# ANALYTICAL EVALUATION OF NATURALLY EXISTING SLIGHTLY WEATHERED ROCK AGAINST BLASTED ROCK AS EMBANKMENT MATERIAL

## (A CASE STUDY OF RIBB DAM)

MSc.THESIS

SELAMAWIT TESFAYE G/HANA

HAWASSA UNIVERSITY, HAWASSA, ETHIOPIA

OCTOBER, 2017

# ANALYTICAL EVALUATION OF NATURALLY EXISTING SLIGHTLY WEATHERED ROCK AGAINST BLASTED ROCK AS EMBANKMENT MATERIAL

## (A CASE STUDY OF RIBB DAM)

BY

SELAMAWIT TESFAYE G/HANA

ADVISOR

**Dr. MOLTOT ZEWDIE**

CO-ADVISOR

**Dr. MULUGETA DADI**

A THESIS SUBMITTED TO THE INSTITUTE OF TECHNOLOGY,

SCHOOL OF WATER RESOURCE ENGINEERING,

GRADUATE STUDIES, HAWASSA UNIVERSITY

HAWASSA, ETHIOPIA

IN PARTIAL FULFILLMENT OF THE

REQUIREMENTS FOR THE DEGREE OF

MASTER OF SCIENCE IN DAM ENGINEERING

OCTOBER, 2017

## SCHOOL OF GRADUATE STUDIES

## HAWASSA UNIVERSITY

## ADVISORS APPROVAL SHEET

This is to certify that the thesis entitled “Analytical evaluation of naturally existing slightly weathered rock against blasted rock as embankment material (a case study of Ribb dam” submitted in partial fulfillment of the requirements for the degree of **Master’s** with specialization in Dam Engineering, the graduate program of the **School of Water resource Engineering,** and has been carried out by Selamawit Tesfaye G/Hana Id. No PGDENG /012/08, under our supervision. Therefore we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

| | | |
|---|---|---|
| Name of Major Advisor | Signature | Date |
| Name of Co- Advisor | Signature | Date |

## SCHOOL OF GRADUATE STUDIES

## HAWASSA UNIVERSITY

## EXAMINERS' APPROVAL SHEET

We, the undersigned, members of the Board of Examiners of the final open defense by Selamawit Tesfaye G/Hana have read and evaluated his/her thesis entitled "Analytical Evaluation of Naturally Existing Slightly Weathered Rock Against Blasted Rock as Embankment Material (A Case Study of Ribb Dam)", and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfillment of the requirements for the degree of Master's of Science, Specialization in Dam Engineering.

| | | |
|---|---|---|
| ______________________ | ________________ | ________________ |
| Name of the Chairperson | Signature | Date |
| ______________________ | ________________ | ________________ |
| Name of Major Advisor | Signature | Date |
| ______________________ | ________________ | ________________ |
| Name of Internal Examiner | Signature | Date |
| Dr. Abdella K | ________________ | June 5, 2017 |
| Name of External Examiner | Signature | Date |
| ______________________ | ________________ | ________________ |
| SGS Approval | Signature | Date |

# Acknowledgement

First of all I would like to praise the almighty GOD and his virgin mother Saint Marry for giving me the strength and finish this thesis.

I would like to thank Dr.Moltot Zewdie for guidance and support through my process of thesis preparation.

Special thanks are extended to all my friends and Colleagues for providing me all the support, advice and guide regarding the Geoslope modeling and through my entire studies.

Finally, I need to express my sincere appreciation to my father Tesfaye G., my mother Tigist B., my sister Meron T. and my Brothers Abel T. and Dagim T., without their support this success will not be possible.

## Table of Contents

# Declaration

I hereby declare that this MSc thesis work is my original work and has not been presented for a degree in any other university or institute, and all sources of material used for this thesis have been duly acknowledged.

Name: Selamawit Tesfaye G/Hana

Signature:

selamawittesfaye2009@gmail.com

## Abbreviations and Acronyms

| | |
|---|---|
| AEL | Alluvium Excavation Level |
| $\alpha$ | Inclination of upstream slope |
| CCL | Construction Crest Level |
| C | Cohesion |
| E | Young's Modulus |
| ETB | Ethiopian Birr |
| ECGD | Earth Core Gravel Dam |
| FEM | Finite Element Method |
| $G_s$ | Specific gravity of rock |
| $\gamma_s$ | Specific unit weight of rock |
| H | Total head |
| $H_s$ | Significant wave height |
| kPa | Kilo Pascal |
| LEA | Limit Equilibrium Analysis |
| LEM | Limit Equilibrium Method |
| M.a.s.l | Mean above Sea Level |
| MDDL | Minimum drawdown level |
| MWL | Maximum Water Level |
| MPa | Mega Pascal |
| NPL | Normal Pool Level |

|  |  |
|---|---|
| $\phi^o$ | Friction angle |
| RBL | River Bed Level |
| USBR | United States Bureau of Reclamation |
| USACE | United States Association of Civil Engineers |
| USSD | United States Society on Dams |
| $\nu$ | Poisson’s Ratio |
| WWDSE | Water Works Design and Supervision Enterprise |
| Y | Elevation head |

## List of Tables

**List of figures**

**List of tables in Appendix**

**List of Figures in Appendix**

# Abstract

*Fundamentally, a good embankment dam design should not only refer to the one which is structurally stable and safe. However, it should also comprise of the actual site condition like use of easily available materials for construction purpose. Utilizing of locally available material for embankment dam will have economic advantages avoids construction difficulty and shortens construction periods. Ribb rock fill dam was designed and constructed with a significant volume of rock material produced by blasting and further degrading process to satisfy the gradation requirements. However, the site was rich with a slightly weather fine rock fill material which can easily be produced and stock pilled with normal production process of using machineries. Hence, this thesis presents the overall stability analysis for revised Ribb dam replacing the rock fill material with slightly weathered rock to show the effect of shallow geotechnical investigation works on dam projects. Numerical modeling software called GEO-Studio 2007 has been used for the analysis. Based on the analysis carried out for different loading conditions, the revised section satisfies all the requirements suggested by standards and improved factor of safeties of 1.801 for steady state seepage, 1.489 for sudden draw down and 1.396for steady state with earthquake has been found. Besides the newly revised design has been found economical since saves 497,536.439.19ETB, less difficult for construction and considerably shortens construction period.*

**Key Words: -***Ribb rocks fill dam, Stability analysis, weathered rock, Numerical modeling, and Geo studio software*

# 1 INTRODUCTION

## 1.1 Background of the study

Dams are generally constructed in river valley for different purposes like irrigation, water supply, power generation, navigation, recreation, flood control and so on. Although the age of first dam constructed by a man is not clearly known however, archeological proofs estimated to around four thousand years old. Dams have allowed people to collect water when it was ample and save it for dry seasons and they are important in modern society in conveying large economic values Jansen, (1988).At present construction of larger rock fill dams has become acknowledged throughout the world and in our country too for their safety, adaptability to widely varying site conditions, particularly of construction simplicity and economy(Jansen, 1988).

To make construction of dams economical and cost effective, intensive investigation of geological and geotechnical works has to be conducted. The purpose of conducting detail investigation on the foundation and construction material helps to obtain necessary information how local materials can be used for the purpose of construction. For dam project availability and suitability of earth material in nearby areas has to be investigated (USBR, 1989)

USBR, (1989) stated that Geological and geotechnical investigation helps to collect all relevant information concerning the site and its surrounding area like: -

- Foundation conditions of the dam site
- Physical properties of various construction materials
- Location and extent of soil and rock strata so that quantities , properties of soil and rock available for embankment construction would be determined before detailed studies of embankment sections are made.

In our country still attention is not given for geological and geotechnical investigations due to lack of awareness, shortage of finance, insufficient time to carry out a proper investigation, insufficient and inadequate supervision and lack of geotechnical expertise. The consequence of these problems is reflected on most dam projects in our country like Tendaho WWDSE, (2005), Kesem WWDSE, (2008), and Gidabo WWDSE, (2009) and so on. Ribb dam project found in South Gondar, Ethiopia is good example constructed from rock fill material which has tedious production process despite having a slightly weather fine rock material which can easily be produced with heavy duty machineries (WWDSE, 2010).

On this thesis, it is tried to show the effect of shallow geotechnical and geological investigation on dam designing and the economic advantage of using locally available construction materials for the construction of dam considering Ribb dam as a case study. According to the original design of Ribb dam WWDSE, (2010) it is zoned rock fill dam with total fill volume of 8.1Mm$^3$. Out of this total volume 3.5Mm$^3$is estimated to be slightly weathered fine rock, 2.5Mm$^3$is blasted rock and the rest accounts for is impervious core and filter materials.

## 1.2 Statement of the problem

Good dam engineering involves make use of materials available at the site. Availability of suitable construction material with in economic haul distance is the most important issue in the selection, design and construction of dams. Zoned rock fill dams mostly composed of central clay core, fine and coarse filter, fine rock and rock.

Due to lack of detail geotechnical investigation the rock fill material for Ribb dam is designed to be produced by blasting and further degradation processes which is time taking and uneconomical. However, during construction period it is found that the site was rich in slightly weathered rock which demands minimum essential processing and underestimated during investigation period.

Considering the effect of shallow geotechnical investigation on construction of embankment dams as identified gap and this thesis intended to show the advantage of using locally available material for construction purpose rather than searching for a material which demands a huge production process regarding to economy, construction time and construction difficulty, as using of easily available material will minimize unnecessary extra cost, avoid extended construction period and construction difficulty.

On the original design stress deformation analysis was not incorporated accordingly this has been found as another gap on Ribb dam projection therefore it has been included in this study.

## 1.3 Objective of the study

### 1.3.1 General objective

- Analyzing the stability of Ribb rock fill dam by replacing blasted rock fill material with easily available fine rock material

### 1.3.2 Specific objective

- To show economic comparison of using fine rock material in place of rock material
- To show how replacing of material minimizes construction completion period of the project
- To show the effect of shallow site investigation on dam projects

## 1.4 Research question

- Is it economical replacing blasted rock material with naturally available fine rock material for rock fill dam of Ribb?
- Will Ribb dam be structurally stable if constructed with fine rock material instead of blasted rock?
- Is construction difficulty eased if one replaces blasted rock with fine rock material?
- Does replacing fine rock in place of blasted rock shorten construction period?

## 1.5 Hypothesis

- Using naturally available fine rock material for construction of Ribb dam does not bring significant economic advantage on the overall project cost and completion period
- Using naturally available fine rock material do not bring significant change on project completion period
- Using naturally available fine rock material do not ease the construction difficulty
- Using naturally available fine rock material does not bring stability problem on the dam

## 1.6 Scope of the study

This thesis is intended to show advantage of dam designing with locally available construction materials rather than searching for a material which demands tedious production process and techniques. Thus this thesis covers the overall design of Ribb dam with locally available weathered rock fill material so that dam dimensioning, seepage modeling and evaluation, static stability, dynamic (Pseudo static) stability and economical evaluation are the main components of the work.

## 1.7 Significance of the study

At the moment construction of rock fill dam has become common practice in our country however, the design of dams especially found around Lake Tana basin are almost similar to one another regarding to their zoning and construction material used regardless of the actual site condition. But, it is understood that designs of two or more dams at different location would have a unique character due the variation of construction material, valley shape, geology, seismic condition etc… from site to site.

This thesis will consider Ribb dam project as a case study and tries to show some key components of a construction project like time; amount of money and unnecessary efforts wasted on this particular dam project due to lack of detail construction material investigation. Besides the outcomes of this analysis can be used as a lesson and input for the dams which going to be constructed for future especially around LakeTana basin (i.e. Jemma dam, Gumera dam and Gilgel Abay dam).

# 2 LITERATURE REVIEW

## 2.1 Geotechnical Investigation of dams

USACE, (1994) and Simons, (2002) stated that dam site investigation includes collection of all relevant information required for designing and construction purpose on the dam site and its surrounding area. These necessary data will be collected by geological and subsurface investigation at the dam foundation and borrow areas. Detail site investigation has to be the first step in planning, designing and construction of any dam projects and the investigation has to be done to the required depth to determine suitability of the foundation for the type of dam, the treatment required and to understand the dominant type of construction material available on the site(USACE, 1994 and Simons, 2002).Construction material assessment focused mainly on the suitability, the quantity and quality of construction materials within reasonable hauling distance (USBR, 1989). Poor site investigation will result great expenses, delay of the project and also construction difficulty. For dam projects, it is a normal practice that at least 3% of the total project cost should be deployed for detail site investigation activities (Briaud, 2013).

Although the construction of dam projects has increased in Ethiopia recently, many projects faces geological and geotechnical challenges during construction phase which leads the projects to extended construction time than expected. According to WWDSE, (2008) Kesem dam project, found in Afar region, Ethiopia was originally scheduled to be constructed in two years' time, but the construction actually take more than eight years due to the delay of foundation treatment work caused by geological complexity of the area which was not seen during investigation. Tendaho dam project is also affected by shallow geological and geotechnical investigation that unforeseen geological structures have been found during water impoundment of the dam in the rims of the reservoir. These problems lead the project to further treatment cost and extended construction period (WWDSE, 2005).

Ribb dam project, located in Amhara region, Ethiopia has been designed to be rock fill dam where the rock material is supposed to be produced in costly and tedious technique (i.e. extracted using blasting and then using jackhammer and excavator processed to the required gradation)as the geotechnical investigation report indicates that there is no other suitable material to the required volume which can be used as shell zone. However, during construction period it was observed that there was plenty of slightly weathered rock material which can be used as shell zone1.5 km from the dam site. This slightly weathered rock could be produced easily with machineries without blasting and other degradation process. This shows how poor site investigation has cost the project regarding to economy and unnecessary construction period (WWDSE, 2007).

Tarhini et al, (2015) has studied the causes of delay on different construction projects in Lebanon. He conducted his investigation by interviewing project managers and site engineers of the projects and made qualitative research. Based on his investigation among various cases for the delay, unavailability of construction material is the main factor. Abdella and Hussien, (2002) also studied that the main causes of delay for construction projects based on the information gathered from construction contractors, consultants and clients working on large construction projects in Jordan indicate among many reasons which brought delay in project: improper planning, poor geotechnical investigation, lack of construction material in good quality and quantity comes in front.

## 2.2 Construction material

According to Dunkin et al, (1987) and Narita, (2000) the materials used for dam's construction cover a wide range from clayey materials to rock and the availability of dam fills material influence dam design considerably. Rock is one of the most preferred fill materials for embankment dam recently use of different fill materials such as sand-gravel has been practiced. Embankment dams are constructed of all types of geologic materials, with the exception of organic soils and peats. Satisfactory construction materials may be obtained from considerable excavation of foundation and spillway or outlet works. When estimating available construction material quantities from borrow area large safety factor should be given because as there may be estimation error and other unforeseen event (Dunkin et al, 1987and Narita, 2000).

The qualifications for pervious rock fill sections generally require that rock has to be sound, well graded and free draining. Clean rock materials with rock content of 60 to 70 % fill materials have sufficient rock to rock contact with the strength of the rock controlling the shear strength rather than the soils or fines (Penman and Charles, 1976 and USBR, 1998).

Dirty rock fill with a hydraulic conductivity less than$1x10^{-3}$ cm/sec may be considered as earth fill because the possibility of developing construction pore pressures, more pervious material may be regarded as clean rock fill. And also for rock to be used as construction material the compressive strength should be with the range between high compressive strength i.e. 70 to 200 MPa, medium compressive strength 17.0 to 70 MPa and low compressive strength 3.5 to 17.0 MPa (USSD, 2011).

According to USSD, (2011) compacted unit weight, hydraulic conductivity, gradation, shears strength and deformation and compressibility are some of the important properties of rock fill material. Compaction is achieved from the traffic of loaded trucks and spreading dozers supplemented by passes of a heavy vibratory roller or other compaction equipment. Rock fills with rock contents of 60 to 70 % can generally be considered free draining shell material. For modern dams, maximum particle size ranges 46 to 122 cm, grading down to fines with 20 to 40% passing a 2.54 cm sieve and 5 to 15 % passing a No. 4 sieve. Experience indicates moduli range from 27 MPa to 128 MPa, depending on the nature of the rock, the grading of the rock fill, lift thickness, compaction and other factors (Cooke, 1990).

Turkmen et al, (2013) studied the effect of construction material on dam type selection and the investigation was carried out on Buyuk Karacay dam in south Turkey. Based on the study the dam was initially designed to be impervious cored rock-fill dam with impervious core however, the borrow area delineated for impervious material during prefeasibility study was turned to urban areas where resettlement could make the project uneconomical. Besides other borrow area for impervious zone could not been found within economical distance so that other types of dams have been considered as an option and concrete face rock fill dam has been selected as best alternative. This study shows that dam designing should be done based on the actual site condition as well as availability of material and the final dam type should be the one which is least expensive.

Good and Parsons, (1985) studied on the suitability of weak rock for the construction of rock fill dam on two different dams. The investigation was carried out on; Kangaroo Creek and Little Para rock fill dams which are 59m and 53m high respectively constructed in Australia. Based on their investigation until the construction of these two dams, hard and durable rock was preferable for rock fill embankment. But Kangaroo Creek was constructed using soft schist and Little Para was constructed with weak dolomitic shale. After investigating the main excavation sites and the principal source of rock, Kangaroo Creek dam site was found that schist was the dominant type of rock. After conducting a series of unconfined compression tests the results indicate that the schist averaged only a third of the strength of the other rock types. Good and Parsons, (1985) stated that other tests were done to determine the effect of saturation on schist and a 30 % loss in strength occurred. If the schist proved to be unsuitable for rock fill it was likely that the site would have to be abandoned. Similarly, in Little Para dam site a number of quarry sites were studied and finally except a single quarry at downstream of the dam site found with a thick band of quartzite, and the remaining quarries consist of a weak dolomite and dolomitic shale with various degree of weathering. As indicated on the study with careful design and construction with material found locally, both dams have performed well in service According to the studies of Good and Parsons, (1985) as far as the required consideration has been taken and implemented all there is no bad material in any construction site.

Refering to the study of Mackenzie and McDonald, (1985) on Mangrove Creek 80m high concrete face rock fill dam in Australia, the dam was constructed largly from low strenght sand stones and siltstones available on site with minimun processingcost. This study showed that soft rocks, usually considered as unsuitable for the construction of large dams with steep slope faces has been disproved and as far as proper design considerations has been made wide ranges of earth materials can be used for damconstruction pourpose.

Wilson and Evans, (1991) studied about Road Ford Dam which is 4lm high rock fill dam in England, constructed of low-grade rockfill materialwith asphaltic concrete membrane on its upstream face as waterproofing element. Site investigations with an exploratory quarry and trial embankmentswas carrid out and the result shows, the rock at the dam site was of low grade and weatherd. This taken into account when designing the embankment and a review of the results indicated that an embankment could be constructed of the less weatherd materials.

## 2.3 Classification and selection of dam type

Embankment dam are constructed of earth and rock fill material and they are most of the time zoned to accommodate and use as much material as possible from structural excavation of appurtenant structures and from borrow areas with economical hauling distance, least waste, minimum production processing and stockpiling, maintain stability and control seepage (USACE, 2004 and Narita, 2000).

Dams with the height greater than 15m are categorized as high dams and about 14,000 high dams have been registered up to the present and more than 70%of them are embankment dams. A recent report on the construction of high dams has also noted that among a number of high dams constructed recently, the majority (i.e. 80 %) of the total dams are embankment and the rest 20 % are accounted for other type of dams(Narita, 2000).

According to Narita, (2000) and Tancev, (2005) embankment dams are favorable as compared to other types of dams for their unique nature that these dams can even be constructed on alluvial deposit and pervious foundation which is totally an acceptable type of foundation for rigid dams. Besides use of easily available material, their flexible nature during earth quake, suitability in wide and flat terrain and easy workman ship are among the advantage which make them first choice.

Based on the construction material used, embankment dams are classified in to two major categories: Earth fill dam and Rock fill dam. Embankment dams are most common type of dam since they are constructed using materials from the required excavations and locally available natural material that requires minimum processing. Embankment dams may be designed with any of a wide range of cross sections and compositions. Although Embankment can accommodate difficult site conditions, they still must be provided with foundation support that will keep deformations within acceptable limits (USBR, 1986 and Jansen, 1988).

In present practice, embankment dam is likely to be designed with a core composed of relatively impervious material enclosed by coarser shells. The impervious part of an embankment serves as a water barrier during life time of the dam and the coarse shell serves structural stability for the dam. Embankment zoning is an important technique to safeguard the embankment against failure. While in embankment zoning proper attention should be given to control seepage lose and erosion of the embankment material through the dam body or through the foundation (Jansen, 1988).

Rock fill in its various forms-dumped, compacted in layers, hand-placed cobbles and masonry, equipment-placed masonry, and wet masonry-has been known since ancient times as a useful, reliable, and durable construction material. Rock fill dams have been successfully constructed to require heights and it is evident that well over 300 m high dam (Rogun dam, 335 m high earth core gravel dam (ECGD) constructed in USSR) can now confidently be designed and constructed (Jansen, 1988).

Moreover USBR, (1992) stated that rock fill dam construction has increased significantly since 1960 and is attributed to the utilization of more remote sites, more economical quarrying and placing operations, the use of excavated material in random zones, better details design, and more general knowledge concerning rock fills.

Based on USBR, (1992) rock fill dams has been proven to be economical and best choice in conditions where rock materials are available during excavation of appurtenant structure, when enough earth fill material is not available on site, when the dam is constructed in wet climate so that placement of earth material is limited and when other dam types are not economically feasible.

## 2.4 Embankment dam selection and design criteria

Turkmen et al, (2013) stated that the quality, quantity, ownership status, hauling distance and accessibility of natural construction material affect the type of dam and also the engineering features of natural construction materials are of the most important factors in the selection of dam type.

As USBR, (1986) recommends the choice of dam type should be done based on adequate study to prevent unnecessary expenses. Selection of the type of dam should be determined by considering various factors like: -

- Topographic considerations: include the surface configuration of the dam site and the reservoir area, accessibility to the site and availability of construction materials. Topography dictates the first choice of type of dam. A Narrow stream flowing between high, rocky walls would naturally suggest a rock fill dam. On the other hand; the low rolling plains would suggest an earth fill dam (USBR, 1986 and Dunkin et al, 1987).
- Geologic considerations: are the determinations of the various types of rock and soil also their suitability as foundation and construction materials. The foundation geology at a dam site often influences the type of dam suitable for the site. Bearing capacity of the foundation, strata thickness, inclination of geological features, water tightness, geological discontinuities, geological structure (fault and fold) are all important on the dam type

selection (USBR, 1986 and Dunkin et al, 1987).

- The availability of materials: for dams of various types, near or at the site determined the type of dam to be selected. Avoiding or reduction of transportation expense for construction materials, particularly those which are used in great quantity will affect considerable reduction in the total cost of the project. The most economical type of dam will often be the one for which materials are to be found in sufficient quantity within a reasonable haul distance from the site. Embankment dams are designed to make the best use of the closet, available and suitable materials (USBR, 1986 and Dunkin et al, 1987).

- Site seismicity: also affect the type of dam suitable for the site. Where foundation movement is expected, a rock fill dam is suitable than an earth fill dam. If the dam lies in an area that is subjected to earthquake shocks, the design must be done in the manner that which can resist the effect of the added loading and increased stresses (USBR, 1986).

- The size, type and location of spillway are controlling factors in the selection of the type of dam (Dunkin et al, 1987).

According USBR, (1987) the design of embankment dams should consider and fulfill the following basic criteria:-

- Embankment dam must be safe and stable during all phases of construction and operation of the reservoir
- The embankment must be safe against overtopping by wave action
- Embankment must be safe against overtopping during occurrence of the inflow design flood by the provision of sufficient spillway and outlet works capacity
- Seepage flow through the embankment foundation and abutment must be controlled to prevent excessive uplift pressure and piping
- The slopes of the embankment must be stable during construction and all conditions of operation, including rapid draw down of the reservoir
- The embankment, foundation, abutments and reservoir rim must be stable and must not develop unacceptable deformations under all loading conditions brought about by construction of the embankment, reservoir operation, and earthquake

## 2.5 Dam zoning

Zoning should contain appropriate impervious zones, transition zones between the core and the shells. It should be done to use as much material as possible from the required excavation and from borrow areas with the shortest hauling distance and the least waste and must be done based on the estimated quantities of required excavation and potential of borrow and quarry source (USBR, 1992).

According to USBR, (1992) towards the outer shell zone, the permeability nature of the embankment should have to increase. In the downstream section, pervious materials are placed to avoid building up pressure from percolation of water and to permit lowering the phreatic line so that it will remain inside the dam body. Furthermore, pervious materials also place in the upstream section to permit dissipation of pressure for sudden draw down condition.

## 2.6 Stability analysis of Embankment Dam

### 2.6.1 Static slope stability analysis

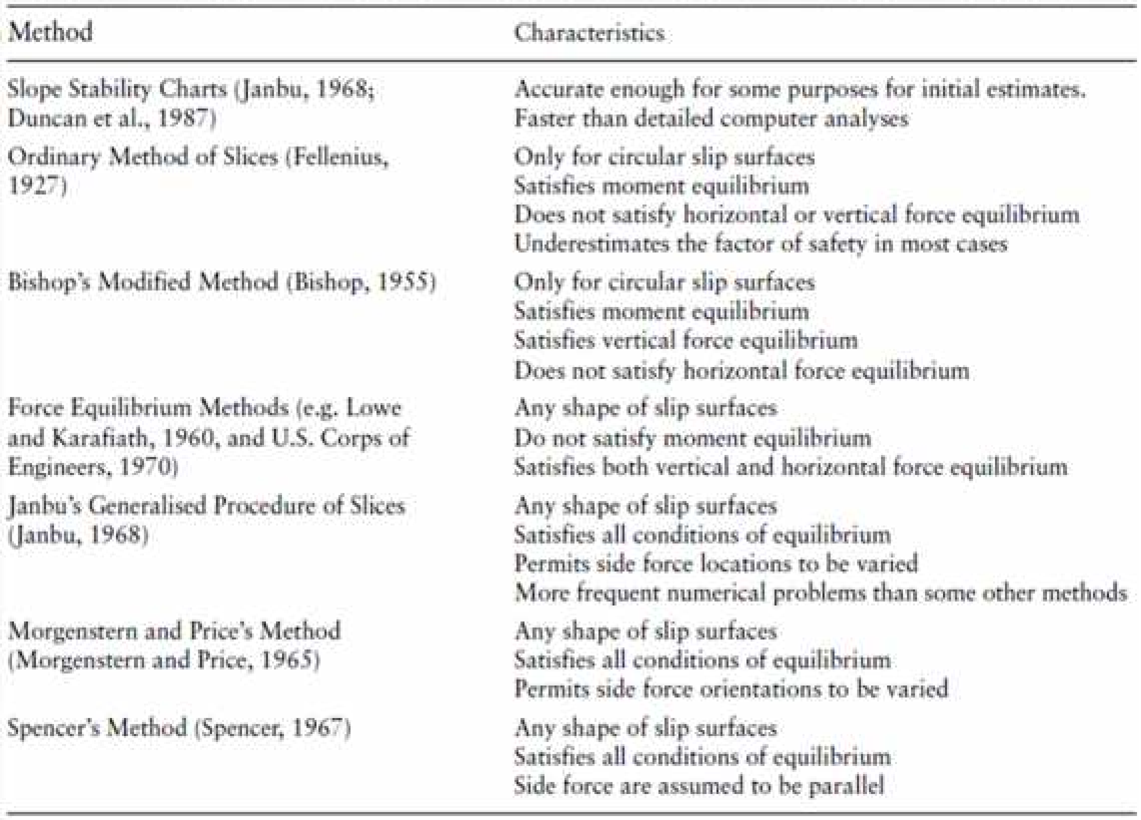
According to Stematiu, (2006) static stability analysis will help to answer basic questions like how the safety of the structure will be and the serviceability of the dam regarding to deformation or slope failure. The basis of the design of earth and rock fill dams is focused on ensuring the stability of the structure under a set of conditions expected to occur during its life. To carry out the analysis there are two methods: the limit equilibrium method (LEM) and finite element methods (FEM).

Limit equilibrium types of analysis have been in use in geotechnical engineering for a long time and are now used routinely in geotechnical engineering practice. Modern graphical software tools have made it possible to gain a much better understanding of the inner numerical details of the method. The fundamental shortcoming of limit equilibrium methods, which only satisfy equations of statics, is that they do not consider strain and displacement compatibility. This limitation can be overcome by using finite element computed stresses inside a conventional limit equilibrium framework. From the finite element stresses both the total shear resistance and the total mobilized shear stress on a slip surface can be computed and used to determine the factor of safety (SLOPE/W, 2008).

Limit equilibrium is a method developed to evaluate adequately the safety against failure for the conditions during and immediately after construction, steady seepage, rapid drawdown and first filling of the reservoir. Characteristics of limit equilibrium methods summarized in table 2-1 and these methods seek to express safety quantitatively by a single number - factor of safety (SLOPE/W, 2008 and Stematiu, 2006).

The analysis of the stability of dams and other slopes is usually carried out using Limit Equilibrium Analysis (LEA). If it is necessary to model the deformations of the dam or the effects of strain weakening, as in progressive failure, numerical methods must be used (SLOPE/W, 2008).

Table 2-1: Characteristics of limit equilibrium methods of slope stability analysis (Duncan 1992)



| Method | Characteristics |
|---|---|
| Slope Stability Charts (Janbu, 1968; Duncan et al., 1987) | Accurate enough for some purposes for initial estimates.<br>Faster than detailed computer analyses |
| Ordinary Method of Slices (Fellenius, 1927) | Only for circular slip surfaces<br>Satisfies moment equilibrium<br>Does not satisfy horizontal or vertical force equilibrium<br>Underestimates the factor of safety in most cases |
| Bishop's Modified Method (Bishop, 1955) | Only for circular slip surfaces<br>Satisfies moment equilibrium<br>Satisfies vertical force equilibrium<br>Does not satisfy horizontal force equilibrium |
| Force Equilibrium Methods (e.g. Lowe and Karafiath, 1960, and U.S. Corps of Engineers, 1970) | Any shape of slip surfaces<br>Do not satisfy moment equilibrium<br>Satisfies both vertical and horizontal force equilibrium |
| Janbu's Generalised Procedure of Slices (Janbu, 1968) | Any shape of slip surfaces<br>Satisfies all conditions of equilibrium<br>Permits side force locations to be varied<br>More frequent numerical problems than some other methods |
| Morgenstern and Price's Method (Morgenstern and Price, 1965) | Any shape of slip surfaces<br>Satisfies all conditions of equilibrium<br>Permits side force orientations to be varied |
| Spencer's Method (Spencer, 1967) | Any shape of slip surfaces<br>Satisfies all conditions of equilibrium<br>Side force are assumed to be parallel |

Stematiu, (2006) stated that, finite element method is more flexible and used for predicting displacements and stresses, for seepage evaluation and for time dependent consolidation analysis. There are features unusual to fill dams which can be modeled by the finite element method. In the process of the analysis certain idealizations and assumptions have to be made about the geometry, loading and the materials properties.

Embankment dam is subjected to various loading conditions in its life time and the response of the dam to such varying conditions can be quite different so is necessary to perform separate analysis for each of them. Currently, stability analyses are by far the most common type of numerical analysis in geotechnical engineering (SLOPE/W, 2008).

#### 2.6.1.1 Static Loading Condition

Based on USBR, (1987) earth and rock fill dams must withstand very different loading conditions that arise during construction and following operations. Loading conditions to be examined should be based on knowledge of construction plan, the emergency and maintenance operation plans, the flood storage and release plan of the reservoir along with the behavior of the embankment and foundation materials with respect to the development of pore pressures in the dam and foundation. For embankment dams, it is necessary to examine the stability of both the upstream slope and the downstream slope, for the most adverse conditions of loading.

**Construction Conditions**

According to USBR, (1987) end of construction condition, as necessary, partial completion of fill conditions, depending on construction schedule and relationship of pore pressures with time should be analyzed. When it is a question of an earth fill or rock fill dam, for the upstream and downstream slope, the state immediately following construction of the dam is critical.

**Steady State Seepage Conditions**

The stability of the downstream slope should be analyzed at the reservoir level that will control the development of the steady-state phreatic surface in the embankment. This reservoir level is usually the top of active conservation capacity but may be lower or higher depending on reservoir operations (USBR, 1987).

Seepage in embankment dams basically analyzed with steady state condition based on the assumption that the variable of time is ignored and the reservoir assumed to be constant for long term to result a stable flow regime. This assumption is somewhat conservative as the water level in the reservoir fluctuates with time and even some dams may not reach to steady state for decades. But with all this limitation this approach has been accepted as an appropriate approach for analyzing of flow quantity, flow gradients and pore-water pressure (USBR, 2014).

### Operational Conditions

If the maximum reservoir surface is substantially higher than the top of active conservation surface the stability of the downstream slope should be analyzed under maximum reservoir loading. The upstream slope should be analyzed for rapid draw down conditions from the top of active conservation capacity water surface to the top of inactive capacity water surface and from the maximum water surface to the top of inactive capacity water surface (USBR, 1987).

### 2.6.2 Upstream slope protection

The upstream and downstream of an embankment should properly be protected against damaging loads which may cause due to wind, wave action, weathering and deterioration, damages from ice as well as floating debris. For upstream slope the most common and cheapest way of slope protection is use of dumped rock (riprap).Usually the upstream embankment slope should be protected in the elevation between maximum water level and minimum draw down as the water in the reservoir fluctuates between these levels (USBR,1992).

According to USACE,(2004) there are different ways of protecting slopes of an embankment dams like dumped riprap, precast and cast-in-place concrete pavements, soil cement, bituminous stabilization, sodding and planting. Moreover, Selection of slope protection method should be based economy and availability of construction material.

# 3 MATERIAL AND METHODS

## 3.1 Description of study area

Ribb Dam is located in the Ribb River, on the eastern side of Lake Tana Basin, in South Gondar Amhara National Regional State. Lake Tana Basin, is one of the major agricultural areas of the country. However, this potential area is under threat because the everincreasing devastation of the natural vegetation, the steep slopes, and traditional land management practices, poorly adapted to land conservation under the prevailing conditions, have resulted in dramatic soil erosion in the area(WWDSE, 2007).

### 3.1.1 Location of the project area

The dam site is found on specific geographic grid reference location bounded between N $12^0$ 02' 30'' and E $37^0$ 59' 45''(Fig.3-1), at an altitude of 1880 m to 1970 m. The left abutment is situated at an altitude of 1943 m, the centre of the dam axis is situated at an altitude of 1873 m, and the right abutment is situated at an altitude of 1966 m. Access to the dam site is possible from the town of Addis Zemen using the existing dry weather road, which is about 40 km long (WWDSE, 2007).

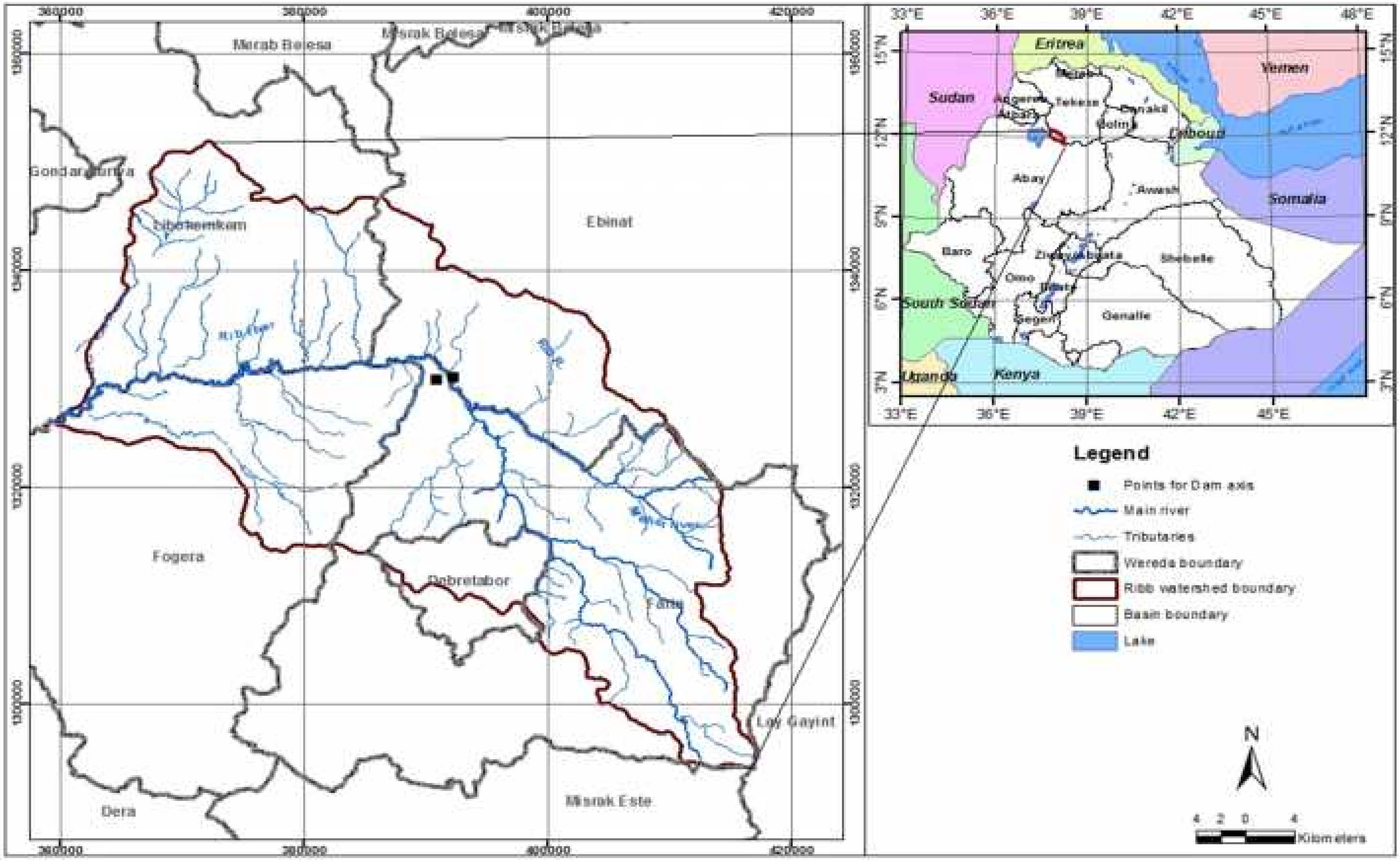


Figure 3-1 : Location map of project area

### Topography

The dam site is characterized by broad and flat flood plains, old bench forming terrace and low to high relief basaltic hills with steep to moderately steep slopes. The right and left abutments of the dam are characterised by steeper slopes with slope angle of 35° to 46°. There are developments of relatively few shallow seated gullies at the reservoir catchments attributed to gully erosion. The peak topography in the area is marked by Shikra Hill, which is at an altitude of 1973 m. The Upper Ribb watershed is characterized as a mountainous, wedge-shaped and steep-sloped (3.6 %) watershed. The highest elevation of the watershed is about 4,100 m in its southeastern part and the lowest topography land is at the dam site, which is at an altitude of 1873 m.(WWDSE, 2007).

### 3.1.2 Silent feature of Dam and reservoir

Ribb rock fill dam is zoned dam containing impervious clay core, transition zones between the clay core and fine rock fill and blasted rock fill material (fig.3-2). The overall estimated volume of Ribb rock fill dam is about 8.1Mm$^3$ as shown in detail in table 3-1(WWDSE, 2010).

Table3-1over all fill volume of Ribb dam (WWDSE, 2010)

| **Fill material** | **Volume in m$^3$** |
|---|---|
| Clay core | 1,118,948.73 |
| Slightly weathered fine rock | 3,455,339.31 |
| Blasted rock | 2,505,367.24 |
| Gravel(alluvium) | 112,624.79 |
| Fine filter (chimney) | 248,279.00 |
| Fine filter ( blanket) | 87,017.50 |
| Fine filter cutoff | 17,832.10 |
| Coarse filter chimney | 254,265.55 |
| Coarse filter blanket | 181,724.06 |
| Coarse filter cutoff | 20,224.02 |
| Riprap | 140,557.09 |
| **Total fill volume** | **8,142,179.39** |

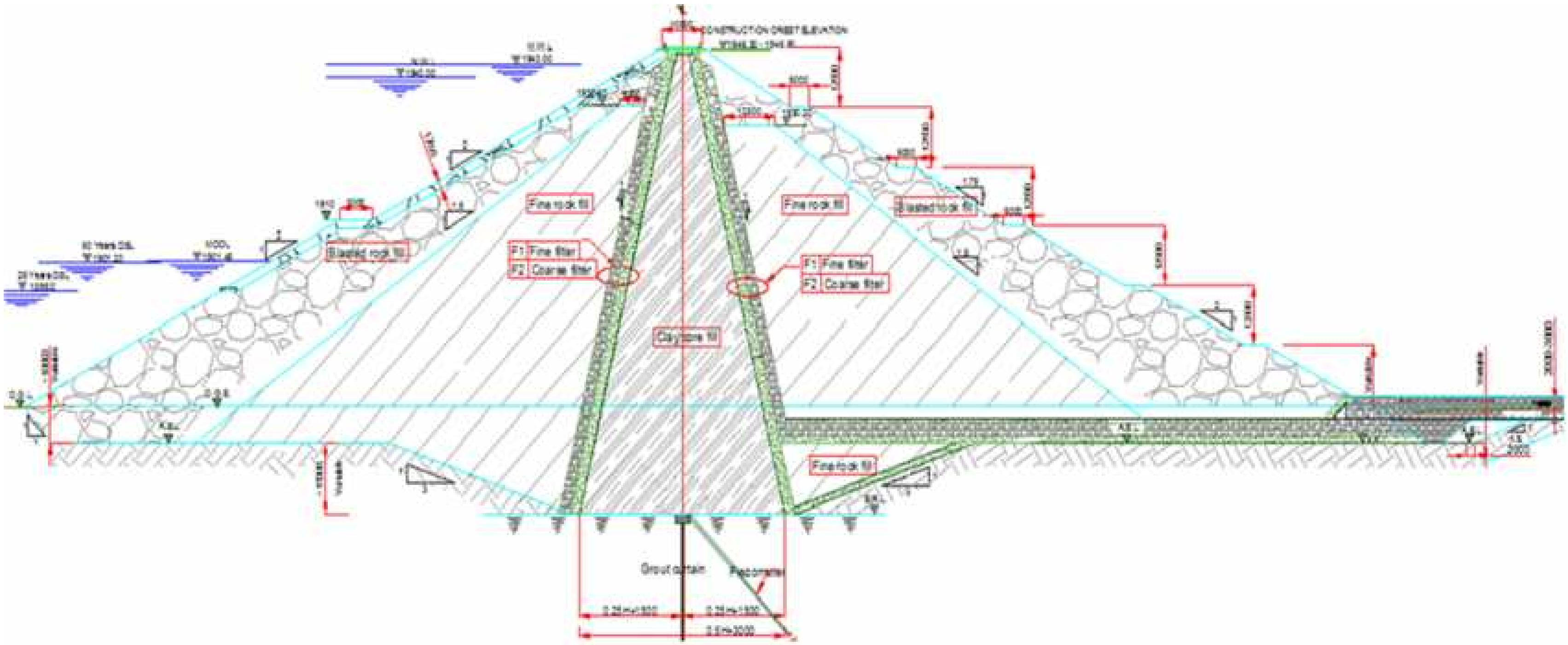


Figure 3-2 Original Dam cross section of Ribb dam (WWDSE, 2010)

### 3.1.3 Geology

The geology of the Ribb Basin is dominated by a huge volcano named Guna Terara. It corresponds to the eruptive events that have occurred during the early Miocene to Pliocene periods, and is classified in the shield group basalt. The common lithotype for this material refers to lenticular basalt with large amount of interbeded scoriaceous lava and basalt agglomerates. Some paleosoils may be interbeded. The other smaller volcanoes located at the north are also considered being active during the same geological period. The lower part of the valley before Lake Tana is completely overlain by recent fluvial depositions, which are mainly formed by silt to clay deposits. Recent volcanic flows have also been noted but they appear to be localized in the lower section of the Ribb plain. (WWDSE, 2007).

### 3.1.4 Climate

The climate of the Ribb Basin is marked by a rainy season from May to September, with monthly rainfall varying from 65 mm in May to 411 mm in July. Temperature variations throughout the year are minor (19 ºC in December to 23 ºC in May), with maximum and minimum temperatures of 30 °C and 11.5 °C, respectively. Humidity varies between 70 % in December and 88 % in August. Wind speed is low, thus minimizing potential evapotranspiration values between 95 mm/month in December and 140 mm/month in April. Sunshine duration is reduced to 6.0–6.5 hr during July and August (WWDSE, 2007).

### 3.1.5 Availability of construction material

Dam should be designed to maximize the use of locally available construction materials. Significant volume of rock fill material for a dam will come from required excavations like from spillway, dam foundation, inlet and outlet works etc. and this will have economical advantag over the effective cost of the rock fill.

According to the geological and geotechnical investigation report of Ribb dam WWDSE, (2007)several borrow areas has been identified for core material which are classified as medium to very high plastic and some borrow areas are also found to be swelling type soil. The Soil samples are highly impervious as their clay content increases and some of the samples at a depth show gravely clay which is semi impervious and most of the soils are non-dispersive that it will not have erosion problem.The relatively better core material have been selected from available borrow areasbased on the laboratory test resultsand found at 1.5 km from the dam site.Potential filter material source have been identified within the reservoir area, 2 km upstream and 13 km away from the dam site.

During feasibility study it was planned to use shell fill material from the expected large alluvium excavtion from the dam foundationbut during construction period,however it was found that the excavted material composed of highly plastic clay soil predominantly black cotton soil with limited volume of sandy gravel material.Hencenew shell quarry has been identified inorder to endure the construction as per the design during re-assesment of material for shell zonenewquarries with high potential have been identified in the visinity of the dam site 1.5 km from it.The alternative sources for rock fill material are blasted aphanatic basalt and ignimbrite from nearby hills 7 km from the dam site.

Riprap is commonly specified material for slope protection and protection against wind and wave erosion. So, dumped riprap which is placed on the upstream section of the dam should be sound rock with sufficient weight to withstand the action of waves, freezing and thawing. This material extracted from ignimbrite rock obtained within the vicinity of the project from rock quarry sites and boulders from Ribb river channel at 2 km and 7 km from the dam site (WWDSE, 2007).

Photo 3-1 : Material production at Ribb dam project (A) slightly weathered fine rock (B) blasted rock

## 3.2 Methods

The aim of this thesis is to show the effect of poor and shallow site investigation on dam projects considering Ribb dam project as case study and the overall procedure followed in the study has been stated as follow.

### 3.2.1 Data collection and analysis

**Primary data:**

- Physical observation on the production of consruction materials
- Relevant photos has been taken on site that have been essential for the study

**Secondary data:**

- Geotechnical parameters like unit weight,cohesion,internal anagle of friction , hydraulic conductivity and pore water pressure ratio for required fill materialshas been collected from geotechnical investigation report, detail dam design and project drawings
- Some missing paramaters required for stress deformation analysis has been collected from typical studies of identical works

#### 3.2.1.1 Materials used for different analysis

Geotechnical parameters and necessary embankment material characterstics datas has been collected from geotechnical investigation report, detail dam design, project drawings and from typical studies of identical worksas shown detail on next section.

### 3.2.2 Dam dimensioning

The rock material which is identified as the most expensive part of the embankment was replaced with easily available finer rock material keeping the core and filter zones as it is. Dam dimensioning and sizing of embankment slope for the newly proposed section has been done based on design manuals, previous studies and literatures.

#### 3.2.2.1 Upstream and Downstream Shell slopes

The upstream and downstream slope of rock fill embankment depends mainly on the size and soil used for construction, foundation condition, seismicity of area, core geometry and location etc... According to USBR, (1986) it would be appropriate to use slopes ranging from 2H: 1V to 4H: 1V if the rock fill dam for central or sloping cores. Based on the above recommendations and economic considerations, different slope trials have been examined and upstream side slope of 2.4H: 1V and downstream side slope of 2.3H: 1V has been selected as an appropriate slope and the analysis has been done accordingly however the original design slope was upstream side slope of 2 H: 1V and downstream side slope of 1.75H: 1V and 2H: 1V.

#### 3.2.2.2 Clay core

Location of core has its own advantage and disadvantage based on the material availability and foundation conditions. The maximum core width is controlled by stability and availability of core material and based on Kutzner, (1997) vertical core are recommended to have a slope of 0.25H:1V to 0.33H:1V thus for the analysis clay core slope of 0.25H:1V has been keptas it is based on original design of the dam(Kutzner,1997).

#### 3.2.2.3 Filter material

Suitable internal filter and drainage blanket should be provided to control saturation and seepage pressure at a safe level and prevent piping in the embankment and foundation. Thickness of filter materials fixed based on the amount of seepage computed from analysis and based on construction equipment used. Considering these criterions and from utilization of construction equipment filters of 2.5 m wide has been adopted from original design by (WWDSE, 2010)for both fine and course filter zone for this study.

#### 3.2.2.4 Provision of berm

For Ribb rock fill dam a total of five berms at 12.5 m vertical interval with 5 m width at downstream are provided, for the safety against slope erosion as both the upstream and downstream slopes will be constructed from slightly weathered fine rock and in order to break the height at convenient levels for later operation and maintenance. An 8 m wide berm at the upstream also provided for the stability of the upstream slope (WWDSE, 2010).

### 3.2.3 Dam and reservoir characteristics

Characteristics level of dam and reservoir of the project used for this study i.e. construction crest level, free board, maximum water level, normal pool level, minimum draw dawn, reservoir storage capacity and so on are listed in table 3-2.Dam and reservoir characteristics levels are adopted from the original design (WWDSE, 2010).

Table 3-2: Dam and reservoir characteristics of the project (WWDSE, 2010)

| Item | Unit | Elevation |
|---|---|---|
| MDDL | m | 1901.45 |
| NPL | m | 1940 |
| MWL | m | 1943 |
| Construction crest level(CCL) | m | 1946.3 up to 1945.5 |
| River bed level (RBL) | m | 1873 |
| General foundation level (AEL) | m | 1863 |
| Free board | m | 3.06 |
| Reservoir capacity | $Mm^3$ | 234 |

### 3.2.4 Seepage Analysis

Once the cross section of the dam has been fixed the amount of seepage through the dam and the foundation has been computed and evaluated using SEEP/W which is a package of Geo Studio. SEEP/W is a numerical model that can mathematically simulate the real physical process of water flowing through a particular medium. In using the SEEP/W software, the term seepage is used to describe all movement of water through soil regardless of the creation or source of the driving force or whether the flow is through saturated or unsaturated soils (SEEP/W, 2008).

There are two fundamental types of finite element seepage analyses, steady-state and transient. Steady-state analysis does not consider how long it takes to achieve a steady condition and that is something which should be understood. The model will reach a solved set of pressure and flow conditions for the given set of unique boundary conditions applied to it and that is the extent of the analysis. A transient analysis by definition means one that is always changing. It is changing because it considers how long the soil takes to respond to the user boundary conditions(SEEP/W, 2008).For this study both steady state and transient types of seepage analysis has been conducted.

Referring to Krahn, (2007) the element size of the mesh to be used in finite element method should be selected carefully. If the element size used in the analysis is too small will result to have many elements and causes unnecessarily time for computation and if large elements are used the result found will be rough and misleading.

For this research a global meshing size of six is used. Besides the shapes of the elements are chosen to be quadrilaterals and triangles as these element shapes are most recent favored approach and appropriate solutions will be found using mostly these patterns(Krahn, 2007).

Impervious clay core and transition filter and drainage zones are considered for modeling of seepage analysis. The shell material (finer rock) has been assumed as a null region because this zone is assumed to be free draining and has no any significance regarding to flow through the embankment.

### Material models used for seepage analysis

### Saturated only model

Saturated only soil model is used because it is very useful for quickly describing a soil region that will always remain below the phreatic surface, but it should not be used for soils that will at some point during the analysis become partially saturated. If this happens ,the model will continue to solve but interpreted in result, that the unsaturated zone can convey water at the same rate as for the saturated soil this will result an over estimation of flow quantity and can result in an unrealistic result(SEEP/W, 2008).The respective material property for saturated only model is:-

- Hydraulic saturated conductivity ($K_{sat}$)

### Saturated / Unsaturated model

Saturated unsaturated material model is used for the materials which are basically above the minimum draw down in the area where the soil is expected to be in saturated as well as unsaturated condition during in the life time of the dam (SEEP/W, 2008).

The respective material properties for saturated/unsaturated model are(SEEP/W, 2008):-

- Hydraulic conductivity function
- Volumetric Water content function

For this study both the above models has been applied that the foundation which is expected always under water attached to saturated model and the embankment material has been attached to saturated unsaturated model as summarized in table 3-3 below.

Table3-3: Material model and properties used for seepage analysis (SEEP/W, 2008)

| **Description of materials** | **Model** | **Material Properties** |
|---|---|---|
| Impervious core | Saturated/Unsaturated | Hydraulic conductivity function |
| | | Volumetric Water content function |
| Fine Filter | Saturated/Unsaturated | Hydraulic conductivity function |
| | | Volumetric Water content function |
| Coarse Filter | Saturated/Unsaturated | Hydraulic conductivity function |
| | | Volumetric Water content function |
| Slightly weathered Fine rock | Saturated/Unsaturated | Hydraulic conductivity function |
| | | Volumetric Water content function |
| Alluvium fill | Saturated/Unsaturated | Hydraulic conductivity function |
| | | Volumetric Water content function |
| Silty clay Foundation | Saturated only | Hydraulic saturated conductivity function |
| Bed rock | Saturated only | Hydraulic saturated conductivity function |

**Material properties**

The value of coefficient of permeability of each material used for the analysis has been adopted from the final design report of Ribb dam WWSDE, (2010) and Final Geological and Geotechnical Report (WWDSE, 2007) is indicated in table 3-4.

Table3-4: Coefficients of Permeability used for seepage analyses (WWDSE, 2010)

| **Description of materials** | **K (cm/sec)** | **Remarks** |
|---|---|---|
| Impervious Core | $1 *10^{-7}$ | Design report |
| Fine Filter | $1 *10^{-4}$ | Design report |
| Coarse Filter | $1*10^{-2}$ | Design report |
| Slightly weathered Fine rock | $1*10^{-1}$ | Design report |
| Alluvium fill | $1*10^{-1}$ | Design report |
| Silty clay Foundation | $6.34*10^{-6}$ | Design report |
| Bed rock | $5.14*10^{-5}$ | Design report |

Typical values of volumetric water content used for this analysis were adopted from sample function for different soil type from engineering book of SEEP/W sample function SEEP/W, (2008) as presented in table 3-5below.

Table3-5: Volumetric water content used for seepage analyses (SEEP/W, 2008)

| **Description of materials** | **Volumetric water content($m^3/m^3$)** |
|---|---|
| Impervious Core | 0.49 |
| Fine Filter | 0.35 |
| Coarse Filter | 0.3 |
| Slightly weathered Fine rock | 0.25 |
| Alluvium fill | 0.25 |

**Boundary conditions**

Boundary conditions are key components in a problem to get solutions. Any solutions or computed result on numerical modeling are the response of the boundary condition applied on .On Geo slope boundary conditions are applied directly on geometry item like region face, region line or points. Boundary condition has been applied as constant and as a function depending on the type of analysis to be carried out. The finite element equation used for seepage analysis is given by the following equation 3.1(SEEP/W, 2008).

$$[K]\{H\}=\{Q\}$$ ----------------------------------------------------------------------Equation 3. 1

Where

[K] = a matrix of coefficient related to geometry and material properties

{H}= a vector of the total hydraulic heads at the nodes

{Q} = a vector of the flow quantities at the nodes

In SEEP/W the total hydraulic head, which is made up of pressure head ($u/\gamma_{w}$) and elevation head (Y) is given by equation 3.2 (SEEP/W, 2008).

$$H = \frac{u}{\gamma_w} + Y \text{ ---------------------------------------------Equation 3. 2}$$

Where

H= total head (in meters)

U = pore –water pressure (in kPa)

$\gamma_w$= unit weight of water (in kN/m$^3$)

Y = elevation head (in meters)

Boundary conditions used for SEEP/Ware (SEEP/W, 2008)

- Total head - which is a function of pore pressure head plus elevation head
- Potential seepage face -in the condition where seepage location is not known
- And zero pressure - inside filter and drainage areas

Expected out puts from SEEP/W analyses (SEEP/W, 2008)

- Flux (quantity of flow or discharge)
- Pore- water pressure distribution and during analysis the pore water distribution found from SEEP/W result is used as an input in SLOPE/W to analyze the respective slope stability analysis of steady state and transient condition.

### 3.2.5 Slope Stability Analysis

The maximum cross-section of the dam has been used for slope stability analysis for different loading conditions using SLOPE/W which is formulated based on principles of limit equilibrium method (LEM) considered as adequate for analyzing slope stability of embankment dams. Basically there are different kind of limit equilibrium methods like Ordinary or Fellenius, Bishop's simplified, Janbu's simplified, Spencer, Morgenstern price, Corps of Engineers-1,Corps of Engineers-2, Lowe-Karafiath, Janbu Generalized, Sarma(SLOPE/W, 2008). Among those Spencer and Morgenstern–price method include all inter slice forces and fulfill equation of static. Therefore Morgenstern-Price has been used for this analysis as it satisfies force and moment equilibrium(SLOPE/W, 2008).For this analysis the original design was done based on limit equilibrium method and the Morgenstern-Price method. Material properties required for SLOPE/W modeling were adopted from final design report of Ribb dam WWDSE, (2010) summarized in the following table 3-6.The stability analysis of upstream and downstream slopes has been analyzed from normal pool level.

Table3-6 :Shear strength parameters used for slope analyses (WWDSE, 2010)

| **Description of materials** | $\gamma$ **(kN/m$^3$)** | **$\phi'$ (degree)** | **c' (kPa)** |
|---|---|---|---|
| Impervious Core | 16 | 20 | 20 |
| Fine Filter | 18 | 34 | 0 |
| Coarse Filter | 18 | 34 | 0 |
| Slightly weathered Fine rock | 18 | 32 | 0 |
| Alluvium fill | 18 | 32 | 0 |
| Silty clay Foundation | 17 | 29 | 5 |

In the analysis of slope stability a single number called factor of safety has been computed for all loading condition to determine the margin of safety of the embankment to guard against failure by determining the total shear resistance and the total mobilized shear stress on a slip surface. Ant the factor of safety has been computed with equation 3.3shown below (Dunkin, 1987).

$$F.S = \frac{\tau}{S} \text{ -----------------------------------------------------------------Equation 3. 3}$$

Where,

F.S = Factor of safety,

τ = Total shear strength

S =Total shear stress

And the Shear strength of the material has been computed with Coulomb's equation expressed as shown on equation 3.4 (Dunkin, 1987).

$\tau = C + \sigma_n t{:}\ \ \phi$------------------------------------------------------------Equation 3. 4

Where

$\tau$ = Sear strength

C = Cohesion

$\sigma_n$=Normal stress

$\phi$= angle of internal friction

During analysis of embankment slope during construction loading condition a pore water pressure ratio ($r_u$) coefficient values that relates the pore water pressure to the overburden stress, has been used to simulate the actual site condition adopting pore water pressure ratio for different materials from the final design report of the project as compile s in table 3-7.

Table3-7: Pore pressure ratio ($r_u$) values used for slope stability (WWDSE, 2010)

| | **Pore water pressure ratio ($r_u$)** | |
|---|---|---|
| **Description of materials** | **During construction** | **End of construction** |
| Impervious Core | 0.5 | 0.4 |
| Fine Filter | 0 | 0 |
| Coarse Filter | 0 | 0 |
| Slightly weathered Fine rock | 0 | 0 |
| Alluvium fill | 0 | 0 |
| Silty clay Foundation | 0.5 | 0.4 |

The pore water distribution found from SEEP/W result has been used as an input in SLOPE/W to analyze the respective slope stability analysis of steady state and transient condition.

As the factor of safety of either embankment slopes for the respective loading condition has been computed the result found on revised section has been compared with standards and original design. According to USACE, (1970) embankment slopes should satisfy the minimum factor of safety recommended depending on the loading conditions. Table 3-8 has presented the minimum required factor of safety for the respective loading conditions.

Table3-8: Minimum required factor of safety (FOS $_{min}$) for different loading conditions (USACE, 1970)

| No | Loading condition | Slope | FOS$_{min}$ |
|---|---|---|---|
| 1 | Steady state seepage | Downstream | 1.5 |
| 2 | Sudden draw down | Upstream | 1.3 |
| 3 | Stage construction | Upstream | 1.3 |
|  |  | Downstream | 1.3 |
| 4 | End of construction | Upstream | 1.3 |
|  |  | Downstream | 1.3 |

- FOS$_{min}$ = minimum required factor of safety

**Upstream slope protection**

The design of riprap used for upstream slope protection has been revised due to the change of rock material with fine rock and modification of dam cross section. The rip rap design revision was done based on considerations and recommendations of USBR, (1992) design manual presented in table 3.9 below.

Table3-9: Parameters used for riprap design (WWDSE, 2010)

| Parameters | Unit | Value | Remark |
|---|---|---|---|
| Specific unit weight of rock ($\gamma_s$) | kg/m$^3$ | 2200 | Design report |
| Significant wave height ($H_s$) | m | 2.63 | Design report |
| Specific gravity of rock($G_s$) | - | 2.65 | Design report |
| Slope angle of upstream slope ($\alpha$) | degree | 22.62 | From revised cross section |

USBR,(1992) stated that earth and rock fill dams require proper slope protection for their upstream and downstream face to protect against erosion caused by wind, weathering, ice, floating debris and wave erosion. The upstream slope requires more treatment than the downstream as it is directly exposed to wave action. Riprap design was done on original design WWDSE, (2010) but on this study the riprap design has been revised due to material replacement of the rock fill material with fine rock material as the riprap design is a function of upstream slope inclination (α). Dumped riprap has been selected for upstream slope protection of Ribb dam considering simplicity and cost.

The design of riprap has been done based on considerations and procedures of USBR, (1992) as presented on the next section by adopting the appropriate data from the original design document WWDSE, (2010) and the revised dam cross section proposed in this thesis.

According to USBR, (1992) riprap is provided to protect the embankment against loads produced by wind generated waves and gravity. The force acting on the embankments is a function of velocity of water, gravity, hydrostatic pressure. While the resisting force offered by the riprap is expressed in terms of riprap volume and its buoyant weight. The required riprap weight for tolerable and zero damage is given by following equations 3.5 and 3.6 (USBR, 1992).

$$W_{50} = \frac{\gamma_r * H_s^3}{4.37(G_s - 1)^3 (\cot \alpha)} \text{------------------------------------------------------Equation 3. 5}$$

$$W_{50} = \frac{\gamma_r * H_s^3}{3.62(G_s - 1)^3 (\cot \alpha)^{0.67}} \text{---------------------------------------------------Equation 3. 6}$$

Where:

$W_{50}$ =Characteristic weight of individual rock fragments necessary to resist wave action

$\gamma_r$= Specific unit weight of rock

$G_s$= Specific gravity of rock

α = Slope angle measured from horizontal

$H_s^3$= Significant wave height

The maximum and minimum riprap sizes can be estimated from the following equations (USBR, 1992)

$$W_{max} = 4W_{50} \text{--------------------------------------------------------------------Equation 3. 7}$$

$$W_{min} = \frac{W_{50}}{8} \text{-----------------------------------------------------------------------Equation 3. 8}$$

Where:

$W_{max}$= 100% of the rock riprap is smaller

$W_{min}$ = Approximately 5% of the rock in the riprap is smaller

The above equation gives the weight of the rock to be used as riprap, however, for construction simplicity; this weight has to be changed to rock volume. So to convert this rock weight to a representative rock diameter (assuming a rock shape between a sphere and a cube), the following equation has beenused (USBR, (1992).

$$Vol = 0.75D_n^3$$ ----------------------------------------------------------------------Equation 3. 9

Where:

VOL = Rock volume ($m^3$)

$$Vol = \frac{W_n}{\gamma_r}$$

$W_n$= Weight of the rock in the riprap where n% is smaller

$\gamma_r$ = Specific unit weight of rock

$D_n$= Representative diameter of the rock where n% is smaller

USBR, (1992) stated below the riprap filter/bedding layer must be provided to prevent loss of embankment material through the riprap. The thickness of bedding layers must be sufficient to provide filter protection, and to provide a supporting bed for the riprap. As shown on table 3-10 the following are suggested bedding layer thicknesses.

Table3-10: Parameters used for riprap design (WWDSE, 2010)

| **Riprap layer, inches {mm}** | **Bedding layer, inches {mm}** |
|---|---|
| 12-24 {300 -600 } | 9 { 225 } |
| 27-36 {325 -900 } | 12 { 300 } |
| over 36 {≥ 900 } | 15 { 375 } |

### 3.2.6 Stress and Deformation Analysis

Stress distribution and deformation of the dam has been analyzed using SIGMA/W, finite element based package which is appropriate for earth structures (SIGMA/W, 2008). Material properties required in SIGMA/W are Young's Modulus (E), Poisson's Ratio ($\nu$), Cohesion(c'), Friction angle ($\phi^o$).As the stress deformation analysis was not incorporated on the final detail design, required material parameters for stress deformation has been taken from literature.Sabatini et al, (2002) and Bowles, (1997) has been used as main source for material parameters as shown in table 3-11.

Table3-11: Material properties usedfor stress deformation analysis (WWDSE, 2010)

| **Description of materials** | $\gamma$ (kN/m$^3$) | **c' (KPa)** | **φ° (degree)** | $\nu$ | **E ,in MPa** | **Remarks** |
|---|---|---|---|---|---|---|
| Impervious Core | 16 | 20 | 20 | 0.3 | 50 | Design report |
| Fine Filter | 18 | - | 34 | 0.25 | 30 | Design report |
| Coarse Filter | 18 | - | 34 | 0.15 | 70 | Design report |
| Slightly weathered Fine rock | 18 | - | 32 | 0.3 | 100 | Design report |
| Alluvium fill | 18 | - | 32 | 0.3 | 100 | Design report |
| Silty clay Foundation | 17 | 5 | 29 | 0.3 | 70 | Design report |
| Bed rock | 25 | 10,500 | 35 | 0.2 | 1.5GPa | Design report |

Among the available soil constitutive models incorporated for stress deformation analysis which are Linear elastic, Elastic plastic (Mohor Coulomb) and Modified Cam clay(MCC)), linear elastic model has been selected based on the suggestion given by (Krahn, 2003) and stated that it is sufficient to use Linear elastic model for earth and rock fill dams and (Yasar, 2010) also stated that Linear elastic model would be appropriate where there is lack information about the real material properties which are required for stress deformation analysis.

### Linear- elastic model

Linear-elastic model is one of constitutive model in SIGMA/W which assumes stresses to be directly proportional to the strain. The proportionality constants are Young's Modulus, E, and Poisson's Ratio, $\nu$.Cohesion and internal angle of friction are also important soil parameters for stress deformation analysis (SIGMA/W, 2008).

### Boundary conditions for SIGMA/W

Like that of Geoslope components SIGMA/W also used boundary conditions to solve the problems. Depending on the type of problem different kinds of boundary conditions may be applied. In case of load deformation analysis it is important to bound the problem by defining some parts of the problem domain with zero displacement boundary condition (SIGMA/W,2008).For this study the problem domain has been bounded at the bottom and side edge to fix the displacement in x and y direction to be zero. Finite Element equation used to solve the unknowns during load deformation analysis is given by (SIGMA/W, 2008):-

$[K]\{\Delta d\} = \{\Delta F\}$ ------------------------------------------------------------Equation 3. 10

Where:

[K] = System stiffness (E), poisons ratio $(\nu)$, Volume or Area (material property)

$\{\Delta d\}$ = Incremental displacements

$\{\Delta F\}$ = Incremental forces

**Analysis type**

In SIGMA/W, different analyses types are incorporated and among this, insitu analysis and stress deformation analysis has been used in this study. Insitu analysis has been carried out to simulate the initial stress condition and stress deformation analysis has been carried out to compute deformation caused by load of the embankment (SIGMA/W, 2008).

**Shear failure checking**

One of the advantage of doing stress-deformation analysis is that ,the embankment clay core can be examined for the occurrence of clay core fracturing and cracking by examining the pore water condition and change in stress distribution. Based onOsuji and Anyata, (2007) study crack or shear failure may happen if the divatoric stress ( $\sigma_1$- $\sigma_3$) at any elemental node in the core is higher than or equal to the Mohr – Columb's failure stresses given by equation 3.12:-

$$\pm = 2 * \left( \frac{c' \cos ¤ + \pm_3}{1 - \sin ¤} \right) \sin ¤$$ -----------------------------------------------------------------Equation 3. 11

Where:-

$\sigma$ =Mohr – Columb's failure stresses

$\phi$= Internal friction angle

c'= Cohesion of material

$\sigma_3$= minimum stresses

Accordingly, the presence of shear failure of core cracking for the revised dam cross section has been analyzed in this study.

### 3.2.7 Cost comparison

After replacing blasted rock fill zone of Ribb dam with easily available fine rock and conducting all necessary design considerations the two dam options has been compared. The comparison has been made based on economy, required construction duration and construction difficulty. For comparison purpose the average fill rate performance of blasted rock and slightly weathered fine rock has been considered for two consecutive construction years.

## 3.3 Model Used

Recently stability of earth structures can be analyzed with geotechnical software and Geo-studio is one of this software, developed by Geo-studio international based on limit equilibrium finite element package developed for the analysis of seepage, slope and deformation of geotechnical structures. For this thesis, Geo Studio 2007 has been used for the analysis and the component SEEP/W, SLOPE/W, and SIGMA/W packages are used mostly here in.

# 4 RESULTS AND DISCUSSION

## 4.1 Dam zoning and geometric analysis

Ribb dam originally designed by water works design and supervision enterprise as a rock fill embankment dam composed of internal clay core, transition filter, internal finer shell and rock fill zones as shown in (Fig.4-1). The rock fill zone provided in the original design demands huge effort for production, mobilization and embankment construction. This research is intended to use the advantage of this easily produced fine rock as an embankment material so that the dam cross section has been revised accordingly based on the criterion listed in section 3.2.2 of this study.

As the main focus of this research has been analyzing the advantages of shell material change, the geometry and dimension of impervious core has been adopted similar with the original design prepared by water works design and supervision enterprise WWDSE, 2010 to have a top width of 3 m and an upstream and downstream slope of 0.25H:1V. Similary the dimension of fine and coarse filiter has been adopted from the design reprt of Ribb dam detail design to have a width of 2.5 m considering utilization of construction equipment.

The newly revised dam cross section replacing the rock fill with finer rock materialhas been shown in Figure4-1.

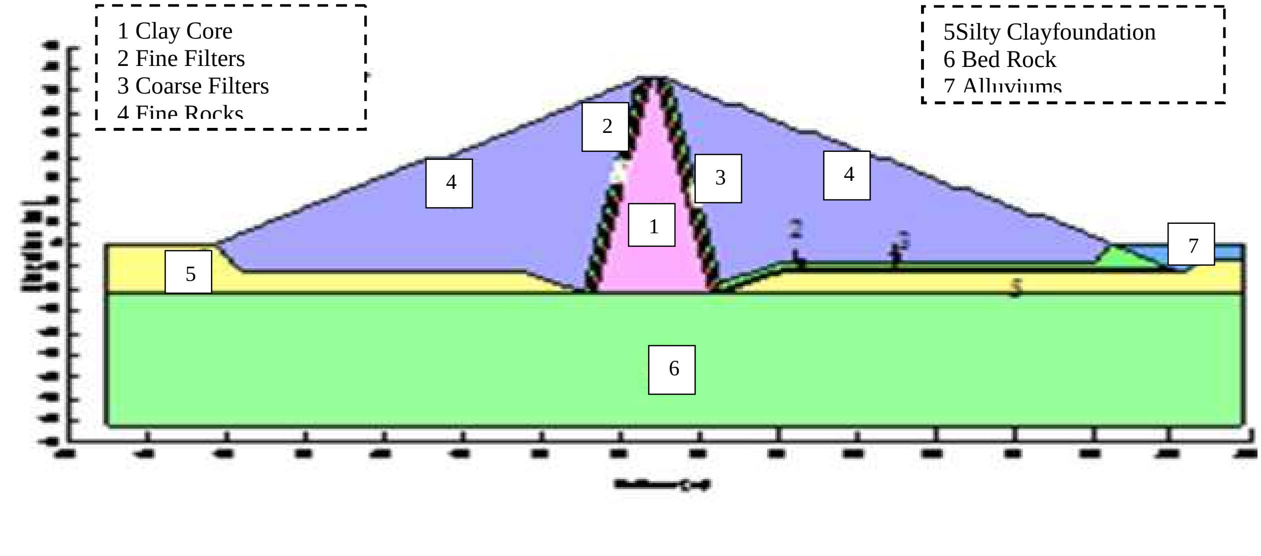


Figure 4-1: Dam cross section of Ribb revised

## 4.2 Seepage analysis

The appropriate hydraulic parameters as shown in section 3.2.4 have been applied to the respective materials. The finite element meshing and boundary conditions and null region of the shell zone applied for seepage analysis in this research is shown in Figure4-2.

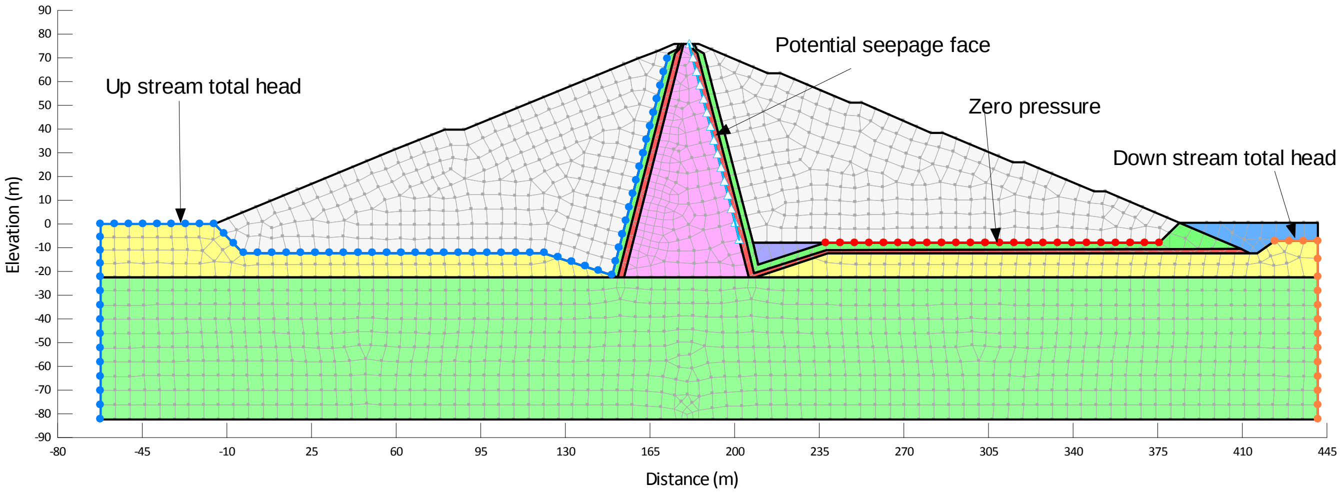


Figure 4-2: Finite element model meshing and boundary conditions used for seepage analysis

Accordingly seepage through the dam, the foundation and the phreatic surface of the seepage has been analyzed and it can be seen that the phreatic line drops down inside the clay core as indicated in figure 4-3 and 4-4 respectively.

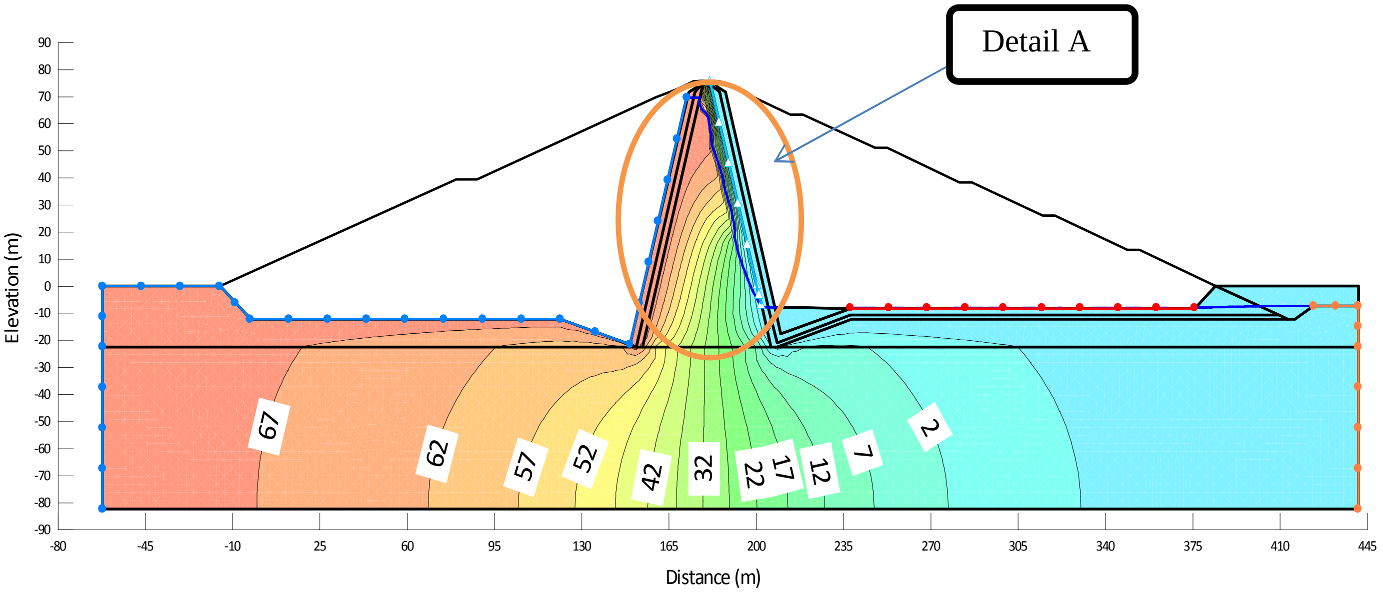


Figure 4-3 :Total head (Isopotential lines) distribution and phreatic line location for the revised section

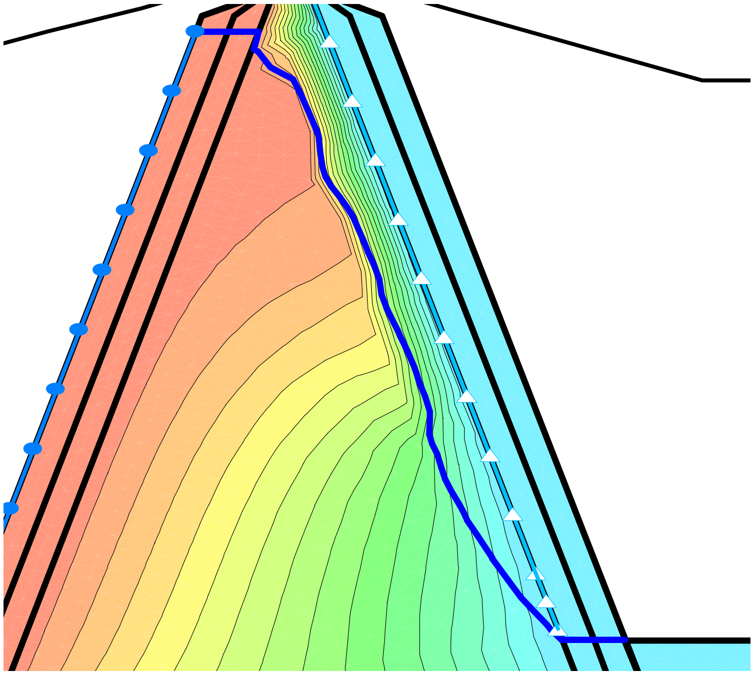


Figure 4-4 :Phreatic line location (Detail A)

The potential drop in the dam body and the phreatic line showed that, there was a total drop in potential within the core of the dam which confirms the performance of the core material against seepage loss as shown in (Fig.4-3).Based on SEEP/W result, the total amount of seepage flux through the dam body and foundation for the substitute embankment material of the dam cross section has been computed and found to be 1.18 lit/min/m as shown in (Fig. 4-5 A).

For the amount of seepage passing through the dam body and foundation Kutzner, (1997) recommends that the flux through embankment dams' body and the foundation should be controlled to minimize migration of base material and subsequent failureand suggestedthat the allowable amount of seepage flux to be within the range between 0.1 to 2 lit/min/m. According to this guideline the amount of flux computed for the revised dam cross section with finer rock is found within the allowable limit so that there will be no excess loss of water.

The amount of flux computed for the dam body and the foundation on the original design WWDSE, (2010) was 0.072 lit/min/m (Fig 4-5 B) which is less than thecomputed in the case of the substitute material despite there is no dimension change of the impervious clay core for the two cross sections. This is due to the reason that in the original design the designer has incorporated the grout curtain in seepage modeling however this grouting zone is not included in the revised case study. Also it is found that as the seepage through the dam body and foundation without foundation treatment is satisfactory, or within the allowable limit of the seepage flux, there is no need of foundation treatment in the revised case.

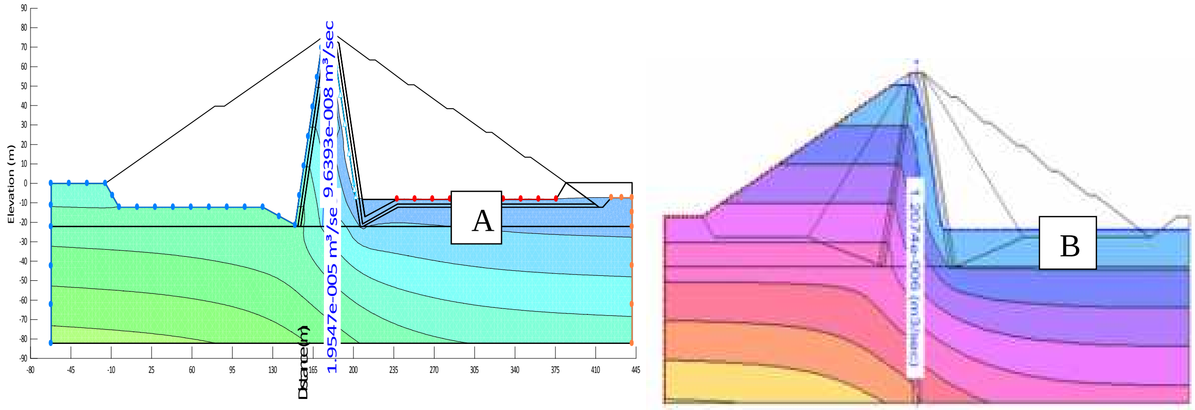


Figure 4-5: Seepage through the dam body and foundation (A) for the revised oneand (B) original design computed by WWDSE

Accordingto Kutzner, (1997) the hydraulic gradient inside the clay core should not exceed 2 to avoid hydraulic fracturing, material erosion and piping. And basedon ouranalysis the horizontal gradient varies from 0.5 to 2 as indicated in Figure4-6 and therefore our result is within the allowable limit.

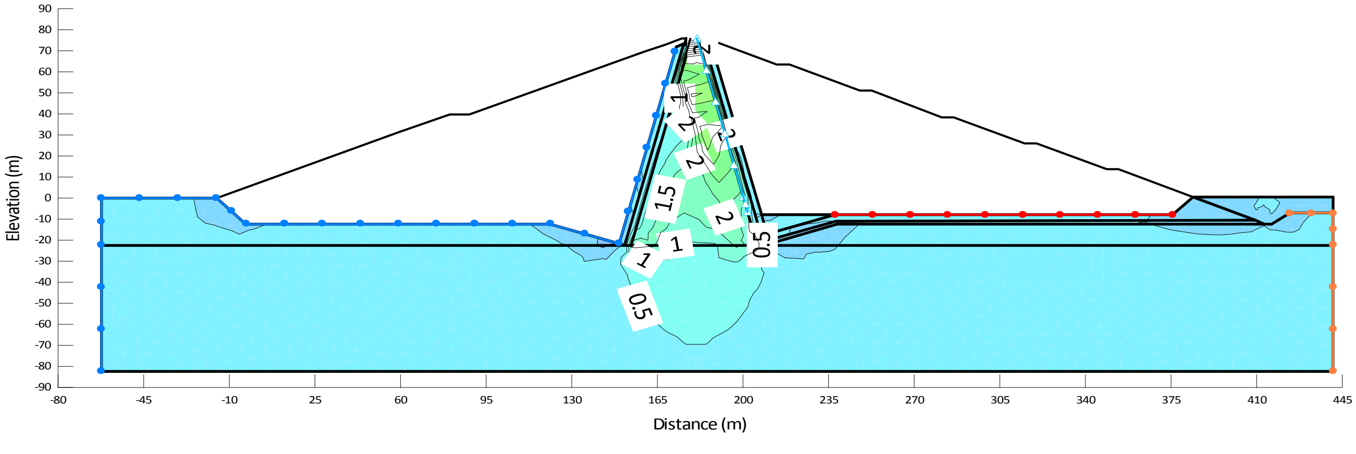


Figure 4-6 :Horizontal gradients through the dam and foundationfor the revised section

Analysis of steady state pore water distribution as indicated in figure 4-7 the pore water pressure below the phreatic line (i.e. zero pressure line) is positive and above the phreatic line it is negative and these results can be used for slope stability analysis in section 4.3.

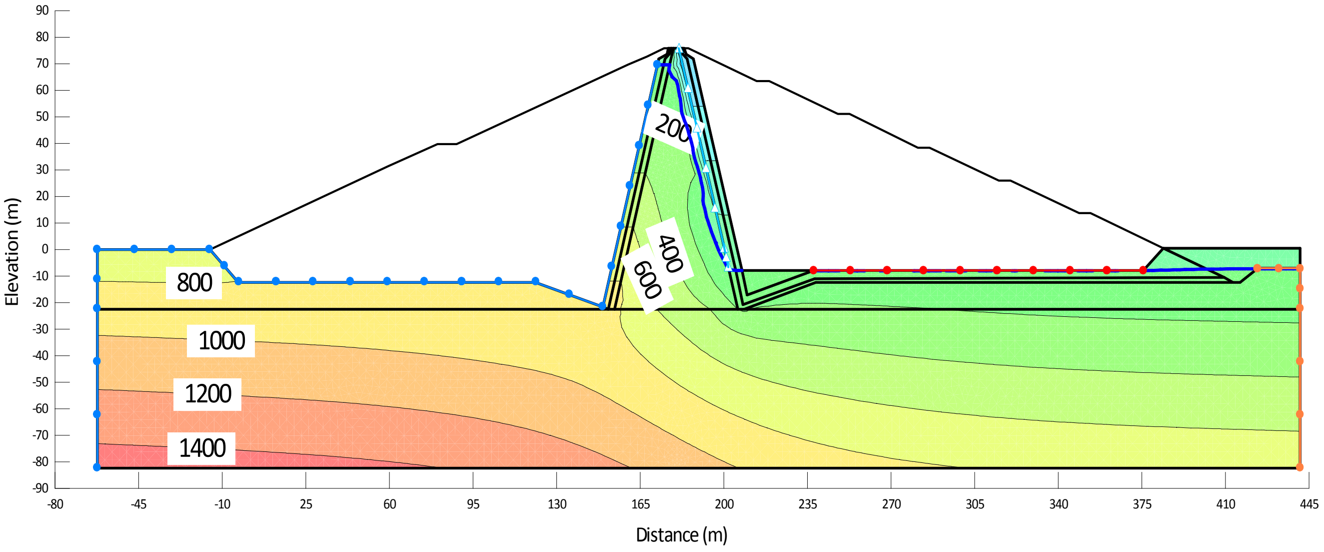


Figure 4-7: Pore water pressure contourfor the revised section

## 4.3 Slope stability analysis

The slope stability of the newly revised cross section of the dam has been used for analysis and the result is found to be satisfactory based on the standards stated in table 3-8.In addition shear strength parameters and pore-water pressure ratio ($r_u$) values presented in section 3.2.5 are applied.

The result of upstream and downstream slope factor of safety computed for modified cross section and results of the original design computed by WWDSE has been presented in (table 4-1). As indicate on summary of stability analysisresult table 4-1 all computed factor of safeties for the revised section has been improved and results more stable structure when compared to original design.

Table4-1: Summary of stability analysis results for different loading condition

| No | Loading condition | Slope | FOS$_{min}$ | FOS Computed by WWDSE | Computed FOS |
|---|---|---|---|---|---|
| 1 | Steady state seepage | Downstream | 1.5 | 1.703 | 1.801 |
| 2 | Sudden draw down | Upstream | 1.3 | 1.364 | 1.489 |
| 3 | During construction (1st stage) | Upstream | 1.3 | 1.367 | 1.503 |
|  |  | Downstream | 1.3 | 1.653 | 1.778 |
| 4 | During construction (2nd stage) | Upstream | 1.3 | 1.276 | 1.484 |
|  |  | Downstream | 1.3 | 1.293 | 1.785 |
| 5 | End of construction | Upstream | 1.3 | 1.297 | 1.503 |
|  |  | Downstream | 1.3 | 1.326 | 1.725 |

### I. Steady State Loading Condition

The factor of safety of the downstream slope at steady state condition without earth quake has been computed and a value of 1.801 has been found as shown in (Fig.4-8 A) and this result shows that the downstream slope is stable during steady state as the minimum requirement for this loading condition is 1.5 (Jansen, 1988).

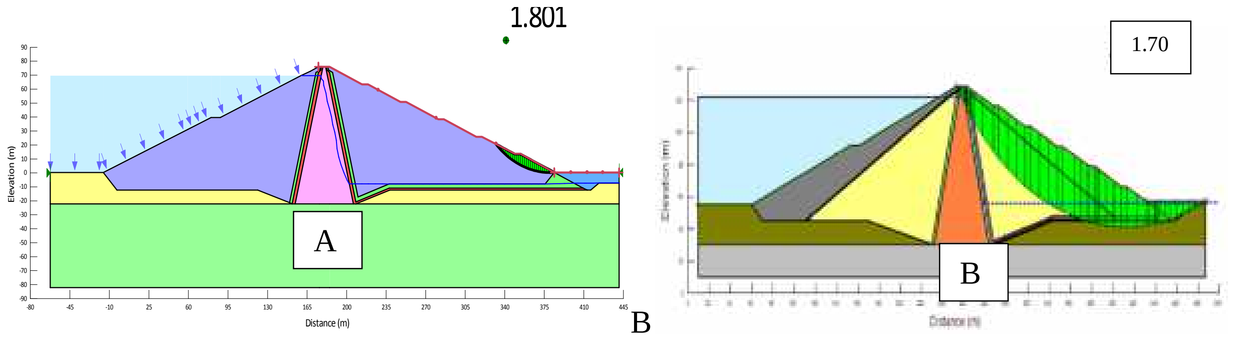


Figure 4-8 :Minimum factor of safety at steady state without earth quake (A) for revised section and (B) old design (WWDSE, 2010)

### II. Sudden Drawdown

For sudden draw down loading condition the stability of the upstream slope of the embankment has been analyzed as the stability of this slope has been compromised due to the evacuation of upstream water suddenly and the stabilizing effect of this water load is deducted and the pore water pressure inside the soil sustained for longer time. The pore water pressure within the embankment dam depends on the permeability and storage capacity of the fill materials. In the revised section of Ribb dam the clay core is expected to drain slowly. The draw down has been assumed to be instantaneous and the analysis has been conducted for the next 10 days. The drop of the phreatic line during analysis period has been presented in Figure 4-9 and Figure 4-10.

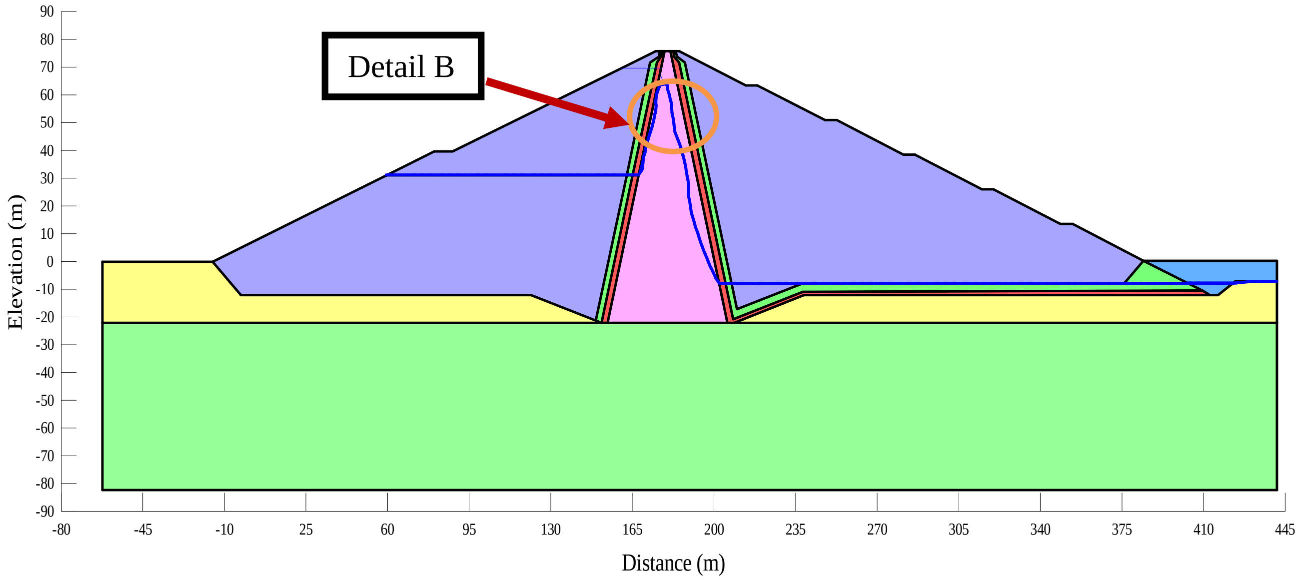


Figure 4-9 : Zero pressure lines of drawdown for different analysis period

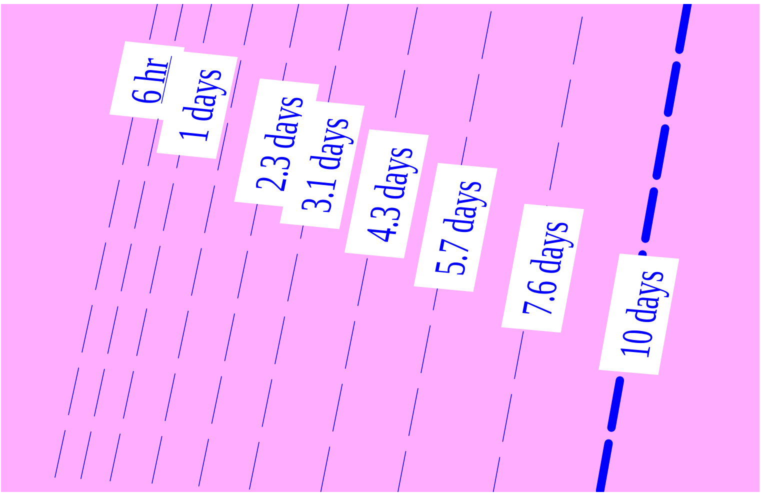


Figure 4-10: Zero pressure lines of drawdown for different analysis period (Detail B)

The status of the upstream slope during sudden draw down has been analyzed for the newly revised dam cross section and a minimum factor of safety of 1.489 (Fig. 4-11 A) at 6 hr after the draw down has been found, which is satisfactory as compare to the minimum requirement 1.3 (Table 4-1). The factor of safety computed for the new cross-section is higher as compare to the original design 1.364 as shown in Figure 4-11B which shows that the modified section is more stable during sudden draw down condition.

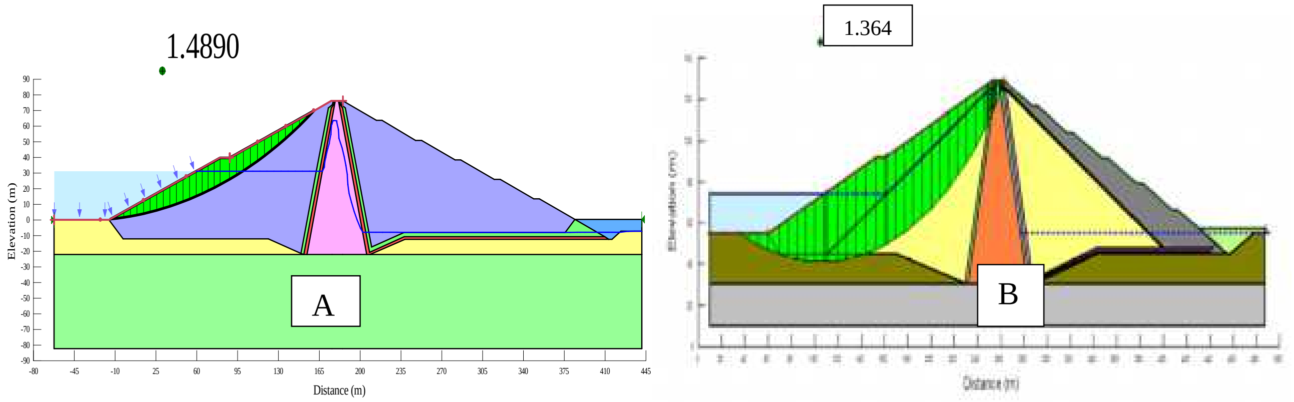


Figure 4-11 : Minimum factor of safety computed (A) for revised section and (B) original design during sudden drawdown at 6 hr

### III. End of Construction

For end of construction loading condition slope stability for both upstream and downstream has been computed and a minimum factor of safety has been found in upstream side and this result satisfies the minimum requirement of 1.3 stated in table 4-1. As it has been shown in Figure4-12A factor of safety of 1.503 has been obtained in addition to this the mass of the embankment section along this critical failure plane is thinas indicated in figure 4-13 so that it can also be easily maintained if failure happens. Downstream slope stability result for the original and revised section has been presented in Figure A - 1 of Appendix A indicated that all estimated factor of safety of downstream slope satisfies the minimum requirement.

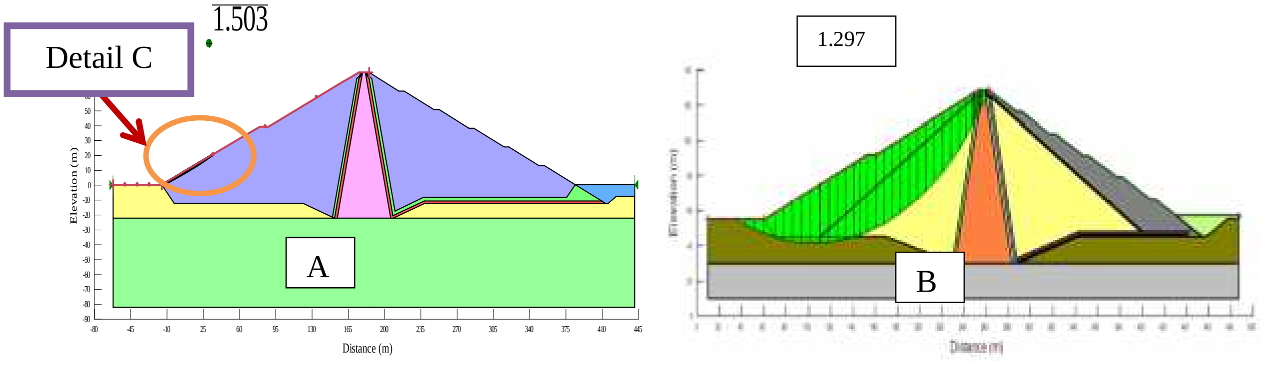


Figure 4-12: Factor of safety of (A) revised and (B) original design section during end of construction

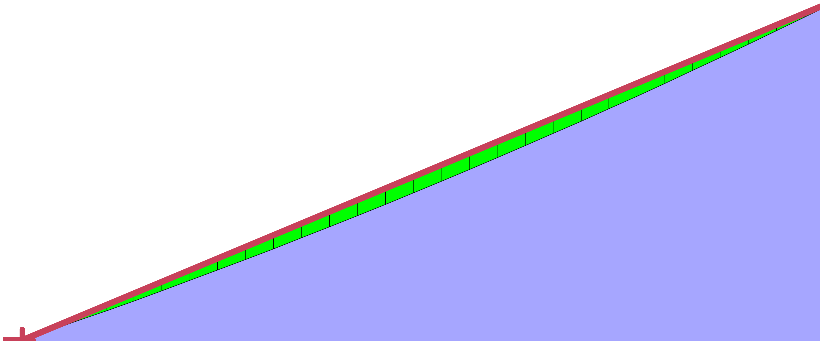


Figure 4-13: Potential failure surface during End of Construction (Detail C)

### Stage Construction

The dam has been analyzed for upstream and downstream slope stability with two construction phases. The first construction stage is assumed to be from foundation to 1910.0 m a s l and the second construction stage to be from 1910 m a s l to crest level 1945.5 m a s l. Based on this construction stage assuming the minimum factor of safety has been found at the upstream embankment slope during the construction of the first stage with the value of 1.503 (Fig. 4-14 A). The attained minimum factor of safety as compared to the threshold value (1.3) is satisfactory under this loading condition.

Factors of safety computed on the original design WWDSE, (2010) during second stage construction for both upstream and downstream slope were 1.276 and 1.293 respectively. It is a little bit less than the minimum required factor of safety of 1.3 and the designer accepts these results as satisfactory but these values are improved for the revised section, where the minimum factor of safety is 1.503 this is due to improvement on upstream and downstream outer shell slopes (relatively flat).

The results computed for this loading condition for the revised and original design has been presented through (Fig. 4-14, Fig. 4-15 and Figure A-2 up to Figure A-4 of Appendix A) for further reference.

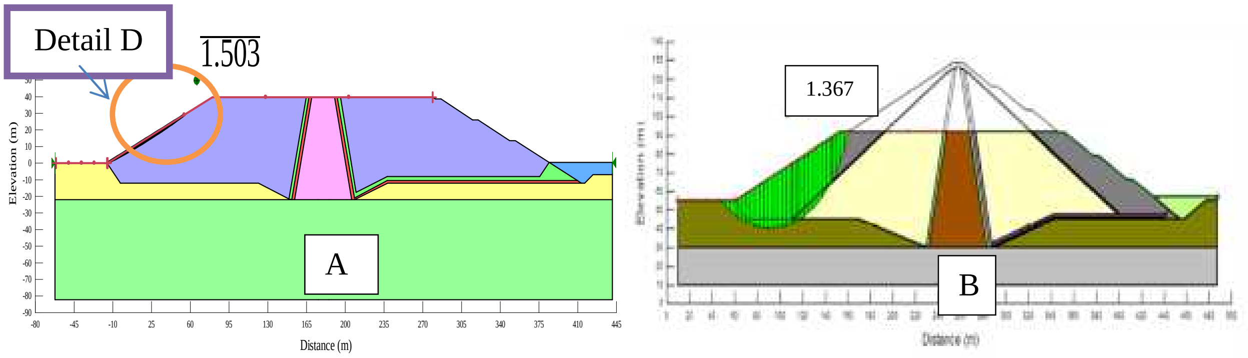


Figure 4-14: Factor of safety of (A) revisedsection and (B) original design for first stage construction

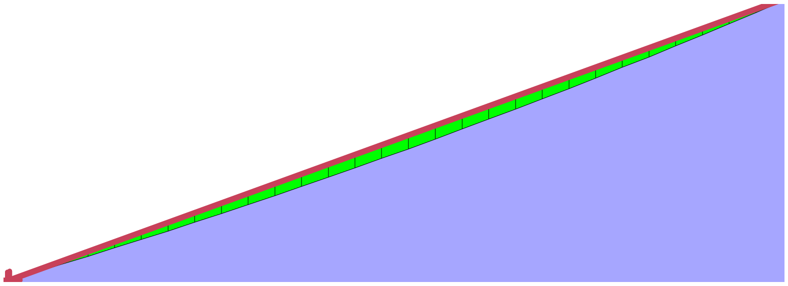


Figure 4-15 : Potential failure surface during first stage construction (Detail D)

## 4.4 Stress –deformation analysis

Stress deformation analysis was not incorporated on the original design of WWDSE 2010 but in this study Stress - deformation of the Ribb dam was analyzed. The analysis has been done for critical condition of stage construction and steady state at normal and maximum water level.

Table4-2: Summary of stress deformation analysis results

| Working conditions | Horizontal displacement (m) | | Vertical displacement (m) | Maximum principal stress (kPa) | Minimum principal stress (kPa) |
|---|---|---|---|---|---|
| | Upstream | Downstream | | | |
| End of construction | -0.06 | 0.06 | 0.535 | 2600 | 1000 |
| Normal pool level | 0.1 | 0.08 | 0.548 | 2800 | 1000 |
| Maximum water level | 0.12 | 0.08 | 0.555 | 2800 | 1000 |

The maximum value of displacement and stress analysis result under different working conditions are shown on (table 4-2).As shown on the above table the estimated horizontal displacement for upstream and downstream side at different loading condition ranges between -0.06 m to 0.12 m. Based on this result wefound the horizontal displacement computed for revised design for different loading conditions is too small as compared to similar projects with respect to dam height; exampleKesem dam projectWWDSE, (2008) which is estimated to have a horizontal displacement up to 0.52 m for static loading condition. The estimated negative and positive horizontal displacement values indicate that materials move to left and right respectively. The vertical displacement estimated for the above mentioned working conditions range from 0.535 m to 0.555 m towards the gravity. In addition the estimated maximum principal stresses for working conditions listed above varies with the range of 2600 kPa to 2800 kPa besides the estimated minimum principal stresses was same for all listed working condition i.e.1000 kPa. As per the standards given by Kutzner,(1977) the estimated stresses and deformation lies within allowable range as shown in detail on the next sections.

### 4.4.1 Displacement analysis

As indicated by Kutzner, (1997) the maximum allowable vertical settlement of embankment dams at the end of construction should not be greater than 2 % of the total dam height. For Ribb dam total height of the dam is 73.3 m from RBL therefore; the maximum vertical settlement should not exceed 1.47 m.

Similarly based onstandard given byICOLD, (1993) and Hunter and Fell (2003) the horizontal displacement of earth rock fill dam should be within the range of 0.1 to 0.5 % of the dam height. Accordingly the maximum allowable horizontal displacement accepted for Ribb revised section would be 0.4 m.

For the case of end of construction condition, the horizontal and vertical displacements have been computed and the results are shown in Figure 4-16 and Figure 4-17. At the end of construction the upstream shell material deform towards the upstream and downstream shell material deforms to the downstream (Fig.4-16). The maximum computed value of horizontal displacement for upstream and downstream were -0.06 m and 0.06 m respectively.(Negative sign shows the direction of deformation) .The computed horizontal displacement shows that it is within the allowable limit.

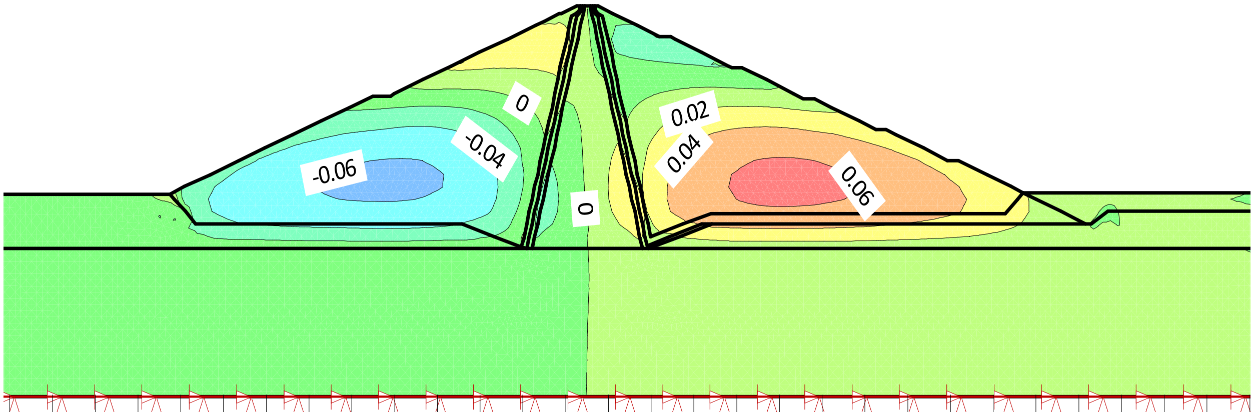


Figure 4-16 : Isoline of horizontal displacement at end of construction

The maximum vertical settlement computed inside the body of the dam is found to be 0.535 m and occurs at the center of clay core (Fig.4-17). Thus the computed maximum settlement as per the above mentioned guideline found to be (0.535/1.47*100) i.e.36.4 % of the total allowable vertical settlement so the result is within the allowable limit.

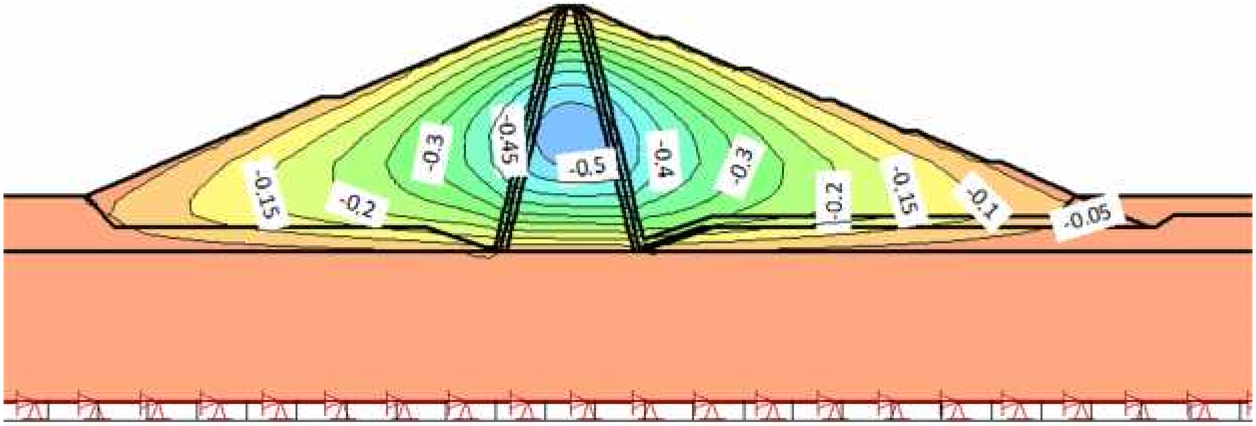


Figure 4-17: Isoline of vertical displacement at end of construction

For steady state condition of normal pool level, the horizontal and vertical displacement was computed and the deformation contour has been rearranged due to the reservoir water load as shown in Figure 4-18 and Figure 4-19.The maximum horizontal deformation in upstream and downstream dam body has increased to be 0.1 m and 0.08 m towards downstream respectively. The high horizontal component of the applied stress vector from the water load acting on the upstream face compared to its vertical component results in greater change in lateral than vertical stress in the core and downstream shoulder, so that deformation has been found dominantly horizontal than vertical moreover the computed horizontal stress lies within the allowable limit.

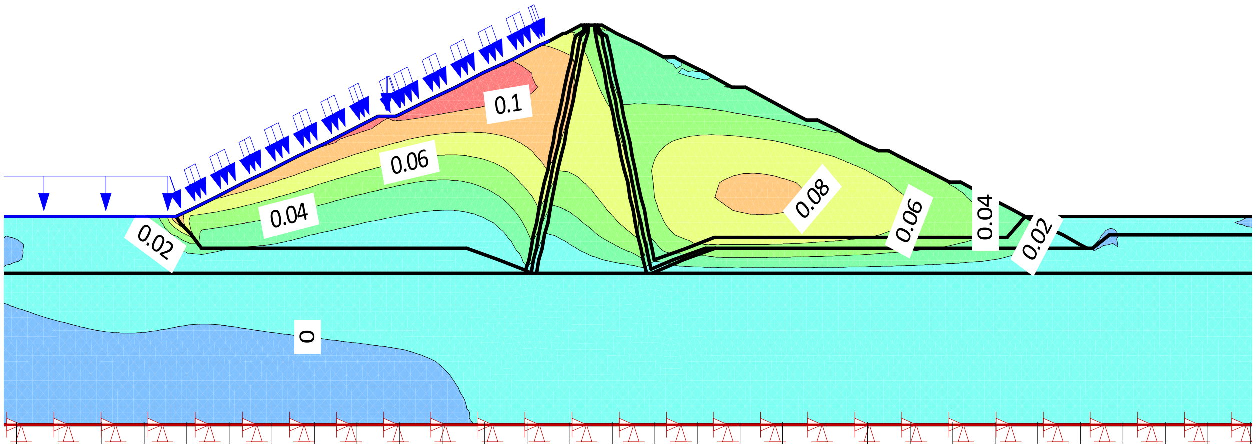


Figure 4-18 : Isoline of horizontal displacement from Normal Pool Level

Under the influence of the seepage pressure, a little vertical displacement of the dam body increased, so the maximum vertical displacement at normal pool level has been computed to be 0.547 m and occurred at center of clay coreas indicated in Figure 4-19. The estimated vertical displacement found to be (0.547/1.47*100) i.e. 37.2 % of the total allowable vertical settlement and based on the criteria suggested above the vertical deformation of the dam is within allowable limit.

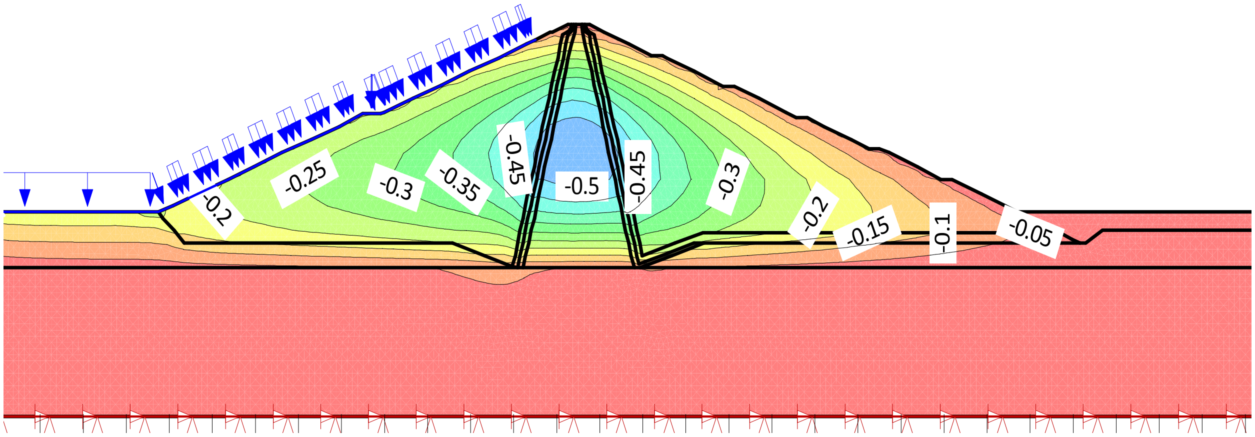


Figure 4-19: Isoline of vertical displacement underNormal Pool Level

Deformation analysis under steady state condition from the maximum reservoir water level has also been analyzed as critical loading condition. As it has been shown on Figure 4-20 and 4-21 the horizontal and vertical deformation has been computed. Based on the analysis the maximum horizontal deformations of 0.12 m and 0.08 m have been found on the upstream and downstream of shell respectively. The result show that as the water level increases the horizontal deformation does not make much change andit became almost the same as that of the result found on maximum horizontal deformation under normal pool level.

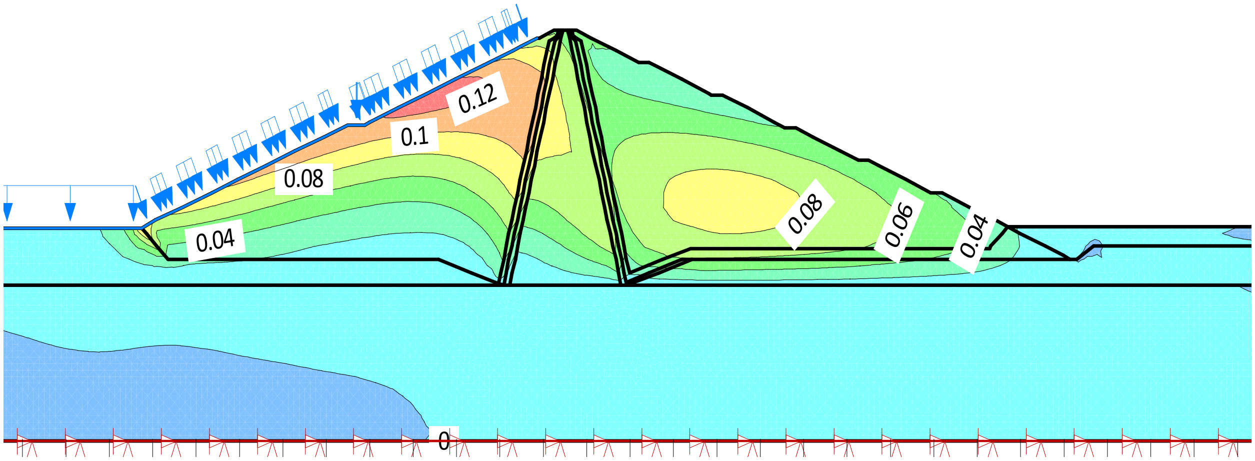


Figure 4-20: Isoline of horizontal displacement underMaximum Water Level

The maximum vertical displacement of the dam for steady state under maximum flood level estimated to be 0.555 m as shown in Figure 4-21 and the estimated vertical displacement found to be (0.555/1.47*100) i.e.37.8 % of the total allowable vertical settlement and based on the criteria suggested above the vertical deformation during maximum reservoir level is still within the allowable limit.

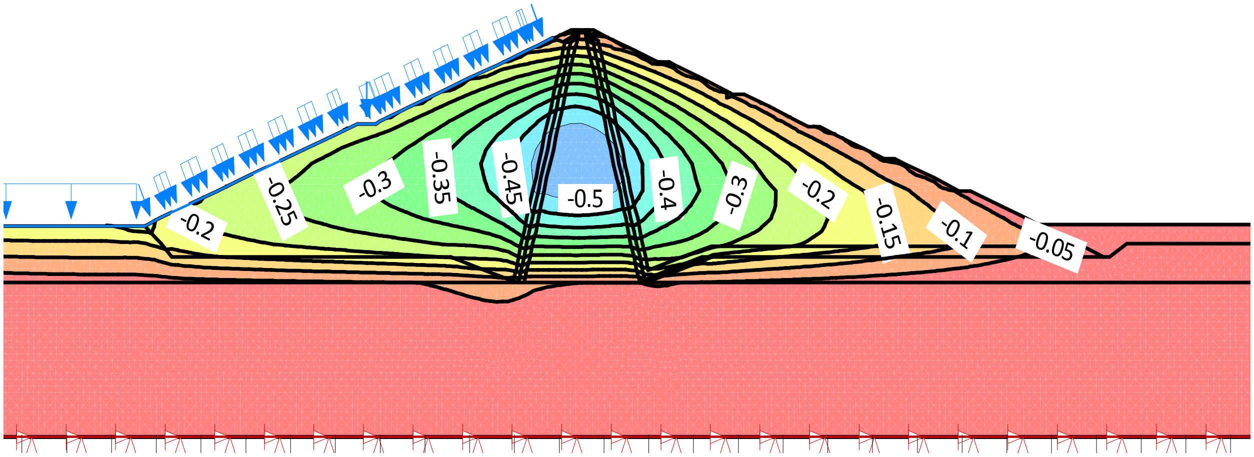


Figure 4-21: Isoline of vertical displacement underMaximum Water Level

Therefore, the dam with the substitute material was safe against both the vertical and horizontal deformation under all possible loading conditions. This ensures slightly weathered fine rock isthe proper alternative material for the construction of the dam with regards to its capability to sustain deformation under all loading conditions.

### 4.4.2 Stress analysis

Failure or cracks may happen in the body of the dam especially in the clay core associated with the stress condition. Crack or failure may happen in the dam body either due to the development of negative stress in the core material or the divatoric stress ($\sigma_1$-$\sigma_3$) at any elemental node in the core is higher than the Mohr-Columb's failure stresses value (Osuji and Anyata, 2007).The distribution of stress in side core material has been analyzed based on this Mohr-Column's stresses criterion. Based on the result the deviatoric stress at every node was found lower than Mohr-Column's stresses, which confirms that no crack is expected in the body of the damas indicated in figure 4-22.

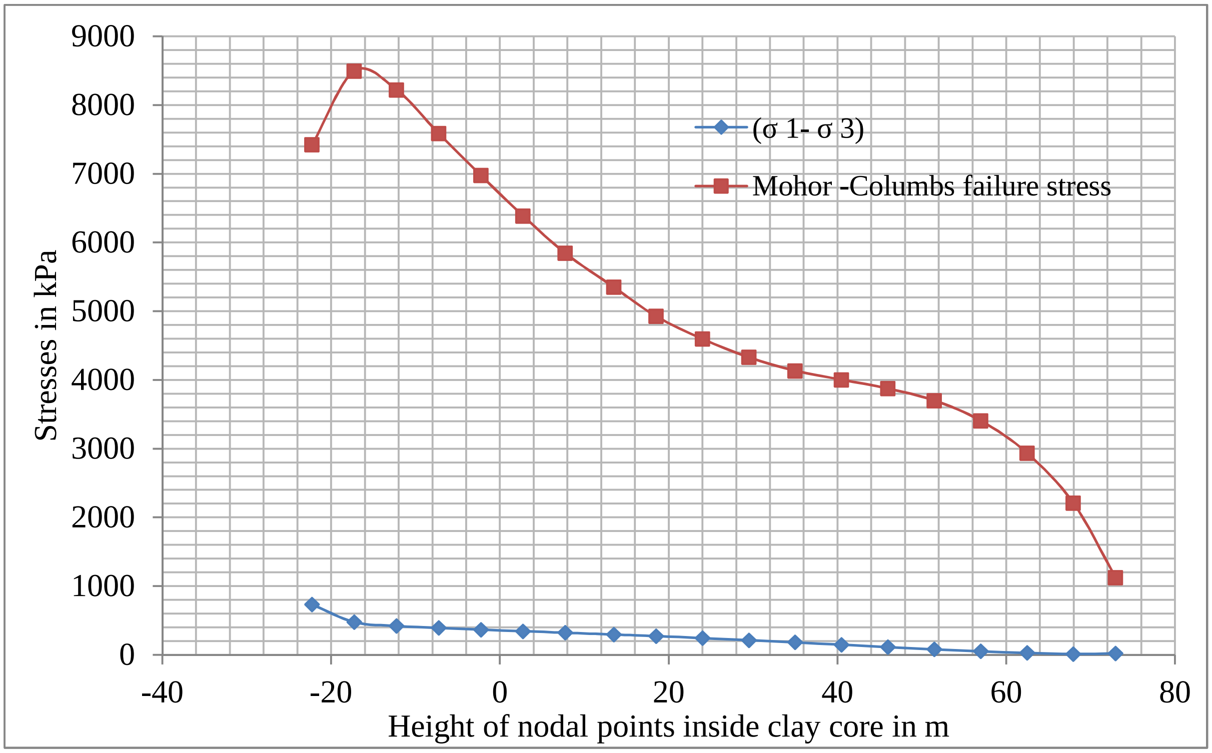


Figure 4-22: Distribution of Mohr-Coulomb's failure stresses at the elements' nodal points in the dam

Thus, Akhtarpour (2012) suggested normal stress theory for evaluation of occurrence of hydraulic fracturing phenomenonand based on his study hydraulic fracturing in the clay core occurs when the hydrostatic pressures is higher than vertical stress at nodal elements. For the case of Ribb dam project with revised material and cross section the total vertical stress at nodal elements has been compared with the respective hydraulic pressure. As it can be seen in figure 4-23 the total vertical stress is higher than the hydraulic pressure thus no risk of core crack will be expected in the body of the dam.

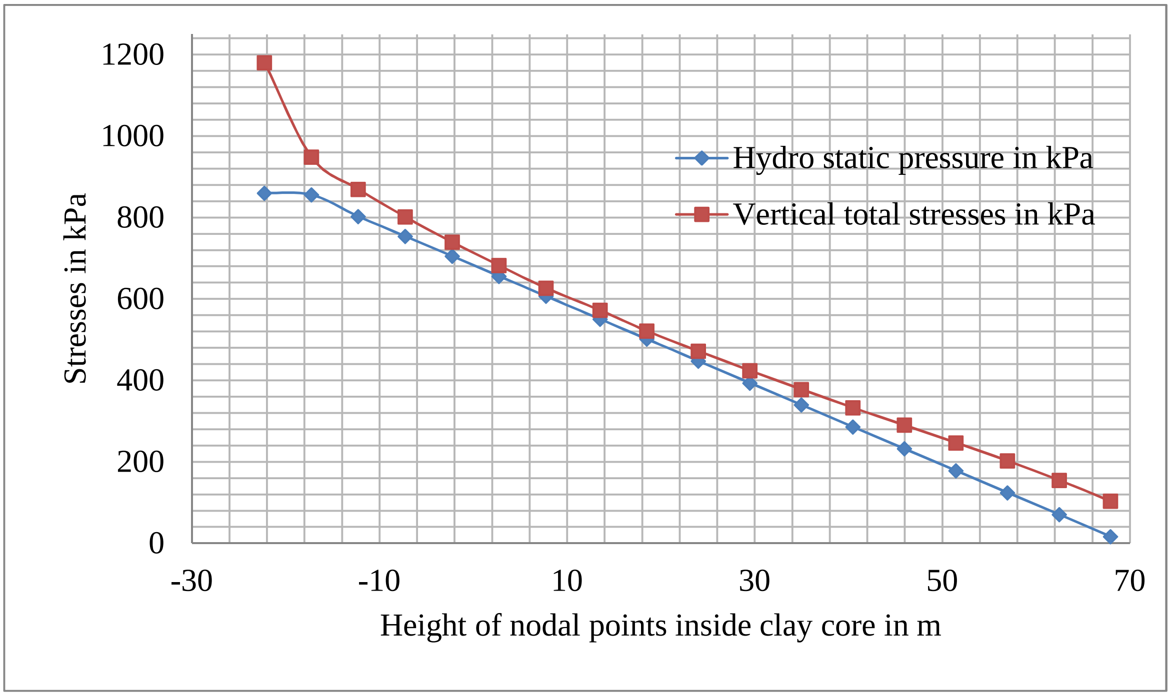


Figure 4-23: Vertical total stress versus hydraulic pressure in the dam core at nodal elements

Stress distribution at the end of construction under normal water level and maximum water level had been analyzed in figure 4-24 and in Figure B - 1 to Figure B - 3 of Appendix B. The maximum total stress at end of constructionin the foundation has been estimated to be with a range of 200 kPa to 2600 kPa as indicated in figure 4-24 and the minimum total stress estimated to be 200 kPa to 1000 kPa as indicated in figure B - 1. As the minimum principal stress inside the dam body is estimated to be greater than zero (none negative) and the clay core is not subjected to tension load (negative stress) which leads to cracking.

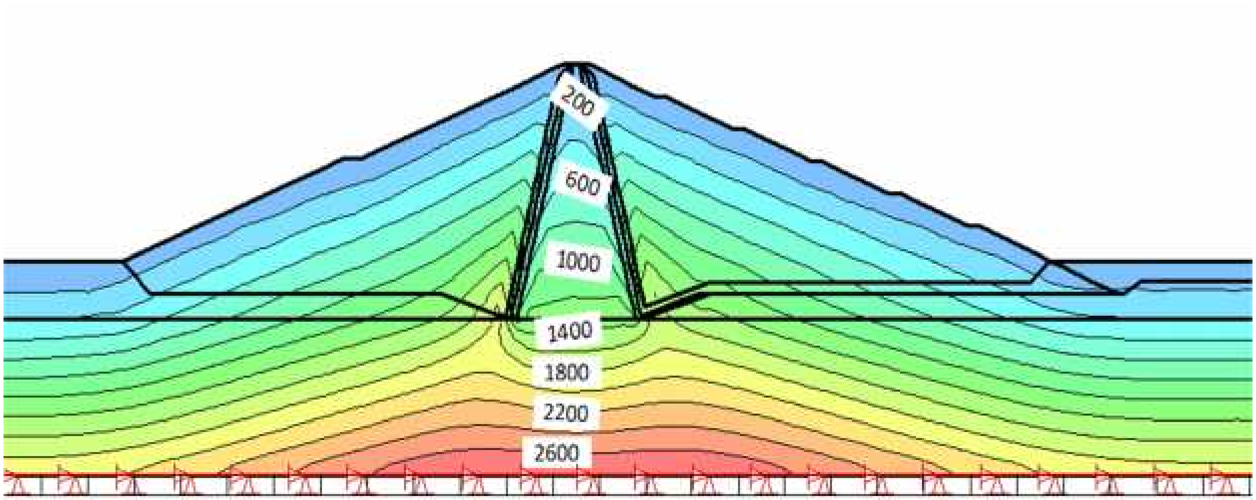


Figure 4-24 : Isoline of maximum total stress at end of construction

The maximum and minimum total stress distribution for normal and maximum water level is similar that the maximum and minimum stress has been computed to be in range between 200 kPa to 2800 kPa as indicated in figure 4-25 and 200 kPa to1000 kPa as indicated in figure B -2 and B - 3 respectively. The maximum total stress has increased relative to end of construction due to the load applied by reservoir water. The maximum stress located in the dam foundation which has high bearing capacity. Besides, the result of minimum total stress indicated that the clay is not subjected to negative stress due to the impounding of water so core crack is not expected.

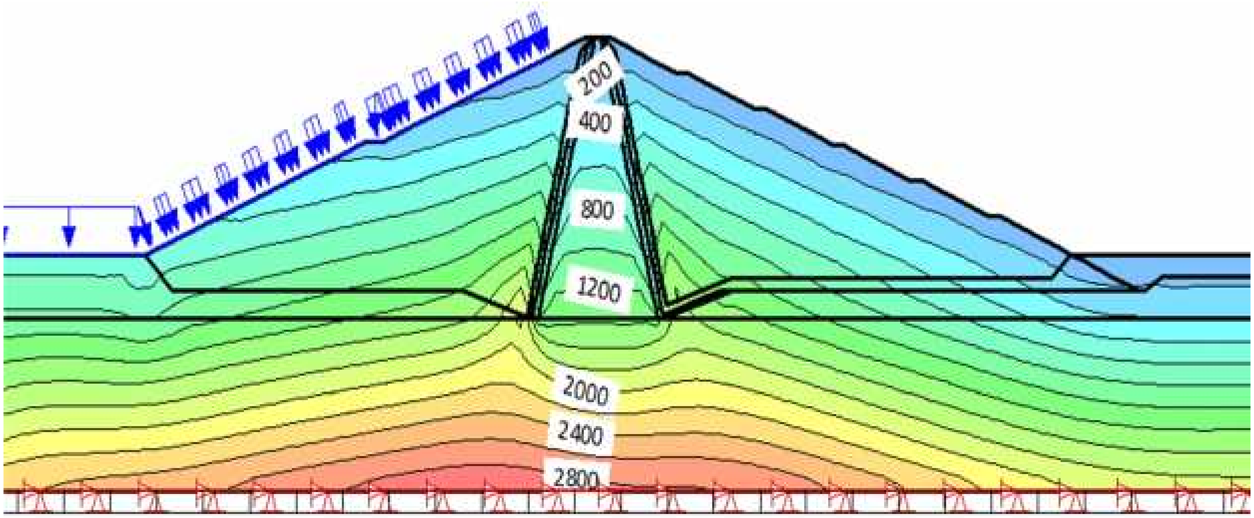


Figure 4-25 : Isoline of maximum total stress at Normal Pool Level

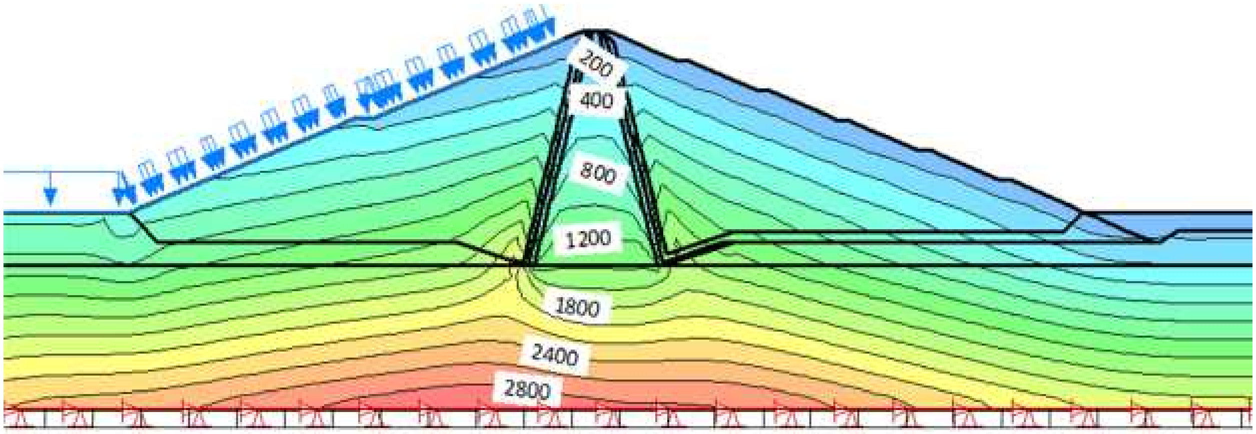


Figure 4-26: Isoline of maximum stress at Maximum Water Level

## 4.5 Riprap design

The original design incorporated a riprap with weight of rock in the riprap where 50% fine ($W_{50}$) range from maximum of 4,087 kg to minimum of 128 kg. In addition based on USBR, (1992) with a rock diameter range from 1.35 m to 0.43 m with a riprap layer thickness of 1.7m. However during this thesis work it is understood that the riprap design for the original design was not important as the upstream shell zone by itself was designed to be a free draining rock material with the size 0.7 m to 1.3 m WWDSE, (2008) and this material can reserve as upstream slope protection as the size used in shell material is almost similar to the required riprap size. But for the revised design as the rock zone is replaced with finer weathered rock it is mandatory to include riprap for upstream.

On this study after blasted rock fill material has been replaced with easily available fine rock fill material it is found necessary to incorporate riprap as the fine rock material are largely composed of gravely soils with sand and 5% fine. So as per the (USBR, 1992) design procedure fortolerable and zero damage, the riprap has been designed and results are presented in Table 4-3.

Table4-3: Summary of riprap design results

<table>
<tr><td></td><td colspan="4"><b>Weight of rock in the Rip rap where 50% is finer</b></td></tr>
<tr><td></td><td>$W_{max}$, in kg</td><td>$W_{50}$, in kg</td><td colspan="2">$W_{min}$, in kg</td></tr>
<tr><td>Tolerable damage design</td><td>3397.86</td><td>849.46</td><td colspan="2">106.18</td></tr>
<tr><td>Zero damage design</td><td>5491.81</td><td>1372.95</td><td colspan="2">171.62</td></tr>
<tr><td></td><td colspan="4"><b>Size of riprap</b></td></tr>
<tr><td></td><td>$D_{max}$ ,in m</td><td>$D_{50}$ ,in m</td><td>$D_{min,}$ in m</td><td>Thickness (2*$D_{50}$),in m</td></tr>
<tr><td>Tolerable damage design</td><td>1.27</td><td>0.80</td><td>0.40</td><td>1.61</td></tr>
<tr><td>Zero damage design</td><td>1.49</td><td>0.94</td><td>0.47</td><td>1.88</td></tr>
</table>

The summary of riprap design result indicated in table 4-3, the size of rock required for tolerable damage design which is 1.6 m smaller than for zero damage design that is 1.88 m. This is due to the assumption that zero damage design is too conservative that it assumes no maintenance and repair are required in the life time of the dam. On the other hand tolerable damage design assumes there would be some maintenance during life time of the embankment. In this study for economic considerationmaintenanceas a requirement, the size of rocks found for the case of tolerable damage has been chosen to use as riprap material with a riprap thickness of 1.6 m.As per the recommendation of USBR, (1992) for the riprap layer thickness of 1.6 m (1,610 mm) it would be appropriate to provide a bedding layer thickness of 375 mm.

## 4.6 Comparison of two construction materials

After structurally stable cross section has been found by replacing the rock fill zone with slightly weathered fine rock material the two dam cross section (designs) has been computed to examine the advantage of changing construction material. The comparison has been made on the following factors which are key for any project.

### 4.6.1 Economy

The total volume of works for the two dam cross section has been computed as shown from table C - 1 to table C - 3 of Appendix C.For this estimation the unit rates has been adopted from the particular dam site of Ribb dam project which is under construction. Based on the computed volume work the new dam cross section with a slightly weatheredmaterial has as increment of

3.78 $Mm^3$, from the original design .But, even if the volume of embankment has increased still the revised section has a significant economic benefit as shown in table 4-4.From the cost analysis the revised design has incurredadditional cost of 671,893,828.11ETB to the project due to the increment of slightly weatheredfine rock material required,however it also omits a considerable amount of activity approximately 2.5$Mm^3$ of blasted rock fill zone which is estimated to cost 1,169,430,267.30 ETB. From these it is shown that the revised dam design would save an amount of 497,536,439.19 ETB. The cost estimation has been done on the actual unit rate adopted from the project and the cost variation for the two design alternatives comes from the wide difference of the unit rate that the two materials have which is a real reflection of construction difficulty. The cost analysis has been presented in table 4-4 below for reference.

Table 4-4: Summary of work volume estimation

| **Activity** | **Unit** | **Unit rate** | **Old Design(with Slightly weathered fine rock and blasted rock)** | | **Revised design with Slightly weathered fine rock** | | **Addition** | | **Omission** | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | **Quantity** | **Amount in birr** | **Quantity** | **Amount in birr** | **Quantity** | **Amount in birr** | **Quantity** | **Amount in birr** |
| Slightly Weathered Fine rock fill material | $m^3$ | 178.32 | 3,455,339.31 | 616,156,106.30 | 7,223,249.97 | 1,288,049,934.41 | 3,767,910.66 | 671,893,828.11 | - | - |
| Blasted rock fill material | $m^3$ | 466.77 | 2,505,367.24 | 1,169,430,267.30 | - | - | - | - | 2,505,367.24 | 1,169,430,267.30 |
| **Total in birr = 1,785,586,373.6** | | | | | **Total in birr =1,288,049,934.41** | | **Saved money due to material replacement, in birr** | | | **497,536,439.19** |

### 4.6.2 Construction duration

Apart from the economical comparisons done, the two designs have been compared based on required construction period. For comparison purpose, the performances of two activity and consecutive construction years have been considered (i.e. 2013 and 2014 G.C) .The fill rate performance for the two years has been adopted from the particular dam site of Ribb dam project which is under construction.

Based on the data analyzed, the average fill rate for blasted rock is found to be 35,098.45 m$^3$/month and average fill rate for slightly weathered rock is 62,293.11 m$^3$/month as shown in table 4-5.With this rate it has been computed that construction of blasted rock and slightly weathered rock requires a construction period of 71.4 and 60.5 months respectively. From this it can be seen that revised section can be completed 11 months earlier as compare to construction with blasted rock fill.

Table4-5: Embankment fill rate performance for 2013 and 2014(from Ribb dam project monthly report)

| Month | Unit | **2013G.C** | | **2014G.C** | |
|---|---|---|---|---|---|
| | | Slightly weathered fine rock | Blasted Rock | Slightly weathered fine rock | Blasted Rock |
| | | 1 | 2 | 3 | 4 |
| January | m$^3$ | 57,224.00 | 0 | 131,835.30 | 50,923.48 |
| February | m$^3$ | 154,340.00 | 27,658.00 | 138,909.87 | 56,532.21 |
| March | m$^3$ | 116,608.00 | 0 | 97,237.58 | 29,480.17 |
| April | m$^3$ | 53,904.00 | 34,920.00 | 14,604.96 | 56,109.42 |
| May | m$^3$ | 79,886.12 | 0 | 27,370.50 | 25,013.80 |
| June | m$^3$ | 135,414.00 | 34,002.00 | 44,380.22 | 35,209.98 |
| July | m$^3$ | 33,335.07 | 39,945.64 | 10,748.55 | 39,604.46 |
| August | m$^3$ | 111,602.61 | 67,968.98 | 2,561.11 | 3,511.97 |
| September | m$^3$ | 104,776.00 | 36,212.00 | 0 | 0 |
| October | m$^3$ | 56,256.52 | 71,523.72 | 3,718.68 | 47,518.60 |
| November | m$^3$ | 30,663.93 | 59,213.81 | 15,010.66 | 61,806.62 |
| December | m$^3$ | 64,417.26 | 36,157.56 | 10,229.72 | 29,050.31 |
| **Total** | m$^3$ | **998,427.51** | **407,601.71** | **496,607.15** | **434,761.02** |
| **Total Fill of Fine rock(1+3),m$^3$** | | | | | **1,495,034.66** |
| **avg. of fine rock,m$^3$/month** | | | | | **62,293.11** |
| **Total Fill of Blasted rock(2+4),m$^3$** | | | | | **842,362.73** |
| **avg. of blasted rock,m$^3$/month** | | | | | **35,098.45** |

### 4.6.3 Construction difficulty

The economic and construction period advantages analyzed earlier are a real reflection of difficulty of blasted rock fill embankment. Thus replacing the blasted rock material with easily available slightly weathered fine rock will avoid construction difficulty and complexity.

### 4.6.4 Disadvantages of revised dam section

The newly revised section of Ribb dam has many advantages as mentioned on previous sections however despite its advantage it has disadvantages when considering the size of the cross section previous design was with bottom width of 352m but on the revised section it has been modified and become bottom width of 428 m. Accordingly this will result in additional excavation works and the area occupied by the revised dam cross section will increase.

# 5 CONCLUSION AND RECOMMENDATION

## 5.1 Conclusion

On this study overall stability analysis and economic comparison has been done for Ribb rockfill dam which is currently under construction. The analysis has been done with replacing ofblasted rock fill material with easily available slightly weathered fine rock fill material and the following conclusions has been made:-

- From seepage analysis the estimated amount of flux computed in this study is slightly higher than the original design as the curtain grouting was not modeled in this study. However, the computed flux is within tolerable limit and expected to decrease as curtain grouting would be conducted on the site.
- From slope stability analysis conducted, replacing the blasted rock fill material with slightly weathered fine rock and re-adjusting the embankment outer slopes, the computed factor of safety for all loading conditions has increased so that the stability of the embankment is more improved and the structure is safe against any loading conditions.
- Even though stress deformation analysis was not conducted for the original design, the result found for the embankment with slightly weathered material is within the allowable limit for both vertical and horizontal deformations. The embankment is also found to be safe and stable for shear failure due to the change in stress occur in the body of the dam.
- From the economic analysis although the volume of embankment work has increased as compare to the original design ,the overall cost of the project decreasedby28% as the newly proposed material is cheap as compared to original design.
- Considering Ribb as a case study, lack of detail geotechnical investigation hugely affects the construction of dam projects economically.

## 5.2 Recommendation

From the conclusion presented above the following recommendations has been presented:-

- Prior to any dam design intensive geotechnical and geological information has to be gathered to model and prepare more realistic design which fits to the actual site condition.
- Proper weight and budget should be allocated for investigation of geological and geotechnical site investigation works.
- All stakeholders (Client, Contractors and Consultants) of the project should have to improve their awareness on the importance of detail site investigation works.
- Designers should consider and exploit all the available materials in the project site so that the procedure should be searching for better design with available material rather than searching for better material for a typical design.
- Apart from structural safety, economy, construction complexity and construction period should be used for design of dams.

Finally, during the process of preparing this thesis the author got massive understanding and major lessons learned on the effect of poor site investigation effect on dam design and construction, the advantage of designing dam using easily available construction materials moreover practice modeling with Geo studio software.

# 6 References

Abdella, M., & Husseien, T. (2002). Causes of Construction Delay:Traditional Contracts. *International Jornal of Project Managment*, 7 p.

Akhtarpour, A. (2012). Evaluation of Hydraulic Fracturing potential in the Inclined Clay Core Dams Constructed in a Narrow Valley. *International Symposium On Dam For A Changing World*, 18p.

Bowles, J. E. (1997). *Foundation analysis and design,Fifth Edition.* Illinois: McGraw- Hill Companies, p.(15-45).

Braja, M. (2008). *Advanced Soil Mechanics.* New york: Taylor and Francis Library , 594 p.

Briaud, J.-L. (2013). *Geotechnical Engineering:Unsaturated and Saturated Soils.* New Jersey: John Wiley & Sons, Inc., Hoboken ,p .(80-100).

Cook, J. (1990). *Rock fill and Rock fill dams,Stan Wilson Memorial Lecture.* Seattle: University of Washington,p.(1-50).

Duncan, J. B. (1992). *State of the art-Static stability and deformation analysis in stability and performance of slopes and embankments,ASCE Geothechnical special pulication No.31,.* p 222-266.

Duncan, J. M., & Wright, S. G. (2005). *Soil Strength and Slope Stability.* New Jersy: Jhon Wiley and Sons,Inc,Hoboken , 309 p.

Dunkin W.C., C. &. (1987). *Design of Small Dams.* Washington DC: United States Bureau of Reclamation , p.(1-312).

Good, R. J., Bain, D. L., & Parsons, A. M. (1985). Weak rock in two rock fill dams. *Concrete-face rock fill dams-design,construction and performances.* New york: ASCE,32 p.

Hunter, G., & Fell, R. (2003). *The deformation behaviour of embankment dams.* Australia : The univerisity of new south Wales ,242 p.

ICOLD. (1993). *Rock materials for rock fill dams.Bulletin No.92.* International Commission On Large Dams.

Jansen, R. B. (1988). *Advanced Dam Engineering for Design,Construction and Rehabilitation.* New York: Van Nostrand Reinhold , p.( 368-408).

Krahn.John. (2003). The Limits of Limit Equilibrium Analysis. *The 2001 R.M.Hardy Lecture:.* Canada: GEO-SLOPE International Ltd.Suit 1400.8 p.

Krahn.John. (2007). Limit Equilibrium, Strength Summation and Strength Reduction Methods for Assessing Slope Stability. *GEO-SLOPE International, Calgary, Alberta, Canada*, 8 p.

Kutzner, C. (1997). *Earth and Rock fill dams,Principles of design and construction.* Netherlands: A.A.Balkema ,342 p.

Narita Kunitomo, D. E. (2000). *Design and Construction of Embankment Dams.* 18 p.

Osuji, S., & Anyata, B. U. (2007). Susceptability of clay core to cracks in rockfill dams by Finite element modeling. *Advanced Material Research*, p 8.

P.R.Mackenzie, & L.A.McDonald. (1985). Mangrove Creek dam:Use of soft rock for rock fill. *Concrete-face rock fill dams-design ,construction and performances.* New york: ASCE ,22 p .

Penman, A. D., & Charles, J. A. (1976). The Quality and Suitability of Rockfill Used in Dam Construction. *ICOLD,12th Congress.* Mexico: p .533-556.

Sabatini P. J, .., Bachus, R. C., Mayne, P. W., Schneider, J. .., & Zettler, T. (2002). *Geothechnical Engineering circular no.5,Evaluation of soil and rock properties.* Virginia: p.108-193.

Seep/W. (2008). *Seepage Modeling with SEEP/W 2007 An Engineering Methodology.* Canada: GEO-SLOPE International Ltd.317 p.

Sigma/W. (2008). *Stress-Deformation Modeling with SIGMA/W 2007 An Engineering Methodology*. Canada: GEO-SLOPE International Ltd.335 p.

Simons Noel, B. M. (2002). *A Short Course In Geothecnical site investigation.* USA: Thomas Telford Publishing ,369 p.

Slope/W. (2008). *Stability Modeling with SLOPE/W 2007 An Engineering Methodology.* Canada: GEO-SLOPE International Ltd.367 p.

Stematiu, D. (2006). *Dam Engineering II volume.* Jhon Wiley and sons,160 p.

Tancev, L. (2005). *Dam and Appurtenant Hydraulic Structures.* Great Britain: A.A.Balkema Publishers Leiden,The Netherlands,amenber of Tylor and Francis group plc,p.(101-368).

Tarhini, Ali Tarhini & Takwa. (2015). Designing guidlines to discover causes of Delays in construction projects,the case of Lebanon. *International Business research*, 17 p.

Turkmen, S., Taga, H., & Ozguler, E. (2013). Effect of Construction Material on Dam Type Selection of the Buyuk Karacay Dam (Hatay, Turkey). *Research Gate*, 14 p.

United States National Research Council (USNRC). (1985). *Safety of Dams : Flood and Earth quake Criteria.* Washington ,DC: National Acadamy Press,18 p.

USACE. (1970). *EM 1110-2-1902 Engineering and Design,Stability of Earth and Rock Fill Dams(Inclusive of Chapyer I).* Wasington DC: United States Association of Civil Engineers , 137 p.

USACE. (1994). *EM 1110-2-2300 Earth and Rock-Fill Dams-General Design and Construction Considerations.* Washington, DC: United States Association of Civil Engineers ,78 p.

USACE. (2004). *EM 1110-2-2300 General Design &Construction Considerations for Earth & Rock fill Dams.* Washington DC: United States Association of Civil Engineers , 130 p.

USBR. (1986). *Design standards embankment dams no.13,Chapter 1 :General Design Standards.* Denver,CO: United States Bureau of Reclamation ,30 p.

USBR. (1987). *Design Standards No 13 Embankment dams Chapter 4: Static Stability analysis.* Denver,CO: United States Bureau of Reclamation,84 p.

USBR. (1989). *Design Standards Embankment dams No.13,Chapter 12 :Foundation and earth materials investigation.* Denver,Colorado: United States Bureau of Reclamation , 92 p.

USBR. (1992). *Design Standards of Embankment Dams No. 13,Chapter 2:Embankment design.* Denver,CO: United States Bureau of Reclamation ,92 p.

USBR. (1998). *Earth manual,Part 1 third edition* (Vol. 1). Denver,CO: United States Bureau of Reclamation ,p. (1-269).

USBR. (2014). *Design Standards No.13, Embankment - Dams Chapter 8: Seepage,Phase 4(Final).* Denver Colorado: United States Bureau of Reclamation,186p.

USSD. (2011). *Materials for Embankment Dams.* USA: United States Society on Dams 145 p.

Willson, A., & Evans, J. (1991). The Use of Low Grade Rock Fill at Roadford Dam. *Preceeding of the sixth confrence of the British Dam Society held in Nottingham.* London: Thomas Telford Ltd 6p.

WWDSE. (2005). *Detai Design Report od Tendaho Earth Fill Dam.* Addis Ababa: Water Works Design and Supervsion Enterprise.

WWDSE. (2007). *Ribb Final Feasibility Report,Volume II Geological and Geotechnical investigations.* Addis Ababa: Water Works Design and Supervsion Enterprise,84p.

WWDSE. (2008). *Kesem Main Dam Design Optimization Report.* Addis Ababa: Water Works Design and Supervision Enterprise,130p.

WWDSE. (2009). *Final detail design report of Gidabo Irrigation project.* Addis Ababa: Water Works Design and Supervision Enterprise,225p.

WWDSE. (2010). *Ribb Detail Design Report.* Addis Ababa: Water Works Design and Supervision Enterprise,169p.

Yasar Zahit, O. (2010). *Deformation Behaviour of Clay Cored Rock Fill Dam In Turkey.* 116 p.

# 7 Appendix

## Appendix A: Slope stability result of end of construction and stage construction

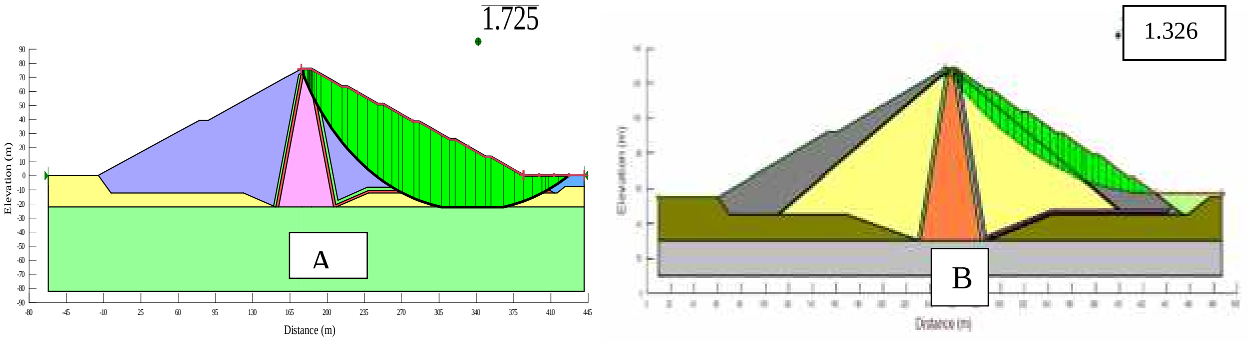


Figure A - 1: Minimum factor of safety for (A) modified section and (B) original design at end of construction

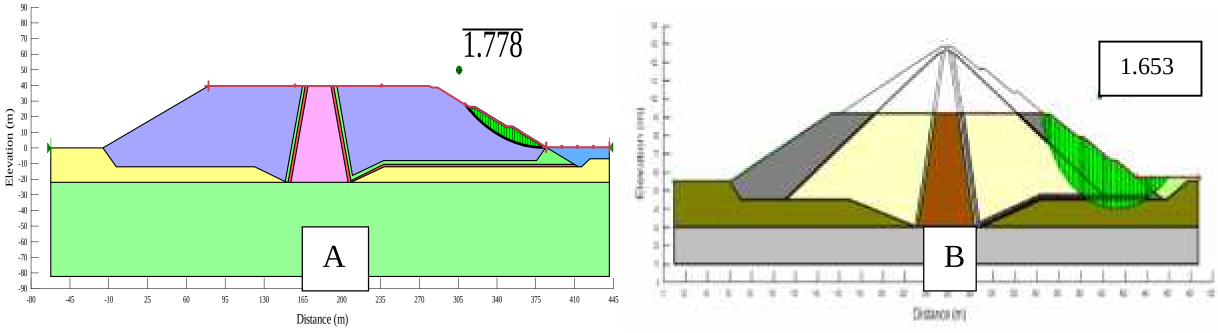


Figure A - 2 : Minimum factor of safety of (A) revised section and (B) original design during first stage construction

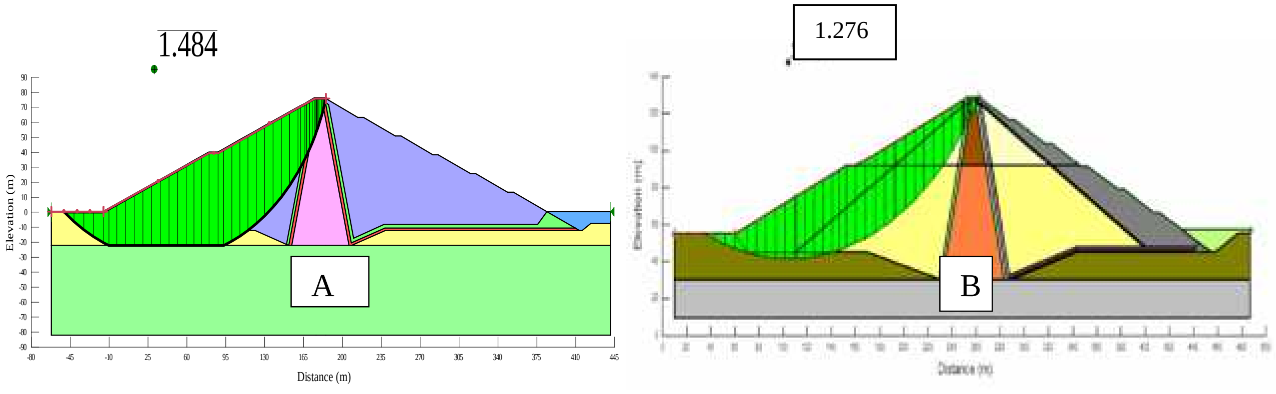


Figure A - 3: Minimum factor of safety of (A) revised section and (B) original design during second stage construction

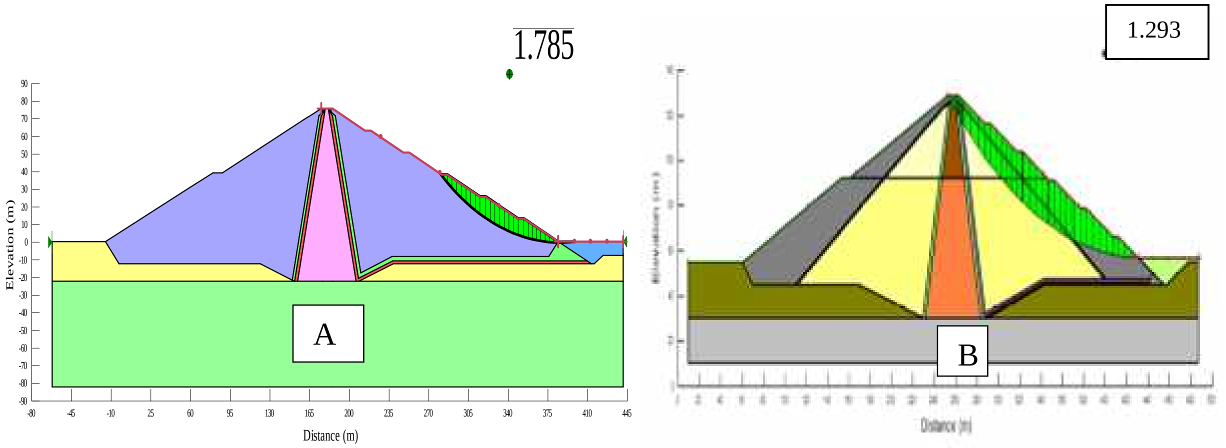


Figure A - 4: Minimum factor of safety of (A) revised section and (B) original design during second stage construction

# Appendix B: Stress deformation analysis results

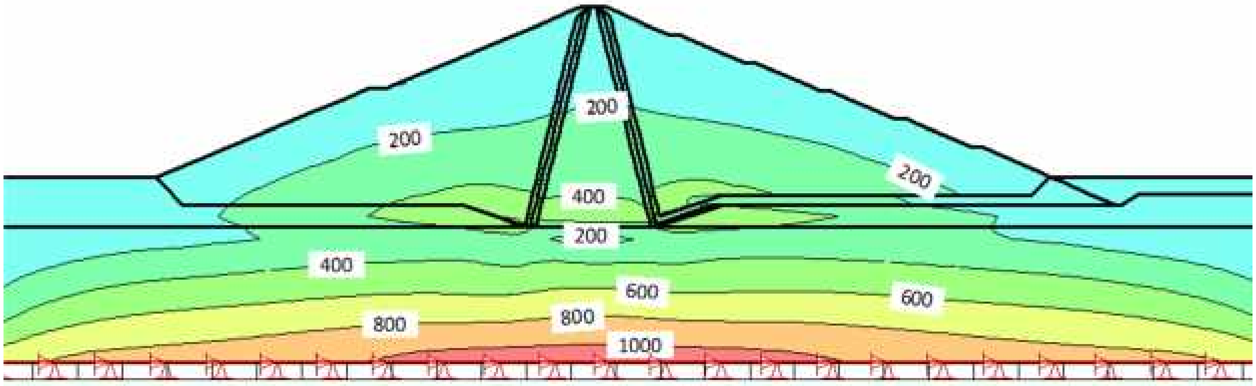


Figure B - 1: Isoline of minimum total stress at end of construction

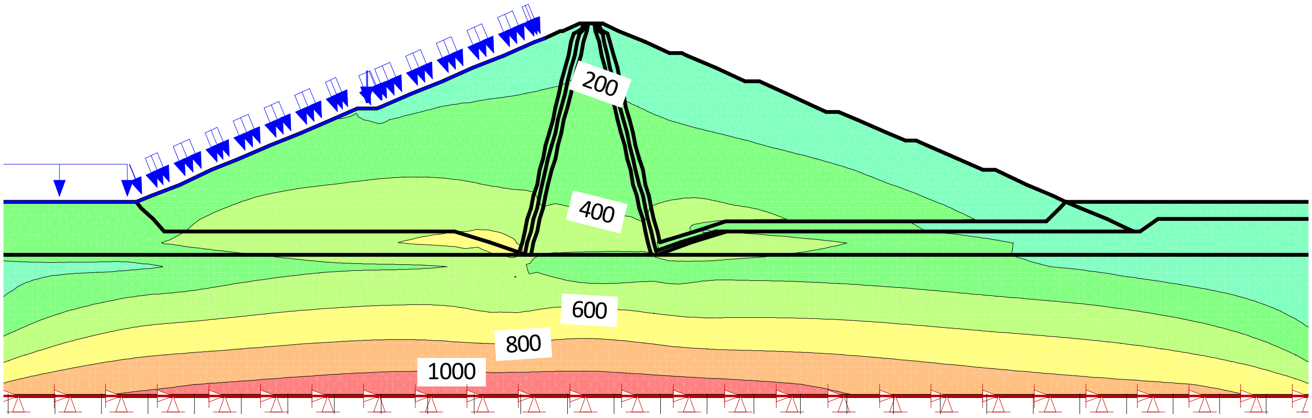


Figure B -2: Isoline of minimum total stress at normal pool level

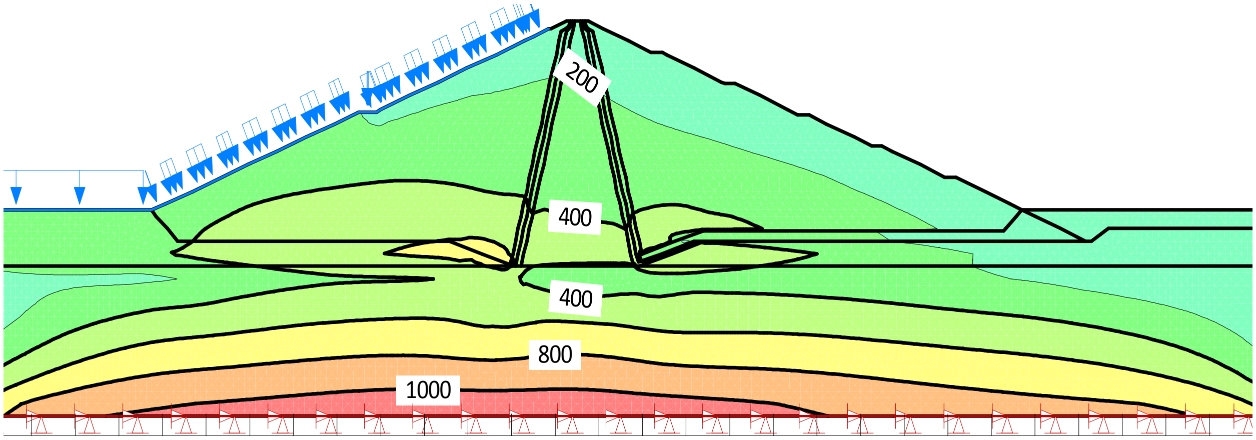


Figure B -3: Isoline of minimum total stress at maximum water level

# Appendix C:  Tables of volume estimation (fill materials for revised and original design)

Table C - 1: Additional volume of work for revised design

| **Chainage** | **Area,m$^2$** | **Average Area,m$^2$** | **Offset ,m** | **Volume,m$^3$** |
|---|---|---|---|---|
| 0+000 | 6.49 | | | |
| 0+020 | 180.30 | 93.39 | 20.00 | 1867.83 |
| 0+040 | 374.09 | 277.19 | 20.00 | 5543.89 |
| 0+060 | 559.29 | 466.69 | 20.00 | 9333.85 |
| 0+080 | 720.84 | 640.06 | 20.00 | 12801.29 |
| 0+100 | 941.63 | 831.24 | 20.00 | 16624.71 |
| 0+120 | 1133.38 | 1037.51 | 20.00 | 20750.16 |
| 0+140 | 1278.46 | 1205.92 | 20.00 | 24118.47 |
| 0+160 | 1531.17 | 1404.82 | 20.00 | 28096.33 |
| 0+180 | 1844.65 | 1687.91 | 20.00 | 33758.16 |
| 0+200 | 2069.39 | 1957.02 | 20.00 | 39140.41 |
| 0+220 | 2147.27 | 2108.33 | 20.00 | 42166.64 |
| 0+240 | 2259.67 | 2203.47 | 20.00 | 44069.35 |
| 0+260 | 2309.81 | 2284.74 | 20.00 | 45694.72 |
| 0+280 | 2302.09 | 2305.95 | 20.00 | 46118.95 |
| 0+300 | 2316.54 | 2309.32 | 20.00 | 46186.30 |
| 0+320 | 2313.40 | 2314.97 | 20.00 | 46299.42 |
| 0+340 | 2310.98 | 2312.19 | 20.00 | 46243.77 |
| 0+360 | 2314.10 | 2312.54 | 20.00 | 46250.79 |
| 0+380 | 2322.35 | 2318.23 | 20.00 | 46364.57 |
| 0+400 | 2310.64 | 2316.50 | 20.00 | 46329.96 |
| 0+420 | 2333.20 | 2321.92 | 20.00 | 46438.47 |
| 0+440 | 2290.28 | 2311.74 | 20.00 | 46234.88 |
| 0+460 | 2290.05 | 2290.16 | 20.00 | 45803.29 |
| 0+480 | 2166.89 | 2228.47 | 20.00 | 44569.33 |
| 0+500 | 2033.92 | 2100.40 | 20.00 | 42008.08 |
| 0+520 | 2038.45 | 2036.18 | 20.00 | 40723.69 |
| 0+540 | 2264.40 | 2151.42 | 20.00 | 43028.43 |
| 0+560 | 2186.88 | 2225.64 | 20.00 | 44512.75 |
| 0+580 | 2631.66 | 2409.27 | 20.00 | 48185.43 |
| 0+600 | 2113.10 | 2372.38 | 20.00 | 47447.66 |
| 0+620 | 2627.46 | 2370.28 | 20.00 | 47405.63 |
| 0+640 | 1675.87 | 2151.66 | 20.00 | 43033.26 |
| 0+660 | 1428.96 | 1552.41 | 20.00 | 31048.29 |
| 0+680 | 1048.67 | 1238.82 | 20.00 | 24776.32 |
| 0+700 | 338.29 | 693.48 | 20.00 | 13869.56 |
| 0+720 | 92.37 | 215.33 | 20.00 | 4306.57 |
| 0+740 | 14.68 | 53.53 | 20.00 | 1070.53 |
| 0+760 | 5.22 | 9.95 | 20.00 | 199.05 |
| 0+780 | 2.87 | 4.05 | 20.00 | 80.93 |
| 0+800 | 1.30 | 2.09 | 20.00 | 41.71 |
| | | | **Total** | **1,262,543.41** |
| | | | **Total in birr** | **225,010,487.27** |

Table C -2: Volume of slightly weathered rock for original design

| **Chainage** | **Area,$m^2$** | **Average Area,$m^2$** | **Offset ,m** | **Volume,$m^3$** |
|---|---|---|---|---|
| 0+000 | 0.00 | | | |
| 0+020 | 98.88 | 49.442 | 20.00 | 988.85 |
| 0+040 | 472.64 | 285.763 | 20.00 | 5715.26 |
| 0+060 | 958.99 | 715.815 | 20.00 | 14316.31 |
| 0+080 | 1459.23 | 1209.112 | 20.00 | 24182.23 |
| 0+100 | 1704.24 | 1581.738 | 20.00 | 31634.76 |
| 0+120 | 2060.17 | 1882.208 | 20.00 | 37644.16 |
| 0+140 | 2628.18 | 2344.175 | 20.00 | 46883.50 |
| 0+160 | 3133.76 | 2880.969 | 20.00 | 57619.38 |
| 0+180 | 3809.64 | 3471.703 | 20.00 | 69434.07 |
| 0+200 | 4226.30 | 4017.971 | 20.00 | 80359.42 |
| 0+220 | 4854.90 | 4540.601 | 20.00 | 90812.03 |
| 0+240 | 4654.28 | 4754.595 | 20.00 | 95091.89 |
| 0+260 | 5033.07 | 4843.677 | 20.00 | 96873.53 |
| 0+280 | 5935.82 | 5484.444 | 20.00 | 109688.88 |
| 0+300 | 6967.17 | 6451.496 | 20.00 | 129029.92 |
| 0+320 | 6949.33 | 6958.252 | 20.00 | 139165.05 |
| 0+340 | 7357.58 | 7153.456 | 20.00 | 143069.11 |
| 0+360 | 7434.66 | 7396.120 | 20.00 | 147922.39 |
| 0+380 | 7778.05 | 7606.354 | 20.00 | 152127.09 |
| 0+400 | 7818.42 | 7798.235 | 20.00 | 155964.70 |
| 0+420 | 7506.95 | 7662.686 | 20.00 | 153253.72 |
| 0+440 | 7710.41 | 7608.681 | 20.00 | 152173.62 |
| 0+460 | 7718.82 | 7714.614 | 20.00 | 154292.28 |
| 0+480 | 7627.77 | 7673.295 | 20.00 | 153465.89 |
| 0+500 | 7305.81 | 7466.794 | 20.00 | 149335.87 |
| 0+520 | 6751.22 | 7028.518 | 20.00 | 140570.36 |
| 0+540 | 6264.24 | 6507.732 | 20.00 | 130154.64 |
| 0+560 | 6020.10 | 6142.170 | 20.00 | 122843.40 |
| 0+580 | 7189.27 | 6604.685 | 20.00 | 132093.71 |
| 0+600 | 7219.82 | 7204.545 | 20.00 | 144090.89 |
| 0+620 | 7192.33 | 7206.074 | 20.00 | 144121.48 |
| 0+640 | 3656.16 | 5424.247 | 20.00 | 108484.93 |
| 0+660 | 3136.85 | 3396.503 | 20.00 | 67930.06 |
| 0+680 | 1511.67 | 2324.255 | 20.00 | 46485.10 |
| 0+700 | 538.40 | 1025.031 | 20.00 | 20500.62 |
| 0+720 | 163.62 | 351.011 | 20.00 | 7020.21 |
| | | | **Total** | **3,455,339.31** |
| | | | **Total in birr** | **616,156,106.30** |

Table C -3: Volume of blasted rock for original design

| **Chainage** | **Area,$m^2$** | **Average Area,$m^2$** | **Offset ,m** | **Volume,$m^3$** |
|---|---|---|---|---|
| 0+000 | 59.54 | | | |
| 0+020 | 385.10 | 222.32 | 20.00 | 4446.41 |
| 0+040 | 685.67 | 535.38 | 20.00 | 10707.67 |
| 0+060 | 1071.53 | 878.60 | 20.00 | 17572.01 |
| 0+080 | 1301.70 | 1186.62 | 20.00 | 23732.35 |
| 0+100 | 1518.88 | 1410.29 | 20.00 | 28205.84 |
| 0+120 | 1900.32 | 1709.60 | 20.00 | 34192.04 |
| 0+140 | 2295.96 | 2098.14 | 20.00 | 41962.85 |
| 0+160 | 2815.87 | 2555.92 | 20.00 | 51118.31 |
| 0+180 | 3333.76 | 3074.81 | 20.00 | 61496.27 |
| 0+200 | 3391.55 | 3362.66 | 20.00 | 67253.11 |
| 0+220 | 3747.41 | 3569.48 | 20.00 | 71389.57 |
| 0+240 | 3819.72 | 3783.56 | 20.00 | 75671.21 |
| 0+260 | 4161.17 | 3990.44 | 20.00 | 79808.81 |
| 0+280 | 4524.94 | 4343.05 | 20.00 | 86861.04 |
| 0+300 | 4977.04 | 4750.99 | 20.00 | 95019.74 |
| 0+320 | 5033.33 | 5005.18 | 20.00 | 100103.62 |
| 0+340 | 5048.12 | 5040.72 | 20.00 | 100814.46 |
| 0+360 | 5044.33 | 5046.23 | 20.00 | 100924.52 |
| 0+380 | 5041.82 | 5043.07 | 20.00 | 100861.48 |
| 0+400 | 5030.81 | 5036.31 | 20.00 | 100726.21 |
| 0+420 | 5011.46 | 5021.13 | 20.00 | 100422.70 |
| 0+440 | 4963.19 | 4987.33 | 20.00 | 99746.51 |
| 0+460 | 4909.66 | 4936.42 | 20.00 | 98728.49 |
| 0+480 | 4677.23 | 4793.45 | 20.00 | 95868.97 |
| 0+500 | 4335.02 | 4506.13 | 20.00 | 90122.58 |
| 0+520 | 4064.20 | 4199.61 | 20.00 | 83992.28 |
| 0+540 | 4397.60 | 4230.90 | 20.00 | 84618.00 |
| 0+560 | 4301.71 | 4349.65 | 20.00 | 86993.07 |
| 0+580 | 5107.60 | 4704.66 | 20.00 | 94093.11 |
| 0+600 | 5094.02 | 5100.81 | 20.00 | 102016.16 |
| 0+620 | 5050.10 | 5072.06 | 20.00 | 101441.11 |
| 0+640 | 2942.52 | 3996.31 | 20.00 | 79926.17 |
| 0+660 | 2520.54 | 2731.53 | 20.00 | 54630.62 |
| 0+680 | 1678.51 | 2099.52 | 20.00 | 41990.47 |
| 0+700 | 609.53 | 1144.02 | 20.00 | 22880.39 |
| 0+720 | 325.65 | 467.59 | 20.00 | 9351.83 |
| 0+740 | 92.66 | 209.15 | 20.00 | 4183.08 |
| 0+760 | 23.40 | 58.03 | 20.00 | 1160.59 |
| 0+780 | 9.95 | 16.68 | 20.00 | 333.55 |
| | | | **Total** | **2,505,367.24** |
| | | | **Total in birr** | **1,169,430,267.29** |