EVALUATION OF IMPACT OF INTERNAL EROSION ON THE STABILITY OF DAM – THE CASE OF ARJO DHIDHESSA EMBANKMENT DAM PROJECT OROMIA REGION, ETHIOPIA

BY: - KETEMA TESFAYE ADUGNA

DEPARTMENT OF HYDRAULIC AND WATER RESOURCES

ENGINEERING

HAWASSA UNIVERSITY INSTITUTE OF TECHNOLOGY

OCTOBER 2019

HAWASSA ETHIOPIA

IMPACT OF INTERNAL EROSION ON THE STABILITY OF DAM – THE CASE OF ARJO DHIDHESSA EMBANKMENT

DAM PROJECT

BY: -KETEMA TESFAYE ADUGNA

ADVISOR: - MOLTOT ZEWDIE (PHD)

CO-ADVISOR: SHEMELIES ASSEFFA (PHD)

MSc. THESIS SUBMITTED TO DEPARTMENT OF HYDRAULIC AND WATER

RESOURCE ENGINEERING.

HAWASSA UNIVERSITY

IN PARTIAL FULFILLMENT OF THE REQUIREMENT FOR THE DEGREE OF MASTER OF SCIENCE IN DAM ENGINEERING

OCTOBER 2019

HAWASSA ETHIOPIA

SHOOL OF GRADUATE STUDIES

HAWASSA UNIVERSITY

ADVISORS' APPROVAL SHEET

This is to certify that the thesis entitled "**Impact of internal erosion on the stability of dam – the case of Arjo Dhidhessa embankment dam project**" is submitted in partial fulfillment of the requirements for the degree of Masters of science with specialization in Dam Engineering, the Graduate Program of the Department of Hydraulic and water resource Engineering, and it has been carried out by Ketema Tesfaye Adugna Id. No PGDEng 004/10 under our supervision. Therefore, we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

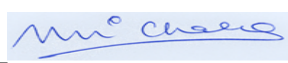
Moltot Zewdie (PhD)	-----------------	----------------
Major Advisor	Signature	Date
Shemelies Asseffa (PhD)	-----------------	---------------
Co- Advisor	Signature	Date

APPROVAL SHEET

DEPARTMENT OF GRADUATE COMMITTEE

HAWASSA UNIVERSITY

As members of the Examining Board of the Final MSc Open defense, We certify that we have read and evaluated the thesis prepared by Ketema Tesfaye entitled as "**Impact of internal erosion on the stability of dam – the case of Arjo Dhidhessa embankment dam project**" and we recommend that it can be accepted as fulfilling the thesis requirement for the degree of masters of science in Hydraulic and water resource Engineering (specialization –Dam Engineering)

Mulgeta Dadi (PhD) ______________	____________________	_________________
Name of Chairman	Signature	Date
Moltot Zewdie (PhD)	_________________	_________________
Major Advisor	Signature	Date
Alemu Osore (PhD)	_________________	_________________
Name of internal Examiner	Signature	Date
Michael Mehari (PhD)_______________	__	_________________
Name of external Examiner	Signature	Date
_______________________	__________________	_______________
SGS Approval	Signature	Date

Final approval and acceptance of the thesis is contingent upon the submission of the final copy of the thesis to the school of graduate's studies (SGS) through the school of graduate committee (SGC) of the candidate's department/school

DECLARATION

I declare that this thesis is my effort work and all sources of materials used for this thesis have been duly acknowledged. I formally declare that this thesis is not submitted to any other institution anywhere for the award of any academic degree, diploma or certificate.

Name: *Ketema Tesfaye Adugna*

Signature ------------------------

Date ------------------------------

Place: Hawassa University

ACKNOWLEDGEMENT

First and foremost, praises and thanks to the God, the Almighty, for His showers of blessings throughout my work to complete the research successfully.

I would like to express my deepest and sincere gratitude to my research Advisors Dr. Moltot Zewdie for his close supervision and guidance throughout this research. Besides my advisor, I would like to thank my co-advisor Dr. Shimelies Asseffa for his encouragement and insightful comments.

I am very much thankful to my wife Samrawit G/Tsadkan and my daughters Asanti and Amen for their love, understanding, prayers and continuing support to complete this research work. Also, I express my thanks to my mom, Gete Amensisa for her valuable prayers.

I would also like to express my appreciation to Hawassa University Institute of Technology staff members and libraries for their provision of the necessary reference books and assistance for my study.

I am very much respectful and honest for Oromiya Water Works Design & Supervision Enterprise (OWWDSE) and Oromiya Water work Construction Enterprise (OWWCE) for providing me all necessary datum.

Finally, my thanks go to all the people who have supported me to complete the research work directly or indirectly.

ABBREVIATION

DSL	Dead Storage Level
EL	Elevation
EMA	Ethiopian Map Agency
FRL	Full Reservoir Level
FEMA	Federal Emergency Management Agency
FEM	Finite Element Modeling
FAO	Food and Agriculture Organization
FS	Factor of Safety
ICOLD	International Commissioning for Large Dams
LIA	Limit Equilibrium Analysis
MWL	Maximum Water Level
MOL	Minimum Operation Level
MDDL	Minimum Drawdown Level)
MoWIE	Ministry of Water, Irrigation and Electric
OWWDSE	Oromiya water work Design & Supervision
OWWCE	Oromiya Water work Construction Enterprise
USACE	United States Army Corps of Engineered
USSD	United States Society on Dams
USDIBR	United State Departments of the Interior Bureau of Reclamation

Table of Contents Page

LIST OF FIGURES

LIST OF TABLES

LIST OF FIGURES IN THE APPENDIX

LIST OF TABLES IN THE APPENDIX

ABSTRACT

Embankment dams encounter several problems in terms of dam safety. One of those problems is called internal erosion. This phenomenon is brought by the movement of fine particles within the dam due to seepage forces. Internal erosion represents a complex combination of several mechanisms related to the failure or near failure of dams and levees. If the dam is not able to self-heal, the eroded zones will increase which will eventually cause the dam to fail. Specially if the discontinuity such us concrete conduit is there in the dam embankment the probability of concentrated flow occurrence through the embankment body is increased. The dam selected for this study Arjo Dhidhessa rock fill dam is currently under construction by MoWIE, Due to regied structure embedded in the body of the embankment dam it is related to the problem of internal erosion within the core. The impact of this internal erosion is analyzed in this thesis with the use of Finite Element Method/Analysis (FEM/A). FEA models simulate the in-situ stresses in the dam and calculate the strength. It also enables the analysis of changing hydraulic conductivity and its effect on the overall effective strength due to changing pore pressure and seepage forces. The analysis using numerical methods was performed in the program PLAXIS2D and SEEP/W while limit equilibrium analysis was done in SLOPE/W. The calculation in PLAXIS2D was performed by using the Mohr-Coulomb constitutive model. The in-situ stresses are initially calculated using gravity loading since this is the preferred method on an uneven terrain instead of a K0-calculation. Then, through a set of phases in the program, zones where erosion is assumed to have occurred are changed. These zones have a higher permeability and will thus affect the pore pressures in the dam following Darcy's law with permeability through a set medium. The increased permeability is set to follow an increased void ratio due to loss of fine material in the core. How this increase of void ratio affects the permeability is investigated through using Ren et al. (2016) proposed equation for calculating permeability with a set void ratio. Their equation, apart from the usually used Kozeny-Carman equation, considers both effective and ineffective void ratio where the ineffective void ratios refers to the volume of pores that is immobile when flow is considered. Conduits through embankment dams are prone to seepage and internal erosion around the surrounding soil. The increased flow in the eroded zones of the core did not seem to impact the strength of the dam in much regard. The phreatic surface and thus the pore pressure did not change enough to influence the overall effective strength of the dam. It raises the question if the stability of an earth-rock fill dam will be affected due to increased pore pressure at all due to its draining properties and if it would rather fail due to increased seepage forces. Throughout design and construction of rockfill dam as much as possible fixing the conduit out of the embankment dam is recommendable to avoid the probability of concentrated flow which potential cause internal piping

Key words: *Seepage, Slope, Internal-erosion, Piping, Arjo-Dhidhessa, Ethiopia*

1 INTRODUCTION

1.1 Background

Water is a crucial entry point for transforming the livelihood and other development activities throughout the entire history of human civilization. The level of agricultural transformation and its sustainability highly depends on how water resources of a country are managed and utilized. Existing literature witness that the national gross domestic product (GDP) of Ethiopia is correlated with the adequacy of seasonal rainfall. Among various uses of water, the largest use of water in the world is for irrigating lands (Garg, 2005). M

Construction of a dam plays a vital role in development of one countrys. Therefore, confidence on safety of earth dams during their construction, impoundment, and operation is one of the most important reasons for monitoring pore water pressure within the dam. Other reasons, which are important to researchers, are better understanding of seepage flow patterns through the dam, dam's deformation, and strength (Kutzner, 1997).

Earth and rock fill embankment dams are structure most commonly used to impound water. For these dams most sites are relatively good and ,since everything from clay to large stones is used in their construction, it requieres low capital investment and can be completed in short period of time with minimum environment impact. Some problems like seepage failure and slope stability are significantly affected by pore water pressure in embankment dams. Usually the designers utilize a two dimensional seepage analysis in a typical cross section of embankment dam (Serafi, and Santos, 1985).

As FEMA (2005) stated conduits convey water from a reservoir through, under, or around an embankment dam in a controlled manner. Conduits through embankment dams serve a variety of purposes. Conduits typically convey releases stored waters to meet downstream requirements, Providing emergency reservoir evacuation, Flood control regulation to release waters temporarily stored in flood control space. Diverting flow into canals or pipelines, Satisfying a combination of multipurpose requirements and Stream diversion during construction. However, conduits represent a discontinuity through an embankment dam and its foundation. This discontinuity can cause settlement to be different adjacent to the conduit than it is in the rest of the embankment dam. Earthfill may also be compacted differently around a conduit than for the rest of the embankment dam. Because it is difficult to compact the core around the conduit by huge mashineris. These factors can cause cracking of the earthfill and lead to other consequences.

According to FAO (2003) stated Failures of embankment dams caused by the uncontrolled flow of water through the dam or foundation are a common problem. A conduit can develop defects from deterioration, cracking from foundation compressibility, or joint separation due to poor design and construction. Water leaking from defects in conduits can contribute to seepage pressures exceeding those that occur solely from flow through soils in the embankment dam from the reservoir. When preferential flow paths develop in the earthfill through which water can flow and erode the fill, severe problems or breaching type failures often result. (FEMA, 2005).

The current study is carried out to find the seepage, slope stability and displacement analysis of rockfill embankment dam using Geo-studio 2012 and PLAXIS 8.2 comercial software. Most of literature shows that seepage models for seepage analysis of embankment dam are more efficient than the other manually calculated methods. Comparing observed data and the estimated data through Geo-studio models, it has been proved that the Geo-studio models show good results and are better than the manually calculated. Plaxis is a finite element package that has been developed specificall for analysis of deformetion, stability and flow in geotechnical engineering projects.

1.2 Statement of Problem

Different studies indicate that due to different reasons the seepage/ leakage occurred in the core of the dam. When preferential flow paths develop in the core and erode the fill internal erosion (piping) occurred which result breaching type failures. Specialy if the discontinuits are initiate in the body of the embankment dam the probablity of formation of preferential flow is scalated. A good expmple of discontinuits are concrete conduite embedded in the body of the dam. Regardless of the material use, a conduit represents a discontinuity through an embankment dam and its foundation.

This discontinuity can cause settlement to be different adjacent to the conduit than it is in the rest of the embankment dam. Inclusion of incompressible structre (culvert, intake conduite, bottom outlet) cause the differential settlement or diformation settlement in dam faundation. Settlement cracks caused by a compressible material in the embankment or foundation can also provide seepage paths (USBR, 1995). Earth fill may also be compacted differently around a conduit than for the rest of the embankment dam. These factors can cause cracking of the earth fill and lead to other consequences. The reason is that the characterstic of embankment dam is flexible and less affected by ground acceleration.

Howeve the chracterstic of conduit embedded in the body of the embankment dam is totally different from other parts of the embankment dam. In the case of Arjo Dhidhessa Large dam, the conduit is constructed from concrete material which is rigid. Therefore there is irregularity of soil compaction in the layer of the core. Arjo Dhidhessa large dam holds two concrete conduit in the embankment,one for intake conduit on the left side and the second is for bottom outlet which is subjected to internal erosion. Therefore, this study envisaged to fill the knowledge gap of the seepage analysis in the core around the conduit embedded in the body of the embankment and with finding scientific cause for the piping occurrence and to recommend remedial measure.

1.3 Objectives

1.3.1 General Objective

The general objective of this study is to analyze the formation of Internal erosion in the core and its impact on the slope stability of Arjo Didessa rock fill dam by using GEO-STUDIO and PLAXIS 8.2 software.

1.3.2 Specific Objective

The specific objectives of this study are.

- To estimate the overall amount of seepage flow rate that may pass through an embankment with considering Conduit and analyze the pore pressures and piping
- To analyze the slope stability and identify the impact of internal erosion on the stability of the Arjo Dhidhessa rock fill dam by using Geo-studio and plaxis.
- To determine Variations in the stability of the dam due to internal erosion measured in terms of safety factor under different parameters.
- To determine the natural deformation of the embankment due to internal erosion

1.4 Research Question

At the end of this study the following questions should be answered

- What is the overall amount of seepage flow rate that may pass through the embankment and pore pressure & piping under different conditions?
- What is the impact of internal erosion on the stability of the Arjo Dhidhessa rockfill dam?
- What is the factor of safety for Arjo Dhidhessa rock fill dam at different level and with changed permeability?
- What is the natural displacement of the Arjo Dhidhessa rockfill dam under different void ratio?

1.5 Scope of the Study

The scope of this study is to assess and study Arjo Dhidhessa embankment Dam in details. Accordingly, probable causes of failures for the project should be identified based on the available data and those collected from the site. Analysis of seepage and slope stability in the earthen dam by the analytical approach will be carried out.

Computation of factor of safety, pore pressure head, flow rate and residual head along the dam embankment and foundation under different conditions were carried out. Seepage analysis and slope stability analysis were determined by using geo-studio and PLAXIS 8.2 software.

1.6 Significance of the Study

Seepage analysis core around the conduit embedded in the body of embankment dam of Arjo Dhidhessa should be analyzed by considering different parameters. Internal erosion & piping of clay core was investigated by different approach and its impact on the stability of the slope was also analyzed. Material characteristic, Soil physical properties, compaction rate, design and construction methodology, depth from the crest and others were also considered.

Therefore, the study has the following significance.

- To assess and Study the Arjo Dhidhessa rock fill embankment dam details for future performance of the project.
- To assess and Study the probable causes of failures of Ardo Dhidhess rock fill embankment dam to consider the remedial measures.
- The output of this study will give us a lesson for future dam development in Ethiopia.

2 LITERATURE REVIEW

2.1 Embankment Dam

2.1.1 Earth Fill Dams

Earth fill dams are simple structures which stand on their self-weight to prevent the sliding and overturning (Craig, 2004). Earth fill dams are the most common type of dams known in the world. At the earlier time the earth fill dams are constructed to divert massive water body and protect the community. Later it was structurally improved and used to construct the reservoirs. Small earth fill dams contain a variety of advantages in both technically and economically (FERM, 2005). These are, Construction materials are readily available, Simple design criteria, less foundation preparation required when compared with other dams, and Quite flexible than other rigid dam structures and suitable for seismic sensitive regions.

As stated by FERM (2005), Earth dam have some disadvantages when compared with other dam types. These are Higher possibility to damage or slides than other dam types, lack of compaction of material leads to increased seepage and Continuous monitoring and assessment is needed to prevent slope erosion, abnormal seepage and growing plants.

An earth dam is made of earth (or soil) built up by compacting successive layers of earth, using the most impervious materials to form a core and placing more permeable substances on the upstream and downstream sides. The earthen dam can be of the following three types: Homogeneous Embankment, Zoned Embankment and Diaphragm Embankment.

2.1.2 Rock Fill Dam

Rock fill dams have characteristics lying somewhere between the characteristics of the gravity dams and those of earthen dams (Garg, 2005). Compacted rock fills have displaced the dumped or sluiced rock fills formerly employed for embankment dam construction. The concept of well-graded and intensively compacted rock fill has developed from appreciation of the importance of grading and the introduction of heavy compaction plant. Compacted density is the principal factor governing rock fill shear strength and settlement (Novak, 2005).

The major advantages of rock fill as an embankment construction material are high frictional shear strength, allowing the construction of much steeper face slopes than earth fill, and relatively high permeability, eliminating problems associated with construction or seepage pore water pressures.

The disadvantages of rock fill lie in the difficulties of controlling the grading of crushed rock, e.g. from excavations and tunnels, and in the construction and post-construction settlements, which are relatively high. This can result in interface problems where rock fill shoulders are adjacent to a compressible clay core. The quality and suitability of rock fill is discussed in (Penman, and Charles, 1975) and (ICOLD, 2018). An exhaustive study of the engineering characteristics of compacted rock fill and of the special large-scale test techniques required is reported in (Marsal, 1973). The shear strength of compacted rock fills is defined by a curved failure envelope with the form (De Mello,1977).

2.2 Conduit

Conduits convey water from a reservoir through, under, or around an embankment dam in a controlled manner. Conduits through embankment dams serve a variety of purposes. Conduits typically convey releases for: Releasing stored waters to meet downstream requirements, providing emergency reservoir evacuation, Flood control regulation to release waters temporarily stored in flood control space, diverting flow into canals or pipelines, providing flows for power generation, Satisfying a combination of multipurpose requirements, Stream diversion during construction (FEMA,2005).

Most conduits through embankment dams are part of outlet works systems. However, some conduits act as either primary or service spillways; auxiliary or secondary spillways to assist the primary spillway structure in passing floods; or power conduits (penstocks) used for the generation of power. The tower-gate/valve chamber can be located in the central part of the dam, or else in the initial part. In the first case, the approach to the tower is simpler; however, the greater part of the conduit is permanently under water. Very often, the gallery for the bottom outlet is utilized for giving passage to the water in the course of the construction of the hydraulic scheme, for which a greater cross-section is necessary. (Ljubomir, 2014).

According to FEMA (2005), Conduits can be classified as either pressurized flow and non-pressurized flow.

No pressurized flow: Open channel flow at atmospheric pressure for part or all of the conduit length. This type of flow is also referred to as “free flow.”

Pressure flow: Pressurized flow throughout the conduit length to the point of regulation or control or terminal structure.

As Ljubomir (2014), stated many types of materials have been used for conduits over the years, such as reinforced cast-in-place and precast concrete, thermoplastic and thermoset plastic, cast and ductile iron, welded steel, corrugated metal, and aluminum. Some early builders of conduits used whatever materials were readily available, such as wood and hand-placed rubble masonry. The conduits are constructed as reinforced concrete galleries or as steel pipe conduits, lined with reinforced concrete. Regardless of the material use, a conduit represents a discontinuity through an embankment dam and its foundation.

Water flowing through conduits can escape through defects in the walls or between separated joints of the conduit. Soils can also be carried into a conduit through these defects. If a conduit is flowing under pressure and water is forced out of defects within the conduit, this can lead to a very serious problem that must be addressed by emergency action, since catastrophic embankment dam failure could result. If a conduit is not flowing under pressure, defects within the conduit may allow soils surrounding it to be carried into the conduit by seepage and hydraulic fracture. Water escaping through defects from within non pressurized conduits will probably have a lower velocity and lower pressure and should be less damaging to the surrounding soils, than if the conduit were pressurized. This may allow for remedial measures to be undertaken in less of an emergency mode. Generally, defects in conduits are much more serious for conduits designed for pressure flow than for non-pressurized flow.

Attempting to place a filter on the outside of a conduit at a defect is likely to be ineffective, particularly for a pressurized conduit. The quantity of flow from the defect in a pressurized conduit will likely exceed the capacity of a filter designed to protect adjacent soils. In a non-pressurized conduit, the filter designed for a given size defect may be inadequate when the defect increases in size. Replacing or renovating conduits with defects are the only reliable long-term solution to preventing damage to surrounding soils

Water flowing through soils surrounding a conduit may also cause failure of the embankment dam. A conduit within an embankment dam is a discontinuity that may create stresses in surrounding soils that are conducive to hydraulic fracture. A conduit may impede uniform compaction of soils in its vicinity. The various ways embankment dams may fail (where conduits are the sole or primary contribution to the failure) are discussed in this chapter. Many other types of failure modes for embankment dams exist that are not associated with conduits and are outside the scope of this document.

As stated by FEMA (2005), there are four main potential failure modes involving conduits through embankment dams. These failure modes are the following.

Failure Mode No. 1: Backward erosion piping or internal erosion of soils into a non-pressurized conduit.

Failure Mode No. 2: Backward erosion piping or internal erosion of soils by flow from a pressurized conduit.

Failure Mode No. 3: Backward erosion piping or internal erosion of soils along the outside of a conduit caused by hydraulic forces from the reservoir.

Failure Mode No. 4: Internal erosion of hydraulic fracture cracks in the earth fill above, below, or adjacent to the conduit.

2.2.1 Intake Conduit

Intake structures, which serve for intercepting water that, afterwards, is transported to users by means of water-conveying structures, depending on their needs. Special structures of water supply systems are: specific intake structures, regular water intake structures, pump stations, water treatment plants, pipework structures for water supply (pressure relief chambers on head race structures, distribution shafts, shafts for aeration valves etc.

Intake structures: These hydraulic structures assist in the process of direct utilization of water from the water source. They are erected directly on the river course, or else in the case of an impounding reservoir, in which case they can be surface or underground, carried out in the dam wall or outside it. During their design and construction certain common problems must be solved, such as: controlled capture of needed water, elimination of sediment, silt and floating objects, the possibility of controlled operation at various water levels in the source of water capture, etc. (Ljubomir,2014)

2.2.2 Bottom Outlet Conduit

The main purpose of an outlet works through an embankment dam is to control the release of water from a reservoir. An outlet works typically consists of a combination of structures. Bottom outlets are openings in the dam used to draw down the reservoir level (Novak, 2005). Bottom outlet works within hydraulic schemes with a reservoir are constructed in order to ensure conveyance of water from the reservoir into the lower part, downstream of the dam, in order to facilitate lowering of the level of the reservoir, or else to ensure water for the biological minimum flow in the riverbed, downstream of the works. In cases of small dams and reservoirs, the bottom outlet structures often also serve for intake of water for some user as well, most often for irrigation.

2.3 Seepage and Piping

2.3.1 Seepage

Seepage occurs through the body of all earthen dams and through their pervious foundation. The amount of seepage has to be controlled in all conservation dams and effects of seepage (i.e. position of phreatic line) have to be controlled for all dams in order to avoid their failure. (Garg, 2005).

Seepage and piping are among the major subsurface erosion processes affecting the stability and long-term performance of landslide dams and other hydraulic structures. Historical records of performance of embankment dams have shown that besides overtopping, seepage and piping have triggered more catastrophic failure of embankment dams (Okeke *et al*.2016; Foster et al., 2000). Internal erosion became an important field of study in the 19th and 20th Centuries when several catastrophic failures of earth dams were recorded. The phenomena naturally occur in all kinds of porous media wherever a difference in total head exists between two points. (Okeke, 2016).

Hence, the existence of anomalous seepage in dams results in the formation of subsurface cavities or channels which trigger the uncontrolled release of impounded water, leading to instability and failure. The mechanisms of seepage and piping failure of landslide dams have not been fully understood, despite the wealth of research on internal erosion in earth dams and in other hydraulic structures. This section presents a critical review of internal erosion processes and the mechanisms of seepage- and piping induced failure of dams and soil slopes, with emphasis on the critical hydraulic conditions that lead to failure of dams and soil slopes.

2.3.2 Internal Erosion

This process involves the movement of water through a continuous crack or defect within a compacted fill, foundation, or at the contact between a fill and a foundation. Internal erosion can be initiated by concentrated leakage, backward erosion, and suffusion.

Fell, *et al*. (2003) divided the various processes of internal erosion and piping into four phases: initiation of erosion, continuation of erosion, progression to form a continuous piping hole and formation of a breach.

2.3.3 Piping

Piping is one of the internal erosion processes which is initiated by backward erosion, or concentrated leakage in a high permeable zone, and results in the development of a continuous tunnel called a “pipe” between the upstream and the downstream side of an earth dam or its foundation. According to Foster, *et al*. (2000); Bonelli, *et al*. (2007). A few of the processes that initiate piping in soils are described below.

2.3.3.1 Backward Erosion

Backward erosion involves the detachment and entrainment of soil particles when the seepage exits to a free surface such as the downstream face of a landslide dam or an embankment dam (Fell, *et al*., 2003). This process gradually progresses backward towards the upstream side of the dam, forming a continuous pipe as the detached particles are carried downstream by the seeping water (Singh, 2006). This kind of internal erosion generally occurs in cases where a roof a competent soil supports the evolution of a bridged opening.

2.3.3.2 Concentrated Leak Erosion

According to Fell, *et al*. (2003), this process refers to the formation of a crack or concentrated leak in an embankment dam or its foundation which originates from the upstream face to an exit point at the downstream face. This type of erosion is usually caused by freeze-thaw cycles, differential settlement, hydraulic cracking, desiccation and differential compressibility’s in sediments resulting in the formation of large voids.

2.3.3.3 Heave (blow out)

This phenomenon occurs in cohesion less soils due to the reduction of the effective stress of the soil by seepage pore pressures such that pore-water pressure equals total stress. Okeke (2016), and

Terzaghi (1943), reported that the phenomenon of heave occurs when a semipermeable barrier is overlain by a pervious zone under a relatively high fluid pressure (Richards and Reddy,2007).

In civil engineering, heave has been known to occur mainly during the filling of reservoirs behind dams and levees, or during the dewatering of deep, pumped excavations. Similarly, "boils" are attributed to the existence of large hydraulic head differentials which result in channelized subsurface flow leading to the erosion of the fluidized material.

Instability caused by heave is usually initiated at a hydraulic boundary where the rate of movement of water through the barrier is smaller than the rate of increase in seepage pressure under steady state transient flow. Heave is characteristically different from seepage erosion or backward erosion in that backward erosion requires a critical seepage velocity sufficient for the detachment and entrainment of soil particles. (Richards, *et al*., 2007)

2.3.3.4 Suffusion (Internal Instability)

As Okeke (2016), stated Suffusion is a form of internal erosion involving the selective erosion of finer particles from the matrix of coarser particles that are in point-to-point contact, such that the finer particles are removed through the interstitial voids between the coarser particles by seepage flow, leaving behind an intact soil skeleton comprised mainly of the coarser particles. Materials which are susceptible to suffusion are termed "internally unstable".

These materials are mostly coarse widely graded or gap-graded soils. Suffusion results in an increase in seepage velocities, higher permeability, and potentially higher hydraulic gradients. Similarly, a related process known as Suffusion occurs when no point-to-point contact exists between the coarser particles resulting in a change in volume of the soils and thus, the development of sinkholes. (Qkeke, 2016)

2.3.3.5 Internal Migration (Stopping)

This phenomenon refers to a scenario where the soil is not capable of forming a continuous piping hole or lacks the capability to support the pipe roof. In this case, the soil particles are transported downwards by gravity-driven movement, or may be distressed by intense seepage which causes the development of ephemeral voids in the periphery of the initiation zone until the pipe roof can no longer be supported, thus resulting in the collapse of the void. This process may be repeated over time until the core is breached or the downstream slope is over steepened leading to collapse.

Stopping can occur in the central core of embankment dams constructed with broadly graded cohesion less materials (glacial till) due to suffusion and internal instability. (Okeke, 2016)

2.3.3.6 Contact Erosion

Contact erosion involves the selective erosion of finer sediments from the contact with a coarse material due to inter granular flow through the coarse material parallel to the contact. This erosion process commonly occurs in zoned embankment dams, at the interface between different materials.

2.3.4 Mechanisms of Internal Erosion and Piping

By way of water flows through a soil from a region of high total head to a region of low total head, the pressure head is dissipated in overcoming the viscous drag forces resisting the flow through the soil micro pores. This dissipation is followed by the generation of high erosive forces which tend to weaken the interlocking bonds binding the soil particles together. If the seepage force becomes equal to or greater than the interconnecting bonds (erosion resisting forces) binding the soil particles together, the soil loses its strength, leading to the initiation of internal erosion and piping. The erosion resisting forces depend on several factors including the plasticity index of the soil, the interlocking effect, gradation, cohesion, and the dispersive characteristics of any clay in the soil as well as the density of the soil particles (Singh,1996).

The formation of a piping hole at the exit face of the dam has been attributed to high hydraulic gradients which must be high enough to cause internal instability in the soil. This development accentuates subsequent entrainment and erosion of the soil at the periphery of the piping hole leading to the enlargement of the piping hole.

Fell and Wan (2005), divided the internal erosion and piping process in embankment dams into four phases: (1) initiation, (2) continuation, (3) progression, 4, breach/failure. The authors illustrious that internal erosion is initiated from the embankment into the foundation through pre-existing cracks and defects or coarse soils such as gravel in a soil foundation from which piping develops under seepage gradients. This kind of erosion involves backward erosion or suffusion but could also be initiated through concentrated leak as a result of a high hydraulic conductivity or poorly compacted zone such as the contact between the embankment and the foundation.

Fell, *et al*. (2005), enumerated four important conditions which must exist for internal erosion and piping to occur through embankment dams or below their foundations. These conditions are: 1.

there must be a seepage flow path and a water source. 2. There must be an erodible material within the flow path and this material must be able to be entrained and transported downstream by the seeping water. 3. There must be an unprotected exit (open and unfiltered), from which the eroded material may escape. 4. The material being piped or the material directly above must be able to form and support the “pipe roof”.

Similarly, Parker (1964), listed four basic conditions for the occurrence of piping, irrespective of climatic and environmental conditions. These conditions are: (1) The presence of a significant amount of water to saturate the soil or bedrock above a base level; (2) Hydraulic head to cause the water to flow through the subsurface conduit; (3) The presence of a permeable, erodible soil or bedrock above a base level; and (4) An unprotected exit for outflow of the eroded sediments.

2.3.5 Seepage Induced Failure

According to Fox and Wilson (2010), seepage erosion involves the entrainment of soil particles due to the flow of water through soil micro pores. When the seepage velocity is sufficiently large, particle mobilization caused by the drag forces exerted by the seepage flux causes undercutting and subsequently induces slope failure. Seepage can trigger erosion and instability in landslide dams and soil slopes through three interrelated mechanisms. First effects of pore-water pressure on the shear stress of the soil, second increase in hydraulic gradient forces acting on the soil and thirdly the detachment and entrainment of soil particles in the direction of the seepage flow.

One seepage process commonly occurring in landslide dams and soil slopes is sapping. The phenomenon involves the exfiltration of water from the exit face of landslide dams and stream banks under a sufficiently high hydraulic gradient induced by the seeping water (Hagerty, 1991). Sapping is usually initiated when exfiltration occurs over a wide area such that multiple piping holes are formed, or larger lenticular cavities appear at the exit point on the dam or soil slope. Under favorable conditions, the process can intensify causing an increase in hydraulic gradient and subsequent erosion of the soil materials, shortening the seepage paths. This could potentially lead to undermining of the downstream slope and breaching of the dams once there is an enough supply of water (Zhang, and Chen, 2006).

2.4 Slope Stability Analysis

According to Sherard (1963), the factor of safety is defined as the ratio of the resisting force or moment to the driving force or moment. Stability analysis is carried out in order to determine the factor of safety of a potential (shear) failure surface. 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.

The selection of minimum factor of safety depends on various factors. The assumptions made concerning soil strength parameters, pore pressures, method of analysis used, the consequences of failures, construction quality control and economics are the governing factors. Different agencies suggest minimum values within the range of 1.2 to1.5 depending upon the risks involved (Lucas, *et al*., 1986).

Earth dam stability analysis requires knowledge of appropriate shear strength parameters of the soil comprising the embankment and the foundation. Two basically different approaches to the stability problem are in common use (Sherard, 1963). They are the effective stress analysis and the total stress analysis.

There are three periods in the life of a dam that may be critical from stability point of view. These are Construction phase, Full reservoir condition and Rapid drawdown condition. At each of these stages, the mechanics of failure is different and slightly different analytic techniques are used.

SLOPE/W is one of the most complete slope stability analysis programs available. Beginning an analysis in this program is through definition of the geometry by drawing regions and lines that identify soil layers. Then analysis method, soil properties and pore-water pressures can be chosen and applied. The pore water pressures developed within the body of the dam and in the foundation under steady state seepage has been initially estimated with the help of the Plaxis 8.2 and SEEP/W software.

The slope stability analysis of the designed dam will be analyzed using slope/w of Geo-studio 2012 software.

Regarding GeoStudio functions, prediction methods which can estimate the hydraulic conductivity function are used. It is also important to review the definition of mv. This parameter is related to the volumetric compressibility coefficient obtained in iodometric test. This coefficient can be estimated from the relationship presented below.

$$M_{v}=\frac{1}{M} \quad \text{---} \quad 1$$

Where mv represents the volumetric compressibility coefficient, and M corresponds to the modulus of elasticity in iodometric compression. Modulus of elasticity in iodometric compression was defined in (Lambe, *et al*., 1979) as constrained modulus.

This property is defined in

$$M = \frac{E(1-\mu)}{(1+\mu)(1-2\mu)} \text{-- } 2$$

where E is the Young's modulus, and μ represents the Poisson's ratio. Parameter mv is relevant only in the case of significant increase or loss of water. Hence, rapid drawdown represents a scenario for which this parameter is relevant. The vertical stress (δ'v) variation can be defined as the difference between soil's weight before and after rapid drawdown.

$$\delta_v = \gamma * h \text{ --} 3$$

Where γ represents unit weight of soil, and h is depth based on crest elevation.

Table 4 below summarizes the values of pore water pressure ratio Ru used for the stability analyses during and at the end of construction conditions.

2.4.1 Method of Slope Stability

The slope stability investigation was carried out using the Slope/W computer program based on the limit equilibrium method and the Morgenstern-Price method was used to obtain the factors of safety. This particular method has been adopted because, unlike Fellenius or Bishop's or Janbu's methods, the Morgenstern-Price method satisfies rigorous methods because they satisfy all three conditions of equilibrium: force equilibrium in horizontal and vertical direction and moment equilibrium condition (Cheng, 2008). Rigorous methods can provide more accurate results than non-rigorous methods. Applicable to virtually all slopes. Rigorous, well-established complete equilibrium procedure. Spencer's method is also applicable to virtually all slopes. The simplest full equilibrium procedure for computing the factor of safety (Duncan and Wright, 2005). Spencer's method also satisfies both moment and force equilibriums and gives factors of safety values very close to those obtained by the Morgenstern-Price method. It is often more conservative than Morgenstern-Price method. The methods require Mohr-coulomb material model and the following material parameters. These are Unit weight, angle of internal friction and cohesion.

The SLOPE/W component of the software uses the limit equilibrium analysis technique in the numeric. Limit equilibrium technique of numerical analysis tries to satisfy the equilibrium of statics. For both the upstream and downstream face of the dam at steady state seepage, the slope stability factor of safety is determined before the earthquake. Table 5below summarizes the loading conditions and corresponding minimum factor of safety (FOS_{min}) requirements proposed by (USACE, 2003) and used worldwide.

Table 1: Various Load Cases and Minimum Required FS (USACE, 2003)

Case	Loading Condition	Critical Slope	FOSmin
i.	During construction	Upstream	1.3
		Downstream	1.3
ii.	End of construction	Upstream	1.3
		Downstream	1.3
ii.	Sudden drawdown	Upstream	1.3
.v.	Steady state seepage	Upstream	1.5
		Downstream	1.5
v.	Steady state seepage with earthquake	Upstream	1.1
		Downstream	1.1

The factor of safety (FS) is calculated as the ratio of the total available shear strength (or resistance) available (s) along a failure surface to the total stress (or driving force) mobilized (τ) along the failure surface:

$FS = s \div \tau$-- 4

For stability analyses of embankment dams, the recommended factors of safety will vary with loading conditions. Long-term loading conditions (i.e., steady seepage) require higher factors of safety while short-term loading conditions (i.e., Steady state seepage with earthquake) will require lower factors of safety (USSD, 2007).

I Loading Conditions

According to USSD (2007), safety factors of dam has been checked for following loading conditions: end of construction stage. These are Steady state seepage with the reservoir at FSL (Full Supply Level), Rapid drawdown of the reservoir from FSL to MOL (Minimum Operating Level) and Steady state seepage plus earthquake

II End of Construction

The '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 dam under this condition, the residual pore water pressures under this condition are considered as based on the pore water pressure ratio R_u, a coefficient that relates the pore water pressure to the overburden stress. The pore water pressure u is given by:

$$U = R_u \gamma_t H_s \quad (5)$$

Where, γ_t = total unit weight

H_s = the height of the soil column

The value of the R_u considered in the analysis is 0.35.

The 'end of construction' condition can be critical for either of upstream or downstream slopes. Analysis has been carried out to determine factor of safety (FOS) for both upstream and downstream slopes.

III 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. In case of Arjo-Dhidhessa dam, rapid drawdown of the reservoir water surface is not possible due to limited discharging capacity of Canal Intakes. Factor of safety for upstream slope has been worked by considering drawdown from FRL to MDDL at the rate of 0.6 m/day.

2.5 Deformation Analysis

Newmark (1965) proposed therefore the important concept that the effects of earthquakes on embankment stability should be assessed in terms of deformations they produce rather than the minimum factor of safety. Newmark's method models the sliding mass of a dam slope by a block subjected to the acceleration time history of the selected earthquake. The block (or the soil body) will start to slide when the shear stresses exceed the shear resistance. The level of acceleration acting on this soil body at the onset of sliding is termed the yield acceleration. The displacements of the body can then be obtained by double integration of the acceleration time history.

Newmark's method is only applicable for materials which do not lose strength in post-earthquake conditions which means that the slope is still stable after shaking has ceased. It is also useful for situations where no changes in pore-water pressure will occur, i.e. when the slope materials are cohesion less. Both linear and non-linear two-dimensional finite element analyses were performed, applying the sequential loading method described by (Clough and Woodward, 1967). The procedure used for approximating the material stress-strain behavior is by successive load increments, within which the material behavior is assumed to be linear.

The simplified input data for the linear elastic model are listed in the following table with respect to material and the maximum shear modulus (G_{max}) is determined by equation 27. Gmax is considered to be constant.

$$G_{max} = \frac{E}{2x(1+v)} \quad \text{-- } 6$$

Where E= Modulus of elasticity, ν= Poisson's ratio

2.6 Modelling Approach

2.6.1 Geo-Studio 2012

There are a number of software products in the world. Well known software products dealing with seepage problems are the software packages SEEP/W (GEO-SLOPE, Canada), DIANA (TNO Company, The Netherlands), SV Flux (Soil Vision Systems, Canada), (Tanchev, 2014). Seep/w software is selected among others since it works based on discretization of the continue flow medium into manageable units and then reuniting or meshing the discrete parts through the use of continuity equations.

Its comprehensive formulation enables to consider analyses ranging from simple, saturated steady-state problems to sophisticated, saturated / unsaturated and time-dependent problems. Geo-Studio is composed of eight software products that enable everything from simple-to-complex analyses.

When integrated, the products offer a broader analytical environment that offers significantly more power and capabilities. The fully integrated software suite includes limit equilibrium stability analysis and seven finite element applications for modeling geotechnical and earth science problems. SLOPE/W Slope stability analysis, SEEP/W Groundwater seepage analysis, SIGMA/W Stress-deformation analysis, QUAKE/W Dynamic earthquake analysis, TEMP/W Thermal analysis, CTRAN/W Contaminant transport analysis, VADOSE/W zone and soil cover analysis, and AIR/W Air flow analysis.

2.6.2 PLAXIS 8.2

PLAXIS2D is a two-dimensional finite element program, developed for the analysis of deformation, stability and groundwater flow in geotechnical engineering.” (Brinkgreve, *et al.*, 2016). Geotechnical application requires advanced constitutive models for simulation of nonlinear, time dependent and anisotropic behavior of soils and/or rock. In addition, since soil is a multi-phase material, special procedures are required to deal with hydrostatic and non-hydro static pore pressure in the soil. PLAXIS software has been optimized to accurately simulate this highly nonlinear behavior. PLAXIS include geometry creation tools and automated setting to allow geotechnical problems to be analyzed the problem. (USDIBR, 2014).

consolidation and ultimate bearing capacity are all examples of these types of calculations that may be performed by hand calculations. With increased computing power it has become possible to perform more advanced calculations, for example by calculating with the definite element method and the finite element method.

3 MATERIAL AND METHODS

3.1 Description of the Study Area

3.1.1 Location

Arjo Dhidhessa rockfill dam project is located in western part of Ethiopia, Oromia Regional State, Jimma Zone & 352 km from the capital, Addis Ababa. The Project is constructed on Dhidhessa River.

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 2013) is responsible for design, construction supervision and contract administration of the project. The project area is located between 8◦-30'-00" to 8◦- 40'- 00'' N Latitude and 36◦22'-00" to 36◦-43'-00" E Longitude.

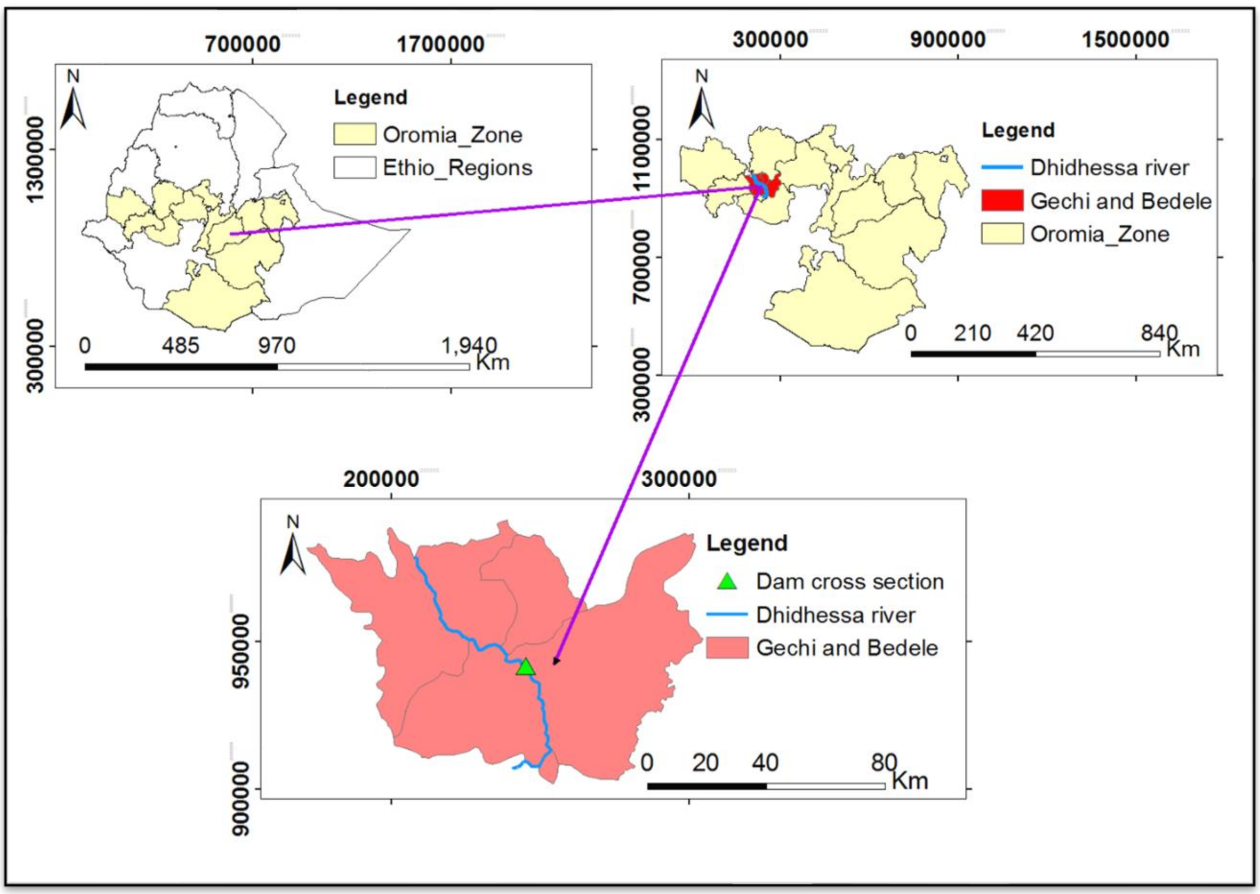


Figure 1: Location of the project area

3.1.2 Topography

Topography of the area indicates that the highest intervening ground reaches El 1365.0m while the top of embankment is at El 1362.0m. Consequently, Topography of the project reservoir area reveals presence of two saddles in the area. Both the saddles are located on the right flank of the main dam. Saddle-1 is closed by a dam of composite section, zoned embankment dam on left and right end and concrete spillway in the middle section of saddle while the saddle-2 is closed by a zoned earth & rock fill dam.

3.1.3 Geology

According to investigation carried out (OWWDSE, 2013) the regional geology shows the rocks available in the area are reported to be Pre-Cambrian metamorphic and Intrusive rocks, Paleozoic deposits, and Mesozoic sediments. An area about 70 km by 80 km neighboring the Dhidhessa river to the NE and SW and within a radius of 35 km from the dam axis was selected for detailed geological investigations, especially potential active faults that might adversely affect this major infrastructure. Tertiary volcanic and quaternary superficial deposits are also present in and around the area.

Alluvial soils are extensively seen along low-lying areas and over lies lower basalt. This deposit is also found over the middle basalt along marshy areas. 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 Sand stones with intercalations of conglomerates, silt stones, etc. (OWWDSE, 2013).

3.1.4 Climate and Hydrology

The Dhidhessa 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. It drains nearly an area of 34,000 square kilometers. The major tributaries of the Dhidhessa River are the Wama entering from the east, dabana from the west, and the Angar from the East. (OWWDSE, 2013). Dhidhessa catchment is situated in the south west of Abbay Basin the catchment area at gauging station near Arjo town is 9981 km2.

The mean annual flow of Dhidhessa river at Arjo station is about 3800 million m^3 having its maximum flow in August and September (52 % of the annual) and minimum flow in February and

March (less than 1.5 % of annual). Most of the rainfall in the Dhidhessa catchment in the June to September period with virtual drought from November through February. Annual totals average from less than 150 cm to more than 200cm.

The Dhidhessa catchment besides reflecting a marked rainfall increase with higher elevation also rainfall heavier annual quantities than most of the catchment in the Abbay basin. Examination of the rainfall records from this area shows that this is due to longer (May through October) rainy season rather than heavier maximum monthly quantities. In the project area, the mean monthly temperature variations throughout the year are $20^{o}c^{o}$ in December to 25.4^{o}C in March.

According to Hurni (1986), classification, the sub basin has five agro climatic zones. These are wet Dega, moist Dega, wet Weyna Dega, moist Weyna Dega and moist Kolla. It has got uni-modal rain fall distribution.

3.1.5 Dam Type

Arjo-Dhidhessa Irrigation Dam is a high rock-fill embankment dam on Dhidhessa River. It comprises of 502.48 m long and 50 m high rock fill dam with central impervious clay core and an ogee spillway on the right bank of river in the Saddle Dam-I. Saddle Dam–I is 710.94m long and 17.0 m (deepest section) high rock fill dam. The length of Saddle is 710.94 m includes spillway, non-overflow blocks and rock fill dam on both flanks of spillway. Saddle Dam–II, 397.04 m long and 10m high rock fill dam adjacent to Saddle Dam I Irrigation sluices for drawing water from the reservoir for the right bank primary canal provided in Saddle Dam-I adjacent to spillway.

3.1.6 Dam Specification

Item	Descriptions
Type of dam:	Rock fill with clay core
Volume of earth fill:	30.8 MCM (including upstream cofferdam volume)
Highest from foundation:	50m
Volume of earth fill under dam body:	7MCM
Crest length:	502.48 m
Reservoir total volume:	5.2 MCM at PMF level and 4.5 MCM at maximum operation elevation
Crest width: 10 m Reservoir area: 96.58 Km at 1362m a.s.l	10m
Crest elevation	1362 m a.s.l
Reservoir length	90 Km at 1362 m. a.s.l
Dead capacity - capacity below DSL	537.15 Mm3
Inactive capacity - capacity between DSL and MDDL	472.61 Mm3.
Active conservation capacity - between FRL and MDDL	1043.20Mm3.
Total capacity - between (FRL - Bed Level)	2052.96 Mm^3.
Surcharge capacity - between MWL and FRL	429.2 Mm3.
Deepest foundation level at distance of about 300 m from left abutment.	1302.7m
Hydraulic height of Arjo-Dhidhessa Dam	1362.0 - 1312.0 = 50 m.
Structural height of Arjo-Dhidhessa Dam	1362.0 – 1302.7= 59.3 m.

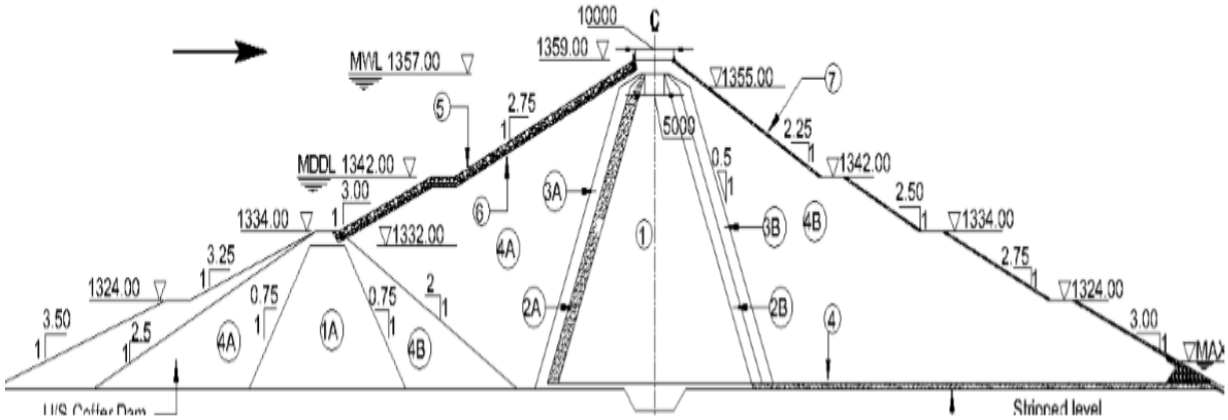


Figure 2: The cross section of Arjo Dhidhessa Large dam project (Source OWWDSE 2013)

3.2 Data Collection

The precision and accuracy of the data included in seepage analysis must be considered in order to predict the failure impact analysis. Some necessary information needs to be collected to analysis the seepage along the conduit embedded in the dam embankment and to determine the effects of internal erosion on the dam stability.

3.2.1 Primary Data

Primary data were collected from the site. These data include topographic survey of the headwork axis, physical observation of the site condition and gathering of necessary photograph and other relevant information.

3.2.2 Secondary Data

All necessary Secondary data were collected from geotechnical investigation and laboratory report used for Geo-studio 2012 such as embankment material property (unit weight, angle of internal friction , cohesion of soil, hydraulic conductivity and condition of soil water content), impervious material, foundation material, aggregate, filter and rip rap.

The detail design report and project drawing of Arjo Dhidhessa large dam project colected from Oromia Water Work Design and supervision Enterprice or ministry of water, Irrigation and Electricity. Topographic map of the Arjo Dhidhessa Large Dam project from Ethiopian Map Agency (EMA).

3.2 Methods

The computer based program, Geostudio 2012 and Plaxis 8.5 software were used in this study for the analysis of the dam. Seep/W was used to conduct seepage analysis, and Plaxis was used to conduct stability and deformation analysis of dam. Deformational analysis, with determination of the displacements and stress distribution through the dam will carried out by Plaxis, finite element methods modeling, under static condition.

3.3 Description of Model Used

3.3.1 GeoStudio

GeoStudio is a platform that enables the user to perform different types of analyses on a soil structure. In this type of analysis, it is necessary to perform a seepage and a stability analysis, thus SEEP/W and SLOPE/W were used.

3.3.1.1 SEEP/W

3.3.1.1.1 Seepage Behaviors

SEEP/W is a numerical analysis program that is able to calculate a phreatic surface, flow and pore pressure in a specified medium and geometry of the users choosing. Fig. (3) shows the result of an experiment performed by (Jennifer Rulon,1985) and how it later on could be calculated in SEEP/W (Geo-Slope, 2012)

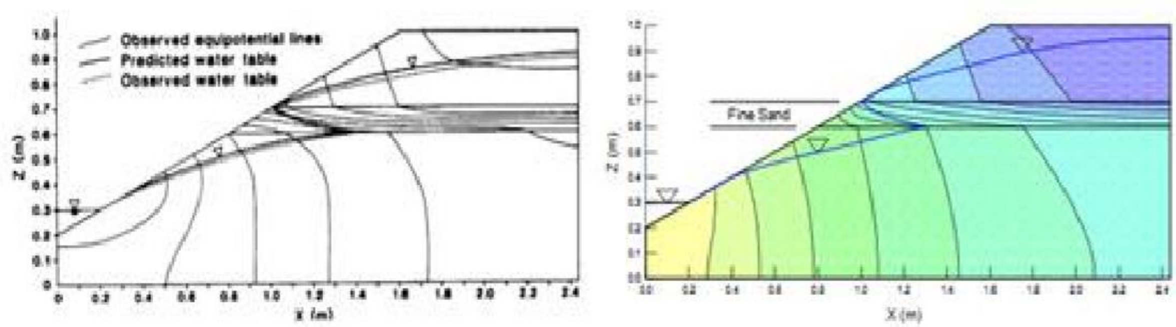


Figure 3: Laboratory results (left) compared to SEEP/W results(right). (Geo-slope, 2012)

SEEP/W uses numerical integration to calculate the seepage. The calculation process is performed in the set of nodes and gauss points that are created in the mesh and with a set of boundary conditions. The mesh can look differently but for this analysis a triangular mesh has been chosen.

The Gauss points inside the triangles calculate material behavior in terms of seepage and extrapolate that information to the nodes at the corner sides of the triangles.

The program then computes the information for each node in the whole model and reveals if the node is either the head (phreatic surface) or flux (below the phreatic surface) Geo-Slope manual (2012). Since the “correct” value is calculated in the Gauss points a finer mesh can yield better results, however the more nodes and points used the longer time the calculation process will take.

Since all dam construction materials are both saturated and unsaturated at some point in the geometry, a saturated/unsaturated model will be used for all materials. For regions over the phreatic surface, as considered for the core and support, matric suction will be considered. However, for the course filter and rockfill it is assumed to have no capillary ability. The suction will create partially saturated zones which has a lower permeability than fully saturated zones. For this a function is needed to simulate the relation between soil water storage and the matric suction.

The soil water storage considers the porosity (n) and the degree of saturation. This is performed by using (Geo-Slope, 2012).

$\Theta w = nS$--7

where $\boldsymbol{\Theta w}$ is the volumetric water content, **n** is the porosity and **S** is the degree of saturation that was set to 100%.

The calculation model Van Genuchten is an equation that takes into consideration the matric suctions resistance of the water flow. What is not considered in the equation is the amount of compaction performed on the soil. The rate of compaction has a direct correlation with void ratio which in itself will link to the capillary rise. However, it is fairly mirrored in the permeability which is low for the materials meaning that there is a low void ratio and thus highly compacted.

3.3.1.1.2 Seepage Analysis

According to GEO-SLOPE International Ltd (2012) there are two fundamental types of seepage analysis: steady state and transient.

A steady-state seepage analysis is an analysis type where water pressures and water flow rates do not change with time. Since steady-state analyses ignore the time domain, it greatly simplifies the equations being solved. 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-Dhidhessa 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.

A *transient analysis* on the other hand, has pressure conditions that change with time. In general, a transient analysis can provide more accurate results when soil conditions are modeled, however, they are significantly more complicated than steady-state analyses.

Both the initial conditions as well as future boundary conditions must be provided. If the initial or future conditions are not accurately represented, the analysis will provide inaccurate results. Since this thesis is considered a primer for understanding seepage analysis, transient analysis is not included in the scope and the steady-state analysis type will be the sole focus.

When developing a numerical steady-state model using SEEP/W, one must determine geometry, assign materials, assign boundary conditions, then review and fine tune the finite element mesh.

According to Geo-Slope International Ltd (2012) “Seepage Modeling with SEEP/W Manual” Soil Geometry The first step to determine soil geometry is to create a scale model of the cross-section of the system being evaluated. The second step is to define soil regions to the cross-section. Both steps are described below.

Creating Cross-section: There are two features that will allow to create a cross-section in SEEP/W. The first feature utilizes the DRAW function to create lines that make up the geometry. This can be an easy tool for creating cross-sections where the user wants to simplify geometry.

The second feature uses the KEYIN function to manually enter points along a cross section. This feature can be more time consuming but is useful if the user wants to define actual contours derived from topographic mapping.

Creating Regions: Regions are created by connecting points and are used to define areas of different material properties and conditions.

Assign Materials: The next step in developing a numerical model in SEEP/W is to assign materials to the regions defined in the previous sub section.

Assign Boundary Conditions: The next step in performing a seepage analysis in SEEP/W is to assign boundary conditions. Setting up the boundary conditions in the model is an essential component as the solution is dependent upon the type of boundary conditions defined in the model. As stated in the ‘’SEEP/W manual’’ provided as a supplementary document along with this thesis, “Boundary conditions can only be one of two fundamental options you can specify H (head) or Q (total flux).”

A simple example of where two boundary conditions would be located can be demonstrated along a cross-section of a dam. The reservoir side would have a boundary condition between the pool and the soil media, and the entire downstream side of the dam would be the potential seepage face.

Fine Tuning the Finite Element Mesh: As stated earlier, SEEP/W uses finite element numerical modeling to solve groundwater problems. In order to solve these problems, SEEP/W divides the entire domain of the model into smaller, simpler parts known as discretization. This discretization is shown by viewing the finite element mesh

3.3.1.1.3 Boundary Conditions

The last thing to do in the model is to set up boundary conditions for the seepage model. This is to set “boundaries” for the analysis in where it is “allowed” to act and to set the initial conditions, such as the reservoir head. What is also set is the seepage face on where it is possible for seepage to exit the geometry created. Boundary conditions are used when calculating differential equations, thus it is only used in SEEP/W and not SLOPE/W.

3.3.1.2 SLOPE/W

After the SEEP/W analysis has been performed, GeoStudio is capable of using that data as a ‘parent’ for the next analysis which in this case is a slope stability analysis. This analysis is performed using SLOPE/W which uses the limit equilibrium method Fig. (4). This is performed by slicing a slope into lamellas with a chosen circular shear surface and a point of moment.

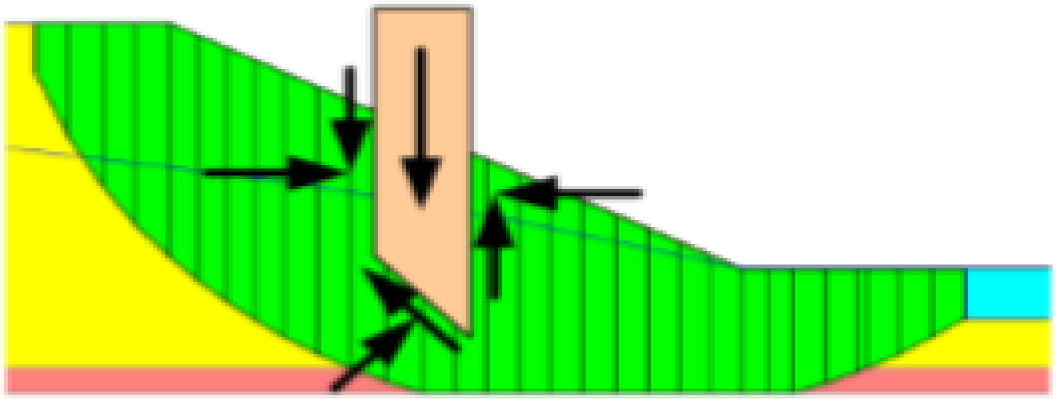


Figure 4: A slope with a visual representation of forces acting on a slice. (Geo-Slope, 2012)

Simply it is the acting unit weight in the lamella that will act on the bottom surface and thus create shear strength. More modern ways of calculating limit equilibrium also includes the action between the slices, so called interslice forces. The shear force and the moment create a global factor of safety for the slope in that geometry. There is an abundance of methods that can be used but in this thesis the Morgenstern-Price method is used due to its inclusion of both force and moment equilibrium in the model. The equation for the factor of safety in regards of moment equilibrium is shown in

$$F_m = \frac{\sum(c^{\wedge\prime}\,\beta R + (N - u\beta)R * tan\,(\phi^{\wedge\prime})}{\sum wx - \sum N_f \pm \sum Dd} \text{---} \quad 8$$

The equation regard of force equilibrium is shown in equation 15

$$F_s = \frac{\sum(c'\beta\cos(\alpha) + (N - u\beta)R * ta\,n(\phi') * \cos(\alpha))}{\sum Nsin(\alpha) - \sum Dcos(\text{ω})} \text{---} 9$$

Table 2: Parameters used in the equations for moment and force equilibrium. (Geo-Slope, 2012)

symbol	Parameters
c’	effective cohesion
ϕ'	effective angle of friction
U	Pore water pressure
N	Slice base normal force
W	slice weight

D	concentrated point load
β, R, x, f, d, ω	geometric parameters
α	inclination of slice base

Morgenstern-Price calculates both the moment and force equilibrium and in SLOPE/W the safety factor is chosen where F_s=F_m as seen in Fig. (5).

Numerical model is a mathematical simulation of a real physical process. It is very different from scaled physical modeling in the laboratory or full-scaled field modeling.

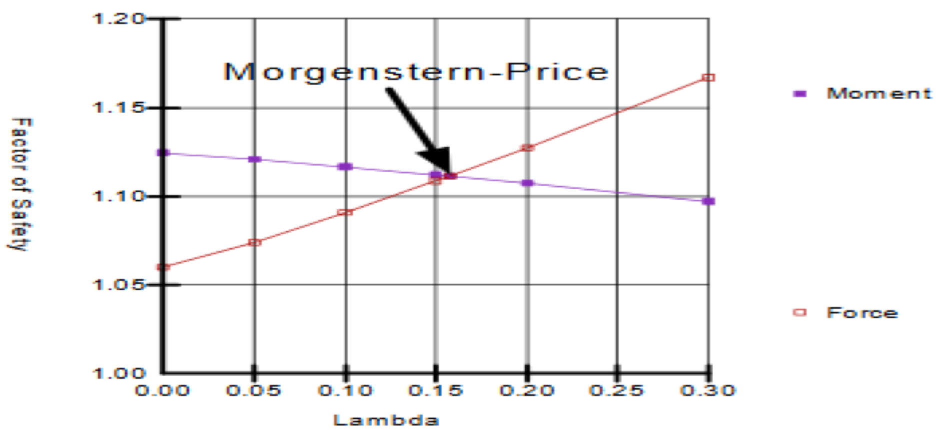


Figure 5: Example of how the SF for Morgenstern-Price is chosen. (Geo-Slope, 2012)

GEO-STUDIO is one of this numerical modeling software developed by Geo-Studio international based on limit equilibrium and finite element principles developed specially for the analysis of seepage, stability and deformation of geotechnical structures.

3.3.2 PLAXIS 2D

PLAXIS2D is a two-dimensional finite element program, developed for the analysis of deformation, stability and groundwater flow in geotechnical engineering." (Brinkgreve, *et al.*, 2016) That is the starting phrase describing PLAXIS2D which defines the relations between different variables that are included in geomechanics. With increased computing power it has become possible to perform more advanced calculations, for example by calculating with the definite element method and the finite element method.

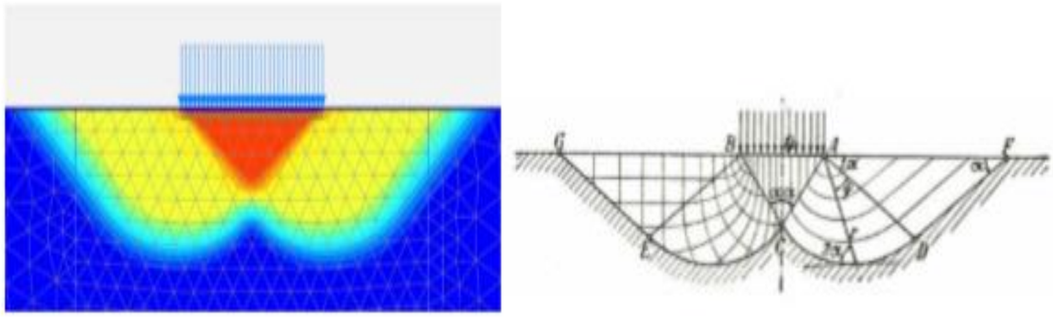


Figure 6: Figure of a typical failure zone in PLAXIS2D

(*left, compliments of Niclas Lindberg) compared to empirically derived Prandtl and Rankine zones (right).*

The older ways of calculating are sufficient in many cases, as seen in Fig. (6) where the Prandtl and Rankine zones are visualized in PLAXIS 2D. However, for more advanced constructions and for research FEM is a better tool to use due to its capability to include more information in the design.

3.3.2.1 Mesh

After the geometry has been created the model is discretized into a mesh of triangles. These triangles are made up of nodes and Gauss (stress) points where the numerical calculations take place as seen in Fig. (7). The more triangles there are in the mesh the more precise the calculation will be performed and with a 15-noded triangle the calculation will yield more realistic results. In this analysis 15-noded mesh triangles are used since the matter of data and memory storage is not of an issue, and also for safety factor calculations 15-nodes are preferred due to that 6-noded triangles over predict failure loads. (Brinkgreve, *et al*., 2016)

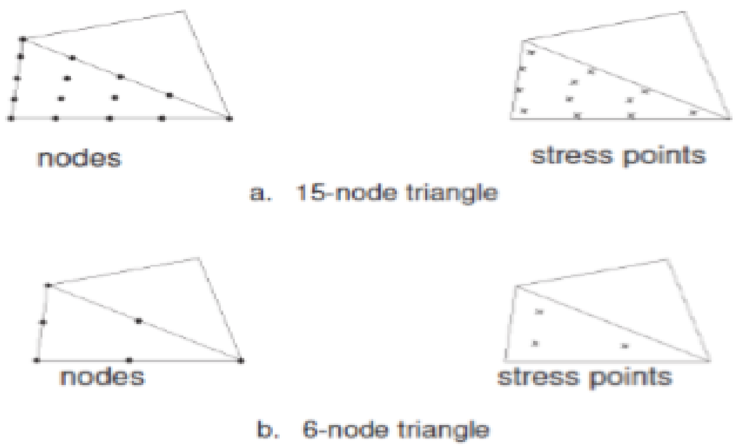


Figure 7: Nodes and stress points with their locations in a mesh triangle. (Brinkgreve, *et al.*, 2016)

As Felippa (2004), stated the stress-strain relation in the model is calculated in the Gauss points where after its results are extrapolated to the nodes for calculating the deformation. Strain and deformation are both variables for 'movement' of the material but since strains are unitless it enables you to put in the materials strain parameters whereas the deformation is the actual length of deformation the user of the program sees. It is also important to receive good quality in the mesh, as seen in Fig. (8)

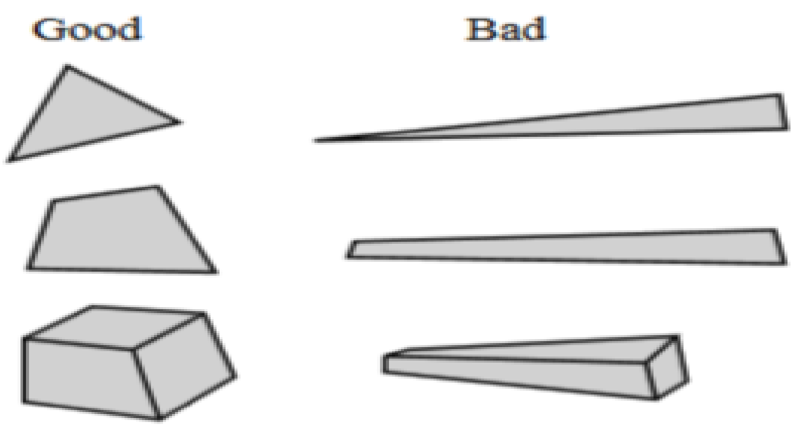


Figure 8: Examples for good and bad mesh quality with different kinds of mesh. (Felippa, 2004)

The integration calculation is subsequently looped until the mesh calculation finds equilibrium in the model in an iterative process which thereafter produces the result of the calculation. (Felippa, 2004)

3.3.2.2 Plane strain modelling

The assumption of plane strain is used when considering an elongated construction or geometry. A dam construction could be seen as such, even with a few discrepancies like depth to bedrock and geometry might differ some. It is up to the engineer to decide if the plane strain assumption can be applied, where strains in z-direction are set as zero (stress is however still considered). An example of a plane strain condition is shown in Fig. (9).

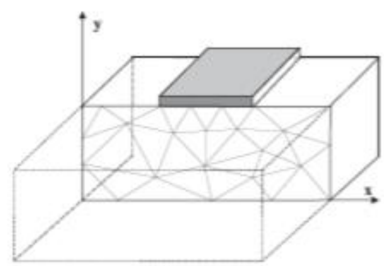


Figure 9: An example of a footing using 3D-modeling (Brinkgreve, et al., 2016)

3.3.2.3 Stress field generation

I K_0 procedure

For soil materials it is important to understand soil's pressure at rest (lateral pressure), meaning the ratio of how vertical pressures induced by gravity and external loads induce horizontal stress, which is shown in the following expression:

$$K_0 = \frac{\sigma' x}{\sigma' y} \qquad 10$$

There is also a relation with the strain parameter Poisson's ratio

$$K_0 = \frac{v}{1-v} \qquad 11$$

and also, the angle of friction, as seen in (Jaky's empirical expression);

$$K_0 = 1 - sin\ (\phi) \text{---} 12$$

For an over consolidated soil, much like the highly compacted soil in a dam it is better to add the over consolidation ratio as a component to receive the correct K0 as shown in Equation (16) (Axelson & Matson, 2016).

$$K_0 = (1 - sin\ (\phi))\ (OCR)^{sin\ (\phi)} \text{---} 13$$

PLAXIS2D K_0-calculation procedure have a drawback when calculating for the geometry that does not have a linear soil stratum and phreatic surface. This is because the full equilibrium is only found for linear horizontal surfaces during the procedure. Examples of such cases are shown in Fig. (10). (Brinkgreve, *et al*., 2016)

Figure 10: Different soil stratum

1.A vertical structure, possibly a lime cement column. 2. A non-horizontal soil strata. 3. A lowering of the phreatic surface, possibly due to a draining well nearby. 4. A slope.

All of the examples observed in the figure can be seen in a dam and for the case of this analysis, K0 procedure cannot be applied. Thus, another type of calculation can be performed called Gravity loading.

II Gravity loadings

Gravity loading calculates the soil based on the materials volumetric weight as opposed to the K0 procedure which adds the soil weight after the calculation is completed. Thus, the initial stresses will be dependent on the use of calculation model, such as Mohr-Coulomb. The amount of stress thus becomes dependent on the used values in Poisson's ratio which has already been seen to have a relation with the K0. (Brinkgreve, *et al*., 2016)

3.3.2.4 Boundary Conditions and Finding a Solution

A finite element analysis is a discretized partial differential equation that is solved by using predetermined parameters. Boundary conditions are the foundation for finding a solution for a finite element analysis (Geo-Slope, 2012). It sets the boundaries for which the initial parameters can act. The program tries to solve the partial differential equations by successive approximation of the solution for the iteration as seen in Equation (14), where the global error is set in PLAXIS2D as a "tolerated error".

Fig. (11) illustrates how the iterative process finds the approximate solution. Fig. (11) illustrates the Newton-Raphson method to approximate the solution. Plaxis however uses the Gaussian method. The figure does however still visualize the relation with "Out of balance forces" well. The Newton-Cotes method is also used in some elements such as geogrids and interfaces.

$$Global\ error = \frac{\sum \text{Out of balance forces}}{\sum \text{Active loads} + \int \frac{\Delta\varepsilon * \Delta\sigma}{\Delta\varepsilon * \text{De}\Delta\varepsilon}(CSP) * \sum \text{Inactive loads}} \quad \text{---------------------------------} \ 14$$

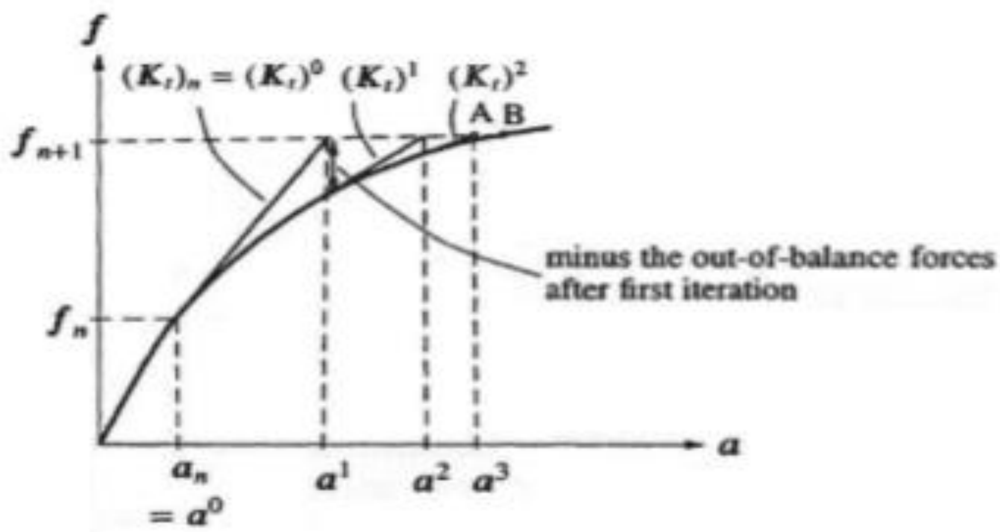


Figure 11: Iterative process for finding the approximate nonlinear solution for a FEA. (Matson, 2016)

I Calculating the Safety Factor

Calculation of the safety factor in PLAXIS2D is performed by using a so called "phi/c reduction". By dividing the initial strength parameters and with a gradually reduced strength in the model, a final global safety can be measured in iterative steps. The equation used for this is shown in (Brinkgreve, *et al.*, 2016)

$$\sum Msf = \frac{\tan(\phi\text{input})}{\tan(\phi\text{reduced})} = \frac{c_{input}}{c_{reduced}} \text{-----------} \quad 15$$

Since PLAXIS2D finds the global safety factor a predetermined circular shear failure surface is not possible to be created like it is possible in SLOPE/W and other programs based on the limit equilibrium method. Thus, only the most critical safety factor will be calculated and shown in the model. If two or more surfaces are found it is an indication that the model is yet to find a solution and more calculation steps may be needed. (Brinkgreve, *et al.*, 2016)

3.3.2.5 Constitutive Model

I Choice of Constitutive Model

Different types of soil behave differently than other materials such as steel and concrete, which behaves linearly. To simulate soil material, models such as Hardening soil and Soft soil can be used. These methods consider strain parameters that enable simulation of deformation in a proper way. However, for a stability calculation, as the case of this study they all follow the Mohr Coulomb failure criterion and thus there is no need to use another model. (Brinkgreve, *et al*., 2016)

II Mohr Coulomb

As Brinkgreve, *et al.* (2016), stated Mohr Coulomb is a linear elastic perfectly plastic constitutive model. The soil will behave according to Hooke's law in the elastic face while behaving perfectly plastic in the plastic phase according to the Mohr-Coulomb failure criterion. An illustration of the behavior is shown in Fig, (12).

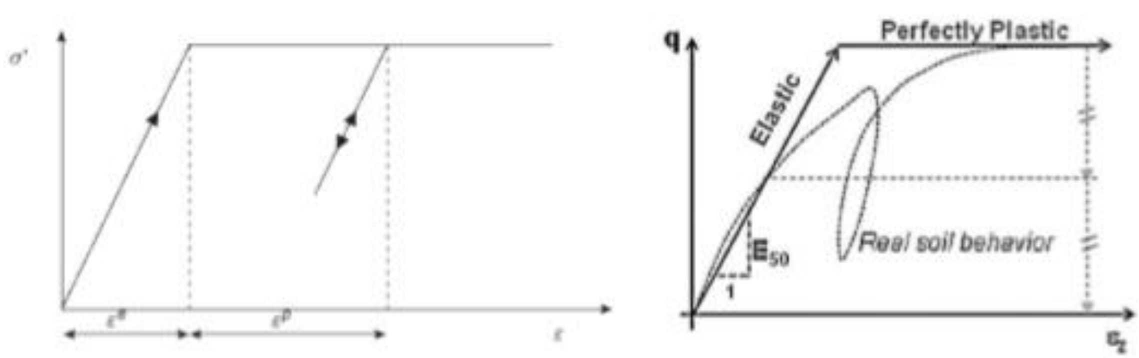


Figure 12: Behavior of a linear elastic perfectly plastic model

(*unloading will perform (left) (Brinkgreve, et al., 2016) The figure to the right compares the Mohr Coulomb model to real soil behavior (Gouw, 2014))*

A perfectly plastic model such as Mohr-Coulomb has a theoretical fixed yield surface thus a constant stiffness. Mohr-Coulomb is usually used for engineers to give a rough estimation of a deformation problem which will be considered in this study. (Brinkgreve, et al., 2016). The Mohr-Coulomb yield surface is described with six functions that include the strength parameters of the soil (cohesion and angle of friction) as well as the effective principal stresses (pore pressure is included, thus effective parameters). These equations visualize the yield surfaces shown in Fig. (13). (Brinkgreve, *et al.*, 2016)

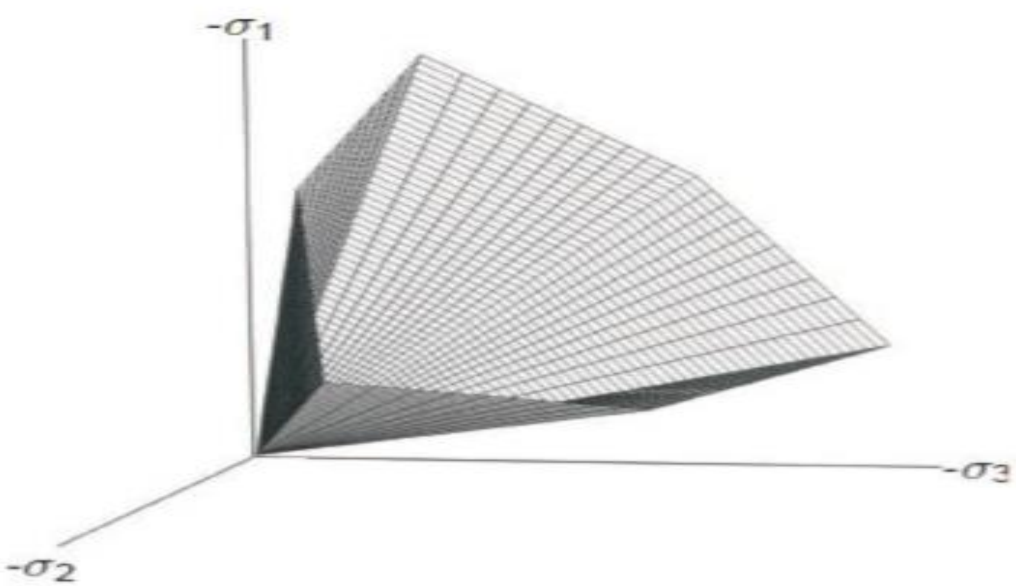


Figure 13: Yield surface for the material model Mohr-Coulomb

(*not that Plaxis programs use negative values for compressive stresses*. (*Source* Brinkgreve, *et al*., 2016)

For positive values on strains another strength parameter is introduced, dilatancy angle (φ). The dilatancy angle is a required parameter if positive values for volumetric strain are considered both for dense or very coarse angular soil.

Parameters used to create Mohr-Coulombs yield surface. (Brinkgreve, *et al.*, 2016), **c** Cohesion **ϕ** Angle of friction **φ** Angle of dilatancy **E** Young's modulus **ν** Poisson's ratio

3.3.2.6 Finished Model

The finished model for the PLAXIS2D analysis is shown in Fig. (14). Note that the phreatic surface inside of the geometry does not matter how it is drawn for a steady state calculation. This is because of input permeabilities where the model tries to find the head using Darcy's law when a gradient is considered. This enables the model to change its ground water head in different phases. (Brinkgreve, *et al*., 2016)

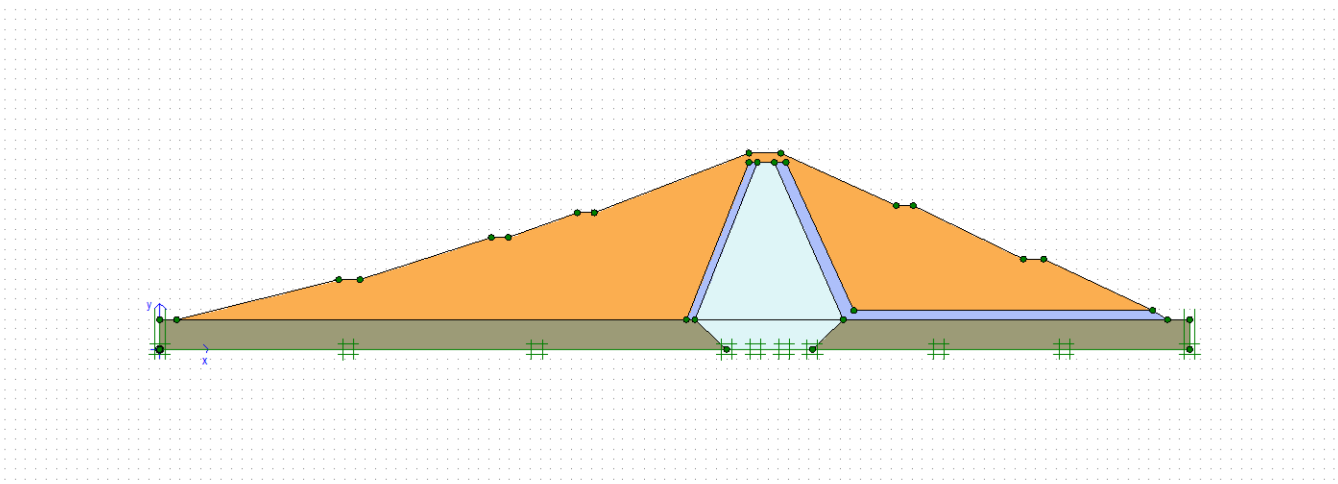


Figure 14: Initial conditions as they are designed in PLAXIS2D.

3.3.2.7 Deformation Analysis

In this study deformation of the dam has been analyzed and present in different approach. The original layout of the output is zoomed in and for detail visibility of the result, the frame and title are trimmed out. shading approach are used to show the deformation pattern of the dam. Unlike Slope/w, which is based on limit equilibrium methods, PLAXIS is described as a Finite Element package for geotechnical analysis that can utilize both two-dimensional and three-dimensional analysis in determining the stability and deformation experienced by slopes. PLAXIS lends itself to modeling more complex geotechnical scenarios as it has the capabilities to simulate inhomogeneous soil properties and time-dependent scenarios. Relative to Slope/w, Plaxis gives more accurate result (Serra, 2013)

3.4 Parameter Considered

Not much data on the material parameters was available. Density and permeability of the core and filter are known, in addition, the reports made during construction of the dam give an indication of the type of materials used. Based on this information a literature was done in order to obtain the additional geotechnical parameters needed to describe the mechanical behavior of the dam materials. RIDAS guidelines also state that the dam should have a safety factor of 1.5 downstream while having a filled reservoir and a safety factor of 1.3 upstream when a rapid drawdown has been conducted. (Svenska Energi, 2012).

3.4.1 Density

The core and the support already have a predetermined density. The filter is made of macadam-sized gravel and the rock fill of cobbles and, according to the documentation for the dam, they are both made from blasted rock taken from the vicinity. The bedrock material nearby is made of intrusive rock such as granite, thus 18kN/m3 dry unit weight for the filter is used Larsson, *et al* (2007). RIDAS recommends 15kN/m3 for the rock fill material if no other data is available. Thus 17kN/m3 is used instead as initial parameter between the previous two values. For the filter, which is based on the same material but with a lower void ratio, 18kN/m3 is assumed. These values were also used in the stability analysis performed by the dam owner.

3.4.2 Poisson's Ratio and Young's Modulus

Poisson's ratio and Young's modulus (E) are the strain parameters in the stress-strain relation used in FEM-calculations. The initial values used in the analysis are the same as used in Vahdat i's work (2014). Even though strain is not being regarded in the stability analysis, it still is an important factor in the gravity loading procedure for initial stresses in PLAXIS2D. It also provides the opportunity to get an indication of main deformations in the model, even though the Mohr-Coulomb constitutive model used in the analyses is not recommended for this observation, (Brinkgreve, *et al.*, 2016).

Table 3: Typical ranges of values for poisons ratio (Bowles, 1996

Types of soil	**poisons ratio**

Clay	0.4-0.5
sandy clay	0.3-0.35
Silt	0.2-0.35
sand (dense)	0.2-0.4
coarse (void ratio 0.4 to0.7)	0.15

3.4.3 Dilatancy

The evaluation of the dilatancy angle follows the simple relation between the friction and dilatancy angle as suggested by Plaxis2D, (Brinkgreve, *et al.*, 2016).

$$\varphi \approx \phi - 30 \quad \text{......} \quad 16$$

Thus, the dilatancy angles are set as 5 degrees for the core and 2 degrees for support and filter. The fill however follows a recommendation to be 15 degrees for angular (not rounded alluvial) blasted rock material which has close to 40 degrees friction angle. (Agahei Araei, *et al*.,2010)

3.4.4 Hydraulic Conductivity

The initial hydraulic conductivity for the core was evaluated to be 1,2*10-7m/s, 4.82E-03 for shell and 6.42E-04 m/s for filter during the construction of the dam and these values are used in the model. The two materials, filter and rock fill, are highly permeable material.

3.4.5 Strength Parameters C and tan ϕ by Iteration

The strength parameters used for the analyses also had to be estimated by iterative process. No cohesion in the dam is considered for any material since they are all based on frictional soil (cohesion less). The base parameters for the friction angle were derived from the dam stability report previously performed.

Table 4: Considered value for the friction angle

Material	**Friction angle ϕ (˚)**
Core	30
Filter	35
Coarse filter	35
shell	42

During a rapid drawdown the upstream safety factor must be at the least 1.3 and while being a full reservoir a safety factor of 1.5 downstream needs to be achieved. When the initial parameters are set an iterative lowering of the dam's strength is performed with the resulting safety factor. This is done with the aim of finding the lowest possible strength parameters of the dam with the ratios between the different parameter's constant. The analysis was performed with a steady state analysis of a full reservoir as well as a rapid draw down analysis (RDD). The lowered strength of the parameters was performed following

$tan{-}1(tan(\varphi)*n)$ 17

3.4.6 Conductivity and Void Ratio

This chapter is largely influenced by the article 'A relation of hydraulic conductivity – void ratio for soils based on Kozeny-Carman equation' by Ren, *et. al*. (2016)

I Void Ratio, Specific Surface and Permeability

Conductivity of soils is a fundamental subject when constructing an embankment dam, thus it is important to understand the behavior of seepage. Seepage is often explained with Darcy's law through a porous medium seen in

$q = Aki$.--18

q is the measured water flow where **A** is the area of the crosscut where the water is being moved through, **k** the permeability and **i** the hydraulic gradient. (Craig, 2004) For a medium such as soil there is a close relation between the void ratio and the permeability. The higher the porosity the higher the permeability is the general rule, which is also the case for the Kozeny-Carman relation shown in

$K = cf\,\frac{1}{s_s^2}\,\frac{yw}{\mu p_m^2}\,\frac{e^3}{1+e}$--- 19

Where again **k** is the calculated coefficient for the permeability, $\mathbf{S_s}$ is the specific surface for the material, $\boldsymbol{\gamma_w}$ the unit weight for the water, **μ** the coefficient for the fluid's viscosity, $\boldsymbol{\rho_m}$ the density of the soil, $\boldsymbol{cf}$ dimensionless shape constant and e the void ratio. The void ratio in this case is the total void ratio for the medium, meaning that there is the assumption of total use of the pores for fluid flow. The fact is that the Kozeny-Carman equation works well for course materials like sand and gravel but loses its coherency for finer soils such as silt and clay. (Ren, *et al*., 2016).

This is because of said assumption of total usage of the pores for fluid transportation, which should be a linear function but is not the case for finer soils due to their electromechanical bond between the particles. Thus, there have been attempts to make a better relation for soils such as clay and silt. (Ren, et al., 2016) One interesting relationship between the logarithm of hydraulic conductivity and the void ratio that was observed by Taylor (1948) and Lambe and Whitman (1969) can be seen in equation (20)

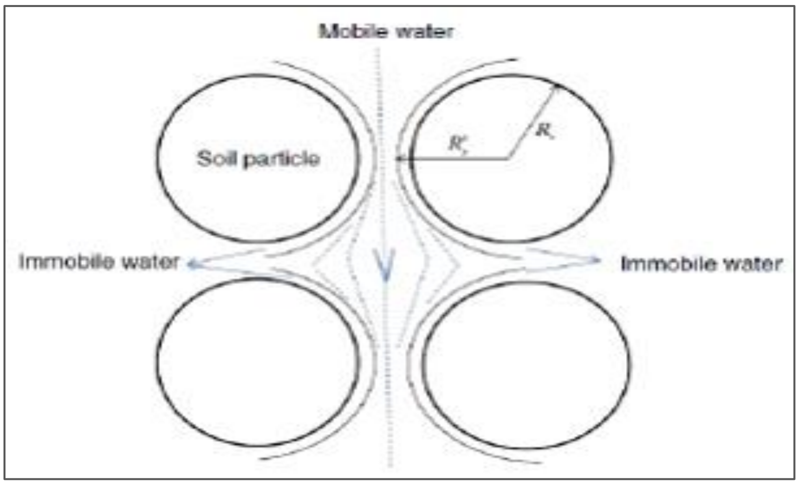


Figure 15: An assumed 100% saturated soil with void ratio

(*That is immobile due to electromechanical bonds and mobile water that flows through the medium*. Ren, *et al., 2016)*

e=$e_o + c_k \log\left(\frac{k}{k_o}\right)$--- 20

This can be rewritten bas

$\log\left(\frac{k}{k_o}\right)=\frac{e-e_o}{c_k}$--- 21

Equation (21) is the equation used in PLAXIS 2D for when soil is consolidated, meaning that there is a change in void ratio and how that affects the permeability. **C_k** is by default set as the compression index given in constitutive models such as Hardening Soil and Soft Soil. However, for the plastic calculation performed in this analysis this cannot be used. Brinkgreve, *et al* (2016). conclude that, there is a difference between void ratios or rather there is a need to separate them.

In the work of Ren, *et al*. (2016) a new concept is introduced called effective void ratio ($\boldsymbol{e_e}$**)** and ineffective void ratio **(**$\boldsymbol{e_i}$**)** which were separate the void volume into different types. Effective void ratio being the parts of the pore which transport of water to take place while the ineffective void ratio "breaks" the flow. The evident relation is shown in (Ren, *et al*., 2016)

$$e_t = e_e + e_i \text{ -- } 22$$

Where e_t meaning the total void ratio which is simply noted as e in most literature. The factor $\boldsymbol{e_i}$, meaning the void ratio of immobilized water, is strongly influenced by geometric characteristics, salinity and cation (the charge). Usually these parameters usually can be translated to specific surface (**Ss**), meaning the total surface area of the particles measured in a medium.

The general rule is that with decreased grain size a higher specific surface needs to be regarded. How this affects the overall permeability with different void ratios for certain materials. (Ren, *et al.*, 2016), As Lambe and Whitman (1969), observed that there was a logarithmic linear relation between void ratio and permeability for several materials so Equation (23) can be used. Ren et al. suggest according

$$K = c\,\frac{e_t^{3m+3}}{(1+e_t)^{\frac{5}{3}3+1} * ((1+e^t)^{m+1} - e_t^{m+1})^{\frac{4}{3}}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 23$$

To remedy this where c is explained in

$$C = \frac{1}{c_f} * \frac{yw}{\mu p_m^2} * \frac{1}{s_s^2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 24$$

In fig. (16) constant c is found to be 2.94E-4*(Ss) $^{-1.45}$ with use of linear regression ion for different soils, thus this value is used.

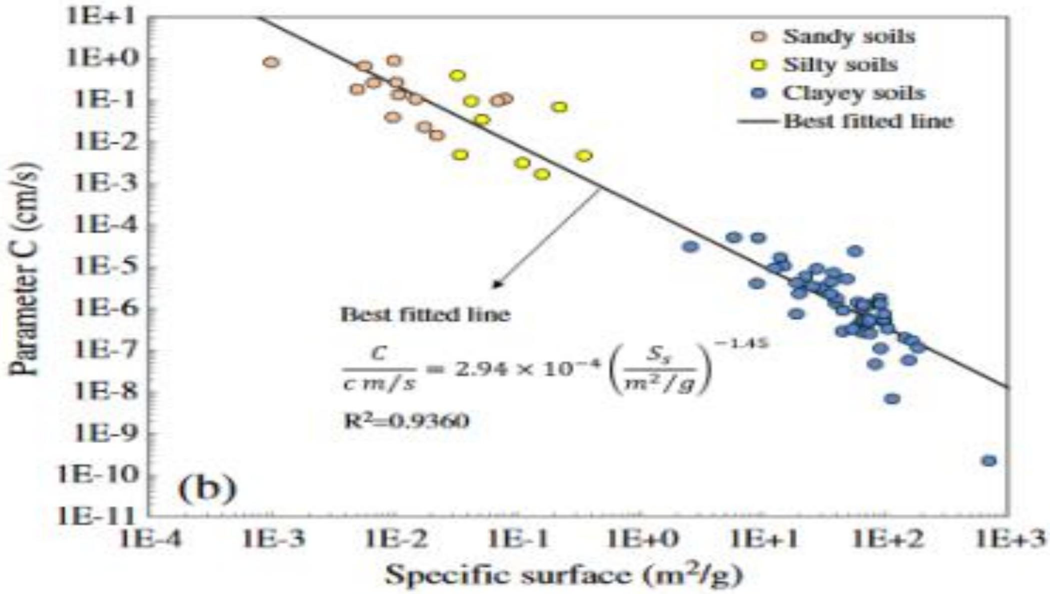


Figure 16: Relation between different soils for parameter C.

(Source Ren, et al., 2016)

The only parameter left is the constant m which refers to the difference in specific surface between materials and was evaluated and shown in Fig. (17)

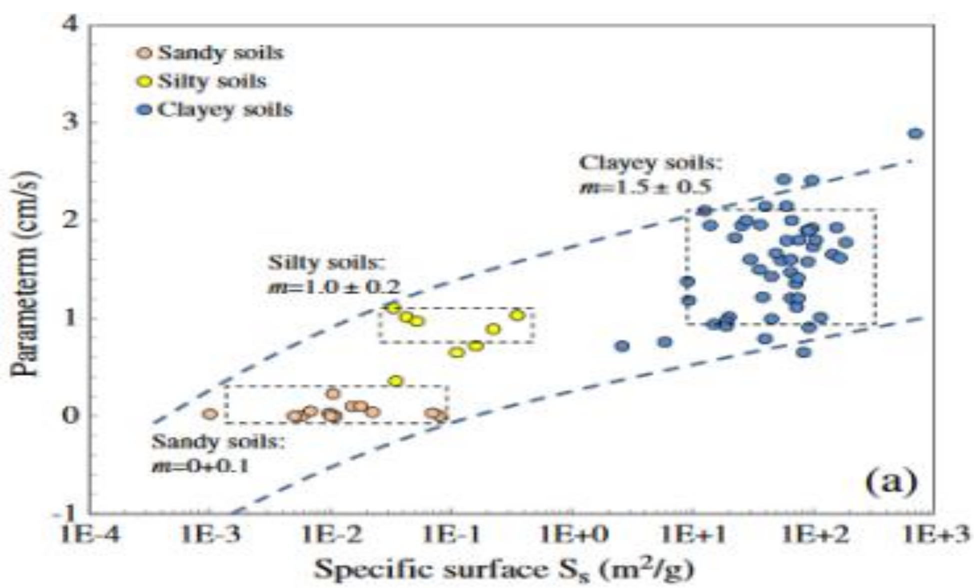


Figure 17: Change of constant m to fit the results of different soils.

(Source Ren, *et al., 2016)*

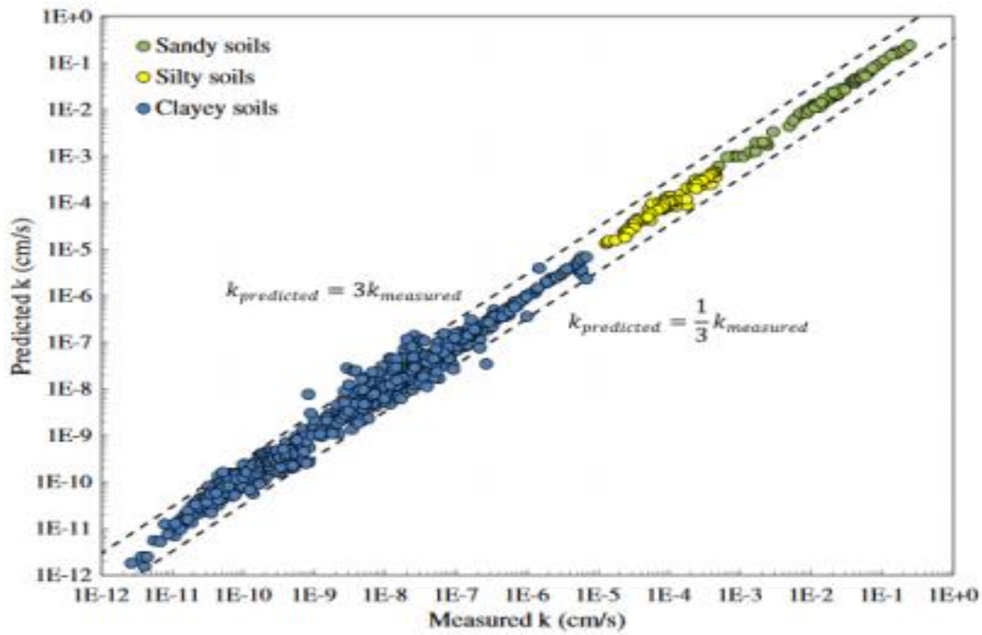


Figure 18: The results after using Equation (6) on different soils

(*where the results tend to span between 3k and 1/3k of measured permeability. (Ren, et al., 2016*)

3.5 Other Parameters

The unit weight for the compact density for the soil material and water was set as 21.68 kN/m3 and 9.81kN/m3. If these values have been set it means other parameters can be calculated. For example, the void ratio for the core which were used in calculations. Note that these are not used as input in either SEEP/W, SLOPE/W or PLAXIS2D. To calculate the void ratio Equation (25) is used.

$$e = \frac{\gamma s - \gamma d}{\gamma d} \quad \text{.......... 25}$$

From this, porosity can also be calculated, as shown in Equation (26)

$$n = \frac{e}{1+e} \quad \text{.......... 26}$$

To calculate the saturated density, Equation (28) is used.

$$t\ \gamma_{sat} = \gamma d + \gamma w * n\ (5) \quad \text{.......... 27}$$

3.6 Input Parameter

Table 5: Properties of Unit Weight for Arjo Rockfill Dam(source OWWDSE Design reports)

No	Properties Unit weight	Unites	Casing/ Shell material KN/m3	Core KN/m3	Foundation KN/m3	Fine Filter	Transition Filter	Core Around Conduit
1	γ moist	KN/m^3	21.5	18	19.5	19	19.5	18.5
2	γ dry	KN/m^4	19.75	19.92	19	18.6	17.92	17.92
2	γ sat	KN/m^5	22.79	21.68	21	21.61	20.14	20.14
3	γ sub	KN/m^6	12	8.5	10	10	10.5	9.5

Table 6: Properties of Strain and Strength for Arjo Rockfill Dam(source OWWDSE)

No	Properties Strain and strength	Unites	Casing/ Shell material KN/m3	Core KN/m3	Foundation KN/m3	Fine Filter	Transition Filter	Core Around Conduit
1	C	t/m^2	0	30	25	1	1	30
2	Ø	Degrees	41	29	30	35	35	30
3	D	KN/m^2	15	5	2	2	2	2
4	E	KN/m^2	120000	83400	170240	170240	170240	83400
5	V		0.15	0.4	0.4	0.35	0.35	0.34

Table 7: Properties of k for Arjo Rockfill Dam(source OWWDSE Design reports)

No	Properties of conductivity	Unites	Casing/ Shell material KN/m3	Core KN/m3	Foundation KN/m3	Fine Filter	Transition Filter	Core Around Conduit
3	Model Used		Van Gen	Van Gen	Van Gen	Van Gen	Van Gen	Van Gen
4	Conductivity	cm/s	4.82E-01	1.07E-06	1.75E-06	0.0642	0.014	0.00006515
5	Conductivity	cm/day	4.16E+04	9.24E-02	1.51E-01	5546.88	1209.6	56.2896
6	Conductivity	m/s	4.82E-03	1.07E-08	1.75E-08	6.42E-04	1.40E-04	6.52E-06
7	Conductivity	m/day	4.16E+02	9.24E-04	1.51E-03	5.55E+01	1.21E+01	0.00092448

γ_d: dry unit weight γ_{sat}: saturated unit weight, k: coefficient of permeability v: Poisson's ratio E: Young's modulus, c: cohesion: ϕ natural friction angle, φ: dilatancy, n; porosity

The parametric study was also carried out by varying the parameters: E' and φ', wherein the E' values of the shell and core were varied without altering the E' value of the subsoil. The φ' was varied from 29° to 41°. The flow functions were different for each of the cases considered Full (high) reservoir level of the dam, Rapid drawdown duration of the dam.

The representative earth dam which was considered for the finite element analysis was 50 m in height with the side slope of 1 in 2 on both the downstream sides and 1 in 3.5 on the upstream. The high reservoir level was 47.5 m high along with 9 m foundation. Suitable hydraulic boundary conditions were assigned to the upstream side, before the start of upstream face and the last portion of the subsoil part. Fig. (2) shows the earth dam section considered in the study.

3.7 Model Assumption

When considering flow inside a medium like cohesion less soil, seepage forces will act on the grains. If not well compacted or well graded a loss of fines will occur, meaning they will follow the flow, thus causing suffusion.

According to Sybille, *et al*. (2015), there are different types of internal erosion. Whereas in this study suffusion is being studied. Some assumptions are used:

1) The internal erosion mechanism is horizontal and is only localized in the core.
2) A plane strain assumption of the erosion is used.
3) No self-healing of the core.
4) No subsidence occurs when lowering the void ratio.
5) Only fine particles (silt) will erode from the soil matrix.

The first hurdle is classifying the material for constant m since it is not specified as clay, silt or sandy material. For void ratio 0.215 which was calculated from in situ bulk density and following the compact density being 2.42 t/m^3. The specific surface was assumed to be 74m^2 /g (Clay).

The constant m was iteratively found to be 1.175 when finding the permeability 1.17E-7 m/s as measured for the core in its construction phase. With the assumption that only silt will erode, the test is set to where the fines are totally gone which is approximately 39% mass which coincidentally makes the void ratio about 1.

This enables the iterative process to have a goal where the m and specific surface has sandy characteristics the iterative process was started where the specific surface was decreased to resemble a sandy material (mass has eroded and thus specific surface has decreased), going from 0.1 to 0.01m2/g.

This was performed with the constant m as well going from 1.175 to 0 which resembles sandy material. The results of this expected study are shown in Fig. (19).

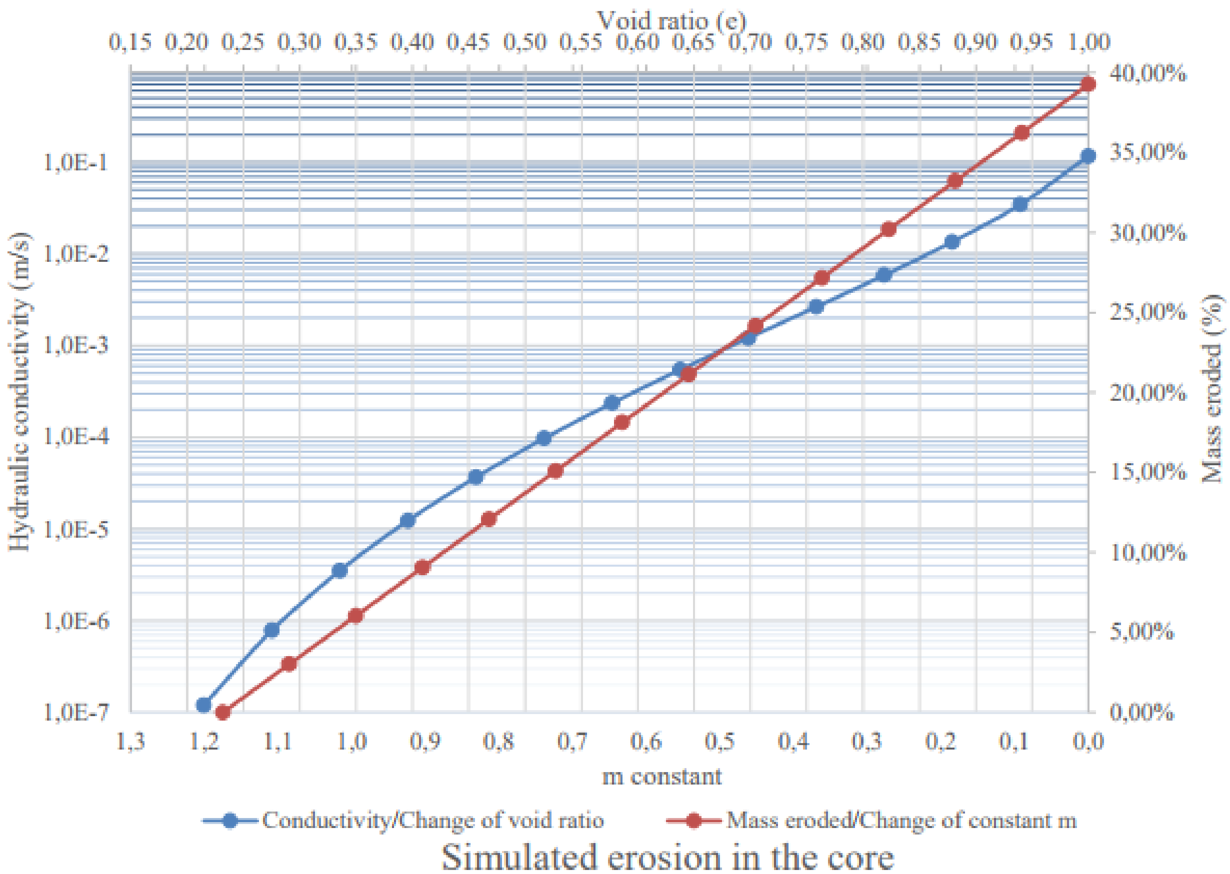


Figure 19: Conductivity change with change of void ratio and how much mass has been eroded.

By regarding Fig. (19) it is possible to extrapolate parameters for conductivity of the eroded zones which are shown in Table 8

$$y(x) = y_1 + \frac{x-x1}{x2-x1} * (y_2 - y_1) \quad \ldots\ldots 28$$

3.8 Hydraulic Conductivity vise Void Ratio

These data show two opposite trends for hydraulic conductivity as a function of void ratio: while the hydraulic conductivity increases with increasing void ratio for any single sediment, fine grained (small pore size) soils exhibit much lower hydraulic conductivity, even at higher void ratios than coarse grained (large pore size) sediments. In fact, the Kozeny-Carman equation highlights the importance of “pore size” rather than “porosity” on fluid transport (anticipated by the Hagen–Poiseuille equation for a single tube).

Table 8: Parameters changed in the core for different void ratios

E	-	0.215	0.25	0.3	0.35	0.4	0.5	0.6	0.94	1
K	m/s	1.17E-07	7.0E-07	1.07E-06	5.0E-06	1.21E-05	6.4E-03	1.28E-02	3.45E-02	1.17E-01
m	-	1.175	1.15	1.07	0.95	0.84	0.7011	0.56	0.09	0
Massa eroded	%	0	0.84	3.51	7.52	11.19	19.99	23.73	36.45	39.25
Dry unit weight	t/m^3	19.92	19.36	18.47	17.92	17.3	16.33	15.12	12.42	12.74
saturated	t/m^3	21.68	21.36	20.8	20.5	20.14	19.46	18.875	17.32	17.1

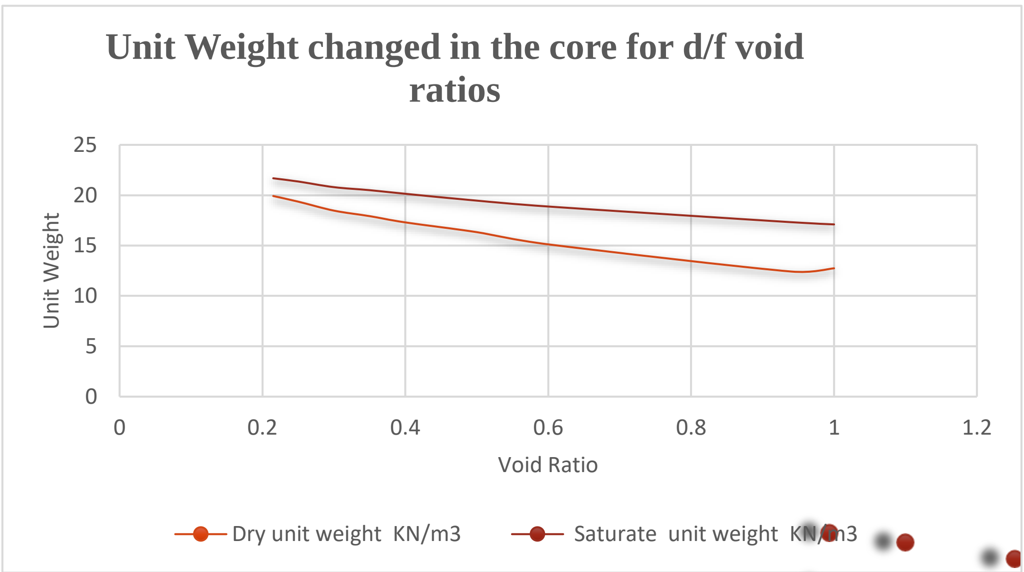


Figure 20: Relation of void ratio and unit weight of core material

4 RESULTS AND DISCUSSION

4.1 Seepage Analysis

Regarding the permeability of soils, in the semi saturation status, because of the prevention of water movement by the air bubbles existed in soil is lower than the saturation status. Seepage analysis resulted for these two types of permeability is differing from each other. In order to prevent this error, a function can be defined in which the permeability can be changed linearity in that range and in outlet of the range, it would be fixed. This function can be defined so that in p=0 kpa, the permeability coefficient equals to the K_{sat} and in p= -100kpa, the permeability coefficient is k_{sat} /100. Fig. (21) show a typical function between pressure and permeability coefficient.

After allocating functions for permeability, boundary conditions for the model are defined. The water height for the permanent seepage is considered as normal water surface in Arjo Dhidhessa rockfill dam is (47.5 m). Total depth of permeability layer considered in foundation for this model is about 5 meters and constructed with impermeable clay soils.

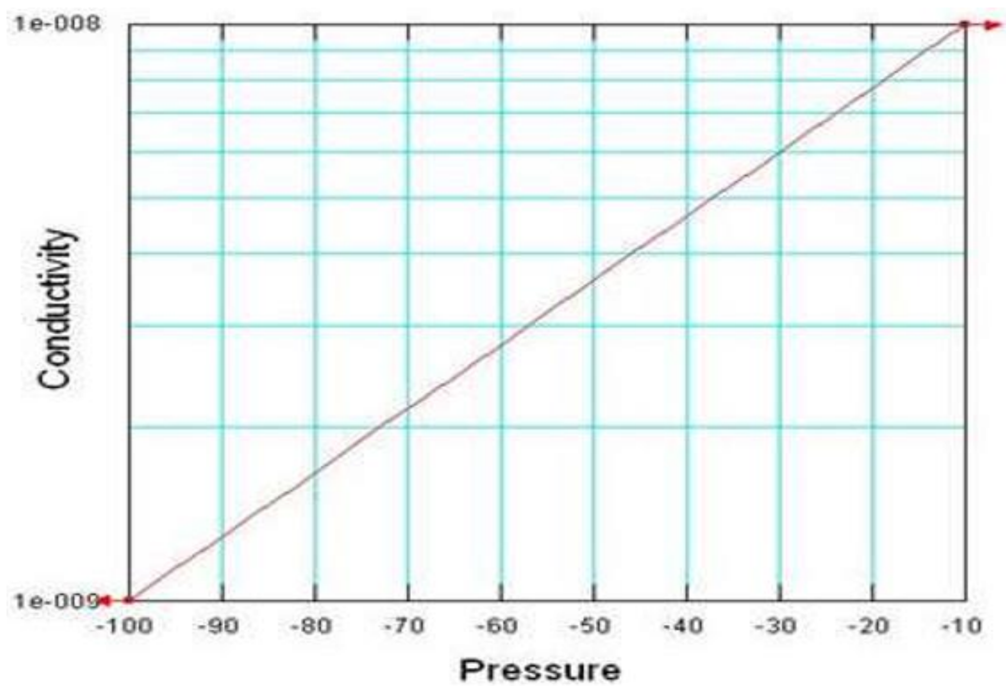
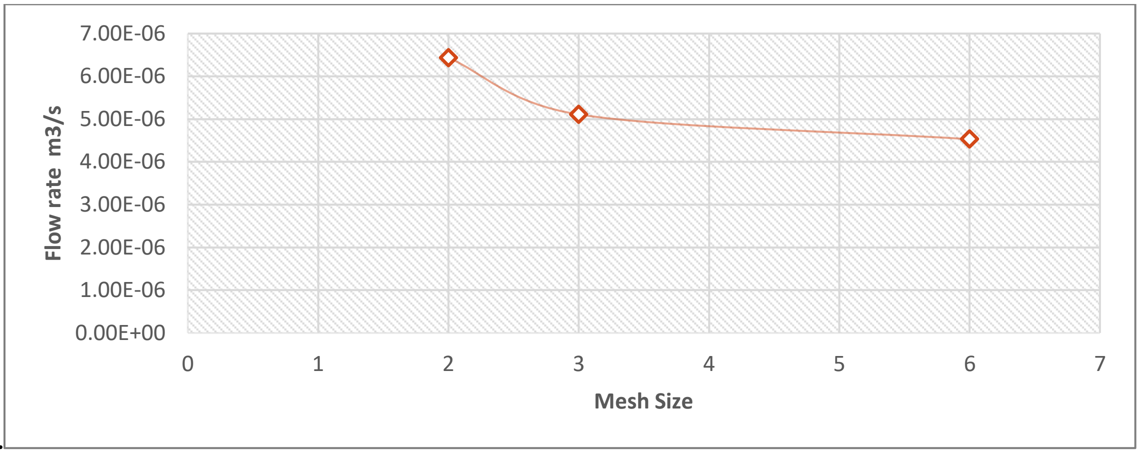


Figure 21: The relation between pressure and permeability

In order to evaluate the type and size of mesh size on the total flow rate and total head through the dam cross section, three mesh size such as coarse, medium and fine mesh is considered fig. (22).

According to Mamizadeh and Karim (2013), stated Width and length of mesh for large, medium and fine are 6 × 6 m, 3 × 3 m and 2 × 2 m respectively.



.

Figure 22: Effect of mesh size on the amount of flow rate of the dam

According to analysis conducted the leakage rate difference is negligible under the different mesh size and flow rate of leakage for Arjo Didessa rockfill dam equal to 6.44E-06 m^3/s, 5.11E-06 m^3/s and 4.53E-06 m^3/s for (2x2,3x3 and 6x6) respectively. For the case of this study to analyze the seepage flow rate 3x3 mesh size is applied to make the calculation more reliable and to minimize the analysis time.

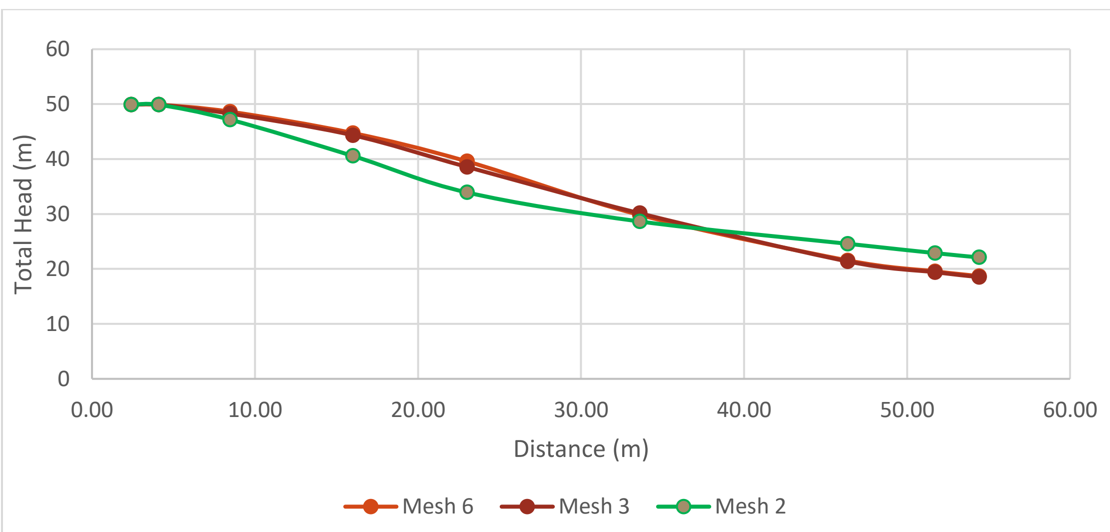


Figure 23: Effect of mesh size on the total head

Fig. (23) indicate the change in mesh size at the time of analysis by GeoStudio has a negligible change on the total head.

In The analysis conducted for seepage flow rate of Arjo Dhidhessa rockfill embankment dam two model were considered based on compaction rate, material properties and physical condition of the site.

Case A: For case A the material properties are classified in to four class for analysis

Core, shell, foundation, filter, this model is more similar with the model analyzed during design period. In this case the fixed structure which is embedded in the body of the embankment dam is not considered by the designer.

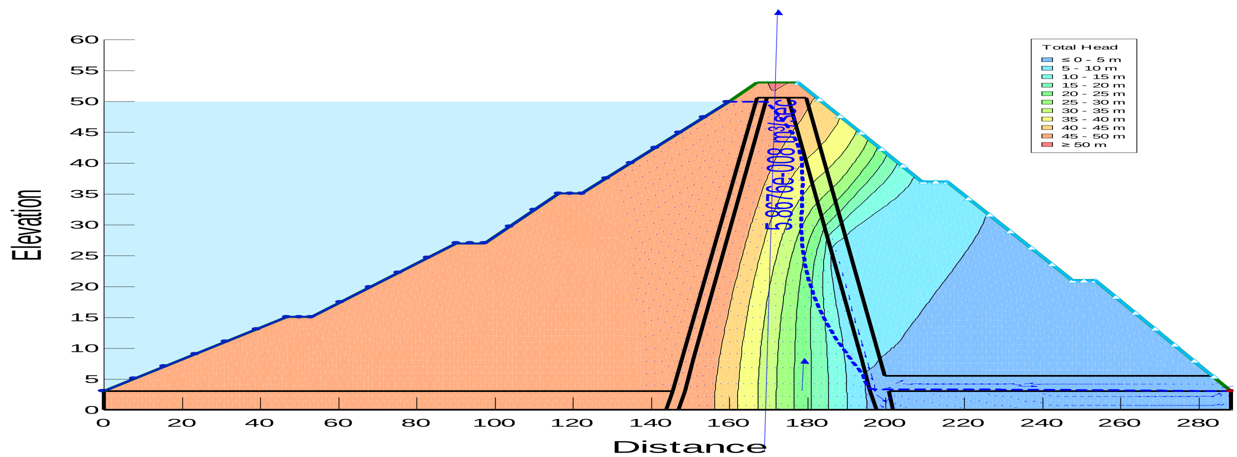


Figure 24: Model considered for case A

Case B: For the case of Case B Model additional region is considered. Which is core around conduit. In the analysis of this model core classified in to two regions based on compaction rate.

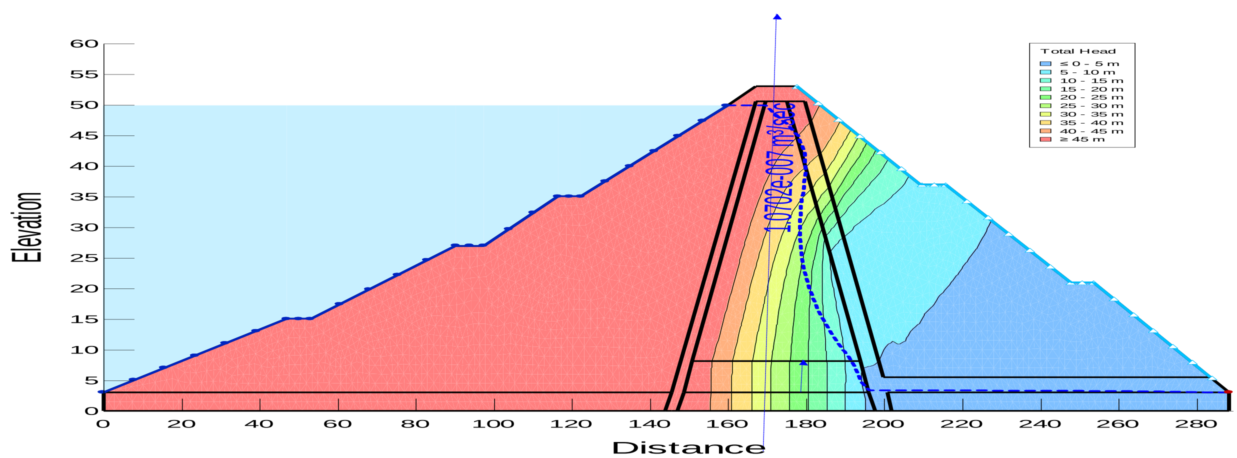


Figure 25: Model considered for case B

As the test conducted on the field indicated the core around conduit at chainage 0+300 from left abutment is not well compacted as to like other parts of the dam. Due to rigid structure embedded in the body of the dam the compaction of the core adjacent to this structure is compacted by manpower. However, the rest of the core part is compacted by 17,000 tone compacter machinery.

As seen in fig. (24 & 25), low permeability clay core with good resistance against water seeping into the dam so that the flow inside the core will drop sharply. Thus, the downstream slope of earth dam always remains dry.

As the analysis indicated for the case of model A is the flow rate is 5.8676e008 m^3/s which less than allowed seepage flow rate for the embankment dam.

For the analysis of case B, the model result shows the flow rate is completely different from the first case A. The flow Rate is 1.1e-007 m^3/s. This means the flow rate is increased from 5.867e-008 m^3/s to 1.1e-007 m^3/s from this we can conclude that the flow rate is increase by 33% due to the less compaction of core layer near the conduit.

4.1.1 Pore Pressure

As determined value indicated on figure (30 up to 35) the pore pressure in the embankment dam were slightly changed. Indicated figures show that, as the void ratio and permeability of the embankment changed due to different reason the contour line of the pore pressure distribution was shifted its position. These situations happened due to increase of void space in the downstream parts of the embankment which is filled with water.

The phreatic line is also gradually raised. This means the saturated part of the dam embankment is increased. At the initial stage the downstream is unsaturated due to fall of the phreatic line in the filter zone. However, as the discharge of the seepage through the dam body is increased the phreatic line also changes its position.

The slight change that occurred in pore pressure may be due to several reasons. The first reason is that boundary conditions do not change because of the steady state calculation. If boundary conditions would change the calculation would have to be performed by using a transient flow analysis.

Since only the core being affected by piping it will not be able to create a great flow in the pipe due to the surrounding soil with lower permeability. Thus, if the pipe would be extended through the filters the results might have become a bit different.

Table 9 Seepage Flow rate result under permeability change (SEEP/W)

E	-	0.31	0.35	0.4	0.5	0.6	0.94	1
K	m/s	1.07E-06	5.00E-06	1.21E-05	6.40E-03	1.28E-02	3.45E-02	1.17E-01
Q	m3/s	1.58E-06	1.90E-06	2.10E-06	3.46E-04	5.83E-04	1.42E-03	1.46E-02

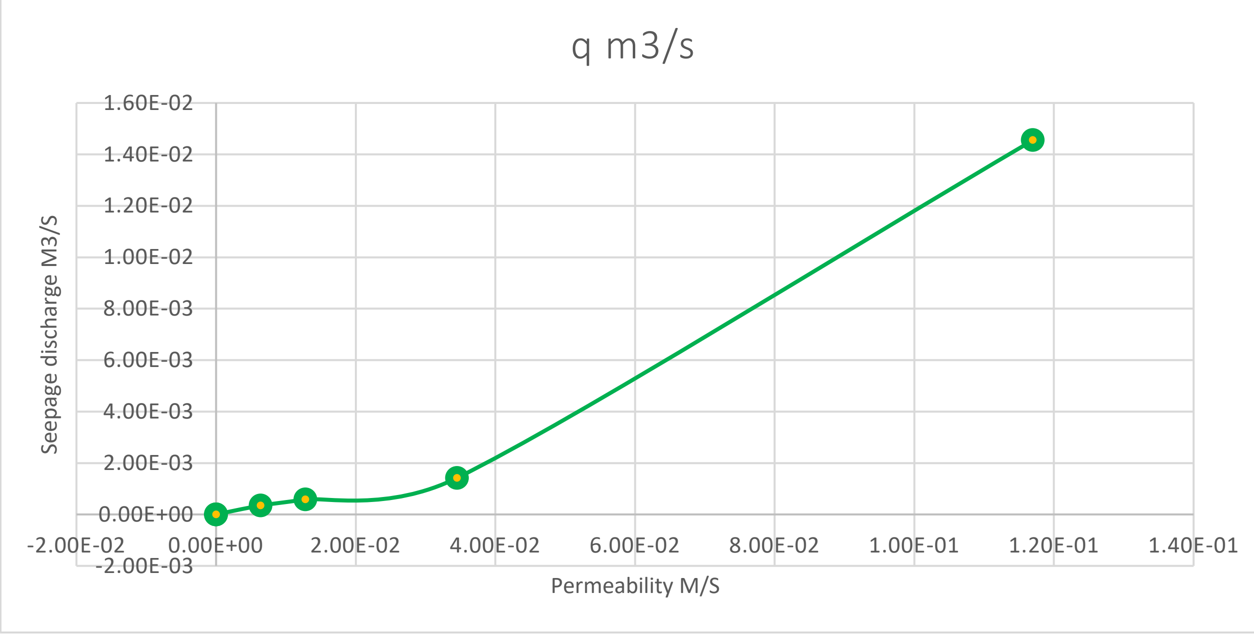


Figure 26: Flow rate increase as the permeability of core around conduit rise (SEEP/W)

Table 9 and fig. (26) shows that the analysis of seepage by Geo studio SEEP/W model the discharge of seepage gradually increased from initial void ratio (0.31) 1.58e-6m^3/s to 1.455e-01m^3/s at 1 void ratio.

At initial void ratio (0.31) the analyzed flow rate value is 1.58E-06 m3/s but after the assumption of internal erosion is takes place due to concentrated flow along the conduit internal void ratio (1)m the flow rate value is 1.46E-02 m3/s

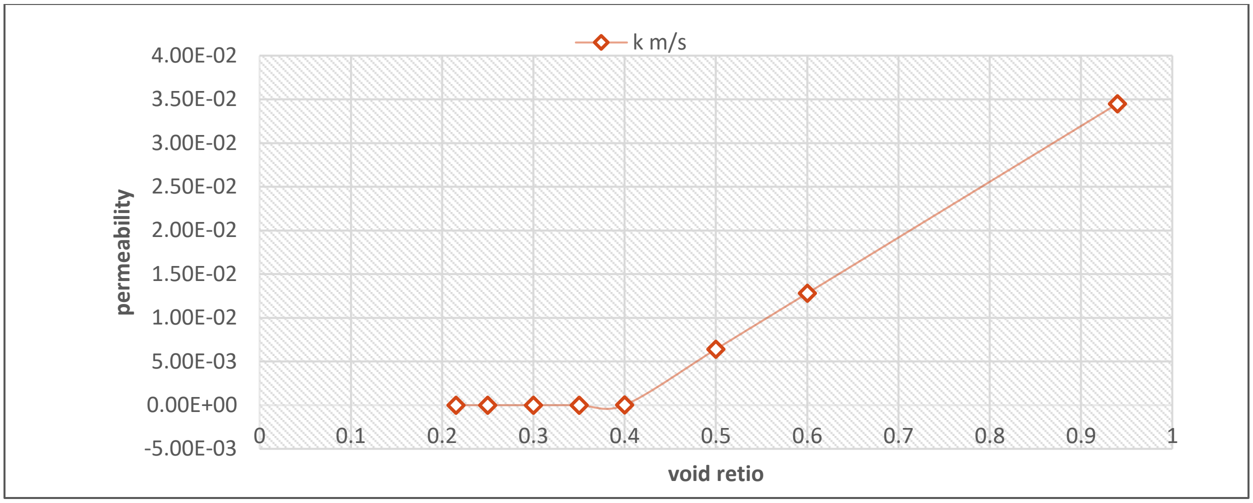


Figure 27: Graph showing the permeability and void ratio for clay core material

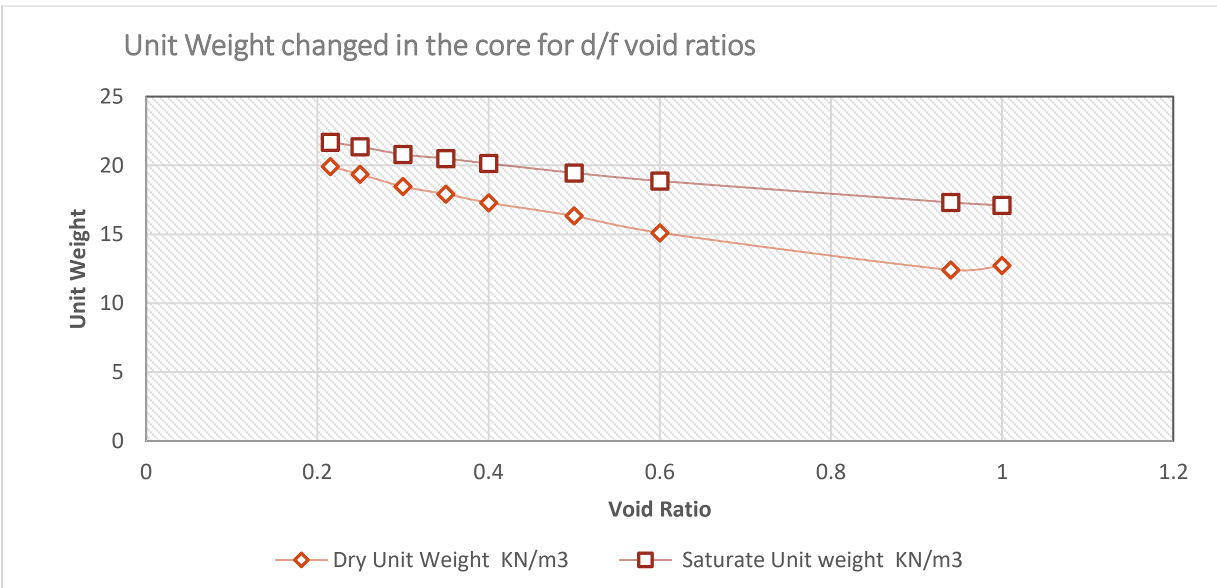


Figure 28: Unit weight and Void ratio relation for clay core

As indicated on fig, (28) the void ratio of the clay core and unit weight of the clay core are inversely proportional. Therefore, as the void ratio increase from 0.215 to 1 due to internal erosion the unit weight of the clay core is decreased from 19.92kn/m3 to 12.74KN/m3 and from 21.68kn/m^3 to 17.1KN/m^3 for dry unit weight and saturated unit weight respectively.

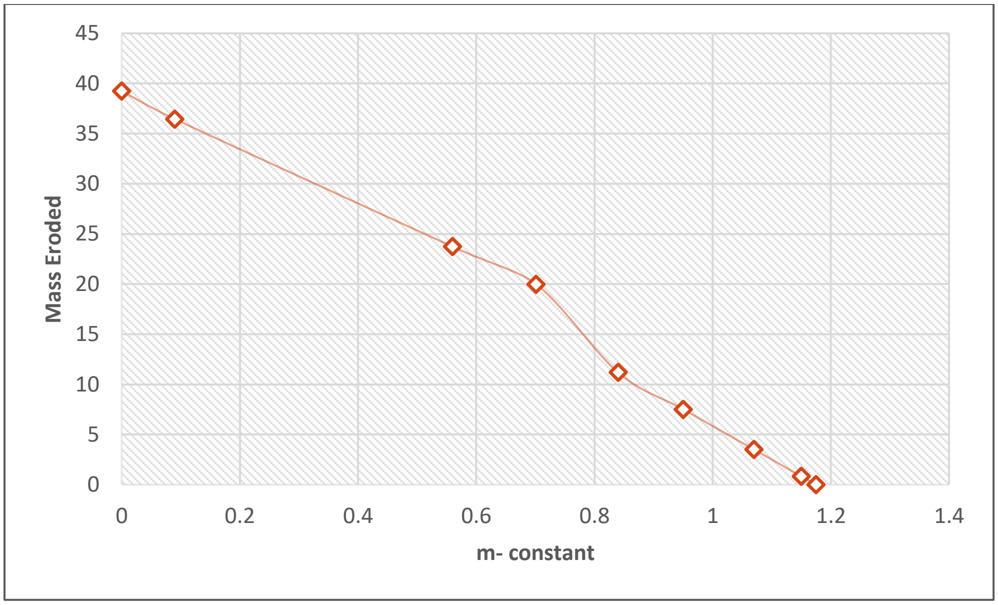


Figure 29: m- constant and mass eroded

constant *m* which refers to the difference in specific surface between materials and was evaluated and shown in Fig. (29). Specific surface area of the particle has significantly influenced the amount of mas eroded from the embankment bod of the dam. As indicated in the by analysis as the m value decrease from 1.2 to 0.1 the percentage of mass eroded raised from 0 to 39%

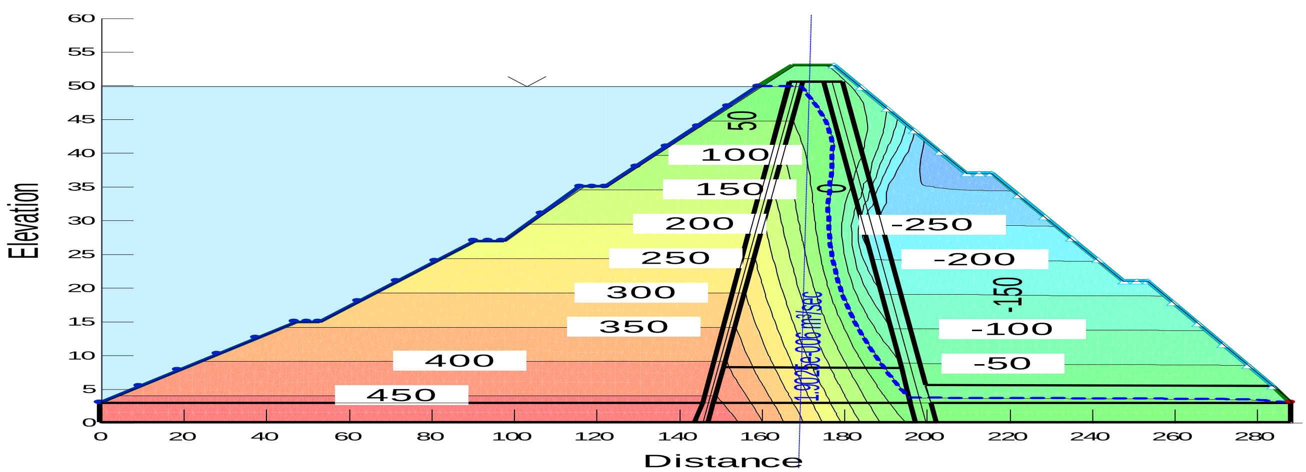


Figure 30: Seepage analysis for the 0.35 void ration core around conduit

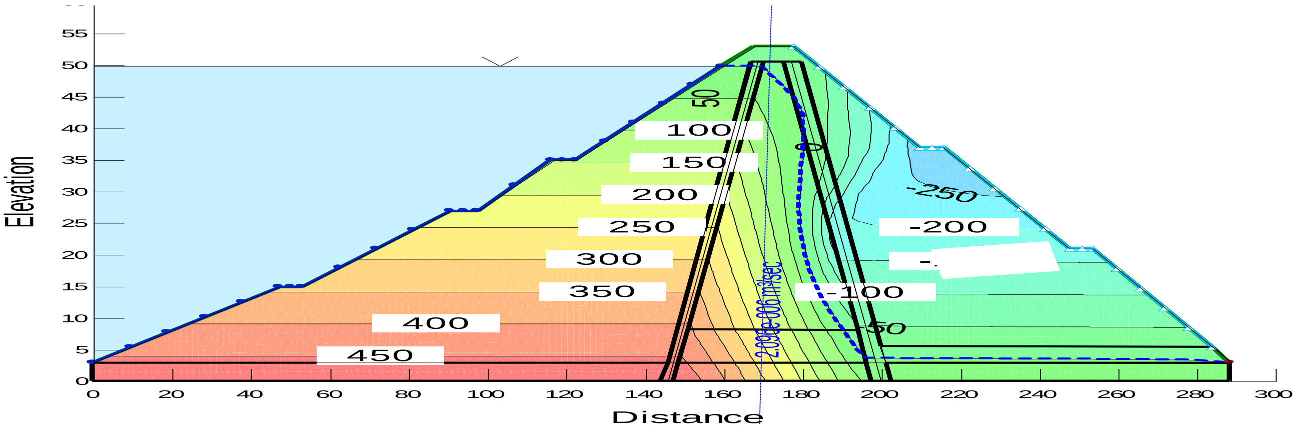


Figure 31: Seepage analysis for the 0.4 void ration core around conduit

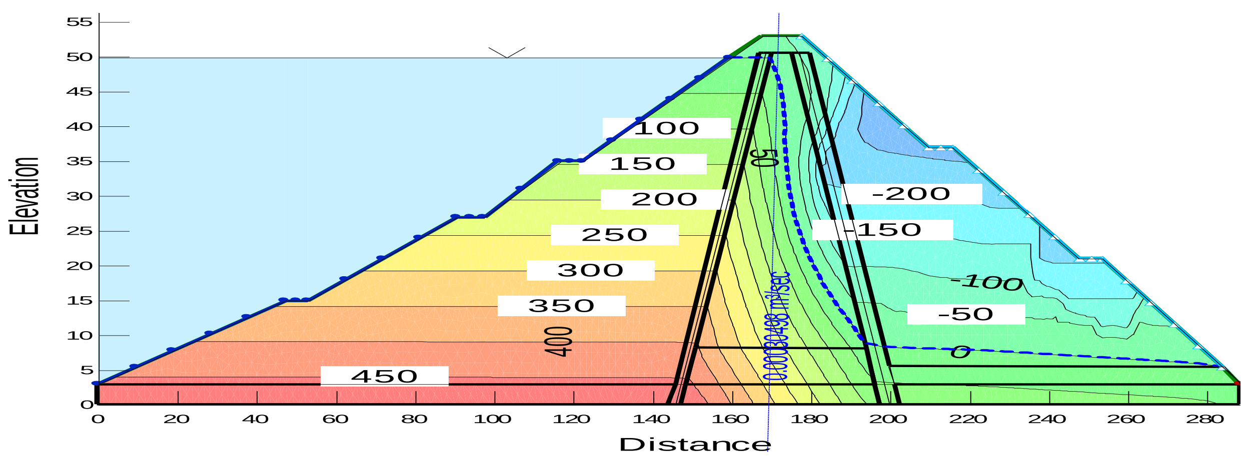


Figure 32: Seepage analysis for the 0.5 void ration core around conduit

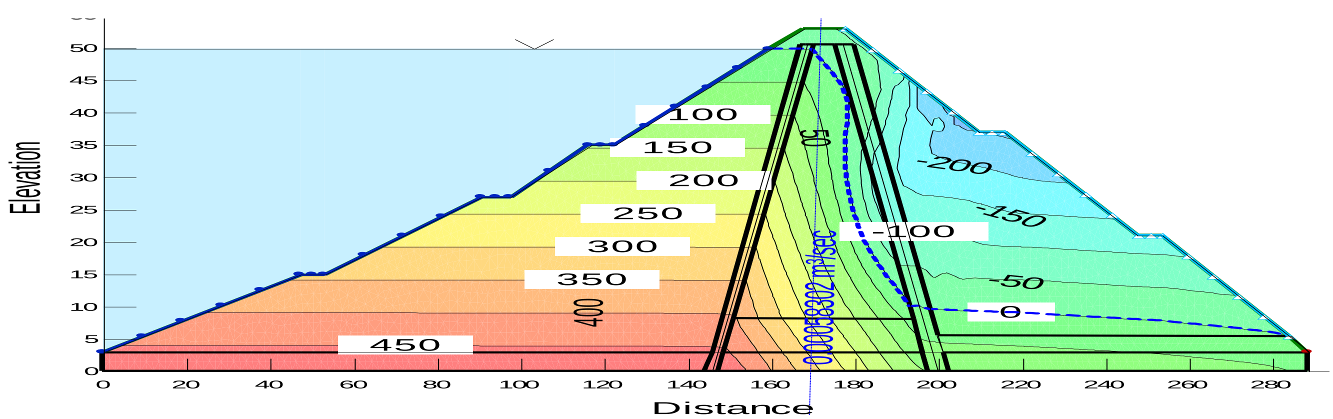


Figure 33: Seepage analysis for the 0.6 void ration core around conduit

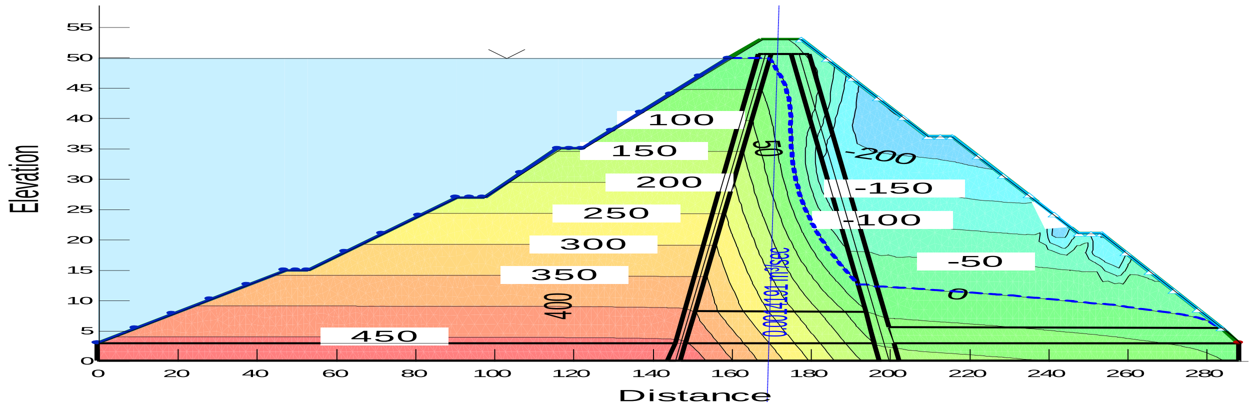


Figure 34: Seepage analysis for the 0.94 void ration core around conduit

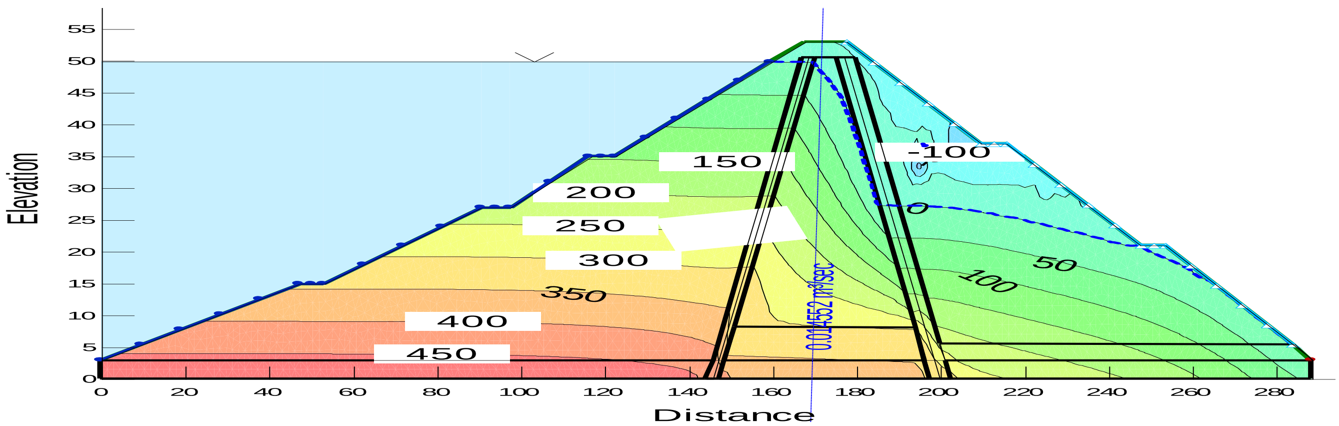


Figure 35: Seepage analysis for the 1 void ration core around conduit

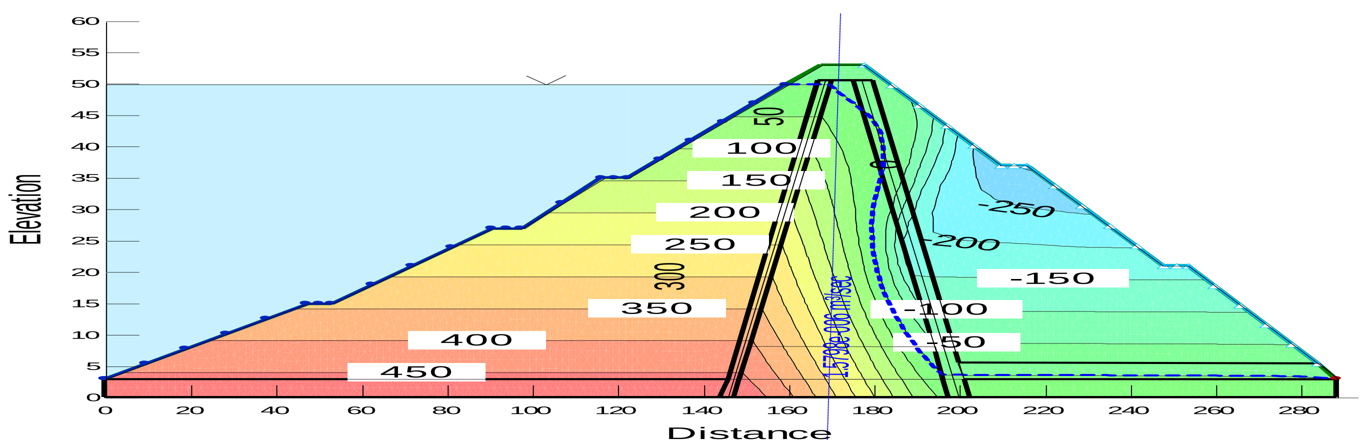


Figure 36: Seepage analysis for the core initial void ration (0.31)

The high permeability where e=1 is rather unrealistic, if the water would be able to flow at 1 m/s in a medium such as soil it would probably bring with all material due to the seepage forces affecting the soil, it does however work in a theoretical calculation such as this. From the calculation conducted to require the permeability at the e= 1 the permeability is 1.17e-01cm/s (the value of k) is obtained.

According to Lambe and Whits (2016) this value is classified in the higher permeability range. See the fig. (56). The changed permeability, connected to the void ratio, was also tested for a lower void ratio (e=0.35), if the pipe would theoretically have undergone settlements. Fig. (34) shows that the permeability is within the area of sands and gravel (k≈0.01m/s) which makes the chosen permeability in the analysis plausible.

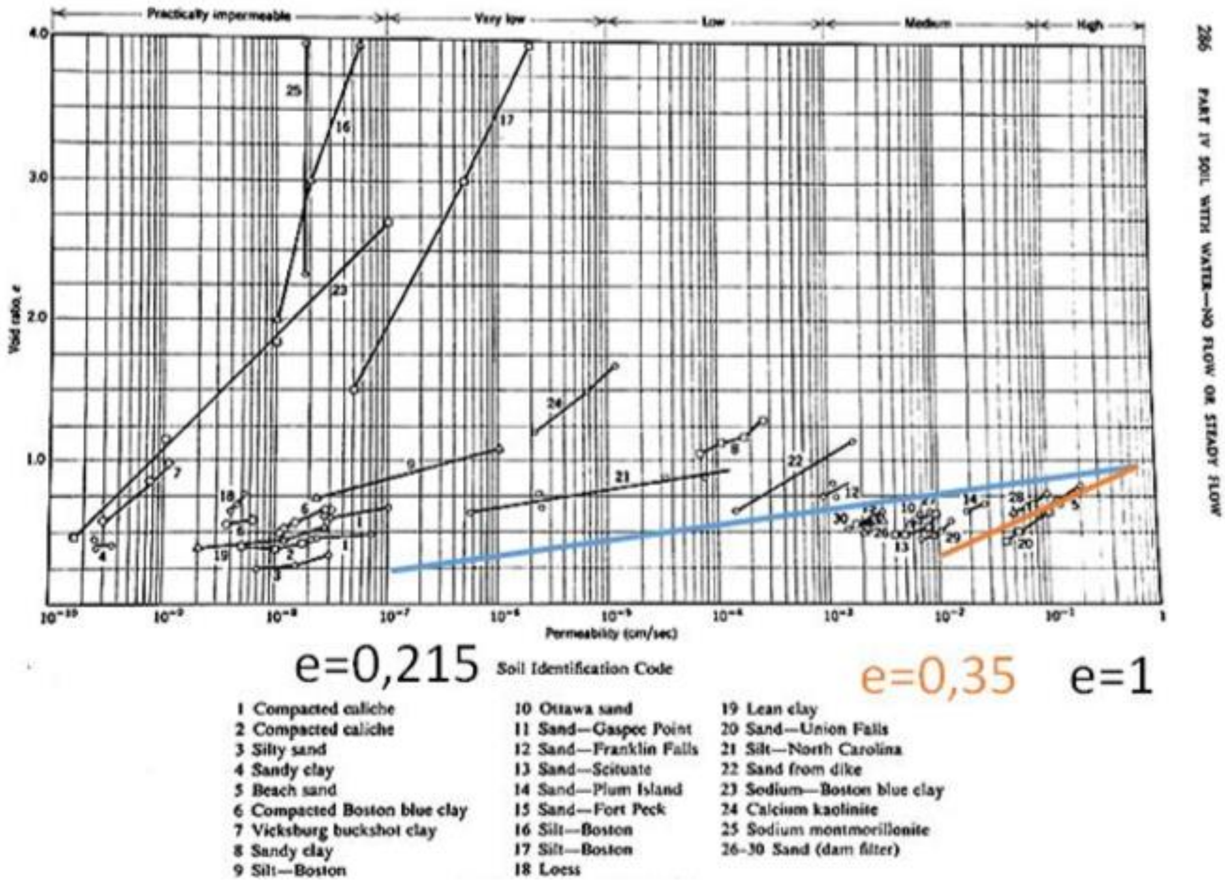


Figure 37: permeability test data.

4.2 Slope Stability

4.2.1 Slope/w

Materials property related to slope stability analysis is shown in table 5. Location of critical slope surface and safety factors obtained by different methods.

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.

4.2.1.1 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.

Also, in the total stress, the core is considered without pore water pressure. As shown on fig. (37 & 38) the analysis conducted by geo studio 1.944 for downstream and 2.835 for upstream value of factor of safety is obtained, according to the obtained values, it was observed that resulting safety factors in upstream are safer than safety factors in downstream.

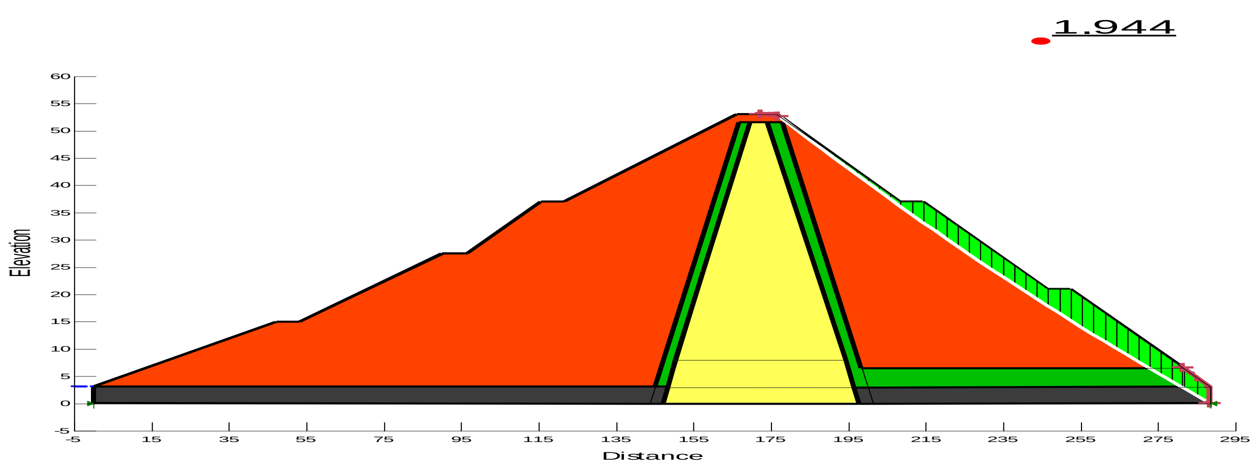


Figure 38: End of construction downstream face

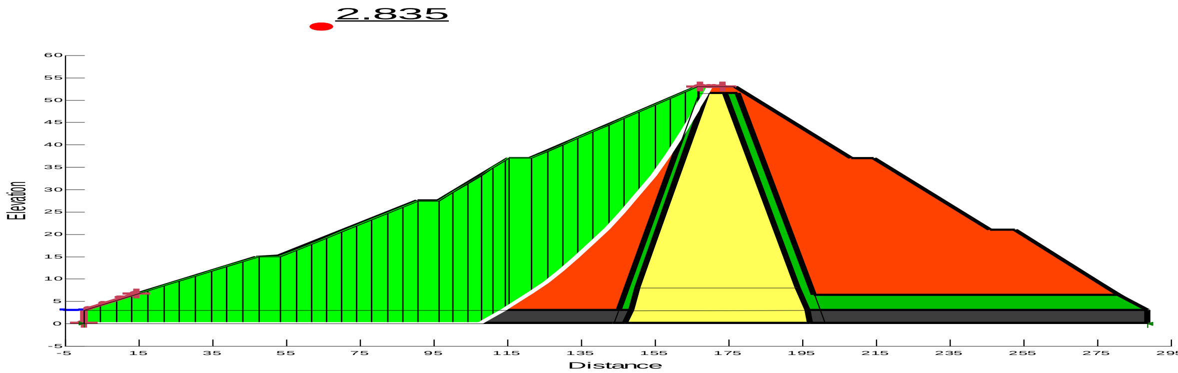


Figure 39: End of construction Upstream face

According to (USACE,2003) the minimum value for end of construction is 1.3 for both upstream and downstream. The results of stability analysis show that the derived values of factor of safety in end of construction condition case are higher than the required minimum values of factor of safety recommended by International standards.

4.2.1.2 Steady-State Seepage Loading Condition

After the construction of the dam and passage of the time required, Steady-State seepage condition were established in dam body and foundation and over time the consolidation will take place in dam body.

The materials used in clay core are examined in consolidation and drained (CD) mode to let the materials to be fully consolidated and also to have the opportunity for drainage. In fig. (40) the phreatic line obtained from SEEP/W is displayed for a dam with a height of 47.5 meters. Fig. (40 & 41) is shown the Steady-State seepage analysis in upstream and downstream for a dam. Accordingly, 2.866 for upstream and 1.961 for downstream factor of safety is obtained.

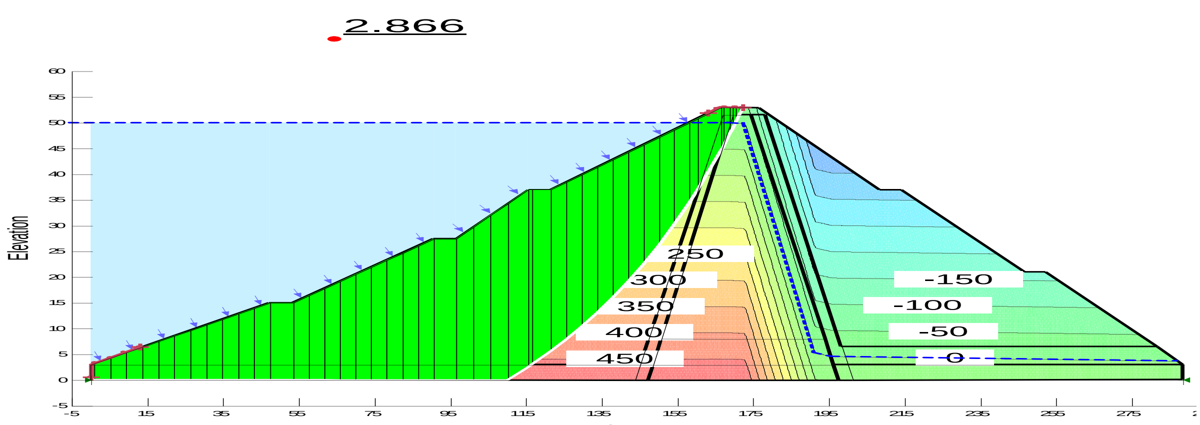


Figure 40: Steady state seepage full reservoir level (upstream slope)

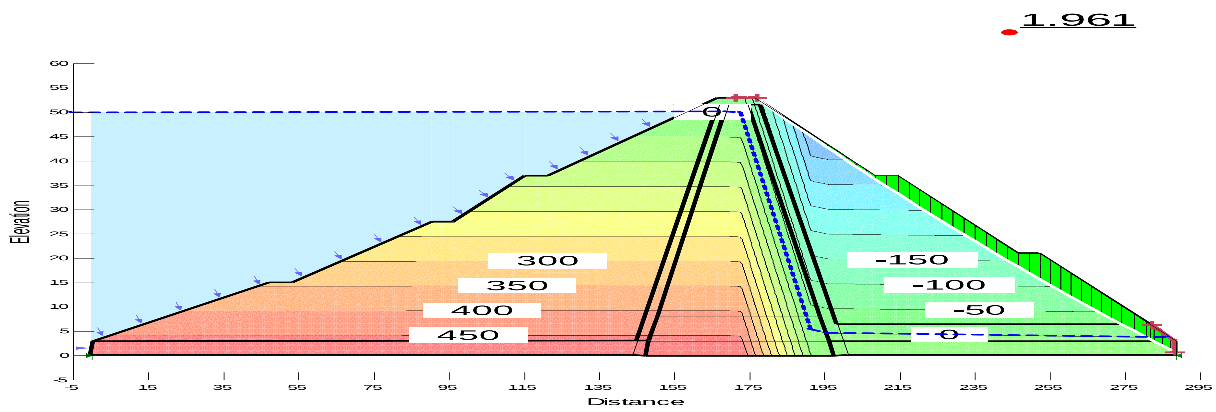


Figure 41: Steady state seepage full reservoir level (downstream slope)

According to (USACE,2003) the minimum value for end of steady state seepage is 1.5 for both upstream and downstream. The results of stability analysis show that the derived values of factor of safety in steady state seepage condition case are higher than the required minimum values of factor of safety recommended by International standards. Therefore, the dam is safe

4.2.1.3 Rapid Drawdown Loading Condition

If the water level behind rapidly 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.

.

In the rapid Drawdown of reservoirs, sharp reduction of the pore water pressure was 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. Accordingly, 1.828 safety factor for upstream RDD.

Based on the result we obtained from the analysis if the reservoir water level is decreased by one third of the dam height the upstream safety factor is decreased by 55.6 %. However, the obtained safety factor value for RDD is higher than the required minimum values of factor of safety recommended by (USACE,2003) which is 1.3.

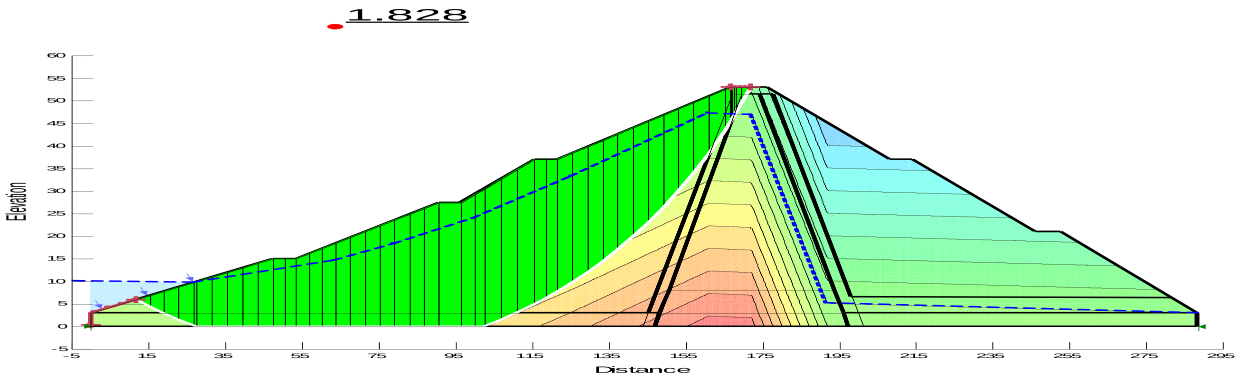


Figure 42: Factor of Safety for Rapid drawdown (upstream slope)

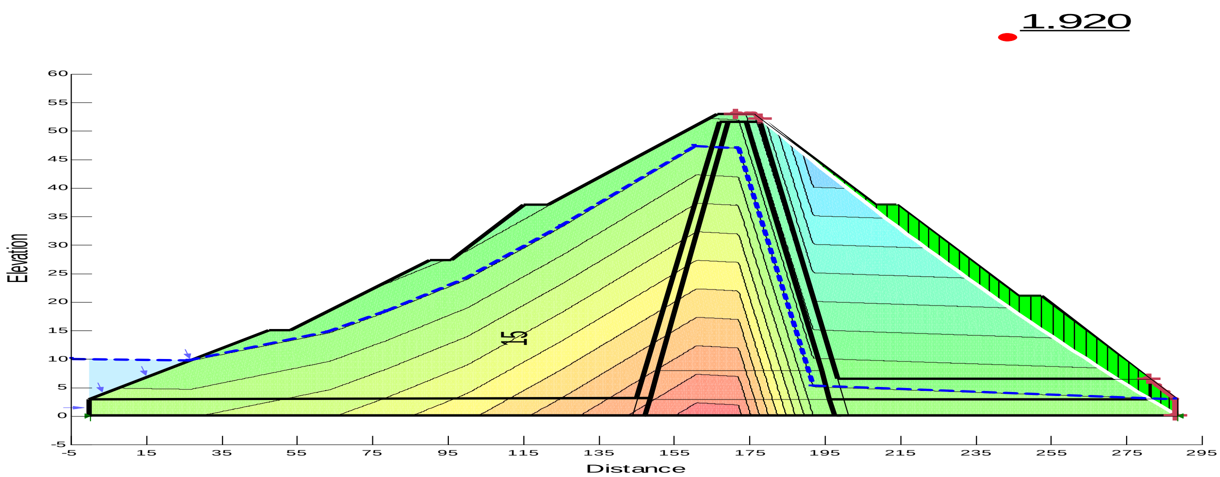


Figure 43: Factor of Safety for Rapid drawdown (downstream slope)

Table 10: Showing factor of safety for slope analysis

Analysis Method	D/S slope End of construction	U/S slope End of construction	D/s Slope in steady sate leakage	U/S slope in steady sate leakage	U/S Sudden Draw Dawn
The Morgenstern	1.944	2.835	1.961	2.866	2.345

As the result indicated the value of factor of safety obtained from the analysis the slope stability of Arjo Didessa Rockfill a dam is full fill the required international standard.

As the result obtained indicated the layer of less compacted clay soil found in the core around conduit does not significantly affect the stability of the dam. Because GeoStudio software is used only the parameter unit weight, cohesion and angle of internal friction. This means seepage change occurred in the core have no direct effect on the stability of the dam. As we observed in the analysis above the factor of safety for different loading condition by geo studio slope/w fulfill the international standard. Even if the layer of core around conduit is less compacted in regard to other place there is no significant impact on the overall stability. Because the thickness of the layer which is less compacted is small when compared to the overall depth

4.2.2 Plaxis 8.2

Stability the overall stability was investigated for the dam with different dimensions on the pipe shown in Fig. (44) with altered void ratio e=1 and two pipes.

4.2.2.1 End of Construction Condition

The stability of the arjo Dhidhessa rockfill dam has also been analyzed for end of construction and the result of the analysis shows the dam achieved the allowable recommended safety factor of 1.3. The safety factor for central clay core dam is shown in fig. (44) shows that, the safety factor of the dam during end of construction is ∑Msf =2.021. But Relative to SLOPE/W (2.835) the factor of safety obtained by PLAXIS 8.2 is lesser.

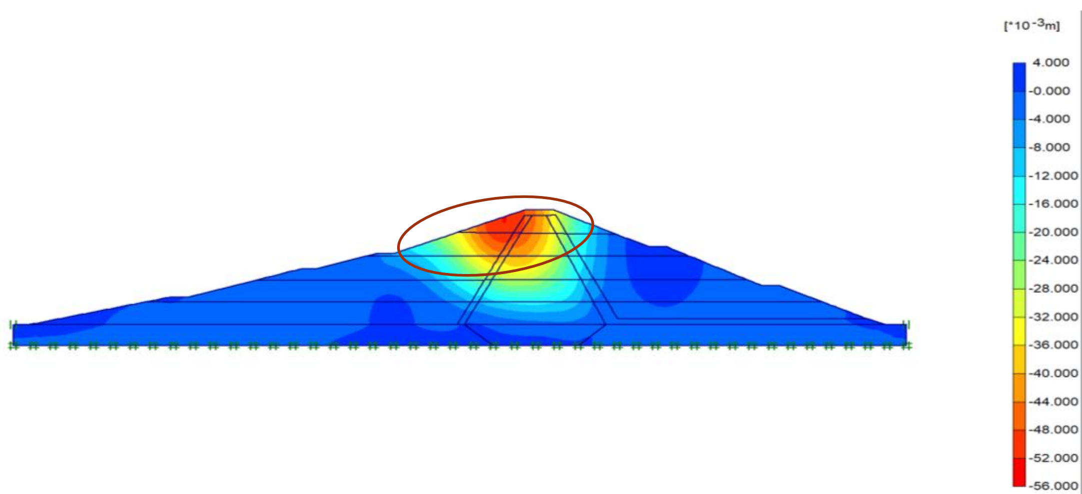


Figure 44: Possible Critical failure surface of dam (End of construction)

PlAXIS 8.2 considers additional parametes such as Elastic module,Poison ratio, Void ratio,cohesion, angle of internal friction ,saturated and dry unit weight of the material to analize the slope stability. However SLOPE/W consider only unit weight, cohesion and angle of internal friction. That is why under the same loading condition (end of construction) safety factor of safety factor by PLAXIS is smaller than that of SLOPE/W.

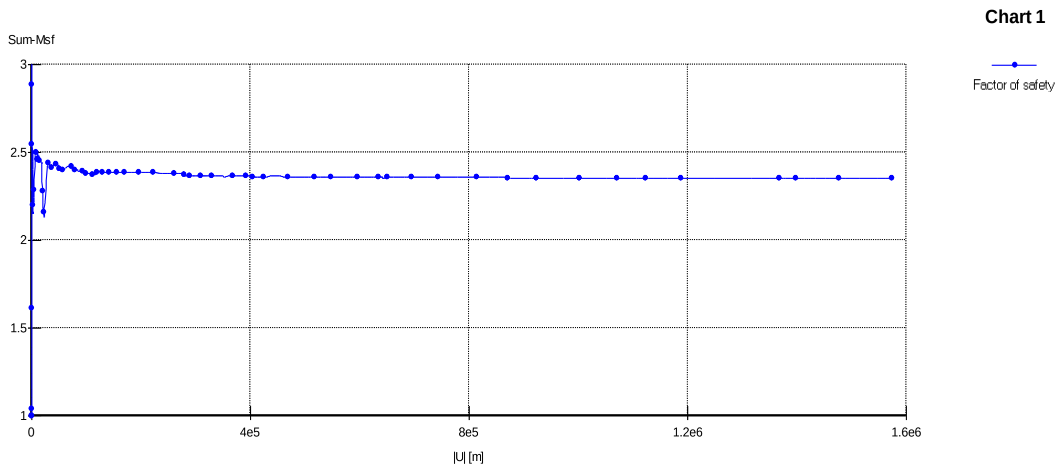


Figure 45: Plots of factor of safety CCD at end of condition

4.2.2.*2* Steady State Condition

Under steady state loading condition, the analysis performed by PLAXIS 8.2 as shown on the fig. (46). Accordingly, the computed factor of safety under this condition was ∑Msf = 1.871 which is satisfied the recommended minimum (1.5), factor of safety requirement suggested by USACE, (2003). Nonetheless relative to SLOPE/W (2.866) the factor of safety obtained by PLAXIS 8.2 is lesser. PlAXIS 8.2 considers additional parametes such as Elastic module,Poison ratio, Void ratio,cohesion, angle of internal friction ,saturated and dry unit weight of the material to analize the slope stability. However SLOPE/W consider only unit weight, cohesion and angle of internal friction. That is why under the same loading condition (end of construction) safety factor of safety factor by PLAXIS is smaller than that of SLOPE/W.

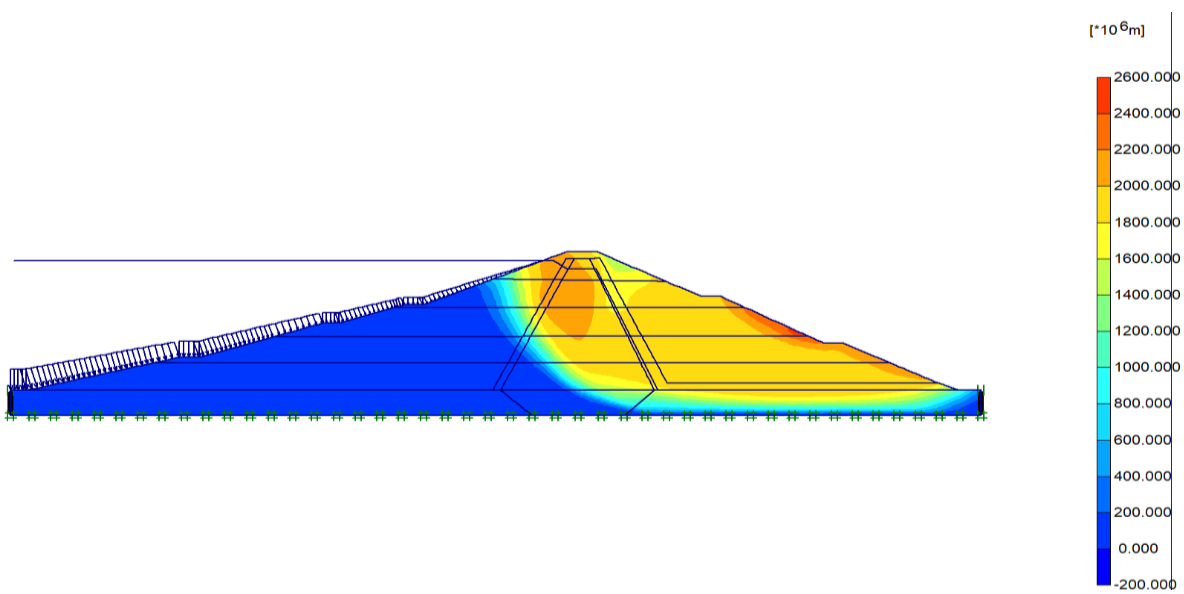


Figure 46: Possible critical failure surface of dam (steady state condition)

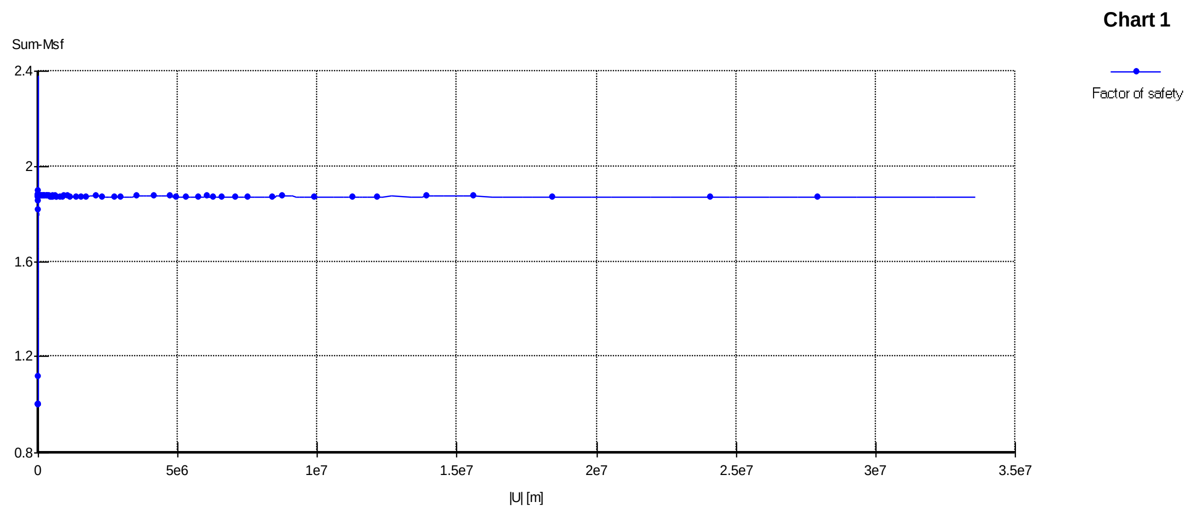


Figure 47: Plots factor of safety of dam during steady state condition

To end, Analysis performed by SLOPE/W and PLAXIS 8.2 software shows that slope stability of the of the Arjo Dhidhessa rockfill dam is higher than required standard. Factor of safety obtained by Slope/w for steady state condition ranges between ∑Msf (2.8-2.866). However, the safety factor obtained by PLAXIS 8.2 are ranges between ∑Msf (1.89 -2.02). The result indicates that, in both cases the dam has reasonably satisfied the minimum factor of safety (1.3) for upstream and 1.5 for downstream requirement recommended by USACE, (2003). The result obtained by PLAXIS 8.2 is more reliable and acceptable than slope/w. Because slope/w consider only three parameters which is cohesion, unit weight and angle internal friction. Nevertheless, PlAXIS 8.2 consider more parameters such us Elastic module, poison ratio, saturated and dry unit weight, void ratio and hydraulic conductivity for each material. These is similar for all loading condition

4.2.3 FS Under Different Permeability

The results for the global stability calculations performed in PLAXIS2D are shown in Table 12.

Table 11: Results of the safety factor calculation performed in PLAXIS2D.

Depth (m)	Diameter (mm)	Void Ratio 0.31	0.35	0.4	0.5	1
		Safety Factor				
15	50	1.8100	1.811	1.81	1.8116	1.812
	100	1.8117	1.8117	-		1.812
	200	1.8119	1.8112	1.8112	-	-
	300	-	-	1.822	1.831	1.833
	500	1.8129	1.821	-	-	1.83
30	50		1.831	1.822		1.825
	100	1.8183	1.821	1.821	-	1.822
	200	-	1.8212	1.822	-	-
	300	1.8233	1.8233	-	1.836	1.861
	500	1.8233	1.843	1.861	-	1.8484
45	50	1.861	1.883	1.902	1.885	1.9031
	100	1.8731	1.882	-	1.886	-
	200	1.875	1.881	-	-	1.883
	300	1.860	-	-	1.913	1.882
	500	1.862	1.784	1.784	1.884	1.917

Depth refers the depth of the dam from the crest and diameter is the diameter of the expected pipe occur in the embankment due to internal erosion.

Table 12 found the most critical failure surface to be on the upstream side of the dam, at lower permeability and lower void ratio however as the permeability and void ratio increase the critical failure surface is shift to the downstream side as seen in Fig. (48).

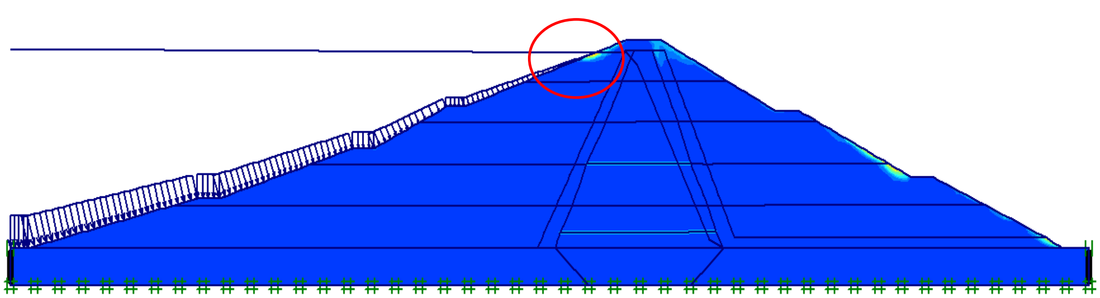


Figure 48: Upstream failure surface found in the model.

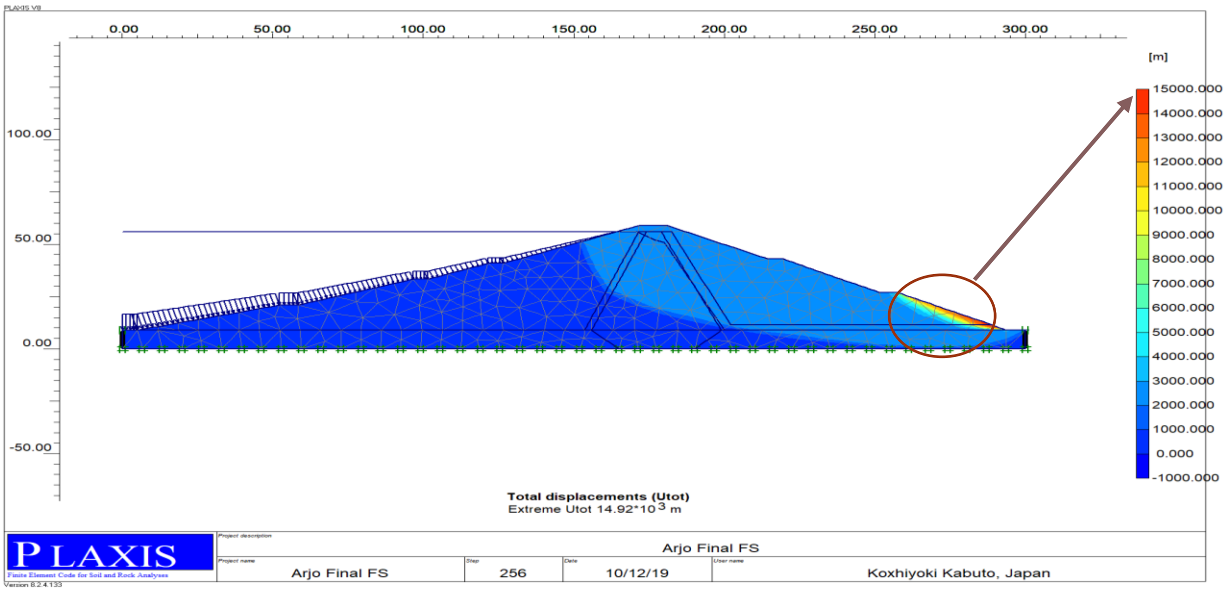


Figure 49: Downstream failure surface found in the model.

As observed in Table 11 there are several points where the safety factor is not indicated. This is due to the program failing to converge, meaning that equilibrium for the flow could not be found (Ultimate state not reached, Error 34 in PLAXIS2D) (Brink greve, *et al*., 2016). The solutions to bypass this problem is to either create a new mesh in the model by making the mesh finer or courser since the mesh might be appropriate for the calculation to converge, or higher the tolerated error described in Equation (15) and Fig. (11). Creating a new mesh does however create new locations for stress points where the calibration takes place, and the model has proven to be sensitive to mesh changes due to soil body collapse if the mesh is “too fine”.

Also, if the mesh is changed it is harder to compare the results due to different stress fields created. Therefore, the solution was to change the tolerated error for the out of balance flux which uses the same equation for the forces but for the flow equilibrium equation instead and keep the mesh coarseness factor same in all models. The results show that there is no change in safety factor with increased permeability in the pipe.

All tests in Table 12 has had their critical shear surface upstream which would not be a concern to the guidelines set by RIDAS since this consider as critical condition for a full reservoir in the downstream slope and minimum safety factor required equal to 1.5. It is also observed that the shear surface tends to be located on materials with less strength than the rockfill (the filters), similar to SLOPE/W's results. It is observed that the safety factor gradually becomes slightly higher in almost all simulations with higher void ratio.

This can be due to several factors, but the most probable cause is the lowering of the pore pressure upstream which yields higher effective stresses. This is confirmed by the extra analysis performed and showed in some extreme cases with higher permeability and void ratio. The most critical surface changed to be downstream where a higher pore pressure is built as well as seepage forces affecting the slope.

4.3 Displacement

Some deformation, were found when piping occurs in the dam, as shown in Fig. (50). Even though the displacement is observed to be found in micrometer it is still interesting to see what initiates the "swelling" phenomena observed.

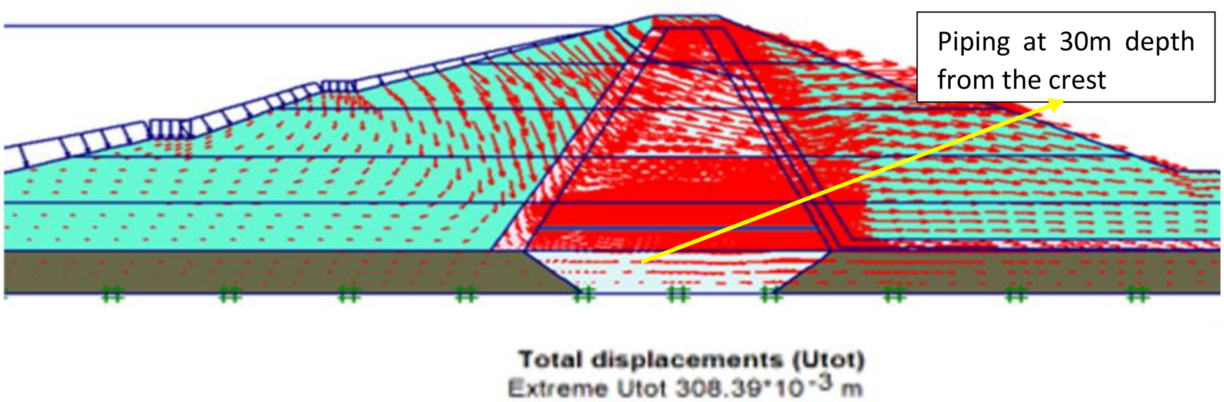


Figure 50: Total displacement in the dam when piping is initiated

(On a 500mm pipe at 30m depth with void ratio e=1)

A crosscut was made through the center of the embankment dam where the primary effective stress was observed through four different analyses. The first analysis is performed on an eroded pipe at 30 meters depth from the crest where both the density and permeability are changed. The second analysis is the same but where the permeability is unchanged. The third analysis only changes the permeability and keeps the density the same.

The fourth one is the dam with no change in both permeability and density. This was done to show the effects on the results due to changes of the mentioned parameters. The results from the four analyses are shown in Fig. 51).

Table 12: Effective stress analysis result under different depth and (Source Plaxis 8.2)

No	Depth in (m)	ES at density change and k constant KN/m^2	ES at No erosion condition KN/m^2	ES at Both density and K changed KN/m^2	ES at Constant density & k change KN/m^2
1	5	-72.69	-78.6	-77.98	-78.77
2	10	-110.92	-163.89	-120.95	-120.95
3	15	-226.41	-218.45	-227.57	-222.49
4	20	-265.69	-265.79	-271.78	-269.75
5	25	-334.75	-336.71	-341.91	-343.53
6	30	-384.01	-403.13	-378.61	-384.98
7	35	-495.31	-498.64	-505.64	-540.44
8	40	-498.03	-511.74	-506.68	-521.03
9	45	-615.79	-639.25	-640.11	-641.05

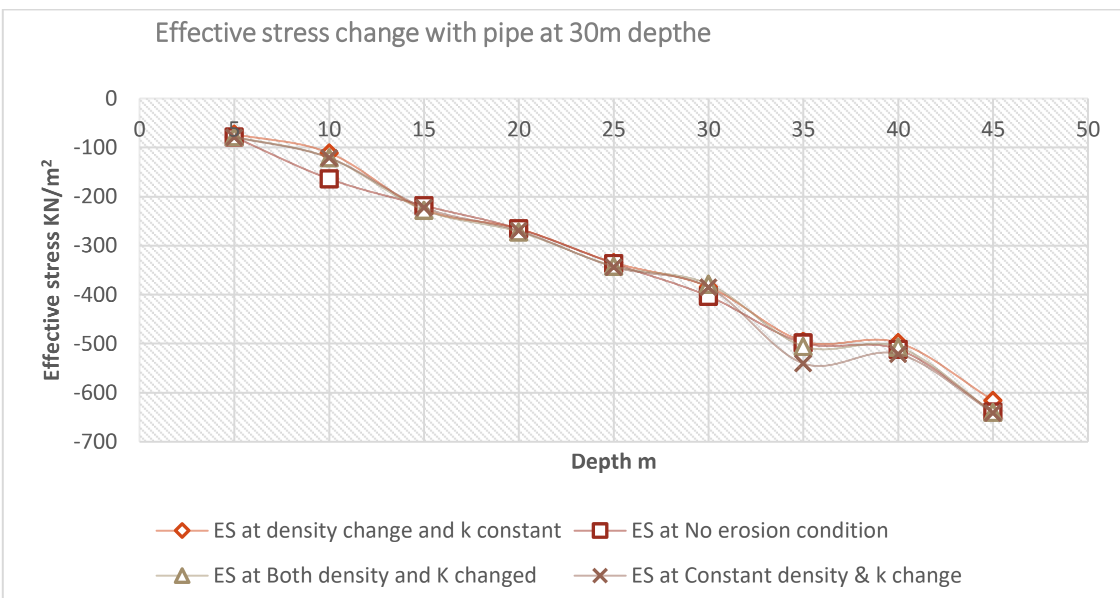


Figure 51: Primary effective stress at different levels in the core with changes to density and flow

Since PLAXIS2D calculates with FEM and thus calculates the stress strain relation of the model it is good to analyze the stresses that have created the displacement due to set strain parameters. The results from the four different cases show that the increased flow and density affects the primary effective stress as seen in Fig. (51). The flow change does not directly affect the stresses but with increased flow there is a lowering of pore pressure that increases the effective stress above the pipe. When the density is lowered due to material being eroded the primary effective stress is decreased below the pipe.

With both combined, in the first analysis it is observed that both affect the dam in this way but since the dam seem to ‘swell’ with increased erosion it means that the loss of density in the pipe affects the primary stresses more than the decrease of pore pressure. This displacement is however minimal and might not be able to be observed if such a method would be used when looking at the dam in terms of dam safety and see if internal erosion has occurred. Also, to note is that the Mohr-Coulomb model is used and thus the displacement may be observed but should not be seen as a good presentation for real in movement.

The total displacement, horizontal displacement and vertical displacement of the rock fill dam subjected for this analysis is significantly changed as the permeability and density of material is changed.

Table 13:The table below shows the value of displacement parameters in different condition

No	Displacement	e= 0.31	e=1
1	Vertical Displacement	-1.22E-01m	-2.55E-01m
2	Horizontal Displacement	1.06E-01m	3.08E-01m
3	Total Displacement	1.22E-01m	3.08E-01m

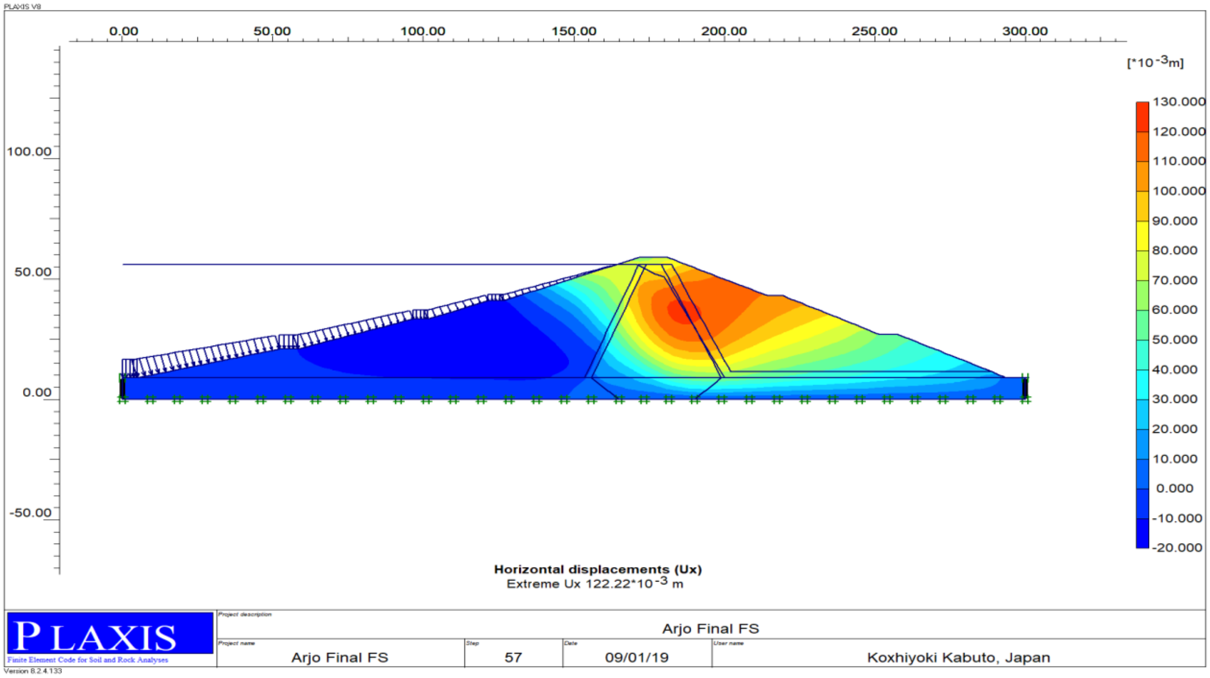


Figure 52: Horizontal displacement for initial void ratio e=0.31 (Plaxis 8.2 Output).

The result of horizontal displacement of Arjo Dhidhessa rockfill dam at initial void ratio (e=0.31) shown above in fig. (52) s, the maximum horizontal displacement of 122.22×10^{-3} m is nearly occurred in the clay core region of the dam.

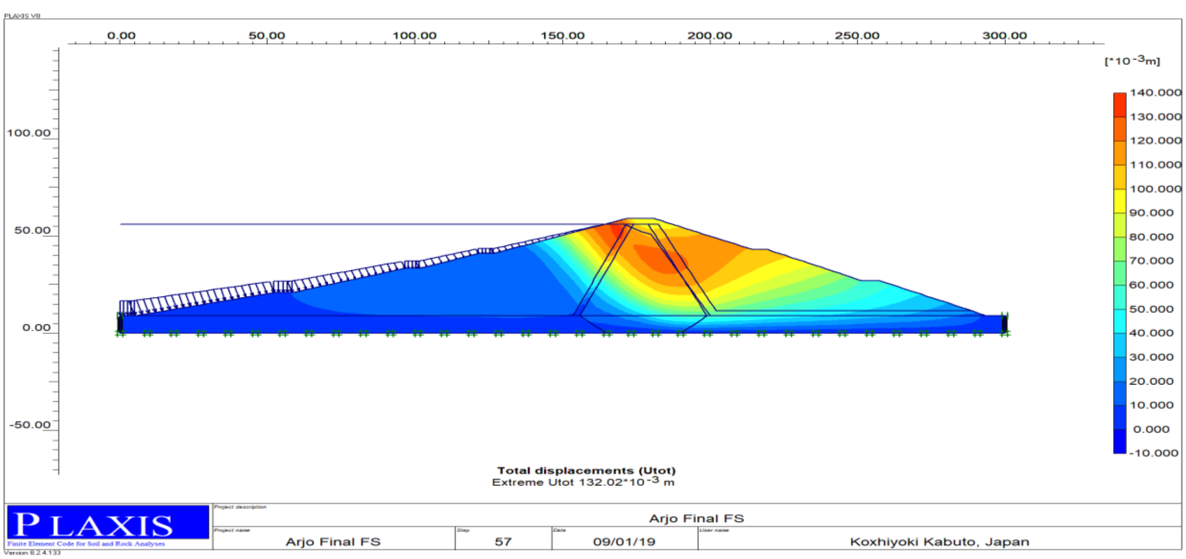


Figure 53: Total displacement for initial void ratio e=0.31 (Plaxis 8.2 Output).

The Arjo Dhidhessa rockfill dam has also been analyzed for the total displacement at extreme void ratio (e=0.31), and from the total displacement result of the dam 132.02*10^{-3}m as shown above 0n fig. (53), the maximum total displacement is also found near the core of the dam,

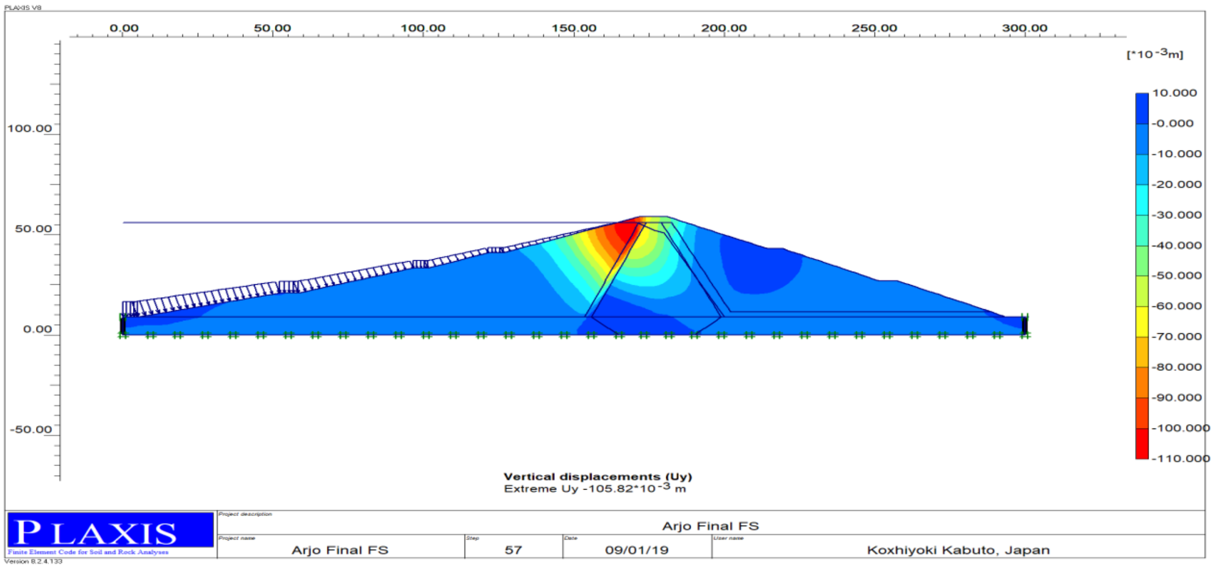


Figure 54: Vertical displacement for initial void ratio e=0.31 (Plaxis 8.2 Output).

The Arjo Dhidhessa rockfill dam has also been analyzed for the vertical displacement at extreme void ratio (e=31), and from the vertical displacement result of the dam shown above in fig. (54) indicate -105.82*10^{-3}m, the maximum vertical displacement is also found near the upstream face close to the crest of the dam. The negative value (-ve) indicate the direction of displacement.

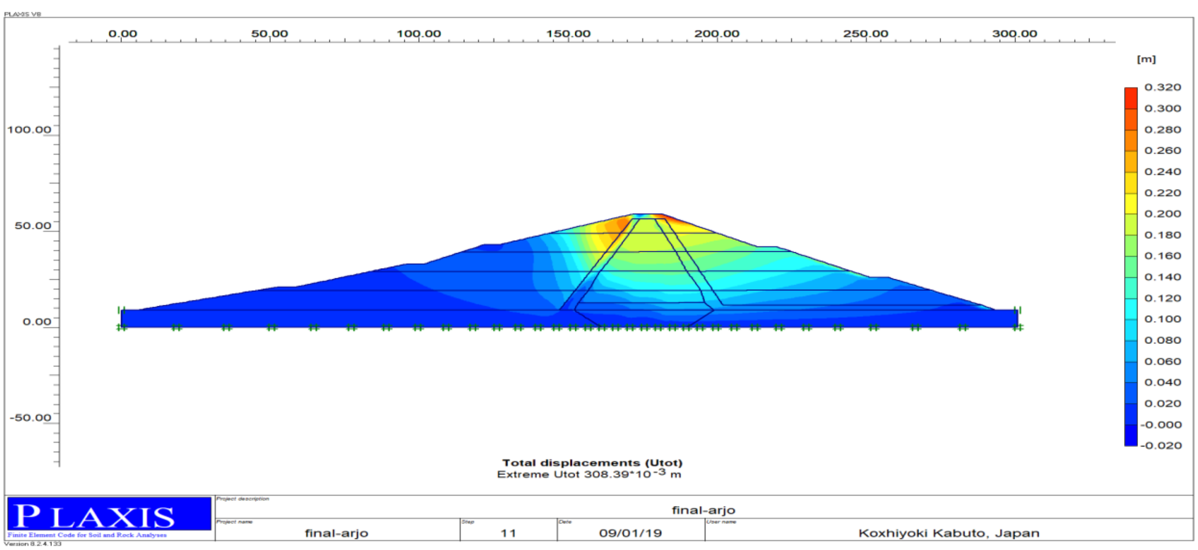


Figure 55: Total Displacement for void ratio e=1 (Plaxis 8.2 Output).

The Arjo Dhidhessa rockfill dam has also been analyzed for the total displacement at extreme void ratio (e=1), and from the total displacement result of the dam 308.39*10^{-3}m as shown above 0n fig. (55), the maximum total displacement is also found near the upstream crest of the dam.

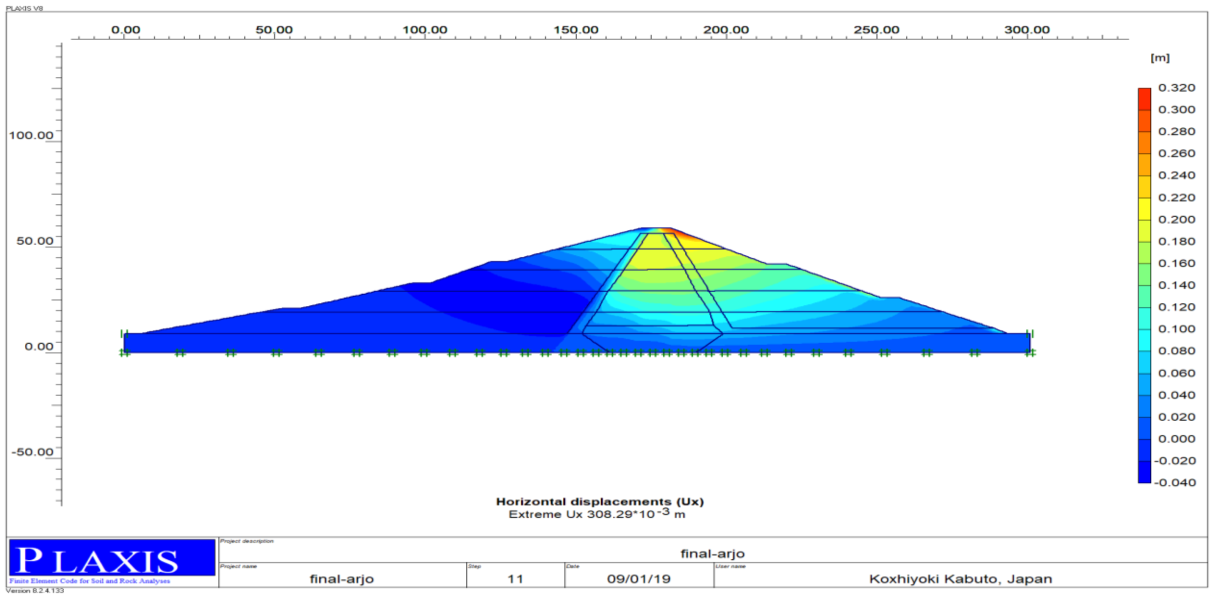


Figure 56: Horizontal Displacement for void ratio e=1 (Plaxis 8.2 Output).

The result of horizontal displacement of Arjo Dhidhessa rockfill dam at maximum void ratio (e=1) shown above in fig. (56) shows that, the maximum horizontal displacement of 308.28x10^{-3} m is nearly occurred in the downstream crest region of the dam.

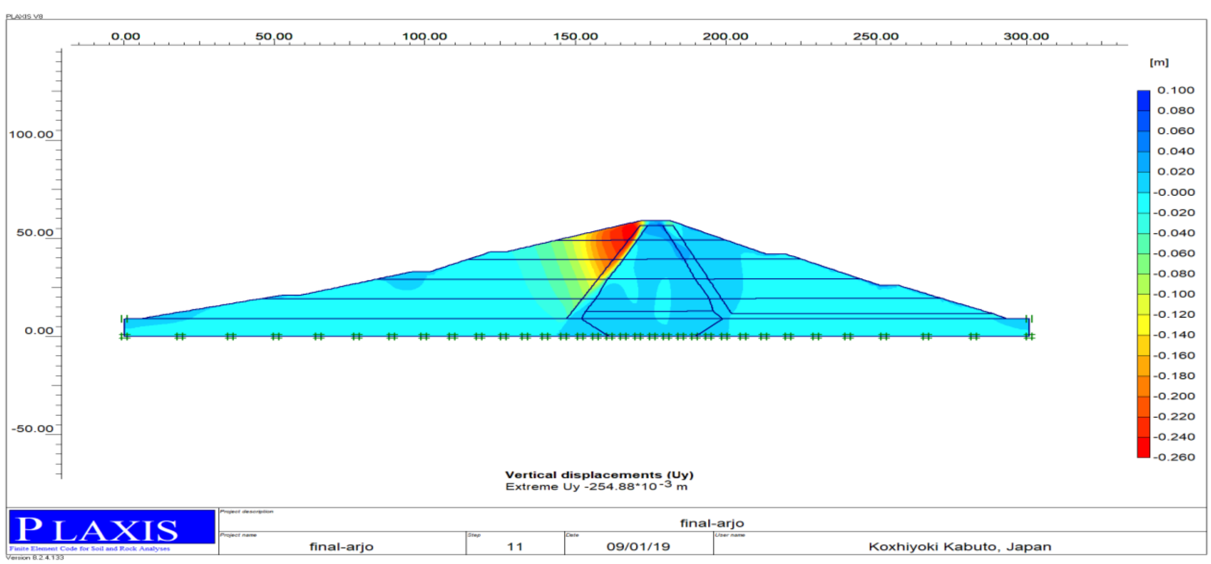


Figure 57: Vertical Displacement for void ratio e=1 (Plaxis 8.2 Output).

The Arjo Dhidhessa rockfill dam has also been analyzed for the vertical displacement at extreme void ratio (e=1), and from the vertical displacement result of the dam shown above in fig. (57) indicate $-254.88*10^{-3}$m, the maximum vertical displacement is also found on the upstream face near crest of the dam. Negative(-ve) sign indicate the direction of displacement

As the result of analysis conducted by Plaxis 8.2 indicate the displacement of the dam is changed as density and permeability of the materials varies. Accordingly, the horizontal displacement occurred in clay core region of the dam is rise from 0.106m fig. (52) to 0.308m fig. (56) which is above the allowable maximum values (0.5%H, Source, USACE, 2003). Consequently, it can be cause crack of the clay which facilitate piping problem in the body of dam.

The fig. (54 &57) shows that, Vertical displacement is also increased from 0.112m to 0.254m. As we observe from the fig. (57) the maximum vertical displacement of 0.254 m is found near crest of dam. As this value is found within allowable recommended range, a vertical displacement of magnitude 0.254 m in clay core dam was not have significant effect on the dam. As the result of analysis conducted by Plaxis 8.2 indicate the displacement of the dam is changed as density and permeability of the materials varies. Accordingly, the horizontal displacement occurred in clay core region of the dam is rise from 0.106m fig. (52) to 0.308m fig. (56) which is above the allowable maximum values 0.5%H (USACE, 2003). Consequently, it can be cause crack of the clay which facilitate piping problem in the body of dam.

Table 14: Maximum and minimum displacement of clay core for e=1

Horizontal Displacement	Magnitude	Standard (USACE.2003)	Remark
Maximum	0.308	0.5%H (0.25m)	
Minimum	0.106	0.05	Not safe

5 CONCLUSION AND RECOMMENDATION

5.1 Conclusion

- There is a good viability in using a FEM program in internal erosion analyses. Even though some calculating problems were encountered during the analysis, the results are still plausible during set circumstances. It should still be considered that FEM is incapable of creating an eroded zone itself due to the reliance of continuum mechanics.
- The pore pressure in the embankment dam were changed if the void ratio and permeability of the clay core of the embankment different due to diverse reason. The contour line of the pore pressure distribution was also shifted its position. These situations happened due to increase of permeability and void space in some part of clay core (around conduit) of the embankment.
- From the analysis conducted by Plaxis 8.2 indicate the displacement of the dam is changed as density and permeability of the materials varies. Accordingly, the horizontal displacement, vertical displacement and Total displacement occurred in clay core region of the dam is rise as the permeability and void ratio of that clay core are increased. Particularly the horizontal displacement rises from 0.106m at (e=031, k=1.07e-06) to 0.308m at (e=1, k=1.17e-01) which is above the allowable maximum values 0.5%H (USACE, 2003). Consequently, it can be cause crack of the clay which facilitate piping problem in the body of dam
- The evaluation of seepage conditions around outlet conduits in dams is a challenging problem. It requires a keen understanding of potential failure modes tied to site geology, design features, and construction methods. To identify whether or not potential failure modes have developed begins with the identification of locations where concentrated and unfiltered seepage leaks can develop.
- Conduits through embankment dams are prone to seepage and internal erosion around the surrounding soil. This may be overlooked by design engineers and the embankment dams may fail due to piping. Due to Potential cracking of embankment and foundation materials as a result of differential settlement and possibly hydraulic fracturing will also be an important element of an overall safety evaluation.
- The safety factor gradually becomes slightly higher in almost all simulations with higher void ratio. This can be due to several factors, but the most probable cause is the lowering

of the pore pressure upstream which yields higher effective stresses. This is confirmed by the extra analysis performed and showed in some extreme cases with higher permeability and void ratio. The most critical surface changed to be downstream where a higher pore pressure is built as well as seepage forces affecting the slope.

- The impact of internal erosion on unit weight and conductivity. The results show that the approach made to calculate the different changes in permeability was a reasonable method. As the void ratio and permeability of the clay core of the embankment dam were increased the unit weight value are of the clay material are gradually decreased. This situation is gradually affecting the slope stability of the dam if the unit weight of the clay reaches minimum than required value.
- The calculated factor of safety evaluated in the model at different sizes of pipes and locations is changing but only to a minimal degree. This is due to only a slight change of the phreatic surface that affects the effective strength in the slopes both downstream and upstream of the dam. As we observe from the results of PLAXIS 8.2 the values of the factor of safety under different permeability lies between 1.88 and 1.77 this means there is low variation are occurred. The calculated factor of safety evaluated in the model at different level (15,30 45m) of the dam and sizes of the of the pipes is changing gradually.

5.2 Recommendation

From the overall result of analysis obtained in this study, I presumed the following recommendations. During design and construction of rockfill dam as much as possible fixing the conduit out of the embankment dam is recommendable to avoid the probability of formation concentrated flow which cause internal piping. However, if it is difficult such as Ajo Dhidhissa rock fill dam considering additional paraments are required.

- Initial void ratio of the Arjo Rockfill dam should not higher than 1. If the void ratio of the clay core of the dam is higher than it induced deformation and settlement that would probably cause crack of clay core and facilitate internal piping in clay core
- The initial void ratio of the clay core around the conduit of the dam should be equal to the initial void ratio of clay core throughout the dam embankment. Therefore, similar compaction rate throughout the layer of clay core should be required.

- The permeability of clay core should be in allowable limit and similar through the clay core embankment. If the permeability values of the clay core are change due to material properties or due to nonuniformity of compaction rate, concentrated flow are happened in the embankment of the dam.
- PLAXIS 8.2 software are more reliable and safer side for slope stability analysis of Arjo Dhidhessa rockfill dam due considering more parameters such us Elastic module, poison ratio, saturated and dry unit weight, void ratio and hydraulic conductivity for each material.
- The authors believe that there is a need further research for both numerical and laboratory on the seepage around conduits through embankment dams.

It should also be noted that this analysis was performed on an earth and rock fill dam which has a highly permeable shell, thus a creation of a higher phreatic surface and thus pore pressure is harder due to its draining capabilities.

REFERENCES

Agahei Araei, A. S. (2010.). Large-Scale Triaxial Testing and Numerical Modeling of Rounded and Angular Rockfill Materials, Sharif: Sharif University of Technology. .

Brinkgreve, R. K. (2016.). Plaxis material model manual, Delft: Plaxis.

Bonelli, S., Brivois, O., Borghi, R., & Benahmed, N. 2006 On modelling of piping erosion, Comptes Rendus de Mécanique

Craig, R. (2004). Basic characteristics of soil, Cragsoil mechanics, London. Taylor and Francis Group.

FAO. (2010). Irrigation and drainage paper N 64.Original edition. Food and Agriculture Organization of United Nations.

Felippa, C. (2004). Introduction to Finite Element Methods (ASEN 5007)-Fall 2004 Department of Aerospace Engineering Sciences, University of Colorado at Boulder,

Fell R., Wan C.F., Cyganiewicz J., Foster M., (2003). Time for Development of Internal Erosion and Piping in Embankment Dams, Journal of Geotechnical and Geoenvironmental Engineering,

FEMA. (2005) Conduits through Embankment Dams Best Practices for Design, Construction, Problem Identification and Evaluation, Inspection, Maintenance, Renovation, and Repair 2005

Fox GA, W. G. (2010). The role of subsurface flow in hill slope and stream bank erosion: a review. Soil Science Society of America Journal. .

Foster, M. & Fell, R., 1999. Assessing embankment dam filters which do not satisfy design criteria. Sydney: University of New South Wales

Garg .S, K. (2005). Irrigation Engineering and Hydroulic structure .Book For Civil Engineering degree student. KHANNA Publisher,2-b NATH market Narsarak,Delhi.

Gouw, T.-L. (2014.). Common mistakes on the Application of Plaxis 2D in Analyzing Excavation Problems, Jakarta: Research In.

Hegerty, D. (1991). Piping/sapping erosion: 1. Basic considerations. Journal of Hydraulic Engineering,.

ICOLD. (2018). Dam Design Criteria, the Philosophy of Their Selection. International Commission on Large Dams.

Jennifer Rulon (1985) The Development of Multiple Seepage Faces on Layered Slopes (water Recourse Research) Published by AGU publication 1985

Lambe, T. W. (1969.). Soil Mechanics - SI version. New York: John Whiley & Sons. .

Geo-Studio International. Ltd., ((2012).). Seepage Modeling with SEEP/W. Canada: GEO-SLOPE International, Ltd. .

Larsson, R. et al., 2007. Evaluation of undrained shear strength of Swedish fine-grained sulphide soils Linköping: Swedish Geotechnical Institute.

Matsson, H. (2016.). An overview of the mathematical background to the nonlinear finite element method. [Sound Recording] (Luleå University of Technology).

OKEKE, C. A. (2016). Internal erosion and piping failure of landslide dams.

OWWDSE. (2013). Final Design Modification Report Volume II. Addis Ababa: Oromia water works design and supper vission enterprise.

Penman A D M and Charles J A (1975) The quality and suitability of rockfill used in dam construction. Publisher Garston, Eng. Building Research Establishment, 1975.

Richards and Reddy, (2008) Experimental investigation of piping potential in earthen structures. Geo sustainability and geohazard mitigation

Ren, X. e. (2016.). A relation of hydraulic conductivity - void ratio for soils based on the Kozeny-Carman equation. Science Direct, .

SIGMA/W.(2008). StressDeformation Modeling with SIGMA/W 2007 An Engineering Methodology. GEO-SLOPE International Ltd. (2007).

Serafi J., and Santos L.,(1985),Three-Dimensional Seepage Through a Dam Foundation , Proceeding of 14th ICOLD, Lausanne

Sibille, L., Marot, D. & Sail, Y., (2015). A description of internal erosion by suffusion and induced settlements on cohesionless granualar matter. s.l.: Acta Geotechnics

Singh, V. (2006). Dam breach modeling technology (Vol. 17). Springer Science & Business Media.

Stephens, T. (2006). Federal Energy Regulatory Commission (FERM). Training aids for dams safety evaluation of seepage. Manual on small earth dams.

Svensk Energi, 2012. Ridas (2012) - Kraftföretagens riktlinjer för dam msäkerhet, s.l.: Svensk Energi.

Terzaghi K, Peck RB, Mesri G . (1996). Soil Mechanics in Engineering Practice, Third Edition: John Wiley & Sons, New York.

Terzaghi, K. (1943). Theoretical soil mechanics: New York, John Wiley and Sons.

Terzaghi, K. (1960). Storage dam founded on landslide debris, Journal of the Boston Society of Civil Engineers.

U.S. Bureau of Reclamation, (1995). Ground Water Manual. 2nd ed. Washington D.C., USA: U.S. Government Printing Office.

Vahdati, P., (2014). Identification of Soil Parameters in an Embankment dam by Mathematical Optimizations, Luleå: Luleå University of Technolog

Zhang LM, Chen Q. (2006).Seepage failure mechanism of the Gouhou rock fill dam during reservoir water infiltration.

APPENDIX A

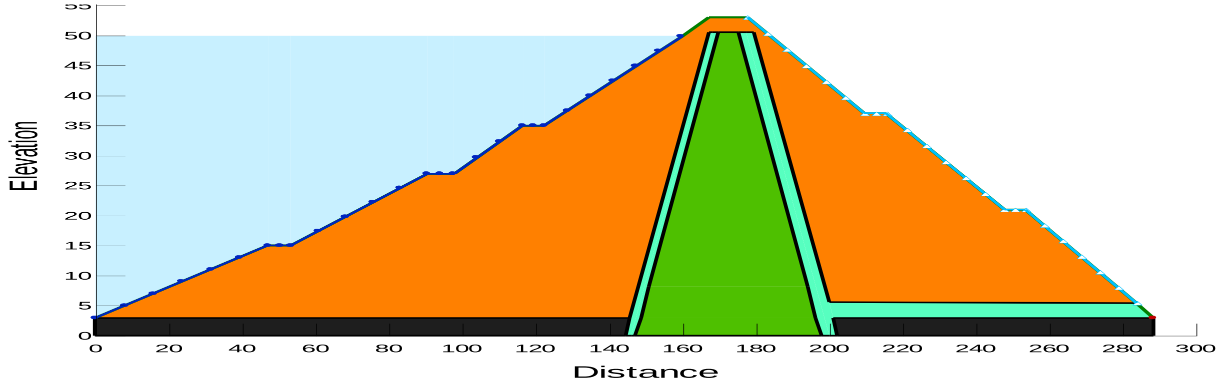


Figure A 1: Model generated for Analysis before mesh

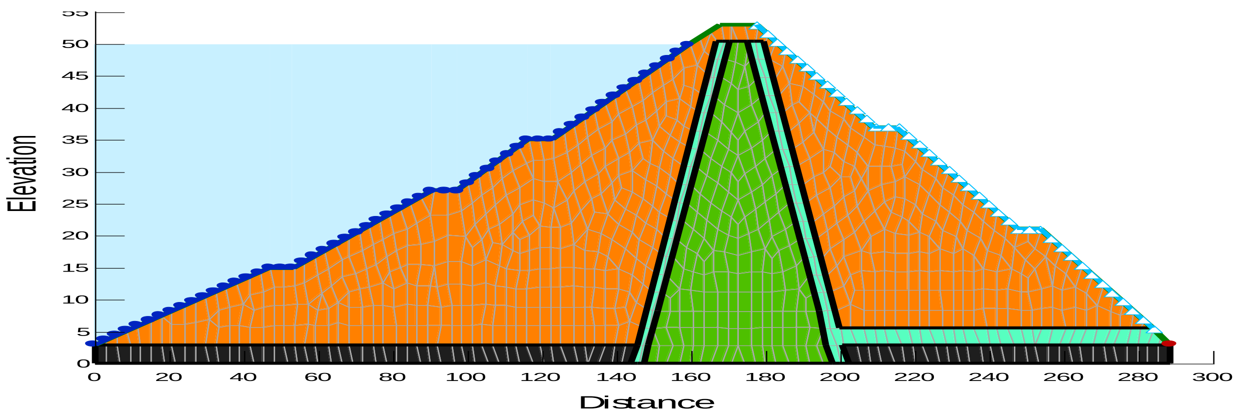


Figure A 2: Model generated for Analysis after mesh

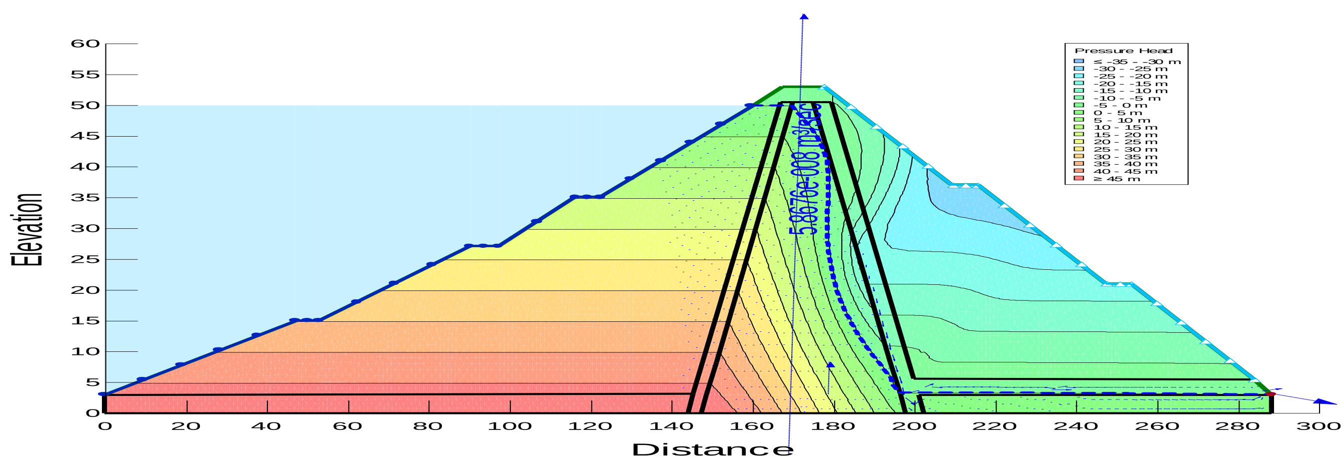


Figure A 3: Flow rate and pore pressure for core without conduit

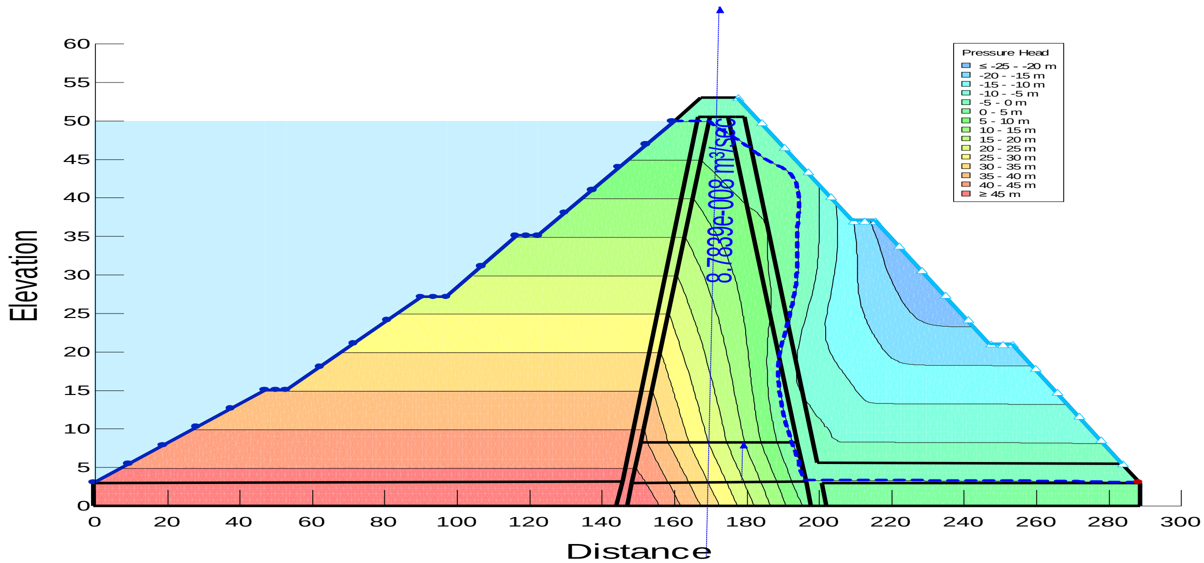


Figure A 4: Flow rate and pore pressure for core with conduit

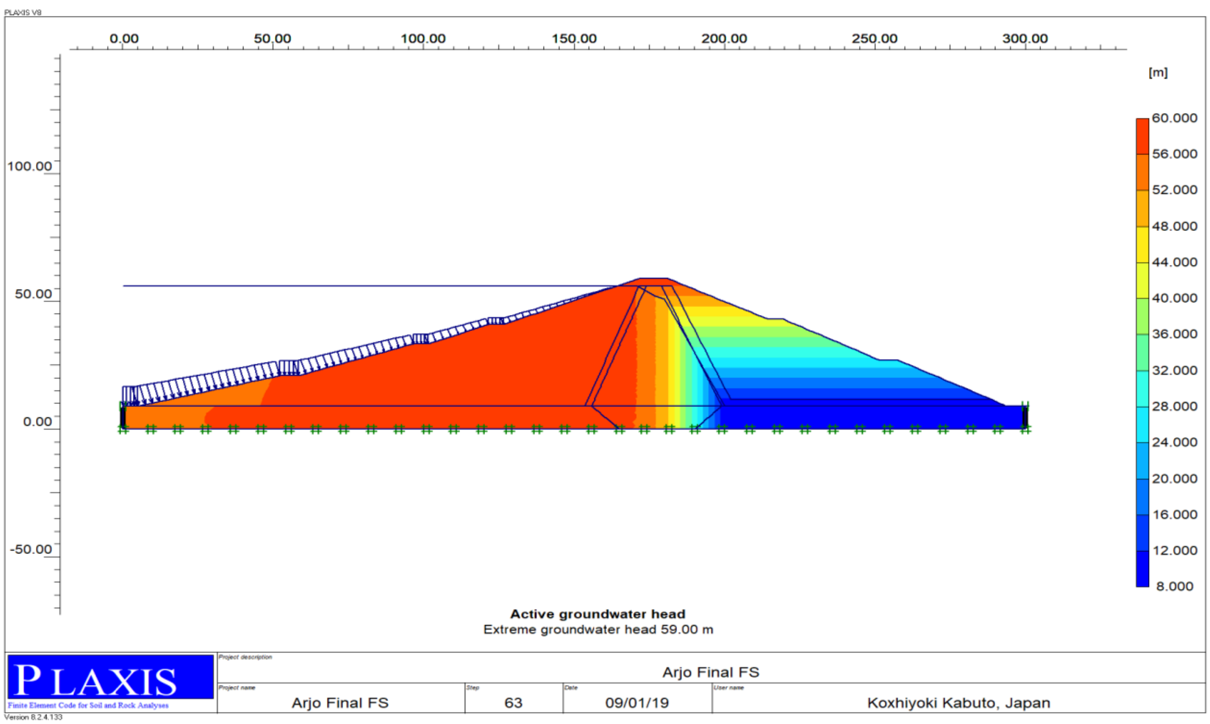


Figure A 5: Active ground water Head for initial void ratio e=0.31

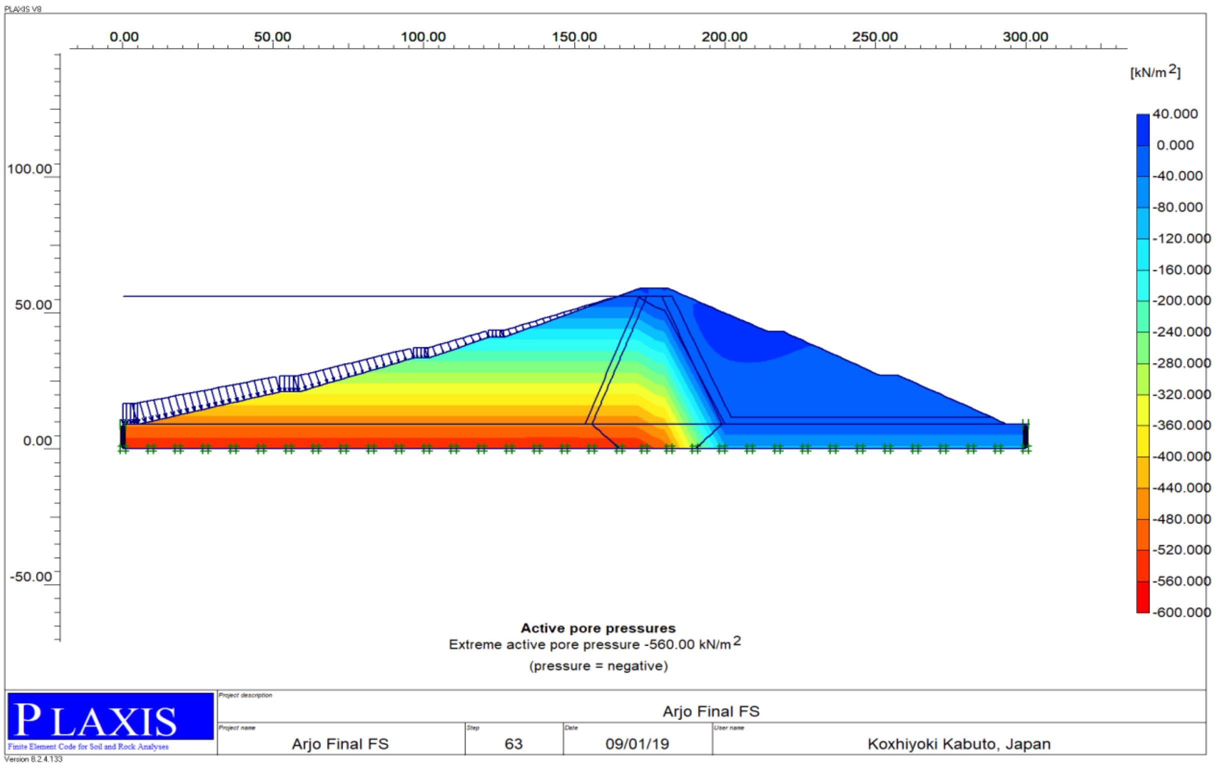


Figure A 6: Active pore pressure distribution for initial void ratio e=0.31

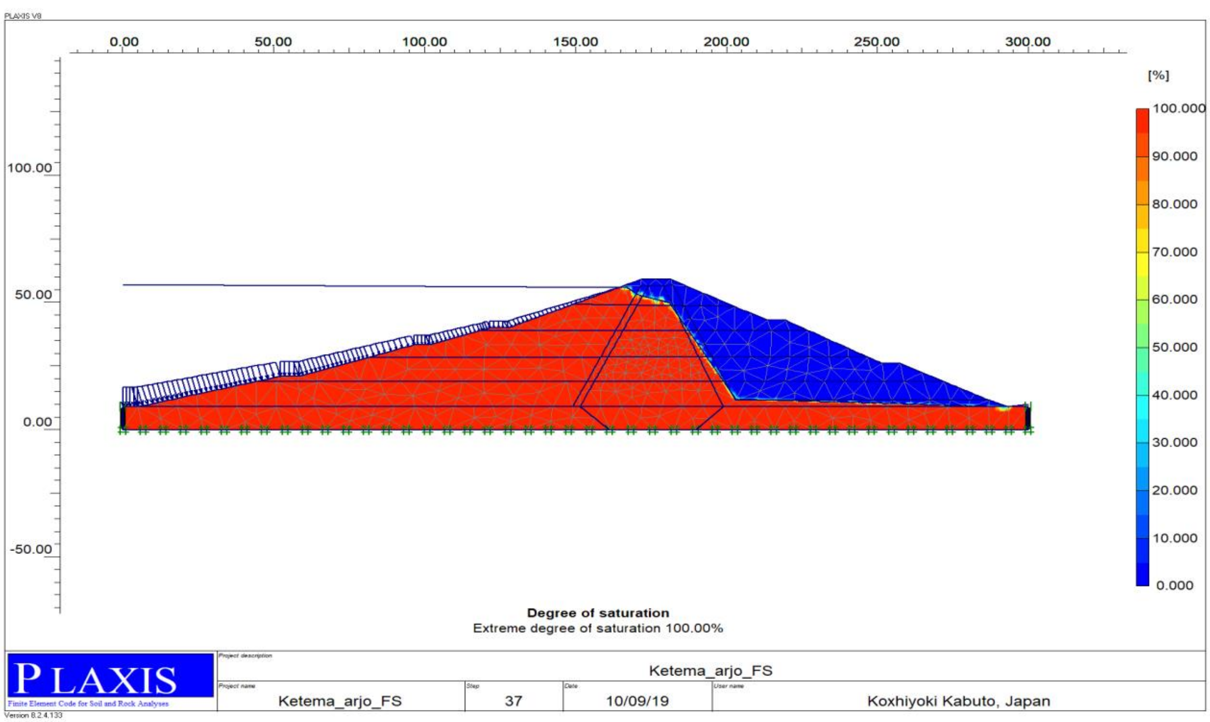


Figure A 7: Degree of Saturation of the dam

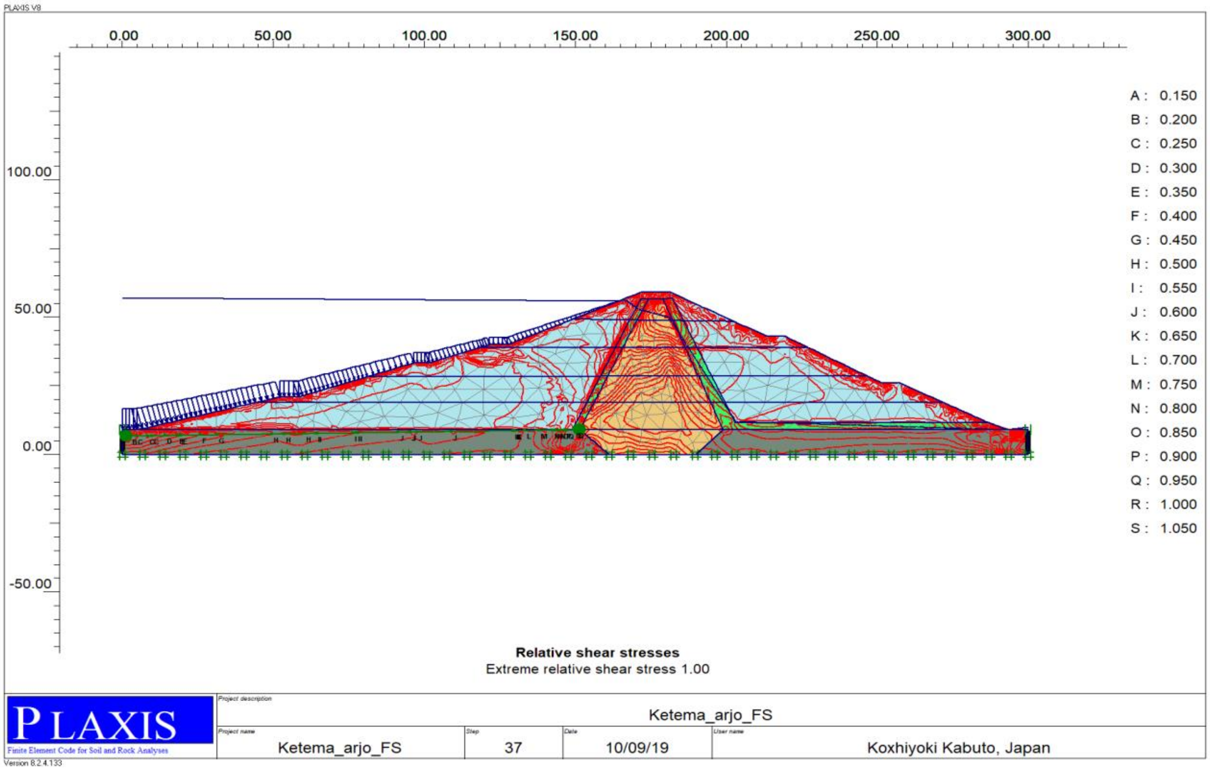


Figure A 8: Relative Shear stress by counter method

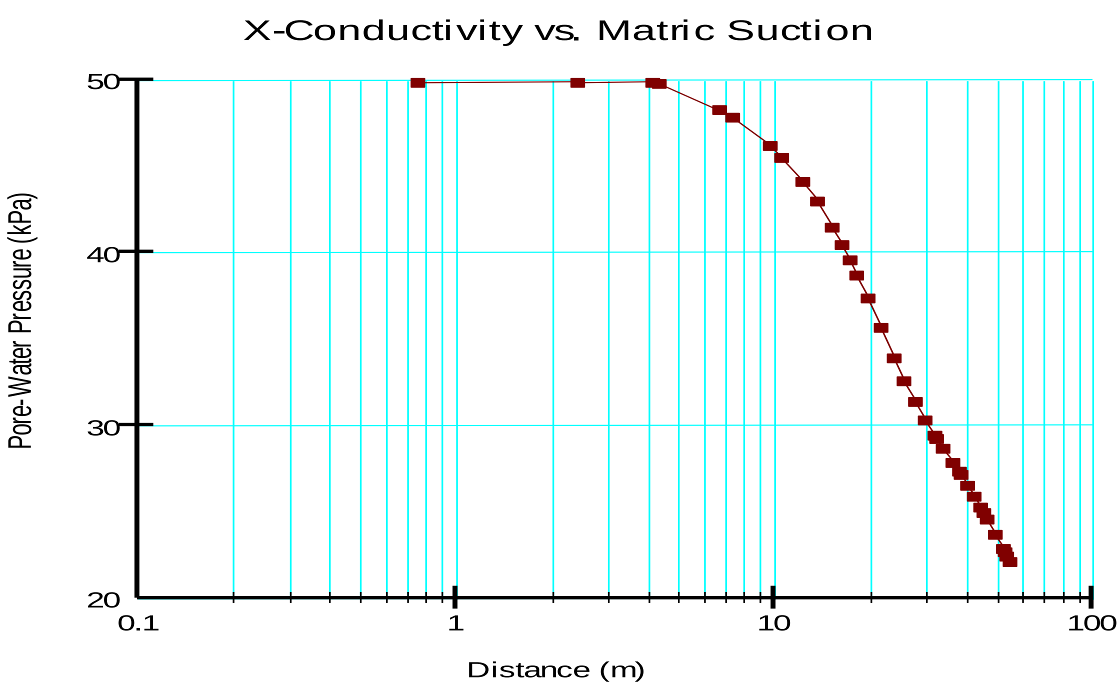


Figure A 9: Pore water pressure vs distance (SEEP/W)

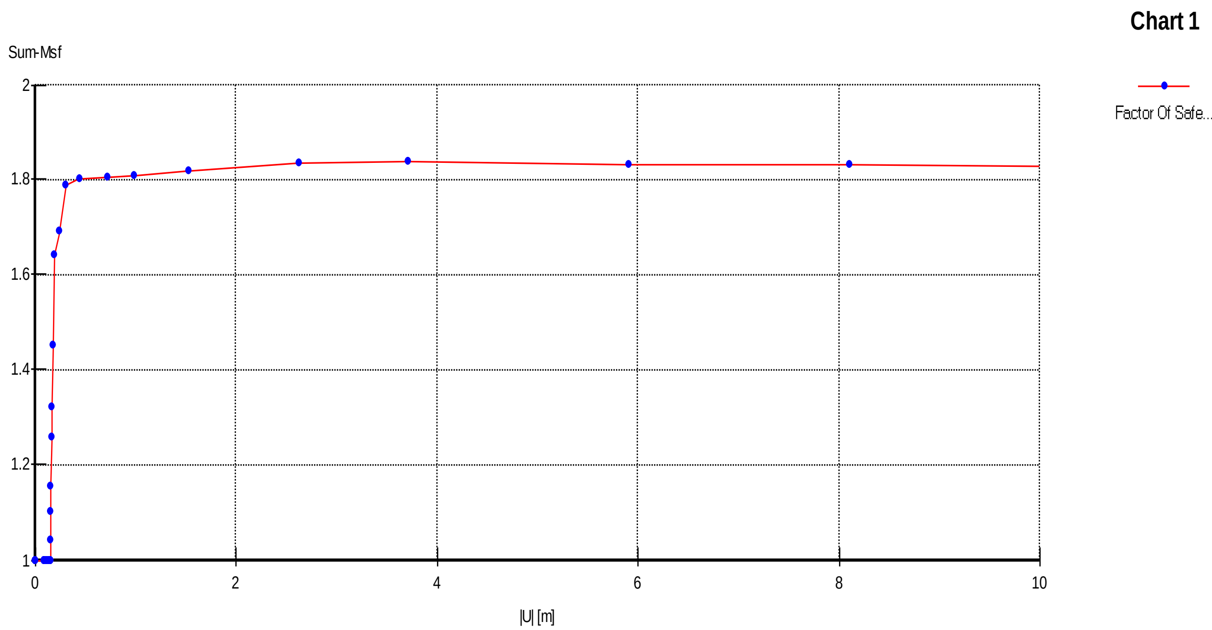


Figure A 10: Plotted Safety Factor

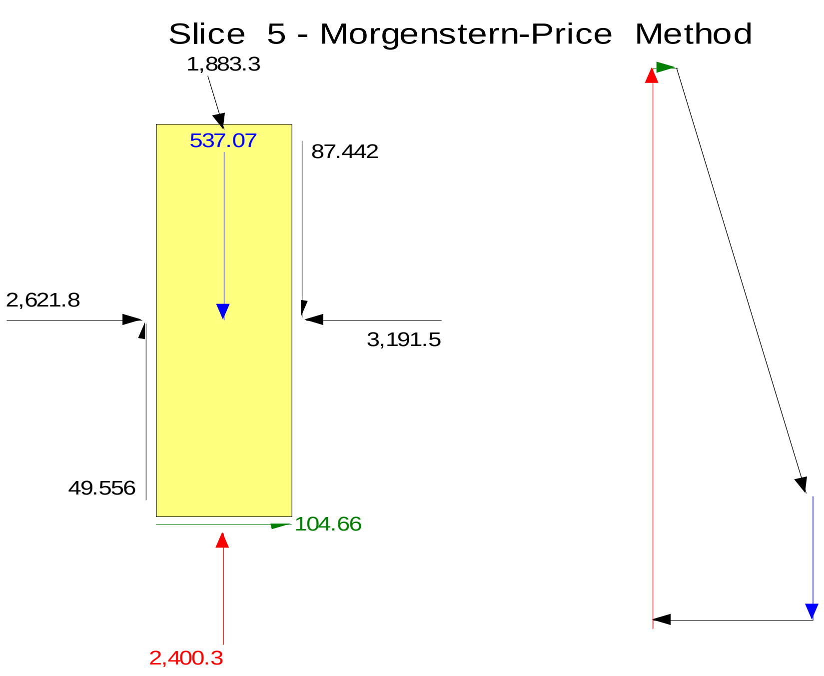


Figure A 11 Free body Diagram and Force polygon

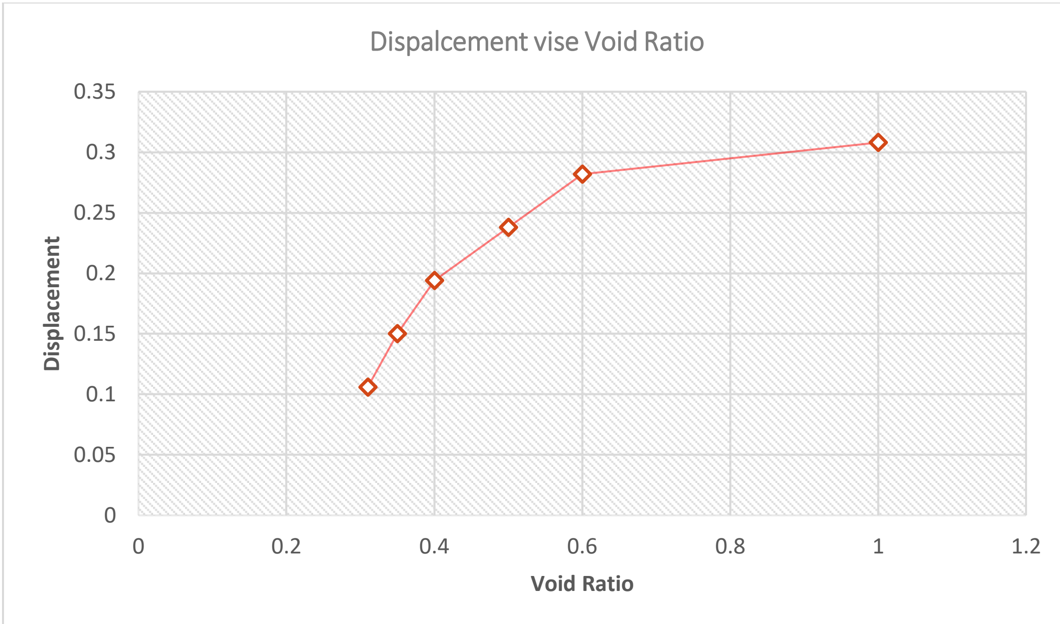


Figure A 12: Displacement of the dam at different Void Ratio

APPENDIX B

Table B 1: Properties of material to be adopted in the stability analysis

No	Properties	Unites	Casing/ Shell material KN/m3	Core KN/m3	Foundation KN/m3	Fine Filter	Transition Filter	Core Around Conduit
1	γ moist	KN/m^3	21.5	18	19.5	19	19.5	18.5
2	γ dry	KN/m^4	19.75	18.5	19	18.6	17.92	17.92
2	γ sat	KN/m^5	22.79	20.8	21	21.61	20.14	20.14
3	γ sub	KN/m^6	12	8.5	10	10	10.5	9.5
4	C	t/m^2	0	30	25	1	1	30
5	Ø	Degrees	41	29	30	35	35	29
6	ϕ	KN/m^2	15	5	2	2	2	5
7	E'	KN/m^2	120000	83400	170240	170240	170240	83400
8	v		0.15	0.4	0.4	0.35	0.35	0.34
9	Model Used		Van Gen	Van Gen	Van Gen	Van Gen	Van Gen	Van Gen
11	Conductivity	cm/s	4.82E-01	1.07E-06	1.75E-06	0.0642	0.014	0.0006515
12	Conductivity	cm/day	4.16E+04	9.24E-02	1.51E-01	5546.88	1209.6	56.2896
13	Conductivity	m/s	4.82E-03	1.07E-08	1.75E-08	6.42E-04	1.40E-04	6.52E-06
14	Conductivity	m/day	4.16E+02	9.24E-04	1.51E-03	5.55E+01	1.21E+01	0.00092448