**Qss (s/y)**	**ln(Qss)**	**Ws (α = 0.320)**
0.0 – 1.0	0.150296	0.849704	104.83	5.379021	-122.03
1.0 – 2.0	0.611243	0.388757	47.96	4.231770	-68.66
2.0 – 3.0	0.911243	0.088757	10.95	2.624451	-15.29
> 3.0	1.000000	0	0	0	0

- Exponential extrapolation

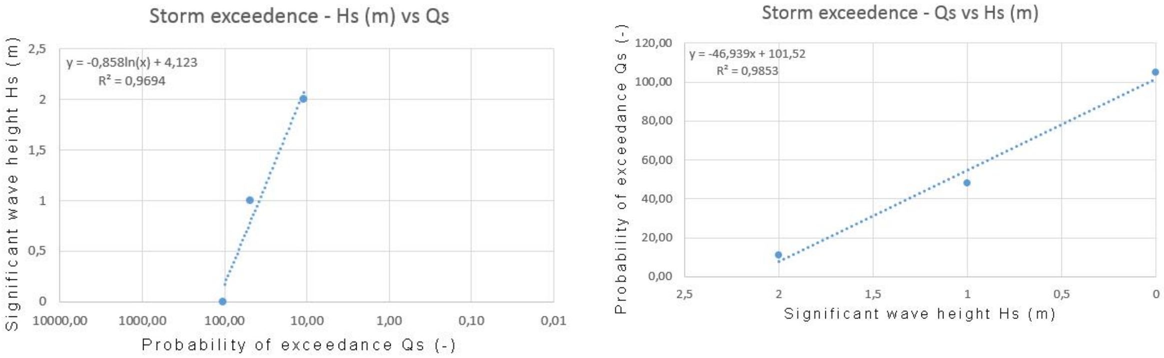


FIGURE B-10 EXTRAPOLATION OF WAVE STATISTICS FROM SOUTHWESTERN DIRECTION (EXPONONTIAL METHOD)

A = 0.469
B = -1.015

H_s (1/1) = 2.16 m
H_s (1/50) = 10.50 m
H_s (1/100) = 11.97 m
H_s (1/150) = 12.84 m
H_S (1/250) = 13.93 m

- Weibull extrapolation

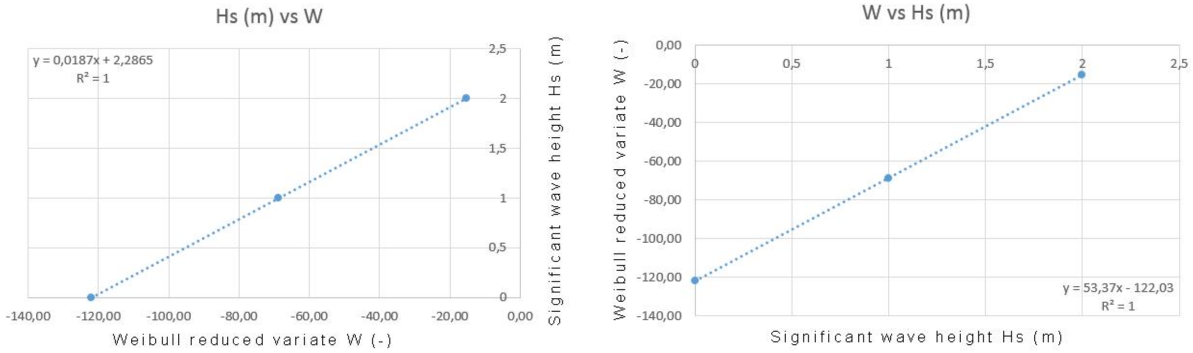


FIGURE B-11 EXTRAPOLATION OF WAVE STATISTICS FROM SOUTHWESTERN DIRECTION (WEIBULL METHOD)

A = 53.370
B = -122.030
α = 0.320
β = 0.019
γ = 2.286

H_s (1/1) = 2.29 m
H_s (1/50) = 3.62 m
H_s (1/100) = 4.50 m
H_s (1/150) = 5.17 m
H_s (1/250) = 6.19 m

TABLE B-11 COMPUTATIONS OF WIND DATA FROM WESTERN DIRECTION

Wind speed (knots)	**P**	**Q**	**Qss (s/y)**	**ln(Qss)**	**Ws (α = 0.710)**
0.0 - 6.0	0.392157	0.607843	67.89	4.217889	-7.59
6.0 - 10.0	0.875817	0.124183	13.87	2.629728	-3.90
10.0 - 14.0	0.967320	0	3.65	1.294727	-1.44
14.0 - 16.0	1.000000	0	0	0	0
16.0 - 19.0	1.000000	0	0	0	0
> 19.0	1.000000	0	0	0	0

- Exponential extrapolation

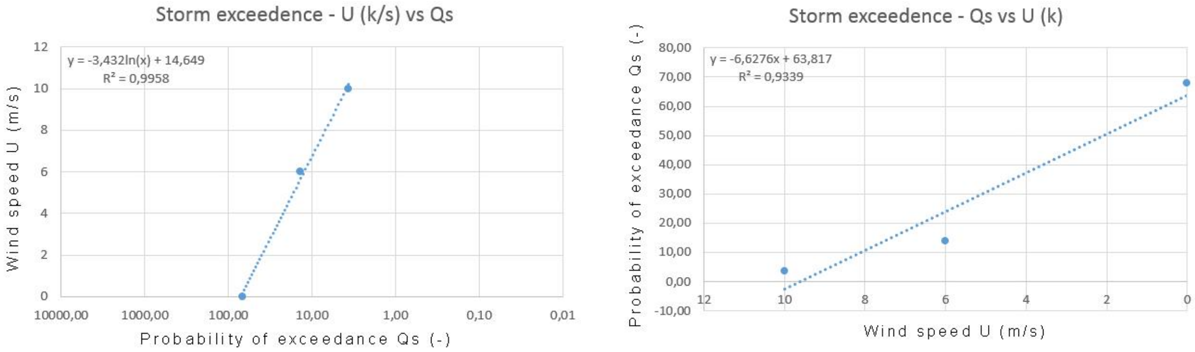


FIGURE B-12 EXTRAPOLATION OF WIND STATISTICS FROM WESTERN DIRECTION (EXPONONTIAL METHOD)

A = 0.066
B = -0.638

U (1/1) = 9.63 m/s
U (1/50) = 39.99 m/s
U (1/100) = 45.37 m/s
U (1/150) = 48.52 m/s
U (1/250) = 52.49 m/s

- Weibull extrapolation

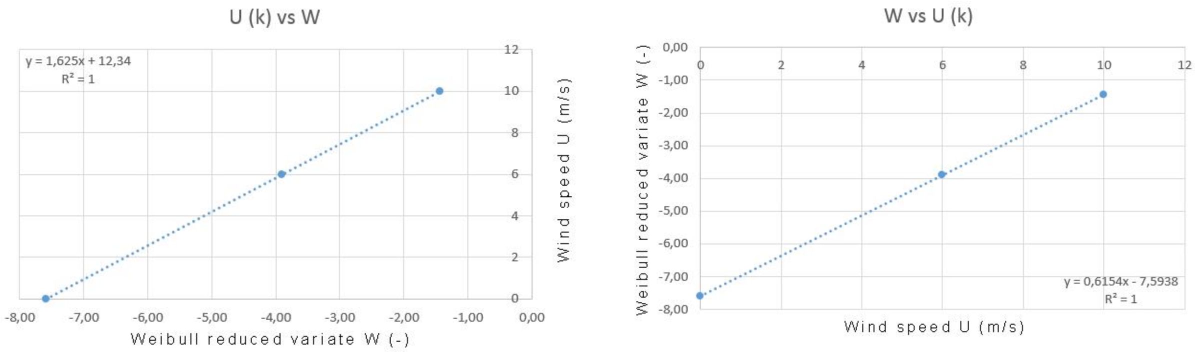


FIGURE B-13 EXTRAPOLATION OF WIND STATISTICS FROM WESTERN DIRECTION (WEIBULL METHOD)

A = 0.615
B = -7.594
α = 0.710
β = 1.625
γ = 12.340

U (1/1) = 12.34 m/s
U (1/50) = 18.05 m/s
U (1/100) = 19.52 m/s
U (1/150) = 20.43 m/s
U (1/250) = 21.61 m/s

SWANONE CALCULATIONS

In Table B-12 the input and output values, based on previous calculations, of the SwanOne calculations are shown per case. The input values of extrapolated wind and wave values differs if one chose the Exponential or Weibull outcomes. The significantly higher values of the Exponential method compared to the Weibull values are to be expected, as the Exponential function always represent the upper boundary. However those values do not approach realistic circumstances. In contrast with that, the Weibull values are quite reasonable as they show realistic values and approaches the curve of the data points accurately. Therefore in this report is chosen to use the Weibull values for the SwanOne calculations. In Table B-12 the extreme values per return period that can possibly occur according to the calculations, are highlighted and will be used in paragraph B.2.1.

TABLE B-12 SWANONE CALCULATION RESULTS

Case	**Input waves height [m]**		**Input wind speed [m/s]**		**Output waves [m]**
1	1/50	3.47	1/1	16.45	3.02
	1/100	4.13	1/1	16.45	3.41
	1/150	4.59	1/1	16.45	3.69
	1/250	5.25	1/1	16.45	4.06
2	1/50	4.01	1/1	16.45	4.34
	1/100	4.55	1/1	16.45	4.77
	1/150	4.90	1/1	16.45	5.05
	1/250	5.39	1/1	16.45	5. 44
3	1/50	3.62	1/1	12.34	3.81
	1/100	4.50	1/1	12.34	4.49
	1/150	5.17	1/1	12.34	5.06
	1/250	6.19	1/1	12.34	5.91

EVALUATION OF THE CALCULATIONS

Now that the outcomes of the SwanOne calculations are known, they will be discussed before comparing them to other measured and calculated wave heights. When observing the three extreme cases, one can see that no single case has the highest values for the four calculated return periods. This could be explained by the fact that the extrapolation of wave heights is based on only three available data points. Therefore significant deviations are not inconceivable. Due to these large uncertainties, it has been decided to take all maximum values per return period. These are combined in one set of wave heights as a compromise for the high probability of large deviations. In this report it is assumed that this new set of wave heights, formed out of case 2 and 3, will represent the design wave heights for the whole harbour area.

This combined set can be compared to calculated or measured wave heights at adjacent or comparable coastlines. In Table B-13 these results are shown.

TABLE B-13 COMPARISON OF DIFFERENT SIGNIFICANT WAVE HEIGHT DETERMINATIONS

Return period (years)	**Observations at Kuta Beach, Bali [m]**	**Statistical Data of the U.S. Navy by JICA (1989), Bali [m]**	**Observation Tipar estuary by BCOM (1993) [m]**	**SwanOne calculations at breaker line [m]**
1	3.03	-	3.00	-
5	4.13	2.70	3.80	-
10	4.53	3.40	4.10	-
30	5.16	4.50	-	-
50	5.44	4.95	4.90	4.34
100	5.80	5.40	5.30	4.77
120	5.89	5.70	-	-
150	-	-	-	5.06
250	-	-	-	5.91

One can observe from the table that the calculated wave heights by SwanOne are significantly lower than the other calculated wave heights or measurements. Most likely the SwanOne calculations underestimate the influence of wind because it's a simple 1D model. However, due to the lack of original measured data it is not possible to find out the exact error.

Observing the other calculations and observations along the South Javanese and Balinese coasts, one can see large similarities in the estimated wave heights. Due to these large similarities at coasts with a large distance in between, it is shown that the wave circumstances are almost equal along the Indian Ocean coastline. Therefore in this report it is chosen to use this wave data instead of the calculated wave heights by SwanOne.

CONCLUSION AND RECOMMENDATIONS OF THE CALCULATIONS

In this paragraph conclusions and recommendations are given about the invested wave data, wind data and bathymetry.

WAVE DATA

Concluding from the previous section, the wave data of the columns 1 to 3 of Table B-13 seems to be valid to for the Glagah beach. The highest values are obtained from the coast of Bali. Because it is not possible to verify the validity of the calculation methods, the highest values are chosen to prevent underestimation of the wave heights. In Figure B-14 the extrapolation of the calculations for a return period of maximum 1/250 years at the coast of Bali are shown.

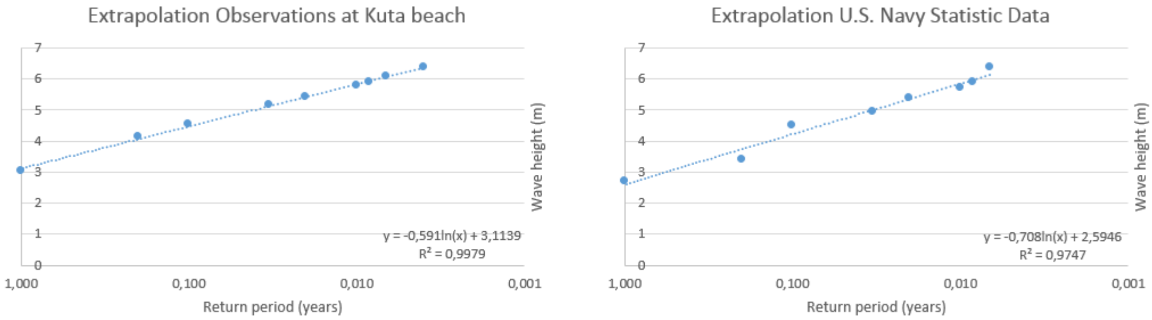


FIGURE B-14 EXTRAPOLATION OF DATA BASED ON THE OBSERVATIONS AT KUTA BEACH (LEFT FIGURE) AND BASED ON THE U.S. NAVY STATISTIC DATA (RIGHT FIGURE)

H_s (1/150) = 6.07 m
H_s (1/250) = 6.38 m

H_s (1/150) = 5.89 m
H_s (1/250) = 6.37 m

The outcomes of both extrapolations indicate that the calculations are pretty accurate as the differences in wave height are minimal. As already stated before, due to unverifiable calculation methods the highest values will be used. This means that the observed wave heights data from the coast of Bali and the extrapolation of that data forms the basis for future calculations. These values are shown in Table B-14.

TABLE B-14 WAVE HEIGHT VALUES USED FOR FUTURE CALCULATIONS PER RETURN PERIOD

Return period [years]	Wave height values used for future calculations [m]
1	3.03
5	4.13
10	4.53
30	5.16
50	5.44
100	5.80
120	5.89
150	6.07
250	6.39

In order to improve the wave height analyses, it is strongly recommended to locate the wind and wave data used for the SJFCSP, performed by the UK Meteorological Office. Furthermore one should also execute measurement nearshore to verify the output of the SWAN model. This can be used to indicate the influence of the wind on the wave climate and therefore improve the accuracy of the wave analyses. To optimize the design, one should initiate permanent wind and wave monitoring stations at the harbour. The long term data collected by such stations can be used to significantly increase the accuracy of the prediction of extreme storm events.

WIND DATA

To achieve a better understanding of the local wave climate and the wave attack on the harbour a more detailed wind analyses is recommended. In this report only yearly averaged data was available distributed over unequal bins that had to be transformed by hand in a table. From the available data the source should be located to increase the accuracy. However, due to a limited timeframe this is out of the scope of this report.

BATHYMETRY

The obtained bathymetry could be improved by getting access to the digital files. Several attempts have been executed, unfortunately without success. Again, it is recommended to locate the data source to improve the calculations.

SIGNIFICANT WAVE PERIOD

From the available data it is not possible to calculate the significant wave period. Therefore it has been decided to use the significant wave period determined by the SJFCSP. The reason behind this choice is the calibration of the SWAN model with wave measurements. From the 'South Java Flood Control Sector Project' it can be concluded that the calculated and measured offshore wave period is about 15 seconds (Asian Development Bank, 2007). Therefore this significant wave period is used during the SwanOne calculations and will be used in further calculations.

B.3 STORM SURGE LEVEL AND WIND SET-UP

The storm surge level and wind set-up level that will be used are based on the findings of Prof. Nur Yuwono. Storm surge level was found to be negligible and a wind set-up of +0.60 m. When sufficient data sources become available, it is recommended to further investigate the determination of these levels.

B.4 SEA LEVEL RISE

Sea level rise can be separated in relative and absolute sea level rise. For the design of the harbour only relative sea level rise is important. To determine the relative sea level rise, one should be in possession of a long term data set of land subsidence and a long term dataset of sea level measurements which are independent of land subsidence. Nowadays the long term sea level measurements could also be replaced by data from global sea level rise models. However, by applying these kind of models one should take into account the accuracy of these models. These models could contain large uncertainties if they are not calibrated on the project location. Therefore its use is neglected in this report.

In this case, there is no data available about land subsidence. Because the project location knows high levels of tectonic activity due to its position along the leading edge coast of Java, it is likely subsidence levels are small or even rise of the land could occur. In this report it is assumed that tectonic activities in the areas have a larger influence than absolute sea level rise on the vertical position of the harbour during its lifetime. Therefore it is chosen to neglect relative sea level rise in the design. However, to exclude its influence, it is recommended to investigate relative sea level rise.

B.5 RIVER

In this section the characteristics of the Serang River are analysed. It is split up into three parts. Part one discusses aspects that influence the river discharge to finally be able to evaluate on the discharge both quantitatively and qualitatively. The second part discusses the presence of sediment in the river. The third part addresses water quality.

DISCHARGE

In order to assess the discharge the extents of the river basin is analysed as well as the river works that may influence the discharge. Besides that, rainfall in the catchment area is studied.

RIVER BASIN CHARACTERISTICS

The Serang River basin extends over 246 km^2 and is located in the Kulon Progo district. Its upper tributaries flow from the Menoreh Hills which mark the border between Central Java and the Special Region of Yogyakarta. The lower tributaries, all joining the Serang River on its right bank, flow from low hills or are simple flat drains.

The upper river basin is mountainous with slopes often larger than 40% and altitudes exceeding 800 masl (meters above sea level). The mid catchment is less steep and generally ranges from 100 to 500 masl in altitude. The lower basin, located downstream from the town of Wates, is dominated by coastal floodplains 5 to 10 km wide, with slopes barely reaching 2%. The boundary with the Progo River basin in the east of the coastal plain is very flat and not clearly defined. Average annual rainfall reaches 2,800 mm in the upper basin and about 1,950 mm in the flood plain.

RIVER FUNCTIONS AND WORKS

Serang River water is used for several purposes. Its main purposes are irrigation and drinking water supply. There are few fish farming enterprises, but river based fishing is practiced mainly as a traditional way of life rather than as a source of income.

At the Ngrancah tributary, an upper western branch of the Serang River, a dam was built and inaugurated in 1996 called 'Sermo Dam'. It was built to answer to the high demand of fresh water in Kulon Progo, to prevent flooding of the Serang River and to reduce the environmental degradation in the surrounding area. The cover dam stores 21.9 million m^3 and has a crest elevation of 142 m. Estimates of its influence in flood reduction say 20% according the construction supervisor mr. Sony [Site Visit]. The sermo reservoir provides the irrigation system Kalibawang (6454 ha) with 0.12 to 1.5 m^3s^{-1} (Aryanti, 2014).

The Kalibawang system is the largest irrigation system in Kulon Progo Regency. The system consists of five irrigation schemes namely Kalibawang (2005 ha), Penjalin (652 ha), Papah (983 ha), Pengasih (2075 ha) and Pekik Jamal (739 ha). Pengasih and Pekik Jamal are supplied from the Serang River via two weirs shown in Figure B-17.

In total Kalibawang intake varies from 5,000 lps (liters per second) to 7,000 lps. The highest discharge of Kalibawang takes place in May or in the second planting season when irrigation requirement is high and water is available in the river. The lowest discharge of Kalibawang occurs in the midst of the dry season in July and August (Hussain, 2004).

The availability of water has major influence on cropping patterns. At the beginning of the rainy season, generally October or November, farmers predominantly crop rice as it can withstand excessive amounts of water. After four months, when rainfall decreases, farmers begin plant their second crop. If sufficient irrigation water is available, which in the Kalibawang system generally is the case, farmers grow rice. If not, other crops that need less water are grown. Four months later these get harvested and the third crop of the year is grown. This time of year is well into the dry season and crops are grown that require little water e.g. chili or soybean (Hussain, 2004).

The Kalibawang system shows a very good performance compared to the rest of java, especially Pekik Jamal. Almost the whole command area is cultivated three times a year, and almost all land is irrigated. Farmers in the Pekik Jamal scheme as well as in the tail of the Pengasih Scheme make use of groundwater for irrigation. The variation in groundwater however can be considered negligible (Hussain, 2004).

For drinking water purposes raw water is extracted from sermo reservoir (60 lps) as well as from Clereng spring (150 lps). The impact of drinking water extraction on the discharge of river Serang is however negligible relative to the irrigational demand. The local water providing company of Kulon Progo does furthermore not expect any shortages in drinking water supply capacity (Sukarelawanto, 2015). An overview of the Serang River basin and its major river structures is provided in Figure B-17.

RAINFALL

Rainfall in the Kulon Progo district is primarily characterized by seasonal monsoons. The south-east monsoon dominates the dry season, generally from the middle of May to October. This period is characterized by little rainfall, low humidity and cloudiness. The rainy season, generally taking place from November to April, is dominated by the north-west monsoon. It is the period of frequent and heavy rainfall, high relative humidity and cloudiness. More than 80 percent of annual rainfall falls in this period. Average monthly rainfall and the maximal daily rainfall data from the Kulon Progo station Kalibawang is shown in Figure B-15 (Hussain, 2004).

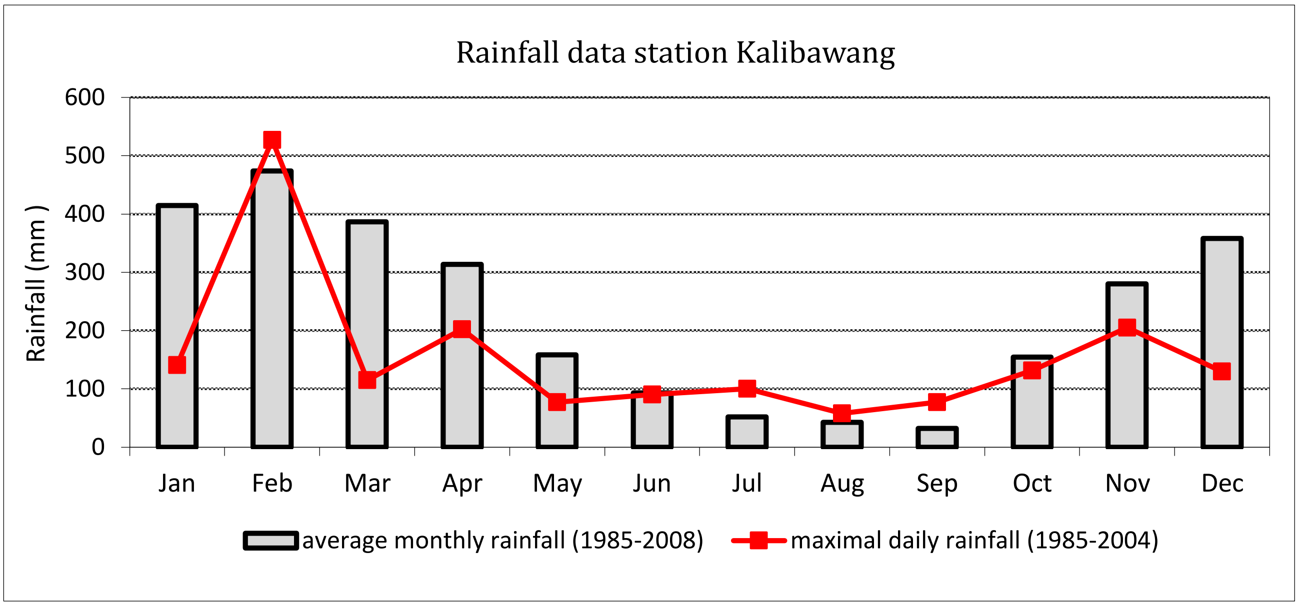


FIGURE B-15 RAINFALL IN THE SERANG RIVER BASIN SYSTEM

DISCHARGE CALCULATION

Discharge and irrigation intake was analysed on the basis of daily discharge data of Pekik Jamal weir and discharge station in the years 1995-2013. The catchment area that is covered by Pekik Jamal discharge station is about 200 km^2 (total catchment area is 281 km^2). Pekik Jamal weir is the last irrigation intake station and discharge station downstream of river Serang. It is the closest approximation for discharge conditions at the harbour mouth with real data.

The Serang River can be classified as a tropical pluvial stream with very low discharge in the dry season and abundant rainfall in the warm season. Its minimum can reach very low values. Combined with irrigation needs, this leads in 50 % of the time to zero flow down stream of Pekik Jamal weir. On average 163 days in a year all available discharge reaching Pekik Jamal weir is completely used for irrigation.

As can be expected with the local climate, the river has rather great variability of discharge during the year; a standard deviation of 12.56 m^3/s was found whereas the average discharge is 4.78 m^3/s downstream of the weir. The maximum discharge that was observed by the discharge station is 303.66 m^3/s. Figure B-16 shows the distribution of discharge over the year for Kalibawang station; it clearly indicates the dry and wet season.

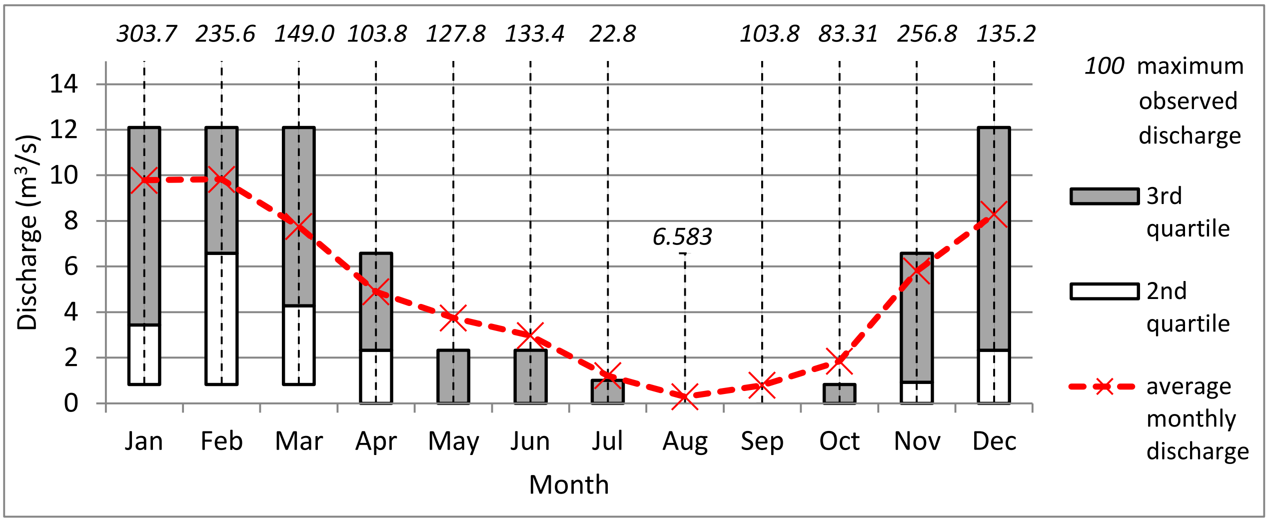


FIGURE B-16 DISCHARGE DATA OVER THE YEAR

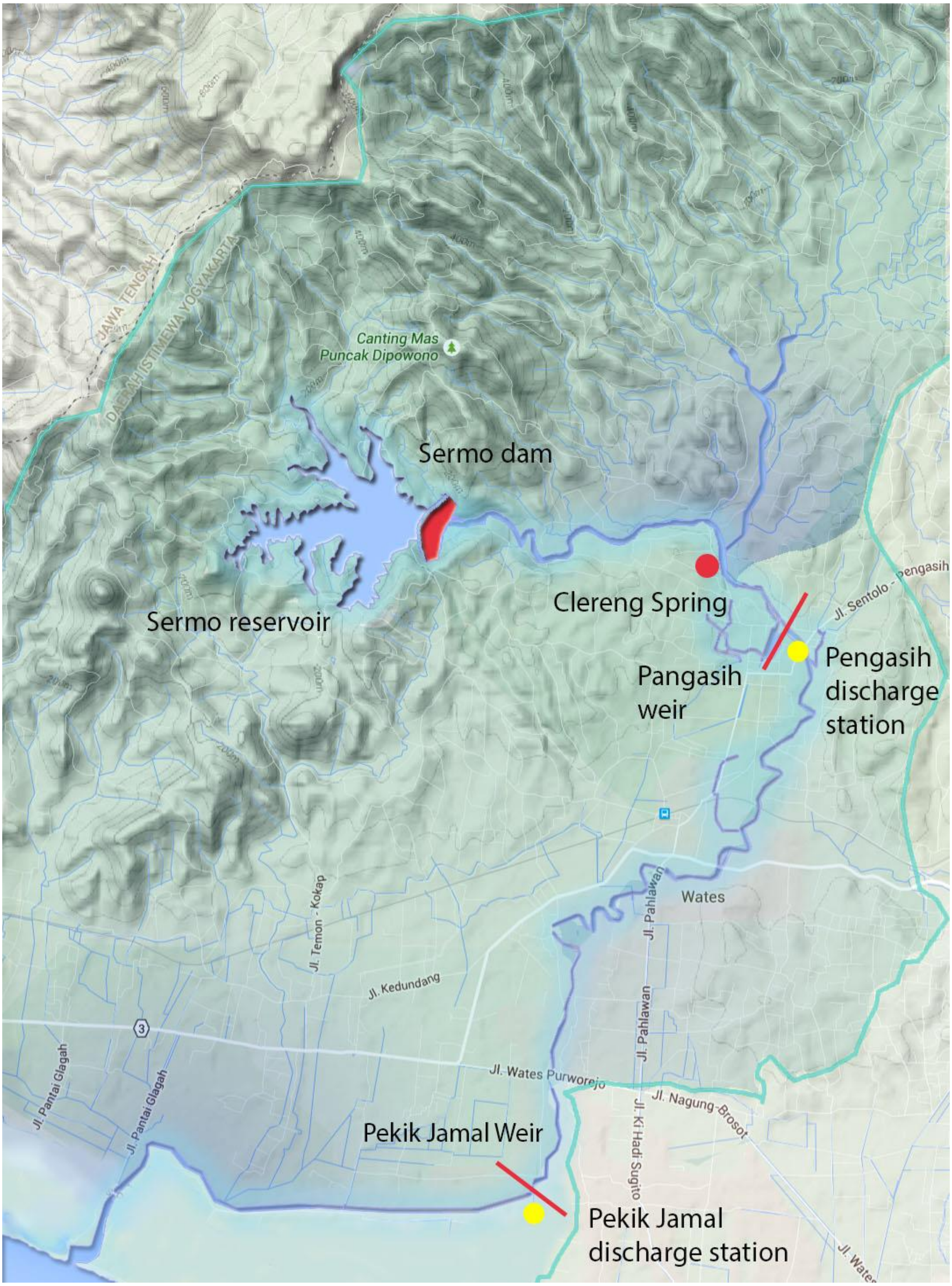


FIGURE B-17 SERANG RIVER BASIN AND MAJOR RIVER WORKS

The discharge also seems to follow the observed rainfall well which indicates an ephemeral stream which flows briefly in direct response to precipitation in the immediate vicinity. This is supported by the fact ephemeral streams go hand in hand with high zero flow index and discharge variance. Rainfall at Kalibawang station and discharge at Pekik Jamal weir in the year 2008 are shown in Figure B-18. In a hydrologic model rainfall could be coupled to discharge however in this report this is not done because of priority deliberations. It is recommended to do in future research.

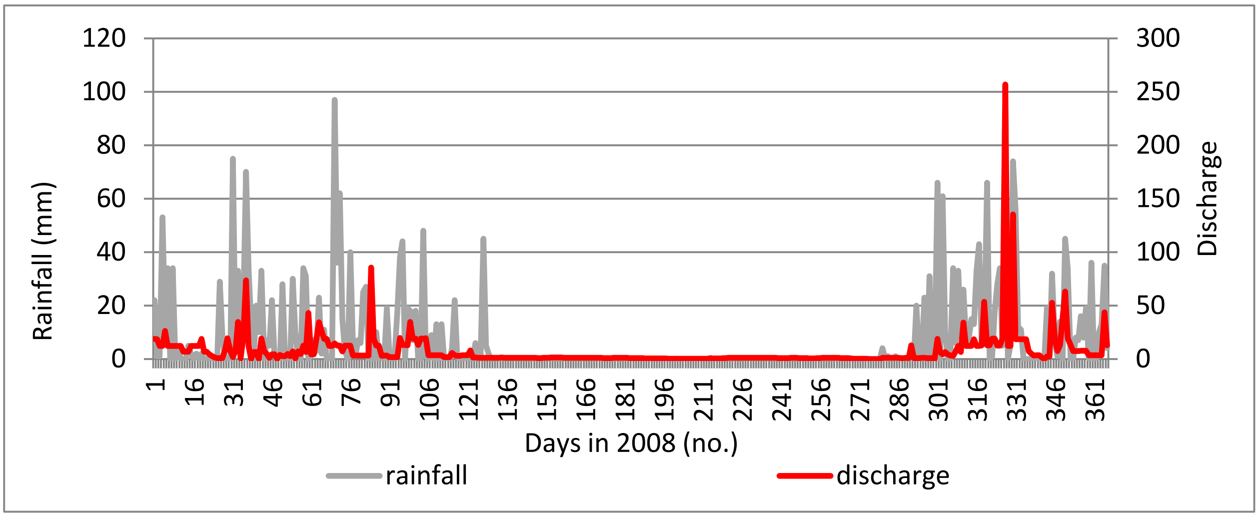


FIGURE B-18 RAINFALL AND DISCHARGE OVER THE YEAR

Hand in hand with high zero flow index and discharge variance. Rainfall at Kalibawang station and discharge at Pekik Jamal weir in the year 2008 are shown in Figure B-18. In a hydrologic model rainfall could be coupled to discharge however in this report this is not done because of priority deliberations. It is recommended river flows are driven by random events such as rainfalls, the discharge patterns observed in the Serang River in both wet and dry season, shown in respectively Figure B-19 and Figure B-20, do not follow clear logarithmic distributions. River flows in general are the realizations of a multiscale nonlinear dynamical system and therefore not necessarily need to be logarithmic distributed.

Firstly, the river is subject to long term trends such as climate change and El Niño which contributes to the non-stationarity of the data. Secondly the river changes naturally over time, processes such as erosion and deposition affect the flow regime of the river. And thirdly, next to natural change, structural change may be human-induced. In the twenty years of data the course of the Serang River has been altered strongly by human works (e.g. sermo dam was built, embankments were put into place, the river mouth was fixed). The river also is and has been affected by irrigation works. Because the river is quite intensively used for agriculture, this effect has been investigated. The correlation between irrigation intake at Pekik Jamal weir and discharge before the weir is shown in Figure B-21. High discharges occur mainly in the wet season where irrigation requirements are generally less. This is likely to be the reason for negative correlation between discharge and irrigation intake.

Compensating for the effect of irrigation in the data, low discharges would be generally higher and high discharges will barely be affected. As shown Pekik Jamal forms only a small part of irrigation intake from the Serang. When all Kalibawang system shows the same pattern as in Pekik Jamal the adaptation may be significant. Irrigation in the Pekik Jamal system is of the order of 1 m3/s and is about a tenth of the size of the total Kalibawang irrigation system. Adaptations of the order 10 m3/s therefore can be expected if the total irrigation system of Kalibawang is taken into account. The irregularity in the distribution for lower values of discharge can therefore be significantly affected. This is amongst other the reason for not including these values in the extrapolation for higher discharges. For the purpose of reliability in estimating likelihood of high discharges extrapolation is only applied for the higher values of discharges. The lower limit was selected in a way that the error in the logarithmic correlation stayed small.

TABLE B-15 EXTREME DISCHARGES IN THE WET SEASON (DECEMBER-MARCH) IN THE PERIOD OF 1995-2013

Start value bin	End value bin	Number of values in bin
103.5	104	1
105	105.5	1
109	109.5	1
110.5	111	1
118	118.5	1
135	135.5	2
148.5	149	1
165	165.5	1
182	182.5	1
217	217.5	1
235.5	236	1
246.5	247	1
303.5	304	1

TABLE B-16 EXTREME DISCHARGES IN THE DRY SEASON (MAY-OCTOBER) IN THE PERIOD OF 1995-2013

Start value bin	End value bin	Number of values in bin
103.5	104	1
105	105.5	1
109	109.5	1
110.5	111	1
118	118.5	1
135	135.5	2
148.5	149	1
165	165.5	1
182	182.5	1
217	217.5	1
235.5	236	1
246.5	247	1
303.5	304	1

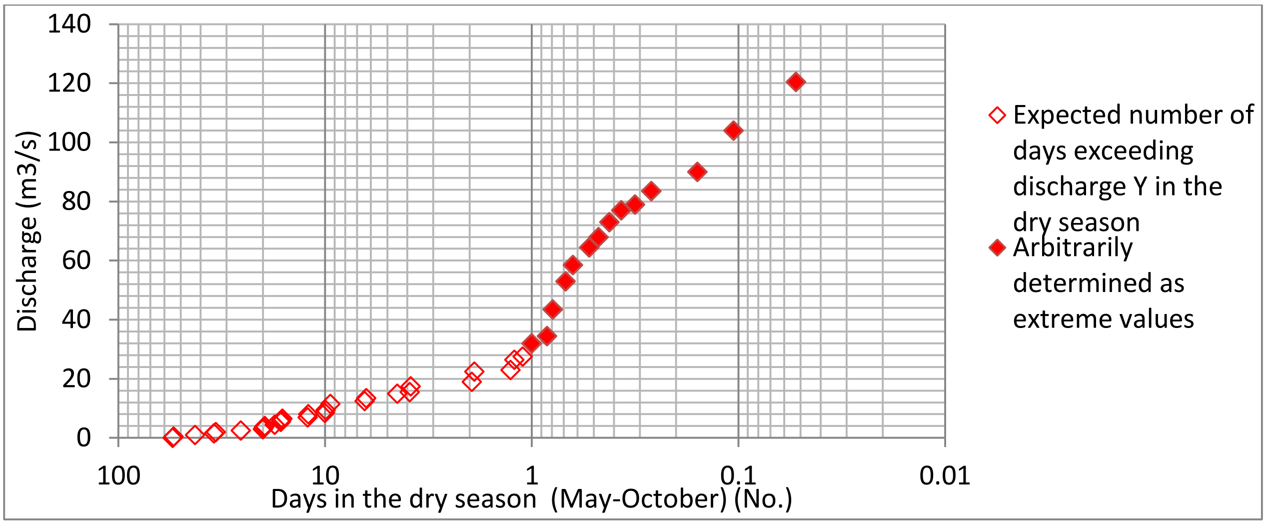


FIGURE B-19 DISCHARGE LEVEL AND EXPECTED EXCEEDENCE RATE IN DAYS PER YEAR IN THE DRY SEASON

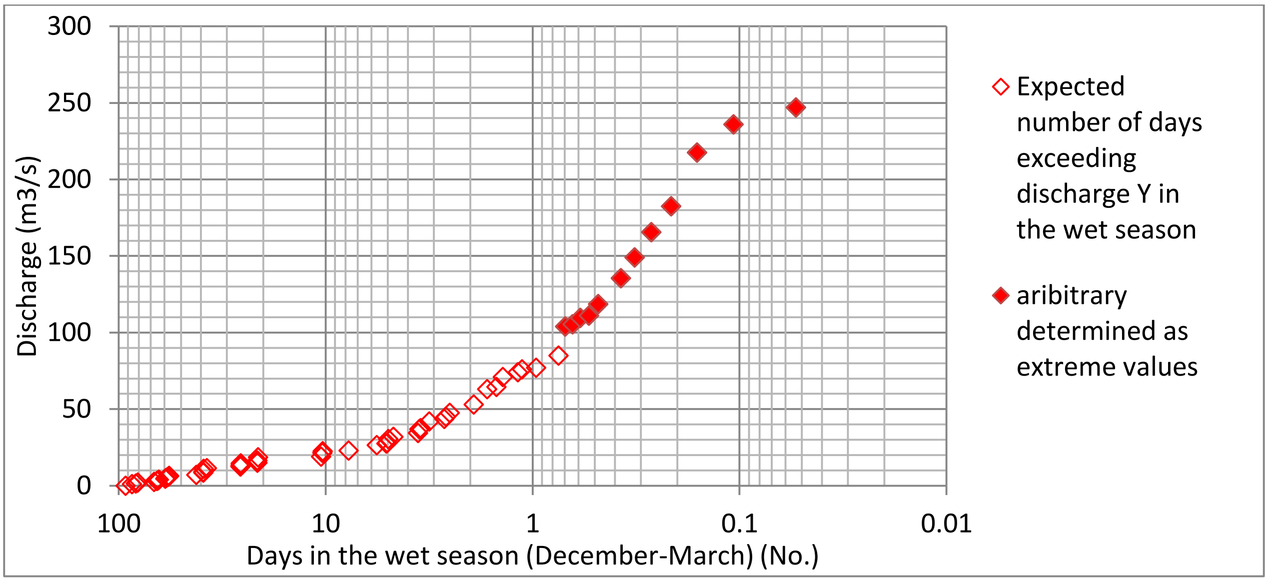


FIGURE B-20 DISCHARGE LEVEL AND EXPECTED EXCEEDENCE RATE IN DAYS PER YEAR IN THE WET SEASON

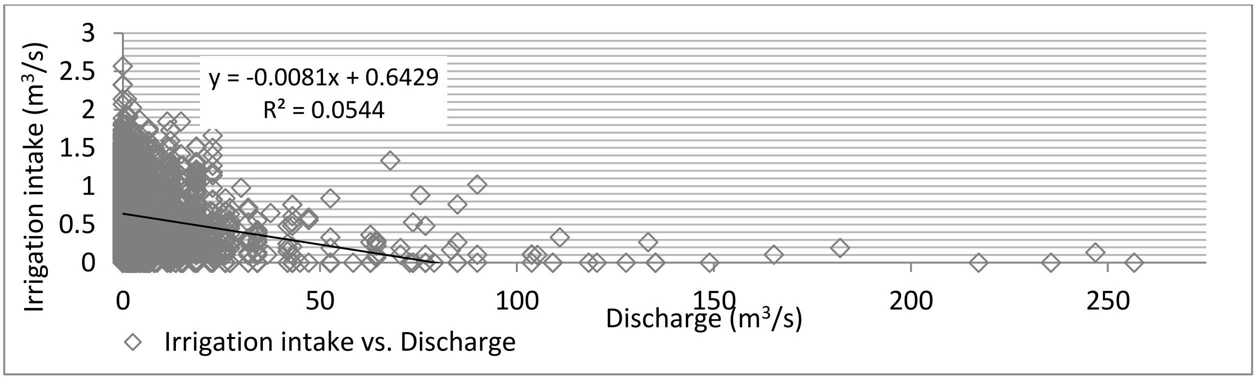


FIGURE B-21 CORRELATION BETWEEN IRRIGATION INTAKE AT PEKIK JAMAL WEIR AND DISCHARGE BEFORE THE WEIR

In order to better foresee flood events one should look at extreme discharge values. The thresholds for the extreme discharge values in the dry and wet season are arbitrarily chosen at 30 m3/s and 100 m3/s respectively. The number of observed extreme discharge values corresponding to these thresholds (Ns) is 0.75 per wet season and 1.1 per dry season on average.

Three distributions were fit to the discharge data: Exponential, Weibull and Gumbel. The distributions were chosen based on the locations of the data points. Table B-17 and Table B-18 show the discharge values calculated for various return periods with optimized parameters for the distributions for the wet and dry season respectively. Table B-17 also shows the discharge as retrieved from the java flood control sector project. As all the errors in the fit of distributions are of the same order it has been decided to continue calculations with the highest values.

TABLE B-17 ESTIMATED RIVER DISCHARGE PER RETURN PERIOD IN THE WET SEASON

Distribution type	Return period (days per year) (Qs) – Wet season						
	1/2	1/5	1/10	1/25	1/50	1/100	1/200
Exponential	122	182	227	287	332	377	422
Weibull	123	189	228	273	304	333	361
Gumbel	125	189	228	276	311	347	382
Highest Value	125	189	228	287	332	377	422
JFCSP	270	352	406	476	527	-	-

TABLE B-18 ESTIMATED RIVER DISCHARGE PER RETURN PERIOD IN THE DRY SEASON

Distribution type	Return period (days per year) (Qs) – Dry season						
	1/2	1/5	1/10	1/25	1/50	1/100	1/200
Exponential	61	88	108	134	154	174	194
Weibull	64	91	107	126	138	150	161
Gumbel	64	90	108	130	147	163	180
Highest Value	64	91	108	134	154	174	194

TABLE B-19 FIT OF DISTRIBUTIONS – PARAMETERS

Distribution type	Equation	Parameters		R^2-value	
		Dry season	Wet season	Dry season	Wet season
Exponential	$\mathrm{Qe} = \gamma + \beta\ln(\mathrm{Qs})$	γ = -28.87 β = 41.15	γ = -65.19 β = 76.99	0.9402	0.9598
Weibull	$\mathrm{Qe} = \gamma + \beta\{\left(-\ln\left(\frac{\mathrm{Qs}}{\mathrm{Ns}}\right)\right)\}^{\frac{1}{\alpha}}$	γ = 15.28 β = 55.56 α = 1.75	γ = 68.34 β = 100.0 α = 1.50	0.9825	0.9601
Gumbel	$\mathrm{Qe} = \gamma - \beta\ln(\frac{\mathrm{Ns}}{\mathrm{Ns} - \mathrm{Qs}})$	γ = 51.44 β = 23.81	γ = 130.1 β = 50.25	0.9779	0.9487

APPENDIX C: STAKEHOLDER ANALYSIS

C.1 STAKEHOLDER IDENTIFICATION

In this section a deep dive is taken into the stakeholders involved in the Glagah Jetty Project. Stakeholders are assessed based on their impact, their opportunities and their concerns. These characteristics yield a support factor, which can be interpreted as "the extent of support for successful completion of the Glagah Jetty Project". Corresponding general strategies have been devised in order to be able to leverage aligned interests or remove concerns.

These sections highlight the most important conclusions that can be drawn from the timeline and the stakeholder meetings. Stakeholders are categorised in:

- **Decision making stakeholders**
  These stakeholders are key to success.
- **High-impact approached stakeholders**
  These stakeholders have been engaged with in meetings and during presentations.
- **High-impact acknowledged stakeholders**
  Stakeholders that are of high-impact to success but are out of the scope of this research.
- **Other stakeholders**
  Relatively low impact stakeholders that will not have significant effect on the policy recommendations.

C.2 DECISION MAKING STAKEHOLDERS

Early stage findings indicate that the high-impact stakeholders for the Glagah Jetty Project are the Ministry of Public Works and Public Housing and the Ministry of Maritime Affairs and Fisheries.

Ministry of Public Works and Public Housing (Min. P.W.)
Menteri Pekerjaan Umum dan Perumahan Rakyat (PUPR)

Min. P.W.

Minister
Dr. Ir. M. Basoeki Hadimoeljono, M.Sc.

Impact	●●●●●	**Support**	−○●○○○+

General Responsibility	Through the Ministry's Balai structure it holds responsibility for the policy of the following aspects of public works: ▪ Maintenance and conservation ▪ Resources development ▪ Flood control ▪ Community development ▪ Information affairs
Organizational structure	**Former Nomenclature** Ministry of Settlements and Regional Development (1999-2000) Ministry of Settlements and Regional Infrastructure (2000-2004) Ministry of Public Works (2004-2014), after which it was combined with the Ministry of Public Housing In 2004, the Public Works division of the Ministry of Public Works and Public Housing was divided into 12 Balai organisations according to the principal: "one river, one plan, one integrated management". A Balai can be responsible for multiple river basins.

Impact on Glagah Jetty Project

- Min. P.W. is by many considered to be the decision-making stakeholder when it comes to further development of the breakwaters.

Opportunities and strategy

- The Glagah Jetty Project is a national prestige project that receives special attention and budget from the Indonesian President which is good for the Project's public image.
  Strategy: Lobby for higher budgets for the Ministry and make Parliament / President involved in this project to the highest extent.

Concerns and strategy

- The Ministry is aware of the Tanjung Adikarto harbour development paradox and is evaluating the proposition that expected prospects of economic activity in Kulon Progo justify large investments for the breakwater lengthening.
  Strategy: Research the economic viability of the harbour concept leverage its potential positive outcome
- Min. P.W. is not responsible for the Glagah Jetty Project as it originated as a desire from the Sultan and turned into a Min. M.A.F. project. Taking responsibility for the project involves significant risk. These risks will be measured at audits.
  Strategy: Stress and prove how the exposure to liabilities from unfinished breakwater construction outweighs the scenario of risks in projects aimed at breakwater repair and improvement.

Conclusion

Despite having constructed the current breakwaters, the Ministry is aware of the Tanjung Adikarto harbour development paradox and is evaluating the proposition that expectated prospects of economic activity in Kulon Progo justify large investments for the breakwater lengthening.

Ministry of Maritime Affairs and Fisheries

Menteri Kelautan dan Perikanan (KKP)

Min. M.A.F.

Minister

Minister Susi Pudjiastuti

Impact	●●●●●	Support	−○○○○●+

General Responsibility

- Planning and execution of programs
- Maintenance of property
- Governance of Ministry policies
- Monitoring of implementation of local projects and provision of guidance
- Execution of technical projects on a national level

Organizational structure

Since October 1999 the Ministry is divided into 14 departments on a national level. Several general directorates have been involved with the Glagah Jetty Project.

Impact on Glagah Jetty Project

- Min. M.A.F. is by many considered to be the decision-making stakeholder when it comes to further development of the harbour.
- Min. M.A.F. got involved through the Prov. M.A.F. department.

	- The project was initiated by the Sultan, however the responsibility of harbour development and coordination lies with the Ministry. - The costs of development of Glagah Harbour are partly borne by the Ministry.
Opportunities and strategy	- The Ministry has the opportunity to develop an economically viable harbour at the South Coast of Java for the first time in Indonesian History which is a boost for its public image. **Strategy**: Stimulate public awareness about this project and educate the local population about the Ministry's effort in developing this harbour. - The Glagah Jetty Project is a national prestige project that receives special attention and budget from the Indonesian President which is good for the Project's public image. **Strategy:** Lobby for higher budgets for the Ministry and make Parliament / President involved in this project to the highest extent. - The Ministry can earn money and knowledge from exploiting this harbour. **Strategy:** Stimulate research into harbour efficiency and development in order to gain maximum profit on long term development.
Concerns and strategy	- The Ministry has insufficient budget for as well as insufficient experience with construction of the breakwater. **Strategy:** Find government institutions such as Min. P.W. that are willing to cooperate and bear the costs.
Conclusion	Min. M.A.F. is aware of the financial gains, the boost in public and project image and advances technical know-how that come with development of the Tanjung Adikarto Harbour, however the Ministry remains hesitant in pushing forward the project as it is unsure whether or not the costs and the project are manageable. Therefore the Min. M.A.F. stakeholder position can be classified as uncertain.

C.3 HIGH-IMPACT ENGAGED STAKEHOLDERS

This section reports the stakeholders that have been actively engaged with during the research. The choice on the extent of the analysis is based on:

- The scope of the research;
- Perceived willingness of stakeholder to cooperate with the investigators.

Provincial Government of the Special Region of Yogyakarta, Department of Maritime Affairs and Fisheries

Dinas Kelautan dan Perikanan (DKP), Daerah Istimewa Yogyakarta (DIY)

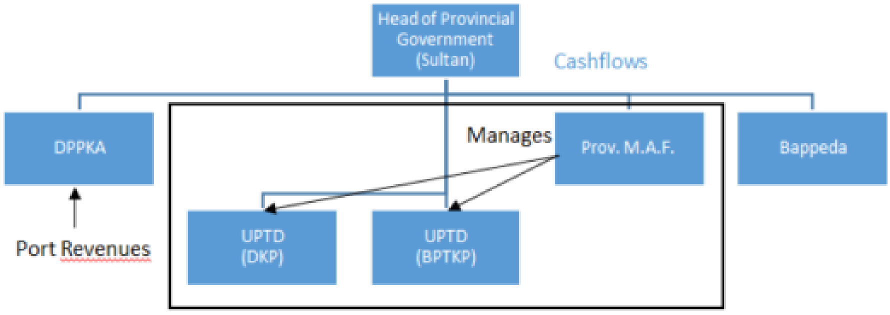
Prov. M.A.F.

Chair

Mr. Andung Prihadi Santosa

Impact	●●●●○	**Support**	−○○○○●+

General Responsibility	- Regulation - Supervision - Construction

	▪ Monitoring ▪ Regulation
Organizational structure	 The Prov. M.A.F. reports to the governor and has a seat in the planning meetings of the Provincial Planning Agency (Bappeda). The Prov. M.A.F. manages the UPTD, which is separated in two different entities. [Prov. M.A.F. 2]
Impact on Glagah Jetty Project	▪ Prov. M.A.F. is responsible for construction, maintenance and supervision of development of the Tanjung Adikarto Harbour. ▪ Prov. M.A.F. is responsible for maintenance of the navigation channel of the harbour. ▪ Prov. M.A.F. is formally responsible for protection of the navigation channel, i.e. the construction of the breakwaters.
Opportunities and strategy	▪ Develop a harbour that is of key importance for the shift to maritime developments. **Strategy**: Raise public awareness about Prov. M.A.F. activities to increase its support base. ▪ Be awarded higher budgets due to higher income from DPPKA (harbour tax revenues). **Strategy:** Involve Prov. M.A.F. in decisions that relate to maintenance costs in order to better estimate provincial incomes from the harbour.
Concerns and strategy	▪ Prov. M.A.F. does not have significant expertise with large-scale projects. [Prov. M.A.F. 1] **Strategy:** Assess Prov. M.A.F. abilities and consider cooperation with other institutions or national level support. ▪ Prov. M.A.F. is uncertain about the available budget for maintenance of the harbour. **Strategy**: Seek cooperation aimed at reducing maintenance costs of dredging works and other maintenance activities.
Conclusion	Despite the realization that Prov. M.A.F. is responsible for development of harbour activities and its implications for construction and maintenance, the department does not pro-actively engage with these activities because it lacks experience and sufficient budgets.

Serayu Opak River Basin Bureau

Balai Besar Wilayah Sungai Serayu Opak (BBWS SO)

Balai P.W.

Chair

Ir. Tri Bayu Adji, MA

Impact	●●●●○	Support	−○●○○○+

General Responsibility	Balai P.W., best characterized as a river basin bureau, holds responsibility for the following aspects of public works: ■ Maintenance and conservation ■ Resources development ■ Flood control ■ Community development ■ Information affairs Balai P.W. is responsible for "one river, one plan, one integrated management". [Start Up].
Organizational structure	A Balai P.W. is technically a national-level river basin bureau that has a work area of one or more river basins. BBWS SO manages the area from the Serayu River to the Opak river. It is represented at the Prov. Bappeda and the Water Council as well as with Prov. M.A.F. with regards to the Glagah Jetty Project. Balai P.W. decided to hire several consults to aid with the designs of the breakwaters.
Impact on Glagah Jetty Project	■ As executing party, Balai P.W. takes execution orders from Min. P.W. ■ Since Balai P.W. is a national level organisation, it is positioned relatively high on the hierarchy ladder. One could argue that this position gives power over other lower level organisations.
Opportunities and strategy	■ The Glagah Jetty Project is a national prestige project that receives special attention and budget from the Indonesian President which is good for the Project's public image. **Strategy:** Lobby for higher budgets from the Ministry and make Parliament / President involved in this project to the highest extent.
Concerns and strategy	■ Balai P.W. is not responsible for the Glagah Jetty Project as it originated as a desire from the Sultan and turned into a Min. M.A.F. project. Taking responsibility for the project involves significant risk. These risks will be evaluated at audits. **Strategy:** Stress and prove how the exposure to risks from previous breakwater construction of the breakwater outweighs the scenario of risks in projects aimed at improvement.
Conclusion	As executing party, Balai P.W. takes execution orders from Min. P.W., is therefore bounded by decisions on a national level but promotes the completion of the Glagah Jetty Project as the Balai P.W.'s interests in economic development are aligned with those of the province.

Provincial Government of the Special Region of Yogyakarta,

Provincial Planning Agency
Bappeda, Daerah Istimewa Yogyakarta (DIY)

Prov. Bappeda

Chair
Mr. Tavip Agus Rayanto

Impact	●●●●○	**Support**	−○○●○○+

General Responsibility	Coordination of general development in Special Region of Yogyakarta.
Organizational structure	Manages general planning and presides over meetings with representatives from departments within the provincial government and Balai P.W.
Impact on Glagah Jetty Project	The Prov. Bappeda is responsible for the guidance of other provincial departments in the process of shifting from an agricultural economy to a more maritime economy. Glagah Jetty is of key importance in this shift. [Prov. Bappeda]
Opportunities and strategy	▪ Prov. Bappeda can coordinate stakeholders and thereby succeed in its task to enable provincial departments to shift the Yogyakarta Province economy slightly towards maritime developments. **Strategy:** Inform Nat. Bappeda about necessity of an economic viability review for the Tanjung Adikarto Harbour.
Concerns and strategy	▪ There is a great concern that Prov. M.A.F will not be able to bear the costs for maintenance of the harbour and thus permission will not be granted. **Strategy:** Inform Nat. Bappeda about necessity of an economic viability review for the Tanjung Adikarto Harbour.
Conclusion	Prov. Bappeda can coordinate stakeholders and thereby succeed in its task to enable provincial departments to shift the Yogyakarta Province economy slightly towards maritime developments.

Government of Kulon Progo Regency,
Department of Maritime Affairs and Fishery,
Water Resources Benefit Section
Pemerintah Kabupaten Kulon Progo,
Departemen Kelautan dan Perikanan,
Sumber air Benefit Bagian

Reg. W.R.

Chair
R. Kuntarso

Impact	●●●●○	**Support**	−○○○○●+

General Responsibility	Water resources (dams, rivers, irrigation policy, flood protection, harbour development).
Organizational structure	In 1999, Law 22 was implemented and the new principle working method of the regency government (Kabupaten) is "Regional Autonomy". The new regency government is responsible for either a district or a town, bounded by governmental administration and can be seen as a regional service for public works. Coordination between regency governments is responsibility of the

	relevant Balai or multiple Balais. A regency government can initiate project work on water resources that are responsibility of higher level institutions but requires permission for this work (BCEOM, 2005). The Kulon Progo regency is one of 5 local governments in the Special Region of Yogyakarta: ▪ Yogyakarta City ▪ Bantul Regency ▪ Genung Kidul Regency ▪ **Kulon Progo Regency** ▪ Sleman Regency Kulon Progo's economy is mainly based on agriculture and therefore the water resources department hosts bi-weekly meetings with representatives from the public and private agriculture sector for planning.
Impact on Glagah Jetty Project	▪ Kulon Progo was chosen for the location of the harbour since reliable power and transport infrastructure was present at the time of assessment. ▪ The breakwaters followed the construction of river revetments that prevented floodings of the Serang river. The agricultural community relies on the structural integrity of the breakwaters and the revetments. ▪ Reg. Kulon Progo is the governmental institution that connects the local population to higher-level institutions and has the opportunity to boost the local economy. ▪ Reg. Kulon Progo is responsible for planning of the export facilities products from the to-be developed maritime economy. An airport for this purpose has been anticipated and Reg. Kulon Progo is responsible for land acquisition.
Opportunities and strategy	▪ Involve the local population in harbour activities, including the construction phase. **Strategy**: Inform the regency government about education programs and investment opportunities. ▪ Act as an exemplary region for harbour development at the Java South Coast. **Strategy:** Leverage the part of the population that embraces the shift to the maritime economy.
Concerns and strategy	▪ Some communities in Kulon Progo do not understand the positive consequences of the economic shift. **Strategy:** Support Kulon Progo Government in developing education programs and allocate budgets for lobbying among the community leaders. In this process, adapt plans to accommodate the integration of community traditions in these plans. ▪ Considering the lagging progress of the harbour construction, morale among the population and government departments has dropped. **Strategy:** Actively engage with the regency government to optimize the soft launch of the harbour [Prov. M.A.F. 2] in order to prevent further skepticism.

Conclusion	The regency of Kulon Progo is at the verge of entering a new era of economic progress, but relies on higher-level institutions for the investments in this future which causes an insecure future of the project.

Several consultants have worked on the construction of the breakwaters and on development studies for the port. In the current situation, the main consultant about breakwater extension is the University of Gadjah Mada (UGM), represented by Prof. Dr. Ir. Nur Yuwono.

University of Gadjah Mada (U.G.M.)
Universitas Gadjah Mada Indonesia (UGM)

Consultants

Lead Professor
Prof. Dr. Ir. Nur Yuwono, Dip. HE

Impact	●●●○○	**Support**	−○○●○○+

General Responsibility	Education and Research on a variety of sciences
Organizational structure	The university comprises of 18 faculties and 27 research centres and is the number one university of Indonesia. Relevant Departments: ▪ Center for Transportation and Logistics Studies (Pustral UGM) ▪ Center for Marine Resources & Technology Studies (Pustek Kelautan UGM) ▪ Research Center for Engineering Science (Pusat Studi Ilmu Teknik UGM, PSIT)
Impact on Glagah Jetty Project	▪ The Engineering Research Centre from UGM conducts research on the coastal dynamics and structural reliability of the breakwater. ▪ The lead consultant at Balai P.W is Prof. Dr. Ir. Nur Yuwono.
Opportunities and strategy	▪ The university has the opportunity to gain knowledge and deliver from the design, cconstruction and maintenance of the Glagah Jetty Breakwater and thereby advance its sciences. **Strategy**: Involve UGM in the whole design and execution process.
Concerns and strategy	-
Conclusion	The University is a neutral consultant and only leverages its insider knowledge to gain ground in the field of breakwater developments at the Southcoast.

Agency for the Assessment and Application of Technology,
Coastal Dynamics Research Center
Balai Pengkajian Dinamika Pantai, Badan Pengkajian dan Penerapan Teknologi (BPPT)

BPPT

Chair
Dr. Ir. Unggul Priyanto, MSc

Impact	●○○○○	**Support**	−○○●○○+

General Responsibility	Research to the field of coastal dynamics. Conducts numerical and physical modelling as it is in possession of a large physical scale model.
Organizational structure	Is part of a large government-owned research institute. Mission: To carry out government duties in the field of assessment and application of technology. The BPPT is a non-departmental government institution under the coordination of the Ministry of Research, Technology and Higher Education. [BPPT] Has been in contact with UGM in the past but is now secluded from many Glagah Jetty Project Stakeholders. [Int. Pres.]
Impact on Glagah Jetty Project	▪ The impact of BPPT research is perceived to be very low as relationships with other institutions are cold [BPPT].
Opportunities and strategy	▪ The BPPT has opportunity to research the breakwaters and finally calibrate its models with the data. **Strategy**: Allow BPPT to conduct research and to gather data.
Concerns and strategy	-
Conclusion	The University is a neutral consultant and only leverages its insider knowledge to gain ground in the field of breakwater developments at the Southcoast.

C.4 HIGH-IMPACT ACKNOWLEDGED STAKEHOLDERS

Since the policy analysis focuses on current issues, the following stakeholders have been identified but are considered out of the scope of this research to engage with.

Sultan of Yogyakarta
Hamengkubuwono

Sultan Yogyakarta

Sultan
BRM Herjuno Darpito

Impact	●●●●●○○	**Support**	−○○○○●+

General Responsibility	Wealth and protection of the Special Region of Yogyakarta, a special province on Java, officially ruled by the Republic Of Indonesia.
Organizational structure	Essentially the Sultan of Yogyakarta acts as the Governor of the province. The Sultan is president of several organisations, including the Yogyakarta Governor Expert Team.

Impact on Glagah Jetty Project	Sri Paku Alam VIII, Sultan of the Special Province of Yogyakarta from 1988 to 1998, had the “gut feeling” that a harbour district should be created at the southern coast of the Yogyakarta Province. A feasibility study for the harbour district was not conducted but the promises envisaged qualified as sufficient for further ordering of the creation of this harbour district. [Prov. M.A.F. 1]
Opportunities and strategy	▪ The Sultan has the potential to realize Yogyakarta’s Province new path to prosperity: from agriculture to maritime developments. This shift potentially gives the region an economic boost. [Balai P.W.] **Strategy**: Inform the Sultan of current developments and leverage the Sultan’s authority to incentivize investments in a maritime economy.
Concerns and strategy	▪ The new Sultan has expressed himself less in favour of the Glagah Jetty Project than the former Sultan. **Strategy:** Efforts should made trying to convince the Sultan to increase the Glagah Jetty Priority. The Sultan can be effective in bringing together the stakeholders, even when interests are not aligned.
Conclusion	The sultanate initiated development of the harbour but is now reluctant to support it. The Sultan’s authority should be leveraged to improve motivation levels for harbour development.

National Planning Agency

Bappeda

Nat. Bappeda

Chair

Impact	●●●●●	Support	−○○○●○+

Impact on Glagah Jetty Project	▪ The natinoal Bappeda considers the Glagah Jetty Prject of national importance. ▪ Nat. Bappeda has to manage the planning between Ministries and is therefore dependant of agreements between Min. M.A.F. and Min. P.W.

Ministry of Finance

Kementerian Keuangan

Min. Finance

Minister

Bambang Brodjonegoro

Impact	●●●●●	Support	−○○●○○+

Impact on Glagah Jetty Project	▪ All Ministries require authentication from the Ministry of Finance fro financial support, meaning that audit commissions have to be positive for budget allocations.

C.5 OTHER STAKEHOLDERS

Other stakeholders are parties that the investigators have not actively engaged with directly.

Residents

Residents

Local Village Officer
Connected to Reg. W.R.

Impact	●●●○○	**Support**	−○○○●○+

Impact on Glagah Jetty Project	▪ The local population has demanded from the Kulon Progo Government to fill the sink holes which are present on their agricultural land. These sinkholes are filled with sand that is being dredged in the navigation channel and in accretion zones close to the breakwater.

Investors

Investors

Representative
No consortium yet

Impact	●●○○○	**Support**	−○○○○●+

Impact on Glagah Jetty Project	▪ The investors recognize the potential of the Tanjung Adikarto harbour. Therefore they see the harbour as a good investment. This potential is enlarged by the plans to build an airport close to the harbour.

Construction Companies

Constr. Comp.

Representative
Connected to Balai P.W.

Impact	●●○○○	**Support**	−○○○●○+

Impact on Glagah Jetty Project	▪ Construction companies are responsible for the construction of the structures in and around the breakwaters. However the main question for these companies is if they are able to construct such structures in such a rough environment.

Eco-oriented Non-Governmental Organizations

Eco NGOs

Representative
Connected to Balai P.W.

Impact	●●○○○	**Support**	−●○○○○+

Impact on Glagah Jetty Project	▪ Eco-orietned NGOs have specifically asked to keep the sediment gained from dredging activities within the project site and not to transport it to any other sites [Prov. M.A.F. 3].

Local fishermen

Fishermen

Representative
Connected to Reg. W.R.

Impact	●○○○○	**Support**	−○○○○●+

Impact on Glagah Jetty Project	▪ The local fishermen are very supportive of the project as fishing is their main income. Nowadays crossing the waves with their small fishing boats is very hard. The harbour will ensure a safe entrance to the sea.

Tourism Industry

Tourism Ind.

Representative
Connected to Reg. W.R.

Impact	●○○○○	**Support**	−○○○●○+

Impact on Glagah Jetty Project	▪ The lagoon west of the breakwaters is a very attractive place for tourism. The future of the lagoon is dependent of the Glagah Jetty Project. The breakwaters it selves are also an attraction for tourists.

Farming Industry

Farming Ind.

Representative
Connected to Reg. W.R.

Impact	●○○○○	**Support**	−○○○●○+

Impact on Glagah Jetty Project	▪ The farming industry supports further development of the Kulon Progo Region. This development stimulates the economy and makes room for agricultural upscaling.

Remaining stakeholders to be considered are:

|5| **General Non-Governmental Organizations**
The Indonesian culture is very open to establishment of NGOs. The impact of NGOs differs largely and cannot be easily measured. [Prov. M.A.F. 2]

|6| **Water council**
The water council is a provincial-level taskforce that combines knowledge from the private and public sector about water resources in the regions. Representation in the water council is 50% public and 50% private. [Reg. W.R.]

|7| **Provincial Irrigation department (Subdin Pengairan Prov. DIY).**
The provincial Irrigation department commenced the fixation of the Serang River mouth and will be in favour of keeping intact the river revetments that have been constructed years ago.

APPENDIX D: PROGRESS-IMPEDING ISSUES RAISED BY STAKEHOLDERS

D.1 BREAKWATER CONSTRUCTION STOP

According to many parties [Balai P.W. 1] [Reg W.R.] [Prov. M.A.F. 2], the Glagah Jetty extension stop is not an engineering problem but it is a political problem. A study to the political structure behind the Glagah Jetty developments would be very interesting and helpful in further developments.

The reason for the construction stop of the eastern breakwater in 2012 has been disputed by the involved parties.

SCENARIO 'MINISTRY OF FINANCE FORESEES DEFICITS'

This scenario has been stated by Public Works [Balai P.W. 2], however these conclusions have been disputed by Prov. M.A.F [Prov. M.A.F. 1].

- In 2012, the Tanjung Adikarta harbour was not operational and the navigation channel and harbour basin were inaccessible for a large period.
- The feasibility study conducted by Prov. M.A.F. projected a return period of 8 years [Prov. M.A.F.], however considering the situation the Min. Fin. questioned the accuracy of this research.
- The Min. Fin. investigates if the involved governmental institutions have sufficient buffer budgets to cover the expenses of maintenance and operations and concludes these budgets are insufficient at Min. M.A.F. and especially at Prov. M.A.F.
- Since no other institutions other than Balai P.W. can carry the burden of maintenance and operation costs of the breakwaters, the Min. Fin. concludes that the lacking maintenance will be a showstopper for port development and that the BUMD (and parent company BUMN) will not earn any revenue in the near future.
- With no port development foreseen in the near future, the Min. Fin. decides to halt further lengthening of the breakwater since it cannot justify expenditure of public budget on unviable projects.

Further lengthening of the breakwaters can be executed as soon as the Prov. M.A.F. proves to have sufficient budget for adequate maintenance works.

SCENARIO 'MINISTRY OF MARITIME AFFAIRS REFUSES MOU'

This scenario has been stated by Public Works [Balai P.W. 1] and confirmed by Reg. W.R. [Reg. W.R.], however these conclusions have been partially disputed by Prov. M.A.F [Prov. M.A.F. 1].

- When in 2005 the construction of the breakwaters commenced, the responsibility of building revetments/breakwaters further than the coastline was transferred from Prov. M.A.F. to Balai P.W. This was due to two main reasons:
  1) The Prov. M.A.F. has insufficient budget for construction of breakwaters.
  2) The Prov. M.A.F. has little to none experience with projects of that magnitude.
- The designs for the breakwaters have been adapted over time, with multiple designers and consultants involved, which resulted in an uncoordinated construction.
- When, according to plan, the breakwaters were constructed up to the level of the 2012 design, the breakwaters proved insufficiently effective for wave breaking.
- An extension had to be build, but legal due diligence required more formal letters and agreements on a national level since the breakwater extensions had little common ground with Balai P.W. core responsibility of Serang River mouth fixation.

- When Min. P.W. proposed a Memorandum Of Understanding (MOU), this proposal was turned down by Min. M.A.F. for reasons unknown to date, even within Prov. M.A.F.
- Ever since, Balai P.W. has been awaiting permission in any legal form to continue construction plans.

As testified in the stakeholder meetings, speculation is that there have only been objections for the MOU by one officer at the office of the General Directorate of Caught Fish of Min. M.A.F. Further lengthening of the breakwaters can be executed as soon as the Min. M.A.F. signs the MOU, properly assigning responsibility to Min. P.W. [Balai P.W. 1].

SCENARIO 'MINISTRY OF PUBLIC WORKS IS AUDITED'

This scenario has been stated by Bappeda. [Prov. Bappeda] and disputed by Prov. M.A.F and Balai P.W.

- The Min. P.W. does regular audits on projects performed by its departments. The Glagah Jetty had been subject to an audit in 2012.
- When in 2012 the Balai P.W. wished to pursuit its construction goals on the eastern breakwater, the Min. P.W. stopped execution.
- The conclusions of the audit were that Balai P.W. had gone far beyond its responsibility without proper legal justification. Upon these conclusions, the Min P.W. ordered the immediate halt of construction.
- The Provincial Governments have always been in favour of signing any agreement that clearly makes a distinction in responsibilities.

Further lengthening of the breakwaters can be executed as soon as the legal due diligence has been done. This can potentially be achieved by the establishment of an MOU between Min. M.A.F. and Min. P.W.

SCENARIO 'MORE RESEARCH REQUIRED'

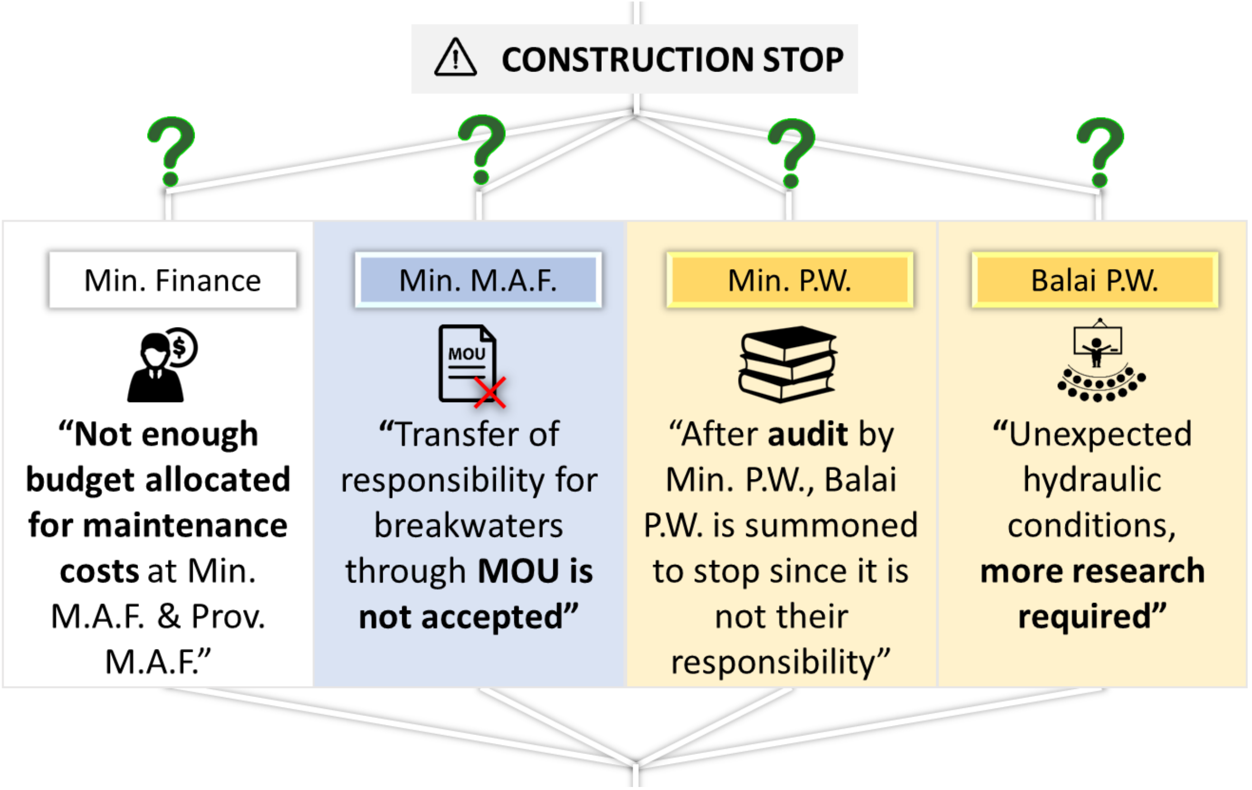
This scenario has been stated by Prov. M.A.F. [Prov. M.A.F. 1] and potentially agreed upon by Prov. M.A.F.

- In the past designs of the breakwaters, the oceanic conditions at Glagah Beach have been underestimated.
- Halfway construction in 2012, it is concluded that the construction process is greatly complicated by the underestimated sedimentation issues in the Serang River estuary.
- In order to be better prepared for future construction, Balai P.W. decides to put a hold to current breakwater lengthening and to consult with new researchers about the effects of dredging around the breakwater.
- In general it can be concluded: more research was required.

Further lengthening of the breakwaters can be executed as soon as the hydraulic conditions have been verified and sedimentation issues have been analysed and put in perspective of future breakwater designs.

CONCLUSION

The following scenarios can be dissected from the explanations given above.



“**Not enough budget allocated for maintenance costs** at Min. M.A.F. & Prov. M.A.F.”

“Transfer of responsibility for breakwaters through **MOU is not accepted**”

“After **audit** by Min. P.W., Balai P.W. is summoned to stop since it is not their responsibility”

“Unexpected hydraulic conditions, **more research required**”

FIGURE D-1 SCENARIOS DISSECTED FROM CONSTRUCTION STOP ANALYSIS

The investigators consider some elements of the scenarios likely and based on verification discussions and historical facts, it can be concluded that the following scenario is true:

“Since there is no MOU on a national level, legal responsibility for maintenance has not been discussed and constructing further would not pass the Balai P.W. audit. Due to the construction stop, the maintenance costs have risen unexpectedly high.”

D.2 LACK OF ADEQUATE DATA COLLECTION AND SHARING

A major issue in the design of the Glagah Jetty is the absence of accurate data and statistics for wave and wind climates. This is acknowledged by the current consultant [UGM]. At the time of writing, the following key information is absent or not shared:

- Front-end engineering guidelines for investment-maintenance ratios based on Indonesian standards for breakwater design;
- Location-specific near-shore wind and wave data and more recent offshore wave data;

SUBISSUE 1: COLLABORATION ISSUES

The following research organizations have conducted experiments, numerical modelling and or physical modelling research on topics related to the Glagah Jetty Project:

- BPPT: Physical modelling is executed in a large laboratory with a scale model of 1:68. The model has been constructed in 2012 and has been used ever since. The newest (2013) extension designs by Prof. Nur Yuwono have not been scaled yet. In addition, BPPT constructs numerical models which are calibrated with physical data.

- UGM: Academic papers on the effects of coastal interruption and structural reliability of the breakwaters. As a consultant to Balai P.W., Prof. Nur Yuwono has created designs for the breakwaters that have not been discussed with BPPT.
- Prov. M.A.F. conducts specific fishing innovation related research focused on the Tanjung Adikarto Port.

Other national level and provincial level institutions have shareable knowledge, but for cultural reasons this information is not easily shared. This resulted in the fact that current building plans for innovation research centres by BPPT and Prov. M.A.F. are now on adjacent properties, but no cooperation strategy has been devised at the time of writing.

Despite attempts to collaborate with other organisations and universities, the information gathered from the BPPT experiments has not been shared effectively, let alone with other institutions. BPPT and UGM have been working together with the in the past, however the last time these parties spoke was about 10 years ago [BPPT] [UGM].

The severity of the lack of collaboration can be characterized by the fact that at least some involved workers at Balai P.W. were not aware of BPPT experiments on the Glagah Jetty Project.

SUBISSUE 2: LACK OF FRONT-END ENGINEERING GUIDELINES

As described in the Design Report (Design Report chapter 2), a solid FEED study can be performed for breakwater extension and construction projects. However, the construction standards and costs in Indonesia that allow for the front-end estimations of project costs and construction methodologies have not been established. Having these standards available would be a great improvement to engineering times across the country, especially when it comes to sharing knowledge. With these standards, better trade-offs can be made between investment and maintenance costs.

SUBISSUE 3: INSUFFICIENT WAVE DATA

Due to the lack of sufficient data for reliable designs, the following issues have been encountered in the design of the breakwaters.

- As a consequence of unrealistic estimations in the past, significant design wave heights have unexpectedly risen from 3 to 5.8 m.
- The origin of the available wave data is located far from the Glagah Jetty site. This resulted in wave direction and height approximations that have not been verified so far, especially since wind set-up data is equally unreliable.

The lacking data resulted in a workflow that is referred to as ‘review upon review upon review’ [UGM] and caused uncertainty regarding the capabilities and responsibilities of the stakeholders.

CONCLUSION

It can clearly be stated that effective research collaboration and data and statistics gathering should be re-evaluated. Large research initiatives and results remain behind closed doors, despite being extremely valuable in some occasions. This led to the common belief among the stakeholders that Project Yogya’s destiny is “review upon review upon review”.

D.3 LACK OF SAFE, DURABLE AND SUSTAINABLE MAINTENANCE DREDGING

Current dredging operations are currently aimed at:

|8| Enabling safe navigability in the navigation channel;
|9| Enabling safe navigability in the harbour basin;
|10| Removing accreted sediment bulks near the toe of the eastern breakwater.

The dredged materials are currently being disposed in onshore locations [Reg. W.R.] [Prov. M.A.F. 1]. A missing, fourth goal should be:

|11| Nourishing of erosion zone west of the western breakwater, as depicted in the figure below. See chapter Design Report Chapter 3 for elaborate discussion about coastal erosion.

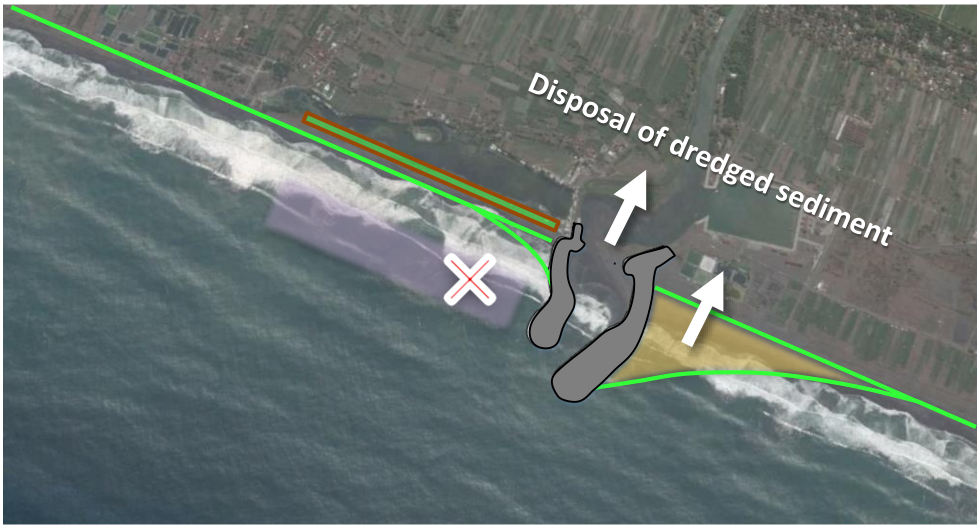


FIGURE D-2 LOCATIONS OF DISPOSAL AND LOCATION OF LACKING NOURISHMENT

SUB-ISSUE 1: KPI CONSTRAINTS

Eco-oriented NGOs have specifically asked to keep the sediment gained from dredging activities within the project site and not to transport it to any other sites [Prov. M.A.F. 3]. These demands follow from nation-wide regulation that prohibits any sand removal from the Javanese coastline. The accountable party for dredging activities of the navigation channel and the harbour basin, Prov. M.A.F., meets the demands from the NGOs and abides the law and this has unfortunately resulted in a sub-optimal situation for dredging operations. It should be noted that Prov. M.A.F. is aware of this situation, which can be best characterized as:

- First of all, it is acknowledged by Prov. M.A.F. that the coastal erosion zone needs to be refilled with sand from the estuary and from the accretion zone.
- When this process started, Prov. M.A.F. knew of an upcoming audit. At the audit there would be an investigation into the dredging activities.
- Since Prov. M.A.F. disposed sand at the coastal erosion zone, there was fear of not being able to prove the sand dumping in full as the nourishment would have also partly eroded.
- The Key Performance Indicators (KPIs) were, and still are, set to measure the amount of disposed sand and its magnitude in relation to the dredged sand volumes.
- The KPIs that would be leading in the audit and would indicate potential corruption as sand would get ‘lost’.
- Legal consequences would be imminent. Prov. M.A.F. decides to dispose sand in onshore sink holes.

This situation resulted in a sub-optimal policy for nourishment dredging as the coastal erosion zone kept, and still keeps, eroding.

SUB-ISSUE 2: SOCIALIZATION RESPONSIBILITY

The dredged sand is currently being transported by a pipeline system to the surrounding areas of the Tanjung Adikarto Port. Reason for this is a request by the local village officer that has been granted by Prov. M.A.F. 'Socialization', as this is called, is one of the responsibilities of Prov. M.A.F.

In the past, Prov. M.A.F. has often discontinued projects that were shut down due to problems with Prov. M.A.F.'s socialization duty. Information about these projects was not provided adequately to the NGOs and other stakeholders [Prov. M.A.F. 3].

Although socialization can be an important responsibility, it should be ranked lower on the list of priorities when compared to the coastline stability, which is currently not the case.

6.7.1 SUB-ISSUE 3: DREDGING EXECUTION PROBLEMS

In addition to policy issues regarding maintenance dredging, findings indicate a series of execution issues [Prov. M.A.F. 2] [Prov. M.A.F. 3].

- There are currently 4 dredgers assigned to dredging activities at the Tanjung Adikarto district.
- The dredging equipment currently used in the harbour is not technologically advanced and the staff might be not sufficiently educated.
- For example, the dredging vessels have selectively cut too deep in the harbour basin. This resulted in 'dredging holes' and its cutting has caused the loss of more time before the completion deadline.

It can be stated that the lack of education, experience and proper material causes irregularities in dredged areas. This implicates a culture where windows of dredging opportunities cannot be met and thus safety is at risk. A direct example of this effect is the observation that the harbour basin was unevenly dredged which could cause risk to vessel stability.

CONCLUSION

Due to sub-optimally formulated KPIs, a dominant socialization responsibility and issues with the execution of dredging activities it can be concluded that Prov. M.A.F. cannot guarantee the a durable, sustainable and safe dredging policy, let alone its governance.

D.4 EXECUTION ISSUES

Major execution issues were revealed in the timeline analysis as well as the stakeholder meetings. Many issues have been acknowledged by both Balai P.W. and its consultants [UGM] [Balai P.W.]. The highest impact issues are presented in this section.

SUBISSUE 1: LITTLE KNOWLEDGE OF IINTEGRAL DESIGN

The Southern coast of Java is considered to be one of the most challenging coastlines worldwide for development of safe harbours. Stakeholder meetings indicate that the Tanjung Adikarto Harbour is the eighth harbour at the Southern coast and prior to its construction the seven reference projects showed severe issues with sedimentation [UGM]. It can be argued that hierarchical pressure was at the basis of the decision to develop the Tanjung Adikarto Harbour [Prov. M.A.F. 1]. Regardless of the cause, it can be stated that Balai P.W. had insufficient knowledge of how to integrate a breakwater design in a difficult dynamical system as present at the Serang River mouth. The consequences of this lacking knowledge in integral design were harmful for many segments of the breakwater.

|12| A major flaw in the integral design of the Glagah Jetty Project is the fact that the western breakwater was constructed first, causing sediment to accrete against it.

|13| Construction of the breakwater was initially started without expert supervision. The construction method was not adapted be flexible for storm scenarios. Upon occurrence of a storm, the core of the breakwater collapsed [UGM] [Balai P.W. 1s].

More in-house knowledge or consultation could have prevented major issues.

SUBISSUE 2: SINGLE-YEAR BUDGETS

Due to this budgeting framework, full technical designs could not be constructed and compromises had to be made regarding the length and height of the breakwater. This resulted in relatively high construction costs per year and significant repair costs in later stages.

In addition to inefficient execution, the engineering designs have been adapted to meet the requirements of Balai P.W. budgeting framework [Start up Presentation].

SUBISSUE 3: LENIENT SUB-OPTIMAL EXECUTION SUPERVISION

A series of issues arose during construction of both the eastern and the western breakwater. A more elaborate analysis has been given in the Analysis Report. This section discusses a two typical examples.

|14| The construction company has wrongly placed the breakwater toes.

|15| Even though construction has started, Balai P.W. perceives that the construction company is unable to construct the heavy tetrapods required in the design.

CONCLUSION

Single-year programs, a lack of in-house knowledge among the executing governmental institutions and lenient execution supervision have resulted in major issues for the Glagah Jetty Project. Although the hiring of external consultants shows good progress at the ministries, the solution has not yet been found.

APPENDIX E: STAKEHOLDER MEETINGS

From the preliminary examinations of Part I, the investigators conclude that the project progress of Glagah Jetty is heavily impeded by policy and governance-related issues. The following series of findings has emerged from Part I:

|1| The breakwaters are unfinished.
|2| The segments of the breakwaters that are currently in place have low structural reliability.
|3| The harbour area and the navigation channel are under development and the initial planning goals have not been achieved.
|4| The construction budgets allocated in the past for the breakwaters of the Glagah Jetty Projects were limited and scaled to allow for 1-year construction and design programs.
|5| In the breakwater design, the costs for investments are relatively low compared to the maintenance budgets.
|6| Many public and private companies, institutions and organisations hold a stake in the Glagah Jetty Project.
|7| It is unclear what governmental or private organisation is responsible for the continued construction of the Glagah Jetty.
|8| Considering the current budget overrun and the status of the Tanjung Adikarto Harbour, the feasibility of a properly-functioning harbour is questioned by many stakeholders.

Stakeholder meetings were executed to discuss these issues. The statements gathered in this appendix are based on 13 stakeholder meetings. The contact information of these stakeholders have been mostly forwarded to Project Yogya by Balai P.W. An approximation of the truth has been found by comparing statements from interviewees and by literature verification. In order to remove the language barrier, at all times a translator was present for translations from English to Bahasa Indonesia. It should be noted that the translator was employed by Balai P.W., but pledged to translate neutrally to the best of his or her ability. In case the translator wished to be involved in the discussion, permission was requested and granted when appropriate.

For the purpose of source protection, the meeting discussions have been anonymised. However, a complete list of the persons that Project Yogya has engaged with is given. Should one have any interest in linking a certain statement to a certain individual, one is advised to contact the investigators with the contact information provided in this report. In consultation with the stakeholder, an assessment will be made of the necessity of revealing stakeholder source information.

The 13 stakeholder meetings are listed below:

|1|	**Dialogue about Breakwater integrity and redesign**	[UGM]
|2|	**Glagah Jetty Project Site Visit**	[Balai P.W. 1]
|3|	**Exploratory dialogue about floods, dredging and the involvement of Kulon Progo Regency**	[Reg. W.R.]
|4|	**Start-up meeting**	[Start-up]
|5|	**Exploratory Dialogue about harbour development**	[Prov. M.A.F. 1]
|6|	**Political structure dialogue about Glagah Jetty development and funding**	[Balai P.W. 2]
|7|	**Exploratory dialogue about Glagah Jetty research and lengthening**	[BPPT]
|8|	**Exploratory dialogue about harbour planning and feasibility**	[Prov. Bappeda]
|9|	**Elaborative dialogue about harbour development**	[Prov. M.A.F. 2]

|10| **Elaborative dialogue about harbour development 2** [Prov. M.A.F. 3]
|11| **Internal meeting about research results** [Int. Pres.]
|12| **First Glagah Jetty and Tanjung Adikarto Symposium** [Symposium]
|13| **Meeting with Minister of Public Works** [Minister]

Complete list of representatives that Project Yogya has engaged with, in alphabetical order:

|1| Ir. Aloysius Bagyo Widagdo, PhD (Mr. Warman), BPPT
|2| Dr. Ir. Arie Setiadi Moerwanto, MSc. (Mr. Arie), Min. P.W.
|3| Dr. Ir. M. Basoeki Hadimoeljono, M.Sc. (Mr. Basoeki), Min. P.W.
|4| Mr. Cahyo, Bappeda
|5| Mr. Catur Nur Amin APi, MMA (Mr. Catur), Prov. M.A.F.
|6| Dr. Ir. Dinar Catur Istiyanto, M.Eng (BPPT)
|7| Mr. Eco, BPPT
|8| Mr. Enyo Riso, BPPT
|9| Mr. Finning, BPPT
|10| Ir. Hanugerah Purwadi, MT (Mr. Hanung), Balai P.W.
|11| Ir. Moch. Silachoeddin, ME (Mr. Sila), Balai P.W.
|12| Dr. Ir. H. Muslikh, M.Sc., M.Phil. (Pak Muslikh), UGM
|13| Prof. Dr. Ir. Nur Yuwono, Dip. HE (Prof Nur Yuwono), UGM
|14| Mr. R. Kuntarso, Reg. W.R.
|15| Mr. Rahadiansyah St. MSc. (Pak Shakti), Balai P.W.
|16| Mrs. Rigakittyndya Tiamono, MBA (Bu Kitty), Balai P.W.
|17| Mr. Ruko, BPPT
|18| Ir. Sigid Santoso, MM (Mr. Sigid), Balai P.W.
|19| Mr. Sony Santoso, ST. (Mr. Sony), Balai P.W.
|20| Dr. Suwarman Partosuwiryo A.Pi., M.M. (Mr. Warman), Prof. M.A.F.
|21| Mr. Suradi, ST. MT. (Mr. Suradi), Balai P.W.
|22| Ir. Tri Bayu Adji, MA (Mr. Tri), Balai P.W.
|23| Mr. Wahyu Endriono, BPPT
|24| Mr. Werno Ripapman, BPPT
|25| Mr. Wisnu, Bappeda

E.1 [UGM] DIALOGUE ABOUT BREAKWATER INTEGRITY AND REDESIGN

Host Hydro Cluster, University Gadjah Mada

Disclaimers

It should be noted that the statements made by the host are interpreted by Project Yogya's team members to the best of their abilities. Due to the nature of interpretations, the information provided to Project Yogya by the host should not be quoted.

According to the host,

1. Design and actual construction differ very much from each other.
2. There is an innate suspicion of using budget because it might lead to wrong usage and accountability for it, therefore 20-40% of budget has been used so far by a committee that has been elected.
3. Core has been washed away from the western breakwater due to rough sea conditions.
4. The breakwater extension was ought to be within the responsibility of the ministry of maritime affairs and fishery. They did not have sufficient resources to extend the breakwaters; therefore the responsibility for construction was given to Ministry of Public Works.
5. Waves during storm conditions have been damaging the breakwaters on many places. The height of the breakwaters is insufficient.
6. He intended earlier to extend the breakwaters in length more than the 250 m and the 300 m as given in his review design for west and east respectively. However as a consequence of limited availability of funds this had been revised.
7. South Java Coast is very dynamic, but shows on the long term regressive behavior. The reason for that might be the intensive mining activities along the rivers which are often illegal. The rivers therefore supply less sediment than before.
8. Setback lines need to be calculated since many houses along the coast are under risk. Local governments should encourage that or do that themselves.
9. Touristic activities are increasing at and around the project site.
10. Wave data is very limited and for the review design therefore wave data from Kutah Beach, Bali was used out of necessity.
11. As a result of the yearly budgeting framework the breakwaters are built in stages and because of that the breakwater is not one smooth aligned structure but has become an irregular shape.

Future Questions

1. What reference projects were used for the review design?
2. What are the implications of sea level rise on the review design?

Recommendations for further investigation

3. Differences between design and construction and its reasons
4. Budgeting framework for the Glagah Jetty Project
5. The susceptibility of the core
6. Transgressive character of the coastal system of which Glagah beach takes part
7. The coastal dynamics of Glagah beach and its global system
8. The risk of damage to construction along the shore
9. The recreational value of the project site

The following documents were provided by the host:

1. Powerpoint: “KONSEP PENANGANAN EROSI PANTAI KUWARU” (unofficial document)
2. Powerpoint: “PERUBAHAN GARIS PANTAI PESISIR DIY DAN USAHA PENGELOLAANNYA” (unofficial document)
3. Powerpoint: “SIMINAR GLAGAH 24 APRIL” (unofficial document)
4. PDF: “DISKUSI REVIEW GLAGAH 18 JUNI 2013” (unofficial document)
5. PDF: “Paparan Jetty Glagah 15 Sept 2015” (unofficial document)

E.2 [BALAI P.W. 1] GLAGAH JETTY PROJECT SITE VISIT

Host	Serayu Opak Watershed Bureau Ministry of Public Work & Public Housing *Hereinafter called "Balai P.W."*

Disclaimers

It should be noted that the statements made by the host are sometimes translated or interpreted by Project Yogya's team members to the best of their abilities. Due to the nature of interpretations, the information provided to Project Yogya the host should not be quoted.

Referenced photos are attached to this the document.

According to the host,

First the visit started at a reference beach (see reference beaches by [E|UGM]) to see the impact of the enormous waves.

1. The waves create a ridge at the treeline which is approximately 0,5 meter high. (picture 1 and 2). The buildings are built close to this ridge and are so vulnerable. The buildings have almost no protection against this wave impact. Some people have heightened the front of their house with sandbags. The beach consist of very fine sediment (picture 3)

After the reference beach we went to the west jetty and the lagoon.

2. The laguna is used for fishing (but there is not many fish left) and for recreational purposes (very busy on Saturday and Monday).
3. The West jetty is built in several stages as can be read in the report of Nur Yunowo. The design is revised several times which leads to a large number of differences between the design and the actual breakwater. The differences between design and construction are sometimes due to hard environmental conditions. (picture 4)
4. One of the largest differences between the design and the actual case is the use of tetrapod's. According to the latest design the tetrapod's have to be 11 ton at the trunk and 14 ton at the head of the breakwater. The actual situation according to mr. Sony is that the trunk consists of 9 ton tetrapod's and the head of 11 ton tetrapod's. On some locations the armour layer consist of more than one layer. (picture 7)
5. The height difference between the design values and the completed construction is 2 meters. The actual height is 6 meters but the design prescribes 8 meters. (according to mr. Sony this will be heightened in the future, an additional prefab wall will be placed on top)
6. Also the design tells us that the trunk of the breakwater has an armour layer of tetrapod's. On location there were several different armour layers that could be divided:
    - Tetrapods (picture 5)
    - Core layer which is breached during the construction phase (picture 6)
    - Concrete cubes from the toe (picture 5)
    - Reinforcement of the inner slope due to ship collision by large concrete blocks (picture 11)
7. The concrete pavement has several gaps between the core and the pavement. (picture 8)
8. The west jetty has to be lengthened by 25 meters according to the design.
9. On the west breakwater a tube was constructed for dredging purposes. This tube is used for the dredging of the navigation channel which is another company than the company that dredges the harbour basin. (picture 11 and 12)

10. At the first time the total project was a maritime ministry project. Due to the lack of funding of the project by this ministry the ministry of public works got involved. In the beginning the ministry of public works only has to construct some flood control construction (these are the perpendicular walls). After this the ministry of maritime also asks public works to construct the west jetty. Later the ministry of public works also constructed the east jetty.
11. The harbour basin is dredged in three months. From the basin a total volume of 118.000 cubic meters has to be dredged. To an average depth of 4.7 meter below LWS. The debit of the dredging pump is 750 cubic meters per hour. A submersible pam is used for dredging because it does not require a minimum depth. This results in a dredging result of 3000 cubic meters of sediment per day that is dredged away. The ministry of maritime is responsible for the dredging works in the basin. (picture 10)
12. A Korean investor is found, who might make use of the harbour in the future.
13. The navigation channel including the sandbank that was created in front of the west jetty is dredged by another company (picture 9). It is dredged using small cutter suction dredgers. The total dredging volume of the navigation channel is 20.000.000 cubic meters. The local government, Kulon Progo, is responsible for the dredging works in the navigation channel.
14. The East jetty (Breakwater timur) has a core height of 4 meter, the height of the pavement is on 6 meter above LWS. The designed height of 8 meter above LWS is reached by constructing a prefab wall on top of the structure.
15. The sedimentation between the jetties can be seen at picture 13
16. The waves impact in the river mouth is shown well in picture 14.
17. Because of the high waves at the entrance between the jetties, the fishing boats can only exit the harbour area during low tide when the waves are less high. They can enter the harbour area by riding the waves, so they can do this during both low and high tide.

1. The ridge due to sediment erosion in the reference harbour.

2. Height of the ridge compared with a 1.80 m person

3. Very fine sediment

4. A picture of the extreme dynamics around the jetty.

5. The concrete cubes from the toe.

6. The core material is exposed to the waves because of the collapse during construction. There are no tetrapods placed on top of the core.

7. At some places there is a double layer tetrapod. This might occur due to collision and slipped off.

8. A gap below the concrete pavement.

9. Large amount of sediment in the “quiet” corner.

10. Also large amount of sediment in the basin.

11. Dredging works seen from the eastern jetty. On this picture the large amount of sediments in the navigation channel can be seen. In the back the repair works from a ship collision can be seen.

12. Tube which is used for the dredging works of the navigation channel.

13. Large sedimentation on the inside of the jetty.

14. Waves at the entrance.

E.3 [REG. W.R.] EXPLORATORY DIALOGUE ABOUT FLOODS, DREDGING AND THE INVOLVEMENT OF KULON PROGO REGENCY

Host	Water Resources Kulon Progo Regency (*Kabupaten Kulon Progo*)

Disclaimers

It should be noted that the statements made by the host are interpreted by Project Yogya's team members to the best of their abilities. Due to the nature of interpretations, the information provided to Project Yogya by the host should not be quoted.

According to the host,

1. Floods around the river mouth could only possibly occur in the event of a high discharge in the beginning of the wet season. The spit that developed would block the river mouth and the river could be blocked.
2. The presence of hyacinths increased the flood risk as they grow on the sand and contribute thereby in blocking the river. The extreme growth of hyacinths was caused by abundant use of fertilizers. In an attempt to protect their crops from inundation farmers in the flood prone area would use the fertilizers to be able to harvest just before the wet season.
3. Local villagers knew how to cope with the floods. They usually dug a channel to guide the river when a high discharge was expected. This had to be done at the exact right time since longshore sediment transport would restore the dug channel if a flood remained absent.
4. Farmers adopted a planting schedule that was adapted to flood risk.
5. The dredging processed 15.000 cubic meters and would cost 11 million rupiah and if the timing was wrong this had to be redone.
6. No serious floods occurred around the river mouth before the construction as well as after the construction. The jetties little influenced the flooding regime of the river serang.
7. The mouth of the Serang river could shift in position within a range of 2 km. The land within that 2 km could not be used for construction.
8. The Jetties only changed the risk on floods and the need for dredging activities. Less dredging was required since the existence of the jetties.
9. The discharge capacity of the Serang river is good as well as its operation and maintenance program which is yearly.
10. In the Progo river more trouble is experienced with floods. These are mainly cause by bad maintenance of drainage systems and river works.
11. Deepening of a connection channel between Progo river and Serang river is being considered to reduce flood issues the Progo river basin.
12. Local floods still occur as a consequence of heavy rainfall and poor drainage. River overbank spilling may also be observed in the upper regions of the river. Local floods however are less than 1500 hectare per year.
13. Sermo dam contributed to the reduction of floods. Estimates show a reduction of 20% flood presence. However, floods occur in different areas. This may also be due to the increase in rainfall over the last couple of years.
14. The river works built by the Java flood Control Sector Projects has diminished salt intrusion in such a way it no longer affect the farmers and local villagers in the area.

The conversation now changes from topic to dredging responsibility and activity in the harbour.

15. The region stimulates fishery by creating education programs for fishermen and apprentices. The education involves mainly activity with small fishing boats.

16. Dredging in the harbour and serang river are within the responsibility of BBWS Serayu Opak. The Kulon Progo Regency only assist in workforce for maintenance and dredging activities.
17. Kulon Progo Regency regrets their small role in the river management and stipulates their wish for role sharing.
18. A clear document that describes the roles of all agencies involved in river management of the Serang river lacks. Responsibilities are clear among agencies, however villagers are often complaining at the wrong agency.
19. Illegal mining does not take place in Serang river, however it is in Progo river.

Future Questions

1. What was the impact of the Java Flood Control project for this region?
2. What is the influence of irrigation on the Serang river flow regime?

Recommendations for further investigation

1. Change in dredging policy after the construction of the breakwaters.
2. Impact of connection channel between Progo river and Serang river on the Serang river flow regime.

E.4 [START-UP] START-UP MEETING

Host	BBWS Serayu Opak

Summary

The analysis phase of the project was reviewed by people who have worked or will work on the Glagah Jetty Project. The morning consisted of a presentation by Project Yogya about the History of the Glagah Jetty Project, a presentation about the design review and a presentation

The information discussed in this meeting is directly reported in the report.

E.5 [PROV. M.A.F. 1] EXPLORATORY DIALOGUE ABOUT HARBOUR DEVELOPMENT

Host	Serayu Opak Watershed Bureau Ministry of Public Work & Public Housing *Hereinafter called "Balai P.W."*

Disclaimers

It should be noted that the statements made by the host are interpreted translated by Mr. Shakti to the best of his ability. Due to the nature of interpretations, the information provided to Project Yogya by the host should not be quoted.

It should be noted that the Prov. M.A.F. requires an official letter from Balai P.W. in order to be able to provide information to Project Yogya. Following the absence of this letter at this meeting, further details on general policy and engineering designs can only be given at a future meeting. Prov. M.A.F. is open to providing this information upon receiving this letter.

According to the host,

1. The South Coast of Java is often neglected when assessing the potential economic gains from maritime industries, even though the whole island is considered both a maritime and agricultural nation.
2. Sri Paku Alam VIII, Sultan of the Special Province of Yogyakarta from 1988 to 1998, had the "gut feeling" that a harbour district should be created at the southern coast of the Yogyakarta Province. A feasibility study for the harbour district was not conducted but the promises envisaged qualified as sufficient for further ordering of the creation of this harbour district.
3. An analysis has been made regarding the location of the harbour. Three rivers were under consideration: the Progo river, the Serang River and the Barong river. It was concluded that the Serang river was most feasible for the construction of a harbour.
4. First engagement of the host with the Glagah Harbour district was in 2011, while construction for the harbour started in 2010.
5. The budget that enabled the development of the Glagah Harbour District was paid for out of three funds:
    i. Prov. M.A.F, whose contribution was/is for the operation of the harbour.
    ii. Reg W.R., whose contribution was/is for land acquisition.
    iii. A newly setup fund for general contribution to the project. The Min. M.A.F. allocated money for this fund as well as the provincial Government.
6. Responsibility for the Jetty lies, has lain, and will lay with Balai P.W. This means that "there is 1 infrastructure for 2 problems". A closer cooperation is necessary between the governmental institutions.
7. The coordination for harbour construction and jetty construction is supervised by an umbrella agency within the Prov. Govt. This agency is called the "Provincial Planning Agency". This agency has been called into life for the coordination of many projects in the province and Maritime developments coordination is the latest addition.

8. Balai P.W. is represented in this Provincial Planning Agency by Pak Tri and Pak Sigid.
9. When the West Jetty was constructed (the first part), there was a need for dredging works as the river mouth was naturally filled by a sand bank.
10. Since the first construction of the (West) Jetty there have been dredging works that facilitate the lengthening of the Jetty or the construction of new parts.
11. The harbour was designed to facilitate the mooring of vessels of 100 Gross Tonnage [edit: not to be confused with GWT].
12. The Jetty height is 8 m [edit: it is believed that sir mentions the original design height from Prof. Nur Yuwono's design].
13. In 2015, the dredging budget was estimated at Rupiah 250 mln. The purpose of these dredging works is to open the navigation channel over a period of several months. In the future, a yearly budget of Rupiah 3 bln is set apart for maintenance dredging.
14. It has been estimated, by studies performed by Prov. M.A.F., that when the East Jetty is lengthened following Pak. Prof. Nur Yuwono's advice, the dredging operations cost would be reduced by about 2/3. This means that the dreding costs can be cut by about Rupiah 2 bln.
15. *(When asked about a monetary contribution from Prov. M.A.F. to the lengthening of the Jetty)*, no funds are, have been, or will be allocated to the lengthening of the East Jetty.
16. Prov. M.A.F. is very interested in working together concerning the Jetty Lengthening. Contributions from Prov. M.A.F. would then include in-house expertise about the harbour and the navigation channel.
17. The Balai P.W. is lacking progress in its ambition to lengthen the Jetty. The Prov. M.A.F. has been waiting for progress for several years now.
18. Initially, the costs for lengthening the Jetty have been estimated at Rupiah 60 bln, but due to the delays for 10 years these costs have rose to 250 bln. Currently, the Balai P.W. budget is insufficient, hindering further development. The delays of execution can be explained by the fact that Balai P.W. has higher priorities.
19. Pak. Prof. Nur Yuwono is a very valued partner to the Balai P.W. and should be included in future plans regarding the Glagah Harbour Projects.
20. When doing anything in Indonesia outside of your formalized responsibility, there is a high chance that you break the law and get sued by another stakeholder.

Project Yogya suggests to create a new entity for the purpose of accelerating the developments that should make the Glagah Harbour future proof. This entity can be based on the working of the "Water council" and can be named "Glagah Harbour Council". A representation of stakeholders should take seat in this council, including the relevant governmental institutions, investors, fisherman representatives etc. This new council can be formalized as part of either the Prov. M.A.F., Balai P.W. or the Provincial Government in general. The host's comments on this proposal are noted below

21. To some extent, there is already an agency such as the one proposed here. There is an informal task force of 'experts' from different backgrounds. The task force is part of the Provincial Planning Agency and has been called into existence by the Sultan, however two issues have risen since its first engagement:

a. The task force has not (yet) been formalized. The reason for this is unknown.

b. The task force has a limited budget

   A meeting with the Provincial Planning Agency is suggested by the host, who will follow up on this suggestion with a meeting proposal. The person in charge of the Provincial Planning Agency is Bu Siwi (Head of monitoring and evaluation and controlling [Bidang Pengendalian]).

Future Questions

1. What is the vessel size of 100 GT based on?
2. Are there any investors lined up to start using the harbour?
3. What Sultan has called into existence the special task force for successful completion of the Glagah Project?

Recommendations for further investigation

1. Get the information request letter and plan another meeting
2. Learn about the general Policy of the provincial government regarding the Glagah Jetty and other cooperations with Balai P.W.
3. Investigate possible sensitive topics between the related governmental institutions
4. Find out what the responsibility is of the Provincial Planning Agency
5. Find out what the status and formality of the Expert Task Force is

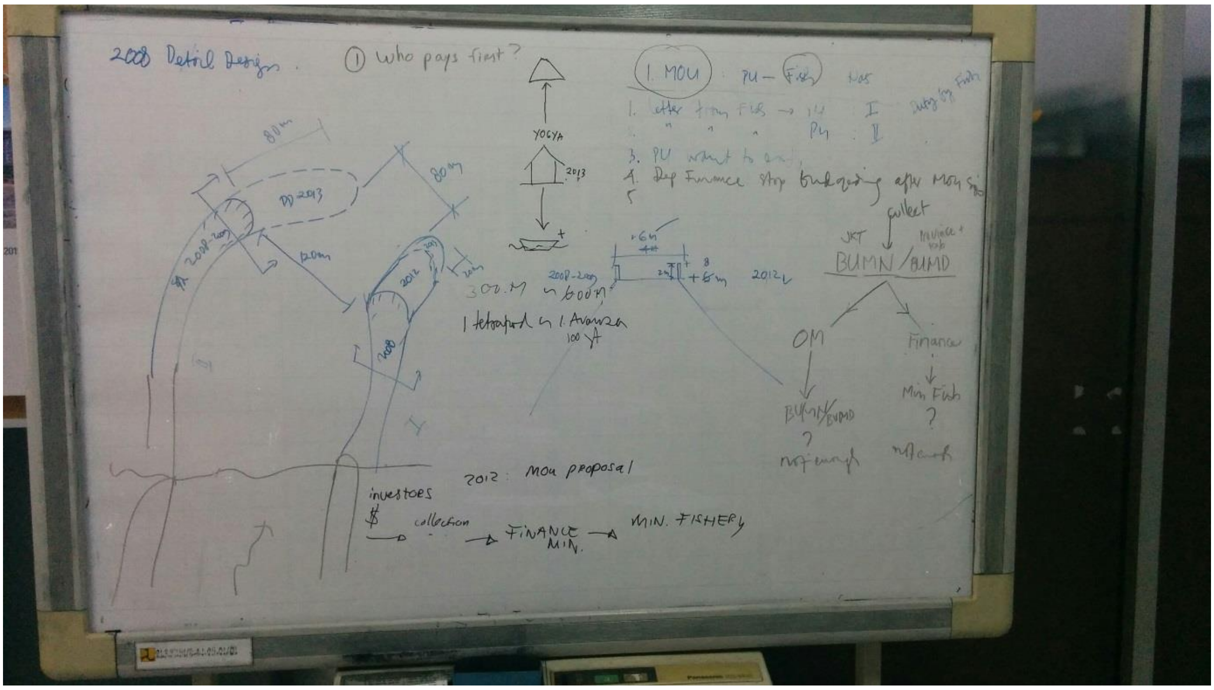
E.6 [BALAI P.W. 2] POLITICAL STRUCTURE DIALOGUE ABOUT GLAGAH JETTY DEVELOPMENT AND FUNDING

Host	Serayu Opak Watershed Bureau Ministry of Public Work & Public Housing *Hereinafter called "Balai P.W."*

Disclaimers

It should be noted that some statements made by the host are interpreted and translated by Mr. Shakti to the best of his ability. Due to the nature of interpretations, the information provided to Project Yogya by the host should not be quoted.

Drawings



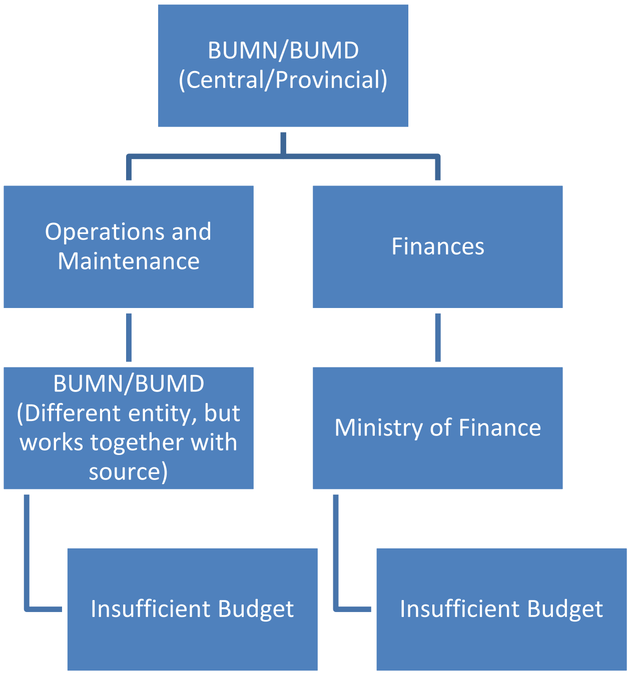
According to the host,

1. In 2007 he got involved with the Glagah Jetty construction as Head of Planning and Programming.
2. Construction of the breakwater was initially started without expert consultants.
3. The Ministry of Public Works is not responsible for the construction of the Jetty for the purpose of harbour development. However, it took responsibility because:
    a. Both Kulon Progo and Prov. M.A. did not have sufficient budgets to construct the jetties
    b. A letter was sent to Balai P.W. requesting the construction of the Western Glagah Jetty and the transfer of responsibility for the Phase 1 works. This letter was accepted, allowing for the start of construction in 2008.
    c. A letter was sent to Balai P.W. requesting the construction of the Eastern Glagah Jetty and the transfer of responsibility for the Phase 2 works. This letter was accepted, allowing for the start of construction in 2008/2009.

4. Prof. Nur Yuwono concluded after the construction of these first parts that the wave height was underestimated and he proposed a redesign. Part of the redesign has been constructed at the West Jetty in 2012. After the first construction steps of this redesign, the Ministry of Finance ordered a stop on further execution. The reason for this was that the Ministry of Finance does not trust the future maintenance and operations to be in good hands as budgets indicate that no responsible institution other than Balai P.W. can carry the burden.

5. A solution for the construction stop is to establish an agreement between Min. M.A.F. and Min. P.W. A proposal for this agreement (MOU: Memorandum of understanding) has been drafted by Min. P.W. in 2012 but was not succesfull.

6. The revenue gathered from harbour operations are subject to the following cashflows:



7. In the flow chart above it can be seen that there is insufficient budget at two important entities. Because of the insufficient budget, Min. Fin. assumes there will be no budget at any entity that covers the costs of operation.

8. The reason why the MOU has not been signed yet is because Min. M.A.F. thinks future budgets are also insufficient for coverage of the operations and maintenance costs. This assumption is based on the feasibility study of the Glagah Harbour.

9. In 2008, phase I of the Glagah Jetty Project started as the West Jetty was built.

10. In 2013, the Yogyakarta Provincial Government decided that future economic developments plans cannot be reached solely with agricultural improvements. It is believed that a shift to maritime developments such as fishery is required for a higher rate of growth.

11. *(When asked about the Task Force Successful Completion of Serang Harbour),* this is not a special task force for the Glagah Harbour, although it has a lot to do with it. This task force was instigated

in order to realize the Yogyakarta Province new path to prosperity: from agriculture to maritime developments. The shift to maritime developments needs to be well supervised and this team has been unofficially assigned to assist this process. The secretary for this commission is Bu Mati.

12. The development costs of the extention of the West Jetty are large. It is believed that 300 tetrapods are required and each tetrapod has a value of 1 Toyota anvanza, equaling about 200.000.000 Rupiah (~13 000 Euro).
13. (*In conclusion)* the Glagah Jetty extention is not an engineering problem but it is a political problem. A study to the political structure behind the Glagah Jetty Developments would be very interesting and helpful in further developments.
14. Budget from Balai P.W. is about Rp 600 Billion. The total costs of the Glagah Jetty extention are at least half, Rp 300 Billion, of the Balai's budget.

E.7 [BPPT] EXPLORATORY DIALOGUE ABOUT GLAGAH JETTY RESEARCH AND LENGTHENING

Host	Agency for the Assessment and Application of Technology, Coastal Dynamics Research Center *Hereinafter called "BPPT"*

Disclaimers

It should be noted that the meeting held at BPPT was on very short notice, so potentially more information can be extracted by both participants on a longer run.

According to the host,

1. BPPT is a hydraulic research division part of the public company BPDP. It is responsible for research into public projects related to disasters and oceanic effects in general.
2. BPPT takes assignments from the government in general, but also takes private assignments if it has timeslots available. The BPPT however should make only little profit and can be considered a government-dependent organization.
3. BPPT closely cooperates with Prov. M.A.F. already, but does not cooperate with Balai PW.

When asked about a potential special taskforce for the successful completion of the Glagah Jetty Project,

4. A harbour council of some kind would be a very good idea and would be fully supported. Other departments at BPDP who are responsible for coastal zone management can be included in this cooperation and significantly add value to the discussion.

When asked if this could be lead by the host,

5. A special task force can indeed be headed by the host and his assistants.

Concerning the Glagah Jetty,

6. There is a full bathymetry map available from the Glagah beach.
7. Two different grain sizes have been and densities have been observed at the coastline.
8. Physical modelling is executed in a large laboratory with a scale model of 1:68. The model has been constructed in 2012 and has been used ever since. The newest (2013) extension designs by Prof. Nur Yuwono have not been constructed yet.
9. The results of physical modelling are used for calibration of the numerical models that have also been produced.
10. BPPT has been working together with Prof. Nur Yuwono, however the last time these parties spoke was about 10 years ago. The reason for a lack of contact Is explained as: "That is unfortunately how Indonesia works".
11. Unfortunately, none of the models that have been produced by BPPT have been verified with actual wave data. The assumption has always been the 5.8m significant wave height.

It is believed this information was provided by Prov. M.A.F.

E.8 [PROV. BAPPEDA] EXPLORATORY DIALOGUE ABOUT HARBOUR PLANNING AND FEASIBILITY

Host	Government of the Special Province of Yogyakarta, Provincial Planning Agency *Hereinafter called "Bappeda"*

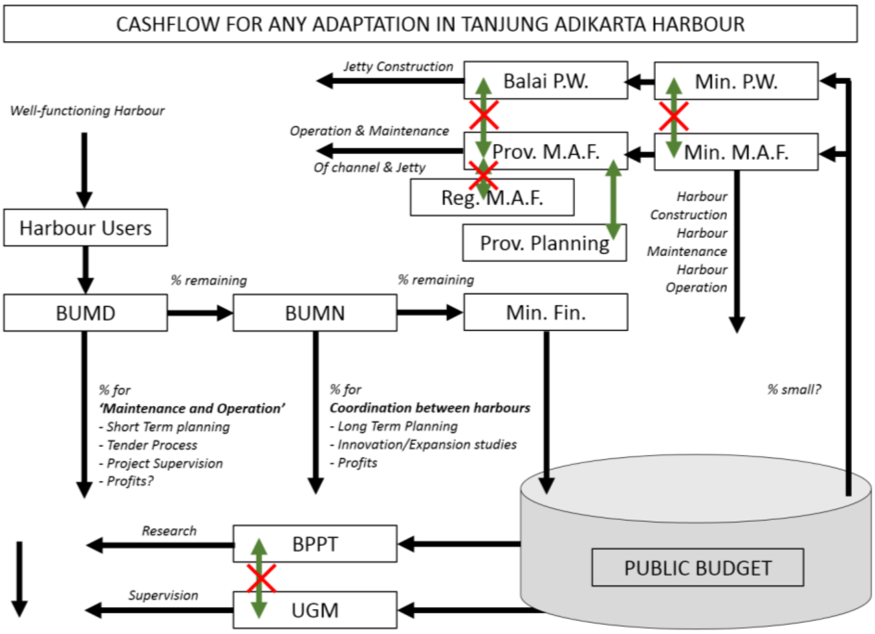
Disclaimers

It should be noted that the statements made by the host are interpreted and translated by Mr. Shakti to the best of his ability. Due to the nature of interpretations, the information provided to Project Yogya by the host should not be quoted.

According to the host,

1. The Tanjung Adikarto Port is prepared for 'soft launch' next year. This means that the main harbour activities will start. They will start with 30 – 50 DWT vessels. Design vessel are maximum 100 GT. 150 – 200 GT is the full potential estimation.
2. Bappeda does budgeting and also takes care of the planning of the construction phases for the harbour. They coordinate all activities in the harbour area.
3. The engineering design is always provided by Dinas Kelautan.
4. Bappeda is also coordinating the airport project for the Kulon Progo Region.

A short introduction to previously gained information is given in order to be verified by representatives from Bappeda.



5. The taxes paid by harbour users will be collected by UPT, not BUMD.
6. UPT is a provincial-level organization below maritime affairs, whereas BUMD is funded by government.
7. UPT is created by the local agency on provincial level *and is only for Tanjung Adikarto*. UPT is responsible for the collection of taxes as well as the execution of harbour maintenance. Generally speaking, the UPT is responsible for the economic process.
8. BUMD's role in the harbour is unclear.

9. A meeting with multiple sectors is planned to come up with budgeting solution for the dredging activities. So far, the Provincial government who is now covering the expenses of all the dredging works, has to find other stakeholders that can pay for dreding.
10. In the future it is not clear yet who is responsible for dredging activites.
11. The harbour construction has so far cost about 300-400 billion, it is expected that this amount will be doubled in order to finish the harbour completely.
12. The Glagah Jetty project is a national pride project.

When asked about the cause of the Glagah Jetty Construction Stop,

13. In 2012, after the Min. Fin. meeting, a meeting between Bappeda, kulon progo, Balai P.W. and Prov. M.A.F. has been held in order to talk about an agreement. A second meeting is planned.
14. Serayu opak stopped construction not because of lack in funds but because they were not responsible anymore.
15. The Harbour is also in national level long term planning.
16. In the future, harbour management is done by ministry of M&F and the private sector.
17. Balai P.W. has been subject to an audit by its own Ministry and by the Ministry of Finance, which led to the construction stop by the Min. P.W.
18. Special and allocation budgeting.
    a. Deconcentration budget.
    b. Tepe, shared duty budget. Money given by national government. Responsibility by provincial.
    c. S&A budget is national.
19. The regency government of Kulon Progo does not share the costs of the Tanjung Adikarto Developments.

Mr. Shakti (translator) requests permission to ask the host to elaborate on the claim about the Audit at BBWS SO.

20. Perhaps this situation is more complicated than as suggested here.

When asked about the Bappeda's opinion on showstoppers for full harbour opening that need to be addressed,

21. Sediment has to be removed in the navigation channel
22. The breakwaters, especially the eastern one, should be extended.
23. Tetrapods should be added and several locations should be inspected for tetrapod maintenance.
24. A soon soft launch of the harbour would also give the project a lot of momentum and will atrackt investors that can afford fees and taxes on the long term.

When asked about the setup of a special task force for succesful harbour developments,

25. ~~The taskforce is a good idea, but it is feared that the taskforce cannot be sustained too long. On provincial level provincial level this taskforce still exists, but is unformal and appears to be inactive momentarily.~~

The above statement has been retracted by the officer in a later stage.

E.9 [PROV. M.A.F. 2] ELABORATIVE DIALOGUE ABOUT HARBOUR DEVELOPMENT 1

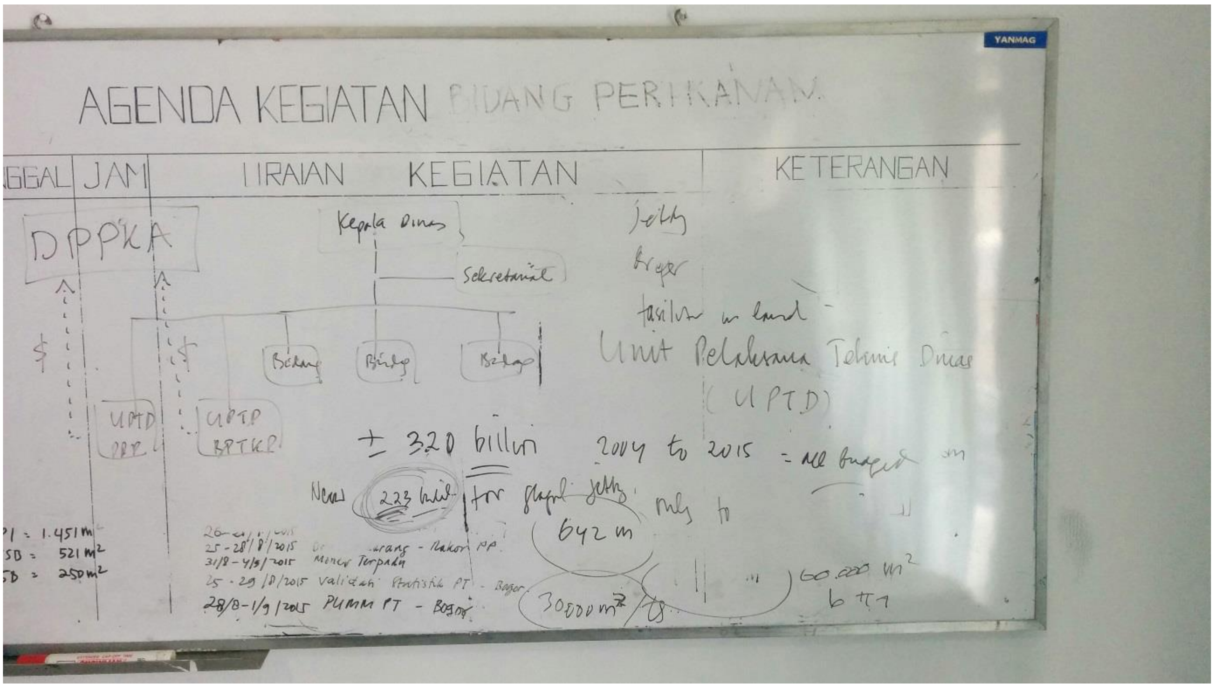
Host	Department of Maritime Affairs and Fishery, Government of the Special Province of Yogyakarta *Hereinafter called "Prov. M.A.F."*

Disclaimers

It should be noted that the statements made by the host are interpreted and translated by Mr. Shakti to the best of his ability. Due to the nature of interpretations, the information provided to Project Yogya by the host should not be quoted.

It should be noted that the following statements have been made during several follow-up meetings over the course of one week due to scheduling difficulties. The information provided in these meetings is therefore bundled.

Drawings



According to the host

1. The size of the harbour pool is dimensioned as 300m on the sides (North and South) and 200m at the back (East).

Concerning the dredging works currently executed in the harbour,

2. Currently, a budget of Rp360 Million is cleared for dredging purposes by the Prov. M.A.F. This budget is used for dredging of the navigation channel and the harbour basin.
3. The navigation channel should be dredged 80m wide at LWL, 3.6m deep and 40m deep at -3.6m below LWL. The navigation channel total length from the breakwater heads to the harbour pool is 642. This distance will be dredged.
4. There are currently 4 dredgers assigned to activity at the Tanjung Adikarto district.
5. The dredging equipments currently used in the harbour is not technologically advanced and the stuff might be not sufficiently educated. For example, the dredging vessels have selectively cut too

deep in the harbour basin. This resulted in 'dredging holes' and its cutting has caused the loss of more time before the completion deadline.

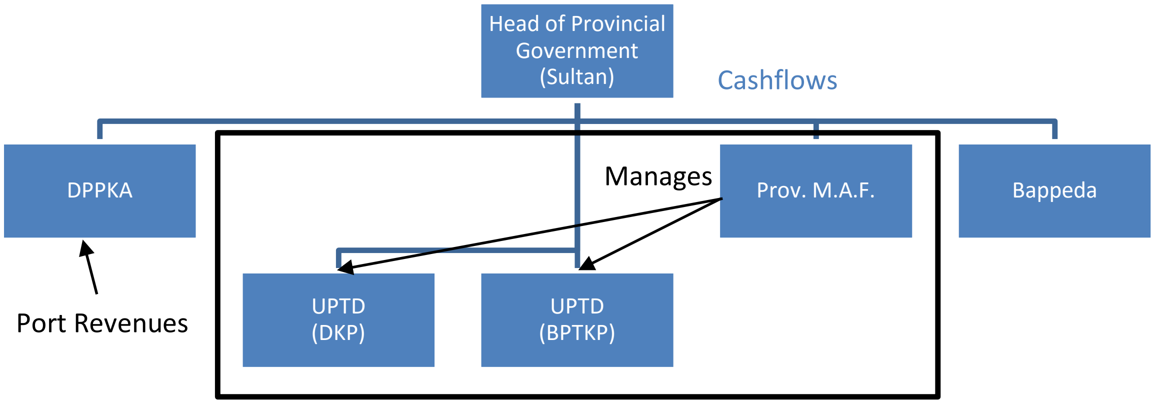
6. The dredging volumes are as follows:
    a. Now that the harbour basin construction will be completed in 2015, the total dredging works for the harbour basin only are 118 000 m^3.
    b. In order to open the navigation channel, the dredging works are considered to require dredging of 150 000 m^3 of sediment.
    c. It is estimated by Dinas Kelautan that the years following harbour opening, maintenance dredging at the channel is required for a volume of 30 000 m^3.
    d. It is estimated that the dredging needs for the Serang River Estuary / navigation channel are approximately 16 000 m^3 afer the Jetties are lengthened.

It is assumed that this information is provided to Prov. M.A.F. by either Prof. Nur Yuwono or BPPT. Following these statements, it can be calculed that the 150 000 m3 dredging activities can be completed in 31.35 days when maximum capacity is reached.

7. The current combined capacity of the dredging vessels is 200 m^3 per hour.
8. For future effectiveness and higher reliability, Prov. M.A.F. prefers to acquire or lease a single dredging vessel.

When asked about the UPT and its function,

9. The real name is UPTD: Unit Pelaksana Tekmis Dinas. The UPTD can be considered a harbour master institution fully owned and funded by the Provincial Government.
    a. The UPTD is under management of Prov. M.A.F.
    b. Funding of UPTD is directly from the Provincial Government.
10. The UPTD is now being setup and formalized, meaning no information is available about this organization yet.
11. As soon as UPTD makes any profit, the Prov. M.A.F. will make UPTD a Public Company. As from that moment, the UPTDs organizational structure will be similar to BUMD or even be assimilated with the BUMD.
12. In fact, the revenues from harbour exploitation are collected by the DPPKA (Dinas …). As soon as revenues are significat, new decisions will be made on the status of the UPTD.
13. The organizational structure of the Provincial Government can be sketched as follows:



When asked about the reason for the current construction stop of the Glagah Jetty,

14. In the past designs of the breakwaters, the oceanic conditions at Glagah Beach have been underestimated.
15. Halfway construction in 2012, it is concluded that the construction process is greatly complicated by the underestimated sedimentation issues in the Serang River Estuary.
16. In order to be better prepared for future construction, Balai P.W. decides to put a hold to current breakwater lengthening and to consult with new researchers about the effects of dredging around the breakwater.
17. In general it can be concluded: more research was required.

At this point in the meeting, the question is asked by both Mr. Jeroen Werkhoven and Mr. Shakti: "Could you elaborate on why there are three scenarios [edit: according to the research, there are actually 4 scenarios] that explain the construction stop and which scenario do you think is closest to the truth?"

18. All three scenarios are true.
    a. In fact, it is true that the Min. M.A.F. has not signed the MOU that was proposed by Min. P.W.
    b. It is complicated, as the Prov. M.A.F. highly favors a formal agreement as such.
    c. The reasons for refusing to sign the MOU are unknown to date. However it is known that there have only been objections for the MOU at the office of the General Directorate of Caught Fish of Min. M.A.F.
    d. It is assumed that there is fear of an upcoming audit for the Glagah Jetty Project. Since the responsibility has been transferred to Min. P.W., potentially this reflect badly on the minstry's own responsibility.
    e. Out of two general secretaries 1 has agreed to sign the MOU, but his counterpart has not.
19. Prov. M.A.F.'s experience with projects of this magnitude is insufficient and it can be safely said that the extension of the Glagah breakwaters is in better hands at BBWS SO considering the capacitiy of this national level organization.
20. Prov. M.A.F.'s trustworthiness in the matter of favorability of the MOU can be derived for its efforts in finding a budget for Glagah Jetty Extension.
21. *Yesterday*, it has been confirmed by the National Planning Agency (Nat. Bappeda) that a budget has been assigned for the extention of the Glagah Jetty.

22. Prov. M.A.F. estimates that the costs for extension of the Glagah Jetty are Rp160 billion. These costs have been subject to high inflation since the 2010 estimation was about Rp100 billion.

"Who should be responsible for the maintenance costs of the Glagah Jetties?"

23. Balai P.W.

When continuing the conversation concerning the other scenarios:

24. The scenario where Balai P.W. would be worried about national-level audits is not true: Balai P.W. has been a valued partner in the process of breakwater extentions at Glagah.

It is mentioned that the responsible officer for this decision will be visiting the Prov. M.A.F. offices this week and the invitation to this informal meeting is extended to Mr. Jeroen Werkhoven. It is stated that a few questions can be asked by Mr. Jeroen Werkhoven.

E.10 [PROV. M.A.F. 3] ELABORATIVE DIALOGUE ABOUT HARBOUR DEVELOPMENT 2

Host	Department of Maritime Affairs and Fishery, Government of the Special Province of Yogyakarta *Hereinafter called "Prov. M.A.F."*

Disclaimers

It should be noted that the statements made by the host are interpreted and translated by Mr. Shakti to the best of his ability. Due to the nature of interpretations, the information provided to Project Yogya by the host should not be quoted.

It should be noted that the following statements have been made during several follow-up meetings over the course of one week due to scheduling difficulties. The information provided in these meetings is therefore bundled.

According to the host,

1. At this point in time, the southern coast of Java knows 8 UPTD institutions of which none make a profit. The provincial governments consider the UPTD as a public serve to enable future economic growth.
2. The Tanjung Adikarta Port is expected to be profitable in 8 years.

Documents supporting this expectation are provided by the host. Since dredging will be a showstopper for launch, this is elaborated upon.

1. According to the contractor responsible for all 4 dredging vessels and all the dredging activities, the dense packed sedimentation caused his vessels to take longer then expecting, endangering the 'soft launch' opening of the harbour.
2. It was assumed at the start that the sand was not densely packed, but this proved to be untrue.
3. Since this conclusion was drawn, two suction cutter dredger vessels have been used.
4. For a significant period of time, one of the suction cutter dregers has been under maintenance after it broke. This put the delivery date of early 2016 under pressure.
5. It is not realistic to think the harbour will be opened at the expected delivery date any longer.
6. The dredged sand is currently being transported by a pipeline system to the hinterland of the Tanjung Adikarto Port.
7. Reason for this is a request by the local village officer (not by Kulon Progo) that has been granted by Prov. M.A.F. 'Socialization', as this is called, is one of the responsibilities of Prov. M.A.F.

When asked why the location of dredged sediment dumping is not in the erosion zone of the western breakwater,

8. There has been placed sediment nourishment at the coastal erosion zone. However there is a problem with dumping sand there.
    a. First of all, it is acknowledged that the coastal erosion zone needs to be refilled with sand from the estuary.
    b. When this process started, Prov. M.A.F. knew of an audit coming up. At the audit, there would be an investigation into the dredging activities.
    c. Since Prov. M.A.F. dumped sand at the coastal erosion zone, there was fear of not being able to prove the sand dumping (in full) as the nourishment would have also partly eroded.
    d. The coastline of Java is protected by law and stealing sand from a project site would be considered a severe felony.
    e. Because of fear of the upcoming audit, it was decided not to continue supplying sand to the coastal erosion zone.

When asked about environmental concerns and the relationships to other stakeholders,

9. Prof. Nur Yuwono is a consultant to Balai P.W. and not directly to Prov. M.A.F.
10. Local villagers support the project, especially since Prov. M.A.F. serves its public goals (such as sink hoal filling).
11. There is an abundance of NGOs in Indonesia, and anyone is entititled to start one. There is a good climate that facilitaties registration and confederation when it comes to environmental concern. NGOs in Indonesia are called LSMs.
    a. For the Glagah Jetty Project, as with any project, a document describing the environmental impact and its mitigation methods needs to be fulfilled.
    b. NGOs have specifically asked to keep the sediment gained from dredging activities within the project site and not to transport it to any other sites.
    c. Since there are many other NGOs there are ´lots of threats’. These have all been coped with.
    d. In the past, Prov. M.A.F. has often discontinued projects that were shut down due to problems with its socialization duty. Information about these projects was not provided adequately to the NGOs and other stakeholders.

When asked about the Glagah Jetty projections,

12. There is no special task force for the successful completion of the Glagah Jetty Project. It is true, however, that any project is accompanied by a special team that focuses on operation smoothness.
13. For the Glagah Jetty, there are many luxurious information gathering facilities. Prov. M.A.F. can rely on research facilitites to the likes of BPPT, UGM and Prov. M.A.F.’s research department.
14. In the near future, BPPT and the internal research department will both be assigned a research centre near the Glagah Jetty [confirmed by BPPT].
    a. BPPT research will focus on disaster mitigation and research related to shifting of the shoreline.
    b. Dinas Kelautan will conduct research on the development of marine and fishery technology with a specific emphasis on use of the Tanjung Adikarto Port. Other research includes the cathing process of fish, the efficient sizing of vessels and how to increase efficiency.
    c. The budgets for these research facilities have been proposed and accepted on a national level already.

E.11 [INT. PRES.] INTERNAL MEETING ABOUT RESEARCH RESULTS

Host	Serayu Opak Watershed Bureau Ministry of Public Work & Public Housing *Hereinafter called "Balai P.W."*

Summary

The goal of this internal meeting, hosted by Project Yogya, consisted of three main subjects:

1. Present the results of the two-month research by Project Yogya in full and with no limitations due to publicity.
2. Discuss the structural integrity of the breakwaters in the current situations and in designs by Prof. Nur Yuwono and by Project Yogya.
3. Discuss a strategic approach for the publication of research conclusions at the October 26th symposium

The meeting was kicked off by an opening speech performed by Mr. Tri Bayu Adji. After the opening the meeting was separated into two parts. Before the break, Prof. Nur Yuwono gave a presentation about his breakwater design.

After the break, Project Yogya showed their two-month research starting with an introduction to the initiation of the project and the key problems that had been observed by the project team during their visit to the Glagah harbour (by Mr. Rogier Burger). Then, the breakwater integrity was presented and discussed by Mr. Maarten Lanters. Third, the coastline stability, followed by a policy analysis, was presented and discussed by Mr. Laurens Leunge and Mr. Jeroen Werkhoven respectively. Finally, the meeting was ended with conclusive remarks by Mr. Tri Bayu Adji.

The relevant discussions from this meeting have been reported in the report.

E.12 [SYMPOSIUM] THE FIRST TANJUNG ADIKARTO PORT AND GLAGAH JETTY SYMPOSIUM

Host Hydro Cluster, University Gadjah Mada

Summary

The goal of this symposium, hosted by Project Yogya, consisted of two main subjects:

1. Present the results of the two-month research by Project Yogya.
2. Get together all of the stakeholders in order to provoke a discussion about the current situation of the Glagah harbour.

The symposium was kicked off by an opening speech performed by Mr. Moch. Silachoeddin. After the opening the meeting was separated into two parts. Before the break, the project introduction about the initiation of the project and the key problems that had been observed by the project team during their visit to the Glagah harbour was presented by Mr. Rogier Burger. Next, the breakwater integrity was presented by Mr. Jorrit Horst and Mr. Maarten Lanters. Both subjects were discussed afterwards during the Q&A. After the break coastline stability and the policy analysis were presented by Mr. Laurens Leunge and Mr. Jeroen Werkhoven respectively. In the following Q&A these subjects were discussed. Finally, the symposium concluded with a speech by Dr. Arie Moerwanto.

According to Dr. Moerwanto,

When asking oneself whether further investments are justified, two arguments can be made.

- *As soon as the harbour shows great potential, investments in the breakwaters are justified*
- *As soon as the breakwaters prove to protect the harbour entrance, investments in the harbour are justified*

This is considered the 'Chicken and Egg' problem. There is no incentive for any governmental institution to make an investment without its counterinvestment being certain.

The relevant discussions from this meeting have been reported in the report.

E.13 [MINISTER] MEETING MINISTER OF PUBLIC WORKS

Host Ministry of Public Works, Jakarta

Summary

An executive summary of the research report was presented to the Minister.

The relevant discussions from this meeting have been reported in the report.

APPENDIX F: DRAFT DESIGN CALCULATIONS

F.1 DETERMINATION OF THE ORIENTATION OF THE BREAKWATERS

The orientation of the breakwaters is mostly dependent of the width of the navigation channel and the length of the breakwaters. First the required width of the navigation channel is calculated. Then the length of each breakwater is calculated based on sediment transport and the position of the breaker zone.

WIDTH OF NAVIGATION CHANNEL

The width of the navigation channel can be calculated with the use of multiple factors determined in the PIANC (PIANC, 1997). These factors are dependent of the vessel speed, wind speed, currents, depths, cargo hazard etcetera. There are four possibilities for the channel width:

- One-way, inner channel protected water
- One-way, outer channel exposed to open water
- Two-way, inner channel protected water
- Two-way, outer channel exposed to open water

The following formulas are used for a one-way and a two-way channel:

One way channel

$$\mathrm{w} = W_{BM} + \sum W_i + W_{Br} + W_{Bg} \tag{F.1}$$

Two way channel

$$\mathrm{w} = 2W_{BM} + 2\sum W_i + W_{Br} + W_{Bg} + W_P \tag{F.2}$$

In which:

W_{BM}	Basic manoeuvring width
W_i	Additional clearances for straight channel sections
W_{Br}	Bank clearance port side
W_{Bg}	Bank clearance starboard side
W_P	Additional for two way traffic

The distinction between the outer channel and the inner channel is made by the determination of the factor for the channel width. The factors for both are shown below.

TABLE F-1 FACTOR FOR CHANNEL WIDTH

Width [w_i]	**Outer channel exposed to open water [-]**	**Inner channel protected water [-]**
Vessel speed (slow)	0.0	0.0
Prevailing cross wind	0.5	0.5
Prevailing cross current	0.0	0.0
Prevailing longitudinal current	0.0	0.0
Significant waves height and length	1.5	0.0
Aids to navigation	0.2	0.2
Bottom surface	0.1	0.1
Depth of waterway	0.2	0.4
Cargo hazard	0.0	0.0
Total $\sum W_i$	2.5	1.2
$\boldsymbol{W_{BM}}$**, good ship manoeuvrability**	1.3	1.3
$\boldsymbol{W_{Br} = W_{Bg}}$**, steep and hard embankments, structures**	0.5	0.5
$\boldsymbol{W_P}$**, additional for two way traffic**	1.2	1.0

When the above factors are filled in in the formulas (F.1) and (F.2) then the following channel widths can be determined:

One-way, inner channel protected water	21.0 m
One-way, outer channel exposed to open water	28.8 m
Two-way, inner channel protected water	42.0 m
Two-way, outer channel exposed to open water	58.8 m

The harbour entrance is clearly exposed to open water. The only decision that has to be made is the decision of a one or two way channel. With an eye on the future it will be better to construct a two way channel as it will be almost impossible to extend the harbour when the breakwaters are in the position of a one way channel.

This results in a width of the navigation channel of 58.8 m; this will be round up to 60.0 m. When in the future the decision is made to expand the size of the vessels above the current maximum dimensions ,as stated in chapter 4.1, the two-way navigation channel requires more than 60.0 m width. To overcome this issue the 60.0 m wide navigation channel can operate as a one-way channel instead of a two-way channel.

LENGTH OF BREAKWATERS

Normally the length of the breakwater is based on the navigability of the harbour. This method is based on the distance to the “breaker zone”. At the project site the slope of the coast is very steep making the distance to the “breaker zone” only about 200 meters. This is obtained from the SwanOne and observations. The sediment transport at the project site is very dynamic and depends on the direction of the sediment transport which changes every season. In case of using breakwaters with a length till the “breaker zone” this will result in sedimentation in the harbour during the dry or the wet season. Therefore in this case the length of the breakwaters is not based on the ‘’breaker zone’’ but is determined by the amount of sediment that has to be blocked each season. The sedimentation against the breakwater will be eroded for the most part in the next season due to the net sediment transport.

Sedimentation against the breakwater can be derived with two different methods. The first method is the single-line method. This method makes the assumption that part of the breakwaters final length is built on a flat bottom. The other method is called the non-parallel accretion method. This method assumes that the slope of the accreted profile is different than the original profile.

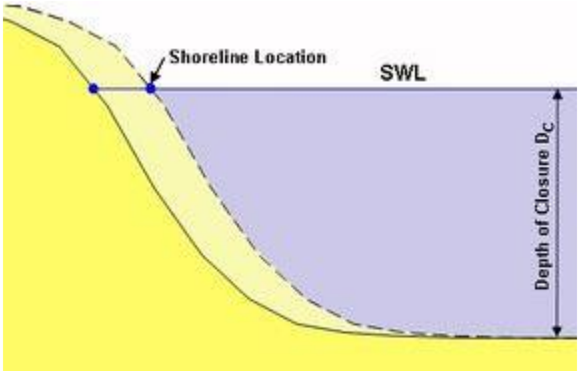


FIGURE F-1 SINGLE LINE METHOD

Assumption made on depth of closure

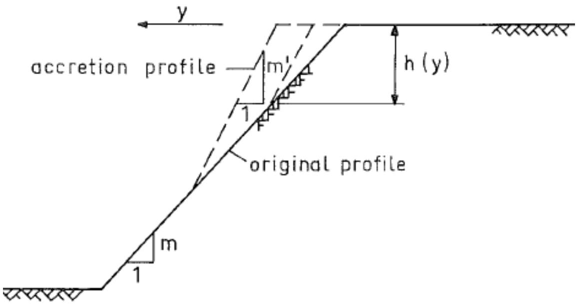


FIGURE F-2 NON-PARALLEL ACCRETION METHOD

Assumption made on slope of accreted profile

The single-line method would require a determination of the closure depth. The calculation of the closure depth is derived from the Hallermeier formula. This formula uses the offshore wave height which is exceeded for 12 hours per year. Due to the rough available wave data a realistic estimation is not possible to make. The wave data only consist of three data points, from here it is not possible to derive the wave height which is exceeded for 12 hours per year.

Hallermeier formula:

$$H = 2.28 \cdot H_{0.137} - 68.5\left(\frac{{H_{0.137}}^2}{g \cdot T^2}\right) \quad \text{(F.3)}$$

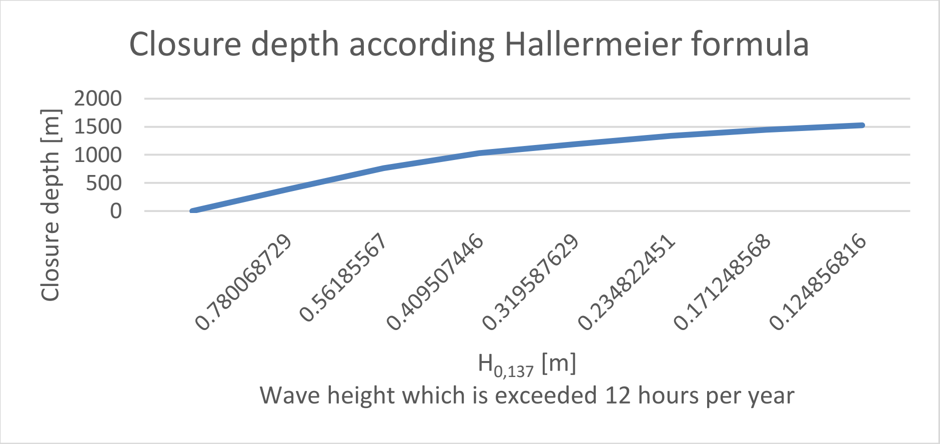


FIGURE F-3 CLOSURE DEPTH ACCORDING HALLERMEIER FORMULA

An assumption for this wave height will be around the 4 to 5 meters this will lead to a closure depth of 8 till 10 meters. This is the same order as the estimated depth of the breakwater. This means there is hardly any estimated horizontal distance before the toe. Therefore it is not advised to make use of the single-line method. For this reason the non-parallel method will be used. This method makes an estimation of the slope of the shoreline.

It is assumed that no sedimentation is allowed within the harbour basin. An extensive cost study in combination with numerical modelling of the project will give more insights. It can be the case that a shorter breakwater in combination with dredging activities is cheaper than a long breakwater. But, again, it is assumed that no sedimentation is allowed in the harbour basin.

In this case it means that the sediment will accrete against the western breakwater. However in the wet season this sediment will erode again because of the change of wave direction. The determination of the length of the western breakwater can be made by the following computation represented by a simplified form with a 10° wave angle with respect to the coastline. This simplification is also used in the length calculations of previous designs. However in the current length calculations there is no slope taken into account.

In the following calculations this simplification does have a slope. This slope is equal to the steepest slope of the current bathymetry. This assumption lies between the boundaries of no slope, as used in the calculations in previous review designs, and a natural slope. In addition a safety factor of 1.25 is used over the maximum sediment volume that has to be trapped.

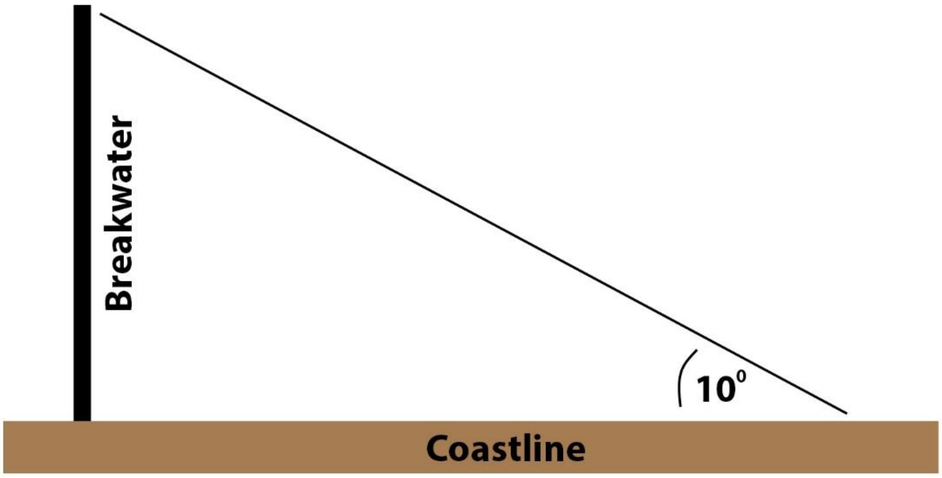


FIGURE F-4 SCHEMATISED ACCRETION ZONE

First the length of the western breakwater will be derived. After that the length of the eastern breakwater will be calculated.

LENGTH OF WESTERN BREAKWATER

The bathymetry of the coast west of the breakwaters is shown in Table F-2.

TABLE F-2 BATHYMETRY WEST OF BREAKWATERS

Depth [m]	Distance from coast [m]
0	0
-1	29.73
-2	38.50
-3	64.62
-4	133.28
-5	189.75
-6	218.48
-7	248.29
-8	266.25
-9	290.55
-10	316.95
-11	335.27
-12	360.14

THE SEDIMENT WILL ACCRETE AGAINST THE BREAKWATER RESULTING IN A FILLED SURFACE AGAINST THE BREAKWATER. THE SURFACE ALONG THE BREAKWATER PER METER DEPTH OF THE BREAKWATER IS SHOWN IN

Table F-5. In this figure it can be seen that the lines per meter depth are not vertical lines. These lines represent the natural slope of sediment which is assumed to be 1:9, or 6.4°. This is equal to the maximum natural slope in the current situation.

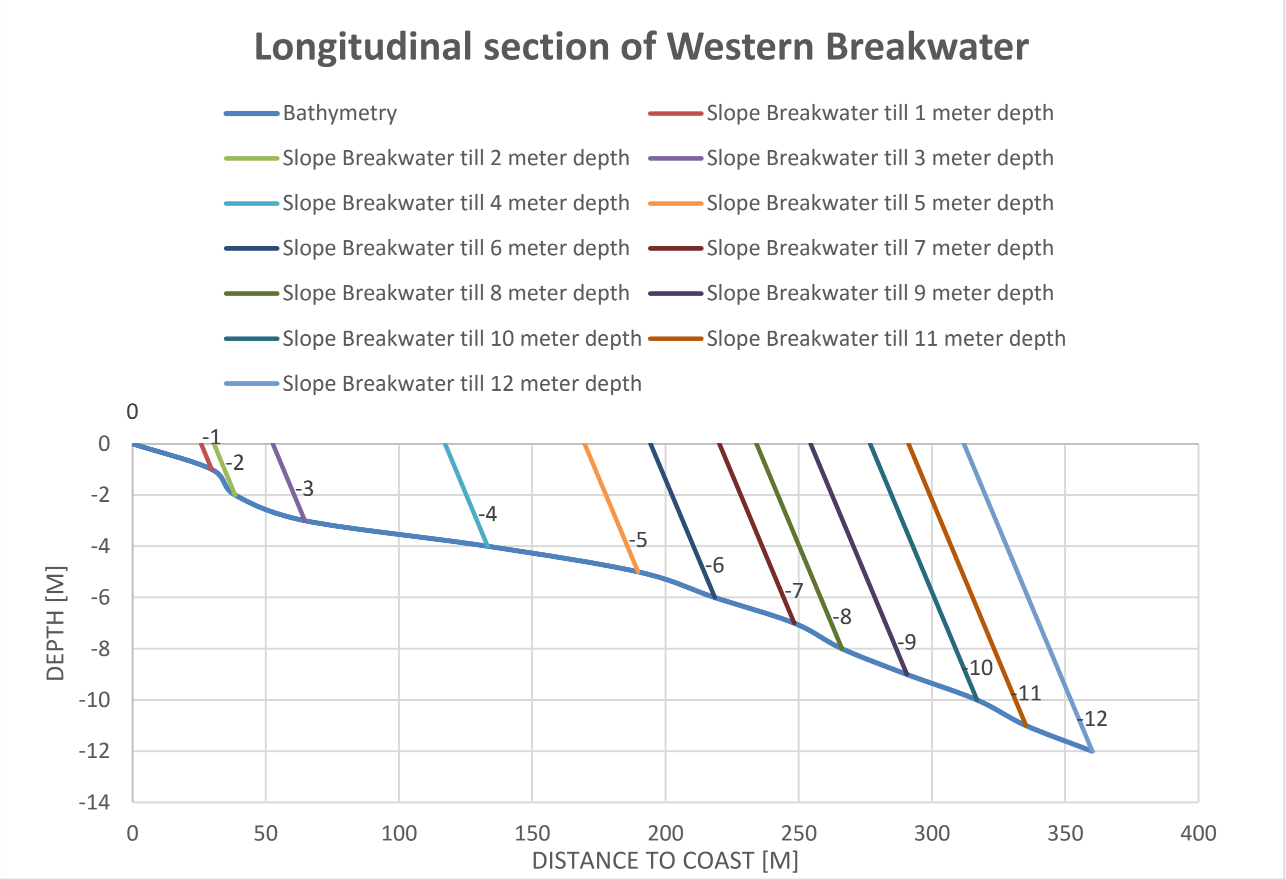


FIGURE F-5 LONGITUDINAL SECTION OF WESTERN BREAKWATER WITH DIFFERENT BREAKWATER LENGTHS

In the chapter 0 it is determined that the extreme amount of sediment coming from the west is 477,000 m^3/year. The sediment that has to be blocked has to be multiplied by a safety factor of 1.25 (Pustek Kelautan Universitas Gadjah Mada, 2003).

$$Volume = 477{,}000 * 1.25 \approx 600{,}000\ m^3 \tag{F.4}$$

The total amount of sediment is calculated by multiplying (1/3) of the surface against the breakwater with the distance of the influenced coast. This shape has the form of a tetrahedron.

$$Volume = \left(\frac{1}{3}\right) * A * B \tag{F.5}$$

In which:

- A is the surface against the breakwater which can be filled with sediment
- B is the length of the influenced coast

Both A and B are dependent of the length of the breakwater. In the following table the results are presented. From here it follows that a with a natural sediment slope of 6.4°, an angle of wave incidence of 10° and a required storage volume of 600,000 m^3 the length of the breakwater has to be 303.56 m. This may be rounded up to 305 m.

TABLE F-3 STORAGE VOLUMES WESTERN BREAKWATER

Depth [m]	Length [m]	A [m^2]	B [m]	Total Volume [m^3 *10^3]
0	0	0	0	0
-1	29.73	10.41	168.63	0.59
-2	38.50	20.67	218.35	1.50
-3	64.62	56.81	366.46	6.94
-4	133.28	195.23	755.85	49.19
-5	189.75	362.94	1076.14	130.19
-6	218.48	494.98	1239.09	204.44
-7	248.29	650.60	1408.10	305.36
-8	266.25	779.69	1509.95	392.43
-9	290.55	946.41	1647.79	519.83
-9.49	303.56	1039.17	1721.60	600.00
-10	316.95	1138.97	1797.50	682.43
-11	335.27	1304.62	1901.42	826.88
-12	360.14	1518.93	2042.44	1034.11

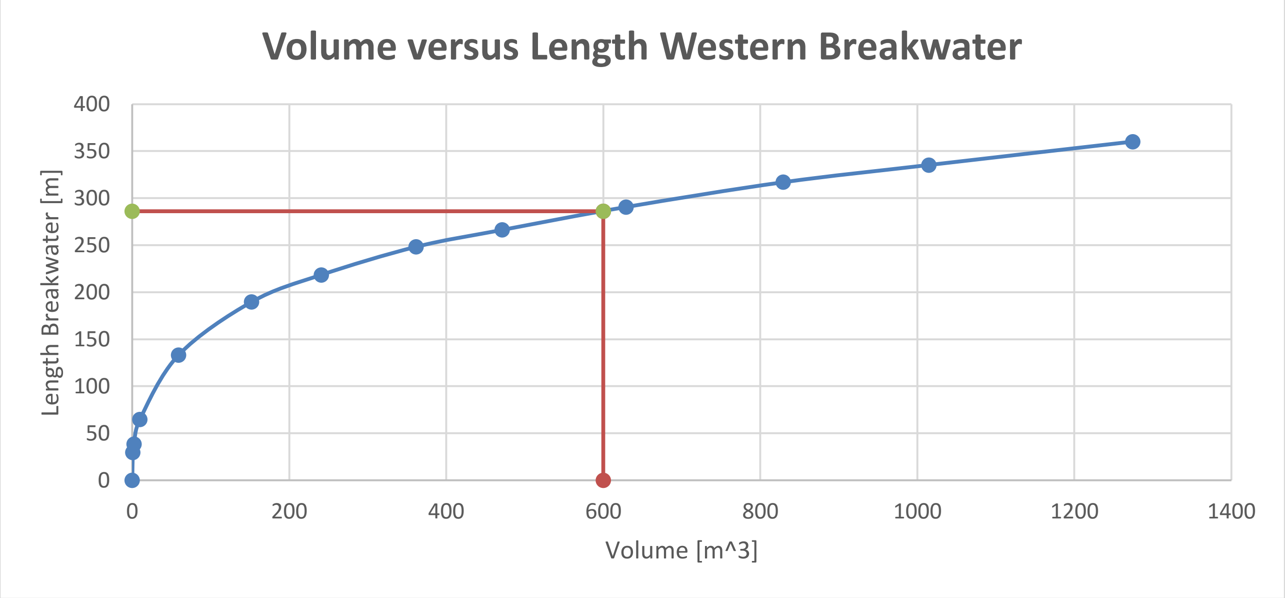


FIGURE F-6 VOLUME VERSUS LENGTH WESTERN BREAKWATER

The longitudinal section of the western breakwater will look like Figure F-7. The surface between both lines is equal to 1039.17 m^2. This is the A which is found by intersection.

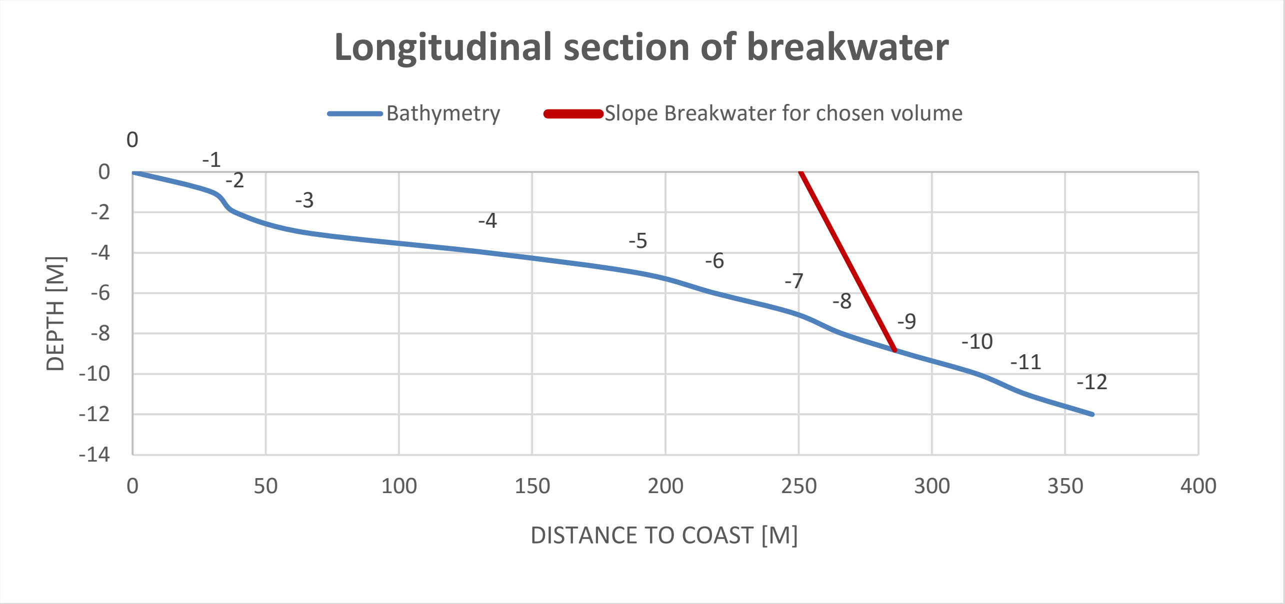


FIGURE F-7 LONGITUDINAL SECTION OF WESTERN BREAKWATER WITH PROPOSED BREAKWATER LENGTH

LENGTH OF EASTERN BREAKWATER

The main wave direction is from the south-east as can be read in Appendix B.2. The net sediment transport is from the east to the west as can be read in chapter 0. The navigation channel needs to be protected from the waves but also the yearly sediment transport has to be blocked. Therefore first the minimum length for sediment blockage will be determined. After this a check will be done if the navigation channel is protected enough.

For the determination of the sediment storage against the eastern breakwater it is assume that the angle of incoming waves is 10°. This is the same as in the calculation of the western breakwater but this time it is from the east instead of the west.

The bathymetry of the coast east of the breakwaters is shown in Table F-4.

TABLE F-4 BATHYMETRY EAST OF THE BREAKWATERS

Depth [m]	Distance from coast [m]
0	0
-1	13.67
-2	17.42
-3	24.99
-4	33.81
-5	48.49
-6	72.80
-7	113.09
-8	130.64
-9	147.35
-10	195.75
-11	224.58
-12	233.57
-13	256.27
-14	273.75

-15	302.85
-16	351.21

The slope of sediment stored against the eastern breakwater is assumed to be 1:4. This is the steepest natural slope in the current situation. Per meter depth this gives the depth contour lines as shown in Figure F-8.

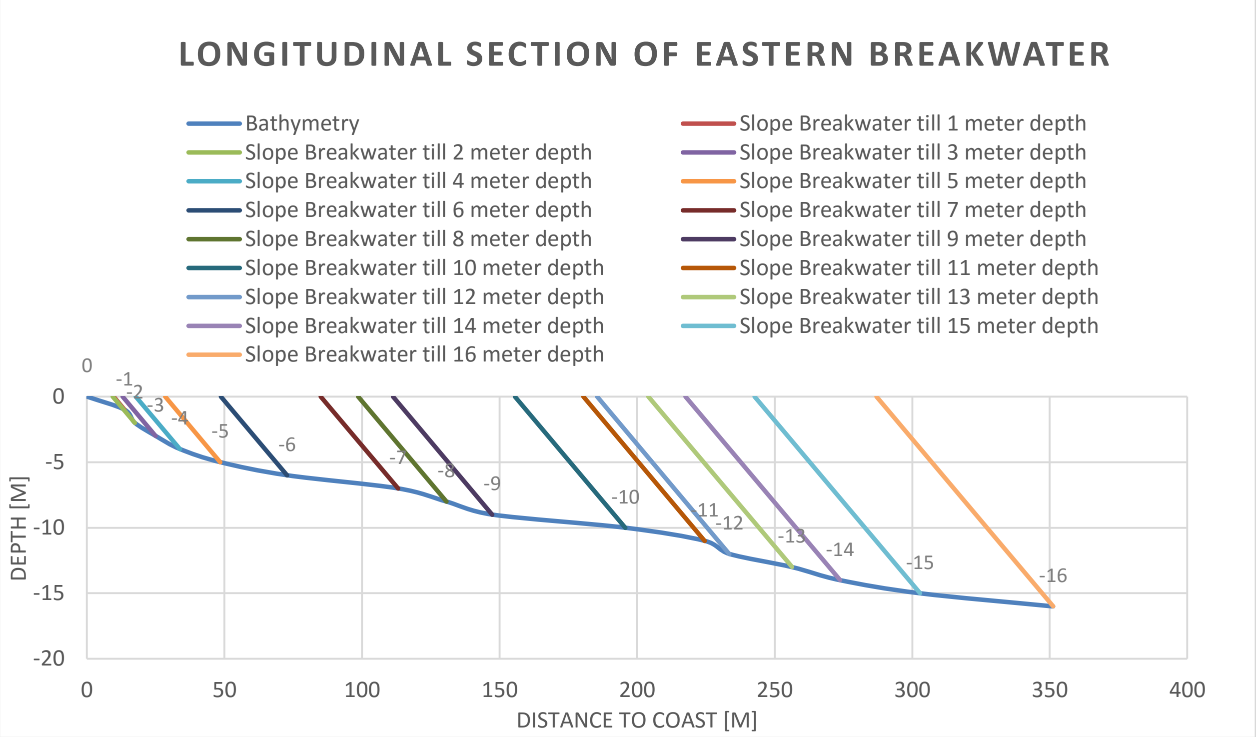


FIGURE F-8 LONGITUDINAL SECTION OF EASTERN BREAKWATER WITH DIFFERENT BREAKWATER LENGTHS

The same safety factor as before is applied to the gross transport from the east to the west.

$$Volume = 1.028.000 * 1.25 = 1.285.000\ m^3 \qquad (F.6)$$

Table F-5 shows us the A and B factors for the determination of the total volume per meter depth. The volume can be derived with the following calculation.

$$Volume = \left(\frac{1}{3}\right) * A * B \qquad \text{(F.7)}$$

TABLE F-5 STORAGE VOLUMES EASTERN BREAKWATER

Depth [m]	Length [m]	A [m^2]	B [m]	Total Volume [m^3 *10^3]
0	0	0	0	0
-1	13.67	4.83	77.54	0.12
-2	17.42	9.40	98.79	0.31
-3	24.99	19.44	141.73	0.92
-4	33.81	35.53	191.74	2.27
-5	48.49	71.08	274.97	6.51
-6	72.80	146.22	412.90	20.12
-7	113.09	297.54	641.35	63.61
-8	130.64	394.23	740.91	97.36
-9	147.35	500.65	835.68	139.46
-10	195.75	778.19	1110.13	287.96
-11	224.58	992.54	1273.66	421.38
-12	233.57	1112.66	1324.66	491.30
-13	256.27	1326.82	1453.36	642.78
-14	273.75	1523.18	1552.51	788.25
-15	302.85	1820.19	1717.57	1042.10
-15.5	327.20	2054.33	1855.63	1285.00
-16	351.21	2296.29	1991.80	1524.59

Both A and B are dependent of the length of the breakwater. From here it follows that a with a natural sediment slope of 14°, an angle of wave incidence of 10° and a required storage volume of 1.285.000 m^3 the length of the breakwater has to be 327.20 m. This can be rounded to 330 m.

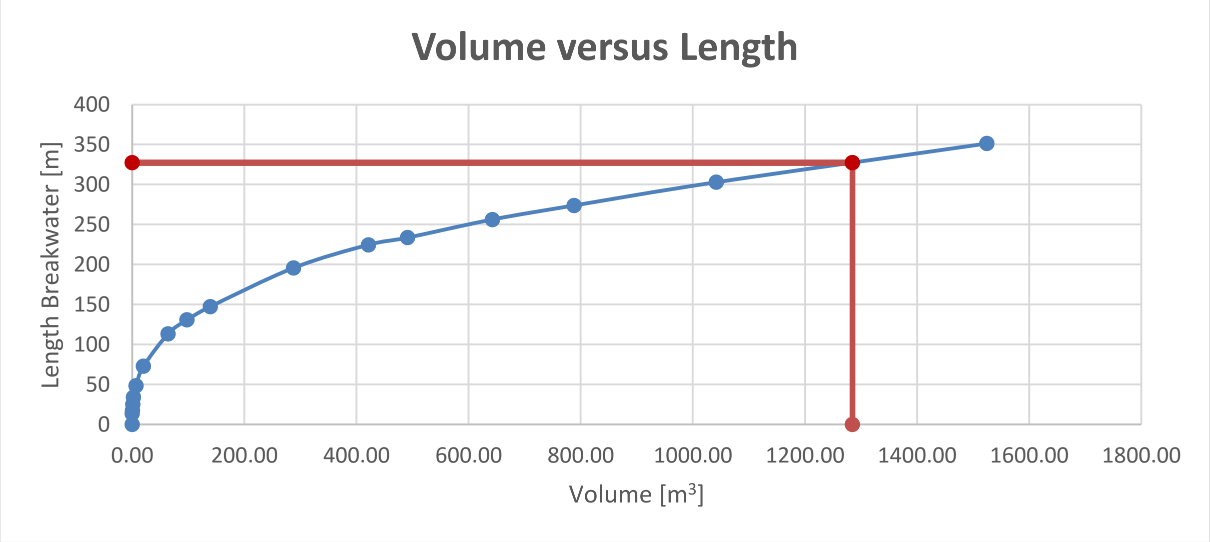


FIGURE F-9 VOLUME VERSUS LENGTH EASTERN BREAKWATER

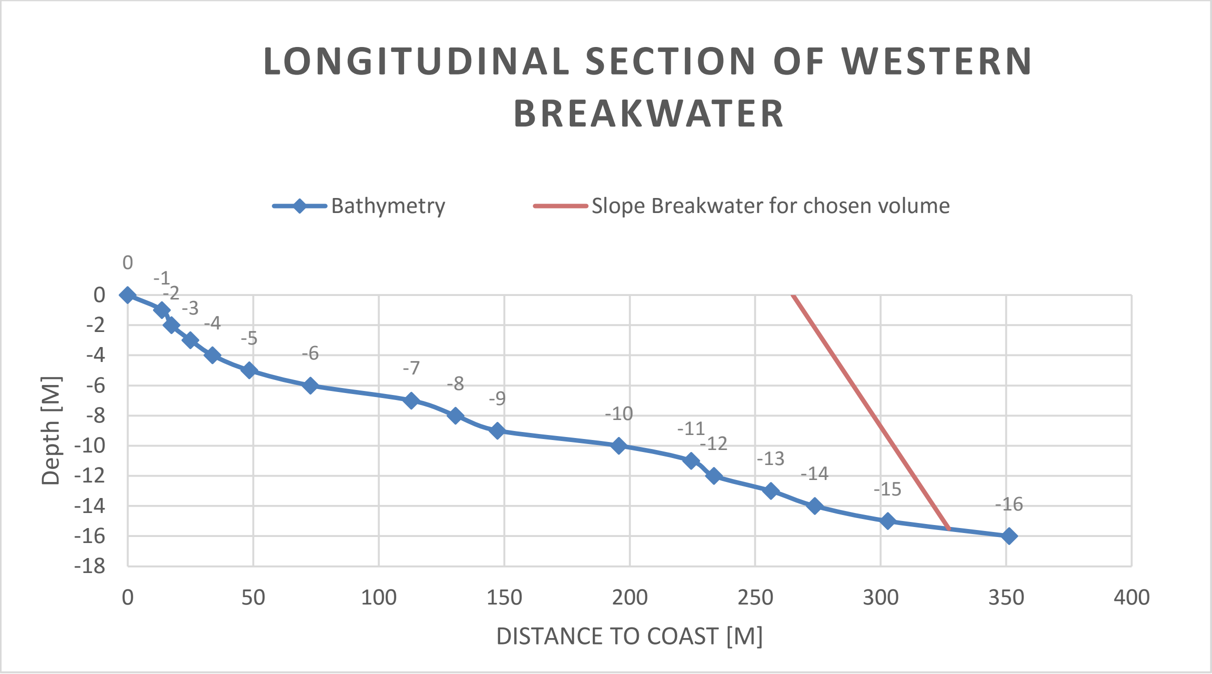


FIGURE F-10 LONGITUDINAL SECTION OF WESTERN BREAKWATER WITH PROPOSED BREAKWATER LENGTH

CONCLUSION ORIENTATION OF THE BREAKWATERS

The minimum breakwater length is derived by the requirement of seasonal sediment storage. In the wet season (April to October) the sediment transport from west to east is normative. In the dry season (November to March) this is the case for the sediment transport from the east to the west. This results in the following minimum breakwater lengths:

Western breakwater	305 m till a depth of 9.50 m
Eastern breakwater	330 m till a depth of 15.50 m

For the eastern breakwater it has to be noticed that this is a minimum required length as this breakwater also has to protect the navigation channel against wave attacks. Because of the change of bathymetry along the coast and the direction of the breakwater to protect the navigation channel against wave attacks the total length of the breakwater will increase. A top view of the final breakwater position is shown in Figure F-11.

FIGURE F-11 OVERVIEW PROPOSED BREAKWATERS

It can be seen that the depth contours shift around the breakwaters. Therefore the desired depth is taken as the guidance for the length. The calculation of the length of the western breakwater is based on bathymetry west of the harbour area and the calculation of the length of the eastern breakwater is based on bathymetry east of the breakwater. Due to the change of bathymetry around the breakwater the western breakwater reaches its desired depth closer to the coast than 305 m. The eastern breakwater however needs to be longer in order to reach its desired depth. The total lengths perpendicular to the coast are:

▪ Western breakwater	272 m till a depth of 9.50 m
▪ Eastern breakwater	370 m till a depth of 15.50 m

First, both lengths are longer than the width of the “breaker zone” as assumed before. This justifies the assumption of taking the sediment transport as normative over the navigability.

Besides that it has to be noted that these lengths are an indication as there are lots of assumptions and simplifications made in this calculation.

- The non-parallel accretion makes a rough assumption about the slope of the accreted profile. As mentioned before an assumption of the closure depth can’t be made in this case.
- It is assumed that the accreted sediment will be eroded and dredged after each cycle of seasons. Therefore every year starts from a zero-case where there is no accreted or eroded shoreline left.

- The shape of the accreted sediment is simplified to a triangle in order to makes this calculation workable by hand. The simplified triangle results in a very conservative length of the breakwater.

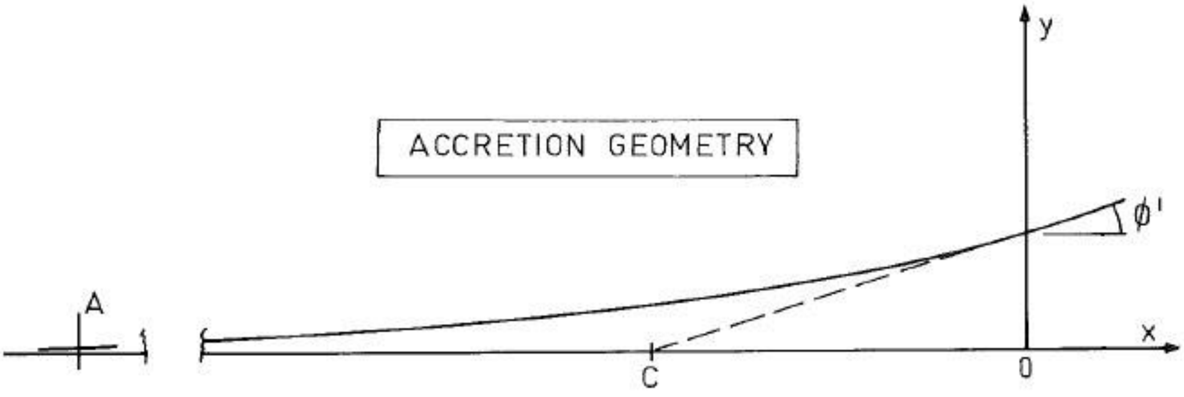


FIGURE F-12 SIMPLIFIED CALCULATION VERSUS COPLEX CALCULATION

No modelling is done during this project. Modelling can give more insights in the behaviour of the sediments around the breakwaters. It is recommended to model this case as it is a very dynamic coast with a varying sediment transport.

F.2 DETERMINATION OF HEIGHT OF BREAKWATERS

Figure F-13 shows all the variables that have to be included in the determination of the height of the breakwaters.

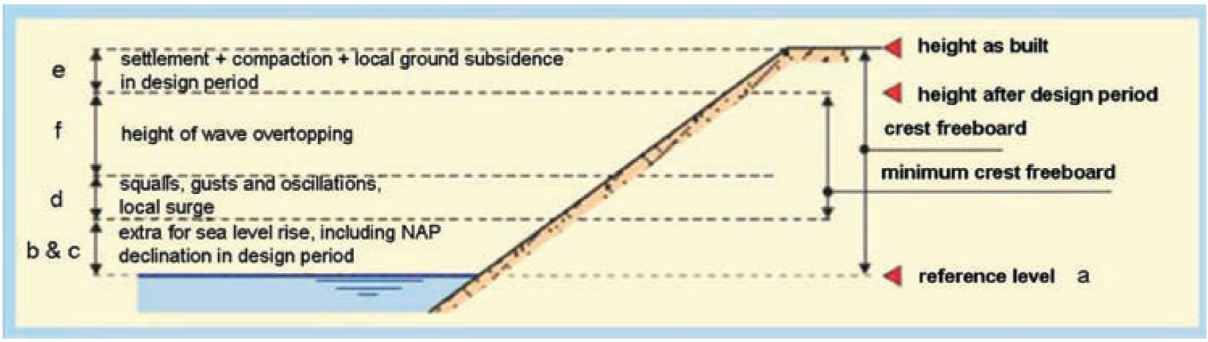


FIGURE F-13 DETERMINATION OVERTOPPING (EUROTOP, 2007)

First the minimum crest freeboard is calculated. A high overtopping discharge can cause damage to seawalls, buildings and infrastructure. It can also be a danger to pedestrians and vehicles on the breakwaters during storm. It is assumed that there are no pedestrians or vehicles on the breakwaters during storm.

The limits of overtopping for vessels and buildings behind the breakwaters is shown in Table F-6 (EurOtop, 2007).

TABLE F-6 LIMITS OF OVERTOPPING FOR VESSELS AND BUILDINGS

Hazard type and reason	**Mean discharge q [l/s/m]**

Significant damage or sinking of larger yachts	50.0
Sinking small boats set 5 - 10 m from wall. Damage to larger yachts	10.0
Building structure elements	1.0
Damage to equipment set back 5 - 10 m	0.4

The limits of overtopping for damage to the crest of the breakwaters or rear slope are shown in Table F-7 (EurOtop, 2007).

TABLE F-7 LIMITS OF OVERTOPPING FOR CREST DAMAGE

Hazard type and reason	Mean discharge q [l/s/m]
Embankment seawalls/sea dikes	
No damage if crest and rear slope are well protected	50.0 - 200.0
No damage to crest and rear face of grass covered embankment of clay	1.0 - 10.0
No damage to crest and rear face of embankment if not protected	0.1
Promenade or revetment seawalls	
Damage to paved or armoured promenade behind seawall	200.0
Damage to grassed or lightly protected promenade or reclamation cover	50.0

The vessels and buildings in Table F-6 are directly behind the concerned breakwater. This is not the case in our situation as the vessels and buildings are in the harbour basin which is quite a distance from the breakwaters. The current rear slope of the breakwaters can be qualified as well protected therefore a well assumption for the maximum acceptable mean discharge over the breakwaters is 50 l/s/m.

The overtopping height of the breakwaters can be derived by the following formula:

$$\frac{q}{\sqrt{g * H_{m0}^3}} = 0.2 * \exp(-2.6\frac{R_C}{H_{m0} * \gamma_f * \gamma_\beta}) \tag{G.8}$$

In which:

- q = Mean overtopping discharge [m^3/s/m]
- g = Gravity
- H_{m0} = Incoming wave height
- R_C = Freeboard
- γ_f = Friction Coefficient
- γ_β = Coefficient for angle of incidence

The maximum allowable mean overtopping discharge is 50 l/s/m and the gravity is 9.81 m^2/s. The incoming wave height is assumed to be 6.38 m as is concluded in chapter 3.2. The friction parameter for tetrapods is 0.38 (CIRIA, 2007) and the coefficient for the angle of incidence is assumed to be 1. From this it results that a freeboard of about 5.6 m is needed.

Together with a storm height of +2.76 m LWS this results in a breakwater height of +8.36 m LWS. This is without taking into account sea level rise and settlement of the structure. Sea level rise is assumed to be zero as explained in Appendix B.4. We assume a total breakwater elevation level of +8.5 m LWS. This means 0.14 m is given for settlement, compaction, and land declination.

In Table F-8 a summary all the elevation levels is given.

TABLE F-8 BREAKWATER ELEVATION LEVEL

	Breakwater Elevation Level	
A	Reference level	+2.76 m LWS
F & D	Freeboard height	5.60 m
B & C & E	Settlement + Compaction + Sea level rise + Land declination	0.14 m
	Total:	+8.50 m LWS

F.3 DETERMINATION CONCRETE ARMOUR LAYER OUTER SLOPE

The required dimensions of the concrete armour units on the outer trunk will be calculated using the Van Der Meer stability formula for tetrapod armour units. This formula is based on the stability formula for rock layers.

TETRAPOD NOMINAL DIAMETER TO WEIGHT RATIO

For our project the determination of the ratio between the nominal diameter and weight of the tetrapod with respect to the height of the tetrapod is important to know as the outcome of the design formula of the tetrapods is the height of the tetrapod.

The standard formula that is used to determine the volume of a tetrapod is: (Ir. Verhagen)

$$V = 0.280 * H^3 \tag{F.9}$$

In which:

- H = overall height of tetrapod [m]

In the following paragraphs the required nominal diameter is calculated. This nominal diameter is not the actual overall height. To convert the nominal diameter into the overall height the following formula is used:

$$H = \frac{D_n}{0.65} \tag{F.10}$$

In which:

- D_n = nominal diameter [m]

Based on the above formulas the weight of a tetrapod can be calculated using:

$$W = 0.280 * \left(\frac{D_n}{0.65}\right)^3 * \frac{\rho_c}{1000} \tag{F.11}$$

In which:

- W = tetrapod weight [ton]
- D_n = nominal diameter [m]
- ρ_c = density of concrete [kg/m^3]

The concrete used for the tetrapods is K-400 concrete. Generally, the density of concrete is around 2600 kg/m^3. The density of this concrete is assumed to be 2400 kg/m^3.

VAN DER MEER FORMULA

The stability formula for tetrapods is given by: (Van der Meer, 1999)

$$\frac{H_s}{\Delta D_n} = \left(3.75\left(\frac{N_{od}}{\sqrt{N}}\right)^{0.5} + 0.85\right) S_{om}{}^{-0.2} \tag{F.12}$$

In which:

- H_s = Significant wave height in front of the structure
- Δ = Relative mass density
- D_n = The size of the cube. For tetrapods D_n = 0.65 D, where D is the height of the tetrapod
- N_{od} = Relative damage, the number of armour units that are displaced related to a width
- N = Number of waves
- S_{om} = Wave steepness

The Van der Meer formula is based on several assumptions. The formula is based on:

- A slope of 1:1.5.
- A two layer system for tetrapod armour units.
- A structure with almost no overtopping (less than 15%).

PARAMETERS

WAVE HEIGHT

As explained in Chapter 2.2 the significant wave height for the breakwater design is equal to 6.38 m for a 1/250 year storm. A normal distribution will be used with a mean value of 6.38 m with a standard deviation of 0.25 to account for the unreliability of the data.

Commonly, a Weibull distribution is used for waves. However, as explained in Appendix B.2.1, the 1/250 years storm wave height is based on an exponential extrapolation. It was possible to perform a Weibull extrapolation due to the lack of data. The required α- and γ-values could not be determined so it was simply not possible to use a Weibull distribution in this case. This is why a normal distribution is chosen as alternative.

RELATIVE MASS DENSITY

The relative density can be derived by using the following formula:

$$\Delta = \frac{\rho_{concrete} - \rho_{seawater}}{\rho_{seawater}} \tag{F.13}$$

In which:

- $\rho_{concrete}$ = The density of concrete
- $\rho_{seawater}$ = The density of seawater

As mentioned before, the density of this concrete is assumed to be around 2400 kg/m^3. The standard deviation is chosen as 100 kg/m^3.

The density of seawater is equal to 1030 kg/m^3. The standard deviation for the density of water is assumed to be 5 kg/m^3.

This leads to a relative density of 1.33.

RELATIVE DAMAGE

N_{od} damage levels for tetrapods which weigh less than 25 tons are classified as follows:

- Start of damage: 0.0
- Initial damage (needs no repair): 0.0 - 0.5
- Intermediate damage (needs repair): 0.5 - 1.5
- Failure (core exposed): > 2.0

Designing for no damage is a strict criterion and will lead to a design with very large tetrapods. A more economical design would be for N_{od} = 0.5. This is a deterministic value.

NUMBER OF WAVES

The number of waves in a storm will be maximized till the number of 7500. At this value the influence of the number of waves will be limited as it reaches an equilibrium. As is told in the citation below:

"After this number of waves the structure more or less has reached an equilibrium. This means that damage for more than 7500 waves is found by using N = 7500 in the equations." (Pilarczyk, 1998)

When assuming an average storm lasts for 12 hours causing waves with periods of about 15 s, a total number of waves during a storm is around 2880.
We assume a triangular distribution with a lower limit of 2880, an upper limit of 7500 and a mode of 7500. This ensures that the distribution does not exceed 7500.

WAVE STEEPNESS

The wave steepness can be derived using the following formula: (Van der Meer, 1999)

$$S_{om} = \frac{2\pi H_s}{g T_p^2} \qquad \text{(F.14)}$$

In which:

- H_s = Significant wave height
- g = Gravity
- T_p = Wave period

When assuming the significant wave height is 6.38 m, gravity 9.81 m^2/s and a wave period of 15 s, the wave steepness is 0.0182. The wave steepness is assumed to be normal distributed with a standard deviation of 0.001.

BLCOK THICKNESS

This is the value that has to be found. A normal distribution is assumed. The blocks are produced accurately so a standard deviation of 0.01 is chosen.

The following table gives a summary all parameters including the mean value and standard deviation.

TABLE F-9 DISTRIBUTIONS PER PARAMETER

Parameter	Type	Mean	Standard deviation
H_s	Normal	6.38	0.25

N_{od}	Deterministic	0.5	-
N	Lognormal	7500	2880, 7500
S_{om}	Normal	0.0182	0.001
$\rho_{concrete}$	Normal	2400	100
$\rho_{seawater}$	Normal	1030	5
D_n	Normal	?	0.01

F.3.4 STABILITY CALCULATION

To compute the required tetrapod size a classical computation, a deterministic approach and a probabilistic approach will be used.

CLASSICAL COMPUTATION

In the classical computation the design formula is directly applied using a design wave height based on a specified storm frequency. The Van der Meer formula is rewritten as:

$$D_n = \frac{H_s}{\left(\left(3.75\left(\frac{N_{od}}{\sqrt{N}}\right)^{0.5} + 0.85\right) S_{om}{}^{-0.2}\right) * \Delta} \qquad \text{(F.15)}$$

Filling in equation (F.15) gives a required D_n of 1.90 m. This leads to a required tetrapod weight of 16.78 tons which would mean a tetrapod of 17 tons is used.

DETERMINISTIC

In the deterministic approach partial safety factors for load (γ_{HSS}) and strength (γ_z) are added to the design formula. The formula becomes:

$$D_n = \frac{H_s}{\left(\left(3.75\left(\frac{N_{od}}{\sqrt{N}}\right)^{0.5} + 0.85\right) S_{om}{}^{-0.2}\right) * \Delta} * \gamma_{HSS} * \gamma_Z \qquad \text{(F.16)}$$

PARTIAL SAFETY COEFFICIENT FOR LOAD

This coefficient can be calculated using the following formula:

$$\gamma_{H_{SS}} = \frac{H_{SS}{}^{tpf}}{H_{SS}{}^{tL}} + \sigma'_{QL}\left(1 + \left(\frac{H_{SS}{}^{3tL}}{H_{SS}{}^{tL}} - 1\right) k_\beta P_f\right) + \frac{0.05}{\sqrt{P_f N}} \qquad \text{(F.17)}$$

In which:

- $H_{SS}{}^{tpf}$ = Significant wave height for the allowable failure during lifetime.
- σ'_{Q_L} = Standard deviation as a function of the type of observations available
- $H_{SS}{}^{tL}$ = Significant wave height for a return period of the design storm equal to one life time
- $H_{SS}{}^{3tL}$ = Significant wave height for a return period of the design storm equal to three life times
- k_β = A design coefficient determined by optimization.
- P_f = The allowable probability of failure during one lifetime.

k_β is given in the PIANC manual (PIANC, 1997) . For the Van der Meer formula using tetrapods k_β is equal to 38. As explained in paragraph 0 P_f is equal to 18.1% or 0.181. This leads to a $\gamma_{H_{ss}}$ of 1.229.

PARTIAL SAFETY COEFFICIENT FOR STRENGTH (Verhagen & D'Angremond, 2012)

This coefficient can be calculated using the following formula:

$$\gamma_z = 1 - \left(k_\alpha \ln P_f\right) \tag{F.18}$$

In which:

- k_α = A design coefficient determined by optimization.
- P_f = The allowable probability of failure during one lifetime.

Just as k_β, k_α is given in the PIANC manual (PIANC, 1997). For the Van der Meer formula using tetrapods k_α is equal to 0.026. Just like in the partial safety coefficient for load the value for P_f is equal to 18.1% or 0.181. This leads to a γ_z of 1.044.

Filling in equation G.16 gives a required D_n of 2.433 m. This leads to a required tetrapod weight of 35.24 tons which would mean a tetrapod of 36 tons has to be used. This is a very large weight.

PROBABILISTIC

The probabilistic calculation will be made using a level III fully probabilistic Monte Carlo Simulation. The results of this calculation are reliable as long as they're done by computer programming software. In order to perform a probabilistic calculation a limit state function needs to be defined. The limit state function is given as Z = R – S in which R represents the strength of the system and S the load that acts on the system. The limit state function is based on the Van der Meer formula.

This leads to the following limit state function:

$$Z = \left(3.75\left(\frac{N_{od}}{\sqrt{N}}\right)^{0.5} + 0.85\right) S_{om}{}^{-0.2} - \frac{H_s}{\Delta D_n} \tag{F.19}$$

With the following Matlab script the Monte Carlo Simulation is performed using the mean values and standard deviations given for each parameters.

Calculation script:

```
function z = probabilistic_x2z(varargin)
samples = struct(...
    'RhoC', [],...       % [kg/m3] RhoC density concrete
    'RhoW', [],...       % [kg/m3] RhoW density water
    'N',    [],...       % [-] Number of waves
    'Nod',  [],...       % [-] Damage level
    'som',  [],...       % [-] Wave steepness
    'Hs',   [],...       % [m] Significant wave height
    'Dn',   []);         % [m] Block size
samples = setproperty(samples, varargin{:});
%% calculate z-values
% pre-allocate z
    z = nan(size(samples.RhoC));
% loop through all samples and derive z-values
for i = 1:length(samples.RhoC)

    %Z-function: Hs/(AA*Delta)
    AA       = (3.75*(samples.Nod(i)*(samples.N(i)^-
0.5))^0.5+0.85)*samples.som(i)^-0.2;     % [-] Part of Van Der Meer
```

```
    Delta   = (samples.RhoC(i) - samples.RhoW(i)) / samples.RhoW(i);
% [-] relative density


    z(i,:)  = AA - samples.Hs(i)/(Delta*samples.Dn(i));

end
```

Input script:

```
function [resultMC] = probabilistic
stochast = struct(...
    'Name', {   % define the stochastic variable names:
    'RhoC'...           % [kg/m3] RhoC density concrete
    'RhoW'...           % [kg/m3] RhoW density water
    'N'...              % [-] Number of waves
    'Nod'...            % [-] Damage level
    'som'...            % [-] Wave steepness
    'Hs'...             % [m] Significant wave height
    'Dn'...             % [m] Block size
    },...
    'Distr', {  % define the probability distribution functions
    @norm_inv...        % [kg/m3] RhoS density concrete
    @norm_inv...        % [kg/m3] RhoW density water
    @trian_inv...       % [-] Number of waves
    @deterministic...   % [-] Damage level
    @norm_inv...        % [-] Wave steepness
    @norm_inv...        % [m] Significant wave height
    @norm_inv...        % [m] Block size
    },...
    'Params', { % define the parameters of the probability distribution
functions
    {2400 100}...       % [kg/m3] RhoS density concrete
    {1030 5}...         % [kg/m3] RhoW density water
    {2880 7500 7500}... % [-] Number of waves
    {0.5}...            % [-] Damage level
    {0.01729 0.001}...  % [-] Wave steepness
    {6.38 0.25}...      % [m] Significant wave height
    {2.00 0.01}...      % [m] Block size
    },...
    'propertyName', { % specify here to call the z-function with propertyname-
propertyvalue pairs
    true...             % [kg/m3] RhoS density concrete
    true...             % [kg/m3] RhoW density water
    true...             % [-] Number of waves
    true...             % [-] Damage level
    true...             % [-] Wave steepness
    true...             % [m] Significant wave height
    true...             % [m] Block size
    } ...
    );
%% main matter: running the calculation
% run the calculation using Monte Carlo
resultMC = MC(...
    'stochast', stochast,...
    'NrSamples', 3e4,...
    'x2zFunction', @probabilistic_x2z);
```

The simulation creates 30.000 samples from the distributions R and S, taking into account the probability distribution. The results of a Monte Carlo simulation are always different because of the random sampling.

Still, the results are very similar. For the stability calculation we are interested in the required size of the tetrapods.

For the stability calculation we are interested in the required size of the tetrapods. The following table shows the failure probability for different block sizes. This failure probability is an average of various simulations for the same block thickness.

TABLE F-10 BLOCK THICKNESS VERSUS FAILURE PROBABILITY

Block thickness D_n (m)	Failure probability (%)
1.90	36.88
1.92	32.57
1.94	28.34
1.96	24.68
1.98	21.16
2.00	17.91
2.02	15.30
2.04	12.67

The design is based on a storm with a return period of 1/250 years, which equals to a probability of exceedance of 18.1%. So, the minimum D_n is also based on a probability of exceedance around this percentage. A block thickness of 2.00 m is required in order to stay within the accepted probability of failure of less than 18.1%. This leads to a required tetrapod weight of 19.57 tons which would mean a tetrapod of 20 tons is used.

A histogram is made of the results using 100 bins, giving the distribution of the limit state function Z. everything left of the vertical line are failure cases. For this situation this is around 18 % of the total amount of samples. Figure F-14 shows this histogram.

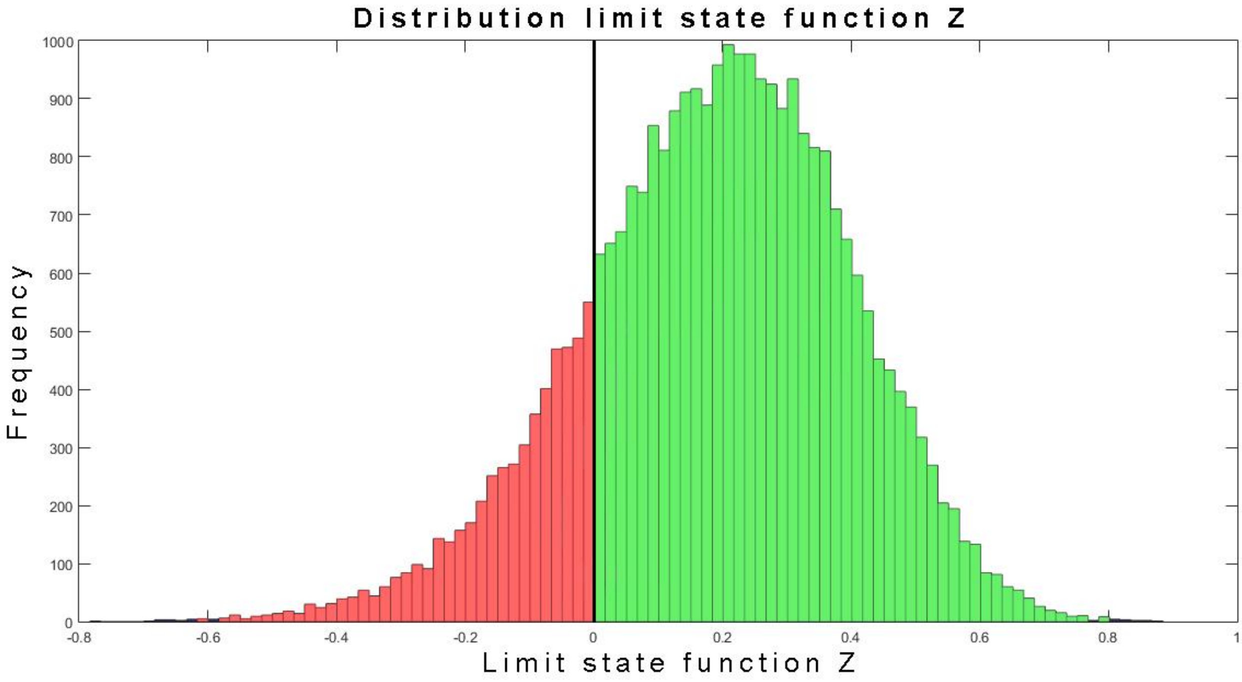


FIGURE F-14 DISTRIBUTION OF MONTE CARLO SIMULATION

SUMMARY CONCRETE ARMOUR CALCULATION

The following table gives the required D_n and weight of the tetrapods for each different calculation. What can be noted is the outlying value of the weight determined with the deterministic approach. This happens because D_n is increased due to the safety factors. Since D_n is multiplied by the power of 3 to get to the weight of the tetrapod the safety factors have a large effect on the tetrapod weight. A conservative tetrapod weight of 20 tons will be used for the design.

TABLE F-11 SUMMARY OF CALCULATIONS

Type of calculation	D_n [m]	Tetrapod weight [t]	Tetrapod weight, rounded up [t]
Classical	1.896	16.78	17
Deterministic	2.433	35.24	36
Probabilistic	2.00	19.57	20

F.4 CONCRETE ARMOUR LAYER CALCULATION INNER SLOPE

The armour layer of the inner slope does not have to be as heavy as the outer slope since it is not directly impacted by waves. The armour size of the inner slope is based on overtopping. The ratio between the freeboard Rc and the size of the armour D_n gives an indication of the safety of the slope. The worst conditions exist for a ratio between 0 and 1. The inner slope is relatively safe when the ratio is larger than 4. This means that for a freeboard of 5.6 m as determined in Appendix F.2 the D_n has to be around Rc / 4 = 1.4 m. This leads to a required tetrapod weight of 6.71 tons which would mean a tetrapod of 7 tons is used.

F.5 CONCRETE ARMOUR LAYER CALCULATION HEAD OF STRUCTURE

The head of the structure is most affected by the waves. Due to the curvature at the head, the tetrapods are less interlocking making them more vulnerable to attacks. It is therefore necessary to increase the weight of the tetrapods at this position. For this design the common design rule is used where the weight of the tetrapods at the head is 1.25 times more than the weight at the trunk. Using 20.0 tons tetrapods for the outer layer leads to a weight at the head of 20.0 * 1.25 = 25.0 tons.

F.6 DETERMINATION OF THE FIRST UNDERLAYER

The first under-layer will be made up out of rocks as they are easily obtainable at the quarries near the project site. The general rule for the first under-layer is that this layer should not be less than 1/10 of the weight of the armour layer. When following the filter rule of Terzaghi, which states a diameter ratio of 4 to 5, the layer should be between 1/64 and 1/125 of the weight of the armour layer. It is recommended to stay between 1/10 and 1/25.

The tetrapods at the outer layer are considered when determining the first under-layer. These tetrapods are 20.0 tons which means that the first under-layer should be between about 0.8 and 2.0 tons. A rock gradation of 1.0 - 2.0 tons will be used for the design.

The thickness of the first under-layer is determined by the basic design rule that it should minimally be 2 times the nominal diameter of the rocks. The largest rock size within the rock gradation is used which leads to a conservative thickness. The D_n of 2.0 tons rocks is 0.75 m this means that the thickness of the first under-layer should be 1.50 m.

F.7 DETERMINATION OF THE CORE

The core will also be made out of rock for the same reason as the first under-layer. The core is chosen to be so that it cannot pass through the first under-layer. For this, the same weight ratio between the layers of

1/10 to 1/25 is often used. A core rock gradation of 0.5 - 1.0 ton will be used. This is heavier than the recommended weight ratio. However, if the core is too light this may give difficulties during construction.

F.8 DETERMINATION OF THE TOE BERM

The toe berm consists of a filter layer with a double layer of tetrapods on top. The filter layer in the original design is equal to 0.20 m. For consistency purposes the same layer thickness is used in the draft design.

The required tetrapods of the toe at the outer slope and the head is determined according the formula of Van der Meer, D'Angremond and Gerding (Verhagen & D'Angremond, 2012). They investigated the relation between the unit weight of toe elements, toe level and damage. The result of this investigation is formula (F.20) for toes which are not too deep (only for h_t/H_s< 2).

$$\frac{H_s}{\Delta d_{n50}} = (6.2 \, (\frac{h_t}{h})^{2.7} + 2)N_{od}^{0.15} \qquad 0.4 < \frac{h_t}{h} < 0.9 \tag{F.20}$$

In which:

- H_s: The significant wave height.
- Δ: Relative density of the toe units.
- d_{n50}: The nominal diameter of the toe units, $d_{n50} = d_n$ because the toe units are made of concrete and the units do not consist of quarry stones.
- h_t: The waterdepth above the toe. This can be rewritten as $h_t = h - 3d_{n50}$. This because the height of a standard toe is equal to $3d_{n50}$.
- h: The waterdepth in front of the toe.
- N_{od}: The damage number

Because the formula is only available for h_t/H_s< 2 and $0.4 < \frac{h_t}{h} < 0.9$ it is not possible to design the total breakwaters over its full length. Therefore some iteration steps have to be made. Formula (G.21) shows the total iteration.

$$d_{n50} = \frac{H_s}{\Delta(6.2 \, (\frac{h - 3d_{n50}}{h})^{2.7} + 2)N_{od}^{0.15}} \tag{F.21}$$

Figure F-15 gives a graphical expression of this formula. On the horizontal axis the nominal diameter of the toe units is shown. On the vertical axis the depth of the breakwaters with respect to LWS is shown.

The figure shows several colours:

- Green These are the values that satisfy the iteration. This means that the input and output value don't differ much.
- White These values have different input and output values. These values don't satisfy the iteration
- Red 3 times the nominal diameter of the toe units exceeds the water depth in front of the toe, this results in the top of the toe above the water surface. The formula does not take this into account.
- Orange This boundary shows the restriction of the formula: $0.4 < \frac{h_t}{h} < 0.9$.
- Yellow This boundary shows the restriction of the formula: h_t/H_s< 2.

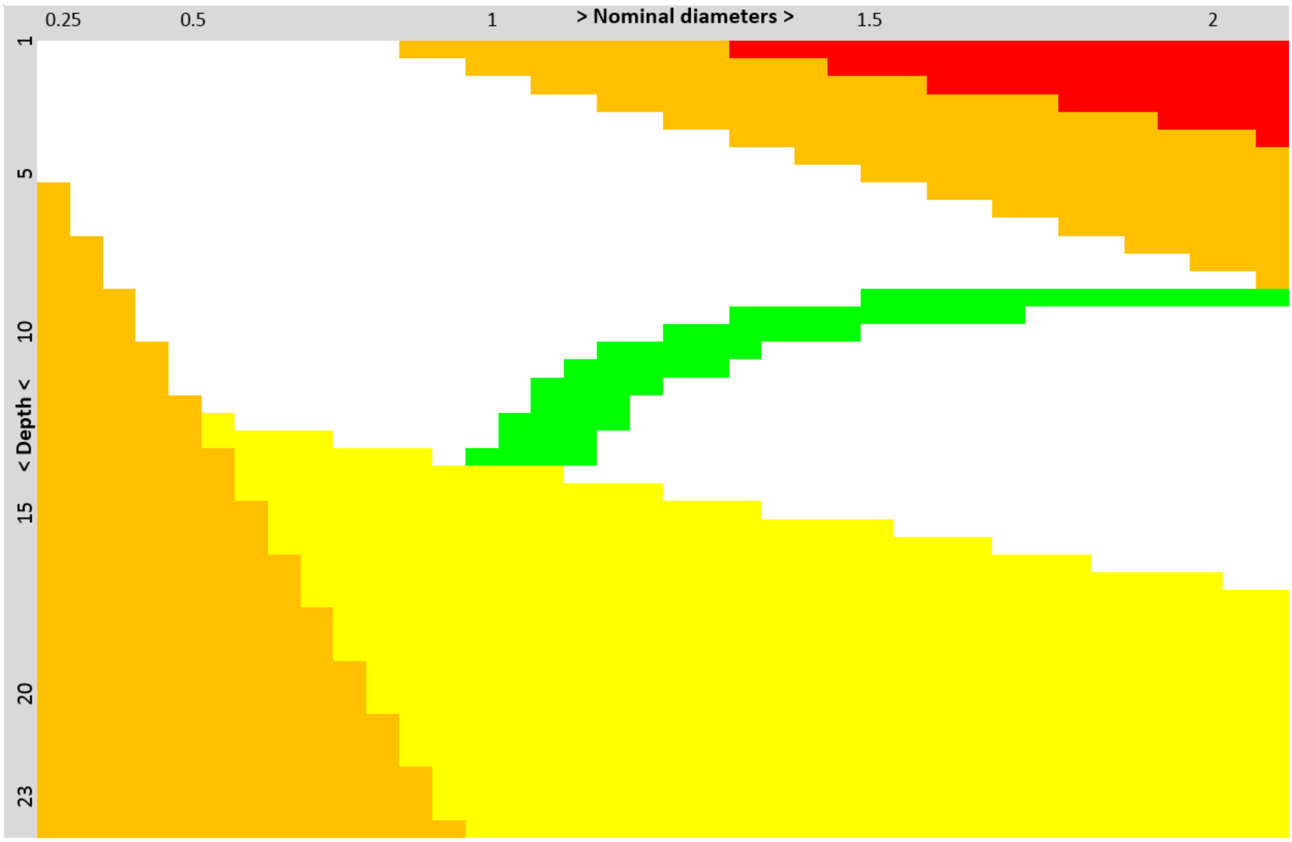


FIGURE F-15 GRAPHICAL EXPRESSION OF TOE FORMULA

From the iteration in the figure several conclusion can be made. These conclusions are shown in Table F-12.

TABLE F-12 CONCLUSIONS OF TOE DIMENSIONS

Section	**Nominal diameter [m]**	**Tetrapod weight [t]**	**Width of toe [m]**	**Remarks**
0 m - 8 m depth	2.00*	20.0	10.0	The toe formula which is used doesn't say anything about the very shallow part of the breakwaters. But this is the part where the waves directly attack the toe. Therefore the same nominal diameter as the armour layer is chosen to be the standard in this section.
8 m - 12 m depth	1.48**	8.0	7.5	The largest nominal diameter in this section is chosen to be the standard toe protection in this section.
12 m - end depth	0.95**	3.0	5.0	The nominal diameter of the toe units decreases as the depth increases. Therefore the nominal diameter of the units at 12 m is chosen to be standard in this section. It is recommended to investigate this further.

*From economical point of view is it chosen to reduce the toe height to 2 times the nominal diameter.

** The height of the toe at these sections is 3 times the nominal diameter.

The width of the toe is determined to be 3 - 5 times the nominal diameter, that is a rule of thumb as stated in (Verhagen & D'Angremond, 2012). A conservative design of 5 times the nominal diameter is used.

The inner slope also requires a toe berm. Inside the breakwaters the waves reduce significantly. This is why it is assumed that for these slopes a toe berm similar to the original design is acceptable. For the original design a toe berm with a length of 15 m of 3.5 tons is used. Closer to the entrance the waves become more significant and the tetrapod sizes as described above should be applied. Further research into the wave height inside of the breakwaters is recommended.

F.9 SUMMARY DRAFT DESIGN

The following table gives a summary of the draft design.

TABLE F-13 DRAFT DESIGN

Storm frequency [years]		1/250
Probability of failure during lifetime [%]		18.1
Length western breakwater [m]		272
Length eastern breakwater [m]		370
Height [+m LWS]		8.5
Armour layer outer [t]		20.0
Armour layer head [t]		25.0
Armour layer inner [t]		7.0
First under-layer [t]		1.0 - 2.0
Core [t]		0.5 - 1.0
Toe berm outer (0 - 8 m)	**Weight [t]**	20.0
	Length [m]	10.0
Toe berm outer (8 - 12 m)	**Weight [t]**	8.0
	Length [m]	7.5
Toe berm outer (12 m - end)	**Weight [t]**	3.0
	Length [m]	5.0

The draft design shows what the cross/sectional build-up should be for a reasonable probability of failure. However, the design also needs to be feasible since a lot has already been built. In the chapter 4.4 a feasible design will be determined. In case that the 1/250 year draft design is not possible to implement there are other options. A draft design based on a 1/150 year storm and a 1/100 year storm is also calculated in the same way as the 1/250 year draft design. The final results are given in Table F-14.

TABLE F-14 RESULTS DRAFT DESIGN

	Draft design 1	**Draft design 2**	**Draft design 3**
Storm frequency [years]	1/250	1/150	1/100
Probability of failure during lifetime [%]	18.1	28.3	39.3
Length western breakwater [m]	272	272	272
Length eastern breakwater [m]	370	370	370
Height [+m LWS]	8.5	8.25	8.0
Armour layer outer [t]	20.0	16.0	12.0
Armour layer head [t]	25.0	20.0	15.0
Armour layer inner [t]	7.0	6.0	5.0
First under-layer [t]	1.0 - 2.0	1.0 - 2.0	1.0 - 2.0
Core [t]	0.5 - 1.0	0.5 - 1.0	0.5 - 1.0
Weight [t]	20.0	16.0	12.0

Toe berm outer (0 - 8 m)	**Length [m]**	10.0	9.5	9.0
Toe berm outer (8 - 12 m)	**Weight [t]**	8.0	8.0	7.0
	Length [m]	7.5	7.5	7.0
Toe berm outer (12 m - end)	**Weight [t]**	3.0	2.0	2.0
	Length [m]	5.0	4.5	4.5
Toe berm inner	**Weight [t]**	3.5	3.5	3.5
	Length [m]	15	15	15

An overview of this table is given in Figure F-16.

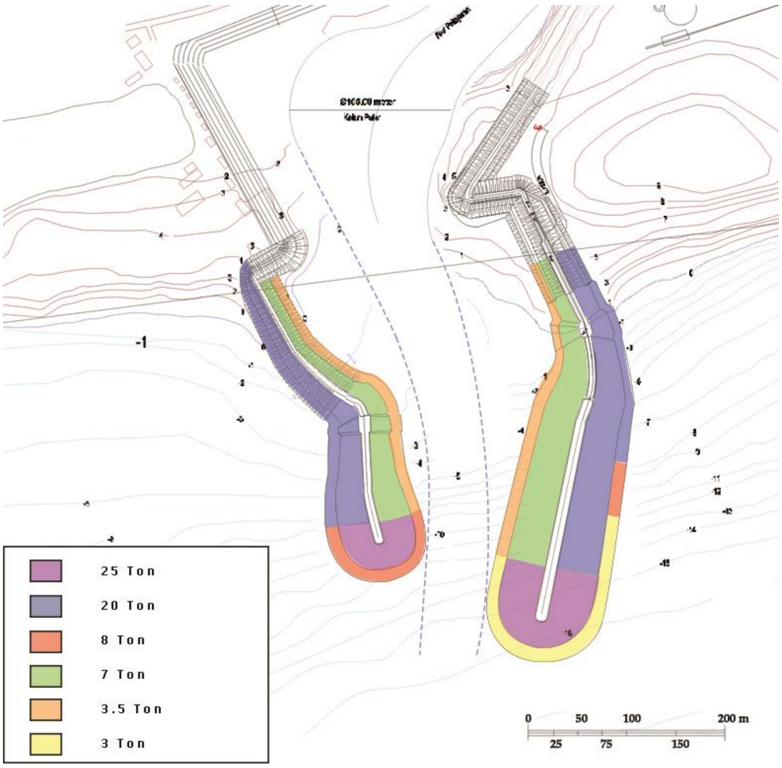


FIGURE F-16 DRAFT DESIGN 1/250 YEAR STORM

The draft design based on a 1/250 year storm is what will be strived for. In paragraph 5.2.3 it is shown what actually can be implemented.

APPENDIX G: REVIEW OF PREVIOUS DESIGNS

G.1 REVIEWS OF PREVIOUS DESIGNS

This section reviews the original design, which is the original design after improvements, and the review design made by Prof. Nur Yuwono. An evaluation of these designs will make clear whether the instability issues are due to a fault in the design process or not.

PROBABILTY OF FAILURE

The original design is based on a storm with a return period of 50 years. (Pustek Kelautan Universitas Gadjah Mada, 2003) This means that, with a lifetime of 50 years, the probability of failure in that period is more than 63%. This is a very high probability. The review design assumes a storm frequency of 1/100. This reduces the probability of failure to 39.3%. Still, this is also a high probability which is not recommended. However, as mentioned before a consideration should be made between investment and maintenance which may explain the chosen probability of failure for the designs. As economical standards are unknown it is hard to say what probability of failure gives an economic optimum. Project Yogya considers a 1/250 year storm frequency corresponding with an allowed probability of 18% a good assumption.

ORIENTATION OF BREAKWATERS

The original design as well as the review design determined a breakwater length of 250 m for the western breakwater and 300 m for the eastern breakwater. The length of the breakwaters in the draft design are 272 m for the western breakwater and 370 m for the eastern breakwater as explained in appendix F.1.2. This is significantly longer. Prof. Nur Yuwono [UGM] explained that the original design and the review design were originally designed to be longer. The length was reduced because the funds to build this length were not available at the time.

The navigation channel in both designs is determined to be 80 m whereas the draft design calculation gives 60 m. The reason for this is most likely because more factor for the width are included. For the draft design it is assumed 60 m is more than enough based on the estimated development of the harbour.

HEIGHT OF BREAKWATER

Both the original design and the review design determined a design height of the structure of +8.0 m LWS. When calculating the required height of the breakwaters in the same way as in appendix F.2 the original design would have had to been around +7.0 m LWS and the review design around +8.0 m LWS. This calculation corresponds well to the actual designs.

CONCRETE ARMOUR LAYER

The following table shows the weights of the tetrapods that were used in the original design and the review design as explained in chapter 2.3.

TABLE G-1 COMPARISON ORIGINAL AND REVIEW DESIGN OF ARMOUR LAYER

	Outer layer [t]	Head [t]	Inner layer [t]
Original design	9.0	11.5	7.0
Review design	11.5/14.0	18.0	9.0

In order to see whether these tetrapods seem reasonable the required dimensions of the concrete armour units are calculated using the Van Der Meer stability formula in the same way as in appendix F.3. The probability of failure and significant wave height corresponding to that specific design are used. The following table gives the results of each calculation.

TABLE G-2 COMPARISON ORIGINAL AND REVIEW DESIGN FOR DIFFERENT CALCULATION METHODS OF ARMOUR LAYER

	Outer layer [t]			Head [t]			Inner layer [t]
	Classical	Deterministic	Probabilistic	Classical	Deterministic	Probabilistic	
Original	6,93	7,43	6.02	8,66	9,29	7,53	2,60
Review	11,83	15,98	12.02	14,79	19,97	15.03	4,63

For the original design, the determined tetrapods are lower than what is used in the design. This means that the original design is over dimensioned. However this design is still not safe enough. The current state of the breakwaters show that the tetrapods are not heavy enough. A design based on a 1/50 year storm is not safe enough. The probability of failure is too high. The review design corresponds reasonably well with the calculations. The weight of the inner layer seems unnecessarily high.

FIRST UNDER-LAYER AND CORE

In appendix F.6 the design rules for the first under-layer and the core have already been explained. The weight ratio between the considered layer and the one above should be between 1/10 and 1/25. The following table gives a summary of the designs along with the weight ratios. All weight ratios stay well within the basic design rules. The weight could actually be lower. However, this may cause problems during construction due to the rough wave climate. It should be noted that in the review design the first under-layer consists of cube blocks instead of rocks. The effectiveness of cube blocks as first under-layer is uncertain. It is proven to be difficult to randomly place cube blocks as they often slide toward each other's sides. This reduces the effectiveness of the layer and the stability of the layer on top of it.

TABLE G-3 COMPARISON ORIGINAL AND REVIEW DESIGN OF UNDER-LAYER

	First under-layer		Core	
	Design	Weight ratio	Design	Weight ratio
Original design	1.0 - 1.5 ton rocks	1/7 to 1/4.67	0.5 - 1.0 ton rocks	1/3 to 1/1
Review design	1.8 ton Cube blocks	1/10 to 1/5	0.5 - 1.0 ton rocks	1/3.6 to 1/1.8

TOE BERM

The original design determined that a 15 m wide toe with 3.5 tons tetrapods is required. The review design uses 7.0, 5.5 and 3.5 tons tetrapods using the same width of the toe berm.

Table G-4 and Table G-5 give the toe berm design following the same design practice as in appendix F.8. The main difference between this calculation and the designs is the width of the toe. An explanation for this is the fact that the calculation below is based on the wave height above the toe and the calculation in the original design and the review design is based on the stability of the armour layer and toe. Further investigation on this is recommended as already stated appendix F.8.

TABLE G-4 ORIGINAL DESIGN TOE DIMENSIONS

	Original design		
Section	**Nominal diameter [m]**	**Weight of tetrapods [t]**	**Width of toe [m]**
0 m - 5.5 m	1.41	7.0	4.5 - 7.5
5.5 m - 9.0 m	1.29	6.0	4.0 - 6.5
9 m - end	0.73	1.0	2.5 - 4.0

TABLE G-5 REVIEW DESIGN TOE DIMENSIONS

	Review design		
Section	**Nominal diameter [m]**	**Weight of tetrapods [t]**	**Width of toe [m]**
0 m - 7 m	1.69	12.0	5.0 - 8.5
7 m - 11 m	1.39	7.0	4.5 - 7.0
11 m - end	0.85	2.0	2.5 - 4.5

INITIAL DESIGN SUMMARY

What can be concluded from this is that both the original plan and the proposal made by Prof. Nur Yuwono are well designed. The main difference lies in the chosen accepted probability of failure and the significant wave height belonging to this probability. This is also what causes the large difference between these designs and the draft design.

APPENDIX H: DRAFT DESIGN IMPLEMENTATION

H.1 IMPLEMENTATION OF YOGYA DRAFT DESIGN

In this section the building method of the breakwaters will be explained. This building method will consist of two main parts: renovation and extension. First the current state of the breakwaters needs to be improved. If nothing is done the breakwaters will deteriorate more and more until they stop functioning properly or even completely collapse. After the improvements the breakwaters can be extended. For this, several building methods are possible. After finding the most promising solutions for the renovation and extension the final design and building plan is given along with the necessary technical drawings.

RENOVATION OF THE BREAKWATERS

As explained in chapter 0 the following table shows what has been constructed so far.

TABLE H-1 CURRENT CONSTRUCTION CHARACTERISTICS

	Construction
Length western breakwater [m]	215
Length eastern breakwater [m]	180
Height	+6 m LSW (+2 m seawall for eastern breakwater)
Armour layer outer [t]	9.0
Armour layer head [t]	11.5
Armour layer inner [t]	7.0
First under-layer [t]	Unidentifiable
Core [t]	0.1 - 0.5 ton rocks
Crest	2 m, K 250 concrete reinforced with D16-150
Toe berm	15 m, 3.5 tons tetrapods
Filter layer	0.1 - 0.5 ton rocks

The armour layer of the inner slope already meets up to the 1/250 year design. This layer does not need to be adjusted.

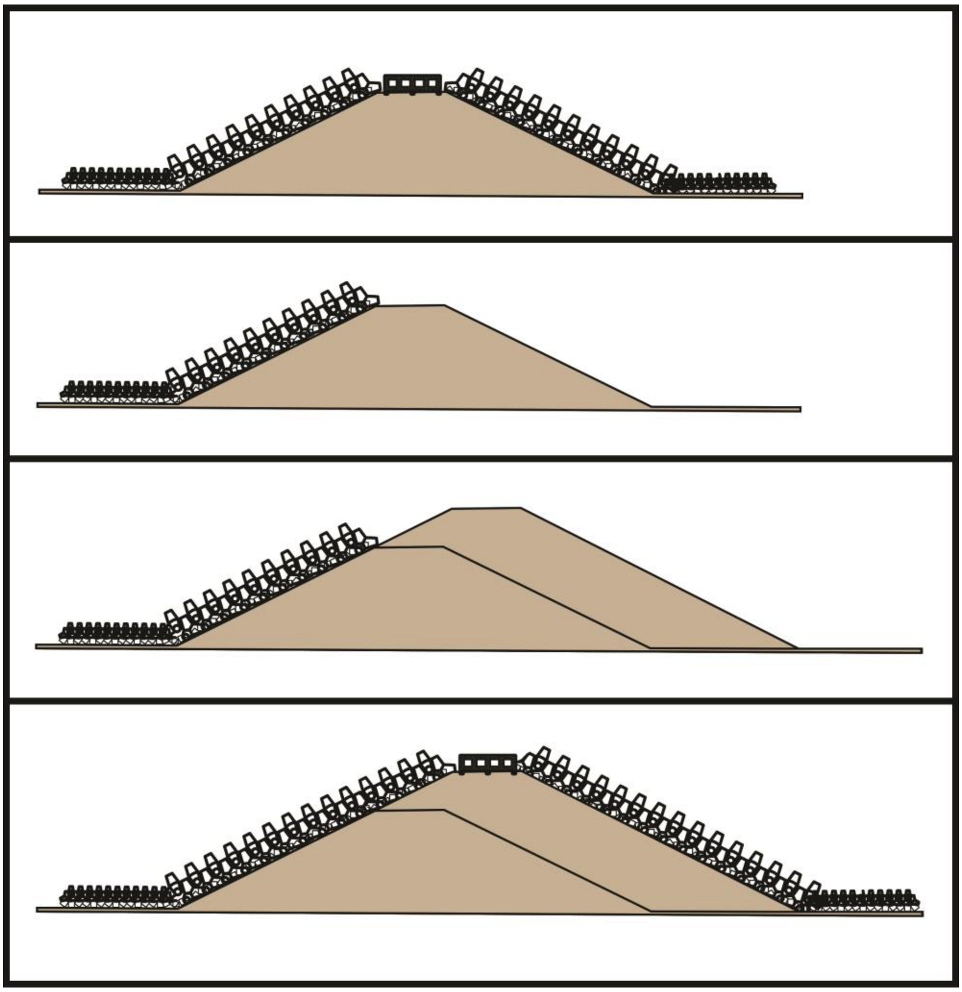
The core layer should be 0.5 - 1.0 ton. Replacing the existing core is not an option because that would mean that the entire breakwaters would have to be removed. This is far too expensive. So the existing core will remain to be 0.1 - 0.5 ton.

There are several options for renovation each with different pros and cons. Each type of renovation will now be described.

FULL RENOVATION

A full renovation means that the current construction is disassembled so that the crest height can be increased from the bottom up. First the tetrapods at the outer layer, the head and at the toe are removed. Also the concrete cap is removed. This means that only the core layer, the first-under layer and the armour layer on the inner slope is left. New core material will now be placed on op top of the remaining structure followed by the first under-layer until the structure is increased to +8.5 m LWS. After this is done the armour layer at the toe and outer slope is placed. The armour layer at the head is only placed if construction of the extension is not planned shortly after completion of the renovation. The four main steps of construction are given in Figure H-1.

The pro of this type of renovation is that after completion the breakwaters are as close to the design as possible. This creates the most certainty of a safe and durable breakwater. The obvious downside of this type of renovation is that it is very costly. Whether the funds are available is the determining factor. It will also take a lot of time to finish the renovation.



Original construction

Removing armour layer and concrete cap

Placing additional core material and first under-layer

Placing new armour layer and concrete cap

FIGURE H-1 CROSS SECTIONS OF DIFFERENT RENOVATION METHODS

ADDITIONAL PLACEMENT RENOVATION

This type of renovation builds on top of the current state of the breakwaters. No tetrapods are removed. One layer of heavier tetrapods, which meet the draft design as close as possible, are placed on top of the existing outer layer and at the toe. Since most tetrapods in the current situation have shifted this method can quickly fill up any gaps as well as create a decent double layer of tetrapod armour. The current layer of tetrapods does not meet the weight requirements for a safe design. As long as the layer on top is heavy enough this will generally give minimal problems. The added risk is that when a storm damages the top layer, the original armour layer will become exposed. This layer is then very likely to also get damaged since it’s less heavy than the layer above.

The fact that the original armour layer is not removed also means that the height of the breakwaters cannot be increased by increasing the thickness of the core. Instead, a seawall is placed to get to the required height. On the eastern breakwater this seawall is already placed up to a height of +8.0 m LWS. For practical reasons this seawall won’t be heightened till 8.5 m as this has no high priority. The extension of the eastern breakwater will be built till a height of 8.5 m. On the western breakwater a seawall will be constructed up to

+8.5 m LWS in order to meet up to the 1/250 year design. Since the breakwater is currently at +6.0 m LWS this means a seawall of 2.5 m high will be placed.

Figure H-2 shows a sketch of the current situation and of the layout after renovation. This type of renovation is easy to implement which makes it appealing. No changes to the existing structure are made so the improvement can start immediately. A downside of this type of renovation is that the stability of the armour layer is more uncertain than for a full renovation.

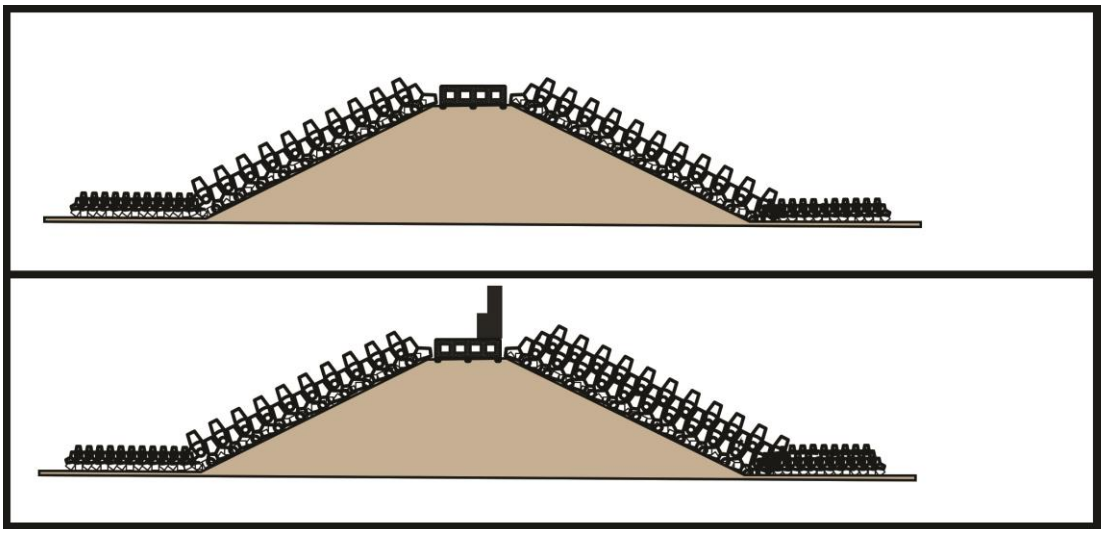


FIGURE H-2 BEFORE AND AFTER RENOVATION, ADDITIONAL PLACEMENT RENOVATION

STABILITY RENOVATION

This renovation is similar to the additional placement renovation in that it also uses the seawall to get to the required height. The difference is that this renovation does not add new tetrapods at the outer layer of the breakwaters. Instead the stability of the outer slope is improved by connecting the existing tetrapods using steel cables. This prevents the tetrapods from shifting. The toe will be improved to make sure that the armour layer does not slide down any further.

The pro of this type of renovation is that only a few new tetrapods are required so the renovation can be done in a short amount of time. The downside is that the current tetrapods at this position are not heavy enough. The steel cables will improve the stability but whether this improves the strength enough is uncertain. Furthermore, the current tetrapod at the breakwaters already shows large signs of corrosion so this might not be a durable solution.

TETRAPOD RENOVATION

This renovation is similar to the additional placement renovation in that it does not remove tetrapods but places a layer of heavier tetrapods, which meet the draft design as close as possible, on top of the existing outer layer and at the toe. However, instead of a seawall this type of renovation also places a double layer of tetrapods over the crest of the western breakwater and one more layer of armour at the inner slope for stability. This way the required height is met. Figure H-3 shows a sketch of the current situation and of the layout after renovation.

This method uses a lot of tetrapods which makes it very expensive. Also, the tetrapods at the crest are a permeable layer so transmission of waves through this part of the breakwater might pose problem for the inner slope. Also the breakwater is not accessible anymore due to the tetrapods on the crest.

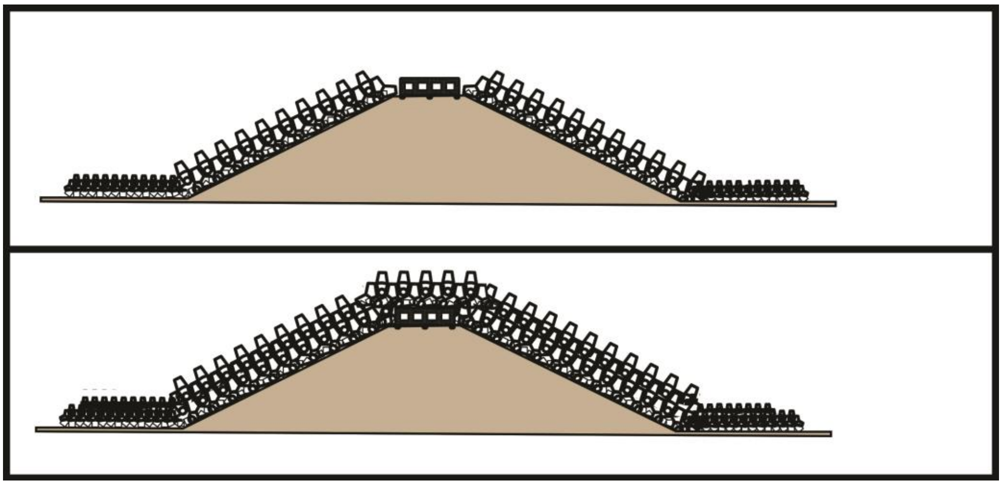


FIGURE H-3 BEFORE AND AFTER RENOVATION, TETRAPOD RENOVATION METHOD

The following table summarizes the advantages and disadvantages of the different types of renovation.

TABLE H-2 EVALUATION OF RENOVATION METHODS

	Advantage	Disadvantage
Full renovation	▪ Breakwater is revised perfectly	▪ Building Time ▪ Relatively expensive
Additional placement renovation	▪ Improvement of armour layer ▪ Improvement can start immediately ▪ Simple construction method	▪ The stability of the armour is less certain
Stability renovation	▪ Relatively Cheap ▪ Improvement can start immediately	▪ Effect of corrosion is unknown ▪ Current tetrapod's are too light
Tetrapod renovation	▪ Simple construction method	▪ Relative expensive ▪ Breakwater not accessible

TABLE H-3 COMPARISON RENOVATION METHODS

	Full renovation	Additional placement	Stability renovation	Tetrapod renovation	Total:
Full renovation	■	0	0	0	0
Additional placement renovation	1	■	1	1	3
Stability renovation	1	0	■	0	1
Tetrapod renovation	1	0	1	■	2

In Table H-2 and Table H-3 it can be seen that the full renovation method is not a good idea as this solution will be very expensive and it will take lots of time. The stability renovation method seems to be a good idea but the uncertainties about this method are too large. Both the additional placement renovation method and the tetrapod renovation method will add tetrapods to the current construction. The best method is the additional placement renovation method as compared to the tetrapod renovation method this method is cheaper because of the use of the seawall. This seawall ensures the crest of the breakwater to be tetrapod free. Also less tetrapods are needed, therefore the additional placement renovation method seems to be a good solution for the renovation of the breakwater.

EXTENSION OF THE BREAKWATERS

There are different manner to construct both of the breakwaters. In this paragraph these building methods are described and evaluated. The building methods that will be evaluated are shown in Figure H-4.

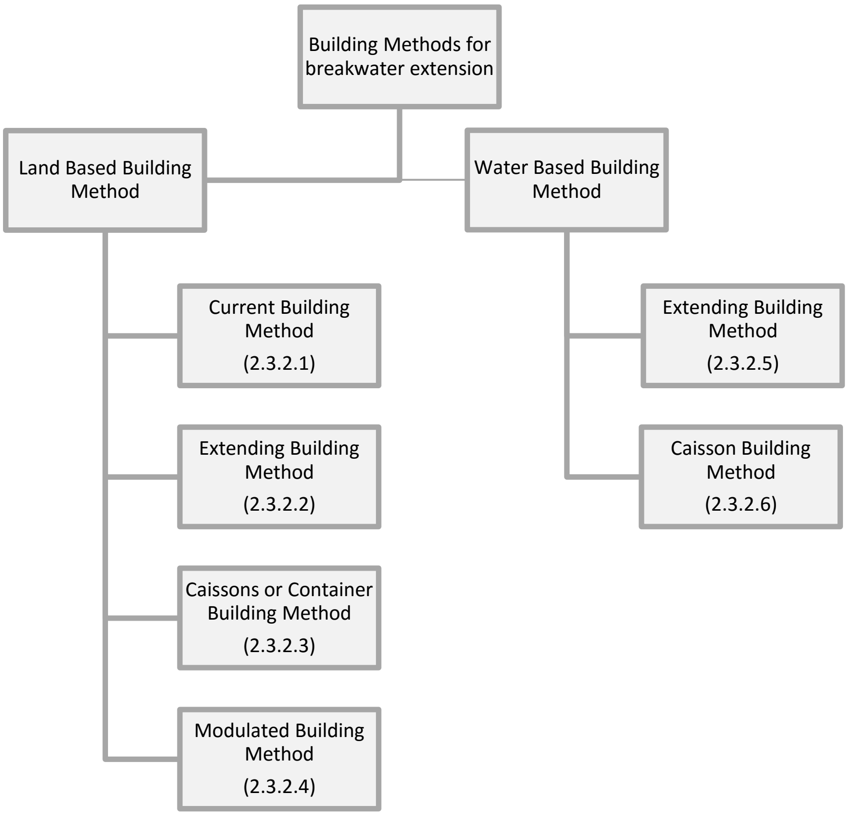


FIGURE H-4 OVERVIEW CONSTRUCTION METHODS

A distinction is made between land based and water based building methods. The construction of the land based building method (LB) will take place from the current breakwaters and will continue from the finished segments. The water based building method (WB) will mainly focus on the construction of the breakwaters from water based vessels.

CURRENT BUILDING METHOD (LB)

This building method is currently used to construct the breakwaters. As can be seen in Figure H-5 the breakwaters are build layer per layer over the full length of the breakwater section. First the core is placed, after that the first under layer is placed on top. The construction of the section is finished by placing the tetrapods as the armour layer of the breakwater.

FIGURE H-5 UNSAFE SITUATIONS DURING CONSTRUCTION

This building method seems to be very practical and cheap. When evaluating this building method several problems occurred. First, the core and under layer cannot withstand the force of the wave attacks so loss of core takes place, this can be seen in Figure H-6. Second, the safety of equipment and workers is a point of improvement. Finally, when finishing a section of the breakwater (all layers are in place) no temporary head is constructed while the construction of the breakwaters can be on hold for months or years. This results in collapsing of the final end of the breakwater. This can be seen in Figure H-6. These pictures are made available by Prof. Nur Yuwono and BBWS Serayu Opak.

FIGURE H-6 LOSS OF CORE AND DAMAGE TO THE HEAD OF THE BREAKWATERS (PROF. NUR YUWONO, 2015)

EXTENDING BUILDING METHOD (LB)

The Extending Building Method is also a land based building method. This means the breakwaters are constantly extended from the already existing structure. From the existing structure a maximum of 10 m core is exposed. After the core immediately the under layer and armour layers are placed. This method differs from the used building method by the length of exposed core and under layer to the waves. But the logistic planning at the head of the breakwater will be somewhat harder because more vehicles are closer to each other at the head. This is schematized in Figure H-7. The head does earlier reach its design height and so it is also safer for vehicles and workers.

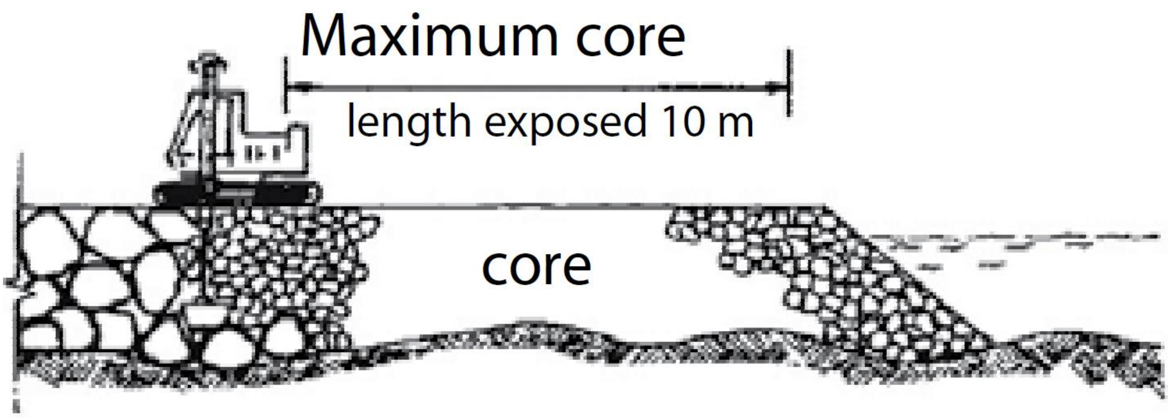


FIGURE H-7 EXTENDING BUILDING METHOD (FOOD AND AGRICULTURE ORGANIZATION OF THE UNITED NATIONS, 2008)

CAISSON OR CONTAINER BUILDING METHOD (LB)

This method consists of replacing the core by caissons or recycled filled containers. This will resolve the problem of core loss due to the wave attacks. However the construction of caissons is a technical challenge. The next challenge is the placement of the caissons on the steep bottom slope. This method is shown in Figure H-8. The caissons or containers are just a replacement of the core this means that the outer layers have to be placed over the units in order to break the waves and absorb the wave energy. It might be the case that the under layer and the armour layer at the inner slope don’t have to be placed because of the limited wave action inside at the inner slope but to confirm this further investigation is recommended.

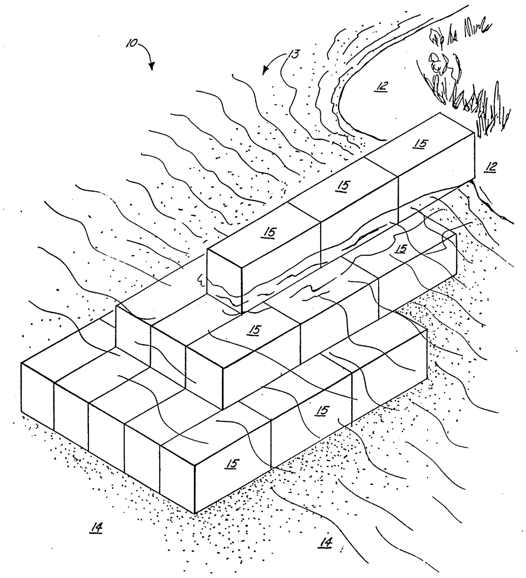


FIGURE H-8 CAISSON OR CONTAINER BUILDING METHOD (USA PATENTNR. US4954012A, 1987)

MODULATED BUILDING METHOD (LB)

The Modulated Building Method looks very similar to the Extending Building Method. Just like the Extending Building Method the layers will be built very close to each other. This results in less exposure of the layers beneath the armour layer to the incoming waves. The difference between the Modulated Building Method and the Extending Building Method is that the size of the breakwaters is divided in several modules. These modules can be built in the relative quiet season. Per breakwater it differs what the relative calm season is due to the wave direction. At the end of a module a temporary head will be constructed. This means that the temporary end of the breakwaters will be protected with an armour layer. When continuing the construction the upper tetrapod's of the temporary head will be removed. The lower tetrapods will stay in place and these tetrapods will be covered by the core and following layers of the next module.

EXTENDING BUILDING METHOD (WB)

This method is almost the same as the land based extending building method. The only difference is that the construction will take place from floating vessels. This can be pontoons on the inner side of the breakwaters and seaborne vessels at the outer side of the breakwaters. As results of this no space for vehicles is needed at the crest of the breakwaters which reduces the amount of material. A disadvantage of this method is the enormous waves at the outer slope of the breakwaters which will lead to an increase of downtime during the construction.

CAISSON BUILDING METHOD (WB)

The Caisson Building Method is based on a large caisson segment that will replaced the core and concrete pavement on top in one. The caisson will be built in a dock and will cover the total height of the breakwaters per piece. The total length of the breakwaters will be divided in several large segments. For example four segments will cover the total length. These segments will be placed at the right position and filled with water and concrete so they will sink to the bottom. It will be a challenge to construct and transport these caissons. Also their placement will be very hard as the bottom profile is very steep.

EVALUATION OF BUILDING METHODS

The evaluation of the building methods is shown in Table H-4.

TABLE H-4 EVALUATION BUILDING METHODS

Land Based Building method	**Advantage**	**Disadvantage**
Current Building Method	▪ Relative cheap ▪ Easy logistics	▪ Loss of core ▪ Safety ▪ No head protection during construction
Extending Building Method	▪ Relative cheap ▪ Limited loss of core ▪ Safety	▪ Hard logistics ▪ No head protection during construction
Caisson or Container Building Method	▪ No loss of core ▪ Head protection during construction	▪ Relative expensive ▪ Hard to construct ▪ Hard to place
Modulated Building Method	▪ No loss of core ▪ Head protection after completing module ▪ Safety	▪ Hard logistics ▪ Loss of tetrapod's
Water Based Building Method	▪	▪
Extending Building Method	▪ Relative cheap ▪ Limited loss of core	▪ Wave height ▪ Hard logistics ▪ No head protection during construction
Caisson Building Method	▪ No loss of core ▪ Head protection during construction	▪ Wave height ▪ Relative expensive ▪ Slope of bottom ▪ Construction challenge

Because of the wave height at the outer slope of the breakwaters a water based building method is assumed to be impossible. The high waves will lead to a large amount of downtime. In Table H-5 a comparison between the different building methods is executed.

TABLE H-5 COMPARISON BUILDING METHODS

	Current Building Method	Extending Building Method	Caisson or Container Building Method	Modulated Building Method	Total:
Current Building Method	■	0	0	0	0
Extending Building Method	1	■	1	0	2
Caisson or Container Building Method	1	0	■	0	1
Modulated Building Method	1	1	1	■	3

The current building method is the worst building method as it is not safe, the core is smashed away quit often and the head collapses due to the lack of a temporary head protection. After that the caisson or container building method is chosen. The technical challenge as well as the costs will make this idea impossible. The extending building method and modulated building method are the best ways of constructing the breakwaters. The best building method of these two options is the modulated building method because the protection of the head when the construction is on hold will prevent the breakwaters from unnecessary damage. It is assumed that the cost of the few tetrapod's that will be covered underneath the core of the next module won't exceed the cost of repairing a collapsed head and the associated loss of core.

H.2 CONCLUSION IMPLEMENTATION

From both the renovation and the extension method analysis it can be concluded that for both parts a good solution can be found. For the renovation of the current breakwaters the additional placement renovation method is the recommended renovation method. This method places a seawall on top of the crest in order to reach the desired height. Also the armour layer will be strengthened by placing heavier tetrapod's on top the current armour layer. The modulated building method is a good method to construct the extension of the breakwaters. This method extends the breakwaters by constructing all layers close to each other. This results in a small part of core and under layer which will be exposed to the waves. After each building segment is finished a temporary head will be constructed and so prevent the head of collapsing.

APPENDIX I: FINAL DESIGN DRAWINGS

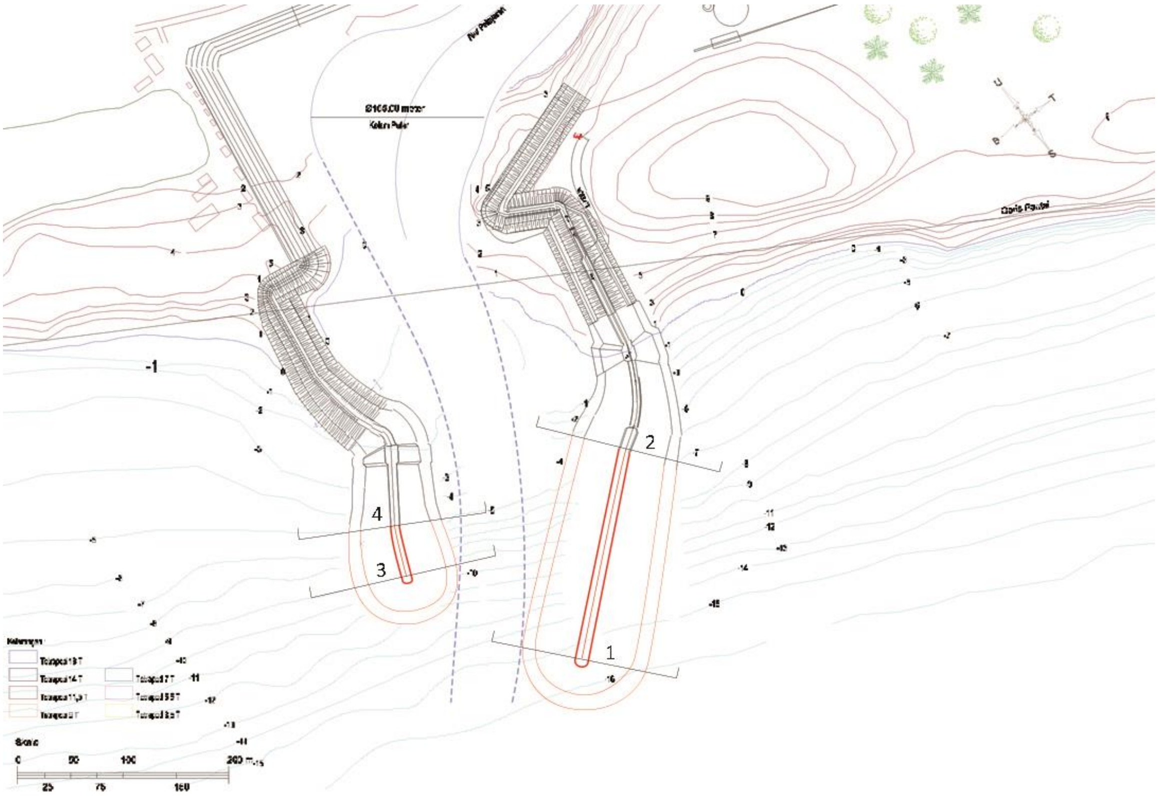


FIGURE I-1 FINAL BREAKWATER DESIGN INCLUDING POSITION OF CROSS-SECTIONS

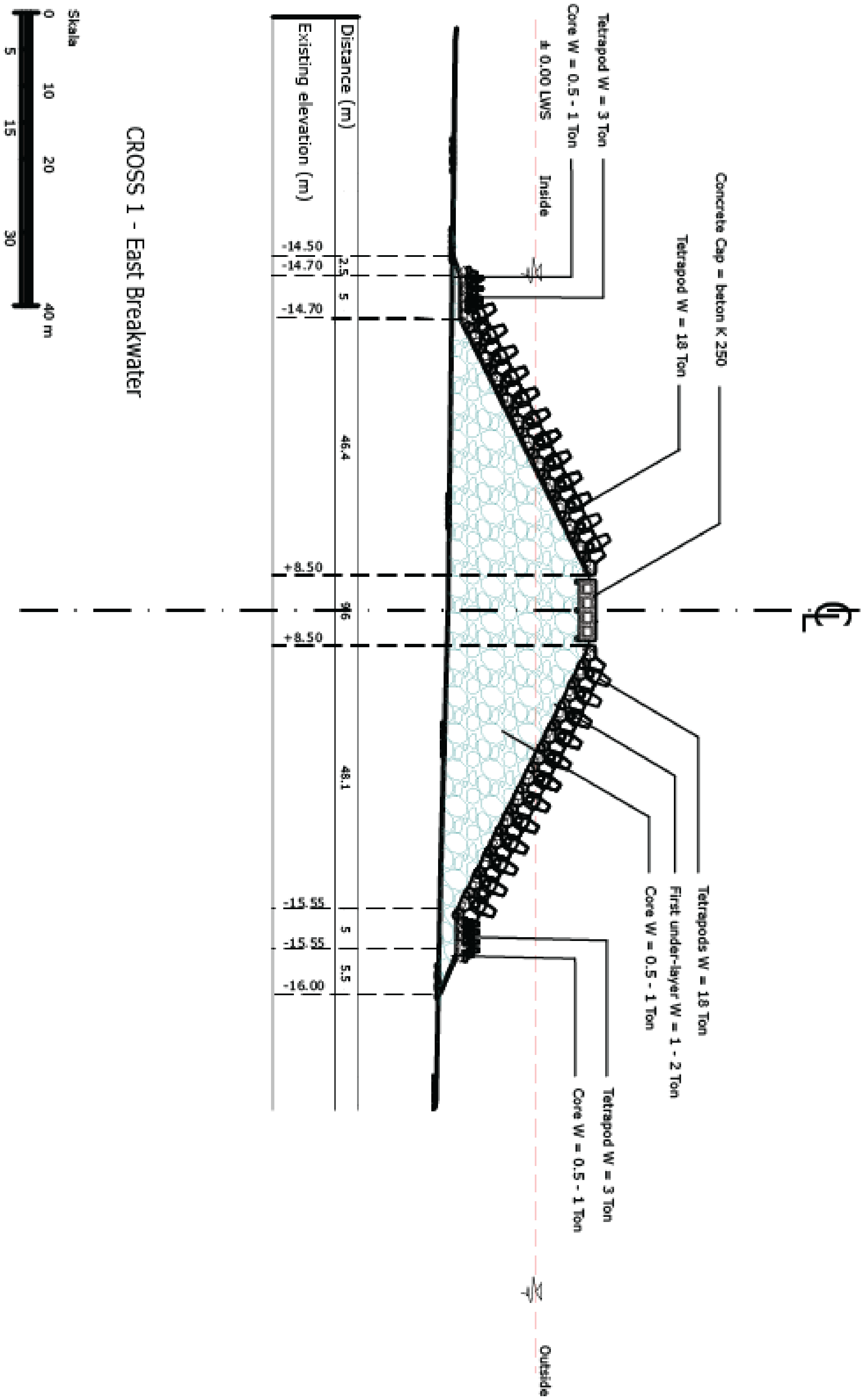


FIGURE I-2 CROSS-SECTION 1

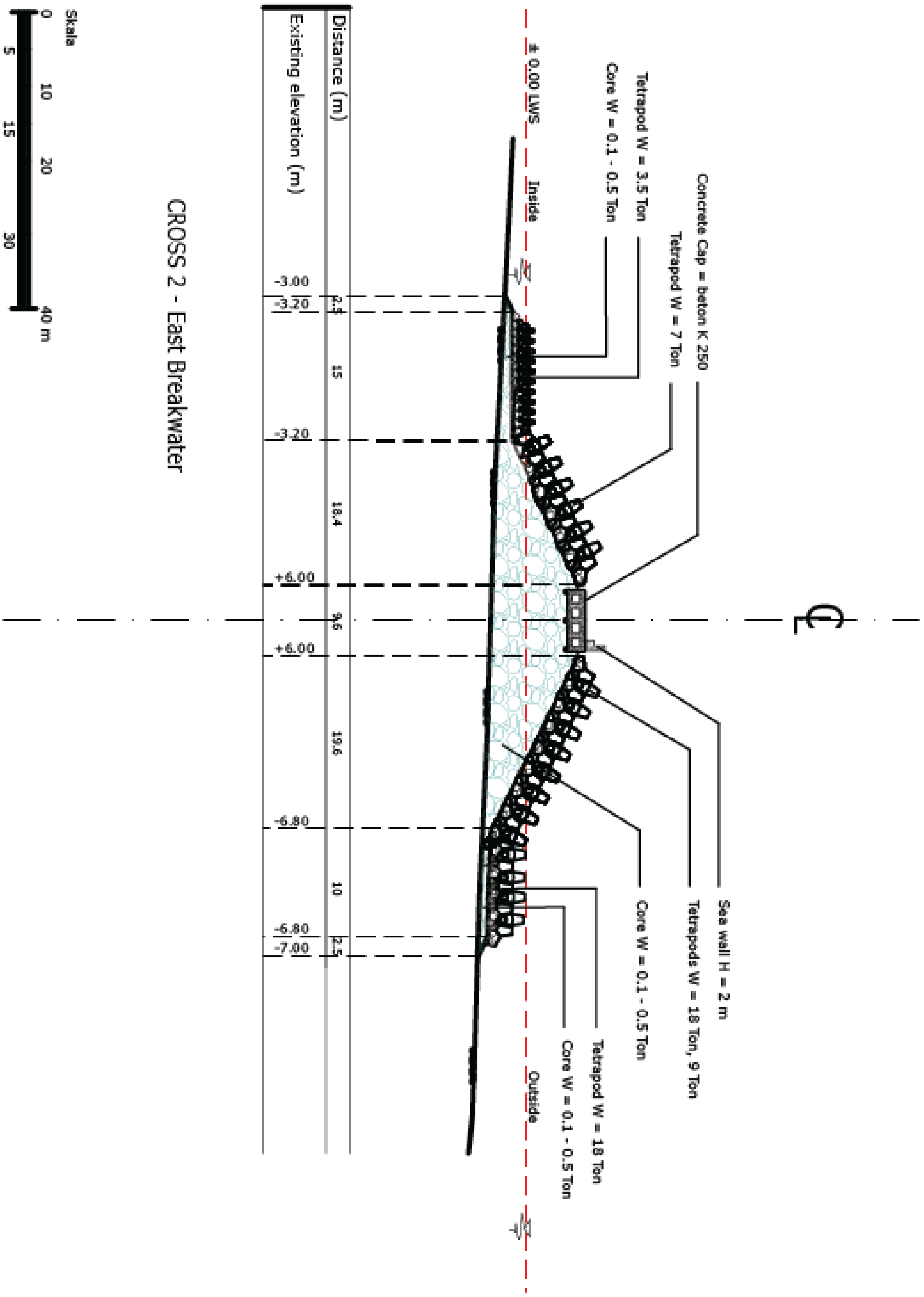


FIGURE I-3 CROSS-SECTION 2

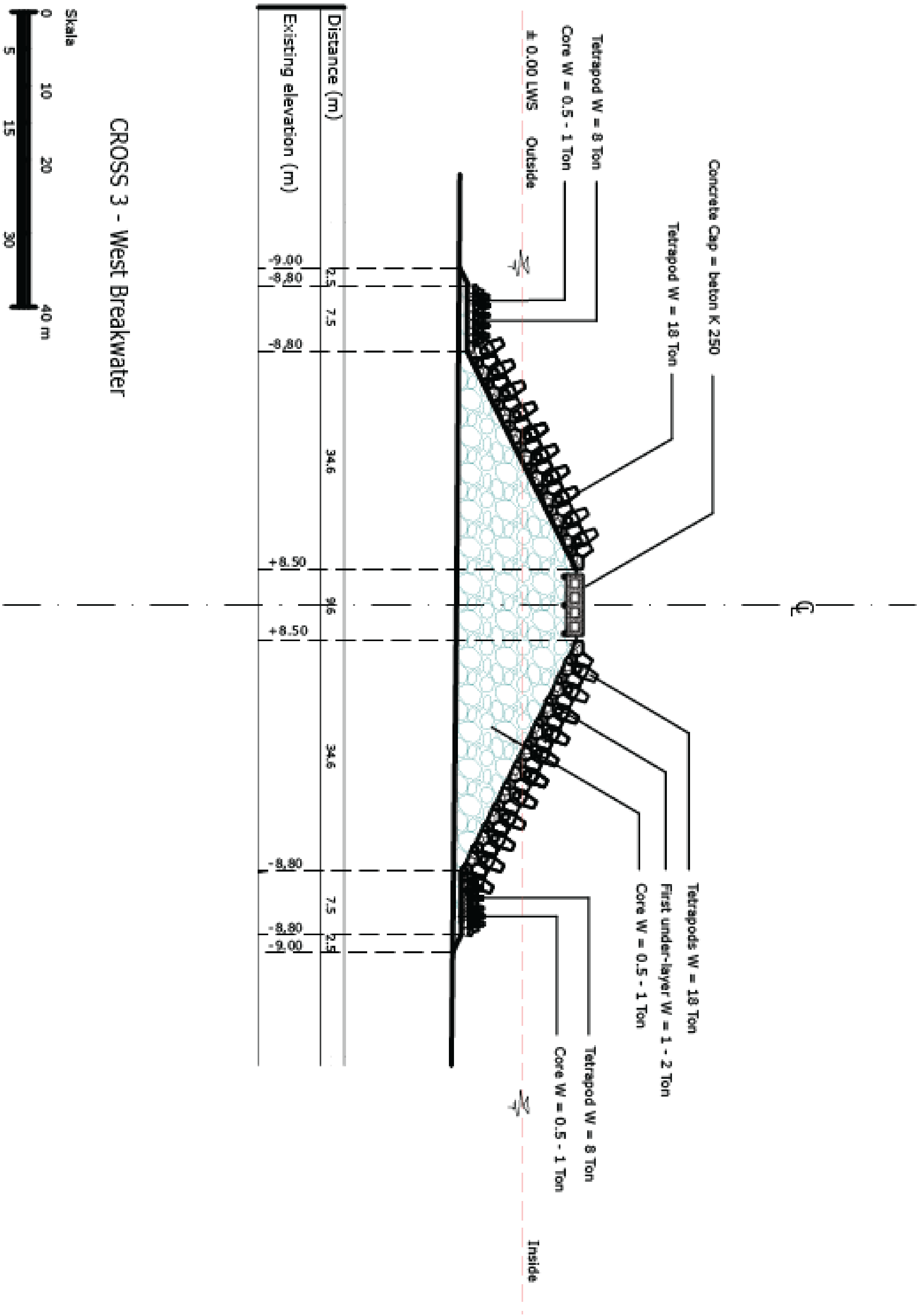


FIGURE I-4 CROSS-SECTION 3

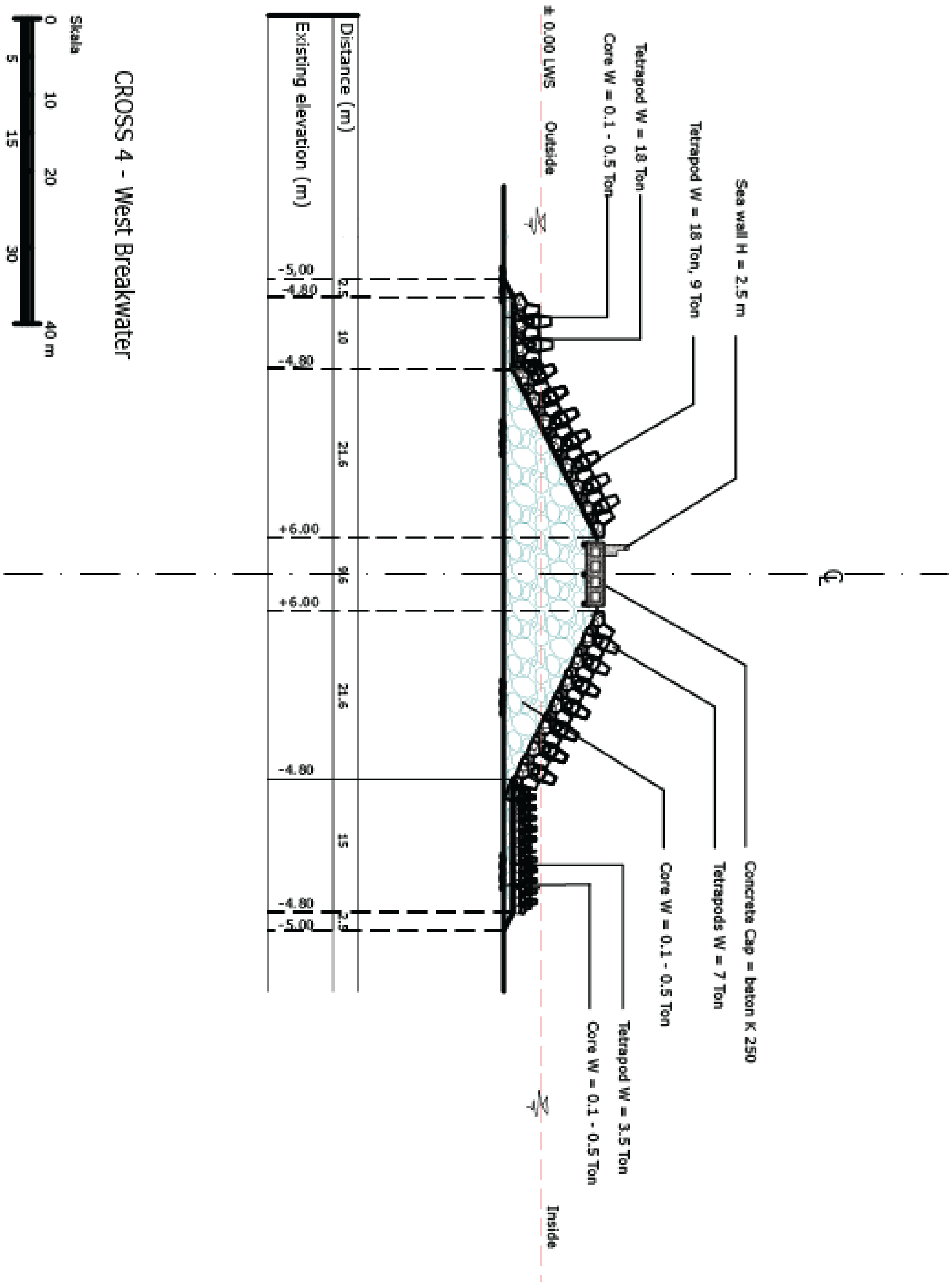


FIGURE I-5 CROSS-SECTION 4

APPENDIX J: COASTAL PROTECTION METHODS

In the following section these solution strategies are discussed, evaluated and eventually incorporated in design options.

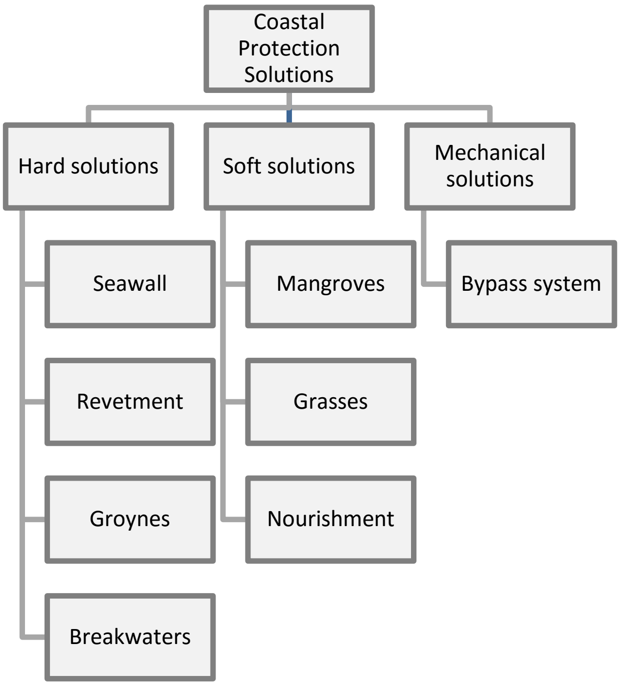


FIGURE J-1 OVERVIEW OF COASTAL EROSION SOLUTIONS

J.1 HARD SOLUTIONS

The hard solutions of Figure J-1 are discussed in the following paragraphs.

SEAWALL

The first solution is to build a seawall on the beach. This wall has to block the wave attack at the sandy beach. In this way the sand will be trapped behind the seawall, preventing erosion of the beach. Stability problems will occur immediately after placing the construction on the beach because erosion will cause a scouring hole at the toe of the structure. This scouring hole can affect the stability which is schematised in Figure J-2. This could be prevented by placing a well design toe protection. Although this solution is able to be executed, another problem will occur. On the long term, after the erosion processes eroded the sand up to the edge of the structure, the sediment for will be taken elsewhere. In the end this means that the erosion problem will occur at the western end of the wall and thus is simply being shifted instead of solved.

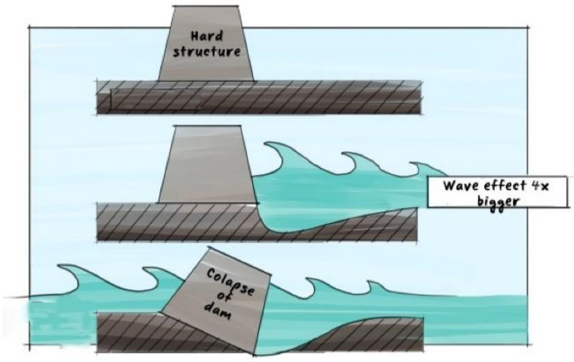


FIGURE J-2 SCOURING PROCESS AT THE TOE OF A SEAWALL (ECOSHAPE, 2015)

REVETMENT

A revetment will cover the entire intertidal area. In this way no sand will be stirred up and transported elsewhere. Although this solution is effective, it is very hard to construct this revetment as it covers almost the total beach. A scouring hole is able to appear on the edges of the protection causing outwash underneath the protection. If this outwash will occur, the protection will be subjected to cracks which leads to even more outwash and cracks, resulting in failure of the whole construction. Like the seawall, this can be prevented by a well-designed toe protection. Revetment is also a hard solution and due to the same reason as is mentioned by the seawall the erosion problem will just be shifted instead of being solved.

GROYNES

Groynes are structures constructed perpendicular to the coastline in order to stop the longshore sediment transport. This halt is a natural process as the trapped sediment will orient the coastline between the groynes in such a way that no longshore sediment transport occurs. This effect is shown in Figure J-3. The appliance of groynes results in local stabilization of the coast. Not only the longshore sediment transport is disturbed, also the currents along the coast are affected by the groynes. Especially around the head these perturbations can result in large scouring holes. Therefore in the design a proper toe protection is required. Groynes are also considered as a hard solution for the same reason is mentioned by the seawall and a revetment the erosion problem will just be shifted instead of being solved. However, a great advantage is the possibility to distribute the erosion over a larger area by a properly designed groyne field. Nevertheless, one should understand that groynes will not provide a strong beach protection during extreme storm events.

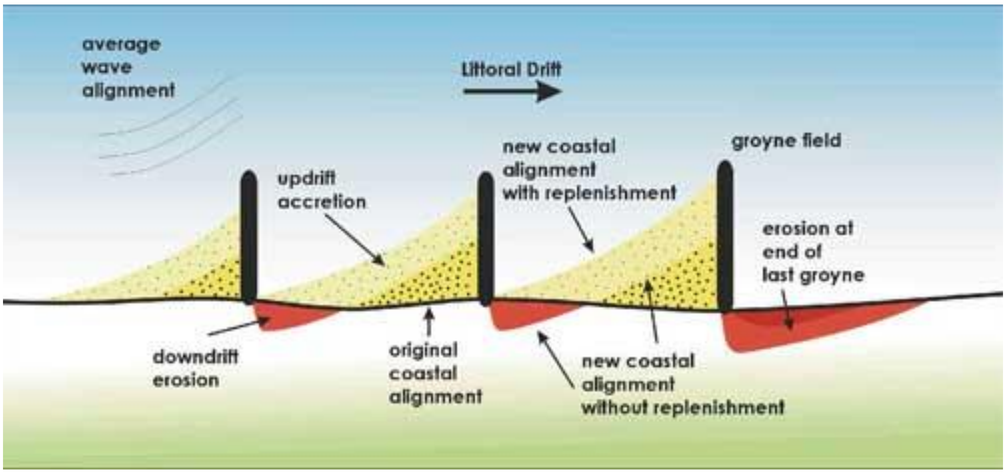


FIGURE J-3 GROYNE FIELD PRINCIPLES (DALLASAMPLITUDES, 2015)

BREAKWATERS

Breakwaters are structures constructed parallel to the coast. Breakwaters can be separated into two types: emerged and submerged breakwaters. The emerged breakwaters are large constructions that are able to break the waves completely. Although it is proven that this types of breakwaters function very effective, the have an enormous atheistic value as they completely block the horizon view from the beach. Submerged breakwaters are in an atheistic way the perfect solution and are smaller in size. However, using this type of breakwater it is not proven that they always meet their design criteria and sometimes even show perverse effects on erosion. Besides that, they can form a large danger for vessels as their location cannot directly be seen most of the time. Breakwaters are also considered as a hard solution and due to the same reason as is mentioned in the description of the seawall, the revetment and the groynes the erosion problem will just be shifted instead of being solved.

J.2 SOFT SOLUTIONS

The soft solutions of Figure J-1 are discussed in the following paragraphs.

MANGROVES

The first soft solution to discuss is the construction of a mangrove forest. A mangrove forest will trap the sediment between its roots and dampen the wave attack. This is a durable and sustainable solution for the erosion problem as a mangrove forest is the natural beach protection in these regions. However, in the first years after placing the young mangrove trees they will be vulnerable to wave attack because the mangrove trees need to be placed in the intertidal zone to vegetate optimal. Implementation of this solution to the project location, a mangrove forest will prevent transport of the sediment. Although it is a durable and sustainable solution, it still will shift the erosion problem instead of solving it.

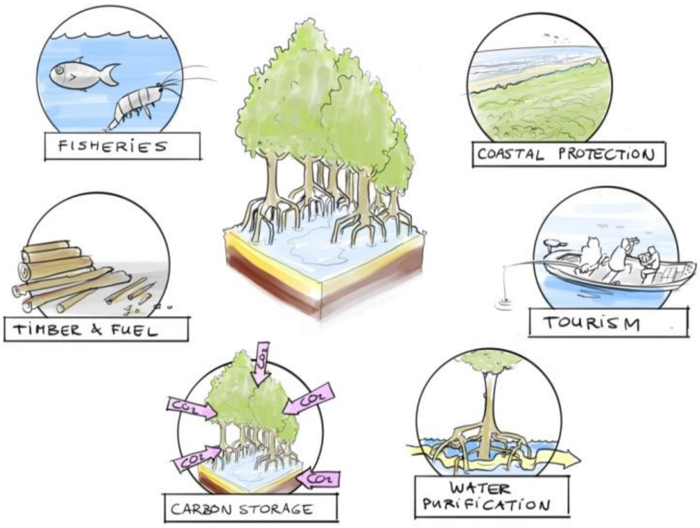


FIGURE J-4 ADVANTAGES OF MANGROVE FORREST (ECOSHAPE, 2015)

GRASSES

The second soft solution is the appliance of grasses that will trap the sediment between their long roots. Considering the Dutch coastal defence strategies, it can be concluded that this type of beach protection may be very efficient. This solution is fast to initiate and besides poses relatively little costs. The grasses will be placed at the top of the intertidal area. Compared to the mangrove forest solution the grasses will be fully grown sooner resulting in a quick solution for a natural coastal protection. Nevertheless, like the mangrove forest it is a durable and sustainable solution but it still will shift the erosion problem instead of solving it.

NOURISHMENT

The appliance of nourishments is a solution for the lack of sediment that is blocked by the breakwaters. Using nourishments, the net longshore sediment transport will be balanced out. This will end the erosion problem. The required sediment can be dredged from the navigation channel, harbour basin and eastside of the eastern breakwater where accretion occurs. Nevertheless, due to the harsh wave conditions the lifespan of a single nourishment is hard to predict. It may be assumed that this solution could solve the erosion problem during normal circumstances, though research is required for precise effect and location of nourishments. Besides that one should also consider the fact that nourishments do not provide a strong beach protection during extreme storm events.

J.3 MECHANICAL SOLUTION

The mechanical solutions of Figure J-1 are discussed in the following paragraphs.

BYPASS SYSTEM

A bypass system is a state-of-the-art mechanical solution that makes use of complex technologies. This system will continuously dredge sediment from one side of the breakwaters by using high pressure jets and deposit it on the other side where the erosion occurs. These jets will bring sediment from the bottom in suspension and then a pump system is transporting the mix of water and sediment. The bypass system will transports the yearly net transport from one side to the other restoring the longshore sediment transport balance. The pipelines that transport the mix of water and sediment across will be placed around or underneath the harbour basin and navigation channel to prevent perturbation of navigability. This system has two main disadvantages. Firstly, due to many sensitive mechanical parts of the system, it is highly dependent on proper and frequent maintenance. Secondly, the pressure jets and also the deposit of sediment by the pipelines will result in a lot of turbidity that will instantly kill all the underwater life in the area. One should also consider the fact that like nourishment, a bypass system solves the erosion problem but does not provide a strong beach protection during extreme storm event.

J.4 OVERVIEW OF SOLUTIONS

A number of solutions has been discussed in previous paragraphs. In these observations many advantages and disadvantages have been identified. In the following table a number of considerations is made based on several aspects like; Working environment, Hydraulic conditions, Erosion shift, etc.

TABLE J-1 COMPARISON COASTAL SOLUTIONS

	Seawall	Revetment	Groynes	Breakwaters	Mangroves	Grasses	Nourishments	Bypass system		**Total:**
Seawall	■	0	0	0	0	0	0	0		0
Revetment	1	■	0	1	0	0	0	0		2
Groynes	1	1	■	1	1	0	0	1		5
Breakwaters	1	0	0	■	0	0	0	0		1
Mangroves	1	1	0	1	■	0	0	1		4
Grasses	1	1	1	1	1	■	0	1		6
Nourishments	1	1	1	1	1	1	■	1		7
Bypass system	1	1	0	1	0	0	0	■		3

From here is can be seen that the solutions can be ranked from nourishments as the best solution and a seawall as the worst option.

APPENDIX K: DETAILED INFORMATION OF CHOSEN COASTAL SOLUTIONS

K.1 GRASS TYPE

The type of grass that could be applied should contain some essential characteristics due to the rough circumstances along the coast. In the Netherland the so called ‘Helm grass’ is applied which satisfy all the needs for the Dutch coastal conditions. The characteristics of the satisfying grass type should be:

- able to vegetate is salt and brackish environment;
- consist of long densely packed roots;
- and should be a native species.

According to the characteristics of the grass ‘Vetiver’, this type will satisfies all the set requirements. Besides that, this type of grass has a spectrum of useful properties as is shown in Figure K-1. It has already proved itself to work in several other projects in Indonesia (Vetiver-Indonesia, 2015).

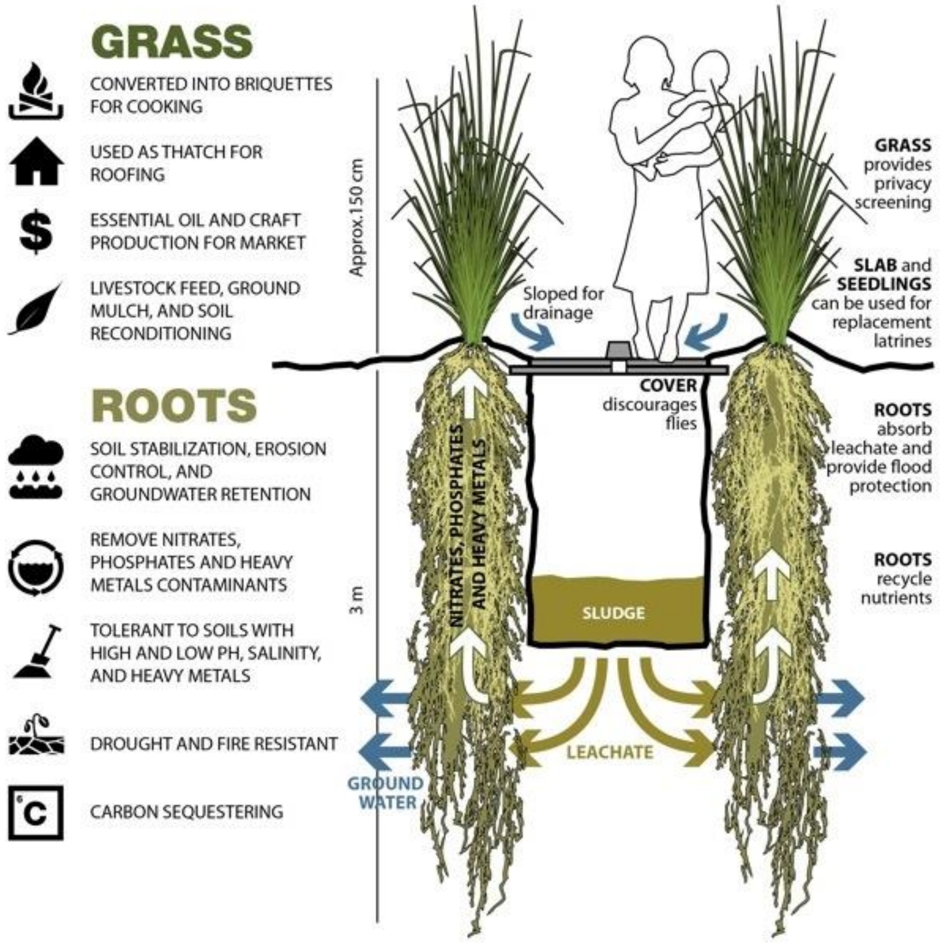


FIGURE K-1 CHARACTERISTICS OF VETIVER GRASS (VETIVERLATRINE, 2015)

K.2 NOURISHMENT TYPE

In this case nourishments can be separated into two categories: beach nourishment and shoreface nourishment. The exact location of the nourishments is theoretically not important as long as they are placed inside the active zone. Then, the cross-shore sediment transport will flatten out the perturbations over the whole beach profile. The effectiveness of both type of nourishments is comparable, however in the execution are large differences.

Applying beach nourishments means that the breaker zone needs to be crossed by the hopper. Since the circumstances in the breaker zone are quit rough, it may get very expensive to apply. This situation can be mitigated by two manners. Firstly, it is possible to construct a pipeline from outside the breaker zone to a fixed point on the beach. The hopper is able to transport a mix of sand and water through the pipeline to the beach, where the sand is being distributed over the beach by bulldozers. This distribution is coupled with a lot of disturbance and noise nuisance. Secondly, the dredged sand can directly be transported to trucks that can bring the sand to the location of deposition. Again location bulldozers are required here and the same nuisance occurs. Besides that, due to the required volume of sediment the frequency and amount of trucks will overload the local roads several times a year.

Applying shoreface nourishments gives the advantage the breaker zone has not to be crossed. Outside the breaker zone the depth is sufficient for the hopper to come close enough to deposit the sediment directly on the location. Since the waves will distribute the sediment no bulldozers are needed at the beach. This gives no perturbations of the recreational function of the beach and is more cost efficient then beach nourishments.

Experience has shown that shoreface nourishments are half as expensive as beach nourishments. Thereby it includes the large advantage that it gives no disturb to beach recreation. Hence, it has been chosen to apply shoreface nourishment in the coming solutions. Due to the rough sea circumstances inside the breaker zone, it will require heavy equipment for dredging the sediment at the accumulation zone. Regarding to the volume of sediments which should be transported at a yearly base, it is not cost efficient to dredge the sediment with a sand hopper that will apply the nourishments as well. Therefore it has also been decided to use excavators that will dig the sand from the eastern beach and load it into trucks. Those trucks will transport the sand towards the harbour where it is loaded into a ship equipped with a rainbow installation. That ship will cross the breaker zone via the harbour entrance, which is secured by the two breakwaters, and will then place the shoreface nourishment at the specific location. Even though this method includes way more proceedings and man power than using just one big sand hopper, it will be more beneficial due to the low cost of labour in Indonesia. The chosen location of the sand mining pit is also valid, because the eastern side of the harbour is only used for industrial purposes and therefore the perturbations of the beach by the excavators and the trucks will be acceptable. In this way it is assumed to obtain the most beneficial method by choosing the most effective nourishment method in combination with optimal land uses.

APPENDIX L: BUILDING SCHEDULE

FIGURE M-1 BUILDING SCHEDULE