Technical University Delft

CIE4061-09 Multidisciplinary Project

Samarinda Water Project

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Preface

This report has been written for the course *CIE4061-09 Multidisciplinary Project* at the Delft University of Technology, commissioned by Arsari Group. The report is a preliminary study for an artificial storage lake to provide the city of Samarinda with clean drinking water.

The Multidisciplinary Project is an MSc. elective at the Faculty of Civil Engineering and Geo-Sciences. The Multidisciplinary Project offers the possibility to perform a project abroad, with multiple students from different educational tracks. The duration of a project is ten weeks full-time, excluding the preparatory work, writing of the report and formalities. The reward for the MSc project (graded a 6,0 or higher) is 10 ECTS. The team exists of six students with backgrounds in Construction Management Engineering, Hydraulic Engineering and Geo-technical Engineering.

The research was done at Pt. ITCI Kartika Utama near Balikpapan in East Kalimantan. The team stayed there for seven weeks to conduct fieldwork, gather data and do research on the different subjects needed for the main research question.

This research considers multiple aspects: The technical aspects regarding the hydrology, geological conditions and the design of the different elements of the storage and transportation of the water. On top of that the general feasibility of the project considering the stakeholders, risks and financial feasibility is reviewed.

This report would not have been possible without the support from a number of people and therefore we want to give a special thanks, firstly to Willy Smits, for giving us the unique opportunity to investigate this problem for Arsari Group and facilitating us with all the necessary help in carrying out the research. Secondly we would like to thank Tjerk Zitman and Rob Schoenmaker for supervising this project. During the preparation phase, while we were in Indonesia and during the aftermath, we could always count on you for guidance, short questions and a good chat on skype, many thanks! During our stay at ITCI we were pleased to have a lot of contact and multiple meetings with Timo Worm (Witteveen+Bos) who was kindly to assist us with the technical questions. Lastly, we want to express our gratitude to all the people at ITCI: pak Tokit, ibu Erika, pak Jan, pak Arsat, pak Tofitor, pak Mora and pak Jurianto. Our stay at ITCI was superb! The food, our accommodation, the help with the visa, the good company and everything else, we are very grateful! Terima Kasih Banyak!

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Executive Summary

The cities of Tenggarong and Samarinda in East Kalimantan are in need of a new source of drinking water, since their current source, the Mahakam river, is said to be contaminated. Arsari Enviro Industri owns a large concession named ITCI of a partly degraded forest 90 kilometers East of Samarinda. This report suggests several design possibilities how to capture water in the ITCI concession area and how to transport it to the aforementioned cities. Additionally, the the financial, technical and social feasibility is evaluated.

The topographic characteristics of ITCI have been investigated to identify catchments and possible dam locations from which the best alternative was selected. The geotechnical characteristics and geotechnical risks have been mapped by means of several geological surveys at the project site. A SOBEK rainfall run-off model of this catchment was made to generate the discharge time series from TRMM precipitation observations. Design alternatives for the dam, pipelines, hydro power and water treatment plants were produced. Subsequently the best alternative was selected after evaluation. The financial feasibility is assessed with the use of a multi-scenario self produced NPV-model and sensitivity analyses. The social feasibility is assessed with the use of multiple stakeholder analyses methods. A general risk and opportunity assessment is performed and mitigation measures are given. The outcome of the financial, risk and opportunity and stakeholder analyses are then used as input for a stakeholder engagement plan.

The catchment named phase 2A has an total area of 7950 ha. Fluctuations of the dry and wet season and the occurrence of El Niño and La Nina have been accounted for in determining the optimal extraction yield of the artifical reservoir of 1,8 m^3/s. The most suitable water level target height has been depicted by not only modelling the behaviour of the reservoir with the modelled 19 year discharge time series, but also taking the extraction yield, construction costs of the dam and hydro power production into account. The design volume of the reservoir will be equal to $41 \cdot 10^6$ m^3. Several scenarios for the fill up time exist: with an discharge it takes 14 months, in a really wet year 10 months, but in dry years just over 26 months. A spillway must be constructed with a capacity of 83 m^3/s, to ensure that no overtopping occurs over the dam during peak rain events with a return period of $R = 1000$ years.

Largest part of the basin floor is expected to be dominated by soils with high clay content, which is a very positive finding. The western ridge of the basin is expected to consist of mainly sandstone and shale. The sandstone layers could present a problem under undesirable circumstances. A fault is assumed to be present and could induce piping of the basin, although not very likely. Limestone units are present in the project area, albeit not at the surface, and therefore presence of karst can prove to be a significant risk if present at unfavourable locations. Eventual steep slopes surround the reservoir which can become unstable and slide as a result of the varying water level within the reservoir.

The functional requirements and a description of the basic design of the dam has been made. Visual geotechnical inspection at the dam site have been conducted and have resulted in a understanding of the local area. The requirements have been formulated in terms of structural, geotechnical and hydraulic components from which design choices have been made for the dam type, membrane type, filter layers, under layers, transition zones, slope cover, freeboard, crest height, crest width, spillway as well as the tunnel outlet.

Similarly, a feasibility design of the water transportation means is made. After deducing the water loss in the water treatment plant a capacity of $Q = 1,7$ m^3/s is required. Several routes, dimensions and materials have been proposed, after which the most favourable options were selected considering the requirements. Next to that, reservoir location and specifications were depicted.

The head difference between the reservoir an the hydro power plant is a potential source of energy, however more study is required to optimise efficiency and energy yield. An indication of the water treatment

plants requirements has been suggested, much as the water quality in the catchment needs to be assessed to resolve the treatment steps needed.
Several issues for the financial and social feasibility were found. The main social issue is that between two critical and indispensable stakeholder groups different problem perceptions occur, causing the governmental stakeholders to neglect the need for the project. The social and financial analysis showed that problems might arise with the financial ownership of separate project phases, The financial analysis showed that financial ownership of the project will decide if it is financially feasible. This was found through analysis of two ownership scenarios: full and partial ownership by Arsari. Additionally, the production capacity does not meet the present and estimated future demand. Moreover, it became apparent that there is a large negative discrepancy between the selling and production price of water. Several opportunities were found, one of which could be used to mitigate some of the financial issues. For the individual issues and opportunity, two main management approach theories and financial opportunities were considered, chosen and adapted to each specific case. From the technical and social analyses it was found the project is feasible. However, during the financial analysis it became apparent if Arsari has financial ownership over the entire project, it is not feasible. Because of the issues for the financial feasibility and the discrepancy between the demand and supply the main recommendation is to look into the possibility of increasing the production capacity through a connection with phase 1 and phase 2 lake B.

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Chapter 1

Introduction

1.1 Project introduction

Borneo is the third largest island in the world and is divided in Malaysia and Brunei in the North and Indonesia in the south (Albelda, 2015). The Borneo rainforest is one of the oldest on our planet and is home to many endemic species of plants and animals.
Kalimantan, the Indonesian portion of Borneo, consists of four provinces: North, South, West and East Kalimantan. East Kalimantan has a population of about 3,5 million people and its capital is Samarinda. The economy of East-Kalimantan is heavily dependent on earth resources such as oilfield exploration, natural gas, coal and gold (Wibisono, 2015; Anderson, Kusters, Obidzinski, & McCarthy, 2015). (Illegal) logging, agriculture, mining, industry all contributed to the removal of much of the original forest cover. East Kalimantan's intact primary forest area has dropped to just 15 percent of the total area of the province (Parker, 2013).

The city of Samarinda and the city of Tenggarong lie on the banks of the Mahakam river, approximately 50 kilometers from the delta, and are among the most populous cities of Borneo with approximately 1 million inhabitants combined. It is forecasted that in 2035 that will grow to close to 1,7 million inhabitants (see table F.1). The illegal forestry, coal-mining activities and agricultural runoff have polluted the Mahakam river (Adri, 2015). Pollution of the river and population growth drive the already growing need for clean drinking water.
Arsari Enviro Industri, through its subsidiary Arsari Tirta Pradana (Henceforward: Arsari), owns a 475.000-acre forest concession east of Samarinda. Now, Arsari is looking into the feasibility of capturing river water and transporting it to Samarinda. Arsari and Witteveen+Bos, are already in an advanced design stage of the infrastructure to capture 120 million m^3 / year of river water in the area Hutan Arsari Lestari and sending it by pipeline to Balikpapan (Henceforward: phase 1).
The water will be captured in an artificial reservoir by the construction of a dam. The water will be transported from the reservoir to the hydro power and water treatment plant by a headrace tunnel and transportation pipelines. Subsequently the water will be transported to various end users in the communities of Tenggarong and Samarinda.
The scope of the report was primarily set to examine two watersheds i.e. two artificial storage lakes, phase 2A and phase 2B respectively. Due to time constraints only phase 2A is taken into consideration. This report was planned to be a preliminary report of various considerations and conclusions for several disciplines, an overview of the main risks and opportunities and recommendations for further developments. Considering the level of detail and thoroughness it is in reality closer to a feasibility study.

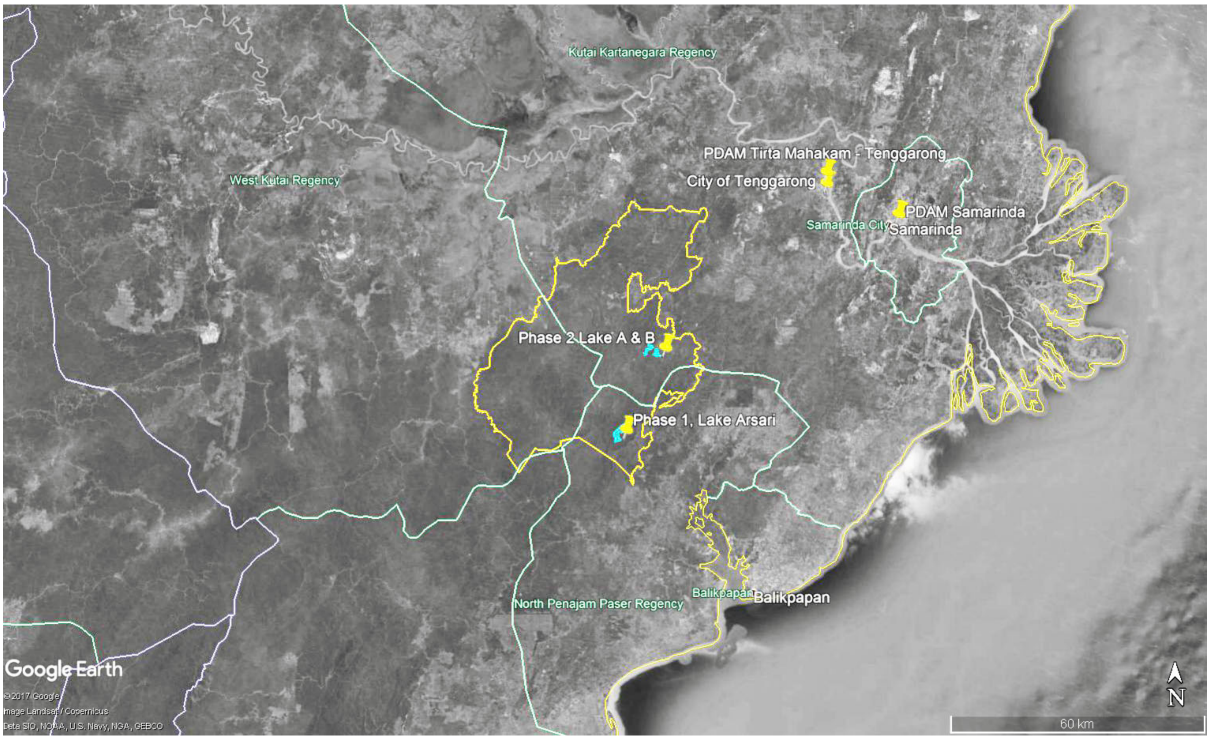


Figure 1.1: Overview of the area of research

1.2 Problem definition

As mentioned above in the 1.1 in the (near) future problems might occur with the clean water supply for the city of Samarinda and Tenggarong due to pollution of the clean water sources and a growing population. Arsari was asked to propose a solution using their concession as a new source of clean water. Arsari currently is unsure if the project is technically, financially and socially feasible. This leads to the research objectives, question and sub-questions below.

1.3 Objectives

Arsari is considering to invest in this project for commercial reasons, financial gain, as well as social reasons through the development of the area. The commercial benefits Arsari wishes to create are necessary for the main objectives of Arsari, namely protection, preservation and advancement of nature both nationally and globally. To make the investment decision, a thorough overview of the technical, financial and social details of the project is needed. This leads to the following objectives:

- A reliable estimation of the available amount and feasibility of the water supply.
- Outline and basic design of the relevant infrastructure and a financial analysis including reliable investment cost estimates.
- Identification and evaluation of possible risks.
- Identification of the stakeholders and an analysis of their characteristics and network together with a comprehensive engagement plan.
- Outline plan for the project

The need for an Environmental Impact Analysis is high. The flora and fauna are unique and under high pressure in that area. Arsari will instruct a local party to conduct this analysis. For this research, based on several sources including the client, it is assumed that the water quality of the Mahakam river is, or will become in the near future, below the standard for drinking water. Similarly, a study to the water sale opportunities and permits is not part of this report, but will be carried out by Arsari.

1.4 Research questions

Main research question:

Is it technically and financially feasible to capture water in the phase 2A catchment area, transport it and sell it to the PDAM of Tenggarong and Samarinda?

- What is the amount of water that can be captured?
  - What are the limits of the catchment area?
  - What is the long-term average?
  - What is the variability of the amount of water that can be captured?
  - What are the extreme values of the amount of water that can be captured?
  - What are the characteristics of the reservoir?
- What are the geotechnical characteristics of the subsurface within the catchment?
  - What are the geotechnical risks?
- What is the optimal dam design for the reservoir?
  - What are the requirements?
  - What is the optimal location for the dam?
  - What is the optimal dam type, filter, slope cover, crest height, dam foundation and spillway design?
- What is the best design for the hydro power plant?
  - What is the best location?
  - What is the optimal hydro power generator?
- What are the stakeholder influences on the project?
  - Who are the stakeholders?
  - What are their characteristics?
  - Who are the critical stakeholders?
  - How are they connected?
  - What are the consequences for the feasibility of the project?
- Will the project be financially feasible?
  - What is the current and future demand of clean water for Samarinda and Tenggarong?
  - What are the total investment costs?
  - What will different break-even scenarios bring forward in realizing financial feasibility within a realistic timespan?

- How should the stakeholders be engaged?
  - What issues exist for the feasibility of the project?
  - What is the appropriate engagement of these issues?
  - What approach could be used in the (stakeholders) subsequent phases of the project?
- What are the risks and opportunities of this project?
  - What are the critical risks?
  - What risks and opportunities exist for the feasibility of the project?
- What are the recommendations?

1.5 Structure of the report

The report is subdivided as follows:

- Chapter 3 deals with the hydrological study. The focus in this chapter is to determine the amount of water that can be constantly extracted from an artificial reservoir in the "Samarinda watershed".
- Chapter 4 describes the process of conducting a geological survey as part of the technical- and financial feasibility study of the Samarinda Water Project.
- Chapter 5 will focus on the dam requirements and the basic design of the feasibility study.
- Chapter 6 gives a short overview of the water transportation from the reservoir to the PDAM of Tenggarong and Samarinda.
- Chapter 7 provides the functional requirements and starting points, design alternatives and basic design of the hydro power and water treatment plant cluster.
- Chapter 8 depicts the most important parts of the complete stakeholder analysis.
- Chapter 9 analyses the financial feasibility of the Samarinda Water project.
- Chapter 10 describes how the issues for the feasibility of the project, found in the stakeholder analysis, financial analysis and the risk and opportunity analysis, can be engaged through stakeholders.
- Chapter 11 presents important risks and opportunities that have been identified during this study.

Chapter 2

Methodology

Figure 2.1 gives an overview of the research methodology used in this report. For this project the first 7 phases have been executed. The last three steps have not been executed because they are outside of the scope of the project. Several iterations between step 2 and 3 were made because of absence of specific data at the start of the project and the need for a problem definition at the start of the project. Each step of the research methodology is depicted by a bullet list of the actions undertaken.

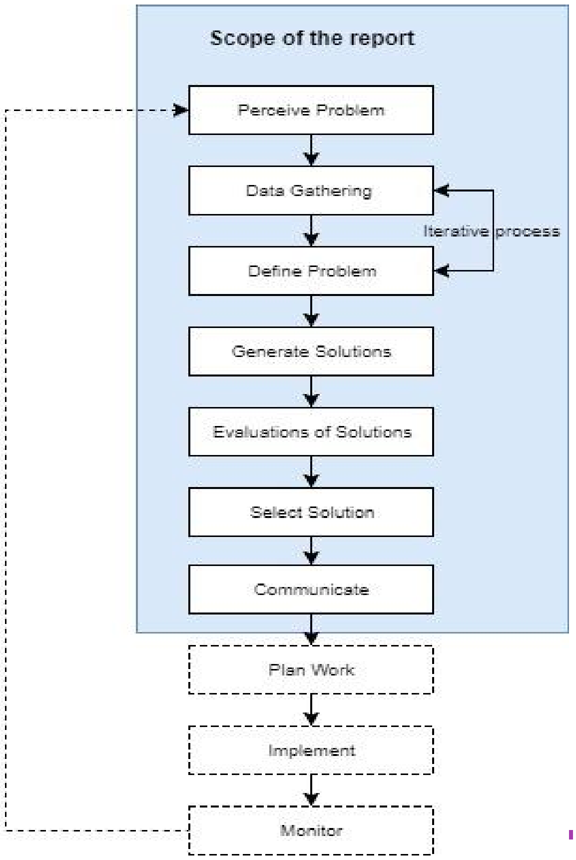


Figure 2.1: Research methodology (Turner, 2009)

2.1 Perceive problem

- Contact with the client
- Literature study
- Field excursions

2.2 Gather data

- Geological survey in the field for subsurface mapping and rock characteristics through expeditions to the project area
- Geological maps of Pertamina and lithological data of Witteveen+Bos
- Hydrological survey in the field
- Installation of measurement equipment
- Visits to the project site
- Interviews with client staff, Witteveen+Bos and stakeholders such as the PDAM
- Literature study
- Data provided by the client
- Open source data (sources are mentioned and evaluated in the relevant sections)

2.3 Define problems

- Literature study
- STQ (Situation - Trigger - Questions)
- Issue tree
- Interviews with client staff (Arsari) and contractor (Witteveen+Bos)
- Stakeholder analysis

2.4 Generate solutions

- The characteristics of the topography of the concession area were investigated through processing DEM data in ArcGIS in order to identify catchments, for which volume and surface area for several target water levels were determined.
- A model is made in SOBEK (Deltares) in order to determine the discharge with 19 years precipitation data, from 1998 until 2016, for the catchment within the watershed in the concession area.
- The time series of discharge rates, generated by the SOBEK rates, are used to simulate the water level within the reservoir for several target heights.
- A dam design for different locations were produced.
- Pipeline routes are proposed. The dimension of the pipelines have been calculated in order to transmission water from the reservoir to the hydro power and water treatment plant to the offtake by Tenggarong and Samarinda by gravity propulsion.

- Hydro power generation and water treatment has been evaluated.
- The installed capacity i.e. demand of water for the cities Samarinda and Tenggarong have been forecasted till 2035.
- The direct costs and added costs are used to estimate the total initial investment costs based on optimistic and pessimistic values derived from empirical studies - P50 cost estimate.
- NPV-analyses have been performed in order to apprehend the average yearly water price (NPV-model with 10 year payback period and average yearly water price as output) and the year in which the project breaks-even (NPV-model with average water price as input) based on two scenarios:
  - Arsari bears all the costs.
  - Arsari invests in the storage lake phase and the PDAMs in the transportation and offtake phase.
- Analyses of possible stakeholder engagement approaches.

2.5 Evaluate solutions

- Different reservoir, dam and pipeline configurations have been evaluated on multiple criteria.
- An evaluation of the geotechnical feasibility of the construction of a dam and the creation of a storage lake at this location.
- The financial feasibility is based on the criteria that the price could be enlarged according to Indonesian law whilst breaking even within a certain period in time.
- A sensitivity analysis is performed to examine the sensitivity and robustness of the NPV-models.
- An evaluation of possible stakeholder approaches with respect to the issues found in the stakeholder analysis.

2.6 Select solutions

- Optimal configuration for reservoir, pipeline and dam recommended.
- Select and apply stakeholder engagement approaches.

Chapter 3

Hydrology Study

The focus in this chapter is to determine the amount of water that can be retained by an artificial reservoir in the "Samarinda watershed". The feasibility of this reservoir depends largely on the amount of precipitation that can be captured, retained and then constantly extracted.
The amount precipitation will fluctuate through the year. Seasons influence the precipitation pattern. So, the artificial reservoir faces the risk of depletion in case of absence of rain during the dry season. The other side of the coin is the wet season, when excessive rainfall might cause the reservoir to over top. Overtopping is not desirable, because of its potential hazards and the waste of water.
Next to seasonality, the reservoir is situated in an area that experiences the climate phoneme El Niño-Southern Oscillation (ENSO). Here, an El Niño is a period of extreme drought and La Niña exactly the opposite, extreme rainfall. These variabilities have to be accounted for in the design. The reservoir is required to deliver a constant discharge, but is allowed to have a fluctuating water level.
The results produced by this report have certain inaccuracies and uncertainties. These mainly originate from the fact that the model hasn't been calibrated yet, because in site measurements are do not exist currently. To tackle this problem we installed a diver in the riverbed in at the downstream end of the catchment and will measure the water level during a year. With these time series the SOBEK-model can be calibrated. Seepage and the open water evaporation are also uncertainties that have to be estimated now. These uncertainties are recommended to be the object of a future report, further investigating the feasibility of this reservoir.
This chapter therefore considers orders of magnitude of the dimensions of the reservoir and the mean abstraction yield. Further research can deliver more exact data. Both in the field, as in the analysis.

3.1 Hydrological analysis

3.1.1 Catchment area

The catchment area of the rivers can be found in the figure 3.1. The catchment area can be divided in 2A and 2B. The main focus in this report will be on area 2A. The total catchment area is 11695 ha, 2A is 7940 ha and 2B is 3755 ha. The catchment area of this project is 3% from the total catchment.

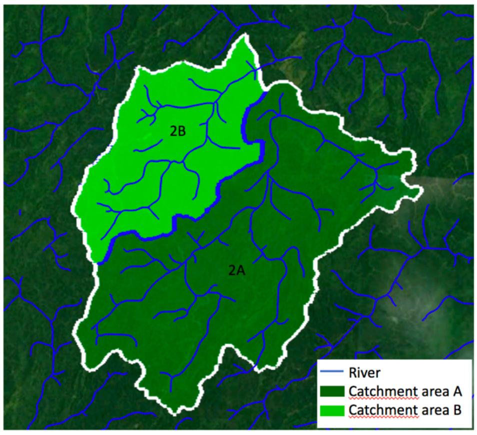


Figure 3.1: Overview of catchment A and B *(own work)*

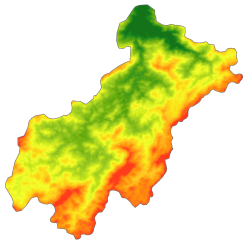


Figure 3.2: Height profile of catchment A *(own work)*

Catchment elevation

The elevation and slope steepness are two important factors influencing the character of the catchment reservoir. They determine the border of the catchment, the amount of surface runoff, the width of the rivers, amongst other factors.
These parameters are obtained from the ASTER GDEM (Global Digital Elevation Model). ASTER GDEM. This model has been produced by both the United States National Aeronautics and Space Administration (NASA) and the Ministry of Economy, Trade and Industry (METI) of Japan. The model accuracy is 20 meters at 95% confidence for vertical data and 30 meters at 95% confidence for horizontal data. As the analysis of the hydrological characteristics of the model as well as the reservoir parameters are based on the relative horizontal and vertical difference, this accuracy will not influence the results.
Figure C.4 shows the total height difference within the watershed of 375 meters, resulting from the peak of 485 m above MSL towards the lowest point in the watershed at 100 m above MSL. This elevation difference in the watershed leads towards a slope in the catchment, these slopes impact the flow velocity in the river, as well as the runoff speed after rainfall.

3.1.2 Rainfall data

The main source of water for the reservoir is the rainfall in the watershed. A certain amount of rain will fall in the catchment area and, like a giant bathtub, end up in the main river through surface runoff, percolation, drainage and small streams. The amount of rainfall is therefore a very important parameter for the model. Three different data sources were presented: the ITCI rainfall data, the BMKG ground station data and the TRMM sattelite data.

The three datasets were evaluated and compared. The data obtained by ITCI cannot be used since the time series is too short. The TRMM data is the most accurate, because it represents the data of the location of the reservoir, whereas the BMKG data is based on a different location 60 kilometres away.
The TRMM data covers an area of nearly $8000ha$. The satellite data is averaged over an area. This does result in less spatial resolution, but in long term simulations the characteristics of this reservoir will level out. This is a major advantage over the BMKG data, which is rain data from one location specific, namely the rainfall measurement station in Balikpapan.

Therefore, the BMKG set is chosen as a tool to validate the TRMM data by comparing them both in terms of yearly totals and month specific averages. The results of this comparison can be found in Appendix A.1.1. As seen in both the figures and the tables, a good correlation is visible between the BMKG and the TRMM data. The SOBEK modelling will therefore be done based on the TRMM rainfall data (1998-2016).

3.1.3 Evaporation

The MOD project (MODIS global Evapotranspiration project) estimated the global terrestrial evapotranspiration form earth and land surface. This is done by using satellite remote sensing data. This project is designed by NASA/EOS.

Evapotranspiration is divided in transpiration and evaporation. The value of the evapotranspiration drops in dry periods due to a smaller evaporation, because there is no 'free' water available and a reduced transpiration. There will always be a positive evapotranspiration value in a tropical forest even during a very dry season.

The daily (24 hours) evapotranspiration data can be found in figure A.3 in the Appendix. From the figure can be seen that the maxima is around 8 mm/day and the minima is around 4,5 mm/day. It can be concluded that the average is at 5,7 mm/day, from Witteveen+Bos. The monthly average evapotranspiration can be found in figure A.4 in the Appendix.

Open water evaporation

The process open water evaporation is present by the artificial lake. There will be always water that evaporate and there is not any transpiration that takes place. In this case, the actual evapotranspiration will be higher than the open water evaporation.

There has been a long-term pan-evaporation test in Singapore. The conclusion of this test was that the open water evaporation rate is around 5,5 mm/day, as said in Witteveen+Bos report. This test had been done with a water temperature from around 31 degrees. The first phase of the project assumes an open water evaporation of around 4,5 mm/day. This is depending on the different factors such as that the lake is more protected from winds and that the reservoir is situated at an elevation of about 300 m+ MSL and so it is expected to be cooler, around 25 degrees.

In this phase, the open water evaporation is calculated with the formula of Penman, (Savenije, 2014; Wang-Erlandsson, Savenije, van der Ent, & Gordon, 2014). This formula has four meteorologist parameters, which are wind speed, relative humidity, temperature of the air and sun hours as a way to measure the radiation (Wang-Erlandsson et al., 2014). In A.1.3 in the Appendix, the open water evaporation calculation can be found.

From the calculation, it can be seen that the open water evaporation is around 4,36 mm/day. This calculation is made with parameters originate from Balikpapan. This city is situated at 0 m + MSL. The artificial lake will be situated at 100 m + MSL and in the jungle, so it is expected to be cooler and more protected from winds. Also, there will be a higher humidity. A conservative (higher) open water evaporation rate is safer for the further calculations. So, the open water evaporation is 4,36 mm/day, which is a conservative open water evaporation.

3.1.4 Seepage

The soil should be extensively investigated for geotechnical purposes, as well as for the hydrological purposes, e.g. seepage rate. The amount of seepage is a very important factor when constructing a large water reservoir. In practice, most of the reservoirs leak. Either at the dam construction or through the natural bed of the reservoir. This factor shouldn't be underestimated.
The majority of the surface of the envisioned reservoir consists of sandstone with smaller clay layers.

Layers of limestone have been found as well. See chapter 4, Geology, for a detailed analysis of the geology of the catchment and reservoir. See table 3.1 for the AQTESOLV permeability bandwith

Table 3.1: Permeability for different sedimentary rocks as given by AQTESOLV

Sedimentary Rocks	
Rock Type	Hydraulic Conductivity [m/s]
Karst and reef limestone	$1 \cdot 10^{-6}$ - $2 \cdot 10^{-2}$
Limestone, dolomite	$1 \cdot 10^{-9}$ - $6 \cdot 10^{-6}$
Sandstone	$3 \cdot 10^{-10}$ - $6 \cdot 10^{-6}$
Siltstone	$1 \cdot 10^{-11}$ - $1.4 \cdot 10^{-8}$
Salt	$1 \cdot 10^{-12}$ - $1 \cdot 10^{-10}$
Anhydrite	$4 \cdot 10^{-13}$ - $2 \cdot 10^{-8}$
Shale	$4 \cdot 10^{-13}$ - $2 \cdot 10^{-9}$

As for this analysis, until in-situ measurements and observations have been performed, we will assume that the reservoir bed mainly consists of sandstone. The AQTESOLV says the permeability of sandstone is between the 0,025 and 500 mm/day. Note, this is a very large bandwidth.

Sensitivity of seepage

Sandstone can be fairly impermeable because of its relative small intergranular pore space as a result from the cementation and compaction of the original substrate: sand. However, older sandstone can contain cracks, fractures and joints. These are prone to create significant flow paths of water, which on its turn can increase the size of these faults.
However, clay can have a decreasing effect on the seepage rate, if these layers of clay prove to be consistent and abundant. Clay can fill up the cracks, faults, joints and fractures and cover the imperfections. This potentially has a decreasing effect on the seepage rate of the reservoir.
Lots of reservoirs have been built on karstic limestones. Whereas most reservoirs function successfully, a few of them cope with large water losses. Generally speaking, the main problem is water tightness by dissolutional widening of karst fractures in a dam. The varied surface landforms are merely a guide to the presence of unpredictable subsurface caves, conduits as well as many different karst underground features. (Bonacci, et al., 2008). Owing to these reasons, karst presents special challenges for the engineer.
Thus, seepage is not a uniform and evenly distributed phenomenon. A 10-fold increase of seepage rates will result in not $3,65 cm/year$, but $36,5 cm/year$.
To conclude, the need for extensive geological research and seepage measurements to investigate the seepage rates cannot be emphasized enough. It must at least include the following:

- Sampling of the sandstone layer at several depths on a specific spatial grid.
- Analysis of the topsoil
- A survey to inspect the extent and presence of joints, faults and fractures.
- Pumping tests to determine the permeability in-situ.

3.1.5 Summary

- The catchment 2A has an area of 7950 ha.
- The elevation and slope steepness are obtained by processing of the ASTER GDEM data using ArcGIS software.
- Three rain data sets were evaluated and compared. The TRMM data proved to be most consistent and reliable and will therefore be the source of the precipitation input for the SOBEK model.

- The evaporation is calculated based on the MOD project at an average of 5,7 mm/day.
- The open water evaporation is determined at an conservative 4,4 mm/day.
- Seepage rates are impossible to determine without proper in-situ measurements and inspections. In this study we assumed that the reservoir mainly consists of sandstone and therefore set at 36,5 mm/year.

3.2 Hydrological Modelling

3.2.1 Model layout

The aim of the SOBEK-1D model is to calculate the rivers-discharge, incorporating the spatial variability of properties of the catchment area. Now the rain, evaporation and seepage data can be translated to input data into SOBEK. In order to introduce the spatial variability of the catchment into the model the following assumptions have been made:

- The entire catchment has the same coverage, secondary dry rainforest.
- The entire catchment is reigned by the same climate conditions.
- The entire catchment consists of the same bedrock material.
- The main variability of the catchment is processes linked to the difference in slope.

The catchment has been divided into sub catchments. The sub catchments have been categorized to the criteria mentioned above. The lay-out is presented in figure 3.3 and in figure 3.4. Sacramento nodes are the modules that model the volume of water per unit time that flows into the channels as a result of rainfall-runoff (hereafter RR) processes in each sub catchment. The 1D river branch network models the rivers channels spatially per unit time. More cross section nodes increase the accuracy of the of the schematic riverbed , more Sacramento nodes increase the accuracy of the RR processes in the catchment. As we are primary interested in water levels, volumes and discharge variabilities, we limit the number of cross sections in this model. This does not impact the accuracy discharge calculations.

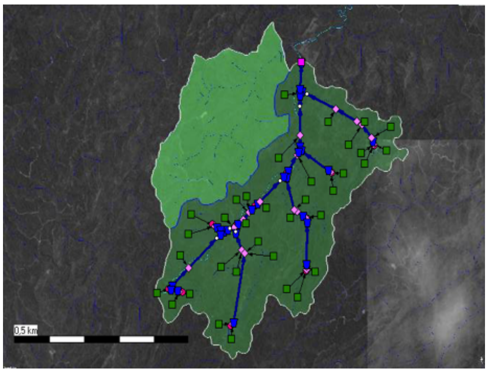


Figure 3.3: Overview of the catchment and the schematic network of SOBEK

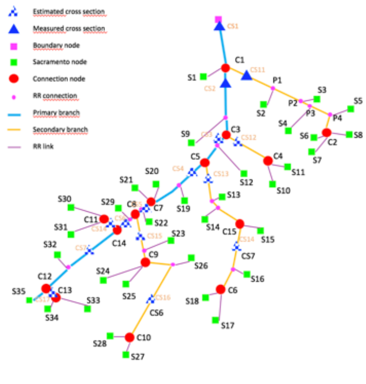


Figure 3.4: The network of branches, nodes and Sacramento nodes in SOBEK

3.2.2 Goal of the model

The goal of the model is to analyse the river's discharge, as well as the consistency and variability. Since we do not possess long term measurements of the river's discharge, we are dependent on results of the model.

3.2.3 Setup of the model

In SOBEK each sub catchment, as described in, was assigned to a Sacramento node (See figure 3.3 and figure 3.4). A variety of parameters for the area covered, amount of percolation, surface runoff, retention, storage, drainage rates and thresholds for overland flow can be specified for each node. The river branches are described by the aforementioned cross sections. The most important branches have been inspected, where others are estimated. The simulation period is from 1st of January 1998 until 30 November 2016.

3.2.4 Scenarios

The scenario that will be simulated is the current situation, a natural catchment. The input rainfall and evaporation data is available from 1998 till 2016 on a daily basis. That is therefore the simulation time and temporal simulation resolution as well.

Input

The input data for the SOBEK simulations are stated in table 3.2.

Table 3.2: Input data and source for the SOBEK simulation.

Data	**Source**
Rainfall data	TRMM [mm/day]
Evaporation data	MOD16 Daily Evapotranspiration data [mm/day]
Rough land use data	Satellite imagery, as well as 4 site visits
River cross-sections	Multiple site visits, photos and measurements
Slope in the catchment	Derived from SRTM elevation data [%]
Topography catchment	SRTM elevation data [m+ MSL]
Calibration data	None yet - diver is installed on October 10th 2017

Calibration

The diver is installed on the 10th of October 2017. At the time writing this report the data series that has been collected is too short in order to calibrate the model properly. In this report, the assumption is made that the RR process as a result from the hills, vegetation and soil characteristics are similar to that of the Arsari catchment calibrated by Witteveen+Bos. We have used the same categorisation and parameters. **The results of the model are not reliable until the model is calibrated**.

3.3 Model results

3.3.1 Model results analysis

In this section, an analysis is conducted on the results from the SOBEK model on the catchment area. The model output is the average discharge over twelve hours in cubic meters per second $[m^3/s]$ at the location at the bottom of the catchment area, which approximately be the location of the dam. The output data is a set of 13816 average discharges, covering 214 months originating from the TRMM Rainfall data.

3.3.2 Long-term average

Looking at the average discharges, it shows a good correlation with the precipitation, as expected. Therefore, it is also strongly correlated with the wet and dry season, leading to a large fluctuation in the monthly averages. In table A.8 in the Appendix A.2.1, the monthly averages are presented.

The long-term average discharge is $1,73\ m^3/s$. In figure F.2 in Appendix A.2.1 the different monthly averages are presented. The wet and dry season are clearly visible. With wet months averaging at 3 m^3/s and dry months as low as $0,5\ m^3/s$ can be seen. This fluctuation indicates the need for a rather large buffer in the reservoir, if a constant yield is to be extracted from the reservoir.

Figure A.6 in Appendix A.2.1 shows the difference in dry and wet seasons each year and also the fluctuation over the years between the seasons. Not only does the discharge fluctuate within the year, but also the years itself show strong differences. With dry season discharges ranging from $0,4\ m^3/s$ to $2,6\ m^3/s$ and in wet seasons even ranging from $1,0\ m^3/s$ to $3,1\ m^3/s$. This fluctuation is can be explained by the existence of climate phenomena such as El Nino and La Nina years. These climate events are each-others counterparts and are irregular periodical variations in the winds and surface temperature of the sea over the tropical eastern Pacific Ocean. The first one causes dry periods and therefore low monthly discharges and the second one exactly the opposite, wet periods causing high discharges.

3.3.3 Peak values

The long-term averages, fluctuation within years caused by the dry and wet season and the significant impact of El Nino and La Nina are important parameters for the eventual constant abstraction yield. The peak values of the data determine the eventual crest height, spillway and other possible emergency measures. The top peak discharges are presented in table A.9 in Appendix A.2.2. The maximum modelled discharge is $53\ m^3/s$.

Not only the maximum peak values are of importance for the reservoir dimensions, also the minimum peak values are relevant, though only when occurring for a significantly long period. This phenomenon can be seen in figure A.7 in Appendix A.2.2, where two periods with no significant peaks and a minimal base flow are shown. If these periods last long enough, they can lead to depletion of the reservoir. When a constant yield is a design requirement, it may not occur that the water level reaches a critical point, where there is no longer enough pressure to extract the water and additional pumps are needed, or worse: no longer being able to supply the City of Samarinda and Tenggarong of drinking water. This requirement is tested in section 3.4.

3.3.4 Extreme value analysis

To investigate the different flood discharges and its return periods, an extreme value analysis is conducted. The Exponential distribution function is used to calculate the extreme values for a given return period. The results are depicted in table 3.3. It can be concluded that the difference in extreme river discharge between higher return periods becomes smaller.

Table 3.3: Peak discharge predictions for given return periods

R [years]	1	2	5	10	15	25	50	100	250	1000
Q [m^3/s]	25,55	31,36	39,04	44,85	48,25	52,53	58,34	64,14	71,82	83,44

3.3.5 Summary

The river discharge has a long-term average of $1,73\ m^3/s$. However, the fluctuation of the dry and wet season and the occurrence of El Nino and La Nina have major impacts on the reservoir configuration.

First of all, the reservoir should be dimensioned so that it does not deplete during long term draughts. Also, the variation in average discharge between the dry and wet season ($1,17\ m^3/s$, $2,13\ m^3/s$) must be considered when configuring the reservoir and thereby the dam dimensions. Secondly, looking at the peak discharges, a spillway must be constructed to ensure that no overtopping occurs over the dam. The spillway is to be designed for a PMF discharge of 83 m^3/s.

3.4 Reservoir configurations

3.4.1 General reservoir process descriptions

In the following chapter the reservoir configuration will be determined. To get a good overview of the different aspects that need to be taken into consideration, the processes within the reservoir itself need to be identified.
The dam will be constructed at a location downstream of the catchment area to capture the water off the river. After the construction is finished, the reservoir fill start filling up. This process is mainly determined by the river discharge, but also affected by seepage and evaporation processes. The discharge is highly correlated with the rainfall, which fluctuates significantly over time. With the volume of the reservoir increasing, the flooded area increases as well. This process continues until the target water level is reached. At that moment, the extraction of a constant amount of water per unit time can start. This will happen at a pre-determined and constant rate, resulting in two dominant mechanisms influencing the water volume inside the reservoir. The inflow of water from the catchment area, controlled by the highly varying rainfall and secondly the constant extraction yield. Here after the water will be transported to a hydro power and water treatment plant, before being transported to the customers.
Apart from these two dominant, open water evaporation and seepage will also influence the volume of the reservoir, and thereby the surface level of the reservoir. Over a longer period of time, the four processes, river discharge, extraction, evaporation and seepage, must be in equilibrium. If this is not the case, the reservoir will deplete or overflow. As mentioned before, one of the requirements for the reservoir is that the extraction yield must remain constant. The other three processes, especially the river discharge, will vary over time. Leading to an either positive, or negative flux in the reservoir. The requirement of a long-term equilibrium can therefore only be accomplished by enough storage volume to 'dampen' out the short-term flux. So, a rise in water level will be seen during the wet season, especially when accompanied with La Nina and the reservoir volume will slowly decrease in the dry season, also being at its low when accompanied with El Nino. The boundary condition for the maximum water level is determined simultaneously with the dimensioning of the spillway, as they affect each other. The minimal allowable water level is the level at which the dam remains operational, implying enough water pressure to 'push' the water out of the lake to the treatment plant.

3.4.2 Dam location

The topography together with the subsurface characteristics of the catchment determine the optimal location of the dam. See figure 3.5 for the elevation map for the downstream end of the catchment. The (thin) black line is the perimeter of the catchment and shows a suitable location for a dam since it is a bottleneck of the catchment. Figure 3.6 shows 3 possible dam locations A, B and C where the dam can be positioned.

The smaller and steeper the gap between the two hills, the less construction material is needed to close the catchment. However, if the slope is too steep, geotechnical problems such as slope instability arise. Figures 3.7, 3.8 and 3.9 show the elevation profiles of cross sections A, B and C. Also, several water levels are drawn in the figures to get an sense of context. Mind the horizontal axes, since they do not have the same step size. Considering the height of the adjacent hills and the narrowness of the gap, cross section B is the best location for the dam.

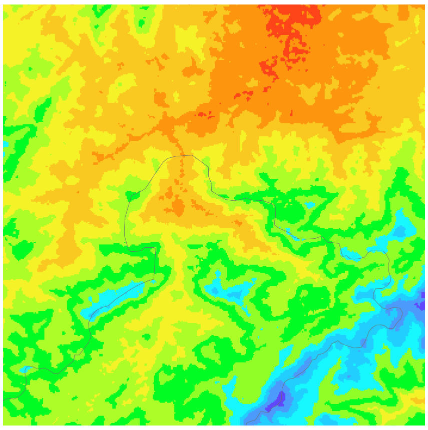


Figure 3.5: Overview of the elevation of the downstream end of the catchment

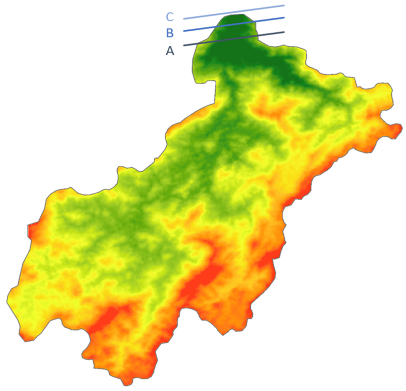


Figure 3.6: Complete overview of the elevation of the catchment

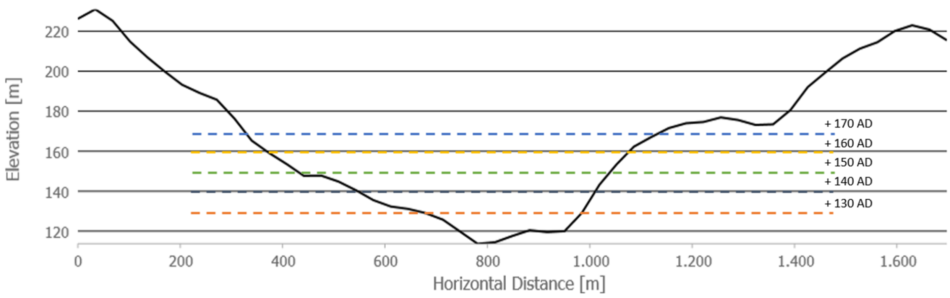


Figure 3.7: Elevation profile of cross section A

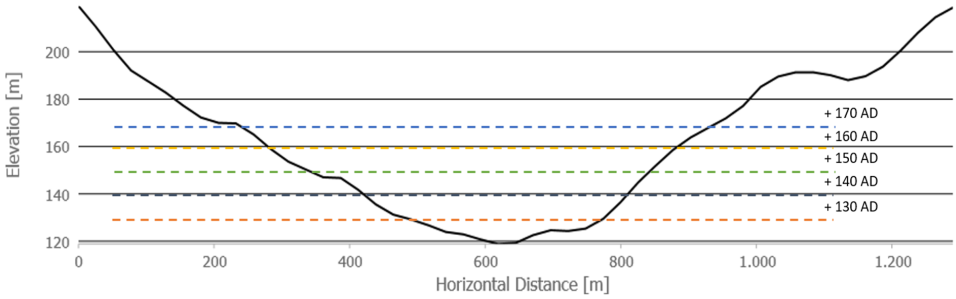


Figure 3.8: Elevation profile of cross section B

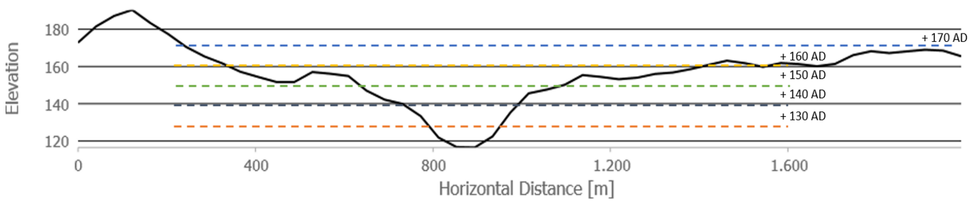


Figure 3.9: Elevation profile of cross section C

3.4.3 Reservoir configuration considerations

The other aspect that greatly influences the volume of the reservoir is the target water surface level. When a fixed freeboard is assumed, the surface level has a major impact on the character and performance of the reservoir. An optimization study is conducted to discuss the optimal surface level for the reservoir. In this study, the following pros and cons were distinguished when one would increase the target water level:

pros

- Increasing the storage volume leads to less fluctuation in water level, due to the dampening effect. As a result, the minimal water level is closer to the mean level, leading to a possibly higher position of the extraction pipe.
- The larger the lake volume, the less chance of depletion during its lifespan.
- The higher the surface level, the more hydro power can be generated. The client has indicated that a hydro power plant is highly desirable.
- Increase in water level leads to less sediment influence on the reservoir. The sediment is brought into the reservoir by the river and will settle on the bottom, with an increase in surface level and therefore volume, this effect will have less impact.
- Less fluctuation of the water surface. Less water would be spilled with the same crest height because the peak discharges have smaller impact on the surface-level.

cons

- With a higher surface level, the total area to be flooded, and therefore the loss of forest, increases.
- Higher surface area of the dam increases the losses due to open water evaporation and seepage.
- When assuming a fixed freeboard, the dam height increases proportionally with the average surface level of the water. This raises the costs of the dam construction, as well as the costs for maintenance.
- The higher dam height also influences the amount of locations where it can be build. The cross-section profile changes to a less high and wider profile more downstream, resulting in a more upstream location of the dam when increasing the dam height. This is associated with loss in storage volume of the reservoir.
- An increased surface level leads to an increased volume, resulting in a longer fill up time. Considering the investment costs, this has a negative impact on the payback period.

With these impacts in mind, the following surface levels have been evaluated on the above criteria.

3.4.4 Reservoir characteristics

For a certain water level the storage volume and surface area of the reservoir are determined by the topography of the catchment. SRTM elevation data is used with a spatial resolution of $90m$. This is sufficient in this stage. Table 3.4 and figure 3.10 present the relation between reservoir surface area and volume for several water surface levels.

They both show that for $110 < h_{water} < 140m$ +MSL the area of the reservoir increases, whereas the volume doesn't expand. Here the floodplain gets flooded. Then for $140 < h_{water} < 180m$+MSL, the surface area does not increase as much as the volume does. This represent a more vertical storing of water in the reservoir, instead of covering more area.
As mentioned before, for a reservoir it is preferred to store in depth, rather than surface area. So, from a hydrological point of view, the characteristics of the reservoir are used optimally with a water elevation greater than $150m$ +MSL and smaller than $180m$ +MSL.

Table 3.4: The volume of the reservoir and the surface area for several water levels in in the reservoir

Water elevation [m +MSL]	**Area [ha]**	**Volume** [$\cdot 10^6\ m^3$]
110	6	0,2
120	49	2
130	108	10
140	156	23
150	204	41
160	253	63
170	304	91
190	448	164

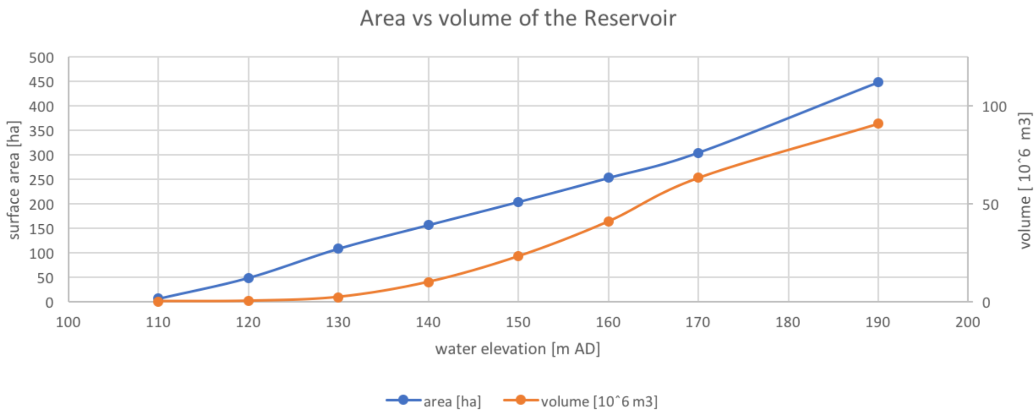


Figure 3.10: the relation between area and volume of the reservoir's topography

3.4.5 Reservoir yield calculations

The optimal reservoir yield is the yield that constantly can be abstracted without depleting the reservoir, even during dry El Niño years. Also, it should be able to handle peak rain events without spilling-over potential drinking water.
The discharge time series of the flow into the reservoir has been calculated previously with the SOBEK model. Now we can simulate the water level in the reservoir for this interval. The time series include 2 extensive El Niño events, but also plenty of heavy rain events. The following processes play a large role in determining the reservoir yield:

- inflowing discharge [$m3/s$]
- constant outlet yield [$m3/s$]
- evaporation rate [mm/day]
- seepage rate [mm/day]
- seepage and evaporation area [ha]
- dam height and spill overs (*neglected)
- rainfall directly on the reservoir [$m/year$]

The parameters and their ranges are summarized in table 3.5.

Table 3.5: Parameters used for the resereservoir yield calculation

Parameter	**Unit**	**Range**
Inflowing discharge	m^3/s	Long term average: 1,73
Constant outlet yield	m^3/s	1,75
Open water evaporation	mm/day	4,4
Seepage	mm/day	order 0,1 (further research!)
Target water level	m +MSL	140-180
Inlet level	m +MSL	110-175
Crest level dam	m +MSL	155-188
Operational pressure	m (above inlet tunnel)	2
Surface area	ha	204-376
Storage volume	$\cdot 10^6\ m^3$	41-127
Rainfall on reservoir	$m/year$	1500 – 3300

3.4.6 Reservoir yield result

The long-term average of the 19-year long time series of the simulated incoming discharge has been calculated at 1,73 m^3/s. The evaporation rate and estimated seepage are of the same order of the amount of rain that falls into the reservoir lake and therefore, they balance each other. Figure 4.12 gives an overview of the long term averages of the fluxes acting on the reservoir. As mentioned before, the assumption that no spill-over occurs has been made.

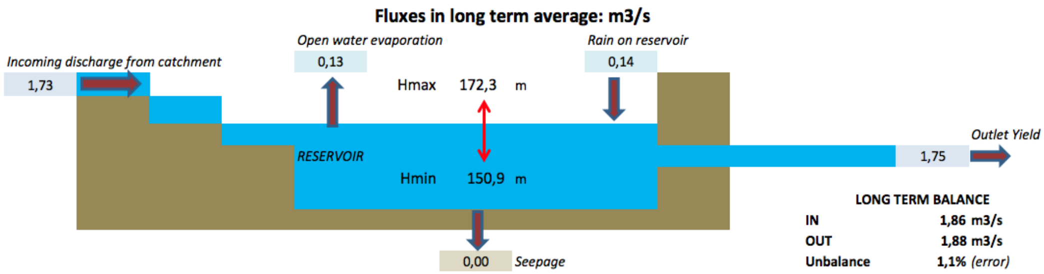


Figure 3.11: The long-term average of all the fluxes in the reservoir

The long-term water balance is one of the key factors in depicting the right extraction yield, the other key factor is the long-term trend of the incoming amount of water. A positive trend of the average simulated water level can be observed when the in- and outgoing fluxes including the extraction yields are simulated over the last 19 year period. Figure 3.12 gives the water elevation within the reservoir over 19 years. The initial water elevation within the reservoir is $h = 160m$ +MSL.
This positive trend has a longer timespan than the average cyclic duration of ENSO-events, thus the increasing amount of precipitation could possibly be linked to increasing temperature as a result climate change. As can be seen in figure 3.14, a neutral trend line is obtained if the extraction yield is equal to 1,78 m3/s.

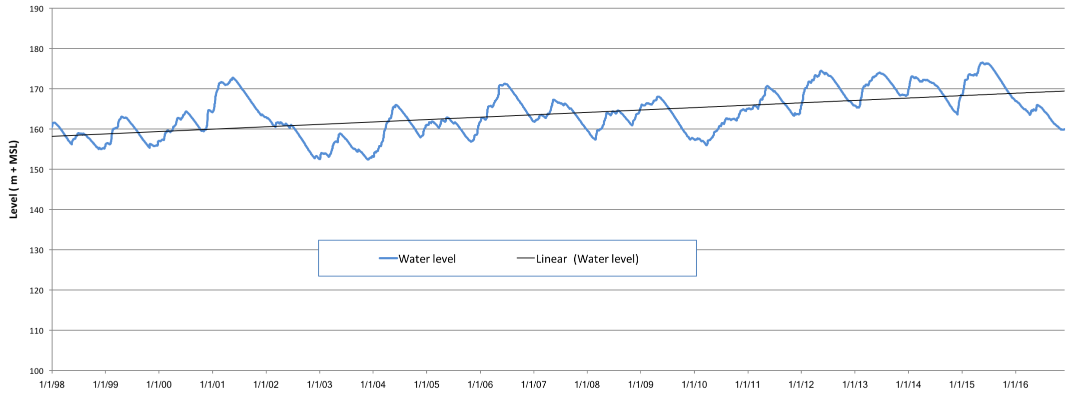


Figure 3.12: When the extraction yield is set equal to the long-term average incoming discharge of $Q = 1,73\ m^3/s$. a strong positive trend of the water level in the reservoir can be observed

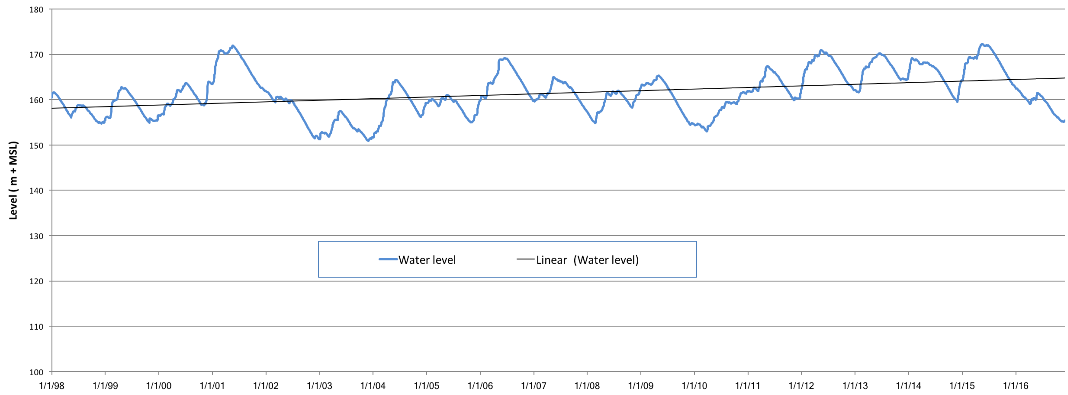


Figure 3.13: An extraction yield of $Q = 1,75\ m^3/s$ still gives a small positive trend of the water level in the reservoir.

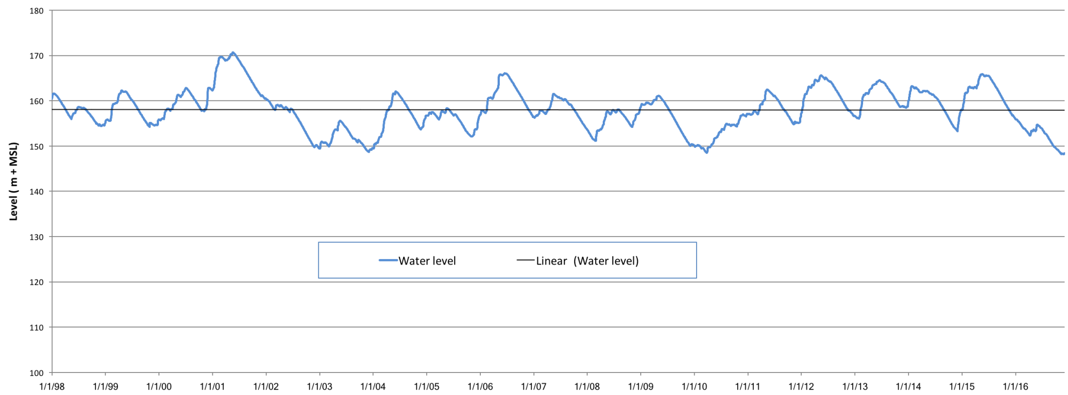


Figure 3.14: If the extraction yield is $Q = 1,78\ m^3/s$ the trend of the water level in the reservoir is horizontal

However, to fully correct to obtain a perfectly neutral trend of the water level in the reservoir is not advised. With a slight positive trend higher seepage rates, extended droughts and climate cyclic events with periods of tenths of years are allowed without hindering reliable operation of the reservoir. The extraction yield is equal to 1,75 m^3/s. Since the model results are not calibrated yet, it is fair to reduce the amount of significant numbers of extraction yield to 1,8 m^3/s.

3.4.7 Reservoir configurations

Reservoir simulations

The goal of the reservoir is to be able to abstract a constant yield of 1,8 m^3/s, without too much fluctuation of the water level or depleting the reservoir. Several simulations of the behaviour of the reservoir are done with the aim to identify the ideal target water level. The target level would be best described as the water level in the reservoir at which the abstraction starts. In figure 3.15 the results of the observed water levels in the reservoir for several target water elevations are presented. The simulation length is 19 years and the input into the (excel) calculations is the time series of the discharge as calculated by SOBEK model.

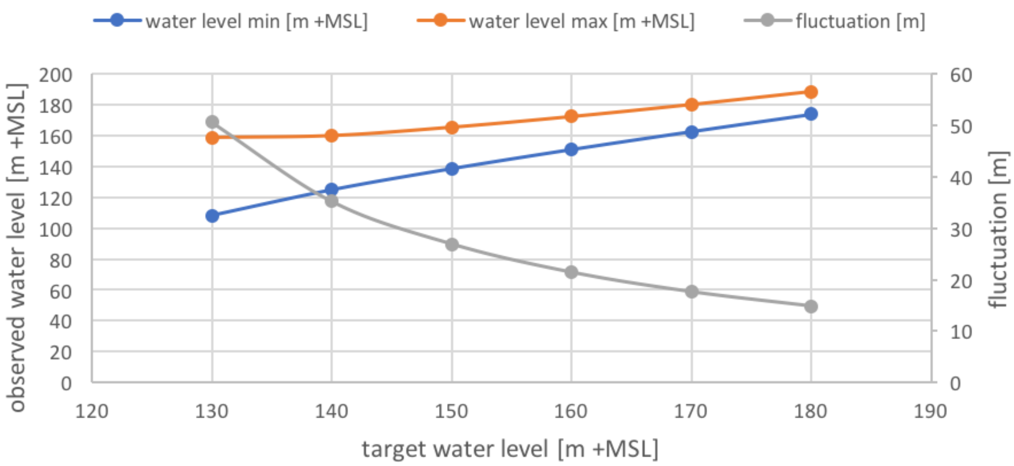


Figure 3.15: The observed minimum and maximum water levels for several design target water levels

The grey line represents the fluctuation of the water level as a function of the target water level. The blue line represents the minimum water level observed in the simulation. The orange line represents the maximum observed water level during the same simulation.
The overall decreasing trend of the grey line shows that for increasing target water levels the observed water levels fluctuate less. This can be explained by the fact that the surface area increases with increasing water surface level, thus more water can be stored in vertical sense.
From figure 3.15 and it is clear that water target levels lower than 150 m +MSL do not meet the requirements. The reservoir in this configuration does not have enough buffer to cope with the extended droughts during El Niño dry seasons. Target water surface level between 155 and 180 m +MSL seem well-favoured from a hydrological perspective.

Target level determination

The target level is the mean water level of the fluctuating water level of the storage reservoir. The hydrological boundary conditions have already been formulated. The determination of the target level does however need an interdisciplinary approach. To summarize the most important multidisciplinary arguments:
The higher the target level,

- the costlier the dam
- the more head available for hydro power
- the higher the absolute maximum water level
- the more area covered in the reservoir
- the higher the seepage rate
- the higher the evaporation rate

However, a qualitative comparison of these factors does not suffice. A quantitative approach has been chosen aiming to gain more insight in the comparison to distinguish the best option. The determination of the optimum target water level cannot be done without considering hydro power production and the costs of the dam construction. So, conclusions of sections that are going to be discussed later in the report, have been stated in this section.
The dam design choices, considerations, etc. are discussed in chapter 5 “Dam”. Chapter 9 “Financial Analysis” discloses the material and construction costs for various dam heights. These costs are based on the final design. Details and configuration of the hydro power plant can be found in chapter 7 “Hydro power and Water Treatment Plant Cluster”.

Table 3.6: Quantitative comparison of several design target water levels

Target water level [m +MSL]	**150**	**160**	**170**	**180**
water level min [m +MSL]	139	151	163	174
water level max [m +MSL]	165	172	180	189
fluctuation [m]	26,9	21,4	17,6	14,8
crest height estimation [m]	169	176	184	192
dam construction costs [million $]	$12,29	$15,88	$24,13	$30,07
available head (with dynamic losses) [m]	29	39	49	59
theoretical constant power production [kW]	423,2	569,1	715,0	861,0
energy [kwh/year]	4,36E+06	5,87E+06	7,37E+06	8,87E+06
gross revenue [million $/10 year]	$7,85	$10,56	$13,26	$15,97

From table 3.6 it can be derived that increasing the water target level will not result in a more cost-effective solution. Increasing the target water level higher than 160 m +MSL does not only lead to a higher dam, but also a wider and longer dam due to the topography and geometry. The costs of increasing the dam height are almost a factor 4 larger than the increasing gross revenue of the hydro power.
A target water level that is smaller than 160 m +MSL leads to larger fluctuations and smaller gross revenue that can be generated by hydro power production. The costs of increasing the height of the dam

for a target water level of 150m +MSL to 160 m +MSL is almost balanced by the amount of increased revenue of the hydro power energy generation.
Lowering the water target level below 160 m +MSL is thus not recommended, but increasing the target water level does not have added benefits other than an increased buffer volume with a high price tag. In this feasibility study the optimum target design water level is thus at 160 m +MSL from a financial, technical and hydrological perspective.

3.4.8 Fill-up time

After the construction phase is finished, the reservoir will fill up. This depends on the river discharge. As discussed earlier, the rivers' discharge fluctuates over the months because of the wet and dry season, but also over the years due to El Nino and La Nina climate events. The following parameters determine the total time:

- The inflow due to precipitation and river discharge
- The target water surface level
- The volume of the reservoir

All three parameters vary in the different scenarios. First of all, three different average river discharges are determined. A wet, an average and a dry year are considered. The wet and dry scenario are based upon the lowest, for the dry scenario, and the highest, for the wet scenario, yearly average of the modelled data. For the mid scenario, the long-term average of the modelled data is used. Here an overview:

- Scenario 1 dry with ($Q_{dry} = 0,91\ m^3/s$)
- Scenario 2 average with ($Q_{average} = 1,73\ m^3/s$)
- Scenario 3 wet with ($Q_{wet} = 2,35\ m^3/s$)

The fill up time for these three scenarios has been computed and stated in table 3.7.

Table 3.7: The fill-up time of the reservoir for a dry (scenario 1), average (scenario 2) or wet year (scenario 3)

target level [m +MSL]	**T1 [months]**	**T2 [months]**	**T3 [months]**	**T1 [year]**	**T2 [year]**	**T3 [year]**
160	26,4	13,8	10,2	2,2	1,1	0,9

Chapter 4

Geological Survey

This chapter describes the process of conducting a geological survey as part of the technical- and financial feasibility study of the Samarinda Water Project. The product of the survey is a geological map of the project area accompanied by three cross-sections that give more detailed information about the composition of the subsurface and the structural geology of the reservoir. Together they form a conceptual geological model of the project area. This model is a synthesis of all the information retrieved from a literature study and observations made in during three field expeditions.

The objective of the geological survey is twofold:

1. To determine the geotechnical characteristics of the subsurface as input for the technical feasibility assessment.
2. To determine the geotechnical risks and opportunities that may have an impact on the technical- and financial feasibility.

Acquiring understanding of the geology of the reservoir is an important part of analysing the technical feasibility of a project. Soil an rock conditions can impact constructions and water flows and need to be documented carefully. This will help to define the risks and opportunities of creating an artificial storage lake. The risks may require mitigating measures which can have an impact of the financial feasibility. Potential influencing factors for a successful outcome of the project are:

- The permeability of the subsurface – this will determine the amount of seepage from the potential storage lake;
- Slope stability – this determines if a location is suitable for construction and will determine the behaviour of the slopes within the storage lake;
- Faults – these may influence seepage from the storage lake;
- Piping – could induce subsurface erosion;
- Presence of construction material – this would lower the costs of construction.

The chapter starts off by introducing the expected sedimentary rocks in section 4.2. It also describes the lithological sequence that Witteveen+Bos have encountered in the Arsari Watershed project. The geological history of the area gives insight on the main structural features that have been active in this area and the composition of the subsurface. Section 4.1.2 describes the important historical events based on a literature study.

The first part of the report accounts for the general aspects of the project area and the second part is focussed on establishing the geotechnical aspects of constructing a dam and creating a reservoir. This part consists of three steps:

1. Making observations on rock characteristics and lithology during expeditions to the reservoir (section 4.3). The method that used is described in detail in Appendix B.3. The documented observations can be found in Appendix B.4.
2. Interpreting these observations by drawing cross-sections and creating a geological map of the area. Appendix B.1 contains the geological map with the expected structural features and surfacing formations. The cross-sections of Appendix B.2 give more detailed insights on the lithology of certain areas in the reservoir. The interpretations are discussed in section 4.4.
3. Finally, combining observations and interpretations to define geotechnical risks. These risks consider the construction of a dam and the filling of the reservoir. The risks concerning the reservoir are discussed in section 4.5.

The geotechnical aspects of the dam construction, a possible tunnel and the seismicity of the area are discussed in section 5.1.3. All geotechnical risks are registered in 11.

4.1 General geology

Knowledge about the general geology of the area will help to interpret observations made in the field. This section will discuss the geological history of the area.

4.1.1 Project location

The project is located to the North-West of Balikpapan, in East-Kalimantan. The area is highlighted on the general geological map of the area in figure 4.1. Based on this map it can be assumed that the there are two formations that are to be expected in the project area; the Pamaluan formation and the Palaubalang formation. Also, it shows the main folded nature of this area and the presence of faults.

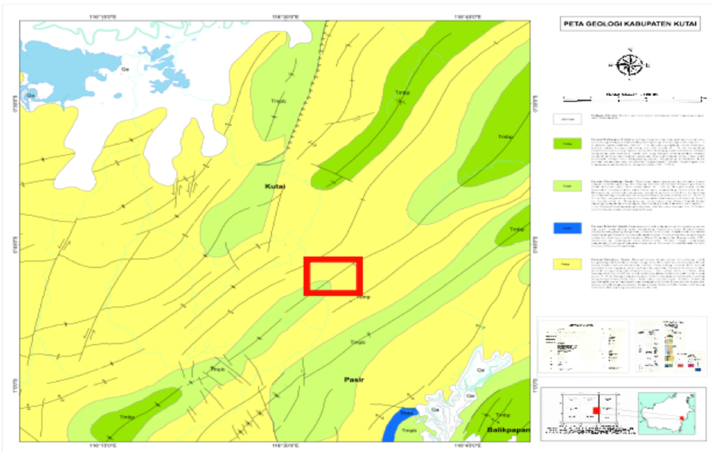


Figure 4.1: Geological map by Pertamina with phase 1 highlighted, provided by ITCI

4.1.2 Geological history

From the general geological map, it can be deduced that the geology in the project area consist of relatively young sedimentary rocks from the Tertiary (Oligocene and Miocene). The rocks were originally deposited in the Kutai basis and have undergone tectonic deformation, resulting in folded synclines and anticlines and major reverse faulting with minor associated faulting. The area consists of sedimentary rocks from the Pamaluan Formation and the Palaubalang Formation. Section 4.2.3 elaborates on the constituents of the formations. The lithology and geological features that have been observed in the phase 2A project area fit in this classification. Thus, the project area can be related to the geological history of the Kutai basin.

Rifting has led to the formation of the Kutai basin during the Eocene and Oligocene. The north-eastern part of the basin submerged rapidly and filled with marine deposits in a deltaic environment. These marine deposits are present everywhere throughout the project area, varying from thick beds of sandstones to shale.

Depending on the continental influx of sediment, the sea level and temperature, sediments with different properties have been deposited over time. Alternating climate conditions have resulted in alternating deposits of sandstone, siltstone, shale and limestone with varying thickness. The sedimentary layers which have been found in the Samarinda Water Project are up to 34 million years old (Oligocene) (de Vries, 2017). Sections 4.2.1 and 4.2.2 will explain in greater detail the characteristics and depositional environments of the sedimentary rocks that can be expected in this area.

After deposition in the Kutai basin, the sedimentary rocks have undergone diagenesis. “Diagenesis is the term used for all the changes that a sediment undergoes after deposition and before the transition to metamorphism” (Mohrig, 2004).

The Samarinda anticlinorium—Mahakam fold-belt is the most prominent geologic structure in the Kutai Basin. It consists of a series of NE-SW directed folds and faults in Miocene deltaic strata that parallel the modern coastline. This deformation happened during the Oligocene to Miocene transition. The compression has also formed reverse faults perpendicular to the direction of compression. Both brittle and ductile deformation has occurred in the Samarinda basin, pushing anticlines upward. These deformations are reverse faulting structures. This process is demonstrated in the schematic figure 4.5 of section 4.4.3.

Uplift and compression occurred during the same period, allowing rapid formation of the present landscape. Tropical climates are known for their fast weathering and erosion rates, contributing to this formation.

4.2 Rock formations

As is elaborated in section 4.1, the rock formations in the project area consist of sedimentary rocks. These are formed by the deposition and lithification of marine sediments in a deltaic environment. Alterations in depositional climates cause different types of sediment to be deposited. Section 4.2.1 describes the characteristics and the depositional environment of the clastic- and the non-clastic sedimentary rocks. The composition of the Pamaluan- and the Palaubalang formations are formulated in section 4.2.3 as they have been encountered by Witteveen+Bos in phase 1.

4.2.1 Clastic sedimentary rocks

Clastic sediments are gravels, sands, silts and clays. The three main sedimentary rocks they form in the project area are sandstone, siltstone and shale.

In the studies of phase 1 by Witteveen+Bos, they refer to a claystone. The clay-containing formations observed during the expeditions in the phase 2A were all fissile and shall thus be referred to as shale. There is no consensus on the difference between a claystone and shale, but in this report a shale is defined as a fissile sedimentary rock containing more than 50% material finer grained than 63 microns.

Sandstone

Sandstone is the most common sedimentary rock to be found in the project area. Quartz is the main component of the mineral content, but it can be accompanied by a wide variety of other minerals. This depends on the origin of the sediment and the path it has taken before deposition. Along this path, they have been subject to chemical and physical weathering. This has impact on the roundness of the grain; long transport result into collisions where sharp angles fail and lead to rounder grains. They are classified from angular to well-rounded grains. Another characteristic of a sandstone that is affected by transport is the sorting. Longer transportation results into better sorted sediments.

Sandstone is known to have a high porosity what enables it to act as an aquifer (layer containing water in the pore spaces) or a reservoir for carbohydrates. It is undesirable for the technical feasibility of the reservoir, if a sandstone formation acts as an aquifer. This will induce seepage from the reservoir. The high porosity makes it an unfit construction material for the dam, even though sandstone has a high strength.

Shale

Shale is a sedimentary rock that forms from the compaction of small particles known as clay. The most important characteristics of shale are that it is fissile and laminated. In a low energetic environment, little particles are enabled to settle and create a laminated structure. The plains along these laminations fail easily, causing shale to be fissile.

A valuable property of shale in the context of this project is its low permeability. Shale has a very small particle size, so the interstitial spaces are very small. Low permeability makes shale a useful bedding for a reservoir, because it acts as a seal. Construction wise, shale is an inconvenient material to build upon. The reason for this is that they can be subject to changes in volume and competence. In between the laminae of shale, water can be stored what can lead to expansion of the volume. When these expansive soils dry out, shrinkage will occur. These processes are very unfavourable for construction. Weathering can cause shale to be transformed into clay rich soils, with a low shear strength. If these layers are located on a slope, they can cause landslides. Construction will cause an extra overburden. This makes weathered layers more prone to slope failure.

Siltstone

Silt represents a group of granular particles with a uniform size range (diameter: 0,0039 mm – 0,0063 mm). The particles are deposited in a mid-energetic climate what causes both mud and sand to settle. These conditions are not very common and the rock units are usually thinner and less extensive than sandstone and shale.

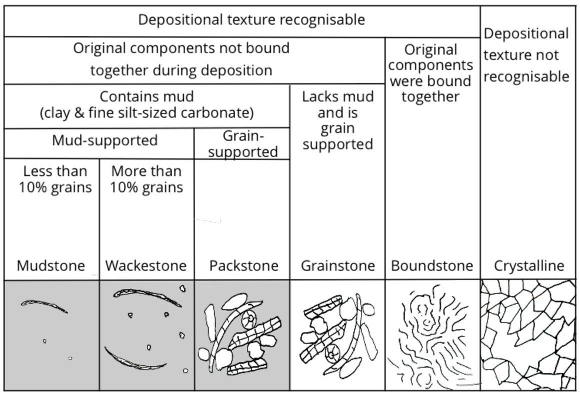
4.2.2 Non-clastic sedimentary rocks

When sedimentary rocks are formed from secretion of organisms, they are referred to as non-clastic sedimentary rocks.

Limestone

Calm, shallow and warm water in the Kutai Basin, combined with a low influx of sediments into the basin, enabled limestone to form. The sedimentary rock is composed primarily of calcium carbonate in the form of the mineral calcite. It is the product of an accumulation of organic material like shells. Classification for the limestone type is called the Dunham classification and it divides them into six main groups.

Figure 4.2 shows the characteristics that must defined to divide them into these categories. The energetic state of the depositional environment will determine the amount of fines taken up in the sediment.



Dunham (1962) Limestone Classification

Figure 4.2: Dunham classification for limestone (Dunham, 1962)

Limestone is a suitable construction material due to its immense strength. Mainly it is used in a crushed fashion, to build roadbeds as is the case in the project area. Throughout the Arsari concession, mines are found at the sites where limestone formations surface. It is also a valuable construction material for the proposed dam.

4.2.3 Formations

In the phase 1 research of Witteveen+Bos, the two prominent formations have been divided into sub-units. In the project area of phase 2A, structures have been encountered that fit this classification. It is assumed that not all sub-units have been observed due to the inaccessibility of the area and the lack of outcrops. The proposed lithology of figure 4.3 is used for the geological interpretation for the project area of the Samarinda watershed.

Formasi palaubalang (Tmpb)

Age: Middle to Late Miocene, about 14 to 7 million years old Thickness: about 900 m

This geological formation consists mainly of the following geotechnical sub-units:

- Tmpb1a: Firm, grey, massive (non-layered), poorly bedded, clayey LIMESTONE.
- Tmpb1b: Firm, grey, fine bedded clayey LIMESTONE including coral fossils and bedded layers with claystone erratic.
- Tmpb2a: Hard brown/grey, fine bedded SANDSTONE (non-calcareous);
- Tmpb2b: Hard, yellow/brown, massive bedded fine grained SANDSTONE (non-calcareous).
- Tmpb3: Medium to hard, very fine bedded, black/brown and red SHALE.
- Tmpb4: Soft to firm, brown and greyish orange to light greyish orange, silty fine to very fine grained sand to SILTSTONE with some fine laminations and sedimentary structures in the rock, carbonaceous laminations, fine to medium bedding planes and iron deposits along laminations, joints and faults.
- Tmpb4b: Weathered soft orange to brown thin bedded, very fine layered fine grained sand to SILTSTONE.
- Tmpb5a: Soft to firm, medium grey to medium olive greenish grey silty, sandy SHALE.
- Tmpb5b: Soft to firm, medium dark grey, medium olive green to light olive greenish grey, thick bedded, silty, slightly clayey, fine to very fine grained SANDSTONE.

Formasi pamaluan (tomp)

Age: Oligocene and Early- to Middle Miocene, about 34 to 14 million years old Thickness: about 1500-2500 m

This geological formation consists mainly of the following geotechnical sub-units:

- Tomp1: Hard to firm brown/black/red Fine bedded SANDSTONE, beds appear to be locally rich in iron.
- Tomp2: LIMESTONE (have not been observed in the field, interpretation is based on client reports of quarry 6000).

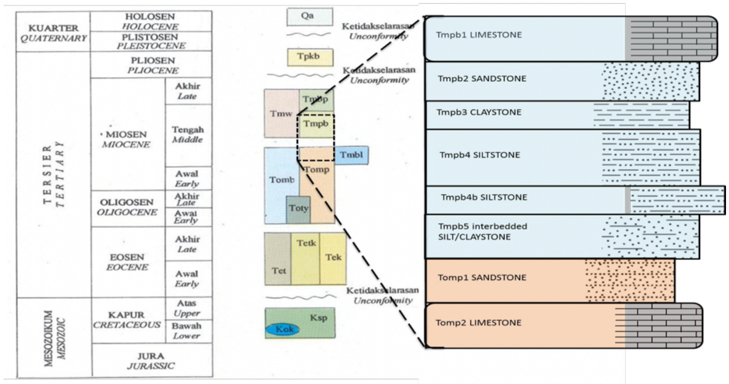


Figure 4.3: Main lithological units as described by Witteveen+Bos in phase 1 (de Vries, 2017)

4.3 Observations

The detailed description and classification of soil and rock are an essential part of the geologic interpretation process and the geotechnical information developed to support design and construction. The description and classification of soils and rocks include consideration of the physical characteristics and engineering properties of the material (Washington State Department of Transportation, 2006). Interpretations should not be included in the descriptions; these are expressed in section 4.4.

Appendix B.3 states the chosen rules and methods for the classification of soils and rocks. All the observed outcrops are described in detail in Appendix B.4. The observations have been made during three surveys in the field. Together with the rock classifications, the appendix also describes the objective of the fieldwork.

4.4 Interpretations

To gain insight on the geology of the project area, three field surveys have been executed. Each trip focussed on a different part of the reservoir. The combined knowledge has enabled us to define the geology of the entire basin of phase 2A and a part of the basin of phase 2B. The three expeditions are described in full detail in Appendix B.4. Here the objective, the expectations and the observations of each survey are described in full detail.

This section will explain the interpretation of the geology that has been based on the observations made during the field surveys and the general geology of the area. The product of the interpretation is a geological map of the project area, see Appendix B.1, accompanied by three cross-sections that uncover the geology of the subsurface, see Appendix B.2.

4.4.1 Cross-sections

The first cross-section (A - A') covers the whole project area, covering both potential storage lakes in a NW – SE orientation, see Appendix B.2.1. This direction was chosen because it is perpendicular to the fault plane and in line with the largest folding system in the area. Also, it gives insight in the geology of storage lake of phase 2B even though no actual surveys have been performed to this region. However, assumptions can be made based on the known geological structures and formations observed in the basin of phase 2A.

The second cross-section (B - B') shows another structural phenomenon; a folded area in the south-eastern part of the project area. This cross-section is also chosen in a direction in line with the dip of this secondary fold structure, see Appendix B.2.2.

The third and final cross-section (C - C') covers the dam-site and is chosen to be parallel to the future dam, see Appendix B.2.3. This is chosen to assess the geotechnical risks that present themselves when a dam is constructed at this location.

4.4.2 Folds

The area has undergone compression from different angles over time, what has led to multiple folds. From the general geological map of the area can be derived that there is a fold-axis between storage lake 2A and storage lake 2B. The axis represents the centre of a syncline structure. The project area of phase 2 is located on the Samarinda anticlinorium—Mahakam fold-belt, like phase 1. This is backed up by evidence of folding in a NW – SE direction.

This syncline structure will be referred to as the main syncline and is illustrated in more detail by the cross-section A – A'. No field surveys have been executed in storage lake B, but the general geological map shows that the orientation of the layers in the north-west of the storage lake is in the opposite direction of the layers in the south-east of storage lake A. The result of the syncline structure is that the relatively younger sub-units of the Palaubalang (Tmpb) formation can be found at the fold-axis. The younger layers of the formation consist mainly of sandstone and siltstone. These sub-units show more resistance to erosion and therefore contribute to the higher topography at the fold-axis. The rise is topography of the south-western side of the basin is created by the hard sandstone of the Pamaluan (Tomp) formation and the oldest parts of the Palaubalang formation.

Next to the main syncline structure, there is evidence of compression perpendicular to the NW – SE compression. This is revealed by a syncline structure, followed up by an anticline to the North-east. Cross-section B – B' shows the structure in more detail. Due to the main syncline structure in the project area, the fold-axis of this second folding is inclined, dipping to the Northwest. A tilted syncline results in V-shaped contacts between different layers. With a syncline, the opening of the V points into the direction of the plunge. An anticline shows the exact opposite; here the sharp end of the V will point in the direction of the plunge. This is known as the 'rule of V's' and is illustrated in figure 4.4.

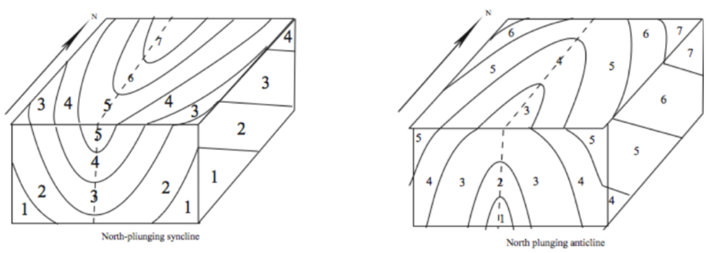


Figure 4.4: Box diagram of plunging anticline- and syncline structures (of New Orleans, 2008)

4.4.3 Faults

During the field surveys, no faults have been observed. However, there is strong evidence of a large reverse fault that cuts through the middle of the basin. Evidence is provided in the form of a lack of lithological succession and the presence of a large quantity of fractured rocks with a very steep dip angle.

An area that has been subject to compression is not only prone to folding, but can also result in faulting. Continuing pressure can cause formations to break when they are not capable of folding any further. Where anticlines and synclines dominate the subsurface, there is a high probability of reverse faults to be present. Alongside the fault plane, one of the sides is thrusted up the other side in a reverse fault. This type of fault is also referred to as a thrust fault and is clearly caused by compression. Figure 4.5 shows the deformation that is characteristic for a compressional region.

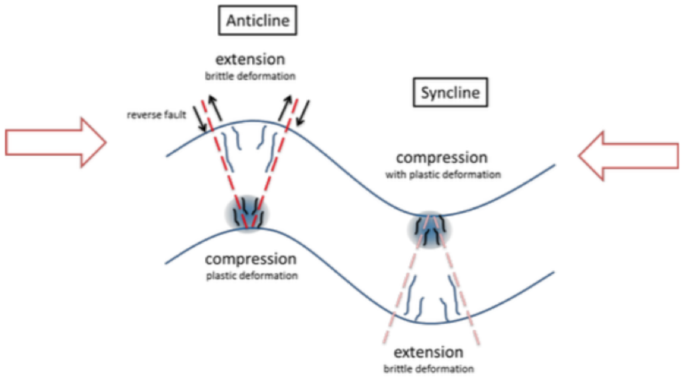


Figure 4.5: Schematic view of folds and reverse faults in a compressional environment (de Vries, 2017)

Extension and compression occur within both the anticline and syncline structures. Reverse faults only occur at the anticline resulting from the lower normal forces at shallow depths. Formations with a lower shear strength will fall subject to faulting first (de Vries, 2017).

The outline of the fault is expected to follow the same path as the main river body that cuts through the basin. The fault is assumed to have a large offset. The reverse fault is expected to be located at the site of dam A, probably perpendicular to the direction of the dam. This hypothesis is based on the depression in the ridge and the bended nature of the thrust fault, whereby erosion along the fault zone has likely generated the depression in the ridge at dam A. Future fieldwork is advised to locate real in-situ evidence of this fault. This discovery produces insight on the orientation of the fault and the actual offset.

4.5 Geotechnical risk assessment

The construction of a dam will create a storage lake from a naturally formed basin, enclosed by high ridges on all sides but north. The previous section discusses the geology and structural features of the basin. This section will discuss the geotechnical risks that are encountered when transforming the basin into a storage lake. The geotechnical aspects of the dam site, will be discussed in section 5.1.3.

The feasibility of the reservoir will depend mainly on the amount of seepage from the storage lake. Seepage is induced by a low permeability of layers covering the base and slopes. Also, the present of faults can play a part in this process. Weathering in the form of piping can contribute to seepage. Another aspect that must be considered is the stability of the slopes in the reservoir. Slope failure could lead to instability of constructions at the dam site.

4.5.1 Permeability

The largest part of the basin floor is expected to be dominated by soils with a high clay content: Tmpb3, Tmpb 4 and Tmpb 5a. This is very desirable because of the low vertical permeability of clay. The western ridge of the basin is expected to consist mainly of sandstone and shale. The sandstone layers could present a problem under certain undesirable circumstances. Permeable sandstones could act as a pathway for water flow. When it starts to act as an aquifer it could increase the seepage significantly.

The assumed fault that through the middle of the reservoir could induce seepage from the basin. The fractured character of the fault could function as a pathway for the flow of water. When water starts to flow through underground channels, there is a probability on the appearance of piping. Piping is described as a subsurface form of erosion which involves the removal of subsurface soils in pipe-like erosional channels to a free or escape exit. The soils that are susceptible to this internal erosion are usually comprised of silts and silty sands with a low clay content (Masannat, 1980). Within the basin there is not a high probability on the occurrence of piping, but it could be induced by the presence of the fault.

Limestone units are present in the project area (Tmpb1 and Tomp2), although they are not presumed to surface. Karst could be a risk if limestone is present because it results in connected cavities varying from ping pong ball sizes up to cave size, due to the dissolution of Calcium Carbonate in the limestone. Caves are present in the area surrounding the basin. It is not known whether the caves are in the limestone that is part of the Pamaluan or Paluabalang formation. If a karst system is present underneath the basin and dam, there is a risk of water flow from the reservoir. It is difficult to detect karst underneath the reservoir because the holes are easily missed by drilling or geophysical methods. Therefore it is needed to extend the geological knowledge of the area as much as possible (de Vries, 2017).

4.5.2 Slope stability

The ridges that surround the reservoir consist of an eventual steep slope. By filling the reservoir with water, the hydrostatic pressure regime will change. Furthermore, the presence of water can induce weathering, what causes the rock strength and discontinuity strength parameters to decrease. Without extensive research on the rock types and water quality it cannot be determined to what extent it alters the mechanical behaviour of a rock mass. These factors could induce slope failures that may impact the stability of the dam or could result in flooding of the downstream area.

4.5.3 Future site investigations

In the preliminary phase of the project, there are a lot of uncertainties concerning the geology and the geotechnical characteristics of the surface and the subsurface. Future site investigations should consider the following factors in order to establish the complete conceptual geological model.

- Soil cover and weathering profile.
- Geotechnical properties of the soil cover.
- Geotechnical properties of the rock mass units.
- Orientation of bedding planes and discontinuities of the rock mass units.
- Presence and character of faults.
- Permeability of the rock mass below the dam.
- Permeability of the rock mass around the dam.
- Ground water level.
- Geotechnical assessment of available geological construction material.

Chapter 5

Dam

The function of reservoirs is to store water and to release it subsequently in a controlled manner. This controlled release functions are either discharge control, water level control or a combination of both. To create a reservoir a river closure needs to be built, to function as dam.
In this study the focus is on the requirements of the dam requirements and its basic design. The (detailed) design should incorporate the local geotechnical conditions based on thorough site investigations. The specific purpose of this document is to:

- Describe the functional requirements of the dam.
- Describe the basic design of the dam and spillway.

5.1 Hydrological study and boundary conditions

The hydrological study and boundary conditions have been formulated in chapter 3. Here we will only recapitulate the results in order to determine the starting points of the dam design – an important step in the creation of the artificial storage lake.

5.1.1 Hydrology and hydraulics

Reservoir configurations

The design mean reservoir level is used to determine the dam crest level. From chapter 3 we have determined for which target water level the characteristics of the reservoir are optimally used: a target water surface elevation of 160m +MSL is selected with a maximum fluctuation of 21,4m and a maximum water level of 172,3m. The target level is the mean of the fluctuating water level, which is dependent on the rainfall, evaporation, planned extraction yield, position of the extraction pipe, etc. Based on the hydrological study a constant extraction yield of 1,8 m^3/s has been determined. Several hydrological simulations of the reservoir and catchment area with TRMM rainfall data of the past 19 years determined the water level (see Chapter 3 for the in-depth analysis). The minima and maxima will govern the crest height, spillway design height, as well as the level of the extraction pipe.

Wind

Wind data at the vicinity of the site is required to estimate the rise of water level due to wind and the wave run-up at the inner slope of the dam, see section 5.3.6. Two sets of wind data have been collected:

- Ground station ITCI, period: 4 Oct 2016 - 20 Dec 2016, can be found in Witteveen+Bos report. (maximum wind speed: 13,3 km/hr = equivalent to Beaufort scale 3).
- For Balikpapan region, period: 1984 – 2006 (Office, 2006) (maximum wind speed: Beaufort scale 7).

5.1.2 Topography

The client has provided ASTER GDEM (Global Digital Elevation Model) topography data. It has been produced as a joint operation of Japan (METI) and United States of America (NASA). The data has a resolution of 20 m with 95% confidence for vertical data and 30m with 95% resolution for horizontal data. The dam location and the elevation of the dam can be seen in figure D.1 and in figure D.2 in the Appendix.

5.1.3 Geotechnical risk assessment

The project area's topography forms a natural basin in which the storage lake is to be created. The most suitable place for a dam is the mouth of the basin where all the water flows to. The dip in topography in the northern part of the reservoir is caused by a fault system and erosion along this fault. In this section, the geotechnical aspects of that location will be discussed.

A thorough knowledge of the present rock and soil types, their characteristics and stratification is necessary for the design and execution of a dam construction. The first step in this process is to define the geological history and the general geology of the area. Then it is key to obtain geotechnical data of the construction site. Laboratory analysis of sample-borings will give insight on the quality of rocks and the lithology. In the field, the soil strength can be tested by the execution of a penetrometer-test (Verhagen, 2016). At the time of writing this report, the potential construction site is still only accessible by an eight-hour hike through extensively vegetated jungle. Such tests are only possible when the infrastructure improves and it is possible to access the location with equipment and in a shorter time. Still, it is possible to give a prediction on the technical feasibility of the construction, based on the general geology and the cross-section of this site.

Slope stability

The most important geotechnical characteristic of the subsurface is the bearing capacity. "Bearing capacity is the ability of soil to safely carry the pressure placed on the soil from any engineered structure without undergoing a shear failure with accompanying large settlements" ("Bearing Capacities of Soils.", 2004). The dam will be built along a depression in the northern ridge of the basin with steep slopes on either side of the construction. Additional surcharge of the dam can contribute to the instability of embankments on these slopes and could result in sliding. The soil characteristics and the angle of the slope are important parameters of the bearing capacity. Filling the basin with water can also affect the stability of the slopes. The presence of water can induce weathering, what causes the rock strength and discontinuity strength parameters to decrease. Without extensive research on the rock types and water quality it cannot be determined to what extent it alters the mechanical behaviour of a rock mass. Additionally, water pressures in a discontinuity reduce the normal stress on the discontinuity and there for reduce the shear strength along the discontinuity (Hack, 1998).

The other side to the presence of water is the effect that an alternating water level has on the extraction discharge. A stable slope at high water level can fail during periods of low water level due to the loss of buoyancy of the construction material. The western bank of construction site for the phase 2A dam, is a peak that consists of moderately strong sandstone at the top, followed by a relatively small layer of shale and then a strong silt formation at the base as can been seen in cross-section C - C' of Appendix B.2. The eastern bank is assumed to consist mainly of sandstone. This bank has been investigated swiftly, but no outcrops were discovered.

The area that could cause the highest risk to construction and thus requires the most attention, is the shale unit on the western bank. During the final field expedition, a shale layer has been observed with an orientation of 090/19. The dip direction is very different from the orientations that have been measured around this outcrop. One explanation for this dip direction could be that the outcrop was part of a failed slope. The general topography at this point (where this outcrop surfaced due to a runoff stream) had an orientation like the shale. The very weak shale layer is located on top of a more competent siltstone layer. For a lower risk to slope instabilities there are several design choices. One is to remove the very weak layer. Another option is to construct on top of the soft soil and press the embankment down, using its own weight. The latter will involve sliding. With an eye on safety it is recommendable to choose the first option. Nevertheless, the second method could also prove suitable, although there is a risk that the shale will be only partly pushed away and that deformation will continue in later stages (Verhagen, 2016). A third option may require that the intake structure be founded at a lower elevation in order to reach competent rock. Furthermore, the slopes are completely covered by vegetation at the time of writing this report. This contributes to the stability of the top soil and the resistance to erosion. Before construction will commence, all trees and plants will be removed and sold for extra revenue. This will affect the slope stability and should be considered in future measurements.

Squeezing

“Squeezing of rock is a time-dependent large deformation which occurs around the underground opening and is essentially associated with creep caused by exceeding a limiting shear stress. The magnitude of the convergences associated with squeezing, the rate of deformation and the extent of yielding zone around the opening depend on the geological conditions, the in-situ stress relative to the rock mass strength, the groundwater flow and pore pressure and rock mass properties” (Marence, 2003). The sedimentary rocks are alternating in strength, for instance in the form of layers of soft shale, siltstone and weak or weathered sandstone in between hard sandstone layers. Under unfavourable conditions and orientations, these softer layers with smaller strength and stiffness could result in squeezing, which should be accounted for in the design of dam and tunnel.

Piping

The assumed fault that is present underneath the dam could prove to be a risk for the technical feasibility. The fractured character of the fault could function as a pathway for the flow of water. If this is the case, water is transported past the dam through an underground pathway, created by the fault. The permeability of the subsurface at the dam will determine the loss of water due to seepage. When water starts to flow through underground channels, there is a probability on the appearance of piping. Piping is described as a subsurface form of erosion which involves the removal of subsurface soils in pipe-like erosional channels to a free or escape exit. The soils that are susceptible to this internal erosion are usually comprised of silts and silty sands with a low clay content. Other characteristics for these soils are a low dry density and a high void ratio (Masannat, 1980).

The foundation rock, directly underneath the dam, consists mainly of silt, but does have a high clay content. It is important to investigate this matter thoroughly, because piping can have disastrous consequences. Throughout history, there have been many situations where piping has been found to induce landslip failures (Jones, 2004). To investigate the leakage, it is required to investigate the permeability of the rocks and the flow pattern underneath this dam. No surfacing limestone formations and thus no evidence of karst has been found in the basin of storage lake A. It is presumed that the limestone layer of the Pamaluan formation does not surface due to the syncline structures in the basin. Areas dominated by synclines, principally have younger layers that surface. Karst is not seen as a probable risk for this construction site.

Seismic hazard and risk

The Indonesian archipelago is located on the conjunction of three major tectonic plates; the Indo-Australian plate, the Pacific plate and the Eurasian plate. Indonesia is at the collision point of these three crustal plates and is therefore subjected to high seismic activity. Relative to the rest of Indonesia, the seismic activity in Kalimantan is reasonably low. Still, the seismic hazard and risk should be considered in the design of the dam.

Seismic hazard and risk are two of the most commonly used terms in engineering design and policy considerations. The potential for dangerous, earthquake-related, natural phenomena such as ground shaking, fault rupture and soil liquefaction are considered seismic hazards. The probability of occurrence of the consequences is referred to as the seismic risk. A seismic hazard assessment should be executed to determine the three parameters that quantify the hazard: the level of severity, the spatial- and the temporal measurements. The means to perform such an assessment are instrumental, historical and geological observations (Wang, 2010). Performing a seismic hazard assessment lies out of the scope of this preliminary study, but should be considered to determine the technical feasibility in a later stage. However, at feasibility study stage, the basic design of the dams against earthquake with incorporating a crest width requirement according to SNI 8062-2015 (table 5.1) is considered sufficient.

5.2 Requirements and failure modes

For a safe and operational structure, numerous requirements must be met; in other words: the dam must be stable under all static and dynamic loading conditions.

5.2.1 Requirements

The requirements are closely correlated with the different failure modes. In general, these can be divided into two different categories. The serviceability limit state (SLS) and the ultimate limit state (ULS). For this dam design, the SLS focusses on the constant extraction discharge of water. If due to failure of some kind, the extraction is no longer possible, this is exceedance of the SLS. The ULS is associated mainly with safety of the construction and its nearby areas. Therefore, exceedance of the ULS is failure of the structures' stability and can be caused by collapse or significant damage with dangerous effects on the nearby areas.

5.2.2 Safety

In terms of safety, three different failure modes are considered for the dam design (CIRIA, 2007).

Structural failure

The dam must remain stable under both static and dynamic loads. If one of these loads causes failure of stability in one or multiple components of the dam, this is considered to be structural failure. More than often this can be linked to filling material or foundation material of poor strength. Also, the wrong determination of the characteristic loads can lead to structural failure. In general, the dam construction must be stable and function properly during construction and operational period, including natural hazard events.

Geotechnical failure

The quality and type of the fill material determine the slope inclination of the dam, such that the dam is stable. The dam cover material must be resistant to water flow. Upstream as a result of waves, downstream mainly as a result from overtopping and surface runoff. Looking at the failure of soil, two types of problems can occur: a strength problem and/or a stiffness problem. To prevent failure, the important soil parameters must be determined and considered throughout the design process. Especially in earthquake prone locations it is key to prevent geotechnical instability by selecting proper material.

Hydraulic failure

Hydraulic failure can be categorised as follows: seepage within the embankment (piping), seepage within the foundation layer and wave overtopping. Seepage in or through the foundation layer can lead to a leaking reservoir, as well as structural instability of the dam. A thorough analysis of the foundation and subsurface condition must be performed to counter act these processes. The last type of hydraulic failure is the wave overtopping. Overtopping of the dam may lead to failure of the dam resulting in serious damage and safety issues to the structure and its nearby areas.

5.2.3 Failure modes

From the types of failure discussed in the previous section, a list of critical failure modes is compiled. For a safe structure the following aspect need to be taken into account (van den Bos, 2017):

- Armour stability, front face
- Toe berm stability
- Armour stability, rear face
- Sea bed scour
- Wave overtopping
- Filter and under layer stability
- Core layer stability
- Sliding
- Piping
- Slipping
- Subsoil settlement

Piping, sliding, slipping and subsoil settlement have been discussed in the previous chapter. The other failure modes are discussed in section D.2.

5.3 Design calculations

As discussed in the previous section, the design must meet a number of requirements regarding construction, operation, stability and safety. In the section D.3 in the Appendix, the different design aspects that determine the main parameters of the final dam design are discussed. The upcoming section elaborates on these considerations.

5.3.1 Dam type

A rock-fill dam is selected as the dam type. This selection is due to the topography of the site which lacks a narrow v-shaped rock valley (for supporting the abutment of a concrete dam). The relatively long dam alignment results in high costs for concrete and makes the choice of rock-fill dam preferred.

Another reason to select hard rock as the main fill material is the availability of by the client owned rock quarries around the area and the stable foundation layer. This consideration will reduce the construction costs significantly. The fill material and the complementary component of a rock dam (filter, transition zone, etc.) should be designed in compliance with the relevant codes and standards.

The considerations for this design are:

- An earth dam is vulnerable in terms of erosion, which can conflict with the integrity of the dam structure.
- A rock dam can achieve a higher stability, so it can be constructed with steep slopes.
- It is economical, due to the availability of rock from a nearby quarry.

5.3.2 Impermeable membrane

The impermeable membrane for the design will be a vertical core in the centre made of clay. The considerations for the positioning of the impermeable membrane in the centre over the upstream slope are:

- It is preferred to use the materials of the nearby area. An upstream slope membrane should use either asphalt, concrete, or steel as the construction material and the materials of a membrane positioned in the centre of the dam are soil materials such as clay.
- It has indirect contact with water, it is protected against weathering and other exposure effect and it can easily adapt against a less favourable foundation layer.

The other consideration is that the impermeable core near the centre of the dam is a vertical core, due to the advantages of:

- Leakage prevention
- Greater stability under earthquake loading
- Better access for remedial seepage control

The greater stability is the decisive factor in this case.

5.3.3 Granular and geotextile filters

The granular geometrically open filter is selected for the design. The material is selected based on its feasibility and economical aspects. A disadvantage of a granular geometrically tight filter is that an uneconomically large number of filter layers are often required and a disadvantage of geotextile layer is that the installation needs to be done with care. It will be hard to do the installation of the geotextile in the middle of the jungle. With these aspects in mind, the granular geometrically open filter is chosen. However, an open filter requires more detailed knowledge of the hydraulic loads on the filters, caused by the water movement along and inside the structure is required. This should be further investigated in the detailed design.

5.3.4 Transition zones

One should keep in mind that embankment zoning is also established for economic reasons, according to the availability of materials. The transition zones should be sufficiently wide, to ensure that they are continuous and constructed with minimal contamination at the outer part of the zone. The width of the grading of the transition zones should be limited to avoid segregation of materials during placement.

Due to the filter criteria and the grading limit, the diameter of the particles need to match with under- and above laying layers to prevent washing out. Therefore, two transition zones are selected. The first filter will consist of gravel, which will be a wide zone due to economic reasons (It is cheaper than sand). The second layer will consist of sand. This filter will contain a thickness of 1,5m.

5.3.5 Slope cover material

Slope protection is needed because the slope is steeper than 1:15. For both the upstream and the downstream slope, quarry-run stones are used as materials. The reason being that the quarry nearby cannot deliver sufficient quality for the rip-rap stones. An advantage from quarry-run stone compared to rip-rap is that its gravel- and sand-size components serve as a filter. As long as the crest of the structure is high enough to prevent overtopping, the armour units on the crest and the rear slope can be much smaller than the armour layer on the front slope. A vegetative cover for the downstream slope is not desirable due to the maximum slope steepness of 1:3.

Due to the relative low wave shoaling and breaking, as well as slope stability during reservoir drawdown, a slope inclination of 1:1,75 (V:H) is selected for the upstream slope with a two-layer quarry stones. For the downstream slope, a steepness of 1:1,5 is selected, consisting of two-layer quarry stone as well.

Stability

Upstream side
The slope-stability is calculated with the van der Meer formula; as the Hudson formula criteria are not met. The calculation can be found in section D.4 in the Appendix. A conservative thickness of 0,60m of the layer is used for the design with a D_50 of 0,32m. A thinner thickness of the layer can be sufficient but this aspect needs to be further optimized during the detailed design stage.

Downstream side
Due to the zero overtopping criteria, the armour layer at the downstream side can have a smaller D_{50}. The D_{50} is selected with 0,2m and a layer thickness of 0,4m.

5.3.6 Crest height/width

At this stage estimating the total settlement of the foundation and the dam during construction and operating phase is difficult since no really good site investigation has been performed yet. The design crest level in this stage will be determined by considering the maximum water level, a rough estimation of the settlement, wind set-up and wave run-up.

From the wind set-up and wave run-up aspects, a preliminary calculation has been performed by using the maximum wind speed (most conservative condition, Beaufort scale 7) as the input data is taken from section 5.1.1.

Total crest height

The crest height is calculated with maximum water level, which is 172,3m +MSL, the wave run-up (0,7m) and the wind set-up (0,005m), see section D.4.2 in the Appendix. The average water level is 160m +MSL leading to a total crest height of:

$$Crestheight = (H_{max} - H_{average} + H_{wave,run_up} + H_{wind,set-up} = (172,3 - 160) + 0,7 + 0,005 = 13,01m$$

At this stage, it is considered sufficient to take a crest height of 16m, taking the foundation and dam settlement into account. This is a conservative number and further optimization can be performed during the detailed design. Further measures against overtopping by constructing a spillway are discussed in section 5.3.10.

- The crest level is at 160+16=176m +MSL
- The minimum bottom profile of the dam is at 120m +MSL
- The crest height is 16m
- The maximum dam height is 176-120 = 56m

Regarding the seismic criteria, as explained in section 5.1.3, the recommended freeboard distance can be used as verification (at this feasibility study stage) whether the determined crest height is sufficient to withstand earthquake action. It is verified that the determined crest height in combination with the considered freeboard, the crest level is already sufficient for seismic criteria see table 5.1.

Crest width

According to Indonesian Standard SNI 8062-2015, the selection of dam crest width for high seismic area can consider a recommended value as given in table 5.1. A crest width of 9m is selected due to the crest height which is between than 31-60 meters. The crest of the dam will be accessible by cars for maintenance during its lifetime. This value is in line with the seismic criteria. The selected width is conservative and can be decreased in the detailed design.

Table 5.1: Crest height/width in line with the seismic criteria

Dam height [m]	Minimum distance between the mean reservoir level and the crest level [m]	Dam crest width [m]
<30	3.5	7.5
31 - 60	4.5	9.0
61 - 90	6.0	10.5
>91	6.0	12.5

5.3.7 Dam foundation

It is expected that hard rock is widely present at the site, thus the dam foundation will be constructed with rock material. A clay cover (topsoil) is present at the left slope surface located on top of a more competent siltstone layer, which has a thickness of a couple of meters. For a lower risk to slope instabilities there are two design choices; either to remove the soft layer or to construct on top of the soft soil and press the embankment down, using its own weight (Verhagen, 2016). With an eye on safety it is recommendable to choose the first option, see section 5.1.3. Before construction will commence, all trees and plants will be removed. This will affect the slope stability and should be considered in future measurements.

5.3.8 Seepage

As discussed in the previous section, there are three general methods to counteract seepage underneath the dam. Considering the rock layer, that was mentioned before, the cut-off trench seems a viable option. At the top clay layer a trench is to be excavated up to the rock layer, were the impermeable foundation is built on. However, looking at the full width of the cross section were the dam is to be built, it shows that the rock layer is highly irregular and jointed. During field studies in the design phase it will be hard to ensure the impermeability of this method. So probably additional jet-grouting is needed during construction. Further investigation on site is needed during the land preparation stage. When the trench is fully excavated, it can be concluded if the additional jet-grouting is needed.

5.3.9 Outlet works

Intake

The intake from the outlet work in the reservoir needs to be high enough to prevent interference from the sediment deposits, but at the same time, low enough to permit either a partial or a complete drawdown below the top of the inactive storage. The intake requires to have an operational pressure from at least 2m water level above the inlet to ‘push’ the water through the tunnel, as been said in Witteveen+Bos report. The intake cannot be at the bottom of the reservoir due to the sedimentation. The minimum water level is 150,90m +MSL, which means that the intake is located at around 145m+ MSL, so that there will be always enough pressure. The intake location is as high as possible due to a shorter tunnel length underneath the dam. The tunnel length underneath the dam will be around 150m. The design for the intake structure needs to be evaluated in the next design phase, as currently not enough specific data for this subject is available.

Pipeline

As can be found in chapter 4, the underground at the dam location is probably a siltstone layer. This siltstone is suitable for a tunnel. Further investigation on site is needed during the land preparation stage to confirm the presence of a siltstone layer. The site is a narrow canyon with steep slopes. Therefore, due to geological and economic reasons, a tunnel type outlet is selected, which is considered the safer and more durable option. Because it is less likely that failure of some portion of a tunnel would cause failure of the dam itself.

The forces that have to be taken into account for the stability and stress analysis are dead load, hydrostatic, uplift, earth and silt, temperature, earthquake forces, wind pressure, ice pressures, debris and trash, wave pressure and operation and maintenance load (Corps, 2003).

The construction method, which is probably the most suitable, is mechanical excavation. This option is considered fast, flexible and doesn’t require much specialist equipment and personnel. If sections of harder rock (harder than 60 to 80 MPa) are encountered, mechanical excavation can easily be combined with drill & blast excavation (Skawina, 2013). Further investigation on this topic is required, as much of the assumptions need to be verified in the field.

A tunnel underneath the foundation of the embankment, through the rocky layer, is considered to be the most viable option. This due to the both the geological conditions, as the local topography. The tunnel is drilled directly underneath the eastern slope of the dam, as the presence of siltstone is expected. Whereas underneath the western slope, a clay layer is expected.

There are a lot of factors that can cause difficulty to the process of constructing a tunnel underneath the dam. Still, these can be avoided by digging through a layer that is closest to containing isotropic characteristics and shows the least amount of variability throughout the layer. The feasibility of tunnel construction can be negatively influenced by the following factors:

- The sedimentary environment in which the sediments were deposited in prehistoric times, a former marine delta, allowed the occasional development of subsea slumps, disrupting the regular depositional sequence. Slump deposits are a risk to the tunnelling operations, since they can cause sudden changes in geotechnical conditions.
- The presence of faults zones where the rocks have been extensively fractured may provide leakage under high water pressure, resulting in piping and instability in the tunnel.
- The sedimentary rocks are generally inclined in different directions and contain different slope angles, due to the structural deformations. The inclined beds could prove unfavourable to the stability of the tunnel.

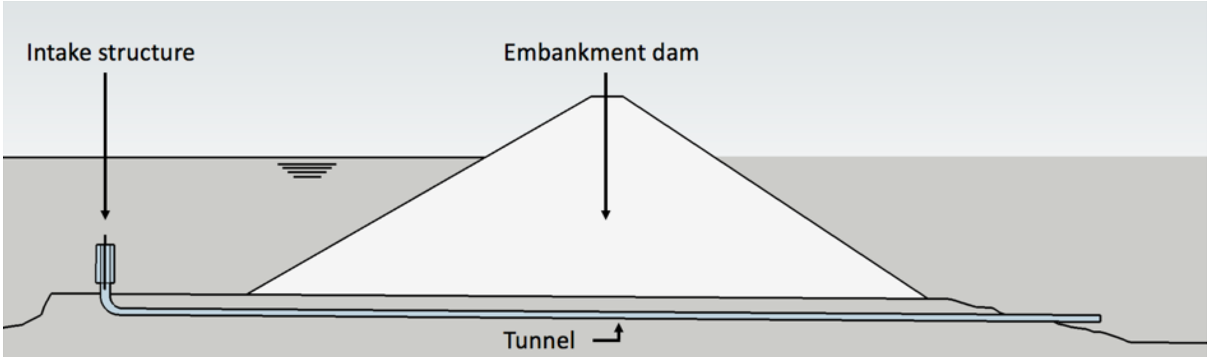


Figure 5.1: Tunnel location

Functional requirements of tunnel:

- The system should convey 60 million m3/year water
- Minimum diameter of the pipeline is 1,3 m (see Chapter 6)
- Life span of 50 years
- Absorption of all stresses from deformations, external (rock) loads and internal loads, including water hammer and pressure fluctuations from hydro (power) operation

5.3.10 Spillway

Flood parameters

The spillway is designed as an emergency spillway, with the sole purpose of regulating flood inflows to minimize the chance of overtopping. It is intended for use under extreme conditions, leading to increase of the water level. As it depends on both the water level and the inflowing discharge from the watershed, two different return periods hold. Firstly, the capacity is defined for the probable maximum flood (PMF) that can occur. As the reservoir is only connected to the watershed, so depends on the rainfall, the extreme discharge analysis is used. With a return period of 1000 years, the maximum discharge the spillway must carry is 83,4 m3/s, as this discharge is the maximum inflow in the reservoir for this return period. Secondly the intake level is determined. The main parameters to take into account are the maximum water level Hmax (172,3m +MSL) and the crest height (176m +MSL). The water level may never reach the height of the crest, so an intake level at approximately 173m +MSL is suitable. As the Hmax was modelled after the discharge from the past 19 years, it is expected that the spillway operates every 20 to 25 years. This return period is desirable, because the water flowing out of the spillway is not treated and cannot be used for drinking water or hydro power (Petras Punys & Šilinis, 2015), so is therefore 'wasted'. The discharge at outflow is hard to determine, because of the time-lag between the rainfall and the discharge in the river, as well as a time-lag between the inflow from the river and the water level rise. Indicating that an outflow of the PMF discharge of 83,4 m3/s is unlikely to happen, which makes it a conservative approach. However, this conservative approach is justifiable because of the potential connection to a nearby water reservoir. Discussed with the client, potentially another reservoir is built in the nearby future. This nearby reservoir will have spillway outflow to the watershed connected to this studies' reservoir. As no definite dimensions and characteristics of this outflow are known, it is to be accounted for in the next design phases.

Location of the spillway

As mentioned in the previous section, there were three options for locating the spillway shown in figure 5.2.

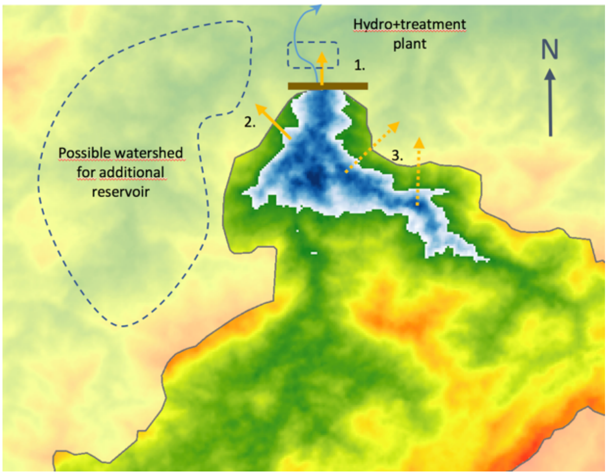


Figure 5.2: Location spillway

A spillway location at the dam site (1), an uncontrolled structure connecting the reservoir through the local topography with a nearby watershed using a short tunnel and pipes (2) or a completely free choice of location that is controlled and connected to an outflow area by means of long tunneling and pipes (3). The last option is very complex and other reasonable alternatives are available. Considering the second option, figure 5.2 shows that it flows into a nearby watershed. The client indicated that at the moment this location (phase 2B) is being examined if it is suitable to serve as an additional clean water reservoir. The topography shows that its size and characteristics are similar to the Samarinda watershed, so supposedly its storage volume and dam dimensions are similar as well. Combined with the fact that the PMF is solemnly depending on the local rainfall in the watershed so the PMF will happen simultaneously at nearby reservoirs, outflow into one of these reservoirs is not a viable solution. With only the dam location left, a spillway integrated with the embankment is chosen. At this location, the presence of hydro power and treatment plant has to be accounted for (Petras Punys & Šilinis, 2015), as well as the boundary of the Arsari territory.

The spillway is designed as a chute spillway with a steep sloped side channel, where the flow, after passing a weir or ogee crest, is carried away by a rectangular concrete channel. This type of spillway is best suited for a non-rigid dam (Tandon, 2014). The dimensions are determined based on the PME discharge of 83,4 m^3/s using the following formula (of Agriculture Soil Conservation Service, 1985):

$$Q = 3,1 * W * [H + \frac{{v_a}^2}{2g}]^{3/2} = 3,1 * He^{3/2}$$

Where:

- Q = discharge of inlet [m^3/s]
- W = width of the chute or inlet [m]
- H = depth of flow over the crest (or floor) of the inlet [m]
- v_a= mean velocity of approach at which the depth H is measured [m/s]
- g = 9,81 m/s^2

With the inlet at 173m +MSL, a freeboard of 3 meter can be used for the spillway. As this could lead to overtopping, the spillway is dimensioned with a maxim depth of flow over the crest of the inlet of 2 meter. With the spillway constructed in the center of the dam, so that the channel can follow the slope of the dam, the width is constrained. A more detailed dam design is needed for a more specific calculation. For now, a width of 10 meter is assumed to be realistic and will fit with the dam. The velocity is an important factor for service and auxiliary spillways, as they are operated more regular and should therefore be resistant to erosion. One method to counteract this process, is decreasing the velocity and thereby the potential energy of the outflow. The maximum velocity for these types of spillway is therefore lower than the critical velocity for an emergency spillway. The critical velocity for an emergency spillway is calculated to be at 3 m/s. For higher velocities, energy dissipaters should be constructed, as the high velocities can cause serious damage to the toe of the dam and nearby structures because emergency spillways are the least robust and erosion-resistant. Again for safety, a velocity of 2 m/s is used for calculating the outflow discharge. Using the formula above, a discharge of 101 m^3/s is found. This is well above the PME discharge of 83 m^3/s, so the spillway suffices.

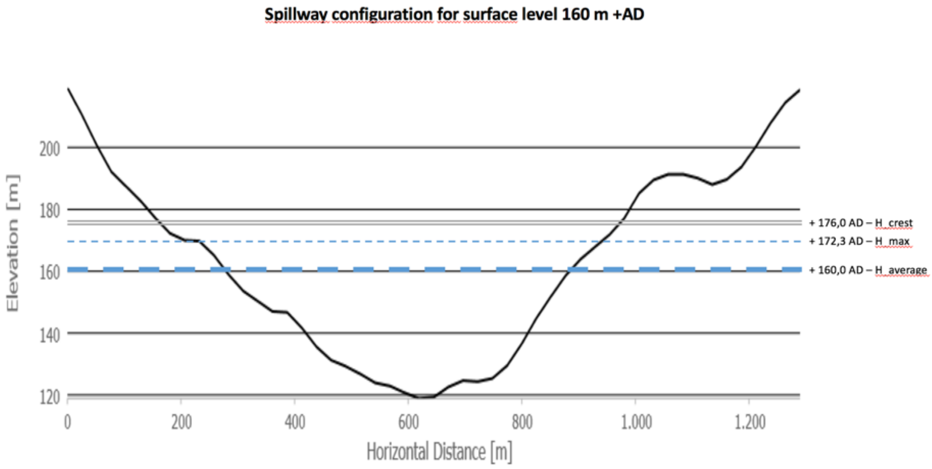


Figure 5.3: Cross section of the dam with characteristic spillway water levels for surface level 160 m +MSL

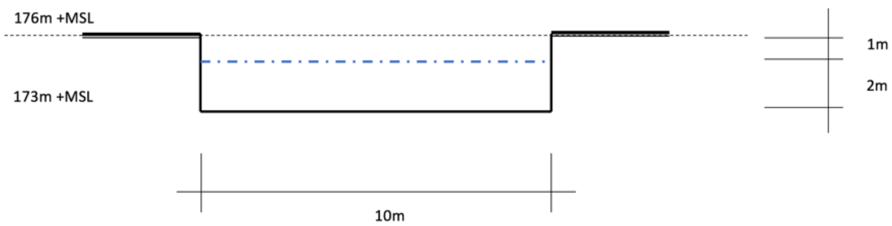


Figure 5.4: Spillway configuration for surface level 160 m +MSL

5.4 Design

5.4.1 Storage Lake

- Surface area of 253 Ha
- Absolute volume of $41 \cdot 10^6\ m^3$
- Average water level of 160 m +MSL
- Maximum water level of 172 m +MSL
- Minimum water level of 151 m +MSL
- Minimum level bottom profile of 120 m +MSL
- Fluctuation between H_{max} and $H_{min}$of 21,4 m
- Outlet yield distraction of $1,8\ m^3/s$
- Fill-up time of 0,9 – 2,2 years

5.4.2 Dam

The final design for the preliminary phase is given in figure 5.5.

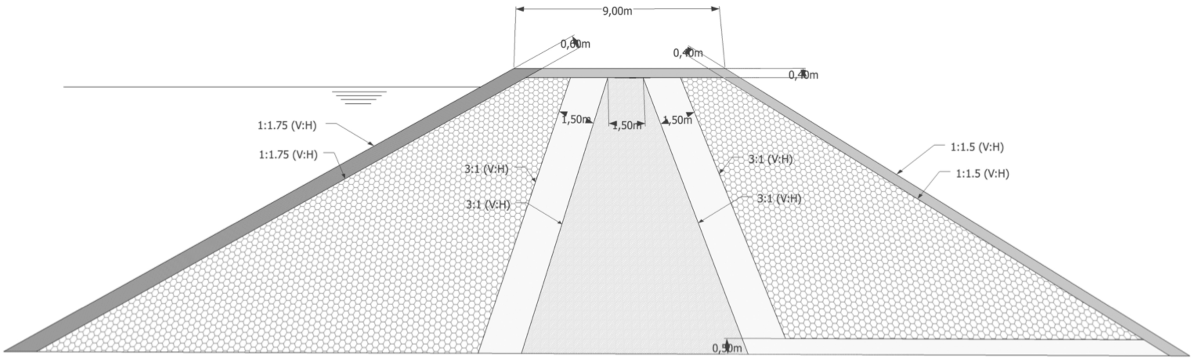


Figure 5.5: Design of the dam of the preliminary phase

In table 5.2, the dimensions of the dam can be found and in table 5.3, together with the considerations for the design decisions.

Table 5.2: Overview of the most important dam dimensions

Core layer	Material type	Clay	
	Slope	3:1 (V:H)	[-]
	Layer thickness	1,5	[m]
Underlayer 1	Material type	Gravel	
	Slope upstream side	1:1,75 (V:H)	[-]
	Slope downstream side	01:01,5	[-]
	Slope core side	3:1 (V:H)	[-]
Underlayer 2	Material type	Sand	
	Slope	3:1 (V:H)	[-]
	Layer thickness	1,5	[m]
Armour layer, upstream side	Material type	Quarry run	
	D_{50}	0,32	[m]
	Weight D_{50}	44,61	[kg]
	Layer thickness	0,6	[m]
	Slope	1:1,75 (V:H)	[-]
Armour layer, downstream side	Material type	Quarry rock	
	D_{50}	0,2	[m]
	Weight D_{50}	10,89	[kg]
	Layer thickness	0,4	[m]
	Slope	01:01,5	[-]
Crest	Material type	Quarry rock	
	$D_{n,50}$	0,2	[m]
	Weight D_{50}	10,89	[kg]
	Layer thickness	0,4	[m]
Tunnel Outlet	Length	150	[m]
	Diameter	1,3	[m]

Table 5.3: Overview of the design choices

		Design choice
Dam type		Rock-fill dam
Impermeable membrane	*Location* *Inclined/vertical*	Centre of dam Vertical
Granular and Geotextile filters		Granular geometrically open filter
Transition zones		Two transition zones of gravel and sand
Slope cover material/armour layer	*Upstream slope* *Downstream slope*	Quarry-run Quarry stones
Maximum crest height		56m
Crest width		9m

5.4.3 Spillway

- The spillway is designed as an emergency spillway, serving the sole purpose of flood discharge control.
- Spillway is constructed at the dam sight, as other options did not suit the boundary conditions.
- Maximum discharge capacity is 83 m^3/s with a return period of 1000 years, which is defined as the probable maximum flood.
- Intake level is at 173m +MSL, which is 3 meter below crest height. Considering the modelled Hmax (172 m +MSL) from a data set covering 19 years, the spillways operating return period is approximately 20-25 years.
- Chute spillway with concrete side channel, with a total width of 10 meters and a maximum depth of flow of 2 meters. Constructed with an ogee curve, together with the maximum depth and width, leading to a velocity below the critical velocity, therefore no energy dissipaters needed.

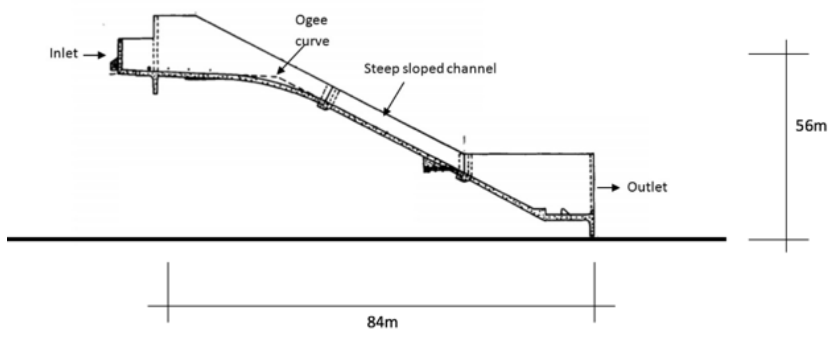


Figure 5.6: Spillway

Chapter 6

Water Transportation

6.1 Functional requirements

The main functional requirement for the water transportation system is transporting water from the reservoir to the PDAM of Tenggarong and Samarinda in a safe and reliable manner. Figure 6.1 shows a schematisation of the situation.

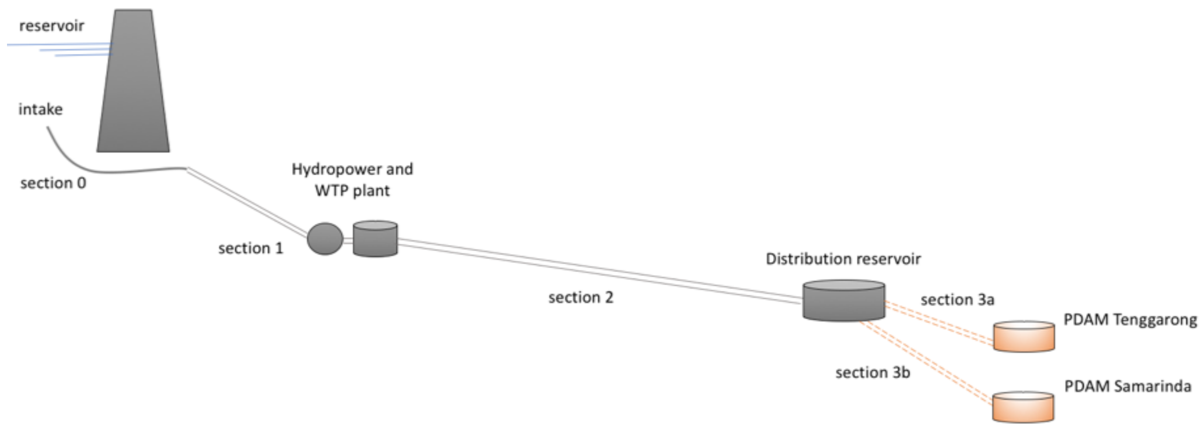


Figure 6.1: A schematisation of the transport of the water from the reservoir to the PDAM's of Samarinda and Tenggarong

Section 0
The requirement of this transmission tunnel is to extract the water from the reservoir and transport it under the dam to the pipelines of section 1.

Section 1
The function of these two pipelines in this section is to transport the water from the outlet tunnel work and to transport it to the hydro power plant. Hereafter it will be treated in the water treatment plant (WTP).

Section 2
In this section two pipelines will transport the treated clean water from the WTP plant to the distribution reservoir, situated near Tenggarong and Samarinda.

Section 3a
The function of these pipelines is to transport the clean water from the distribution reservoir to the PDAM of Tenggarong. The client states that the PDAM of Tenggarong will have the responsibility of the construction of these offtake pipelines. However, in order to optimise the total project, these trajectories need to be accounted for in an early stage.

Section 3b
The function of these pipelines is to transport the clean water from the distribution reservoir to the PDAM of Tenggarong. The client states that the PDAM of Samarinda will have the responsibility of the construction of these offtake pipelines. However, in order to optimise the total project, these trajectories need to be accounted for in an early stage.

Distribution reservoir
The functions of this clean water reservoir are:

- to serve as buffer for fluctuations of the water demand of the cities of Tenggarong and Samarinda.
- to create sufficient storage volume to ensure continuous water supply during failure or maintenance operations.
- to supply the downstream users using gravity only, therefore elevation should be high enough.

6.2 Starting points

6.2.1 Water supply

In subsection 3.4.6 the constant (untreated) water discharge has been determined at $1,8m^3/s$. The raw water has to be treated in a water treatment plant to make it drinkable. A 5% water loss is conventional in the water treatment plant. Consequently, the resulting drinking water production capacity is $1,7m^3/s$.

6.2.2 Water demand

Table 6.1 summarizes the water demand per user group.

Table 6.1: Water demand per city

User	**Water demand** $[l/s]$
PDAM Samarinda	1440
PDAM Tenggarong	220

6.2.3 Supply security

pipeline redundancy

To increase the supply security and reliability of the water transport system, the pipelines shall be redundant. A redundancy of 2 times 50% is chosen, i.e. two pipelines will be constructed with each a capacity of 50% of the total discharge. The extraction tunnel shall not be redundant. The likelihood of an event happening in the ground, maintenance or sabotage is very small.

Buffer capacity at the distribution reservoir

Bearing in mind that the double pipelines give an extra supply security, a reservoir volume of 4 hours at maximum flow is recommended. This entails 2 hours of reaction time in case of accidents and 2 hours for balancing the feed versus demand. The key to effective supply security is to have proper maintenance plan, as well as emergency protocols. The PDAM of Samarinda and the PDAM of Tenggarong have additional reservoirs which they currently use, so additional reservoir capacity is not expected to be necessary.

6.2.4 Elevations

The elevations along the water transmission greatly influence the hydraulic properties of the system. Table 6.2 summarizes the elevations of start and ending points of each of the pipe sections.

Table 6.2: A summary of the elevations of key points along the route of the pipeline.

pipe section	starting elevation [m +MSL]	end elevation [m +MSL]
section 0	146	145
section 1	145 (reservoir outlet level)	120
section 2	110	56 (high level)
section 3a	52 (low level)	11
section 3b	52 (low level)	27

6.2.5 Pipeline diameters

Table 6.3: A summary of the pipeline diameter requirements.

pipe section	criteria
section 0	The diameter of the tunnel should be large enough to ensure the production of hydro power does not decrease significantly due to friction.
section 1	The diameter of the tunnel should be large enough to ensure the production of hydro power does not decrease significantly due to friction.
section 2	The diameter of the pipes should be large enough that transmission from the WTP to the distribution reservoir can be established only through gravity flow.
section 3a	The diameter of the pipes should be large enough that transmission from the distribution reservoir to the PDAM of Tenggarong can be established only through gravity flow.
section 3b	The diameter of the pipes should be large enough that transmission from the distribution reservoir to the PDAM of Samarinda can be established only through gravity flow.

6.3 Design options

6.3.1 Pipeline material considerations

The material of the pipeline has a large impact on the hydraulics, costs, robustness, and possibly degradation of the water transport system on hand. Several piping materials have been assessed for the water transport system (GRP, HDPE, steel, iron). Steel and high density polyethylene (HDPE) have been shortlisted because they are:

- widely available in Indonesia
- widely used in Indonesia, i.e. contractors do have experience with these materials

Table 6.4: Pros and cons of steel and HDPE pipes

Steel	**HDPE**
+ available in large diameters + assembly can be done with local labour + reliable unit prices + resistant to high pressures + easy to repair	+ Less maintenance + more flexible + more earthquake resistant
− maintenance intensive − Steel prize subject to supply and demand	− high material costs for outer diameters larger than DN >800mm − The strength class SDR21 is limited to a max pressure of 8 bar. − Indopipe can only supply up to DN900 with SDR21 and no reliable prices for large diameters

In case of small pipe diameters are needed HDPE clearly gets the advantage over steel. When the pipe is required to have relative large diameters and/or high pressures steel is the superior material.

6.3.2 Pipeline construction alternatives

Here the different options are compared and quantified. The requirements and elevations stated in the starting points are met. The lengths of section 0 and 1 are in the order of 100 meters, thus really short compared to section 2, 3a and 3b and therefore not stated in the comparison overview of table 6.5.
The length of sections 2, 3a and 4a are based on the routes as proposed in section 6.3.3. Now the head losses due to dynamic processes can be calculated and the diameters of the pipe determined to allow flow by gravity only. Steel, welding and coating prices are based on rates of contractors and provided Witteveen+Bos. HDPE pipe costs for the available pipes at IndoPipes are based on rates provided by international manufactures, since local contractors were not able to provide details. Further research needs to specify accurate prices, especially for large diameter HDPE pipe fusion.

Table 6.5: Cost comparison for different design alternatives for present production capacity of Q = 1,7 m^3/s

Production capacity	**section 2**		**section 3a**		**section 3b**	
present Q = 1,7 m^3/s	**HDPE**	**Steel**	**HDPE**	**Steel**	**HDPE**	**Steel**
number of pipes	3	2	2	2	3	2
inner diameter [m]	0,81	0,92	0,36	0,3	0,81	0,95
outer diameter [m]	0,89	0,95	0,4	0,33	0,89	0,98
wall thickness [mm]	38,6	12,7	17,1	12,7	38,6	12,7
gross material cost (incl. welding and coating) [1 000$ / 100m]	132	132	12	45	132	136

Table 6.6: Cost comparison for different design alternatives for a hypothetical future production capacity of Q = 3,0 m^3/s

Hypothetical production capacity future	**section 2**		**section 3a**		**section 3b**	
Q = 3,0 m^3/s	**HDPE**	**Steel**	**HDPE**	**Steel**	**HDPE**	**Steel**
number of pipes	4	2	2	2	4	2
inner diameter [m]	0,90	1,13	0,36	0,37	0,90	1,18
outer diameter [m]	1,0	1,16	0,40	0,40	1,0	1,21
wall thickness [mm]	42,9	12,7	17,1	12,7	42,9	12,7
gross material cost (incl. welding and coating) [1 000$ / 100m]	229	161	12	54	229	168

For the present calculated discharge, HDPE pipelines does have a small price advantage over steel in section 2 and section 3b for the material use. However, this can be deceiving since several factors need to be accounted for:

- The difficulty of adequate pipeline fusion for large diameters can be underestimated here. A specialised crew and company is needed and can turn out to be costlier than estimated now.
- The availability of large diameter HDPE pipelines in Indonesia is not consistent.
- Larger than 900mm (outer diameter) SND21 HDPE pipes are not commercially available, so multiple pipes are needed to meet the requirements. This however increases handling, transport and storage costs.
- Deeper and wider holes need to be dug to cover the 3 pipes, a 'hidden' cost factor.

Steel is slightly more expensive for sections 2 and 3b, however:

- Pipeline thickness is conservative.
- The coating of the pipeline is a costly affair, though not proven to be necessary yet. That depends on the water quality and treatment in the WTP. This might decrease the cost per 100m with 5 000$ to 10 000$

As mentioned before, the aim is to combine phase 2A and 2B and possibly even phase 1 in the future to increase the discharge. With an increased discharge and thus larger pipe diameters, steel proves to be more cost efficient (see table 6.6).

6.3.3 Routing alternatives

As requested by the client for this report, the transportation phase (pipeline section 2) and the offtake phase (pipeline sections 3a and 3b) are considered the responsibility of the PDAMs of Tenggarong and Samarinda. Therefore, this chapter describes the selection of the advised route for the transportation phase, from the water treatment facility to the distribution reservoir, and two possible routes for the offtake phase, from the distribution reservoir to the PDAMs of Tenggarong and Samarinda. Several factors have been considered in the selection of the pipeline route because of their significant influence on transport hydraulics, construction, operation and maintenance cost. The pipeline should be constructed without additional pumps, as mentioned before, see table 6.7 for the overview.

Table 6.7: A summary of the elevations of key points along the route of the pipeline.

Pipeline Section	**Height**
Starting point (Hydro power/WTP)	Approximately 120 m +MSL
Distribution Reservoir	Approximately 52 m +MSL
Tenggarong PDAM	11 m +MSL
Samarinda PDAM	27 m +MSL

The selection of the route is based on a balance between several factors. Firstly, the length of the pipeline influences the transport hydraulics and cost of construction, operation and maintenance. Secondly, constructing the pipeline along existing roads saves construction cost and time. However, this should be avoided when the road has a high number of bends, large height increases or greatly increases the length of the pipeline. Additionally, river crossings and river beds should be avoided as much as possible due to the high cost of construction and because of the fact that the pipeline will be above ground, thus susceptible for sabotage and illegal tapping. Lastly, the land use plans for the area between the ITCI concession and Tenggarong have been used to evade areas in which the pipeline cannot be constructed or land ownership could create difficulties.

Pipeline route

Two alternative pipeline routes have been assessed. These alternatives have the same route for the first 22 kilometres. The selection of the route for the first 22 kilometres is based on avoiding large height increases and is thus for both alternatives similar.

Alternative A: straight route

The economically most advantageous route (section 2A) is 55 kilometres long and starts at the water treatment plan location at approximately 120m +MSL and ends in the outskirts of Tenggarong at approximately 52 m + MSL. This route has been selected on the shortest connection and avoidance of "problematic areas" according to the land use plans provided by the client see figure 6.3. Although this route is economically the most advantageous, it can result in difficulties regarding land procurement and the construction of the *right-of-way* (ROW) due to inhospitable terrain.

Alternative B: follow existing roads

The second alternative (section 2B) is 67,2 kilometres long and also starts at the water treatment plant location at approximately 120 m +MSL and ends in the outskirts of Tenggarong at approximately 52 m +MSL. The remainder of the pipeline route to the distribution reservoir was mainly determined using the presence of already existing roads and land use plans, while preventing large height increases, see figure 6.2 and 6.3 .

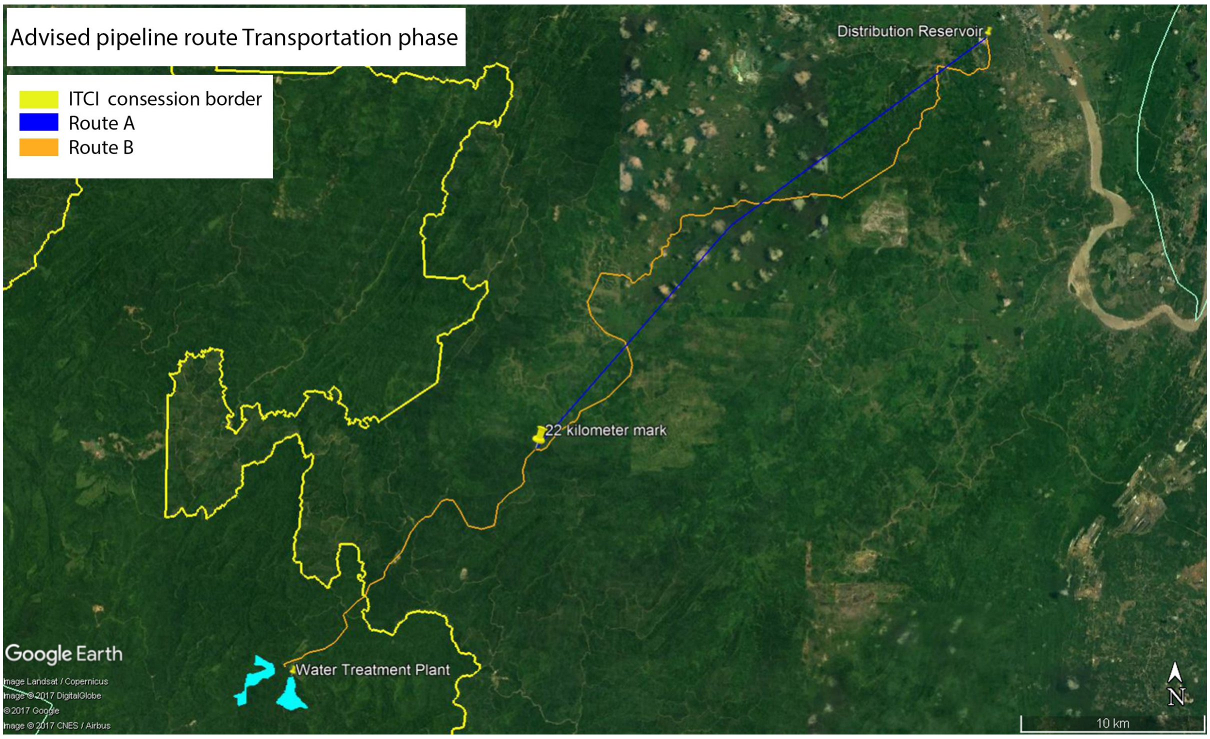


Figure 6.2: A overview of route alternatives A and B for section 2. Satellite image source: Google Maps

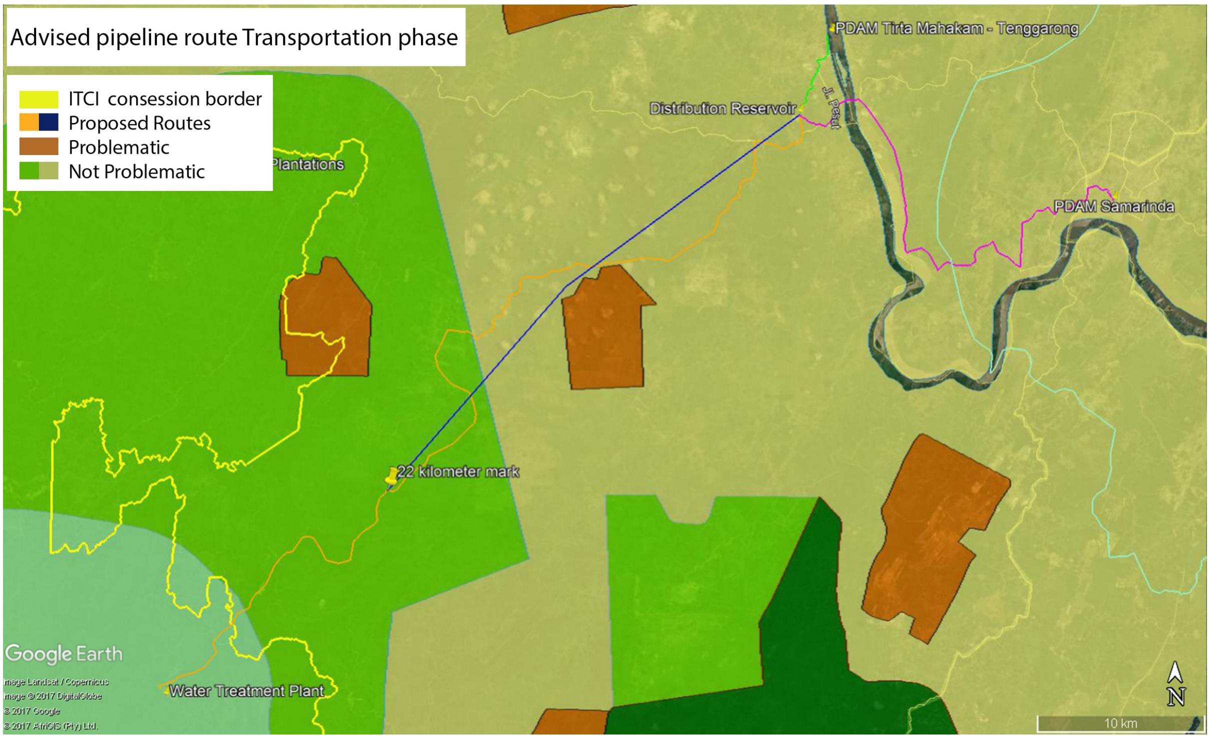


Figure 6.3: A overview of route alternatives A and B for section 2 and the land use plans.

The pipeline routes for the offtake phase to Tenggarong and Samarinda, sections 3a and 3b, were determined based on the topography and the presence of roads. The pipeline between the distribution reservoir and the reservoir of the PDAM in Tenggarong is 6,0 kilometres with a starting point at 52 m

+MSL and an endpoint at 11 m +MSL. The pipeline between the distribution reservoir and the reservoir of the PDAM in Samarinda is 34,8 kilometres with a starting point at 52m + MSL and an endpoint at 27m +MSL, see figure 6.8 for an overview.

Figure 6.4: A overview of the elevation of pipe section 2a. Height profile image source: Google Maps

Figure 6.5: A overview of the elevation of pipe section 2b. Height profile image source: Google Maps

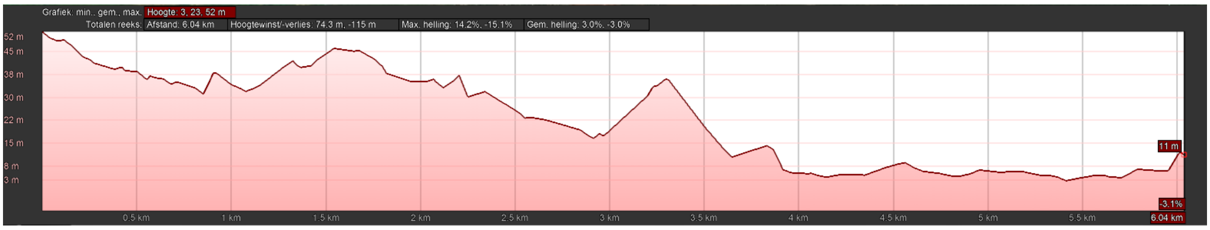


Figure 6.6: A overview of the elevation of pipe section 3a. Height profile image source: Google Maps

Figure 6.7: A overview of the elevation of pipe section 3b. Height profile image source: Google Maps

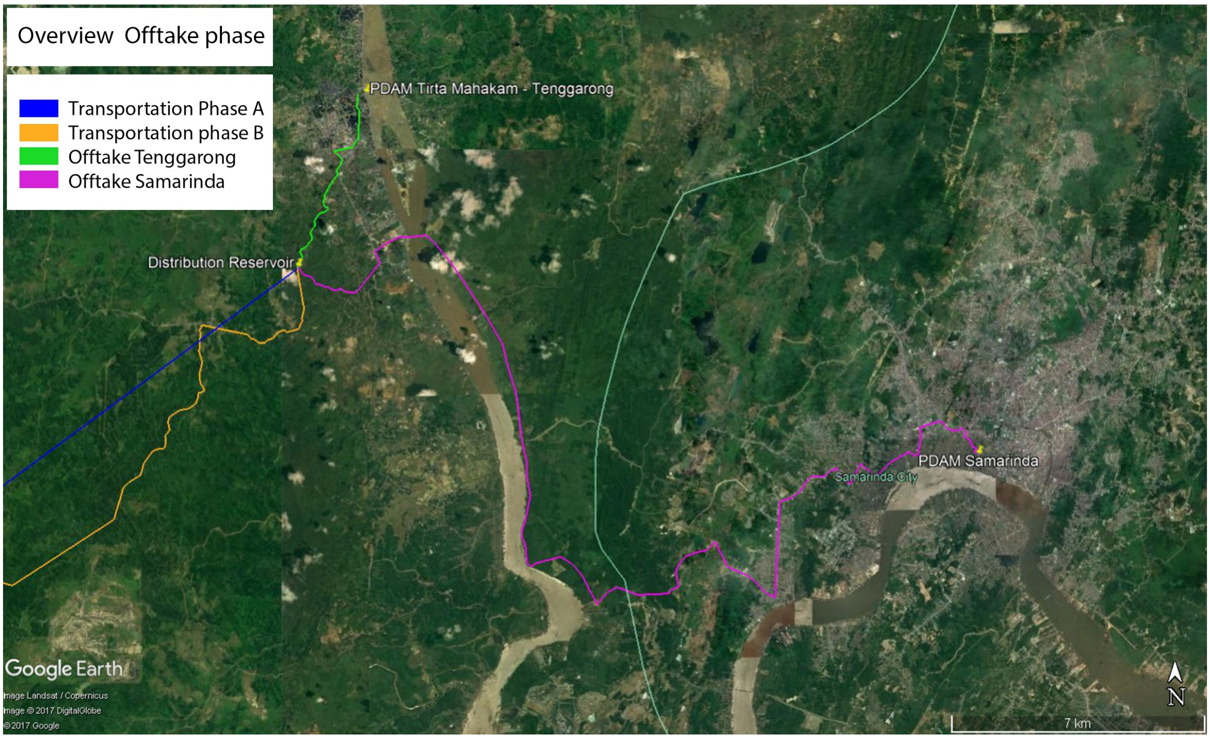


Figure 6.8: An overview of the offtake pipes to Tenggarong (section 3a) and Samarinda (section 3b). Height profile image source: Google Maps

6.4 Basic design

6.4.1 Design overview

The water will flow from the artificial reservoir into the hydro power plant, which is located on the North side of the reservoir after the dam. This location is favourable because of the access roads, that have to be built for the dam construction, and secondly the elevation of approximately 120m + MSL. After the hydro power plant, the water will be transported by gravity into the water treatment plant which is located directly next to the hydro power plant. Then through section 2A, hereafter called section 2, the clean water flows to the distribution reservoir near Tenggarong conveyed by only gravity and subsequently in separate pipelines to the PDAM of Tenggarong and Samarinda. All the pipelines have a 50% redundancy. See fig 6.4, fig 6.6 and fig 6.7 for an approximate height profile of the routes.

6.4.2 Major crossings

One major crossing exists in the design route. The pipeline from the distribution reservoir towards the PDAM of Samarinda crosses the Mahakam river at the location of the Kartanegara bridge. The span of the bridge is 580 meter and the width of the Mahakam river is 350 meters. The proposed method of crossing is an above water river crossing attached to the construction of the bridge.

6.4.3 Pipe specifications

The pipeline specifications are presented in table 6.8. For section 0 and 1 there is chosen for steel since the large height differences and steep slopes requires high tensile strengths in the pipelines. Steel is also going to be used for section 2 and 3b. Section 3a is going to be fitted with HDPE pipes.
A comprehensive evaluation of phases 1, 2A and 2B to determine the exact water quantities and pipeline routes is needed to configure the optimal pipeline route.

Table 6.8: The specifications for all the pipeline sections.

section	**quantity**	**internal diameter [mm]**	**length [m]**	**Max internal pressure [bar]**	**Material**
section 0	1	1300	150	2	steel
section 1	2	900	100	5	steel
section 2	2	920	55000	6,4	steel
section 3a	2	360	6000	3,1	HDPE (SDR21)
section 3b	2	950	34800	1,5	steel

6.4.4 Reservoir specifications

The net storage capacity is based on a 4-hour supply and will be divided in 3 compartments. The net storage volume, evidently, should be at least 24 000 m3. In this study, it is assumed that the distribution reservoir is constructed of reinforced concrete instead of steel, as steel has a shorter lifespan. The reinforced concrete construction is expected to have a lifespan similar to that of the coated steel pipes (>15 years). The reservoir is planned to be constructed on a location close to Tenggarong with an elevation of 52 +MSL. The exact location should be determined in cooperation with the PDAM and municipality of Tenggarong since the most preferable location is within proximity of the aforementioned city.The main design parameters of the reservoir are stated in table 6.9.

Table 6.9: Design parameters for the distribution reservoir.

Distribution reservoir	**value**	**unit**
length	80	m
width	75	m
height	4,8	m
water depth	4	m
total gross surface	6000	m2
net volume reservoir	24000	m3

6.5 Construction method

6.5.1 Right-of-way

The right-of-way (ROW) is the land alongside a pipeline needed for construction, maintenance and operation during the entirety of the pipelines' life cycle. The pipeline in the Samarinda water project is an underground pipeline, meaning that after construction, the land above the pipeline will have to be clear of vegetation and if not already present a road has to be constructed in order to create access to the pipeline for inspection and maintenance. A right of way (see figure 6.9 consists of two sides, the working side used for access and working equipment, and the spoil side used for the pipeline itself and the excavated soil. Taking into account two parallel pipes the ROW is estimated to have a width of 35-45 meters during construction and 15-20 meters when it has been installed.The first 22 kilometres of the pipeline the ROW has to be fully constructed in highly uneven terrain with steep slopes and drops due to the fact no existing roads exist along the advised route. After the first 22 kilometres, the advised path mostly runs alongside roads that can be used for the working side of the ROW.

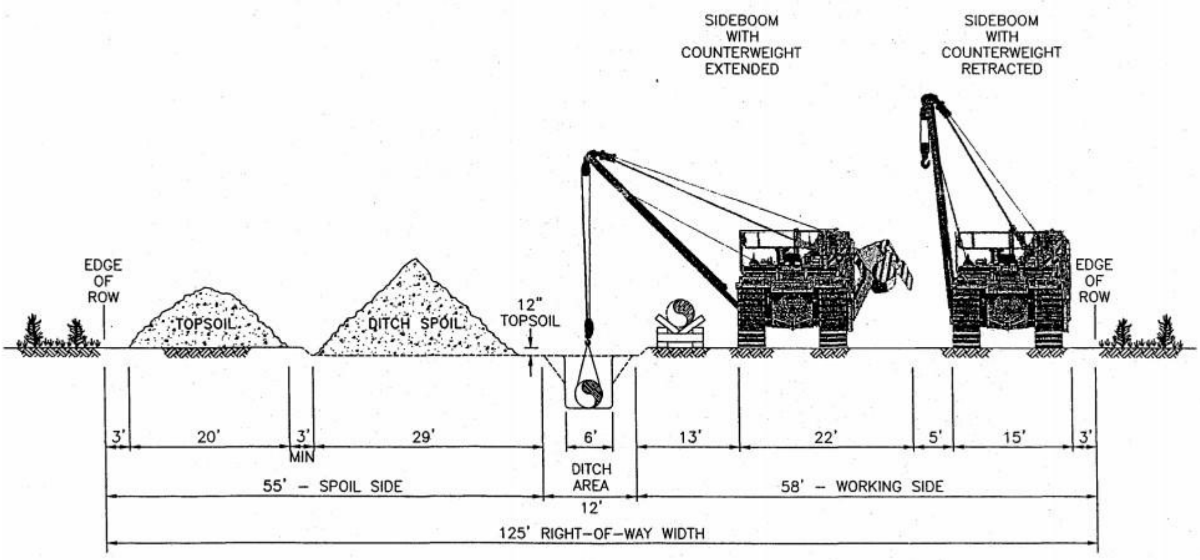


Figure 6.9: Right of Way diagram. Source: Gulf Interstate Engineering. (1999). Temporary Right-of-Way Width Requirements for Pipeline Construction

Chapter 7

Hydro power and Water Treatment Plant Cluster

It is of importance that the water that is delivered into the cities of Tenggarong and Samarinda is of potable drinking water quality. The client demands the water treatment plant (WTP) to be located within their concession area. The water in the reservoir contains lots of potential energy, that can be harvested by water turbines and generators, they can provide the WTP with consistent energy influx.

7.1 Functional requirements and starting points

7.1.1 Hydro power plant

The potential energy of the water trapped by the dam can be captured by turbines and generators. The discharge is assumed at 1,8 m^3/s with a net head of approximately 40 m. The net head is the sum of the dynamic head, i.e. friction of wall and valves, and the static head. $H_{net} = H_{static} - H_{dynamic} = 39,8\ \ m$
The hydro power station will be fitted with two turbines. Each one, independently, capable of working at the capacity of 50% of the total discharge. Each turbine needs to be regulated from $1,8 - 0,18m^3/s$ aiming to optimise the efficiency. A by-pass has to be constructed in order to establish a constant feed of the 'raw' water into the water purification plant when the turbines are in maintenance.

7.1.2 Water treatment plant

The requirement of the water treatment plant is to purify the raw incoming water into drinking water that complies with the Indonesian Drinking Water Standard which is regulated by PERMENKES No.492 Th.2001. The water quality parameters should be the deciding factors for the design of the water treatment plant.
The following starting points have been formulated:

- The WTP will be built for maximum reservoir discharge rate of 1,8 m^3/s.
- The WTP will be built in several parallel streams, so in case of failure of one stream it will not lead to the entire failure of the WTP. Also, maintenance per stream has a smaller effect if the number of streams are higher.
- The WTP will not be equipped with back-up power because of the proximity of the hydro power plant with two independent turbines and generators.

An analysis of the present-day water quality cannot be made since no water samples have been taken from the catchment area. The area is expected to have water of good quality.

7.2 Design alternatives

7.2.1 Hydro power plant

Several design options exist for these head difference and discharge rates, the most conventional being the Kaplan Turbine, the Cross-Flow Turbine and the Francis Turbine. In as much the proportion of the turbine to the total project is fairly small, a thorough comparison of the several turbines hasn't been made. It is however safe to assume that these turbines work within an efficiency range of 85-90%.

7.2.2 Water treatment plant

The conventional alternatives for the design of the WTP are associated to sedimentation, filtration and sludge treatment. The parameters that need to be used for a good comparison are:

- Treated water quality
- Required footprint
- Investment cost
- Operational costs
- Robustness and ease of operation

The client has stated that he is developing his own way of purifying the water, but that does require further research and tests. It is outside the scope of our expertise and this report to evaluate these alternatives.

7.3 Basic Design

7.3.1 Hydro power plant

The hydro power plant will consist of two main sections, being the mechanical plant and the electrical plant.

Mechanical plant

Two identical turbines will be installed, each turbine fitted with its own generator. The turbine will most likely be a cross-flow turbine type. General expected flows and the conservative efficiencies are stated in table 7.1

Table 7.1: Expected flow and power generation efficiency

admission	**100%**	**90%**	**80%**	**70%**	**60%**
flow [l/s]	1750	1575	1400	1225	1050
efficiency [-]	85%	85%	85%	85%	85%
power [kW]	581	523	465	407	348

A bypass should be constructed so that the water can be transported around the WTP in case of maintenance of one or both of the turbines. Also, drop weight valves need to be installed in case of interruption of the electricity demand or grid failure.

Electrical plant

The electrical plant will exist of the following 4 sections:

- a grid connection
- electrical transportation
- connection to hydro power generators
- connection to the WTP

All these sections need specific switchboards and transformers. The local used grids and means of electricity transport should be looked into as to define the conventional voltage levels. Also, the redundancy needs to be evaluated.

7.3.2 Water treatment plant

The proposed treatment is based on conventional, well proven technologies. A detailed study to the water quality spatially and temporal must be conducted in order to formulate how the WTP should be designed in detail in order to treat the water properly.

The treatment scheme selected for WTP consists of the following process steps:

1. pre-pH correction
2. coagulation
3. flocculation/sedimentation
4. sand filtration
5. wash water ponds
6. sludge drying beds
7. primary disinfection
8. final pH correction
9. secondary disinfection

In table 7.2 assumptions based on calculations of Witteveen+Bos concerning the surface area needed for each process step.

Table 7.2: Surface area needed for each process step of the water treatment plant based on data provided by Witteveen+Bos

Process step	**Surface area** $[m^2]$
coagulation	not significant
flocculation/sedimentation	3840
sand filtration	1440
wash water ponds	2900
sludge drying beds	10800
primary disinfection	not significant
total	19000

7.3.3 Cluster overview

The total surface area of the hydro power, water treatment plant and supporting structures should be located within ITCI concession area. Supporting structures such as:

- raw water intake
- meter chamber
- office building
- chemical building
- electrical control panel
- energy building
- access road
- fencing

Its elevation is 110m +MSL. The total area needed is estimated and specified in table 7.3.

Table 7.3: Estimation of the total surface area needed for the hydro power plant, water treatment plant and supporting structures based on data provided by Witteveen+Bos

Unit	**Surface area** $[m^2]$
water treatment plant	19000
hydro power plant	150
auxiliary structures estimation	1400
total	20500

In figure E.11 there is an overview of the planned location of the cluster. Note the elevation. The cluster is positioned just above the bed of the possible river that can occur after the use of the spillway in case of excessive rainfall.

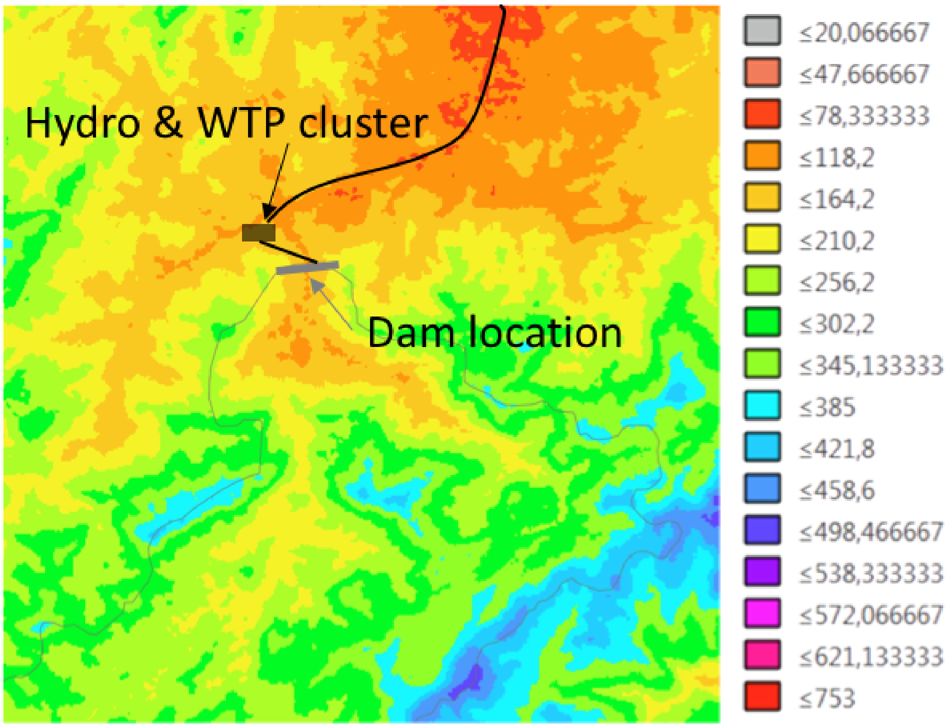


Figure 7.1: A possible location for the hydro power and water treatment plant cluster. Note the bed level and location - within range for gravitation forced transport to the Distribution reservoir. The elevation data is provided by the client.

Note that the combination and location of each of the stages in the cluster need to be optimized. The location on a hilly slope lends itself to make transmission between the stages by gravity only possible. Costly expenses as a pump house and pumps can be prevented. This does, however, require intensive analysis and detail.
Also, the location of the cluster is fairly isolated. The need for regular maintenance and water quality inspections cannot be emphasised enough. This may lead to complex operation of the cluster and should therefore be considered beforehand by the client.

7.4 Summary

The head difference between the reservoir and the hydro power plant is a potential source of energy. More study is required to explore the possibilities and turbine types to optimise efficiency and energy yield. A part of the electricity will be used to purify water to Indonesian drinking water standards in the water treatment plant and then the water will transported to Samarinda and Tenggarong. The water quality in the catchment needs to be assessed systematically to determine the process steps needed in the water treatment plant.

Chapter 8

Stakeholder Analysis

In phase 1, The Arsari water project, the offtaking stakeholders gave a clear indication of the importance and quantity of their need, thereby showing their interest in a productive and successful collaboration. In the phase 2 project, The Samarinda Water Project, the offtaking stakeholders have not yet given such an indication and one of two offtaking stakeholders has denied the need for a new source due to pollution of their main source of water. This creates the need for clear, characteristics based stakeholder engagement. In order to give recommendations for the engagement of stakeholders in a preliminary study phase the following steps are taken. First all possible stakeholders are listed and a stakeholder analysis is performed. A stakeholder analysis is used as input for the engagement of stakeholders and can be used for problem identification and problem resolvement.
For this stakeholder analysis any person, group, or organization that can place a claim on the organization's attention, resources, output, or is affected by that output is identified (Bryson, 2004). Subsequently the stakeholders interest and power in the project, problem perception, goals and resources are defined. Their problem perception and goals are used to define a possible positive or negative attitude towards the project. Their interest, power and resources are used to classify the stakeholders as critical and noncritical. Lastly the relations between the stakeholders are analysed. This network analysis combined with the attitude and criticality of stakeholders is the basis for the recommendations for the engagement of stakeholders. In this chapter the most important parts of the full stakeholder analysis are described. The full stakeholder analysis and extensive stakeholder descriptions can be found in Appendix E.

8.1 List of stakeholders

The Samarinda Water project has been divided into three phases. The artificial storage lakes phase, the transportation phase and the offtake phase. This demarcation will also be used for the stakeholder analysis because of the large distance between the three parts, the large difference in geographical landscape such as forest and urban areas, different political districts and the difference in activities, functionality and impact of the three phases on their surrounding. The stakeholders involved in the different phases are listed below.

Stakeholders for the storage lake phase

- PT Arsari Tirta Pradana *(Client, company under Arsari Enviro Industri)*
  - Project Owner
- ITCI *(Company under Arsari Enviro Industri)*
  - Landowner
- Government

  - National
  - Province of East Kalimantan
- Contractors
- Secondary product buyers
- Future employees

Stakeholders for the transportation phase

- PT Arsari Tirta Pradana *(Client, company under Arsari Enviro Industri)*
  - Project Owner
- Government
  - National
  - Province of East Kalimantan
  - District of Kukar
  - Landowner and regulator
- PDAM of the Kutai Kartanegara District *(Kukar)*
- PDAM of the city of Samarinda
- Private landowners
- Villages along the pipeline
- Contractors
- River users
  - industry and transportation companies
  - Inhabitants and fisheries

Stakeholders in the offtake phase

- PT Arsari Tirta Pradana *(Client, company under Arsari Enviro Industri)*
  - Project Owner
- National and provincial Government
  - Landowner
  - Regulator
- Municipality of Samarinda
- District of Kutai Kartanegara *(Kukar)*
  - City of Tenggarong
- PDAM of Samarinda
- PDAM of Kutai Kartanegara
  - City of Tenggarong
- Industrial customers of the PDAMs
- Non-industrial customers of the PDAMs

- PDAM supplied bottles water companies
- Bottled water companies
  - With a market share in Samarinda and Tenggarong
- PDAM supplied water trucks

8.2 Stakeholder characteristics and criticality

Stakeholders can be assessed based on their 'interest in', 'problem perception of', 'Goal for' and 'resources for' the Samarinda Water Project. Based on these characteristics certain predictions about stakeholder behaviour, reasoning, decision making, long and short term ambitions can be made and their influence on the project process can be assessed (Hans de Bruijn, 2010). Important for the feasibility phase of this project is to find out which stakeholders have, interests, problem perceptions or goals that contradict that of the client. These stakeholder can become problematic for the project feasibility if they are critical to project success. Criticality is assessed based on their power over, interest in the project and the project's dependency on and replace-ability of their resources. The tables below lists the critical stakeholders that have a contradicting or opposing problem perception for each phase of the project.

Table 8.1: Artificial storage lake phase

Stakeholder	**Problem perception**
National/district government	Currently the government does not officially recognize a problem with the clean water supply in these cities. However, nationwide attention for and acknowledgement of pollution problems is growing

Table 8.2: Transportation phase

Stakeholder	**Problem perception**
National/district government	Currently the government does not officially recognize a problem with the clean water supply in these cities. However, nationwide attention for and acknowledgement of pollution problems is growing
Municipality of Samarinda	Although the government does not officially recognize a problem with the clean water supply, within the municipality of Samarinda some recognition exists for a possible water supply problem
District of Kukar	Currently the district of Kutai Kartanegara does not recognize a problem with the clean water supply
PDAM Kukar District and PDAM city of Samarinda	The PDAM does not recognize a pollution problem of the current water supply

Table 8.3: Offtake phase

Stakeholder	Problem perception
National government	Currently the government does not officially recognize a problem with the clean water supply in these cities. However, nationwide attention for and acknowledgement of pollution problems is growing
Municipality of Samarinda	Although the government does not officially recognize a problem with the clean water supply, within the municipality of Samarinda some recognition exists for a possible water supply problem
District of Kukar	Currently the district of Kutai Kartanegara does not recognize a problem with the clean water supply
PDAM Kukar District and PDAM city of Samarinda	The PDAM does not recognize a pollution problem of the current water supply

From the characteristics and criticality analysis it can be concluded that currently several critical governmental stakeholders have to be engaged in order to change and adapt their problem perception. In the following section, the network of these critical stakeholder will be assessed to better understand their interdependence, variety and closedness towards the project. The full characteristics and criticality analysis can be found in Appendix E.

8.3 Stakeholder network

The network that exists between the stakeholders is analysed on three factors; variety, closedness, and interdependence (de Bruijn, 2008). Although independent stakeholders often have a hierarchical structure, this is especially true for Indonesian organisations (Simarmata, 2012), the networks these stakeholders perform in differ from many of the hierarchical characteristics, the most important ones described above: variety, closedness and interdependence. Within networks, sub-network can exist. These sub-networks can have different characteristics and have a, positive or negative, influence on the entire decision making process. The full network analysis and theoretical background can be found in Appendix E.

8.3.1 Variety of stakeholders

The variety, or the amount of difference between the network stakeholders, depends on factors such as organisational size, problem perception, influence and resources (de Bruijn, 2008). For the Samarinda Water project the variety of critical stakeholders is low and influenced mostly by the fact that there are many governmental stakeholders, all of which all critical and the other two critical stakeholders are the client company and ITCI, both owned by Arsari and strongly collaborating. Because of this two sub-networks exist in the Samarinda Water Project. The first sub-network consists of the Client (PT Arsari Tirta Pradana) and the landowner (ITCI) and is illustrated in figure 8.1.

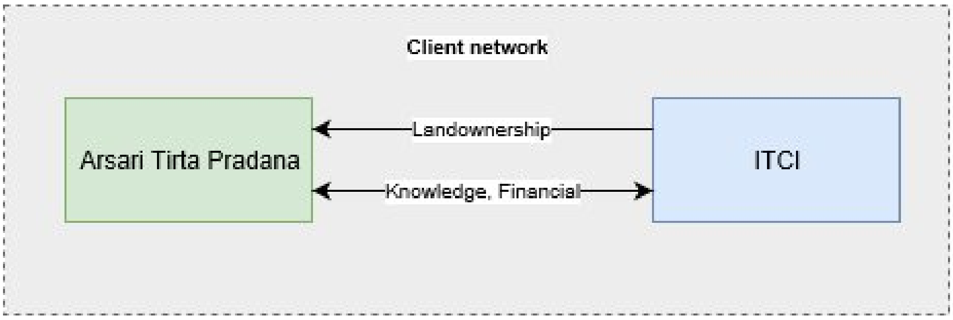


Figure 8.1: Client network *(own work)*

The second network consists of the national government, the provincial government, the municipality of Samarinda, the district of Kukar and the PDAM of both Samarinda and Kukar. This network, illustrated in figure 8.2, has a strong hierarchical structure. The existence of these sub-networks and low variety creates different opportunities and implications for the Samarinda water project.

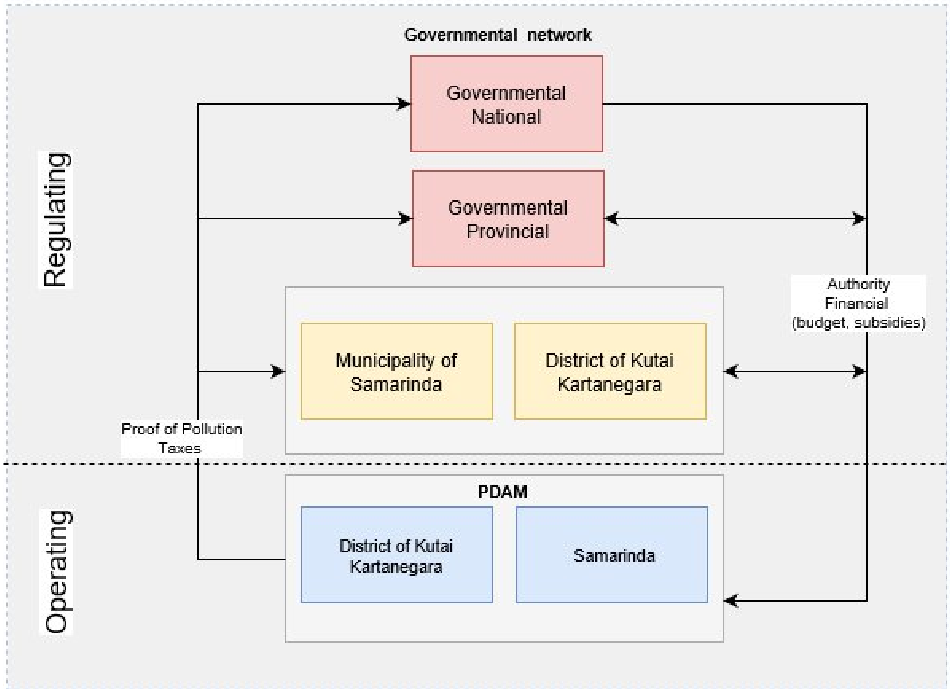


Figure 8.2: Government Network *(own work)*

An opportunity the sub-network produce is the reduction of complexity in three ways:

- Increase the reach of an intervention
- Decreases the need for tailor-made approaches
- Less room for reinterpretation

(de Bruijn, 2008)

A problem the governmental sub-network produces given the low variety and the hierarchical collaborative and authoritarian structure is a high probability for collaboration. Although collaboration can have positive effects, if they collaborate against the interests or goals of the client it can become problematic. This will be further discussed in subsection 8.3.3, Interdependencies between stakeholders.

8.3.2 Closeness of stakeholders

Depending on their goal, interest and problem perception, stakeholders can be characterized as closed. This entails that they are not sensitive, or open, to external interventions. A closed stakeholders attitude towards an intervention can manifest in several different ways but comes down to two types of attitude, ignoring and resisting (de Bruijn, 2008).
Several critical stakeholders of the Samarinda Water Project, in the sub-network of governmental institutions, currently have a closed attitude towards the Samarinda Water Project. Until their attitude towards the project changes, several scenarios on their susceptibility and acceptance of interventions from the client exists. Although these scenarios differ in likelihood, they are all possible.

Most likely

- Interventions supported by the higher governmental institutions are resisted or sabotaged by lower governmental institutions, despite the strong hierarchical structure

Less likely

- All government institutions ignore the problem and any intervention, including the the creation of awareness, will fail.
- Lower level governmental institutions ignore interventions supported by higher level governmental institutions.
- Higher level governmental institutions ignore interventions supported by lower level governmental institutions.

(de Bruijn, 2008)

With closed stakeholder there is a high upside al well. Closedness can arise from genuine doubts about the project and process. Acknowledging and working with these doubts can lead to better decision making and a more successful project (de Bruijn, 2008). Additionally, partnerships based on mutual agreement are stronger than partnerships based on forced compliance (de Bruijn, 2008). Therefore, in a scenario in which a governmental stakeholder is unwilling to collaborate, it is better to focus on ‘winning them over’ than forcing them to comply.

8.3.3 Interdependencies between stakeholders

The last characteristic used to analyse the stakeholder network is interdependencies. These interdependencies exist in different forms such as authority over another stakeholder, landownership and financial resources (de Bruijn, 2008). The interdependencies of the stakeholder involved in The Samarinda Water Project have been mapped in formal maps. In figure 8.3 the formal map for the critical stakeholders in the entire Samarinda Water Project can be seen.

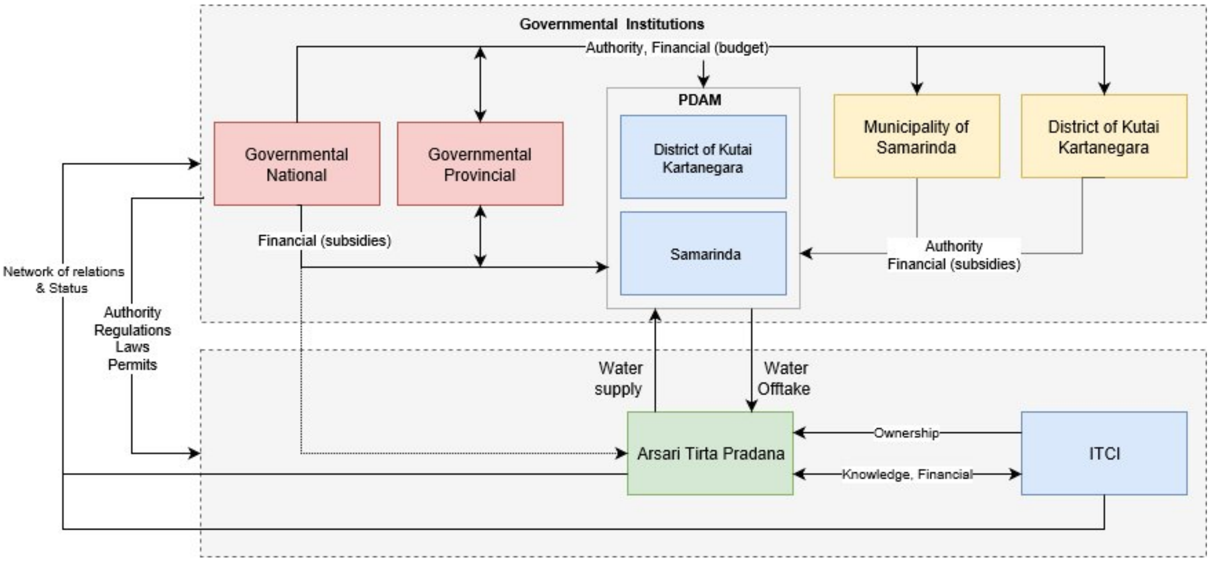


Figure 8.3: Critical stakeholders for the entire project *(own work)*

** Through strong collaboration in their sub-network the PDAMs of Kukar and Samarinda could indirectly have the resources 'Authority, Regulations, Laws and Permits' over PT Arsari tirta pradana and ITCI.*
*** Subsidies are shown as a resource directly towards the PT Arsari tirta pradana and the PDAMs. This is because it is currently unknown if and how subsidies could be ascribed to the project.*

Based on the available resources and with the use of the formal map several opportunities and risks for the project arising from the interdependencies can be described.

- As long as the need for clean water is denied, the water supply and water offtake dependencies are not valid. This means there is currently little incentive for moderate and collaborative behaviour.
- Water supply is a valuable resource for the client when negotiations start. The value or weight of this resources depends on the availability of alternatives and the extent of the pollution of the current clean water sources and both are currently unknown.
- Two levels of authority over were identified within the sub-network of governmental institutions. The national government has authority over all other governmental stakeholders. The the District of Kukar and the municipality of Samarinda have authority over the PDAMs. This poses both a risk and an opportunity:
  - Authority can lead to poor decision making
  - Authority can be used to force a subordinate stakeholder to take the right decision

(de Bruijn, 2008; Hans de Bruijn, 2010)

See Appendix E for the full description of the interdependencies

8.4 Conclusion

From the stakeholder analysis several conclusions can be drawn for the stakeholder engagement in chapter 10. In the stakeholder analysis it was found that two sub-networks exists. The second sub-network, consisting of all governmental stakeholders, has a high probability of collaboration due to its hierarchical set up and authoritarian resource distribution. This, among others, is a reason the client should ensure a strong party from that network is willing to collaborate. Within this network, hierarchy and authority can be used to force unwilling or closed stakeholders. Forcing a stakeholder can have a negative impact on further collaboration, which in long term projects such as the Samarinda Water Project can lead to severe problems.

Lastly, from a meeting held with the PDAM of Tenggarong and audited reports from both the PDAM of Samarinda and the PDAM of Tenggarong is was concluded they do not publically acknowledge a pollution problem with the current sources. Therefore they currently are considered closed stakeholders not directly willing to collaborate in and for the Samarinda Water Project. This closedness, in combination with the low level of variety and strong interdependencies in the governmental sub-network create an environment in which little collaboration with the client is expected until a problem with the current sources of water is acknowledge and no better alternatives are present.

The stakeholder analysis should be seen as a constant and iterative process due to several reasons. First and foremost, the stakeholders and the network they form are dynamic. The position of stakeholders and their resources may change. The variety, closedness and interdependencies of the network can change and these characteristics on their own are interdependent as well. Due to the dynamic character of the network, threats and opportunities change and new ones may arise (de Bruijn, 2008). In order engage threats and use opportunities, they have to be monitored and controlled throughout the project. A stakeholder analysis is project specific. This entails that if the project changes, the stakeholders and their network may change as well. Because this is a preliminary study, only the critical stakeholder have been thoroughly assessed. As the project progresses, new stakeholders can become involved in the project and the position of stakeholders can change, from non critical to critical and from proponents to opponents. It is therefore recommended that as the project develops into next phases, all stakeholders are taken into account.

Chapter 9

Financial Analysis

This chapter is about the financial feasibility of the Samarinda Water project. The goal of this analysis is to examine if the project will break even in a realisable timespan. To analyse commercial viability of the Samarinda Water Project multiple crucial variables are put forward. First, to obtain the current and future demand of the cities of Samarinda and Tenggarong, the offtake is estimated until 2035. Then, the direct cost of the necessary investments are defined for each of the three phases. Subsequently, the total amount of the initial investment is estimated by using the P50-method. With the estimated demand, the investments and the income, a break-even analysis will be performed with the use of NPV-models. In this section the inputs for the NPV-model will be explained in detail and will be analysed on the basis of two scenarios: Arsari bears all the costs and the PDAMs pay for the transportation and offtake phase.

9.1 Production capacity and offtake estimations

This section will focus on the offtake estimations of the cities: Samarinda and Tenggarong. The production capacity is calculated in Chapter 3 and is set on 1,7 m^3/s for phase 2A. The capacity from phase 2B and the overspill/connection with phase 1 could contribute to a larger capacity, however, these quantities are unknown and are not incorporated in this analysis. Therefore, the focus will solely lie on the production capacity of phase 2B. Thus, one should keep in mind that this estimated capacity is not the maximum capacity. In section 9.1.2 a forecast of the offtake is made to see what the current and future demand of Samarinda and Tenggarong is. A more detailed analysis can be found in Appendix F.

9.1.1 Assumptions for the offtake estimations

To estimate the demand of Samarinda and Tenggarong till 2035 a forecast of the population growth is made. The demand estimation is based on several assumptions which are mentioned below:

- The production capacity of Arsari Enviro Industri to supply clean water to the cities of Samarinda and Tenggarong is based on the production capacity of phase 2A calculated in chapter 3.
- The offtake of the PDAMs till 2035 is based on the installed capacity. The installed capacity is the full theoretical capacity the PDAMs possess before production and distribution of water.
- It can be assumed that the population in these cities increases with the the urban growth rate estimated for East Kalimantan, due to the fact both are large cities.
- The demand till 2035 is solely based on the growth of the population, as there was a lack of data surrounding the growth of industry in both cities. Therefore it is assumed that the industry grows with the same rate as the estimated population growth rate.

9.1.2 Forecast of the water demand

Currently, the city of Tenggarong has a demand of 0,4 m^3/s and the city of Samarinda 2,6 m^3/s (PDAM Samarinda, 2017; PDAM Tenggarong, 2017). Thus, the total current offtake is 3,0 m^3/s in 2017. In table 9.1, it can be seen that the demand grows till 2035 to a demand of 0,67 m^3/s and 4,3 m^3/s, respectively. Resulting in a total water demand of 5,0 m^3/s in 2035. In Appendix F.1, the calculation of the urban growth rate and the population growth of both cities can be found.

Table 9.1: Installed Capacity for Tenggarong and Samarinda till 2035

Year	Urban Growth Rate (%)	Installed Capacity Tenggarong (m3)	Installed Capacity Samarinda (m3)	Combined Installed Capacity (m3)
2015				
2016	3,2%	0,4	2,6	3,0
2017	3,2%	0,4	2,7	3,1
2018	3,2%	0,4	2,8	3,2
2019	3,2%	0,5	2,8	3,3
2020	2,9%	0,5	2,9	3,4
2021	2,9%	0,5	3,0	3,5
2022	2,9%	0,5	3,1	3,6
2023	2,9%	0,5	3,2	3,7
2024	2,9%	0,5	3,3	3,8
2025	2,6%	0,5	3,4	3,9
2026	2,6%	0,5	3,5	4,0
2027	2,6%	0,6	3,6	4,1
2028	2,6%	0,6	3,6	4,2
2029	2,6%	0,6	3,7	4,3
2030	2,1%	0,6	3,8	4,4
2031	2,1%	0,6	3,9	4,5
2032	2,1%	0,6	4,0	4,6
2033	2,1%	0,6	4,1	4,7
2034	2,1%	0,7	4,2	4,9
2035	2,1%	0,7	4,3	5,0

9.1.3 Conclusion of the production capacity and offtake estimations

Deriving from the production capacity of phase 2A of 1,7 m^3/s and the current demand of 3,0 m^3/s. It can be concluded that with the current production capacity the full demand could not be delivered. It should be noted that this is not the maximum production capacity Arsari Enviro Industri could offer. With the addition of phase 2B and the connection/overspill of phase 1, the production capacity will be enlarged. However, when looking at the demand in 2035 of in total 5,0 m^3/s, the addition in production of 3,4 m^3/s is a large gap. Additionally, industry might outgrow the population in water demand leading to a larger total demand. Thus, by adding phase 2B and the overspill/connection of phase 1, the supply is likely to fail in meeting the demand in 2035. Consequently, as the supply is thought not to be sufficient, alternatives to supply clean water to both cities have to be sought after. Additionally, to know the exact deficiencies in supply versus demand, further research should be carried out in order to obtain the production capacities of both phase 2B and the overspill/connection with phase 1.

9.2 Cost of investment

In this chapter the initial costs of investment are calculated. The initial costs of investment are based on the direct costs i.e. cost of equipment, activities and materials and the added costs i.e. allowances, indirect costs and contingencies. Eventually, with the use of P50 method the final investment costs are estimated. In Appendix F.2, the full description and calculations of the costs is given.

9.2.1 Assumptions for the initial investment costs

To obtain the most exact and robust estimation of the initial investment costs, the costs were assessed without value-added taxes (VAT), divided over three phases (storage lake, transportation and offtake) and the following assumption was made:

- The total initial investment consist of multiple activities that will be invested over unequally over a certain period of time. However, in this analysis it is assumed that the full amount will be appropriated at the start of the project.
- Assumptions are inherent to the costs calculations. Which are not needed to be noted in this chapter, however, they are explained in Appendix F.2 in detail.

9.2.2 Direct and added costs

The direct costs and added costs are derived from the Witteveen+Bos financial data and adjusted for the project specific characteristics of the Samarinda Water Project. In Appendix F.2.1 the costs are specified and explained in detail.

Direct costs

In table 9.2, the direct costs are portrayed of the specific investments that are needed to finalise the project. The direct costs are based on material costs, equipment costs and labour amongst other things.

Table 9.2: Direct Costs of the Investments (x million)

Detailed Scope List	Direct Costs $
1. Storage Lake	
1.1 Lake A	0,3
1.2 Dam A	15,8
1.3 Tunnel	2,2
1.4 Pipelines Section 1	0,1
1.5 Water Treatment Facility	44
1.6 Water Treatment BioChar	-
1.7 Hydro Plant Power	2,6
2. Transportation	
2.1 Pipelines Section 2	67,5
2.2 Reservoir	5,6
2.3 Pumps	-
2.4 Land Procurement	-
3. Offtake	
3.1 Pipelines Section 3a	3,8
3.2 Pipelines Section 3b	43,6
3.3 Pumps	-

The storage lake phase consist of preparing the current surface of phase 2A for an artificial storage lake. Additionally, a dam will be constructed with an accompanying tunnel to subtract the water from the artificial lake. The pipeline section 1 traverses a short distance of roughly 100 meters from the dam to the hydro power plant, which continues to the water treatment facility where the water will be purified. The transportation phase has two costs items: pipeline section 2 and the distribution reservoir. The clean water has to be transported over a distance of about 55km from the water treatment facility to the distribution reservoir. The reservoir will be built in order to separate the water flows to Tenggarong and Samarinda and to create a buffer in case of short unavailability of water supply. In the offtake phase, two sections of pipelines are built, one to Tenggarong, section 3a, and one to Samarinda, section 3b.

As seen in table 9.2, there are several big expenditures: water treatment facility, pipelines section 2 and pipelines section 3b. The water treatment facility is based on the costs provided by Witteveen+Bos and altered according to the production capacity. The pipelines section 2 and 3b are large expenses due to the large distance with uneven topography that has to be covered. The material costs of steel and the road access are the largest contributors in the costs. In Chapter 6, a comparison is made between steel- and HDPE-pipelines. For long distances where gravitational powers are used for transport, HDPE is not suitable. Thus, the material of the pipelines in section 1, 2 and 3a are steel and section 3b HDPE.

Furthermore as seen in table 9.2, several investments are not quantified. Water treatment with biochar is not yet specified, because as of yet there is no data available on biochar efficiency and capacity. However, that is currently being examined. If the biochar yields the same quality, this could drive down costs and also act as a more sustainable alternative to the current water treatment facility. Furthermore, the pumps in both the transportation phase and the offtake phase are not quantified because in the current proposition, gravitational force propulses the water transport. Additionally, no quantification is given for procurement of land, as there was no data present on the prices of land that will be surpassed. The client mentioned that this will not be substantial costs.

Added costs

In the preliminary phase, certain costs are difficult to ascertain in detail, for that reason several estimations are made in order to calculate the amount of the initial investment costs. These estimations are derived from the financial report of Witteveen+Bos and backed by literature. The additional costs are mentioned and elaborated upon below and added to the direct costs in table 9.3.

- **Allowances:** 5%. Allowances are namely administrative costs, such as: costs of postage, holding meetings, telephone calls etc. It is custom to set in on 5% with new projects (Benge, 2014). Also, Witteveen+Bos incorporates an allowance of 5%.
- **Indirect Costs:** 15%. Derived from Witteveen+Bos, which contains i.e. site facilities and site organization amongst other things.
- **Contingency:** 20%. Contingency is the amounts of fund added to the base costs to cover uncertainties and risks (Baccarini, 2004). As the project is in the preliminary phase, a high contingency rate is necessary to have a sufficient buffer for high uncertainties. Further in the project, when certain risks are surpassed, a lower rate could be incorporated. Witteveen+Bos set their contingency estimate on 10% in their feasibility study (in the preliminary phase the rate was also 20%).
- **Remaining/Engineering Costs:** 11%. Derived from Witteveen+Bos, which contains i.e. project management and engineering and contractor surveys.

Table 9.3: Added Costs on top of the Direct Costs (x million)

Detailed Scope List	**Direct + Added Costs $**
1. Storage Lake	
1.1 Lake A	0,5
1.2 Dam A	25,1
1.3 Tunnel	3,4
1.4 Pipelines Section 1	0,1
1.5 Water Treatment Facility	68,8
1.7 Hydro Plant Power	4,1
2. Transportation	
2.1 Pipelines Section 2	106,9
2.2 Reservoir	8,9
3. Offtake	
3.1 Pipelines Section 3a	6,0
3.2 Pipelines Section 3b	69,0
Total Initial Investment Costs (excl. 10% VAT)	**292,8**

9.2.3 Investment cost estimation

From the direct and added costs, a total initial investment is derived. However, nine out of ten large infrastructure projects have a cost overrun (Bent Flyvbjerg & Buhl, 2003). Therefore, a cost estimation will be made to pinpoint the initial investment costs based on empirical studies. By incorporating the PERT (Project Evaluation and Review Technique), a more accurate initial investment cost can be obtained. The PERT is applied to calculate the average total initial investment μ and the accompanying standard deviation σ. By using a costs estimation based on the estimation and actual costs of similar projects a more specific and robust cost estimation for this project is derived.

$$Average(mean) = \mu = \frac{a + 4 * m + b}{6} \tag{9.1}$$

$$StandardDeviation = \sigma = \frac{b - a}{6} \tag{9.2}$$

The average or mean estimate is based on the most likely costs or the known direct costs and added costs (m), seen in table 9.3. An optimistic (a) and a pessimistic (b) value are derived from empirical studies. To perform a P50-analysis optimistic and pessimistic values for the eventual costs have to be given to the project's activities. The activities: lake, dam, tunnel, water treatment facility and hydro dams have a pessimistic value of 80% and an optimistic value of 10%. The pipeline activities have a pessimistic value of 60% and an optimistic value of 40% (Awojobi & Jenkins, 2015; Rui et al., 2012). In Appendix F.2.3 the optimistic and the pessimistic values are elaborated on. On the next page in table 9.4 the estimated averages and standard deviations are calculated, summing up to a total estimated P50 investment of \$312 million with a standard deviation of \$47 million. Although, the values are not specific for Indonesian projects, it gives a good indication how certain project can vary in estimated costs and actual costs.

Table 9.4: P50-Estimate based on the average of an optimistic and pessimistic value (x million)

Detailed Scope List	**Variability**			**Probability**	
	Optimistic (a)[$]	**Most Likely (m)[$]**	**Pessimistic (b)[$]**	**Average (μ)[$]**	**Standard Deviation (σ)[$]**
1. Storage Lake					
1.1 Lake A	0,4	0,5	0,9	0,5	0,07
1.2 Dam A	22,6	25,1	45,2	28,0	3,8
1.3 Tunnel	3,1	3,4	6,2	3,8	0,5
1.4 Pipelines Section 1	0,07	0,1	0,19	0,1	0,02
1.5 Water Treatment Facility	61,9	68,8	123,9	76,8	10,3
1.7 Hydro Plant Power	3,7	4,1	7,4	4,6	0,6
2. Transportation					
2.1 Pipelines Section 2	64,1	106,9	171,0	110,4	17,8
2.2 Reservoir	8,0	8,9	16,0	10,0	1,3
3. Offtake					
3.1 Pipelines Section 3a	3,6	6,0	9,5	6,2	1,0
3.2 Pipelines Section 3b	41,4	69,0	110,5	71,4	11,5
P50-Estimate				**312**	**47**

9.2.4 Conclusion of the estimation of the investment costs

With the P50 analysis the sum of the total initial investment costs is obtained. Based on the production capacity of 1,66 m^3/s, the initial investment of the project is set on $312 million with a standard deviation of $47 million. This is what the project will initially cost when incorporating the direct costs, the added costs and the uncertainties that are inherent with these kinds of projects. As mentioned, the optimistic and the pessimistic value in the P50 analysis are based on projects all over the world and could, therefore, differ from values in Indonesian projects. However, it gives a good indication in how estimations and actual costs can differ in comparable projects. Furthermore, by incorporating the P50 analysis as well as the implementation of 20% contingency, two methods are used to calculate for uncertainties i.e. possible cost overruns, therefore, the initial investment is a conservative estimate.

9.3 Break-even analysis

In this section, the focus lies on the inputs and the outputs of the NPV-model. In this analysis two NPV-models are used. One with a payback period of 10 years, as requested by the client, wherefrom the price of the produced water will be derived and the other NPV-model calculates what year the project will break-even with the price of water as input. Unfortunately, the full possible production capacity is not known as the phase 2B and the connection/overspill is not yet defined in technical and financial detail. However, with the use of current data several conclusions can be drawn. A variety of variables that are needed to perform a NPV-analysis are are depicted below and explained in detail in Appendix F.3.

9.3.1 Assumptions and inputs for break-even analysis

A NPV-analysis is used to perform the break-even analysis. The NPV-analysis has a variety of inputs and assumptions that are briefly elaborated on below.

- The total initial investment is $312 million. It will be appropriated on the start of the project, because it is unknown when the separate investments will be allocated. This gives the NPV-analyses a worst-case scenario, as the investments will not be discounted.
- The production estimate is set on 1,7 m^3/s.
- It is assumed that the project will be finished in year 3. This entails, that from year 3 revenue of clean water and electricity will be made.
- The operation & maintenance costs are assumed to be 4,5% of the total initial investment: $5 million. In year one, the O&M Cost will be at 25%, year two at 50% and when the project is finalized in year three: 100%. The amount is only derived from operating and maintaining the storage phase activities. The O&M Costs of the transportation phase and offtake phase are appointed to the PDAMs in this analysis.
- A nominal discount rate of 13,1% is incorporated in this analysis. This is based on the average interest and a inflation rate of the last 10 years (Trading Economics, 2017b).
- The cashflows in the NPV-analysis will be corrected on the basis of the average inflation rate of the last 10 years. The average inflation rate is 5,7% (Trading Economics, 2017a).
- The electricity revenue has a yearly income set on $0,6. This is derived from the amount of electricity that is produced from the hydro power plant and the average costs of electricity. As electricity is needed in some operational activities, not all electricity will be sold, but, no need for procuring electricity elsewhere. Therefore, it is assumed that the produced electricity will serve as electricity revenue.
- The operation and maintenance costs, electricity revenue, nominal discount rate, inflation rate and the production estimate in this analysis have the same value over the years.

- The water prices in Samarinda and Tenggarong have tariffs for different customers. The average water price in Samarinda is $0,32 per m^3 and the average water price in Tenggarong is $0,34 per m^3. The starting average water price in the NPV-analysis is $0,33 per m^3.
- Two NPV-analysis are performed. One with a payback period of 10-years, where the average yearly price will be derived from and one with the average water price as input with a maximum timespan of 20 years, starting at a average water price of $0,33 per m^3 in the latter NPV-model.
- The PDAM of Samarinda has a profit of $0,016 per m^3 and Tenggarong has a profit of $0,006 per m^3, as was derived from client's data. Addionally, the client indicated they PDAMs are not allowed to create profit for themselves. Therefore, it can be said the PDAMs have little to no aspirations to make a large profit from selling water. Thus, it is assumed the calculated average asking water price for Arsari Enviro Industri will be the same price that the customers will pay.
- Indonesian law states that a PDAM can increase the average water price with a maximum of 10% per year. A raise of 10% could not be maintained indefinitely, therefore, an increase of 1% is incorporated after the project's NPV is positive (break even).
- Two scenarios are examined in this analysis. Both scenarios are two extremities, being that Arsari bears all the investment costs of the project and the PDAM pays for the transportation and offtake phase, resulting in a high amount for the PDAM in the second scenario. In the table 9.5, the investment amounts are depicted.

Table 9.5: Two Scenarios of bearing the initial investment (x million)

	Arsari [$]	**PDAMs[$]**
Scenario 1: Arsari bears all the costs	312	0, -
Scenario 2: PDAM pays for the tranportation and offtake phase	114	198

- The average water prices derived from the analyses are calculated from the costs inflicted by Arsari Enviro Industri. Also in the second scenario, where the PDAMs bear the costs of the tranportation and offtake phase. Meaning, the costs made by the PDAMs will not be calculated onto the customers in this analysis. It is solely derived from the investments made by Arsari Enviro Industri.

9.3.2 NPV-Analyses with scenarios

In the upcoming analysis two scenarios will be worked out: Arsari bears the costs and the PDAM pays for the 2nd and 3rd phase of the project. These scenarios will be implemented in two different NPV-models. The scenarios and the above-mentioned input variables will be implemented in the two different NPV-models. The criteria if the project is commercially viable or financially feasible is based on the ability of the customers to pay the increase in price.

NPV-Analysis with average water price as output: 10-year payback period

This analysis is based on a 10-year payback period. Therefrom, as all the variables are known except the average water price, a yearly water price can be calculated in order to have the project paid back in 10 years. In formula 9.3 stated below the detailed NPV is shown.

$$NPV = -C_0 + \frac{(0,25 * -O\&M) * (1+k)}{1+r} + \frac{(0,5 * -O\&M) * (1+k)^2}{(1+r)^2} + \frac{((Prod * Price) + Er - O\&M) * (1+k)^3}{(1+r)^3} + \cdots + \frac{((Prod * Price) + Er - O\&M) * (1+k)^{10}}{(1+r)^{10}} = 0 \tag{9.3}$$

NPV = Net Present Value ($)

C_0 = *Total Initial Investment Costs ($)*

O&M = Operation and Maintenance Costs ($/year)

Prod = Production Capacity (m^3*/year)*

Price = Average Yearly Water Price ($/year)

ER = Electricity revenue ($/year)

r = Nominal discount rate (%)

k = Inflation rate (%)

Given the fact the water price is the only unknown variable, the yearly water price could be calculated. As the yearly price is calculated by solving formula 9.3. Since it only has one value, the price merely acts as an indication for what has to be earned in order to break even in 10 years. Though, it gives important and valuable information about the commercial viability project.

Scenario 1: Arsari Enviro Industri bears the total initial investment costs

In this scenario Arsari bears the costs for all the three phases: storage lake, transportation and offtake. As mentioned, the full amount is $312 million. The first two years no income is made as seen in figure 9.1. In year three the project is finished and revenue is made from the production of water and electricity.

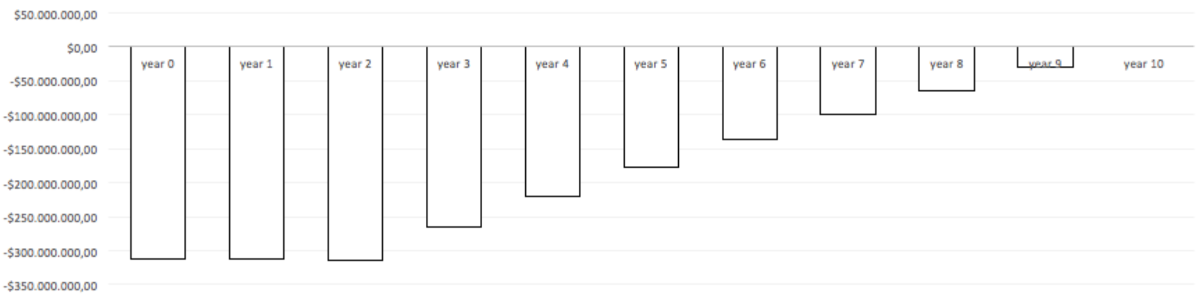


Figure 9.1: 10-year payback period: Arsari bears all the costs

In order to break-even in 10 years, a yearly price is needed of $1,3 per m^3 with a yearly discount rate of 13,1%. Which is much higher than the current average price of $0,33 per m^3. Therefore, it can be concluded that a project with a payback period of 10-years where Arsari bears all the costs is not feasible as an average water price increase of this magnitude is not possible to calculate onto the customers. Additional possibilities, such as Public-Private-Partnerships and Subsidy schemes exist. This is further discussed in chapter 10 *Stakeholder Engagement* and Appendix G

Scenario 2: PDAMS bear the costs for the transportation and offtake phase

This scenario the PDAM's pay for the transportation phase and the offtake phase. Consequently, the costs of Arsari are \$114 million and the costs of the PDAM's are \$198 million. This results in the following figure 9.2 for a 10-year payback period NPV-model.

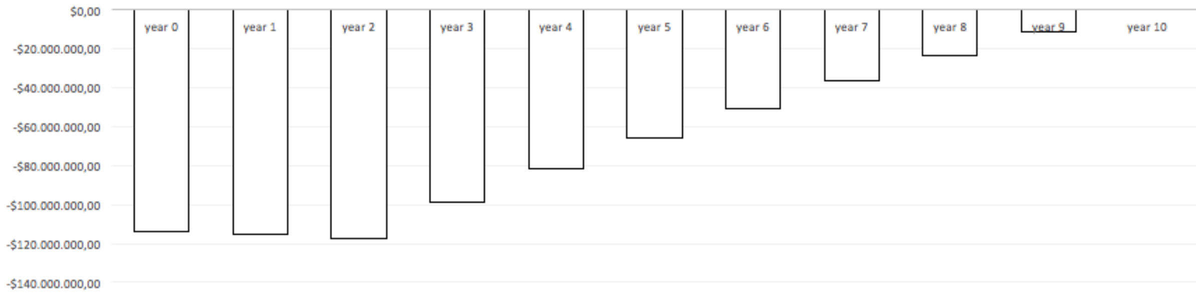


Figure 9.2: 10-year payback period: PDAM bears the costs for the 2nd and 3rd phase

A yearly average water price is needed of \$0,52 per m3 for Arsari to break even in 10 years, calculated with a discount rate of 13,1%. As mentioned, the government can increase their water price with 10% per year. This entails, starting at \$0,33, after 10 years the average water price is \$0,64 per m^3 with a net present value of -\$17 million. Seen in figures 9.3 and 9.4 below, this project is not feasible within 10-years even if the PDAMs will bear the costs for the transportation and offtake phase. Furthermore, as assumed in this analysis, the PDAM will not calculate the investment costs onto the customers.

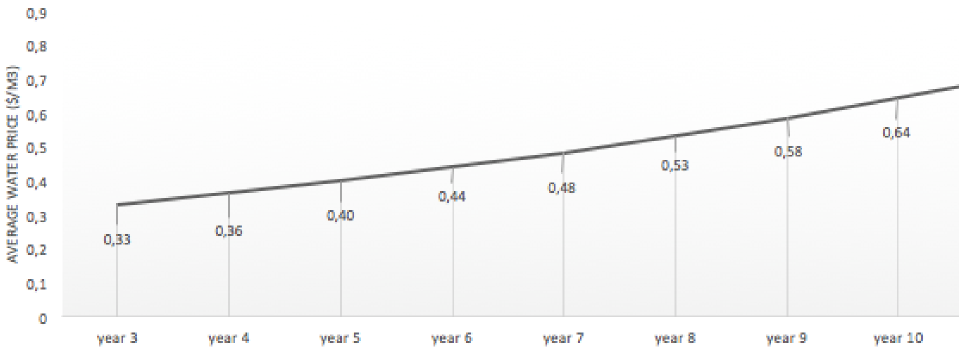


Figure 9.3: Price with a 10% increase per year before breaking even

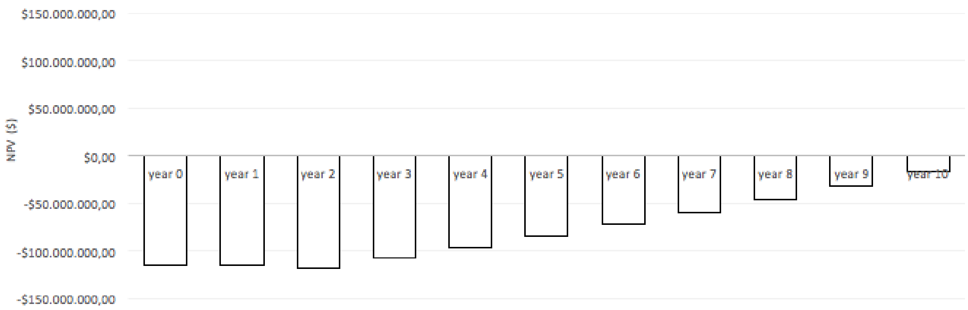


Figure 9.4: Time span of 10 years with a negative NPV on year 10

NPV-Analysis with average water price as input: payback period calculation

A 10-year payback period was requested by the client, however, as apparent in the previous section, a payback period of 10 years is not feasible. Therefore, an NPV-model with a timespan of 20 years will be analysed, where the average water price will serve as input. Whereas in the previous section the water price was subtracted from the NPV-analysis, this NPV-model will insert the water price and subtract in which year the project will break-even. The starting point of the model is the price the customers are currently paying on average \$0,33 per cubic meter. The model has a timespan of 20 years. An assumption is made that the project should break-even in 20 years for it to be feasible. The 20-year NPV-model is portrayed below in formula 9.4.

$$NPV = -C_0 + \frac{(0,25 * -O\&M) * (1+k)}{1+r} + \frac{(0,5 * -O\&M) * (1+k)^2}{(1+r)^2} + \frac{((Prod * Price_3) + ER - O\&M) * (1+k)^3}{(1+r)^3} + \cdots + \frac{((Prod * Price_{20}) + Er - O\&M) * (1+k)^{20}}{(1+r)^{20}} \tag{9.4}$$

NPV = Net Present Value (\$)

C_0 *= Total Initial Investment Costs (\$)*

O&M = Operation and Maintenance Costs (\$/year)

Prod = Production Capacity (m^3*/year)*

Price = Average Water Price (\$/year)

ER = Electricity revenue (\$/year)

r = Nominal discount Rate (%)

k = Inflation rate (%)

Scenario 1: Arsari bears the total initial investment costs

In this scenario Arsari bears the costs for all the three phases: storage lake, transportation and offtake. As mentioned, the full amount is \$312 million. The first two years no income is made as seen in figure 9.6. In year three the project is finished and revenue is made from the production of water and electricity. Furthermore, as seen in figure 9.5 the starting water price is \$0,33 per m^3 in year three.

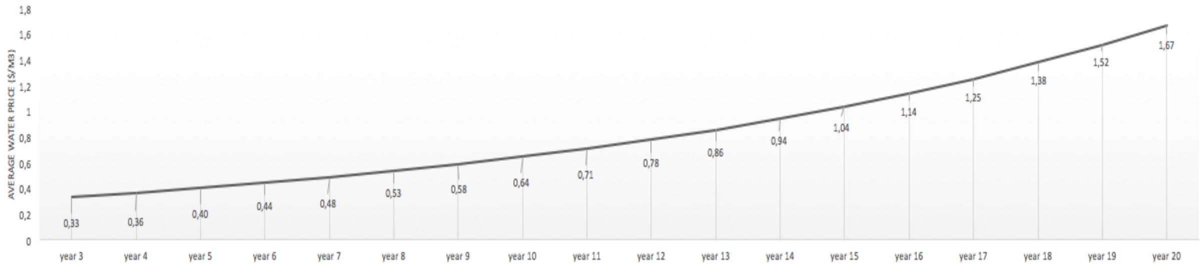


Figure 9.5: Average water price with a 10% increase per year before breaking even

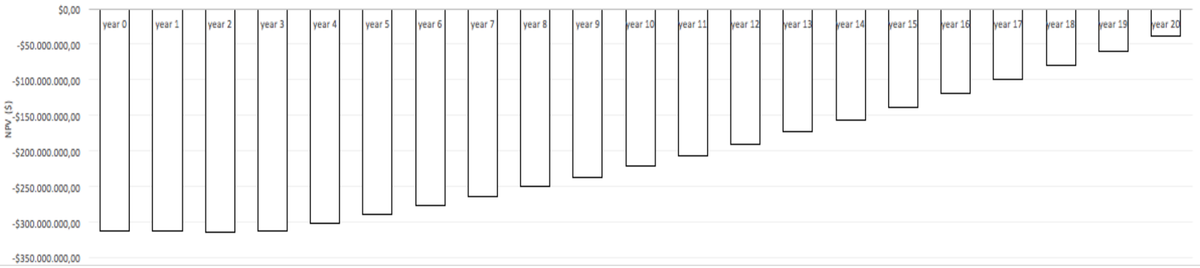


Figure 9.6: Time span of 20 years with a negative NPV at year 20

Derived from the figures 9.5 and 9.6, it can be seen that with a yearly increase of 10% starting at the current water price \$0,33 per m^3 with a discount rate of 13,1%, the initial investment costs will not be paid back in a timespan of 20 years. The price increases to \$1,67 per m^3 in year 20 and with of around \$40 million. Therefore, it could be concluded that if Arsari alone bears all the costs, the project is not feasible in 20 years.

Scenario 2: PDAMS bear the costs for the transportation and offtake phase

This scenario Arsari bears the cost of the storage lake phase: \$114 million and the PDAMs will finance the transportation and offtake phase: \$198 million. Therefrom, the below portrayed figures 9.7 and 9.8 can be derived.

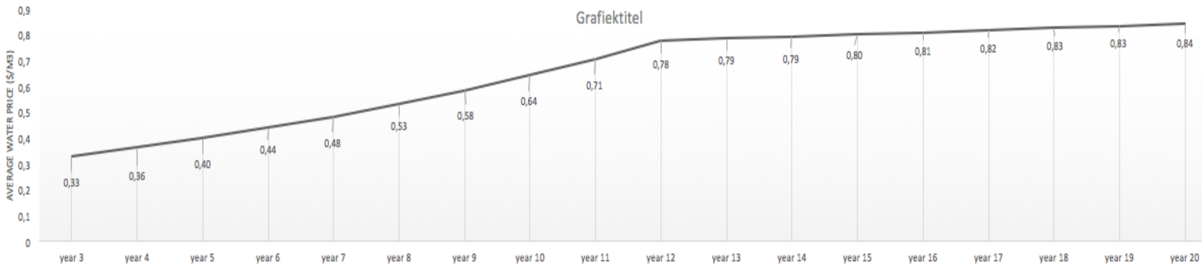


Figure 9.7: Average water price with a 10% increase per year breaking even in year 12

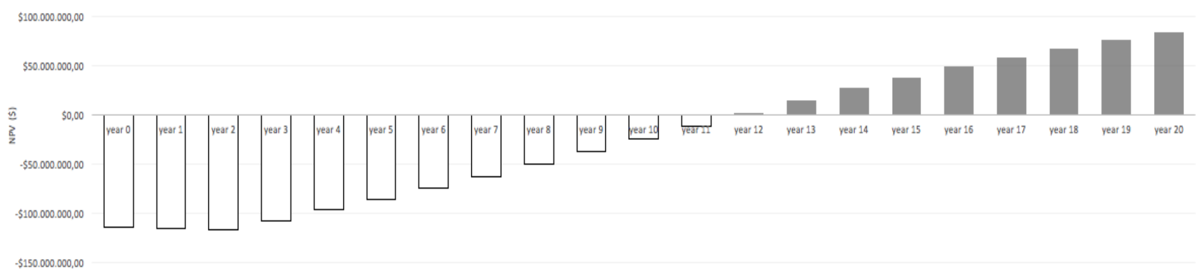


Figure 9.8: Time span of 20 years with a break even point in year 12

With a starting point of an average water price of \$0,33 per m^3, a 10% yearly increase before breaking even, a 1% increase after a positive NPV and with a discount rate of 13,1%, it can be concluded that the project is paid back in year 12. In year 12, the average price of water is \$0,78 m^3 and increases with 1% per year till \$0,84 per m^3 in 2035. Bearing in mind, that these average water prices are what the consumers pay, therefore, this analysis makes the assumption that the PDAMs will not calculate their financial expenses onto the customers. Consequently, when the PDAM will bear the costs of the latter two phases and will not calculate the prices onto their customers the project is feasible for Arsari Enviro Industri.

9.3.3 Conclusion of the break-even analysis

In this section two NPV-models were constructed: a model with a payback period of 10 years, wherefrom the average yearly price is derived and a model where the price will serve as input with the calculation of the payback period with a maximum of 20 years. Both models use the discount rate of 13.1% calculated with the use of the historical average of the interest and inflation rates. Besides that, each model portrays two scenarios: Arsari bears all the investment costs and the PDAM bears the cost for the transportation and offtake investments. The current average price the customers of the PDAMs pay is \$0,33 m^3 and as Indonesian law states, it can be increased with 10% yearly. However, in the net present value model an increase of 1% is incorporated after the net present value is positive. A 10 year payback period was requested by the client, however, from this analysis one can concluded that the project is not feasible within this timespan. Therefore, a NPV-model with a maximum timespan of 20 years is performed. In the scenario where Arsari alone invests in the storage lake, transportation and offtake, the projects net present value is -\$38 million, hence, not feasible within 20 years . Furthermore, in the scenario, where Arsari only pays for the storage lake and the PDAMs for the tranportation and offtake, the project breaks even in the twelfth year at \$0,78 per m^3 and increases with 1% to \$0,84 per m^3 after 20 years with a net present value of \$115 million. Important to note that the costs incurred by the PDAMs are not calculated onto the customers in this analysis.

9.4 Sensitivity and scenario analysis of the NPV-model

The net present value models are based on a variety of assumptions that could alter in reality, therefore, a sensitivity analysis is performed. As the variables could differ in the real world the sensitivity analysis obtains certain scenario characteristics. Through this analysis, the impact on the results by altering several crucial input variables is apprehended. This analysis will make use of the 20-year net present value model where Arsari bears the total investment costs.

9.4.1 Sensitivity analysis variables

The following input variables are adjusted to analysis the sensitivity of the NPV-model. Below are the variables depicted with their current value.

- Nominal discount rate: 13,1 %
- Inflation rate: 5,6 %
- Interest rate: 7%
- Production capacity: 1,66 m^3/s
- Average water price (starting point): \$0,33 per m^3
- Growth of the price till break even: 10%

The above-mentioned input variables will be altered with a range of $+/-$ 25%. Furthermore, the impact will be tested on the net present value that is conceived in year 20 and in what year the project breaks even. This analysis will change the appointed input variable, whilst keeping other input variables constant. The calculations not only show the relationship between output and input but it also tells how sensitive output is to each input.

9.4.2 Sensitivity analysis

In table 9.6 the effects are depicted of the changed input variables on the output variable: net present value. The current value of the net present value in year 20 is - $38 million with a average water price of $1,67 m^3/s.

Table 9.6: Sensitivity analysis where the current values are altered $+/-$ 25%

	Current Expected Value	**Changed Value**	ΔNPV[$ $\cdot$ 1000]	ΔNPV[%]	**Break-Even [year]**	**Price in year 20** [$/$m^3$]
Nominal Discount Rate [%]	13,1	16,375 9,825	- 121.212 83.643	-220 +320	>20 18	1,67 1,41
Inflation Rate [%]	5,6	7 4,2	- 38.096 - 37.953	-0,1 +2,4	>20 >20	1,67 1,67
Interest Rate [%]	7	8,75 5,25	- 76.050 28.465	-100 +75	>20 20	1,67 1,67
Production Capacity [m^3/s]	1,66	2,075 1,245	22.630 - 96.976	+159 -155	20 >20	1,67 1,67
Average Water price [$/$m^3$]	0,33	0,4125 0,2475	40.764 - 119.130	+207 -213	19 >20	1,91 1,25

Nominal discount rate

As seen in table 9.6, changing the nominal discount rate hugely affects the net present value. The nominal discount rate in this model is dependent upon the inflation rate, by keeping the inflation rate constant, the cash flow is inflated with the same pace as the current model, therefore not correcting it, and thus, resulting in a huge effect. Subsequently, it is of utmost importance to have a trustworthy and well-calculated discount rate incorporated in the NPV-model.

Inflation rate

The inflation rate has little to no effect on the NPV-model. This is due to the fact that when the inflation increases or decreases both the discount factor and the cash flows are corrected. Thus, the NPV-model is not sensitive with respect to the inflation rate.

Interest rate

The same goes for the interest rate as goes for the nominal discount rate. By alternating the interest rate, the discount rate changes, but the inflation rate stays constant. Thus, not correcting the cash flow accordingly. As seen in table 9.6, this also greatly affects the outcome of the NPV-model.

Production capacity

A 25% increase and decrease in production capacity has a large effect on the net present value. As this has a direct link to the most revenue being made. Important to note, in reality an increase in production capacity through addition of phase 2b and the overspilling/connection with phase 1 does not only lead to an increase in production capacity, thus, revenue, but inherent to addition of the two water sources is an increase in investment costs. Not only investment costs will be incurred for realizing both additions, however, also the cost of the current analysed infrastructure, like the water treatment facility, hydro power plant and the pipelines will increase.

Average water price

The average water price of \$0,33 per m^3 is a known and fixed amount in 2017. It is unknown when the project will begin to make revenue and, also, when the PDAMs will increase their water prices with 10%. Thus, changing this value and see what affect it has on the NPV could be beneficial. By increasing the starting average water price a net present value increases with 207%, which is beneficial as it breaks even in year 19, however, due to a 10% yearly increase the price after 20 years is \$1,91 per m^3. Which is unrealistic as it increases with 570% in 20 years. A decrease in price is unrealistic as well, as the PDAMs would not lower their price. It yields the same information as from an increase in starting price, being it has a large effect on both the net present value as the eventual average water price.

9.4.3 Conclusion of the sensitivity and scenario analysis

By increasing and decreasing certain input variables by 25% it puts forward the effect is has on the net present value. Several conclusions can be drawn from this analysis. The effect of different discount rates and interest rates on the net present value is large, this is partly due to the fact that the inflation rate stays constant over time, thus, cashflows are unchanged in respect to the original NPV-model. Mostly, it gives a good indication that the discount rate is crucial in a NPV-model. Furthermore, the effect of the inflation rate is examined, as the inflation rate both influences the cashflow as the discount rate, no significant changes became apparent. Additionally, the production capacity is changed with $+/-$ 25%. The production capacity will increase as addition of two water sources will be realized, however, the capacity and costs have yet to be defined. The effect of changing the production capacity is high, as the revenue is directly linked to it. However, as mentioned, in reality increasing production capacity will also increase the investment costs, which is currently unknown. In the self-made NPV-model the costs of the infrastructure as the water treatment facility and the hydro power plant change automatically, however, the pipeline infrastructure as it is dependent on flow through, water pressure and other characteristics has to be changed manually. Moreover, the average water prices are altered with $+/-$ 25%. Although, the current average water price is known, it is unknown when the project will start and at what average water price. Deriving from this alteration, it can be concluded that is has not only a large effect on the net present value, but also on the average water prices are affected. With a higher starting price the project breaks even in year 19, however, the price after 20 years becomes \$1,91 per m^3, which is unrealistic.

9.5 Financial Model

As mentioned before the financial analysis performed for this project has been based primarily on assumptions and estimates. This is due to the limited amount of useful data available on (total) production capacity, statistical data about the stakeholders (inhabitants, industry, etc.), current state of production (PDAMs) and so forth. To ensure the client can use the research provided in this report for an extend period of time until new methods are needed a financial model has been created. This model, an Excel dashboard, has been created for a financial assessment of the Samarinda Water Project. The model is created to ensure that a connection to phase 1 or phase 2, lake B can be directly implemented and used for calculations. In the following section the tool and its use will be explained . *The tool itself is send to the client and the TU Delft supervisors.*

9.5.1 Dashboard

The main page of the model is the project *Dashboard* which gives an overview of the main in- and outputs of the model. Two NPV models are present on the Dashboard. The first NPV model has the water price

as output for a payback period of 10 years. The second model has, among others *(listed below)*, the water price as input variable. All project specifics can be found on the dashboard and several important project inputs, grey cells, can be changed and selected on the dashboard.
The available inputs on the dashboard are:

- Type of Water Treatment
  - *Regular Water treatment plant* (WTP) or *Biochar Water treatment plant (Biochar)*
- Financial ownership of transportation and offtake phase
  - *Arsari* or *PDAM*
- Nominal Discount rate
  - *Several options exist and can be changed by the user*
- Total Offtake
- Production capacity
  - *Phase 1*
  - *Phase 2 Lake A*
  - *Phase 2 Lake B*
- The second NPV model with water price as a input variable, which has the following inputs.
  - *Water Price*
  - *NPV time period*
  - *Discount Rate*
  - *price growth until break-even*
  - *Price growth after break-even*
  - *Offtake*

9.5.2 Break-even analysis

The second sheet is the input sheet for the break even analysis. This sheet shows the same two NPV calculations as described above, with all possible inputs. This sheet is used to calculate the average price of water using the solver as described on the Dashboard of the model.

9.5.3 Investment cost

The investment cost sheet is used to calculate the direct costs, allowances, known costs, contingencies, remaining engineering costs, total costs and the P50 estimates. This sheet works with information derived from the next sheet *Investment cost specifications*. The investment cost sheet has not direct inputs. In case additional cost items are identified, they can be added on row 16, 22, and 27.

9.5.4 Investment cost specification

In this sheet all investment costs are specified in high detail including possible addition of phase 1 and phase 2 lake B. Users can enter inputs on quantity and unit rates. It must be noted that cost items such as the pipeline are dependent on hydraulic characteristics such as production capacity which influences the type op pipe. This is not automated and should be entered by hand.

9.5.5 Operation & maintenance cost

The operation & maintenance sheet allows for separate specification of O&M costs per phase and per Asset *(Dam, Pipeline, Hydro Power, etc.)*. Currently the O&M costs are specified at 4.5% of the investment cost of the artificial storage lake phase *(see section 9.3.1)*.

9.5.6 Production estimations

Currently, the production estimations are derived from the total water supply capacity calculated in section 6.2.1. As soon as more or other information becomes available about the supply capacity of phase 1, phase 2 lake A or lake B this can be entered on this sheet.

9.5.7 Average price of water

The average price of water sheet is currently not being used. Due to the lack of specific data, not demarcation is made between industrial customers and non-industrial customers in the price of water and offtake estimations. This sheet allows for this demarcation to be made as soon as the information becomes available so it can be used as input for price schemes, break-even calculations, etc.

9.5.8 Offtake estimations

The offtake estimations sheet the seperate and combined offtake of Tenggarong and Samarinda is derived from the*estimated Total Urban population of East Kalimantan*, taken from (Badan Pusat Statistik Indonesia, 2013) and the PDAMs installed capacity of 2016. Similarly as with the average price of water sheet, inputs are available to demarcate between industrial and non-industrial customers when this data becomes available.

9.6 Conclusion of the financial analysis

The focus of the financial analysis is threefold: firstly, the demand of clean water of the cities of Samarinda and Tenggarong are forecasted and compared with the current production capacity of phase 2A i.e. the clean water supply. Secondly, the total investment costs are calculated and eventually estimated by using the P50-estimate. And finally, break-even analyses are performed to obtain if the Samarinda Water Project will be financially viable.

Several conclusions can be drawn from the financial analysis. Through the calculation of the future clean water demand of both cities it became apparent that the supply of phase 2A will not suffice. As the current demand is 3,0 m^3/s and the production capacity of phase 2A is 1,7 m^3/s. In 2035 the combined demand of Samarinda and Tenggarong is forecasted on 5,0 m^3/s. This gap will most likely not be fulfilled by addition of the two other potential water reservoirs. Thus, alternative clean water sources have to examined to gratify the full demand of Samarinda and Tenggarong. Furthermore, the amount in capacity of phase 2B and the overspill/connection with phase 1 has to be obtained to know the whole production capacity Arsari is able to supply.

In addition, the total initial investment costs are calculated and estimated. The costs are calculated on the basis of the direct costs i.e. equipment, materials and labour plus the added costs i.e. allowances, indirect costs, contingency and remaining/engineering costs. With the use of the P50-estimate the total investment costs are estimated by incorporating optimistic and pessimistic values of the actual costs in comparison with the estimated costs based on similar projects. With a production capacity of 1,7 m^3/s, the investment is estimated to a total of $312 million with a standard deviation of $47 million. Two methods in this analysis take risks and uncertainties into account: contingency and P50-estimate.

Moreover, a break even analysis is performed with two NPV-models. An NPV-model with a 10-year payback period with the average water price as output and a NPV-model where the average water price serves as input starting at \$0,33 per m^3 with a yearly increase till break-even of 10% according to Indonesian law with a maximum timespan of 20 years. Two scenarios are incorporated in both analysis: Arsari bears all the costs and the PDAMs pay for the transportation and offtake phase. From this analysis it became apparent that in both scenarios a 10-year payback period is unrealistic as the average water price is too high for the customers or it is not conform with a 10% yearly increase stated in Indonesian law. In the NPV-analysis with a maximum timespan of 20 years the scenario where Arsari bears all the costs is not realistic, thus, not feasible as the project has a net present value of -\$38 million in year 20. However, when the PDAMs pay for the transportation phase and offtake phase, a sum of \$ 198 million, thus a sum for Arsari of \$114 million, the break even point is in year 12 with a net present value of \$115 million and with an average water price of \$0,84 per m^3 after 20 years. Bearing in mind, that the PDAMs do not calculate the price onto the customers and, also important to note, that the sum which the PDAMs will incur could perhaps also be invested in other alternatives for a clean water supply. To conclude, when Arsari bears the costs of the whole project, the project is not feasible, the PDAMs pay for the transportation and offtake phase the project breaks even in 12 years considering the previous mentioned conditions.

Finally, the calculated production capacity of 1,7 m^3/s is not the maximum production capacity Arsari could offer the cities of Samarinda and Tenggarong. The production capacity will be increased by addition of the phase 2B lake and a connection/overspill of phase 1. The production capacity is directly linked to the revenue made by selling the clean water. A financial model is set up to insert the additional production capacity and the supplementary costs where the net present value analyses and results adjust accordingly. This tool is useful for future usage as the quantities are easily inserted and yields information about the financial feasibility of the project instantly. Almost all production capacity related activities, such as the water treatment facility and the hydro power plant, alter automatically in the model. However, the pipeline infrastructure investment costs have to be altered manually as it depends upon multiple factors and could, therefore, not be implemented in this financial model. Besides the above-mentioned alterations, other input variables could be changed as well. Such as, the discount rate, starting average water price, the yearly average water price increase before breaking even and after breaking even and what party (Arsari or PDAM) will pay for what cost items.

Chapter 10

Stakeholder Engagement

In networks, stakeholders are dependent on each other in complex and intricate ways. These dependencies can be a risk for the project but can also offer opportunities (de Bruijn, 2008). In the Samarinda Water Project several risks for the feasibility can be identified. The following chapter will describe different possibilities for approach of these risks. Two different approaches were assessed on their applicability, project based and process based. The entire assessment can be found in Appendix G.1. Recommendations for the engagement in successive phases are given in Appendix G.2.

10.1 Feasibility issues

The next section will elaborate on the risks found in the stakeholder analysis, financial analysis and risk and opportunity analysis and which can be solved through stakeholder engagement. A short list of these risks is given below to give an overview:

Stakeholder Analysis

- Different problem perceptions between two sub-networks of critical stakeholders. One of these sub-networks has a strong hierarchical structure and a high probability for collaboration.

Financial Analysis

- Large difference between the Average Water Selling Price of the PDAMs and the Estimated Production Cost of the client.
- The highest investment needed is for the transportation phase and the client indicated they prefer that the governmental institutions take operational and financial responsibility for the pipelines.

Risk and Opportunity Analysis

- International grants and investments and private party collaboration could be an opportunity for the financial feasibility. Additionally, private party involvement can lead to an increase in project and performance and willingness to collaborate.

10.2 Stakeholder engagement

Different problem perceptions

In the stakeholder analysis it was found that several government institutions do not officially recognize pollution of the current clean water sources, which leads to several implications.
Firstly, there currently is no basis for collaboration with all governmental stakeholders. Individual governmental institutions will not collaborate on a project for a new clean water source without first

recognizing the current sources are polluted.
Secondly, because there is no basis for collaboration, important data regarding factors such as the expected future offtake, future average selling price of water, financing possibilities (subsidies, public-private partnerships) and user statistics (inhabitants/industry distribution) are not available. This leads to uncertainty regarding the commercial viability which, until access to the necessary data is provided, is based on assumptions and estimates that cannot be validated.
Thirdly, without recognition of a pollution problem, no intent for offtake exists which in turn means that no interest participation in the Samarinda Water Project exists. Additionally, these critical stakeholders form a sub-network with a high probability of collaboration and a strong hierarchical structure and individual stakeholders within this network have resources on which the project is highly dependent.
Although no official acknowledgements has been given, several institutions do acknowledge a possible (future) problem with pollution in the Mahakam river. An example of this acknowledgement is the request made by the Governor of Kukar to the client to look into possibilities of new clean water supply and the "state of the Environment" report of the Ministry of Forestry and Environment which stated the rivers in East Kalimantan are heavily polluted.

Suitable approach for the different problem perceptions

The starting point for the engagement approach of this risk is the lack of a unilateral problem perception. The advised approach for this problem is the process approach focused on unilateral recognition of the pollution while ensuring all critical actors are willing to collaborate in future stages of the project. Given the client's strong network of relations and the fact that the Governor of Kukar and the mayor of Samarinda have asked for help it can be expected that the provincial government and the municipality of Samarinda are willing to cooperate. The strong hierarchical structure of the governmental network can be used to create a situation in which the PDAMs of both cities, the District of Kukar and the Municipality of Samarinda agree on information and sources about the current state of the clean water sources and how this will develop in the coming years. Water quality can be examined and defined by experts in an objective way with exact results, leaving no room for stakeholders to interpret the information in any other way (de Bruijn, 2008). In this approach it is important to ensure all parties agree on who gathers the data, how they gather the data and how the development of the water quality is assessed. When information and the method of interpretation of this information is agreed upon upfront, resistance of stakeholders, as described in section 8.3.2, becomes unlikely.

Average water selling price

The financial analysis showed that with the current investment cost, production capacity and the PDAM average water selling price the project is not financially feasible. Several steps can be taken by the client to lower the cost of production. For now, the financial feasibility is calculated for for the entire transportation phase infrastructure and solely the production phase infrastructure for lake A. This leads to a high production price and large difference between supply and demand. The production capacity can be increased by including lake B and/or a connection to the Arsari lake of phase 1, leading only to the added investment cost of the production phase infrastructure for lake B and/or the connection to lake Arsari and larger dimensions for the pipeline. The current price of the PDAM is, as the client indicated, on the low side of the national average and with an increase of 10% per year, which is the maximum price increase they can legally apply, a large difference between both prices remains a problem for the financial feasibility of the project. With a price increase another stakeholder group will become involved and critical, the industrial and non-industrial clients of the PDAMs who can be opposed to a price increase, especially because there is a probability the price increase precedes the improvement in water quality. For the non-industrial customers a subsidy system is in place since 2006. This system works with customer classification based on economic background. Lower income customers are compensated through cross-subsidies, meaning the higher income customers pay the a higher tariff in order for the lower income customers to pay a lower tariff (The Water Dialogues, 2009). Because cross-subsidies only distributes costs among customers, a long lasting price increase is still likely to be ill received. Because, depending on the payback period, the deficit is likely to remain even with a price increase of 10% per year, and a price increase is likely to be ill received and create problems for lower income customers the price increase should be subsidized, by governmental institutions or investors. There are several possibilities for subsidies. The first possibility is that the regional governments allocate budget to subsidize the water price increase. It is not known what the current priority of the clean water supply in the project area is among regional governments. With the implementation of decentralisation of governmental functions in 2001 the responsibility of clean water supply to the inhabitants has been fully relocated to the regional governments (of The Republic of Indonesia The Hague, 2013). Nationally it has been found that due to competing priorities, large debts among PDAMs, small budgets for large necessary investments and the abundant presence of both social and technical difficulties in the process of clean water supply the regional governments give a low priority to clean water and sanitation (ASB, 2016; for International Development, 2009). However, in 2010 a new bureaucratic reform was implemented to increase the national government's role as regulator and facilitator (ASB, 2016). Part of the reform is a national Integrated Water Resources Management plan which created parliamentary responsibility in providing political and financial support in the form of budget allocation for improvement projects (ASB, 2016). As found in the risk and opportunity analysis additional possibilities for financial support lie with private party participation such as Public Private Partnerships with national organisation. International grants provided by multi-nationals such as the World Bank, Unicef and the Asian Development Bank and countries such as Japan, Australia, the United States of America and the Netherlands whom are already involved in water access and water supply improvement projects in Indonesia (ASB, 2016; for International Development, 2009; The Water Dialogues, 2009).

Suitable approach for the average water selling price

As described above, the problem for the price increase is rather complex and involves a multitude of stakeholders, directly and indirectly related to the project. To facilitate for this complexity and the large number of stakeholders a process approach is advised. Through a process approach, a large variety and number of stakeholders can be involved and included in finding the best solution because it allows for a broad problem definition (de Bruijn, 2008). The involvement of more stakeholders will increase complexity, which is problematic when using a project approach (de Bruijn, 2008). Raising the complexity has two main advantages. A more attractive decision making process because there are more problems looking for a solution, this lead to more opportunities for collaboration and deal making (de Bruijn, 2008; Hans de Bruijn, 2010). Next to that, it creates more room manoeuvre for stakeholders which entails they can change their point of view and problem perception without 'losing ground' (de Bruijn, 2008). Lastly, with a process approach goals can be formulated during the process and the scope can evolve (de Bruijn, 2008). This does however entail more control has to be exercised on the process by the client.
For the actual engagement of stakeholders it is advised to start with a broad problem definition in order to create distance between the client and the price increase issue to ensure the PDAMs take responsibility. When a stakeholder takes ownership of a risk, it becomes difficult for them to distance themselves from it later on in the project (Hans de Bruijn, 2010). Additionally, they are the party in the best position to solve the price increase issue. To ensure this responsibility is accepted, a sense of urgency is necessary. This sense of urgency will arise from the recognition of pollution. This creates a context in which the governmental stakeholders are sensitive to a particular problem perception, the necessity of a subsidized price increase to ensure financial feasibility without the non-industrial customers paying for the deficit (de Bruijn, 2008).

Operational and Financial ownership of the transportation phase

The client has made it clear that they want the regional government and PDAM to take operational and financial ownership of the transportation and offtake infrastructure. It is sensible that the PDAMs take operational ownership for the infrastructure. Operation and maintenance of clean water distribution infrastructure is one of their main activities and falls within the scope of their day to day business. This makes them the most knowledgeable and competent party. For the financial ownership several risks are present. As mentioned in subsection Average water selling price, many PDAMs struggle with high debt (ASB, 2016). It is not known what the financial condition of the regional government and PDAM of

Samarinda and Tenggarong with respect to large infrastructure investments is. Considering the size of the estimated investment, it is unlikely it is readily available. Furthermore, in their Indonesian Water Assessment the Asian Development Bank (ASB, 2016) found that, nationally, clean water supply often has a low priority.

Suitable approach for the operational and financial ownership of the transportation phase

Ownership of operation and maintenance should be a put forward as a process requirement by the client. It is clear that it should be the responsibility of the PDAM when considering the additional investment, yearly cost and risks of procuring, educating, controlling and monitoring employees it if would be done by the client. If the PDAMs or the regional governments refuse to accept this as a requirement they can be persuaded through compensation (de Bruijn, 2008). An example of compensation is that the client provides provides the technical knowledge for the construction of the pipeline, in addition to already having the financial and operational ownership of the storage lake phase. For the financial ownership, the arguments are less evident. From a process design perspective, no arguments can, and should, be made without thorough financial analysis based on exact and final numbers as backing. Currently, such an assessment is not possible and exact predictions cannot be made. Therefore, the advised approach is to commission a financial expert to create financial scenarios based on the detailed engineering design. From these scenarios, question about the financial ownership and which party is most capable to bear the costs of investment can be answered. With these arguments an approach can be chosen to ensure a process outcome that remains true to the value's, goal and interests of the client.
Two suggestions for this approach can however already be made. The first is to create more value for the money invested. Value adding is the practice in which a project is reframed and the scope of the project is expanded to create sufficient political and societal support (Hertogh, n.d.). In light of the main practices and values of the client, extra value for the Samarinda water project could be environmental protection and education. The client could commit to investing parts of the profit created with the project in environmental protection and education projects in the district of Kukar and the municipality of Samarinda.
A second suggestions is to explore the possibilities for a Public-Private Partnership (PPP) between the client or other private organisations and the regional government. The application of PPP in clean water supply has obtained more interest over the past few years due to poor performance of the governmental water sector on cost recovery, cooperation between regions, creditworthiness of local governments, project preparation and execution and infrastructure operation and maintenance (ASB, 2016; Roesly, 2014). PPP can be applied to the entire project or individual phases and with a commitment to increase performance through complementary collaboration, financial and political support of the government becomes more likely (ASB, 2016; Roesly, 2014).

10.2.1 Conclusion

Because of the dynamic context of the project, the level of uncertainty, the difference in problem perceptions and the requirements of the client, such as the operational and financial ownership of the transportation and offtake phase a process approach is advised. A process approach allows for difference of opinion at the beginning of the project without withholding progress. Through the process approach, the parties can agree on a problem definition without agreeing on a final solution while maintaining the incentives for cooperative and solution seeking behaviour. Alternatives can be discussed and decided on through incremental growth and based on information obtained during the process instead of information agreed upon at the beginning, leading to high quality alternatives. It is assumed an urgency for cooperation will present itself when the pollution of the raw water sources is acknowledged. This is a valuable incentive for progress and a valuable tool for negotiation for the client. It is however advised to start engagement before the pollution of the raw water sources creates a need to fast track the project because this can negatively influence the behaviour and decision making of the governmental institutions responsible for the clean water supply.
Three main risks for the feasibility of the project were identified in the stakeholder analysis, financial analysis and the risk and opportunity analysis. Important to note is that the engagement of these risks

is based on the current state of the project and this should be re-evaluated based on the stakeholder characteristics when the actual engagement will commence. Furthermore, we recommend that as the the project progresses into more detailed phases, more explicit methods of analysis and engagement are used, such as Active Threat and Opportunity Management (ATOM), Core Value Analysis and Engagement and Actor-network analysis.
Because of the absence of critical financial information, no direct engagement can be set out for the financial ownership of the transportation and offtake phase. It is however recommended to look into the possibility of private party collaboration within this ownership due to the benefits regarding project performance and governmental support. The scope of this engagement was the feasibility of the Samarinda project. For successive phases several key elements of successful stakeholder engagement have been set out, this can found in Appendix G.2. It is recommended to ascribe priority and necessity of these elements based on the criticality of the involved stakeholders as the project progresses. This prevents process deadlocks and deviation due to low interest stakeholders given to much influence over the project. As the project progresses into design, construction and operational phases a shift towards a project based approach is advised because the need for social management decreases and the need for strict requirements, robust and predetermined knowledge and project goal realisation increases (de Bruijn, 2008; Hans de Bruijn, 2010).

Chapter 11

Risk and Opportunities

Several risks and opportunities have been identified during this study. Several issues, risks with a probability of 100%, have been identified and discussed in the previous chapters as well. The following chapter will list and shortly describe the risks found that are critical, i.e. a high probability or impact or a medium probability and impact. Mitigation measures are described with each risk. Many risks resulting from insufficient data or possible poor execution are present. These will however not be elaborated on in this chapter, simply because they can be mitigated by carefully and thoroughly executing the tasks presented in the recommendations. The risks and its mitigations are divided into six different categories. Namely, Political, Economical, Social, Technical, Environmental and Legal. The complete table of the identified risks can be found in Appendix H.

11.1 Risk response

As can be seen in the risk register 4 approaches are used. These four approaches, table 11.1, include all possible ways a risk or opportunity can be engaged.

Table 11.1: Possible risk approaches

Risk	**Opportunity**
Accept	Accept
Reduce	Enhance
Transfer	Share
Avoid	Exploit

Accept entails that, for now, nothing will or can be done about the risk or opportunity and that it will be monitored until action is needed. Reducing or Enhancing means action will be undertaken to change the probability of impact of a risk or opportunity. Transferring or sharing involves the risks being transferred to someone best capable of bearing it or the opportunity being shared with other parties. Avoiding a risk means to ensure the probability becomes zero by for instance doing something completely different by building somewhere else. Exploiting means ensuring the probability becomes 100% to ensure the opportunity is seized (Nicholas, 2008).

11.2 Critical risks per category

The following section describes the critical risks found per category (PESTEL).

Table 11.2: Critical political risks

Risk	Mitigation
Because the PDAMs conduct the official test for water quality and they currently deny pollution levels exceeding government standards the governmental institutions may not have interest in the project.	Use Independent experts to prove the pollution has reached harmful levels for consumers and use this to apply pressure for acknowledgement.
Because of the strong hierarchical structure of the Indonesian government, hierarchy may be used to force subordinates which can lead to poor substantive decision making impacting project quality.	Adapt the process design to combat hierarchical pressure. For instance through negotiated knowledge and equal influence in decisionmaking.

Table 11.3: Critical economical risks

Risk	Mitigation
The Samarinda Water Project requires a high investment and a cheaper alternative might be found which would make the Samarinda Water Project superfluous .	Conduct alternatives research, both for supply and demand and monitor other offtake possibilities to ensure relevance of the project.
The Samarinda Water Project requires a high investment and the Regional government or PDAMs might not be able to afford this investment.	Create political support on a national level and/or involve private sector parties.

Table 11.4: Critical social risks

Risk	Mitigation
Misaligned problem perceptions can lead to an arduous collaboration which negatively affects the project outcome.	*See "stakeholder engagement: different problem perceptions".*
A sharp price increase will take place which the customers of the PDAM might not agree to leading to public uproar and project failure.	*See "stakeholder engagement: Average water selling price". And "stakeholder engagement: Financial ownership".*
Several stakeholders have different interest, they may pursue their own interest which negatively affects process progress, quality and trust.	Ensure all stakeholders agree that the joint end goal is more important than individual interests and install incentives for collaborative behaviour.

Table 11.5: Critical technical risks

Risk	**Mitigation**
Seepage occurs at dam foundation and this might rise above the maximum seepage levels leading to hydraulic failure of the dam..	Perform thorough and repetitive analysis of the subsurface and soil to find and counteract seepage at the dam foundation.
Because water levels in the reservoir differ, construction materials might lose buoyancy which leads to structural failure of the dam.	Remove top, weak soil, and build on the stronger siltstone formation.
Because shale has a maximum carrying capacity, adding extra weight might cause the shale to rupture leading to structural failure of the dam.	Remove Shale.
The area is prone to earthquakes which can lead to structural failure of the dam.	Strengthen dam construction.
Due to decreasing of cohesion in the slope, slope instability might occur leading to structural failure of the dam.	Remove the top soil layer.
Due to the forming of Karst seepage may occur which endangers the technical feasibility.	Fill up the Karst connections.
The Dam, Water Treatment plant and hydro power are planned in a remote area and emergency response time could be very long due to bad weather and time of day.	Create better infrastructure to the plant to cut travel time and emergency response time.
Mudslides occur in the region which might destroy the pipeline.	Ensure the pipeline route is outside of mudslide prone areas.
Due to slumps in the rock layers, layers may settle differently which may cause structural failure of the tunnel.	Enforce tunnel structure and research rock layers to find the best tunnel route.
Due to climate change the precipitation patterns change Less rainfall might occur leading to depletion of the reservoir.	In depth study of future weather patternt to adapt the design of the reservoir to what is expected.

Table 11.6: Critical environmental risks

Risk	**Mitigation**
Currently no environmental risks or opportunities have been identified. Our recommendation is to perform a thorough environmental impact assessment when a detailed engineering design is available.	

Table 11.7: Critical legal risks

Risk	Mitigation
Many permits are required for a project this size and governmental stakeholders may use their political influence to block permits applications delaying the project.	Ensure mutual collaboration with governmental stakeholders before permit applications.
Many permits are required for a project this size and governmental stakeholders may use their political influence to block permits applications delaying the project.	The client can use their network of relations to persuade superior governmental stakeholders to apply pressure on subordinate governmental stakeholders.

11.3 Conclusion

Several risks were identified during the research for the Samarinda Water Project of which the most critical have listed in this chapter. Most risks found are technical risks, of which many can be mitigated by executing the task they are associated with well and thoroughly. Therefore they do not directly classify as a risk, they were however added because they show where attention is needed in the hydraulic and geological research if the project progresses. Several social and economical risks were identified, of which the most critical have been elaborated on in the financial analysis and the stakeholder engagement. There are some categories, such as Environment and Legal, for which only a small amount or no risks were found. No extensive analysis was performed in these categories, because no motive for further research was present. It is recommended to use this risk analysis as a basis and continuously update the register and monitor the critical risks.

Conclusion

The main objective of this research project was to create insight into the feasibility of the Samarinda Water Project, to find out if further research would be valuable. Several disciplines came together in this research project to ensure the problem, *a need for clean water in the city of Tenggarong and Samarinda*, would be engaged in a broad manner and with thorough research performed in three distinct disciplines. The short answer to the research question for this project, *is it technically and financially feasible to capture water in the phase 2 A catchment, transport it and sell it to the PDAM of Tenggarong and Samarinda?* is maybe. Therefore, a longer answer will be given.
From a technical perspective, taking into account the hydrological, hydraulic and geological research, the project is feasible. No issues without mitigation possibilities were identified for the technical feasibility. Several side notes have been made about the technical research, these have been elaborated on in the report and will be elaborated on further in the Discussion.

From a financial point of view, no final conclusion about feasibility can be given. This was anticipated on during the research and therefore the financial model was created, to ensure further research with conclusive arguments can take place. Two likely scenario's have been researched leading to two conclusions. If Arsari bears the cost for the entire project, with the current researched set-up, it is not feasible. If Arsari bears the cost only for the construction, operation and maintenance of the artificial reservoir, and the rest would be covererd by the PDAM, several (private) parties or subsidized and if Arsari receives the same Average Water Selling Price with a yearly price increase of 10% for twelve years until break-even, it would be feasible. With this scenario it must be noted that without even meeting the total demand such a large investment might be put into cheaper or more easily accessible alternatives.

In addition to the technical and financial feasibility, the soft or social side has been researched. Although several issues were identified from the social side, it was also found that through the engagement several financial issues can be resolved. If the pollution of the Mahakam river is found to be in excess of safety norms there will be a high demand for projects such as the Samarinda Water Project. From a social perspective, despite the issues found in the stakeholder analysis, the project is feasible.

Discussion

The central question in this study was whether it is technically and financially feasible to capture water in the 2A catchment, transport it and sell it to the PDAM of Tenggarong and Samarinda. The client requested a pre-feasibility study, we did, however, deliver a study with the character of a feasibility study. It created great insight in a situation that is hard to understand, since very little data is available. As of now, good estimations and forecasts are made available which clarify the situation on hand. In the discussion not only the relevance and importance of the study is discussed, but also the research is assessed critically to indicate where improvements can be made. First the technical subjects will be adressed, after that the socal issues and finally the financial matters.

The discharge time series of the catchment modelled by SOBEK has not been calibrated yet. Field measurements are important to test the validity of the output of the model. For now, the model output can only be considered an educated estimation. As a result, the extreme value analysis has this uncertainty as well, since it is based on the modelled time series. Early in our study this issue was recognised, therefore, in-situ instruments are installed by the team so in future studies the model output can be validated.

Seepage has been accounted for in the model for the behaviour of the reservoir, although for 'typical' sandstone values. Most of the basin consist of clay and shale which have a lower seepage rate. However, if karst or faults are present at unfavourable locations, major seepage rates can make the reservoir inoperative. Also, only qualitative assessments of the subsurface are the body of the geotechnical study for the dam foundation, outlet works and spillway. This remains an uncertainty as long no thorough geotechnical research is conducted. One must note that the project site is very isolated, largely in unpassable terrain. These in-field measurements will be expensive, labour intensive and complex measurement instruments are needed, all were simply not available for our team.

For the calculation of the dam, the Dutch safety standards are used instead of the Indonesian standards, due to the limited availability an English translation of the Indonesian standards. This has an influence on the calculations of the dam dimensions, the slope, the crest height and the materials which are used. The crest width is estimated with the Indonesian standard to account for the risk of an earthquake event. The quarry nearby the dam site, is expected to have hard rock with sufficient availability and quality, but this assumption has not been validated yet. Both factors impact the dam dimensions and the financial feasibility of the project.

The approach of the water transportation design was that no pumps should be needed for the transmission of water from the water power and treatment cluster to the distribution reservoir to increase reliability and robustness. Large pipelines diameters were needed in order to keep friction rates low. Since the pipelines account for more than half of the total projected investment cost, this approach should be compared to a scenario with pumps.

From a management point of view, the following matters need to be evaluated. Indonesia and Dutch culture vary on many points, this influences our understanding or the way Indonesian stakeholder behave and interpret behaviour of others. Therefore our analysis of and engagement for the stakeholders could be off.

Furthermore, no interviews were conducted with stakeholder apart from the PDAM of Tenggarong who proved to be unwilling to have a realistic conversation about pollution of the Mahakan river. Therefore, most stakeholder perceptions are based on general knowledge of the client company and available data. This might possibly be a result from the difference in culture. That is also the reason why only critical stakeholder have been thoroughly assessed. This does give a skew vision of reality because issues, opinion, perceptions etc. that might become critical later are left out.

Looking at the financial analysis in retrospect, several discussion points arises, which will be depicted below.
The increase in water demand of the cities Samarinda and Tenggarong is based solely on the growth of the urban population of East-Kalimantan till 2035, leaving the industry growth aside of both cities. The growth rate of the population is stagnant till 2035, however, the industry is still growing, giving an upwards effect on the needed water supply. Consequently, the forecasted water demand will be most likely higher in reality.
It was chosen not to use the value-added taxes in the total initial investment costs for further analysis as it was also done in the financial report of Witteveen+Bos. Therefore, in reality a higher investment could exist.
The P50-estimate is based on the difference between the estimated costs and actual costs of similar projects, however, these projects are undertaken in different countries than Indonsia i.e. different environment and characteristics. Thus, the pessimistic and optimistic values could have different values when looking at Indonesian project. Unfortunately, similar Indonesian projects costs estimates have not been found.
Although, by appropriating the full initial investment amount at the start of the project the NPV-model is at a worst-case scenario, it is not realistic. The investments time per cost item should be made explicit as it could be discounted accordingly. This could make the net present value more accurate.
With financing the project, only two scenarios are put forward: Arsari bears all the costs; PDAMs pays for the transportation and offtake phase. Thus, looking at two extremities. No other external investors or institutions that are willing to invest in this project are put forward in the financial analysis.
The full electricity production is incorporated in the net present value analysis as an income. This is partly true, as a part of the produced electricity will be used for own consumption, thus, decreasing the net produced electricity. However, on the other side of the medal, the project is not dependent on external electricity sources, which will have a price tag as well.
In the financial analysis the pipeline construction from the water treatment facility to the reservoir is calculated without the use of pumps as only gravitational powers are used in this analysis. As the pipeline construction is one of the highest cost items, it could have been beneficial to incorporate pumps in the infrastructure as it would change the route and, therefore, reducing the distance, thus, leading to a decrease in pipeline construction costs. Bearing in mind that the pumps will add additional costs (investments and operation and maintenance costs) and consumes electricity yielded form the hydro power plant.
The scenario of a higher production capacity could have been more elaborate by also increasing the investment costs and operation and maintenance costs. Giving a more accurate indication of the feasibility of the project through addition of the phase 2B and the overspill/connection of phase 1.
From the risk and opportunity analysis some noteworthy considerations and observations are mentioned below.
Many issues with incomplete or possibly inaccurate data occurred. As an effect of incomplete data, many of the research is based on assumptions. Although it is clearly indicated which parts are based on assumptions and the assumptions have been substantiated, this does leave room for mistakes. As an effect of possibly inaccurate data, due to time constraints or inaccurate data sets, some models have not been validated. This has been put in the risk register as risks, which in fact does not belong in a risk register. We thought of this as necessary because validating the data and reassuring correct data is used is essential for further research and the outcome of further research is at risks if this is not done.
Due to a lack of expertise, time constraints and research scope no environmental impact assessment has been performed and thus no environmental risks or opportunities have been identified. The client has a high level of expert knowledge in this field and therefore is best suited to perform an environmental impact and risk assessment.

Recommendations

Throughout the report recommendations for further research, improvement of current research and project approaches were made. These suggestions have been summarized and listed below.

- Perform alternatives research. The Samarinda Water Project requires a very high investment. This high investment can be used as a benchmark value for alternatives that can be developed by other organisations or by Arsari.
- Calibrate the SOBEK model. To do this, primarily discharge and precipitation data is needed. The first is to be obtained from the installed diver at the dam location. Install precipitation measurement equipment near the diver, to obtain the second. This will provide reliable data from which a more accurate representation of the actual discharge can be made.
- Perform more in-depth site investigations in order to establish a complete, conceptual geological model. The focus should lie on the validation of seepage values and finding evidence for the presence and character for potential faults.
- Check whether quarry rock for the rock fill dam is indeed available and of sufficient quality as informed by the client.
- Compare the scenario for water transportation without gravity with the scenario with gravity. If pumps are used, smaller pipeline diameters can be used which will be cheaper. Since it is a large portion of the total investment cost, even small advantages in price will have an significant impact.
- Investigate the need to bury the pipelines. If the technical, operative and social risks prove to be low, a considerable amount of money can be saved.
- Investigate the differences between Indonesian and Dutch building standards for dams to find out whether differences for key components exist.
- Obtain knowledge about the growth of the industry of Samarinda and Tenggarong to get a accurate estimation of the future clean water demand of both cities.
- To meet the future demand of the cities of Samarinda and Tenggarong alternative clean water sources have to be sought as the current and (most presumably) future production capacity will not suffice.
- To ensure financial feasibility Arsari needs financial support from other institutions. Most preferably, contribution in the form of subsidies from the government or participation from private parties. Therefore, these stakeholder have to be engaged properly and duly to ensure (financial) participation.
- To reduce the costs of the largest cost items: water treatment facility and the pipeline section 2 alternatives have to be examined. For instance, water purification of biochar and implementing pumps.
- The water prices are based on tariff schemes for specific customers and the offtake of those customers. Therefore, it is not known what the exact offtake is of the inhabitants and what price they are paying, the same goes for the industry and other clients. It would be beneficial to obtain that knowledge for future analysis on how to change the water prices for the customers, so that the inhabitants could still be able to afford it.

- In order for the PDAM to take financial ownership of parts of the project, it is recommended to start with a broad problem definition.
- Use a process approach to handle the dynamic character of the Samarinda Water Project stakeholders and (Indonesian) hierarchy structure.
- To ensure acknowledgement of pollution ensure objective experts set the standard and prove the pollution.
- Operation and maintenance ownership by the PDAM should be put forward as a process requirement by the client.
- The stakeholder analysis should be performed repeatedly throughout to process. After engaging stakeholder, reassessing the engagement is advised.
- Expand, monitor and update the risk register on a regular basis to ensure all risks are known and can be mitigated.

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Appendices

Appendix A

Hydrology Study

A.1 Hydrological analysis

A.1.1 Rainfall data

ITCI rainfall data

The location which was closest to the catchment area with rainfall data was ITCI, the location from where this study was conducted. The precipitation was monitored at ITCI from October 2016 until December 2016. The data will be used for validation of the other two data sets. Other analysis like the monthly averages or extreme highs and lows cannot be derived from this data set. The measuring period was not nearly long enough for this purpose.

BMKG ground station data

The second data set that was used for the precipitation data was from the *Badan Meteorologi Klimatologi dan Geofisika* (BKMG). This station had measurements available from 2000 until 2014, see table A.1 and is located in the city of Balikpapan, almost $60km$ away from the catchment area. The city experiences a different climate because of its significant differences in topography. The catchment area concerned is located in an elevated and hilly rainforest and therefore experiences more rainfall than Balikpapan, an area situated next to the ocean. So, the data from BMKG is can be used for a comparison, but not reliable enough to draw hard conclusion of it.

Table A.1: BKMG ground station rainfall data 2000-2014 [mm]

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
2000	397	221	245	369	431	66	244	126	104	286	190	243	2922
2001	371	338	254	296	99	343	324	251	200	227	382	228	3313
2002	444	301	233	572	139	97	219	90	172	66	243	314	2890
2003	299	134	294	148	145	239	133	282	99	42	320	278	2413
2004	325	132	402	250	441	361	229	405	224	118	180	141	3208
2005	237	320	424	176	198	108	215	2	131	116	251	282	2460
2006	172	232	271	253	259	102	212	151	36	280	271	247	2486
2007	229	375	166	385	245	610	81	94	254	12	122	315	2888
2008	276	258	144	199	250	378	393	199	336	98	88	205	2824
2009	71	224	324	256	259	454	705	309	292	220	346	325	3785
2010	226	169	223	258	258	276	194	108	266	181	113	262	2534
2011	120	191	264	258	132	268	69	90	211	141	-	314	2058
2012	254	136	190	227	103	185	155	122	70	169	188	186	1985
2013	224	388	207	219	220	148	197	333	129	138	282	-	2485
2014	200	98	256	272	147	246	242	187	21	164	146	422	2401

TRMM satellite data

The third precipitation data set is the *Tropical Rainfall Measurement Mission* (TRMM) of exact location of the reservoir. TRMM is data obtained by a satellite in a collaboration of both NASA and JAXA with the aim to study tropical rainfall. A specific satellite is needed because tropical precipitation is difficult to measure, due to the large temporal and spatial variation. The data has a daily temporal resolution and available online. For this report, the daily rainfall from 1998 until 2016 is used of specifically the project catchment area, see table A.2.

Table A.2: TRMM satellite rainfall data 1998-2016 [mm]

	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**	**Total**
1998	40	5	4	96	308	278	224	176	174	182	232	291	2010
1999	196	331	304	190	184	112	130	149	167	288	198	273	2522
2000	183	298	193	300	203	269	95	119	148	207	435	175	2625
2001	390	220	183	272	154	67	115	21	154	147	193	197	2113
2002	152	149	277	187	168	216	83	46	25	98	264	203	1868
2003	288	144	359	242	210	149	75	225	221	180	187	245	2525
2004	272	263	373	236	313	81	148	3	182	29	295	322	2517
2005	218	131	233	245	243	146	184	103	109	270	309	319	2510
2006	184	348	255	261	303	212	21	85	121	66	144	232	2232
2007	317	229	216	311	202	222	223	98	162	164	154	184	2482
2008	125	299	266	332	181	256	274	168	172	195	362	284	2914
2009	235	164	265	246	137	84	102	72	45	184	143	271	1948
2010	233	138	224	248	263	214	282	186	217	274	219	189	2687
2011	219	171	301	309	170	197	108	70	202	158	299	293	2497
2012	356	225	299	220	181	160	171	107	79	146	241	186	2371
2013	196	407	211	295	236	170	134	133	171	153	291	291	2688
2014	260	151	219	207	195	176	84	81	63	96	237	405	2174
2015	342	202	221	322	166	211	63	39	21	48	168	190	1993
2016	157	185	167	276	240	150	155	85	266	11	256	-	-

Validation and comparison

This comparison is shown in figure A.1 andA.2 and table A.3 and A.4

Table A.3: Yearly total comparison

	TRMM [mm]	**BMKG [mm]**	**Difference**
2000	2625	2922	11%
2001	2113	3313	57%
2002	1868	2890	55%
2003	2525	2413	-4%
2004	2517	3208	27%
2005	2510	2460	-2%
2006	2232	2486	11%
2007	2482	2888	16%
2008	2914	2824	-3%
2009	1948	3785	94%
2010	2687	2534	-6%
2011	2497	2058	-18%
2012	2371	1985	-16%
2013	2688	2485	-8%
2014	2174	2401	10%
Total	36151	40652	12%

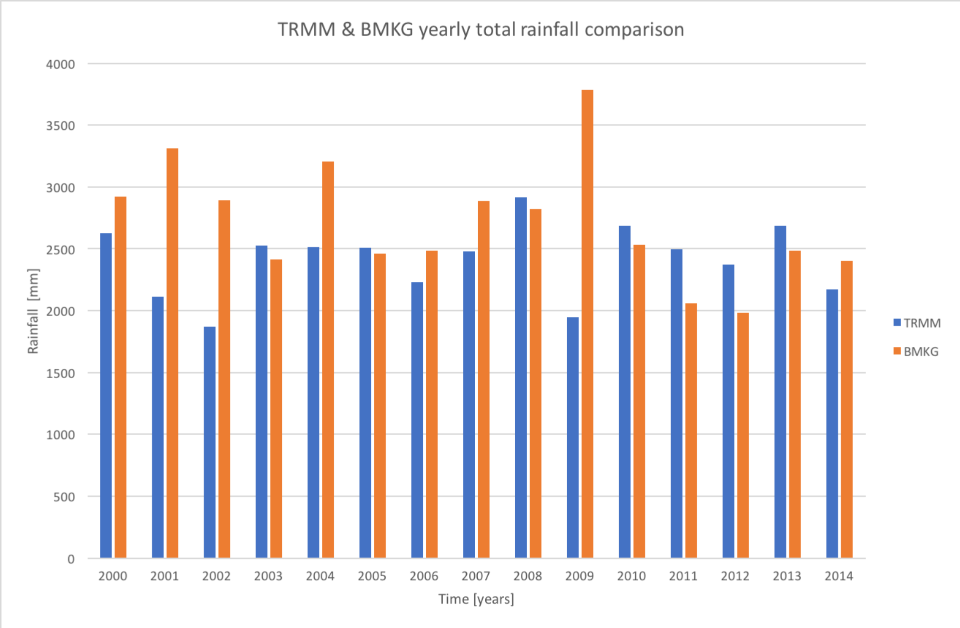


Figure A.1: Yearly total rainfall comparison

Table A.4: Comparison of the TRMM and BKMG monthly averages

	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**	**Average**
TRMM	242	222	258	261	211	175	140	101	138	158	252	253	201
BMKG	256	234	260	276	222	259	241	183	170	151	208	251	226

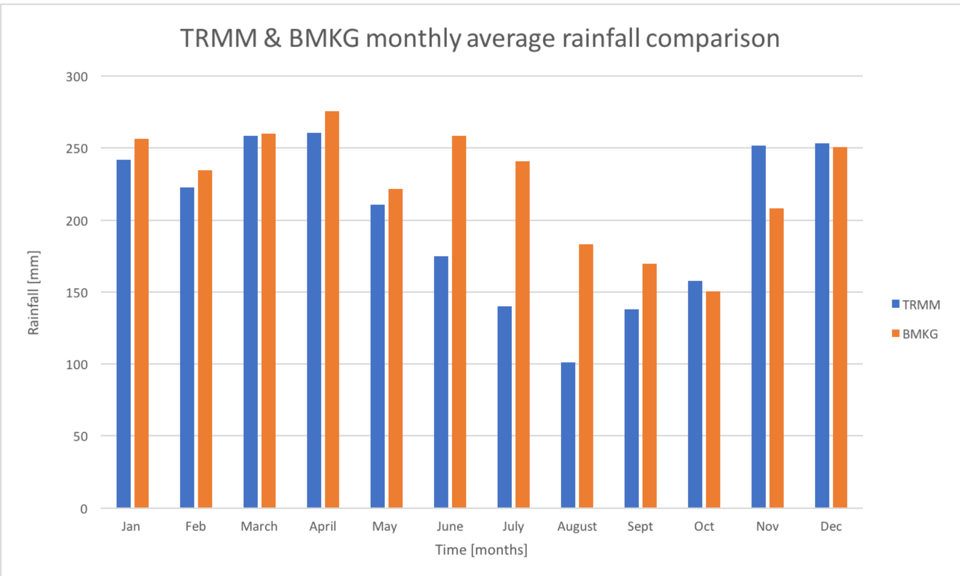


Figure A.2: Monthly average rainfall comparison

As seen in both the figures and the table, a good correlation is visible between the BMKG and the TRMM data. On annual basis, the BMKG is on average 12% higher than the satellite data, with an extreme difference of 94% in 2009. This was a El Nino year, which should indicate a more dry year, but the Balikpapan data shows the opposite, which could indicate a flaw in the data. With this in mind, the TRMM data on annual basis shows good correlation with the BMKG data. The monthly comparison shows that in the dry season, June, July and August, the TRMM data is a bit lower than the Balikpapan data.

Looking at both comparisons, it is not what was expected. Usually satellite data (TRMM) gives higher annual totals and monthly averages than a ground station (BMKG). This can be explained by the fact that satellite data is an average of a larger area, so therefore local peaks are less accurately measured, but overall, it is better able to capture all the rainfall than a ground station. A ground station can accurately describe a peak in precipitation, but because it's small measurement area, it can easily miss large parts of the overall rain, and gives therefore lower averages than satellite data. Although this does not hold for our data, it does not implicate the data is incorrect.

A.1.2 Evaporation

The daily (24 hours) evapotranspiration data can be found in figure A.3. The data set is on a daily basis and is from 1998 till November 2016. The monthly average evapotranspiration can be found in figure A.4.

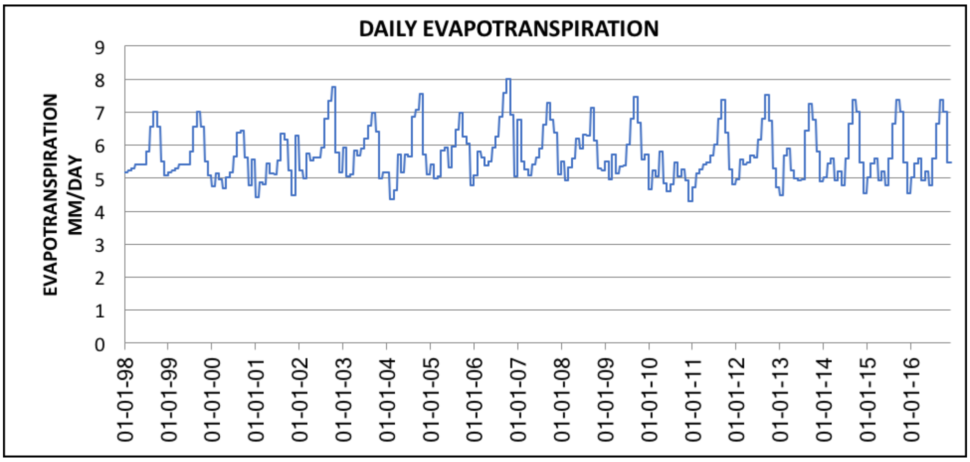


Figure A.3: Daily evapotranspiration data

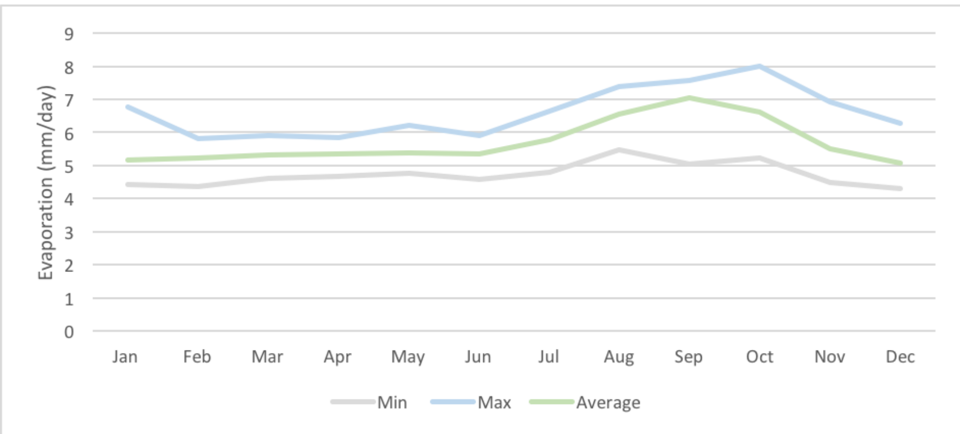


Figure A.4: Montly average evapotranspiration data

A.1.3 Open water evaporation

The open water evaporation rate depends on many physical conditions such as:

- Air temperature
- Relative humidity
- Wind speed
- Radiation
- Total sun hours
- Possible sun hours
- Stefan constant
- Evaporation heat
- Psychrometric constant

The open water evaporation is calculated with the Penman formula:

$$E_0 = \frac{\frac{s*R_n}{\lambda*\rho} + \frac{c_p*\rho_a}{\lambda*\rho} * \frac{e_s - e_a}{r_a}}{s+\gamma}$$

In which

- R_n: net radiation on the surface of the earth [J d^{-1} m^{-2}]
- λ : evaporative heat ($\lambda = 2.45$ MJ/kg) [J kg^{-1}]
- s: slope of the vapor pressure curve [kPa K^{-1}]
- c_p: Specific heat of air at constant pressure (1004 J kg^{-1} K^{-1}) [J kg^{-1} K^{-1}]
- $\rho_a$density of air (1.205 kg/m^3) [kg m^{-3}]
- ρ: density of water (1000 kg/m^3) [kg m^{-3}]
- e_a: actual vapor pressure [kPa]
- e_s saturation vapor pressure [kPa]
- γ: psychrometric constant (γ = 0.066 kPa/° C) [kPa K^{-1}]
- r_a: aerodynamic resistance [d m^{-1}]

Table A.5

Parameters for the storage lake from Witteveen+Bos		
Air temperature	26,7	°C
Relative humidity	0,77	-
Wind speed	3,8	m/s
latitude	0	°C
Sun hours (n)	3,7	hr/day

Saturation vapor pressure (e_s)

$$e_s(T) = 0,61 * exp(\frac{19,9*T}{273+T}) = 3,59Kpa$$

Slope of the vapor pressure curve (s)

$$s = \frac{5430 * e_s}{273+T^2} = 0,22Kpa$$

Actual vapor pressure (e_a)

$$e_a = h * e_s = 2,77kPa$$

In which:

- h: relative humidity [-]

Aerodynamic resistance (r_a)

$$r_a = \frac{245}{0,54 * u_2 + 0,5} * \frac{1}{86400} = 0,0011dag/m = 1,11 * 10^{-6}dag/mm$$

- u_2 is the wind speed at 2m [m/s]

Table A.6: Average short wave radiant energy (R_A) per month for degrees lattitude (Savenije, 2014)

$°\phi$	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sept**	**Oct**	**Nov**	**Dec**	**Aver.**
60	1,4	3,6	7,0	11,1	14,6	16,4	15,6	12,6	8,5	4,7	2,0	0,9	8,2
52	3,2	5,5	8,8	12,0	15,4	16,6	16,0	13,6	10,2	6,7	3,9	2,6	9,5
50	3,7	6,0	9,2	12,7	15,5	16,6	16,1	13,7	10,4	7,1	4,4	3,1	9,9
40	6,2	8,4	11,1	13,8	15,9	16,7	16,3	14,7	12,1	9,3	6,8	5,6	11,4
30	8,1	10,5	12,8	14,7	16,1	16,5	16,2	15,2	13,5	11,2	9,1	7,9	12,7
20	10,8	12,4	12,0	15,2	15,7	15,8	15,8	15,4	14,4	12,9	11,3	10,4	13,5
10	12,8	13,9	12,8	15,2	15,0	14,8	14,9	15,0	14,8	14,2	13,1	12,5	14,1
0	14,6	15,0	15,2	14,7	13,9	13,4	13,6	14,3	14,9	15,0	14,6	14,3	14,5
10	15,9	15,7	15,1	13,9	12,5	11,7	12,0	13,1	14,4	15,4	15,7	15,8	14,3
20	16,8	16,0	14,5	12,5	10,7	9,7	10,1	11,6	13,6	15,3	16,4	16,9	13,7
30	17,2	15,8	13,5	10,9	8,6	7,5	7,9	9,7	12,3	14,8	16,7	17,5	12,7
40	17,3	15,1	12,2	8,9	6,4	5,2	5,6	7,6	10,7	13,8	16,5	17,8	11,4
50	16,9	14,1	10,4	6,7	4,1	2,9	3,4	5,4	8,7	12,5	16,0	17,6	9,9
60	16,5	12,6	8,3	4,3	1,8	0,9	1,3	3,1	6,5	10,8	15,1	17,5	8,2

As can be found in table A.6, the mean short wave radiant energy will be:

$$R_a = 14,5kg/m^2/d = 14,5mm/day$$

Table A.7: Short wave radiant energy for different locations (Savenije, 2014)

Location	**Short wave radiant energy** $[R_C]$
The Netherlands	$(0.20 + 0.48n/N) \cdot R_A$
Average climate	$(0.25 + 0.50n/N) \cdot R_A$
New Delhi	$(0.31 + 0.60n/N) \cdot R_A$
Singapore	$(0.21 + 0.48n/N) \cdot R_A$

The location of Singapore has been chosen to calculate the short wave radiant energy.

$$\frac{R_c}{\lambda * \rho} = (0,21 + 0,48\frac{n}{N}) * R_A = 5,18 mm/day$$

In which:

- n is the amount of sun hours.
- N is the amount of possible sun hours, which is 12 hours for 0 degrees of latitude.

Long wave radiant energy (R_B)

$$R_B = \sigma * 273 + T^4 * (0,47 - 0,21 * \sqrt{e_a}) * (0,2 + 0,8 * \frac{n}{N}) = 2,13 MJd^{-1}m^{-2}$$

In which:

- σ is the Stefan-Boltzmann constant $\sigma = 4,9 * 10^{-3} Jd^{-1}m^{-2}K^{-4}$

$$\frac{R_B}{\lambda * \rho} = 0,87 mm/day$$

Net radiant energy (R_N)

$$\frac{R_N}{\lambda * \rho} = (1 - 0,006) * \frac{R_c}{\lambda * \rho} - \frac{R_B}{\lambda * \rho} = 4,00 mm/day$$

Open water evaporation (E_0)

$$E_0 = \frac{\frac{s * R_n}{\lambda * \rho} + \frac{c_p * \rho_a}{\lambda * \rho} * \frac{e_s - e_a}{r_a}}{s + \gamma} = 4,36 mm/day$$

A.2 Model results

A.2.1 Long-term average

Table A.8: Monthly average discharge from SOBEK model [m^3/s

	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**	**Yearly average**
1998					2,3	3,2	1,5	1,0	0,6	0,4	1,3	2,4	1,6
1999	2,2	5,6	3,4	2,8	1,3	0,5	0,4	0,2	0,3	1,4	1,2	2,9	1,8
2000	1,9	4,1	1,7	4,0	2,4	3,4	0,9	0,4	0,3	0,8	6,5	1,5	2,3
2001	6,6	4,1	1,2	2,6	2,1	0,5	0,4	0,2	0,2	0,2	0,6	1,2	1,6
2002	0,8	0,3	2,5	1,4	1,4	1,7	0,4	0,2	0,1	0,1	0,5	1,4	0,9
2003	3,0	1,1	3,9	2,8	3,5	0,6	0,4	0,8	1,4	0,6	0,6	2,5	1,8
2004	2,9	3,9	5,9	3,3	4,6	1,1	0,5	0,3	0,3	0,1	1,1	4,2	2,4
2005	2,2	1,5	1,2	2,9	2,4	0,6	1,2	0,3	0,3	0,7	3,3	4,8	1,8
2006	2,3	5,1	1,9	3,6	4,6	2,0	0,5	0,3	0,3	0,2	0,1	0,4	1,7
2007	2,2	2,7	1,6	3,8	3,1	1,2	1,6	0,7	0,8	0,3	0,3	0,4	1,5
2008	0,3	1,2	3,8	4,1	2,9	2,3	2,0	0,7	0,4	0,4	4,5	3,2	2,2
2009	2,4	1,9	2,5	2,6	0,9	0,4	0,2	0,2	0,1	0,2	0,2	1,6	1,1
2010	1,8	1,3	1,7	2,6	3,2	2,9	3,3	1,8	1,7	2,6	3,1	1,9	2,3
2011	1,7	2,3	3,5	3,9	2,2	1,0	0,6	0,3	0,5	0,2	1,7	3,7	1,8
2012	5,9	3,2	3,1	1,7	2,5	1,2	0,9	0,3	0,2	0,2	0,7	0,8	1,7
2013	1,6	6,7	2,1	3,6	2,5	1,8	1,0	0,4	0,4	0,2	1,6	3,0	2,0
2014	4,7	1,4	1,0	1,9	1,3	1,0	0,7	0,2	0,2	0,1	0,5	6,3	1,6
2015	5,2	3,1	1,8	4,0	2,0	1,8	0,5	0,3	0,2	0,1	0,2	0,5	1,6
2016	1,0	0,5	0,7	2,6	2,8	0,9	0,6	0,3	0,5				1,1
Monthly average	2,7	2,8	2,4	3,0	2,5	1,5	0,9	0,5	0,5	0,5	1,6	2,4	**1,7**

The long-term average discharge is $1,73\ m^3/s$. In figure F.2 the different monthly averages are shown. The wet and dry season are clearly visible. With wet months averaging at 3 m3/s and dry months as low as $0,5\ m^3/s$ can be seen. This fluctuation indicates the need for a rather large buffer in the reservoir, if a constant yield is to be extracted from the reservoir.

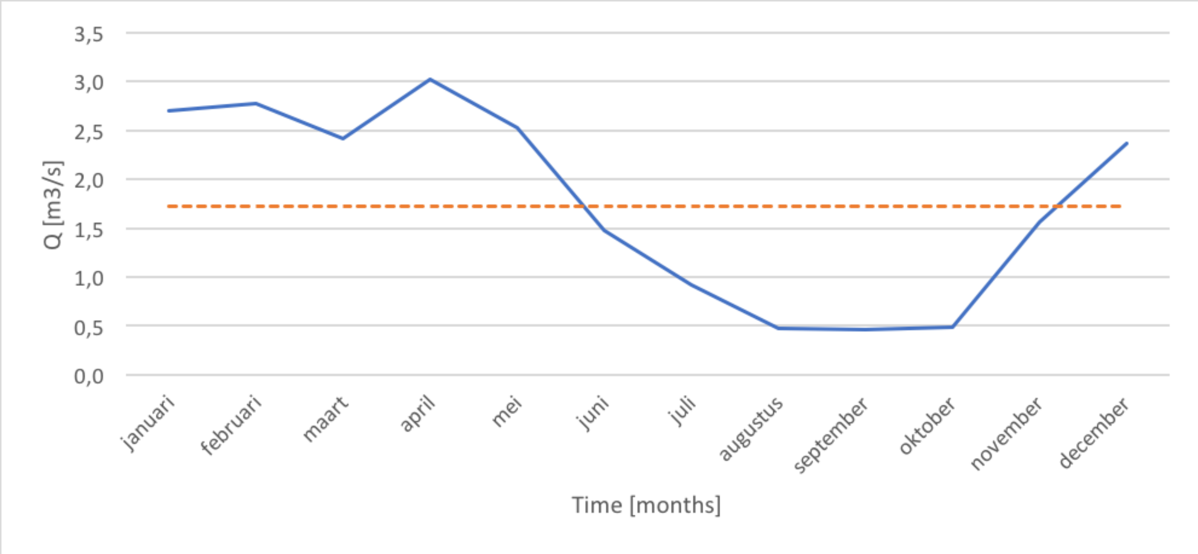


Figure A.5: Average monthly discharge

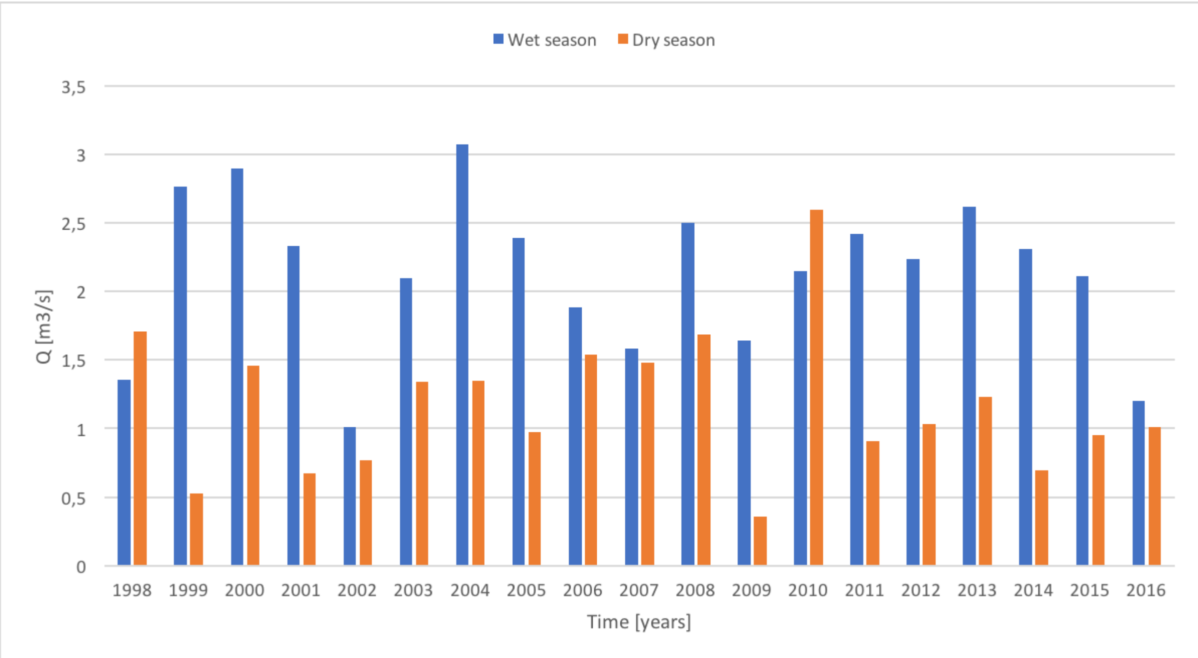


Figure A.6: Average monthly discharge

A.2.2 Peak values

Table A.9: Extreme peak discharges

Date	Q $[m^3/s]$
07-05-06	52.84
29-12-13	44.87
12-12-05	40.03
28-04-11	36.39
06-01-03	36.32
20-01-01	35.74
05-05-03	35.61
30-10-99	34.69
06-05-06	33.89
23-12-11	33.79

In figure A.7 the daily average discharge for the period between 2014 and 2016 is presented. The data shows that the peaks occur only during a short period of time, but can have a discharge of more than thirty times the annual average discharge of $1,73\ m^3/s$.

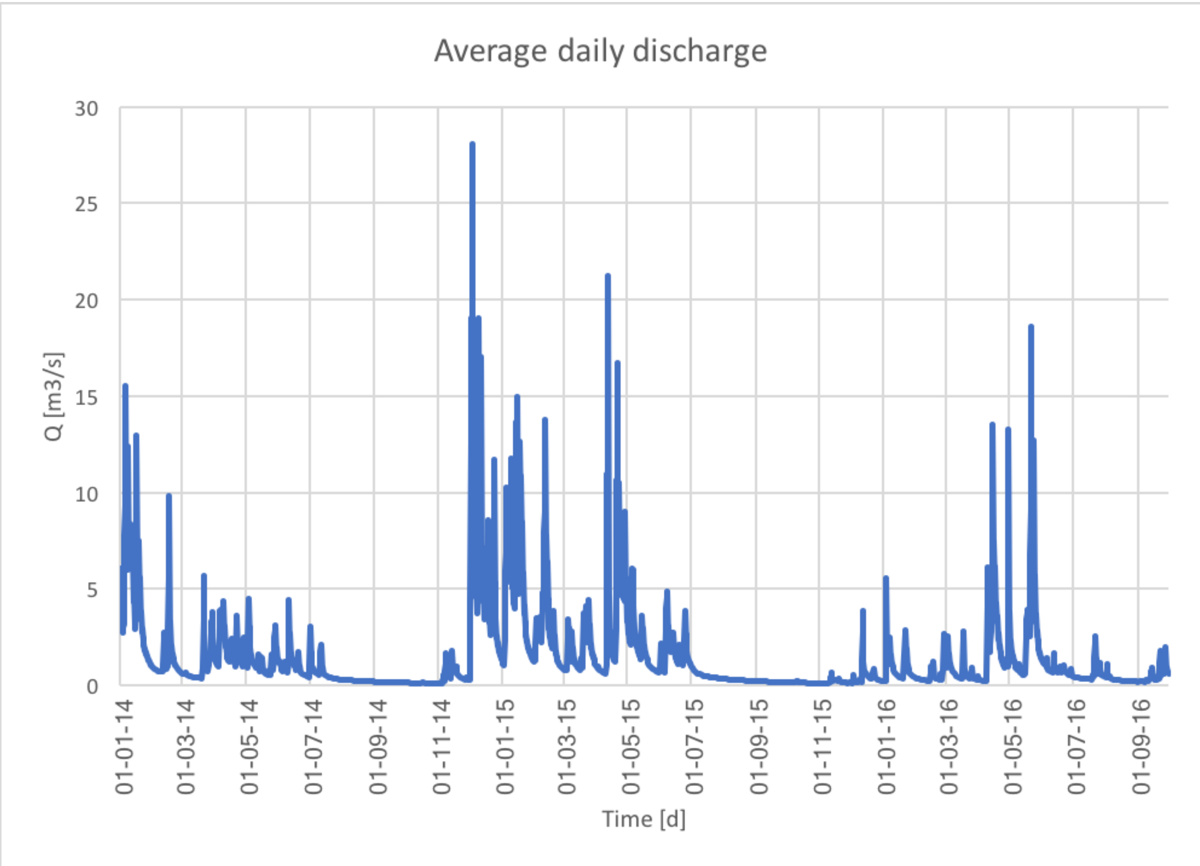
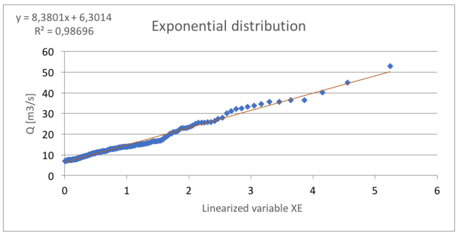


Figure A.7: Average daily discharge over a two year period

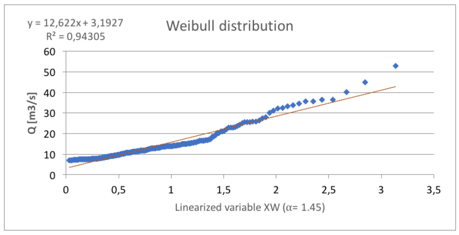
A.2.3 Extreme value analysis

The top discharges displayed in table A.9 already gives an indication on the extreme values. To investigate the different flood discharges and its return periods, an extreme value analysis is conducted. First of all the number of 'significant' peaks is determined using the *peak-over-threshold method*. As a rule of thumb, around 10 events per year is a good indication. Therefore, the threshold value for the discharge is set at $7,0\ m^3/s$. This gives a total of 189 independent discharge peaks of different rain events. Aiming to extrapolate these values with an extreme value distribution, first the most appropriate distribution must be selected.

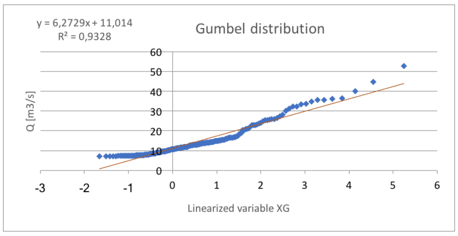
This is done by finding the best 'fit' for the data with one of the following distributions functions: *Exponential*, *Weibull*, *Gumbel* and the *Generalised Pareto* (GDP) using the linear regression method. The choice for a distribution is based on the coefficient of determination, the R^2, which shows how good the 'fit' of the model is or better said, the error between the linear regression line and the linearised peak discharge values. An R^2 value of 1, means a perfect fit with the modelled data from the SOBEK model. The following graphs shown in figure A.8, present the fit of all four distribution functions and its R^2.



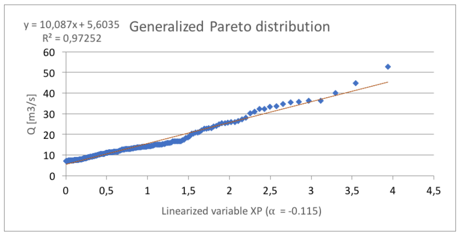
(a) Exponential



(b) Weibull



(c) Gumbel



(d) Generalized Pareto

Figure A.8: Result from linear regression

As presented in figure A.8 the Exponential distribution has the best R^2 value (0.987).The peak values with its return periods is displayed in table A.10.

Table A.10: Peak discharge predictions for given return periods

R [years] Q [-]	1 $1 \cdot 10^{-1}$	2 $5 \cdot 10^{-2}$	5 $2 \cdot 10^{-2}$	10 $1 \cdot 10^{-2}$	15 $6 \cdot 10^{-3}$	25 $4 \cdot 10^{-3}$	100 $1 \cdot 10^{-3}$	1000 $1 \cdot 10^{-4}$
Expon.	**25,6**	**31,4**	**39,0**	**44,9**	**48,3**	**52,5**	**64,1**	**83,4**
Gumbel	25,1	29,6	35,5	39,8	42,4	45,6	54,3	68,8
Weibul	25,6	30,1	35,5	39,4	41,5	44,2	51,0	61,5
Pareto	26,6	32,0	38,4	42,9	45,3	48,2	55,3	64,9

From table A.10, it can be seen that the extreme value analysis fits the data very good. As the highest peak in 19 years modelled by SOBEK was $53\ m^3/s$. This matches the extreme value analysis where a peak discharge of $53\ m^3/s$ happens every 25 years. The extreme value analysis is used for the determining spillway dimensions, where the Probable Main Flood (PMF) is used, which is the flood discharge with a return period of 1000 years, in this case 83 m^3/s.

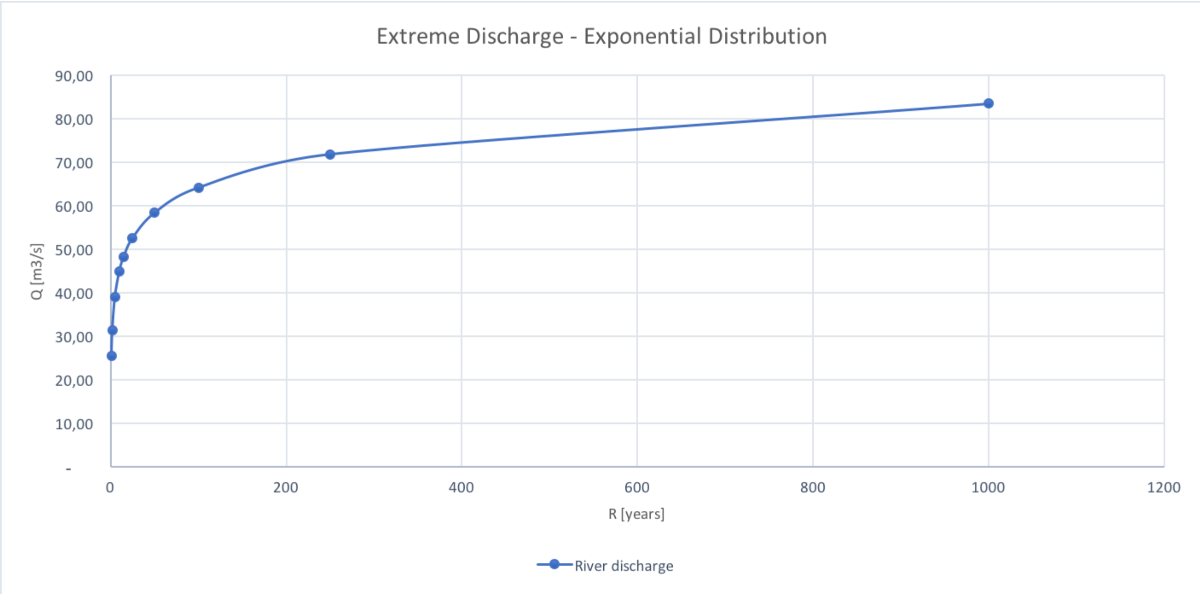


Figure A.9: Result from extreme value analysis

The Exponential distribution function is used to calculate the extreme values for a given return period. The results are depicted in figure A.9 and table A.10. It can be concluded that the difference in extreme river discharge between higher return periods becomes smaller.

Appendix B

Geological Survey

B.1 Geological Map

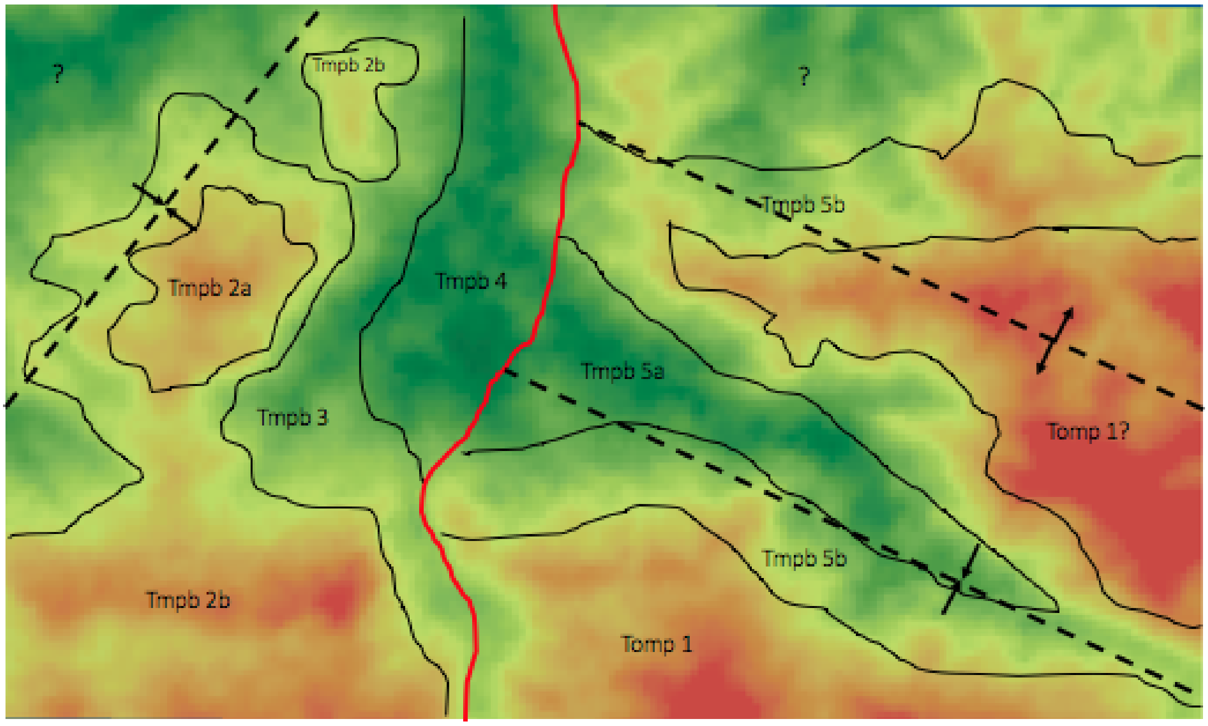


Figure B.1: Geological Map of project area phase 2A

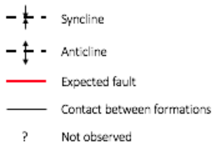


Figure B.2: Legend - Geological Map

B.2 Cross-sections

B.2.1 Cross-section A - A'

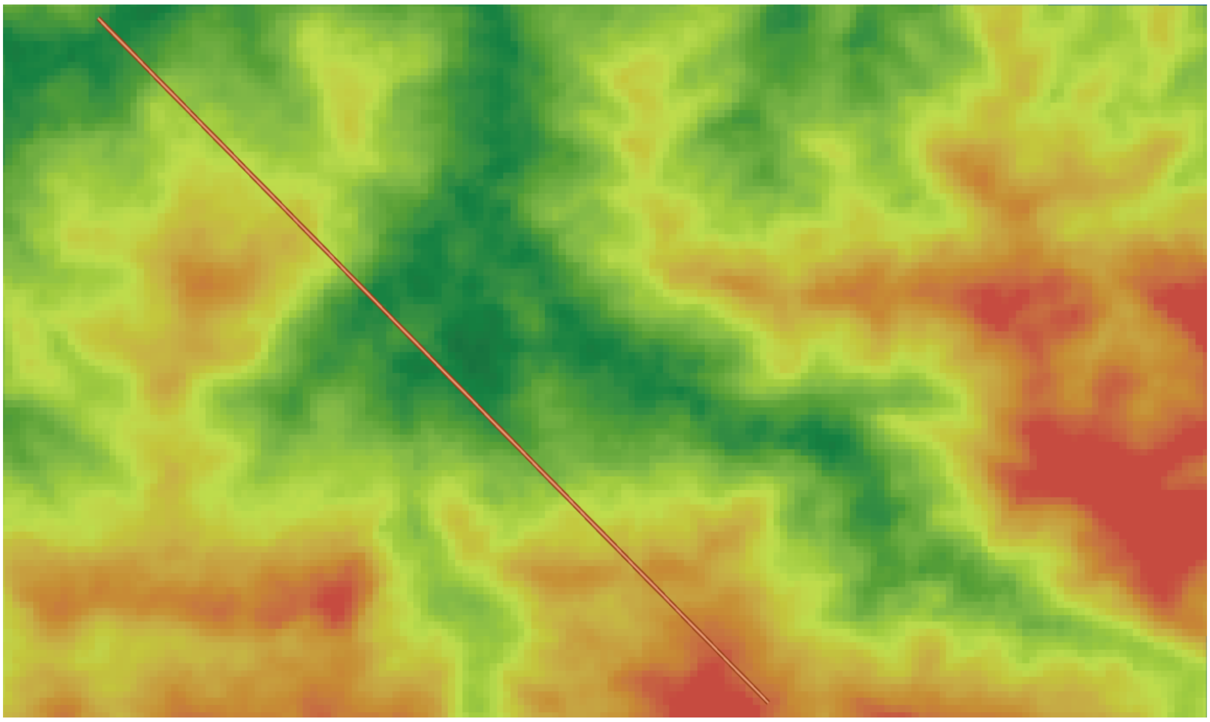


Figure B.3: Location of cross-section A - A'

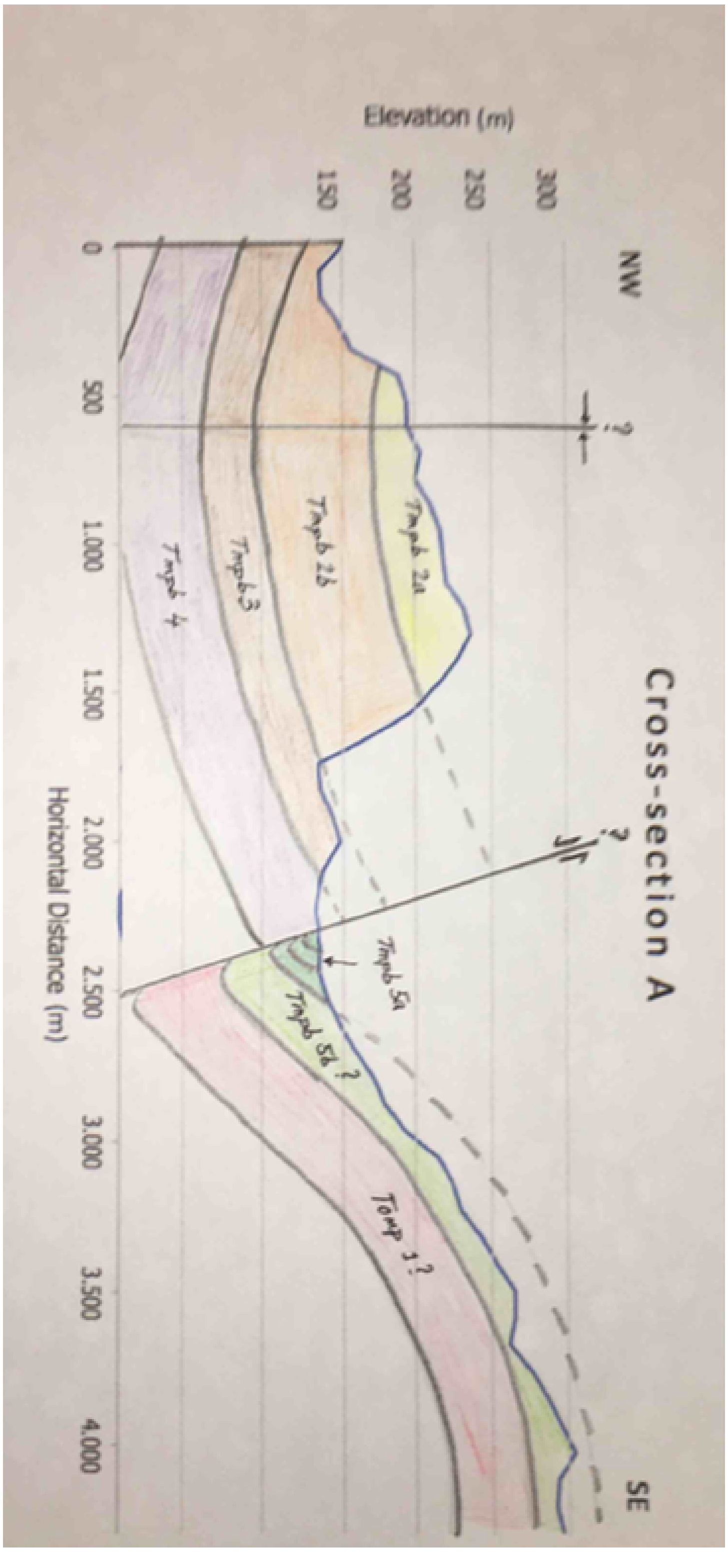


Figure B.4: Cross-section A - A'

B.2.2 Cross-section B - B’

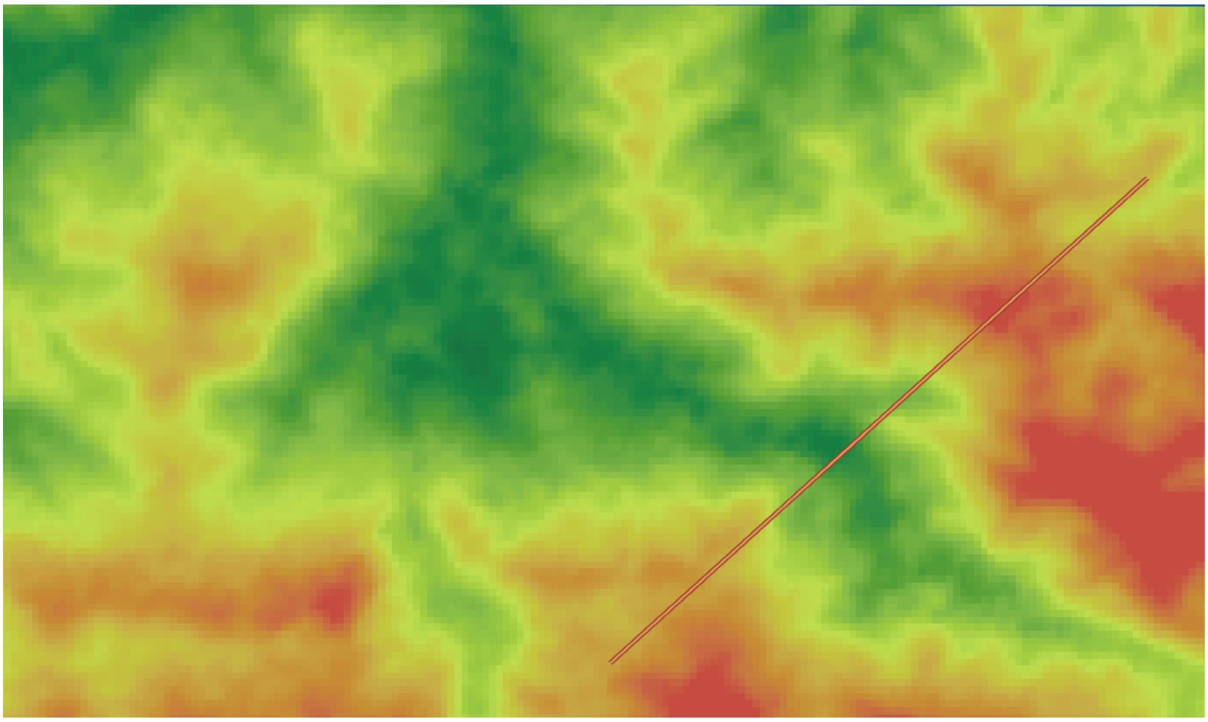


Figure B.5: Location of cross-section B - B’

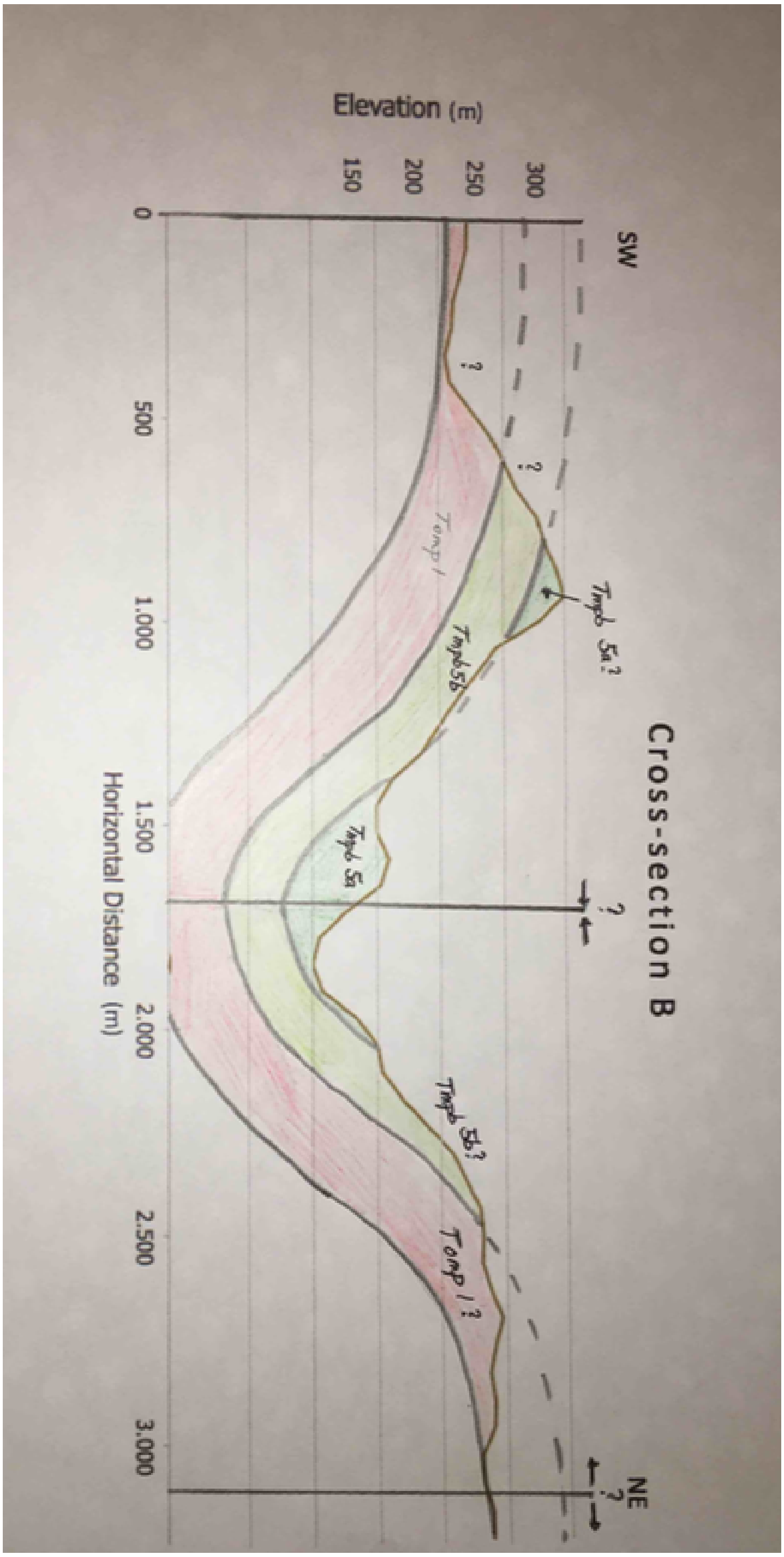


Figure B.6: Cross-section B - B'

B.2.3 Cross-section C - C'

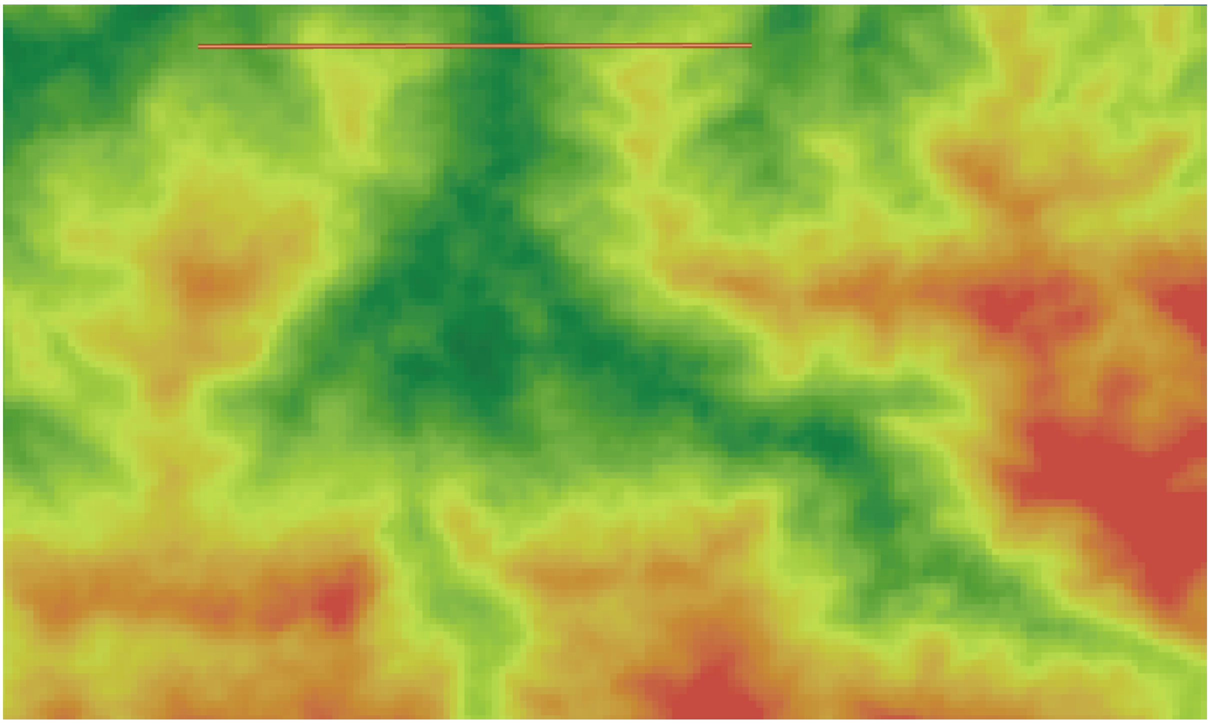


Figure B.7: Location of cross-section C - C'

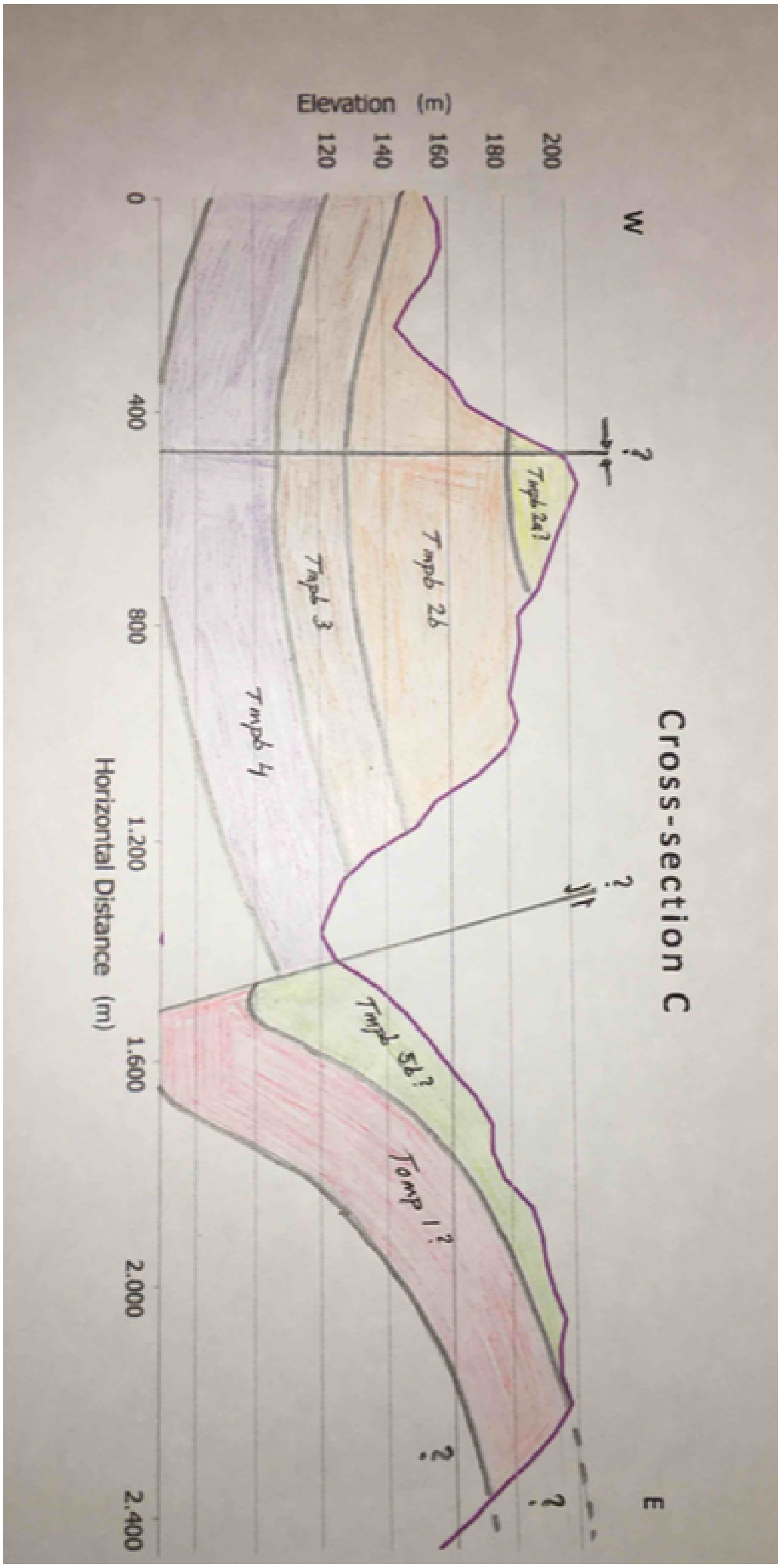


Figure B.8: Cross-section C - C'

B.3 Classification Method

An important facet of soil and rock classification is the determination of what constitutes a rock, opposed to extremely weathered material that approaches soil in its character and engineering characteristics. Extremely soft or decomposed rock that is easily crumbled, and can be reduced to gravel size or smaller by normal hand pressure, should be classified as a soil (Washington State Department of Transportation, 2006).

B.3.1 Soil Classification

Constituents

Soil Constituent	Description
Boulder	Particles of rock that will not pass through a 12 in. opening.
Cobble	Particles of rock that will pass through a 12 in. opening, but will not pass through a 3 in. opening.
Gravel	Particles of rock that will pass through a 3 in. opening, but will not pass a 0.19 in. (4.75 mm) opening.
Sand	Particles of rock that will pass through a 0.19 in. (4.75 mm) opening, but will not pass a 0.003 in. (0.075 mm) opening.
Silt	Soil that will pass through a 0.003 in. (0.075 mm) opening that is *non-plastic* or very slightly plastic and exhibits *little or no strength* when air-dried.
Clay	Soil that will pass through a 0.003 in. (0.075 mm) opening that can be made to exhibit *plasticity* (putty-like properties) within a range of water contents, and exhibits *considerable strength* when air-dried.
Organic Soil	Soil that contains enough organic particles to influence the soil properties.
Peat	Soil that is composed primarily of vegetable tissue in various stages of decomposition usually with an organic odor, a dark brown to black color, a spongy consistency, and a texture ranging from fibrous to amorphous.

Figure B.9: Soil Constituent Classification (Washington State Department of Transportation, 2006)

Angularity

Description	Criteria
Angular	Coarse grained particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	Coarse grained particles are similar to angular description but have rounded edges
Subrounded	Coarse grained particles have nearly plane sides but have well rounded corners and edges
Rounded	Coarse grained particles have smoothly curved sides and no edges

Figure B.10: Criteria for the Field Description of Angularity (Washington State Department of Transportation, 2006)

Relative Density

SPT N (Blows/Foot)	Relative Density
0 to 4	Very Loose
5 to 10	Loose
11 to 24	Medium Dense
25 to 50	Dense
Over 50	Very Dense

Figure B.11: Relative Density of Non-Cohesive Soils (Washington State Department of Transportation, 2006)

Colour

The predominant colour of the soil should be described, based on the "Munsell Soil Color Charts" (Washington State Department of Transportation, 2006).

Moisture

Moisture Description	Criteria
Dry	Absence of moisture; dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water

Figure B.12: Criteria for Describing Moisture Condition (Washington State Department of Transportation, 2006)

Structure

Description	Criteria
Stratified	Alternating layers of varying material or color with layers at least 0.25 in. thick; note thickness and inclination.
Laminated	Alternating layers of varying material or color with layers less than 0.25 in. thick; note thickness and inclination
Fissured	Breaks along definite planes of fracture with little resistance to fracturing.
Slickensided	Fracture planes appear polished or glossy, sometimes striated.
Blocky	Cohesive soil that can be broken down into smaller angular lumps which resists further breakdown.
Disrupted	Soil structure is broken and mixed. Infers that material has moved substantially - landslide debris.
Homogeneous	Same color and appearance throughout.

Figure B.13: Criteria for Describing Soil Structure (Washington State Department of Transportation, 2006)

B.3.2 Rock Classification

Colour

The colour of the rock is based on the "Geological Society of America Rock Colour Charts" (Washington State Department of Transportation, 2006).

HCl Reaction

HCL Reaction Description	Criteria
No HCL Reaction	No visible reaction
Weak HCL Reaction	Some reaction with bubbles forming slowly
Strong HCL Reaction	Violent reaction with bubbles forming immediately

Figure B.14: Rock Reaction to Hydrochloric Acid (Washington State Department of Transportation, 2006)

Grain Size

Grain Size	Description	Criteria
Less than 0.04 inches	Fine grained	Few crystal boundaries/ grains distinguishable in the field or with a hand lens.
0.04 to 0.2 inches	Medium grained	Most crystal boundaries/ grains distinguishable with the aid of a hand lens.
Greater than 0.2 inches	Coarse grained	Most crystal boundaries/ grains distinguishable with the naked eye.

Figure B.15: Grain size classification (Washington State Department of Transportation, 2006)

Weathered State of Rock

Term	Description	Grade
Fresh	No visible signs of rock material weathering; perhaps slight discoloration in major discontinuity surfaces.	I
Slightly Weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering, and may be somewhat weaker externally than in its fresh condition.	II
Moderately Weathered	Less than half of the rock material is decomposed and/or disintegrated to soil. Fresh or discolored rock is present either as a continuous framework or as corestones.	III
Highly Weathered	More than half of the rock material is decomposed and/or disintegrated to soil. Fresh or discolored rock is present either as discontinuous framework or as corestone.	IV
Completely Weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	V
Residual Soil	All rock material is converted to soil. The mass structure and material fabric is destroyed. There is a large change in volume, but the soil has not been significantly transported.	VI

Figure B.16: Weathered state of rock (Washington State Department of Transportation, 2006)

Relative Rock Strength

Grade	Description	Field Identification	Uniaxial Compressive Strength (Approx)
R0	Extremely Weak Rock	Indented by thumbnail	0.04 to 0.15 ksi
R1	Very Weak Rock	Specimen crumbles under sharp blow with point of geological hammer, and can be cut with a pocket knife.	0.15 to 3.6 ksi
R2	Moderately Weak Rock	Shallow cuts or scrapes can be made in a specimen with a pocket knife. Geological hammer point indents deeply with firm blow.	3.6 to 7.3 ksi
R3	Moderately Strong Rock	Specimen cannot be scraped or cut with a pocket knife, shallow indentation can be made under firm blows from a hammer point.	7.3 to 15 ksi
R4	Strong Rock	Specimen breaks with one firm blow from the hammer end of a geological hammer.	15 to 29 ksi
R5	Very Strong Rock	Specimen requires many blows of a geological hammer to break intact sample.	Greater than 29 ksi

Figure B.17: Relative rock strength (Washington State Department of Transportation, 2006)

B.4 Observations

B.4.1 Primary fieldwork, September'17 23th

Objective

The objective of the first fieldwork was to go to the enormous outcrop, located in the main river that cuts through the basin. This large outcrop was quite easily accessible and seemed like a good starting point for geological surveys. From this outcrop, we followed the river downstream for half a kilometer.

Expectations

The big slab of rock that was discovered during an earlier walk through the area was expected to consist of limestone. By following the river downstream, we hoped to find a contact between this rock and another sub-unit.

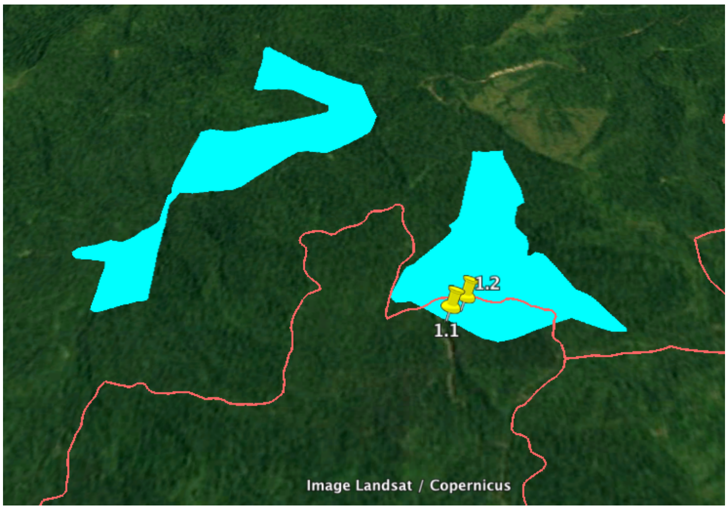


Figure B.18: Outcrops observed during primary fieldwork

Outcrop details			
Day	1	Coordinates	0°46'35.66" S – 116°37'59.34" E
Stop	1	Interpretation Tmpb 4	
Outcrop	1		
Orientation	315/19		
Observation	A large outcrop has been cut out by the main river. A siltstone formation surfaces here and shows clear orientation. The colour is 3/1 and sometimes slightly red due to local presence of Iron. The main outcrop consists of solid slabs of siltstone. When moving further downstream, there are boulders of the same material. These start off with a diameter of 2 meters and decrease in size up to a diameter of 20 centimetres. Fine grained. No HCl reaction.		
Weathering	Slightly weathered		
Strength	Strong rock		
Remarks			
Photo			

Figure B.19: Outcrop 1.1

Outcrop details			
Day	1	Coordinates	0°46'31.45" S - 116°38'2.03" E
Stop	1	Interpretation Tmpb 3	
Outcrop	2		
Orientation	315/19?		
Observation	We one follows the river from outcrop 1.1 on will first encounter a lot of siltstone boulders, as mentioned in outcrop report 1.1. Here the river runs underground and when it surfaces again there is a shale outcrop. The orientation is like that of outcrop 1.1. The river has cut through this unit and makes a right turn. The shale encountered is very fissile. Fine grained. No HCl reaction.		
Weathering	Highly weathered		
Strength	Very weak rock		
Remarks	No picture is taken and no exact orientation is measured here. Although it is certain that the dip direction is towards the NW. Angle is assumed to be the same as outcrop 1.1.		
Photo			

Figure B.20: Outcrop 1.2

B.4.2 Secondary fieldwork, October'17 02th

Objective

The objective for the expedition was to walk the length of the supposed path on the southern ridge of the basin. This path was chosen, because roads are a good source for outcrops due to excavations that need to be made to create a level path through the jungle. The available outcrops give an insight in the geology of the eastern and southern flanks of the potential storage lake.

Expectations

We expected to find evidence of an E-W anticline through the center of the basin with limestone in the center. This was based on the basins that had been investigated in phase 1 by Witteveen+Bos.

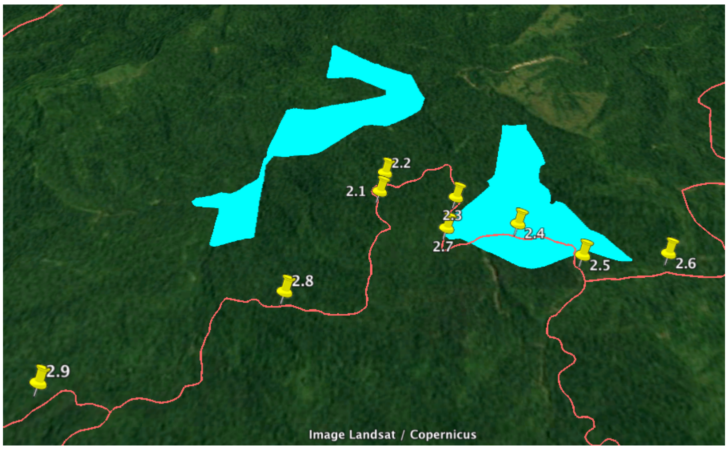


Figure B.21: Outcrops observed during secondary fieldwork

Outcrop details			
Day	2	Coordinates	0°46'20" S - 116°37'27" E
Stop	1	Interpretation Tmpb 2a	
Outcrop	1		
Orientation	300/15		
Observation	An outcrop of approximately 6 meters high and stretches along most of the hill ridge. Consists mostly of layers of sandstone (80%) with small layers of shale in between (20%). At the bottom of the outcrop, the average thickness of sandstone layers is 20-40 cm and shale layers of approximately 5-10 cm. Sandstone layers become thicker moving towards the top and shale becomes less frequent. Colour is 5/8. Medium grained. No HCl reaction.		
Weathering	Moderately weathered		
Strength	Moderately strong rock		
Remarks	The layer is located on the top of the hill. The ridges are quite steep and it seems that the layer stretches along the complete hill top. We have checked this on several places and this seems to be in order.		
Photo			

Figure B.22: Outcrop 2.1

Outcrop details			
Day	2	Coordinates	0°46'09" S - 116°37'29" E
Stop	2	Interpretation Tmpb 2a	
Outcrop	2		
Orientation	315/15		
Observation	Small outcrop of sandstone with very thin interbedded shale (approx. 2 mm). A fresh sample colour is 5/3 with no signs of iron. The shale colour is 3/1 and is very fissile. Medium grained. No HCl reaction.		
Weathering	Moderately weathered		
Strength	Strong rock		
Remarks			
Photo			

Figure B.23: Outcrop 2.2

Outcrop details			
Day	2	Coordinates	0°46’18” S - 116°37’50” E
Stop	3	Interpretation Tmpb 2b	
Outcrop	3		
Orientation	-		
Observation	Down a slope, under the layers of stop 2.2 we find a thick layer of sandstone, with little interbedded shale. Outcrop was found on path, where water cut a road through overlaying soil. No orientation discovered. Colour 7/1. Medium grained. No HCl reaction.		
Weathering	Slightly weathered		
Strength	Very strong rock		
Remarks	The sandstones seem to become harder as we move down through the layers. Also found on top of a steep slope, reaching down a couple of meters down. Difficult terrain, dangerous to move down and retrieve more information.		
Photo			

Figure B.24: Outcrop 2.3

Outcrop details			
Day	2	Coordinates	0°46'27" S - 116°38'08" E
Stop	4	Interpretation Tmpb 5a	
Outcrop	4		
Orientation	325/40		
Observation	Outcrop at the foot of riverbed of at least 4 meters high. Consists only of shale with a dark grey colour (3/1). Very fissile and has a steep slope compared to the other outcrops. Fine grained. No HCl reaction.		
Weathering	Completely weathered		
Strength	Very weak rock		
Remarks	Has not been seen on the other side of the river, but that does not mean that it cannot be present there. Hard to go down and look for it.		
Photo			

Figure B.25: Outcrop 2.4

Outcrop details			
Day	2	Coordinates	0°46'38" S - 116°38'24" E
Stop	5	Interpretation Tomp 1	
Outcrop	5		
Orientation	-		
Observation	A large outcrop at the top of the hill with 100% sandstone (white colour) and no orientation visible. At least 2 meters thick. Coarse grained. No HCl reaction.		
Weathering	Slightly weathered		
Strength	Extremely weak rock		
Remarks			
Photo			

Figure B.26: Outcrop 2.5

Outcrop details			
Day	2	Coordinates	0°46'38" S - 116°38'49" E
Stop	6	**Interpretation** Tmpb 5a	
Outcrop	6		
Orientation	315/80		
Observation	At least 15 meters of shale at the riverbed where the road crosses the river. Can be seen along both sides of the river - has cut through the shale. Colour varies from grey to very dark grey. All the outcrops were in contact with the water. Fine grained. No HCl reaction.		
Weathering	Highly weathered		
Strength	Moderately weak rock		
Remarks	The shale is similar to the outcrop found at 2.4 but under a much steeper slope. Probably result of folding.		
Photo			

Figure B.27: Outcrop 2.6

Outcrop details			
Day	2	Coordinates	0°46'52" S - 116°37'02" E
Stop	7	Interpretation Tmpb 2a	
Outcrop	7		
Orientation	325/12		
Observation	Underneath the path, a long stretch of sandstones surfaces. Light brown/red colour (5/3) on the in- and outside. No apparent layering; only the top is visible. Medium grained. No HCl reaction.		
Weathering	Slightly weathered		
Strength	Very hard rock		
Remarks			
Photo			

Figure B.28: Outcrop 2.7

<table>
<tr><td colspan="4">Outcrop details</td></tr>
<tr><td>Day</td><td>2</td><td>Coordinates</td><td>0°46'30" S - 116°37'46" E</td></tr>
<tr><td>Stop</td><td>8</td><td colspan="2" rowspan="3">Interpretation
Tmpb 4b</td></tr>
<tr><td>Outcrop</td><td>8</td></tr>
<tr><td>Orientation</td><td>310/15</td></tr>
<tr><td>Observation</td><td colspan="3">A siltstone layer when walking down a slope with on top a thick sandstone outcrop. A river runs 20 meters below the sandstone layer and exposes this outcrop. Small layers are visible due to weathering. Fine grained. No HCl reaction.</td></tr>
<tr><td>Weathering</td><td colspan="3">Highly weathered</td></tr>
<tr><td>Strength</td><td colspan="3">Moderately weak rock</td></tr>
<tr><td>Remarks</td><td colspan="3"></td></tr>
<tr><td>Photo</td><td colspan="3"></td></tr>
</table>

Figure B.29: Outcrop 2.8

Outcrop details			
Day	2	Coordinates	0°46'56" S - 116°36'44" E
Stop	9	Interpretation Tomp 2	
Outcrop	9		
Orientation	-		
Observation	Large outcrop of solid limestone. 15 meters high and visible along roadside for at least 50 meters. Has been used to harden roads in the area. Strong HCl reaction.		
Weathering	Lot of karst		
Strength	Very strong rock		
Remarks	Water streams through the cracks, what indicates that the karst is in contact with another river. Should be looked further into.		
Photo			

Figure B.30: Outcrop 2.9

B.4.3 Tertiary fieldwork

Objective, October 10th

This expedition was headed to the potential location of the dam. This area was to be inspected to characterize the rock conditions and to be able to draw a cross-section of that location. The long path to this location (four hours through thick vegetation) covered the western ridge of the basin, right up to the dam location. The objective was to retrieve more information about this region.

Expectations

The western ridge was expected to be part of the syncline structure that dominates the area. At the dam location, it was expected to find evidence of a fault that would run through the depression in the topography.

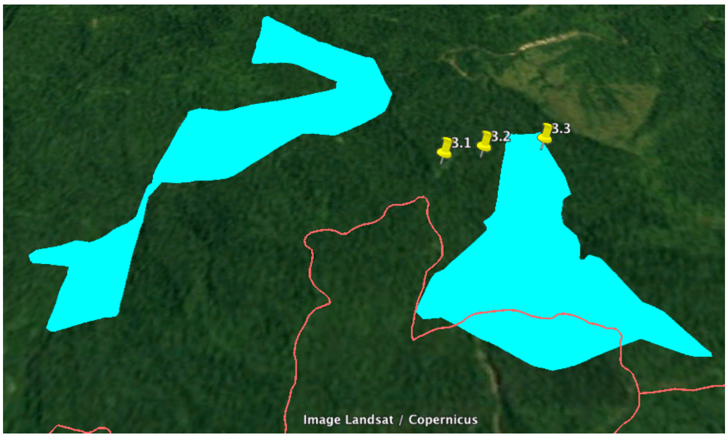


Figure B.31: Outcrops observed during tertiary fieldwork

<table>
<tr><td colspan="4">Outcrop details</td></tr>
<tr><td>Day</td><td>3</td><td>Coordinates</td><td>0°45'54.29"S - 116°37'51.02"E</td></tr>
<tr><td>Stop</td><td>1</td><td colspan="2" rowspan="3">Interpretation
Tmpb 2b</td></tr>
<tr><td>Outcrop</td><td>1</td></tr>
<tr><td>Orientation</td><td>350/08</td></tr>
<tr><td>Observation</td><td colspan="3">River has cut through the top soil to uncover five meters of sandstone. Colour is 5/6. Medium grained. No HCl reaction. Only a small outcrop visible due to abundant soil cover.</td></tr>
<tr><td>Weathering</td><td colspan="3">Moderately weathered</td></tr>
<tr><td>Strength</td><td colspan="3">Strong rock</td></tr>
<tr><td>Remarks</td><td colspan="3"></td></tr>
<tr><td>Photo</td><td colspan="3"></td></tr>
</table>

Figure B.32: Outcrop 3.1

<table>
<tr><td colspan="4">Outcrop details</td></tr>
<tr><td>Day</td><td>3</td><td>Coordinates</td><td>0°45'48.57"S - 116°38'4.87"E</td></tr>
<tr><td>Stop</td><td>2</td><td colspan="2" rowspan="3">Interpretation
Tmpb 3</td></tr>
<tr><td>Outcrop</td><td>2</td></tr>
<tr><td>Orientation</td><td>090/19</td></tr>
<tr><td>Observation</td><td colspan="3">A small river cut out a small shale outcrop. A little farther uphill are a lot of sandstone boulders. Colour 2/1. No HCl reaction. Fine grained.</td></tr>
<tr><td>Weathering</td><td colspan="3">Highly weathered</td></tr>
<tr><td>Strength</td><td colspan="3">Very weak rock</td></tr>
<tr><td>Remarks</td><td colspan="3">The orientation is very different from outcrops found close to it. Might indicate a landslide.</td></tr>
<tr><td>Photo</td><td colspan="3"></td></tr>
</table>

Figure B.33: Outcrop 3.2

Outcrop details			
Day	3	Coordinates	0°45'43.90"S - 116°38'13.05"E
Stop	3	Interpretation Tmpb 4	
Outcrop	3		
Orientation	350/06		
Observation	A solid slab of siltstone is cut out by the river. Orientation is visible and found along the river bed. No HCl reaction. Colour is 4/1. Fine grained.		
Weathering	Moderately weathered		
Strength	Strong rock		
Remarks			
Photo			

Figure B.34: Outcrop 3.3

Appendix C

Fieldwork

C.1 Introduction

After a week of literature studies and defining our research questions at ITCI, we have gone to the catchment area of interest to collect data. During the first week, we came up with a planning for the week, but we had to adjust this on Tuesday morning when it became clear that a large part of the necessary equipment had been delayed. Here we will give a daily description of our objectives, the fieldwork and the conclusions we could make accordingly.

C.2 Field report

C.2.1 Tuesday September 12th

Objective

The objective was to pick a small catchment area within the phase 2 catchment area. This area will be called the Test Site (TS) and should help us to find a correlation between actual precipitation data and discharge + sediment load. To define the TS we planned on following a small river, upstream, to its source and then tracking the borders of the catchment area.

Fieldwork

Driving down from the quarry we chose the second river for our TS because the discharge was significantly larger than the first river we crossed and because there was a lot of building material present at the site to construct the V-notch. We (Timo Worm, Tim, Scipio, Stephanie, Piebe and Mick + Ahmat and Dap) followed the river upstream for a kilometer until we came upon a point where the river split into two streams of (assumed) equal discharge. At this point we discussed the route we should take with the guides and they informed us that both paths would continue for about 10 kilometers before we would find the source of the river. We decided that the TS would become too large and that the hike would take too long for us to accomplish. We turned around and decided to track the shorter, smaller river we crossed earlier that morning. After lunch, we drove to the first river, again determined the location of the V-notch and started the upstream hike in order to discover the source. Ten minutes after our departure, it started to rain heavily and we saw the effect this had on the discharge first hand. It affected the amount of discharge significantly and the sediment load of the water. We did not have any equipment in place to measure the change in discharge, but we assumed that it was tenfold its original volume. The discharge kept growing with continuing rainfall. Multiple side inlets were detected along the way – all with a smaller discharge that the main stream, though not insignificant. After two hours, we arrived at a road that crossed the river. This was not the starting point of the river, but we decided to head back to the quarry and investigate the path of the river with the online maps of the area.

Conclusion

From this hike, the main thing we can conclude is that the river characteristics are primarily influenced by precipitation. The two factors it influences are sediment load and discharge:

- The sediment load of the river water is much higher than before. To determine the exact difference, we have taken a sample of the water. By continuing taking samples over a larger timeframe we can determine the exact influence of precipitation on sediment load.
- Discharge was significantly higher after the storm than before. To determine the exact impact, we will install a V-notch.

Because we haven't followed the river to the source, we will have to define the river path in google maps and locate the outline of the catchment area.

(a) Start at roadside

(b) First part of the river

(c) Large rocks present

(d) Approximately $1km$ upstream

(e) Side branch

(f) Side branch

Figure C.1: Pictures of investigating the first river

(a) Start at roadside

(b) First part of the river

(c) Underground stream

(d) Underground stream

(e) Last part of the river

(f) Last part of the river

Figure C.2: Pictures of investigating the second river

C.2.2 Wednesday September 13th

Objective

Objective was to find an enormous drop in river topography at location C and to search for a location along the drop that was suited for a hydro power plant.

Fieldwork

Our hike started at the end of the road, the point where the car couldn't go any further. We continued to follow the road by foot for an hour or so, and then turned east through the jungle in search for the river. The river was different from anything we have seen so far; a forty-meter-wide stretch of solid rock without any plant habitation that ran through the jungle like a highway. The total discharge was made up out of a couple of small streams that ran over the rock surface. Each of these streams was just half a meter wide and only a couple of centimeters deep, with water running at a medium velocity. We should take into consideration that there had been no precipitation that day. On the map, the point where we first found the river was called C-old. We were looking for a point more downstream, where google maps predicted the drop would be. When we followed these coordinates, we found that there had been a mistake with the notation of the coordinates of that point and that C-old was actually A2 (elevation 140 meters). We then followed the river upstream to see if we could locate any kind of drop. We continued to follow the river up to an elevation of 160 meters, but the path had a very gradual inclination. With only an hour before sunset and more than four kilometers of rough, jungle terrain to cover, we decided it was best to head to the quarry.

Conclusion

The path of river that has been drawn into google maps is incorrect; the presumed drop of 100 meters is not there. There could still be a drop we don't know about, but that will be a lot smaller than presumed. If we wish to be absolutely sure about the presence of a possible drop, we will have to follow the river all the way upstream to A1. Although we can say for certain that the area is less suited/not suited for a hydro power plant than Willie presumed. The river is bordered by very steep slopes. When drawing the river path in google maps, an error of only 20 meters, could lead to huge errors in topographical information. We assume this caused the mistake.

Treats of nature

- While descending through the river, jumping over the boulders, we discovered a four-meter-plus snake, crawling between the boulders in the river;
- The sight of a twenty-odd meter tree coming down naturally;
- A tree split into half by a lightning bolt, blackened from the inside all the way down to the roots, while leaving the bark untouched.

C.2.3 Wednesday September 13th, V-Notch Test Setup

Objective

The objective was to build a V-notch on a test site, we found the day before. With this V-notch we are able to calculate a discharge which we can use for the correlation between the actual precipitation.

Fieldwork

On the 13th of September we started building the V-notch with the use of sand bags, plywood, stones and wooden sticks. The first step was to fill 15 sand bags with clay, which we took from the surrounding area, and place them on the location of the weir. After, we repeated this for another 15 sandbags. The next step was to cut the plywood in dimensions of 1 x 1,5 meter and put wooden sticks on the side and bottom of them. Then, we created the weir by placing the plywood on the location of the V-notch. We waited a short while for the water level to raise and cut an angle of around 90 degrees at the 'V' of the notch, so that the cut would be exactly 2 cm above the water level. We could measure the water height with the measurement lines on the plywood. The final step was to fill up the last 30 sand bags and put them on the correct places on the V-notch. We used rocks for extra support. During 14 days, we will make measurements at the V-notch including the water level upstream, the water level at the notch and sediment samples. Due to turbulence, we are not able to measure the downstream water level.

Dimensions

Before placing the V-notch:

Water level: 18cm
Width: 130cm

After placing the V-notch:

Water level upstream: 41cm
Bottom V till bottom river: 43cm
Free fall of water downstream: 16cm
Plywood: 4mm
Degree of V: 90 degrees

Inaccuracies:

- Not all the discharge through the V of the V-notch, but also on the side (+10%) (C_u)
- The angle is not exactly 90 degrees (+3%) (C_a)
- The plywood bend, so it is not in a straight line (+3%) (C_b)
- Wrong interpretation of the water level

Theory

The water levels and flow velocities can be solved by using a combination of equations:

"Preservation of discharge (volume and mass)":
$$Q = u * A = constant$$
"Preservation of energy (Bernoulli)":
$$H = h + (u^2)/(2g) = constant$$

The second formula can be used at places where you can neglect turbulence and large wall friction losses. Due to losses occurring in a V-notch, because of edges of the weir and contractions in the area of flow, we have take a discharge coefficient C_d into account. Which forms the formula $Q_(actual) = Q_(theory) * C_d$, with the average discharge coefficient C_d = 0,60 for a V-notch weir. This coefficient has been determined through dimensional analysis and experiments. The values of the eddy near the gate depend on the shape of the edge of the gate and cannot be calculated precisely.

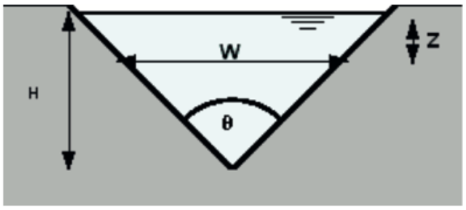


Figure C.3: Front view V-Notch

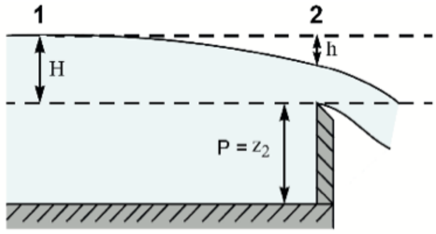


Figure C.4: Side view V-Notch

The width of the notch can be calculated with either 'H' and 'z' or 'h'. For our calculation we use the parameter 'z', which makes the equation for the width as follows:

$$w = 2 * (H - z) * \tan(\frac{\theta}{2})$$

For calculating the discharge of the notch we used the following equation, taken into account that $u_1 << H$:

$$Q = \frac{8}{15} * \sqrt{2 * g} * \tan(\frac{\theta}{2}) * H^(\frac{5}{2})$$

Results and conclusion

With the use of good results and a catchment area a model can be set up to calculate the correlation between the actual precipitation. Unfortunately, there are no useful results from the measurements due to the absence of rainfall, which makes it impossible to make a conclusion of this results. Our V-notch will be probably fail during heavy rainfall since it already showed a slight bending after a couple of days. This is the result of using thin and weak plywood, so in order to get useful results we will need to use stronger material and more data.

Manual

During 14 days, we will make measurements at the V-notch including the water level upstream, the water level at the notch and sediment samples. Local people will preform the measurements with the use of this manual.

Indonesian version

Curah hujan
Langkah 1: Lakukan setiap hari pada jam 9 pagi.
Langkah 2: Menuju lokasi dengan membawa gelas ukur, buku dan pulpen.
Langkah 3: Tuangkan air yang terdapat di dalam botol ke dalam gelas ukur untuk diukur.
Langkah 4: Baca jumlah air menggunakan gelas ukur.
Langkah 5: Catat di buku.

Level air 1
Langkah 1: Lakukan setiap hari pada jam 9 pagi.
Langkah 2: Menuju ke lokasi 1 dengan membawa buku dan pulpen.
Langkah 3: Baca level air di point 1 dengan melihat ketinggian air yang tampak pada tiang kayu.
Langkah 4: Tulis pada selembar kertas.

Level air 2
Langkah 1: Lakukan setiap hari pada jam 9 pagi.
Langkah 2: Menuju ke lokasi 2 dengan membawa buku dan pulpen.
Langkah 3: Baca level air di point 2 dengan melihat ketinggian air yang tampak pada tiang kayu.

Langkah 4: Tulis pada selembar kertas.

Contoh sedimentasi
Langkah 1: Lakukan setiap hari pada jam 9 pagi.
Langkah 2: Menuju lokasi 1 dan bawa 1 botol aqua 600 ml kosong.
Langkah 3: Isi botol dengan air dari sungai lokasi tersebut.
Langkah 4: Tutup botol dan bawa ke camp.
Langkah 5: Beri sticker penanda pada botol (tulis tanggal, waktu dan "location D").
Langkah 6: Letakkan botol pada tempat yang aman dan tandai pada buku bahwa telah selesai.

Langkah terakhir
Ambil foto catatan tersebut setiap hari.

English version

Rainfall
Step 1: Do this every day in the morning at 9am.
Step 2: Go to the place with the measurement cup, paper and pen.
Step 3: Empty the bottle in the measurement cup.
Step 4: Read the amount of water in the measurement cup.
Step 5: Write it on the piece of paper.

Water level 1
Step 1: Do this every day in the morning at 9am.
Step 2: Go to location 1 with the paper and pen.
Step 3: Read the water level on point 1 on the stick.
Step 4: Write it on the piece of paper.

Water level 2
Step 1: Do this every day in the morning at 9am.
Step 2: Go to location 2 with the paper and pen.
Step 3: Read the water level on point 2 on the stick.
Step 4: Write it on the piece of paper.

Water level 3
Step 1: Do this every day in the morning at 9am.
Step 2: Go to location 3 with the paper and pen.
Step 3: Read the water level on point 3 on the stick.
Step 4: Write it on the piece of paper.

Sediment sample
Step 1: Do this every day in the morning at 9am.
Step 2: Go to location 1 take an empty bottle of water from 600ml.
Step 3: Fill the bottle of water with water from the river at the location.
Step 4: Close the bottle of water and take it back to the house.
Step 5: Place a sticker on the bottle and write down the date, time and 'Location D'.
Step 6: Place the bottle on a save place and write on the paper that it is done.

Last step:
Take a picture of the paper.

(a) Situation before

(b) First step

(c) Second step

(d) Third step

(e) Fourth step

(f) Fifth step

Figure C.5: Pictures of the V-notch

C.2.4 Thursday September 14th

Objective

The objective was to follow the river down from C, all the way to point A1. This way we would be able to map the path of the river exactly, determine if there was a suitable drop for hydro power and finally be able to describe the possible dam location of A1. We would then walk towards location B, also a possible dam location, to do the same there. The necessary data at the dam locations that had to be collected was:

- A sample of the water to determine the water quality;
- A sample of the water to define the sediment load;
- The average width and depth of the river;
- A stretch of river with laminar flow where we could perform velocity measurements and give an estimation of the discharge at that given time;
- A height profile perpendicular to the river flow;
- An estimation of the subsurface composition to determine the suitability for construction.

Fieldwork

We (Stephanie, Piebe, Mick + Ahmat and Dap) set of along the same path as we did the day before. Following the former road, then turning east until we came upon the river at the most southern point of the potential storage lake. We then continued to track the river downstream. The characteristics of the river changed rapidly:

- We started with 300 meters of solid limestone, smoothened by the river;
- It then changed into a scattering of enormous boulders for 500 meters;
- These made way for a shallow muddy base with a scattering of smaller rocks (30-centimeter diameter) for 500 meters;
- The boulders disappeared and were replaced by a sandy riverbed (with muddy slopes) with varying water depth from 10 centimeters to half a meter;
- Just after the bridge a smaller river joined the main river (bifurcation), adding approximately half of the discharge to the main river;
- The river then continued to grow in depth, becoming wider with a varying depth between 50 centimeters and 1 meter;
- Eventually it wasn't possible to walk through the river because of the depth (1 meter – 2 meter-plus), the shores were steep and there were a lot of fallen trees in the river.

Overall the river velocity was low. The last heavy rainfall had taken place two days ago on Tuesday. There were a lot of little streams that joined the river along the way. The path of the river overall was lightly meandering.

Then we arrived at A1 – the potential location of the primary dam – and performed the data collection. Also, a qualitative description of the area:

- The subsurface composition of the slopes and the surrounding area was made up out of sandy clay. The riverbed contained debris from limestone layers further up the hill;
- Steep slopes bordered the river; from this we may assume that B is approximately equal to Bs;
- The riverbed was not smooth, but disturbed by various stones and rocks.

Finally, we continued our journey to B – the potential location of the secondary dam. Darkness was coming fast, so we could not describe the area in the detail that we could at A1.

- The area at B was completely covered with large boulders – not only the riverbed, but also the borders.
- We can only account for a small stretch of the river, that already showed large variety in bedding, width and depth.
- The velocity measurements were conducted at a stretch with relatively laminar flow, but turbulence was still present.
- Area was to inaccessible to walk a cross-section at B perpendicular to the river, as we have done at A1.

When we finished the data collection at B we had to head back to the quarry because sunset was approaching. We had huge difficulty with finding the old road and only discovered it when almost all the light was gone. Then we still had a 1,5-hour journey ahead of us, guided by the stars, catching sight of the car at 19:30.

Conclusion

We hoped to determine a possible location for a water treatment plant and a tunnel connection between the two potential storage lakes. We have concluded this is impossible due to the thick vegetation, that disabled our mobility and made it impossible to determine an exact topographical profile. We think that a DED is only possible when the forest is cleared at the site.

(a) Potential dam location A1

(b) Water velocity measurements at location A1

(c) Potential dam location B

(d) Potential dam location B

(e) Measuring width and depth at location B

(f) Measuring width and depth at location B

Figure C.6: Pictures of investigating possible dam location of A1 and B

Appendix D

Dam

D.1 Topography

The dam location and the elevation of the dam can be seen in figure D.1 and in figure D.2.

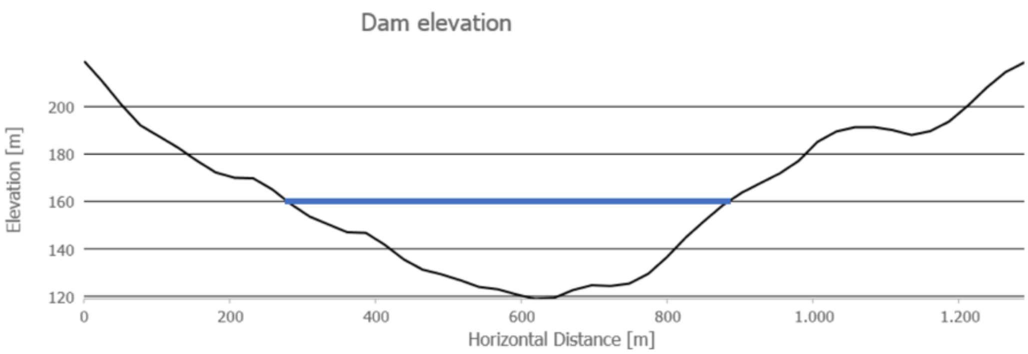


Figure D.1: Dam elevation

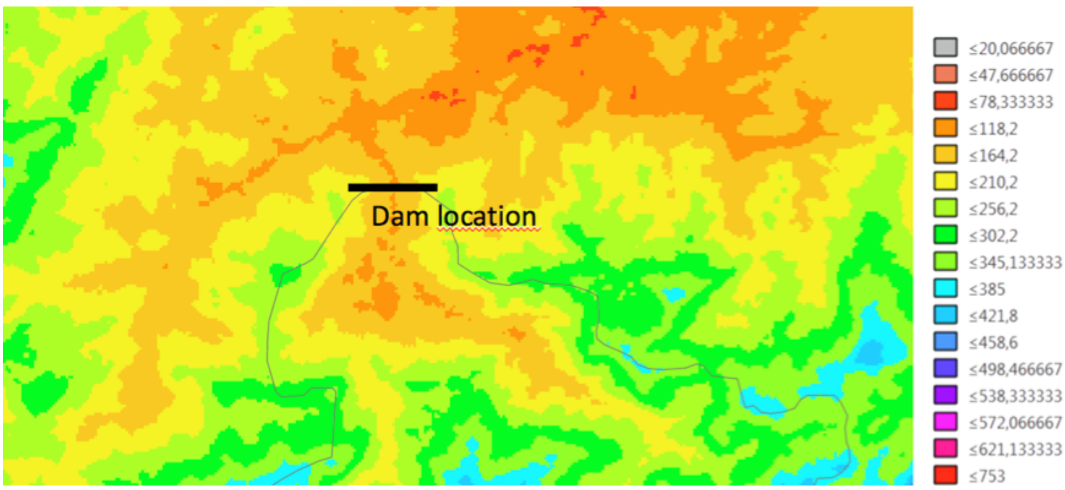


Figure D.2: Dam Location

D.2 Failure modes

In this section the different failure modes of section 5.2.3 are discussed.

Armour stability, front face

In order to determine the stability of the concrete armour you need the average stone weight, diameter, density of the stone, slope angle, interlocking capacity, wave height and the density of the water. There are several types of failure of the armour layer, such as rocking, settling, displacement and sliding.

Toe berm stability

There is a strong interaction between the stability of the toe berm and the stability of the armour layer. Meaning that failure of the toe berm stability could lead to failure in the armour layer. When the toe berm is designed with similar loads and similar dimensions, this risk is minimized. Also, when the toe berm is damaged moderately, this will not have a significant influence on the stability of the armour layer. However, when the toe berm is severely damaged, it will have a big negative influence on the stability of the armour layer.

Armour stability, rear face

If the dam is designed for overtopping, the rear face of the dam should be designed for eroding conditions.

Scour

Scour is the removal of sediment underneath the base of the dam, caused by the flow underneath the structure. This can result in gradual dislocation of a sill and can decrease the geotechnical stability of the structure. It is an interaction between the hydrodynamics and the geotechnical properties of the soil under the base of the dam. To prevent scour under the core or the toe, protection should be applied. The width of the protection should be equal or somewhat larger than the scour depth to avoid soil mechanical stability problems.

Wave overtopping

Wave overtopping is one of the most critical failure modes for a reservoir dam. It can seriously endanger the structure because it can cause erosion and softening of the foundation, but also the effects on the surroundings is considered to be highly undesirable. The waves in the reservoir are mainly caused by the wind set-up and wind waves. The amount of overtopping depends on the wave height, the wave steepness, the slope and the existing freeboard. Therefore, to prevent overtopping, the design of the dam should include a sufficient freeboard above the maximum water level.

Return period

For a proper design, the failure modes must be coupled to return periods. As this is a preliminary study, the report will focus on the return periods considered for the Ultimate Limit State and the Serviceability Limit State. The ULS is related to failure of the dam by serious damage of collapse. As the consequences are severe and considered to be unacceptable, the limit state is given a return period of 1000 years. Which is considered safe, when taking the absence of populated areas near the reservoir into account. The SLS is defined as the state in which the constant extraction yield can no longer be met for a significant period of time. The drinking water will provide a large part of the total water consumption for a number of cities. Failure is therefore, highly undesirable and may not occur frequent in the dams' lifespan. The return period of the SLS is therefore determined at 50 years (Ponce, 2017).

D.3 Design considerations

The design must meet a number of requirements regarding construction, operation, stability and safety. In this section, the different design aspects that determine the main parameters of the final dam design, see section 5.3, are discussed.

D.3.1 Dam material

The dam material is defined by the type of dam, the following four types are the most common for a reservoir (CIRIA, 2007):

- Earth dam
- Rock dam
- Concrete dam
- Masonry dam

Taking the local topography, soil and reservoir characteristic into consideration, the concrete dam and masonry dam are left out of this report, as they are less suited for this type of area. The earth and rock dam are discussed in the following sections.

Earth-fill dam

The earth dam is commonly selected if suitable soils can be obtained from nearby borrow areas. The soil will serve as fill material, preferably sand and clay. To minimize costs, the dam should be designed to maximize use of the most economical materials available, including materials which must be excavated for the dams' foundation and for secondary structures. The main advantage of the earth dam, is that it can adapt to a weak foundation. On the other hand, an earth dam is vulnerable against erosion which can reduce the integrity of the dam structure (Engemoen, 2012). That is why when choosing an earth-filled dam, the erosion of the dam and foundation is thoroughly tested, explored and taken into account for the final design.

The earth-filled dam can be classified into homogeneous, zoned or homogeneous with a relatively thin core. When applying an armour layer, this can be done on one of the following locations (Corps, 2004):

- On the upstream slope, together with an underlying filter as a protection against wave and current attack (near spillways and outlets).
- On the downstream slope with an underlying filter, as a protection against rainfall runoff.
- In filter drains near the downstream heel of the dam.

Usually the armour layer consists mainly of rocks. The different types of earth dams can be found in figure D.3.

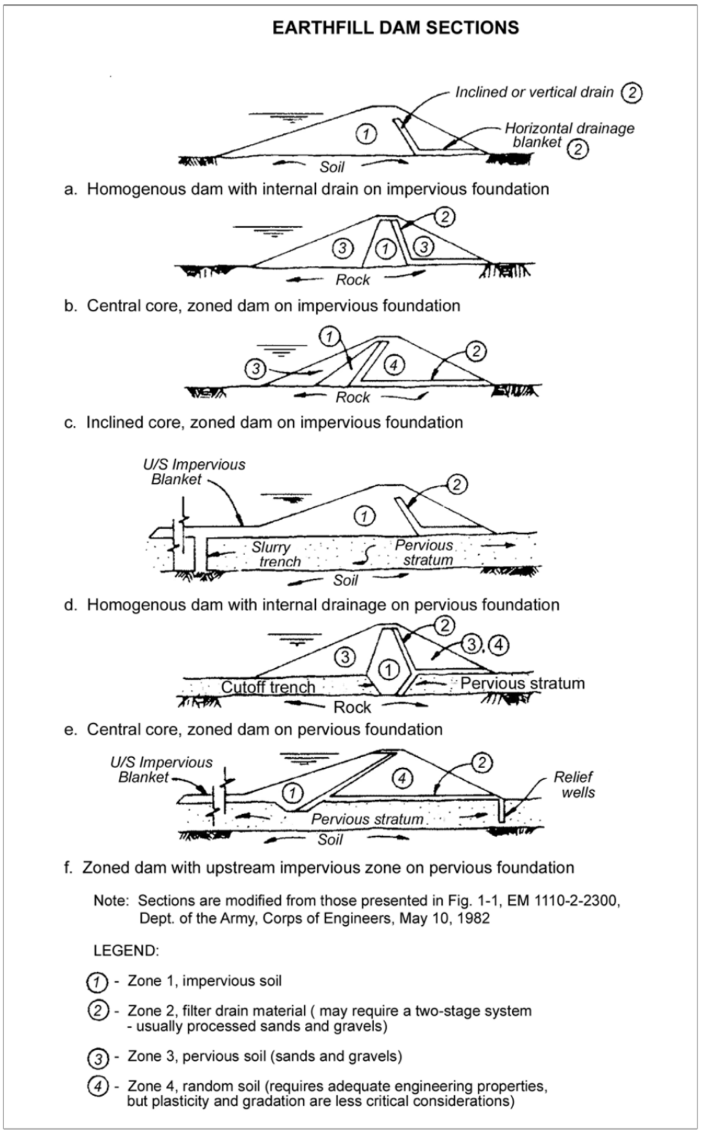


Figure D.3: Earthfill dam sections

Rock-fill dam

A rock-fill dam consists of an outer layer constructed from boulders and cobbles and an impervious core. The materials of a rock dam can range from free-draining rock to the more friable materials such as sandstones and silt-shales (Engemoen, 2012). When using sandstones and silt-shales, the embankment design is similar to earth dams, due to the seepage considerations, stability of undrained conditions and lack of shear strength. The core of the rock dam and its outer layer are separated by different transition zones. These transition zones are build-up from material with a pre-determined grading.

Due to the permeability of the boulders, an impermeable layer as a membrane is crucial so that excessive seepage can be prevented. A rock dam with a steep slope requires better foundation conditions than an earth dam, due to the minimum settlement that is allowed to keep the membrane in a good condition. A foundation built from rock or compacted gravelly sand are preferred for a rock dam.

An advantage of a rock-fill dam is that it can achieve a higher stability compared to an earth-fill dam, so it can have steeper slopes. The slopes can vary between 1:1,5 and 1:1,75 (V/H) for the upstream slope and 1:1,3 to 1:1,8 (V/H) for the downstream slope (Engemoen, 2012). A rock-fill dam can be an economical option when large quantities of rock are available from nearby borrow sources. Therefore, a rock dam is suitable for sites with a good access to nearby rock quarries, with the presence of hard rock near the surface for the foundation purposes and limited suitable local soil material. The different types of rock dams can be found in figure D.4.

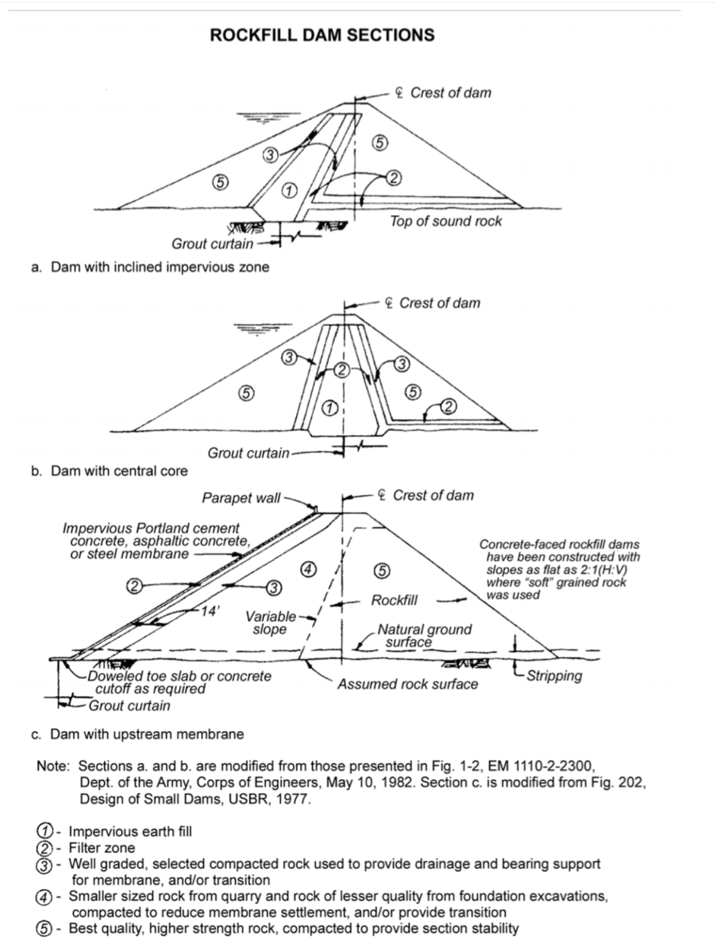


Figure D.4: Rockfill dam sections

D.3.2 Impermeable membrane

The impermeable membrane layer can be on the surface of the upstream slope or in the dam core. This membrane layer is critical for a rock dam and can be built of (clayey) soils, concrete, asphalt paving concrete, geomembrane or a steel plate. The first option requires a filter zone to be constructed between the outer layer and the impermeable core. This filter zone is built-up from an even grain size gradation in order to prevent water intrusion in the outer layer. For placing the membrane, two configurations are considered (Engemoen, 2012):

- Positioned in the centre of the dam.
- Positioned on the surface of upstream slope: called facing.

Positioned in the centre of the dam

There are two different ways to design the impermeable core near the centre of the dam; an inclined core or a vertical core (Engemoen, 2012). The advantage of the vertical core is that the former provides higher contact pressure between the core and foundation. This principle has the advantages of:

- To prevent leakage
- Greater stability under earthquake loading
- Better access for remedial seepage control

The advantage of an inclined upstream core is that it allows that the downstream portion of the embankment can be placed first. Another advantage is that an inclined upstream core reduces the possibility of hydraulic fracturing. The minimum thickness of the core is depending on different factors, such as (Mohammadi, 2011):

- Tolerable seepage loss
- Minimum width which will allow proper construction
- The type of material chosen for the core of the dam
- Design of proposed filter layers
- Past experience on similar projects

For high dams in steep-walled canyons, the topography of the location is the overriding consideration. The objective in this consideration is to fit the core to the topography in such a way to avoid divergence, abrupt topographic discontinuities, and serious geologic defects. The different design of the core can be found in figure D.5.

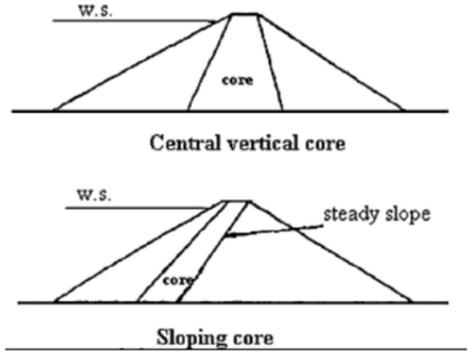


Figure D.5: Core sections

Facing membrane

This configuration covers the complete upstream side of the dam, making it more accessible for service and maintenance. The main difference with the membrane positioned in the centre is the soil conditions inside the dam body. Due to the placement at the 'front', the soil within the dam is considered to be mostly unsaturated. Upstream slope membrane should use either asphalt, concrete, or steel as the construction material (Engemoen, 2012).

Consideration

Advantages for placing the membrane in the centre of the dam are the following:

- No direct contact with water.
- Protected against weathering.
- Flexible when designing a foundation with less favourable soil conditions.

On the other hand, the placement of the membrane in front of the dam on its upstream slope is considered a good thing because of:

- Easier service and inspection.
- Flexible in construction because it can be installed during and after rock filling.
- Unsaturated soil conditions inside dam body, which gives more stability in both static and dynamic condition.
- Upstream membrane can serve multiple purposes

D.3.3 Granular and geotextile filters

Filter instability can occur when a flow within the pores of a layer conveys fine particles of granular materials or subsoil particle. These particles can flow through the pores of coarse granular materials or through those of geotextiles, which can cause deterioration of the structure or change its permeability. There are three types of filter instability (CIRIA, 2007):

- Internal erosion; This only occurs in filters with wide-graded materials. This phenomenon causes the finer particles to be transported through the voids connected with the coarse particles within the same layer.
- Interface instability with granular filters; When the particles of one base layer are transported through the pores to another filter layer where they blend with the particles of the other layer.
- Interface instability with geotextile filters; When the particles of the base layer are transported through the pores of a geotextile filter.

Granular filters

The filters should prevent the erosion of the fine grains. There are two different traditional design criteria (Fema, 2011):

- Geometrically tight (or closed)
- Geometrically open

Geometrically tight implies that the pores or opening sizes are too small to allow the fine grains to pass through. An advantage of these filters is that it is relatively simple to design due to the limited pre-required knowledge to make a design. Only the grain size distributions and the pore or opening size distributions of the filter need to be known. A disadvantage of this approach is that an uneconomically large number of filter layers are often required. It is important to note that a flexible approach should be adopted in the specification of granular filter layers, taking into account possible limitations of the local supply quarries (Kunitomo, 2000).

Geometrically open filters are based on the principle that the hydraulic load is small so it does not initiate erosion of the base (fine) material. The main disadvantage of this type is that more detailed knowledge of the hydraulic loads on the filters is required, because the water movement along and inside the structure.

Every filter has two functions (Verhagen, 2016):

- Prevention of the transport of fine particles.
- Filter permeability, a filter must allow for the transport of water, mainly to prevent excess pore pressures.

Geotextile layer

Geotextiles are permeable fabrics that have the ability to separate, filter, reinforce, protect or drain the different layers. It is mostly made out of polypropylene or polyester and three different types can be distinguished: woven, needle punched or heat bonded. Geotextile filters can only fulfil their function if the following two requirements are met (Fema, 2011):

- The layer is installed properly and with care. This prevents damage and ensures good overlap between panels. Extra attention is required for the following characteristics of the geotextile: elongation at maximum strength, absorbable energy, resistance to static puncture and resistance to dynamic perforation.
- Sufficient durability is guaranteed to maintain its initial functional properties, characterized by the long-term filtration performance and the resistance to aggressive environments (so as to maintain the initial functional characteristics).

D.3.4 Transition zones

The filter design for internal zoning of a dam is a critical part of the embankment design. The individual particles in the embankment need to be held in place. The transition zones can be varying for an earth and rock dam. They can have a function of a geotextile layer. The transition zones are usually more self-healing, so an increased width of these zones can be beneficial. Transition zones are necessary to prevent washing out. They should have sufficient thickness to control seepage through the layer, permit efficient placement with normal hauling and compacting equipment and minimize effect of differential settlement and possible cracking. It is less likely that a leak will develop when the width of contact area between the impervious fill and rock is at its highest. Different filters for the transition zones are given below (Engemoen, 2012).

- Drainage filters (Class I) – This type of filter is used mainly against the seepage within the dam and the foundation. It carries away the seepage as well as small particles. A uniformly grading is chosen consisting usually of two layers. The drainage filter has to meet the requirements for both drainage and small particle movement.
- Protective filters (Class II) – To protect the dam against erosion and from material at the base to wash away, a protective filter is considered. A protective filter may also serve as a drainage filter to control the pore pressure that can build up inside the dam. The grading of the protective filter has two main alternatives. The first one is the grading in zones and uniformly. The second alternative is usually chosen when a better transition with the bas material is required, this is done by grading the material broader so that the number of zones is reduced.
- Choke (inverted) filters (Class III) – Mainly used in emergency situations, with a main purpose of overlying fill from moving. Not further discussed because irrelevant for our design phase.
- Seismic crack stoppers (Class IV) – As the location of the dam is in a seismic active area, seismic crack stoppers should be considered in the design. Its function is protecting the core of the dam against cracks, which are mainly an effect of large deformation as a result of seismic loading. Design of the layer concerns mainly the expected displacements due to the seismic activity. Permeability is less of a requirement as it does not influence the other layers significantly, but no fines are allowed in the filter because it does not sustain a crack. When fines are possible, a second layer should be considered that can sustain cracks or the propagation of one. This second layer can also be used as a transition zone to a coarser material.

Table D.1: Transition zones (Engemoen, 2012)

Class	**Filter type**	**Uses**	**Multiple stages required**	**Uniform gradation required?**	**Permeability or drainage required?**
I	Drainage	Toe drains, relief wells, drain fields	Yes	Yes	Yes
II	Protective	Downstream chimneys, blankets, transition zones	Frequently	No	Yes
III	Choke	Foundation filters, sinkhole backfill	No	No	No
IV	Crack Stopper	Upstream and downstream chimneys	Frequently	Yes	No (although the filter should not sustain a crack)

D.3.5 Slope cover material

Slope protection is needed for all earth and rock-fill dams as protection against wind and wave erosion, weathering, ice damage, and potential damage from floating debris. There are different materials which can be used for the slope cover, such as (CIRIA, 2007):

- Dumped rip-rap (graded heavy rock fragments)
- Quarry-run stones
- Pavements of concrete
- Asphaltic concrete
- Soil cement
- Bituminous soil stabilization
- Sodding
- Planting

Factors that influence the type of slope protection are embankment damage, required excavated material, availability and quality of nearby quarries and turfs. Due to the high costs of the slope cover, an efficient design should be accomplished during the survey studies to establish a reliable cost estimate.

Upstream slope

The upstream slope needs to be protected mainly from wave action. Rip-rap is a common material for the slope cover, which is tight, strong and durable (Braen, 2017). It has the characteristics of a wide grading, generally bulk placed, often placed as a cover layer and frequently used in estuarial and river bank application. An advantage of a rock cover is the availability, price, the flexibility and damage progression. Good materials for this cover type are igneous, metamorphic rock and strong limestone (Kutzner, 1997). The thickness of rip-rap layer depends on the considered maximum wave height, which is given in several literatures based on empirical approach and summarized in table D.2. The table is only valid for 1:1 steepness slope, less steep slope may require different criteria (Kutzner, 1997).

Table D.2: Slope cover material (Kutzner, 1997)

Maximum wave height	**Layer thickness**	**Mean values of individual pieces**		**Maximum values of individual pieces**	
[m]	[cm]	[kg]	[cm]	[kg]	[cm]
<0,6	40	20 - 50	20 - 25	80 - 100	30 - 35
0,6 - 1,2	50	50 - 100	25 - 35	150 - 300	40 - 50
1,2 - 1,8	60 - 70	150 - 250	40 - 45	400 - 800	55 - 65
1,8 - 2,4	80 - 90	250 - 500	45 - 55	1000 - 1500	70 - 85
2,4 - 3,0	90 - 120	500 - 1000	55 - 70	1800 - 2500	85 - 100
>3,0	>120	1200 - 1500	75 - 85	3000 - 4000	100 - 115

A different approach for the upstream slope is to use quarry-run stones. This approach can consist stones that may be of a less quality than rip-rap. The thickness of this protection layer is bigger than the thickness of the rip-rap protection layer (CIRIA, 2007). The thickness of the quarry-run stone layer depends on the material quality and size. An advantage of quarry-run stone l compared to rip-rap is that its gravel- and sand-size components serve as a filter.

The other materials require a different design approach and the maintenance is costly. Slopes less steep than 1:15 seldom require slope protection. Slopes that have a steepness of 1:6 and flatter can be traversed easily by construction and maintenance equipment(Kutzner, 1997). In a rock-fill dam a slope can be varying between 1:1,5 to 1:1,5 (V:H) (CIRIA, 2007). The use of high-density rock will result in a smaller armour stone and hence a reduced layer thickness.

Rear side slope

As the dam design will consider no wave overtopping, so the downstream slope should only be protected from damage and erosion from surface runoff due to rainfall. There are a lot of different slope protection types for the downstream slope. The vegetative cover is usually the most desirable type. For this type of protection slope a slope steepness of approximately 1:3 (V/H) is the steepest on which mowing and fertilizing equipment can operate efficiently (CIRIA, 2007).

An alternative to grass is a rock-fill material slope protection. This material can have a steeper slope than a vegetative cover. A rock toe on the rear side slope is beneficial for the stability and erosion. The slope can be varying between 1:1,3 to 1:1,8 (V/H) (CIRIA, 2007). However, this option is costlier than the grass cover.

D.3.6 Crest height/width

Crest height

The crest height is depending on different factors, such as:

- The expected local ground subsidence during the design period
- Settlement due to seismic event
- The expected decrease in crest height due to settlement of the dike body and the foundation soils during the design period
- Lake level increase during the design period
- Water level rise during storm condition (Probable Maximum Flood (PMF) event)
- The wave run-up height and the wind set-up height

The wind set-up and wave run-up has to be taken into account by the design of the crest level of the dam, see figure D.6. The crest level of the dam will be designed with aspect to a zero overtopping and a spillway. Also, the consideration of the settlement of the dam material or foundation either due to static dead load or seismic load has to be taken into account.

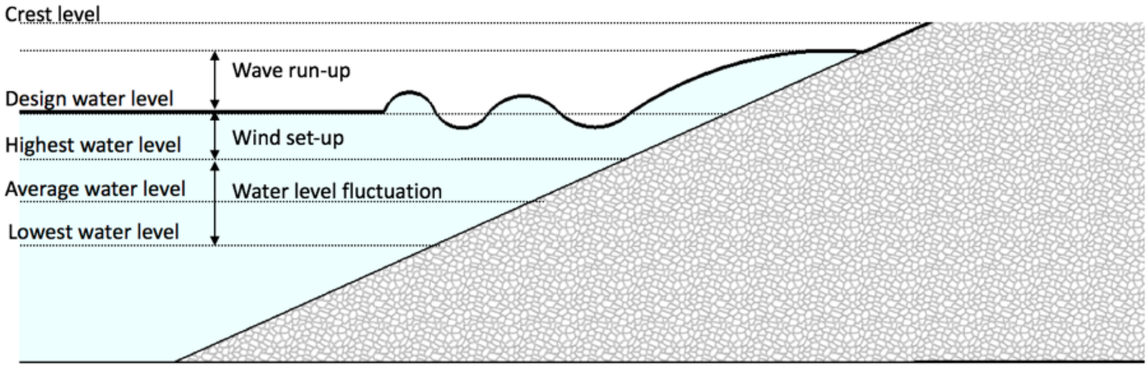


Figure D.6: Crest height

Crest width

In terms of stability, the width of the dam is not considered to be a major influence. Its main purpose is increase the strength of the dam against static loading due to the added weight that comes from a wider dam crest. Taking the seismic activity into account, increased width leads to more safety in the event of seismic activity. After such an event sliding can occur endangering the reservoir, with a wider crest more dam will remain to contain the water body inside the reservoir. Another important aspect that determines the design width is its operational use. This operational use consists of activities such as maintenance and inspection, but can also consist of roads for public use.

D.3.7 Dam foundation

The dam foundations are divided into three classes (Engemoen, 2012):

- Foundations consisting of rock
- Foundations consisting of coarse-grained material (sand and gravel)
- Foundations consisting of fine-grained material (silt and clay)

The last two classes are classified as earth foundations. Many foundations are not characterized by a single material, but by a combination of structural arrangement and physical characteristics of their constituent materials.

Rock foundations

Although rock-foundations should be built on a surface without loose fragments and clean from projecting knobs, in order to avoid differential settlement, they are generally considered more competent than the ones made of soil. Also, the cracks, joints and openings beneath the core need to be filled with lean concrete or mortar, depending on the size of them. An earth dam can be constructed on a 'cracked' rock foundation, but it is essential to prevent the embankment material from pouring into the openings of the rock. Excessive reservoir seepage or possibly piping can occur when embankment material flows into the joints. Weak zones or layers need to be located so that the foundation's stability, under the applied load of the embankment and reservoir, will be maintained. It is important that the search for weak layers of clay during exploration work happens carefully, since they could be only a few millimeters thick. The advantages from a rock dam are a very large load bearing capacity, resistance against erosion and reduced permeability. Even 'weaker' rock foundations are often preferred over earth foundations (Engemoen, 2012).

Earth foundations

The design of dams on earth foundations is based on the in-situ shear strength of the soil materials (CIRIA, 2007). The method of an earth foundation under a rock-fill dam is the same as that for an earth-fill dam. When there is a weak foundation, strengthening or excavation of undesirable material is mostly more economical than using flat slopes or stability berms. After the surface has been stripped, in order to prepare for the foundation, it will need to be compacted due to the loose condition it will have at that moment. When differential settlement of an embankment takes place, a crack along the longitudinal axis could emerge due to tension zones along the upper portion of the dam. This possible cracking can occur in the presence of steep abutment slopes, closure sections or where thick deposits of unsuitable foundation soils have been removed. Also, transverse cracks in the embankment can occur by differential settlement along the dam axis, which can lead to undesirable seepage conditions. The surface layer of the foundation beneath the downstream rock-fill dam needs to either meet the filter gradation criteria, or a filter layer must be provided, so that seepage from the foundation does not cause the material to flow into the rock joints.

D.3.8 Seepage

The seepage through the foundation and underneath the dam must be controlled and collected to ensure safe operation. This is done to avoid excessive uplift pressure, piping of materials and sloughing removal of material by solution or erosion of this material intro cracks, joints and cavities. To determine the design of the protection against seepage a clear image of the subsoil must be retrieved by boring an in-situ permeability tests. In general, there are three different measures against seepage:

1. Cut-off trench
   The principle off the cut-off trench, see figure D.7, is based on a shallow soil layer with beneath that layer a hard rock present. When excavating a trench out of the top layer, a shallow foundation can be built on the hard rock layer. The trench is usually along the dam alignment with a certain slope inclination which increases stability. The shallow foundation is generally made from impermeable core material to counteract against the seepage. This material is usually layered and compacted until it is at ground level. The main risk is the connection between the rock layer and the foundation, therefore care must be taken for the excavation.
2. Diaphragm wall
   When no hard rock is present closely underneath the soft top soil, or the soil underneath the dam is thick and hard to excavate, a diaphragm wall is used, see figure D.8. This type of seepage protection can be constructed in two different ways. When an impermeable layer is present in the soil underneath the dam, it is possible to construct a sufficiently long impermeable structure, so that it reaches from the bottom of the dam until the impermeable layer. The second method consists of calculating the maximum seepage length. Now a single impermeable wall of sheet pile can be inserted if sufficient, otherwise two separate structures can suffice. However, this is usually the case with relatively long horizontal seepage lengths, which is not the case for this dam.
3. Jet grouting
   This method is mainly used for locations and subsoil where the first two options cannot be constructed due the presence of old or operational structures in the subsoil. Jet grouting, see figure D.9, is a form of soil injection, where a seepage cut-off wall is grouted in overlapping columns. As the design area has no history of human presence, the subsoil contains no former structures.

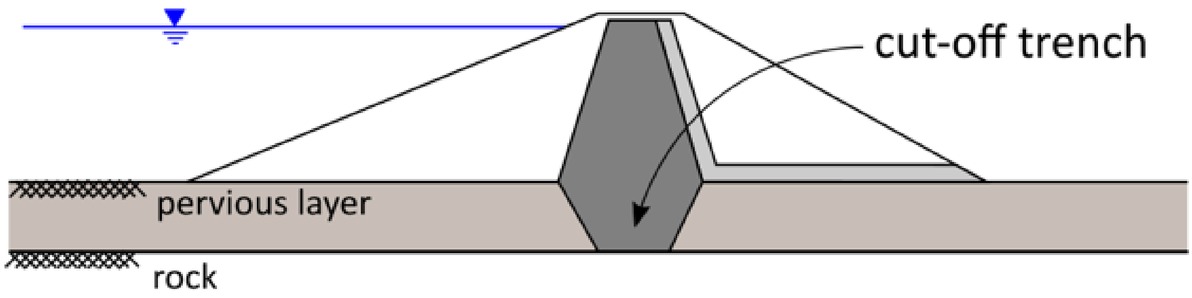


Figure D.7: Illustration of cut-off trench in a central core earth dam

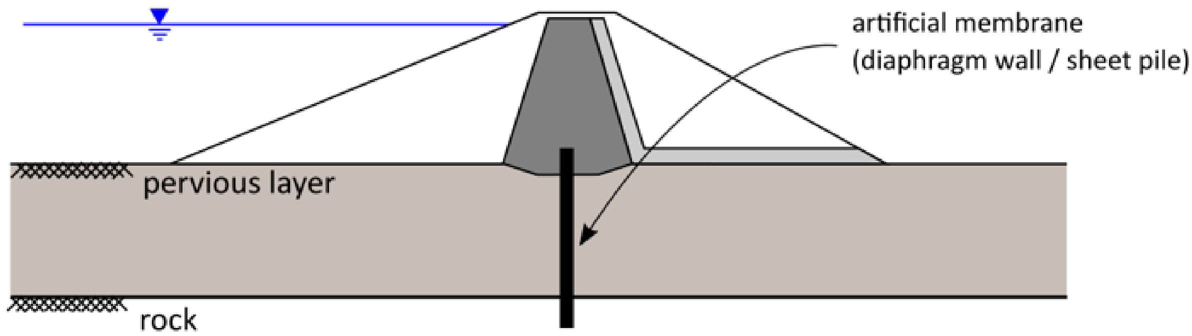


Figure D.8: Illustration of vertical structure in a central core earth dam

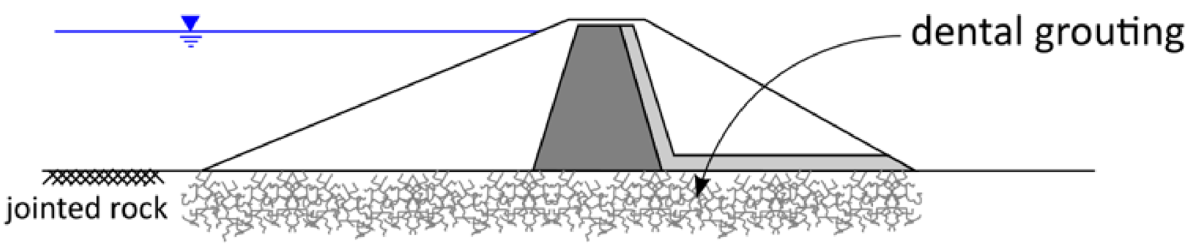


Figure D.9: Illustration of vertical structure in a central core earth dam

D.3.9 Outlet works

Outlet works are designed to control the release of outflow discharge from the reservoir. Functions of an outlet work include flood control, navigation, irrigation, water supply, hydro power, regulating low-flow requirements, diversion and drawdown. Generally for embankments, conduit or tunnel type of outlet work is considered.

An outlet work can consist of an intake structure, a conduit or a tunnel, approach channel, a control gate chamber, an exit chute, an energy dissipater, and a discharge channel (Corps, 2003). Selecting the type of outlet work structure should be based upon the costs of operation and total maintenance costs. When designing the outlet works, one could consider reducing the frequency of spillage by increasing the capacity. However, this is not done in the design. The main reason being that the reservoir and the extraction yield is dimensioned so that a maximum constant discharge can be achieved, without the reservoir depleting. When the outflow is increased, over time, the reservoir will deplete. Implicating that the outlet work should be designed for this constant discharge and regulated because of the fluctuating water head. For the preliminary study mainly the pipeline, tunnel or conduit, and intake structure are discussed.

Intake

Within the reservoir, the water level will fluctuate over time, however the discharge is required to be constant. Therefore, a control system is needed at the intake structure. This can either be done by gates, valves or other control mechanisms.

Pipeline

The transport from the intake, across the embankment to the other side of the dam is done by pipes. For constructing the pipes, two main types are generally used. The tunnel type and the conduit type. The topography, foundation conditions, hydraulic requirements, costs of construction and geology of a site determine the design choice.

Tunnel type
An outlet tunnel is drilled underneath or aside the structure and therefore completely detached from the dam or the foundation. The main upsides of this type being the small chance of foundation settlement, differential movement, and structural displacement (Kharagpur, 2017). Also, it is not likely that failure of some portion of a tunnel would cause failure of the dam. However, depending on the subsoil characteristics, the chance of seepage increases when the tunnel is constructed in a permeable layer. Therefore, this type of outlet is mainly used in rocky subsoil, where it is a safe and durable option. When properly designed and constructed, a lifespan of approximately 50 years can be achieved. If there is unfavorable foundation geology present, such as deep over-burdens or inferior foundation rock, a tunnel scheme cannot be selected. On the other hand, sites in narrow canyons with steep slopes may make a tunnel outlet the only choice (Kharagpur, 2017). The selection of the number and size of tunnels depends on hydraulic requirements, the maximum size of tunnel for the rock conditions encountered, and overall economics (Corps, 2003). Excessive movement in the surrounding rock can be prevent by limiting the size of the tunnel in yielding rock. The larger the tunnel within practicable driving limits is, the more economical it is per unit flow capacity.

Conduit type
The conduit type is, other than the tunnel type, part of the dam structure. Either the conduit is part of the embankment itself, or it is placed within the foundation (Corps, 2003). This type is generally defined as a cut-and-cover conduit, as it is created before construction of the embankment using the cut-and-cover method. A conservative design is necessary due to strong connection with the embankment. This can cause leakage within the dam, that eventually can lead to piping. The need for a conservative design has been demonstrated by numerous failures of earth dams caused by improperly designed or constructed cut-and-cover outlet conduits. The conduit needs to be placed on suitable rock to keep settlement as small as possible. Cut-and-cover conduits are economical for low-head embankment dams because of relatively low construction costs, as high-head dams will require a thick concrete section to resist high embankment loads, increasing construction costs. A cut-and-cover conduit can be built in almost any size if it is precast of cast-in-place. The dimensions of the conduit are merely determined by the construction requirements, instead of the hydraulic aspects (Kharagpur, 2017).

Foundation properties

The principal foundation properties for outlet works design are bearing capacity, deformation and elastic modulus, and shear strength. These properties determine suitability of the rock mass to support the structure placed on it and the resistance to sliding at the structure/foundation interface. Dynamic elastic modulus and shear strength are important factors in the seismic design of these structures. Also very important to stability analysis is whether any weak zones exist within the foundation where sliding failure could occur. Such zones may consist of shears, faults, or other discontinuities along which material of low strength such as clay gouge may be present in an otherwise sound rock mass. Significant voids such as those produced by dissolving of some limestone formations also influence foundation suitability. The foundation properties must be investigated early in the design phase of the project to determine design parameters and any remedial treatment that may be necessary during excavation and construction. The structural design and contract drawings should include provisions for unforeseen conditions which may require that the intake structure be founded at a lower elevation in order to reach competent rock.

In order to check the technical feasibility of the tunnel structure, future site investigations should focus on the following factors (de Vries, 2017):

- Geotechnical properties of the rock mass units at tunnel level and at tunnel portals.
- Orientation of bedding planes and discontinuities of the rock mass units.
- Soil cover and weathering profile at tunnel portals.
- Presence and character of faults.
- Permeability of the rock mass.
- Ground water level.

D.3.10 Spillway

A spillway is an important part of the dam, constructed to dispose surplus floodwater safely which cannot be stored in the reservoir. It is necessary to construct a spillway with sufficient capacity. Because the dam is designed for no overtopping, the spillway must avoid this from happening, as overtopping may lead to failure of the dam. A spillway must meet the following requirements:

1. A sufficient capacity, avoiding the reservoir volume to exceed the crest height.
2. Safe disposal of the excess water, without causing toe erosion or other damage to the dam.
3. The spillway must remain stable under both static and dynamic loads, as well as be resistant to erosion caused by the high scouring.
4. The energy, created by the outflow of the excess water, should be dissipated on the downstream side of the dam by a dissipation work.

There are two main types of spillways to distinguish, the first one is a controlled spillway which regulates the outflow using gates or valves. The second option is an uncontrolled spillway. This configuration is static and is only put into operation when the surface level inside the reservoir reaches a critical level. From this point, all the excess water is extracted from the reservoir through the spillway. Therefore, the spillway design must include the water level at which it will start functioning. For an uncontrolled type, this parameter is pre-determined and static. The controlled type can vary in the critical level at which is starts operating.

Another important parameter in the spillway is the outflow capacity. This parameter is strongly related to a peak discharge with a long return period. As the dam is designed with no overtopping, exceedance of the spillway capacity is highly undesirable and therefore a peak discharge with a long return period should be chosen.

Picking a location depends on merely two main aspects. The first one is quite trivial, the water cannot have an outflow location upstream of the reservoir, as it will flow right back in. This implies that the southern half of the reservoir does not qualify as outflow location. Secondly, the storage capacity of the outflow location is to be considered. When the outflow is to a nearby reservoir or its watershed, one has to take into account the storage capacity of that reservoir, as well as its spillway configuration. When both are similar or smaller than the Samarinda reservoir, outflow into the nearby reservoir is undesirable, as the flood is likely to be also present in the nearby reservoir, as the flood is mainly completely caused by the local rainfall. When considering outflow to a different location, it is best to choose an unpopulated and unbuilt location, as the discharge will flow out freely. This option is usually the case with emergency spillways, with a long return period.

There are three options main options for the spillways' location (LaBoon, 2014). Either the spillway is located at the dam location or elsewhere in the reservoir, either a controlled option with gates or valves, or a free inflow and uncontrolled structure like for example a morning glory. The first one is a more expensive and more extensive option as it requires a lot more construction, safety measures, design effort and money. The spillway at the dam location needs to be designed in such a way that the dam and none of the nearby structures such as a possible hydro power generator and treatment plant are affected by the excess water. The second option concerns a natural outflow to a nearby area. When choosing the uncontrolled morning glory option, the design and construction are a lot simpler and consist mainly of an inlet for the water at a predetermined level and a tunnel or pipeline to location outside the watershed where the excess water does no harm to the population and environment. As mentioned before, a tunnel to a nearby area can also be regulated by a control structure. This is far more expensive, but gives the opportunity to have a free choice in location, as it can be beneath the water surface without water flowing out.

Figure D.10: Side channel spillway

Figure D.11: Culvert spillway type

Figure D.12: Morning glory spillway

D.4 Design calculations

These calculations are used in section 5.3.

D.4.1 Slope stability

The criteria of the Hudson formula are:

- The use of regular waves only
- No account of the wave period and the storm duration
- No description of the damage level
- The use of non-overtopped and permeable structures only.

The structure will have an impermeable core and so the Hudson formula cannot be used.

The D_{50} for the armour layer can be calculated with the van der Meer formula (van den Bos, 2017):

$$\frac{H_s}{\Delta D_{n50}} = 0,7 * (K_D * cot\alpha)^{1/3} * S_d^{0,15}$$

In which:

- H_s is the wave height, which is 0,4 [m]. See section 5.3.6.
- Δ is the relative buoyant density, which can be calculated with $\Delta = \frac{\rho_r}{\rho_w} - 1$
  - ρ_r is mass density of the rock, which is 2600 [kg/m^3].
  - ρ_w is the mass density of water, which is 1000 [kg/m^3].
- D_{n50} the average diameter from the amour layer [m].
- K_D the Hudson value, which is for structures with an impermeable core: $K_D = 1$ and structures with a permeable core: $K_D = 4$. So for this design $K_D = 1$.
- α is the degrees of the slope [°], which is 1:1,75 (V:H) on the upstream side and 1:1,5 (V:H) on the downstream side.
- S_d is the damage level parameter [-], 0 to 5% damage generally referred to as the no damage condition. The S_d values are given in table D.3. So for this design $S_d = 2$ [-].

Table D.3: Damage with corresponding damage level (van den Bos, 2017)

Armour type	**Relative wave height**	**Damage D (per cent) with corresponding damage level S_d**						
		0-5 ($S_d = 2$)	5-10 ($S_d = 6$)	10-15 ($S_d = 10$)	15-20 ($S_d = 14$)	20-30 ($S_d = 20$)	30-40 ($S_d = 28$)	40-50 ($S_d = 36$)
Smooth	$H_s/H_{s;D=0}$	1,00	1,08	1,14	1,20	1,29	1,41	1,54
Angular	$H_s/H_{s;D=0}$	1,00	1,08	1,19	1,27	1,37	1,47	1,56

With this formula, it follows that the D_{n50} is:

$$D_{n50} = \frac{\frac{H_s}{\Delta}}{0,7*(K_D*cot\alpha)^{1/3}*S_d^{0,15}} = 0,27m$$

$$D_{50} = 1,2 * D_{n50} = 0,32m$$

The dynamic stability number N_s, needs to be between around 1 for statically stable structures. This is a classification and not a design formula. This can be calculated with (van den Bos, 2017):

$$N_s = \frac{H_s}{\Delta D_{n50}} = \frac{0,4}{1,6*1,27} = 0,94[-]$$

The calculated D_{n50} satisfied the criteria of the dynamic stability.

The effective thickness for the armour layer is calculated with:

$$t = n * k_t * D_{n50}$$

In which 't' is the effective thickness, 'n' the number of stones in the layer, k_t is the layer thickness coefficient (-) and D_{n50} the nominal diameter. The layer thickness coefficient (-) can be found in table D.4.

Table D.4: Layer thickness coefficient (van den Bos, 2017)

		Reference survey method		**Highest point survey method**	
Layer and placement type	**Parameter**	Blocky rock *BLc=0.65*	Irregular rock *BLc=0.50*	Blocky rock *BLc=0.65*	Irregular rock *BLc=0.50*
Single dense	k_t, k_{thp}	0,84	0,77	0,89	0,82
Single dense	n_v, n_{vhp}	32	35	36	40
Double standard	k_t, k_{thp}	0,91	0,87	0,96	0,92
Double standard	n_v, n_{vhp}	32	35	36	40
Double dense	k_t, k_{thp}	0,91	0,87	0,96	0,92
Double dense	n_v, n_{vhp}	31	32	35	36

$$t = 2 * 0,87 * D_{n50}$$

D.4.2 Wave height

The wave height is calculated using the maximum wind speed (most conservative condition, Beaufort scale 7, 50–61 km/h). The wind height is calculated with a limited Fetch. This result in a wind height of ((van der Meer; Allsop ;Bruce; De Rouck; Kortenhaus; Pullen, 2016):

$$H_{m0} = 5,112 * 10^{-4} * u_a * F^{\frac{1}{2}}$$

In which:

- u_a is the wind speed [m/s], which is 16,94 m/s
- F is the fetch [m], which is 2000m

$$H_{m0} = 5,112 * 10^{-4} * 16,94 * 2000^{\frac{1}{2}}$$

The period of the wave is calculated with:

$$T_m = 6,238 * 10^{-2} * u_a * F^{\frac{1}{3}} = 2,02s$$

D.4.3 Wind set-up

The wind set-up can be calculated with the following formula: (Voorendt, 2017):

$$\frac{dS}{dx} = C_2 * \frac{u^2}{g*d}$$
$$S = \frac{dS}{dx} * F = 2,56 * 10^{-6} * 2000 = 0,005m$$

In which:

- C_2 is constant around $3,5 * 10^{-6}$ to $4,0 * 10^{-6}$ [-]
- S is the total wind set-up [m]
- d is the water depth [m], which is 40m average (see chapter 3)
- u is the wind velocity [m/s]
- F is the Fetch [m]

Wave run-up

The wave run is calculated with (van den Bos, 2017):

$$\frac{q}{\sqrt{g*H_{mo}{}^3}} = 0,2 * exp(-2,3 * \frac{R_c}{H_{m0}*\gamma_f*\gamma_\beta}$$

In which:

- R_c is the wave run-up height [m]
- q is the overtopping discharge, which is calculated with 0,0001 $[m^3/s]$
- H is the wave height, which is 0,4 [m]
- γ_f is the roughness factor, which is 0,55 for quarry stone [-]
- γ_β is the berm-reduction factor, which is 1 [-]

The wave run up height is 0,7m.

Appendix E

Stakeholder analysis

E.1 Stakeholders description

Arsari Tirta Pradana and ITCI

Arsari Tirta Pradana and ITCI are both subsidiary companies of Arsari Enviro Industri. Arsari Tirta Pradana is the owner of the clean water projects, such as the Samarinda Water Project and the Arsari lake for the city of Balikpapan. ITCI, International Timber Corporation Indonesia, a former American and Indonesian military owned timber company is the owner of the concession on which the reservoir lakes, of phase 1 and 2, are to be built. The concession, demarcated with a yellow line in figure E.1 consists of 70% degraded forest and 30% primary forest. As can be seen in figure E.1, the concession lies in several different Districts in the province of East Kalimantan. Including a possible connection to phase 1, the phase 2 project will take place in the District of Kutai Kartanegara and the District of North Penajam Pasar.

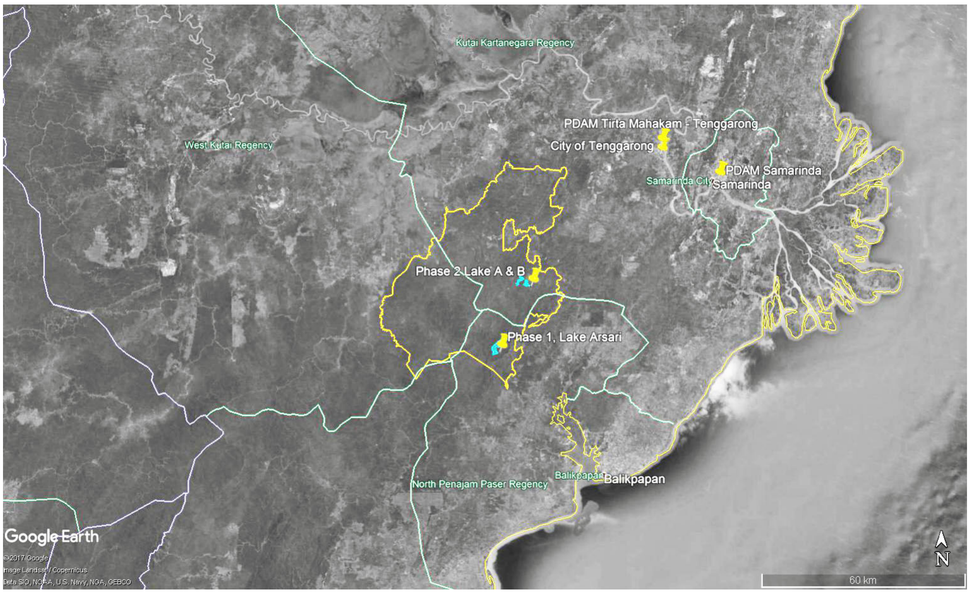


Figure E.1: ITCI Consession, Arsari lake (phase 1) and the Samarinda water project lake (phase 2) *(Google Earth)*

National government

Within the national government several ministries share a responsibility for the clean water provision (ASB, 2016; The Water Dialogues, 2009). The Ministry of Public Works determines policies, standards, norms, regulations, guidelines and manuals known in Indonesia as the NSPM for all national water resources, water supply and sanitation (The Water Dialogues, 2009).

The Ministry of Home Affairs issues guidelines on water tariffs, PDAM management and performance assessment and monitoring. Additionally it is responsible for the supervision and monitoring of local governments and their performance and they are responsible for the overall management and support of local clean water supply organisations (The Water Dialogues, 2009).

The Ministry of Finance is the owner of all water supply development related assets that have been funded by the state. Additionally, they govern and supply any government related loan to clean water supply organisations (ASB, 2016; The Water Dialogues, 2009).

The Ministry of Health is responsible for the water quality standards and the quality of water supplied by the clean water supply agencies. The clean water providers have to comply to these standards and the Ministry has the right to inspection and monitoring of their clean water supply (The Water Dialogues, 2009). Between 20010 and 2013 the Ministry of Health recorded a decrease of approximately 20% in households with access to clean water in East Kalimantan (ASB, 2016).

The Ministry of Environment and Forestry is responsible for monitoring water quality and pollution control of the clean water sources (ASB, 2016) Additionally they are responsible for all policies, implementation of environmental programs, environmental affairs with public participation in regard to water pollution and control and the operation of the Environmental Impact Management Board (The Water Dialogues, 2009). The Ministry of Environment and Forestry has rated the river water quality of East Kalimantan with"Polluted" in their "State of the Environment" report of 2012. The Ministry of Mines and Energy is responsible for granting of exploration permits and groundwater exploration for mines and energy production (The Water Dialogues, 2009). The National Planning Agency is responsible for all medium and long term development programs in regard to their program coordination, budget, effectiveness and future program planning (The Water Dialogues, 2009).

Provincial government

The province of East Kalimantan consists of six districts and three cities (Kartanerga, 2016). For the Samarinda Water Project the District of Kutai Kartanegara, in which the city of Tenggarong lies, and the city of Samarinda and its municipal governing body are of importance. Since the implementation of decentralisation in 2001 the regencies and municipalities are the head administrative units (of The Republic of Indonesia The Hague, 2013). The provincial government describes a decrease in water quality from 2012 until 2014 and a slow increase in water quality in the period 2014-2016 (Sidata, 2016). The client was requested by the current Governor of East Kalimantan, Dr. H. Awang Faroek Isaac, to look into the possibility of creating a new clean water source on the concession of ITCI with the purpose of providing the cities Tenggarong and Samarinda with clean water.

Regional government: *District of Kutai Kartanegara & Municipality of Samarinda*

As previously mentioned, since the implementation of decentralisation in 2001, the regional governing bodies, districts and municipalities, have become the key administrative units (Sidata, 2016; of The Republic of Indonesia The Hague, 2013). Due to the decentralisation, most matters regarding permits, land ownership and financial aspects such as a water selling price increases and subsidies have to be engaged through the District and Municipal governing bodies. Samarinda is the capital of East Kalimantan and it is divided into Samarinda and Samarinda Seberang by the Mahakam River. Although the city of Samarinda is completely surrounded by the District of Kukar, it has its own governing body. Tenggarong is the capital city of the district of Kukar. Tenggarong is also divided into Tenggarong and Tenggarong Seberang by the Mahakam river. The combined size of the population of Tenggarong and Samarinda is 1.019.484, which is expected to rise to over 1.600.000 by 2035 (Badan Pusat Statistik Indonesia, 2013).

Local government owned water company: *Perusahaan Daerah Air Minum (PDAM)*

The PDAM are local, government-owned utility organisations responsible for all clean water services provided to local inhabitants and industry (ASB, 2016; The Water Dialogues, 2009). They are responsible for quality control on the clean water supply, on which the Ministry of Health is allowed to perform inspections and monitoring for quality assurance ((BPPSPAM), 2016; The Water Dialogues, 2009). In this responsibility, quality control of water sources is also present, from the national Government, the Ministry of Environment and Forestry carries the right and responsibility for inspection and quality assurance (ASB, 2016). For both PDAMs, the main source of clean water is the Mahakam river, of which the quality is described in contradicting ways. Figures E.2 and E.4 show the current main water intakes for the City of Tenggarong and Samarinda.

Figure E.2: PDAM Mahakam River pumping station
(own work)

Figure E.3: Samarinda PDAM office entrance
(own work)

Figure E.4: PDAM Tenggarong Mahakam River pumping station
(Google Earth)

Figure E.5: Tenggarong PDAM office
(Own work)

Contractors

There will be multiple stages in constructing the artificial storage lake. For now, the area is still highly vegetated, therefore no infrastructure is present. A road has to be constructed to access the area. This will presumably done by Arsari itself. Other activities are cutting down forest, constructing a dam, hydro power, water treatment facility and constructing a pipeline. It is not known who will be the constructing, operating and/or maintaining contractor for these activities and how the contracts for these will be set up.

Secondary product buyers

No farmland has to be relocated as a result of the deforestation of the construction site. However, biomass will be obtained by deforesting the artificial storage area. The acquired biomass has a special purpose. The goal of the obtained biomass is to produce biochar. Biochar among other things helps greatly with increasing the fertility of the soil and diminishes the need for artificial fertilizer, contributing to the productivity and sustainability of agriculture. The biochar will be mostly given/sold to farmers that cultivate Sugar Palms. Subtraction of sugar oil is labour intensive work, therefore it will add to an increase in local jobs.

Future employees

Arsari has a strong focus on collaboration with local inhabitants and involvement and local provision of work is an important part of their vision because it allows for better cooperation and nature protection and preservation. Arsari not only trains their employees for their daily tasks and activities but educates them on how to implement and spread the research and work done and executed by Arsari. This gives extra weight to the stake the future employees have in this project because their involvement is part not only of the success of the Samarinda Water Project but also of the entire philosophy with which Arsari wants to create a better future.

Figure E.6: Arsat (left) and Joe (right) two current Arsari (ITCI) Employees *Fotograph by Pieter Zaalberg*

Private landowners

The water will be transmitted from the artificial storage lake to the cities of Tenggarong and Samarinda. It will be transported with the use of a pipeline infrastructure and the water will traverse approximately 80 km on land. This land is owned by either the government or private parties. The land of private ownership will have to be bought. Regulation about the government land has to be checked. The route proposed in chapter 6 does not cross privately owned land or area's of agricultural interests such as palm oil plantations.

Villages along the pipeline

With the current proposed pipeline route no villages are near the pipeline path. They are considered a stakeholder because in case the pipeline does cross their villages, connecting them to the pipeline can be found necessary to prevent sabotage and public uproar. Some rural villages are not or not sufficiently connected to water infrastructure and often depend on raw water sources for untreated water such as wells and rain (ASB, 2016).

Figure E.7: Rainwater being captured for household use *(own work)*

River users

Different types of usage can be defined. For the stakeholder analysis the types of river usage is defined as *water based transportation* and local inhabitants using the river for economic and social purposes. These users might experience negative consequences because of the construction of the river crossing part of the pipeline. In the proposed route the pipeline is attached to the Kartanegara bridge shown in figure E.8. This will cause partial or complete closing of the river during construction. The distance between the pillars of the bridge is 250 meters. Because of this large distance, partial closure is a possibility, which would have a limited impact on the water based transportation. Local inhabitants use the river for different purposes such as fishing, recreation and washing. Their usage can be of importance in forcing the PDAM to cooperate or to create a sense of urgency for the proof of pollution (Darma, 2017).

Figure E.8: Kartanegara Bridge

Industrial customers

There are many different industrial customers in both Tenggarong and Samarinda. Many of these industrial customers have a need for clean water, not for consumption but for industrial purposes. The industrial customers present in Tenggarong and Samarinda are:

- Agriculture, forestry, hunting and fisheries
- Mining and Quarrying
- Manufacturing Industries
- Municipality electricity & gas
- Construction
- Wholesale trade & retail trade
- Transportation, warehousing & communication
- Financial, Insurance, Real estate and other business services
- Community, social and personal services
- Chemistry and pulp industry

(PDAM Samarinda, 2017; PDAM Tenggarong, 2017)

Non-industrial customers

Many non-industrial customers are present in both Samarinda and Tenggarong. Among these are not only inhabitants but all customers of the PDAM who need this water for consumption purposes. This is the most vulnerable group of stakeholder. This group will be influenced the most if large price increases take place or in case of heavy pollution. Among the non-industrial customers are the most vulnerable groups such as children, elderly and sick inhabitants of both cities. Currently, many of the non-industrial customers rely on bottled water, the water trucks or provisional solutions such as can be seen in figure E.9 to full fill their need for clean water. Non-industrial customers in Tenggarong and Samarinda are:

- Religious institutions
- Public institutions
    - Social/health institutions
    - Hospitals
- Inhabitants
- Hotels and restaurants

(PDAM Samarinda, 2017; PDAM Tenggarong, 2017)

Figure E.9: Water tower in Kenangan, Indonesia *(Own work)*

PDAM supplied bottled water companies

The PDAM has its own bottled water subsidiary. Although the PDAM is not allowed to create a profit, according to the client it's subsidiaries are. The PDAM sells its treated water to this bottled water company who treats the water once again using reverse osmosis. Depending on the type of pollution and the ability of reverse osmosis to filter it, acknowledgement of pollution will be problematic for this bottled water subsidiary. The relation between the PDAMs and the subsidiary bottled water companies can cause a conflict of interest, which can attribute to the problems (i.e. denial) regarding the acknowledgement of pollution through additional pressure exercised on the PDAM.

Bottled water companies

Due to the poor availability and reliability of the water distribution infrastructure there is a large market for bottled water Indonesia (The Water Dialogues, 2009; ASB, 2016). International, national and local companies compete on the bottled water market and resistance might be expected from these companies. Additionally, because of the high accessibility to bottled water (it is sold on every street corner) they are direct competition and prices should reflect this competition in case of large price increases.

PDAM supplied water trucks

The poor availability and reliability has also led to a large network of PDAM supplied water trucks. These trucks are not owned by the PDAM but they do have their own, relatively low, water price and are considered an extension of the network of the PDAM. Considering the small loss in possible revenue with this lower price and the large investment needed to connect the entire city to the clean water network the water trucks form a cheap alternative.

E.2 Stakeholder Characteristics

In the following Appendix the stakeholders will be assessed based on their 'interest in', 'problem perception of', 'goal for' and 'resources for' The Samarinda Water Project. For reasons of clarity a definition of the characteristics described above will be given. Interest is reflected in the reasons to undertake or participate in a project, this can be used to understand and predict decision making processes (Hans de Bruijn, 2010). Individual problems perceptions are important in understanding the difference in behaviour of actors regarding the 'true' problem. It describes the way a stakeholder perceives the problem and challenges in a project. Problems might arise when stakeholder problem perceptions differ. It can also be the case that no problem perception can be defined or they differ because stakeholders are not affected by the project (Hans de Bruijn, 2010). A stakeholders goal can be seen as a long-term ambition. Although similarities between interests and goals exist, interest are short-term steps taken to reach a goal. All activities, actions and decision of a stakeholder eventually lead to accomplishment of their goal. Defining a goal creates insight in the form and probability of these activities, actions and decisions. Resources determine the amount of influence and actor has on a project in order to realize their interests and accomplish their goals. Resources vary from funds and authority to reputation and network of relations (de Bruijn, 2008).

	Interest	**Problem perception**	**Goal**	**Resources**
PT Arsari Tirta Pradana(project owner)	Construction, operation and maintenance in a sustainable manner. Development through a (close to) zero-waste process.	Polluted water source and a (future) lack of clean water of Tenggarong and Samarinda.	Creating a clean water supply for the cities of Samarinda and Tenggarong.Creating revenue for preservation and conservation projects.	Financial resources, knowledge, legal, lobby, network of relations, status, employment
ITCI(landowner)	Keeping water reservoir size as efficient as possible. As little service infrastructure as possible. Low level of disturbance to environment.	Possible construction of artificial water reservoir on ITCI grounds. Destroying part of the forest.	Exploit the concession in a sustainable way while creating profit.	Financial, resources, knowledge, legal, network of relations ownership
Government (Government & Provincial)	Revenue from taxes, assessing and creating new possibilities to satisfy the growing demand in clean water. Monitoring and controlling compliance with regulations.	Currently the government does not officially recognize a problem with the clean water supply.	A well regulated clean water supply for the inhabitants and industries of the Kutai Kartanegara district and City of Samarinda. Ensuring all activities are in compliance with the laws and regulations.	Authority, law, regulations, permits

	Interest	Problem perception	Goal	Resources
Contractors (constructing, operating, maintaining)	Obtaining the contract/participate in project.	An artificial storage lake has to be constructed, operated and maintained. There is no direct reason for the contractor to perceive the clean water problem as their own problem.	Making profit and successfully completing projects.	Knowledge, legal, network of relations
Secondary Product buyers	Biochar usage for agriculture	N.A. there are no farmers close enough to the project area of the artificial storage lakes.	Enhancing their agricultural activities	Future customers of secondary product (through the acquisition of Biochar.)
Inhabitants of the surrounding area	Possible water supply, work opportunities	Project can cause disturbance in the local area (sound, light, traffic	Preserve and enhance their livelihood and living environment	Protesting/Appeal through the house of representatives-Labour,Sabotage
Future employees	Obtaining an income	N.A.The future employees not necessarily have a deviating problem perception. Their interest and goal are obtaining an income through the project.	Preserve and enhance their livelihood	Labour, protesting, knowledge

Table E.1: Stakeholder characteristics of the artificial storage lakes phase

Stakeholders	Interest	Problem perception	Goal	Resources
PT Arsari Tirta Pradana (project owner)	Construct a pipeline efficiently (Technical, sustainable) while maintaining commercial feasibility(supply water to the surrounding villages, social)	Polluted water source and a (future) lack of clean water of Tenggarong and Samarinda.	Creating a clean water supply for the cities of Samarinda and Tenggarong.Creating revenue for preservation and conservation projects.	Financial resources, knowledge, legal, lobby, Network of relations, Status employment
Governmental (national, district & provincial) (regulating, landowners)	Revenue from taxes.Assessing and creating new possibilities to satisfy the growing demand in clean water.Monitoring and controlling compliance with regulations.	Currently the government does not officially recognize a problem with the clean water supply	A well regulated clean water supply for the inhabitants and industries of the district Kutai Kartanegara district and city of Samarinda.Ensuring all activities are in compliance with the laws and regulations.	Authority, law, regulations, permits, land ownership, financially (subsidies)
PDAM Kutai Kartanegara District/Samarinda	Supplying water at a profitable but reasonable price.Securing sufficient clean water sources.Creating and maintaining enough infrastructure capacity for the water supply.	The PDAM does not recognize a pollution problem of the current water supply.	Ensuring a profitable and sufficient clean water supply.	Authority, sole right to water supply in their district, official right of proof to water quality,
Private landowners	Maximizing profit from selling land	Interference from pipeline with other activities	Creating revenue with activities on their land	Land ownership
Villages along the pipeline	Obtain a clean water supply	Potential pipeline through their village	Preserving and enhancing their livelihood	Protest/Appeal, sabotage of pipeline (construction), illegal water tapping
Contractors	Obtaining the contract/participate in project.	A pipeline has to be constructed, operated and maintained.There is no direct reason for the contractor to perceive the clean water problem as their own problem.	Making profit and successfully completing projects.	Knowledge, legal, network of relations

Stakeholders	Interest	Problem perception	Goal	Resources
River users (industry and transportation companies)	No hindrance due to construction of the pipeline	Pipeline crossing the Mahakam river	Using the river a as method of transportation, source of income and source of water.	Lobby,Financial, Network of Relations
River users (inhabitants and fisheries)	No hindrance due to construction of the pipeline	Pipeline crossing the Mahakam river	Using the river a as method of transportation, source of income and source of water.	Protest/Appeal, Sabotage of pipeline (construction)

Table E.2: Stakeholder characteristics of the transportation phase

Stakeholder	Interest	Problem perception	Goal	Resources
PT Arsari Tirta Pradana (project owner)	Construct a pipeline efficiently (Technical, sustainable) while maintaining commercial feasibility(supply water to the surrounding villages, social)	Polluted water source and a (future) lack of clean water of Tenggarong and Samarinda.	Creating a clean water supply for the cities of Samarinda and Tenggarong.Creating revenue for preservation and conservation projects.	Financial resources, knowledge, legal, lobby, network of relations, status
Governmental (National & Provincial) (Regulating)	Revenue from taxes.Assessing and creating new possibilities to satisfy the growing demand in clean water.Monitoring and controlling compliance with regulations.	Currently the government does not officially recognize a problem with the clean water supply	A well regulated clean water supply for the inhabitants and industries of the district Kutai Kartanegara district and City of Samarinda.Ensuring all activities are in compliance with the laws and regulations.	Authority (over the PDAM, District of Kutai Kartanegara and Municipality of Samarinda), law, regulations, permits, financial (subsidies)
Municipality of Samarinda	Supplying water at a profitable but reasonable price.Securing sufficient clean water sources.Creating and maintaining enough infrastructure capacity for the water supply.	Currently the government does not officially recognize a problem with the clean water supply	A well regulated clean water supply for the inhabitants and industries of the city of Samarinda.Ensuring all activities are in compliance with the laws and regulations.	Regulating, authority, law, permits
District of Kutai Kartanegara (Tenggarong)	Supplying water at a profitable but reasonable price.Securing sufficient clean water sources.Creating and maintaining enough infrastructure capacity for the water supply.	Currently the government does not recognize a problem with the clean water supply	A well regulated clean water supply for the inhabitants and industries of the district Kutai Kartanegara district.Ensuring all activities are in compliance with the laws and regulations.	Regulating, authority, law, permits

Stakeholder	Interest	Problem perception	Goal	Resources
PDAM of Samarinda	Supplying water at a profitable but reasonable price.Securing sufficient clean water sources.Creating and maintaining enough infrastructure capacity for the water supply.	Currently the government does not recognize a problem with the clean water supply	Ensuring a profitable and sufficient clean water supply.	Authority, sole right to water supply in their district, official right of proof to water quality,
PDAM of Kutai Kartanegara	Supplying water at a profitable but reasonable price.Securing sufficient clean water sources.Creating and maintaining enough infrastructure capacity for the water supply.	Currently the government does not recognize a problem with the clean water supply	Ensuring a profitable and sufficient clean water supply.	Authority, sole right to water supply in their district, official right of proof to water quality,
Industrial customers of PDAM (Samarinda & Tenggarong)	Clean water supply	Polluted water source and a (future) lack of clean water	Preserving and enhancing their commercial value.	Financial (customers), protest/appeal, network of relations
Non-industrial customers of PDAM (Samarinda & Tenggarong)	Clean water supply	Polluted water source and a (future) lack of clean water	Preserving and enhancing their livelihood	Financial (customer), protest/appeal
PDAM supplied bottled water companies (Samarinda & Tenggarong)	Clean water supply	Polluted water source and a (future) lack of clean water.For the PDAM bottled water company it is not of importance where the water comes from.	Create revenuefrom bottled water sales	Financial (customer), contract, network of relations
Bottled water companies	Creating and maintaining a large group of customers.	Increase in clean water supply quality might decrease their market share.	Create revenue from bottled water sales	Financial (competition), network of relations,
PDAM supplied water trucks (Samarinda & Tenggarong)	Clean water supply	Polluted water source and a (future) lack of clean water.	Create revenue from secondary water distribution.	Financial (customer), econdary distribution (network).

Table E.3: Stakeholder characteristics of the offtake phase

E.3 Critical Actor Table

Early recognition of critical stakeholder is necessary for correct engagement of stakeholders throughout the project, from initiation to decommissioning. Although stakeholder analysis in such an early phase does not guarantee successful stakeholder management throughout the entire project without repeating the analysis, it creates insight in the different possibilities of approach for the project. Additionally, it might be necessary to include various stakeholders in the project at an early phase to ensure project support and participation. To assess which stakeholders are of importance for the project, and thus must be taken into account early on in the project, a critical actor table is presented below. The criticality of stakeholder is assessed on 4 characteristics, power, interest, and the replaceability of and dependency on their resources. Stakeholder are categorized on these four characteristics using a '*high*' or '*low*' categorization. Based on their stake and goal in the project a stakeholder can have a high or low interest in the project. Similarly they can have a high or low amount of power based on their resources and influence on the project outcome. The replaceability of a stakeholder is derived from the necessity of their participation for project completion and the presence of other stakeholders capable of fulfilling their role. The dependency shows the importance of a stakeholder based on how much the project depends on their resources and participation in the project (de Bruijn, 2008).

Table E.4: Criticality analysis of stakeholders in the artificial storage lake phase

Stakeholder	Power	Interest	Resrouces	Replaceability	Dependency	Critical stakeholder
PT Arsari Tirta Pradana (project owner)	High	Low	Financial Knowledge Legal Lobby Network of relations Status Employment	Low Low Low Low Low HighLow Low	High High High High High High Low/High	**Yes**
ITCI (land owner)	High	High	Financial Knowledge Network of relations Ownership	Low Low Low Low	High High High High	**Yes**
Government (National & Provincial)	High	Low	Authority Law Regulations Permits	Low Low Low Low	High High High High	**Yes**
Contractor	Low	High	Knowledge Legal Network of relations	High High Low	High Low Low	No
Secondary product buyers	Low	High	Future customers of secondary product	High	Low	No
Inhabitants of surrounding area	Low	High/Low	Protesting/appeal Labour Sabotage	High High High	Low Low Low	No
Future employees	Low	High/Low	Labour Protesting	High High	Low Low	No

Table E.5: Criticality analysis of stakeholders in the transportation phase

Stakeholder	Power	Interest	Resources	Replaceability	Dependency	Critical stakeholders
PT Arsari Tirta Pradana (project owner)	High	Low	Financial Knowledge Legal Lobby Network of relations Status Employment	Low Low Low Low Low Low Low	High High HIgh HIgh HIgh HIgh Low/High	**Yes**
Government (National, District & Provincial)	High	Low	Authority Law Regulations Permits Landownership Financial (subsidies)	Low Low Low Low Low Low	High High High High High High	**Yes**
PDAM Kutai Kartanegara District	High	High	Authority Sole right to supply of clean water Right to proof of water quality	Low Low High	High High High	**Yes**
Waterway Regulating Governmental Institution	High	Low	Authority Law Regulations Permit	Low Low Low Low	High HIgh High High	**Yes**
Private Landowners	High	Low	Ownership	Low	High	**Yes**
Villages along the pipeline	Low	Low	Protest/appeal Sabotage of pipeline (construction) Illegal water tapping	High High High	Low Low Low	No
Contractors	Low	HIgh	Knowledge Legal Network of relations	High High Low	Low Low Low	No
River users (industry & transport companies	Low	Low	Lobby Financial Network of relations	High Low HIgh	Low Low Low	No
River users (Inhabitants & fishers	Low	Low	Protest/appeal Sabotage of pipeline	HIgh High	Low Low	No

Table E.6: Criticality analysis of stakeholders in the offtake phase

Stakeholder	Power	Interest	Resources	Replaceability	Dependency	Critical stakeholder
PT Arsari Tirta Pradana (project owner)	High	Low	Financial Knowledge Legal Lobby Network of relations Status	Low Low Low Low Low Low	High High HIgh HIgh HIgh HIgh	**Yes**
Government (National & Provincial)	High	Low	Authority Law Regulations Permits Financial (subsidies)	Low Low Low Low Low	High High High High High	**Yes**
Municipality of Samarinda	High	High	Authority Regulation Law Permits	Low Low Low Low	High High High HIgh	**Yes**
District of Kutai Kartanegara	High	High	Authority Regulation Law Permits	Low Low Low Low	High High High High	**Yes**
PDAM of Samarinda	High	High	Authority Sole right to water supply Right to proof of water quality	Low Low High	High HIgh High	**Yes**
PDAM of Kutai Kartanegara	High	High	Authority Sole right to water supply Right to proof of water quality	Low Low High	High HIgh High	**Yes**
Industrial Customers of PDAM	High/Low	High	Financial (customers) Protest/Appeal Network of Relations	Low Low Low	Low Low Low	No
Non-industrial customers of PDAM	Low	High	Financial(customers) Protest/Appeal	Low Low	Low Low	No
PDAM supplies supplies bottled water companies	Low	Low	Financial (customer) Network of relations Contracts	High Low Low	Low Low High	No
Bottled water companies	Low	Low	Financial Network of relations	High High	Low Low	No
Water trucks	Low	High	Financial(customer) Secondary distribution(network)	High High	Low Low	No

E.4 Stakeholder Network

In the following appendix the relations between the critical stakeholders are analysed based on three different factors. The first factor being the variety of stakeholders, followed by the closedness of stakeholders and lastly the interdependence between the actors (de Bruijn, 2008). Although independent stakeholders often have a hierarchical structure, this is especially true for Indonesian organisations (Simarmata, 2012), the networks these stakeholders perform differ from many of the hierarchical characteristics, the most important ones described above: variety, closedness and interdependence. This Appendix focuses on the influence of these characteristics on the decision making in networks. Hierarchical decision making is often well-structured and stable. Network decision making, however, often is a chaotic and dynamic process (de Bruijn, 2008). Because the stakeholder analysis as a whole is meant to provide input for the stakeholder engagement plan, the network is analysed from the client point of view and focused on the critical stakeholders. Within networks, sub-network can exist. These sub-networks can have different characteristics and have a, positive or negative, influence on the entire decision making process.

Variety of stakeholders

Networks are often characterized by a high level of variety instead of hierarchical uniformity of stakeholders. One problem with a high level of variety is that it creates a need for more single-stakeholder focused strategies, which increases complexity, and have a smaller span of control (de Bruijn, 2008). Many factors add to the variety of a network such as organisational size, problem perception, influence and resources.

Within the network of The Samarinda Water Project, two sub-networks exist, one of which has strong hierarchical characteristics. The first sub-network is composed of the client (PT Arsari Tirta Pradana) and the landowner (ITCI). This sub-network exists is because of the common goal, common problem perception, possibility of collaboration based on their interest and the nature of their involvement (de Bruijn, 2008). The common nature of involvement for the client and the landowner is based on the fact that they are collaborating companies both owned by the Arsari group.

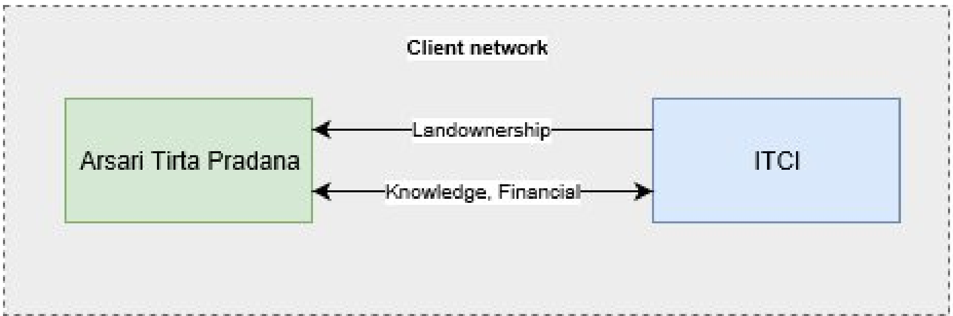


Figure E.10: Client network *(own work)*

The second sub-network consists of the national government, the provincial government, the municipality of Samarinda, the district of Kukar and the PDAM of both Samarinda and Kukar. The reason this sub-network exists is because of the way the government is structured and the strong hierarchical connection between the individual institutions.

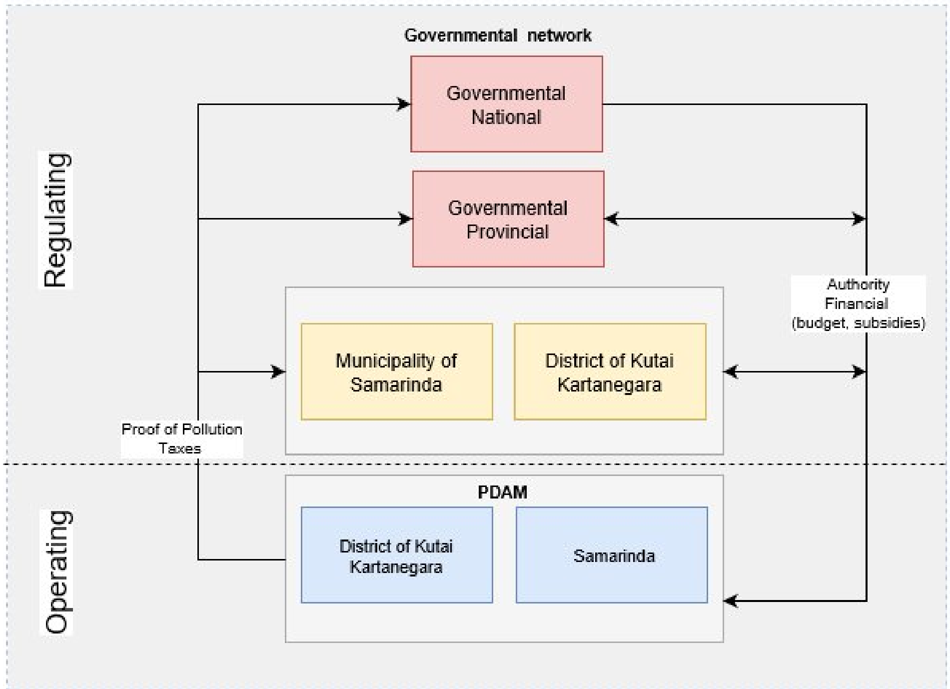


Figure E.11: Government Network *(own work)*

The following part of this section will focus on the implications and opportunities that arise due to these two sub-networks.

Within the entire network these two sub-networks decrease the overall complexity of the decision making process is several ways. Firstly, they improve the reach of an intervention (de Bruijn, 2008). For the first sub-network, interventions from the client to the landowner can be dealt with mainly through maintaining a good relationship and through contract management. For interventions between the client and for instance the contractor, a non-critical stakeholder likely to become become critical in later phases the emphasis should lie on contract management on the basis of a good relationship.
For the second sub-network, interventions from the client to this network are complicated because of of differences in stakeholder characteristics. However, the strong hierarchical structure of the governmental institutions entails that if an intervention is received and agreed upon at a the top of the hierarchy, the rest of the network is likely to follow, even if they don't necessarily agree with the intervention.
The second way complexity is decreased by the sub-networks in The Samarinda Water Project is because the need for tailor-made stakeholder approaches decreases (de Bruijn, 2008). Because of the strong hierarchical structure, there is lower need for tailor made approaches. However, when engaging individual stakeholders in the second sub-network, it is important to understand how the intervention might be received by the other stakeholders. For instance when engaging the Municipality of Samarinda and the District of Kukar it should be taken into account that they have the same power over the stakeholders below, the PDAMs, but seen from the national and provincial government, different levels of governing powers exist within the Municipality and the District.
A third way the sub-networks decrease complexity is that there is less room for reinterpretation of individual stakeholder interventions. Because the governmental stakeholders of the second network have a clear scope concerning their goals and tasks, predicting how an intervention is received, interpreted and executed, becomes easier.

For The Samarinda Water Project there is a positive characteristic of networks that becomes a possible threat because of the second sub-network. When there is a high level of variety in a network there is a low possibility for cooperation. However, in this project there is a low variety due to the second sub-network which exist solely because of their hierarchical collaborative and authoritarian structure. Because of this, collaboration is likely and the second sub-network should be managed closely based on their interdependencies. This will be elaborated on further in Interdependencies between stakeholders.

Closedness of stakeholders

Depending on their goal, interest and problem perception, stakeholders can be characterized as closed. This entails that they are not sensitive, or open, to external interventions. A closed stakeholders attitude towards an intervention can manifest in several different ways but comes down to two types of attitude, ignoring and resisting (de Bruijn, 2008). Within The Samarinda Project there are several stakeholder of which is known they currently have contradicting interests, goals and problem perceptions because of ignorance and possibly corruption. This indicates a possible closed attitude towards the project (de Bruijn, 2008). Several of these, the governmental institutions such as the Municipality of Samarinda, the District of Kukar and the PDAMs of Tenggarong and Samarinda, are critical to project success. Until they change their attitude towards the project several scenarios exist, all having a negative impact on project success. The first, most likely, scenario is that interventions supported by the a number of governmental institutions are resisted by other governmental institutions, despite the strong hierarchical structure. This resistance can manifest either through false compliance with interventions, by transforming interventions to suit their own agenda or by evading real compliance at every possible opportunity, all eventually leading to high probability of failure of the intervention (de Bruijn, 2008). False compliance is most likely to happen with interventions accepted by higher level stakeholders who force it on lower level stakeholders. A second scenario is that all government institutions ignore the problem and any intervention, including the the creation of awareness, will fail (de Bruijn, 2008). This is scenario is unlikely because the Governor of Kukar and the Mayor of Samarinda have asked Arsari to look into the possibility of a new clean water supply. This entails that within the municipality of Samarinda awareness about the poor quality of water exists. Additionally, general awareness about the poor quality of the water sources is growing. A third scenario is that is that interventions supported by the national, municipal and district government are ignored by the PDAMs and fail (de Bruijn, 2008). Because of the strong hierarchical structure it is unlikely that interventions that are accepted by higher governmental institutions will be ignored by lower governmental institutions. This can however lead to the the first scenario. A fourth scenario, in which the PDAMs acknowledge the problem and the higher level governmental institutions do not, is highly unlikely because the PDAM are the official governmental sources on water quality.

Although a high level of closedness makes the engagement of stakeholders more complex and often leads to a high probability of failure there is a positive impact as well. Resistance not only comes forth from ignorance and unwillingness. Resistance can originate from genuine doubts about decision making which in turn can lead to a better decision making process and successful project completion (de Bruijn, 2008). Obtaining support of closed stakeholders because they agree with the intervention after they have raised and exercised their doubts creates stronger partnerships than forced compliance with interventions (de Bruijn, 2008). This approach, focused on collaboration, can be important in the engagement of closed governmental stakeholders critical to project success.

Interdependencies between critical stakeholders

The last characteristic used to analyse the stakeholder network is interdependencies. These interdependencies exist in different forms such as authority over another stakeholder, landownership and financial resources (de Bruijn, 2008). The interdependencies of the stakeholder involved in The Samarinda Water Project have been mapped in formal maps. These formal maps give an overview of the resources a stakeholder has to exercise influence over another stakeholder. The formal maps for the three project phases containing all stakeholders can be found in figure E.13, E.14 and E.15. The following appendix will focus on the critical stakeholders of all three phases.

In figure E.12 the formal map with the critical stakeholders, and the sub-networks they form, for the entire Samarinda Water Project is shown. Important to note is that the PDAMs of Kukar and Samarinda do not directly have the governmental resources “Authority, Regulations, Laws and Permits” towards Arsari Tirta Pradana and ITCI. They are however part of the the sub-network containing the the national, district and municipal government and through strong collaboration they can have indirect access to these resources. Additionally, the financial resource of subsidies is directly linked to Arsari, the PDAMs and indirectly linked to the PDAMs through the District and Municipal government. It it currently unsure how possible subsidies will be provided and to whom. It is however known that subsidies are available to cover differences between the Arsari water selling price and the PDAM selling price to prevent sudden price surges for the customers of the PDAM.

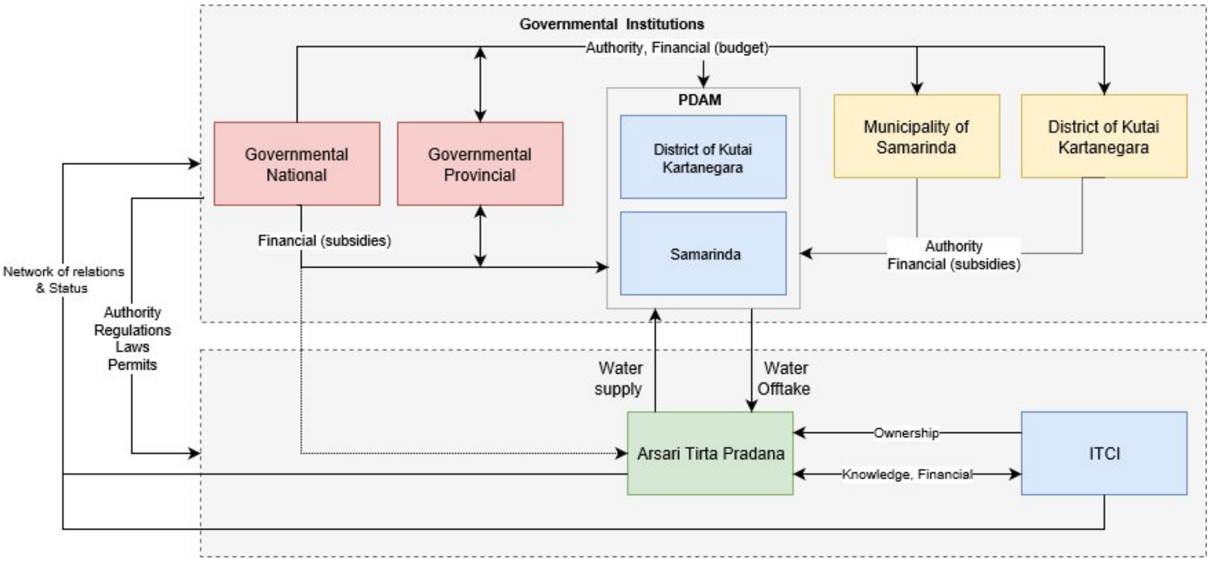


Figure E.12: Critical stakeholders for the entire project *(own work)*

Several opportunities and threats can be ascribed to the interdependencies described above. Firstly, although 7 critical stakeholders have been identified there is little incentive for moderate behaviour until the governmental sub-network agrees on the necessity of the project due to pollution of the water sources as described in 'closedness of stakeholders'. The water supply and water offtake dependencies are not valid until the entire governmental sub network, or higher governmental institutions, acknowledges a need for clean water. When this is acknowledged, an incentive for moderate behaviour arises which can lead to cooperation.

Secondly, a valuable resource for the client towards the sub-network of governmental institutions is “water supply”. The weight of this resource for negotiation and interventions depends on the availability of alternatives. During our research no direct alternatives were identified. However, which sources are polluted and the extent of this pollution was outside the scope of this research. If other useful sources become available to the PDAMs of Samarinda and Tenggarong or current sources are found not to be polluted, the weight of “water supply” as a resource for negotiation decreases.

Lastly, two levels of “authority over” can be identified. The first is the authority of the national government over the provincial government, the municipality of Samarinda and the district of Kukar and the PDAMs. The second level is the authority of the Municipality and the District over The PDAMs. This can be both a risk and an opportunity for the project. Authority can lead to poor substantive decision making when decisions are forced, but not acknowledged by the subordinate stakeholder party or when better options are available but not acknowledged by the superior stakeholder (de Bruijn, 2008). However, in the same way but with exactly the opposite as result, authority may be used to force unwilling parties to take the right decision. However, a party that was forced to take a certain decision but does not receive anything in return or that has his core values affected is not likely to become a strong cooperator or collaborative partner in the following phases the project (Hans de Bruijn, 2010). Therefore this is a approach that should be exercised with caution and preferably not in or before a phase in which the subordinate stakeholder will be critical.

Interdependencies between all stakeholders

The following formal maps (figure E.13, E.14, E.15) illustrate the interdependencies between all stakeholders that have been identified for the Samarinda Water Project per phase.

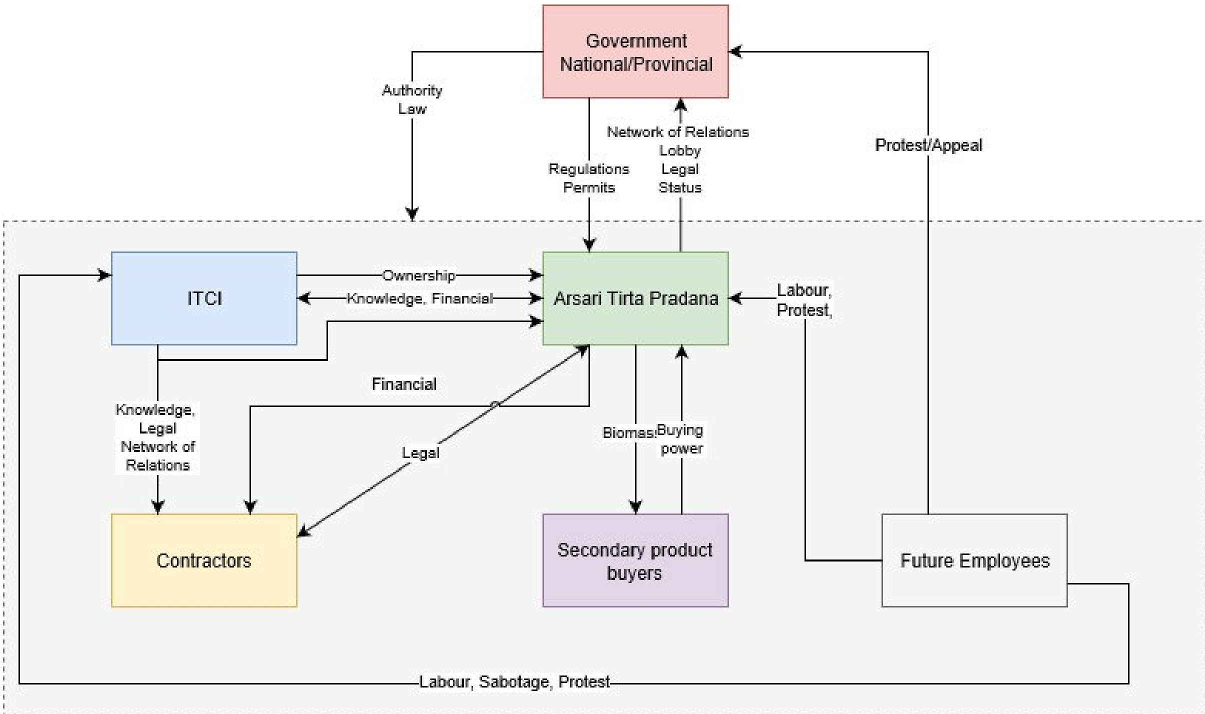


Figure E.13: Formal map for the artificial storage lake phase *(own work)*

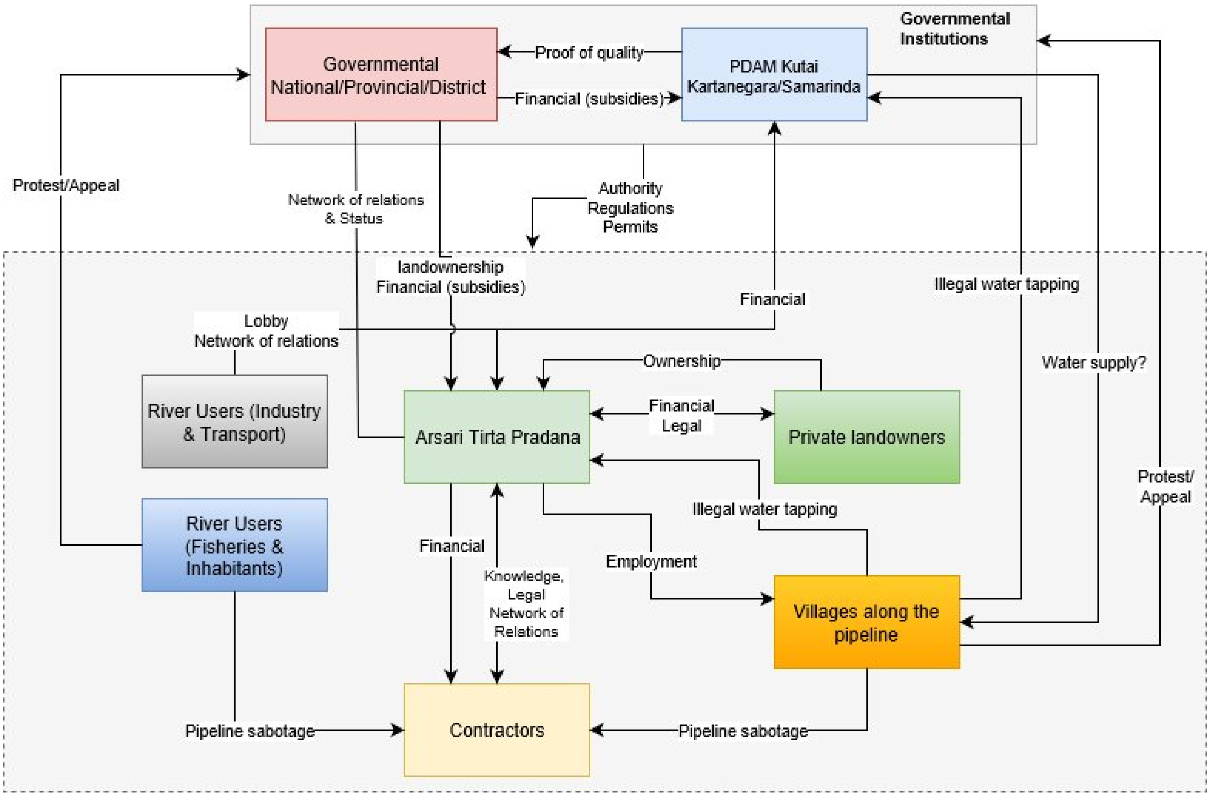


Figure E.14: Formal map for the Transportation phase *(own work)*

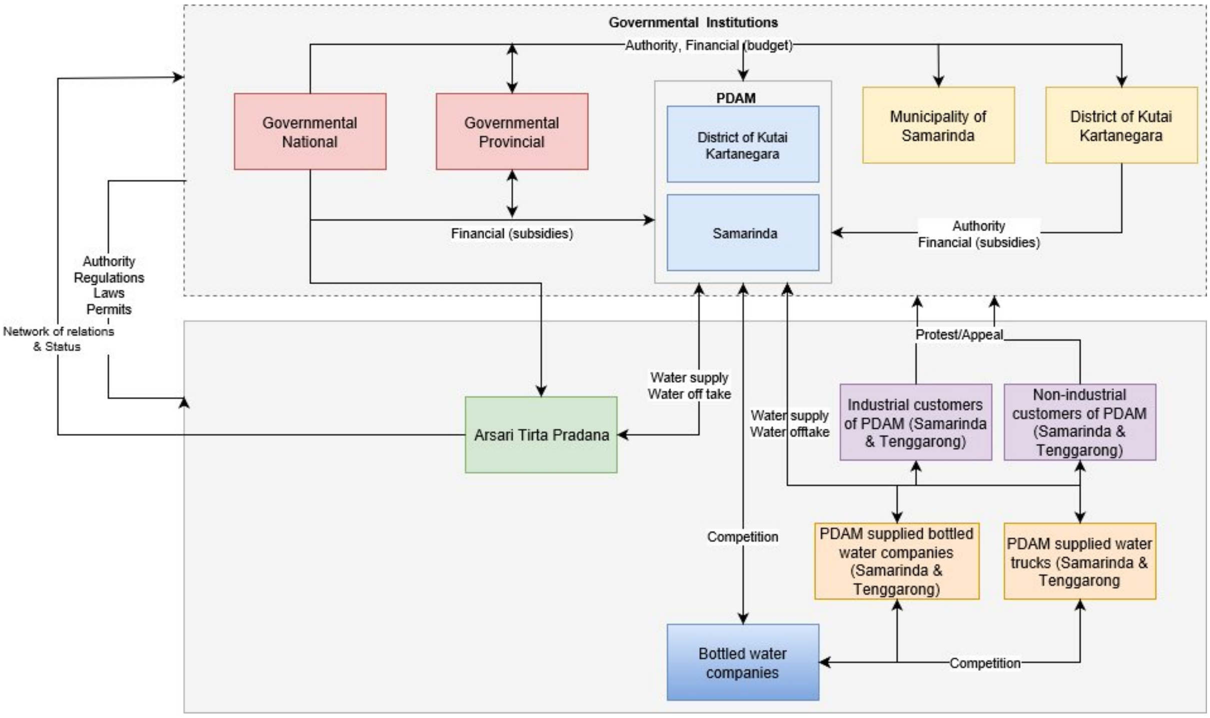


Figure E.15: Formal map for the Offtake phase *(own work)*

Appendix F

Finance

F.1 Offtake Estimations

The current offtake in Tenggarong is 0,4 m^3/s and in Samarinda it is 2,6 m^3/s (PDAM Tenggarong, 2017; PDAM Samarinda, 2017). That is in total an offtake of 3,0 m^3/s in 2017. Due to growth of inhabitants and industry in those cities, the demand will increase. No data can be found on what the exact growth will be in water demand till 2035 and also no data is present for the growth of the industry in both cities. Therefore, an estimation for the future water demand will be made on the basis of growth of the urban population in East-Kalimantan till 2035, seen in table F.1. The city of Samarinda is the capital with the largest population in East-Kalimantan and Tenggarong is the capital of Kutai Kartanegara, a district in East-Kalimantan. Therefore, it can be assumed that the population growth of both cities follows the urban growth of East-Kalimantan. As seen in table F.1, the urban population share of East-Kalimantan is estimated every 5 years and increases till 2035. From the population increase in urban areas the growth rate per year can be can be calculated:

$$\textit{Yearly growth rate of urban areas (\%)} = \left(\left(\frac{Population_{t+dt}}{Population_t}\right)^{\frac{1}{dt}} - 1\right) * 100\% \tag{F.1}$$

Table F.1: Estimation of East-Kalimantan Urban Population Growth (Badan Pusat Statistik Indonesia, 2013)

Year	Population Size (x1000)	Population Growth (%)	Urban Population Volume (%)	Urban Population Size (x1000)	Growth of Urban Areas (%)
2015	4069		66%	2685	
2016	4168	2,4%			3,2%
2017	4267	2,3%			3,2%
2018	4365	2,2%			3,2%
2019	4464	2,2%			3,2%
2020	4659	2,2%	68,9%	3143	3,2%
2021	4756	2,1%			2,9%
2022	4852	2,0%			2,9%
2023	4947	2,0%			2,9%
2024	5041	1,9%			2,9%
2025	5134	1,9%	71,8%	3619	2,9%
2026	5226	1,8%			2,6%
2027	5317	1,8%			2,6%
2028	5408	1,7%			2,6%
2029	5497	1,7%			2,6%
2030	5585	1,6%	74,8%	4112	2,6%
2031	5673	1,6%			2,1%
2032	5759	1,6%			2,1%
2033	5759	1,5%			2,1%
2034	5845	1,5%			2,1%
2035	5929	1,4%	77,7%	4565	2,1%

Population growth

The current population size of Tenggarong and Samarinda is 197.295 and 854.792, respectively (PDAM Samarinda, 2017; PDAM Tenggarong, 2017). The population growth till 2035 of both cities is not known. It is derived from the East-Kalimantan population growth and the urban population size, as seen in the table F.1. In the following table F.2, the population of both cities will be forecasted on the basis of the urban population growth in East-Kalimantan.

Table F.2: Population growth of Tenggarong and Samarinda till 2035

Year	Urban Growth Rate (%)	Population Tenggarong	Population Samarinda
2015		185744	812597
2016	3,2%	191181	828303
2017	3,2%	197295	854792
2018	3,2%	203605	882129
2019	3,2%	210116	910340
2020	2,9%	216835	939453
2021	2,9%	223041	966337
2022	2,9%	229424	993991
2023	2,9%	235989	1022437
2024	2,9%	242743	1051696
2025	2,6%	249689	1081793
2026	2,6%	256143	1109754
2027	2,6%	262763	1138437
2028	2,6%	269555	1167862
2029	2,6%	276522	1198047
2030	2,1%	283669	1229012
2031	2,1%	289670	1255013
2032	2,1%	295798	1281563
2033	2,1%	302056	1308675
2034	2,1%	308446	1336361
2035	2,1%	314971	1364633

F.1.1 Installed water capacity of the PDAMs

The future demand of clean water is estimated using the growth of the population of both cities. These estimations are, thus, merely based on the increase in population, whilst ignoring the growth of industry located in both cities. The reason that the industries are left aside, is due to the lack of data. Thus, it is assumed that the future demand is based solely on the growth of inhabitants. The water demand will be based on the installed capacity that the PDAM's currently possesses. To elaborate on that, the installed capacity is the full theoretical capacity the PDAM's carry. Although, the production capacity and distribution capacity are more exacter due to efficiency losses, there was no useful data present on both. There is no uniform percentage loss in efficiency to the production capacity and distribution capacity, therefore it cannot be calculated what these capacities contains. Consequently, the installed capacity will be incorporated as the water demand in the analysis. The demand of Tenggarong is 405 L/s and 2587,5 L/s for Samarinda, combined it is 2992,5 L/s (PDAM Samarinda, 2017; PDAM Tenggarong, 2017). In table F.3, the installed capacity is estimated till 2035, resulting in a combined offtake of 4930,2 L/s in 2035.

Table F.3: Installed Capacity for Tenggarong and Samarinda till 2035

Year	Urban Growth Rate (%)	Installed Capacity Tenggarong (m3)	Installed Capacity Samarinda (m3)	Combined Installed Capacity (m3)
2015				
2016	3,2%	405	2587,5	2992,5
2017	3,2%	417,95	2670,25	3088,2
2018	3,2%	431,32	2755,64	3187,0
2019	3,2%	445,11	2843,77	3288,9
2020	2,9%	459,35	2934,72	3394,1
2021	2,9%	472,49	3018,70	3491,2
2022	2,9%	486,01	3105,09	3591,1
2023	2,9%	499,92	3193,95	3693,9
2024	2,9%	514,23	3285,35	3799,6
2025	2,6%	528,94	3379,37	3908,3
2026	2,6%	542,62	3466,71	4009,3
2027	2,6%	556,64	3556,31	4113,0
2028	2,6%	571,03	3648,23	4219,3
2029	2,6%	585,79	3742,53	4328,3
2030	2,1%	600,93	3839,26	4440,2
2031	2,1%	613,64	3920,48	4534,1
2032	2,1%	626,62	4003,42	4630,0
2033	2,1%	639,88	4088,11	4728,0
2034	2,1%	653,42	4174,60	4828,0
2035	2,1%	667,24	4262,92	4930,2

F.2 Cost of Investment

The initial costs of investment are based on the direct costs i.e. cost of equipment, activities and materials; added costs i.e. allowances, indirect costs and contingencies. Eventually, with the use of P50 method the final investment costs are estimated. It is not known when what investment will be made, therefore the assumption is made that the full amount of the investment cost will be appropriated at the start of the project. Also, this way the NPV-analysis, which will be elaborated on in section F.3, is at a worse-case scenario, for the reason that none of the investment costs are discounted.

F.2.1 Direct costs specifications

In this section, the costs will be specified and elaborated qualitatively in detail. The direct costs are divided in three sections: storage lake, transportation phase and the offtake phase. In any scenario, the storage lake will be an investment of the client. Most preferably from the client perspective, the PDAM's should bear the costs of the two other phases. To give a full overview of the initial investment costs, all three phases are examined thoroughly.

Storage lake

Lake A

The total cost of preparing the basin of phase 2A for the artificial storage lake is based on the cost of the removal of the top soil, the land clearance and levelling & site measurement. The first step in this process is to remove the biomass and top soil. Costs for this process are set on zero, because of the revenues generated by the harvested biomass. Then, the land should be levelled in order to allow construction. The cost for the levelling process are linked to the topography of the area, as steep slopes will induce higher costs. Due to the similarities in land cover and topography with phase 1, the same costs per hectare will be appropriated in phase 2.

Dam A

The materials of the dam will consist of rock, sand, clay and concrete injection. The volume of the different materials which the dam consist of is stated in Chapter 5. On the concession of ITCI, a quarry is located where rocks can be obtained, therefore no procurement costs for rock is inflicted. The costs for obtaining the rock is based on blasting and grading. Furthermore, the rocks have to be transported and placed on the dam site. Also, sand and clay can be subtracted from the ITCI concession. Thus, only consisting of transport and placement costs. The distance between the quarry and the dam site is roughly 7 km. The transport costs are based on the cycle time: time of the transport from and to the dam site and the un- and reloading time. The transport costs are derived from Witteveen+Bos distance to the phase 1 dam site, which is 36,5 km. Notably, the abundance in rock, clay and sand is based on the client's convictions. Further research should be carried out.

Tunnel A

The total direct costs of the tunnel consist of preparation work and the construction. In order to prepare the construction of the tunnel, access roads have to be constructed, the construction site has to be levelled, two tunnel portals have to be procured and a prefab structure of the intake is needed. These costs are based on lump sum investments. After the preparation, the tunnel will be constructed in a conventional manner by using a roadheader, as mentioned in Chapter 5. These costs are based on the length of the tunnel: 150m. Furthermore, to transport the water through the tunnel, a steel pipeline will be implemented. The dimensions of the pipeline are stated in Chapter 6 and the material costs are based on the price of steel per kilogram. To ensure no degradation of steel and seepage will occur, an internal epoxy and external asphalt coating is applied. Moreover, due to the standard size and, therefore, easily obtainable, the length of the pipeline is set on 12m. This entails the parts have to be welded. In addition, the costs of pipeline installation, pipe fitting and transfer of soil surplus will be incurred.

Pipelines Section 1: Dam to Hydro power

The hydro power plant will be constructed in proximity of the dam due to the efficiency of the power generating capabilities, whilst remaining the needed head. Therefore, the pipeline construction of section 1 will only be around 100 meters. The material costs of the steel pipes are based on the dimensions of the pipes. In Chapter 6, the dimensions of the pipelines are stated. Furthermore, the costs consist internal and external coating to ensure degradation and seepage are minimized. As mentioned in the tunnel section, the 12m pipes will be welded. As the dimensions of the pipeline differ from phase 1, it is converted to the unit costs per meter for welding, whereas in Witteveen+Bos financial data it is stated in number of joints. In addition, the road access is calculated with the assumption that the right-of-way will be 30 meters wide, times the length of the pipeline. Further investment costs are the costs of pipe installation by crane, transport and instalment of sand to give the pipeline a robust foundation. Two pipes will be constructed to ascertain redundancy.

Water Treatment Facility Plant and Hydro power Plant

In the phase 1 financial report, the costs from the Water Treatment Facility plant (WTP) and the hydro power Plant (HP) are based on the production capacity of the phase 1. As the same WTP and HP will be used for both the projects and the capacity is known of phase 2A, the cost of the WTP and hydro power of phase 2 will be estimated by the use of scaling on analogy project ((Nicholas, 2008), p.301). The costs depicted in phase 1 are all lump sum, for the detail design is not yet executed. The formulas to incur the costs for phase 2 are written below:

$$Cost_{WTPphase2} = Cost_{WTPphase1}\left(\frac{Capacity_{phase2}}{Capacity_{phase1}}\right)^{\frac{2}{3}} \tag{F.2}$$

$$Cost_{HPphase2} = Cost_{HPphase1}\left(\frac{Capacity_{phase2}}{Capacity_{phase1}}\right)^{\frac{2}{3}} \tag{F.3}$$

Water Treatment biochar

There is another option to purify water and that is with the use of biochar. This is a suggestion made by the client, however, there is, as of yet, no proof of biochar yielding the same quantities of water with the same quality as the water treatment plant. The water treatment capabilities of biochar are currently being examined. It can, however, be said that the costs will lower drastically and it is much more sustainable than building a water treatment facility plant. Due to the lack of data on water treatment with biochar, it will not be incorporated in this analysis, although in the future it might be a valuable option.

Transportation

Pipelines section 2: WTP to the Reservoir

The section 2 pipeline will traverse from the water treatment plant to the reservoir that will be located at 6 km from the city of Tenggarong. The length of the pipeline is about 55km in total and will transport the water through terrain with a complex height profile, which is depicted in Chapter 6. When setting the route for the pipeline in Chapter 6, the focus lies on remaining enough head to transport the water by using solely gravitational powers, thus, casting away the use of pumping stations. Also, the shortest route would be optimal, for the price of steel is expensive. The first section of this pipeline, will traverse 22km with a small detour around a hilltop. After that, the pipeline will be constructed in an almost straight line to the water reservoir, whilst remaining enough head and avoiding lands that could not be crossed. Thus, no detour is needed in the second section, resulting in a decrease in material usage, consequently, lowering the costs of the pipeline. Although, in practice, a straight line is not possible, however, this will approximately be the optimal pipeline route. Since, the pipeline will cross land, a road will be constructed with a width of 30 meters at a length of 55 km. This section will be built underground to minimize sabotage by inhabitants in surrounding areas, thus, investing in excavation and backfilling activities. The excavation costs are calculated by stating the depth and the slope of the ground that is necessary for achieving underground construction and backfilling is the volume of the excavation minus the volume of the pipeline. As in the prior pipeline sections, the length of the pipeline parts will be 12 meters and the material usage will be steel, as examined in Chapter 6. Also, the pipelines will be coated internally and externally and the cost unit of welding per meter will be incurred. Further costs are pipe installation by crane, transport and instalment of sand to obtain a robust foundation for the pipeline. For redundancy reasons, two pipes will be constructed.

Water Reservoir

The water reservoir will be constructed at 6 km from the city of Tenggarong and 34,8 km from the city of Samarinda at a height of 57 meters, as mentioned in Chapter 6. This distribution reservoir will function as the hub for clean water supply of both cities, be used as back-up capacity for maintenance on the pipeline section 2 and in case of emergency a two-hour buffer to balance the supply and demand. Some of the costs will be identical as with the reservoir used in phase 1, as it also contains the construction of offices and entrances. Besides that, investments will be made to build the basin of the clean water reservoir. These costs will also be estimated by using the scaling of analogy projects as it is based on the capacity of the similar clean water distribution reservoir in phase 1 (Nicholas, 2008), p.301.

$$Cost_{Reservoirphase2} = Cost_{Reservoirphase1}\left(\frac{Capacity_{phase2}}{Capacity_{phase1}}\right)^{\frac{2}{3}} \tag{F.4}$$

Pumping Station

To supply the clean water to the reservoir the pipeline will traverse complex landscapes with different height profiles. Therefore, pumps may be needed to transport the water to the reservoir to overcome the height differences. However, the route of the pipeline examined in Chapter 6 gives a route where the water can be transported using only gravitational powers. Giving the route, no need for any pumping station. Although, making no use pumping stations will be an optimal solution, the route is based on Google earth, where errors can occur and, therefore, can lead to wrong heights of the landscapes. Further research has to be done to ensure no pumping stations will be needed. The pumping station is based on a lump sum estimate and based on scaling of analogy in comparison with Phase 1 pumping station prices.

Land Procurement

The section 2 pipeline will cross land sections that need to be procured. The owners of the land are either governmental or private and is used for different functions i.e. palm oil plantations and mining agriculture amongst other things. In Chapter 6, the pipeline is routed to surpass landscape that will not be available for future usage. According to the client, it will not be a threshold to ascertain these lands. In addition, at this moment no procurement costs are known.

Offtake

Pipeline section 3a: Reservoir to Tenggarong

The clean water will be transported to the reservoir of the Tenggarong's PDAM, which is located in the city centre, from there on out the distribution network of the PDAM will be used. The pipeline to Tenggarong will be about 6 km and will also have two pipes for redundancy. Due to the fact, the offtake of Tenggarong is relatively small in comparison to the total production capacity and the presence of a large height difference in reservoirs, the dimensions of the pipeline decrease for this section, which leads to the option of using HDPE pipelines. In Chapter 6, a comparison is made between steel and HDPE, where HDPE costs less than the steel option. Therefore, this trajectory will utilize HDPE pipelines. These costs accompanying HDPE pipelines differ from steel. Firstly, the material costs are significantly less expensive. Secondly, there is no need for internal and external coating. Thirdly, fusion i.e. welding HDPE pipeline, is less expensive than welding of steel. Furthermore, costs for excavation, backfilling, transport & instalment of sand and road access are the same as with the usage of steel.

Pipeline section 3b: Reservoir to Samarinda

Section 3a transports clean water from the reservoir to the PDAM's reservoir in the city centre of Samarinda. The distance the pipeline traverses to Samarinda is almost 35 km and has enough height difference to flow by gravitational powers, however, the diameter of the pipeline is larger than in section 1 & 2, for the available head is less. Samarinda is located at the other side of the Mahakam river, thus, the pipeline has to cross the river. This will be done by making use of the bridge in Tenggarong.

Pumping Station to the cities

Due to the height difference with the water reservoir and Tenggarong no pump will be needed to supply the clean water to the city of Tenggarong. However, the trajectory to Samarinda is much longer and will traverse landscape with complext height profiles. As said earlier, an optimal solution is a pipeline without pumps, but further research is needed to give a valid conclusion.

F.2.2 Identification of costs

As mentioned, the initial investment costs of phase 2A are based on the financial data made available by Witteveen+Bos. The majority of the costs of phase 1 could be applied to the phase 2A area as it has no different variables. However, several investments are based upon dissimilar quantities in variables. In the upcoming section the alternated costs will be stated based on the changed variable.

Production capacity

Multiple costs of phase 1 are derived from the production capacity the phase 1 lake offers. Therefore, the amount of these investments has to be alternated according to the production capacity of phase 2A. The investments that are based on the production capacity are the water treatment facility and the hydro plant. These costs are both based on lump sum and will be estimated with the use of scaling on analogy project (Nicholas, 2008), p.301.

$$Cost_{phase2} = Cost_{phase1} \left(\frac{Capacity_{phase2}}{Capacity_{phase1}} \right)^{\frac{2}{3}} \tag{F.5}$$

Furthermore, the water reservoir is also based on the production capacity, however, it is not procured with a lump sum, but is defined more in detail. Thus, one cannot alter the detailed prices of for instance a concrete floor calculated based on cubic square meters. Consequently, the same scaling on analogy formula is used, but for the quantities required for building the water reservoir.

$$Quantities_{phase2} = Quantities_{phase1} \left(\frac{Capacity_{phase2}}{Capacity_{phase1}} \right)^{\frac{2}{3}} \tag{F.6}$$

When changing the production capacity, the dimensions of the pipeline change as well. However, an increase in production capacity does not mean the dimensions of the pipeline will follow linearly. The dimensions of the pipeline are dependent upon also pressure and the amount of head amongst other things. Therefore, in this analysis a link between the production capacity and the dimension of the pipeline construction, thus the costs, is not incorporated automatically and has to be done manually.

Distance from Quarry to the Dam Site

The quarry, where the rock, sand and clay will be harvested, lies at about 36,5 km for the dam site of phase 1. The distance to the dam site of phase 2A, however, is about 7 km. The transport costs are based on the cycle time: time of the transport from and to the dam site and the un- and reloading time. Deriving solely from difference in distance the transport costs of rock, sand and clay will be calculated for the phase 2A dam site, thus, not incorporating the similar un- and reloading time.

Costs of the Pipeline Material

Whereas Witteveen+Bos calculated the material costs of the pipelines per kilometer, in this report, the weight in kilogram is used from the material to calculate the investment costs of the pipeline for the steel pipelines, as the Indonesian costs per kilogram steel is known. Furthermore, with the construction of HDPE pipelines the costs are calculated using the length of the pipeline construction.

Coating Cost of the Pipeline

Due to the differences in production capacity of phase 2, for example the length of the pipelines and the topology of the pipeline route, the dimensions of the pipe line construction needed to be altered from the phase 1 pipelines. Thus, changing the size of the surface of the pipeline. Thus, revising the costs of the coating of the pipeline accordingly.

Costs of Welding

In this analysis, the length of the welding is used for calculating the steel pipeline construction and the number of joints for the HDPE pipeline construction. This differs from the Witteveen+Bos calculations, where they also used joints for the steel pipeline construction. In the financial data of Witteveen+Bos, the length of the circumference of a pipeline with a diameter of 1200mm is 3,85m. Therefore, it can be calculated what the price is per meter. Due to the differences in dimensions, the circumferences of the pipelines used in this analysis are not the same as in phase 1. Hence, using the cost of length of the steel welding is more practical.

Table F.4: Direct costs per cost item and its accompanying added costs

Detailed Scope List	**Investment Costs**						
	Direct Costs	Allowances (5%)	Indirect Costs (15%)	Known Costs	Contingency (20%	Remaining/ Engineering Costs (11%)	Total Costs
1. Storage Lake							
1.1 Lake A	$304.000,00	$15.200,00	$47.880,00	$367.080,00	$73.416,00	$40.378,80	$480.874,80
1.2 Dam A	$15.881.057,13	$794.052,86	$2.501.266,50	$19.176.376,49	$3.835.275,30	$2.109.401,41	$25.121.053,20
1.3 Tunnel	$2.162.100,98	$108.105,05	$340.530,90	$2.610.736,94	$522.147,39	$287.181,06	$3.420.065,39
1.4 Pipelines Section 1	$73.739,26	$3.686,96	$11.613,93	$89.040,15	$17.808,03	$9.794,42	$116.642,60
1.5 Water Treatment Facility	$43.502.379,25	$2.175.118,96	$6.851.624,73	$52.529.122,94	$10.505.824,59	$5.778.203,52	$68.813.151,05
1.6 Hydro Power Plant	$2.614.949,20	$130.747,46	$411.854,50	$3.157.551,15	$631.510,23	$347.330,63	$4.136.392,01
2. Transportation							
2.1 Pipelines Section 2	$67.553.944,71	$3.377.697,24	$10.639.746,29	$81.571.388,24	$16.314.277,65	$8.972.852,71	$106.858.518,60
2.2 Reservoir	$5.611.801,58	$280.590,08	$883.858,75	$6.776.250,41	$1.355.250,08	$745.387,54	$8.876.888,03
3. Offtake							
3.1 Pipelines 3a	$3.767.162,71	$188.358,14	$593.328,13	$4.548.848,97	$909.769,79	$500.373,39	$5.958.992,15
3.2 Pipelines 3b	$43.644.460,34	$2.182.223,02	$6.874.002,50	$52.700.685,86	$10.540.137,17	$5.797.075,44	$69.037.898,48
Total Investment Costs (excl. 10% VAT)							**$292.820.476,31**

F.2.3 Investment costs estimation

The optimistic and pessimistic valuation of the activities in this project vary from each other as they are derived from specific empirical scientific studies. Although, data for similar specific Indonesian projects is lacking, an exact percentage is sought-after. Thus, these values are just an indication in the costs estimates, and are not specific for Indonesian projects.

Hydro dam projects

In a study based on hydro power dam infrastructure projects worldwide, it is found that the median lies around 27%. With the actual costs being on average 80% higher than the initial estimated costs, when looking at the distribution in figure F.1 (Awojobi & Jenkins, 2015). Furthermore, when looking at the bar chart an optimistic value can be derived of 10%. In this analysis, not only hydro power dams were analysed, but also water supply dams. There is no data present in the analysis about the presence of water treatment facility, however, due to lack of data on cost estimations for a water treatment facility, the optimistic and the pessimistic value will also be implemented for the water treatment facility. The same argument is given for the lake and the water reservoir.

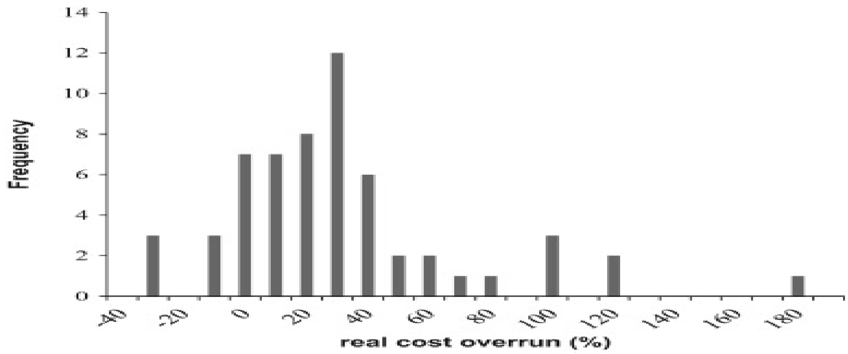


Figure F.1: Actual Costs of hydro dam project in comparison with the estimated costs (Awojobi & Jenkins, 2015)

Pipeline construction projects

In an American research, 412 onshore pipelines are analysed on the estimated costs and their eventual actual costs. In that research five cost components were analysed: material, labour, miscellaneous, right of way (ROW) and the total costs. The bandwidth of the total costs will be implemented in this analysis. The 412 onshore pipeline projects have an average of 7% cost overrun. When, looking at the bar chart a distribution is shown from about -40% to 60% difference from the estimated cost with the actual cost. Therefore, in this analysis an optimistic value of 40% and a pessimistic value of 60% will be taken (Rui et al., 2012).

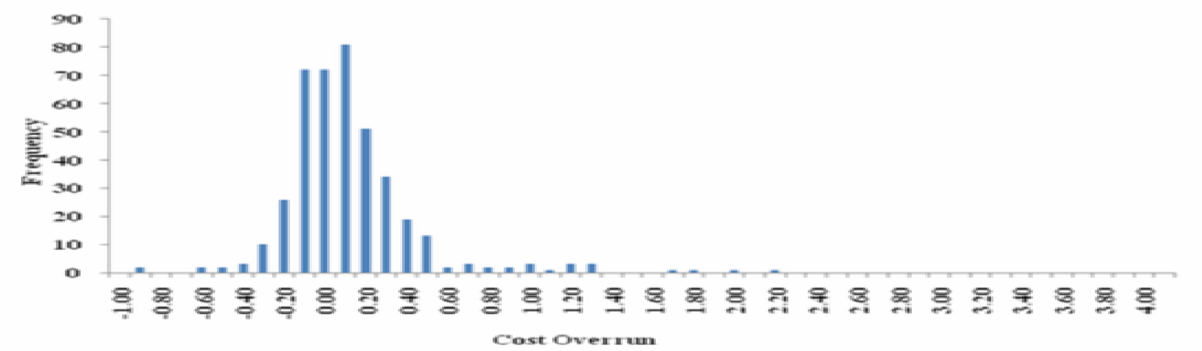


Figure F.2: Actual costs of pipeline construction projects in respect with the estimated costs (Rui et al., 2012)

F.3 Net Present Value Analysis Input

This section puts forward the variables that serve as input for these particular NPV-analysis. Due to the differences in input in NPV's, every section will mention for which NPV-analysis it serves as input. As mentioned in the introduction of this chapter, the inserted data can differ as it is not known which party will pay for what activity.

F.3.1 Total initial investment costs

Derived from the direct costs, the added costs and the P50 estimate, the total investment costs are estimated. The total investment costs that is needed to start this project is $311.818.527,56. This sum will be appropriated at the start of the project: T_0. As this report is written in an early phase of the project, it is not known who is going to allocate which costs. There are two scenarios put forward in this analysis: Arsari pays the full amount or Arsari pays for the first phase and the governmental institutions will pay for the second two phases. In the table F.5, as seen below both scenarios are depicted.

Table F.5: Two Scenarios of Bearing the Initial Investment

	Arsari	**PDAMs**
Scenario 1: Arsari bears all the costs	$311.818.527,56	$ 0, -
Scenario 2: PDAM pays for the tranportation and offtake phase	$113.988.746,39	$197.829.781,17

As is obvious in the above depicted table, the second two phases: transportation and offtake are much higher in investment costs than the first phase: storage lake. The two latter phases are outside the concession of ITCI and, therefore, ideally the PDAM bear the costs for with respect to ownership. In both the analysis's these scenarios will be incorporated.

F.3.2 Production estimates

The production estimate is calculated and is set on 1,7 $m3/s$ for phase 2A. As mentioned, the capacity of phase 2B and overspill/connection with the phase 1 is not yet defined. Thus, in both NPV-analysis's only the production estimate of phase 2A will be used for this analysis.

F.3.3 Operational & maintenance costs

The operational and maintenance costs are inherent with this project. The operational and maintenance costs are based on the multiple variables that comes to show with the different activities, for instance, energy consumption of a water treatment facility or personnel costs for maintaining the dam. However, due to the lack of data in these detailed costs, an estimation is made derived from the initial investment costs. As recommended by Witteveen+Bos, the operation and maintenance costs will be set at 4,5% of the total initial investment costs: $5.130.000,-. Furthermore, it is estimated that the project will be finished at year three. Deriving from that estimation, in the first two years several subprojects will still be in construction, but also several subprojects will be finished. Therefore, in the first year 25% of the operational and maintenance costs will be inflicted and in the second year 50%. The third year will be at the operation and maintenance costs will be at a full amount of 4,5% of the total initial investment costs. Besides that, in this analysis the operational and maintenance costs are based on solely the storage lake phase as the operational and maintenance costs of the transportation phase and the offtake phase will be under the responsibility of the PDAM's of Samarinda and Tenggarong. In both the NPV-analysis these costs will be incorporated.

F.3.4 Discount rate

To perform a NPV-analysis a well thought of discount rate is essential (Jawad & Ozbay, 2006). Different discount rates can have a large effect on the ultimate result. Normally, when calculating a discount rate for a corporation it is based upon how the company's assets are financed i.e. equity and debt. This discount rate is called the weighted average cost of capital (WACC), however, due to no data available, the discount rate is calculated with the use of Indonesia's interest rate and inflation rate from 2017. The rates are 3,75% and 4,50% in 2017, respectively (Trading Economics, 2017a, 2017b). As this is a long-term project and the rates have proven to be volatile in the past, an average will be taken of the historical interest and inflation rates of the last 10 years as depicted in table F.6.

Table F.6: Average of inflation and interest rate of the last 10 years

	2007	2008	2009	2010	2011	2012	2013	2014	2015	2016	2017	**Average**
Interest Rate	9.5%	8.0%	8.6%	6.5%	6.5%	6.0%	5.75%	7.5%	7.75%	7.5%	4.25%	**7,0%**
Inflation Rate	6.40 %	10.21 %	4.45 %	5.12 %	5.38 %	4.28 %	6.40 %	6.42 %	6.38 %	3.53 %	3.88 %	**5,7%**

$$NominalDiscountRate = \Big((1 + InflationRate) * (1 + InterestRate)\Big) - 1 \tag{F.7}$$

From the formula, a nominal discount rate of 13,1% is subtracted. As, both rates are fairly low in comparison with historical rates and these rates followed a volatile path, it is not certain that the calculated discount rate will be the most robust. Therefore, several scenario's will be put forward to perform a sensitivity analysis. In these scenario's, the above-mentioned discount rate, interest rate and inflation rates are changed $+/-$ 25%.

F.3.5 Electricity revenue

More than 20% over whole Indonesia and even 100% in West Kalimantan of the power generation is from diesel or fuel oil, the most expensive fuels for power generation ((Bank, 2010)). Therefore, opportunities arise when power can be generated with the construction of the artificial storage lake. Electricity generation costs in West Kalimantan are on average $0,25 per kWh, while neighbouring countries can produce for rates as low as $0,03 - 0,11 kWh. In order to estimate the possible revenue that could be generated by constructing a hydro power plant, preliminary calculations have been made.

For the water target level of 160 m +MSL and the level at which the hydro power plant is planned to be constructed a gross revenue of $10,6 million / 10 years is estimated. Note that this assumes full production all year round and a selling price of $0,18 per kWh. Also, the fluctuating water level and thus the varying energy production must be accounted for.
Other than revenue, the hydro power plant can reliably supply energy to the planned water treatment plant. This may be an invaluable asset to the client and must be considered.

F.3.6 Average water price

The average water price is currently set on is $0.32\$/m^3$ in Samarinda and in Tenggarong $0.34\$/m^3$ cents per m3, derived from data provided by the client. The average water price is based on different tariff-schemes. The industry, for example, has higher tariffs for "clean water" than the inhabitants or hospitals. Based on the average water prices the NPV-analyses differ from each other. In the NPV-analysis with a payback period of 10 years, the price, Arsari should receive to break-even in 10 years, is subtracted. In the other NPV-analysis the average water price serves as input, thus, one can set the price and calculate what year the project will break-even. As said, the current average water price is 32 and 34 cents per m3 in 2017. In Indonesian law, it states the PDAM can raise the water price with a maximum of 10% per year, as told by the client. It is assumed, that inherently to the 10% increase, the government has corrected it on the basis of inflation. Consequently, the starting point for the second NPV-analysis is with a water price of 33 cents per m3 and an increase of 10% per year until it breaks even. For the reason that an 10% increase is rather high and could not be maintained in perpetuity, an increase of 1% is incorporated in this analysis after the project breaks even.

Appendix G

Stakeholder engagement

G.1 Project based VS Process based for the engagement of stakeholders.

In a perfect world, stakeholder engagement for large scale projects would consist of one meeting that results an agreement on a problem formulation, common goals, boundaries and scope, a project plan and planning, a budget estimation that ends up to be on the high side and one meeting at the end of the project where all stakeholders conclude it the project is completed successfully. In reality stakeholder engagement in large scale projects, such as the Samarinda Water Project, consists of a multitude of meetings in which the stakeholders use strategies to realize their individual goals and in the end, hopefully, all stakeholders are satisfied with the end result. For the engagement of stakeholders two different approaches exist, a project based approach and a process based approach (Hans de Bruijn, 2010). The project-based approach allows for clear distinction between projects phases, creating overview and reducing the perceived complexity (Hans de Bruijn, 2010; Nicholas, 2008). Therefore, a short comparison of both approaches based on the clear distinction of the project approach is given in table G.1.

Table G.1: Project-based VS Process-based
(Nicholas, 2008) p 577-587 (Hans de Bruijn, 2010) p.41-56 (de Bruijn, 2008) p.60-84

	Project-based	**Process-based**
Problem Formulation	Joint problem definition with user requirements.	Individual problem perceptions with individual engagement.
Establishing goals, boundaries and scope	Well defined up front. Focused on factors and content	Dynamic and defined through learning processes. Focused on stakeholders and process.
Information gathering	Robust, intersubjective knowledge, based on the problem content.	Negotiated knowledge, used strategically. Focused on relations.
Decision making	Right-wrong based decision making. One-issue focused	Gain-loss based decision making. Multi-issue focused.
Implementation	Follows the decisions	Decisions follow the actions already taken or decision making is avoided.
Evaluation of project success	Based on realisation of goals at the end of the project.	Based on stakeholder satisfaction and stakeholder relations, continuous process.

The difference between both strategies as shown in table G.1 shows that both approaches are suitable for different situations. Seen from both perspectives, the project initiator has the advantage of designing and initiating the engagement of stakeholders, creating the engagement agenda and involving stakeholders based on analysis of the project (Hans de Bruijn, 2010; Maylor, 2010). For the client, this entails they can design the start of the process to address the risks currently making the project infeasible.

G.2 Stakeholder Engagement in successive phases

The previous section focused on risks for the feasibility and initiation of the Samarinda water project. In this section recommendations for the stakeholder engagement and management approach in the phases following the initiation are given.

(Hans de Bruijn, 2010) define 4 requirements for a good process. They define a good process as "an open process in which parties are offered security through protection of their core values, which offers sufficient incentives for progress and momentum, and which offers sufficient guarantees for the substantive quality or the results" (Hans de Bruijn, 2010). From these requirements several recommendations for the Samarinda water project can be made. An open process entails that all stakeholders involved in a process are also involved in drafting the agenda of the process and that the process and the process management are transparent (Hans de Bruijn, 2010). Therefore, before initiating the process, all possible decisions to be made and stakeholders relevant to these decisions should be mapped in order to involve them. In principle this is a good requirement, there is however tension between openness of and control over the process. When to many stakeholders are involved, allowing them all to influence the process agenda can mean the process is slowed down and small issues become substantive content (Hans de Bruijn, 2010). It is therefore recommended that a thorough assessment is made to define who to involve and how they can be involved. This assessment should go beyond critical stakeholders, but also assess for whom the substance of decision is relevant. The protection of core values entails that regardless of the process outcome, the key interest that stakeholders find so important that if they are affected they would not participate, are protected. This ensures that parties do not withdraw out of the process when they do not fully satisfy or accomplish their interests of goal because a certain amount of protection is offered (Hans de Bruijn, 2010). These core value can be straightforward. An example might be the political accountability for an individual with political responsibilities or face value with respect to the inhabitants for the PDAMs (Hans de Bruijn, 2010). Protection of these core values benefits the commitment of stakeholders to the process rather than to a specific result because they know their key interests are protected. It may well be the case that key interests of stakeholders are problematic for the process (Hans de Bruijn, 2010). In the stakeholder analysis for the Samarinda water project no problematic key interest have been identified. Incentives for progress and momentum are used to ensure the process reaches a conclusion and decision are made instead of side-tracked. Several methods to create progress and momentum are possible. Applicable for the Samarinda water project is the method in which conflicts are handled in the periphery of the process to enable decision making to progress (Hans de Bruijn, 2010). An example of this is the water price increase. Although it is an important part of the process, it can severely slow down the process when all stakeholders are focussing their resources on one complex issue. In section 10.2,suitable approach, it was recommended to create distance between the client and the risks involved in the price increase and to ensure the PDAMs take ownership of this issue. This would allow the client to focus on the process rather than on individual problems. Furthermore, an important source of urgency is becomes present in the process when pollution of the raw water sources is acknowledged. With that acknowledgement, it is in the best interest of the governmental institutions to ensure progress is made which reduces the probability of governmental stakeholders knowingly slowing down the process. For the substance of the project it is important that there is tolerance towards a large variety of solutions (Hans de Bruijn, 2010). Therefore, the process should be open to all solutions at the beginning. At predetermined moments in the process, relevant stakeholders take part in selection of these solutions in order to cast-off infeasible solutions and adapt the remaining solutions to the newest requirements. Although this might seem as a natural course of events, it is emphasized because it creates two valuable opportunities for the process. It improves the quality of the entire decision making process because it restrains stakeholders from developing tunnel vision and not participating in development of the alternatives because they are involved in the selection process and it is in their best interest to fully understand all options ((Hans de Bruijn, 2010). Furthermore it ensures that the selected solutions have support of a critical mass of stakeholders and individual stakeholders cannot question selected alternatives solely on the basis of their own interest (Hans de Bruijn, 2010).

G.2.1 Management of Stakeholders

To create a rough sense of direction for individuals engagement approach for stakeholders a power-interest grid can is used (Maylor, 2010). In this grid, all involved stakeholders are mapped based on their power over and interest in the project. The end result gives an indication of the priority and type of management for individual stakeholders. Figure G.1 shows the classification for the all stakeholder involved in the Samarinda water project.

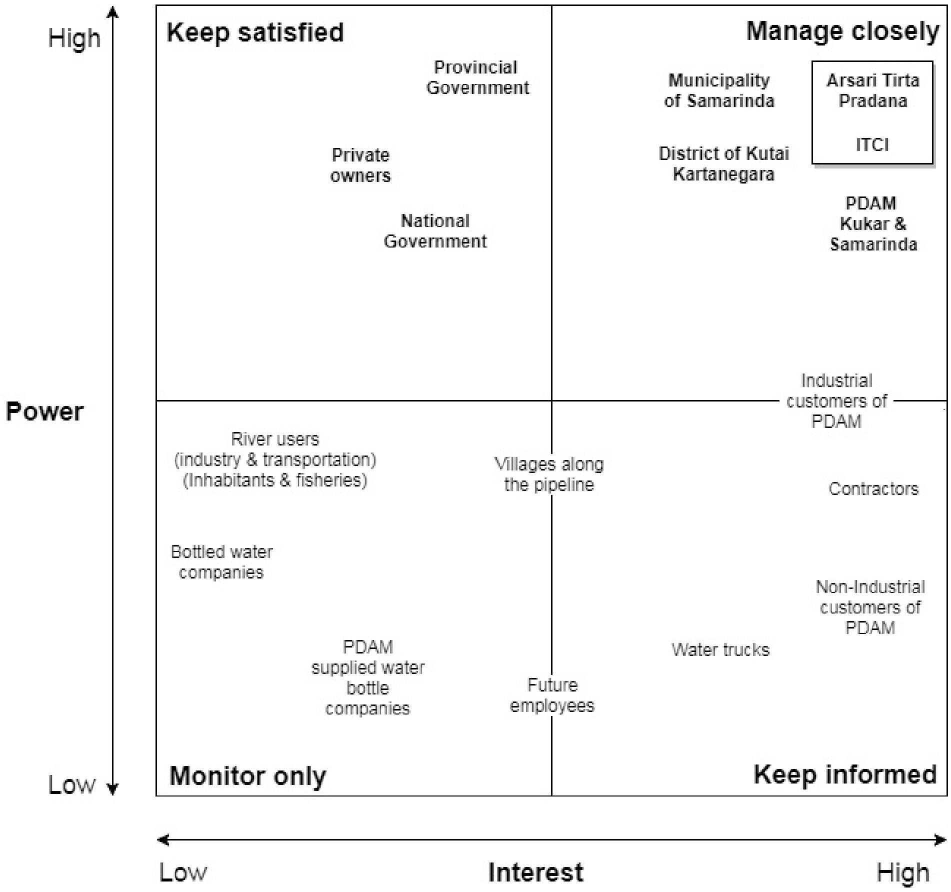


Figure G.1: Power-interest grid *(own work)*

Monitor only

Monitor only stakeholders have the lowest priority and only entail monitoring of the stakeholder development (Maylor, 2010). This is true for the river users, bottles water companies and the PDAM supplied bottled water companies. The Villages along the pipeline and future employees currently are still only to be monitored. However, as the project progresses the villages along the pipeline may become more involved in the project and employees will have to be trained, paid and if necessary provided with housing and transportation which increases their interest in the project (Maylor, 2010).

Keep Informed

Keep informed stakeholders have a moderate priority and their interest should be sustained. They should be involved when their participation in the project becomes useful (Maylor, 2010). The water trucks are customers of the PDAM that function as a secondary distribution network. There are large gaps in the distribution system of the PDAM, therefore they are of importance for the PDAM in providing as much of the inhabitants as possible with clean water. However, they will become less relevant when the distribution capacity increases. The non-industrial customers of the project will be involved in the project and their resources can be used to force stakeholders in a certain direction. Until this is the case, they remain 'keep informed' stakeholders. The contractors are only in need of the most updated information during the first phases (if they are present) and during procurement. They will become critical and 'manage closely' stakeholder, of which the extent depends on the type of contract used, when construction starts.The Industrial companies are of importance because, as is the case in Balikpapan, they can pay a higher fee for clean water to ensure commercial feasibility of the project. Therefore it might be necessary to manage them closely in certain moments of the project.

Keep Satisfied

Stakeholders in this category have a moderate priority and have to be sufficiently involved in the project to be satisfied (Maylor, 2010). For private landowners this encompasses they have to be satisfied with the compensation they receive for eventual use of their land. Because of the implementation of decentralisation in 2001 the regional governments have more responsibilities and power over the project then the national and provincial government (of The Republic of Indonesia The Hague, 2013). However, due to the hierarchical structure and the implementation of the Integrated Water Resources Management plan, part of the 2010 democratic reform the national government has become an important regulator and facilitator through political and financial support (ASB, 2016). They requires them to be informed and satisfied with the progress made in the project.

Manage closely

Stakeholders in the last category, Manage closely, have the highest priority and should be managed through active engagement (Maylor, 2010). This involves the stakeholder most critical to the project, namely the municipality of Samarinda, the district of Kukar and the PDAMs of Samarinda and Tenggarong. For the active engagement it is advised to regularly monitor the internal and external progress of the process and how this influences their position and attitude toward the project. Based on this quick analysis, a adapted strategy can be made and applied accordingly (Turner, 2009).

Appendix H

Risk & Opportunity Analysis

H.1 Risk Register

Category	Risk Description			Pre response assessment		Risk response description		Post-response assessment	
PESTEL	**Cause**	**Risk event**	**Consequence**	**Probability**	**Impact**	**Response**	**Description**	**Probability**	**Impact**
				Political					
	The PDAMs conduct the official test for water quality	The PDAMs deny pollution of Mahakan exceeding government standards	No interest in the Samarinda Water project	Medium	High	Reduce	Use independent experts to prove pollution is harmful for consumers and use this to apply political pressure	Low	High
	The Indonesian Government has a strong hierarchical structure	Hierarchy may be used to force subordinates into a decision	Poor substantive decision making having an impact on project quality	Medium	Medium	Reduce	Adapt the process design to combat hierarchical pressure. For instance through negotiated knowledge and equal influence in decision making.	Low	Medium

Category	Risk Description			Pre response assessment		Risk response description		Post-response assessment	
PESTEL	**Cause**	**Risk event**	**Consequence**	**Probability**	**Impact**	**Response**	**Description**	**Probability**	**Impact**
				Economical					
	A high investment is needed for the Samarinda Water Project	A cheaper alternative is found	No interest in the Samarinda Water Project	Low	High	Accept	Conduct alternatives research for both supply and demand and monitor to see if other offtake possibilities arise. For instance, the demand outgrowing production capacity in Phase 1.	Low	High
	A high investment is needed for the Samarinda Water Project	The regional government and PDAMs might not be able to pay the investment	Other sources of investment have to be found	Medium (high uncertainty)	Medium	Transfer	Create political support on a national level / construct a PPP / ensure private investments	Low	Medium

Category	Risk Description			Pre response assessment		Risk response description		Post-response assessment	
PESTEL	**Cause**	**Risk event**	**Consequence**	**Probability**	**Impact**	**Response**	**Description**	**Probability**	**Impact**
				Technical					
Hydrology									
	Satellite rainfall data has inconsistent accuracy	Rainfall data for project location is inaccurate	The modelled discharge is inaccurate	High	Medium	Avoid	Install accurate rainfall measurement equipment	Low	Low
	The modelled discharge is not calibrated	Modelled discharge is inaccurate	Constant inflow discharge in the reservoir is inaccurate	High	High	Avoid	Use the data from the installed diver to calibrate the model	Low	Low
	Rainfall volume is currently increasing	The rainfall volume keeps increasing	Underdimensioning of the reservoir, increased spillage	Medium	Low	Reduce	Conduct a quantitative meteorological research to better predict the rainfall in the coming design phases	Medium	Low
	Only a very rough subsoil survey has been conducted	Faults and cracks are present in the reservoir area	Increased seepage which could lead to depletion of the reservoir	Low	High	Reduce	Conduct a more thorough geological survey	Low	High

Category	**Risk Description**			**Pre response assessment**		**Risk response description**		**Post-response assessment**	
PESTEL	**Cause**	**Risk event**	**Consequence**	**Probability**	**Impact**	**Response**	**Description**	**Probability**	**Impact**
	The evaporation has been calculated by a standard formula, in this formula the height and humidity have not been taken in to the calculation	The calculated evaporation value might not be correct	The water level will increase or decrease over time, resulting in different Hmax, Hmin as well as the constant extraction yield	High	Low	Accept	n.a.	n.a.	n.a.
Dam									
	ASTER GDEM topography maps are used, is known to be inaccurate	Data derived from ASTER GDEM is inaccurate	Will lead to different dam volume	High	Low	Reduce	Detailed topography survey campaign - required to be performed for conceptual and detailed design	Low	Low

Category	Risk Description			Pre response assessment		Risk response description		Post-response assessment	
PESTEL	**Cause**	**Risk event**	**Consequence**	**Probability**	**Impact**	**Response**	**Description**	**Probability**	**Impact**
	High quality rock is necessary for the construction of the Dam	Insufficient quality control in quarry	Will lead to structural failure of the dam	Medium	High	Reduce	Detailed site investigation and laboratory test required to be performed for conceptual and detailed design	Low	High
	High quality clay and sand is necessary for the construction of the Dam	Insufficient quality control in quarry	Poor quality leads to structural failure of the dam	Medium	High	Reduce	Detailed site investigation and laboratory test required to be performed for conceptual and detailed design	Low	High
	Data is used for wind speed, is known to be inaccurate	Incorrect data	Poor quality leads to different dynamic loads	Medium	High	Reduce	Detailed investigation required for the detailed design	Low	High

Category	Risk Description			Pre response assessment		Risk response description		Post-response assessment	
PESTEL	**Cause**	**Risk event**	**Consequence**	**Probability**	**Impact**	**Response**	**Description**	**Probability**	**Impact**
	Seepage occurs by the foundation	Seepage rises above maximum	Will lead to hydraulic failure of the dam	Medium	High	Reduce	A thorough analysis of the foundation and subsurface condition must be performed to counter act these processes	Low	High
	The reservoir will contain waves	Maximal wave overtopping at the dam	Will lead to hydraulic failure of the dam	Low	High	Reduce	A thorough analysis of the waves must be performed to counter act these processes to ensure minimal wave overtopping	Low	Low
	Strong variation in water level of the reservoir	Might cause construction material to lose buoyancy	Sliding of supporting formation leading to structure failure of the dam	Medium	High	Reduce	Remove soil cover to build on stronger siltstone formation	Low	High

Category	Risk Description			Pre response assessment		Risk response description		Post-response assessment	
PESTEL	**Cause**	**Risk event**	**Consequence**	**Probability**	**Impact**	**Response**	**Description**	**Probability**	**Impact**
	Weathering by water decreases strength of surfacing rock formations	Drops below critical carrying capacity	Structure failure of the dam	Low	Medium	Accept		Low	Medium
	Shale has a maximum bearing capacity.	Adding surcharge to shale formation on western slope the bearing capacity might drop under the critical value	Structure failure of the dam	Medium	High	Reduce	Removal of shale	Low	High
	There are Interbedded weak layers in subsurface	Squeezing events in subsurface	Damage to structure of dam and tunnel	Medium	Medium	Accept		Medium	Medium
	Seepage occurs along fault plane	Induced internal erosion/piping	Structure failure of the dam	High	High	Reduce		Medium	High
	The area is know to have heave seismic activity	Occurrence of earthquake	Structure failure of the dam	High	High	Reduce	Strengthen dam construction	High	

Category	Risk Description			Pre response assessment		Risk response description		Post-response assessment	
PESTEL	**Cause**	**Risk event**	**Consequence**	**Probability**	**Impact**	**Response**	**Description**	**Probability**	**Impact**
	Changing hydrostatic regime can cause decreasing of cohesion in the slope (further induce by removal of vegetation)	Slope instability might occur	Structure failure of the dam	High	High	Avoid	Removal of top soil	n/a	n/a
Reservoir									
	Seepage from reservoir due to karst	Karst connections between basin and other areas	Endangers technical feasibility of the reservoir	Low	High	Reduce	Fill up the Karst connections		
	Seepage from reservoir due to fault	Faults acts as pathway for water flow	Endangers technical feasibility of the reservoir	Medium	Medium				
	Due to climate change the precipitation patterns change	Less rainfall might occur	Leading to depletion of the reservoir	Medium	High	Reduce	In depth study of future weather pattern to adapt the design of the reservoir to what is expected		

Category	Risk Description			Pre response assessment		Risk response description		Post-response assessment	
PESTEL	**Cause**	**Risk event**	**Consequence**	**Probability**	**Impact**	**Response**	**Description**	**Probability**	**Impact**
Hydro power									
	Average discharge hasn't been calibrated yet	Extraction yield may be different than expected	Less power production	Medium	Medium	reduce	In-situ measurement are needed to calibrate the model outputs	Low	medium
	Not enough trained personnel to maintain the equipment	Longer downtime because of inefficient maintenance	Less energy production	Medium	Medium	Reduce	Train personnel in house	Low	Medium
	A reservoir has a chance of overtopping	Flooding of the power plant	Short circuiting and breakdown in the power plant	(very) Low	(very) High	Reduce	Segment the building, but very low probability and costly –>Blackswan?	Low	Low
	Hydro power plant is built in a remote area	Long emergency response time	Failure of operation	Medium	Medium	Reduce	Better infrastructure to the plant to cut travel time to the hydro power plant	Low	Medium

Category	Risk Description			Pre response assessment		Risk response description		Post-response assessment	
PESTEL	**Cause**	**Risk event**	**Consequence**	**Probability**	**Impact**	**Response**	**Description**	**Probability**	**Impact**
Water treatment									
	Water treatment plant is built in a remote area	long emergency response time	Failure of operation	Medium	Medium	Reduce	Better infrastructure to the plant to cut travel time to the hydro power plant	Low	Medium
	A reservoir has a chance of overtopping	Flooding of the power plant	short circuiting and breakdown in the power plant	(very) Low	(very) High	reduce	Segment the building, but very low probability and costly –>Blackswan?	Low	Low
Transport									
	The region is known for heavy rainfall and erosion	Mudslide or geo-instabilities can occur	Destruction of pipeline	Medium	High	Avoid	Build the pipeline in an area where landslides are not possible	n/a	High
	Infrastructure can be sabotaged and stolen	Material theft	Operational failure	Low	Medium	Reduce	CCTV, motion triggered alarms, patrol, surveillance	Low	Medium

Category	Risk Description			Pre response assessment		Risk response description		Post-response assessment	
PESTEL	**Cause**	**Risk event**	**Consequence**	**Probability**	**Impact**	**Response**	**Description**	**Probability**	**Impact**
	Pipeline welds, fittings and narrow bends are prone to failure under high pressure	Leakage	Operational failure	Low	Medium	Reduce	Pre-instalment tests and detailed hydraulic assessment	Low	Medium
Tunnelling									
	Presence of slumps causes varying rock characteristics for tunnel	Layers may settle differently affecting the pipeline	Structure failure of the tunnel	Low	High	Reduce	Enforce tunnel structure and research rock layers to find the best tunnel route.	Low	Low
	Pipeline welds and fittings are prone to failure under high pressure	Leakage	Operational failure	Low	Medium	Reduce	Pre-instalment tests and detailed hydraulic assessment	Low	Medium
	Interbedded weak layers in subsurface	Squeezing events in subsurface	Damage to structure of the tunnel	Low	Medium	Accept		Medium	Medium

Category	Risk Description			Pre response assessment		Risk response description		Post-response assessment	
PESTEL	**Cause**	**Risk event**	**Consequence**	**Probability**	**Impact**	**Response**	**Description**	**Probability**	**Impact**
				Social					
	Aligned problem perceptions are crucial for project success	Governmental institutions maintain a different problem perception	Difficult cooperation with negative effects on project quality	Medium	High	Reduce	See Engagement approach report "Different problem perceptions"	Low	High
	Due to the high investment, a increase in the Average Water Selling Price is necessary	The non-industrial customers might not accept this price increase	Public uproar and a negative attitude towards the project	High (current estimates indicate a very sharp price increase)	High	Avoid	See Engagement approach report "Average Water Selling Price"	N/A	High
	The client wants to ensure the PDAM and regional government take ownership over the operational phase of the project	The PDAM and/of regional government might not agree to this requirement	Impact of project cost for the client due to extra investment cost for operational phase	Low	Medium	Reduce	See Engagement approach report "Financial and Operational Ownership"	Low	Medium

Category	Risk Description			Pre response assessment		Risk response description		Post-response assessment	
PESTEL	**Cause**	**Risk event**	**Consequence**	**Probability**	**Impact**	**Response**	**Description**	**Probability**	**Impact**
	Stakeholder pursue their own interest	They may resist interventions that do not serve their own interests.	Has a negative effect on process progress, quality and trust.	Medium	Medium	Reduce	Ensure all stakeholders agree that the joint end goal is more important than individual interests and install incentives for collaborative behaviour.	Low	Medium

Category	Risk Description			Pre response assessment		Risk response description		Post-response assessment	
PESTEL	**Cause**	**Risk event**	**Consequence**	**Probability**	**Impact**	**Response**	**Description**	**Probability**	**Impact**
				Legal					
	Many permits are required for land acquisition and construction	Governmental stakeholders may use their political influence to block permit applications	The project is delayed	Medium	High	Reduce	Ensure mutual collaboration with governmental stakeholders before permit applications	Low	High
						Avoid	The client can use their network of relations to persuade superior governmental stakeholders to apply pressure on subordinate governmental stakeholders.	Low	High

Table H.1: Risk register

H.2 Opportunity Register

Category	Risk Description			Pre response assessment			Opportunity response description
PESTEL	**Cause**	**Opportunity event**	**Consequence**	**Probability**	**Impact**	**Exploit, Share, Enhance, Accept**	**Description**
				Political			
	The Indonesian Government has a strong hierarchical structure	Hierarchy may be used to force subordinates into a decision	Unwilling subordinate institutions are forces to cooperate	Medium	Medium	Accept	The hierarchical structure can be used as a tool. However, forcing parties to cooperate can have a bad influence on their future performance and willingness to cooperate. It should therefore be used with caution and not exploited, enhanced or shared.
	The Indonesian government is supportive of the development of private party involvement in large scale infrastructure projects.	Private party involvement can increase performance for infrastructure projects through: higher level of professionalism, value for money, efficiency and quality of services (Asian Development Bank, 2016)	Increase in performance can increase national, regional and local support for and interest in the project.	Medium	Medium	Enhance	Enhance the probability of occurrence through active engagement of private parties and governmental parties to ensure private party involvement is possible.

Category	Risk Description			Pre response assessment			Opportunity response description
PESTEL	Cause	Opportunity event	Consequence	Probability	Impact	Exploit, Share, Enhance, Accept	Description
			With private project involvement or ownership difficulties arising from decentralisation and governmental partition can be overcome.	Medium	Medium	Enhance	Enhance the probability of occurrence through active engagement of private parties and governmental parties to ensure private party involvement is possible.
			Economical				
	The Indonesian Government is actively facilitating and regulating large water supply infrastructure projects	They may give political and financial support to the Samarinda project	Receiving subsidies greatly improves the financial feasibility of the Samarinda water project	Low	High	Enhance	To enhance the probability of political and financial support of the Samarinda water project the government has to be actively engaged and satisfied as soon as possible.
	Government is actively supporting Public Private Partnerships.	PPP may be applicable for the Samarinda water project for investment and cost sharing possibilities	Improving the financial feasibility of the Samarinda water project.	Low	High	Enhance	Actively engage the regional governments, PDAM and potential investors with the intention to create a PPP.

Category	Risk Description			Pre response assessment			Opportunity response description
PESTEL	**Cause**	**Opportunity event**	**Consequence**	**Probability**	**Impact**	**Exploit, Share, Enhance, Accept**	**Description**
	The Mahakam river is polluted	PDAM's may desperately need a clean water supply and may be willing to bear the initial investment costs	Significantly less investment costs	Medium	High	Exploit	If the pollution is acknowledged engage the PDAMs as described in the stakeholder engagement for the financial ownership.
	Biochar is a cheaper and more sustainable than the WTP	Biochar may yield the same water quality as a WTP	Significantly less investment costs	High	High	Exploit	Conduct research into biochar as an alternative to the current set up. In that way Arsari is still the owner of the project through an alternative.
	The PDAM has industrial and non-industrial customers and the Industrial customers pay a higher tariff for clean water	The industry in Samarinda and/Tenggarong may grow faster than predicted	More revenue created from industrial customers	Medium	Medium	Exploit	The price for industrial customers could be increased faster than non-industrial customers
						Accept	Due to the increase in growth of industrial customers, more revenue is already created leading to higher income/year

Category	Risk Description			Pre response assessment			Opportunity response description
PESTEL	**Cause**	**Opportunity event**	**Consequence**	**Probability**	**Impact**	**Exploit, Share, Enhance, Accept**	**Description**
	The Indonesian government is supportive of the development of private party involvement in large scale infrastructure projects.	Private party involvement can increase performance for infrastructure projects through: higher level of professionalism, value for money, efficiency and quality of services (Asian Development Bank, 2016)	More value for money and a higher efficiency leads to a higher financial feasibility.	Medium	Medium	Enhance	Enhance the probability of occurrence through active engagement of private parties and governmental parties to ensure private party involvement is possible.
				Technical			
Hydrology							
	On the other side of the concession, the phase 1 reservoir is being built.	Phase 1 might have excess capacity which could be used as extra input for phase 2 lake A.	Increase of water supply and hydro power production.	Medium	Medium	Enhance	If an excess capacity is present, redirect is and use it to its full extent.
	There is room for additional artificial reservoirs	A second reservoir might be built and connected to phase 2 reservoir A.	Increase of water supply and hydro power production.	High	High	Accept	If the second reservoir is build, use it as it is needed to meet the demand.

Category	Risk Description			Pre response assessment			Opportunity response description
PESTEL	**Cause**	**Opportunity event**	**Consequence**	**Probability**	**Impact**	**Exploit, Share, Enhance, Accept**	**Description**
Dam							
	The current starting point is a conservative approach regarding overtopping.	More detailed insight in the “failure criteria” in respect to construction material and water level fluctuations could lead to lower crest levels	Decrease in quantities and lower investment costs	High	High	Enhance	Perform detailed design for the dam
	There are available resource within the storage lake and at the dam	Take out all the available resource within the storage lake before filling	Both subsoil and trees can be re-used within the ITCI compound. This potentially gives a profit.	High	Medium	Enhance	Research to the available sources & project planning
	There are available resource within the storage lake and at the dam	Take out all the available resource within the storage lake before filling	It includes an additional interface / milestone to the project planning	High	Medium	Enhance	Research to the available sources & project planning

Category	Risk Description			Pre response assessment			Opportunity response description
PESTEL	**Cause**	**Opportunity event**	**Consequence**	**Probability**	**Impact**	**Exploit, Share, Enhance, Accept**	**Description**
Hydro power							
	High availability on various turbine types	Get greater value for money	Higher production, with lower investment	Medium	Medium	Accept	Perform detailed evaluation of the set-up and do compare various companies with various turbine types
	Isolation of hydro power plant	better connection to the grid	Lower investment, lower transport costs	High	Medium	Enhance	As client demands the hydro power plant within proximity of the concession area, it is up to them to make use of this opportunity
Transport							
	Steel pipes are vulnerable to corrosion	Good quality water lead to less corrosion	No extra coating needed/or less, so to	Medium	medium	Exploit	If the water quality is found to be good, no coating is necessary
	Coated steel pipes are durable and reliable	Single pipeline may be used instead of two	Cheaper transport	Medium	Medium	Accept	At this moment water supply security is assumed to be more important that the financial benefits of using one pipe
	HDPE is widely available	Fusion and material costs might become cheaper in large quantities	HDPE material is cheaper, less maintenance intensive pipelines	Medium	Medium	Exploit	Research the cost price of HDPE and the cost of fusion

Category	Risk Description			Pre response assessment			Opportunity response description
PESTEL	**Cause**	**Opportunity event**	**Consequence**	**Probability**	**Impact**	**Exploit, Share, Enhance, Accept**	**Description**
	Pressure in the pipeline varies over the length of the pipeline	Pressure may be lower in certain sections, enabling reduction of the thickness of the pipewalls	Thinner pipewalls, less steel needed leading to lower investment cost	High	Medium	Exploit	Research the pressure levels more detailed along the trace of the pipe
	Currently it is assumed that sabotage and geo-instability are a threat to the pipeline	Sabotage and geo-instability might not be a threat, enabling above the ground installation	Cheaper solution and easier to reach maintenance	Medium	High	Exploit	Engaging the surrounding inhabitants on a social level, incentivising them to not sabotage the pipeline. Or supply clean water to the surrounding villages.
				Social			
	The Mahakan river is polluted	The inhabitants of Tenggarong and Samarinda may be outraged and use their resources to demand clean water from the PDAM and regional government	Finding a new, reliable and durable clean water sources becomes urgent	Medium	Medium	Exploit	Use the urgency to benefit the full fill client requirements such as the PDAM agreeing to financial and operational ownership of the transportation and offtake phase

Table H.2: Opportunities register