NTNU – Trondheim
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Study of Hydropower Project in Zambia

Selection and Optimization of Solutions in

Muchinga HPP

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Preface

This master thesis represents the closing part of the five-year education at the Norwegian University of Science and Technology, NTNU, under the Department of Hydraulic and Environmental Engineering. The thesis is an extension of the introductory work conducted in the fall of 2012.

Working with a project situated in an environment so different from Norway has been a challenge. It has been an incredible learning experience, forcing me into both making my own assessments and to validate them. The field trip to Zambia was both useful as direct input for this thesis, but also a wonderful experience for me personally. For that I am thankful.

I have been fortunate enough to be working with some very experienced people. They have been eager to share their knowledge and to guide me in the process. I would like to thank Siri Stokseth at Statkraft/NTNU for continuous supervision and guidance, Helge Saxegaard at Veidekke AS for guidance and education regarding the asphaltic core dam, and supervisor Lars Ødegård and professor Leif Lia at SN Power and NTNU respectively. Furthermore, I would also like to thank Ånund Killingtveit, for taking his time to teach me nMAG, and Lunsemfwa Hydro Power Company for their warm welcome and commitment during the field trip to Zambia.

It has been a great experience writing this master thesis. I wish you, the reader an equal great experience reading it. Enjoy!

Trondheim, June 9 2013

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Abstract

The supply of electricity in Zambia is mainly based on the production of hydropower. Currently only 1780 MW out of 6000 MW of the hydropower potential is being utilized. Zambia has in recent years experienced economical growth, and the demand for power is increasing. However, it is expected that there will be a shortage of power supply until 2017. Lunsemfwa Hydro Power Company ltd. (LHPC) is currently making a feasibility study of the Muchinga Hydro Power Project. Several consultant engineers have been working on the project, which constitutes parts of the background material for this master thesis.

This report examines the optimal solution of the dam, waterways and powerhouse. A simulation of the future production in the Muchinga HPP is also conducted.

The background materials have been thoroughly assessed and the proposed solutions have been adapted to optimize the overall result of this report. The approach to best assess the overall solution has been based on literature reviews, experience based conversations, a field trip to Zambia and an asphaltic core test project.

This report highlights the Asphaltic Core Embankment Dam (ACED), with a central core as the best solution for the dam type. Structures built in Zambia should be sought to be made maintenance free and an ACED with central core matches this requirement better than a RCC dam. The ACED is in addition 16,5 M USD cheaper than the proposed RCC dam, costing 38,4 M USD in total.

The optimal waterway solution has been identified to consist of an underground powerhouse, head- and tailrace tunnel (8200 m and 1540 m respectively) and a 455 m vertical unlined pressure shaft. An optimization with respect to minimizing the head loss, highlights the cross sectional areas to be 40 m^2 and 18 m^2 for the head- and tailrace tunnel and pressure shaft respectively. The total cost of the waterway and powerhouse is estimated to be 46,7 M USD.

The total head loss is calculated to be 9,4 m. This implies that the net head for hydropower production is 480,6 m. As a consequence the total installed capacity is estimated to be 228 MW, resulting in an annual power production of 1395 GWh.

Sammendrag

Tilførselen av elektrisitet i Zambia er i hovedsak basert på produksjon av vannkraft. Foreløpig er kun 1780 MW av totalt 6000 MW av vannkraftpotensialet blitt utnyttet. Zambia har de siste år opplevd en økonomisk vekst og etterspørselen etter strøm øker. Det forventes imidlertid at det vil være mangel på strøm frem til 2017. Lunsemfwa Hydro Power Company ltd. (LHPC) gjør nå en mulighetsstudie av Muchinga Hydro Power Project. Flere rådgivende ingeniører har jobbet med prosjektet og deres arbeid utgjør deler av bakgrunnsmateriale for denne rapporten.

Denne rapporten undersøker den optimale løsningen av dam, vannvei og plassering av kraftverk. En simulering av fremtidig produksjon i Muchinga HPP er også gjennomført.

Bakgrunnsmaterialet har blitt grundig vurdert og de foreslåtte løsninger har blitt tilpasset for å optimalisere det samlede resultatet. Tilnærmingen til å best kunne vurdere den generelle løsningen er basert på litteraturstudier, erfaringsbaserte samtaler, en ekskursjon til Zambia samt deltagelse på et asfaltkjerne prøveprosjekt.

Denne rapporten belyser Asfaltkjerne dam (ACED) med sentral tetning, som den beste løsningen for valg av dam type. Konstruksjoner i Zambia bør etterstrebes å bli gjort vedlikeholdsfrie. En Asfaltkjerne dam med sentral tetning oppfyller dette kravet bedre enn en RCC dam. I tillegg er totalkostnaden på en Asfaltkjerne dam 38,4 M USD, 16,5 M USD billigere enn en RCC dam.

Den optimale løsningen på vannveien er blitt identifisert til å bestå av en kraftstasjon i fjell, trykk- og avløpstunnel (på henholdsvis 8200 m og 1540 m) og en 455 m vertikal uforet trykksjakt. En optimalisering med hensyn på å minimere falltapet fremhever tversnittsarealet til å være 40 m^2 og 18 m^2 på henholdsvis trykk- og avløpstunnel og trykksjakt. Den totale kostnaden på vannveien og kraftstasjon i fjell har blitt beregnet til 46,7 M USD.

Det totale falltap er beregnet til 9,4 m. Dette medfører at netto fallhøyde for kraftproduksjon er 480,6 m. Som en konsekvens er samlet installert kapasitet beregnet til 228 MW, noe som resulterer i en årlig kraftproduksjon på 1395 GWh.

Contents

Glossary and Abbreviations

Glossary

ICOLD	The International Commission on Large Dams. ICOLD is a non-governmental, international organization, which provide a forum for the exchange of knowledge and experience in dam engineering [1]
NVE	The Norwegian Water Resources and Energy Directorate is a directorate under the Ministry of Petroleum and Energy. NVE's main work areas is development of legal framework for hydropower design [2]
SP	Studio Pietrangeli, Italian consulting engineering company. Responsible for providing most of the background material
Firm power	Term used for energy supplied with a certain degree of reliability
Occasional power	Excess power produced, when it is beneficial to produce more than the need for firm power should indicate

Abbreviation

ACED	Asphaltic Core Embankment Dam
CFRD	Concrete Faced Rockfill Dam
RCC	Roller Compact Concrete
M	Equals to a million (1.000.000)
m.a.s.l	meters above sea level

Chapter 1

Introduction

This chapter gives a short presentation of the thesis. Background material, supporting documents, objective, purpose, structure and clarification of interfaces are presented.

1.1 Background

The supply of electricity in Zambia is mainly based on the production of hydropower. The overall hydropower potential is approximately 6000 MW of which only 1780 MW is currently utilized. Zambia has in recent years had a good economical growth, and the demand for power is increasing. Even though there are many hydropower projects currently at a feasibility stage, it is to be expected that there will be a shortage of power supply until 2017.

The Zambian company Lunsemfwa Hydro Power Company ltd. (LHPC) is currently making a feasibility study of the Muchinga Hydro Power Project. LHPC's two main shareholders is Agua Imara (a subsidiary of SN Power) with 51% and a locally owned company with 49 %.

This master thesis is being conducted in cooperation with SN Power.

Location of the project

The project is located in Zambia, about 85 km east of Kabwe, and 135 km north-east of the capital Lusaka.

Figure 1.1: Placement of project [3]

Hydropower potential and project characteristics

Muchinga Hydro Power Project is thought to be an expansion of the already existing Lunsemfwa Hydro Power scheme, to better utilize the hydropower potential in the area. The expansion consists of the plans to implement the water from the Mkushi river. See Appendix A for graphical representation.

The project area is characterized by large catchment areas, sufficient amounts of rainfall and almost 500 meter of head. This makes for an ideal placement for a hydropower project. The three different catchment areas Mita Hills, Lunsemfwa and Mkushi are situated on a plateau just north of an escarpment, which provides the potential head for the preliminary placement of the powerhouse. See figure 1.2.

1.2 Background material

The following supporting documents provide part of the basis for the input parameters used in this master thesis.

1.2.1 Conceptual schemes report

The Italian consultant company Studio Pietrangeli (SP) was hired by SN Power to conduct a feasibility study for this project. The results was presented in the *Conceptual schemes report* from 2010. See Appendix A for graphical representation of the overall layout proposed by SP.

The following key features was presented as the best alternatives for each component in this project:

Dam

- RCC dam
- Height = 120 m
- Length of dam crest = 350 m
- Volume = 0,71 Mm^3

Headrace tunnel

- Tunnel capacity of 60 m^3/s
- Length = 6,5 km
- Gross area 29 m^2 with lining. Diameter = 5 m

Penstock

- Number = 3
- Length = 1,3 km
- Diameter = 2,2 m

Power house

- Type = Open air
- Turbine type = Pelton
- Number of turbines = 3
- Net Head = 488 m
- Maximum turbine discharge = 57 m^3/s
- Annual production = 1630 GWh/year
- Plant factor = 0,73
- Total installed capacity = 255 MW

1.2.2 Hydrological report

Summarized this report highlights:

- Input value for future estimation of power production (See table 1.1)
- Flood analysis of Mkushi river (see table 1.2)

Table 1.1: Mean annual flows

Mita Hills	990 Mm^3/year
Lunsemfwa	142 Mm^3/year
Mkushi	507 Mm^3/year
Total inflow to the system	1639 Mm^3/year

Regional statistical analysis has been applied in the calculating the design floods. Consequently, the input assumed for the assessment of the spillway solution is as follows:

Table 1.2: Flood analysis of the Mkushi river

Return period [years]	**Design floods [m^3/s]**
1000	469
10000	782
PMF	1864

An assessment of the climatic and physical conditions of the project area has been conducted, leading to clarification of catchment areas and corresponding characteristics see figure 1.2.

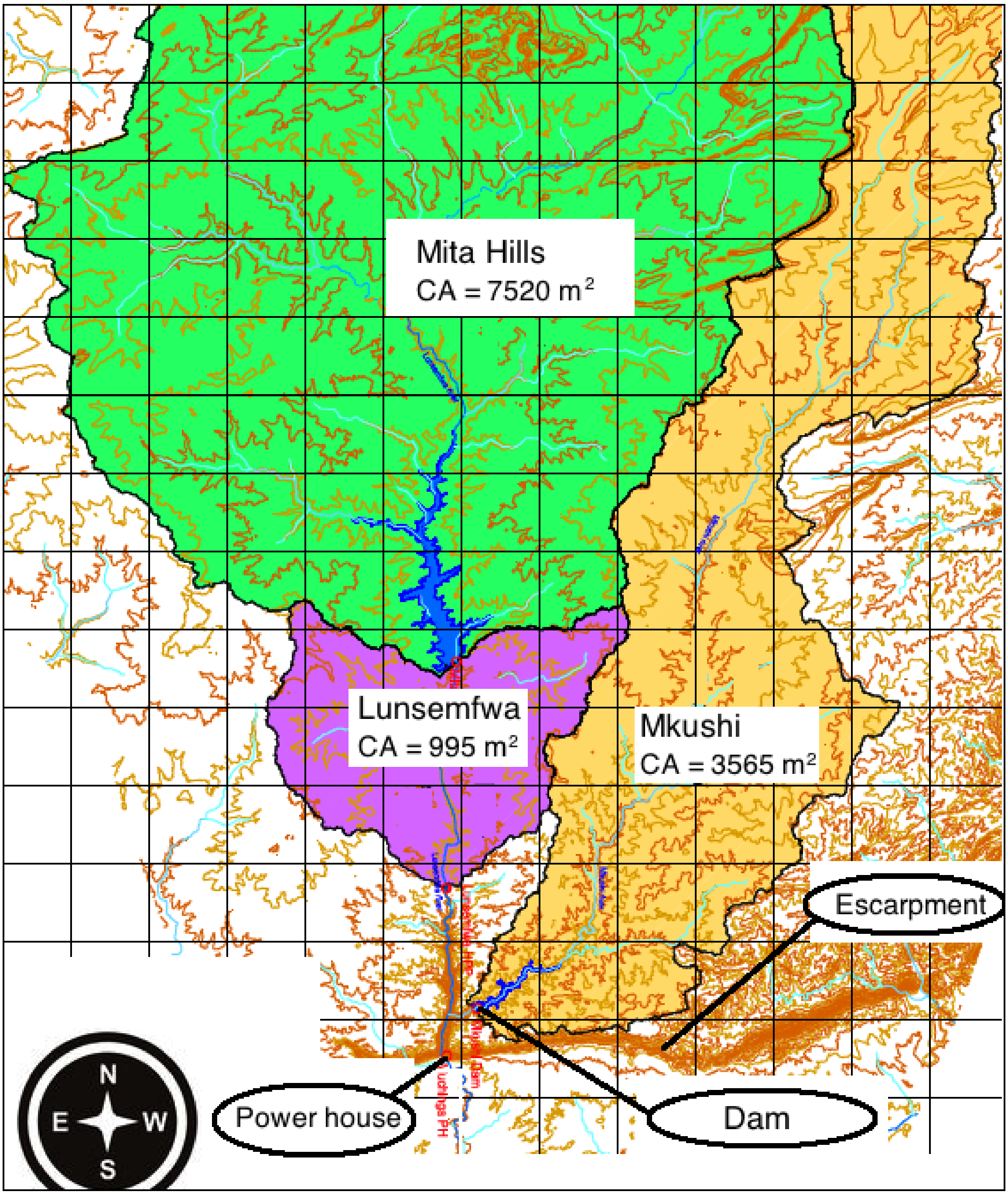


Figure 1.2: Catchment areas [4]

1.2.3 Geological report

Evaluating this project, SP conducted a geological report that highlights the geological properties of the project area. The overall geological map is presented in figure 1.3

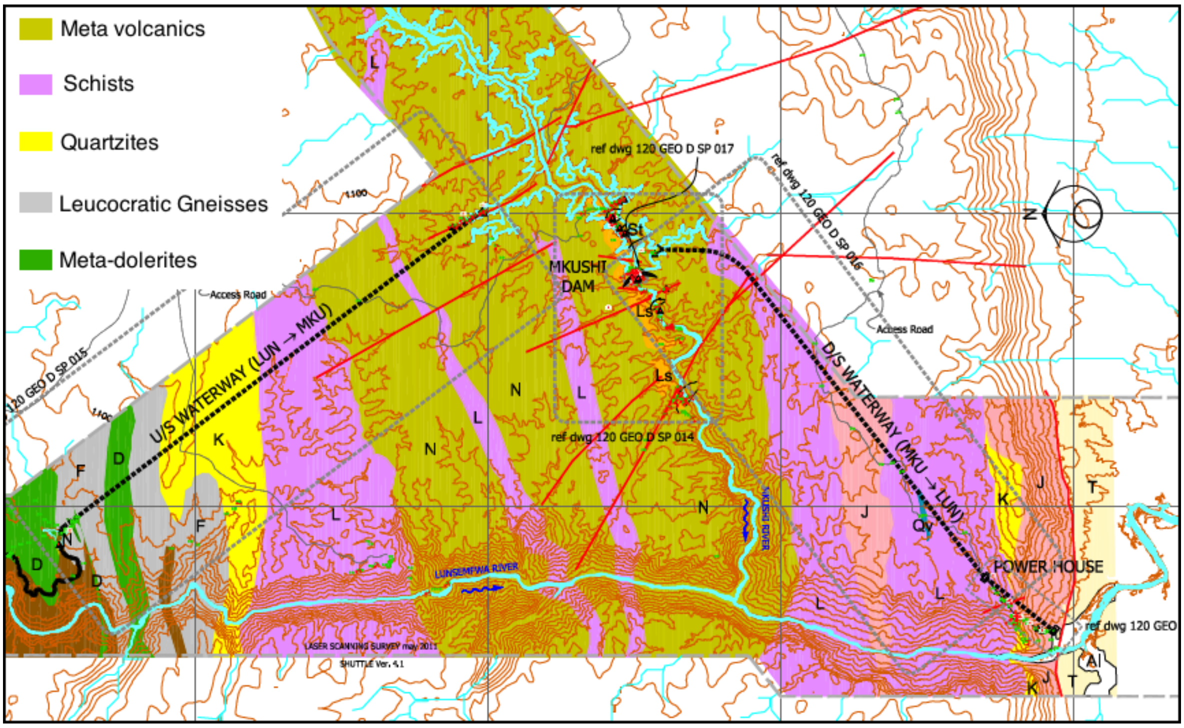


Figure 1.3: Geological map, overview [5]

Dam site

According to SP, the dam site consists mainly of metavolcanics of the Muva supergroup. The left bank of the Mkushi river is only metavolcanics, while the right bank is similar but with a layer of Lufubo schist at the very top of the bank (above 990 m.a.s.l). This Lufubo schist layer runs parallel to the river the whole stretch. Preliminary site investigations suggest the rock at the bottom of the valley to be strong and unweathered with Unconfined Compression Strength (UCS) of 100-250 MPa. This suggests a very strong foundation for the construction of the dam.

At the current dam site, the right side (seen in the stream direction) is much steeper than the left side. This is because of the stratification at the dam site. On the right side the dip direction of the strata is almost perpendicular to the mean slope of the bank. On the left side the strata is almost parallel to the mean slope of the bank, see figure 1.4. This presents the left side as the potential surface of weakness.

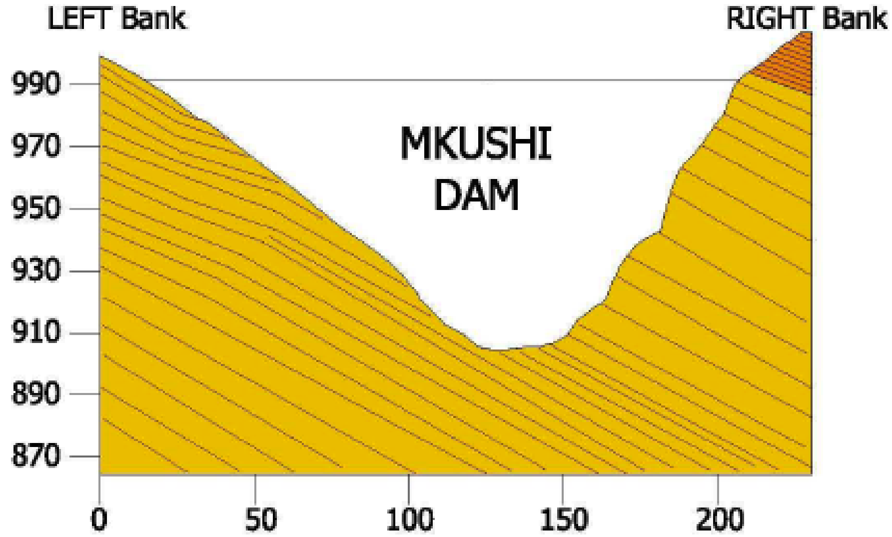


Figure 1.4: Principle drawing of the stratification of the dam site [4]

The preliminary site investigations indicate that the foundation is strong enough to support most dam types. The depth and degree of the weathering of the rock, does not indicate a change of this fact. Regardless, due to limited site investigations, this must be investigated further. In future assessments in this thesis, the fact that the rock at the dam site seems strong enough to support most dam types, will be assumed.

Waterway and Power house

SP presents the following geological map to illustrate the geological composition between Mkushi dam site and the power house, see figure 1.5.

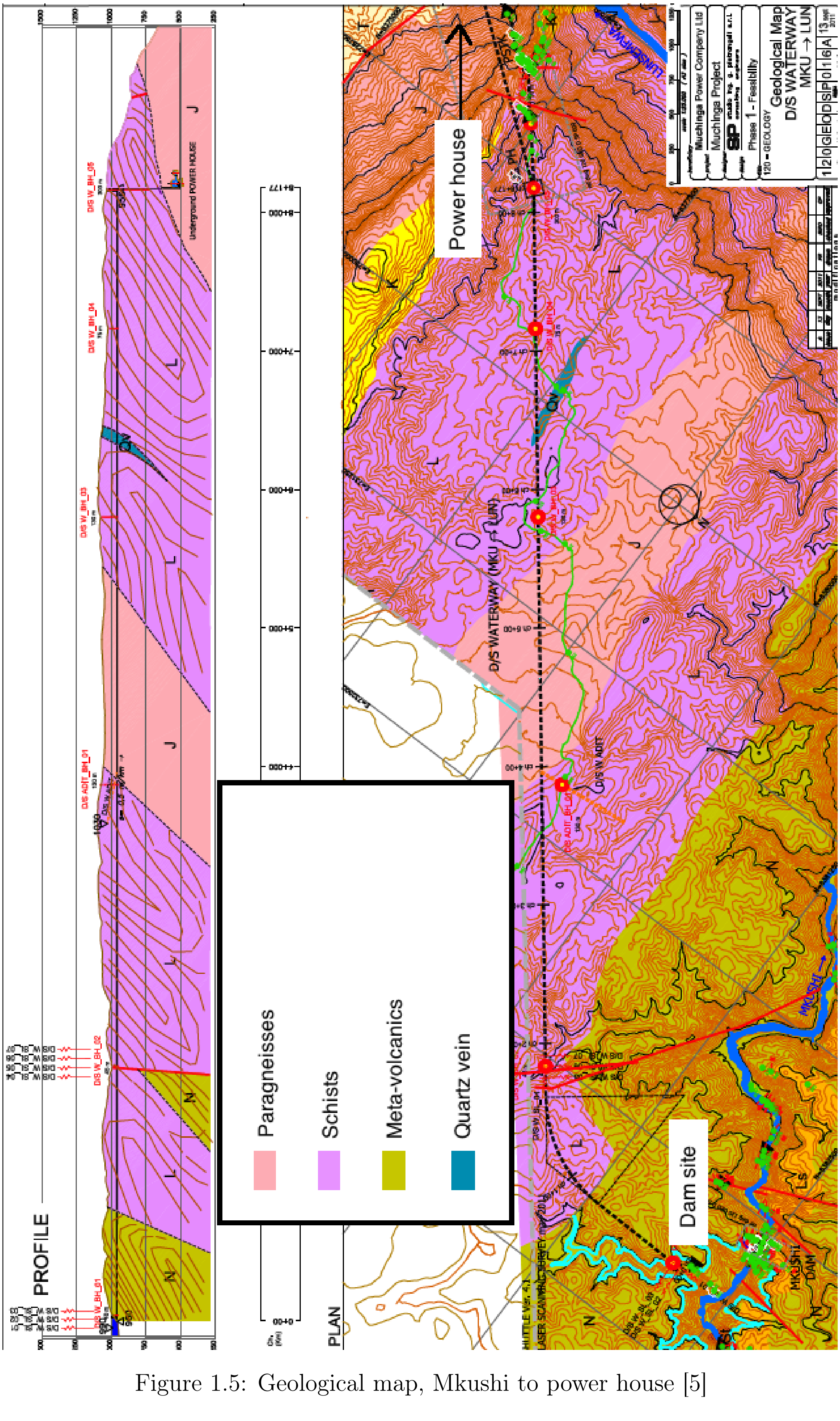


Figure 1.5: Geological map, Mkushi to power house [5]

Figure 1.5 illustrates the different rock types that could be encountered in the making of the headrace tunnels and powerhouse. There are mainly three different types of rock to be encountered; Meta-volcanics, Schist and Paragneisses.

The reason the headrace tunnel has a bend is based on the wish to cross the fault zone (situated south of the dam) perpendicular to its own axis. This will reduce the amount and methods of rock support.

Furthermore, borehole sampling of the headrace tunnel was conducted and as a result a borehole log was constructed by Arcus Gibb. This borehole log indicates that there in all likelihood is a comprehensive zone of weathered rock along the stretch [6]. This proposes the main difference in geological properties compared to Norway, where erosion (from e.g. glaciers) have removed the weathered rock. Because of this, it should be considered higher costs for rock support in areas with low overburden, since this has a significant impact on the total cost of tunneling.

Figure 1.6: Borehole sample

1.2.4 Additional material

Introductory project report

As an introduction to this master thesis, an introductory project report was conducted, *Studie av vannkraftprosjekt i Zambia - hvordan stedlige forhold og anleggsteknikk påvirker priser og tekniske løsninger*. The introductory project report highlights to some extent principles and assessments made, and will to some degree also provide a basis for assumption in this thesis.

Field trip to Zambia

In January 2013 a field trip to Zambia was carried out. The field trip was conducted over a time span of almost three weeks and consisted of the following main activities:

- Close cooperation with the employees at LHPC and surveying of affiliate networks
- A site visit to Mita Hills reservoir and dam with corresponding spillway solution
- Site visit to the existing Mulungushi and Lunsemfwa powerhouse
- Inspection of dam site - Mkushi
- Two day field trip to the Copperbelt - Gathering of information surrounding experiences made in the mining industry
- Assessment of road conditions and availability at dam site

Unfortunately, because the field trip was conducted during the rainy season, we were not able to visit the potential placement of the powerhouse due to the bad weather conditions.

The experiences made on this field trip help provide a basis for future assumptions made in this master thesis.

Asphaltic core test project

In May 2013, a test project regarding the placement and compaction of the asphaltic core embankment dam (ACED) was conducted. This test project helped clarify the method for construction of the ACED.

Parallel conducted thesis by Marius Jentoftsen

Parallel to the development of this master thesis, an ongoing report is being conducted by Marius Jentoftsen. His main focus will be on the waterways; primarily the canals, transmission- and headrace tunnels. These two reports should be seen in close context and will to some degree build on one another. Consequently, there will be some input parameters that will rely on the works of Jentoftsen (2013).

The cost function used calculating the cost of tunneling has largely been designed by Jentoftsen (2013). This cost function is based partly on an Excel-based simulation tool named TunSim, for the conventional driven tunnel, and partly on a cost estimating function (both developed at NTNU) used for lining and rock support. Both simulation tools has been modified to better fit the local conditions [7].

1.3 Purpose

The purpose of this master thesis is to the extent possible, to highlight the economical feasibility of the project along with a more thorough assessment of certain components within this project.

Normally, hydropower consultants would perhaps kept the detailed design phase going a bit further to better see the optimal overall solution of the project. Furthermore, it is natural not to design all the elements to the smallest detail, but rather incorporate these costs as a percentage of the total cost. However, due to limitations in time and scope of this thesis, the decision to more thoroughly assess certain components that seem promising, is made in order to maximize the overall output for the client.

1.4 Objective

The objective of this master thesis is to give a preliminary suggestion of the most optimal solution of the different components in question.

The main focus will be:

- development of a cost basis
- the placement of the dam axis and optimization of volume
- selection of dam type
- optimization of waterway and placement of powerhouse
- simulation of production

1.5 Structure

There may often be several satisfactory design solutions for a hydropower project. However, an optimization of the project as a whole is often conducted in order to find the best solutions. To obtain this result is often done through an iterating process between all of the components in the project, making the road to the final result rather long.

In this thesis, it has been deemed necessary to limit the optimization to the components alone, in order to be able to later assess the chosen solution as a whole. This is done not to exceed this thesis scope.

As a result, this thesis may roughly be divided into the following sections:

1) Preliminary assessment of dam type

This section will on a general level assess and conclude with what is the most preferable dam type for this project. Furthermore, a calculation of parameters used in further assessments is established.

2) Feasibility study of the Asphaltic Core Embankment Dam (ACED)

A thorough assessment of the ACEDs feasibility is conducted along with a total cost estimation of the construction works of the dam.

3) Assessment of waterways and powerhouse

A preliminary optimization of the waterways is preformed, based on the established parameters, providing the values for the simulation of production. Assessment of the overall preferred layout is conducted, along with a total cost estimation of the waterways and powerhouse solution.

4) Simulation of production

Simulation of production is conducted based on the parameters derived from earlier works. Average annual production based on the input hydrology is estimated, along with the rate of utilization of the water.

5) Conclusion and presentation of the best overall solution

Finally, the results are analyzed, discussed and a conclusion is reached. The best overall solution for the components assessed in this project is presented.

1.6 Clarification of interfaces

In order to limit this thesis' extent, some interfaces needs to be established. First of all, this thesis will assess and use the same overall layout as proposed by SP. Furthermore, this thesis main focus will be:

- the construction and total cost of the dam
- headrace tunnel and power house
- simulation of production

As a consequence elements which forms part of the project is disregarded, such as:

- an upgrade of the road system
- construction of power lines and switchyard

Before an assessment can begin, the following pre-established parameters is introduced:

Dam

- Placement of the dam site is locked. Coordinates 14°37' South and 29°10' East
- Highest regulated water level (HRWL) and lowest regulated water level (LRWL) in Mkushi is 990 m.a.s.l and 987 m.a.s.l respectively

Chapter 2

Selected Approach

The purpose of this chapter is to describe the procedure used in order to give a valid assessment of the project. The method for gathering, and validation of information is described.

2.1 Gathering of information

Access to good information is key to solve the objective of this thesis. The information obtained to best assess this project is gathered through:

- literary reviews, with searches in databases (i.e. BIBSYS)
- literature recommended by supervisors and other people of importance
- search in the bibliography of gathered information

Supplemented methods such as correspondence with fellow students, conversations with people of importance and published articles from known institutions, has helped provide an adequate theoretical basis. For more details see Appendix B.

2.2 Analysis of the collected data

In general, the same volume of data pre-design phase cannot be expected in Zambia as in Norway. Furthermore, the present data for this project can be expected to be less precise compared to the similar projects conducted in Norway. The Hydrological basis in this project is provided based on incomplete gauge series. As a consequence hydrological scaling must be conducted, which introduces a level of uncertainty. In Norway, at least to some degree, a reliable data set without extensive scaling could be expected.

The field trip to Zambia revealed no indication of an institution comparable to The Geological Survey of Norway (NGU)[8]. NGU carries out mapping, map compilation and research on Norway's bedrock geology [9]. As a consequence, the same reliability in the geological basis cannot be expected as in Norway. Norway is furthermore a country with a solid basis of statistics such as e.g. cost indexes. This has not been identified in Zambia, making the overall transferability between Norway and Zambia somewhat complicated.

The geological data is based on site visits (gathering of rock samples and surface survey), a few borehole samples and laser scanning. Even though more extensive borehole testing (especially at the dam site) should be conducted, the preliminary basis is regarded as sufficient, but should be treated with some degree of uncertainty.

2.3 Design method

In contrast to Norway, Zambia has no evident rules or guidelines for designing hydropower projects. The design of hydropower projects relies to some degree on principles proposed by the International Commission on Large Dams (ICOLD) [8]. Consequently, the approach selected to best assess the project as a whole, is based on the Norwegian way of designing hydropower projects. Design criteria provided by NVE is therefore assumed in the design process of this thesis.

2.4 Development of cost basis

To best assess the overall costs of certain components in this thesis, a cost basis applicable to Zambia was made. Se table 2.1

The cost basis has been used to the extent possible. Certain calculations have however required more information than this cost basis can provide. Consequently the cost basis provided by NVE has been assumed. Even though this is based solely on projects in Norway, and as a result not necessarily is best applicable to Zambia, it is considered to provide the best alternative. However, this will introduce some uncertainty to the overall calculated results.

The cost basis proposed by NVE, highlights both the unit cost of the components in question, and provides the assumptions of which the unit cost is based. It is therefore, to some extent, applicable to base the assumptions made in this thesis on the basis provided by NVE, but with revised unit cost.

Table 2.1: Cost basis

Material	Price [NOK/USD]	Unit
Cement	1380/236	ton
Aggregates		
40 mm	173/30	ton
20 mm	202/35	ton
10 mm	219/38	ton
Sand	81/14	ton
Fine sand	69/12	ton
Concrete	950/162	m^3
Reinforcement		
High tensile bar	5853/1000	ton
Mild tensile bar	5268/900	ton
Mesh reinforcement	2341/400	ton
Formwork		
From plywood	470/80	m^2
Formwork timber	2630/450	m^3
Fuel		
Diesel	8,9/1,5	litre
Petrol	9,4/1,6	litre
Bitumen	6600/1150	ton
Labour	1000/170	pr month
Rigging and operation	25-30	%

Explanation of the development of the cost basis, please see Appendix C.

2.5 Economic assessment base

Revenue basis

In the energy market, the price of electricity will fluctuate. Producing power is therefore most profitable when the price is at its top. This however is not always possible to do. Not every power plant owner has the luxury of producing only at the peak of the electricity demand. In Zambia compared to Norway, there is a notable deficit of energy supply. It is therefore not possible to save the water at low energy price levels, and supply the energy demand from other energy sources, only to start producing at high energy price levels.

Given limited information of the fluctuating market prices in Zambia, the price of electricity is assumed to be 9 USc/kWh*.

Investment risk

Determining the interest rate has great impact on the projects economical feasibility. Eventually, it is the risk the investor is willing to take that influence the total decision. A higher interest rate returns the investment faster, but results in smaller dimensions. This results in a greater head loss and hence lower cash flow over time. In Norway a 7% interest rate is normally used (may be even lower), but developing countries with capital shortages normally is encouraged to use 10-12% interest rate [10].

2.6 Simulation of production

The program system nMAG is a model for simulating reservoir operation and power production in a hydropower system [11].

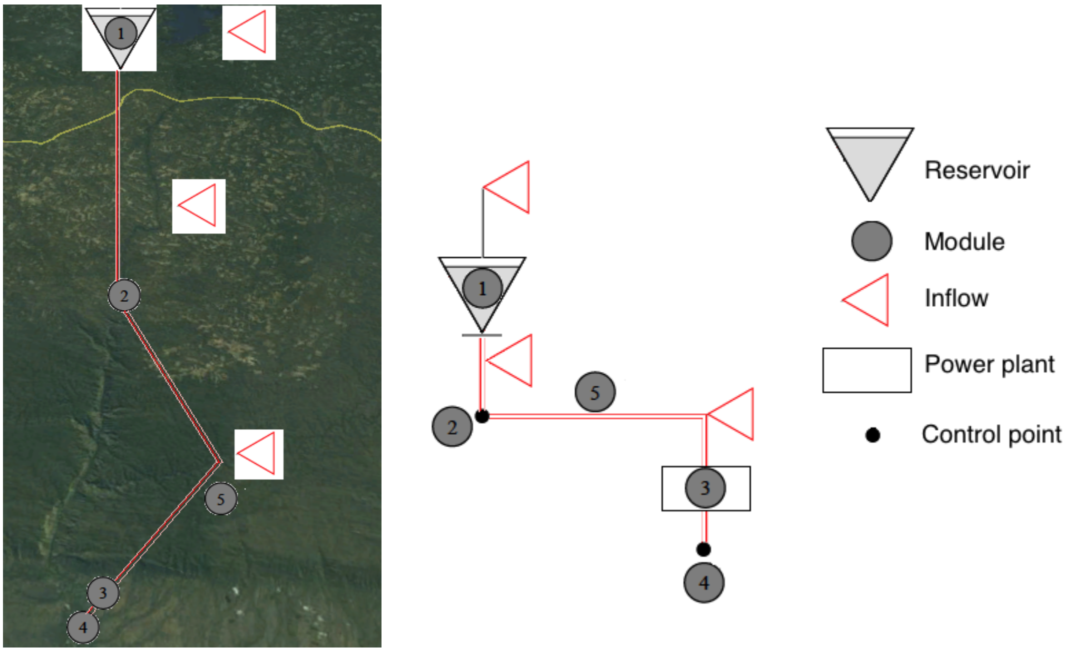
A hydropower system consists normally of the following components:

- Power plant
- Regulation reservoir
- Water transfer

In nMAG these components are referred to as modules. In order to function as a competent simulation model, nMAG has to both deal with these modules, and describe the connections/links between them. The links describe the way the water flows.

Figure 2.1 schematically illustrates how the modules in this project are connected together.

*Communicated by Lars Ødegaard March 2013.



(a) System illustrated on map [12] (b) System illustrated in nMag model

Figure 2.1: Illustration of waterflow

Each module is given a number. Entering the number of the next module in an address block connects the modules. There are three ways the water may be addressed:

- Released water
- Bypass water
- Spill water

Based on correct hydrological input parameters, nMAG should then be able to calculate the water available for production over a predetermined amount of time.

Time steps

Simulation is preformed by dividing each year into a number of time steps. Each time step is kept so small that both the inflow and the energy market may be considered to be constant within each time step. However the use of long time steps e.g a month, may lead to systematic underestimation of flood loss in the system and a systematic overestimation of energy production in the system [13].

Output

The relevant output produced in nMAG is:

- total power production based on a predefined demand of delivered firm power
- the percentage of the water utilized
- demand coverage
- reservoir operation

Chapter 3

Selection of Dam Type - Technical Economical Optimization

This chapter will try to clarify the best solutions, with emphasis on the technical and economical aspect, for the dam and associated solutions.

3.1 Project area

3.1.1 Cost of transportation

According to an article published by the World Bank, Meeuws (2004) concludes that the cost associated with transportation are very high compared to the price level in the region [14]. This results in huge costs associated with the import of materials that can not be obtained locally in Zambia.

3.1.2 Access

Kabwe to project area

From Kabwe and to the project area, there are two alternative routes, the northern and southern route. See figure 3.1.

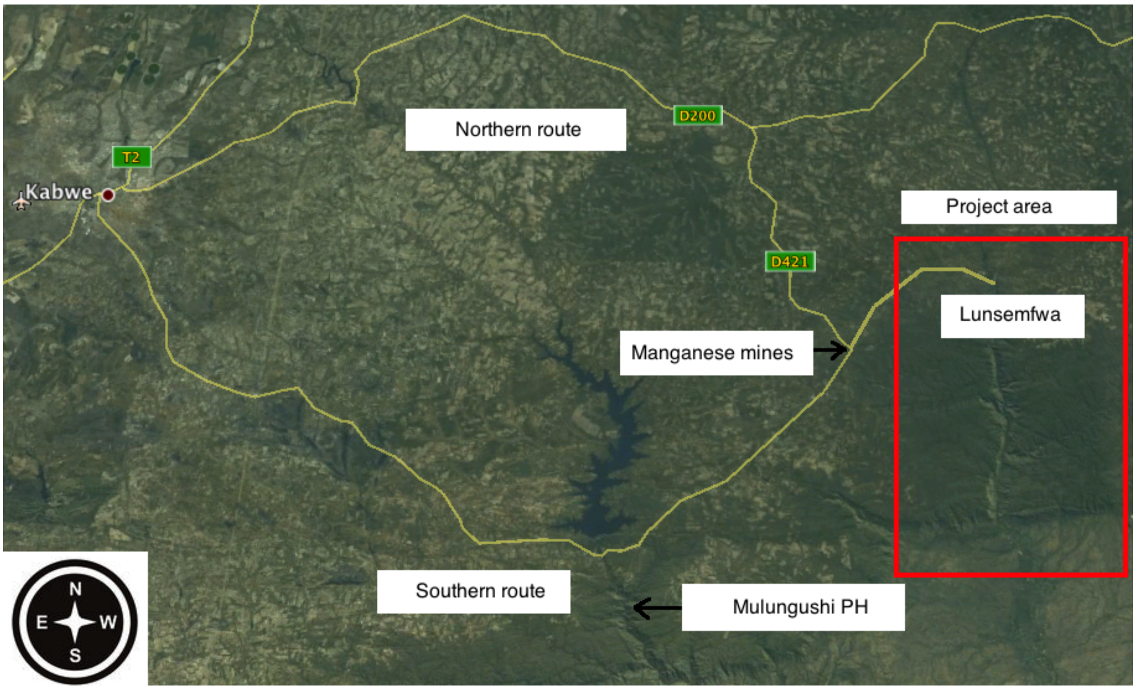


Figure 3.1: Access to the project area [3]

The two different roads are of fairly similar condition. Both are made of compacted earth fill, sand and gravel. Drainage systems does not exist, making the roads hard to travel during the rainy season, see figure 3.2. The roads definitely need to be upgraded to ensure access to the project area through the whole year. That being said, the roads to Lunsemfwa was used for construction purposes as late as 2012, when a Chinese contractor installed a fourth turbine in the powerhouse, though outside the rainy season.

Figure 3.2: Road conditions

It is currently an ongoing feasibility study for an upgrade of the Mulungushi powerhouse, see figure 3.1. Similar requirements for the road conditions associated with future construction works can contribute to synergy effects for both projects. The same argument may to some extent also interest the owners of the Chinese operated manganese mine just south of the project area.

Lunsemfwa to dam site

There are currently several possible routes to the dam site, see figure 3.3.

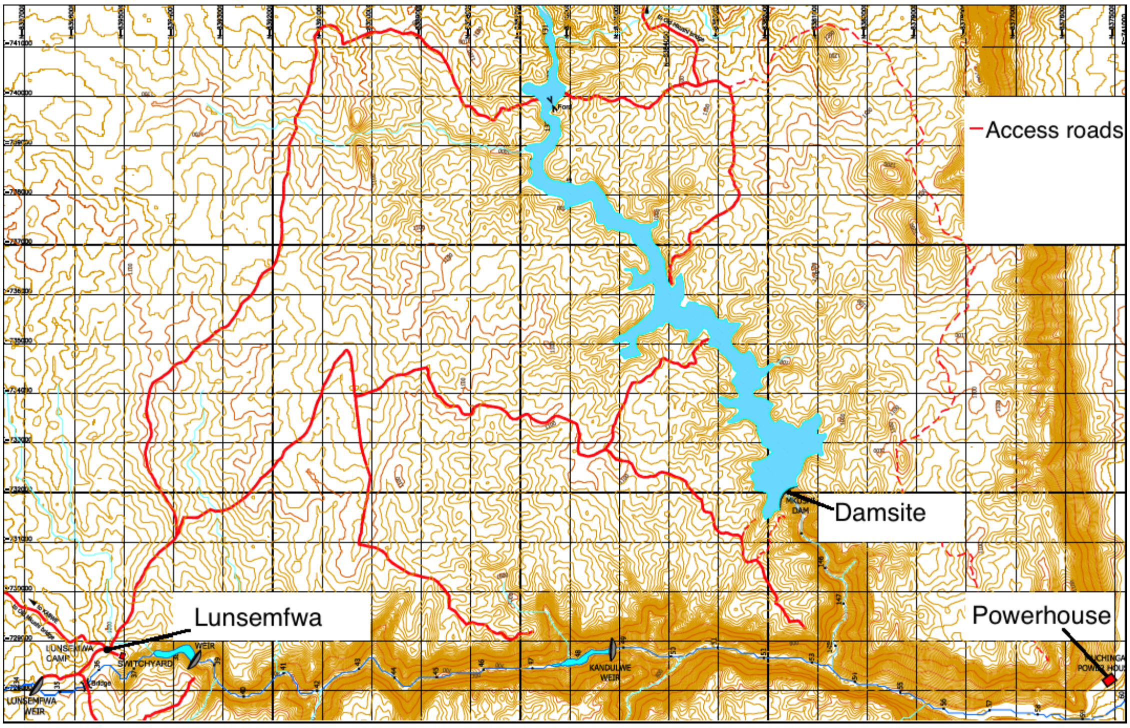


Figure 3.3: Access roads [4]

These routes are technically more equal to a path than to a road, so the need for upgrade is definitely present. Compared to the road conditions from Kabwe to Lunsemfwa, this stretch does not have the equal capacity to transport heavy equipment to the dam site, regardless of the season. See figure 3.4.

Figure 3.4: Road to dam site

Summarized, it is safe to assume that the road network will have to be upgraded to ensure transportation of heavy equipment to the project area throughout the whole year.

3.1.3 Availability of materials

With an upgrade of the road network, it should be no problem importing materials. Cement is produced locally in Lusaka or the Copperbelt, and the manufacturers guarantees delivery on cite. Rebar and explosives are also available due to the well-developed mining industry in the Copperbelt. Aggregates can either be produced locally at the dam site or be imported (maximum transportation distance 150 km from Kapiri M'poshi). As a result it is safe to assume a stable delivery at a cheap price for the most commonly used materials in the project. However as previously mentioned, this does not apply to materials that must be imported from outside of Zambia.

3.2 The purpose of the reservoir

The purpose of the future reservoir at Mkushi is not to save large amounts of water; it is simply to maintain the head for hydropower production. Furthermore, the argument of filling up the reservoir before the completion of the dam is finalized is neglected because of the fact that this is a small reservoir, which will be quick to fill up.

As mentioned in section 1.6, the height of the regulated water is supposed to be of only three meters. This in turn presents the fact that the varying load effects from the water results in lower impact compared to a reservoir where the reservoir was drained completely [15].

3.3 Dam site

3.3.1 Location of dam site

As previous mentioned, the placement of the dam site is locked to coordinates 14°37' South and 29°10' East, see figure 3.5.

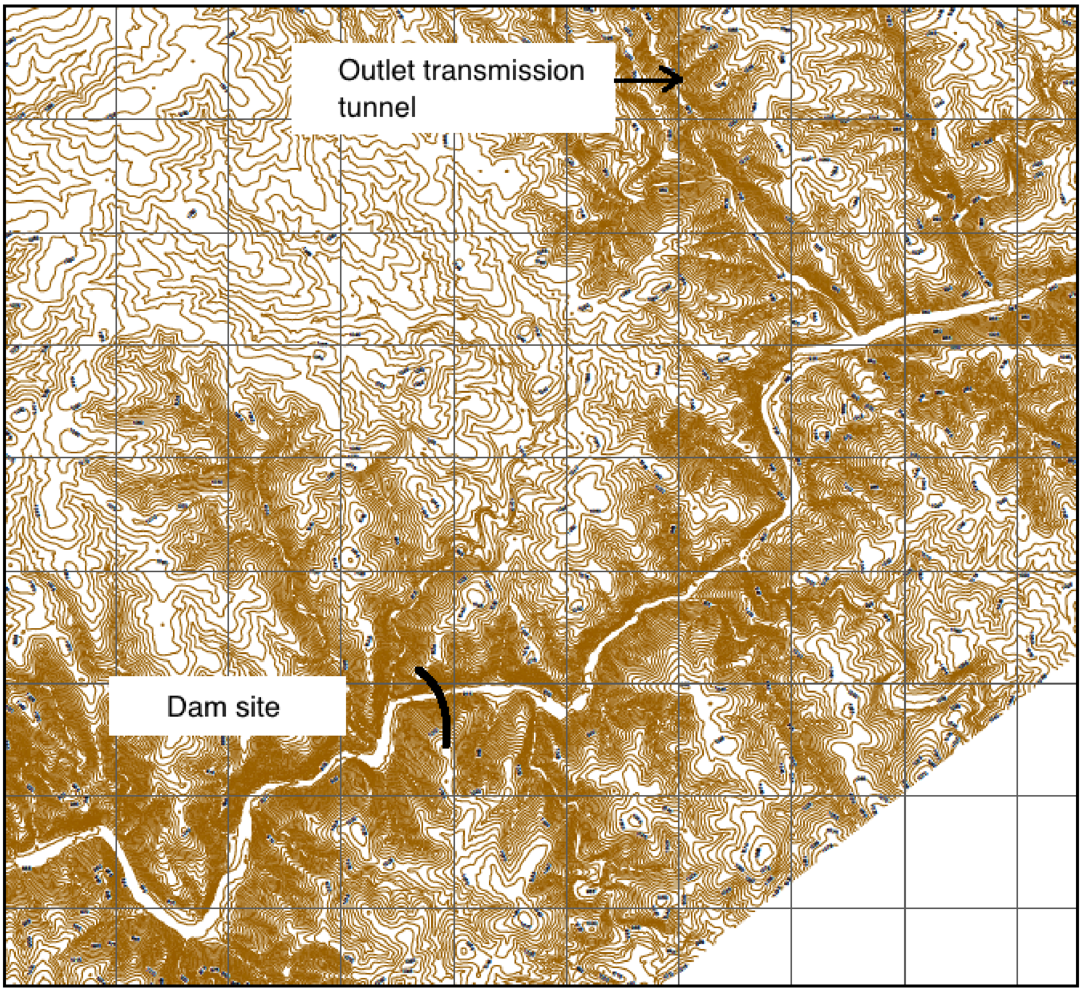


Figure 3.5: Location of dam site [4]

3.3.2 Optimization of placement

Normally an optimization between volume of reservoir and placement of the dam is considered. However in this case, this optimization will not be conducted since the dam site is locked to the aforementioned coordinates. Even though the placement itself has been decided, it should be conducted an optimization of the placement of the dam axis in order to save volume and hence reduce the total cost of the dam. In this thesis the following three sections was further examined to find the optimal placement. See figure 3.6.

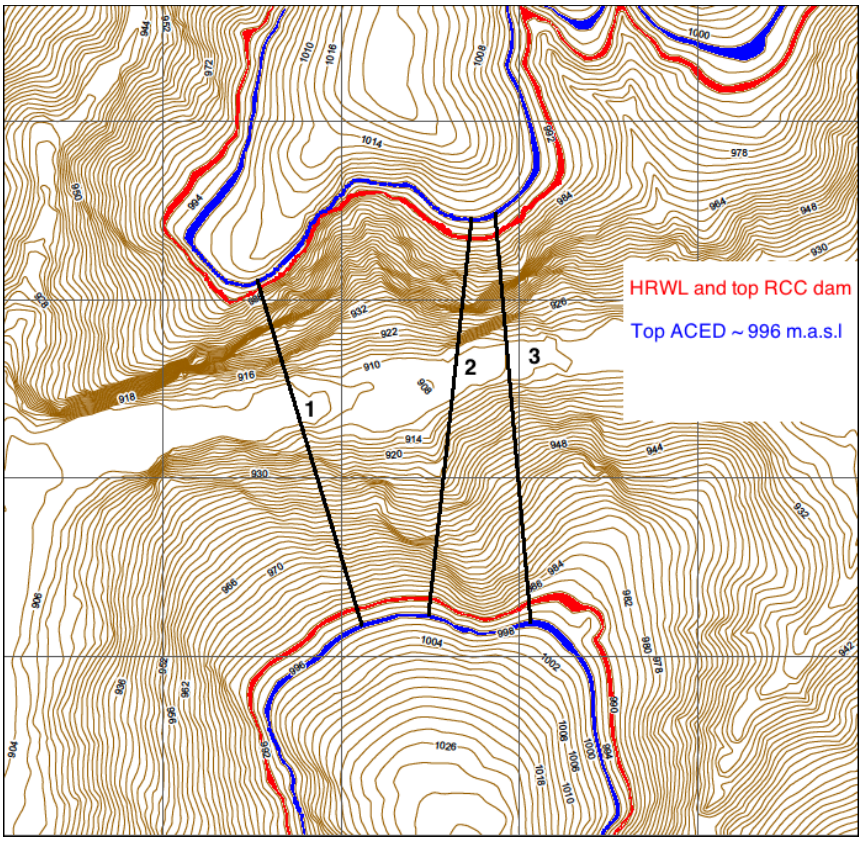


Figure 3.6: Optimization of placement [4]

The three sections presents the following simplified elevation profiles, see figure 3.7.

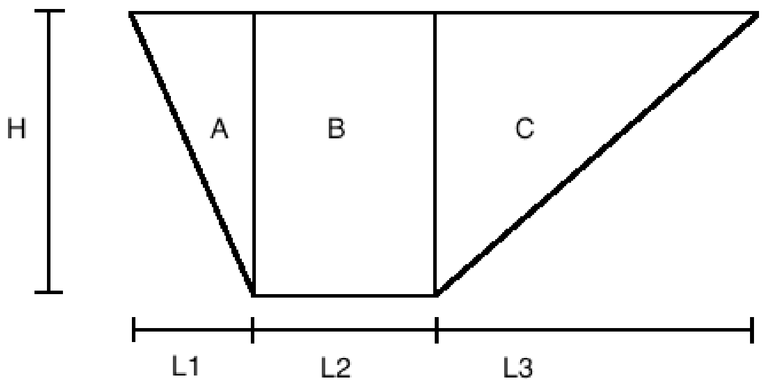


Figure 3.7: Elevation profile

Table 3.1: Optimization of dam placement

	Section 1	Section 2	Section 3
H	**92m**	**88m**	**86 m**
L1	50 m	71 m	71 m
L2	42 m	29 m	21 m
L3	105 m	117 m	133 m
Lentgh of damcrest	**197 m**	**217 m**	**225 m**
Surface	**11000 m^2**	**10800 m^2**	**10600 m^2**

As a simplification the abutments will be assumed to be constant equal to the section illustrated in figure 3.7. Based on this assumption it is section 3 that is the most preferable one, with the lowest surface area and hence provides the optimal volume.

3.4 Type of dam

It is of great interest to conduct a preliminary assessment of the dam type that is most preferable in this project. This is done to simplify the further work regarding a total assessment of the dam type.

A simple economical analysis of total costs of dams was conducted by ICOLD [16]. According to this analysis it is the average unit cost of labour, which is the one factor that varies much more than the others. Furthermore, the bulletin highlights the following facts:

- for a given dam type the cross-sections and therefore the quantities remain the same

- the total cost of supplies and equipment per m^3 of a given dam type is similar

Consequently, the two most competitive dam types is the SP proposed Roller Compact Concrete (RCC) dam and a mechanical placed embankment dam, see figure 3.8.

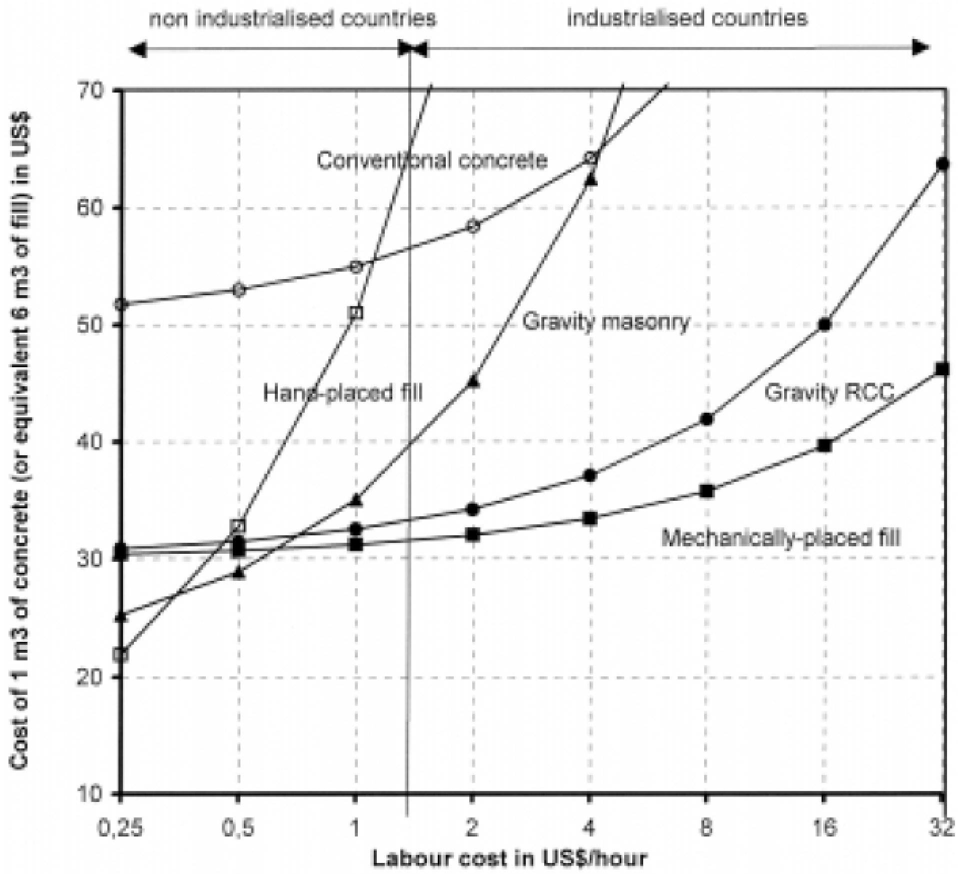


Figure 3.8: Labour cost vs. unit cost, competitive dam types [16]

3.4.1 Comparison of technical arguments

Based on experiences made during the field trip to Zambia, the following argument is being presented:

A preferable structure situated in this region should be sought to be made virtually maintenance-free.

Furthermore, the different advantages and disadvantages for each dam type is presented:

RCC dam

Advantages

- Low construction time, 2-4 m per week (6-12 months in total) [17]
- Integrated spillway solution
- Possible to implement energy dissipator directly into the body of the dam
- The tunnels made for bypass and diversion of water during construction is presumably shorter
- Limited use of rebar, joints and a low cement content contribute to a lower price, compared to other concrete dams

Disadvantages

- Bigger environmental impact (the use of concrete)

Asphaltic core embankment dam

Historically, for embankment dams, the material used as a core is often found in the vicinity of the dam site. Furthermore, if there are no available materials sufficient enough to preform as a core material, different solutions should be considered. A concrete faced rockfill dam (CFRD) has in many cases been chosen. In this case the argument that the sealing element is placed out in the open, and therefore will be subjected to temperature changes and weathering in general, combined with the fact that the structure should be made maintenance-free, excludes this option. Furthermore, since there are no suitable materials found in the vicinity of the dam site, it is to be assumed that an asphaltic core embankment dam (ACED), with a central core, will be a sufficient option.

Preliminary investigation of asphaltic core embankment dam (ACED) has revealed the following arguments [15][18]:

Advantages

- Virtually impermeable
- Earthquake resistant, hence no deterioration of properties
- The properties of the core can be tailored to best fit the local demands
- Self-sealing material. No permanent cracks
- The construction phase is fairly independent of weather conditions
- Core adjusts to dam foundation and deformations

- Method of construction is simple and robust (much simpler than a CFRD construction)
- No core erosion; therefore no strict filter criteria
- Can resist overtopping erosion during construction
- Approximately 12 months construction time, after diversion dam and tunnels are finished

Disadvantages

- Needs separate spillway solution

The fact that the ACED is robust enough to resist overtopping during construction suggests that the expenses associated with construction of the temporary diversion dam does not present a difference compared to the construction of the RCC- dam.

3.4.2 Roller Compact Concrete dam

Main economic components

According to preliminary cost estimations [19] conducted in this thesis predecessor, the key unit cost regarding the total cost of constructing a RCC- dam is the cost of concrete. To further evaluate the cost of concrete used in a RCC dam, it is essential to establish the unit prices of its components. Appendix D presents the following main economic components in the RCC dam, with the associated total cost of concrete:

Table 3.2: Unit cost of concrete in RCC

Cement	36 USD/m^3
Aggregates	52 USD/m^3
Pozzolan	68 USD/m^3
Total unit cost for concrete in RCC	**156 USD/m^3**

The use of Pozzolan is mainly because of the intent of reducing the total consumption of cement [20]. Reducing the amount of cement has two advantages, both economical and environmental. However, pozzolan is not produced locally in Zambia, which implies a rather high unit cost.

Preliminary economical estimation

Normally a RCC dam is constructed with a vertical side upstream and an inclination of approximately 1:0,8 on the downstream side [21]. At the same time, it could be of interest to utilize the top of the dam as a road, so that the dam construction itself works as a bridge. This will help improving the accessibility in the project area and also benefit the local community. A 5 m wide dam crest is assumed in further calculation.

Total volume of RCC dam is about 307.000 m^3, see Appendix D. Compared to the solution proposed by SP, this is a reduction of 400.000 m^3, eventually reducing the cost.

This results in the preliminary cost estimation of the RCC dam to be approximately 47,9 M USD.

3.4.3 Asphaltic Core Embankment Dam

Main economic components

The main cost contributors to assess for the total cost of the ACED option is as follows:

- Construction of dam
- Spillway solution

It is been assumed that the total cost associated with construction of bypass/diversion tunnel is fairly similar to that of the RCC dam. This assumption is based on the fact that it is possible to construct the diversion dam as part of the future dam for an ACED, see figure 4.7. For a RCC dam however, there is a need to build the diversion dam to make room for the future construction work. Since the RCC is approximately 80 m wide at the bottom, the diversion dam has to be adjusted to the required workspace. This results in the conclusion that the total length of bypass/diversion tunnel and hence cost is not a difference maker.

A preliminary assessment of the total cost of the ACED construction was conducted based on the cost basis presented by NVE [21].

Table 3.3: Preliminary unit cost of ACED

Core (Zone 1)	631 USD/m^3
Filter (Zone 2)	48 USD/m^3
Transition zone (Zone 3)	19/28 USD/m^3 *
Supporting fill (Zone 4)	9/15 USD/m^3 *

The design of slope protection is dimensioned by the criteria that it should be able to withstand the effect of waves, ice, snow etc. [22]. Due to the fact that the

*Dependent on whether or not crushed rock from tunneling can be utilized.

climatic conditions likely does not produce any snow or ice, this dimension criteria can be ignored. The materials used in the transition zone or the supporting fill comes either from a quarry in near vicinity to the dam site (Meta volcanics), or as tunnel excavated materials. Regardless, the geological properties of the proposed used rock, can be assumed to be of good quality [7]. This means that with the proper execution, with a bulldozer, the body of the dam should be sufficient to protect the dam [15]. The ACED can also, as previously mentioned withstand overtopping to some degree. Even though the effect of a wave encounter should be considered, the construction of slope protection is not taken into account. Another argument for why the slope protection is not taken into account, is the fact that there are no contractors with the skill level or the expertise to conduct this work properly in the region [15]. This in turn proposes the need of importing a competent contractor, which leads to an increase in total cost.

Proposed spillway solution

A potential spillway solution has been identified as illustrated in figure 3.9. This solution consists of an inlet basin, a spillway tunnel (inclination of about 1:9), and an energy dissipator.

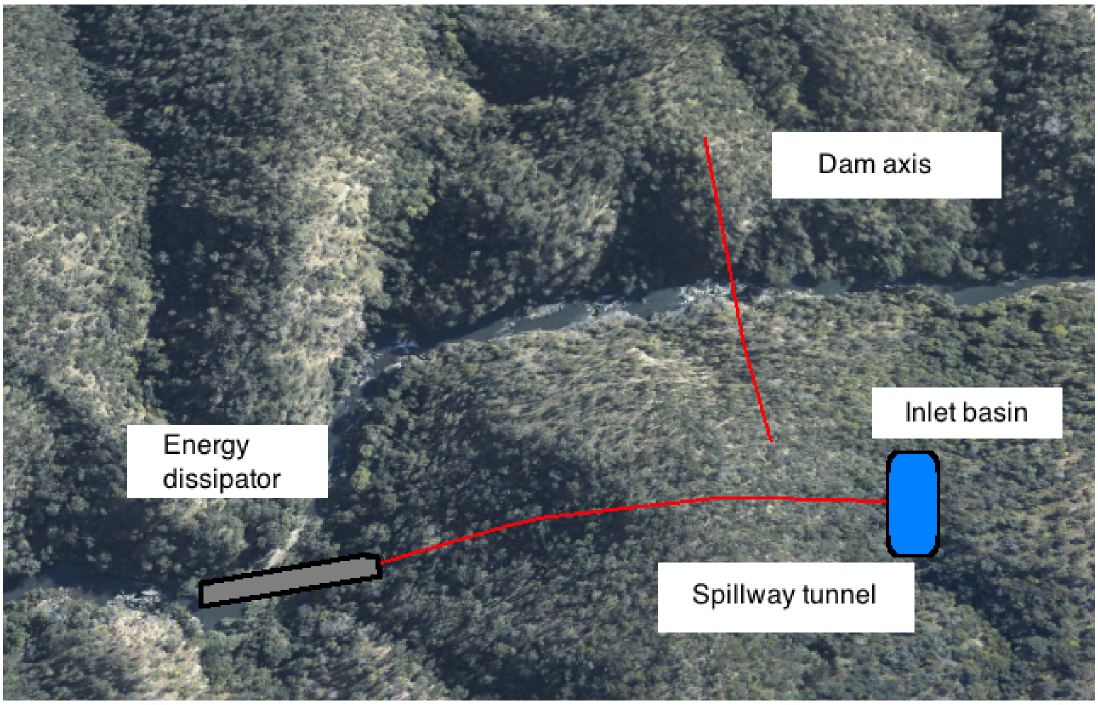


Figure 3.9: Proposed spillway solution [12]

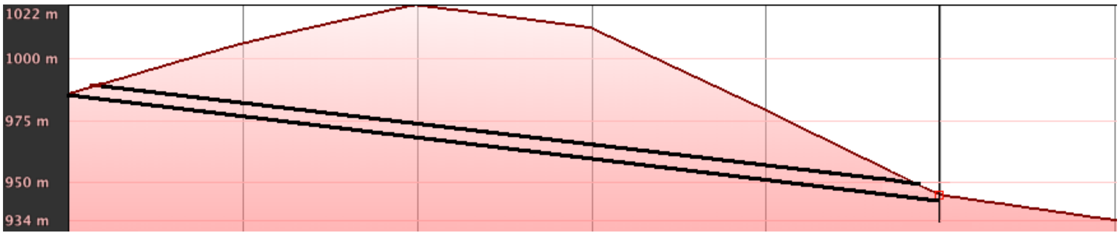


Figure 3.10: Elevation profile of proposed spillway tunnel [12]

The inclination of the spillway tunnel is within the normal inclination of tunnel construction (1:7)*. Furthermore, the inlet basin is thought to be excavated before the construction of the threshold for the spillway. At last, to complete the total construction of the spillway solution, an energy dissipator is needed. This identifies the following cost elements for the spillway solution:

- Excavation of intake basin
- Construction of energy dissipator and threshold for the spillway (concrete works)
- Construction of tunnel

The total cost for the spillway is estimated to be about 6,2 M USD. The design and cost estimation is discussed more thoroughly in Appendix F.

Preliminary economical estimation

The thickness of the core is reduced from 0,9 m (proposed by NVE [21]) to 0,65 m at the bottom and 0,5 m at the top [15]. This results in reduced total volume of the core. The reduction is essential because of the great costs associated with the construction of the core.

The core volume along with the calculated volumes for zone 2, 3 and 4 is presented in table 3.4 (see Appendix E for calculations):

Table 3.4: Volume ACED

Core (Zone 1)	5.440 m^3
Filter (Zone 2)	29.400 m^3
Transition zone (Zone 3)	64.070 m^3
Supporting fill (Zone 4)	1.016.000 m^3

*Communicated by Amund Bruland May 2013.

A preliminary cost estimation highlights the following total cost of constructing an ACED:

Table 3.5: Preliminary cost estimation ACED

Core (Zone 1)	3,4 M USD
Filter (Zone 2)	1,4 M USD
Transition zone (Zone 3)	1,8 M USD
Supporting fill (Zone 4)	15,0 M USD
Total cost	**21,6 M USD**

Consequently the total cost for the option of an ACED is preliminary calculated to be about 27,8 M USD. This is 20,1 M USD cheaper than the RCC option.

3.5 Summary and selection of further study

- ACED is by far the cheaper option
- Both dams will be conducted in a time span of approximately one year
- Possible greater environmental impact by construction of RCC
- ACED is virtually maintenance free, resulting in longer life expectancy

Pozzolan is not produced locally in Zambia, and the cost of import is regarded as very high. Consequently, the purpose of using pozzolan cease to exist. Without the use of pozzolan the amount of cement needed in concrete structures increases significantly, which eventually leads to non environmentally friendly structures. This presents the argument to reduce the total amount of concrete used in the project.

Based on the preliminary assessment, it is evident that the ACED option is the most preferable one. Consequently, a more thorough assessment of this option will be conducted.

Chapter 4

Asphaltic Core Embankment Dam - ACED

This chapter investigates the advantages of the preferred dam type, ACED, further. Cost analysis, construction method, requirements and feasibility is considered.

4.1 Why ACED?

As mentioned in 3.4.1 *Comparison of technical arguments*, the following technical arguments is presented:

- Virtually impermeable
- Earthquake resistant, hence no deterioration of properties
- The properties of the core can be tailored to best fit the local demands
- Self-sealing material. No permanent cracks
- The construction phase is fairly independent of weather conditions
- Core adjusts to dam foundation and deformations
- Method of construction is simple and robust (much simpler than a CFRD)
- No core erosion; therefore no strict filter criteria
- Can resist overtopping erosion during construction
- About 12 months construction time, after diversion dam and tunnels are finished

Virtual impermeable core

Years of experience in building ACED, and thorough field monitoring, have revealed an excellent performance record with negligent seepage or leakage through the core or even through the core-plinth interface [15]. This confirms the proof of being virtually impermeable. This experience is backed up by extensive laboratory testing, which proves that with an air void content of less than 3% (of total volume), the core is virtually impermeable (10^{-10} m/s) [23]. Because of this, there is no risk for core erosion which supports the fact that there is no need for a strict filter criteria.

The fact that the core is virtually impermeable makes the downstream side of the dam virtually dry. Dry material has more compressive strength than wet material, proposing the opportunity of constructing the downstream side of the dam with a steeper inclination than the upstream side. This proposes a reduction of the total volume of the dam.

Laboratory testing of asphalt concrete

For each new dam and site, tests are preformed to determine the optimal asphalt concrete mix. Quality control of materials, and mixing of the asphalt concrete mix is carried out in a field laboratory [24]. The asphalt concrete mix consists of:

- Aggregates 0-18mm
- Filler material 0-0,075 mm
- 6,5-7,5 % Bitumen (the grade available)

The goal of laboratory testing is to achieve a flexible core with low permeability and ductile stress-strain behavior with the required strength [18].

Designing the asphalt concrete mix is done, by using Fuller's grain size curve. A theoretical curve used to minimize the void in the mix. In reality the asphalt mix deviates a bit from Fuller's grain size curve in the lower parts to make room for the binder (Bitumen) and the filler, to obtain the preferred properties [15]. With a Bitumen content of approximately 6,5-7,5 % the mix is easy to place and compact to the required air void content (3%) [23].

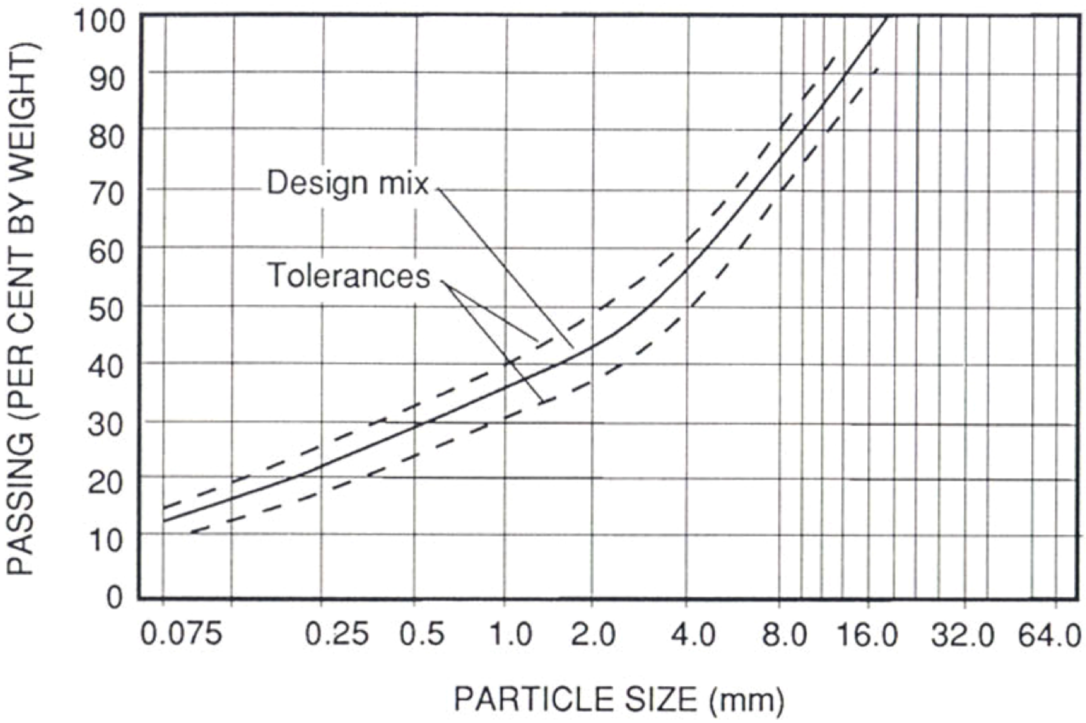


Figure 4.1: Fuller's grain size curve [18]

The use of Bitumen results in the fact that the mix acts as a low viscosity liquid, easy to form, and in need of immediate side support during construction. The construction of the core is therefore a critical face in the production of the dam.

Self-sealing material

The core is constantly subjected to shear stresses and tension stresses caused by loads, imposed displacement or in some regions rapid temperature reduction. As a result fissures and cracks may occur. Because the core acts as a low viscosity liquid, it has the ability to close the cracks and fissures by itself, hence the argument of the self-sealing properties.

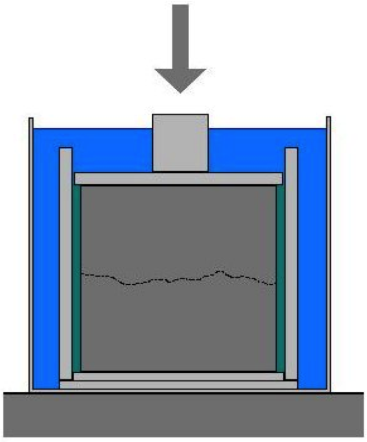


Figure 4.2: Regain of tensile strength and sealing of crack [18]

A lot of testing has been conducted on the matter. And the results show that it is possible to regain the tensile strength when subjected to compressive stress. The principle is illustrated in figure 4.2. Since the core always is subjected to compressive stress (it increases with depth), it will always have the possibility to regain its tensile strength and seal of potential cracks. It only becomes a matter of how fast this process is being preformed. How fast this process is carried out has also been investigated.

Several independent investigations have been conducted. The procedure of the experiment with some associated results is presented below.

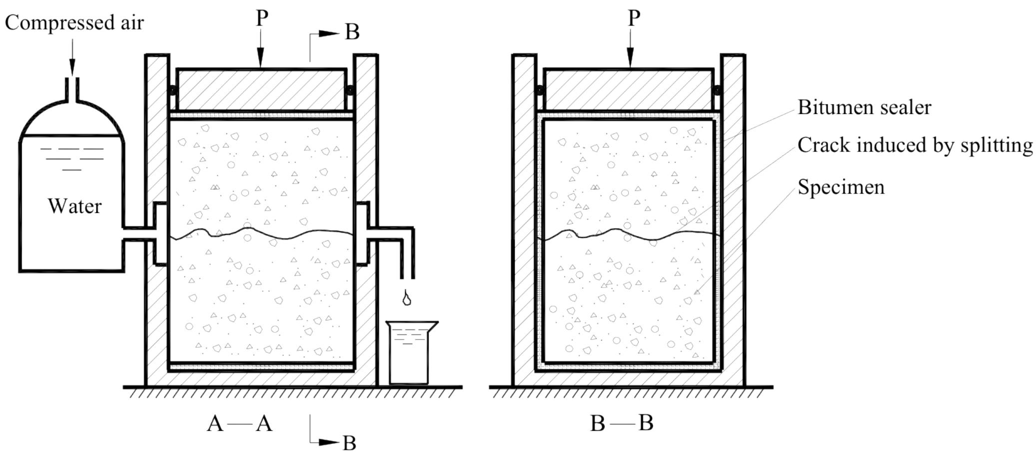


Figure 4.3: Investigation of self healing of crack [18]

Figure 4.3 illustrates a specimen with a crack. The crack self-healing is evaluated by measuring the rate of the water leakage through the crack as a function of stress level, temperature and time. The results of the experiment in question is presented in figure 4.4.

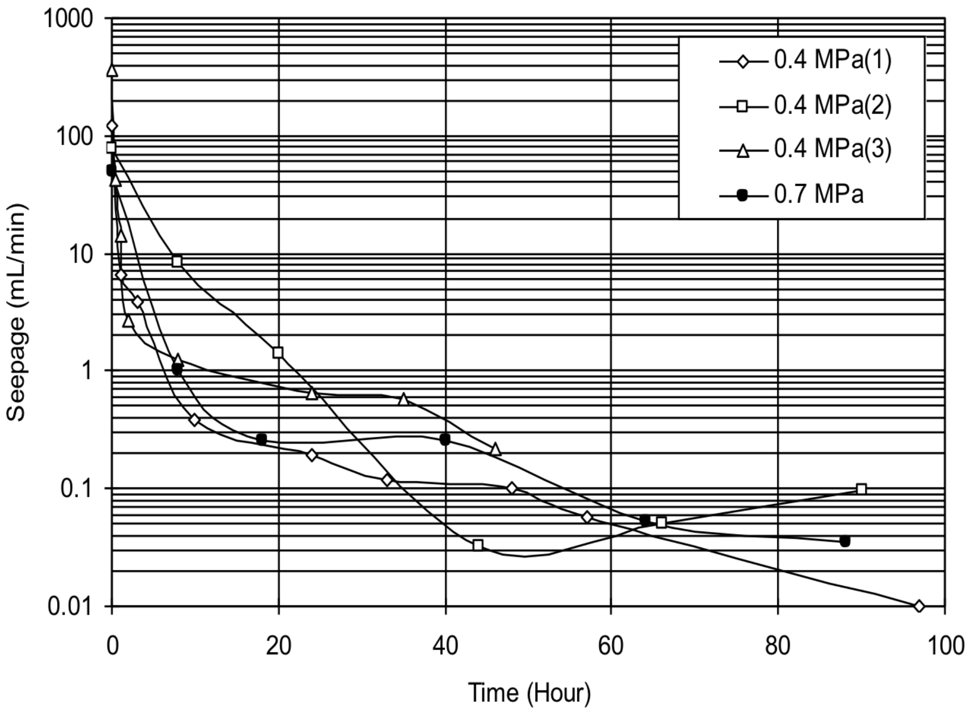


Figure 4.4: Self-healing vs. time [18]

As can be read from figure 4.4, in approximately 20 hours or less, the seepage ratio has dropped two orders of magnitude, and the seepage ratio continue to drop. Independent studies have concluded that the average regained tensile strength of the specimens is approximately 55 % after only 24 hours [24]. These results highlight the fact that the asphaltic concrete mix has self-sealing properties.

Adjustability

The ACED has become a very attractive option in many projects. As a result, there are continuous development and monitoring of performance of the ACED. Experience from other projects (e.g. Eberlaste dam and Feistritzbach dam in Austria) highlights the fact that the ACED has an excellent record of adjusting to large differential settlements, tension stresses and shear strains without cracking [24]. This illustrates the material ductility and even its self-sealing ability further.

Figure 4.5: Ductility testing of the asphaltic core

If for some reason, large deformations because of limited foundation abilities could be suspected, it is easy to alter the concrete mix to better fit the future deformation. A softer grade Bitumen and a higher Bitumen content (e.g. above 7%) could be sufficient.

Overtopping

An ACED will to some degree have the ability to resist overtopping. After the asphaltic core has cured, it will be able to stand as an individual construction, should the event of an overtopping occur. The surrounding masses will of course run the risk of being washed away, but the core should stand. After the construction area is drained after the flood, normal construction may commence after refilling of the surrounding masses.

Construction Speed

Constructing the core, historically, two layers of 20 cm compacted thickness has been placed on average per day, totaling 40 cm/day. However, if required, it is possible to use significantly higher rates of construction speed than previously practiced [24]. Several independent experiments have been conducted, with the result presented by Helge Saxegaard in 2002 as the most astonishing. He built a full scale dam with 6.7 % Bitumen (type B180). In two different cases he was able to place four layers of 20 cm and three layers of 30 cm, totaling 80 and 90 cm/day respectively. On top of this remarkable result, the quality of the core was not diminished in any way.

The fact that an ACED can be constructed as fast as 80-90 cm/day, makes it less vulnerable to the consumption of time. Resulting in an extremely reliable and predictable option in regards to the fact that time might often be one of the biggest cost contributors.

4.2 Requirements

The construction of core and filter (zone 1 and 2) as well as the construction of the concrete plinth, has to be constructed by experts [15]. To even consider building an ACED, this fact needs to be highlighted. It is crucial for the total quality of the construction works. If the construction of the two zones is not done according to strict specifications, with daily quality control, the core can no longer be guaranteed to be virtually impermeable. Should this property lapse; the probability of core erosion would increases considerably. This is because the design proposes no strict filter criteria. However, the requirement that there has to be a specialized contractor to construct zone 1 and 2, proposes several new advantages. First of all it ensures the aforementioned arguments. Second, compared to a regular embankment dam the quality of the adjacent zones does not need to be of equal quality. This results in the fact that the business involving crushing and sorting of rock at the dam site does not have to be as extensive. Furthermore, the construction of the remaining zones can be conducted by local contractors, who should possess the advantage of utilizing the local rates for labour, without any extensive risk.

4.3 Method of construction

The description of the method of construction will mainly revolve around the construction of the core and the immediate side support.

4.3.1 Normal sequence for construction

The method of construction is normally conducted in the following sequence [15]:

1. Construction of bypass/diversion tunnels
2. Construction of diversion dam
3. Preparation of foundation
4. Construction of concrete plinth, with deep grouting and interface grouting (between rock and concrete)
5. Construction of the dam

The sequence of construction is illustrated in figure 4.6

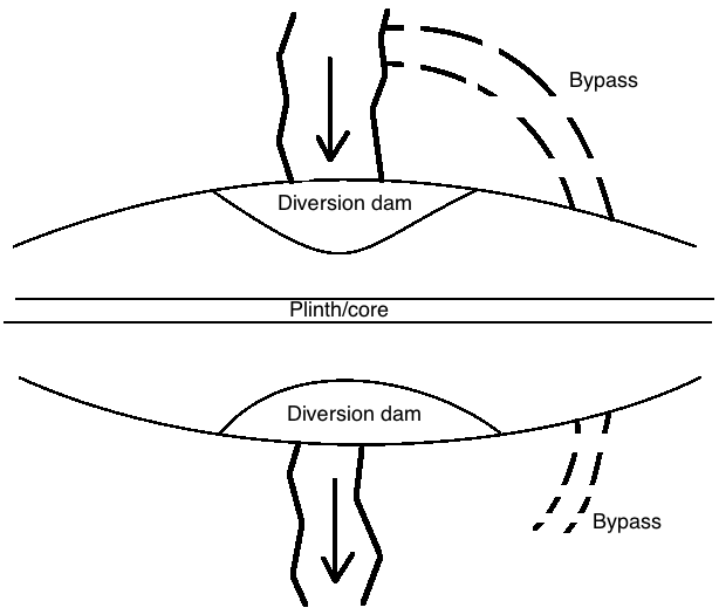


Figure 4.6: Sequence of construction of ACED

The reason the diversion dam is built is because the need of keeping the construction area dry. However, the construction of the diversion dam does not have to impose further costs compared to building an RCC dam. Upon this construction, it is possible to simply construct it as a future part of the remaining dam, see figure 4.7. The only extra cost is associated with the implementation of a impervious PVC-membrane.

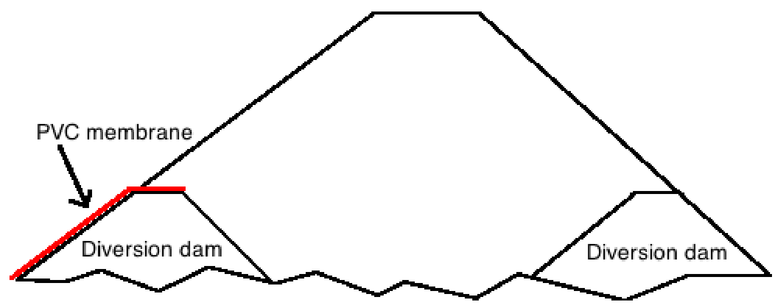


Figure 4.7: Construction of diversion dam with PVC-membrane

4.3.2 Concrete plinth, core and side support

Construction of the core normally requires 5 people. Of these 5, only three will have to be skilled [15]. Normally, before the construction of the dam itself commences, a trial section is placed in near vicinity. This proposes several advantages:

- Education of local labour
- Testing of asphalt mix

As earlier mentioned, competent contractors must conduct the construction of the core and filter zone. Before construction of the core commences however, preparation of the foundation and construction of the concrete plinth must be conducted in order to make sure to minimize the seepage through the ground.

Construction of concrete plinth

As a normal concrete structure the concrete plinth is reinforced with rebar, and anchored to the ground. After the concrete plinth has completed its curing process, both deep grouting (often 50-60 meters depending on foundation properties [15]) and interface grouting between the plinth and rock is completed, see fig 4.8. In addition, holes for hydraulic jacking are installed to simulate the future hydraulic pressure.

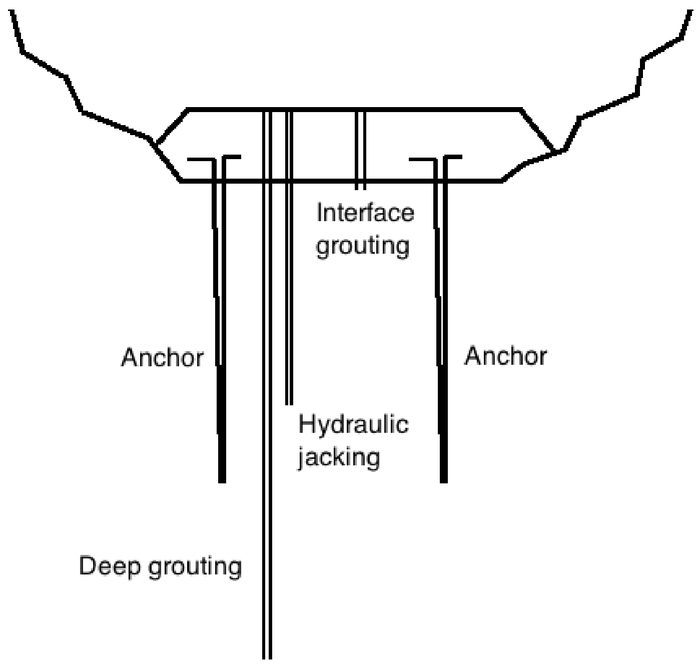


Figure 4.8: Concrete plinth

Before the construction of the core and immediate side support can begin, a thin layer of mastic is placed on top of the concrete plinth, in order to create good adhesion between the two.

Construction of core

Once the mastic has been placed, the first layer of core may be completed. The equipment used for such a placement is shown in figure 4.9. Furthermore the principle behind the placement process is illustrated in figure 4.10.

Figure 4.9: Asphalt core paver

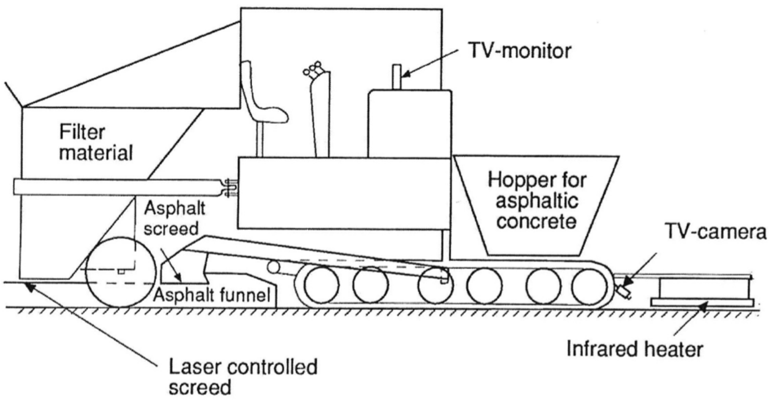


Figure 4.10: Principle behind asphalt core paver [18]

The asphalt core paver places three layers of materials simultaneously. One layer of asphalt and two layers of side support, see figure 4.11. The asphalt is transported from a local established asphalt production plant, while the side support may be materials from either excavation of tunnel or a nearby quarry.

Figure 4.11: Placement of three layers simultaneously

Immediately after placement, rollers conduct simultaneous compaction of the layers. See figure 4.12. Normally the compaction is finalized when the binder starts to pile up at the top [15].

Figure 4.12: Simultaneous compaction of core and filters

Even though machine placement can be done throughout most of the stretch, some hand placement will always be required at the abutments. As the dam construction progresses, the various zones is raised simultaneously. To avoid any accumulation of water, it is advised to always keep the core and filter level above the level of the adjacent zones [23].

4.4 Cost of construction

This section will highlight the applicability of the masses from tunnel construction and the cost of the construction of the core and filter. The work has been conducted in close cooperation with Helge Saxegaard.

4.4.1 Identification of expenses

The different expenses in regards to constructing the core and filter is identified, and presented below [15]:

1. Establishment of asphalt plant on site
2. Establishment of asphalt laboratory
3. Asphalt production on site
4. Running of asphalt laboratory
5. Production and transportation of transition zone material to the dam site
6. Placement of asphalt
7. Administrative services

Furthermore, in order to complete a total assessment of the cost of constructing the core and filter (zone 1 and 2), the following input parameters had to be established:

- Price of Bitumen available
- Price of cement
- Labour cost
- Price of diesel
- Total construction time

For input values, please see the cost basis in chapter 2.1 .

4.4.2 Cost of core and filter

As previously stated the approximate construction time is assumed to be 12 months. Based on the input parameters the total price for constructing the core and filter is calculated to be **8,4 M USD**. See Appendix G for calculations.

The thorough assessment of the construction of core and filter, is 3,6 M USD more expensive than preliminary calculated based on the cost basis provided by NVE. This assessment however should be considered to be much more reliable.

This results in a new revised total cost of the dam solution equal to 31,4 M USD.

4.4.3 Applicability of masses from tunneling

The supporting fill and the transition zones require rock of a decent mechanical stability. Schist, may be utilized unless it is considered to be decayed [15]. The geological report from SP point out, that the quality of the schist is of a variable quality. However, the preliminary borehole samples indicate that the possibility of using unweathered schist may be utilized as materials for the dam. As a consequence 50 % of the schist excavated from tunneling is assumed usable. This result in the total volume of applicable masses from tunneling, to be approximately 700.000 m^3 [7].

Even though it has not been assessed if the masses from tunneling is any cheaper than the prices proposed by the cost basis, see section 2.1, it is anticipated to propose a reduction of the unit cost.

4.5 Overall assessment of the ACED option

The selection of the preferred type of dam relies on a total assessment of the preferences specified for the particular project. In this project it is the dam, with the associated bypass solution (both during and after construction), spillway solution and energy dissipator which roughly constitutes the basis for selection. The assessment of the feasibility and total cost of the dam is thoroughly conducted. However, the assessment of the spillway solution should be treated as a suggestion only. There are some uncertainties regarding the feasibility of the proposed solution, which should be investigated further. The same applies for the economical assessment of the spillway. This is roughly based on the cost basis provided by NVE, with a minimum of adjustment to better fit the cost parameters applicable in Zambia.

Even though the level of accuracy in obtained cost estimations is somewhat inconsistent, the main cost component (i.e. the dam) is thoroughly assessed. A cost difference of about 16,5 M USD means that the total cost of the ACED needs to increase with 53%, in order to cost the same as a RCC dam. Even though the cost of a spillway and a bypass solution is expected to deviate from the calculations made in this thesis, an increase of 16,5 M USD is considered to be unlikely. Consequently, the ACED is still regarded as the best option for selection of dam type in this project.

Chapter 5

Waterway and Powerhouse - Technical Economical Optimization

This chapter will assess two proposed layouts of the waterways and powerhouse, eventually selecting the overall layout used for simulation of production.

5.1 Potential schemes of waterway and powerhouse

To assess a potential solution of the waterway and powerhouse, two different options are presented. Through the *Investigation Plan and Tech. Specifications* from 2012, SP proposed two layouts with similar headrace tunnel, but different powerhouse solution. See Appendix H for overall layout of headrace tunnel. Figure 5.1 illustrates the following principle layout, option A and option B [25]:

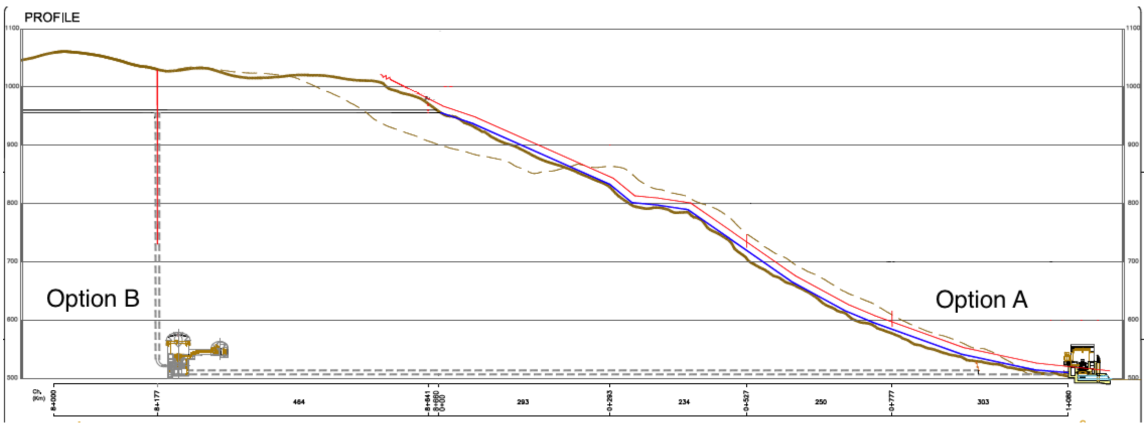


Figure 5.1: Option A and Option B [25]

Option A has a surface powerhouse right by the river and Option B has a underground powerhouse with an associated tailrace tunnel.

In this project, there may be several preferable layout schemes, both with different total length for the waterway and potential head for hydropower production. Normally a more thorough assessment of several of these schemes should be conducted. This master thesis will only be assessing the two proposed SP schemes, option A and B. However, should a more thorough assessment be conducted in the future, it is worth to mention that it exist a game reserve south of the escarpment, that may constraint the placement of the powerhouse.

This thesis does not consider an option of an unlined high-pressure tunnel with a closed chamber with an air cushion. This layout has grown more popular in Norway since 1975 [26], but due to e.g. the geological properties along the waterway, this is considered to be a too comprehensive option [7]. Jentoftsen (2013) gives a more thorough assessment and justification of why this layout has been disregarded.

5.2 Basis for comparison

In designing and estimating the solutions for the waterway and powerhouse, the following main components has been identified:

- Waterway - Head/Tailrace tunnel. Penstock/pressure shaft
- Powerhouse, surface/underground with associated access tunnels
- (Electrotechnical installations)
- (Mechanical installations)

The purpose for this assessment is to analyze which of the two options is the most preferable one, and hence simplifications have been made. Certain cost elements have not been taken into account (e.g. power supply during construction, new power lines, intake solution etc.), regarding comparison of the two options. It is to be assumed that the cost of the mechanical and electrotechnical installations is fairly similar and will not be assessed when comparing the two options.

5.2.1 Selection of type and number of turbines

With a gross head of approximately 490 m, it becomes apparent that the decision stands between a Pelton and a Francis turbine [27]. However, further assessments have to be conducted.

Number of turbines

A stop in production is crucial and may cause significant loss of income to the owner. The power plant is therefore vulnerable to the installation of only one aggregate. Installation of two aggregates however, minimizes the probability of a total breakdown (and so on for the further installations). Installation of several aggregates, will at the same time propose an opportunity to run the power plant at a better efficiency. Even though installation of multiple aggregates ensures the safety of production, it will also increase the total cost. It is natural therefore to assess this even further in later phases of the project. For further assessments, installation of 3 aggregates is assumed.

Comparison of turbines

Francis

- Cheaper than Pelton
- Runs on higher speeds than Pelton, resulting in a cheaper solution for generators
- Higher peak efficiency than Pelton, see figure 5.2
- Lower efficiency at lower outputs
- Runs down to 30 % of max capacity

Pelton

- Flat efficiency curve. Runs better at lower outputs
- Possible to run it down to 5 % of max capacity
- Easy accessible for maintenance

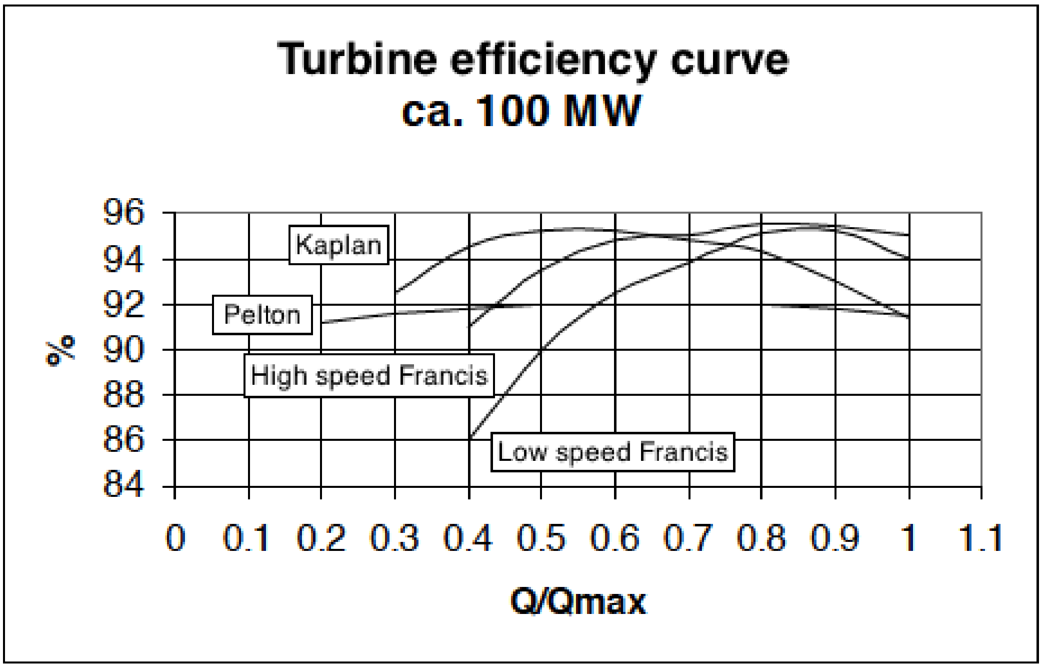


Figure 5.2: Typical turbine efficiency curves [21]

This project is characterized by its large gross head, and considerably large discharge (assumed max turbine discharge equal to 57 m^3/s). Furthermore, the fact that Mita Hills reservoir is of a considerable size (live capacity of 658 M m^3 [28]) implies a smooth regulation of the discharge. Figure J.1 in Appendix J illustrates the running of the power plant. The running is to be conducted at a high relative discharge for most of the time. This suggests the use of a Francis turbine. The Francis turbine is both cheaper and has higher peak efficiency than a Pelton turbine. Installation of more than one turbine ensures higher peak efficiency compared to relative discharge, making the Francis turbine the preferable choice.

5.2.2 Placement of powerhouse - technical arguments

The layout of this project is as mentioned characterized by a long headrace tunnel. This introduces a great flexibility of placement of the underground powerhouse. Consequently a more thorough survey of the final placement of the powerhouse, in regards of the geological aspects, should be conducted. However, the current placement of option B will be assumed in the following. This places it in the paragneiss zone, providing it with good geological properties. See figure 1.5.

Edvardsson and Broch (2002) states that an underground solution in many cases are preferable [26]. In principle, it is the steel parts that are the most costly sections of the headrace. A solution of a powerhouse out in the open implies longer steel parts. Furthermore, an optimal design gives a higher specific loss in steel parts than the waterway in general. This in turn (if equal optimization criteria is applied), results in the fact that an underground solution has a higher total plant efficiency factor [26].

Other technical arguments for choosing the powerhouse solution is as follows [26]:

Safety reasons An underground powerhouse present a more reliable option considering protection against sabotage

Structural design A surface powerhouse require more concrete to achieve appropriate safety against sliding or uplift failure. Compared to the underground powerhouse who is fixed in its cavern

Operational reliability Risk of accidents and their consequence require special attention in an underground power plant

Maintenance Outdoor structures is exposed to climatic variations, hence subjected to deteriorating effects. For penstocks this implies maintenance of:

1. external corrosion coating
2. expansion joints
3. erosion protection

Overall design An underground solution proposes unlined high-pressure tunnels and shafts, which reduce the cost of tunneling [29].

In Norway, a general guideline is to minimize the visual impact of the construction of the powerhouse, hence the construction of the underground powerhouse. In Zambia however, the construction of a surface powerhouse is regarded as a visual sign of industrial growth.

5.3 Option A. The SP proposed solution

Option A corresponds to the overall solution presented in SPs conceptual schemes report.

Layout characteristics

To best review the options proposed by SP, an assessment of the total cost and potential head loss should be conducted. This is done to create a basis for comparison with option B.

Option A has the following characteristics:

- Length of headrace tunnel 8650 m
- Gross conventional driven area 29 m^2 with lining (A = 19,6 m^2)
- Length of penstock approximately 1300 m
- Diameter 2,2 m
- 3 penstocks
- Powerhouse at 500 m.a.s.l

Cost estimation of option A

The cost function provided by Jentoftsen (2013) highlights the cost of tunneling (Option A) pr meter to be approximately 5.160 USD, included the lining [7]. As a result the cost of tunneling should be 44,6 M USD.

The price functions provided by NVE introduce large uncertainties regarding the construction of penstocks and a surface powerhouse. The cost basis (NVE) introduces a +60% to -40% error of margin of the total cost estimations of the penstocks. Furthermore, the cost estimations for a surface powerhouse is conducted based on an inconsistent base of experience, as well as being adjusted for a fairly low hydraulic head compared to this project [21]. Consequently, in order to make a fair assessment of option A, input parameters regarding total cost of the penstock and surface powerhouse have been obtained from Mott MacDonald. Which in this case is regarded as having a better basis for assessment.

- Penstock cost: 42 M USD [30]
- Civil works of surface powerhouse: 12,3 M USD [30]

The cost of option A, which provides a basis for comparison, is estimated to be 98,9 M USD.

Calculation of head loss

The total calculated head loss for option A is 35,1 m. See Appendix I for calculations.

5.4 Option B

Layout characteristics

The layout of Option B has the following characteristics:

- Length of tunnel approximately 8200 m
- Tunnel cross section not dimensioned
- Length of vertical pressure shaft 455 m
- Underground powerhouse, outlet at 500 m.a.s.l
- Length tailrace tunnel 1540 m

In order to find the proper dimensions of the waterway, an optimization, with respect of minimizing the head loss, should be conducted.

5.4.1 Optimization of option B

Life expectancy and discount rate has to be determined before the optimization. Even though the discount rate for projects conducted in developing countries normally is about 10-12%, a discount rate of 15 % is assumed. Life expectancy is set to be 30 years.*

Optimization of head- and tailrace tunnel

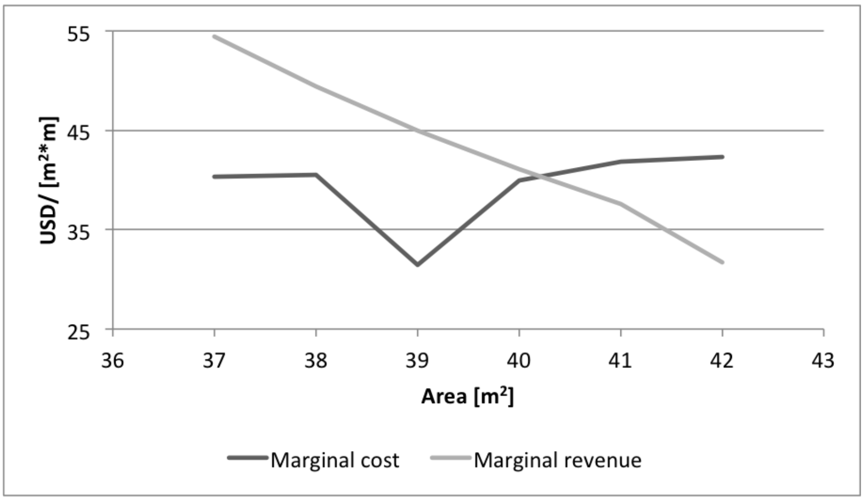


Figure 5.3: Optimal area of headrace tunnel

*Communicated by Lars Ødegård. May 2013.

In order to ensure adequate capacity, equal cross section of the head- and tailrace tunnel is assumed. This yields as figure 5.3 illustrates an optimal area of approximately 40 m^2. See Appendix J for calculations.

An area of 40 m^2 cost about 3010 USD/m of tunnel. This brings the total cost of head- and tailrace tunnel to be 29,3 M USD.

Sensitivity of input

Choosing the different input parameters may have great impact on the output of the optimization. A short analysis of the following input parameters is given below:

- Life expectancy
- Discount rate
- The price of electricity

As figure 5.4 illustrates, a change in the assumption of the life expectancy does not yield a difference in optimal area.

Furthermore, figure 5.4 also illustrates the sensitivity in the change of price of electricity and discount rate. It is apparent that optimal area absolutely is dependent on assumed discount rate and changing electricity price, spanning from about 38 m^2 to almost 42 m^2. A similar sensitivity will naturally apply to the optimization of the pressure shaft, being conducted by the same approach.

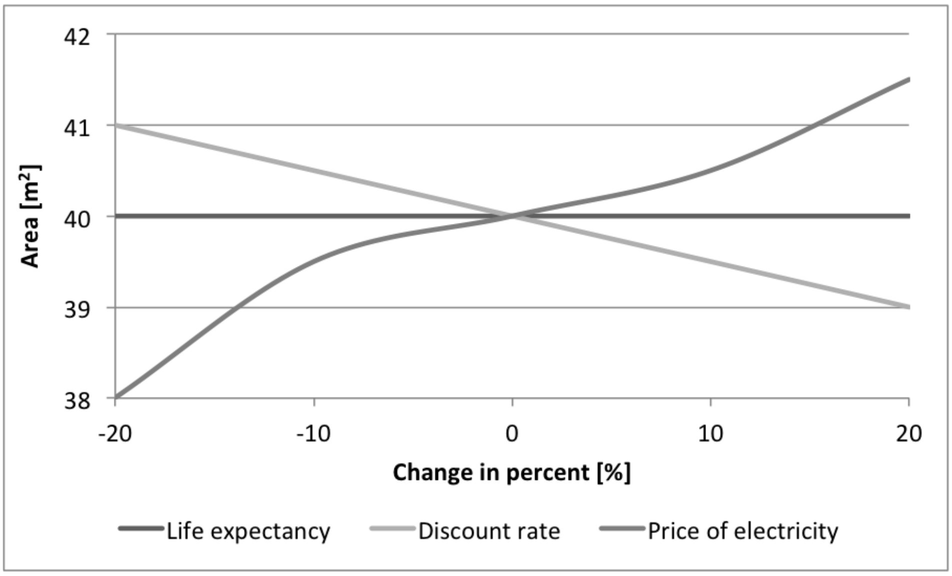


Figure 5.4: Sensitivity of input

Optimization of pressure shaft

The optimal area of the pressure shaft is found by the same approach:

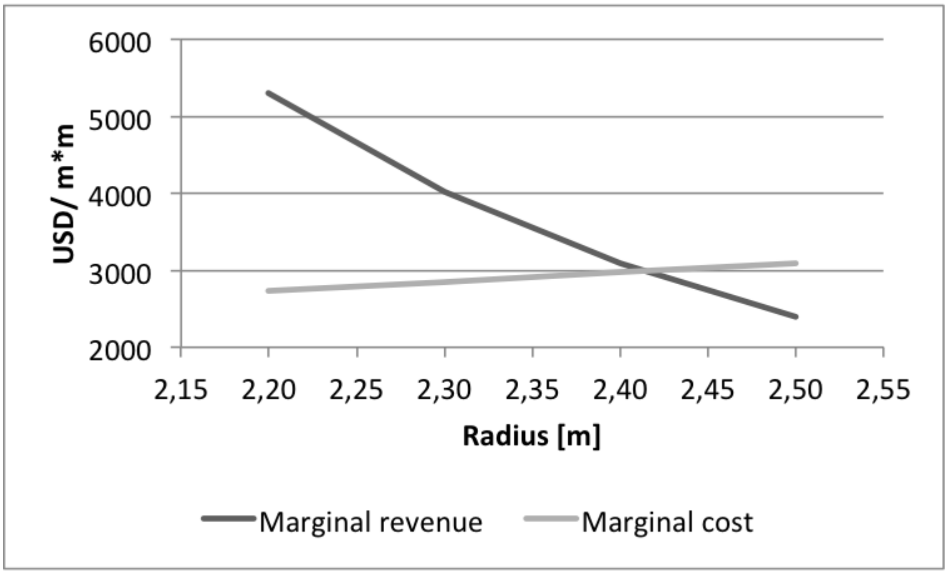


Figure 5.5: Optimal radius of pressure shaft

As figure 5.5 illustrates the optimal radius of the pressure shaft is about 2,4 m, which gives an area of approximately 18 m^2. This yields a cost of about 7.640 USD/m, which result in a total cost of the pressure shaft to be 3,5 M USD.

Total cost of Waterway

With an optimal area of head- and tailrace tunnels of 40 m^2, and the vertical pressure shaft of 18 m^2, the total cost of the waterways is 32,8 M USD.

Calculation of head loss

The cross sectional area of the waterways in Option B was designed based on an optimization with the intention of minimizing the head loss. As a consequence, lower head loss for option B can be expected, compared to option A. Total head loss in Option B is about 9,4 m, see Appendix I for calculations.

5.4.2 Underground power house

For the construction of an underground powerhouse (civil works) there are two cost contributors that distinguish itself:

- Construction of the power house, 6,3 M USD
- Access tunnel, 7,6 M USD

The total cost of the civil works of the underground powerhouse is calculated to be 13,9 M USD, see Appendix J.

Total cost of Option B

The total cost of option B, consisting of the construction of the underground powerhouse, pressure shaft and the head- and tailrace tunnels is equal to 46,7 M USD.

5.5 Comparison of option A and B

The basic principle of hydropower production is to utilize the potential head of the water. Larger head provides a bigger potential for production of power. The two factors that differ between Option A and B, is the total cost and the head loss through the system. Thus constituting the basis for comparison.

Table 5.1: Comparison of Option A and B

Component	**Option A**	**Option B**
Total head loss	35,1 m	9,4 m
Waterways	86,6 M USD	32,8 M USD
Powerhouse	12,3 M USD	13,9 M USD
Total Cost	**98,9 M USD**	**46,7 M USD**

As table 5.1 illustrates, the head loss of Option A is considerably greater than Option B. Option B is at the same time significantly cheaper, making it the overall layout worth using as a basis for simulation of production.

Construction of tunnel

The big difference in cost between the two options is mainly because of the large expense concerning the construction of the penstocks. There is at the same time also a big difference in the cost per meter tunnel for the two proposed options. SP suggests a lining in the tunnel, with an associated unit cost of 5.160 USD/m. Jentoftsen (2013) however, argues that this will not be necessary and concludes with an unit price of 3010 USD/m [7]. The two different unit costs rests on the fact that it exist a different construction philosophy, regarding tunnel making, in Norway compared to other places. Norway has through high labour costs been forced to optimize their method of construction.

The Norwegian philosophy of constructing tunnels rests on the following idea:

Utilize the properties of the rock, by keeping main focus on supporting it after excavation.

However, the lining proposes a more extensive rock support, which could prove advantageous in areas with non-sufficient geological properties. The lining will in addition smoothen out the cross section, minimizing the head loss. Jentoftsen (2013) concludes that the implementation of lining in this case is a technical exaggeration and not economically beneficial [7].

Powerhouse

Even though the cost of constructing the surface powerhouse isolated is somewhat cheaper than constructing the underground powerhouse, it needs to be seen in a bigger context. As mentioned, it is the steel parts of the penstock, which is the biggest contributor to the overall cost. Constructing a surface powerhouse without extensive use of penstocks may prove difficult.

Chapter 6

Simulation of Production

6.1 General

To assess whether or not a potential project is profitable in the long run, an overview over the investment- and income flow should be conducted. A way to estimate the income flow is to simulate the production in the future and hence calculate the income.

A simulation model should be capable of simulating the following conditions [13]:

- Hydrological conditions
- Composition of the production system
- The consumer system
- Operation strategy

Simulation of production was conducted by nMAG, a software developed at NTH/SINTEF [13]. Furthermore, in order to represent a realistic simulation model, four elements are especially important [13]:

- Hydrological conditions (inflow)
- Power market (power demand and price)
- The hydropower production system (reservoir, tunnels, power plants etc.)
- Operation (concession rules, operating restrictions, strategy etc.)

The hydropower production system is thoroughly described in section 2.6.

6.2 Hydrological input

The hydrological data used in this simulation is a preliminary series based on a relatively short time period (8 years, from 1.1 to 31.12, nMAG does not work with hydrological years). The data is modified to better fit the hydrological conditions today. The original series was from 1.10-1973 to 30.9-1985, with missing input from 1.10-1976 to 30.9-1979. This is not an ideal basis for simulation, but will have to make due being the best basis obtainable.*

The data is retrieved from Mita Hills, and is adjusted to also be applicable for Lunsemfwa and Mkushi by scaling it according to the mean annual inflow of the respective areas.

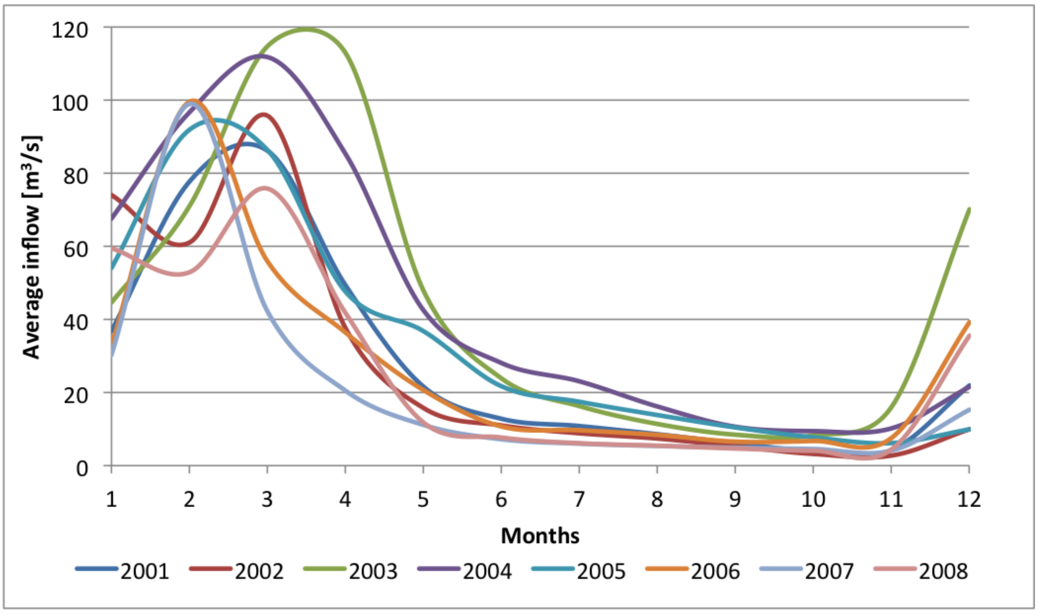


Figure 6.1: Annual variation of the average inflow to Mita Hills

*Communicated by Lars Ødegård. March 2013.

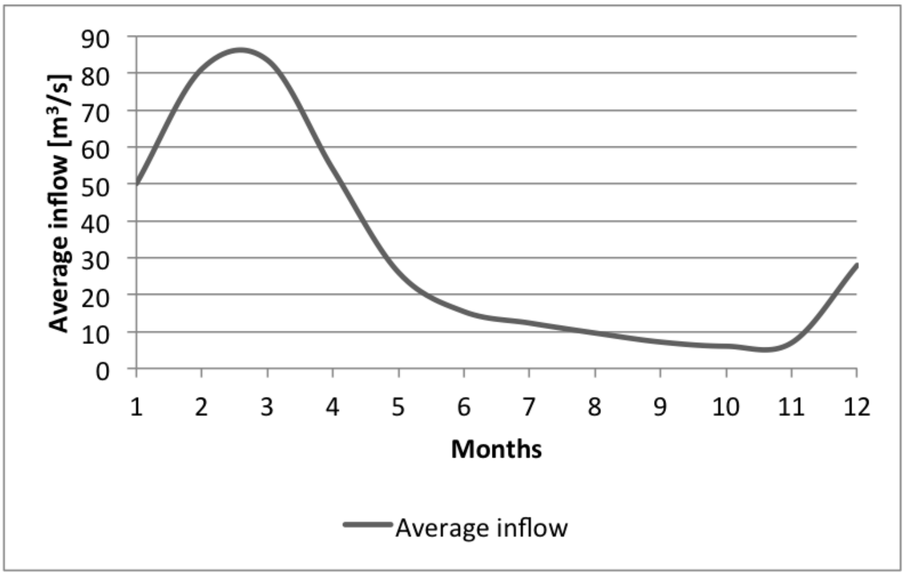


Figure 6.2: Average inflow of hydrology series (Mita Hills)

The average inflow derived from the data series provided, equals to 998 Mm^3/year, which is in good resemblance to the SP proposed inflow (990 Mm^3/year) of Mita Hills.

6.3 The power market

Firm power

Firm power is a term used for energy supplied with a high degree of reliability [13]. In Norway it is normal to define firm power as the amount of energy that can be delivered in 9 out of 10 years. Elsewhere, firm power of 95-99% is often defined.

In this case, being that the hydrological data series is based on less then 10 years, the definition of firm power is determined as the power that can be delivered to the system with a reliability of 95%.

Occasional power

The firm power consumption normally varies with the seasons. In Norway the consumption is high during winter when inflow is low and lower during summer when inflow is high. During periods with high inflow and low consumption (summer in Norway, rainy season in Zambia) it may be profitable to produce more power than the need for firm power should indicate. Such power is called Occasional power, and is often supplied on short term agreements and at a lower price [13].

Power demand in Zambia

The Norwegian power market has access to a well constructed power grid system, making it somewhat easy to allocate power. Depending on the seasons and the power market, optimizing the production of the power at the hydro power plant is considered more feasible in Norway than in Zambia. In Zambia, a country with a shortage of power, the need for a predictable power supply is considered to be the most important aspect. This fact makes up the chosen requirement for simulation in this case:

Produce as much power as possible, with a reliability above 95%.

6.4 Plant efficiency factor

If the head and discharge in the system only varies a little, it is common practice to preform simulation with a constant plant efficiency factor. This is a good approach for power plants with good regulation [13].

In this case, the head of Mkushi reservoir varies only 3 m, from 987-990 m.a.s.l. This is considered to be fairly little compared to the gross head (490 m). Mita Hills will assure a steady inflow to Mksuhi even in periods with low unregulated inflow. Even though the Mkushi reservoir proposes some storage capacity, it has been ignored in the simulations as a simplification.

In the *Conceptual schemes report*, SP concludes with an overall plant factor of 0,73. The arguments presented are that a lower plant efficiency factor minimizes the spill of water, maximizing the total production (firm + occasional power). However, considering Zambia's shortage of power, it is assumed, as stated in section 6.3, to be most profitable to maximize the production of firm power alone, ignoring the upside of the potential extra occasional power.

Therefore, a plant efficiency factor of 0.85 is assumed in this case.

6.5 Restrictions

Not all the water in the system goes directly to the production of power. There may be several other addresses that claim part of the water, putting restrictions on the system. In this project, the following restrictions has been identified:

- Mandatory release from Mita Hills - 480.500 m^3/d to agriculture and industrial rights [28]

- Mandatory minimum discharge from Mkushi dam, 1,6 m^3/s *
- 260,4 mm/year evaporation from Mita Hills [28]
- Min/max operation level in Mita Hills - 1095/1120 m.a.s.l

6.6 Operation Strategy

The object of simulation is to resemble, as accurately as possible, the real operation of the hydropower system. The main challenge in simulating operation is to decide on the management of the reservoir, i.e. the amount of water that should be released for production in each time step. Normally this is dominated by the requirement for desired firm power production [13]. Two different ways to run the simulation in nMAG is described below.

The Reservoir Guide Curve Model

The hydrological conditions in the watercourse follow more or less the same pattern each year. There is a filling period and a release period. The hydrological history may then be used to constitute a *guide curve*, which shows normal reservoir contents over the year. As a consequence the reservoir may be filled and emptied accordingly in order to be used correctly, taking both the future inflow and the power market into account. The use of this method may reduce the risk of rationing, which is an expensive operation.

The Rule Curves Method

The rule curves method is an expansion and improvement of the guide curve method. Two curves are established for running of the reservoir. The upper curve, *the surpluss curve*, defines when additional release for occasional power is to be implemented. The lower curve, *the rationing curve*, defines when rationing should be implemented, see figure 6.3 [13].

*Communicated by Lars Ødegård. April 2013.

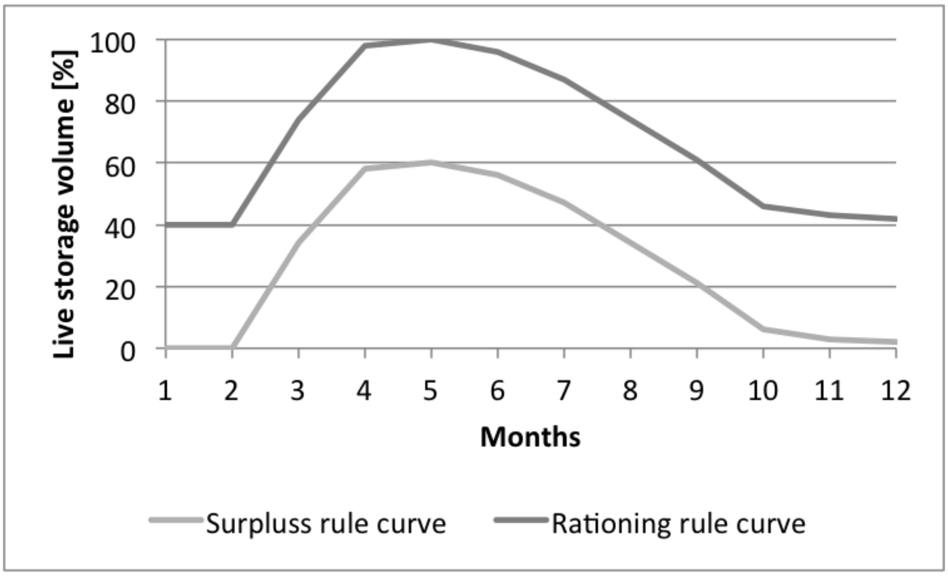


Figure 6.3: Optimized rule curves for Mita Hills

If rationing is inevitable, it is considered cheaper to even out the deficit over time, compared to dealing with the deficit in a short time period. This because the marginal cost of rationing increases with the increase of firm power deficit [13].

6.7 Theoretical limit for production

To create a basis for comparison of the future simulated production, a theoretical limit for production is calculated. This calculation is based on the following established input:

- Plant factor 0,85
- Net head 480,6 m
- Energy equivalent 1.113 kWh/m^3
- Total inflow to the system 1639 Mm^3/year

The total amount of water available for production equals to the total inflow subtracted evaporation, irrigation and minimum discharge in Mkushi. This is calculated to be 1407 Mm^3/year. Consequently, the theoretical limit for production, given no spill from the system is equal to 1566 GWh. It is important to highlight this value as a theoretical maximum based on a fixed plant efficiency (used in this case). However, if the simulation were conducted with a varying efficiency curve, it would perhaps be possible to obtain a somewhat higher maximum value.

6.8 Result of simulation

6.8.1 Firm power delivery and demand coverage

The results obtained through simulation was based on the following input:

- Q = 57 m^3/s, max turbine discharge
- Firm power demand = 1496 GWh

Several simulations and adjustments of the rule curves where conducted in nMAG, highlighting the total production of 1395 GWh firm power as the best result. The firm power is obtained with a reliability of 95,1% (demand coverage), see Appendix K for results.

Table 6.1: Main result from simulation

Firm power demand	1496 GWh
Demand coverage	95,1 %
Utilization †	93,4 %
Average production	**1395 GWh**

6.8.2 Spill water

The results obtained in this simulation highlights the following spill of water before and in the Mkushi reservoir:

- Transfer system 78,3 Mm^3
- Mkushi 88,8 Mm^3

Spill in the transfer system

There may be two reasons for the spill in the transfer system:

1. The rule curves may be further optimized in order to operate the reservoir even smoother. The rule curves are based on monthly input proposing an improvement by shortening the increments (e.g. daily based)
2. The dimensions of the transfer system is not able to handle the peak floods

†Power production utilizes 93,4 % of the inflow to the Mkushi reservoir.

In order to be able to handle the peak floods, an expansion of the capacity of the transfer system, should be considered. A preliminary assessment considers this to be costly, based on the length of the tunnels [7]. However, as earlier clarified, a more thorough assessment of the transfer system is outside this thesis scope.

Spill in the Mkushi reservoir

The spill associated with the Muchinga powerhouse is located in the Mkushi reservoir. It is a result of limited capacity of the headrace tunnel and the maximum turbine discharge. Even though it has not been assessed in this thesis, an optimization of increased capacity should be considered. An immediate assessment assumes the increase not to be economically beneficial, based on the total length of the head- and tailrace tunnels (9740 m).

6.9 Validation of results

The obtained result of 1395 GWh is 171 GWh short of the theoretical maximum production. This is considered to be within fair limits because of the water lost from the system through spill.

The yearly average delivered production (1395 GWh) does not meet the required firm power demand (1498 GWh) set for this simulation. The firm power demand is set to the respective value, only to maximize the total production from the system. Lowering the demand for firm power however, would results in a decrease of total production, and also possibly result in the production of occasional power.

In industrialized countries such as Norway, electricity is traded in a similar fashion as to the stock market. This makes the optimization of the total production (firm + occasional power) interesting.

However, this assessment has not been conducted in this thesis based on the following reasons:

- the market for occasional power in Zambia, is not identified well enough
- the shortage of power supply in Zambia indicates the need for a reliable production of firm power

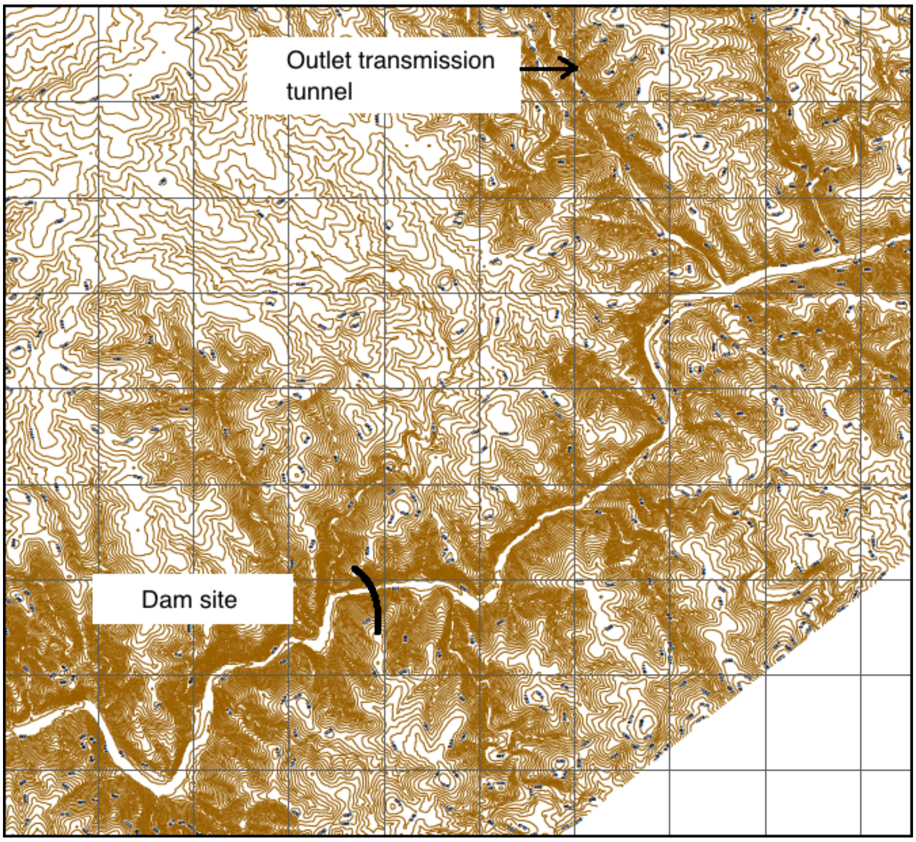
Chapter 7

Conclusion

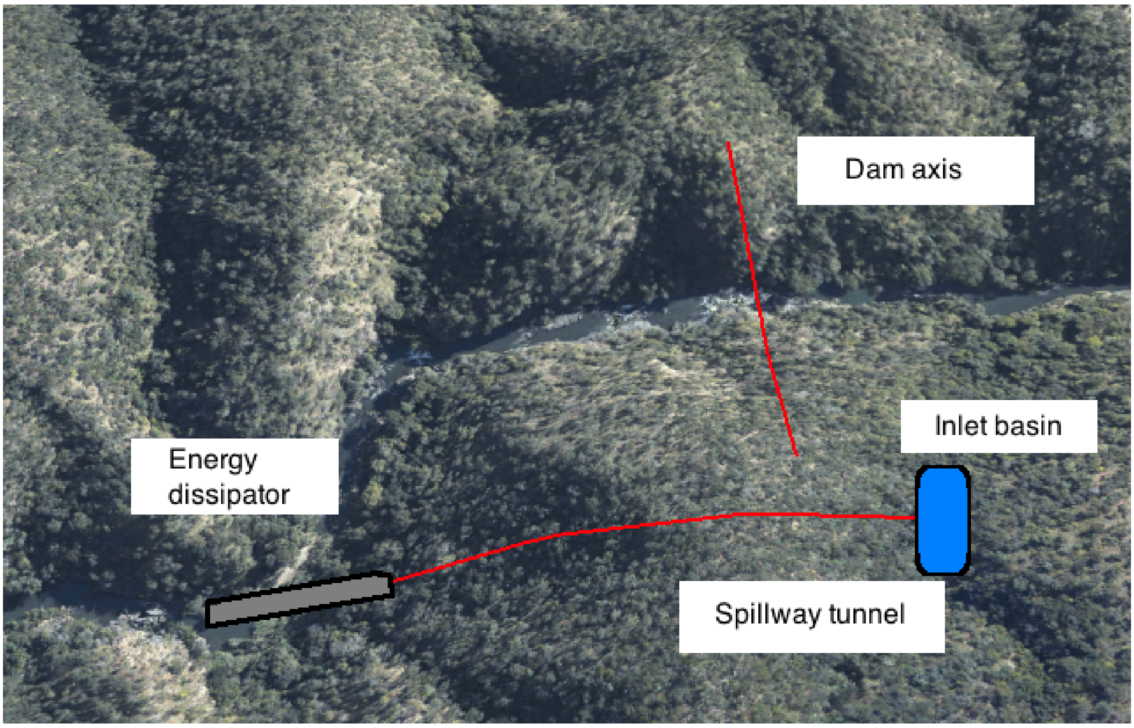
7.1 Presentation of the optimal solution

The best overall solution for production of power in the Muchinga HPP, may be summarized as follows (see figures 7.1 and 7.2 for graphical representation):

- A 86 m tall, asphaltic core embankment dam
- Associated spillway solution
- Underground powerhouse, 228 MW
- Headrace tunnel, 8200 m, area 40 m^2
- Pressure shaft, 455 m, area 18 m^2
- Tailrace tunnel 1540 m, area 40 m^2

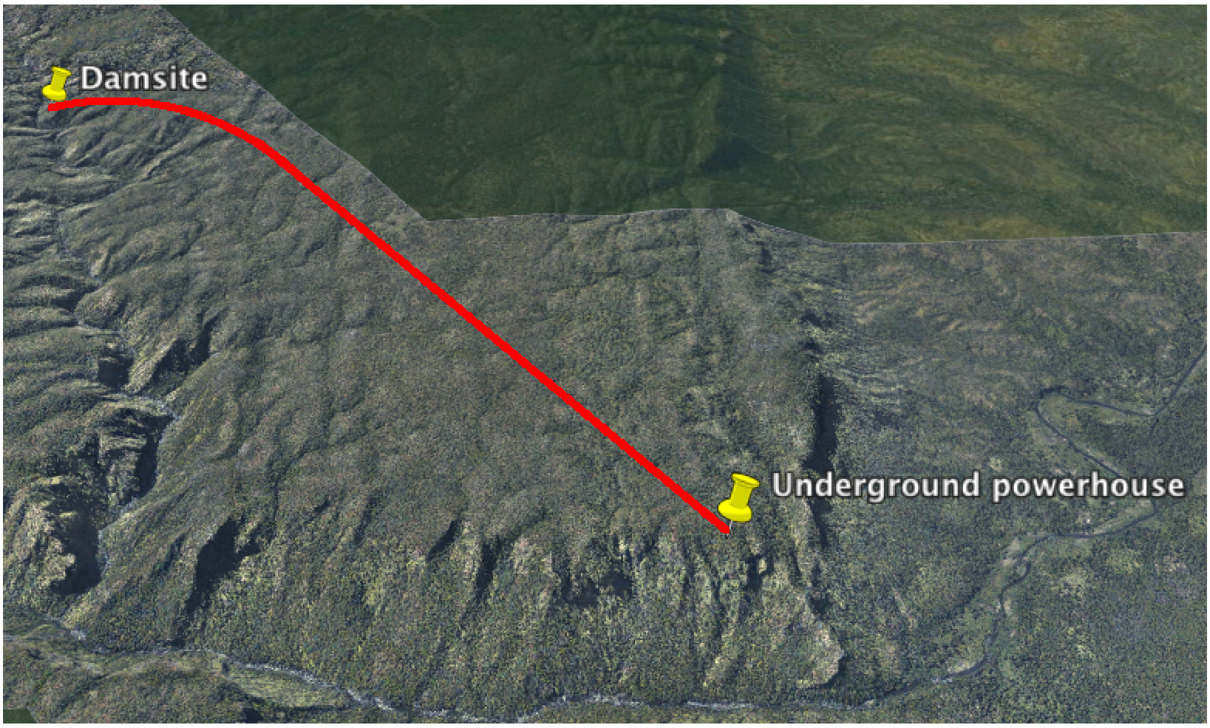


(a) Dam site [4]

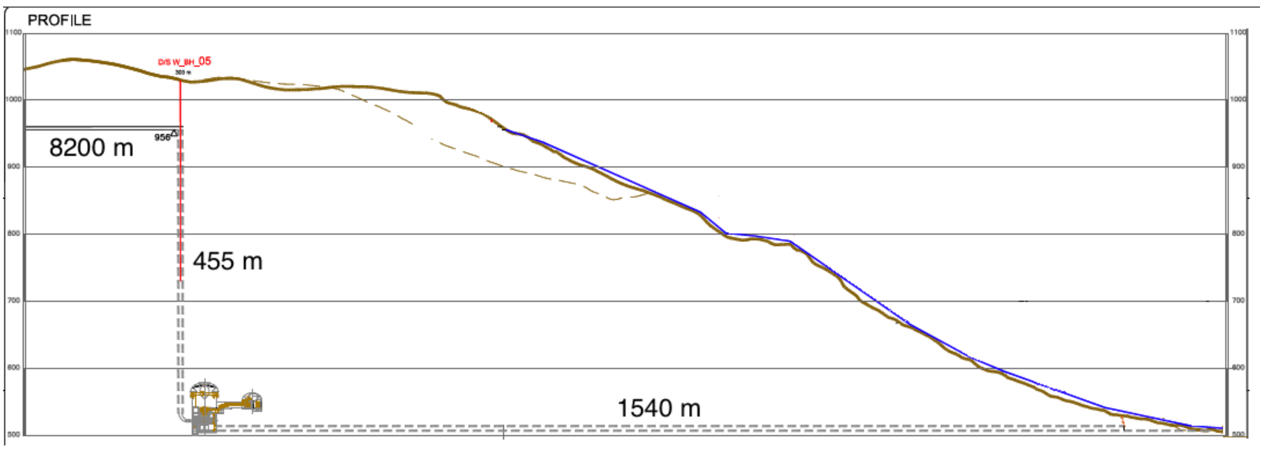


(b) Proposed spillway solution [12]

Figure 7.1: Proposed dam solution



(a) Headrace tunnel [12]



(b) Underground powerhouse [25]

Figure 7.2: Pressure shaft, underground powerhouse and tailrace tunnel

7.2 Comparison with the Studio Pietrangeli (SP) solution

Dam solution

The SP proposed solution of a RCC dam, is considered to be a less attractive option both in regards to the overall cost, but also because the technical arguments an ACED proposes. The overall height of the dam is reduced from 120 m to 86 m regardless of type. As a consequence the total cost of the dam is greatly decreased. The total cost of the overall dam solution is calculated to be approximately 38,4 M USD (including preparation of foundation and bypass solution, see Appendix E).

This thesis has not thoroughly assessed the bypass and spillway solution, being only preliminary suggestions. Isolated, the ACED is considered to be the best option by far, but the selection of this dam type is also an assessment of the associated solutions. The probability of further increased costs related to the construction of the bypass/spillway solution is present. Even though this thesis does not regard an increase of 16,5 M USD to be probable, a further and more thorough assessment of the construction of these components needs to be conducted.

Waterways

The total costs of the waterways are equal to 32,8 M USD, reducing the SP proposed solution by 53,8 M USD. This is primarily due to the cost of penstocks and to the different construction philosophies used in Norway compared to other places.

The head- and tailrace tunnels and pressure shaft have been optimized with respect of minimizing the head loss. As a consequence the head loss has been reduced significantly from the SP proposed solution. Even though the lining proposed used contributes to reduced head loss, the cross section in question evens out the advantage..

The overall head loss in the system is 9,4 m, which implies that the net head for hydropower production is 480,6 m.

Powerhouse and production

An underground powerhouse proposes great flexibility in its placement, minimizing its visual impact. Its current placement in paragneisses offers good geological properties reducing the cost of rock safety compared to a placement in schist.

However a more thorough assessment of the placement in regards of a potential optimization of the length of the tailrace- and access tunnel should be conducted.

Furthermore, a powerhouse out in the open requires a more extensive use of concrete. As mentioned the total use of concrete used in this project should be kept to a minimum, due to the fact that pozzolan is not produced locally and needs to be imported.

Installed capacity

SPs suggestion of a total installed capacity of 255 MW is considered to be an overestimation. An installation of 255 MW suggests an increase of the maximal turbine discharge. However, this would also require an increase of the capacity of the head- and tailrace tunnels. Even though this will utilize more of the water, it may be a costly operation, outweighing the benefits. As a consequence the installed capacity is calculated to be 228 MW, resulting in cheaper solutions for the electrotechnical and mechanical installations.

Total cost of proposed solutions

The total cost of the proposed solutions is summarized in table 7.1:

Table 7.1: Total Cost

Component	**[M USD]**
Dam	38,4 M USD
Waterways	32,8 M USD
Powerhouse	13,9 M USD
Total Cost	**85,1 M USD** *

7.3 Suggestions for further work

Survey of power market

It is expected that Zambia will have a shortage of power supply until 2017. Beyond this, it would be of interest to survey the potential that exists in the future market, regarding speculation in power production (firm + occasional power).

*Excluded electrotechnical and mechanical equipment in powerhouse.

Continued operation of Lunsemfwa Hydro Power Station

Even though it has not been conducted in this thesis, an incorporation of the existing Lunsemfwa Hydro Power Station, during simulation of production should be considered. Based on current simulations, a significant portion of the spill water is located at Lunsemfwa. Even though the contribution from the spill water not necessarily will contribute to the overall production of the firm power, it would be interesting to see it in relation to the survey of the Zambian power market.

Incorporation of Mkushi reservoir in simulation of production

An incorporation of the Mkushi reservoir in the simulation model would perhaps contribute to a reduction of the spill in the reservoir. Expanding the simulation model should therefore be conducted in later assessments.

Continuous development of the cost basis

Continuous development of the cost basis should be conducted in order to minimize the uncertainties regarding some of the cost calculations. The cost of materials provided from the construction of tunnels and opening of a quarry, should be surveyed better than at present time.

Assessment of the upgrade required for the road networks

This thesis highlights the possibility of a synergy effect regarding the upgrade of the road network between the Chinese operated Manganese mine, the upgrade of the Mulungushi powerhouse, and this project. A thorough assessment of the required upgrade should be conducted in order to present an overall assessment of the feasibility of this project.

More thorough geological inspection at dam site

Even though the geological properties look sufficient at the current time, a more thorough inspection of the dam site should be conducted. A thorough inspection of the feasibility of the bypass and spillway solution should also be conducted. Furthermore, there is uncertainty about the quality of the rocks in the Lufubo schist, on the right abutment, which may affect the overall dam selection.

More thorough assessment of the placement of the powerhouse

This thesis proposes a placement of the powerhouse in the paragneisses zone, offering good geological properties for construction. Being that the placement of the powerhouse only has been preliminary assessed, an optimization of the placement should be conducted. This should be done both in regards of surveying the best location regarding the optimal length of the tailrace- and access tunnels, but also in regards of the geotechnical properties of the suggested placement.

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List of Figures

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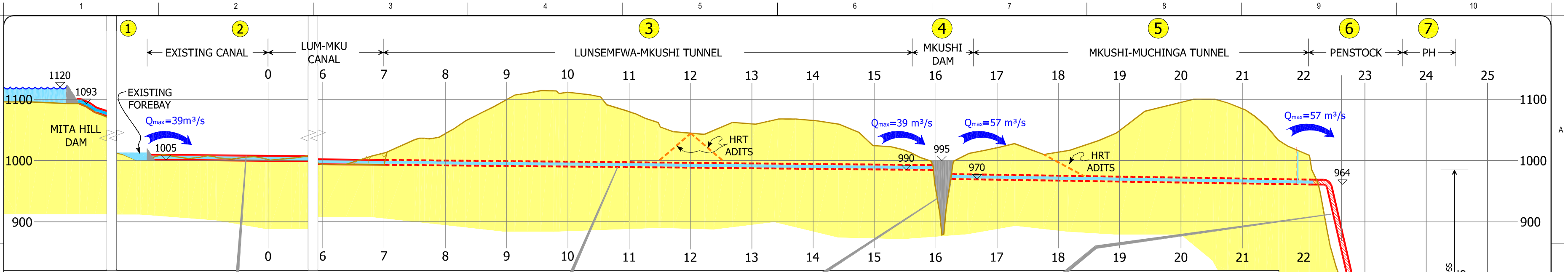
Appendix A

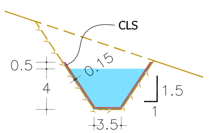
Overall Layout



1 2 HEADRACE CANAL

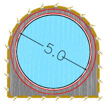
LUNSEMFWA-MKUSHI

L (km)	= 2 + 7
i	=0.8 m/Km
V_{EX}	=0.42 Mm³
V_{CLS}	=21'000 m³
W_{bars}	=1'400 ton



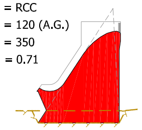
3 5 HEADRACE TUNNEL

		3	5
L	km	8.5	6.5
Q	m³/s	50	60
D	m	5.0	5.0
i	m/km	0.9	1.4
V_{ex}	m³	272'000	208'000
V_{cls}	m³	105'000	80'000



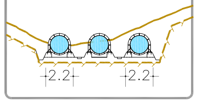
4 MKUSHI DAM

C A (km²)	= 3'565
Reservoir (km²)	= 5
(Mm³)	= 135
DAM	
Type	= RCC
Height (m)	= 120 (A.G.)
CL (m)	= 350
Volume (Mm³)	= 0.71



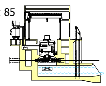
6 PENSTOCK

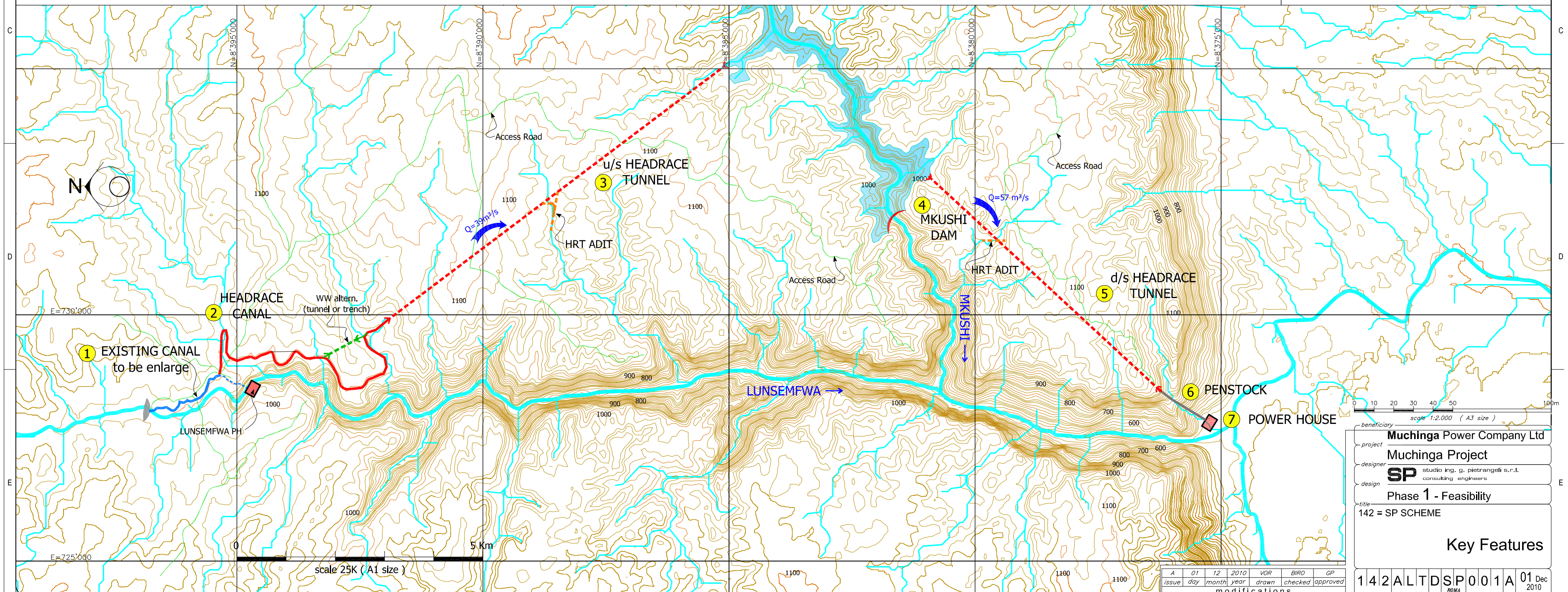
number	= 3
L (Km)	= 1.2
D (m)	= 2.2
S (mm)	= 27
W_Steel (ton)	= 3'100



7 POWER HOUSE

Type	= OPEN-AIR
Turbines	
numbers	= 3
type	= Pelton
HN (m)	= 488
Q (m³/s)	= 57
E_{PH} (GWh/y)	= 1'630
LF	= 0.73
IP (MW)	= 255
Units (#xMW)	= 3 x 85





scale 1:2.000 (A3 size)

beneficiary	**Muchinga** Power Company Ltd
project	Muchinga Project
designer	SP studio ing. g. pietrangeli s.r.l. consulting engineers
design	Phase 1 - Feasibility
title	142 = SP SCHEME
	Key Features

A	01	12	2010	VDR	BRO	GP
issue	day	month	year	drawn	checked	approved

modifications

1 4 2 A L T D S P 0 0 1 A 01 Dec 2010

section / kind works / type / office ROMA / serial n° / rev. / date

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Appendix B

Supplemented methods

People of importance

Assessments made in this thesis are often made in cooperation with well-experienced people. These people have shared their knowledge, making it possible to make fair assessments regarding solutions presented in this thesis.

Siri Stokseth	PhD Statkraft, and associated professor at NTNU. Main supervisor for this master thesis.
Lars Ødegård	Project Director in SN Power, supervisor.
Helge Saxegaard	Senior Advisor, Veidekke - Expert in asphalt core dams, supervisor.
Leif Lia	Professor NTNU, supervisor.
Ånund Killingtveit	Professor NTNU, developer of nMAG.
Øystein Røneid	Consultant SWECO. Participated in the making of the cost basis provided by NVE.
Amund Bruland	Professor NTNU, expert in tunnel construction.

Institutions of importance

Known and respected institutions may often provide publications of relevant information.

NVE The Norwegian Water Resources and Energy Directorate is a directorate under the Ministry of Petroleum and Energy. NVE's main work areas is development of legal framework for hydropower design [2].

ICOLD The International Commission on Large Dams. ICOLD is a non-governmental, international organization, which provide a forum for the exchange of knowledge and experience in dam engineering [1].

HYDROPOWER DEVELOPMENT Hydropower Development is a book series published by NTNU. It consists of 17 volumes. The authors of the volumes are all leading professionals within their fields [13].

Mott MacDonald The Mott MacDonald Group is a diverse management, engineering and development consultancy [31]. Mott MacDonald was hired by SN Power to preform a feasibility study parallel to the conduction of this thesis.

Rankin Engineering Rankin Engineering is a consulting firm based in Lusaka, Zambia. Rankin Engineering possesses great knowledge about the unit prices of certain components used in this project.

Appendix C

Developement of cost basis

How to make a cost basis

Approach

In theory there are two different ways to develop a cost basis. One way is through gathering of information about unit prices. Another way is to gather information about similar projects and use their experience of total costs to anticipate the future expenses of your own project. One of the most important parameters that influence the total cost of a project, is the total amount of time the project is ongoing*. It is of course difficult to quantify the unit price of time, but it is important to highlight this as a latent cost contributor.

The challenge of making a well functioning cost basis is to adapt the unit prices in question to better fit the proportion it constitutes of the total cost. This approach however implies that it exist a decent basis for comparison in terms of available conducted projects. In Norway this approach proved somewhat difficult to conduct, because the majority of the approached owners did not submit information about their projects*.

Adaption to the project

Based on the fact that no information about similar projects in the area was available, the natural choice for approach to making a cost basis was through the clarification of unit prices. Through the field trip conducted to Zambia, contact with certain suppliers and local contractors were established. The information they

*Communicated by Øystein Røneid. March 2013.

provided together with the experience made in Zambia, where used to approximate a cost basis.

Cost basis

Table C.1: Cost basis

Material	Price [NOK/USD]	Unit
Cement	1380/236	ton
Aggregates		
40 mm	173/30	ton
20 mm	202/35	ton
10 mm	219/38	ton
Sand	81/14	ton
Fine sand	69/12	ton
Concrete	950/162	m^3
Reinforcement		
High tensile bar	5853/1000	ton
Mild tensile bar	5268/900	ton
Mesh reinforcement	2341/400	ton
Formwork		
From plywood	470/80	m^2
Formwork timber	2630/450	m^3
Fuel		
Diesel	8,9/1,5	litre
Petrol	9,4/1,6	litre
Bitumen	6600/1150	ton
Labour	1000/170	pr month
Rigging and operation	25-30	%

The table C.1 Cost basis is based on the following input:

- Imported materials
- Conversion between NOK-USD is based on the conversion rate from mid april (2013), 1 USD = 5,85 NOK

Even though the cost basis in general relies on imported materials, it can be assumed that local production of aggregates results in a cheaper price. There has not been estimated how much cheaper at present time. Furthermore the cost basis is based on:

Cement Price from Chilanga Lafarge Cement in Lusaka, 250 km from the project area. Available in 50 kg bag and bulk. Stable supply are guaranteed by contract [32]

Aggregates Import from Kapiri M'poshi, approximately 150 km from project area [32]

Concrete Based on normal value of concrete density equal to 2400 kg/m^3, and 50/50 aggregate composition of 0-8 mm sand and 8-24 mm coarse aggregate [20]. Price includes 30 % in addition (establishment and operation of concrete production plant), excluded admixtures

Reinforcement Price based on import from the major operators in Copperbelt (Ndola, Kitwe) [32]

Formwork Average price based on an average within different sizes (9,12,18 mm) [32]

Fuel Based on local experience

Bitumen Delivery from South-Africa [33]

Labour Based on experience gathered from visit to Copperbelt and personal communication. Normal work load is 6 days a week, 12 hours shift [8]

Rigging and operation Communicated by Øystein Røneid. March 2013.

Appendix D

RCC Dam

Unit price of concrete

The cost basis developed by NVE states the following in regards to the total cost of concrete in a RCC- dam [21]:

Table D.1: Unit cost of concrete in RCC, based on cost basis from NVE

Cement	120 USD/m^3
Aggregates	26-94 USD/m^3
Pozzolan	75 USD/m^3
Total unit cost for concrete in RCC	**214-282 USD/m^3**

Concrete properties for a RCC- dam [21]

- Concrete quality B 25-35
- Amount of cement is 150 kg/ (m^3 concrete)
- Amount of pozzolan is 80 kg/ (m^3 concrete)
- Amount of water (100 l) is adjusted to the low cement content

Because the concrete used in RCC dams have a lower cement content [21], a separate unit price will have to be calculated. For a concrete quality of B 30 (Norwegian standard) similar to that of a RCC-dam, the following approximation can be assumed for RCC:

- 50/50 % of 0-8 mm sand and 8-24 mm of coarse aggregates [20]
- 2100 kg/m^3 Aggregates, normal consumtion in RCC dam [21]

These assumptions combined with the cost basis developed in Appendix C, highlights the unit cost for aggregates to be 52 USD/m^3. At the same time the new unit cost for cement in use in a RCC is approximately 36 USD/m^3. Pozzolan is not produced locally in Zambia, and hence presents the need of import. It has not been possible to gather the unit price for pozzolan from any external providers and it will be assumed to be equal to the price stated in the cost basis by NVE for further calculations.

Summarized the new unit cost for concrete in RCC dams is:

Table D.2: New unit cost of concrete in RCC

Cement	36 USD/m^3
Aggregates	52 USD/m^3
Pozzolan	68 USD/m^3
Total unit cost for concrete in RCC	**156 USD/m^3**

Calculation of volume

Based on the optimized dam section, the volume of the dam is calculated by height increments of 1 meter. The total volume is approximately 307.000 m^3.

Appendix E

ACED

Core volume

Based on the previous optimized dam section the volume of the dam is calculated by height increments of 1 meter. The top of the central core should be at least 0,5 m above the design flood water level (DFWL) [22], resulting in a 82 m tall core.

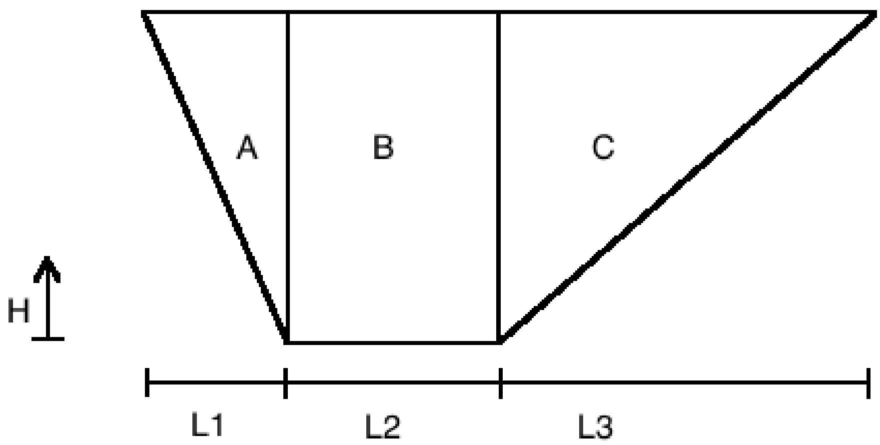

Figure E.1: Elevation profile

The volume of the core was calculated to be approximately 5440 m^3.

Volume zone 2, 3 and 4

The following theoretical cross section of an ACED is presented, see figure E.2:

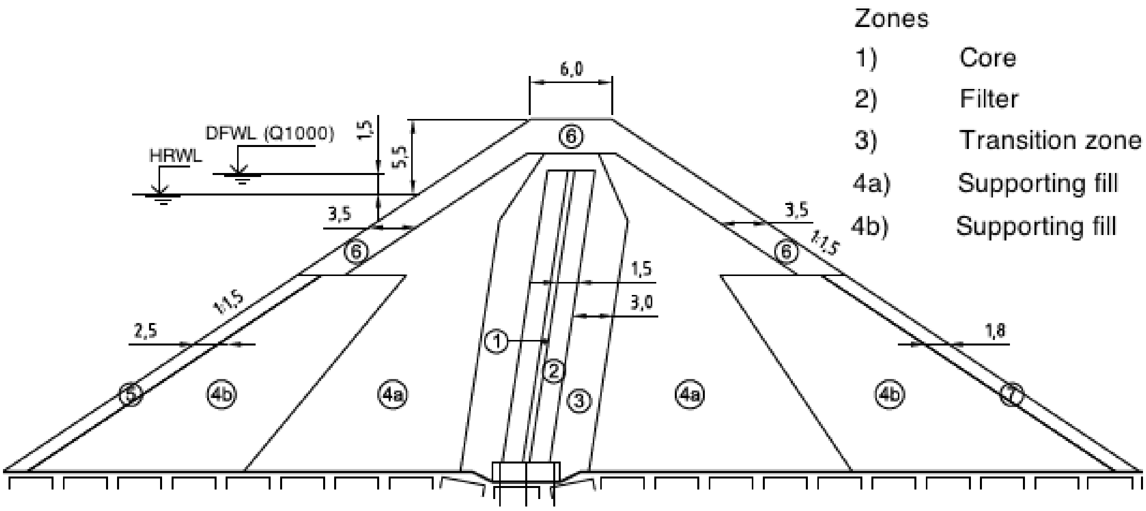

Figure E.2: Cross section ACED [21]

Furthermore, to obtain an approximation of the total volume of the different zones the calculation is based on the same approach as for the core. The width of zone 2 and 3 is assumed vertical and equal to 3 m and 6 m, respectively, for simplification. The width of zone 4 follows the inclination of 1:1,5.

The following calculations is presented:

Table E.1: Calculation of volume ACED

Core (Zone 1)	5.440 m^3
Filter (Zone 2)	29.400 m^3
Transition zone (Zone 3)	64.070 m^3
Supporting fill (Zone 4)	1.016.000 m^3

Cost of preparation of dam foundation

The chosen dam placement highlights the need of foundation preparation length to be 292 m. Furthermore, with a total height of the dam equal to 86 m, the following cost function may be assumed in order to calculate the total cost for preparation of the dam foundation. This includes construction of plinth, grouting and excavation of soils:

- 11.965 USD/m [21]

This equals a total cost for foundation preparation to be 3,5 M USD.

Cost of bypass solution

Dimensions of the bypass tunnel calculated in the introductory project report, will be assumed in the further calculations [19]:

- Area of bypass tunnel 53 m^2

With an area of 53 m^2 the unit price for tunneling is 3555 USD/m [7]. The bypass solution is also assumed constructed with two gates, for inspection reasons. The total cost of these gates is about 1,7 M USD [21].

Furthermore, the width of the ACED is about 264 m at the bottom. In order to create a bypass tunnel a length of 500 m is assumed in order to get by the dam itself, with reasonable embedded safety.

This makes the total cost of the bypass solution to be 3,5 M USD.

Appendix F

Spillway solution

In designing the spillway solution a simplified approach has been chosen. This has been done in order to limit the extent of the calculations and to keep focus on the general overview.

Threshold - Design and price estimation

To make an inlet basin, excavating the existing rock before constructing a threshold must be conducted. The proposed inlet basin is assumed to be of rectangular shape, with two corners, see figure F.1.

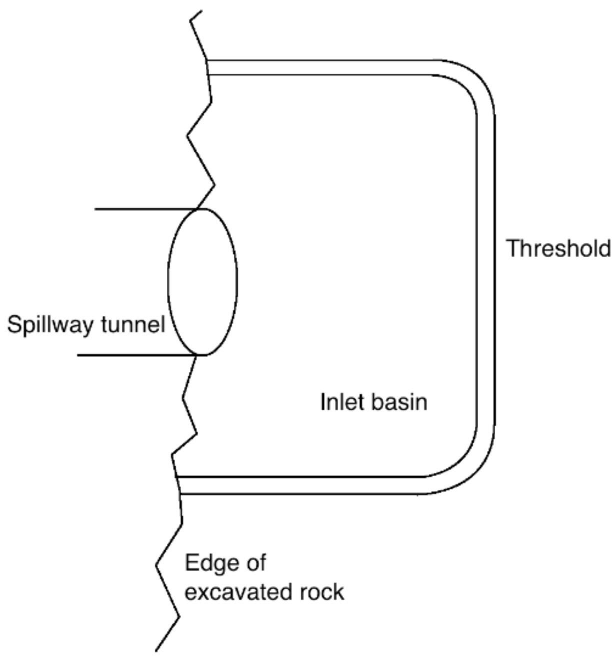

Figure F.1: Principle design of inlet basin, seen from above

To figure out the length of the threshold, the following equations was used [34]:

$$Q = C * L_{eff} * H_0 \tag{F.1}$$

$$L_{eff} = L_{tot} - 0,1 * n * H_0 \tag{F.2}$$

Where:

$\mathbf{H}_0$	Design height - assumed 1,5 m
C	Threshold coefficient - assumed equal to 2
Q	Total capacity [m^3/s]
$\mathbf{L}_{eff}$	Effective length [m]
$\mathbf{L}_{tot}$	Total length [m]
n	Number of contractions (corners) - equal to 2

Based on the assumption presented above, the total length of the threshold is approximately 128 m.

The following shape for the threshold is assumed [34]:

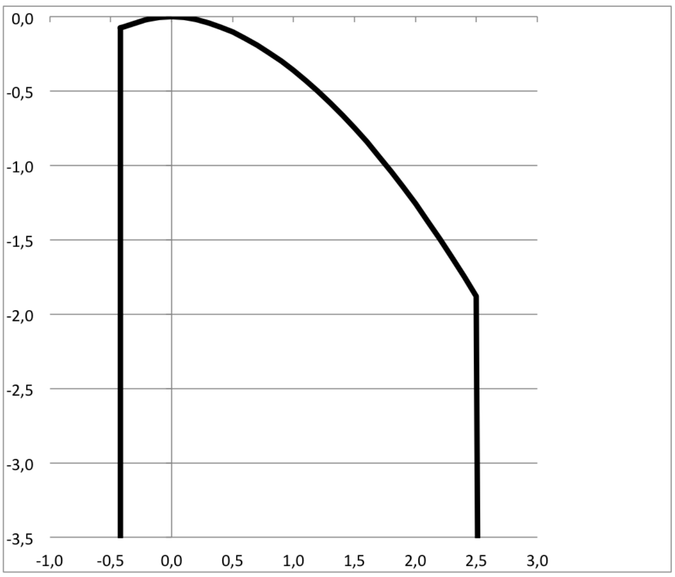

Figure F.2: Profile of threshold

This shape makes out an area of approximately 8,6 m^2. This results in the total volume of the concrete made threshold, to be around 1100 m^3.

The cost for constructing the threshold is about 0,2 M USD.

Inlet basin - Design and price estimation

When excavating the rock for the making of the inlet basin, an inclination towards the inlet of the spillway tunnel should be ensured.

For further calculations the following price estimations is applied [21]

- Blasting, loading and transport to dump - 41 USD/m^3
- Mass transport 19 - USD/m^3
- Total cost 60 - USD/m^3

The case of reducing the total price due to the lower labour cost could be made, but then again it is to be assumed that the effectiveness in Zambia is not comparable to Norway. Since the purpose of figuring out the cost of the spillway solution simply is to create a comparable alternative, this reduction is neglected.

Furthermore the design of the threshold should be constructed so that the sides do not go to far into the reservoir. To limit the extent of excavation the width of the inlet basin is kept considerably small (e.g. 10 meters). The inclination of the slopes in the actual area for the inlet basin is almost 1:1. The following is assumed for calculation of the excavation volume:

- width 10 meters
- length 100 meters
- an inclination of about 2 meters towards the inlet of the tunnel

This yields the approximate total excavated volume of 6000 m^3, resulting in the total cost of excavation of the inlet basin to be about 0,4 M USD.

Spillway tunnel - Design and price estimation

The proposed solution suggests two 360 m long tunnels with an inclination of 1:9. The solution of the spillway tunnel is calculated and presented below. An area of 68 m^2 yields a unit price of tunnel making of approximately 4.890 USD per meter of tunnel length [21]. The cost basis provided by NVE proposes a somewhat high unit price for tunnel making, compared to the relative low labour cost in Zambia. Then again, extra cost for rock support and a probable lower efficiency rate in Zambia could compensate for this. Consequently the unit price provided by NVE is still being assumed. As a result, the total cost of the spillway tunnel is about 3,5 M USD.

Spillway tunnel

Theta	14,64	70% of area
Q-1000	469	
Q-PMF	1864	
Inclintion (I)	0,326	
Manning	33	
Area (A) * r^2	2,5	
Perimeter (P)	4,51	
A/P=R	0,55	
r(m)	4,38	
Q	932,00	Two tunnels
Velocity (v)	19,43	

Head loss in waterway	**k or M faktor**	**Area [m^2]**	**Perimeter P [m]**	**R [m]**	**Velocity V [m/s]**	**Length [m]**	**Head loss [m]**
Inlet	0,5	68			13,71		4,79
Spillway tunnel	33	68	31,28	2,17	13,71	360	22,05
Outlet	1	68			13,71		9,57
Total loss							36,41
Allowed loss							37

Energy dissipator - Design and price estimation

An energy dissipator is essential to make sure the environmental effects of the construction of the hydraulic structures remains at a minimum. This section will not propose a selection of an energy dissipator solution. It will simply make a rough estimate of the total cost of construction.

From the tunnel outlet, it is approximately 100 meters to the natural flow of the river. As calculated when estimating the total area of the two spillway tunnels, the radius of the tunnels, with conventional cross section, equals to about 4,4 meters. This again proposes a width (included a buffer width) of the concrete works of 26 meters. An average depth of 3 meters is also assumed.

The total amount of concrete is about 7800 m^3, resulting in the total cost for the energy dissipator to be about 2,1 M USD (including reinforcement, formwork and excavation of soils).

Total cost of proposed spillway solution

The total cost of the spillway solution is 6,2 M USD.

Appendix G

Price estimation of ACED (core and filter)

Muchinga FS, Mkushi dam, Zambia. Heigth = 86 m, crest = 225 m.					
Prices excluding food, lodging, surveyor services, taxes and duties					
Prices as for 2013/14					
Core thickness = 0.65 -0.5m					
Approx volume of asphalt = 5440 cu.m or approx.16200 tons incl over consumption					
Construction time dam and asphalt: 12 months, effective 20 days/ month					
5 days work per week, 10 hours work per day					
Total working days, asphalt = 12x20=240 days, 2400 hours					
Items	amount	amount	unit price	Period	NOK
1.Establishmenet of asphalt plant on site					
Foundation for asphalt plant					100 000
Freight to site, asphalt plant					400 000
Establishment of asphalt plant, crane, people					300 000
Establishment of bitumen tanks, generator					100 000
Demobilisation					300 000
Total establishment, demob. of asphalt plant					**1 200 000**
2.Establisment of asphalt laboratory (container laboratory or in building on site)					**500 000**
3.Asphalt production on site					
Diesel for asphalt production, 12 litres per ton (litres)	12	16200	8,9		1 730 160
Hire of generator for electricity at plant, new price 550.000 NOK	0,2		550000	1	110 000
Electrisity from a 500KW generator, 250 KW running 0,2 liters per KW per hour , it is 50 liters per hour , 8 hours day 400 l per day	400	240	8,9		854 400
Asphalt aggregates prodused and delivered in 4 fractions on site		16200	70		1 134 000
lime stone/ cement as filler for asphalt, 7% of total quantity in ton	0,07	16200	1370		1 553 580
Bitumen delivered to site, 7,0%	0,07	16200	6600		7 484 400
Machinery hire of asphalt plant, 20% of value for 1 year +	0,2		12000000	1	2 400 000
Various repair and maintenance of asphalt plant					300 000

Lubricants for asphalt plant			10000	1	50 000
Hire of wheel-loader without operator, Cat 950 or similar, 10 hours per day 240 days	10	240	400		960 000
Skilled operator asphalt plant, 5000 NOK / month	12		5000		60 000
Two assistant at asphalt plant and operator wheel loader for asphalt plant, 10 hours per day 240 days	30	240		7	50 400
Total asphalt production					**16 686 940**
4.Running of asphalt laboratory					
Hire of equipment, value = 900000 NOK	0,25		900000	1	225 000
Repair/maintenenance of lab equipment					50 000
Chemicals and various materials					50 000
1 Laboratory engineer one year			30000	1	30 000
2 assistants (unit price per month)	2	12	2000	1	48 000
Total asphalt laboratory					**403 000**
5.Production and transportation to the dam of transition zone					
Production of (0-60 mm) from screened tunnel rock + 20% over consumption	1,2	30000	40		1 440 000
Transportation to dam in lorries/dumpers, unit price inclusive driver 3 lorries, 10 hours per day, 240 days.	3	240	10	200	1 440 000
Total transition zone					**2 880 000**
6.Placement of asphalt					
exclusive of specialized contractors supervision and machinery					
5 operators/labourers, min 3 skilled, 240 days in total	5	240	10	8	96 000
Hire of 1 pay loader Cat 980 or similar without operator, unit price per hour 10 hours per day , 240 days	10	240		700	1 680 000
Operator for above, unit price per hour	10	240		10	24 000
Hire of 1 excavator Cat 325 without operator, unit price per hour	10	240		500	1 200 000
Operator for above, unit price per hour	10	240		10	24 000

Hire of 3 small rollers (2,5 - 3 tons) , one spare, unit price per year Investment per roller150000 NOK	3	0,2	150000	1	90 000
2 tippers with operators for asphalt delivery to dam, unitprice per hour 2 lorries, 10 hours per day 240 days	2	240	10	200	960 000
Repair and maintenance per year for above units					60 000
Gas (propane), for paving maskin 2 bottles per day, 240 days	2	240		200	96 000
Diesel for above units 5 lorries, 3 pick ups, 2 pay loaders , one excavator , one core paver , 3 rollers, total consumtion 370 liters per hours effective 6 hour/day	300	240	6	8,9	3 844 800
Lubricants for all machinery units					50 000
Total cost placement exclusive specialized contractor cost					**4 230 000**
7.Administrativ services					
4x4 pickups for supervisory personnel , 3 vehicles	3	0,25	300000	1	225 000
One 4x4 pickups for laboratory	1	0,25	300000	1	75 000
Running expenses (5000 per month per vehicle)	4	12		5000	240 000
Sum					**540 000**
Specialized contractor, hire of specialized machinery, personnel and administration					
Lump sum for one year services on site + preliminary costs, excluding tax/ custums					**16 000 000**
Sum net					**42 439 940**
Contingency 15%	15 %				**6 365 991**
Total					**48 805 931**

Appendix H

Layout of Option A and B

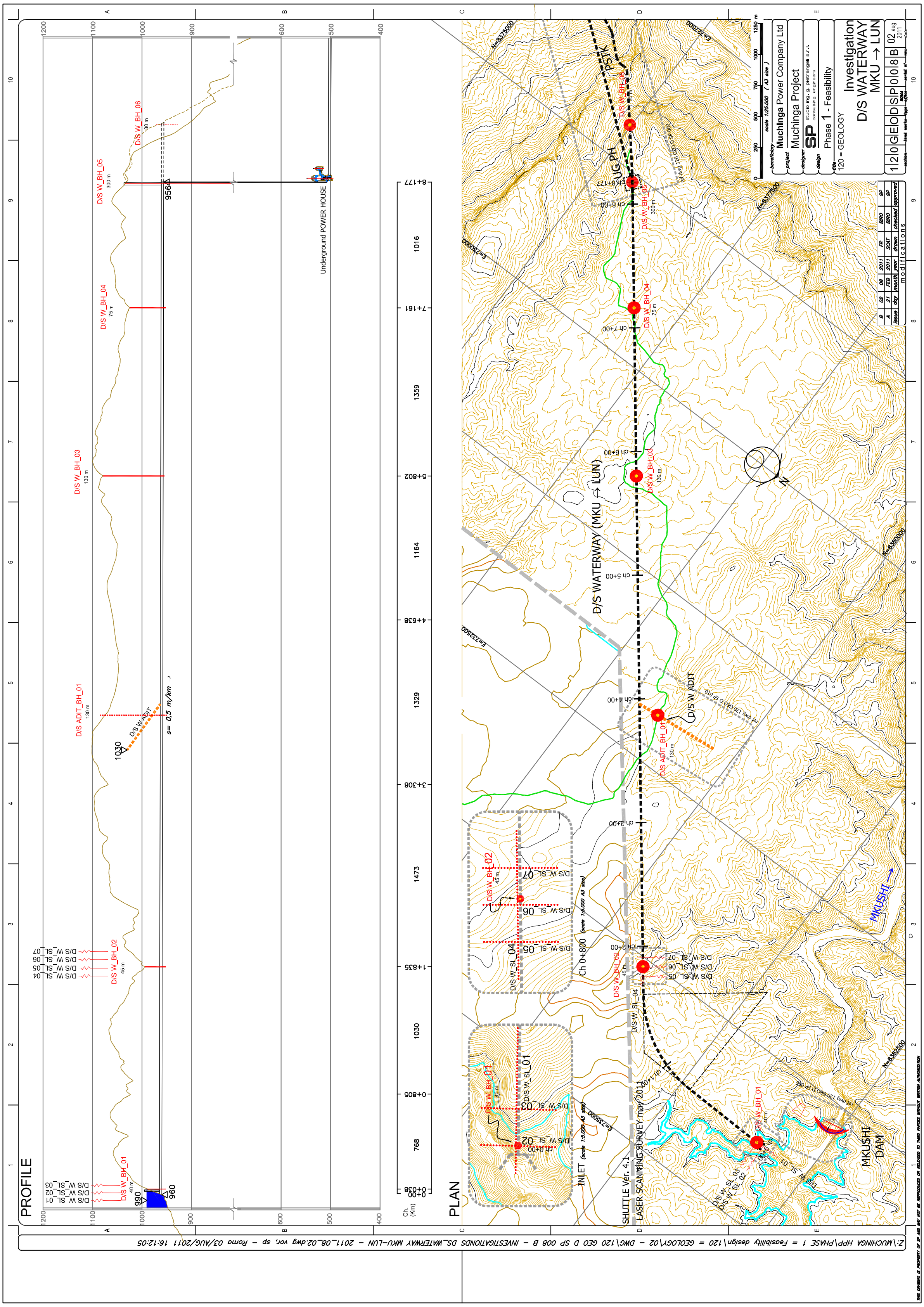

Figure H.1: Headrace layout [25]

Appendix I

Calculation of head loss

To calculate the total loss in the waterways the Darcy-Weissbachs equation (see I.1) is used for the penstocks only, as the Manning formula (see I.2) is applied for the tunnels and the pressure shaft. Since friction losses in tunnels and shafts tend to dominate the total hydraulic loss [10], the minor losses is neglected for simplification.

$$h_f = f * \frac{L * Q^2}{D * 2g * A^2} \tag{I.1}$$

$$h_f = \frac{L * Q^2}{M^2 * R^{4/3} * A^2} \tag{I.2}$$

Where:

h$_f$ Friction loss

f Friction coefficient

L Length of tunnel

Q Discharge

D Diameter

g Gravity of Earth, 9,81 m/s^2

A Area

M Manning number

R Wet perimeter

The friction coefficient was in this case found using an approximation of the Colebrook-White equation (see I.3). It could also be found using the Moody diagram.

$$\frac{1}{\sqrt{f}} = -2 * Log(\frac{k}{3,7 * D} + \frac{5,1286}{Re^{0,89}}) \tag{I.3}$$

Where:

k Absolute roughness

Re Reynolds number, Re=D*V/ν

V Velocity

ν Kinematic viscosity, 1*10^{-6} at 20° C

Option A

Tunnel	Area	Manning	R	Q	L	Friction loss	
	19,6	64	1,25	57	8650	13,23	m

Penstock	Area	Manning	R	Q	L	Friction loss	
(3 pipes)	3,80	85	0,55	19	1300	21,88	m
	f	**Re**	**k**	**D**			
	0,0097	11000000	0,045	2200			

Total	Friction loss	
	35,11	m

Option B

Headrace tunnel	Area	Manning	R	Q	L	Friction loss	
	40	34,4	1,676	57	8200	7,07	m

Pressure shaft	Area	Manning	R	Q	L	Friction loss	
	18	60	1,197	57	455	1,00	m

Tailrace tunnel	Area	Manning	R	Q	L	Friction loss	
	40	34,4	1,676	57	1540	1,33	m

Total	Friction loss	
	9,39	m

Appendix J

Option B

Optimization of Option B

Head loss coefficient is the efficient time the power production is operated at its maximum, to produce the same head loss over a year where the operation rate varies. It is calculated by the following equation:

$$T' = \sum \frac{Q^3}{{Q^3}_{max}} \tag{J.1}$$

Equation J.1 is used in context with the running of the power plant, see figure J.1.

Running of power plant

Depending on the season, the running of the power plant follows approximately the following illustrated trend lines*:

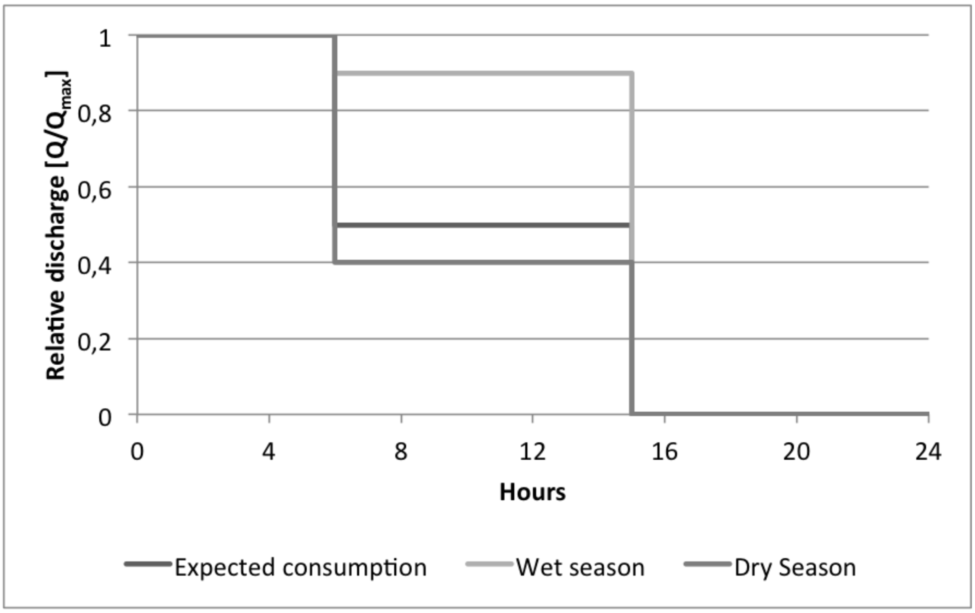

Figure J.1: Expected consumption and running of power plant

Surge tank/chamber is neglected in the headrace tunnels of both options since this does not propose the biggest difference. In later assessments of this project, when the level of detail needs to be more accurate, the need of a surge chamber in the tailrace tunnel should be investigated. This will however not be conducted in this report.

*Communicated by Lars Ødegård. April 2013.

Headrace tunnel

Manning	34	
Length	8200	
Head loss coeff	2600	hours
Q	57	m^3/s
Plant efficiency	0,85	
Price	0,09	USD/kWh
Present value factor	6,57	
Interest	15	%
Life expectancy	30	year

Area [m^2]	R [m]	Loss of energy [W]	Present value of the head loss [USD/m]	Total cost [USD/m]	Marginal cost [USD/m^2*m]	Marginal revenue [USD/m^2*m]
37	1,61	516,28	793,24	2896		
					40,3	54,5
38	1,63	480,84	738,79	2936		
					40,5	49,4
39	1,65	448,66	689,34	2977		
					31,5	45,0
40	1,68	419,37	644,34	3008		
					40,0	41,1
41	1,70	392,65	603,28	3048		
					41,9	37,5
42	1,72	368,21	565,73	3090		
					42,3	31,7
45	1,78	306,33	470,66	3217		

Pressure shaft

Q	57	m^3/s
Present value factor	6,57	
Price	0,09	USD/ kWh
Interest	15	%
Life expectancy	30	year
Plant efficiency	0,85	
Head loss coeff	2600	hours
Correction for length	1,25	
Addition	10 %	
Length	455,00	m
Manning	60,00	

Area [m^2]	Radius [m]	R [m]	Loss of energy [W]	Present value of the head loss [USD/m]	Total cost [USD/m]	Marginal cost [USD/m*m]	Marginal revenue [USD/m*m]
12,57	2	1,00	2716	4174	4137		
						2489	9562
13,85	2,1	1,05	2094	3217	4386		
						2611	7069
15,21	2,2	1,10	1634	2510	4647		
						2732	5299
16,62	2,3	1,15	1289	1981	4921		
						2854	4022
18,10	2,4	1,20	1027	1578	5206		
						2975	3088
19,63	2,5	1,25	826	1270	5504		
						3096	2396
21,24	2,6	1,30	670	1030	5813		

Underground powerhouse

To calculate the total volume of the underground powerhouse the following equation is being used [21]:

$$V = 78 * H^{0,5} * Q^{0,7} * n^{0,1} \tag{J.2}$$

Where:

V	Total volume
H	Net head
Q	Max discharge
n	Number of aggregates

With a maximum discharge of 57 m^3/s, net head of 480,6 m and three aggregates, the total volume is about 32.400 m^3.

NVE present the following estimations [21]:

Blasting	39 USD/m^3
Reinforcement	60 kg/m^3 concrete
Form work	2,1 m^2/m^3 concrete
Concrete volume	20 % of blasted volume
Rock support	15% of cost of blasting
Masonry works	5% of blasting and concrete cost
Decoration	15% of blasting and concrete cost
Unforeseen	10%
Rigging and operation	30%

This yields the following quantities and associated prices:

Table J.1: Total cost of the civil works of an underground powerhouse

Component	**Unit**	**Price [M USD]**
Blasting	32.400 m^3	1,3
Concrete	6.480 m^3	1,1
Reinforcement	390 kg	0,4
Form work	13600 m^2	1,1
Addition		0,7
Total		**6,3**

Access tunnel

Cross sectional area for access tunnels depend on type and size of the chosen aggregate solution. A normal value of the area however is 30-40 m^2 [21]. An area of this size yields an unit price for tunneling of approximately 5.035 USD/m including rock support, culvert and ventilation [21][7].

The length of the access tunnel may vary, depending on accessibility in the area and placement of powerhouse. Given the topography in the area, and a tailrace tunnel of 1540 m, an access tunnel of approximately 1500 m is assumed.

This yields a total cost of the access tunnel to be about 7,6 M USD.

Electrotechnical and mechanical installations

A simplified price estimation of the total cost of the electrotechnical and mechanical installations in the powerhouse, is estimated to be 51,3 M USD [21].

Appendix K

Result of Simulation

Based on the optimization of the waterways, the following input for simulation of production was established:

- Net Head equal to 480,6 m
- Energy equivalent equal to 1,113

Based on the stated demand of 95% reliability in the firm power supply, the following total firm power production of 1395 GWh was calculated. See figure K.1.

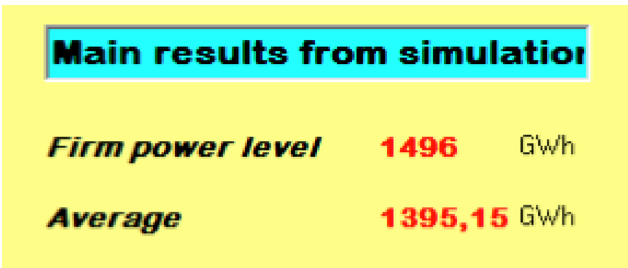

Figure K.1: Main results of simulation

MAIN SYSTEM DATA
DATASET : one year. File: C:\nMag2004\muchinga_Q_57_BYP_distr.set
RUN : Muchinga PP in Zambia
SIMULATION FOR YEARS 2001 TO 2008

NO	NAME	R e	P p	I t	C o	RESVOL Mm3	PMAX MW	QMAX m3/s	EEKV kWh/m3	ADR. 1 2 3	PROD GWH	UTIL %
1	Mita Hills	X				658.0	0.000	10000.0	0.000	2 2 2	0.0	0.0
2	Lusemfwa				X	0.0	0.000	10000.0	0.000	5 5 5	0.0	0.0
3	Muchinga		X			0.0	$\boxed{228.388}$	57.0	1.113	4 4 4	$\boxed{1395.2}$	$\boxed{93.4}$
4	Utløp				X	0.0	0.000	10000.0	0.000	0 0 0	0.0	0.0
5	Transfer C			X		0.0	0.000	39.0	0.000	3 4 4	0.0	0.0

(a) Total production

STATISTICS OF DEFICIT:
======================

DATASET : one year. File: C:\nMag2004\muchinga_Q_57_BYP_distr.set
RUN : Muchinga PP in Zambia

SIMULATION FOR YEARS 2001 TO 2008

YEAR / VOLUME OF DEFICIT:

2007	350.80
2008	167.27
2006	41.66
2002	20.54
2001	4.16

AVERAGE DEFICIT = 73.1 GWh/yr
DEMAND COVERAGE = $\boxed{95.1\ \%}$

(b) Demand coverage

WATER BUDGET FOR EACH MODULE IN THE SYSTEM:
DATASET : one year. File: C:\nMag2004\muchinga_Q_57_BYP_distr.set
RUN : Muchinga PP in Zambia
SIMULATION FOR YEARS 2001 TO 2008

NO	MODULE NAME	QLOCAL Mm3	QTOTAL Mm3	THROUGH Mm3	BYPASS Mm3	SPILL Mm3	SPILL %
1	Mita Hills	990.0	990.0	996.2	0.0	0.0	0.0
2	Lusemfwa	142.0	1138.2	1138.2	0.0	0.0	0.0
5	Transfer Channel	0.0	1138.2	835.3	224.6	78.3	6.9
3	Muchinga	507.0	1342.3	1253.5	0.0	88.8	6.6
4	Utløp	0.0	1645.2	1645.2	0.0	0.0	0.0

(c) Water balance

Figure K.2: Results form simulation in nMAG