

A COMPARATIVE EVALUATION OF SEEPAGE, SLOPE STABILITY AND
DEFORMATION IN EMBANKMENT DAM :- THE CASE OF AGUAT WUHA
MICRO EARTHEN DAM, ETHIOPIA



SOLOMON MELAKU SHUME

A THESIS SUBMITTED TO THE DEPARTMENT OF WATER RESOURCES
ENGINEERING

SCHOOL OF CIVIL ENGINEERING AND ARCHITECTURE
IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF
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(SPECIALIZATION IN HYDRAULIC ENGINEERING)

OFFICE OF GRADUATE STUDIES
ADAMA SCIENCE AND TECHNOLOGY UNIVERSITY

JUNE, 2023

ADAMA, ETHIOPIA

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ADVISOR :- DEREJE BIRHANU (PhD)

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DECLARATION

I, hereby declare that this Master Thesis entitled “ A Comparative Evaluation of Seepage, Slope Stability and Deformation in Embankment Dam :- The case of Aguat wuha micro earthen dam” is my original work. That is, it has not been submitted for the award of any academic degree, diploma or certificate in any other university. All sources of materials that are used for this thesis have been duly acknowledged through citation

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RECOMMENDATION OF ADVISOR

I, the advisor of this thesis, hereby certify that I have read the revised version of the thesis entitled “A Comparative Evaluation of Seepage, Slope Stability and Deformation in Embankment Dam :- The case of Aguat wuha micro earthen dam” prepared under my guidance by Solomon Melaku Shume submitted in partial fulfillment of the requirements for the degree of Master’s of Science in Hydraulic Engineering. Therefore, I recommend the submission of a revised version of the thesis to the department following the applicable procedures.

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APPROVAL PAGE OF M.S.C.THESIS

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LIST OF ACRONYMS AND SYMBOLS

ADSWE	Amhara Design and Supervision Works Enterprise
A.M.S.L	Above Mean Sea Level
BEM	Boundary Element Method
BM#	Bench Mark
DEM	Digital Elevation Model
E	Horizontal Side Force
FDM	Finite Difference Method
FEM	Finite Element Methods
FEMAUS	Federal Emergency Management Agency of the United States
FoS	Factor of Safety
Ff	Factor of Safety with Respect to Force
Fm	Factor of Safety with Respect to Moment
GPS	Geographical Position System
GS	Ground Surface
ICOLD	International Commission on Large Dam
LE	Limit Equilibrium
K	Hydraulic Conductivity
L	Length of Slice Base
LEM	Limit Equilibrium Method
Mw	Water Storage (the Slope of the Soil-Water Characteristic Curve)
MoLS	Minister of Labour and Skill
N	Slice Base Normal Force
NFL	Normal Flood Level
Quake/W	Limit Equilibrium Software for Dynamic Analysis of Structure
SEEP/W	Finite Element Software for Ground Water (Seepage) Analysis
SIGMA/W	Limit Equilibrium Software for Stress-Strain Analysis
SLOPE/W	Limit Equilibrium Software for Slope Stability Analysis
S	Slice of Slide
U	Pore Water Pressure Force Acting on the Base
USACE	United States Army Corps of Engineers
USBR	United States Bureau of Reclamation
UTM	Universal Transverse Mercator
W	Weight of the Slice
WWDSE	Water Works Design and Supervision Enterprise
X	Vertical Side Force
σ	Normal Stress

ν	Poisson's Ratio
γ	Unit Weight
μ'	Pore Pressure
c'	Cohesion
E	Modules of Elasticity
ϕ	Friction Angle of Soil Material
τ	Available Shear Strength
τ_f	Mobilized Shear Strength
ϕ'	Friction Angle of Soil Materials
E	Younge's Modulus
VES	Vertical Electrical Resistivity Method
M	Poison's Ratio
E_n	Horizontal Force
$M^2/s/m$	Square Meter per Second per Meter
γ_w	Unit Weight of Water
I	Total Head Gradient
H	Total Pressure Head

ABSTRACT

This study presents an evaluation and comparative study on the seepage, slope and deformation analysis of Aguat wuha micro earth dam which is found in the Amhara Region, south Wollo Zone, Saint Ajibar Woreda. The main problem of Aguat wuha micro earh dam is anticipated seepage through the dam body, foundation, and surrounding outlet construction of the dam. The aim of this study is to identify the problem of Aguat Wuha Embankment dam by evaluating seepage, slope stability, and stress deformation. The main significance of this study is protecting the loss of stored water and safe guarding the safety of the dam and thus protecting down stream dwellers and property from hazards. The analysis was performed using limit equilibrium (Geo Studio) and finite element (Plaxis 2D 8.2) models. Field data measurements and observatory investigations were also carried out. Seepage analysis such as:- seepage magnitude, pore water pressure distribution, and location of the phreatic line have been determined using the limit equilibrium analysis of SEEP/W and finite element analysis of PLAXIS models. From the simulated results of SEEP/W and PLAXIS models at 2874m a.m.s.l, the average seepage rate for the entire length of the dam at full reservoir level was found to be $1.17 \times 10^{-5} \text{ m}^3/\text{s}$ and $4.987 \times 10^{-3} \text{ m}^3/\text{s}$ respectively;- , whereas for 2868m a.m.s.l, reservoir level was equal to $5.5 \times 10^{-6} \text{ m}^3/\text{s}$ and $3.576 \times 10^{-3} \text{ m}^3/\text{s}$ respectively;- , and the practical measured seepage value (rectangular lined masonry canal) using manning equation at the downstream of the dam body at 2874m and 2868m a.m.s.l, was $7.89 \times 10^{-3} \text{ m}^3/\text{s}$ and $6.23 \times 10^{-3} \text{ m}^3/\text{s}$ respectively. where as using calliper container method the measured value was $9.4 \times 10^{-3} \text{ m}^3/\text{s}$ and $8.1 \times 10^{-3} \text{ m}^3/\text{s}$ at 2874m a.m.s.l and 2868m a.m.s.l respectively. The stability analysis results for the designed conditions show that the slopes are unstable under some of the loading conditions of the Plaxis 2D 8.2 model which is the factor of safty result is less than 1.3 but for Geo Studio 2012 the slopes are stable under all loading conditions which means the factor of safty value is greater than 1.3. However, the actual site condition of the dam stability analysis indicated that the downstream slope is stable. The results of stress-deformation analysis (maximum horizontal and vertical displacements), carried out using both SIGMA/W and PLAXIS models, for end-of-construction and full reservoir conditions show almost similar values. The maximum vertical displacements 0.1718 m occurs at the crest of the dam, which is 0.625% of the dam height. This study shows that the Plaxis 2D 8.2 model simulates seepage better than Seep/W, whereas the two models have close capability in simulating deformations. From this study, we also understand that the dam safety and performance currently is not satisfactory (high leak, slope instability and deformation problems). Hence, it is recommended to conduct critical monitoring and surveillance of the dam and It is also better to use updated PLAXIS as an alternative to LEM because it presents several advantages that are no assumptions are needed concerning the failure surface, no assumptions on the inter-slice side forces are needed, since there is no concept of slices and soil behavior can be modeled in terms of stresses-strains.

Keywords:- Seepage, Slope Stability, Stress-Deformation, Embankment dam, Geo Studio, Plax's 2D

1 INTRODUCTION

1.1 Background

A dam is a hydraulic structure made of impervious material built across a stream or river for impounding water on its upstream side for different purposes such as hydropower generation, irrigation, flood control, water-supply, recreation, fishing, and navigation. Based on the purpose given dams can be constructed for a multipurpose or single purpose (Chen 2015).

The upstream and downstream face slopes of embankment dams are similar and of a moderate angle, resulting in a wide section and a high construction volume in relation to height. Contrarily, depending on the type of dam, concrete dams have relatively slim profiles and differing face slopes, such as a high upstream slope and a virtually vertical downstream slope. Embankment dams, which make up an estimated 85–90% of all dams constructed, are numerically dominating for both technical and economic reasons, according to The International Commission of Large Dams (Pham et al. 2013).

Instability, seepage and deformation-related issues in engineered as well as natural slopes are common challenges to both researchers and professionals. Embankment dam projects, placed into operation, observations, surveillance, inspections, and continuing evaluation, are required to assure the satisfactory performance of the dam (USACE 1994).

Periodic inspection of seepage, slope stability and deformations is the main and most important reason to maintain the overall safety of earthen dams. Seepage, slope stability, stress-deformation are much more dangerous after embankment dam construction than after concrete dam construction because embankments are typically built on soft material that is subject to scour and is also susceptible to influx of water; in contrast, a concrete dam is typically built on rock that is not worn away as quickly by the scouring action of water; even then, a malfunctioning dam will not necessarily be endangered by passage of water. Therefore, it has been emphasized throughout this study to draw attention to the issues related to earth-fill dams (Vardaro et al. 2016).

Earth fill dams need to be reliable at all times. These key factors, such as inappropriate design, lack of thorough investigations, inadequate care in operation, and bad maintenance, must be addressed because these types of dams are more susceptible to hydraulic, seepage, and structural failures. Geotechnical engineering's slope stability studies will change as soil and rock mechanics develops. It can be difficult for theoreticians and practitioners to evaluate the stability of both naturally occurring and manufactured slopes. Man or nature could

upset the balance of a natural slope, posing stability issues. Natural slopes that have been stable for many years may abruptly fail as a result of topographical changes, seismic activity, groundwater flows, a loss of shear strength, changes in stress, and weathering (Vardaro et al. 2016).

The slopes of the embankment must also be stable during construction and under all circumstances involving reservoir operation, and the upstream slope must be protected against erosion by wave action, while the crest and downstream slope must be protected against erosion caused by wind and rain (Vardaro et al. 2016).

Generally, ensuring the stability of the dam against slope failure, seepage and stress deformation is an essential component of the design. Besides stability, the dam also must be able to exhibit satisfactory serviceability (stress-strain) where the dam should be able to perform the intended function throughout the service life (USACE 1994).

But during the building process, during the first impoundment, and throughout long-term operation, embankment dam performance and safety must be monitored. It is well acknowledged that monitoring actual behaviors during the first years of operation and throughout the service life of the structure is as important to good design and construction when it comes to ensuring safety (ICOLD 2014).

All dams are designed and constructed to meet specific criteria such as a dam should be built from locally available materials wherever possible, the dam must remain stable under all conditions, during construction, and ultimately in operation, both at the normal reservoir operating level and under all flood and drought conditions. The dam and foundation must be sufficiently watertight to control seepage and maintain the desired reservoir level. The dam materials and foundation must be strong to resist load and the dam must have sufficient spillway, outlet works capacity and freeboard to prevent flood water from overtopping the dam crest (Ratnayaka et al. 2008).

For many embankment slopes the most critical condition is at the end of construction. In some cases, however, there may be intermediate conditions during construction that are more critical. In some fill placement operations, including some waste fills, the fill may be placed with a slope geometry such that the stability conditions during construction are more adverse than at the end of construction (Duncan and Thomas 2014).

Seepage through the embankment, base, and abutments is a possibility with all earth and rock-fill dams. Controlling seepage is essential to preventing high uplift pressure, sloughing

of the downstream slope, piping through the embankment and foundation, and erosion of material via loss into open joints in the foundation and abutments. The permitted amount of seepage may be restricted according on the project's goals, such as long-term storage, flood control, etc (Duncan and Thoma 2013).

Slope stability is an important aspect of geotechnical engineering. In general, linear problems such as the prediction of settlements and deformations, and the calculation of flow quantities due to steady and transient seepage are all highly amenable to solution by finite element method. The uses of finite element analysis of slope stability, seepage and deformations have gained popularity in recent years due to their capability to handle complex problems. The primary focus of this research is to study the influence of various water levels on the dam slope by using finite element analysis, to investigate failure mechanisms and recommend remedial measures. Slope stability, seepage analysis and stress deformations using computers are easy tasks for engineers when the slope configuration and the soil parameters are known. However, the selection of the slope stability method is not an easy task and the field data should be collected as the failure observations to understand the failure mechanism. Therefore, the theoretical background of each slope stability method should be investigated in order to analyze the slope failure and assess the reliability of the results (Pham et al. 2013).

Seepage and safety evaluations are vital to determine the status of a dam relative to its structural and operational safety. Therefore, this study is conducted to analyze the overall seepage, slope stability and deformation of Aguat wuha micro earthen dam utilizing FEM-based PLAXIS 2D and Geo Studio (LE) software which is helpful in the evaluation of the performance of embankment dams.

This study's major objective is to examine the seepage, slope stability, and deformation analysis of Aguat wuha micro earthen dam in order to assess the state of the water leakage issue, suggest difficulties with dam stability, and recommend beneficial corrective steps. Additionally, the study contrasts Geo Studio 2012 and Plaxis 2D 8.2 as models for studying and designing embankment dams.

1.2 Statement of the Problem

The main issue with the Aguat wuha embankment dam is anticipated seepage through the dam body, foundation, and surrounding outlet construction of the dam. Given the constant

flow of water and color change of the reservoir water at the downstream face via the dam body.

Nowadays, in our country, numerous embankment dams are being constructed particularly in various drought-prone areas. Most of the failure of earth-fill dams is due to excessive leakage via the reservoir side abutment, bottom, and foundation of the dam (Abay and Claudia 2011).

According to Tefera (2006), it is reported that out of the 14 micro earthen dams in the Amhara region, Ethiopia seepage failure is estimated at about (58 %), therefore, we can conclude that seepage and slope failure are considered among the most common types of failures in an embankment dam.

Many embankment dams are constructed in Ethiopia most of which are used for irrigation purposes. However, their capacity reduces frequently before their design life time due to several reasons. The main causes of capacity reduction are hydrological, structural, and hydraulic failures in the Amhara region (Tefera 2006).

In Ethiopia, many researchers have tried to study the seepage and slope stability analysis of the earthen dam including (Abera 2016), (T/Haymanot 2017), and (Zewdu 2019). These researchers focused only on one method it may be limited equilibrium, finite element, and finite-difference. But, instead of using one method, it is good using different methods for embankment analysis finally come up with the best methods for seepage, slope and deformation analysis.

In this study area specifically, the analysis of the seepage and slope stability analysis was carried out by (Habtom Reda 2016). He tries to address only the seepage and slope stability problem but, in addition to this, there is a deformation problem specifically in the upstream side. Even more, the failure of the upstream face of the dam is higher than before. Therefore, in this study area, it is necessary to have a clear understanding of seepage, slope stability, and deformation analysis by using different embankment dam analysis methods and field practical measures. According to the International Commission of Large Dams (ICOLD 1973), Embankment dam failures are classified into three major categories: overtopping by flood, foundation defects, and piping or seepage. From these three major categories of embankment dam failure, overtopping by flood (35%), foundation defects (21%), piping and seepage (58%) and others 6%. This indicates that piping and seepage are the most chronic dam failure in history. Due to the nature of the variable foundation conditions and the variety

of qualities of the materials available for use in the embankment, the design and construction of earth and rock-fill dams are difficult. Conducting thorough geological and subsurface explorations that identify the foundation, abutments, and probable borrow regions is the initial stage. The following step is to carry out a study of the kind and physical characteristics of the materials that will be used in the embankment, including all studies, testing, analyses, and evaluations to make sure that the embankment satisfies all technical requirements. Construction supervision, management, and monitoring of the embankment and ancillary structures are crucial components of the overall project management plan (Shano 1994). The development of soil mechanics, research into the behavior of earth dams, and the development of better construction methods have all contributed to the confidence needed to evaluate the outcomes of embankment dam seepage, slope stability, and stress-deformation processes, though more recent methods and research studies have been used (Abhilasha and Balan 2014).

The amount of seepage passing through the dam body and foundation, which is useful for assessing the current performance of embankment dams, was also measured. These results were compared to actual figures that would be measured in the field. Throughout recorded history, there is evidence that man has been afraid of the destructive power of water. It exerts incredible forces from within soil pores and rock crevices, tearing down mountain sides and destroying man-made structures. Furthermore, it is acknowledged that unchecked seepage can result in significant monetary losses as well as the loss of numerous lives (P. Novak 2001).

Generally, this study is mainly focused on a comparative evaluation of seepage, slope stability, and stress deformation analysis based on the geotechnical properties of the construction materials by finite element (Plaxis 2D8.2) and limit equilibrium (Geo Studio) method, and to put remedial measures of the Aguat wuha micro earthen dam.

1.3 Research Objectives

1.3.1 General Objectives

The General objective of the study is to a comparative evaluation of seepage, slope stability, and deformation analysis of Aguat Wuha micro earthen dam.

1.3.2 Specific Objectives

- To quantify seepage through the main body of the dam and foundation
- To evaluate the slope stability of the dam, and find out the factor of safety

- To evaluate the static stress-deformation of the existing dam at the end of construction and under water loading conditions
- To compare Geo Studio and PLAXIS 2D models for earthen dam design and analysis
- To investigate the existing condition of the dam

1.4 Research Questions

- How much amount of seepage is release from dam body and foundation ?
- In which condition are the the slope stability of the dam is stable and unstable?
- In which conditions and models are the stress-deformation value of the dam is larger one?
- Which one model has better value interms of seepage, deformation, and stress analysis?
- How do we know the existing performance of Aguat wuha micro earthen dam?

1.5 Scope and Limitation of the Research

The scope of this research will focus on the evaluation of seepage, slope stability and stress-deformation on Aguat Wuha micro earthen dam at the end of construction and under water loading conditions using Geo Studio 2012 and Plaxis 2D8.2 softwares and put the curative reclamation measure for the problem. The analysis incorporated the fully coupled effect of seepage over slope stability and stress-deformation. But, the study will not include analysis of gate failure and rainfall effects.

They are considered two loading conditions that are highly critical to the stability of slopes of the earth dam i.e., at the End of construction and Maximum water level with Steady State Seepage and Rapid draw down.

1.6 Significance of the Study

The nation as a whole will benefit from further attempts to evaluate the safety of dams given their current state. Special and unique issues arise while evaluating the safety of embankment dams, both new and old. In this way, preserving human life, minimizing property damage, and maintaining services are the key driving forces behind dam safety review. As the result The most appropriate and are becoming more and in-demand. The technique developed in this study can be utilized to increase public knowledge of dam safty evaluation, making regulation a more feasible effort. Rehabilitation of our aging dams will be highlighted as a crucial duty (USACE 2014).

The evaluation of safety of both new and existing embankment dams presents special and unique problems. In this way, the main motivation of safety evaluation of dams is to save

human lives, reduce property damages and provide continuous services. Therefore, the most suitable and reliable performance evaluation methods have great scope and thus, they are increasingly demanding (USACE 1994).

Some of the most valuable learning has come from projects where failure has been clear in retrospect. The basic factor in the ability of today's relatively stable dams is their ability to learn from the former failures (Mahdi and Alireza 2016).

Another significance of the study is to use the finite element method as a good alternative for seepage, slope stability and deformation analysis (CO-SAERAR 1993). Besides, the study may also be utilized by the later researchers intending to work on the same study area. Moreover, this study will be a guideline for designers and professionals of this area in which the design, analysis and maintenance of failed and existing embankment dam.

So it is always essential to study causes of failure and mitigation measures of failed dams. The results and findings of the proposed research work would be expected to have a great importance to evaluate the performance (seepage, slope stability and deformation analysis) of the existing Aguat wuha embankment dam, which was constructed mainly for irrigation purpose.

1.7 Structure of the Thesis

This Thesis is organized into six Chapter

Chapter One deals with the general introduction, statement of the problem and objective of the study.

Chapter Two gives Literature Review on the Evaluation of Seepage, Slope Stability and Deformation in Aguat Wuha Embankment Dam under Static and dynamic Condition.

Chapter Three gives an overview of the methodology of the thesis, it includes the location of the study area, data collection, modeling and analysis.

Chapter Four is the Result and Discussion part of the work namely the Evaluation of Seepage, Slope Stability and Stress Deformation of the dam.

Chapter Five is about Conclusion and Recommendation.

Chapter Six list of the References and Appendices are included.

2 LITERATURE REVIEW

2.1 Dam Safty

Safety is the primary and most important reason for monitoring seepage, stability and deformations of dams. Nowadays, dam safety draws increasing attention from the public (FEMA 2000). This is because inundations resulting from dam failures can lead to devastating disasters with tremendous loss of life and property, especially in downstream areas. Obviously, of dam safety evaluations and failure analysis is of critical importance for disasters prevention and mitigations. Hence, a robust understanding and assessment of the characteristics of the different types of dams, examine the primary failures (e.g., failure mode, cause, and key influence factors), and explain what to look for when performing a safety inspections is needed (Narita 2000).

2.1.1 Embankment Dam Safety Issues and Failure Causes

To avoid high uplift pressures, piping, sloughing, material loss by solution, and the development of cracks, joints, and cavities, seepage through the embankment, foundation, and abutments must be collected and controlled. The design should take into account seepage prevention techniques such foundation cutoffs, sufficient and strong impervious zones, transition zones, drainage blankets, upstream impervious blankets, and relief wells. The requirements for a safe design must be so detailed as to account for every potential failure mode.

The foundation treatment, abutment stability, seepage conditions, stability of slopes next to control structure approach channels and stilling basins, stability of reservoir slopes, and capacity of the reservoir to hold the stored water are the main design elements of an embankment dam. Before beginning thorough stability or seepage assessments, these qualities should be researched in relation to field conditions and different alternatives. Because the foundation's unknown material properties make the construction of an embankment dam difficult, this study is necessary. Earth dams should be created to best take advantage of the material that is at hand and to fit the site's actual characteristics.

Earth dams should be designed to utilize available material to the best advantage and to conform to actual conditions at site. “ the characteristics of the particular site have a great influence on the design of an earth dam than they do on many other engineering structure (Sherma 1973 and Narita 2000).”

The strengths of the foundation and building materials may occasionally have a significant impact on the detail design, but seepage considerations usually lead to the abandonment of

the fundamental characteristics (P. Novak 2000).

2.1.2 Causes of Embankment Dam Deformations

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

The volumetric changes are due to either an increase in the normal stresses on a soil element causing a decrease in void volume or expansion of soil elements undergoing shear. Lateral spreading and shear displacements are due to squeezing, distorting, and localized shear failures of material elements as the materials adjust to the stress conditions imposed by constructing the embankment and operating the reservoir. The rate at which these deformations happen depends on the dissipation rate of excess pore pressures and the rate at which steady-state seepage conditions develop (USBR 2014). The magnitudes and directions of embankment deformations are controlled by foundation and embankment material properties, abutment and embankment geometry, reservoir loading conditions, type of construction equipment used and embankment placement rates, and stress distribution within the various zones or layers of the embankment and its foundation, Compute the distribution of minimum factor of safety.

2.1.3 Patterns of Embankment Dam Deformations

The first filling of a reservoir may cause significant movement of the crest, but there after deformation will slack off (Chugh 2014). Some areas of deformation are vertical (settlement), upstream and downstream (normal to the dam axis) and cross-valley (parallel to the dam axis). For the maximum section of the dam, the general pattern of deformations for the upstream surface is down and upstream, while the downstream surface, moves down and downstream. On the other hand, the crest of the dam moves down and upstream during first filling, and down and downstream as reservoir water begins to penetrate the dam.

The magnitudes of horizontal deformations (into and down valley) are relatively small compared to the vertical settlement. The exact ratio between the magnitudes varies with geometry, dam zoning, and material properties.

2.1.4 Effects of Deformations and Mitigations

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

In general, the main causes of embankment dams are as follows:

1. Overtopping of the reservoir water due to flooding or loss of free board
2. Seepage through the dam and foundation, causing pore water pressures in the fill and the foundation
3. Settlement of dam and foundation
4. Deformations due to internal and external stresses
5. Slope stability (both upstream and downstream)

Table 2.1 Failure Causes of Embankment Dams (Narita 2000)

The condition of an earthen dam	Failure Causes
During Construction	➤ Pore-water pressure built-up during construction ➤ Reduction of shear strength due to thixotropic property
After Construction	➤ Overtopping ➤ Hydraulic fracturing /Internal erosion/ Piping ➤ Excess hydrostatic pressure due to rapid draw-down ➤ Reduction in shear strength, swelling of compacted soil ➤ Settlement and cracking ➤ Earthquake forces

The following criteria are commonly accepted for safe design of embankment dams.

1. There shouldn't be a chance of the dam portion being overtopped.
2. The seepage line must to be entirely within the downstream face of the dam section.
3. There shouldn't be any chance of "piping" through the foundations or the embankment.
4. There shouldn't be any space where water might easily escape from the u/s and d/s faces.
5. Under the most extreme circumstances to which they may be subjected, the u/s & d/s slopes of the dam should be secure against sliding.
6. The reservoir's waves will be present on the u/s face.
7. Under all construction and reservoir operation circumstances, including seismic ones, the embankment, base, and abutments must be stable.
8. Free board must be enough to prevent wave overtopping and, when applicable, must also account for foundation and embankment settlement as well as seismic effects (USACE 2003).

2.1.5 Deformation and Failures of Embankment Dams

2.1.5.1 Deformation Analysis

An embankment dam's deformation can be divided into three categories:

1. The deformation produced during the building phase
2. The distortion brought on by reservoir encumbrance
3. Long-term deformations brought on by creep and consolidation

Because a dam has a higher confidence coefficient than any other dam prior to impoundment as a result of the reduction in pore water pressure distribution, the building phase is not taken into consideration in the stress-strain calculations. The initial catchment, however, must occur once the dam's construction is complete and the necessary amount of time has passed. During the dam impoundment process in heterogeneous dams, there may be differences in the movements of various materials. As a result, this action is crucial.

Embankment dams in the catchment stage can deform in one of two ways:

1. Deformation caused by the increased water load on the body.

2. Deformation brought on by flooding's destruction of the soil structure in the upstream crust. Leakage in the dam body and foundation will be identified at this point, and it will take time for them to get consolidated.

2.2 Earthen Dam Seepage and Failure Modes

Earthquakes, unstable foundations, deformation and degradation, seepage pipes, and deformation and degradation can all lead to earth dam failure. However, seepage issues constitute a factor in the majority of documented failures worldwide. Pre- and post-construction monitoring are crucial to preventing failure of earth dams caused by seepage, settlement, and piping. Continuous field observations of pore water pressures and deformation are required during the building of earth dams, and field observations of seepage and the piezometric head are typically made following completion. Without the observations, a dam failure might occur suddenly, causing a substantial amount of water to be released abruptly and sometimes with little to no notice, leading to significant losses in human life and property damage. Earthen dam seepage and failure mechanisms. Failure of an earth dam might have many different causes. According to numerous statistics, seepage or piping were the primary causes of earth dam failure, so it is generally advised that the safety of the dam be controlled by monitoring seepage through the earth dam (Bulletin 1995).

2.2.1 Effects of Uncontrolled Seepage on Embankment Dams

2.2.1.1 Seepage Failure

Monitoring seepage, stability, and deformation of dams is done primarily and most importantly for safety reasons. Dam safety is receiving more and more public attention today. (FEMA 2000). This is due to the fact that flooding brought on by dam failures can result in catastrophic events that cause a great deal of property damage and loss of life, especially in downstream areas. Obviously, failure analysis and dam safety assessments are crucial for disaster mitigation and prevention. Consequently, it is necessary to have a thorough understanding and assessment of the features of the various types of dams, examine the principal failures (such as failure style, cause, and key influencing variables), and describe what to look for when conducting safety inspections (Narita 2000). According to an analysis of past dam collapses, internal erosion brought on by seepage has been the cause of over half of all embankment dam failures.

Piping:- Water seeping through the dam's body and carrying away small fragments of dam material causes piping, a failure mode where the gap continues to grow bigger. The solution

might be possible if the original piping can be found before it reaches the crucial stage. Slope collapse may result from water leaking into the dam body. Appropriate instrumentation is required to determine the rate of infiltration within an embankment in order to prevent this type of failure (USACE 2004). Piping happens when soil erosion starts at a seepage exit point and proceeds backward through the foundation or dam, with the surrounding soil acting as support (a roof) to maintain the forming pipe open traditionally.

Internal Migration:- When the soil cannot support a roof or a conduit, internal migration can happen. A temporary void forms as a result of soil erosion and expands until a roof can no longer hold it, at which point the void collapses. Progressively, this technique is used until the void shortens the seepage channel, causing unchecked erosion and, eventually, a dam breach (USACE 2004).

Scour:- Scour occurs when tractive seepage forces along a surface (e.g., a crack within the soil, adjacent to a wall or conduit, or along the dam/foundation contact) are sufficient to move soil particles into an unprotected area. Once this begins, failure from a process similar to piping or progressive erosion could occur. Suspected examples of scour at reclamation facilities include the internal erosion events at Steinaker and Fontenelle dams, as well as the failure of Teton Dam. Internal Instability - are internal erosion mechanisms that can occur with internally unstable soils. Suffusion typically involves little or no change in volume of the soil mass whereas suffusion differs from suffusion because the coarser materials are not in point-to-point contact and, therefore, when they erode out, volume change is observed (e.g., sinkholes, depressions) (USACE 2004).

In fact, the most harmful process in embankment dams is piping, which can be brought on by cracking and micro infiltration and refers to the progressive development of internal erosion by seepage that shows downstream as a hole discharging water (Avella 1993).

2.2.1.2 Development of Pore Pressures and Seepage Forces

Uncontrolled or poorly controlled seepage can lead to the formation of high pore pressures and/or high seepage forces in an embankment dam or its foundation. The presence of high pore pressures in the embankment or foundation results in a decrease of effective stress/strength in those soils, which makes the embankment more vulnerable to a slope failure. Similarly, seepage passing through an embankment generates seepage forces acting along the flow lines (USACE 1994). For seepage exiting at or above the downstream toe, these seepage forces add to the driving forces and reduce the safety factor against stability.

In some cases, failure results not from a deep-seated slope failure, but from progressive sloughing of the downstream slope. This is a key reason that embankments are zoned or contain internal filters and drains to ensure that the downstream portion of the embankment is out side the seepage zone (remains unsaturated and strong) and can provide support for the central portion of the dam.

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

2.2.2 Seepage Analysis Through Embankment Dam

Seepage flow of water through porous media depends on the soil media, type of flow, properties of a liquid and hydraulic gradient. Seepage piping accounts for approximately 50% of all earth dam failures. Many methods have been developed to solve seepage problems, these methods can be classified as analytical, experimental, and approximate methods. Dupuit, Schaffernak, Pavlovsky, Cassagrande, and Polubarinova-kochina, developed approximate methods to calculate the seepage quantity and the free surface seepage line of earth dams.

The Finite Element Method (F.E.M.) is important to the solution of the steady state seepage problems through multi-zoned non homogeneous porous media (S. Ismael et. al 2011).

Bulletin (1995) used experimental models to determine discharge, pore water pressure, seepage face and free surface profile. Most of mentioned methods are valid for simple geometries, isotropy and homogeneous soil media, but with complex geometry, anisotropy, non-homogeneous and non linear material behavior these methods are difficult to be used and may limit the solution accuracy.

Therefore, during the last 30 years, considerable advances have been made in the theory and practice with the help of high speed digital computers. As a result, the numerical methods (finite element, finite difference and boundary element) and their applications are widely used for the solution of many engineering problems (Bahru 2014).

The finite element method (FEM) is one of these numerical methods that can be used for accurate solution of many complex engineering problems. Over the years, the FEM seems to be the most powerful and versatile tool over other numerical methods for solving a wide variety of practical problems efficiently.

2.2.3 Estimation of Seepage

2.2.3.1 Numerical Model Method

For seepage analysis discretize the continuous flow medium into manageable units and then reunite the discrete parts through the use of continuity equations and describe the internal mechanism of flow by the Darcy equation. Computerized numerical methods are recommended because of;

- Complex systems can be modeled easily
- No transformation of dimensions
- Parameter variation is much more expedient and
- Results are printed digital form for each node

The computer program is 2D SEEP/W finite element program

Much computer software has come in general use, and any hard computations and simulation can be carried out through them by giving them appropriate inputs and data. SEEP/W is a finite element which is capable enough to solve steady state vertical section modes of the embankment and or foundation. The numerical model SEEP/W can be employed to carry out simulation of seepage and Phreatic surface. The SEEP/W program is capable enough to simulate quite effectively seepage rates and phreatic surfaces in homogenous and non- homogenous dams, groundwater flow, seepage and excessive pore water pressure problems

within the porous media such as soil and rock (Krah 2012).

2.3 Modes of Slope Failures and Stability Analysis

2.3.1 Slope Stability Analysis

Typically, seepage issues precede or follow stability issues in embankment dams. Prior to performing a stability study, it is crucial to understand the seepage that is occurring through the dam and its foundation. The best indicators of a dam's safety are measures of the pore-water pressure and seepage (USACE 2006).

The most used methods and approaches to analyze and discuss the stability of slopes are the limit equilibrium method (LEM) and finite element method (FEM). The principal difference between these two approaches is that the LEM is based on the static equilibrium whereas FEM utilizes the stress-strain relationship or constitutive law. The FEM is more versatile and leads to good approximation instability analysis than that of the most widely used LEM (Abdullah 1999).

When material is forced downward by gravity and shear pressures more than it can be forced upward by shear strength, slope failure results. As a result, the likelihood that a slope may fail increases when factors that tend to raise shear loads or lower shear strength are present. Accordingly, embankment dams are highly subjected to slope failures due to:-

1. The gravitational forces that try to push down parts of the soil mass adjoining the slope
2. The excessive and abrupt increase of pore-water pressure, such as the one built-up during construction and the residual one due to rapid drawdown of the reservoir, may cause sliding failures in embankment (Duncan and Wright 2005).

During the static slope stability analysis of the embankment dam recommended minimum factors of safety for various slip surfaces is to be provided for criteria. In order to determine the stability analysis applicable parameters such as loading conditions, geometry, material properties, and pore pressures are to be considered.

2.3.1.1 Limit Equilibrium Method (LEM)

The stability of earth slopes has been studied using a variety of limit equilibrium methods for many years (Bishop 1954). Improvements of the standard or Swedish method of slices, which is introduced by (Pham et. al 2013).

Due to the ability to more easily manage the iterative procedures that come with the method made available by the development of electronic computers in the 1960s, mathematically more rigorous formulations, such as those created by Morgenstern price and displayed in Figure 2.1 below were produced (Duncan et. al 2014).

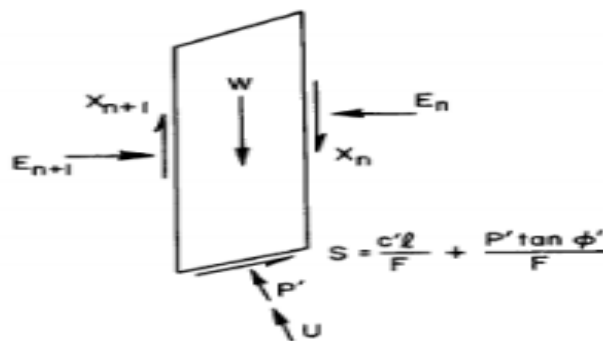


Figure 2.1 Slice Discretization and Slice Forces in a Sliding Mass

Fellenius (1936) introduced the first method, referred to as the Ordinary or the Swedish method, for a circular slip surface.

Bishop (1955) advanced the first method by introducing a new relationship for the base normal force.

Janbu (1954) created a more straightforward method for non-circular failure surfaces at the same time by cutting a prospective sliding mass into many vertical slices. Simultaneously with the advancement of the simplified approach, the generalized procedure of slices (GPS) was established (Janbu 1973).

With new assumptions for the inter-slice forces, (Morgenstern Price 1965) later created more contributions. As a development of the Spencer and Morgenstern-Price procedures, (Chugh 1986) created a general limit equilibrium (GLE) procedure that satisfies the conditions for moment and force equilibrium (Krahn 2004). All LE methods are based on certain assumptions for the inter slice normal (E) and shear (T) forces, and the basic difference among the methods is how these forces are determined or assumed. In addition to this, the shape of the assumed slip surface and the equilibrium conditions for calculation of the FOS are among the others. These methods typically involve cutting the slope into fine slices and applying appropriate equilibrium equations (equilibrium of the forces and moments). The differences between these methods depend on which inter slice forces are included and satisfied by the equations of statics, which inter slice shear and normal forces are included, and what is assumed to be the relationship between the inter slice shear and normal forces (Bishop 1954).

Table 2.2 Summary of LE Methods (Abramson et al. 2002).

Methods	Moment Equilibrium	Force Equilibrium	Assumptions for T and E
Ordinary or Fellenius	Yes	No	Neglects both E and T
Bishop Simplified	Yes	No	Considers E, but neglects T
Janbu's Simplified	No	Yes	Considers E, but neglects T
Spencer	Yes	Yes	Constant inclination, $T = \tan\theta E$
Morgenstern.-Price	Yes	Yes	Defined by $f(x)$, $T = f(x) \cdot \lambda \cdot E$
Corps of Engineers-1	No	Yes	Resultant inclines at, $\theta = \frac{1}{2}(\alpha_1 + \alpha_2)$
Corps of Engineers-2	No	Yes	Resultant inclines at, $\theta = \frac{1}{2}(\alpha_1 + \alpha_2)$

Limit equilibrium (LE) methods use the Mohr-Coulomb failure criterion to determine the shear strength along the slip surface. A state of limit equilibrium exists when the mobilized shear stress is expressed as a fraction of the shear strength. At failure, the shear strength is fully mobilized along the critical slip surface. The factor of safety against slope failure is calculated as the ratio of the available shear strength to the mobilized shear strength. The available shear strength depends on the type of soil and the effective normal stress, whereas the mobilized shear stress depends on the external forces acting on the soil mass.

In LE analysis, the sliding mass is divided into slices, determination of the shear and normal inter-slice forces is made, and appropriate force and/or moment equilibrium equations are satisfied for static equilibrium conditions. The first LE method for around slip surface was presented by Fellenius (2003).

Bishop (1955) Later developed a revised method of circular slip analysis. Janbu (1954) Presented a technique for non-circular failure surfaces that isolated a potential sliding mass into a few vertical cuts. Later techniques were developed by (Morgenstern and Price 1936), (Spencer 1967), (Sharma 1973) and a few others to make further advances with regards to the various assumptions about inter-slice forces.

2.3.1.2 The LE Method Requires the Following Assumptions

Each block within the slip surface has the same FOS, inter-slice forces are assumed, the soil acts as a Mohr-Coulomb material, and all of the soils involved have the same FOS for the cohesive component of strength and the frictional component of strength. These factors combine to make the problem determinate (Aryal 2008).

2.3.1.3 Vertical Slices

The LE method utilizes the method of vertical slices, the vertical slice method is where “the entire sliding mass is divided into a reasonable number of slices and the inter-slice forces are calculated based on an assumed inter-slice force functional relationship” (Aryal 2008).

Slip surfaces are assumed and the static equilibrium equations are used to calculate the stresses and FOS on each slice (Chen and Liu 1999).

The slip surface is a surface where sliding is assumed to occur; this slip surface may be circular, or a shape defined by straight lines.

Duncan and Wright (2005) States that when using the LE method the Morgenstern-Price procedure should be adopted as it satisfies all requirements for static equilibrium requirements for both forces and moments. The Morgenstern-Price procedure creates ‘blocks’ dividing the soil above the slip surface.

2.3.2 The Finite Element Method (FEM)

In geotechnical engineering analysis, numerical modeling has grown in popularity as a useful technique for resolving a variety of engineering issues. Finite element (FE) and finite difference (FD) methods are the two types of numerical methods that are used the most frequently. The finite element method uses the soil stress-strain behavior for slope stability modeling. Unlike LE methods, there is no pre-assumption regarding the shape and location of the failure slip surface. This is considered an advantage of this method over the traditional LE methods where FOS are calculated for a pre-determined failure surface. Since there is no concept of slices in FE analysis, there is no need for assumptions about the lateral inter-slice forces between adjacent slices. The FE method preserves global equilibrium until failure is reached, and is able to monitor progressive failure up to and including overall shear failure (Vinod et al. 2017).

The FE approach divides the model into a number of pieces or elements of a mesh. Stresses and strains are calculated using the constitutive laws for materials comprising the slope stability model. Failure occurs naturally through the zones in which the soil shear strength is unable to sustain the applied shear stresses. Ultimately, a reduction factor (RF) can be calculated for finite element methods using the ‘ ϕ -c reduction’ procedure. This approach requires incrementally reducing the soil strength parameters until the failure occurs. The shear strength reduction technique enables the FE method to calculate FOS (equivalent RF) for slopes

PLAXIS 2D program was used, which has a series of advantages (Plaxis 2010):- such as Ability to deal with excess pore-pressure phenomena. Excess pore pressures are computed during plastic calculations in undrained soil, Better insight into soil-structure interaction, It can reproduce advanced constitutive soil models for simulation of non-linear behavior. It includes more advanced material models or Mohr-Coulomb, Soil Creep.

Dupuit, Schaffernak, Pavlovsky, Cassagrande, and Polubarinova-kochina, developed approximate methods to calculate the seepage quantity and the free surface seepage line of earth dams.

Table 2.3 Comparison of Limit Equilibrium and Finite Element Methods (Aryal 2008)

Slope analysis method	Limit equilibrium method	Finite element method
Analysis principle	Slice method, equilibrium of force and moments, simple theory	FEM, boundary condition and constitutive equations, elasto-plastic theory
Input data	Cohesion, friction angle	Constitutive model data
Analysis results	Safety factor, critical section, no displacement and stress, assumptions to failure surface	Deformed mesh, stress distribution, safety factor, displacement

2.3.3 Drained and Undrained Conditions

It is essential considering the concepts of drained and undrained conditions at the starting point of this discussion of soil mechanics principles since they are of fundamental importance in the mechanical behavior of soils. Drained is the condition under which water is able to flow into or out of a mass of soil in the length of time that the soil is subjected to some change in load. Under drained conditions, changes in the loads on the soil do not cause changes in the water pressure in the voids in the soil, because the water can move in or out of the soil freely when the volume of voids increases or decreases in response to the changing loads. Undrained is the condition under which there is no flow of water into or out of a mass of soil in the length of time that the soil is subjected to some change in load. Changes in the loads on the soil cause changes in the water pressure in the voids, because the water cannot move in or out in response to the tendency for the volume of voids to change (Duncan 2005).

2.3.4 Parameters of Static Loading Condition

The various loading conditions to which an embankment and its foundation may be subjected and which should be considered in analyses are designated as follows:-

1. End-Of-Construction Stability

Depending on the permeability of the soil, analysis of slope stability during construction and afterward uses either drained or undrained strengths. There is little drainage during construction since a lot of fine-grained soils are adequately impermeable. For clays in particular, this is true. Undrained shear strengths are employed for these fine-grained soils, and total stresses are used to describe the shear strength. Drained strengths are employed for

soils that drain easily, shear strengths are stated in terms of effective stresses, and pore water pressures are determined using either water table data or a suitable seepage analysis. In the same analysis, undrained strengths for some soils and drained strengths for others can be used.

2. Steady State Long-Term Stability

The soil on slopes may either swell (with an increase in water content) or consolidate (with a decrease in water content) over time following construction. To reflect the circumstances after these changes have taken place, long-term stability evaluations are carried out. Effective stresses are used to represent shear strengths, and pore water pressures are evaluated using the most damaging ground water and seepage scenarios projected over the slope's lifespan. Depending on the complexity of the cross section, seepage assessments can be carried out using either graphical methods (flow nets) or numerical methods (finite element, finite difference).

3. Rapid (Sudden) Draw down

The stabilizing effect of the water pressure on the upstream slope is lost when the reservoir level behind an embankment dam is decreased. The slope is less stable if the water level is reduced quickly enough that the pore pressures in the slope do not have time to attain equilibrium with the new reservoir water level.

Two fundamentally distinct approaches have been used to examine stability after the conclusion of fast drawdown:

- Utilizing stress-reduction techniques
- Utilizing complete stress techniques

Free-draining materials are handled the same way by both techniques. Depending on the specific slope, the pore water pressures are computed assuming either continuous seepage or hydrostatic conditions, and the strengths of free-draining materials are described in terms of effective stresses.

2.3.5 Material Properties and Pore Water Pressure Parameters

Shear Strength

Shear strength is defined by Duncan and Wright (2005) as, “the maximum shear stress that the soil can withstand.” The proper assessment of shear strength for slope stability analyses is a critical aspect of understanding and predicting embankment slope stability performance.

Shear strengths are typically assessed by laboratory experiments carried out on materials generated by laboratory compaction or undisturbed samples collected from fast growing soil borings. The data from the laboratory tests may be complemented with results from on-site field tests and correlations between shear strength parameters and other soil characteristics including grain size, plasticity, and Standard Penetration Resistance (N) values (Hunter and Fell 2003).

A Mohr-Coulomb failure envelope is used to represent shear strength for all slope stability analyses discussed in general. This failure envelope links shear strength to either the total or effective normal stress on the failure plane. When there are total stresses, the shear strength is written as follows:-

Shear strength (available):

$$\tau_f = c' + (\sigma - \mu') \tan(\phi') \dots \dots \dots 2.1$$

Shear stress (mobilized):

$$\tau = \frac{\tau_f}{\text{FoS}} = \frac{c' + (\sigma - \mu') \tan(\phi')}{\text{FoS}} \dots \dots \dots 2.2$$

Where FoS, σ , μ' , c' , τ_f and ϕ' are factor of safety, normal stress, pore water pressure, effective cohesion, and shear stress required for equilibrium and effective friction angle respectively.

The recommended minimum factor of safety (FoS) provided for slope stability analysis is expressed as the ratio between the shear strength, and the shear stress required for equilibrium, and this becomes:-

$$\text{FoS} = \frac{\tau_f}{\tau} = \frac{c' + (\sigma - \mu') \tan(\phi')}{\tau} \dots \dots \dots 2.3$$

Finer soils become weaken their shear strength conditions due to many factors. Among the factors causing reduced strength of a soils mass, increased pore pressure is very important.

2.4 Comparison from Other Studies

In the world a number of studies have been made and compared the results of slope stability analyses using the LE and FE methods. Griffiths and Lane (1999) Compared results of six simplified slope models generated using FE analysis to other traditional LE methods. It was

concluded that the FE method was powerful alternative to traditional LE methods and its wide spread use should now be standardized in geotechnical practice.

Another study Hammah, Yacoub, Corkum, and Curran (2004) compared FE analysis of slopes to several LE methods on a wide range of slope cases. Although the author of this study recommended adopting, the FE method using shear strength RF as an additional robust and powerful tool for design and analysis, conducting further research using this method was also recommended (Chowdhury 1978).

Chowdhury (1978) performed slope stability analysis of homogeneous and non homogeneous simplified slope cases with various material properties in order to compare the LE results with the FE method results. The study concluded that the results are generally in good agreement for homogeneous slopes. It was further concluded that both the LE and FE methods have their own merits and limitations, and the use of FE method is not superior to the use of LE methods in routine analysis and design. While both methods provide an estimation of the FOS and probable failure mechanism, engineers should appreciate the limitations of each method when assessing the results of the respective analysis.

Khabbaz et.al. (2012) presented the results of a study comparing the limitations and advantages of the LE and FE methods. The findings indicated that when a simple homogeneous slope is considered, the difference in the FOS and locations of the critical slip surface are minimal and both methods generate indistinguishable results. Furthermore, this study states that differences between the FE and LE methods are evident when a heterogeneous slope is analyzed.

Vinod et.al (2017) concluded that FOS differences between LE and FE methods are very small. A recent study Zein and Karim (2017) compared results of LE and FE slope stability analysis of simple slope models founded on untrained clay soils. Results of this study indicated that although LE methods are simple and relatively fast, relatively accurate and reliable results can be produced, and such methods may be applied with confidence in routine design for slopes with simple geometry. Lastly, the study concluded that the FE method is an advanced and valuable analysis technique, especially while modeling design cases with complex slope geometry, heterogeneous soil behavior and different loading patterns. Now a days in Ethiopia different researches have done using GeoStudio software to evaluate seepage slope stability and deformation under static condition.

Hasani et al. (2013) did a research related to seepage analysis in Ilam earth fill dam by using seep/w software. In order to evaluate the type and size of mesh size on the total flow rate and

total head through the dam cross-section, four mesh sizes such as coarse, medium, fine and unstructured mesh are considered. The result showed that average flow rate of leakage under the different mesh size for Ilam dam equal 0.836 liters per second for the entire length of the dam.

Sitharam and Hegde (2017) tried to study on stability analysis of the rock-fill tailing dam constructed at Rampura Agucha zinc mine in Rajasthan, India using limit equilibrium method and finite element methods. The results from the limit equilibrium approaches were validated with the shear strength reduction technique using finite element based PLAXIS 2D analysis. The factors of safety values calculated from the different approaches were found to be in satisfactory agreement with each other. But, the limit equilibrium method overestimated the factor of safety as compared to the finite element method.

Shivamant et al. (2015) investigated the stability analysis of a dyke using LEM and FEM approaches. For this purpose, analyses have been carried out using Slope/W and PLAXIS 2D for different construction and operating conditions. FOS result obtained from LEM was 1.75 whereas FOS value of PLAXIS 2D was 1.57. In this study result, the LEM gives about 12 % bigger safety factor than the FEM approach.

Wood (2000) showed that a numerical study of the seismic behavior of earth fill dams. It mainly focuses on seismic amplification at the dam crest under real earthquake records with frequency content close to the natural frequency of the dam. Since the 1971 San Fernando earthquake in California (Ming & Li 2003), major progress has been achieved in the understanding of the earthquake action on dams.

Recently, the FEM program PLAXIS 2D has become popular for slope stability analysis because of its excellent capability to model hydraulic and geotechnical engineering stability problems and its extended programming ability (Plaxis 2010).

Khabbaz et.al (2012) Presented the results of a study comparing the limitations and advantages of the LE and FE methods. The findings indicated that when a simple homogeneous slope is considered, the difference in the FOS and locations of the critical slip surface are minimal and both methods generate indistinguishable results. Furthermore, this study states that differences between the FE and LE methods are evident when a heterogeneous slope is analyzed

According to data from Hunter and Fell (2003), there may be empirical relationships between the lateral displacements in earthen fill and zoned earth dams and their proportion to dam

height. When a reservoir fills for the first time, embankment dams normally move laterally by between 0.02% and 0.20% of the embankment height, or 50 mm upstream to 200 mm downstream. Displacements are normally less than 0.2% of the embankment height for the downstream slope (mid to upper region), and they typically range from 0 to 200 to 250 mm.

Mohsen and Hamid (2015) used SIGMA/W software's finite element analysis to investigate the impact of embankment dam height variation on the analysis of stress and strain on heterogeneous embankment dams. Two heterogeneous dams were the subjects of the investigation, and the materials used in both were thought to have similar properties. Based on an analysis of the total stress, shear stress, and strain values on the dam, it can be determined that higher dams experience greater stresses and strains than shorter dams, demonstrating that height plays a role in increasing the stresses and strains generated in the dam. Additionally, the parameters of stress, strain, and displacement in each dam are greater when acting vertically than when acting horizontally.

2.5 Previous Study Cases in Ethiopia

Nowadays in Ethiopia different research have done using Geo Studio and Plaxis 2D software to evaluate seepage, slope stability and deformation under static condition.

Ksatie (2016) has attempted to study the evaluation of seepage, slope stability and deformation in the Grindeho embankment dam using Geo-Studio software. The result showed that the quantity of seepage flow rate through the dam at the current reservoir condition is equal to $0.000618 \text{ m}^3/\text{s}$. However, the actual seepage recorded at the downstream toe of the dam is $0.00124 \text{ m}^3/\text{s}$. The researcher concludes that the difference in seepage quantity is come from the left and right abutments, from the dam body and its foundation. The researcher tries to determine the factors of safety using SLOPE/W software and the FOS result was within the acceptable limit based on USACE recommendation. However, the researcher tries to determine the static stress-deformation analyzed using SIGMA/W software and the Maximum horizontal and vertical displacement results were within the acceptable limit based on Hunter and Fell's recommendation.

T/Haymanot (2017) analyzed the earthen dam (on the Ribb earthen dam) using FEM software for seepage, slope stability, and earthquakes. The analyses were performed using finite element software, and the results showed that the safety of the dam is not at risk from the perspective of seepage, and all factors of safety, as well as displacements, are safe at all critical loading conditions with the exception of when the earthquake and rapid drawdown

conditions are assumed to occur at the same time, in which case the factor of safety is found to be about 0.996.

Melka (2015) has attempted to study performance evaluation and remedial measures of Kesem embankment dam using Geo-Studio software. The result showed that the average flow rate of seepage for the entire length of the dam at the current reservoir condition is equal to $0.0159 \text{ m}^3/\text{s}$. However, the actual seepage recorded at the downstream toe of the dam is $0.2084 \text{ m}^3/\text{s}$. The researcher concludes that the difference in seepage quantity comes from the hot spring, reservoir water and rising of ground water.

Fitsum (2017) has studied the seepage, slope stability and deformation analysis of earthen dam using FEM software (on Rib earthen dam). The analyses were done using finite element software and the result showed that the dam safety is not endangered from the seepage point of view and all the factor of safety, and displacements are safe at all critical loading conditions.

Abera (2016) has studied the stability of the slope and seepage of the Dora earth fill dam using a numerical model. The result showed that the average flow rate of seepage under the case using different mesh size including foundation seepage was equal to $0.079 \text{ m}^3/\text{s}$ for the entire length of the dam. This value was compared with the quantity of seepage practically measured it was $0.072 \text{ m}^3/\text{s}$. However, the researcher tries to determine the factors of safety using the Slope/w model and the FOS result was within the acceptable limit based on the USACE recommendation. But, Slope/w is not a numerical model because of it uses a limit equilibrium method for slope stability analysis.

Eshete (2016) has attempted to study slope stability analysis of the Gidabo earthen dam using FEM software. The researcher has used a geogrid material to reinforce the strength of the dam and to decrease the cost of construction material. But, the author tries to attempt only to clear the slope stability. After lime treatment, the liquid limit, plasticity index and swelling potential decreased. The permeability also decreased by more than one order of magnitude from 4.47×10^{-6} to 3.63×10^{-7} .

However, many scholars are researched seepage, slope stability, and deformations of embankment dam using Geo Studio they are lacking identifying the seismic hazard zone of the specific area and don't incorporate seismic actions to the dam slope stability under different loading condition's and operations.

Additionally, they do not interpret the stress deformation (Σ/w) results in relation to the general foundation of a dam and its bearing capacity, nor do they interpret the phreatic line property in the seep/w results in relation to the filter and down stream shell of the dam. However, in this research seepage, slope stability, and deformation analysis are performed under static and dynamic analysis and the relationship between stress deformation result and general foundation condition bearing capacity is well analyzed.

The seepage and slope stability of the Koga earth-fill dam are presented in similar studies using Plaxis 2D software. However, they do not take earthquake consideration into account. The flow rate, pore water distribution, and the location of the phreatic line were all analyzed for seepage in this study. Additionally, measurements of actual field data as well as observatory research were done. According to the results of the simulation, the average seepage flow rate along the full length of the dam's body at normal pool level was equal to $0.06085 \text{ m}^3/\text{s}$, while the figure through the dam's foundation was $0.01937 \text{ m}^3/\text{s}$. At the current reservoir level, total seepage through the dam's main body was $0.04982 \text{ m}^3/\text{s}$, whereas $0.04644 \text{ m}^3/\text{s}$ of measured seepage collected at the dam's downstream toe. The similarity between the measured and modelled seepage discharges is 93.2% (Zewdu 2019).

There is no research done in this research study area regarding to seepage, slope stability, and deformation analysis except the design report which has been analyzed using limit equilibrium (ADSWE 2015) and the design report analyses only seepage and slope stability using LEM “SEEP/W” and “SLOPE/W” software respectively. The slope stability both upstream and down stream conditions has the factor of safety value is 2.496 and 2.025 respectively. So the dam condition is safe slope and material composition. The second condition is at maximum and minimum Draw Down condition the factor of safety value for shell dry and shell wet condition is 1.757 and 1.324 respectively. So the dam condition is not safe this implying that the core material should not be gravel with clay. But our materials are black cotton and with clay, which practically impervious stratum. The third condition is at the end of construction the factor of safety value for up stream and down stream is 2.185 and 2.026 respectively. From the design document the amount of seepage at 2875.25, 2874, and 2868 m a.m.s.l, were $1.05 \times 10^{-6} \text{ m}^3/\text{s}$, $1.23 \times 10^{-6} \text{ m}^3/\text{s}$, and $5.24 \times 10^{-7} \text{ m}^3/\text{s}$ respectively.

3 MATERIALS AND METHODS

3.1 Description of the Study Area

Geographical Location

The project area is located at about 580 km from Addis Ababa, 200 km far from kombolcha town, and 5 km from Sayint ajibar (the capital city of Amhara Sayint Woreda). The study area is located in Amhara National Regional State, South Wollo Zone Administration, and Amhara Sayinit woreda bonded with geographical coordinates of $10^{\circ} 54' 0''$ to $11^{\circ} 2' 0''$ N Latitude and $38^{\circ} 50' 0''$ to $38^{\circ} 55' 0''$ E longitude with in the basin of Abbay and specifically Sub basin of Beshilo as shown in fig 3.1: below. The headwork structures are specifically located at an altitude of about 2849 m.a.s.l and geographical coordinates of 486122.144E (UTM) & 1213807.134N (UTM) (ADSWE 2015b)

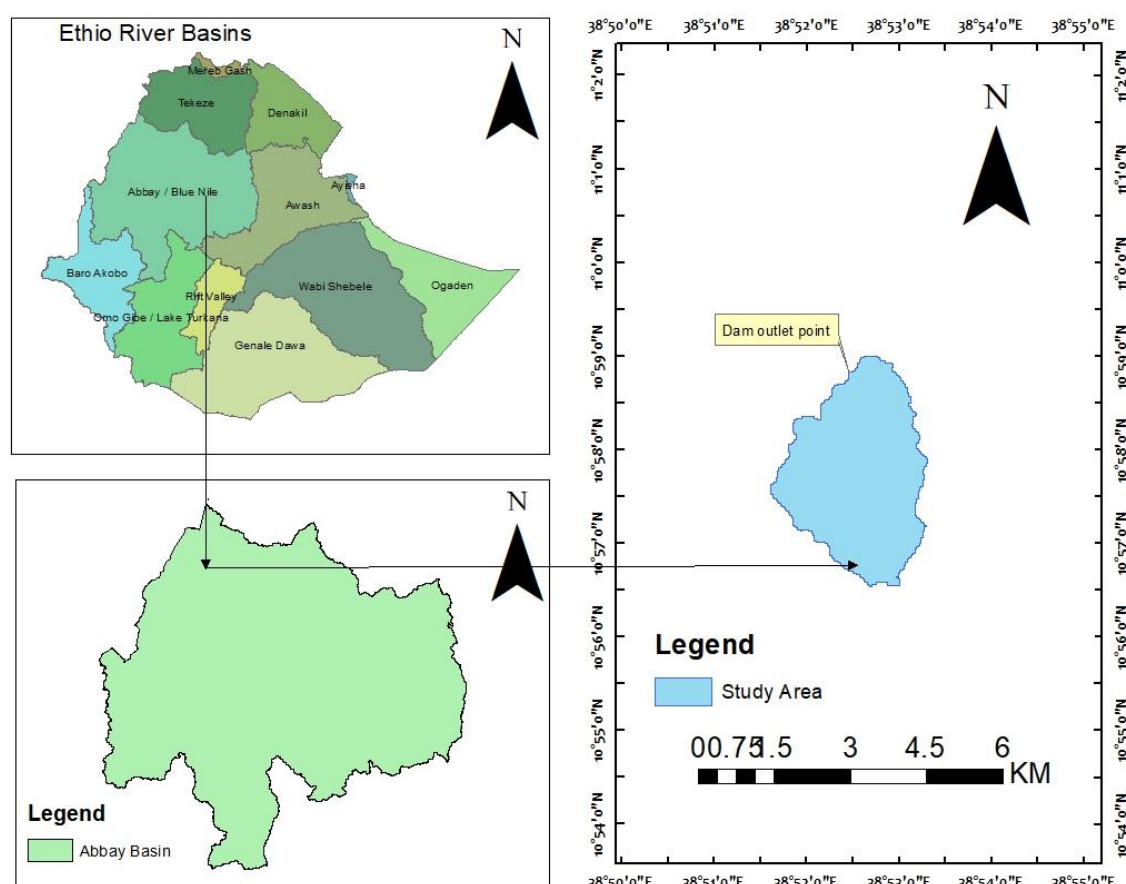


Figure 3.1 Location of Dam Site

Aguat Wuha Micro-Embankment dam is a Heterogeneous Embankment dam with a central clay core followed by a layer of filter on one side and supported by sand and gravel shell (ADSWE 2015a).

Hydrologists and designers are faced with a lack of good or non-recorded hydrometric data on the target stream/river and on local weather and climate conditions. Stream gauging stations are non-existent in remote rural areas of the region and even meteorological stations are almost rare near small rivers in the region. Likewise, at the watershed of Aguat Wuha River, there is a meteorological station near the project area which is Sayint Ajibar meteorological station (ADSWE 2015b).

3.1.1 Climate Condition of the Dam Site

The elevation of the project area ranges from 2856 to 3538 meters above sea level. The watershed lay within two elevation classes, 2300-3200 and 3200-3700 meters, based on which the watershed is set in highland class or Dega and Wurch traditional agro-climatic zones (ADSWE 2015b).

Important climatic variables such as temperature and rainfall in the watershed comprise a single Temperature Range and cold zones. The watershed comprises a single rainfall zone with a minimum rainfall of 900 mm and maximum of 1200 mm and the monthly mean minimum and maximum temperature are 10 °C and 17.5 °C respectively (ADSWE 2015b).

3.1.2 Soil

Land shapes and soil properties are closely related to one another. There are several terrain formations in the watershed. The soil qualities vary depending on the different types of land uses in the watershed for the majority of the mapping units. There are three different soil types, Eutric Cambisols, Lithosols, and Vertic Cambisols. The depth of soil in the watershed ranges from shallow in hills and mountains to very deep in plains (ADSWE 2015a).

3.1.3 Topography

The rate and volume of runoff, and sediment yield from the watershed have much to do with slope and other parameters of the land scape such as contour and elevation. The Watershed has marked topographic variation. Four types of slope classes are present. Moderately steep (15–30%) slopes, which occupy 41.77% of the total area, are the dominating slope class, followed by steep (30–50%) slopes, which account for 26.29% of the total area. There are four different types of slopes: steep (8-15%), gentle (3-8%), extremely steep (>50%), and flat or nearly flat (0-3%) which account for 24.39%, 4.34%, 2.84%, and 0.38% respectively (ADSWE 2015a).

Contours contain information about the relief, shape and form of the land and the relative distribution in space of the components of the land scape of the watershed. The elevations of land marks and reference points, such as hill tops and mountain peaks, are also printed on topographic maps. Following contour lines, it is possible to delineate micro-watersheds within the watershed on a contour map and enable the gradient or slope between two points to be measured. Closely spaced contours on the contour map of the watershed indicate that steeper slopes occur in those locations relative to areas that have less closely spaced contours; whereas sparse contour lines indicate plains (ADSWE 2015b).

3.1.4 Geology and Geotechnical Condition of the Dam Site

Geotechnical and geological factors play a major role in the design, construction and operation of dams. Water tightness, stability and bearing capacity are highly dependent on these factors. The nature and impact of the geological and geotechnical conditions of the dam site and reservoir area are mainly controlled by the nature of the rocks and soils of the area. Feasibility study and design report of Aguat Wuha dam (ADSWE 2015) indicated that the bed rocks are found in the shallow depth both in the left and right valley floor area including the abutments. Whereas it is covered with dark brown clay soil and flood plain silty soil with some recent alluvial deposit. From natural exposures investigation and excavated test pits, this rock is expected within 3 to 5 m depth, where covered with the three horizons within the valley floor area of the dam site the rock is very strong, fresh and massive, and also impervious at most places. Where it is exposed on the surface, both the general and cut-off foundation of the dam can easily lie. The topography and engineering geological foundation conditions of both abutments are different. The right abutment is characterized by relatively steeper slope. It is made up of entirely reddish clay soil on the top (about 0.5m in depth) and Aphanitic basalt bed rock underlies by the soil units.

The left abutment, on the other hand, is characterized by relatively moderate slope. It is made up of top layers of dark brown clay soil and highly weathered aphanitic basalt rock is visible after 4.5 m. The depth of this weathered zone reaches 4.5 to 6 m, below these units fresh parts of rocks units are expected. The valley floor region includes the stream bed and the adjacent moderate slopes up to the points of intersection on each side. The surface of the valley floor area of Aguat wuha dam site is covered by three types of engineering geological/geotechnical materials/layers. These are:- Recent Alluvial deposit, Colluvial soil and Aphanitic basalt.

A) Recent Alluvial Deposit

This soil layer is found at a most portions of the Aguat wuha stream bed surface and underneath a small amount of boulders and cobbles are visible around the bank of the stream. It is a recently deposited cobble having gray color, and loose density. Where out crops of the bed rock are found near the right and left banks along the axis. The bed rock is covered at the valley floor area by the clay soil. Here, the sub surface water flows over the stream bed surface that bed rock is expected at shallow depth and visible at the surface.

B) Colluvial Soil

This soil deposit is found on both sides of the Aguat stream course covering gentle farm lands. It is a flood plain deposit covering both the alluvial, Colluvial and underlying bed rock. At the left side of the stream four test pits (AGFOTP-1, AGFOTP-2, AGFOTP-3 and AGFOTP-4) were dug to depths of 3 m and 5 m the first two and the next two each respectively. The top soil layer is a low plasticity, dark brown clay soil, according to the pits' log. The top is rigid and dry, but the depths become too damp or saturated. It's thickness is variable, and decreases from the river side towards the right abutment feet. It ranges from 5 m to 1 meter and most of these soil units exist in the left valley floor area while small existents of soil units are visible in the right valley floor area at a meter (ADSWE 2015).

The soil is underlain by aphanitic basalt near the stream side, and central right valley floor areas, but by jointed bed rock visible at the stream beds. The contact of the two soil types is marked by sub surface water, especially near the stream bank, and central areas (here the depth to sub surface water is at about 4.5 m about the valley floor area at the left site of the dam). A sample (AQFO-2 and AQFO-3) has also been taken from this soil horizon for laboratory testing (from test pit AQFOTP-2 and AQFOTP-3 from depth of 1 m to 5m) (ADSWE 2015).

C) Aphanitic Basalt Bed Rock

At the valley floor area, the lower bed rock of the site is exposed at the major areas inside the Aguat wuha stream bed. Along the dam axis, the aphanitic basalt out crops are visible at the stream beds. The bed rocks are found in the shallow in both the left and right valley floor area including the abutments. Whereas it is covered with dark brown clay soil and flood plain silty soil with some recent alluvial deposits. From natural exposures investigation and excavated test pits, this rock is expected within 3 to 5m depth, The rock is extremely durable where it is covered by the three horizons in the valley floor area of the dam site, fresh and

massive, and also impervious at most places. Where it is exposed on the surface, both the general and cut-off foundation of the dam can easily lie on it (ADSWE 2015).

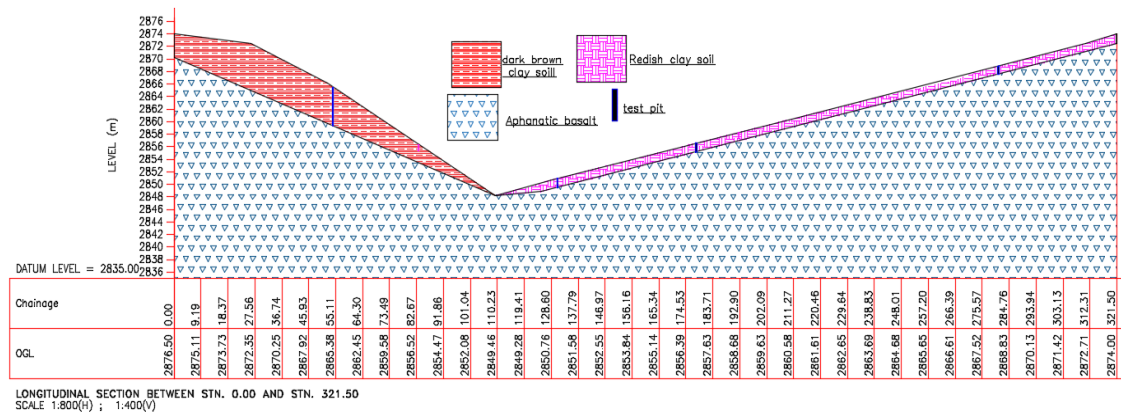


Figure 3.2 Geological Cross-Section of Aguat Wuha Dam (ADSWE 2015)

3.1.5 Effect of An Earthquake on Embankment Dams

The earth quake hazards refer to the whole dam and appurtenant structures system. When it comes to the seismic hazards faced by the body of an embankment dam itself, the most important effects are the following (PLAXIS 2D V8.2 2010).

- Slope instabilities
- Liquefaction flow failures due to excess pore pressure generation
- Longitudinal cracks occur near the crest due to shear sliding deformations and large tensile strains during lateral oscillations
- Differential crest settlements and loss of free board possibly resulting from lateral sliding deformations or soil densification
- Transverse cracks caused by tensile strains from longitudinal oscillations or by different lateral responses near the abutments and near the central crest zone

3.1.6 Seismicity of Project Area

According to the seismic hazard map of Ethiopia, the country has been sub divided in to five seismic Zones as:- Zone (o), Zone (1), Zone (2), Zone (3) and Zone (4), the no hazard, the low hazard, the moderate hazard, the higher hazard and the highest hazard zones, respectively (EBCS-8 1995). The project area is found within the zone 2 where seismic activities are active that dynamic design approach is also important for the proposed engineering structures (ADSWE 2015).

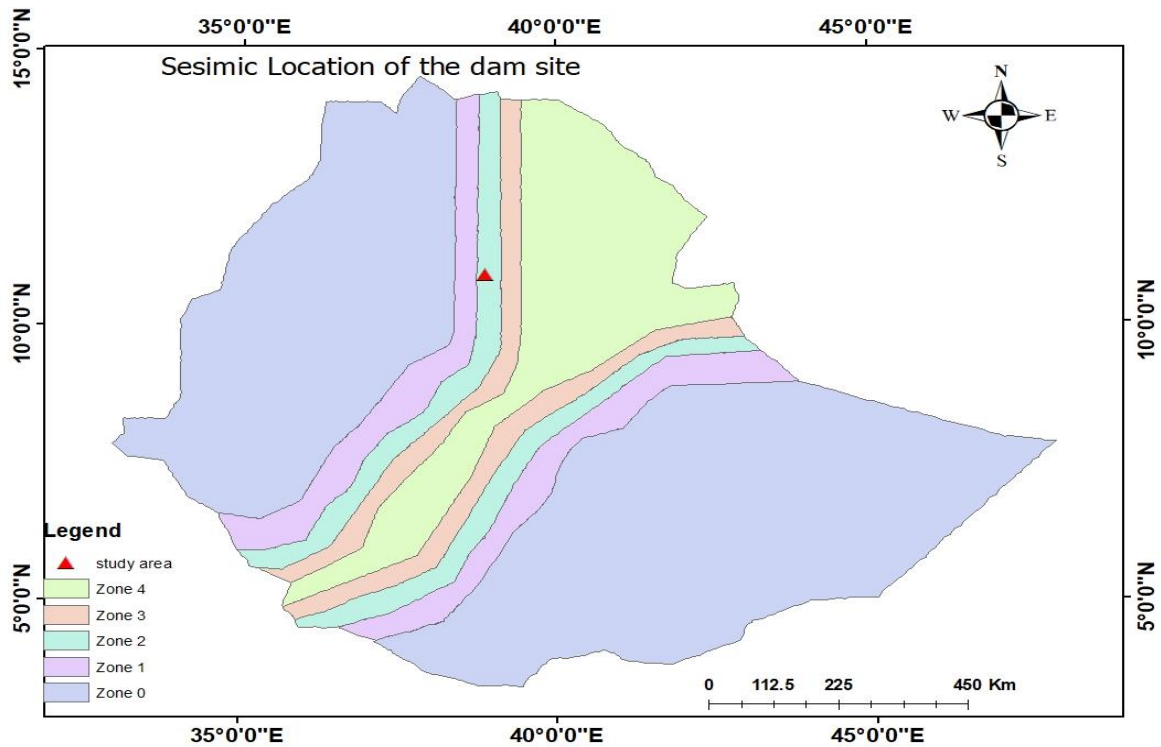


Figure 3.3 Seismic Zone of the Dam Site

3.1.7 Seismic Slope Stability

The stability of slopes can be decreased due to the presence of seismic load. The earth quake ground motions are capable of destabilizing internal forces in soil matrices and also initiate the excess pore water pressures. When the shear strength decreases, the stability of the slope, will be affected. In general, four methods of analysis have been proposed for the evaluation of the stability of slopes during earth quakes which are the Pseudo static method, New mark's displacement method, Post-earth quake stability, and Dynamic finite element analysis (PLAXIS 2D V8.2 2010).

3.1.8 Earth Quake Analysis

Our knowledge of how soil responds to cyclic stress, how earth dams behave dynamically during earthquakes, and how to conduct dynamic analyses dynamically to evaluate seismic stability have all undergone significant advancements. Since the fall of the lower San Fernando Dam (PLAXIS 2D V8.2 2010), which sparked development in this field, there has been a significant amount of study on earth dams using dynamic analytic approaches.

There are some important cases that caused damage and failures after earth quake to some days Lower San Fernando Dam in USA is the first example that failed as a result of the liquefaction phenomenon under earthquake loading conditions. In the case of the May 12,

2008, Wenchuan earth quake in China, many dams and reservoirs had been subjected to strong ground shaking. In general, strong ground shaking can result in the instability of the embankment and loss of strength at the foundations (PLAXIS 2D V8.2 2010).

Table 3.1 The Salient Features of Aguat Wuha Embankment Dam

Dam		Spill way	
Types of earth fill dam	Zoned earth fill dam	a) Type	Ungated chute spill way
Type of dam fill material	Clay core,sand Silt with gravel shell	b) Crest length	8 m
Dam height	27.5 m	c) Design discharge	7.08 m ³ /s
Dam crest level	2876.5 m	d) Type of energy dissipate	Hydraulic stilling
Crest length	321.5 m	Freeboard	1.25 m
Foundation cut-off trench	5 m	Total free board	2.5 m
Dam crest width	5 m		
Minimum river bed level	2849 m	water head over the spillway crest	1.245 m
Maximum reservoir level	2875.25 m		
Normal reservoir level	2874 m		
Reservoir capacity	819,480 m ³		
Embankment slope			
Up stream Slopes of the Shell (casing) material (V: H)	(1:3)	Between burms, top level of dam and levels,burm level and dam foundation level, and b/n berms	U/s = 1:3 D/s = 1:2.5 For all condition
Down stream slope (V: H)	(1:2.5)	Slopes of the Core material	U/s=1:1 D/s=1:1
Berm width	3 m		
Core slope of up stream and down stream (V: H)	(1:1)		
Cut of trench slope (V: H)	(1:1)		
Up stream dam of the riprap thickness	0.3 m		

Source (ADSWE 2015).

Description of the Dam Site

The Aguat Wuha dam is constructed along the Aguat River. It is a zoned earth-fill dam with a central clay core that is constructed in 2018 G.C for the purpose of irrigation. The dam is found in the Abay River Basin having a storage capacity of 819,480 m³ water from a catchment area of about 959 ha that is at maximum reservoir level (ADSWE 2015b).

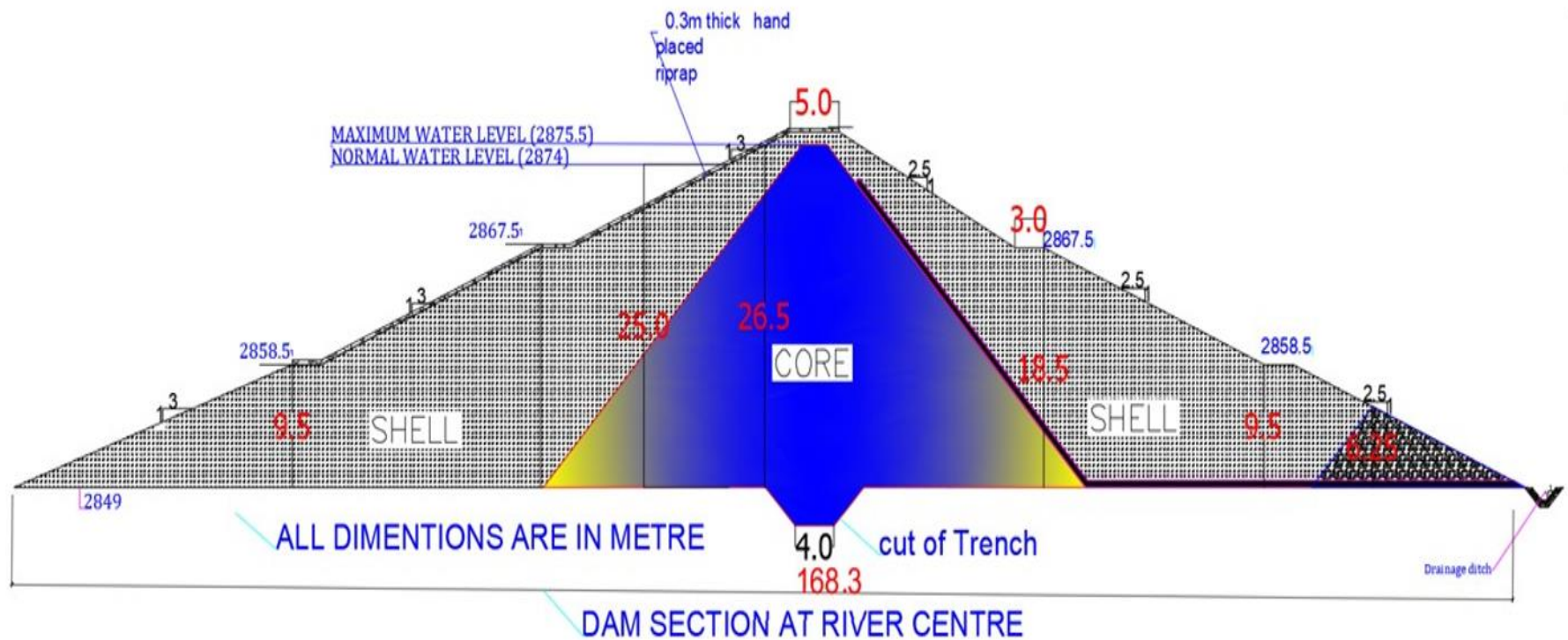


Figure 3.4 Aguat Wuha Dam Profile Section-Model (ADSWE 2015b)

3.2 Methodology Used

In this study, the examination of the dam was done using the computer programs Geo-Studio 2012 and Plaxis 2D V 8.2 models. Using the Geo-Studio 2012 model, Seep/W used to analyze seepage, Slope/W for slope stability, and Sigma/W for deformation study, including seismic threats, in relation to the current performance of the Aguat Wuha Embankment dam. determining the displacements and stress distribution within the dam, in addition. Similar to how the aforementioned parameters were modeled using Plaxis, the outcomes were compared and assessed.

3.2.1 Materials, Equipments, and Models Required

Materials

The performance assessment of the Aguat wuha earthen dam in terms of seepage, slope stability, and Stress-deformation was carried out in this study primarily using Geo Studio 2012 and Plaxis 2D models. Auto CAD 2018 was used to create the dam cross section and was imported into both programs to assign regions and materials for the dam.

Three of the products from Geo Studio-2012 are used. SLOPE/W, SEEP/W, and SIGMA/W are utilized for slope stability analysis, seepage analysis, and stress-deformation analysis respectively.

For Study area delineation and location map preparation Ethiopian Digital Elevation Model (DEM), ARC GIS 10.5, and arc hydro tools are used respectively.

In this research surveying instruments (Total Station), physical tools/equipment and materials such as Light vehicle, differential Global Positioning Systems (DGPSs), laptop computer, Digital camera, discharge measuring tools, Tape-meter, and different design report documents, and as built documents Such as-engineering design report, Geological and soil survey data , geotechnical report and watershed report, etc. will be used.

The software and models required in the proposed thesis are: ArcGIS 10.3, Auto CAD, Microsoft Office, Geo Studio 2012, PLAXIS 2D, and Arc Swat to delineate watersheds. In selecting these materials and models among alternative ones, different aspects were considered and these are discussed in detail in a subsequent section.

3.2.2 Material Parameters Used in a Different Analysis

Mohr-Coulomb Model

A well-known model used to approximate soil dynamics is the Mohr-Coulomb model. Deformation modulus Young's modulus (E), Poisson's ratio (ν), dilation angle (ϕ), cohesion (c), friction angle (ϕ), dilation angle (c), unit weight (γ), and deformation modulus (E) are the six factors that must generally be input into this model. Saturated and unsaturated unit weights, as well as the horizontal and vertical coefficient of permeability, were computed as crucial input factors.

This is the simple model to represent the soil behavior. It is an elastic perfectly plastic soil model. Following the recommendation of (Griffiths and Lane 1999) a dilation angle of 0° was assumed for slope stability analysis. The effect of increasing the soil angle of dilation (ψ) to be equal to the soil angle of internal friction (ϕ) results in only 3% increase in the safety factor and this also revealed by the results of (Nordal et. al. 2004). Young's modulus and Poisson's ratio are of insignificant importance in slope stability analysis by FEM using strength reduction method, due to the nature and mathematical formulation of the method. As a result, slope stability analysis by shear strength reduction method using Mohr Coulomb model required only three soil parameters (γ , ϕ , and c), same as the limit equilibrium method.

The soil was described using Mohr Coulomb's model. This model was chosen for the following reasons:-

- 1) It is well-established and widely used by engineers
- 2) As soon as the stability analysis is started (performed using the strength reduction technique), any model defined with the same strength parameters would behave like the Mohr-Coulomb model (i.e. Stress dependent stiffness is excluded). When the Phi-c reduction method is applied to advanced soil models, it follows Mohr-Coulomb failure criteria. Triangular elements with 15- nodes were used to construct meshes in 2D analysis assuming plane strain conditions. The Plane Strain model is useful when the loading is over a particular length in the z direction. It is assumed that the displacements and strains in the z-direction are zero, but the normal stresses are fully accounted for (PLAXIS 2010).

Young's Modulus (E , kPa)

The fundamental stiffness modulus used by PLAXIS in both the elastic and Mohr-Coulomb models is the Young's modulus. The Tri-axial unconsolidated undrained compression or Tri-

axial consolidated undrained compression tests are laboratory procedures used to calculate the soil modulus. The pressure meter test, the plate load test, the cone penetration test, and the standard penetration test (SPT) are the four most popular field tests.

$$E = \frac{\sigma}{\epsilon} = \frac{(F/A)}{(\Delta L/L)} \dots\dots\dots 3.1$$

Where, E is Young’s Modulus, σ is the stress and ϵ is the strain.

Poisson’s Ratio (ν)

The ratio of lateral or transverse strain to the longitudinal strain is defined as Poisson’s ratio

$$\nu = (\text{Lateral strain}) / (\text{Axial strain}) \dots\dots\dots 3.2$$

The experimental values of Poisson’s ratio for several clays, based on experimental values are expressed by (PLAXIS 2D V8.2 2010).

$$\nu = 0.25 + 0.00225 (PI) \dots\dots\dots 3.3$$

Where, PI is plasticity index. For granular soils, a general range of Poisson’s ratio is expressed by approximated for drained soils as follows:-

$$\nu = 0.1 + 0.3 ((\phi - 25) / (45 - 25)) \dots\dots\dots 3.4$$

Where, ϕ is friction angle in Tri-axial compression test.

Cohesion (C, kPa)

The cohesion of a soil's molecules or other similar particles is what binds them together. In the laboratory, the direct shear and tri-axial tests are typically used to calculate cohesion, c.

Friction Angle (ϕ , °)

The angle at which shear failure takes place on a given soil's shear stress and normal effective stress plot is known as the angle of internal friction. The direct shear or triaxial tests are used in the laboratory to evaluate the angle of internal friction,. The formula for rating rock masses, which is presented as follows, can also be used to determine the angle of friction of a rock mass (PLAXIS 2D V8.2. 2010).

$$\phi = 0.5 \times RMR + 5 \dots\dots\dots 3.5$$

Where, RMR is rock mass rating

Dilatancy (Ψ , °)

When granular materials are subjected to shear deformations, dilatancy is the volume change that is noticed (PLAXIS 2D V8.2. 2010). The value of the dilatancy angle is calculated as follows for non-cohesive soils (sand and gravel) with the angle of internal friction $> 30^\circ$:

$$\Psi = \phi - 30 \dots\dots\dots 3.6$$

The answers to the above empirical questions or the plot of volumetric strain against axial strain are used to determine the dilation angle.

Permeability (K, m/s)

Because there may be a large variation between horizontal and vertical permeability in some types of soil, such as peat, PLAXIS distinguishes between a horizontal permeability (K_x) and a vertical permeability (K_y).

Darcy's empirical Law establishes the coefficient of permeability, often known as Darcy's Coefficient:-

$$k = Q/Ai \dots\dots\dots 3.7$$

Where, Q is the volume of water flow through the soil per unit time, A is the cross-sectional area of the soil, k coefficient of permeability (m/s) and i is the hydraulic gradient.

The constant-head test, the falling head test, and the indirect determination from consolidation test are laboratory techniques for figuring out the coefficient of permeability of soils. (PLAXIS 2D V8.2 2010).

For more porous granular materials, the constant head test is appropriate, and for fine-grained soils, the falling head permeability test is utilized. The consolidation test is frequently used to evaluate the coefficient of permeability of clay soils in the following ways:-

$$K = \frac{T_v \times \gamma_w \times H^2}{t \Delta\sigma (1+e)} \dots\dots\dots 3.8$$

For more permeable granular materials, the constant head test is appropriate, and the falling head permeability test is employed for fine-grained soils. Frequently, the consolidation test yields the following results in determining the coefficient of permeability of clay soils (PLAXIS 2D V8.2, 2010).

$$K = C \times D_{10}^2 \dots\dots\dots 3.9$$

Where, C factor, usually taken as 0.01 and D_{10} effective grain size in mm and k is permeability in m/sec.

Unit Weight (γ , kN/m³)

By weighing and measuring a field-collected soil sample's weight and volume, unit weight is calculated in the lab. Sand cone testing may be used to determine the unit weight of soil directly in the field.

Table 3.2 Geotechnical Summarized Data Sets Used (Mott MacDonald 2006).

Material Type	Y_{un-sat} ($\frac{KN}{M^3}$)	Y_{sat} ($\frac{KN}{m^3}$)	K_x ($\frac{m}{d}$)	K_y ($\frac{m}{d}$)	E (Mpa)	ν	C ($\frac{KN}{m^2}$)	ϕ (°)	ψ (°)
Clay	18.4	19.2	3.456×10^{-3}	3.456×10^{-3}	5.7	0.3	5	30	0
Fine filter (F ₁)	17.3	20.4	17.28	17.28	10	0.22	0	33	3
Coarse filter (F ₂)	17.8	20.9	12960	12960	12.25	0.25	0	35	5
Com-pacted fill	18	19.8	118273	543456	50	0.175	0	30	0
Rip Rap	19	22	543456	118273	5000	0.4	0	45	15
Rip Rap bedding	19	22	86400	86400	5000	0.4	0	45	15
Basalt foundation	19	22	0.438912	0.438912	73.2	0.325	25	40	10

3.2.3 Model Simulation and Analysis

In this study, the Aguat wuha micro earthen dam's performance was assessed in terms of seepage, slope stability, and earthquake analysis using Geo studio and FE-based PLAXIS 2D software.

A. Using Geo-Studio Soft Ware

The Geotechnical dam design report of the Aguat whua earthen dam served as the source for the geotechnical material parameters used in the Geo Studio study. The foundation was modeled with a linear elastic model utilizing Young's modulus and Poisson's ratio values in Geo Studio 2012. A non linear elastic model was also used to simulate the dam's body. Through transverse sections of the dam, the Geo-Slope International Ltd. 2012 computer software was utilized to evaluate slope stability, seepage, and settlement conditions. Additionally, it was used to calculate displacement and stress distribution throughout the dam under static conditions using SIGMA/W FEM modeling. Slices Method analysis was used to analyze the dam's slope stability in both static and dynamic circumstances.

Seepage Analysis Using SEEP/W Model

2D SEEP/W finite element program is the computer program. Many pieces of computer software are now widely used, and by providing them with the right inputs and information, they may be used to perform any difficult calculations and simulations. SEEP/W is a finite element that has the ability to resolve foundation and/or embankment stable steady vertical section modes. You can simulate seepage and phreatic surfaces using the numerical model SEEP/W. The SEEP/W program has the ability to model ground water flow, seepage, and issues with excessive pore water pressure in porous medium like soil and rock rather successfully. It can also simulate seepage rates and phreatic surfaces in homogeneous and nonhomogeneous dams (Krah 2012). A range of seepage flow conditions in embankment dams and ground water movement can be modeled by a number of software programs. Software packages like SEEP/W, PLAXIS, ANSYS, SVFLUX, etc. that deal with seepage issues are well-known (Tanchev 2014). Software called SEEP/W was chosen above competing products because it operates by discretizing a continuous flow medium into smaller, more manageable components and then joining or meshing those discrete sections with continuity equations. Its comprehensive approach enables consideration of studies ranging from complex, saturated/unsaturated, and time-dependent problems to simple, saturated steady-state problems. It provides the capacity to simulate axisymmetric radial flow issues and has the option of computing seepage forces (Krah 2012). The primary goals of embankment dam design are to protect against internal erosion, leaking, and the buildup of excessive pore pressures (Narita 2000). The Seep/w and Plaxis software applications, which are parts of the Geo Studio and Plax packages, respectively, were employed in this research because of their

strength. SEEP/W is numerical modeling software that will be used to solve practical seepage problems. This is a part of the most popular geotechnical software called Geo Studio. The seepage theory and finite element approach were combined to create the SEEP/W program, which focuses on saturated and unsaturated soil regions. The actual physical process of water moving through a particle medium can be numerically simulated. It is based on computer software with finite elements that can handle issues with seepage, excess pore water pressure, and ground water movement in porous medium like soil and rock. Material properties for each zone with proper geometry are entered into the software in the appropriate manner, and each region has been verified in line with this. Because natural soils are heterogeneous and boundary conditions fluctuate, it is never simple to model practical seepage issues using numerical methods. The water surface boundary as well as the entrance and exit conditions are typically the boundary conditions for seep/w.

Steady State Seepage

Laminar flow is assumed and satisfies the Laplace equation in SEEP/W, which is regulated by Darcy's law. The water intake and outflow are assumed to be equal in the SEEP/W steady-state study. Transient seepage, where the inflow and outflow are not equal, is employed in the investigation of fast drawdown stability. The fundamental tenet on which nearly all seepage analyses are based is Darcy's equation. It is the simplest one for the seepage rate by:

$$q = k \times i \times A \dots\dots\dots 3.10$$

Where q is the flow rate, k is the hydraulic conductivity, i is the hydraulic gradient and A is the cross-sectional area, can be used when a gross estimate of flow is needed, or when a back calculation of average hydraulic conductivity is needed. The general governing differential equation for two-dimensional seepage under steady-state conditions can be expressed as:

$$(\partial^2 H)/(\partial x^2) + (\partial^2 H)/(\partial y^2) = 0 \dots\dots\dots 3.11$$

Where: H = the total head

The Laplace equation is satisfied by SEEP/W and is governed by Darcy's law. Hydraulic conductivity, saturated water content, and reservoir water content serve as the research's boundary conditions and inputs for the SEEP/W model. The amount of seepage and pore water pressure fluctuate depending on hydraulic conductivity and reservoir water level changes. All materials in the dam body of this typical section have their corresponding hydraulic, volumetric, and other soil property parameters allocated to them in order to simulate the materials. Fine elements and nodes connect together to form the cross-section. A node's

seepage and pore water are calculated using the Seep/W software. For two-dimensional seepage in steady-state conditions, the basic governing differential equation can be expressed using the Laplace notation as (Geo Slop 2012).

$$\frac{\partial}{\partial x} \left(K_x \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left(K_y \frac{\partial h}{\partial y} \right) + Q = 0 \dots\dots\dots 3.12$$

The changes in volumetric water content are dependent on changes in the stress state and the properties of the soil. For steady-state seepage, only the coefficient of permeability is required because the time-dependent term disappears and the water storage term drops out.

SEEP/W is formulated for conditions of constant total stress; that is, there is no loading or unloading of the soil mass. SEEP/W also assumes that the pore-air pressure remains constant at atmospheric pressure during transient processes. Since the elevation is a constant, the derivative of y with respect to time disappears, leaving the following governing differential equation used in SEEP/W finite element formulation:-

$$\frac{\partial}{\partial x} \left(K_x \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left(K_y \frac{\partial h}{\partial y} \right) + Q = m_w \gamma_w \left(\frac{\partial h}{\partial t} \right) \dots\dots\dots 3.13$$

Where “Q” = the applied boundary flux,

“ γ_w ” = the unit weight of water

K_x , and K_y , = coefficient of permeability of soil in x, and y direction,

“t” = time

“h”= total fluid head and “ m_w ”= water storage (the slope of the soil-water x-cs curve).

Seepage study was carried out in a specific steady-state flow situation during dam operation. Various reservoir water levels were taken into consideration. Eight different types of materials make up the dam body and its foundation, and the saturated/unsaturated state was taken into account when modeling the components. In order to input the hydraulic parameters, the functions of volumetric water content variation and permeability variation should be used.

Numerical Model Method

- Discretize the continuous flow medium into manageable pieces for the seepage analysis. Then, put the discrete pieces back together using continuity equations, and utilize the Darcy equation to describe the internal workings of the flow.

- Computerized numerical approaches are advised due to the following reasons: Easy modeling of complex systems
- No dimension transformation
- Faster parameter variation
- Results printed in digital form for each node

1. Volumetric Water Content

As water flows through soil, certain amounts of water are stored or retained within the soil structure, being these functions of the pore-water pressure and the characteristics of the soil structure. Since it can sometimes be difficult or time consuming to obtain a volumetric water content function, it may be of benefit to be able to develop an estimation of the volumetric water content function using either a closed-form solution that requires user-specified curve fitting parameters, or to use a predictive method that uses a measured grain-size distribution curve. SEEP/W has four methods available to develop a volumetric water content function. One is to estimate a data point function using a predictive method based on grain size, one is to base your function of a sample set of functions built into the software, and two are closed form equations based on known curve fit parameters (Krah 2012).

Volumetric water content was used as the "Data point function" for this study. This function is estimated using "Grain-size data," which in this case depends on various dam bodies and the materials used in their construction. The initial condition of water volume value in "Saturated WC" must then be entered. This value was established based on the porosity of the material. The soil is loaded with water when it is saturated according to the coefficient of compressibility. For clay soil, it ranges from 10^{-4} 1/k to 10^{-3} 1/k pa, while for another type of soil, it goes up to 10^{-6} 1/k pa (Dr. Arora 2004). The residual soil moisture content, which the soil retains under high pressure, ranges from 5 to 12% of the volumetric moisture content, which is a very modest proportion (Jose-Navar 2016). Since it is based on volumetric water content or porosity, it differs for all material models.

2. Hydraulic Conductivity

The amount of water held in unsaturated soil changes according to the matric suction, which is caused by the difference between air and water pressure. To take this into consideration, the sample function in the software is employed by entering the materials' saturated water content. Nearly comparable to the soil's porosity is the saturated water content. There are mathematical models that link the characteristic curves of hydraulic conductivity and soil

water retention. The van Genuchten makes use of the Mualem theory and offers a more rounded illustration of the inflection point in the characteristic curve close to saturation. The definition of hydraulic conductivity of materials also considers the matric suction. the boundary condition of the static water pressure reaching the usual water level (Krah 2012).

The Van Geunchten method is used to estimate the "Data point function" of permeability from the moisture function. As the Mualem theory is used in the Van Geunchten approach. It allows for a smoother portrayal of the characteristic curve's inflection point close to saturation.

3. Boundary Conditions

For modelling the seepage analysis using the finite element method, SEEP/W boundary conditions are essential. In seepage analysis with SEEP/W, there are two fundamental boundary conditions namely, the head-dependent boundary condition and the flux-dependent boundary condition. Accordingly, in the seepage simulation of Aguat Wuha dam, two different scenarios of head boundary conditions were been used. Computations are carried out for the two important reservoir water levels which are the current reservoir level (i.e. Elevation 2868m a.m.s.l), and Normal pool level scenarios (i.e. Elevation 2874m a.m.s.l). Both in the current reservoir water level and normal reservoir water level practical seepage measurements were carried out. The water level in the reservoir and downstream face of the dam was taken as a head boundary condition and potential seepage face respectively and finally the earthen dam was simulated for the steady state condition.

In case of sudden drawdown, the elevation differences in the reservoir water level are due to rapid drawdown. The central clay core was considered as a flux boundary condition and the downstream face as a potential seepage face and finally, the earthen dam was simulated for the transient state condition. After the result of the model, the location of the phreatic line, seepage flux (discharge), and pore water pressure distribution for the above head-boundary conditions were determined.

4. Determination of Phreatic Lines, Quantity of Seepages, and Pore Water Pressures

Phreatic line or seepage line is the line at the upper surface of the seepage flow at which the pressure is atmospheric. For Aguat Wuha dam determination of seepage through the dam body, the determination of phreatic line and pore water distribution was done using the finite elements SEEP/W model. SEEP/W model for steady state and transient seepage analysis was

used as a parent for the SLOPE/W and SIGMA/W to analyze the slope stability and stress deformation.

SLOPE/W

SLOPE/W is the most common and popular software application which will be used for the stability analysis of a slope under different load conditions. This is part of the Geo Studio software application. The very important advantage of the SLOPE/W analysis is it allows handling all possible slices in the same model with the corresponding factor of safety.

SIGMA/W

SIGMA/W is capable of doing both a straight forward study of linear elastic deformation and a more complex analysis of nonlinear elastic-plastic effective stress. You can represent a wide variety of soils or structural materials using a number of constitutive soil models. SIGMA/W can also simulate the formation and dissipation of pore-water pressure in a soil structure in response to external loads. Stress and deformation assessments of earth constructions will be carried out using the finite element software program SIGMA/W. Its thorough structure enables the analysis of both straight forward and extremely complicated issues.

QUAKE/W

For the dynamic study of earth structures subjected to earthquake shaking and other abrupt impact loadings, the geotechnical finite element software program Quake/w is employed. Complete integration exists between Quake/W and additional parts like Slope/W, Seep/W, and SIGMA/W. It is possible to undertake several analyses using Quake/W; however, the main constraint is that the model requires a time history of earth quake for a given duration which is lacking in our country. The model also requires other dynamic parameters to successfully understand the behavior of the embankment during and after the earth quake.

Creating Dynamic Model

The dynamic model will be created with the following

- i. Add Quake/W
- ii. Name: Dynamic Quake/W (downstream slope)
- iii. Parent: Initial Static
- iv. Analysis Type: Equivalent Linear Dynamic
- v. Initial Stress Condition from: Parent Analysis

vi. Initial PWP Conditions from: Parent analysis

vii. Insert the time series of earth quake data:

B. Finite Element Analyses PLAXIS 2D Soft Ware

PLAXIS is a finite element program for geotechnical applications in which soil models are used to simulate the soil behavior, developed by PLAXIS in cooperation with several universities including DUT in the Netherlands and NTNU in Norway. The computer program is applicable to many Geotechnical problems, including stability analyses and steady-state ground water flow calculations. This software contains several FE models and four main sub-routines. These sub-routines are inputs, calculations, outputs and curve plots. The FOS versus displacement is plotted from the curve plots sub-routine (PLAXIS 2D V8.2 2010). The numerical modeling is carried out by means of the Finite Element method as it allows for modeling complicated non linear soil behavior and various interface conditions, with different geometries and soil properties.

PLAXIS 2D program was used, which has a series of advantages (Plaxis 2010):-

1. Ability to deal with excess pore-pressure phenomena
2. Excess pore pressures are compute during plastic calculations in undrained soil
3. More knowledge of how soil interacts with structures
4. It can replicate complex constitutive soil models for nonlinear behavior simulation.
5. It contains more sophisticated material models with certain properties such as Mohr Coulomb model, hardening-soil model, jointed rock model, soft soil creep model and soft soil model.

The initiation of the PLAXIS Finite Element Program was held at Delft University of Technology Netherland by Pieter Vermeer in 1974. PLAXIS name was derived from Plasticity Axisymmetric. It is the finite element software for soil and rock that has been used by geotechnical engineers and researchers. The software was first developed by The Technical University of Delft in 1987 to analyze soft soils of the low lands of Holland (PLAXIS 2010).

It was then extended to cover all aspects and applications of geotechnical engineering simulation using a user friendly interface with the power of finite element. The first version of PLAXIS was commercially available in 1998. PLAXIS is described as a finite element package for geotechnical analysis that can utilize both two dimensional and three-

dimensional analyses in determining the stability and deformation experienced by slopes. It lends it self to modeling more complex geotechnical scenarios as it has the capabilities to simulate in homogeneous soil properties and time dependent scenarios (Brinkgreve and Vermeer 2002).

The program has a simple graphical input procedures enabling quick generation even of complex finite element models, in addition to great facilities that provide a detailed presentation of the out put computational results. The calculation program is fully automated and based on robust numerical procedures (PLAXIS 2010).

The FE method preserves global equilibrium until failure is reached, and is able to monitor progressive failure up to and including overall shear failure (Vinod et.al 2017).

Different soil models are incorporated in PLAXIS with a versatile library of structural elements. The automated mesh generation tool in the program makes the creation of soil models easy and practical since 6-node as well as 15-node triangular elements are available. The program uses the $c - \phi$ method to calculate the factors of safety of slopes.

One of the 2D finite element geotechnical analysis tools is PLAXIS 2D. It can do slope stability analysis using the strength reduction approach in addition to its plastic deformation analysis and other analysis choices. It mostly consists of two components. These programs have input and output. Five sub-sections make up the input program: soils, structures, mesh, flow conditions, and staged construction. Units, workspace dimensions, model type (axisymmetric or planar strain), and finite element type (6 or 15 nodes) can all be specified in the project properties section.

Materials can be created and assigned to the soil regions by using the material set options. In there, soil parameters, type of drainage, constitutive material model, seepage, and ground water conditions can be defined. In the mesh section, geometry is divided into finite elements (forming a mesh) with a desired level of fineness. The soft ware has five options for mesh dimensions. These are very fine, fine, medium, coarse, and very coarse. After that, the ground water level (e.g. phreatic level) can be added to the flow conditions section.

The material set choices can be used to construct materials and allocate them to the different soil zones. It allows for the definition of soil parameters, drainage methods, constitutive material models, seepage, and ground water conditions. Geometry is separated into finite components (forming a mesh) in the mesh section, with the required amount of fineness. The software offers five different mesh size options. They range from extremely fine to fine,

medium to coarse, and extremely coarse. After that, the section on flow conditions can include the ground water level (for example, phreatic level).

After the input geometry is complete, the necessary level of calculation steps can be built in accordance with the analysis's goal. There should be at least two stages in a slope stability analysis (Akbas 2015).

In the output, several analysis results, including deformations and stresses, can be viewed for individual stages. After going to the strength reduction phase results, one can obtain the failure surface zone from the “Incremental Cartesian Strain” option viewing the incremental shear strains, and reached SRF value can be seen from the “Calculation Information” (Akbas 2015).

Elements

Both 6-or 15-node triangular elements, which are the only types of elements available in PLAXIS 2D, can be selected to model soil or other materials; and by using these elements, displacements and stresses can be simulated.

The 15-node triangular element provides a fourth order interpolation for displacement and numerical integration involving twelve stress points (gauss points). It gives a high quality stress results for difficult problems, but leads to longer calculation times. Numerical integrations involving three stress points and second order interpolation for displacement are provided by the 6-node triangular element. Compared to the 15-node element, the 6-node produces less precise estimates of deformation but can provide acceptable results for standard deformation analysis.

Interfaces

Interfaces are usually placed at the union between the soil and structure in PLAXIS 2D and incorporate a virtual thickness used to define material properties within the region. This virtual thickness allows for elastic deformation to be generated, and more elastic deformation can occur when the virtual thickness is larger.

The roughness of the interaction between the soil and geo grid can be simulated by the use of a strength reduction factor (R_{inter}). This factor allows modeling of the soil strength parameters (cohesion, friction angle and dilatancy angle) to be related to the interface strength such as the wall friction and adhesion.

A rigid interface between the soil and structure is integrated when a value of 1.0 for R_{inter} is chosen; and as a result, PLAXIS does not reduce the strength of the surrounding soil.

Consequently, the cohesion, friction angle and dilatancy angle for the material interacting within the interface is not changed; however, ν is altered to a value of 0.45 (PLAXIS 2011). When using interfaces in a ground water calculation, the interfaces are, by default, fully impermeable. In this way interfaces may be used to block the flow perpendicular to the interface, for example to simulate the presence of an impermeable screen. On the other hand, if the interfaces are present in the mesh it is the user's intention to explicitly avoid any influence of the interface on the flow and the pore pressure distribution. In such a case the interfaces should be de-activated in the water conditions mode. This can be done separately for a consolidation analysis and a ground water flow calculation. For inactive interfaces the pore pressure degrees-of-free dom of the interface node pairs are fully coupled whereas for active interfaces the pore pressure degrees-of-free dom are fully separated (PLAXIS 2011).

Plaxis Calculation Types

Initial Conditions

It is necessary to specify the initial stress state and the initial configuration after the geometry model has been built and the finite element mesh has been formed. The initial conditions section of the input program is where this is done. Two separate modes make up the beginning conditions: one for creating the initial water pressures (the water conditions mode) and another for specifying the initial geometry configuration and creating the initial effective stress field (the geometry configuration mode). A plastic calculation, consolidation analysis, dynamic calculation, and phi-c reduction are the four computations that the finite element PLAXIS 2D software application offers for each building phase (PLAXIS 2D V8.2 2010).

Plastic Calculation

In order to perform an elastic-plastic deformation analysis without taking the deterioration of surplus pore pressures with time into consideration, a plastic calculation was chosen. A plastic calculation, with the exception of when the soft soil creep model is utilized, does not account for time effects even though a time interval can be given.

Due to the quick loading of water-saturated clay-type soils, the un-drained option in the material data sets may be utilized to perform a plastic calculation for the limiting situation of entirely un-drained behavior. However, conducting a roughly drained study can evaluate the settlements over the long run.

Consolidation Analysis

When it's important to examine the development or dissipation of excess pore pressures in clay-type soils that have been saturated with water over time, consolidation analysis should be used. Analyses of real elastic-plastic consolidation are possible with PLAXIS. After an undrained plastic calculation, a consolidation analysis without further loading is often carried out. A consolidation analysis also allows for the application of loads.

Dynamic Analysis

For significant dams, where collapse could result in substantial levels of risk, dynamic analysis is advised. The finite element approach is mostly used in the dynamic analysis to estimate how an earth dam will deform (Carlo 2015).

The earthquake is modeled by imposing a prescribed displacement at the bottom boundary. A dynamic analysis uses a different time parameter than other types of calculations. The time parameter in dynamic analysis is the dynamic time parameter, which is always expressed in seconds (s). Ground motion attenuation equations are used to determine the level of acceleration as a function of distance from the source and magnitude of the earthquake. The following widely used attenuation equation was developed by Esteva and Rosenblueth (Carlo 2015). Based on international standards, the vertical PGA has been taken as ½ of the horizontal component (Carlo 2015).

$$A = \frac{2000 e^{0.8M}}{R^2} \dots\dots\dots 3.14$$

Where, A is expected peak ground acceleration in % of g, M is earthquake magnitude and R is focal distance. For a given site located at an epicenter distance X km from an earthquake source at focal depth Z km, the focal distance R is given by:-

$$R = \sqrt{X^2 + Z^2} \dots\dots\dots 3.15$$

Phi-c Reduction (Safety Analysis)

A safety analysis in PLAXIS can be executed by reducing the strength parameters of the soil. A safety analysis can be performed after each individual calculation phase and thus for each construction stage. However, note that a phi-c reduction phase cannot be used as a starting condition for another calculation phase because it ends in a state of failure.

When performing a safety analysis, no loads can be increased simultaneously. Phi-c reduction is a special type of plastic calculation. In the Phi-c reduction approach, the strength

parameters ϕ and c of the soil are successively reduced until failure of the structure occurs. The factor of safety is given by:

$$FOS = \frac{c + \sigma_n \tan \phi}{c_r + \sigma_n \tan \phi_r} \dots\dots\dots 3.16$$

Where; c and ϕ are the input strength parameter and σ_n is the actual normal stress component. The parameter c_r and ϕ_r are reduced strength parameters that are just large enough to maintain equilibrium.

The $c - \phi$ method is based on the reduction of the shear strength parameters; cohesion (c) and the tangent of the friction angle ($\tan\phi$) of the soil. These parameters are reduced in steps until the soil mass fails. PLAXIS uses a factor to relate the reduction in the parameters during the calculation at any stage with the input parameters according to the following Equation (Hammah et.al 2004).

$$RF = \tan\phi \text{ input} / \tan\phi \text{ reduced} = C \text{ input} / C \text{ reduced} = \Sigma Msf \dots\dots\dots 3.17$$

Where RF = the reduction factor at any stage during calculations, $\tan\phi$ input and C input are the input parameters of the soil, $\tan\phi$ reduced and C reduced are the reduced parameters calculated by the program. In PLAXIS factor of safety (F.S) is indicated in terms of sum of incremental multiplier (ΣMsf). ΣMsf is set to 1 at the start of calculation after that the analysis for safety calculation starts through load advancement number of steps procedure. The incremental multiplier (Msf) with the default number of 0.1 at the beginning is applied in order to determine the amount of strength reduction and the reduction in strength parameters continues until all the additional steps are calculated and analyzed. Finally at the failure stage of the slope, the total safety factor is given as follows:-

$$F.S = \text{Available strength} / \text{Strength at failure} = RF \text{ at failure} \dots\dots\dots 3.18$$

In this method it can be seen that the factor of safety is independent of the stress level and therefore, the modulus of elasticity and the Poisson's ratio will have negligible effect on the obtained factors of safety.

FEM RESULTS

The calculation results allowed the identification of:-

- Deformation at the end of the construction phase
- Deformation due to first filling
- Safety factors in 4 different loading conditions

As described, during the calculation different stability analyses were performed using the "phi – c reduction" method. This analysis computes a safety factor using the approach of

reducing the strength parameters “ $\tan\phi$ ” and c' of the soil until failure of the structure occurs. The total multiplier ΣM_{sf} is used to define the value of the soil strength parameters at a given stage in the analysis:

$$\sum M_{sf} = \frac{\tan \phi_{input}}{\tan \phi_{reduced}} = \frac{C'_{input}}{C'_{reduced}} \dots\dots\dots 3.19$$

Where the strength parameters with the subscript “input” refer to the properties entered in the material sets and the parameter with the subscript “reduced” refer to the reduced values used in the analysis.

The incremental multiplier ΣM_{sf} is used to specify the increment of the strength reduction of the first calculation step and the final ΣM_{sf} value is the stability factor at failure. This method resembles the calculation of stability factor as conventionally adopted in slip circle analysis because it is the ratio between the available strength and the strength at failure

3.3 Data Source

3.3.1 Primary Data Collection

The collection of primary data from the field was based on

- Measuring flow data downstream of the dam site
- Taking elevation at the crest of the dam along the crest length at different chainage using differential Global Positioning Systems.
- Taking elevation upstream of the dam at different water level
- Taking elevation at the crest of the spill way

Observation and practical field visits had been carried out with the purpose of:-

- Measuring the quantity of seepage at the downstream toe (using a masonry Rectangular canal with a dimension of width 0.4 m and depth 0.4 m by applying manning equation.

Aguat Wuha Dam Seepage Measured at Full Reservoir Level (2874 m a.m.s.l)

The collected seepage data are representative monitored seepage data taken for repeated seepage data at a single reservoir level on different dates.

The discharge through the dam body and around the periphery of the outlet is combined downstream of the dam via Rectangular lined masonry canal sections and the discharge is measured by using Manning’s equation at different elevation levels as shown below.

$$Q = \frac{1}{n} AR^{2/3} S^{1/2} \dots\dots\dots 3.20$$

Where Q = discharge measured at rectangular section

Q = Design discharge, m³/s A = Wetted cross-sectional area of canal, m²

P = Wetted Perimeter, m R = A/P = Hydraulic radius of the canal, m

n = Manning's roughness coefficient = 0.018 S = Bed slope of the canal

The elevations of the bed of the discharge measuring rectangular channel starting point and end point are:-

Elevation @ start point = 2845.75 m

Elevation @ end point = 2845.73 m

Length of bed = 15 m

$$\text{Slope}(S) = \frac{2845.75\text{m} - 2845.73\text{m}}{15\text{m}} = \frac{0.02\text{m}}{15\text{m}} = 0.00133$$

B = 0.4 m (measured value)

D = 0.07 m (measured value)

A = 0.4 m × 0.07m = 0.028 m²

P = 2D+B = 2 × 0.07 m + 0.4m = 0.54 m

$R = \frac{A}{P} = \frac{0.028}{0.54} = 0.0518\text{m}$ and $R^{2/3} = 0.139$

$S^{1/2} = (0.00133)^{(1/2)} = 0.0365$

S = 0.0365

$$\frac{1}{n} = \frac{1}{0.018} = 55.55$$

$$Q = \frac{1}{n} AR^{2/3} S^{1/2} = 55.55 \times 0.028 \times 0.139 \times 0.0365 = 0.00789 \text{ m}^3/\text{s}$$

Aguat Wuha Dam Seepage Measured at Reservoir Level (2868 m a.m.s.l)

The discharge is measured at the same place but at a different time when the reservoir water level is at 2874 and the average water depth is 3 cm.

Elevation @ start point = 2845.75 m

Elevation @ end point = 2845.73 m

Length measured = 15 m

$$Slope(S) = \frac{2845.75m - 2845.73m}{15m} = \frac{0.02m}{15m} = 0.00133$$

B = 0.4 m (measured value)

D = 0.06 m (measured value)

$$A = 0.4 m \times 0.06 m = 0.024 m^2$$

$$P = 2D + B = 2 \times 0.06 m + 0.4 m = 0.52 m$$

$$R = \frac{A}{P} = \frac{0.024}{0.52} = 0.046 m \text{ \& } R^{2/3} = 0.128$$

$$S^{1/2} = (0.00133)^{(1/2)} = 0.0365$$

$$\frac{1}{n} = \frac{1}{0.018} = 55.55$$

$$Q = \frac{1}{n} A R^{2/3} S^{1/2} = 55.55 \times 0.024 \times 0.128 \times 0.0365 = 0.00623 m^3/s$$

Table 3.3 Average Measured Seepage on the Dam Body

Reservoir water level (a.m.s.l)	The measured seepage(Rectangular lined canal manning)
2874 m	7.89*10 ⁻³ m ³ /s
2868 m	6.23*10 ⁻³ m ³ /s

Table 3.4 Vertical Settlement on the Crest of the Dam due to Vertical Deformation (Own Measured Data)

Chainage (m)	0+00	0+20	0+40	0+60	0+80	1+00	1+20
Surficial settlement (m)	0.28	0.32	0.31	0.28	0.26	0.12	0.29
Chainage (m)	1+40	1+60	1+80	2+00	2+20	2+40	2+60
Surficial settlement (m)	0.17	0.42	0.71	0.46	0.25	0.14	0.38
Chainage (m)	2+80	3+00	3+20				
Surficial settlement (m)	0.41	0.19	0.13				

3.3.2 Secondary Data

Essential input data for the Geo-Studio 2012 and Plaxis 2D are:-

- The dam's geometrical design.

- Geotechnical parameters of the foundation materials and embankment from the geotechnical design report
- The Amhara Design and Supervision Works Enterprise provided geological reports, maps, and data regarding the dam site, reservoir interfaces, and reservoir water levels. (ADSWE 2015).

These secondary collected data of geotechnical parameters of the embankment and foundation materials which were mainly used for the model simulation were collected and summarized in tables (3.5) and (3.6) below.

Table 3.5 Geotechnical Material Properties for SEEP/W (ADSWE 2015)

PARAMETERS	UNIT	EMBANKEMENT MATERIAL			
		SHELL	CORE	FILTER	ROCK TOE
Saturated water content	m ³ /m ³	0.29	0.5	0.29	0.2
Soil type		Gravel	clay	Sand	gravel
K at saturation	m/s	9.20*10 ⁻⁰⁷	3.30*10 ⁻⁰⁹	1.30*10 ⁻⁰⁴	2.50*10 ⁻⁰⁷
Residual water content	%	0	0.25	0	
conductivity ratio		1	0.1	1	1

Table 3.6 Input Material Properties for SLOPE/W & SIGMA/W (ADSWE 2015)

Material Type	Unit weight (k N/m ³)	Cohesion(C) (k Pa)	φ (°)	E(k Pa)	ν
Shell	20.35	14.78	26.5	38000	0.33
Clay core	18.45	16	34.9	320000	0.38
Transition (D/S Filter)	19	0	34	80000	0.31
ROCK TOE	20.00	40.00	0	33000	0.34
Foundation Rock	21	19	40	310000	0.32
Foundation rock	Bedrock (Impenetrable)	20	40	1500000	0.29

3.3.3 Analysis Limitations

All observed behaviors of an embankment cannot be accurately analyzed because of the complicated nature of soil activity. In these situations, it is typically seen wise and appropriate to have confidence in the engineering judgment, which is based on the professional experience of those accountable engineers.

3.4 Method Slope Stability Analysis

Many different solution techniques for slope stability analyses (computer program, charts, and hand calculations) have been developed over the years. These methods of analysis should be selected according to the complexity of the site or job and the data available to define the site conditions. Where conditions are complex, significant amounts of data are available slope stability analyses using computer program is recommended. Due to the lengthy computational procedures, computer programs provide a means for efficient and rapid detailed analysis of a wide variety of slope geometry and load conditions (Chowdhury 1978). A large number of commercially available slope stability programs are in use by engineers all around the world. These computer programs uses limit equilibrium as well as finite element techniques to evaluate the side resistance of the sliding mass and the searching technique for the most critical slip surface.

The elements of safety were determined utilizing the slope stability inquiry work performed in addition to Plaxis using the Slope/W computer program based on limit equilibrium method analysis performed by the Morgenstern-Price technique. Contrary to Fellenius, Bishop, or Janbu's methods, the Morgenstern-Price method has been adopted because, unlike those methods, it satisfies all three conditions of equilibrium: force equilibrium in both the horizontal and vertical directions, as well as the moment equilibrium condition (Cheng 2007). Additionally, Spencer's approach works on almost all slopes. The simplest full equilibrium method (Duncan and Wright 2005) for calculating the safety factor.

Additionally, Spencer's method offers factors of safety values that are extremely similar to those found by the Morgenstern-Price method and satisfies both moments and force equilibriums. Compared to the Morgenstern-Price technique, it is frequently more conservative.

1. Morgenstern-Price method (LEM)

This method was developed by Morgenstern and Price (1965), which consider not only the normal and tangential equilibrium but also the moment equilibrium for each slice in circular

and non-circular slip surfaces. In this method, a simplifying assumption is made regarding the relationship between the inter-slice shear forces (X) and the inter-slice normal forces (E) as: shown in equation (3.21).

$$X = \lambda \cdot f(x) \cdot E \quad \dots\dots\dots 3.21$$

Where: $f(x)$ is an assumed function that varies continuously across the slip, and (λ) is an unknown scaling factor that is solved for as part of the unknowns.

The unknowns that are solved for in the Morgenstern and Price method are the factor of safety (F), the scaling factor (λ), the normal forces on the base of the slice (P), the horizontal inter slice force (E), and the location of the inter slice forces (line of thrust). Once the above unknowns are calculated using the equilibrium equations, the vertical component of the inter- slice forces (X) is calculated from equation 3.22.

$$F_f = \frac{\sum (C' + (N - ul) \tan \phi') \sec \alpha}{\sum (W - (T_2 - T_1)) \tan \alpha + \sum (E_2 - E_1)} \quad \dots\dots\dots 3.22$$

$$F_m = \frac{\sum [C' l + (p - u) \tan \phi']}{\sum W \sin \alpha} \quad \dots\dots\dots 3.23$$

Where, u = pore pressure,

l = slice base length, and, α = inclination of slip surface at the middle of the slice

The method suggests assuming any type of force function, for example half-sine, trapezoidal or user defined. For a given force function, the inter-slice forces are computed by iteration procedure until, F_f is equaled to F_m in Equations' (3.22) and (3.23) (Morgenstern and Price 1965).

In summary, the Morgenstern price method

- ✓ Considers both inter-slice forces
- ✓ Assumes an inter-slice force function, $f(x)$
- ✓ Allows selection for inter-slice force function
- ✓ Computes FOS for both force and moment equilibrium.

2. Strength Reduction Factor Method (FEM)

The stability of the dam was also analyzed by the finite element method (FEM) with the shear strength reduction technique, which was proposed by (Zienkiewicz et.al 1975). This method is effective for assessing the performance together with the safety factor of slope and

locating the failure surface that were proven by past studies (Griffiths and Lane 1999), (Lane and Griffiths 2000), and (Cai and Ugai 2004). The essence of FEM with shear strength reduction technique is the reduction of the soil strength parameters ($\tan \phi$, C) until the soil fails of the structure occurs (Huang and Jia 2008). The FOS produced by ϕ this method is known as the strength reduction factor (SRF).

An embankment dam's stability depends on:

- The foundation's characteristics
- Unit weight and cohesion angle of internal friction are examples of fill materials.
- Considering the embankment section's geometry
- Additional elements, including the presence of water, loading circumstances, etc.

The procedures for static slope stability are well established, and have been concisely documented by (Duncan et al. 1987). The stability of Aguat Wuha earthen fill dam is analyzed using GeoStudio SLOPE/W (LEM) and Plaxis 2D software.

3.4.1 Acceptable Factors of Safety

The majority of dams are planned, and acceptance criteria for safety considerations are used to evaluate the safety of existing dams. Acceptable values of factors of safety for existing dams may be less than those for design of new dams, considering the benefits of being able to observe the actual performance of the embankment over a period of time. The factor of safety required will have an effect on determining whether or not remediation of the dam slope is necessary.

3.4.2 Computation of FOS

To analyze and find the critical slip surface and minimum FOS in different search options: grid and radius, entry and exit and auto-locate the critical slip surface. The selected search options is entry and exit were used in the study. The entry and exit option were used in this study applied to the surface of the model. The software searched for the circular critical slip surface among the number of trial surfaces within the given entry and exit ranges. The FOS is indicated at the axis of rotation, around which, the moment equilibrium is satisfied.

Loading Conditions for Analysis

The stability of the upstream and downstream slopes of an embankment dam is analyzed for the most critical or severe loading conditions that may occur during the life of the dam. The

most critical loading conditions of embankment dams, which should be considered in analyses are end of construction, steady state seepage, and rapid drawdown (USACE 2003).

Table 3.7 Various Load Cases and Minimum Required Factor of Safety (USACE 2003)

Slope	Load condition	Reservoir characteristics	Minimum factor of safety
U/s	End of construction	Reservoir empty	1.3
D/s	End of construction	Reservoir empty	1.3
D/s	Steady-state seepage	Reservoir at normal maximum operating level (Full Supply Level)	1.5
D/s	Maximum surcharge pool	Reservoir at maximum level	1.4
U/s	Rapid drawdown to critical level	Rapid drawdown to critical level	1.3

These safety considerations apply to the design of new high consequence of failure dams, on strong foundations, with low permeability zones made of soil that is not strain weakening, using reasonably conservative shear strengths and pore pressures developed from in-depth geotechnical investigations of borrow areas, laboratory testing and analysis of the results, and using the methods of analysis described above. It is envisaged that throughout construction, surface settlement points will be used to monitor deformations.

Table 3.8 Minimum Required Factors for Settlement (Fellenius et al. 2003)

Factor	Loading condition	Influence on Likelihood		
		Less Likely	Neutral	More Likely
Settlement as percentage of embankment height	During construction	1% or less	1-3%	3-5%
	post-construction	0.25% or less	0.25 - 1%	1-2%

3.4.3 Effects of Seepage Water on Slope Failure

The slope stability may also get reduced by increased shear stress due to changed loading conditions. The shear stress could be increased by:-

1. Increased stresses acting in directions being driving; external loads at the top of the slopes, water pressure in cracks at the top of the slope, increased soil weight due to increased water content etc.
2. The shear stress could also be increased by factors making the resistant forces of soil reduced any kind of removal of soil at the bottom of the slope, and drop in water level at the slope base (Duncan and Wright 2005).

3.4.4 General Considerations of Slope Stability Analysis Procedures for LEM

Limit Equilibrium Analyses (SLOPE/W) Procedures:-

The investigation of the Aguat Wuha earthen fill dam's static slope stability under typical loading circumstances was conducted using the Geo-Studio program of SLOPE/W. The following steps are taken into account when doing the slope stability analysis.

First, regions or n-sided closed polygons were used to define the geometry of the Aguat Wuha earthen fill dam. There can only be one type of soil in a given area. A lot of distinct regions can be given the same kind, but each region can only have one kind of soil. The geometry of the dam was imported using procedures that were similar.

In order to define a material's property in terms of soil strength, Mohr-Coulomb strength parameters C and ϕ were used. Depending on the given pore-water pressure circumstances, the parameters may be total or effective. Making a zero indicates the strength of an undrained force. The geotechnical study of the Aguat wuha dam served as the source of the property data used in the analysis. A slip surface command was established once the material inputs and SEEP/W linked pore water pressure were assigned. The entry and exit approach was chosen from among the many methods that are frequently used to describe the slip surface for the circular failure mode. This is because the entrance and exit command, when compared to the other instructions, is relatively accurate and enables the user to identify slip surfaces without difficulty by giving the assumed section of the surface where the slip surface will enter and exit. The Aguat Wuha earthen fill dam and its foundation were studied and built to resist collapse due to slope instability after the slip surface specified the allotted loading condition. For all possible reservoir arrangements, seepage circumstances, and both the current reservoir water level condition and the full reservoir condition, consideration of loading

conditions that may result in instability must be made. The end of construction, steady state seepage, and fast drawdown are the typical loading situations that are looked at in the static stability study. The limit equilibrium approach involves slicing the slope into several slices and applying the suitable equilibrium equation (force and/or moment equilibrium) after the loading condition has assigned the select analysis method. The Morgan Stern-Price approach is utilized in LE for the stability study of both upstream and downstream slopes.

As a last step in the demonstration analysis, the SLOPE/W program computes the slip surface's shear stress in relation to its shear strength to arrive at the safety factor. The analysis's findings display the minimum safety factor and the critical slip surface.

Geometry of Slopes

Important parameters of slope geometry affecting its stability include height and angle of slope. The critical height of the slope depends on the shear strength, density, and bearing capacity of the slope foundation. Slope stability generally decreases with an increase in the height of the slope. As the slope height increases, the shear stress within the toe of the slope increases due to added weight. Shear stress is also related to the mass of the material and the slope angle.

3.5 Design Seismic Parameters

Analysis of seismic stability of slopes is complicated by the need to consider the effects of:-

1. Dynamic stresses induced by earth quake shaking
2. The effects of those stresses on the strength and stress-strain behavior of the slope materials

Seismic forces reduce the margin of safety of an embankment dam. When an embankment dam is located in a seismic region, the stability analysis must consider earthquake forces. During an earthquake, the ground surface oscillates randomly in different directions. This motion can be represented by horizontal and vertical components. An earth dam should be treated as a flexible structure for determining dynamic pressure due to earthquakes. However, a simple method to account for earthquake forces in the design of structures is based on seismic coefficients. In this method, basic seismic coefficients or earthquake acceleration coefficients are used as given by equation 3.24

$$\alpha = \frac{\text{earth quake acceieration}}{g} \dots\dots\dots 3.24$$

Where α = seismic coefficient

α_g = Earthquake acceleration in a particular direction

Inertial force due to Earthquake is given by:

$$W_g \cdot \alpha_g = W \cdot a = F_e \dots \dots \dots 3.25$$

Slope stability is done by both a Limit Equilibrium software product and finite element analysis, use for stability analysis of earth slopes through Geo Studio and Plaxis. It can effectively analyze problems for a variety of slip surface shapes, pore-water pressure conditions, soil properties, analysis methods and loading conditions.

Using limit equilibrium, Slope/W can model heterogeneous soil types, complex stratigraphic and slip surface geometry, and variable pore-water pressure conditions using a selection of soil models.

Slope stability analyses can be performed using deterministic or probabilistic input parameters. Stresses computed by a finite element stress analysis were used. One of the most comprehensive programs for slope stability analysis is available. In this program, defining the geometry by creating regions and lines that define soil layers is the first step in starting an analysis. You can choose and use the analysis method, the soil characteristics, and the pore-water pressures. Using the slope/w feature of the Geo-Studio 2012 software and Plaxis 2D, the slope stability study of the designed dam will be examined.

The analysis of soil slopes is essential to evaluate their performance and stability under seismic loading. The over stress of a slope or the reduction in the shear strength of the sloped soil may cause the soil matrix to displace and thus jeopardizes public safety. In general, two indicators of performance are considered by the engineers in the analysis of the stability of soil slopes under seismic loading: the lateral deformation of the soil slope and the safety factor against failure.

3.6 Software Used for Deformation Analysis

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

3.6.1 Methods of Deformation Analysis

Predicting the deformation of embankment dams and ancillary facilities is a critical in safety consideration. The response of the embankment dam under the action of static loading is complex issue in the analysis of dam deformation. However, the development of two different methods of deformation surveys: geometric and physical analysis are commonly utilized to describe by physical laws, and to estimate by numerical models (Avella 1993).

In the first case, the information on the acting forces and stresses, and on the physical properties of the body are of no interest to the interpreter whereas, in the second case, the load-deformation relationship may be modelled using either a statistical or deterministic method.

Geometric analysis is used to describe the state of a deformable body, and its change in shape and dimension. This method is of particular importance when the deformable structure must satisfy certain geometrical conditions, such as the crest alignment of a dam, or the knowledge on the developed strains in the material is required.

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

The important parameters settlement and stresses distribution of Aguat Wuha dam have been extracted from the numerical analysis. The numerical analysis was conducted based on the FEM model SIGMA/W. In this study, the simulated results were compared to the actually measured vertical settlement resulted from a collected surveying data at the dam crest.

3.6.2 General Considerations and Analysis Procedures

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

3.6.2.1 Determination of Boundary Conditions

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

To fix the boundary condition of the problem, an approximate method or Boston rule (2V:1H) was applied for this work by supporting with FEM model. (Available at (<http://www.civil.uwaterloo.ca/maknight/course/CIVE353/Lecture/stress-load.pdf>)).

3.6.2.2 Specifying Initial Conditions

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

3.6.2.3 Constituting Material Models

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

3.7 Model Selection and Setup

Nowadays numerous finite element-based software is implemented to evaluate the performance of embankment dams, natural engineering slopes, and ground water flow.

In order to achieve the objectives of this study, finite element-based Geo Studio-2012 and Plaxis 2D software was employed. Those models are selected on their ability to solve complicated geometries, loadings, and material properties when compared with other applications. Geo Studio 2012 is finite element method which is now commonly adopted for modeling civil and hydraulic engineering structures mainly:-

- | | |
|----------------------------------|--------------------------------|
| 1. Embankment Dams. | 2. Reinforced walls and slopes |
| 3. Excavation and open pit mines | 4. Roads and bridges |
| 5. Earthquake deformation | |

It is a powerful tool for the analysis of many geotechnical structures. The sub program of Geo Studio 2012 software is used in seepage modeling in embankment dams, ground water seepage analysis, and for excess pore water pressure dissipation problems, slope modeling in an embankment dam. PLAXIS 2D is a software product based on finite element code that can be used to evaluate the performance of dams and levees with varying levels of complexity and it can consider analyses like stress-strain, seepage, slope stability, dynamic analysis, and also fast water drop in the reservoir. Plaxis has an excellent capability to model geotechnical engineering-related problems and its flexible programming capability based on its application, availability, loadings, and material properties when compared with other applications.

3.7.1 Seepage Estimation Using SEEP/W and Plaxis Model

The Figures below correspond to the finite element models used in the analyses and the estimated discharges. The usage of clay core as a water-resistant material is quite effective, as shown by the SEEP/W results, since the phreatic level steeply decreases. Making sure that the water is adequately drained away and that the amount of water left behind is manageable and minimal is one of the challenges of design and construction. Based on the seepage flux through the embankments and divided by the typical dam crest length, the seepage flow was computed. The amount of total discharge through the dam body is calculated by:-

$$Q = q \text{ flux (m}^3\text{/s/m)} \times \text{Average Length of the dam crest (m)} \dots\dots\dots 3.26$$

3.8 Conceptual Framework of the Study

In this research the process of analyzing the dam seepage, slope stability and deformation was conducted by using primary data collection at the field level such as conducting a survey at the dam top crest and upstream water level and measuring the amount of seepage at downstream of the dam site and then secondary data was collected such as Geotechnical parameters, normal reservoir water level, and geometry of dam section at the office. After that the dam section is drawn using the Auto CAD 2018 software and saved as a format of dxf and imported into Geo Studio software to assign each regions and materials to the dam section and to module the process of seepage, slope stability and deformation and then the simulated result and the field measured data are compared to evaluate the performance of the dam, the same procedure also done for plaxis 2D software, as shown in Figure 3.5 below.

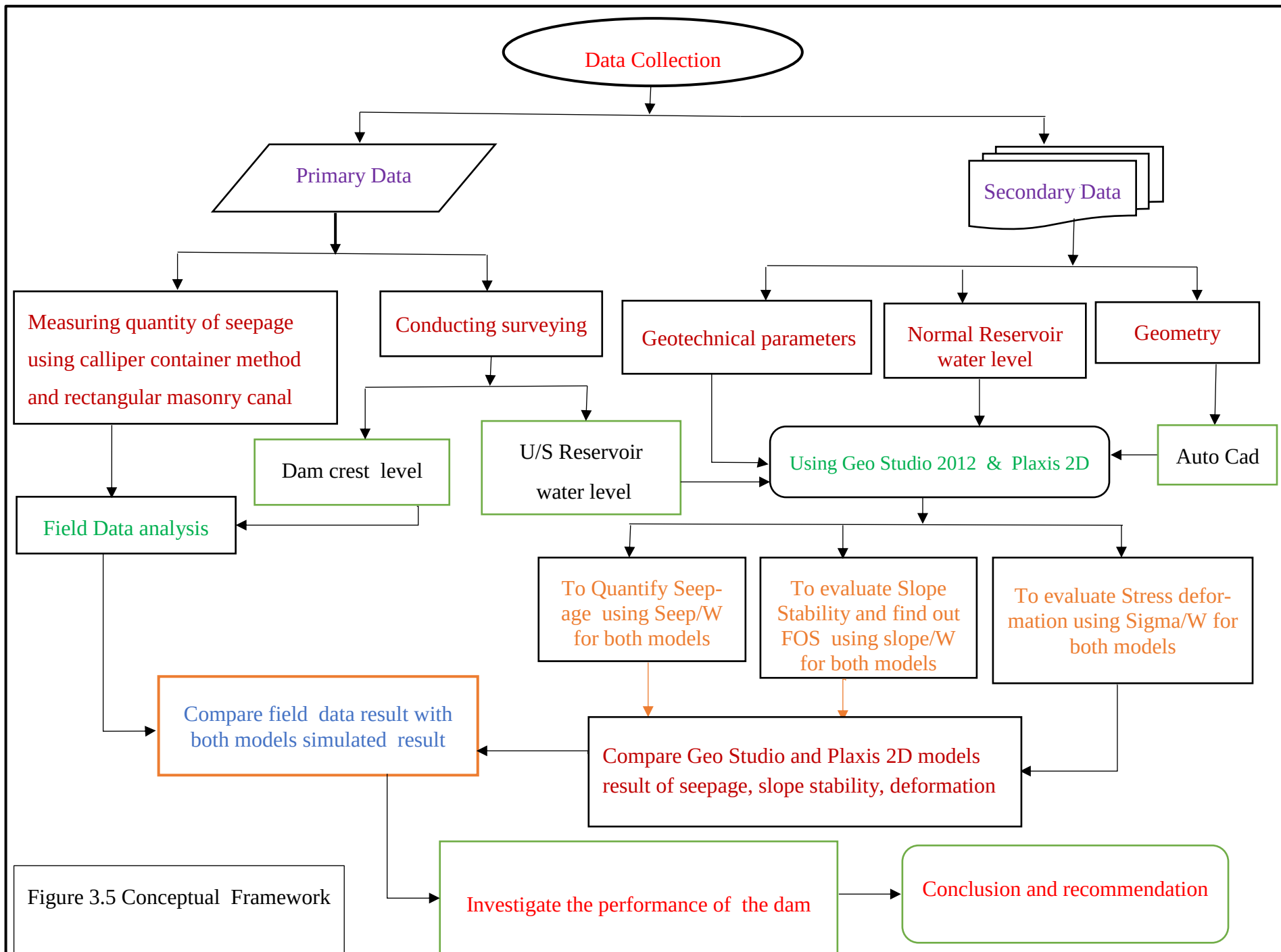


Figure 3.5 Conceptual Framework

4 RESULTS AND DISCUSSION

One of the most important parts of embankment dams is the study of discharge flow-through construction materials as well as the hydraulic balance of the dam and the amount of water discharging from the foundation and body of the dam (Rezaee et al. 2014).

In this study, seepage analysis was carried out based on the three reservoirs water levels or impoundment conditions. These conditions are maximum reservoir water level at an elevation of 2875.25m a.m.s.l Normal reservoir level at an elevation of 2874m a.m.s.l and at reservoir level of 2868m a.m.s.l.

4.1 Seepage Analysis and Results

4.1.1 Steady Seepage Analysis at Normal Reservoir Level (Elevation of 2874m a.m.s.l)

At this level, actual field measurement data of flow discharge was taken at downstream of the dam site using a rectangular channel and compared with the simulated flow obtained at this water level boundary condition using the seep/w software. This impoundment condition (at Elevation 2874 m a.m.s.l) is the maximum active storage. During the impoundment, there is fluctuation in the water level of the reservoir due to various reasons. This might be the main reasons for seasonal inflow variation, evaporation and seepage losses. The numerical analyses of seepage flux (discharge), phreatic surface, exit gradient and maximum seepage velocity for the entire pond level are computed with similar procedures of seepage analysis at the required reservoir level using SEEP/W model as shown Figure 4.1 below.

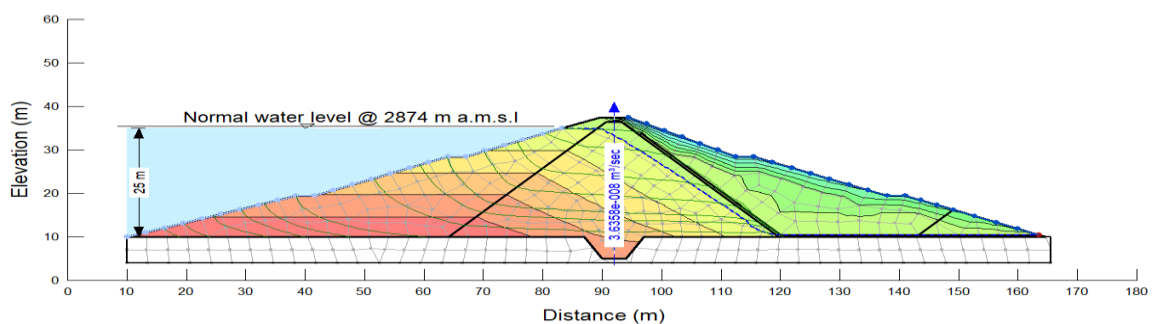


Figure 4.1 Seepage Through the Dam Body and Location of the Phreatic Line

The amount of total discharge through the dam body is $Q = 0.0000117 \text{ m}^3/\text{s}$

$q \text{ flux} = 3.64 \times 10^{-8} \text{ m}^3/\text{s}/\text{m}$ and Average Length of the dam crest = 321.5 m

4.1.2 Steady Seepage Analysis at Full Reservoir Level (Elevation = 2875.25m a.m.s.l).

The maximal active storage is at this impoundment situation (Elevation = 2875.25m a.m.s.l) The reservoir's water level changes throughout the impoundment for a number of causes. According to calculations, the embankment's computed seepage quantity (qflux) was $5.16 \times 10^{-8} \text{ m}^3/\text{s}/\text{m}$ per meter of dam length and average dam crest level is 321.5 m. So, The amount of seepage flow rate through the dam is $Q = 1.66 \times 10^{-5} \text{ m}^3/\text{s}$ at full reservoir level.

Similar to the analysis mentioned above, the pressure distribution showed the expected minimum at the top and maximum at the bottom as the depth of the water increased.

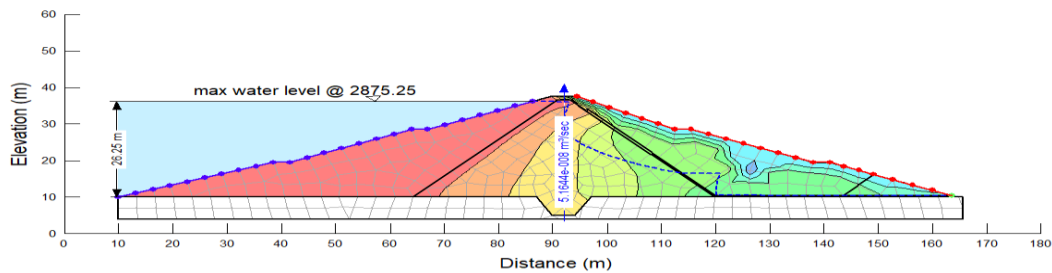


Figure 4.2 Total Head Distribution, Flow Path, and Location of Phreatic Line

4.1.3 Steady Seepage Analysis at Reservoir Water Level (at Elevation 2868m a.m.s.l)

At this elevation field measurement data was taken for the second time at the upstream of the reservoir level and at the downstream of the dam body to compare and contrast the actual flow through the dam body with the simulated one. The numerical analyses of seepage flux (discharge), phreatic surface, exit gradient and maximum seepage velocity for the entire pond level are computed with similar procedures of seepage analysis at this reservoir water level using SEEP/W model as shown in Fig 4.3 below. The amount of total discharge through the dam body is $Q = 5.5 \times 10^{-6} \text{ m}^3/\text{s}$

q flux = $1.715 \times 10^{-8} \text{ m}^3/\text{s}/\text{m}$ and Average Length of the dam crest (m) = 321.5 m

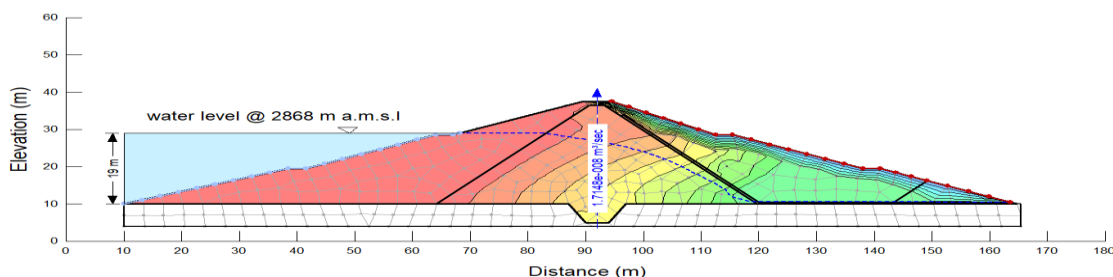


Figure 4.3 Seepage Through the Dam Body and Location of the Phreatic Line

When the quantity of seepage is multiplied by the average crest length of the dam, or 321.5 m, the results from the seep/w model simulation at this reservoir level are 0.0000055 m³/s. In this study practical seepage measurement was done at normal reservoir level (2874 m a.m.s.l) and at elevations 2868 m a.m.s.l at downstream rectangular canal using Manning's equation and calliper container method , and compared with the seep/w model result and summarized in Table 4.1 below.

Table 4.1 Comparison of Seepage Model Output and Field Measured Values

Upstream water level	Seep/W result	Rectangular canal	Calliper container
At Full level (2875.25 m)	$1.66 \times 10^{-5} \text{ m}^3/\text{s}$		
At Normal level (2874 m)	$1.17 \times 10^{-5} \text{ m}^3/\text{s}$	$7.89 \times 10^{-3} \text{ m}^3/\text{s}$	$9.4 \times 10^{-3} \text{ m}^3/\text{s}$
At Reservoir level (2868 m)	$5.5 \times 10^{-6} \text{ m}^3/\text{s}$	$6.23 \times 10^{-3} \text{ m}^3/\text{s}$	$8.1 \times 10^{-3} \text{ m}^3/\text{s}$

Seepage Source Identification

Good understanding of the sources/causes of the observed excessive seepage certainly plays a major role in providing a recommendation against problems concerned with structural safety or water loss. During the elevation of 2874 m impoundment, a sign of leakage occurred in the downstream face of the dam, especially at the bottom outlet conduit joints. it is located in the middle of the clay core section with the bottom outlet. Thus, suggestions clue to the considerable difference in seepage discharge quantity using the numerical analysis and actual measurements. Even though field measurement data was not taken at the an elevation of maximum water level (2875.25m a.m.s.l) the module shows that the phreatic line crosses the downstream filter section and passes through the shell material section. Therefore, it can be concluded that the downstream face of the shell erodes and high seepage flow rates is observed especially between interfaces of the outlet conduit joints and the dam body. Because the dam filter is not appropriate and the tunnel steel layer is not provided water proofing material. According to USACE (2014) recommendation, the probability of failures of the dam based on seepage discharge will likely happen because the seepage water originating from the dam body is not pure (there is material transport along the flow path) and this indicates that there is scouring problem.

4.2 Seepage Estimation Using Plaxis Model

One of the most important parts of both earth and rock fill dams, is the study of discharging flow through construction materials as well as the hydraulic balance of the dam and the amount of water discharging from the foundation and body of the dam. The result of seepage quantity and pore water distribution is presented below.

4.2.1 Steady Seepage Analysis at Reservoir Level (Elevation 2868m a.m.s.l)

The scenario water level condition is at this reservoir level (Elevation of 2868 a.m.s.l.). In this scenario, the Plaxis model's computed seepage quantity through the dam and its foundation is simulated and displayed in the image below. The amount of seepage passing through the dam's body and foundation was $1.1123 \times 10^{-5} \text{ m}^3/\text{s}/\text{m}$ per meter of the dam's length. At that time, the amount of seepage flow rate through the dam is $Q = 3.57610^{-3} \text{ m}^3/\text{s}$

$q_{\text{flux}} (\text{m}^3/\text{s}/\text{m}) = 1.1123 \times 10^{-5} \text{ m}^3/\text{s}/\text{m}$ and Average Length of the Dam Crest = 321.5 m

At the Reservoir level (2868m a.m.s.l), the computed seepage quantity exceeded the Plaxis model evaluation at the current water level seepage quantity by $3.5705 \times 10^{-3} \text{ m}^3/\text{s}/\text{m}$ per meter length. The pressure distribution likewise came out as expected in the study of the Seep/w model discussed above, with a low at the top and a maximum at the bottom as the water depth increased. And the location of the seepage face (phreatic line) for both model analysis and the impounding condition fall within the downstream filter. It can be said this as a positive side because the seepage line is adjoined to the provided horizontal drains without intersecting the downstream shell material. The seepage quantity through the dam body and foundation is $90.21 \times 10^{-3} \text{ m}^3/\text{d}/\text{m}$ and the exit velocity is $10.55 \times 10^{-3} \text{ m}/\text{day}$.

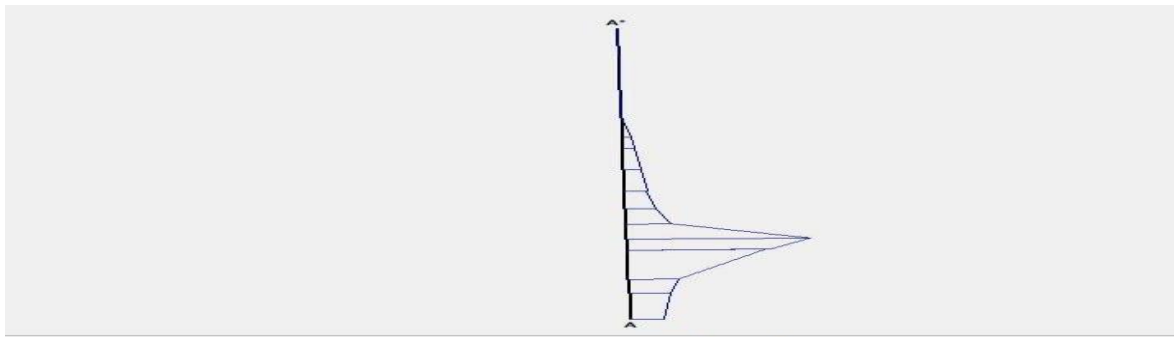


Figure 4.4 Seepage Quantity Through Dam Body and Foundation @ 2868 m.a.s.l

The Finite element mesh of the model in Figure 4.5, presents the phreatic line and seepage quantity through the dam body and foundation. The phreatic surface (zero pressure line) can

be viewed as the blue line that crosses the dam. The location of the phreatic line is fallen within the downstream transition filter. A vertical line (flux section line) has been drawn through the center of the clay core and foundation of the dam and the total entire seepage through the dam body and its foundation is computed at the different cross section.

The phreatic line as seen in Figure 4.5 below has emerged below the toe of the dam. This indicates that the dam is safe against sloughing problems, which is the main cause of failure of most downstream faces of dams.

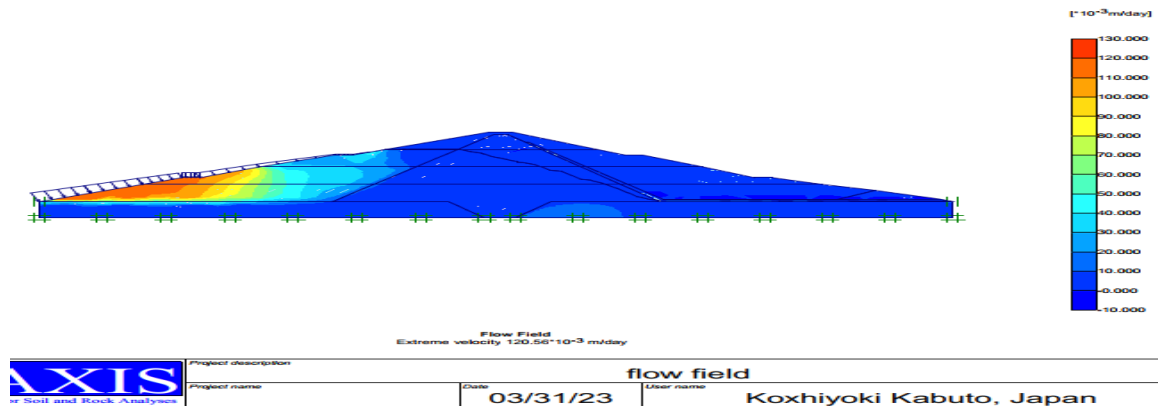


Figure 4.5 Flow Field = 120.56×10^{-3} and Location of Phereatic Line during Current WLC

4.2.2 Steady Seepage Analysis at Normal Reservoir (Elevation 2874m a.m.s.l)

The maximal active storage is at this impoundment situation (Elevation = 2874 m a.m.s.l.). The Plaxis model is used to calculate the numerical analyses of seepage through the dam's body and foundation, the placement of the phreatic line, and the flow field (extreme velocity) throughout the entire pond level. The embankment's computed seepage quantity was calculated to be $1.55117 \times 10^{-5} \text{ m}^3/\text{s}/\text{m}$ per meter of the dam's length. The amount of seepage flow rate passing through the dam is $Q = 0.004987 \text{ m}^3/\text{s}$.

$$q_{\text{flux}} = 1.55117 \times 10^{-5} \text{ m}^3/\text{s}/\text{m} \quad \text{and} \quad \text{Average Length of the Dam Crest} = 321.5 \text{ m}$$

At the Normal reservoir level, the computed seepage quantity exceeded the Plaxis model evaluation at the current water level seepage quantity by $4.9753 \times 10^{-3} \text{ m}^3/\text{s}$ per meter length, and at the full reservoir level using the Plaxis model analysis. The computed seepage quantity exceeded the Seep/w model analysis results at the same water level condition (2875.25 m a.m.s.l) seepage quantity by $1.11 \times 10^{-5} \text{ m}^3/\text{s}$ per meter length.

Similar to that, this Section computes the normal reservoir level at an elevation of 2874m a.m.s.l. and the reservoir level at 2868m a.m.s.l the ascertained seepage extrema velocity

determined by the Plaxis model. At a reservoir level of 2868m a.m.s.l., the lowest extreme seepage velocity was seen, while at a reservoir level of 2874m a.m.s.l., the highest extreme seepage velocity is experienced.

Similar to the analysis mentioned above, the pressure distribution showed the expected minimum at the top and maximum at the bottom as the depth of the water increased. Additionally, the downstream filter takes into account the location of the seepage face (phreatic line) for both model analysis and the impounding condition. This is a benefit because the seepage line does not cross the shell material farther downstream and is adjacent to the provided horizontal drains. The extreme velocity is 20.35×10^{-3} m/day, and the amount of seepage through the dam body and foundation is 295×10^{-3} m³/d/m.

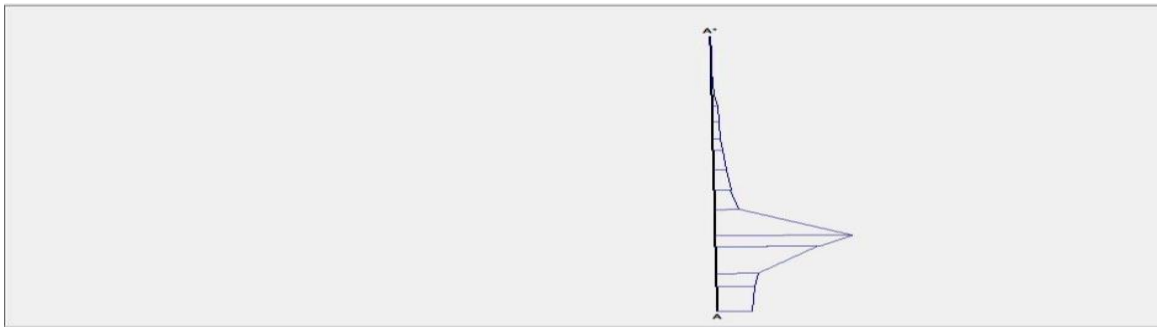


Figure 4.6 Seepage Quantity Through the Dam Body and Foundation @ 2874 m.a.s.l.

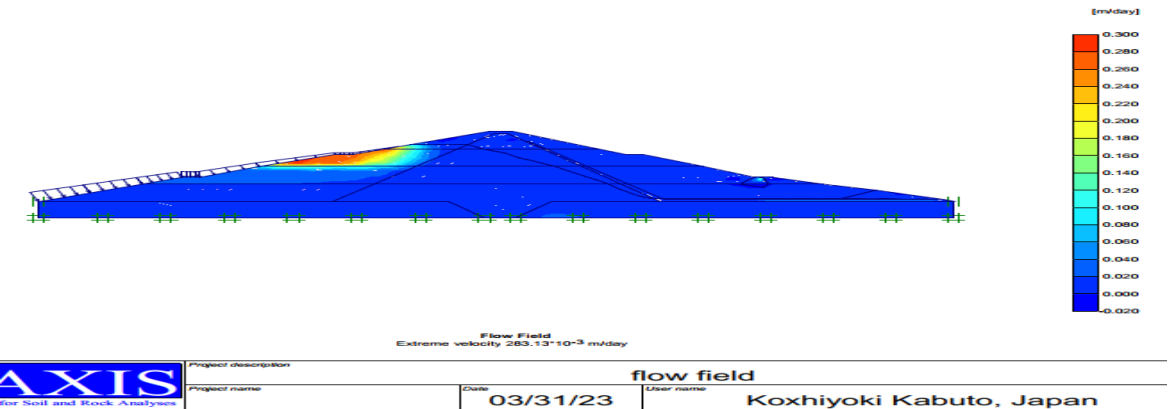


Figure 4.7 Aguat Wuha Dam Flow Field (Extreme Velocity = 283.13×10^{-3}) and Location of Phreatic Line during NWL 2874 a.m.s.l Condition

The amount of seepage derived from the Plaxis model simulations for the full reservoir level situation (Elevation = 2874 m a.m.s.l) was $0.004987 \text{ m}^3/\text{s}$. This was fairly accurate in terms of the measured seepage magnitude. Practical seepage measurement was difficult in this

study because it was hard to measure seepage at the outlet conduit and there was no previously built regular weir close to the outlet conduit that was intended to measure unexpected seepage. The amount of seepage from the dam body has been attempted to be measured using the wade flow recording technique at the dam's downstream toe.

Table 4.2 Summarized of Seepage Analysis at the Two Important Impoundment Conditions of Various Locations of the Dam Body and its Foundation

Seepage analysis/ measured Result		At an elevation of 2868 m a.m.s.l	At an elevation of 2874m a.m.s.l
1.Quantity of seepage (m ³ /s)	Using SEEP/W	$5.5 \times 10^{-6} \text{ m}^3/\text{s}$	$1.17 \times 10^{-5} \text{ m}^3/\text{s}$
	Using PLAXIS 2D	$3.576 \times 10^{-3} \text{ m}^3/\text{s}$	$4.987 \times 10^{-3} \text{ m}^3/\text{s}$
	Rectangular lined masonry canal	$6.23 \times 10^{-3} \text{ m}^3/\text{s}$	$7.89 \times 10^{-3} \text{ m}^3/\text{s}$
	Calliper container (taken average)	$8.1 \times 10^{-3} \text{ m}^3/\text{s}$	$9.4 \times 10^{-3} \text{ m}^3/\text{s}$
2. Location of seepage line		Under the d/s filter	Under the d/s filter

According to Table 4.2, in all cases, the measured seepage exceeds the software's projected value. When a PLAXIS seepage evaluation result provides more information about diagrammatically illustrating the effect of seepages in a different body of a dam, these things provide researchers or engineers in the design phase with additional knowledge about seepage. The amount of seepage measured by both limit equilibrium and finite element method compares to show its result similar because both methods that are SEEP/W and PLAXIS 8.2 using a finite method analysis type of seepage result evaluation. As said, the foundation, abutments, and portion of the reservoir area are mostly made of alluvial, colluvial, and aphanitic basaltic bed-rock, which generally indicates that the Aguat wuha micro earthen dam has excessive seepage through the dam body and foundation, which may cause seepage failure. Discontinuities of all kinds, such as bedding contact, fracturing/joint, tectonic joint, etc., have a significant impact on these units. When seepage is measured practically, a lot of it registers because of discontinuities and heavily fractured shale rocks. Thus, the aforementioned factors result in a dam Problems with leakage and stability are anticipated, especially on the abutments.

4.3 Slope Stability Analysis

The static stability of the upstream and downstream slopes of Aguat Wuha dam was simulated, using the leading slope stability CAD product software SLOPE/W. The slope stability against sliding risk is represented by the safety factor (FOS). The computed out comes of this study for the dam slope stability analysis using the limit equilibrium (Morgenstern-Price) and finite element method with the shear strength reduction technique were considered. In this analysis, five different cases of operation; at the end of construction, steady-state at current reservoir water level, steady-state at maximum reservoir water level, rapid/sudden draw-down from current reservoir water level, and rapid drawdown from maximum reservoir water level are considered.

In most cases, the stability of the downstream slope is critical at the end of construction and steady seepage. On the other hand, the upstream slope may have its critical factor of safety at the end of construction and during rapid drawdown (Mahdi and Alireza 2016).

4.3.1 Slope Stability Analysis Using Geo-Studio 2012

The figures below show the computed critical slip surfaces and their accompanying factors of safety for the various loading circumstances.

The shear strengths of the dam materials and the foundation control slope stability (Coulomb 1776).

According to Kaare (1997), the slope itself is the critical slip plane for a homogeneous dam on an analogous foundation, provided that the shear strength is provided only by friction.

If the shear strength is provided by friction and cohesion, on the other hand, the critical plane breaks through the dam. For the zonal dam type, the crucial slip plane hits the foundation line and the core of the dam has a shear strength that is equal to or greater than the foundation's clay material's shear strength.

The slip plane may not always contact the foundation line if the foundation is stiffer than the embankment, depending on the rigidity of the foundation. The usage of free-draining material to address the issues brought on by this pore water pressure development is strengthened by the fluctuations in pore water pressure at different situations when the upstream dam body is subjected to variable stress conditions as a result of various reservoir water level activities.

A permeability of $k \ 10^{-2}$ m/s and free draining materials are required across the upstream shell during an earthquake (Kaare 1997).

4.3.2 End of Construction Slope Stability Analysis

Significant pore pressure growth will be expected in either the foundation or the embankment at the conclusion of construction, when the reservoir is empty, and during the embankment's construction (USACE 2004).

At the conclusion of construction, the stability of the upstream and downstream slopes is crucial in this situation. By taking into account the starting pore water pressure condition from the parental analysis of SEEP/W, the undrained shear strength concepts criterion was utilized to predict the behavior of the embankment materials and the upstream and downstream slope stability analysis of Aguat wuha micro dam.

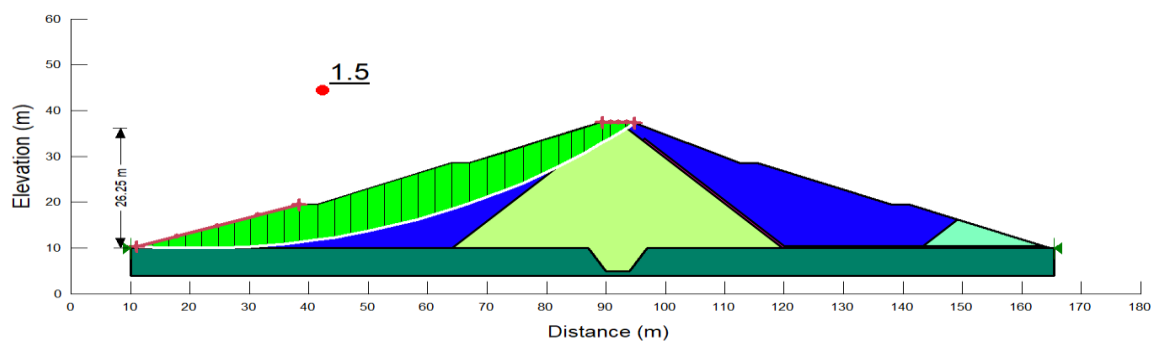


Figure 4.8 FOS of UpStream Slope at End of Construction

4.3.3 Slope Stability Analysis Under Steady-State Seepage

In this condition, the reservoir is full long enough for seepage water to percolate through the embankment; the pressure in the pore water in the downstream portion reaches its highest values. The downstream slope may have its lowest factor of safety against sliding (Krishna 2006).

The phreatic surface within the embankment has been developed to compute the slope stability analysis. The steady state seepage state is first established by seepage analysis. The Mohr-Coulomb failure criterion was used to calculate the drained (effective) stress strength parameter in accordance with the norms set by the various authorities. The results of the limit equilibrium analysis using the SLOPE/W model are coupled input for the steady state seepage analysis using SEEP/W. The stability of the downstream slope is typically crucial for steady seepage, and Figure 4.9 - 4.10 below illustrates the findings of the Morgenstern-Price method's factor of safety calculation with its slip surfaces.

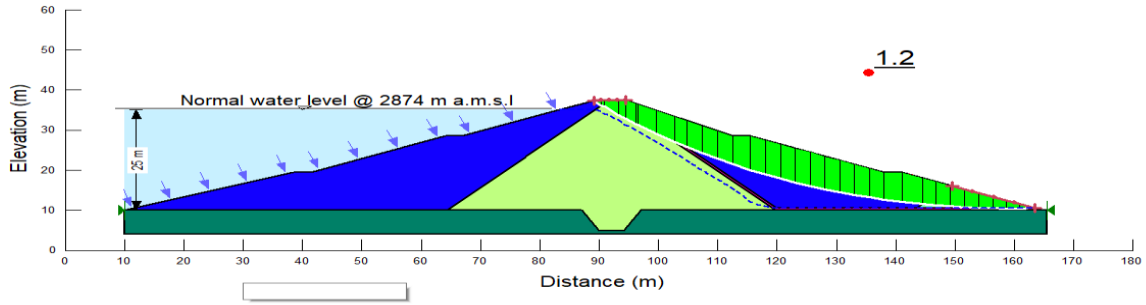


Figure 4.9 FOS of the UpStream Slope during NWL with Earth Quake

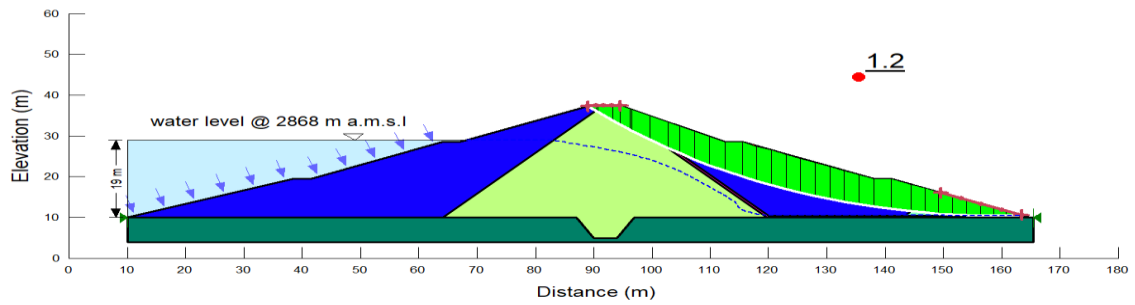


Figure 4.10 FOS of Down Stream Slope at 2868 Reservoir Level with Earth Quake

4.3.4 Rapid Draw Down Condition Stability Analysis

The upstream slope may fail by sliding once the reservoir is decreased, especially if seepage water has entered the embankment and the reservoir has theoretically been operating as a critical decline or totally empty for some time. So, during the sudden drawdown, the upstream slope has a crucial role to play in terms of safety. In this instance, the reservoir is depleted more quickly than steady-state seepage conditions can be established, which causes the pore pressures inside the dam's body to rise. In an un-drained unloading scenario, total stresses fall but shear stresses within the embankment increase (Duncan et al. 1997). This is due to the rapid removal of the sustaining water load from the upstream face of the embankment along with changes in pore pressure.

In this condition, According to the various agencies; standard, an effective stress parameter using the Mohr-Coulomb failure criterion was utilized. The upstream slope should be analyzed for rapid drawdown conditions from the top of the active conservation capacity water surface to the top of the inactive capacity water surface and from the maximum water surface to the top of the inactive storage water surface (USACE 2014). In order to do this research, it was assumed that the reservoir would operate uncontrollably and that the stored

water would evaporate in 5.79 days, or 500,256 seconds, and that a rapid drop of 14.5 m depth between the full reservoir level and the intake structure would be taken into account.

The reservoir storage above the center of the out let (2859.5 m) is shown below and the out let diameter is 0.6 meters with a length of 100m

Table 4.3 Reservoir Depletion Time

ELEVATION	(Δ)vol. (m³)	H (m)	Q (m³/s)	75% flow	dep. Time (days)
2874.0	50040.26	14.50	2.86	2.15	0.27
2873.5	47996.37	14.00	2.81	2.11	0.26
2873.0	45965.67	13.50	2.76	2.07	0.26
2872.5	43958.79	13.00	2.71	2.03	0.25
2872.0	41947.33	12.50	2.66	1.99	0.24
2871.5	39888.14	12.00	2.60	1.95	0.24
2871.0	37782.61	11.50	2.55	1.91	0.23
2870.5	35267.32	11.00	2.49	1.87	0.22
2870.0	33033.17	10.50	2.43	1.83	0.21
2869.5	31475.66	10.00	2.38	1.78	0.20
2869.0	29932.80	9.50	2.32	1.74	0.20
2868.5	28159.97	9.00	2.25	1.69	0.19
2868.0	26437.21	8.50	2.19	1.64	0.19
2867.5	25016.76	8.00	2.13	1.59	0.18
2867.0	23367.77	7.50	2.06	1.54	0.18
2866.5	21808.60	7.00	1.99	1.49	0.17
2866.0	20627.22	6.50	1.92	1.44	0.17
2865.5	19523.57	6.00	1.84	1.38	0.16

2865.0	18471.31	5.50	1.76	1.32	0.16
2864.5	17456.86	5.00	1.68	1.26	0.16
2864.0	16486.57	4.50	1.59	1.20	0.16
2863.5	15568.26	4.00	1.50	1.13	0.16
2863.0	14671.97	3.50	1.41	1.05	0.16
2862.5	13726.73	3.00	1.30	0.98	0.16
2862.0	12750.71	2.50	1.19	0.89	0.17
2861.5	11832.96	2.00	1.06	0.80	0.17
2861.0	11008.20	1.50	0.92	0.69	0.18
2860.5	10244.00	1.00	0.75	0.56	0.21
2860.0	9476.54	0.50	0.53	0.40	0.28
2859.5	8230.35	0.00	0.00	0.00	
762,153.68		5.79			

Table 4.4 The Computed FOS against Slope Stability Under Different Loading Conditions

Loading Condition	Computed Factors of Safety			Status
	FoS _{min.(usce)}	Upstream	Downstream	
➤ End of construction	1.3	1.5		Stable
➤ Steady-state seepage at 2874 m a.m.s.l With earth quake	1.1		1.2	Stable
➤ Rapid drawdown (Transient seepage 2874)	1.3	2.273	---	Stable
➤ Steady-state seepage With earth quake at 2868m a.m.s.l	1.1		1.2	Stable
➤ Rapid drawdown (Transient seepage 2868ma.m.s.l)	1.3	2.0	----	Stable

The reason for the difference in FOS is primarily related to the normal stress distribution along the cross-section. According to a study carried out by (Krahn 2004), a significant difference in normal stress distribution, downstream area, can be found.

Janbu (1973) also suggests studying the shear stress concentrations in critical zones by using a stress-strain relationship, instead of the LE equations. These wide spread experiences highlight the computational difficulty of realistic stress distribution, which can be more precisely computed by modern FE analyses.

4.4 Stability Analysis Using Plaxis V 8.2

The stability of an embankment dam depends on the pore developed either in the dam body or in the foundation, characteristics of the foundation and fill materials, the geometry of the embankment section, and additional factors such as the presence of water, loading conditions etc. The slope stability against sliding risk is represented by a safety factor (FOS). In this analysis, three different cases of operation; the end of construction, steady-state seepage, and rapid drawdown are considered. The soil parameters like cohesion, friction angle and unit weight of the soil have a greater role in the stability of slopes. They have an inversely proportional effect on relative displacements, which means that the increase in the value of soil parameters leads to smaller displacements.

It is crucial to take into account both the stability during construction and the final stability when designing an embankment dam. The construction staging (2 lifts), with the first stage being the dam foundation in 6 meters below ground level and the second stage being the dam body above on the OGL 27.5 meters, was chosen for the deformational and stability assessments.

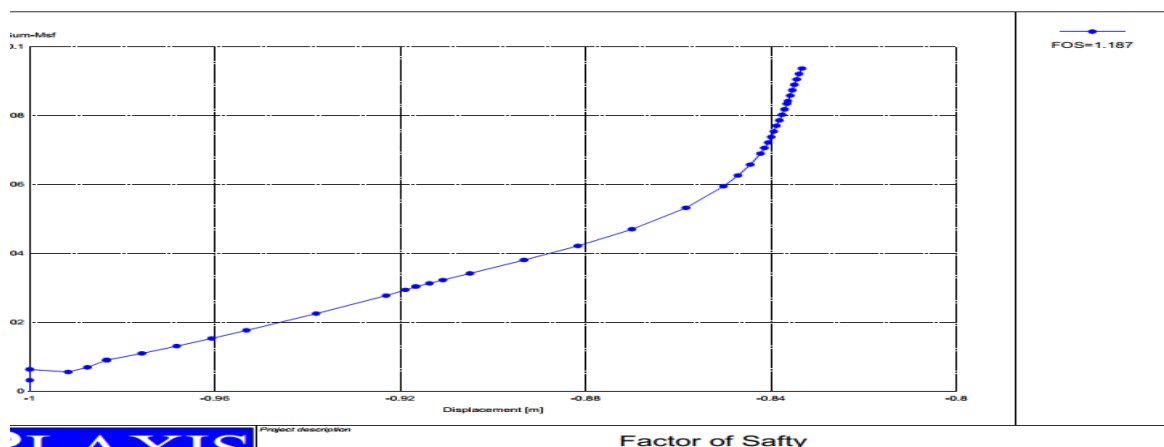


Figure 4.11 Displacement (x) vs Sum-Msf (y), FoS = 1.187 during Steady State @ 2874 m

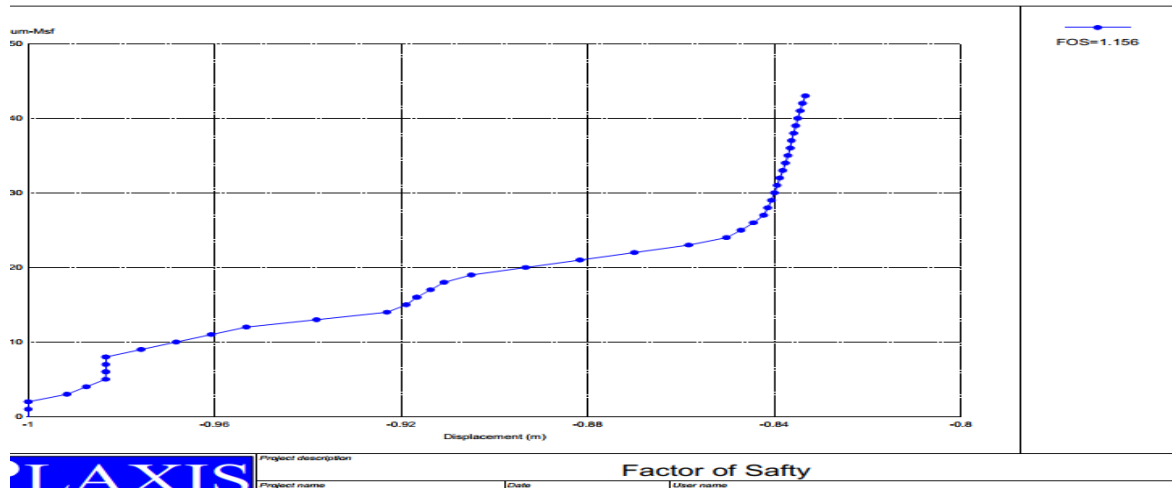


Figure 4.12 Displacement (x) vs Sum-Msf (y), FoS = 1.156 during Steady State @ 2868 m

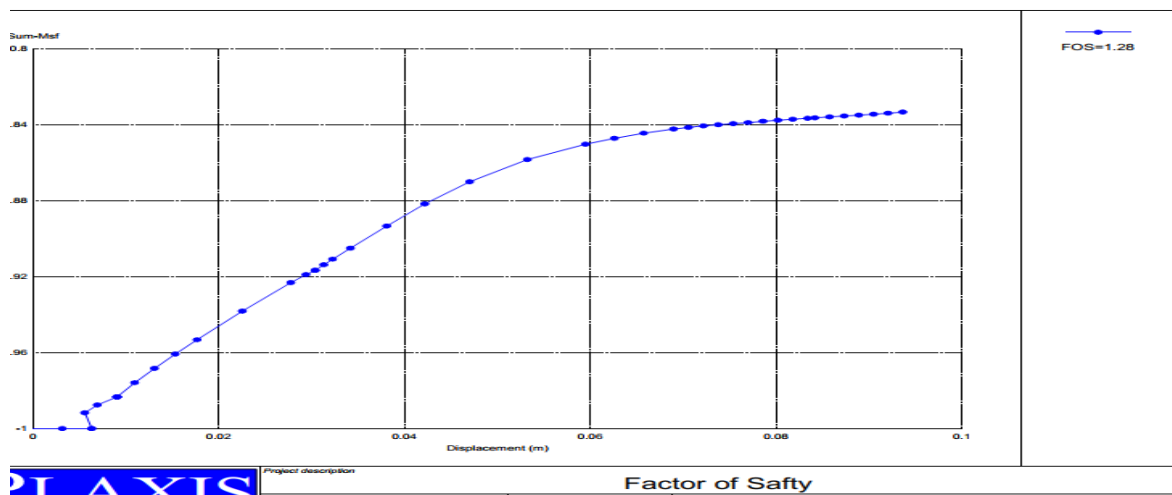


Figure 4.13 Displacement (x) vs Sum-Msf (y), FoS = 1.28 during End of Construction

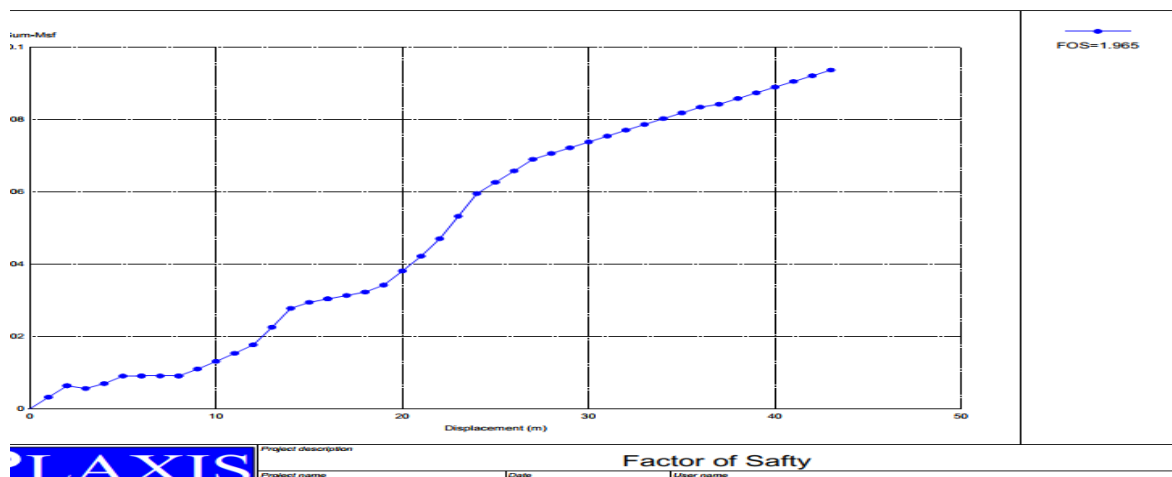


Figure 4.14 Displacement (x) vs Sum-Msf (y), FoS = 1.965 during Rapid Draw Down

Table 4.5 Factor of Safty Result Made by Plaxis

Case	Loading Condition	Type of analysis	Factor of Safty		Status
			FoS min.	FoS Computed	
Case 1	End of Construction	Static Analysis	1.3	1.281	Fail
Case 2	Steady-state seepage @ 2868.0	Static Analysis	1.5	1.156	Fail
Case 3	Steady-state seepage @ 2874.0	Static Analysis	1.5	1.187	Fail
Case 4	Rapid drawdown (from NWL) 2868	Static Analysis	1.3	1.965	Pass

Table 4.6 Comparison of the FoS Analysis Made with Geo Studio and Plaxis

Case	Loading Condition	Critical Slope	SLOPE/W	Plaxis
1	End of construction	Upstream	1.5	1.281
		Down stream	1.3	
2	Steady-state seepage @ 2874 a.m.s.l	Upstream, with earth quick	1.3	
		Down stream with earth quick	1.2	1.156
3	Steady-state seepage @ 2868 a.m.s.l	Upstream with out earth quick	1.3	
		Down stream with earth quick	1.2	1.187
4	RDD from 2868 a.m.s.l	Upstream with out earth quick	2.0	1.965
		Down stream with earth quick	1.2	

Table 4.6 shows a comparison of the results from LE and FE analysis. The result shows FEM has been shown to be a reliable and robust method for assessing the factor of safety of slopes. One of the main advantages of the FEM is that the factor of safety emerges naturally from the analysis without the user having to commit to any particular form of the mechanism. The FE approach for determining the factor of safety of slopes has satisfied the criteria for effective computer-aided analysis. The factor of safety differences between the finite element method and limit equilibrium method results are small. Any failure mode develops naturally, there is no need to specify a range of trial surfaces in advance. Multiple failure surfaces evolve naturally. LEM has been the primary method used in estimating the stability of slopes for decades. However, due to some of the shortcomings of LEM, the FEM is a great tool to model the non-linear stress-strain behavior of materials and understand the stability based on deformations. There is no concept of slices in the FE approach, there is no need for assumptions about slice side forces.

4.5 Static Stress-Deformation Analysis

It is important to recognize that all embankment dam in service deforms and settle. In general, excessive deformations in embankment dams may result in aesthetically acceptable surficial appearance such as reductions or loss of free board, and internal and /or external cracks (Chen and Liu 1990). Under static loads, embankments can deform due to volumetric changes, lateral spreading, or shear displacements in the materials that make up the embankment and its foundation. The FEM model SIGMA/W served as the foundation for the numerical analysis. In this study, two conditions of analysis were carried out to compute the deformation processes during the end of construction and full reservoir level conditions. To compare the computed numerical analysis of Aguat wuha micro earthen dam, field recorded data interpretations and measurements using GPS, Total Station surveying instruments had conducted. Results for displacement in the horizontal and vertical directions were nearly identical in SIGMA/W and Plaxis. In order to avoid having to crack through the core, the horizontal displacement must be no more than 0.5 m, even if the impermeable layer varies in thickness from 0.5 m at the top to 1.5 m at the bottom.

4.5.1 Static Stress-Deformation Analysis

Static Deformation Analysis Using Geo Studio 2012

The magnitudes and direction of the deformations of the embankment are controlled by the properties of the foundation material and the embankment, the geometry of the abutment and

embankment and the rates of embankment placement, the conditions of reservoir loading and the distribution of stresses within the different zones or layers within the embankment and its foundation (USACE 2014). In this study, the contours and graphs of magnitude and distribution of the vertical and horizontal displacements are computed in two conditions, the end construction and full reservoir condition. For the SIGMA/W-based computed results, the horizontal displacement direction towards the upstream is negative, and the horizontal displacement towards the downstream direction is positive.

4.5.2 Settlement Analysis at the End of Construction

At end of the dam construction, the maximum downstream horizontal displacements occur at about one-fifth of the dam's height in the downstream central part of the shell, and the maximum computed value is 0.12 m for downstream sides, and the maximum upstream horizontal displacements computed value is 0.108 m for upstream sides. Whereas the maximum vertical displacements 0.1718 m occurs at the crest of the dam, which is 0.624% of the dam height which is conceded to the USBR "1 percentage rule" standard. The change in the maximum vertical settlement and downstream horizontal displacement at the end of dam construction with the number of simulation contours and graphs are shown in 4.15 below.

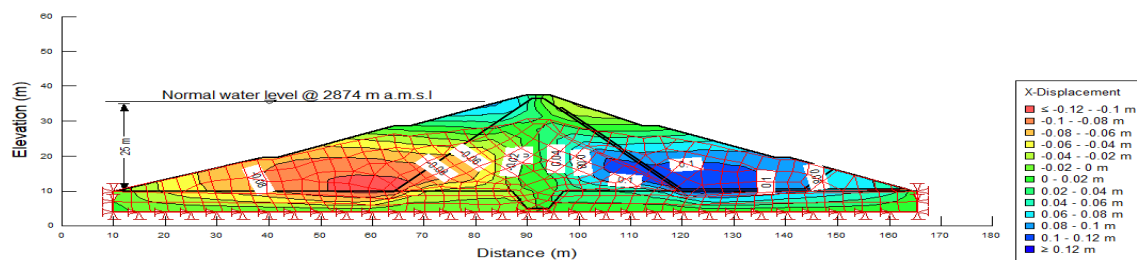


Figure 4.15 Vertical Settlement (End of Construction Condition)

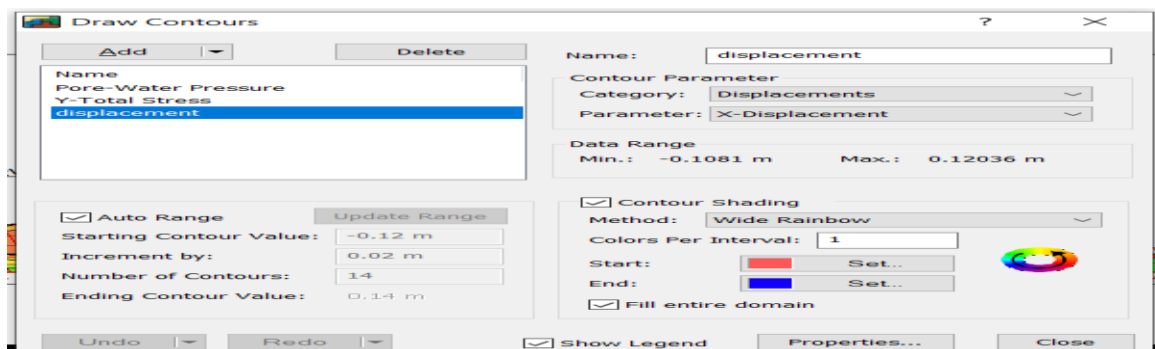


Figure 4.16 Horizontal Displacement (End of Construction Condition)

Whereas the maximum vertical displacements of 0.68 m occurs at the crest of the dam. The change in the maximum vertical settlement and downstream horizontal displacement at the end of dam construction with the number of simulation contours and graphs are shown in Figure 4.17 below.

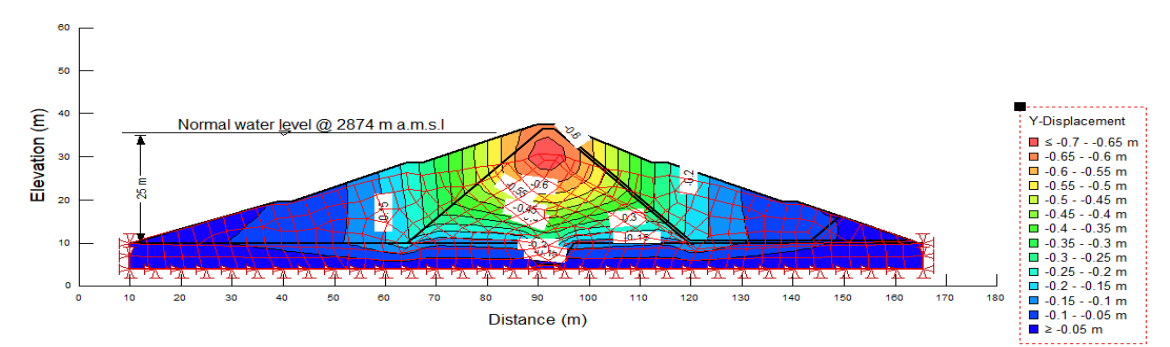


Figure 4.17 Maximum Vertical Settlement (End of Construction Condition)

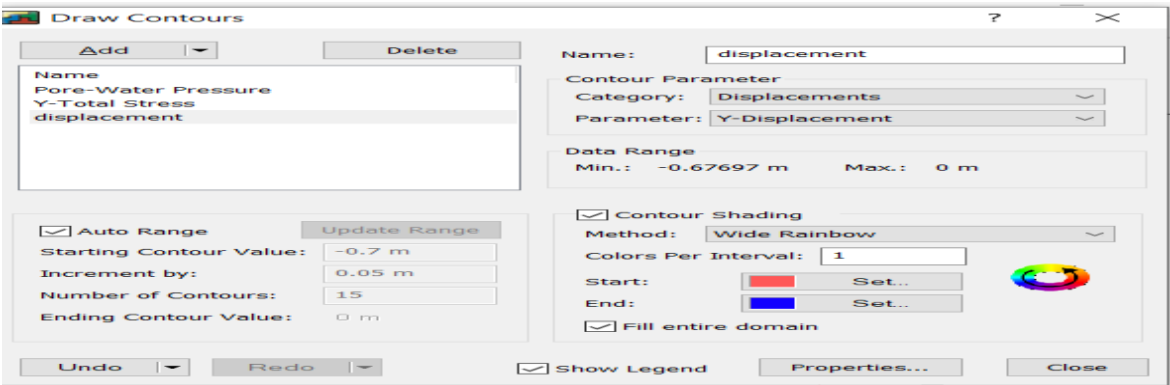


Figure 4.18 Horizontal Displacement (End of Construction Condition)

As we observe from Figure 4.18 Maximum deformation of the dam body is 0.68 m and the dam height is fixed considering settlement allowance of 2%, = 0.54 m, and road fills work over the dam crest with additional recommended fill depth of 0.5 m; totally 1.04 m, therefore the dam height is safe against deformation.

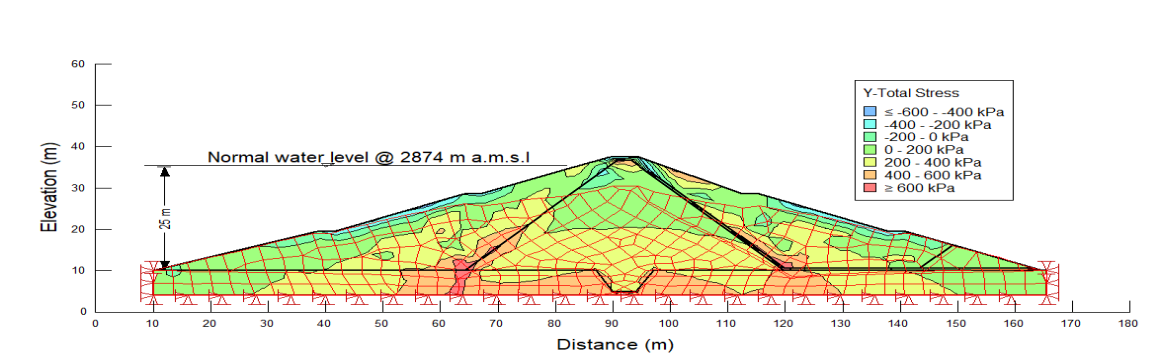


Figure 4.19 Maximum Stress at Dam Foundation (End of Construction Condition)

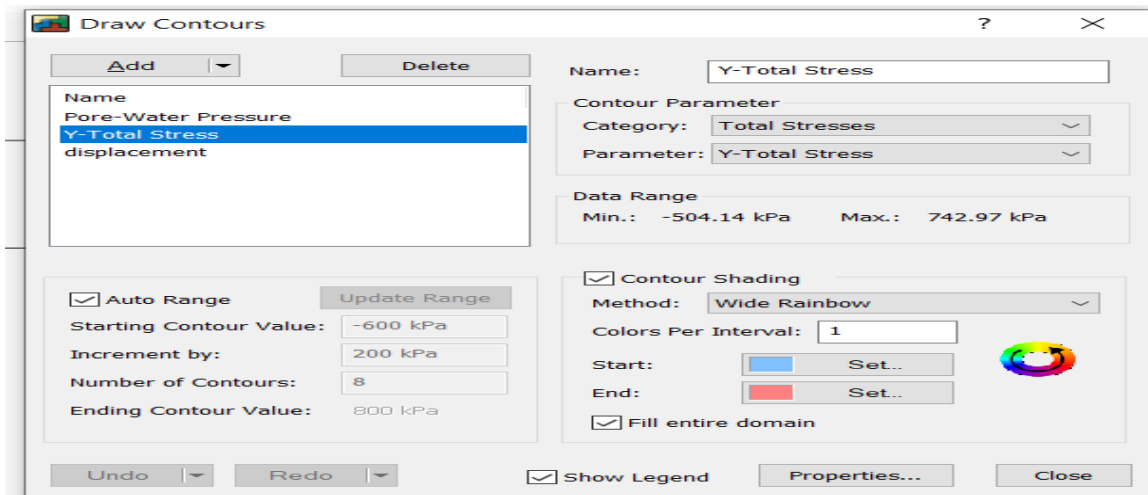


Figure 4.20 Y- Total Stress at Dam Foundation (End of Construction Condition)

As shown in Figure 4.20 The maximum stress developed by the dam and reservoir water on the dam foundation is 742.97 K pa from SIGMA/W analysis, but as per the geotechnical report information the General foundation of the dam lies on basaltic rock and the bearing capacity of the foundation is greater than the maximum stress and, this implies that the foundation is safe against total vertical stresses.

The analysis results found from SIGMA/W applications is shown in Table 4.7 below.

Table 4.7 Deformation Analysis Made with SIGMA/W

Loading Condition (m)	MWL Condition	
	Vertical (m)	Horizontal (m)
SIGMA/W	0.68	0.12

The maximum vertical settlement measured by a total station at the dam crest was 0.71 m at the river bed location this is greater than the simulated result by 0.03 m.

Hunter and Fell (2003) Determined that a significant portion of the post-construction lateral and longitudinal displacement occurs during first filling, particularly for embankments with thin to medium-width central cores and permeable upstream fill (mainly central core earth and earthen fill dams). Thus, for the further deformation inquiry of Aguat Wuha dam, field data are collected and analyzed based on the field observation at the various external surface of the dam crest using surveying instruments (DGPSs instrument).

4.5.3 Settlement Analysis Under Full Reservoir Condition

The rate at which these deformations occurred depends on the dissipation rate of excess pore pressures and the rate at which steady-state seepage conditions develop. When the water fully fills the reservoir, the upstream horizontal displacements decrease and the downstream displacements increase due to the effect of the water load and the pore water pressure (Chen, Qun Zou, Yu Hua Tang, Min He, and Chang Rong 2014). A maximum vertical settlement of 0.0115 m occurs near the dam crest.

Chugh (2001) mentioned that, after the first filling the reservoir may cause the crest to move upstream while at the same time the lower portion of the impervious core defect downstream. This probe highly conceded with this study simulated results as shown in Figure (4.23). Differential settlements between the core and its abutment zone happen in the case of zonal dams because the two components are dissimilar in stiffness. The resulting phenomena of bending is brought on by this element. Internal erosion on the dam core results from cracks that may appear during the first impounding due to differential settlements between the core and shell that start out as subsurface fractures (Zhang and Xu 2009).

According to the various agencies deformation safety standards, the horizontal displacement was 0.284% of the dam height. Based on the (Hunter and Fell 2003) recommendation this settlement value is slightly greater than 0.2%. Thus, during rapid drawdown, the impervious core material is not furtherly safe against the inside cracking and the probability of hydraulic fracturing becomes critical when the reservoir reaches its top level quickly. The vertical settlement occurred under 1% of the embankment height. From this, the rate of vertical settlement is found in the locality of the designed free board. So finally, this might be concluded that the dam couldn't be affected by the overtopping problem through the impact of the current vertical displacement.

The vertical and horizontal displacement at the full reservoir impoundment and under the action of steady state condition with the number of simulation contours and graphs are shown in the Figures 4.21- 4.24 below.

Settlement Analysis Under Full Reservoir Condition

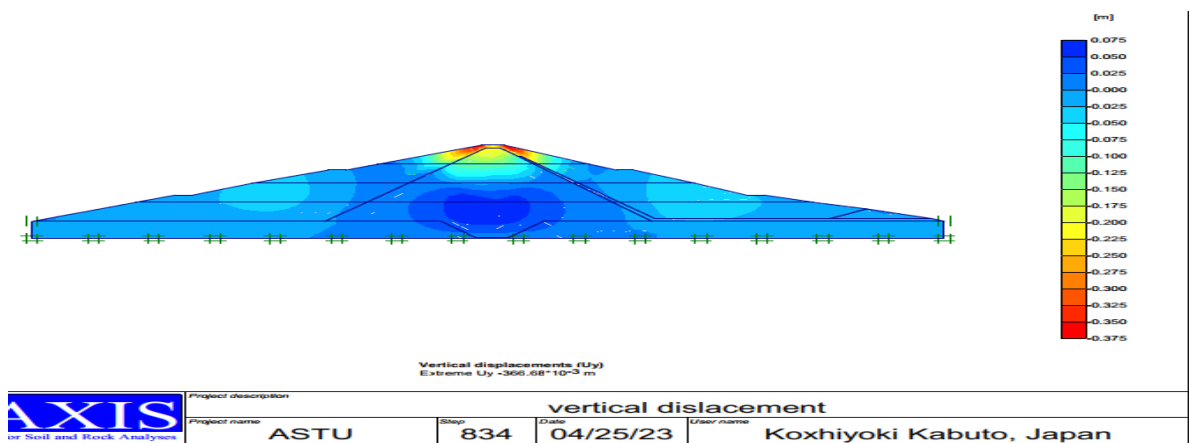


Figure 4.21 Vertical Displacement ($U_y = 306.68 \times 10^{-3}$ m) in the Dam Body and Foundation (Steady State Condition)

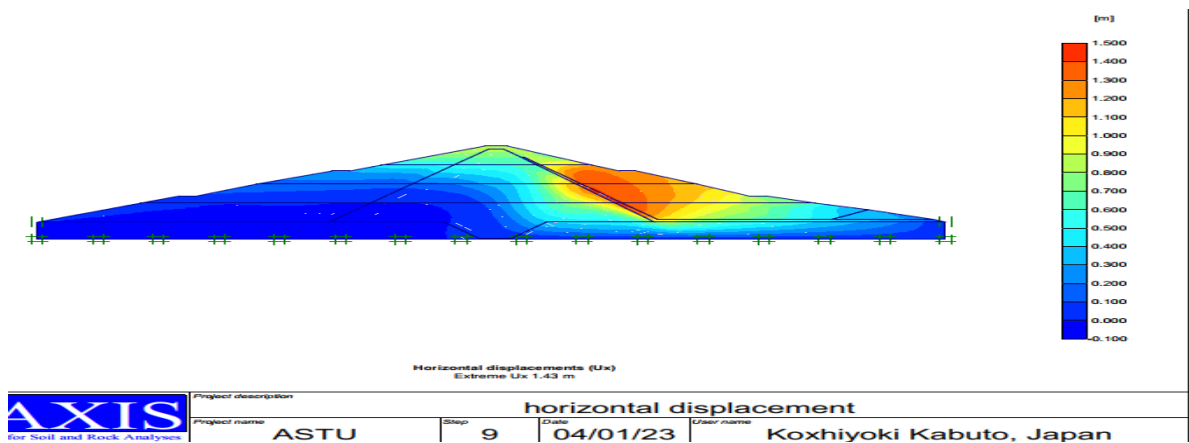


Figure 4.22 Horizontal Displacement ($U_x = 1.43 \times 10^{-1}$ m) in the Dam Body and Foundation (Steady State Condition)

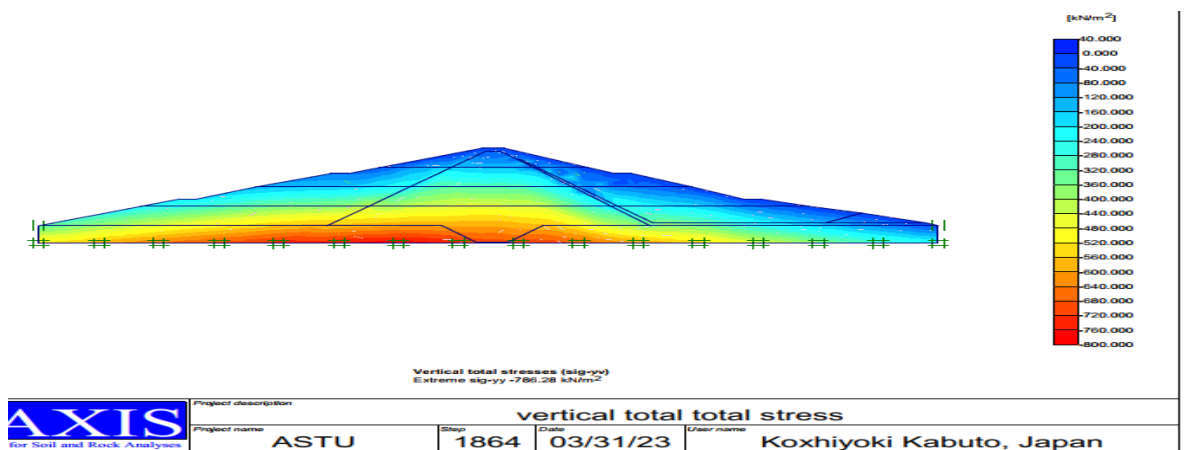


Figure 4.23 Y- Total Stress = 786.28 KN/m² in the Dam Body and Foundation

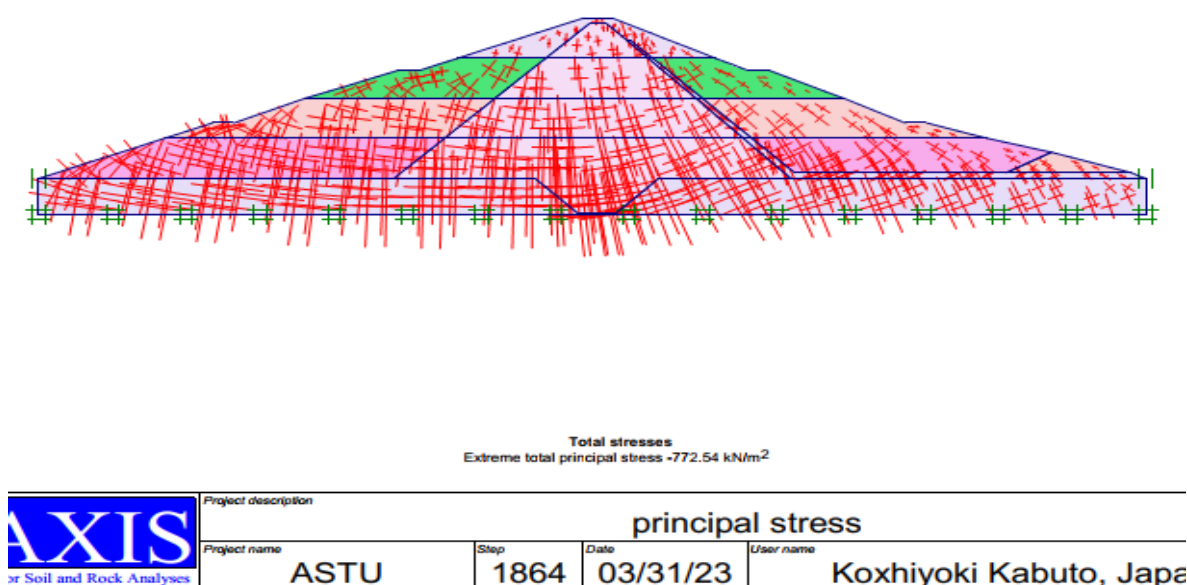


Figure 4.24 Maximum Total Stress = 772.54 KN/m² in the Dam Body and Foundation

4.5.4 Deformation Analysis Using Plaxis V8.2

This application's deformation analysis closely matches the methods used in actual embankment construction. In the following figures for the maximal cross-section, the size and distribution of the vertical and horizontal displacements under various situations are shown.

The total displacements at end construction were 488.91×10^{-3} m vertical and 446.89×10^{-3} m horizontal, respectively. As shown in Figures 4.25 to 4.28 below and Appendices A5, A6, A7, A8, and A9, the maximum horizontal settlement is near the left side of the embankment, and the maximum vertical displacement is at the dam's crest. The outcomes shown in both instances as vertical settlement are consistent with USBR's advice for rock fill dams.

In Figure 4.21- 4.24 is possible to see the deformation when the water fills the reservoir, the upstream horizontal displacements decrease and the downstream displacements increase due to the effect of the water load and the pore water pressure (Chen, Qun Zou, Yu HuaTang, Min He, and Chang Rong 2014). In this case, the simulated maximum vertical displacement occurred at the crest of the dam at 0.30668 m and the maximum horizontal displacement are 0.143 m is created on the downstream side of the dam.

Settlement Analysis by PLAXIS at End of Construction

4.5.5 End of Construction

At the end of construction, significant pore pressure development is anticipated in the foundation and embankment body, either as a result of compaction water or rainfall. At the end of the construction, in this situation, the stability of the upstream and downstream slope is crucial.

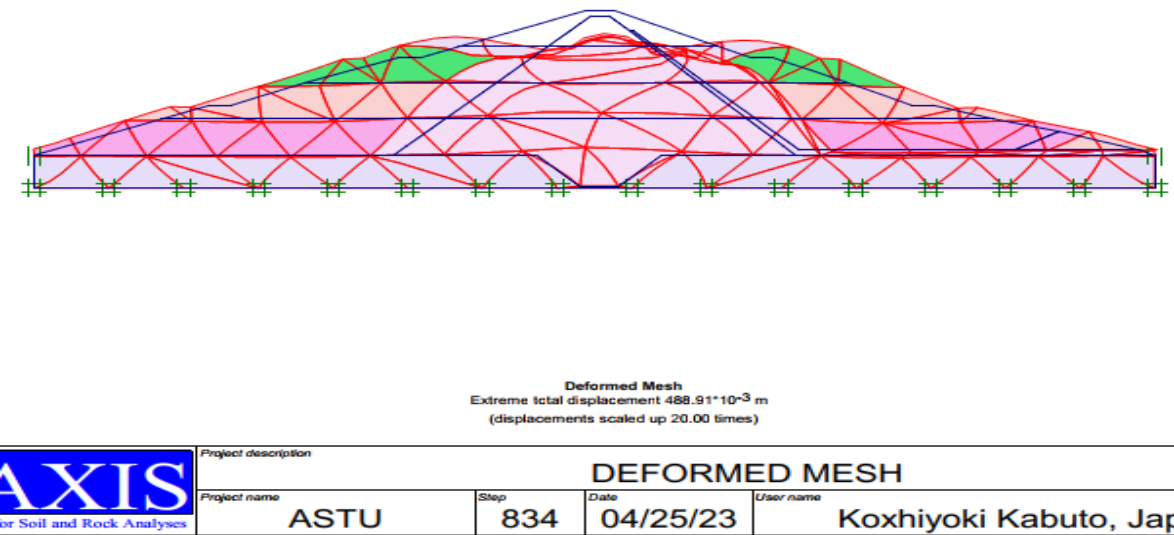


Figure 4.25 Vertical Displacement = 488.91×10^{-3} m

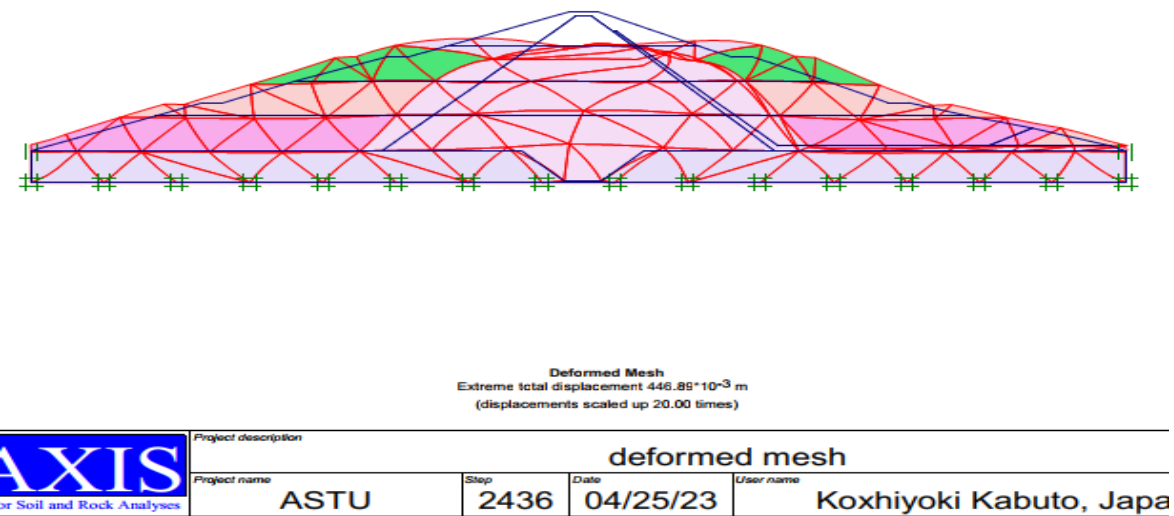


Figure 4.26 Horizontal Displacements = 446.89×10^{-3} m

4.5.6 Steady-State Seepage

When the reservoir is full of water and some steady state seepage into the earth embankment dam is created, this was another essential condition that was investigated. The phreatic surface within the embankment has been established and is essential for the downstream slope for long-term operation.

4.5.7 Rapid Draw Down

This situation arises when the reservoir's water level drops rapidly nevertheless the soil on the upstream slope doesn't drain at the same time. In this case, the pressure balance between the pore water and the water behind the dam would be lost as a result of the removal of the water pressure. Due to the materials on the slope being saturated in this case, the water seepage will shortly be shifted to the upstream side. This will provide a critical situation for the stability of the upstream slope (Carlo 2015).

It is assumed that the phreatic line of the dam body does not drop from the normal pool level. For analysis purposes, it was assumed that draw down is very fast, and no drainage occurs in materials with low permeability.

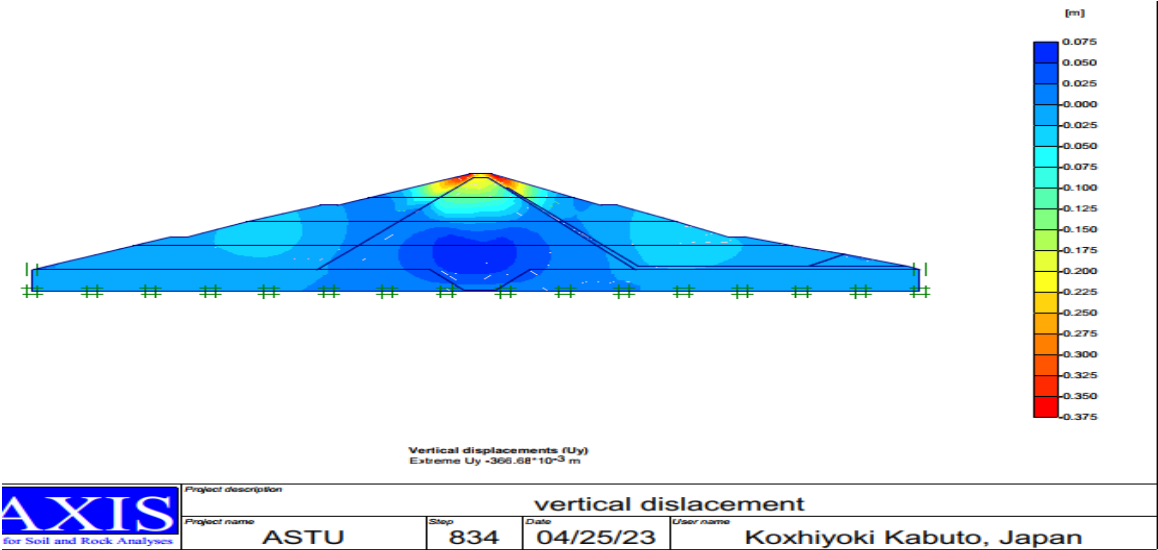


Figure 4.27 Plot Vertical Displacements 366.68×10^{-3} (Arrows)-(Phase:7,Plastic Calculation)

In this analysis, the pore water pressure distribution was also simulated as expected with a minimum at the top and maximum at the bottom proportionally with the depth of water increment. It is particularly higher below the reservoir bed.

The magnitudes of pore water pressure at Full reservoir level (2875.25 m) and bottom of boundary condition or Normal reservoir level (2874 m) a.m.s.l are zero and 303.00 KN/m², respectively. In the figures shown below the pore water pressure distribution and groundwater head are displayed.

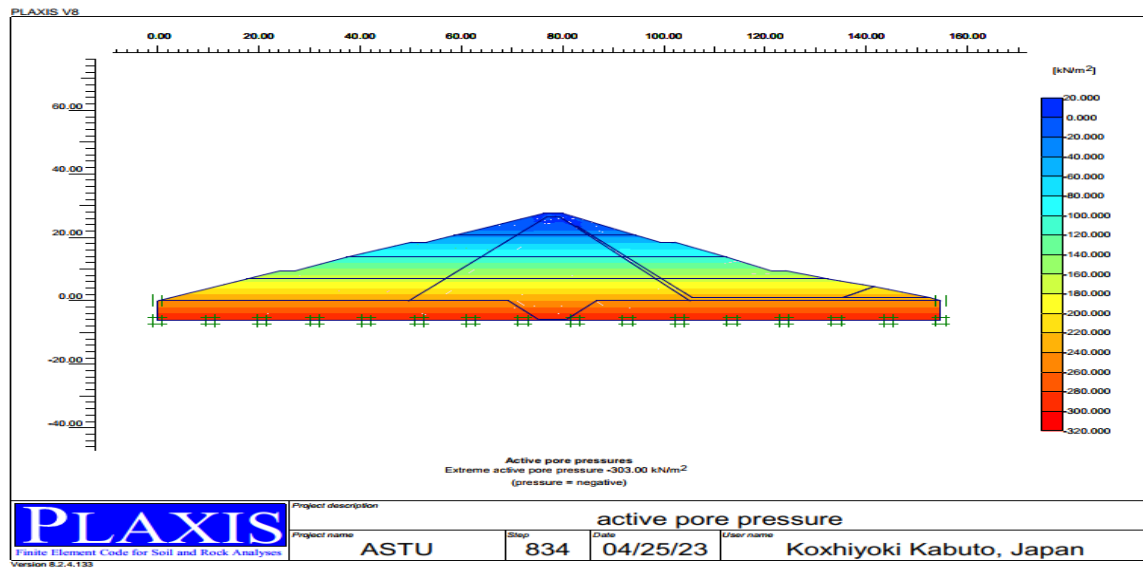


Figure 4.28 Active Pore Pressure = 303 KN/m² (Shading) (Phase:-10 Plastic Calculation)

The analytical findings from these two Sigma/w and Plaxis applications appear to be very similar. Some of them are shown in the following Table.

Table 4.8 Comparison of Deformation Analysis Made with SIGMA/W & Plaxis

Loading condition	SIGMA/W		Plaxis		Vertical settlement using Total station Reading (m)
	Vertical (m)	Horizontal (m)	Vertical (m)	Horizontal (m)	
End of Construc- tion	0.12	0.108	0.48891	0.44689	0.71
MWL Condition	0.68	0.12	0.30668	0.143	

The values shown in Table (4.8) above are calculated from the actual field measurements using levelling equipment is higher than others. In this task, precised levelling was run following the proper surveying chainage route from the referenced benchmark. These surficial elevation difference on the top of the dam are calculated based on the reference point which is found on the top surface of spillway wall bank. During construction of any

dam, the top crest of the dams complete as level and properly drained surface. From these points of view, it can be concluded that these values indicates the surficial deformation or depressions resulted of a dam is from the various settlement patterns. From these obtained measurements of the crest of the dam, the greatly depressed part is found at 210 m distance from the spillway wall bank. It is logically important to recognize that deformation of embankment dams may result in aesthetically unacceptable surficial appearance and this excessive deformation can result in reduction or loss of freeboard, internal and/or external cracks. These consequences of settlement and deformation can lead to dam failure. The crest surficial deformations or depressions as shown in the above figure are important observations but are not always associated with an internal erosion potential failure mode. Localized settlement of limited loose zones in the embankment could result from sinkholes or depressions, slumps, cracks, and formations of cavities. Up on the current field observations, measurements and result findings of the crest surficial deformation of the case study; it can be concluded that the dam might be safer from a dam breach by overtopping, and reservoir wave actions.

Figure 4.29 below shows the PLAXIS 2D finite element model for the largest part of the dam and soil profile, including bed rock. The model had 3956 plane-strain elements, 4189 stress points, and 3287 node points. The base and vertical sides of the model were taken into consideration, along with standard rigidity characteristics.

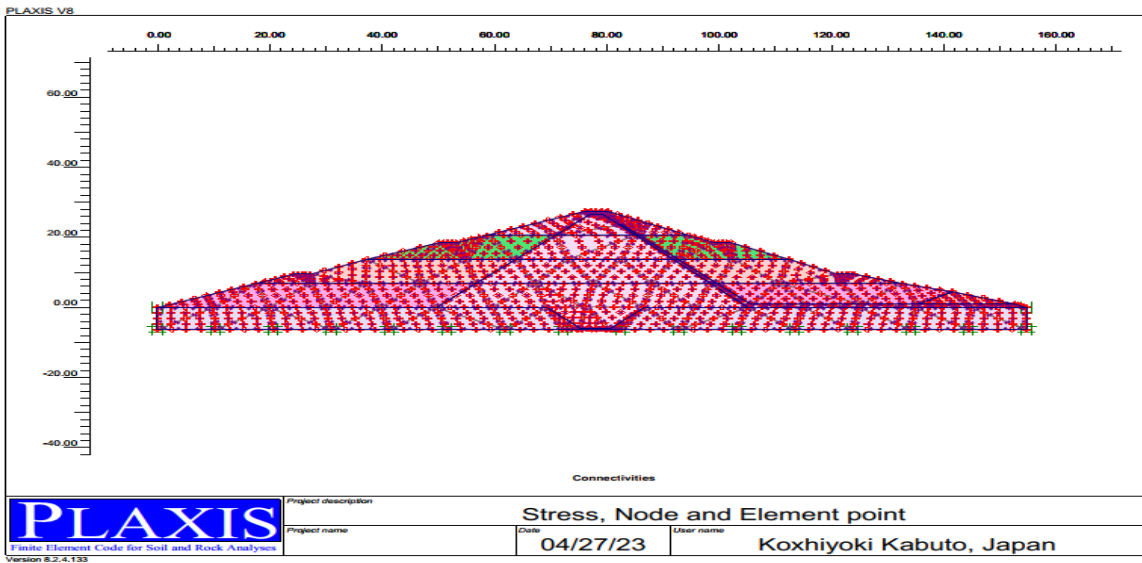


Figure 4.29 Finite Element Model of the Embankment Dam

The plastic points are the stress points in a plastic state, displayed in a plot of the undeformed geometry. A red open square indicates that the stress lies on the surface of the Coulomb failure envelope. A white solid square indicates that the tension cutoff criterion was applied. The Mohr-Coulomb plastic points were particularly useful to check whether the size of the mesh is sufficient. If the zone of coulomb plasticity reaches a mesh boundary then this suggests that the size boundary is too small and the calculation is recommended with a larger model (Plaxis 2010). Therefore, the boundary used for the analyses was sufficient enough.

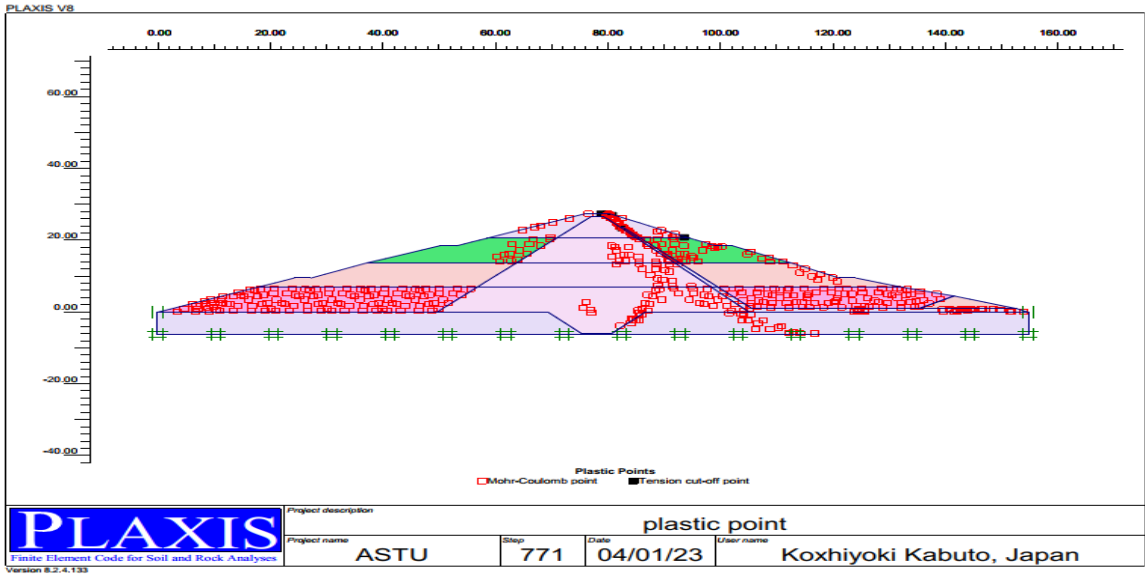


Figure 4.30 Plastic Points during Normal Pool Level

5 CONCLUSIONS AND RECOMMENDATIONS

This study, is done based on the constructed Aguat Wuha micro earthen dam which has a problem of excessive seepage. Static slope stability analysis was performed using the limit equilibrium SLOPE/W software based on the Morgenstern-Price and plaxis 2D model. For the undertaking, mathematical modeling utilizing a professional version of the popular geotechnical software Geo Studio 2012, Plaxis 8.2, and practical field measurements have been performed. These results and discussions of the analysis are summarized below. The whole dam body, including the 6 m foundation depth and the 11 m bed rock, was examined. The studies involved seepage, slope stability, and stress deformation.

5.1 Conclusions

- Seepage analysis was carried out using Limit equilibrium analysis of SEEP/W and Finite equilibrium analysis of the Plaxis method based on the two important impoundment conditions (Normal reservoir condition at an elevation of 2874m a.m.s.l, and Reservoir (water) level at an elevation of 2868m a.m.s.l.) to locate free surface seepage line, determine the pore water pressure distribution, and quantify the seepage through the dam. From both analyses of limit equilibrium and finite equilibrium the result simulated that the average flow rate of leakage for the entire length of the dam at the Normal reservoir water level of 2874m a.m.s.l is $1.17 \times 10^{-5} \text{ m}^3/\text{s}$ and $4.987 \times 10^{-3} \text{ m}^3/\text{s}$ respectively and at reservoir level of 2868m a.m.s.l is $5.5 \times 10^{-6} \text{ m}^3/\text{s}$ and $3.576 \times 10^{-3} \text{ m}^3/\text{s}$. But, the actual measured seepage (rectangular lined masonry canal using manning equation) at the downstream toe of the dam at 2874m a.m.s.l and at 2868 m a.m.s.l was $7.89 \times 10^{-3} \text{ m}^3/\text{s}$ and $6.23 \times 10^{-3} \text{ m}^3/\text{s}$ respectively. Where as for calliper container method at down stream toe of the dam at 2874m a.m.s.l and 2868m a.m.s.l was $9.4 \times 10^{-3} \text{ m}^3/\text{s}$ and $8.1 \times 10^{-3} \text{ m}^3/\text{s}$ respectively. This implies that the measured value is much more than the allowable seepage value and there is visible material transport through the dam.

These results and discussions of the analysis are summarized below:-

- During the rapid drawdown, the stabilizing effect of the water on the upstream face is lost, but the pore-water pressures within the embankment remain high. As a result, the stability of the upstream face of the dam can be much reduced.
- Stability evaluations by using the pore pressure distribution from seepage analyses indicate that the downstream of the dam may have critical conditions in wet seasons.

- It may be improper fill material utilization during construction and insufficient provision of filter material.
- According to a USACE, recommendation the slope stability and stress deformation are safe, but the probability of failures of the dam based on seepage discharge will be likely to happen because the seepage water originates from the dam body contains transport material (there is material transported along the flow path) and this indicates that there is scouring problem.

The main suggestible reasons for the difference in the seepage flow rates are:-

- Due to leaks from the dam's body's contacts with its base and abutments. because of the lack of a provision for grouting operations for the treatment of seepage and degradation of the surface on the dam foundation and abutment edges.
- The fill section of the dam contains the alignment of the outlet structure.
- The contribution of spring seepage that occurred from the dam's right and left abutments. The majority of the bedrock is made up of shale-marl-limestone, which is susceptible to solutions, water leaks, and may be adjacent to the downstream toe drain. It can be due to insufficient filter material provision and incorrect fill material use during construction.
- Based on SIGMA/W model the downstream maximum vertical settlement of 0.68 m and maximum horizontal displacement of 0.12 m full reservoir condition is created near the crest of the dam..
- The vertical direction displacement analysis results that were found from SIGMA/W model should be considered in free board calculation of the dam.
- All results of the settlement indicates that the impervious core material is safe against the inside cracking due to the horizontal displacement and the dam could not be affected by the overtopping problem through the impact of current vertical settlements.
- The maximum stress developed by the reservoir and dam water on the dam's foundation from SIGMA/W is 742.97K pa and from Plaxis 2D is 786.28 Kpa analysis, but as per the geotechnical report information the General foundation of the dam lies on basaltic rock and the bearing capacity of the foundation is greater than the maximum stress and, this implies that the foundation is safe against deformation.
- Even though, model analyses result showed that the dam safety is good from seepage, Slope stability analysis for all loading conditions and deformations,
- From a seepage point of view the field measured data and field visual inspection of the actual recorded data is much more than the allowable seepage and this shows that the dam

is endangered unless immediate remedial measures taken. This study is used to take remedial measures before the dam fail and early warn the downstream dwellers before the dam breach.

The advantage of stress-strain modeling using finite element analysis is that it gives the most accurate picture of what happens in the slope during different loading conditions. A calculated F.S value greater than 1.3 represents the slope being stable under the given conditions (resisting forces > driving forces), and a F.S value less than 1.3 represents that the slope is unstable (failing); that is the driving forces out way the resisting forces. As the result of the According to FEM research, the safety factor of the dam varies depending on the model and the loading conditions. Safety factors simply cannot be used for evaluating a structure's stability. A value of factor of safety greater than 1.3 indicates that capacity exceeds demand and that the slope will be stable because the FEM is a good approximation analysis method for maintaining a force of equilibrium, compatibility equation, and constitutive equations and simulates actual slope failure without any assumptions.

- The analysis of the dam was carried out in four main cases in both limit equilibrium and finite equilibrium models. Accordingly, the safety of the LEM analysis for case one, case two, case three and case four were found to be 1.5, 1.2, 1.2, and 2 and accordingly the safety of the FEM analysis for all cases are 1.281, 1.156, 1.187 and 1.965 respectively. Since both factor of safety of geo studio and three factor of safety of Plaxis are less than the minimum factor of safety which is 1.3 on the required loading condition so, the dam was of failures due to the above reasons.
- The safety factor of the dam obtained by SLOPE/W is 2.00, when the water level in the reservoir drops quickly and the earth materials of the upstream slope do drain simultaneously (case four), and 1.965 get by the Plaxis Model, respectively which is greater than the minimum requirement (1.3).
- The result of Static Stress-Deformation was analyzed using the finite element method model (SIGMA/W) and Plaxis 8.2 in the two cases; during the empty and full reservoir conditions. The result shows that in both models relatively the same value because the two models consider a finite element analysis method. In practical measurement result rate of vertical settlement is 0.71 m this implies that the dam could be affected by an overtopping problem through the impact of current vertical displacement and which is the reason for dam failures.

- Finally, the study's findings show that quick, easy, and accurate results can be obtained using LE slope stability analysis techniques. Such techniques can be applied confidently in routine slope design for slopes with straight forward geometry built on undrained clays.
- The FE approach is a sophisticated, trustworthy, and valuable analysis methodology for modeling design instances with complex slope geometry, diverse soil behavior, and various loading patterns. FEM is used as an alternative to LEM because it presents several advantages that are no assumptions are needed concerning the failure surface, no assumptions on inter-slice side forces are needed, since there is no concept of slices and soil behavior can be modeled in terms of stresses/strains. The finite element method of slope stability is a vital method of analysis that is now available for engineering practice. The use of the finite element method yields more detailed information on the stress state in the soil than is available from conventional limit equilibrium methods. This information can assist an engineer in the design of slopes and adjust their material properties.

5.2 Recommendations and Possible Remedial Measures

5.2.1 Recommendations

The following suggestions and corrective actions are given in light of the findings:-

- The design should be reviewed with multidisciplinary expertise before conducting construction.
- Dam safety instruments should be incorporated in the design for monitoring and surveillance of the safety of the dam to manage the risk associated with operation and maintenance.
- Dam breach analysis should be part of the design document.
- The dam engineer and Geotechnical engineer should work closely to fill the information gap between the dam report and the geotechnical report and revise the general and cut of the foundation after the dam stress is simulated on the Geological cross-section of the dam.
- The maximum stress developed by the dam and reservoir water on the dam foundation from SIGMA/W and Plaxis 2D analysis; should be less than the bearing capacity of the foundation to be safe the dam.
- The vertical direction displacement analysis results that were found from the SIGMA/W

model should be compared with the settlement allowance added the in free-board calculation of the dam and should be taken the greater value of the two.

- The irrigation out let should be placed in cut to reduce the leakage around it and a special construction methodology should be followed to join the rigid structure with earthen material.
- To reduce the amount of seepage first the stored water should be vanished using the irrigation out let and provide an upstream face geo-membrane blanket to reduce the quantity of seepage and to safeguard the dam from failure.
- The original stability analysis of Aguat wuha micro earthen dam was done using LEM and FEM , the factor of safety of LE was found to be greater than 1.3. But, this method gives less approximation results than that of FEM with more assumptions. Therefore, the CO-SAERAR can use the finite element method as a good alternative for the stability analysis of such earthen dams. It is also better to use updated PLAXIS software which works un-expected flow, to see the actual time-dependent flow safety analysis.
- The seepage water is measured once a week and they should also check the quality of the seepage water.

5.2.2 Possible Remedial Measures

Utilizing the information from the design document and actual measurements, the seepage and stability study was performed. There are no problems with the design document's output. The field measurement result, however, indicates that seepage downstream has been severely impacted, and the actual observed value was higher than the seepage estimate. Based on the findings, the key recommendation that follows is made:-

- Impervious blanketing at the upstream part of the dam is tied into the impervious core of the dam to seal the upstream seepage sources. This method aids in reducing undesirable seepage quantities and pressures beneath the embankment.
- Providing appropriate drainage filters at the downstream toe of the dam to reduce uplift pressure and minimize the foundation and embankment weakening.
- Practically obtained measurements indicate a settlement issue in the dam crest level, requiring the removal of the dam's weak point and failure, as well as improving the dam's height and normal pool level after evaluating and correcting the foundation. This is because of the high ground-water stored in the d/s portion of a dam. If these settlement leads when a probable maximum flood will increase the water level and lead to total dam failure by overtopping.

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APPENDIX



Appendix A1 Down Stream Slope of Dam



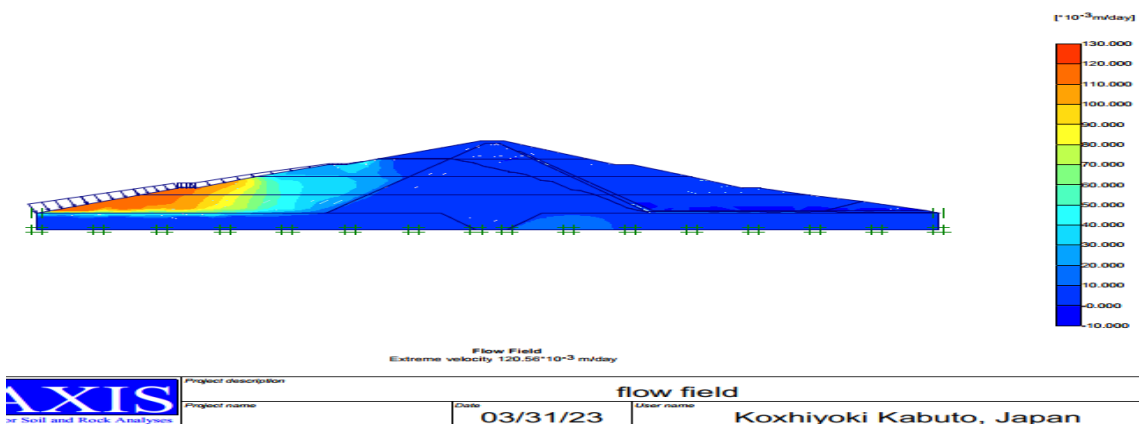
Appendix A2 Aguat Wuha Dam Crest and U/s Water Level Measurement (18/04/2023)



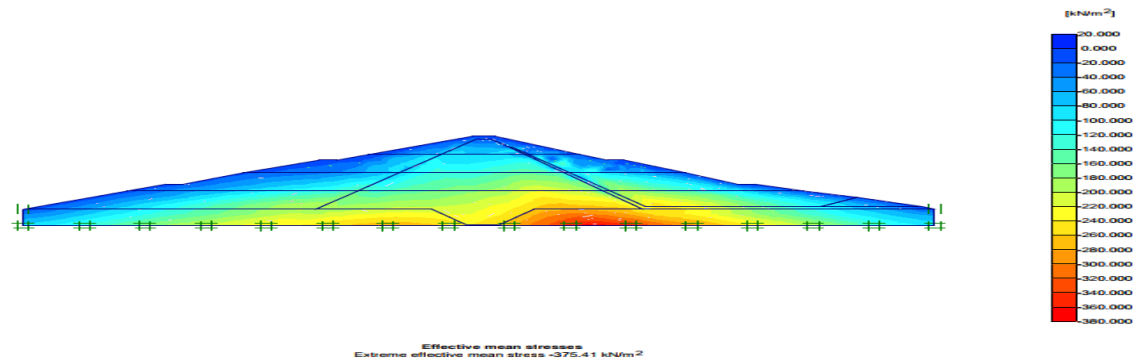
Appendix A3 Aguat Wuha Dam Measurement of Seepage and Location of Energy Dissipator (18/04/2023)



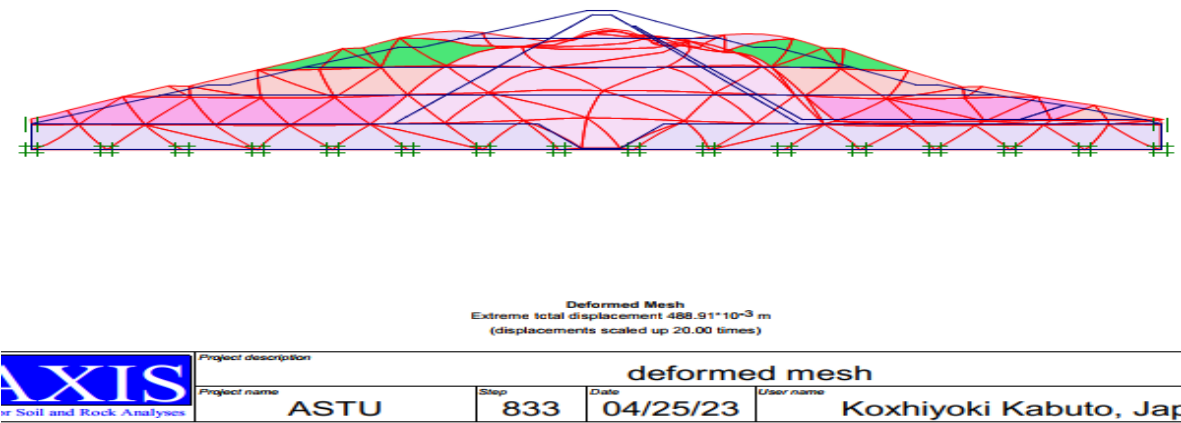
Appendix A4 Photo Taken at Normal Reservoir Water Level (18/04/2023)



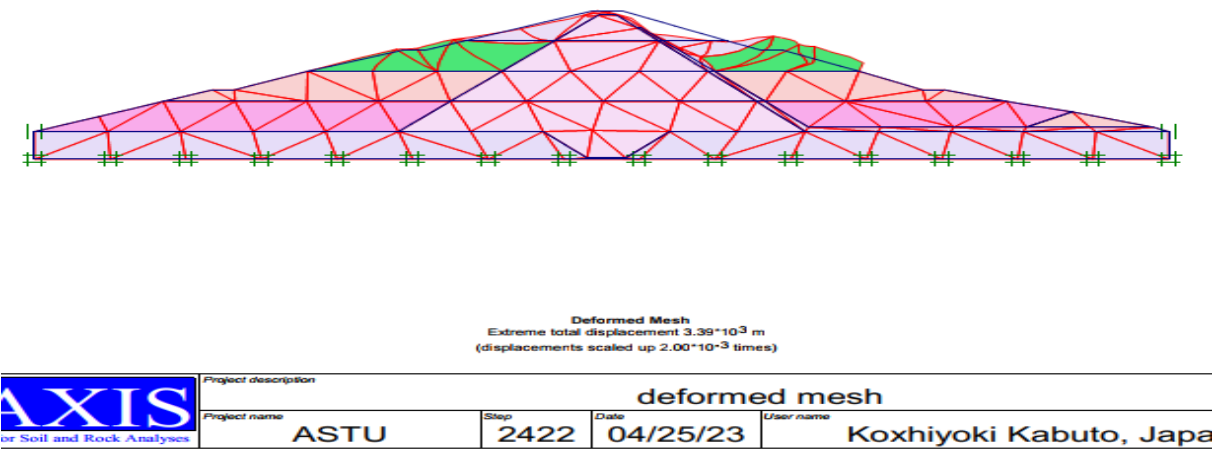
Appendix A5 Aguat Wuha Dam Current NWL Flow Field = $120 \times 10^{-3} \text{ m/day}$



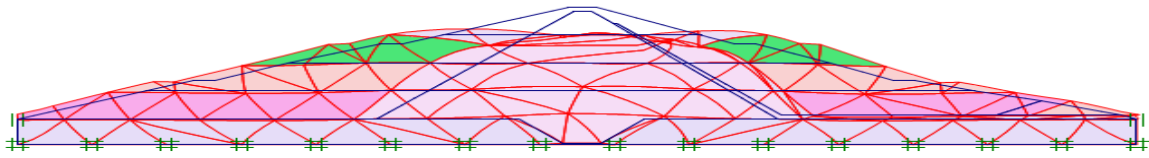
Appendix A6 Aguat Wuha Dam Current NWL Effective Mean Stresses = 375.41KN/m²



Appendix A7 Aguat Wuha Current NWL Total Deformed Mesh 488.91*10⁻³m After phase 1



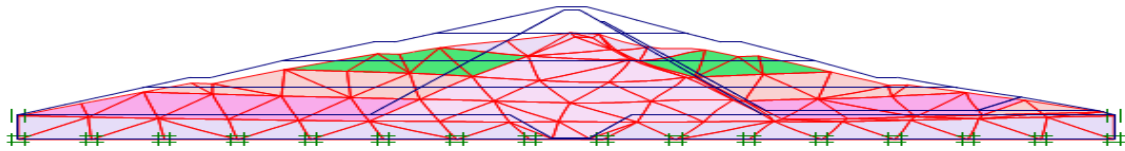
Appendix A8 Aguat Wuha Dam Current NWL Total Deformed Mesh = 0.0033m After Phase 2



Deformed Mesh
Extreme total displacement 455.28*10⁻³ m
(displacements scaled up 20.00 times)

LAXIS ment Code for Soil and Rock Analyses	Project description				deformed mesh	
	Project name	Step	Date	User name		
	ASTU	519	04/25/23	Koxhiyoki Kabuto, Japa		

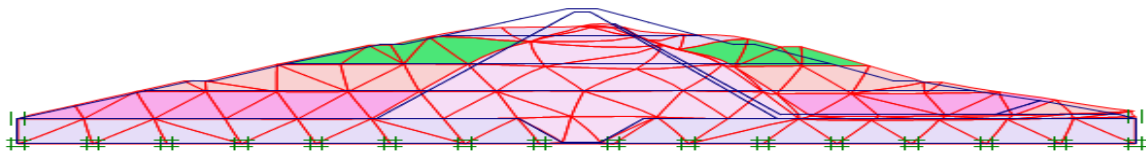
Appendix A9 Aguat Dam Current NWL Total Deformed Mesh = 0.0045 m After Phase 3



Deformed Mesh
Extreme total displacement 3.71 m
(displacements scaled up 2.00 times)

LAXIS or Soil and Rock Analyses	Project description				deformed mesh	
	Project name	Step	Date	User name		
	ASTU	9	04/01/23	Koxhiyoki Kabuto, Japar		

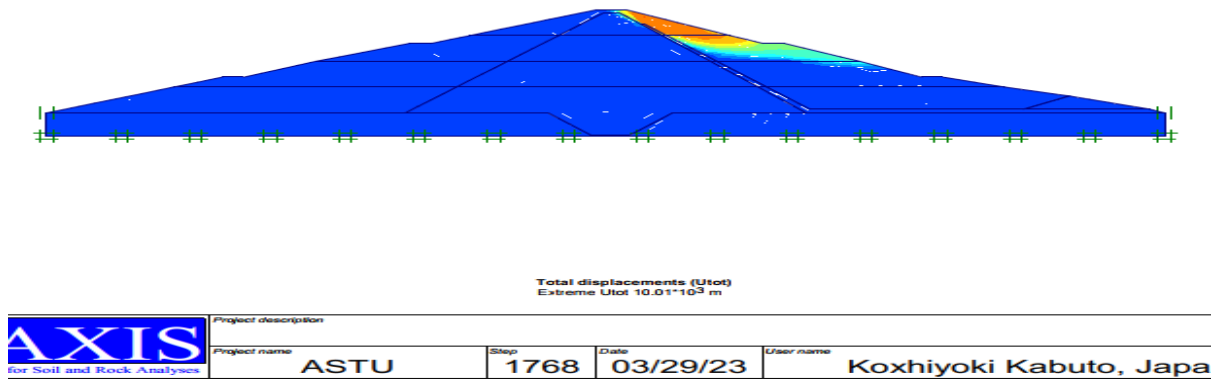
Appendix A10 Aguat Wuha Dam Current NWL Total Deformed Mesh = 3.71 m After phase 4



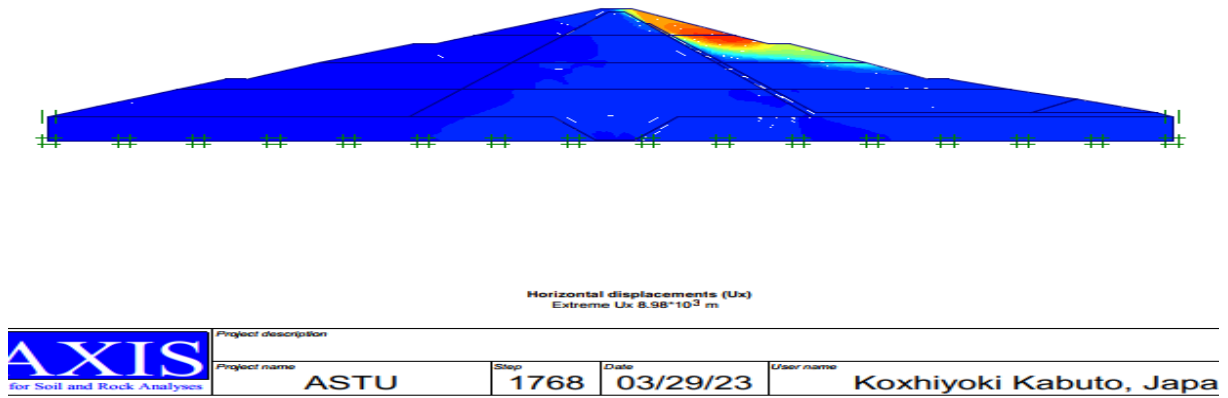
Deformed Mesh
Extreme total displacement 1.36 m
(displacements scaled up 5.00 times)

LAXIS for Soil and Rock Analyses	Project description				deformation	
	Project name	Step	Date	User name		
	ASTU	1259	04/25/23	Koxhiyoki Kabuto, Japa		

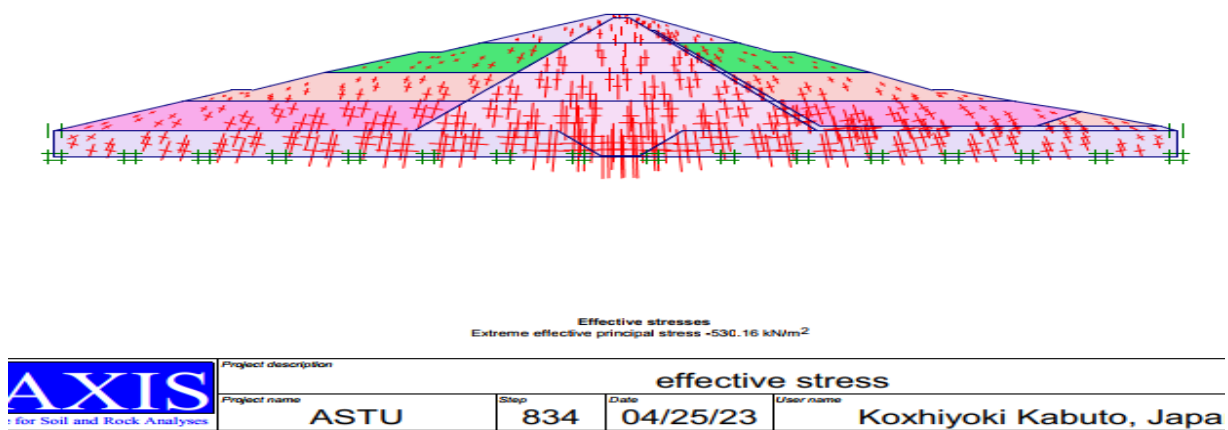
Appendix A11 Aguat Wuha Dam Current NWL Total Deformed Mesh = 1.36 m After Phase 5



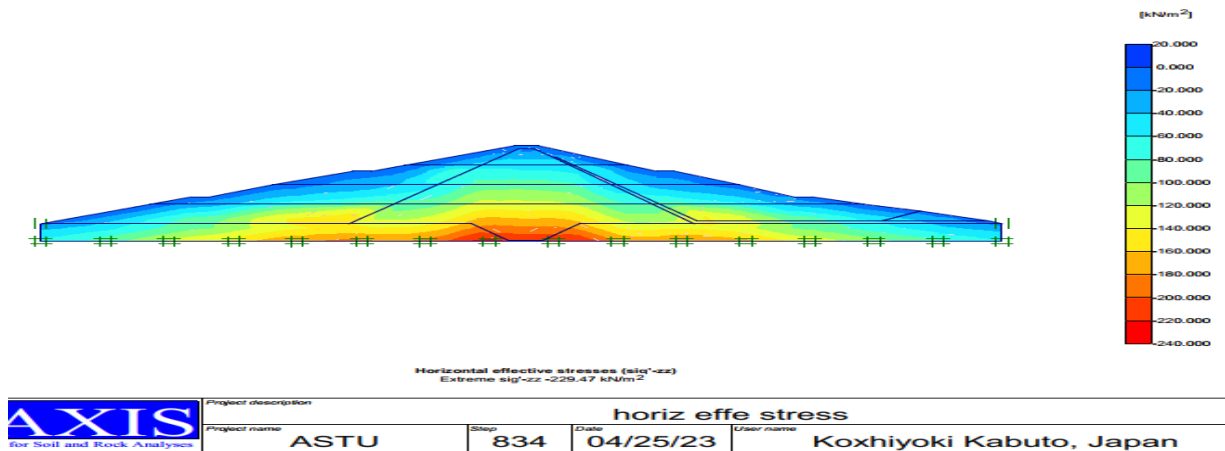
Appendix A12 Plot of Total Displacements = 0.001m (Shadings) @ 2868 a.m.s.l



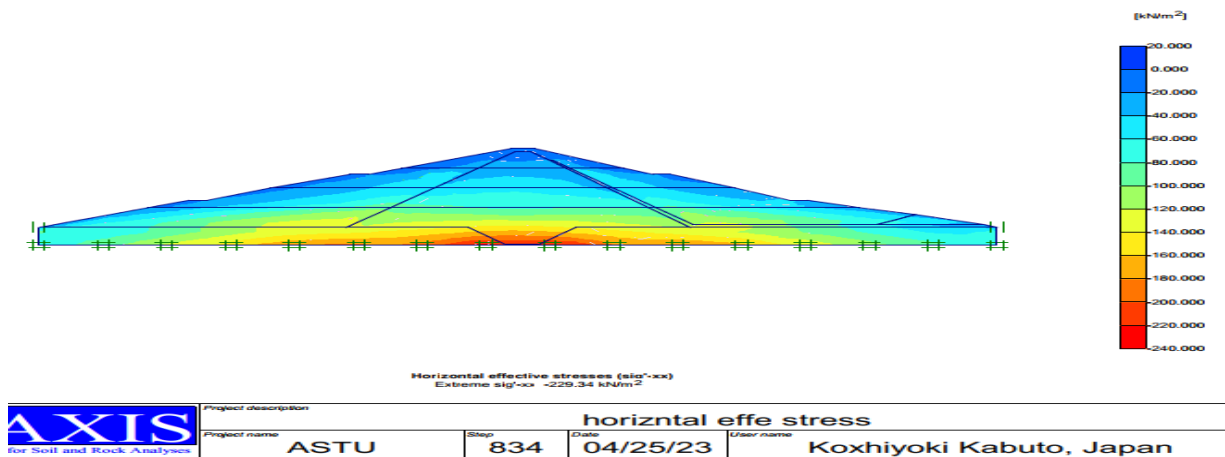
Appendix A13 Plot of Horizontal Displacements = 0.00856 m (Shadings)



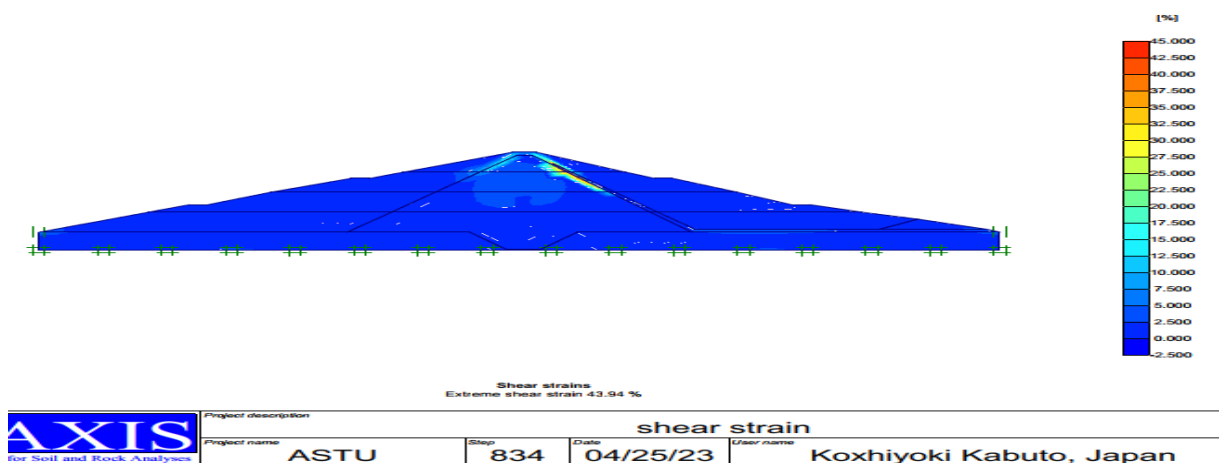
Appendix A14 Effective Stress = 530.16 KN/m² in the Dam Body and Foundation



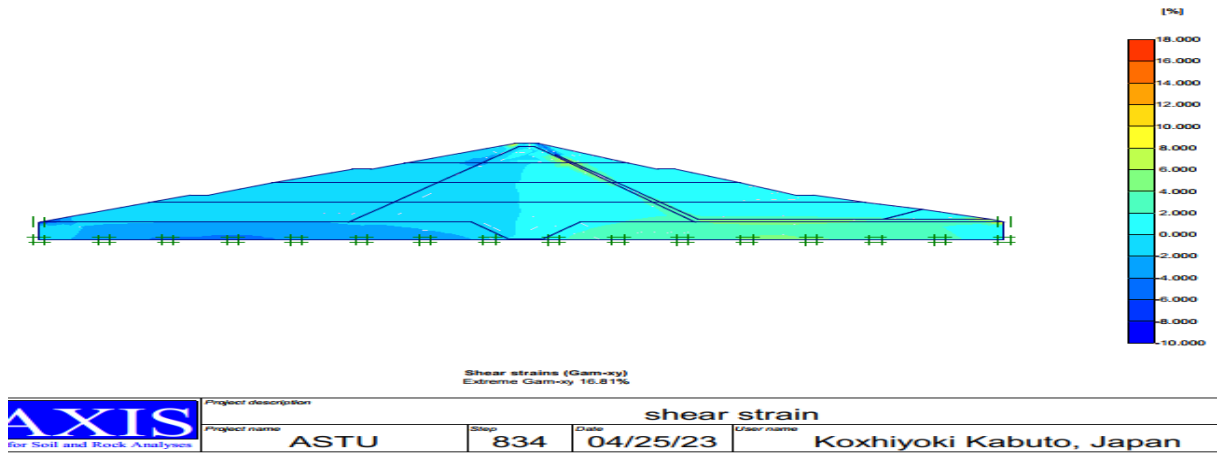
Appendix A15 Horizontal Effective Stress = 229.47 KN/m² in Dam Body and Foundation



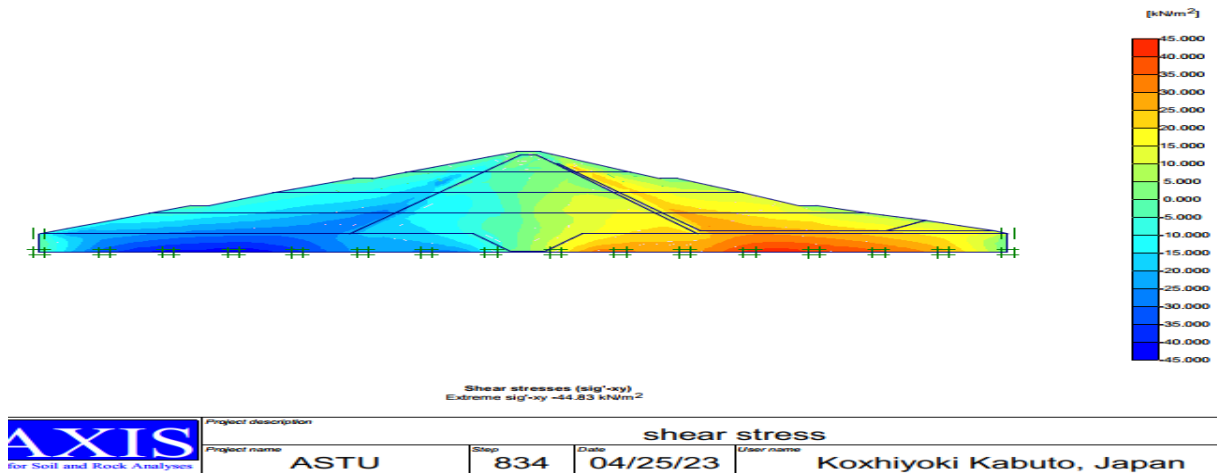
Appendix A16 Horizontal Effective Stress = 229.34 KN/m Dam Body and Foundation



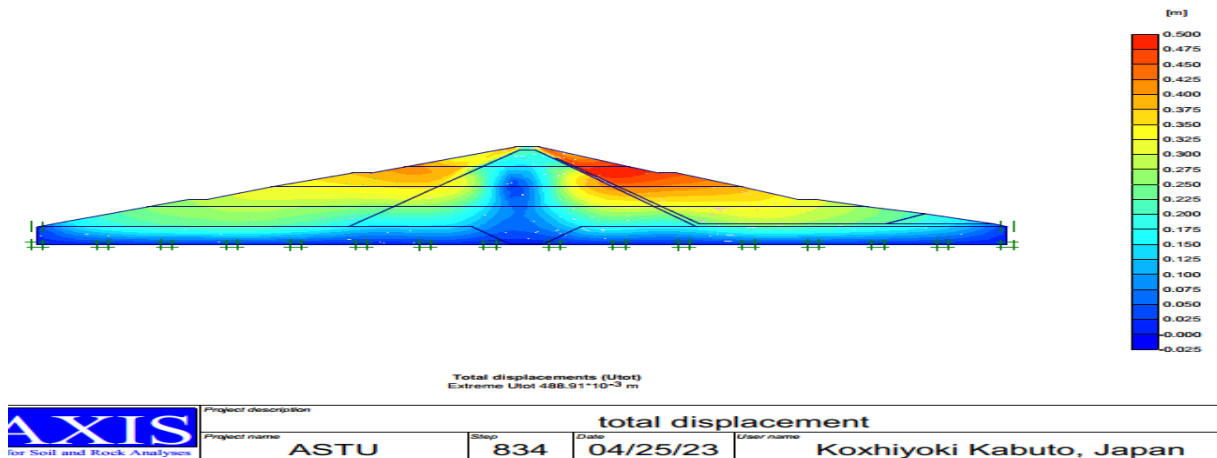
Appendix A17 Shear Strain = 43.94 % in the Dam Body and Foundation at 2874



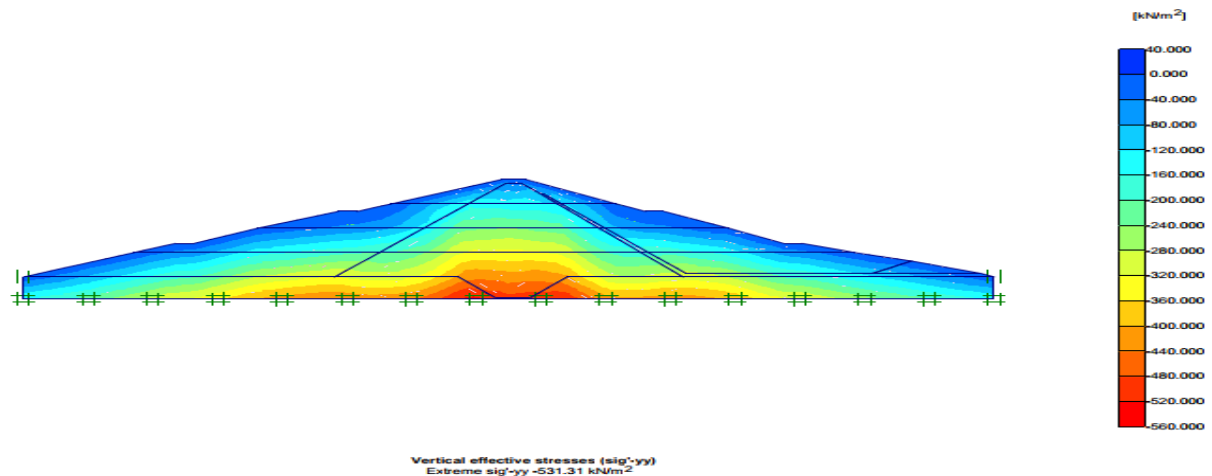
Appendix A18 Shear Straine = 16.81% in the Dam Body and Foundation at 2868



Appendix A19 Shear Stress = 44.83 KN/m² in the Dam Body and Foundation



Appendix A20 Total Displacement (Utot = 488.91*10⁻³m) in Dam Body and Foundation



AXIS for Soil and Rock Analyses	Project description			
	vert effec stress			
Project name	ASTU	Step	834	Date
				04/25/23
User name	Koxhiyoki Kabuto, Japan			

Appendix A21 Vertical Effective Stress in the Dam Body and Foundation

Appendix B1: Data for slope/w

Embankment description	Unit weight (kN/m ³)	Cohe-sion (kPa)	Phi (°)
Core	19	35	16
Fine Filter	18	0	34
Coarse Filter	18	0	35
Shell	21	20	28
Rock toe	21	0	40
Decomposed rock	19	5	34
Foundation rock	Bedrock (Impenetrable)		

Appendix B2: Data for Seep/W

Embankment description	Soil type	Saturated WC	Saturated Ks (m/s)	Estimation method	Grain size data		
					Dia. (mm) at 10% passing	Dia. (mm) at 60% passing	Liquid limit (%)
Core	Silty Clay	0.475	8.4×10^{-9}	Grain size	0.015	0.07	50.0
Fine Filter	Sand	0.35	0.002	Grain size	0.3	1.20	0.0
Coarse Filter	Sand	0.30	0.005	Grain size	3.0	10.0	0.0
Shell	Gravely silt/Silt sand	0.38	1×10^{-6}	Sample Function	--	--	
Rock toe	Crushed rock/Gravel	0.25	0.005	Sample Function	--	--	
Decomposed rock	Silty sand	0.38	5×10^{-6}	Sample Function	--	--	
Foundation rock	Rock/Clay	0.25	1×10^{-10}	Sample Function	--	--	

Appendix B3: The Two borrow areas are almost within the reservoir area. Representative geographic co-ordinates of the borrow areas, together with their given name is presented below.

Borrow Area Name	Co-ordinates	
	E	N
Core-1	486061	1213901
Core-2	486173	1213703
Core-3	486194	1213632

Appendix B4: Calculation Part for Plaxis Software

PLAXIS Finite Element Code for Soil and Rock Analyses Project description: soil and interface data User name : Koxhiyoki Kabuto, Japan Project name : ASTU Output : Soil and Interfaces Info - Mohr-Coulomb Date : 4/25/2023 Step : 0 Page : 1											
ID	NAME	TYPE	ϕ	Γ_{unsatu} (KN/m ³)	Γ_{satur} (KN/m ³)	K _x (m/day)	K _y (m/day)	V (-)	E _{ref}	C _{ref}	R _{inter}
1	No name	Un-drained	30	25	27	0.001	0.001	0.1	29.7	5	1
2	Lesson 1: sand	Un-drained	40	35	40	2	2	0.22	3334.5	3	1
3	Lesson 2: clay	drained	35	20	25	3	3	0.25	2516.8	10	0.5
4	Lesson 3: Peat	Un-drained	20	15	20	0.001	0.001	0.3	3537.1	15	1
5	Lesson 4: Fill	drained	30	20	25	0.003	0.003	0.3	1485.7	5	0.65

6	Lesson 5: Loam	drained	30	25	30	0.02	0.02	0.3	5000	10	1
7	Lesson 6: Deep clay-sand	drained	25	25	30	0.003	0.003	0.26	2463.8	10	0.7
8	Lesson 7: Deep-sand	φdrained	18	25	30	1.5	1.5	0.02	4996.1	5.5	0.7

Appendix B5: Finite Element Code for Soil and Rock Analyses

PLAXIS - Finite Element Code for Soil and Rock Analyses									
Project description : effective stress PLAXIS 8.0									
User name : Koxhiyoki Kabuto, Japan									
Project name : ASTU							Date : 3/31/2023		
Output : Soil element effective stresses							Step : 1864		Page : 3
Cluster	Soil Element	Stress Point	X	Y	σ_{xx}	σ_{yy}	σ_{xy}	σ_{zz}	Status
		[m]	[m]	[kN/m ²]	[kN/m ²]	[kN/m ²]	[kN/m ²]		
	3	25	93.786	-5.505	-299.450	-514.594	145.055	-224.260	Elastic
	Lesson -6	26	1036.742	-0.397	-240.269	-396.993	85.563	-164.981	Elastic

		27	96.242	- 0.397	- 259.57 4	- 410.17 0	100.64 2	- 175.95 2	Elasti c
		28	96.637	- 3.159	- 279.34 5	- 478.64 5	136.18 1	- 207.84 6	Elasti c
		29	99.733	-1.571	- 255.54 3	- 427.14 4	95.311	- 179.01 6	Elasti c
		30	97.401	-1.571	- 256.71 1	- 435.90 7	109.45 2	- 183.88 6	Elasti c
		31	96.792	-4.010	- 297.92 8	- 501.09 4	145.21 3	- 219.42 4	Elasti c
		32	100	-1.955	- 250.61 1	- 431.20 6	96.951	- 180.57 1	Elasti c
		33	101.577	-0.335	- 251.17 9	- 395.02 9	87.286	- 167.29 9	Elasti c
		34	98.560	-0.335	- 257.82 9	- 399.07 0	93.811	- 171.14 8	Elasti c
		35	95.401	-1.955	- 260.22 0	- 453.50 0	126.33 7	- 195.22 0	Elasti c
		36	94.413	-4.010	- 301.11 1	- 497.86 3	146.29 9	- 219.31 9	Elasti c

Appendix B6: Summary of Dam Crest Elevations

Dam Option	Spillway bed level @ control point(masl)	Still water rise (m)* (1)	Freeboard (m)* (2)	settlements allowance (m) (3)	Total depth above spl (m) (1)+(2)+(3)	Dam top level (masl) **
Dam axis	2874	1.245	0.773	0.34	2.35 take 2.5	2876.5

Appendix B7: Design Slopes of the Dam

Slopes of the shell material		U/s side	D/s side
Between the top level of the dam and the level		1:3	1:2.5
Between burms		1:3	1:2.5
Between the burm level and dam foundation level		1:3	1:2.5
height location	from Top to the burm level(m)	9.0	9.0
Height location	From 2 nd burm to Ogl level	9.5	9.5
	- Width (m)	3	3

Appendix B8 Table: Summary of Laboratory Test Results of Over Burden

S/N	Soil Parameters	Values
1	Grain Size Distribution	
1.1	Gravel,%	12.34
1.2	Sand, %	45.80
1.3	Fines, %	41.86
2	Soil Classification	

2.1	D10, mm	0.001
2.2	D60, mm	0.5
2.3	D30, mm	0.02
2.4	Liquid Limit	48
2.5	Plastic Limit	28.11
2.6	Plasticity Index	19.89
2.7	USCS Soil Name	MC
3	Compaction Characteristics	
3.1	MDD, gm/cc	1.46g/cc
3.2	OMC, %	29.39
4	Permeability @ MDD, cm/s	1.88099×10^{-7}

Appendix B9: Summary of Laboratory Test Results of Core Material

Parameters	Units	Embankment Soils
		Core and Cut-off
Unit Weight	KN/M3	
Permeability, K	Cm/s	3.3×10^{-9}
Soil Type/Group		
γ dry	g/cc	1.57
Bulk density		2.01

Liquid limit	%	46.0
Plastic limit	%	27.03
Plastic index	%	18.97
Moisture Content		27.72
D10		0.001
D60		0.002

Appendix B10: Summary of Laboratory Test Results of Shell Material

Parameters	Units	Shell Material Parameters
Unit Weight	KN/M3	
Permeability, K	cm/s	9.2×10^{-7}
Soil Type/Group		
γ Submerged	KN/M3	2.03
γ Bulk	KN/M3	2.03
gravel	%	34.93
Sand	%	52.84
finer	%	12.23
Moisture Content	Dec.	19.41
D10	mm	0.07
D60	mm	8

Size of D15 of filter material for chimney drain

The required size of D15 = 0.45 of filter material considering D85 of the core material is determined using the following criteria

Upper limit

D15 of filter < 5x D85 core material

Where D85 of core material = 0.0015mm (from Laboratory)

Therefore, D15 of filter < 5x0.06mm, 0.06mm is the appropriate core material size to be used in the dam body core component.

✓ D15 of filter < 0.30mm

Lower limit

D15 of filter > 5xD15 of core material

Where D15 of core material = 0.015mm

➤ D15 of filter > 5 x 0.015mm

➤ D15 of filter > 0.075mm

Therefore the required size of D15 of filter material will be in the order of 0.075mm to 0.30mm which means $0.075\text{mm} < \text{D15 filter} < 0.30\text{mm}$.

a/ Determination of Rock toe height

To improve the drainage in the dam body and to minimize the effect of tail water on d/s side of the dam 25% of reservoir head (h) rock toe height is provided.

And therefore, Rock toe height at maximum dam height

$$= 25\% \times h, \quad h = 25$$

$$= 0.25 \times 25.00\text{m}$$

$$= 6.25\text{m}$$

b/ Determination of rock toe slope

The recommended internal rock toe slope of 1:1 and external slope of 3:1 (similar to the d/s slope)

has been selected as shown in the drawing.

Appendix B11: Summary of laboratory results used f geo slope analysis

PARAMETERS	DAM MATERIALS					
	UNIT		SHELL	CORE	FILTER	Rock toe
UNIT WEIGHT	KN/M3		20.35	18.45	19	20.00
ANGLE OF REPOSE	DEGREE		26.5	34.9	0	0
COHESION	KPA		14.78	16	34	40.00

Appendix B12: Input Data for Geo Slope

A	FOR SEEP/W ANALYSIS							
SN	PARAMETERS	VALUES						REMARKS
				SHELL MATE RIAL	CORE MATE RIAL	FILTE R MATE RIAL	ROCK TOE MATE RIAL	
		UNIT						
hyd. water content	saturated water content	m3/m3		0.29	0.5	0.29	0.2	
	soil texture			grave l	clay	sand	gravel	
	D10	Mm						
	D60	Mm						
	Liquid limit	%						
hyd.Per- meability	K at saturation	m/s		9.2E- 007	3.3E- 009	1.3E- 004	2.5e- 007	
	Residual water content	%		0	0.25	0		
Material	conductivity ration			1	0.1	1	1	

Appendix B13: Calculated coordinate points of Aguat wuha micro-earth dam

points	X (m)	Y (m)
Point 1	0	0
Point 2	24.225	9.5
Point 3	27.225	9.5
Point 4	50.125	18.5
Point 5	53.175	18.5
Point 6	76.125	27.5
Point 7	80.125	27.5
Point 8	99.125	18.5
Point 9	102.115	18.5
Point 10	121.105	9.5
Point 11	124.105	9.5
Point 12	154.7	0
Point 13	105.1	0
Point 14	86.6	0
Point 15	80.6	-6
Point 16	75.35	-6
Point 17	69.35	0
Point 18	49.6	0

Point 19	0	-6.3
Point 20	154.7	-6.3
Point 21	76.875	26.5
Point 22	79.375	26.5
Point 23	82.325	23.55
Point 24	82.825	23.55
Point 25	105.6	1
Point 26	135.263	1
Point 27	141.513	4.5
Point 28	152.59	1
Point 29	36.14933	13.007379
Point 30	-14.8805	13.013451
Point 31	69.7	25
Point 32	-14.8805	25
Point 33	-14.8805	5
Point 34	13	5

Appendix B14: Generated line report from SEEP/W analysis at full reservoir level

	Start Point	End Point	Hydraulic Boundary	Length (m)	Angle (°)
Line 1	1	2	reservoir head=25m	26.021	21.4
Line 2	2	3	reservoir head=25m	3	0
Line 3	4	5	reservoir head=25m	3.05	0
Line 4	6	7		4	0
Line 5	7	8	Potential Seepage Face	21.024	-25.3
Line 6	8	9	Potential Seepage Face	2.99	0
Line 7	9	10	Potential Seepage Face	21.015	-25.4
Line 8	10	11	Potential Seepage Face	3	0
Line 9	26	25		29.663	0

Appendix B15 : Generated region report from SEEP/W analysis at full reservoir level

	Material	Points	Area (m ²)
Region 1	shell	1;2;3;29;4;5;6;7;8;9;10;11;27;26;25;24;23;22;21;18	1,227.3
Region 2	core	18;21;22;23;13;14;15;16;17	837.14
Region 3	foundation	1;18;17;16;15;14;13;12;20;19	907.11
Region 4	sand filter	23;13;12;28;26;25;24	71.207
Region 5	Rock Toe	26;28;27	30.322

