INVESTIGATING THE CAUSE OF FAILURES OF EMBANKMENT DAM (A CASE STUDY OF GRINDEHO EARTH FILL DAM, NORTHERN, ETHIOPIA)

SELEMAWIT ABADI GEBRU

MSc. THESIS SUBMITTED TO

HAWASSA UNIVERSITY

INSTITUTE OF TECHNOLOGY

SCHOOL OF WATER RESOURCE ENGINEERING

IN PARTIAL FULFILLMENT OF THE

REQUIREMENTS FOR THE

DEGREE OF MASTER OF SCIENCE IN DAM ENGINEERING

OCTOBER, 2017

HAWASSA, ETHIOPIA

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BY

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UNDER THE SUPERVISION OF

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HAWASSA UNIVERSITY INSTITUTE OF TECHNOLOGY
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HAWASSA, ETHIOPIA

ADVISORS’APPROVAL PAGE

This is to certify that the thesis entitled “**Investigating The Cause of Failures of Embankment Dam: A Case Study Of Grindeho Earth Fill Dam**” **Northern, Ethiopia** submitted in partial fulfilment of the requirements for the degree of master of science with specialization in Dam Engineering, the Graduate Program of the Department of **School of Water Resource Engineering,** and has been carried out by **Selemawit Abadi Gebru** Id. No Deng/013/08, under my supervision. Therefore I recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department for defense.

Moltot Zewdie (PhD)	________________	_____________
(Major- Advisor)	**Signature**	**Date**
Sirak Tekleab (PhD)	___________________	_____________
(Co-Advisor)	**Signature**	**Date**

EXAMINERS’ APPROVAL

We, the undersigned, members of the Board of Examiners of the final open defense by **Selemawit Abadi Gebru** have read and evaluated her thesis entitled **“Investigating The Cause of Failures of Embankment Dam: A Case Study of Grindeho Earth Fill Dam**” **Northern, Ethiopia,** and examined the candidate’s oral open presentation. This is, therefore, to certify that the thesis has been accepted in partial fulfilment of the requirements for the degree of Masters of Science in Dam Engineering.

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Thesis Approved by

Stamp of SGS Date: ___________

DECLARATION

I, the undersigned person, declare that this thesis is my original work and that all sources of materials used for this thesis have been accordingly acknowledged.

Name: Selemawit Abadi Gebru

Signature ____________________

Place Hawassa University

Institute of Technology

Date of submission: October, 2017

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LIST OF ABBREVIATIONS AND SYMBOLS

a.m.s.l	Above Mean Sea Level
ASTM	American Society for Testing and Materials
BDS	British Dam Society
BE&CoMU	Business Enterprise and Consultancy Office of Mekell University
c' (kN/m2)	effective (true) cohesion
CoSAERT	Commission for Sustainable Agriculture and Environmental Rehabilitation in Tigray
C_u	Coefficient of uniformity
D/S	down stream
FoS	Factor of Safety
GD	Group discussion
H	Total Head
Ha	Hectares
ICOLD	International Commission on Large Dam
K	Hydraulic Conductivity
Km	Kilometers
LE	Limit Equlibrium
LL (%)	Liquid limit
NFL	Normal Flood Level
PI (%)	Plasticity index
PL (%)	Plastic limit
SC	Clayey Sand

SEEP/W	Finite Element Software for Ground Water (seepage) Analysis
SIGMA/W	Limit Equilibrium Software for Stress-Strain Analysis
SLOPE/W	Limit Equilibrium Software for Slope Stability Analysis
SP	Poorly graded SAND with GRAVEL
TWRB	Tigray Water Resource Bureau
U/S	Upstream,
USACE	United States Army Corps of Engineers
USBR	United States Bureau of Reclamation
USCS	Unified Soil Classification Systems
σ (kN/m^2)	Total normal stress
τ (kN/m)	Shear strength per meter width of a slice of length

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ABSTRACT

Grindeho earth fill dam located at Agulae near Wukro town, Tigray Region, North Ethiopia. Grindeho earth fill dam planned to harvest 4.2MCM of water from a catchment area of about 78.03 km^2 to introduce a modern irrigation practice downstream side of the dam. The planned net irrigable area was about 400ha. However, the dam performs under capacity and does not give service yet as a result of high water loss. Therefore, the main objective of this study was to focus on the investigation of cause and type of failures of Grindeho earth fill dam. To achieve the objectives of this study, primary and secondary data has been collected from the study site, design documents, internet sources related research, references and books for emphasis the literature review, and interviewing the site engineer, hydro geologist and others. The analysis was conducted using GEO-SLOPE software to analyze the seepage flux, slope stability and stress deformation. The amount of Seepage losses through the abutments of the dam based on the design document and laboratory test data are $8.9 *10^{-4}$ m^3/sec and $1.1*10^{-2}$ m^3/sec, respectively, however the actual seepage losses of the dam based on measurement of the seepage flow at the downs stream side is 0.221 m^3/sec which is much greater than the expected as per the design, the dam has been exposed for piping failure.

The stability analysis results based on design document data as well as laboratory test data indicated that the critical upstream and downstream slope of the dam are safe for the possible loading and operation cases.

The stress-deformation analysis during the empty and full reservoir conditions and the maximum horizontal and vertical displacements are calculated. The maximum horizontal and vertical settlement for full reservoir conditions is found to be 0.20% and 0.94% of the dam height respectively which is within the safe limit as to deformation is considered. The impervious core material is not safe against the inside cracking and hydraulic fracturing due to the horizontal displacement, however, the dam couldn't be affected by overtopping problem through the impact of current vertical settlements.

Key word: Embankment Dams; Seepage; Slope Stability; Stress-deformation.

CHAPTER ONE: INTRODUCTION

1.1 Background

For the storage and management of water in watersheds, dams are a critical and essential part of the Nation's infrastructure. Water storage in dams started early in history and dams are one of the oldest manmade constructions. Nowadays, dams are used to store water for purposes such as human consumption, food production, electricity production, industrial use, and flood protection. Today as in the past, the embankment dam is the most common type of dams, principally because its construction involves utilization of its natural construction material with little processing (USBR, 1992).

Embankment dams are classified into two main categories based on the material used for construction, such as earth fill dams and rock fill dams. The main body of rock fill dams, which should have a structural resistance against failure, consists of rock fill shell and transition zones, and core and facing zones have a role to minimize leakage through embankment. Filter zone should be provided in any type of rock fill dams to prevent loss of soil particles by erosion due to seepage flow through embankment. In earth fill dams, on the other hand, the dam body is the only one which should have both structural and seepage resistance against failure with a provided drainage facilities.

Loss of life and damage to structures, utilities and crops may result from a dam failure. Dam failures and incidents involve unintended releases or surges of impounded water. These may come from poor initial design or construction, lack of maintenance and repair, or the gradual weakening of the dam through the normal aging processes [www.leg.wa.gov/rcw/index.cfm].

Dams are valuable assets; problems can worsen however, and can become more expensive to repair if they are not solved promptly. A minor problem can turn into a major reconstruction project or even result in a complete dam failure. Most dams have seepage through or around the embankment because of water moving through the soil structure. The rate at which water moves through the embankment depends on the characteristics of soil in the embankment, how well it is compacted, the foundation and abutment preparation, and the number and size of cracks and voids within the embankment. Many seepage problems and failures of earth dams have occurred because of inadequate seepage control measures or poor/incomplete cleanup and preparation of the foundations and abutments. Seepage can lead to piping and embankment sloughing or sliding, both of which can lead to dam failure. If seepage occurs without dislodging and removing soil particles, no structural damage will result. However, if soil particles are washed away in seepage, severe problems may develop [www.waterpowermagazine.com].

Many embankment dams are constructed in Ethiopia most of which are used for irrigation purpose. However, their capacity reduces frequently before their design life time due to a number of reasons. The main causes of capacity reduction are hydrological, structural and hydraulic failure of which hydraulic failure, is the major contributed (Asmelash, 2010).

In the northern parts of Ethiopia, particularly in Tigray, rainfall is more seasonal and erratic. To achieve their goal, the Regional Government of Tigray established the 'Commission for Sustainable Agriculture and Environmental Rehabilitation for the Tigray Region (Co-SAERT)'in 1994.The target of the commission were to bring food self-sufficiency to the area mainly by development of irrigated agriculture through planning, designing and construction of dams. However, the construction of the dams did not proceed as planned. Because of different practical problems such as siltation, excessive seepage, lack of appropriate dam sites and technical problems. According to Haregeweyn et al. (2006), there are four possible reasons or sources of failure for the dams of which about 60 per cent of the studied reservoirs lose water is by

Excessive seepage through the reservoir bottom and through the dam foundation into the bedrock. This excessive seepage is related to the geological formations on which the reservoirs were built. About 50 per cent of the dams are located on thin Agulae shales, overlaying a highly jointed Antalo Limestone.

1.2 Problem Statement

Agriculture has been the backbone of Ethiopia's economy and the country has given much emphasis on this sector. However, the agricultural practices in the most part of the country are dependent on rainfall which makes it the sector running at the lower rate than expected because of two reasons; the erratic nature of rainfall on one hand and the amount and the variability owing to high spatial distribution on the other hand impede production and productivity in the majority of the regions. To this end, irrigation development is one way of making the agricultural sector sustainable through combating climate change variability. To achieve food security and to enhance agricultural production, the Tigray Region has constructed many micro dams out of which only few dams are functional at the time being.

About 70% of the total dams constructed have water retention problem. Out of these failed dams, 60% to 65% are related to geological and geo-technical reasons as indicated by various researchers as well as by the preliminary assessment of the Tigray Water Resource Bureau (Solomun, 2012). Although the magnitude of the problems differ from one project to another, the major technical problems observed in many of the micro dams include: excessive seepage, insufficient inflow, early sedimentation and structural and dam stability problems before service related to geology and geo-technical condition of the dam sites.

Moreover, most of the constructed dams were found to be economically inefficient, technically the structures deteriorated before their service life and some of them are left unused due to construction problem and lack of awareness of beneficiaries. And if so this problem is to continue without providing any remedial measures, a lot of investment will be lost in the future

Grindeho earth fill dam is constructed in year 2006-2007E.C to upgrade economic and improving of living standards of the people living on Hadnet Kebele by irrigating about 400ha gross command area in the irrigation command and surrounding areas. But now farmers and professionals have already started to complain about the performance of the dam due to increasing water loss from the dam. It is speculated that increasing leakage and seepage on dam body and from both abutments are the main problem occurred on the dam. Identifying and quantifying the seepage losses and other structural problems in the dam is vital from the perspective of dam safety and in turn enable to achieve the goals set during the design period. Sudden breakdown of the dam might have occurred if the problem is not identified and remedial measures are taken.

Thus research is initiated to investigate failure of the dam with respect to hydraulic or structural problems and to recommend remedial measures in order to make the dam irrigation project sustainable and increasing its service life.

1.3 Objective of the Study

1.3.1 General Objective

The general objective of this research was to identify probable causes of seepage and other problems in the Grindeho earth fill dam.

1.3.2 The Specific Objectives

To attain the general objective, the following specific objectives were undertaken;

- To investigate the current condition of Grindeho earth fill dam.
- To investigate the cause for the seepage problem of the dam.
- To undertake seepage, stability and Static deformation analysis.

1.3.3 Research Questions

This research will try to answer the following research questions

- How effective is the existing dam structure system?
- What will be the main problem for the seepage failure of dam?
- Is that stable the existing dam based on stability, Static deformation analysis and seepage lose estimation?

1.4 Significance of the Study

According to Haregeweyn et al. (2006), in Tigray (Northern Ethiopia), significant achievements were made, mainly from 1994 to 2002, on the development of agriculture through irrigation by employing seasonally harvested runoff using earth dams. However, most of the implemented schemes are not serving the intended purpose well because of constraints associated with both pre-project and post-project implementation. Most of the reservoirs are under risk of insufficient inflow, excessive seepage and sediment deposition. These problems are mainly attributed to the use of a poor database on hydrology and sediment yield, and the lack of adaptable methodologies for assessing controlling factors at the planning stage. Therefore, the significance of this study is;

- Investigating the causes of failure of the dams and propose remedial measures to improve the overall performance and service life of the existing dams.

- It will be extremely important to serve as a lesson for the future dam infrastructure construction in reducing the number of avoidable dam failures in the Tigray and other regions in Ethiopia.

1.5 Scope of the study

The scope of this research is focused on the probable causes of seepage and other problems of grindeho earth fill dam. The work is done based on primary and secondary data collection of engineering properties of construction materials from feasibility design reports as well as from laboratory test result. The analysis is conducted on seepage, slope stability and deformation under static condition using GeoStudio 2007 software and put the curative reclamation measure for the problem. The analysis incorporated the fully coupled effect of seepage over slope stability and static deformation. But, this study was not including the dynamic stability, static stress analysis, hydrological data analysis, field surveying and geological and geotechnical investigation of the dam.

CHAPTER TWO: LITERATURE REVIEW

2.1 Description of Earth Fill Dam

A dam containing more than 50 percent, by volume, earth fill materials (fill composed of soil and rock material's that are predominantly gravel sizes or smaller) is known as earth fill dam. The three main types of earth-fill dam's classification Based on methods of construction are hydraulic-fill dams, rolled-fill dams, and Semi-hydraulic -filled dams.

Mostly common used type of earth-fill dam are rolled-fill type of dam, the major portion of the embankment is constructed in successive, mechanically compacted layers. The material from borrow pits and that suitable from required excavations for the dam and other structures is delivered to the embankment, usually by trucks or scrapers. It is then spread by motor graders or bulldozers and sprinkled, if necessary, to form lifts of limited thickness having the proper moisture content. These lifts are then thoroughly compacted and bonded with the preceding layer by means of power rollers of the proper design and weight. Rolled-fill dams consist of three types: diaphragm, homogeneous, and zoned (USBR, 1987).

2.1.1 General description of zoned earth fill dam

The most common type of a rolled earth fill dam is zoned earth fill dam typically has a central impervious core flanked by upstream transition zones, downstream filters and drains, and then outer zones or shells composed of gravel fill, rock fill, or random fill, which are considerably stronger than the core. The shells support and protect the impervious core, transition zones, filters, and drains; the upstream pervious zone provides strength for stability against rapid drawdown; and the downstream zone provides strength to buttress the core and filters so that steeper (more economical) slopes can be used. The upstream transition zone, if necessary because of a very pervious shell, provides protection against internal erosion or washout of the core during rapid drawdown, and protection against cracking of the core. The downstream filters and drains control seepage and leakage and prevent sediment transport through any cracks in the central impervious core (USBR, 2012).

These filter-drainage layers must meet filter criteria with adjacent fill and foundation materials. They are sometimes multilayered for capacity requirements (USBR, 1987). In any case, filter criteria must be met between the impervious zone and the downstream shell and between the shell and the foundation. For most effective control of through seepage and drawdown seepage, the permeability should progressively increase from the center of the dam out toward each slope.

The pervious zones may consist of sand, gravel, cobbles, rock, or mixtures of these materials. For purposes of this text, the dam is considered to be a zoned embankment if the horizontal width of the impervious zone at any elevation equals or exceeds the height of embankment above that elevation in the dam. The maximum width of the impervious zone will be controlled by stability and seepage criteria and by the availability of material.

A dam with an impervious core of moderate width composed of strong material and with pervious outer shells may have relatively steep outer slopes, limited only by the strength of the foundation, the stability of the embankment itself, and maintenance considerations. Conditions that tend to increase stability may be decisive in the choice of a section even if a longer haul is necessary to obtain required embankment materials. If a variety of soils are readily available, the type of earth fill dam chosen should always be the zoned embankment because its inherent advantages will lead to more economical construction as it has an advantage over other types of earth dams (USBR, 1987).

2.2 Cause of Earth Fill Dam Failure

2.2.1 General

In the event of a dam failure, the energy of the water stored behind the dam is capable of causing rapid and unexpected flooding downstream, resulting in loss of life and great property damage. A devastating effect on water supply and power generation could be expected as well. Due to this reason for instance the terrorist attacks of September 11, 2001in United States of America generated increased focus on protecting the country’s infrastructure, include ensuring the safety of dams (HMPU, 2011-2016).

On a worldwide scale, it is clear that the objective of constructing stable dams is not always achieved. During the 1900–1965 periods, for example, about 1% of the 9000 large dams in service throughout the world have failed, and another 2% have suffered serious accident (ICOLD, 1986).

Failures of earthen embankment dams or dikes can generally be grouped into three classifications: hydraulic, seepage and structural (Johnston et al., 1999).

2.2.2 Hydraulic Failures

Overtopping is a form of hydraulic failure and accounts for approximately 30 % of all reported earth fill embankment failures throughout the UK (Hughes & Hoskins, 1994).

Overtopping is type of failure of embankment dams it may occur for different reasons such as: excessive inflows into the reservoir due to heavy rainfalls or the failure of upstream reservoir, landslide into reservoir, extreme waves and surges, inadequate design structure and maintenance of the structure, debris blockage of outlet or spillway and flood channel, and settlement of the embankment crest (Duricic Jasna, 2014).

Hydraulic failures from the uncontrolled flow of water over and adjacent to the embankment are due to the erosive action of water on the embankment slopes. Earth embankments or dikes are not normally designed to be overtopped and therefore are particularly susceptible to erosion. A well vegetated earth embankment or dike may withstand limited overtopping if its top is level and water flows over the top and down the face in an evenly distributed sheet without becoming concentrated in any one area. The failure under Hydraulic may occur due to by over topping, erosion of upstream face, cracking to forest action, erosion of down steam face by gully formation, erosion of downstream toe (Cameron, 2010).

Tigist(2008)assessed hydraulic failure of micro embankment dams and remedial measures (case study: Zana dam,Amhara Region, Ethiopia). The main objective of the study was to identify the causes of hydraulic failure of Zana Micro Earth Dam (MED), giving emphasis to seepage. An attempt has been made to identify the main causes of hydraulic failure of the Zana dam giving emphasis to seepage. Darcy’s phreatic line, Flow Net and SEEP/W software model were used to estimate the quantity of seepage. The biggest value of all these three methods, compared with the original estimated quantity of seepage. The author concluded that no major problem in quantification of seepage and the study identified the problem of seepage at the d/s berm is due to lack of practical experience of the designer or improper design of the filter media.

2.2.3 Seepage Failures

According to Johnston et al. (1999) failure of earth dams can be caused by seepage piping, foundation instability, deformation and deterioration, and earthquakes. However, most of the recorded failures around the world are related to seepage problems. Failure due to seepage or leakage flow must always be considered, as it accounts for approximately 40 % of all embankment failures.

To avoid failure of earth dams due to seepage, settlement, and piping, observations before and after construction are essential. During construction of earth dams, continuous field observations of deformation and pore water pressures have to be made while field observations after construction normally include seepage and the piezometric head. Without the observations, the dam may suddenly fail, and the losses of life and property damage will be great because of sudden release of a large volume of water, often with little or no advance warning. There are many causes of failure of an earth dam. From many statistics, the failure of earth dams were mainly due to seepage or piping and it is widely recommended that the monitoring of seepage through an earth dam will control the safety of the dam (Bharat and Vershney, 1995).

At most dams, some water will seep from the reservoir through the foundation. Where it is not intercepted by subsurface drain, the seepage will emerge downstream from or at the toe of embankment .If the seepage forces are large enough, soil will be eroded from the foundation and be deposited in the shape of a cone around the outlet. A continues or sudden drop in the normal pool level may be an indication that seepage is occurring (Foster, 1999).

The International Commission of Large Dams (ICOLD) also discussed in (FEMA, 2006), conducted a study on the causes of dam failure. According to the results of this study, for embankment or earth fill dams, the major structural cause for failure of fill or embankment dams was piping or seepage. Piping is the progressive erosion and subsequent removal of soil grains from within the body of the dam or foundation of the dam. Sloughing is the progressive of soil from the wet d/s face.

More than 1/3 of the earth’s dams have failed due to Piping through foundation, Piping through the dam body, and sloughing of downstream toe (Johnston et al., 1999).

2.2.4 Structural Failures

In earth dams are generally shear failures leading to sliding of the embankments or the foundations. Structural and slope instability also can cause embankment dam failure. Instability of a dam is a Serious problem and can cause sliding and movement of the embankment or foundation. Sliding can occur in embankment dams from slopes that are too steep, high pore pressures due to inadequate drainage, and loss of shear strength due to liquefaction of loose granular materials. Dams constructed on clay-shale foundations are also at risk of sliding, likely causing the embankment to fail. Seepage, different forms of erosion, and poor maintenance can also cause problems with stability. (Imbrogno, David F. 2014).

Structural failures involve the separation (rupture) of the embankment material and/or its foundation. This type of failure is more prominent in large embankment dams. However, it is not exclusive to large dams and similar occurrences may be seen on earthen embankments or dikes in New Hampshire. Structural failure of an earthen embankment may take on the form of a slide or displacement of material in either the downstream or upstream face. Sloughs, bulges, cracks or other irregularities in the embankment or dike generally are signs of serious instability and may indicate structural failure (*Environmental fact sheet, 2011*).

2.3 Seepage In Embankment Dams

Major features of the design of Embankment dam are required foundation treatment, abutment stability, seepage conditions, stability of slopes adjacent to control structure approach channels and stilling basins, stability of reservoir slopes, and ability of the reservoir to retain the water stored. These features should be studied with reference to field conditions and to various alternatives before initiating detailed stability or seepage analyses. (ICOLD, 1995).

Excessive seepage from a dam can indicate serious problems. According to (Ashok, 2005) the seepage problem can lead to failure as a result of excessive pore pressure, and/or saturation causing sloughing heave, or blowout, internal erosion of base material and soluble rock piping.

Burrows or holes created by animals such as the groundhog, woodchuck, or muskrat creates voids in the embankment or dike, which weaken the structure and may serve as a pathway for seepage. Tree roots can provide a smooth surface for seepage to travel along. When trees die, their decaying roots may leave passageways for seepage to concentrate in. pipes through the embankment may also provide smooth surfaces for seepage to concentrate along as well (Ghanbari, 2013).

There are many cause of failure of an earth dam. From many statistics, the failure of earth dam was mainly to seepage or piping and it is widely recommended that the monitoring of seepage through an earth dam will control the safety of the dam. (Vershney, 1995)

All earth dams have seepage resulting from water percolating slowly through the dam body and its foundation. Seepage must, however, be controlled in both velocity and quantity. If uncontrolled, it can progressively erode base soil from embankment and its foundation, resulting in rapid failure of dam. Erosion of the soil begins at the downstream side of embankment, either in the dam proper or the foundation, progressively out spreads toward the reservoir, and eventually develops a direct conduit (piping) to the reservoir. Piping action can be recognized by an increase seepage flow rate, the discharge of muddily or discolor water, sinkholes on or near the embankment, and a whirlpool in the reservoir. Once a whirlpool (eddy) is observed on the reservoir surface, complete failure of the dam will probably follow. As with overtopping, fully developed piping is virtually impossible to control and will likely cause failure (Mattsson.2008).

Seepage can be affect the stability of the embankment dam, namely the saturated materials, concentrated seepage, and progressive erosion towards the upstream slope of the embankment dam, accelerates the failure process. Various alternatives have been suggested to far to prevent this problem. In this regard, the application of horizontal drain has been a common method to control the seepage flow since it can alleviate the pore pressure and lower the phreatic line in the embankment. On other hand, the transient seepage due to rapid drawdown of water level in the reservoir is another critical state in an embankment (Mattsson.2008).
Detail design sometimes will be influenced heavily by the strengths of foundation and construction materials, but the basic features are usually ditched by seepage considerations (Novak, 2001).

The design should consider seepage control measures such as foundation cut-offs, adequate and Non brittle impervious zones, transition zones, drainage blankets, upstream impervious blankets and relief wells Criteria for safe design have to be so specified that they cover all possible cause of failure (USACE, 1993).

2.3.1 Effects of uncontrolled seepage on embankment dams

a) Seepage failure

Piping - occurs when soil erosion begins at a seepage exit point and erodes backwards through the dam or foundation, with surrounding soil providing a support (roof) to keep the developing pipe open classically. If seepage is concentrated through materials such as sands or cohesion-less slits, the force of the flowing water can start to remove materials at the exit point, and cause progressive erosion known as piping.

The contacts between the downstream slope and the abutments (or groins) are especially prone to seepage because the embankment, and therefore less watertight. The embankment fill near the abutments is less dense because compaction is difficult along the embankment/abutment interface. Also, improperly sealed porous abutment rock can introduce abutment seepage into and along the Embankment (Narmada, 2009).

Difficulties with compaction also make areas around conveyance structures like outlet works, conduits, or siphon barrels more susceptible to uncontrolled seepage problems. Seepage existing from around conveyance structures is particularly alarming because it may also indicate that there is a crack or opening in the structure that is allowing reservoir water under pressure into the embankment. Rapid erosion and an eventual breach of the embankment can result (Narmada, 2009).

Internal Migration - Internal migration can occur when the soil is not capable of sustaining a roof or a pipe. Soil particles are eroded, and a temporary void grows until a roof can no longer be supported, at which time the void collapses. This mechanism is repeated progressively until the void shortens the seepage path and leads to uncontrolled erosion and, ultimately, to breach of the dam.

Scour - Scour occurs when tractive seepage forces along a surface (e.g., a crack within the soil, adjacent to a wall or conduit, or along the dam/foundation contact) are sufficient to move soil particles into an unprotected area. Once this begins, failure from a process similar to piping or progressive erosion could occur. Suspected examples of scour at reclamation facilities include the internal erosion events at Steinaker and Fontenelle dams, as well as the failure of Teton Dam.

Internal Instability (Suffusion, and Suffosion) - are internal erosion mechanisms that can occur with internally unstable soils. Suffusion typically involves little or no change in volume of the soil mass whereas suffosion differs from suffusion because the coarser materials are not in point-to-point contact and, therefore, when they erode out, volume change is observed (e.g., sinkholes, depressions).

b) Development of pore pressures and seepage forces

Uncontrolled or poorly controlled seepage can lead to the formation of high pore pressures and/or high seepage forces in an embankment dam or its foundation. The presence of high pore pressures in the embankment or foundation results in a decrease of effective stress/strength in those soils, which makes the embankment more vulnerable to a slope failure. Similarly, seepage passing through an embankment generates seepage forces acting along the flow lines (USACE, 2004).

c) Excessive water losses

Excessive seepage flow without soil erosion is usually not a structural or dam safety related failure mode; however, it could be considered a project failure if it results in a serious loss of project water and benefits. Downstream flooding or destructively high ground water levels could also result from excessive seepage.

According to the investigatory geological data of the study area, the objective of the investigation was to collect important geotechnical data for design of the dam and identification and detail investigation of natural construction materials. Geotechnical and engineering geological condition of the dam foundation was investigated by the help of hand dug test pits and conventional field observation. But the investigation/ study was relatively shallow toward engineering geology and geotechnics than pure geology. The investigatory concluded that Grindeho dam site is generally suitable for an earth fill dam only from topographic perspectives, but in terms of subsurface engineering geological and geotechnical perspectives additional subsurface investigation (drilling) is required to help for better decision (TBoWR feasibility, 2011).

2.4 Seepage Analysis Through Embankment Dams

Usually the main objectives of a seepage analysis are to obtain a plot of pore-water pressure distribution and to determine the quantity of seepage.

Dams must be designed and maintained to safely control seepage. Excessive seepage leads to dam safety issues, if not treated carefully. Seepage analyses are carried out for the following reasons:

To estimate the phreatic surface within an embankment, the amount of seepage flow that may pass through an embankment or foundation, the relative effectiveness of various seepage reduction measures, exit gradients and/or uplift pressures at the toe of an embankment, the amount of seepage flows intercepted by drainage features and to size and optimize the configuration of these types of drainage features, and to simulate seepage through soil saturated or unsaturated soil, to evaluate the relative effectiveness of various seepage reduction measures, the effectiveness of, or to aid in the design of, dewatering systems.
All these are factors taken into consideration while suggesting remedial measures in existing dams**.** (www.iosrjournals.org)

2.4.1 Selecting an appropriate method

Darcy's Law

Darcy (1856) formulated governing flow through porous media. The formula, now known as Darcy’s law, was based on the study of water flow through vertical filters in laboratory experiments.
Darcy’s law was valid if the flow through soil is laminar. The flow of water through soil depends the dimension of interstices, which in turn, depend up on the particle size. The fine grained soil, the dimension interstices are very small and the flow is necessary laminar. In coarse grained soils, the flow is generally laminar. However, in a very coarse grained soils, such as coarse gravels, the flow may be turbulent. For flow through soil it has been found that, the flow is laminar if the Reynolds less than unit.
Darcy’s law applies equally to unsaturated flow as to saturated flow and not useful in studying flow through cracks or fractures and similar features in rocks or soil.

Flow nets

The analysis of seepage using flow net starting with drawing two sets of orthogonal (intersecting at right angles) curves. One set of curves represents flow paths (flow lines) through the porous media, while curves at right angles to the flow paths show the location of points within the porous media that have the same piezometric head (equipotential lines).

If the number of division point increases and the person in drawing flownets is skilled the result become more accurate therefore it depends on that. The flow net is a singular solution to a specific seepage condition; in other words, there is only one family of curves that will solve the given geometry and boundary conditions (Training Aids for Dam Safety Evaluation of Seepage Conditions, 2009).

SEEP/W Software

Geo-Studio is one of the most popular geotechnical software and SEEP/W is a part of this Software. SEEP/W is numerical modelling software which used to solve the practical seepage problems. The SEEP/W program is created with the combination of seepage theory and finite element method and working on saturated/unsaturated soil region (Fell et al., 2005)

The practical seepage problems are never easy to convert into a numerical modelling because of the heterogeneity of the natural soils and the varying boundary condition. Generally the boundary conditions for a seepage problem never being as same as found in the initial stage.

Therefore the Seepage analysis in SEEP/W program is divided into two categories. **Transient analysis**; - is used to know how long the embankment takes to responds for a given boundary condition. Therefore the fundamental flow properties (pressures and water flow rate) will vary with time. The analysis required an initial boundary condition as well as a destination boundary condition The SEEP/W program has ability to read the initial condition from another analysis (may be SEEP/W) and generally obtained from a steady-state analysis (John, 2010).

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. Examples of transient analyses include predicting the time it takes the core of a dam to "wet up" when the reservoir is filled quickly; or predicting where the seepage will exit the face of a dam if a heavy rainfall event is applied over the ground surface (John, 2010).

It is important to recognize that the initial conditions for a transient analysis can have a significant effect on the solution. Unrealistic initial conditions will lead to unrealistic solutions that may be difficult to interpret, especially in the early stage of the transient analysis. It is also plain to see that changing the time step magnitude can change the solution of the computed heads. Small time steps can cause overshoot in the calculation of the new heads and time steps that are too large can result in poor averaging of material properties at the midpoint of the time step. Elements that are too large can also result in poor material property averaging, while elements that are too small can lead to overshoot problems as well (Cedergren, 1989).

Steady - state analysis; - In the steady state the fundamental water flow properties such as water pressure and water flow rates never going to be changed. Practically achieving steady state is impossible. The purpose of the steady-state analysis is only to know how the initial input parameters respond to a given boundary condition. This analysis never state that how long it takes to reach a steady state. It returns a set of solved values for water pressures and water flow parameters for particular boundary conditions. A constant pressure (H) and a constant flux rate are the important boundary conditions used for a steady-state analysis (Fredlund, 1974).

Steady state analysis is selected for SEEP/W analysis. Because, the flow line in a steady state analysis is useful to follow the path of a particle of water from its entrance into the geometry to its exit. The entire path is known, because once the flow is established, it is at a steady value and therefore is fixed at that position (Fell et al., 2005).

The flow paths are simply a line based on velocity vectors in an element that a drop of water would follow under steady-state conditions; and slight variations between a flow path and a flow line should not be of concern because they are computed in entirely different ways. At best, the SEEP/W flow path should be viewed as a reasonable approximation of the flow lines within a flow net. But, in a transient analysis you cannot plot the flow path of a particle of water across the entire geometry. This is because there are no single flow lines in a transient process. Sure, water is flowing somewhere, but a single particle of water at one point in a mesh has an infinite number of possible places to flow with time, and so it is not known what the path of the particle will be. Finally, flow nets are sort of a historical reference point and having the ability to view SEEP/W results (Alberta, 2012). Hence, SEEP/W numerical modelling software will be applied in this study work.

2.5 Slope Stability Analysis of Earth Fill Dam

Slope stability analysis is performed to assess the safe design of a human-made or natural slopes (e.g. embankments, road cuts, open-pit mining, excavations, landfills etc.) and the equilibrium conditions. It is the resistance of inclined surface to failure by sliding or collapsing. The main objectives of slope stability analysis are finding endangered areas, investigation of potential failure mechanisms, determination of the slope sensitivity to different triggering mechanisms, designing of optimal slopes with regard to safety, reliability and economics, designing possible remedial measures, e.g. barriers and stabilization (Fell et al., 2005).

Slope stability analysis of earth dam is very important to ascertain the stability of the structure. The stability of earth dam depends on its geometry, its components, materials, properties of each component and the forces to which it is subjected. The design of earth dams involves many considerations that must be examined before initiating detailed stability analyses. Such as geological and subsurface explorations, the earth and/or rock-fill materials available for construction should be carefully studied (IJCIET, 2012)

Computer-assisted graphical viewing of data used in the calculations makes it possible to look beyond the factor of safety. For example, graphically viewing all the detailed forces on each slice in the potential sliding mass, or viewing the distribution of a variety of parameters along the slip surface, helps greatly to understand the details of the technique. From this detailed information, it is now becoming evident that the method has its limits and that it is perhaps being pushed beyond its initial intended purpose. Initially, the method of slices was conceived for the situation where the normal stress along the slip surface is primarily influenced by gravity (weight of the slice). Including reinforcement in the analysis goes far beyond the initial intention (Alberta, 2008).

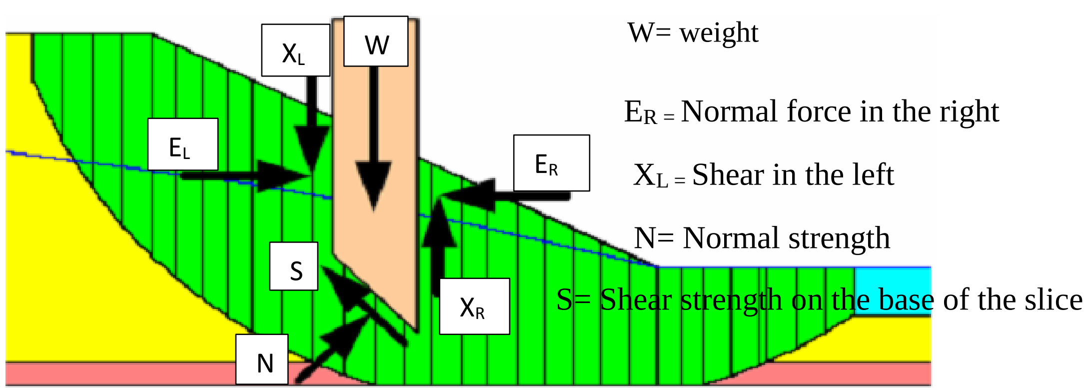

Figure 2.1: Slice discretization and slice forces in a sliding mass

2.5.1 Critical Loading Conditions Considered For Stability Analysis of Zoned Earthen Dams

Stability analysis should be carried out for various situations. Generally this analysis made on three different stages of dam construction (Kjærnsli et al., 1992).

a) End of Construction

The end-of-construction loading condition is usually analyzed for embankments that include fine grained soils, and are constructed on fine-grained saturated foundations that may develop excess pore pressures from the loading of the embankment. The embankment is constructed in layers with soils at or above their optimum moisture content that undergo internal consolidation because of the weight of the overlying layers. Both the upstream and downstream slopes of the embankment are analyzed for this condition (Novak et al., 1996; USBR, 2011).

b) Steady State Seepage at Full Reservoir

After a prolonged storage of reservoir water, water percolating through an embankment dam will establish a steady-state condition of seepage. The upper surface of seepage is called the phreatic line. According to (BDS, 1994) & (USBR, 2011), It is general practice to analyze the stability of the downstream slope of the dam embankment for steady-state seepage (or steady seepage) conditions with the reservoir at its normal operating pool elevation (usually the spillway crest elevation) since this is the loading condition the embankment will experience most.

C) Rapid Drawdown of Reservoir Level

This case represents the condition whereby a prolonged flood stage or even normal storage saturates much of the upstream portion of the embankment, and then flood load or the reservoir falls faster than the soil can drain. This can cause both higher pore pressures directly, and by causing excess pore water pressure to develop from un-drained shear. This can result in the upstream dam or water side levee slope becoming unstable. Rarely does this loading condition lead to embankment breach, but it may need to be considered in the risk assessment. (USBR, 2014).

2.5.2 Selecting an appropriate method

Limit equilibrium method and finite element method are commonly methods of stability analysis method. The limit equilibrium method is selected for analyzing stability of earth structures remains a useful tool for use in practice, in spite of the limitations inherent in the method. Care is required, however, not to abuse the method and apply it to cases beyond its limits. To effectively use limit equilibrium types of analyses, it is vitally important to understand the method, its capabilities and its limits, and not to expect results that the method is not able to provide. Since the method is based purely on the principles of statics and says nothing about displacement, it is not always possible to obtain realistic stress distributions. This is something the method cannot provide and consequently should not be expected. Fortunately, just because some unrealistic stresses perhaps appear for some slices, does not mean the overall factor of safety is necessarily unacceptable. The greatest caution and care is required when stress concentrations exist in the potential sliding mass due to the slip surface shape or due to soil-structure interaction (Alberta, 2008).

Limit equilibrium analyses applied in practice should as a minimum use a method that satisfies both force and moment equilibrium, such as the Morgenstern-Price or Spencer methods. With the software tools now available, it is just as easy to use one of the mathematically more rigorous methods than to use the simpler methods that only satisfy some of the statics equations (USACE, 2003).

On the other hand, limit equilibrium method is applied in my work due to:

➢ Cracking is always appeared on gravity dams rather than earth fill dam because earth fill dam is made up of earthen materials (USACE, 2003). Due to this reason most earth fill dams are not considered the effect of earth quake.

SLOPE/W software model

SLOPE/W is the most common and popular software application which used for the stability analysis of a slope. SLOPE/W is a software product that uses limit equilibrium theory to compute the factor of safety of earth and rock slopes. This application is created based on limit equilibrium method and included several types of methods like Ordinary, or Fellenius method, Bishop's simplified method, General limit equilibrium method, Janbu generalized method, Spencer method and Morgenstern-Price method (Sivakugan and Das, 2009).

Slope stability analysis by using limit equilibrium method in 2D and 3d shows two dimensional limit equilibrium methods will always estimate lower factors of safety compared to three dimensional methods. Slope side effect should be considered in 3D limit equilibrium methods to produce more realistic simulation of the problem. Side resistance using the at-rest earth pressure coefficient should be considered can produce lower bound 3D factors of safety while using the active earth pressure coefficient can produce upper bound factor of safety. Upstream side slope stability under sudden drawdown condition by computer programming shows the effect of horizontal drains on upstream slope of earth fill dams during rapid drawdown using finite elements and limit equilibrium methods. Changing of pore water pressure, outpouring seepage flow and factor of safety are inspected (IJCIET, 2012).

Modern limit equilibrium software such as SLOPE/W is making it possible to handle ever increasing complexity in the analysis. It is now possible to deal with complex stratigraphy, highly irregular pore-water pressure conditions, a variety of linear and nonlinear shear strength models, virtually any kind of slip surface shape, concentrated loads, and structural reinforcement.

Limit equilibrium formulations based on the method of slices are also being applied more and more to the stability analysis of structures such as tie-back walls nail or fabric reinforced slopes, and even the sliding stability of structures subjected to high horizontal loading arising, for example, from ice flows (IJCIET, 2012).

The results of stability analysis from the SLOPE/W can be obtained as both visuals and numbers. The visually interpreted results make it possible to easy understand of the results in numbers. The very important advantage of the SLOPE/W analysis is it allows handling all possible slides in a same model with the corresponding factor of safety. SLOPE/W computes the factor of safety for all specified trial slip surfaces. For probabilistic analyses, the Monte Carlo technique is used to compute the distribution of minimum factor of safety.

Table 2.1: Summary of characteristics of commonly used methods of limit equilibrium analysis (Modified after Duncan and Wright, 2005)

No.	Procedure	Equilibrium Condition Satisfied	Shape of slip surface	Assumptions	Unknowns Solved for
1.	Swedish Circle (ø = 0) Method	Moment Equilibrium about center of circle	Circular	• The slip surface is circular and • The friction angle is zero.	1 FoS = 1 Total unknown
2.	Ordinary Method of slices	Moment Equilibrium about center of circle	Circular	• The forces on the sides of the slices are neglected. • The normal force on the base of slice is Wcosα and the shear force is Wsinα.	1 FoS = 1 Total Unknown
3.	Simplified Bishop Method	Vertical equilibrium and overall moment equilibrium	Circular	• The side forces are horizontal (i.e., all inter-slice shear forces are zero.	1 FoS n Normal force on the base of slices (N)= n + 1Total unknowns
4.	Janbu's GPS Procedure	All conditions of Equilibrium	Any shape	• Assumed a position of the line of the thrust of the inter-slice forces. .	1 FoS n Normal force on the base of slices (N), n-1 Resultant inter-slice forces (Z) =2n
5.	Spencer's Method	All conditions of equilibrium	Any shape	• Inter-slice forces are parallel (i.e., all have the same inclination). • The normal force (N) acts at the center of the base of the slice.	1 FoS 1 inter-slice force inclination (θ) n Normal force on the base of slices (N), n-1 Resultant inter-slice forces (Z),n-1 Location of side forces (line of thrust) = 3n total unknowns
6.	Morgenstern and Price's Method	All conditions of equilibrium	Any shape	• Inter-slice shear force is related to interslice normal force by: $X = f(x) E$ • The normal force acts at the centre of the base of the slice.	1 FoS 1 inter-slice force inclination "scaling factor" (λ), n Normal force on the base of slices (N) n-1 Horizontal inter-slice forces (E), n-1 Location of inter-slice forces (line of thrust) = 3n Total unknowns

2.6 Deformation and Failures of Embankment Dams

2.6.1 Causes of embankment dam deformations

Results of deformations directly concern with the human life and safety (Kalkan, 2007). Earth fill dams are subject to external loads that induce deformations to the structure and the foundations. The self-weight of a dam and the reservoir water pressure are primarily responsible for the increase of stresses within the dam body, which in time result in horizontal and vertical displacements, mostly of a permanent character (Gikas and Sakellariou, 2008).

The volumetric changes are due to either an increase in the normal stresses on a soil element causing a decrease in void volume or expansion of soil elements undergoing shear. Lateral spreading and shear displacements are due to squeezing, distorting, and localized shear failures of material elements as the materials adjust to the stress conditions imposed by constructing the embankment and operating the reservoir. The rate at which these deformations happen depends on the dissipation rate of excess pore pressures and the rate at which steady-state seepage conditions develop (USBR, 2014).

2.6.2 Patterns of embankment dam deformations

Pattern of deformations for the upstream surface is down and upstream, while the downstream surface moves down and downstream. On the other hand, the crest of the dam moves down and upstream during first filling and down and downstream as reservoir water begins to penetrate the dam. Surface movements at the abutments contain an additional horizontal component of movement into the valley. The magnitudes of horizontal deformations (into and down valley) are relatively small compared to the vertical settlement. The exact ratio between the magnitudes varies with geometry, dam zoning, and material properties. (USBR, 2014)

2.6.3 Effects of deformations and mitigations

The major effects of deformations are loss of freeboard, damage to appurtenant structures located within or upon the dam, loss of confidence in the dam due to swayback appearance, cracking of the embankment (most detrimental to the impervious core), development of localized zones susceptible to hydraulic fracturing, and failure of instrumentation (USBR, 2014). The effects of deformation can usually be mitigated by designing features based on experience gained from studying historical performance of existing dams without the need for performing any elaborate analyses. For most situations, simple “rules of thumb” and/or basic settlement calculations to determine the amount of over-build or camber to place on top of a dam and settlement estimates for appurtenant structures yields satisfactory results.

According to Ksatie, (2016) the result of static stress-deformation of Grindeho dam was analyzed using the finite element method model (SIGMA/W) in the two cases; during the empty or after end of construction and full reservoir conditions. The author concluded that the impervious core material is not safe against the inside cracking and hydraulic fracturing. And the vertical settlement was occurred 0.92% of the dam height seldom exceeded to the 0.5% of the embankment height. However, during construction of Grindeho dam the USBR standard “1 percent rule” for the camber design had been considered. From this, the rate of vertical settlement is found in the vicinity of the designed freeboard. So this implied that the dam couldn’t be affected by overtopping problem through the impact of current vertical displacement.

2.6.4 Methods of deformation analysis

Predicting deformation of embankment dams and ancillary facilities is a critical in safety consideration. The response of the embankment dam under action of static loading is complex issue in the analysis of dam deformation. However, the development of two different methods of deformation surveys: geometric and physical analysis are commonly utilized to describe by physical laws, and to estimate by numerical models (Avella, 1993). In the first case, the information on the acting forces and stresses, and on the physical properties of the body are of no interest to the interpreter whereas, in the second case, the load-deformation relationship may be modelled using either a statistical or deterministic method. Geometric analysis is used to describe the state of a deformable body, and its change in shape and dimension. This method is of particular importance when the deformable structure must satisfy certain geometrical conditions, such as the crest alignment of a dam, or the knowledge on the developed strains in the material is required.

In the physical analysis, the load-deformation relationship is modelled. Commonly, this method is accomplished by using the deterministic models (Avella, 1993).The deterministic model uses the information on the sizes and locations of loads, properties of the materials (e.g., modulus of elasticity, poison ratio), and physical laws governing the stress-strain relationships to derive the relationship between the causative quantities and effects. From this relationship, the deterministic model provides information on the expected deformations using the finite element method (FEM).

2.6.4.1 SIGMA/W software:

SIGMA/W is a powerful finite element method tool that can be used to model a wide range of stress-strain problems. Its comprehensive formulation makes it possible to analyses both simple and highly complex problems of stress and deformation analysis of earth structures. A SIGMA/W model fundamentally differs from other products in Geo-Studio because of the uses of an incremental load formulation. When coupled with SEEP/W, (another GEOSLOPE software product), it can also model the pore-water pressure generation and dissipation in a soil structure in response to external loads.

2.7 Seepage Controls In Earth Fill Dam

2.7.1 General

Seepage control is necessary to prevent excessive uplift pressures, instability of the d/s slope, piping through the embankment and /or foundation, and erosion of material by migration into open joints in the foundation and abutments (Robert, 1988).

Filters are used to prevent migration of fines between various zones and foundations of embankment dams. Seepage transport of soil particles between zones can lead to serious consequences and, in extreme cases, failure of an embankment dam. Filters are also designed to prevent particle movement from internal erosion along cracks, anomalies, or defects in the embankment. Preferential flow paths can occur in earth embankments, their foundations, or at contacts between the fill and concrete structures or bedrock. In this mechanism of soil erosion, soil particles are detached by slaking along the preferential flow path (i.e., along the walls of a crack in the base soil), and the soil is subsequently eroded by water flowing at relatively high velocity (compared to the velocity of flow in inter-granular flow). The eroded particles are then carried through the preferential flow path to the filter face. Most soils are subject to erosion from this mechanism, and modern filter criteria also control this type of erosion. (USBR).

A downstream berm can be used as a remedial treatment against seepage forces and uplift pressures on the downstream face of the dam. A berm may prevent blowout by increasing the overlying weight sufficiently to resist the uplift pressures. If the berm is of low permeability, the seepage will be forced to exit further downstream. (Training Aids for Dam Safety Evaluation of Seepage Conditions)

Toe, drains drainage trenches, relief walls, horizontal drains, drainage galleries and tunnels conduit filters envelops and are some methods of seepage control measures. Toe drains basically used as the collection system for the internal drainage system in embankment, such the like a drainage source for foundation soils. In addition advantage of toe drains is for quantitative measurement of seepage to aid in observation of seepage-related behavior (Engemeon, 2012).

Drainage trenches provide to relief of pressure and a filtered outlet for seepage flow that are located at a greater depth than would be encountered with a typical toe drain. This tranches are placed in excavated and filled with filter/drainage materials of particular gradation to prevent piping of adjacent foundation soils in to the trench (Engemeon, 2012).

Relief wells are one among the seepage control methods provided to reduce excessive pore pressure in pervious foundations to a tolerable level and provide safety against high exit gradient or uplift pressures. Commonly, it is used to reduce artesian pressures in confined aquifers (Engemeon, 2012).

Drainage galleries and tunnels are features typically consists of tunnels bored into the foundation from which a series of drain holes fan out into the foundation (which typically is rock) provide to relieve pore pressures and remove and control seepage flows from beneath the embankment, often with a focus at the embankment/foundation contact. Because, potentially high gradients might develop in the immediately surrounding soils, drains that are installed near the embankment/foundation contact or in erodible rock should be filtered to minimize any potential for piping of soils into the drains (Engemeon, 2012).

CHAPTER THREE: MATERIALS AND METHODS

3.1 Description of the Study Area

3.1.1 General

Grindho dam is a 30.5 m height with a crest length of 376 m zoned type earth fill dam which is constructed in 2006 – 2007 E.C for the purpose of irrigation water supply. The dam is found in the Tekeze basin having a storage capacity of 4.28 Mm^3 water from a catchment area of about 78.03 km^2 that can irrigate 400ha of land during the dry season (TBoWR feasibility, 2011).

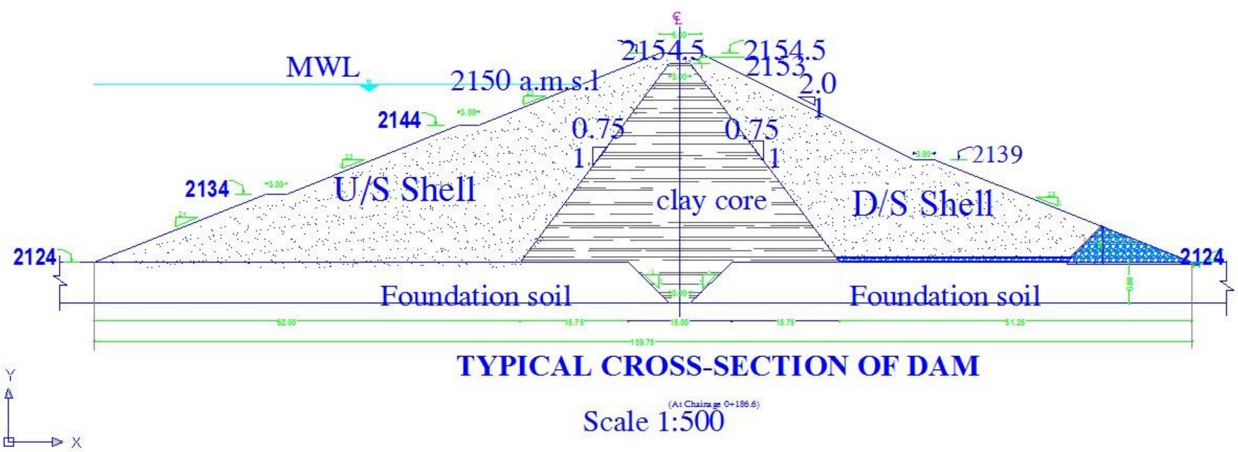

Figure 3.1: Typical cross sectional view of Grindeho dam (by: TBoWR, 2012)

Table 3.1: Structural geometry of the dam body and its appurtenant structures (TBoWR, 2012)

Main dam body		Spillway	
Dam height	30.5 m	Spillway Type	Un-gated, Ogee shaped
Dam crest length	376m	Crest length	15m
Dam crest width	6m	Design discharge:	151.20m^3/s
Upstream slope(V:H)	(1:2.5)	Flow depth over the spillway crest	Hd = 2.845m
Downstream slope(V:H)	(1:2.5), (1:2) for the 15 m and 15.5m height respectively from bottom to top	Normal water level of the reservoir:	2150.00 m a.m.s.l
		Outlet pipe level (U/s inlet):	2136.60m a.m.s.l
Foundation depth (m)	6m		

3.1.2 Location

Grindeho dam site, reservoir and its command area is located in the Northern part of Ethiopia, Tigray region. The geographic location of the dam site is 0571209 mE and 1508393 m-N. Specifically it is located in the eastern zone of Tigray Regional state, Woreda- Kelet Awlaelo and Tabia Hadent. The area is covered by topographic sheet of scale 1:50,000 numbered as 1339 B3 (Agulae). The location of Tigray Regional Government, Kelet Awlaelo woreda and the project area is shown in the figure below (Fig.3.2). Grindeho dam site is located in the south eastern side of Agulae village/town at about 27 km (Design Document BE&COMU, 2012).

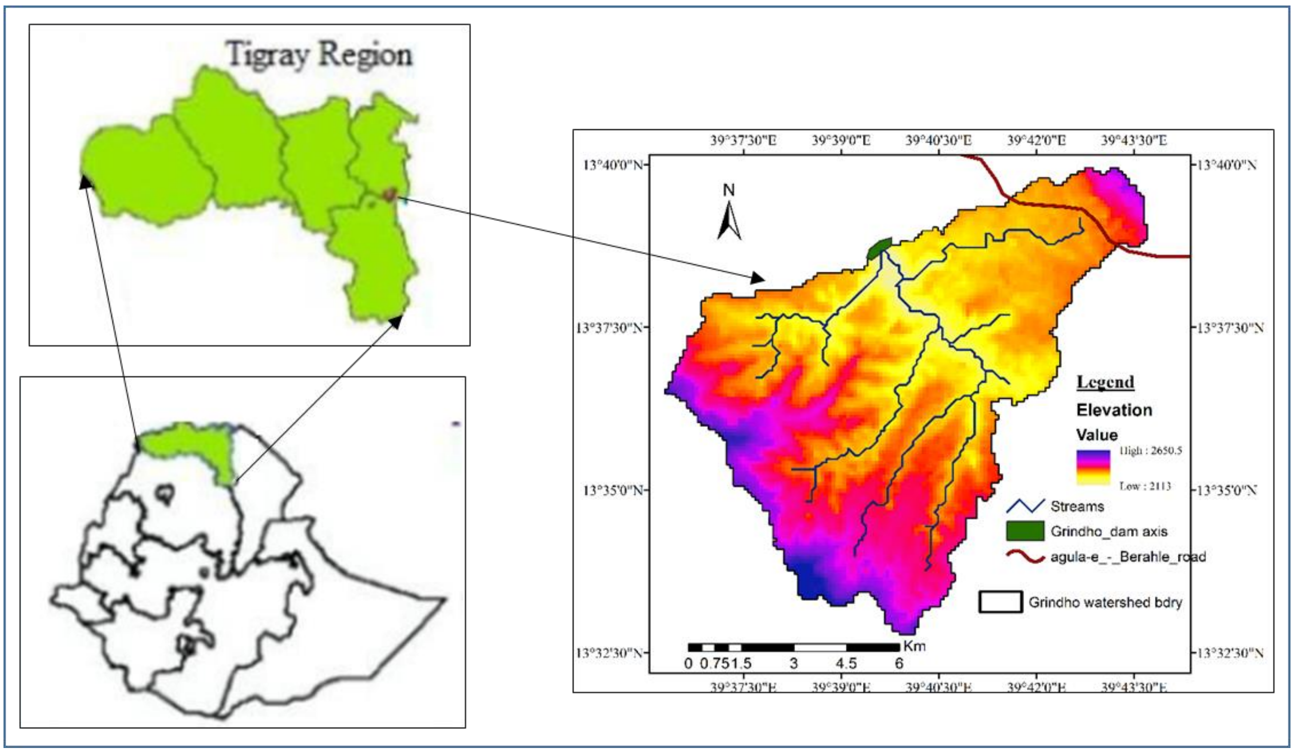

Figure 3.2: Location map and its catchment of the study area.

3.1.3 Topography

The Topography of Grindeho dam-site is Very steep sloped to undulating plane is found at this moderately degraded catchment area. Past deforestation and overgrazing practices have caused to have bare land on some steeper parts. The dominant topography in this watershed is sloping terrains having a slope range of 8-15% and covers about 39.02% of the total area. Such topographic feature is commonly found at lower & middle part of the watershed (fig.3.3). The steep landscapes are largely found at the middle and upper part of the catchment that is commonly used as grazing lands. Such topographical feature covers 24.40% of the total area. The moderately steep topographies that are commonly found at the middle and some at lower of the watershed occupy 17.35% of the total area. The gentle sloping terrain covers 15.95% of the total area and is situated on middle & lower part of the watershed (Design Document BE & COMU, 2012).

Figure 3.3: General view of Grindeho dam site, Google earth

3.1.4 Climate

Climatic condition of the area is generally classified as semi-arid (Wayna Dega) zone with an altitude that ranges from 2040 to 2840m a.m.s.l, and a mean annual temperature that varies from 12 - 26 °C. The rainy season is an erratic for about three month (June-August) with an annual rain fall that varies from 238.7-868.7 mm. 67% of the total annual rain fall is contributed from these months while the remaining rainfall is from April, May, June and September. The annual actual evapotranspiration rate of the study area is 394 mm/annum and potential evapotranspiration of 1092 mm/annum (Daniel, 2007). Tsebat river is the source of inflow or runoff for the dam which have an annual flow and sediment yield of 12.84 Mm^3 and 157,936 m^3 respectively (Vanmaercke, Amanuel *et al*. 2006).

3.1.5 Geology and Geomorphology

The geology and geomorphology of Grindeho dam site of the central foundation is composed of alluvial deposit and limestone shale-marl intercalation. The alluvial deposit is composed of clay, silt, sand and fine gravel. Within the alluvial succession there are a number of sand layers or lenses. These cyclic layers of sand are responsible for excessive leakage through foundation and also result for settlement. In general unsuitable foundation soil conditions include organic matter such as swamp deposit; low-density materials such as loose deposits of silt and sand; some clays classified as highly plastic and active/swelling and soils in a soft and saturated condition. Such materials (loose sandy deposit and highly plastic clays soils) are observed in the central foundation. In the central foundation as a whole differential settlement is also expected due to lateral variation of the materials along the foundation (Design Document BE&COMU).

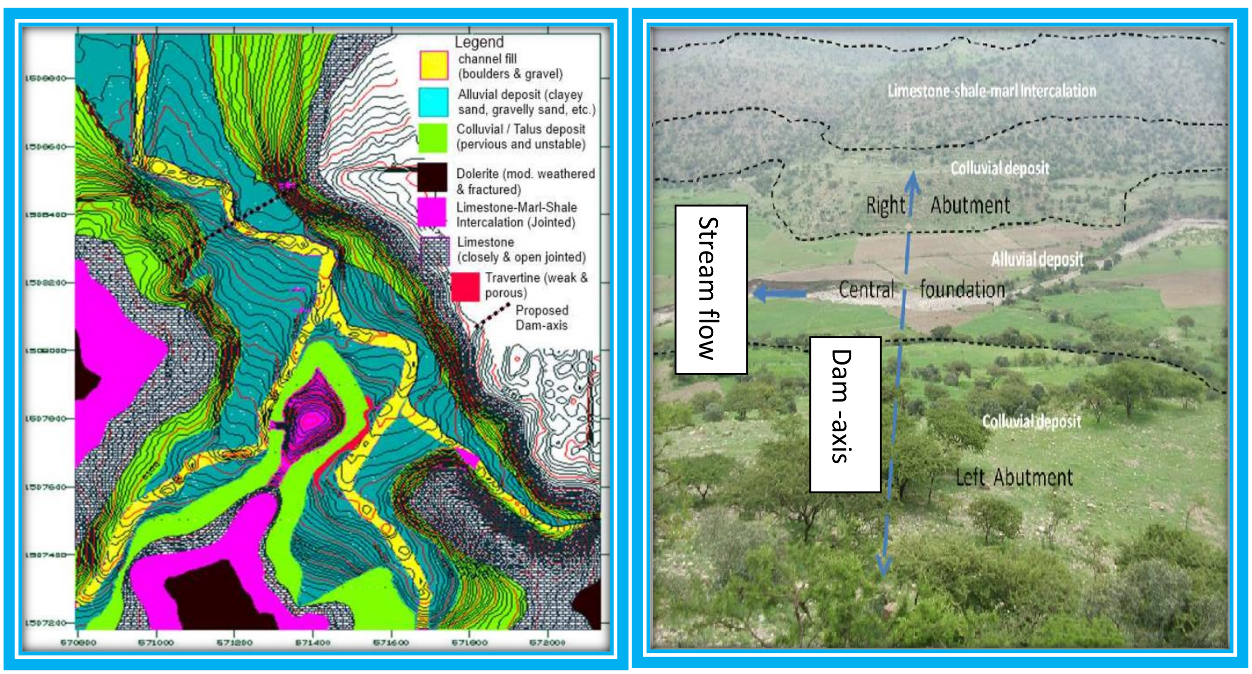

Figure 3.4: A) engineering geological map of dam site and reservoir area. B) General overview of central foundation and both abutment, (BE&COMU, 2012).

The Right Abutment; - is generally step and covered by talus/colluvial deposit. It is a loose deposit with mixture of boulder; gravel and sand of limestone origin. This deposit extends towards upstream and downstream side following the cliff forming limestone. To minimize leakage problem through the colluvial deposit and the underlying intercalation unit relatively deep cut-off trench was recommended along foundation. This recommendation has to be confirmed by drilling and packer testing during detail design. But the area that will be covered by shell material should be blanketed during construction period.

Table 3.2: Summary of laboratory results from right abutment (Design Document BE&COMU)

Pit No.	Depth (m)	LL	PL	PI	%gravel	%sand	%fines	USCS
TP01	0.6 – 1.8	46	22	24				
TP02	0.3 – 1.5	29	22	7	0.0	61.5	38.5	SC: clayey Sand
TP03	1.0 – 3.5	50	24	26	0.0	20.0	80.0	CH: Fat clay with sand

Soils of SC type are good to fair in shear strength, generally impervious and good in piping resistance, while soils of CH type are poor in shear strength (low shear strength) and impervious (10^{-6} to 10^{-8} cm/sec.) and excellent in resistance to piping.

The Left Abutment; - is relatively gentle as compare to the right abutment. The left abutment is also covered by colluvial deposit with big boulders of limestone and considerable proportion of fines. This deposit starts at the foot of cliff forming limestone and extends in the upstream and downstream side of the abutment. The other possibility is that the coarser talus/colluvial material might be deposited on top of thick clay deposit. Leakage and stability problems are expected on this abutment too.

Table 3.3: Laboratory results from left abutment

Pit No.	Depth (m	LL	PL	PI	%gravel	%sand	%fines	USCS
TP08	0.3 – 1.5	49	26	23	28.8	69.9	1.4	SW: well graded SAND with GRAVEL

Soils of SW type are excellent in terms of shear strength, but extremely previous (more than 10-3c cm/sec and fair with problems associated with piping.

LL= Liquid limit PL= Plastic limit PI= Plasticity index

3.2 Materials Used

In order to achieve the objectives, the material or tool adopted to carry out the analysis of this thesis work is Geo Studio 2007 software, field seepage measurement using vessel and calibration flow meter, with visual inspection, observation the failure of the dam, information was collected using digital camera, laboratory tests on Grain Size Analysis, Atterberg Limits, and unconfined compression test by taking the sample of material from study area. The Geo Studio software includes eight products from these eight suite products of GeoStudio-2007 three of them are utilized in this research work. SEEP/W was used for seepage analysis, SLOPE/W for slope stability analysis and SIGMA/W for stress deformation analysis.

3.3 Methodology

To carry out this thesis work, the procedures of the methodology that are followed are;

- Primary data is gathered from the study area such as with visual inspection, observation the failure of the dam, information was collected using digital camera, field seepage measurement using vessel and calibration flow meter, sample of construction material taken for laboratory tests on Grain Size Analysis, Atterberg Limits, and unconfined compression test and then characterizing the samples based on unified soil classification due to laboratory test analysis result. The other one is secondary data collection of the engineering properties of the construction materials and foundation materials (unit weight, cohesion, angle of internal friction, and hydraulic conductivity) from the design document, literatures and Tigray Regional State Bureau of Water Resources other sectors (WWDSE, 2012). After that input the analyzed data's that are taken from laboratory test and secondary data's in to GeoStudio-2007. And then analysis of the different scenarios of seepages, slope stabilities and stress-deformation based on the laboratory test result and based on design document. Due to this obtained results, a detail discussion is carried out for each analysis result, field seepage measurement and visual inspection, observation the failure of the dam by compering each other.

  Finally, conclusions and recommendations are presented based on the obtained results.

The generalized flow chart of the methodological approach deployed for this investigation is described in Figure 3.5.

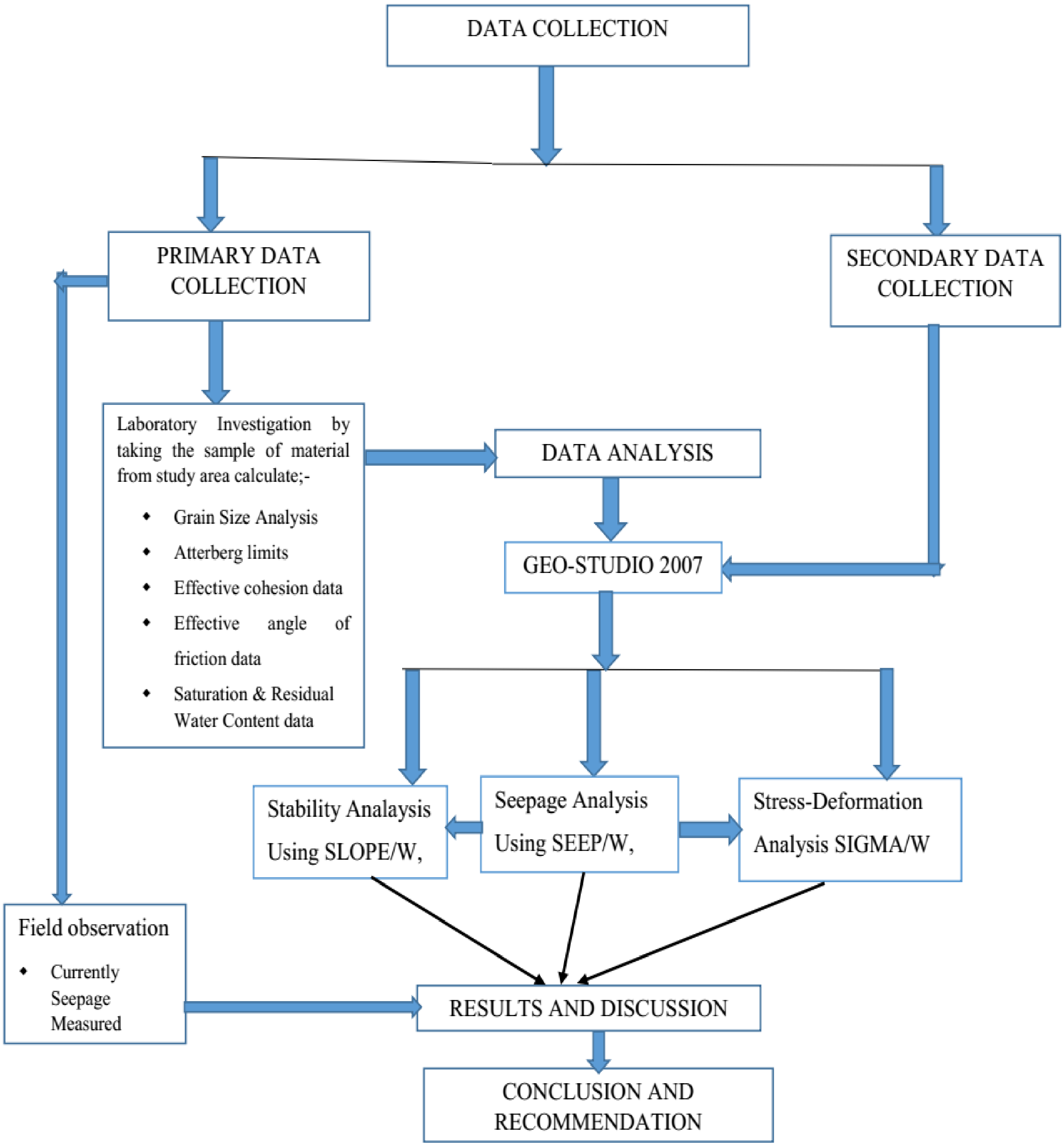

Figure 3.5: Methodology flow chart followed in this research

3.3.1 Data Collection

Primary data collection

- With visual inspection, observation the failure of the dam.
- Information was collected using digital camera.
- Currently seepage has been measured on Grindeho earth fill dam, using surface flow technique and calibration of flow meter.
- The following Laboratory Investigation was made by taking the sample of material from study area.
  - ✓ Grain Size Analysis
  - ✓ Atterberg limits
  - ✓ Effective cohesion data
  - ✓ Effective angle of friction data

Saturation & residual water content

- The geological formation of the study area has been assessed by taking an image during construction and after construction.
- Interviewing (unofficial group discussion) of the residential engineers, engineering and hydro geologists of the dam, dam operator and beneficiaries of the command area corresponding to the construction and current performance of the dam.

3.3.1.1 Currently seepage losses measured on Grindeho dam

a) Seepage measured at the left abutment using surface flow technique

Practically, the current condition of seepage on Grindeho earth fill dam is difficult to measure accurately. But as much as possible the measurement is conducted at different body of the dam fig.3.6.

Figure 3.6: current seepage measured on Grindeho dam at the left abutment (February, 2017)

Three trials were taken for an actual volume of flow measurement using a measuring vessel of 21liter size. The total discharge per unit time was determined by dividing the volume of water collected in the measuring vessel by the time taken to fill the vessel.

Trial one

The volume of the measured material is 21L

The time taken to fill this material is 12.892second

Discharge (Q) can be calculated using the formula of, Q=volume/time

Q=21L/12.892sec=1.6289 L/sec.

Trial two

Q=21L/7.23sec= 2.9 L/sec

b) Seepage measured of Grindeho dam using Current Stream flow meter

Calibration flow meter is operation that, under specified conditions establishes a relation between the quantity values (with measurement uncertainties) provided by measurement standards and corresponding indications (with associated measurement uncertainties).the calibration chart for water velocity is shown in Fig. 3.7 below.

Discharge (Q) can be calculated using the formula of,

(Q) m^3/s = Cross-sectional Area (A) m^2* Flow velocity (V) m/s

Where: Area (A) m^2 = Width (w) * Depth (d) and

Flow velocity (V) m/s = 0.000854C * 0.05 where C is counter number per minute.

According to Geopacks Operation Manual Stream Flow meters & Anemometers the method of measuring and plotting the channel for meaningful stream velocity recording at an appropriate point within each column, the flow velocity would be measured with an impeller.

Width of stream = 4.27 m and the section has been divided in to 8 columns 0.53m wide and maximum depth is 23cm. Each columns has been divided in to rectangles and/or triangles as shown fig.3.7 below.

Figure 3.7: Current seepage measured on Grindeho dam using stream flow meter (February, 2017).

3.3.1.2 Laboratory Investigation

To engineers engaged in the design and construction of earthwork for dam, the physical properties of soils such as unit weight, permeability, shear strength, and compressibility, and their interaction with water are of primary concern. The laboratory analysis is conducted by taking the sample of material from quarry area. Grindeho earth fill dam was started in 2006 and finished in 2007.The quarry site is not disappeared because it has been finished before one year. According the geological report, Clay core source is selected on the upstream side of the reservoir at about 5 km far (Fig. 3.8) below. The USCS provides a method of classifying and grouping unconsolidated earth materials according to their engineering properties (appendix Figure A2, A3 and A4).

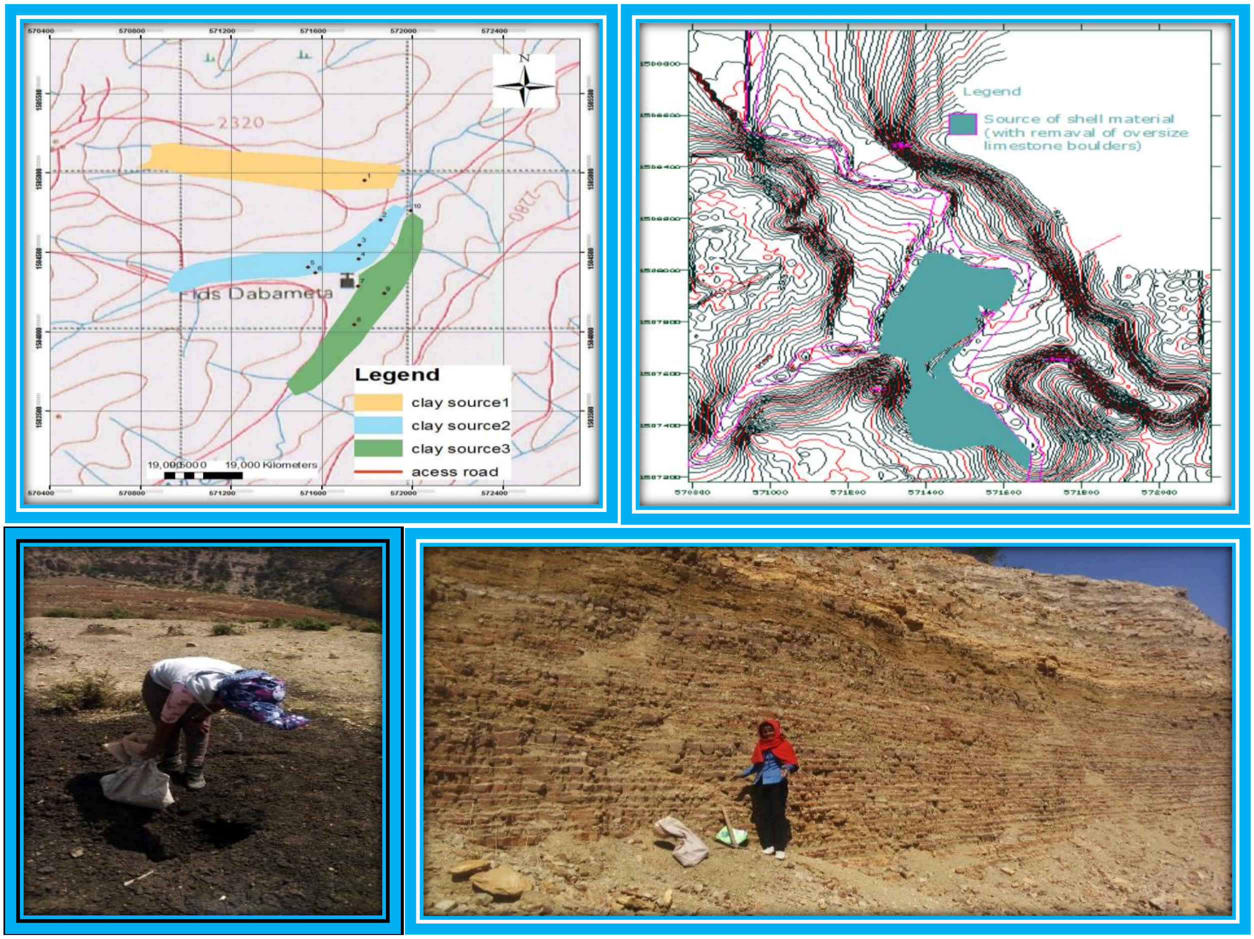

Figure 3.8: Location map of clay source for core and shell material adopt from geological report and exact location, 2017.

I. Grain size analysis

This test method covers the quantitative determination of the distribution of particle sizes in the soils. The distribution of particle sizes larger than 75µm (retained on the No. 200 sieve) is determined by sieving, while the distribution of particle sizes smaller than 75µm is determined using hydrometer analysis.

From sieve analysis and the grain-size distribute curve determine the percent passing. It is advantageous to have a standard method of identifying soils and classifying them into groups that have distinct engineering properties.

Well graded Soils that have a wide range of particle sizes and a good representation of all particle sizes between the largest and the smallest and value of $Cu > 4$ and $1 < Cc < 3$, and Poorly graded

Soils in which most particles are about the same size or have a range of sizes with intermediate sizes missing (skip grades) whereas value of $Cu < 4$ and/or $1 > Cc > 3$ (appendix Figure A2).

Where: $Cu = D60/D10$ and $Cc = (D30)^2 / (D60 * D10)$

Shell and Clay material

The procedures of sieve analysis for Shell and Clay material is as follows in fig.3.9 below.

Figure 3.9: Procedures of sieve analysis for shell and Clay material.

II. Atterberg limits

Atterberg limits are water contents at defined transitions in soil consistency, as measured by standardized tests. The procedures of Atterberg Limit test for clay material is as follows in fig.3.10 below and the same as to shell material.Soil sample is taken in an evaporating dish. It is mixed with distilled water till it becomes plastic and can be easily moulded with fingers. The plastic soil ball is rolled with fingers, the water content at which the soil can be rolled in to a thread 3mm in diameter without crumbling is plastic limit.

Figure 3.10: Procedures of Atterberg limit test for clay material.

III. Unconfined compression test

The primary purpose of this test is to determine the unconfined compressive strength, which is then used to calculate the unconsolidated undrained shear strength of the clay and shell under unconfined conditions. According to the ASTM standard, the unconfined compressive strength (qu) is defined as the compressive stress at which an unconfined cylindrical specimen of soil will fail in a simple compression test. In addition, in this test method, the unconfined compressive strength is taken as the maximum load attained per unit area, or the load per unit area at 15% axial strain, whichever occurs first during the performance of a test (Krishna, 2002). ASTM D 2166 – Standard Test Method for Unconfined Compressive Strength of Cohesive Soil.

The procedures of unconfined compression test for clay and shell material is as follows in fig.3.11.

Figure 3.11: Procedures of Unconfined compression test for clay and shell material.

Secondary data

- Essential input data for the Geo-Studio 2007 such as; Geometrical design of the dam, Geotechnical parameters of the embankment and foundation materials from laboratory testing, Geological reports and maps of the dam site and reservoir interfaces are collected from the Tigray Regional State Bureau of Water Resources in collaboration with Mekelle University office of Business and Consultancy and Defence Construction Enterprise. Additional data such as reservoir water level recorded data, photo images taken during construction, guide lines and legislations are also collected from Water Works Design and Supervision Enterprise (WWDSE, 2012).
- Secondary data has been collected from related research, references and books for emphasis the literature review, result and discussion and conclusion and recommendation.

  These secondary collected data's of geotechnical parameters of the embankment and foundation materials which are mainly used for the model simulation are amassed and summarized in the tables (3.4) and (3.5) below.

Table 3.4: SEEP/W input data's based on design document and literature review (TWRoB).

Material Type	Ks (m/s)	Liquid Limit (%)	Saturated WC (m^3/m^3)	Residual WC (m^3/m^3)	Dia. 10 % passing	Dia. 60 % passing
Shell	8.097×10^{-5}	21.5	0.41	0.057	0.002	4
Clay core	2.57×10^{-7}	30.5	0.36	0.07	0.0014	0.003
Transition Filter	0.019	0.1	0.43	0.035	0.09	0.5
Drain Filter	0.0085	0	0.40	0.018	0.2	1.1
Rock toe	0.076	0	0.0001	0.00001	200	500
Overburden	2.01×10^{-7}	28	0.38	0.1	0.002	0.05
Shale and Marl found.	1.63×10^{-8}	42	0.34	0.16	0.001	0.002
Bed rock Foundation.	4.32×10^{-9}	0	0.0001	0.000009	700	1200

Table 3.5: For SLOPE/W and SIGMA/W analysis input data's based on the design document

Material Type	γ (kN/m^3)	C' (kPa)	ϕ' (0)	ψ (°)	E (kPa)	ν
Shell	17	27.5	33.3	3	38,000	0.34
Clay core	13.7	33.7	20.96	0	32,000	0.381
Transition Filter	16.3	15	30	0	60,000	0.28
Drain Filter	21	0	38.5	8.5	75,000	0.26
Rock toe	22.2	0	42	12	240,000	0.18
Overburden	16.5	29.2	31	1	36,000	0.31
Shale and Marl found.	17	29.2	30	0	31,000	0.30
Bed rock foundation.	21	0	41.5	11.5	140,000	0.2

Where: Ks, C, γ, ϕ', k_s, ν and E represents Hydraulic conductivity, Cohesion, Unit weight, Permeability, Internal friction angle, Poison's Ratio and Young's Modulus respectively (TWRoB).

3.3.2 Analysis of data

3.3.2.1 Seepage analysis

Analysis of seepage through dam has an advantage to maintain the safety of the dam and minimizing loss of water. The collected data analyzed by Geo studio software model so as to analyze the safety and quantity of seepage of the earthen dam.

SEEP/W is one of the GEO-STUDIO software which uses a numerical model that can mathematically simulate the real physical process of water flowing through a particulate medium. Numerical modelling is purely mathematical and which is different from scaled physical modelling in the laboratory or full-scaled field modelling. The software is an extremely powerful calculator, obtaining useful and meaningful results from this useful tool depend on the guidance provided by the user. It is the user's understanding of the input and their ability to interpret the results that make it such a powerful tool. The software only provides the ability to do highly complex computations that are not otherwise humanly possible (Alberta, 2012).

An analysis of the expected quantity of seepage through the embankment and dam foundation using SEEP/W software model requires the sets of parameters like; model section of the dam, permeability coefficient of material, the piezometer reading and boundary conditions (Cedergren,1977).

The principle of SEEP/W fine element is based on Darcy's law and the rule that the sum of the rate of change of flow the x- and y- direction plus the external applied flux is equal to the rate change of volumetric water content with respect to time (krahn, 2004).

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial H}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_y \frac{\partial H}{\partial y}\right) + Q = \frac{\partial \theta}{\partial t} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots..3.1$$

Where: H- the total head

K_x and K_y - the hydraulic conductivity in the x and y direction

Q - The applied bounder flux

θ- The volumetric water content and, t- time.

SEEP/W General Considerations and Analysis Procedures

The following input data were used in SEEP/W software model (John, 2010):

- ✓ Model the cross sectional area of the dam
- ✓ Insert hydraulic conductivity and volumetric water content of the material.
- ✓ Insert boundary condition that influence the seepage, head of water above it, and the location of seepage exit where pressure head will be zero
- ✓ Locate the fluxes section where the result will be labelled
- ✓ Verify/optimize the data given
- ✓ If the data have no error, solve the problem

3.3.2.2 Slope stability analysis

Stability of an embankment slope depend on the height of the slope (H), slope angle (β) and the shear strength parameters such as cohesion (C) and the friction angle (ϕ). Among these three parameters, the height and the slope angle reduces the stability with respect to increased amount but, increasing shear strength parameters giving a more stable slope (Sivakugan and Das,2009).

Slope stability analysis using SLOPE/W software is an easy task for engineers when the slope configuration and the soil parameters are known.

However, the selection of the slope stability analysis method is not an easy task and effort should be made to collect the field conditions and the failure observations in order to understand the failure mechanism, which determines the slope stability method that should be used in the analysis. Therefore, the theoretical background of each slope stability method should be investigated in order to properly analyze the slope failure and assess the reliability of the analysis results (IJCIET, 2012)

Investigation of the stability of Grindeho earthen dam's embankment was analyzed using the limit equilibrium method of SLOPE/W-2007 software program from Geo-Slope International Ltd. with the coupled input data of SEEP/W. The analytical method for evaluating the static stability of an embankment method had been utilized in the analysis is consistent with the anticipated mode of failure, dam cross section, and soil test data.

Accordingly, the stability analyses had been carried out in order to determine the factor of safety for various critical slip surfaces. This stability of the upstream and downstream slopes of the embankment is analyses for the most critical or severe loading conditions that may occurred during the life of the dam. These loading conditions typically include: (1) End of Construction, (2) Steady-State Seepage, (3) Rapid (or Sudden) Drawdown.

General Considerations and Slope stability Analysis Procedures

The critical safety factors are computed using the most common limit equilibrium methods such as Ordinary, Bishop's Simplified, Janbu's Simplified, Morgenstern-Price and Spencer methods. In order to achieve the slope stability analysis the following procedures are considered.

a) Defining Slip Surface for Circular Failure Model

After the material inputs and SEEP/W coupled pore water pressure was assigned, a slip surface command was defined. From these several methods that commonly used to define the slip surface for the circular failure mode the entry and exit method was selected. This is due the entry and exit command is relatively accurate when compared with the other commands and it allows the user to identify slip surfaces without difficulties by specifying the assumed portion of the surface where the slip surface will enter and exit.

b) Safety Criteria

Safety evaluation of embankment dams should satisfy the recommended criterion by safety regulations or codes issued by authorized agencies. Among the numerous dam safety regulation, both (USACE, 2003) and (BDS, 1994) criterion are considered as the standard for their broad area of validation in the present studies. From these various standard of safety criterion, the most commonly utilized standards (USACE, 2003), (USBR, 2011) and (BDS, 1994) are used to validate the factor of safety of this study which is indicated in table 3.6.

Tables 3.6 shown below present the safety criteria of stability factor of safety for different cases of operation.

Table 3.6: Safety criteria standard of various enterprises

Agency	Loading condition	Stress parameter	FoS
(USACE, 2003)	End of Construction	Total and Effective	1.3
	Long term (Steady seepage condition)	Effective stress (Drained)	1.5
	Rapid drawdown	Total and Effective	1.2
(USBR, 2011)	End of Construction	Total and Effective	1.3
	Long term (Steady seepage condition)	Effective stress (Drained)	1.5
	Rapid drawdown	Effective stress (Drained)	1.3
(NRCS, 2005)	End of Construction	Total stress for impervious layer effective for pervious layer	1.4
	Long term (Steady seepage condition)	Total and effective stress (Drained and Un-drained)	1.5
	Rapid drawdown	Total and effective stress (Drained and Un-drained)	1.2
(BDS, 1994)	End of Construction	Total stress (Un-drained)	1.3 to1.5
	Steady seepage	Effective stress (Drained)	1.3 to1.5
	condition		
	Rapid drawdown	Total stress (Un-drained)	1.2 to1.3

C) Calculating safety factors

The factors of safety criteria presented in this thesis work are based on the slope stability analysis being performed by limit equilibrium method using Morgenstern-price analysis procedures. Because, Morgenstern-Price methods are possible to more readily handle the iterative procedures using the limit equilibrium methods and this lead to mathematically more rigorous formulations which include all inters lice forces and satisfy all equations of statics. Hence, slope stability analysis are performed to many scenarios until a stable slope is achieved based on table 3.6 which describes the summary of critical conditions and minimum recommended safety factors at different normal loading conditions for each zoned earthen dam categories.

I. Morgenstern-Price method

This method considers not only the normal and tangential equilibrium but also the moment equilibrium for each slice in circular and non-circular slip surfaces.
The equations are written for a slice of infinitesimal thickness as:
The factor of safety with respect to force equilibrium is indicated in equation 3.2 and 3.3 respectively:

$$F_f = \frac{\sum(c'l+(p-ul)\tan\emptyset')\cos\alpha}{\sum p\sin\alpha} \quad \text{..........3.2}$$

And, the factor of safety with respect to moment equilibrium is:

$$F_m = \frac{c'l+(p-ul)\tan\emptyset'}{W\sin\alpha} \quad \text{..........3.3}$$

In most of the homogeneous embankment dams failure occurred along the most critical slide surface with corresponding lower value of factor of safety.

3.3.2.3 Static deformation analysis

Magnitudes and directions of embankment deformations are controlled by foundation and embankment material properties, abutment and embankment geometry and embankment placement rates, reservoir loading conditions, and stress distribution within the various zones or layers within the embankment and its foundation (USBR, 2014). In this study, the contours and graphs of magnitude and distribution of the vertical and horizontal displacements are computed in two conditions, the end construction and full reservoir condition. For the SIGMA/W based computed results, the horizontal displacement direction towards the upstream is negative, and the horizontal displacement towards the downstream direction is positive.

General Considerations and Analysis Procedures

For embankment stress evaluation, the total stress and the effective stress analysis is required. The total stress analysis with the aid of SIGMA/W software was used in the existed embankments for loading conditions during empty condition of the dam and full reservoir level. Prior to simulation, the following analysis procedures are considered.

I. Determination of Boundary Conditions

For the incremental finite element formulation of stress deformation multiple boundary condition types are implemented to support virtually all load deformation modelling scenarios. Thus, SIGMA/W analysis of the dam the respective boundary conditions are applied using the node lines for displacement, stress and water pressure. For the earthen dam stress and deformation analysis the model is bounded with zero stress-displacement along the edges.

II. Specifying initial conditions

For both cases empty and full reservoir condition the initial pore water pressure was utilized from the SEEP/W parent analysis to examine the effective stress and settlement distribution. This helps to compute the effective stress redistribution for the situations of a change in pore-pressure which is graphically represented as:

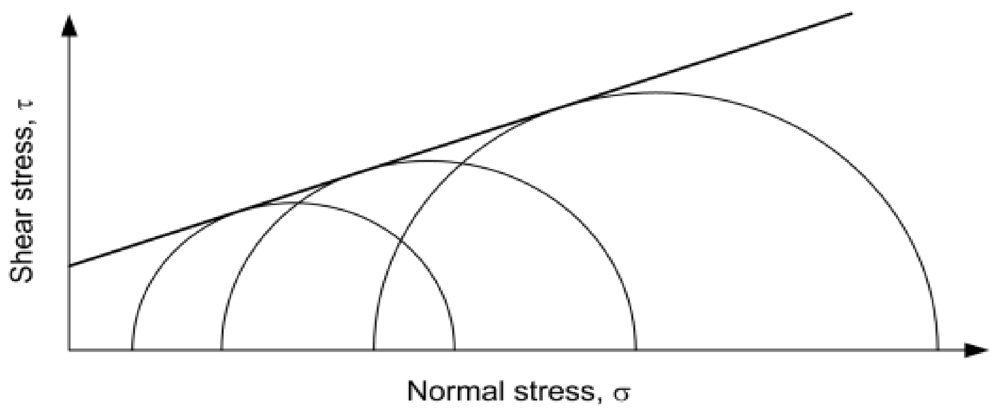

Figure 3.12 Over-stressing due to the increment of pore-pressure:

Constituting Material Models

For this analysis these embankment and foundation materials are modeled as linear elastic and elasto-plastic constitutive soil models for the non-cohesive and cohesive materials. Two-dimensional plane is applied to formulate the SIGMA/W problems and to compute the stress-strain and displacement effects of the dam body and foundations.

CHAPTER FOUR: RESULTS AND DISCUSSIONS

This chapter presents the results of the research work and discusses & analysis the probable failure of Grindeho earth fill dam with respect to the major problems encountered seepage analysis, stability analysis and Static Stress-Deformation Analysis.

4.1 Current Condition of Grindeho Earth Fill Dam

Field observation

Grindho Earthen dam is one of the few recently constructed dams in Tigray region. In interview the dam has lived and passed two wet and rainy seasons. The first summer or rainy season was 2007 E.C when the construction of the dam had not completed. In this summer no significant volume of water has been reached to the dam because of the drought or El Nino. And therefore there had no significant challenge seen that could have put the dam in risk.

When we look to the second rainy season 2008 summer history has been totally changed. This summer season was good enough to harvest runoff in the dam. As the result of this good summer the dam has got enough runoff to fill and had been spilling for certain days at final days of the summer or at the beginning of September the reservoir was full as shown in figure 4.1below. That means according to design document (appendix Figure A1) at elevation of 2150m a.m.s.l with storage capacity 4,283,238.9 m^3 but at this time the reservoir elevation head is at 2136.60m a.m.s.l it is around the intake about 13.4 m elevation difference is happened means only 517,862.73 m^3 is present and about 3,765,376.17 m^3 it is maximum live storage of the dam lost within six months there for the probable reasons for this lost water are listed below.

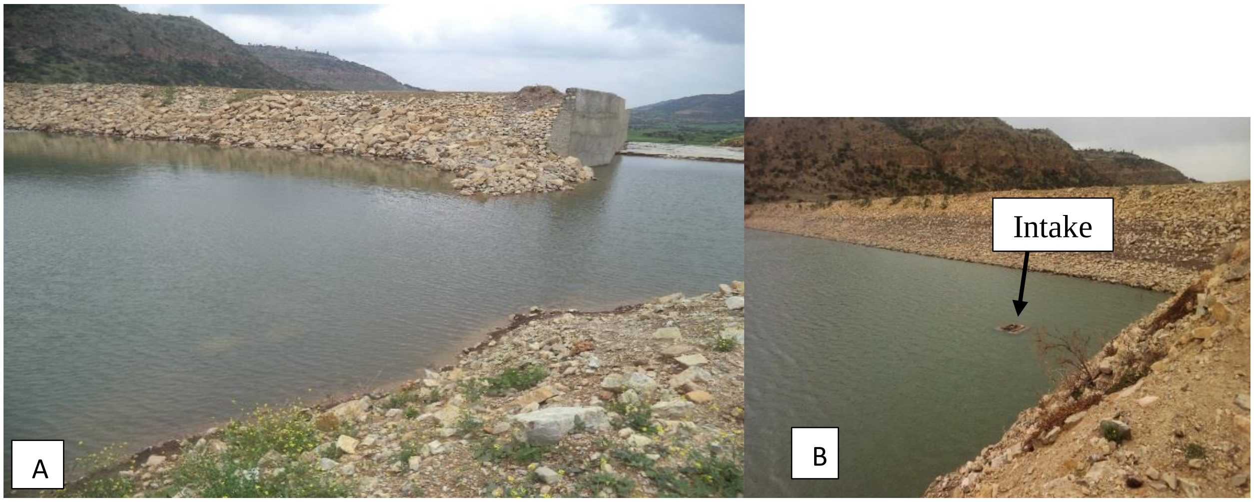

Figure 4.1: Grindeho earth fill dam storage capacity A) when the reservoir full B) progress within six months, 2017.

History has been completely changed when the dam got full. Problems that could lead someone can doubt about the efficiency of the dam start to arise. The biggest issue everybody who pay field visit can question is the amount of seepage flowing at different clusters in the body of the dam.

4.1.1 Seepage

It was observed that the site is faced seepage problem in both abutments and the downstream of spillway before reaching the terminal structure. The spillway is part of the dam in right side and seepage is seen in both sides of the spillway. In right side of spillway the seepage is seen around 120m downstream of control section in the abutment side of joints.

There may be different reasons for seepage seen in the right abutment, among them:-

1. The geotechnical condition of foundation of the spillway is weak and without any foundation surface treatment was done as shown figure 4.2 below.
2. Construction Quality is poor and the upstream cut off is not extended up to the guide bank.
3. Depth of cut off of the upstream is very short it may not be keep the scouring depth.

Figure 4.2: Grindeho earth fill dam geotechnical condition and Construction Quality.

4.1.1.1 Seepage at right abutment of the dam

The dam cut off trench doesn't reach to the impervious geological end laterally. That is why seepage has been observed to pass round outside the dam body. The springs emerge in natural parent rock material as shown figure 4.3 below. The material which springs emerged form is fractured limestone. Systematic and unsystematic joints have shattered the limestone severely. The talus or slope sediment has nothing to do with seepage as deposit left above the pool level of the dam. The spillway is greatly at risk as the foundation lied down on friable and highly weathered shale material. There is also seepage along the right guide wall foundation with contact to the spillway structure. It is understood that the origin of shale rock is fine material mostly clay. The behavior of the weathered shale is tremendously varies with moisture amount they got. Facing with moisture may soften the material and this can either let sliding or deformation happening on foundation of the spillway. When one get closer to present the ground level the geological complexity become severe. The sedimentary origin means the shale, marl and limestone resides side to side sharing sub vertical contact. Here an observable minor fault is experienced that could display similar layer the reddish shale to exist at different elevation.

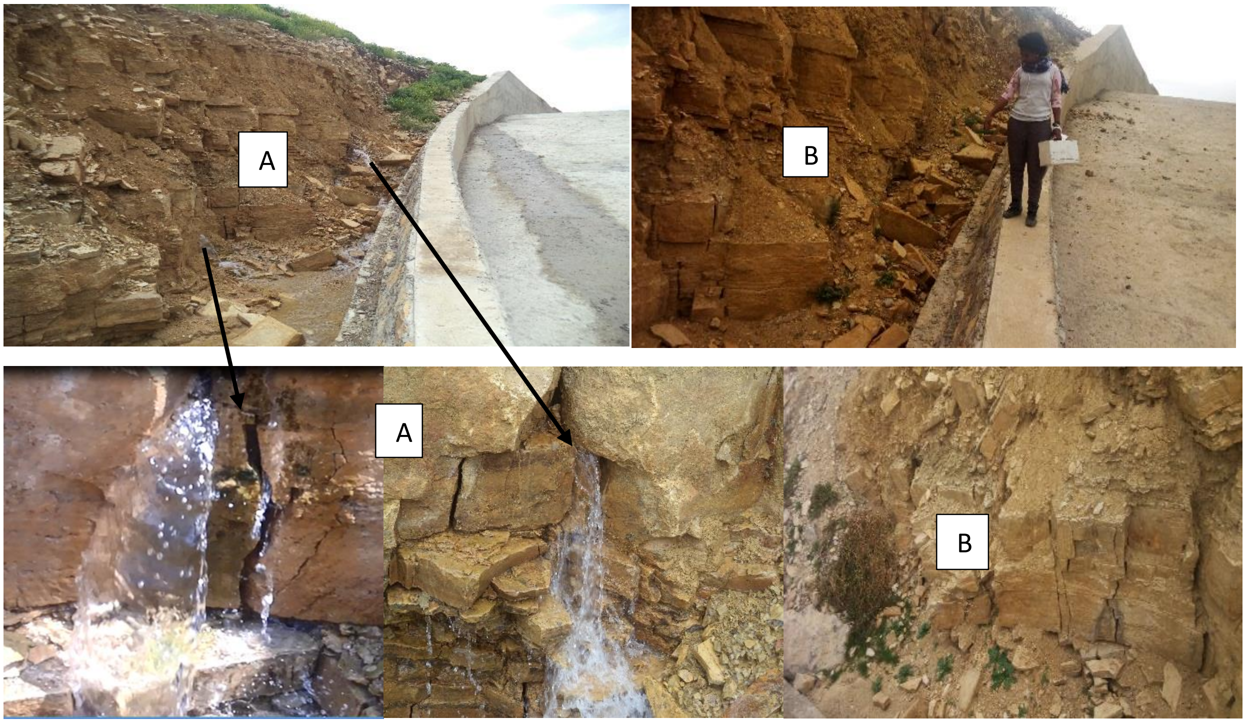

Figure 4.3: Seepage at right abutment A) when the reservoir full and B) current condition

4.1.1.2 Seepage at left abutment of the dam

Both abutments and central foundation are covered by pervious and unstable Colluvial /talus deposit and susceptible to excessive leakage. Seepage from this location look like springs and emerge far from the dam body and water flows like in open conduits. The geological formation on which seepage has been hosting is fractured limestone similar to springs around the right abutment and bedding contact limestone with friable shale.

4.1.2 Slope instability

Land slide has been happened at two specific positions. The smaller one which happened near to the spillway descends down ward and it has 13m burst edge length. The bigger land slide which consist about 200m edge length went down about 5-8m displacement as shown in Fig.4.4 below. The instability has triggered by the excessive seepage flowing through the left abutment. Up on to Fostere Mark et al. (2008), well compacted, cohesive materials, material likely to hold a crack.

Figure 4.4: Land slide situation near the spillway and the out let

4.1.3 Structures of the dam at great risk

The spillway, out let and canal of the dam are severely in danger. A landslide has happened already. A wide crack near the canal shows the land has started to slide is portrayed.

Due to the seepage part of the abutment starts sliding and because of this erosion occurs in right side of dam body. At the end of spillway before reaching the terminal structure (settling basin) and in both sides of retaining wall seepage is happened.

.

Figure 4 5: situation foundation of the spillway and the canal around the out let

Observatory inspections by professionals and another correspondents implied that the emerged seepage from these two abutments and the underneath spillway structure leakage rate becomes worsen from time to time. At that time the seepage is uncontrolled piping problem is occurred in the abutment interface and underneath scouring problems might be serious safety issue. And this consequent piping problem might lead to further structural instability and excessive losses of the stored water.

4.1.4 Down Stream of the dam axis

The geologic materials excavated from the foundation and spillway is deposited adjacent downstream bottom of the shell. These disposal soil may Cause blocks the free movement of water that drains out from the dam body and sloughing of downstream toe may occurred hence; gradually it may cause sliding downstream face of the dam.

4.2 Causes For Seepage And Instability Failure of Grindeho Dam

During field visit it is observed that the source potential seepage emerging from the parent rock might be the likely cause failure due to seepage. The rock which is responsible to high potential seepage is the highly fractured limestone. Almost the springs emerge from this formation. The large percentage of opening to volume ratio and the wide aperture joints aggravate the flow comparable to pumped water flow through a pipe. I have observed talus deposit in both side banks of the valley that extended to both abutments. There are some evidences that show the slope sediment deposits only cover the surface with insignificant depth on both sides of the abutments. The previous study (Grindeo earth fill dam) has foreseen and predicted a failure that could come from such a deposit. The talus's being poorly sorted and less degrees of compaction were the factors that had been thought to be problematic entity for future.

In case of the Grindeho dam, geological and geotechnical challenges happened. The foundation and abutment interfaces are dominantly covered by the Marly limestone (MLst), Colluvial/talus deposit and shale, weak or loose soil respectively which is mainly characterized by moderately fractured and weathered, affected by rare solution cavities. Hence, the susceptibility for excessive seepage and hydraulic fracturing due to pore water pressure with the expected hydraulic head of the impounded water could be greater in the foundation and abutment interfaces of the dam. The siltation of the reservoir with time will tend to diminish under seepage. However, the right and left abutments is partisan to excessive leakage. Generally the orientation of the discontinuities in the dam site and reservoir are favorable for leakage to occur along the abutments and beneath foundation.

According to the investigatory geological data of the study area, failure has foreseen and predicted as shown in Table 4:1. The foundation of the dam is composed of alluvial deposit and both Abutments have Limestone-shale-marl intercalation. The alluvial deposit is composed of clay, silt, sand and fine gravel. Within the alluvial succession there are a number of sand layers or lenses.

These cyclic layers of sand are responsible for excessive leakage through foundation and also resulted in settlement. In addition the Limestone-shale-marl intercalation is highly affected by discontinuities due to fracturing/joints, bedding contact, intrinsic behavior of the rock units etc.

Table 4.1: Grindeho Earth dam and reservoir: Initial Engineering Geological Hazard and risk assessment (TWROB, 2012).

No	Potential Hazard	Undesirable Event	Consequence	Response
1	Jointed Limestone on abutments and rim of reservoir	High hydraulic conductivity/leakage and weak rock mass strength/sliding/rock falling, etc.	Excessive leakage and reservoir slope stability problem, rock fall and slide, etc.	No mitigation measures has been done
2	Weak or loose soils at Foundation	High Settlement	Structural damage and overtop of dam	
3	Colluvial/talus deposit on abutments	Stability problem/sliding	Structural instability	
4	Dominant joints perpendicular to dam axis	High hydraulic conductivity along the direction of joints/leakage	Excessive leakage and dry up of reservoir	

Due to these Geological and geotechnical Hazard challenges happened piping problem is occurred in the abutment interface and this consequent piping problem might lead to further structural instability.

The bed rock foundation and reservoir rim of Grindeho earthen dam is rich in carbonate (travertine exposures), karstification and shale which is inherently weak, permeable zone so the unit is strongly susceptible to defects and further weathering processes in the presence of moisture through the seepage action as a result it trigger to excessive seepage or prone to collapse . The right and left abutments of the dam are dominantly composed of calcareous shale (Sh) which is highly weathered, poor rock. The shell material of the dam body is constructed from the local material collected from near the dam site which is the known geological setting of Agula’a shale. Research findings by Abay and Claudia, (2011), Yagiz, (2011), Bryson et.al, (2011), Laksiri, (2007) and Zhang, (2014) observed that embankments behavior made up of these geological materials are highly triggered to karstification, low durability and low to medium swelling potentials.

4.3 Seepage Analysis

The results of the seepage analysis of Grindeho dam conducted in two ways first seepage analysis was investigated based on secondary collected data's of geotechnical parameters of the embankment and foundation materials which are mainly used for the model simulation that are amassed and summarized in the tables 3.4 means from the design document, which is shown under section 4.3-4.5. Secondly the seepage analysis was conducted based on the laboratory investigation, which is provided under section 4.6. The seepage analysis was also made using SEEP/W to compare the actual observed seepage flux with the design value identify the source of the problem.

4.3.1 SEEP/W analysis based on the design document data

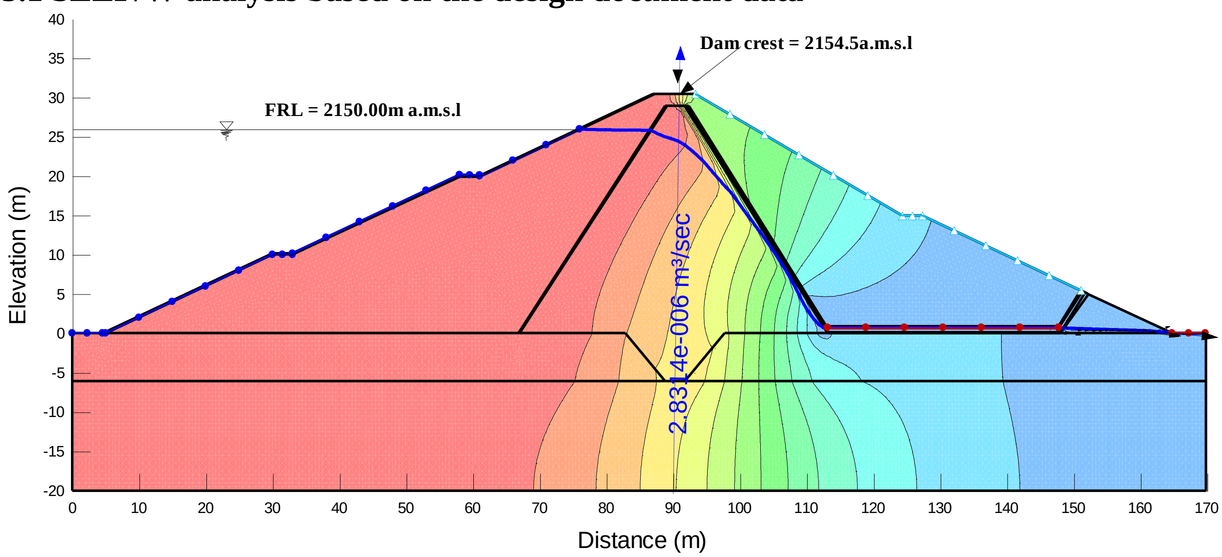

Figure 4.6: Seepage analysis through zoned dam based on design report

Based on the design data the seepage through the dam body is analyzed. It is found that seepage for the embankment was estimated as $2.83*10^{-6}$ m^3/s/m length of the dam. The total seepage over the average length of the dam crest 315 m is estimated to be $8.9*10^{-4}$ m^3/sec. Based on the SEEP/W result as shown in Figure 4.6, but the total amount of seepage flux through the dam body and foundation is calculate in the design document to be $7.09*10^{-5}$ m^3/sec . Therefore, the amount of seepage flux Grindeho earth fill dam is different from design document which indicates there was a gap in design document.

4.3.2 Seepage losses analysis using surface flow technique and stream flow meter

The amount of seepage measured using surface flow technique at the left abutment of the dam over the average length of the dam crest 315 m is $1.63*10^{-3} – 2.9*10^{-3}$ m^3/sec and still have different amount of seepage quantity when compared to the seepage analysis determined based on design document.

Total Seepage is also measured of Grindeho dam using Current Stream flow meter as it is presented in table 4.2.

Table 4.2: Measurements and calculations for the cross-section

	Col.1	Col.2	Col.3	Col.4	Col.5	Col.6	Col.7	Col.8	Cols 1-8
Station or width (w) c.m	0-53	53-106	106-159	159-212	212-265	265-318	318-371	371-427	427
Depth (d) c.m	13	15	15.5	23	18	16	14	10	d_{aveg}15.5
Area (A) m^2	0.069	0.079	0.082	0.122	0.095	0.085	0.074	.053	
Counter (c) No	403	249	414	619	202	241	257	233	
Water velocity (V) m/s	0.39	0.263	0.403	0.578	0.222	0.256	0.269	0.249	V_{mea}=0.33
Discharge (Q) m^3/s	0.027	0.021	0.033	0.07	0.021	0.022	0.02	0.013	0.221

Generally, the amount of seepage at the junction of abutments and embankment body

Q_T = <u>0.221m^3/s</u>

The expected quantity of seepage is estimated with SEEP/W software model that includes foundation seepage and both abutments based on design document and the laboratory test data's over the average length of the dam crest 315 m is $8.9*10^{-4}$ m^3/sec and $1.1*10^{-2}$ m^3/sec respectively. But the amount of seepage at the design document is $7.09*10^{-5}$ m^3/sec. Those quantities of seepage indicate that there is difference as compared each other in terms of quantity. A leakage of 0.03 m^3/sec through the embankment and dam foundation is generally acceptable provided that proper filter material, drainage system and relief wells are incorporated (Jansen, 1988). In comparison to this recommendation the quantity of seepage loss through the dam body and foundation calculated in this research lies on the allowable range. So, the soil sample is safe against leakage and actually represented for material (core and shell) used during construction time. However, the currently seepage flow over the average length of the dam crest 315 m is 0.221 m^3/sec Therefore, the currently quantity of seepage value at Grindeho dam is high as compared with others. It shows that the dam has been exposed for piping failure.

4.4 Slope Stability Analysis

The results of the slope stability analysis of Grindeho dam conducted in two ways first stability analysis was investigated based on secondary collected data's of geotechnical parameters of the embankment and foundation materials which are mainly used for the model simulation that are amassed and summarized in the tables 3.5 means from the design document, which is shown under section 4.3-4.4. Secondly the stability analysis was conducted based on the laboratory investigation, which is provided under section 4.6. The static stability of the upstream and downstream slopes of Grindeho earth fill dam is to be analyzed for the most critical or severe loading conditions that may occur during the life of the dam using numerical models (SLOPE/W) of Geo Studio software product.

4.4.1 SLOPE/W analysis based on the design document data

SLOPE/W is a software product that uses limit equilibrium theory to compute the factor of safety. For Grindeho dam based on the design document data is given in table (3.5) above.

The structural failure of the dam was serious/ crucial under steady state seepage in the downstream face of the dam. Based on the steady seepage analysis the minimum factor of safety on the downstream face of the dam is found to be 2.014 (Fig.4.7).

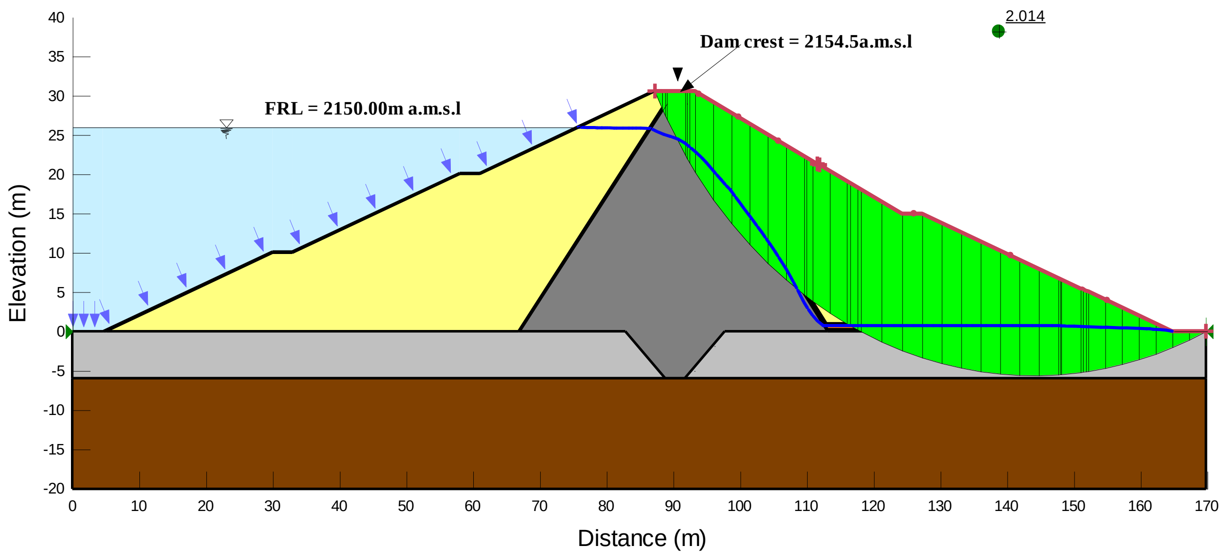

Figure 4.7: Steady state (Reservoir full) at downstream slope

The minimum factor of safety against sliding under normal loading condition is 1.5, which shows that the dam is safe as the minimum factor of safety calculated for the slip surface under given loading condition is higher than the threshold value.

Based on the steady seepage analysis the minimum factor of safety on the upstream face of the dam is found to be 2.252 (Fig.4.8).

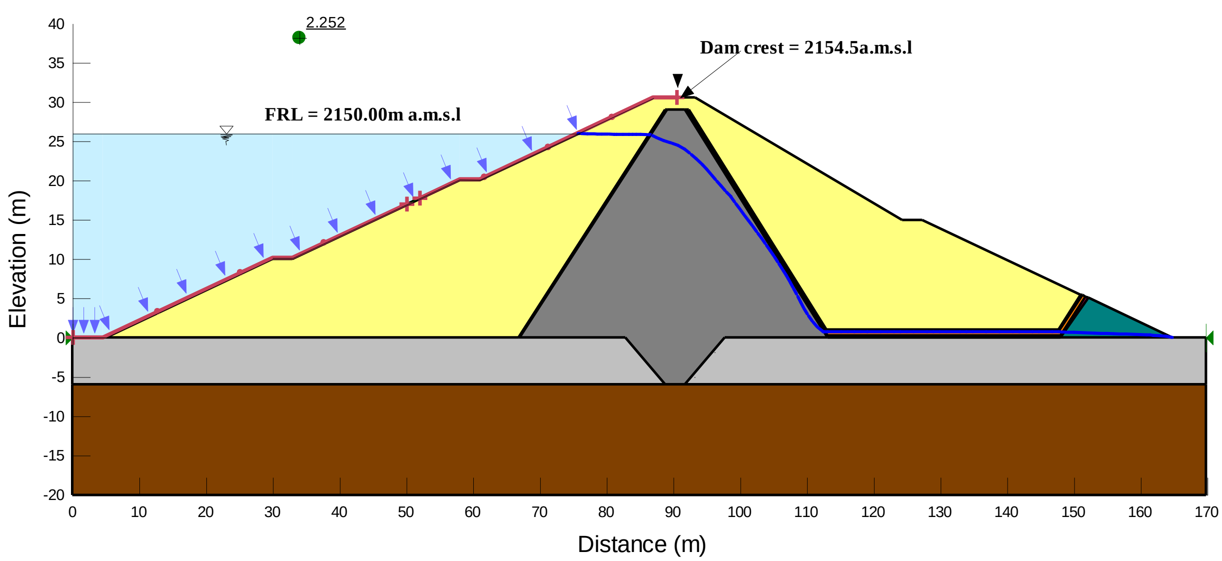

Figure 4.8: Steady state (Reservoir full) at upstream slope

The minimum factor of safety against sliding under normal loading condition is 1.5, which shows that the dam is safe as the minimum factor of safety calculated for the slip surface under given loading condition is higher than the threshold value.

Based on the Rapid drawdown analysis the minimum factor of safety on the upstream face of the dam is found to be 1.915 (Fig.4.9).

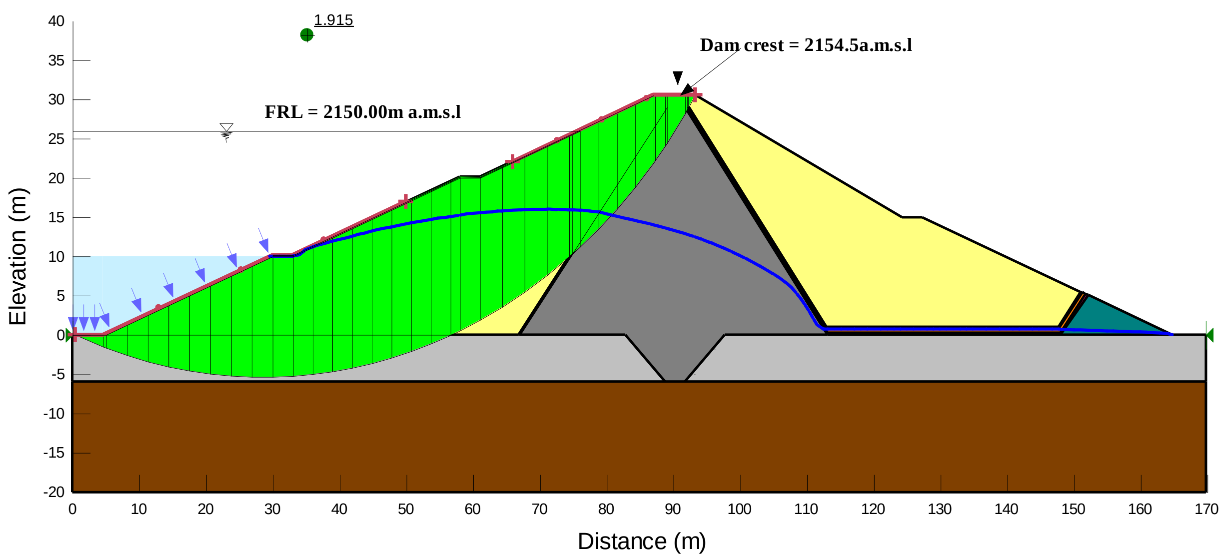

Figure 4.9: upstream slope sudden drawdown at day 10

The minimum factor of safety against sliding under rapid drawdown loading condition is 1.2, which shows that the dam is safe as the minimum factor of safety calculated for the slip surface under given loading condition is higher than the threshold value.

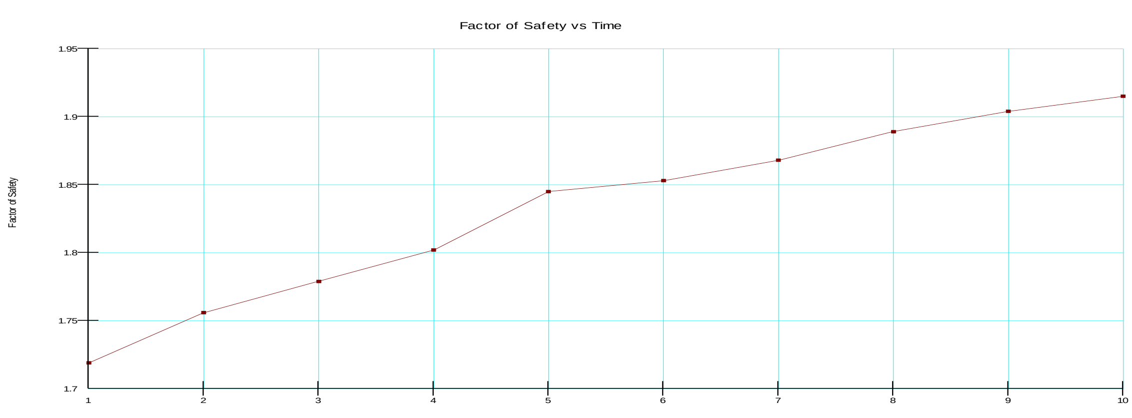

Figure 4.10: Factor of safety vs Time graph after instantaneous drawdown based on design document.

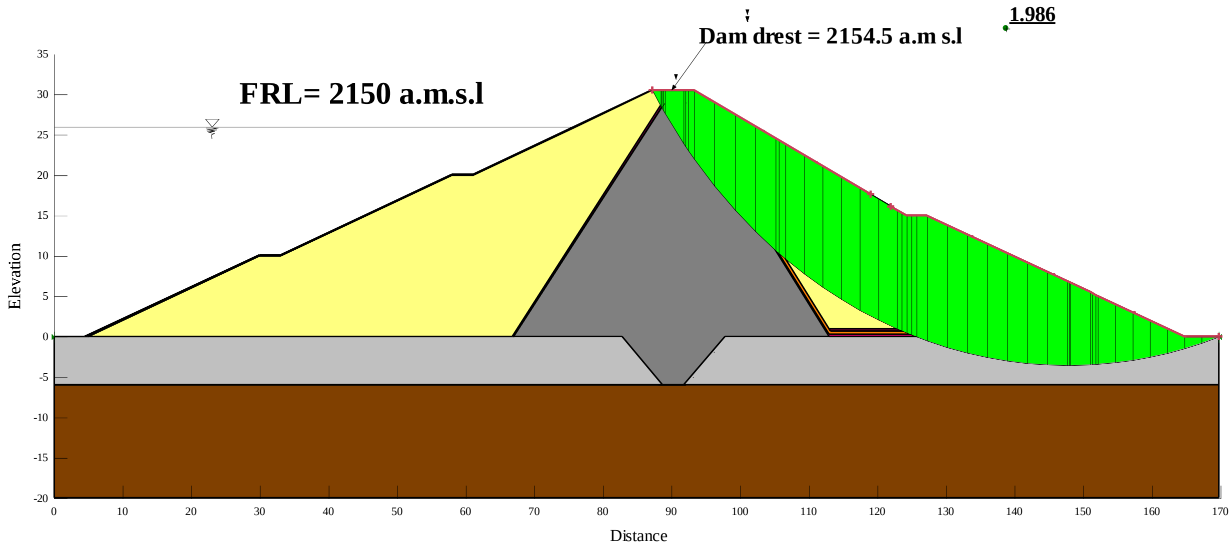

Figure 4.11: End of construction stability for the downstream

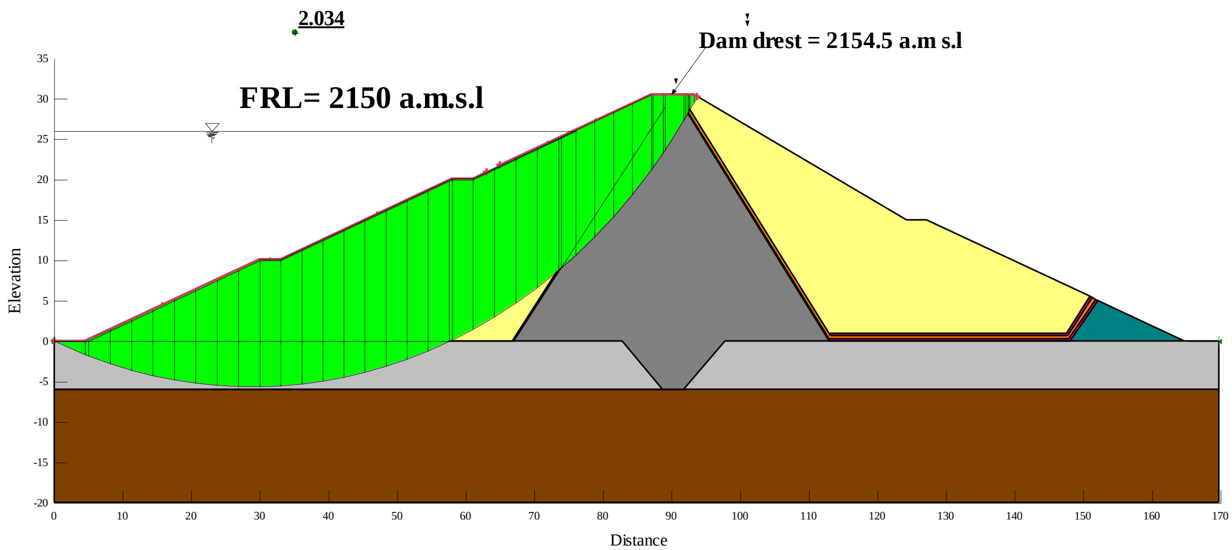

Figure 4.12: End of construction stability for the upstream

Based on the above result the factor of safety value Grindeho earth fill dam is greater than as compared to the baseline recommended minimum acceptable factors of safety USACE (2003) and BDS (1994) hence safety of the dam is checked; and the dam is stable.

4.5 Static Deformation Analysis

Magnitudes and directions of embankment deformations are controlled by foundation and embankment material properties, abutment and embankment geometry and embankment placement rates, reservoir loading conditions, and stress distribution within the various zones or layers within the embankment and its foundation (USBR, 2014). In this study, the contours of magnitude and distribution of the vertical and horizontal displacements are computed in two conditions, the end construction and full reservoir condition. From the SIGMA/W based computed results, the horizontal displacement direction towards the upstream is negative, and the horizontal displacement towards the downstream direction is positive.

4.5.1 End of construction Settlement Analysis

At the end of the dam construction or empty condition, the maximum upstream and downstream horizontal displacements occur at about one fifth of the dam's height in the upstream and downstream central part of the shell, and the maximum computed value is 0.03m for both sides. Whereas the maximum vertical displacements of 0.223 m created at the crest of the dam, which is 0.73% of the dam height which is conceded to the USBR "1 percentage rule" standard. The change in the maximum vertical settlement and upstream and downstream horizontal displacement at the end of dam construction with the quantity of simulation contours are shown in figures (4.13 - 4.16).

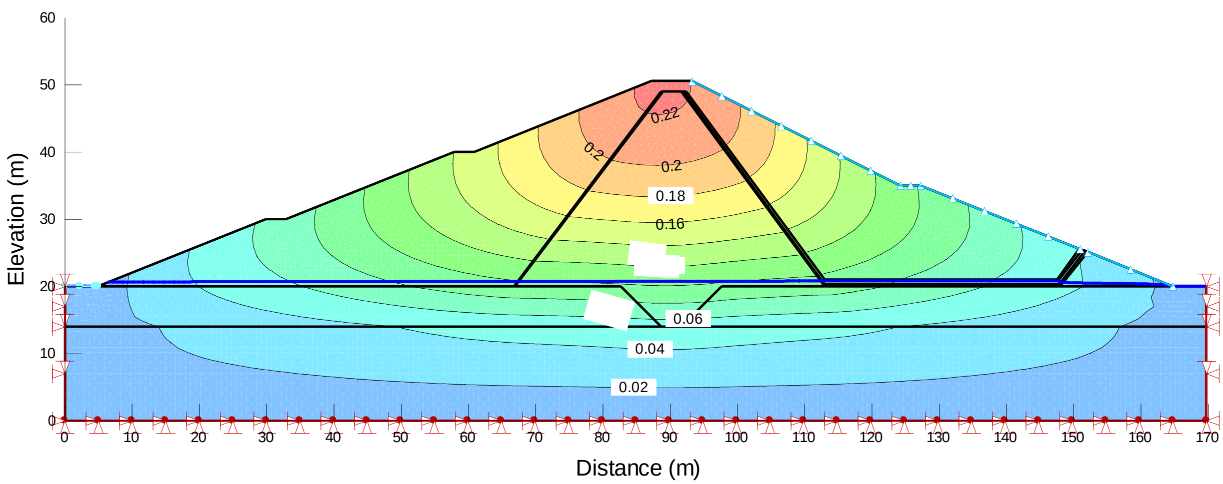

Figure 4.13: Vertical settlement in the dam body and foundation

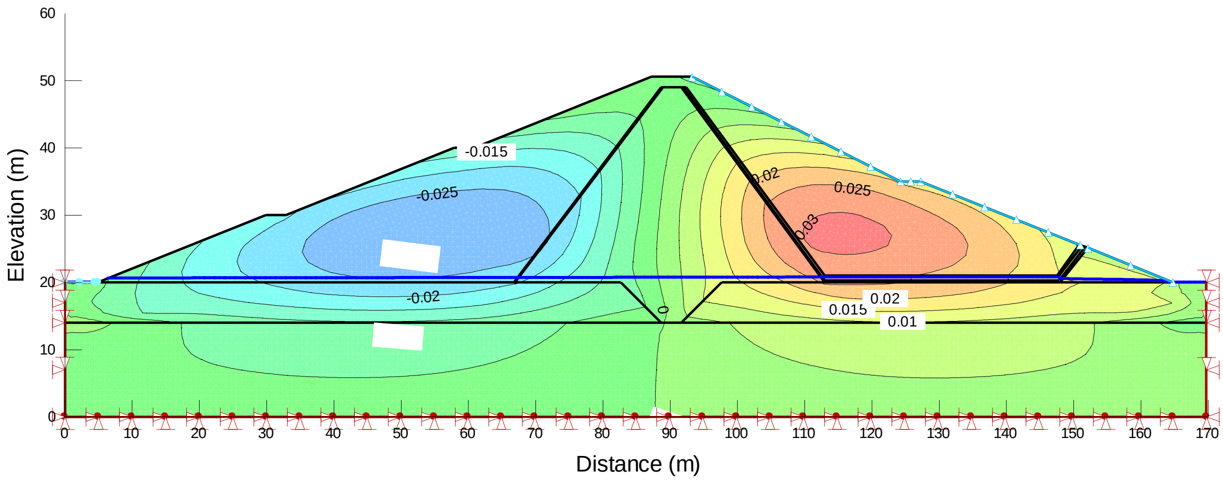

Figure 4.14: Horizontal settlement in the dam body and foundation

According to Petkovski *et al* (2006) SIGMA/W numerical model uses and recommended that the horizontal displacements along the core axis is obtained as deformed shape, with maximal displacements around 0.2% of the dam height, located above the foundation at approximately 60% of the dam height, and reduced downstream displacement in the crest, estimated at 70% of the maximal horizontal displacement. In this case, the maximum horizontal displacement value is 0.03m for both sides, which is 0.098% of the dam height. This value is less than 0.2%. Hence, at the end of the dam construction or empty condition the dam is within the safe limit as to deformation is considered and against the inside cracking.

4.5.2 Settlement Analysis under steady state condition

In the case of Grindeo earth fill dam, the simulated maximum upstream displacement distribution is 0.0252 m which is created in the upstream face of the dam and the maximum downstream displacement of +0.063 m is created inside the impervious core of the dam. The positive value indicates that the displacement direction is towards the downstream side of the dam. The rate at which these deformations occurred depends on the dissipation rate of excess pore pressures and the rate at which steady-state seepage conditions develop. According to (Chen, Zou *et al.*, 2014) when the water fully fills the reservoir, the upstream horizontal displacements decrease and the downstream displacements increase due to the effect of the water load and the pore water pressure.

According to the various agencies deformation safety standards, the horizontal displacement was 0.207% of the dam height. Based on the Hunter and Fell (2003) recommendation this settlement value is slightly greater than 0.2%. Based on the laboratory test result materials of Grindeho dam, effective cohesion value increases are not safe against crack. Thus, during rapid drawdown, the impervious core material is not further safe against the inside cracking and the probability of hydraulic fracturing becomes critical when the reservoir reaches its top level quickly. The vertical settlement was occurred 0.94% of the dam height seldom exceeded to the 0.5% of the embankment height. However, during construction of Grindeho dam the USBR standard "1 percent rule" had been considered. From this, the rate of vertical settlement is found in the vicinity of the designed freeboard. So finally, this might be concluded that the dam couldn't be affected by overtopping problem through the impact of the current vertical displacement.

The vertical and horizontal displacement at the full reservoir impoundment and under the action of steady state condition with the quantity of simulation contours are shown in figures below.

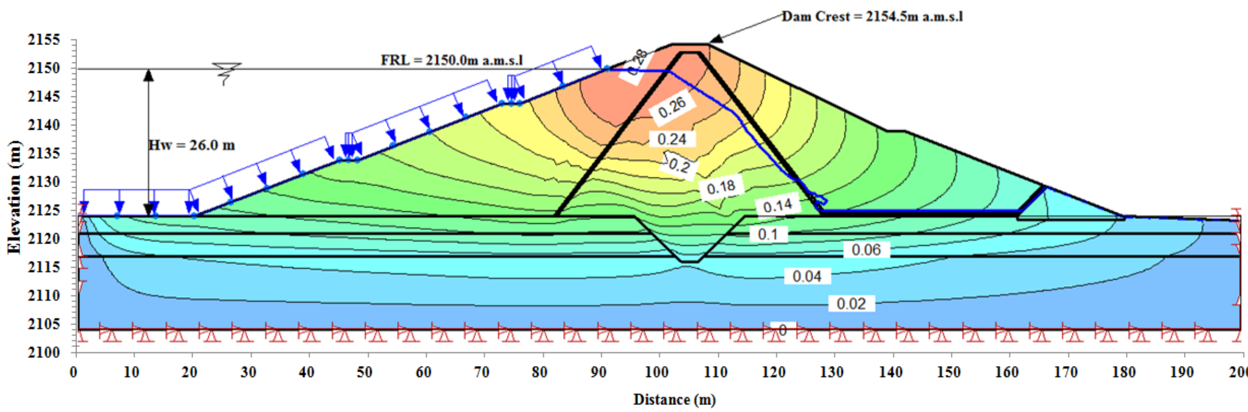

Figure 4.15: Vertical settlement contours in the dam body and foundation (Steady state condition)

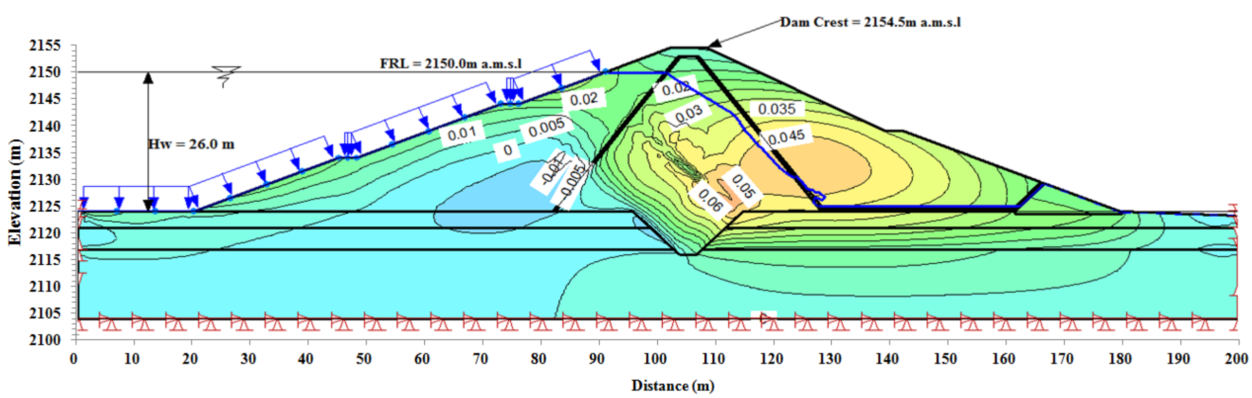

Figure 4.16: Horizontal displacement in the dam body and foundation.

Localized settlement of limited loose zones in the embankment could result from sinkholes or depressions, slumps, cracks, and formations of cavities. Up on FEMA (2015) recommendation dam sites with karstic foundations of limestone or other soluble bedrock are especially detected by surface deformation. In view of that, for this study the crest surficial measurements and inspection was carried out. Because, the geological formation including the bedrock foundation of Grindeho dam foundation is composed of the carbonate rich succession (Limestone-shale-marl unit) which is inherently weak and strongly susceptible to defects and further weathering processes in the presence of moisture through the seepage action.

Based on the current field observations and of the case study; it can be concluded that the dam might be safer due variable engineering behavior of the geological conditions of the dam foundation the future lifespan of the dam, further sinkholes and depressions may be resulted from seepage through internally unstable soils that allow material transport to occur. Therefore, the static- deformation analysis result of this research is fully agree with kisate, (2016) static-deformation analysis result.

4.6 Laboratory Investigation

4.6.1 Grain size analysis

ASTM D422-90, 1988 standard was used in tests performed to determine the grain size analysis of the samples under investigation and the results are presented in tables and figures. The grain size analysis test results for shell materials are tabulated in (Appendices Table A1 & Figure 4.17).

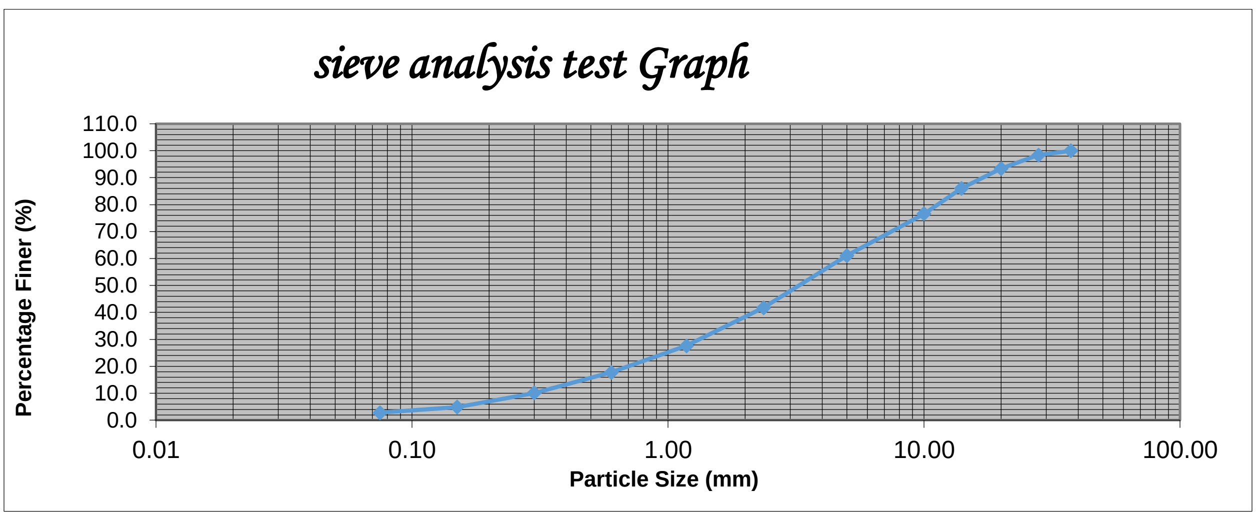

Figure 4.17: Grain size analysis graph for shell material

The grain size analysis test result for clay material are tabulated on (Appendices Table A4 & Figure 4.18).

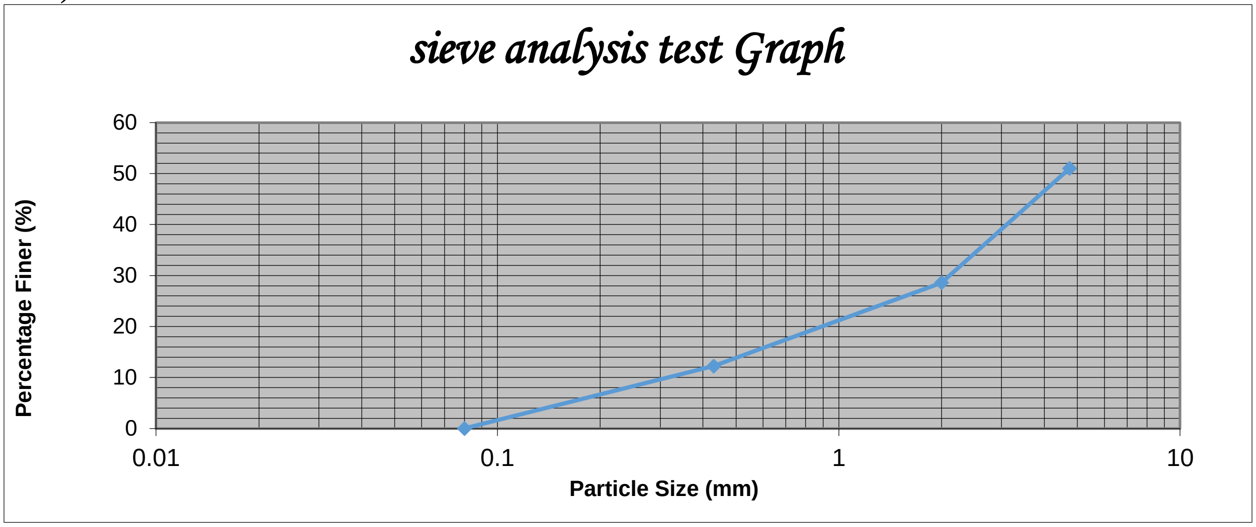

Figure 4.18: Grain size analysis graph for clay material

If the soil is fine-grained (≥50% passes number 200) follow the guidelines for fine-grained soils If the soil is coarse-grained (<50% passes number 200 sieve) follow the guidelines for coarse grained soils.

A flat S-curve represents a soil which contains the particles of different size in good proportion such a soil is called a well- graded (or uniformly graded) soil and/or classify the soil as a well-graded gravel, GW, or well-graded sand, SW, if Cu is greater than or equal to 4.0 for gravel or greater than 6.0 for sand and Soils with a value of Cu less than 2 are uniform soils (Arora 1997, Appendices figure A2). Therefore, the result of particle size distribution a curve of the shell material indicates they are course- grained soil or soil which contains the particles of different sizes in good proportion and the clay core materials are fine-grained. Coarse-grained soils are pervious, easy to compact, have excellent Shear strength, excellent as construction material and little affected by moisture or frost action. Fine grained soils have lower permeability, lower shear strength, and higher compressibility. Excess pore pressures often develop during rapid construction of fine-grained fill zones, resulting in reduced shear strength and potentially unstable conditions during or shortly following construction.

4.6.2 Atterberg limits

Atterberg limits are regarded as useful indices for determining the characteristics of most clay. This is true because parameters depend on the amount of water a soil tries to imbibe. A typical soil mass have three constituents: soil grains, air, and water. In soils consisting largely of fine grains, the amount of water present in the void has a pronounced effect on the soil properties.

In describing these soil states, it is customary to consider only a fraction of soil smaller than the No.40 (0.425mm) sieve size. For this soil fraction, the water content in percentage of dry weight at which the soil passes from the liquid state to the plastic state is called liquid limit. Similarly, the water content of the soil at the boundary between the plastic state and solid state is called the plastic limit. The difference between the liquid limit and the plastic limit corresponds to the range of water content within which the soil is plastic, and is called plasticity index (PI). Soils with high plasticity have high value of PI. These limits of consistency, known as "Atterberg limits" are used in the plasticity chart as the basis in laboratory differentiation of materials of appreciable plasticity (clays) and slightly plastic or non-plastic materials (silts).

ASTM D4318-95 standards, the Atterberg test results obtained are tabulated in (Appendices Table A3) and (Appendices Table A4**)** for shell and core materials respectively.

Table 4.3: Summary of Laboratory results of samples and the USC.

Material type	S. No.	Grain size				Atterberg limits			USC
		Gravel [%]	Sand [%]	Silt [%]	Clay [%]	PL [%]	LL [%]	PI [%]	
Shell	1	15	49.1	27.4	8.5	18.4	37.1	18.7	CL
Clay Core	2	0.0	3.8	46.8	49.4	26.5	53.8	27.3	CH

From the results shown the shell materials have inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, and lean clays (CL). And the clay core materials also have highly plastic inorganic clays as compared with design document and classified as a fat clay, CH, if the liquid limit is 50 or greater. Fat clay the mixed clay must be out of CH group (preferably its relative desirability as impervious core of embankment material is greater or equal to that of CL soil.) and have high compressibility. Accordingly, to USCS Classified the core material has been obtained to lie above the A- Line, in the area for the soils that are classified as inorganic clay (Appendices figure A3).

4.6.3 Unconfined compression test

According to the ASTM standard, the unconfined compressive strength (qu) is defined as the compressive stress at which an unconfined cylindrical specimen of soil will fail in a simple compression test. In addition, in this test method, the unconfined compressive strength is taken as the maximum load attained per unit area, or the load per unit area at 15% axial strain, whichever occurs first during the performance of a test (Krishna, 2002). ASTM D 2166 –Standard Test Method for Unconfined Compressive Strength of Cohesive Soil **(**Appendices Table 4&Figure 4).

The result indicates that changes of sensitive parameters influenced for factor of safety. As the effective cohesion value increases the dam safer. There is a difference in shear strength value. Less value of effective cohesion(C) and friction angle (ϕ) on design report (Table 3.5 :) as compare with laboratory results (Table 4.9 :) and indicators of for instability of the dam. According to Fostere Mark et al. (2008), well compacted, cohesive materials, material likely to hold a crack.
Cohesive, clayey soils have particles which are thin and flaky, like a sheet of paper. Soils composed of flaky particles are highly compressible. These soils deform easily under static loads, like dry leaves, or loose papers in a basket subject to a pressure. However, such soils are relatively more stable when subjected to vibration (Arora 1997).

4.6.4 SEEP/W Analysis Based on the Laboratory Test Data

The main purpose to conduct laboratory test on Atterberg limit and grain size analysis is to analyze the slope stability and seepage on Geo-studio software by Grain size data method. The important Grain size data from the laboratory test given on the table 4.8 and SEEP/W analysis based on laboratory tests results data is given as below.

Based on the result of laboratory test the embankment materials have high value of Liquid limit, Permeability and saturated water content as compared with original design. These values are affected the SEEP/W analysis result as shown table 4.4 below.

Table 4.4: SEEP/W input data based on the laboratory test results data

Material	Saturated water content (m^3/m^3)	D10(mm)	D60(mm)	Liquid limit (%)	Residual water content (m^3/m^3)	Permeability Ks (m/s)
Shell	0.40	0.43	8	37.1	0.058	1.03×10^{-5}
Core	0.66	1.0	5	53.8	0.102	7.87×10^{-7}
Foundation	0.34	0.1	2	42	0.0230	$6.98*10^{-4}$

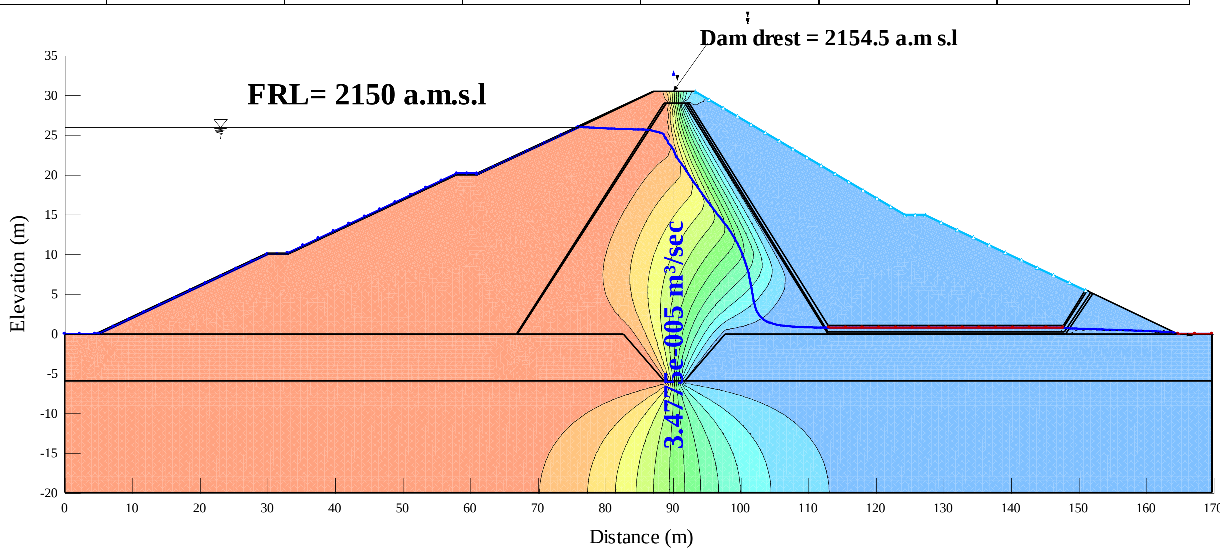

Figure 4.19: Seepage analysis through zoned dam based on laboratory test data

The calculated seepage for the embankment was estimated as 0.0348 l/s per meter length of the dam. The quantity of seepage flow rate through the dam over the average length of the dam crest 315 m is: $Q = 1.1*10^{-2}$ m^3/sec.

Quantity of Seepage flow (m^3/sec) through the dam body and foundation of the dam over the average length of the dam crest 315 m is listed below in table 4.5.

Table 4.5: Summarized of estimated seepage analysis using SEEP/W and actual seepage flow measured.

SEEP/W RESULT			Current or actual seepage flow (m^3/sec)		Recommended quantity of seepage (Jansen, 1988) (m^3/sec)
From design document (m^3/sec)	Based on design document (m^3/sec)	Based on laboratory tests data (m^3/sec)	Based on reservoir storage capacity after six month (m^3/sec)	Using current flow meter (m^3/sec)	
$7.09*10^{-5}$	$8.9 *10^{-4}$	$1.1*10^{-2}$	0.24	0.22	0.03

From the above table 4.5 both the quantity of seepage flow estimated by SEEP/W result are lies on the allowable range, which is 0.03 m^3/sec (Jansen, 1988) so the embankment materials are safe against water retaining . However, the current or actual seepage flow measured using current flow meter and the amount of lost water within six months since the dam doesn't give service yet after first fill there exist a quantity of water flow which is 0.22 and 0.24 m^3/sec respectively. Hence, the actual seepage flow and the amount of lost water within six months as shown fig 4.1 is almost similar and these values are above the recommended value which is 0.03 m^3/sec (Jansen, 1988) consequently the dam is exposed for piping failure due to geological and geotechnical challenges happened on both abutments.

4.6.5 SLOPE/W Analysis Based on Laboratory Test

Slope Stability analysis for Grindeho dam from laboratory tests**:** The data from the unconfined compression test for shell and clay material is given on table 4.6. In the case Grindeho earth fill dam, the factor of safety on loading conditions such as during Steady state seepage (downstream slope, upstream slope), during/end of construction (upstream, downstream slopes), sudden drawdown (upstream slope) are conducted to investigate the cause of failure. And Slope Stability analysis for dam Grindeho based on the laboratory test data is given as follows.

Table 4.6: SLOPE/W input data based on the laboratory test results data

Property strength type	Shell material	Core material	Foundation material
γ(kN/m^3)	16.3	16.2	17
C (kN/m^2)	78.75	86.19	29.2
ϕ (degrees)	31.36	35.14	30

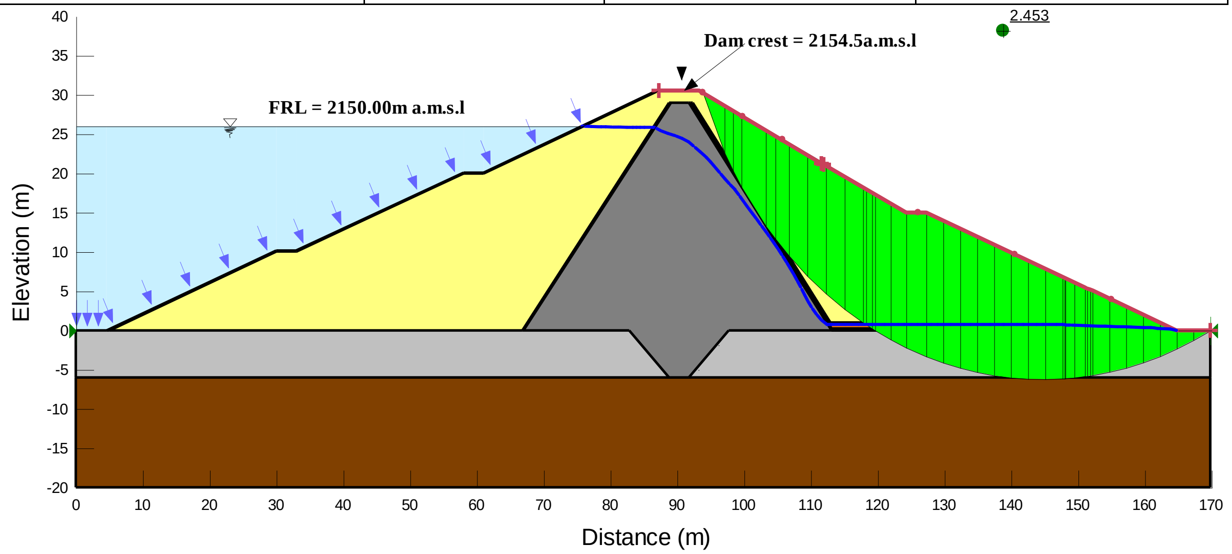

Figure 4.20: Steady state (Reservoir full) at downstream slope

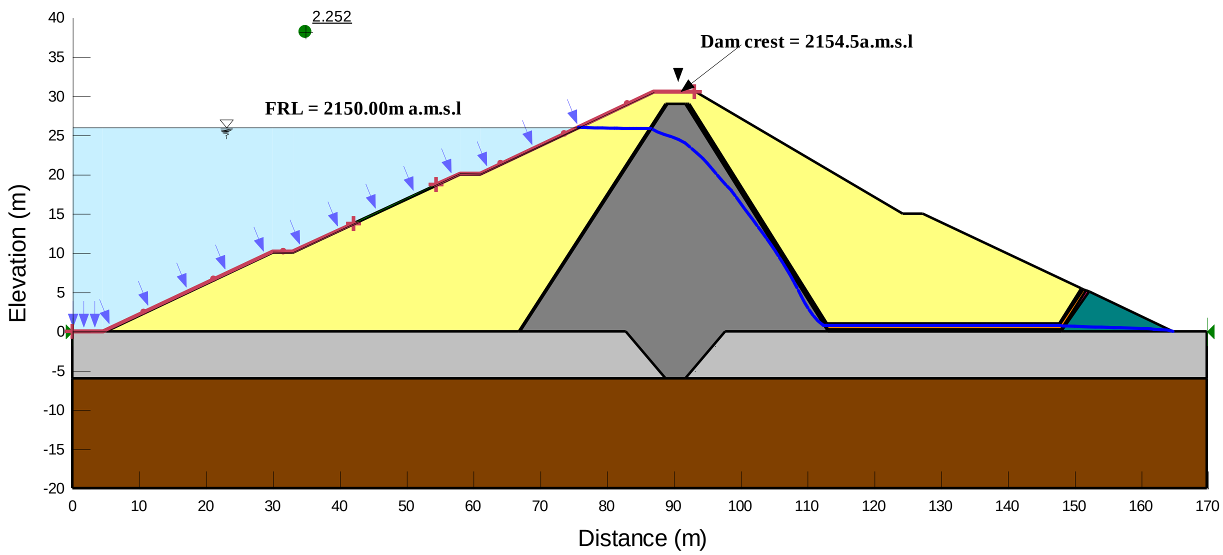

Figure 4.21: Steady state (Reservoir full) at upstream slope

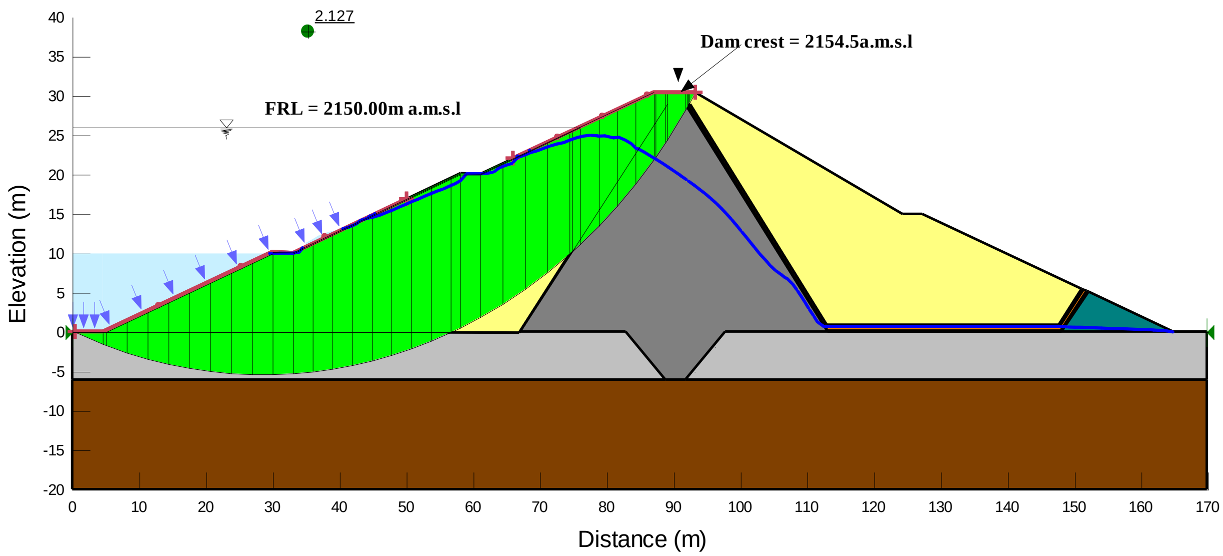

Figure 4.22: upstream slope sudden drawdown at 0.25day

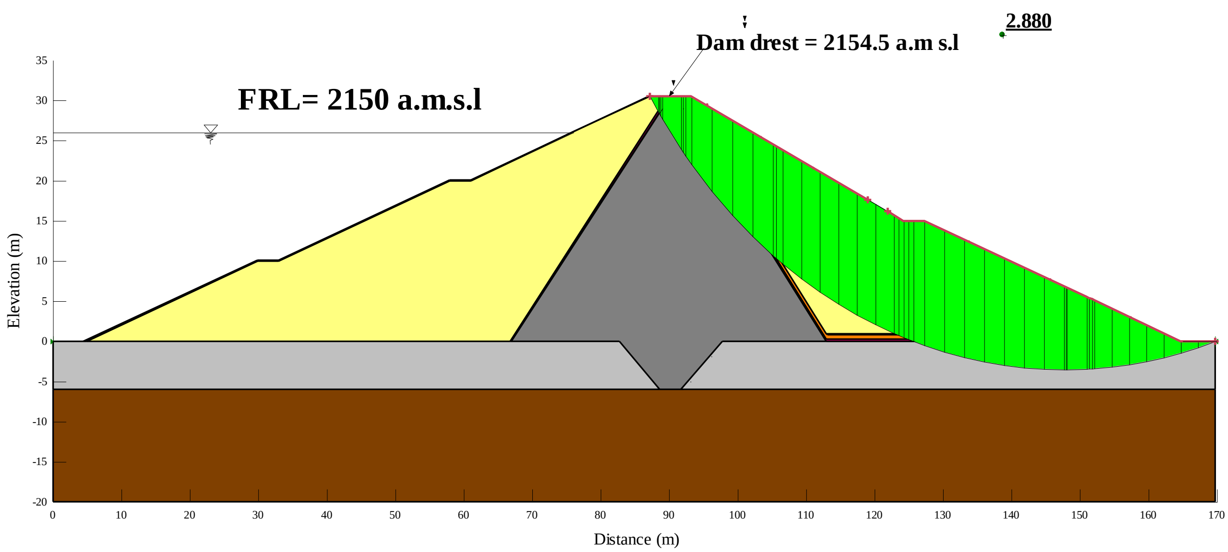

Figure 4.23: End of construction stability for the downstream

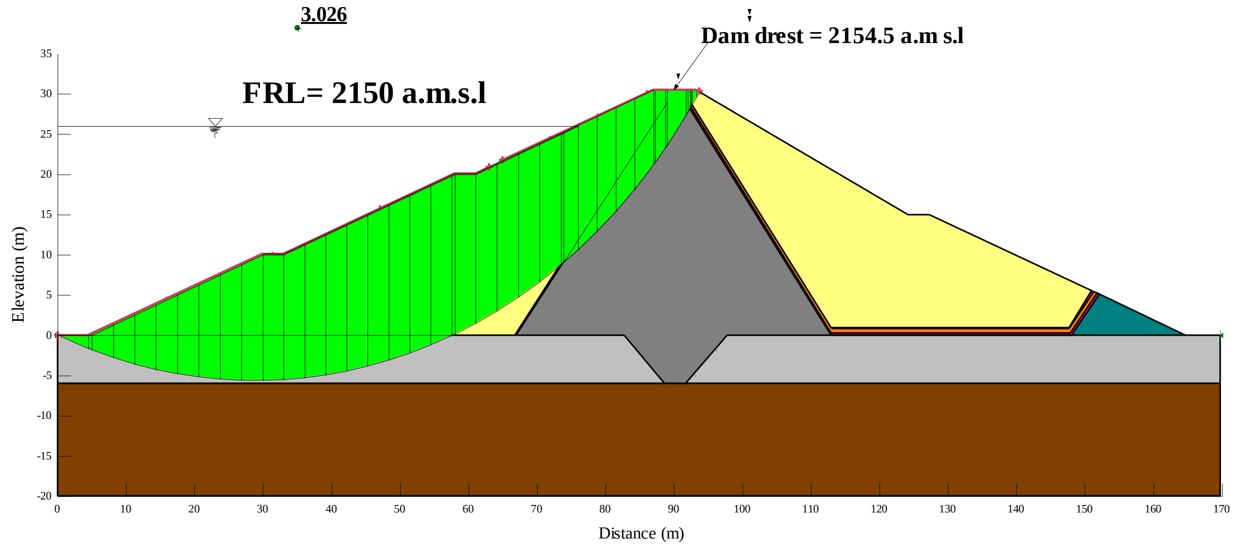

Figure 4.24: End of construction stability for the upstream

Factor of safety using the data based on design document and laboratory tests results is given on the table 4.7.

Table 4.7: Slope Stability analyses from design document for the following load cases :(BE&Comu, 2012).

Loading Condition	factor of safety Result using GLE/M-P				Recommended Factor of Safety		Remark
		From design document	Based on design document	Based on laboratory tests data	USACE (2003)	BDS (1994)	
Steady state seepage	U/S D/S	3.122 2.107	2.252 2.014	2.252 2.453	1.5 1.5	1.5-1.3 1.5-1.3	Stable
End of construction	U/S D/S	1.660 1.484	2.034 1.986	3.026 2.880	1.3 1.3	1.5-1.3 1.5-1.3	Stable
Sudden drawdown	U/S	1.668	1.915	2.127	1.2	1.3-1.2	Stable

In the case of Grindeho earth fill dam, the factor of safety using data from the design document and based on laboratory test are above the recommended minimum acceptable factor of safety USACE (2003) BDS (1994) and the values are acceptable. The factor of safety value Grindeho earth fill dam is greater than as compared to the baseline recommended minimum acceptable factors of safety hence safety of the dam is checked; and the dam is stable. Therefore, there may be used excess amount of construction materials during construction and that is not economical.

The results of this analysis (Table 4.7) indicate that acceptable factor of safety. Therefore, the soil sample is actually represented for material (core and shell) used during construction time. The shear strength (effective cohesion and effective angle of internal friction) and unit weight (γ) are sensitive parameters during slope stability analysis. The laboratory result indicates that changes of sensitive parameters influenced for factor of safety. As the effective cohesion value increases the dam safer. There is a difference in shear strength value. Less value of effective cohesion (C) on design report (Table 3.5) as compare with laboratory results (Table 4.6) and indicators of instability of the dam. Cohesive soils often develop excess pore pressures during rapid draw dawn, which is resulting in reduced shear strength when saturated and potentially unstable conditions during or shortly following construction.

CHAPTER FIVE: CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Earth fill dam is constructed using local construction materials for water supply and irrigation purpose. In order to assess the causes of failure of earth fill dam, the investigation has been conducted at Grindeho dam. To attain the overall objective several activities were undertaken including seepage, stability and Static Stress-deformation analysis using Geo-studio software, field seepage measurement, laboratory tests on Grain Size Analysis, Atterberg Limits, and unconfined compression test. The analysis has covered the whole dam body, including 20m of foundation depth.

Seepage analysis was carried out using SEEP/W based on data on the design document and the laboratory test results of the construction material data collected from the site. The analysis was conducted considering full reservoir condition (at an elevation of 2150.0m). It was found that the seepage flux of $8.9 * 10^{-4}$ m^3/sec and $1.1*10^{-2}$ m^3/sec respectively for the two cases while amount of seepage at the design document is $7.09*10^{-5} m^3$/sec. However, the current or actual seepage flow of the dam computed is $0.221 m^3$/s which is almost the whole reservoir storage. This quantity of seepage value at Grindeho dam is high or it depicts that the dam fail against seepage.

Based on the results the following main conclusions were drawn.

- The difference in seepage quantity (almost all the seepage springs) is expected to come from the parent rocks in the area shale and limestone contact or through fractured limestone. This means the discontinuities of the parent rocks that let water pass through are the opened fractured in the limestone and the bedding contact of limestone with shale.
- Construction quality is poor and the upstream cut off is not extended up to the guide bank, which facilitate the seepage loss
- There was a piping problem at both abutments which could also be the result of the complicated geological conditions; insufficiency of the depth of upstream cut off. Regardless of such problem no mitigation measures has been done during that aggravate the condition.
- The technical design documents (i.e. the geological survey and the engineering survey) are not in agreement. The geological report of Grindeho Dam predicted/states that the bed rock

is found at a depth of 8 to 9 meters from the top/river bed at the right side of the river and it goes down to 20 meters depth on the left side of the river. It suggests a deep cut-off trench to be applied. However, the engineering survey report of dam geometry indicates only 6m cut off. Hence, there was a gap between the designers and geologists which may be the source of the problem.

Static slope stability analysis was performed using the limit equilibrium SLOPE/W software using the data based on document and data from the laboratory test (the sample material from quarry area).Based on the results the following main conclusions can be drawn.

- Both the analysis results based on design document and laboratory test data's give factor of safety which is above the baseline recommended minimum factor of safety justify the safety of the dam. However, due to these geological and geotechnical hazard challenges happened piping problem is occurred in the abutment interface and this consequent piping problem might lead to further structural instability. A good indicator for this is the land slide that has been happened at two specific positions (abutments). Possibly these instability has triggered by the excessive seepage flowing through both abutment.

According to the various agencies deformation safety standards, the horizontal displacement was 0.207% of the dam height. Based on the Hunter and Fell (2003) recommendation this settlement value is slightly greater than 0.2%. And the laboratory result of Grindeho dam have cohesive materials are not safe against crack or not satisfy the self-healing requirements. Thus, during rapid drawdown, the impervious core material is not further safe against the inside cracking and the probability of hydraulic fracturing becomes critical when the reservoir reaches its top level quickly. The vertical settlement was occurred 0.94% of the dam height seldom exceeded to the 0.5% of the embankment height. However, during construction of Grindeho dam the USBR standard "1 percent rule" had been considered. From this, the rate of vertical settlement is found in the vicinity of the designed freeboard. So finally, this might be concluded that the dam couldn't be affected by overtopping problem through the impact of the current vertical displacement. According to Fostere Mark et al. (2008), well compacted, cohesive materials, material likely to hold a crack.

5.2 Recommendations and Possible Remedial Measures

Based on the findings of this research study, the following recommendations and remedial measures are provided:

5.2.1 Recommendations

Based on the findings of this research study, the following points are recommended.

- The output of this work is true only for sites out of earth quack zones or seismic action was not considered. Even though earthen dam are not that sensitive to earth quakes loading compared to other types of dams (USBR, 1986), there could be further a need of checking the specific dam sites which are lied in area of that is subjected to earth quake shocks.
- Based on the results observed in this study, it is recommended that Grindeho earth dam is at great risk so continual monitoring and regular surveillance, close inspection is required after that put an appropriate possible remedial solutions to the dam safety, further geotechnical investigations are important that are not included in this thesis.
- Quick mitigation measures for the problems on the spillway should be taken seriously. Construction upstream cut off to some extent in depth weighing the property of material foundation of the spillway can stop the seepage.
- The geologic materials excavated from the foundation and spillway that deposited adjacent downstream bottom of the shell should be carted away.
- Due to the seepage from the left abutment the area is sliding and affecting the area for further displacement. Therefore the area should be reclaimed by providing proper drainage to safely pass the flow coming from the hill.
- In site investigation all professional design report must have be share their report as a team.

5.2.2 Possible Remedial Measures

Upstream impervious blanket

The proposed reservoir large area covered by the talus/colluvial. It is loose, weak, and generally unstable, composed different size rock fragments and poorly sorted. This deposits extend along the foot of the limestone cliffs. It is composed of blocks, boulders, cobbles, gravel, sand and fines of different rock types. These deposits are poorly sorted or erratic and extremely variable in nature.

The materials mainly covered the moderate to steep slopes of the dam site and reservoir area following between the limestone cliff and the alluvial deposit.

The bed rock foundation and reservoir rim of Grindeho earthen dam is rich in carbonate (travertine exposures) and shale which is inherently weak, permeable zone and they are highly affected by discontinuities of different type, like bedding joint, contact, tectonic joint, etc. So the unit is strongly susceptible to defects and further weathering processes in the presence of moisture through the seepage action. The right and left abutments of the dam are dominantly composed of calcareous shale (Sh) which is highly weathered, poor rock. As a result upstream impervious blanketing was recommended in the design document but no remedial measures taken during construction.

An impervious blanket immediately upstream of a dam can be used to seal the reservoir bottom and sides and thereby reduce seepage quantities and pressures beneath a dam. If the dam contains a permeable shell, the impervious blanket must be extended up the embankment slope to effectively control the seepage problem. If blankets will be exposed by pool fluctuation, they must be protected against erosion from wave action and runoff, from desiccation or drying and cracking, from mechanical damage, and from piping into coarse granular or fractured rock subgrades (Training Aids for Dam Safety Evaluation of Seepage Conditions, April, 2009).

- The right abutment should be protected by providing wing walls and trench field with impervious clay for approximately 200m from normal pull level (2150 masl) to 2141 masl the level that seepage disappears.
- The upstream cut off trench in both wing ends must be extended up to the guide bank.
- The geotechnical condition of foundation of the spillway is weak therefore, Construction upstream cut off to some extent in depth weighing the property of material foundation of the spillway can stop the seepage.
- The seepage noticed at the end of chute part and side walls of spillway can be resolved by reconstructing jointly the chute part with the terminal structure and side walls with apron selectively in the area where the problem is identified.

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APPENDICES

I. Appendices to Chapter Four

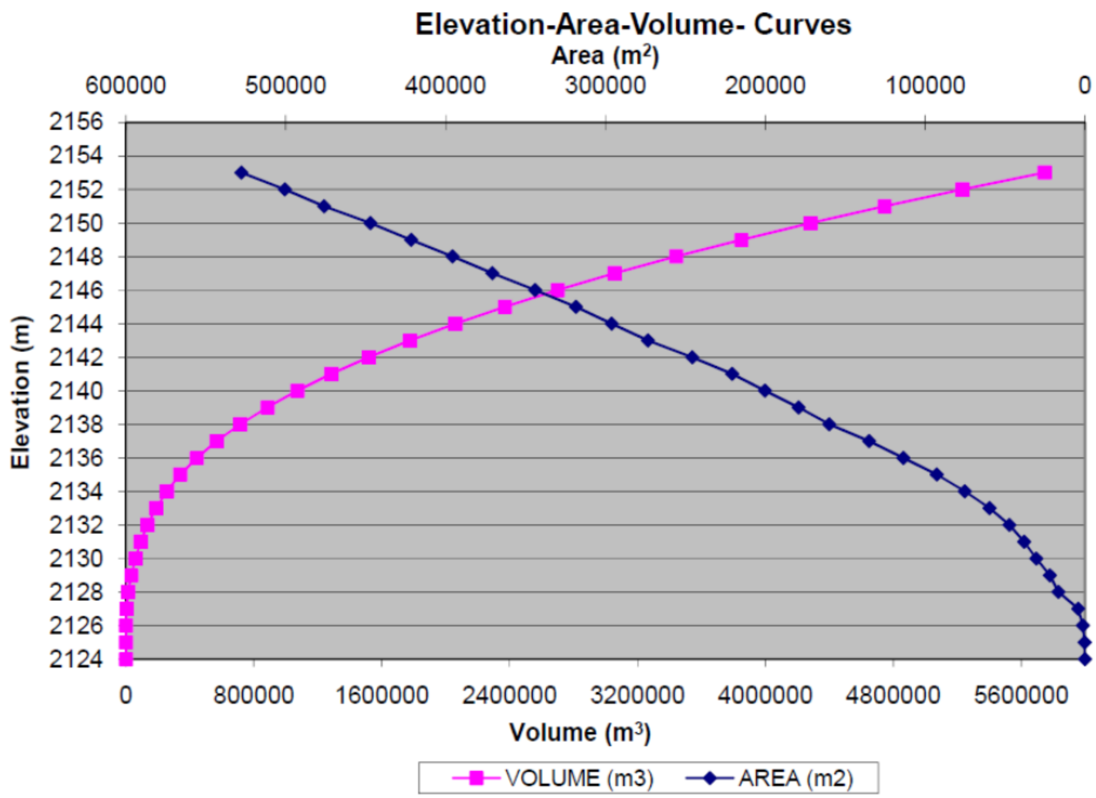

Figure A1: Area-Capacity curve for Grindeho reservoir (TBoWR, 2012)

Figure: Grain size distribution graph

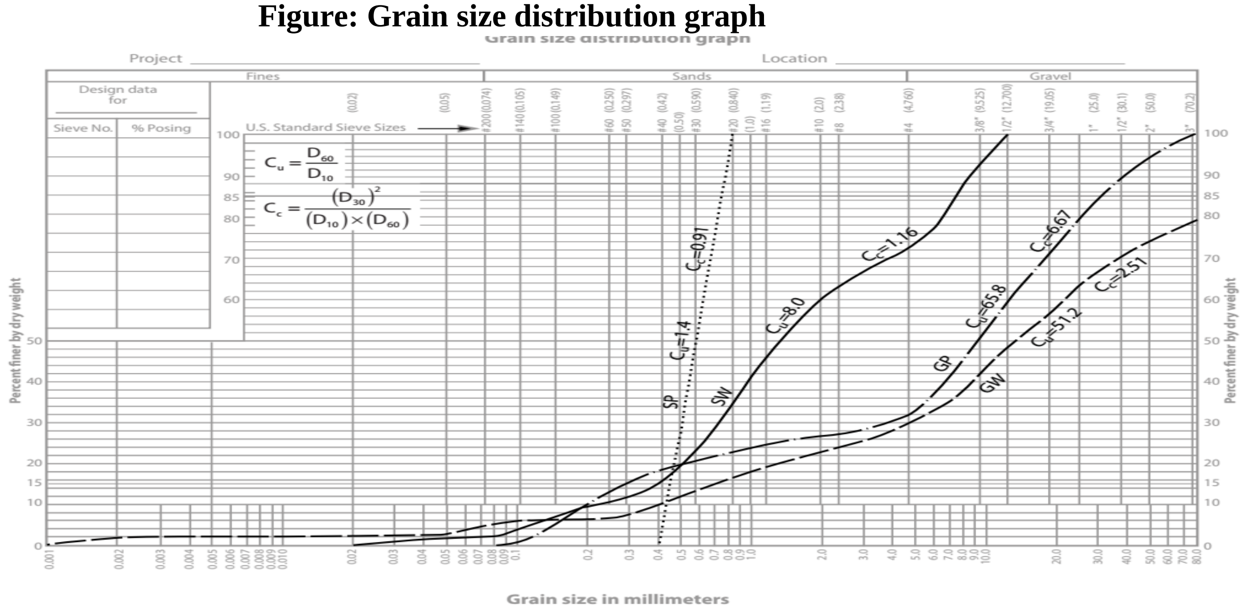

Figure A2: Grain size distribution graph

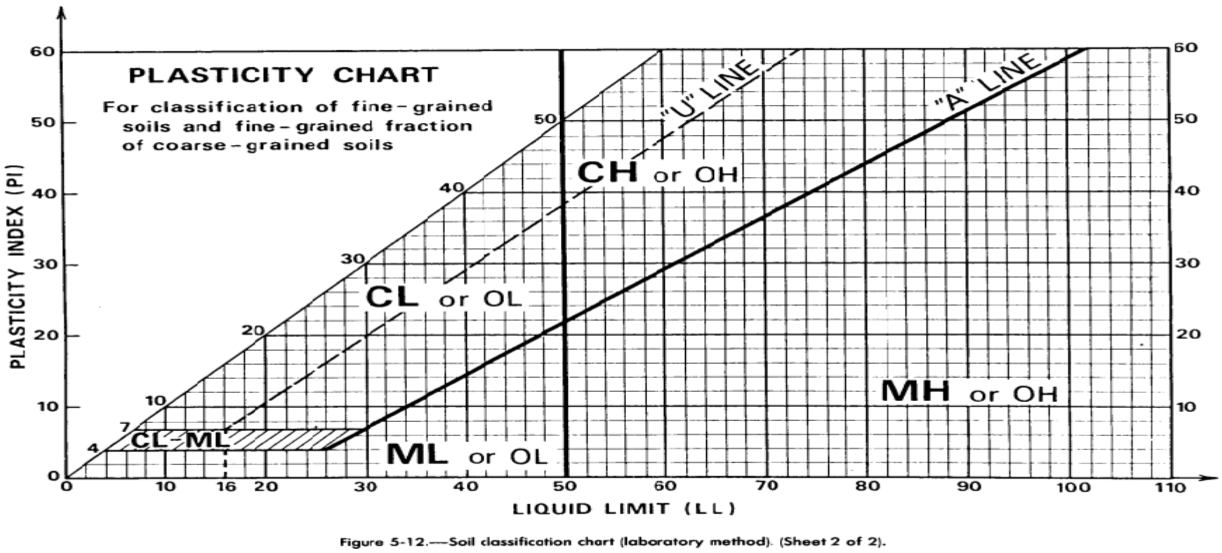

Figure A3: plasticity chart

Table A1: Grain size analysis result for shell material

Sieve size (mm)	Weight Of sieve (gm)	wt of sieve + retained soil (gm)	Mass Retained (gm)	Percent retained (%)	Com. % retained	Percentage Finer (%)
37.50	327	327	0.00	0.00	0.00	100.00
28.00	352	396	44.00	1.62	1.62	98.38
20.00	337	472	135.00	4.98	6.61	93.39
14.00	295	494	199.00	7.35	13.95	86.05
10.00	307	563	256.00	9.45	23.40	76.60
5.00	538	956	418.00	15.43	38.83	61.17
2.36	442	966	524.00	19.34	58.18	41.82
1.18	493	876	383.00	14.14	72.31	27.69
0.60	390	656	266.00	9.82	82.13	17.87
0.30	361	569	208.00	7.68	89.81	10.19
0.15	342	482	140.00	5.17	94.98	5.02
0.075	418	475	57.00	2.10	97.08	2.92
Pan	301	377	76.00	2.81	99.89	0.00

Table A2: Grain size analysis result for clay material

Sieve size (mm)	Weight Of sieve	wt of sieve + retained soil	Mass Retained (gm)	Percent retained (%)	Com. % retained	Percentage Finer (%)
4.75	449	473	24	48.98	48.98	51.02
2.00	434	445	11	22.45	71.43	28.57
0.425	358	366	8	16.33	87.76	12.24
0.075	418	424	6	12.24	100	0
Pan	301	301	0	0	100	0

Table A3: Atterberg limit test results for shell material

Container no.	Weight of can	Weight of Can +wet soil	Weight of can +dry soil	Moisture Content (%)	No. Of blow
1	121	149	141	40.00	18
2	130	166	156	38.46	22
3	130	163	154	37.50	29
4	109	131	125.2	35.80	39

Table A4: Atterberg limit test results for clay material

Container no.	Weight of can	Weight of Can +wet soil	Weight of can +dry soil	Moisture Content (%)	No. Of blow
1	121	151	139	66.67	15
2	124	156	144	60.00	21
3	131	157	148	52.94	32
4	124	149	141	47.06	39

Table A5: unconfined compression test (UCS Test) of clay core material for Grindeho Dam

A1:I22B1A

Unconfined Compression Test (UCS Test)of clay core material for grideho dam

Unconfined Compression Test (UCS Test)								
Calibration factor 5N per division								
Dimensions of mold Do = 50mm and Ho = 100mm			**Cross sectional area Ao = [V/H] = 1963.495mm2**			Value ox X (cm) = 2.8, 3		
Volume of mold, V = 196349.54mm3						Value ox Y (cm) = 2.2,1.5		
VDDR	**Δl =VDDR*0.01**	**Ɛ = Δl/Lo (%)**	**Value PR1**	**value PR2**	**Ac =Ao/(1 - Ɛ)**	**Vertical load = P = VPR*Calib number**	**Deviator stress = [P/Ac) (Kpa)**	**Remark**
0.00	0.00	0.00	0.00	0.00	1,963.50	0.00	0.00	
20.00	0.20	0.20	4.00	7.00	1,967.43	27.50	13.98	
40.00	0.40	0.40	9.00	12.00	1,971.38	52.50	26.63	
60.00	0.60	0.60	14.00	17.00	1,975.35	77.50	39.23	
80.00	0.80	0.80	19.00	22.00	1,979.33	102.50	51.79	
100.00	1.00	1.00	24.00	28.00	1,983.33	130.00	65.55	
120.00	1.20	1.20	29.00	32.00	1,987.34	152.50	76.74	
140.00	1.40	1.40	34.00	39.00	1,991.37	182.50	91.65	
160.00	1.60	1.60	39.00	45.00	1,995.42	210.00	105.24	
180.00	1.80	1.80	44.00	50.00	1,999.49	235.00	117.53	
200.00	2.00	2.00	59.00	59.00	2,003.57	295.00	147.24	
220.00	2.20	2.20	63.00	65.00	2,007.66	320.00	159.39	
240.00	2.40	2.40	68.00	68.00	2,011.78	340.00	169.00	
260.00	2.60	2.60	64.00	75.00	2,015.91	347.50	172.38	
280.00	2.80	2.80	58.00	70.00	2,020.06	320.00	158.41	
300.00	3.00	3.00	54.00	64.00	2,024.22	295.00	145.74	
320.00	3.20	3.20	50.00	57.00	2,028.40	267.50	131.88	
340.00	3.40	3.40	40.00	49.00	2,032.60	222.50	109.47	
360.00	3.60	3.60	36.00	41.00	2,036.82	192.50	94.51	
380.00	3.80	3.80	32.00	32.00	2,041.06	160.00	78.39	
400.00	4.00	4.00	28.00	22.00	2,045.31	125.00	61.12	
420.00	4.20	4.20	24.00	12.00	2,049.58	90.00	43.91	

Unconfined Compression Test graph

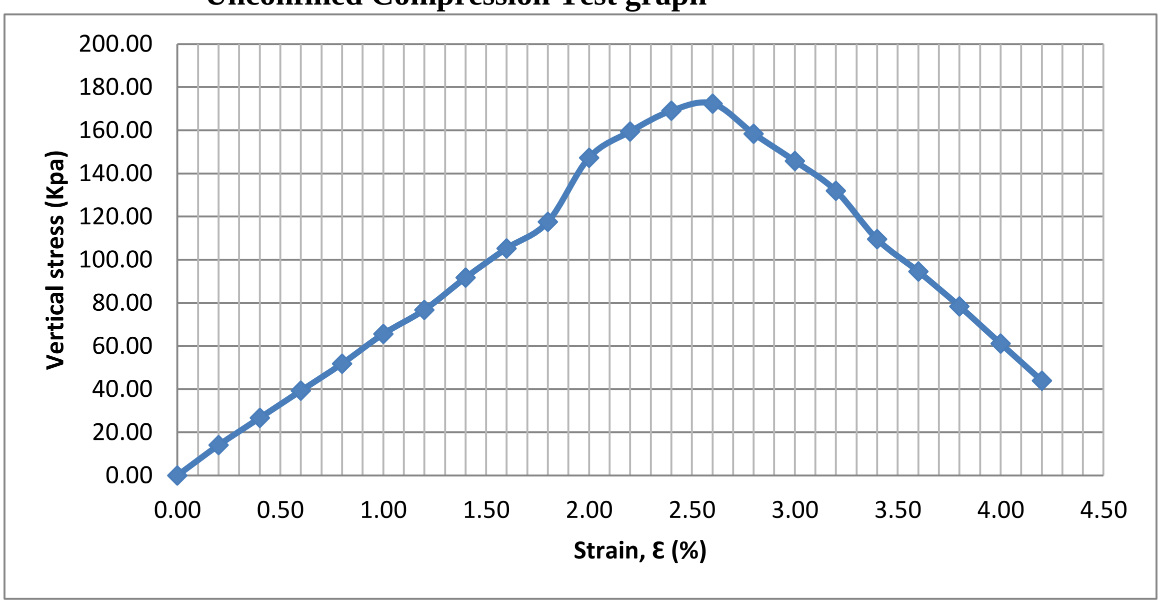

Figure A4: unconfined compression test graph of clay core Grindeho dam

II. Appendices: Geo-Studio Software Results

Transient Seepage Analysis

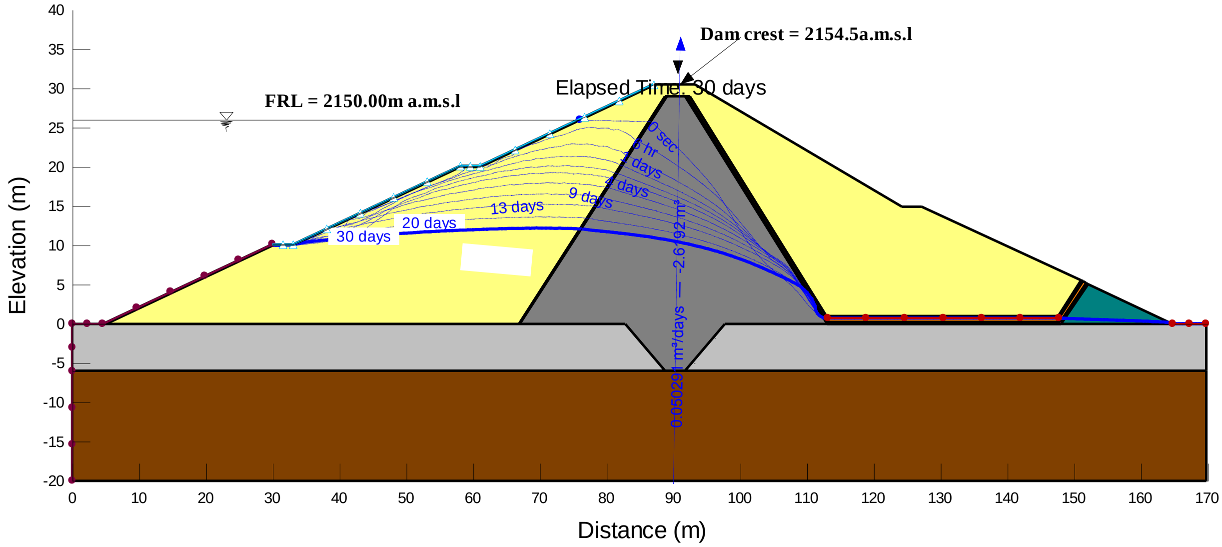

Figure A5: Transient seepage Analysis of grindeo dam

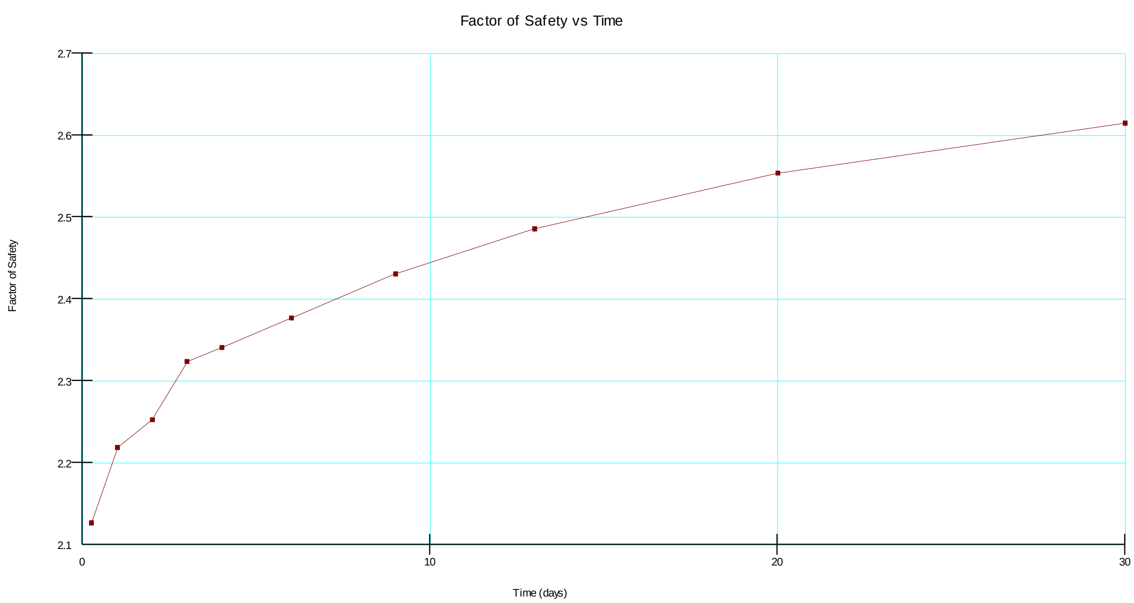

Figure A6: Factor of safety vs Time graph.

Steady-State Seepage

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File Information

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Revision Number: 179

Last Edited By: selam

Date: 5/5/17

Time: 1:02:47 PM

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Project Settings

Length (L) Units: meters

Time (t) Units: Seconds

Force (F) Units: kN

Pressure (p) Units: kPa

Mass (M) Units: g

Mass Flux Units: g/sec

Unit Weight of Water: 9.807 kN/m^3

View: 2D

Analysis Settings

Steady-State Seepage

Kind: SEEP/W

Method: Steady-State

Settings

Include Air Flow: No

Control

Apply Runoff: Yes

Convergence

Maximum Number of Iterations: 50

Tolerance: 0.1

Maximum Change in K: 1

Rate of Change in K: 1.1

Minimum Change in K: 0.0001

Equation Solver: Parallel Direct

Potential Seepage Max # of Reviews: 10

Time

Starting Time: 0 sec

Duration: 0 sec

Ending Time: 0 sec

Materials

Shell material

Model: Saturated / Unsaturated

Hydraulic

K-Function: shell Hydraulic conductivity

Vol. WC. Function: shell vol. Water content

K-Ratio: 1

K-Direction: 0 °

Clay Core Material

Model: Saturated / Unsaturated

Hydraulic

K-Function: clay HC

Vol. WC. Function: core vol. water content

K-Ratio: 1

K-Direction: 0 °

Foundation Material

Model: Saturated / Unsaturated

Hydraulic

K-Function: Foundation H.C

Vol. WC. Function: Found. VWC

K-Ratio: 1

K-Direction: 0 °

Riprap Material

Model: Saturated / Unsaturated

Hydraulic

K-Function: Rock toe H.C

Vol. WC. Function: Rip rap vol. water content

K-Ratio: 1

K-Direction: 0 °

Transition filter Material

Model: Saturated / Unsaturated

Hydraulic

K-Function: transition Filter Hydraulic conductivity

Vol. WC. Function: transition Filter Vol. content

K-Ratio: 1

K-Direction: 0 °

Drain Filter Material

Model: Saturated / Unsaturated

Hydraulic

K-Function: drain filter H.C

Vol. WC. Function: Drain Filter vol. water content

K-Ratio: 1

K-Direction: 0 °

Rock toe Material

Model: Saturated / Unsaturated

Hydraulic

K-Function: Rock toe H.C

Vol. WC. Function: Rock toe vol. water content

K-Ratio: 1

K-Direction: 0 °

Bed Rock Foundation

Model: Saturated / Unsaturated

Hydraulic

K-Function: bed rock foundation H.C

Vol. WC. Function: Bed Rock Foundation vol. water content

K-Ratio: 1

K-Direction: 0 °

Boundary Conditions

Zero Pressure

Type: Pressure Head 0

Potential Seepage Face

Review: true

Type: Total Flux (Q) 0

Reservoir head

Type: Head (H) 26

Flux Sections

Flux Section 1

Coordinates

Coordinate: (90, -20.402533) m

Coordinate: (91, 36.554538) m

K Functions

Shell Hydraulic conductivity

Model: Data Point Function

Function: X-Conductivity vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

K-Saturation: 8.09e-006

Data Points: Matric Suction (kPa), X-Conductivity (m/sec)

Data Point: (0.01, 8.09e-006)

Data Point: (0.018329807, 7.2428378e-006)

Data Point: (0.033598183, 6.2707551e-006)

Data Point: (0.061584821, 5.1832017e-006)

Data Point: (0.11288379, 4.0090564e-006)

Data Point: (0.20691381, 2.8142098e-006)

Data Point: (0.37926902, 1.7137962e-006)

Data Point: (0.6951928, 8.5422114e-007)

Data Point: (1.274275, 3.3059704e-007)

Data Point: (2.3357215, 9.82666e-008)

Data Point: (4.2813324, 2.3538493e-008)

Data Point: (7.8475997, 4.9012018e-009)

Data Point: (14.384499, 9.4521177e-010)

Data Point: (26.366509, 1.7551571e-010)

Data Point: (48.329302, 3.2019545e-011)

Data Point: (88.586679, 5.7943865e-012)

Data Point: (162.37767, 1.0447726e-012)

Data Point: (297.63514, 1.8807405e-013)

Data Point: (545.55948, 3.3831402e-014)

Data Point: (1000, 6.0837335e-015)

Estimation Properties

Volume Water Content Function: shell vol. Water content

Hydraulic K Sat: 8.09e-006 m/sec

Hyd. K-Function Estimation Method: Van Genuchten Function

Maximum: 1000

Minimum: 0.01

Num. Points: 20

Residual Water Content: 0.057 m³/m³

Clay HC

Model: Data Point Function

Function: X-Conductivity vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

K-Saturation: 2.57e-007

Data Points: Matric Suction (kPa), X-Conductivity (m/sec)

Data Point: (0.01, 2.57e-007)

Data Point: (0.018329807, 2.547086e-007)

Data Point: (0.033598183, 2.5155665e-007)

Data Point: (0.061584821, 2.4724046e-007)

Data Point: (0.11288379, 2.413434e-007)

Data Point: (0.20691381, 2.3331816e-007)

Data Point: (0.37926902, 2.2246056e-007)

Data Point: (0.6951928, 2.0789538e-007)

Data Point: (1.274275, 1.8860932e-007)

Data Point: (2.3357215, 1.6360428e-007)

Data Point: (4.2813324, 1.3232688e-007)

Data Point: (7.8475997, 9.5693742e-008)

Data Point: (14.384499, 5.777997e-008)

Data Point: (26.366509, 2.6470639e-008)

Data Point: (48.329302, 8.4465831e-009)

Data Point: (88.586679, 1.8795756e-009)

Data Point: (162.37767, 3.224459e-010)

Data Point: (297.63514, 4.7963807e-011)

Data Point: (545.55948, 6.6803803e-012)

Data Point: (1000, 9.0497167e-013)

Estimation Properties

Volume Water Content Function: core vol. water content

Hydraulic K Sat: 2.57e-007 m/sec

Hyd. K-Function Estimation Method: Van Genuchten Function

Maximum: 1000

Minimum: 0.01

Num. Points: 20

Residual Water Content: 0.07 m³/m³

Foundation H.C

Model: Data Point Function

Function: X-Conductivity vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

K-Saturation: 1.63e-008

Data Points: Matric Suction (kPa), X-Conductivity (m/sec)

Data Point: (0.01, 1.63e-008)

Data Point: (0.018329807, 1.6279126e-008)

Data Point: (0.033598183, 1.6247117e-008)

Data Point: (0.061584821, 1.6198199e-008)

Data Point: (0.11288379, 1.6123435e-008)

Data Point: (0.20691381, 1.6009266e-008)

Data Point: (0.37926902, 1.5835199e-008)

Data Point: (0.6951928, 1.5570402e-008)

Data Point: (1.274275, 1.5169019e-008)

Data Point: (2.3357215, 1.4564129e-008)

Data Point: (4.2813324, 1.3661012e-008)

Data Point: (7.8475997, 1.2334802e-008)

Data Point: (14.384499, 1.0446133e-008)

Data Point: (26.366509, 7.9162636e-009)

Data Point: (48.329302, 4.9346493e-009)

Data Point: (88.586679, 2.2175471e-009)

Data Point: (162.37767, 6.3049803e-010)

Data Point: (297.63514, 1.1328592e-010)

Data Point: (545.55948, 1.4928761e-011)

Data Point: (1000, 1.6898405e-012)

Estimation Properties

Volume Water Content Function: Found. VWC

Hydraulic K Sat: 1.63e-008 m/sec

Hyd. K-Function Estimation Method: Van Genuchten Function

Maximum: 1000

Minimum: 0.01

Num. Points: 20

Residual Water Content: 0.16 m^3/m^3

Transition Filter Hydraulic conductivity

Model: Data Point Function

Function: X-Conductivity vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

K-Saturation: 0.019

Data Points: Matric Suction (kPa), X-Conductivity (m/sec)

Data Point: (0.01, 0.019)

Data Point: (0.018329807, 0.018999859)

Data Point: (0.033598183, 0.01899918)

Data Point: (0.061584821, 0.018995923)

Data Point: (0.11288379, 0.01898033)

Data Point: (0.20691381, 0.01890505)

Data Point: (0.37926902, 0.018542569)

Data Point: (0.6951928, 0.016842225)

Data Point: (1.274275, 0.010369233)

Data Point: (2.3357215, 0.0014089806)

Data Point: (4.2813324, 2.2057379e-005)

Data Point: (7.8475997, 1.5164059e-007)

Data Point: (14.384499, 9.1902645e-010)

Data Point: (26.366509, 5.4877754e-012)

Data Point: (48.329302, 3.271315e-014)

Data Point: (88.586679, 1.9496797e-016)

Data Point: (162.37767, 1.161969e-018)

Data Point: (297.63514, 6.925078e-021)

Data Point: (545.55948, 4.1271947e-023)

Data Point: (1000, 2.4597322e-025)

Estimation Properties

Volume Water Content Function: transition Filter Vol. content

Hydraulic K Sat: 0.019 m/sec

Hyd. K-Function Estimation Method: Van Genuchten Function

Maximum: 1000

Minimum: 0.01

Num. Points: 20

Residual Water Content: 0.035 m^3/m^3

Drain filter H.C

Model: Data Point Function

Function: X-Conductivity vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

K-Saturation: 0.0085

Data Points: Matric Suction (kPa), X-Conductivity (m/sec)

Data Point: (0.01, 0.0085)

Data Point: (0.018329807, 0.0084999998)

Data Point: (0.033598183, 0.008499999)

Data Point: (0.061584821, 0.0084999946)

Data Point: (0.11288379, 0.0084999705)

Data Point: (0.20691381, 0.0084998397)

Data Point: (0.37926902, 0.0084991303)

Data Point: (0.6951928, 0.0084952765)

Data Point: (1.274275, 0.0084741474)

Data Point: (2.3357215, 0.0083588768)

Data Point: (4.2813324, 0.0077384656)

Data Point: (7.8475997, 0.0049919172)

Data Point: (14.384499, 0.0006535874)

Data Point: (26.366509, 7.7277039e-006)

Data Point: (48.329302, 3.8391217e-008)

Data Point: (88.586679, 1.6965708e-010)

Data Point: (162.37767, 7.4076451e-013)

Data Point: (297.63514, 3.2303717e-015)

Data Point: (545.55948, 1.4085458e-017)

Data Point: (1000, 6.141635e-020)

Estimation Properties

Volume Water Content Function: Drain Filter vol. water content

Hydraulic K Sat: 0.0085 m/sec

Hyd. K-Function Estimation Method: Van Genuchten Function

Maximum: 1000

Minimum: 0.01

Num. Points: 20

Residual Water Content: 0.035 m³/m³

Rock toe H.C

Model: Data Point Function

Function: X-Conductivity vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

K-Saturation: 0.076

Data Points: Matric Suction (kPa), X-Conductivity (m/sec)

Data Point: (0.01, 0.076)

Data Point: (0.018329807, 0.0026002963)

Data Point: (0.033598183, 6.3257927e-006)

Data Point: (0.061584821, 9.9148296e-009)

Data Point: (0.11288379, 1.5032733e-011)

Data Point: (0.20691381, 2.2741184e-014)

Data Point: (0.37926902, 3.4396912e-017)

Data Point: (0.6951928, 5.2026104e-020)

Data Point: (1.274275, 7.8690658e-023)

Data Point: (2.3357215, 1.190227e-025)

Data Point: (4.2813324, 1.8004449e-028)

Data Point: (7.8475997, 2.7241124e-031)

Data Point: (14.384499, 4.1890537e-034)

Data Point: (26.366509, 9.4378353e-037)

Data Point: (48.329302, 1.501543e-038)

Data Point: (88.586679, 3.5324778e-039)

Data Point: (162.37767, 1.1674567e-039)

Data Point: (297.63514, 4.5265943e-040)

Data Point: (545.55948, 3.0333988e-040)

Data Point: (1000, 6.6104724e-041)

Estimation Properties

Volume Water Content Function: Rock toe vol. water content

Hydraulic K Sat: 0.076 m/sec

Hyd. K-Function Estimation Method: Van Genuchten Function

Maximum: 1000

Minimum: 0.01

Num. Points: 20

Residual Water Content: 1e-005 m^3/m^3

Bed rock foundation H.C

Model: Data Point Function

Function: X-Conductivity vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

K-Saturation: 4.32e-009

Data Points: Matric Suction (kPa), X-Conductivity (m/sec)

Data Point: (0.01, 4.32e-009)

Data Point: (0.018329807, 4.0010611e-009)

Data Point: (0.033598183, 3.6599345e-009)

Data Point: (0.061584821, 3.2980852e-009)

Data Point: (0.11288379, 2.9174206e-009)

Data Point: (0.20691381, 2.5213282e-009)

Data Point: (0.37926902, 2.1151198e-009)

Data Point: (0.6951928, 1.7067046e-009)

Data Point: (1.274275, 1.3075247e-009)

Data Point: (2.3357215, 9.3348717e-010)

Data Point: (4.2813324, 6.0500516e-010)

Data Point: (7.8475997, 3.4406587e-010)

Data Point: (14.384499, 1.655353e-010)

Data Point: (26.366509, 6.5763044e-011)

Data Point: (48.329302, 2.1694447e-011)

Data Point: (88.586679, 6.1666032e-012)

Data Point: (162.37767, 1.585972e-012)

Data Point: (297.63514, 3.845701e-013)

Data Point: (545.55948, 9.0327693e-014)

Data Point: (1000, 2.0870677e-014)

Estimation Properties

Volume Water Content Function: Bed Rock Foundation vol. water content

Hydraulic K Sat: 4.32e-009 m/sec

Hyd. K-Function Estimation Method: Van Genuchten Function

Maximum: 1000

Minimum: 0.01

Num. Points: 20

Residual Water Content: 9e-006 m³/m³

Vol. Water Content Functions

Shell vol. Water content

Model: Data Point Function

Function: Vol. Water Content vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

Mv: 0 /kPa

Saturated Water Content: 0 m³/m³

Porosity: 0.41110549

Data Points: Matric Suction (kPa), Vol. Water Content (m³/m³)

Data Point: (0.01, 0.40999894)

Data Point: (0.018329807, 0.40999894)

Data Point: (0.033598183, 0.40999894)

Data Point: (0.061584821, 0.40999894)

Data Point: (0.11288379, 0.40999894)

Data Point: (0.20691381, 0.40999894)

Data Point: (0.37926902, 0.40999894)

Data Point: (0.6951928, 0.40998434)

Data Point: (1.274275, 0.40777969)

Data Point: (2.3357215, 0.3700382)

Data Point: (4.2813324, 0.303891)

Data Point: (7.8475997, 0.25364895)

Data Point: (14.384499, 0.22010093)

Data Point: (26.366509, 0.19512174)

Data Point: (48.329302, 0.1742049)

Data Point: (88.586679, 0.15537424)

Data Point: (162.37767, 0.13770752)

Data Point: (297.63514, 0.12077627)

Data Point: (545.55948, 0.10449651)

Data Point: (1000, 0.089041838)

Estimation Properties

Vol. WC Estimation Method: Grain Size Function

Sample Material: Clay

Saturated Water Content: 0.41 m^3/m^3

Liquid Limit: 22.5 %

Diameter at 10% passing: 0.002

Diameter at 60% passing: 4

Maximum: 1000

Minimum: 0.01

Num. Points: 20

Core vol. water content

Model: Data Point Function

Function: Vol. Water Content vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

Mv: 0 /kPa

Saturated Water Content: 0 m^3/m^3

Porosity: 0.35931465

Data Points: Matric Suction (kPa), Vol. Water Content (m^3/m^3)

Data Point: (0.01, 0.35903644)

Data Point: (0.018329807, 0.35903644)

Data Point: (0.033598183, 0.35903644)

Data Point: (0.061584821, 0.35903644)

Data Point: (0.11288379, 0.35903644)

Data Point: (0.20691381, 0.35903644)

Data Point: (0.37926902, 0.35903644)

Data Point: (0.6951928, 0.35903644)

Data Point: (1.274275, 0.35903644)

Data Point: (2.3357215, 0.35903644)

Data Point: (4.2813324, 0.35903644)

Data Point: (7.8475997, 0.35903644)

Data Point: (14.384499, 0.35903644)

Data Point: (26.366509, 0.35835929)

Data Point: (48.329302, 0.32270043)

Data Point: (88.586679, 0.25632056)

Data Point: (162.37767, 0.20788804)

Data Point: (297.63514, 0.17803426)

Data Point: (545.55948, 0.15731824)

Data Point: (1000, 0.14077062)

Estimation Properties

Vol. WC Estimation Method: Grain Size Function

Sample Material: Clay

Saturated Water Content: 0.36 m^3/m^3

Liquid Limit: 30 %

Diameter at 10% passing: 0.0014

Diameter at 60% passing: 0.003

Maximum: 1000

Minimum: 0.01

Num. Points: 20

Found. VWC

Model: Data Point Function

Function: Vol. Water Content vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

Mv: 0 /kPa

Saturated Water Content: 0 m³/m³

Porosity: 0.34032746

Data Points: Matric Suction (kPa), Vol. Water Content (m³/m³)

Data Point: (0.01, 0.33983221)

Data Point: (0.018329807, 0.33983221)

Data Point: (0.033598183, 0.33983221)

Data Point: (0.061584821, 0.33983221)

Data Point: (0.11288379, 0.33983221)

Data Point: (0.20691381, 0.33983221)

Data Point: (0.37926902, 0.33983221)

Data Point: (0.6951928, 0.33983221)

Data Point: (1.274275, 0.33983221)

Data Point: (2.3357215, 0.33983221)

Data Point: (4.2813324, 0.33983221)

Data Point: (7.8475997, 0.33983221)

Data Point: (14.384499, 0.33983221)

Data Point: (26.366509, 0.33983221)

Data Point: (48.329302, 0.33983221)

Data Point: (88.586679, 0.32769642)

Data Point: (162.37767, 0.29177623)

Data Point: (297.63514, 0.25494246)

Data Point: (545.55948, 0.22554889)

Data Point: (1000, 0.20166504)

Estimation Properties

Vol. WC Estimation Method: Grain Size Function

Sample Material: Clay

Saturated Water Content: 0.34 m³/m³

Liquid Limit: 42 %

Diameter at 10% passing: 0.001

Diameter at 60% passing: 0.002

Maximum: 1000

Minimum: 0.01

Num. Points: 20

Transition Filter vol. content

Model: Data Point Function

Function: Vol. Water Content vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

Mv: 0 /kPa

Saturated Water Content: 0 m^3/m^3

Porosity: 0.43373262

Data Points: Matric Suction (kPa), Vol. Water Content (m^3/m^3)

Data Point: (0.01, 0.42999885)

Data Point: (0.018329807, 0.42999885)

Data Point: (0.033598183, 0.42999885)

Data Point: (0.061584821, 0.42999885)

Data Point: (0.11288379, 0.42999885)

Data Point: (0.20691381, 0.42999885)

Data Point: (0.37926902, 0.42999885)

Data Point: (0.6951928, 0.42999885)

Data Point: (1.274275, 0.41382794)

Data Point: (2.3357215, 0.24718198)

Data Point: (4.2813324, 0.088898094)

Data Point: (7.8475997, 0.040592273)

Data Point: (14.384499, 0.02976742)

Data Point: (26.366509, 0.025272104)

Data Point: (48.329302, 0.021748147)

Data Point: (88.586679, 0.018607885)

Data Point: (162.37767, 0.015807197)

Data Point: (297.63514, 0.013335821)

Data Point: (545.55948, 0.011173698)

Data Point: (1000, 0.0092938228)

Estimation Properties

Vol. WC Estimation Method: Grain Size Function

Sample Material: Clay

Saturated Water Content: 0.43 m³/m³

Liquid Limit: 0.1 %

Diameter at 10% passing: 0.09

Diameter at 60% passing: 0.5

Maximum: 1000

Minimum: 0.01

Num. Points: 20

Drain Filter vol. water content

Model: Data Point Function

Function: Vol. Water Content vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

Mv: 0 /kPa

Saturated Water Content: 0 m³/m³

Porosity: 0.43948651

Data Points: Matric Suction (kPa), Vol. Water Content (m³/m³)

Data Point: (0.01, 0.42474027)

Data Point: (0.018329807, 0.42474027)

Data Point: (0.033598183, 0.42474027)

Data Point: (0.061584821, 0.42474027)

Data Point: (0.11288379, 0.42474027)

Data Point: (0.20691381, 0.42474027)

Data Point: (0.37926902, 0.42474027)

Data Point: (0.6951928, 0.42474027)

Data Point: (1.274275, 0.42474027)

Data Point: (2.3357215, 0.42474027)

Data Point: (4.2813324, 0.42474027)

Data Point: (7.8475997, 0.42474027)

Data Point: (14.384499, 0.2461559)

Data Point: (26.366509, 0.082613798)

Data Point: (48.329302, 0.04826525)

Data Point: (88.586679, 0.039266114)

Data Point: (162.37767, 0.033416434)

Data Point: (297.63514, 0.028341264)

Data Point: (545.55948, 0.023835793)

Data Point: (1000, 0.01987232)

Estimation Properties

Vol. WC Estimation Method: Grain Size Function

Sample Material: Clay

Saturated Water Content: 0.43 m^3/m^3

Liquid Limit: 0.1 %

Diameter at 10% passing: 0.09

Diameter at 60% passing: 0.05

Maximum: 1000

Minimum: 0.01

Num. Points: 20

Rock toe vol. water content

Model: Data Point Function

Function: Vol. Water Content vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

Mv: 0 /kPa

Saturated Water Content: 0 m^3/m^3

Porosity: 0.002146308

Data Points: Matric Suction (kPa), Vol. Water Content (m^3/m^3)

Data Point: (0.01, 0.002146308)

Data Point: (0.018329807, 0.00071500841)

Data Point: (0.033598183, 0.00053925005)

Data Point: (0.061584821, 0.00046306998)

Data Point: (0.11288379, 0.00040258368)

Data Point: (0.20691381, 0.00035013554)

Data Point: (0.37926902, 0.00030408565)

Data Point: (0.6951928, 0.0002636286)

Data Point: (1.274275, 0.00022812296)

Data Point: (2.3357215, 0.00019699834)

Data Point: (4.2813324, 0.00016974464)

Data Point: (7.8475997, 0.0001459072)

Data Point: (14.384499, 0.00012508215)

Data Point: (26.366509, 0.00010691119)

Data Point: (48.329302, 9.1076762e-005)

Data Point: (88.586679, 7.7297809e-005)

Data Point: (162.37767, 6.5325573e-005)

Data Point: (297.63514, 5.494013e-005)

Data Point: (545.55948, 4.5947126e-005)

Data Point: (1000, 3.8174864e-005)

Estimation Properties

Vol. WC Estimation Method: Grain Size Function

Sample Material: Clay

Saturated Water Content: 0.4 m^3/m^3

Liquid Limit: 0 %

Diameter at 10% passing: 200

Diameter at 60% passing: 500

Maximum: 1000

Minimum: 0.01

Num. Points: 20

Rip rap vol. water content

Model: Data Point Function

Function: Vol. Water Content vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

Mv: 0 /kPa

Saturated Water Content: 0 m^3/m^3

Porosity: 9.9148484e-005

Data Points: Matric Suction (kPa), Vol. Water Content (m^3/m^3)

Data Point: (0.01, 9.9148484e-005)

Data Point: (0.018329807, 9.9148484e-005)

Data Point: (0.033598183, 9.9148484e-005)

Data Point: (0.061584821, 9.9148484e-005)

Data Point: (0.11288379, 9.9148484e-005)

Data Point: (0.20691381, 9.9148484e-005)

Data Point: (0.37926902, 9.9148484e-005)

Data Point: (0.6951928, 9.9148484e-005)

Data Point: (1.274275, 9.9148484e-005)

Data Point: (2.3357215, 9.9148484e-005)

Data Point: (4.2813324, 9.9148484e-005)

Data Point: (7.8475997, 9.9148484e-005)

Data Point: (14.384499, 9.9148484e-005)

Data Point: (26.366509, 9.9148484e-005)

Data Point: (48.329302, 9.9148484e-005)

Data Point: (88.586679, 9.9148484e-005)

Data Point: (162.37767, 9.9148484e-005)

Data Point: (297.63514, 9.9148484e-005)

Data Point: (545.55948, 9.9148484e-005)

Data Point: (1000, 9.9148484e-005)

Estimation Properties

Vol. WC Estimation Method: Grain Size Function

Sample Material: Clay

Saturated Water Content: 0.0001 m³/m³

Liquid Limit: 0 %

Diameter at 10% passing: 200

Diameter at 60% passing: 500

Maximum: 1000

Minimum: 0.01

Num. Points: 20

Bed Rock Foundation vol. water content

Model: Data Point Function

Function: Vol. Water Content vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

Mv: 0 /kPa

Saturated Water Content: 0 m³/m³

Porosity: 9.9683477e-005

Data Points: Matric Suction (kPa), Vol. Water Content (m³/m³)

Data Point: (0.01, 9.9523475e-005)

Data Point: (0.018329807, 9.9523475e-005)

Data Point: (0.033598183, 9.9523475e-005)

Data Point: (0.061584821, 9.9523475e-005)

Data Point: (0.11288379, 9.9523475e-005)

Data Point: (0.20691381, 9.9523475e-005)

Data Point: (0.37926902, 9.9523475e-005)

Data Point: (0.6951928, 9.9523475e-005)

Data Point: (1.274275, 9.9523475e-005)

Data Point: (2.3357215, 9.9523475e-005)

Data Point: (4.2813324, 9.9523475e-005)

Data Point: (7.8475997, 9.9523475e-005)

Data Point: (14.384499, 9.9523475e-005)

Data Point: (26.366509, 9.9523475e-005)

Data Point: (48.329302, 9.9523475e-005)

Data Point: (88.586679, 9.9523475e-005)

Data Point: (162.37767, 9.4392306e-005)

Data Point: (297.63514, 7.9781899e-005)

Data Point: (545.55948, 6.6911654e-005)

Data Point: (1000, 5.5686127e-005)

Estimation Properties

Vol. WC Estimation Method: Grain Size Function

Sample Material: Gravel

Saturated Water Content: 0.0001 m^3/m^3

Liquid Limit: 0 %

Diameter at 10% passing: 700

Diameter at 60% passing: 1200

Maximum: 1000

Minimum: 0.01

Num. Points: 20

Regions

	Material	Points	Area (m^2)
Region 1	Clay Core Material	21,20,19,18,24,16,25,22	760.89947
Region 2	Transition filter Material	18,26,17,24	5.7174653
Region 3	Riprap Material	2,3,4,5,6,7,8,32,33,34,35,36,37	12.417437
Region 4	Foundation Material	23,1,37,2,26,18,19,20	514.40879
Region 5	Foundation Material	22,25,39,12,13,14,21	449.97353
Region 6	Transition filter Material	16,27,28,29,31,38,39,15,25	16.866362
Region 7	Rock toe Material	39,38,12	42.180432

Region 8	Drain Filter Material	27,40,45,41,42,43,31,29,28	33.049386
Region 9	Transition filter Material	43,30,44,45,41,42	10.358067
Region 10	Shell material	2,3,4,5,6,7,8,9,10,11,30,44,45,40,27,16,24,17,26	1652.7857
Region 11	Bed Rock Foundation	20,21,14,47,46,23	2376.5

Lines

	Start Point	End Point	Hydraulic Boundary
Line 1	21	20	
Line 2	20	19	
Line 3	19	18	
Line 4	18	24	
Line 5	24	16	
Line 6	16	25	
Line 7	25	22	
Line 8	22	21	
Line 9	18	26	
Line 10	26	17	
Line 11	17	24	
Line 12	16	27	
Line 13	27	28	
Line 14	29	31	
Line 15	2	3	Reservoir head
Line 16	3	4	Reservoir head
Line 17	4	5	
Line 18	5	6	
Line 19	6	7	Reservoir head

Line 20	7	8	
Line 21	8	32	
Line 22	32	33	
Line 23	33	34	Reservoir head
Line 24	34	35	Reservoir head
Line 25	35	36	
Line 26	36	37	
Line 27	37	2	
Line 28	9	10	Potential Seepage Face
Line 29	10	11	Potential Seepage Face
Line 30	11	30	Potential Seepage Face
Line 31	26	2	
Line 32	23	1	
Line 33	1	37	Reservoir head
Line 34	12	13	Zero Pressure
Line 35	13	14	
Line 36	14	21	
Line 37	12	39	
Line 38	39	25	
Line 39	31	38	
Line 40	38	39	
Line 41	39	15	
Line 42	15	25	
Line 43	38	12	
Line 44	29	28	
Line 45	31	43	

Line 46	43	30	
Line 47	27	40	
Line 48	41	42	Zero Pressure
Line 49	42	43	
Line 50	40	45	
Line 51	45	41	
Line 52	30	44	
Line 53	44	45	
Line 54	8	9	
Line 55	23	20	
Line 56	14	47	
Line 57	47	46	
Line 58	46	23	

Points

	X (m)	Y (m)
Point 1	0	0
Point 2	5	0
Point 3	30	10
Point 4	33	10
Point 5	58	20
Point 6	61	20
Point 7	76	26
Point 8	87.25	30.5
Point 9	93.25	30.5
Point 10	124.25	15

Point 11	127.25	15
Point 12	164.75	0
Point 13	169.75	0
Point 14	169.75	-6
Point 15	113.5	0
Point 16	91.75	29
Point 17	88.75	29
Point 18	67	0
Point 19	82.75	0
Point 20	88.75	-6
Point 21	91.75	-6
Point 22	97.75	0
Point 23	0	-6
Point 24	88.994895	28.994359
Point 25	113.00156	-0.00079
Point 26	66.85247	-0.00313
Point 27	92.03009	29.00039
Point 28	112.9824	0.2430224
Point 29	147.9258	0.2440644
Point 30	150.9782	5.536664
Point 31	151.79885	5.209971
Point 32	87.01625	30.50267
Point 33	61.00442	20.12906
Point 34	58.01546	20.15597
Point 35	32.99891	10.1513
Point 36	29.96237	10.16768

Point 37	4.50668	0.00233
Point 38	152.12759	5.0604946
Point 39	148.08016	-0.0002544845
Point 40	92.40293	28.9961
Point 41	113.12597	0.74918
Point 42	147.74354	0.74957
Point 43	151.32588	5.398257
Point 44	147.65852	0.99917
Point 45	112.94108	1.0012004
Point 46	0	-20
Point 47	169.75	-20