



ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

ADDIS ABABA INSTITUTE OF TECHNOLOGY
DEPARTMENT OF CIVIL AND ENVIRONMENTAL
ENGINEERING

**EVALUATING THE GEO-MECHANICAL PROPERTIES OF
THE JIMMA VOLCANIC AND MAKKONNEN BASALTIC
ROCK FORMATIONS: A SPECIFIC GUIDELINE FOR USE
IN THE CONSTRUCTION INDUSTRY**

(A CASE STUDY ON WABE HYDROPOWER PROJECT, ETHIOPIA)

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**A THESIS SUBMITTED TO THE SCHOOL OF GRADUATE STUDIES
OF ADDIS ABABA UNIVERSITY IN PARTIAL FULFILMENT OF THE
REQUIREMENT FOR THE DEGREE OF MASTER OF SCIENCE IN CIVIL
ENGINEERING (GEOTECHNICAL ENGINEERING)**

JUNE, 2020



Addis Ababa University

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Declaration

I hereby declare that this thesis is my original work under the supervision of Yodit Ayallew Asmare (Ph.D.) submitted to the School of Graduate Studies of Addis Ababa University in Partial Fulfilment of the Requirement for the Degree of Master of Science in Civil Engineering (Geotechnical Engineering). I further declare that this work has not been submitted to any other institutions for the award of any degree or diploma and all sources of materials used for the thesis have dully acknowledged.

Mihret Wujira

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2020

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Abstract

In the present study, the engineering properties of the Jimma volcanic and Makkonnen basaltic rock formations located in the south-western part of Ethiopia in Wolkite Town are assessed. It is aimed to propose a guideline useful in the early stages of site investigation. A systematic methodology is followed which includes review of secondary reports on investigations and design and; collection of primary data from subsurface and surface outcrops of the geological formation.

The physical and mechanical properties of rock materials such as water absorption, specific gravity, porosity, flakiness index (FI) and elongation index (EI), Uniaxial Compressive Strength (UCS), Aggregate Crushing Value (ACV), Aggregate Impact Value (AIV) and Los Angeles Abrasion (LAAV) have been determined with reference to the various specifications and standards (AASHTO, ASTM and BS) in terms of suitability in the construction industry. WA value ranges from 0.2-5.52%, porosity values range from 1.17-13%, SG values range from 2.02-2.92%, FI and EI values range from 5.26-32.24% and 14.65-22.57% respectively; UCS value ranges from 1.37-237.29MPa, ACV value ranges from 16.23-20.33%; AIV values range from 10.02-12% and LAAV values range from 24.32-29%.

The data analysis from the geological logs and the seismic refraction survey suggests that the bedrock is located at a deeper depth represented by fresh, massive and slightly fractured basalt of high quality (RMR>65) represented by high P-wave velocity ($V_p > 5000\text{m/s}$). Some very soft tuff rocks and heavily jointed media have $V_p < 1500\text{m/s}$ and $\text{RMR} < 35$. The majority of the rock mass can be classified as an excellent foundation material.

The drilled boreholes are also checked for in-situ rock mass permeability conditions using packer test. According to the test results the test sections have a very low to medium permeability Lugeon value ranging from 0.03-22.8. The relatively high lugeon values are obtained at BH-07 (21 and 22.8) which is located at the right abutment of the dam site

The rock mass along the tunnel section is divided into a number of geological structural units in order to determine the RMR and support requirement. Accordingly, GC1 (geotechnical class one) has an RMR of 85 and categorized under a very good rock mass class; GC2 (geotechnical class two) has an RMR of 71 and categorized under a good rock mass class; GC3 has an RMR of 81

and categorized under a very good rock mass class; GC4 has an RMR of 66 and categorized under a good rock mass class and GC5 has an RMR of 13 and categorized under a very poor rock mass class.

Finally, on the basis of the findings of the present research, a specific guideline is proposed based on rock index property tests and geotechnical parameters. Initially, the rock mass is divided into five rock grades (A, B, C, D and E) using weathering (alteration), observed number of joint, joint spacing together with RQD as a judgment criterion. A rock mass classification guideline is then proposed based on the combination of the geotechnical parameters that is based on the overall rock mass quality and the intensity of P-wave velocity for the mapped geological units.

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Abbreviations

AASHTO-American Association of Static Highway and Transportations Officials

ACV-Aggregate Crushing Value

AIV-Aggregate Impact Value

ASL-Above sea level

ASTM-American Society for Testing and Material

BH-Borehole

BS-British standards

C-Cohesion

D-Disturbance Factor

D_e-Equivalent Dimension

ECDSWCo. -Ethiopian Construction Design and Supervision works Cooperation

E_a-Dynamic Elastic Modulus

ENE-East- North-East

ESR-Excavation Support Ratio

FAO-Food and Agriculture Organization

GSI-Geological Strength Index

GSE-Geological survey of Ethiopia

HR-Hammer Rebound

ISO-International Organizational for Standardization

ISRM- International society of Rock Mechanics

LAHV-Los Angeles Abrasion Value

LU-Lugeon Value

M_b-Material Constant

Mc-Mohr Coulomb

NEH-National engineering Handbook

NNE-North-North-East

NRCS-Natural resources Conservation services

NW-North-West

PH-Potential of Hydrogen

Q_i-Quality index

RMR-Rock Mass Rating

RQD-Rock Quality Designation

RSR-Rock Structure Rating

SE-South-East

SHV-Schmidt Hammer Rebound value

SMR-Slop Mass Rating

SNNPR-Southern Nations Nationalities and People Regional State

SRF-Stress Reduction Factor

UCS-Unconfined Compressive Strength

UTM-Universal Traverse Mercator

V_p-p-Wave velocity

V_s -S-Wave Velocity

V_t -Theoretical velocity

WNW-West-North-West

σ_1 - Major principal stress

σ_2 - Intermediate principal stress

σ_3 - Minor principal stress

σ_{ci} - Uniaxial Compressive strength of the intact rock material

E_d -Deformation modulus

P-Density

γ -Dry Rock Density

CHAPTER ONE

1. Introduction

1.1. General Insight

This is an MSc research conducted on assessing the geo-mechanical properties of rocks at a preliminary geotechnical investigation stage or phase on a geologic formation, prevailing at a site of a local hydropower project. The overall assessment process in general requires recognizing the basic difference between rock material and rock mass engineering properties and producing a guideline for future use based on the parameters obtained.

The systematic and in-depth characterization and classification of rocks for engineering purposes is a major part of geotechnical investigations of major projects. Having a site specific geo-mechanical property of rocks is mandatory for engineering structures, but having a general geo-mechanical property of rocks (guideline) for major geological formations will give us the general idea of how the geology and the intended civil engineering structure will behave with each other. This is very valuable especially in the early stages of site investigation where there are very limited to none geotechnical data for deciding the successive site investigation stages. The presence of such data will considerably decrease the time and cost of the geotechnical investigation which in turn will reduce the cost of the general project.

However, in Ethiopia, we have extremely scarce guidelines in terms of geo-mechanical or geotechnical properties of specific geological formations that has led the preliminary stages ineffective that it required more attention. Some of the reasons include:

- Lack of proper specific guidelines that can provide precise information of the intended material to be used.
- Adopting other countries design scheme instead of creating one's own.
- Inaccuracy in fulfilling the requirements intended in initial stages of investigation process in construction.

The term “specific” in this research context refers in general to the guideline over which the particular properties of the formation are specifically discussed and engineering properties of the rock mass as well as the rock material depending on the geological, weather, hydrogeologic and likely conditions of the locations are assessed.

Engineering properties of rocks include all those properties of rocks that are relevant to engineering application either at in situ or processed state. The first state includes all those qualities of a natural bed rock as and where it exists so as to determine its suitability as a construction site for a certain proposed engineering project. The second state include all those properties for which a rock must be tested for selection as a material for construction such as a building stone, road stone or aggregate for concrete making.

The determination of the engineering properties of rocks basically requires activities such as field observation, laboratory testing of samples and empirical analysis.

Rock classification in general is a product of rock characterization, which provides a basis for understanding the characteristic behavior and relates to experiences gained in rock conditions at one site to another. Rock deformation and strength properties can also be estimated alongside with rock classification schemes.

The importance of rock characterization and classification at early stages of any construction process is not limited to the rock mass, rock materials could also be characterized when assessing a site for any engineering purpose and using rocks as a construction material.

For nearly any engineering structure built upon a rock mass or incase of the rock to be used as a construction material, the behavior of the rock mass under the applied loads will have important implications upon both the construction costs and operation of the structure and may even dictate the feasibility of the entire project.

In light of this, the present study evaluates the geo-mechanical properties on two of the prominent volcanic formations in Ethiopia in terms of occurrence, i.e., the Jimma volcanic and Makkonnen basaltic rock formations to be used as a general guideline for engineering projects for early site

investigations stages such as preliminary and pre-feasibility phases wherever the formations exist in Ethiopia with the proper precaution.

1.2. Problem Statement

In Geotechnics, the characteristics and behavior of various soils has been covered to much better extent especially from deformation and strength point of view. While for rock, a prominent geotechnical material, the process of detailed investigation and characterization has not gone as expected. This might partly be due to the insufficient information available.

In theory, rocks have the strength and capability of withstanding loads much greater than soils, especially in considering large structures, though the capacity depends on the characteristics of the rock mass. The proper determination of geotechnical parameters for design requires a detailed characterization of the rock mass as well as the rock material. Intact rock characteristics which are usually employed under a confined condition are way different from mass characteristics which the overall rock property will eventually depend on.

Unlike in soil, characterization of rocks usually requires drilled boreholes at larger depth and integration of the data obtained with surface outcrops and seismic refraction data. In case of large structure schemes like tunnels, caverns, dams and powerhouses, rock characterization is essential for the long-life span of the project and for proper selection of suitable construction materials.

1.3. Previous Works

In order to conduct the intended study at the proposed project site, previous works related to the formation have been reviewed. Richard Woodroffe & Associates (1993) have indicated that the study area has previously been mapped by the Omo-Gibe River Basin Development Master Plan. Accordingly, the project area is covered by Cenozoic rocks of the Jimma Volcanics, Makkonnen Basalt and Tarmaber-Megezez Formation. A one to quarter million (1:250,000 scaled) geological map developed by the Geological Survey of Ethiopia (GSE) as part of the geological map of Akaki map sheet (Beshawered, 2012) has also indicated that the study area is well represented by Cenozoic rocks of welded to partially welded pyroclastic flows (referred to as Nazret Series), and Tarmaber Megezez Formation.

The regional geology of the study area as well as the surrounding have been best represented by the above studies. But there exists a limitation in obtaining a well-organized data regarding the engineering geological and structural condition and geotechnical parameters of the rock formation which can aid the intended study.

Depending on the vast dominancy of the formation throughout the country and its importance in various engineering works, there still exists a limitation in obtaining data regarding the geo-mechanical properties of the rock formation suitable in proposing a site-specific guideline that can aid the preliminary geotechnical investigation phase.

1.4. Research Objectives

1.4.1. General Objective

The main objective of this research is to evaluate the geo-mechanical properties of the Jimma volcanic and Makkonnen basaltic rock formations, for use as a technically assisting guideline for the early stages of geotechnical study or investigation of major civil engineering projects. Moreover, this study is performed in the hope that it will play its part in advancing the understanding of the formation for engineering application.

1.4.2. Specific Objectives

The specific objective of the research includes the following:

- Engineering geological characterization of the Jimma volcanic and Makkonnen basaltic rock formations.
- Geo-mechanical classification of the Jimma volcanic and Makkonnen basaltic rocks to be used in engineering structures, i.e., dams, tunnels, roads, etc.
- Increase scientific understanding of the Jimma volcanic and Makkonnen basaltic rocks for engineering use.
- To give a general insight on the processes of evaluating the properties of rocks at initial stages of a geotechnical investigation process for major works.
- Integrate borehole data with seismic refraction survey to characterize the rock mass within the project site.

- To determine the shear strength and deformability characteristics of the rock mass in the study area.
- To assess the permeability, bearing capacity and deformability characteristics of the rock mass within the project site.
- To divide the rock formation into geotechnical regions/classes based on lithology, structure, and strength properties and determine the overall RMR.
- To conduct laboratory tests on both core and aggregate rock samples and to assess its suitability as a core construction material for the locality as well as the specific project at hand.

1.5. Significance of the Research

For any civil engineering project that involves rocks either as construction or foundation materials, characterization of the rock mass plays an important role beginning from assessing the suitability of the project site up to the detailed design and actual construction stage.

When proposing a certain project of a given purpose, a comprehensive and thorough investigation of the suitability of the project site for the intended purpose is crucial. Preliminary geotechnical investigation helps in adopting proper design parameters which soon will be improved by the detailed investigation process. Both engineering and geological parameters obtained from the subsurface helps the engineer to understand the suitability of the proposed site for further investigations.

Rock mass characterization should be aided with geophysical investigations such as seismic refraction survey methodology in addition to borehole log data for better understanding of the engineering properties of subsurface material.

The present research is intended to demonstrate the importance of acquiring design parameters at early investigation stages by geotechnical engineers through rock characterization and classification. Furthermore, it intended to show how engineering parameters of rocks could be obtained from index properties and could be a relative and practical solution to the construction industry.

Eventually, the research could also be used as a practical guide and source of information for future researchers and projects where the specific formation is prevalent.

1.6. Methodology

To accomplish the aforementioned objectives discussed above systematic and thorough techniques were employed. The specific tasks performed include the following:

Throughout the study available literature dealing with the subject matter under consideration is reviewed. The reviewed sources include published and unpublished reports, text books, journal articles, previous works and case studies. The geological map of the area initially prepared for the purpose of this project is used to identify the lithology of the rock units, possible geological structures (faults and shear zones) and identify surface discontinuity as well as weathered zones.

Appropriate data necessary for the study are categorized as primary and secondary data. Secondary data incorporates seismic refraction data obtained from geophysical surveying. Primary data include core log data, water level measurement data, in-situ permeability test data, in-situ strength measurement data (Schmidt hammer test data) and representative sample test data.

A detailed rock mass classification system based on the in-situ properties of the rock mass as per Bieniawski's Rock Mass Rating (1989) rock mass classification scheme is employed. The RMR together with seismic refraction survey data is used as a tool for characterizing the rock mass within the vicinity.

The present study focuses on methods that the engineer should employ in preliminary geotechnical investigation, with empirical analysis methods being the backbone of the study. Field observation as well as inspection and laboratory analysis are the basic tools in the whole characterization process.

The assisting tools for the research include but not limited to AutoCAD, Google Earth, ArcGIS, RockLab, Geostereonet and Stereonet.

1.7. Limitation of the Study

- The present study has aimed at covering rock units located at the proposed tunnel and dam section, but due to limited data, time and finance, it only focused on small portion of the study area incorporating both sections.
- The basic constraints are also, locating surface outcrops at both sections to employ rock mass rating (RMR), locating active quarry site to employ rock material tests, insufficiency in laboratory equipment to conduct the tests and insufficiency in literature and documented data on the subject matter.

However, maximum effort is made to fill the gaps due to these limitations.

1.8. Thesis Outline

This research work is carried out in south-western part of Ethiopia where the intended project is to be constructed. The content is categorized into six chapters.

Chapter 1 briefly discusses the generalities of the research work. It includes a general insight on the whole project, problem statement, significance of the research, overall objective, methodologies used to conduct the research and limitations.

Chapter 2 reviews relevant literature in regard to both rock material and rock mass properties and characterization techniques. It also concentrates on reviewing in-situ permeability tests and rock mass bearing capacity determination at preliminary stages and few rock mass characterization methodologies adopted in different countries. This chapter reviews documents into two categories: rock material and rock mass characterization.

Chapter 3 provides an overview of the general geology and landscape of the study area; it focuses on describing the regional geology and geological structures within the study area and local geology and structures within the vicinity.

Chapter 4 addresses the major in-situ and laboratory methods and principles of investigation used in this research for characterization of both basaltic rock mass and rock material. The principles also include empirical methodologies proposed by researchers to determine rock mass bearing

capacity and in-situ permeability. It mainly discusses the principles and procedures, standards and the necessity of the physical and mechanical tests to be conducted.

Chapter 5 discusses briefly the results obtained from characterizing the basaltic rock mass and rock material. The chapter is devoted to focus on two phases. Phase-I focuses on material characterization from index and mechanical properties and inducing correlation among parameters to observe the relationship that could be obtained in circumstances where it is impossible to easily access laboratories and the cost for testing is unaffordable. Phase-II discusses rock mass characterization, specifically rock mass rating system (RMR), rock mass strength and failure criteria, deformability characteristics, in-situ permeability and bearing capacity determination. Moreover, it reviews seismic refraction data together with borehole data to characterize the rock mass.

Chapter 6 is spared to briefly summarize and conclude the research output. It also recommends some further necessary works on an unattainable problem and matters in this research work.

Chapter Two

2. Literature Review

2.1. Estimating the Engineering characteristics of rocks

2.1.1. Introduction

Subsurface engineering and construction projects require a detailed rock mass classification. This in return is a means of effective communication among geologist, laboratory and design engineers, field engineers and contractor.

The responsibility for presenting an accurate description of materials to be encountered rests within the design engineer and engineering geologists aside with conveying the contractor on the characteristics of the rocks which will influence the construction methods (Deer and Miller, 1966).

For engineer, the appropriate classification of both the rock material and the rock mass would greatly assist in making the initial evaluation of the project, and in pointing to the areas where additional engineering information must be obtained for use in arriving at the final solution of the problem (Coates, 1964).

Engineering properties of rocks are basically dealt under the field of rock mechanics, which is a sub-division of geomechanics concerned with the mechanical responses of all geological materials including soils.

As a general trend, infrastructure construction such as roads, hydro power and irrigation dams, high rise buildings, etc. is increasing in Ethiopia. All of these activities need a thorough and project specific engineering geological understanding of rocks and soils for safe, economical and, long-lasting civil engineering structures.

Rocks can be used either as building materials so that the structure will be made of rock, or a structure will be built on it or a structure will be built within the rock. A primary division of rocks based on their mode of origin or genesis is almost universal in usage (igneous, sedimentary and

metamorphic rocks), while on engineering basis, rock is often referred to as either intact or in situ (Hunt, 2007).

Intact rock refers to a block or fragment of rock free of defects, in which its hydraulic and mechanical properties are controlled by the petrographic characteristics of the material, whether in the fresh or decomposed state.

In situ rock refers to a rock mass that normally contains defects, such as fractures or cavities, which separate the mass into blocks of intact rock and control the hydraulic and mechanical properties.

To use rock in engineering application, certain properties of the rock must be assessed to reasonably predict its performance in the as-built condition. The properties of rock fall into two broad classes: rock material properties and rock mass properties.

2.1.2. Rock Material Properties

Rock material properties are properties which directly relate to the physical properties of the rock forming minerals and the type of mineral bonding.

Various techniques can be used to determine the properties, beginning with hand specimens, core sections, drill cuttings and outcroppings to qualitative procedures, simple classification tests or laboratory tests using disturbed samples (Palmstorm, 1995).

Typical classification elements include rock type, rock unit identification (mineralogy and primary porosity), hardness, strength, dry unit weight, color and particle size (NEH, 2012).

1. Rock unit identification

Rock unit is consistent in its mineralogical composition, geological structures and hydraulic properties. It is traced in the field by surface and subsurface mapping techniques.

Each rock unit at the site can be identified either by its proper formation name or by alpha numeric designation whichever is the most useful. In the stratigraphic section rock unit should be described by station elevation and position (Fig 2.1).

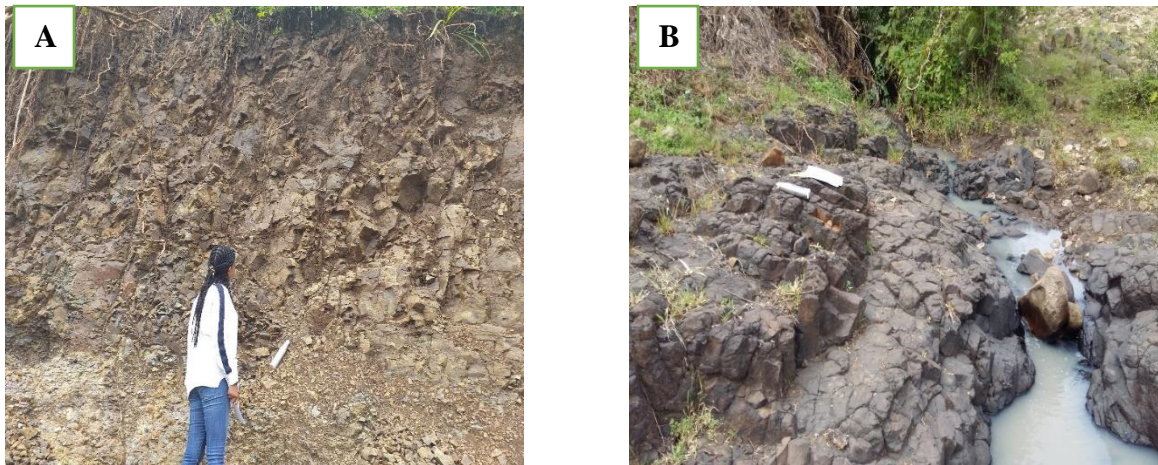


Fig 2.1: Rock units identified from (A)Cut-section (unit Tb5) and (B)outcrops (Unit Tb4) (Location: Project Site, Proposed Power house and Surge shaft section)

2. Rock Type

Rock type is a simplified geological classification of rock based on its generic category, structure, composition and grain size. Geological names and geological classifications may not always relate directly to engineering properties of rocks, but are useful for identification and correlation purposes (NEH, 2012). Besides, rock types may also indicate what processes acted on the rock during and after its formation (Fig 2.2). Rock type can indicate mineralogical and textural characteristics which may provide insight into the physical and chemical interaction between the grains.



Fig 2.2: Stratified rock of varying structure, composition and grain size (Tuff) (Zone of unconformity) (Location: Proposed project site, Proposed Spillway Section)

3. Hardness

It is a subjective description which indicates the resistance of an earth material to permanent deformation specifically by impact (indentation) or abrasion (scratching). It is simply a qualitative expression of earth material strength (Fig 2.3). The hardness of rock material is a function of the individual rock type, but may be modified (weakened) by chemical or physical weathering.



Fig 2.3: Relatively Hard and Strong Tuffaceous Rock material (Location: Project site, Proposed Spillway Section)

Hardness categories provide reasonable estimates of rock material strength for classifying earth material as rock in excavated auxiliary spillways (NEH, 2012). Therefore, care should be taken in considering the characteristics of the rock mass before reaching in to decision on alignment and location.

4. Strength

It is the ability of material to resist deformation induced by external forces; the strength of a material is the amount of applied stress at failure. Laboratory uniaxial (unconfined) compressive strength is the standard strength parameter of intact rock material (ASTM D653).

The strength of rock is influenced by the mineralogical composition, shape of grains, texture, crystallinity, stratification, and lamination, modification by heat or pressure. Besides, secondary processes such as cementation and weathering strongly influence rock strength (NEH, 2012).

Rock material strength and rock material hardness are used in classifying earth material as rock in earth spillway (NEH, 2012). According to NEH628.52, rock material strength can be reasonably estimated from the rock hardness scale without conducting a laboratory strength test.

5. Color

According to ASTM 1986, rock color is an attribute of visual perception that can be described by color names. Color is difficult to describe because a perceived color greatly depends not only the spectral power distribution of the color stimulus, but also on the size, shape, structure and envelope of the stimulus area.



Fig 2.4: Variation in color of exposed rock unit under dry and weight condition (Location: Project site, Proposed Inlet Section of the Tunnel Route)

Recording the color of each rock unit in both its wet and dry states and indicating whether the sample is fresh or in an altered condition is important since these conditions can affect color (Fig 2.4 and 2.5).

6. Particle size and texture

Particle size refers to the size of the particles that makeup a sedimentary or pyroclastic rock while texture refers to the crystallinity and granularity of igneous and crystalline metamorphic rocks (NEH, 2012).

The particle sizes used in the description of rocks for engineering purposes should be consistent with those used for soils (ASTM D422, D2488, and D653). A hand lens is usually sufficient in identifying particle size and rock texture.

The basic engineering significance behind particle size is that, bonding strength between particles that constitute rock material may determine particle size and texture with mechanical or chemical weathering of the rock.



Fig 2.5: Highly weathered basalt based on color indication (Location: Project site, left of abutment proposed Dam section)

The term classification could easily substitute characterization in the case of rock material characterization. Generally rock material characterization is more of classifying the rock material based on the physical and mechanical properties. Figure 2.6 below illustrates the physical and mechanical properties used to characterize a rock material.

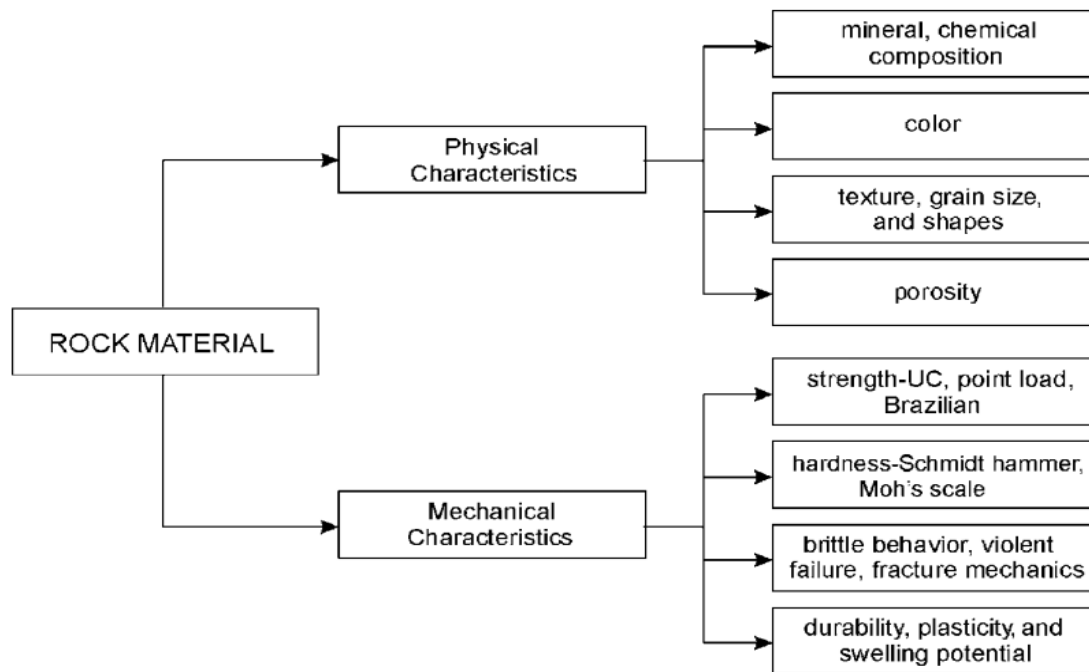


Fig 2.6: Material Characteristics of rocks

2.1.3. Rock Mass Properties

A rock mass is a material quite different from other structural materials used in civil engineering. It is heterogeneous and quite often discontinuous, but is one of the materials in the earth's crust, which is most, used in man's construction (Palmstorm, 1995).

Ideally a rock mass is composed of a system of rock blocks and fragments separated by discontinuities forming a material in which all elements behave in mutual dependence as a unit (Matula and Holzer, 1978). The material is characterized by shape and dimensions of rock blocks and fragments, by their mutual arrangement within the rock mass, as well as by joint characteristics such as joint wall conditions and possible fillings.

According to Einstein and Baecher (1982), uncertainties about geological conditions and geotechnical parameters are the basic challenges that the field is facing regarding rock mass in general. Important in all rock mechanics, rock engineering and design are the quality of the geo-data that form the basis for the calculations and estimates made.

Rock mass properties are measurable or describable lithology properties, characteristics or features of the rock mass that are evaluated on a macroscopic scale in the field. They include fractures, joints and faults (NEH, 2012).

The strength and mechanical behavior of the rock mass are commonly dominated more by the nature of its mass properties than by its material properties (NEH, 2012).

A rock mass comprised of even the strongest intact material is greatly weakened by the occurrence of closely spaced discontinuities. Material properties may also control the strength of the rock mass if discontinuities are widely spaced or if the intact rock material is inherently weak or altered. Discontinuities within a rock mass, therefore, reduce its strength and stability and reduce the energy required to excavate or erode it.

Typical classification elements in the case of rock mass properties include: Permeability, consolidation, rock texture, shearing resistance, rock structure, attitude and discontinuities (NEH, 2012).

1. Permeability

A low porosity rock mass may be highly permeable due to fractures and joints. Fractures and joints have a great influence on the permeability of a rock mass. Jointing is not restricted to any particular type of rock, but certain types of rocks may locally exhibit larger or more closely spaced joints (NEH, 2012).

Differential weathering may be found in many types of igneous and metamorphic rocks and certain sedimentary rocks. Therefore, it is important to note that the rate of permeability and the depth and direction of water movement be determined, as closely as possible to determine the requirement for many underground engineering works.

Field investigation may require angular test borings, pressure testing, use of dyes and other tracer compounds, or other methods to properly determine permeability of rocks.

2. Consolidation

The bearing strength of rock is normally adequate to support dams and other structures (designed by Natural resources conservation service (NRCS) and many other subsequent researches). Consolidation may be a problem in certain types of rocks such as weakly cemented shale's and silt stones and rocks that have been altered to clay minerals. In such cases samples of questionable materials must be obtained for laboratory analysis (NEH, 2012).

3. Rock Texture

Texture is the geometric aspects of the component particles of a rock including size, shape and spatial arrangement while in the case of unconsolidated material it is an alternate description of particle gradation (NEH, 2012).

According to NEH, it is important for field identification purposes and for predicting behavior of rock under load. Descriptions of mineral constituent, conditions of weathering, fracture system and other properties influence engineering properties.

4. Shearing Resistance

Usually problems related to shear may result from poorly cemented shale's and silt stones or highly weathered rock of low shear strength. Materials that dip in an adverse direction and are subject to saturation or unloading of toe supports by excavation are of particular concern, and this include strata dipping downstream basically in foundation and strata dipping towards the center line which is parallel to the slope of the abutment specially in auxiliary spillway excavation (NEH, 2012).

5. Rock structure

The structure of rocks includes holes, cavities, joints, bedding planes, fractures, cleavage, schistosity, lenses and other similar features. Rock structure has important influence on the geological conditions of a site and the ultimate stability and safety of an engineering structure. Besides, it is an important factor affecting the amount and direction of groundwater flow as well as actual sliding or slipping of any embankment under investigation (NEH, 2012).

6. Attitude

Attitude is the orientation of strata, faults, fractures and other features relative to a horizontal plane. It is usually expressed in terms of measured dip and strike (Fig 2.7).

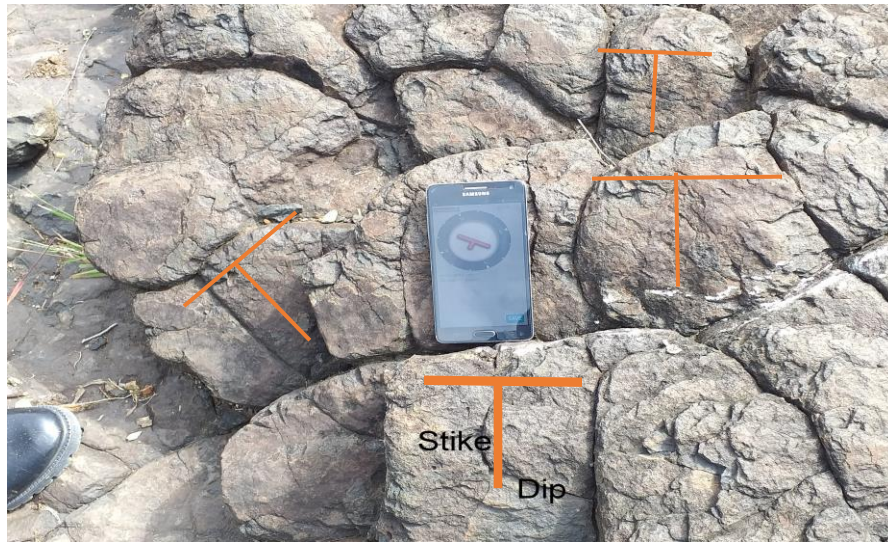


Fig 2.7: Attitude (Strike and Dip orientation) of a rock mass (Location: Project site, Proposed Intake section of Tunnel Route)

7. Discontinuities

Any structural or geological features that changes or alters the homogeneity of a rock mass can be considered as discontinuity (Singh and Goel, 2011).

The roughness, nature of their contacts, degree and nature of weathering, type and amount of gouge and susceptibility to ground water flow will vary greatly from one type of discontinuity to another since their cause, age and history of development are fundamentally different.

According to their mode of formation, discontinuities are classified as stratigraphic and structural (NEH, 2012). Stratigraphic discontinuities represent a significant interruption of the orderly sequence of deposition, most frequently marked by a considerable time interval of erosion or non-deposition.

Structural discontinuities are discontinuities that develop after the initial formation of the rock mass as a result of external process acting on it. These features are produced by the mechanical

deformation or displacement of rock by natural stresses within the earth's crust, which include fractures of all types, planes or zones of weakness faults and shear zones, most of which have little to no tensile strength.

Faults are breaks along which there has been displacement of the sides relative to one another parallel to the break. It can occur as a single break or as a fault zone. A fault zone consists of numerous sub-parallel and interconnecting, closely spaced fault surfaces. Faults may be detected by discontinuity of strata and by surface features (NEH, 2012).

Faulting may bring together materials with different engineering properties and also modify ground water conditions.

The characteristics of fractured rocks include orientation, joint spacing, and aperture width of joint face surfaces, type of infilling material, linear persistence and type of joint ends.

Orientation

Orientation is the establishment of the correct relationship in direction, usually with reference to points of the compass and a geological compass can be used to measure the orientation of joints and fractures (Fig 2.8).



Fig 2.8: Measuring the orientation of a Joint face (Location: Project site, Proposed Inlet Section of the Tunnel Route)

Joint Spacing

It is the average spacing of joints within a joint set expressed in meters (millimeters). It also determines the size and shape of discrete rock particle (Fig 2.9).



Fig 2.9: Joint Spacing of rock outcrop at a certain rock mass (Location: Project site, Proposed Intake section of the Tunnel Route)

Aperture width of joint face surfaces

Aperture refers to the opening between opposing faces of a joint, fracture or fault. The aperture width of a joint affects the movement of water into the opening, the wider the aperture, the greater the potential for movement of particles by uplift forces and pore pressure (Singh and Goel, 2011) (Fig 2.10).



Fig 2.10: Aperture Width of rock outcrop (Location: Project site, Proposed Intake section of the Tunnel Route)

Infilling (Gouge material)

Infilling is the material occupying the aperture between joint faces and the thickness usually the same as the aperture width. Brekke and Howard (1972) in Hoek and Brown (1980) presented seven groups of discontinuity infillings or gouges that have significant influence upon the engineering behavior of rock masses (Fig 2.11).

- Joints, seams, and sometimes even minor faults may be healed through precipitation from solutions of quartz or calcite. In this instance, the discontinuity may be “welded” together. Such discontinuities might, however, have broken up again, forming new surfaces. It should be emphasized that quartz and calcite may be present in a discontinuity but may not always be healing it.
- Clean discontinuities include those without fillings or coatings. Many of the rough joints or partings have a favorable character. Close to the surface, however, it is imperative not to confuse clean discontinuities with “empty” discontinuities from where filling material has been leached and washed away from surface weathering.
- Calcite fillings may dissolve due to seepage during the lifetime of an underground opening, particularly when they are porous or flaky. Their contribution to the strength of the rock mass then disappears. This is a long-term stability (and sometimes fluid flow) problem easily overlooked during design and construction. Gypsum fillings may behave the same way.
- Coatings or fillings of chlorite, talc, and graphite make very slippery (i.e., low strength) joints, seams, or faults, particularly when wet, due to the loss of cohesion.
- Inactive clay material in seams and faults naturally represents a very weak material that may squeeze or wash out.



Fig 2.11: Calcite infillings obtained from drilled rock core sample (Location: Project site, BH-07)

If the gouge consists of montmorillonite clay mineral, variation in its moisture content may bring about catastrophic instability of the rock slope. Any clay gouge in a sloped discontinuity makes the rock mass slide easily, and when such a gouge becomes wet it promotes sliding of the rock blocks. In either case, the presence of a significant thickness of gouge has a major influence on the stability of a rock mass (Hoek & Bray, 1981).

Linear persistence

Linear persistence is the extent to which an individual fracture can be traced within a plane. It is one of the most important factors in rock performance evaluation, however, it is usually difficult to measure adequately because joints often extend beyond the outcrop area, it can be quantified by measuring the discontinuity trace lengths on surface of exposures (NEH, 2012) (Fig 2.12).

2.1.4. Properties related to both rock material and rock mass

There exist physical properties that are common or related to both the rock material and rock mass, where each of which have a direct effect on the hydraulic erodibility and excavation characteristics of the rock. Common properties include:



Fig 2.12: Persistence (Joint Length) of rock outcrop (Location: Project site, Proposed Power House section of the Tunnel Route)

1. Seismic velocity

As the name indicates seismic velocity is the velocity of propagation of stress wave through a rock mass. It is a function of many rock material properties including density, porosity, mineral composition and the degree of cementation and consolidation and rock mass properties including degree of fracturing and degree of weathering (NEH, 2012).

Seismic refraction surveys are routinely conducted during preliminary site investigations to provide a rapid assessment of depth to rock, configuration of the rock surface and indication of the relative integrity of foundation materials (Gay, 2001).

The results of a seismic survey must be considered provisional until supplemented with conventional drilling and excavation techniques in subsequent detailed investigations.

2. Weathering

Weathering is the physical disintegration or chemical decomposition of earth materials resulting in changes in color, texture, composition, density or form with little or no transport of the loosened or altered material while its effect tends to diminish with depth and is best assessed on a macroscopic scale in the field (NEH, 2012).

The rate and type of rock weathering depend on climate, topography, vegetation, time and the physical and chemical composition of the rock.

3. Secondary Cavities

Secondary cavities are open holes and voids such as pits and vesicles that form as a result of chemical or mechanical process acting on the rock mass after its formation. These types are exclusive of fracture, jointing and other open planar secondary features.

4. Geohydrologic property

Geohydrologic properties are attributes of rock units that affect the mode of occurrence, location, distribution and flow characteristics of subsurface water within the unit and it includes material and mass properties but also account for the interaction and behavior of subsurface water within the rock mass.

2.2. Rock mass Characterization

Rock mass is generally an in-situ medium which comprises intact rock blocks separated by discontinuities. Rock masses are generally discontinuous and often have heterogeneous and anisotropic properties.

Rock Mass Classification is the process of placing a rock mass into groups or classes on defined relationships (Bieniawski, 1989) and assigning a unique description (or number) to it on the basis of similar properties/characteristics such that the behavior of the rock mass can be predicted.

Rock mass classifications form the backbone of the empirical design approach and are widely employed in rock engineering.

Engineering rock mass classifications have recently been quite popular and are used in feasibility designs. Many researches have shown that the classification approach is the only practical basis for the design of complex underground structures (Bieniawski, 1989).

2.2.1. Rock mass Classification

Rock mass classification schemes owe their origin to the late 18th century devising as an empirical designing method in tunnels for finding out support requirements (Hoek, 2007). The rock mass classification schemes that are often used in rock engineering for assisting in designing underground structures are rock mass rating (RMR) system, rock quality index (Q) system and geological strength index (GSI) system (Palmstrom, 2003).

According to Abbas and Konietzky (2015) citing Bieniawski's (1989) original work, engineer's observations, experience, and judgment are correlated and consolidated more effectively by an engineering (quantitative) classification system.

An engineering classification system has considerable application in an overall assessment of the rock quality. The classification approach also helps in the organization of knowledge, and thus an ideal application of engineering rock mass classification occurs in the planning of hydroelectric projects, tunnels, caverns, bridges, silos, building complexes, hill roads, rail tunnels, and so forth.

On the basis of mode of characterization, these systems can be grouped as qualitative and quantitative. Qualitative i.e. descriptive systems include Geological Strength Index (GSI), Rock Load and Schweizerischer Ingenieur and Architekten-Verein (SIA 199). Rock quality index (Q) system, rock mass rating (RMR) system, rock structure rating (RSR) and rock quality designation (RQD) system are quantitative. After Coasr (2004), some of the well-recognized rock mass classification systems are listed in table format in Figure 2.13.

2.2.1.1. Rock Quality Designation (RQD)

In earlier stages rock quality information was usually available only from geologists' descriptions and the percent of rock recovery (Deere et al., 1967). Rock quality designation is a terminology useful to define the percentage of intact core pieces longer than 100 mm from the total length of core having core diameter of 54.7 mm (Hoek, 2007).

RQD is a measure of degree of fracturing of the rock mass and is aimed to represent the in-situ rock mass quality. The greater the RQD value the better the rock mass quality.

RQD is used as an input parameter in RMR and Q systems. Though, it does not reflect fully the rock mass quality as it only considers the extent of fracturing of the rock mass and does not account for the strength of the rock or mechanical and other geometrical properties of the joints (Singh and Goel, 2011).

The RQD was originally developed for predicting tunneling conditions and support requirements. Its application was extended to correlation with in-situ rock mechanical properties and within the upcoming years after the development, for forming a basic element of several classifications. Its greatest values remain uncontroversial where it serves as a mechanism to identify low-RQD zones.

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)	Bieniawski, 1973 (last modification 1989-USA)	South Africa	Tunnels, mines, (slopes, foundations)
Modified Rock Mass Rating (M-RMR)	Ünal and Özkan, 1990	Turkey	Mining
Rock Mass Quality (Q)	Barton et al., 1974 (last modification 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)	Palmström, 1996	Sweden	Tunneling
Geological Strength Index (GSI)	Hoek and Brown, 1997	Canada	All underground excavations

Fig 2.13: Table illustrating the major rock mass classification systems (Cosar, 2004)

Eq.2.1 is illustrated after Deere (1967). Also, Palmström (1982) demonstrated that the *RQD* may be estimated from the number of discontinuities per unit volume, which are exposed on the outcrops using the following relationship (Eq.2.2) for clay-free rock masses (Where J_v , known as the volumetric joint count, is the sum of the number of joints per unit length for all joint sets).

$$RQD = \frac{\sum \text{Length of core pieces} > 100\text{mm}}{\text{Total length of core run}} * 100\% \quad (2.1)$$

$$RQD = 115 - 3.3J_v \quad (2.2)$$

RQD is dependent on the orientation of the borehole. The use of the volumetric joint count can be quite useful in reducing this directional dependence.

2.2.1.2. Rock mass Rating (RMR) rock mass classification system

Rock mass rating (RMR) rock mass classification system, also known as the Geo-mechanical classification system was first developed by Bieniawski during 1972-1973 in South Africa to assess the stability and support requirements of tunnels (Bieniawski, 1973b).

The advantage of this system is that only a few basic parameters relating to the geometry and mechanical conditions of the rock mass are used. Though in situ stress conditions are not included among the parameters, it is yet the basic and well-established system till date (Bieniawski, 1989).

The classification system incorporates six basic parameters in which the summation of the weighted parameters (Eqn.2.3) in return gives the RMR for the rock mass. The parameters are namely:

- a. Uniaxial compressive strength(σ_c) or the point load of the intact rock material
- b. Rock quality designation (RQD)
- c. Spacing of discontinuities
- d. Conditions of discontinuities
- e. Ground water condition
- f. Orientation of discontinuities

$$RMR = a + b + c + d + e + f \quad (2.3)$$

One of the basic accomplishments of the RMR system is that properties of rock mass based on their rated rock class can be obtained. Table 2.1 below illustrates rock mass properties like friction angle, cohesion and average stand-up time with their respective rock class.

For the present research work, the rock mass around the Wabe hydropower project will be characterized based on the Geomechanics classification system or the rock mass rating system as proposed by Bieniawski (1976) and Bieniawski (1989) versions.

Table 2.1: Rock mass properties and parameters with their respective rock class (after Bieniawski, 1989)

Properties and Parameters Of rock mass	Rock Mass Class (Rating)				
	100-81	80-61	60-41	40-21	< 20
Ratings	100-81	80-61	60-41	40-21	< 20
Rock mass classification	Very Good	Good	Fair	Poor	Very Poor
Cohesion of rock mass	>400KPa	300-400KPa	200-300KPa	100-200KPa	<100KPa
Friction angle of rock mass	>45°	35°-45°	25°-35°	15°-25°	<15°
Average stand-up time	10 years for 15m Span	6 Month for 8m Span	1 Week for 5m Span	10 hours for 2.5m Span	30 Minutes For 1m Span

Drawbacks

Although the RMR system for rock mass classification, when compared to most classification systems, is widely acceptable, the effects of overstressing conditions like rock burst and swelling

are not included in any of the parameters (Palmstrom, et al., 2006). In addition, the output of RMR system can lead to overdesign of support systems because it is conservative (Bieniawski, 1989).

RMR system cannot also be used reliably in weak rock masses because it is mostly based on case histories of competent rocks (Singh and Geol, 1999). Therefore, this system is not preferable for deciding excavation methods for very weak rock masses.

2.2.1.3. The Q- Rock mass classification system (Rock Tunneling Quality Index)

The Q-system is also another quantitative rating system proposed on the basis of analysis made on various hard rock tunnel case histories.

RMR and Q-Systems use essentially the same approach but different log-scale ratings, as Q-value is the product of ratio of parameters while RMR is the sum of parameters (Hoek, 2007). The Q-rating is developed by assigning values to six parameters that are grouped into three quotients (Eq.2.4) (Singh and Goel, 1999). It is calculated as follows:

$$Q = \frac{RQD}{J_n} * \frac{J_r}{J_a} * \frac{J_w}{SRF} \quad (2.4)$$

Where,

RQD= Rock Quality Designation

J_n= Joint set number (number of discontinuity sets)

J_r= Joint roughness number (roughness of discontinuity surface)

J_a= Joint alteration number (degree of alteration or weathering or filling of discontinuity surface)

J_w= Joint water reduction number (Pressure and inflow rate within the discontinuities)

SRF= Stress reduction factor (presence of shear zone, stress concentration and swelling rocks)

Barton et al., 1974 suggested that RQD/J_n reflects block size, J_r/J_a reflects friction angle and J_w/SRF reflects effective stress conditions.

One of the widely known applications of the Q- system is that, Q-value can be applied to estimate the support measure for a tunnel of a given dimension, and the usage of excavation by defining the equivalent dimension (D_e) of the excavation (Barton et al., 1974).

According to Barton et al (1974), the parameter, the Equivalent Dimension, D_e , is introduced to relate the value of the index Q to the stability and support requirements of underground excavations. This dimension is obtained by dividing the span, diameter or wall height of the excavation by a quantity called the Excavation Support Ratio, ESR. Hence: (Hook, 2007) (Eq.2.5).

$$D_e = \frac{\text{Excavation span } (s), \text{ diameter } (d) \text{ or height } (m)}{\text{Excavation support ratio } (ESR)} \quad (2.5)$$

The value of ESR depends upon the intended use of the excavation and the degree of its safety demanded (Singh and Geol, 1999). Hence, in the case of wall support in the D_e equation, height of wall is usually used. Table 2.2 below clearly describes the value of ESR related to the intended use of the excavation and to the degree of security after (Barton et al 1974).

Table 2.2: The Value of Excavation Support Ratio (ESR), (Barton et al. 1974)

	Excavation category	ESR
A	Temporary mine openings.	3-5
B	Permanent mine openings, water tunnels for hydropower (excluding high pressure penstocks), pilot tunnels, drifts and headings for large	1.6
C	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels.	1.3
D	Power stations, major road and railway tunnels, civil defense chambers, portal intersections.	1
E	Underground nuclear power stations, railway stations, sports and public facilities, factories.	0.8

Based on the relationship between the index Q and the equivalent dimension of the excavation, 38 different support categories have been suggested (Figure 2.14), and permanent support has been recommended for each category in the support tables (Barton et al., 1974).

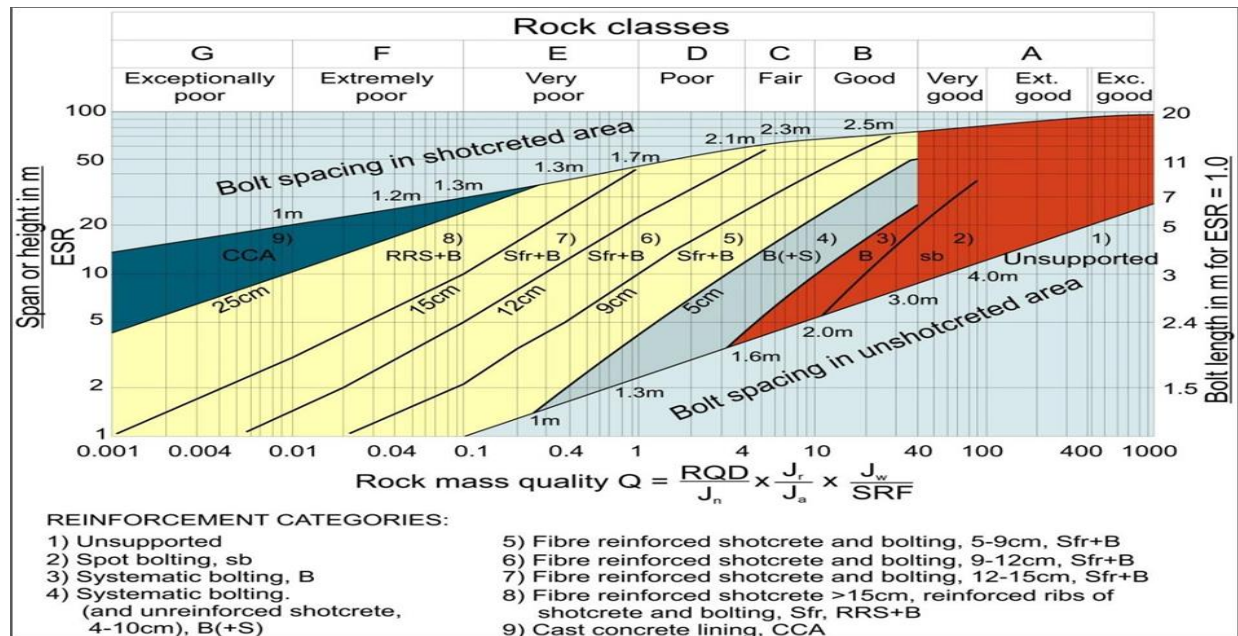


Fig. 2.14: Support categories for different rock mass classes as per the values of Rock Quality Index Q and the support width or height (Grimstad and Barton, 1993)

Another application of Q-system is that, it can be used to estimate deformation modulus of the rock mass (E_m) of good quality by using Eq. (2.6), (2.7) and (2.8) below (Grimstad and Barton, 1993).

$$E_m = 25 \log Q, Q > 1 \quad (2.6)$$

$$E_m = 10 \left(\frac{Q \sigma_c}{100} \right)^{1/3} \quad (2.7)$$

$$E_m = 10^{(15 \log Q + 40)/40} \quad (2.8)$$

Drawbacks

Though the Q-system is relatively sensitive to minor variations in rock properties, the stress reduction factor does not cover a wide range of in situ-stress for rocks of certain strength and this in turn has led to ineffectiveness in rock engineering design.

The Q-system is relatively difficult to apply especially for inexperienced users. According to Palmstrom and Broch (2006), the ratio RQD/J_n does not provide a meaningful measure of relative block size and the ratio J_w/SRF is not a meaningful measure of the stresses acting on the rock mass to be supported.

2.2.1.4. Geological Strength Index (GSI)

Hoek and Brown (1997) introduced the Geological Strength Index (GSI) for both hard and weak rock masses for use in rock engineering. The GSI is a combining effect of observation obtained from the rock mass condition that is a detailed visual inspection with experience obtained from the RMR-system.

According to Hoek (1998) and Marinos and Hoek (2000), a range of values of GSI (or RMR) should be estimated instead of just a single value. This practice has a significant impact on the design of slopes and excavations in rocks.

The aim of the GSI-system is to determine the properties of the undisturbed rock mass. For disturbed rock masses, compensation must be made for lowest GSI-values obtained from the same locations. Fig.2.15 below shows a chart used in RocLab software to estimate the Geological Strength Index based on field geological description.

2.3. In-situ Rock mass Permeability Test

In-situ rock mass permeability tests are usually done by using water pressure tests (Packer test) (Fig.2.16), where the test consists of injection of water into a borehole at a constant flow rate and pressure. Water enters the rock mass along the entire length of the borehole or along the interval of the borehole (test section) which has been sealed off by one or more packers. Water pressure tests can be conducted in media above or below the ground water table and anisotropic permeability can be estimated by orienting test boreholes in different directions.

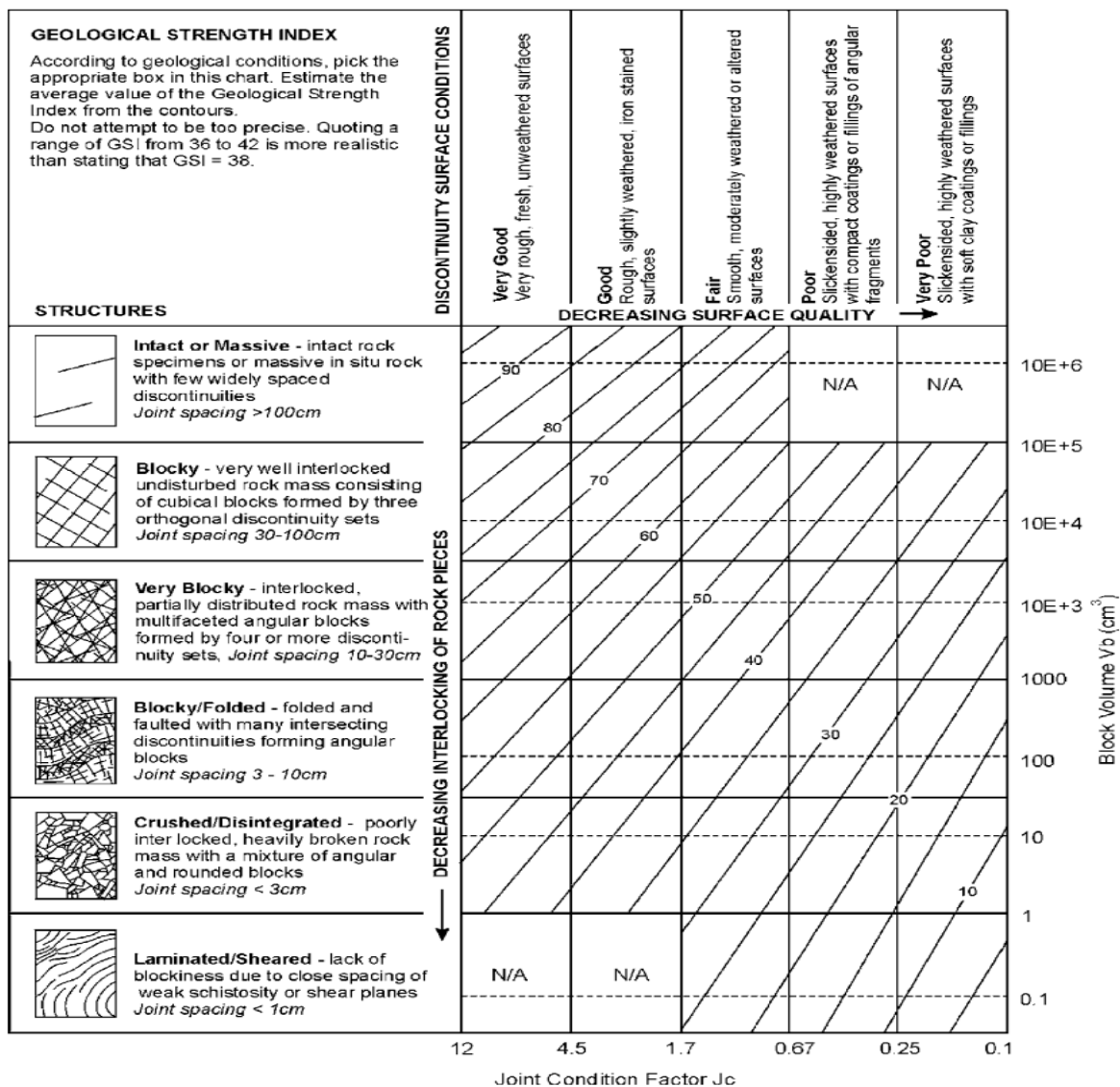


Fig 2.15: Estimate of GSI based on visual inspection of geological conditions (from Roclab, 2006), after Marinou and Hoek, 2000)

Prior to the beginning of the test, a maximum test pressure, P_{MAX} , is defined. P_{MAX} is chosen such that it does not exceed the confinement stress (σ_3) expected at the depth where the test is being conducted, thus avoiding the development of hydraulic fracturing or hydraulic jacking. As a rule of thumb, P_{MAX} is usually established using Eq. (2.9a), where D is equal to the minimum ground coverage – depth in the case of a vertical boring in a flat site or minimum lateral coverage in the case of a test conducted in a hillside.

$$P_{\max} = D * \frac{1MPa}{m} \quad (2.9a)$$

The test is conducted in five stages, with a particular water pressure magnitude associated with each stage. A single stage consists of keeping a constant water pressure at the test interval for 5-10 minutes by pumping as much water as required. The first stage is held at a low water pressure, increasing the pressure in each subsequent stage until reaching P_{MAX} . Once P_{MAX} is reached, pressures are decreased following the same pressure stages used on the way up, thus describing a “pressure loop”.

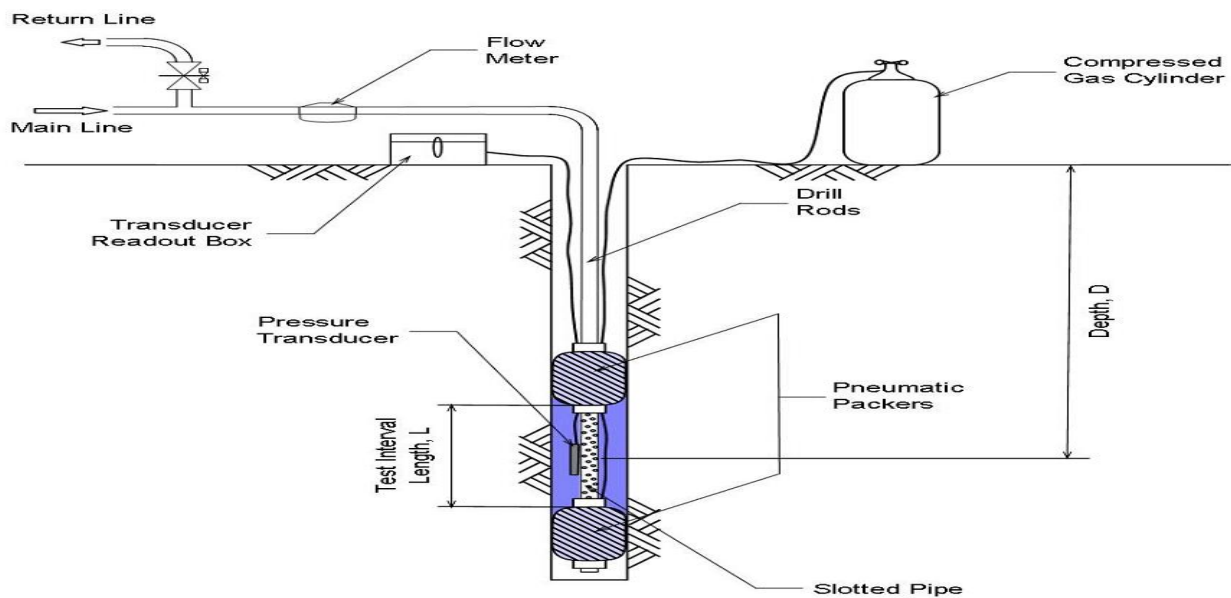


Fig 2.16: Water pressure (Packer test) Test Configuration

During the execution of each stage, both water pressure (P) and flow rate (q) values are recorded every minute. Subsequently, average values for P and q are then used to compute the hydraulic conductivity for each stage. The hydraulic conductivity is expressed in terms of the Lugeon value, which is empirically defined as the hydraulic conductivity required in achieving a flow rate of 1 liter/minute per meter of test interval under a reference water pressure equal to 1 MPa (Eq.2.9b). Where P_0 corresponds to a reference pressure equal to 1MPa

$$Lugeon\ Value = \alpha * \frac{q}{L} * \frac{P_0}{P} \quad (2.9b)$$

A typical precision used to report the Lugeon values are summarized in Table 2.3 below associated with rock mass discontinuities.

Table 2.3: Condition of rock mass discontinuities associated with different Lugeon values (after Fell et al., 2005)

Lugeon Range	Classification	Hydraulic Conductivity Range (cm/sec)	Condition of Rock Mass Discontinuities	Reporting Precision (Lugeons)
<1	Very Low	$<1 \times 10^{-5}$	Very Tight	<1
1-5	Low	$1 \times 10^{-5} - 6 \times 10^{-5}$	Tight	± 0
5-15	Moderate	$6 \times 10^{-5} - 2 \times 10^{-4}$	Few Partly Open	± 1
15-50	Medium	$2 \times 10^{-4} - 6 \times 10^{-4}$	Some open	± 5
50-100	High	$6 \times 10^{-4} - 1 \times 10^{-3}$	Many open	± 10
>100	Very High	$>1 \times 10^{-3}$	Open closely spaced	>100

Once a Lugeon value has been computed for each of the five test stages, a representative value of hydraulic conductivity is selected based on the trend observed throughout the test.

The current Lugeon interpretation practice is mainly derived from the work performed by Houlsby (1976). In his work, geared towards establishing grouting requirements, Houlsby proposed that representative hydraulic conductivity values should be selected based on the behavior observed in the Lugeon values computed for the different pressure stages. Fig.2.17 below illustrates a graphic summary of the five behavior groups defined by Houlsby (1976), as well as the representative Lugeon value that should be reported for each group.

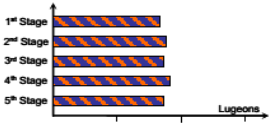
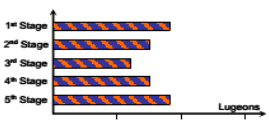
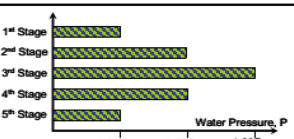
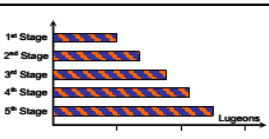
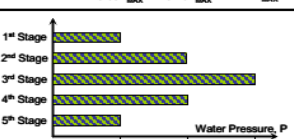
BEHAVIOR	PRESSURE STAGES	LUGEON PATTERN	DESCRIPTION	REPRESENTATIVE LUGEON VALUE
LAMINAR			All Lugeon values about equal regardless of the water pressure	Average of Lugeon values for all stages
TURBULENT			Lugeon values decrease as the water pressures increase. The minimum Lugeon value is observed at the stage with the maximum water pressure	Lugeon value corresponding to the highest water pressure (3 rd stage)
DILATION			Lugeon values vary proportionally to the water pressures. The maximum Lugeon value is observed at the stage with the maximum water pressure	Lowest Lugeon value recorded, corresponding either to low or medium water pressures (1 st , 2 nd , 4 th , 5 th stage)
WASH-OUT			Lugeon values increase as the test proceeds. Discontinuities' infillings are progressively washed-out by the water	Highest Lugeon value recorded (5 th stage)
VOID FILLING			Lugeon values decrease as the test proceeds. Either non-persistent discontinuities are progressively being filled or swelling is taking place	Final Lugeon value (5 th stage)

Fig 2.17: Lugeon Interpretation table (Houlsby, 1976)

2.4. Rock mass Characterization using seismic refraction Technique

2.4.1. Introduction to Seismic Method

Geophysical methods have proven useful as rapid means of obtaining subsurface information and as economical supplements to borings in exploratory programs for civil engineering purposes. This technique also supply information for bed rock profiling, define the limits of granular borrow areas and large organic deposits and yeild a general definition of subsuface conditions including the depth to ground water (Gay, 2001).

The seismic method of exploration are based on the fact that schok waves travel at different velocities through different types of materials. In this technique , artificial impulses are produced either by the detonation of explosives or a mechanical blow usually with a heavy hammer at ground

surface or at shallow depth within a hole. The artificial shocks generate three types of waves namely compressional waves, shear wave and surface waves. In general, only compression (longitudinal) waves are observed.

According to Gay (2001), the longitudinal waves are further classified as direct, reflected, or refracted waves. Waves that are turned back when they encounter such a boundary are called reflected waves; those that undergo a change or a bending in the direction of propagation are refracted.

It is mainly refraction and reflection seismic methods that are used for subsurface profiling in engineering exploration (Redpath, 1973). The refraction method is most commonly used for civil engineering purposes. In addition to subsurface profiling, seismic methods have also been used to determine engineering properties of soil and rock. A typical field set up for seismic refraction test is illustrated in Fig 2.18 below.

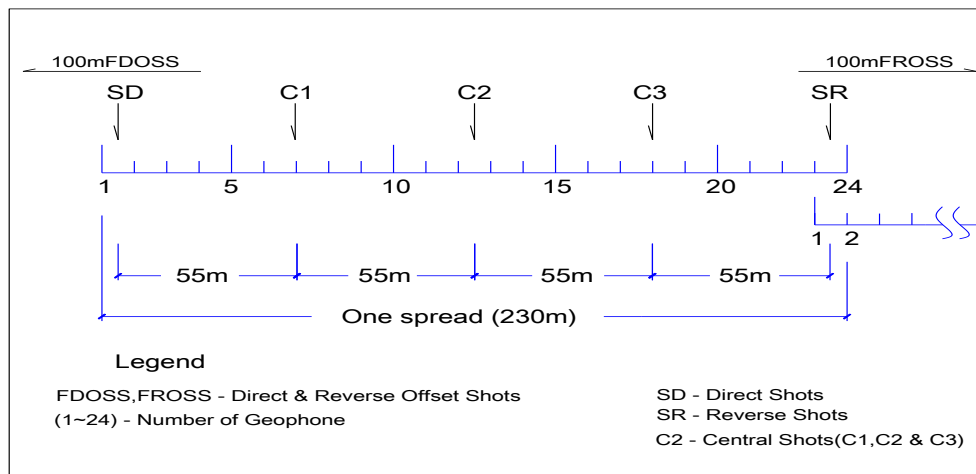


Fig 2.18: Field setup for seismic refraction test (after Wabe seismic report ECDSWC, 2019)

2.4.2. Seismic Refraction Method in characterizing Rock mass

Seismic refraction methods have been used to investigate subsurface conditions from ground surface to depths of approximately 300 m (Gay, 2001).

Measurement of subsurface conditions by the seismic refraction method requires a seismic energy source, trigger cable (or radio link), geophones, geophone cable, and a seismograph (ASTM

D5777). The geophone(s) and the seismic source must be placed in firm contact with the soil or rock. The geophones (Fig 2.19) are usually located in a line, sometimes referred to as a geophone spread.

As discussed earlier the seismic source may be a sledge hammer, a mechanical device that strikes the ground, or some other type of impulse source. Explosives are used for deeper refractors or special conditions that require greater energy. Geophones convert the ground vibrations into an electrical signal and this electrical signal is recorded and processed by the seismograph (2.20). The travel time of the seismic wave (from the source to the geophone) is determined from the seismic wave form.



Fig 2.19: A single Geophone along with geophone cable

The seismic energy source generates elastic waves that travel through the soil or rock from the source. When the seismic wave reaches the interface between two materials of different seismic velocities, the waves are refracted according to Snell's Law. Seismic refraction methods of exploration are based on the fact that shocks waves travel at different velocities through different types of materials.

The velocity of wave propagation depends on density, moisture, texture, void space and elastic constants, and from such properties the nature and stratification of subsurface materials can be determined. This in return indicates rock mass can be characterized through seismic refraction surveys; this is because different rocks have their own distinct seismic velocities. The type of

discontinuities (joints), orientation, spacing, persistence, aperture and filling, roughness, waviness, etc. are some of the parameters that influence the physical properties of the rock mass (Palmstrom, 1996).

Aside from the inherent rock properties in fresh (unweathered) rock mass, the stress acting, presence of joints and infilling materials within the joints, presence of ground water and degree of joints determine the in situ longitudinal velocity.

On the other hand seismic refraction measurements cannot be used to assess the condition of the joint itself. The seismic velocity might not be influenced by the presence of infilling materials within joints that usually cause instability. One or two open joints that may not have any effect on the stability of an opening can significantly lower the seismic velocity and give the impression of low-quality rock (Cecil, 1971).



Fig 2.20: Seismograph (Seis-24) used to measure the compressional wave velocity

The density of the rock is one factor, which affects the velocity of longitudinal seismic waves. In the elastic wave theory, the velocity decreases with increasing density. According to Gay (2001), P-wave velocities are generally greater for:

- Denser rocks than lighter rocks
- Older rocks than younger rocks;
- Igneous rocks than sedimentary rocks

- Solid rocks than rocks with cracks or fractures;
- Unweathered rocks than weathered rocks

The fundamental limitation of seismic refraction method is that when refraction measurements are made over a layered earth, the seismic velocity of the layers is assumed to be uniform and isotropic. If actual conditions in the subsurface layers deviate significantly from this intended idea, then any interpretation also deviates from the idea.

In addition, the range of the velocity of wave propagation for a material may overlap that of the material immediately above or below and it may be difficult, if not impossible, to distinguish between the two strata.

There also are some strata in which the velocity of wave propagation continuously increases with depth and causes difficulty in the determination of the actual wave velocity. Therefore, borings should always be performed in conjunction with seismic surveys to clarify the interpretation of the data and to insure maximum reliability (Gay, 2001).

2.5. Rock Mass Failure Criteria

2.5.1. Failure Criteria

Bieniawski (1984) defines criteria of failure as an algebraic expression of the mechanical condition under which a material fails by fracturing or deforming beyond some specified limit. The specification can be in terms of load, deformation, stress, strain and other parameters.

In engineering, the stress condition by which ultimate strength is reached is referred to as failure criterion. It is often expressed in terms of the major principal compressive stress σ_1 that rocks sustain for given values of the other two principal stresses, σ_2 and σ_3 that is $\sigma_1 = f(\sigma_2, \sigma_3)$ or $f(\sigma_1, \sigma_2, \sigma_3) = 0$.

The “Mohor- Coulomb” and “Hoek- Brown” are the most frequently used failure criteria for rock while both incorporate only the major and minor principal stresses and not the intermediate principal stress.

2.5.2. Mohr- Coulomb failure criterion

The criterion can be considered as the first criterion available in rock mechanics. The MC criterion is generally a set of linear equations in principal stress space describing the conditions for which an isotropic material will fail, with any effect from the intermediate principal stress σ_2 being neglected. When all the principal stresses are compressive, experiments demonstrate that the criterion apply reasonably well to rock.

The Mohr-Coulomb criteria for rock failure can be described as in the following relation (Eq.2.10) (Singh et al., 2002)

$$\tau_{cr} = S_i + \sigma \tan \phi \quad (2.10)$$

Where, τ_{cr} is the shear strength, S_i is the cohesion which is the intercept with the τ axis of the linear envelop, ϕ is the angle of friction which is the slope angle of the linear envelope of failure.

The criterion allows for a curved shape of the failure envelop, and this nonlinear behavior is exhibited by many rock types. It has been established that rock fails in compression by shearing along a failure surface oriented at an angle ϕ with respect to σ_1 that is specific for a particular rock while θ stays the same regardless of the confining pressure applied. The MC criterion can also be expressed in terms of the principal stress (Eq.2.11) as;

$$\sigma_1 = \frac{2 \cos \theta}{1 - \sin \phi} + \frac{1 + \sin \phi}{1 - \sin \phi} \sigma_3 \quad (2.11)$$

The advantages of the MC failure criterion are its mathematical simplicity, clear physical meaning of the material parameters and general level of acceptance, while its shortcoming is that it does not consider the effect of the intermediate principal stress σ_2 (Fig 2.21).

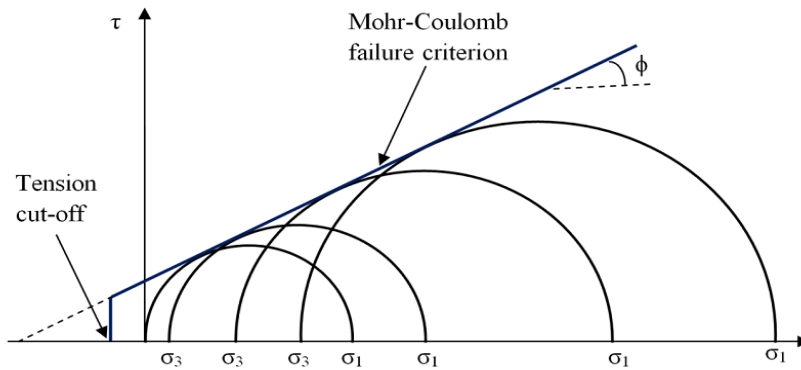


Fig 2.21: The MC failure criteria for rock

2.5.3. Hoek-Brown Failure Criterion

The Hoek–Brown criterion was developed as a means to estimate rock mass strength by scaling the relationship derived according to the geological conditions present (ISRM, 2014).

The failure criterion was 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 on the basis of the characteristics of joints in a rock mass (Hoek, et al. 2002).

The Hoek-Brown model can be used to represent intact rock or highly fractured rock masses. However, it should not be used 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, 1989). The original Hoek-Brown criterion in terms of normal stresses for intact rock is given by Eq.2.12:

$$\sigma_1' = \sigma_3' + \sigma_{ci} \left(m \frac{\sigma_3'}{\sigma_{ci}} + S \right)^{0.5} \quad (2.12)$$

Where,

σ_1' and σ_3' : major and minor effective principal stresses at failure;

σ_{ci} : the uniaxial compressive strength of the intact rock material and

m and s: material constants, where s = 1 for intact rock.

In applying the original Hoek-Brown failure criterion to jointed rock masses, the predicted strengths are found to be acceptable where the rock mass is subjected to conditions in which the minor principal effective stress (σ_3) has a significant compressive value. For low values of σ_3 the criterion predicts too high uniaxial strength and also a finite tensile strength.

Most rock mechanics engineers consider that the type of jointed rock mass to which the Hoek-Brown failure criterion applies should have zero tensile strength. For the past 30 years, finite element numerical models for use in rock mechanics have included a 'no tension' option which allows tensile stresses developed in the model to be transferred onto adjacent elements (Hoek et al., 1992).

In view of this deficiency in the original Hoek-Brown criterion when applied to jointed rock masses, it was decided that a modified criterion should be developed. This criterion should conform to the strength predictions given by the original criterion, for compressive stress conditions, and should predict a tensile strength of zero for the rock mass. A modified criterion, which satisfies these conditions, was developed by Shah (1992) and can be expressed in the following form (Eq. 2.13):

$$\sigma_1' = \sigma_3' + \sigma_c \left(m_b \frac{\sigma_3'}{\sigma_c} \right)^a \quad (2.13)$$

Where, m_b and a are reduced value of the material constants

In addition to the changes in the equations, it was also recognized that the Rock Mass Rating of Bieniawski was no longer adequate as a means for relating the failure criterion to geological observations in the field, particularly for very weak rock masses. This resulted in the introduction of the Geological Strength Index (GSI) by Hoek, Wood and Shah in 1992. This index was subsequently extended for weak rock masses in a series of papers.

The Generalized Hoek-Brown criterion aimed at adjusting the shape of the principal stress plot or the Mohr-envelope by means of introducing a variable coefficient 'a' in place of the square root term in the original equation.

2.6. Rock mass Deformation characteristics

Rock mass deformation defined by Palmström (et al., 2001) clearly indicates that, deformation is the closure of discontinuities, plastic and elastic deformation of the intact rock that comprises rock mass, under applied static or dynamic load. The deformation characteristics of the rock mass can be measured using the modulus of deformation, E_d (Jonson, 1988).

Deformability is characterized by a modulus describing the relationship between the applied load and the resulting strain. The fact that jointed rock masses do not behave elastically has prompted the usage of the term modulus of deformation rather than modulus of elasticity or Young's modulus (Palmström and Singh, 2001).

Empirical determination of the deformability of the rock mass is given by different scholars. In the 1960's several attempts were made to use Deere's RQD for estimating in-situ deformation modulus, however, this approach is seldom used today (Deere, 1988).

Based on the case histories of mostly dam foundations, Serafim and Pereira (1983) developed an empirical relation by back analysis of E_d from the measured deformation; the relation works well for the rock mass for which RMR lies between 10 to 50 and UCS of intact rock $q_c > 100$ MPa.

The Serafim and Pereira relation for $RMR_{89} < 50$ is expressed in Equation 2.14 (Serafim & Pereira, 1983). Where RMR_{89} represents RMR obtained using Bieniawski (1989) rock mass classification system.

$$E_d = 10^{(RMR-10)/40} \quad (2.14)$$

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

Bieniawski (1978) proposed an empirical relation (Eq.2.15) to determine the modulus of deformation " E_d " by using RMR. For the rock mass having RMR higher than 55 and UCS of intact rock greater than 100 MPa the relation is in close agreement with the tested values. For $RMR > 50$ (Bieniawski, 1978),

$$E_d = 2RMR - 100 \quad (2.15)$$

Barton et al (1980), Barton et al (1992) and Grimstad and Barton (1993) proposed an empirical relation (Eq.2.16) to determine E_d by using Q value;

$$E_d = 25 \log Q \quad (2.16)$$

Hoek and Brown (1997) proposed an empirical relation (Eq.2.17) to work out modulus of deformation of rock mass. This relation works well for rock mass having UCS less than 100 MPa.

$$E_d = \left(\frac{q_c}{100} \right)^{0.5} * 10^{\left(\frac{GSI-10}{40} \right)} \quad (2.17)$$

Where; “ q_c ” is the UCS of intact Rock at natural moisture content given in MPa, E_d is modulus of deformation given in MPa and GSI is the Geologic Strength Index. For $RMR_{76} > 18$, $GSI = RMR_{76}$ and $RMR_{89} > 23$, $GSI = RMR_{89} - 5$. Where RMR_{76} represents RMR obtained using Bieniawski (1976) rock mass classification system.

Deer and Miller (1996) presented an empirical classification of the Modulus of Deformation with its deformability characteristics (Table 2.4). Rocks having a modulus of deformation < 5 have a corresponding very high deformability and on the other hand rocks having a modulus of deformation > 60 the rock exhibit very low deformability.

Table 2.4: Classification of intact rock based on modulus of deformation (After, Deere and Miller, 1966)

Class	Modulus of Deformation (GPa), Hoek and Brown	Deformation characteristics
1	<5	Very High
2	5-15	High
3	15-30	Moderate
4	30-60	Low
5	>60	Very Low

2.6. Rock mass Shear strength Characteristics

The assessment of rock mass strength is a key element in any rock excavation for both open pit and underground excavations. When numerical models are used as a tool of analysis, this strength is defined in terms of a strength envelope. The rock mass strength is controlled by the composite structure of the intact rock and the discontinuities, fissures, and jointing.

Based on results of a number of projects this, criterion was modified by Hoek & Brown in (1988) and later by Hoek et al. (1992). For this empirical method Hoek and Brown utilized Bieniawski's Rock mass rating system (RMR) to work out the material constants. Bieniawski (1989) proposed direct empirical relations to workout shear strength parameters i.e. Cohesion and Angle of internal friction by using Rock Mass Rating System (RMR). Where, C should be in KPa (Eq.2.18) and (Eq.2.19).

$$\text{Cohesion, } c = 0.05(RMR) \quad (2.18)$$

$$\text{Angel of internal friction, } \phi = 0.5(RMR) + 5 \quad (2.19)$$

2.7. Rock Slope Stability

2.7.1. Slope Mass Rating

For evaluating the stability of rock slopes, Romana (1985) proposed a classification system called the "slope mass rating" (SMR) system. SMR is obtained from Bieniawski's rock mass rating (RMR) by subtracting adjustment factors of the joint-slope relationship and adding a factor depending on the method of excavation (Eq.2.20).

$$SMR = RMR_{basic} + (F_1 * F_2 * F_3) + F_4 \quad (2.20)$$

Where, RMR_{basic} is evaluated according to Bieniawski (1979, 1989) by adding the ratings of five parameters. F_1 , F_2 , and F_3 are adjustment factors related to joint orientation with respect to slope orientation, and F_4 is the correction factor for method of excavation (Table 2.5).

Table 2.5: Values of Adjustment Factor F_4 for method of excavation (After Romana 1985)

Method of Excavation	Value of F_4
Natural Slope	+15
Pre-Splitting	+10
Smooth Blasting	+8
Normal Blasting or Mechanical excavation	0
Poor Blasting	-8

F1- depends upon parallelism between joints and slope face strikes. $F_1 = (1 - \sin A)^2$, $A = (\alpha_s - \alpha_j)$, A denotes the angle between the strikes of the slope face (α_s) and that of the joints (α_j)

F2- refers to joint dip angle (β_j) in the planar failure mode. $F_2 = \tan \beta_j$

F3 – depends upon relationship between joint dip or plunge of line of intersection of two wedge forming planes and slope inclination. $(\beta_j - \beta_s)$ or $(\beta_i - \beta_s)$ where, α_s is the inclination of the slope.

F4. depends on the adjustment for the method of excavation. It includes the natural slope, or the cut slope excavated by pre-splitting, smooth blasting, normal blasting, poor blasting, and mechanical excavation.

The minimum and maximum values of SMR from Eq.2.20 are 0 and 100, respectively.

Romana (1985) used planar and toppling failures for his analysis. The wedge failures have been considered as a special case of plane failures and analyzed in forms of individual planes, and the minimum value of SMR is taken for assessing the rock slopes. Dip, β_i and dip direction, α_i , of the intersection of these planes should be taken as β_j and α_j respectively; that is, $\beta_j = \beta_i$ and $\alpha_j = \alpha_i$ where wedge failure is likely to occur. Values of adjustment factors F_1 , F_2 , and F_3 for different joint orientations are given in Table 2.6.

Table 2.6: Values of Adjustment factor for different joint orientation (after Romana 1985)

Case of slope Failure		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
P	$(\alpha_j - \alpha_s)$	$>30^\circ$	$30 - 20^\circ$	$20 - 10^\circ$	$10 - 5^\circ$	$<5^\circ$
T	$(\alpha_j - \alpha_s - 180^\circ)$					
W	$(\alpha_j - \alpha_s)$					
P/W/T	F_1	0.15	0.40	0.70	0.85	1.00
P	(β_j)	$<20^\circ$	$20 - 30^\circ$	$30 - 35^\circ$	$35 - 45^\circ$	$>45^\circ$
W	(β_j)					
P/W	F_2	0.15	0.40	0.70	0.85	1.00
T	F_2	1.0	1.0	1.0	1.0	1.0
P	$(\beta_j - \beta_s)$	$>10^\circ$	$10 - 0^\circ$	0°	$0 - (-10^\circ)$	$<-10^\circ$
W	$(\beta_j - \beta_s)$					
T	$(\beta_j + \beta_s)$	$<110^\circ$	$10 - 120^\circ$	$>120^\circ$	$>120^\circ$	-
P/W/T	F_3	0	-6	-25	-25	-60

The effect of future weathering on the slope stability cannot be assessed with rock mass classification, because it is a process that depends mostly on the mineralogical conditions of rock and the climate. In certain rock masses (e.g., some marls, clays, and shales), slopes are stable when excavated but fail sometime afterward, usually one to two years later. According to the SMR values, Romana (1985) defined five stability classes. These are described in Table 2.7 below.

Table 2.7: Stability classes as per SMR Values (after Romana 1985)

Class No.	V	IV	III	II	I
SMR value	0–20	21–40	41–60	61–80	81–100
Rock mass description	Very bad	Bad	Normal	Good	Very good
Stability	Completely unstable	Unstable	Partially stable	Stable	Completely stable
Failures	Big planar or soil-like or circular	Planar or big wedges	Planar along some joints and many wedges	Some block failure	No failure
Probability of failure	0.9	0.6	0.4	0.2	0

2.7.2. Discontinuity Analysis

It is important to categorize discontinuities according to the manner in which they are formed, essentially in geotechnical engineering, since discontinuities within each category usually have similar properties as regards both dimensions and shear strength properties that can be used in the initial review of stability conditions of a site (Wyllie and Mah, 2005).

In rock weakness of structural planes are generally characterized by the indication of discontinuities. According to Hoek and Bray the stability condition of rock slopes depends on the existence of this discontinuity planes.

The most readily apparent influence of the orientation of discontinuities on rock mass strength is evident in the failure of rock slopes along one or more discontinuities. The first step in the investigation of discontinuities in a slope is to analyze their orientation and combine the data using procedures that are easy for analysis. This is done under stereographic presentation of the structural data commonly referred as kinematic analysis. Kinematic analysis will allow to examine the direction in which a block will slide aside with the stability conditions.

2.8. Correlation among Geotechnical Parameters and Geophysical Techniques

The amount of information available for a rock mass can be increased by geophysical techniques, which can provide an indirect assessment of its engineering properties. Rock mass characterization needs comprehensive geotechnical, geophysical and engineering geological investigations of the ground conditions to assess the relevant parameters to be considered in the design of the structure.

The most sophisticated geophysical method for investigating rock mass properties is the seismic refraction method (Sjögren *et al*, 1979). This method is used widely for engineering purposes to provide an indirect assessment of the geo mechanical properties of rock masses. Many attempts have been made to relate empirically the geophysical properties of the rocks to their mechanical behavior (Knill, 1970; McCann *et al.*, 1990).

some rock mechanical parameters, such as fractur frequency and rock quality designation (RQD) has been correlated with compressional wave velocity (V_p) and an average regression curve was

derived for different rock types from an extensive study of borehole log samples (Sjögren *et al.* 1979).

The geomechanics classification or rock mass rating (RMR) and tunneling quality index (Q) are two systems used to assess the geo-mechanical characterization of the rock masses. Many attempts have been made to correlate the engineering classification systems, RMR, Q and RQD, with the seismic quality index (QI) which is defined as the ratio between the theoretical seismic velocity (calculated on the basis of the mineralogical composition of the rock) and the seismic velocity measured in situ.

According to a case study made on an investigation for a proposed dam in Wadi Mujib, Jordan, the following relation has been obtained (Fig 2.22), the relationship between the seismic velocity and RQD cannot be generalized because the former is a function of the actual stress state of the rock mass, while the latter is an index of the fracture state of the rock mass (Vuillermin, 1991 cited in El-Naqa (1996))

However, it seems to be more useful to correlate the seismic velocities with geomechanics classification systems because these classifications take into consideration not only RQD and fracture frequency but also other parameters of the discontinuity characteristics such as alteration, roughness, spacing, aperture, etc. that affect the geomechanics behavior of the rock masses.

The values of RQD, RMR and Q were correlated with the Quality Index (QI) (Eq.2.21) proposed by the French Group AFTES (Vuillermin, 1991 cited in El-Naqa (1996)), which is defined as:

$$QI = 100 \left(\frac{V_p}{V_t} \right) \quad (2.21)$$

Where, V_t is the theoretical velocity calculated on the basis of the mineralogical composition of the rock, and V_p is the seismic velocity measured near the boreholes. Improved results may be obtained by replacing the theoretical velocity by the actual velocity of un-weathered or unfractured rocks (Vuillermin, 1991 cited in El-Naqa (1996)). The correlation between the QI and different engineering classification systems, Deere's (1964) RQD, RMR, Q and Terzaghi (1946) are illustrated in Fig.2.22 below.

Deer (RQD)	Bieniawski (RMR)	Barton (Q)	Terzaghi classification	AFTES Group (QI)
1. 90–100 excellent	1. 81–100 very good	1. 400–1000 exceptionally good	1. Intact rock	1. 100–75 good
2. 75–90 good	2. 61–80 good	2. 100–400 extremely good	2. Stratified rock	2. 100–75 good
3. 50–75 fair	3. 41–60 fair	3. 40–100 very good	3. Moderately fractured	3. 75–50 fair
4. 25–50 poor	4. 21–40 poor	4. 10–40 good	4. Blocky and seamy rock	4. 50–25 poor
5. 0–25 very poor	5. < 21 very poor	5. 4–10 fair	5. Crushed rock	5. < 25 very poor
		6. 1–4 poor very poor	6. Squeezing rock	
		7. 0.1–0.4 very poor	7. Swelling rock	
		8. 0.01–0.1 extremely poor		
		9. < 0.01 exceptionally poor		

Fig 2.22: Correlation among the seismic quality index (QI) and common engineering geo mechanics classification systems (after Wadi Mujib dam, Jordan)

2.9. Genesis of methodology for the present study

A review on the background knowledge regarding the present research was undertaken as the backbone of this research topic. The literature review includes but not limited to published and unpublished reports of investigations, case studies, text books and journals which were found related and relevant to the research topic.

The aforementioned literature review, has clearly improvised the idea and the thorough knowledge regarding the rock properties both material and mass as well as the evaluation techniques in the feasibility stages. Safe design of large scale (mega) projects, require complete understanding of pertinent geological features and geotechnical information of the actual surface and subsurface materials.

The material characterization is also used to select a suitable construction site for the upcoming proposed project as well as for the surrounding areas with similar geologic setting and properties.

The rock mass characterization used an integrated approach mainly the rock mass classification and seismic refraction survey as a tool to characterize the rock mass while the rock material characterization basically used the time-honored manuals (ASTM, AASHTO and BS).

Chapter Three

3. Overview of the Study Area

3.1. General Physiography

Ethiopia can be divided into four major physiographic regions; the north western plateau, the south eastern plateau, the Main Ethiopian Rift and the Afar depression. The lowest elevation in Africa is located in the Afar depression, Dallol, at -126m below sea level, while the highest elevation is, Ras Dashen, at 4620m above sea level. The study area is found in the north section of the south western plateau.

3.2. Location and Accessibility

The current study area is located in south-western Ethiopia, in the Southern Nations, Nationalities, and Peoples Regional State, Gurage zone, Wolkite town. Geographically the study area is located between UTM coordinates of 341747-395826 mE and 900340-925228 mN.

The project area is accessed by Addis Ababa – Weliso– Wolkite-Jimma asphalt road. The newly constructed Wolkite-Gubre-Hosaina and Butajira asphalt road, and other gravel roads branching from the main asphalt road provide a direct access towards the study area. Fig.3.1 below illustrates the location map of the study area with respective test bore hole locations along the dam and tunnel section.

3.3. Climate, Vegetation and Land use

3.3.1. Climate

3.3.1.1. Temperature

According to the National Meteorological Agency, the wolkite station is one of the meteorology stations set in the southern Nations Nationalities and Peoples Regional State (SNNPR).

The whole area, lying in a tropical climate, classified as humid region (Ethiopian Meteorological Authority, 1981 cited in Dawd (2013)). In general, based on local climatic classification, Wolkite town is grouped under woeina-dega climatic zones.

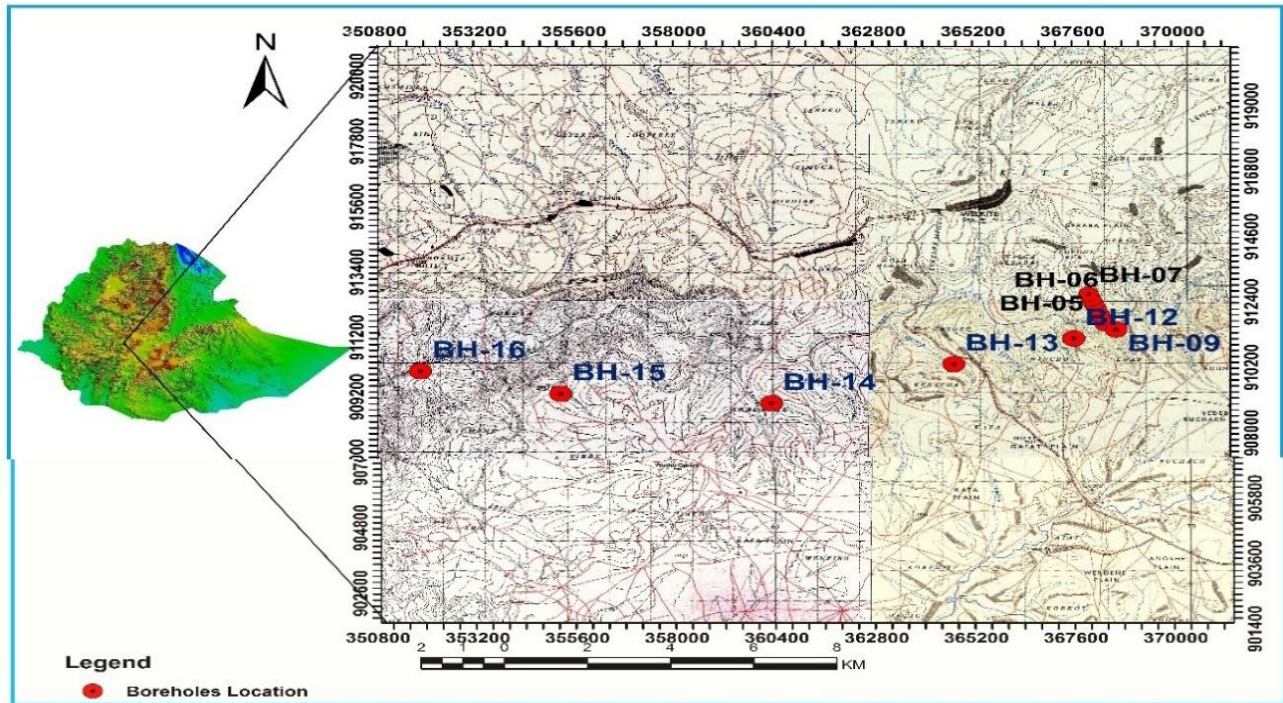


Fig 3.1: Location map of the study area (with respective boreholes)

Data obtained for air temperature within this station from the year 1993-2017 EC, for a range of twenty-five years at an altitude of 2000m has indicated a mean annual value of 17.28 C⁰, while the mean monthly maximum and minimum temperatures values are illustrated in Fig.3.2.

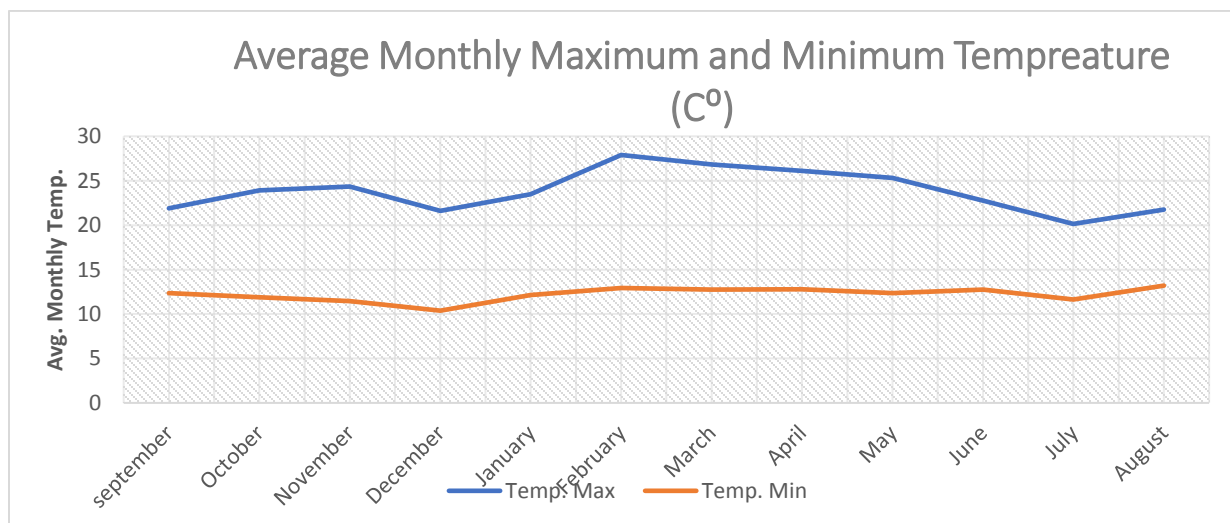


Fig 3.2: Average monthly maximum and minimum temperature of the study area (Ethiopian Meteorological Authority, 1993-2017 EC)

3.3.1.2. Rainfall

According to the National Meteorological Agency, Wolkite station is categorized under class-3, where only precipitation and temperature data are recorded. The mean annual precipitation for the year 1993-2017 EC, twenty-five years ranges, is 840.9mm. Around 90% of the annual precipitation occurs during the wet season, which is from late May to early September, while November, December, January and February are the driest months. This is summarized in Table 3.1 and Fig.3.3 below.

Table 3.1: Mean Monthly Rainfall of wolkite station for Twenty-five years

Station: Wolkite											
Elevation: 2000											
Latitude: 8.13											
Longitude: 37.45											
Period of observation:1993-2017											
Type of Data: Precipitation (mm)											
Month											
Sep.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.
1181	388.64	146.92	74.6	210.44	207.44	616.3	868.5	976.6	1630	2049	1735.7
Mean Annual Temperature= 840.39mm											

Based on the Ethiopian agro-ecological classification the Wabe site is located in the Woyena Dega ecological zone (Teshome, 2018). Regarding seasonal rainfall variability in Southern Ethiopia, Shone and Wolkite have shown a decreasing trend over the study period of 1983-2012 (Negash and Eshetu, 2016). In contrast to this, Abiy et al. (2014) reported that annual rainfall showed an increasing trend by 3.93 mm/year in Indibir station over the study period of 1982-2012.

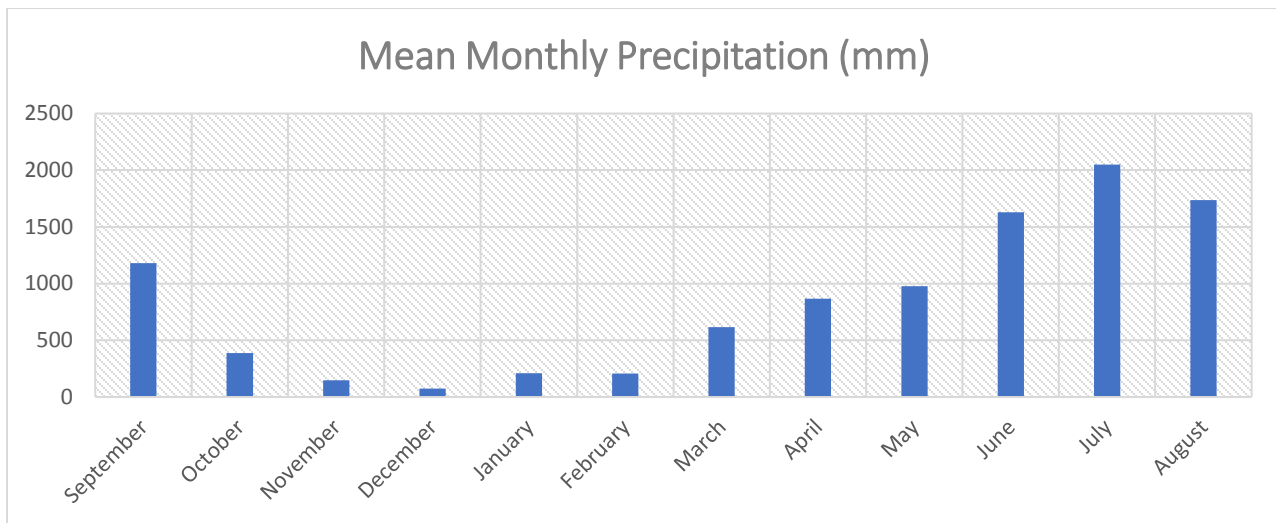


Fig. 3.3: Mean Monthly Precipitation (Wolkite Station, 1993-2017 EC)

3.3.2. Land use and land cover

Wolkite town has been built up on rugged terrain. Its elevation ranges from 1619 to 1868 m a.s.l dominated by undulating flat topography and low-lying areas. Gentle slope surface with a range of 0-2.8% accounts for about 55.51% area coverage and slope in between 2.9% - 24% accounts for about 42.78% while a slope greater than 24% covers about 1.7% of total land coverage (Beshir, 2014).

The land coverage of wolkite town according to Hailu et al. (2018), is categorized under the Land use land cover classes of Gibe Sheleko National Park, which includes forest, cultivated, shrub and grazing land. The study has showed, in 1990 EC the total forest land coverage in the town especially in the study area was 18,600 ha (46.5%), shrub land comprised of 12,600 ha (31.5%), cultivated land and grazing land covered 7,400 ha (18.5%) and 1,400 ha (3.5%) respectively.

3.3.2.1. Soil coverage

A detailed pedological study was conducted for the purpose of characterization and classification of soils in wolkite university research sites. A representative pedon was opened at the Wabe site with a depth >150cm and clay textural class. The PH of the surface soil ranged from strongly acidic (4.5) to moderately acidic (5.6), the soil has also medium (2.60%) to high (3.84%) organic carbon content (Yitbarek, Jembere and Kerebeh, 2018).

The moist color of surface horizons of the Wabe site varied from brown to dark brown, whereas the color of the subsurface horizons varied from dark reddish brown to very dark gray (Yitbarek, Jembere and Kerebeh, 2018). The surface horizons were darker as compared to the subsurface horizons that mainly could be due to accumulation and decomposition of organic materials, as it was discussed by previous studies (Mulugeta and Sheleme, 2010).

3.4. Regional Geology and structures

3.4.1. Regional Geology

The Southwestern Ethiopia plateau volcanic is separated from the Northwestern Ethiopia Plateau volcanic in the north by the E-W rift transversal structure called Addis Ababa-Nekemte tectonic line (Abbate and Sagri, 1980) or Yerer-Tulu Wollel Volcano Tectonic Lineament (YTVL) (Abebe et al., 1998).

The site lies within the upper reach portion of Omo – Gibe River Basin in the Southwestern Ethiopian Plateau (Richard Woodrooffe & Associates, 1993), which is characterized by flat-topped plateau, narrow to wide elevated ridges, low-lying plains and wide to narrow deep valley formed due to dissections of Gibe and Wabe Rivers and their tributaries. The geology of the project area is covered by Cenozoic rocks of the Jimma Volcanics, Makkonnen Basalt and Termaber-Megezez Formation

Another important work in understanding the geology of the area was conducted by the Geological Survey of Ethiopia (GSE) as part of the geological map of Akaki map sheet at a scale of 1:250,000 (Beshawered, 2012).

The geology of the project vicinity is represented by Cenozoic rocks of welded to partially welded pyroclastic flows (referred to as Nazret Series), and Tarmaber Megezez Formation.

According to a scaled map of 1:25,000, which is part of the prefeasibility investigation of the Wabe Hydropower project (ECDSWCo., 2019), the area generally covers contrasted physiographic areas that are formed during past geotectonic evolution of the southwestern Ethiopian Plateau (Fig.3.4).

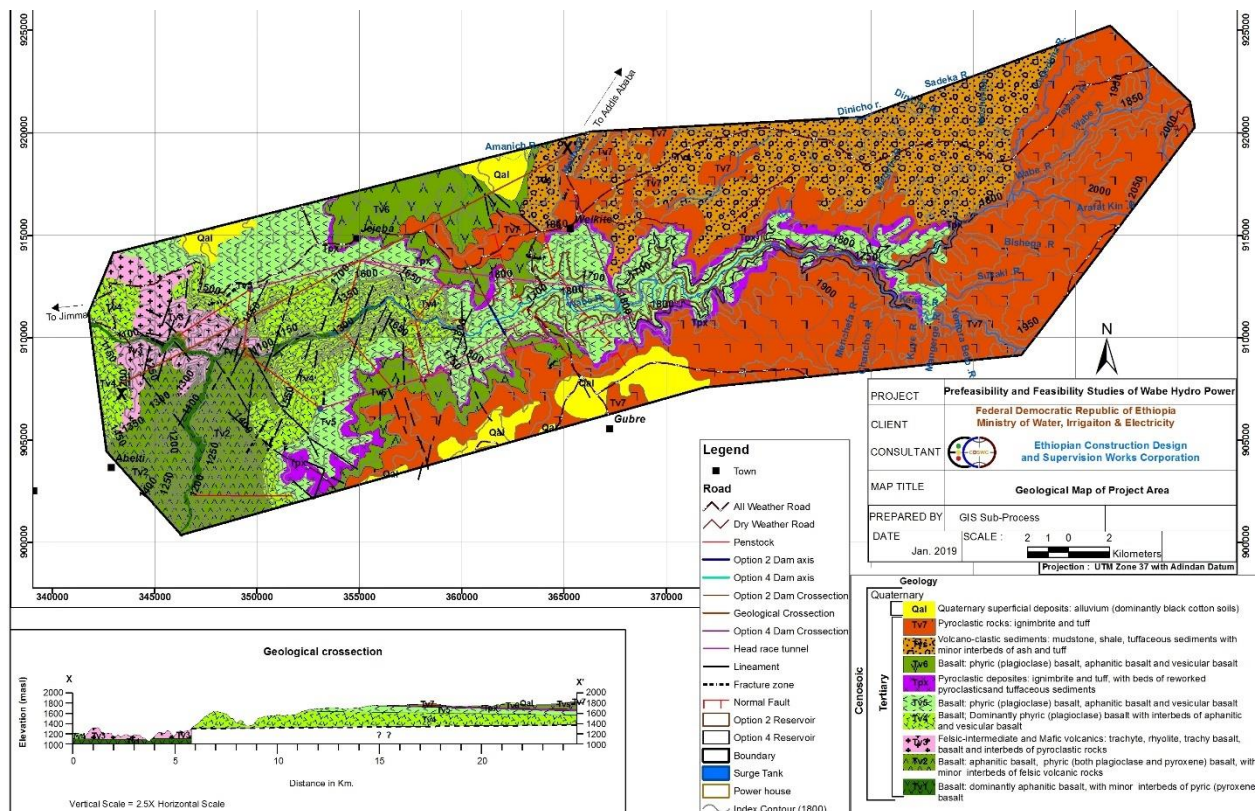


Fig 3.4: Geological map of the Wabe Hydropower project (Source: ECDSWCo., 2019)

Furthermore, the report has clearly implied, the present-day contrasting topography prevalent at the mapped area is the result of recurring and prolonged geo-tectonics and geomorphological processes; such as faulting/uplift/subsidence, weathering and erosion that heavily modified the landform for the present topographic scene. The elevation varies between 1059 m.a.s.l (dissected valley of the Gibe River) and 2063 m.a.s.l in plateaus.

The Cenozoic volcanic rocks of western and southwestern Ethiopia consist of dominantly basalts with subordinate silicic rocks, with thickness varying from 700 to 2000 m (Davidson, 1983; Berhe, 1987; Tefera et al., 1996).

3.4.2. Rocks and Stratigraphy

Until now, no unified litho-stratigraphic divisions are put forward for the volcanic of southwestern Ethiopia plateau. Various researchers subdivided the southwestern Ethiopian flood basalts based

on the origin of sequences. The most recent divisions of southwestern Ethiopian flood basalts include:

1. Tefera et al. (1996) identified two volcanic units in the southwestern Ethiopian flood basalts. (1) The Jimma Volcanics and (2) the Makonen Basalts.
2. Alemu (2014), classified the volcanics of southwestern Ethiopia into five units. From oldest to youngest, these are: - (i) the Gibe Group, (ii) the Limu Genet Group, (iii) the Arjo Group, (iv) the Jibat Group, and (v) trachyte flow and plugs.

3.4.3. Regional Physiography

As discussed earlier, the present study area generally covers contrasted physiographic areas that are formed during past geotectonic evolution of the southwestern Ethiopian Plateau. Besides the topography prevailing at the present study area which is mapped accordingly is the result of recurring and prolonged geo-tectonics and geomorphological processes including faulting/uplift/subsidence, weathering and erosion which in return has heavily modified the landform for the present topographic scene.

According to the prefeasibility and feasibility study report of Wabe hydropower project, ECDSWCo. (2019), on the basis of differences in altitude, prevalent topographic and geological features, the project area can broadly be divided into four major physiographic divisions These are:

1. Dissected valley, gorges and ridges

This physiographic division is characterized by rugged ridge/hills and dissected valley/gorges with contrasting topographic scenery of the Gibe and Wabe River valley/gorge. Incision which is accompanied by set of faults creating extensive fractures are followed by intense weathering and deep erosion finally forming the existing dissected landscape of the valley/river gorge.

They are formed by faulting, prolonged erosion and incision exposing rock units while cutting through, and producing topographic elevation differences in the order of 300 to 400m.

2. Sides of incised valley and steep cliffs

This physiographic division is characterized by narrow to broad ridges and domical hills comprised of basaltic flows and pyroclastic rock successions. It is dissected by streams and gullies, which formed dendritic to minor rectilinear drainage a pattern that flows in branching courses with many random bends and joining the Gibe and Wabe Rivers. Landslides are seen in some areas.

3. Gently rolling to steep cliffs, ridges and plains

This area is characterized by flat to low-lying terrain consists of plains and narrow stretches of steep cliffs, and rolling to hilly ridges. The plains formed flat topped and composed of rounded to sub-rounded pebble to cobble size basaltic rocks lying with the surrounding residual soils.

4. Flat-topped plateaus

This area is characterized by flat-topped gently undulatory plateaus with some ridges and hills. Elevations in this physiographic region vary between 1822 and 2063 m.a.s.l.

3.5. Local Geology and Structures

The geology of the Wabe Hydropower Project in general is characterized by Cenozoic rocks consisting of Tertiary (Neogene -Paleogene) volcanics and sediments, and Quaternary superficial deposits (ECDSWCo., 2019).

The Tertiary volcanics make up the largest part of the area are represented by mappable units, Tertiary basaltic unit 1-5 (Tb1, Tb2, Tb3, Tb4 and Tb5). The Tertiary sediments are volcanoclastic sediments (TPx), which are comprised of interlayered sandstone, mudstone with frequent inter-beds of reworked ash and tuff. From oldest to youngest the tertiary rock units are discussed below according to the above-mentioned report of ECDSWCo. (2019).

The geology of the reservoir section (dam site) has been scaled in 1:10,000 map, for the purpose of this work the geological cross section is illustrated in (1:25,000 scale) map (Fig.3.4).

3.5.1. Tertiary Volcanic Rock Units

3.5.1.1. Basalt (Tb1)

This rock unit is exposed in the river bed of Wabe River, which is composed of aphanitic and phyric basalts with subordinate vesicular and amygdaloidal basalts. The dominant rock type is the Aphanitic basalt which is fresh, grayish black, massive and attains an exposed thickness of 10 to 30 m. It is jointed with joints striking west-northwest and east-northeast and spacing measured between 10 and 20 cm while the phyric basalts are dominantly composed of plagioclase phenocrysts with minor amounts of olivine and pyroxene.

3.5.1.2. Basalt (Tb2)

The largest part of the reservoir area is occupied by this basaltic rock unit. It is comprised of highly weathered to decomposed interlayered aphanitic and vesicular basalts, which is attaining maximum thickness of 100 m. The aphanitic basalts are moderately to highly weathered and decomposed and appeared four to five times in sections interlayered with the vesicular basalt ranging in thickness from 10 to 15 m. The vesicular basalt also appeared three to four times in stratigraphy interlayered with the aphanitic basalt measured in thickness ranging from 3 to 20 m.

The vesicular basalt is moderately to highly weathered and decomposed and consists of vesicles. It makes up 20-30% of the volume of the rock. The vesicles are circular to sub-circular in shape, shallow and ranging in length from 0.5 to 3 cm.

3.5.1.3. Basalt (Tb3)

This rock unit is composed of slightly to moderately weathered interlayered phyric (plagioclase) basalt and aphanitic basalt with subordinate amygdaloidal (calcite-filled) basalt, which attains a thickness of 30-40 m. A paleosol (1-2 m thick), separating it from the underlying basalt (Tb2) exists. Its contact with the overlying basalt (Tb4) is marked by development of lateritic soil.

The phyric (plagioclase) basalt, slightly weathered to fresh and frequently observed interlayered with the aphanitic basalt, which attains a thickness of 5-10 m. The basalt is characterized by massive and sheet jointed varieties. In the amygdaloidal basalts the amygdules are calcite in composition and make up 10-20% volume of the rock.

3.5.1.4. Basalt (Tb4)

The mapped unit basically occupies flat topped plateau, and it is highly observed at around the tunnel route specifically in the head race tunnel alignment. It is unconformably overlies Unit Tb3 and underlies Unit Tb5. A laterite horizon separating the flow from the underlying basalt flow of Unit Tb3, and a pyroclastic deposit (Unit Tpx) is separating this flow from the overlying basalt flow of Unit Tb5.

3.5.1.5. Basalt (Tb5)

This unit attains an average thickness of 50 to 60m and forms flat topped ridges and plateaus. It is mainly comprised of fresh to slightly weathered phyrlic (plagioclase) basalts with subordinate aphanitic basalt and minor layers of amygdaloidal (calcite-filled) basalts.

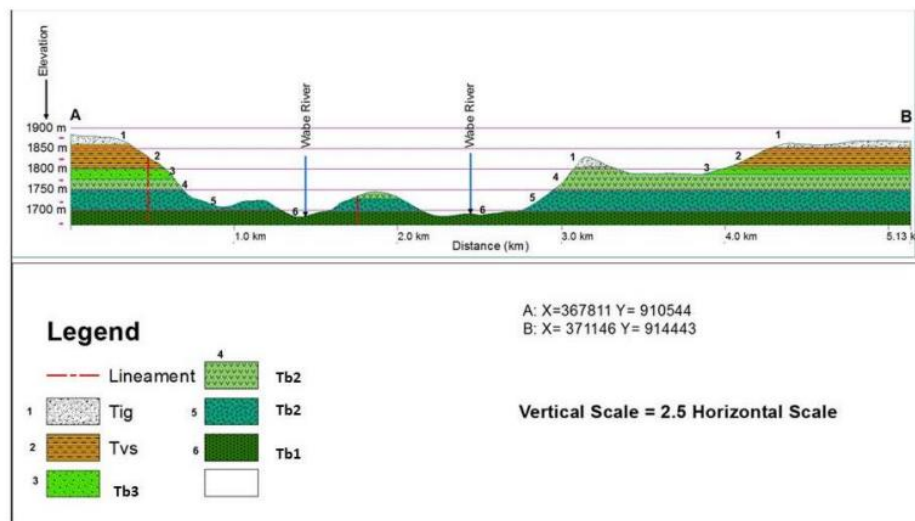


Fig.3.5: Geological cross section of reservoir section (dam site) (ECDSWC, 2019)

3.5.2. Tertiary Sediments

3.5.2.1. Volcano-Clastic Sediments (Tvs)

The volcano-sediments are comprised of interlayered sandstone, mudstone with minor interbeds of ash and reworked tuff, which is attaining a thickness of 10-20 m. Volcano-clastic sediments are unconformable overlying the basaltic rocks (map Unit Tb3), where the unconformity is marked by development of laterite soils. However, its contact with the overlying Tertiary ignimbrite and tuff (Tig) seems gradational.

3.5.2.2. Ignimbrite and Tuff (Tig)

This unit is represented by varying proportion of crystal, lithic and vesicular tuff and ignimbrite, which is attaining a thickness of 20-30 m. Its contact with underlying volcano-clastic sediments (Tvs) seems gradational. The ignimbrite is represented by crystal and lithic varieties. Crystal ignimbrite consists of phenocrysts of feldspar, which are aligned to define a flow texture. The lithic ignimbrite is moderately to poorly weld with pumice fragments.

The tuff is grey to light grey, and contains angular gravel to boulder size fragments of basalt, rhyolite, and pumice in tuffaceous matrix. The thickness laterally varies from few meters to 10 m at places.

3.5.3. Quaternary superficial deposits (Q)

The Quaternary superficial deposits are alluvial deposit mainly covers plains and base of gentle sloped hills. Mostly they are clay to silt size, less compacted and dark brown to black cotton soil. Usually they are thin-coated soil but at places considerable amount of thickness (about 5 m) were mapped.

3.6. Local Structures

The structures in the Wabe Hydropower Project area are characterized by brittle deformation, which resulted in the development of fractures, faults and joints. The intensity and distributions of these structures in space, and their geometric characteristics and relationships across the geology of the mega-structure differ instantly (ECDSWCo., 2019).

The fractures/joints in the dam site are grouped into three systems. These are: - (i) NW-SE to WNW-ESE, (ii) NNE- to NE, and (iii) ENE- to E-W systems. The NW-SE to WNW-ESE striking fracture systems represent the largest part of the dam site. They vary in strike from 15° to 40° W and 60° to 70° W and dip between 75° and 90° and oriented parallel to the NW-striking lineaments. In the case of the tunnel route, the fractures/joints in the area are grouped into three systems. These are: - (i) NW (330-335), (ii) WNW (290-305), and (iii) NNE- to NE (015-030). The NW-SE to WNW-ESE striking fracture systems is representing the largest part of the domain.

3.7. Seismicity

3.7.1. Seismicity of the region

The East African Rift System is a 50km to 60km wide zone of volcanoes and faults that extend north to south in Eastern Africa for more than 3000km (1864 miles). It cuts through Ethiopia in a NE-SW direction (Ayele & Kuthanek, 2007). In East Africa there are three main zones of seismic weaknesses each with specific geological, seismic and tectonic characteristics in the crustal segments (Haile, 2004).

The Afar Triangle (Afar Depression): The East African rift system, the Gulf of Aden, and the Red Sea make up the Afar Triangle (Ayele & Kuthanek, 2007). Extended faults of the main Ethiopian rift system are highly observed in this triangle with Small to intermediate magnitude earth quakes being expected. The 1969 earth quake with an intensity of 6.4 Mercalli scale occurred to the town of Sardo is the most notable earthquake in this zone with high levels of damage.

The main Ethiopian rift system: the zone is characterized by NE-SW trending fault systems (mainly the rift margins). Small to intermediate magnitude earthquakes are generated from this seismic source zone. The 1906 Langano earth quake with magnitude of 6.75 (UNISDR, 1999) and the 1987 rift valley area earth quake with magnitude of 6.2 (Kinde, 2002) are the notable earthquakes in this zone.

The Escarpment: this zone is dominated by N-S running faults, it is one of the most seismically active zones in the region. Small to intermediate magnitude earth quakes are also expected in this seismic zone. The 1961 Kara Kore earth quake with an intensity of 7 (UNISDR,1999) on the Mercalli scale is the most notable earth quake within this zone.

3.7.2. Seismicity of the study area

East African Rift especially its segment in southern part of Ethiopia is not adequately explored. Only very rough estimation of seismic hazard is available. the territory of Ethiopia is divided to five seismic zones (Gouin, 1979).

Earthquakes of big magnitudes are located mainly to the North part of the rift and to the Red Sea. However, it is due to the fact that written testimonials were better preserved in northern part of

Ethiopia. Certainly, it does not exclude the possibility of big earthquake in the southern part of the rift. Gouin, 1979 states the existence of former localities presumably destroyed by seismic tremors and now covered up by lakes. The legends are heard in the rift valley from Lake Zwai to Abbaya, and on its western escarpment in Gemu-Gofa.

The oldest documented earthquakes inside the rift in South Ethiopia are two events represented by strong earth quakes with magnitude of 6.6 and 6.8 in 1906. During this period of seismic activity, a geyser (hot spring water) started on the island in the northeastern bay of Lake Langano. As stated by Gouin (1979), earthquake with magnitude 6 was observed at the scarp of the rift, westward of lake Abay in the period of 1928. In 1960 the earthquake with magnitude of 6.3 was located inside the rift near Chabbi volcano. It was felled up to the distance of 200 km. Historical catalog from 1960 to 1980 contains also several smaller earthquakes with magnitudes between 4 and 5 (Gouin, 1979).

3.7.3. Seismic Hazard of the study area

Ground motion parameters are essential in describing and characterizing the ground motion intensity and severity. At a given seismic region the ground motion intensity is described in terms of peak ground acceleration (PGA). The PGA is expressed using seismic hazard of a region defined as the probability that a certain level of this parameter will be exceeded at a given place and within a given time period. The seismic hazard of the current study area is described in terms of the peak ground acceleration (PGA) as defined in Ethiopian Building Code Standard, ES EN 1998:2015 and Global Seismic Hazard Assessment Program (GSHAP).

3.7.3.1. Ethiopian Building Code Standard, ES EN 1998:2015

The seismic hazard map ES EN 1998:2015 was developed in the use for a 475-year return period (exceedance rate of 10% within 50 years) based on the European building code standard. As discussed earlier, the seismic hazard areas are characterized in terms of the ratio of peak ground acceleration at rock level to gravitational acceleration, g (bedrock acceleration ratio). Figure 3.6 illustrates the seismic hazard map of the study area.

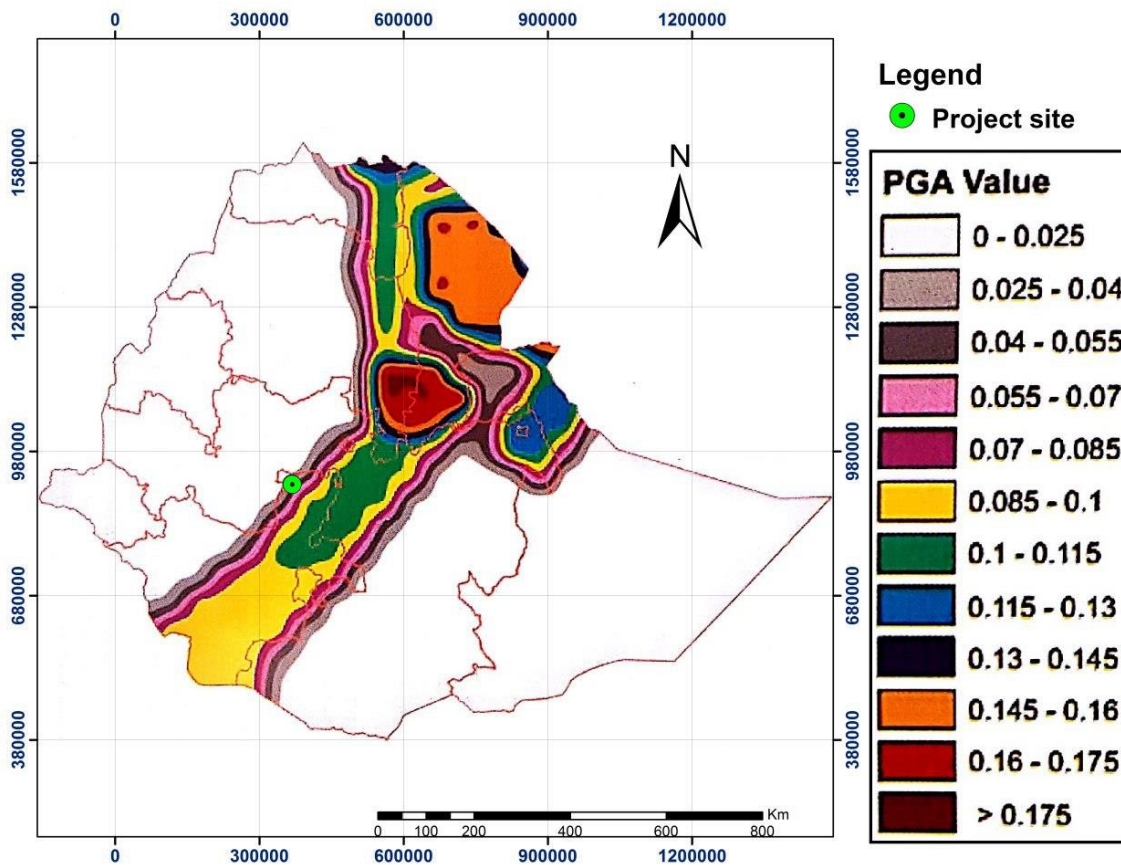


Fig.3.6: Seismic hazard map of the project site in terms of PGA (ES EN 1998:2015)

The code groups the country into six distinct seismic zones based on bedrock acceleration ratios as shown in Table 3.2.

Table 3.2: Bedrock Acceleration Ratio a_0 (ES EN 1998:2015)

Zone	5	4	3	2	1	0
$a_0 = a_g/g$	0.20	0.15	0.10	0.07	0.04	0.00

Wolkite town with an approximate location of 341747-395826 mE and 900340-925228 mN is assigned in zone 2 with PGA of 0.07. However, there is a significant inconsistency in the values of PGA obtained from the hazard maps and from the table listing described above. PGA for

Wolkite town obtained from the map (seismic hazard map along the Horn of Africa) is approximated to 0.04-0.05g (Fig.3.7) which is less than the value found earlier.

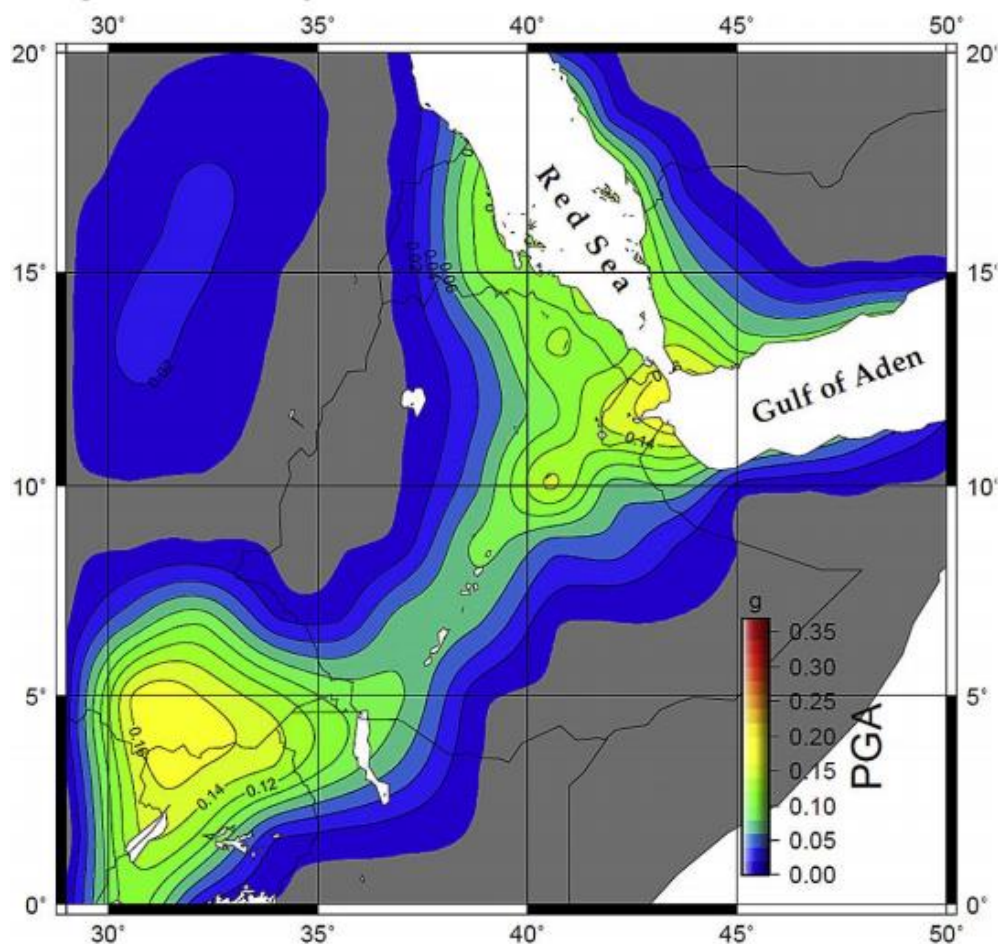


Fig.3.7: Seismic Hazard map along the Horn of Africa

3.7.3.2. Global Seismic Hazard Assessment Program (GSHAP)

The GSHAP was launched in 1992 by the International Lithosphere Program (ILP) with the support of the International Council of Scientific Unions (ICSU), and endorsed as a demonstration program within the frame work of the United Nations International Decade for Natural Disaster Reduction (UN/IDNDR). The GSHAP was designed to provide a useful global seismic hazard frame work and serve as a resource for any national or regional agency for further detailed studies.

The GSHAP report accompanies the publication of the GSHAP global seismic hazard map adjacent to regional maps produced for different GSHAP regions and test areas.

The seismic hazard map introduced by GSHAP is in terms of PGA values for 10% probability of exceedance in 50 years (return period of 475 years). The seismic hazard map extracted from this source for the region containing Ethiopia is depicted in Figure 3.8. Wolkite is characterized by a PGA between 0.04-0.05g.

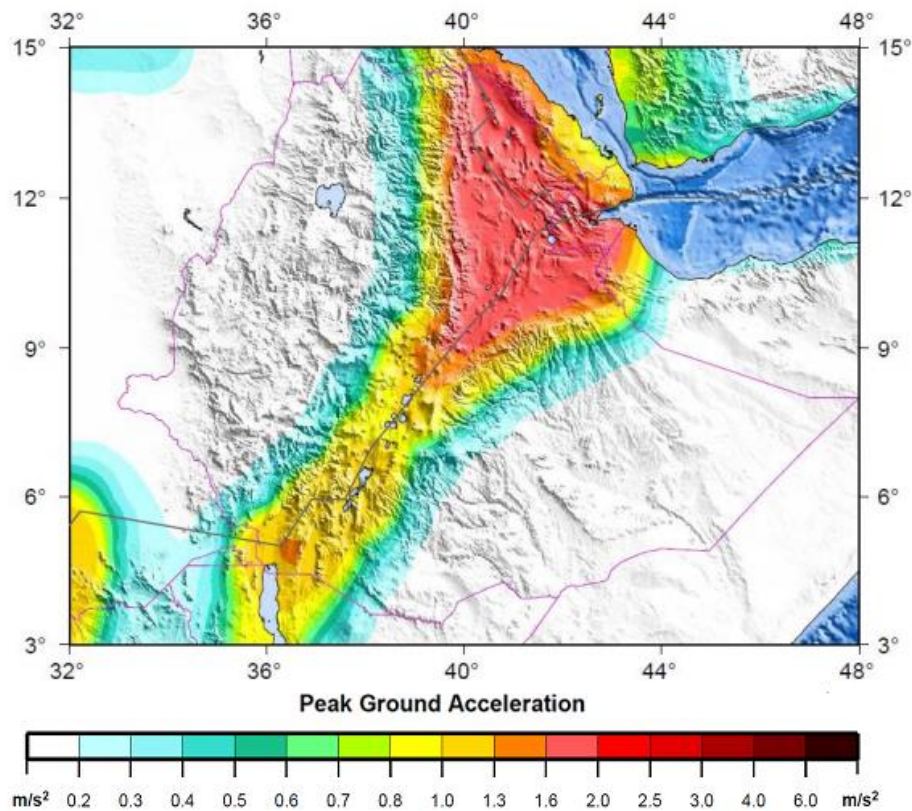


Fig 3.8: Seismic hazard map of Ethiopia based on the GSHAP data for a return period of 475 years

The PGA value for the widely accepted seismic hazard assessment program, GSHAP, should be adopted for further assessment of the intensity of seismic hazard in this area, since the new code ES EN 1998:2015 appears to overestimate the seismic hazard of wolkite town.

Chapter Four

4. Methodology and Data Collection

4.1 Rock Material Characterization Methodology

4.1.1 Introduction

The term rock mechanics refers to the basic science of mechanics applied to rocks while the term rock engineering refers to any engineering activity involving rocks (Hudson and Harrison, 2000).

The present section of this study deals with the techniques that will be used to characterize and evaluate both the rock material and rock mass that have been retrieved from active quarries and natural outcrops within the study area. In the first section of the study, methodologies followed to classify rock materials that is crushed and intact rock. In the second section, methodologies followed to characterize rock mass along the dam and tunnel section.

4.1.2 Field investigation and Analysis of Rock Material under in situ Conditions

4.1.2.1 Introduction

Behavior of rock material and rock mass in general cannot be determined purely from laboratory tests. The prediction of engineering properties of the rock mass always insists in situ tests.

In-situ rock tests are performed to determine various field parameters e.g. in-situ stresses and deformation properties of the rock/rock mass, shear strength of jointed rock mass or critically weak seams within the rock mass, residual stresses within the rock mass, anchor capacities, and rock mass permeability.

Large-scaled and number of in-situ tests tend to average out the effect of complex interactions. In-situ tests in rock are usually expensive. Well-conducted tests may be useful in reducing overly conservative assumptions.

4.1.2.2 Schmidt Hammer (Rebound Hardness) Test

The Schmidt hammer rebound hardness value (R_a) is the most frequently used index in rock mechanics practice for estimating the uniaxial compressive strength (UCS) and the modulus of elasticity (E) of intact rock both in laboratory conditions and in situ (ISRM, 1978a).

The harder the surface, the shorter the penetration time (i.e. smaller impulse) or depth (i.e. lesser work or energy loss) and hence the greater the rebound i.e. smaller momentum change. The distance traveled by the piston after rebound (expressed as a percentage of the initial extension of the key-spring) is called the hammer rebound value (HR), which is considered to be an index of surface hardness (ISRM, 2007-2014). Fig.4.1 illustrates Schmidt hammer test on the core samples retrieved from drilling.

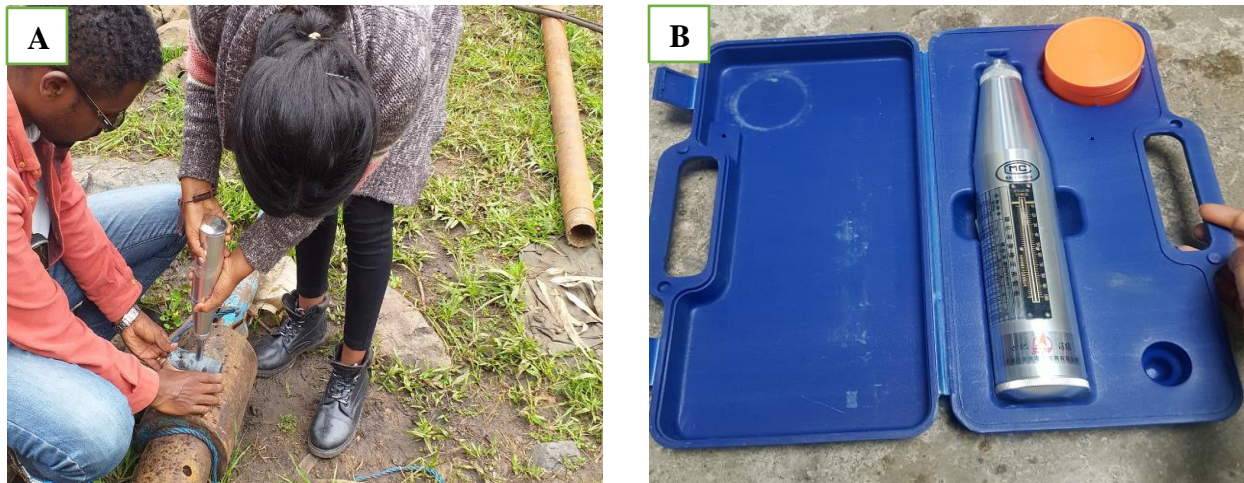


Fig 4.1: Schmidt hammer test on drilled core sample (A) and Typical Schmidt hammer (B)

4.1.2.3 Rock Quality Designation (RQD)

Generally, the RQD values provide a basis for making preliminary design decisions involving estimation of required depths of excavation for foundations of structures (Singh and Goel, 2011). It also provides an indication of rock quality in quarries for concrete aggregate, rock fill, or large riprap.

Low RQD values should be considered as a “red flag” for further action (Singh and Goel, 2011). Poor drilling techniques, core breakage upon handling, stress-relief, closely jointed zone and zone

of poor rock conditions with shearing and weathering are considered the reason for low RQD values.

RQD is a directionally dependent parameter and its value may change significantly, depending upon the borehole orientation therefore its application in rock mass classification systems should be strengthened with parameters that best represents the in-situ condition of the rock mass.

4.1.3 Analysis of Rock Material under Laboratory Conditions

4.1.3.1. Introduction

Laboratory tests have the limitations like variability and sample disturbance. Testing is done on small specimens and extrapolation of the measured properties for the entire site is often challenging. In contrast, in situ tests are the most acceptable and essential part of any geotechnical site investigation and design process.

The change in rock properties and behavior when acted upon by a certain type of loading is determined under testing. For the present work analysis under laboratory condition is dealt by considering physical (index) and mechanical property of the rock material.

4.1.3.2. Physical (Index) Property Tests of Rock Material (Intact Rock)

The index property tests performed include specific gravity, bulk density, water absorption, porosity and flakiness and elongation index.

4.1.3.2.1. Specific Gravity (Relative Density)

The specific gravity test of the rock materials to be used as a coarse aggregate for the purpose of dam foundation material and intact core recovered from drilling operation for the purpose of material characterization are performed according to AASHTO T-85 standards. (Fig.4.2).

4.1.3.2.2. Water Absorption and Porosity

The water absorption potential of the rock units to be used as coarse aggregate for the purpose of dam foundation materials in this case is performed according to ASTM-C-127-88 standards.



Fig 4.2: Soaked (A) and SSD (B) aggregate sample to determine Relative density and Water Absorption

Porosity is one of the governing factors for permeability. It is a fraction between 0 and 1. Porosity provides the void for water to flow-through in a rock material. High porosity, therefore, naturally leads to high permeability. Density and porosity are often related to the strength of rock material. A low density and high porosity rock usually, has a low strength.

4.1.3.2.3. Bulk Density (Unit weight)

For objects that consists of irregular shapes, that are highly fractured and made up of many pieces can cause difficulties in determining an accurate volume and thus the density (ASTM C-127-88).

4.1.3.2.4. Flakiness and Elongation Index

Shape tests of aggregate particles for use in various vicinities within the project are performed according to BS-812 by sampling fresh rock materials from one available active quarry site for seven consecutive production dates.

4.1.3.3. Mechanical Property Tests of Rock Material

Rock materials which are intended for aggregate purpose require strength tests, which are the basic mechanical property tests in stones. It includes satisfactory resistance to crushing under the roller during construction and adequate resistance to surface abrasion under traffic.

Strength, durability/toughness and soundness of rock aggregates will be tested based on standard testing procedures using acceptable strength and durability parameters such as, Aggregate Crushing Value, Aggregate Impact Value, Durability (Los Angeles Abrasion Value) and uniaxial compressive strength (UCS).

4.1.3.3.1. Aggregate Impact and Aggregate Crushing value

The aggregate impact value gives a relative measure of the resistance of the aggregate to sudden shock or impact while the aggregate crushing value provides a relative measure of resistance to crushing under a gradually applied compressive load.

To achieve a high-quality pavement, aggregate possessing low aggregate crushing value should be preferred. Fig.4.3 below illustrates sample preparation and test conduction procedures for aggregate strength test.



Fig 4.3: Sample Preparation for crushing value test (A) and Performing impact value test (B)

4.1.3.3.2. Los Angeles Abrasion Value (Durability test)

The ability of an aggregate to resist the impacting and grinding forces applied during manufacturing, placing, and compacting is so called toughness. The tests to measure the toughness of aggregate particles are described in ASTM C131 (Fig.4.4).



Fig 4.4: Prepared sample for LAA test

4.1.3.3.3. Uniaxial Compressive strength test

Compressive strength is the capacity of a material to withstand axially directed compressive forces. The most common measure of compressive strength is the uniaxial compressive strength or unconfined compressive strength. The UCS test requires preparation of sample as per ISRM.

UCS of rock material and deformation behavior under loading is verified by applying compressive load until failure using high capacity universal testing machine. The behavior of rock is best presented in a stress-strain curve (Fig.4.5).

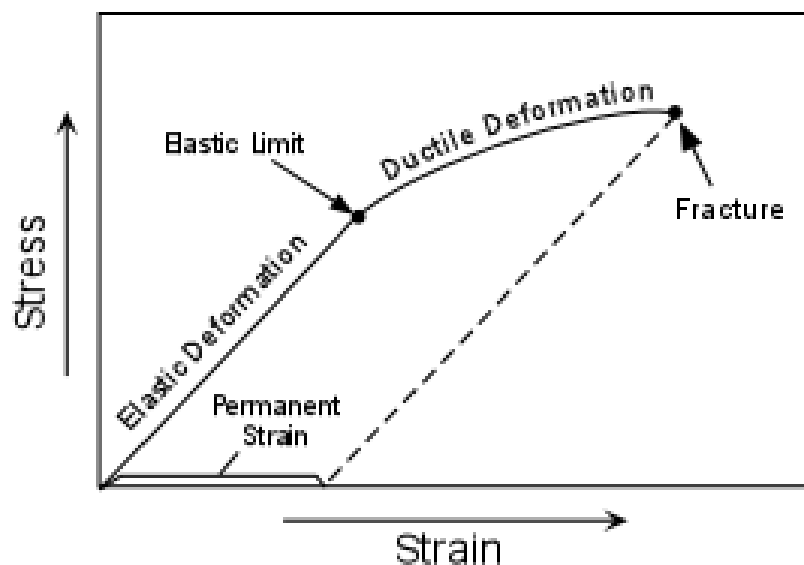


Fig 4.5: Ductile deformation mechanism for rock under uniaxial compressive load

4.1.4 Dynamic Elastic Modulus (Young's), Ultrasonic P-wave Velocity and Uniaxial Compressive Strength

Seismic wave velocity gives a physical measurement of the rock material. It is also used to estimate the Elastic Modulus of the rock material.

The compressional P-wave velocity (V_p) of a rock material is related to the Dynamic Elastic Modulus (E_d) and the density (ρ) of the material (Pierce, 1989). A correlation can be obtained using Eq.4.1 to solve for the dynamic elastic modulus (E_d) (Eq.4.2).

$$V_p = \sqrt{\frac{E_d(1-\nu)}{\rho(1+\nu)(1-2\nu)}} \quad (4.1)$$

From the above equation the dynamic elastic modulus can be solved as follows:

$$E_d = V_p^2 \frac{[\rho(1+\nu)(1-2\nu)]}{(1-\nu)} \quad (4.2)$$

Where, E_d : Dynamic Young's Modulus, V_p : P-wave Velocity, ρ : Density of the material and ν : the Poisson Ratio.

Young's Modulus of rock materials varies widely with rock type. For extremely hard and strong rocks, Young's Modulus can be as high as 100GPa. Poisson's Ratio measures the ratio of lateral strain to axial strain, at linearly-elastic region. It is 0.15-0.4 for most rocks.

4.2. Rock mass characterization Methodology

4.2.1 Introduction

This portion of the study will focus on characterizing the rock mass located within the vicinity of the dam site and throughout the proposed tunnel alignment. In preliminary design stages of large underground structures, in rock slope stability problems and dam projects, rock mass characterization plays an important role by acting as a valuable systematic design aid.

Rock mass characterization allows to understand the characteristic behavior of the rock mass and to relate the experiences gained in rock conditions at one site to another. Besides, in feasibility and preliminary design stages of a project, comprehensive information related to the rock mass

parameters, its stress, and hydrogeologic characteristics is mostly unavailable. Thus, rock mass classification proves helpful at this stage for assessing rock mass behavior.

4.2.2 Primary Data Collection

4.2.2.1 Geological Data

The aim of prefeasibility and feasibility study of the Wabe hydropower project is to set a geological map of scale 1:25,000 which allows in establishing the origin and distribution of rocks and overburden materials as well as the relationship between lithological and structural data and understand their significances in evaluating the suitability of the site for the design of the dam and its appurtenant structures (ECDSWCo., 2019).

In order to achieve the objectives of this study in general the geological data obtained include but not limited to:

- ✓ Litho-stratigraphic succession of the project area,
- ✓ A detail structural map and analysis of lithological and structural data for the suitability of the hydropower schemes at a scale of 1:10,000 at the reservoir area and 1:2,000 at the dam site (ECDSWCo., 2019).
- ✓ Geological cross-section at potential dam sites and along alternative tunnel alignments including power house

4.2.2.2 RMR Data Collection at surface outcrops

In situ rock mass properties at the project vicinity are obtained by collecting data from the field at surface exposures. Based on engineering geological judgment and the accessibility of the rock mass for visual description twelve representative sites are selected from both tunnel and dam sites. The location of the points is given in Table 4.1.

The geomechanics classification system as discussed in section (2.2.1.2) requires the determination of the six parameters which in turn requires dividing the rock mass into a number of geological structural units.

RQD is determined using relation proposed by Palmstrom (1982, 2005). For long and flat blocks clay free rock masses (Eq.4.3) or cubical shaped rock masses (Eq.4.4) is used. The uniaxial compressive strength (UCS) is determined by using Schmidt Hammer rebound number (R_a) along with the Barton and Choubey (1977) relation (Eqn.4.5).

Table 4.1: Field RMR data collection points

RMR ID	Easting	Northing	Elevation(m)	Rock Unit	Remark
OC-BH-16-1	0354182	0909233	1627.00		Tunnel section
OC-BH-16-2	0353817	0909097	1568.00	“	
OC-BH-15	0355376	0908594	1712.00	“	
OC-BH-12-1	0367345	0909740	1814.00	“	
OC-BH-12-2	0367369	0909954	1808.00	“	
OC-BH-13	0363851	0910404	1741.00	Aph. Basalt	
OC-Spillway	0367338	0912593	1874.00	Tuff	Dam Section
OC-BH-04	0367814	0911792	1687.00	Aph. Basalt	
OC-BH-06	0367479	0912337	1691.00	“	
OC-BH-07	0367515	0912311	1811.00	“	
OC-BH-09-1	0368191	0911241	1594.00	“	
OC-BH-09-2	0368204	0911294	1611.00	“	

Discontinuity spacing is measured using measuring tape among adjacent discontinuities. The condition and orientation of discontinuity are determined by inspection techniques such as visual observation and expert advice and suggestion.

$$RQD = 115 - 3.3J_v \quad (4.3)$$

$$RQD = 115 - 2.5J_v \quad (4.4)$$

$$\log_{10} \sigma_c = 0.00088\gamma R + 1.01 \quad (4.5)$$

In the above equations J_v is the discontinuities count, more than 10cm long, in 1m x 1m exposed rock surface, ' σ_c ' is the uniaxial compressive strength in MPa, ' γ ' is the Dry rock density in KN/m³ and 'R' is the Schmidt rebound number. Figure 4.6 illustrates a map showing field RMR data collection points.

Discontinuity data like dip, dip direction, strike and slope direction are measured in the field at selected outcrops using geological (Brunton) compass. Ground water condition is determined by the actual characteristics and geological conditions of the rock masses.

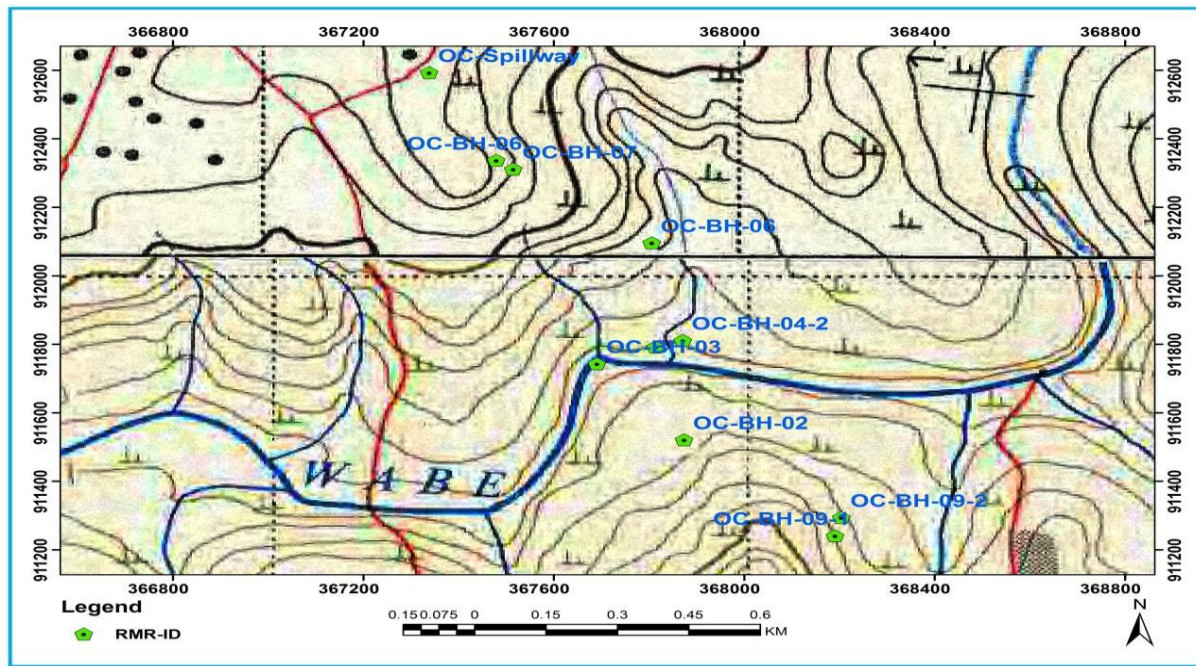


Fig 4.6: Figure showing field RMR data collection points

4.2.2.3 RMR Data Collection from drilled boreholes

The geological description on logging addresses the rock type identification, color and textural description, the degree of weathering, rock strength class, intensity of rock fracturing, and the type and extent of secondary infilling minerals. The RQD data are collected following the empirical relation proposed by Deer (1964) and expressed in Equation 2.1. Figure 4.7 illustrates in situ strength measurement on core sample.



Fig 4.7: Field In situ strength test on drilled core sample

During the current investigation, the core samples obtained from drilling operation are obtained with double tube core barrel, and proper precautions are taken to avoid or minimize core sample disturbance. Core breaks caused by the drilling process were evidenced by rough fresh surfaces thus fitted together and considered as one piece for RQD data collection.

4.2.3 Secondary Data Collection

4.2.3.1 Borehole Data

In characterizing the rock mass, borehole data were used as secondary data sources. A total of seven boreholes are used to characterize the rock mass within the dam section, (ECDSWCo., 2019). The boreholes are designated as BH-01 to BH-07 (Table 4.2 and Fig.4.8).

BH-09, BH-12, BH-13, BH-14, BH-15 and BH-16 are located along the tunnel alignment. Few boreholes from the tunnel section are not yet ready at full extent for investigation. Therefore, the available boreholes are used for investigation.

Table 4.2: Drilled borehole location and total depth along the Dam axis

Borehole ID.	Location	Total Depth(m)	UTM Coordinates		Elevation(m)
			Easting	Northing	
BH-01	Dam axis (Left-side)	90m	368010.75	911288.213	1794
BH-02	Dam axis (Left-side)	71.6m	367943.928	911455.379	1750
BH-03	Dam axis (Left -side)	70m	367877.279	911622.546	1712
BH-04	Dam axis (River-bed)	110m	367810.53	911789.713	1682
BH-05	Dam axis (Right-side)	90m	367743.782	911956.879	1710
BH-06	Dam axis (Right-side)	71m	367677.034	912124.046	1762.5
BH-07	Dam axis (Right-side)	89.5m	367610.286	912291.212	1807.00
Total drilled depth (m)		592.1			

The proposed tunnel section with the sub-components alongside has eighteen-meter (18m) length including the powerhouse, (ECDSWCo., 2019). The investigation process has represented each sub component along the section with vertical boreholes of varying and yet suitable depth. The number of drilled boreholes with their respective UTM coordinates and maximum drilling depth are summarized in Table 4.3 below.

Table 4.3: Drilled borehole location and total depth around the Tunnel Alignment

Borehole ID.	Location	Total Depth(m)	UTM Coordinates		Elevation(m)
			Easting	Northing	
BH-09	Intake	50	368273.7	911105.7	1812.0
BH-12	Inlet	25	367255.5	910799.7	1815.0
BH-13	Alignment	156	364371.9	909933.1	1765.0
BH-14	Alignment and middle shaft	(Under investigation)	359987.5	908615.4	1725.0
BH-15	Surge Shaft	174	354911.6	908939.0	1695.0
BH-16	Tail race tunnel and outlet Portal	400.5	351542.3	909698.9	1355.0
Total drilled depth (m)		805.5			

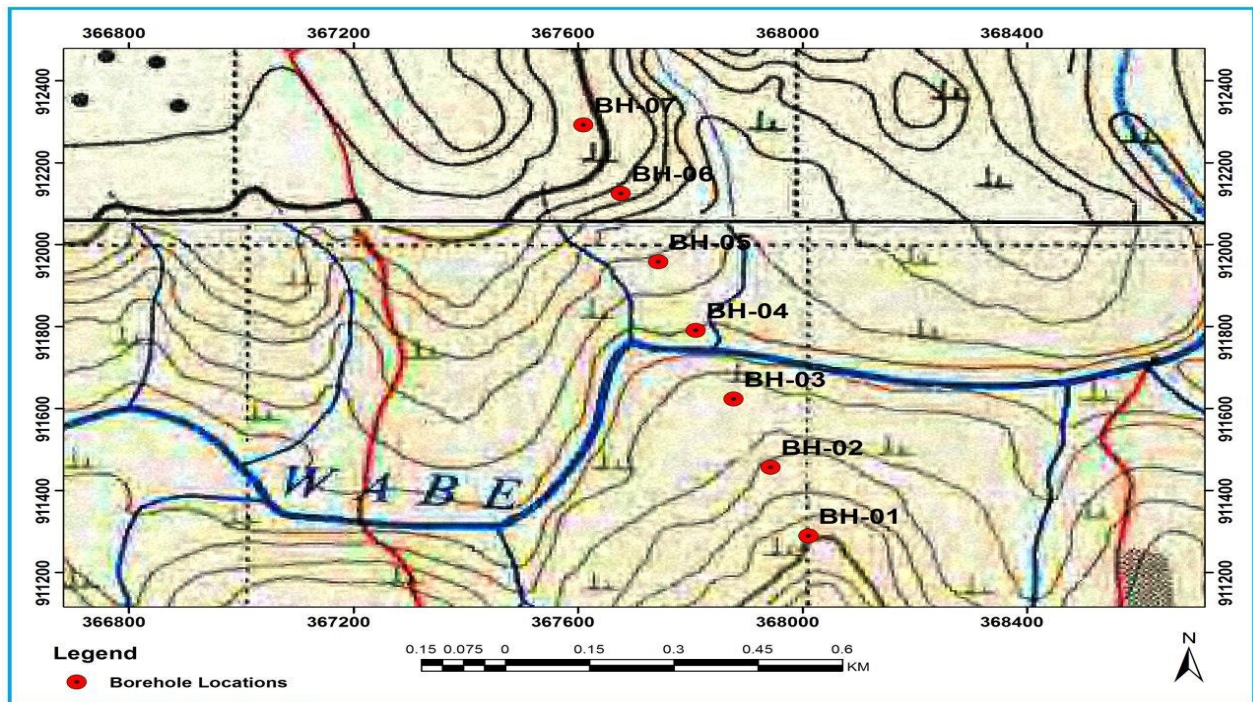


Fig 4.8: Drilled borehole location along the Dam section

In the case of this project in-situ permeability tests (Packer test) is conducted in each bore hole at every five meters interval, allowing the rock to inflate the gland. Induced pressure (P_{MAX}) is applied in an increasing and decreasing sequence in an order of R-S-T-S-R where R=one-third full of the induced pressure, S= two-third full of the induced pressure, T= three-third of the induced pressure= induced pressure.

4.3. Seismic Refraction Data Collection

The seismic refraction survey is conducted using the instrument called Seis-24 which is a specialized engineering seismic wave prospecting instrument. A total of 24 pieces of geophones are set, at equal interval with 10m interval along a straight line driven into the ground and connected with multi-channel cable to the main instrumentation. The shot points were marked at every 55 m of spacing, along the survey lines.

A total of seven lines which measured about 8847m length have been surveyed on the dam site. The survey line along the tunnel route consists of six lines with a total of 9045m length (ECDSWCo., 2019).

4.4 Rock Characterization Methodology for Foundation Rock Mass

In general, geotechnical investigation processes which intend to aid the preliminary investigation processes of mega structures usually follow a certain step that include standardized and acceptable methodologies. As for this specific research and for future enhancement of the whole project, rock foundation condition is analyzed by employing standard techniques specifically within the dam section.

4.5. Foundation Characterization

4.5.1 Rock Mass classification

In characterizing the rock mass within the vicinity of the dam site, rock mass rating (RMR) Geomechanics classification scheme developed by Bieniawski in 1989 is used. To apply the geomechanics classification system, the site is divided into certain geotechnical structural units in such a way that each type of rock mass is represented by a separate geotechnical structural unit with the six parameters determined for each structural unit separately.

The geotechnical units used in dividing the rock mass are based on the intensity of the compressional wave velocity, the fracture density of the rock mass and the overall rock quality (RQD) of the rock.

4.5.2 Rock Mass Strength and Failure Criteria

There are various failure/strength criteria applied to compare rock stresses and strength to describe rock failures. For this specific research the most common and widely accepted failure criteria, Hoek-Brown criterion has been utilized to estimate the strength criterion as well as the required shear strength parameters.

4.5.3 Deformation Assessment

The deformability characteristics of the rock mass within the vicinity of the project area is characterized using the most recent empirical method introduced by Serafim and Pereira (1983), that compromised of correlations imposed by Bieniawski (1978), Agarwal et al (1991), Barton et al. (1980), Hoek and Brown (1997) and Hoek et al. (2002).

4.5.4. In-situ Permeability Assessment

For the present study the permeability condition of the rock mass is determined as per Fell et al. (2005) after Lashkaripour and Ghafoori (2002), and the corresponding hydraulic domain classifications is determined as:

- ✓ Impervious, if Lugeon Value is 0-3,
- ✓ Low permeability, if Lugeon Value is 3-10,
- ✓ Medium permeability, if Lugeon Value is 10-30,
- ✓ High permeability, if Lugeon Value is 30-60,
- ✓ Very high permeability, if Lugeon Value is >60

4.5.5. Bearing capacity Estimation

Bowels (1996) suggested a correlation among UCS and RQD (Eq.4.6) to obtain the safe load bearing capacity of rock mass intended for the purpose of foundation material using an optimum factor of safety. The value so obtained are not reliable unless combined with the results obtained from Hoek-Brown Criteria (2002) and RocLab software developed by Rock Science.

$$\text{Safe load bearing capacity} = \frac{UCS * (RQD)^2}{\text{Factor of safety}} \quad (4.6)$$

4.5.6. Seismic refraction in characterizing the rock mass

The compressional wave velocity obtained from seismic refraction survey is used to characterize the rock mass along the dam site in terms of rock quality, degree of weathering and inferred fault zones. The seismic data in conjunction with the borehole data is also used to characterize the foundation rock mass.

4.6. Rock Characterization Methodology for rock mass along the tunnel route

The design and construction of underground structures such as tunnels, the basis of the rock mass classification and characterization is highly indebted to in-situ rock parameters obtained at preliminary investigation phase. Though what will be encountered during excavation could bring

out new parameters for investigation and design, yet initial stages are dependent on parameters obtained from characterization scheme.

In this study, the Rock Mass Rating (RMR) or Bieniawski's Geomechanics Classification (1988) and Rock Mass Quality Index (Q-Index) (Barton et al.,1974) are used to characterize and classify the rock mass along the proposed tunnel section.

The geological units identified and described in chapter three of this study has been categorized as per chainage and elevation of which it exists among the engineering structures of the tunnel section or the tunnel route. Each representative borehole along the tunnel section is composed of geological units which are carefully identified and mapped by geologists from ECDSWCo.

Intake Structure (Around BH-09)

The predominant geological unit around the proposed intake structure at chainage 0+495m, from elevation 1807 to 1756m, is **Tb4**, which occupies a flat-topped plateau and covering an extensive area of the head race tunnel as well. This unit is underlain by another geological unit, Tb3, unconformably, while the unconformity is represented by the existence of a separating layer, in this case a laterite horizon.

Head Race Tunnel (Around BH-13)

The geological unit dominating around the proposed tunnel route (specifically at around BH-13) which is considered as the alignment section at chainage 0+2753 to 0+5250 and from elevation 1700 to 1639 is **Tb3**. This unit is comprised of dark grey and grayish black, fresh to slightly weathered massive to weakly jointed Aphanitic basalt and phyric (plagioclase) basalts observed towards the top part of the succession as interlayers.

Vertical Shaft (Between BH-15 and BH-16)

The geological unit found at the location of the vertical shaft at chainage 0+14505m and elevation from 1215m to 1161m is **Tb2**. This unit is exposed and is fresh to slightly and moderately weathered phyric (plagioclase) basalt and interlayered with aphanitic basalt. Vesicular basalt where also observed within this unit.

Power House (Around BH-16)

Tb1 is the geological unit found at around the power house section within the chainage 0+14505 to 0+17462 from elevation 1148 to 1098. The dominant rock type (**Tb1**) within this unit is a fresh, grayish black, and commonly massive at some places jointed, aphanitic basalt.

Table 4.4 summarizes the lithological units along the proposed tunnel route by considering few of the engineering structures within the section. However, other engineering structures exist with their respective geological units over a proposed chainage and elevation along the tunnel route that are not dealt with in this work.

Table 4.4: Summary of Chainage and Elevation of the different civil structures with respect to the geologic units

Engineering Structures	Intake Structure		Head Race Tunnel		Vertical Shaft		Power House	
Dominant BH-Location	Around BH-09		Around BH-13		Between BH-15 and BH-16		Around BH-16	
Geological Structures	Tb4		Tb3		Tb2		Tb1	
(in meter)	Chain.	Elev.	Chain.	Elev.	Chain.	Elev.	Chain.	Elev.
	0+493	1807 to 1756	0+2753 to 0+5250	1700 to 1639	0+14505	1215 to 1161	0+14505 to 0+17462	1148 to 1098

Chapter Five

5. Result Interpretation and Discussion

5.1. Rock material characterization

5.1.1. In situ Investigations

5.1.1.1. Schmidt Hammer

The Schmidt hammer rebound value (Ra) of the intact rock samples obtained from both the borehole and natural outcrops within the vicinity are presented in Fig. (5.1) and Fig. (5.2). The Schmidt hammer rebound number is high for fresh and unfractured intact samples of the aphanitic and amygdaloidal basalts (64.9 and 53 respectively). Relatively, the slight to moderately fractured phyrlic and vesicular basalts show a maximum rebound value reaching up to 51 and 47 respectively.

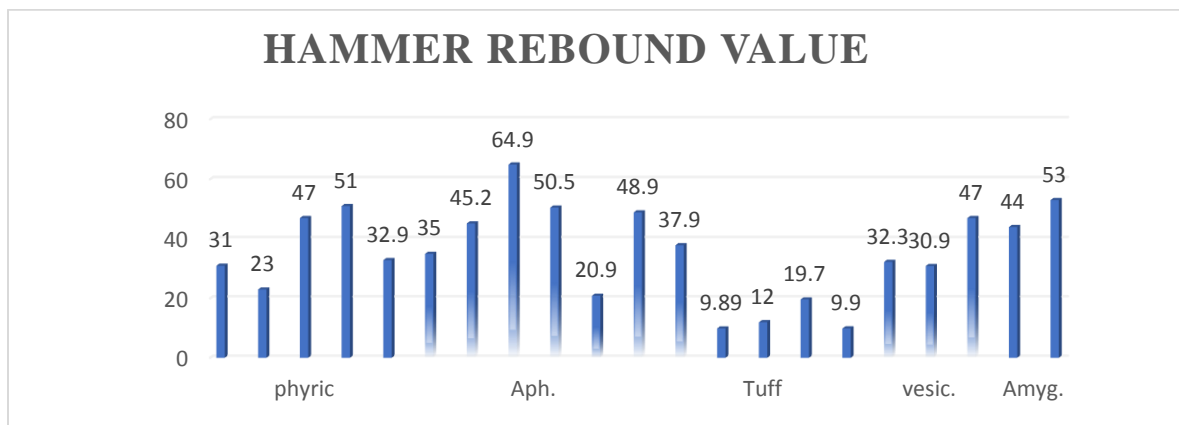


Fig 5.1: SHV of basalt and tuff rock core samples

For the case of the vesicular basalt and tuff samples obtained, most cores recovered were under a highly weathered zone which has led the rebound values to be very small and some even have a value of zero (BH-03, BH-04 & BH-07).

The rebound (Ra) value obtained from the field outcrop for a highly weathered basaltic rock unit is 11.67. Aphanitic basalts outcropped around the river bed have Ra of 64.5.

The overall implication of the obtained results is that Schmidt hammer rebound number increases as density increases. The fracture density and intactness of the rock material have a significant influence on the ability of the material to resist permanent deformation.

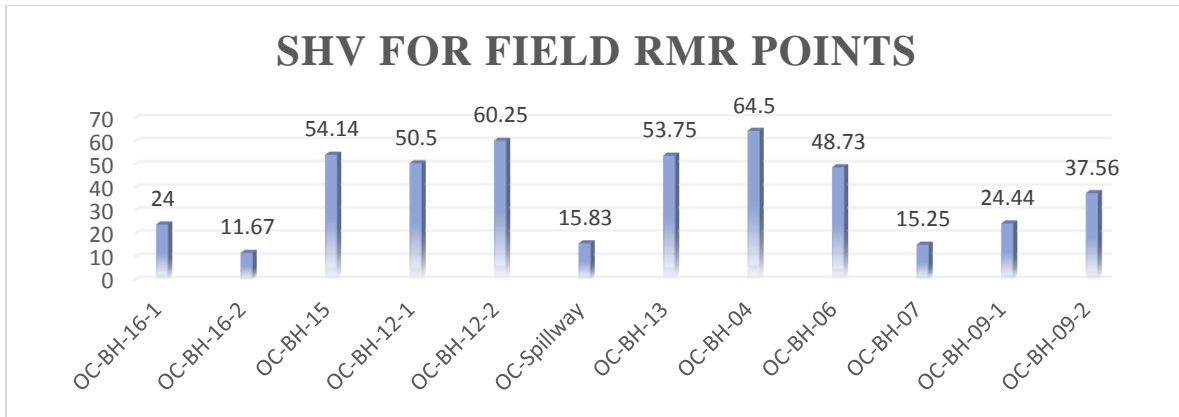


Fig 5.2: SHV of rocks from field RMR points

Schmidt hammer rebound hardness tests are insensitive to strength changes and the results are strongly influenced by variation in testing techniques (WES, 1982). Therefore, it is important to consider the mean Ra value in estimating in-situ strength of the rock material.

5.1.1.2. Rock Quality Designation (RQD)

The RQD values obtained for Aphanitic basalts ranges from 6.2-94.2%; for amygdaloidal basalts the values range from 39.4-92%; for vesicular basalts RQD value ranges from 23-76.3%; and for the case of phyrlic basalts and tuffs it ranges from 3.3-90% and 5-54% respectively (Fig.5.3).

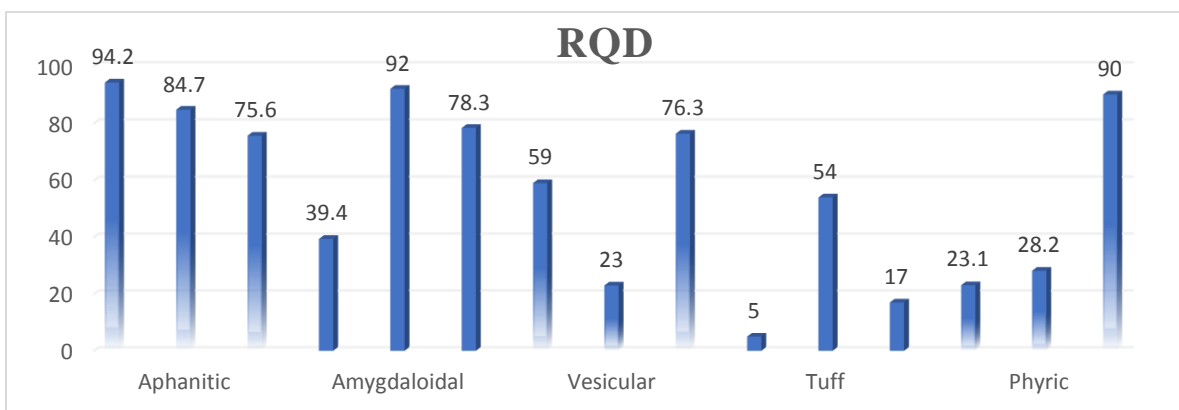


Fig 5.3: RQD of the intact rock core samples obtained

Based on the data obtained the RQD and the engineering rock mass quality can be related according to Deer (1968); accordingly, the tested rock material samples fall in fair (50-75%) to good (75-90%) rock quality.

The Schmidt hammer rebound value and RQD do not sufficiently describe the in-situ behavior of the rock material and should be taken as indicative parameters in describing the rock quality.

5.1.2. Laboratory Test Results

5.1.2.1. Introduction

The availability of construction materials for mega structures like dams usually require a nearby quarry site, where materials can easily be mobilized towards the construction site. As for the case of the Wabe hydropower project, quarry site is selected at a preliminary level. For the purpose of this study, rock material characterization has been made from borehole core samples and from an active quarry which is located within the vicinity of the project site which is currently serving as a local quarry site and is also likely to serve the project.

5.1.2.2. Physical properties

5.1.2.2.1. Water Absorption (WA) and Specific Gravity (SG)

The water absorption for the fresh and unfractured to the highly weathered and fractured aphanitic basalt samples varies from 0.47-8.56% respectively. The amygdaloidal and vesicular basalts which are retrieved at a limited section during drilling have a WA of 0.37% and 2.56% respectively. The quarry materials have a WA ranging from 0.102-0.241% (Fig.5.4).

Generally, the aggregate rock samples have a water absorption falling within the standard range for use in road surfaces and base coarse aggregates that is 0.1-4.0%.

The specific gravity of the basaltic rock core samples ranges from 2.51-2.9% while, the tuffs have a specific gravity of 2.02%. The aphanitic basalt yields the highest specific gravity due to its compact and massive nature which has also a direct relation with the strength. For the case of the materials obtained from the local quarry site the SG value ranges between 2.35-2.74% (Fig.5.5).

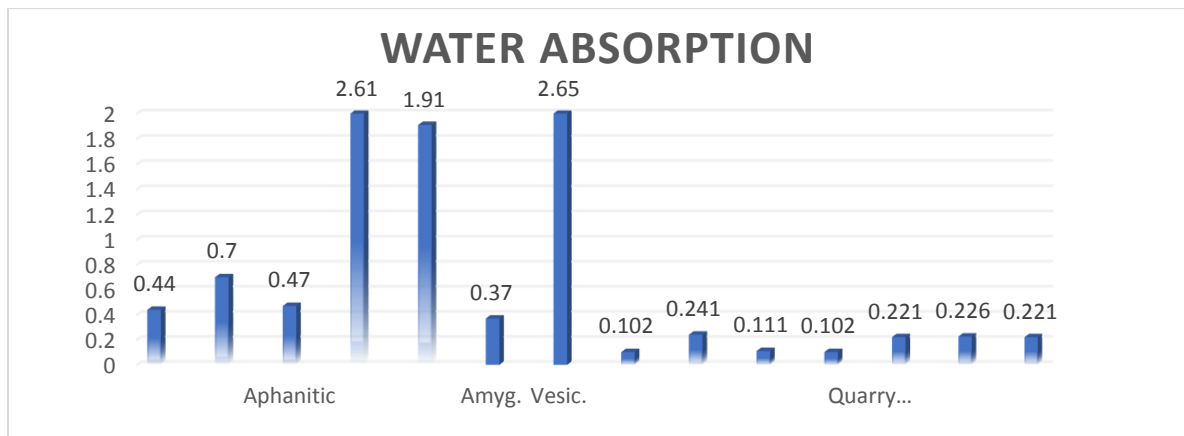


Fig 5.4: Water Absorption of some basaltic rock core samples

Though, high specific gravity of aggregate is considered as an indication of its high strength, it is not possible to judge the suitability on this basis alone without finding other mechanical properties like aggregate abrasion value or other aggregate mechanical properties.

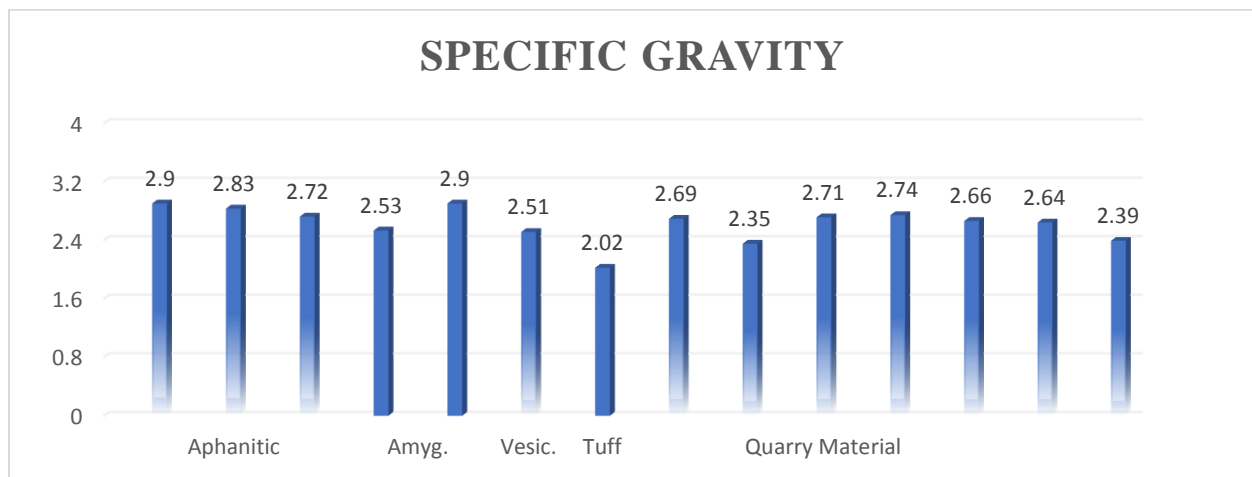


Fig 5.5: Specific gravity of some tuff and basaltic rock core samples

5.1.2.2.2. Porosity and Bulk unit weight

Porosity is the most important factor to properties of many rocks and which in return has an effect on the physical property of the rocks. The range of values obtained for porosity from the drill rock core samples are presented in Fig.5.6.

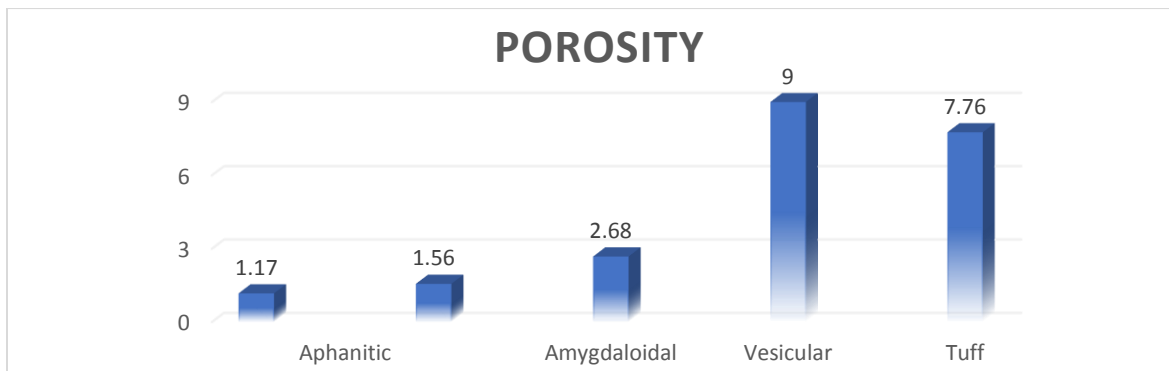


Fig 5.6: Porosity of tuff and basaltic rock core material

Based on the obtained porosity values a relation can be made regarding strength and deformability after Anon (1979) and Dearman (1983). Therefore, the basaltic core samples have a low porosity while the tuff and vesicular basalt have relatively high porosity values.

Similarly, according to Dearman (1993) (Table 5.1), the samples fall in the range of strong to extremely strong with a very slight deformability characteristic. Therefore, the basaltic units within the vicinity can be used as good sources of construction materials.

Table 5.1: Relationship between porosity, compressive strength and deformability (after Turk and Dearman (1983))

Porosity (%)	Strength	Deformability
0–5	Extremely strong	Very slightly deformable
5–10	Very strong	Very slightly deformable
10–20	Very strong	Slightly deformable
20–30	Strong	Slightly deformable
30–50	Strong	Moderately deformable
>50	Moderately strong	Very deformable

The values of the bulk unit weight for the drilled core sample ranges from 1.87 g/cm³ for the tuffs, 2.79 g/cm³, 2.82 g/cm³ and 2.37 g/cm³ for the aphanitic, amygdaloidal and vesicular basalts respectively (Fig.5.7).

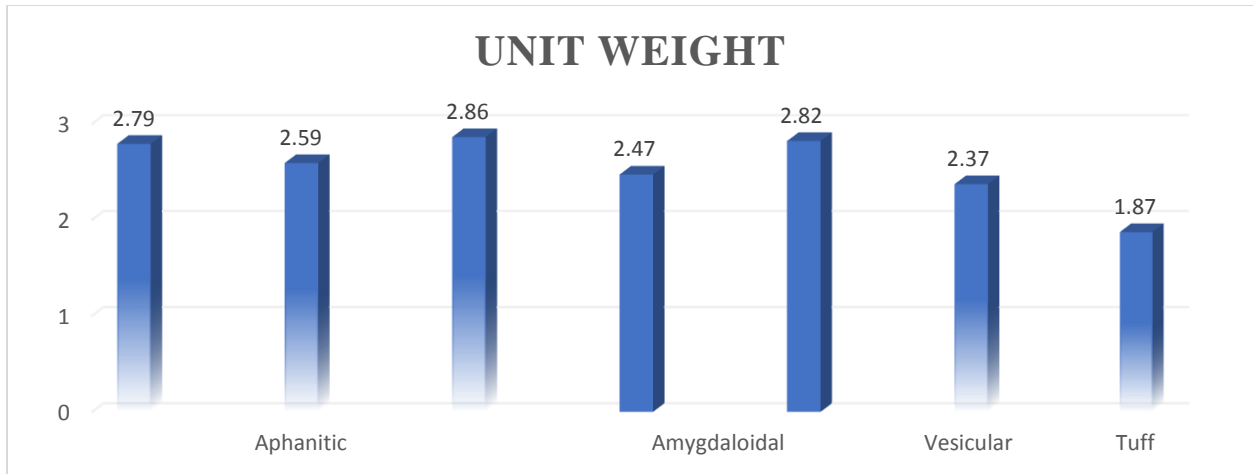


Fig 5.7: Unit weight of some rock and tuff core samples

5.1.2.2.3. Flakiness index and Elongation index

The shape tests are performed on aggregates sampled for seven consecutive production dates. The values of the elongation and flakiness index of the rock material obtained as a crushing product of the crusher are illustrated in (Fig.5.8) below. It is to be recalled that the quarry rock is mainly Aphanitic basalt.

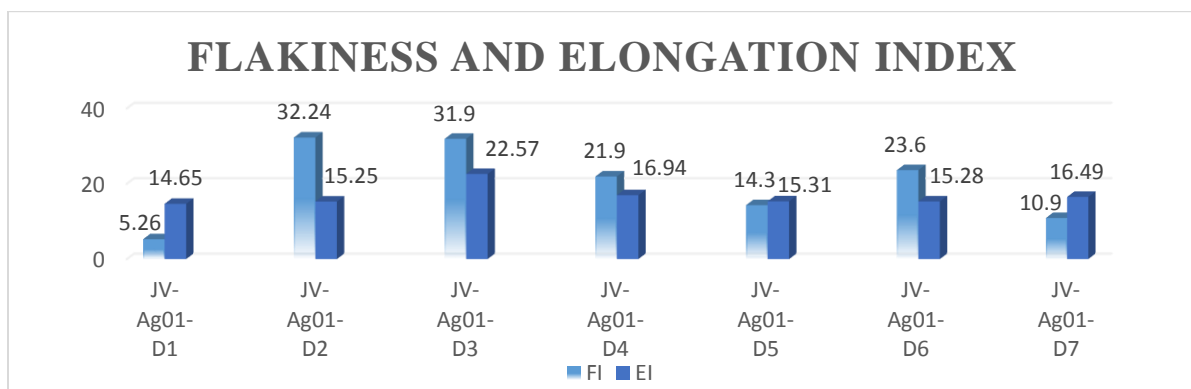


Fig 5.8: EI and FI of quarry material found near the locality

Some FI values are beyond the limit stated by BS-812 (32.24% and 31.9%), and this is a clear indication of either the reduction ratio or the type of crusher used during the crushing process. Either or both of the factors has an impact on the compaction process which in return affects the end product of concrete mix by inducing segregation effect.

5.1.2.3. Mechanical properties

5.1.2.3.1. Determination of uniaxial compressive strength

The laboratory uniaxial compressive strength (UCS) values of the intact basaltic samples range from 11.89 MPa for highly weathered aphanitic basalt to 237.29MP for a fresh aphanitic basalt. The amygdaloidal, vesicular and phytic basalts which were retrieved at a limited section during drilling have a UCS of 118.72MPa, 12.03MPa and 52.30Mpa respectively. The pyroclastic natured rocks (Tuffs) have a very low uniaxial compressive strength ranging up to just 1.38 MPa (Fig.5.9).

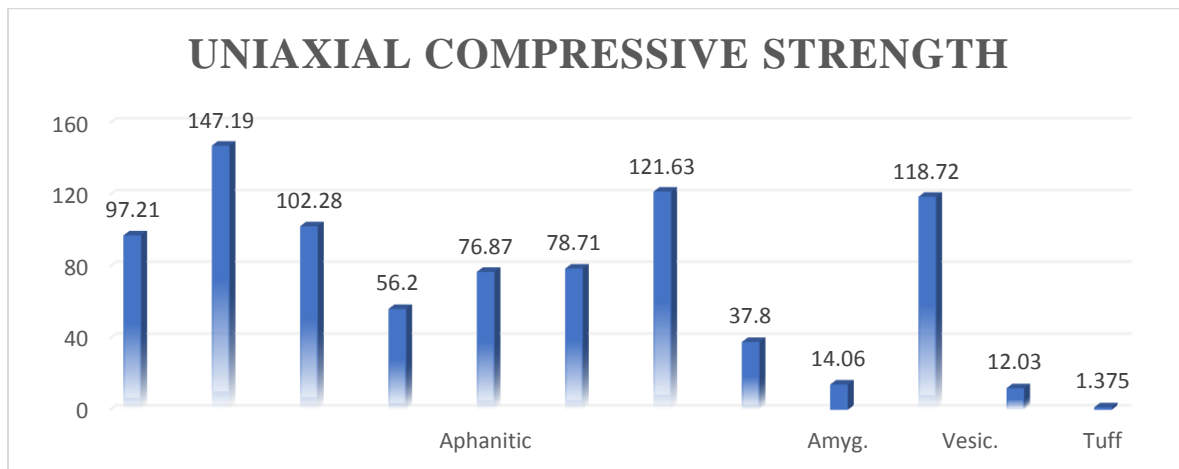


Fig 5.9: Laboratory UCS of tuff and basaltic rock core samples

5.1.2.3.2. Determination of abrasion resistance (Los Angeles abrasion value)

In order to determine the abrasion resistance of the aggregates found from the existing quarry, the Los Angeles abrasion test was carried out for seven consecutive production dates. The range of values obtained is listed in Table 5.2 and illustrated in Fig.5.10 below.

Table 5.2: Los Angeles abrasion values of the tested samples

Sample Id.	JV-Ag01-D1	JV-Ag01-D2	JV-Ag01-D3	JV-Ag01-D4	JV-Ag01-D5	JV-Ag01-D6	JV-Ag01-D7
LAHV (%)	28%	29%	26.11%	24.79%	25.11%	24.32	28.31

The LAHV of aggregates for use in pavement construction (road surface and base coarse aggregates) as specified by AASHTO is less than 45%. The tested samples are within the range specified by AASHTO and can be used in any kind of pavement construction.

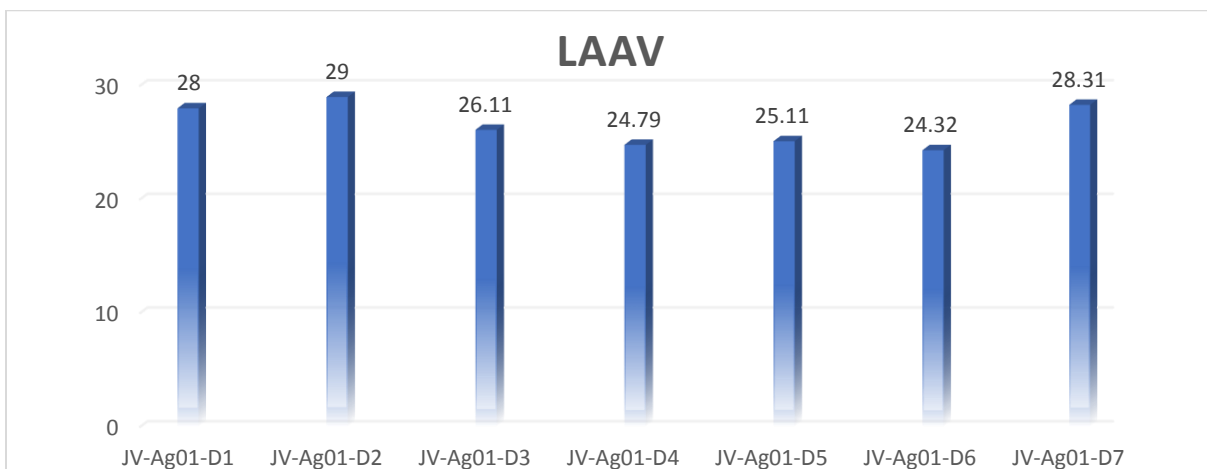


Fig 5.10: LAHV of quarry rock materials

5.1.2.3.3. Determination of Aggregate Impact value and Aggregate Crushing value

The aggregate crushing value (ACV) for the tested samples ranges from 16%-20% (Table 5.3). According to many researches the ACV less than 30% can be used at the surface for pavement construction in this particular case the crushed rock aggregates are suitable in pavement construction.

The aggregate impact value (AIV) for the tested samples range from 10.02-12% (Table 5.3) and this indicates the crushed rock aggregates are under the range of strong aggregate and meets the criteria for use in road construction and concreting operation.

Table 5.3: LAAV, ACV and AIV of tested samples

Sample No.	Los Angeles Abrasion Value (LAAV)	Aggregate Crushing Value (ACV)	Aggregate Impact Value (AIV)
JV-Ag01-D1	28%	18.83%	10.96%
JV-Ag01-D2	29%	20.33%	12%
JV-Ag01-D3	26.11%	16.23%	10.67%
JV-Ag01-D4	24.79%	18.19%	10.84%
JV-Ag01-D5	25.11%	18.57%	10.02%
JV-Ag01-D6	24.32%	19.21%	10.94%
JV-Ag01-D7	28.31%	19.46%	11.12%

5.1.3. Correlation of Physical and Mechanical Tests

It is important to correlate physical parameters obtained from various engineering tests with that of mechanical parameters especially in circumstances where it is impossible to easily access laboratories and the cost for testing is unaffordable.

Physical properties such as water absorption and specific gravity and; mechanical properties such as AIV, ACV and LAAV are correlated. The correlation is based on Linear, logarithmic, exponential and power curve fitting approximations while, the best approximation equation with highest correlation coefficient was determined for each regression. The correlation could also be strengthened with future studies.

5.1.3.1. Water absorption and Specific gravity

The ASTM: C29 and 127, AASHTO: T-85, BS: 812 standards set a limit for an acceptable value of water absorption (0-2%) and specific gravity (2.6-3.0%) of an aggregate (Fig.5.11).

The correlation coefficient value for WA and SG is 0.90 and this indicates that a strong correlation can be obtained between the parameters.

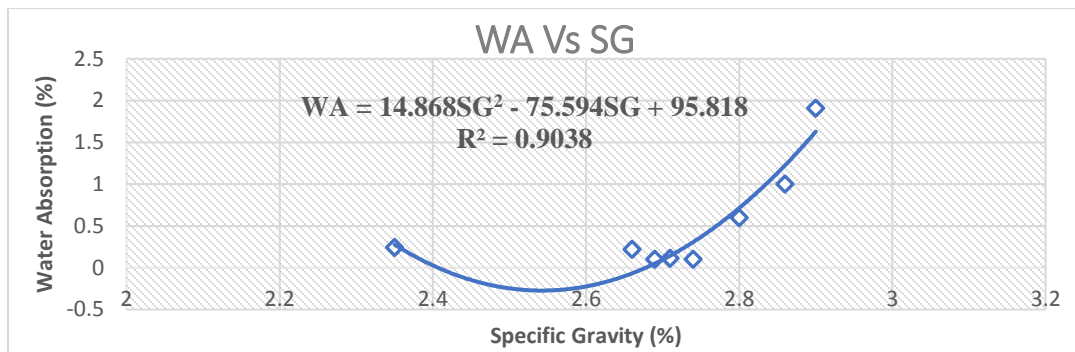


Fig 5.11: Correlation between water absorption and specific gravity

5.1.3.2. Aggregate Impact value and Aggregate Crushing value

The BS 812: Part 112 and BS 812: Part 110 set a standard limit for aggregates used in construction. The ACV and AIV values obtained all fall within the range specified by BS that is 5%-25% (Fig.5.12). The correlation coefficient value for AIV and ACV is 0.72 which is comparatively weak but if the sampling increases a strong correlation could possibly be obtained between the parameters.

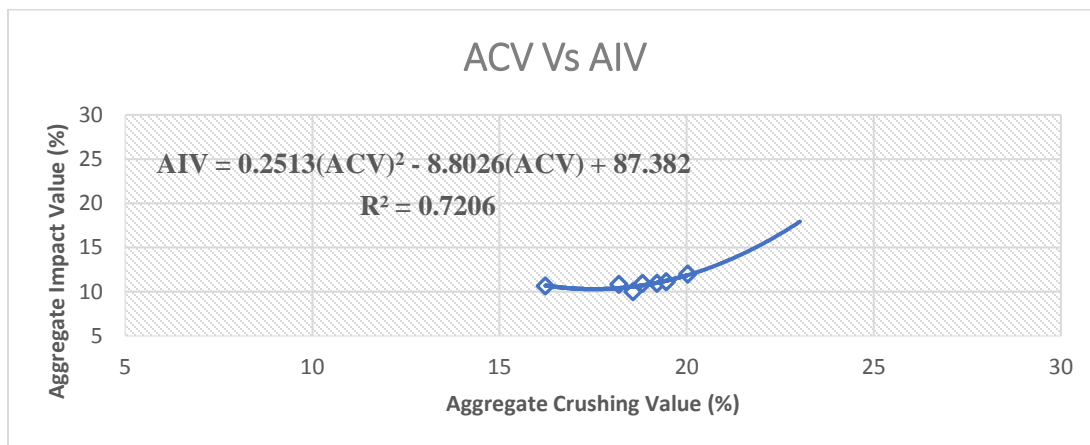


Fig 5.12: Correlation between ACV and AIV

5.1.3.3. Aggregate Crushing Value and Los Angeles Abrasion Value

The correlation among Aggregate crushing value and Los Angeles Abrasion value as referred by different researchers (Engidasew, 2014) is good as far as the values obtained after sampling fall in between the specified range as indicated by BS 812:1990. As for the case of this research the values obtained are within the acceptable limit with $R^2=0.8293$ (Fig.5.13).

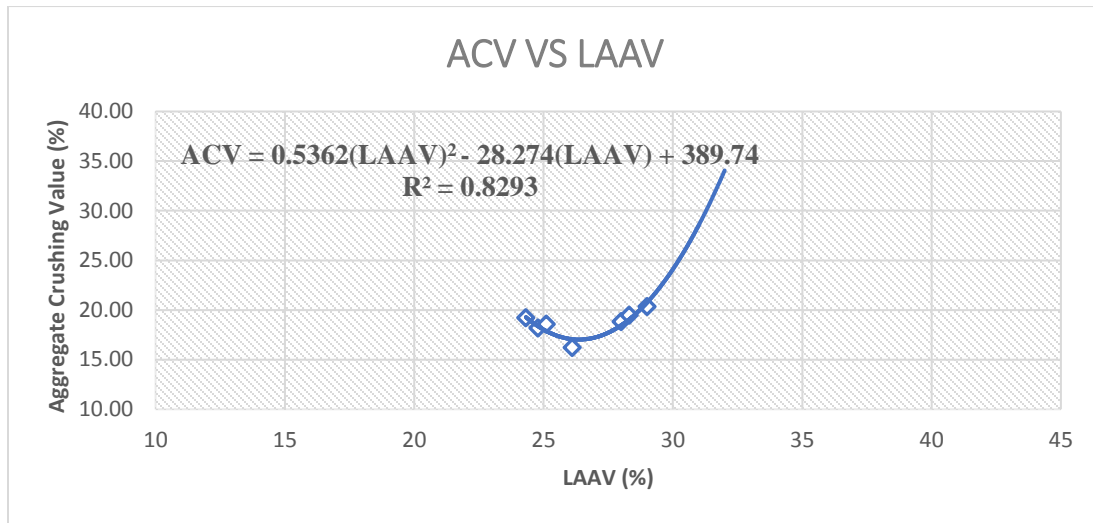


Fig 5.13: Correlation between LAAV and ACV

5.1.3.4. Aggregate Impact Value and Los Angeles Abrasion Value

The same is true for the case of AIV and LAAV relation. As specified by BS the correlation among these two important mechanical parameters is good as long as the values obtained lie in the range specified. The range for which all Los Angeles abrasion values will lie for Aggregate Crushing Value of less than 25% is within the acceptable limit that is in between 10% to 35% (Fig.5.14).

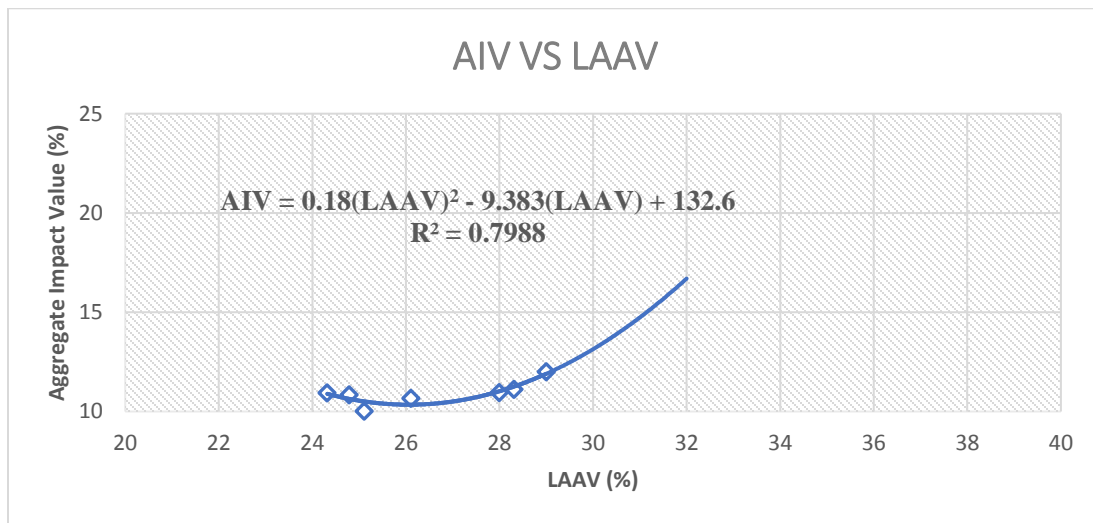


Fig 5.14: Correlation between LAAV and AIV

5.1.4. Simple Relation among UCS, P-wave velocity, Bulk Density and Elastic Dynamic Modulus (E_d)

Elastic modulus is a measure of the stiffness of the rock material which in return determines the strength of the rock material. An attempt has been made to correlate the dynamic elastic modulus (E_d) with the Uniaxial Compressive Strength and bulk density of the rock material.

The seismic wave velocity which is the physical measurement of the rock material is used to estimate the elastic dynamic modulus (E_d). An average P-wave velocity obtained from seismic refraction test for rock mass at an average depth has been considered since it is impossible to obtain the exact ultrasonic P-wave velocity of each intact core sample under consideration (Table 5.4).

Table 5.4: Elastic modulus obtained from average Vp and Bulk density

Sample-ID	Depth (m)	UCS (MPa)	Average Vp (m/s)	Bulk Density (g/cm ³)	Elastic Dynamic Modulus (GPa)
BH-01 RCS 1	26.40-26.68	97.21	1720	2.84	7
BH-01 RCS 4	87.65-87.92	118.72	5390	2.82	77
BH-02 RCS 1	33.00-33.40	175.87	2300	2.83	14
BH-02 RCS 3	65.10-65.40	237.29	4300	2.86	50
BH-03 RCS 1	16.50-16.75	147.19	1110	2.87	3
BH-03 RCS 3	34.00-34.40	150.07	2330	2.79	14
BH-03 RCS 4	58.60-59.00	102.28	3560	2.8	33
BH-04 RCS 6	52.20-52.80	76.87	3630	2.8	34
BH-04 RCS 9	86.74-87.00	36.16	5630	2.7	81
BH-04 RCS 10	96.00-96.30	76.49	6000	2.86	97
BH-05 RCS 1	15.00-15.25	54.38	970	2.93	2
BH-05 RCS 3	26.36-26.69	121.63	1720	2.91	8
BH-05 RCS 6	62.70-63.00	67.54	4170	2.71	44
BH-05 RCS 9	80.45-87.68	100.69	5390	2.92	80

The correlation coefficient of E_d and ρ (Fig.5.15a) and E_d and UCS (5.15b) is 0.89 and 0.93 respectively. This indicates that a strong correlation can be obtained among the parameters.

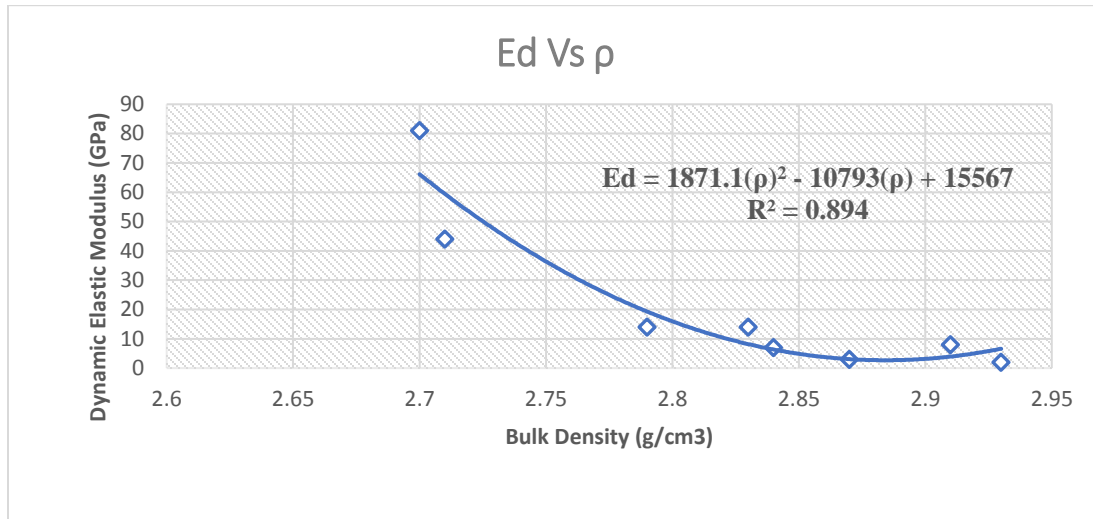


Fig 5.15a: Correlation between Bulk density and E_d

The Elastic Dynamic Modulus obtained from earlier defined equation might deviate from the result that could be obtained from actual V_p of the rock material this is due the variation in the compressional wave velocity (V_p) which is obtained at an average depth from seismic refraction survey.

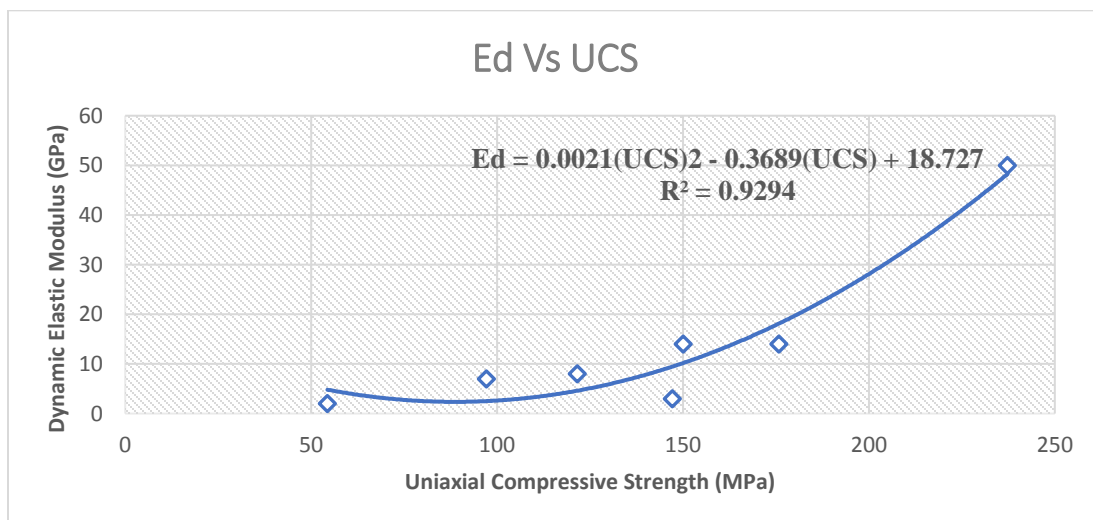


Fig 5.15b: Correlation between UCS and E_d

Now a days V_p can be obtained exactly from intact core samples due the introduction of digitalized recording equipment's. The E_d obtained from the above correlation could be used neither for further relation nor to signify the stiffness of the rock mass within the study area.

The characterization technique employed for rock materials both in the intact and aggregate form basically focused on determining the major physical and mechanical parameters that are important at early stages of preliminary geotechnical investigation process. The parameters are the representative values of basaltic rocks of varying texture and tuffs. Characterization at early stages allows to identify the index properties of rocks and potential quarry sites that will serve the proposed project in the near future as well as the local construction industry.

5.2. Rock mass Characterization

5.2.1. Introduction

This section of the present study in detail discusses rock mass characterization mechanisms for the purpose of evaluating the rock mass along the project site, which will be used as a basis to develop a guide for the geo-mechanical characterization of the Jimma Volcanic and Makkonnen basaltic formations, in terms of rock quality, deformation characteristics, stability analysis, bearing capacity measurement and permeability conditions by integrally applying geotechnical and geophysical investigation techniques and testing approaches.

5.2.2. Rock Mass Classification

5.2.2.1. Rock mass classification from surface outcropped rocks

The geomechanics rock mass classification system in general requires data obtained at different surface exposures in relation to drilled borehole data. This section discusses RMR obtained from different surface exposures and road cut slopes within the tunnel alignment and dam axis sections.

5.2.2.1.1. Intact Rock Strength

The UCS for surface exposed rocks are determined indirectly using Schmidt rebound value under the empirical relation proposed by Barton and Choubey, 1977 (Eqn.4.6) and the value ranges from 22-160MPa. The variation in strength of the intact rock is due to rock weathering, condition of the discontinuities and composition of the rock units.

5.2.2.1.2. Rock Quality Designation (RQD)

The RQD for the surface exposed rock mass is determined by using Palmstrom's Volumetric Count method (Palmstrom, 1982) (Eqn.2.2). Accordingly, the RQD obtained from both the dam and tunnel section ranges from 35.8-95.2%. Based on the classification given by Deer (1964), the rock mass in general falls in fair (50-75%) to excellent (90-100%) rock class.

5.2.2.1.3. Spacing and Conditions of Discontinuities

The condition and spacing of the discontinuities for the exposed rock mass within the vicinity of the dam and tunnel section are obtained according to the manual and specification proposed by ISRM (1981). The discontinuity spacing ranges from 60mm-2.0m while discontinuity aperture has a value ranging less than 10mm.

5.2.2.1.4. Orientation of Discontinuities

The orientation of the discontinuities is measured at and near the tunnel and dam section. All the orientations of the discontinuities fall under fair orientation as per the Bieniawskis RMR classification system (Bieniawski, 1989).

5.2.2.1.5. Ground Water Condition

Both borehole and site investigation data indicate that some exposures are damp enough due to high rainfall intercepting at shallower depth while most surface exposures are characterized under dry condition. Table 5.5 shows few of the RMR location points, the parameters used, the ratings assigned and the total RMR values for the corresponding rock mass class.

5.2.2.2. Rock Mass Classification from Boreholes

5.2.2.2.1. Rock Quality Designation (RQD)

The rock quality for the proposed project area (dam axis and tunnel section) ranges from poor (25%-50%) to very good (90%-100%), with the quality increasing at depths where more intact and stronger core is retrieved.

As suggested by Deer (1988), a low RQD value should be considered a "red flag" for further action, poor drilling techniques, core breakage upon handling, stress-relief or air staking, thinly

bedded or closely jointed zone, or zone of poor rock conditions with shearing and weathering are the basic reasons behind low RQD value (Fig.5.16).



Fig 5.16: Core sample with poor RQD value

Highly fractured and weathered rock core samples are likely to be retrieved with an increase in depth indicating zone of poor rock conditions with close joints susceptible to shearing and weathering. Fig 5.17a and 5.17b show the RQD values of selected boreholes with respective depth.

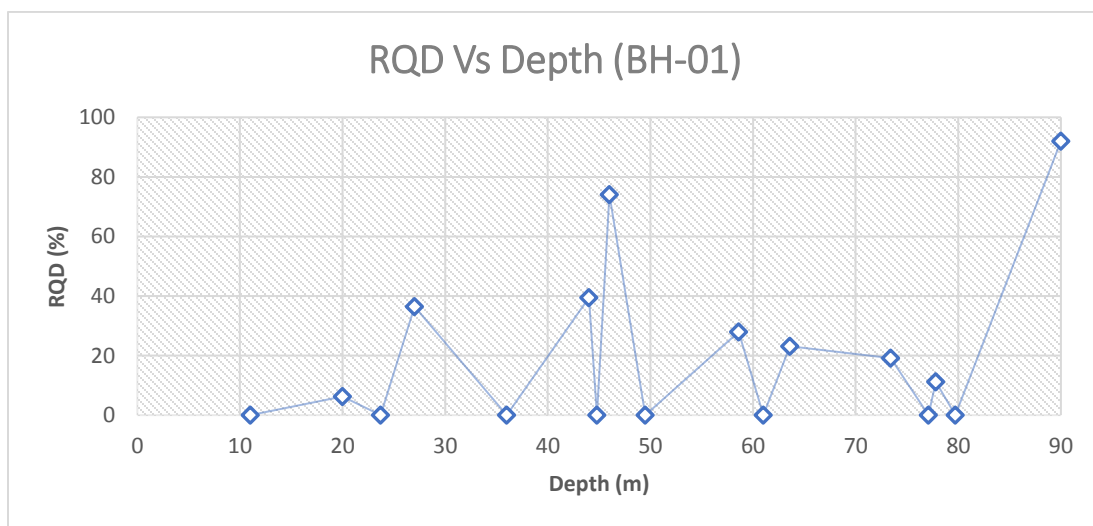


Fig 5.17a: Relation between RQD and Avg. Depth at BH-01

The rock units encountered during field survey are mainly basalts of aphanite texture from highly weathered and fractured to fresh and un-weathered characteristics.

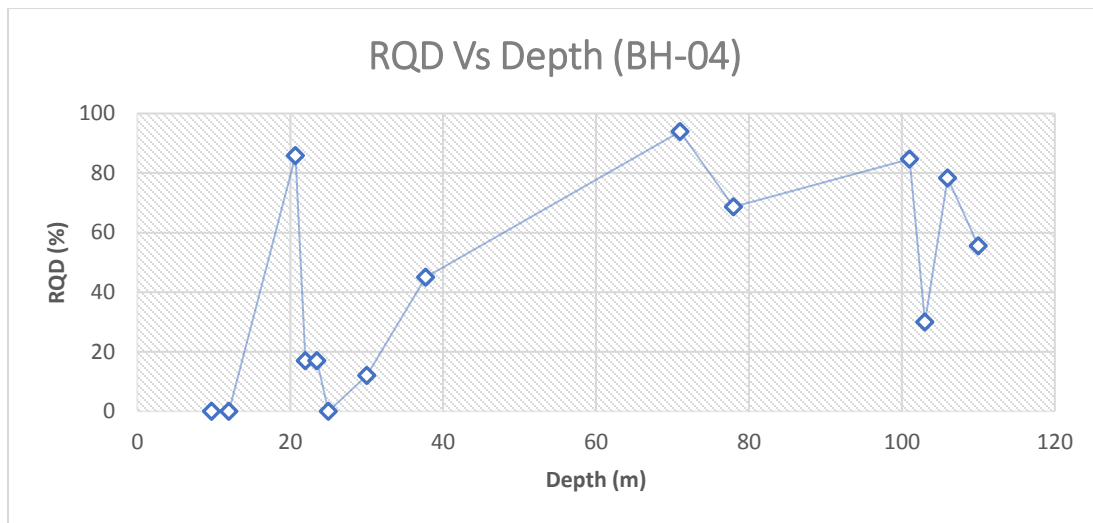


Fig 5.17b: Relation between RQD and Avg. Depth at BH-04

5.2.2.3. Estimation of Rock Mass Quality

5.2.2.3.1. Rock mass Quality estimation using Geological Strength Index (GSI)

Based on the GSI chart (Marinos and Hoek, 2000), the surface RMR points dominantly corresponds to the structural category two and three that is blocky (a well interlocked undisturbed rock mass consisting of cubical blocks formed by orthogonal discontinuity sets) and very blocky (an interlocked partially disturbed rock mass with angular blocks formed by four or more discontinuity sets) (Fig.5.18).



Fig 5.18: Very Blocky and Blocky Rock mass observed from field RMR points

Table 5.5: RMR data from exposed rock mass around dam and tunnel section

RMR Data Points	Parameter Ratings										RMR	Rock Mass Class
	UCS			RQD			Spac.	Cond.	GWC	Ori.		
	SHV	UCS	Ra	J _v	RQD	Ra						
Dam Section												
OC-Spill.	15.03	30.19	4	22	42.4	8	10	15	15	-7	45	Fair
OC-BH04-1	64.5	159.37	12	12	75.4	17	15	19	15	-7	71	Very Good
OC-BH07	15.25	29.06	4	11	78.7	17	8	15	15	-7	52	Fair
OC-BH02	46	113.95	12	22	42.4	8	15	17	15	-7	60	Fair
OC-BH03	31.33	77.93	7	12	75.4	17	15	14	15	-7	61	Good
Tunnel Section												
OC-BH16-1	24	45.26	4	24	35.8	8	10	10	10	-5	37	Poor
OC-BH15	54.14	133.94	12	20	49	8	15	18	15	-5	63	Good
OC-BH12-2	60.25	148.94	12	9	85.3	17	15	20	10	-5	69	Good
OC-BH13	53.75	132.98	12	12	75.4	17	15	18	15	-5	72	Very Good
<p>UCS- Uniaxial Compressive Strength, SHV- Schmidt Hammer Value, JV- Volumetric counting, RQD- Rock Quality Designation, Spac.- Spacing of discontinuity, Cond.- Condition of discontinuity, GWC- Ground Water Condition, Ori- Orientation of discontinuity, Ra- Rating, RMR- Rock mass Rating</p>												

As an input for the GIS chart, the zone is in between 40 to 70 corresponding to a fair to good quality rock mass.

It is also observed that the space between each joint is filled with gouge materials such as calcite fillings, inactive clay minerals and iron oxide. Fig 5.19 shows gouge material infilling some joint sets.

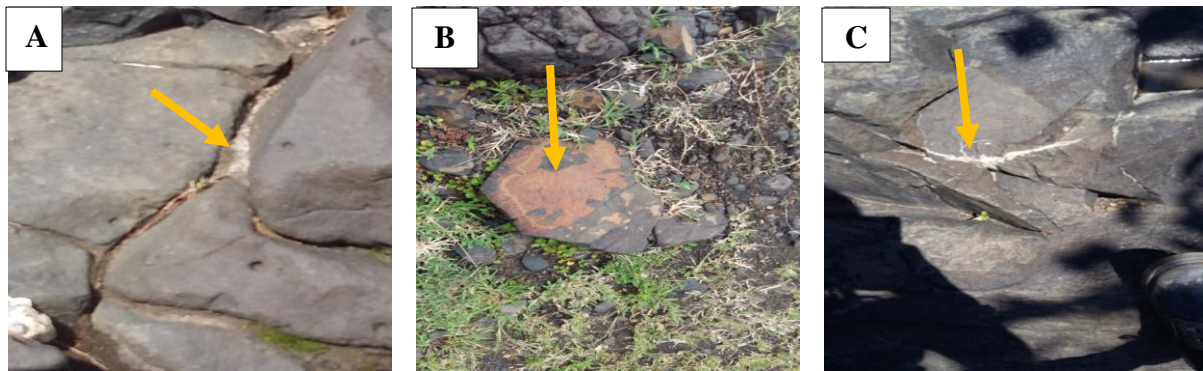


Fig 5.19:Gouge materials infilling joint sets (Inactive clay (A), iron oxide (B) and calcite minerals (C))

5.2.2.3.2. Weak zones

The weak zones within the rock mass are mostly decomposed having low strength and high permeability from rebound hammer and in-situ packer permeability test data, respectively.

Vesicular basalts obtained from the right abutment of the dam site during drilling are good indications for a highly decomposed and low strength weak zone in a rock mass. Fig.5.20 shows a rock core with decomposed weak zones at contact sections.



Fig 5.20: Zone of weakness in a rock core, highly decomposed and fractured at contact zones (BH-07, Dam section)

5.2.3. Estimation of Deformation

The modulus of deformation ‘ E_d ’ of the rock mass is determined by using empirical relations proposed by Serafim and Pereira (1983) for $RMR < 50$ and Bieniawski (1978) for $RMR > 50$ (Table 5.6). The shear strength parameters (c & ϕ) are also determined according to Bieniawski (1989) (ref. Chapter 2).

Table 5.6: Modulus of Deformation ‘ E_d ’ and Shear strength parameters from RMR

RMR Data Points	RMR	Elevation(m)	Modulus of Deformation 'Ed' (Kg/cm ²)	Shear Strength Parameters	
			Serafim and Pereira (1983) and Bieniawski (1978)	c (KPa)	ϕ (°)
OC-Spillway	45	1874	7.49	225	27.5
OC-BH-04	71	1687	42	355	40.5
OC-BH-06-1	49	1691	9.44	245	29.5
OC-BH-07	52	1811	4	260	31
OC-BH-02	60	1768	20	300	35
OC-BH-03	61	1704	22	305	35.5
OC-BH-04	74	1698	48	370	42
OC-BH-06-2	51	1781	2	255	30.5
OC-BH-16-1	37	1627	4.73	185	23.5
OC-BH-16-2	50	1568	10	250	30
OC-BH-15	63	1712	26	315	36.5
OC-BH-12-1	69	1814	38	345	39.5
OC-BH-12-2	69	1808	38	345	39.5
OC-BH-13	72	1741	44	360	41
OC-BH-09-1	57	1808	14	285	33.5
OC-BH-09-2	64	1810	28	320	37

5.2.4. Estimation of shear strength

5.2.4.1. Hoek and Brown failure criteria

Rock parameters useful in determining shear strength characteristics of the rock mass from the generalized Hoek-Brown failure criteria are listed in Table 5.7. The Hoek-Brown classification parameters (input parameters) include the intact rock material constant (m_i), disturbance factor (D) and the geological strength index (GSI).

The parameters are calculated and simulated from the generalized Hoek-Brown failure criterion Hoek et al. (2002) using the spread sheet version of the Roc Lab software developed by Raghuvanshi, (2017).

Table 5.7: Rock mass strength of the Wabe hydropower project Dam and Tunnel Section, from UCS values obtained at selected boreholes (Based on Raghuvanshi, 2017 spread sheet)

BH-ID	Rock Material Properties		Hoek-Brown Classification Parameters			Rock mass Parameters Generalized Hoek-Brown Criterion			
	σ_c (MPa)	γ (MN/m ³)	m_i	D	GSI	E_m (MPa)	m_b	s	a
Dam Section									
BH-03	70.5	0.026	25	0	35	3541	2.453	0.0007	0.516
BH-07	86.3	0.026	25	0	69	27734	8.263	0.032	0.501
Tunnel Section									
BH-09	207.02	0.026	25	0	87	84140	15.715	0.2359	0.500
BH-13	155.59	0.026	25	0	75	42170	10.237	0.0622	0.501

The generalized Hoek-Brown criteria (output parameters) essential in estimating the shear strength characteristics of the rock mass are rock mass modulus (E_m), reduced value of the material constant m_i (m_b) and constants s and a which depend on the characteristics or quality of the rock mass.

A relation has also been developed between the major and the minor principal stresses incorporating both Mohr-coulomb and Hoek-Brown criteria (Fig.5.21a and Fig.5.21b).

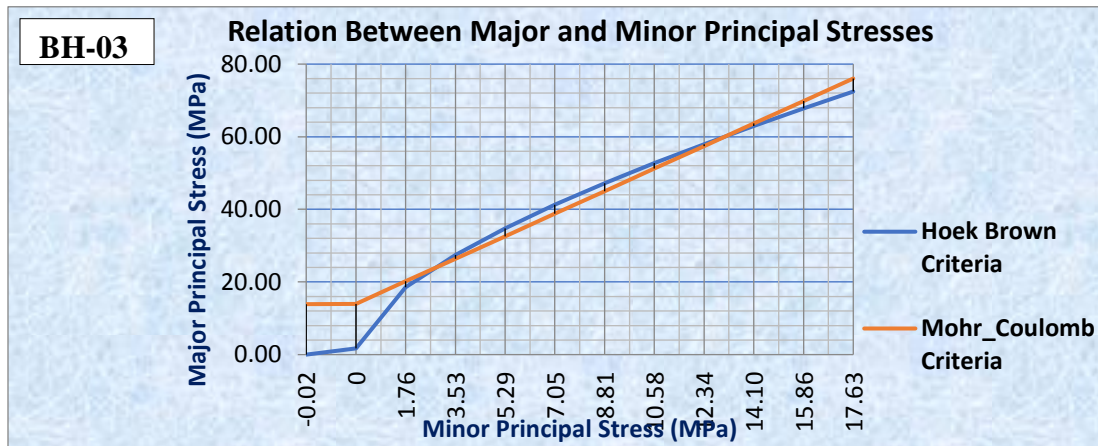


Fig 5.21a: Rock mass strength based on estimated m_i and UCS value for BH-03

The relations indicate that at a very small confining stress that is at elastic deformation stages the strength envelope relation is linear. The Mohr-Coulomb criteria is not a satisfactory peak strength criterion in rock since it overestimates the strength at higher confining stress as illustrated in Fig.5.21a and Fig.5.21b.

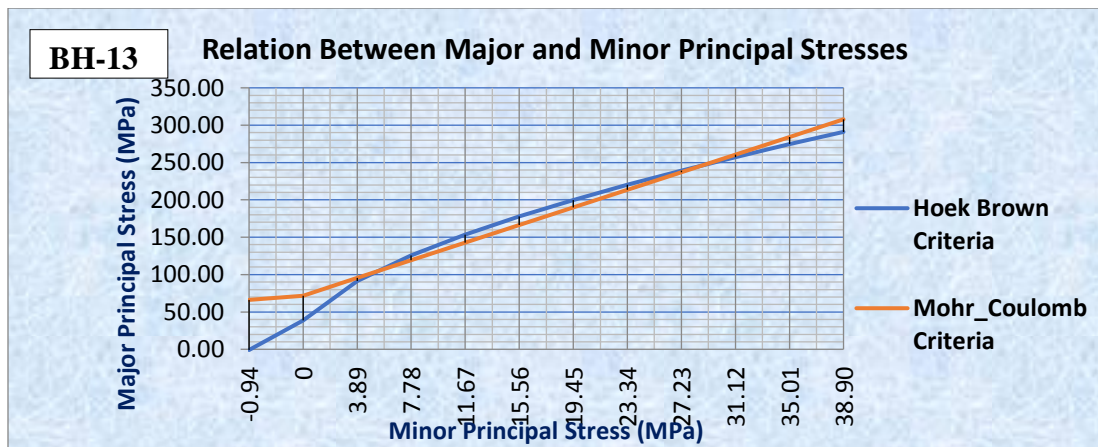


Fig 5.21b: Rock mass strength based on estimated m_i and UCS value for BH-13

The strength envelope observed from most experimental data's in rock are non-linear after reaching the elastic limit (elastic deformation stage), the Mohr-Coulomb criterion deviates such relation by considering a linear relation.

As depth increases confining pressure increases where rocks are less likely to fracture since the pressure of the surroundings tends to hinder the formation of fractures. In general, Hoek-Brown criteria is a theoretical strength-failure criterion suitable to estimate shear strength characteristics of rock mass.

5.2.5. Slope Stability Analysis

5.2.5.1. Discontinuity Analysis

In the present study the influence of joint orientation on the stability of the rock masses prevailing at around the proposed tunnel section where there exists a high head variation has been evaluated.

The first step of the evaluation mechanism is to analyze the structural data of the slope obtained in instability of the slope under consideration using stereographic presentation technique (Kinematic analysis).

At RMR point OC-BH-16-2, at around the proposed power house section, the exposed outcrop length is about 9m. As illustrated in Fig.5.22 the rock underwent fracturing and weathering processes that is likely due to intense tectonic action which produced faulting.

In order to work out the preferred orientations of these discontinuity planes, structural data, mainly, joints have been collected from the road cut slope. The joint sets obtained along the road cut has the following values of strike direction and dip direction respectively: N67E/82W (J₁), N37E/79E (J₂), N76E/82N (J₃) and N89W/41N (J₄) (Fig.5.23).



Fig 5.22: RMR Point OC-BH-16-2, proposed power house section

5.2.5.2. Kinematic Check

Structural data, along with slope inclination and a 'phi circle' corresponding to angle of friction of the rock mass has been plotted on equal area projection 'Schmidt Net'. The angle of friction has been estimated from the RMR data.

According to Duncan and Christopher (2005), the failure of a rock slope will only occur if the following conditions are satisfied (Eq.5.1 and Eq.5.2).

$$\text{Wedge failure} \rightarrow \alpha_f > \alpha_i > \varphi \quad (5.1)$$

$$\text{Plane failure} \rightarrow \alpha_f > \alpha_p > \varphi \quad (5.2)$$

Where; α_f is the slope angle, α_p is the dip of the potential failure plane, α_i is plunge of the line of intersection and φ is the angle of internal friction of the two wedges forming plane.

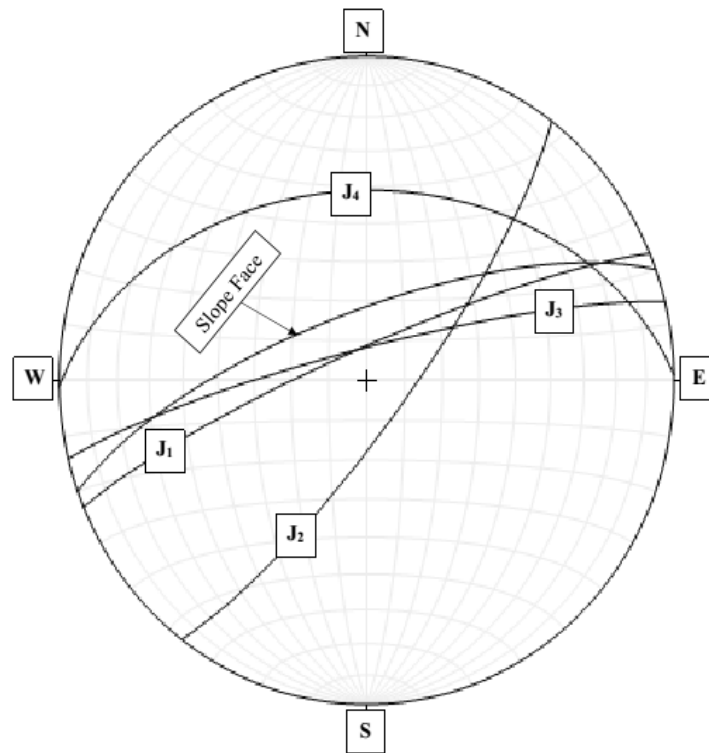


Fig 5.23: Preferred orientations of discontinuity planes as observed on the power house section (OC-BH-16-2)

It is found that the road cut slope satisfies the kinematics condition for plane mode of failure defined by joints J₁, J₃ and J₄ while J₂ satisfies wedge mode of failure (Fig.5.24).

5.2.5.3. Slope stability analysis using slope mass rating (SMR)

Initially SMR was developed to account for only plane and toppling mode of failure. Later, it was modified by Anbalagan et al. (1992) to account for wedge mode of failure.

$$SMR = RMR_{basic} + (F_1 * F_2 * F_3) + F_4 \quad (5.3)$$

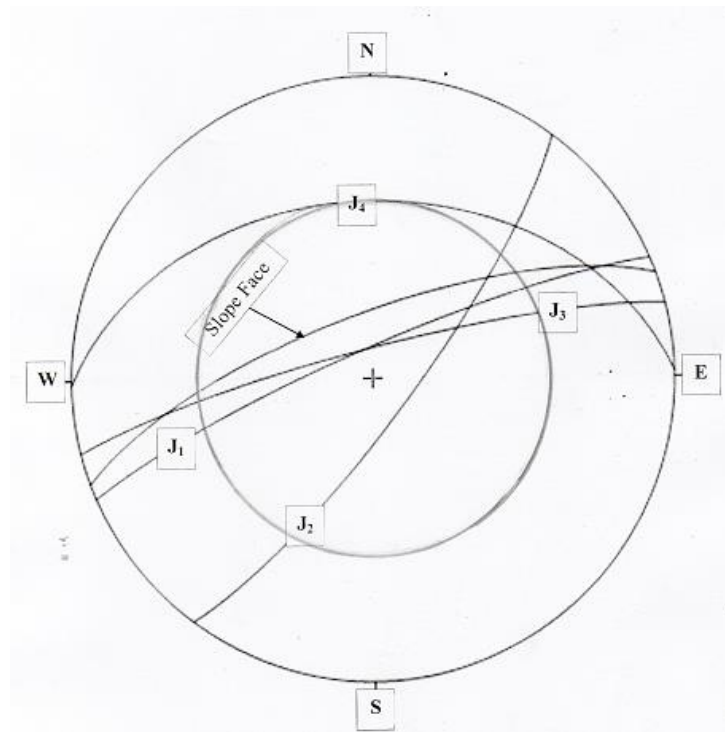


Fig 5.24: Kinematic check for potential mode of failure around the power house section

Where, RMR_{basic} is evaluated according to Bieniawski (1979, 1989) by adding the ratings of five parameters. F₁, F₂, and F₃ are adjustment factors related to joint orientation with respect to slope orientation, and F₄ is the correction factor for method of excavation (refer Chapter 2).

According to the classification of Romana (1985), the road cut slope at around the power house section falls into Class I, the rock mass description is very good; the stability condition is completely stable with no failure blocks. Table 5.8 shows the parameter rating and the calculated SMR for the road cut slope around power house section for joint set four (N89W/41N).

Table 5.8: Values and ratings given for the parameters for the power house section slope

Cause of slope failure	Rating
P ($\alpha_j - \alpha_s$)	19
T ($\alpha_j - \alpha_s - 180$)	
W ($\alpha_i - \alpha_s$)	
P/T/W	
F1 Rating	0.7
P(β_j)	41
W (β_i)	
P/W	
F2 Rating	0.85
P ($\beta_j - \beta_s$)	-31.5
W ($\beta_i - \beta_s$)	
T ($\beta_j + \beta_s$)	
P/W/T	
F3 Rating	-60
F4 Rating	0
RMR basic	50
Total SMR	85.7

The Kinematic analysis for the road cut slope has indicated the possible mode of failure based on the orientation of joint sets however, the SMR is classified within class I, which is described as very good, completely stable with no failure blocks.

The analysis is based on limited data and it is recommended to make a detailed slope stability analysis during excavation and construction period, where there will be rock exposure and detail structural data since it is likely for artificial (manmade) and natural actions to trigger the above-mentioned rock instabilities and failure modes.

5.2.6. Permeability Assessment in Foundation Rock mass

Unlike soils, where seepage takes place through a series of small, closely spaced, interconnected pore spaces, seepage through rock masses occurs mostly along discrete planar discontinuities. In rock masses the conductivity depends on the aperture, spacing and infilling characteristics of its discontinuities (Goodman, 1980).

The permeability test results for the foundation rock mass (BH-04 and BH-07) and tunnel alignment (BH-13) at selected depths is summarized in Table 5.9 with the respective degree of permeability and rock mass condition. Under ideal conditions (i.e., homogeneous and isotropic) one Lugeon is equivalent to 1.3×10^{-5} cm/sec (Fell et al., 2005). The condition of the rock mass is assigned based on the typical behavior observed in Lugeon values (flow type).

The Lugeon values obtained at the specific depths within each bore hole can also indicate the permeability zones. The dam foundation rock mass fall from very low ($Lu < 1$) to moderate ($5 < Lu < 15$) permeability zones, especially rocks around the dam axis (BH-04) fall under variety of permeability zones (Fig.5.25).

Rocks found around the right of the dam foundation (BH-07) are categorized under low permeability zones except at specific depths were the lugeon values are 21.00 (35.00-40.00 m) and 22.8 (80.00-89.50 m), which are categorized under medium permeability zones where there exist some openings that might have occurred due weak zones.

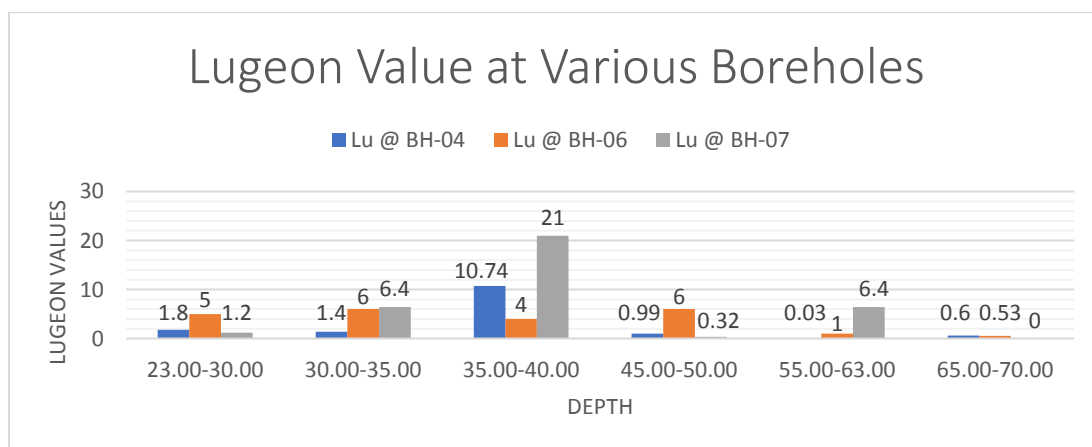


Fig 5.25: Lugeon value at various boreholes obtained within the same Depth

Table 5.9: In situ permeability conditions at selected boreholes

BH-ID	Test Section (m)	In situ Permeability Condition Fell et al., (2005)			In situ Permeability Condition Bell (2007)	
		Lu Value	Flow Type	Condition of Rock mass	Coefficient of Permeability k (m/sec)	Degree of Permeability
BH-04	28.00-33.00	1.4	Dilation	Tight	1.82×10^{-7} m/s	Slightly Permeable
	36.00-40.00	10.74	Dilation	Few Partly Open	13.96×10^{-7} m/s	Slightly Permeable
	40.00-45.00	0.74	Laminar	Very Tight	0.962×10^{-7} m/s	Slightly Permeable
	45.00-50.00	0.99	Dilation	Very Tight	1.29×10^{-7} m/s	Slightly Permeable
	55.00-60.00	0.03	Laminar	Very Tight	0.39×10^{-6} m/s	Slightly Permeable
BH-07	35.00-40.00	21	Void Filling	Some Open	27.3×10^{-7} m/s	Slightly Permeable
	40.00-45.00	3.4	Wash-Out	Tight	4.42×10^{-7} m/s	Slightly Permeable
	45.00-50.00	0.32	Laminar	Very Tight	0.416×10^{-7} m/s	Slightly Permeable
	50.00-55.00	0.4	Turbulent	Very Tight	0.52×10^{-7} m/s	Slightly Permeable
	80.00-89.50	22.8	Void Filling	Some Open	29.64×10^{-7} m/s	Slightly Permeable
BH-13	135.00-140.00	1	Dilation	Tight	1.30×10^{-7} m/sec	Slightly Permeable
	140.00-145.00	1	Laminar	Tight	1.30×10^{-7} m/sec	Slightly Permeable

The lugeon value in relation to the depths obtained are also illustrated in Fig.5.26 and 5.27 below. For instance, at borehole five (BH-05) at initial depth the packer test indicates a relatively high lugeon value and this shows again relatively a high-water intake of the rock units but as test continues, with an increase in depth, lugeon value decreases.

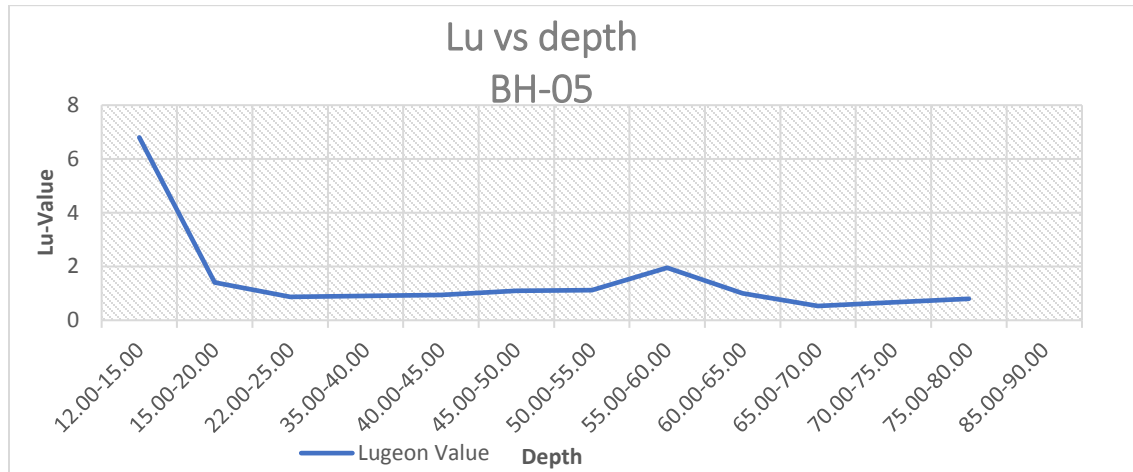


Fig 5.26: Lugeon Value at BH-05

In case of borehole seven (BH-07), the initial water intake capacity of the rock units at lower depths is relatively low while there exists a depth where the water intake capacity is high due to highly fractured and weak zones.

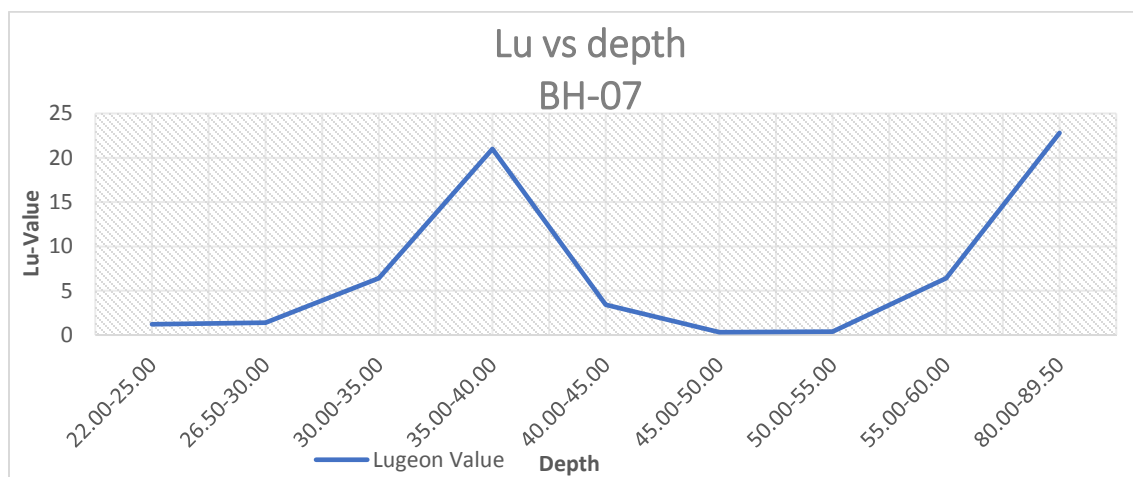


Fig 5.27: Lugeon Value at BH-07

Even though there exists no complete water loss from surface to certain depths, a relatively high lugeon values were observed at the right of the abutment and the dam axis. This might not bring a serious of seepage problem, yet the joint systems and the moderately weathered rocks within this specific depth require a treatment so as to avoid time-based seepage problems.

5.2.7. Evaluation of Dam site Rock mass Bearing Capacity

According to Bowels, (1996) the safe load bearing capacity of rock mass based on UCS and RQD has been given empirically (ref.Eq.4.6)

Where: UCS is the uniaxial compressive strength of the rock mass, RQD is the rock quality designation (from drilled core sample and joint volumetric count) and FS being the factor of safety.

The result obtained for the safe load bearing capacity of the foundation rock within the dam section (Table 5.10) in general ranges from 6 to 425 KPa.

Table 5.10: Safe Load Bearing Capacity of Wabe hydropower Project Dam Foundation

Source	Source ID	Location	UCS (MPa)	RQD	FS	Safe Load Bearing Capacity (KPa)
Borehole Points	BH-4.1	Along Dam Axis	50.7	85.8	3	124.411
	BH-4.2		101.1	93.9	3	297.139
	BH-4.3		36.8	78.3	3	75.206
	BH-4.4		83	55.5	3	85.220
Surface Points	OC-BH-02	Dam Section	113.95	42.4	3	68.285
	OC-BH-03		77.93	75.4	3	147.682
	OC-BH-06.2		97.74	45.7	3	68.043
	OC-BH-07		29.06	78.7	3	59.996
	OC-Spillway		30.194	42.4	3	18.094
	OC-BH-04-1		159.37	75.4	3	302.015
	OC-BH-04-2		140.96	95.2	3	425.842
	OC-BH-06.1		119.642	12.7	3	6.432

On average large values are obtained from borehole log data indicating load bearing capacity of rock mass increases as depth increases where more intact and sounder rock exists.

For most structural foundations, the minimum acceptable factor of safety is 3 with a structural load comprises of the full dead load plus live load. A factor of safety of 3 has been used for this specific research as well.

5.2.8. Rock mass characterization using seismic refraction test results

Based on the investigation the formation has been categorized into four seismic units as per the intensity of the compressional wave velocity (P-wave velocity) of the rock units. The seismic units with their corresponding p-wave velocity is summarized in Table 5.11 below after the appropriate interpretation has been made.

5.2.9. Overall interpretation of seismic refraction data in relation to geotechnical methods

The results from seismic refraction test, together with borehole log data has shown a relation that can be interpreted in terms of the fracture density and penetration strength of P-wave velocity.

According to Geomatrix (2016), the relation among the fracturing density of the rock mass and the penetrating velocity of P-wave is in general indirect.

Table 5.11: Generalized P-wave velocity of the various rock units of the study area

No.	Rock-Type	P-Wave Velocity (m/sec)
1	Residual material, brown shed grey, dense, sandy silt with some gravel material, variegated color, highly to completely weathered and disintegrated to sandy gravel size	<1500
2	Intensively to moderately weathered and fractured, basalt and tuff with medium strength	1000-3000
3	Moderately weathered and fractured basalt, composed of slightly weathered and fractured, with strong and dense basalts	2500-3500
4	Slightly weathered and slightly fractured and fragmented basalt composed of fresh basalt with greyish in color	>3500

It is also anticipated that there may be geological structures at certain stations of most survey lines along the dam section and tunnel alignment (Fig.5.28).

Most of the drilling penetrated up to 110m depth and shows predominantly basalt with various degree of fracturing and weathering and also a pyroclastic formation is found with few meters thickness. Soil and/or completely weathered basalt rock have been identified in the first 10m depth.

As illustrated in Fig.5.29 BH-04 and BH-07 are within the survey line WSD4-2. A highly weathered and fractured rock unit inferred by the upper most seismic layer has a very low P-wave penetration strength reaching as small as 300m/s while the log data has also shown the same result that is the layer is mainly recognized by a highly to completely weathered and decomposed rock with, gravel (overburden).

Unusual zones also referred as anomaly zones are identified by their appearance either in layers where the intactness of the seismic layer is much less than the small layer (anomaly zone) that has appeared in between or it could be identified by its appearance in layers where the intactness of seismic layer in which it exists is much greater than it (anomaly zone). This in general can be referred as high and low anomaly zones respectively.

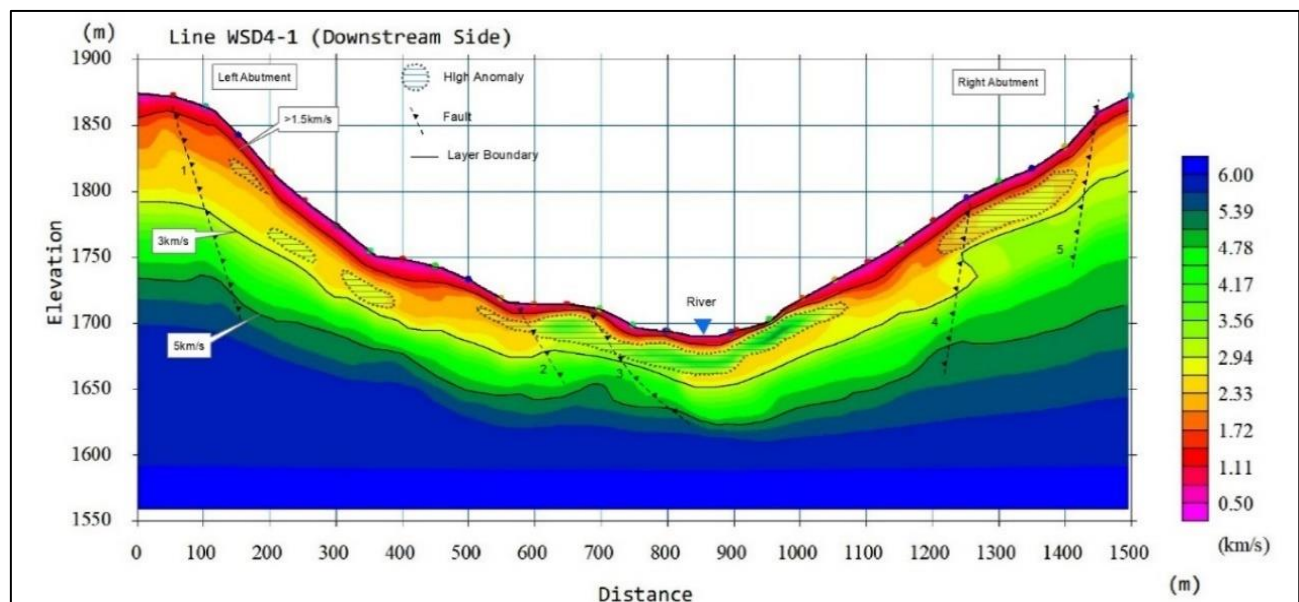


Fig 5.28: Seismic Tomographic section with inferred faults along the dam section (Survey line WSD4-1) (Source: Wabe Seismic refraction test, ECDSWC, 2019)

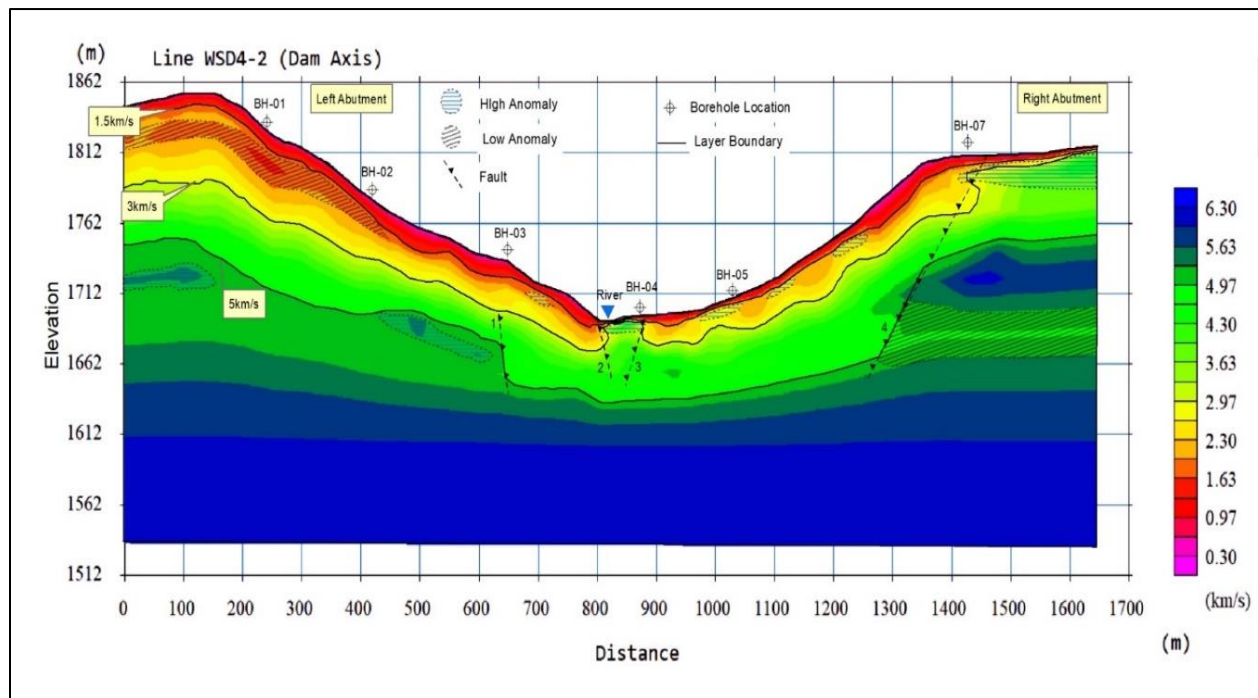


Fig 5.29: Seismic velocity section along the dam axis (Survey line WSD4-2) (Source: Wabe Seismic refraction test, ECDSWC, 2019)

5.2.10. Correlation between Seismic velocity and Rock mass rating (RMR)

The correlation between RMR values obtained from drilled borehole log and compressional wave velocity obtained from seismic refraction test allows understanding the penetration strength of P-wave velocity in relation to the quality of rock mass.

From the borehole log, the average RMR data can be obtained easily, RMR has been chosen instead of RQD to determine the quality of rock mass since RMR is a combination of parameters which determines the overall rock mass quality. Previous works have shown a correlation between RQD and seismic wave velocity (Gebremichael, 2017).

Table 5.12 shows the calculated RMR values with the corresponding seismic velocity at target depths, and Fig.5.30a and 5.30b show the correlation between RMR and seismic velocity at BH-02 and BH-04.

Table 5.12: Avg. Vp and Avg. RMR at target depths

Depth (m)	BH-02		BH-04	
	Avg. V _P (m/s)	Avg. RMR (%)	Avg. V _P (m/s)	Avg. RMR (%)
10	300	0	300	0
20	970	0	970	36
30	1630	63	1630	6.75
40	2300	63	2300	35
50	2970	68	2970	85
60	3630	83	3630	85
70	4300	83	4300	85
80	4970	-	4970	63
90	5630	-	5630	67
100	6300	-	6300	50.67

From the figures, there exists a linear relation between the two parameters that is for unfractured and sound rock mass the penetration strength of P-wave velocity is high. The fracture density of rocks in general has an influence on the penetration strength of compressional wave velocity (V_p).

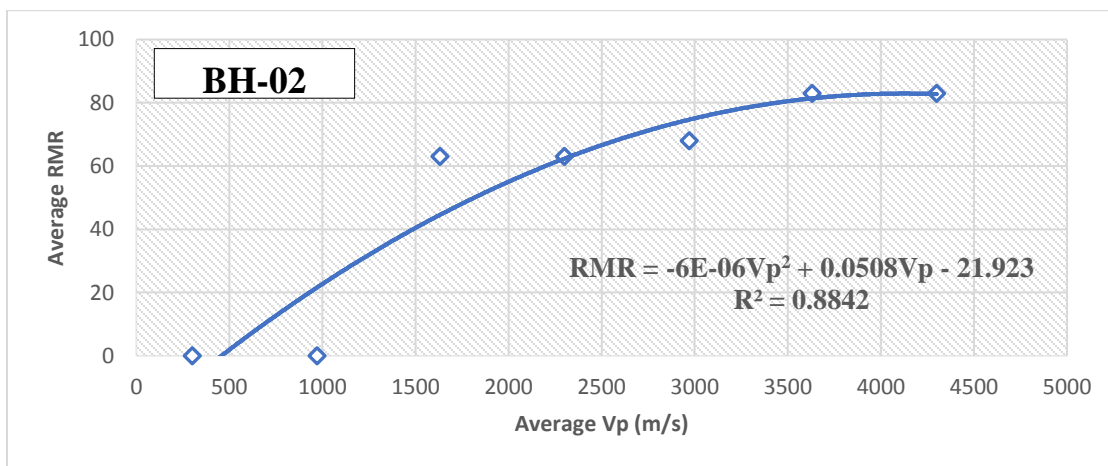


Fig 5.30a: RMR and Seismic velocity relation at BH-02

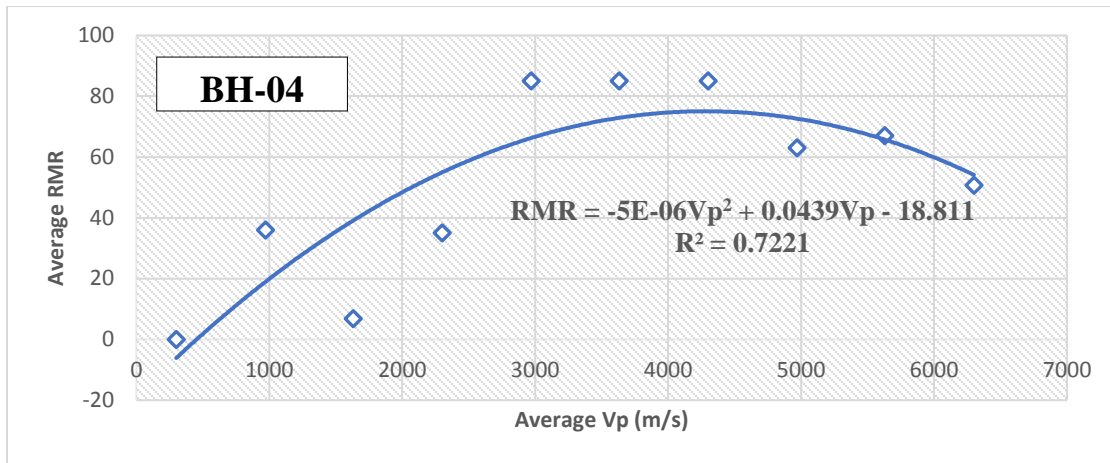


Fig 5.30b: RMR and Seismic velocity relation at BH-04

For this specific research, Table 5.13 below gives the inferred correlation between P-wave velocity and RMR value for depths up to 100 meters.

Table 5.13: Overall correlation between P-wave velocity and RMR value

Seismic Layer	Description rock	P-wave Velocity (m/s)	RMR Value
1	Soil (over Burden) Alluvial, Highly or Completely Weathered Basalt	<1500	<35
2	Moderately Weathered and Fractured Basalt, Pyroclastic Rock	1500-3000	35-45
3	Slightly Fractured Basalt	3000-5000	45-65
4	Fresh Basalt with Minor Fractures	>5000	>65

5.2.11. Geotechnical Domains and Rock mass Classification

In order to apply the geomechanics classification system, the rock mass along the tunnel section is been divided into a number of geological structural units in such a way that each type of rock mass is represented by a separate structural unit. Seismic refraction survey together with drilled bore hole data enabled to identify the continuity and uniformity of the geological features.

To determine the RMR of the waterway, the tunnel section is divided into geotechnical regions/classes based on lithology, structure, and strength properties. The different geotechnical regions/classes are described below.

Geotechnical Class 1, GC1

This class has been categorized according to RMR parameters obtained from seismic refraction survey, geologic and geotechnical data of BH-09. This unit is found in Chainages 0+00 to 0+551m and 0+581m to 0+1085m. The RMR parameters and corresponding values are summarized in Table 5.14 below.

Table 5.14: RMR for the geotechnical class around the tunnel route 1, GC-1

Description	UCS (MPa)	RQD	Spacing of Discontinuities (mm)	Condition of Discontinuities	GWC	Adjustment for joint Orientation
Value	207 (47.1-47.4m)	91	600-2000	Un-weathered surface	Dry	Tunnel
Rate (1989)	12	20	15	30	15	-5
RMR	85					
Class	I (Very good rock)					
Average Stand up time in tunnel	20 Years for 15m span					
Cohesion of the rock mass	>400 KPa					
Friction angle of the rock mass	>45 Degree					

Geotechnical Class 2, GC-2

Four chainages from 0+1085 to 0+2715, 0+2745 to 0+5265, 0+5290 to 0+6435, 0+6646m to 0+7171m, based on interpretations from seismic refraction geophysical survey results, geological, and geotechnical bore hole data from BH-09 and BH-13 have shown similar geotechnical properties along the tunnel route. The RMR parameters and corresponding values are summarized in Table 5.15 below.

Table 5.15: RMR for the geotechnical class around the tunnel route 2, GC-2

Description	UCS (MPa)	RQD	Spacing of Discontinuities (mm)	Condition of Discontinuities	GWC	Adjustment for joint Orientation
Value	155.6 (141.15-141.34m)	89	600-2000	Un-weathered surface	Wet	Tunnel
Rate (1989)	12	17	15	25	7	-5
RMR	71					
Class	II (Good rock)					
Average Stand up time in tunnel	1 Year for 10m span					
Cohesion of the rock mass	300-400 KPa					
Friction angle of the rock mass	35-45 Degree					

Geotechnical Class 3, GC-3

The same interpretation from geologic, seismic survey and geotechnical borehole data from BH-13 and BH-15) have been made to obtain the RMR parameters for this geotechnical class. This unit is found in chainage from 0+7201 to 0+9795, 0+9830 to 0+12189, and 0+12219 to 0+13932. The RMR parameters and corresponding values are summarized in Table 5.16 below.

Table 5.16: RMR for the geotechnical class around the tunnel route 3, GC-3

Description	UCS (MPa)	RQD	Spacing of Discontinuities (mm)	Condition of Discontinuities	GWC	Adjustment for joint Orientation
Value	200	92	>2000	Un-weathered surface	Wet	Tunnel
Rate (1989)	12	20	20	30	7	-5
RMR	84					
Class	I (Very good rock)					
Average Stand up time in tunnel	20 Years for 15m span					
Cohesion of the rock mass	>400 KPa					
Friction angle of the rock mass	>45 Degree					

Geotechnical Class 4, GC-4

This unit is found in Chainages from CH: 0+14,626 - 0+16,937; CH: 0+16,967- 0+17,555, CH 0+17,585-0+17650. The RMR value for this Geotechnical Class is determined by interpreting data from boreholes (boreholes 9, 15, and 16), seismic and geological interpretations. The RMR parameters and corresponding values are summarized in Table 5.17 below.

Table 5.17: RMR for the geotechnical class around the tunnel route 4, GC-4

Description	UCS (MPa)	RQD	Spacing of Discontinuities (mm)	Condition of Discontinuities	GWC	Adjustment for joint Orientation
Value	156	66	>2000	Slightly weathered	Wet	Tunnel
Rate (1989)	12	13	15	25	7	-5
RMR	66					
Class	II (Good rock)					
Average Stand up time in tunnel	1 Year for 10m span					
Cohesion of the rock mass	300-400 KPa					
Friction angle of the rock mass	35-45 Degree					

Geotechnical Class 5, GC-5

This unit is found in Chainages from 0+00,551 - 0+00,581; CH:0+02,715 - 0+02,744; CH:0+05,265 - 0+05,290; CH:0+07,171 - 0+07,201; CH:0+09,795 - 0+09,825; CH:0+12,189 - 0+12,219; CH:0+13,932 - 0+13,963; CH:0+16,997- 0+17,027 and CH 0+17,555-0+17585.

Geological structures compromising this unit include normal fault zones and one reactivated normal fault. The geotechnical class is obtained by interpreting data from geological maps and sections, geotechnical boreholes (boreholes 9, 13, 15, and 16), and seismic data. The RMR parameters and corresponding values are summarized in Table 5.18 below.

Table 5.18: RMR for the geotechnical class around the tunnel route 5, GC-5

Description	UCS (MPa)	RQD	Spacing of Discontinuities (mm)	Condition of Discontinuities	GWC	Adjustment for joint Orientation
Value	1	<25	<60	Decomposed	Wet	Tunnel
Rate (1989)	0	3	5	30	7	-2
RMR	13					
Class	V (Very poor rock)					
Average Stand up time in tunnel	30 minutes for 1m span					
Cohesion of the rock mass	<100 KPa					
Friction angle of the rock mass	<15 Degree					

5.2.11.1. Hoek-Brown Criterion

The Generalized Hoek-Brown Criterion parameters as well as the Hoek-Brown classification parameters for each Geotechnical Class (GC1-GC5) are estimated from the generalized Hoek-Brown failure criterion (Hoek et al 2002) using the spread sheet version of the RockLab software (Table 5.19) developed by Raghuvanshi, (2017). Similarly, Fig.5.31 illustrates the relationship between the major and minor principal stress by incorporating the Hoek-Brown and Mohr-coulomb failure criteria.

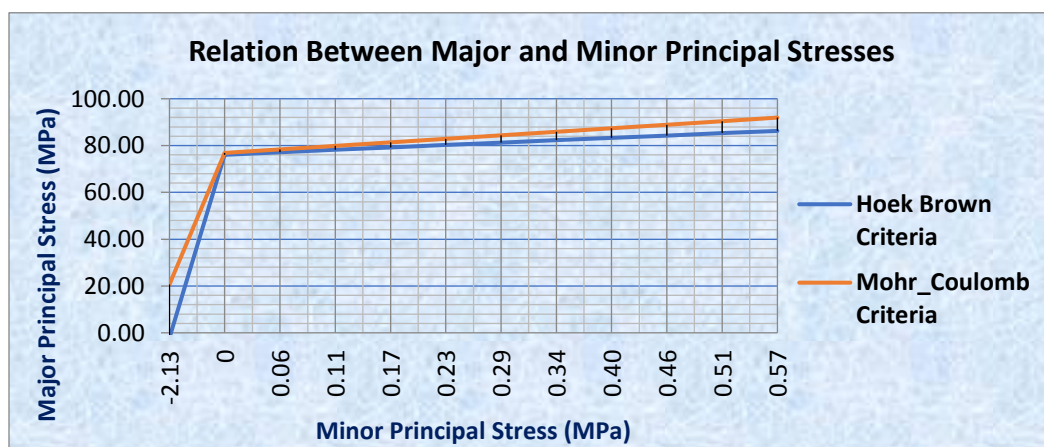


Figure 5.31: Rock mass strength based on estimated m_i and UCS value for BH-03

Table 5.19: Hoek-Brown Classification and Generalized Hoek-Brown Criterion Parameters

Geotechnical Class	Rock Material Properties		Hoek-Brown Classification Parameters			Rock mass Parameters Generalized Hoek-Brown Criterion			
	σ_c (MPa)	γ (MN/m ³)	m_i	D	GSI	E_m (MPa)	m_b	s	a
GC-1	207	0.026	25	0	82	63096	13.145	0.1353	0.500
GC-2	155.6	0.026	23	0	66	25119	6.829	0.0229	0.502
GC-3	200	0.026	25	0	79	53088	11.809	0.0970	0.501
GC-4	156	0.026	23	0	61	18836	5.712	0.0131	0.503
GC-5	1	0.026	13	0	19	167.88	0.7204	0.0001	0.547

5.2.11.2. Rock Tunneling Quality Index, Q- System

The initial goal of the Q-system developed by Barton et al. (1974) is to characterize the rock mass for preliminary empirical design of the support system of tunnels and caverns. The relationship of RMR and Q system is given by Eq.5.4. Table 5.20 summarizes Q-values of the rock mass for the geotechnical class.

$$Q = e^{((RMR-42)/10.5)} \quad (5.4)$$

Table 5.20: Q index Values for the Different Geotechnical Classes

Geotechnical class	RMR	Rock mass Class (Q)	Q-Index	Support Recommendation
GC-1	87	Very Good	72.65	Unsupported
GC-2	71	Good	15.8	Spot bolting, sb
GC-3	84	Very Good	54.59	Unsupported
GC-4	66	Fair	9.83	Systematic bolting, B
GC-5	13	Extremely Poor	0.063	Fiber reinforced shotcrete and bolting, 12-15cm, sfr + B

5.2.11.2.1. Estimation of Support Requirements from Q-system

Based on the Q-index values obtained for each geotechnical class and the equivalent dimension (De), support requirement has been suggested as per reinforcement categories recommended (ref. Chapter 2). The support requirements for each geotechnical class along the proposed tunnel (Wabe Tunnel) is summarized in Table 5.21.

Table 5.21: Estimated Support requirements for the proposed tunnel route based on (Grimstad and Barton 1993).

Geotechnical unit	Chainage (m)	Equivalent Dimension (De)	Q- index	Support Recommendation
GC-1	CH:0+00 to 0+551 and CH:0+581 to 0+1085	6.25	72.6	1) Unsupported
GC-2	CH:0+1085 to 0+2715, CH:0+2745 to 0+5265, CH:0+5290 to 0+6435 and CH:0+6646 to 0+7171		54.6	2) Spot bolting, sb
GC-3	CH:0+7201 to 0+9795, CH:0+9830 to 0+12189 and CH:0+12219 to 0+13932		15.8	1) Unsupported
GC-4	CH:0+14,626 to 0+16,937, CH:0+17,027 to 0+17,555 and CH 0+17,585 to 0+17650		9.8	3) Systematic bolting, B
GC-5	CH:0+00,551 to 0+00,581, CH:0+02,715 to 0+02,744, CH:0+05,265 to 0+05,290, CH:0+07,171 to 0+07,201 CH:0+09,795 to 0+09,825, CH:0+12,189 to 0+12,219, CH:0+13,932 to 0+13,963, CH:0+16,997 to 0+17,027and CH:0+17,555to 0+17585		0.06	7) Fiber reinforced shotcrete and bolting, 12-15cm, sfr + B

5.2.12. Overall assessment of the investigation process and Guideline preparation

The present study in general has focused on rock characterization methodology for the Jimma volcanic and Makkonnen basaltic rock formations.

The rock characterization methodology provides a basis for understanding the characteristic and behavior of rocks, and relate to experiences gained in rock conditions at one site to another. Table 5.22 summarizes the geological units with respect to intact rock and rock mass properties.

In designing any engineering structure on a rock mass, the engineering properties of rock material and discontinuities should be taken into account. Rock grade classification is based on the strength of the cylindrical core sample obtained from the rock mass, total number of joints (blocks), joint spacing and degree of weathering (Table 5.23).

The penetration strength of compressional wave velocity (V_p) depends on the fracture density of the rock mass. The fracture density in return is highly dependent on the overall rock mass quality represented by the geomechanics parameters.

The overall rock mass qualities of the rock units are classified in to four categories from “a” to “d” from good to very poor quality, and four stages of their V_p intensity as shown in Table 5.24a and Table 5.24b.

A general geo-mechanical property of rocks (guideline) for major geological formations will give the general idea of how the geology and the intended civil engineering structure will behave with each other, with this aim a rock mass classification guideline is proposed based on the combination of geotechnical parameters for the mapped geological units (Table 5.25 and Table 5.26).

The guideline will be valuable with proper caution especially in the early stages of site investigation, where there are very limited-to-none geotechnical data. It could also be revised as long as the study is continued, with new parameters being added.

Table 5.22: Intact rock and Rock mass properties of the Geological units

Properties	Intact Rock Properties					Rock mass Properties				
	Parameters									
Geological units	WA (%)	SG (%)	P (%)	ρ (g/cm ³)	UCS (MPa)	RQD (%)	RMR	V _p (m/s)	Em (GPa)	Rock mass Class
Tb1	0.44-1.74	2.72-2.86	1.17-1.56	2.80-2.86	>237.29	94.6-85	>65	>5000	>30	I (Very Good)
Tb2	0.60-1.91	2.80-2.91	<1.97	2.79-2.91	>150.07	89-70	35-45	1500-5000	4.22-30	III(Fair)
Tb3	0.2-2.44	2.80-2.92	1-2.27	2.72-2.92	>118.72	92-75	45-65	3000-5000	7.5-30	II(Good)
Tb4	>2.61	2.53-2.71	>8	2.32-2.60	11.89-27.12	20-45	<35	<1500	1.78-2.37	Poor (IV)
Tb5	0.63-1.31	2.86-2.92	2.27-2.34	2.79-2.92	33.5-103.9	89.9-70	45-65	3000-5000	4.22-30	II(Good)
Tpx	>5.52	2.02-2.06	7.76-13	<1.87	1.37-4.7	<20	<35	<1500	<2.37	Very Poor (V)

Table 5.23: Classification for Rock Grade

Class	Criteria for Judgement	Drilled Core sample and surface outcrop rock mass	Weathering (Alteration)
A	Rock piece cannot be broken easily when struck by hammer, with metallic sound.	RQD is b/n 85-95.2%, with total number of observed joints in b/n 6-9. Joint spacing is in b/n 900mm-2.0m	Fresh (no alteration observed)
B	Metallic resonant sound when struck by Schmidt hammer. A few traces of deteriorated minerals.	RQD is b/n 70-92%, with total number of observed joints in b/n 9-13. Joint spacing is in b/n 800mm-1.0m. Very small number of fragmental pieces of rock is recovered.	Slightly (Alteration of limited portion observed)
C	Rock often becomes broken when struck by hammer. Rock pieces keep almost intact when broken.	RQD is b/n 75-89%, with total number of observed joints in b/n 11-12. Joint spacing is in b/n 300mm-600mm. Some fragmental pieces of rock are recovered.	Moderately (Alteration in color is comparatively small)
D	Broken by hand and slightly penetrated by hammer blow. Joints are not seen clearly.	RQD is b/n 20-45%, with total number of observed joints in b/n 17-24. Joint spacing is in b/n 200mm-500mm.	Highly (Alteration in color relatively large)
E	Broken by hand and finger by applying force (intense squeezing).	RQD is less than 20%, with total number of observed joints >31. Joint spacing <200mm.	Completely (from confirmation of the texture of rock to non-confirmation)

Table 5.24a: Compressional Wave Velocity, V_p (m/s)

Class	Judgment of Criteria
I	>5000m/s
II	3000 to 5000 m/s
III	1500 to 3000 m/s
IV	<1500 m/s

Table 5.24b: RMR (Combination of Geomechanics Classification Parameters

Class	Judgment of Criteria
a	$RMR \geq 65$
b	$45 < RMR < 65$
c	$35 < RMR \leq 45$
d	$RMR \leq 35$

Table 5.25: Classification of Rock mass based on combination of parameters

	A				B				C				D				E			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
a	Tb1	Tb1	Tb3	[Tb3]	Tb3	Tb3	Tb5	[Tb5]												
b	Tb1	Tb3	Tb3	Tb5	Tb5	Tb5	Tb5	[Tb2]	[Tb2]	Tb2	Tb2	Tb2								
c	Tb3	Tb3	Tb5	[Tb5]	Tb5	Tb5	Tb2	[Tb2]	Tb2	Tb2	Tb2	Tb4	Tb4	Tb4	Tb4	Tb4				
d													Tb4	Tb4	Tpx	Tpx	Tpx	Tpx	Tpx	Tpx

Where, [] indicates the rock unit is encountered in a limited conditions

Table 5.26: Rock mass Classification Guideline

Rock mass Classification	Condition of Rock mass
Tb1	Fresh and massive, no deterioration in the rock-forming minerals. Vp larger than 5000m/s. Rock unit is hard and compact in greyish black color with RMR greater than 65.
Tb2	Slightly to moderately weathered, often broken when struck by hammer. Vp is in between 1500 to 3000m/s. Relatively hard with RMR ranging in between 35 to 45.
Tb3	Fresh to slightly weathered, massive to weakly jointed. Vp is in between 3000 to 5000m/s. Rock unit is hard in dark grey and greyish black color with RMR ranging between 45 to 65.
Tb4	Highly weathered and decomposed and can be broken by hammer. Vp is less than 1500 m/s with RMR less than 35.
Tb5	Fresh to slightly weathered, relatively hard with a few highly fractured and weathered rocks observed. Vp is in between 3000 to 5000 m/s. RMR ranging in between 45 to 65
Tpx	Broken by hand, mostly becomes sand like remaining minerals or powdered. Vp less than 1500m/s. Massively bedded and compacted greyish white to white in color with RMR less than 35.

Chapter Six

6. Conclusions and Recommendations

2.1. Conclusions

Preliminary investigation stages in geotechnics require a thorough classification and characterization of the geotechnical materials present within the project vicinity.

Rock material characterization has focused on a naturally occurring rock formations, Jimma volcanic and Makkonnen basaltic rock formations which is dominant within the south western part of the country. A better understanding of the engineering properties of rocks can provide a base for a more rational approach to use them in civil engineering works. Based on this understanding the following conclusions have been drawn.

The formation within the study area is made up of various textured basalts, mainly composed of Aphanitic basalt, Phyric Basalt, Amygdaloidal Basalts, Vesicular Basalts and Tuffs.

The range of values for the physical and mechanical property tests (water absorption, specific gravity, porosity, uniaxial compressive strength, abrasion resistance etc.) of the basaltic and tuff rock samples are summarized in Table 6.1:

Table 6.1: Physical and mechanical rock material tests with range of values obtained

Physical and mechanical property	Range of values
Porosity	1.17-13%
Water Absorption	0.2-5.52%
Specific Gravity	2.02-2.92%
Bulk Unit Weight	1.87-2.92g/cm ³
UCS	1.37-237.29MPa
ACV	16.23-20.33%
AIV	10.02-12%
LAHV	24.32-29%

Physical and mechanical property	Range of values
FI	5.26-32.24%
EI	14.65-22.57%

The second section dealt on rock mass characterization of the Jimma volcanic and Makkonen basaltic formations. The characterization technique included both surface rock mass classification at exposed rocks and drilled rock samples. The Geomechanics properties are summarized in Table 6.2:

Table 6.2: RMR parameters with range of values

RMR Parameters	Range of values
UCS (obtained from SHV)	22.525-159.37MPa
RQD (obtained from drilled core sample)	18%-94.6%
RQD (obtained from surface RMR points)	12.7%-95.2%
Spacing of discontinuities	60-2000mm
Condition of discontinuities	Un-weathered to highly weathered
Ground water condition:	Dry, Damp and Wet

Rock mass from the exposed rock surface at around the proposed dam and tunnel sections have been characterized. Accordingly, 31.25% of the rock mass has RMR value ranging from 55-65 and 65-75; 25% of the exposed rock mass have RMR value in between; 45-55 and the rest 12.5% have 35-45 RMR value.

Results from borehole as well as exposed surface revealed that the left side of the proposed dam section is relatively strong or competent rock this is also true for rock mass located within the dam axis.

The core logging from drilled borehole has also revealed the RMR class fluctuates instantly showing a variation in the strength of the intact rock sample. As depth increases a relative lowering of permeability values were observed at the proposed dam section.

The rock mass exposed at the proposed dam site falls into fair (60%-41%) to good quality (61%-80%) for the surface outcrops, whereas rock mass at depth, as observed from borehole logs, falls into fair (60%-47%) to very good quality (100%-81%) as per the Bieniawski's (1989) Rock mass classification rating system.

The seismic refraction survey and core logged data analysis at the proposed dam and tunnel site has indicated the bedrock around the dam foundation section and tunnel route respectively are represented by compressional wave velocity $>5000\text{m/s}$ and $3000\text{-}5000\text{ m/s}$ for good quality rocks.

An attempt has also been made to correlate the overall rock quality with its seismic velocity. The approach initially followed determining the average velocity from the seismic refraction survey and then determining the average RMR from drilled boreholes. A promising result leading to good correlation was found between the seismic velocity and the rock quality. Besides, the study also shows a significant correlation between seismic P-wave velocity changes due to their penetration strength.

The water pressure tests conducted at abutments indicates from low to very low permeability conditions except in some bore holes where the condition exceeds to moderate condition. In general, based on the surface and subsurface explorations the seepage condition along the abutments may be summarized as;

- Along both abutments variety of rocks are present, which are partly weathered and highly jointed. The permeability result indicates that in the top reaches the rocks are highly permeable.
- The permeability value on the right abutment, BH-07 borehole, demonstrated a relatively high permeable condition at specific depths where the lugeon value ranges from 21.00 (35.00-40.00 m) to 22.8 (80.00-89.50 m).

- Aside from its condition, excessive seepage may take place through this zone as it extends further downstream.

The current research presents a conceptual framework which puts forward a vision for rock characterization for the specific formation. The output of the characterization will, intend to provide future resource references and illustrate the suitability of the proposed sections for the intended mega structures to be constructed.

It may be concluded that the current research would provide results which would be highly beneficial to the local construction industry and the intended project. It has also tried to indicate problems mainly related to permeability and rock mass quality which require adequate treatment to overcome the problems.

2.2. Recommendations

So far, there exists no available guideline that is used to employ the basalt and tuff rock materials for use as a local construction material (Aggregate case) within the study area. A detailed physical and mechanical characterization methodology is important to save the failure of engineering structures before the design life. Further the construction materials could also be used as a dimension/building stone which in this day has a greater acceptance when considering both strength and aesthetics.

The present study identified important physical and mechanical properties of the rock formation in general. However, the following recommendations are given for future consideration:

- The tuffs which are located at around the spillway section for the proposed dam site, are the more preferred varieties for cobble stone and masonry even though the water absorption and porosity is a bit higher than the specified limits.
- The rock index property tests obtained in the laboratory are all within the permissible limit as per AASHTO, ASTM and BS standards for use in the construction industry as an effective construction material in specifically in concrete mix, in road construction, as a slab in sidewalks and as a cobble stone.

- All basaltic rocks are not always excellent sources of aggregates for either concrete or asphalt mix. Therefore, a mere attention should be given to the constituent mineral components of the rocks to identify deleterious components.
- In cases where it is impossible to locate material laboratories near newly developed quarry sites, available data obtained from quarry sites of similar formation could be gathered as a form of guideline and used as a characterization mechanism.
- Some important rock material tests such as Alkali silica reactivity (ASR) test has been left out due lack of laboratory equipment. The proposed guideline could be sufficiently fulfilled and more effective in the future with the addition of new and related parameters.
- In order to develop the trend of preparing effective guide lines such as this, standardized and well-equipped material laboratory should be developed and guided by a concerned literate personal.
- Though Ethiopia is a country rich in construction material (rock formation), proper usage of the resources has to be given more attention since there exists no recycling technology for construction material wasted during the process of concrete making. Therefore, care has to be taken for resource management and employment.

From the results and findings of the present study obtained from characterizing the rock mass, the following recommendations are forwarded which could be considered for future works as well as for design and construction of the proposed structures.

- Before designing and construction stages in mega structures proceed, a well-established and detailed preliminary investigation process must be improvised, with specific guidelines as an aiding tool, to give a general insight on the conditions of the geotechnical structures.
- From design, structural and material abundance point of view, an embankment dam specifically a rock fill dam will suit the steep slopes of the valley gorges of the Wabe river. Crushed and loss basalts and tuffs could be used as a loose rock fill dump which forms the bulk of the dam and resist the thrust of the reservoir.
- All overburden materials including gravely and clayey alluvial deposits, highly weathered tuffs and paleosols should be removed from the location of the proposed dam section

especially at around the dam axis since unconsolidated could result in degradation of the shear strength of the surrounding as well as foundation material.

According to seismic refraction and bore hole data, the basement rock which is sound and fresh and practically suitable for foundation rock and tunnel route is found at deeper depth while overlying layers are dominantly fractured and weathered. Therefore, based on the design, safety and economic point of view the design engineer could use either of the recommendations listed below:

- If considering the foundation at shallower depth, a Sevier and well improvised engineering solution is needed that is, grouting using cement paste. The drilling data especially at around the dam axis showed a sound rock at shallower depth as compared to the abutments therefore, consolidation grouting could seal the fissures located at around the river bed to a maximum depth of 15 meters.
- If considering the foundation at deeper depth due degraded and fractured rocks existing at intermediate depths, then removing the portion of highly fractured zone by excavation is a good engineering solution. Though, the excavation method should be to the best interest of the features of the foundation design scheme and the characteristics of the rock mass to be excavated.
- The foundation excavation level could be 40-50m at around the river bed and island areas, while at the around the abutment's sections 50-60m stripping depth is recommended below ground level. Excavation at around the dam axis section could be at a maximum of 35-40m depth. The right abutment is composed of weak rock mass relative to the left abutment therefore the maximum stripping level could increase up to 60-70m depth.
- A well improvised excavation methods can be employed based on the rock mass modulus (E_m) and ultimate bearing capacity (Q_{ult}) of the rock units. Excavation techniques such as easy digging for highly weathered rock units to ripping and controlled blasting for moderately weathered and highly jointed strong rock units respectively can be employed. Controlled blasting methods such as presplitting and smooth blasting are the best suggested methods for excavation. Hence the above recommendation is based on the characteristics of the rock mass obtained from the present study, though the suggested methods may alter

based on the size and shape of the foundation design, the tolerance required along the excavation line and the time allowed for the excavation to be completed.

- The left side of the dam foundation area and spillway section is surrounded by colluvial materials specifically at the sloppy sections where a series of earth slides could occur; therefore, the material shall be removed respective areas.
- During site survey, a possible cross drainage was observed at limited sections of the dam and tunnel section. The encountered cross drainage has no direct connection with the drilled boreholes. Therefore, it was impossible to conduct packer test but a pumping test could be employed to check the ground water level as well as permeability condition of the rock mass and to determine the anisotropy of the rock mass aside from identifying possible unfavorable geologic structures (dykes, shear zones, faults, folds, etc.) which has a remarkable effect on the support condition of the rock mass.
- The kinematic analysis made over the proposed structure is with a limited point structural data, therefore it is recommended to make a detailed slope stability analysis during excavation and construction period, where there will be rock exposure and detailed structural data on both of the abutments and tunnel route (inlet, power house and middle shaft) where there exists high head as well.
- The geotechnical classes obtained by dividing the rock mass along the tunnel route in to geological structural units are a useful means of identifying geotechnical parameters essential for design. Therefore, it should be adopted in many other early geotechnical investigation processes.
- The support guidelines for the proposed tunnel section, which has been recommended as per the guideline provided by the RMR system (Bieniawski, 1988), should be applied under proper caution, further investigations could strengthen the recommendation while what will be encountered during excavation should be kept under consideration.
- Geotechnical parameters obtained from RMR and Compressional wave velocity (V_p) test has been used to propose a guideline for the mapped rock units of the Jimma volcanic and Makkonnen basaltic rock formation. The proposed guideline should be used under proper caution and can be summarized and reviewed as long as new and reliable parameters are profound.

The present study has focused on characterizing the Jimma volcanic and Makkonnen basaltic rock formations and proposing a guideline specified to the proposed project vicinity which is based on limited number of samples and small area coverage because of time and financial limitations. Therefore, it is recommended to make further analysis by making observations, using number of samples and in-situ tests, covering large area over the project vicinity and where ever the formation exists so as to address a feasible and practicable indigenous guideline aside with suitability of the rock mass for the intended purpose.

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Annexes

Appendix A:

Rock Material Tests

Source: Wabe Hydropower Project Geotechnical investigation process prepared by ECDSWCo.
and the author

Summary of test results of rock core samples

Sample No.	BH-ID	Location	Lithological Description	Sampling Depth	Rock Core tests Performed in the Laboratory				
					UCS (MPa)	Unit Weight (gm/cm ³)	Specific Gravity (SSD)	Water Absorption (%)	Porosity (%)
01	BH-01	Left of Abutment	Aphanitic Basalt	26.40-26.68	97.21	2.84	2.90	-	1.97
02			Amygdaloidal Basalt	38.00-38.25	14.06	2.47	2.53	-	-
03			Aphanitic Basalt	66.05-66.35	11.89	2.60	2.71	1.91	-
04			Amygdaloidal Basalt	87.65-87.92	118.72	2.82	2.90	0.37	2.68
05	BH-02	Left of Abutment	Aphanitic Basalt	33.00-33.40	175.87	2.83	2.86	1.00	1.17
06			Aphanitic Basalt	42.70-43.00	46.089	2.72	2.80	-	-
07			Aphanitic Basalt	65.10-65.40	237.29	2.86	2.91	0.44	1.56
08	BH-03	Left of Abutment	Aphanitic Basalt	16.50-16.75	147.19	2.87	2.91	0.98	-
09			Tuff	27.70-28.00	1.3756	1.87	2.02	-	7.76
10			Aphanitic Basalt	34.00-34.30	150.07	2.79	2.80	0.60	-
11			Aphanitic Basalt	58.60-59.00	102.28	2.80	2.82	1.10	-
12	BH-04	DAM Axis	Aphanitic Basalt	12.74-13.00	56.2	-	-	0.47	-
13			Aphanitic Basalt	19.05-19.29	46.3	-	-	1.22	3
14			Aphanitic Basalt	31.35-31.80	26.2	2.32	-	2.61	-
15			Aphanitic Basalt	41.18-41.40	27.1	-	-	-	-
16			Aphanitic Basalt	46.00-46.40	36.99	-	-	3.32	8
17			Aphanitic Basalt	52.20-52.80	76.87	2.8	2.84	-	8
18			Aphanitic Basalt	60.50-60.73	48.90	-	-	2.44	6
19			Aphanitic Basalt	76.64-77.00	32.33	2.59	-	0.7	-
20			Aphanitic Basalt	86.74-87.00	36.16	2.7	2.83	5.52	13
21			Aphanitic Basalt	96.00-96.30	76.49	2.86	-	1.38	4

Sample No.	BH-ID	Location	Lithological Description	Sampling Depth	Rock Core tests Performed in the Laboratory				
					UCS (MPa)	Unit Weight (gm/cm ³)	Specific Gravity (SSD)	Water Absorption (%)	Porosity (%)
22	BH-05	Right of Abutment	Vesicular Basalt	104.75-105	12.03	2.37	-	2.65	9
23			Aphanitic Basalt	107.20-107.45	35.48	-	-	-	5.56
24			Aphanitic Basalt	15.00-15.25	54.38	2.93	-	1.91	-
25			Aphanitic Basalt	21.74-22.00	78.71	-	2.84	3.8	9
26			Aphanitic Basalt	26.36-26.69	121.63	2.91	-	-	-
27			Aphanitic Basalt	41.00-41.28	52.41	-	2.72	1.74	4
28			Aphanitic Basalt	46.24-46.59	53.29	2.79	-	-	-
29			Aphanitic Basalt	62.70-63.00	67.54	2.71	-	4.21	10
30			Aphanitic Basalt	66.00-66.28	51.98	2.81	-	-	3.91
31			Aphanitic Basalt	72.18-72.43	37.80	-	-	4.5	-
32			Aphanitic Basalt	80.45-80.78	100.69	2.92	-	-	1
33			Aphanitic Basalt	87.38-87.68	57.06	-	-	8.56	-
34			BH-11	Spillway	Aphanitic Basalt	14.15-14.50	39.03	2.83	2.87
35	Phyric Basalt	26.70-27.00			52.30	-	2.92	-	-
36	Aphanitic Basalt	34.00-34.28			86.33	2.9	-	-	-
37	Aphanitic Basalt	37.13-37.43			88.72	-	2.89	-	-
38	Aphanitic Basalt	42.00-42.36			62.31	2.8	-	-	-
39	BH-09	Intake	Aphanitic Basalt	13.56-13.90	168.59	2.78	2.87	-	3.14
40			Aphanitic Basalt	47.10-47.40	207.02	2.85	2.92	0.31	2.27
41	BH-13	Surge shaft	Aphanitic Basalt	135.65-135.85	155.59	-	2.87	0.2	-
42			Aphanitic Basalt	141.15-141.34	100.52	-	2.86	0.44	-

Sample No.	BH-ID	Location	Lithological Description	Sampling Depth	In- Situ Rock Core tests		
					RQD (%)	SHV (Ra)	UCS (from Barton and Choubey (1977) relation)
1	BH-01	Left of Abutment	Aphanitic Basalt	11.00-20.00	6.2	13	4.7
2			Aphanitic Basalt	23.70-27.00	36.4	38	53.3
3			Amygdaloidal Basalt	36.00-44.00	39.4	44	74.3
4			Aphanitic Basalt	44.80-46.00	74	39.5	58.1
5			Phyric Basalt	61.00-63.60	23.1	31	33.5
6			Phyric Basalt	77.10-78.80	11.2	23	17
7			Amygdaloidal Basalt	79.70-90.00	92	53	113.4
8	BH-02	Left of Abutment	Aphanitic Basalt	21.70-39.50	66.5	45.2	79
9			Aphanitic Basalt	41.60-46.60	69.2	46.8	85.4
10			Aphanitic Basalt	48.80-70.60	73.8	46.4	83.8
11	BH-03	Left of Abutment	Phyric Basalt	1.70-12.50	3.3	47	86.3
12			Aphanitic Basalt	12.50-23.50	84.6	54.9	123
13			Tuff	23.50-31.00	5	9.89	2.5
14			Tuff	31.00-34.15	54	12	3.9
15			Aphanitic Basalt	34.15-41.00	77.4	49.4	96.6
16			Phyric Basalt	48.75-54.60	28.2	43	70.5
17			Phyric Basalt	54.60-56.20	90	51	103.9

Sample No.	BH-ID	Location	Lithological Description	Sampling Depth	In- Situ Rock Core tests		
					RQD (%)	SHV (Ra)	UCS (from Barton and Choubey (1977) relation)
18	BH-04	DAM Axis	Aphanitic Basalt	56.20-70.00	94.2	64.9	180
19			Tuff	9.70-12.00	0	19.7	12
20			Aphanitic Basalt	12.00-20.70	85.8	37.2	50.7
21			Tuff	22.00-23.50	17	9.9	2.51
22			Phyric Basalt	25.00-30.00	12	12	3.9
23			Aphanitic Basalt	37.70-71.00	93.9	50.5	101.1
24			Aphanitic Basalt	71.00-78.00	68.6	13.7	5.2
25			Aphanitic Basalt	103.00-106.00	84.7	32.3	36.8
26			Aphanitic Basalt	106.00-110.00	55.5	46.2	83
27	BH-05	Right of Abutment	Aphanitic Basalt	10.00-29.00	85.3	39.8	59.13
28			Phyric Basalt	38.00-40.00	0	9.9	2.51
29			Phyric Basalt	47.50-59.80	0	9.9	2.51
30			Phyric Basalt	59.80-62.00	20.1	9.9	2.51
31			Aphanitic Basalt	62.00-81.00	91	51.2	104.8
32			Aphanitic Basalt	81.00-83.50	25	20.9	13.8
33	BH-06	Right of Abutment	Phyric Basalt	9.00-14.30	0	23.9	18.7
34			Aphanitic Basalt	16.70-30.00	41	48.5	92.7
35			Phyric Basalt	30.00-35.80	6.4	32.9	38.6
36			Aphanitic Basalt	35.80-54.00	73.1	47.5	88.4
37			Aphanitic Basalt	61.30-71.00	74.7	48.9	94.8
38	BH-07	Right of Abutment	Aphanitic Basalt	21.40-33.00	91	47.9	90.5
39			Aphanitic Basalt	33.00-39.00	0	0	0
40			Vesicular Basalt	39.00-42.00	59	30.9	33.5
41			Tuff	67.50-69.60	18.3	17	8.6
42			Aphanitic Basalt	69.60-80.30	84.2	41.9	66.8
43			Vesicular Basalt	85.70-89.50	76.3	47	86.3

Summary of test results of Aggregate rock material

Observation sheet for Water Absorption and Specific Gravity Tests (Production Day-One) (16/09/2019)			
Description	Sample-I	Sample-II	Sample-III
Weight of sample	1000gm	1000gm	1000gm
Weight of vessel+ sample+ water (A)	3605gm	3740gm	3615gm
Weight of Vessel + water (B)	3110gm	3120gm	3120gm
Weight of SSD Sample (C)	868.26gm	1109.83gm	847.67gm
Weight of oven dry sample (D)	787.89gm	998.05gm	776.25gm
SG (Apparent)=(D/D-(A-B)) *100	2.69	2.64	2.76
WA=((C-D)/D) *100	0.102	0.112	0.092
Avg. Apparent SG %	2.69%		
Avg. WA %	0.102%		

Summary of test results of Aggregate rock material (Cont'd)

Observation sheet for Water Absorption and Specific Gravity Tests (Production Day-Two) (17/09/2019)			
Description	Sample-I	Sample-II	Sample-III
Weight of sample	1000gm	1000gm	1000gm
Weight of vessel+ sample+ water (A)	3620gm	3610gm	3735gm
Weight of Vessel + water (B)	3122gm	3120gm	3120gm
Weight of SSD Sample (C)	1165.9gm	1018.82gm	1281.63gm
Weight of oven dry sample (D)	913gm	825.62gm	1057.45gm
SG (Apparent)=(D/D-(A-B)) *100	2.20	2.46	2.39
WA=((C-D)/D) *100	0.277	0.234	0.212
Avg. Apparent SG %	2.35%		
Avg. WA %	0.241%		

Summary of test results of Aggregate rock material (Cont'd)

Observation sheet for Water Absorption and Specific Gravity Tests (Production Day-Three) (18/09/2019)			
Description	Sample-I	Sample-II	Sample-III
Weight of sample	1000gm	1000gm	1000gm
Weight of vessel+ sample+ water (A)	3624gm	3620gm	3740gm
Weight of Vessel + water (B)	3115gm	3118gm	3128gm
Weight of SSD Sample (C)	878.72gm	879.29gm	1104.28gm
Weight of oven dry sample (D)	803.22gm	800.81gm	967.82gm
SG (Apparent)=(D/D-(A-B)) *100	2.73	2.68	2.72
WA=((C-D)/D) *100	0.094	0.098	0.141
Avg. Apparent SG %	2.71%		
Avg. WA %	0.111%		

Summary of test results of Aggregate rock material (Cont'd)

Observation sheet for Water Absorption and Specific Gravity Tests (Production Day-four) (19/09/2019)			
Description	Sample-I	Sample-II	Sample-III
Weight of sample	1000gm	1000gm	1000gm
Weight of vessel+ sample+ water (A)	3743gm	3741gm	3740gm
Weight of Vessel + water (B)	3128gm	3120gm	3120gm
Weight of SSD Sample (C)	1106.12gm	1097.19gm	1028.19gm
Weight of oven dry sample (D)	1009.23gm	988.46gm	934.72gm
SG (Apparent)=(D/D-(A-B)) *100	2.56	2.69	2.97
WA=((C-D)/D) *100	0.096	0.110	0.100
Avg. Apparent SG %	2.74%		
Avg. WA %	0.102%		

Summary of test results of Aggregate rock material (Cont'd)

Observation sheet for Water Absorption and Specific Gravity Tests (Production Day-Five) (20/09/2019)			
Description	Sample-I	Sample-II	Sample-III
Weight of sample	1000gm	1000gm	1000gm
Weight of vessel+ sample+ water (A)	3620gm	3631gm	3615gm
Weight of Vessel + water (B)	3115gm	3125gm	3118gm
Weight of SSD Sample (C)	973.19gm	998.94gm	979.36gm
Weight of oven dry sample (D)	800.32gm	826.25gm	791.08gm
SG (Apparent)=(D/D-(A-B)) *100	2.71	2.58	2.69
WA=((C-D)/D) *100	0.216	0.209	0.238
Avg. Apparent SG %	2.66%		
Avg. WA %	0.221%		

Summary of test results of Aggregate rock material (Cont'd)

Observation sheet for Water Absorption and Specific Gravity Tests (Production Day-Six) (21/09/2019)			
Description	Sample-I	Sample-II	Sample-III
Weight of sample	1000gm	1000gm	1000gm
Weight of vessel+ sample+ water (A)	3743gm	3629gm	3741gm
Weight of Vessel + water (B)	3126gm	3114gm	3129gm
Weight of SSD Sample (C)	1217.08gm	1007.82gm	1219.22gm
Weight of oven dry sample (D)	988.69gm	823.38gm	996.91gm
SG (Apparent)=(D/D-(A-B)) *100	2.66	2.67	2.59
WA=((C-D)/D) *100	0.231	0.224	0.223
Avg. Apparent SG %	2.64%		
Avg. WA %	0.226%		

Summary of test results of Aggregate rock material (Cont'd)

Observation sheet for Water Absorption and Specific Gravity Tests (Production Day-Seven) (22/09/2019)			
Description	Sample-I	Sample-II	Sample-III
Weight of sample	1000gm	1000gm	1000gm
Weight of vessel+ sample+ water (A)	3625gm	3633gm	3635gm
Weight of Vessel + water (B)	3114gm	3121gm	3133gm
Weight of SSD Sample (C)	1050.59gm	1084.48gm	1072.13gm
Weight of oven dry sample (D)	845.21gm	888.19gm	894.19gm
SG (Apparent)=(D/D-(A-B)) *100	2.53	2.36	2.28
WA=((C-D)/D) *100	0.243	0.221	0.199
Avg. Apparent SG %	2.39%		
Avg. WA %	0.221%		

Summary of test results of Aggregate rock material

Observation sheet for Water Absorption and Specific Gravity Tests (Production Day-One) (16/09/2019)			
Description	Sample-I	Sample-II	Sample-III
Weight of sample	1000gm	1000gm	1000gm
Weight of vessel+ sample+ water (A)	3605gm	3740gm	3615gm
Weight of Vessel + water (B)	3110gm	3120gm	3120gm
Weight of SSD Sample (C)	868.26gm	1109.83gm	847.67gm
Weight of oven dry sample (D)	787.89gm	998.05gm	776.25gm
SG (Apparent)=(D/D-(A-B)) *100	2.69	2.64	2.76
WA=((C-D)/D) *100	0.102	0.112	0.092
Avg. Apparent SG %	2.69%		
Avg. WA %	0.102%		

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Checked By: [Signature]

Approved By:

Amala Getaw Adugna

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Summary of test results of Aggregate rock material (AIV)

Observation Sheet for Aggregate Impact Value (Production Day-One) (26/08/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Total weight of Dry Sample (W ₁) gm	365	363.5
Wt. of portion passing 2.36mm IS sieve (W ₂) gm	40.22	39.62
Wt. of cylinder (gm)	1040	1040
Wt. of cylinder + Aggregate (gm)	1405	1403.5
Wt. of Aggregate retained on 2.36mm IS sieve (W ₃) (gm)	324.78	323.88
AIV= (W ₂ /W ₁) *100	11.02	10.89
Avg. AIV	10.96%	

Summary of test results of Aggregate rock material (AIV) Cont'd

Observation Sheet for Aggregate Impact Value (Production Day-Two) (27/08/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Total weight of Dry Sample (W_1) gm	375	373.21
Wt. of portion passing 2.36mm IS sieve (W_2) gm	44.78	45.01
Wt. of cylinder (gm)	1040	1040
Wt. of cylinder + Aggregate (gm)	1415	1413.21
Wt. of Aggregate retained on 2.36mm IS sieve (W_3) (gm)	330.22	328.20
AIV= (W_2/W_1) *100	11.94	12.06
Avg. AIV	12%	

Summary of test results of Aggregate rock material (AIV) Cont'd

Observation Sheet for Aggregate Impact Value (Production Day-Three) (28/08/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Total weight of Dry Sample (W_1) gm	375	377.8
Wt. of portion passing 2.36mm IS sieve (W_2) gm	40.91	39.40
Wt. of cylinder (gm)	1040	1040
Wt. of cylinder + Aggregate (gm)	1415	1417.8
Wt. of Aggregate retained on 2.36mm IS sieve (W_3) (gm)	334.09	338.4
AIV= (W_2/W_1) *100	10.91	10.43
Avg. AIV	10.67%	

Summary of test results of Aggregate rock material (AIV) Cont'd

Observation Sheet for Aggregate Impact Value (Production Day-Four) (29/08/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Total weight of Dry Sample (W_1) gm	364.5	369
Wt. of portion passing 2.36mm IS sieve (W_2) gm	39.00	40.51
Wt. of cylinder (gm)	1040	1040
Wt. of cylinder + Aggregate (gm)	1404.5	1409
Wt. of Aggregate retained on 2.36mm IS sieve (W_3) (gm)	325.50	328.49
AIV= (W_2/W_1) *100	10.7	10.98
Avg. AIV	10.84%	

Summary of test results of Aggregate rock material (AIV) Cont'd

Observation Sheet for Aggregate Impact Value (Production Day-Five) (30/08/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Total weight of Dry Sample (W_1) gm	370.1	368.4
Wt. of portion passing 2.36mm IS sieve (W_2) gm	36.94	37.06
Wt. of cylinder (gm)	1040	1040
Wt. of cylinder + Aggregate (gm)	1410.1	1408.4
Wt. of Aggregate retained on 2.36mm IS sieve (W_3) (gm)	333.16	331.34
AIV= (W_2/W_1) *100	9.98	10.06
Avg. AIV	10.02%	

Summary of test results of Aggregate rock material (AIV) Cont'd

Observation Sheet for Aggregate Impact Value (Production Day-Six) (31/08/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Total weight of Dry Sample (W_1) gm	359.4	363.1
Wt. of portion passing 2.36mm IS sieve (W_2) gm	39.64	39.39
Wt. of cylinder (gm)	1040	1040
Wt. of cylinder + Aggregate (gm)	1399.4	1403.1
Wt. of Aggregate retained on 2.36mm IS sieve (W_3) (gm)	319.76	323.71
AIV= (W_2/W_1) *100	11.03	10.85
Avg. AIV	10.94%	

Summary of test results of Aggregate rock material (AIV) Cont'd

Observation Sheet for Aggregate Impact Value (Production Day-Seven) (01/09/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Total weight of Dry Sample (W_1) gm	368.7	368.4
Wt. of portion passing 2.36mm IS sieve (W_2) gm	40.82	41.15
Wt. of cylinder (gm)	1040	1040
Wt. of cylinder + Aggregate (gm)	1408.7	1408.4
Wt. of Aggregate retained on 2.36mm IS sieve (W_3) (gm)	327.88	327.25
AIV= (W_2/W_1) *100	11.07	11.17
Avg. AIV	11.12%	

Summary of test results of Aggregate rock material (AIV) Cont'd

Observation Sheet for Aggregate Impact Value (Production Day-Five) (30/08/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Total weight of Dry Sample (W_1) gm	370.1	368.4
Wt. of portion passing 2.36mm IS sieve (W_2) gm	36.94	37.06
Wt. of cylinder (gm)	1040	1040
Wt. of cylinder + Aggregate (gm)	1410.1	1408.4
Wt. of Aggregate retained on 2.36mm IS sieve (W_3) (gm)	333.16	331.34
AIV = $(W_2/W_1) * 100$	9.98	10.06
Avg. AIV	10.02%	

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Checked By: 

Approved By: _____

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Amalu Gelaw Adugna

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Summary of test results of Aggregate rock material (ACV Test)

Observation Sheet for Aggregate Crushing Value (Production Day-One) (26/08/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Total weight of dry sample taken (W_1) gm	3000	3000
Weight of Cylinder	6500	6500
Weight of portion passing 2.36mm IS sieve (W_2)	553.80	576
Weight of aggregate retained on 2.36mm IS sieve	2446.2	2424
Aggregate Crushing Value (W_2/W_1) (%)	18.46	19.20
Avg. ACV (%)	18.83%	

Summary of test results of Aggregate rock material (ACV Test) (Cont'd)

Observation Sheet for Aggregate Crushing Value (Production Day-Two) (27/08/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Total weight of dry sample taken (W_1) gm	3000	3000
Weight of Cylinder	6500	6500
Weight of portion passing 2.36mm IS sieve (W_2)	615	604.8
Weight of aggregate retained on 2.36mm IS sieve	2385	2395.2
Aggregate Crushing Value (W_2/W_1) (%)	20.50	20.16
Avg. ACV (%)	20.33%	

Summary of test results of Aggregate rock material (ACV Test) (Cont'd)

Observation Sheet for Aggregate Crushing Value (Production Day-Three) (28/08/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Total weight of dry sample taken (W_1) gm	3000	3000
Weight of Cylinder	6500	6500
Weight of portion passing 2.36mm IS sieve (W_2)	479.70	494.10
Weight of aggregate retained on 2.36mm IS sieve	2520.3	2505.9
Aggregate Crushing Value (W_2/W_1) (%)	15.99	16.47
Avg. ACV (%)	16.23%	

Summary of test results of Aggregate rock material (ACV Test) (Cont'd)

Observation Sheet for Aggregate Crushing Value (Production Day-Four) (29/08/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Total weight of dry sample taken (W_1) gm	3000	3000
Weight of Cylinder	6500	6500
Weight of portion passing 2.36mm IS sieve (W_2)	549.60	541.80
Weight of aggregate retained on 2.36mm IS sieve	2450.4	2458.2
Aggregate Crushing Value (W_2/W_1) (%)	18.32	18.06
Avg. ACV (%)	18.19%	

Summary of test results of Aggregate rock material (ACV Test) (Cont'd)

Observation Sheet for Aggregate Crushing Value (Production Day-Five) (30/08/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Total weight of dry sample taken (W_1) gm	3000	3000
Weight of Cylinder	6500	6500
Weight of portion passing 2.36mm IS sieve (W_2)	556.20	558
Weight of aggregate retained on 2.36mm IS sieve	2443.8	2442
Aggregate Crushing Value (W_2/W_1) (%)	18.54	18.60
Avg. ACV (%)	18.57%	

Summary of test results of Aggregate rock material (ACV Test) (Cont'd)

Observation Sheet for Aggregate Crushing Value (Production Day-Six) (31/08/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Total weight of dry sample taken (W_1) gm	3000	3000
Weight of Cylinder	6500	6500
Weight of portion passing 2.36mm IS sieve (W_2)	575.70	576.90
Weight of aggregate retained on 2.36mm IS sieve	2424.3	2423.1
Aggregate Crushing Value (W_2/W_1) (%)	19.19	19.23
Avg. ACV (%)	19.21%	

Summary of test results of Aggregate rock material (ACV Test)

Observation Sheet for Aggregate Crushing Value (Production Day-Seven) (01/09/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Total weight of dry sample taken (W_1) gm	3000	3000
Weight of Cylinder	6500	6500
Weight of portion passing 2.36mm IS sieve (W_2)	581.40	586.20
Weight of aggregate retained on 2.36mm IS sieve	2418.6	2413.8
Aggregate Crushing Value (W_2/W_1) (%)	19.38	19.54
Avg. ACV (%)	19.46%	

Summary of test results of Aggregate rock material (LAAV Test)

Observation Sheet for LAAV (Production Day-One) (09/09/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Original wt. of the sample (W_1) gm	5000	5000
Wt. of aggregate retained in 1.70mm IS sieve (W_2) gm	3613	3587
Wt. of Fraction passing 1.70mm IS Sieve (W_2-W_1) gm	1387	1413
Percentage Wear ($(W_1-W_2)/W_1$) *100	0.2774	0.2826
Los Angeles Abrasion Value (LAAV)	27.74	28.26
Avg. LAAV	28%	

Summary of test results of Aggregate rock material (LAAV Test) (Cont'd)

Observation Sheet for LAAV (Production Day-Two) (10/09/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Original wt. of the sample (W ₁) gm	5000	5000
Wt. of aggregate retained in 1.70mm IS sieve (W ₂) gm	3553.8	3546.5
Wt. of Fraction passing 1.70mm IS Sieve (W ₂ -W ₁) gm	1446.50	1453.5
Percentage Wear ((W ₁ -W ₂)/W ₁) *100	0.2893	0.2907
Los Angeles Abrasion Value (LAAV)	28.93	29.07
Avg. LAAV	29%	

Summary of test results of Aggregate rock material (LAAV Test) (Cont'd)

Observation Sheet for LAAV (Production Day-Three) (11/09/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Original wt. of the sample (W ₁) gm	5000	5000
Wt. of aggregate retained in 1.70mm IS sieve (W ₂) gm	3693	3696
Wt. of Fraction passing 1.70mm IS Sieve (W ₂ -W ₁) gm	1307	1304
Percentage Wear ((W ₁ -W ₂)/W ₁) *100	0.2614	0.2608
Los Angeles Abrasion Value (LAAV)	26.14	26.08
Avg. LAAV	26.11%	

Summary of test results of Aggregate rock material (LAAV Test) (Cont'd)

Observation Sheet for LAAV (Production Day-Four) (12/09/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Original wt. of the sample (W ₁) gm	5000	5000
Wt. of aggregate retained in 1.70mm IS sieve (W ₂) gm	3754	3767
Wt. of Fraction passing 1.70mm IS Sieve (W ₂ -W ₁) gm	1246	1233
Percentage Wear ((W ₁ -W ₂)/W ₁) *100	0.2492	0.2466
Los Angeles Abrasion Value (LAAV)	24.92	24.66
Avg. LAAV	24.79%	

Summary of test results of Aggregate rock material (LAAV Test) (Cont'd)

Observation Sheet for LAAV (Production Day-Five) (13/09/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Original wt. of the sample (W_1) gm	5000	5000
Wt. of aggregate retained in 1.70mm IS sieve (W_2) gm	3751.5	3737.5
Wt. of Fraction passing 1.70mm IS Sieve (W_2-W_1) gm	1248.5	1262.5
Percentage Wear $((W_1-W_2)/W_1) * 100$	0.2497	0.2525
Los Angeles Abrasion Value (LAAV)	24.97	25.25
Avg. LAAV	25.11%	

Summary of test results of Aggregate rock material (LAAV Test) (Cont'd)

Observation Sheet for LAAV (Production Day- Six) (14/09/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Original wt. of the sample (W_1) gm	5000	5000
Wt. of aggregate retained in 1.70mm IS sieve (W_2) gm	3785.5	3782.5
Wt. of Fraction passing 1.70mm IS Sieve (W_2-W_1) gm	1214.50	1217.50
Percentage Wear $((W_1-W_2)/W_1) * 100$	0.2429	0.2435
Los Angeles Abrasion Value (LAAV)	24.29	24.35
Avg. LAAV	24.32%	

Summary of test results of Aggregate rock material (LAAV Test) (Cont'd)

Observation Sheet for LAAV (Production Day- Seven) (15/09/2019)		
Parameters to be obtained	Sample-ID	
	Sample-I	Sample-II
Original wt. of the sample (W_1) gm	5000	5000
Wt. of aggregate retained in 1.70mm IS sieve (W_2) gm	3587	3582
Wt. of Fraction passing 1.70mm IS Sieve (W_2-W_1) gm	1413	1418
Percentage Wear $((W_1-W_2)/W_1) * 100$	0.2826	0.2836
Los Angeles Abrasion Value (LAAV)	28.26	28.36
Avg. LAAV	28.31%	

Summary of test results of Aggregate rock material (Shape Test) (Cont'd)

Observation Sheet for Shape Test (Production Day- one) (23/09/2019)					
IS Sieve Size(mm)		Sample Weight (gm)	Weight of agg. Passing Thickness Gauge (gm)	Weight of non agg. Retained on Thickness Gauge (gm)	Weight of agg. Retained on Elongation gauge (gm)
Passing	Retained				
63.0	50.0				
50.0	40.0				
40.0	31.50				
31.50	25.0				
25.0	20.0	4163	174	3989	520
20.0	16.0	2617	111	2506	313
16.0	12.5	1482	101	1381	221
12.5	10.0	831	92	739	208
10.0	6.3				
Total		A=9093	B=478	C=8615	D=1262
FI=(B/A) *100		5.26%			
EI=(D/C) *100		14.65%			

Summary of test results of Aggregate rock material (Shape Test) (Cont'd)

Observation Sheet for Shape Test (Production Day- Two) (24/09/2019)					
IS Sieve Size(mm)		Sample Weight (gm)	Weight of agg. Passing Thickness Gauge (gm)	Weight of non agg. Retained on Thickness Gauge (gm)	Weight of agg. Retained on Elongation gauge (gm)
Passing	Retained				
63.0	50.0				
50.0	40.0				
40.0	31.50				
31.50	25.0				
25.0	20.0	5321.2	1377.2	3944	594.3
20.0	16.0	5001.5	1370.5	3631	524.1
16.0	12.5	4834.4	1845.1	2989.3	434.0
12.5	10.0	3911.6	1555.6	2356	418
10.0	6.3				
Total		A=19,068.7	B=6148.4	C=12,920.3	D=1970.4
FI=(B/A) *100		32.24%			
EI=(D/C) *100		15.25%			

Summary of test results of Aggregate rock material (Shape Test) (Cont'd)

Observation Sheet for Shape Test (Production Day- Three) (25/09/2019)					
IS Sieve Size(mm)		Sample Weight (gm)	Weight of agg. Passing Thickness Gauge (gm)	Weight of non agg. Retained on Thickness Gauge (gm)	Weight of agg. Retained on Elongation gauge (gm)
Passing	Retained				
63.0	50.0				
50.0	40.0				
40.0	31.50				
31.50	25.0				
25.0	20.0	3821.3	901.7	2919.6	611.9
20.0	16.0	3206	1112	2094	546
16.0	12.5	2675.5	804.5	1871	385
12.5	10.0	1793.1	849.1	944	224
10.0	6.3				
Total		A=11,495.9	B=3667.3	C=7828.6	D=1766.9
FI=(B/A) *100		31.9%			
EI=(D/C) *100		22.57%			

Summary of test results of Aggregate rock material (Shape Test) (Cont'd)

Observation Sheet for Shape Test (Production Day- Four) (26/09/2019)					
IS Sieve Size(mm)		Sample Weight (gm)	Weight of agg. Passing Thickness Gauge (gm)	Weight of non agg. Retained on Thickness Gauge (gm)	Weight of agg. Retained on Elongation gauge (gm)
Passing	Retained				
63.0	50.0				
50.0	40.0				
40.0	31.50				
31.50	25.0				
25.0	20.0	3919	1055.6	2863.4	556.3
20.0	16.0	3140	324	2816	414
16.0	12.5	2611.3	398.3	2213	309
12.5	10.0	1643	700	943	217
10.0	6.3				
Total		A=11,313.3	B=2477.9	C=8835.4	D=1496.3
FI=(B/A) *100		21.9%			
EI=(D/C) *100		16.94%			

Summary of test results of Aggregate rock material (Shape Test) (Cont'd)

Observation Sheet for Shape Test (Production Day- Two) (24/09/2019)					
IS Sieve Size(mm)		Sample Weight (gm)	Weight of agg. Passing Thickness Gauge (gm)	Weight of non agg. Retained on Thickness Gauge (gm)	Weight of agg. Retained on Elongation gauge (gm)
Passing	Retained				
63.0	50.0				
50.0	40.0				
40.0	31.50				
31.50	25.0				
25.0	20.0	5321.2	1377.2	3944	594.3
20.0	16.0	5001.5	1370.5	3631	524.1
16.0	12.5	4834.4	1845.1	2989.3	434.0
12.5	10.0	3911.6	1555.6	2356	418
10.0	6.3				
Total		A=19,068.7	B=6148.4	C=12,920.3	D=1970.4
FI=(B/A) *100		32.24%			
EI=(D/C) *100		15.25%			

Prepared By: Mihret Wujira

Checked By: 

Approved By:


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Head, Department of Construction
Technology & Management

Summary of test results of Aggregate rock material (Shape Test) (Cont'd)

Observation Sheet for Shape Test (Production Day- Three) (25/09/2019)					
IS Sieve Size(mm)		Sample Weight (gm)	Weight of agg. Passing Thickness Gauge (gm)	Weight of non agg. Retained on Thickness Gauge (gm)	Weight of agg. Retained on Elongation gauge (gm)
Passing	Retained				
63.0	50.0				
50.0	40.0				
40.0	31.50				
31.50	25.0				
25.0	20.0	3821.3	901.7	2919.6	611.9
20.0	16.0	3206	1112	2094	546
16.0	12.5	2675.5	804.5	1871	385
12.5	10.0	1793.1	849.1	944	224
10.0	6.3				
Total		A=11,495.9	B=3667.3	C=7828.6	D=1766.9
FI=(B/A) *100		31.9%			
EI=(D/C) *100		22.57%			

Prepared By: Mihret Wujira

Checked By: 

Approved By:

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Amala Gelaw Adugna

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Appendix B:

Rock Mass Characterization Methodologies

Source: Wabe Hydropower Project Geotechnical investigation process prepared by the author

Table: RMR Data through Tunnel Route

Rock Parameters for RMR_{Basic}			
RMR Data Point= OC-BH-16-1		Photo No.= 01	Elevation: 1627m Easting: 0354182 Northing: 0909233
No.	Parameters	Value	Rating
1	Point Load Lump Strength (I_L)= UCS (q_c)= $15I_L$ = SHV= $R_a=(264)/11= 24$ $\gamma= 2.095$ $\text{Log}_{10}(\sigma_c) = 0.00088\gamma R + 1.01= 45.256$	$\sigma_c=45.256$	4
2	Rock Quality Designation (RQD) in % $J_{v1}=14$ $J_{v(a)} = \Sigma (1/ (J_{v1}+ J_{v2}\dots + J_{v5}))$ $J_{v2}=10$ $J_{v(a)}=24$ $J_{v3}=$ $\text{RQD}=115-3.3J_{v(a)}=35.8$ $J_{v4}=$ $J_{v5}=$	35.8	8
3	Spacing of discontinuities	$(7700)_=550\text{mm}$ 14	10
4	Condition of Discontinuities		
	(i) Discontinuity Length	8.20m	1
	(ii) Aperture	5mm-0.5cm	1
	(iii) Roughness	Rough	5
	(iv) Infilling thickness	clay	2
	(v) Degree of Weathering	Highly weathered	1
5	Ground Water Condition	Damp	10
6	Orientation of discontinuities	Tunnel case	-5
	Total RMR _{basic}	37 (IV, Poor Rock)	
RMR Data Point= OC-BH-16-2		Photo No.= 02	Elevation: 1568m Easting: 0353817 Northing: 0909097
No.	Parameters	Value	Rating
1	Point Load Lump Strength (I_L)= UCS (q_c)= $15I_L$ = SHV= $R_a= (35)/3= 11.67$ $\gamma= 2.095$ $\text{Log}_{10}(\sigma_c) = 0.00088\gamma R + 1.01= 22.525$	$\sigma_c=22.525$	2
2	Rock Quality Designation (RQD) in % $J_{v1}=9$ $J_{v(a)} = \Sigma (1/ (J_{v1}+ J_{v2}\dots + J_{v5}))$ $J_{v2}=3$ $J_{v(a)}=12$ $J_{v3}=$ $\text{RQD}=115-3.3J_{v(a)}=75.4$ $J_{v4}=$ $J_{v5}=$	75.4	17
3	Spacing of discontinuities	$(3350)_=335\text{mm}$ 10	10
4	Condition of Discontinuities		
	(i) Discontinuity Length	9.0m	2
	(ii) Aperture	0.1mm-0.5mm	4
	(iii) Roughness	Rough	5
	(iv) Infilling thickness	Weathered material	4
	(v) Degree of Weathering	Highly weathered	1
5	Ground Water Condition	Damp	10
6	Orientation of discontinuities	Tunnel case	-5
	Total RMR _{basic}	50 (III, Fair Rock)	

Rock Parameters for RMR _{Basic}			
RMR Data Point= OC-BH-13		Photo No.= 07 Elevation: 1741m Easting: 0363851 Northing: 0910404	
No.	Parameters	Value	Rating
1	Point Load Lump Strength (I_L)= UCS (q_c)= $15I_L$ = SHV= $R_a = (430)/8 = 53.75$ $\gamma = 2.79$ $\text{Log}_{10}(\sigma_c) = 0.00088\gamma R + 1.01 = 132.977$	$\sigma_c = 132.977$	12
2	Rock Quality Designation (RQD) in % $J_{v1} = 8$ $J_{v(a)} = \Sigma (1 / (J_{v1} + J_{v2} \dots + J_{v5}))$ $J_{v2} = 4$ $J_{v(a)} = 12$ $J_{v3} =$ RQD= $115 - 3.3J_{v(a)} = 75.4$ $J_{v4} =$ $J_{v5} =$	75.4	17
3	Spacing of discontinuities	$(10250) / 12 = 854.2\text{mm}$	15
4	Condition of Discontinuities		
	(i) Discontinuity Length	14.0m	1
	(ii) Aperture	<0.1mm	5
	(iii) Roughness	Rough	5
	(iv) Infilling thickness	clay	2
	(v) Degree of Weathering	Fresh-Slightly weathered	5
5	Ground Water Condition	Dry	15
6	Orientation of discontinuities	Tunnel case	-5
	Total RMR _{basic}	72 (I, Very good Rock)	
RMR Data Point= OC-BH-12-2		Photo No.= 05 Elevation: 1808m Easting: 0367369 Northing: 0909954	
No.	Parameters	Value	Rating
1	Point Load Lump Strength (I_L)= UCS (q_c)= $15I_L$ = SHV= $R_a = (241)/4 = 60.25$ $\gamma = 2.79$ $\text{Log}_{10}(\sigma_c) = 0.00088\gamma R + 1.01 = 148.936$	$\sigma_c = 148.936$	12
2	Rock Quality Designation (RQD) in % $J_{v1} = 4$ $J_{v(a)} = \Sigma (1 / (J_{v1} + J_{v2} \dots + J_{v5}))$ $J_{v2} = 5$ $J_{v(a)} = 9$ $J_{v3} =$ RQD= $115 - 3.3J_{v(a)} = 85.3$ $J_{v4} =$ $J_{v5} =$	85.3	17
3	Spacing of discontinuities	$(10200) / 9 = 1133\text{mm}$	15
4	Condition of Discontinuities		
	(i) Discontinuity Length	7.30m	2
	(ii) Aperture	1-2cm	5
	(iii) Roughness	Rough	5
	(iv) Infilling thickness	Weathered material	2
	(v) Degree of Weathering	Fresh	6
5	Ground Water Condition	Damp	10
6	Orientation of discontinuities	Tunnel case	-5
	Total RMR _{basic}	69 (II, Good Rock)	

Rock Parameters for RMR _{Basic}			
RMR Data Point= OC-BH-15		Photo No.= 03 Elevation: 1712m Easting: 0355376 Northing: 0908594	
No.	Parameters	Value	Rating
1	Point Load Lump Strength (I_L)= UCS (q_c)= $15I_L$ = SHV= $R_a = (379)/7 = 54.14$ $\gamma = 2.79$ $\text{Log}_{10}(\sigma_c) = 0.00088\gamma R + 1.01 = 133.935$	$\sigma_c = 133.935$	12
2	Rock Quality Designation (RQD) in % $J_{v1} = 20$ $J_{v(a)} = \sum (1 / (J_{v1} + J_{v2} \dots + J_{v5}))$ $J_{v2} = 20$ $J_{v(a)} = 20$ $J_{v3} = 20$ $RQD = 115 - 3.3J_{v(a)} = 49$ $J_{v4} = 20$ $J_{v5} = 20$	49	8
3	Spacing of discontinuities	$(12210)/20 = 610.5\text{mm}$	15
4	Condition of Discontinuities		
	(i) Discontinuity Length	12.0m	1
	(ii) Aperture	0.1mm-0.5mm	5
	(iii) Roughness	Rough	5
	(iv) Infilling thickness	clay	2
	(v) Degree of Weathering	Fresh-slightly weathered	5
5	Ground Water Condition	Dry	15
6	Orientation of discontinuities	Tunnel case	-5
	Total RMR _{basic}	63 (II, Good Rock)	
RMR Data Point= OC-BH-12-1		Photo No.= 04 Elevation: 1814m Easting: 0367345 Northing: 0909740	
No.	Parameters	Value	Rating
1	Point Load Lump Strength (I_L)= UCS (q_c)= $15I_L$ = SHV= $R_a = (404)/8 = 50.5$ $\gamma = 2.79$ $\text{Log}_{10}(\sigma_c) = 0.00088\gamma R + 1.01 = 124.998$	$\sigma_c = 124.998$	12
2	Rock Quality Designation (RQD) in % $J_{v1} = 9$ $J_{v(a)} = \sum (1 / (J_{v1} + J_{v2} \dots + J_{v5}))$ $J_{v2} = 8$ $J_{v(a)} = 17$ $J_{v3} = 17$ $RQD = 115 - 3.3J_{v(a)} = 58.9$ $J_{v4} = 17$ $J_{v5} = 17$	58.9	13
3	Spacing of discontinuities	$(10360)/17 = 609.4\text{mm}$	15
4	Condition of Discontinuities		
	(i) Discontinuity Length	6.0m	2
	(ii) Aperture	0.1mm-0.5mm	5
	(iii) Roughness	Rough	5
	(iv) Infilling thickness	clay	2
	(v) Degree of Weathering	Fresh-slightly weathered	5
5	Ground Water Condition	Dry	15
6	Orientation of discontinuities	Tunnel case	-5
	Total RMR _{basic}	69 (II, Good Rock)	

Rock Parameters for RMR _{Basic}			
RMR Data Point= OC-BH-09-1		Photo No.= 11	Elevation: 1808m Easting: 0368191 Northing: 0911241
No.	Parameters	Value	Rating
1	Point Load Lump Strength (I_L)= UCS (q_c)= $15I_L$ = SHV= $R_a = (220)/9 = 24.44$ $\gamma = 2.79$ $\text{Log}_{10}(\sigma_c) = 0.00088\gamma R + 1.01 = 61.015$	$\sigma_c = 61.015$	7
2	Rock Quality Designation (RQD) in % $J_{v1} = 6$ $J_{v(a)} = \Sigma (1 / (J_{v1} + J_{v2} \dots + J_{v5}))$ $J_{v2} = 6$ $J_{v(a)} = 6$ $J_{v3} = 6$ RQD = $115 - 3.3J_{v(a)} = 95.2$ $J_{v4} = 6$ $J_{v5} = 6$	95.2	20
3	Spacing of discontinuities	$(5550) / 6 = 925\text{mm}$	15
4	Condition of Discontinuities		
	(i) Discontinuity Length	14.0m	1
	(ii) Aperture	1-5mm	1
	(iii) Roughness	Slickenside	0
	(iv) Infilling thickness	Soft infilling	2
	(v) Degree of Weathering	Highly weathered	1
5	Ground Water Condition	Dry	15
6	Orientation of discontinuities	Tunnel case	-5
	Total RMR _{basic}	58 (III, Fair Rock)	
RMR Data Point= OC-BH-09-2		Photo No.= 12	Elevation: 1810m Easting: 0368204 Northing: 0911294
No.	Parameters	Value	Rating
1	Point Load Lump Strength (I_L)= UCS (q_c)= $15I_L$ = SHV= $R_a = (338)/9 = 37.56$ $\gamma = 2.79$ $\text{Log}_{10}(\sigma_c) = 0.00088\gamma R + 1.01 = 93.227$	$\sigma_c = 93.227$	7
2	Rock Quality Designation (RQD) in % $J_{v1} = 5$ $J_{v(a)} = \Sigma (1 / (J_{v1} + J_{v2} \dots + J_{v5}))$ $J_{v2} = 6$ $J_{v(a)} = 11$ $J_{v3} = 6$ RQD = $115 - 3.3J_{v(a)} = 78.7$ $J_{v4} = 6$ $J_{v5} = 6$	78.7	17
3	Spacing of discontinuities	$(5820) / 11 = 529.1\text{mm}$	10
4	Condition of Discontinuities		
	(i) Discontinuity Length	9.0m	2
	(ii) Aperture	<0.1m	5
	(iii) Roughness	Rough	5
	(iv) Infilling thickness	clay	2
	(v) Degree of Weathering	Fresh	6
5	Ground Water Condition	Dry	15
6	Orientation of discontinuities	Tunnel case	-5
	Total RMR _{basic}	64 (II, Good Rock)	

Table: RMR Data through Dam Section

Rock Parameters for RMR_{Basic}			
RMR Data Point= OC-BH-11 (Spillway section)		Photo No.= 06	Elevation: 1874m Easting: 0367338 Northing: 0912593
No.	Parameters	Value	Rating
1	Point Load Lump Strength (I_L)= UCS (q_c)= $15I_L$ = SHV= $R_a=(95)/6= 15.83$ $\gamma= 2.095$ $\text{Log}_{10}(\sigma_c) = 0.00088\gamma R + 1.01= 30.194$	$\sigma_c=30.194$	4
2	Rock Quality Designation (RQD) in % $J_{v1}=13$ $J_{v(a)}= \sum (1/(J_{v1}+ J_{v2} \dots + J_{v5}))$ $J_{v2}=9$ $J_{v(a)}=22$ $J_{v3}=$ $\text{RQD}=115-3.3J_{v(a)}=42.4$ $J_{v4}=$ $J_{v5}=$	35.8	8
3	Spacing of discontinuities	$(\frac{5260}{20})=263\text{mm}$	10
4	Condition of Discontinuities		
	(i) Discontinuity Length	8.0m	2
	(ii) Aperture	0.1-0.5mm	5
	(iii) Roughness	Rough	5
	(iv) Infilling thickness	Weathered Material	2
	(v) Degree of Weathering	Highly weathered	1
5	Ground Water Condition	Dry	15
6	Orientation of discontinuities	Foundation case	-7
	Total RMR _{basic}	45 (III, Fair Rock)	
RMR Data Point= OC-BH-04-1		Photo No.= 08	Elevation: 1687m Easting: 0367814 Northing: 0911792
No.	Parameters	Value	Rating
1	Point Load Lump Strength (I_L)= UCS (q_c)= $15I_L$ = SHV= $R_a=(516)/8= 64.5$ $\gamma= 2.79$ $\text{Log}_{10}(\sigma_c) = 0.00088\gamma R + 1.01= 159.370$	$\sigma_c=159.370$	12
2	Rock Quality Designation (RQD) in % $J_{v1}=12$ $J_{v(a)}= \sum (1/(J_{v1}+ J_{v2} \dots + J_{v5}))$ $J_{v2}=$ $J_{v(a)}=12$ $J_{v3}=$ $\text{RQD}=115-3.3J_{v(a)}=75.4$ $J_{v4}=$ $J_{v5}=$	75.4	17
3	Spacing of discontinuities	$(\frac{5000}{12})=416.7\text{mm}$	15
4	Condition of Discontinuities		
	(i) Discontinuity Length	10.0m	1
	(ii) Aperture	<0.1mm	5
	(iii) Roughness	Rough	5
	(iv) Infilling thickness	Calcite infilled	2
	(v) Degree of Weathering	Fresh	6
5	Ground Water Condition	Dry	15
6	Orientation of discontinuities	Foundation case	-7
	Total RMR _{basic}	71 (II, Good Rock)	

Rock Parameters for RMR _{Basic}			
RMR Data Point= OC-BH-06-1		Photo No.= 09 Elevation: 1691m Easting: 0367479 Northing: 0912337	
No.	Parameters	Value	Rating
1	Point Load Lump Strength (I_L)= UCS (q_c)= $15I_L$ = SHV= $R_a=(536)/11= 48.73$ $\gamma= 2.79$ $\text{Log}_{10}(\sigma_c) = 0.00088\gamma R + 1.01=$ 119.642	$\sigma_c=119.642$	12
2	Rock Quality Designation (RQD) in % $J_{v1}=13$ $J_{v(a)}=\Sigma (1/(J_{v1}+J_{v2}\dots+J_{v5}))$ $J_{v2}=18$ $J_{v(a)}=31$ $J_{v3}=\text{RQD}=115-3.3J_{v(a)}=12.7$ $J_{v4}=\text{RQD}=115-3.3J_{v(a)}=12.7$ $J_{v5}=\text{RQD}=115-3.3J_{v(a)}=12.7$	12.7	3
3	Spacing of discontinuities	$(11070)/21=527.1\text{mm}$	10
4	Condition of Discontinuities		
	(i) Discontinuity Length	12.0m	1
	(ii) Aperture	>0.5mm	5
	(iii) Roughness	Rough	5
	(iv) Infilling thickness	clay	2
	(v) Degree of Weathering	Moderately weathered	3
5	Ground Water Condition	Dry	15
6	Orientation of discontinuities	Foundation case	-7
	Total RMR _{basic}	49 (III, Fair Rock)	
RMR Data Point= OC-BH-07		Photo No.= 10 Elevation: 1811m Easting: 0367515 Northing: 0912311	
No.	Parameters	Value	Rating
1	Point Load Lump Strength (I_L)= UCS (q_c)= $15I_L$ = SHV= $R_a=(122)/8= 15.25$ $\gamma= 2.095$ $\text{Log}_{10}(\sigma_c) = 0.00088\gamma R + 1.01= 29.058$	$\sigma_c=29.058$	4
2	Rock Quality Designation (RQD) in % $J_{v1}=7$ $J_{v(a)}=\Sigma (1/(J_{v1}+J_{v2}\dots+J_{v5}))$ $J_{v2}=4$ $J_{v(a)}=11$ $J_{v3}=\text{RQD}=115-3.3J_{v(a)}=78.7$ $J_{v4}=\text{RQD}=115-3.3J_{v(a)}=78.7$ $J_{v5}=\text{RQD}=115-3.3J_{v(a)}=78.7$	78.7	17
3	Spacing of discontinuities	$(2180)/11=198.2\text{mm}$	8
4	Condition of Discontinuities		
	(i) Discontinuity Length	6.5m	2
	(ii) Aperture	0.1-0.5mm	5
	(iii) Roughness	Rough	5
	(iv) Infilling thickness	Weathered Material	2
	(v) Degree of Weathering	Highly weathered	1
5	Ground Water Condition	Dry	15
6	Orientation of discontinuities	Foundation case	-7
	Total RMR _{basic}	52 (III, Fair Rock)	

Rock Parameters for RMR _{Basic}			
RMR Data Point= OC-BH-02		Photo No.= 13	Elevation: 1768m Easting: 0367874 Northing: 0911521
No.	Parameters	Value	Rating
1	Point Load Lump Strength (I_L)= UCS (q_c)= $15I_L$ = SHV= $R_a = (368)/8 = 46$ $\gamma = 2.79$ $\text{Log}_{10}(\sigma_c) = 0.00088\gamma R + 1.01 = 113.949$	$\sigma_c = 113.949$	12
2	Rock Quality Designation (RQD) in % $J_{v1} = 8$ $J_{v(a)} = \sum (1 / (J_{v1} + J_{v2} \dots + J_{v5}))$ $J_{v2} = 14$ $J_{v(a)} = 22$ $J_{v3} =$ RQD = $115 - 3.3J_{v(a)} = 42.4$ $J_{v4} =$ $J_{v5} =$	42.4	8
3	Spacing of discontinuities	$(15690) = 784.5\text{mm}$ 20	15
4	Condition of Discontinuities		
	(i) Discontinuity Length	9.0m	2
	(ii) Aperture	<0.1m	5
	(iii) Roughness	Rough	5
	(iv) Infilling thickness	Calcite infilling	2
	(v) Degree of Weathering	Moderately weathered	3
5	Ground Water Condition	Dry	15
6	Orientation of discontinuities	Foundation case	-7
	Total RMR _{basic}	60 (III, Fair Rock)	
RMR Data Point= OC-BH-03		Photo No.= 14	Elevation: 1704m Easting: 0367691 Northing: 0911743
No.	Parameters	Value	Rating
1	Point Load Lump Strength (I_L)= UCS (q_c)= $15I_L$ = SHV= $R_a = (282)/9 = 31.33$ $\gamma = 2.79$ $\text{Log}_{10}(\sigma_c) = 0.00088\gamma R + 1.01 = 77.93$	$\sigma_c = 77.93$	7
2	Rock Quality Designation (RQD) in % $J_{v1} = 9$ $J_{v(a)} = \sum (1 / (J_{v1} + J_{v2} \dots + J_{v5}))$ $J_{v2} = 3$ $J_{v(a)} = 12$ $J_{v3} =$ RQD = $115 - 3.3J_{v(a)} = 75.4$ $J_{v4} =$ $J_{v5} =$	75.4	17
3	Spacing of discontinuities	$(12100) = 1008.\text{mm}$ 12	15
4	Condition of Discontinuities		
	(i) Discontinuity Length	11.0m	1
	(ii) Aperture	<0.1m	5
	(iii) Roughness	Rough	5
	(iv) Infilling thickness	Tight weathered material	2
	(v) Degree of Weathering	Highly weathered	1
5	Ground Water Condition	Dry	15
6	Orientation of discontinuities	Foundation case	-7
	Total RMR _{basic}	61 (II, Good Rock)	

Rock Parameters for RMR _{Basic}			
RMR Data Point= OC-BH-04-2		Photo No.= 15 Elevation: 1698m Easting: 0367871 Northing: 0911811	
No.	Parameters	Value	Rating
1	Point Load Lump Strength (I_L)= UCS (q_c)= $15I_L$ = SHV= $R_a = (456)/8 = 57$ $\gamma = 2.79$ $\text{Log}_{10}(\sigma_c) = 0.00088\gamma R + 1.01 = 140.96$	$\sigma_c = 140.96$	12
2	Rock Quality Designation (RQD) in % $J_{v1}=6$ $J_{v(a)} = \Sigma (1 / (J_{v1} + J_{v2} \dots + J_{v5}))$ $J_{v2}=26$ $J_{v(a)}=26$ $J_{v3}=26$ $RQD = 115 - 3.3J_{v(a)} = 95.2$ $J_{v4}=26$ $J_{v5}=26$	95.2	20
3	Spacing of discontinuities	$(\frac{6.2}{6}) = 1.03\text{m}$	15
4	Condition of Discontinuities		
	(i) Discontinuity Length	13.0m	1
	(ii) Aperture	0.1-0.5m	5
	(iii) Roughness	Rough	5
	(iv) Infilling thickness	clay	2
	(v) Degree of Weathering	Fresh	6
5	Ground Water Condition	Dry	15
6	Orientation of discontinuities	Foundation case	-5
	Total RMR _{basic}	74 (II, Good Rock)	
RMR Data Point= OC-BH-06-2		Photo No.= 16 Elevation: 1781m Easting: 0367806 Northing: 0912096	
No.	Parameters	Value	Rating
1	Point Load Lump Strength (I_L)= UCS (q_c)= $15I_L$ = SHV= $R_a = (394)/10 = 39.4$ $\gamma = 2.79$ $\text{Log}_{10}(\sigma_c) = 0.00088\gamma R + 1.01 = 97.74$	$\sigma_c = 97.74$	7
2	Rock Quality Designation (RQD) in % $J_{v1}=10$ $J_{v(a)} = \Sigma (1 / (J_{v1} + J_{v2} \dots + J_{v5}))$ $J_{v2}=11$ $J_{v(a)}=21$ $J_{v3}=21$ $RQD = 115 - 3.3J_{v(a)} = 45.7$ $J_{v4}=21$ $J_{v5}=21$	45.7	8
3	Spacing of discontinuities	$(\frac{7940}{21}) = 378.1\text{mm}$	10
4	Condition of Discontinuities		
	(i) Discontinuity Length	11.0m	1
	(ii) Aperture	0.1-0.5m	5
	(iii) Roughness	Rough	5
	(iv) Infilling thickness	Weathered Material	2
	(v) Degree of Weathering	Slightly weathered	5
5	Ground Water Condition	Dry	15
6	Orientation of discontinuities	Foundation case	-7
	Total RMR _{basic}	51 (III, Fair Rock)	

Appendix C:

Rock Mass In-Situ Tests

Source: Wabe Hydropower Project Geotechnical investigation process prepared by ECDSWCo.
and the author

Table: RMR from drilled rock core

BH-ID	Zoning Depth(M)	Location	Rock Description	UCS (Mpa)	Average RQD (%)	Spac. Disc (mm)	Cond. Disc	GWC	Total RMR	Rock class no.	Description
BH-01	23.70-27.00	Dam Axis (Left Abutment)	Aphanitic Basalt	53.3	36.8	60-200	1-5mm separation (Silckensided surface)	Dry	48	III	Fair
			Rating	7	8	8	10	15			
	61.00-63.60		Moderately to highly weathered Phyric Basalt	33.5	23.1	60-200	Separation <1mm	Dry	50	III	Fair
			Rating	4	3	8	20	15			
	79.70-90.00		Fresh to slightly Weathered Amygdaloidal Basalt	113.4	92	600-2000	Slightly Weathered<1mm	Dry	87	I	Very Good
Rating		12	20	15	25	15					
BH-02	21.70-39.50	Dam Axis (Left Abutment)	Moderate to highly jointed Aphanitic Basalt	79	66.5	200-600	Un-weathered	Damp	63	II	Good
			Rating	7	13	10	30	10			
	41.60-46.60		Moderate to highly jointed Aphanitic Basalt	85.4	69.2	200-600	Un-weathered	Dry	68	II	Good
			Rating	7	13	10	30	15			
	48.80-70.60		Slightly to moderately jointed Aphanitic Basalt	83.8	73.8	600-2000	Un-weathered	Dry	83	I	Very Good
Rating		7	13	25	30	15					

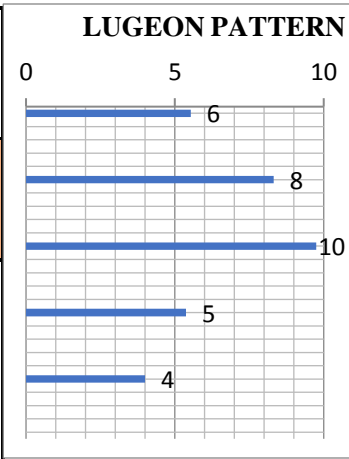
BH-03	12.50-23.50	Dam Axis (Left Abutment)	Slight to moderately jointed, fresh Aphanitic Basalt	123	84.6	600-2000	Slightly weathered<1mm	Dry	84	I	Very Good
			Rating	12	17	15	25	15			
	48.75-54.60		Highly fractured to fragmented Phyric Basalt	70.5	28.2	60-200	1-5mm separation (Slickensides surface)	Wet	40	IV	Poor
			Rating	7	8	8	10	7			
	56.20-70.00		Fresh to slightly jointed Aphanitic Basalt	180	94.2	600-2000	Un-weathered	Dry	92	I	Very Good
			Rating	12	20	15	30	15			
BH-04	22.00-23.50	Dam axis (Dam Seat)	Extremely Weathered Tuff	1-5	17	60-200	Separation>5mm	Damp	22	IV	Poor
			Rating	1	3	8	0	10			
	103.00-106.00		Fresh to slightly weathered Vesicular Basalt	36.8	78.3	200-600	Slightly weathered<1mm	Dry	71	III	Good
			Rating	4	17	10	25	15			
	106.00-110.00		Fractured to highly jointed Aphanitic Basalt	83	55.5	200-600	Un weathered	Wet	67	II	Good
			Rating	7	13	10	30	7			
BH-05	40.00-47.50	Dam Axis (Right Abutment)	Moderately to highly weathered Phyric Basalt Rating	108.60 12	89.9 17	600-200 15	Slightly weathered<1mm 25	Dry 15	84	I	Very Good

BH-05	59.80-62.00	Dam Axis (Right Abutment)	Fresh to Slightly weathered Aphanitic Basalt	2.70	20.1	60-200	Highly Weathered	Dry	47	III	Fair
	Rating		1	3	8	20	15				
	83.50-90.00		Fresh to Slightly weathered Aphanitic Basalt	54	90.9	>2000	Un-weathered	Dry	98	I	Very Good
			Rating	3	20		30	15			
BH-06	9.00-14.30	Dam Axis (Right Abutment)	Moderately to highly weathered Phyric Basalt	18.70	0	<60		Wet	10	V	Very Poor
	Rating		0	3	5	0	7				
	16.70-30.00		Fresh to Slightly weathered Aphanitic Basalt	92.7	41	200-600	Slightly weathered<1mm	Damp	53	III	Fair
	Rating		7	8	10	25	10				
	35.80-54.00		Fresh to Slightly weathered Aphanitic Basalt	88.4	73.1	200-600	Slightly weathered<1mm	Dry	63	II	Good
Rating	7	13	10	25	15						
BH-07	39.00-42.00	Dam axis (Right Abutment)	Fresh to slightly weathered Vesicular Basalt	33.5	59	60-200	Highly Weathered	Damp	55	III	Fair
			Rating	4	13	8	20	10			

BH-07	69.60-80.30	Dam axis (Right Abutment)	Fresh, less jointed Aphanitic Basalt	66.8	84.2	600-2000	Un-weathered	Dry	84	I	Very Good
			Rating	7	17	15	30	15			
	85.70-89.50		Fresh, less jointed Vesicular Basalt	86.3	76.3	200-600	Slightly weathered<1mm	Dry	74	II	Good
			Rating	7	17	10	25	15			
BH-09	13.56-13.90	Intake (Tunnel Alignment)	fresh to moderately weathered Aphanitic Basalt	168.59	74.21	600-2000	Un-weathered surface	Dry	85	I	Very Good
			Rating	12	13	15	30	15			
	47.10-47.40		fresh to moderately weathered Phyrlic Basalt	207.02	91	600-2000	Un-weathered surface	Dry	92	I	Very Good
			Rating	12	20	15	30	15			
BH-13	135.65-135.85	Surge Shaft (Tunnel Alignment)	fresh to moderately weathered Aphanitic Basalt	155.59	68.41	600-2000	Un-weathered surface	Damp	80	II	Good
			Rating	12	13	15	30	10			
	141.15-141.34		fresh to moderately weathered Aphanitic Basalt	100.52	89	600-2000	Slightly weathered<1mm	Wet	76	II	Good
			Rating	12	17	15	25	7			

In-Situ Permeability Test										
Borehole Id		BH-4			Testing Method			wireline packer		
Borehole Location		Dam axis			Borehole Diameter (mm)			46mm		
Easting (m)		367810.53			Ground Water Level from surface (m)			3.4m		
Northing (m)		911789.713			Tested by					
Top Elevation (a.s.l.)		1683.112			Supervised by					
Test Section (m)		17-22			Testing Date					
Test Section length (m)		5								
Testing time			Water Meter Reading (L)		Rate (L/min)	Testng Pressures (bar)			Lugeon Value	
Start (HH:MM)	Finish (HH:MM)	Elapsd time (min)	Start	Finish		Applied Gauge Pressure	Column pressure	Total Pressure		
Elapsed Time			Water meter Reading			Testing Pressure			Average lugeon value	
Start (min)	finish (min)	Elapsed Time (min)	Start	finish	Rate (lit/min)	Applied Gauge pressure	Column Pressure	Total Pressure (bar)		
3:00	3:02	0:02	18353.8	18371.4	8.8	0.8	1.95	2.75	6	$Lu = (10*Q)/(P*L)$ LU = Lugeon Value Q = average water intake in L/min P = Total pressure in bar L = Length of test sectionin meter
3:02	3:04	0:02	18371.4	18384.9	6.8					
3:04	3:06	0:02	18384.9	18396.8	5.9					
3:06	3:08	0:02	18396.8	18412.6	7.9					
3:08	3:10	0:02	18412.6	18429.9	8.7					
3:10	3:12	0:02	18429.9	18466.7	18.4	1.6	1.95	3.55	8	Lugeon Pattern Graphical form calculated for 10 minutes run
3:12	3:14	0:02	18466.7	18492.6	12.9					
3:14	3:16	0:02	18492.6	18523.2	15.3					
3:16	3:18	0:02	18523.2	18548.4	12.6					
3:18	3:20	0:02	18548.4	18577.5	14.5					
3:20	3:22	0:02	18577.5	18624.9	23.7	2.5	1.95	4.40	10	
3:22	3:24	0:02	18624.9	18664.6	19.8					
3:24	3:26	0:02	18664.6	18704.5	20.0					
3:26	3:28	0:02	18704.5	18746.5	21.0					
3:28	3:30	0:02	18746.5	18791.9	22.7					

Testing time			Water Meter Reading (L)		Rate (L/min)	Testng Pressures (bar)			Lugeon Value
Start (HH:MM)	Finish (HH:MM)	Elapsed time (min)	Start	Finish		Applied Gauge Pressure	Column pressure	Total Pressure	
Elapsed Time			Water meter Reading			Testing Pressure			Average lugeon value
Start (min)	finish (min)	Elapsed Time (min)	Start	finish	Rate (lit/min)	Applied Gauge pressure	Column Pressure	Total Pressure (bar)	
3:30	3:32	0:02	18791.9	18804.8	6.4	1.6	1.95	3.55	5
3:32	3:34	0:02	18804.8	18824.9	10.1				
3:34	3:36	0:02	18824.9	18844.6	9.8				
3:36	3:38	0:02	18844.6	18865.5	10.5				
3:38	3:40	0:02	18865.5	18887.2	10.9				
3:40	3:42	0:02	18887.2	18892.6	2.7	0.8	1.95	2.75	4
3:42	3:44	0:02	18892.6	18896.6	2.0				
3:44	3:46	0:02	18896.6	18914.5	9.0				
3:46	3:48	0:02	18914.5	18925.4	5.5				
3:48	3:50	0:02	18925.4	18935.3	4.9				
Lu= 6.60									



Coordinates and work volumes of seismic survey lines at Dam Site (ECDSWCo., 2019)

No.	Survey line	Coordinates (UTM)				Length of survey line (m)	Depth of investigation (m)	Remark
		Starting		Ending				
		Easting	Northing	Easting	Northing			
1	WSD4-1	367390	912235	367964	910900	1503	100	Downstream
2	WSD4-2	367529	912492	368143	910980	1647	100	Dam axis
3	WSD4-3	367746	912600	368364	911079	1656	100	Upstream
4	WSD4-4	367274	911933	367870	912570	900	100	Right bank
5	WSD4-5	367382	911632	368152	912314	999	100	Right bank
6	WSD4-6	367539	911239	368179	911961	945	100	Left bank
7	WSD4-7	367477	910800	368324	911587	1197	100	Left bank
						8847		

Coordinates and work volumes of seismic survey lines Tunnel section (ECDSWCo., 2019)

No.	Survey line	Coordinates (UTM)				Length of survey line	Depth of investigation	Remark
		Starting		Ending				
		Easting	Northing	Easting	Northing	(m)	(m)	
1	WSLT-1	355261	908854	351542	909700	3969	100	Penstock
2	WSLT-2	358090	908216	356928	908456	1170	100	Penstock
3	WSLT-3	360243	908724	359284	908412	999	100	Tunnel
4	WSLT-4	361928	909199	361562	909082	405	100	Tunnel
5	WSLT-5	364604	910015	364229	909891	405	100	Tunnel
6	WSLT-6	368250	911098	366309	910510	2097	100	Tunnel
Total						9045		

Geological formation classification based on the seismic refraction result Dam section (ECDSWCo., 2019)

Layer	Description	P-wave velocity (m/sec)
A	Soil, Alluvial, Highly or Completely Weathered Basalt	<1500
B	Moderately Weathered and Fractured Basalt, Pyroclastic Rock	1500-3000
C	Slightly Fractured Basalt	3000-5000
D	Fresh Basalt with Minor Fractures	>5000

Generalized P-wave velocities of various rock types Tunnel section (ECDSWCo., 2019)

No	Rock Type	P-wave velocity (m/sec)
1	Residual material, brown shed grey, dense, sandy silt with some gravel material, variegated color, highly to completely weathered and disintegrated to sandy gravel size	<1500
2	Intensively to moderately weathered and fractured basalt and tuff extremely to moderately fractured basalt composed of moderately to slightly weathered, fractured, medium strong	1000-3000
3	Moderately weathered and fractured basalt and, composed of slightly weathered and fractured, strong and dense	2500-4000
4	Slightly weathered and slightly fractured and fragmented to brown, fresh basalt	4000<

