

**Engineering Geological Investigation of Katar Dam
Foundation and Abutment Slope Stability Analysis, Arsi
Zone, Oromia, Ethiopia**



Dawit Belete Legesse

A Thesis Submitted to the Department of Applied Geology

School of Applied Natural Science

Presented in Partial Fulfillment of the Requirement for the Degree of
Master of Science in Applied Geology

Office of Graduate Studies

Adama Science and Technology University

March 2024
Adama, Ethiopia

**Engineering Geological Investigation of Katar Dam
Foundation and Abutment Slope Stability Analysis, Arsi Zone,
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Declaration

I hereby declare that this Master Thesis entitled “**Engineering Geological Investigation of Katar Dam Foundation and Abutment Slope Stability Analysis, Arsi Zone, Oromia, Ethiopia**” is my original work. That is, it has not been submitted for the award of any academic degree, diploma, or certificate in any other university. All sources of materials used for this thesis have been duly acknowledged through the citation.

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We, the advisors of this thesis, hereby certify that we have read the revised version of the thesis entitled “**Engineering Geological Investigation Of Katar Dam Foundation And Abutment Slope Stability Analysis, Arsi Zone, Oromia, Ethiopia**” prepared under our guidance by Dawit Belete Legesse submitted in partial fulfillment of the requirements for the degree of Master of Science in Applied Geology. Therefore, we recommend the submission of revised version of the thesis to the department following the applicable procedures.

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LIST OF ACRONYMS AND ABBREVIATIONS

ASTU	Adama Science and Technology University
ASTM	American Society of Testing and Materials
DEM	Digital Elevation Model
EBCS	Ethiopian Building Code Standard
ECDSWC	Ethiopian Construction Design & Supervision Works Corporation.
ECO	Engineering Corporation of Oromia
FAO	Food and Agriculture Organizations
FOS	Factor of Safety
GDP	Gross Domestic Product
GIS	Geographical Information System
GSI	Geological Strength Index
GPS	Global Positioning System
ICOLD	International Commission of Large Dams
ISRM	International Society of Rock Mechanics
JCS	Joint Compressive Strength
JRC	Joint Roughness Coefficient
LEM	Limit Equilibrium Method
LL	Liquid limit
MER	Main Ethiopian Rift
MMS	Moment Magnitude Scale
MoWR	Ministry of Water Resources
PI	Plasticity Index
PL	Plastic Limit
RMC	Rock Mass Classification
RMR	Rock Mass Rating
RQD	Rock Quality Designation
SDI	Slake Durability Index
UCS	Uniaxial Compressive Strength
USBR	United States Bureau of Reclamation
UTM	Universal Transverse Mercator

ABSTRACT

The primary pillar of the Ethiopian economy in terms of income, employment, and export earnings generation is agriculture. About 80 percent of the population live in rural, agriculture-dependent areas but the majority of the population is still impoverished and faces food insecurity. The Katar dam site is a 35m height rainwater-harvesting hydraulic structure implemented by the Ethiopian government in order to enhance irrigation and bolster food security in the region. Despite this, the dam site has engineering geological challenges due to the presence of weak, extensively fractured, and weathered ignimbrite rock, as well as weak unwelded to welded tuff rock mass near the dam base and abutments. Engineering geological investigation of the dam foundation and abutments on the dam site's rocks and soils addressed these challenges. To achieve these objectives, geological mapping, sampling and laboratory analysis of samples, discontinuity surveying, in-situ strength test analysis, core drilling, geophysical investigation, in-situ permeability test, and slope stability study were carried out. From geological mapping, the principal lithological unit of the research region were residual soil deposits, ignimbrite rock, welded tuff rock unit and unwelded tuff unit with varied degree of fracturing and weathering. At the dam site, intact rock strength test indicated that the uniaxial compressive strength value ranges from 8.97MPa to 22.87MPa and the UCS estimated from the in-situ strength test shows a range of 24.15MPa to 33.88MPa showing the rock mass constituting dam site have very low to low strength rock mass. At the dam site, the subsurface rock quality designation ranged from very poor to excellent in terms of rock mass quality. According to the packer test results, there should be significant leakage at the intake conduit borehole and right abutment due to hydraulic conductivity with representative Lu values of ranging from 9.08-12.07 and 2.14-13.23, respectively. The majority of the soils in the reservoir and dam site were clayey silt and sandy silty soils, with hydraulic conductivity values indicating low to impermeable soil mass and plasticity index values ranging from 6.4% to 31.9% with low to high and 24% with high plasticity of soil. The allowable load-bearing capability of the rock formations at the dam's abutment and intake conduit varies between 0.02MPa and 16.11MPa, and between 0.36MPa and 2.3MPa, respectively. The slope sections were unstable under fully saturated static and dynamic conditions, according to the slope stability analysis. This research recommends curtain grouting for the permeable rock layer, further detailed study of soils at the reservoir area, additional evaluation of design and implementation of remedial measures such as rock bolts, and intensive and adequate dam foundation design.

Keywords: Katar Dam site, Foundation Assessment, Slope Stability analysis, Lugeon Value, Grouting .

CHAPTER ONE

1. INTRODUCTION

1.1. Background

Dams are civil engineering structures built across a river valley or natural depression in order to impound enormous quantities of water for numerous uses, including power generation, irrigation, flood control, ground water recharge, and water diversion (Sissakian et al., 2020). The provision of affordable hydroelectricity and freshwater facilitated by dams essentially increased food production via irrigation, powered industrial expansion, and enhanced health outcomes via the distribution of potable water and sanitation services (Piesse, 2018; Sissakian et al., 2020; CLG, 2023). Dams were also partly responsible for the massive increase in the global food supply. Between 1950 and 2000, the amount of irrigation increased almost threefold, primarily as a result of the construction of storage dams. At that time, the portion of arable land that was irrigated accounted for approximately 17% of the total, yet it yielded 40% of the global food supply. Despite the presence of numerous potential dam sites, Africa, for the most part, did not partake in this economic growth (Piesse, 2018).

According to World Bank (2020), the primary pillar of the Ethiopian economy in terms of income, employment, and export earnings generation is agriculture. About 80 percent of the population live in rural, agriculture-dependent areas but the majority of the population is still impoverished and faces food insecurity (World Bank, 2020; FAO, 2022). The development of irrigation systems has been recognized as a key strategy for promoting rural and economic development. According to estimates from Bekele et al. (2007) and the CSA (2015), 3.8 million hectares of land in Ethiopia have the capacity to be irrigated. Irrigation development has been identified as a key strategy for promoting rural and sustainable economic growth as well as a pillar of food security and poverty alleviation in government policy documents (MoWR, 2002; MoFED, 2006; Yohannes, 2020). By encouraging agricultural activity through varied dimensions irrigation schemes, water impoundment structures like Micro-Dam reservoirs (MDRs) play a significant role in addressing such challenges, notably of food security (Garrido et al., 2006; Scanlon et al., 2006). Food security poses a significant challenge for emerging economies such as Ethiopia, where fluctuations in weather patterns

and prolonged periods of drought jeopardize the availability of food and water (De Hamer et al., 2008).

The majority of dam failures occur as a result of inadequate engineering and geological research conducted before construction. Engineering geological investigation must be completed prior to the building of the engineering structure in order to ensure the safety of the dam against such issues (Sissakian et al., 2019). The two main factors that are examined using these techniques are hydraulic conductivity and the geo-mechanical characteristics of rock masses (Gurocak and Alemdag, 2011).

The experience of building dams all around the world has demonstrated that many dams are impacted by many engineering geological issues, such as leakage, uplift pressure, differential settlement, abutment instability, and others (Garo and Meten, 2021; Asfaw et al., 2022). During catastrophic events, most dams, regardless of their structure or design, suffer higher stresses that raise the likelihood that they will fail (Genetu, 2007). Several dams in Ethiopia have encountered challenges stemming from geological and geotechnical factors. Micro dams, constructed from the 1970s onwards to address drought and enhance food security, have also encountered instances of failures and operational difficulties (Tiruneh, 2005; Berhane et al., 2016; Berhane et al., 2017). Research indicates that the performance of these studies has been subpar due to issues such as seepage or water leakage, slope instability, and other obstacles (Berhane et al., 2013). Constructing dams on inappropriate terrains, such as the sedimentary terrain of the Mekelle Outlier, has led to problems such as leakage, sedimentation, and hydrological issues. Engineering geological investigations are essential to guarantee the safe and efficient operation of dams. Dams built on volcanic terrains encounter notable geological and engineering geological obstacles, such as seepage, bearing capacity, and slope stability concerns (Asfaw et al., 2022).

Performing engineering geological and geotechnical surveys at dam sites allows for the assessment of the hydraulic and geomechanical characteristics of the geological materials comprising the dam's foundation and its immediate environment. The investigations carried out are thorough and include discontinuity mapping, geotechnical drilling, geophysical investigation, in situ analysis, and laboratory testing (Nezhad et al., 2012; Berhane et al., 2017; Garo and Meten, 2021; Asfaw et al., 2022). Asfaw et al.(2022) utilized discontinuity surveys, rock strength testing, and other geological criteria to evaluate the engineering properties of the rock mass and categorize the geological materials beneath dam foundations.

Researchers typically use the kinematic approach, the limit equilibrium method, and numerical approaches when evaluating slope stability (Eberhardt, 2003; Khadija et al., 2012; Karaman et al., 2013; Shaz et al., 2019; Asfaw et al., 2022). Grouting and impermeable coatings are the methods that are often used to stop reservoir water leaks (Warner, 2004). The installation of rock reinforcement (such as rock bolts, rock anchors, and shotcrete), modification of the slope's contour, and laying out drainage holes are the best corrective measures utilized to stabilize the rock slope at the dam site (Hoek and Bray, 1981 and Abramson et al., 2002).

Igneous rocks exhibit both primary and secondary porosity, along with joints, fractures, and fault lines, which increase the probability of seepage. The seepage can lead to water depletion from the reservoir, as well as instability and deformation of the slope (Asfaw et al., 2022). Moreover, volcanic rock undergoes significant weathering, which has an impact on the rock's ability to support weight (Cabria, 2015; Chala and Rao, 2021). The process of weathering diminishes the mechanical characteristics of rocks, which can lead to failures or substantial settlements in various rock-engineering structures (Gupta and Seshagiri, 2000). These challenges imply that the construction of a dam necessitates comprehensive engineering geological investigations in the country to ensure the long-term functionality and overall safety of the dams.

The primary goal of the engineering geological research for the dam project is to offer exact and dependable data and information to ensure the dam's specified lifespan. The primary goal of this study is to assure the dam's long-term viability. This will be accomplished by an engineering geological evaluation designed to gather extensive knowledge and information on the dams. This evaluation will include soil characterization and classification, geomechanical rock analysis, determining the bearing capacity of foundation rocks, assessing the deformability of rock masses, analyzing the permeability of foundation rocks or seepage, evaluating slope stability for both abutments, and determining the reservoir's watertightness. These objectives can be accomplished through a site-specific geological and engineering geological investigation that includes activities such as mapping geological outcrops, conducting geophysical tests, performing geotechnical drilling, surveying joints and discontinuities, conducting in situ tests, collecting rock and soil samples, and analyzing samples in a laboratory (Asfaw et al., 2022).

The Katar dam site is a 35 m height rainwater-harvesting hydraulic structure implemented by the Ethiopian government in order to enhance irrigation and bolster food security in the region. The objective of this research was to tackle engineering geological challenges by conducting a comprehensive analysis and offering remedial measures for the dam site, with the aim of guaranteeing the long-term safety and sustainability of the dam.

1.2. Location and Accessibility of Study Area

The Katar small-scale dam irrigation development project is located in Degeluna Tijo worda, Arsi zone of Oromia regional state at about 36km southeast of Asela Administrative city of zone. Sagure and Digelu are two of the towns in Degeluna Tijo Wereda. Sagure is 123km far from the Adama town. Only 7km to 8km is required to walk to arrive at dam site from Sagure. It is bounded between 39° 01' 5.2755" to 39° 11' 58.5663" E longitude and 7° 7' 38' 25.7153" to 7° 55' 29.6163" N latitude. It is accessed through Addis Ababa to Asela to Sagure through asphaltic road and the remaining branched road with all and dry weather road up the project site. It was located about 251 kilometers to the southeast of Addis Ababa (Figure 1.1).

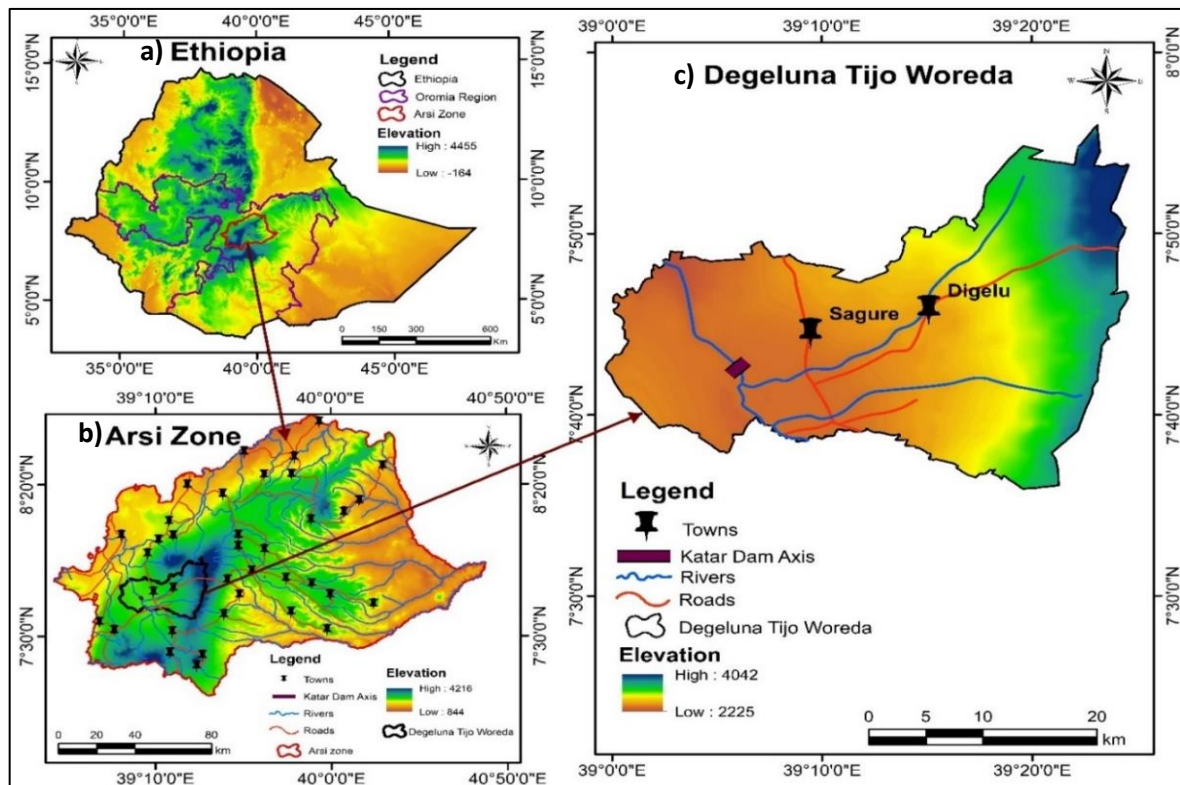


Figure 1. 1. Location Map of Katar Dam site. (a) Ethiopia map with Oromia regional state and Arsi zone boundary with elevation; (b) Arsi zone and Degeluna Tijo worda with their

Katar dam site, roads, towns, river and elevation; (c) Degeluna Tijo woreda and katar dam site with their surrounding towns, rivers, roads, and elevation.

1.3. Climate and Vegetation of the Study Area

The meteorological data were obtained from nearby Segure Stations. The average annual temperature and rainfall are 15.61°C and 63.94 mm respectively. There is much higher rainfall in summer than winter, 7.27 mm of precipitation in December which is much less than 143.70 mm rainfall in July. March is the warmest month with an average temperature of 25.64 °C. On the contrary January has the lowest average temperature of the year which is 5.01°C. The monthly average temperatures of the area vary from 14.17 °C to 16.94 °C. The study area is covered by abundant trees and vegetation with warm and temperate climatic conditions.

Table 1. 1. Rainfall and Temperature data of Katar dam site from Sagure station (1985-2022) (Source: Ethiopian Metrological Agency).

Element	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Average
PRECIP	11.38	32.76	53.42	85.64	81.99	90.59	143.70	140.96	75.14	35.70	8.66	7.27	63.94
TMPMAX	24.37	25.23	25.64	24.00	23.49	22.06	20.09	19.35	21.41	23.09	23.80	23.56	23.01
TMPAVG	15.24	16.50	17.26	16.94	16.60	15.81	14.91	14.93	15.23	15.20	14.58	14.17	15.61
TMPMIN	5.01	7.56	8.52	9.77	9.59	9.38	9.73	9.66	8.97	7.26	5.37	4.74	7.96

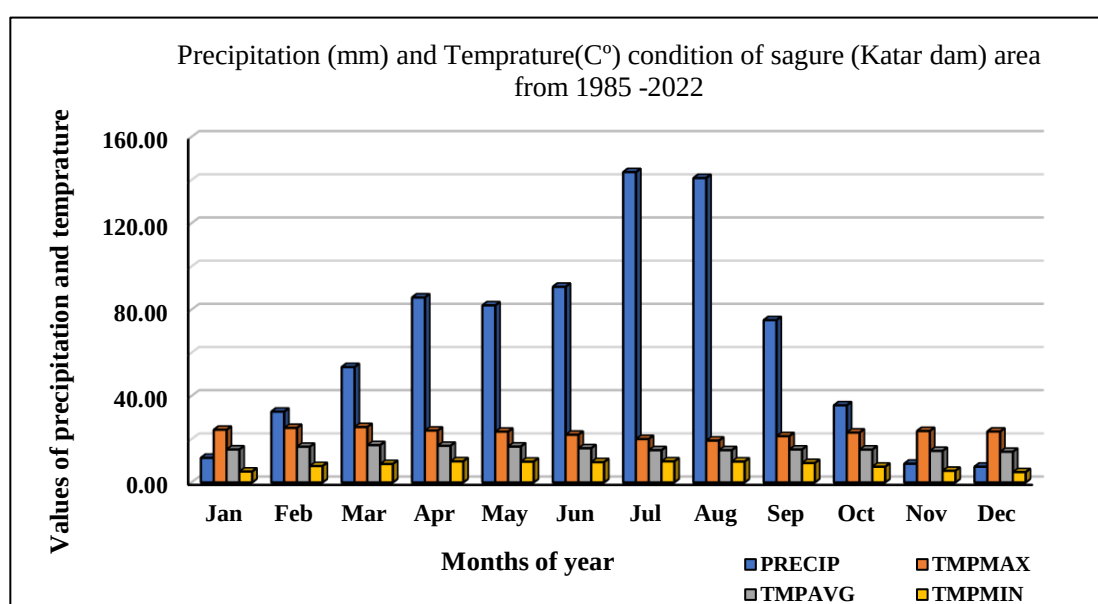


Figure 1. 2. Monthly average precipitation and temperature of Katar dam site (1985-2022).

1.4. Statement of Problem

Dam failures are usually caused by poor engineering geological investigation, emphasizing the significance of geological inquiry before dam construction. Dam sites have different geological qualities and geotechnical concerns; therefore, any dam site may have bad geological circumstances. Engineering geological evaluation and corrective procedures are needed to understand geological conditions and their impact on the proposed dam.

The Katar irrigation dam project aims to irrigate downstream farms. Safe and reliable operation are crucial to the dam project's irrigation success in the study area. Geological and geotechnical assessments are essential to dam safety studies. The dam site abutments and foundation must be thoroughly investigated, explored, and studied to estimate the engineering geological condition. This ensures that the dam site can bear loads and remain stable and impermeable. The new dam's safety must be assessed, among other things.

According to the Engineering Corporation of Oromia (2023), the foundation of the proposed Katar dam is difficult because there is a weak, highly weathered, and fractured rock mass formation in the area. The dam site is situated in a region characterized by the prevalence of pyroclastic rocks. The geotechnical drilling conducted on the left and right abutments of the dam, as stated in this report, discovered the presence of an inferred fault in the rock mass. Field surveys provided evidence of geological structures, such as joints and faults, exhibiting diverse conditions. The dam site is located in the main Ethiopian rift. Consequently, the dam's location may become more vulnerable to earthquakes and seismic activity. The aforementioned area is susceptible to leakage, seepage, and foundation problems, rendering the construction of a dam in that location exceedingly challenging. It is anticipated that this situation will lead to the failure of the dam's abutment and the instability of its foundation. This research aims to address the existing knowledge gap by providing comprehensive information on the geological and engineering geological properties of the materials surrounding the dam site, specifically in relation to abutment slope stability and foundation analysis.

1.5. Research Questions

This research work expected to answer the following research questions:

- What are the geological and geotechnical condition of the dam site?

- What are the index and engineering properties of the soils and rocks of the dam site?
- How about the suitability of foundation and abutment condition based on geotechnical investigation?
- What are the remedial measures that should be taken for these Foundation and abutment slope instability problems?

1.6. Objectives of the Study

1.6.1. The General Objectives of the study

The main objective of this study is to conduct engineering geological appraisal of the dam foundation and abutment slope stability problems.

1.6.2. The Specific Objectives of the Study

The specific objectives of this study are:

- To determine engineering properties of rock and soil of the dam site,
- To evaluate seepage potential and bearing capacities of the dam foundation,
- To carry out slope stability analysis on both side of the dam abutment,
- To produce a geological map at scale of 1:10,000 and engineering geological map of dam site at scale 1:5,000, and
- To suggest possible effective remedial measures for engineering geological problems on the dam site.

1.7. Significance of the study

A dam is an engineering structure that requires a comprehensive engineering geological assessment at the construction site. In order to prevent issues with the stability of the foundation and abutment slopes. The objective of the ongoing research project is to assess whether the foundation conditions of the dam are adequate in preventing seepage and deformation of the foundation material caused by the loads imposed by the dam. This analysis aims to identify any adverse foundation conditions that may lead to complications during or after the construction phase. Hence, specific remedial measures have proven to be successful based on the underlying circumstances. This facilitates the economic strategizing and implementation of the Katar irrigation projects, while also ensuring their safety.

In general, the outcome of this research has been crucial for ensuring the safety and cost-effectiveness of the dam's design. The research findings will be beneficial for both the construction and post-construction stages of the dam. This research study offers valuable

data for future researchers interested in working within the same problem domain. The outcome and findings of this study will enhance the data in the specific area and the broader region, providing valuable resources for the scientific community as a whole.

CHAPTER TWO

2. LITERATURE REVIEW

2.1. Introduction

The construction of a dam to produce a water reservoir is a critical infrastructural improvement for the town, representing a complex and major technological undertaking. Many dam failures occur around the world as a result of insufficient research and parameter characterization. As a result, the building of these massive structures involves extensive site assessment to assure their safe operation. Dam accidents and failures can be caused by a variety of geological reasons, including poor surface and subsurface geological mapping, variable degrees of rock weathering and solubility, permeability, seismic activity, structural faults, and relics of previous slope failures. This chapter extensively reviewed various literature works, such as recent scientific journals, articles, guidelines, and books, pertaining to the analysis of gravity dam foundation and abutment slope stability under different conditions. The review focused on the estimation methods for factor of safety, considering the most recent and updated dam assessment guidelines, standards, criteria, and personal studies conducted by other researchers in this field.

2.2. Engineering Geological Problems Related to Dams

The engineering geology challenges at a dam site can manifest in various ways. Various incidents have occurred worldwide due to geological complications related to the foundation and abutment of dams. Varying locations exhibit distinct geological formations for the placement of dams, along with corresponding geotechnical obstacles. Gangopadhyay (1993) identifies several geotechnical challenges encountered during dam construction, including the thickness of overburden, and weathered rock, the presence of soft layer strata, and structural defects such as faults, thrusts, shear zones, and joints.

A dam is deemed to have collapsed when water is permitted to flow in the opposite direction, towards its original location. Dams have the potential to undergo either abrupt or gradual disintegration. Concrete dams have a higher propensity for abrupt failure compared to other varieties of dams. In recent years, there have been numerous documented instances of dam failures worldwide. These failures have been attributed to major factors such as leakage, differential settlement, abutment instability, and other foundation defects (Barzegari, 2017; Garo and Meten, 2021).

One of the prevalent engineering geological challenges encountered at dam sites is the instability of abutment slopes (Kanik and Ersoy, 2019; Garo and Meten, 2021). Considerable effort has been devoted to resolving the slope stability problem, which is a critical concern in the fields of soil and rock engineering. Various approaches have been utilised in this endeavor, including kinematic analysis for identification of potential mode of failures, limit equilibrium methods for determination of forces that are acting on slope section in terms of factor of safety, and also numerical methods like finite element methods (Asfaw et al., 2022). Such instability primarily results from adverse slope geometries, geological defects, a high degree of weathering and fracturing of slope materials, human activity, heavy rainfall, and earthquakes at the dam site (Basahel and Mitri, 2017).

Numerous endeavors have been undertaken to compile and evaluate statistics on dam failure. The International Commission on Large Dams (ICOLD) made significant efforts on a global level in 1974, 1983, and 1995. According to ICOLD (1995), foundation issues are the primary reasons for failure in concrete dams, with internal erosion and foundation shear strength each accounting for 21% of the failures. The primary cause of failure in earth and rockfill dams is overtopping, accounting for 31% of cases, followed by internal erosion within the dam body (15% as primary cause and 13% as secondary cause), and in the foundation (12% as primary cause and 5% as secondary) (ICOLD, 1995). Table 2.1 presents different statistics on the causes of dam failure, as provided by the International Commission on Large Dams (ICOLD, 1995), the US Army Corps of Engineers (USACE), and (Novak et al., 2017).

Table 2. 1. Statistics on causes of dam failure (Novak et al., 2003).

Source	Overtopping	Foundation defects	Internal erosion	Others
USACE (2006)	34%	30%	20%	6%
Novak et al. (2003)	30-35%	No data	30-35%	
ICOLD (1995)	31% Primary Cause 18% Secondary Cause	No data	27% primary Cause 18% secondary Cause	

The foundation problems include slope instability, differential settlement, high uplift pressure, and foundation seepage. Internal erosion also refers to piping and seepage of the

water through dam body and geological structures at the dam foundation. Prior to commencing construction on a dam site, it was necessary to conduct an engineering geological assessment in order to determine the site's diverse geological characteristics. Typically, these investigations are carried out in sequential phases. Subsequently, the site undergoes an engineering geological mapping process to accurately identify and document the lithological and structural composition, as well as any noticeable geological faults on the surface (Gangopadhyay, 1978). Engaging experts to conduct these examinations can decrease the likelihood of dam failure.

2.3. Engineering Geological Investigation of Dam Site

The dam site can undergo an engineering geological investigation by utilizing various techniques, including in-situ and laboratory testing. These techniques primarily focus on achieving the strength, deformation, water-tightness, and stability of the slopes formed by the materials present at the site under investigation. The strength and deformation parameters can be acquired through in-situ methodologies. Nevertheless, their determination can primarily be achieved by adopting rock mass classification systems and subsequently utilizing empirical equations. The water-tightness of a structure is primarily assessed by conducting in-situ permeability tests, such as the Packer test. On the other hand, slope stability can be evaluated using various methods including slope mass rating, kinematic analysis, limit equilibrium analysis, finite element analysis, and numerical methods (Abramson et al., 2002).

2.3.1. Permeability Analysis of Dam

Dam design and construction require intense engineering geological investigation. Assessing the water tightness of dam foundations, abutments, and reservoir areas is crucial for their structural integrity and functionality. Analyzing geological parameters of the dam site ensures impermeable foundations, preventing water seepage and leaking (Qureshi et al., 2013; Qi and Dashui, 2019; Garo and Meten, 2021).

Fell et al.(2014) stated that engineering geological investigation for dam seepage issues identifies geological hazards and analyses foundation permeability-affecting soil and rock properties. Water can seep through faults, fractures, weak geological formations, and other features (Asfaw et al., 2022). To find these hazards, engineers must map and characterize the site's subsurface geology. Geological material analysis at the dam site and reservoir includes geotechnical drilling, in-situ tests, soil and rock sampling, and lab testing. They

determine material index and engineering properties. Engineering geological conditions must be understood for safe dam design because dam sites are unique (Uromeihy and Barzegari, 2007; Barzegari, 2017).

Water seepage from dam reservoirs can result from rapid sedimentation and slope instability, according to Asfaw et al. (2022). Solution channels, fractures, joints, fault zones, and buried channels allow localized water flow and leakage. Water losses create downstream streams. Alonso et al.(2010) define seepage as a smaller-scale and widespread flow that causes foundation material saturation or rock weakening, leading to sliding failure. Investigating reservoir seepage and leakage requires considering the basin's water tightness and reservoir bank stability. Water tightness can be assessed, and groundwater control measures designed after examining groundwater conditions and geology. Some leakage or seepage is acceptable if it doesn't threaten dam safety or hydraulics (Fell et al., 2014). After the identification of permeability condition of dam foundation, permeability controlling system should be implemented with precision and adequate design. The installation of a seepage control system is a prevalent method for enhancing the dam's watertightness. As with a slurry trench or concrete cut-off, this may involve the installation of a grout curtain, which consists of a sequence of precisely spaced drill holes filled with cement or another material with a low permeability. The purpose of the grout curtain is to prevent seepage along the foundation (Malkawi and Al-Sheriadeh, 2000; Uromeihy and Barzegari, 2007; Gurocak and Alemdag, 2011; Azimian and Ajalloeian, 2014; Hailu, 2020; Asfaw et al., 2022).

The dam foundation and its surroundings' permeability are determined by in-situ Packer tests of geological materials' depth, discontinuity characteristics during field investigations, and laboratory soil mass permeability tests. Vaskou et al.(2019) found that Lu's relative permeability changes with depth, aiding hydro stratigraphical lithological unit characterization. The Lugeon test estimates a rock mass's water absorption capacity and equivalent isotropic coefficient of permeability/hydraulic conductivity around a borehole. This data measures rock mass hydraulic behaviour by depth and location to determine grouting needs for dam foundations, abutments, and underground rock excavations. The test also determines excellent grouting materials and pre-grouting injection pressures (Gurocak and Alemdag, 2011; Gurocak and Alemdag, 2012).

2.3.2. Bearing Capacity Evaluation of Foundation Rocks

The process of determining the safe load-bearing capacity of the foundation rocks before constructing the dam foundation is known as the bearing capacity evaluation of the foundation rocks. The foundation material's bearing capacity is a crucial parameter in dam design. Various empirical techniques were employed to determine the secure load-bearing capacity of foundation rocks. Shirzad and Nikkhah (2021) found that rock structure design requires accurate rock mass strength and deformation estimates. Settlement and stress from structure loading should not exceed soil and medium bearing capacity and settlement. Since rocks are stronger than concrete, rock foundations no longer have bearing capacity issues (Duncan and Hancock, 1966; Wyllie and Mah, 2004). The bearing capacity of rock masses depends on their quality, and discontinuities can significantly reduce it. A rock structure has different conditions than an earth structure. Rock and soil media differ in two ways: rock strength at higher loads and discontinuities in rock masses that affect rock strength (Carter and Kulhawy, 1992). Thus, subsurface strata engineering performance and bearing capacity affect foundation design. Bearing capacity was calculated from several equations using the shear strength parameter derived from the Hoek-Brown empirical failure criterion.

The engineering performance of the subsurface strata and its bearing capacity are crucial factors in determining the construction of a foundation. Bowles (1996) provides an equation (1) and (2) to determine the ultimate bearing capacity (Q_{ult}) and allowable bearing capacity value (q_{all}) of a rock mass.

$$Q_{ult} = UCS_{av} \left(\frac{RQD_{av}}{100} \right)^2 \quad (1)$$

$$Q_{all} = \frac{Q_{ult}}{FOS} \quad (2)$$

Where Q_{ult} = Ultimate bearing capacity, Q_{all} = allowable bearing capacity, FOS = factor of safety, RQD_{av} = average rock quality designation, and UCS_{av} = average uniaxial compressive strength. A safety factor of 3 was utilised to calculate the safe load-bearing capacity.

However, this equation has its own limitations as it does not consider the shear strength parameters, slope of the ground, width of the foundation etc. Hence, the bearing capacity of the foundation rocks was further determined using equations forwarded by Wyllie (1992).

Conditions vary depending on whether a structure is built on rock or on soil. There are two major distinctions between rock and soil media: the strength of the rock with greater loads and the presence of discontinuities in rock masses that result in a different strength level than the complete rock (Carter and Kulhawy, 1992).

According to Carter and Kulhawy (1992), the equation for calculating the ultimate bearing capacity of a rock mass can be suggested as the following:

$$q_u = UCS[S^a + (m_b S^a + S)^a] \quad (3)$$

Wyllie (1992) devised an equation to calculate the permissible bearing capacity (q_a) for rocks that are tightly fractured and weak and are situated on a mostly flat surface. This equation (4) is based on the Hoek-Brown strength criteria.

$$q_a = \frac{C_{f1} S^{0.5} UCS [1 + (m_b S^{-0.5} + 1)^{0.5}]}{FOS} \quad (4)$$

Where UCS is the uniaxial compressive strength of the intact rock, FOS is the safety factor.

Assume that the foundation is situated on a sloped surface. Therefore, it is necessary to adjust the bearing capacity factors to consider the reduced lateral resistance caused by the smaller rock mass on the downslope side of the foundation. This information is based on the research conducted by Bell (2007) as referenced in Wyllie (1992). The permissible working load of the foundation is typically determined by the bearing capacity or settlement for shallow slopes with an angle of repose less than $\phi/2$. When the slope angles exceed $\phi/2$, it is usually unnecessary to verify the bearing capacity because the primary factor is the stability of the slope (Shen, 1998; Shen et al., 2012). The calculation for the allowable bearing pressure for foundations on sloped surfaces is as follows (equation 5):

$$q_a = \frac{Cf_1 C N_{cq} + Cf_2 \left(\frac{B\gamma}{2}\right) N_{\gamma q}}{FOS} \quad (5)$$

Cf_1 and Cf_2 are correction factors, N_{cq} and $N_{\gamma q}$ are bearing capacity factors and depend on the slope angle (β), C is the cohesion, B is the width of the foundation, FOS is the factor of safety, and γ is the unit weight of the rock. H is the slope height and N_o is the stability number, which is calculated as follows:

$$N_o = \frac{\gamma * H}{c} \quad (6)$$

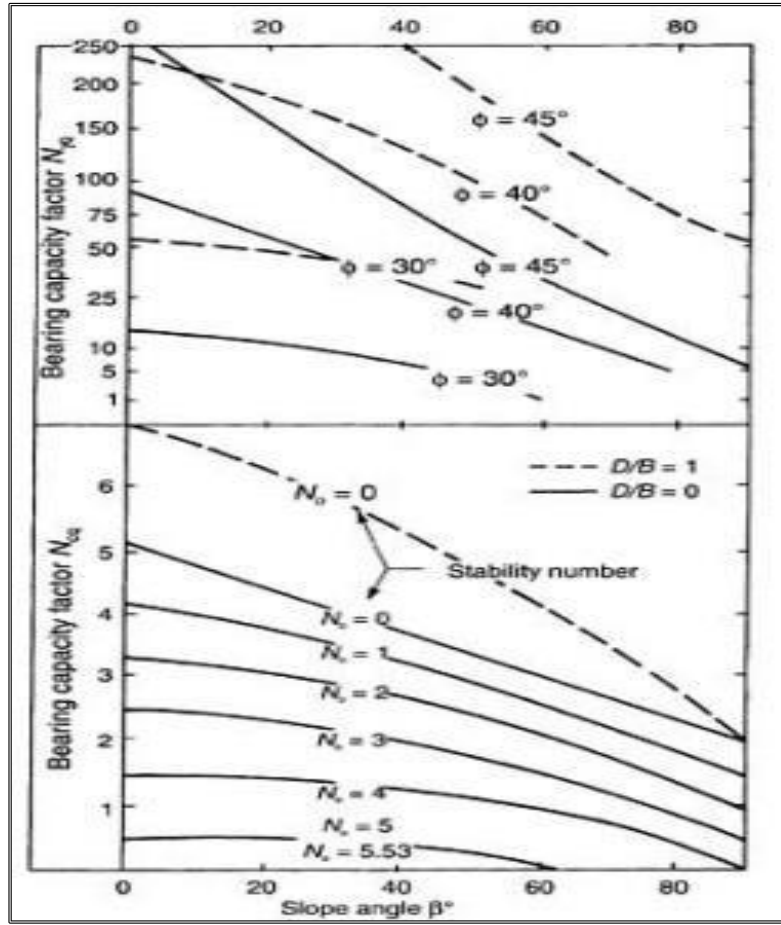


Figure 2. 1. Bearing capacity considerations for sloping ground surface footings (US Department of the Navy, 1982).

Bearing capacity problems usually related to the details of the structural geology so that bearing capacity of foundation should be described based on the geological condition of the site. Accordingly, the bearing capacity of fractured and weathered condition, shallow dipping bedded plane, layered formation and karstic formations behaves in different ways (Garo and Meten, 2021).

2.3.3. Rock Mass Strength and Failure Criteria

Evaluating the strength of rock masses is crucial for any type of rock excavation, whether it is an open-pit or underground operation. The strength of numerical models as an analytical tool is characterized by a strength envelope. The strength of the rock mass is determined by the combined structure of the solid rock and its discontinuities, such as cracks and joints. Bieniawski (1989) introduced empirical equations to determine shear strength parameters,

specifically cohesion and angle of internal friction, through the utilization of the Rock Mass Rating System (RMR).

Bieniawski (1979) provides a definition of failure criteria as a mathematical representation of the mechanical state in which a material breaks or deforms beyond a predetermined threshold. The failure criterion is the stress condition in engineering where the ultimate strength is reached. The expression of rock strength is commonly described in relation to the primary compressive stress, σ_1 , which the rocks can withstand under specific values of the other two principal stresses, σ_2 and σ_3 . This can be represented as $\sigma_1 = f(\sigma_2, \sigma_3)$ or $f(\sigma_1, \sigma_2, \sigma_3) = 0$. The "Mohr-Coulomb" and "Hoek-Brown" failure criteria are commonly employed in rock mechanics. However, it is important to note that these criteria consider only the major and minor principal stresses, neglecting the intermediate principal stress.

The Hoek–Brown criterion developed as a means to estimate rock mass strength by scaling the relationship derived according to the geological conditions present (Hoek and Brown, 1980). The failure criterion first introduced in an attempt to provide input data for the analysis required for the design of underground excavations in hard rock and, derived from the results of research into the brittle failure of intact rock and on model studies of jointed rock mass behavior. The criterion started from the properties of intact rock and then introduced factors to reduce these properties based on the characteristics of joints in a rock mass (Hoek et al., 2002; Hoek and Diederichs, 2006). The Hoek-Brown model used to represent intact rock or highly fractured rock masses. However, it should not use for massive rocks with only one or two joint sets. In the case of intact rock masses having only one to two joint sets, wedge analyses should be used (Goodman, 1991). The original Hoek-Brown criterion in terms of normal stresses for intact rock given by Equation (7):

$$\sigma_1 = \sigma_3 + \sigma_{ci} \left(m + \frac{\sigma_3}{\sigma_{ci}} + S \right)^{0.5} \quad (7)$$

σ_1 and σ_3 represent the major and minor effective principal stresses at failure, respectively. σ_{ci} refers to the uniaxial compressive strength of the intact rock material. The variables m and s are material constants, with S being equal to 1 for intact rock.

When applying the original Hoek-Brown failure criteria to connected rock masses, the predicted strengths are accurate as long as the minor principal effective stress (σ_3) has a significant compressive value. When σ_3 is small, the criterion overestimates the uniaxial strength and also predicts a non-zero tensile strength.

Rock mechanics engineers assert that the Hoek-Brown failure criterion is applicable to jointed rock masses that possess no tensile strength. Over the last three decades, numerical models for rock mechanics have incorporated a 'no tension' feature, enabling the transfer of tensile stresses from the model onto neighboring elements (Hoek et al., 1998).

Considering the inadequacy of the original Hoek-Brown criterion when applied to rock masses with joints, it was determined that a revised criterion should be formulated. The new criterion must align with the original criterion's predictions for compressive stress conditions and accurately forecast a tensile strength of zero for the rock mass. The expression for a modified criterion that fulfils these conditions is given by equation (8):

$$\sigma_1 = \sigma_3 + \sigma_c \left(mb \frac{\sigma_3}{\sigma_c} \right)^a \quad (8)$$

Where, mb and a are reduced value of the material constants.

Furthermore, it was acknowledged that the Rock Mass Rating system developed by Bieniawski was no longer sufficient in establishing a connection between the failure criterion and geological observations in the field, especially for extremely fragile rock masses. In 1992, Hoek, Wood, and Shah introduced the Geological Strength Index (GSI).

The Generalized Hoek-Brown criterion seeks to modify the shape of the principal stress plot, also known as the Mohr-envelope, by replacing the square root term in the original equation with a variable coefficient ' a ' (Hoek and Brown, 1980).

2.3.4. Rock Mass Deformation Characteristics

Rock mass deformation, as defined by Palmström et al., refers to the changes in shape or structure that occur in a collection of rocks. Rock mass deformation, as defined by Palmström and Singh (2001), refers to the process of discontinuities closing and the intact rock within a rock mass undergoing plastic and elastic deformation when subjected to a static or dynamic load. The measurement is conducted by calculating the modulus of deformation (E_d), which encompasses both plastic and elastic deformations resulting from the closure of joints in the rock mass (Johnson and DeGraff, 1988).

Deformability is defined by a modulus that quantifies the correlation between the load applied and the resulting strain. The non-elastic behavior of jointed rock masses has led to the adoption of the term "modulus of deformation" instead of "modulus of elasticity" or "Young's modulus" (Palmström and Singh, 2001).

Various scholars provide empirical assessments of the rock mass's deformability. In the 1960s, there were several efforts to utilize Deere's RQD for estimating the deformation modulus in its original position. However, this method is rarely employed in modern times (Deere and Deere, 1988). Seraphim and Pereira (1983) conducted a back analysis of the measured deformation in dam foundations and developed an empirical relation. This relation is applicable to rock masses with a Rock Mass Rating (RMR) between 10 and 50, and an unconfined compressive strength (UCS) of intact rock greater than 100 MPa. The Serafim and Pereira relation for $RMR_{89} < 50$ is expressed in Equation 9 (Seraphim and Pereira, 1983). Where RMR_{89} represents RMR obtained using Bieniawski (1989) rock mass classification system.

$$E_d = 10^{(RMR-10)/40} \quad (9)$$

Where, E_d is in situ modulus of deformation in GPa and RMR is rock mass rating.

Bieniawski (1978) introduced an empirical equation (Equation 10) to calculate the modulus of deformation " E_d " based on the Rock Mass Rating (RMR). The relationship between the rock mass with a Rock Mass Rating (RMR) above 55 and a Uniaxial Compressive Strength (UCS) of intact rock greater than 100 MPa is highly consistent with the tested values. When the Rock Mass Rating (RMR) is greater than 50, according to Bieniawski's classification in 1978:

$$E_d = 2RMR - 100 \quad (10)$$

Hoek et al.(1998) introduced an empirical equation (Equation 11) for calculating the modulus of deformation of a rock mass. This relationship is effective for rock masses with an Unconfined Compressive Strength (UCS) of less than 100 MPa.

$$E_d = \left[\frac{q_c}{100} \right]^{0.5} \times 10^{\left(\frac{GSI-10}{40} \right)} \quad (11)$$

The equation relates the uniaxial compressive strength (UCS) of intact rock at its natural moisture content, given in megapascals (MPa), to the modulus of deformation (E_d) and the Geologic Strength Index (GSI). If RMR_{76} is greater than 18, then GSI is equal to RMR_{76} . If RMR_{89} is greater than 23, then GSI is equal to RMR_{89} plus 5. RMR_{76} denotes the RMR value obtained through the utilization of the Bieniawski (1989) rock mass classification system.

Hoek et al. (2002) introduced an empirical categorization of the modulus of deformation and its characteristics related to deformability (Table 2.2.). Rocks with a modulus of deformation less than 5 demonstrate a high level of deformability, whereas rocks with a modulus of deformation greater than 60 display a very low level of deformability.

Table 2. 2. Classification of intact rock based on modulus of deformation (After, Hoek et al., 2002).

Class	Modulus of Deformation (GPa), (Hoek et al., 2002)	Deformation Characteristics
1	<5	Very High
2	5-15	High
3	15-30	Moderate
4	30-60	Low
5	>60	Very Low

2.4. Slope Stability Analysis

Slope stability analysis attempts to evaluate the secure design of both artificial and natural slopes, as well as the conditions of equilibrium (Abramson et al., 2002). The slope refers to the level of resistance exhibited by an inclined surface when it comes to the risk of sliding or collapsing, potentially resulting in the loss of human life and property. Consequently, it is crucial to assess the stability of the suggested slopes (Abramson et al., 2002).

The slope of the earth can be categorized into two types: natural slopes, which are formed by natural factors and occur in nature, and man-made slopes, which are created by human activities such as the sides of cuttings, embankments for roads, railway lines, canals, and earth dams for water storage (Murthy, 2003). Salunkhe et al.(2017) classified natural and manmade slopes into two categories: finite slopes and infinite slopes, based on their degree of inclination. When a slope has a limited length of inclination, it is categorized as a finite slope. On the other hand, if a slope represents the boundary surface of a semi-infinite soil mass, or if the inclination extends to an unlimited length and the soil properties remain constant at all depths below the surface, the slope is referred to as an infinite slope. The stability of a slope can be assessed by considering the factor of safety. This can be done by analyzing either the factor of safety for shearing strength, which is the ratio of shear strength (resistance to shearing) to the shear stress (actual shear strength) along the sliding surface, or the factor of safety for cohesion, which is the ratio of the available cohesion intercept to the mobilized cohesion intercept (Abramson et al., 2002; Murthy, 2003).

Additionally, there exists a friction safety factor that represents the ratio between the available strength of friction and the strength of friction that is actually utilized. Multiple techniques and methodologies exist for the assessment of slope stability. The methods can be classified into four categories: kinematic analysis, limit equilibrium, numerical modeling, and empirical methods (Gurocak et al., 2008; Basahel and Mitri, 2017; Asfaw et al., 2022; Tesfaye et al., 2023).

2.4.1. Kinematic Analysis

The stereo net projection method is a highly effective technique used in kinematic analysis to predict potential structural failure modes, such as planar, wedge, and toppling. It serves as a valuable tool for systematic data collection and visualization (Siddique et al., 2017; Asfaw et al., 2022; Tesfaye et al., 2023). According to Gischig et al.(2011), kinematic analyses are valuable for studying possible failure modes of rock masses that include discontinuities. This approach presupposes that all discontinuities are inflexible, completely enduring, devoid of moisture, and lacking cohesion. Furthermore, the blocks encounter additional stresses that were not considered in the analysis, in addition to the lateral limitations. Failures involving movements of rock blocks on discontinuities combine the plane, wedge, and toppling modes of failure, which can occur due to the presence of unfavorably oriented discontinuities. Kinematic analysis may be ineffective in cases where the rock mass exhibits closely spaced and low-persistence joints, as well as in situations involving rotational mode failures. Kinematic analysis is conducted by employing stereographic projection plots, which can be manually created using a stereo net or by utilizing Dips 6.0 software (Rocscience, 2004).

2.4.2. Limit Equilibrium Methods

The limit equilibrium method (LEM) quantifies the factor of safety for slope instability in rock and soil. It does this by assessing the magnitudes of the driving and resisting forces acting along the sliding planes, where longitudinal or rotational movements occur on various failure surfaces (Eberhardt, 2003; Faramarzi et al., 2017).

An analysis of limit equilibrium was conducted for the structures where the potential for kinematic instability had been identified. The shear strength parameters, specifically the friction angle (ϕ) and cohesion (c), were then utilized to determine the factor of safety (FOS) (Park and West, 2001). The FOS, or Factor of Safety, is a quantitative measure that represents the ratio of resisting forces to driving forces. A factor of safety (FOS) of 1.0 is considered the critical point at which the stability of the rock slope transitions from stable to unstable.

When the Factor of Safety (FOS) is less than 1.0, the driving forces exceed the resisting forces, resulting in slope movement (Ureel and Momayez, 2014). Conversely, if the FOS is greater than one, the resisting forces are greater than the driving forces. Limit equilibrium analysis involves dividing the sliding mass into multiple slices and calculating the shear and normal interslice forces. This is done by considering the force and moment equilibrium for each individual slice (Memon, 2018).

The LEM methods commonly employed for slope stability analysis comprise Fellenius (1936), Bishop (1956), Janbu (1955), Morgenstern and Price (1965), and Spencer (1967). Limit equilibrium method computes the factor of safety by considering the inter-slice normal and shear forces. Nevertheless, the fundamental distinction lies in the method by which these forces are established, and the calculations for Factor of Safety (FOS) are contingent upon the configuration of the crucial slip surface (Memon, 2018).

The LEM has the capability to assess a broad spectrum of failure modes in relation to safety factors. Nevertheless, Faramarzi et al. (2017) argued that the LEM failed to account for the stress and deformation in the rock mass and did not calculate the displacement and velocities of the failure surface (Ureel and Momayez, 2014). The limitation of the limit equilibrium method is addressed in slope stability analysis through the application of the numerical Finite Element Method (FEM). This method utilizes the shear strength reduction technique, as described by Jiang et al. (2015).

Presently, engineering practices extensively employ limit equilibrium slope stability analysis software, such as Slide software, Swedge software, and RocPlane software, which are developed within the Rocscience computer programme (Rocscience, 2004). This computer programme is utilised for conducting slope stability analysis for both rock and soil slopes. The Slide computer software is a 2D slope stability programme used to assess the stability of circular or non-circular failure surfaces in soil or highly weathered and fractured rock slopes (Rocscience, 2004). The factor of safety was determined using Slide software, utilizing the Mohr-Coulomb failure criterion excluding tension cracks (Rocscience, 2004). Swedge and RocPlane software were employed to analyze the stability of wedge and planar failure modes, respectively, in two dimensions.

2.4.3. Numerical Methods

Numerical modelling techniques provide solutions to problems involving intricate slope geometries and failure processes that cannot be effectively addressed using kinematic and limit equilibrium methods (Eberhardt, 2003). In addition, numerical modelling considers the rock mass deformation, the intricate slope behaviour, and stress distribution. It also accounts for the influence of stress distribution and displacements on the slope's behaviour. Limit equilibrium and kinematic analysis, as described by (Jiang et al., 2015), do not consider these factors.

Hence, the numerical model assesses the stability of the slope in engineering applications by offering a direct quantification of stability in relation to the factor of safety for the failure planes (Gurocak et al., 2008). Eberhardt (2003) classify the numerical methods used for rock slope stability assessments into three approaches: continuum modelling, discontinuum modelling, and hybrid modelling. Continuum approaches encompass both the finite difference and finite element methods. Both of these methods involve the division of the problem domain into a collection of sub-domains or elements. Finite Difference Methods (FDM) utilize the differential equations of equilibrium, the strain-displacement relations, and the stress-strain equations to determine the solution procedure. Alternatively, the procedure can utilize approximations of the connectivity of elements and the continuity of displacements and stresses between elements, similar to the Finite Element Method (FEM). One advantage of Finite Element Method (FEM) compared to traditional Limit Equilibrium (LE) methods is that it allows for the calculation of Factor of Safety (FOS) based on a predetermined failure surface. The Finite Element (FE) method maintains overall balance until failure occurs and is capable of detecting gradual failure, including complete shear failure (Shivananda et al., 2017). The continuum method is the most appropriate and commonly employed approach for analyzing the stability of rock slopes consisting of massive rock, intact rock, weak rock, and highly jointed rock masses (Eberhardt, 2003). These methods have a primary benefit in that they enable material deformation and failure, incorporating concepts of factor of safety.

The process of finite element analysis is conducted utilizing Phase 2 software. Additionally, it provides the user with an overview of the overall structure of the rock slope and anticipated instances of failure. The discontinuum method is suitable for rock slopes that consist of multiple joint sets, which govern the failure mechanism. Discontinuum methods view the

problem domain as a collection of separate bodies or blocks that interact with each other and experience external loads. These bodies are expected to undergo significant motion over time (Eberhardt, 2003). According to Eberhardt (2003), there is a growing trend in using hybrid approaches for rock slope analysis. This involves combining limit equilibrium stability analysis with finite element groundwater flow and stress analysis. The shear strength reduction (SSR) approach is a widely used method for conducting slope analysis using the finite element method (FEM) (Griffiths and Lane, 1999; Gurocak et al., 2008).

2.5. Engineering Geological Characterization of Rocks

Rocks are utilized as construction materials to construct a structure that is entirely comprised of rock, a structure that is erected on a rock, or a structure that is built within a rock. Intact rock is a term used to describe a solid piece of rock that is free from any flaws or imperfections. Its hydraulic and mechanical properties are determined by the petrographic characteristics of the material, regardless of whether it is in a natural or decomposed state. In situ rock refers to a rock mass that typically contains imperfections, such as fractures or cavities, which divide the mass into intact rock blocks and influence the hydraulic and mechanical properties. The characterization of rock mass involves the assessment of both the properties of intact rock material and the presence of discontinuities. Bieniawski (1989) asserts that rock engineering classification systems have a crucial impact on the characterization and design of structures.

The foundation or stability of a slope is entirely contingent upon the physical attributes of the rock mass. To assess the appropriateness of the rock mass, it is crucial to classify rocks according to specific engineering attributes such as strength, deformability, and permeability. This may also assist in implementing appropriate corrective actions to mitigate the potential detrimental impacts associated with the strength and deformability properties of the rock mass (Johnson and DeGraff, 1988).

2.5.1. Discontinuity Characterization

A survey of discontinuities in rock outcrops provides crucial data on the geological structure and mechanical characteristics of rocks. This information aids in evaluating the stability of rock slopes, estimating the quality of the rock mass, and predicting how the rock mass will respond to excavation and other construction activities. To gain a comprehensive understanding of the impact of discontinuities on rock mass behavior, including

permeability, deformability, and strength, it is essential to characterize and quantify the relevant discontinuity survey. Therefore, the systematic mapping of geological structures on the surface helps comprehend their response to various natural and human-induced forces. This process involves mapping and measuring fractures, joints, bedding planes, and other discontinuities within rock masses. Parameters such as spacing, orientation (dip amount and direction), condition of discontinuities (length/persistence, separation/aperture, roughness, infilling materials, and weathering), and groundwater conditions of dam site rock masses are measured. Additionally, the measurement of slope height and orientation is included (Hoek and Bray, 1981; Wyllie and Mah, 2004).

2.5.2. Determination of Uniaxial Compressive Strength (UCS) of rocks

One of the most essential parts of geotechnical and geological engineering is determining the Uniaxial Compressive Strength (UCS). This study sheds light on the mechanical behaviour of rocks under axial loading. The uniaxial compressive strength of intact rock material can be measured using laboratory testing of rock cores. Nonetheless, direct measurement of uniaxial compressive strength (UCS) is very expensive. However, it can be approximated by utilizing empirical formulas (Broch and Franklin, 1972) based on the point load strength index $Is(50)$, as well as by conducting laboratory and in situ tests to measure the Schmidt hammer rebound hardness value (R) (Aydin, 2009).

2.5.2.1. Point Load Index Strength Test

The point load test has often been reported as an indirect measure of compressive strength. Many researchers have investigated the relationship between uniaxial compressive strength (UCS) and point load index (Is) for different rock types. Broch and Franklin (1972) described the relation between uniaxial compressive strength and point load as follows in equation 12.

$$UCS = 23 \times Is(50) \quad (12)$$

Where $Is(50)$ is the size corrected point load strength.

2.5.2.2. Schmidt Hammer Rebound Value

The Schmidt hammer rebound value is the predominant indicator utilized in rock mechanics practice to estimate the uniaxial compressive strength (UCS) and modulus of elasticity (E) of undamaged rock (Aydin, 2009). During the test, it is important to consider various factors that can influence the Schmidt hammer rebound value. These factors include the types of hammers used, the direction of hammer impact, weathering, moisture, specimen

requirements (free of any visible crack), testing, data gathering and reduction, and analysis procedures (Aydin and Basu, 2005).

Barton and Choubey (1977) stated that uniaxial compressive strength (σ_c) of the rock can be estimated by multiplying the rebound number and the dry density of the rock using correlation as shown in equation 13.

$$\log_{10}(\sigma_{ci}) = 0.00088\gamma R + 1.01 \quad (13)$$

Where, σ_{ci} is the uniaxial compressive strength in MN/m^2 , γ is the dry rock densities in KN/m^3 and R is the average Schmidt rebound value.

2.5.3. Rock Mass Classification Systems

Goodman (1991) defines rock mass classification as a method of assigning a numerical rating to the quality and performance of a rock mass, using measurable parameters. Engineering rock mass classifications have gained significant popularity in recent times and are employed in feasibility designs. The purpose of a rock mass classification system is to categorize the rock mass according to its strength, discontinuity condition, weathering, structural orientation, and other factors. This classification system serves as a foundation for estimating the deformability and strength properties of the rock mass, providing quantifiable data for support estimation, and facilitating communication between exploration, design, and construction teams (Bieniawski, 1989).

The development of rock mass classification schemes can be traced back to the late 18th century when they were first devised as an empirical method for designing tunnels and determining support requirements (Hoek, 2007). The commonly employed rock engineering classification systems include RMR (Rock Mass Rating), Q-system, and GSI (Geological Strength Index) systems (Palmstrom, 2009). Table 2.3 presents several renowned classification systems.

Table 2. 3. The primary rock mass classification systems as documented by Coşar (2004).

Rock Mass Classification System	Originator	Country of Origin	Application Areas
Rock Load	Terzaghi (1946)	USA	Tunnels with steel Support
Stand-up time	Lauffer (1958)	Australia	Tunneling
New Austrian Tunneling Method (NATM)	Pacher et al. (1964)	Austria	Tunneling

Rock Quality Designation (RQD)	Deere et al. (1967)	USA	Core logging, tunneling
Rock Structure Rating (RSR)	Wickham et al. (1972)	USA	Tunneling
Rock Mass Rating (RMR) Modified Rock Mass Rating (MRMR)	Bieniawski (1973) (last modification in 1989-USA) Ünal and Özkan (1990)	South Africa Turkey	Tunnels, mines, (slopes, foundations) Mining
Rock Mass Quality (Q)	Barton et al. (1974) (last modification in 2002)	Norway	Tunnels, mines, foundations
Strength-Block size	Franklin (1975)	Canada	Tunneling
Basic Geotechnical Classification	ISRM (1981)	International	General
Rock Mass Strength (RMS)	Stille et al. (1982)	Sweden	Metal mining
Unified Rock Mass Classification System (URCS)	Williamson (1984)	USA	General
Communication Weakening Coefficient System (WCS)	Singh (1986)	India	Coal mining
Rock Mass Index (RMI)	Palmstrom (1996)	Sweden	Tunneling
Geological Strength Index (GSI)	Hoek and Brown (1997)	Canada	All underground excavation-

2.5.3.1. Rock Quality Designation

The Rock Quality Designation (RQD) is a crucial parameter utilized in rock mass classification systems, numerical modeling, and engineering evaluations of rock masses. The determination of RQD can be achieved through different approaches, including both direct and indirect methods (Singh and Goel, 2011). Deere and Deere (1989) states that the rock quality designation index (RQD) can be calculated using the direct method. This involves summing up the intact rock pieces larger than 10cm in each core run, as indicated in equation (14). Additionally, the quality of the rock mass can be determined by referring to Table 2.4 for its engineering classification.

$$RQD = \frac{\text{sum of core piece} \geq 10\text{cm}}{\text{total drill run}} * 100 \quad (14)$$

RQD can be indirectly estimated by utilizing seismic wave velocities and the JV value in cases where the core is unavailable. JV is determined by considering both joint spacing and frequency as indicators of the primary joint system. Palmstrom (1982) proposes an empirical equation (equation 15) to estimate RQD indirectly.

$$RQD = 115 - 3.3 J_v \quad (15)$$

Where; J_v is the volumetric count; it is a measure for the number of joints within a unit volume of rock mass defined by equation 16 as follows:

$$J_v = \sum_{i=1}^J \left(\frac{1}{S_i} \right) \quad (16)$$

Where S_i is the average joint spacing in meters for the i^{th} joint set. Random joints may also be considered by assuming a 'random spacing'. Experience indicates that this should be set to $S_r = 5\text{m}$ (Palmström, 1996). Thus, J_v can also be generally expressed by equation 17:

$$J_v = \sum_{i=1}^J \left(\frac{1}{S_i} \right) + \frac{N_r}{5} \quad (17)$$

Where; N_r is number of random or irregular joints.

After the RQD value is determined from J_v value, the rock mass quality of the rock mass is determined by the criteria given by (Deere and Deere, 1988). From J_v value also the degree of jointing and fracturing could be determined (Table 2.5).

Table 2. 4. Engineering classification of rock mass quality according to Deere and Deere (1988)

RQD	90 – 100	75 – 90	50 – 75	25 - 50	< 25
Rock mass quality	Excellent	Good	Fair	Poor	Very poor

Table 2. 5. Classification of the degree of joint based on J_v .

	Degree of Jointing					
	Very low	Low	Moderate	High	Very High	Crushed
J_v Value	<1	1-3	3-10	10-30	30-60	>60

Joint spacing refers to the perpendicular separation between two joints that belong to the same joint set. It is utilised as one of the six input parameters in the RMR (rock mass rating) system, as described by Palmstrom in 2005. The spacing between joints plays a significant role in the classification of rock masses. The existence of joints in a rock mass inherently weakens its strength, and the extent of this weakening is determined by the spacing between the joints (Bieniawski, 1973).

2.5.3.2. Rock Mass Rating

The Rock Mass Rating (RMR) is an empirical method utilized to assess the quality and behavior of rocks, relying on the inherent and structural parameters of the rock mass (Stille and Palmström, 2003). The RMR classification system was originally devised by Bieniawski (1979) at the South African Council of Scientific and Industrial Research to assess the integrity of rock formations for subterranean endeavors. The Rock Mass Rating System (RMR) utilizes six parameters as input. The parameters used to estimate RMR are the

uniaxial compressive strength of rocks (UCS), rock quality designation (RQD), spacing of discontinuities, condition of discontinuities, groundwater conditions, and orientation of discontinuities (Bieniawski, 1989). Each parameter is categorized and assigned a specific rating to indicate the quality of the rock mass.

Table 2. 6. Rock mass classification based on RMR-value (After Bieniawski, 1989)

Rock mass Rating	100 - 81	80 – 61	60 - 41	40 - 21	20 - 0
Rock class	I	II	III	IV	V
Classification	Very good	Good	Fair	Poor	Very poor

Multiple classification systems should be used, according to Bieniawski (1989), to allow for comparison and reliability verification of the results obtained. The range in which a rock mass's cohesion and angle of friction will be found is specified by the RMR. For this purpose, Bieniawski (1976) suggested the following relationships in order to determine the cohesion (equation 18) and angle of friction (equation 19) for a specific RMR values as follows:

$$C = 0.05RMR \quad (18)$$

$$\Phi = 0.5RMR + 5 \quad (19)$$

Where, Φ = angle of internal friction (degree), and C = cohesion (MPa).

Discontinuity characteristics which are used as a parameter of the basic RMR such as; spacing between discontinuities and condition of discontinuities (aperture, persistence, roughness, infilling, and alteration/weathering) that directly influence the characteristics of the rock mass by reducing the strength and the parameters were described and classified from surface exposure or drill core samples (Bieniawski, 1973). Furthermore, groundwater conditions can strongly influence rock mass behavior. The groundwater rating varies according to the conditions encountered and the general conditions can be described as completely dry, damp, wet, dripping and flowing with a higher rating for a completely dry rock mass (Bieniawski, 1989).

2.5.3.3. Geological Strength Index

The Geological Strength Index (GSI) is a classification system used to describe the mechanical properties of rock masses with numerous joints (Hoek et al., 1998). Recently, GSI has become a crucial tool in different engineering fields such as tunneling, mining, and underground structure design. Its importance lies in guaranteeing the safety, efficiency, and cost-effectiveness of many engineering projects (Marinos and Hoek, 2000). The technique

serves as a crucial method for assessing the quality of a rock mass in terms of its strength, deformation characteristics, and overall performance (Cai et al., 2004).

In 1998, Hoek and his colleagues utilized Bieniawski's RMR (Rock Mass Rating) System to compute the material constants through this empirical methodology. The method demonstrated efficacy for rock masses possessing a Rock Mass Rating (RMR) exceeding 25. However, it was not successful for individuals whose rock mass rating (RMR) was below 25.

$$\text{For RMR}_{76} > 18 \quad \mathbf{GSI} = \mathbf{RMR}_{76} \quad (20)$$

$$\text{RMR}_{89} > 23 \quad \mathbf{GSI} = \mathbf{RMR}_{89} - 5 \quad (21)$$

According to Wyllie and Mah (2004), the GSI value is determined by assessing the weathering condition of the discontinuity surface, the structure of the rock mass, and the interlocking characteristics of rock blocks, especially in rock outcrops. In order to determine the characteristics of a rock, it is necessary to initially assess the Geological Strength Index and m_i , and subsequently utilize the following equation 22.

$$\sigma_1 = \sigma_3 + \sigma_c \left(mb \frac{\sigma_3}{\sigma_c} + S \right)^a \quad (22)$$

Where mb is the reduced value of the material constant m_i and is given by:

$$mb = m_i \exp\left(\frac{GSI-100}{28-14D}\right) \quad (23)$$

m_i for intact rock is determined from the laboratory triaxial test or the tabulated data proposed by Hoek et al. (2000) for different rock types.

S and a are the constants for the rock mass given by the following expression.

$$S = \exp\left(\frac{GSI-100}{9-3D}\right) \quad (24)$$

$$a = \frac{1}{2} + \frac{1}{6} \left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}} \right) \quad (25)$$

D is a variable that depends on the level of blasting damage and stress relaxation that the rock mass has experienced. It ranges from 0 for in situ rock mass that hasn't been disturbed to 1 for highly disturbed rock mass.

Marinos et al., (2007) developed a failure criterion for rock masses with joints, and also proposed an empirical method to assess the strength of these rock masses. The criterion was revised based on the findings from two distinct projects: Hoek and Brown (1988) and Hoek et al. (1992). In addition, through visual examination of the outcrop, it is possible to

approximate the Geological Strength Index (GSI) of rock formations. The following diagram illustrates the fundamental GSI chart as presented by Marinos and Hoek (2000).

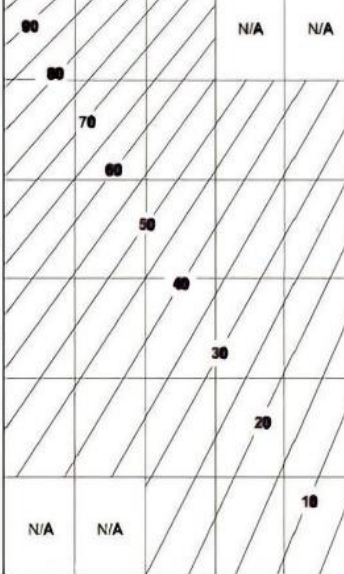
GEOLOGICAL STRENGTH INDEX (GSI)		JOINT SURFACE CONDITION				
According to rock mass structure and discontinuity surface conditions observed on the rock mass at site, estimate the average or range of the GSI value.		VERY GOOD – very rough, fresh, unweathered joint surfaces GOOD – rough, slightly weathered, stained joint surfaces FAIR – smooth, moderately weathered, and altered surfaces POOR – slickensided, highly weathered surfaces with coating or fillings or fragments VERY POOR – slickensided, highly weathered, surfaces with soft clay coating or filling				
ROCK MASS STRUCTURE		Decreasing of Surface Quality →				
	MASSIVE – mass in situ rock mass with few widely spaced joints					
	BLOCKY – very well interlocked undisturbed rock mass consisting of cubical blocks formed by three orthogonal joint sets					
	VERY BLOCKY – interlocked, partially disturbed rock mass with multi-faced angular blocks formed by four or more joint sets.					
	BLOCKY/FOLDED – folded and faulted with many intersecting discontinuities forming angular blocks.					
	DISINTEGRATED – poorly interlocked, heavily broken rock mass with a mixture of angular and rounded blocks.					
	SHEARED/LAMINATED – lack of blockiness due to close spacing of weak schistosity or shear zones.	N/A	N/A			10

Figure 2. 2. Estimation of Geological strength index GSI based on geological descriptions (Marinos and Hoek, 2000).

2.6. Engineering Characterization and Classification of Soils

The characterization and classification of soils are vital components of geotechnical engineering, as they furnish indispensable data for the design of foundations, analysis of slope stability, and other geotechnical structures. Soil classification aids engineers in comprehending the response of soils to various loads and environmental circumstances. Soils are commonly employed as a primary material in the construction of various earthworks, such as dams, highways, and other structures. Soils in dam projects have two main functions: they serve as embankments for the dam bodies and act as protective layers for permeable reservoir areas. Soil engineering classification entails the classification of soils into distinct groups and subgroups according to their engineering characteristics. There are several methods available for the engineering classification of soils, including textural soil

classification, the American Association of State Highway and Transportation Officials (AASHTO), and the Unified Soil Classification System (USCS) (Das, 2002, 2010, 2011).

The textural soil classification method focuses only on soil texture and does not consider plasticity, which makes it less applicable for engineering classification. AASHTO considers both texture and plasticity when classifying soils, making it especially useful for categorizing soils in the context of highway construction. Unlike AASHTO, the USCS integrates both textures and plasticity, making it the most essential system for a wide range of engineering applications. The American Society for Testing Materials (ASTM) has also adopted this system, which is widely acknowledged as the primary approach for tackling various engineering problems related to soils (Arora, 2008).

Basically, in soil mechanics properties of soil are broadly classified as Index Properties and Engineering Properties. The properties of soil which are used for identification and classification are called index properties of soil such as specific gravity, water content, particle size distribution and Atterberg limits (Jadar, 2018). Accordingly engineering properties of soils were permeability, shear strength, compressibility. Moisture content can also be defined as the natural moisture content is the ratio of the weight of water to the weight of the solids in a given mass of soil. This ratio is usually expressed as a percentage. By determining the moisture content, the amount of water present in a soil sample can be known and the compaction characteristics of soils can be critically evaluated. The specific gravity is the ratio of the weight of soil solids to the weight of an equal volume of water content at standard temperature. Specific gravity measures the density of soil particles compared to the density of water and helps in calculating void ratios and porosity. Particle size analysis characterizes soils in determining the distribution of particle sizes in a specific soil sample. The common methods include sieve analysis for coarse-grained and hydrometer analysis for fine-grained soils. Atterberg limits consist of three limits including liquid limit, plastic limit, shrinkage limit, and plasticity index. These limits indicate the range of moisture contents over which the soil behaves differently. The aforementioned properties of the soils are index properties of soils (Das, 2002, 2010, 2011; Jadar, 2018).

The suitability of a soil for a specific application is determined by its response to that particular use. The suitability of the soil is typically determined by one or more of its engineering properties. Engineering analysis is necessary to accurately assess the engineering characteristics of soil and forecast its performance for a specific application. The three main engineering properties of soils are shear strength, compressibility, and

permeability. The permeability of soils is an indicator of how easily water can flow through them. Estimating seepage discharge through earth masses necessitates its use. Soil deformation under compressive loads is linked to its compressibility. The structural composition of settlements built on soils must possess compressive properties. The shear strength of soil, which refers to its ability to withstand shear stresses, has a direct impact on the stability of slopes, the bearing capacity of soils, and the earth pressure on retaining structures (Arora, 2008; Das, 2010). From the engineering properties of soils, permeability of soil mass is discussed in section below.

2.6.1. Soil Permeability Characteristics

Soil permeability pertains to the flow of water through the soil, and it directly affects the functioning of engineering structures. Evaluating soil permeability is essential for determining structural stability and integrity. Laboratory permeability studies are mainly employed to evaluate the appropriateness of soils for various projects. The soil's permeability directly influences the speed at which water moves through it, which has a notable influence on several matters including excavations in sandy areas below the water table, seepage through dam bodies and beneath dam foundations, and the overall stability of engineering structures' foundations (Ranjan and Rao, 2005; Arora, 2008). Soil permeability tests can be categorized into two types: constant head tests and falling head tests. The constant head test is primarily employed to ascertain the permeability of coarse-grained soils, such as sand and gravel. Conversely, the falling head permeability test is commonly utilised for fine-grained soils (Das, 2002; Arora, 2008).

Soils are composed of individual particles, forming a complex network of interconnected uneven tubes. In a solid-liquid system, the gaps are typically filled with water, allowing unrestricted movement when a voltage difference is applied. The resistance to water flow is higher in soils with smaller pores and disorganized channels. Soils with larger voids or well-defined channels experience reduced flow resistance. Permeability varies among soils; sands are more permeable than silts, which, in turn, are more permeable than clays. Gravels exhibit even greater permeability than sands. Additionally, soil composition influences permeability, with loose sand being more permeable than compact sand, and clay soil with a flocculated structure having higher permeability than clay soil with a dispersed structure (Ranjan and Rao, 2005; Arora, 2008). The falling head permeability test is commonly utilised for fine-grained soils. The determination of permeability in the laboratory is achieved by employing either a constant head or a falling head permeameter (Arora, 2008; Fell et al., 2014).

2.6.1.1. Constant Head Permeability Test

Arora (2008) and Budhu (2011) state that the constant-head permeability test is used to determine the hydraulic coefficient of permeability for soils with high permeability ($k > 10^{-4}$ cm/s), such as coarse-grained sands and gravels. The constant-head permeameters used adhere to the specifications outlined in I.S.: 2720 (Part XVII). These permeameters are equipped with a metallic mould that has an internal diameter of 100 mm, an effective height of 127.3 mm, and a capacity of 1000 ml. The mould used for soil compaction is equipped with a detachable extension collar measuring 100 mm x 60 mm. The mould consists of a drainage base plate that contains a recess made of porous stone, as well as drainage caps equipped with air release and inlet valves. The drainage base and cap incorporate mould mounting fittings. Prior to inserting the soil sample into the mould, it is necessary to remove air from both porous discs, which have a permeability ten times greater than that of the soil. It is necessary to eliminate air from the water tubes.

Murthy (2003) states that soil can be introduced into the permeameter and compressed to the prescribed density, or it can be reshaped and compressed using a standard proctor to achieve the same density as found in nature for sample preparation. Once the sample is compressed in the mould, a dummy plate with a thickness of 12 mm and a diameter of 108 mm is installed on the base. Complete saturation of the sample is necessary for testing. Immerse the soil in a permeameter filled with water and allow water to flow upwards from the bottom into the mould. This is achieved by establishing a connection between the constant-head reservoir and the drainage base. Following the removal of air, there is a sustained upward flow for a sufficient duration (Arora, 2008; Budhu, 2011).

During testing, the sample's length (L) and cross-sectional area (A) are kept constant under a continuous head (h). The rate at which water flows through the sample is quantified by measuring the water discharge (Q) as a function of time (t) as given in equation (26). Darcy's Law is employed to determine soil permeability (k) as shown in equation (27).

$$Q = k \frac{h}{L} A t \quad (26)$$

By performing calculations for a given value of k , we will obtain:

$$k = \frac{QL}{hAt} \quad (27)$$

The variables in the equation are defined as follows: k represents the hydraulic conductivity, Q represents the discharge (volume flow rate) in m^3/sec , L represents the length of the soil specimen in meters, A represents the cross-sectional area of the soil specimen in m^2 , h represents the head difference in meters, and t represents the time in seconds.

2.6.1.2. Falling Head Permeability Test

The falling head permeability test is mainly employed for soils with low permeability, particularly fine-grained types such as silts and clays, which have permeability values ranging from 10^{-2} to 10^{-6} cm/s (Budhu, 2011). This testing method is similar to the constant head test, but the main difference is that it allows for a decrease in the head difference as water permeates the soil specimen, instead of keeping the head level constant (Murthy, 2003; Arora, 2008).

The permeameter mold employed in the falling head permeability test remains consistent with that used in the constant head permeability test. It features a vertical, graduated standpipe of specified diameter at the top, and the soil sample is positioned between two porous discs within the mold. Preceding the initiation of the test, the entire apparatus is placed within a constant head chamber filled with water. Prior to sample placement, the essential process of de-airing is conducted for both the water tubes and porous discs. If an in-situ, undisturbed sample is available, it is employed; alternatively, soil is obtained and compacted to the prescribed density within the mold.

To eliminate air from the soil, the valve located at the drainage base is sealed, and gradual vacuum pressure, reaching 700 mm of mercury, is applied through the drainage cap, sustained for approximately 15 minutes. Throughout the vacuum stage, de-aired water is permitted to ascend from the drainage base, saturating the soil sample. Once saturation is achieved, both the top and bottom outlets are sealed, and water is introduced into the standpipe until it attains the desired height (Murthy, 2003; Arora, 2008; Budhu, 2011).

The experimental procedure commences by allowing water in the standpipe to percolate through the soil sample into the constant-head chamber until it overflows. As this process unfolds, the water level in the standpipe gradually decreases. The test involves calculating the time it takes for the water level to drop from an initial head (h_1) to a specified final head (h_2), as specified by Arora in 2008. The head is determined by the quantity of water present in the constant-head chamber. The permeability of the soil can be ascertained by observing

changes in the head and measuring the time it takes for the head to transition from its initial to final value, employing the equation provided by Das in 2010:

$$k = 2.303 \times \frac{a(L)}{A(t_f - t_i)} \times \log_{10}\left(\frac{h_1}{h_2}\right) \quad (28)$$

The variables in the equation are as follows: A represents the cross-sectional area of the sample, a represents the cross-sectional area of the standpipe, L represents the length of the specimen, t_f and t_i represent the final and initial time intervals, respectively, h_1 represents the initial head, and h_2 represents the final head.

2.7. Comprehensive Remediation Approaches at Dam Sites

The construction of dams is a sophisticated engineering project that frequently encounters geological obstacles necessitating thorough remediation strategies. The outcome of these projects depends on effectively reducing the inherent risks linked to the geological conditions of the site. Geological challenges may arise from unstable soil, rock formations, or groundwater conditions, potentially jeopardizing the structural stability of the dam and posing safety hazards. In order to maintain the structural integrity of the dam and ensure that it serves its intended purpose, intensive consideration must be taken when building a dam and creating a reservoir in geologically difficult areas. Proactive corrective remediation steps become necessary when site investigation studies indicate the existence of unfavorable geological conditions (Haider et al., 2000; Bruce et al., 2012).

Comprehensive remediation approaches require a deep understanding of the geological characteristics of the site, which can be achieved through comprehensive site investigation studies. Subsequently, specialized engineering geological methods and technologies are utilized to tackle particular obstacles. Reinforcement measures, such as soil stabilization and ground improvement, are commonly employed to augment the overall stability of the construction site. Furthermore, the integration of monitoring systems guarantees immediate evaluation and timely identification of potential problems. Engineering geologists have a crucial role in the decision-making process, assessing the feasibility of corrective measures and considering alternative strategies if necessary. The cost-effectiveness and feasibility of these remediation methods are pivotal considerations in the decision-making process. If the challenges become impossible to overcome or too costly to address, the possibility of moving the dam site may be considered in order to prioritize safety and ensure long-term sustainability. Generally, it is crucial to implement comprehensive remediation strategies in

order to ensure the successful construction of dams, especially when dealing with geological difficulties. By integrating thorough geological analyses with inventive engineering solutions, these methods guarantee the construction of dams with the highest level of safety and durability, thereby promoting the development of sustainable infrastructure (Das, 2011; Asfaw et al., 2022).

Leakage and seepage issues at the dam foundation and reservoir area, as well as slope instabilities in the dam abutments and rims, are common issues that pose a challenge to the structural integrity, stability, and operational life of dam construction. As a result, it is necessary to approach these difficult problems in terms of their associated problems, which are covered below (Garo and Meten, 2021).

2.7.1. Dam Foundation Remedial Measures

Dam foundation treatment methods are techniques used to address specific vulnerabilities and engineering geological obstacles. The treatment of dam foundations is a crucial factor in guaranteeing the stability and security of dams, which are essential for the storage of water and the generation of energy. The choice of treatment for a dam's foundation depends on a number of factors, such as the type of foundation (soil or rock), where different techniques work better with different materials, the foundation's weaknesses (voids, cracks, or permeability), the size and type of the dam (the scale and load-bearing requirements of the dam influence treatment approach), and the cost and feasibility of the various options (different methods have different costs and implementation complexities) (Houlsby, 1991; Kazemian and Huat, 2009). The two main techniques used are grouting and constructing cut-off walls (Kazemian and Huat, 2009; Garo, 2020; Asfaw and Meten, 2021).

2.7.1.1. Grouting Methods

The grouting method is a commonly used technique for improving the ground in the area surrounding a dam foundation (Wahlstrom, 1974; Houlsby, 1991). The process involves drilling holes in the dam foundation, typically starting from the cutoff level, and injecting them with cement slurry or chemicals. This is done to fill geological defects in the rock, such as open joints, fractures, bedding parts, and faults (Wahlstrom, 1974). It is primarily conducted as a preventive measure against issues such as seepage through the foundation of a dam or other areas within its vicinity, as well as to address any uneven movement or settling of the foundation caused by geological faults. Consequently, for the safe operation of newly

built dams, it is typically necessary to enhance the stability of the ground by applying grouting techniques, particularly in the foundation area (Turkmen, 2003).

Grouting involves injecting cement or chemicals into the foundation to fill voids, consolidate soil, and reduce permeability. Cement grouting strengthens the foundation by forming a solid mass, while chemical grouting stabilizes fine-grained soils (Houlsby, 1991 ;Kazemian and Huat, 2009; Njock et al., 2018; Celik, 2019).

The execution of grouting is primarily determined by the outcome of the permeability (water pressure) test, which is used to divide the foundation rock mass into distinct zones based on their varying rock mass quality. These zones then require different levels of treatment (Sadeghiyeh et al., 2013). The grout curtain and the relatively impermeable rock strata must have a permeability (q) that satisfies the specified criteria, measured in Lugeon/Lu (USBR, 1977):

- For dams with a height exceeding 100 meters, the value of q falls within the range of 1Lu to 3Lu.
- Dams with heights ranging from 100m to 50m have a discharge rate (q) that falls within the range of 3 times the upstream length (Lu) and 5(Lu).
- For dams with a height below 50m, the permeability (q) is equal to 5(Lu). For pumped-storage power stations or reservoirs located in areas with limited water resources, the minimum value of the mentioned ranges will be used as the lower limit.

The most commonly implemented types of grouting methods are consolidation and curtain grouting. Consolidation grouting is the process of injecting a mixture containing cement into the ground in order to enhance the strength of the soil and decrease settlement. It is especially beneficial for stabilizing unstable or fragile soil beneath structures. Conversely, curtain grouting involves the injection of grout into a vertical or inclined surface to create a barrier that effectively seals off areas that are permeable, thus preventing the seepage of water. Both methods are essential for improving the stability and integrity of structures, such as dams and foundations. Customizing grouting techniques to suit particular geological conditions guarantees efficient soil remediation and enduring structural integrity (Cambefort, 1977; Houlsby, 1991; Bell, 2004; Kazemian and Huat, 2009; Fell et al., 2014; Njock et al., 2018; Celik, 2019).

2.7.1.2. Cut-off Walls

Cut-off walls, such as diaphragm walls or sheet piles, create impermeable barriers in the foundation to prevent water seepage. These walls intercept and redirect water flow, enhancing the dam's overall structural integrity. The selection of treatment methods depends on site-specific geological conditions, and often a combination of grouting and cut-off walls is employed for comprehensive foundation stabilization and seepage control in dam construction (Telling, 1975; Liu et al., 2016; Javanmard et al., 2018; Bestuzheva and Anakhaev, 2020).

2.7.2. Slope Stabilization Methods

Slope stabilization is essential for preventing slope failures and guaranteeing the security of communities and infrastructure. To improve slope stability, a number of techniques are used, such as shotcrete, rock bolts, rock anchors, and slope geometry modification. Modifying slope geometry entails making changes to the shape of the slope in order to achieve a stable arrangement. This may involve decreasing the incline, constructing terraces, or implementing retaining structures. These modifications alter the distribution of forces exerted on the slope, thereby reducing the likelihood of landslides. Rock bolts are extensively employed for the purpose of fortifying rock formations and mitigating the occurrence of rockfalls. These steel bolts are drilled into the rock and firmly anchored. They offer supplementary reinforcement and prevent the possible displacement or collapse of rock formations. Rock bolts are highly effective in stabilizing slopes in mountainous areas where rock instability is frequently a concern. Rock anchors are analogous to rock bolts, but they are specifically engineered for more profound or extensive stabilization purposes. The installation process involves the creation of holes in the rock, followed by the insertion of robust cables or bars that are secured in place by grouting. This effectively anchors the unstable rock mass. Rock anchors bolster the overall stability of the slope by enhancing cohesion and diminishing the likelihood of mass movements. Shotcrete, a technique that utilizes the application of high-velocity concrete or mortar, is used to strengthen and safeguard slopes. It creates a long-lasting, erosion-resistant coating on the uncovered rock surface, enhancing its durability and stability. Shotcrete is frequently employed in conjunction with rock bolts or anchors to establish a comprehensive slope stabilization system (Abramson et al., 2001; Abramson et al., 2002; Eberhardt, 2003; Wyllie and Mah, 2004; Salunkhe et al., 2017).

The choice of slope stabilization techniques is contingent upon the particular geological circumstances, slope attributes, and project specifications. Geotechnical engineers perform comprehensive site investigations to evaluate factors such as soil composition, rock type, and water infiltration, which inform the selection of suitable stabilization measures. Ultimately, the alteration of slope geometry, implementation of rock bolts, rock anchors, and application of shotcrete are essential elements of slope stabilization strategies. When used carefully, these techniques help to enhance the durability and security of slopes, reducing the likelihood of landslides and maintaining the stability of the surrounding environment and infrastructure (Abramson et al., 2001; Abramson et al., 2002; Tiedeu et al., 2020; Zhang et al., 2020).

2.8. Reviews on Selected Dam Site Investigations and Remedial Measure Recommendations

Ethiopia built micro-dams in the late 1970s to reduce drought (Tiruneh, 2005). Over 70 earth-fill dams have been built in Tigray regional state since the early 1990s, indicating a rise in construction activity. Dam foundations are mostly carbonate and shale. However, many small dam reservoirs have seepage and structural integrity issues (Abdulkadir et al., 2009 ; Abdurahman, 2009). In other regions, micro-dams around the Main Ethiopian Rift (MER) escarpment have had leakage issues (Abay and Meisina, 2015). Researchers have extensively studied dam foundation rock engineering geological properties to identify and describe harmful geological factors. To solve these problems, they propose solutions. A brief discussion of a few of these studies will be held in the following paragraphs.

In order to ascertain the engineering geological characteristics of the carbonates and shales at this dam site and how they relate to the construction of dams, Abay and Meisina, (2015) also conducted research. Consequently, this study has demonstrated that carbonates dominated by calcite are highly fractured, karstified, deformable, and have a rock mass quality rating of poor to fair. They also have a medium to high permeability rating, which has the potential to cause excessive leakage. Conversely, solution cavities with a moderate to high permeability range hardly ever affected those carbonates with lower calcite contents. Additionally, because of its extreme degree of weathering, the calcareous shale of this site was found to be watertight but to have very low rock mass quality (bearing capacity). Despite the lack of solution cavities or openings in the core logs, this study also showed that gypsum-shale intercalation occurs deep below the foundation and poses a risk to the dam's safety.

According to Abay and Meisina, (2015), an analysis of the rock properties at this dam site revealed that, while calcareous shale makes a good reservoir site due to its watertightness, its low bearing capacity makes the limestone layer unsuitable for construction.

Asfaw et al. (2022) conducted an analysis of leakage and abutment slope stability at the Arjo Didesa dam site in western Ethiopia. The purpose of the analysis was to assess the potential for leakage and evaluate the stability of the slope sections at the dam site. The assessment of leakage was conducted using data obtained from lugeon tests. The analysis of slope stability was determined through the utilization of kinematic analysis, limit equilibrium methods, and finite element methods. The study found that more than 57% of the Lugeon test results along the dam axis had values greater than 3, indicating a potential for excessive leakage and the need for grouting treatment. About 19.04% of these values were classified as high to very high permeability, suggesting that a considerable amount of rock masses along the dam axis are affected by interconnected open joints, which could lead to serious leakage problems. The analysis of Lugeon values with depth showed that appropriate curtain grouting to depths of about 40 m at the left abutment, 53 m at the central foundation, and 43 m at the right abutment is recommended to control potential excessive leakage. The slope stability analysis results indicated that both abutments of the dam site are unstable during saturated conditions for both structurally controlled (planar mode of failure) and non-structurally controlled (circular or near circular mode of failure) conditions. For structurally controlled conditions, the FOS was less than 1 during fully saturated slope conditions. For non-structurally controlled conditions, the FOS was as low as 0.132 during saturated conditions, indicating instability. Both abutments were stable during dry conditions for both types of failures. Rock bolting and removal of overhanging rock mass were recommended for planar mode failures, while providing proper drainage holes and reducing slope angle were recommended for slope sections with circular mode failures for those potentially unstable slopes at the Arjo Didesa dam site.

CHAPTER THREE

3. METHODS AND MATERIALS

3.1. Methods

The Methods used for this research to achieve the research objectives can be summarized as follows: desk work; data collection (including primary data collected from detailed fieldwork, discontinuity surveying, in-situ testing, geological mapping, and sampling (soils and rocks), and laboratory testing); and secondary data such as borehole drilling results, packer test results, geophysical survey data and laboratory analysis of soil and rock samples from core logging (specific gravity, water absorption, unit weight, strength test (point load index test, UCS), porosity and slake durability); data processing, analysis and interpretations. Primary and secondary data were used to evaluate, summarize, and present the findings.

3.2. Desk Work

Throughout this stage of the research, existing data from the study area, such as topographic maps (from the Ethiopian Space Science and Geospatial Information Institute), regional geology reports and maps (from the Geological Survey of Ethiopia), photographs, satellite imagery such as Digital Elevation Model (DEM), and photographs were collected and thoroughly reviewed to obtain information about the physiography, structure, and geological setup of the study area. Consequently, the study area's location and accessibility, geomorphological setting, and drainage map were created using Arc-GIS 10.8, and Surfer 21 software. Additionally, topographic maps were utilized to create a base map for geological and engineering geological mapping. Literature review of published journals and unpublished reports related to gravity dam failure modes, characterization, and classification of soils and rock masses, engineering geological evaluation of dam sites and reservoir areas like water tightness of evaluation of dam sites and reservoir area, and slope stability analysis methods were carried out to have the comprehensive understanding of engineering geological condition of the study area. Essential data for the current investigation were gathered from secondary and primary sources. Meteorological data such as precipitation(monthly precipitation data) and temperature (monthly temperature data) were gathered from the National Meteorological Agency of Ethiopia (NMAE). The primary data were generated from detailed fieldwork and laboratory investigations from the samples collected from the dam site and reservoir areas.

3.3. Geological Mapping and Data Collection

The second approach to this research work was detailed fieldwork and sample collection. Primary data were obtained from the study area, and a detailed geological and engineering geological evaluation was carried out at the dam site and reservoir areas. This step includes extensive discontinuity surveying, digging test pits and soil sampling for laboratory tests, in-situ strength tests, field descriptions of soils and rocks, and mapping. Potential modes of slope instabilities were identified during the field investigation based on the factors influencing failure mechanisms; either structurally controlled failures or non-structurally controlled failures (rock mass-controlled failures).

All field survey data and sample collections are compiled to make up the method of primary data collection referred to as Geological Mapping. This method is critical to determine if the dam site and its surroundings are suitable for the construction of dam projects in terms of stability, operational longevity, and overall structural integrity of the dam. The abovementioned approach was used to precisely determine the foundation's geological formation, abutments, and reservoir areas. Different rock types were classified based on their genetic type and degree of weathering. The lithology's thickness and the geological structure's orientation were measured in the field at different study area outcrops. To carry out extensive geologic mapping, representative traverse lines connected several lithological units and notable geological structures. Field methods were used to detect crucial slope sections from dam abutments and reservoir rims of the study areas, including identifying stress fracture development on the upper slope face, slope face tilting, and discontinuity orientation, favouring rock slope failures.

During the fieldwork, testing such as in-situ strength tests on rocks using a Schmidt hammer, field descriptions of soils and rocks, mapping and sampling for laboratory analysis, and adequate discontinuity survey parameters were carried out. Visual observation was used to determine the engineering properties of rocks and soils. The rock type, spacing, roughness, orientation, aperture, persistency, degree of weathering, infilling materials, and groundwater condition of the dam abutments were all noted on the rocky outcrops. Topographic features, surface soil deposit type, and depth were recorded in the field.

3.3.1. Discontinuity Survey

A discontinuity survey of rock outcrops provides important geological and mechanical data that helps assess rock slope stability, rock mass quality, and rock mass response to excavation and other construction activities. Characterizing and quantifying the

discontinuity survey is necessary to understand how discontinuities affect rock mass behavior including permeability, deformability, and strength. Thus, geological structure surface mapping helps explain their reaction to natural and human-induced stresses. It involves mapping and measuring fractures, joints, bedding planes, and other discontinuities within rock masses to determine their spacing, orientation (dip amount and direction), condition (length/persistence, separation/aperture, roughness, infilling materials, and weathering), groundwater conditions, and slope height and orientation.

Discontinuity surveys were primarily conducted on both the dam abutments. Potentially unstable slopes were identified during the field investigation based on the factors affecting the failure mechanism, either structurally or non-structurally controlled failures (rock mass-controlled failures). Structurally controlled and non-structurally controlled instabilities are mainly related to the discontinuity orientation of geological structures and the degree of rock mass weathering, respectively (Wyllie and Mah, 2004). Those discontinuity sets with favourable orientations for rock slope instability were given specific attention throughout the field measurement. This evaluation was primarily conducted along the excavation area. This discontinuity survey provided the critical input parameters for rock mass classification and characterization, which were then used for slope stability analysis (Hoek and Bray, 1981).

The clinometer compass was used to determine the orientations of discontinuities and the dam site's slope face. Similarly, a tape meter was used to measure the spacing of discontinuities, aperture, and persistence of discontinuity sets, and other discontinuity parameters such as surface roughness, infill material, weathering condition, and associated groundwater condition were observed and estimated visually during the fieldwork at the dam site. using International Society of Rock Mechanics (ISRM, 1978) standard method. The degree of weathering of the rock outcrops was assessed using geological hammer blows following the International Society of Rock Mechanics (ISRM, 1978).

3.3.2. The Sampling of Soils and Rocks

Soil and rock sampling provides comprehensive and critical information about a site's subsurface properties and significantly impacts the design and construction of numerous engineering projects. Because of the wide variety of soils and rocks in depth, location, and formation processes, thorough and representative sampling is required for effective classification. Sampling soils and rocks during fieldwork is critical to providing essential information on the index and engineering properties of soils and rocks. Prior to the start of construction of the dam, the Engineering Corporation of Oromia (ECO) collected and totally

analyzed twelve soil samples where (5 (five) soil samples, 6 (six) soil samples and one soil samples from right abutment of the dam site left abutment dam site, and reservoir area, respectively) to study the soil around the dam site and reservoir area.

For this study, test pits were excavated to obtain soil samples in the reservoir area (one test pit on the right bank), and one representative soil samples was collected to determine the index and engineering properties of the soil samples. The soil samples were packed in plastic bags to preserve their natural moisture content. In addition, the rocks that make up the dam and reservoir were described in terms of their engineering properties. Subsequently, three rock samples (one right abutment, one left abutment and one from the river center of the dam site) were collected from the surface area for laboratory testing. Based on visual observation techniques, the soils encountered during fieldwork in the study area can be described in terms of color.

3.3.3. In-situ Strength Determination

The uniaxial compressive strength (UCS) of representative fresh and weathered rock outcrops was tested in situ using a portable tool known as the Schmidt hammer. The Schmidt hammer is used to determine the uniaxial compressive strength of rock masses, which is then utilised as an input parameter in the calculation of shear strength parameters for joints. A Schmidt hammer was used to determine the in-situ rock strength of the dam's right and left abutments. The test was conducted per the (Aydin, 2009). ASTM (2001) recommends taking at more than ten (10) individual impact readings, excluding those that deviate from the average by more than seven (7) units, and averaging those that remain. The rebound value, often referred to as the rebound number, corresponds to the strength of the rock. The procedures followed during the utilization of the Schmidt hammer to estimate the in-situ strength rock mass discussed in below paragraph.

The first procedure of the insitu test was specimen preparation. The rock surface must be petrographically homogenous and representative of the rock mass domain to ensure accurate measurements. Smooth, clean, and dry; if not, note the degree of moistness of the surface and the specimen as wet, moist, or damp. Then testing of the insitu strength was conducted by using the hammer rebound value. While holding the hammer perpendicular to the test surface, the plunger is pressed forcefully against the rock until the impact is triggered. Multiple readings (typically 10-12) are taken at different positions and orientations across the surface to account for variability. Then, lock the Schmidt hammer by pressing the button after the impact and read the gauged rebound values from the instrument. Maintaining

constant pressure on the rock during testing is crucial to obtain consistent results as shown in the figure below (Figure 3.1).



Figure 3. 1. The type L Schmidt hammer test on the left abutment ignimbrite rock mass.

3.3.4. Soil Laboratory Test

After the test pits from the right bank of the dam reservoir areas was dug up and soil samples are collected, it was brought to the laboratory for testing and determining the index and engineering properties of the soil sample. The laboratory tests conducted on soil samples include water content, the specific gravity of solid soils, particle size distribution tests (sieve analysis), consistency of soils (liquid and plastic limit), compaction test (standard proctor test), and permeability test. The tests were conducted at the Gia Geotechnical Laboratory Pvt. Ltd. Co.

The Procedures followed for each soil laboratory test are elaborated on in the sections below.

3.3.4.1. Water Content Determination

The ASTM D 2216 (1998) standard was utilized to conduct the test, which used the oven-drying method, one of the most extensively used methods in laboratories for assessing soil water content due to its simplicity and accuracy. The process involves drying a weighed damp soil sample for 24 hours at a controlled temperature (105–110°C) (Das, 2002). First the mass of the container with lid was measured on the weight balance (M_1) as shown in Figure 3.2a. Then add the soil sample to the container and the mass of the container, lid and soil sample was weighed (M_2). Furthermore, the soil sample in the container with lid was placed in the oven and dried in an oven for 24 hours at temperatures ranging from 105 to 110°C. The dried soil sample with container is weighed (M_3).

The water content of the soil sample is calculated from the following equation:

$$w = \frac{M_w}{M_s} = \frac{M_2 - M_3}{M_3 - M_1} \times 100 \quad (29)$$

Where M_1 is the mass of the container with lid, M_2 is the mass of the container, lid and wet soil, and M_3 is the mass of the container, lid and dry soil.

3.3.4.2. Specific Gravity

According to ASTM D 854 (1998) standard, the test was conducted. By utilizing a pycnometer, this test technique involves figuring out the specific gravity of soils. In the pycnometer method, according to ASTM D 854 (1998), a clean and dry pycnometer was weighed, and its weight was recorded as M_1 . A representative soil sample was then oven-dried at a temperature of around 110°C for 24 hours to eliminate any water content. The dry soil was placed in the pycnometer, and its weight was again recorded as M_2 . The pycnometer was then carefully filled with water until it overflowed, ensuring no air bubbles were trapped in the soil sample. The weight of the pycnometer filled with soil and water was recorded as M_3 , and the weight of the pycnometer filled with water alone was measured as M_4 as shown in Figure 3.2a. The following formula can determine the specific gravity of the soil:

$$G_s = \frac{(M_2 - M_1)}{(M_4 - M_1) - (M_3 - M_4)} \quad (30)$$

Where M_1 is mass of empty pycnometer, M_2 is the pycnometer with dry soil M_3 is mass of the Pycnometer and soil and water, M_4 is the pycnometer filled with water only. G_s is Specific gravity of soils.

3.3.4.3. Grain Size Analysis

The sieve analysis was used to determine the distribution of particle sizes bigger than 75 μm (retained on the No. 200 sieve), whereas sedimentation using a hydrometer analysis determines the distribution of particle sizes lower than 75 μm .

The sieve analysis was carried out according to ASTM D 422 and the procedure involves several steps to ensure the accurate determination of soil particle size distribution. The samples are oven-dried at $110\pm 5^\circ\text{C}$ for at least 24 hours to remove all moisture content. The agglutinated/clumped/conglomerated parts of the soil samples were crushed to the correct size using a crushing device such as a pestle. The dry samples were then weighed and the notation of the initial soil mass. The next stage of the procedure involves selecting an appropriate series of sieves. A standard set of sieves specified by the American Society for Testing and Materials (ASTM) is typically employed, ranging in size from 75 μm to 75mm. The sieves are stacked in descending order, with the largest openings at the top and the smallest at the bottom, with a receiving pan at the bottom as shown in Figure 3.2b. The prepared soil sample was placed on the top sieve, and the entire stack was mounted on a sieve shaker, which mechanically agitates the sieves for a predetermined period (typically 10 to 30 minutes). The shaking process separates soil particles according to size through the sieve openings. Upon completion of the shaking process, each sieve was carefully removed, and the soil retained on the sieve was weighed. The cumulative weight retained on each sieve and the receiving pan was recorded.

3.3.4.4. Consistency Limits

The test was conducted according to ASTM D 4318 (1998) standard. To determine the liquid limit of the soil, sample the following procedures are considered.

First, a 30g sample of soil passing through a 425 micron IS sieve was mixed with water to form a paste. Then, a portion of the paste was placed in the cup of a Casagrande device, and a vertical groove was created with a standard grooving tool. The device was rotated at 2 revolutions per second while counting the number of blows required for a 12 mm closure of the groove at the bottom as shown in Figure 3.2c. A representative slice of the soil was collected with a spatula for water content determination. This procedure was repeated for different numbers of blows ranging from 15 to 35, with the consistency of the paste being adjusted accordingly. The moisture content of each sample collected was determined and

plotted on semi-log paper, with the number of blows on the X-axis (using a log scale) and the water content on the Y-axis (using an arithmetic scale). The liquid limit was then calculated by finding the water content for 25 blows from the resulting graph.

To determine the plastic limit of a soil sample, an 8g portion was taken and formed into a ball. The ball was then rolled on a glass plate with sufficient pressure to create a thread of uniform diameter. Rolling continued until the thread began to crumble, which was noted when the diameter reached 3 mm or larger as shown in Figure 3.2d. At this point, a small amount of water was added to the sample, and the kneading and rolling process resumed until the thread crumbled at 3 mm diameter. This process was repeated two or three times to obtain an average value for the plastic limit (PL). The plasticity index (PI) is the difference between liquid limit and plastic limit ($PI = LL - PL$).

3.3.4.5. Standard Proctor test

The test was carried out according to ASTM D 698 (1998) standard. In general, compaction is removing air from soil to make it denser, which takes mechanical effort. The dry unit weight of the soil is used to determine the degree of compaction. The procedure comprises applying varied compaction efforts at various moisture levels to compact a soil sample in layers inside a cylindrical mould. A typical compaction effort with a specific energy of 592kN-m/m³ compacts the soil. Engineers can establish the optimal compaction parameters by using the results of tests to construct a graph showing the relationship between dry density and moisture. The soil is compacted in a mold with a volume of 943.3 cm³ in the Standard proctor test. The mold has a diameter of 101.6 mm. The soil is combined with variable quantities of water before being crushed in three equal levels using a hammer that strikes each layer 25 times. The hammer has a drop of 304.8 mm and weighs 24.4 N (mass =2.5 kg). The wet unit weight of compaction and dry unit weight may be estimated for each test as follows:

$$\gamma = \frac{W}{V} \quad , \quad \gamma_d = \frac{\gamma}{1+w} \quad (31)$$

Where, W = weight of the compacted soil in the mold V= volume of the mold 944 cm³. The values of γ_d determined from the above equation can be then plotted against the corresponding moisture contents to obtain the maximum dry unit weight and the optimum moisture content for the soil.

3.3.4.6. Falling Head Permeability Test

The test was carried out in accordance with the ASTM D 2434 (1994) standard. The falling head method is used in this test procedure for measuring the coefficient of permeability. The soil sample was compacted in a standard mold and to begin the test, the water in the standpipe is allowed to pass through the sample and into the constant-head chamber, where it overflows and spills out. The water level in the standpipe decreases as the water percolates through the soil. To calculate the coefficient of permeability of the soils for the A is the sample's cross-sectional area, a is the standpipe's cross-sectional area, L is the specimen's length, t_f and t_i are the time interval (T) final and initial, respectively, h_1 is the initial head, and h_2 is the final head, the equation (28) was used for the course of the test.



Figure 3. 2. Laboratory Soil Sample Analysis indicating some of the procedures in the tests. a) water content determination and specific gravity, b) grain size analysis, c) liquid limit determination using the Casagrande apparatus, d) plastic limit determination.

3.3.5. Rock Laboratory Test

The rock samples collected during the field investigation are brought to the laboratory for test and analyzed accordingly. Laboratory tests on rock samples are the determination of the unit weight, point load index test, and water absorption test. The tests were conducted at the Gia Engineering Pvt. Ltd. Co.

The procedures followed to test the rock in the laboratory are described in the following section.

3.3.5.1. Unit Weight Test

The unit weight test is a common laboratory test used to determine a rock sample's density for slope stability study and strength evaluation. The ISRM (1978) standard lists the basic test processes. Firstly, the sample must be free of visible fractures or other structural flaws that might compromise the accuracy of the test findings. The sample must be typical of the rock formation under consideration. The next stage in the technique was to determine the mass of the rock sample after it has been oven-dried for 24 hours at 105° C and recorded as (Md). After the mass of the rock samples were determined, the method moved on to determine its volume and saturated weight. The next procedure was filling a container (typically a graduated cylinder) with a predetermined amount of water, a rock sample was immersed in the water to saturate it with water and recording the saturated weight (Ms). The subsequent rise in water level was measured, which corresponds to the rock sample volume (V_1 and V_2 in ml). The density of the rock samples (ρ) was computed using the formula: $\rho = \text{mass/volume}$. The result was then stated in quantities such as Kg/m^3 . The final stage in determining the unit weight test was computing by multiplying the value of the dry and saturated density of the samples and the acceleration due to gravity to determine its unit weight (γ).

$$\gamma = \rho \times g \quad (32)$$

Where γ is the unit weight of the rock sample (KN/m^3), ρ is the density of the sample (g/cm^3), and g is the acceleration due to gravity (m/s^2).

3.3.5.2. Point Load Index Test

The test was carried out in accordance with ASTM D 5731 (1995) specifications to measure the strength of irregular rock samples collected from the field investigations, and it was also carried out on an irregular rock sample with natural moisture content. The procedure of the point load index test is described as follows (as shown in figure 3.3a below).

The PLI test technique begins with collecting a representative rock from the fieldwork, often in the shape of an irregular lump. It is crucial to ensure that the specimen is free of visible weaknesses such as bedding planes or fractures. Irregular rock samples were broken down into an appropriate shape and size as the scale of the point load testing apparatus using a geological hammer and chisel. Before beginning the test, the irregular rock samples' height (H), length (W1), and width (W2) were measured, and it is placed into the Point Load Testing Machine between two conical loading platens. Conical platens apply a force to the specimen perpendicular to the loading axis until failure occurs, resulting in a pair of fragments. Upon the occurrence of this failure, the failure load (P) was recorded, whereas the equivalent core diameter (D) was obtained from sample dimension during the test. Then, uncorrected point load index (Is) was determined from failure load (P). The data is then used to calculate the Point Load Strength Index ($Is_{(50)}$) and consequently, the Uniaxial compressive strength (UCS) of the rock samples from the equation (12).

3.3.5.3. Water Absorption Test

The test was conducted according to the standard by American Society for Testing and Materials (ASTM D6473-10, 2010). In the test, the weight of the basket in air is measured and noted as W_1 . A 2-kilogram sample of irregularly broken rock is properly cleaned to eliminate fines, then drained, put in a wire basket, and submerged in distilled water for at least 5 cm above the top of the basket at a temperature between 22 and 32 °C as shown in figure (3.3b) below. After immersion, the sample's trapped air is evacuated by raising the basket holding it 25 mm above the tank's bottom, and letting it fall at a rate of roughly one drop per second. For the following 24 hours, the basket and rock sample must be entirely submerged in water.

A temperature range of 22° to 32°C is used to weigh the basket and the sample while they are submerged in water. It is noted that the weight when suspended in water = W_2 . The rock samples are transferred to the dry absorbent fabrics once the basket and rock samples have been removed from the water and have allowed enough time to drain. The empty basket is then jolted 25 times in the water tank before being weighed there = W_3 . The rock samples

applied to the absorbent clothing are surface dried until the cloth is unable to absorb any more moisture. The rock samples are then moved to a second dry cloth and spread out in a single layer, where they are left to dry for at least 10 minutes or until the surface is totally dry. After that, the aggregate is weighted = **W₄**. The irregularly broken rock sample is placed in a shallow pan and kept in a 110° C-maintained oven for 24 hours. Once it has finished cooling in an airtight container, the oven is turned off = **W₅**.

Where weights of saturated and basket irregularly broken rock suspended in water are **W₂** and **W₃**, respectively. The weights of saturated surface dry irregularly broken rock suspended in air and the weight of oven-dry aggregate are **W₄** and **W₅**, respectively. Weight of the saturated aggregate in water = **W₂ – W₃**, Volume of the aggregate in water = **W₄ - (W₂ – W₃)**. The Water absorption, bulk specific gravity, bulk specific gravity (SSD), and apparent specific gravity are determined by using the following equation:

$$\text{Water Absorption} = \frac{W_3 - W_1 - W_5}{W_5} \times 100 \quad (33)$$

$$\text{Bulk Specific Gravity(SSD)} = \frac{W_3 - W_1}{(W_3 - W_1) - (W_4 - W_2)} \quad (34)$$

$$\text{Bulk Specific Gravity} = \frac{W_5}{(W_3 - W_1) - (W_4 - W_2)} \quad (35)$$

$$\text{Apparent Specific Gravity} = \frac{W_5}{W_5 - (W_4 - W_2)} \quad (36)$$



Figure 3. 3. Rock sample analysis in the laboratory. a) point load index test, and b) water absorption determination.

3.4. Secondary Data Collection

Secondary data from the dam site's subsurface investigation, including geophysical investigations, borehole logging and core sampling performed during the geotechnical drilling process, permeability data (packer test results and falling head permeability test), and laboratory investigation results such as specific gravity, water absorption, unit weight, strength test (point load index test), porosity and slake durability, were collected from the Engineering Corporation of Oromia (ECO). As a result, three boreholes of various depths were drilled for direct exploration of the subsurface condition using section logging and permeability testing. The location and depth of boreholes are shown in Table 3.1. Consequently, core logging, RQD, and geological data for each borehole were extracted from the relevant project's borehole geological log data. These boreholes' falling head permeability and water pressure tests were also utilized to investigate the dam site's water-

tightness condition. The packer test was used for generally competent rock formations, whereas the falling head permeability test was employed for highly weathered and fractured rock formations and soils. In addition to the subsurface investigation, the secondary data of surface investigation of the reservoir area including the natural moisture content, specific gravity, grain size analysis, activity of soils, atterberg limits, and permeability test.

Table 3.1. Location and depth of drilled boreholes at the dam site.

Borehole Site	Location	BH-ID	UTM coordinates (Adindan Zone 37N)		Depth(m)
			Easting	Northing	
Dam Axis	Left Abutment	KABH-01	511156	852312	66
	Right Abutment	KABH-03	511271	852423	53.30
Conduit	Intake Conduit (Left Side)	KABH-02	511198	852343	15
Total Borehole Depth					134.30

3.5. Rock Mass Classification Methods

Although there are other ways to classify surface rock outcrops in the study area, this research will employ the rock quality designation (RQD) from the volumetric joint count method, rock mass rating (RMR), and geological strength index (GSI) approach to characterize the rocks at the dam site and reservoir area. In order to determine the surface rock exposure quality in terms of RQD, the indirect method volumetric joint count method was used (Palmström, 1982; Bieniawski, 1989). In determining the RQD of rock outcrops using the volumetric joint count method (J_v), the number of joints as joint sets by counting and joint spacings were determined using a ruler and tape meter. Then the J_v values were calculated using equation (16), and the RQD values were estimated using equation (15).

After determining the RQD values of the rock mass, the following rock mass classification is the rock mass rating (RMR). During the field investigations, the Schmidt hammer rebound values were used to determine the uniaxial compressive strength of rock mass. The point load index test on rock samples was conducted to determine the uniaxial compressive strength of intact rocks in the laboratory. In the case of utilizing the UCS values of the rocks for rock mass classifications, the UCS values obtained from the point load index test were used as the input parameter for the rock mass rating (RMR) classification system since the UCS values estimated from the Schmidt hammer rebound values can be compromised by the influences of the discontinuity conditions beneath the tested surface of the rock masses. The discontinuity survey data such as spacing, orientation (dip amount and direction),

condition of discontinuities (length/persistence, separation/aperture, roughness, infilling materials, and weathering), and groundwater condition that were collected during the fieldwork were used as the input parameters for rock mass classification of rock outcrops at the dam site (Bieniawski, 1989).

Consequently, the data such as RQD, discontinuity survey data, and UCS from the point load index test are then changed into the rated parameters. The RMR basic was by adding up the rated input parameters.

According to Hoek and Marinos (2000), visual observation and assessment of the surface rock quality can be used for the characterization of rock masses in terms of geological strength index (GSI) using the GSI basic chart. GSI can also be determined by using the equation (21). Further, GSI value can be used in determining the shear strength parameters in the RocData software.

3.6. Determination of Shear Strength and Deformation Modulus Parameters

Depending on the rock materials' strength and the discontinuities' features, shear strength parameters are the most significant factors that influence the stability of the rock slope. RocData software was used to evaluate the shear strength and modulus of deformation of dam site rock masses using Generalized Hoek-Brown Failure criteria (Hoek et al., 2002). This software employs GSI, material constant of intact rock (m_i), UCS of intact rock, and disturbance of factor (D) to determine the geo-mechanical properties of the rock mass. As a result, in this case, GSI was estimated from field observation, and the UCS of intact rock was calculated using the point load index as input data. Similarly, the material constant of intact rock (m_i) was taken from the RocData software's standard table. The disturbance factor value (D) is determined by the degree of disturbance in the rock mass caused by the excavation method. Its values range from 0 to 1 for undisturbed in situ rock masses and 1 for extremely disturbed rock masses, as built into the RocData software. In this study, the disturbance factor used is 0.7.

RocData software (Rocscience, 2004b) was also used to calculate the rock joints' shear strength parameters (cohesion and friction angle) along the discontinuity failure plane for structurally controlled slope sections, based upon the empirically derived correlation described by (Barton, 1973) which has already been integrated into RocData software given in the equation below.

$$\tau = \sigma_n \tan \left[JRC \log_{10} \left(\frac{JCS}{\sigma_n} \right) + \phi_r \right] \quad (37)$$

Where JRC is the joint roughness coefficient, JCS is the joint wall compressive strength, and τ is the peak shear strength (in MPa), σ_n is the effective normal stress (in MPa), and ϕ_r is residual frictional angle.

According to the shear strength criteria for rock joints (Barton and Bandis, 1990), the basic input parameters of the RocData software (Rocscience, 2004b) include the residual friction angle (ϕ_r), the joint roughness coefficient profile (JRC) profile, the joint wall compressive strength (JCS), the slope height, and the unit weight of the rock. The uniaxial compressive strength (UCS), determined from the Schmidt rebound value (R) on an un-weathered rock surface with unit weight (γ), represents the joint wall compressive strength of the rock (JCS) (Barton and Bandis, 1990). According to Sharma et al. (1999), the joint wall compressive strength becomes a fraction of the uniaxial rock compressive strength of approximately equal to UCS, 0.75UCS, 0.5UCS and 0.25UCS for unweathered, slightly weathered, moderately, and highly weathered joint wall of rocks. In this case, JCS was determined considering the rock joint wall condition of highly weathered joint walls and moderately weathered joint walls with value of 0.25UCS and 0.5UCS, respectively. The JRC value was determined visually during fieldwork by comparing the appearance of the discontinuity surface to the standard joint roughness profile developed by (Barton and Choubey, 1977). The base friction angle value (ϕ_b) for ignimbrite ranges from 32.50 for most smooth and un-weathered rock surfaces (Barton and Choubey, 1977).

The residual friction angle (ϕ_r) was calculated using Equation 38, as suggested by (Barton and Choubey, 1977). The in-situ Schmidt hammer test was used to determine the input parameters for this equation, which included dry rebound (R) and wet rebound (r). Ulusay and Karakul (2016) proposed a fundamental friction angle (ϕ_b) of 32.50 for ignimbrite rock, which is exposed in all slope portions.

$$\phi_r = (\phi_b - 20^\circ) + 20(r/R) \quad (38)$$

3.7. Determination of Bearing Capacity of Rock Mass

In this research work, around four empirical equations were used to determine the bearing capacity of the rock masses at the dam site. In case of the determining the bearing capacity of rock mass using empirical equations (equation 3) forwarded by Carter and Kulhawy (1992) were calculated using the input parameters like shear strength parameters (cohesion

and frictional angle and Hoek-Brown failure criteria constants (m_b , S , and a). Using the parameters such as Hoek-Brown constant (m_b), UCS and C_f1 with value of 1, were used in the equation (4) forwarded by Wyllie (1992). Similarly, for the equation (equation 5) suggested by Bell (2007) the bearing capacity factors (N_o , N_{cq} and $N_{\gamma q}$) which were determined using the chart shown in Figure 2.1 (which in turn depends on the slope angle and height, width of foundation, unit weight of rock, and cohesion), and correction factors (C_f1 and C_f2) with value of 1. From the borehole at the dam site, 7 geotechnical/lithological units were identified at the left abutment borehole KABH-01 with the variation in rock type, UCS value, and RQD value or degree of fracturing and weathering was identified. At the right abutment borehole KABH-03, 9 geotechnical rock units with variation in rock type, UCS value, and RQD value or degree of fracturing and weathering was identified. At the intake conduit borehole KABH-02, 2 (two) geotechnical units which are the welded tuff and ignimbrite rock unit. Therefore, the bearing capacity of rock mass at the left and right abutment was calculated as seven and nine rock units, respectively. Similarly, the bearing capacity of rock mass was also calculated for 2 geotechnical units were also calculated at the intake conduit borehole. The geotechnical rock unit identified at the dam site from the drilled boreholes varies with depth in terms of strength and degree of weathering and fracturing. Using the equation (equation 1 and 2) given by Bowles (1996), The ultimate and allowable bearing capacity of these geotechnical units were calculated by using the input parameters of average RQD and UCS of each rock unit and the bearing capacity factors are determined using charts as given in Appendix 14.

3.8. Water tightness Assessment Methods

The soil sample from the reservoir area was subjected to laboratory permeability testing. This was followed by packer tests from the in-situ test. 13 (thirteen) representative soil samples 6 (six) from left abutment, 5 (five) from right abutment, and two from the reservoir area were collected from the test pits were subjected to a laboratory permeability test to ascertain the soil deposits' permeability of coefficient by ECO and the researcher of this study. ECDSWC (2022) also conducted packer or water pressure tests in the three boreholes at the dam axis and intake conduit to evaluate dam site water tightness using packer test and falling head permeability tests of stronger rock formations and weak formation throughout the boreholes, respectively.

3.9. Slope Stability Analysis Methods

Kinematic analysis, and limit equilibrium method were used to analyze the stability of the rock slope for this study. The factor of safety and deformation of the slope sections were established for both dry and saturated situations following the collection of essential data from fieldwork, laboratory, and correlation. When the situation is dry, stability analysis is carried out for this condition under static and dynamic circumstances, considering that the slope section is entirely dry, and that water has no impact on the instability of the slope. The slope was considered saturated, and when water forces formed within the slope, the material determines the safety factor in wet conditions. This water significantly impacts the condition of the slope's instability.

3.9.1. Kinematic Analysis Method

In this study, kinematic analysis of the abutment slopes was carried out for rock slope sections with distinctive discontinuity orientation and strength where possible failure modes were assessed during the fieldwork. Regarding analyzing the rock slope of the abutments in terms of mode of failures, Dips 6.0 software (Rocscience, 2004a) was utilized by inputting the discontinuity data like the orientation of discontinuity and slope faces, and the friction angle, which was measured and collected from the field.

3.9.2. Limit Equilibrium Method

Since the slope stability of the dam site was structurally controlled, after the mode of failures of the slope section are identified and the limit equilibrium method using the deterministic method was used to determine the stability of slope in terms of factor of safety by correlating the force leading the rock mass to fail along the failure surface (driving force) and the force resisting the rock mass to fail (resisting force) (Wyllie and Norrish, 1996; Aprilia and Indrawan, 2014). The modes of rock slope failures are usually identified as discussed earlier kinematic analysis (Wyllie and Norrish, 1996).

In case of determining the factor of safety for planar mode of failure, RocPlane 2.0 software (Rocscience, 2004c) was utilized by inputting the slope orientation, upper slope orientation, failure plane orientation, bench width, slope height, unit weight of rock, waviness, shear strength parameters of joints (friction angle and cohesion), and forces (water pressure and seismic coefficient). Similarly, for wedge mode of failure, Swedge 4.0 software (Rocscience, 2004d) was used by providing the input parameters like slope angle, potential failure plane

orientation, slope height, bench width, unit weight of rock, and shear strength parameters (cohesion and friction angle) the factor of safety of slope section with wedge mode of failure was computed.

3.10. Materials Used

For purposes of this research work, the materials used and exploited are described as the following.

The Topographic map, inclinometer compass, GPS, geological hammer, sample bag, tape meter and notebook with pens, pencil, coloured pencils and markers were used for geological modelling and discontinuity surveying during the field work. In addition, mobile phone camera with 48M Quad camera was used to take field photographs and rock outcrops with an enhanced resolution. The type L Schmidt hammer apparatus and point load index (PLI) machine were used to determine the uniaxial compressive strength (UCS) of rock mass in the field and intact rock samples in the laboratory. Unit weight determination and water absorption apparatus were used in the laboratory analysis of rocks. The water content determination, specific gravity, grain size analysis (sieve analysis and hydrometer analysis), Atterberg limits (consistency limits), standard compaction test, and falling head permeability test apparatus were used in the analysis of the soil samples in the laboratory.

To produce location map and drainage map, and 3D physiographic map of the study area, the ArcGIS 10.8 software and Surfer 21 software were used respectively. The Strater 5 software was used to prepare the lithological log from the drilled borehole data at the dam site. The RocData 2.0 software was used to determine the shear strength parameters of rock mass and joints. For the slope stability analysis of the study area, Dip 6 software, RocPlane 2.0 software, and Swedge 4.0 software were used.

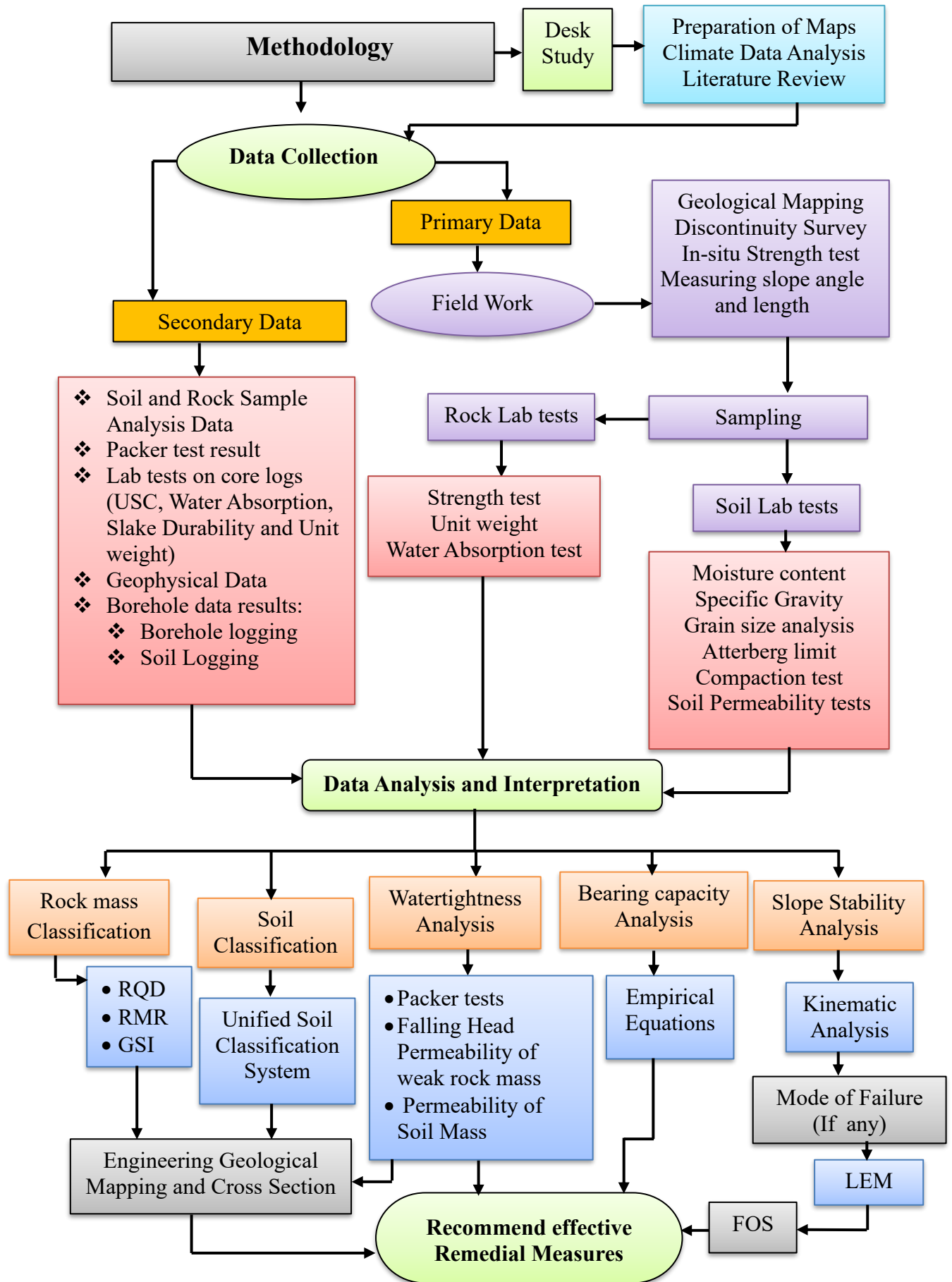


Figure 3. 4. Flowchart of the Research Methodology.

CHAPTER FOUR

4. GEOLOGY OF THE STUDY AREA

4.1. Introduction

Ethiopian geology comprises the Precambrian basement rocks to recent volcanic rocks (S. Tadesse et al., 2003). These rocks are categorized into the following geological formations: Precambrian rocks, Paleozoic-Mesozoic sedimentary rocks, and Cenozoic volcanic rocks and associated sediments (Tadesse and Allen, 2005). Geologic map of Ethiopia was first compiled by (Kazmin et al., 1978) at scale of 1:2,000,000, and was modified later by (Chernet, 1992) provide some broad and basic information about the geology of Ethiopia. Later on, the regional geological map of the Asela area was compiled by (Workineh et al., 2014).

Geology of Asela area is mainly plateau volcanics belonging to the southeastern Ethiopia and associated lowlands. Minor areas occurring to the northwestern part belong to the Main Ethiopia Rift (MER). Later young Quaternary lava flows (mainly basalts) covered localized areas of the rift. The physiographic region of the MER possesses down-faulted plateau volcanics, especially the upper pyroclastic. Linear micro-horsts and micro-grabens form subparallel ridges and troughs. The horsts are formed by pyroclasts, and the grabens are at places filled with Quaternary alluvial sediments and lacustrine sediments. The area is mostly covered by volcanic mountains of younger Plio-Pleistocene trachyte flows, scoria cones and Quaternary basalt flows. Quaternary volcanics and sediments occur in the MER to the NW part (Workineh et al., 2014). The study area is found within the Main Ethiopia Rift (MER) and its evolution will be briefly discussed in the next section.

4.1.1. Regional Geology

The regional geology of the Asela area is mainly plateau volcanics found within the Main Ethiopia Rift (MER). Regional geology of the area readily falls in to two major categories. These are Cenozoic Tertiary Volcanics successions (palocene and Goba trachyte, palocene, highland and plateau basalt, middle and lower flows, and upper pyroclasts) to Quaternary volcanics and sediments (alluvial deposit, scoria and basalt). And Mesozoic Jurassic (upper and lower limestone) to cretaceous sedimentary successions (upper sandstone). The Tertiary and Quaternary Volcanics comprise of Ginner Basalt, Chillalo Volcanics, Nazeret groups, Alkali Trachyte and Basalt flow, Alkali Trachyte flow and Arsi-Bale basalts from young to old stratigraphical orientation. The Volcanics crops out throughout the map area except the north-eastern part of the study area which is covered by sedimentary units. The Volcanics

are thought to be Oligocene to Holocene in age and unconformable over the sedimentary units. The sedimentary units in the Northeastern part of the map area are Amba Aradam formation, Urandab formation and Hamanle formations successively overlain from young to old. The sedimentary formations overlay the basement unit that is not outcropped in the area.

The upper Paleozoic, Pangaea, fragmentation, led to the Mesozoic marine invasion, on the down-warped Horn of Africa. The Mesozoic Sea deposited, mainly coastal or shelf facies, the lower sandstone (the Adigrat Formation), is followed by marine fossiliferous Jurassic limestone. The sea transgressed from the SE to NW through time, as a diachronous surface, overlapping the lower sandstone. These limestones are known as the Hamanlei Formation, first deposited in SE Ethiopia, later reached the central Ethiopia. In the central and northern Ethiopia, the limestones are known as the Antalo limestone (Mohr, 1963; Merla et al., 1979, and Colby et al., 2022).

Late in the Cretaceous, uplift in the Horn caused the Mesozoic Sea regression, depositing the Upper Sandstone (The Ambaradom Formation). Continued uplift in the Eocene, alongside with tensional plate motion, resulted in the linear fractures and / or faults, through which various lava flows outpoured (Workineh et al., 2014).

The volcanics of central Ethiopia are not only flood basalts, but also include, thick succession of pyroclastic flows and falls, mainly capping the major known flood basalts. There are also pyroclastic flows or pyroclastic falls within the major episodic flood basaltic volcanism. Also occur paleosoils and wood fossils, at various stratigraphic levels, indicating hiatus in volcanism (Workineh et al., 2014). Therefore, volcanism in central Ethiopia, didn't commence and ended within short period of time as some investigators concluded. The earlier age data, probably lack systematic sampling following stratigraphy or lack of fresh samples leading to erroneous results.

The earlier classification of volcanic successions was modified later into formational names. Accordingly in the gorges of Wabe Shebele, the lower most unit in the map area, the Hamanlei Formation, is exposed. This is followed upward by shaly, Urandab Formation and by the Antalo limestone. The sedimentary units were ended by Cretaceous Upper sandstone (Ambaradom Formation).

The Mesozoic sedimentary rocks are unconformably overlain by the lower lava flows correlable to the Ashanghi Formation (Mengesha and Haro, 1996). Upward in the section comes the middle lava flows, correlable to the Aiba Formation of northern Ethiopia. Thick succession of pyroclastics is lacking in the southeastern plateau. The Aiba basalts are

unconformably overlain by plateau basalts and pyroclastics correlable to the Alajae basalts and rhyolites of northern Ethiopia (Workineh et al., 2014). These later ones cover large parts of Arsi and Bale plateau. The rhyolite in this case is related to the silicic upper pyroclasts. These pyroclasts are previously known as the Nazareth series. They were wrongly interpreted, as if they were formed by rift tectonics. The series don't match with MER formation but erupted in much older time (Kazmin et al., 1978). The rift floor was also previously mapped as Dino Formation. This mapping unit was erroneously separated from the series. They are now grouped as the upper pyroclasts, down-faulted by the MER tectonics (Workineh et al., 2014).

The silicic upper pyroclasts were overlapped by central type series of basalt eruption. The central type of basaltic volcanism is correlable with the Tarmaber Formation, covering most of the highlands of Bale and Arsi Zone. This formation was locally ended with trachyte flows. These basalts are previously known as Arsi and Bale basalts (Merla et al., 1979 and Mengesha and Haro, 1996). The whole volcanic terrain to the east of Asela, except the trachytic peaks, were mapped as Ashanghi Group (Kazmin, 1973). This mixes different formations and lumps them together.

After the formation of the Main Ethiopian Rift, which down-faulted the upper pyroclasts and early formed lava flows, localized younger central type eruption formed the highlands of Arsi and Bale areas (Workineh et al., 2014). These first erupted basalt lava, followed by trachytes. These are previously mapped as Chilalo Formation (Mengesha and Haro, 1996). Mountain peaks such as Chilalo, Galema, Kaka, Berole, and the top part of Bale mountains were mapped as Chilalo Formation. Most of the bottom part of these peaks first erupted basalt flows of central type, on older Tarmaber Formation. The Chilalo Formation is Pliocene in age (Workineh et al., 2014). The Bale mountains within the map area covered by highland basalt, is pierced with multiple trachytic dike swarms. These dike swarms seem to be the root source of the trachyte there.

On the plateau of Bale, Quaternary basaltic fissure eruptions occur. These are previously known as Ginir Formation. Locally both the plateau of Arsi and Bale, are pierced with scoria fall (scoria cones). Quaternary basalts also occur within the floor of the MER. These later ones have origin with tectonics of the Wonji Fault Belt (Mohr 1967 in Kazmin et al., 1978). On the Bale Mountain National Park occur, collapsed caldera formed during Quaternary (Workineh et al., 2014). This also emitted younger Quaternary basalts.

4.1.2. Regional Structural Setting

Regional structural setting of the study area has two categories which is the primary structures and secondary structures. Primary structures observed are bedding and laminations encountered in the Mesozoic sedimentary rocks, the upper limestone and upper sandstone, in northeastern part of the study area.

The Mesozoic sediments are horizontal and don't show tilting. Generally, the Mesozoic sedimentary rocks are horizontally bedded, continuous, planar beds. The bedding ranges from thinly laminated horizons to very thickly bedded. Laminations occur mostly on lime-mudstones interfingering between limestones (Workineh et al., 2014). The limestone is medium to very thickly bedded.

Most of the volcanic rocks are massive but some of the lower lava flow and plateau basalts show columnar structures. In addition to these local columnar structures are also observed in the middle lava flow and the upper pyroclasts (ignimbrite). Mostly, the columnar joint structures show vertical fractures in multi-direction. The upper pyroclastic rocks (ignimbrites) around Asela area show weakly to moderately columnar structures.

Secondary structures encountered include mainly faults, fractures, shear zones and joints which are mostly related to extensional tectonics of the MER. In addition to these there are also dykes swarms in the Highland of Bale Mountain. Minor localized deformation structures like micro fold, and deformed fiammaes on some of the ignimbrites in the upper pyroclasts also occur on localized places. All these structures are observed in the Tertiary volcanic rocks of the highland and plateau of Arsi and Bale areas and Wabe Shebele valleys (Workineh et al., 2014). Columnar structures are characteristics of the lower lava flows found in the Wabe gorges and in other highlands.

Even though, the study area mainly belongs to the southeast plateau; the northwest of the area belongs to the main Ethiopian rift (MER). This contains down-faulted plateau volcanics of upper pyroclasts. Here the plateau volcanic, mainly ignimbrites, with ash fallouts, tuffs, ash-flow tuffs pyroclastic fallouts, volcanic breccias, belonging to the Nazareth series have been affected by steep normal fault facing to the northwest. This occurred during the Miocene rifting producing the Main Ethiopian Rift. The rift pinches out and gently inclined to the NW of Etaya.

The normal faults strike to NE and have vertical to subvertical dip. In the region of the MER, there are micro-horsts and micro-graben trending also to the northeast. The micro grabens have been filled with younger Quaternary sediments and Quaternary basalts while the micro-horsts contain the upper pyroclasts.

The main striking lineaments of the area are shown by numerous systems of faults. They are characterized by three distinct set of linear features trending dominantly NE but also have NW and NS direction. In addition to this several subordinate jointing and fracturing are common cutting the different rock type. Mostly the faults are normal faults but have minor sinistral strike slip faults around southern part of the map sheet. The normal fault, direction of displacement is to NW with amount of displacement varies from 15m to 130m. Some of the major faults of the eastern escarpment of the MER show displacement up to 300 m. Many step faults from highland plateau to MER axis also form back tilting.

Most faults in the MER area show northeasterly trend. This fault mainly cuts and down throw the pyroclastic rocks. At the center of the rift there is also rift-in-rift structure which forms horst and graben. This is known as Wonji Fault Belt (WFB) forming axial graben. This rift-in-rift faults also show NE strike similar to MER with vertically displacement. Following the deep-seated reactivated fault younger Holocene Lake such as Hora are surrounded with Quaternary basalt flows. The basalt flow is related to the reactivated WFB (Workineh et al., 2014).

Voluminous magmatic outflows of central type eruption and lava pile load at places created deformation on some volcanic rocks especially, the pyroclasts. Deformation of ignimbrite is observed due to magma outflow creating uplift. This produced antiforms and synforms on the ignimbrite and locally monoclonal sloping surfaces (Workineh et al., 2014). In the Asela area well deformed fiammae and vesicles in the ignimbrite show lenticular shape due to this phenomenon. Southeast of Asela gentle uplift due to internal movement of the crust caused local deformation producing local sinusoidal minor synform and antiform showing linear basins and hills.

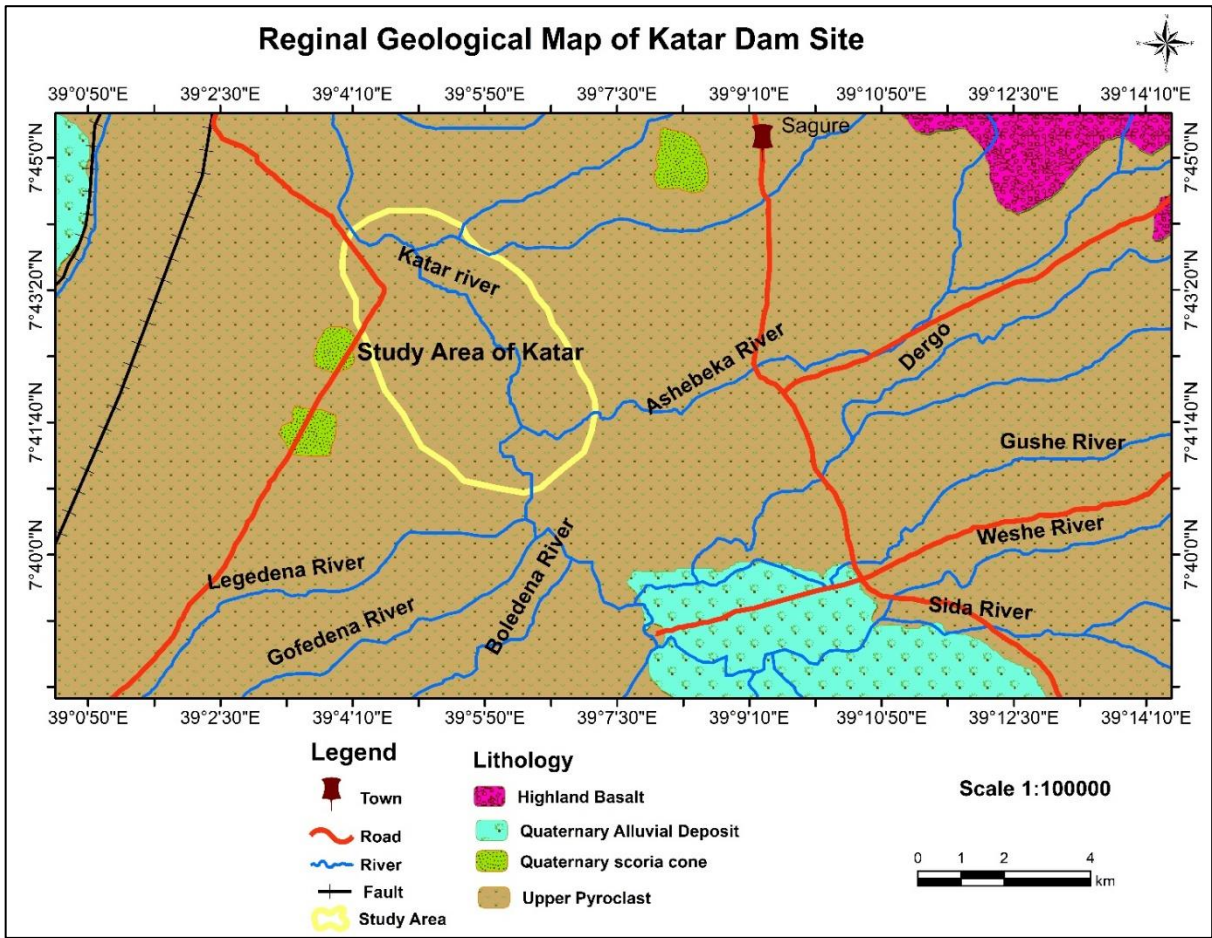


Figure 4. 1. The Regional Geological and Structural Setting of the Study Area (Workineh et al., 2014).

4.1.3. Local Geology

The dam site and the surrounding area covered by different geological units. These are upper pyroclasts, exposed in the majority part of the project area and surrounding and alluvial/residual sediments observed from few centimeters to two-meter average depth in the right and left abutment and in very vast area of the dam site. The above geology units are described clearly as follows.

4.1.3.1. Ignimbrites

The upper pyroclasts cover vast plateau of Asela to Sagure and Melka Wakena areas. It is whitish in color and moderates to highly weathered in the degree of weathering. These units include ignimbrites and different form of tuff. The site major geologic units which composed the dam site are described below.

The ignimbrite is usually found below the tuff, ash-fall or Ash-flow deposits. Exposures of Ignimbrite flow layers were appearing and covers vast portion of the dam area including the right and left abutments, abutment slopes, reservoir, flat/gentle slopes, riverbeds/banks of

Katar River and intake area forming flat to gently sloping topography and are commonly found overlain by thick volcanic tuff sequences. it formed relatively thick sub- horizontally layered sheets of hardened rock units.

Particular to the dam site, exposure this rock units are limited and are exposed due to the removal of top volcanic tuff layers and occupies hill sides, stream/river cut sections and depression areas as sheets of strongly welded and jointed rocks.

Majorly, these rock units are hexagonally fractured (columnar jointing) and frequently affected by penetrative, persistent & tight sub vertical systematic tectonic joint sets. Tension cracks manifested by hexagonal columnar jointing are probably resulted as a result of instantaneous cessation of the flow that caused local compression and extension. Both the primary cooling jointing and the secondary tectonic jointing favored quarrying activities for dimension stone production.

Megascopically, these rocks are well welded/hardened pyroclastic flow rocks made up of crystal (glassy crystals) and rock fragments (medium to coarse sized lithic fragments of rocks of various composition) within dark grey to light green glass sheared groundmass. The ignimbrites mapped in the dam area exhibit welding, giving the ignimbrite the texture of a solid rock mass. Based on field rough estimation the lithic fragments and are irregularly shaped, poorly sorted and of various composition. Compositionally, these rocks are felsic.



Figure 4. 2. Ignimbrite at the katar dam axis.

4.1.3.2. Tuff Unit

Unconsolidated to relatively consolidated massive voluminous pyroclastic deposits cover riverbanks of Katar River and their tributaries majorly overlain by unconsolidated loose quaternary alluvial/residual deposits. This unit is relatively dense, hard, relatively consolidated to unconsolidated, massive, light green to light yellowish colored pyroclastic deposit composed of some rock fragments (at places minor glassy crystals) within fine grained ash groundmass. It is believed that compaction of volcanic ash ejected/deposited from a vent during a volcanic eruption developed more compacted/consolidated solid rock and is believed to be fall out origin rather than flows (that resulted in welded ignimbrites). The temperature of erupted mass was sufficiently hot at the time of deposition so that welding of rock fragments and fine groundmass occurred to produce relatively more solid rocks.

Megascopically, this unit is relatively consolidated to unconsolidated pyroclastic deposit made up of medium to coarse size lithic fragments within voluminous dark green to light yellowish fine groundmass (volcanic ash). The proportion of lithic fragments versus fine groundmass varies laterally/vertically and at places can be rich by lithic fragments. Moreover, the size, shape and composition of lithic fragments and glassy crystals also varies laterally and vertically. Similar to the other formations, irregularly networked erosional surfaces due to selective dissolutions are common. Moreover, to the other units in the area, this unit is also jointed by two sets of systematic, penetrative and tight sub-vertical joints that parallels and transverse the rift system.



Figure 4. 3. Welded tuff of the study area.

4.1.3.3. Alluvial Deposit

Besides to locally exposed organic soil, there are two types of soil exposure observed in the study area. These are soil transported by water (Alluvial) and soil originally formed from weathered bed rock of the area (residual).

Alluvial soil is typically made up of a variety of materials, including fine particles of silt clay, clay silty and larger particles of sand and gravel and alluvial deposit is also formed after these transported material accumulated and lithified at a particular area. Cotton black color transported soil and remained in the riverbed. However, the river passed through a narrow and steep slope valley therefor most of the alluvial soil mostly not remain at dam site.

The Quaternary alluvial deposits also occur on local river valley of Katar and Ashebeka junction, in their tributaries and some grabens formed by rift within the main rift around the dam site area. Alluvial soil is typically made up of a variety of materials, including fine particles of silty clay, clayey silt and coarser particles of sand and gravel. It is dark brown in color, moderately plastic, fine grain size clayey silt to round sandy gravel ignimbrite and basaltic rock composition.



Figure 4. 4. Alluvial soil at the study area.

4.1.3.4. Residual Deposit

Residual soil developed by the weathering of bedrock and some organic material without much lateral movement. This type of soil covers much part of the dam site and reservoir from the left and right abutment of the river valley with thickness ranging from few

centimeters up to one and half meter. Hence, This type of soil unmappable likes to the alluvial soil exposure due to its depth and coverage incompatibility to the required scale.

4.1.4. Geological Structure of Study Area

Structures are common causes of engineering geological problems of reservoir and dam site as they highly influence rock masses properties which contribute to weakening and increasing rock mass strength and permeability. Therefore, construction of the dam and formation of associated water reservoirs require cautious identification and evaluation of geological structures and their incorporation into the design. Accordingly, an attempt was made in this study to differentiate various geological structures and evaluate their influence on the proposed dam. The study area is part of the main Ethiopian rift (MER). Hence, rift-related geological structures were observed in Katar dam site which are characterized mostly by fractures, faults, and joints.

4.1.4.1. Joints

Joints are the dominant geological structures in the study area, which is observed mostly in slightly weathered to moderately weathered pyroclastic rock. There are different joint sets oriented on both abutment side of the dam site. Discontinuity conditions, orientation, spacing, aperture, persistence, type of infill material, and degree of wall weathering were determined in the field.



Figure 4. 5. Joints and their representative description of the dam site (joint sets at the right abutment).

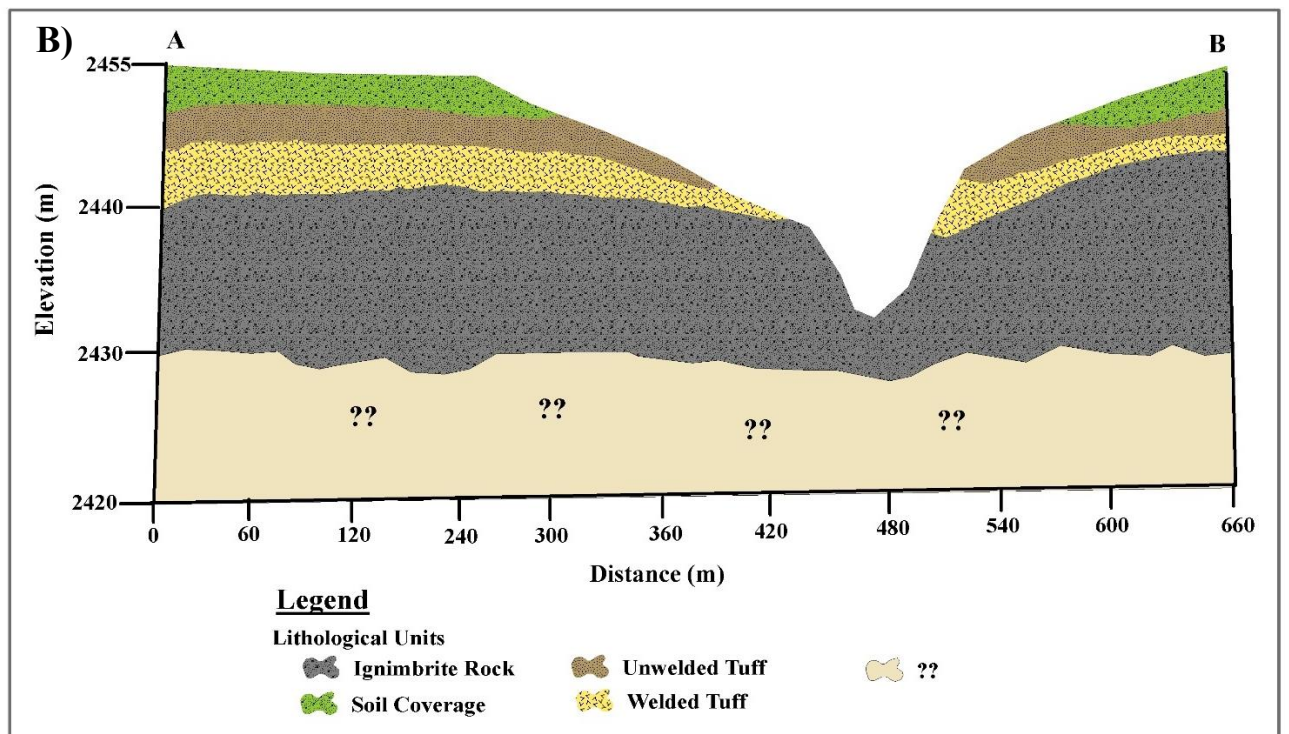
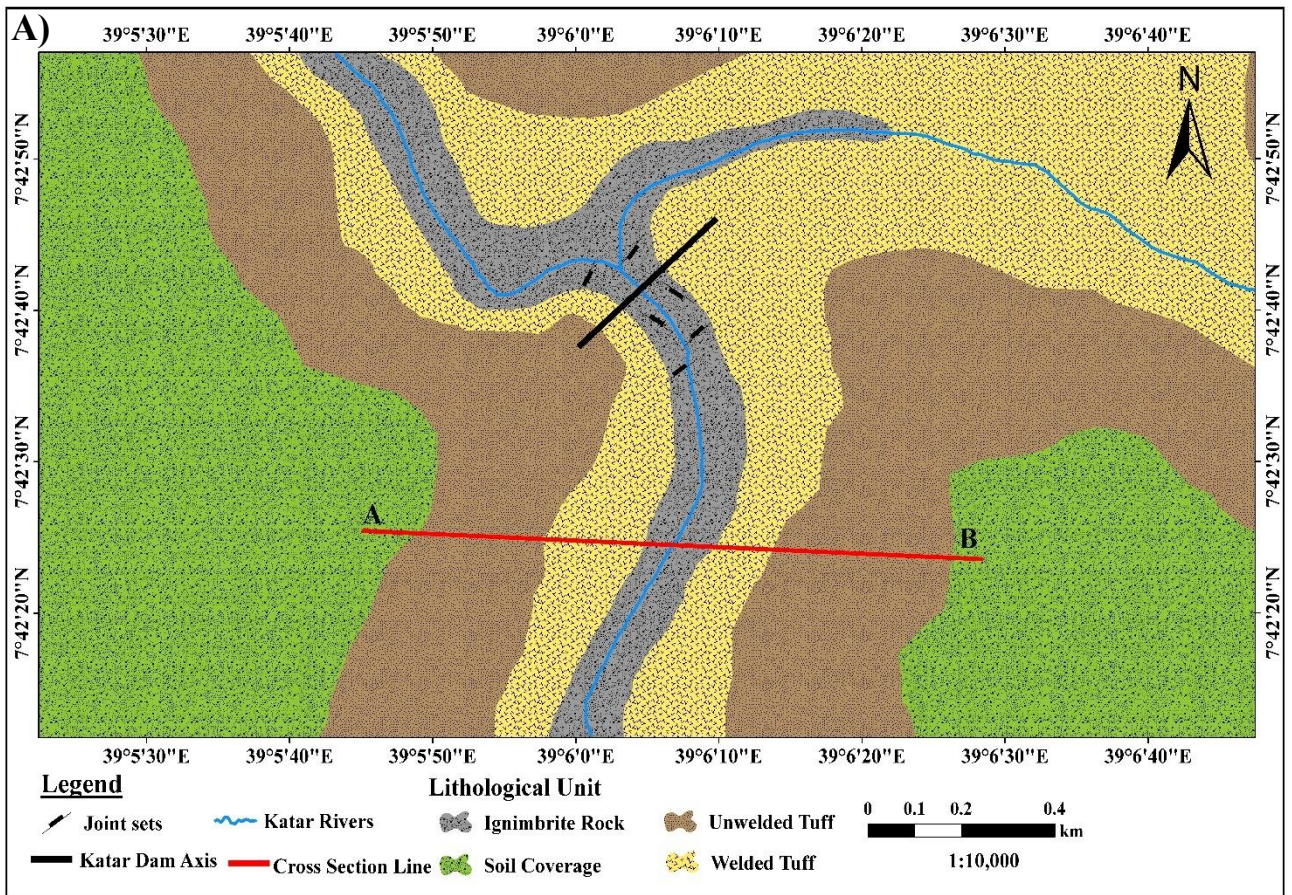
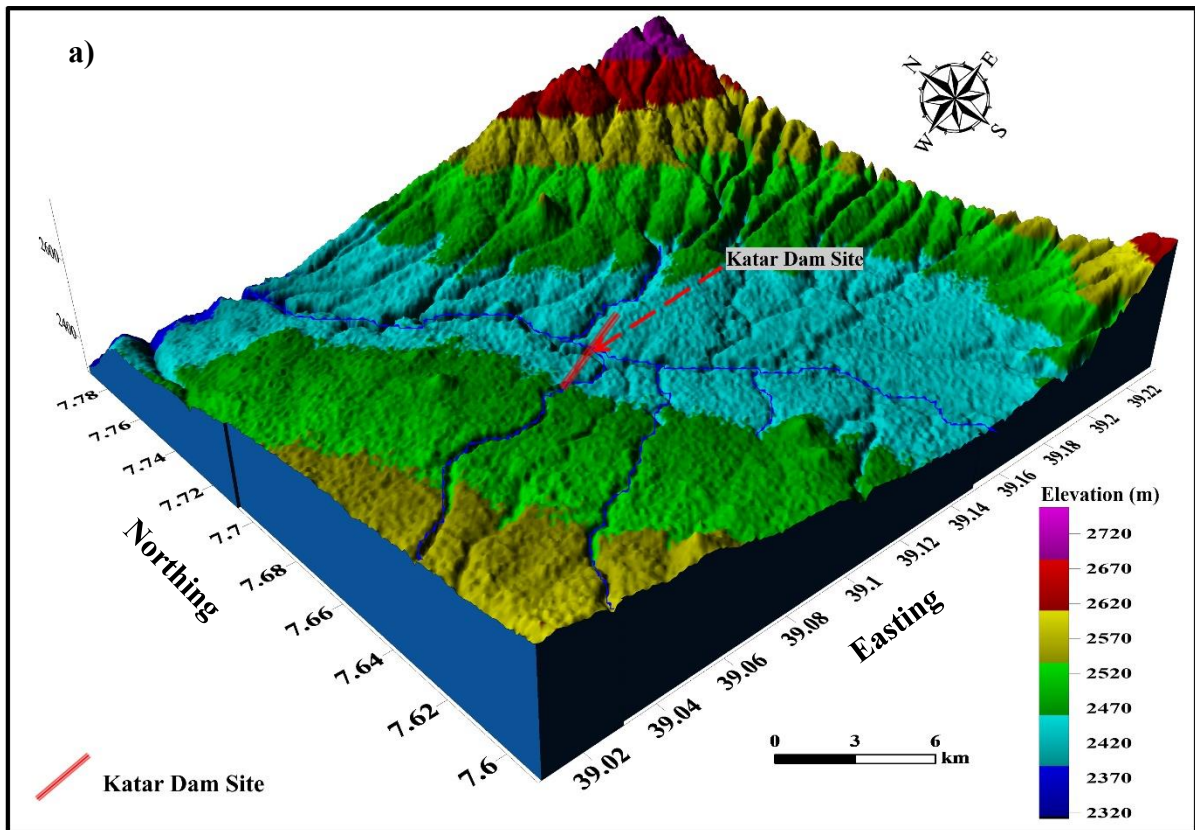


Figure 4. 6. A) Geological map of katar dam site at the scale of 1:10,000 and B) Geological cross section of the geology map of the study area.

4.2. Physiographic and Drainage Characteristics

In Arsi Zone the plateau surface begins from rift escarpments and extends to Sagure and surrounding areas. These plateau surfaces are built of pyroclastic rocks and to a lesser extent by basalts. All of the river flow to the main katar river from the eastern and southern corners of the highland of the area (Figure 4.7). The command side generally situated within the low laying portions of this meandering stream extension. The topographic variation ranges from 3250 to 2350m.a.s.l. and generally defines residual plain for the command area. Topography of the proposed dam axis is relatively gentle to steep, narrower and moderately strong ignimbrite and locally basalt rock exposer. Both ta he left and right banks of the streams have the stable and gentle to steep slope. The study area has a dense drainage network featuring parallel, sub-parallel, and dendritic drainage patterns.



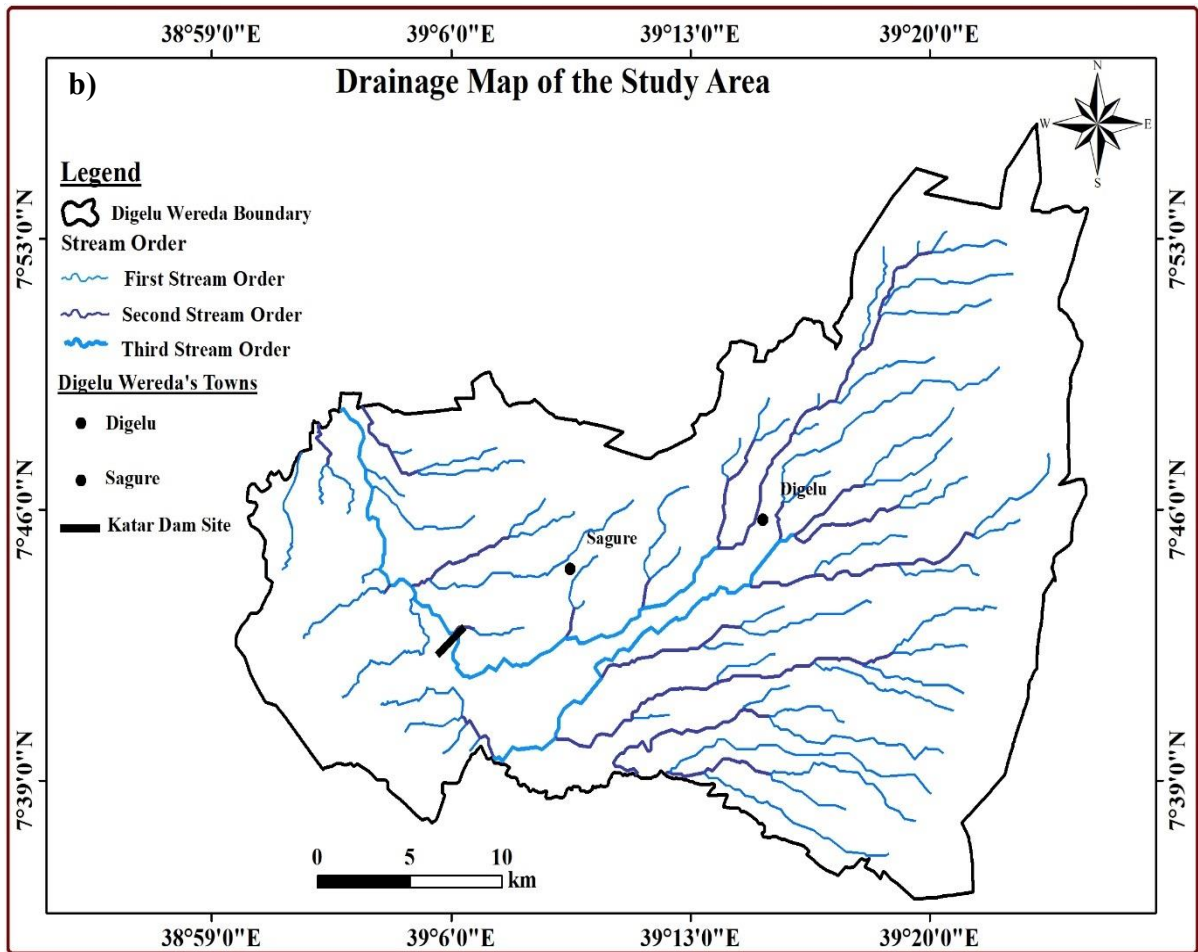


Figure 4. 7. a) 3D Physiographic map; (b) Drainage map of the study area.

4.3. Seismicity of the Study Area

Ethiopia covers a region which is prone to a significant level of seismic hazard due to the presence of the East African rift system. The rift valley has famed for its seismic and volcanic activity (Corti, 2009). Small earthquakes are commonly felt and record in tectonically active regions like the Main Ethiopian Rift Valley. Different sized earthquakes can occur throughout the rift and its surroundings. There is ground fissuring throughout the rift.

A number of destructive earthquakes, some causing loss to life, have been reported during this century. Realizing this major threat in the region, which covers an area of approximately 5.5 million square kilometers, the region's capacity in earthquake preparedness and hazard mitigation needs to be improved significantly. The prerequisite in any hazard mitigation program is to investigate the earthquake hazard potential in the region through an assessment of seismic hazard using state-of-the-art techniques. Concern of earthquakes over their

occurrence at the dam project relates to, The stability of the dam foundation, The stability of the reservoir sides/rims and the possibility of seepage in the reservoir.

The location of Katar Dam site is at the eastern margin of the Main Ethiopian Rift (MER), which is characterized by active tectonics. The active geodynamic nature of the MER makes the dam site vulnerable and even susceptible to seismic (earthquake) and volcanic hazards. According to the general seismic hazard map of Ethiopia (ES EN 19981:2015), the Katar Dam area lies in 0.085-0.1 m/s^2 of seismicity (Figure 4.8), Areas that fall in this Zone are the most prone to seismic hazards. This zone comprises all areas in the Main Ethiopia Rift and Afar Depression.

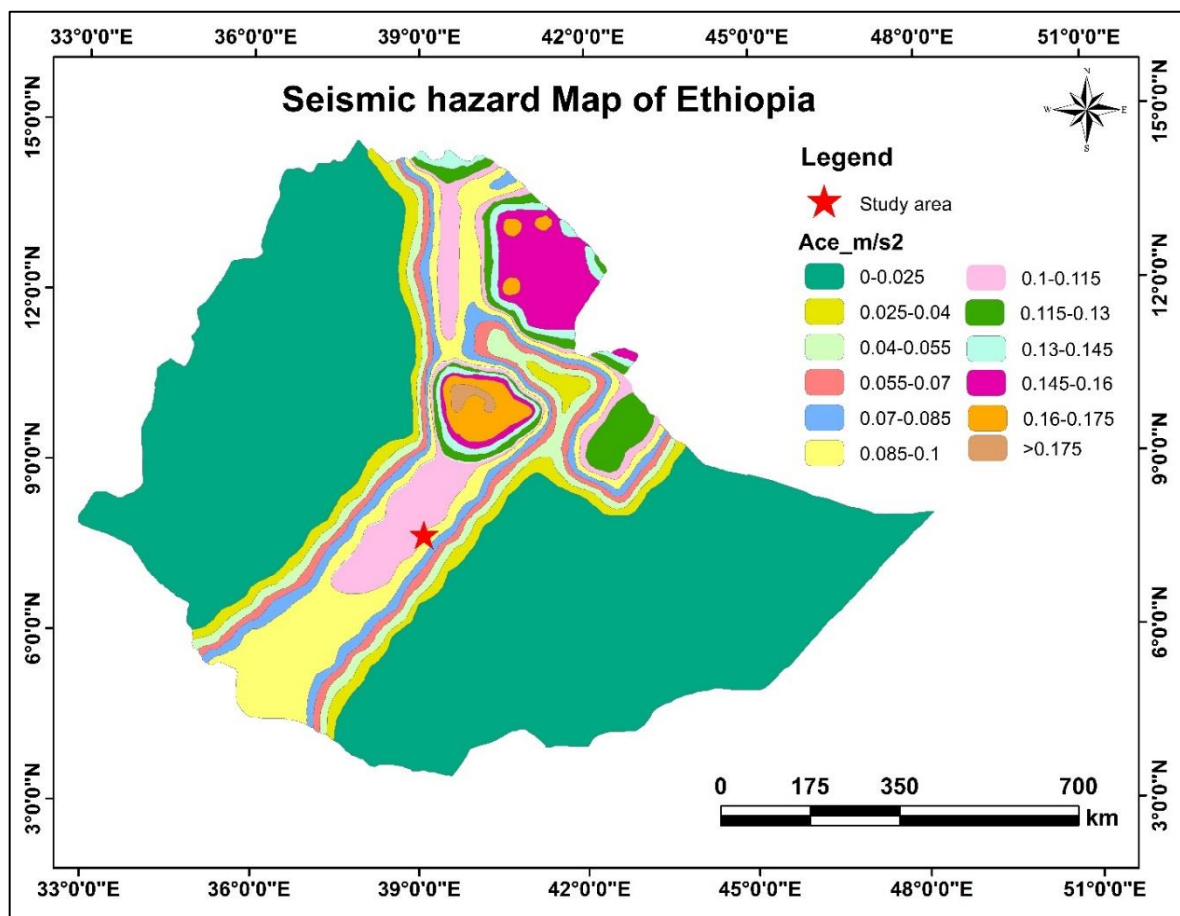


Figure 4. 8. Ethiopia's Seismic hazard map in terms of peak ground acceleration (ES EN 19981:2015).

CHAPTER FIVE

5. RESULTS AND DISCUSSIONS

5.1. Introduction

The primary goal of dam site investigations is to characterize the engineering geology of the soils and rocks in the chosen research area. This entails a rigorous evaluation of geological materials, including their physical and mechanical properties, as well as an assessment of terrain characteristics to establish the site's appropriateness for dam construction. This approach helps to anticipate and manage potential geological and geotechnical difficulties that may affect the dam's safety, functionality, and cost-effectiveness. The study employs engineering geological evaluation methodologies to examine the geological features of rocks and soils at the dam site, as well as a geological investigation of the larger study region during fieldwork. Prior to assessing the engineering qualities of soils and rocks, fieldwork was required to determine regional and local geology, as well as geological structural conditions. The dam site was thoroughly investigated using field mapping, drilling, sampling, and laboratory testing. The obtained data was analyzed to classify soils and rocks, understand their stratigraphic and structural properties, and assess material durability, compatibility, and the possibility of slope instability.

An engineering geological evaluation was conducted at the Katar dam site, using a comprehensive methodology that included geological, geophysical, and geotechnical approaches. The goal was to determine the impact of soils and rocks on the dam construction process. The stability of engineering constructions is largely dependent on the engineering properties of rocks, which include strength, bearing capacity, permeability, and deformability. This chapter summarizes the outcomes of the research study, which included a mix of secondary data, field and laboratory activities designed to acquire critical data for the assessment.

5.2. Geological Description of the Dam Site and Reservoir Areas

Based on field observations, the study area is topographically characterized by steep slope ridges that form the dam site's abutments, a V-shaped valley, and highly elevated ground in both the upstream and downstream directions. Tuff and ignimbrite rock, two types of pyroclastic rock, make up the dam's abutments. Tuff and leftover soil deposits cover the

exposed ignimbrite rock at both abutments. Rock masses that were jointed, fractured, and moderately to highly weathered made up the ignimbrite rock that was exposed at the abutment. Additionally, the rock mass exhibits a great deal of weathering and fracturing, as evidenced by the numerous tiny to large openings that are visible. Additionally, the reservoir area is mostly covered in residual and alluvial soils, with pyroclastic rocks also present in some areas. The geology of the foundation and both abutment sides of the dam is primarily composed of pyroclastic rock, specifically ignimbrite and tuff, and is primarily covered by thin layers of residual and alluvial soil deposit, according to a thorough geological study of the dam site.

5.3. Engineering Geological Classification and Characterization of Rock Mass

The engineering geological evaluation of rock masses plays a crucial role in ensuring the stability and safety of structures, particularly at dam sites. At the heart of this assessment is the comprehensive classification and characterization of rock masses, both on the surface and subsurface levels. Surface engineering geological characterization involves a meticulous process that includes rock mass classification and a detailed examination of engineering properties. Utilizing rock mass classification systems and conducting discontinuity survey analyses contribute to a deeper understanding of the rock masses. The engineering properties extracted from field investigations and laboratory analyses of representative rock samples are integral in shaping the overall evaluation. The subsurface characterization of rock masses is equally vital and involves the use of secondary data. Geophysical investigation results, geotechnical drillings, and packer tests conducted at the dam site provide valuable insights into the subsurface conditions. This multifaceted approach ensures a comprehensive understanding of the geological composition, allowing engineering geologists to make adequate decisions about dam design, construction, and long-term stability. In essence, engineering geological evaluation becomes the cornerstone for successful dam projects, safeguarding against potential risks and ensuring the durability of these critical infrastructure elements.

5.3. Engineering Geological Classification and Characterization of Surface Rock Mass

5.3.1. Discontinuity Characterization and Orientation

The discontinuity analysis was conducted in accordance with the criteria set by the International Society of Rock Mechanics (ISRM, 1978) to assess the rock mass conditions of the dam abutments. Discontinuity data, including joint spacing, orientation, conditions (apertures, infill material, and roughness), and the number of joints, were gathered and analyzed from visible rock formations on the slope sides of both abutments during the fieldwork. The discontinuity data is utilized for evaluating water tightness, as fundamental input parameters for characterizing rock masses, assessing foundation bearing capacity, and analyzing slope stability. Moreover, both left abutment and right abutment majority of joint sets are characterized by closely spaced, vertical to sub-vertical and planar oriented, slightly rough to rough surfaces, tight to open with fine in filled materials were also observed and slicken sided surface were also observed. Table 5.1 provides a concise overview of some representative metrics of discontinuity.

Table 5. 1. Summary of Representative Discontinuity Characteristics.

Location	Discontinuity Characteristics						
	JS	Spacing (cm)	Persistence (m)	Aperture (mm)	Roughness	Infilling material	Degree of Weathering
Left Abutment	JS1	16-40	0.15- 0.38	0.5	Slightly Rough	None	Moderately Weathered
	JS2	9-50	0.1-0.45	0.1	Slightly Rough	None	Slightly Weathered
	JS3	19-97	0.35 – 1.5	0.25	Smooth	None	Slightly Weathered
Right Abutment	JS1	11- 38	0.19- 0.35	0.36	Slightly Rough	None	Slightly Weathered
	JS2	8-45	0.2-0.85	0.1	Slightly Rough	None	Slightly Weathered
	JS3	10-76	0.31-1.4	0.2	Smooth	None	Slightly Weathered
JS1, JS2, and JS3 are joint set 1, joint set 2 and joint set 3 respectively and JS is joint set.							

The investigation of the discontinuities showed that the right abutment had three main sets of joints with strike directions of NW-SE (JS1), NE-SW (JS2), and NNW-SSE (JS3). On the other hand, the left abutment had joints striking in the directions of NW-SE (JS1), NE-

SW (JS2), and NNW-SSE (JS3). Based on these findings, it can be inferred that JS2 for both right and left abutment rock outcrops of the dam site have orientations that are highly conducive to leaking through the discontinuities due to their orientations of strike direction are nearly parallel to the direction of the river flow at the right and left abutments, respectively. This situation is also linked to the possibility of water seepage from the reservoir. Table 5.1 demonstrates that joint set 2 (JS2) and joint set 2 (JS2) at the right and left abutment lack infilling, which may lead to water leakage due to their integration with the joint orientation. These joint sets can also impact the overall structure and operating lifespan of the dam. Hence, it is imperative to implement specific measures at both abutments during the construction phase to mitigate the risk of potential leakage.

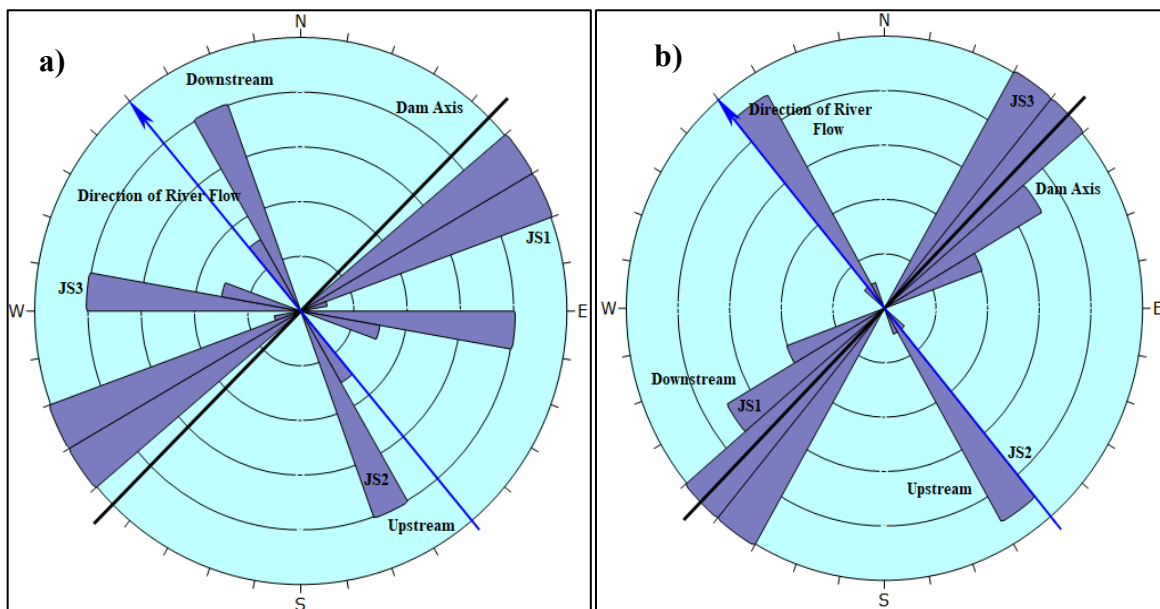


Figure 5. 1. The Rosette Diagram of the discontinuities with respect to the Dam Axis and River Flow Direction. a) Right Abutment, b) Left Abutment.

5.3.2. Rock Sample Analysis Findings

Three rock samples were subjected to laboratory analysis, including the point load index, unit weight, and water absorption tests, in order to analyze the characteristics of surface rock masses.

A unit weight test was performed on rock samples taken from the left and right abutment as well as the reservoir sections. The test was conducted according to the ISRM (1978) standard, which recommended the use of buoyancy techniques. The unit weight of the rock samples can serve as an input parameter in analyzing the stability of rock slopes and in

estimating the unconfined compressive strength (UCS) value based on the Schmidt rebound value. The result showed that the unit weights are 15.36 kN/m³, 15.18 kN/m³ and 19.77 kN/m³ on the right abutment, riverbed, and left abutment, respectively. The results of unit weight were used as input parameters in the rock slope stability analysis.

Table 5. 2. Result of unit weights of rocks determined in the laboratory.

Location	Sample code	ρ_{dry} (gm/cm ³)	ρ_{sat} (gm/cm ³)	$\gamma = \rho_{sat}$ × g (kN/m ³)	$\gamma = \rho_{dry}$ × g (kN/m ³)	Rock unit
Right Abutment	RA-1	1.566	1.586	15.55	15.36	Highly Weathered Ignimbrite
Riverbed	RB-1	1.548	1.555	15.25	15.18	Moderately Weathered Ignimbrite
Left Abutment	LA-1	2.015	2.036	19.97	19.77	Moderately Weathered Ignimbrite
Where M_{dry} is oven dried mass of rock, ρ_{dry} is dry density, γ is Bulk unit weight of rock						

In addition to the determination of UCS of rocks using Schmidt hammer test, the intact rock strength was also estimated through the point load strength test on representative samples in the laboratory. The test was conducted according to ASTM D 5731 (1995) standards on irregular rock samples to determine the uniaxial compressive strength of intact rocks. The rocks' uniaxial compressive strength (UCS) values obtained are 8.97 MPa, 21.52 MPa, and 22.87 MPa for the right abutment, riverbed, and left abutment correspondingly. This information may be found in Table 5.3. In general, the findings of the point load strength test on the rock sample indicate that the ignimbrite rock exhibits a significantly low strength.

Table 5. 3. Summary of the UCS determination of rock samples.

Location	Sample code	$I_{s(50)}$ MPa	UCS (MPa)	Classification according to ISRM 1978	Rock Unit
Right abutment	RA-1	0.43	8.97	Very low strength rock	Highly Weathered Ignimbrite
Riverbed	RB-1	1.03	21.52	Very low strength rock	Moderately Weathered Ignimbrite
Left abutment	LA-1	1.10	22.87	Very low strength rock	Moderately Weathered Ignimbrite
RA-1 is sample taken from right abutment, RB-1 is sample taken from riverbed, LA-1 sample taken from left abutment.					

According to the International Society for Rock Mechanics (ISRM) in 1978, strong rock is characterized by a water absorption rate ranging from 1% to 2%. Medium strength rock, on the other hand, has a water absorption rate between 2% and 3%. Weak rock is defined as having a water absorption rate over 3%. The water absorption test was performed on rock samples in order to ascertain the quantity of water absorbed by the rock samples. The test results indicate that the ignimbrite rock sample taken from the riverbed (RB-1) has a water absorption value of 14.56%, indicating a weak rock mass. Similarly, the ignimbrite rock samples from the right abutment and left abutment have water absorption values of 24.69% and 15.84% respectively, suggesting that these rocks are also weak. These findings are summarized in Table 5.4.

Table 5. 4. Summary of the water absorption value of rock sample analysis.

Location	Sample Code	Water Absorption (%)	Rock Description (ISRM, 1978)	Rock unit
Right Abutment	RA-1	24.69	Weak	Highly Weathered Ignimbrite
Riverbed	RB-1	14.56	Weak	Moderately Weathered Ignimbrite
Left Abutment	LA-1	15.84	Weak	Moderately Weathered Ignimbrite
RA-1 is sample taken from right abutment, RB-1 is sample taken from riverbed, LA-1 sample taken from left abutment.				

In addition to this, Engineering Corporation of Oromia (ECO, 2023) have also conducted the laboratory analysis of determining the unit weight, specific gravity, water absorption, soundness by using sodium sulphate, and unconfined compressive strength (UCS) of representative ignimbrite rock samples from both left and right abutment at the dam axis.

Hence, the unit weight and specific gravity of the rock sample was 21.2 kN/m³ and 2.44 and 22.4 kN/m³ and 2.52 for ignimbrite rock sample from right and left abutment of the dam axis, respectively. The water absorption value of the rock samples was 7.11% and 6.81% for the ignimbrite rock sample from the right and left abutment, respectively indicating weak rock mass considering the water absorption value of the rock samples was greater than 3 according to ISRM, 1978). The soundness value of the rock samples was 9.22 and 8.44 for the ignimbrite rock sample from the right and left abutment, respectively indicating weak rock mass considering the soundness value of the rock samples was very low according to

ISRM, 1978). In addition to determining the durability and soundness of rock samples, the UCS value of each rock sample was also determined. The UCS value of the rock samples were 20.4MPa and 29.79MPa for the ignimbrite rock sample from the right abutment and left abutment indicating the very low strength rock mass (further see Table 5.5).

Table 5. 5. Summarized results of the ignimbrite rock sample from right and left abutment of the dam axis.

Rock Sample ID	Unit weight (kN/m³)	Specific Gravity	Water Absorption (%)	Soundness by (Sodium Sulphate)	UCS (Mpa)
KADR-1	21.2	2.44	7.11	9.22	20.4
KADL-1	22.4	2.52	6.81	8.44	29.76
KADR-1 is the ignimbrite rock sample from the right abutment, and KADL-1 is the ignimbrite rock sample from the left abutment					

5.3.3. Insitu Strength Test Results

Furthermore, the assessment of the geological characteristics in the designated area involved a thorough analysis of the uniaxial compressive strength. This crucial parameter was determined through the meticulous utilization of Schmidt hammer rebound values, coupled with the concurrent consideration of the dry unit weight of meticulously collected rock samples within the laboratory setting.

In the present study, the in-situ strength of the rocks has been estimated using L-type Schmidt hammer following the standard suggested by ASTM (2001) to determine the strength of intact rock adjacent to joint walls at the dam abutments. Accordingly, the test was conducted on the fresh and slightly weathered rocks and for each observation point at least eleven Schmidt rebound values were collected (Table 5.6). The obtained rebound values were first averaging and then discarding those differing from the average by more than 7 units. Then the remaining rebound values were converted into uniaxial compressive strength (UCS) of the rocks using equation 13. Table 5.6 below presents some representative UCS of rocks of the study area determined from the Schmidt hammer test. The UCS values of the right and left abutment rock outcrop range from 24.15MPa to 33.88MPa. According to Deere and Miller (1966), The UCS values of both abutments imply a rock mass with a very low to low strength.

Table 5. 6. Schmidt Rebound value and the determined UCS values.

Location	Schmidt Rebound Value	Average RN		UCS (MPa)	According to Deere and Miller (1966)
RAS	28, 27, 31, 34, 29, 31, 27, 29, 31, 32, 27	29.63	27.63	24.15	Very Low Strength Rock
	25, 31, 30, 26, 19, 24, 26, 27, 25, 25,26	25.81			
	24, 30, 26, 24, 28, 23, 30, 24, 25, 27, 29	26.36			
	28, 31, 30, 26, 29, 32, 33, 27, 25, 35,20	28.72			
LAS	29, 31, 32, 29, 28, 28, 29, 34, 33, 29, 35	30.64	29.77	33.88	Low Strength Rock
	27, 29, 27, 32, 33, 28, 34, 28, 30, 29, 32	29.91			
	25, 29, 27, 23, 28, 30, 28, 24, 26, 29, 26	26.82			
	29, 30, 28, 36, 34, 32, 29, 33, 28, 36, 34	31.72			

5.3.4. Rock Quality Designation

Rock Quality Designation was estimated from joint volumetric count (JV) for rock mass that are exposed at the dam abutment using equation 15 as suggested by Palmstrom (2005). The Volumetric Joint count (Jv) is an average measurement for the rock mass volume expressed as the number of joints occurring in one m³ of volume. In order to have good representative value for RQD determination through Jv, it is important that as many measurements are taken. Accordingly, in this study, Jv measurement was taken at 12 different discontinuity stations along abutments of the dam axis. The measurements were taken from ignimbrite unit. Interpretation of determined Jv values was made based on Palmstrom (2005). Consequently, rocks with a Jv value less than 1 exhibit a very minimal level of jointing, while those with a Jv value between 1.0 and 3.0 have a low level of jointing. Rocks with a Jv value between 3.0 and 10.0 have a moderate level of jointing, and those with a Jv value between 10.0 and 30.0 exhibit a high level of jointing. 30-60 refers to a significant level of jointing, whereas anything greater than 60 indicates complete crushing. Table 5.7 presents a summary of some representative discontinuity measurements.

Table 5. 7. RQD is estimated from Surface Discontinuity Frequency.

Location	Observation Section	JV	RQD (%)	Rock Mass Quality According to Deere and Deere (1988)
Right Abutment	RAS	14	69	Fair
		12	75	Good
		11	79	Good
		22	42	Poor
		24	36	Poor
		23	39	Poor
Left Abutment	LAS	13	72	Fair
		16	62	Fair
		9	85	Good
		18	56	Fair
		23	39	Poor
		15	66	Fair

According to the analysis, the average number of joints in the study region varies from 11 to 24 on the right abutment and from 9 to 23 on the left abutment of the dam site. The typical RQD values range from 36% to 79% at the right abutment and from 39% to 85% at the left abutment.

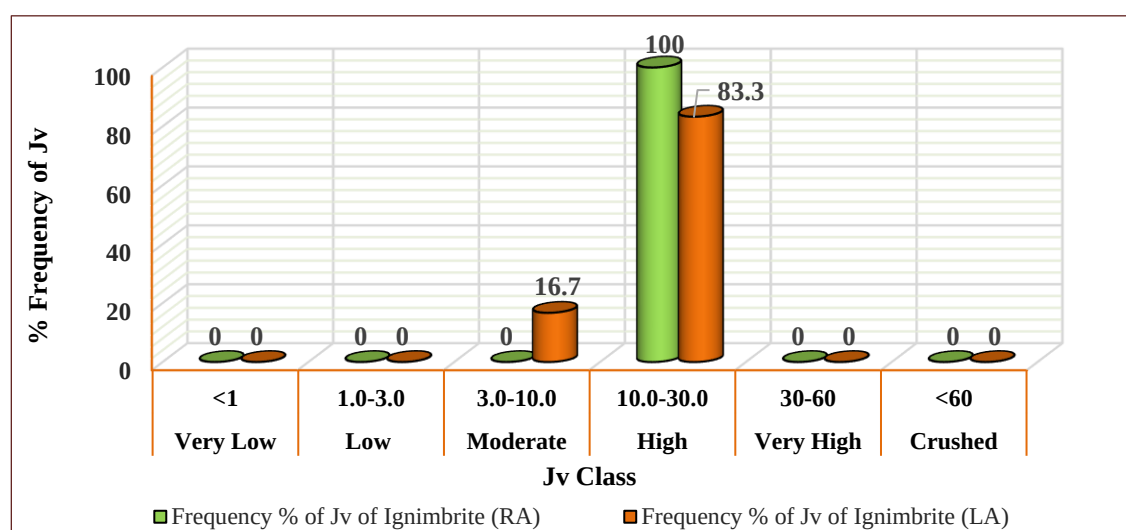


Figure 5. 2. The percentage frequency of Jv of rocks of the study area in each Jv class as per Palmstrom (2005).

The statistical distribution of the determined Jv value of the right abutment rock outcrop (RA) rocks in the study area indicates that all Jv values fit under the category of high degree of jointing. 83.3% of the joints at left abutment rock outcrops (LA) exhibit a high degree of jointing, whereas 16.7% show a moderate degree of jointing (Figure 5.2.).

According to Deere (1968), the statistical distribution of the RQD class of surface rocks at the right abutment rock outcrop (RA) is as follows: 50% fall into the poor class, 16.67% fall into the fair class, and 33.33% fall into the category of good rocks, as illustrated in Figure 5.3. Similarly, at the left abutment rock outcrop (LA), 16.67% of the data falls into the poor class, 66.67% falls into the fair class, and 16.66% falls into the good class. According to Deere (1968), the rocks exposed at both abutment slopes can be classified as having varying quality, ranging from poor to good. These findings suggest that the rock mass at both abutments is extensively worn and impacted by multiple joints and fractures.

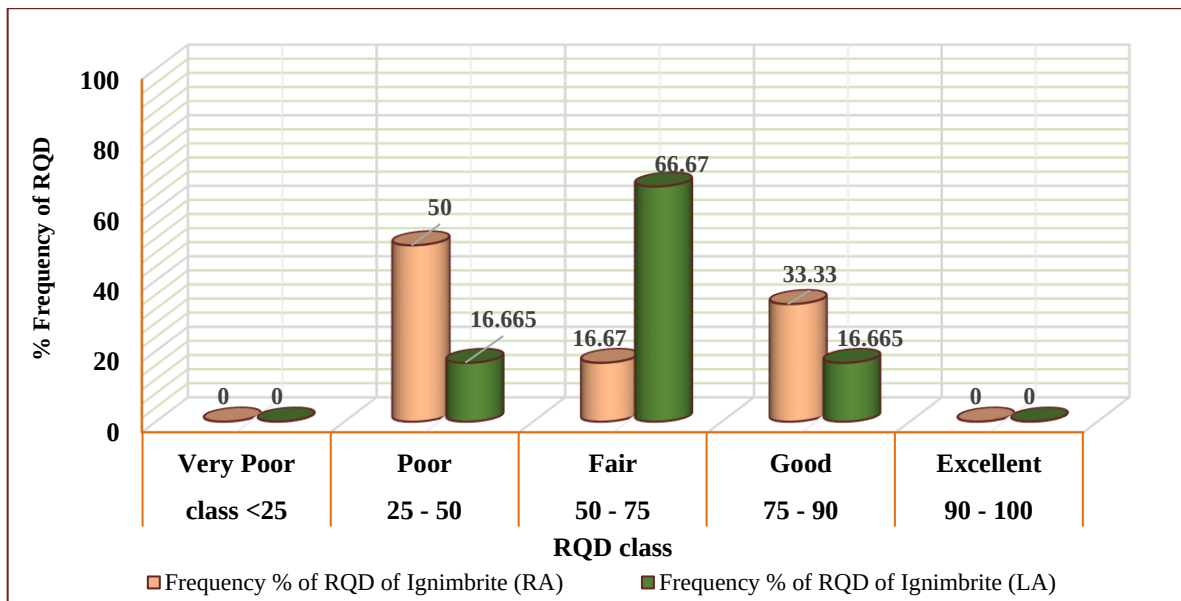


Figure 5. 3.The Percentage of RQD variation of Ignimbrite rocks of both abutment in each RQD class of Deere and Deere (1988).

5.3.5. Rock Mass Rating

Engineering classification of the rock mass is used to characterize and determine the quality of the rock mass. In this study, two methods of rock mass classification are used namely rock mass rating (Bieniawski, 1989) and geological strength index (Marinos and Hoek, 2000) for rock mass characterization and to estimate the rock mass quality at the study area. The RMR system consists of five basic parameters that represent different conditions of rock mass and discontinuities such as UCS of intact rock, RQD, spacing between discontinuities, condition of discontinuities and groundwater condition. The UCS results determined through the point load strength test (Table 5.3) was preferred to those determined through the Schmidt hammer test for the classification and RQD determined from volumetric joints (Table 5.7) were used in rock mass classification. Joint spacing, condition of discontinuity and ground water condition used in the rock mass classification were determined in the field. The rating was

assigned to each of these mentioned earlier five parameters according to Bieniawski's (1989) and were added together to obtain basic RMR. To do RMR classification, the study area was divided into different structural regions. In this study, structural aspects for the classification were based on the change in rock type, degree of fracturing and weathering.

The rock mass quality assessment of the left abutment yielded a favourable result, indicating good rock quality with a rock mass rating value of 59. Similarly, the rock quality on the right abutment was classified as fair, with a rock mass rating value of 55. The specific parameters of discontinuity, along with their corresponding rating values and the computed RMR, can be shown in Table 5.8.

Table 5. 8. Rock mass rating (RMR) values and classification of rocks at the dam site (After Bieniawski, 1989).

Input Parameters		Location			
		Right Abutment Survey (RAS)		Left Abutment Survey (LAS)	
Discontinuity Parameters		Condition	Rating	Condition	Rating
USC (MPa) ^a		8.97	2	22.87	2
RQD (%) ^a		56.66	13	63.33	13
Spacing (cm)		5 -29	8	4 - 35	8
Discontinuity Condition	Aperture (cm)	0.2 – 0.7	1	0.1 – 0.5	1
	Persistence (m)	0.7 – 1.5	6	0.2 – 1.9	6
	Roughness	Slightly Rough	3	Rough	5
	Filling Material	None	6	None	6
	Degree of Weathering	Highly Weathered	1	Moderately Weathered	3
Groundwater Condition		Dry	15	Dry	15
RMR Rating Values		55		59	
RMR Class		III		III	
Description		Fair Rock		Fair Rock	
Where RAS is the right abutment survey, LAS is the left abutment survey, ^a ; is average value.					

5.3.6. Geological Strength Index

Additionally, the GSI values were calculated from RMR using empirical equation (21) (Hoek and Brown, 1998). Subsequently, these results were compared with GSI values obtained in the field from eye observation using the quantitative GSI chart proposed by Marinos and Hoek (2000). The evaluation of the Geological Strength Index (GSI) of the rock mass was conducted by examining the exposed surfaces of the rock at the surface and the slopes that

were excavated on the abutments. The GSI value was determined by considering the state of the discontinuity surfaces and structures within the rock masses. Accordingly, the GSI results revealed that the rock mass on both abutments can be structurally classified as very blocky, which is well interlocked undisturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets. The GSI range field estimation at the right abutment (RAS section) 50 - 55, whereas at the left abutment (LAS section) was 55-60. Similarly, GSI determined from RMR₈₉ for right abutment survey (RAS) was 50 and for left abutment survey (LAS) was 54 that show fair and good rock mass conditions with very blocky rock mass structures, respectively.

The results of estimated GSI values are given in Table 5.9 and plotted in Figure 5.4 with blue and green shades showing the GSI values of the right and left abutments respectively. Note that, the GSI values plotted in the Figure 5.4 were estimated in the field. Moreover, the GSI values estimated in the field was used to determine the rock quality, and as the main input parameter for rock mass shear strength parameters, deformation modulus, limit equilibrium slope stability analyses methods.

Table 5. 9. Estimated GSI values at right and left abutment rock outcrops.

Location	Survey Location	RMR₈₉	GSI from RMR₈₉-5 (Hoek et al., 1998)	GSI from Field Estimation (Marinos and Hoek, 2000)	Rock Unit Description
Right Abutment	RAS	55	50	50-55	Highly Weathered Ignimbrite
Left Abutment	LAS	59	54	55-60	Moderately Weathered Ignimbrite
Where, RMR is the Rock mass rating, and GSI is the Geological Strength Index.					

GEOLOGICAL STRENGTH INDEX FOR JOINTED ROCKS From the lithology, structure and surface conditions of the discontinuities, estimate the average value of GSI. Do not try to be too precise. Quoting a range from 33 to 37 is more realistic than stating that GSI = 35. Note that the table does not apply to structurally controlled failures. Where weak planar structural planes are present in an unfavourable orientation with respect to the excavation face, these will dominate the rock mass behaviour. The shear strength of surfaces in rocks that are prone to deterioration as a result of changes in moisture content will be reduced if water is present. When working with rocks in the fair to very poor categories, a shift to the right may be made for wet conditions. Water pressure is dealt with by effective stress analysis		SURFACE CONDITIONS				
STRUCTURE		DECREASING SURFACE QUALITY →				
		VERY GOOD Very rough, fresh, unweathered surfaces	GOOD Rough, slightly weathered, iron stained surfaces	FAIR Smooth, moderately weathered and altered surfaces	POOR Slickensided, highly weathered surfaces with compact coating or fillings of angular fragments	VERY POOR Slickensided, highly weathered surfaces with soft clay coatings or fillings
	INTACT OR MASSIVE- Intact rock specimens or massive in-situ rock with few widely spaced discontinuities	90 80			N/A	N/A
	BLOCKY - Well interlocked undisturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets		70 60	50 40		
	VERY BLOCKY - Interlocked, partially disturbed mass with multi-faceted angular blocks formed by 4 or more joint sets			30 20		
	BLOCKY/DISTURBED/SEAMY - Folded with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes or schistosity				10	
	DISINTEGRATED - Poorly interlocked, heavily broken rock mass with mixture of angular and rounded rock pieces					
	LAMINATED/SHEARED - Lack of blockiness due to close spacing of the weak schistosity or shear planes	N/A	N/A			

Figure 5. 4. Rock mass classification based on GSI system (after Marinos and Hoek, 2000).

5.3.7. Determination of Shear Strength Parameters and Deformation Modulus

The RocData program was utilized to assess the geomechanical properties of the rock mass in the structurally controlled slope portions in the current study. The RocData software employs input parameters such as the Geological Strength Index (GSI), Uniaxial Compressive Strength of intact rocks (UCSi), material constant of intact rock (mi), and Disturbance factor (D) to determine the shear strength parameters, deformation modulus of the rock mass, and Hoek-brown criterion constants. In this case, The GSI values were determined from the field observation. The value of Uniaxial Compressive Strength of rocks (UCS) determined from point load index test and unit weight of rock (γ) were determined in the laboratory. Similarly, the value of the material constant for intact rock (mi) was obtained

from published sources (Rocscience, 2002). For ignimbrite rock, the specific value of m_i is predetermined as 22.5 within the RocData software. Subsequently, the shear strength characteristics of the rock masses were employed to compute the bearing capacity of the rocks at the dam site.

Table 5. 10. Input parameters of RocData software.

Input Parameters			
RAS		LAS	
UCSi (MPa)	8.97	UCSi (MPa)	22.87
GSI	52.5	GSI	57.5
m_i	22.5	m_i	22.5
D	0.7	D	0.7
RAS is the highly weathered ignimbrite rock unit at the right abutment, and LAS is the moderately ignimbrite rock unit at the left abutment.			

Table 5. 11. Shear strength parameters of rock masses and computed deformation modulus using RocData software.

Output of RocData software	RAS	LAS
mb	1.655	2.178
S	0.0010	0.0021
a	0.505	0.0503
C (MPa)	0.436	1.229
Φ (Deg)	30.46	32.77
UCS _{rm} (MPa)	0.278	1.030
E _{rm} (GPa)	2.24807	4.78681
mb, S, a are Hoek-brown shear strength constants, C is cohesion, Φ is friction angle, UCS _{rm} is the uniaxial compressive strength of rock mass, E _{rm} is the deformation modulus.		

The analysis indicates that the highly weathered ignimbrite rock (RAS) in the right abutment has a strength of 0.278 MPa, which is lower than the strength of the moderately weathered ignimbrite rock (LAS) with a value of 1.030 MPa. The cohesiveness of the highly weathered ignimbrite rock at the right abutment (RAS) is 0.436 MPa, which is lower than the cohesion of the moderately weathered ignimbrite rock at the left abutment (LAS), which is 1.229 MPa. The friction angle of the highly weathered ignimbrite rock was 30.46°, while the moderately weathered ignimbrite rock had a friction angle of 32.77°.

The deformation modulus of the highly weathered ignimbrite rock at the right abutment was found to be 2.24807 GPa, which is lower than the deformation modulus of the moderately weathered ignimbrite rock at the left abutment, which is 4.7868 GPa.

The RocData software, developed by Rocscience in 2004, was utilized to determine the shear strength parameters (cohesion and friction angle) of rock joints along the failure plane of structurally controlled slope sections. These parameters were calculated using the empirically derived correlation described by Barton in 1973, which has already been incorporated into the RocData software. The shear strength parameters of joint sets (RAS and LAS) can be determined using the input parameters listed in Table 5.12 and Appendix 15. The analysis utilized the predicted uniaxial compressive strength derived from the Schmidt Rebound values shown in Table 5.6. Therefore, the joint roughness coefficient was measured on-site using the joint roughness profile chart, and the joint wall compressive strength, which is equivalent to 25% and 50% of the uniaxial compressive strength of the complete rock for right and left abutment slope section rock outcrop (RAS and LAS) calculated using a Schmidt hammer rebound value, was utilized in the analysis, respectively. The value of basic friction angle was used to determine the residual friction angle considering the wet and dry Schmidt rebound value. The unit weight of rock samples was determined by laboratory analysis using buoyancy techniques, as shown in Table 5.2.

Table 5. 12. Input and output parameters of the shear strength of joints.

Input Parameters		RAS			LAS		
		JS1	JS2	JS3	JS1	JS2	JS3
ϕ_b (°)		32.5	32.5	32.5	32.5	32.5	32.5
Φ_r (°)		28.38	28.5	23.93	29.07	28.38	28.06
JRC		9	11	7	7	9	13
JCS (MPa)		0.5575	0.56	0.6075	1.685	1.5	1.84
Unit Weight (γ), (MN/m ³)		0.01536	0.01536	0.01536	0.01977	0.01977	0.01977
Slope Height (m)		24	24	24	28	28	28
Outputs of RocData software	C (MPa)	0.012	0.021	0.011	0.020	0.026	0.042
	Φ (°)	19.40	27.87	24.05	30.90	30.23	31.49
Φ_r is the residual friction angle, ϕ_b is the basic friction angle, C is Cohesion, Φ is friction angle, JRC is the joint roughness coefficient, and JCS is the joint compressive strength.							

5.4. Engineering Geological Classification and Characterization of Subsurface Rock Mass

The subsurface geological condition of the Katar dam site was assessed through the use of geophysical data for preliminary investigation, as well as geotechnical drilling of boreholes to extract core samples from the subsurface. In addition, in-situ tests were conducted within the boreholes to further investigate the subsurface conditions. Boreholes are drilled to evaluate the subsurface geological strata and perform laboratory testing on soil and rock core samples. Subsequently, the foundation stability conditions and permeability, deformability, and bearing capacity of the underlying ground are assessed through the verification of Lugeon or water pressure tests and empirical equations, respectively.

5.4.1. Analysis Geophysical Survey Findings

Geophysical methods are non-direct techniques employed to characterize the subsurface geological material. It provides a representation of the underlying conditions beneath the surface. At the Katar Dam site, geophysical methods were employed alongside drilling investigation to assess the thickness of the overburden material, analyze the vertical distribution of subsurface layers, examine the condition of the rock mass, and determine the depth of the sound rocks.

The geophysical examination revealed that the location was about 470 meters to the left of the river and had a depth of approximately 104.8 meters. The majority of the area had a resistivity value ranging from 129 to 380 $\Omega\text{m. m}$, shown by the colour green. The geophysical examination conducted around the dam axis revealed the following findings: the upper part consists of mildly weathered ignimbrite with resistivity ranging from 129 to 3380 $\Omega\text{m. m}$, the majority of the area is covered by massive ignimbrite with resistivity ranging from 380 to 3285 $\Omega\text{m. m}$, and there are extensively fractured and weathered ignimbrite sections with resistivity ranging from 15 to 44 $\Omega\text{m. m}$. The extensive ignimbrite discovered by geophysical analysis can be described as having a poor water conductivity. The presence of extensively fragmented and eroded ignimbrite serves as a clear sign of groundwater, which is observed at a depth of 68m below the surface, ranging from 50 to 150m. At the terminus of the dam axis, there is evidence of a fault (see Figure 5.5).

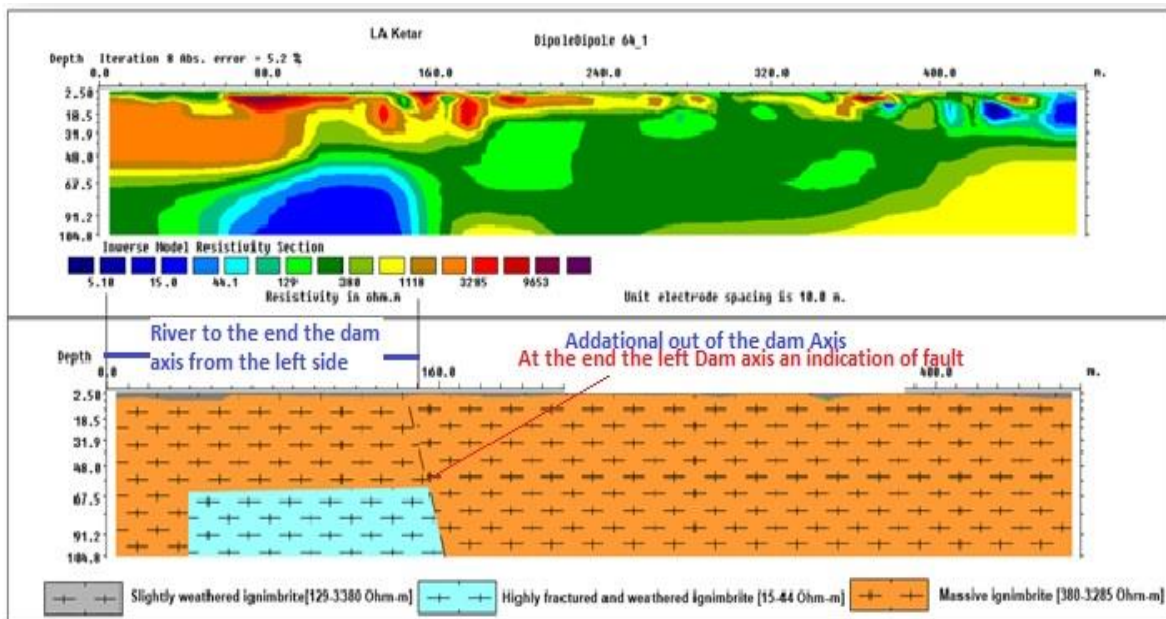


Figure 5. 5. Geophysical investigation results at the left abutment of the dam site (ECO,2023).

The geophysical examination revealed that the location was situated 470 meters away from the river on the right side, with a depth of approximately 104.8 meters. The majority of the land has a resistivity value ranging from 598 to 1444 $\Omega\text{hm. m}$. The geophysical examination along the dam axis revealed that the upper half consists of weathered ignimbrite with a thin layer of soil, which has a resistivity of 7.41 $\Omega\text{hm. m}$. The majority of the studied region is composed of extensive ignimbrite deposits, with resistivity values ranging from 598 to 1444 $\Omega\text{hm. m}$. At a distance of 0 to 150m from the right side of the river center, there is evidence of groundwater at a depth of 67m below the surface. The groundwater is found in severely fractured and weathered ignimbrite, with a resistivity ranging from 41 to 100 ohm-meters. The extensive ignimbrite discovered through geophysical analysis exhibits a notable feature of having poor water conductivity. There is an indication of a fault at the end of the dam axis. The right abutment section exhibited a higher degree of weathering and fracturing, along with a lower resistivity value (see Figure 5.6).

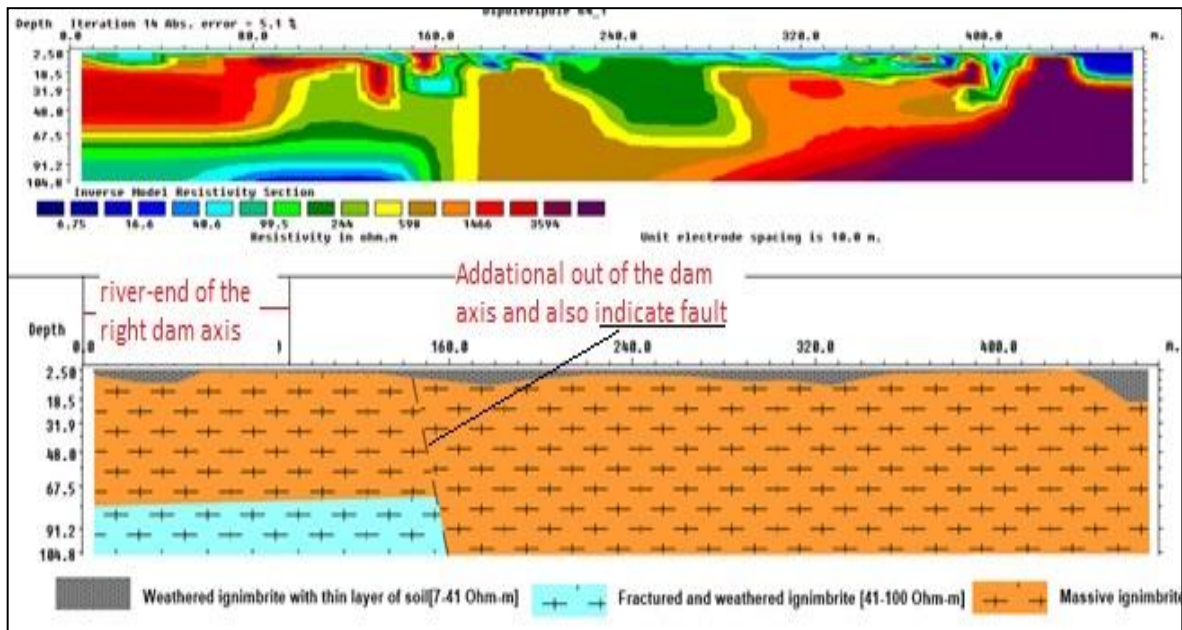


Figure 5. 6. Geophysical investigation results at the left abutment of the dam site (ECO,2023).

5.4.2. Borehole Logging and Intact Rock Properties

An engineering geological examination was conducted to assess the geological conditions of the foundation along the dam axis and intake structure sites. This investigation involves the use of three boreholes, namely KABH1 at the left abutment, KABH3 at the right abutment, and KABH2 at the left abutment conduit intake. These boreholes were drilled from the surface to depths of 66m, 53.3m, and 15m respectively, resulting in a total depth of 134.30m. Four distinct geological units were found along the dam axis from the three boreholes. The geological units are categorized based on rock masses that share comparable geo-mechanical characteristics. The homogeneous "geotechnical units" were identified. The geotechnical units identified along the dam axis are outlined below:

The Geotechnical Unit One (GTU-1) refers to the topsoil layer, which is the uppermost layer on the site. The soil is characterized as dark brown, with a dry and cohesive to non-cohesive texture. It is stiff and consists of clay and silt. The thickness of the soil ranges from 0.3 meters at the right abutment to 1.50 meters at the left abutment.

The Geotechnical Unit Two (GTU-2) consists primarily of weathered and decomposed material from un-welded tuff and lithic tuff. It has been altered to a silty-sand, low plasticity silt, and high plasticity silt soil mass. The soil mass contains a significant amount of coarse grains of sand and a minor amount of gravel soil mass derived from ash and sediments. The

GTU-2 is heterogeneous in nature, ranging in colour from light grey to brownish grey, as well as pinkish grey to pink. It is highly weathered, decomposed, and fragmented. The sediment consists of sand, gravels, cobbles, and boulders of varying sizes. The sediment also contains pores. Additionally, there are deposits of Scoria, which are pyroclastic rocks. These rocks are intercalated with phyrlic minerals such as Plagioclase and pyroxene. The core stones of the sediment can be described as scoriaceous/vesicular, meaning they have a porous and frothy texture, and amygdaloidal, meaning they include filled cavities of silica and clay.

The Geotechnical Unit Three, also known as GTU-3, is a unit of rock masses with fair characteristics. The material is a brownish-grey to dark grey welded and lithic tuff. It is moderately weathered and has moderately weak. The joints are oblique and horizontal, and there is a moderate amount of fracturing. The core contains randomly distributed lithic fragments of light and dark colors, ranging in size from sand to gravel.

The Geotechnical Unit Four (GTU-4) consists of a rock mass that is greenish grey to dark grey in colour. The rock is ignimbrite, which is fresh to slightly weathered. It is characterized by oblique and horizontal joints/beds, as well as moderate fracturing. The core of the rock contains randomly distributed sand-sized lithic fragments that are dark in colour and have a rounded and flattened shape. Additionally, there is dark grey aphanitic basalt present, which is fresh to slightly weathered. This basalt is massive and has widely spaced oblique joints and slight fracturing.

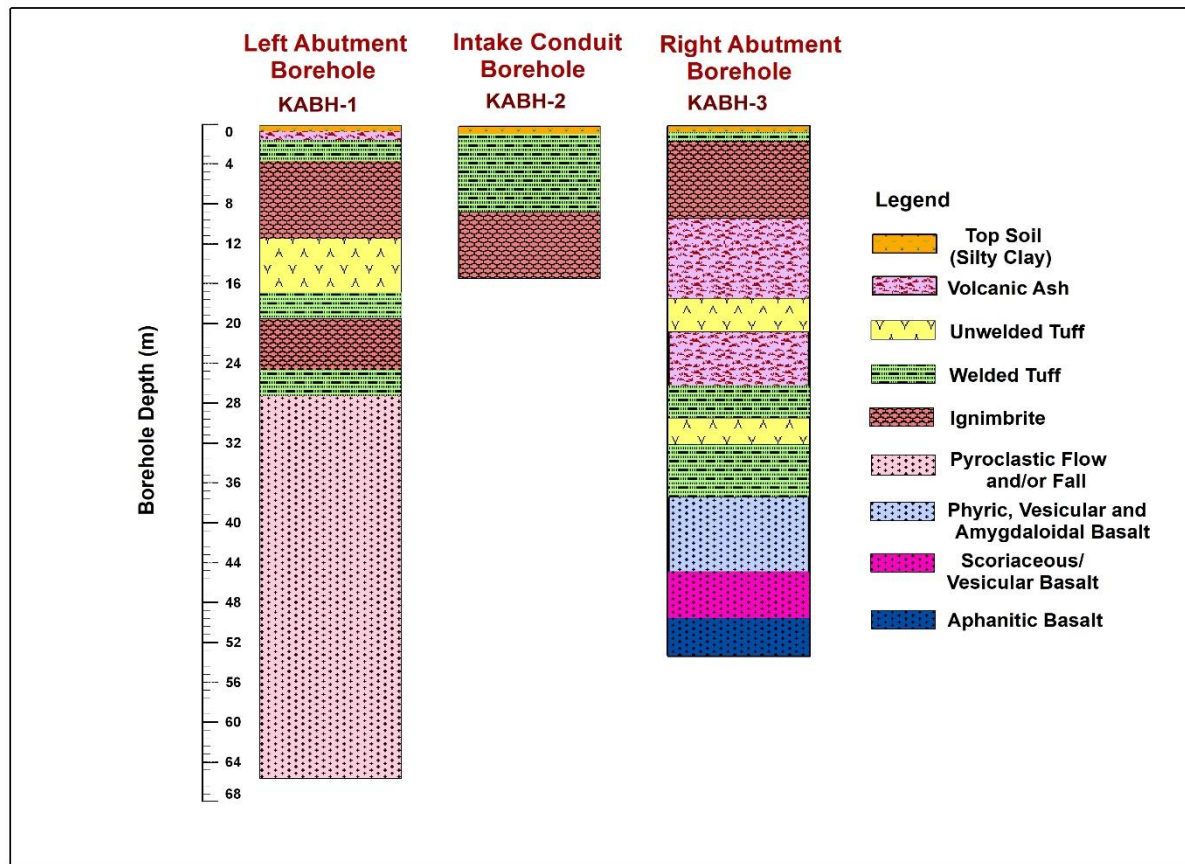


Figure 5. 7. Lithological log of the borehole at the left abutment KABH-1, conduit intake KABH-2, and right abutment KABH-3 of the dam site of the study area.

In addition to employing the geotechnical drilling investigations to understand further the subsurface geological conditions, 21 rock core samples (10 (ten), 3 (three), and 8 (eight) from the left abutment borehole, conduit intake borehole, and right abutment borehole, respectively) were taken to a laboratory for further analysis. The laboratory tests were specific gravity, porosity, slake durability index, water absorption, unconfined compressive strength (UCS), and unit weight as shown in Table 5.13.

The ten rock core samples from the left abutment borehole were collected for further laboratory analysis. The results indicated that variation in UCS value of the core samples. The ignimbrite core rock samples taken from the left abutment from the borehole depth range of 5.10m to 5.40m, 8.30m to 8.55m, and 21.70m to 22.00m have a UCS value of 10.08MPa, 15.36MPa, and 20.88MPa, respectively. These UCS value of left abutment core rock sample indicated the rock has a very low strength according to Deere and Miller (1966).

The ignimbrite core rock samples from the left abutment from the borehole depth range of 5.10m to 5.40m, 8.30m to 8.55m, and 21.70m to 22.00m have a specific gravity (G_s) value 1.80, 2.38, and 2.29 and unit weight of 6.4 kN/m^3 , 23.2 kN/m^3 , and 21.7 kN/m^3 , respectively. Again, the ignimbrite core rock samples from the left abutment from the borehole depth range of 5.10m to 5.40m, 8.30m to 8.55m, and 21.70m to 22.00m have a water absorption value of 5.94%, 2.5%, and 3.31% indicating weak rock, medium strength rock, and weak rock respectively, according to ISRM (1978) and slake durability index of 99%, 96.2%, and 99.3%, respectively indicating high class rock mass with less than 15% of material loss according to Bryson et al. (2011). These rock core samples have a porosity value of 36.28%, 5.42%, and 15.08% at the depth range from 5.10m to 5.40m, 8.30m to 8.55m, and 21.70m to 22.00m, respectively.

The unwelded tuff rock core samples taken from the left abutment from the borehole depth range of 12.75m to 12.95m have a UCS value of 16.56MPa indicating the rock core sample have a very low strength according to Deere and Miller (1966). These rock core samples also have water absorption value of 6.25% indicating that the rock mass is weak rock since the value is greater than 3 according to ISRM (1978) and slake durability index of 99.2% showing high class rock mass with less than 15% of material loss according to Bryson et al. (2011).

The Welded tuff rock core samples taken from the left abutment from the borehole depth range of 18.60m to 18.70m and 24.75m to 25.00m have a UCS value of 10.32MPa and 5.52MPa, respectively indicating the rock core sample have a very low strength according to Deere and Miller (1966). The Welded tuff rock core samples taken from the left abutment from the borehole depth range of 18.60m to 18.70m and 24.75m to 25.00m have a specific gravity value of 2.06 and 1.76, respectively. The unit weight of the rock core samples from the borehole depth range of 18.60m to 18.70m and 24.75m to 25.00m was 20.2 kN/m^3 and 11.2 kN/m^3 , respectively. The rock core samples taken from the left abutment from the borehole depth range of 18.60m to 18.70m and 24.75m to 25.00m also have analyzed water absorption value of 27.27% and 28.02%, respectively indicating that the rock mass is weak strength rock since the value is greater than 3 according to ISRM (1978). These core rock samples have a slake durability index of 88% and 71.64% showing medium class rock mass with less than 15% of material loss and medium rock class with 15%-40% material loss, respectively according to Bryson et al. (2011). Again, these rock core samples have a porosity

value of 44.12% and 38.57% at the depth range from 18.60m to 18.70m and 24.75m to 25.00m, respectively.

The scoriaceous or vesicular basalt rock core samples taken from the left abutment from the borehole depth range of 32.60m to 32.70m, 53.05m to 53.20m, 59.50m to 59.65m, and 63.75m to 63.90m have a specific gravity and unit weight value of 2.56, 2.36, 2.57, 1.81 and 23.7 kN/m^3 , 23.4 kN/m^3 , 24.8 kN/m^3 , and 13.5 kN/m^3 , respectively. These core rock samples from the borehole depth range of 32.60m to 32.70m, 53.05m to 53.20m, 59.50m to 59.65m, and 63.75m to 63.90m have a UCS value of 53.76MPa, 28.3MPa, 27.6MPa and 28.6MPa, respectively indicating slight variation rock strength of low to very low strength according to Deere and Miller (1966). Again, the water absorption value of the core rock samples from the depth range of 32.60m to 32.70m, 59.50m to 59.65m, and 63.75m to 63.90m was 6.38%, 3.83%, and 23.08%, respectively indicating that the rock mass is weak rock since the value is greater than 3 according to ISRM (1978). The water absorption value of the core rock samples from the depth range of 53.05m to 53.20m was 2.98% indicating that the rock mass is medium strength rock since the value is between 2% and 3% according to ISRM (1978). The slake durability index of the vesicular basalt rock core samples from the borehole depth range of 32.60m to 32.70m, 53.05m to 53.20m, 59.50m to 59.65m, and 63.75m to 63.90m have 98.3%, 98.8%, 97.7%, and 97.6% indicating that the rock mass is weak strength rock since the value is greater than 3 according to ISRM (1978). Again, the scoriaceous or vesicular basalt rock core samples from the depth of 53.05m to 53.20m, 59.50m to 59.65m, and 63.75m to 63.90m have the porosity value of 41.27%, 16.91%, and 33.87%, respectively.

The welded tuff rock core samples taken from the intake conduit borehole from the borehole depth range of 3.70m to 3.95m have a UCS value of 3.84MPa indicating the rock core sample have a very low strength according to Deere and Miller (1966). This rock core samples from the borehole depth range of 3.70m to 3.95m have a specific gravity of 1.76, unit weight of 17.6 kN/m^3 , and porosity of 35.29%. The water absorption value of this rock core sample was 25.14% indicating medium strength rock since the value was greater than 3% according to ISRM (1978). The slake durability index of welded tuff rock core sample was 85.2% indicating high class rock mass with less than 15% of material loss according to Bryson et al. (2011).

The ignimbrite rock core samples taken from the intake conduit borehole from the borehole depth range of 9.40m to 9.65m and 12.00m to 12.20m have a UCS value of 43.68MPa and

16.32MPa indicating the rock core sample have a very low to low strength according to Deere and Miller (1966). The rock core samples taken from the intake conduit borehole from the borehole depth range of 9.40m to 9.65m and 12.00m to 12.20m have a specific gravity of 2.43 and 2.12, unit weight of 24.8 kN/m^3 and 19.5 kN/m^3 , and porosity value of 10.38% and 26.48%, respectively. These rock core samples have a water absorption value of 4.55% and 14.2% for sample from the depth of 9.40m to 9.65m and 12.00m to 12.20m, respectively. The slake durability index of ignimbrite rock core sample was 99.1% and 99.3% for sample from the depth of 9.40m to 9.65m and 12.00m to 12.20m, respectively indicating high class rock mass with less than 15% of material loss according to Bryson et al. (2011).

The ignimbrite rock core sample was taken from the right abutment borehole from the borehole depth range of 6.80m to 7.00m have a UCS value of 46.32MPa indicating the rock core sample have a low strength according to Deere and Miller (1966). These rock core samples have a specific gravity of 2.43, unit weight of 26.6 kN/m^3 , and porosity value of 6.35%. The water absorption value of this rock core sample was 2.68% indicating that the rock mass is medium strength rock since the value is between 2% and 3% according to ISRM (1978). The slake durability index of the rock samples was 99.3% indicating high class rock mass with less than 15% of material loss according to Bryson et al. (2011).

The unwelded tuff rock core sample was taken from the right abutment borehole from the borehole depth range of 19.80m to 20.00m have a UCS value of 19.2MPa indicating the rock core sample have a very low strength according to Deere and Miller (1966). These rock core samples have a specific gravity of 1.58, unit weight of 15.6 kN/m^3 , and porosity value of 14.53%. The water absorption value of this rock core sample was 10.12% indicating that the rock mass is weak rock since the value is greater than 3% according to ISRM (1978). The slake durability index of the rock samples was 99.8% indicating high class rock mass with less than 15% of material loss according to Bryson et al. (2011).

The welded tuff rock core sample was taken from the right abutment borehole from the borehole depth range of 27.75m to 28.00m and 35.00m to 35.50m have a UCS value of 11.76MPa and 5.28MPa indicating the rock core sample have a very low strength according to Deere and Miller (1966). These rock core samples have a specific gravity of 1.79 and 1.85, unit weight of 15.9 kN/m^3 and 15.5 kN/m^3 , and porosity value of 37.17% and 40.57%, respectively. The water absorption value of this rock core sample was 25.86% and 28.03%

indicating that the rock mass is weak rock since the value is greater than 3% according to ISRM (1978).

The amygdaloidal basalt rock core sample was taken from the right abutment borehole from the borehole depth range of 41.50m to 41.80m have a UCS value of 8.16MPa indicating the rock core sample have a very low strength according to Deere and Miller (1966). These rock core samples have a specific gravity of 2.34, unit weight of 21.1kN/m³, and porosity value of 12.65%. The water absorption value of this rock core sample was 5.83% indicating that the rock mass is weak rock since the value is greater than 3% according to ISRM (1978). The slake durability index of the rock samples was 84.2% indicating medium class rock mass with medium rock class with 15%-40% material loss according to Bryson et al.(2011).

The vesicular basalt rock core sample was taken from the right abutment borehole from the borehole depth range of 42.65m to 42.80m and 46.65m to 46.80m have a UCS value of 10.32MPa and 19.8MPa indicating the rock core sample have a very low strength according to Deere and Miller (1966). These rock core samples have a specific gravity of 2.5 and 2.22, unit weight of 23.9kN/m³, and porosity value of 12.5% and 21.48%. The water absorption value of this rock core sample was 5.21% and 19.24% indicating that the rock mass is weak rock since the value is greater than 3% according to ISRM (1978). The slake durability index of the rock samples was 95.8% and 97.9% indicating high class rock mass with 15% material loss according to Bryson et al.(2011).

The aphanitic basalt rock core sample was taken from the right abutment borehole from the borehole depth range of 50.15m to 50.40m have a UCS value of 69.6MPa indicating the rock core sample have a medium strength according to Deere and Miller (1966). These rock core samples have a specific gravity of 3.10, unit weight of 31.4kN/m³, and porosity value of 4.08%. The water absorption value of this rock core sample was 1.26% indicating that the rock mass is strong rock since the value is between 1% and 2% according to ISRM (1978).

Table 5. 13. Summarized results of rock core samples analysis in terms of Gs, WA, unit weight, porosity, UCS and SD.

No	Sample ID	Depth (m)	Gs	WA (%)	Unit Weight (kN/m ³)	Porosity (%)	IS ₍₅₀₎ , MPa	UCS, MPa	SDI (%)	Rock Unit
1	KABH-01/RS-01	5.10 - 5.40	1.80	5.96	16.4	36.28	0.42	10.08	99	Ignimbrite
2	KABH-01/RS-02	8.30 -8.55	2.38	2.5	23.2	5.42	0.64	15.36	96.2	Ignimbrite
3	KABH-01/RS-03	12.75 - 12.95	-	6.25	-	-	0.69	16.56	99.2	Unwelded tuff
4	KABH-01/RS-04	18.60 - 18.70	2.06	27.27	20.2	44.12	0.43	10.32	88	Welded tuff
5	KABH-01/RS-05	21.70 - 22.00	2.29	3.31	21.7	15.08	0.87	20.88	99.3	Ignimbrite
6	KABH-01/RS-06	24.75 - 25.00	1.76	28.02	11.5	38.57	0.23	5.52	71.64	Welded tuff
7	KABH-01/RS-07	32.60 - 32.70	2.56	6.38	23.7	-	2.24	53.76	98.3	Scoriaceous/ Vesicular Basalt
8	KABH-01/RS-08	53.05 - 53.20	2.36	2.98	23.4	41.27	1.2	28.8	98.8	
9	KABH-01/RS-09	59.50 - 59.65	2.57	3.83	24.8	16.91	1.15	27.6	97.7	
10	KABH-01/RS-10	63.75 - 63.90	1.81	23.08	13.5	33.87	1.19	28.56	97.6	
11	KABH-02/RS-01	3.70 -3.95	1.76	25.14	17.6	35.29	0.16	3.84	85.2	Welded tuff
12	KABH-02/RS-02	9.40 - 9.65	2.43	4.55	24.8	10.38	1.82	43.68	99.1	Ignimbrite
13	KABH-02/RS-03	12.00 - 12.20	2.12	14.2	19.5	26.48	0.68	16.32	99.3	Ignimbrite
14	KABH-03/RS-01	6.80 -7.00	2.43	2.68	26.6	6.35	1.93	46.32	99.3	Ignimbrite
15	KABH-03/RS-02	19.80 - 20.00	1.58	10.12	15.6	14.53	0.8	19.2	98.8	Unwelded tuff
16	KABH-03/RS-03	27.75 - 28.00	1.79	25.86	15.9	37.17	0.49	11.76	-	Welded tuff
17	KABH-03/RS-04	35.00 - 35.50	1.85	28.03	15.5	40.57	0.22	5.28	-	Welded tuff
18	KABH-03/RS-05	41.50 - 41.80	2.34	5.83	21.1	12.65	0.34	8.16	84.2	Amygdaloidal Basalt
19	KABH-03/RS-06	42.65 - 42.80	2.5	5.21	23.9	12.5	0.43	10.32	95.8	Vesicular Basalt
20	KABH-03/RS-07	46.65 - 46.80	2.22	19.24	-	21.48	0.825	19.8	97.9	Vesicular Basalt
21	KABH-03/RS-08	50.15 - 50.40	3.10	1.26	31.4	4.08	2.90	69.6	-	Aphanitic Basalt

5.4.3. Soil Logging Analysis Findings

In addition to the rock core sample analysis, Soil core samples were also taken to laboratory analysis for further analysis. Laboratory tests such as grain size analysis test, Atterberg limit (liquid limit/LL), natural moisture content, and specific gravity were utilized for the analysis of the soil core samples. 2 (two) and 3 (three) soil core sample taken from the left abutment borehole and right abutment borehole, respectively. At the left abutment borehole, soil core samples from the depth range from 29.0m to 29.45m have a specific gravity of 2.85 and natural moisture content of 23.13%. Soil core samples at the depth range from 39.5m to 40.0m have a specific gravity of 2.86 and natural moisture content of 0.51. From grain size analysis results the soil core samples grain size distribution was analyzed in terms of gravel percentage, sand percentage, silt percentage and clay percentage for both left and right abutment boreholes. Soil core samples at the depth range from 29m to 29.5m have a 0% of gravel particle distribution, 71.23% of sand particle distribution, 28.6% silt particle distribution, and 0.17% of clay particle distribution (Figure 5.9. and Table 5.14). Soil core samples at the depth range from 39.5m to 40.0m have a 84.02% of gravel particle distribution, 6.84% of sand particle distribution, 9% silt particle distribution, and 0.15% of clay particle distribution (Figure 5.9. and Table 5.14).

At the right abutment borehole, soil core samples from the depth range from 14.40m to 14.85m have a specific gravity of 2.38 and natural moisture content of 30.23%. Soil core samples at the depth range from 17m to 17.45m have a specific gravity of 2.45 and natural moisture content of 12.11%. Soil core samples at the depth range from 24.50m to 25m have a specific gravity of 2.38 and natural moisture content of 2%. Soil core samples at the depth range from 14.4m to 14.85m have a 1.49% of gravel particle distribution, 55.54% of sand particle distribution, 42.03% silt particle distribution, and 0.94% of clay particle distribution (Figure 5.9. and Table 5.14). Soil core samples at the depth range from 17.0m to 17.45m have a 1.23% gravel particle distribution, 48.21% of sand particle distribution, 49.07% silt particle distribution, and 1.5% of clay particle distribution (Figure 5.9. and Table 5.14). Soil core samples at the depth range from 24.50m to 25m have a 0% gravel particle distribution, 25% of sand particle distribution, 74.51% silt particle distribution, and 0.49% of clay particle distribution (Figure 5.9. and Table 5.14).

Table 5. 14. Summarized table of soil core samples laboratory analysis results.

Sample ID	Depth	Grain Size Analysis (%)				Atterberg Limit	NMC	Gs
		Cl	Si	Sa	Gr	LL		
KABH-01/SS-01	29 – 29.45	0.17	28.6	71.23	0	27.45	23.13	2.85
KABH-01/SS-02	39.5 -40	0.15	9	6.84	84.02	19.5	0.51	2.86
KABH -03/SS-01	14.4–14.85	0.94	42.03	55.54	1.49	33.58	30.23	2.38
KABH -03/SS-02	17 – 17.45	1.5	49.07	48.21	1.23	36.75	12.11	2.45
KABH -03/SS-03	24.5 - 25	0.49	74.51	25	0	63.55	2	2.38

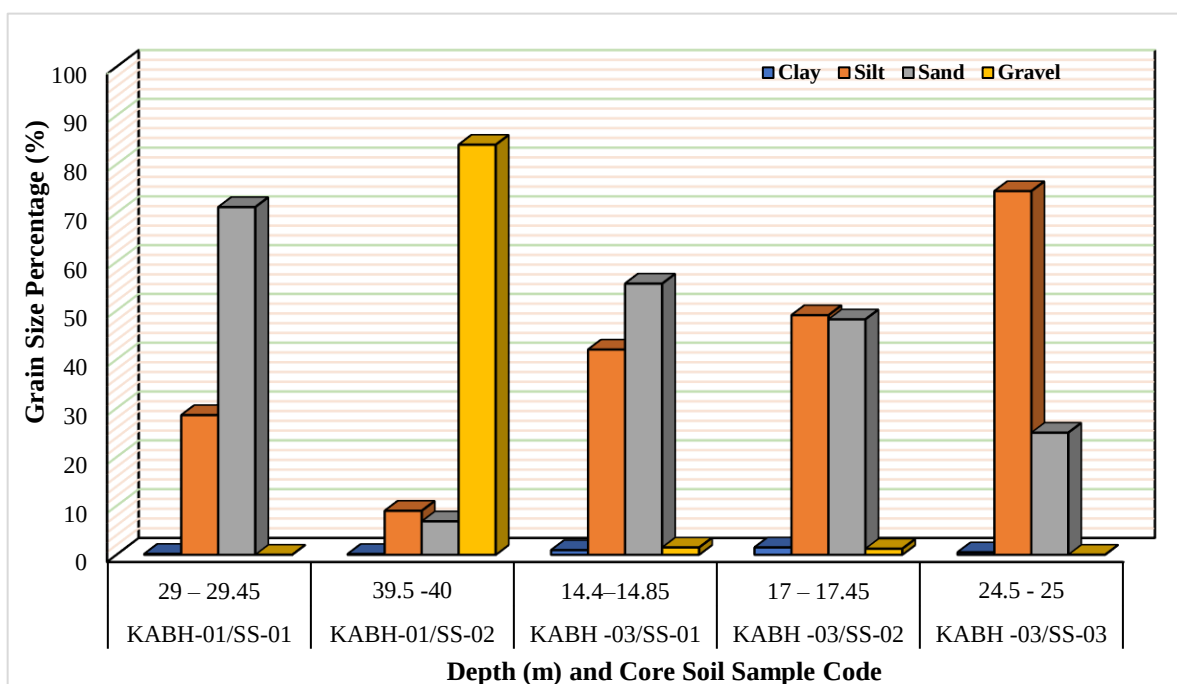


Figure 5. 8. Graphical Representation of Grain size distribution of the soil core samples.

5.4.4. Packer Test Result Interpretations

To evaluate the water-tightness condition of the dam site, the permeability of the rock masses was estimated using a packer test and falling head permeability tests. The Falling head field permeability tests were conducted for weak formations while the packer test was utilized for relatively hard formation.

5.4.4.1. Analysis Falling Head Permeability Test Findings

The hydraulic conductivity of the uppermost, highly weathered, and fractured weak rock masses forming the abutments of the dam was determined by the in-situ falling head permeability test. A total of ten (10) falling head field permeability tests were performed in boreholes KAHB-1 and KABH-3 along the dam axis at various depths, as indicated in Table

5.15. Consequently, six (6) falling head permeability tests were carried out at the left abutment of the dam axis, specifically at the KABH-1 borehole, ranging from a depth of 28.0m to 56.0m. As to the USBR (1987) study, the findings indicate that hydraulic conductivity ranges from 1.80E-04 cm/sec to 4.96E-04 cm/sec, placing it in the semi-previous permeability class. Likewise, the examination took place at the KAHB-3 borehole, specifically at the right abutment of the dam site, spanning a depth range of 12.0m to 32.0m. The test result indicated that the hydraulic conductivity ranges from 5.10E-05cm/sec to 1.92E-04cm/sec, placing it within the category of a semi-pervious permeability class.

Table 5. 15. Summary of falling head permeability test and permeability class according to (USBR, 1987).

BH ID	Falling head permeability test (cm/s)		
	Test section (m)	Permeability value (K in cm/s)	USBR rating (1987)
KABH-01 (Left Abutment)	28.0-33.0	3.20E-04	Semi-previous
	33.0-37.0	4.96E-04	Semi-previous
	37.0-41.0	3.81E-04	Semi-previous
	41.0-46.0	4.83E-04	Semi-previous
	46.0- 51.0	2.74E-04	Semi-previous
	51.0- 56.0	1.80E-04	Semi-previous
KABH-03 (Right Abutment)	12.0-17.0	1.92E-04	Semi-previous
	17.0-22.0	5.10E-05	Semi-previous
	22.0-27.0	2.80E-04	Semi-previous
	27.0- 32.0	3.38E-04	Semi-previous

5.4.4.2. Analysis of Packer Test: Rock Permeability Evaluation

The adequate assessment and interpretation of permeability results at a dam site are crucial in determining the amount of seepage through the foundation and valley sides, as well as in deciding if ground treatment is necessary. At three borehole locations, Engineering Corporation of Oromia conducted the test (ECO, 2023). Rock mass portions for all drilled boreholes have undergone Single Packer Permeability Tests. A total of ten 13 tests were done in the current investigation at the Katar Dam site. The test was performed within geotechnical drilled boreholes located along the axis of the dam. Consequently, the outcome of the packer test reveals that the foundation geological units exhibit distinct Lugeon values and flow behavior, as illustrated in Table 5.16.

Houlsby (1976) classified the typical flow behaviors observed in packer test practice into five different groups. Depending on his classification, the in-situ permeability result of the dam site displays different flow types in different sections of boreholes drilled at the

proposed dam foundation and abutments. Some representative flow behaviors are selected from different borehole sections as shown in Table 5.16 below. The description and selection of representative values for each flow type are discussed below with their observation section.

In the left abutment and intake conduit borehole of the dam site, the test was conducted at these boreholes with borehole code of KABH-01 and KABH-02, the tests were conducted in 5 and 2 sections respectively. In the left abutment borehole KABH-01, the determined permeability of rock mass at the test section depth range of 2.00m to 7.00m was 0.34 and characterized by dilation water flow pattern. The determined permeability of rock mass at the test section depth range of 7.00m to 13.00m, 13.00m to 18.00m, 18.00m to 23.00m, 23.00m to 28.00m was 0.021, 0.35, 0.29, 0.034 and characterized by dilation, void filling, washout, dilation water flow pattern, respectively. In the left abutment borehole KABH-01, there is no potential of leakage of water through the rock mass considering the Lu values at each test section are less than 1 indicating very tight condition of rock mass discontinuities according to Fell (2005).

In the intake conduit borehole with code of KABH-02 the test was conducted in 2 sections. The lugeon values for this specific borehole at the depth range of 5.50m to 10.50m, and 10.00m to 15.00m were 12.07 and 9.08, respectively. The test section at this borehole has the water flow pattern of turbulent. According to Fell (2005), both tests at borehole KABH-02 have moderate permeability class indicating that some open joint conditions exists within the test depth. This higher permeability or lugeon is frequently linked to turbulent flow during water injection tests. The turbulent and erratic motion of water suggests enhanced interconnection among fractures, resulting in an increased Lugeon value.

In the right abutment borehole with borehole code of KABH-03 of the dam site, the test was conducted in six sections. The lu value of test section at the depth 2.00m to 7.00m was 2.14 which can be classified as low permeability class inferring presence of tight condition of rock mass discontinuities according to Fell (2005). The water flow pattern observed in this test portion exhibited dilation. The observed water flow pattern suggests the existence of fractures that have the potential to expand. However, these fractures may not have a significant impact on the overall water flow, either due to limited connectivity or the presence of less permeable zones. The lu value of test section at the depth 7.00m to 12.00m and 38.00m to 43.00m was 13.23 and 4.81 which can be classified as moderate permeability

class inferring presence of few partly open condition of rock mass discontinuities and low permeability class indicating that presence of tight condition of rock mass discontinuities according to Fell (2005). The water flow pattern observed in this test part displayed turbulent. Turbulent flow implies the presence of interconnected cracks, allowing water to move chaotically. LU values in the low to moderate range suggest that the rock mass has limited permeability. This could be due to a lack of extensive fracture networks, partially obstructed or rough-surfaced fractures that impede water flow, and smaller apertures in the fractures, all of which contribute to the rock mass being low to moderately permeable. Considering the presence of the highly weathered and fractured geological material which is also weak, the falling head permeability test was utilized to determine the permeability of this geological material at the depth range of 12.00 to 33.00m. The lu value of test section at the depth 32.00m to 38.00m was 0.754 which is further classified as very low permeability class indicating that presence of very tight condition of rock mass discontinuities according to Fell (2005). The water flow pattern observed in this test part displayed washout. Considering the presence of the highly weathered and fractured geological material, which is also weak, the falling head permeability test was utilized to determine the permeability of this geological material at the depth range of 12.00 to 33.00m. The lu value of test section at the depth 43.00m to 48.00m and 50.30m to 53.30m was 0.327 and 0.328 which is further classified as very low permeability class indicating that presence of very tight condition of rock mass discontinuities according to Fell (2005). The test component exhibited dilatation in the measured water flow pattern. Dilation refers to the ability of fractures to expand or enlarge when water is injected into them. A low LU value indicates that the permeability of the rock mass is limited, despite the possibility of expansion. This limitation is caused by fractures that are either not well-connected or form isolated channels, as well as the presence of narrower openings and a complicated fracture network.

In addition to determining the permeability of rock mass, the types of flow were also determined based on the packer test results. The flow type can be utilized to identify discontinuity and grouting-type features. At left abutment, the dominant flow type is dilation with distribution percentages of 60%, washout and void filling with distribution percentages of 20% each. The dominant flow pattern at the right abutment is dilation which constitutes 50% of the flow pattern, 33.33% of the flow pattern is turbulent, and 16.67% is washout flow types. At intake conduit abutments, the dominant flow type is turbulent with distribution percentages of 100% as shown in Figure 5.10.

Table 5. 16. Summarized packer test results and Lu values with respect to the depth.

BH ID	Test section (m)	Packer test Hydraulic Conductivity (cm/sec)	Representative Lugeon value (Lu)	Flow Pattern
KABH- 01 (Left Abutment)	2.00-7.00	4.34E-06	0.34	Dilation
	7.0-13.00	2.81E-07	0.021	Dilation
	13.00-18.00	4.53E-06	0.35	Void Filling
	18.00-23.00	3.82E-06	0.29	Washout
	23.00-28.00	4.46E-07	0.034	Dilation
KABH- 02 (Conduit Intake)	5.50-10.50	1.57E-04	12.07	Turbulent
	10.00-15.00	1.18E-04	9.08	Turbulent
KABH- 03 (Right Abutment)	2.00-7.00	2.78E-05	2.14	Dilation
	7.00-12.00	1.72E-04	13.23	Turbulent
	32.00-38.00	9.80E-06	0.754	Washout
	38.00-43.00	6.25E-05	4.81	Turbulent
	43.00-48.00	4.25E-06	0.327	Dilation
	50.30-53.30	4.26E-06	0.328	Dilation

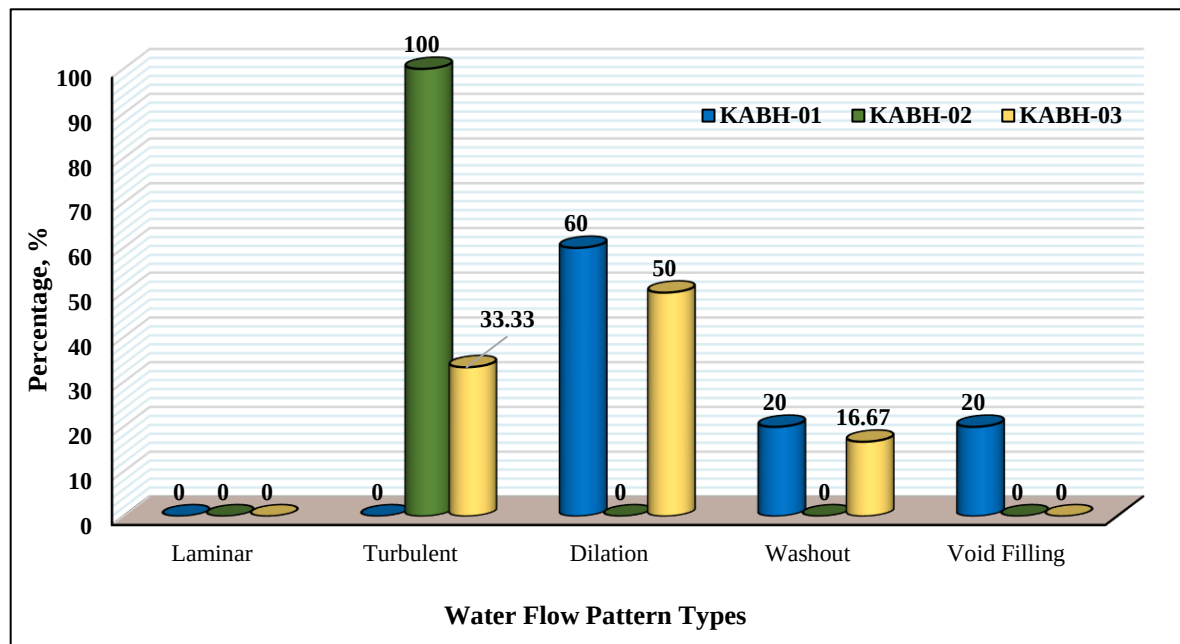


Figure 5. 9. Flow Pattern Distributions of the three boreholes (KABH-01, KABH-02, and KABH-03).

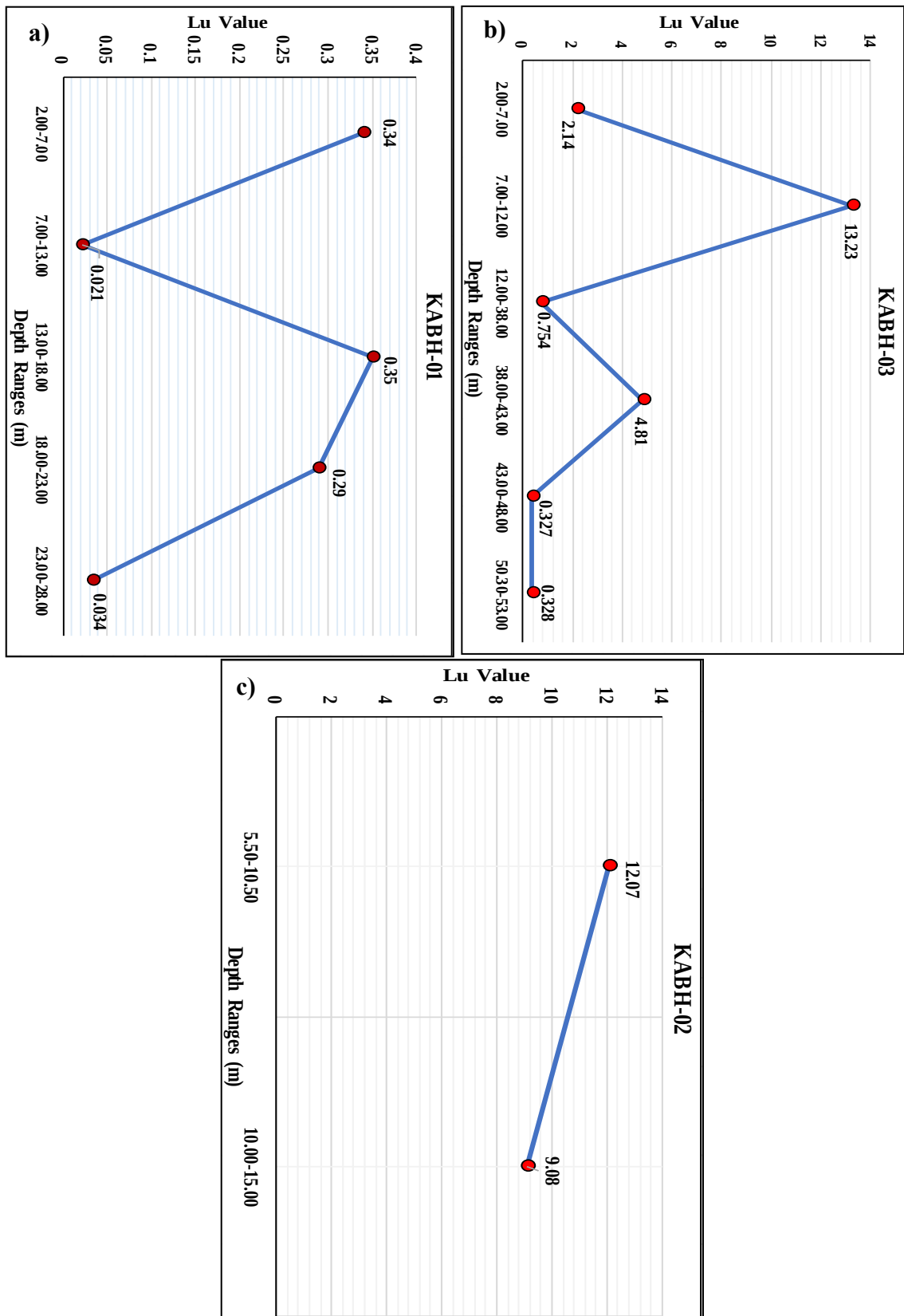


Figure 5. 10. Lu values with respect to depth. a) Left Abutment (KABH-1), b) Right Abutment (KABH-3), and c) Conduit Intake (KABH-2).

5.4.5. Correlation between RQD and Depth

It is possible for the rock quality, as determined by RQD, to change significantly with depth due to the presence of fractures, bedding planes, or other geological structures, as well as changes in the types of rocks that are present. Despite this, there is a possibility that the association between RQD and drilled depth exhibits some general movements or patterns. It is possible, for instance, that RQD will decrease with depth if deeper rocks are more fragmented or have experienced more substantial weathering than shallower rocks. There is also the possibility that RQD will rise with depth if deeper rocks are more competent and have fewer fractures than rocks that are shallower.

The welded tuff rock unit in the left abutment borehole of the dam site at the depth range from 1.50m to 3.45m have a RQD value of 33% where the rock mass quality of the rock unit was plotted to be in the poor rock mass quality class indicating that the rock unit is weathered and fractured rock mass according to Deere and Deere (1988). The Welded tuff rock unit at the depth range from 17.0m to 19.20m have an average RQD value of 47.5% where the rock mass quality of the rock unit was plotted to be in the fair rock mass quality class indicating that the rock unit is highly weathered and fractured rock mass according to Deere and Deere (1988). At the depth range from 17.0m to 19.20m, the welded tuff rock unit have the minimum value of RQD of 11% and maximum value of RQD of 84% where the rock mass quality of the rock unit was plotted to be in the very poor and good rock mass quality according to Deere and Deere (1988). The degree of jointing and weathering of the rock unit was weathered and fractured for the RQD of 11% and relatively sound to sound for the RQD of 84% according to Deere and Deere (1988). The Welded tuff rock unit at the depth range from 24.3m to 27m have an average RQD value of 45% where the rock mass quality of the rock unit was plotted to be in the fair rock mass quality class indicating that the rock unit is highly weathered and fractured rock mass according to Deere and Deere (1988). At the depth range from 24.3m to 27m, the welded tuff rock unit have the minimum value of RQD of 0% and maximum value of RQD of 90% where the rock mass quality of the rock unit was plotted to be in the very poor and good rock mass quality according to Deere and Deere (1988). The degree of jointing and weathering of the rock unit was weathered and fractured for the RQD of 0% and relatively sound to sound for the RQD of 90% according to Deere and Deere (1988).

The unwelded tuff rock unit at the depth range from 11.6m to 17m have an average RQD value of 7.25% where the rock mass quality of the rock unit was plotted to be in the very poor rock mass quality class indicating that the rock unit is highly weathered and fractured rock mass according to Deere and Deere (1988). At the depth range from 11.60m to 17m, the unwelded tuff rock unit have the minimum value of RQD of 5% and maximum value of RQD of 13% where the rock mass quality of the rock unit was plotted to be in the very poor and good rock mass quality according to Deere and Deere (1988). The degree of jointing and weathering of the rock unit was weathered and fractured for the RQD of 5% and 13% according to Deere and Deere (1988).

The ignimbrite rock unit at the depth range from 3.45m to 11.60m have an average RQD value of 70.9% where the rock mass quality of the rock unit was plotted to be in the fair poor rock mass quality class indicating that the rock unit is moderately weathered and fractured rock mass according to Deere and Deere (1988). At the depth range from 3.45m to 11.60m, the ignimbrite rock unit have the minimum value of RQD of 44% and maximum value of RQD of 85% where the rock mass quality of the rock unit was plotted to be in the poor and good rock mass quality according to Deere and Deere (1988). The degree of jointing and weathering of the rock unit was weathered and fractured for the RQD of 44% and relatively sound to sound for RQD value of 85% according to Deere and Deere (1988). The ignimbrite rock unit at the depth range from 19.2m to 24.30m have an average RQD value of 85% where the rock mass quality of the rock unit was plotted to be in the fair poor rock mass quality class indicating that the rock unit is moderately weathered and fractured rock mass according to Deere and Deere (1988). At the depth range from 19.20m to 24.30m, the ignimbrite rock unit have the minimum value of RQD of 82% and maximum value of RQD of 89% where the rock mass quality of the rock unit was plotted to be in the good rock mass quality according to Deere and Deere (1988). The degree of jointing and weathering of the rock unit was relatively sound to sound for the RQD of 82% and 89% according to Deere and Deere (1988).

The scoriaceous/vesicular pyroclastic rock unit at the depth range from 27m to 66m have an average RQD value of 6.6% where the rock mass quality of the rock unit was plotted to be in the very poor rock mass quality class indicating that the rock unit is highly weathered and fractured rock mass according to Deere and Deere (1988). At the depth range from 27m to 66m, the ignimbrite rock unit have the minimum value of RQD of 0% and maximum value

of RQD of 42% where the rock mass quality of the rock unit was plotted to be in the very poor and poor rock mass quality according to Deere and Deere (1988). The degree of jointing and weathering of the rock unit was weathered and fractured for the RQD of 0% and RQD 43% according to Deere and Deere (1988).

The RQD values of the rock units at the left abutment decreases with increases borehole depth which indicates the high degree of weathering and fracturing as shown in Figure 5.12a. This decreasing trend of RQD values could be problematic considering the poor rock mass quality for engineering applications which is due to the variation in rock types and degree of weathering and fracturing with the drilled borehole depth. According to Berhane and Walraevens (2013), it is quite difficult to rely on the RQD values in this scenario for the purpose of determining the subsurface geological material conditions, as well as the groutability of the section and the depths. In order to provide preliminary information on the subsurface conditions of the geological materials and the degree of the necessity for ground treatment in order to limit the likelihood of engineering geological issues, the depths of the low RQD values are provided. Consequently, in order to ascertain the depth of the sections and the groutability of the grout, lugeon tests must be utilised in conjunction with an integration of RQD values.

The welded tuff rock unit in the right abutment borehole at the depth range from 0.30m to 1.50m have an RQD value of 99% where the rock mass quality of the rock unit was plotted to be in the excellent rock mass quality class indicating that the rock unit is low weathered and fractured rock mass according to Deere and Deere (1988) as shown in Figure 5.12b. The welded tuff rock unit in the right abutment borehole at the depth range from 26.65m to 29.50m have an RQD value of 66% where the rock mass quality of the rock unit was plotted to be in the fair rock mass quality class indicating that the rock unit is moderately weathered and fractured rock mass according to Deere and Deere (1988). The welded tuff rock unit at the depth range from 32.0m to 37.40m have an average RQD value of 44.33%% where the rock mass quality of the rock unit was plotted to be in the poor rock mass quality class indicating that the rock unit is highly weathered and fractured rock mass according to Deere and Deere (1988). At the depth range from 32.0m to 37.40m, the welded tuff rock unit have the minimum value of RQD of 26% and maximum value of RQD of 65% where the rock mass quality of the rock unit was plotted to be in the poor and fair rock mass quality according to Deere and Deere (1988). The degree of jointing and weathering of the rock unit

was highly weathered and fractured for the RQD of 26% and moderately weathered and fractured for the RQD of 65% according to Deere and Deere (1988). The ignimbrite rock unit at the depth range from 1.50m to 9.40m have an average RQD value of 82.67% where the rock mass quality of the rock unit was plotted to be in the good rock mass quality class indicating that the rock unit is slightly weathered and fractured rock mass according to Deere and Deere (1988). At the depth range from 1.50m to 9.40m, the ignimbrite rock unit have the minimum value of RQD of 50% and maximum value of RQD of 100% where the rock mass quality of the rock unit was plotted to be in the fair and excellent rock mass quality according to Deere and Deere (1988). The degree of jointing and weathering of the rock unit was moderately weathered and fractured for the RQD of 50% and relatively sound to sound for the RQD of 100% according to Deere and Deere (1988). The volcanic ash rock unit at the depth range from 9.40m to 17.75m have an average RQD value of 8.33% where the rock mass quality of the rock unit was plotted to be in the very poor rock mass quality class indicating that the rock unit is highly weathered and fractured rock mass according to Deere and Deere (1988). At the depth range from 9.40m to 17.75m, the volcanic ash rock unit have the minimum value of RQD of 0% and maximum value of RQD of 50% where the rock mass quality of the rock unit was plotted to be in the very poor and fair rock mass quality according to Deere and Deere (1988). The degree of jointing and weathering of the rock unit was highly weathered and fractured for the RQD of 0% and moderately weathered and fractured for the RQD of 50% according to Deere and Deere (1988). The volcanic ash rock unit at the depth range from 20.3m to 26.65m have an average RQD value of 12.5% where the rock mass quality of the rock unit was plotted to be in the very poor rock mass quality class indicating that the rock unit is highly weathered and fractured rock mass according to Deere and Deere (1988). At the depth range from 20.3m to 26.65m, the volcanic ash rock unit have the minimum value of RQD of 6% and maximum value of RQD of 10% where the rock mass quality of the rock unit was plotted to be in the very poor rock mass quality according to Deere and Deere (1988). The degree of jointing and weathering of the rock unit was highly weathered and fractured for the RQD of 6% and 10% according to Deere and Deere (1988).

The unwelded tuff rock unit at the depth range from 17.45m to 20.30m have an average RQD value of 3% where the rock mass quality of the rock unit was plotted to be in the very poor rock mass quality class indicating that the rock unit is highly weathered and fractured rock mass according to Deere and Deere (1988). At the depth range from 17.45m to 20.30m, the unwelded tuff rock unit have the minimum value of RQD of 0% and maximum value of

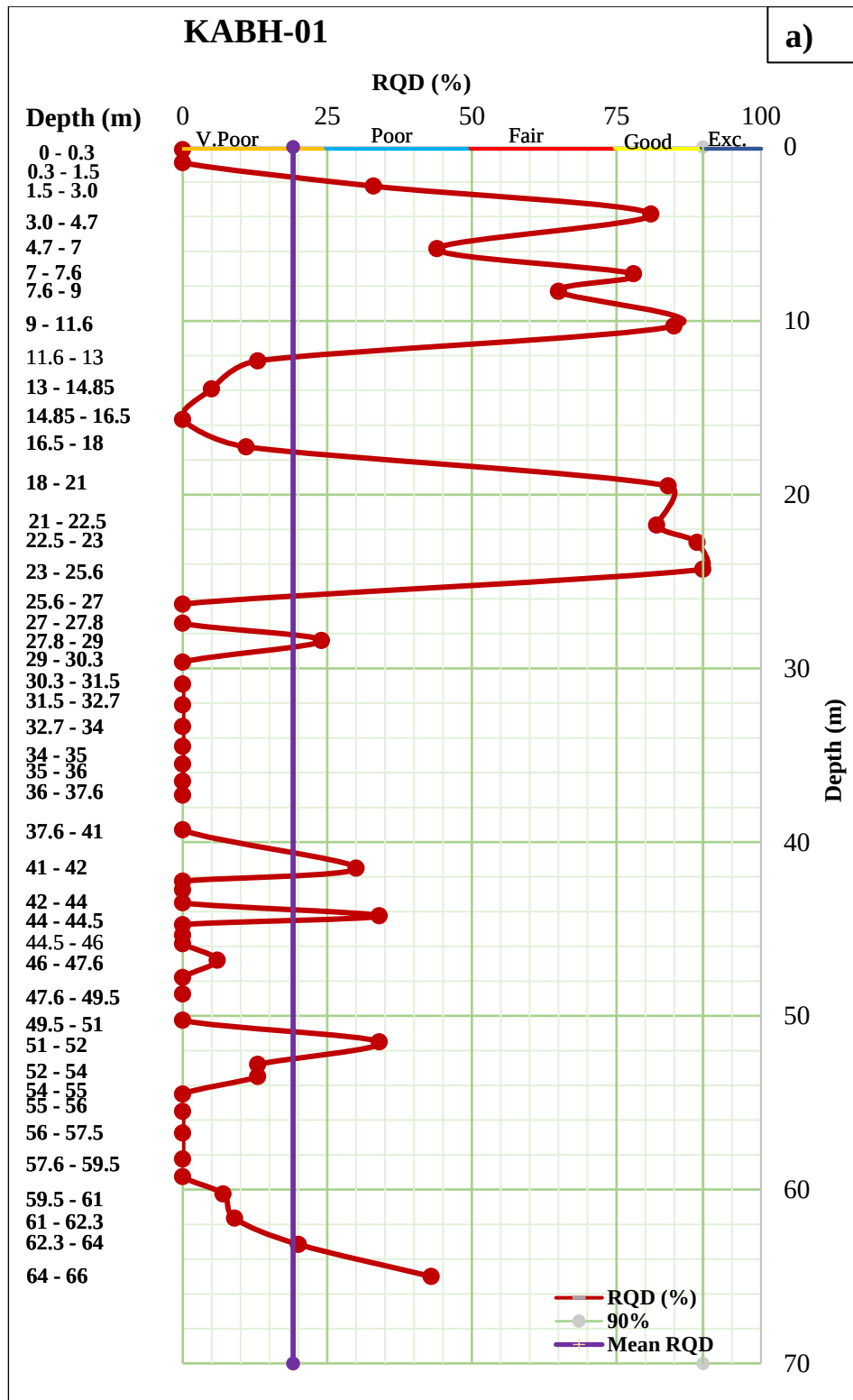
RQD of 9% where the rock mass quality of the rock unit was plotted to be in the very poor rock mass quality according to Deere and Deere (1988). The degree of jointing and weathering of the rock unit was highly weathered and fractured for the RQD of 0% and 50% according to Deere and Deere (1988). The unwelded tuff rock unit at the depth range from 29.5m to 32.0m have an average RQD value of 33% where the rock mass quality of the rock unit was plotted to be in the poor rock mass quality class indicating that the rock unit is highly weathered and fractured rock mass according to Deere and Deere (1988). At the depth range from 29.5m to 32.0m, the unwelded tuff rock unit have the minimum value of RQD of 0% and maximum value of RQD of 66% where the rock mass quality of the rock unit was plotted to be in the very poor and fair rock mass quality according to Deere and Deere (1988). The degree of jointing and weathering of the rock unit was highly weathered and fractured for the RQD of 0% and moderately weathered and fractured for the RQD of 66% according to Deere and Deere (1988).

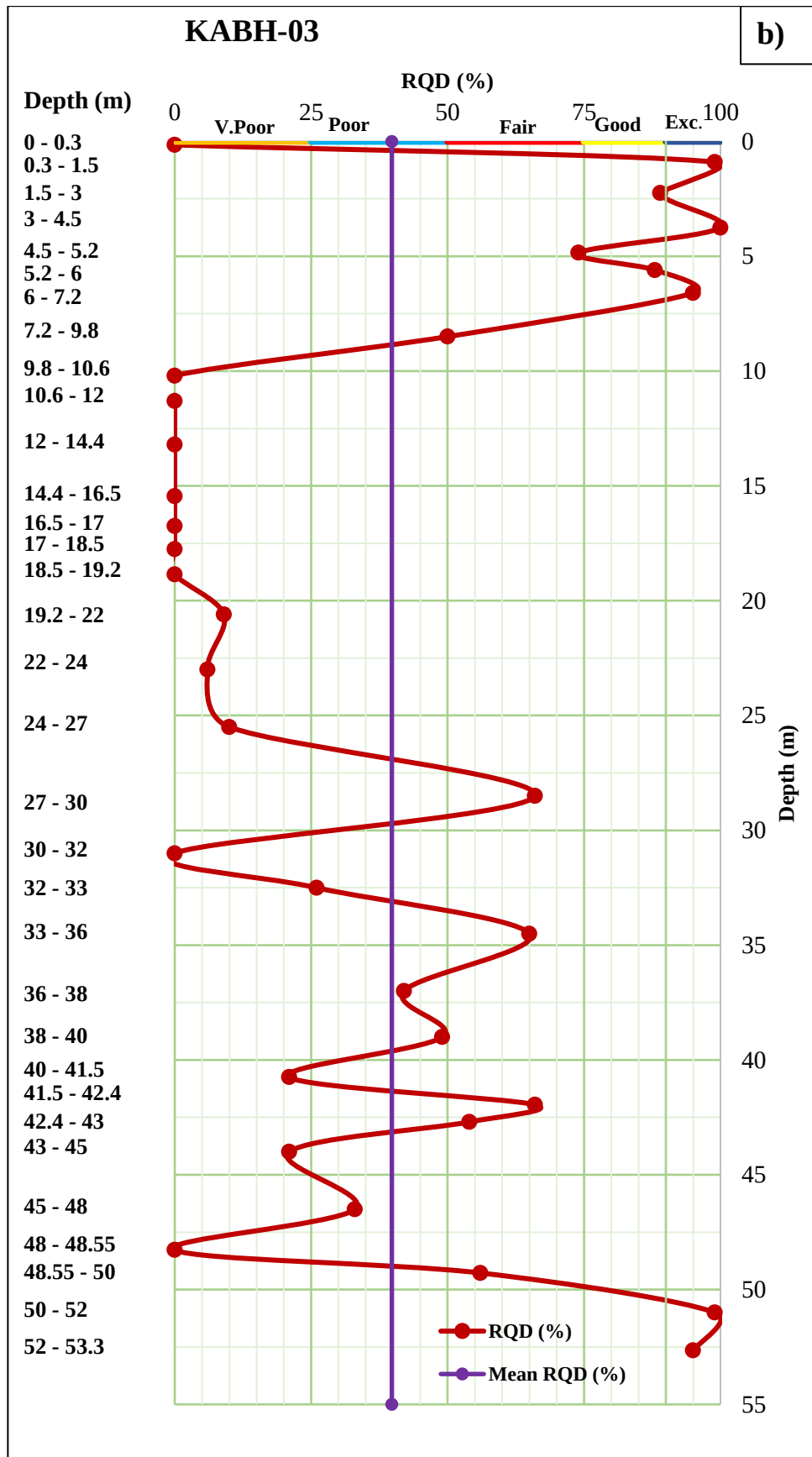
The phyric and amygdaloidal basalt rock unit at the depth range from 37.40m to 45.00m have an average RQD value of 42.17% where the rock mass quality of the rock unit was plotted to be in the poor rock mass quality class indicating that the rock unit is highly weathered and fractured rock mass according to Deere and Deere (1988). At the depth range from 37.40m to 45.00m, the phyric and amygdaloidal basalt rock unit have the minimum value of RQD of 21% and maximum value of RQD of 66% where the rock mass quality of the rock unit was plotted to be in the very poor and fair rock mass quality according to Deere and Deere (1988). The degree of jointing and weathering of the rock unit was highly weathered and fractured for the RQD of 21% and moderately weathered and fractured for the RQD of 66% according to Deere and Deere (1988). The scoriaceous or vesicular basalt rock unit at the depth range from 45.00m to 49.15m have an average RQD value of 29.67% where the rock mass quality of the rock unit was plotted to be in the poor rock mass quality class indicating that the rock unit is highly weathered and fractured rock mass according to Deere and Deere (1988). At the depth range from 45.00m to 49.15m, the scoriaceous or vesicular basalt rock unit have the minimum value of RQD of 0% and maximum value of RQD of 56% where the rock mass quality of the rock unit was plotted to be in the very poor and fair rock mass quality according to Deere and Deere (1988). The degree of jointing and weathering of the rock unit was highly weathered and fractured for the RQD of 0% and moderately weathered and fractured for the RQD of 56% according to Deere and Deere (1988).

The aphanitic basalt rock unit at the depth range from 49.15m to 53.3m have an average RQD value of 83.33% where the rock mass quality of the rock unit was plotted to be in the good rock mass quality class indicating that the rock unit is slightly weathered and fractured rock mass according to Deere and Deere (1988). At the depth range from 49.15m to 53.3m, the aphanitic basalt rock unit have the minimum value of RQD of 56% and maximum value of RQD of 99% where the rock mass quality of the rock unit was plotted to be in the fair and excellent rock mass quality according to Deere and Deere (1988). The degree of jointing and weathering of the rock unit was moderately weathered and fractured for the RQD of 56% and relatively sound to sound for the RQD of 99% according to Deere and Deere (1988). The higher RQD values indicate a reduced level of weathering and jointing, suggesting that the rock mass is composed of sound rock. These values were found to be larger than 75%. The RQD value in this borehole has a positive correlation with the depth of drilling since the degree of weathering and jointing decreases with increasing depth. Given the evident pattern of RQD values rising as depth increases, the depth at which lower RQD values occur serves as an initial indicator of the necessary level of section treatment to mitigate anticipated difficulties. In general, the majority of the rock formation in the right abutment exhibits a favourable quality of rock mass for engineering purposes.

The welded tuff rock unit in the intake conduit borehole (left side the dam site) at the depth range from 0.60m to 8.8m have an average RQD value of 53.71% where the rock mass quality of the rock unit was plotted to be in the fair rock mass quality class indicating that the rock unit is moderately weathered and fractured rock mass according to Deere and Deere (1988). At the depth range from 0.60m to 8.80m, the welded tuff rock unit have the minimum value of RQD of 35% and maximum value of RQD of 96% where the rock mass quality of the rock unit was plotted to be in the poor and excellent rock mass quality according to Deere and Deere (1988). The degree of jointing and weathering of the rock unit was highly weathered and fractured for the RQD of 35% and relatively sound to sound for the RQD of 96% according to Deere and Deere (1988). The ignimbrite rock unit in the intake conduit borehole (left side the dam site) at the depth range from 8.8m to 15.0m have an average RQD value of 48% where the rock mass quality of the rock unit was plotted to be in the poor rock mass quality class indicating that the rock unit is highly weathered and fractured rock mass according to Deere and Deere (1988). At the depth range from 8.8m to 15.0m, the ignimbrite rock unit have the minimum value of RQD of 0% and maximum value of RQD of 86% where the rock mass quality of the rock unit was plotted to be in the very poor and good rock mass

quality according to Deere and Deere (1988). The degree of jointing and weathering of the rock unit was highly weathered and fractured for the RQD of 0% and relatively sound to sound for the RQD of 86% according to Deere and Deere (1988). The RQD values of the rock units at the intake conduit borehole decreases with increases borehole depth which indicates the high degree of weathering and fracturing as shown in Figure 5.12c. This decreasing trend of RQD values could be problematic considering the poor rock mass quality for engineering applications which is due to the variation in rock types and degree of weathering and fracturing with the drilled borehole depth.





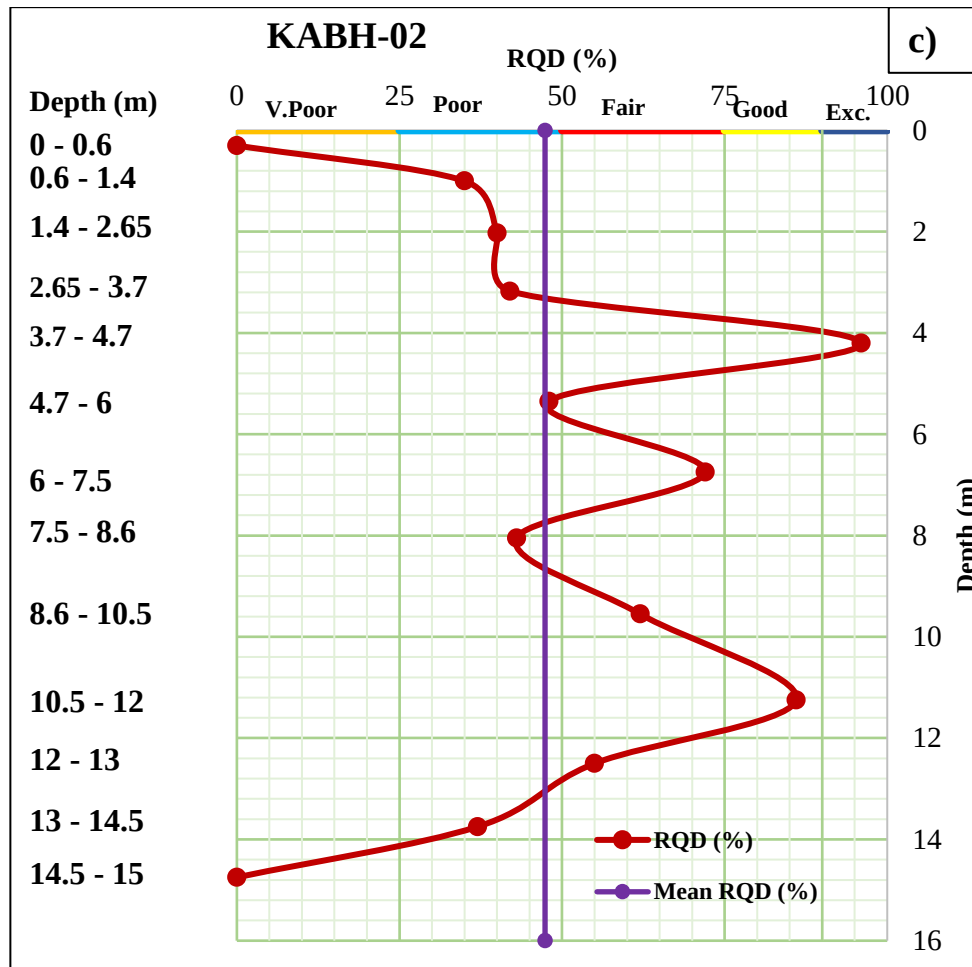


Figure 5. 11. Variation of RQD with depth at the dam site of the study area. a) left abutment (KABH-01), b) right abutment (KABH-03), and c) intake conduit borehole (KABH-02).

5.4.6. Correlation between Lu Values and Depth and RQD

The statistical relationship between the lugeon values and the drilled borehole depth was done for the dam abutments and intake conduit borehole. According to the analysis, at the left abutment borehole even if the Lu values are less than 1 it tends to decrease with increasing depth and reaches 0.034 after a particular depth as shown in Figure 5.13a and 5.13b. The decrease in Lu values with drilled depth can be related to the decrease in the degree of jointing and weathering. At the right abutment borehole, the Lu values are 12.07 at the depth range from 5.50m to 10.50m and this Lu value decreases to 9.08 Lu value at the depth range from 10.50m to 15.00m. At the intake conduit borehole, the Lu values are 2.14 at the depth range from 2.00m to 7.00m and this Lu value increases to 13.23 Lu value at the depth range from 7.00m to 12.00m. The Lu value decreases to 0.754 at the depth range from

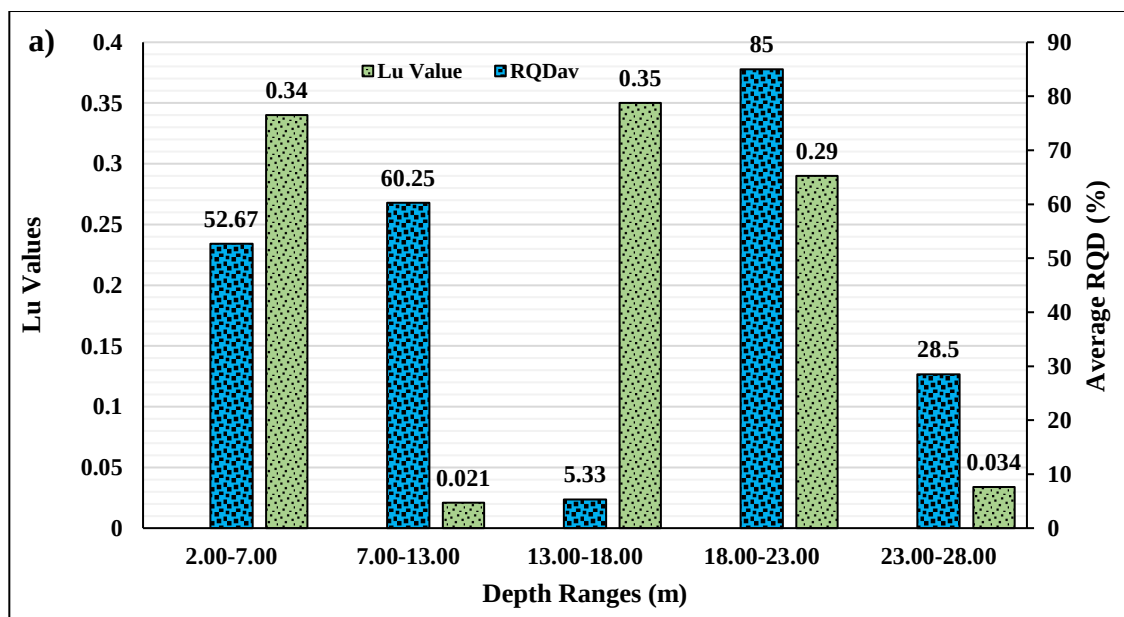
12.00m to 38.00m. The Lu value increases to 4.81 Lu value at the depth range from 38.00m to 43.00m. The Lu value decreases to 0.327 and 0.328 at the depth range from 43.00m to 48.00m and 50.30m to 53.00m, simultaneously.

Lu values and RQD can provide insight information about the rock mass quality and permeability. In some cases, a lower RQD value may be associated with the higher Lu value as more fractured rock mass can have greater permeability (higher Lu values) according to Hedayati et al. (2012) and Garo and Meten (2021). At the left abutment borehole at the depth range of 2.00m-7.00m, 7.00m-12.00m, and 18.00m-23.00m, there is a clear correlation between the Lu value and RQD with Lu value of 0.34, 0.021, and 0.29 and 52.67%, 60.25%, and 85%, respectively. Here the rock mass quality of given RQD value is plotted to be in the fair rock mass quality. With fair rock mass quality, lower Lu values are reasonable. In contrast, there is no correlation between Lu value and RQD where the Lu value is 0.35 and 0.034 at the depth range of 13.00m to 18.00m and 23.00m to 28.00m considering the RQD value of 5.33% and 28.5%, respectively. With very poor to poor rock mass quality, lower Lu values are not reasonable due to the case of lower RQD values exhibit higher Lu values.

At the right abutment borehole, Lu values show decreasing trend with increasing borehole depth at the depth range of 43.00m to 48.00m with Lu value of 0.327. At the depth range of 2.00m to 7.00m, there is a no correlation between Lu value and RQD since the higher Lu value of 2.14 have higher RQD values of 89.2% with good rock mass quality according to the Deere and Deere (1988) considering the case higher RQD values exhibit lower Lu values. At the depth range of 7.00m to 12.00m, there is a clear correlation between Lu value and RQD since the higher Lu value of 13.23 have lower RQD values 16.67% with very poor rock mass quality according to the Deere and Deere (1988) considering the case lower RQD values exhibit higher Lu values. At the depth range of 32.00m to 38.00m, there is a no clear correlation between Lu value and RQD since the lower Lu value of 0.753 have lower RQD values of 17.23% with very poor rock mass quality according to the Deere and Deere (1988) considering the case lower RQD values exhibit higher Lu values. At the depth range of 38.00m to 43.00m, there is a clear correlation between Lu value and RQD since the higher Lu value of 4.81 have lower RQD values of 46.4% with poor rock mass quality according to the Deere and Deere (1988) with considering the case lower RQD values exhibit higher Lu values. At the depth range of 43.00m to 48.00m, there is a no clear correlation between Lu value and RQD since the lower Lu value of 0.327 have lower RQD values of 27.5% with

poor rock mass quality according to the Deere and Deere (1988) with considering the case lower RQD values exhibit higher Lu values. At the depth range of 50.30m to 53.00m, there is a clear correlation between Lu value and RQD since the lower Lu value of 0.327 have higher RQD values of 95% with excellent rock mass quality according to the Deere and Deere (1988) with considering the case higher RQD values exhibit lower Lu values.

At the intake conduit borehole, at the depth range of 5.50m to 10.50m, there is unclear correlation between Lu value and RQD with Lu value of 12.07 and 64.2%, respectively. Here the Lu value is higher with fair rock mass quality and moderately weathered and fractured rock mass. This causes unclear correlation between Lu value and RQD. At the depth range of 10.50m to 15.00m, the Lu value is 9.08 and the RQD value is 44.5 with poor rock mass quality and highly weathered and fractured rock mass causing clear correlation between Lu value and RQD with Lu value. The reason of the absence for correlation between Lu value and RQD was justified by numerous studies that have shown higher RQD values does not always show lower Lu values indicating that there are some conditions where lower RQD values result in the lower Lu values (Foyo et al., 2005; Ghafoori et al., 2011; Berhane and Walraevens, 2013). This shows that the inconsistency of the correlation which reveals permeability of rock mass can also depend on the aperture and continuity of fractures, the presence of clay minerals (presence of infilling material and type) and other factors that can affect the pathways and flow rate of water within the rock mass and failure of RQD to not consider the joint continuity and presence of infilling material.



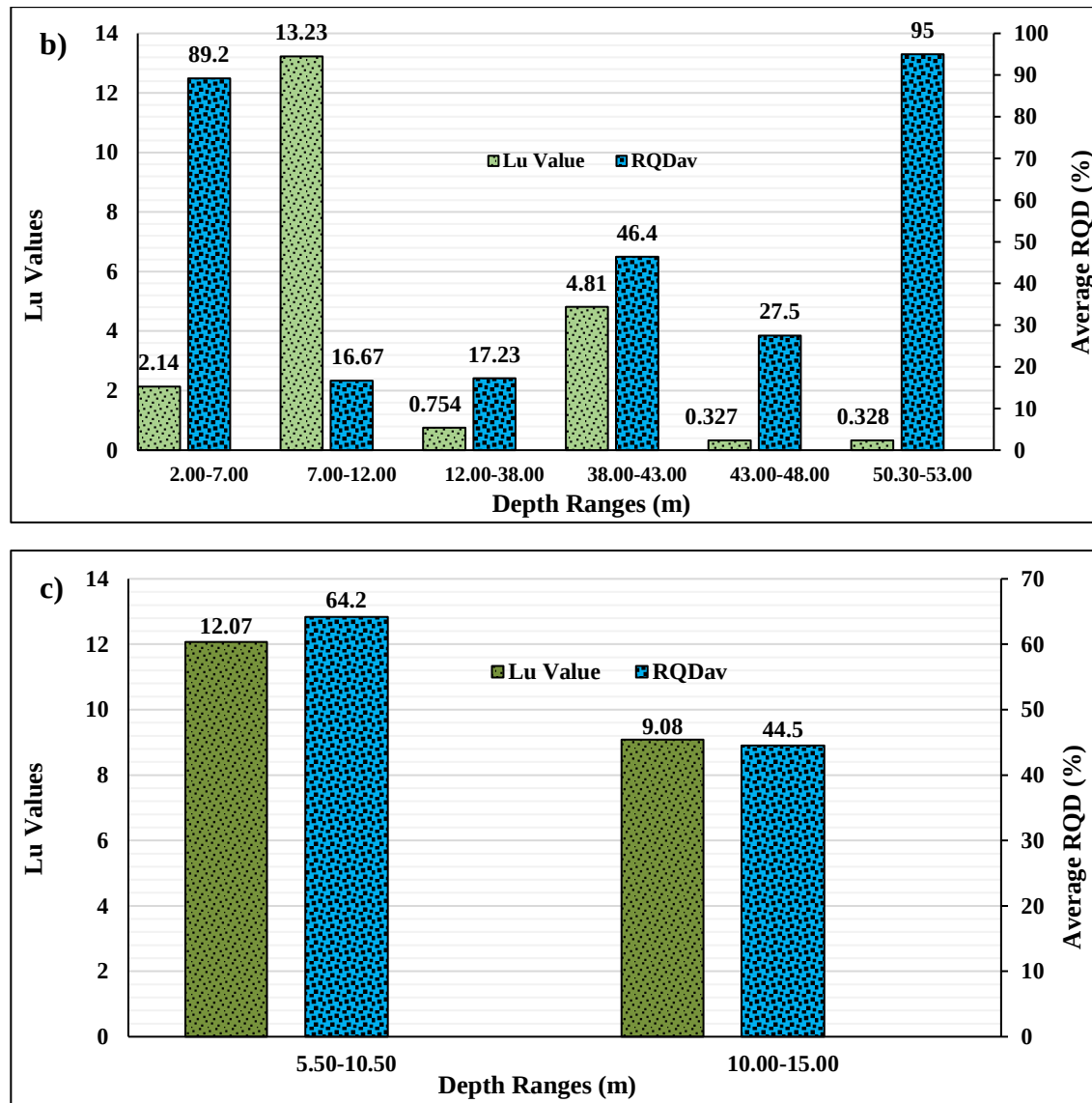


Figure 5. 12. Correlation between Lu values and RQD and depth. a) left abutment (KABH-01), b) right abutment (KABH-03), and c) Intake Conduit Borehole (KABH-02).

5.5. Bearing Capacity Analysis of Rock mass

The load-bearing capacity of the rock masses in the dam site were determined with respect to the degree of weathering and fracturing surface for surface rock mass exposures using the empirical equation suggested by Carter and Kulwahy (1992) using equation (3), Wyllie(1992) using equation (4), Bell (2007) for rock on foundation sloped surface using equation (5), Bowles (1996) using equation (1) and (2).

The load-bearing capacity of the rock mass was determined using the Carter and Kulhawy (1992) method in order to illustrate the impact of weathering conditions on the rock mass's ability to bear weight. Chala and Rao (2021) and Lamplmair et al.(2021) found that

weathering reduces rock mechanical characteristics. Weathering reduces rock bulk strength and deformation modulus. Several studies (Singh and Goel, 2011; Ramamurthy, 2004; Fookes et al., 1988; Ogunsola et al., 2017) have shown that weathered rocks have lower unconfined compressive and tensile strengths. Declining mechanical properties affect rock mass's load-bearing capacity. Lamplmair et al. (2021) discovered a high correlation between rock mechanical characteristics and bearing capacity. The rock mass's bearing capacity was calculated using RocData S, mb, and strength properties. The rock mass's bearing capacity was assessed by utilizing the rock mass strength characteristics, including S, mb, and a, generated by RocData software. The ultimate bearing capacity of the ignimbrite rock at the dam site was estimated by applying the parameters specified in the empirical equations proposed by Carter and Kulhawy (1992).

The findings indicate that the highly weathered ignimbrite rock (HWI) on the right abutment has an ultimate bearing capacity of 2.28 MPa and an allowable bearing capacity of 0.76 MPa, while the moderately weathered ignimbrite rock (MWI) on the left abutment has an ultimate bearing capacity of 8.21 MPa and an allowable bearing capacity of 2.74 MPa. The bearing capacity of the ignimbrite rocks in the left abutment is greater than that of the ignimbrite rocks in the right abutment, according to the results. Table 5.17 and Figure 5.14 and Figure 5.15 summarize the findings.

Table 5. 17. presents a concise and calculated assessment of the load-bearing capacity of the Ignimbrite rock formation at the dam site, based on the methodology outlined by Carter and Kulhawy in 1992 and Wyllie (1992).

Carter and Kulhawy (1992)							
Rock Unit Code	Input Parameters				Computed Outcome		
	UCS(MPa)	mb	S	a	qu (MPa)	FOS	Qa (Mpa)
HWI	8.97	1.655	0.0010	0.0505	2.28	3	0.76
MWI	22.87	2.178	0.0021	0.0503	8.21	3	2.74
Wyllie (1992)							
Rock Unit Code	Input Parameters				Computed Outcome		
	GSI	UCS (MPa)	mb	S	qu (Mpa)	FOS	qa (Mpa)
HWI	52.5	8.97	1.655	0.0010	2.36	3	0.79
MWI	57.5	22.87	2.178	0.0021	8.35	3	2.78
qu is the ultimate bearing capacity of the rock mass, qa is the allowable bearing capacity of the rock mass, HWI is the highly weathered ignimbrite rock unit at the right abutment, and MWI is the moderately weathered ignimbrite rock unit at the left abutment.							

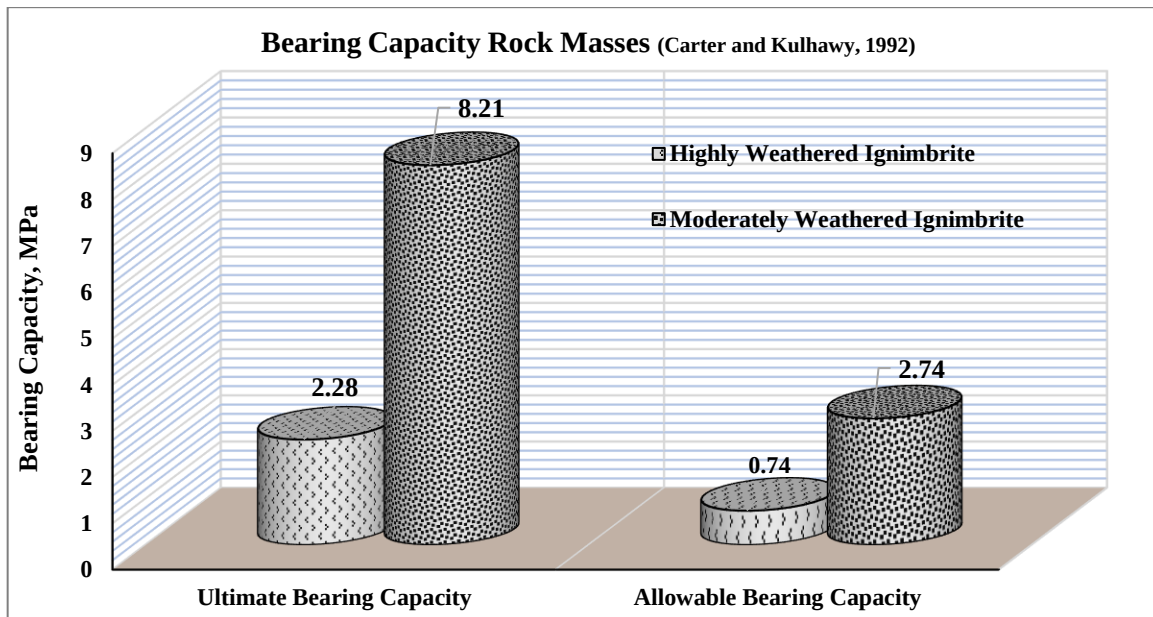


Figure 5. 13. The ultimate and allowable load-bearing capacity of Ignimbrite rock formation at the dam site (Carter and Kulhawy, 1992).

The Wyllie (1992) empirical equation with an input parameter of UCS, S, mb, Cfl1, and FOS was used to evaluate the bearing capacity of the Ignimbrite rock formation at the dam site, taking into account the fragmented rock surface. The Ignimbrite rock formation in the left and right abutment exhibits an ultimate bearing capacity of 8.35MPa and 2.36MPa, and an allowable bearing capacity of 2.78MPa and 0.79MPa, respectively. The results indicate that the ignimbrite rock on the right abutment has lower bearing capacity values compared to the ignimbrite rock on the left abutment.

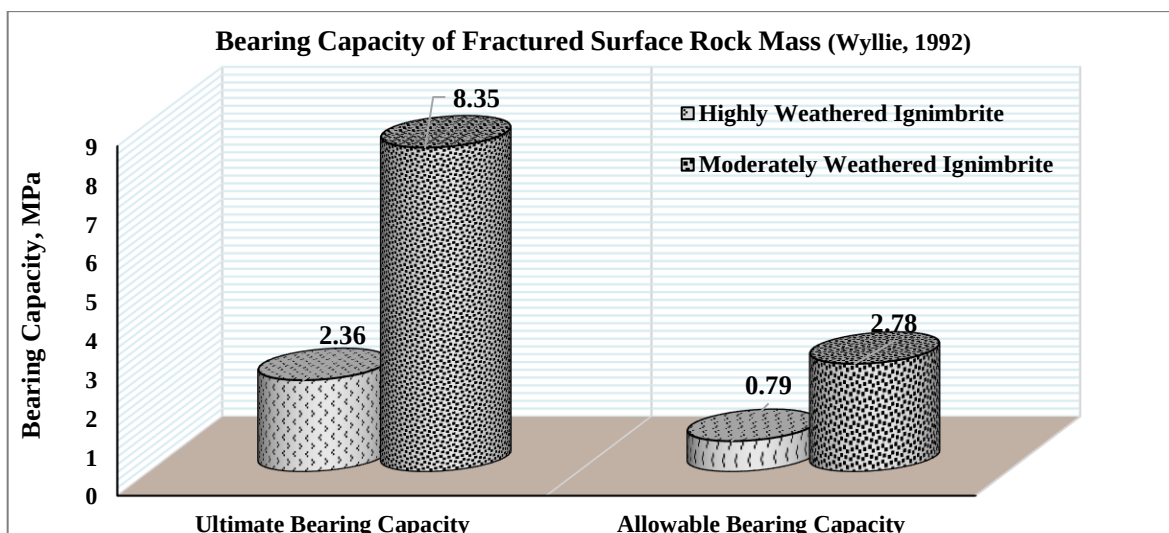


Figure 5. 14. The ultimate and allowable load-bearing capacity of Ignimbrite rock formation at the dam site (Wyllie, 1992).

The bearing capacity of ignimbrite rocks on sloped ground surfaces is primarily influenced by the slope inclination, as the lesser mass of the rock on the downstream side of the footing reduces its resistance (Wyllie, 1992). The equation proposed by Bell (2007) was utilized to ascertain the bearing capacity of the ignimbrite rocks at the dam site. The Ignimbrite rock formation that is located in the left and right abutments has a maximum bearing capacity of 7.98MPa and 3.9MPa, respectively, and an acceptable bearing capacity of 2.7MPa and 1.3MPa, respectively. On the basis of the findings, it is possible to draw the conclusion that the ignimbrite rock that is situated on the right abutment has lower bearing capacity values in comparison to the ignimbrite rock that is situated on the left abutment accordingly.

Table 5. 18. Summarized and estimated the bearing capacity of Ignimbrite rock mass at the dam site according to Bell (2007).

Rock Unit	Input Parameters									Computed Outcome	
	C (MPa)	Φ (°)	Bearing Capacity Factor			β (°)	H (m)	B (m)	γ (MN/m ³)	qu (MPa)	qa (MPa)
			No	N _{cq}	N _{γq}						
HWI	0.436	30.46	0.855	4.75	29.50	69	24	8	0.01536	3.9	1.3
MWI	1.229	32.77	0.455	4.90	24.80	64	28	8	0.01977	7.98	2.7

qu is the ultimate bearing capacity of the rock mass, qa is the allowable bearing capacity of the rock mass, HWI is the highly weathered ignimbrite rock unit at the right abutment, and MWI is the moderately weathered ignimbrite rock unit at the left abutment.

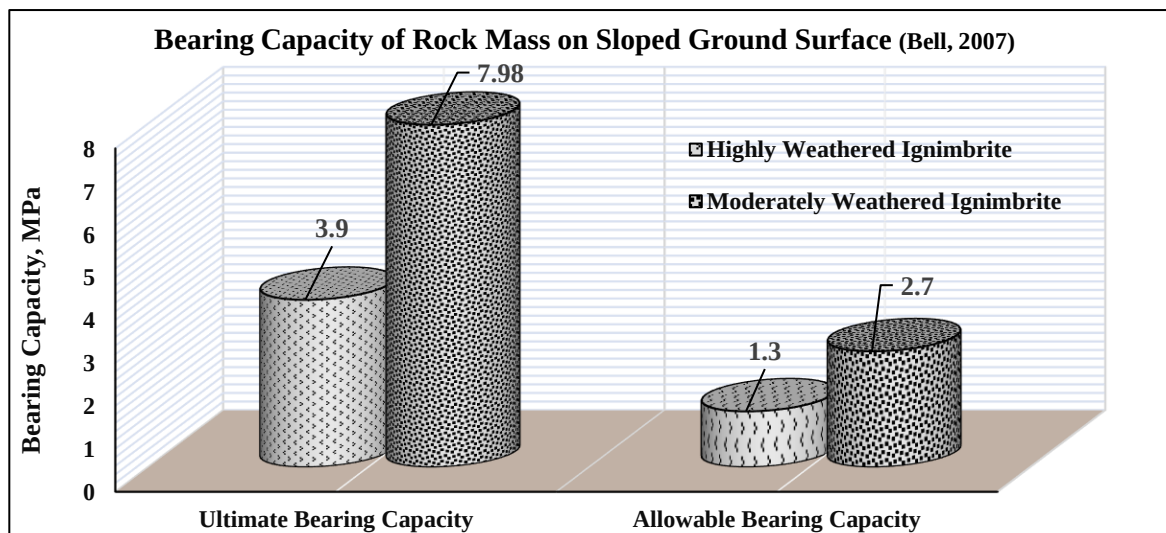


Figure 5. 15. The ultimate and allowable load-bearing capacity of Ignimbrite rock formation at the dam site (Bell, 2007).

The Bowles (1996) equation was used to assess the bearing capacity of the rock mass in the river center and both abutments borehole units. This equation considers the RDQ and the uniaxial compressive strength of the rocks, taking into consideration the depth and lithological unit. Within the borehole located at the left abutment of the dam site, seven geotechnical units (LAW, LAI, LAuW, LAW, LAI, LAW, and LAVB) were distinguished based on the type of rock, variations in RQD, UCS values, and the depth of the boreholes. Within the intake conduit borehole at the dam site, two geotechnical units (IBW and IBI) were distinguished based on the rock type, variations in RQD (Rock Quality Designation), UCS (Unconfined Compressive Strength) values, and the depth of the boreholes.

Within the borehole located at the right abutment of the dam site, nine geotechnical units (RAW, RAI, RAuW, RAW, RAuW, RAW, RAAMB, RAVB, RAAPB) were distinguished based on the kind of rock, variations in RQD (Rock Quality Designation), UCS (Unconfined Compressive Strength) values, and the depth of the boreholes. Consequently, the ultimate and allowable load-bearing capacity of the rock mass in the boreholes was established.

At the left abutment borehole, the welded tuff rock unit at the depth range of 1.50m to 3.45m, 17.0m to 19.20m, and 24.3m to 27.00m have the ultimate bearing capacity and allowable bearing capacity of 0.39MPa, 3.54MPa, and 1.1278MPa and 0.13MPa, 1.18MPa, and 0.37MPa, respectively. The unwelded tuff rock unit at the depth range of 11.60m to 17.0m have the ultimate bearing capacity and allowable bearing capacity of 0.07MPa and 0.02MPa, respectively. The ignimbrite rock unit at the depth range of 3.45m to 11.60m and 19.20m to 24.30m have the ultimate bearing capacity and allowable bearing capacity of 6.34MPa and 15.14MPa and 2.11MPa and 5.05MPa, respectively. The vesicular basalt rock unit at the depth range of 27.0m to 66.0m have the ultimate bearing capacity and allowable bearing capacity of 0.15MPa and 0.05MPa, respectively.

The Intake conduit borehole, the welded tuff rock unit at the depth range of 0.60m to 8.80m have the ultimate bearing capacity and allowable bearing capacity of 1.08MPa and 0.36MPa, respectively. The ignimbrite rock unit at the depth range of 8.80m to 15.0m have the ultimate bearing capacity and allowable bearing capacity of 6.9MPa and 2.30MPa, respectively.

At the right abutment borehole, the welded tuff rock unit at the depth range of 0.30m to 1.50m, 26.65m to 29.5m, and 32.0 to 37.4m have the ultimate bearing capacity and allowable bearing capacity of 3.49MPa, 5.158MPa, and 1.04MPa and 1.16MPa, 1.72MPa, and

0.35MPa, respectively. The unwelded tuff rock unit at the depth range of 17.45m to 20.3m, 29.5 to 32.0m have the ultimate bearing capacity and allowable bearing capacity 0.017MPa, 2.09MPa, and 0.0057MPa, 0.69MPa, respectively. The Amygdaloidal basaltic rock unit at the depth range of 37.40m to 45.0m has the ultimate bearing capacity and allowable bearing capacity of 1.64MPa, and 0.55MPa, respectively. The vesicular basaltic rock unit at the depth range of 45.0m to 49.15m has the ultimate bearing capacity and allowable bearing capacity of 1.74MPa and 0.58Mpa, respectively. The aphanitic basaltic rock unit at the depth range of 49.15m to 53.30m has the ultimate bearing capacity and allowable bearing capacity of 48.34MPa, and 16.11MPa, respectively as shown in Table 5.19 and Figure 5.17. The bearing capacity of the rock units in the left abutment borehole at the dam site is generally lower than the bearing capacity of the rock units in the right abutment borehole at the dam site, as indicated by The Bowles (1996) equation.

Table 5. 19. presents a concise overview of the ultimate and allowable load-bearing capacity of the rock formations at the location of the dam, determined by rock unit, RQD, UCS values, and depth.

S. No	Depth (m)	Rock Unit Code	RQD		UCS _{av}	qu (MPa)	qa (MPa)	Rock Unit Description
			RQD _{av}	Decimal				
1.	1.50-3.45	LAW	33	0.33	3.66	0.39	0.13	Welded Tuff (W)
2.	3.45-11.6	LAI	70.6	0.706	12.72	6.34	2.11	Ignimbrite (I)
3.	11.60-17.0	LAuW	7.25	0.0725	13.485	0.07	0.02	Unwelded Tuff (uW)
4.	17.0-19.2	LAW	47.5	0.475	15.68	3.54	1.18	Welded Tuff (W)
5.	19.20-24.3	LAI	85	0.85	20.95	15.14	5.05	Ignimbrite (I)
6.	24.3-27	LAW	45	0.45	5.52	1.1278	0.37	Welded Tuff (W)
7.	27.0-66.0	LAVB	6.61	0.0661	34.71	0.15	0.05	Vesicular Basalt (VB)
8.	0.60-8.80	IBW	53.71	0.5371	3.75	1.08	0.36	Welded Tuff (W)
9.	8.80-15.0	IBI	48	0.48	30	6.9	2.3	Ignimbrite (I)
10.	0.3-1.50	RAW	99	0.99	3.56	3.49	1.16	Welded Tuff (W)
11.	1.50-9.40	RAI	82.67	0.8267	46.32	31.66	10.55	Ignimbrite (I)
12.	17.45-20.3	RAuW	3	0.03	19.2	0.017	0.0057	Unwelded Tuff (uW)
13.	26.65-29.5	RAW	66	0.66	11.82	5.158	1.72	Welded Tuff (W)
14.	29.5-32	RAuW	33	0.33	19.2	2.09	0.69	Unwelded Tuff (uW)
15.	32.0-37.40	RAW	44.33	0.4433	5.29	1.04	0.35	Welded Tuff (W)
16.	37.40-45.0	RAAmB	42.17	0.4217	9.215	1.64	0.55	Amygdaloidal Basalt (AmB)
17.	45.0-49.15	RAVB	29.67	0.2967	19.8	1.74	0.58	Vesicular Basalt (VB)
18.	49.15-53.3	RAApB	83.33	0.8333	69.62	48.34	16.11	Aphanitic Basalt (ApB)

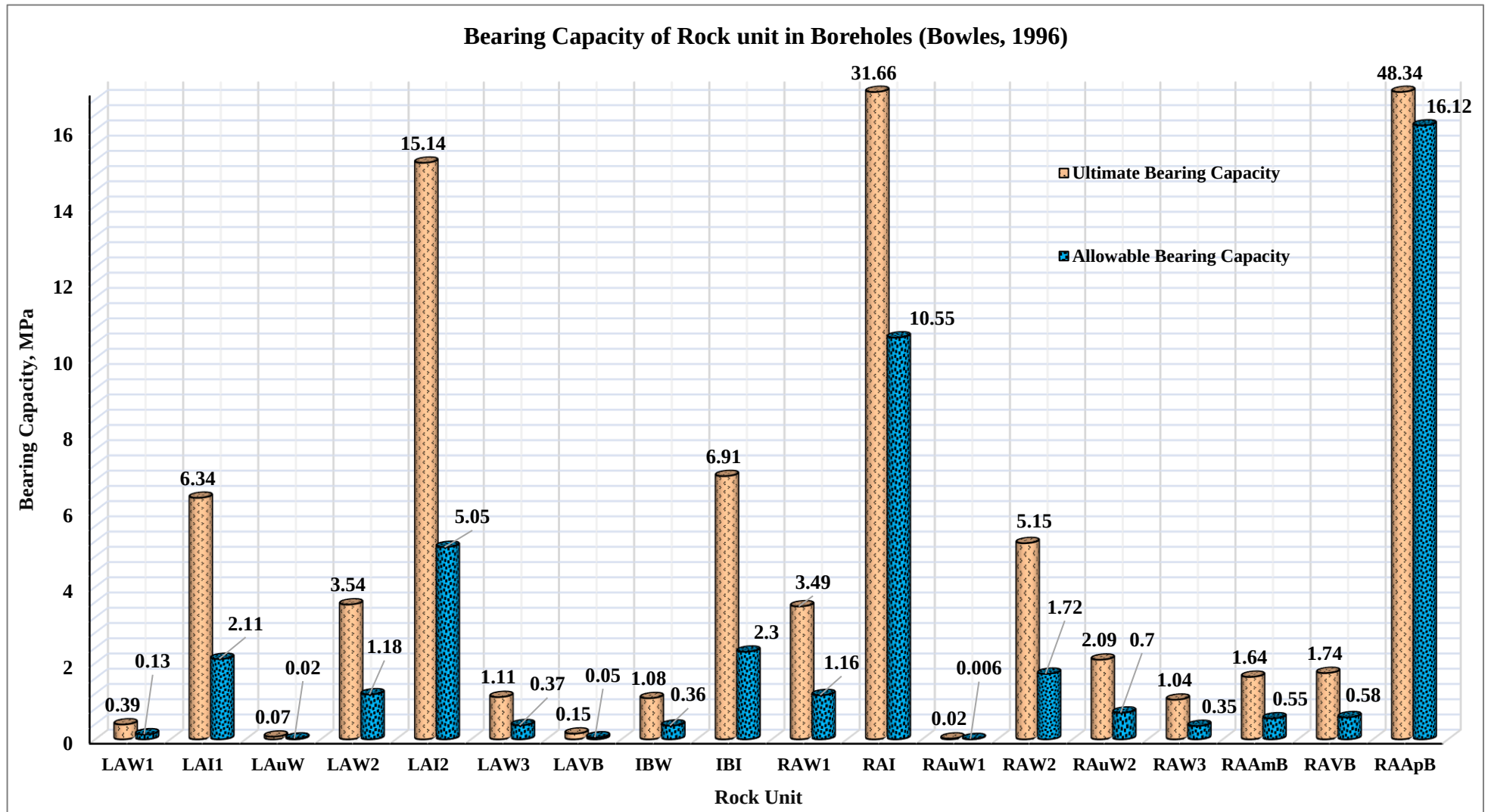


Figure 5. 16. The ultimate and allowable bearing capacity of rock units throughout the borehole at the dam site according to Bowles (1996).

5.6. Engineering Geological Classification and Characterization of Soils

5.6.1. Natural Moisture Content

Some soils especially fine-grained soils show a significant change in their property with change in their water content (Arora, 2004). In view of this, the natural moisture content of soils that constitute the proposed foundation of the dam axis and parts of the reservoir area was determined. The test was carried out in accordance with ASTM D 2216 (1998). The obtained result indicates that the natural moisture content of the soils was 35% around the dam axis (Appendix 5).

5.6.2. Specific Gravity

Specific gravity is an important parameter of soils as it is used for the determination of void ratio and particle size (Arora, 2004). Therefore, in this study the specific gravity of soils was determined according to the ASTM D 854 (1998) standard for each sample. The analysis result showed that the average specific gravity of soils of the study area is 2.57 (Appendix 6). In addition to this, Engineering Corporation of Oromia (ECO, 2023) have also conducted the laboratory analysis of determining the specific gravity of soil samples from both left and right abutment. The specific gravity of the soil samples from the left abutment of the dam site ranges from 2.5 to 2.632, from right abutment of the dam site ranges from 2.44 to 2.632, from the reservoir area the specific gravity of 2.38 (as given in Table 5.21).

5.6.3. Grain Size Analysis

Grain size analysis was undertaken for each soil sample to identify the type of soils which constitute the study area. The analysis was done according to the standard ASTM D 422 (1998). From the analysis result, particle size distribution curve (percent finer versus particle diameter) was plotted (Appendix 7) for each soil sample and the corresponding percentage of soil particles are computed from the curve. Moreover, the soil samples in the dam site area were studied in different localities by ECO (2023) to analyze and investigation site suitability. From sieve analysis results the soil samples revealed 3.11% Gravel, 9.43% sand and 87.46% silt and clay at the dam axis as given in Figure 5.18.

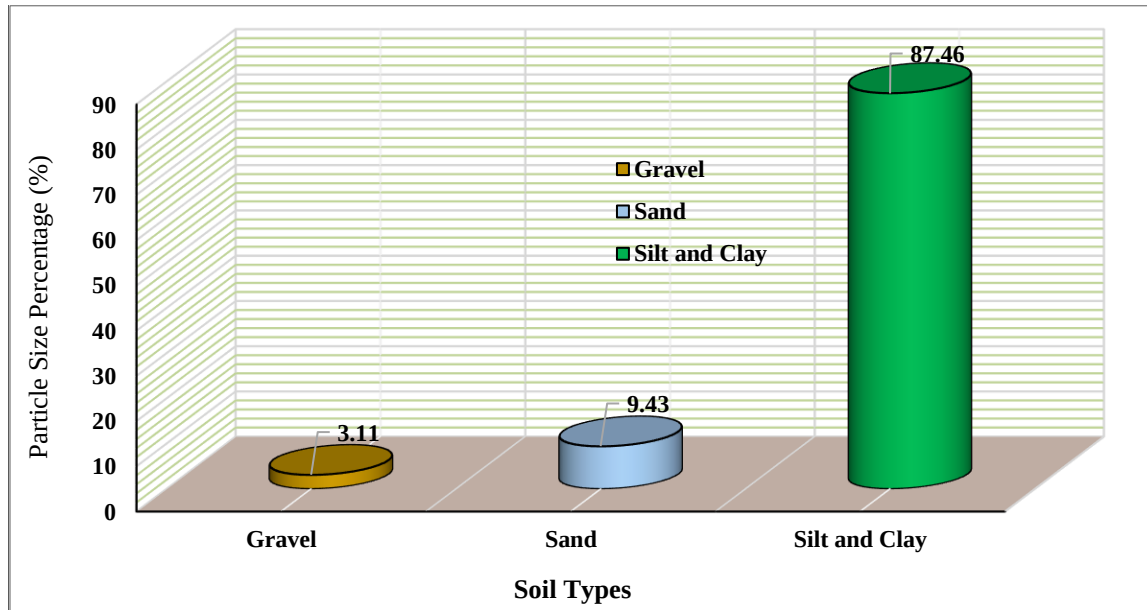


Figure 5. 17.. Grain Size distribution of soil samples.

In addition to this, Engineering Corporation of Oromia (ECO, 2023) have also conducted the laboratory analysis of determining the grain size distribution of soil samples from both left and right abutment. From grain size analysis results the soil samples grain size distribution was analyzed in terms of gravel percentage, sand percentage, silt percentage and clay percentage. From the grain size distribution test, all soil samples from the left abutment have 0% of gravel size soil. Soil samples from left abutment at the depth of 0.0m to 0.50m have sand particle of 10%, at the depth of 0.0m to 1.0m have sand particle distribution 15%, at the depth of 0.70m to 1.50m have sand particle distribution of 41%, at the depth of 0.30m to 1.0m have sand particle distribution of 33%, at the depth of 0.30m to 1.50m have sand particle distribution of 11%, at the depth of 0.30m to 1.50m have sand particle distribution of 22%. Soil samples from left abutment at the depth of 0.0m to 0.50m have silt particle distribution of 33%, at the depth of 0.0m to 1.0m have silt particle distribution 36%, at the depth of 0.70m to 1.50m have silt particle distribution of 34%, at the depth of 0.30m to 1.0m have silt particle distribution of 45%, at the depth of 0.30m to 1.50m have silt particle distribution of 53%, at the depth of 0.30m to 1.50m have silt particle distribution of 50%. Soil samples from left abutment at the depth of 0.0m to 0.50m have clay particle distribution of 57%, at the depth of 0.0m to 1.0m have clay particle distribution 49%, at the depth of 0.70m to 1.50m have clay particle distribution of 15%, at the depth of 0.30m to 1.0m have clay particle distribution of 22%, at the depth of 0.30m to 1.50m have clay particle distribution of 36%, at the depth of 0.30m to 1.50m have sand particle distribution of 28% (as given in Table 5.21). The grain size distribution analysis of the soil samples taken at depths

ranging from 0.0m to 0.50m and 0.0m to 1.0m showed the presence of silt clay soil. The grain size distribution analysis of the soil samples taken at depths of 0.70m to 1.50m indicated the soil is silt sandy, 0.30m to 1.50m indicated the soil is sand silty, 0.30m to 1.50m, and 0.30m to 1.50m revealed that the soil is composed of clay silty.

From the grain size distribution test, all soil samples from the right abutment have 0% of gravel size soil. Soil samples from right abutment at the depth of 0.0m to 1.50m have sand particle of 12%, at the depth of 0.30m to 1.50m have sand particle distribution 47%, at the depth of 0.30m to 1.50m have sand particle distribution of 15%, at the depth of 0.30m to 1.50m have sand particle distribution of 47%, at the depth of 0.30m to 1.50m have sand particle distribution of 39%. Soil samples from right abutment at the depth of 0.0m to 1.50m have silt particle of 43%, at the depth of 0.30m to 1.50m have silt particle distribution of 37%, at the depth of 0.30m to 1.50m have silt particle distribution of 51%, at the depth of 0.30m to 1.50m have sand particle distribution of 30%, at the depth of 0.30m to 1.50m have sand particle distribution of 37%. Soil samples from right abutment at the depth of 0.0m to 1.50m have clay particle distribution of 45%, at the depth of 0.30m to 1.50m have clay particle distribution of 16%, at the depth of 0.30m to 1.50m have clay particle distribution of 34%, at the depth of 0.30m to 1.50m have sand particle distribution of 23%, at the depth of 0.30m to 1.50m have sand particle distribution of 24%, (as given in Table 5.21). The grain size distribution analysis of the soil samples taken at depths ranging from 0.0m to 1.50m and 0.30m to 1.50m showed the presence of clay-silty soil. The grain size distribution analysis of the soil samples taken at depths of 0.30m to 1.50m, 0.30m to 1.50m, and 0.30m to 1.50m revealed that the soil is composed of silty sand.

From the grain size distribution test, soil samples from the reservoir area at the depth of 0.0m to 0.50m have 0% of gravel size soil, sand particle distribution of 40%, silt particle distribution of 34%, and clay particle distribution of 26%.

5.6.4. Atterberg Limits

The consistency of soil is largely influenced by the moisture content in it. At very high moisture content, soil behaves more like a liquid state and as the moisture content gradually decreases in the soil, the soil behavior changes from one state to the other state. Atterberg limit tests such as liquid limit and plastic limit tests were conducted. In the present study, the representative soil samples were collected from test pits dug at the dam site. The results of the Atterberg limit test are presented in Appendix 8 and table 5.7. According to the result

obtained from the liquid and plastic limit of soils in the study area is 53.74% and 29.5% respectively, which indicated high plasticity soils (Ramana, 1993) and the plasticity index (PI) value is 24.24% showing a high plastic soil.

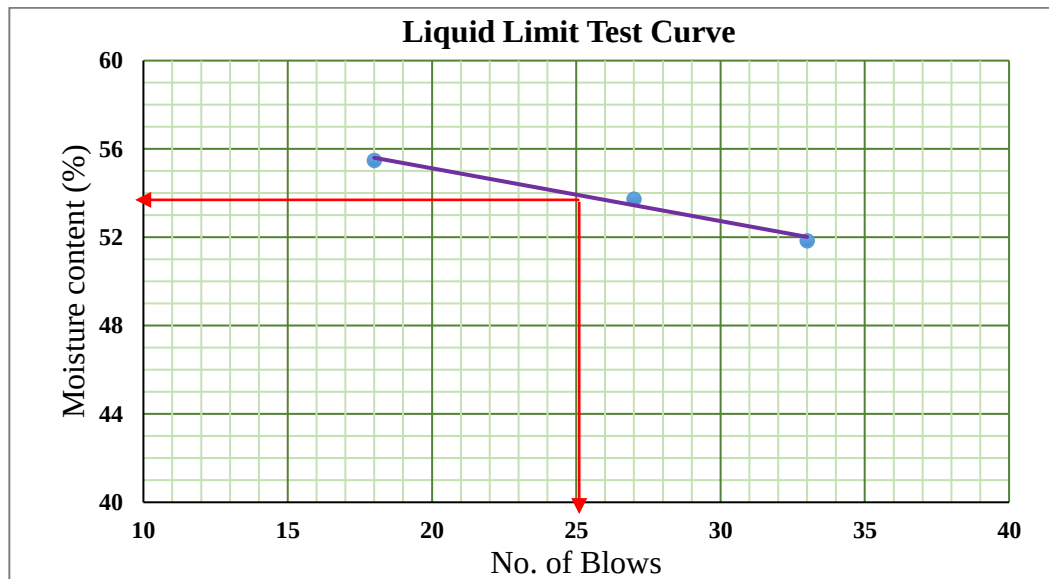


Figure 5. 18. Atterberg limit curve of soil samples.

Furthermore, Engineering Corporation of Oromia (ECO, 2023) has performed laboratory study to determine the consistency limitations (Atterberg limits) of soil samples from both the left and right abutment. These limits include the liquid limit (LL), plastic limit (PL), and plasticity index (PI). The soil samples taken from the left abutment at depths ranging from 0.0m to 0.50m have a liquid limit of 62.7%, a plastic limit of 33.9%, and a plasticity index of 28.8% indicating a high plasticity of the soil (Ramana, 1993). The liquid limit value for the depth range of 0.0m to 1.0m is 53.4%, the plastic limit value is 31.3%, and the plasticity index value is 22.1% indicating a high plasticity of the soil (Ramana, 1993). The liquid limit value of the soil at a depth of 0.70m to 1.50m is 45.3%, the plastic limit value is 38.9%, and the plasticity index value is 6.4% indicating a low plasticity of the soil (Ramana, 1993). The soil layers at depths ranging from 0.30m to 1.50m have a liquid limit value of 50.6%, a plastic limit value of 32.8%, and a plasticity index value of 24.9% indicating a high plasticity of the soil (Ramana, 1993). Similarly, at the same depths, the soil layers have a liquid limit value of 65.9%, a plastic limit value of 34.1%, and a plasticity index value of 31.9% indicating a high plasticity of the soil (as given in Table 5. 21).

The soil samples taken from the right abutment at depths ranging from 0.0m to 1.50m and 0.30m to 1.50m have a liquid limit of 53.1%, a plastic limit of 32%, and a plasticity index of

19% and have a liquid limit of 52.3%, a plastic limit of 29.2%, and a plasticity index of 23% indicating a high plasticity of the soil (Ramana, 1993).

5.6.5. Compaction Test Analysis Results

The results of the compaction test carried out on the samples from the dam site. For these tests, as indicated in the table, the Optimum Moisture Content was 24.5% while the Maximum Dry Density was 1.405 g/cm³ (Appendix 9). Generally, the soil with low activity is compacted to the high density at lower moisture content.

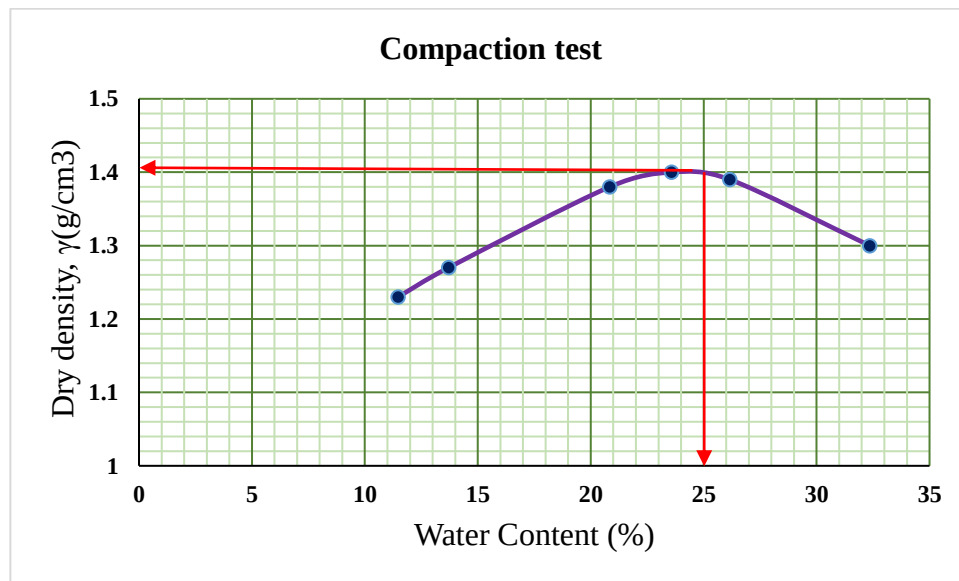


Figure 5. 19. Maximum dry density of the soil samples.

5.6.6. Permeability of Soil Samples

Water can flow through interconnected voids of soil from points of high energy to the points of low energy (Das, 2012). This property of soil which allows passage of water through it is called permeability (Arora, 2004). In the construction of water retaining structures such as dams, the permeability of soil is a very important engineering property to determine the rate leakage below dam foundation and within the reservoir area. Having this in context, the permeability test was conducted as a representative sample from the foundation of the dam axis. The analysis result showed the foundation soil has a coefficient of permeability in the order of 1.53×10^{-6} cm/sec. According to Bell (2007), the soil permeability class was very low permeability class. The soils of the site contain a significant amount of fine fraction (silt and clay particle sizes). This accounts for the very low permeability of the materials. So that it can be generally stated that the permeability of the soils at the Katar reservoir site is low which is enough to retain water in the reservoir after impoundment.

In addition to this, Engineering Corporation of Oromia (ECO, 2023) have also conducted the laboratory analysis of determining the permeability of soil samples from both left and right abutments, and also from the reservoir. Four soil samples, six soil samples, and one soil samples were taken to laboratory for further analysis which were taken at different depth from the dam axis and reservoir.

The black cotton silty clay soil samples from the right abutment of the dam site at the depth range from 0.50m to 1.50m have hydraulic conductivity/permeability value of 2.92×10^{-5} cm/sec indicating a very low degree of permeability according to Bell (2007) as shown in Table 5.20. The silty sandy residual soil samples taken at the depth range from 0.30m to 1.50m have a permeability value of 1.29×10^{-3} cm/sec showing a low degree of permeability according to Bell (2007). The black cotton silty clay soil samples taken at the depth range from 0.0m to 0.50m have a permeability value of 2.56×10^{-6} cm/sec showing an impermeable degree of permeability according to Bell (2007) as shown in Table 5.20. The sand dominant with gravel soil samples taken at the depth range from 0.30m to 1.50m have a permeability value of 3.46×10^{-3} cm/sec showing a low degree of permeability according to Bell (2007) as shown in Table 5.20.

The organic soil, silty dominant with trace clay soil sample taken from left abutment of the dam axis at depth range from 0.00m to 0.50m have a permeability value of 6.3×10^{-3} cm/sec indicating a low degree of permeability according to Bell (2007) as shown in Table 5.20. The black cotton soil, silty clay soil sample taken at depth range from 0.00m to 1.00m have a permeability value of 4.43×10^{-5} cm/sec indicating a very low degree of permeability according to Bell (2007) as shown in Table 5.20. The silt sandy with trace gravel soil sample taken at depth range from 0.70m to 1.50m have a permeability value of 2.77×10^{-3} cm/sec indicating a low degree of permeability according to Bell (2007) as shown in Table 5.20. The sandy dominant with gravel residual soil sample taken at depth range from 0.30m to 1.50m have a permeability value of 1.85×10^{-3} cm/sec indicating a low degree of permeability according to Bell (2007) as shown in Table 5.20. The organic soil, clay dominant soil sample taken at depth range from 0.0m to 0.50m have a permeability value of 2.55×10^{-5} cm/sec indicating a very low degree of permeability according to Bell (2007) as shown in Table 5.20. The clay dominant residual soil sample taken at depth range from 0.30m to 1.50m have a permeability value of 2.38×10^{-5} cm/sec indicating a very low degree of permeability according to Bell (2007) as shown in Table 5.20.

The silty sandy residual soil sample taken from the reservoir area at depth range from 0.00m to 0.50m have a permeability value of 3.85×10^{-3} cm/sec indicating a low degree of permeability according to Bell (2007) as shown in Table 5.20.

Table 5. 20. Summarized results of permeability analysis of soils samples from the dam axis and reservoir area.

ID	Depth (m)	Permeability (K) (cm/sec)	Permeability Class (Bell, 2007)	Type of Soil
KADA-01 Right L-1 OP-2	0.50-1.50	2.92×10^{-5}	Very low	Black cotton soil, silty clay
KADA-01 L-2 Right OP-2	0.30-1.50	1.29×10^{-3}	Low	Silty sandy residual soil
KADA-01 (L-1) Right op-3	0.00-0.50	2.56×10^{-6}	Impermeable	Black cotton silty clay
KADA-02 L-2 Right OP-2	0.30-1.50	3.46×10^{-3}	Low	Sandy dominant with gravel
KADA-01/Left/L-1/OP-4	0.00-0.50	6.3×10^{-3}	Low	Organic soil, silty dominant with trace clay
KADA-02 Left (L-1); OP-3	0.00-1.00	4.43×10^{-5}	Very Low	Black cotton, silt clay
KADA-02/Left (L-2); OP-3	0.70-1.50	2.77×10^{-3}	Low	Silt Sandy with trace gravel
KADA-02 Left (L-2); OP-4	0.30-1.50	1.85×10^{-3}	Low	Sandy dominant with gravel residual soil
KADA-03 (L-1) Left; OP-2	0.00-0.50	2.55×10^{-5}	Very Low	Organic, clay dominant
KADA-03- (L-2) Left; OP-2	0.30-1.50	2.38×10^{-5}	Very Low	Clay dominant residual soil
KAR-01 (L-1) Right	0.00-0.50	3.85×10^{-3}	Low	Silty sandy /Residual soil
Where KA-Katar, DA-Dam axis, L-Layer, OP- Dam option, R-reservoir.				

Table 5. 21. Summarized laboratory results in terms of specific gravity, grain size distribution, Atterberg limits, and activity of the soil samples.

Soil Sample Id	Depth (m)	Gs	Gr	Sand (%)	Silt (%)	Clay (%)	LL (%)	PL (%)	PI (%)	Plasticity	Activity	Type Of Soil
KADA -01 Right L-1 op-3 (Black cotton)	0.00-1.50	2.632	0	12	43	45	53.1	34.2	19	H-plastic	Inactive (0.42)	Clay Silty
KADA -01 Right L-2 op-2 SAWG	0.30-1.50	2.5	0	47	37	16	NP	NP	NP	NP	Inactive	Silt Sandy
KADA -01 Right L-1 op-2 (SILT SANDY)	0.30-1.50	2.5	0	15	51	34	52.5	29.2	23	H-plastic	Inactive (0.68)	Clay Silty
KADA -01 Right L-2 op-4	0.30-1.50	2.44	0	47	30	23	NP	NP	NP	NP	Inactive	Silt Sandy
KADA -02 Right L-2 op-2 (SILT SAND)	0.30-1.50	2.564	0	39	37	24	NP	NP	NP	N-plastic	Inactive	Silt Sandy
KAR -01 Right L-1 (SILT SANDY)	0.00-0.50	2.38	0	40	34	26	NP	NP	NP	NP	Inactive	Silt Sandy
KADA -01 Left L-1 op-4 (Organic soil)	0.00-0.50	2.564	0	10	33	57	62.7	33.9	28.8	H-plastic	Inactive (0.51)	Silt Clay
KADA -02 Left L-1 op-3 (Black cotton)	0.00-1.00	2.564	0	15	36	49	53.4	31.3	22.1	H-plastic	Inactive (0.45)	Silt Clay
KADA -02 Left L-2 op-3 (SILT SANDY)	0.70-1.50	2.564	0	41	34	15	45.3	38.9	6.4	L-plastic	Inactive (0.43)	Silt Sandy
KADA -02 Left L-2 op-4 (SANDWG)	0.30-1.00	2.5	0	33	45	22	NP	NP	NP	N-plastic	Inactive	Sand Silty
KADA -03 Left L-1 op-2 (Black cotton)	0.30-1.50	2.632	0	11	53	36	57.6	32.8	24.9/KA	H-plastic	Inactive (0.69)	Clay Silty
KADA -03 Left L-2 op-2 (Black cotton)	0.30-1.50	2.56	0	22	50	28	65.9	34.1	31.9/KA	H-plastic	Normal (1.4)	Clay Silty
(KA-Katar, DA-Dam axis, L-Layer, OP- Dam option, R-reservoir, NP- Nonplastic, LP-Low plastic, MP- Medium plastic, HP-Highly plastic												

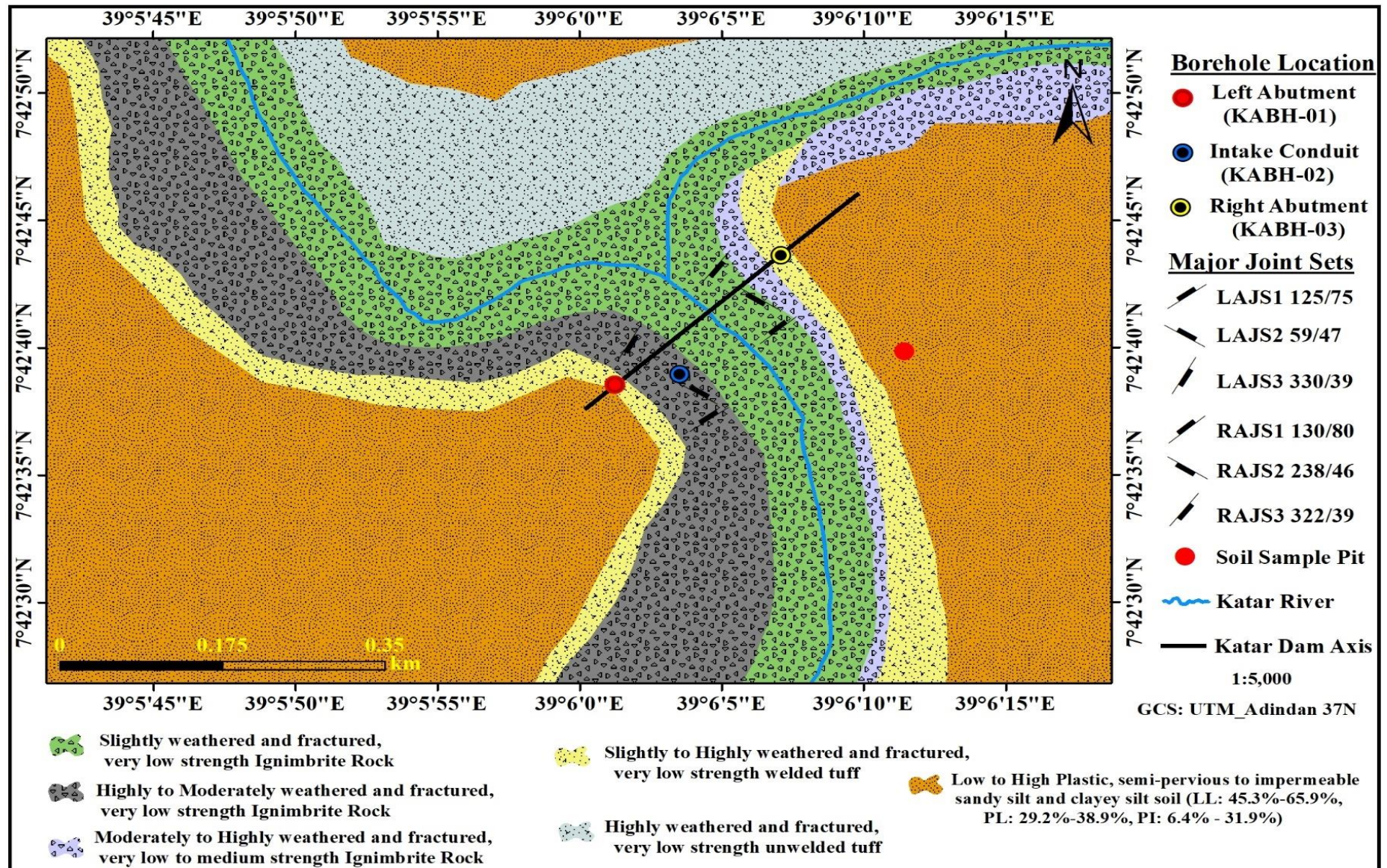


Figure 5. 20. Engineering geological map of the Katar dam site and its surroundings at a scale of 1: 5,000.

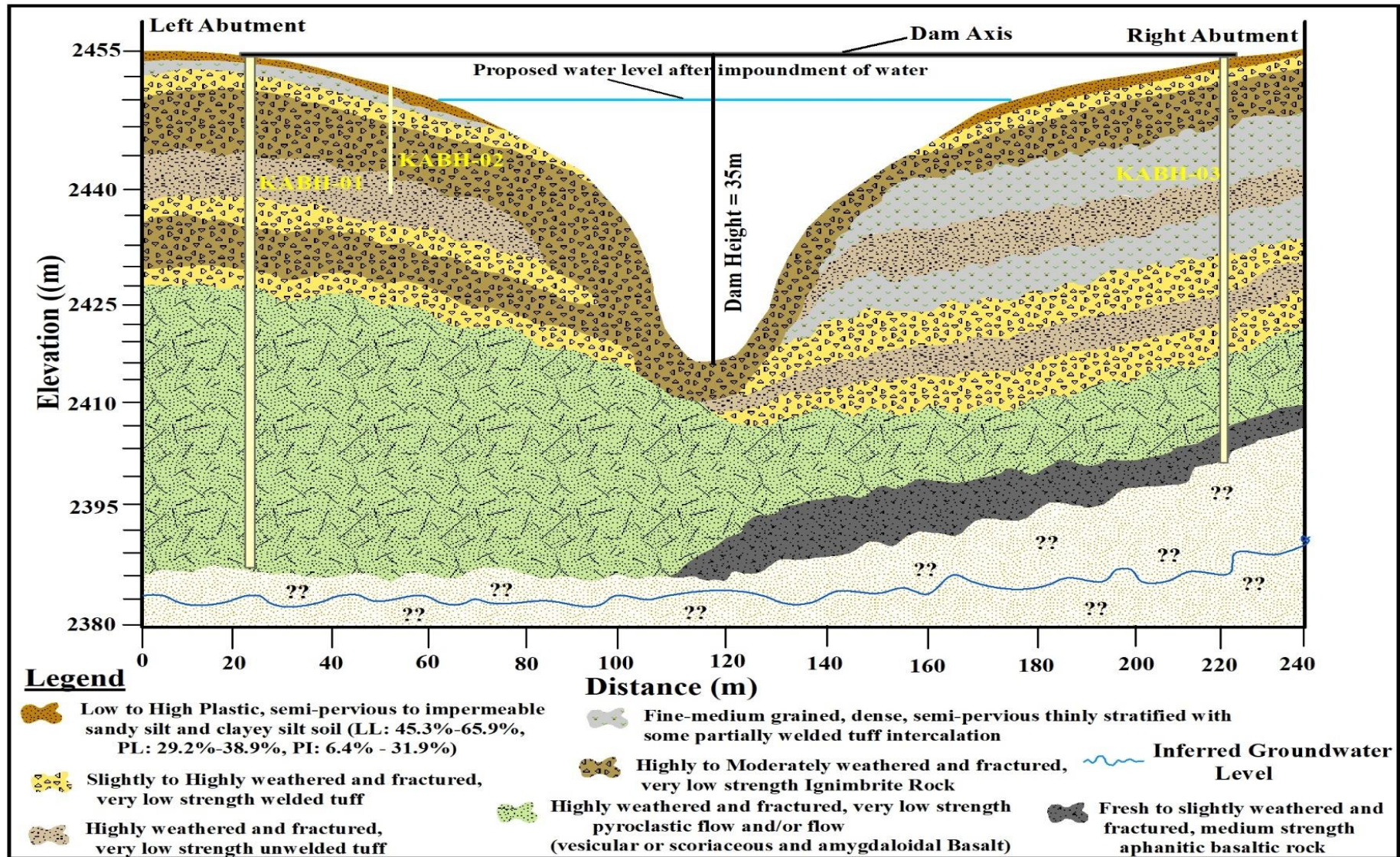


Figure 5. 21. Engineering geological cross-section along the dam axis (location of borehole, inferred groundwater level and proposed mean water level after impoundment of the dam were shown).

5.7. Rock Slope Stability Analysis

The hilly sections at the dam abutments and reservoir rim may present challenges to the safe functioning of the dam. Detecting these potentially problematic slopes at the reservoir rim and support in the early stages of the inquiry can aid in determining suitable corrective measures. During the field investigation, it was identified possible slope-critical portions by considering the components that determine the failure mechanism, which can either be structurally controlled failures or non-structurally controlled failures (rock mass-controlled failures). The investigation reveals that the failures occurring at the dam abutments are mostly influenced by the structural characteristics of the critical slope sections.

5.7.1. Kinematic Analysis

Kinematic analyses were carried out for the structurally controlled slope failure on the right abutment slope sections (RAS) and the left abutment slope sections (LAS), as seen in Figure 5.22. These analyses were carried out with the software known as Dips 6 (Rocscience, 2004). Input data that the Dips software takes into consideration includes the dip angle, slope and dip direction of joints, as well as the slope and friction angle, as shown in Table 5.22. The dip and dip direction of joints and slope were measured on-site, while the friction angle along the failure plane and cohesion were determined based on the Barton-Bandis failure criterion, which is utilized to calculate the shear strength properties of the discontinuities using the RocData software (Table 5.12).

Table 5. 22. Input parameters of the geometry of joints and slope face for the Dips 6.0 Software.

Input parameters for Kinematic Analysis								
Right Abutment Slope Sections (RAS)					Left Abutment Slope Sections (LAS)			
SO and FPO	JS1	JS2	JS3	Slope Face	JS1	JS2	JS3	Slope Face
DD	130°SE	238°SW	322°NW	219°SW	125°SE	59°NE	330°NW	52°NE
DA	80°	46°	39°	68°	75°	47°	39°	69
Φ (°)	24°				31°			
SO is the slope orientation, FPO is the failure plane orientation, DD is the dip direction, DA is the dip amount, and Φ (°) is the friction angle in degrees.								

In the left abutment slope section (LAS), there are expected failure modes such as planar failure modes. This is due to the fact that joint set 2 (JS2) is plotted in the critical zone of the planar mode of failure, as shown in Figure 5.22a. Additionally, wedge failure modes are

anticipated to occur due to the fact that the intersection of JS1 and JS2 and the intersection of JS2 and JS3 are plotted on the critical zone of the wedge mode of failure, which preconditions such an occurrence. This is demonstrated in Figure 5.22b. There are failure modes that are anticipated to occur in the right abutment slope section (RAS), such as planar failure modes. This is due to the fact that joint set 2 (JS2) is plotted in the critical zone of the planar mode of failure, as demonstrated in Figure 5.22c. Additionally, wedge failure modes are anticipated to take place, as the intersection of joint set 1 and joint set 2 is plotted in the critical zone of the wedge mode of failure, which preconditions the occurrence of such a failure mode, as demonstrated in Figure 5.22d.

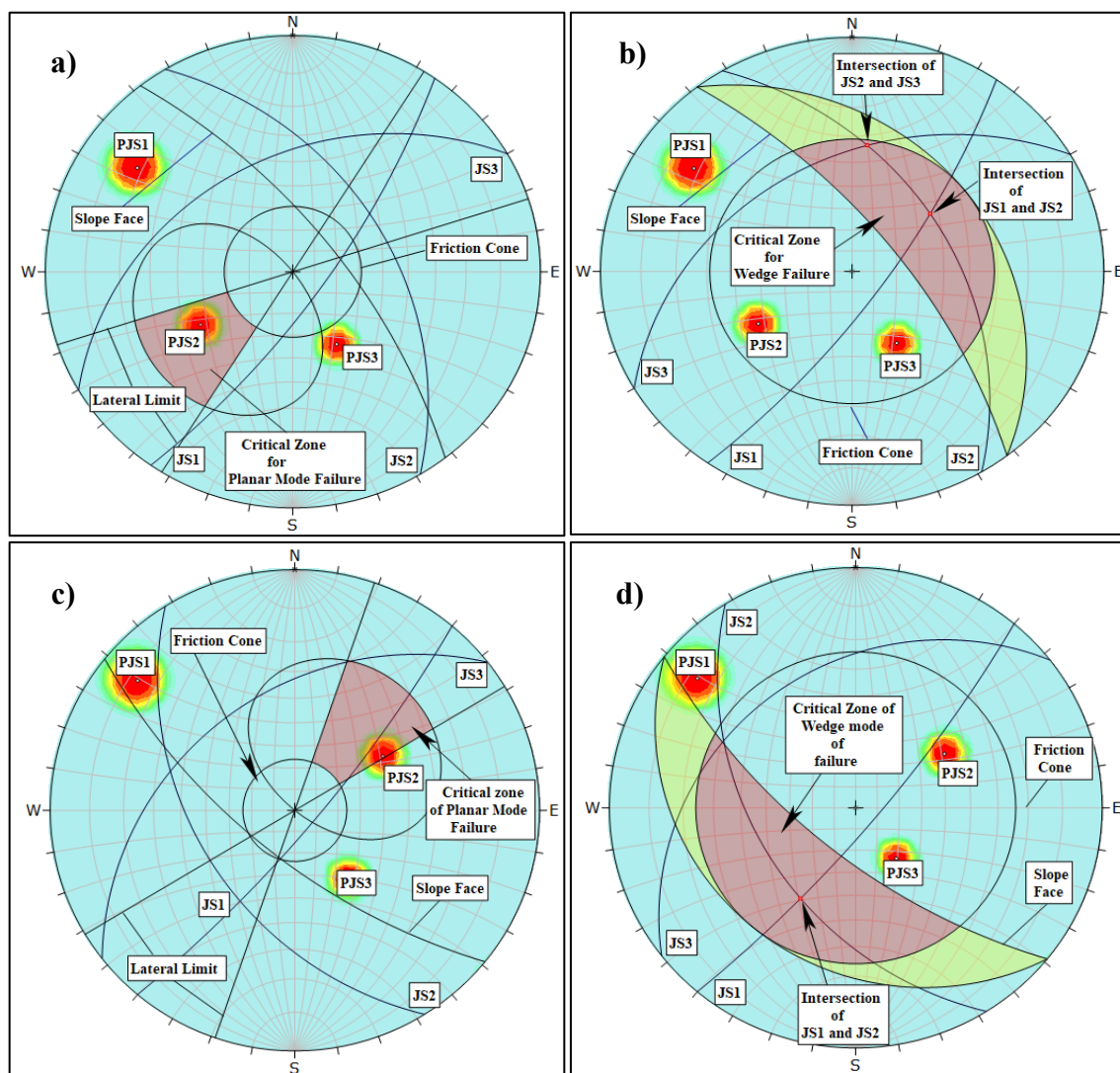


Figure 5. 22. Kinematic analysis for Right Abutment Slope Section and Left Abutment Slope Section using Dips 6.0 software; (a) Checking for Planar mode of failure for LAS (b) Checking for wedge mode of failure for LAS (c) Checking for planar mode of failure for RAS (d) Checking for Wedge mode of failure for RAS.

5.7.2. Limit Equilibrium Method

According to the findings of the kinematic analysis, a planar failure mode is anticipated to occur in the left abutment slope section (LAS) and the right abutment slope section (RAS) as a consequence of the discontinuity set JS2, which is depicted in Figure 5.22. As a result, the deterministic method was applied further for stability analysis with regard to FOS. This study was conducted in order to assess the probability of kinematic failure modes (Park and West, 2001). In context of this, the factor of safety (FOS) was calculated for every single planar failure mode, and the RocPlane software was utilized to perform the analysis (Rocscience, 2004). As shown in Table 5.23, the RocPlane software makes use of the input parameters that are specified. Due to the fact that the Barton-Bandis failure criterion is utilized in the process of determining the shear strength parameters of discontinuities with the RocData program (Table 5.12), the shear strength parameters were derived in accordance with this criterion. The geometry and discontinuity orientation of the slope were ascertained throughout the field investigation. Waviness was assessed in the field for this investigation. The consideration of waviness is important in determining the shear strength of the failure plane. The text describes the irregularities on the surface of the failure plane, which can be observed at distances ranging from 1 to 10 meters (Rocscience, 2004). To quantify the degree of waviness, the average dip of the failure plane is adjusted by subtracting the minimal dip. The shear strength of the failure plane increases proportionally when the waviness angle exceeds zero.

Table 5. 23. Specified input parameters for the joint geometry, slope face, shear strength, and forces (including water forces and dynamic forces) in the RocPlane Software.

Critical Slope Section	PFJS	Slope Geometry							Shear Strength Parameters		Forces	
		SA (°)	USA (°)	FPA (°)	B _w (m)	SH (m)	γ (t/m ³)	w (°)	φ (°)	C (t/m ²)	u _w (t/m ³)	α(g)
LAS	JS2	69	15	47	63	28	2.02	8	30.23	2.65	1	0.0925
RAS	JS2	68	20	46	55	24	1.57	6	27.87	2.14	1	0.0925

Where PFJS is potential failure joint sets, SA, USA, and FPA stand for slope angle, upper slope angle, and failure plane angle, respectively; B_w is bench width, γ is unit weight of rock, φ is frictional angle, w represents waviness; C is cohesion; u_w is water pressure; SH is slope height and α is the seismic coefficient.

When determining the stability of the rock slope of the abutments, the safety factor is taken into consideration. Within the context of the potential failure surface, the safety factor takes into account the equilibrium between the forces that contribute to instability (referred to as driving forces) and the forces that oppose instability (referred to as resisting forces). This was accomplished by modifying the percent water pressure in the slope section, with dry conditions at 0% fill, moderate saturation at 50% fill, partial saturation at 75% fill, and full saturation at 100% fill for both static conditions (without seismic loading) and dynamic conditions (with seismic loading) as shown in Table 5.24. The FOS at the left abutment and the right abutment was estimated using this method.

In this study, the FOS was interpreted according to Norrish and Wyllie (1996) and stated that when $FOS = 1$, the driving and resisting forces are balanced; $FOS > 1$ indicates a stable slope; and $FOS = 1$ corresponds to a crucial state of equilibrium. After entering the input parameters into the RocPlane software, the factor of safety for the planar mode of failure was calculated. The RocPlane software was utilized to produce a 2D and 3D picture of the joint set JS2 failure plane in the left abutment slope section LAS, as shown in Figure 5.23. Under the assumption that the slope was completely saturated with water and taking into account the changing conditions, the seismic forces were anticipated to occur with a magnitude of 48.06t, which was exerted on the failure plane. The factor of safety is 0.22, given that the resistive force is 89.83t/m and the driving force is 412.78t/m. Under dry static conditions, the factor of safety was 0.99, indicating that the slope section was unstable. However, during fully saturated static conditions, the factor of safety decreased to 0.32. This suggests that the pore water pressure is a crucial factor in the instability of the slope section. This was justified by elevating the pore water pressure level from 0% to 100%, thus the pore water pressure exerts an influence on the opposing force, thereby resulting in a reduction in the factor of safety.

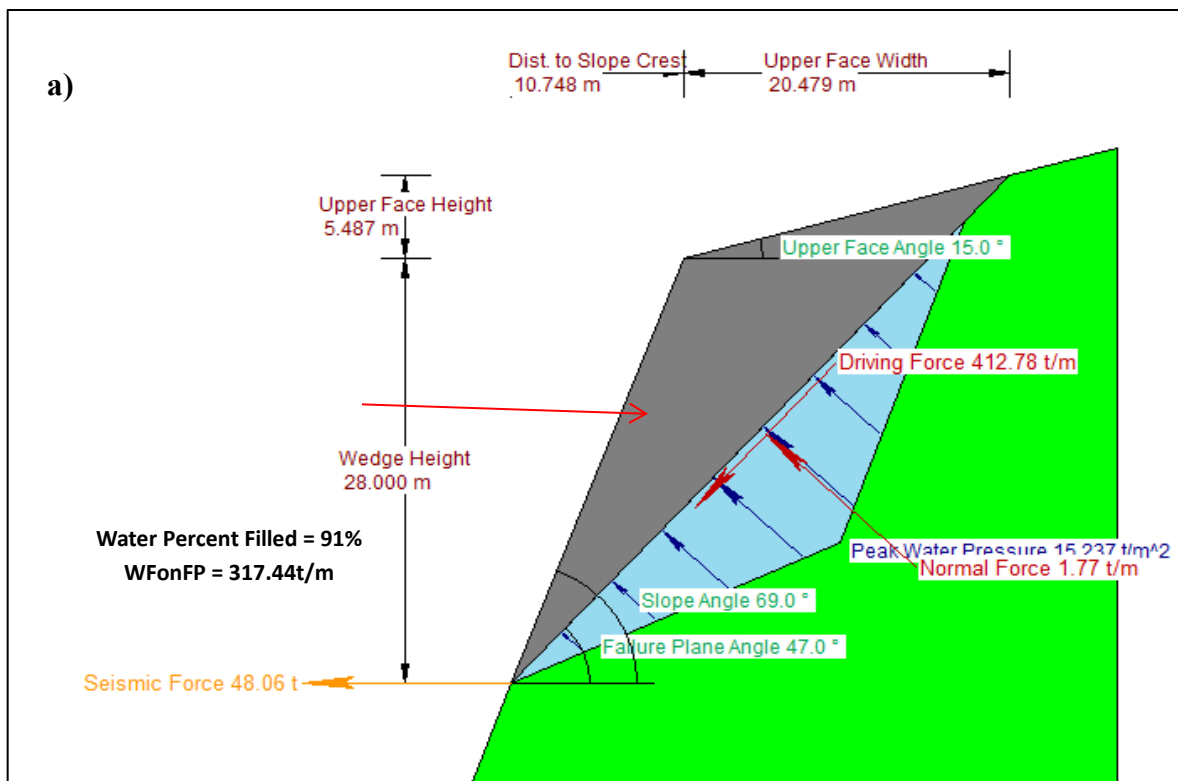
Table 5. 24. The factor of safety has been established for the crucial slope portions that exhibit a planar mode of failure in the LAS.

Static Condition				Dynamic Condition			
Water Pressure Level (%)	Resisting Force (t/m)	Driving Force (t/m)	FOS	Water Pressure Level (%)	Resisting Force (t/m)	Driving Force (t/m)	FOS
0 (Dry)	377.63	380	0.99	0 (Dry)	319.42	412.78	0.77
50	308.32	380	0.81	50	250.11	412.78	0.61
75	221.68	380	0.58	75	163.47	412.78	0.40
100	122.12	380	0.32	100	88.83	412.78	0.22

At the dry static condition (0% of water percent filled and without the consideration of the seismic loading), the slope section at the left abutment was unstable due to the factor of safety of 0.99 with the resisting and driving forces of 377.63t/m and 380t/m, respectively. when the slope section is moderately saturated water percent filled of 50%, the slope section is still unstable with decreased factor of safety of 0.81 having the resisting and driving forces of 308.32t/m and 380t/m, respectively. when the slope section is partially saturated water percent filled of 75%, the slope section is still unstable with decreased factor of safety of 0.58 having the resisting and driving forces of 221.68t/m and 380t/m, respectively. when the slope section is fully saturated water percent filled of 100%, the slope section is still unstable with decreased factor of safety of 0.32 having the resisting and driving forces of 122.12t/m and 380t/m, respectively as given in Table 5.25.

At the dry dynamic state (0% water percentage filled and seismic loading considered), the slope section at the left abutment was unstable due to a factor of safety of 0.77 with resisting and driving forces of 319.42t/m and 412.78t/m, respectively. When the slope section is 50% filled with moderately saturated water, it remains unstable with a lowered factor of safety of 0.61, resisting, and driving forces of 250.11t/m and 412.78t/m, respectively. When the slope portion is partially saturated with 75% water, it remains unstable with a decreased factor of safety of 0.40, resisting at 163.47t/m and driving at 412.78t/m, respectively. The slope section remains unstable with a decreased factor of safety of 0.22 and resisting and driving forces of 88.83t/m and 412.78t/m, respectively, even when fully saturated with water. as given in Table 5.25.

The wedge weight formed by the slope angle and failure plane angle was 519.59t/m, whereas the wedge volume was 257.22m³/m. The normal force in dry and fully saturated conditions was 319.21t/m and 1.77t/m, respectively. The shear strength in the dry and fully saturated conditions was 307.35t/m² and 122.37t/m², respectively. This decreasing trend of a factor of safety with an increasing percentage of water pressure level indicates that a minimum amount of water pressure was required to displace the slope's wedge weight of 519.59t/m, which is greater than the driving and resisting forces, resulting in slope section instability. According to the sensitivity analysis, the following factors contributed to the slope section's failure: water pressure level, slope angle, failure plane angle, and seismic coefficient. The water pressure level necessary to displace the wedge will be minimal since the difference between the slope and failure plane angles is minimal, as is the wedge weight formed as a result of that difference. The failure plane angle, slope angle, upper face angle, and seismic coefficient, along with the water pressure level, resulting in a factor of safety of less than one. Aside from the water pressure level, additional factors that contribute to the slope section's instability include the failure plane angle and seismic coefficient.



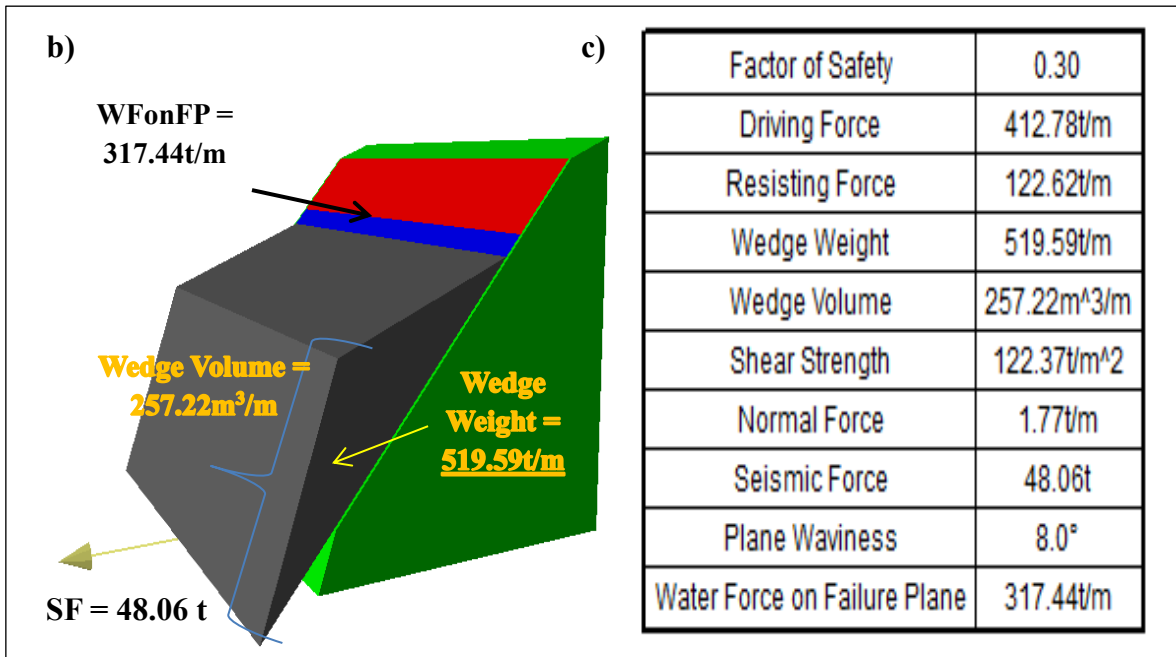


Figure 5. 23. The Deterministic method of slope stability analysis by RocPlane software under 91% water pressure level and dynamic condition at both LAS slope section. a) 2D side view with forces acting on the plane of failure of JS2 for LAS. b) 3D view of failure plane of JS2 for LAS. c) The factor of safety at 91% of water pressure level under dynamic condition.

Similar to the left abutment slope section (LAS), the RocPlane programme was used to generate a 2D and 3D image of the joint set JS2 failure plane in the right abutment slope section (RAS), as illustrated in Figure 5.24. Under the assumption that the slope was completely saturated with water and taking into account the changing conditions, the seismic forces were anticipated to occur with a magnitude of 32.28t, which was exerted on the failure plane. The factor of safety is 0.0, given that the resistive force is 0.0t/m and the driving force is 273.46t/m. Under dry static conditions, the factor of safety was 0.88, indicating that the slope was unstable. However, with fully saturated static circumstances, the factor of safety decreased to zero. This shows that pore water pressure plays an important role in the instability of the slope section. This was justified by increasing the pore water pressure from 0% to 100%, which exerts an influence on the opposing force, resulting in a decrease in the factor of safety.

Table 5. 25. The factor of safety has been established for the crucial slope portions that exhibit a planar mode of failure in the RAS.

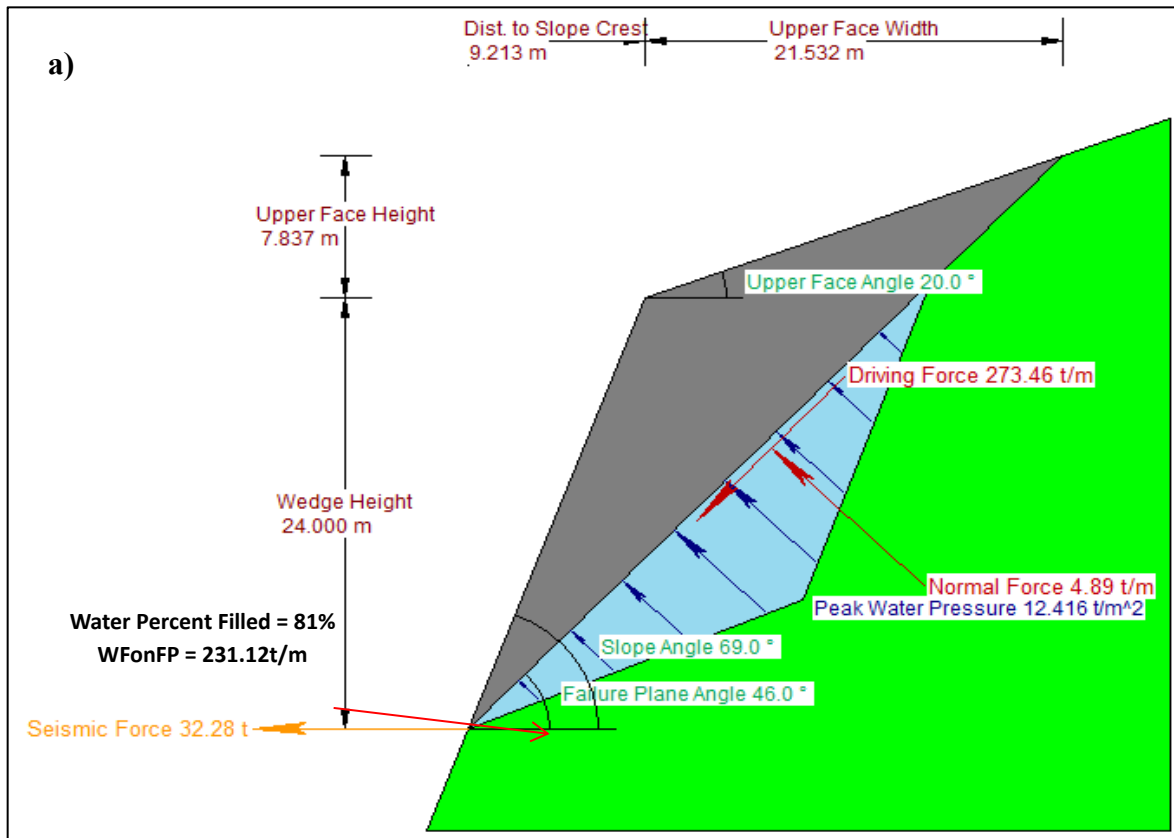
Static Condition				Dynamic Condition			
Water Pressure Level (%)	Resisting Force (t/m)	Driving Force (t/m)	FOS	Water Pressure Level (%)	Resisting Force (t/m)	Driving Force (t/m)	FOS
0 (Dry)	248.39	251.04	0.99	0 (Dry)	233.46	273.46	0.85
50	192.56	251.04	0.77	50	177.84	273.46	0.65
75	122.78	251.04	0.50	75	108.06	273.46	0.40
100	0	251.04	0	100	0	273.46	0

As shown in above Table 5.25, At the dry static condition (0% of water percent filled and without the consideration of the seismic loading), the slope section at the right abutment was unstable due to the factor of safety of 0.99 with the resisting and driving forces of 248.39t/m and 251.04t/m, respectively. when the slope section is moderately saturated water percent filled of 50%, the slope section is still unstable with decreased factor of safety of 0.77 having the resisting and driving forces of 192.56t/m and 251.04t/m, respectively. when the slope section is partially saturated water percent filled of 75%, the slope section is still unstable with decreased factor of safety of 0.50 having the resisting and driving forces of 122.78t/m and 251.04t/m, respectively. when the slope section is fully saturated water percent filled of 100%, the slope section is still unstable with decreased factor of safety of 0.0 having the resisting and driving forces of 0.0t/m and 251.04t/m, respectively as given in Table 5.25.

At the dry dynamic state (0% water percentage filled and seismic loading considered), the slope section at the left abutment was unstable due to a factor of safety of 0.85 with resisting and driving forces of 233.46t/m and 273.46t/m, respectively. When the slope section is 50% filled with moderately saturated water, it remains unstable with a lowered factor of safety of 65, resisting, and driving forces of 177.84t/m and 273.46t/m. When the slope portion is partially saturated with 75% water, it remains unstable with a decreased factor of safety of 0.40, resisting at 108.06t/m and driving at 273.46t/m, respectively. The slope section remains unstable with a decreased factor of safety of 0.0 and resisting and driving forces of 0t/m and 273.46t/m, respectively, even when fully saturated with water as given in Table 5.25.

The wedge weight formed by the slope angle and failure plane angle was 348.98t/m, whereas the wedge volume was 222.28m³/m. The normal force in dry and partially saturated conditions was 219.20t/m and 4.89t/m, respectively. The shear strength in the dry and fully saturated conditions was 210.63t/m² and 97.30t/m², respectively. This decreasing trend of a

factor of safety with an increasing percentage of water pressure level indicates that a minimum amount of water pressure was required to displace the slope's wedge weight of 348.98t/m, which is greater than the driving and resisting forces, resulting in slope section instability. The sensitivity analysis identified several parameters that contributed to the failure of the slope section, including the water pressure level, slope angle, failure plane angle, and seismic coefficient. The water pressure required to displace the wedge will be very low due to the small difference between the angles of the slope and the failure plane, as well as the resulting weight of the wedge. The combination of the failure plane angle, slope angle, upper face angle, seismic coefficient, and water pressure level leads to a factor of safety that is less one. In addition to the water pressure level, other elements that contribute to the instability of the slope section include the angle of the failure plane and the seismic coefficient.



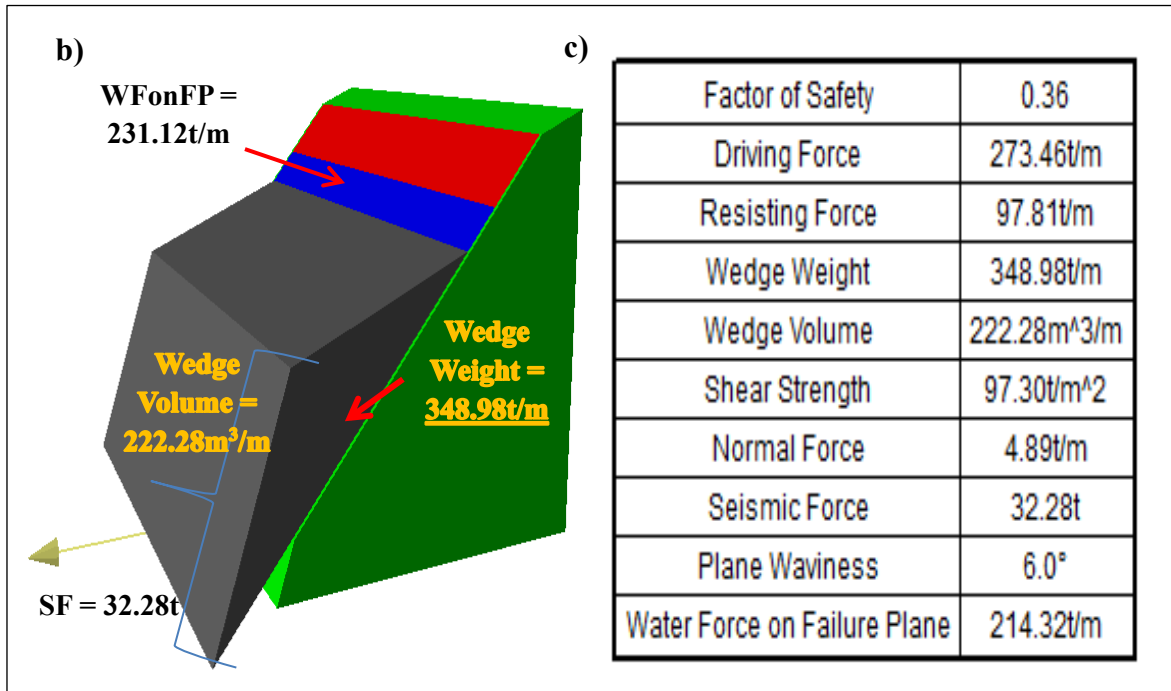


Figure 5. 24. The Deterministic method of slope stability analysis by RocPlane software under 81% water pressure level and dynamic condition at both RAS slope section. a) 2D side view with forces acting on the plane of failure of JS2 for RAS. b) 3D view of failure plane of JS2 for RAS. c) The factor of safety at 81% of water pressure level under dynamic condition.

To summarize, the deterministic method slope stability analysis indicated that the left and right abutment slope sections were unstable in both the saturated static and saturated dynamic conditions for the planar mode of failure. The pore water pressure is a critical factor in the destabilization of the slope section, beyond the influence of the seismic coefficient and other factors (Pan et al., 2020). This indicates that water pressure level combined with slope geometry and discontinuity orientation, seismic loading contributes a significant role in destabilizing the slope sections. Consequently, it was necessary to implement appropriate corrective actions for these particular sections of the slope to prevent the dam abutments from collapsing.

The intersection of JS1 and JS2 and intersection of JS2 and JS3 at the left abutment slope section and the intersection of JS1 and JS2 are expected to cause wedge mode failure in addition to planar failure. Swedge 4.0 program assessed the deterministic approach of slope stability analysis for the wedge mode of failure. The wedge software's input parameters include the discontinuity orientation, slope geometry, unit weight, and shear strength

parameters obtained with RocData software. Table 5.26 shows the input parameters for the Swedge software.

Table 5. 26.. The input parameters for determining the factor of safety at the left abutment critical slope sections having a wedge mode of failure.

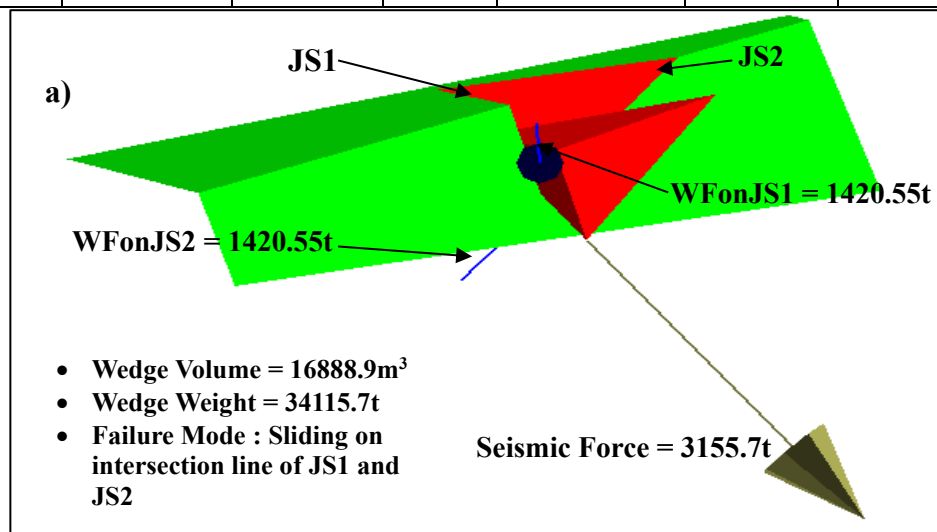
Critical Slope Section	Potential Failure Plane	Discontinuity Orientation		Slope Geometry				γ (t/m ³)	Shear strength parameters	
				Slope Angle		SH (m)	Bw (m)		C (t/m ²)	Φ (deg)
		DD (°)	DA(°)	DD (°)	DA (°)					
LAS	JS1	125	75	52	69	28	63	D = 2.02 S = 2.04	2.03	30.90
	JS2	59	47						2.65	30.23
	JS3	330	39						4.28	31.49
RAS	JS1	130	80	219	69	24	55	D = 1.57 S = 1.59	1.22	19.40
	JS2	238	46						2.14	27.87
D is dry unit weight, S is saturated unit weight, DA is the dip angle of the joints and slope face, DD is the dip direction of the joints and slope face, SH is slope height, Bw is the bench width, γ is the unit weight of the rock, C is cohesion and Φ is frictional angle.										

The factor safety of the slope section was ascertained in the left abutment slope section (LAS) following the input parameters' input into the swedge software. The Factor of Safety (FOS) was determined by adjusting the water pressure percentage in the slope section. Dry conditions were represented by 0% fill, moderate saturation by 50% fill, partial saturation by 75% fill, and full saturation by 100% fill. These conditions were evaluated for both static conditions (without seismic loading) and dynamic conditions (with seismic loading), as indicated in Table 5.27. The factor of safety analysis of wedge mode of failure at the left abutment slope section (LAS) due to the intersection of JS1 and JS2 showed that the slope section was unstable with the factor of safety of less than 1 during the dry static and dry dynamic condition as given in Table 5.27. During the static condition without considering the water pressure, the factor of safety was 0.90 with the resisting force and driving force of 22546.7t and 24852t respectively. Therefore, the slope section was unstable according to Norrish and Wyllie (1996). Nevertheless, the factor of safety diminishes progressively as the water pressure level rises and the resisting forces decrease. The water pressure reduces the factor of safety by weakening the resisting/opposing forces and intensifying the driving force as shown in Table 5.27. At the moderate water pressure level of 50%, The factor of safety shows slight decreasing change to 0.84 with resisting force of 20907.5t and driving force of 24852t. The factor of safety decreases 0.68 at partial water percent fill of 75% with the resisting force and driving force of 17014.3t and 24852t respectively. As the water pressure

level increases to 100% the slope section becomes unstable with the drastic decrease of factor of safety of 0.41 with the resisting force of 10204t and driving force of 24852t. When taking into account the seismic coefficient of the research area, the factor of safety for the slope section falls as the driving forces increase while the resisting forces remain the same. During the dry dynamic condition, the factor of safety was 0.80 with the resisting force and driving force of 22546.7t and 28007.8t respectively. Under a water pressure level of 50%, the factor of safety exhibits a declining trend, reaching 0.75. The resistive force is 20907.5t, while the driving force is 28007.8t. The factor of safety reduces by 0.61 when the water fill percentage is 75%, with a resisting force of 17014.3t and a driving force of 28007.8t. When the water pressure reaches 100%, the slope section becomes unstable due to a significant reduction in the factor of safety to 0.36. This decrease is caused by a resistive force of 10204t and a driving force of 28007.8t. The seismic force acting on the slope section is 3155.7t, the water force on JS1 is 1420.55t and the water force on JS2 is 23918.9t. The failure mode is sliding on intersection line of JS1 and JS2. The wedge weight and wedge volume is 34115.7t and 16888.9m³, respectively.

Table 5. 27. The factor of safety determined at the left abutment critical slope sections (LAS) having a wedge mode of failure due to the intersection of JS1 and JS2.

Static Condition				Dynamic Condition			
Water Pressure Level (%)	Resisting Force (t)	Driving Force (t)	FOS	Water Pressure Level (%)	Resisting Force (t)	Driving Force (t)	FOS
0 (Dry)	22546.7	24852	0.90	0 (Dry)	22546.7	28007.8	0.80
50	20907.5	24852	0.84	50	20907.5	28007.8	0.75
75	17014.3	24852	0.68	75	17014.3	28007.8	0.61
100	10204	24852	0.41	100	10204	28007.8	0.36



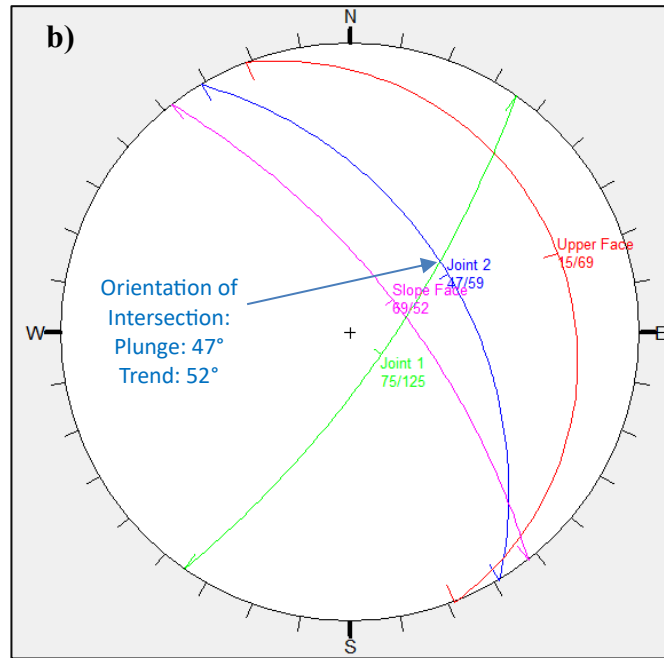


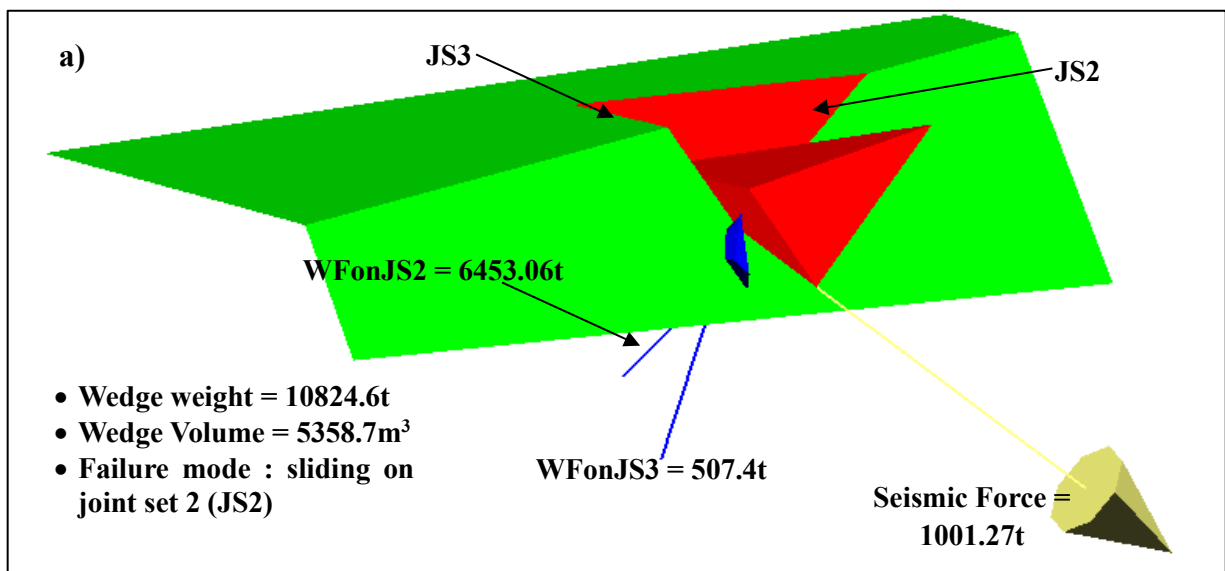
Figure 5. 25.. The Deterministic method of slope stability analysis by Swedge software under saturated dynamic condition at LAS (JS1 and JS2). a) 3D view of the joint sets with forces acting on the plane of failure for wedge mode of failure. b) The stereographic projection of the joint sets with slope face.

The determination of the factor of safety for the wedge mode of failure at the left abutment slope section (LAS) due to the intersection of JS2 and JS3 revealed that the slope section was stable, with a factor of safety greater than 1, under both dry static and dry dynamic conditions, as indicated in Table 5.28. Under the dry static condition, the slope section was stable with the factor safety of 1.65 having the resisting force and driving force of 9790.3t and 5909.6t, respectively. At the moderate water pressure level of 50%, The factor of safety shows slight decreasing change to 1.59 with resisting force of 9363.9t and driving force of 5909.6t. The factor of safety decreases 1.41 and still slope section is stable at partial water percent fill of 75% with the resisting force and driving force of 8351.27t and 5909.6t, respectively. As the water pressure level increases to 100% the slope section becomes unstable with the drastic decrease of factor of safety of 0.46 with the resisting force of 2890.13t and driving force of 6255.4t. Considering the seismic coefficient of the study area, the factor of safety for the slope section decreases as the driving forces increase, while the resisting forces remain constant. In the dry dynamic condition, the factor of safety was 1.42, with a resisting force of 9790.3t and a driving force of 6910.9t. At a water pressure level of 50%, the factor of safety exhibits a minor decrease to 1.35, with a resisting force of 9363.9t and a driving force of 6810.94t. The factor of safety reduces to 1.20, but the slope section remains stable at a partial water percent fill of 75%. The resistive force is 8351.27t and the

driving force is 6910.9t. When the water pressure reaches 100%, the slope section becomes unstable due to a significant fall in the factor of safety, which is 0.40. The resistive force is 2890.13t, while the driving force is 7208t. This indicates that water pressure influences the instability of the slope section compared to the seismic coefficient. The seismic force acting on the slope section is 1001.27t, the water force on JS2 is 6453.06t and the water force on JS3 is 507.4t. The failure mode is sliding on joint 2 JS2. The wedge weight and wedge volume are 10824.t and 5358.7m³, respectively.

Table 5. 28.. The factor of safety determined at the left abutment critical slope sections (LAS) having a wedge mode of failure due to the intersection of JS2 and JS3.

Static Condition				Dynamic Condition			
Water Pressure Level (%)	Resisting Force (t)	Driving Force (t)	FOS	Water Pressure Level (%)	Resisting Force (t)	Driving Force (t)	FOS
0 (Dry)	9790.3	5909.6	1.65	0 (Dry)	9790.3	6910.9	1.42
50	9363.9	5909.6	1.59	50	9363.9	6910.9	1.35
75	8351.27	5909.6	1.41	75	8351.27	6910.9	1.20
100	2890.13	6255.4	0.46	100	2890.13	7208	0.40



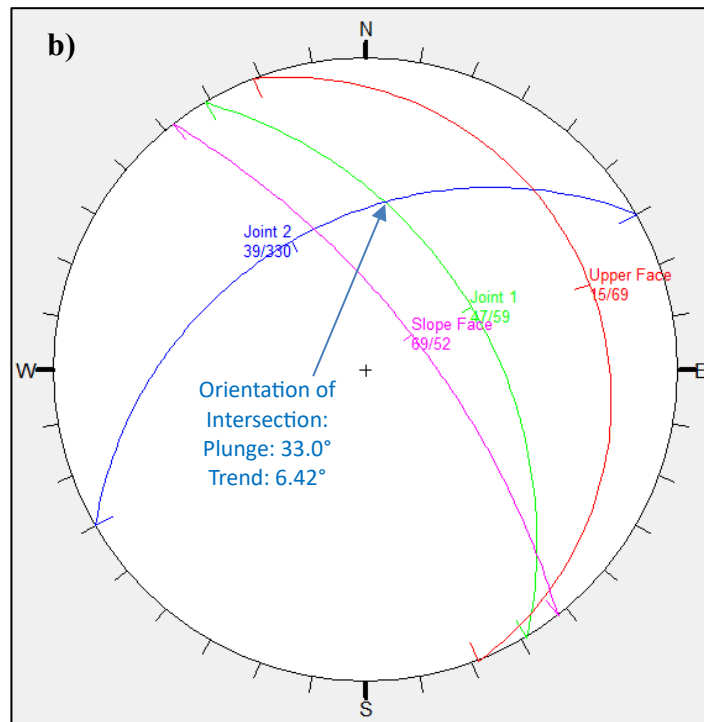


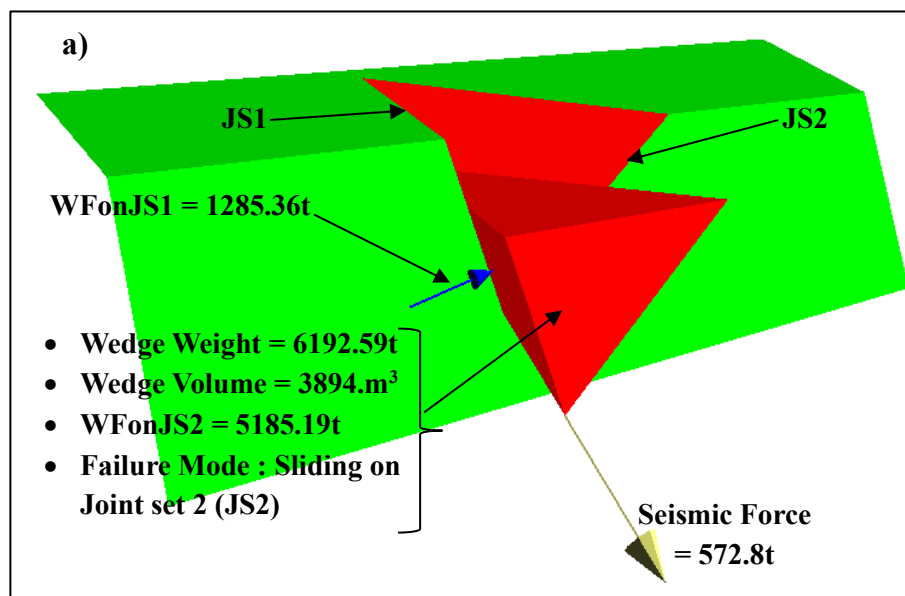
Figure 5. 26. The Deterministic method of slope stability analysis by Swedge software under saturated dynamic condition at LAS (JS2 and JS3). a) 3D view of the joint sets with forces acting on the plane of failure for wedge mode of failure. b) The stereographic projection of the joint sets with slope face.

Similar to the left abutment slope section (LAS), beside the planar mode of failure wedge mode of failure was also prominent and expected to occur due to intersection between the JS1 and JS2 as plotted and interpreted in the kinematic analysis. The Factor of Safety (FOS) was obtained by modifying the proportion of water pressure in the slope segment. The representation of dry conditions was indicated by a fill of 0%, while moderate saturation was represented by a fill of 50%. Partial saturation was indicated by a fill of 75%, and full saturation was represented by a fill of 100%. The conditions were assessed under both static conditions (without seismic loading) and dynamic conditions (with seismic loading), as specified in Table 5.29. The analysis of the factor of safety for the wedge mode of failure at the right abutment slope section (RAS) due to the intersection of JS1 and JS2 revealed that the slope section was stable, with a factor of safety greater than 1, under both dry static and dry dynamic conditions, as indicated in Table 5.29. Under the dry static condition, the slope section was stable with the factor safety of 1.28 having the resisting force and driving force of 5398.4t and 4192.5t, respectively. At the moderate water pressure level of 50%, The factor of safety shows slight decreasing change to 1.04 with resisting force of 4999.05t and driving force of 4192.5t. The factor of safety decreases 0.97 and slope section is unstable at partial

water percent fill of 75% with the resisting force and driving force of 4050.66t and 4192.5t, respectively. As the water pressure level increases to 100% the slope section becomes unstable with the drastic decrease of factor of safety of 0.09 with the resisting force of 419.6t and driving force of 4192.5t. Considering the seismic coefficient of the study area, the factor of safety for the slope section decreases as the driving forces increase, while the resisting forces remain constant. In the dry dynamic condition, the factor of safety was 1.13, having the resisting force and driving force of 5398.4t and 4765.3t, respectively. At a water pressure level of 50%, the factor of safety exhibits a minor decrease to 1.04, with a resisting force of 4999.05t and a driving force of 4765.3t. The factor of safety reduces to 0.85, but the slope section becomes unstable at a partial water percent fill of 75%. The resistive force is 4050.7t and the driving force is 4765.3t. When the water pressure reaches 100%, the slope section becomes unstable due to a significant fall in the factor of safety, which is 0.086. The resistive force is 416.6t, while the driving force is 4821.1t.

Table 5. 29. The factor of safety determined at the right abutment critical slope sections (RAS) having a wedge mode of failure due to the intersection of JS1 and JS2.

Static Condition				Dynamic Condition			
Water Pressure Level (%)	Resisting Force (t)	Driving Force (t)	FOS	Water Pressure Level (%)	Resisting Force (t)	Driving Force (t)	FOS
0 (Dry)	5398.4	4192.5	1.28	0 (Dry)	5398.4	4765.3	1.13
50	4999.05	4192.5	1.19	50	4999.05	4765.3	1.04
75	4050.7	4192.5	0.97	75	4050.7	4765.3	0.85
100	416.6	4255.8	0.09	100	416.6	4821.1	0.086



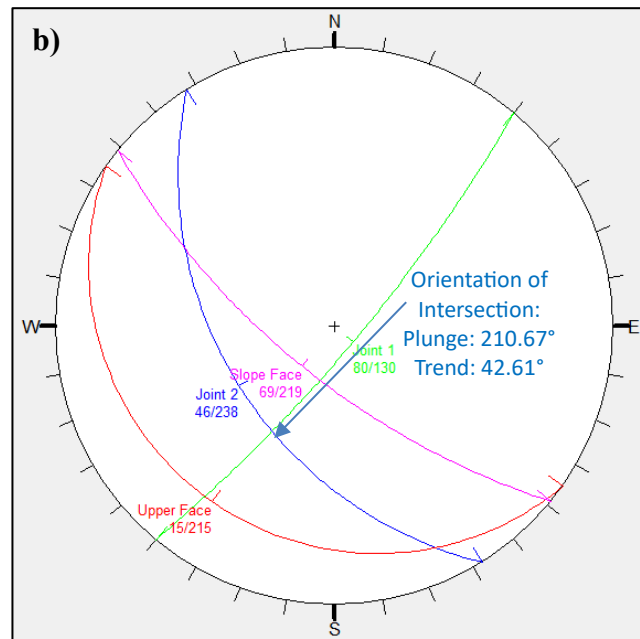


Figure 5. 27. The Deterministic method of slope stability analysis by Swedge software under saturated dynamic condition at RAS (JS1 and JS2). a) 3D view of the joint sets with forces acting on the plane of failure for wedge mode of failure. b) The stereographic projection of the joint sets with slope face.

5.7.3. Design of Possible Remedial Measures

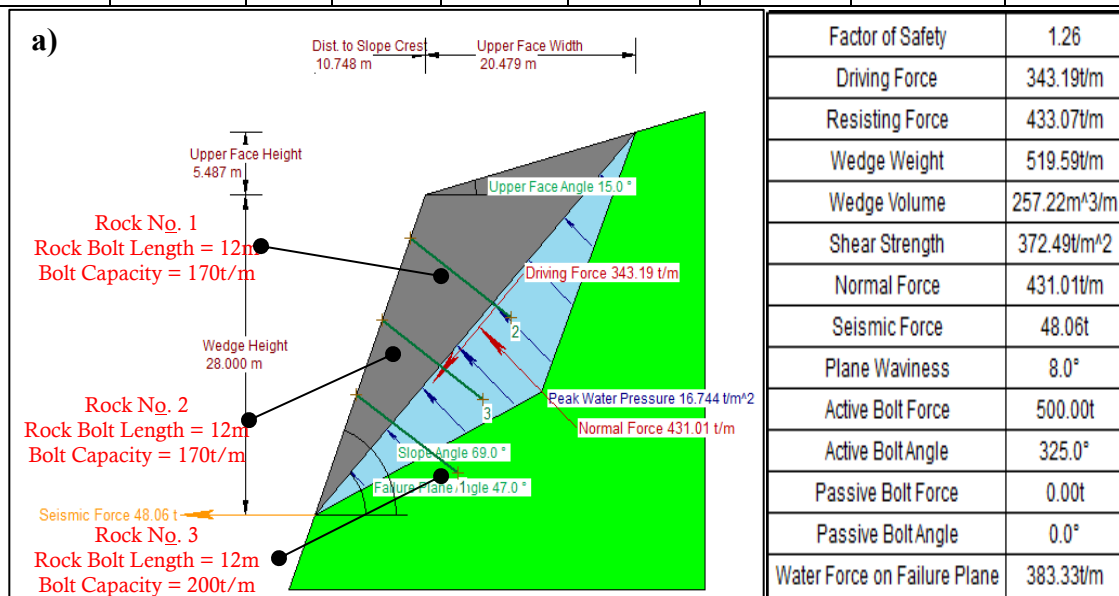
Due to the instability of the slope sections in the worst condition (saturated dynamic state), it is necessary to stabilize the rock slope portions. Neglecting to do so may potentially jeopardize structural stability. Hence, rock bolts are necessary to prevent planar collapse, while a mixture of shotcrete and rock bolts is employed to stabilize wedge failure. Properly implementing rock bolts and shotcrete can lead to an increase in resistance force and shear strength, which are derived from the cohesion and frictional strength of sliding planes, as well as a decrease in the driving force (including the mass accelerated by gravity, seismic force, and water pressure). The rise in the resisting and driving forces leads to a corresponding increase in the factor of safety. In this research work, the stabilization of planar mode of failure was done by employing RocPlane 2.0 software (RocPlane 2.0 Rocscience 2004 a, b, c, d) through the implementation of rock bolts that were installed across the failure plane to the relatively sound rock. Similarly, the stabilization of wedge mode of failure at the left abutment was done using the Swedge 4.0 software (Swedge 4.0 Rocscience 2004a, b, c, d) via the application of rock bolts exceeding the failure plane and shotcrete to slope face. All the stabilization design was carried out by considering the worst condition of the slope sections (i.e. dynamic saturated condition).

Similarly, about 3 rock bolts with length of 12m each were applied to the unstable slope section at the left abutment with planar mode failure due to JS2 at angle of 35° each , bolt capacity of 170t/m each and resultant active bolt force of 500t/m. After the rock bolts with these properties are applied to the slope section, the factor of safety has increased from 0.22 to 1.26 with the resisting force of 433.07t/m and driving force of 343.19t/m as shown in Figure 5.28a and Table 5.30.

Similarly, about 2 rock bolts with length of 12m each were applied to the unstable slope section at the left abutment with planar mode failure due to JS2 at angle of 35° each , bolt capacity of 200t/m each and resultant active bolt force of 400t/m. After the rock bolts with these properties are applied to the slope section, the factor of safety has increased from 0.00 to 1.24 with the resisting force of 260.8t/m and driving force of 210.89t/m as shown in Figure 5.28b and Table 5.30. The significance of rock bolts in the stabilization of rock slope sections is demonstrated by the fact that the factor of safety has increased as a result of the implementation of rock bolts.

Table 5. 30. Remedial measures for Planar Mode Failure and the Increased FOS for both LAS and RAS.

Slope Section	FOS without support	Bolt No.	Angle (Deg)	Bolt Length (m)	Anchor Length (m)	Bolt Capacity (t/m)	Resisting Force (t/m)	Driving Force (t/m)	FOS
LAS	0.22	1.	35	12	7.72	170	433.07	343.19	1.26
		2.	35	12	2.19	170			
		3.	35	12	5.11	170			
RAS	0.00	1.	35	12	3.67	200	260.8	210.89	1.24
		2.	35	12	6.74	200			



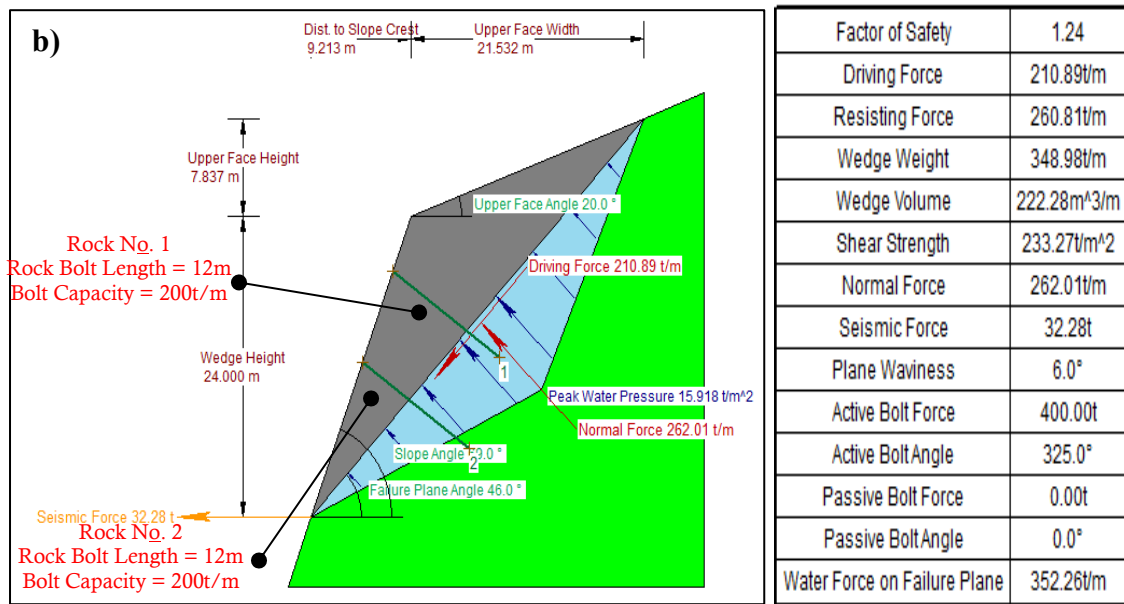


Figure 5. 28. Countermeasures for Planar Mode Failure and Factor of Safety (FOS) include a) addressing the slope section on the left abutment (LAS) and b) addressing the slope section on the right abutment (RAS).

To stabilize the wedge mode of failure due to the intersection JS1 and JS2 at the left abutment slope section (LAS), about 3 (three) rock bolts were applied to the slope section where each one of them have trend of 165° and plunge of 45°, bolt length of 15m, bolt capacity of 14800t. Additional to the rock bolts, the shotcrete was also implemented stabilize the slope section with thickness of 0.1m and shear strength of 200t/m² which is expected to apply the shotcrete force of 4837.83tons. These remedial measures increased the factor of safety from 0.36 to 1.25 with the resisting force and driving force of 52919.4t and 42320.4t respectively as shown in Table 5.31 and Figure 5.29.

Table 5. 31. Developed remedies for addressing wedge mode failure at the left abutment slope section (LAS) caused by JS1 and JS2, as well as an enhanced factor of safety.

Slope Section	FOS without support	Bolt No.	Orientation		Bolt Properties		
			Trend (°)	Plunge (°)	Bolt Length (m)	Anchored Length (m)	Capacity (tons)
LAS (JS1 and JS2)	0.36	1.	165	45	15	4.46	14800
		2.	165	45	15	4.97	14800
		3.	165	45	15	5.04	14800
		Shotcrete Properties					
		Thickness(m)	Shear strength(t/m²)			Shotcrete Force (t)	
		0.1	200			4837.83	
FOS with support			Resisting Force (t)			Driving Force(t)	
1.25			52919.4			42320.4	

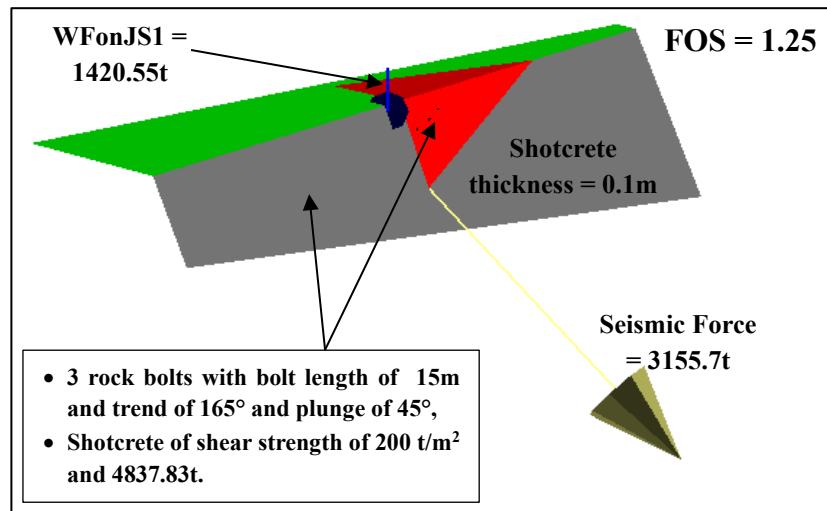


Figure 5. 29. Countermeasures for Wedge Mode Failure and Factor of Safety (FOS) include addressing the slope section on the left abutment (LAS) due to the intersection of JS1 and JS2.

Again, to stabilize the wedge mode of failure due to the intersection JS2 and JS3 at the left abutment slope section (LAS), about 3 (three) rock bolts were applied to the slope section where each one of them have trend of 165° and plunge of 45°, bolt length of 15m, bolt capacity of 2000t. Additional to the rock bolts, the shotcrete was also implemented stabilize the slope section with thickness of 0.1m and shear strength of 150t/m² which is expected to apply the shotcrete force of 2323.45tons. These remedial measures increased the factor of safety from 0.40 to 1.27 with the resisting force and driving force of 7716.28t and 6071.1t respectively as shown in Table 5.32 and Figure 5.30.

Table 5. 32. Developed remedies for addressing wedge mode failure at the left abutment slope section (LAS) caused by JS2 and JS3, as well as an enhanced factor of safety.

Slope Section	FOS without support	Bolt No.	Orientation		Bolt Properties		
			Trend (°)	Plunge (°)	Bolt Length (m)	Anchored Length (m)	Capacity (tons)
LAS (JS2 and JS3)	0.40	1.	165	45	15	8.22	2000
		2.	165	45	15	7.21	2000
		3.	165	45	15	8.53	2000
		Shotcrete Properties					
		Thickness(m)	Shear strength(t/m ²)			Shotcrete Force (t)	
		0.1	150			2323.45	
FOS with support			Resisting Force (t)			Driving Force(t)	
1.27			7716.28			6071.1	
FOS- Factor of Safety.							

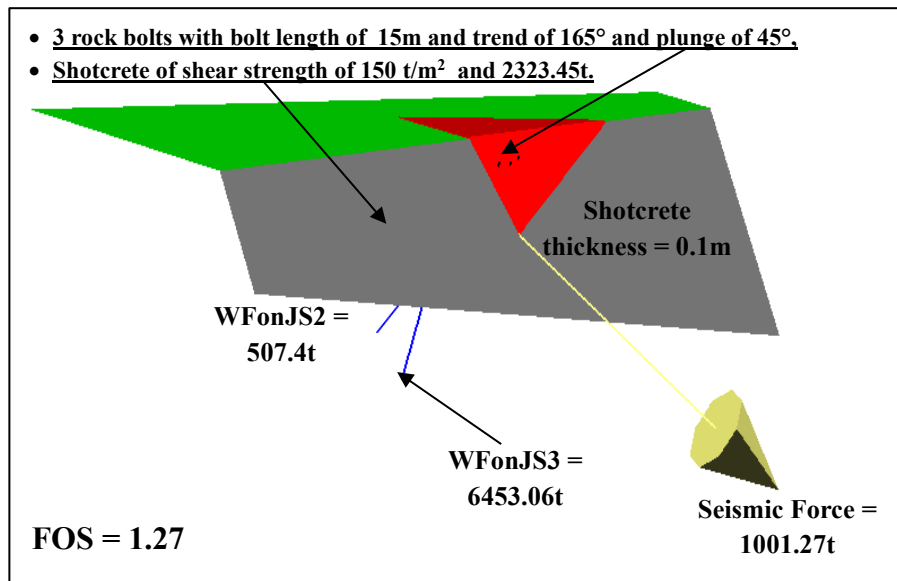


Figure 5. 30. Countermeasures for Wedge Mode Failure and Factor of Safety (FOS) include addressing the slope section on the left abutment (LAS) due to the intersection of JS2 and JS3.

Similar to the left abutment slope section (LAS), to stabilize the wedge mode of failure due to the intersection JS1 and JS2 at the right abutment slope section (RAS), about 3 (three) rock bolts were applied to the slope section where each one of them have trend of 90° and plunge of 45°, bolt length of 15m, bolt capacity of 8000t. Additional to the rock bolts, the shotcrete was also implemented stabilize the slope section with thickness of 0.1m and shear strength of 300t/m² which is expected to apply the shotcrete force of 2258.2tons. These remedial measures increased the factor of safety from 0.086 to 1.256 with the resisting force and driving force of 15598.1t and 12417.2t respectively as shown in Table 5.33 and Figure 5.31.

Table 5. 33. Developed remedies for addressing wedge mode failure at the right abutment slope section (RAS) caused by JS1 and JS2, as well as an enhanced factor of safety.

Slope Section	FOS without support	Bolt No.	Orientation		Bolt Properties		
			Trend (°)	Plunge (°)	Bolt Length (m)	Anchored Length (m)	Capacity (tons)
RAS (JS1 and JS2)	0.086	1.	90	45	10	1.25	8000
		2.	90	45	10	6.07	8000
		3.	90	45	10	6.44	8000
		Shotcrete Properties					
		Thickness(m)	Shear strength(t/m ²)			Shotcrete Force (t)	
		0.1	300			2258.2	
FOS with support			Resisting Force (t)			Driving Force(t)	
1.256			15598.1			12417.2	

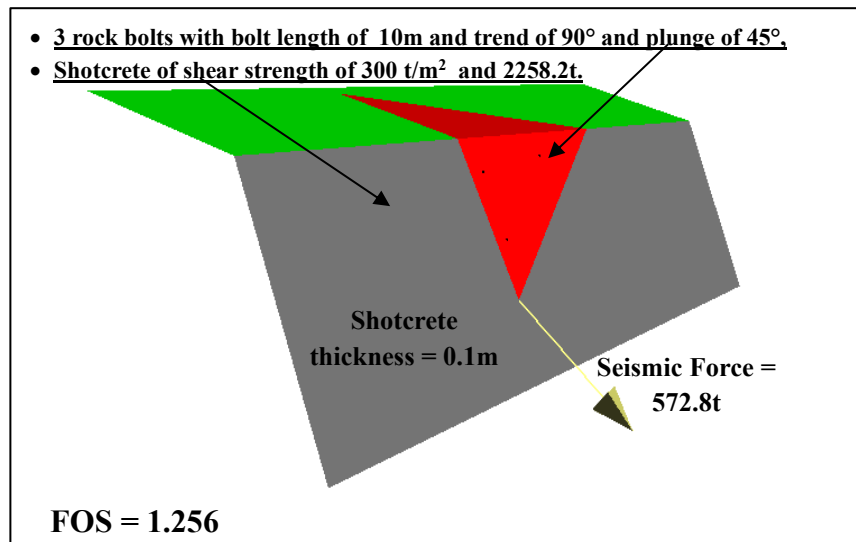


Figure 5. 31. Countermeasures for Wedge Mode Failure and Factor of Safety (FOS) include addressing the slope section on the right abutment (RAS) due to the intersection of JS1 and JS2.

CHAPTER SIX

6. CONCLUSIONS AND RECOMMENDATIONS

6.1. Conclusions

- The geological mapping of the study area entails that area constitutes residual soils, ignimbrite rock, unwelded tuff and welded tuff.
- From the discontinuity surveying, from three major joint sets two joints have orientation favourable for leakage of water due to the joints are in parallel with the direction of river flow.
- The soil at the dam site and reservoir area with thickness of 1.50m at the reservoir area are mostly covered with clayey silt and sandy silt with medium to high plasticity and semi-pervious permeability class. These permeability values show the soil deposits area suitable for the impoundment of water at the dam site and reservoir area with additional investigation for the reservoir area.
- The surface and subsurface engineering geological mapping indicated that the dam site constituting rocks are ignimbrite rock, volcanic ash, scoriaceous basaltic rock, amygdaloidal basaltic rock, unwelded to welded tuff and aphanitic basaltic rock.
- The RQD estimated using volumetric joint count shows the rock mass at the right and left abutment have values ranging from 36%-79% with poor - fair rock mass quality and 39% - 85% with poor - good to fair rock mass quality, respectively.
- The rock mass quality at the right and left abutment of the dam site was fair rock mass class with RMR value was 55 and fair rock mass class with RMR value of 59. The GSI values revealed that the rock outcrop at the dam site is blocky – very blocky rock masses.
- At the dam site, intact rock strength test indicated that the uniaxial compressive strength value ranges from 8.97MPa to 22.87MPa and the UCS estimated from the in-situ strength test shows a range of 24.15MPa to 33.88MPa showing the rock mass constituting dam site have very low to low strength rock mass.
- The evaluation core drilling revealed that RQD shows a clear trend of decreasing with increasing drilled depth at both left abutment, and intake conduit boreholes. However, there is a positive correlation of RQD variation with drilled borehole depth in right abutment.

- The water pressure test revealed that there is potential for the occurrence of leakage of water at the dam site in right and intake conduit boreholes with Lu value of greater than 1 and further shows a negatively influence of the dam's safety and stability.
- The ultimate and allowable bearing capacity of ignimbrite rock at the dam site (left, intake conduit and right abutment) ranges from 2.28MPa to 31.66MPa and 0.76MPa to 10.55MPa, respectively. The ultimate and allowable bearing capacity of welded tuff rock at the dam site (left, intake conduit and right abutment) ranges from 0.39MPa to 5.158MPa and 0.13MPa to 1.72MPa, respectively. The ultimate and allowable bearing capacity of unwelded tuff rock at the dam site (left and right abutment) ranges from 0.017MPa to 2.09MPa and 0.0057MPa to 0.69MPa, respectively. The ultimate and allowable bearing capacity of vesicular basaltic rock at the dam site (left and right abutment) ranges from 0.15MPa to 1.74MPa and 0.05MPa to 0.58MPa, respectively. The ultimate and allowable bearing capacity of amygdaloidal basaltic rock at the dam site (left and right abutment) ranges from 1.64MPa and 0.55MPa, respectively. The ultimate and allowable bearing capacity of aphanitic basaltic rock at the dam site (left and right abutment) ranges from 48.34MPa and 16.11MPa, respectively.
- The kinematic analysis outcomes showed that the slope sections of the dam site were potentially unstable via planar mode of failure and wedge mode of failure for left abutment slope section and right abutment slope section.
- The limit equilibrium method using deterministic slope stability analysis method result showed that at the left and right abutment slope section, the slope was unstable in saturated static and dynamic conditions. Therefore, water pressure, failure plane angle, and slope angle significantly influenced the destabilizing of these slope sections in contrast to the seismic coefficient.
- The possible remedial measures to stabilize the planar and wedge mode failure resulted in the increased the factor of safety to be greater than 1.20.

6.2. Recommendations

- Packer test results and falling head permeability strong and weak, and determined bearing capacity rocks revealed the potential for leakage and differential settlement along the foundation of the dam axis respectively. Therefore, ground improvement or treatment is required mainly in terms of curtain grouting along the foundation of the dam axis.
- RQD should not be utilized as criteria for deciding test sections for packer test during detailed design stages and construction phase of the dam as it showed no clear trend with Lugeon value and depth.
- The geological investigation, discontinuity survey, geotechnical borehole drilling and permeability tests have all shown that the dam site is characterized by complex geological and structural setup and therefore it requires close monitoring throughout detailed design stages and construction phases.
- The dam site has low bearing capacity considering the geotechnical units at the dam site. Therefore, intensive, and adequate foundation design will be necessary to ensure the dam's stability and safety over the long-term.
- Due to time and financial constraints, this investigation concentrated solely on surface soil investigation, with soil samples collected from 1m to 2m. The soils of the reservoir area should be further studied in detail if they are planned for construction purposes.
- Considering the reservoir area is covered by clayey silt soil which could cause siltation of the reservoir. Hence this research work recommended further assessment of reservoir issues in order to enhance the operational life of the dam and reservoir.
- This study recommends that additional evaluation for the design and implementation of remedial measures such as rock bolts since for the unstable slope sections, stabilization of the slope section resulted in the increase of factor of safety.
- The study further recommends the assessment of the effect of surface and subsurface water on the dam abutment slope stability conditions.

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APPENDIXES

Appendix 1: Rock Rating System (After Bieniawski, 1989)

Parameters		Ranges of Values						
Strength of intact material	PLSI (MPa)	>10	4-10	2-4	1-2	For the low range, uniaxial compression test is preferred		
	UCS (MPa)	>250	100-250	50-100	25-50	5-25	1-5	<1
Rating		15	12	7	4	2	1	0
(RQD) (%)		90-100	75-90	50-75	25-50	<25		
Rating		20	17	13	8	3		
Spacing of Discontinuities		>2m	0.6-2m	200-600mm	60-200mm	<60mm		
Rating		20	15	10	8	5		
Condition of Discontinuities		Very rough surfaces	Slightly rough surfaces	Slightly rough surfaces	Slickensided surfaces	Soft gouge >5mm thick		
		Not continuous (No separation)	Separation < 1mm	Separation < 1mm	Gouge < 5mm thick Separation 1-5mm, continuous	Separation >5mm		
		Unweathered wall rock	Slightly weathered walls	Highly weathered walls				
Rating		30	25	20	10	0		
Groundwater condition		Completely dry	Damp	Wet	Dripping	Flowing		
Rating		15	10	7	4	0		
RMR rating		100 - 81	80 - 61	60 - 41	40 - 21	< 21		
Rock Class		I	II	III	IV	V		
Classification		Very good rock	Good rock	Fair rock	Poor rock	Very poor rock		

Appendix 2: Unit Weight Laboratory test data sheets.

Sample Station (RS-ID)	RAS (RA-1)	LAS (LA-1)	RB-01
Mass of dry rock sample (gm)	156.60	201.50	309.50
Mass of saturated rock sample (gm)	158.60	203.60	311.00
Mass of water (gm)	2.0	2.10	1.50
Water Initial Volume (cm ³)	400	350	400
Water Final Volume (cm ³)	500	450	600
Volume of rock sample (cm ³)	100	100	200
Saturated density(g/ cm ³)	1.586	2.036	1.555
Saturated unit weight, (kN/m ³)	15.55	19.97	15.25
Dry Density (g/ cm ³)	1.566	2.015	1.548
Dry unit weight, (kN/m ³)	15.36	19.77	15.18
RB = River bed rock Sampling Station, RAS = Right Abutment Rock Sample, LAS = Left Abutment Rock Sample			

Appendix 3: Classification of UCS of rock according to Deere and Miller (1966)

Unconfined compressive strength (UCS) (MPa)	Description of strength
Over 224	Very High strength
112-224	High strength
56-112	Medium strength
28-56	Low strength
Less than 28	Very low strength

Appendix 4: Coefficient of permeability of soils according to (USBR, 1987)

coefficient of permeability(mm/sec)	Description
Greater than 10 ⁻³	Previous
10 ⁻⁵ -10 ⁻³	Semi-previous
Less than 10 ⁻⁵	impervious

Appendix 5: Natural water content determination test

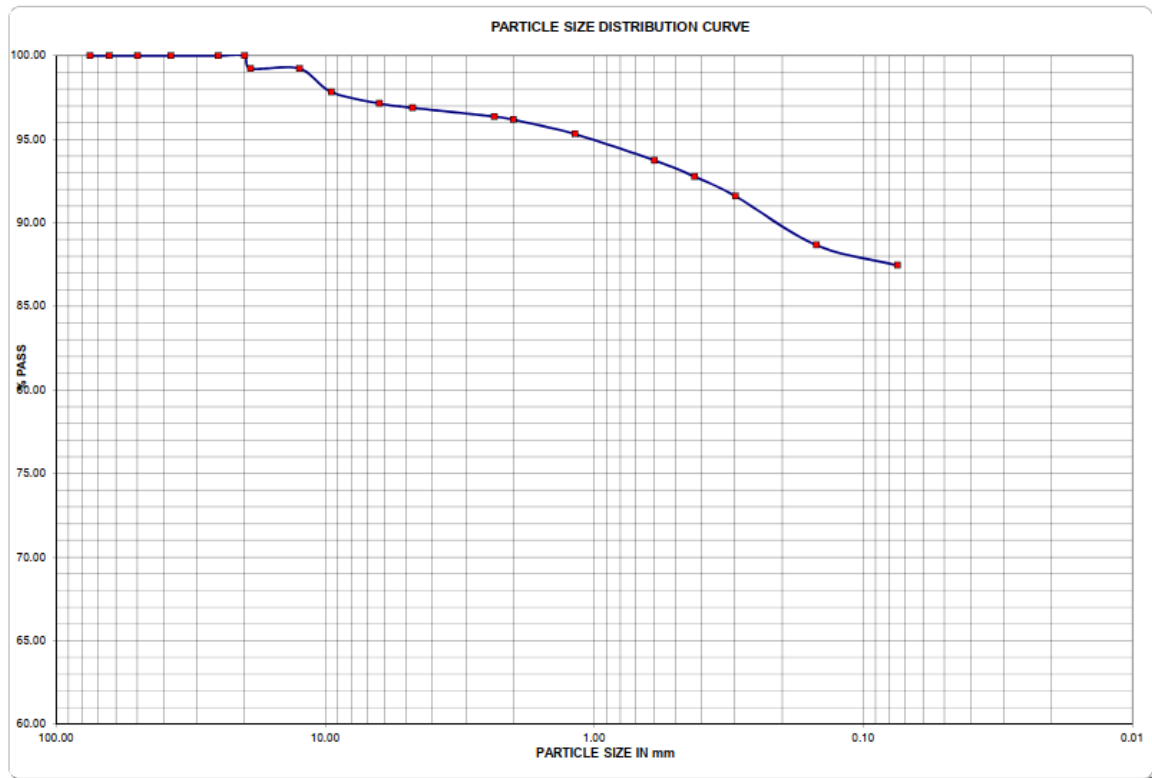
BH-ID	-
Sample depth (m)	-
Can No.	Gs
Wt.of cont. + wet soil (gm)	157.105
Wt.of cont. + dry soil (gm)	122.154
Wt. of water (gm)	34.951
Wt. container (gm)	22.144
Wt. dry soil (gm)	100.01
Wt. wet soil (gm)	134.961
Wt. dry soil (gm)	100.01
Moisture content (%)	34.95
NMC (%)	35

Appendix 6: Specific Gravity determination test

 ጀ.ዩ ኢ.ንጂነሪንግ ሥላ. የተወ. የግል ማህ. Engineering Private Limited Co. Geotechnical Engineering Service, Mobile No: +251 - 912 01 60 24 / +251 - 921 33 12 51, E-mail: gilaengineering@ymail.com		
Project	-	
Client	Dawit	
Location	-	
[A] Calibration of pycnometer		
Pycnometer No.	P5	P6
Weight of dry, clean pycnometer, w_p (g)	38.237	35.93
Weight of pycnometer + water, w_{pw} (g)	138.45	137.363
Observed temperature of water, T_i (°C)	20	20
[B] Specific Gravity Determination		
Depth (m):		
Determination No.	1	2
Pycnometer No.	P5	P6
Weight of pycnometer + soil + water, W_{pws} (g)	144.278	143.254
Temperature, T_x (°C)	20	20
Weight of pycnometer + water at T_x , $W_{pw}(at T_x)$ (g)	138.21	137.12
Weight of dry soil, w_s (gm)	10	10
Conversion factor, K	1	1
Specific gravity of soil at 20°C.	2.55	2.59
Average specific gravity of soil	2.57	
Reported by	Approved by	
_____	_____	

Appendix 7: Particle size distribution determination test

PARTICLE SIZE DISTRIBUTION REPORT



SIEVE SIZE (MM.)	% PASSED
75.00	100.00
63.50	100.00
50.00	100.00
37.50	100.00
25.00	100.00
20.00	100.00
19.00	99.23
12.50	99.23
9.50	97.83
6.30	97.14
4.75	96.89
2.36	96.36
2.00	96.17
1.18	95.31
0.60	93.75
0.425	92.77
0.30	91.61
0.15	88.67
0.075	87.46

Soil Description

Elastic Silt

Atterberg Limit

LL 53.7

PI 24

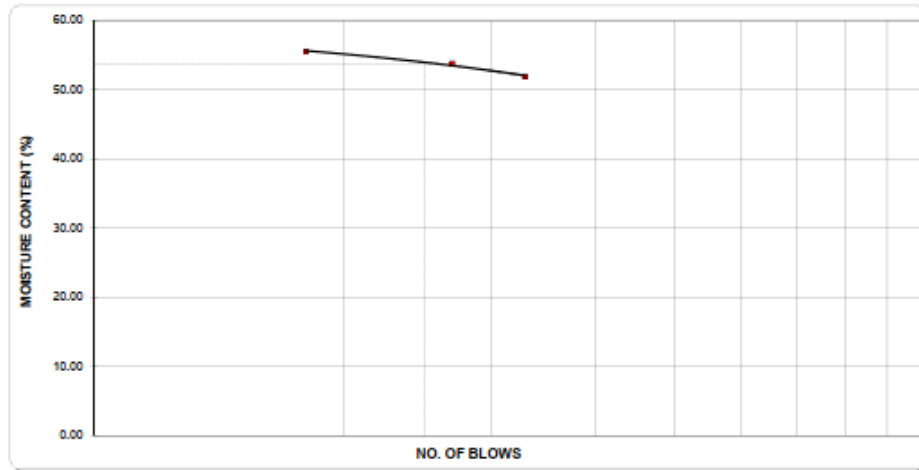
Classification

USCS MH

% COBBLE	% GRAVEL	% SAND	% FINE
-	3.11	9.43	87.46

Appendix 8: Atterberg limit test

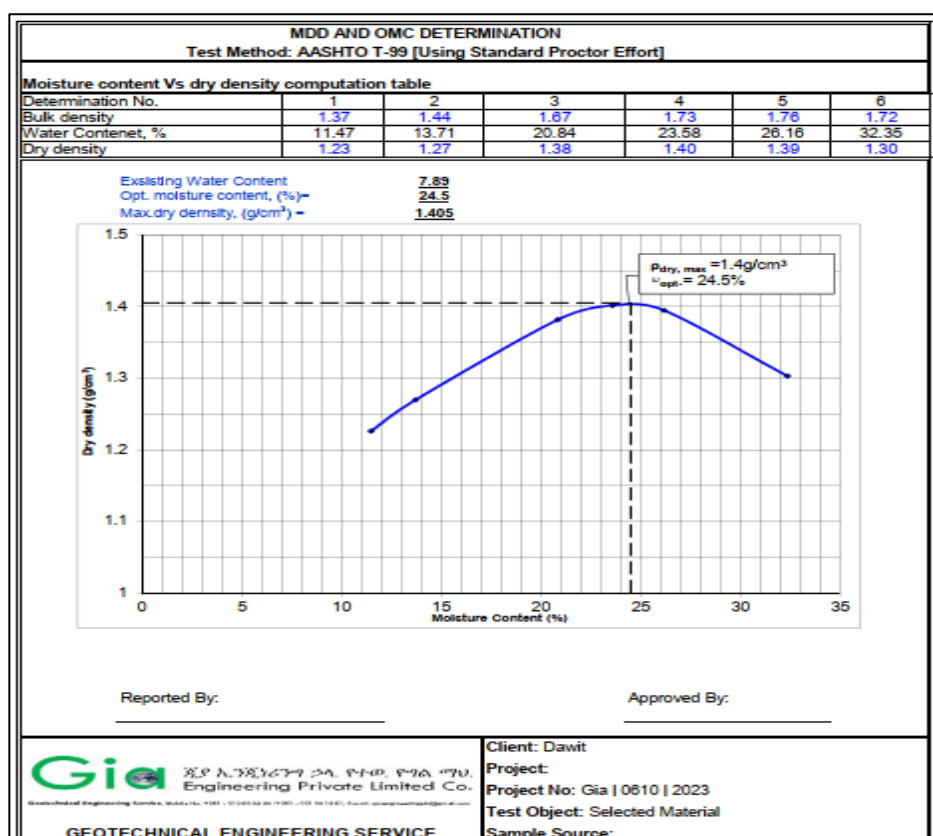
LIQUID AND PLASTIC LIMITS TEST REPORT



No. Blows	LIQUID LIMIT			PLASTIC LIMIT	
	33	27	18		
Wt. wet soil (g.)	18.44	18.56	19.05	2.92	2.21
Wt. dry soil (g.)	12.15	12.07	12.25	2.25	1.70
Moisture content (%)	51.85	53.73	55.49	29.54	29.46
				AV. PL (%)	29.5

Liquid Limit LL(%)	Plastic Limit PL(%)	Plasticity Index PI	WET SEIVE ANALYSIS, % PASS		
			4.75mm	2.00mm	0.075mm
53.74	30	24	96.89	96.17	87.46

Appendix 9: MDD and OMC Determination



Appendix 10: Soil Permeability test (Falling head test)

PERMEABILITY TEST REPORT (FALLING HEAD TEST)										
No	Sample ID	MDD (gm/cm ³)	OMC (%)	Stand Pipe Area, a (cm ²)	Initial Head, h _i	Final Head, h _f	Elapsed Time, t (Sec)	Sample Length, L (cm)	Sample Area, A (cm ²)	Permeability, k (cm/sec)
1	-	1.4	24.5	1.131	92	47	66000	4	30	1.63E-06
2										
3										
4										
REPORTED BY: _____ APPROVED BY: _____										
Gia ጂያ ኢንጅነሪንግ ፓላ. የተወ. የግል ማህ. Engineering Private Limited Co. Geotechnical Engineering Service, Addis Ababa, Ethiopia. Tel: +251 11 225 02 34 / 11 225 11 12, Email: giageoengineer@gmail.com GEOTECHNICAL ENGINEERING & MATERIAL TEST SERVICE Tel: +251 118 886343 +251 912 036024 +251 921 331251; E-mail: giageoengineerplc@gmail.com ; Addis Ababa, Ethiopia					Client: Dawit Consultant: - Project: - Sample Type: Disturbed (Remolded) Soil Project No: Gia 0610/2023 Sample Source: Refer Table Above Test Type: Permeability (Hydraulic Conductivity) Ordered on: 1/05/2023 Reported on: 16/05/2023					

Appendix 11: Degree of Jointing and Weathering with respect to the RQD value according to Deere and Deere (1988).

RQD values (%)	Degree of Jointing and Weathering
10-50	Weathered and Jointed bedrock
<50	Highly jointed
50-75	Moderately jointed
>75	Relatively sound to sound

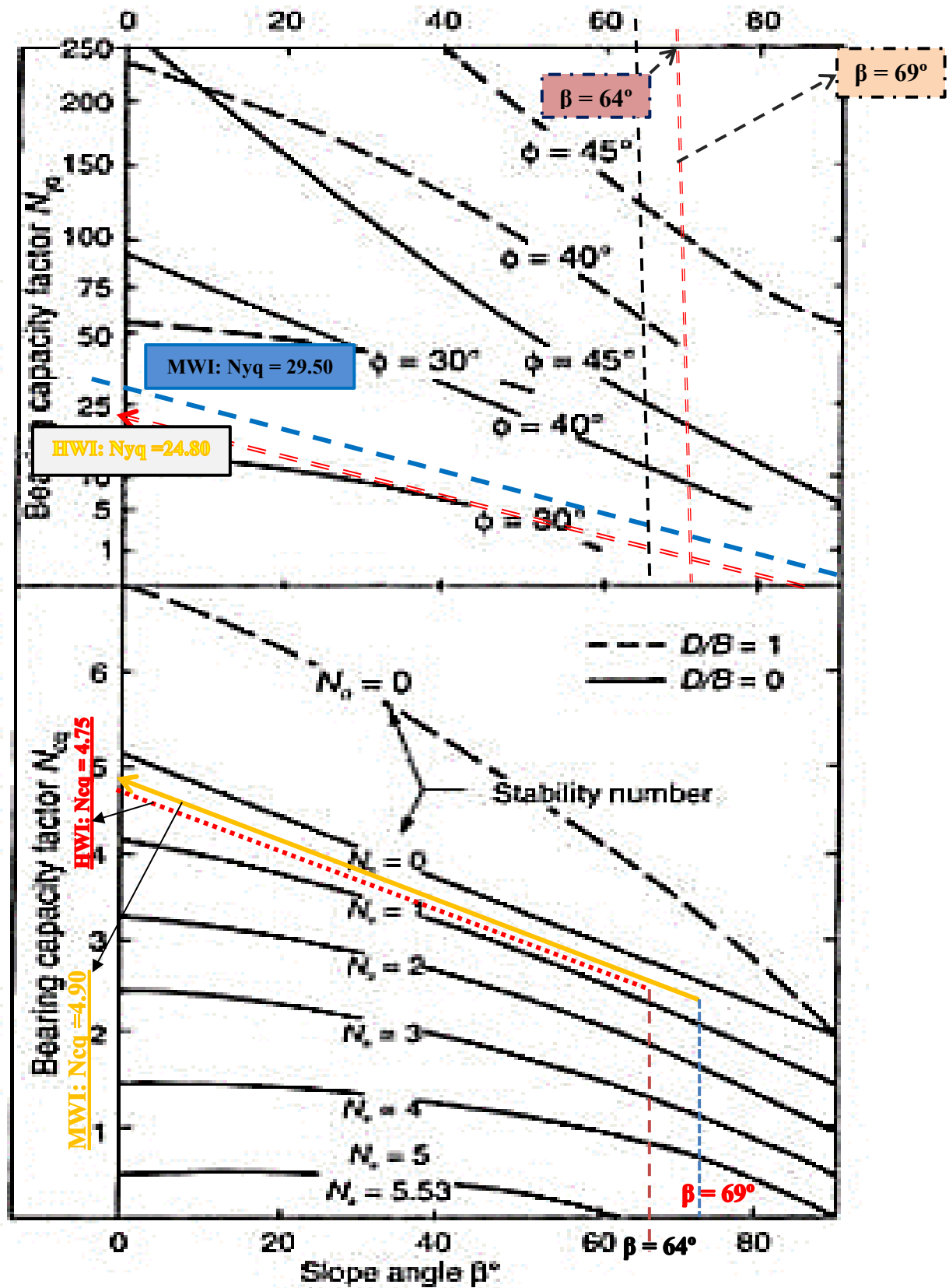
Appendix 12: Laboratory test standard applied for the soil samples.

S.No.	Soil Laboratory tests		Standard Test Method
1.	Water Content Determination		ASTM D2216 (1998)
2.	Specific Gravity		ASTM D854 (1998)
3.	Grain Size Analysis	Sieve Analysis	ASTM D422 (1998)
		Hydrometer Analysis	
4.	Atterberg Limits (Consistency Limits)		ASTM D4318 (1998)
5.	Standard compaction test		ASTM D698 (1998)
6.	Permeability test: Falling Head permeability test		ASTM D2434 (1994)

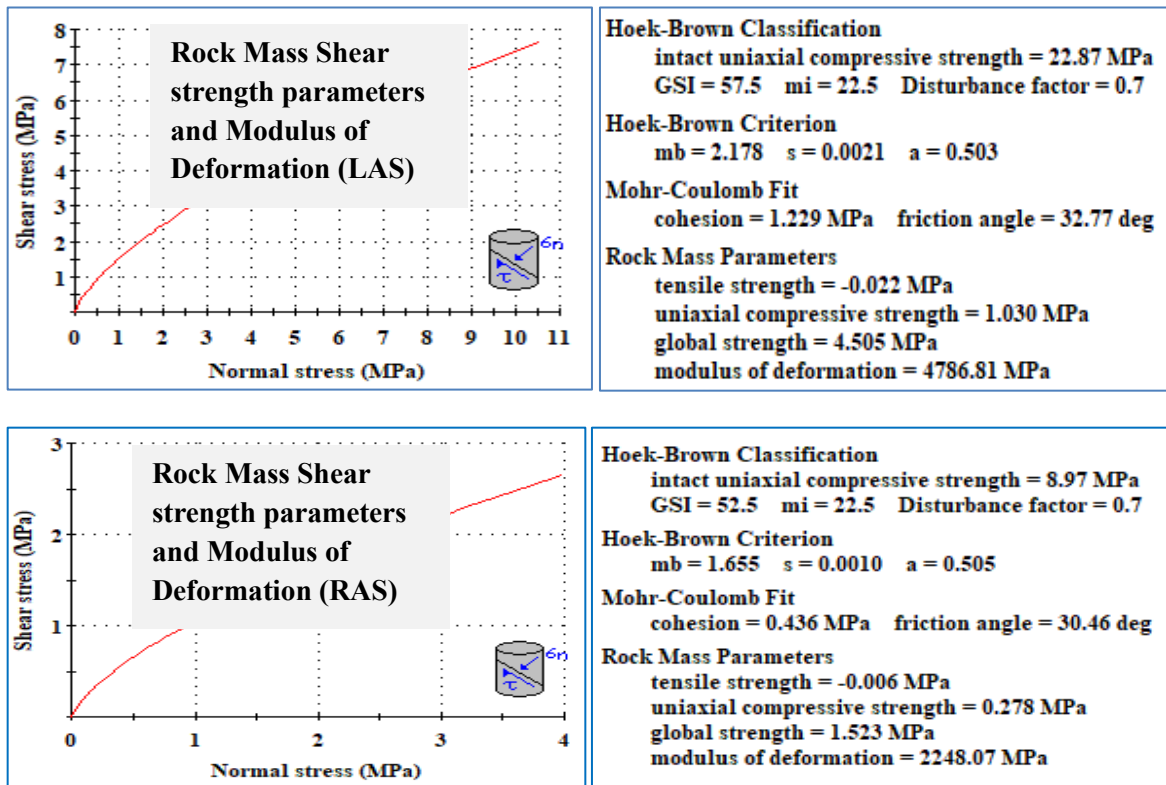
Appendix 13: Laboratory test standard applied for the soil samples.

S.No.	The Conducted Rock Laboratory Test Types	Standard Followed
1.	Point Load Index Strength test	ASTM D 5731 (1995)
2.	Unit Weight	ISRM (1978)
3.	Water Absorption	ASTM D6473-10 (2010)

Appendix 14: Determined bearing capacity factors according to Bell (2007).

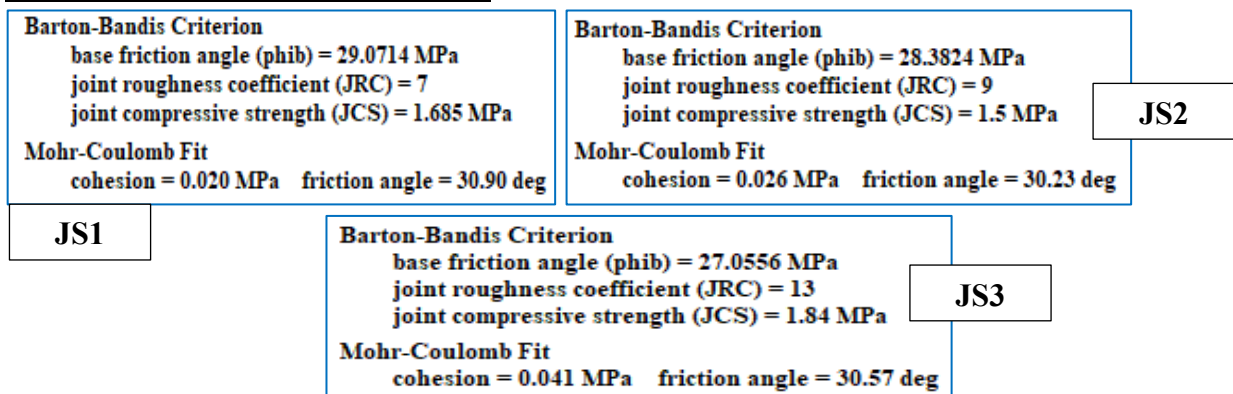


Appendix 15: Analysis of shear strength parameters of rock mass and rock strength using RocData software according to Hoek-Brown Classification.

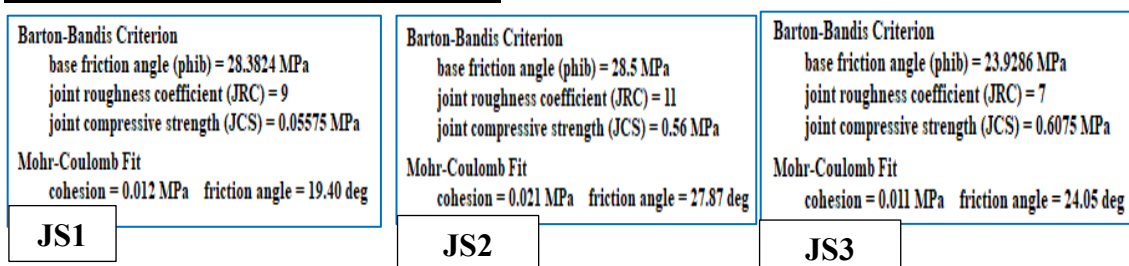


Appendix 16: Analysis of shear strength of joints using RocData software according to Barton-Bandis failure criteria.

Left Abutment Slope Section (LAS)



Right Abutment slope section (RAS)



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