JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

HYDROLOGY AND HYDRAULIC ENGINEERING CHAIR

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EMBANKMENT DAM SEEPAGE AND SLOPE STABILITY ANALYSIS: THE CASE OF ARJO-DEDESSA DAM, ETHIOPIA

A Thesis Submitted to the School of Graduate Studies of Jimma University in Partial fulfillment of the requirements for the Degree of Masters of Science in Hydraulic Engineering.

By: Ayansa Gamachu Aga

JANUARY, 2020

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Main Advisor: Tamene Adugna (Ph.D)

Co- advisor: Deme Betele (MSc.)

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DECLARATION

This thesis is my original work and has not been presented for a degree in any other university. All sources of materials used for the research have been duly acknowledged.

Ayansa Gamachu	____________________	_______________
Candidate	Signature	Date

APPROVAL PAGE

The thesis entitled "**Embankment dam seepage and slope stability analysis: The case of Arjo-dedessa dam, Ethiopia**" submitted by Ayansa Gamachu Aga is approved and accepted as a Partial Fulfillment of the Requirements for the Degree of Masters of Science in Hydraulic Engineering at Jimma Institute of Technology.

Tamene Adugna (Ph.D) ________________ ______________
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Deme Betele (MSc.) ___________________ _______________

Co- advisor: Signature Date

As members of the examining board of MSc. thesis, we certify that we have read and evaluated the thesis prepared by Ayansa Gamachu Aga. We recommend that the thesis could be accepted as a Partial Fulfillment of the Requirements for the Degree of Masters of Science in Hydraulic Engineering.

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Kiyya Tesfa (MSc.) _______________________ _______________

Internal examiner Signature Date

Tolera Abdisa (MSc.) _______________________ _______________

Chairman Signature Date

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ABSTRACT

Dams have one of the most important roles in utilizing water resources. An embankment dams are more common than any other type of dams. Seepage and slope stability failures cause completely failure of embankment dams. Dam failures not only risk public safety, they also can cost our economy millions of dollars in damages. The prime objective of this research was to assess Arjo-Dedessa embankment dam, highlighting on seepage and slope stability analysis. The study area is found in Oromia regional state between East Wollega and Bunno Bedele Zone. The dam is located on Dedessa River, tributary of Blue Nile River. Analysis of seepage quantity was done by Darcy's phreatic line, and SEEP/W software model for both homogeneous and zoned dam. The expected quantity of seepage estimated with the SEEP/W software model analysis that includes foundation seepage is $6.6*10^{-6} m^3/sec$ which was compared with the quantity of seepage estimated at the designed document that is $4.16*10^{-5} m^3/sec$. Therefore, the design document has no problem of quantifying the expected quantity of seepage. But seepage is visible at the downstream berms of dam and downstream face of the existing Upstream Coffer Dam. The document stated that, the shell material used for the design was larger than the required size of shell material which may create Wet spots or seepage problem in the embankment. Even though, the dam has a problem data on construction history. Slope stability analysis is to contribute to the safe and economic design of upstream and downstream slope of the dam. Factor of safety was calculated under the standard loading conditions for limit equilibrium methods using entry and exit trial slip surface. The factory of safety obtained by Morgenstern-Price method for end construction is (FOS=1.99), for steady state (FOS=2.03) and rapid drawdown (FOS=2.91).The result shows that dam was safe when compared with international standards and design document. However, at the design document shell material hadn't been designed properly. From field observation, there was also the oversized stone of shell materials and wrong placement of stones in the downstream of the dam which may leads to Face (slope) failure on the downstream slope of the dam. Finally the paper concluded that the possible remedial measures for seepage controls are impervious blanket, grouting, and for downstream slope failure by removing the weak zone and fill with similar graded material, regular maintenance and cover the downstream slope with grass.

Key words*: Limit equilibrium methods, phreatic line, seep/w, slop/w, steady state*

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ACRONMYS AND ABBREVIATION

BIS	Bureau of Indian Standard
Cc	Effective Cohesion
CWL	current water level
D/S	Down Stream
EL	Elevation
F_m	Moment Equilibrium
FOS	Factor of Safety
FRL	Full Reservoir Level
GPS	Global Positioning System
GLE	General Limit Equilibrium
Ha	Hectare
H	Height of Dam
Hd	Head over Crest
ICOLD	International Commission on Large Dams
IS	Indian Standard
Kh	Permeability along horizontal
Kv	Permeability along vertical
L_b	Length of Basin
MWIE	Ministry of Water, Irrigation and Electricity
MWR	Ministry of Water Resources
MWL	Maximum Water Level
MDD	Maximum Dry Density
MDDL	Minimum Draw down Level
NMC	Natural Moisture Content
OMC	Optimum Moisture Content
OWWDSE	Oromia Water Works Design & Supervision Enterprise
Seep/w	seepage for windows
U/S	Upstream
USBR	United States Bureau of Reclamation
WWDSE	Water Works Design & Supervision Enterprise

1. INTRODUCTION

1.1 Background

A dam is a barrier that blocks the flow of water and produces a reservoir. The water stored in the reservoir is used for various purposes, such as irrigation, municipal and industrial supply, hydropower and recreation. Dams may also be constructed for flood control, retention of debris, navigation and various other purposes. A dam and a reservoir are complements of each other. Dam failures and incidences have been taking place all over the world over a long period of time in history. Reports on failure of dams are common things nowadays. Effects of dam failure on man and environment are well known. These require both preventive and mitigation measures. Dam failures may occur due to a variety of causes. The most common causes of dam failure are leakage and piping, overtopping, spillway erosion, excessive deformation, sliding, gate failure, faulty construction, and earthquake instability (Umaru , *et al*., 2014).

Historical study of dams conceived in earlier times is essential. To continue advancing, the engineering profession must periodically review past problems and the lessons that they taught. Candid sharing of information on failures as well as successes is needed. In fact, some of the most valuable learning has come from projects where errors have been clear in review. Past dam failure disasters showed that the loss of life in the event of a dam failure is directly related to the warning time available to evacuate the population at risk downstream of the dam. Earth and rock fill dams are widely used throughout the world, and most of the dam failures involve such dams. To speak about failures of dams without a brief account of these happenings in the dam world is not possible (Sharma, *et al., 2013)*.

A dam failure is commonly defined as an incident of structural failure that involve unintended releases or surges of impounded water or incidents that lead to the loss of the dam. In some developed parts of the world, the problem of dam failures has always been of great importance because of their economic and environmental attributes. Therefore, the problem has always given rise to a particular interest among hydraulic engineers in estimating downstream valley that are risk of inundation in instances of dam failures (Kolala, *et al*., 2015).

Dam construction represents a major investment in basic infrastructure within all nations. The annual completion rate for of all sizes continues at a very high level in many countries. Dams are individually unique structures. Irrespective of size and type they demonstrate great complexity in their load response and in their interactive relationship with site hydrology and geology. In recognition of this, and reflecting the relatively indeterminate nature of many major design inputs, dam engineering is not a stylized and formal science. As practiced, it is a highly specialist activity which draws upon many scientific disciplines and balances them with a large element of engineering judgment; dam engineering is thus a uniquely challenging and stimulating field of endeavor. It is also important to recognize that many major dams are now necessarily built on less favorable and more difficult sites. A proportion of sites developed today would, in the past, have been rejected as uneconomic or even as quite unsuitable for a dam (Novak, *et al*., 2007).

Most dams have some seepage through or around the environment as a result of water moving through the soil structure. If the seepage forces are large enough soil could be eroded from the environment or foundation. Many seepage problems and failure of earth dams have occurred because of inadequate seepage control measures or poor clean up and preparation of the foundations and abutments. Excessive seepage not picked up by an embankment or foundation drain will be noticed as wetness, spring or boils on the lower back slope and toe of the dam. A change in vegetation is another indicator of seepage. Seepage causing problems that can lead to dam failure are piping through dam body or foundation and sloughing of downstream side of dam (Omofunmi, *et al*., 2017).

Slope stability analysis of earthen dam is dependent to many parameters which must consider in design and construction. Stability of these structures is composed of many ambiguities relevant to lack of precise geotechnical parameters. Because of the importance of dam construction and its related expenses, determination of dam behavior has an important result for makers. By considering the uncertainties of geotechnical parameters, applying risk analysis is unavoidable in dam construction. Investigating of slope stability of earthen dam slopes is an advanced procedure which has been shown to lead to a more economical design by many researchers which states that the cases that computed safety factor without modeling uncertainties is more than reality value. These methods do not consider many uncertainties in their computations (Yazdanian, *et al*., 2017).

Embankment dam failures may occur due to different reasons such as slope instability and seepage through body of dam and its foundations. The prime objective of this thesis was to assess the embankment dam failure focusing on seepage and slope stability analysis for Arjo-Dedessa dam. Earth dam failures are mainly caused by improper design, lack of thorough investigations, and inadequate care in construction and poor maintenance. Therefore in this study, dam failure analysis, seepage and slope stability was done using analytical and numerical methods for the standard or critical loading condition to assess the previous and current condition of the dam.

1.2 Statement of the problem

All embankment dams must functions safely under routine everyday operation as well as unusual conditions. The practical seepage problems are not easily convertible into an equivalent numerical counterpart because of the heterogeneity of the natural soils and the varying boundary conditions. Based on the parametric sensitivity analysis, both the seepage and stability studies have brought out the importance of considering the coupled effects on the overall stability of the earth dam. It is concluded that the coupled analysis is a prerequisite for the design and performance evaluation of the earth dam under all conditions of seepage and stability (Athania, *et al*., 2015).

Seepage problems can occur in either concrete or embankment dams as well as through or along the foundations. Uncontrolled seepage through an embankment dam can cause the movement of soil to unprotected exits, creating voids, and leading to "piping" failures. Stability problems in embankment dams are almost always preceded or accompanied by seepage problems. It is therefore essential to understand the seepage occurring through the dam and its foundation prior to doing stability analysis. Pore-water pressure and seepage measurements are the best indicators of dam safety condition. Both excessive seepage and slope instability may cause failure of the dam which may damage the infrastructures and greatly affect the life of several people and also causes socio-economic problems (Mekonnen, 2017).

Arjo-Dedessa project is designed for the irrigation purpose to provide irrigation facility to 80,000 ha of land for sugar cane development, to promote and encourage sustainable agricultural production. To attain this, dam failure due to the quantity of seepage and slope instability will be effectively mitigated and analyzed using various methods. When a dam fails, resources must be devoted to the prevention and treatment of public health risks as well as the resulting structural consequences (OWWDSE, 2017).

1.3 Objective of the study

1.3.1 General objective

The general objective of this research is to assess seepage and slope stability analysis of Arjo-dedessa embankment dam.

1.3.2 Specific objectives

1) To assess the quantity of seepage through the dam body and foundation.
2) To assess the static stability of the upstream and downstream of the dam.
3) To recommend the appropriate remedial measures for the result of the study.

1.4 Research questions

1) What is the quantity of seepage through the dam body and foundation?
2) What is the static stability of upstream and downstream of the dam?
3) What are the various remedial measures that can be taken?

1.5 Scope of the Study

This study is a step towards assessing embankment dam failure, highlighting to seepage and slope stability analysis. In addition, this study analysis the performance of the dam, to put appropriate remedial measure for the result of study and checks the seepage quantity and slope stability analysis based on the design document that kept on Oromia Region Water Resource Development Office. The research does not include the hydraulic failure analysis (overtopping,) and structural failure analysis (settlement and deformation analysis and earthquake analysis).

1.6 Significance of the Study

The study provides relevant information for the decision makers in the design of the dam and other developmental activities and various beneficial to the community at the dam. This information helps those bodies for taking appropriate decision making and under take effective remedial action to minimize failure of the dam and increasing the agricultural production for the return period of the dam and to be used as a guide in planning, designing new similar projects and redesign and maintenance of failed and existing dams. The main significance is the safe guarding of safety of the dam.

1.7 Limitation of the study

The problem faced through the study of this research is lack of primary data. When I collect secondary data the problem I come across are: Some of the government employees are not interested to give the documented data and full documented data is not available in their office.

1.8 Organization of the Thesis

The research is organized into five chapters. The first chapter is the introductory part which discusses the overall objective of the research, problem statement of research, scope of the study, Significance of the Study and limitation of the study. The second chapter discusses the literature review of the research. The third chapter discusses the methodology used to conduct the research. The result and discussions are discussed in chapter four. The fifth chapter concludes the study with the points of recommendation. Finally the list of references and appendix are included.

2. LITERATURE REVIEW

2.1 Embankment dam failure

An embankment dam is a massive artificial dam that are more common than any other type of dams because of various reasons like the use of ordinary technology construction method utilizing cheap raw soil materials and subsurface materials, no need of a particular valley shape etc. The geometry of embankment dams depends on burrowed soil materials, subsurface conditions and type of construction. It is typically created by the placement and compaction of a complex semi-plastic mound of various compositions of soil, sand, clay and/or rock. It has a semi-pervious waterproof natural covering for its surface and a dense, impervious core. This makes such a dam impervious to surface or seepage erosion (Redda, 2016).

Earth Dams are constructed where the foundation or underlying material or rocks are weak to support the masonry dam. Earth dams are less rigid and hence more susceptible to failure. The various causes leading to the failure of earth dams can be grouped into three categories (Saluja, *et al*., 2018).

2.1.1 Hydraulic failure

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. About 40% of earth dam's failures have been attributed to hydraulic failure. Hydraulic failures include: overtopping, erosion of u/S face, erosion of d/S face and erosion of d/S toe (Getachew, 2018).

2.1.2 Seepage failure

Most embankments exhibit some seepage. However, this seepage must be controlled in velocity and quantity. Seepage occurs through the earthen embankment or dike and/or through its foundation. Seepage, if uncontrolled, can erode fine soil material from the downstream slope or foundation and continue moving towards the upstream slope to form a pipe or cavity to the pond or lake often leading to a complete failure of the embankment. More than 1/3rd of the earth dams have failed because of these reasons. Seepage failures are piping through the body of the dam, piping through the foundation of the dam and Sloughing of downstream toe (Anyemedu, 2007).

2.1.3 Structural failure

Structural failures involve the separation (rupture) of the embankment material and/or its foundation. 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 Structural failures can occur in either the embankment or the appurtenances. About 25% of the dam failures have been attributed to structural failures. The structural failures are slides in embankments (u/s & d/s slope failures), foundation slides (spontaneous liquefaction), failure due to earthquake, earthquake loading can lead to failure of the dam itself but also of the foundation and the appurtenant structures (Getachew, 2018).

2.2 Embankment dam considerations

Most of the time locally available embankment material governs the type of embankment dam. If the site is dominated by only one type of material (soil) the design will consist of a homogeneous embankment. If it is impervious soil, a homogeneous embankment with only small amount of pervious material to control the internal erosion will be selected. If it is pervious material (sand or gravel), a dam with a very thin core may be used where enough impervious material is available to make a core; otherwise an impervious facing may be constructed. In homogeneous fill dams the slopes are flattened, which contributes to the seepage control by descending the velocity of the percolating water. For the case where varied material is available at the site a zoned dam which incorporates the material available on the site into the embankment will be selected. A zoned dam is a rolled fill dam composed of several zones that increases in permeability from the core towards the outer slopes (Tumoro, 2010).

The criteria commonly accepted for safe design of embankment dams are sufficient spillway capacity and freeboard are provided so that there is no danger of overtopping of the dam, Seepage flow through the embankment is controlled so that the amount lost does not interfere with the objective of the dam and there is no erosion or sloughing of soil, in this respect, seepage line should remain well within the downstream face of the dam and the portion of the dam on downstream side of the impervious core should be well drained, uplift pressure due to the seepage underneath is not enough to cause piping, the slopes of the embankment are stable under all conditions of reservoir operation, including rapid drawdown and during steady seepage under full reservoir, the stresses imposed by the embankment upon the foundation are less than the strength of material in the foundation with a suitable factor of safety, the upstream face is properly protected ((stone pitching, riprap, revetment) against erosion caused by wave action, and the downstream face is protected against the action of rain (Anyemedu, 2007).

2.3 Seepage in embankment dams

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 (Anteneh, 2008).

Embankment dam stability must be assessed in relation to the changing conditions of loading and seepage regime which develop from construction through first impounding into operational service, including reservoir drawdown. Seepage is always present within the body of any dam. Seepage flows and their resultant internal pressures must be directed and controlled. Internal drainage systems for this purpose are therefore an essential and critical feature of all modern dams. In embankments drainage is affected by suitably located pervious zones leading to horizontal blanket drains or outlets at base level. Serious under seepage is unlikely to be a problem in extensive and uniform deposits of competent clay. It is important, however, to identify and consider the influence of interbedded thin and more permeable horizons which may be present (Novak, *et al.*, 2007).

For embankment dams the main safety concern is seepage. Methods for monitoring the seepage and for detecting internal erosion give essential information for the safety evaluation of earth embankment dams. Together with overtopping, internal erosion through the dam or the foundation is the most frequent reason for embankment dam failures. Internal erosion in the embankment or in the foundation of the dam may reach an advanced stage before any sign is visible on the outside of the dam. The first indication may be higher seepage rates, a visually observable concentrated leak at the downstream toe or high turbidity in the seepage water (Sjödahl, *et al*., 2009).

2.3.1 Modes of Seepage failure

The failure mode of an embankment dam is directly connected with the type of cause of failure and the type of the dam. Abnormal increases of seepage quantity and leakage of turbid water are the visual indication of ongoing erosion. In some cases, internal erosion and piping may appear similar because, the induced force is common for both that obtained from the water flow with higher hydraulic gradient. But, both have completely different mechanisms. Piping effect is a result from the intergranular flow of water. Internal erosion is a very common cause of embankment failure in hydraulically fractured structures such as cracks and joints (Rajeeth, 2011).

Earth dams have their embankments constructed of soil and rock. Properly constructed earth dams usually have a life span of more than 25 years. However, improperly constructed dams usually fail. A dam failure is a catastrophic type of failure characterized by the sudden, rapid and uncontrolled release of impounded water or the likelihood of such an uncontrolled release. Major causes of failure of earth dams worldwide include construction flaw, seepage/ piping, overtopping and siltation. Dam failure is normally viewed in the context of the risk that is posed to life and property downstream of dams. This is usually so for large dams constructed directly above large population centers. These are capable of causing catastrophic losses. Dam failure can cause loss of life, property damage, cultural and historic losses, environmental losses as well as causing social impacts (Nyoni, 2013.).

2.3.2 Seepage analysis methods

The seepage pattern is the same irrespective of the material (sand, clay, loam) of the dam, though the rate of seepage will depend on soil type. The emergence of seepage lines on the downstream slope tends to make the downstream slope unstable. Either the downstream slope has to be made very flat or the seepage must be diverted away from the downstream slope. There are various methods of Seepage analysis this are;

a) **By Darcy's law -phreatic line analysis**

Phreatic line (seepage line) or saturation line is the line at the upper surface of the seepage flow at which the pressure is atmospheric. The location of the seepage line in earth dam is required for the following purposes: It gives us a divide line between the dry and submerged soil, The soil above the seepage line will be taken as dry and soil below the seepage line shall be taken as submerged for computation of shear strength of soil, It represents the top stream line and hence, helps us in drawing the flow net and The seepage line determination helps us to ensure that it does not cut the downstream face of the dam. This is extremely necessary for preventing softening or sloughing of the dam (Garg, 2005).

Assumptions to be made in seepage analysis:

The rolled embankment and the natural soil foundation of the earth dam are incompressible porous media. The size of the pore spaces do not change with time, regardless of water pressure (Isotropic),The seeping water flows under a hydraulic gradient which is due to only gravity head loss, or Darcy's law for flow through porous medium is valid, There is no change in the degree of saturation in the zone of soil through which the water seeps and the quantity flowing into any element of volume is equal to quantity which flows out in the same length of time (Steady flow) and The hydraulic boundary conditions at entry and exit are known (Garg, 2005).

I. Phreatic line for a homogeneous Earth dam with horizontal Drainage blanket

Figure below shows a homogeneous earth dam with horizontal drainage blanket FK at its toe. The phreatic line in this case coincides with the base parabola ADC except at the entrance. The basic property of the parabola which is utilized for drawing the base parabola is that the distance of any point p from the focus is equal to the distance of the same point from the directrix. Thus;

Distance PF = Distance PR where, PR is the horizontal distance of P from the directrix. EG.

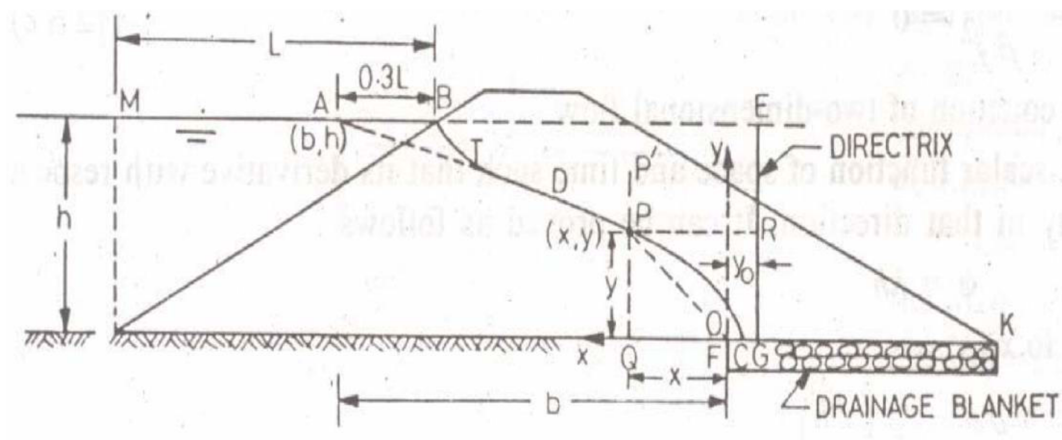

Figure 2-1 Homogenous dam with horizontal drainage

Source: (Garg, 2005).

Graphical method Steps:

Starting point of base parabola is at A; AB = 0.3L

F is the focal point

Draw a curve passing through F; center at A

Draw a vertical line EG which is tangent to the curve

EG is the directrix of the base parabola

Plot the various points P on the parabola in such a way that PF = PR

Analytical method

PF=PR

$\sqrt{x^2+y^2}$= $x+y_{o}$, from point A, x=b and y=h

Y_0=$\sqrt{b^2+h^2}$-b

$\sqrt{x^2+y^2}$=x+$\sqrt{b^2+h^2}$-b equation of parabola………………..……………….. 2.1

Using Darcy's Law

q = k i A……………………...2 .2

q=k $\frac{dy}{dx}$y from parabola equation y=$\sqrt{2xy_o{}^2 + y_o{}^2}$

q=k$\frac{d\sqrt{y_o{}^2+2xy_o}}{dx}\sqrt{y_o{}^2 + 2xy_o}$

q=k ($\frac{y_o}{\sqrt{y_o{}^2+2xy_o}}$) $\sqrt{y_o{}^2 + 2xy_o}$

q=ky$_o$..2.3

Whereas q- seepage quantity y_o-focal distance

K-hydraulic conductivity x-horizontal distance

II. Phreatic line for a homogeneous Earth dam without horizontal Drainage

The analysis for a homogeneous dam without any drainage system and its angle of inclination less than $30^0$is, Casadragde has shown that the phreatic line coincides with the base parabola, provided the slope of the d/s face is flat. Schaffernake and Van Iterson gave an approximate analytical solution for determination of the distance a, the phreatic line cuts the d/s face from the toe, for the slope angle α < 30^0

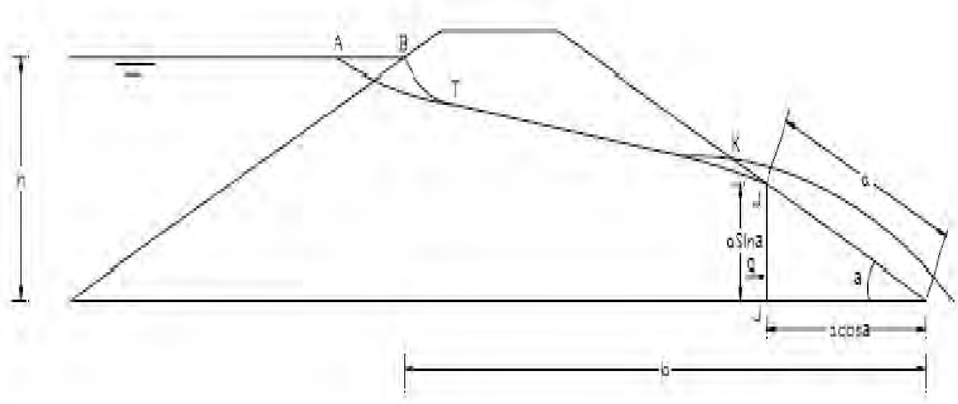

Figure 2-2 Homogenous dam without horizontal drainage for the slope angle α < 30^0

Source: (Hordofa, 2015)

Using Darcy's Law

q=kiA

$i=\frac{dy}{dx}=tan\alpha$ …………………………………………………………………. 2.4

Where i= assumed hydraulic gradient α = angle of inclination at the d/s face

Y, unit area =a sinα k= hydraulic conductivity

From equation (2.2) & (2. 4)

q=k a tan α sin α……………………………………………………...………. 2.5

To find the value of "a" from equation (2.2) and (2.5) atan αsin α dx $=d_y y$, integrating between the limits of x= a cos α and y=a sin α to x=b and y=h

$a=\frac{b}{cos\alpha}-\sqrt{\frac{b}{cos\alpha^2}^2+\frac{h^2}{sin\alpha^2}}$ …………………………………………………..…2. 6

Analytical solution of Casagrande For slope angle $30^0 < \alpha < 60^0$

For steeper slopes, the deviation from correct values increases rapidly beyond tolerable limits. Casagrande suggested the use of sin α instead of tanα. In other words, it should be taken as (dy/ds) instead of (dy/dx), where s is the distance measured along the phreatic line.

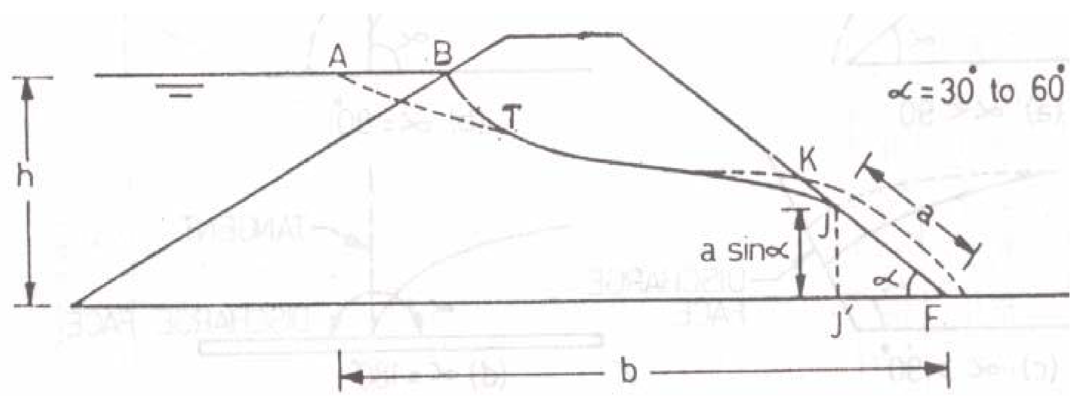

Figure 2-3 Homogenous dam without horizontal drainage for slope angle $30^0 < \alpha < 60^0$

Source: (Anteneh, 2008).

Can be obtained as follows

q=k a $\sin^2$ α……………………………………………………………….…...2.7

a= $\sqrt{b^2+h^2}-\sqrt{b^2-h^2cot\alpha^2}$ ……………………….…………...…..…………2.8

General solution by Casagrande

Figure below shows a homogeneous dam with no horizontal drainage filter at the d/s side. The focus in this case will be the lowest point F of the d/s slope.

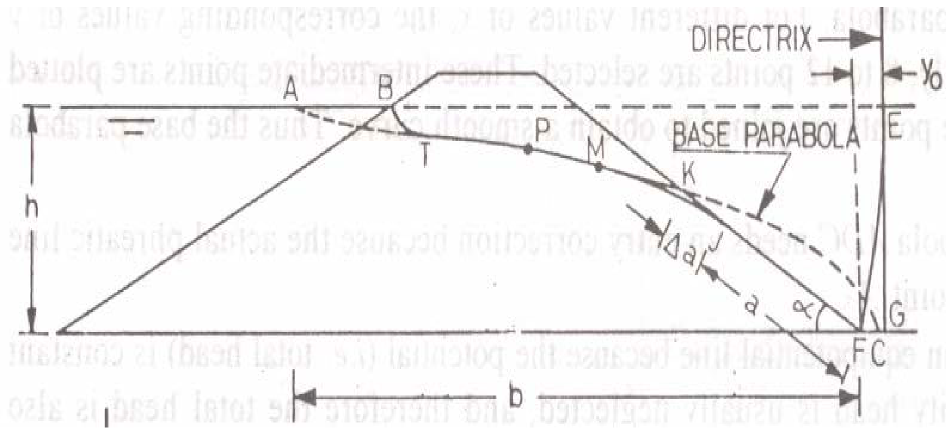

Figure 2-4 Dams without drainage filter.

Source: (Garg, 2005).

The base parabola BKC will evidently cut the d/s slope at K and extend beyond the limits of the dam, as shown by dotted line. However, according to exit conditions, the phreatic line must emerge out at some point M, meeting the d/s face tangentially at J. The portion JF is then known as discharge face and always remains wet. The correction Δa, by which the parabola is to be shifted downwards, is found by the value of $\frac{\Delta\alpha}{\alpha+\Delta\alpha}$ given by Casagrande for various values of the slope α of the discharge face. The slope angle α can even exceed the value of 90^0. Thus we observe that

$\frac{\Delta\alpha}{\alpha+\Delta\alpha}$=Value found from the table

a+ Δa=KF   Solving the above equation the value α and Δa can be found.

Table for the value of $\frac{\Delta\alpha}{\alpha+\Delta\alpha}$ with slope angle α

Table 2-1 Cassagrande value for various inclination angle of discharge face with horizontal

α in degrees	$\frac{\Delta\alpha}{\alpha+\Delta\alpha}$
30	0.36
60	0.32
90	0.26
120	0.18
135	0.14
150	0.10
180	0

Source: (Behailu, 2006).

α is the angle which the discharge face makes with the horizontal. a and Δa can be connected by the general equation;

$$\Delta a = (a + \Delta a)\left[\frac{180^o - \alpha}{400^o}\right]$$

b) **By Flow Net Analysis**

A flow net is a graphical representation of a flow field and comprises a family of flow lines and equipotential lines. The flow net must be drawn by considering appropriate boundary conditions and adhering to characteristics of flow net in order to estimate quantity of seepages. The analysis of seepage by flow nets contributes to the proper design and construction of many dams. The analysis of seepage using flow net starting with drawing a flow net diagram with subjective division of equipotential line and flow line. The Guidelines for drawing flow nets are determine flow conditions at the boundaries, Equipotential and flow lines must meet at right angles and make curvilinear squares, Flow lines should always be perpendicular to a constant head boundary, and equipotential lines are always parallel to it, Provide some guidelines for entrances and exits and particular areas within the flow region, the intersection of two flow lines and two equipotential lines is a square, a circle, tangent to each of the sides, may be inscribed within the square, For calculation of seepage quantity only a crude flow net is required. Accurate flow nets are required to determine pressure distribution (Brown, 1993).

i. For isotropic soils:-If the soil is an isotropic soil; its permissibility is constant in all direction, horizontal permissible is equal to the vertical permissible i.e. Kh = Kv. The amount of seepage through it can also be computed from the flow net analysis. The flow net is drawn by free hand sketching and making suitable adjustment and corrections until to draw the flow line and equipotent line intersect at right angle. The seepage rate (q) can be computed from the flow net.

Using Darcy's law

$\Delta q = kiA$

$=K\,(\frac{\Delta H}{\Delta x})\,\Delta y*1$ (considering unit thickness)

Where ΔH is the energy drop between the two equipotential lines

Δx is horizontal distance between the flow lines

Δy is Vertical distance between the equipotential lines

K is hydraulic conductivity

If N_d = total number of potential drops in the complete flow net,

Then $\Delta H = \frac{H}{Nd}$

$\Delta q = K\,(\frac{H}{Nd})\frac{\Delta y}{\Delta x}$ since $\Delta x = \Delta y$

$=K\,(\frac{H}{Nd})$

Total flow per unit width across each flow channel,

q=ΣΔq*number of flow channels.

$q = K\,(H\frac{Nf}{Nd})$……………………...2.9

ii. For anisotropic soil: - If the soil is non- isotropic the permeability of horizontal direction is not equal to the vertical direction ($kx \neq k_y$) therefore the seepage quantity is estimated using the effective permeability (ḱ). All horizontal dimensions shall be reduced by multiplying them by a factor equal to $k' = \sqrt{khkv}$

$q=\sqrt{khkv}\ (H\frac{Nf}{Nd})$……………………………………………………………… 2.10

c) **By SEEP/W Software Model**

SEEP/W is numerical modeling software which used to solve the practical seepage problems. This is a part of the most popular geotechnical software called GeoStudio. The SEEP/W program is created with the combination of seepage theory and finite element method and working on saturated/unsaturated soil region. 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. The practical seepage problems are never easy to convert into a numerical modeling 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.

i. 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.

ii.Transient analysis: - 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 (Rajeeth, 2011).

2.3.3 Purpose of seepage analysis

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 to estimate the phreatic surface within an embankment, to estimate pore pressures within an embankment or foundation, to estimate exit gradients and/or uplift pressures at the toe of an embankment, to estimate the amount of seepage flow that may pass through an embankment or foundation, to evaluate the relative effectiveness of various seepage reduction measures, to estimate the amount of seepage flows intercepted by drainage features and to size and optimize the configuration of these types of drainage features and to evaluate the effectiveness of, or to aid in the design of dewatering systems (Redda, 2016).

2.3.4 Seepage control in embankment dams

The need for seepage control will depend on the quantity, content and location of the seepage. Reducing the quantity of seepage that occurs after construction is difficult and expensive. Typical methods, using to control the quantity of seepage, are grouting or installation of an upstream blanket. Controlling the content of the seepage or preventing seepage flow from removing soil particles is extremely important. Various methods of seepage reduction and/or control can be used, depending on the requirements for preventing uneconomical loss of water and the likelihood that the foundation will transmit water forces and pressures related to seepage, which can contribute to static instability and cause internal erosion, heave, or blowout. Thereby reducing related water pressures so that static instability, heave, blowout, and internal erosion are adequately controlled in the downstream zones of the foundation (William, 2012).

If seepage is detected on a dam embankment or foundation, it should be closely monitored on a regular basis until it is corrected. If seepage flows increases or embankment soil are showing signs of instability, corrective action should be taken quickly. A qualified geotechnical engineer or dam safety professional should be contacted for inspection and advice for all high dam seepage problems. The reservoir level should be lowered if serious piping or embankment sledding or sloughing is occurring. Sloughing and sliding due to seepage at the two of the embankment may be corrected by removing the unstable soil and constructing a two drain with filter out of permeable soil. Seepage, piping and boils in existing dams may be corrected or slowed by intercepting the water before it exits on the downstream side of the dam.

The blankets may be deployed on the floor of the reservoir to prevent foundation seepage. All cracks and erosion rills on the embankment should be filled, re-graded and re-seeded. Borrowing rodents should be eliminated from dams and any damage created should be repaired by back filling a soil or filtered drain (Omofunmi, *et al*., 2017).

a) Seepage Control through dam embankment

The three methods for seepage control in embankment are:-

i. Use of filters: - Every seepage discharge face, both internal and external, that could be susceptible to piping and heave, must be covered with filters that permit the water to escape freely but hold the particles in place. The filters have two main functions: To prevent internal erosion by blocking migration of soil particles from the base soil and to facilitate internal drainage of seepage flows without built-up of excessive seepage forces and hydrostatic pressures in filters or drains.
ii. Use of impervious Core: - Seepage reduction in embankment dams is done by providing an impervious core at the middle of the section. Most of the energy due to the stored water is consumed by seepage through the core, which is subject to excessive seepage forces in the process. The core is, therefore, used in combination with filter and drainage; the filter protects the core against piping resulting from excessive seepage forces, and the drainage prevents the seepage from entering the downstream shell.
iii. Use of drain**:**-Chimney drains the most effective seepage control measure in earth dams, extending along the d/s face of the impervious core in a zoned dam section, or placed in the heart of a homogeneous dam section, and connected to the d/s drainage blanket for drainage. (Hordofa, 2015).

b) Seepage Control through dam foundations

The foundation and abutment of dams, which are usually stable under the influence of natural ground water flow, may develop a tendency to internal erosion and piping due to the change ground-water regime on reservoir impoundment. The measure for under-seepage control through the foundation include a positive cutoff formed in an excavation up to an impervious stratum and backfilled with compacted impervious material ,concrete cutoffs walls ,grout curtain ,slurry trench cutoff (earth backfilled)

,sheet piles ,u/s impervious blanket ,vertical drains, relief wells and filter trenches. The effective control of seepage requires that the earth embankment, its foundation, and the adjoining structure should behave as one unit. If the foundation of an earth dam consists of an impervious stratum, generally, no specific measures are required to reduce the seepage. However, in rock foundations, grouting and some surface treatment may be required. On the other hand, methods are commonly used to control seepage through pervious, Seepage reducing methods comprise trench cut-offs, u/s impervious blankets, concert diaphragms, slurry trench cut-offs, and grout curtains (Hordofa, 2015).

2.4 Stability of embankment dam

2.4.1 Slope Stability Analysis

Slope Stability analysis is carried out in order to determine the factor of safety of a potential (shear) failure surface. The factor of safety is defined as the ratio of the resisting force or moment to the driving force or moment. The computations for the factor of safety should be based on the most unfavorable condition under which the tests for the determination of the material properties (parameters) are to be carried out. The greatest uncertainties in stability problems arise in the selection of pore pressure and strength parameters. Earth dam stability analysis requires knowledge of appropriate shear strength parameters of the soil comprising the embankment and the foundation. (Tadesse, 2017).

2.4.2 Purpose of Stability Analysis

The slope stability problems have been encountered throughout history, when slopes have been created or disturbed. The design of a foundation must consider slope movement. The need for engineered structures on construction projects continues to increase, as well as the need for advanced analysis methods such as computer modeling, investigative tools, and stabilization methods to solve slope stability problems. Stability problems most often occur when an embankment is built upon soft soils, such as clays with low bearing capacity, silts or organic soils (kiser, *et al*., 2013).

The Purpose of Slope Stability Analysis was to: understand the development and form of natural slopes and the processes responsible for different natural features assess the stability of slopes under short-term (often during construction) and long-term conditions, evaluate the possibility of landslides involving natural or existing engineered slopes, analyze landslides and understand failure mechanisms and influence of environmental factors, to redesign failed slopes and plan for the design of preventive and remedial measures, where necessary and study the effect of seismic loading on slopes and embankments (kiser, *et al*., 2013).

2.4.3 SLOPE/W software

SLOPE/W, developed by GEO-SLOPE International Canada, is used for slope stability analysis. This software is based on the theories and principles of the limit equilibrium methods. In this study, SLOPE/W has been applied separately and together with SEEP/W, other software program, which computes the pore pressure distributions, based on finite elements mesh and groundwater seepage analyses. Finally, the pore pressure distributions were coupled with slope stability analysis and FOS was determined. The software SLOPE/W computes factor of safety for various shear surfaces. The stability of the dry slope was first analyzed in SLOPE/W. The minimum FOS and critical SS searched by entry and exit option. Similarly, a Mohr-Coulomb soil model was chosen, without the feature of tension cracks. A half sine function was selected to compute the inter-slice forces with tolerance error of 1%. Moreover, the selection of a half-sine function was based on the assumption that the inter-slice shear forces could be at maximum in the middle of the CSS and zero at the entry and exit points (Aryal, 2006).

2.4.4 Static Instability Indicator

A need for evaluating the static stability of an existing embankment dam and its foundation is indicated if; - there is an appropriate slope stability failure, there are longitudinal cracks on the dam crest or slopes, there is erosion or sloughing near the downstream toe of the dam resulting in local over steepening of the downstream slope, surface measurement points indicate movements and internal instrumentation indicates movements (Redda, 2016).

2.4.5 Slope Stability Analysis methods

Many numerical analysis methods have evolved in the last six decades for solution of complex engineering problems due to advent of high speed computers. Out of various available numerical techniques, finite difference method (FDM), finite element method (FEM), finite volume method (FVM), boundary element method (BEM) and meshless method have become more popular among scientists and engineers. FEM is applied to very large and complex problems, and it is very important that the solution process remains efficient and economical. From Engineer's point of view, FEM can always be made more efficient and easier to use with sophisticated pre and post processing tools (Athania, et al., 2015).

There are several methods available for circular arc slope stability analysis for embankments built upon soft ground. These techniques can generally be classified into three broad categories e.g., limit equilibrium methods, limit analysis, and finite element methods. Many of the methods for stability analysis fall into the limit equilibrium category. The method of slices is commonly used in limit equilibrium solutions. The soil mass within the slip surface is divided into several slices, and the forces acting on each slice is considered. The limit equilibrium method does not account for load deformation characteristics of the materials, whereas the limit analysis method considers yield criteria. The finite element method is used in more complex problems where earthquake and vibrations are part of the total (kiser, *et al*., 2013).

In the limit equilibrium method, the available shear strength along a potential sliding surface is reduced by a factor of safety so that the mass contained within the sliding surface and the free surface is in a state of equilibrium. The limit equilibrium methods do not determine the displacement within the soil and waste mass. The finite element method gives the stress-strain response of the mass caused by the forces that are imposed on it. This method is more accurate and considers estimation of stresses and deformations. This method has become successful because of the incorporation of representative stress-strain parameters. The stress-strain parameters for waste needed to perform finite element analysis are more difficult to obtain than the strength parameters needed in the limit equilibrium analysis. The three main types of limit equilibrium analysis often used in practice are; the method of slices, wedge method and the infinite slope method. The software which would be used for the limit equilibrium analysis for the slope stability works with the method of slices. (Omari, 2012).

The limit equilibrium is statically indeterminate analysis. As the stress strain relationship along assume surface are not known, so necessary that system becomes statically determinant and it can be analyzed easily using the equation of equilibrium.

The assumptions made in the limit equilibrium are the stress system is assumed to be two-dimensional, it is assumed that the column equation for shear strength is applicable and the strength parameters c and φ are known, it is further assumed that the seepage conditions and water level are known, and the corresponding pore water pressure can be estimated, the condition of plastic failure as assumed to be satisfied along the critical surface in other word shearing strains at all points of the critical surface are large enough to mobilize all the available shear strength, depending upon the method of analysis some additional assumption is made regarding the magnitude and distribution of forces along various planes *(* Salunkhe, *et al.,* 2017).

The limit equilibrium formulation is very useful for understanding what is happening behind the scenes and Understanding the reasons for differences between the various methods. It is not necessarily a method for routine analyses in practice, but it is an effective supplementary method useful for enhancing your confidence in the selection and use of the other more common methods. The importance of the inters-lice force function is related to the slip surface shape. As it turns out, the moment factor of safety is not sensitive to the assumed inter-slice force function when the slip surface is circular. Several methods are available for slope stability calculation (GEO-SLOPE International, 2018).

2.4.5.1 Morgenstern-Price method

Morgenstern and Price (1965) developed a method similar to the Spencer method, but they allowed for various user-specified inter-slice force functions. The inter-slice functions available in SLOPE/W for use with the Morgenstern-Price (M-P) method are: Constant, Half-sine, Clipped-sine, Trapezoidal and Data-point specified.

Selecting the Constant function makes the M-P method identical to the Spencer method.

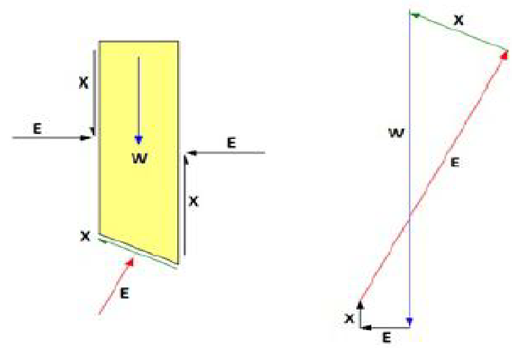

Figure 2-5 Free body and force polygon for Morgenstern-Price method

Source: (GEO-SLOPE International, 2018).

The Morgenstern-Price method:

- Considers both shear and normal inter-slice forces,
- Satisfies both moment and force equilibrium, and
- Allows for a variety of user-selected inter-slice force function

2.4.5.2 Ordinary method

This method is also sometimes referred to as the Swedish method of slices. This is the first method of slices developed and presented in the literature. The simplicity of the method made it possible to compute factors of safety using hand calculations. In this method, all inter-slice forces are ignored. The slice weight is resolved into forces parallel and perpendicular to the slice base. The force perpendicular to the slice base is the base normal force, which is used to compute the available shear strength. The weight component parallel to the slice base is the gravitational driving force. Summation of moments about a point used to describe the trial slip surface is also used to compute the factor of safety. The factor of safety is the total available shear strength along the slip surface divided by the summation of the gravitational driving forces (mobilized shear).The simplest form of the Ordinary factor of safety equation in the absence of any pore-water pressures for a circular slip surface is:

$$FS = \frac{\sum[c\beta + N\tan\phi]}{\sum W\sin\alpha} = \frac{\sum S_{resistance}}{\sum S_{mobilized}}$$ -- 2.11

Where

c = cohesion, β = slice base length

N = base normal (W cos α), φ = friction angle,

W = slice weight, and α = slice base inclination.

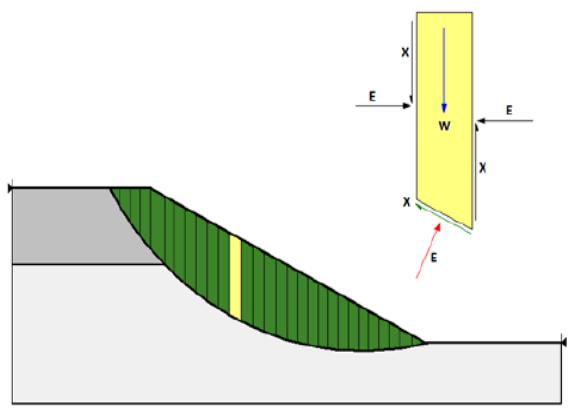

Figure 2-6 A circular slip surfaces and the inter-slice forces with a free body diagram

Source: (GEO-SLOPE International, 2018).

The Ordinary method (OM):

- Satisfies moment equilibrium condition,
- Neglects the inter-slice normal and shear forces,
- Gives the most conservative FOS, and
- Is useful only for demonstrations

Limitation of OM:

- Inaccurate FOS for flat slopes with high pore pressures
- Only for circular slip surfaces
- Assumes that normal force on the base of each slice is Wcosα

- One equation (moment equilibrium of entire mass) one unknown (factor of safety)
- Due to poor force polygon (not closed in free body diagram) it gives unrealistic FOS and consequently should not be used in practice.
- It used for historic reasons and for teaching purpose

2.4.5.3 Bishop's simplified method

Bishop developed an equation for the normal at the slice base by summing slice forces in the vertical direction. The consequence of this is that the base normal becomes a function of the factor of safety. This in turn makes the factor of safety equation nonlinear (that is, FS appears on both sides of the equation) and an iterative procedure is consequently required to compute the factor of safety. A simple form of the Bishop's Simplified factor of safety equation in the absence of any pore-water pressure is:

Bishop's simplified method (BSM)

- Satisfies moment equilibrium for FOS,
- Satisfies vertical force equilibrium for N,
- Considers inter-slice normal force,
- More common in practice, and
- Applies mostly for circular shear surfaces

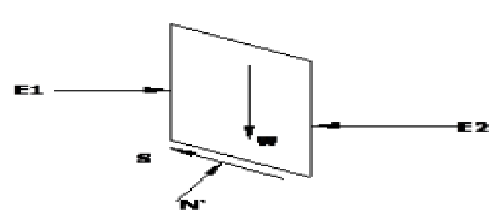

Figure 2-7 Free body diagram for the inter-slices of Bishop's simplified method

Source: (Rajeeth, 2011).

Where N' = normal force

S = shear resistance/shear strength of the soil

E1, E2 = inter-slice normal force

$$FS = \frac{1}{\sum W \sin\alpha} \sum \left[\frac{c\beta + W \tan\phi - \frac{c\beta}{FS} \sin\alpha \tan\phi}{m_\alpha} \right] \quad \text{--------------------------------2.12}$$

$$m_\alpha = \cos\alpha + \frac{\sin\alpha \tan\phi}{FS}$$

To solve for the Bishop's Simplified factor of safety, it is necessary to start with a guess for FS. In SLOPE/W, the initial guess is taken as the Ordinary factor of safety. The initial guess for FS is used to compute *mα* and then a new FS is computed. Next the new FS is used to compute *mα* and then another new FS is computed. The procedure is repeated until the last computed FS is within a specified tolerance of the previous FS. Fortunately, usually it only takes a few iterations to reach a converged solution.

2.4.5.4 Janbu's simplified method

The Janbu's Simplified method is similar to the Bishop's Simplified method except that the Janbu's Simplified method satisfies only overall horizontal force equilibrium, but not overall moment equilibrium. The Janbu's Simplified factor of safety falls on the force equilibrium curve where lambda is zero. Since force equilibrium is sensitive to the assumed inter-slice shear, ignoring the inter-slice shear, as in the Janbu's Simplified method, makes the resulting factor of safety too low for circular slip surfaces. The Janbu's Simplified method considers normal inter-slice forces, but ignores inter-slice shear forces, and satisfies over all horizontal force equilibrium, but not over all moment equilibrium (GEO-SLOPE International, 2018).

Janbu's Simplified method (JSM):

- Satisfies both force equilibriums,
- Does not satisfy moment equilibrium,
- Considers inter-slice normal forces, and
- Is commonly used for composite shear surface

2.4.5.5 Fellenius - jumikis method

The Fellenius - Jumikis method was used in order to obtain a very approximate indication of the location of the most critical slip circle centre in the Earth Embankment Dam. Since the determination of the minimum factor of safety for a slope is very crucial for the design of the Earth Embankment Dam, it is important to locate the most critical slip circle with as few trials as possible. In a random trial and error approach, the three geometric parameters, namely, the centre of rotation, the radius of slip circle and the distance of intercept in front of the toe are varied and the minimum factor of safety obtained. This requires a very large number of trials, but computers have made the method feasible. However, it is known that there is a certain pattern in slip circle behavior and knowledge of this pattern can be used to advantage and the number of trials reduced.

For instance, it is known that the most critical circle passes through the toe of the slope when (a) the angle of shearing resistance φ is greater than 3°, and (b) the slope angle β exceeds 53°, irrespective of the value of φ. The most critical circle intersects the slope in front of the toe if φ is less than 3° and $\beta < 53°$.

Fellenius (1936) proposed an empirical procedure to find the centre of the most critical circle in a $\varphi_u = 0$ soil. The centre O for the toe failure case can be located at the intersection of the two lines drawn from the ends A and B of the slope at angles α and ψ .The angles α and ψ vary with the slope β. (Sachpazi, 2013).

The center of most critical circle may lie anywhere on the line AB or its extension. However, its exact position can be obtained only after conducting the stability analysis for different slip surfaces. The centers of the trial circles are marked as O_1, O_2, etc. on the line AB. The corresponding factors of safety F_1, F_2, etc. are plotted at the corresponding centers as perpendicular ordinates on the line AB. The curve of factor of safety is obtained by joining the ends of these ordinates. The center O corresponding to the minimum factor of safety is the center of the most critical circle

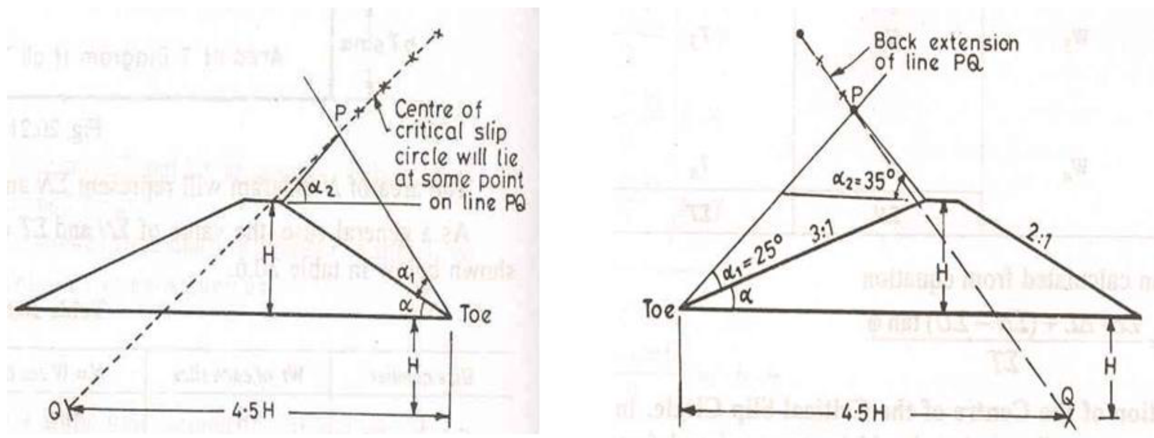

Figure 2-8 Location of center of critical slip circle for upstream and downstream slope

Source: (Behailu, 2006)

Table 2-2 Fellenius's criteria for locating the most critical slip surface

Slope	*Directional angles*	
	α_1 in degrees	*α_2 in degrees*
1:1	27.5	*37*
2:1	25	*35*
3:1	25	*35*
4:1	25	*35*
5:1	*25*	35

Source: (ARORA, 2003)

2.4.6 Standard Loading Condition

Embankment dams during construction and operation are exposed to a variety of stresses that should be designed and implemented safe to withstand these stresses. The basis of limiting equilibrium methods is on determining the imposed stress and mobilized resistance in a hypothetical fractured surface in embankment slope and then determining the safety factor. In examining the stability of embankment dams, forces that cause slope instability include: gravity and leakage. Given that the basic calculations are based on effective stress need to know the pore pressure, therefore, to design three critical steps should be considered (Yazdanian, *et al.*, 2017).

1. End of Construction Loading Condition

End of Construction phase has always higher safety factor than other phases, prior to the start of catchment, due to the reduction of pore water pressure distribution. In this case, the stability of the dam should be examined in the case of effective stress and total stress. For effective stress analyses, pore water pressures must be defined and their values must be specified. For total stress analyses using computer program, hand calculation, or slope stability charts, pore water pressures are defined as zero, actually, the pore pressures are not equal zero. This is essential because of all computer programs for slope stability analyses subtract pore pressure from the total normal stress at the base of the slice. In end of the construction condition, both downstream and upstream slope of the embankment dam is in critical condition. In this case, the materials used in the clay core are undrained and unconsolidated (UU).

2. Steady-State Seepage Loading Condition

After the construction of the dam and passage of the time required, Steady-State seepage condition will be established in dam body and foundation and over time the consolidation will take place in dam body. So the stability of upstream and downstream slope should be analyzed based on effective stress analysis. The basic equation of groundwater motion is obtained as two-dimensional under conditions of saturated and unsaturated flow by combining the Darcy's law and continuity equation. In Steady-State seepage condition, the upstream and downstream of dam is analyzed after the dam catchment and the stability safety factor will be controlled. Steady-state seepage loading condition should be performed using effective stress shear strength parameters joined with measured or estimated embankment and foundation pore pressures.

3. Rapid Drawdown Loading Condition

If the water level behind the dam can be lowered, the seepage in the dam body will change and seepage line will be reversed. That's why the study of upstream slope against rapid Drawdown is important. The reservoir rapid Drawdown can cause instability and incidence slip in the upstream slope which results is remaining the dam materials in saturation and starting the leakage flow towards upstream slope. If the reservoir discharge is done at a rate that in time of drop in water level, the pore water pressure inside the body is not changed and phreatic line remains in its previous position, this process is called reservoir rapid Drawdown. In the rapid Drawdown of reservoirs, sharp reduction of the pore water pressure will be created and with sharp and abrupt decrease in pore pressure, the pressure imposed to the on the dam body will increase and cause rupture in the dam body.

2.4.7 Factor of safety

An analysis of slope stability begins with the hypothesis that the stability of a slope is the result of downward or motivating forces (i.e., gravitational) and resisting (or upward) forces. The resisting forces must be greater than the motivating forces in order for a slope to be stable. The relative stability of a slope (or how stable it is at any given time) is typically conveyed by geotechnical engineers through a factor of safety. The factor of safety is the ratio between the forces/moments resisting movement and the forces/moments motivating movement. This method satisfies both equilibrium conditions. In addition, the inter-slice force relationship is assumed as a linear Mohr-Coulomb expression. The inter-slice forces are adjusted until the Factor of Safety for force and moment equilibrium is satisfied (Salween, *et al.*, 2016). $FS=\Sigma R/\Sigma M$--2.13

Table 2-3 Minimum values of factor of safety as recommended by International Standards

Case No.	**Loading condition**	**Critical Slope**	**Minimum Factor of Safety**
I	End of construction condition	U/S & D/S	1.0
II	Sudden draw down	U/S	1.3
III	Steady seepage of Dam with reservoir full	D/S	1.5
IV	Steady seepage with seismic loading	D/S U/S	1.0 1.0

Source: (Garg, 2005).

2.4.8 Remedial measures of Seepage

Seepage problem with an existing dam has been identified, investigated, analyzed, and that some type of remedial action is deemed necessary. Also assume that the seepage problem is not an imminent threat to the safety of the dam and sufficient time is available to design and construct a permanent remedial measure. Because seepage is often difficult to evaluate, the precise location and extent of remedial control may be difficult to define. Therefore, it is necessary to monitor the "fix" to see if it achieves its objective. In fact, the design of the remedial measure should be flexible to permit change as construction reveals actual conditions, or monitoring indicates the need to supplement the remedy. Remedial action can range from continued or additional monitoring to the extremes of substantially rebuilding or decommissioning and removing the dam. Factors affecting the type of treatment needed include: geological/Geotechnical environment risk**,** amount of correction require, feasibility of correction. The remedial actions described are: monitoring seepage and seepage control measures, lowering the reservoir**,** grouting, cutoff walls**,** upstream impervious blankets**,** downstream berm and drainage (chugh, 2007).

2.4.9 Remedial measures for the static stability of embankment dam

Features which have been used to improve the static stability of embankment dams include: repairing over steepened embankment slopes, buttressing unstable embankment slopes with additional fill, sealing cracks in embankment to prevent rain fall infiltration, sealing the U/S slope with a membrane or other seepage barrier, removing and replacing weak embankment material, adding drainage zones, adding toe drains and rehabilitating existing toe drains (chugh, 2007).

3. MATERIALS AND METHODS

3.1 Description of the Study area

3.1.1 Location and Topography

Arjo-Dedessa dam is found in between East Wollega and Jimma Zones of Oromia Regional State. The project area is located between 8^{o}-30’-00” to 8^{o}- 40’- 00’’ N Latitude and 36^{o}-22’-00” to 36^{o}-43’-00” E Longitude. The catchment is characterized by mountainous, highly rocky and divided topography with deep slopes. The lowest part of the catchment is characterized by valley floor with flat to gentle slopes. Arjo-Dedessa catchment area up to the proposed dam is about 5,632.64 square kilometers and the catchment area up to confluence with Abay River 34,000 square kilometers .There is an irrigable command area of 80,000 hectare on both left and right banks at 40 km downstream of the dam (OWWDSE, 2017).

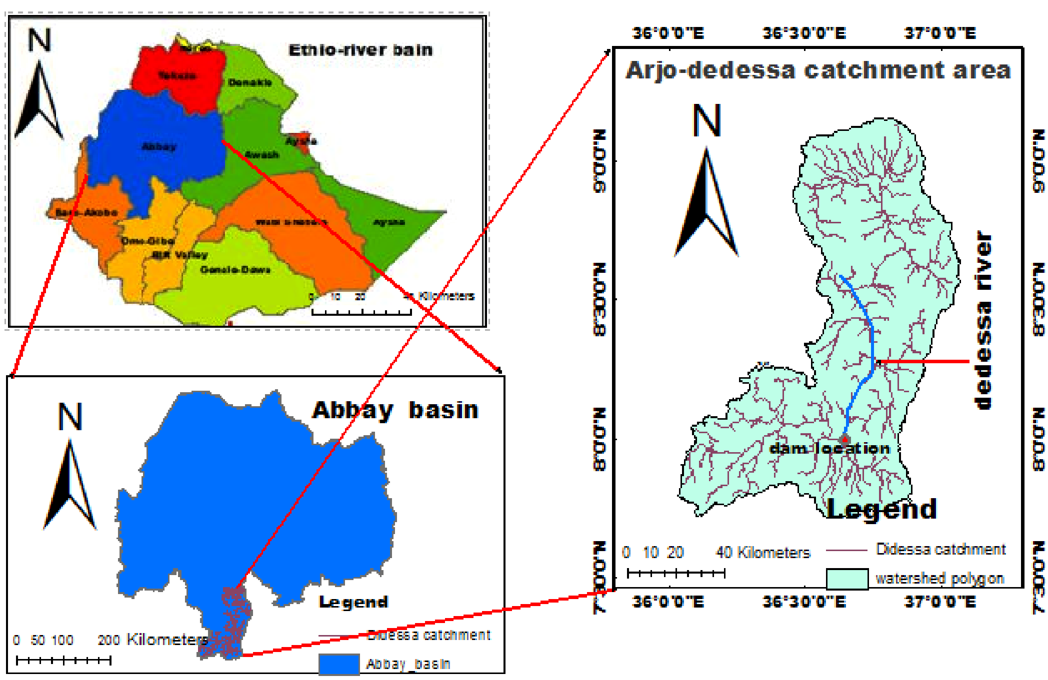

Figure 3-1 Geographical Location of the Study Area

3.1.2 Climate

The Dedessa River is the largest tributary of the Abbay River in terms of the volume of water contribution to the total flow of Abbay at the Sudan border. The major tributaries of the Dedessa River are the Wama, entering from the east, the Dabana from the west, and the Angar from the east. Most of the rainfall in the Dedessa Catchment is concentrated in the June to September period with virtual drought from November through February. The annual rainfall of the area is 2013 mm (Dembi) to 1417 mm (Agaro) and the average annual rainfall is 1453mm. From five meteorological observation stations located in and around the Arjo-Dedessa Irrigation Project area (i.e. Jimma, Bedele, Dedessa, Agaro, and Dembi), station which includes observations of rainfall, temperature, relative humidity, sunshine duration, wind speed, and evaporation for 50 years is taken from which meteorological information relevant to the project area has been derived. In the project area, the mean monthly temperature variations throughout the year are 20.0^0 C in December to 25.4^0 C in March (OWWDSE, 2017).

3.1.3 Geology

The dam axis is located at the foot of land parallel to a trachyte dike overlying a basaltic unit which can act as a natural water barrier, hence minimizing the chance of leakage out of the reservoir. In the downstream course of the Dedessa valley from the dam axis, the river cuts through Precambrian granite gneisses and granites. On the upstream side, up to its head waters north of Gojeb Valley, the river cuts through Tertiary volcanic rocks, belonging to the Limu Genet and Arjo volcanic. The entire volcanic pile has a cumulative thickness of about 1000 m. East and west of Arjo town, the Precambrian gneisses and granites are locally overlain by Paleozoic-Mesozoic sandstones with intercalations of conglomerates, silt stones, etc. (OWWDSE, 2017).

3.2 Tools

The tools used for embankment dam failure analysis focusing on seepage and slope stability are:-

a) Digital camera-to collect the dam body and its appurtenant structural pictures,
b) Tape meter- for measuring length, slope and area of the basin
c) Global positioning Satellite (GPS) – to check coordinates of points.
d) Geo- studio-2012 software-is products of Seep/W and slope/w software models-to analyze both seepage and slope stability
e) AutoCAD-2016-To plot the geometry of the dam
f) Ms. Excel-To compute the points on the phreatic line

3.3 Data collection method

3.3.1 Primary data collection

With visual inspection about the current performance of the dam, Interviewing of the beneficiaries and operator about the past condition of the dam and collect pictures that show the dam body and its appurtenant structures with digital camera.

3.3.2 Secondary data collection

Secondary data's are design documents, Topographic map of the dam site, Manuals, guidelines and standards for the design and analysis of embankment dams which are collected from the Water Resource Development Bureau of Oromia Region. This design documents contain the main hydrological, structural & soil data.

In order to achieve the objective of this study, the main data taken from the design document are the dam profile and property of construction materials and foundation.

a) Dam profile:- the top and bottom width, the height of dam, the u/s and d/s slope, the normal, maximum and minimum water level of the dam, the length horizontal filter, depth of cut-off.
b) Property of material: - the laboratory test result of the core, shell material and foundation (dry, saturated and submersed unit weight, cohesion of soil, angle of internal friction, permeability/ hydraulic conductivity of the soil).

Tests were made on different construction and foundation materials. The test results are analyzed and the stability analysis was performed using these foundation and construction material physical and shear parameters.

Table 3-1 Properties of material to be adopted in design document for the analysis

Zone	Unit	Casing	Core	Foundation	Filter
$K_{.sat}$	cm/sec	$5*10^{-5}$	$1.6*10^{-7}$	10^{-5}	$5*10^{-3}$
γ_{moist}	kN/m^3	21.5	18.0	19.5	-
γ_{sub}	kN/m^3	12.0	8.5	10.0	-
Υ_{sat}	KN/m^3	22	18.5	20	21
c	kPa	0	26	22	3.3
ϕ	Degree	41^o	27^o	30^o	34^o

Source: (OWWDSE, 2017).

Where $K_{.sat}$- saturated hydraulic conductivity; γ_{moist} - moist unit weight

γ_{sub}- submerged unit weigh; Υ_{sat}- saturated unit weight

c- Unit cohesion; ϕ- angle of internal friction

3.4 General description of the dam and Finding out the problems

Arjo-Dedessa dam project comprises of high rock fill with central impervious clay core of length 537 .11m and height of 50 m. The project aims to provide irrigation facility to 80,000 ha of land for sugar cane development. The Ministry of Water, Irrigation and Electricity (MoWIE) is the responsible authority of the project. Oromia Water Works Design & Supervision Enterprise (OWWDSE) is responsible for design, construction supervision and contract administration of the project. The stored water will be diverted into canal system on both the right and left bank for providing water through a network of canal system for irrigation. The embankment dam is zoned type dam with an appurtenance structure of ogee type spillway at the right side of it and stilling basin of 25m length as energy dissipation structure. The volume of water stored at full reservoir level (elevation of FRL=1355.2m) is 2052.96Mm^3. The total reservoir capacity2.5Bm^3 and its catchment area were 12,000 ha.

The main problems were identified through the dam site visiting and by interviewing the operators and observing the design document. As shown in figure3.2, 3.3 and 3.4. The main problems are;-Water was seen to be emerging out of downstream face of the coffer dam close to the diversion conduits and seepage has been physically observed at the berm of downstream face of the dam. But from the design document the seepage analysis was missed for cofferdam. An increase in seepage flow or increased turbidity may be a signal of distress and need to be carefully watched. This result in high reduction of water storage on the upstream and endanger for the dam. In addition to this, the loss of water on the reservoir have an impact on the daily life of the beneficiary's whose life depend on the designed command area. Since the time of field visit was rainy season and the right outlet and the spillway was on construction, we couldn't measure the amount of seepage quantity at the downstream of dam and at the spillway.

At the design document shell material hadn't been well graded. The document stated that, the shell material used for the design was larger than the required size or oversized of shell material. From field observation, at downstream slope side there was the oversized stones of shell material and wrong placement of shell material. Shell material placed in downstream zone had an oversized of stones higher than the required size, Such a practice needs to be discouraged as it leaves the possibility of material of undesired grade inadvertently going into the dam fill at some time creating a wet spot in embankment body. This Wet spots or seepage appearing at new locations downstream from an embankment could also indicate a seepage problem and wrong placement of the stone were occurred at downstream side slope, due to this the stones moving down from the top slopes of the dam. This may leads to Face (slope) failure on the downstream slope of the dam. The gradation of the placed horizontal / coarse filter material laid on d/s side of the main dam was not looking appropriate and particle sizes were over size than the specifications. The ungraded patches were also laid in layers creating weaker zones. So the analysis was done to identify the cause, to determine whether or not an observed or perceived problem in serious and represents an unacceptable risk and, to develop effective remedial/action at reasonable cost.

Figure 3-2 Large size stones in shell material

Figure 3-3 Water seeping out of Coffer Dam

Figure 3-4 Slope face failure at the downstream

3.5 Procedures and Data analysis

In order to accomplish the objective of the research, flow chart prepared for indicating the procedure/ steps and the methods used in the analysis of seepage as shown below

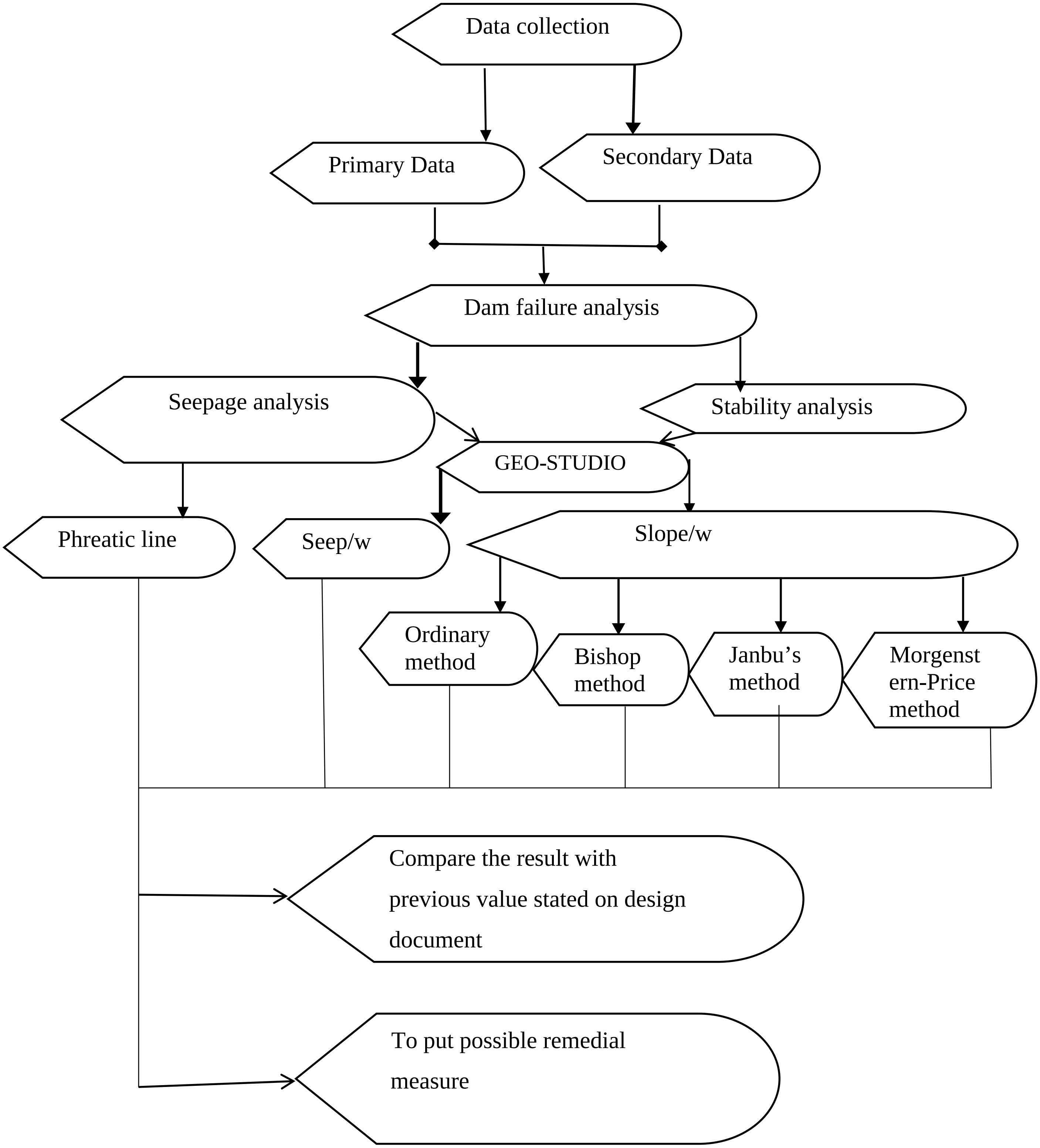

Figure 3-5 Flow chart diagram of the work

3.6 Analysis of Data

Analysis of the Arjo-dedessa embankment dam failure has been carried out by using analytical and numerically by Geo studio software (SEEP/W and SLOPE/W) for both seepage and stability analysis. For the current condition of the dam, the analysis was done using the above methods at full reservoir level and current water level. The full reservoir level of the Arjo-Dedessa reservoir has been fixed by optimizing the 'active storage' capacity of the reservoir so as to meet the irrigation water demand, limiting the chances of failure. The choice to use the different analysis method should be based on complexity of the conditions to be analyzed and the objective of the analysis. Computer programs are rapid methods and provide a means for detailed analysis of seepage and stability.

3.6.1 Analysis of Seepage

3.6.1.1 Analysis by phreatic line

Phreatic line / seepage line / Saturation line is the line at the upper surface of the seepage flow at which the pressure is atmospheric. Seepage line is used to determine the quantity of water passing through the body of the dam and foundation. Now the analysis has been carried out both for Homogeneous and Zoned embankment dams. But actually Arjo-dedessa dam is a zoned dam with horizontal filter. The available data obtained from design document for estimating seepage are geometry of the dam and permeability coefficient of core and shell material.

3.6.1.2 Analysis by Seep/W software model

SEEP/W (Seepage for Windows) is a useful tool that can be used to model the movement and pore-water pressure distribution within porous materials such as soil and rock.

The procedures followed to analyze the problem using this model are:-

i) Importing the geometry of the dam and draw region

The geometry of the dam is the profile of the dam that is, the impervious core and shell material, the dam height, the upstream and downstream slope of the dam, the top and bottom width, the maximum, normal and minimum water level, and horizontal filter length and an inclined chimney drain.

ii) Insert hydraulic conductivity of the material.

The ability of a soil to transport or conduct water under both saturated and unsaturated conditions is reflected by the hydraulic conductivity function. In a saturated soil, all the pore spaces between the solid particles are filled with water. SEEP/W has built-in predictive methods that can be used to estimate the hydraulic conductivity function once the volumetric water content function and a Ksat value have been specified.

iii) Insert boundary condition that influence the seepage, head of water above it, and the location of seepage exit where pressure head will be zero.

Specifying conditions on the boundaries of a problem is one of the key components of a numerical analysis. This is why these types of problems are often referred to as "boundary-valued" problems. Being able to control the conditions on the boundaries is also what makes numerical analyses so powerful. Solutions to numerical problems are a direct response to the boundary conditions. Without boundary conditions it is not possible to obtain a solution. The boundary conditions are, in essence, the driving force.

iv) Locate the fluxes section where the result will be labeled .The key point to note when defining a flux section is to make the flux section go completely through an element if you want the value associated with that element to be included in the flux summation. Flux sections can be used in many ways, because they can be drawn any place across which you want to know the flux.

v) Verify/optimize the data given

Each analysis and input data are verify automatically before solving, and any errors are reported on the verification tools. Verify tool checks for errors in the overlapping geometry lines, input data and region that do not have any data points.

vi) If the data have no error, solve the problem. The final step is solving the problem and analyzing the results and make conclusions. Seepage analyses are often conducted for three major applications: calculating flow rates, gathering hydraulic gradient data for determining factors of safety against piping and to be used as a parent analysis for a slope stability analysis.

3.6.2 Analysis of slope stability

Stability analysis of selected section of dam embankment has been carried out by using software SLOPE/W developed by GEO-SLOPE International Ltd. The software uses the general limit equilibrium (GLE) formulation for estimating the factor of safety in stability of upstream and downstream slopes for various loading conditions. The procedure of slope stability will be listed below.

i. Defining the geometry

In defining a slope stability problem, it is convenient to first prepare a sketch of the problem dimensions. This sketch is a useful guide for drawing the geometric elements of the problem.

ii. Specify the analysis methods

In case of Arjo-Dedessa dam, slope stability was analyzed by different approaches like Ordinary, Bishop, Janbu and Morgenstern's Price. From design document FOS derived by Morgenstern's Price method has been adopted.

iii. Specify slip surface analysis options

Determining the shape and position of the critical slip surface is a crucial step in executing a slope stability analysis. In SLOPE/W, the critical slip surface is located through the calculation of trial slip surfaces that can be more or less controlled by the user. The software computes the factor of safety for the trial slip surfaces where upon the slip surface generating the minimum factor of safety is considered as the critical. The Analysis Options - for the study, two options used for determining the position of slip surface i.e, entry-exit and grid and radius for Fellenius method.

iv. Define Soil Properties

Mohr-Coulomb strength parameters C (cohesion) and Ø (phi) are available to describe the property of material in terms of soil strength. The parameters may be total or effective depending on the pore-water pressure conditions specified. Un-drained strengths are specified by making Ø zero.

v. Solve and analyze result

The part of an analysis is to use the SLOPE/W solve function to compute the factors of safety. The Slope/W software computes the shear stress of slip surface corresponding to shear strength to determine the factor of safety. The analysis result shows the critical slip surface and the minimum factor of safety.

4. RESULT AND DISCUSSION

4.1 Analysis of seepage

Analysis has been carried out to assess the embankment dam failure with highlighting on seepage and slope stability problem of the dam.

4.1.1 Darcy's-law phreatic line

Although Arjo-dedessa embankment dam is zoned type dam; the analysis has been used to estimate the expected quantity of seepage for homogeneous and zoned dam with drainage system. The phreatic line and the total estimated seepage through the dam body was done on Ms Excels as shown on the appendix B and C. The phreatic line for both homogeneous and zoned dam at full reservoir level was drawn as shown in figure 4.1 and 4.2.

The coordinate points on the phreatic line were done on Ms Excels worksheet as shown on the appendix B and C.

i. Homogeneous Dam

Homogeneous dam analysis has been carried with provision of drainage system. The salient features of the dam and its appurtenant structures are described on the appendix A. The Permeability of the shell materials Kshell=$5*10^{-5}$ cm/sec.

The equation of parabola can be written as:

$\sqrt{x^2+y^2}= x+y_o$ for x=b and y=h

$y_{0}=\sqrt{b^2+h^2}-b$

h= full reservoir level-river bed level

h=1355.2-1312=43.2m

The distance between A&B=0.3L=0.3*147.3=44.19m

b=0.3L+17+10+21.15=92.34m

At point A, b=92.34m and h=43.2m

$Y_{0}=\sqrt{b^2+h^2}-b$

$$Y_0 = \sqrt{92.34^2 + 43.2^2} - 92.34 = 9.6m$$

The amount of seepage per unit length, q

$q = ky_0 = 5*10^{-5} *10^{-2}* 9.6$

$q = 4.8*10^{-6}\ m^3/sec/m$

Full length (L) =537.11m

The total seepage (Q) $=4.8*10^{-6}\ m^3/sec/m *537.11m=2.6*10^{-3}m^3/sec$

The coordinate points on the phreatic lines are;

$$y = \sqrt{2xyo + yo^2}$$

$$y = \sqrt{2 * 92.34 * 9.6 + 9.6^2} = 43.12m$$

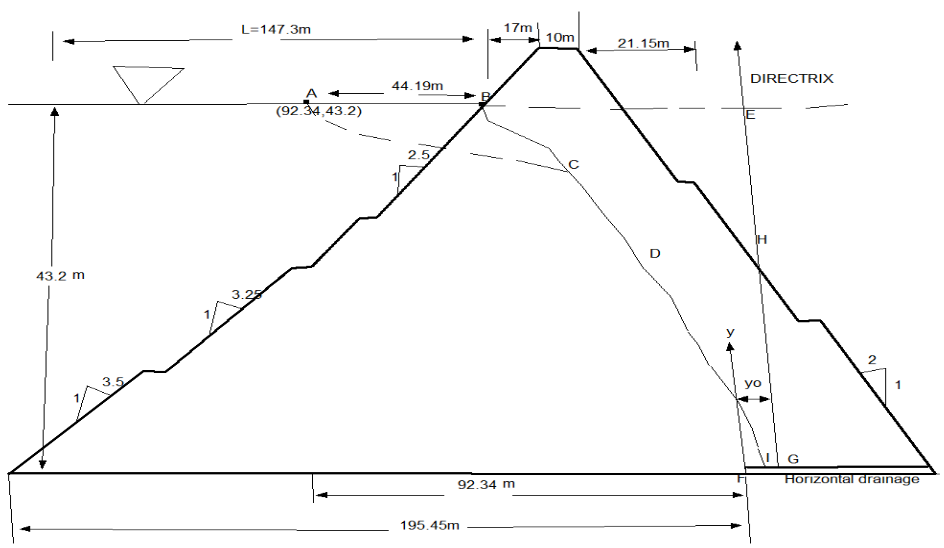

Figure 4-1 Phreatic line at FRL for homogeneous dam

ii. Zoned Earth Dam

The most common type of an earth dam usually adopted in the zoned earth dam as it leads to an economic & more stable design of the dam. In a zoned earth dam, there is a central impervious core which is flanked by zones of more pervious material. In case of zoned dam analysis, the impact of permeability of shell material with the core material should be checked first. And if the ratio of permeability of shell material with core material (Ks/Kc) is greater than 20, the effect of shell material on core is negligible (Anteneh, 2008).Permeability coefficient which was obtained from the design document for:

-Shell material, ks = $5*10^{-5}$ cm/sec

- Core material, kc= 1.6*10-7cm/sec

The ratio of (ks/ kc) = $3.13*10^{2}$*>>>20

From the above result, the effect of shell material on the core is neglected. For zoned dam, the presence of a chimney drain and horizontal filter drain within the seepage model, the phreatic surface changed as compared to the analysis was done for homogeneous. The core material is highly plastic clay with permeability of $1.6*10^{-7}$ cm/sec.

L-cassagrande solution for slope angle $30^0 < \alpha < 60^0$

For the core material b=25.94+35.94=61.88m and h=43.2m

For $\alpha=59^0$ at b=61.88m, and h=43.2m

$$a = \sqrt{b^2 + h^2} - \sqrt{b^2 - h^2 cot\alpha^2}$$

$$a = \sqrt{61.88^2 + 43.2^2} - \sqrt{61.88^2 - 43.2^2 cot59^2}$$

a=75.47-35.9=39.6

$q = k\ a\ \sin^2 \alpha$

$q=1.6*10^{-7} *10^{-2}*39.6* \sin^2 59$

$q= 4.7*10^{-8}\ m^3/sec/m$

Full length of (L) =537.11m

The total seepage (Q) = $4.7*10^{-8}\ m^3$/sec/m*537.11m =$2.52*10^{-5} m^3$/sec

The coordinate points on the phreatic lines were obtained as shown on appendix C.

$Y_0 = \sqrt{b^2 + h^2} - b$ for b=61.88m and h=43.2m

$Y_0 = \sqrt{61.88^2 + 43.2^2} - 61.88 = 13.6$m

$y = \sqrt{2xyo + yo^2}$

$y = \sqrt{2 * 61.88 * 13.6 + 13.6^2} = 43.2$m

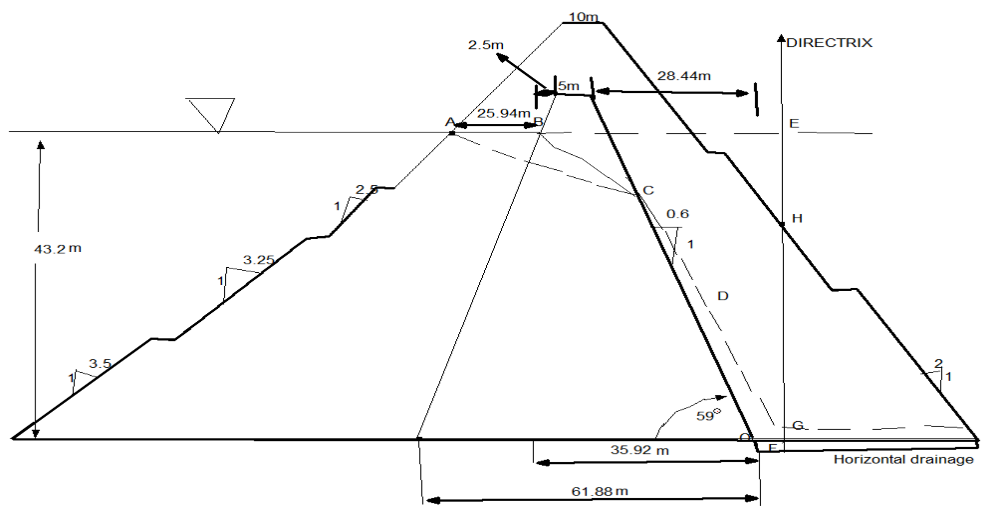

Figure 4-2 Phreatic line at FRL for zoned dam

4.1.2 Analysis by Seep/W software model

For homogeneous dam, the result of the analysis using SEEP/W software model for the case of homogeneous dam is presented below. The analysis considered the dam with filter drainage system and neglected the impact of foundation seepage. Using SEEP/W software model requires the sets of parameters like; model section of the dam, permeability coefficient of material, the piezometer reading, boundary conditions and saturated water content. Its permeability coefficient has a value of $5*10^{-5}$cm/sec. As shown from the figure 4.3 that has been estimated.

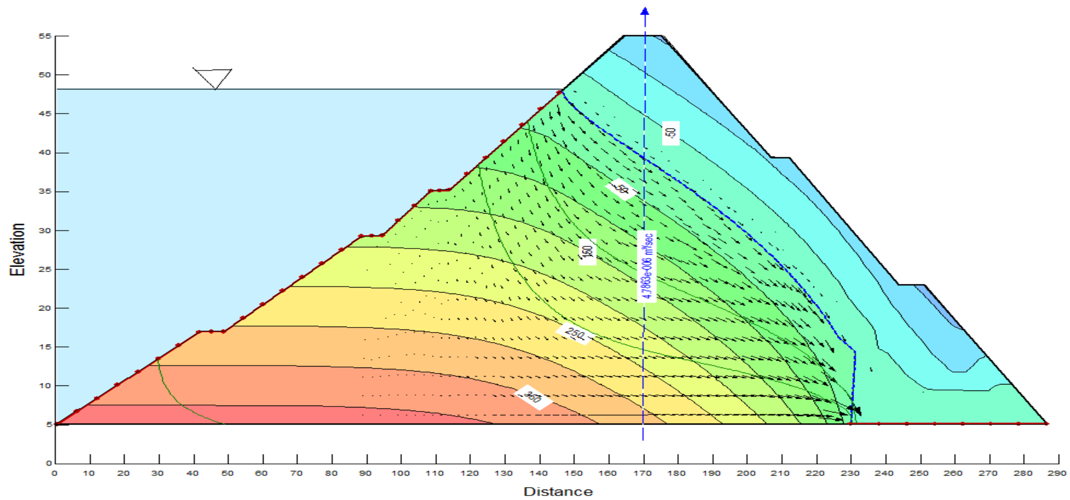

Figure 4-3 Seepage through the homogeneous dam with horizontal filter

Zoned Earth Dam -Contains materials of different kinds in different parts of the embankment. In a zoned earth dam, there is a central impervious core which is flanked by zones of more pervious material. The analysis of zoned dam using SEEP/W software model is differ with the other method used above. In this case the dam is analyzed with filter drainage system and foundation seepage .However, the analysis is considered the impact of shell material with the core, chimney filter and foundation.

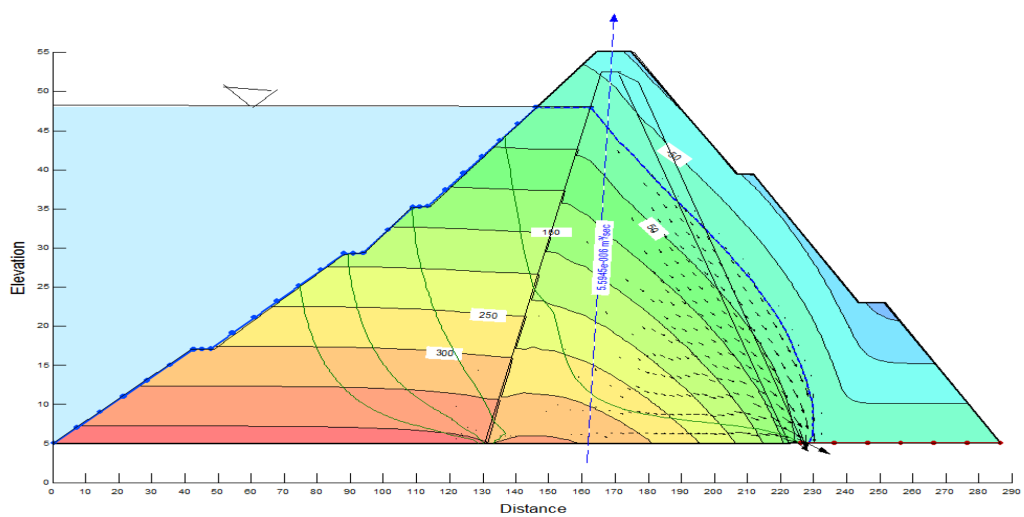

Figure 4-4 Seepage through zoned dam with chimney and horizontal filter

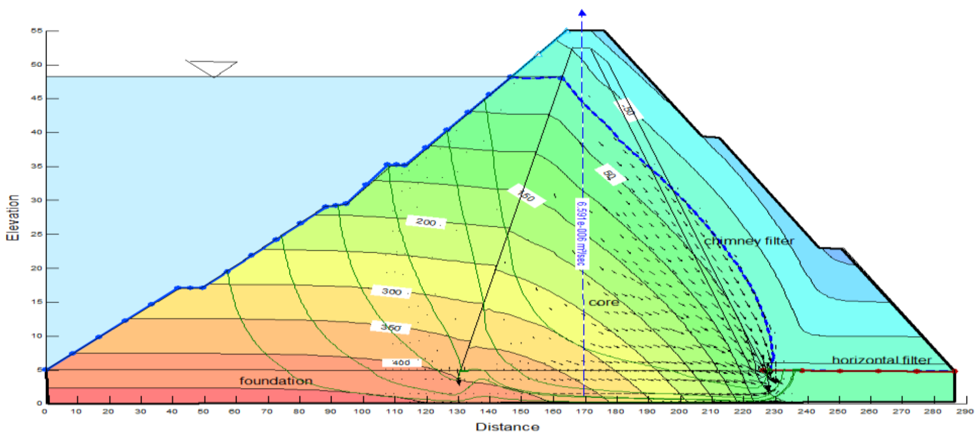

Figure 4-5 Seepage through zoned dam with foundation at FRL

To compare the computed result of seepage analysis with the previous study, the result for both analytical and numerical methods was shown.

Table 4-1 Result of seepage computed at Full reservoir level

Methods	Homogeneous dam with horizontal filter (m^3/sec)	zoned dam with chimney and horizontal filter (m^3/sec)	Zoned dam with foundation Including with chimney and horizontal filter (m^3/sec)
Analytical by phreatic line	$2.6*10^{-3}$	$2.52*10^{-5}$	-
SEEP/W	$4.8*10^{-6}$	$5.6*10^{-6}$	$6.6*10^{-6}$
At design document	-	-	$4.16*10^{-5}$

As shown from the above table 4-1, the expected quantity of seepage estimated with these different methods. The study result show that the maximum seepage passing through the dam body calculated using Seep/W software for homogeneous dam with horizontal filter is $4.8*10^{-6}m^3$ /s. However, Arjo-dedessa dam was a zoned dam. In this study, the maximum value of seepage between zoned dam with horizontal filter including foundation($6.6*10^{-6}$ m^3 /s) should be compared with the value stated at the design document($4.16*10^{-5}$ m^3/s) through the dam as per the SEEP/W software model. This result shows that the value of seepage estimated in the design document is higher than the calculated value of seepage. This means the design document estimate safe amount of seepage passing through the dam body and foundation. Therefore, the design document has no problem of quantifying the expected quantity of seepage through the dam body and foundation.

During the current condition evaluation, the value of seepage evaluated using Seep/W software at current elevation 1332 m or at a water depth of 20 m shown in figure 4.6.

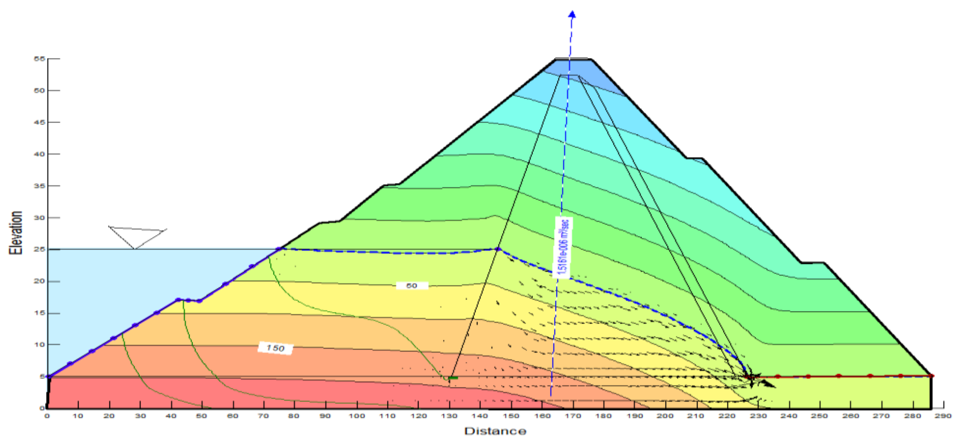

Figure 4-6 Seepage through zoned dam with foundation at current water level

The two scenarios at different water level conditions where full reservoir level and current water level. It can be noticed that the quantity and velocity of seepage during full reservoir level ($6.6*10^{-6}$ m^3/sec/m)greater than in case of current water level ($1.52*10^{-6}$$m^3$/se/m) which indicate that the quantity and velocity of seepage increase with increasing height of water level in the upstream of the dam. This result shows that the value of seepage estimated in the design document ($4.16*10^{-5}$$m^3$/sec/m) is higher than the computed value of seepage at both full reservoir level and current water level. Therefore, the design document has no problem of quantifying the expected quantity of seepage through the dam body and foundation.

4.2 Slope stability analysis

The determination of factor of safety for the dam slope stability, under different cases of operations, is vital to ascertain the dam overall safety.The considered analytical methods in this study are Ordinary, Bishop's Simplified, and Janbu's Simplified and Morgenstern-Price methods. Many of the methods for stability analysis fall into the limit equilibrium category. The method of slices is commonly used in limit equilibrium solutions. The soil mass within the slip surface is divided into several slices, and the forces acting on each slice is considered. The limit equilibrium method does not account for load deformation characteristics of the material. For static stability analysis, Stability analysis of selected section of dam embankment has been carried out by using software SLOPE/W developed by GEO-SLOPE International Ltd. The software uses the general limit equilibrium (GLE) formulation for estimating the factor of safety in stability of upstream and downstream slopes for various loading condition. From the design document factor of safety derived the Morgenstern-price method has been adopted. In this document the value of factor of safety analyzed using various methods but only the Morgenstern –price method was shown as below. The other methods were listed on the appendix D. To investigate the stability of dam slopes, the dam was simulated, using Geostudio software, three different cases of operation are considered, as follows: after construction, Steady-state seepage and Rapid drawdown.the parameters used as input datas are saturated unit weight, unit cohesion and angle of internal friction. Factor safety of Arjo-Dedessa dam has been checked for the following loading conditions:

a) At end of construction:-At end of construction condition represents a situation when the dam is just constructed and the pore pressures developed as a result of dam material compression due to the overlying fill are not dissipated or are only partly dissipated. The residual pore water pressures depend on the moisture content and the compaction efforts imparted during construction as well as on the rate of rise of the dam. For stability analysis of Arjo-Dedessa dam under this condition, the residual pore water pressures under this condition are considered. The 'end of construction' condition can be critical for either of upstream and downstream slopes. Analysis has been carried out to determine factor of safety (FOS) for both upstream and downstream slopes.

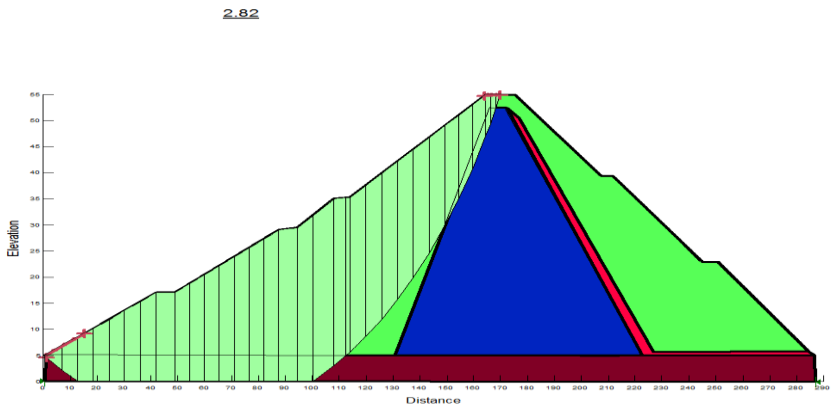

Figure 4-7 the minimum FOS for Upstream at the end of construction

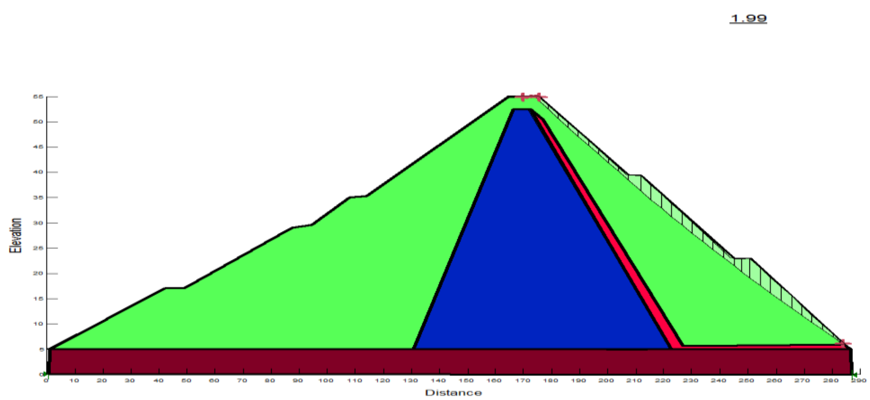

Figure 4-8 the minimum FOS for Downstream at the end of construction

From figure 4.7 and 4.8, the minimum factor of safety calculated for end of construction at upstream (FOS=2.82) and at downstream (FOS=1.99) compared with the minimum acceptable safety factor provided by international standards (FOS=1).The result shows that the dam was stable at the end of construction

b) Steady Seepage state

This condition is developed when the water level is maintained at a constant level for sufficiently long time establishing seepage lines in the earth dam. Stability analysis of Arjo-Dedessa dam for steady seepage state has been carried out by considering head pond level at FRL and no water at tail of the dam. The stability of downstream slope under steady state has been checked without consideration of seismic loading.

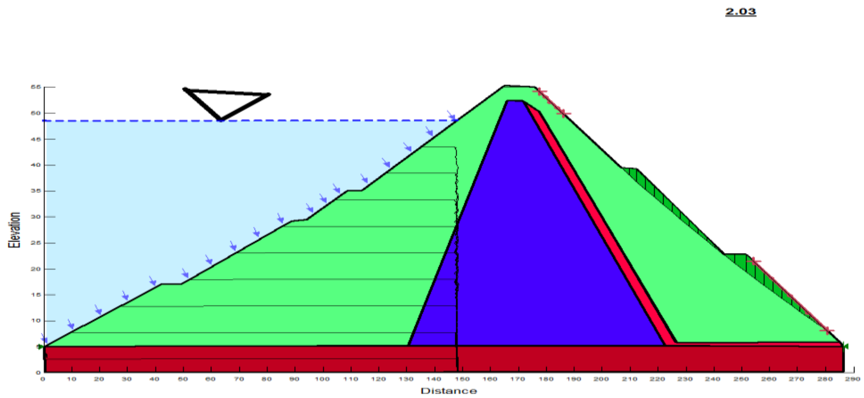

Figure 4-9 the minimum FOS for steady seepage state of dam with FRL

c) Sudden Drawdown

In projects that incorporate an outlet of sufficiently large discharging capacity, either for irrigation releases downstream or for emergency depletion of reservoir, there may be a sudden drop in reservoir water level after a condition of steady state seepage is established in the embankment and foundations. Such rapid drawdown of the reservoir water surface does not allow the pore water pressures in the embankment fill to dissipate at the same rate as the fall in reservoir water surface. This condition then becomes critical for stability of upstream slope.

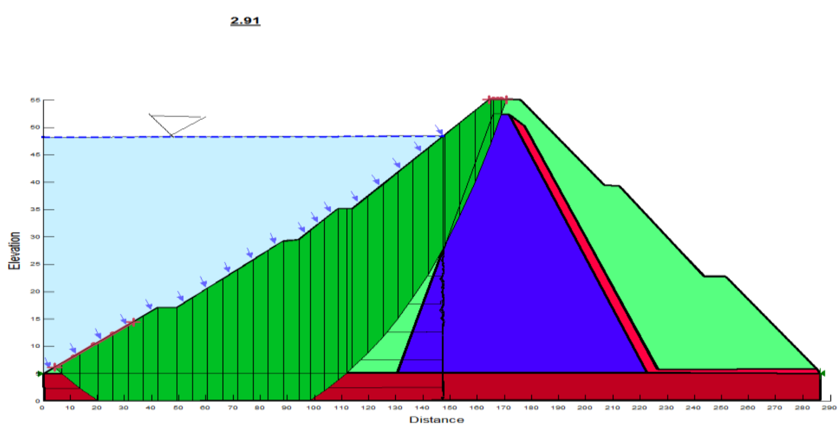

Figure 4-10 the minimum FOS for Rapid draw down of the dam with FRL

For assessing the previous result, the static slope stability analysis by Geo studio product of SLOPE/W for the different limit equilibrium method using entry and exit trial slip surface at standard loading condition is summarized as shown in table 4-2

Table 4-2 Computed minimum FOS of at the previous study of slope stability at FRL

Analytical methods	Minimum factor of safety			
	End of construction		Rapid drawdown	Steady–state seepage
	U/S	D/S	U/S	D/S
Morgenstern-Price	**2.82**	**1.99**	**2.91**	**2.03**
Bishop’s Simplified	2.83	1.99	2.95	2.00
Janbu’s Simplified	2.45	1.74	2.68	1.74
Ordinary	2.48	1.79	2.60	1.79

Table 4-3 the minimum FOS for different Loading conditions at design document

Loading Condition	Critical For	Ordinary	Bishop	Janbu	Morgenstern's Price
End of construction	U/s	1.234	1.433	1.316	**1.416**
End of construction	D/s	1.587	1.59	1.587	**1.589**
Steady seepage state	D/s	2.018	2.021	2.018	**2.020**
Rapid drawdown	U/s	1.874	2.398	2.16	**2.346**

The results of stability analysis show that the derived values of factor of safety in each case are higher than the required minimum values of factor of safety recommended by International standards in the literature review in table2.3. For stability analysis of embankment dams, the recommended factors of safety will vary with loading conditions. The factor of safety for a short term (end of construction) and a long term operation (steady state and rapid draw down) should be greater than the minimum acceptable value stated by International standards.

Table 4-4 Comparison of the minimum FOS for different loading conditions

No	Loading condition	Critical Slope	Minimum Factor of Safety		
			Computed at the previous study(Morgenstern -price method)	International standards	At design document(by Morgenstern-price method)
I	End of construction	U/S	2.82	1.0	1.416
		D/S	1.99	1.0	1.589
II	Sudden draw down	U/S	2.9	1.3	2.346
III	Steady seepage reservoir full	D/S	2.03	1.5	2.020

From Table4-4, the result shows that the calculated minimum factor of safety when compared with the minimum acceptable safety factor provided by International standards and the previous result in the design document, the dam was safe for all loading conditions. From the field observation the downstream slope failures occur at the side slope of the dam. But At the design document installation of the Soil Strain Meter is not proposed for Arjo-dedessa dam. This Instrument, designed to measure axial deformations in soil as might occur at the time of slope failures in dam embankments, may be useful at the slope sections which are as indicated potentially prone to landslides with a low factor of safety by the slope stability analysis.

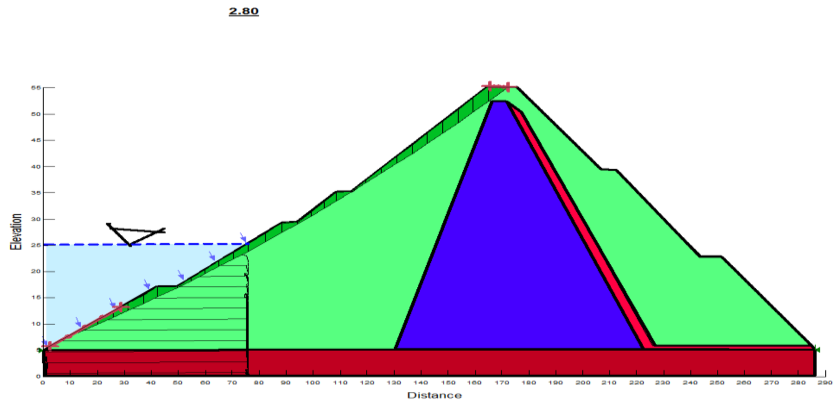

Figure 4-11 the minimum FOS for Rapid draw down of the dam with CWL

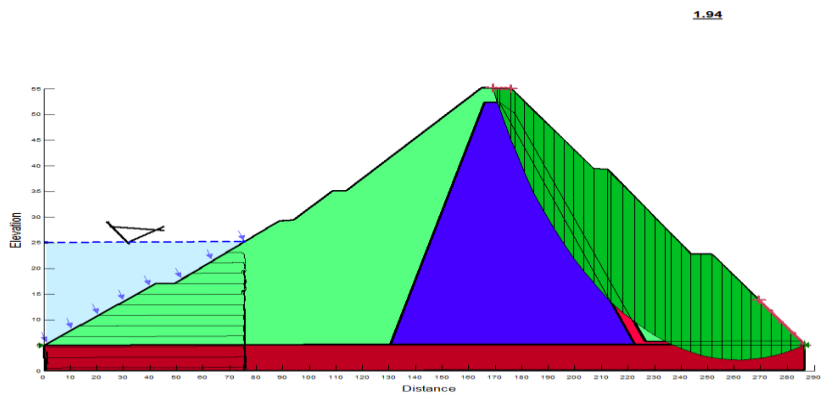

Figure 4-12 the minimum FOS for seepage steady state of the dam at CWL

Table 4-5 Computed minimum FOS of at the current study of static slope stability at CWL

Loading Condition	Critical For	Ordinary	Bishop	Janbu	Morgenstern's Price
Steady seepage state	D/s	1.78	1.93	1.73	**1.94**
Rapid drawdown	U/s	2.48	2.80	2.46	**2.80**

For the stability of upstream slope under sudden draw down condition the derived factor of safety is dependent upon the rate of reservoir draw down and the rate of dissipation of pore water pressures. The analysis of current water level was done to check the stability of a zoned earth dam to examine during rapid draw down under different water levels in the reservoir, the insufficient stability may occur in the upstream slope as soon as the water is lower than the drawdown level of 1/3 of the dam height. The results of stability analysis show that the derived values of factor of safety for rapid draw down in each case are higher than the required minimum values of factor of safety recommended by Indian or International standards and stated in design document. In case of Arjo-Dedessa dam, the reservoir draw down can take place only by flow through irrigation outlets. The discharging capacity of irrigation outlet being limited, the drawdown of the reservoir would be at a very slow rate which is not likely to be critical for Arjo-Dedessa dam. Therefore the dam was safe in this loading condition.

4.3 Remedial measures of the case study

The seepage and stability analysis was done using the available data from the design document. From the field observation shows seepage has been physically observed at the berm of downstream face of the dam and Water was seen to be emerging out of downstream face of the coffer dam close to the diversion conduits and also the downstream side slope failure was observed. In order to reduce the risk of embankment failure the remedial measure has been taken as below.

4.3.1 Remedial measures for seepage

During field visit shows seepage has been physically observed at the berm of downstream face of the dam and Water was seen to be emerging out of downstream face of the coffer dam close to the diversion conduits. Upstream coffer dam, that is to form an integral a part of main dam, has already been constructed embodying these diversion conduits. Seepage water is observed to be coming out of the downstream face of the cofferdam from a point located just above the embedded diversion conduit. Besides this concentrated seepage, water was seen to be coming out of cofferdam body at many other points, though in small quantities. The seepage flow from the body of the cofferdam is likely to increase considerably when the dam is completed and water stands at full reservoir level. The seeping water from point near diversion conduit was observed to be turbid. Such an increase in seepage flow or increased turbidity may be a signal of distress. The field visit was conducted during rainy season to conform whether the water at the berm is from seepage or not. The

Farmers and Development Agents (DA) of the area said that, about the general situation of the dam the water at the berm is not only rain water and further explained the problem occurred at downstream face of the dam close to the diversion conduits during dry season even when the water level reached at the normal pool level of the reservoir. The water seeping throughout the year that has an impact on their life, Therefore, it has fundamental importance to limit these problems related with seepage through it.

1 Grouting at the contact between the embankment and the abutment. The contact zone can be treated by cement grout with suitable admixture
2 Reasonable thickness of impervious blanket of appropriate length is placed over the soft seepage areas at the downstream face of the dam as one of the remedial measures. This adds weight and provides a working platform for installation of relief wells at points of excessive seepage.
3 Berm control seepage is made by increasing the weight of the top stratum so that the weight of the berm plus top stratum is sufficient to resist uplift pressure and the water will not rise to the berm. Again, a seepage analysis must be made to determine the resisting load required of the berm. Downstream slope stability of the embankment will normally increase because of the resistance to sliding provided by the berm.

4.3.2 Remedial measure for slope failure

1 Removing the oversized material around the failure part and proper filling and compacting with same materials /well graded gravel or shell material at the top and downstream slope failure part of the embankment dam.
2 Berms will improve the stability of an embankment dam, a one possible means to prevent such stability problems of this dam is to raise and re-construct a stabilizing berm of coarse material along the d/s toe of the dam.
3 The downstream slope should be protected against the erosive action of rain and its runoff. So, cover the exposed area with grass and regular maintenance need on the downstream slope of the dam.

5. CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The main objective was to assess the Arjo-Dedessa embankment dam failure, focusing on seepage and slope stability analysis and finally to come up with the possible remedial measure. Analysis of seepage quantity was done using seepage analysis methods by Darcy's phreatic line for zoned dam and seep/w software model analysis for zoned dam with chimney drain and horizontal filter including foundation compared with the value stated at the design document. The result shows that the value of seepage estimated in the design document is higher than the computed value of seepage at both full reservoir level and current water level. Then the design document has no problem of quantifying the expected quantity of seepage through the dam body and foundation. But during field visit seepage has been physically observed at the berm of downstream face of the dam and Water was seen to be emerging out of downstream face of the coffer dam close to the diversion conduits. An increase in seepage flow or increased turbidity may be a signal of distress and need to be carefully watched. This result in high reduction of water storage on the upstream and endanger for the dam. The causes for excessive seepage are problem on laboratory test and result, under compaction, wrong placement filter materials in different zones, if not properly done. The existence of one or more problems, lead to a great loss of reservoir water moving through the dam body and foundation into the downstream, which result in dam failure by seepage

Slope/W software was used to calculate the factor of safety under the standard loading conditions for limit equilibrium methods(Morgenstern-Price, Bishop's, Janbu's and Ordinary method) using entry and exit trial slip surface and compared with the international standards and stated in design document. The result shows the dam was safe under this loading condition. However, from the field observation, the downstream side slope was failed. The causes of instability of the side slope in the downstream are the use of oversized stones of shell material and wrong placement of stones in the downstream of the dam.

5.2 Recommendation

The analysis was carried out to assess seepage and slope stability using the available data from the observed and design document. Therefore the following recommendations should be under consideration:

Identification on the source seepage failure needs an extensive re- analysis with help of frequent field visit and at change on upstream water level, the study area has exposed for such failure, therefore, for further detail investigation the amount of seepage should be measured at downstream of the dam and Installing dam monitoring instruments, frequent instrument reading and making observation use to provide data and information which can be used to assess the performance of the dam under normal and extreme loading condition and to manage the risk associated with operation and maintenance.

The Arjo-Dedessa dam was highly exposed to soil erosion at the slope surface problem due to the absence of soil conservation structures on the areas. In order to prevent soil erosion at the slope surface, the vegetation and trees on the existing slopes must be preserved and Surface water must be directed away from the slope or carried down the slope in suitable conduits.

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APPENDICES

Appendix A- the salient features of the dam and its appurtenant structures are described below.

Dam body

- Embankment level =1362m, Max water level =1358.9m
- Full reservoir level =1355.2m, River bed level =1312m
- Top width =10m, Crest length =537.11m

Upstream (U/S)

- Slope =1:2.5 from El 1362 to El 1342m and from El 1342 to 1336.4m
- Slope 1:3.25 from El 1336.4 to El 1324m
- Slope 1:3.5 from 1324 to river bed
- Berm length=6m at El 1342,1336.4 & El1324m(U/S)

Downstream (D/S)

- Slope =1:2 from El 1362 to 1346m, from El 1346 to 1330m and from El 1330 to river bed
- Berm length =6m at El 1346m & El 1330m(D/S)

Spillway (Un-gated)

- Type of Spillway-Un-gated Ogee type
- Probable Maximum Flood (PMF) at dam=4580 m3/sec
- Crest Level of Ogee at EL 1355.2m
- Design Flood :421 m^3/sec
- Total width of spillway crest =86 m

Property of dam material

Core material	Shell material
Type= clay soil	Type= Graded rock fill
Specific gravity=2.7	Specific gravity=2.7
Natural moisture content=16%	Optimum moisture content=15.4%
Optimum moisture content=37%	Porosity (Ƞ) =0.51
Porosity (Ƞ) =0.5	

Appendix B- the coordinate points on the phreatic lines for homogeneous dam

X	Y
0	9.6
2.5	11.83892
5	13.71714
7.5	15.3675
12.5	18.22526
15	19.49769
17.5	20.69203
20	21.82109
22.5	22.89454
25	23.91987
27.5	24.90301
30	25.84879
32.5	26.76117
35	27.64344
37.5	28.49842
40	29.32848
42.5	30.13569
45	30.92184
47.5	31.68848
50	32.43702
52.5	33.16866
55	33.88451
57.5	34.58555
60	35.27265
62.5	35.94663
67.5	37.25802
70	37.8967
72.5	38.5248
75	39.14282
77.5	39.75123
80	40.35046
82.5	40.94093
85	41.52301
87.5	42.09703
90	42.66333
92.34	43.18666

Appendix C-the coordinate points on the phreatic lines for zoned dam

x	Y
0	13.6
2.5	15.90472
5	17.91536
7.5	19.72207
10	21.37662
12.5	22.91201
15	24.35077
17.5	25.70914
20	26.99926
22.5	28.23048
25	29.4102
27.5	30.54439
30	31.63795
32.5	32.69495
37.5	34.71253
40	35.67856
42.5	36.61912
45	37.53612
47.5	38.43124
50	39.30598
52.5	40.16167
55	40.99951
57.5	41.82057
60	42.62581
61.88	43.22148

Appendix D-SLOPE/W result by other methods

1) At end of construction:

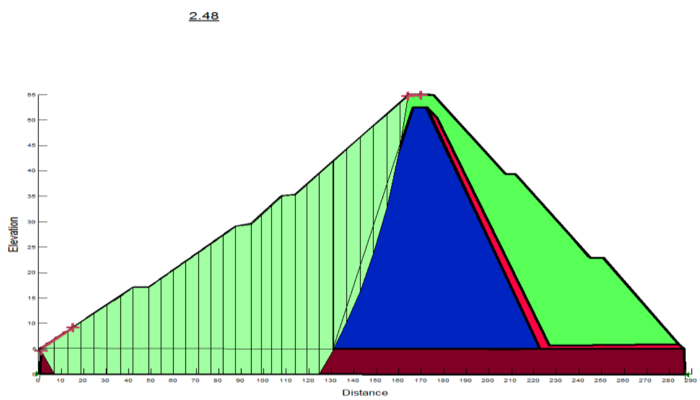

Figure 1 -the minimum FOS for Upstream using Ordinary method

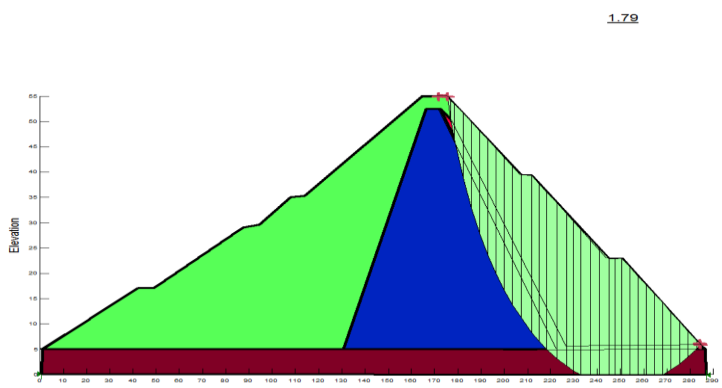

Figure 2-the minimum FOS for downstream using Ordinary method

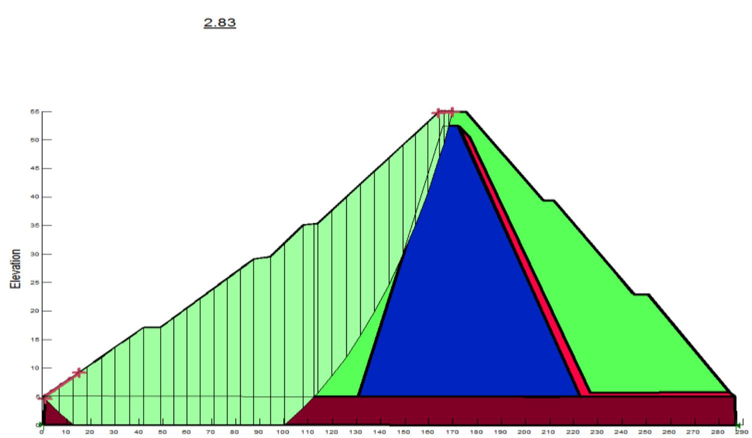

Figure 3-the minimum FOS for Upstream using Bishop method

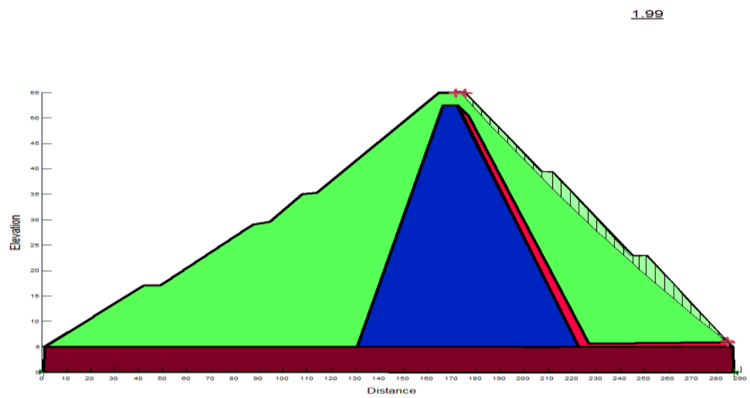

Figure 4-the minimum FOS for downstream using Bishop method

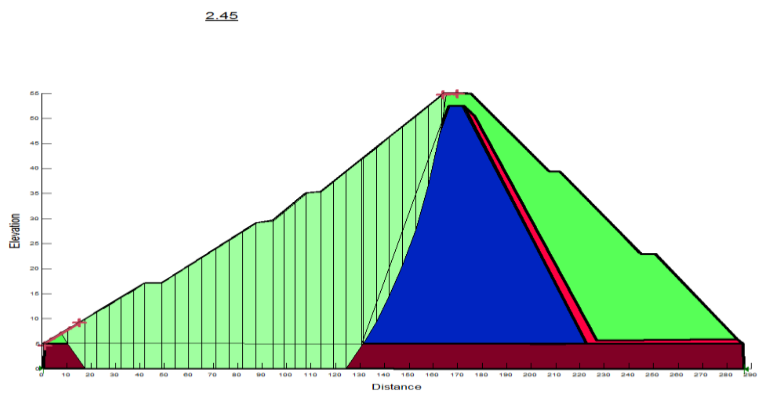

Figure 5-the minimum FOS for Upstream using Janbu's method

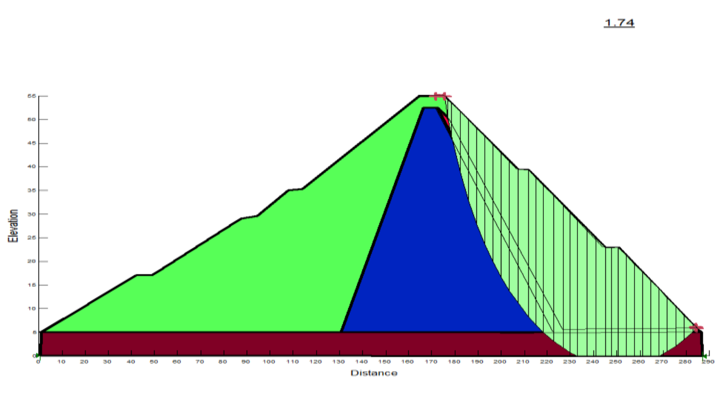

Figure 6- the minimum FOS for downstream using Janbu's method

2) At steady seepage state

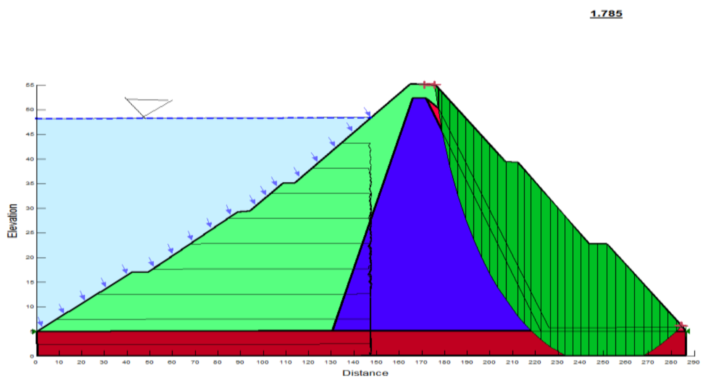

Figure 7- the minimum FOS for downstream using Ordinary method

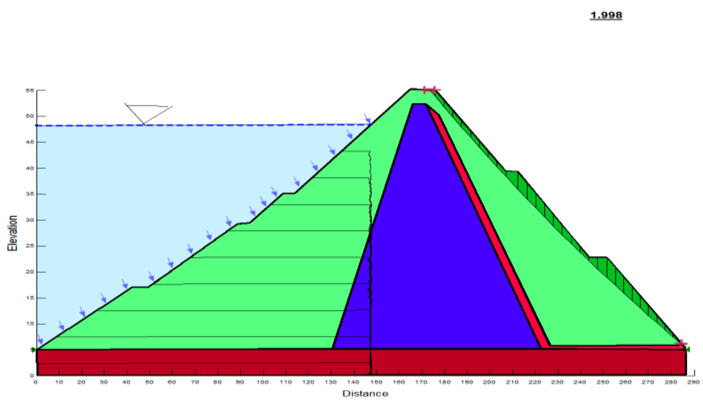

Figure 8-the minimum FOS for downstream using Bishop Method

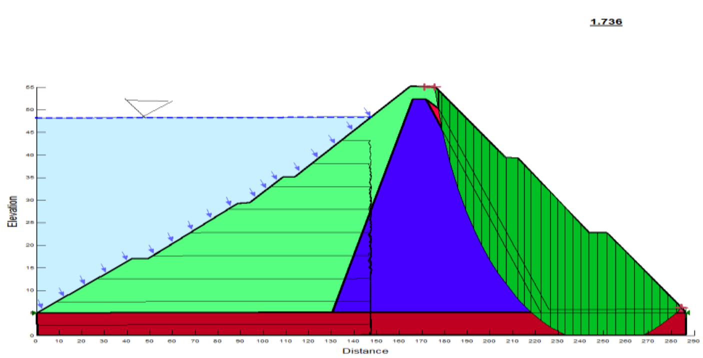

Figure 9-the minimum FOS for downstream using Janbu's method

3) Sudden drawdown

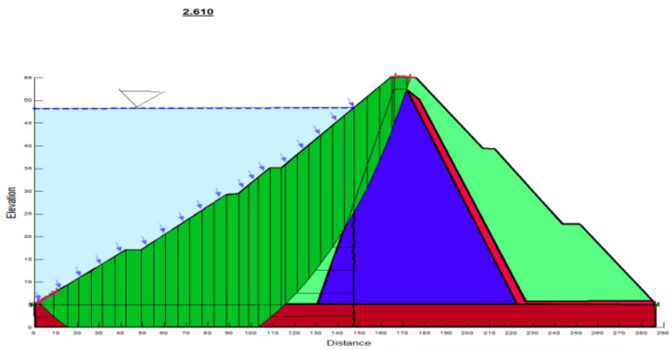

Figure 10-the minimum FOS for Upstream using Ordinary method

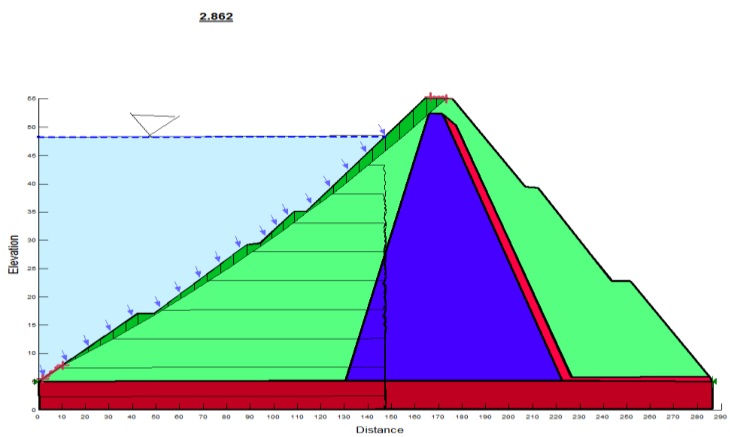

Figure 11-the minimum FOS for Upstream using Bishop Method

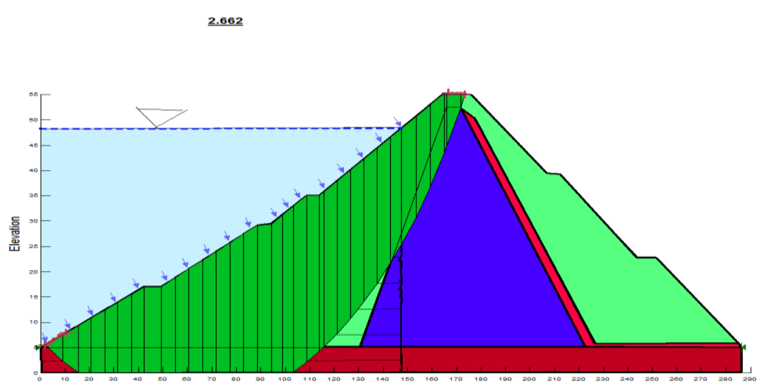

Figure 12-the minimum FOS for upstream using Janbu's method

1) The minimum FOS for rapid drawdown at CWL:

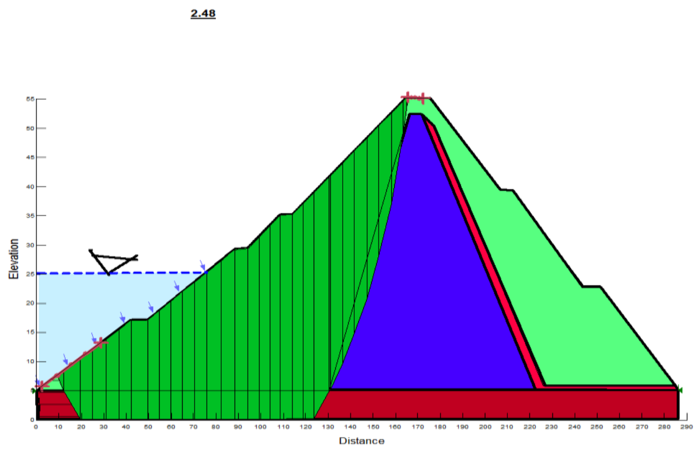

Figure 13- the minimum FOS for rapid drawdown using Ordinary method

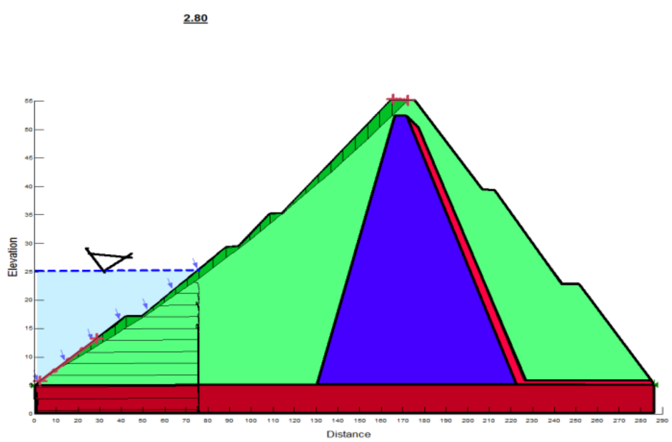

Figure 14-the minimum FOS for rapid drawdown using Bishop Method

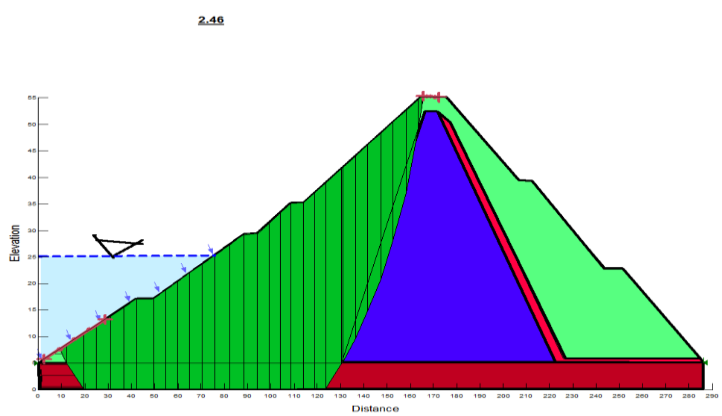

Figure 15-the minimum FOS for rapid drawdown using Janbu's method

2) The minimum FOS for steady seepage state at CWL:

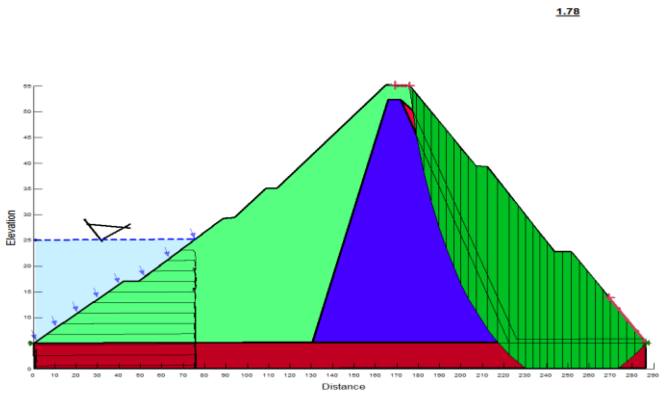

Figure 16-the minimum FOS for steady seepage state using Ordinary method

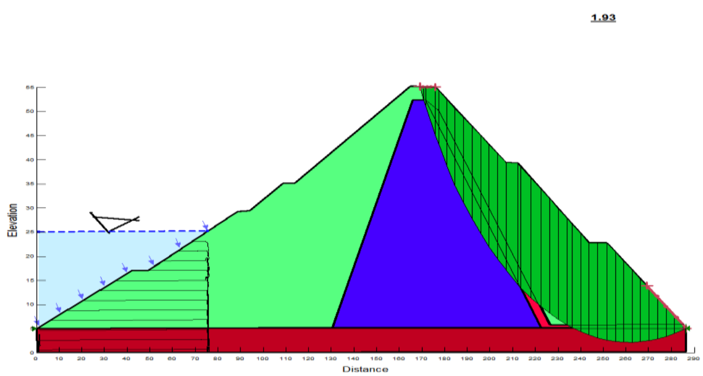

Figure 17-the minimum FOS for steady seepage state using Bishop Method

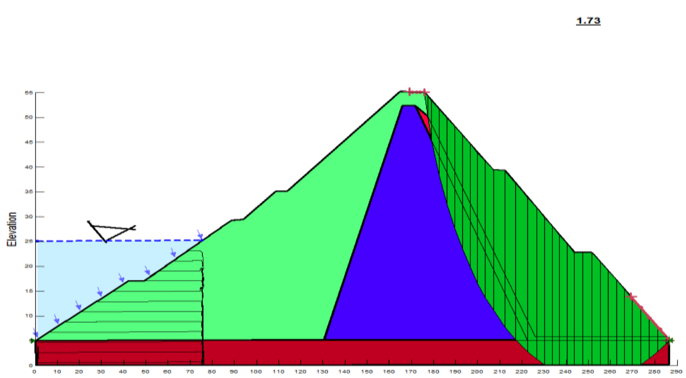

Figure 18-the minimum FOS for steady seepage state using Janbu's method