EVALUATION OF SEEPAGE AND SLOPE STABILITY ANALYSIS OF EMBANKMENT DAM: A CASE STUDY OF CHEL-CHEL DAM, OROMIA, ETHIOPIA.

BY

JEMAL ALIYI TUFA

A THESIS SUBMITTED TO THE DEPARTMENT OF WATER RESOURCES ENGINEERING

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Evaluation of Seepage and Slope Stability Analysis of Embankment: A Case Study of Chel-Chel Dam, Oromia, Ethiopia.

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Jemal Aliyi Tufa

Advisor:

Dr.Dereje Birhanu (Ph.D)

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APPROVAL SHEET

I hereby certify that the recommendations and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis titled "Evaluation of Seepage and Slope Stability Analysis of Embankment Dam: A Case Study of Chel-chel Dam" by Jemal Aliyi.

Dereje Birhanu (Ph.D)	________________________	____________________
Major Advisor	Signature	Date

DECLARATION

I hereby declare that this Master Thesis entitled "Evaluation of Seepage and Slope Stability Analysis of Embankment: A Case Study of Chel-Chel Dam, Oromia, Ethiopia." is my original work and has not been submitted to any university for a similar purpose. Proper citations duly recognize the references used in this proposal.

Jemal Aliyi	____________	______________
Name of Student	Signature	Date

RECOMMENDATION

I the advisor(s) of this thesis, hereby certify that I have read the revised version of the thesis entitled "Evaluation of Seepage and Slope Stability Analysis of Embankment Dam: A Case Study of Chel-chel Dam." prepared under my guidance by Jemal Aliyi Tufa submitted in partial fulfillment of the requirements for the degree of Mater's of Science in Hydraulic Engineering.

Therefore, I recommend the submission of the revised version of the thesis to the department following the applicable procedures.

Dereje Birhanu (Ph.D)	______________________	__________________
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We, the undersigned, members of the Board of Examiners of the thesis by Jemal Aliyi, have read and evaluated the thesis entitled ‘’Evaluation of Seepage and Slope Stability Analysis of Embankment Dam: A Case Study of Chel-chel Dam.” And examined the candidate during the open defense. This is, therefore, to certify that the thesis is accepted for partial fulfillment of the requirements of the degree of Master of Science in Hydraulic Engineering.

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BDS	British Dam Society
BEM	Boundary Element Method
BS	British Standard
DRIP	Dam Rehabilitation and Improvement Project
D/S	Down Stream
E	Modules of Elasticity
EBCS	Ethiopia Building Code of Standard
ECO	Engineering Corporation of Oromia
FAO	Food Agricultural Organization
FEM	Finite Element Methods
FoS	Factor of Safety
GDP	Gross Domestic Product
GS	Ground Surface
GTL	Geotechnical Layer
GWP	Global Water Partnership
H	Horizontal
ICOLD	International Commission on Large Dam
LEM	Limit Equilibrium Method
K	Hydraulic Conductivity
MDC	Main Dam Center
M-P	Morgenstern Price

N	Slice Base Normal Force
NFL	Normal Flood Level
O	Common Point (Origin)
SEEP/W	Finite Element Software for Ground Water (seepage) Evaluate
SLOPE/W	Limit Equilibrium Software for Slope Stability Evaluate
U	Pore Water Pressure Force Acting on the Base
UCS	Uniaxial compressive strength
USACE	United States Army Corps of Engineers
USBR	United States Bureau of Reclamation
USSD	United States Society of Dam
U/S	Upstream
V	Vertical
W	Weight of the slice

ABSTRACT

This research was undertaken to evaluate seepage and slope stability analysis of the Chel-chel embankment dam located in south-eastern Ethiopia, with the goal of identifying a list of useful methods to prevent occurrences. The Chel-Chel Embankment dam, situated on a limestone foundation, encounters significant challenges related to seepage and structural instability due to the solutioning processes inherent in such geological formations. This study undertakes a comprehensive analysis of seepage and slope stability to evaluate the dam's performance under various conditions and propose remedial measures to enhance its safety and functionality. SEEP/W and SLOPE/W were conducted using Geo-Studio 2018, by incorporating the dam's different parameters. The analysis assessed seepage rates with and without treatment structures, revealing that untreated seepage ranged from 2.14 to 6.56 m³/day per dam length. Implementation of foundation treatment reduced seepage to 1.41–5.98 m³/day and the result confirm that, effective reduction in seepage when further foundation treatment is applied. Slope stability analysis was also performed under diverse loading conditions, including steady-state seepage, construction phases, rapid drawdown, and earthquake loading. The Morgenstern-Price technique was used to evaluate both force and moment balance. Results indicate that the dam meets or exceeds recommended safety thresholds, with computed factors of safety of 1.71 for steady-state seepage, 1.68 and 1.57 for upstream and downstream slopes during construction, and 1.44 for rapid drawdown conditions. Pseudostatic earthquake analysis, considering a peak ground acceleration of 0.1g, yielded a factor of safety of 1.24, confirming the dam's stability under seismic conditions. To mitigate seepage and improve stability, constructing a slurry trench or concrete cutoff in the dam central foundation section is the recommended proposed remedial measure.

Key Words: *Geo-Studio, Finite Element Method, Stability, Chel-chel dam, Pseudostatic earthquake*

1. CHAPTER ONE: INTRODUCTION

1.1. Background of the study

Dams are barriers that create reservoirs by obstructing water flow. Numerous uses exist for the water held in the reservoir, including irrigation, hydropower, industrial and municipal supplies, and recreation. Construction of dams can also be done for navigation, debris retention, flood control, and other uses. There has been a lengthy history of dam breakdowns and incidents occurring all over the world. Reports of dam failure are not uncommon these days. Human and environmental impacts of dam failure are widely recognized. Both preventative and mitigating actions are necessary for these. Numerous factors that can lead to dam collapses are Leakage and piping, overtopping, spillway erosion, severe deformation, sliding, gate failure, poor construction, and earthquake instability are the most frequent reasons for dam collapse (Umaru et al., 2014).

Earth and rock-fill dams are the two primary types of embankment dams, depending on the fill material that is utilized. A rock-fill dam is composed mostly of rock pieces and has an impermeable core. The core and the rock shells are separated by a sequence of transition zones made of appropriately graded material. An impermeable earth core should only be replaced with a membrane composed of concrete, asphalt, or steel plate when there is not enough impervious material available (USACE, 2004).

Embankment dams are commonly employed due to their relatively inexpensive construction costs, ease of managing natural resources, and minimal equipment requirements. Furthermore, they have more flexible site and foundation requirements than other types of dams (Talukdar,p. Dey, 2019). They can be utilized for a single or several uses, including as water storage, hydropower generation, flood control, deepening navigation channels, and recreation. However, despite their obvious advantages, embankment dams can fail, resulting in devastating results. Dam collapses can occur due to inadequate design, lack of knowledge of material characteristics, construction flaws, age, and inadequate defect monitoring (Omofunmi, 2020).

The requirements for ensuring safe embankment dam designs are as follows: 1) the embankment slopes must be stable throughout construction and under all reservoir operating circumstances. 2) Seepage through the embankment, foundation, and abutments must be controlled to avoid internal erosion. 3) The wave movement must not be able to reach the upstream slope. In contrast, the summit and downstream slope must be protected from erosion caused by wind and rain.4) the dam's embankment zoning design must ensure its safety in terms of stability, seepage, and cracking. 5) A suitable freeboard must be supplied above the NPL to prevent overtopping due to wave movement. 6) The decision is based on material availability and the most cost-effective handling method. 7) The dam core and shells must have sufficient strength, a minimal drop in strength characteristics as strain progresses, quantifiable settlement, and sufficient effective stress to avoid hydraulic fracturing (ICOLD, 2011).

The SEEP/W and SLOP/W models in the Geo-Studio application utilize a numerical model to assess the stability of the structures and mathematically simulate the real physical process of water moving through particle matter. Numerical modeling is completely mathematical, as opposed to full-scale field modeling or scaled physical modeling in a lab. The software is a very powerful calculator, and whether or not the tool produces useful and relevant results depends on the user's guidance. The user's ability to understand the input and interpret the results is what makes it such a powerful tool. The program can only do very complicated calculations that are otherwise impossible for humans to do (G.U.S. Manual, 2012).

Chel-chel dam construction is one of the large-scale irrigation development construction programs undertaken in Ethiopia and launched by the Ministry of Irrigation and Lowlands for sustainable agricultural development in the Oromia Region to address food security issues in the drought-affected area of the East Bale Zone Rayitu woreda. The primary objective of the dam is to facilitate irrigation projects that promote sustainable agricultural crop production throughout the region. The dam was designed to irrigate a command area of 4146 hectares. To ensure the long-term effectiveness of the dam, it is crucial to analyze/evaluate its stability against slope and seepage failures, as these factors are integral to the design of earth fill dam.

This study focuses on conducting seepage and slope stability analysis of the Chel-chel embankment dam using the Finite Element Method based Geo-Studio software. The ultimate goal is to identify existing water leakage issues, highlight potential dam instability concerns, and propose effective remedial measures for the sustainable utilization of the dam. On the other hand, the study result aids in putting a mark for further study, design, and construction of similar infrastructure development programs.

1.2. Statement of the problem

Ethiopia is building many rock-fill and earth-fill dams. Careful and thorough engineering geological investigations are needed before building a water impoundment structure on top of unbearing rock types. All embankment dam needs to be able to operate securely in both normal and extraordinary circumstances. The variability of natural soils and the fluctuating boundary conditions make it difficult to transform the actual seepage problems into an equal numerical counterpart. Both the seepage and stability studies have shown the significance of taking into account the coupled effects on the overall stability of the earth dam, as indicated by the parametric sensitivity analysis. The design and performance assessment of the earth dam under all seepage and stability situations are determined to require the analysis (Athania et al., 2015).

This study considers the Chel-chel embankment dam seepage and slope stability that is constituted by cutting through various geological conditions. Chel-chel embankment foundation is laid on an accumulated alluvial deposit. The geotechnical investigation undertaken during the commencement of the construction reveals the presence of unseen geological and topographical conditions that are not incorporated in the preliminary design provided. In the recent construction process, most of the dam site area, especially the central part has been cleared to the proposed general foundation level during the previous design stage. However, even after reaching the proposed general dam foundation level of the design, it was found out that the tick alluvial deposit is not completely removed even at the bottom of test pits excavated up to 5meter depth using the excavator to check the depth of alluvial deposit; but the bedrock was not reached to this depth.

Because of this fundamental issue that occurred in the foundation region, the dam is primarily vulnerable to seepage and slope instability issues and it is necessary to verify the seepage and slope's stability against collapse using the proper approach. The seepage and stability of slopes upstream and downstream was investigated in accordance with the geotechnical properties of the building materials utilized in this specific project. For rock fill dams on karst foundations, careful seepage and dam slope stability assessments are required. Determining probable failure reasons and putting the right preventative measures in place need an understanding of how the dam and its surroundings behave when the water level fluctuates. According to statistics, inadequate foundations and piping through dams are the primary reasons why dams fail (Aberra et al., 2018).

1.3. Objectives

1.3.1. General objective

The main objective of this study is to evaluate the Chel-chel embankment dam's seepage and slope stability and suggest a potential corrective action.

1.3.2. Specific objective

- To assess and measure seepage in different embankment dam zones.
- To examine the slope stability of the embankment dam under various loading scenarios.
- To provide suitable corrective measures for the outcomes of the study.

1.4. Significance of the study

This study provides relevant information for the decision-makers in the design of embankment dams and other relevant developmental activities, delivering numerous benefits to the surrounding communities. The study focuses on the seepage and slope stability analysis of the Chel-chel embankment dam, whose foundation is challenging and is laid on an alluvial deposit. By identifying and analyzing seepage patterns and slope stability conditions specific to this site, this study aims to provide empirical insights that can prevent catastrophic failures, thereby safeguarding lives and property.

The study will yield specific recommendations regarding design considerations and mitigation strategies tailored to embankment dams in the country facing similar geological challenges.

1.5. Scope of the study

The scope of this study is to provide a comprehensive analysis of seepage and slope stability which advances our understanding of embankment dam failure mechanisms by using the Finite Element Method (FEM) through the use of Geo-Studio software. The study does not assess other types of embankment dad failure like hydraulic failure (overtopping) and structural failure (settlement, deformation, and earthquake failures).

2. CHAPTER TWO: LITERATURE REVIEW

2.1. General

Dams continue to be essential hydroelectric, irrigation, flood control, and water storage infrastructure. Recent studies emphasize the need for sustainable dam design and operation, especially in the face of climate change and aging infrastructure. According to Reed (2024), modern dam engineering is shifting toward predictive maintenance, smart sensor integration, and eco-friendly designs to enhance safety and environmental compatibility. These innovations aim to address sedimentation, structural fatigue, and biodiversity loss, which are increasingly pressing concerns.

The two main kinds of dams are concrete dams and embankment dams. How the dam resists erosion and seepage pressures or counteracts the main hydraulic forces depends critically on its design and construction materials. Earth dams have traditionally been linked to seepage since they impound water. Seepage poses a safety risk when water flow shifts the materials of the foundation or embankment, or when high water pressure accumulates on the dam or its foundation (Ali Beheshti, 2013).

Failures of earthen dam heads can typically be categorized into three main types: hydraulic, seepage, and structural. In Ethiopia, numerous dam heads have been constructed, primarily for irrigation purposes. However, their capacity consistently diminishes before reaching their intended lifespan due to various factors. The primary reasons for this reduction in capacity include hydrological, structural, hydraulic, and seepage failures. In the Amhara region, seepage failures account for 58% of these issues (Tefera, 2006).

2.2. Embankment dam

A dam is a structure that blocks water or subterranean streams. Although other structures like floodgates or levees (also known as dikes) are used to restrict or halt water flow into certain geographical areas, dams are often employed primarily to contain water. Dams are frequently combined with hydropower and pumped-storage hydroelectricity to produce energy (The BP Statistical Review of World Energy, 2012). Good design, careful planning, analysis, and the use of the right construction techniques may bring out a dam's potential utility and beauty (Doherty, 2019).

Natural materials that have been dug or found nearby without any binding are used to build embankment dams. Depending on the materials utilized for the embankment, there are two primary types of embankment dams: rock fill dams and earth fill dams. If more than half of the material used in an embankment dam is made up of compacted soils, the dam might be classified as an earth fill dam. The main materials used to build an earth fill dam are specific engineered soils that have been compacted evenly, thoroughly, in relatively thin layers, and with a regulated moisture content. To keep the impermeable core from washing away, they often have drains and filters built of sandy and gravelly soils, as well as an impermeable core made of clayey soils (Novak et al, 2007).

The study must include the determination of the quantities of different types of materials that will be available and the sequence in which they are available, and a thorough understanding of their physical properties is required. In theory, soil traps can be designed so that any type of soil can be used. In practice, large organic soils (peat) are not selected because of their low shear strength and high compression. Due to construction difficulties, highly plastic inorganic clays (CH soils) are not desirable unless other materials are not available. Based on the configuration of the dam section, rock dams can be classified into: core rock dams, inclined core rock dams and membrane rock dams (Narita, 2007).

2.3. Embankment Dams Failures

An embankment dam is a large artificial dam that is more prevalent than any other kind of dam for a number of reasons, including the use of low-cost subsurface and raw soil resources in standard technological building methods, the lack of a specific valley form, etc. The kind of construction, subsurface conditions, and the components of the soil that are buried all affect the geometry of embankment dams. Usually, a complicated semi-plastic mound of different soil, sand, clay, and/or rock compositions is placed and compacted to make it. It features a solid, impenetrable core and a semi-pervious, waterproof natural coating on the outside. This renders the dam immune to seepage or surface erosion Redda (2016).

When the underlying rocks or foundation are too weak to support the masonry dam, earth dams are built. Earth dams are more prone to collapse because they are less rigid. Three categories may be used to classify the many reasons why earth dams fail (Saluja et al., 2018).

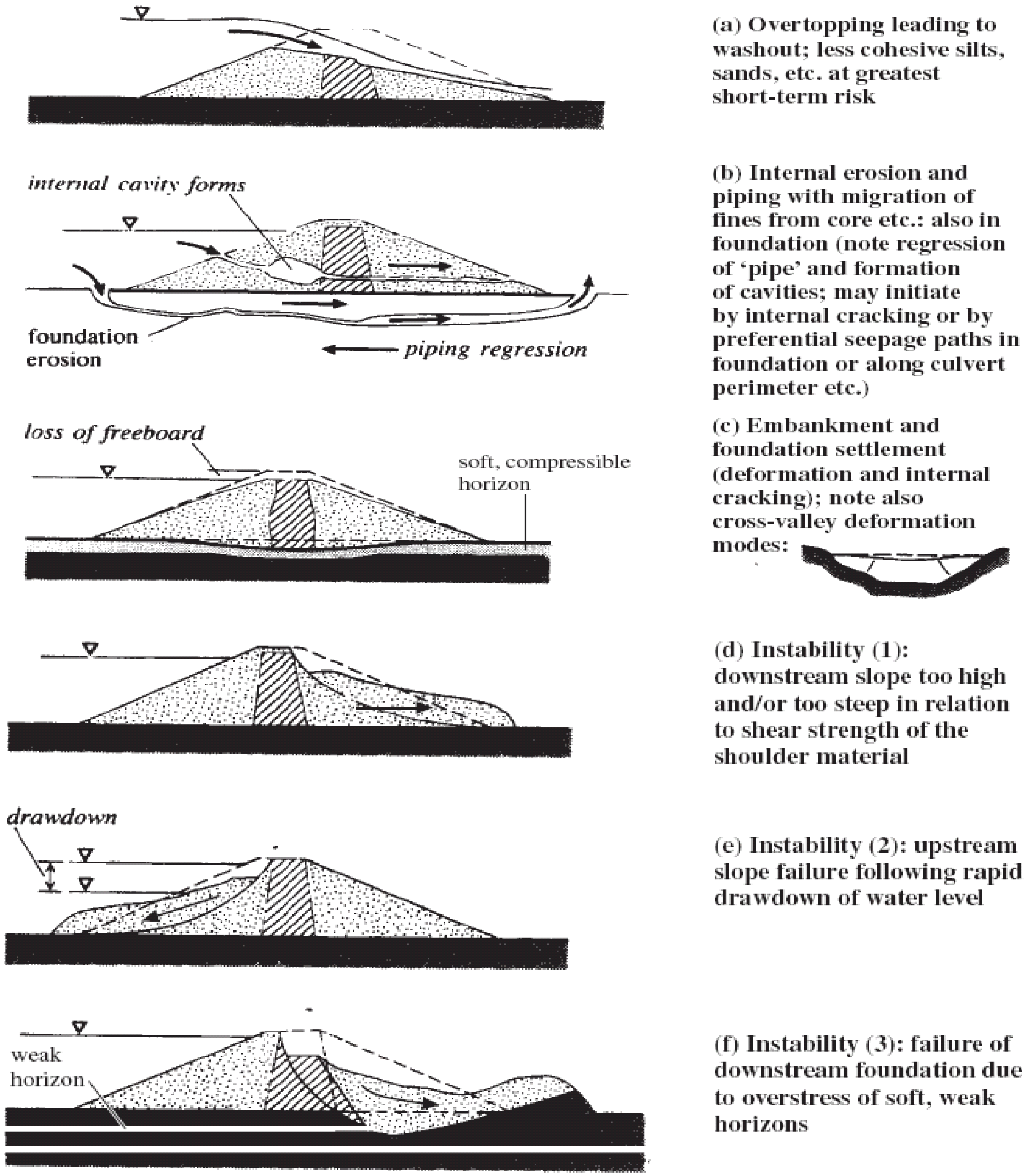


Figure 2.1: Causes of Dam Failure Mechanism (Novak et al., 2003).

2.4. Typical failure mode of embankment dams

2.4.1. Hydraulic failure

Hydraulic failures due to uncontrolled flow of water in and near the embankment are due to the erosive action of water on the embankment slopes. The vulnerability of earth embankments to erosion is greater since they are usually not designed to droop. Failure in these categories can be directly or indirectly related to the cause of overflow, upstream face erosion, ice cracking, erosion of the upstream face of the stream by the formation of a ravine and downstream toe erosion (NHDES, 2011).

The erosive action of water on the embankment slopes causes hydraulic failures from the uncontrolled flow of water over and next to the embankment. Approximately 40% of earth dam collapses have been attributed to hydraulic failure. Overtopping, u/s face erosion, d/s face erosion, and d/s toe erosion are examples of hydraulic failures (Getachew, 2018).

2.4.2. Structural failure

The separation (rupture) of the embankment material and/or its foundation is a structural failure. An earthen embankment's structural collapse might manifest as a slide or material displacement in the upstream or downstream faces. Sloughs, bulges, fractures, or other anomalies in the dike or embankment typically signal structural collapse and significant instability. Either the appurtenances or the embankment may experience structural problems. About 25% of dam disasters have been attributed to structural failures. The structural failures include foundation slides (spontaneous liquefaction), u/s and d/s slope failures in embankments, and seismic loading, which can cause the foundation and appurtenant structures in addition to the dam itself to fail (Getachew, 2018).

2.4.3. Seepage failure

Most embankments show signs of seepage. Nonetheless, it is crucial to regulate both the quantity and pace of this seepage. It occurs within the embankment itself and/or through its underlying structures. If left unchecked, seepage has the potential to wash away fine soil particles from the downstream slope or foundation, and can migrate upstream to create a pipe or cavity leading to a pond or lake, often resulting in the failure of the embankment. This phenomenon is referred to as "piping." As defined by (ASTM, 2002), piping is the gradual extraction of soil particles from a mass due to the percolation of water, resulting in the formation of channels. Seepage-related failures account for roughly 40 percent of all embankment failures (NHDES, 2011).

Seepage is seen in most embankments. However, the amount and velocity of this seepage need to be managed. Seepage happens through the foundation of the dike or earthen embankment. If left unchecked, seepage can remove fine soil particles from the foundation or downstream slope and continue to move upstream, creating a hollow or conduit to the pond or lake and frequently resulting in the embankment's total failure. These factors have contributed to the failure of over one-third of earth dams. Sloughing of the downstream toe, piping through the dam's base, and piping through the dam's body are examples of seepage failures (Anyemedu, 2007).

1. **Piping through the foundation**

If the foundation contains highly permeable cracks, fissures, or layers of coarse sand or gravel, water may begin to seep through it at a very rapid rate. These activities may cause the soil to erode or allow a lot of water to seep through it, which will lead to soil erosion and the development of pipes. Causing a surge of soil and water, which will cause hollows beneath the foundation and cause the dam to collapse or settle, ultimately failing.

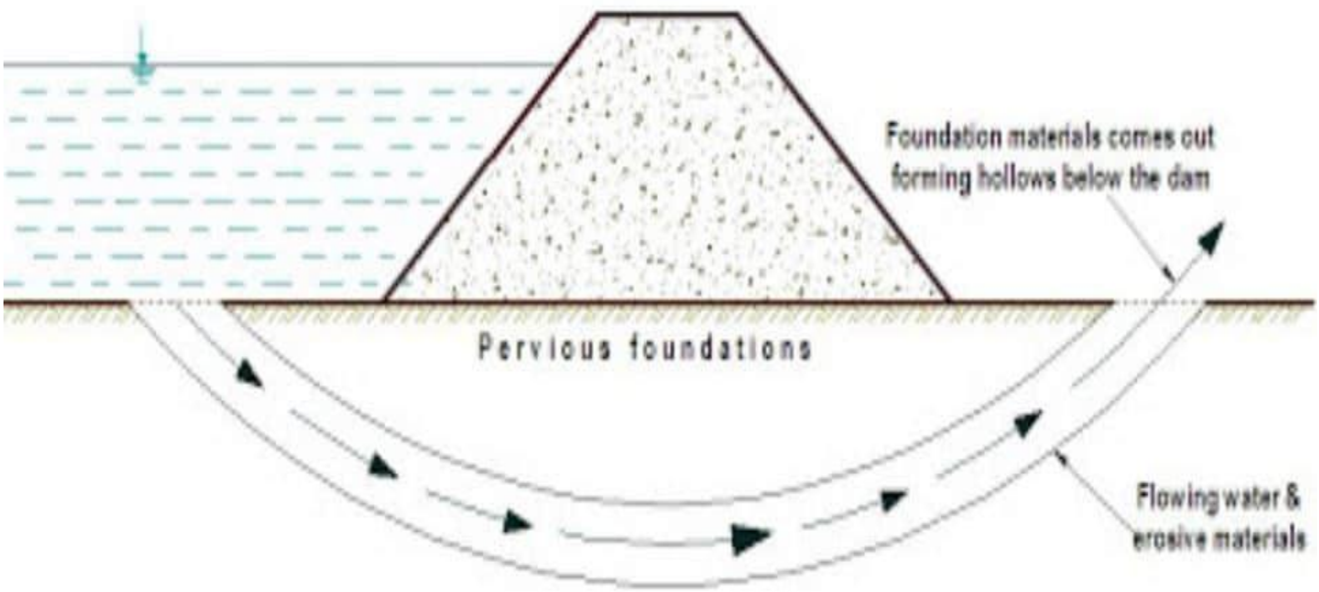


Figure 2.2: shows piping through the dam foundation.

2. **Piping through the dam body**

Piping through the dam body is a significant internal erosion process that threatens the structural integrity of embankment dams, often developing under high seepage gradients. Rising phreatic surfaces—especially near the downstream toe—can initiate backward erosion tunnels. .

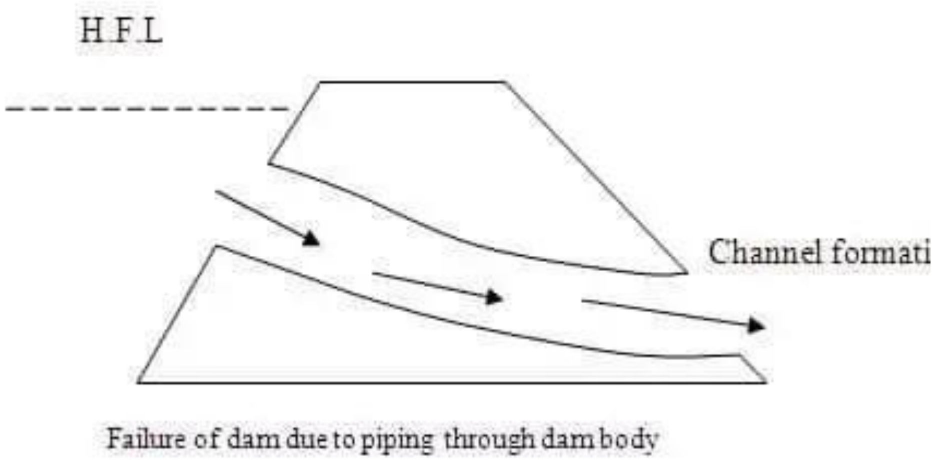


Figure 2.3: Shows piping through the dam body (Garg, 1987).

3. **Sloughing of dawn stream toe**

Sloughing at the downstream toe of an embankment dam typically results from erosion caused by seepage or concentrated flow, which weakens the soil structure and leads to progressive slope failure. This condition is often exacerbated when outlet releases or inadequate energy dissipation create scour holes at the toe, undermining the dam's stability. If left unaddressed, such sloughing can steepen the downstream face and potentially trigger a full-scale failure of the embankment (DOWL, 2018).

2.5. Seepage analysis methods

The seepage pattern is consistent regardless of dam material (sand, clay, or silt), however the seepage rate varies depending on soil type. The presence of seepage lines on the downstream slope tends to render it unstable. This is required for both flattening the downstream slope and diverting seepage away from it. To avoid collapse, all storage dams must regulate seepage and limit the effect of infiltration (i.e. water loss). It is necessary to confirm the location of the phreatic line. The seepage analysis is used to calculate the volume of water going through the dam's body and foundations, as well as to estimate the pore pressure distribution. There are various ways for determining the volume of water moving through a dam, including analytical, numerical, and graphical methods.

2.5.1. Analytical method /Darcy's Low – phreatic line Analysis

The line at the top surface of the seepage flow where the pressure is atmospheric is known as the pheneatic line, seepage line, or saturation line. For the following reasons, the seepage line's position in an earth dam is necessary: It allows us to distinguish between dry and submerged soil. For the purposes of calculating the soil's shear strength, dirt above the seepage line is considered dry, while soil below the seepage line is considered submerged. Because it shows the top stream line, it helps with the flow net drafting. The seepage line determination allows us to ensure that it does not cut the downstream face of the dam. This is crucial to keeping the dam from sloughing or softening (Garg, 2005).

1. **Estimation of discharge and phereatic line determination for homogenous earth dam with horizontal drainage blanket**

Except at or near the entry to the upstream face, as shown in Fig.2.5, the phreatic lines in this case correspond with the fundamental parabola. It has been found from experiments that the infiltration line is pushed by the filter. Consider a basic parabola with focus at F which is designed and fabricated to intersect the water surface at point A. Casagrande demonstrated that for a dam with a relatively level slope, AB = 0.3HB. With the focus at F, the parabola of the base AIJC can be drawn after knowing the point A, and from the upper face of the dam GB an equipotential line is made completely covered with water, the infiltration line must be perpendicular to this face near its point of the crucifixion. B. The curve BI is then rectified such that it is perpendicular to GB, and ultimately BIJC represents the infiltration line.

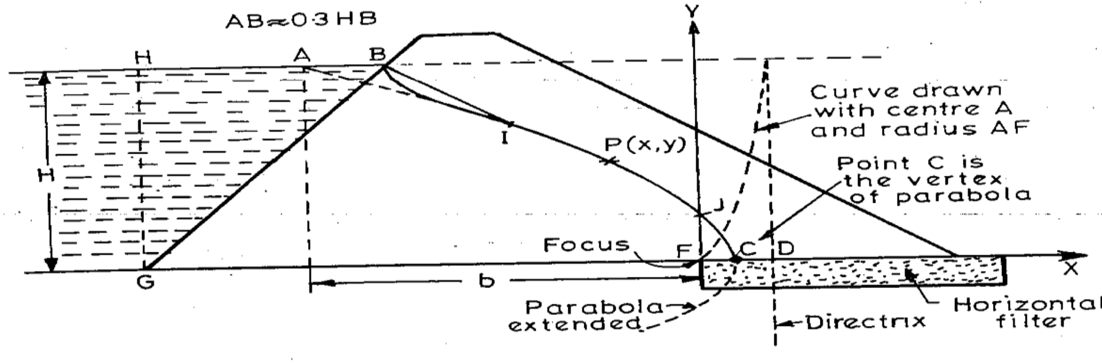


Figure 2.4: Phreatic line for a homogenous earth dam with horizontal drainage blanket

The fundamental characteristic of the parabola used to build the base parabola is that the distance of any point p from the focus equals the distance of the same point from the directrix. Using focus F as the origin, the basic parabola equation is given as.

$$\sqrt{x^2 + y^2} = x + FD \quad \text{..1}$$

$$y = \sqrt{s^2 + 2xs} \quad \text{..2}$$

Where FD is the distance of focus from the directrix, it is represented by S or y0, these is because of when x = 0, S = y0 hence FJ will be equal to S = y0 and it can be determined as

$$FD = S = \sqrt{b^2 + H^2} - b \quad \text{..3}$$

Where From point A (known), x = b and y = ht

Then by working out a few more coordinate points from the equation 2 the parabola can be easily drawn and corrected for the curve from the upstream face of the dam with known point A.

Discharge through the body of the earth dam can also be calculated as follows:

According to Darcy's low, consider a unit length of the dam

$$v = ki \quad \text{..4}$$

$$q = vA = kiA \quad \text{..5}$$

$$i = \frac{dy}{dy} \quad \text{..6}$$

$$q = k\frac{dy}{xy}y * 1 \quad \text{..7}$$

The discharge crossing any vertical plane across the dam section will be the same when steady conditions have reached. Where q = seepage discharge per unit width of the dam,

i = hydraulic gradient,

k = coefficient of permeability,

A = cress sectional area.

From the parabola equation 2

$$y = \sqrt{s^2 + 2xs} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.8$$

Then by substitute equation 4 from 7 and derivate it the discharge q can be computed as

$$q = k\frac{d(\sqrt{s^2+2xs}}{dx}\ (y = \sqrt{s^2 + 2xs}) \ldots\ldots\ldots\ldots\ldots\ldots\ldots.9$$

$$q = k\frac{s}{\sqrt{s^2+2xs}}\ (\sqrt{s^2 + 2xs})$$

$$q = ks \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.10$$

Where k = coefficient of permeability and s = focal distance, then the discharge q can be easily computed.

2. **Estimation of discharge and phereatic line determination for homogenous earth dam without horizontal drainage blanket**

The primary focus in this example will be on the d/s slope's lowest point, F. The base parabola BIJC will visibly cut the d/s slope at J and extend beyond the dam's restrictions to point C (the parabola's vertex), as seen by the dotted line in Figure 2.6. The phreatic line must, however, emerge at a certain position k, where it will tangentially intersect the d/s face, in order to satisfy exit conditions. Figure 2.6: A homogeneous dam section without a filter. The part KF is therefore referred to as the discharge face, which is usually moist. Casagrande provides a graphical solution for the adjustment Δa, which shifts the parabola downwards, based on different slope α values of the discharge face.

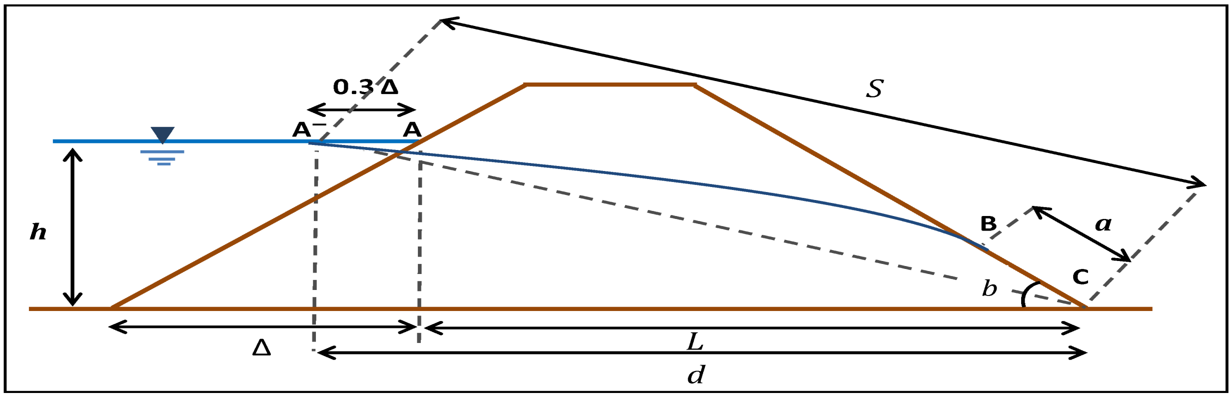


Figure 2.5: Homogenous dam section without filter.

The portion KF is then known as discharge face and always remains wet. The correction Δa, by which the parabola is to be shifted downwards, is found graphical solution by the value of Δa/a+ Δa given by Casagrande for various values of the slope α of the discharge face.

The slope angle α can even exceed the value of 900. a+ Δa Is the distance FJ and is known as shown from the table 1. Then Δa can be evaluated and a and Δa can be connected by the general equation.

$$\Delta a = (\mathrm{a} + \Delta\mathrm{a})\left[\frac{180° - a}{400°}\right] \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 11$$

Table 2.1: value of a with different angle of inclination defined by Casagrande.

a in degree	$\frac{\Delta a}{a + \Delta a}$	Remark
30	0.36	Note, intermediate value can be interpolated, or read out from the graph between and $\frac{\Delta a}{a+\Delta a}$ plotted with the value given here.
60	0.32	
90	0.26	
120	0.18	
135	0.14	
150	0.10	
180	0.0	

To find the value of an analytically, Schaffernak and van Iterson assumed that the energy gradient $i = \tan\alpha = \frac{dy}{dx}$ This means that the gradient is equal to the slope of the line of seepage, which is approximately true so long as the slope is gentle (i.e. <300). With the base of this assumption the value of a can be easily driven.

2.5.2. Graphical/Flow Net Analysis

A flow network is constructed from up of both equipotential and perpendicular flow lines. The stream channel is the segment between two consecutive flow lines, whereas the field is the closed section between two consecutive equipotential lines and subsequent flow lines. Analysis of seepage through flow networks contributes to the proper design and construction of many dams. The flow network is drawn freehand and appropriate adjustments and corrections are made until the flow line and the equipotential line intersect at right angles. The seepage rate (q) can be calculated from the flow network, using Darcy's law. To conduct seepage analysis using a flow network, first design a flow network diagram with a subjective separation of the equipotential line and the flow line. The outcome rises as the number of separation points grows (Garg, 1987).

1. Computation of rate of seepage from flow net through the isotropic soil

When the soil is isotropic, the horizontal permeability is equal to the vertical permeability KH = Kv, the amount of drainage can be easily calculated from the flow network using Darcy's law as follows:

Assuming a unit width cross section of the dam as shown in Figure 10, the flow through the field ABCD and applying the continuity principle of Darcy's law

$$q = kiA \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.... 12$$

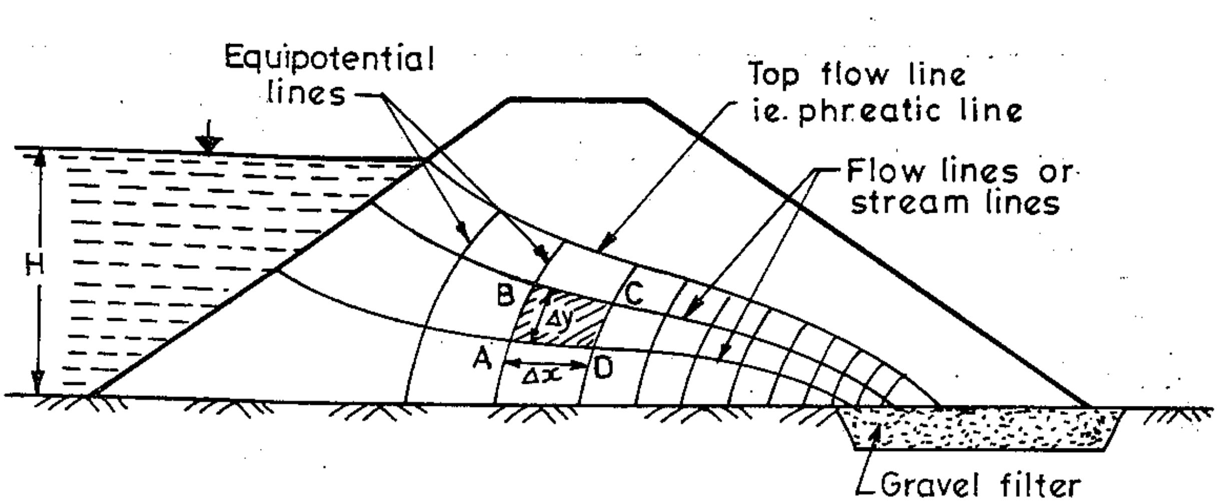


Figure 2.6: flow net in the earthen embankment dam.

$$\Delta q = k\frac{\Delta H}{\Delta x}\Delta y * 1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 13$$

But $\Delta H = \frac{total\ head\ causing\ flow}{number\ of\ drops\ in\ the\ complet\ flow\ net} = \frac{H}{Nd}$

After substituting and simplified the required expression, representing discharge passing through a flow new within the isotropic soil is

$$q = \sum\Delta q = \frac{kH}{Nd}Nf \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 14$$

Where: ΔH = head drop through the field,

Δq = discharge passing through the flow channel,

H = total head causing flow,

Nf = number of flow channel

Δx = is horizontal distance between the flow line

Δy = is Vertical distance between the equipotential lines

2. Computation of rate of seepage from flow net through the non-isotropic soil

Non-isotropic soil has different permeability in the vertical and horizontal directions. To illustrate the flow in this situation, the cross-section across anisotropic soils is plotted to a natural scale in the y-direction, but to a modified scale in the x-direction; all dimensions parallel to x- axis being reduced by multiplying by the factor   Field of transformed section will be a square one, while the field of actual section (retransformed) will be a rectangular one having its length in x direction equal to factor times the width in y direction. After that, the discharge may be calculated using the formula.

$$q = \sqrt{k_H k v} * \frac{N_f}{N_d} * H \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 16$$

2.5.3. By SEEP/W Software Model

A numerical modeling program called SEEP/W is used to address real-world seepage issues. The SEEP/W program is created by combining drainage theory and the finite element method and working in a saturated/unsaturated soil region. An analysis of the expected amount of seepage through the embankment and the foundations of the dam using the SEEP/W software model requires sets of parameters such as the Dam cross section, material permeability coefficient, piezometer data, and boundary conditions. Practical seepage problems are never easy to convert into numerical models due to the heterogeneity of natural soils and variable boundary conditions. In general, the boundary conditions of a seepage problem are never the same as those found at the initial stage. Therefore, seepage analysis in the SEEP/W program falls into two categories.

I. Steady - state analysis: - in the steady state, the basic properties of the water flow such as water pressure and water flow velocity never change. Reaching a stable state is nearly impossible. The sole purpose of steady-state analysis is to find out how the initial input parameters respond to a given boundary condition. The amount of time needed to achieve a steady state is never displayed in this analysis. I. It returns a set of resolved values for the water pressure and water flow parameters for the particular boundary conditions. Constant pressure (H) and constant flow are the important boundary conditions used for steady-state analysis.

ii. Transient analysis: - Transient analysis is used to find out how long it takes for the soil to respond to a given boundary condition. Therefore, the basic flow properties (water pressure and flow) change with time. The analysis required an initial boundary condition as well as a target boundary condition (Rajeeth, 2011).

2.6. Purpose of seepage analysis

Dams need to be planned and maintained in order to securely manage seepage. If excess drainage is not managed properly, dam safety issues can arise. seepage analyses are used to determine the amount of seepage flow that can pass through an embankment or foundation, to estimate the water level in an embankment, to estimate the pore pore pressure in an embankment or foundation, to estimate outlet gradients and/or uplift pressures at the foot of an embankment, to assess the effectiveness of various seepage reduction measures, to estimate the amount of seepage flows captured by drainage elements, to size and optimize the configuration of these types of drainage elements and to evaluate the effectiveness of drainage systems that "drain or help with their design (Redda, 2016).

2.7. Seepage control in embankment dams

The requirement to regulate seepage will vary depending on the amount, substance, and location of seepage. After construction, seepage is hard to reduce and costly. Typical methods used to control the amount of seepage are grouting or installing an upstream cover. It's critical to manage the composition of the seepage or stop it from displacing soil particles. Various methods can be used to reduce and/or control seepage, depending on the requirements to avoid uneconomical water losses and the likelihood that the foundation will transmit water forces and pressures associated with seepage, which can contribute to static instability and cause internal erosion, heave, or blowout. This reduces the associated water pressure so that static instability, heave, blowout and internal erosion are adequately controlled in the areas below the foundation (William, 2012).

If seepage is identified in a dam's fill or foundation, it must be closely monitored on a regular basis until repaired. If the seepage rate increases or the embankment soil shows signs of instability, remedial measures must be taken quickly. A qualified geotechnical engineer or dam safety professional should be contacted for inspection and advice on all high seepage dam problems. The storage level must be lowered in the event of a serious slide or collapse of the piping or embankment. Subsidence due to seepage in both embankments can be corrected by removing unstable soil and constructing a two-way drain with a permeable soil filter. Seepage, piping, and boils in existing dams can be corrected or slowed by capturing water before it exits the downstream side of the dam. To stop foundation leaks, lay the blanket on the storage floor. All cracks and erosion channels on the slope should be filled, leveled, and replanted. Boring rodents should be eliminated from the dams and any damage created should be repaired by filling with soil or a filter drain (Omofunmi, et al. (2017).

2.7.1. Seepage Control through dam embankment

The three methods for seepage control in embankment are:-

i. Use of filters: - Every seepage discharge surface, internal and external, which may be subject to piping and heave, should be covered with filters that allow water to escape freely but retain particles in place. Filters have two main functions: to prevent internal erosion by blocking the the migration of soil particles from the base soil, and to facilitate the internal drainage of seepage flows without the accumulation of excessive seepage forces and hydrostatic pressures in filters or drains.

ii. Use of impervious Core: - seepage reduction in embankment dams is achieved by providing an impermeable core in the middle of the section. Most of the energy for the stored water is consumed by the drainage through the core, which is subjected to excessive seepage forces in the process. Therefore, the core is used in combination with a filter and drain; the filter protects the core against piping resulting from excessive seepage forces and the drainage prevents seepage from entering the downstream shell.

iii. Use of Drainage: The best way to prevent seepage in earth dams is to install chimney drains, which can be placed in the core of the dam and can be positioned along the d/s face of the impermeable core in a designated region of the dam, or they can be attached to the d/s blanket for drainage (Hordofa, 2015).

2.7.2. Seepage Control through dam foundations

Because the groundwater regime changes during reservoir closure, internal erosion and piping may become more likely to occur in dam foundations and abutments, which are normally stable under the influence of natural groundwater flow. Foundation drainage control measures include a positive cut formed in an excavation in an impermeable layer and filled with compacted impermeable material, concrete shear walls, grout curtains, mud channel interruptions (soils), piles, impermeable covers, vertical drains, drain wells and filter trenches.

Effective seepage control requires that the embankment, its foundation and the adjacent structure behave as a single unit. If the foundation of an earthen dam consists of an impermeable layer, there is generally no need for specific measures to reduce seepage. However, on rock foundations, grouting and surface treatment may be necessary. On the other hand, methods are commonly used to control seepage through percolators, seepage reduction methods include trench cuts, underwater impermeable covers, concert diaphragms, slurry trench, and grout curtains (Hordofa, 2015).

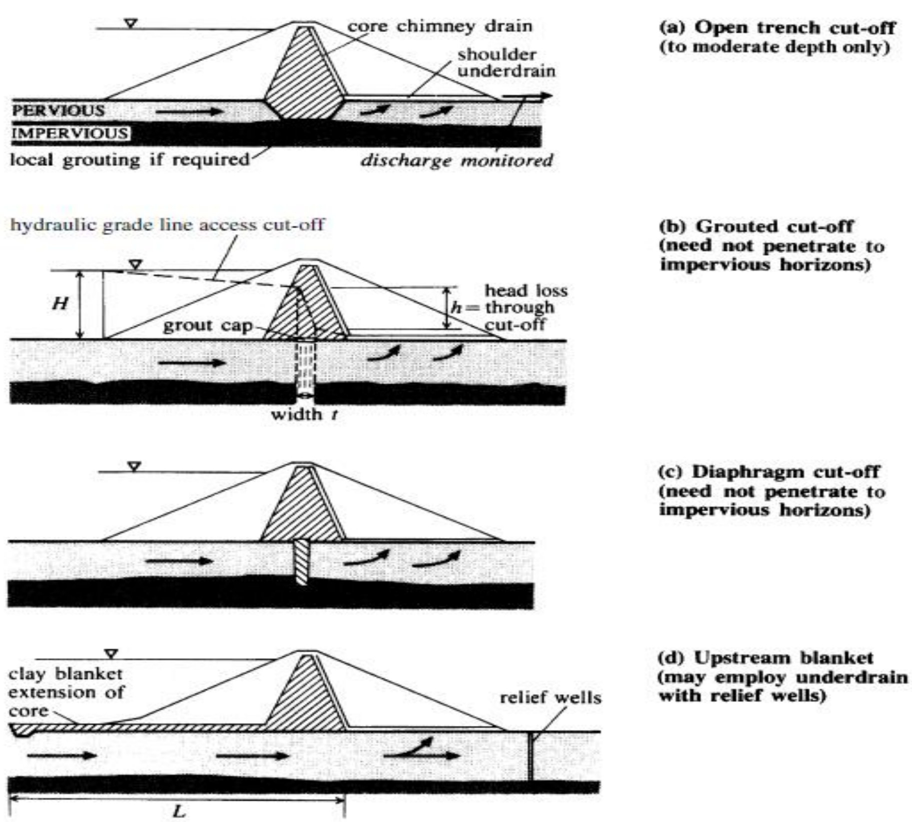


Figure 2.7: Seepage Control through dam foundations

2.8. Stability of embankment dam

2.8.1. Slope Stability Analysis

A probable failure surface's factor of safety (in shear) is ascertained by slope stability analysis. The safety factor is computed as the ratio of resisting force to driving force, or moment. The factor of safety should be calculated using the most adverse conditions in which tests will be conducted to establish the material's attributes (parameters). The most unknown aspects of stability difficulties come when pore pressure and strength parameters are chosen. The stability analysis of an earthen embankment involves knowledge of the soil's proper shear strength characteristics, including the embankment and base. (Tadesse. 2017).

2.8.2. Purpose of Stability Analysis

The purpose of Slope Stability Analysis was to understand the growth and shape of natural slopes, as well as the mechanisms responsible for various natural characteristics. Assess slope stability under short-term (often during construction) and long-term conditions, evaluate the possibility of landslides involving natural or engineered slopes, analyze landslides to understand failure mechanisms and the influence of environmental variables, redesign failing slopes and plan for the building of preventative and remedial measures, and study the effect of seismic stress on slopes and embankments (Kiser et al., 2013).

2.8.3. SLOPE/W software

SLOPE/W, developed by GEO-SLOPE International Canada, is used to assess slope stability. This curriculum is based on the principles and concepts of limit equilibrium techniques. In this study, SLOPE/W was utilized both alone and in combination with SEEP/W, another tool that calculates pore pressure distributions using groundwater infiltration analysis and a finite element mesh. Finally, the FOS was calculated using the pore pressure distributions and slope stability analysis. The SLOPE/W software calculates the factor of safety for various cutting surfaces. The dry slope's stability was initially assessed using SLOPE/W. Minimum FOS and crucial SS were sought based on the input and output parameters. Similarly, a Mohr-Coulomb soil model with no distinguishing stress fractures was used. The forces between the slices were calculated using a half-sine function with a tolerance of 1%. Furthermore, the half-sine function was chosen under the premise that shear pressures between the slices are maximal at half the CSS and zero at the entrance and exit sites (Aryal, 2006).

2.8.4. Static Instability Indicator

If there is an appropriate slope stability failure, longitudinal cracks on the dam crest or slopes, erosion or sloughing near the dam's downstream toe causing local over steepening of the downstream slope, surface measurement points indicate movements, or internal instrumentation indicates movements, the static stability of an existing embankment dam and its foundation must be evaluated (Redda, 2016).

2.8.5. Slope Stability Analysis methods

Many numerical analytic approaches have emerged during the last six decades to address complicated engineering issues as a result of the introduction of high-performance computers. Among the various numerical techniques available, the finite difference method (FDM), the finite element method (FEM), the finite volume method (FVM), the boundary element method (BEM) and the meshless are become more popular among scientists and engineers. FEM is used to solve very big and complicated problems, and it is critical that the solution process stays efficient and inexpensive. From the engineer's perspective, FEM can be more efficient and easier to use with sophisticated pre- and post-processing tools (Athania, et al., 2015).

The stability of curved slopes in embankments constructed on soft soil may be evaluated using a variety of techniques. These methods may be broadly divided into three categories: finite element methods, limit analysis, and limit equilibrium methods. Many stability analysis approaches come under the limit equilibrium group. The slicing strategy is frequently used to address boundary equilibrium problems. After separating the soil mass into slices, the forces acting on each slice on the sliding surface are determined. While the limit analysis technique takes performance requirements into account, the limit equilibrium approach disregards the materials' deformation characteristics. Earthquakes and vibrations are among the increasingly complex problems that are resolved using the finite element method (Kiser et al., 2013).

The limit equilibrium approach reduces the shear resistance available along a possible sliding surface by a safety factor, resulting in an equilibrium between the mass contained in the sliding surface and the free surface. Limit equilibrium approaches do not predict the movement of soil and debris masses. The finite element approach calculates the stress-strain response of a mass generated by forces applied to it. This approach is more accurate since it considers stress and deformation. This strategy has been effective because it includes representative stress-strain characteristics. The residual stress-strain parameters necessary for finite element analysis are more difficult to obtain than the force parameters needed for limit equilibrium analysis.The three main types of limit equilibrium analysis that are often used in practice are: the slice method, the wedge method, and the infinite slope method. The software to be used for limit equilibrium analysis for slope stability works with the slice method. (Omari, 2012).

The limit equilibrium formulation is very useful for understanding what is going on behind the scenes and for understanding the reasons for the differences between the different methods. This is not necessarily a routine method of analysis in practice, but it is an effective complementary method, useful for your reinforcement. The significance of the interfacial force function is related to the form of the sliding surface. When the sliding surface is circular, the moment safety factor appears to be unaffected by the postulated interface force function. There are numerous approaches to calculating slope stability (GEO-SLOPE International, 2018).

1. **Morgenstern-Price method**

Morgenstern and Price (1965) created a system similar to Spencer's, but with the option for users to select different power functions. SLOPE/W includes the following cross-slice functions for use with the Morgenstern-Price (M-P) method: constant, half-sine, truncated sine, trapezoidal, and data-point-specific. The Morgenstern-Price approach: allows for a variety of user-selected inter-slice force functions; ensures moment and force equilibrium; and considers both shear and normal inter-slice forces.

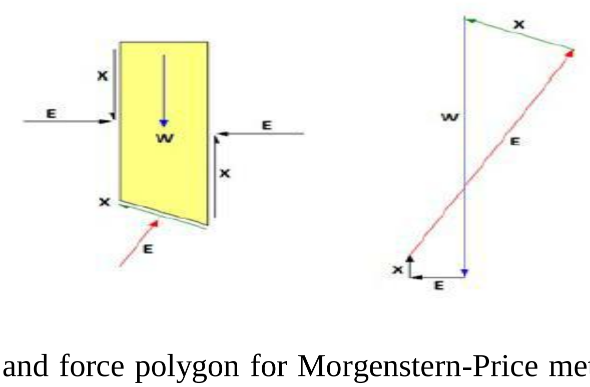


Figure 2.8: Free body and force polygon for Morgenstern-Price method: Source: (GEO-SLOPE International, 2018).

2. Bishop's simplified method

By adding together slice forces vertically, Bishop created an equation for the normal at the slice base. This has the effect of making the base normal a function of the safety factor. Because of this, the factor of safety equation becomes nonlinear (FS occurs on both sides of the equation), necessitating an iterative process to calculate the factor of safety. When there is no pore-water pressure present, the Bishop's Simplified factor of safety equation can be expressed simply as follows:

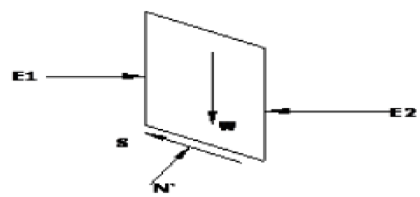


Figure 2.9: Free body diagram for the inter-slices of Bishop's simplified method: Source: (Rajeeth, 2011).

Bishop's simplified method (BSM):

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

Where N' = normal force, S = shear resistance/shear strength of the soil, E1, E2 = inter-slice normal force.

$$FS = \frac{1}{\sum W sin\alpha} \sum \left[\frac{c\beta + wtan\emptyset - \frac{c\beta}{FS} sin\alpha \; tan\emptyset}{m\alpha} \right] \text{....}\ldots\ldots\ldots\ldots 17$$

Where FS = Factor of Safety.

$\sum$W sinα = Sum of the downslope components of the weights of the slices (driving forces).

Cβ = Cohesion component acting along the base of a slice, adjusted for slice inclination.

W tan∅ = Shear strength due to friction (w = normal force, ∅ = internal friction angle).

Cβ / FS sinα tan∅ = A correction term that accounts for the interaction between cohesion and friction under the assumed FS.

Mα = A moment arm or geometric factor related to the slice's base inclination.

To calculate the Bishop's Simplified factor of safety, we must first make an estimate of FS. SLOPE/W uses the first guess as the Ordinary factor of safety.

After computing mα using the first FS estimation, a new FS is determined. The new FS is then used to determine mα, followed by another new FS calculation. The method is continued until the final determined FS is within a specified tolerance of the previous FS. Fortunately, a convergent solution is usually achieved within a few iterations.

3. Janbu's simplified method

The simplified Janbu method is similar to the simplified Bishop method, except that the simplified Janbu method only satisfies the general horizontal force balance, but not the general moment balance. The simplified Janbu FoS lies on the force balance curve, when lambda is zero. Because the force balance is sensitive to the assumed interslice shear, disregarding it, as in the simpler Janbu technique, results in an insufficient factor of safety for circular slip surfaces. The simplified Janbu approach takes into account normal shear forces but ignores shear forces between slices, thus it achieves the general horizontal force balance but not the general moment balance (GEO-SLOPE International, 2018). Janbu Simplified Method (JSM): Satisfies both force balances, does not satisfy moment balance, Considers normal forces between slices, and commonly used for composite shear surfaces.

2.8.6. Standard Loading Condition

Throughout their construction and operation, embankment dams are subjected to a range of stresses, thus they must be safely designed and built to withstand them. Limiting equilibrium techniques are based on calculating the mobilized resistance and applied stress in a fictitious broken surface on an embankment slope, followed by the safety factor. Gravity and leakage are two factors that contribute to slope instability while evaluating the stability of embankment dams. Three crucial phases should be taken into consideration when designing, since the fundamental computations rely on effective stress and require knowledge of pore pressure (Yazdanian et al., 2017).

1. End of Construction Loading Condition

The conclusion of before the catchment begins, the construction phase always has a higher safety factor than later phases because the distribution of pore water pressure is reduced. In this instance, it is necessary to investigate the dam's stability under both effective and total stress. It is necessary to identify and specify pore water pressures in order to conduct effective stress evaluations. Pore water pressures are defined as zero for total stress studies that are performed by computer programs, manual calculations, or slope stability charts; nevertheless, the pore pressures are not equal to zero. This is crucial since pore pressure is deducted from the total normal stress at the slice's base by all computer algorithms used for slope stability calculations. When construction is finished, both downstream and upstream slope of the embankment dam is in critical condition. In this case, the materials used in the clay core are undrained and unconsolidated.

2. Steady-State Seepage Loading Condition

Steady-State seepage conditions will be formed in the dam body and base following construction and the necessary amount of time, and the consolidation of the dam body will occur over time. Effective stress analysis should therefore be used to evaluate the stability of the upstream and downstream slopes. By combining the continuity equation and Darcy's law, the fundamental equation of groundwater motion is produced in two dimensions under saturated and unsaturated flow conditions. Following dam catchment and stability safety factor control, the dam's upstream and downstream are examined in steady-state seepage conditions. Effective stress shear strength characteristics along with measured or anticipated embankment and foundation pore pressures should be used to conduct steady-state seepage loading conditions.

3. Rapid Drawdown Loading Condition

The seepage in the dam body will alter and the seepage line will be inverted if the water level behind the dam can be decreased. Studying the upstream slope against rapid drawdown is crucial because of this. Rapid reservoir drawdown may result in upstream slope instability and incidence slip, leaving the dam materials saturated and initiating the leakage flow towards the upstream slope. Reservoir quick drawdown is the act of releasing water from a reservoir at a rate that, when the water level drops, does not alter the pore water pressure inside the body and the phreatic line stays in its original position. Rapid reservoir drawdown will result in a quick drop in pore water pressure, and when this drop occurs suddenly and sharply, the pressure applied to the dam body will rise and cause the dam body to rupture.

2.8.7. Factor of safety

The premise of a slope stability analysis is that downward or motivating (gravitational) and upward (resisting) forces are responsible for a slope's stability. For a slope to be stable, the opposing forces must be stronger than the motivating forces. Geotechnical engineers usually use a factor of safety to communicate the relative stability of a slope, or how stable it is at any particular time. The ratio of the pressures and moments that encourage mobility to those that hinder it is known as the factor of safety. Both of the equilibrium conditions are met by this approach. Furthermore, a linear Mohr-Coulomb equation is assumed for the inter-slice force interaction. The inter-slice forces are modified until the force and moment equilibrium Factor of Safety is satisfied (Salween, et al., 2016).

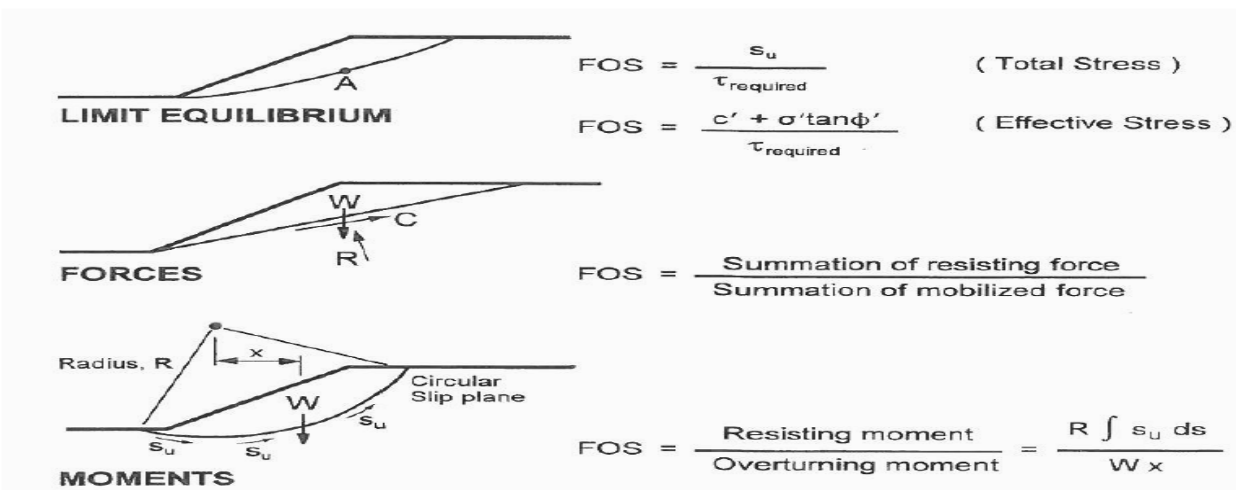


Figure 2.10: adopted of Factor of Safety (FOS) (from Abramson et al., 2002)

FS=ΣR/ΣM--18

Where ΣR = refers to the sum of resisting forces or moments that prevent failure.

ΣM = refers to the sum of driving forces or moments that contribute to failure.

Table 2.2: Minimum values of factor of safety as recommended by International Standards.

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

2.9. Remedial measures of Seepage for embankment dam

An existing dam's seepage issue has been located, examined, and evaluated, and remedial action of some kind is judged required. Additionally, assume that there is enough time to plan and build a long-term corrective solution and that the seepage issue does not pose an immediate threat to the dam's safety. It might be challenging to pinpoint the exact size and scope of remedial management since seepage is frequently hard to assess. As a result, it is essential to keep an eye on the "fix" to determine whether its goal is met. Actually, the remedial measure's design should be adaptable enough to allow for modifications as monitoring shows that the treatment needs to be supplemented or as actual conditions are revealed during construction. From further or ongoing monitoring to the drastic of completely rebuilding or decommissioning and demolishing the dam, remedial intervention can take many forms. The risk to the geological and geotechnical environment, the amount of correction necessary, and the feasibility of the correction are some of the criteria that impact the type of treatment required. Remedial procedures include grouting, cutoff walls, upstream impermeable blankets, downstream berm, drainage, reducing the reservoir, seepage monitoring, and seepage control measures (Chugh, 2007).

2.10. Remedial measures for the static stability of embankment dam

Repairing overly steepened embankment slopes, buttressing unstable embankment slopes with additional fill, caulking embankment cracks to stop rainwater infiltration, caulking the U/S slope with a membrane or other seepage barrier, removing and replacing weak embankment material, adding drainage zones, adding toe drains, and rehabilitating existing toe drains are some features that have been used to increase the static stability of embankment dams (Chugh, 2007).

3. CHAPTER THREE: METHOD AND MATERIALS

3.1. Description of the study area

Location and accessibility

The study area is found in the East Bale Zone of Oromia National Regional State between Ginir and Raytu Woreda, located some 20 kilometers before Raytu town. It is situated geographically between 572635E to 590558E and between latitude 6° 55' 42.5356" and longitude 41° 01' 08.0535" in this region.

Access to the project region is feasible via the 400-kilometer Addis Abeba-Asela-Dodola-Robe asphalt road, which is followed by either the Robe-Gasera-Ginir-Raytu all-weather gravel road, which is about 200 kilometers long, or the Robe-Goro-Ginir-Raytu gravel road, which is around 170 kilometers long. Another route branching from Goro and passing via Sof-Omar village connects the Dawe Qachen Woreda capital with the Zone, however it is unpaved after turning left from this road at Sof-Omar until it meets the Ginir-Raytu road.

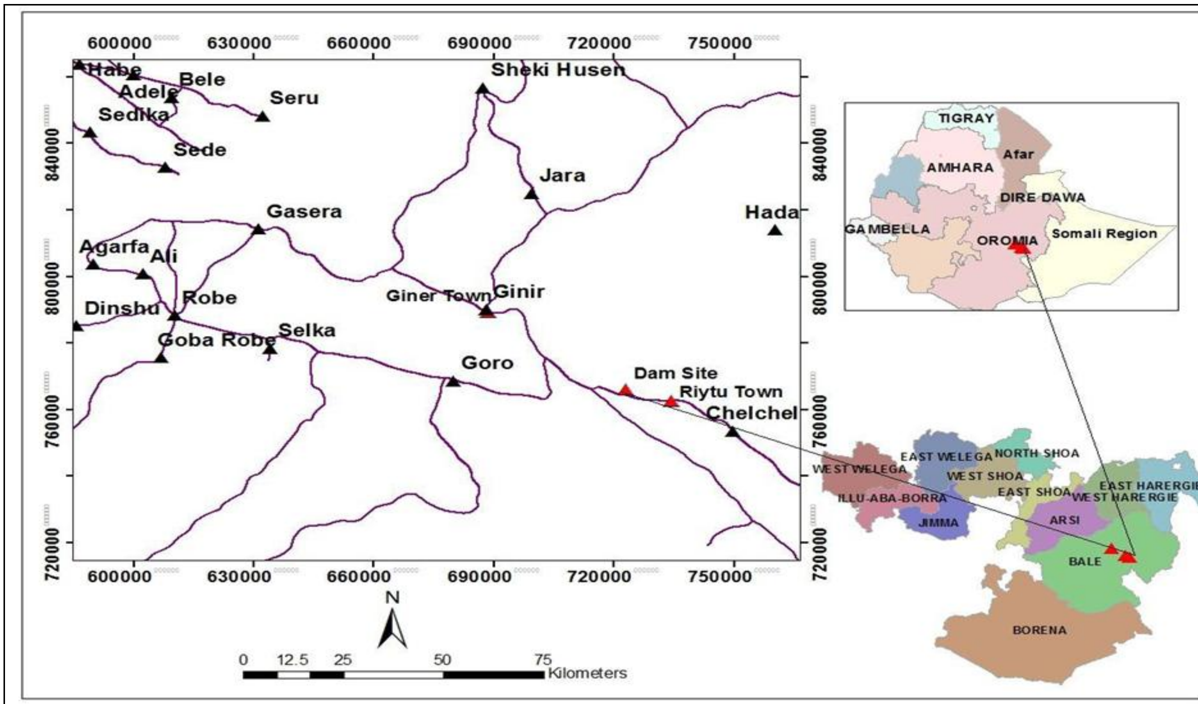


Figure 3.1: location of the study area

3.1. Description of the dam site

Major activities that were constructed under the Chel-Chel dam irrigation development construction project include the following:

Table 3.1: Dam major component

S.N	Description/types of structure	Type/Dimension
1	**Main Dam**	
	Dam Type	Rock fill Dam with central impervious core
	Dam Height	46.5m (1109 m - 1155.5m)
	Dam Crest Length	685.5m
	Crest width	10 m
2	**Coffer Dam**	
	Type	Rock fill dam with central impervious core
	Height	16.5 m
	Crest length	420
	Crest width	12
3	**Bottom outlet construction**	
	Length	155m within the dam body
	Capacity	6.14 m3/s
4	**Spillway**	
	Location	Left Bank
	Type	Un-gated Ogee type
	Design flood (PMF)	1290 m3/sec
5	**Irrigation Intake construction**	
	Length	114
	Size of Irrigation outlet	2.0 m x 2.0 m RC Duct
	Capacity	5 m3/s

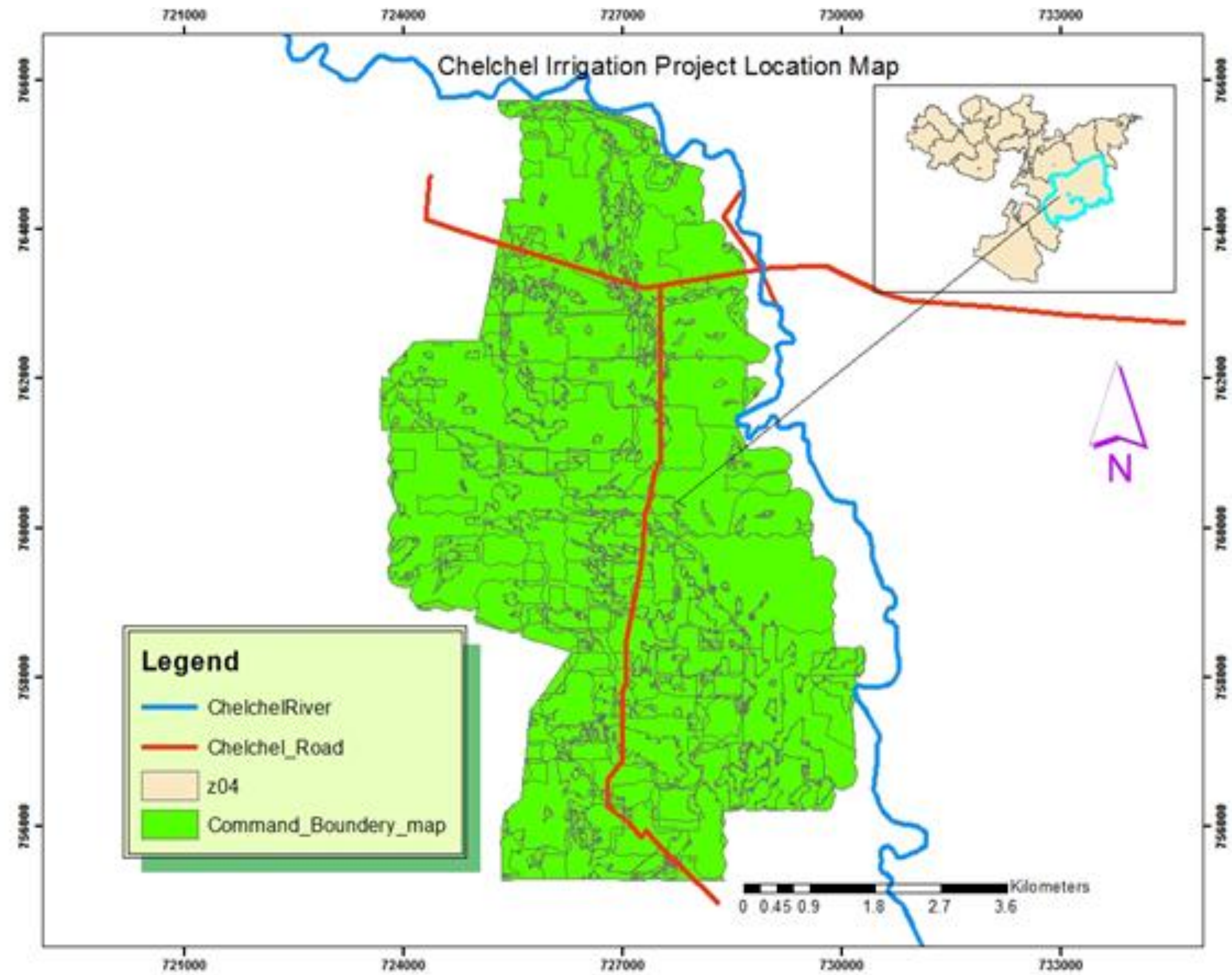
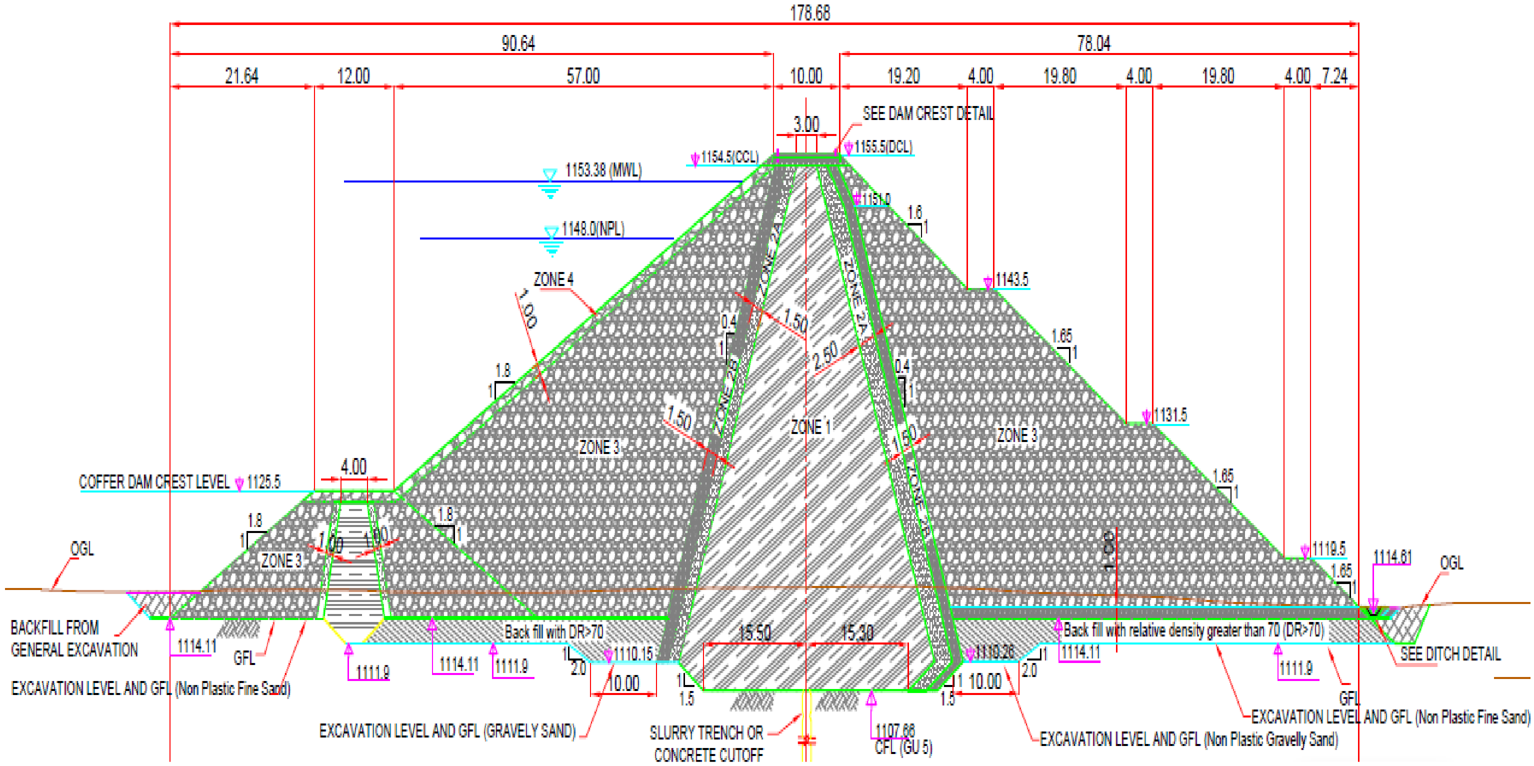


Figure 3.2: Map of the Project Area



Zoning Scheme

Zone	Designation	Material Description
1	Clay Core	Clayey Silt Soil
2A	Fine Filter	Within filter band with filter maximum size 25mm, Percent of fine (<0.075mm) not exceeding 5 % (preferably less than 2%), If any, fines should be non- plastic.
2B	Coarse Filter	Within filter band, 70mm maximum size and no fines (0.075mm)
3	U/S and D/S Shell	Graded crushed rock with maximum size 600mm
4	Dumped rock riprap	Hard, dense rock, maximum size 1000mm
5	Rock Toe Drain	Crushed rock or Natural gravel, size not exceeding 80mm
	Back fill soil replacement	River deposit to be produced by blending and after compaction it should be with DR>70% with fines less than 5%.

Figure 3.3: Dam zoning for rock fill with central clay core

3.2. Topography and Climate condition of the study area

The project area is found in the south-eastern lowlands, of the south-eastern highlands, and associated lowlands of the physiographic subdivision of Ethiopia. It is located close to the foothills of the highlands where a sharp topographic fall separates the lowlands from the highlands that are raised by the thick Tertiary Volcanic Cover. Elevation in the area gradually decreases as one goes in the southeast direction Elevation in the Project area varies between 1217 m to 1112 m above sea level represented characterized by a flat environment, ridges, and a growing river valley. The ridges when observed on the other side are flat-lying showing that they are continuations of the plain whose topography is under the process of development by the forces of nature.

The climate of the area is generally hot with little or scanty rainfall. It is characterized by bimodal rainfall distribution ranging from mid-September to the end of October, and April-May. The nearest and physiographically similar meteorological station of Sof-Omer registered mean annual rainfall of 465 mm the highest precipitation being recorded in October, April, and May.

3.3. Geological setting of the study area

3.3.1. Regional Geology

The rocks belonging to the Gabredarre and Korahe formations of the Mesozoic sediments of Ethiopia, and the Ashangi formation of the Cenozoic Volcanic rock are exposed in and around the project area. The upper Gabredare unit (Jg2) which is limestone with shaley and gypsiferous units is exclusively exposed in the project area and its surroundings except for some patches of basalt overlying the sedimentary rock.

3.3.2. Local Geology

According to the Feasibility Geological and Geotechnical Investigation Report of the project, the project area is exclusively underlain by the Gabredarre formation of the Mesozoic sedimentary rocks of Ethiopia with some patches of the Ghinir basalts overlying them. The dominant geological features are Marley Limestone, Sandstone, Ghinir basalts, Eluvia Deposits, and alluvial deposits.

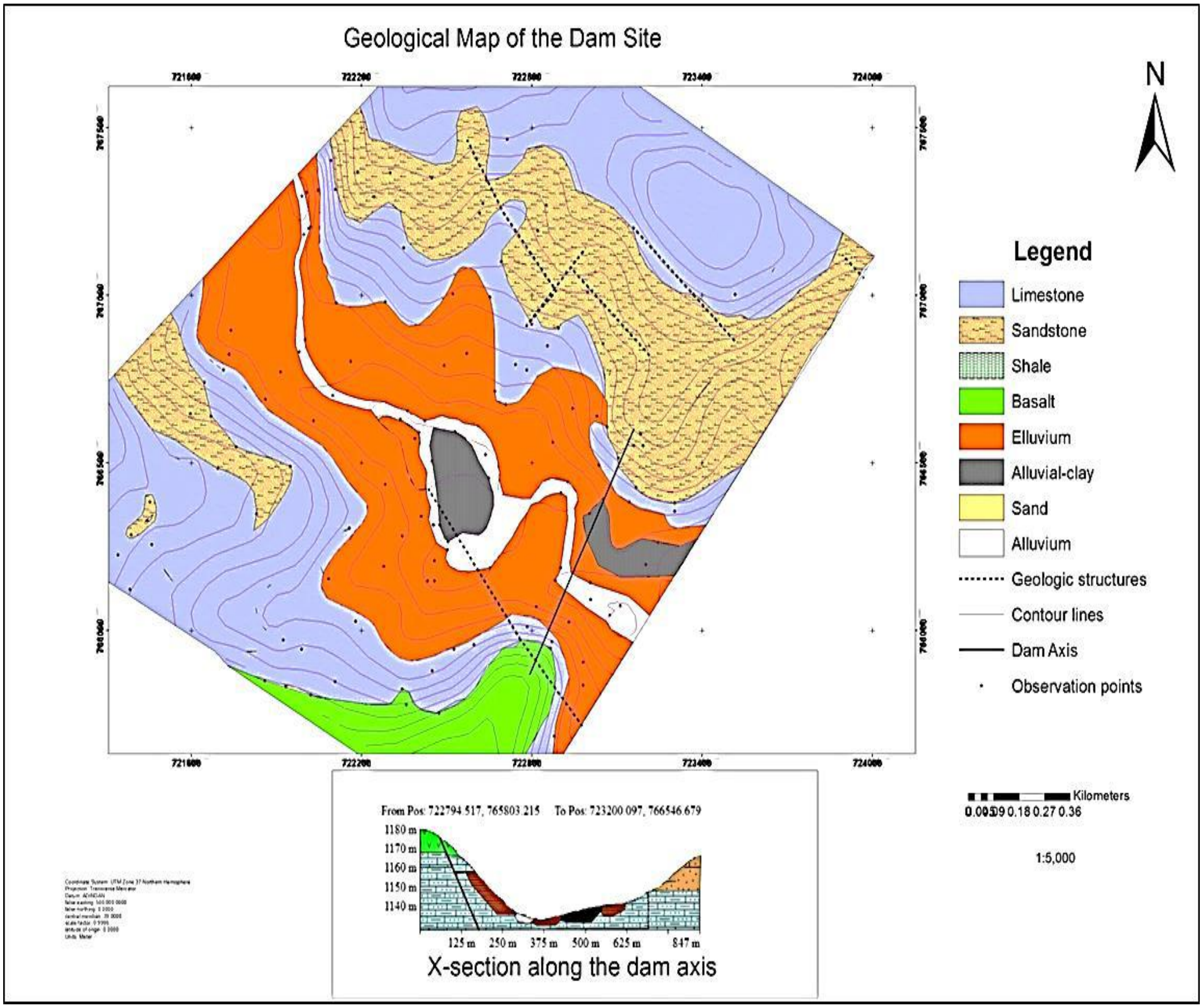


Figure 3.4: Geological Map of Chel-chel Dam Project Area

3.4. Seismicity of the Project Area

An assessment of the seismicity around the site is made based on the seismic hazard map of Ethiopia produced by the Geophysical Observatory of Addis Ababa University. As per this map, the Chel-chel project site lies in Zone-0, which corresponds with low seismic hazard zones.

The important implication here is that the location of the Chel-chel Dam Site is far from the currently active rift margins but can be affected by distant and damaging earthquakes that can be sourced from the rift margin border faults. Therefore, due consideration needs to be given in the design to avert damage to the structure, if such a phenomenon occurs. Besides, even though the site is located in a zone with no seismicity before it may be susceptible to induced seismicity after the water load. This needs monitoring with appropriate instruments before and after the Dam is finalized and filled with water.

The recent seismic hazard zoning map shown above places the project site in the lowest seismic hazard zone concerning other locations in the country, designated with potential ground acceleration (PGA) of 0 to 0.025 (ECO, 2022).

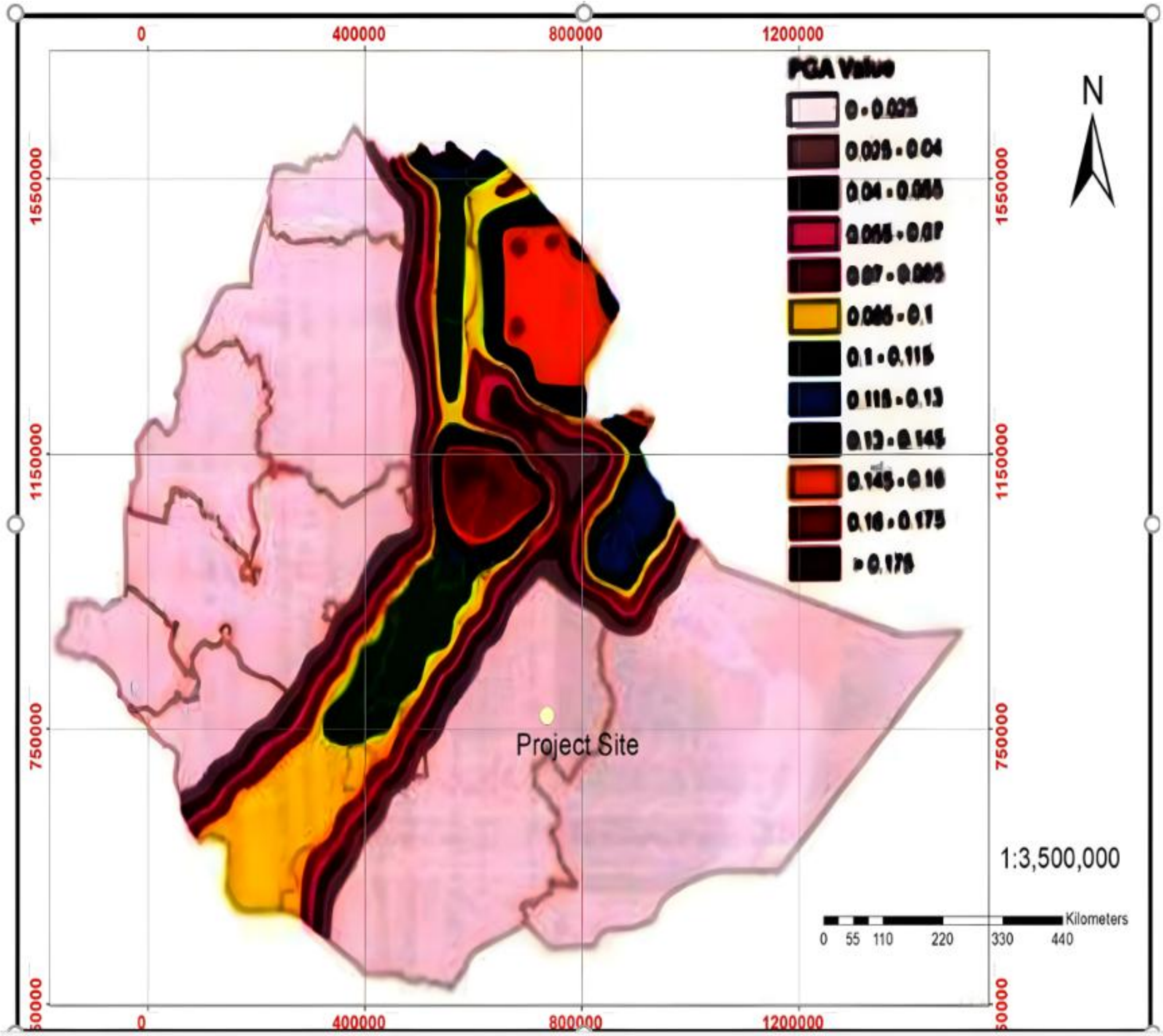


Figure 3.5: Seismic Hazard zoning map of Ethiopia showing the project's location

3.5. Geotechnical Characterization of the Dam Foundation

Based on data from test pits and drilled boreholes along the dam axis, the foundation of the project Chel-chel dam site was identified, and five geotechnical units were subsequently identified (ECO, 2020).

GEOTECHNICAL UNIT-01 (Colluvial Soil): CH: 0+000 to 0+070 (at the right abutment) and 0+200 to 0+680 (at the left abutment). Generally expected to be non-uniform in the abutments hill slopes and form heterogeneous deposits at the bases of the slopes, with low permeability and soft consistency.

GEOTECHNICAL UNIT-02(Alluvial Soil): CH: 0+150 to 0+450. It is a very thick deposit of the site where its reduced level reaches 1105+-3.4 m in the valley bottom. Results showed that this soil deposit is pervious and non-uniform strength both vertically and laterally. Due to formation topography and condition, the deposit is found generally below the groundwater table.

GEOTECHNICAL UNIT-03 (Sandstone): This unit bedded and occasionally intercalated with shale and limestone layers, observed at different layer and along the whole length.

GEOTECHNICAL UNIT-04: (Shale mudstone) The fourth lithological unit identified is highly to moderately weathered sandstone and limestone inter bedded with decomposed shale (GTL-4), the top part of the two abutments sandstone intercalated with decomposed while the lower elevation of the abutments and dam foundations the intercalations of weathered limestone with decomposed shale. Generally, the mudstone unit is formed from prolonged time weathering and decomposition of shale-sandstone-limestone unit. This mudstone unit is the dominated lithological unit comprises the dam foundations, intake and spillway foundation and right abutment. The mudstone unit is characterized by variegated color (light pink, brown, and whitish yellow) since it is a package of shale-sandstone –limestone unit. The units as package are very compacted and stiff. The average falling head water test result coefficient of permeability of K (cm/s) =1.97E-06 and this indicate the mudstone units are Semi-Pervious degree of Permeability. Laboratory test of mudstone rock samples revealed that bulk unit weight of 20.6 to 21.5kN/m3, and uniaxial compressive strength in the range of 4.56 to 1.76MPa. Modulus of deformation in the range of 548MPa.

GEOTECHNICAL UNIT-05: This unit observed by intercalating at both abutments with shale and sandstone and the continuous bed limestone found below 1080 elevation. This lower limestone is characterized with slightly to fresh degree of weathering, slight fracturing and very strong in strength. Rock mass rating has revealed fair quality (RMR =55). The Laboratory tests of the limestone unit shows unit weight of 25.68 to 28.05kN/m3, uniaxial compressive strength in the range of 31 to 72MPa. Estimated cohesion between 1.5MPa, angle of internal friction ranging from 17 degrees, and modulus of deformation in the range of 4820MPa.

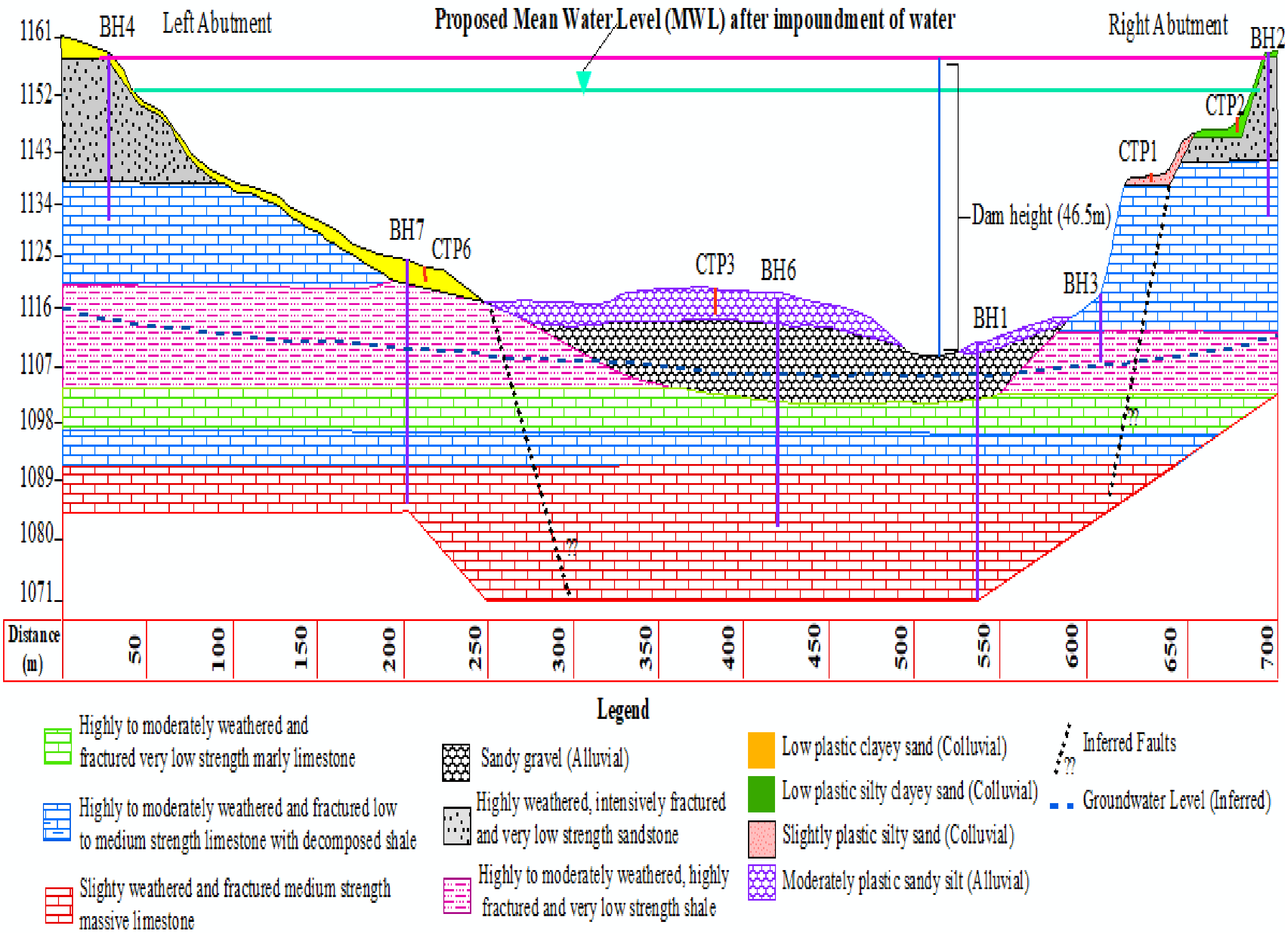


Figure 3.6: Geotechnical Section and Engineering Properties along the Dam Axis (ECO, 2020)

3.6. Materials

Different materials will be used during the implementation of this study;

- ✓ A topographic and geological map will be used to aid the site data collection
- ✓ Garmin GPS, Schmidt Hammer, sampling bag, and other field equipment.
- ✓ Sand cone and digital balance
- ✓ HP laptop computer and a Samsung digital camera
- ✓ Different software (Geo-Studio, Arc GIS 10.1, Global Mapper, MS EXCELL)
- ✓ Printer and scanner
- ✓ Other stationaries

3.7. Methodology

Initially, data was collected directly from the Chel-chel dam site and the corresponding office. This included visual inspections and observations related to the dam's failure. A digital camera was used for documentation and collecting previously sampled laboratory tests. The tests performed included Grain Size Analysis, Atterberg Limits, and the unconfined compression test. The findings of these tests were utilized to classify the samples based on the Unified Soil Classification System.

Following this, secondary data were collected from sources such as the design documents and the Ministry of Irrigation and Lowlands, as well as the Engineering Corporation of Oromia. The analyzed laboratory data, along with the secondary information, were then input into GeoStudio-2018. Thereafter, various scenarios regarding seepages and slope stability were analyzed, which were based on the laboratory results and the design documents.

As a result of this analysis, a thorough discussion was conducted for each outcome and the dam's failure was assessed by making comparisons. Finally, conclusions and suggestions were made based on the results acquired.

The methodology for this thesis work is illustrated in the following Fig.

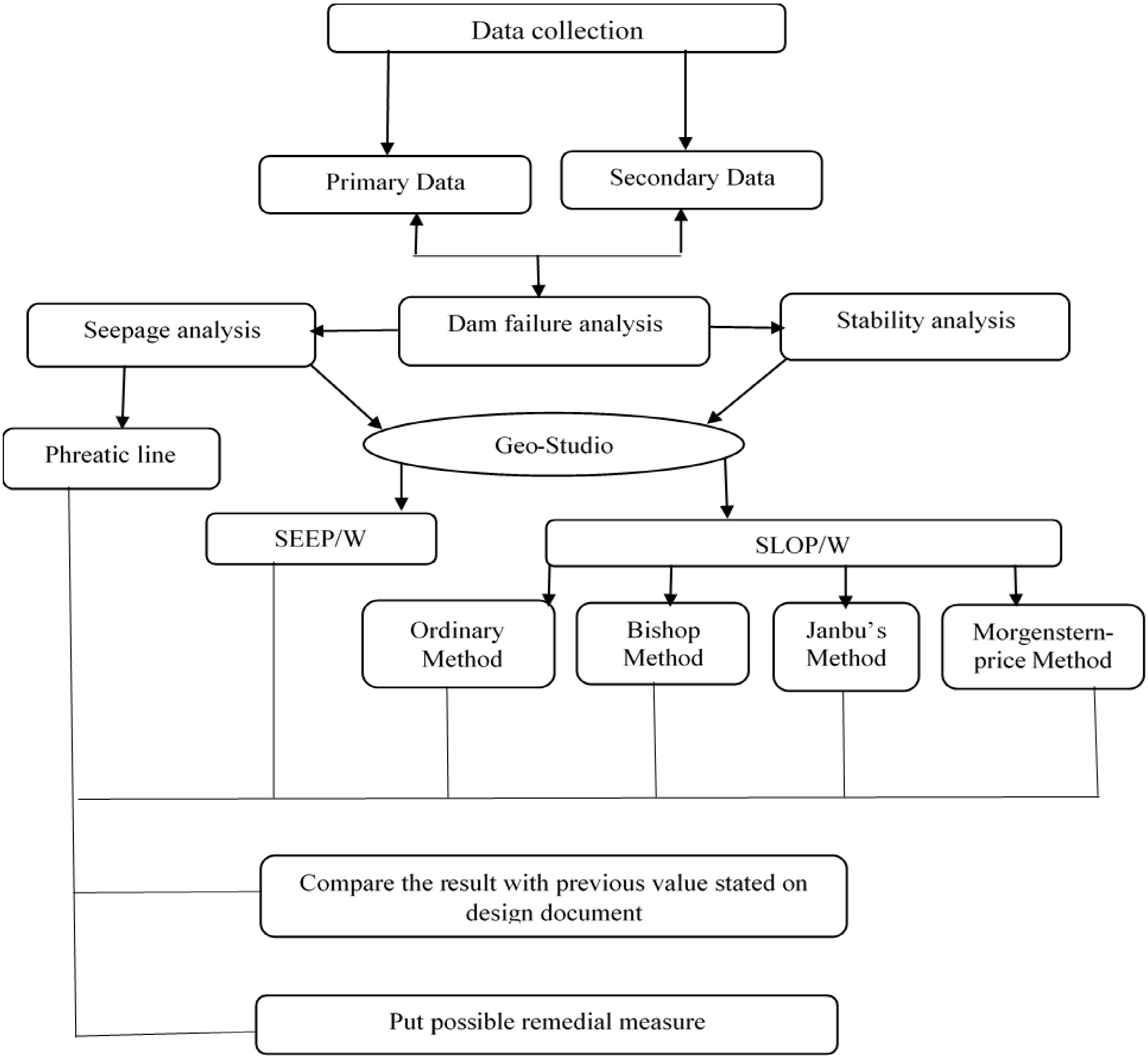


Figure 3.7: Flow chart, conceptual formwork

3.8. Data Collection

3.8.1. Primary Data

The primary data was gathered from the Chel-chel dam site, including the following information, and then verified by comparing it with the secondary data collected from the Ministry of Irrigation and Lowlands and Engineering Corporation of Oromia (ECO). The data collection methods involved;

1. Capturing an image of the dam body and its appurtenant structures.
2. Conducting interviews with contractors and consultant engineers.
3. Collecting Construction material laboratory tests conducted by the contractor, both in the field and at the office level.

3.8.2. Secondary Data

Secondary data were collected from the Ministry of Irrigation and Lowlands and the Chel-chel Dam site office. The data are;

- ✓ Detail dam design report and laboratory test result
- ✓ Material properties of the dam’s
- ✓ Essential input data for the Geo-Studio software such as the geometrical design of the dam, Geotechnical parameters of the embankment, foundation materials from laboratory testing, Geological reports, and reservoir interfaces.

Figure 3.8: Clay core material compaction test conducted

Figure 3.9: Clay core section excavation and preparation

The dam profile and the characteristics of the building materials and foundation are the primary data extracted from the design document in order to accomplish the goal of this study.

a) **Dam profile:** the dam's height, width at the top and bottom, u/s and d/s slope, normal, maximum, and minimum water levels, horizontal filter length, and cut-off depth.

b) **Material properties: -** the results of laboratory tests on the foundation, shell, and core (dry, saturated, and submerged unit weight, soil cohesiveness, internal friction angle, and soil permeability/hydraulic conductivity).

The foundation and construction materials' physical and shear properties were used to examine the test findings and conduct a stability study.

Table 3.4: Summary of embankment fill and foundation material characteristics (ECO, 2020)

Zone	Function	Unit Weight (KN/m3)	Permeability (m/s)	C'(Kpa)	F'(0)
1	Core	15.6	1.77*10^-0.7	23.94	18.06
2A	Fine filter	19	5*10^-5	0	30
2B	Coarse filter	19	5*10^-4	0	32
4	Rock fill	22	0.01	0	38
5	Riprap	22	0.01	0	40
	Geological unit one	18	1*10^-6	25.65	10.48
	Geological unit two	18	5.32*10^-6	0	38
	Geological unit three	18.8	1.61*10^-8	186	35.5
	Geological unit five	23	1.61*10^-8	135	24.4
	Concrete cutoff	24	1*10^-9	10	38
	Geological unit seven	26.1	1.61*10^-8	285	22

3.9. Data analysis

3.9.1. Analysis of Seepage

I. Analysis by Seep/W software model

The GEO-STUDIO application, SEEP/W, employs a numerical model to mathematically replicate the actual physical process of water traveling through particulate matter. Numerical modeling is completely mathematical, as opposed to full-scale field modeling or scaled physical modeling in a lab. The software is a very powerful calculator, and whether or not the tool produces useful and relevant results depends on the user's guidance. The user's ability to understand the input and interpret the results is what makes it such a powerful tool. The program can only do very complicated calculations that are otherwise impossible for humans to do (G. U. s. Manual, 2012).

Darcy's law, which characterizes the rate of change in volumetric water content over time as the sum of the rate of change of flow in the x and y directions plus the externally applied flux, serves as the foundation for the SEEP/W fine element (Krahn, 2004).

$$\frac{\partial}{\partial x}\left(kx\frac{\partial H}{\partial x}\right)+\frac{\partial}{\partial y}\left(ky\frac{\partial H}{\partial y}\right)+Q=\frac{\partial \theta}{\partial t} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots 16$$

Where: H- the total head

Kx and Ky - the hydraulic conductivity in the x and y direction

Q - The applied bounder flux

θ- The volumetric water content and,

t- time

SEEP/W General Considerations and Analysis Procedures

The following input data were used in SEEP/W software model

a. Types of Analysis

There are two fundamental types of finite element seepage analyses: transient and steady-state. Numerous additional limitations and specifications may apply to each basic type (Krahn, 2004). The analysis type employed in this study is steady state since the total amount of seepage passing through the dam body and foundation does not change over time.

1. Importing dam Geometry and draw the region

The dam's geometry is determined by its profile, which includes its height, material, upstream and downstream slope, top and bottom widths, maximum, normal, and minimum water levels, and horizontal filter length.

There are two techniques for importing the dam's geometry: direct drawing and the dam profile. The second option imports the dam profile from AutoCAD.

The second method was used to import the geometry. - The areas of the closed polygon are defined by connecting points and are used to identify locations with different material properties and conditions.

2. Assign resources

Understanding the significance of soil properties and their influence on the results produced is crucial. The subsequent step in creating a numerical model of SEEP/W involves designating the types and properties of the materials in the area based on specific criteria.

- Click on MATERIALS using the KEYIN function.
- Add a new material, give it a name (foundation, shell, or core), and then give it a color.

Choose an item from the material model dropdown menu.

A. Saturated Only: Use this option if a domain that will stay saturated throughout the simulation is the subject of a steady state analysis.

B. Saturated/Unsaturated: Use if you anticipate unsaturated zones. For this thesis, this was utilized.

3. Boundary functions

The SEEP/W formulation is capable of handling a wide variety of boundary conditions.In a steady-state analysis, all boundary conditions are expressed as either constant flow values or set head (or pressure) values. However, the boundary conditions in a transient analysis might also be time-dependent or react to flow amounts entering or leaving the flow regime. SEEP/W can handle a number of distinct border functions. Krahn (2004).

4. Locate the flux section.

For a steady state or transient study, the instantaneous seepage volume rate that flows across a user-defined segment was calculated using SEEP/W. Calculating the overall flow across a vertical segment of the element is the goal. Because they may be drawn anywhere you wish to know the flux, flux sections can be used in a variety of ways.

5. Verify and optimize the data.

Before being solved, every analysis and piece of input data is automatically checked, and any mistakes are noted on the verification tools. Check the tool for mistakes in input data, regions without data points, and overlapping geometry lines.

6. Analysis the result

The final phase of SEEP/W was to solve the challenges, analyze the outcomes, and make judgments.

II. Analysis by phreatic line

The phreatic line, also known as the seepage line or saturation line, is the line at the upper surface of the seepage flow when the pressure is atmospheric. The seepage line determines how much water passes through the dam's body and foundation. The study is now complete for both homogeneous and zonal embankment dams. However, the Chel-chel Dam is a zonal dam with a horizontal filter. The available data for predicting seepage from the design document are the dam's shape and the permeability coefficients of the core and shell materials.

3.9.2. Slope stability analysis by SLOP/W

An embankment slope's stability is determined by its height (H), slope angle (β), and shear strength factors like cohesiveness (C) and friction angle (ϕ). According to Sivakugan (2009), of these three characteristics, height and slope angle decrease stability as the amount increases, however, increasing shear strength parameters results in a more stable slope.

When the slope configuration and soil parameters are available, engineers may easily do computerized slope stability analysis. But choosing the right slope stability analysis method is not a simple task; in order to understand the failure mechanism, which dictates which slope stability method should be utilized in the study, field circumstances and failure observations must be gathered. In order to properly investigate slope failures and assess the reliability of the analysis outcomes, it is essential to examine the theoretical foundations of each slope stability method (Tatewar & Pawade, 2012).

The stability of the Elidar earthen dam's embankment was investigated using the limit equilibrium approach of Geo-Slope International Ltd.'s SLOPE/W-2012 program using linked input data from SEEP/W. The analysis's application of an analytical technique to evaluate the static stability of an embankment is in line with information from dam cross sections, soil testing, and anticipated failure modes.

In order to ascertain the factor of safety for different critical slip surfaces, stability analyses were conducted. The stability of the embankment's upstream and downstream slopes is examined for the most severe or critical loading scenarios that could arise over the dam's lifetime. Usually, these loading circumstances consist of: (1) Construction End; (2) Steady-State Seepage; and (3) Rapid (or Sudden) Drawdown.

I. Procedures for Slope Stability Analysis and General Considerations

The most popular limit equilibrium techniques, including Ordinary, Bishop's Simplified, Janbu's Simplified, Morgenstern-Price, and Spencer methods, are used to calculate the key safety factors. The following steps are taken into consideration to complete the slope stability study.

a. Defining Slip Surface for Circular Failure Model

A slip surface command was defined following the assignment of the material inputs and SEEP/W linked pore water pressure. The entry and exit approach was chosen from among these various widely used techniques to determine the slip surface for the circular failure mode. By indicating the presumed area of the surface where the slip surface would enter and exit, the entry and exit command enables the user to easily identify slip surfaces and is comparatively accurate when compared to the other commands.

b. Safety Criteria

The required criterion by safety regulations or codes issued by authorized agencies should be met by the safety evaluation of embankment dams. Both the USACE (2003) and BDS (1994) criteria are regarded as the gold standard for their wide range of validation in the current studies among the several dam safety regulations. The most widely used safety criteria standards (USACE, 2003; USBR, 2011) and (BDS, 1994) are used to validate the study's safety factor, as indicated in the table below, which presents the safety criteria of stability factor of safety for various operation scenarios.

Table 3.5: Safety criteria standard of various enterprises

Agency	Loading condition	Stress parameter	FoS
USACE, 2003	End of construction	Total and Effective	1.5
	Steady seepage condition	Effective stress (drained)	1.3
	Rapid drawdown	Total and Effective	1.2
USBR, 2011	End of construction	Total and Effective	1.3
	Steady seepage condition	Effective stress (drained)	1.5
	Rapid drawdown	Effective stress (drained)	1.3
NRCS, 2005	End of construction	Total stress for impervious and effective for pervious layer	1.4
	Steady seepage condition	Total and Effective stress(drained and un-drained)	1.5
	Rapid drawdown	Total and Effective stress(drained and un-drained)	1.2
BDS, 1994	End of construction	Total stress (un-drained)	1.2 to 1.3
	Steady seepage condition	Effective stress (drained)	1.3 to 1.5
	Rapid drawdown	Total stress (un-drained)	1.2 to 1.3

c. Calculating safety factors

The slope stability study carried out by the limit equilibrium method employing Morgenstern-price analysis processes serves as the foundation for the factors of safety criteria that are described in this thesis work. Because limit equilibrium methods can more easily handle iterative procedures, Morgenstern-Price methods can produce mathematically more rigorous formulations that meet all equations of statics and incorporate all in-terslice forces. Because Table 4 summarizes the critical criteria and minimum suggested safety factors for various typical loading conditions for each zoned earthen dam category, slope stability analysis is therefore carried out in a variety of scenarios until a stable slope is reached.

Morgenstern –price method

This method considers the moment equilibrium for each slice in both circular and non-circular slip surfaces, in addition to the normal and tangential equilibrium. The following is how the equations are expressed for a slice of minuscule thickness: Equations 17 and 18, respectively, show the factor of safety in relation to force equilibrium:

$$Ff = \frac{\sum(c'l+(p-ul)\tan \emptyset\prime)cos\emptyset}{\sum psin\alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots17$$

And the factor of safety with respect to moment equilibrium is;

$$Fm = \frac{(c'l+(p-ul)\tan \emptyset\prime)}{Wsin\alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots18$$

The most critical sliding surface was where the most homogenous embankment dam collapses occurred, resulting in a lower factor of safety value. The sample is sheared along a horizontal plane in a direct shear test. The failure plane is horizontal as the name implies. The normal stress (σ) on this plane is calculated by dividing the external vertical force by the sample's area. The external lateral force divided by the sample's area is the shear stress at failure (Murthy, 2003).

The Chel-chel earth-fill dam's static slope stability under typical loading circumstances was examined using Slope/W's Geo Studio program. The following is a list of actions used to guarantee slope stability.

1. **Defining the geometry**

Before generating a slope stability problem, it is a good idea to design the problem dimensions. This sketch serves as a helpful reference for sketching the problem's geometric components.

2. **Specify the analysis methods**

Slope stability in the Chel-chel dam instance was examined using a variety of methods, including Ordinary, Bishop, Janbu, and Morgenstern's Price. Morgenstern's Price technique has been used to derive the FOS from the design document.

3. Specify slip surface analysis options

An important step in carrying out a slope stability study is figuring out the location and form of the critical slip surface. By calculating trial slip surfaces that are mostly within the user's control, SLOPE/W determines the critical slip surface. The software calculates the safety factor for each test slip surface, and the slip surface with the lowest safety factor is considered essential.

The study's analysis choices include entry-exit and grid and radius for the Fellenius method, which are utilized to determine the location of the slip surface.

4. Define Soil Properties

The criteria for Mohr-Coulomb strength Soil strength can be characterized by C (cohesion) and Ø (phi). Depending on the pore-water pressure conditions, the parameters might be total or effective. To indicate undrained strengths, set Ø to 0.

5. Solve and analyze result

The SLOPE/W solution function calculates the safety factors in an analysis. The Slope/W software calculates the factor of safety by comparing the slip surface's shear stress and shear strength. The analytical findings reveal the minimal safety factor and essential slip surface.

4. CHAPTER FOUR: RESULT AND DISCUSSION

4.1. Dam Seepage Analysis

The basic challenge in lime-stones foundation area is mainly seepage and at the presence of more seepage it aggravates the solutioning process resulting structural instability. Therefore, based on the remedial practice and geologic investigation, the seepage amount numerically computed. The computation was undertaken with and without the seepage control structure. The first was done to assess the requirement of seepage reduction to attain the required service and stability. For this analysis GeoStudio 2018 was utilized.

The model was defined based on the geological cross section, and the existing dam cross section from drawing Album, and the hydrogeological parameter was adopted from packer test result. From the drawing Album cross section at chainage 0+40, 0+200 and 0+600 was selected for evaluation which believed to be representative. Accordingly, for with and without seepage reduction structure was defined as it was shown in the figure below at the critical or maximum section can also can be referred.

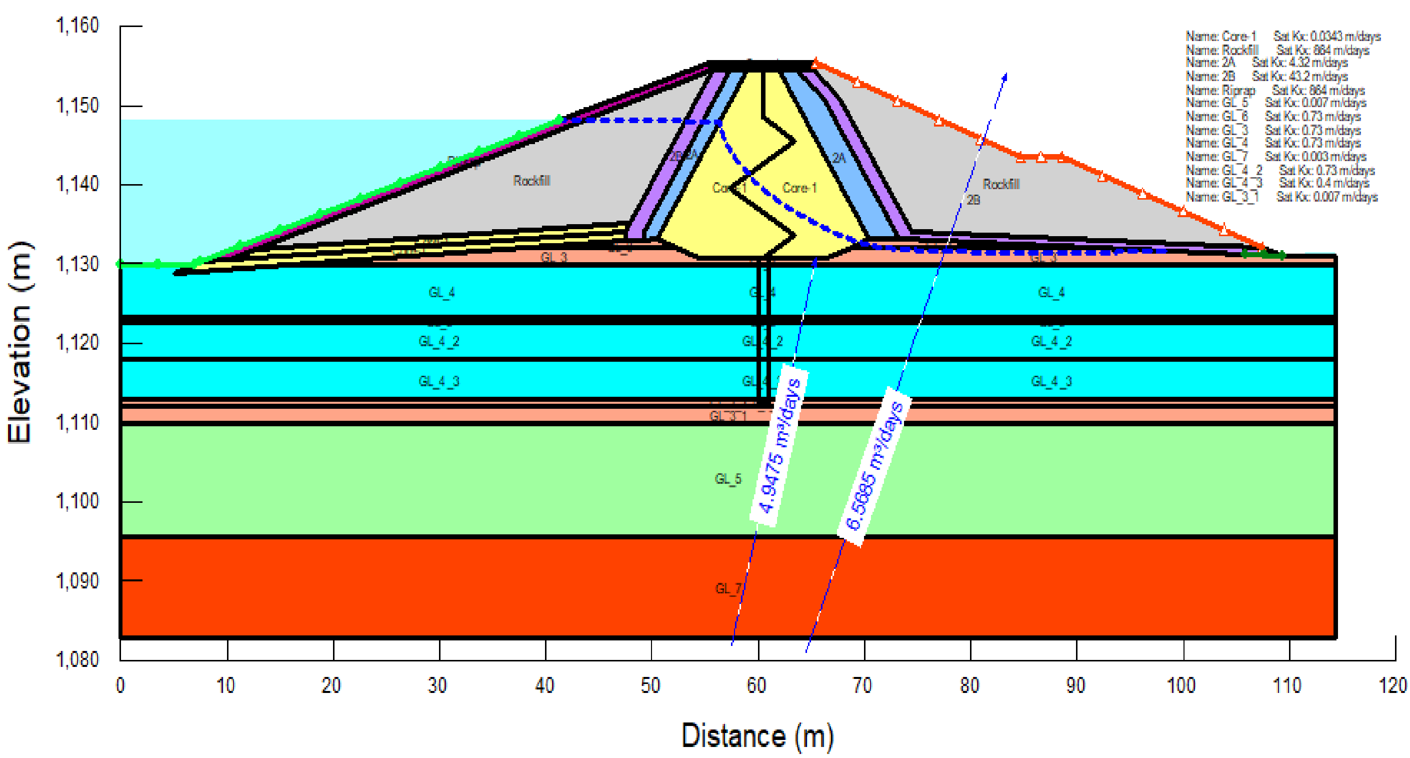


Figure 4.1: Selected, at Chainage 40 without Treatment

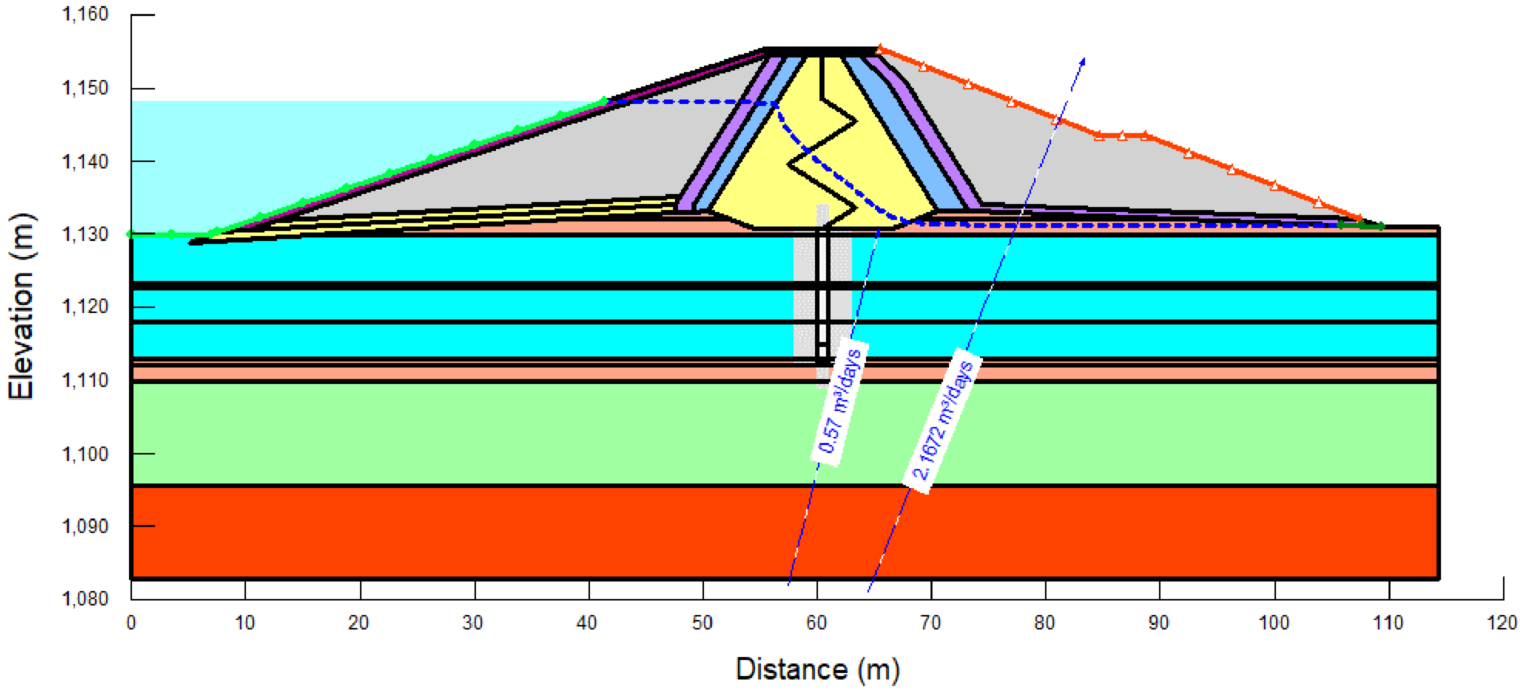


Figure 4.2: Selected, at Chainage 40 with foundation Treatment

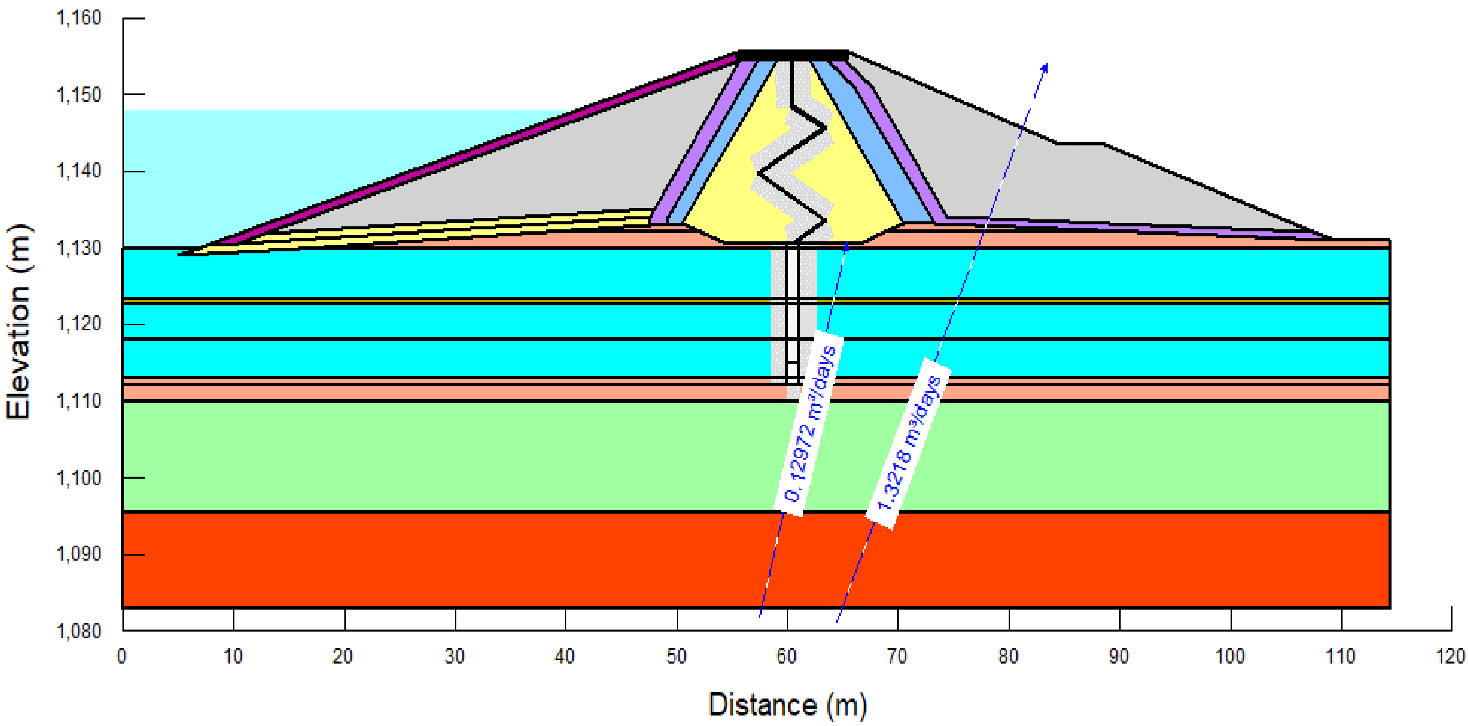


Figure 4.3: at Chainage 40 with further Foundation and Embankment Treatment

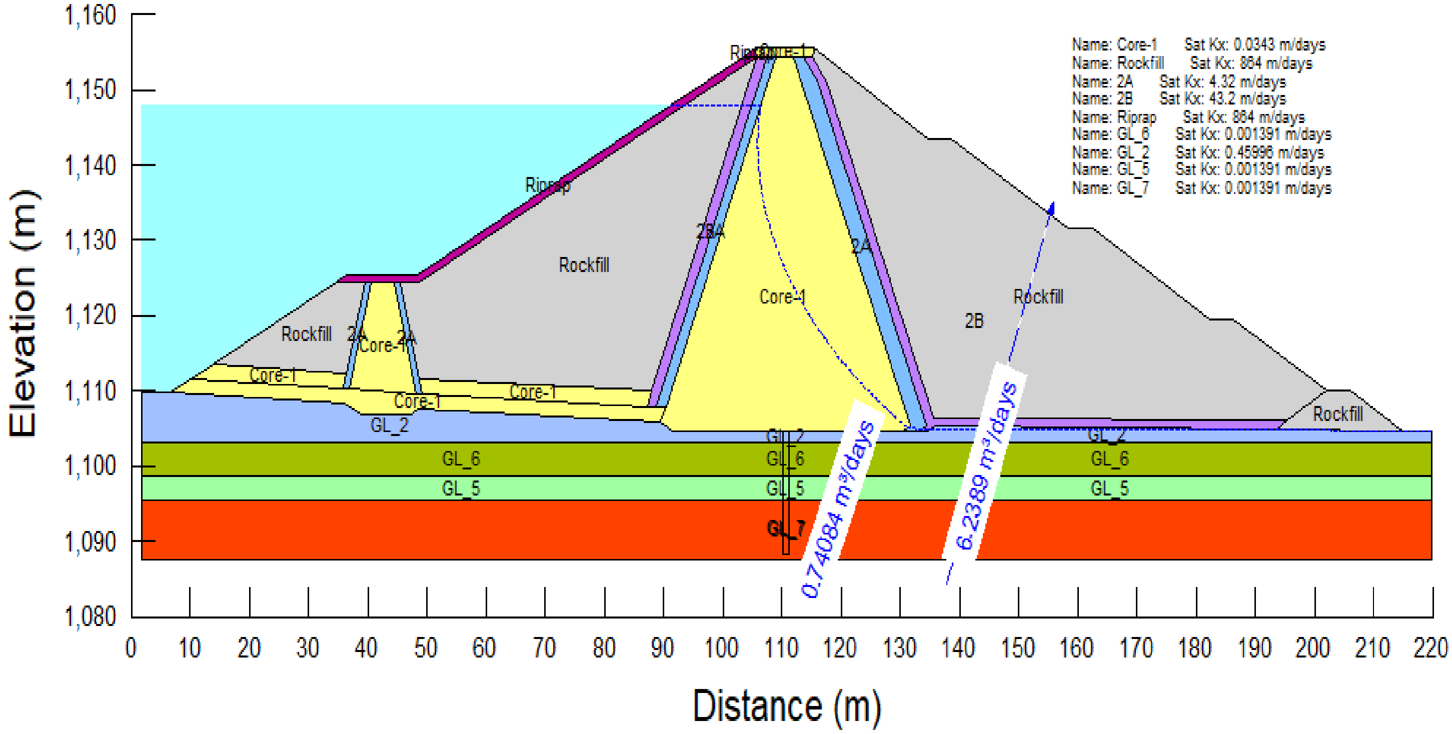


Figure 4.4: Selected, at Chainage 200 without Treatment

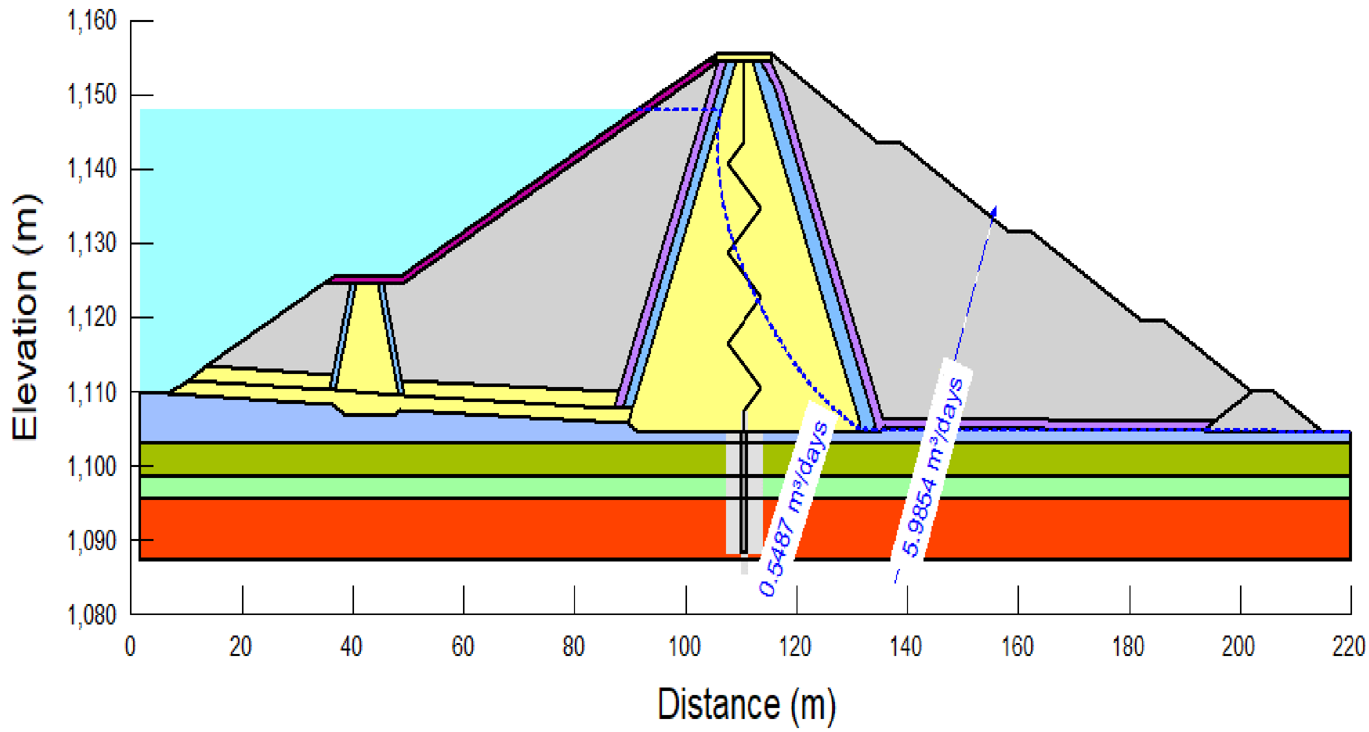


Figure 4.5: Selected, at Chainage 200 with foundation Treatment

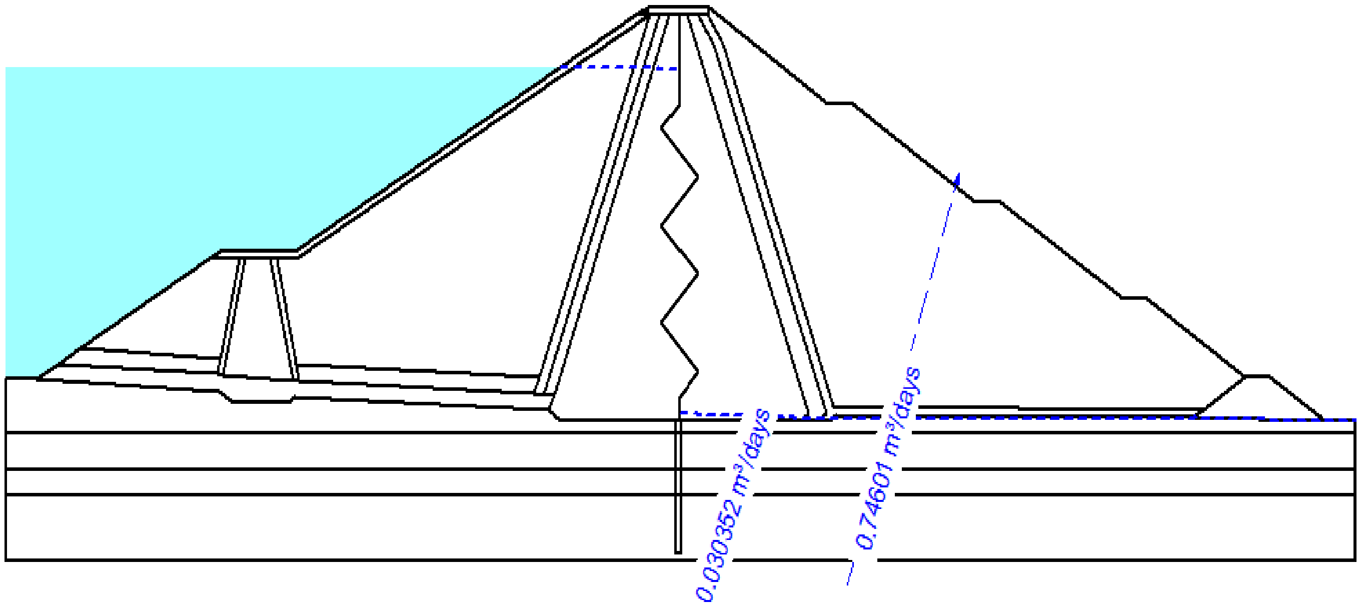


Figure 4.6: at Chainage 200 with further Foundation and Embankment Treatment

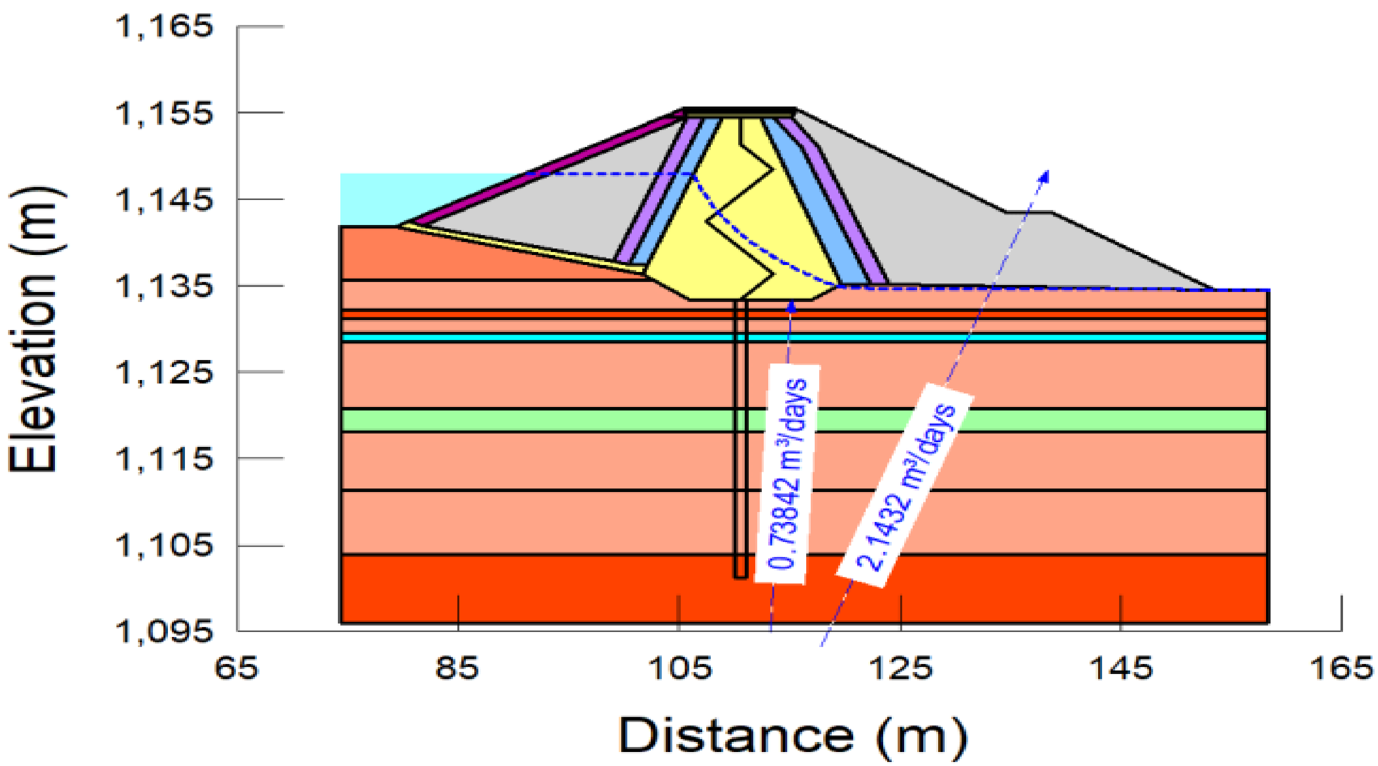


Figure 4.7: Selected, at Chainage 600 without Treatment

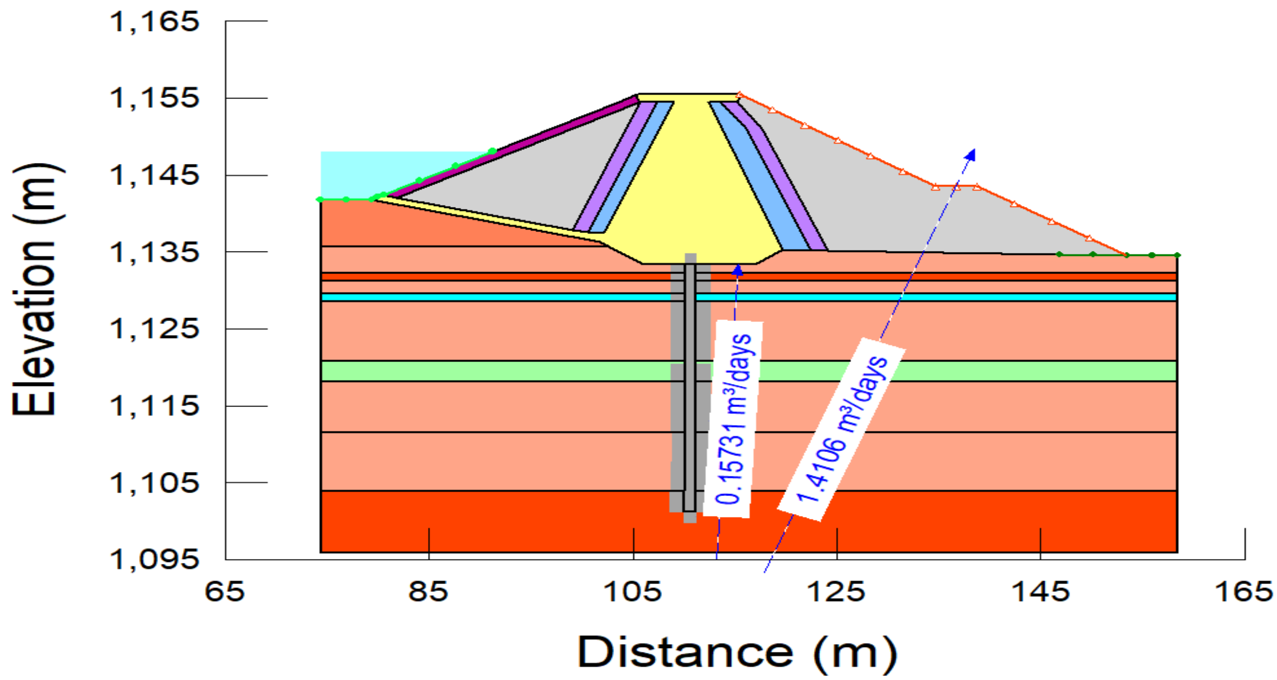


Figure 4.8: Selected, at Chainage 600 with foundation Treatment

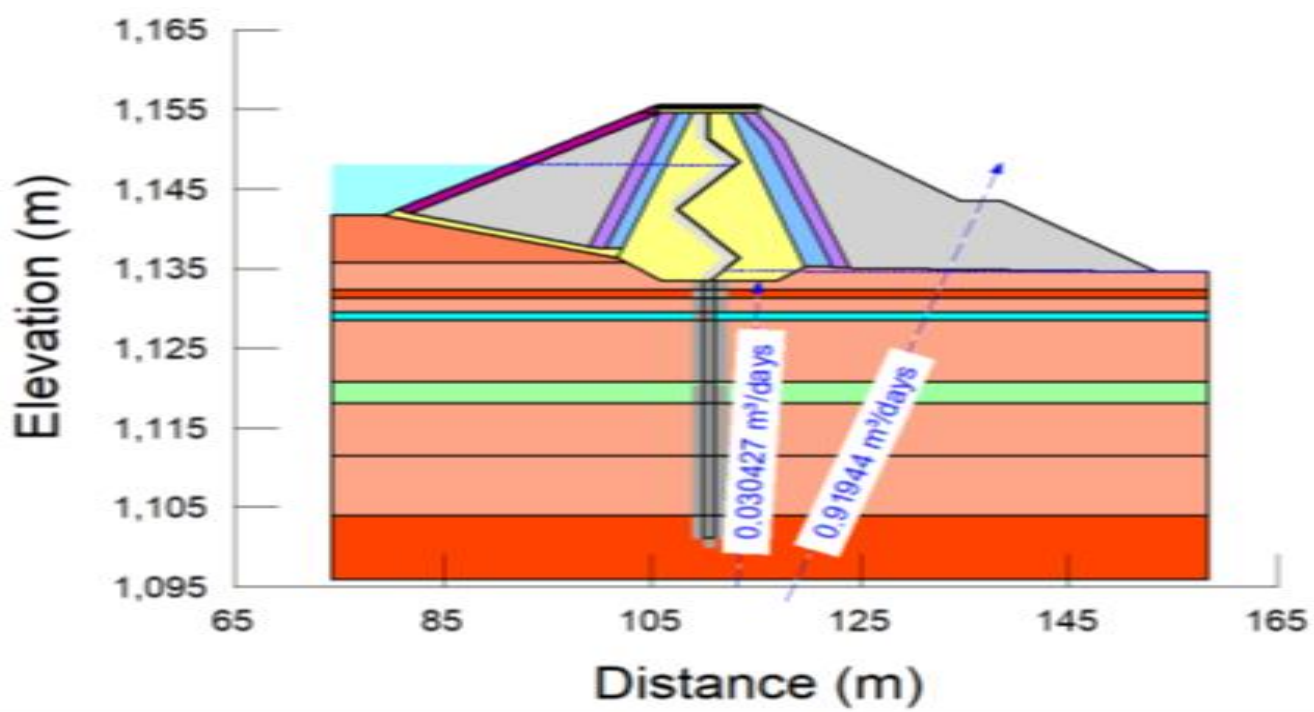


Figure 4.9: at Chainage 600 with further Foundation and Embankment Treatment

The model analysis result shows without treatment the foundation releases 2.14 to 6.56 m3/day per dam length at different section but by providing foundation treatment the seepage will be reduced to 5.98-1.41 m3/day/ dam length. Further treatment on the embankment the seepage will be reduced to 1.32 to 0.75m3/day/ dam length. The analysis also indicated the need of improvement of the filter and drainage system.

Considering the maximum seepage 6.56 m3/day/dam length, for 500m dam length the seepage amount will be 37.96 litter per second this value as compared with the above constructed in similar lime stone area the amount was relatively small without treatment but since any seepage reduction increases the stability and availability of water, especially in such unfavorable foundation any treatment which enables reduction was adopted. Accordingly, it was recommended to construct a seepage barrier of soil slurry trench/ concrete cutoff in the central foundation section and the reduction was evaluated.

Table 4.1: Summary of seepage analysis a different selected section

Chainage	**Treatment**		
	Non	**Foundation**	**Further treatment on Embankment**
40	6.56	2.16	1.32
200	6.23	5.98	0.746
600	2.14	1.41	0.92

Generally, the foundation and embankment seepage of Chel-chel rock fill dam can be controlled significantly with the provision slurry trench and concrete cutoff curtaining of the embankment which is consistent with the current foundation treatment in the limestone foundation moreover during the construction of slurry trench cutoff, the existence of caves and any sinkholes may be prevails and relevant treatment will be adopted.

4.2. Dam Slope Stability

As a result of increased general and cutoff foundation due to additional excavation the structural of the dam has increase and thus it's necessary to recheck the slope stability analysis of the dam in line with the foundation levels and foundation materials. For impervious representative value was considered among the different samples. Accordingly the parameters adapted for the analysis are shown in the table below. The analysis done at the critical section i.e.at selected chainage and the foundation materials indicated in the table are representative formations of the Chel-Chel dam foundation.

Table 4.2: Summary of embankment fill and foundation material characteristics used in seepage and stability analysis (Zoned rock fill dam with central clay core) source MILLS, 2020

Zone	**Function**	**Unit Weight (KN/m3)**	**Permeability (m/s)**	**C'(Kpa)**	**ϕ'(o)**
1	Core	16.0	7.76*10-7	16	20
2A	Fine filter	19.0	5*10^-5	0.0	30
2B	Coarse filter	19.0	5*10^-5	0.0	32
3	Rockfill	22.0	0.01	0.0	38
4	Riprap	22.0	0.01	0.0	40
	High plastic silty clay	18.0	2.95*10^-7	48	20
	Plastic clayey silt	20.0	3.05*10^-7	75	20
	Non plastic fine sand	16.0	2.39*10^-6	0	28
	Non plastic gravely sand	16.0	5.32*10^-6	0	32
	Foundation granular back fill	19.0	1*10-5	0	34
	Geological unit four	18.0	7.9*10-6	200	19
	Geological unit five	18.0	5*10-5	212	28
	Geological unit seven	22.0	1.61*10-8	367	42

4.1.1. Slope Stability Analysis

Typical slope stability analysis results of for both static and with earth quake loading conditions. The minimum factor of safety is presented with four method of analysis i.e. Morgenstern-Price, Ordinary, Bishop and Janbu. However for my case the Morgenstern-Price method is selected which accounts both the force and moment equilibrium.

I. Steady state analysis

The steady state slope stability analysis has been checked for the downstream slope which is expected to be the worst case compared to the upstream face. The hydraulic conductivity values indicated in the table above are used as input SEEP/W of Geo studio 2018 to generate the water table and pore water pressure required for the slope stability analysis. The shear strength parameters shown in the table are used for slope stability analysis using SOPE/W of Geo studio 2018 package. The analysis is made when the water level is at Normal Pool Level (1148.00m).

The minimum computed factor of safety for the downstream slope when the reservoir water is at normal pool level is found to be 1.710 for no earthquake loading

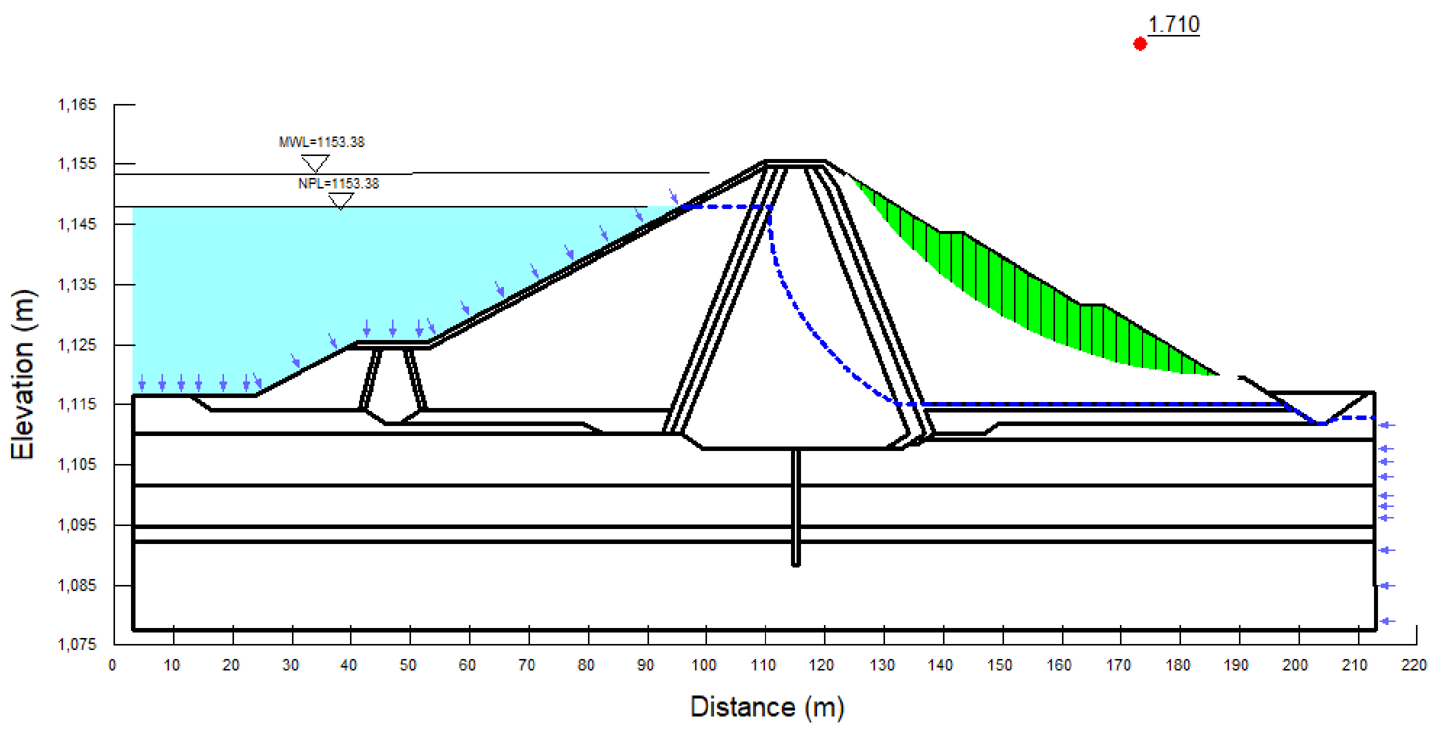


Figure 4.10: Model output for steady seepage when the water level is at full reservoir level without earth quake loading condition

II. During/End of construction

In non-free-draining materials, stability was computed during and after construction using drained strengths. For materials that drain slowly, the following two alternatives are used: total stress analysis with undrained strengths and zero pore water pressure or effective stress analysis modeling a partially saturated condition with pore water pressure.

For the case of Chel-Chel dam, the second alternative is used, with pore water pressure during construction being higher than at the end of construction. The table below summarizes the values of the pore water pressure ratio Ru used for the stability analysis during and at the end of construction conditions.

Table 4.3: Pore water pressure ratio Ru values for construction conditions

Material Zone	**Porewater pressure ratio Ru during construction**	**Porewater pressure ratio Ru at the End of construction**
Impervious Core	0.40	0.35
Filters	0.10	0.10
Rock Fill	--	--
Riprap	--	--
Highly Plastic Silty Clay (Foundation soil)	0.40	0.35
Plastic Silty Clayey (Foundation soil)	0.35	0.30
Non-plastic fine sand (Foundation soil)	0.25	0.20
Non plastic gravelly sand (Foundation soil)	0.15	0.10
Foundation Rock	--	--

Accordingly the computed results during construction are presented in below Figure for downstream and upstream slopes respectively. For downstream slope the minimum factor of safety obtained for end of construction and during construction is 1.685. The minimum factor of safety obtained for the upstream slope is 1.382 during and end of construction case.

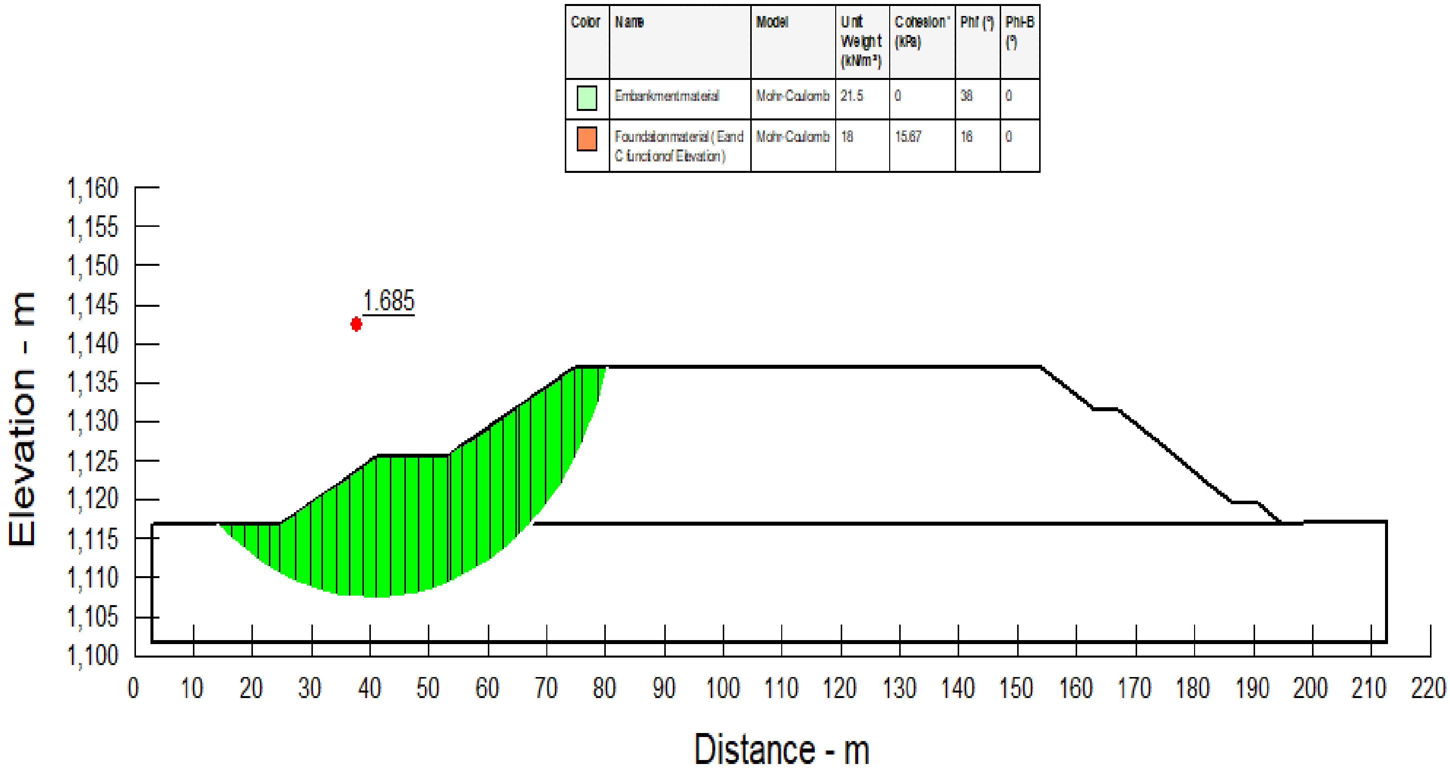


Figure 4.11: Slope stability analysis result for downstream slope during construction

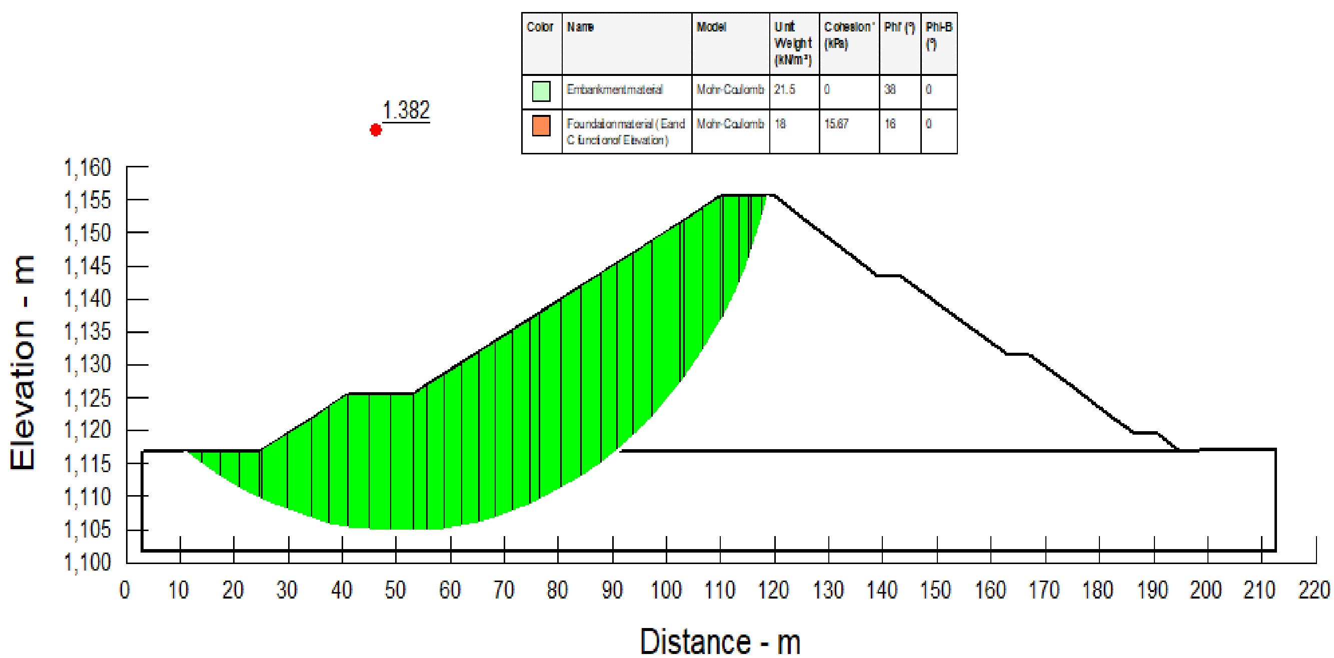


Figure 4.12: Slope stability analysis result for upstream slope at end of construction

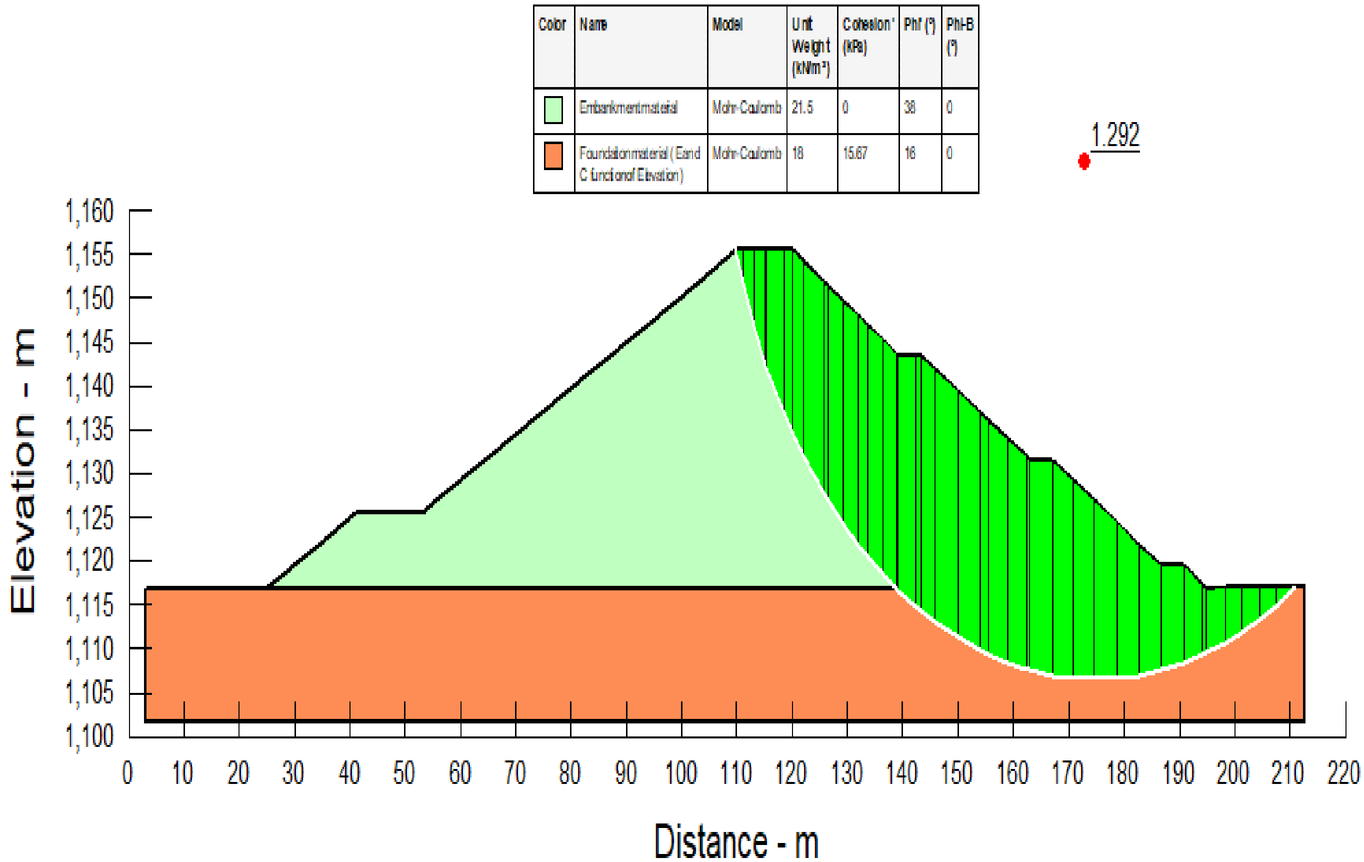


Figure 4.13: Slope stability analysis result for downstream slope at end of construction

III. Upstream Drawdown

Rapid drawdown stability simulations were done for the upstream slope under situations occurring when the water level next to the slope is dropped fast. For analytical reasons, it was assumed that the drawdown is very quick and no drainage occurs in materials with limited permeability. It is intended to have outlets on two distinct floors. The 1126.3m exit is for irrigation, while the lower outlet suggested at 1117m is largely for flow evacuation below the irrigation outlet during maintenance, flow diversion during construction, and silt removal as needed. As a result, the combined drawdown time is expected to be around 21.5 days. The computed factor of safety is 1.44 with Morgenstern-price method of analysis and the pore water is analyzed considering transient analysis.

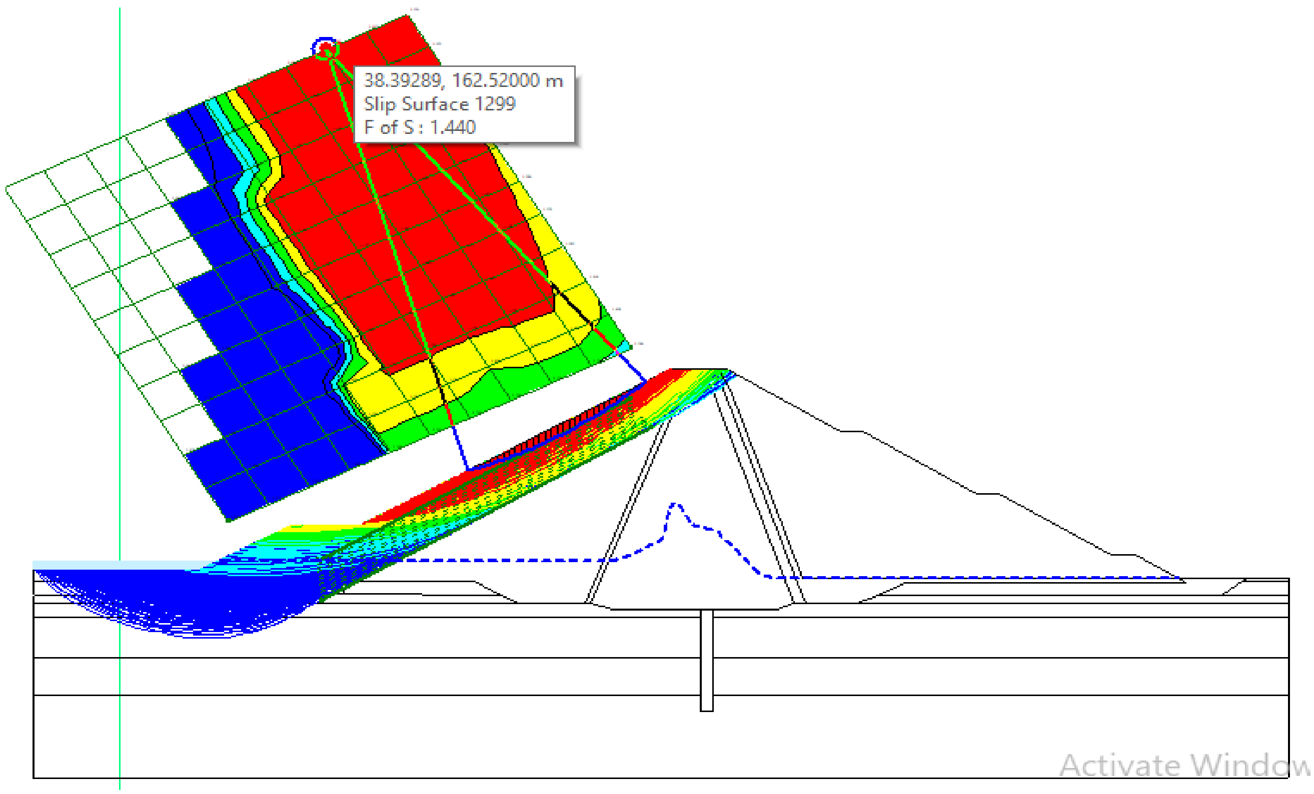


Figure 4.14: Upstream drawdown analysis result

IV. Slope stability analysis with earthquake loading

The Pseudostatic approach is commonly used in analysis for relatively well-built dams on stable soil or rock foundations with projected peak ground accelerations of less than 0.2g. According to the USNRC (1985), a dynamic study should be done in places where peak ground accelerations may exceed 0.2g, as well as dams with embankment or foundation soils that may lose a considerable portion of their strengths due to earthquake shaking. The project region is located in a less active seismic zone, with an anticipated ground acceleration of 0.1g. Therefore, for Chel-chel Dam design, Pseudostatic analysis is sufficient, and thus the analysis is discussed below.

Pseudostatic analysis represents seismic shaking as a permanent body force that is added to the force body diagram of a traditional static limit-equilibrium analysis. The project region is located in a less seismically active zone, hence a PGA value of 0.1g is used for SEE (Safety evaluation earthquake). It is customary practice to consider vertical acceleration as half of horizontal acceleration, i.e 0.05g. Accordingly, the slope stability analysis results considering the Pseudostatic analysis for the steady seepage with earthquake loading for the downstream slope is presented in Figure 3-17 for SEE loading. Results of the analysis depicts that the minimum factor of safety for the SEE loading is 1.244. The output for the earthquake analysis is greater than the minimum required and thus safe.

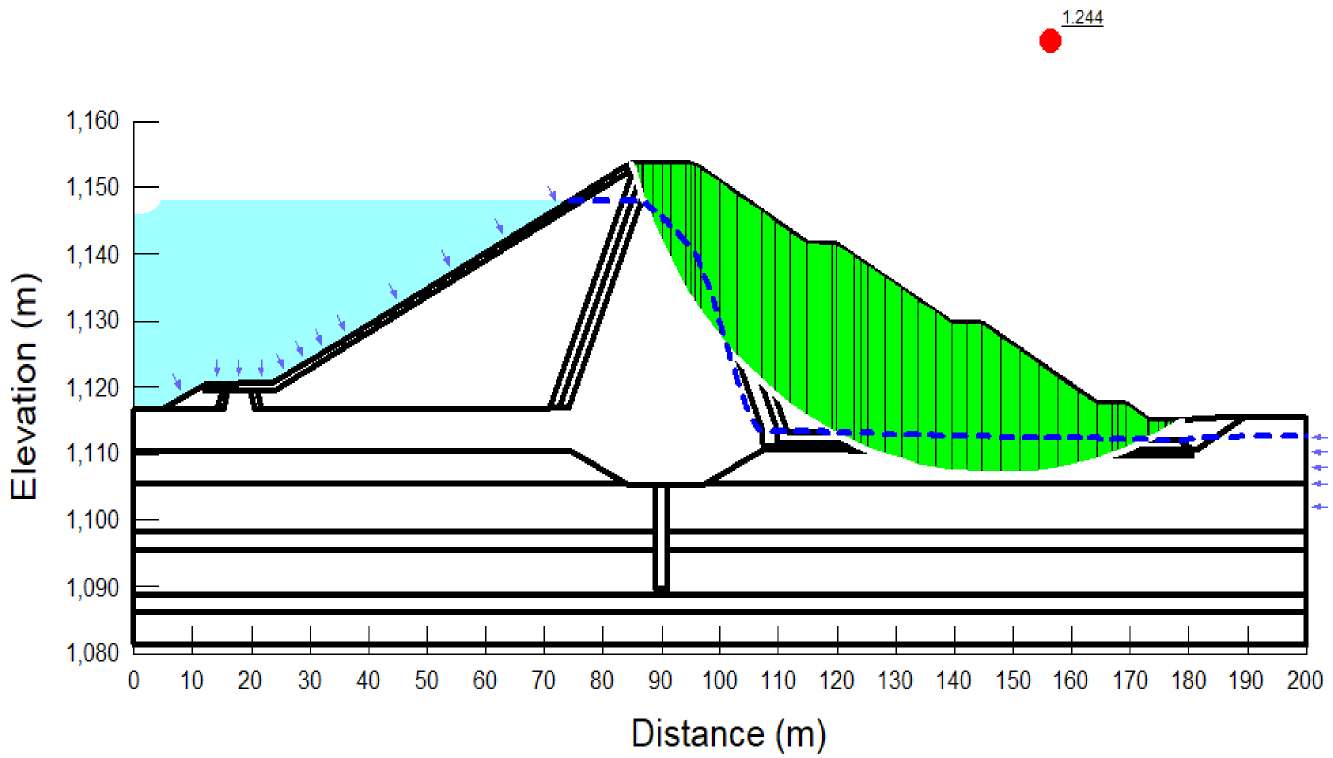


Figure 4.15: Pseudostatic stability analysis result for SEE loading condition

In summary, the computed minimum factor of safety is presented in Table 8 in comparison to the recommended minimum factor.

Table 4.4: Recommended and Computed Factor of Safety (FoS) for different Loading Conditions

Loading condition	Recommended minimum FoS	Computed FoS	
		Upstream	Downstream
During construction	1.3	1.685	
End of construction	1.3	1.382	1.292
Steady state seepage at normal level	1.5	-	1.710
Sudden drawdown	1.25	1.44	-

Based on analysis results, the stable slopes for the proposed dam zoning under all loading conditions are upstream slope of 1V:1.8H and 1V:1.65H for the downstream slope below 1143.50m and 1V:1.6H above 1143.50m.

4.3. Remedial measures

In the first half of the twentieth century, dams like the Hales Bar, Wolf Creek, and Great Falls were built on karstic foundations without adequate foundation explorations, resulting in either foundation leakage or piping issues. Understanding the nature of the problems that must be addressed is crucial for any successful construction on a karstic foundation. Chel-chel embankment dam is also one of the dam proposed to construct on such types of foundation and in the context of our country its implementation is so difficult.

In general, modern practice for the successful treatment of karst foundations necessitates a method of reducing seepage, techniques to prevent the dissolution of soluble materials that may be present in the foundation, and methods to ensure that the foundation has adequate capacity to withstand post-impoundment loadings without excessive settlement.

In the Chel-chel case, Main seepage and leakage in the foundation and abutments is anticipated to occur due to washout of decomposed and friable materials (which would also not be effectively grouted by cement) in the foundation and abutments. Therefore, effective seepage control shall be planned. In this case, an engineering remedy aiming to protect both excessive seepage and foundation washout is protected by create a relatively impervious stratum beneath a dam foundation, utilization of concrete wall and soil bentonite/slurry cutoff are recommended.

The vertical and horizontal drains are also required to control seepage through the embankment by preventing material eroded through a crack in the core impervious part from washing into the downstream shell by seepage water under reservoir head. Moreover because of the often-variable characteristics of borrow materials, it is frequently advisable to provide vertical and horizontal drains within the downstream section of the embankment, in order to ensure satisfactory seepage control.

5. CHAPTER FIVE: CONCLUSION AND RECOMMENDATION

5.1. Conclusion

The main goal of this study was to evaluate seepage and slope stability analysis of Chel-chel Embankment dam and finally come up with the possible remedial measures. As stated in statement of the problem, the Chel-Chel Embankment dam, situated on a limestone foundation, encounters significant challenges related to seepage and structural instability due to the solutioning processes inherent in such geological formations.

Excavation of the alluvial material together with the unconsolidated sediments from the foundation part of the dam body is essential. The removal and replacement of overburden material has no significant impact on the stability and deformation as a result, it was recommended to remove only the core section and weak black soil which enables as well for the construction of cutoff.

This basic challenge happened in the foundation area making the dam exposed mainly to seepage and slope instability and the presence of more seepage aggravates the solution process resulting in structural instability. Excessive seepage flow through dam body and foundation should be seen from economic and dam safety point of view.

Finite Element modeling is utilized in this thesis to simulate seepage and slope stability problems for earth dams using Geo-Studio 2018 software product of SEEP/W and SLOPE/W model, by incorporating dam's different parameters.

From the result simulated the seepage rates with and without treatment structures, revealing that untreated seepage ranged from 2.14 to 6.56 m^3/day per dam length. Implementation of foundation treatment reduced seepage to 1.41–5.98 m^3/day. And the simulated slope stability analysis result are, computed factors of safety of 1.710 for steady-state seepage, 1.685 and 1.566 for upstream and downstream slopes during construction, and 1.44 for rapid drawdown conditions.

Finally, the result of seepage and slope stability analysis with the existing geological formation and property shows the seepage amount was manageable with provision of further foundation treatment, but it's recommended to provide horizontal and toe drains to safely discharge the seepage amount with the recommended safety factor. The slope stability result also indicate that the dam meets or exceeds recommended safety thresholds under all evaluated conditions.

5.2. Recommendations

Dam construction on limestone terrains is complex, and hence necessary precaution as well as strict quality control is mandatory. In our case the following points are recommended;

- The removal and replacement of overburden material has no a significant impact on the stability and deformation as a result it was recommended to remove only the core section and Weak black soil allows for the building of cutoff.
- Providing a fine filter in the contact between clay core and the general foundation of the dam (GTL-2) to protect clay particle movement to the foundation.
- Practices have shown that the soil Bentonite slurry trench offers a good seepage barrier at a far cheaper cost than a diaphragm concrete wall, while being soft for a long period and settling more than the neighboring foundation. As a result, concrete cutoff walls are the ideal option for such foundation circumstances because they provide more control over shear stress in the shale zone.
- Any decomposed material shall be covered with at least a 1m thick clay blanket before the dam fill shell material (Zone 3) for the upstream face is placed. This shall be decided phase by phase during construction. Special focus shall be made to shale formation during foundation excavation and preparation.
- If it is proposed, grouting pattern and spacing shall be confirmed by exploratory holes and test grouting.
- Dam settlement analysis will be the future research area.
- Design report should be prepared and reviewed by a panel of experts or other advisors and geotechnical engineering consultants before any activities of construction work.

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7. APPENDINCES

Company Name

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Ethiopian Construction Design & Supervision Works Corporation

Title: **Geotechnical Laboratory Report** | Document No: OF/ECDSWC/0096 | Issue No: 1 | Page No: 1 of 56

Lab No : 1755/13-1760/13 — Client Ref : -

Submitted by : Defense Construction Enterprise — Received on: 16/12/2020

Chel-Chel Dam and Appurtenant structures construction project (LOT-1)

Project : Dinik - ChelChel Dam and Appurtenant structures — Reported on: 28/1/2021

Station : Borrow Site

Test Requested : As Stated Below

Reported to : Defense Construction Enterprise

No	Tests	Tests methods	Soil Test Results					
			CDWC E:712659 N:766707 1755/13	CDBC E:719472 N:766370 1756/13	CDCC E:720829 N:766279 1757/13	CDDC E:724341 N:763763 1758/13	CDRC E:724203 N:768179 1759/13	CDHC E:716077 N:766620 1760/13
1	Grain size Analysis Clay% Silt% Sand % Gravel %	ASTM D 422	14.36 54.45 31.20 0.00	18.86 48.62 32.51 0.00	13.48 44.47 37.88 4.17	29.49 39.34 31.17 0.00	22.95 38.94 38.12 0.00	47.92 34.16 17.92 0.00
2	Atterberg Limit Liquid Limit% Plastic Limit% Plastic Index%	ASTMD 4318	30.60 17.64 12.96	27.20 14.02 13.18	27.60 14.16 13.44	24.85 16.00 8.86	29.30 15.54 13.76	64.00 29.34 34.66
3	Specific Gravity	ASTM D 854	2.59	2.65	2.51	2.50	2.59	2.55
4	Bulk Unit weight(g/cc)	ASTM D 7263	1.70	1.41	1.63	1.45	1.78	1.39
5	Dry Unit weight(g/cc)	ASTM D 7263	1.52	1.26	1.53	1.29	1.59	1.24
6	Free swell (%)	IS 2720	15	20	10	10	15	85
7	Direct Shear C(kPa) Ø(Degree)	ASTM D3080	27.00 23.27	31.50. 27.25	26.50 23.03	32.00 16.70	25.50 22.54	45.00 7.97
8	Dispersion by double hydrometer (%)	ASTM D4221	19	11	17	10	14	13
9	Consolidation(Cc)	ASTM D2435	0.210	0.256	0.144	0.156	0.184	0.405
10	Permeability (cm/sec)	ASTM D2434	1.19×10^{-5}	9.16×10^{-6}	6.14×10^{-5}	1.16×10^{-5}	8.22×10^{-6}	3.89×10^{-6}
11	Standard Compaction MDD(g/cc) OMC(%)	ASTM D698	1.74 17.20	1.82 16.00	1.84 16.00	1.78 16.90	1.89 15.00	1.53 23.00

REMARK: -The samples were collected and submitted to the laboratory by the client.

Reported by Bethlehem. B
Senior Geotechnical Engineer

Checked by Biruk. A
Senior Geotechnical Engineer

Approved by Getu. D
Geotechnical Lab S/P Manager

Among the major services rendered by the Geotechnical and Material Laboratory Testing s/processes of Ethiopian Construction Design & Supervision Works Corporation are:

- *In Geotechnical Laboratory: Testing the engineering properties of Soil Mechanics and Rock Mechanics.*
- *In Material Testing Laboratory: Testing the engineering properties of various Construction materials, such as Aggregates, Asphalts/Bitumen, Cements, Rocks, Water, Reinforcement steel bars, Hollow Blocks, Bricks, Ceramics, Tiles, Asphalt & Concrete Core Tests, Concrete Mix Designs, Asphalt Mix Designs, Sampling of the soil and construction materials, and so on.*

Please make sure that this document is the correct version before use

Company Name

በኢትዮጵያ የኮንስትራክሽን ዲዛይንና ሱፐርቪዥን ሥራዎች ኮርፖሬሽን

Ethiopian Construction Design & Supervision Works Corporation

Title:

Geotechnical Laboratory Report	Document No OF/ECDSWC/0996	Issue No. 1	Page No 2 of 56

Lab No : 1761/13-1765/13

Submitted by : Defense Construction Enterprise

Chel-Chel Dam and Appurtenant structures construction project (LOT-1)

Project : Dinik - ChelChel Dam and Appurtenant structures

Station : Representative to the quarry

Test Requested : As Stated Below

Reported to : Defense Construction Enterprise

Client Ref: -

Received on: 16/12/2020

Reported on: 28/1/2021

No	Tests	Tests methods	Rock Test Results CDSL 1761/13	CDNL 1762/13	CDNIL 1763/13	CDCAB 1764/13
1	Specific Gravity (SSD)	ASTM C127	2.59	2.65	2.78	2.79
2	Water Absorption (%)	ASTM C127	2.17	1.84	2.01	1.37
3	UCS(MPa)	ASTM D2938	53.99	52.97	47.55	78.05
4	Point load(MPa)	ASTM D5731	2.25	2.21	1.98	3.25
5	Slake durability(%)	ASTM D4644	99.10	99.10	98.95	-
6	Soundness by Sodium Sulphate(%)	ASTM C88	1.39	2.40	1.36	0.78
7	LAA(%)	ASTM C131	31.37	42.45	32.35	31.37
8	Bulk unit weight(g/cc)	ASTM D 7263	2.56	2.61	2.76	2.72

REMARK: - The samples were collected and submitted to the laboratory by the client.
- Six Compaction laboratory test result attached.

Reported by Bethlehem. B
Senior Geotechnical Engineer

Checked by Biruk. A
Senior Geotechnical Engineer

Approved by Getu D
Geotechnical Lab S/P Manager

Among the major services rendered by the Geotechnical and Material Laboratory Testing s/processes of Ethiopian Construction Design & Supervision Works Corporation are:

- *In Geotechnical Laboratory:- Testing the engineering properties of Soil Mechanics and Rock Mechanics.*
- *In Material Testing Laboratory:- Testing the engineering properties of various Construction materials, such as Aggregates, Asphalts/Bitumen/, Cements, Rocks, Water, Reinforcement steel bars, Hollow Blocks, Bricks, Ceramics, Tiles,Asphalt & Concrete Core Tests, Concrete Mix Designs, Asphalt Mix Designs, Sampling of the soil and construction materials, and so on.*

Please make sure that this document is the correct version before use

P.O. Box 62668 Addis Ababa, Ethiopia Tel : 011439 06 93 /011439 10 65 / 011439 16 17 / 011439 17 33 Fax. 0114391230 / 0114391617 E-Mail saba.eng@ethionet.et /info@sabaeng.com	Page No. 1 of
Company Name: SABA ENGINEERING PLC	Effective: April 17,2015
Document Title: Summary Test Result of Earth Material	Doc. Number: OF/MTL/SE/455

LAB. NO : 257/20
CLIENT : Defence Construction Enterprise
PROJECT : Chel-Chel Dam and Appurtenant Structure Construction Lot -1
SAMPLE OF : **Impervious Material (Soil)**
SAMPLE AND TEST SUBMITTED BY : The client ON 6/6/2020
SPECIFIED BY : The client ON 6/6/2020
TEST FOR : Quality
TEST RESULT REPORTED TO : The client ON 20/10/20

Summary Test Results of Impervious Material (Soil)

Sample ID	Source Location	Soil Chemical Test				Specific Gravity	Natural Moisture Content	Unit Wieght	Free Swell	Direct Shear Strength		Proctor Densith AASHTO T 99 & T180				Swelling Pressur	Permeability
												Standard AASHTO T 99		Modified AASHTO T180			
		Chloride Content	Sulphate Content	Total Disolved Solids	PH Value		%	Kg/m^3	%	C	Φ	OMC%	MDD g/cm^3	OMC %	MDD g/cm^3	Kpa	K20 cm/Sec
CDWC	West clay	11.5	7	305	7.09	2.303	5.44	1225.71	20	41.7	5.71	Sample insufficient for Standard Proctor test		16.200	1.960	3.82	2.31E-04
CDBC	Bullo clay	19.4	8	447	7.11	2.234	5.55	1270.60	0	50.7	5.71			14.100	2.000	2.3	2.95E-04
CDCC	Camp clay	12.2	9	427	7.47	2.351	5.86	1283.45	0	36.3	4.57			11.300	2.190	3.83	3.12E-04
CDDC	Dhobu clay	7.6	5	413	7.39	2.313	9.33	1080.18	20	29.3	7.12			12.400	2.050	2.1	2.724E-04

REMARK :

Reported by

SABA ENGINEERING Pvt. Ltd. Co. CENTRAL LABORATORY

Approved by
Ayehutsega Awgechew
Soil and Const. Mat. Testing
Dept. Manager

SABA Engineering

P.O. Box 62668 Tel. 39 36 63 / 39 10 63 / 39 16 17 / 39 17 33 Fax 39 12 30 / 391617 E-mail saba.eng@telecom.net.et Addis Ababa, Ethiopia

Lab No.	:	257/20
Client	:	Defence Construction Enterprise
Project	:	Chel-Chel Dam and Apprutenant Struccture Construction Lot 1
Sampled by	:	The Client
Test for	:	Summary test result of Atterberg Limits,Sive Analysis, Hydromoter Analysis
Submitted by	:	The Client ON :06/06/20
Specified by	:	The Client ON :06/06/20
Reported to	:	The Client ON: 20/10/20

SUMMARY TEST RESULTS of Impervious Material (Soil)

Sr. No.	Samples No	Sample Source	Sieve Analysis,AASHTO T 27-84									Hydrometer Analysis ASTM D 422-63							Atterberg Limits ASTM D4318-93		AASHTO soil Classification	Unified Soil Classification ASTM D 2487-93
			37.5 mm	25 mm	19.0 mm	12.5 mm	9.50 mm	4.75 mm	2.0 mm	0.425 mm	0.075 mm	Particle Size Smaller Than 0.020	0.002	0.001	Particle Size,mm > 2mm	Sand% 2-0.075mm	Silt% 0.075-0.002mm	Clay% < 0.002mm	LL	PI		
1	CDWC	West clay					100	99	97	94	76	35.3	27.1	23.2	2	54	22	22	31	10	A-4(3)	ML
2	CDBC	Bulio clay						100	96	93	75.1	41.8	31.4	27.6	4	41	26	29	28	11	A-6(8)	ML
3	CDCC	Camp clay					100	98	95	93	69.4	37.6	18.6	14.7	4	52	35	14	23	7	A-4(0)	ML
4	CDDC	Dhobu clay						100	97	95	79.2	36.6	14.7	8.5	2	49	35	11	27	7	A-4(0)	ML

Remark :

Reported by

SABA ENGINEERING Pvt. Ltd. Co. CENTRAL LABORATORY

Approved by

Ayehutsega Awgechew
Soil and Constr Mat. Testing
Dept. Manager

P.O. Box 62668 Addis Ababa, Ethiopia Tel / 011439 06 93 /011439 10 65 / 011439 16 17 / 011439 17 33 Fax. 0114391230 / 0114391617 E-Mail saba.eng@ethionet.et /info@sabaeng.com

Document Title	SABA ENGNEERING PLC	
Issue No. 1	Summary Test Result of Earth Material	

LAB NO : 257/20

CLIENT : Defence Construction Enterprise

PROJECT : Chel-Chel Dam and Appurtenant Structure Construction Lot -I

SAMPLE OF : **Natural Filter Material/Coarse Sand**

SAMPLE AND TEST SUBMITTED BY : The client ON : 6/6/2020

SPECIFIED BY : The client ON : 6/6/2020

TEST FOR : Quality

TEST RESULT REPORTED TO : The client ON : 20/10/20

Summary Test Results of Natural Filter Materials/Coarse Sand

Sample ID	Source Location	Particle Grading Analysis								Fineness modules	Silt and Clay Content	Specific Gravity		Water Absorption	Organic Content	Total Dissolved Salts	Permeability
		9.5 mm	4.75 mm	2.36 mm	1.18 mm	0.60 mm	0.30 mm	0.15 mm	0.075 mm		%	Bulk	Apparent	Water Absorption %			K20 cm/Sec
CDFS-1	Fejis River	100	70	34	18	10	6	3	2.6		1.71	2.514	2.567	1.36	1.60	535.00	1.92E-03
CDFS-2		100	80	48	27	20	15	5	2.8		2.34	2.542	2.611	1.68	1.50	568.00	2.153E-03
CDFS-3		100	94	84	71	49	24	6	2.1		1.55	2.496	2.543	1.22	1.30	734.00	2.15E-03
Natural Aggregate /Fine Sand																	
CDCS-1	Chelchel River				100	95	53	4	1.8		0.95	2.527	2.584	1.44	0.50	562.00	2.22E-03
CDCS-2		100	99	98	94	65	27	5	2.1		1.12	2.567	2.618	1.24	1.00	578.00	2.15E-03

REMARK :

Reported by

SABA ENGINEERING Pvt. Ltd. Co. CENTRAL LABORATORY

Approved by

Ayehutsega Awgechew
Soil and Const. Mat. Testing
Dept. Manager

BH ID	Test section	Test type	Flow pattern	Hydraulic Conductivity (cm/se)
MDC	0.0 – 2.45	Falling head	Laminar	0.0000082
	2.50 – 5.50	Falling head		0.000031
	5.00 – 8.12	Falling head		0.000039
	28.0 – 33.0	Packer		0.00000161

Table: Packer permeability tests result

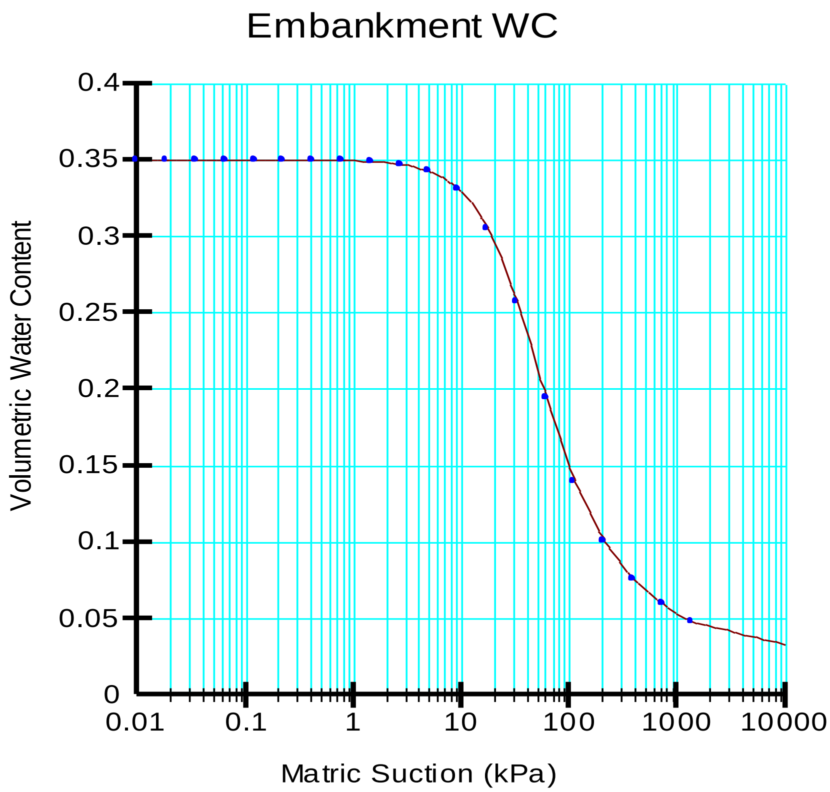

Figure 1: Volumetric water content of the embankment

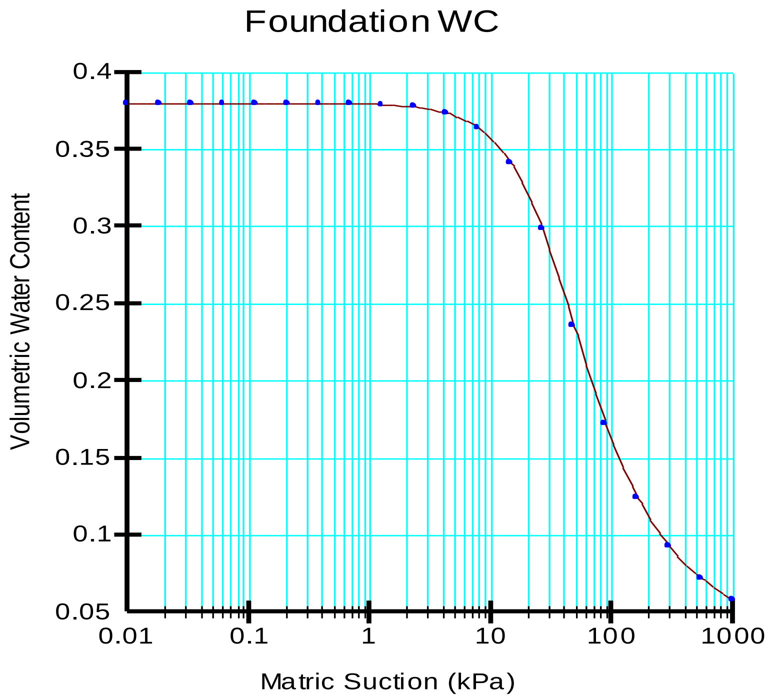

Figure 2 : Volumetric water content of the foundation

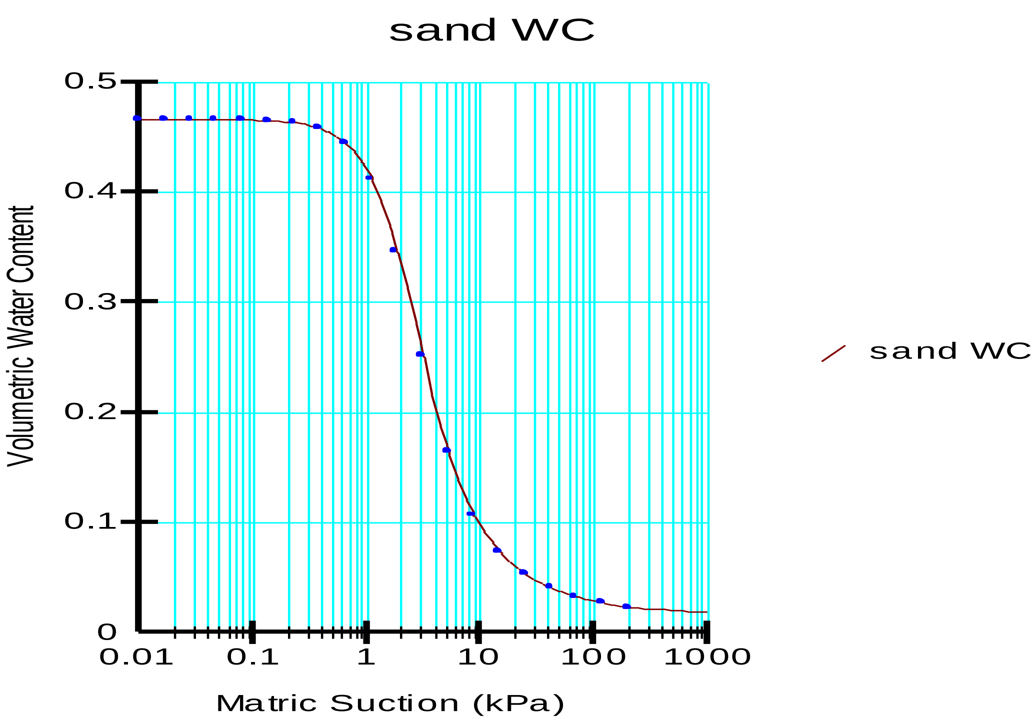

Figure 3: Volumetric water content of the filter materials

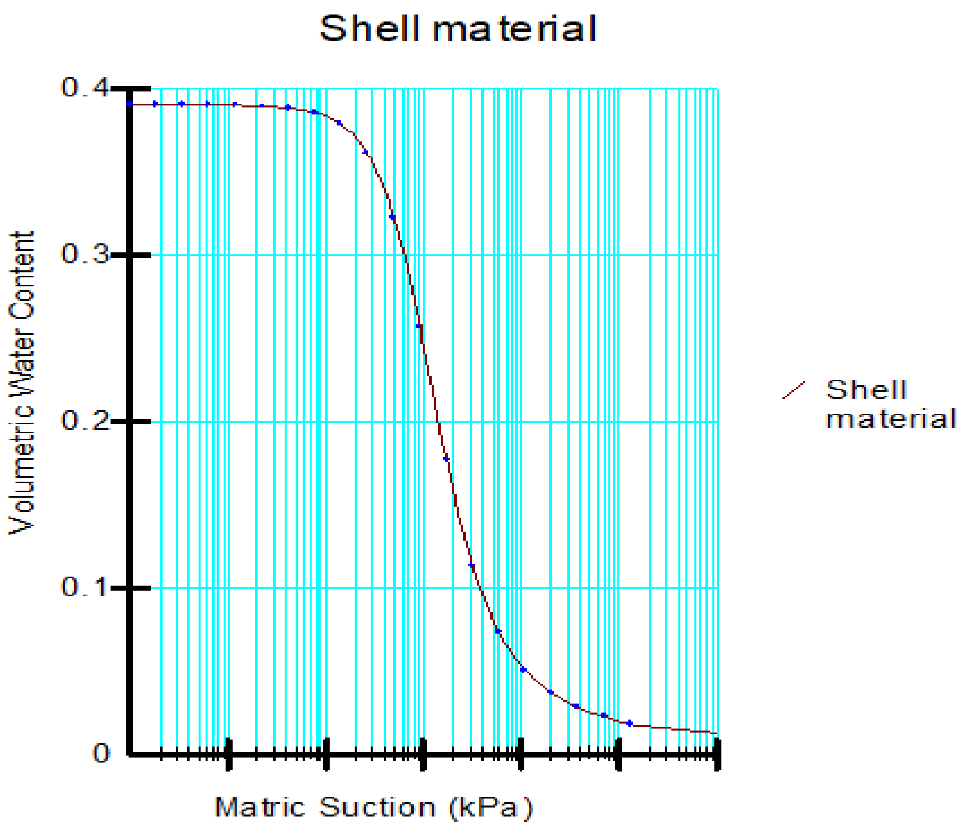

Figure 4: Volumetric water content of the shell material