ABSTRACT

The Tekeze River and its tributaries have been selected by Ethiopian Sugar Corporation as one of the key irrigation development area for sugar cane crops cultivation. As such, Zarima Dam is constructing across Zarima River in lower Tekaize River Basin to irrigate about 40,000 hacters land for Woilkaite Sugar Development Project in western Tigray Region of northern Ethiopia. The irrigation head works comprise; 153 meters high central asphalt core rockfill dam, diversion conduit, central clay core rockfill coffer dam, spillway, irrigation intake tunnel and dam mid-level outlet tunnel of 600 meters in length.

The mid-level outlet tunnel is designed to be excavated along left side slope of Zarima dam abutment, and the principal aim of providing this tunnel is to assure the possibility to accelerate the reservoir impounding program with control of the overtopping during dam construction works. The tunnel has an overburden rock of thickness reaching from 12 m to 105 m. The geology through which the tunnel is excavated is composed of Meta volcanic rock of low grade metamorphic unit, affected with variable degree of rock masses weathering.

Engineering geological studies along the tunnel alignment is the main scope of the present research work. This includes geological mapping at the scale of 1:5000, geostructural surveying, rock mass classifications, and determination of rock properties. From the results of the engineering geological studies, it is found that the full length of the tunnel will be excavated through Meta-volcanic rocks of low grade metamorphic unit. Based on the engineering properties of the rocks which are directly related to the degree of rock mass weathering, nature of discontinuities and intact rock strength, three geotechnical units (Gtu.1, Gtu.2 and Gtu.3) have been identified along the tunnel alignment.

The results of the rock mass classifications conducted in the study area suggests five types of underground excavation rock supports such as; support type A, B, C, D, and E. These rock support design is slightly more on the safe side if compared with the one suggested with the Q system for selecting temporary rock supports.

Key Words: Detailed geological mapping, geo-structural survey, rock mass classifications and laboratory rock property tests

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Table of Contents

List of Figures

List of Plates

List of Tables

List of Appendices

Chapter 1 Introduction

1.1. General

Tunnels are underground structures which are recognized as a means of attaining convenient transport through conditions posing natural difficulty or special hazards. Surmounting such natural obstacles as mountainous terrain, rivers and seas by tunnels allows safe and convenient transport at all times irrespective of weather conditions. According to Whittaker and Frith (1990), tunnels are a major part of everyday life for the populations of most developed countries, and their service and general range of application are broadly categories as traffic tunnels, water conveyance tunnels and mining tunnels.

In tunnels, geological conditions which may result in decreased competence of the rocks surrounding the excavation often result in increasing tunneling costs in addition to affecting operational and safety aspects. Consequently, in depth appreciation of the geological conditions plays an important role from design and planning, through to construction and eventual commissioning and operation of the tunnels.

Many tunneling problems are caused by unexpected changes in the strength or deformability of the rock masses where it is being excavated. When such a mass is disturbed, it undergoes a re-distribution of stresses, accompanied by a deformation of rock mass is very often. These changes can be either inconsequential or catastrophic, depending on the distribution of stresses in the rock, its strength, deformability etc. Early assessment of such changes in tunneling projects can be of great importance in identifying potential unstable zones, and also in devising appropriate remedial measures (Kolymbas, 2005).

In Ethiopia, tunnels are continuously constructed for the water conveyance systems of hydroelectric projects. There are more than 7 hydropower projects with rock tunnels of totaling 83 km long have been completed to date. The largest power tunnel length is 26 km and maximum tunnel diameter is 14 meters. Table 1.1 gives general information about well-known tunnels constructed in Ethiopia in the past 15 years.

Table 1.1: Major rock tunnels constructed in Ethiopia from the year 2002 to 2014

Tunneling Projects	Total lengths	Geologic Environments	Construction methods
1. Gibe1	12.5 km	Volcanic Rocks	Drill and Blast
2. Gibe2	26 km	Volcanic Rocks	TBM
3. Beles	18.5 km	Volcanic Rocks	TBM
4. Gibe 3	3.8 km	Volcanic Rocks	Drill and Blast
5. Tekeze	2.7 km	Metamorphic Rocks	Drill and Blast
6. Genale Dawa-3	17 km	Metamorphic Rocks	TBM
7. NESHI (Fincha)	2.3 km	Sedimentary Rocks	Drill and Blast

Tunneling collapses occurred in Gibe1, Gibe 2 and Gib 3 hydroelectric projects are particular geological problems which have been recorded in underground construction industry of Ethiopia. According to ENEL and ELC (2004) and Samuel Kinde and Samison Engida (2010), these tunneling problems had been resulted due to the unforeseen geological conditions encountered during the construction works, which significantly affected the project's progress for a year and increased the construction cost unexpectedly.

The Zarima dam project is constructed across the Zarima River that will develop 40,000 ha land for sugar cane cultivation. The project head work comprises rock fill dam of 153 m height, spillway, diversion conduit and middle level outlet tunnel of 600 m in length (SG and SC, 2014). Thus, middle level outlet tunnel has been selected for the present research work in order to conduct an engineering geological appraisal. As such, detailed site geological investigations, geotechnical evaluations and design aspects related to were carried out in the present study. In addition, suggestions have been forwarded for the appropriate tunneling method and underground excavation supports.

1.2. Problem Statement

The geologic and geotechnical constraints which had been encountered during the construction of GIBE 1/2/3 tunnels are the challenges of the construction industry of Ethiopia in the past 15 years. The major geotechnical problems of these underground projects occurred during construction works, which indicated that a proper attention should be given for a special site investigations and geotechnical design of water conveyance tunnels for hydropower and irrigation projects.

Tunnels of Hydropower and Irrigation projects involve high risks and are very expensive. In the absence of a knowledge about the geology and rock mass behavior, the underground excavation design would necessary be conservative. A tunnel design with a good understanding of the rock stresses and deformation behavior can result in potentially large cost saving in the future construction works. Analysis of stability is a principal geotechnical task for underground excavation of tunnels.

Factors influencing the stability of underground excavations in rock are stratigraphy, geological structures, ground water conditions, strength of the rock masses, geometry, excavation methods, and type of support systems and method of support installations (EM 1110-2-2901, 1997). In shallow tunnels, the tunnel excavation stability is usually controlled by the presence and orientation of discontinuity in the rock mass and the presence of water under pressure in these discontinuities. Failures most frequently occur as a result of sliding or separation along discontinuities (Hoek et al., 1995)

This research is focused on the engineering geological appraisal for the 600 m long mid-level outlet tunnel of Zarima Dam situated in Tigray Region of Northern Ethiopia, which is proved to be constructed in low grade Metamorphic Units of Meta-Volcanic Rocks. Thus, the research work has investigated the potential geological features which could affect the construction of tunnels in the study area such as; faults, foliations, rock fractures and weathered shear zones, the engineering characteristics of the rock mass to be met along the tunnel alignment. The study suggests the types of tunnel rock reinforcements with respect to appropriate tunneling methods.

1.3. The Study Area

1.3.1. Location and Accessibility

The present study area is located in the Western Tigray Region of Ethiopia in the lower Tekeze River Basin, along the left bank of Zarima River, at the boundary of Wolkayite and Tselemt Woreda. The UTM coordinates of the project at the river center are Easting 370206 m and Northing 1518890 m.

The Zarema River is one of the major tributaries of the Tekeze River, originating in the northern side of the Semien Mountains. Figure 1.1 shows the location map of the study area. The project site can be reached by about 975 km Addis Ababa – Gonder - Humera main asphalt road and later by the 180 km Humera - Maytemen - Maygaba Asphalt and partially gravel road. The study area is located about 17 km to the east of Maygaba town, the nearest major settlement center in the area, and can be reached by a recently constructed all weather access road. Overall the project site is about 1172 km from Addis Ababa via Gondar and Humera towns. In addition the main asphalt road connecting Addis Ababa – Shire –Humera is the alternative access that can be used to reach to the study area.

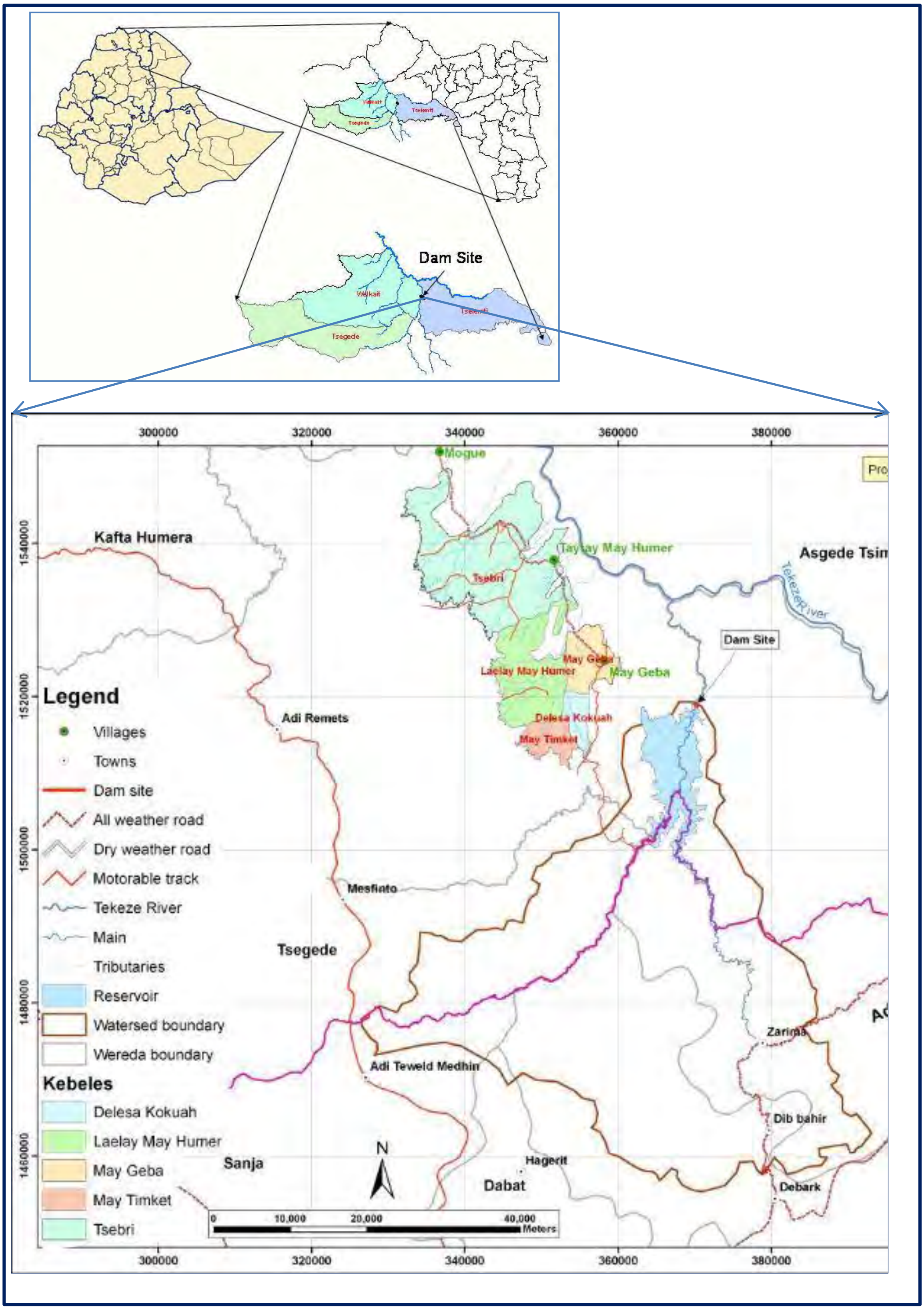


Figure 1.1: Location Map of the Study Area

1.3.2. Hydrologic Characteristics

According to the project design report (SGI and SC, 2013), the Zarima basin consists of two sub-catchment areas; Zarima and Denkuku. The Zarima dam has been strategically located downstream of the river confluence to maximize the available water resources. The overall basin area at the dam site is around 2115 km^2.

The project area exhibits the highest rain fall amount in the region, and totals higher than 2000 mm/yr have been recorded in the past. The Zarima basin, due to the presence of Siemen Mountain range, is frequently exposed to intense and rapid orographic storms, which generate high flash flood along the steep topography (WWDSE and ELC, 2012). The seasonal pattern of the Zarima project area is classified as Mono-Modal, i.e. dominated by single maxima with the wet period clearly marked from June/July to August/September. The dry season extends from October to January under the influence of dry and cool north-easterly winds. A small rainy season with erratic rainfalls can be defined from Feb. to May.

Rain Fall Distribution

The rainfall distribution in the area has been estimated from stations within and around the study area as Zarima, Adi Rametse, Shire-Endesilase and Humera stations. The table1.2 indicates the mean monthly rainfall recorded at the selected stations in the past 12 years.

Table 1.2: Mean monthly rainfalls at selected stations of the study area (from 2000 to 2012 years)

Station	Mean monthly rainfall (mm)											Annual mean
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	
Zarima	5.17	8.50	16.39	40.35	84.96	377.74	445.46	449.89	218.41	63.41	20.39	9.91	1740.58
Humera	0.11	0.00	0.30	2.90	24.40	86.90	172.50	190.30	97.90	20.80	0.50	0.00	596.61
Shire Endeselassie	3.90	1.90	14.90	20.70	35.50	80.80	267.30	241.60	134.70	45.20	17/90	5.40	869.80
Adi Rametse	0.67	0.00	2.14	11.15	57.10	193.74	348.29	387.85	191.97	53.47	1.47	0.06	1247.91

As can be seen from the table 1.2, the Zarima and Adi Rametse areas are having relatively longer precipitation period from June to September. The rainfall distribution during this period varies between 377.74 to 218.41 mm at Zarima Dam Site. This period is contributing about 86% of the annual average rain fall in the study area (Fig 1.2).

This situation indicates that the tunnel portal site is exposed to local landslide problems due to the high intensity of rainfall in combination with the local topography, and highly fractured and weathered rock mass conditions of the study area. In addition to this, rain water seepage is highly suspected in shallow tunnel section which will be excavated through permeable rock masses, and care need to be given for implementing appropriate drainage and slope treatment measures during the construction works.

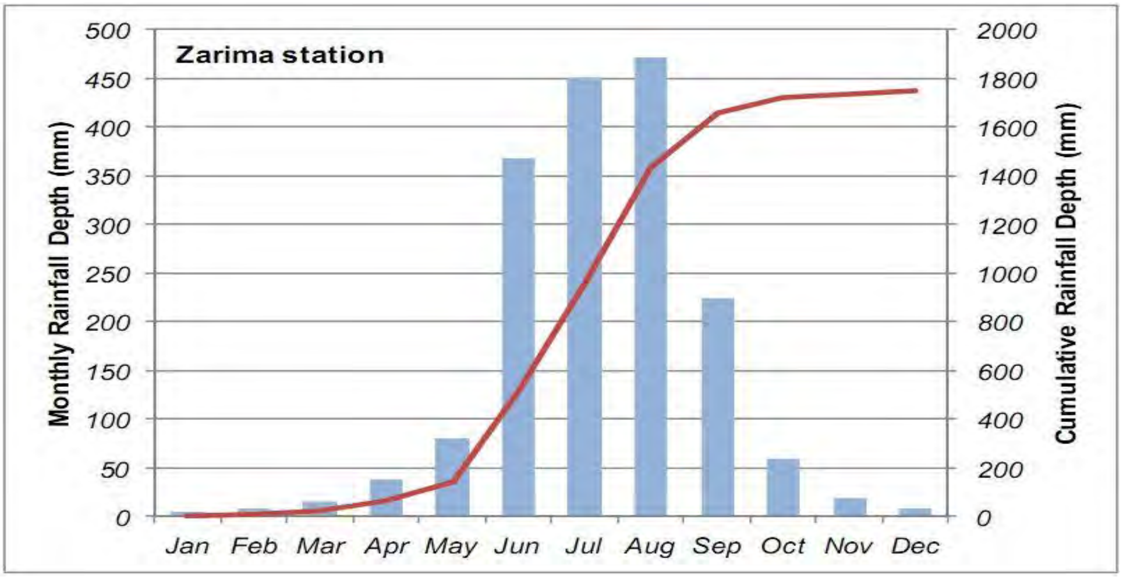


Figure 1.2: Average monthly and annual cumulative rainfall within and arround the study area (from 2000 to 2012 years)

1.3.3. Physiography

The project under study is located on the Zarima River some 5 km upstream from its confluence to Tekeze River. The river alignment exhibits sharp turns, dictated by structural features (faults).

The physiographic characteristics at and near the study area is dominated by a flat plateau, deeply cut by water streams. The elevation of the plateau is around or slightly above 1000 m a.s.l. The plateau is topped by volcanic hills, the remnants of large structures now partly demolished. At Zarima dam site, the valley is some 150 to 200 m deep, below the plateau elevation and about 1200 m wide. The typical geometry of the valley shows a 20 m high cliff all around the edge of the plateau, followed by a nearly 40^0 to 45^0 slope down to the river. The lower portion of the profile is mantled by the accumulation of detritus and of weathering products originated from all local rocks, basalt, sedimentary and metamorphic. In general, the study area is covered by sparse vegetation of semi- arid regions.

The tunnel alignment will go along the valley slopes having a minimum ground surface slope of 45^o. As such, the tunnel may encounter instability by virtue of the nature of the slope ground surface overlying the tunnel excavation particularly at outlet and inlet portal sections. In other situation, the tunnel excavation may introduce changes in the stability condition of the valley slope.

1.4. Description of Zarima Dam Project

Ethiopian Government is currently focusing on development of the country's irrigation potential from medium to large scale schemes. The Tekeze River and its tributaries have been selected by Ethiopian Sugar Corporation as one of the key irrigation development area for sugar cane crops cultivation. As such Zarima Dam is constructing across Zarima River at lower Tekaize River Basin to irrigate about 40,000 ha land for Woilkaite Sugar Development Project in western Tigray Region of Ethiopia. According to the feasibility study report (WWDSE and ELC, 2012), the project irrigation head works comprise; central asphalt core rockfill dam, diversion conduit, central clay core rockfill cofferdam, spillway, irrigation intake tunnel and dam mid- level outlet tunnel. Figure 1.3 shows the general layout of the Zarima Dam Project.

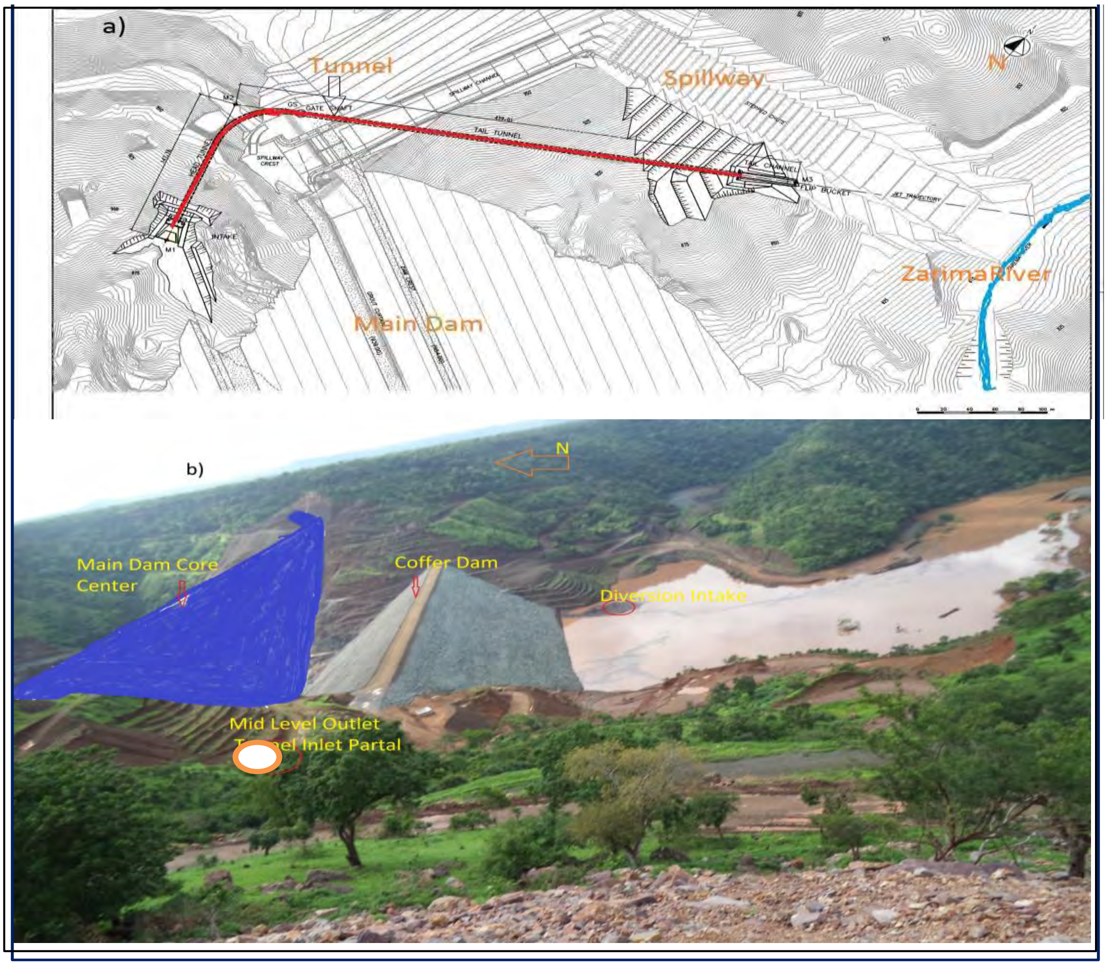


Figure 1.3: General Layout of the Middle Level Outlet Tunnel of Zarima Dam

1) Centeral Asphalt Core Rock Fill Dam;

Zarima Dam, currently under construction, is a 153 m high central asphalt core rock fill dam with crest length of 1100 m and embankment volume of 12 Mm^3 (Fig.1.4). The dam has a trapezoidal shape and the embankment is filled with quarry-run sedimentary rock except for a relative small portion, on both sides of the core, where scalped or processed rock will be used. The bituminous core has 1.2 m thick at the bottom and 0.80 m at the top, which is located some 40 m upstream of the axis of the embankment and will therefore daylight on the upstream slope at el. 939 m, where a berm has been foreseen for possible future maintenance.

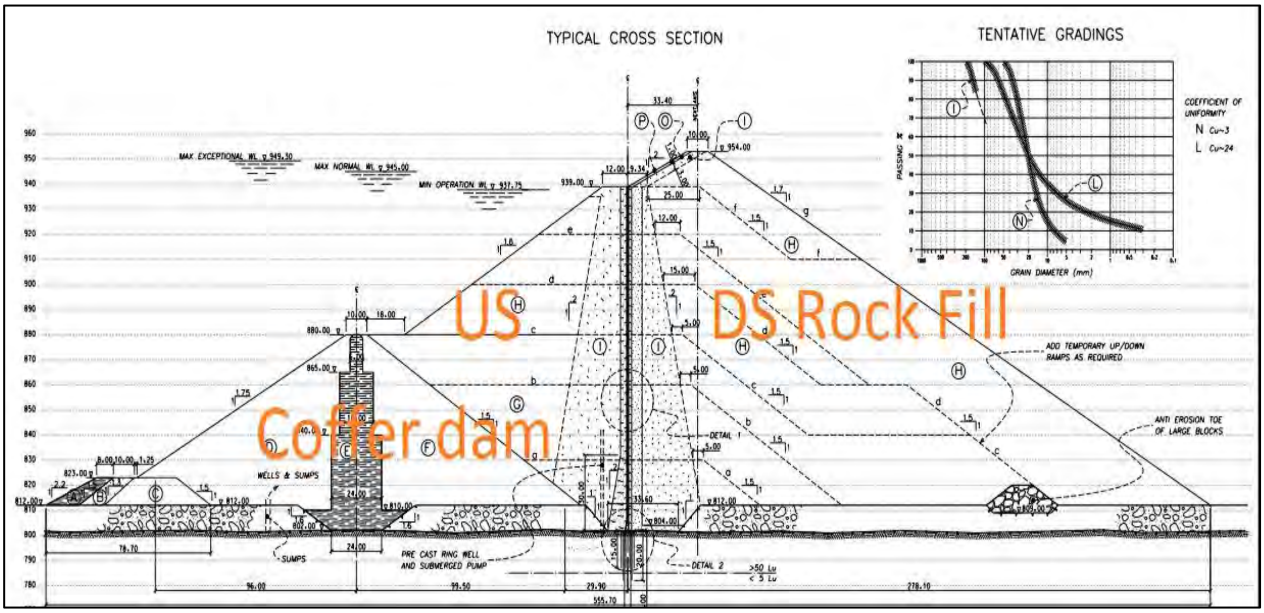


Figure 1.4: Zarima Dam Cross Section (Source SGI and SC, 2014)

2) Mid- Level Outlet Tunnel

The mid-level outlet tunnel alignment is foreseen to be excavated along left side slope of Zarima dam abutment (Fig 1.5). The principal aim of providing this tunnel is to assure the possibility to accelerate the reservoir impounding program with control of the overtopping during dam construction works. In addition, it can also be used to drawdown the reservoir water below the minimum operating level during the reservoir operation, for future maintiance of irrigation intake structure and top polymer face of the dam.

The final design review which has been made by SGI and SC (2014) suggested that the mid-level outlet tunnel will be a circular tunnel, with 3.5 m concrete lining inner diameter, for the first 160 m upstream section from the gates. Downstream from the gates, the tunnel will be a 3.5 m horse shoe, concrete lined, and some 340 m long (Fig.1.5). The inlet and outlet structures are located at elevations of 885 m.a.s.l and 852 m.a.s.l respectively. The mid-level outlet tail will be a steep canal 104 m long, ending in a sky jump. The discharge capacity will be 160 m^3/s at El. 946 m.a.s.l of maximum water level. This geometrical design has been considered for the present research work.

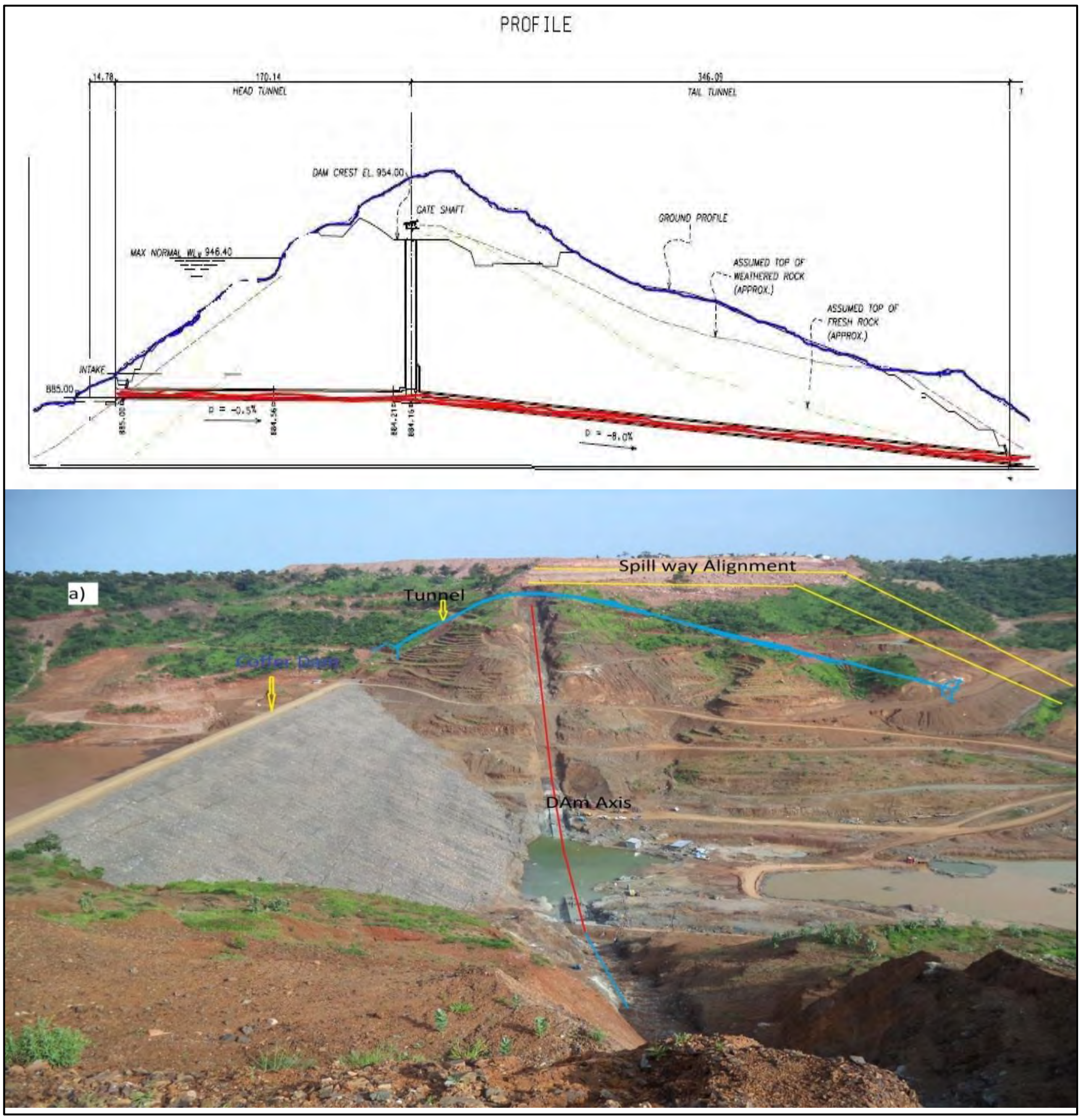


Figure 1. 5: The tunnel alignment veiwing from rigth bank to left dam abutment

1.5. Objectives of the present study

1.5.1. General objectives

The main objective of this research is to study the site geological conditions and geotechnical design considerations for rock tunnel in order to give an engineering geological appraisal for 600 m long middle outlet tunnel of Zarima Dam Project.

1.5.2. Specific objectives

To meet out the aforementioned objectives, the following specific objectives were formulated;

- ✓ Study the geological features, and mechanical properties of rocks to be encountered along the tunnel alignment
- ✓ Identifying the potential geological problems that could affect the construction of tunnels in the study area.
- ✓ Conducting an engineering geological evaluation on the site investigation results and producing relevant geotechnical parameters for underground design purpose.
- ✓ Providing the preliminary geotechnical design of underground excavation support systems.
- ✓ Assessing adverse ground tunneling conditions and extending recommendations for appropriate remedial measures.

1.6. Methodology of the study

To accomplish the research successfully an integrated methodology of literature review, engineering geological mapping, field rock sampling, rock mass characterizations, laboratory rock property testing, geotechnical data analysis, and interpretation of the results have been conducted. In general, the followings methodology has been adopted to achieve the objectives of the present study;

Dusk Study and Literature Review; the literature review was undertaken in order to get a framework on understanding of the geological features and geotechnical conditions associated with such tunneling project. Emphasis has been given to review the previous works on site investigations and engineering geological considerations for designing water conveyance tunnels for hydropower and irrigation projects. Literature sources included maps and reports, books, journals and on-line materials from various websites. An experience of several researchers' works in similar geological environments has been referred in some parts of the study to build up the procedures and methods dealing with engineering geological appraisal of rock tunnels.

Engineering Geological Studies; detailed engineering geological studies were carried out in the present study area. These include geological mapping at a scale of 1:5000, discontinuity characterizations, rock mass classifications and field rock property tests and collecting representative rock samples for laboratory tests and analysis.

Geotechnical Data Interpretation and Presentation; after completion of the field engineering geological studies and laboratory rock property tests, geotechnical data analysis, interpretation and verifications have been carried out using Rock-science software such as Dips, AutoCAD, GIS and Rock Lab.

1.7. Significance of the study

The research may provide information about the site investigation and engineering geological evaluations and design considerations for tunnels in meta-volcanic rocks. The research work may improve the knowledge of site investigation and geological aspects of rock tunneling projects in Ethiopia. The major output of the research is the identification of the potential geological/ geotechnical problems which can affect the construction of tunnels in the study area, provides geotechnical design parameters, suggestions for appropriate underground excavation supports, and devises possible remedial measures for adverse ground tunneling conditions.

1.8. Limitation of the present study

The present research was focused only on one tunnel site and is based on limited number of samples. All efforts are being made to carry out the present study in a very systematic and organized way, well supported by the actual field data, laboratory tests and the secondary data obtained from various sources. However, these efforts were made under the limitations of time, resources and the financial constraints. All laboratory tests were made on the limited number of samples and may contain instrumental and operational errors. Thus, the quality of the results and the findings of the present study may be affected to certain degree of inaccuracy.

1.9. Chapter Schemes

The research project presents systematical detailed geotechnical site investigation, engineering geological evaluation and design consideration for middle outlet tunnel of Zarima dam. The present study comprises following six chapters;

Chapter I covers the introduction of rock tunnels, describes location and accessibility of the study area, problem statements, objectives, methodologies, application of the result and limitations of the research and finally scheme of presentation.

Chapter II presents geological and tectonic settings of the study area, which includes presentation of regional geology, tectonic setup, local geology, geological structures, geomorphological conditions and seismicity of the study area.

Chapter III presents literature review which includes detailed discussions on the influence of geological conditions in the construction of rock tunnels, geotechnical site investigation considerations and tunnel design using empirical approaches.

Chapter IV offers the detailed site investigations, engineering geological evaluations and geotechnical design considerations for rock tunnel in the study area.

Chapter V gives the descriptions of rock failures mechanism in tunneling, determination of rock load pressures, suggestions of underground excavation supports and devises possible remedial measures for adverse ground tunneling conditions in the study area.

Chapter VI is totally deals with conclusion and recommendations that includes the researchers' scientific and logical recommendations.

Chapter 4 Engineering Geology of Mid-Level Outlet Tunnel

4.1. Permeable

To successfully plan, design and construct a tunnel project requires various types of investigative techniques to obtain a broad spectrum of pertinent topographic, geology, hydrogeological and geological structures information and data. Although most of the techniques and procedures are similar to those applied for other civil engineering projects, the specific scope, objectives and focuses of the investigations are considerably different for tunnel and underground projects, and can vary significantly with subsurface conditions and tunneling methods. Records of geological observations and their interpretation play an important rule during all the stages of tunnel project, from preliminary design considerations through design evaluation and final design selection, to assessment and choice of construction method, and finally through to commissioning and operation of the tunnel (Whittaker and Frith, 1990).

In the present study, engineering geological investigations and evaluations have been made for the middle level outlet tunnel of Zarima Dam, since the geological conditions are decisive for the choice of the alignment and the correct construction methods; they greatly affect the cost and safety of construction operations of the tunnel.

4.2. Geotechnical Site Investigations

A geotechnical investigation program for a tunnel project must use appropriate means and methods to obtain necessary characteristics and properties as basis for planning, design and construction of the tunnel and related underground facilities, to identify the potential geotechnical risks, and to establish realistic cost estimate and schedule. In the present study, the geotechnical site investigations for the middle level outlet tunnel of Zarima dam have been carried out with the following methodologies;

1. Existing Information Collection and Study

Before staring detailed geotechnical site investigations, review on previous works was undertaken first in order to understand the geological features and geotechnical conditions associated with such tunneling project. Emphasis has been given to review of pertinent topographic data, boreholes drilling logs, geophysical investigations, geological maps and design geotechnical reports of the Zarima dam project.

The feasibility study which was conducted by WWDSE and ELC (2012), and presented in section 2, has focused on the geology of the dam site with a special emphasis given on the evaluations of the dam foundation and abutment rocks. By this study, the tunnel alignment was not assessed even though there was apple data of boreholes drilling and geophysical investigations with the project. Later, SGI and SC (2013) proposed additional geotechnical investigation like boreholes drilling and laboratory tests to be carried out at the tunnel site. Accordingly, about 4 boreholes with a total depth of more than 200 m were drilled along the tunnel alignment and the geotechnical logs are presented in appendix 2. Besides, laboratory tests like unconfined compressive strength, unit weight, specific gravity and petrography analysis were conducted on representative rock samples and the data are presented in appendix 1.

Following the results of the additional geotechnical investigations, the tunnel design shown in figure 4.1 was produced considering two rock mass classes, namely; GSI < 40 represents fractured rocks and GSI > 40 fair to good quality of rock mass. As such, the temporary underground rock supports given in the Table 4.1 below has been suggested for the proposed tunnel

Table 4.1: Middle level outlet tunnel temporary rock supports according to SGI and SC (2014)

Support Type	GSI	Temporary Rock Supports
Type A1 and C2	GSI <40, Fractured Rocks	Systematic Bolts and Steel Ribs with 60mm thick reinforced shotcrete
Type A2 and C1	GSI > 40, Fair to Good Rock Masses	Local Bolts if required and Unreinforced shoctrete with 60mm thickness

As it is observed from figure 4.1 of the tunnel design given in SGI and SC (2014), the geological features of the tunnel alignment was not assessed as per the requirements of the tunnel excavation stability, and the rock support system has been produced based on the general support guide line of Bieniawski (1989). Thus, detailed engineering geological investigations and evaluations of the rock masses status at tunnel grade have been done along the tunnel alignment in the present study.

2. Geological Mapping and Geostructural Survey

For the present study, detailed field geological mapping has been done along the tunnel alignment with in 200 m corridor at a scale of 1:5,000. In addition, geostructural survey was conducted to identify the pertinent geological structures such as faults, shear zone, foliations and joints which can affect the stability of tunnel excavations in the study area. More than 95 rock joint orientations were collected, and structural analysis has been done by using Rock Science software, and the results obtained are presented in section 4.3 and section 4.5.

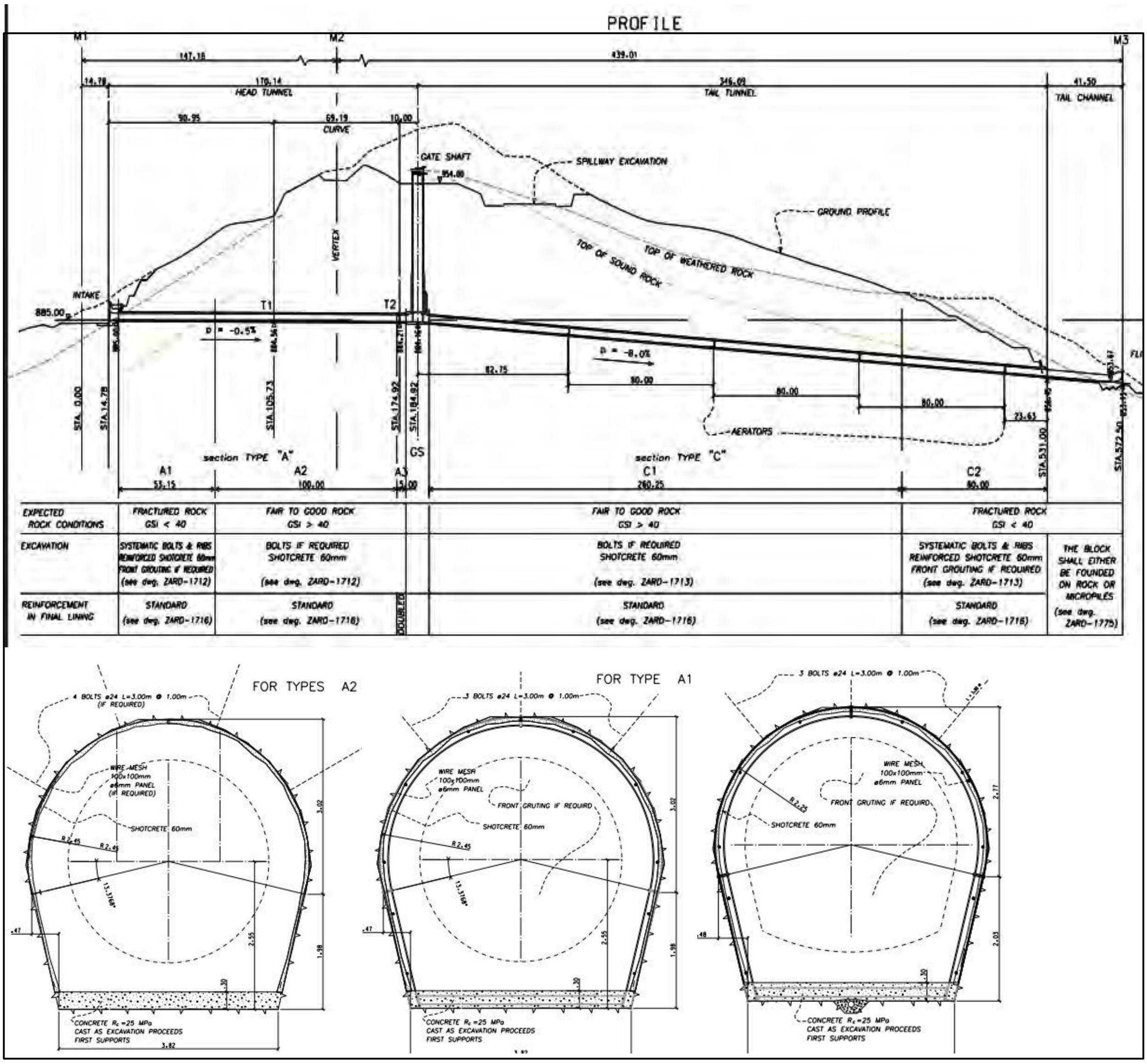


Figure 4.1: Middle level outlet tunnel design according to SGI and SC (2014)

3. Discontinuity Characterizations:

In the present study, discontinuity characterizations were performed on rock exposures at seven stations using Scan line Methods, and the important geotechnical properties have been described according to the standard given in ISRM (1981) and the rock mass conditions have been evaluated with respect to the tunnel grades obtained from the assessment of Rock Mass Weathering, Intact Rock Strength, Spacing of Discontinuity, Conditions of Discontinuity (roughness, alteration, joint infilling, opening and persistence) and Orientation of key discontinuity planes.

By using rock mass characterization systems established in ISRM (1981), in order to understand the engineering properties of the rock masses to be encountered along the tunnel alignment, the following rock mass classifications have been performed;

- ✓ Rock Mass Rating(RMR) from bore holes log and surface rock exposures
- ✓ Geological Strength Index(GSI) from RMR_{89} parameters
- ✓ Tunnel Quality Index(Q-system) from borehole logs and surface rock exposures

All the results obtained from the rock mass classification systems are presented in section 4.5 and the station and description of geostructural surveying are provided in appendix 3.

Table 4. 2a: Classification of Rock Mass Weathering (ISRM, 1981)

Term	Description
Fresh	No visible sign of rock material weathering; perhaps Slight discoloration on major discontinuity surface
Slightly weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering
Moderately weathered	Less than half of the rock material is decomposed or disintegrated to a soil. Fresh or discolored rock is present either as a continuous framework or as core stones
Highly weathered	More than half of the rock material is decomposed or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as core stones
Completely weathered	All rock material is decomposed and or disintegrated to soil. The original mass structure is still largely intact
Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported

4. Rock Property Tests

In order to understand the engineering properties of the rock mass to be encountered along the proposed tunnel route, field rock strength and laboratory rock tests have been conducted in the present study. In addition an effort has been made on evaluations of the existing data of the laboratory tests such as petrography analysis, dry unit weight/specific gravity, water absorption/ porosity, uniaxial compressive strength tests. The field rock strength tests were performed in accordance with the standard given in table 4.1a. All the results obtained from the rock property tests are provided in section 4.4, and raw data of the laboratory tests are given in appendix 1.

Table 4.2b: Descriptions of Rock Strength in Accordance with ISRM (1981)

Term	Description	Unconfined comp. Strength(Mpa)
Extremely weak rock	Indented by thumbnail	0.25 – 1.0
Very weak rock	Crumbles under firm blows with joint of geological hammer, can be peeled by a pocket knife	1.0 – 5.0
Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	5.0 – 25
Medium strong rock	Cannot be scrapped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer to fracture it.	25-50
Strong rock	Specimen requires more than one blow of geological hammer to fracture it	50 – 100
Very strong rock	Specimen requires many blows of geological hammer to	100 – 250

4.3. Tunnel Geology

4.3.1. Pertinent Geological Features

For the present study, a detailed geological mapping has been carried out along the tunnel alignment with in 200 m corridor in scale of 1:5000. Figure 4.2 shows the geological map of middle level outlet tunnel of Zarima dam, and three rock units are observed in the study area namely; Sedimentary Rock, Meta-volcanic Rock and Meta-ultramafic Rock from top to bottom. The detailed geological description of these rock units have been discussed in the previous section (chapter 2). In this study, it has been found that the full length of mid-level outlet tunnel will be excavated through low grade metamorphic rock of chlorite schist, represented as Meta- volcanic Rock, having very poor to good rock mass properties. Based on the nature and degree of rock mass weathering, three geotechnical zones have been identified along the tunnel alignment with respect to the tunnel excavation stability point of view (Plate 4.1 to Plate 4.3)

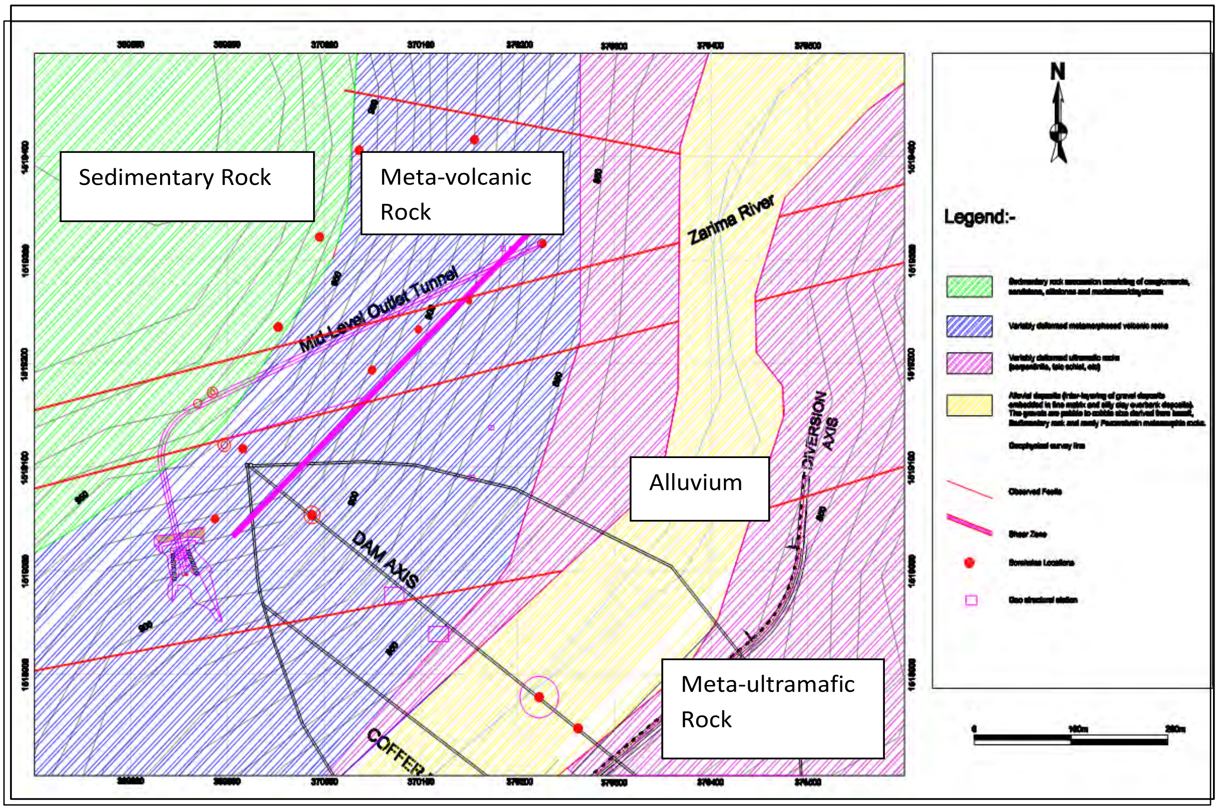


Figure 4.2: Engineering Geological Map of Middle Level Outlet Tunnel of Zarima Dam

Highly Weathered Rocks; the meta volcanic rock in this weak zone are highly weathered and locally decomposed to soil and at some places overlain by colluvial deposit (Plate 4.1). According to the field rock strength test conducted in accordance with ISRM (1981), it is classified as extremely weak to weak rock (Table 4.7). The evidences obtained from the evaluation of geophysical survey (Fig.4.3) and existing boreholes log (appendix 2) reveals that the highly weathered rock just overlain by colluvial deposit covers a significant length of the tunnel section at inlet and outlet portal area (Fig.4.5), and care shall be given for the implementation of appropriate construction method and rock support measures.

Moderately Weathered Rocks; the meta-volcanic rock in this zone is characterized by yellowish grey, fine grain, moderately weathered and closely jointed rock masses (Plate 4.2) with intact rock strength ranging from 20 Mpa to 30 Mpa. The joints spacing measurement (appendix 3) and evaluation of exiting boreholes log (appendix 2) confirm that the average RQD value is reached to 42 % with fair to poor geotechnical properties. As shown in figure 4.5, this rock mass will cover the tunnel excavation for more than 60 m length on both inlet and outlet portal of the tunnel section just next to the highly weathered zone, and care shall be given for the design of appropriate underground excavation supports.

Plate 4.1: Highly weathered and partially fragmented rock masses in the study area

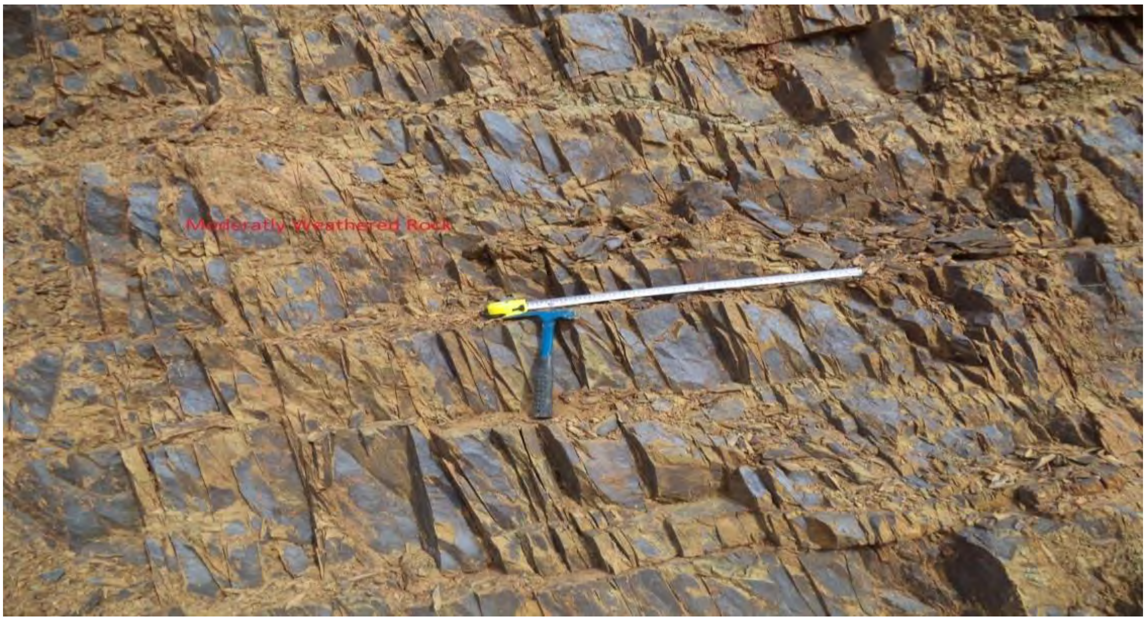


Plate 4.2: Moderately to highly weathered and closely jointed rock masses in the study area

Slightly Weathered to Fresh Rocks; as shown in plate 4.3, this rock mass is characterized by greenish grey to yellowish grey, fine grain, fresh to slightly weathered and closely to widely jointed rock masses with intact rock strength ranging from 35 Mpa to 68 Mpa. The joints spacing measurement and evaluation of existing boreholes log indicate that the average RQD is reached to 65 % with fair to good geomechanical properties. The resistivity model (Fig.4.3) shows that the rock mass will cover the central part of the tunnel section just next to the moderately weathered rocks zone.

The results of geophysical/ resistivity survey as shown in figure 4.3 indicates three geological structures that may cross the tunnel alignment, and there is a wide weakness zones with low apparent resistivity, which is considered that the Meta-volcanic rocks are shattered and weathered widely and deeply at the top part. Besides, the top soil which is usually 5 m to 10 m thick covers the weathered rock masses.

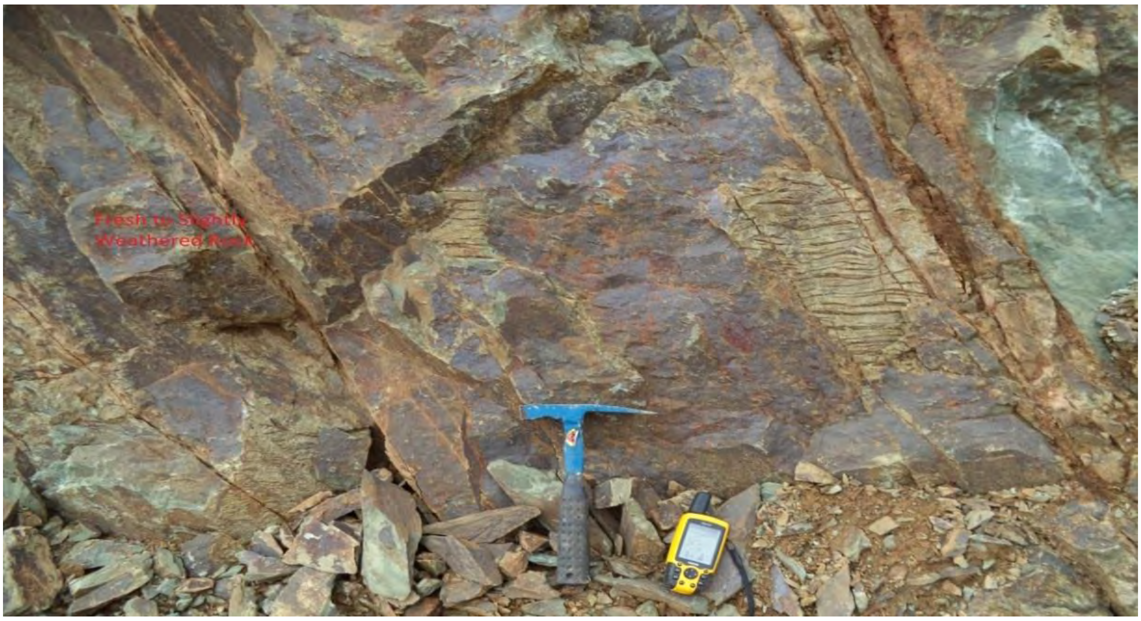


Plate 4.3: Fresh to slightly weathered and widely jointed rock masses in the study area

4.3.2. Faults and Shear Zones

Faults; the general characteristics of a fault are essentially that of the effect of differential stress producing rupture and subsequent displacement along the plane of failure. Various classifications of faults exist but they can be broadly identified under normal faults, reverse faults and strike-slip faults. Normal faults result in relative movement along the plane of failure causing beds to displace laterally from each other, reverse fault causes lateral displacement of beds to overlap each other by virtue of rock movements being thrown over their previously matching beds. Strike slip faults are characterized predominantly horizontal relative dislocation (Davis and Reynolds, 1996).

Wahlstrom (1973) has forwarded the following points on the influence of faults in the design and stability of tunnels;

- Relative movement of the rock masses produces scratches, grooves and polished interfaces. These can indicated movement direction but they are of special significance to tunneling in representing planes of very low friction with the ability to readily encourage detachment and sliding of rock into tunnel excavations.

- Faults and interconnecting structures allow circulation of ground water to penetrate deep below the surface and laterally into side walls. This can produce wall rock alternation and result in deep seated weathering zone with consequent loss of competency of rocks appreciably below the ground surface.
- Breccia filling and fault gouge in association with ground water can cause progressive collapse in to tunnel excavations. They have also minor or insignificant bonding strength and exhibits poor stand up time.

Thus, the orientation of faults in relation to the tunnel lines vitally important since this governs the length of tunnel affected by the fault and its accompanying fault zone.

In the present study area, from the geological map and resistivity profiles (Fig. 4.2 and Fig. 4.3), a total of three geological structures have been identified along the tunnel alignment. These major fracture zones are oriented NNE-SSW (shear zone) to ENE-WSW (Faults) as shown in figure 4.4. Their orientations are fair to unfavorable with respect to the tunnel axis as they cut the tunnel excavation obliquely. As it is observed in the boreholes log presented in appendix 2, minor weakness zone in addition to the ones identified from the geological map should also be expected.

The NNE trending shear zones, which initially underwent ductile type of deformation, were later reactivated to produce more brittle deformation. It is noted that later reactivation of the shear zones coupled with cross-cutting relationships produce shattering of the rocks, which become weaker and susceptible to mechanical and chemical weathering resulting in partial to complete decomposition.

The ENE-WSW trending fault/ fracture set is prominent and display close spacing which varies from few centimeters-where the intensity of deformation is high to several meters. There are also other subsidiary structures striking NW-SE (296°) and dipping at angle of 80° towards east, which are considered as pinnate structures associated with the ENE-WSW striking structures.

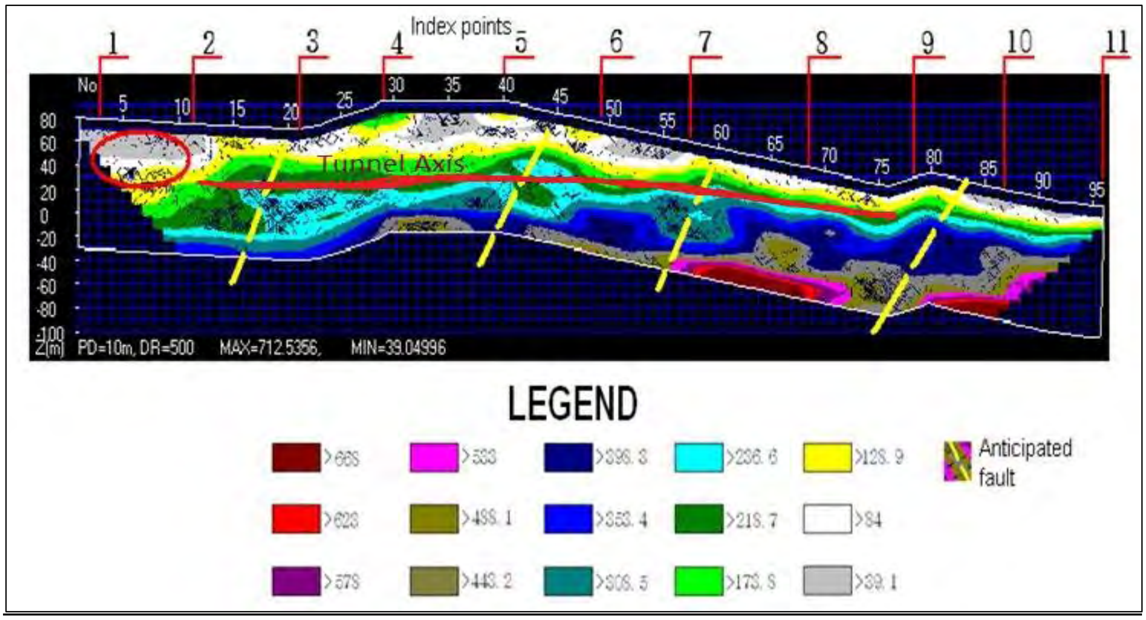


Figure 4.3: Resistivity model along tunnel alignment (Modified after WWDSE, 2011)

4.3.3. Rock Joint Structures

Rock Join Structures; where a rock fracture result in no significant visible displacement at the plane of fracture, then this is commonly referred as a joint. Conversely a fault is a plan of failure in rock which exhibits relative displacement (ISRM, 1981). Joints in rock structure have originated primarily due to the regional tectonic history. Their frequency and orientation are related to the nature of the stress field with tensional and compressive states coupled with folding and faulting playing important roles (Davis& Reynolds, 1996).

According to Whittaker and Frith (1990), joint patterns represent structural weakness in rock masses and can be substantially influence the stand-up time of different rock types. They are likely to influence the mode of rock failure and character of its collapse potential during tunnel excavations. Consequently, the joint patterns require special considerations when giving attention to the choice and application of rock support, particularly for temporary support measures.

In the study area, Meta-volcanic rock like chlorite schist may contain four or more sets of regularly spaced, extensive joints. Typically, there is one set of joints parallel to the original rock structures, one set parallel to the foliation and two or more sets of fracture surfaces in other direction. Table 4.3 shows average orientations of joint structures in the study area, and at some places the meta-volcanic rocks are strongly foliated.

Table 4.3: Preferred orientations of joints as observed on 12 geostructural stations

Joints	Preferred joint planes orientations	
	Dip Amount	Dip Direction
J_1	25^0	195^0
J_2	80^0	343^0
J_3	83^0	095^0

In the present study, discontinuities data was collected from rock exposures found at quarry opening, road cutting, dam key trench and spillway excavation areas, and stereographic analysis has been done using Rock science software (Dips Program) and the results obtained are presented in figure 4.4. As can be seen from the stereographic analysis, the key joint structures found in the tunnel site show cross cutting relationships, and this can affect the stability of the tunnel excavation by inducing rock wedge failures.

4.3.4. Rock Overburden and Ground Water Conditions

a) Rock Overburdens;

The rock overburden conditions of the tunnel alignment will range from 12 m to 105 m as shown in the geological profile (Fig.4.5). At the inlet and outlet portal sections, the overburden is reached from 12 m to 45 m and at central section of the tunnel it will range from 72 m to 105 m. This may result gravity stress induced rock failures as the tunnel will be excavated in shallow depth through jointed rock mass.

b) Ground Water Conditions;

The presence of ground water in very large quantities is recognized as a major hazard in addition to causing operational difficulties in respect of tunnel construction works. Zaruba & Mencl (1976) states that inflow of water into the tunnel increases construction costs by about 20 %; as water proofing of the lining and often also drainage behind the lining are necessary. Encountering large quantities of water in weak ground conditions can also lead to rapid formation of cavities around the tunnel excavation. This can produce the potential for significant quantities of wet and loose ground to flow into the tunnel excavation.

Whittaker and Frith (1990) point outs that long term stability problems can also arise due to the high gravity loading of tunnel supports by fractures and weathered rock materials above the tunnel crown. Potential problems from ground water inflow during tunneling can be predicted to a large extent by a comprehensive site investigation from employing deep boreholes and probe drilling. The rock types representing potential aquifers with in the rock sequence through which the tunnel is to be driven can be generally identified and appropriate provisions made to either control or deal with the water inflow problem.

In the present study area, the boreholes drilled along the tunnel alignment confirm that the ground water level is 70 m and 37 m below the inlet and outlet tunnel inverts respectively, and the full length of the tunnel will be excavated in dry ground water conditions. However, significant amount of rain water seepage is highly suspected during rainy season, as the tunnel will pass through highly fractured/ weathered rock mass. Particularly at inlet and outlet portals for about 128 m tunnel lengths with overburden thickness less than 40 m, the water may reduce the shear strength of the rock mass and increase gravity loadings on the underground excavation supports.

The water pressure tests reported in SGI and SC (2014) indicated that the in-situ permeability of meta-volcanic rocks is reached to maximum of 51 Lu ($6.15 * 10^{-7}$ m/s) for the top 41 m depth of over burden thickness, and the rock mass permeability decreases as the depth increases.

According to Hagen (2012), the ground water inflow in to the tunnel excavation in jointed rock masses can be predicted by using the following equation of Darcy law;

$$Q = \frac{2\pi K L \Delta h}{2.3 log(\frac{2\Delta h}{r})}$$

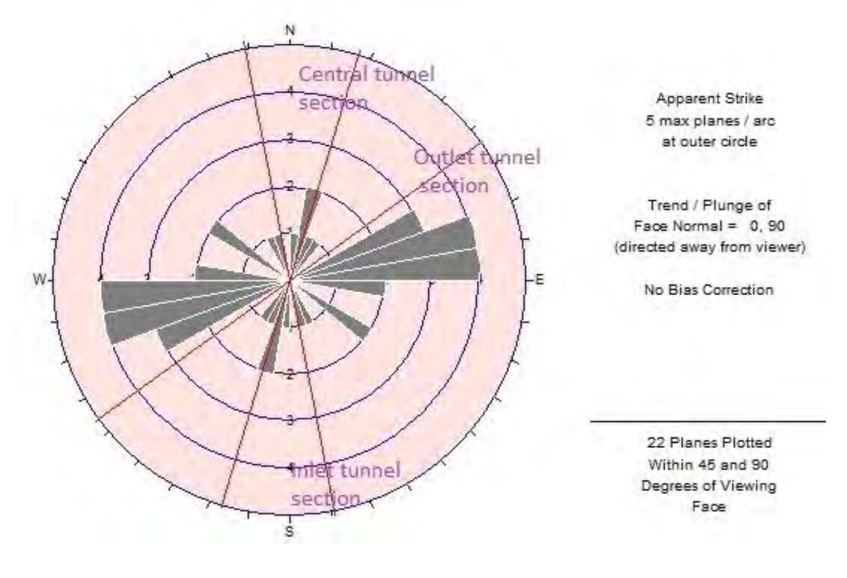
Where; ***Q*** is the inflow/ leakage rate in *m^3/s*, ***K*** is the hydraulic conductivity in vertical pane of flow in *m/s*, ***L*** is the tunnel length in meter under considerations, ***h*** is the hydraulic head in *m* and ***r*** equals the radius of a tunnel in *m.*

Thus, for mid-level outlet tunnel of Zarima dam, considering $K = 6.15 * 10^{-7}$m/s, L = 128 m, h = 41 m and r = 2.5 m, the rain water seepage in heavy rainy season can be estimated as;

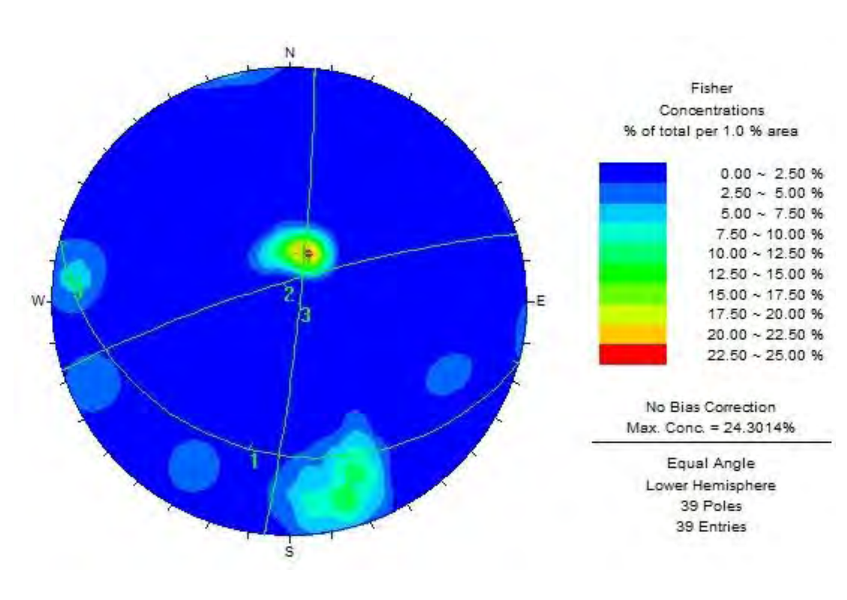
$$Q = \frac{2\pi K L \Delta h}{2.3 log(\frac{2\Delta h}{r})} = \frac{2*3.14*(6.15*10^{-7})*128*41}{2.3\log(2*\frac{41}{2,5})} = 5.79 * 10^{-3} m^3/s$$

$$Q = 5.79 * 10^{-3} m^3/s \quad = 5.79\ L/s$$

Accordingly, a maximum of about *5.79 L/s* rain water seepages may affect both the tunnel outlet and inlet portals excavation during heavy rainy season.

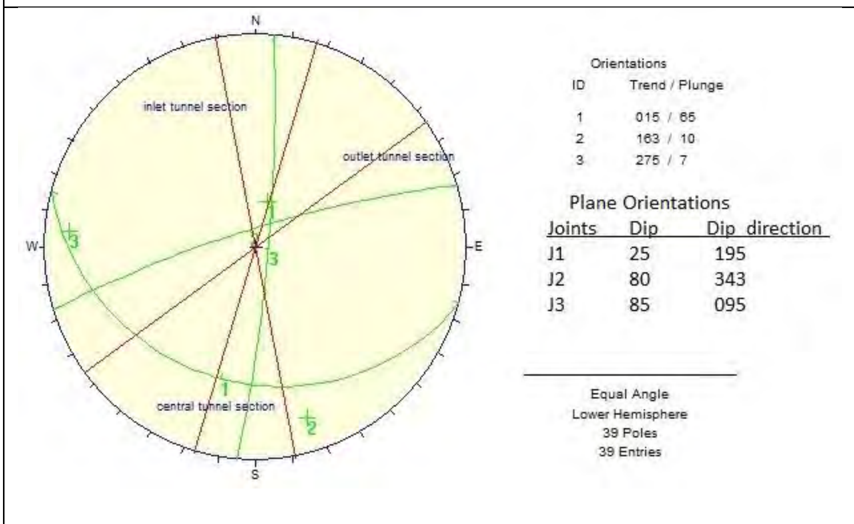


a) Joints rosettes; the strikes of key joints are parallel to oblique with respect to the tunnel axis. Their orientations are fair to unfavorable as they cut the tunnel excavation at high angle.



b) Joints Pole; the preferred orientation of the pole of key joint structures;

ID	Trend	Plunge
J1	015^0	65^0
J2	164^0	10^0
J3	275^0	7^0



c) Joints plane orientation; the joints show cross cutting relationships. They are likely to influence the mode of rock failure and character of its collapse potential during tunnel excavation.

Figure 4.4: Stereographic analysis of Joint orientations using Dips Program; The tunnel alignment is indicated by the red solid line.

4.4. Rock Mass Weathering and Intact Rock properties

4.4.1. Rock Mass Weathering

The natural processes of weathering produce rock alteration which can be of major importance to tunneling operations. Weathering reduces the strength of rock masses and can be extend to considerable depth due to the action of ground water circulation. Climate and topographical features in addition to rock type and structure influence the depth of weathering. Pockets of highly weathered rocks may contain water under appreciable head and can possess the ability to rapidly flow in to an excavation if disturbed by underlying or adjacent tunnel activities.

In the study area, as it is observed from the boreholes log and rock exposures presented in appendix 3, the rocks exposed along the tunnel alignment are highly affected by different degree of rock mass weathering. Along the tunnel route three rock mass weathering zones are observed with such conditions; the meta-volcanic rock, which is found at the inlet and outlet area, is highly weathered. Whereas, in the other part of the tunnel section the rock mass is slightly to moderately weathered, and fresh at the central section of the tunnel.

Table 4.4: Relationships between compressive strength and rock mass weathering

Sample Source	Depth Interval(m)	RQD (%)	Unit Weight (gm/cm^3)	Compressive Strength (Mpa)	Geological Descriptions
JVO.01	16.80 - 17.02	20	2.44	20.2	Dark grey moderately weathered and fractured Meta-volcanic rock
	20.55 - 20.70	25	2.46	40.3	Closely jointed, slightly weathered to fresh meta-volcanic rock
	32.95 - 33.23	25	2.52	46.5	
	43.40 - 43.70	74	2.51	29.60	Widely jointed moderately weathered Meta- volcanic rocks
JVO.02	28.84 - 29.09	12	2.50	13.8	Moderately to highly weathered and fractured meta-volcanic rock
	36.05 – 36.33	35	2.68	68.2	Closely to widely jointed, slightly weathered to fresh meta-volcanic rock
JVO.03	29.32 – 29.56	76	2.71	31.7	Closely to widely jointed, slightly weathered meta-volcanic rock
	38.50 – 38.80	87	2.54	37.2	Widely jointed slightly weathered to fresh meta volcanic rock
JVO.04	29.85 – 30.05	10	2.33	12.6	Highly weathered fractured and brecciated meta-volcanic rock
Note: JVO.01, JVO. 02, JVO.03 and JVO.04 are boreholes drilled along the tunnel alignment					

Table 4.4 and Table 4.6 show the relationships between RQD, the compressive strength and rock mass weathering as evaluated from rock exposures and boreholes log. In the study area, weathering has three significant effects on the engineering properties of the rock mass, namely; reduction of intact rock strength, increasing rock mass porosity and changes the intrinsic rock fractures like foliation to mechanical joints. These are the dominant factors in affecting the quality of rock masses along the proposed tunnel alignment. Thus, the effect of rock mass weathering on meta-volcanic rocks of the study area has been assessed in accordance with the standard in Table 4.1a by ISRM (1981), and incorporating with relevant rock mass properties to determine the geotechnical parameters as described in details in this research work.

4.4.2. Intact Rock Properties

Mechanical and physical properties of the intact rocks were evaluated from laboratory tests performed on several samples obtained from the boreholes and field measurements (Table 4.4). The specific gravity, porosity, water absorption, water content and uniaxial compressive strength were determined from these tests (Table 4.5). Accordingly, the uniaxial compressive strength of the rocks are ranging from 13 Mpa to 68 Mpa, and the rock classifications according to intact rock strength is weak to strong rock as given in table 4.7. In addition, petrographic analysis was done on selected rock samples and the results reveal that the Meta-Volcanic Rock in the study area is compositionally Calcite-Epidote-Chlorite Schist (Table 4.5).

Table 4.5: Laboratory tests analysis of intact rock properties of Meta-volcanic Rocks

Intact Rock properties	Porosity (%)			Unit Weight(gm/cm^3)			Compressive Strength (Mpa)		
	Min.	Max.	Av.	Min.	Max.	Av.	Min.	Max.	Av.
Highly Weathered Meta-Volcanic Rock	15.08	17.89	**16.47**	2.33	2.50	**2.42**	12.6	13.8	**13.2**
Moderately Weathered Meta-Volcanic rock	5.85	6.69	**6.27**	2.44	2.51	**2.48**	20.2	29.60	**24.90**
Slightly weathered to fresh Meta-Volcanic Rock	2.43	2.51	**2.47**	2.46	2.71	**2.58**	31.7	68.2	**44.78**

Petrographic analysis of the rock samples according to WWDSE and ELC (2012)

Main Rock Units	Samples Description	Rock Name
1. Meta-Volcanic Rock	Greenish grey, fine grained	Calcite-Epidote-Chlorite-Schist
2. Meta-Volcanic Rock	Greenish grey, fine grained	Calcite- Epidote- Chlorite-Sschist
3. Highly Weathered Meta-Volcanic Rock	Greenish grey and fine grained	Quartz-Plagioclase-Chlorite-Schist

4.5. Tunnel Geotechnical Units

4.5.1. Geotechnical Units along tunnel alignment

From the results of the geotechnical site investigations carried out in the present study, a tunnel geotechnical modeling has been done to understand the engineering properties of the rock masses encounter along middle level outlet tunnel. As such three major tunnel geotechnical units (Gtu1, Gtu2 and Gtu3) have been identified in the study area. These tunnel units are basically classified based on their engineering properties directly related to the degree of rock mass weathering, nature of discontinuities and intact rock strength, through which the important geotechnical parameters for the in-situ rock masses could be understood with respect to the stability of tunnel excavations. Accordingly the following geotechnical zones can be possibly foreseen for along the mid-level outlet tunnel of Zarima dam project;

Geotechnical Unit-1 (Gtu.1): This tunnel geotechnical unit is characterized by slightly weathered and fresh rock masses with closely to widely spaced joints. The intact rock strength is reached to maximum of 68 Mpa and average RQD value is 65 %. As can be observed from table 4.6 and figure 4.5, this unit will cover 42 % of the tunnel excavation length (253 m).

Geotechnical Unit-2 (Gtu.2): This tunnel unit is characterized by moderately weathered and closely jointed rock masses with maximum intact rock strength are reached to 29 Mpa and average RQD is 42 %. As shown in figure 4.5, it will cover 30 % of the tunnel excavation length.

Geotechnical Unit-3 (Gtu.3): It is characterized by highly weathered and fractured rock masses with intact rock strength is 13 Mpa, and will cover 28 % of the tunnel length (Table 4.6).

Table 4.6: Percentages of tunnel geotechnical units encounter in the study area

Geotechnical Units	Approximate Tunnel Lengths	Percentage	Stations(m) starting from inlet portal
Gtu.1	252m	42%	101 to 353
Gtu.2	179m	30%	51 to 101 353 to 482
Gtu.3	169m	28%	0.00 to 51 482 to 600

Figure 4.5 shows the extent of the tunnel geotechnical units to be encountered along the tunnel alignment and the corresponding geotechnical properties are described in detail in the next section of the report.

Table 4.7: Rock classification based on intact rock strengths (according to ISRM, 1981)

Description	Uniaxial Compressive Strength (Mpa)	Rock classification
1. Highly Weathered Meta-volcanic Rock	10 -13.2	Weak Rock
2. Moderately Weathered Meta-volcanic Rock	25 – 30	Medium Strong Rock
3. Slightly weathered to fresh Meta-volcanic Rock	45 -68	Medium strong to Strong Rock

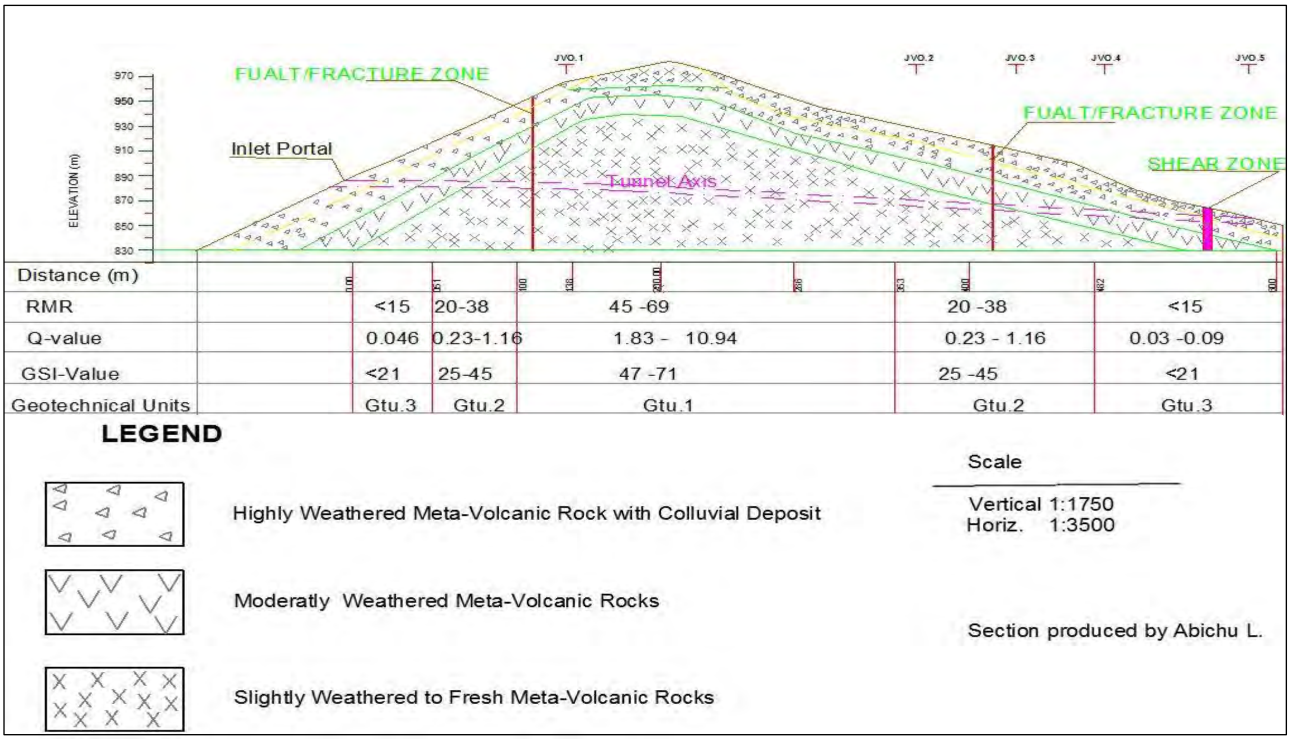


Figure 4.5: Longitudinal geological and geomechanical profile of middle level outlet tunnel

4.5.2. Rock Mass Rating(RMR)

Using rock mass ratings it is possible to estimate support requirement of rock tunnels. The Rock Mass Rating system which was developed by Bieniawski (1989) is based on six parameters, namely;

1. Uniaxial compressive strength of rock material
2. Rock Quality Designation(RQD)
3. Spacing of Discontinuities
4. Conditions of Discontinuities
5. Ground Water Conditions
6. Orientation of Discontinuities

Details of the individual ratings are given in Bieniawski (1989) in a series of tables. The way to apply this system is to divide the rocks in to a number of structural regions in such a way that certain features are more or less uniform with in each region.

For the present study, the average uniaxial compressive strength estimated in table 4.5, RQD values given in table 4.4, joint characteristics and ground water conditions from field measurement/observations (appendix 3) are used as input parameters for the determination of RMR along the tunnel route, and the results obtained are presented in table 4.8. As shown in this table and figure 4.5, for 600 m tunnel length it is estimated that the RMR value will be occurred in the following percentages;

- RMR 44 to 69, fair to good rock masses: 42 %
- RMR 20 to 38, poor rock masses; 30 %
- RMR 00 to 15, very poor rock masses; 28 %

Table 4.8: Summary of RMR values estimated for middle level outlet tunnel

Rock Mass Rating according to Bieniawski (1989) (from drilled core and surface observations)	Tunnel Geotechnical Units				
	Gtu.1		Gtu.2		Gtu.3
	Values	Rate	Values	Rate	Rate
1. Intact Rock Strength (Mpa) from Lab. test	40 – 72	4 – 7	11 -28	2 - 4	1
2. RQD(%) from drilled core and field measurement	25- 82	7 -17	20 -46	3 - 8	3
3. Spacing of discontinuities (surface observations)	6 – 50cm	10 -12	5-10cm	5 - 8	5
4. Conditions of discontinuities(surface observations)	Smooth to rough	15 - 25	joint infilling 1-7mm	5 -10	0
5. Ground Water; dry (however, damp to wet in rainy season as the tunnel is in shallow depth)	Dry	15	Dry	15	12
6. Discontinuity orientations (surface observations); cross cutting relationships of the joints may result unstable underground excavations in the study area	Fair to Unfavorable	-7	Fair to unfavorable	-7	-7
RMR- Results	**44 - 69**		**20 - 38**		**14**
RMR-basic(*for GSI computations*)	**51- 76**		**27 - 45**		**24**
GSI - Values	**46 -71**		**22 - 40**		**19**

4.5.3. Tunnel Quality Index (Q- System)

The Norwegian Geotechnical Institute Index, Q-system, was proposed by Barton etal (1974; as cited in Bieniawski, 1989) and is based on the evaluation of some 200 case histories of tunnel support in Scandinavia. It is probably the most widely used rock mass classification system today. The Q-system takes numerical account of the following parameters;

- Rock Quality Designation: the absolute value is employed; $0 \leq RQD \leq 100$
- Joint Set Number (J_n): this is a measure of the number of joint sets with in the rock mass and has a range of values of 0.5 (representing massive rock with few joints) to 20 (crushed rock)
- Joint Roughness Number (J_r): the range of values is 0.5(smooth, planer, slickensided joints) to 4 (rough, undulating and discontinues joint)
- Joint Alteration Number (J_a):this takes account of infilling and has a range of values from 0.75(no infilling) to 20(thick band of crushed rock in-filled with clay materials)
- Joint Water Reduction (J_w): this takes in to account the presence of water under pressure affecting the shear strength of joints and has the range of values 0.05(high pressure) to 1 (zero pressure)
- Stress Reduction Factor (SRF): this takes account of several factors such as; loosing of the rock mass as a result of presence of shear zone and clay bearing rocks, rock stress problems in competent rock, and loads induced by swelling and squeezing ground conditions. The ranges of value cover from 1 to 15 and depend upon the nature of the problem.

The detailed descriptions of the Q- parameters are presented in Bieniawski (1989) in a series of tables. Using these standard tables, for the present study, Q system has been determined based on the evaluation of existing boreholes log in combinations with field measurements described in appendix 3, and the results obtained are presented in table 4.9. From the tunnel quality index (Q-system) evaluations, it is found that the rock masses encounter along the tunnel route has Q values ranging from lowest 0.03 to highest 10.94. These Q values can be considered for selecting temporary rock support of the tunnel.

Table 4.9: Determination of Q-system for middle level outlet tunnel

Q-Values according to eq. 3.3 given in section 3(from drilled core and surface observations)	Tunnel Geotechnical Units		
	Gtu.1	Gtu.2	Gtu.3
	Q- Rating	Q- Rating	Q-Rating
1. Rock Quality Designation(RQD)	55-82	27-52	10-15
2. Joint set number (J_n)	6- 9	9- 12	12- 20
3. Joint roughness (J_r)	1.5- 2	1.0- 1.5	1
4. Joint alteration (J_a)	1- 2	3- 4	6
5. Joint water reduction(J_w)	1	1	1
6. Stress reduction factor(SRF)	2.5	2.5	2.5
Q-values	**1.83-10.94**	**0.23 – 1.16**	**0.03– 0.09**

4.5.4. Geological Strength Index (GSI)

Proper estimation of the strength and deformability characteristics of the rock mass is essential in the numerical analysis of the tunnels. Since the mechanical properties of the rock mass can be hardly evaluated directly, empirical relations based on the rock mass classification systems are used for this purpose. As such the Geological Strength Index (GSI), produced by Hoek et al. (1995) can be used for estimating the rock mass strength for different geological conditions. Using the Rock Mass Rating of Bieniawski (1989), GSI = RMR'89 – 5, where RMR'89 has the "groundwater rating" set to 15 and the "adjustment for joint orientation" set to zero.

Thus, in the present study, GSI values for three tunnel geotechnical tunnel units mentioned above has been computed from RMR values given in table 4.8, and the results obtained are summarized in table 4.10 below.

Table 4.10: Summary of rock mass classification results of Mid- Level Outlet Tunnel

Rock Mass Classifications	Tunnel Geotechnical Units		
	Gtu.1	Gtu.2	Gtu.3
RMR-values	44 - 69	20 -38	14
RMR- basic	51 -76	27 - 45	24
Q-system	1.83- 10.94	0.23 – 1.16	0.03 – 0.09
GSI- value	47 - 71	25 - 45	19
Descriptions	Fair to good rock mass	Poor to fair rock mass	Very poor rock mass
Estimated percentages	42 %	30 %	28 %

As it is observed from the GSI values determined in table 4.10, the rock masses in the study area has the GSI values is reached from 19 to 71, which is categorized as very poor to good quality of rock masses. As such these GSI values should be considered in the determination of the strength of jointed rock masses for numerical modeling of tunnel support analysis, and the details are presented in section 4.5.5 here after.

4.5.5. Strength of the Rock Masses

For the present study an attempt has been made to empirically estimate the strength of the rock masses. Hoek and Brown (1980) developed an empirical approach to determine the strength of the jointed rock mass and formulated a failure criterion for jointed rock mass. The formulas of Hoek-Brown rock failure criteria are described in detail in section 3.4. However, in the present section rock science software (Rock Lab) has been utilized to determine the rock mass strength parameters. For this, the intact rock strength given in table 4.5, GSI value estimated in table 4.10 and intact rock material constant (m_i) presented in Hoek etal. (1995) were used as input parameter for computing rock mass strengths using Rock Lab software. Thus, for the three tunnel geotechnical units, the strength of rock masses have been determined accordingly, and presented here under

1. Rock Mass Strength of Geotechnical Unit.1 (*Gtu.1*)

- *Average intact rock strength;* δ_c = *44.78 mpa*
- *Average Geological Strength Index; GSI = 59*
- *Intact rock material constant(for schist);* m_i *= 10*
- *Summary analysis of the rock mass strength using RockLab is given in table 4.11*

Accordingly, for tunnel geotechnical unit-1 (***Gtu.1***), the rock mass uniaxial compressive strength is estimated to be 4.55 Mpa, global strength 9.6 Mpa and modulus of deformation is 11.262 Gpa (see Fig.4.6)

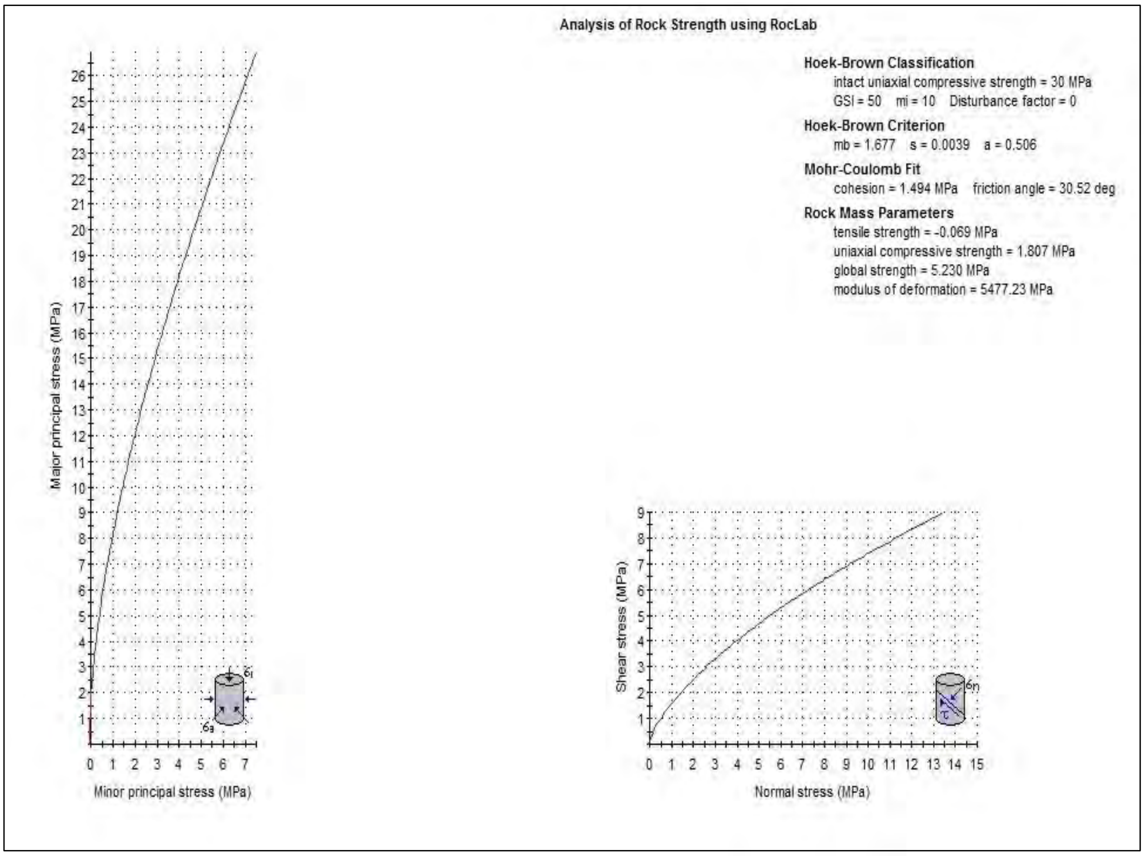


Figure 4.6: Rock mass strength of tunnel geotechnical unit-1 in the study area

2. Rock Mass Strength of Geotechnical Unit.2 (*Gtu.2*)

- *Average intact rock strength;* δ_c = *24.90 Mpa*
- *Average Geological Strength Index; GSI = 35*
- *Intact rock material constant(for schist);* m_i *= 10*
- *Summary analysis of the rock mass strength using RockLab is given in table 4.11*

Thus, for tunnel geotechnical unit-2 (***Gtu.2***), the rock mass uniaxial compressive strength is estimated to be 0.6 Mpa, global strength 3.1 Mpa and modulus of deformation is 2.104 Gpa (see Fig.4.7)

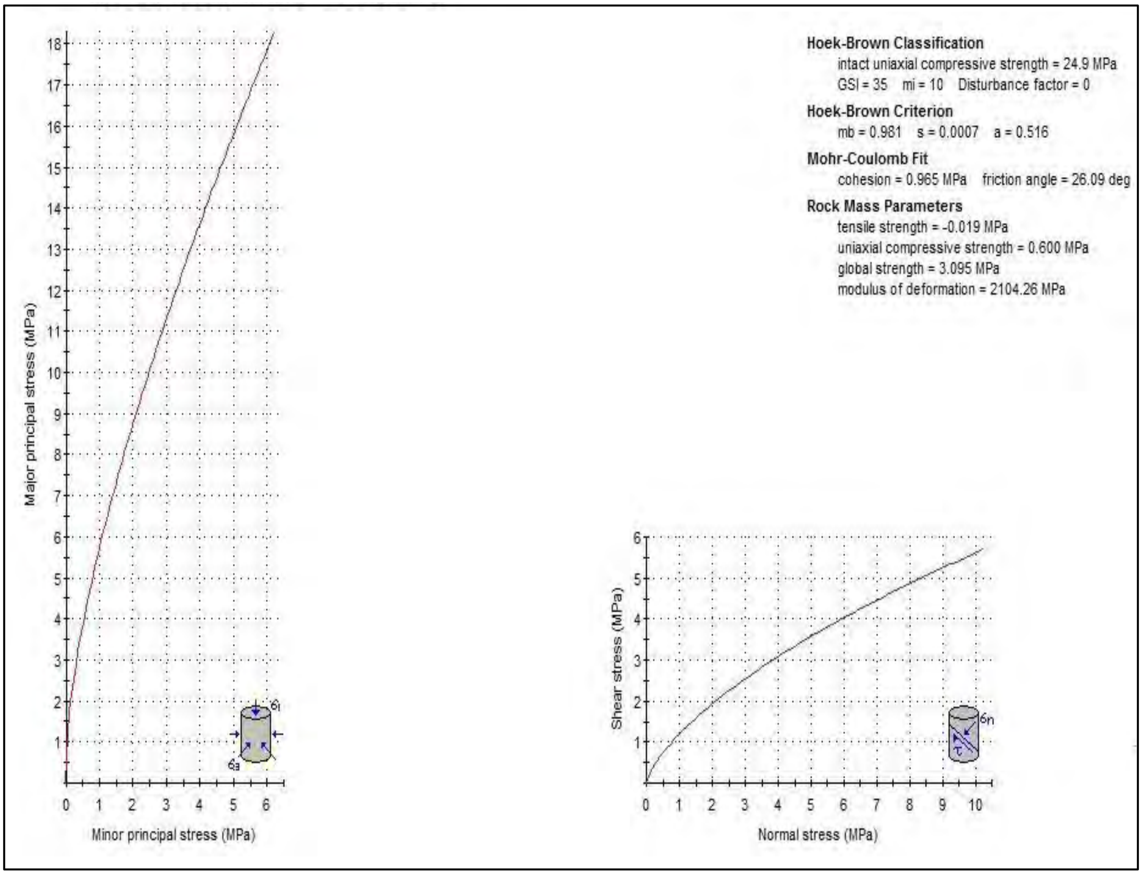


Figure 4.7: Rock mass strength of tunnel geotechnical unit-2 in the study area

3. Rock Mass Strength of Geotechnical Unit.3 (*Gtu.3*)

- *Average intact rock strength;* δ_c = *13.2 Mpa*
- *Average Geological Strength Index; GSI = 21*
- *Intact rock material constant(for schist);* m_i *= 10*
- *Summary analysis of the rock mass strength using RockLab is given in table4.11*

As can be seen in figure 4.8, for tunnel geotechnical unit-3 (***Gtu.3***), the rock mass uniaxial compressive strength is estimated to be 0.114 Mpa, global strength 1.12 Mpa and modulus of deformation is 0.684 Gpa.

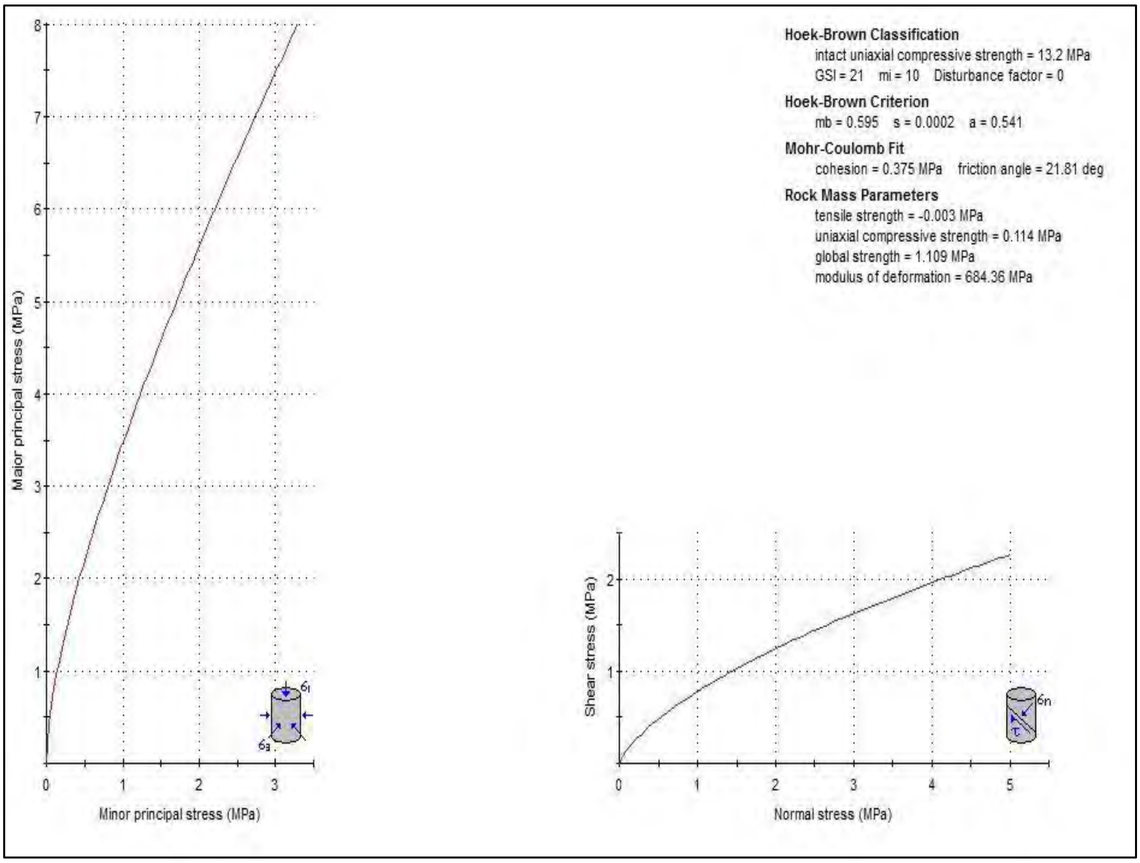


Figure 4.8: Rock mass strength of tunnel geotechnical unit-3 in the study area

Table4.11: Summary of rock mass strengths analysis using Rock Lab software

Geotechnical Unit-1(Gtu.1	*Geotechnical Unit-2(Gtu.2)*	*Geotechnical Unit-3(Gtu.3)*
Hoek Brown Classification *sigci 45MPa* *GSI 59* *mi 10* *D 0* *Hoek Brown Criterion* *mb 2.31243* *s 0.0105087* *a 0.503051* *Failure Envelope Range* *Application General* *sig3max 11.25MPa* *Mohr-Coulomb Fit* *C 2.59766MPa* *Phi 33.1556deg.* *Rock Mass Parameters* *sigt -0.204499MPa* *sigc 4.54936MPa* *sigcm 9.59964MPa* *Em 11261.8MPa*	*Hoek Brown Classification* *sigci 24.9MPa* *GSI 35* *mi 10* *D 0* *Hoek Brown Criterion* *mb 0.981333* *s 0.000730178* *a 0.51595* *Failure Envelope Range* *Application General* *sig3max 6.225MPa* *Mohr-Coulomb Fit* *c 0.965234MPa* *phi 26.0915 deg.* *Rock Mass Parameters* *sigt -0.0185273MPa* *sigc 0.599633MPa* *sigcm 3.09489MPa* *Em 2104.26MPa*	*Hoek Brown Classification* *sigci 13.2MPa* *GSI 21* *mi 10* *D 0* *Hoek Brown Criterion* *mb 0.595209* *s 0.00015412* *a 0.540887* *Failure Envelope Range* *Application General* *sig3max 3.3MPa* *Mohr-Coulomb Fit* *c 0.375453MPa* *phi 21.813 deg.* *Rock Mass Parameters* *sigt -0.00341794MPa* *sigc 0.114455MPa* *sigcm 1.10935MPa* *Em 684.364MPa*

4.6. Findings of Engineering Geological Studies

The middle outlet tunnel of Zarima Dam will have excavation diameter of 5meters and extends for 600 m long and is designed along the left abutment of Zarima dam. It is proposed to be excavated by drill and blast tunneling methods from both outlet and inlet portal sides. The tunnel is buried in the rock overburden of ranging from 12 m to 105 m, and the geology through which the tunnel is excavated is composed of Meta volcanic rocks of low grade metamorphic unit affected with variable degree of rock masses weathering.

Engineering geological studies of the tunnel route are main scope of this thesis work. This includes geological mapping in scale of 1:5000, geostructural surveying, rock mass classifications, and rock mechanics tests. From the results of the engineering geological studies, it is found that the full length of the tunnel will be excavated through Meta-volcanic rocks of low grade metamorphic unit. As such three geotechnical units (Gtu.1, Gtu.2 and Gtu.3) have been identified along the tunnel alignment. These tunnel units are basically classified based on their engineering properties directly related to the degree of rock mass weathering, nature of discontinuities and intact rock strength, For each unit, RMR, GSI, Q values and Rock Mass Strength have been determined at tunnel grades.

Based on the geomechanical evaluations done along the tunnel route, for about 600 m tunnel stretch, it is estimated that the rock classes according to Bieniawski's Classification will be occurred in the following percentages;

- Geotechnical Unit-1; RMR 44 to 69, fair to good quality of rock masses; 42 %
- Geotechnical Unit-2; RMR 21 to 38, poor quality of rock masses; 30 %
- Geotechnical Unit-3; RMR 00 to 20, very poor quality of rock masses; 28 %

As it is observed from the GSI determination, the rock masses in the study area has the GSI values ranging from 19 to 71, which is categorized as very poor to good quality of rock masses. The global strength of the jointed rock masses, as determined by using Rock Lab software, is reached to minimum 1.12 Mpa and maximum 9.6 Mpa with intact rock strength ranging from 13 Mpa to 45 Mpa. Besides, from the evaluations of tunnel quality index (Q-system), it is found that the rock masses encounter along the tunnel route has Q value of lowest 0.03 and highest 10.94. These Q values can be considered for selecting temporary rock support of the tunnel. Thus, the results of the rock mass classifications will suggest the possible underground excavation rock supports for the proposed tunnel and the details are presented in the next section (Chapter 5).

Furthermore, the geological map and resistivity profiles indicate that a total of three geological structures will cross the tunnel axis. These major fracture zones are oriented NNE-SSW (shear zone) to ENE-WSW (Faults), and their orientations are fair to unfavorable with respect to the tunnel axis as they cut the tunnel excavation obliquely. In addition, the stereographic analysis done in the present study reveals the rock joint structures found in the tunnel site show cross cutting relationships, and this can affect the stability of the tunnel excavation by inducing rock wedge failures.

Chapter 5 Underground Excavation Rock Supports

5.1. General

When a tunnel is excavated in competent ground conditions, it is inevitable consequence that some form of support will be required if the tunnel is to retain adequate stability and maintain sufficient dimensions to facilitate its use in the intended manner. In rock tunnels where the ground has insufficient standup time to allow the construction of permanent support some distance behind the face, then some form of temporary ground support applied at the tunnel face is required (EM 1110-2-2901, 1997). Accordingly initial ground support is installed shortly after excavation in order to make the underground opening safe until permanent support is installed. The initial ground support may also function as the permanent ground support or as a part of the permanent ground support.

In the present study, two methodologies have been employed in order to selecting initial ground support; one is the empirical rules produced from Rock Mass Rating and Q-systems; and the other is assessment of the rock failure mechanism and identification of potential wedge failures by using stereographic analysis. As such, five types of underground excavation rock supports have been determined for the study tunnel, and the details are presented here in after.

5.2. Rock Failure Mechanisms

Understanding the failure mechanism of a rock mass surrounding an underground excavation is essential in the design of support systems for the tunnels. The failure mechanism depends on the in-situ stress level and characteristics of the rock masses. At shallow depths, where the rock mass is blocky and jointed, the tunnel stability problems are generally associated with gravity falls of wedge from the roof and side walls since the rock confinement is generally low. As the depth below the ground surface increases, the rock stress increases and may reach the level at which the failure of the rock mass is induced (Hoek, 2007). In the study tunnel, the in-situ stress field is commonly of at low magnitude and the rock mass degradation high due to the shallow depth involved and the tunnel instability is more likely to be related to the rock structure aspects. As such, two major rock deformations are identified with respect to stability of the tunnel in the study area;

1. Instability resulting from the rock structures

This includes the natural features of the rock structure such as foliations, joints, fault planes in addition to the movement of discrete blocks under the action of gravity. Plate 5.1 shows the nature of rock mass failures occurred in the study area. In the present study, it has been found that this type of rock failure mechanism may be pronounced in the tunnel section where the tunnel excavation will pass through tunnel geotechnical unit.1 and geotechnical unit.2. The rock masses behavior of these tunnel units are as following;

Tunnel Geotechnical Unit-1 (Gtu.1): This tunnel unit is characterized by slightly weathered and fresh rock masses with closely to widely spaced joints. The intact rock strength is reached to maximum of 68 Mpa with an average RQD value is 65 %.

Tunnel Geotechnical Unit-2 (Gtu.2): The rock masses in this tunnel unit is characterized by moderately weathered and closely jointed rocks, the maximum intact rock strength are reached to 29 Mpa with an average RQD value is 42 %.

The stereographic analysis presented in chapter 4 confirm that the rock joint structures found in these geotechnical units show cross cutting relationships, and this can affect the stability of the tunnel excavation by inducing rock wedge failures.

Plate 5.1: Rock mass failures occurred in the study area due to crosscutting relationships of the joint structures

2. Effect of semi - failed state of ground prior to tunnel excavations

This situation can also exist where by the rock mass is highly fractured and weathered and is in effect, already in a semi-failed state. In these conditions, even with low values of in-situ stress, large values stress induced closures can occur although the strength of intact rock may be high significantly higher than the level of in-situ stress. Plate 5.2 shows the nature of rock mass failures occurred in highly fractured and weathered rocks of the study area. In the study tunnel site, this problem is highly suspected at the inlet and outlet portal sections where the tunnel is situated in highly fractured and weathered rock masses.

The meta- volcanic rock situated at the inlet and outlet tunnel portals are highly weathered and locally decomposed to soil and at some places overlain by colluvial deposit (Fig 4.3). The field rock strength tests conducted in accordance with ISRM (1981) confirm that it is extremely weak to weak rock masses (Table 4.7). As it is observed from the geophysical/ resistivity survey, there is a wide weakness zones with low apparent resistivity, which is considered that the Meta-volcanic rocks are shattered and weathered widely and deeply at the top part. Besides the top soil which is usually 5 to 10 m thick covers the weathered rock masses.

Plate 5.2: Rock mass failure occurred in highly fractured and weathered rocks of the study area

5.3. Determination of Rock Support Pressures

The key to successful design and support of tunnel is to evaluate the ground loads exert on the excavation openings during the site investigation stage of the project such that appropriate supporting measures can be utilized during tunnel construction. Sinha (1989) states for underground excavations in rocks, the most important loading comes from the host ground itself. In competent host ground, the rock loading on underground structures is quite insignificant, where as in incompetent rocks, it may be quite significant. The host ground rock pressure is dependent on several factors such as the relative stiffness of the structure and the host ground, the elapsed time between the excavation and installation of support, the characteristics of the host ground, the in-situ stress, the size of opening, the ground water conditions and adopted method of construction.

The rock support load for underground excavations in the study area has been estimated using empirical approaches from RMR values in accordance with the formulas expressed by Eq.3.1 and Eq.3.2 given in chapter 3, and the summary data are presented in table 5.1. Hence, the following results are obtained for 5 m excavation diameter of the tunnel;

1. **Rock Support Load for Geotechnical Unit.1 (Gtu.1)**

For average RMR = 56, and the rock density is 2580kg/m^3

$$\textit{Rock support pressure; } \boldsymbol{P} = \frac{\mathbf{100-56}}{\mathbf{100}}\mathbf{2580} * \mathbf{5} = \mathbf{5.7}\boldsymbol{ton/m2}$$

$$\textit{Rock load height; } \boldsymbol{Ht} = \left(\frac{\mathbf{100-56}}{\mathbf{100}}\right)\mathbf{5} = \mathbf{2.2}\boldsymbol{m}$$

2. **Rock Support Load for Geotechnical Unit-2(Gtu.2)**

For average RMR = 29 and the rock density is 2480kg/m^3

$$\textit{Rock support pressure; } \boldsymbol{P} = \frac{\mathbf{100-29}}{\mathbf{100}}\mathbf{2480} * \mathbf{5} = \mathbf{8.8}\boldsymbol{ton/m2}$$

$$\textit{Rock load height; } \boldsymbol{Ht} = \left(\frac{\mathbf{100-29}}{\mathbf{100}}\right)\mathbf{5} = \mathbf{3.55}\boldsymbol{m}$$

3. **Rock Support Load for Geotechnical Unit.3 (Gtu.3)**

For average RMR = 14, and the rock density is 2420kg/m^3

$$\textit{Rock support pressure; } \boldsymbol{P} = \frac{\mathbf{100-14}}{\mathbf{100}}\mathbf{2420} * \mathbf{5} = \mathbf{10.4}\boldsymbol{ton/m2}$$

$$\textit{Rock load height; } \boldsymbol{Ht} = \left(\frac{\mathbf{100-29}}{\mathbf{100}}\right)\mathbf{5} = \mathbf{4.3}\boldsymbol{m}$$

Table 5.1: Rock support load determinations for underground excavations in the study area.

Geotechnical Units	Rock Mass Properties					
	Unit Weight (gm/cm^3)	Av. RMR	Av. GSI	Av. Q-value	Max. Rock Support Load(ton/m^2)	Max. Rock Load Height (m)
Gtu.1	2.42	56	59	6.39	5.7	2.2
Gtu.2	2.48	29	35	0.70	8.8	3.55
Gtu.3	2.58	14	21	0.06	10.4	4.3

From the above rock load analysis, it has been found that the rock support pressure for the three tunnel geotechnical units are estimated to be in the range of 5.7ton/m^2 and 10.4ton/m^2, and the rock load height is from 2.2 m to 4.3 m respectively.

5.4. Identification of Potential Wedge Failure

In tunnels which are excavated through jointed rock mass at relatively shallow depth, the most common types of failure are those involving wedges falling from the roof or sliding out of the side walls of the openings (Fig.5.1). These wedges are formed by intersecting structural features, such as foliation planes and joints, which separate the rock mass into discrete but interlocked pieces. When free face is created by the excavation of the openings, the restrained from the surrounding rock is removed. One or more of these wedges can or slide from the surface if the bounding planes are continuous or rock bridges along the discontinuities are broken (Hoek et al., 1995). The rock mass classification scheme discussed in the previous section cover a wide range of tunneling geotechnical environments. With respect to rock bolting, however, they do not identify critical blocks of the large mass which need individual consideration in relation to their support. Thus, stereographic projection can be used to solve such kind of problems for small size tunnels situated at shallow depth.

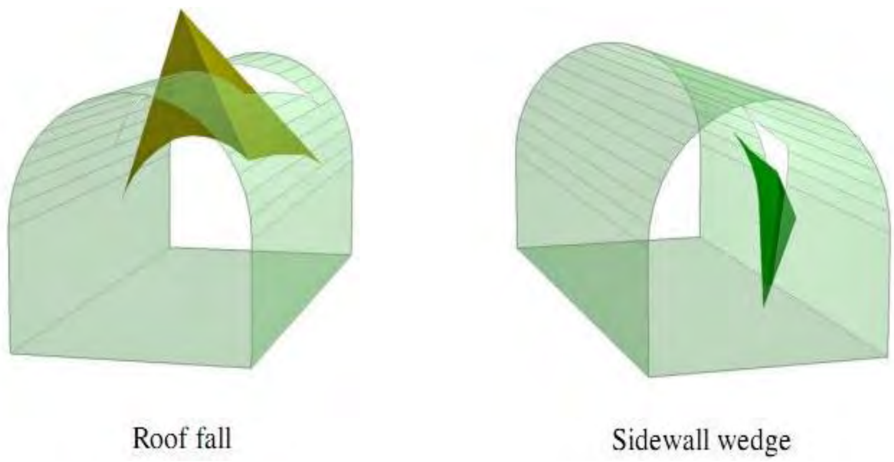
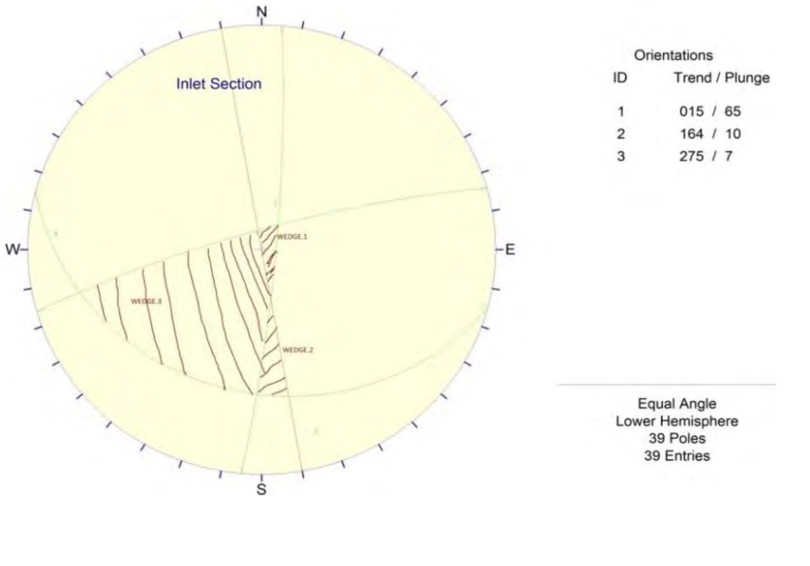


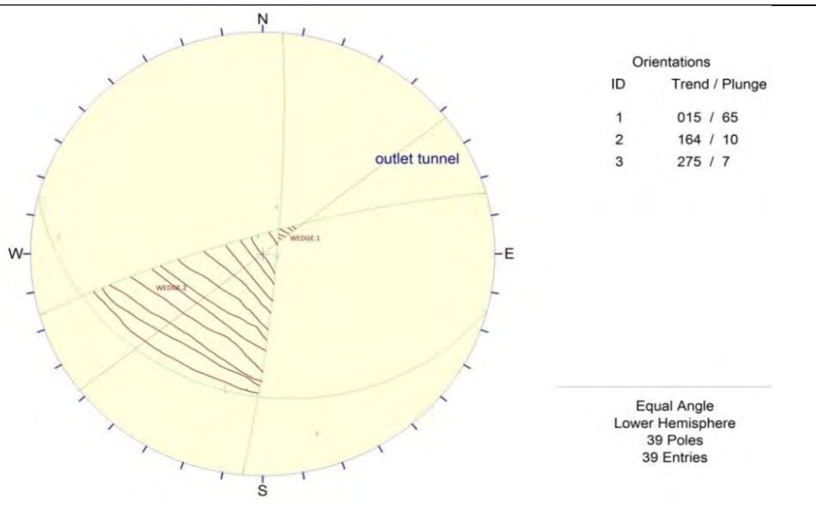
Figure 5.1: Modes of Wedge failure in underground excavations in jointed rock mass (Hoek,2007)

In the present study, stereographic analysis has been done using Rock science software (Dips Program) in order to identify the potential wedge failures, and three strongly developed joint sets occur in the study area. The ranges of their dips and dip directions of these joint sets are shown as great circle in figure 5.2. From the stereographic analysis, it has been found that the rock joint structures found in the study tunnel site show cross cutting relationships. Hence, local sidewall wedges and significant roof wedge failures are highly suspected at outlet and inlet tunnel portion, as the two tunnel portals are situated at shallow depth in highly jointed and weathered rock masses with 12m to 42m thickness rock overburden conditions.

Joint Set	Dip	Dip Direction
J1	20 – 30°	185 – 205°
J2	75 - 85°	333 – 353°
J3	77 – 87°	085 – 105°



a) Inlet Tunnel Section: three wedges are observed such as wedge.1, wedge.2 and wedge.3 due to the intersection of joint set J1, J2 and J3. Significant side wall wedges and roof wedge are highly suspected at inlet tunnel section



b) Outlet Tunnel Section: two wedge failures are observed due to the intersection of joint set J1, J2 and J3. Roof wedge is highly suspected at outlet tunnel section

Figure 5.2: Stereographic analysis showing potential rock wedge failures in the study tunnel

5.5. Preliminary Rock Support Analysis

5.5.1. Preliminary Analysis: Q - System

For the present study, in order to determine the tunnel excavation rock supports, a preliminary analysis has been performed by means of the rock classification systems (RMR, Q-system). The empirical rules suggested by Bieniawski (1989), shown in table 3.3 of section 3.4.1, can be considered only as a general reference being referred to a 10 m span rock tunnels excavated with a horse shoe shape, quite far from the study tunnel having 5m excavation diameter. Hence, the Q-system has been applied basing on the following approximate conversion table extracted from section 4 with relevant rock classes considering;

Rock Mass Classifications	Tunnel Geotechnical Units						
	Gtu.1			Gtu.2		Gtu.3	
RMR -values	44 - 70			20 - 40		10 - 18	
GSI- values	> 61	45- 61	31- 44	31- 44	21 - 30	21 - 30	< 21
Q-system	> 9.5	1.8 – 9.5	0.23-1.7	0.23 – 1.7	0.09- 0.23	0.09- 0.23	0.03- 0.09

Considering an average tunnel height of 5m and excavation support ratio ESR = 1.6 (water conveyance tunnel), the equivalent dimension of the tunnel (De) have been evaluated as;

$$\boldsymbol{De} = {}^{Hieght}/_{ESR} = {}^{5}/_{1.6} = 3.13$$

The length of the rock bolts have been estimated from the excavation width B and the excavation support ratio ESR, as given below:

$$L = {}^{(2+0.15B)}/_{ESR} = {}^{(2+0.15*5)}/_{1.6} = 1.72\ m$$

The bolt length is estimated to be 1.72m which is relatively short to resist the rock pressure exerting on the tunnel opening that will be passed through Gtu.2 and Gtu.3, as the rock load height computed in table 5.1 is higher than the bolt length in shallow tunnel conditions. Thus, the bolt length shall be fixed to 2.5m by considering the average of the two situations.

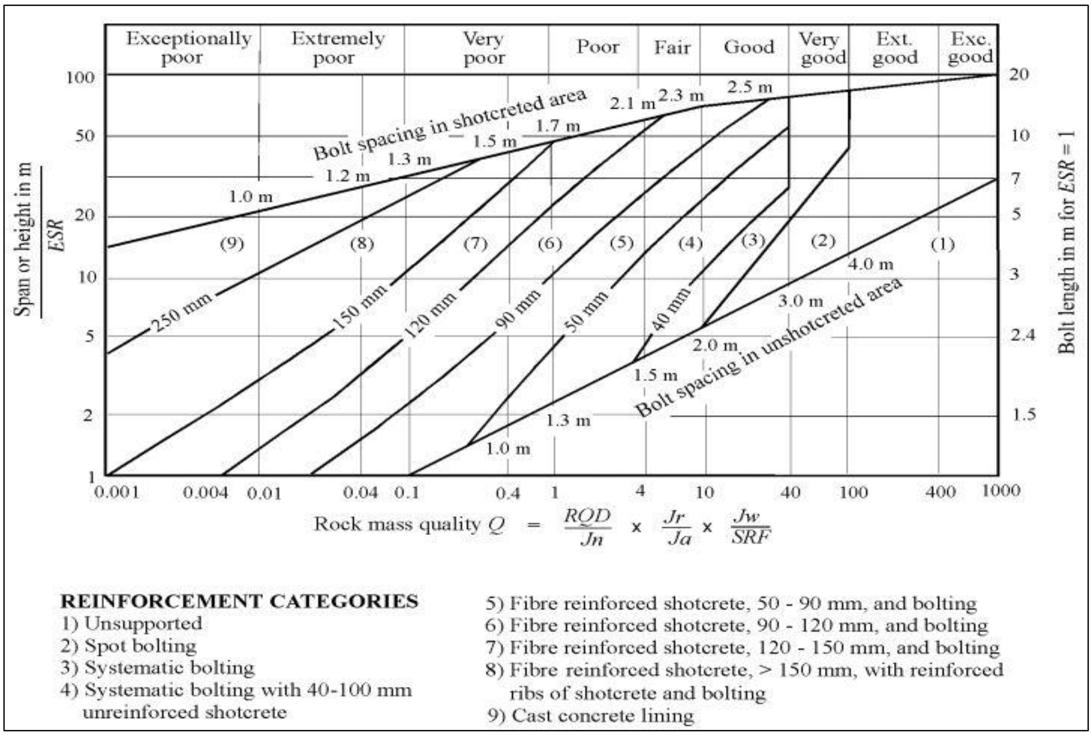
Maximum Unsupported Span

Using the values of Q system, Barton etal (1974; as cited in Bieniawski, 1989) gives empirical relationships for the maximum unsupported span as expressed by the equations given here under;

$$\text{Maximum span} = 2(\text{ESR})\ \text{Q}^{0.4}$$

Accordingly, for ESR=1.6 (water conveyance tunnels), the maximum unsupported span for underground excavations in Gtu.1, Gtu.2 and Gtu.3 are 8.33 m, 2.77 m and 1m respectively

Based on the recent estimated support category suggested by Barton and Grimstad (1993; as cited in Hoek, 2007) illustrated in the figure 3.7, and as shown below, the tunnel rock support is assessed for each rock class, considering the lowest Q value for each class for determining the foreseen tunnel support.



5.5.2. Suggested Underground Excavation Supports

The results of the preliminary analysis of the Q system in combination with the determined rock load pressures suggest five types of underground excavation rock supports for the study tunnel, as given in table 5.2 and the details are presented here after;

1. SUPPORT TYPE- A (Q > 9.5)

Spot bolting is foreseen, as required, for supporting the possible local unstable wedges forming on the crown or along the side walls.

2. SUPPORT TYPE- B (Q = 1.8 – 9.5)

SHOTCRETE; 40 mm to 50 mm thick unreinforced shotcrete layer is required at the crown and sidewalls.

PATTERN ROCK BOLTING; Pattern rock bolts with 24 mm diameter and 2.5 m long are required along crown and sometimes at sidewalls with 1.7 m spacing center to center and between each row

UNSUPPORTED SPAN; Maximum unsupported span is 8 meters

These rock supports have been proposed taking in to account the followings

- A shotcrete arch layer is foreseen for protection against rock weathering, local raveling phenomena and structural control instability.
- The pattern rock bolting is envisaged to help controlling the possible structural local instability.

3. **SUPPORT TYPE- C** (Q = 0.23 - 1.7)

SHOTCRETE; 50 mm to 90 mm thick layer of reinforced shotcrete is required all along the excavation (crown + sidewalls)

PATTERN ROCK BOLTING; Pattern rock bolts with 24mm diameter and 2.5m long are required along crown and sidewalls with 1.4m spacing center to center and between each row

UNSUPPORTED SPAN; Maximum unsupported span is 4 meters

These rock supports have been proposed taking in to account the followings

- A shotcrete arch layer is foreseen for protection against rock weathering and/or local raveling phenomena, controls the tunnel convergence, and allows a control of local structural controlled instabilities.
- The pattern rock bolting is foreseen to help controlling the structural instability encountered during the excavation and the control of the tunnel convergence by means of the shotcrete.

4. **SUPPORT TYPE- D** (Q = 0.09- 0.23)

SHOTCRETE: 90 mm to 120 mm thick layer of reinforced shotcrete is required all along the excavation (crown + sidewalls).

PATTERN ROCK BOLTING; Pattern rock bolts with 24 mm diameter and 2.5 m long are required along crown and sidewalls with 1.2 m spacing center to center and between each row

UNSUPPORTED SPAN; Maximum unsupported span is 2 meters.

These supports have been proposed taking in to account the followings;

- A shotcrete arch layer is obviously required for protection against rock weathering and local raveling phenomena.
- The pattern rock bolting is helps controlling the structural local instability.

5. SUPPORT TYPE- E (Q < 0.09)

SHOTCRETE; 120 mm to 150 mm thick layer of reinforced shotcrete is required all along the excavation (crown + sidewalls)
STEEL RIBS; Medium Weight Steel Ribs with 1- 1.2 m spacing center to center are required with the reinforced shotcrete.

These rock supports have been proposed taking in to account the followings;

- The rock mass behavior is potentially risk situation
- A shotcrete arch layer is foreseen for protection against rock weathering and local raveling phenomena.
- The steel ribs are envisaged to reduce the tunnel closure by making confinement on the excavation opening.

Table5.2: Suggested underground rock supports for middle level outlet tunnel

Descriptions	Tunnel Geotechnical Units						
	Gtu.1			Gtu.2		Gtu.3	
RMR	44 - 70			20 - 40		10 - 18	
GSI	> 61	45- 61	31- 44	31- 44	21 - 30	21 - 30	< 20
Q-value	> 9.5	1.8 – 9.5	0.23-1.7	0.23 – 1.7	0.09- 0.23	0.09- 0.23	0.03- 0.09
Suggested Support Type	**A**	**B**	**C**	**C**	**D**	**D**	**E**
	Tunnel Support Design According to SGI and SC (2014)						
GSI-values	GSI > 40, fair to good rocks			GSI < 40, fractured rocks			
Supports	▪ Reinforced Shotcrete, 60mm ▪ No. 3 bolts at the crown with 24mm diameter and 1m spacing			▪ 60mm thick Reinforced shotcrete ▪ No. 4 bolts at the crown with 24mm diameter and 1m spacing ▪ Steel ribs with 1m spacing center to center			

In table 5.2, it is observed that the rock support systems suggested by SGI and SC (2014) is based on the general support guideline produced by Bieniawski (1989), for 10m diameter horseshoe tunnels excavated by drill and blast methods, which is conservative support for the proposed tunnel as the tunnel has 5m excavation diameter.

The rock masses in the study area are characterized by very poor to good quality of rocks having GSI values ranging from 20 up to 71 and Q values are from 0.03 up to 10.94. In addition, as the tunnel will be excavated through shallow depth, the crosscutting relationships of the joint structures are more pronounced in the study area, and then rock wedge failures are highly suspected during excavation works. Thus, by considering these situations, the underground rock support suggested by the present study illustrates that:

- The shotcrete thickness becoming in the design is generally slightly higher than the one suggested for temporary rock supports with Q methodology, as the Q values modified to $Q^1 = 5Q$ for selecting temporary rock supports
- The design bolt length is generally substantially higher (2.5 m vs. 1.72 m) and having smaller spacing.

It can be therefore conclude that, broadly speaking, the proposed underground excavation rock support design is slightly more on the safe side if compared with the one suggested with the Q system for temporary rock supports.

5.5.3. Limitations and uncertainties of the tunnel rock support analysis

It needs to be emphasized that the temporary rock support analysis is based on a limited amount of field data. Results should be used with care and considered as estimates.

Selection of representative rock geotechnical parameters are a difficult task since the geological conditions at tunnel depth are to a high degree of unknown during site investigation stages. Accurate estimation of variables requires field mapping at tunnel depth during construction works. Since that is beyond the scope of this thesis, the input parameters used are based on measurements carried out at rock exposures near by the tunnel location, or when such measurement does not exist, average common values assessed to be represent the expected rock conditions are used.

The role of geotechnical data which has been previously assessed are as one of particular importance in respect of the support of tunnels. It is of major importance in the design of rock reinforcement systems in views of the latter needing to match and complement the rock structure as closely as possible. Emphasis requires to be placed on rock structural data obtained during site investigation stage, and should be verified and reviewed in the light of further information obtained during the construction of the tunnel. As a result, the tunnel rock support design should be updated as new geotechnical data become available following new rock exposures obtained by inspection of underground excavations.

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Appendices

Appendix-1a: Results of laboratory rock property tests according to JWHC (2013)

SABA Engineering PRIVATE LIMITED COMPANY
Box 62668 Addis Ababa , Ethiopia. Tel.4- 39 06 93/ 39 10 65/439161 // 4391733 / 4393004/ Fax. 4391230 / 43

LAB NO.	: 2376/13
CLIENT	: Jiangxe Water And Hydropower Consteruction
PROJECT	: Grouting Works Under Wolkite Sugar Developi
SAMPLE OF	: Rock Core
SAMPLED BY	: The Client
TEST SPECIFIED BY	: The Client ON:09/11/13
TEST ORDER SUBMITTED BY	: The Client ON:09/11/13
Test For	Uniot Weight
RESULT REPORTED TO	: The Client ON:30/11/13

TEST RESULTS

SR NO	Sample Source	Depth (m)	Dia cm	Height cm	Area cm^2	Volume cm^3	Mass (gm)	Load (KN)	Unit Weight (gm/cc)	Commpressive Strength	
										Kg/Cm^2	Mpa
1	JVS - 01	29.36 - 29.60	7.0	14	38.47	538.51	1324.00	71.0	2.46	188.16	18.8
2	JVO - 03	29.32 - 29.56	7.0	14	38.47	538.51	1460.00	119.8	2.71	317.48	31.7
3	JVO - 02	28.84 - 29.09	7.0	14	38.47	538.51	1354.80	49.9	2.52	132.24	13.2
4	JVD - 03	40.27 - 40.75	7.0	14	38.47	538.51	1374.30	146.4	2.55	387.98	38.8
5	JVD - 01	15.76 - 16.00	7.0	14.0	38.47	538.51	1450.70	71.9	2.69	190.54	19.1
6	JVD - 02	25.75 - 25.95	7.0	14.0	38.47	538.51	1450.00	148.8	2.69	394.34	39.4
7	JVO - 03	38.50 - 38.80	7.0	14.0	38.47	538.51	1369.00	140.4	2.54	372.08	37.2
8	JVD - 02	13.44 - 13.71	7.0	14.0	38.47	538.51	1251.00	24.7	2.32	65.46	6.5
9	JVO - 02	36.05 - 36.33	7.0	14.0	38.47	538.51	1443.00	257.4	2.68	682.14	68.2
10	JVO - 04	29.85 - 30.05	7.0	10.0	38.47	384.65	908.00	47.7	2.36	126.41	12.6
11	JVO - 01	16.86 - 17.02	7.0	9.0	38.47	346.19	808.00	64.2	2.33	170.14	17.0
12	JVO - 01	20.55 - 20.74	7.0	10.0	38.47	384.65	938.00	151.9	2.44	402.55	40.3
13	JVD - 01	9.80 - 10.00	7.0	8.0	38.47	307.72	709.00	82.9	2.30	219.69	22.0
14	JVD - 05	9.68 - 9.88	7.0	10.0	38.47	384.65	866.00	69.7	2.25	184.71	18.5
15	JVO - 05	27.81 - 28.06	7.0	14.0	38.47	538.51	1377.00	139.7	2.56	370.22	37.0
16	JVD - 03	9.85 - 10.00	7.0	12.0	38.47	461.58	1270.00	21.8	2.75	57.77	5.8
.7	JVQL - 02	5.27 - 5.43	7.0	11.0	38.47	423.12	852.40	64.3	2.01	170.40	17.0
18	JVD - 01	19.64 - 19.89	7.0	14.0	38.47	538.51	1133.00	92.5	2.10	245.14	24.5
19	JVO - 01	32.95 - 33.23	7.0	14.0	38.47	538.51	1355.00	175.3	2.52	464.57	46.5
20	JVO - 01	43.40 - 43.70	7.0	14.0	38.47	538.51	1331.00	111.8	2.47	296.28	29.6
21	JVD - 02	42.30 - 42.58	7.0	14.0	38.47	538.51	1497.00	217.3	2.78	575.87	57.6
22	JVD - 02	17.21 - 17.41	7.0	14.0	38.47	538.51	1436.00	155.7	2.67	412.62	41.3
23	JVQL - 02	15.53 - 15.85	7.0	14.0	38.47	538.51	1357.00	63.02	2.52	167.01	16.7
24	JVQL - 02	18.69 - 18.93	7.0	14.0	38.47	538.51	1125.00	154.3	2.09	408.91	40.9
25	JV0 - 05	26.52 - 26.67	7.0	14.0	38.47	538.51	1295.00	125.1	2.40	331.53	33.2

Appendix-1b: Results of laboratory rock property tests conducted in the present study

Sur Construction
Zarema May-Day dam project
Laboratory section

Sample date:- 15 August, 2014	Type of Material: Rock	Material Sourse- Zarema River
Test date:- 18 August, 2014	Location:- Zarema River	Sampled by:
	Tested by:	

Data's		Sample ID.	Sample Location	Description	Results: Bulck Volume(Cm^3)	Grain Volume(Cm^3)	Grain Density (gm/cm^3)	Grain Volume of original sample (Cm^3)	Porosity (%)	Dry Unit Weght	Specific Gravity
1- Weght of clean dry sample (W_{dry})	2293	MWmr.2	Road cutting down stream of spillway E:0370178 N:1519699	Moderately weatherd Meta volcanic rock	849	99	2.87	799.32	5.85	2.93	2.78
2- Weght of saturated sample in air (W_{sat})	2359										
3- Weght of saturated sample immersed in water(W_{imm})	1510										
4- Weght of water displaced($W_{displaced\ wtr}$)	849										
1- Weght of dry crushed sample in air(W_{dry})	284										
2- Weght of pycnometer filed with water(W_{py+wtr})	1635										
3- Weght of crushed sample pycnometer and water($W_{py+wtr+sample}$)	1820										
4- Weght of displaced water($W_{displaced\ wtr}$)	99										

sampled by
Name Abichu Lule
Sig.

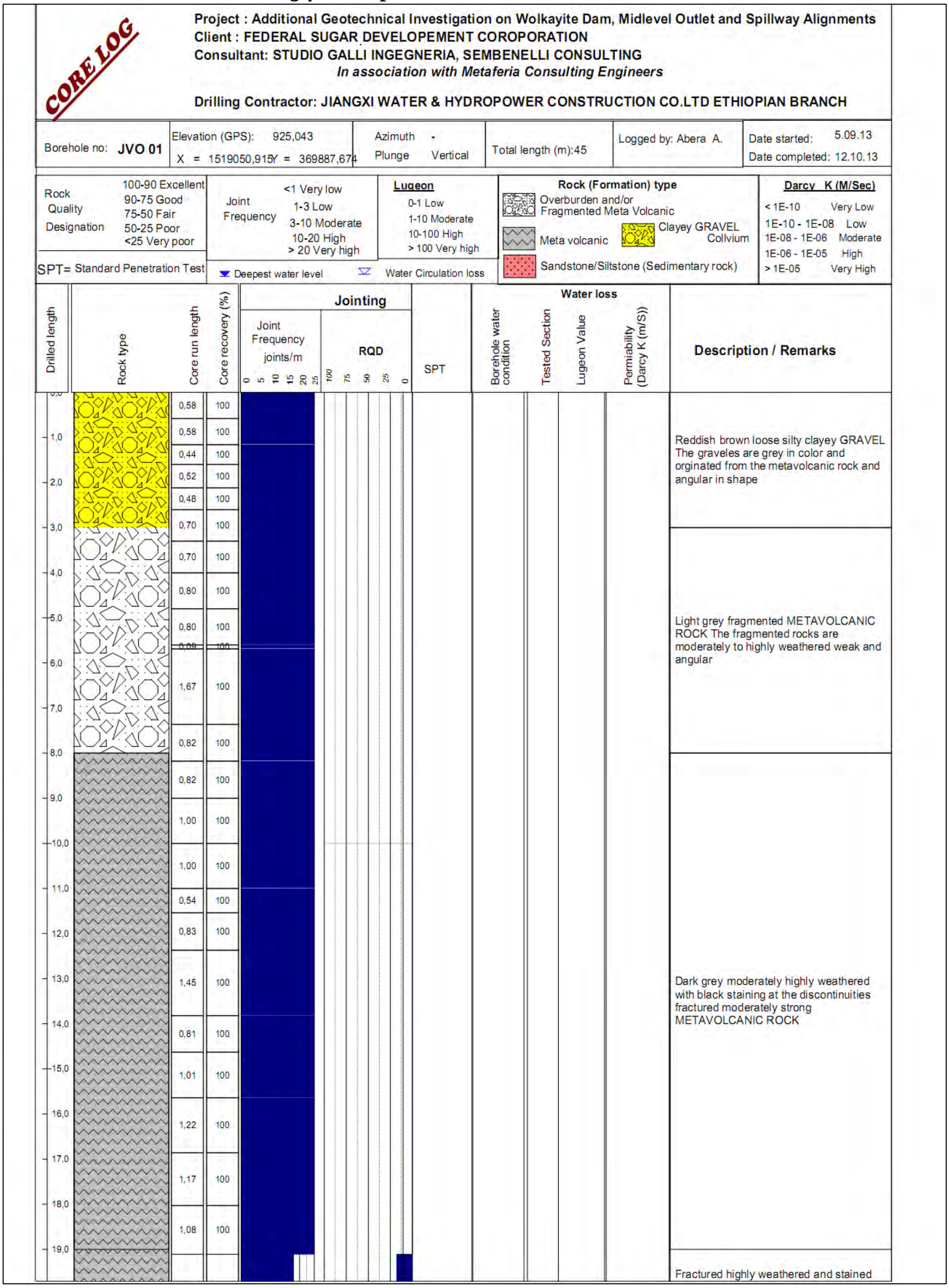
Tested by
Name Zemane
Sig.

Approved by(Material Engineer)
Name Ambachew
Sig.

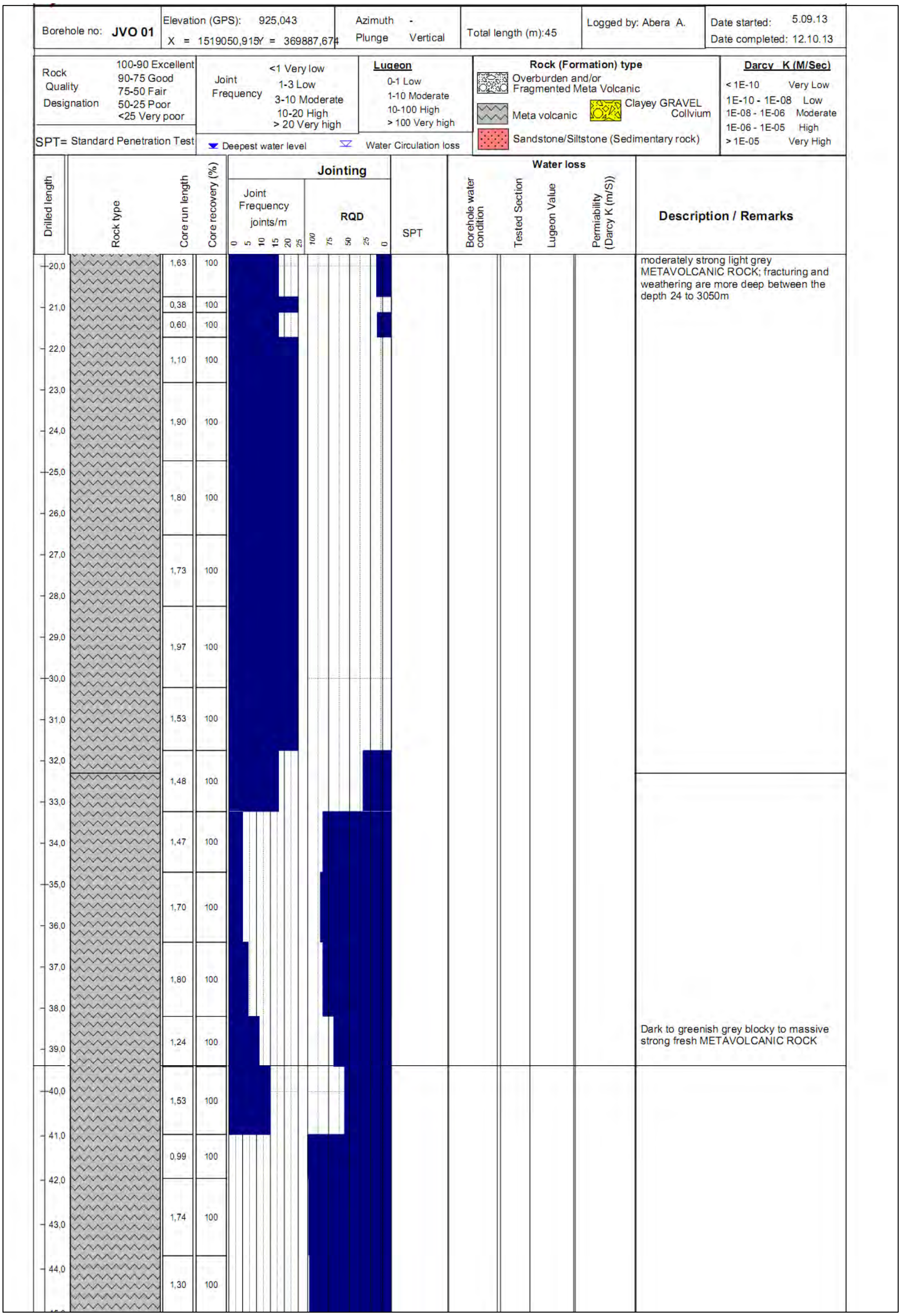
Sur Construction PLC Zarema May-Day Dam Construction Project

Appendix-2: Geotechnical logs of boreholes drilled along tunnel alignment according to JWHC (2013)

JVO.1: Geotechnical core log from depth 0.00 to 20m



JVO.1: Geotechnical core log from depth 20 to 45 m



JVO.2: Geotechnical core log from depth 0.00 to 20m

CORE LOG

Project : Additional Geotechnical Investigation on Wolkayite Dam, Midlevel Outlet and Spillway Alignments
Client : FEDERAL SUGAR DEVELOPEMENT COROPORATION
Consultant: STUDIO GALLI INGEGNERIA, SEMBENELLI CONSULTING
In association with Metaferia Consulting Engineers

Drilling Contractor: JIANGXI WATER & HYDROPOWER CONSTRUCTION CO.LTD ETHIOPIAN BRANCH

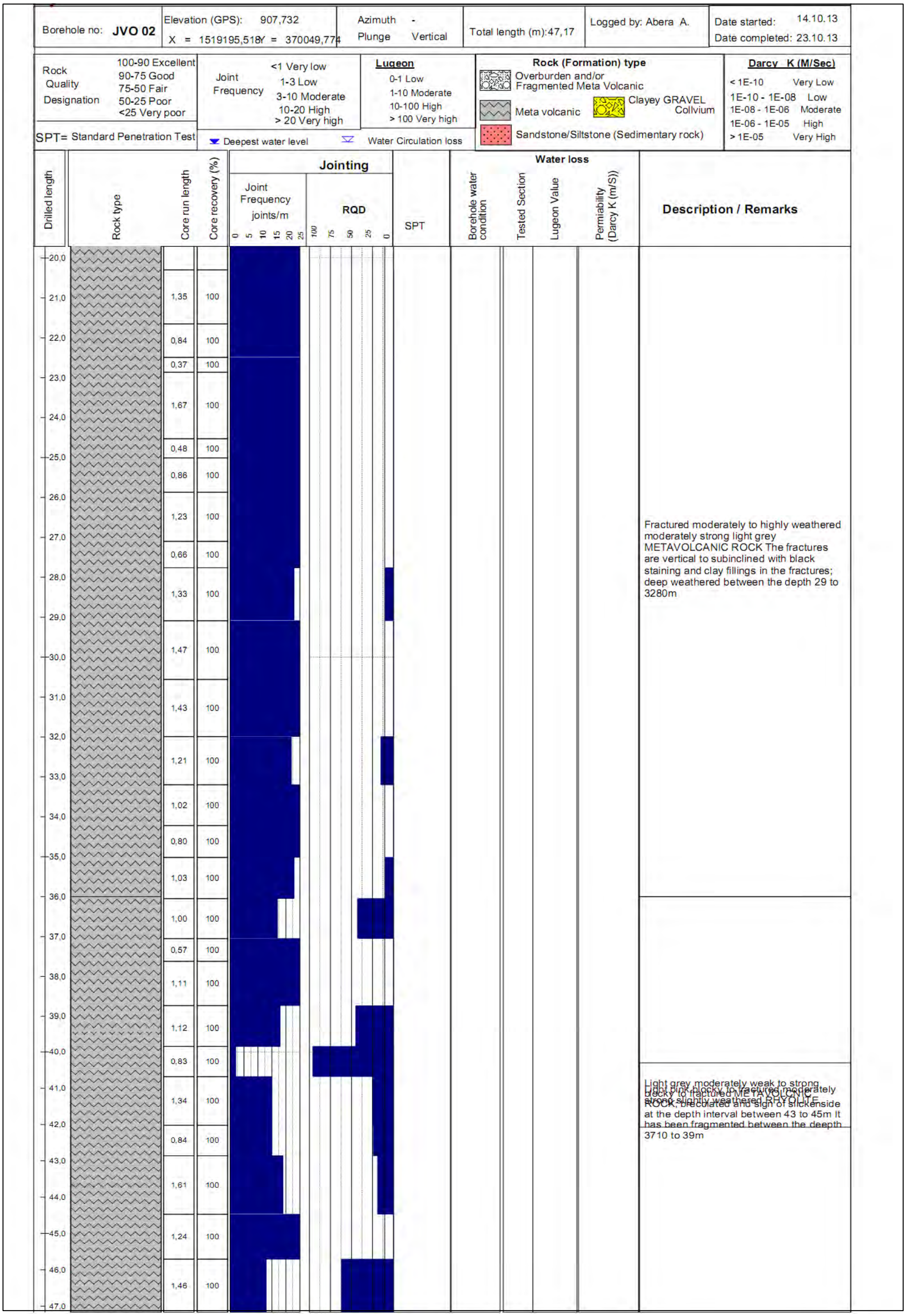
Borehole no: **JVO 02**	Elevation (GPS): 907,732 X = 1519195,518Y = 370049,774	Azimuth - Plunge Vertical	Total length (m):47,17	Logged by: Abera A.	Date started: 14.10.13 Date completed: 23.10.13

Rock Quality Designation	Joint Frequency	**Lugeon**	**Rock (Formation) type**	**Darcy K (M/Sec)**
100-90 Excellent 90-75 Good 75-50 Fair 50-25 Poor <25 Very poor	<1 Very low 1-3 Low 3-10 Moderate 10-20 High > 20 Very high	0-1 Low 1-10 Moderate 10-100 High > 100 Very high	Overburden and/or Fragmented Meta Volcanic Meta volcanic Clayey GRAVEL Collvium Sandstone/Siltstone (Sedimentary rock)	< 1E-10 Very Low 1E-10 - 1E-08 Low 1E-08 - 1E-06 Moderate 1E-06 - 1E-05 High > 1E-05 Very High

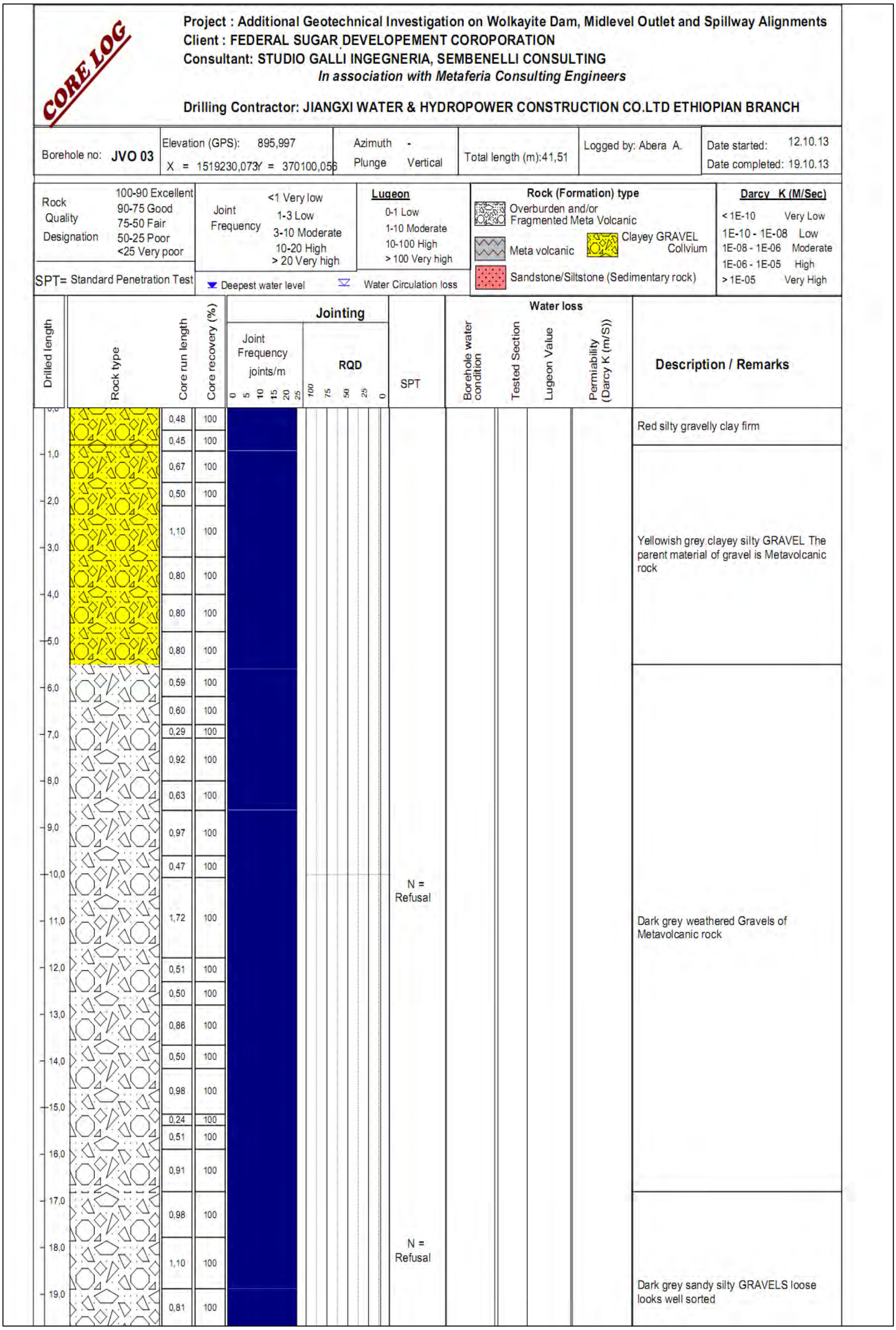
SPT= Standard Penetration Test | Deepest water level | Water Circulation loss

Drilled length	Rock type	Core run length	Core recovery (%)	Jointing: Joint Frequency joints/m (0 5 10 15 20 25)	Jointing: RQD (100 75 50 25 0)	SPT	Water loss: Borehole water condition	Water loss: Tested Section	Water loss: Lugeon Value	Water loss: Permiability (Darcy K (m/S))	Description / Remarks
0,0		0,70	100								
1,0		0,60	100								
		0,63	100								
2,0		0,68	100								
3,0		0,59	100								
		0,75	100								
4,0		0,66	100								
5,0		0,70	100								
6,0		0,68	100								
		0,60	100								Yellowish to reddish grey firm to loose silty sandy GRAVEL Resulted from the decomposition Metavolcanic rock
7,0		0,60	100								
		0,40	100								
8,0		0,41	100								
		0,80	100								
9,0		0,70	100								
10,0		0,90	100			N = Refusal					
11,0		0,90	100								
12,0		1,00	100								
13,0		0,88	100								
14,0		0,88	100								
15,0		0,88	100								Dark grey fractured to fragmented moderately to highly weathered moderately strong METAVOLCANIC ROCK with black staining and deeep weathering at the discontiniuties
		0,85	100								
16,0		1,15	100								
17,0		0,65	100								
		0,12	100								
18,0		1,36	100			N = Refusal					Light grey sandy silty gravel loose more uniform
19,0		1,23	100								

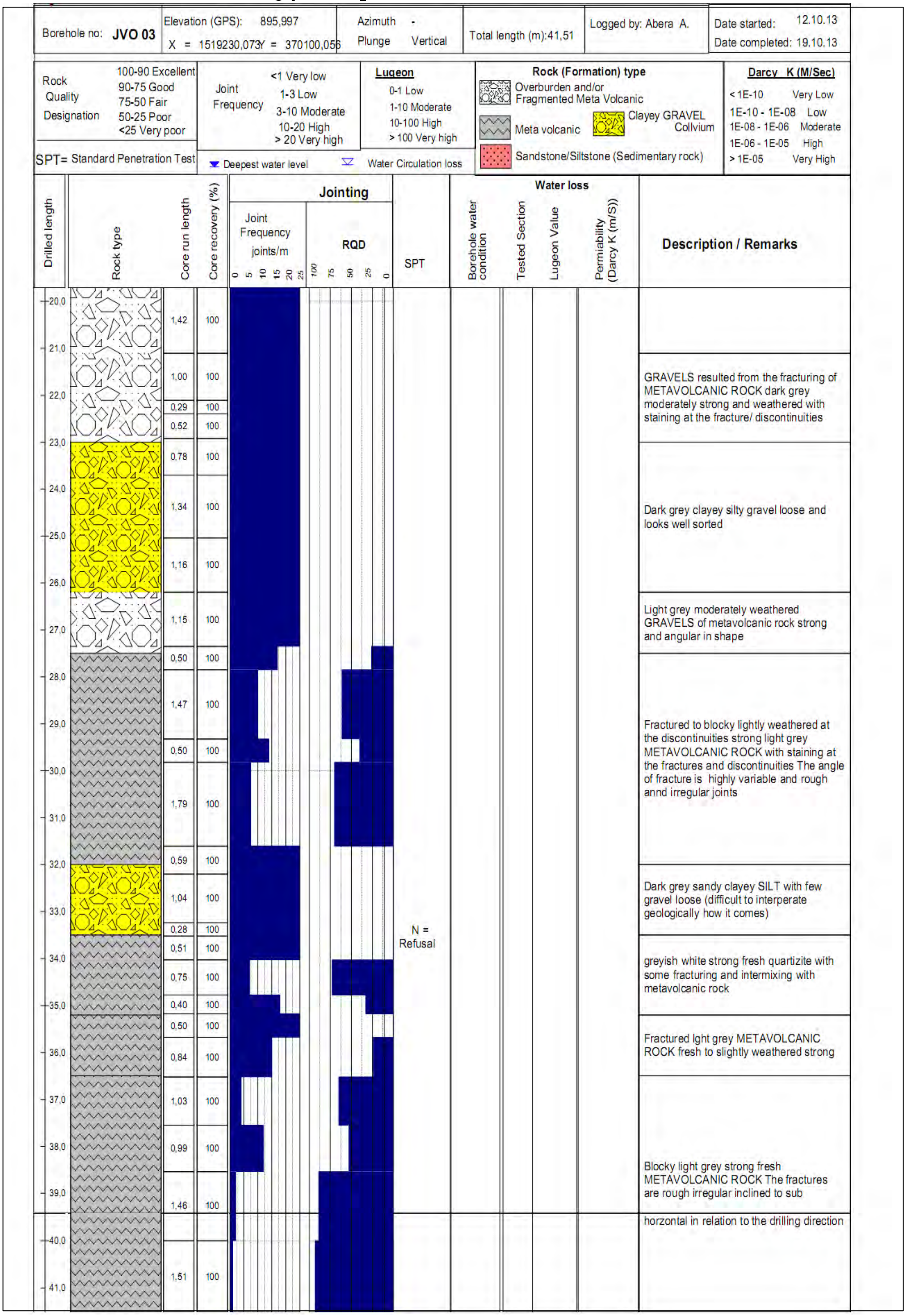
JVO.2: Geotechnical core log from depth 20 to 47m



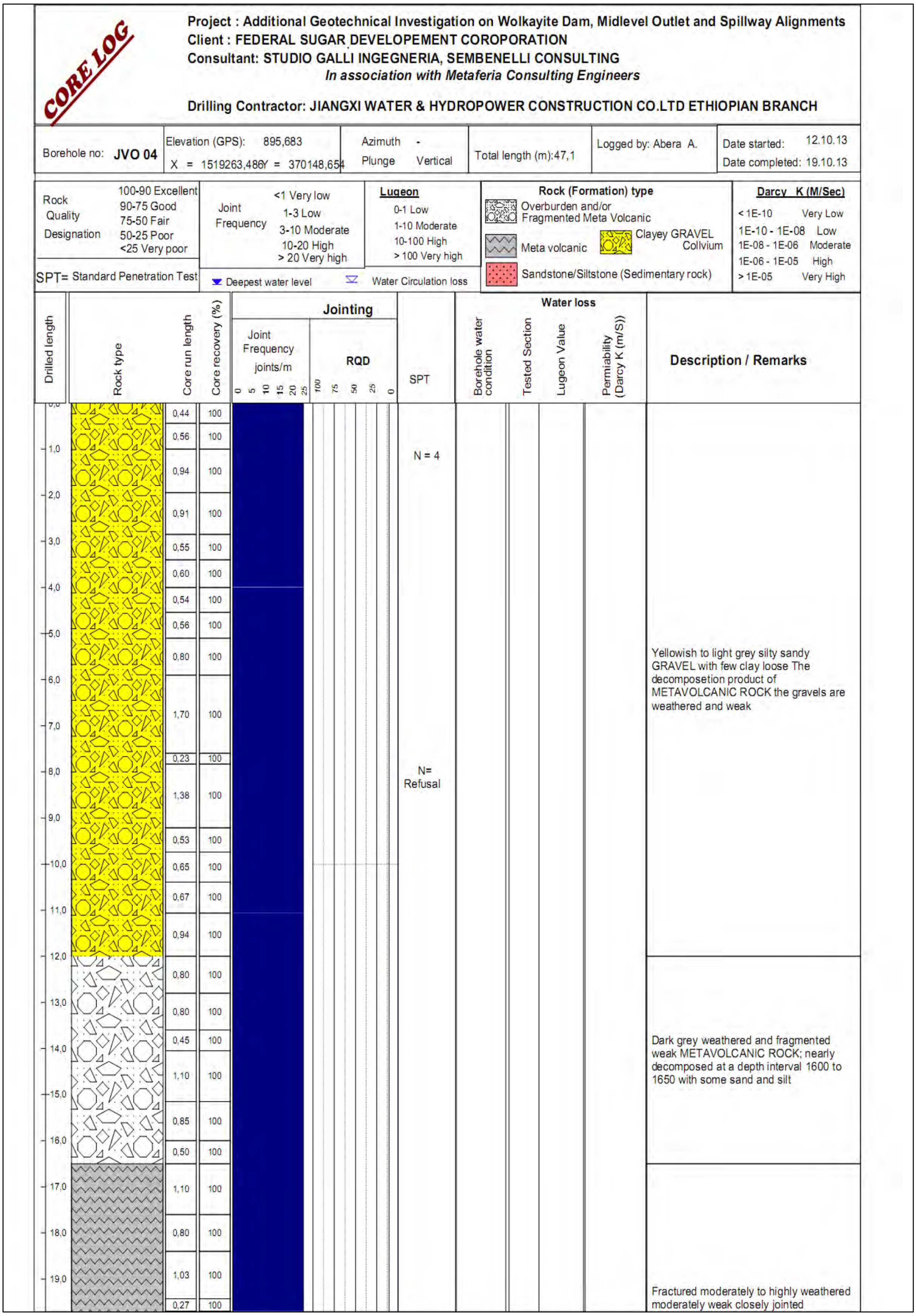
JVO.3: Geotechnical core log from depth 0.00 to 20m



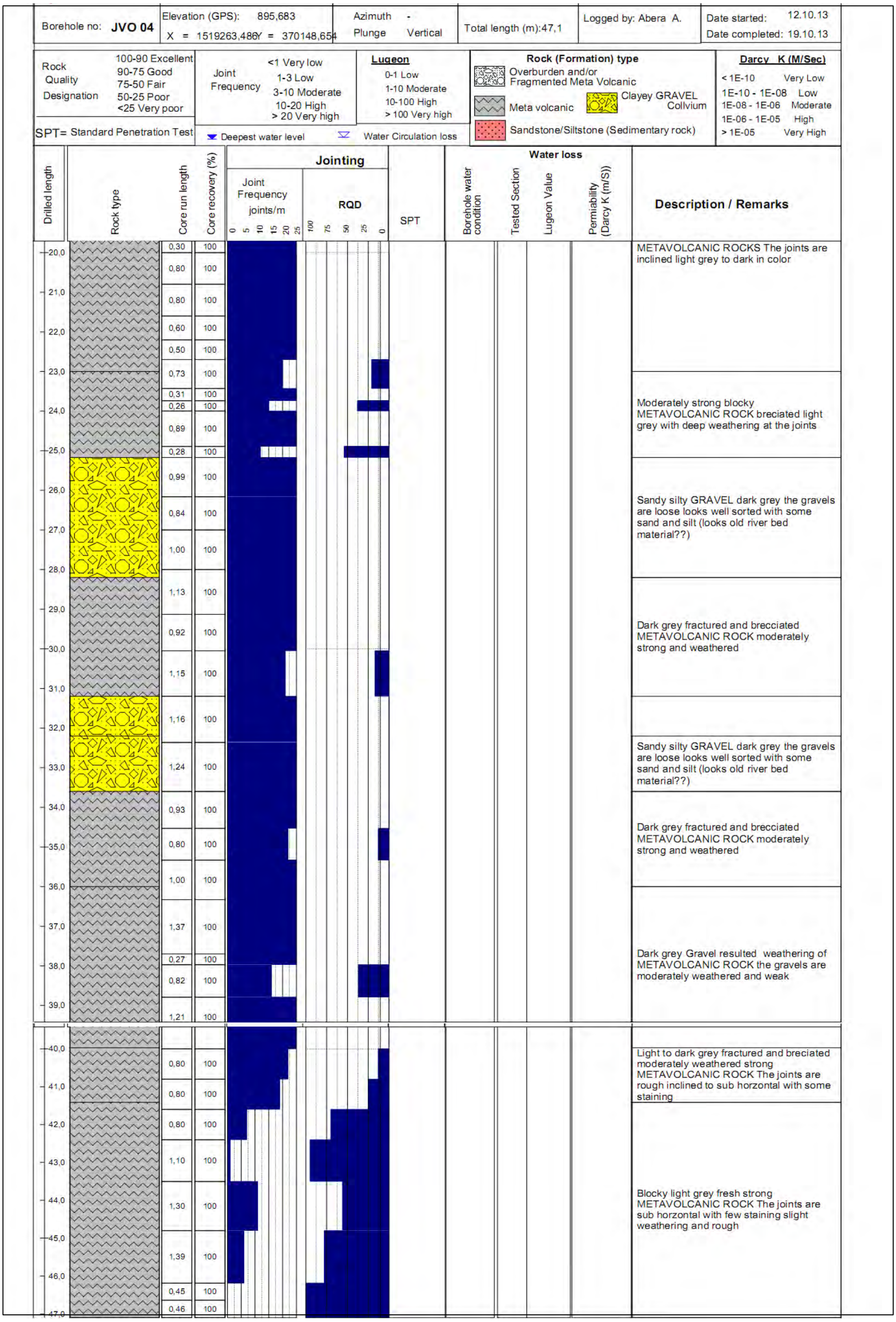
JVO.3: Geotechnical core log from depth 20 to 42m



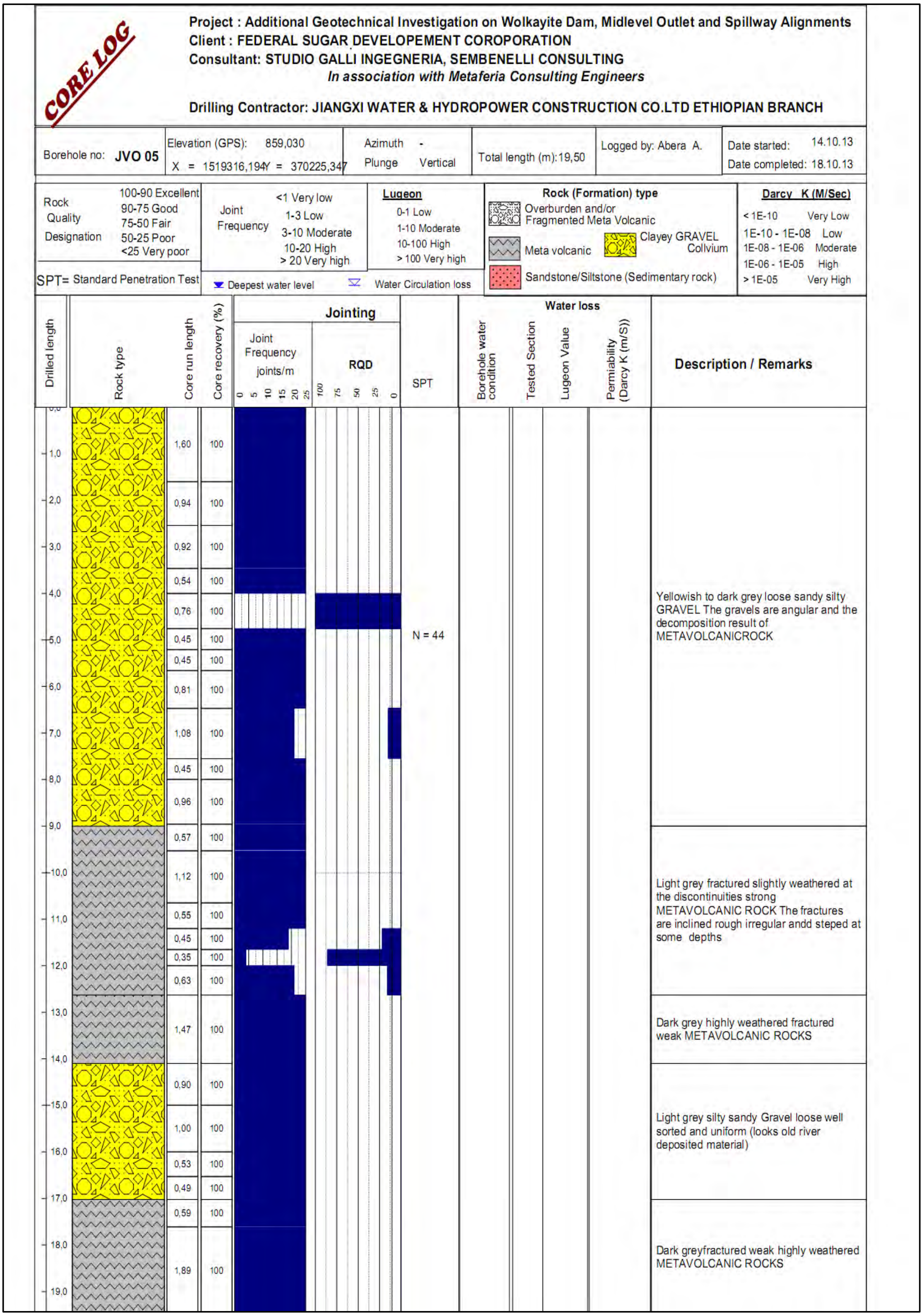
JVO.4: Geotechnical core log from depth 0.00 to 20m



JVO.4: Geotechnical core log from depth 20 to 47m



JVO.5: Geotechnical core log from depth 0.00 to 19m



Appendix -3: Descriptions of geostructural survey conducted at the study area

Location	Major Joints Characteristics	Geological Description
Station.1: tunnel outlet portal area. E.0370185, N.151931	Shear zone with 5-6m width & orientation $48^0/030^0$	The rock masses in the shear zone is highly fractured & weak, friable by finger, and field strength tests indicating that UCS is below 10Mpa
Outlet Portal: right side of the shear zone/ station.1	There are two set of joints with orientations 1. $81/100^0$ with 5-20cm spacing 2. $75^0/030^0$ The joint surfaces are smooth to planar	The meta volcanic rock is yellowish grey to greenish grey, fine grained, moderately to highly weathered with intact rock strength ranging from 10Mpa (for highly weathered rocks) to 25Mpa(for moderately weathered rocks)
Station.2: 70m from station 1 towards Zarima river E.0370194, N.1519238	Closely jointed rock masses with 2-10cm joint spacing. Three joint sets are observed with orientation 1. 72/335o 2. 80^0/030o 3. $32/200^o$ In general the joint surface is slightly rough to smooth planar	The rock mass is moderately weathered with intact rock strength of 12 to 25Mpa , and the joint surface is smooth to planar
Station.3: 100m towards coffer dam from station.2	Jointed rock masses with joint spacing of 3-10cm , slightly rough & planar joint surface The orientations of the joints are 1. $72/355^0$ (5-10cm spacing) 2. $58/330^o$ (with 20cm spacing locally) 3. $85/340^0$ 4. $36/180^0$ 5. $30/210^0$ 6. $25/200^0$ (20cm to 1m spacing) The joints are sometimes filled by 0.5cm thick quartz. The rock mass is easily friable by finger Some 50m towards coffer dam from station.3, 30cm thick shear zone is observed with orientation of $62^0/330^0$. Characterized by fractured rock and quartz mineral filled	Meta-volcanic rock: greenish grey to yellowish grey in color, fine grained, easily broken by finger pressure (5-10Mpa), highly to moderately weathered rock. The weathering changed the intrinsic joint to mechanical joints. RMR Parameters; Rock Strength 5-12Mpa Joint Spacing 3-10cm and >20cm Joint Opening 1mm Weather; highly weathered Roughness; .slightly rough to smooth Dry condition in groundwater conditions
Station 4: GPS Locations E:0370149, N:1519092	1. $68^0/340^0$ 2. $74^0/360^0$ (3-10m spacing) 3. $780^0/350^0$ 4. $26^0/200^0$ 5. $22^0/220^0$ (10cm to 30cm) 6. $27^0/210^0$	Meta volcanic rock; yellowish to greenish grey in color, fine grained, highly weathered with intact rock strength of 5-12Mpa
Geostructural survey conducted in between station.5 & station.5 orientation of joints	Orientations of the joints 1. $82^0/340^0$, $85^0/350^0$, $83^0/360^0$ 2. $75^0/290^0$, $70^0/300^0$ 3. $27^0/190^0$, $22^0/210^0$	
Station.5: dam key trench area at ch.0+370: E:0370063, N:1518963	Closely to moderately jointed rock masses with joint spacing of 20-50cm and orientations 1. $75/350^0$,$85/340^0$,$70/360^0$,$85/355^0$,$72/350^0$	Meta volcanic rock; yellowish to greenish grey in color, fine grained, fresh to slightly weathered with rough & planar joint surface; RMR Parameters Strength: 50-70Mpa ROD: 70-75%

	2. 85/090^0, 87/100^0,82/070^0,85/065^0 3. 27/150^0,28/170^0,22/150^0,23/200^0,18/190^0	Joint spacing: 20-50cm Weathering: slightly to fresh Roughness: rough & planar Ground water: Dry conditions. Joint opening about 1mm
Station.6; Top of dam key trench excavation area; E:0369904 and N:1519035	Shear zone with orientation; 52^0/300^0	Shear zone with 7-10m width characterized by highly fractured & weathered rock masses
Station .7:top of dam key trench, near spillway excavation area E:0370004, N:1519035	Sheared & fractured zone with 10m width & orientation of 60^0/290^0	Highly weathered and fractured sheared zone
Towards spillway excavation, the joint orientations	1. 78^0/340^0 2. 22^0/190^0,27^0/180^0, 3. 68^0/350^0 The dominant joints are rough & undulating with 10cm to30cm joint spacing, tight & partially filled with hard minerals.	Mata- volcanic rock, greenish grey to yellowish grey, fine grained, moderately to slightly weathered, medium strong rock
Station.8; Top of dam key trench excavation area, E:0369959 and N:1519072	Major joint orientation 1. 86^0/350^0(10-30cm spacing 2. 22^0/170^0 (50cm-1m spacing) The joint surfaces are smooth & planar	Meta volcanic rock, greenish grey, fine grained, fresh to slightly weathered with intact rock strength of 60-70Mpa, joints are partially open and filled with quartz

Chapter 2 Geological and Tectonic Settings

2.1. Regional Geology and Tectonic Setup

The north-western Ethiopia is characterized by the presence of low-grade Precambrian Basement assemblages, Phanerozoic sedimentary rock successions, and various Tertiary volcanic rocks.

The Precambrian Basement rocks, generally a part of the northern Ethiopia Basement, include low-grade volcano-sedimentary assemblage with intervening mafic-ultramafic belts and basic to acidic intrusive rocks. According to the geological report of Ethiopia (Mengesha Tefera, Tadiwos Chernet and Workineh Haro, 1996), the northern Ethiopian basement rocks are classified into six tectono- stratigraphic blocks; namely, Shiraro, Adi Hageray, Adi Nebrid, Chila, Adwa, and Mai Kental from west to east. The Adi Hageray block consists of low-grade (green schist facies) meta-sedimentary and meta-volcanic rocks; whereas the Adi Nebrid block contains such lithologic components as basic–intermediate meta-volcanics, meta-pyroclastic, and associated volcanoclastic meta-sediments. The Zager mafic-ultramafic belt occurs between the Adi Hageray and Adi Nebrid blocks and is reported to extend for considerable distance.

The present study area, Zarima dam site, is located to the south of the Adi Hageray and Adi Nebrid tectono-stratigraphic blocks to the south of Tekeze River. As such, the metamafic-ultramafic rocks and associated meta-volcanics encountered in the project site and surrounding areas can be interpreted as the southern extension of the Zager mafic-ultramafic belt.

The Phanerozoic sedimentary succession including sandstones, siltstones, and conglomerates mapped along the Tekeze valley and its major tributaries are considered as Adigrat Sandstone. This sedimentary succession is overlain unconformably on the Precambrian basement rocks of the study area (Hailu Tefera, 1979, and Kebede Kidane, 1982; as cited in WWDSE, 2011).

According to the site geological map presented in the feasibility report of WWDSE and ELC (2012), the Tertiary volcanic rocks in and around the project area includes Ashange basalt, Aiba basalt and Alaji basalts and rhyolites, which are outcropped on top of the sedimentary rocks of the project area (Fig 2.1). Ashange represents the oldest flood basalts in north and northwest plateau unconformably overlying the Mesozoic sedimentary succession and probably the Nubian sandstones. The porphyritic-aphanitic basalt and amygdaloidal/ vesicular basalts occur in the project area are part of Ashange flood basalt.

Structurally the study area is located in southern part of the Arabian Nubian Shield in the East African Orogen; and as such the Precambrian basement rocks, which include the low-grade metavolcano-sedimentary assemblage with ophiolitic mafic-ultramafic belts and basic – acidic intrusive rocks, are characterized by shear zones, thrust faulting, and different folding with overprinting (Tadesse Takele, 1997). However, the structures particularly the shear zones and thrust faults were reactivated during the Phanerozoic uplifting of the east Africa as a result of which considerable thicknesses of the Paleozoic-Mesozoic sedimentary rock successions and Tertiary volcanic sequences with rare inter-volcanic sedimentary deposits were formed, uplifted, and tilted at places. The sedimentary successions and volcanic sequences then are affected by the reactivated older Precambrian structures as well as Phanerozoic structures (WWDSE and ELC, 2012). As such the study area is out of the Main Ethiopian rift system.

Landsat image of the northern Ethiopia indicate extension of structures found to occur around the Lake Tana area to the project area. These structures generally trend north-south and NNW-SSW and caused structural depression/trough that formed the Wolkayite low-lying area, where the entire irrigation command area and the reservoir area are located (WWDSE, 2011). As can be observed from Figure 2.1, these structures are intercepted by other fault/ fracture sets mainly trending northeast-southwest and ENE-WSW to ESE-WNW producing a characteristically rugged topography in the catchment areas of the Zarima and adjoining areas. As such the area is denuded due to the interplay of the underlying geological structures (which controls the overall geomorphology), climate, weathering, erosion, etc.

2.2. Local Geology of the Study Area

The study area is underlain by the Precambrian Basement rocks in the narrow valley portions of Zarema River, and followed by Phanerozoic sedimentary succession and Tertiary volcanic rocks in the plateau area (Fig. 2.2). According to the field geological mapping conducted in the present study and feasibility report of WWDSE and ELC (2012), the local geology of the project site, from old to young, are described as follows.

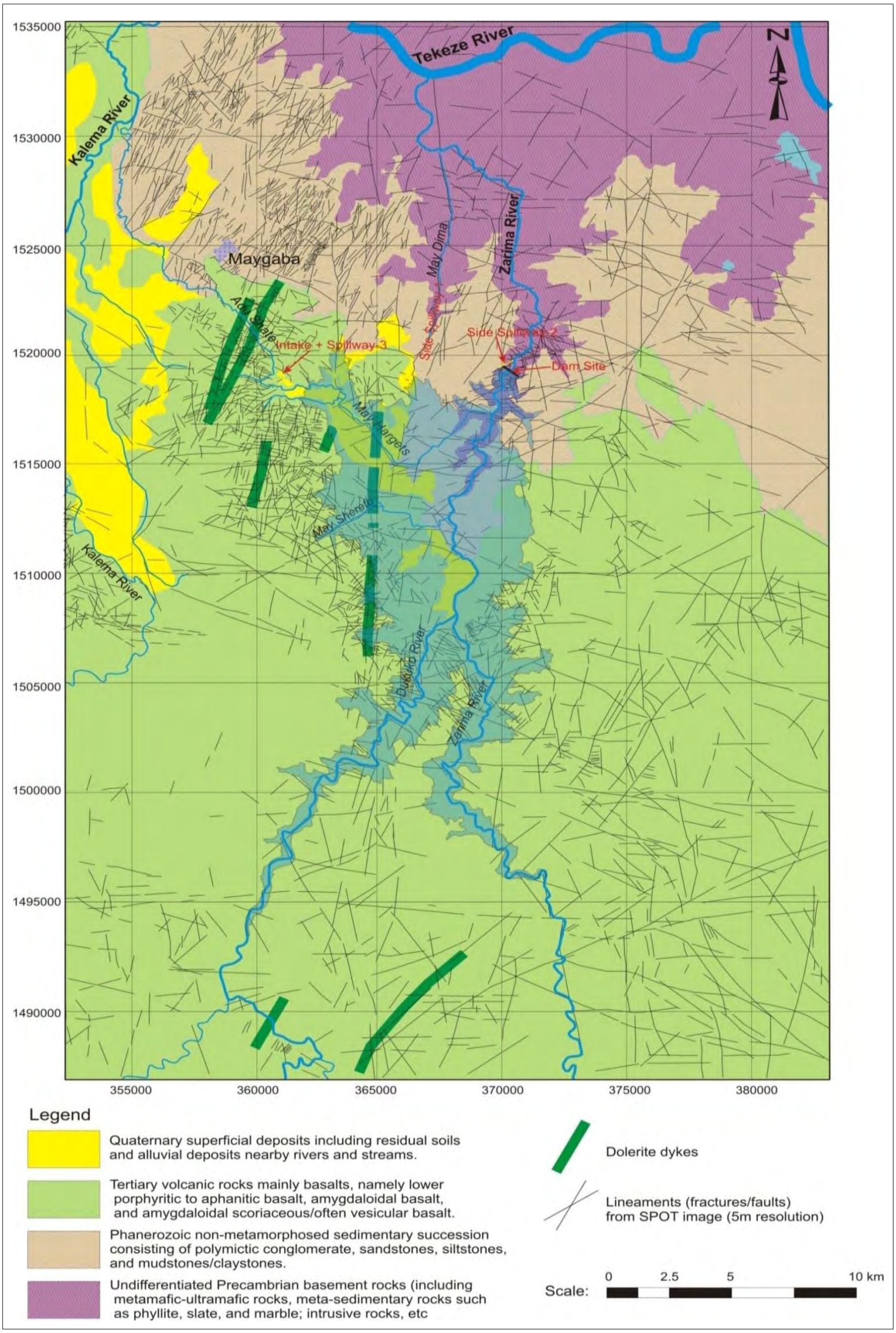


Figure 2.1: General geological map of the project site and surrounding areas (After WWDSE and ELC, 2012)

2.2.1. Precambrian Basement Rocks

The Precambrian basement rock exposures occur in Zarima River valley and blanketed by the Phanerozoic sedimentary rock succession away from the valleys (Fig. 2.2). As observed at the road cut, left abutment key trench area, the flat-lying sedimentary rocks are separated from the underlying Precambrian basement rocks by unconformable contact zone of variable thickness. The unconformable contact zone is marked by loose unconsolidated material, which grades to weathered basement rocks.

The abutment slopes are overlain by colluvial deposits of variable thickness, which is thicker in the depression areas and thin on relatively steeper and ridge forming parts. At depth the large portion of dam site is underlain by meta-basic/ultrabasic rocks intercalated with minor meta-sediments such as banded ironstone, which at places grades to ferruginous sandstones or quartzite, calc-silicate rocks, and minor marbles as fracture fillings (SGI and SC, 2013). According to WWDSE (2011), the meta-basic/ ultrabasic rocks include greenstones mainly composed of epidote, chlorite, and amphiboles; serpentinites containing talc, meta-gabbros, and meta-pyroxenites. These rocks are briefly described as follows.

i. Meta-Ultramafic Rocks

The meta-ultramafic rocks including serpentinite, chlorite schist, thin slivers of talcose rock, and relict pyroxenite are found to occur in the dam site area mainly downstream and upstream of the dam axis at the left and right banks of the Zarima River. The general foliations/fabrics have wavy natures. The intensely deformed portions are largely transformed to schistose rock-chlorite schist.

The ultramafic rocks are at places affected by grain-size reduction due to deformation and associated mineral transformation/alteration and hence easily confuse with meta-basic volcanic rocks. Where relict minerals or rocks are not preserved and weathering intensity is high, demarcation between the meta-basic and meta-ultramafic rocks is difficult. Hence, the lithologic contacts between the meta-basic volcanic and meta-ultramafic rocks are approximate as based on exposures observed at limited localities.

Moreover, meta-ultramafic rock exposed along access road cut on the left side slope (UTM coordinate 370077 E and 1519232 N) seems to contain talc and requires to be verified during excavation. The rock is affected by inhomogeneous deformation as rocks are in variably deformed and metamorphosed to show undeformed or slightly deformed to strongly deformed varieties even in a single exposure (Fig. 2.1 and Fig.2.2).

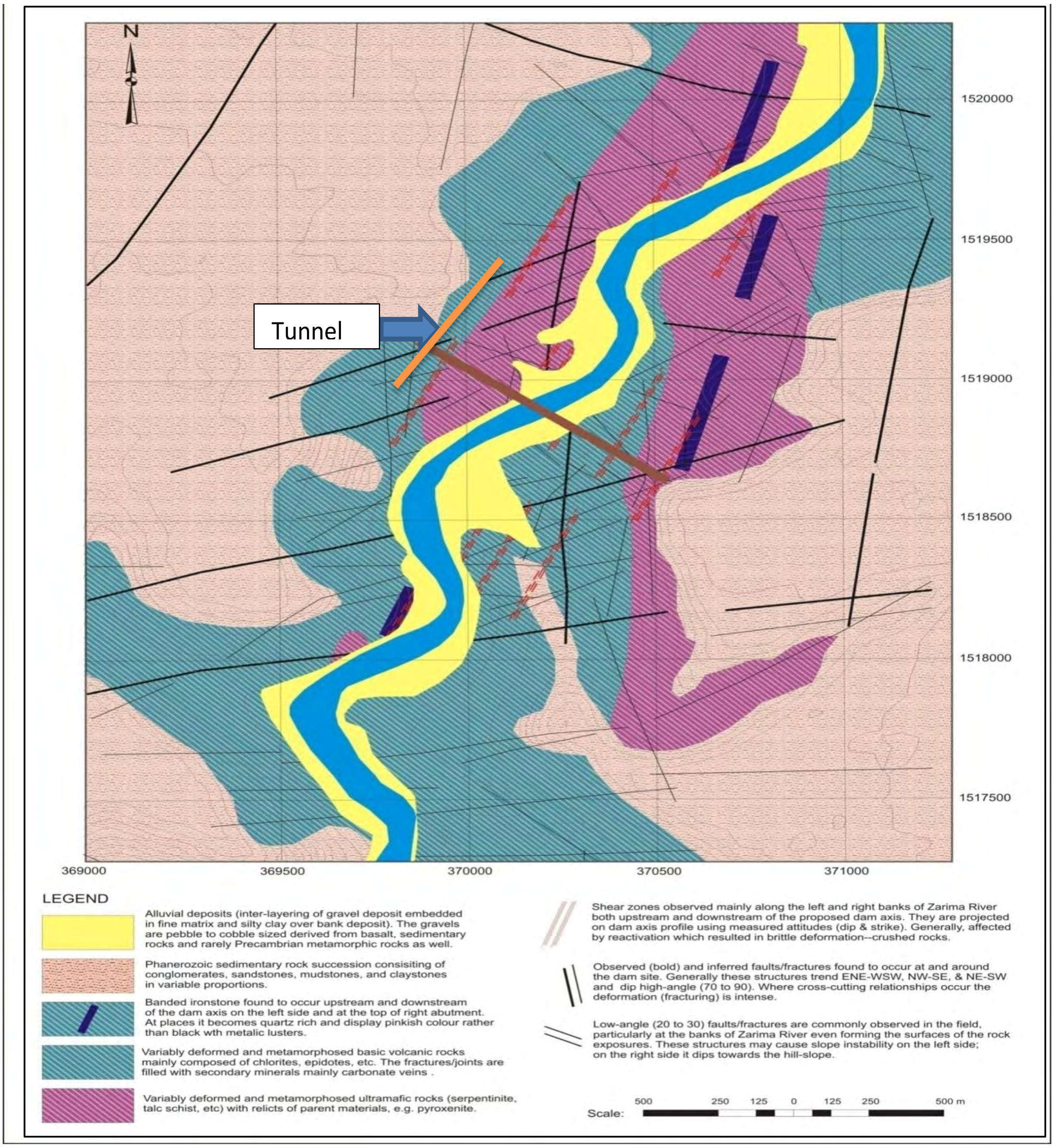


Figure 2.2: Local Geological Map of the Study Area (Modified after WWDSE, 2011)

ii. Meta-Volcanic/Meta-Basic Rocks

The meta-basic volcanic rocks occur extensively in the tunnel site area. Generally on the slopes along the tunnel route the rocks are covered by thick colluvial deposits and extremely weathered equivalent.

These rocks are fine-grained, pale green rocks mainly composed of chlorite, plagioclase, epidotes and probably actinolite. The sheared portions are characteristically epidotized and display a yellowish green color. The epidote minerals crystallized in the shear zones (ductile deformation) are later reactivated (brittle deformation) to produce dilatation cracks/ fractures commonly encountered in the area (Fig. 2.2).

iii. Meta-Banded Ironstone

The meta-banded ironstone is encountered at about 740 m upstream and 1050 m downstream of the dam axis on the left side of Zarima River (Fig. 2.2). Similarly, the rock is encountered at access road cut on the right side towards top of the dam axis and downstream at the right bank of Zarima. The rock is dense (heavy) with metallic luster at places grade to quartz rich varieties; the presence of the meta-banded ironstone is usually marked by occurrence of ubiquitous blocks distributed every which way. The quartz-rich varieties display alternating bands of black and pinkish layers. This unit may represent tectonic slivers within the meta-basic volcanic and meta-ultramafic rocks.

As such it can be considered as a marker layer to infer structural displacements. As observed at the right side slope downstream of the dam axis (UTM coordinate 370540 E and 1518704 N) the banded ironstone trends north-northeast parallel to the general foliation and dips 80/116^{o}.

2.2.2. Phanerozoic Sedimentary Rocks

The Phanerozoic sedimentary successions are found to occur in the northern-half of the mapped area overlying the Precambrian basement rocks (Fig.2.2.). The left and right abutments are characterized by relatively steep slopes. The top of the proposed tunnel route above el. 960 m is covered by Phanerozoic sedimentary successions consisting of such lithological components as basal conglomerate, inter-layered sandstones, siltstones, and mudstones. The mudstones which generally form thick bedding are dominant among the sedimentary rocks.

In the study area the sedimentary rocks successions which consist of a polymictic basal conglomerate, sandstones, siltstones, mudstones, and clay stones unconformable overlie the Precambrian meta-basic to meta-ultramafic basement rocks (Plate 2.1). As observed at top of the spillway excavation left side slope face the base of the sedimentary rock succession is formed by polymictic conglomerate and followed by sandstone that grades to fine sandstone and siltstone, light gray mudstone, and reddish brown mudstone/clay stone (Plate 2.1).

These sedimentary rocks lie nearly horizontal; however, dipping bedding surfaces (up to 2° towards west and southwest) are observed elsewhere in the area. Furthermore, bedding parallel faulting mainly encountered in the reddish brown mudstone/clay stone dips gently towards south west of the study area as measured elsewhere, for instance along spillway excavation area. Toppled blocks of the sedimentary rocks mainly the breccia/conglomerate are abundant on the slopes forming scree/ colluvial deposits and also in the creeks draining to Zarima on both left and right sides.

2.2.3. Superficial Deposits

The superficial deposits encountered at the study area are generally colluvial/ scree deposits on both slopes of Zarima valley, in situ weathering products of the basement rocks and alluvial deposits along the banks of Zarima River and its tributaries; and are briefly described as follows.

i. Colluvial Deposits

Almost all area mapped as metamafic-ultramafic rocks (Fig. 2.1.) is overlain by colluvial deposits of variable thicknesses. The colluvial deposits are mainly composed of reddish lateritic soils and yellowish soils; both of which contain embedded rock clasts of variable sizes (sand to boulder), which are generally composed of non-metamorphosed sandstones and metamorphic rock fragments ranging in size from large blocks to sand and gravel. The thicknesses of the colluvial deposits vary from place to place and as such ranges from 0 to 8 m (Plate 2.2a,b), which is thinner where the slope is steeper. The small creeks and topographic lows joining Zarima from both left and right sides have thick colluvial deposits with large debris of sandstones and conglomerates.

ii. In-situ Weathering of the Basement rocks

The meta-basic volcanic and meta-ultramafic rocks are generally composed of ferromagnesian minerals which are susceptible to alteration and chemical decomposition. Hence, the depth of weathering is presumed to be deeper and becomes more aggravated by the presence of structural defects such as joins/ faults. In deeply weathered meta-ultramafic and meta-basic volcanic rocks at places the fabrics of the original rock are preserved but tilted due to creep movement; and as such care was taken while taking measurements of attitudes (dip and strike) of planar structures.

Where the rocks are sheared or intensely crushed and cross-cutting relationships are observed at the depth and intensity of weathering are greater. Extremely weathered zone without traces of the parent rock is encountered at UTM coordinates 369676 E & 1519018 N and continued to 369656 E & 1519001 N for a width of about 25 m. These zones seem to be fault zones filled by gouge - highly pulverized almost clay material.

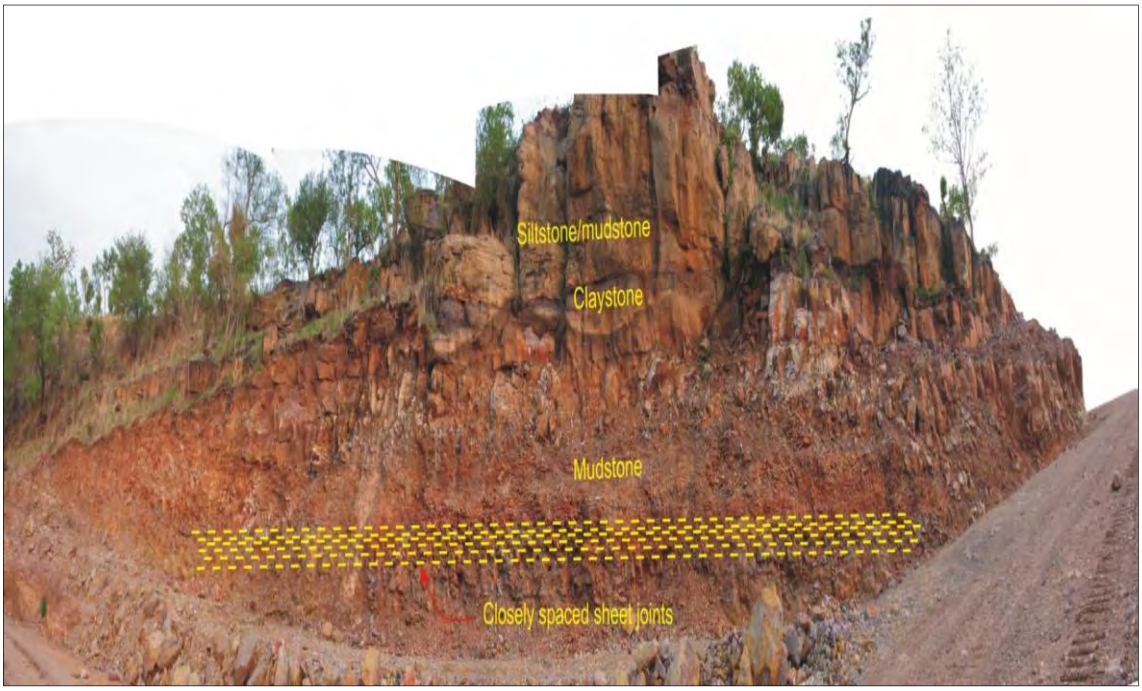


Plate 2.1: Sedimentary rock successions at the top of the left abutment of Zarima Dam Project

iii. Alluvial Deposits

The dam is located at a relatively straight portion of the Zarima channel; which at the dam site is about 90 m wide and is underlain by gravel deposits consisting of mainly well rounded pebbles and cobbles with rare boulders derived from the Tertiary volcanic rocks, sedimentary succession and Precambrian basement rocks as well. Away from the river banks old alluvial deposits (terrace deposits) consisting of mixtures or inter-layering of well-rounded gravel deposits and silty/ sandy clay soils overlie the meta-mafic/ultramafic rocks generally encountered at both the left and right banks of Zarima River. Extensive and thick (up to 7 m) alluvial deposits occur downstream and upstream of the dam axis on the left and right sides, respectively, at concave portions of the Zarima river channel (Fig. 2.2).

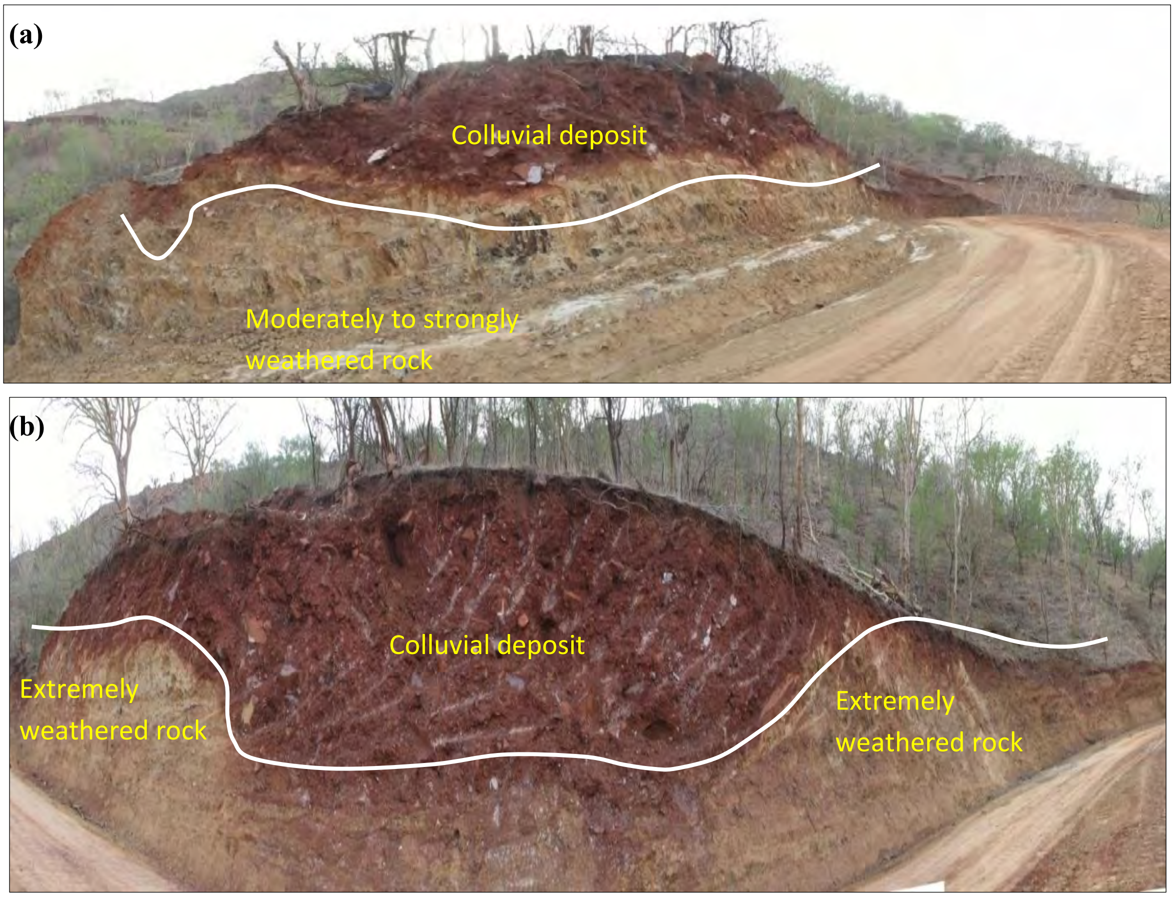


Plate 2.2: (a) and (b) Lateritic colluvial deposit as exposed along the access road cut. Note that the thicknesses of the colluvial deposits considerably vary from place to place.

2.3. Geological Structures

The study tunnel site and surrounding areas are underlain by Precambrian basement rocks comprising basic meta-volcanic rocks, meta-ultramafic rocks and minor banded meta-ironstone at places rich in quartz; Phanerozoic sedimentary rock succession, and superficial deposits (colluvial and alluvial deposits). The tunnel alignment is proposed along the middle part of left abutment and will be fully passed through meta-volcanic rocks. The geologic structures associated with these rock units of the study area are described as follows.

i. Foliations

In the study area, the general foliation of the basement rocks trends NNE-SSW and generally dips sub-vertical towards ESE; however, at places the foliation swings and dips towards west-northwest as well. The basement rocks have experienced inhomogeneous deformations which resulted in variable grain-size and schistosities of the rocks. The variations are even observable on single outcrops at the banks of the Zarima River, where fresh rock exposures are preserved.

Two foliation surfaces were encountered in the area (Plate 2.3). The older foliation surfaces are folded and obliterated by later deformation events. The second and younger foliations generally trend north-northeast (N10E to N30E) and dip at high angle (>75°) towards ESE. However, at places the foliation swings and as such WNW dipping surfaces were observed. These foliations/schistosities as present in the basic meta-volcanic rocks and meta-ultramafic rocks define the regional or general structural features in the study area. The shear zones encountered at the proposed tunnel site are oriented parallel to the regional structural features suggesting contribution of the shearing deformation in developing the later foliation surfaces.

The structural measurements of the weakly foliated surfaces indicate moderately to steeply dipping (55° - 87°) towards both southeast and northwest. Few measurements also have NNW strikes with dips of 60° - 80° towards northeast. The rose diagram of the foliations is shown in Figure 2.3 indication major trend in NE-SW and minor in NNW-SSE directions. The central part of the tunnel sections are oriented sub parallel to the strike of these foliation structures

Plate 2.3.: Contorted/ folded older foliation surfaces encountered in the mafic-ultramafic rocks.

ii. Shear Zones

The Precambrian basement rocks are generally affected by ductile shear zones as encountered along right and left banks of Zarima River and access road-cuts along the dam axis (Fig. 2.2). According to WWDSE (2011), the NNE trending Precambrian shear zones, which initially underwent ductile type of deformation, were later reactivated to produce more brittle deformation as observed in more fresh rock exposures along both the left and right banks of the Zarima River. It is noted that later reactivation of the shear zones coupled with cross-cutting relationships produce shattering of the rocks. Such shattered and intensely fractured zones will become relatively weaker, susceptible to mechanical and chemical weathering which may result in partial to complete decomposition of the rocks (Plate 2.4).

The basic meta-volcanic rock exposures encountered at top of left bank dam key trench excavation area show cross-cutting relationships of various fractures (Plate 2.4) which are gradually dominated by the N20E striking shear zone, which is about 5 - 7 m wide. The shear zone displays ductile to brittle deformation features. These structures are intersected by nearly east-west trending fault/fracture set that consistently dips (70° to 75°) towards NNW to N and a low angle (15° to 30°) fault/fracture set trending ENE which generally dips towards SSE to S at both the left and right abutments.

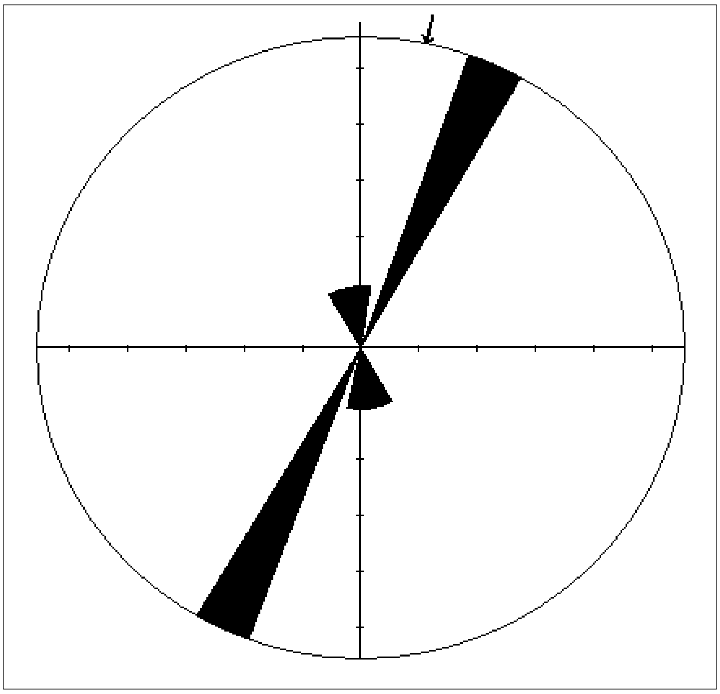


Figure 2.3: Rose diagram of foliations in the Precambrian Meta Mafic-Ultramafic Rocks

Plate 2.4: The rock mass found on top of left abutment key trench excavation area is affected by shear zone and weathering actions

iii. Faults and Fractures

East-West and ENE-WSW trending faults/fractures: As commonly observed in the tunnel site East-West to ENE-WSW striking fault/fracture set affected the basement rocks. This fault/ fracture set is prominent and generally dips 70° to 80° towards north-northwest. The fracture spacing varies from few centimeters-where the intensity of deformation is intense to several meters. These structures have cross-cutting relationships with foliations and shear zones described above.

The intersection areas of this fracture set with strongly deformed shear zones, later reactivated as brittle fractures, trending north-northeast produce strongly deformed crushed and fractured rocks. Such exposures are found to occur along the proposed tunnel route at left banks of Zarima River. Moreover, the East-West to ENE-WSE trending fractures/ joints are commonly encountered at the left and right abutment slopes along the access road-cuts. There are also other subsidiary structures striking NW-SE (N296) and dipping 80° due East, which are considered as pinnate structures associated with the ENE-WSW striking structures.

The ENE trending encountered at the top part of the left abutment extensively affected the basic meta-volcanic rocks. These fault fractures dip 70-78° towards north-northwest.

Moreover, the deformation is complicated by the occurrence of low-angle faults/fractures which dips towards South (Fig. 2.8).

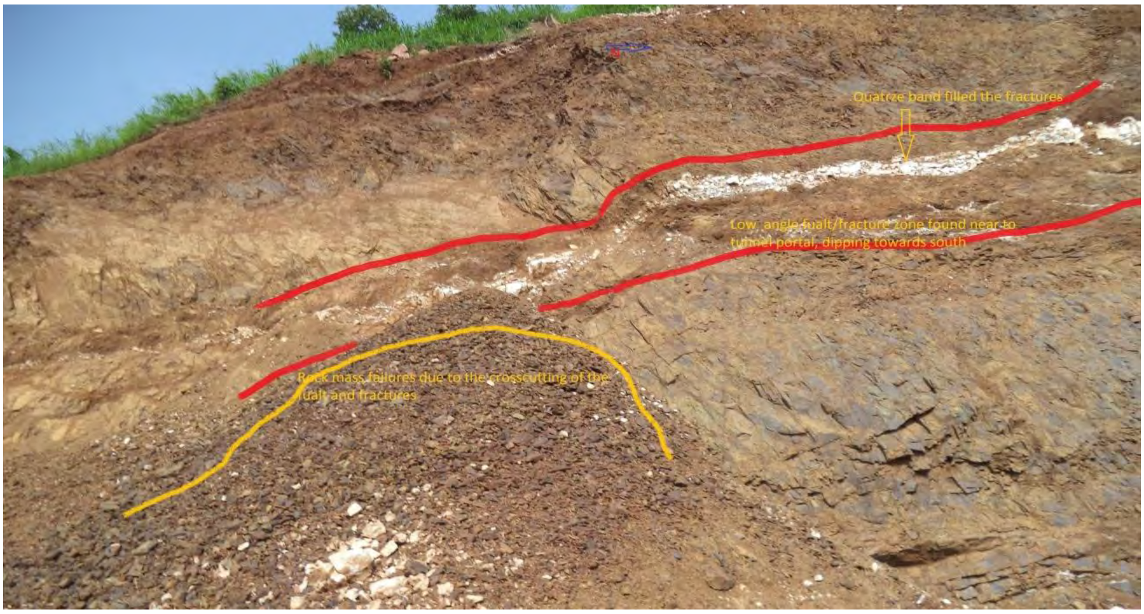


Plate 2.5: Low angle fault/ fracture zones encountered at the outlet tunnel portal area, dipping 20^0 to 25^0 towards south east.

Low-angle Faults/Fractures: Low-angle fault/fracture surfaces are commonly encountered in the Precambrian basement rocks exposed at and around the tunnel site. These surfaces generally dip 15° to 30° towards the south (165° to 206°). The low-angle planar structures display recrystallization of secondary minerals such as quartz (Plate.2.5). These fractures dip towards the valley on the left side and dip in to the slope on the right side of Zarima River. As such these fault/ fracture surfaces may daylight on the left side slopes and their effect on slope stability at the tunnel portals has to be assessed to take necessary engineering remedial measures.

2.4. Seismicity of the Study Area

The Zarima Dam site is located approximately 185km from Mekelle town, thus more than 225 km far away from active rift margin. Figure 2.4 shows the location of the Zarima Dam site with respect to seismic hazard location of Ethiopia. Accordingly, the study area falls within Zone 0 of seismic hazard zonation of Ethiopia. From the figure 2.5, it results that almost all the epicenters of the Earthquakes occur near the broad western margin of the main Ethiopian rift and beneath the northwest plateau, with no much earthquakes recorded with in the surrounding area of the Zarima project site.

The peak ground acceleration derived from WWDSE and ELC (2012) and presented in table 2.1 confirmed that the study area is characterized by a low value of peak ground acceleration (PGA) ranging from 0.06 to 0.074g for soils in a return period of 50 years. This is probably due to the far location of the Zarima Dam Site from the active rift margin, and the Red Sea category or western Ert Ale is the one contributing most to the seismic hazard at the study area

Table 2.1 Ground motion amplitude determination for rocks and soils at Zarima Dam Site (Source WWDSE and ELC, 2012)

Return Period in Years	Ground Motion Amplitude in % of g					
	Period = 0.2 sec		Period = 1.0 sec.		Period = 2.0 sec	
	Rock	Soil	Rock	Soil	Rock	Soil
50	4.587	6.686	3.192	3.869	2.338	2.731
100	4.976	7.446	3.490	4.259	2.545	2.997
200	5.398	8.293	3.816	4.689	2.771	3.290
500	6.06	9.562	4.294	5.323	3.101	3.721
1,000	6.521	10.65	4.695	5.86	3.377	4.084

In conclusion the results of the seismicity study excludes that the area of Zarima dam site is subject to a significant seismic hazards.

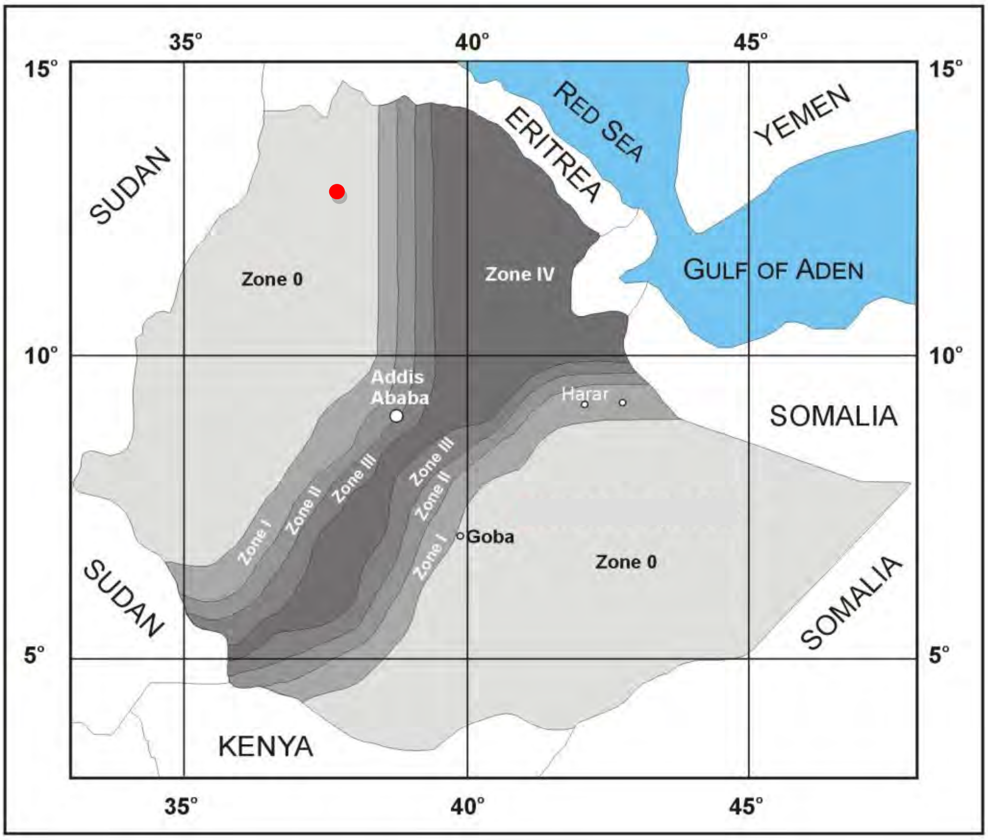


Figure 2.4: Seismic Hazard Zonation of Ethiopia (Source EBCS 8, 1995); the red circle shows the location of the study area

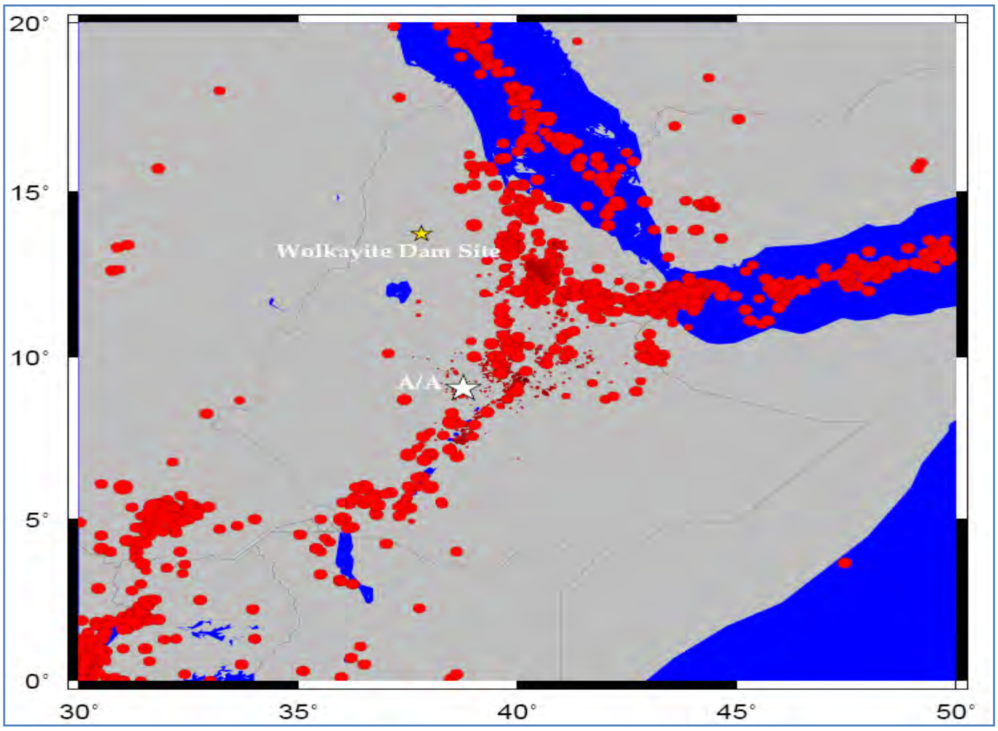


Figure 2.5: Seismicity for the Horn of Africa; red circles represent earthquakes that occurred for the last 110 years in the region and size of the circles is proportional with magnitude (Modified after AAU-Geophysics Observatory, 2012). The yellow star shows the location of the Zarima Dam Site and the white star shows the location of Addis Ababa.

2.5. Summarized Geological Setup of the Tunnel

Broadly the northern and northwestern Ethiopia is underlain by low-grade Precambrian basement assemblages, Phanerozoic sedimentary rock successions and various Tertiary volcanic rocks (WWDSE, 2011). The regional geology of the study area comprises the following major units from old to youngest:

- ✓ The low-grade Precambrian basement assemblages
- ✓ Phanerozoic sedimentary succession
- ✓ The Tertiary volcanic rocks

In the study area, the Precambrian Basement rock which consist of Meta-volcanic and Meta-ultramafic rock exposures occur in Zarima River valley and blanketed by the Phanerozoic sedimentary rock succession away from the valleys. The sedimentary rocks are separated from the underlying Precambrian basement rocks by unconformable contact zone of variable thickness. The unconformable contact zone is marked by loose unconsolidated material, which grades to weathered basement rocks.

The geology through which the proposed tunnel will be excavated comprises mainly low grade metamorphic units of meta-volcanic rocks underlain by the meta-ultramafic rocks. These rocks are blanketed by variable thicknesses of colluvial deposit and extremely weathered rocks. The field rock strength tests reveal that the intact rock strength of the rocks is reached to maximum of 70Mpa in fresh state and the strength decreases to 10Mpa as the degree of rock mass weathering increases.

In general, the rocks are affected by shear zone and brittle deformations. The shear zones strike N14E to N23E and dipping vertical to sub-vertical to southeast and the brittle structures trend nearly east-west and dip in opposite directions resulting in detached rock slabs. Accordingly, the low-angle faults/fractures, similar to that discussed in section 2.3.1, dip 22/183^{o} to 30/176^{o}, whereas the high-angle faults/fractures dip 73/352^{o}. There is also another set of fault/fracture dipping 44/047 to 50/042^{o}.

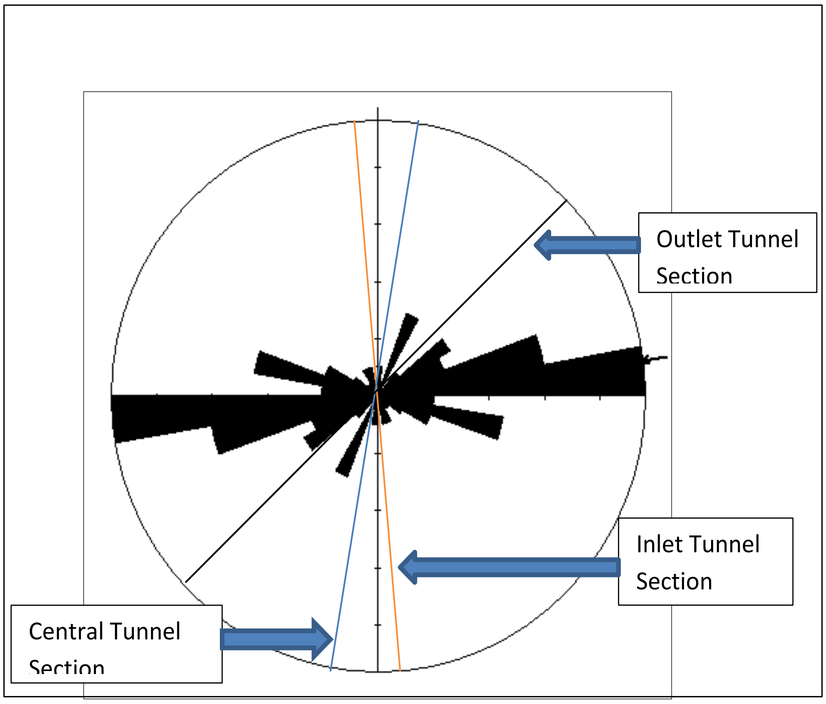


Figure 2.6: Rose diagram of planar structures (including foliations/shear zones and fractures) encountered along the tunnel alignment. The tunnel will be excavated in three directions; inlet tunnel section (SSE - NNW), central tunnel section (SSW – NNE) and outlet tunnel section (SW – NE)

The tunnel alignment is oriented oblique to the strike of these geologic structures. Rose diagram (Fig. 2.6) of all planar structures such as shear zones, foliations and fault/fractures measured at and around the tunnel site show that the ENE-WSW fracture set is quite dominant. Moreover, the northeast-southwest (foliation) and the WNW-ESE fractures are also important. Their orientations are fair to unfavorable with respect to stability of tunnel excavations in the study area. It is observed in the field that where these structures cross-cut complex fracture patterns develop. Such intense fracturing may result a significant decreasing of the engineering properties of the rock masses. Hence, thorough geotechnical site investigations including engineering geological mapping, rock mass characterizations and rock mechanics tests are suggested to be carried out in the present study and the details of the geological- geotechnical evaluations of the tunnel route are presented in section 4.

Furthermore, the assessment of seismicity of the study area display that the peak ground acceleration (PGA) is ranging between 0.06g and 0.074g for soil formation in return period of 50 years, which is considered as low value to produce significant seismic hazards in Zarima Dam site.

Chapter 3 Review of Literature Works

Tunneling projects should manage successfully the challenges exerted by the encountered range of geological difficulties. The nature of geological setting of a tunnel site has a major influence on the choice of construction methods, safety, and design and construction operation. Bell (1980) in discussing subsurface excavation and particular geological investigation for tunnels, regards geology as the most important factor in determining the nature, form and cost of a tunnel, in most situations the route design and construction aspects are mainly governed by the geological considerations.

Experience gained from several tunneling projects has enabled the designers of tunnels to have an improved understanding of the behavior of such constructions in different geological conditions. Geological factors, however, vary considerably from site to site, and even on the same site .Tunneling sites often exhibit some geological unique features (Sing, B. and Goel, R., 2006).

In ITA (2010), it has been stated that site investigations for tunneling projects lead to projected section of the geological setting through which the tunnel is planned to pass. Irrespective of the skill of the engineering geologist, exploration programs still involve interpretation and judgment, and consequently a degree qualitative and quantitative appraisal is involved. There is always likely to be a degree of risk in encountering unanticipated ground conditions in tunneling. The impact of such risks on tunneling progress, however, can be greatly reduced by careful planning, monitoring and reviewing of the ground conditions and related aspects.

History has witnessed the consequence of lack of appreciation of geological factors associated with tunneling projects from the design stage, through to construction and to eventual operation. Site investigation both as a preliminary stage and as simultaneous with construction provides the base for success of tunneling projects (Whittaker and Frith, 1990).

Several tunnel design methods are available but it would appear that empirical methods still have a major role to play in the modern underground construction industry. According to Hoek (2007), the empirical design approaches for shallow tunnels are basically derived from rock mass classification systems. The role of rock mass classification schemes has contributed significantly to the characterization of rock structures sufficient to give guidance on the selection of a tunnel supporting methods in both the short and long term.

The first step in designing of the underground structures is to evaluate the ground loads exert on the excavation openings. In Sinha (1989), it is stated that for underground structures, the most important loading comes from the host ground itself. In competent host ground, the rock loading on underground structures is quite insignificant, where as in incompetent rocks, it may be quite significant. The host ground rock pressure is dependent on several factors such as the relative stiffness of the structure and the host ground, the elapsed time between the excavation and installation of support, the characteristics of the host ground, the in-situ stress, the size of opening, the ground water conditions and adopted method of construction.

Several researchers have tried to characterize the rock and there by assess the rock load that they are capable of exerting on the tunnel support structures. As such, empirical rules constructed from experience records of satisfactory past experience have been produced. Empirical design rules for the selection of underground excavation supports are basically originated from the rock mass classification systems, namely;

- ✓ Rock Load Classification Method
- ✓ Stand-Up Time Classification
- ✓ Rock Quality Designation Index(RQD)
- ✓ Rock Structure Rating (RSR)
- ✓ Geomechanics Classifications(RMR-system)
- ✓ NGI- Tunneling Quality Index(Q-system)

These empirical design approaches produced base on the rock mass classification parameters of RMR and Q-systems are widely used in the modern underground construction industry, and a reviewed has been given on the applications of these rock classifications systems for the design of rock support of tunnels in civil engineering projects.

a) Geomechanics Classification (RMR**- system)**

The Rock Mass Rating system was developed by Bieniawski in South Africa in early 1972's and has been modified over the years as more data become available (Bieniawski, 1989). The classification is based on six parameters, namely;

1. Uniaxial compressive strength of rock material

2. Rock Quality Designation(RQD)
3. Spacing of Discontinuities
4. Conditions of Discontinuities
5. Ground Water Conditions
6. Orientation of Discontinuities

Details of the individual ratings are given in Bieniawski (1989) in a series of tables, the sum of which gives an overall RMR which has a maximum value of 100. Table 3.1 shows the rock classes and descriptions in relation to RMR value.

Table3.1: Summary of RMR rating related to rock mass conditions

Characteristics	RMR-89 Ratings
1. Strength of intact rock material (UCS)	0 to 15
2. Drilled core quality, Deere's RQD	3 to 20
3. Spacing of discontinuities	5 to 20
4. Conditions of discontinuities; Length(0-6), Aperture(0-6), Roughness(0-6), Infilling(0-6) and Weathering(0-6)	0 to 30
5. Ground Water	0 to 15
6. Orientation of Discontinuities	0 to -12

Conditions of Rock Masses					
Sum of Rating	81 - 100	61 - 80	41 - 60	21 - 40	0 -20
Class no.	1	2	3	4	5
Description	V. good rock	Good rock	Fair rock	Poor rock	Very poor rock
Average Stand-up time	20yr for 15m span	1yr for 10m span	1wk for 5m span	10hr for 2.5m span	30min for 1m span
Cohesion of the rock mass (kpa)	> 400	300 - 400	200 - 300	100 - 200	< 100
Friction angle of the rock mass (deg.)	> 45	35 - 45	25 - 35	15 - 25	< 15

The RMR concept has been used in a variety of applications including to obtain a class description of the rock mass, an average stand-up time (dependent upon unsupported span) and approximate values of the cohesion and friction angle for the rock mass under question (Table 3.1). For tunnels, information can be obtained on stand-up time and maximum stable excavation span for a given RMR value (Fig. 3.1) as proposed by Bieniawski (1989).

In terms of application of RMR system to tunneling operation, Table 3.2 provides guidelines for the selection of rock support for tunnels in accordance with the RMR system. This guideline has been produced depending on the factors such as in-situ stress, tunnel size and shape, and the method of excavation. It should be noted that the guideline is only applied to a

10m diameter tunnel excavated using drill and blast method, and no indication is provided as to how to extend this to other sizes of tunnel. In addition to tunneling, the RMR system has also applied to foundations, slopes, and mining operations.

Unal (1983; as cited in Bieniawski, 1989) has suggested an equation to determine the probable rock support load (P) exerts on underground structure based on RMR considerations and is expressed by the following formula;

$$P = \frac{100-RMR}{100} rB = rHt \quad 3.1$$

Where P = the support load in KN;

B = the tunnel width in meter;

H_t = *Rock load height in M, given by* $\boldsymbol{Ht} = \left(\frac{\mathbf{100-RMR}}{\mathbf{100}}\right)\boldsymbol{B}$ 3.2

$r = the\ rock\ density, kg/m^3$

Table 3.2: Suggested excavation and support of rock tunnels in accordance with RMR system (after Bieniawski, 1989); Tunnel shape: horseshoe: width = 10m; vertical stress <25mpa: construction: drill and blast

		Support		
Rock mass class	*Excavation*	*Rock bolts (20 mm dia. fully grouted)*	*Sprayed concrete*	*Steel sets*
Very good rock I RMR: 81–100	Full face. 3 m advance.	Generally, no support required except for occasional spot bolting		
Good rock II RMR: 61–80	Full face. 1.0–1.5 m advance. Complete support 20 m from face.	Locally, bolts in crown 3 m long, spaced 2.5 m, with occasional wire mesh.	50 mm in crown where required.	None
Fair rock III RMR: 41–60	Top heading and bench 1.5–3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.	Systematic bolts 4 m long, spaced 1.5–2 m in crown and walls with wire mesh in crown.	50–100 mm in crown and 30 mm in sides.	None
Poor rock IV RMR: 21–40	Top heading and bench 1.0–1.5 m advance in top heading. Install support concurrently with excavation 10 m from face.	Systematic bolts 4–5 m long, spaced 1–1.5 m in crown and wall with wire mesh.	100–150 mm in crown and 100 mm in sides.	Light to medium ribs spaced 1.5 m where required.
Very poor rock V RMR: < 20	Multiple drifts 0.5–1.5 m advance in top heading. Install support concurrently with excavation. Sprayed concrete as soon as possible after blasting.	Systematic bolts 5–6 m long, spaced 1–1.5 m in crown and walls with wire mesh. Bolt invert.	150–200 mm in crown, 150 mm in sides, and 50 mm on face.	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert.

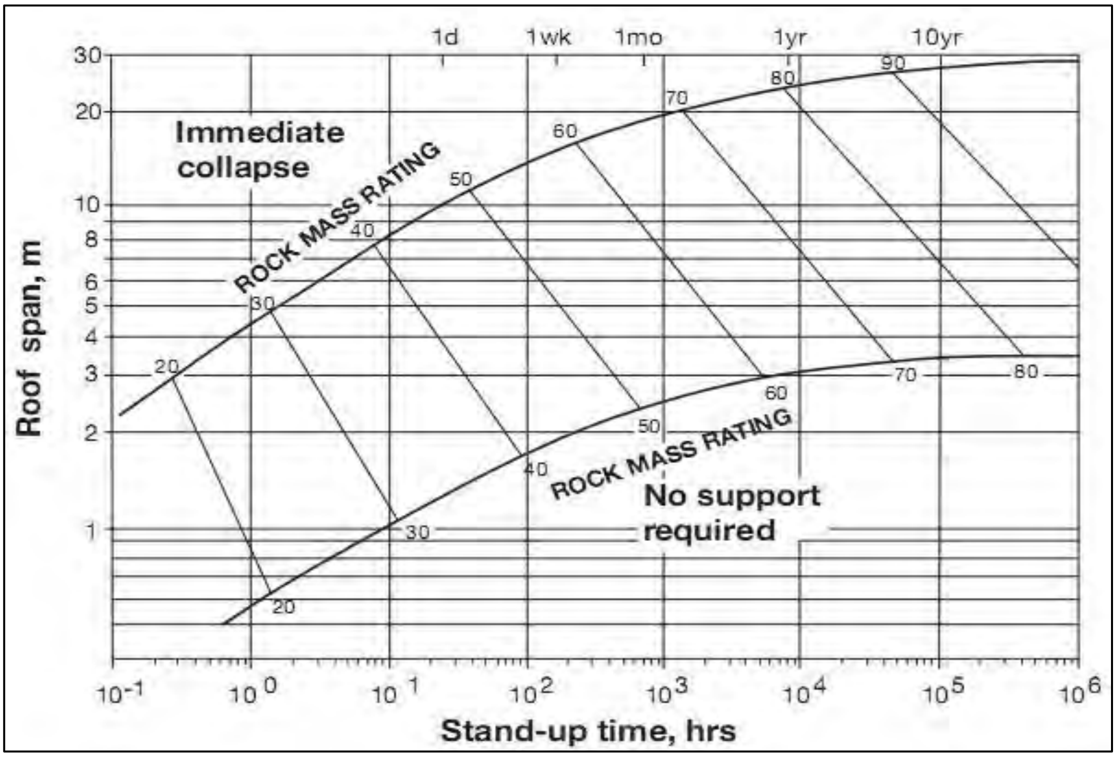


Figure 3.1: Relationship between the stand-up time and roof span for various rock masses (after Bieniawski, 1989)

b) Tunnel Quality Index (Q-system)

The Norwegian Geotechnical Institute Index, Q-system, was proposed by Barton et al. (1974) and is based on the evaluation of some 200 case histories of tunnel support in Scandinavia. It is probably the most widely used rock mass classification system today. The concept, upon which the Q-system is based, depends upon three fundamental requirements:

1. Classification of the relevant rock mass quality
2. Choice of the optimum dimensions of the excavation with consideration given to its intended purpose and the required factor of safety.
3. Estimation of the appropriate support requirements for that excavation.

The Q-system takes numerical account of the following parameters

- ✓ Rock Quality Designation: the absolute value is employed; $0 \leq RQD \leq 100$
- ✓ Joint Set Number (J_n): this is a measure of the number of joint sets with in the rock mass and has a range of values of 0.5 (representing massive rock with few joints) to 20 (crushed rock)
- ✓ Joint Roughness Number (J_r): the range of values is 0.5(smooth, planer, slickensided joints) to 4 (rough, undulating and discontinues joint)

- ✓ Joint Alteration Number (J_a): this takes account of infilling and has a range of values from 0.75(no infilling) to 20(thick band of crushed rock in-filled with clay materials)
- ✓ Joint Water Reduction (J_w): this takes in to account the presence of water under pressure affecting the shear strength of joints and has the range of values 0.05(high pressure) to 1 (zero pressure)
- ✓ Stress Reduction Factor (SRF): this takes account of several factors such as; loosing of the rock mass as a result of presence of shear zone and clay bearing rocks, rock stress problems in competent rock, and loads induced by swelling and squeezing ground conditions. The ranges of value cover from 1 to 15 and depend upon the nature of the problem.

The detailed descriptions of the Q- parameters are presented in Bieniawski (1989, from Barton et al, 1974) in a series of tables. For the application of Q system to tunneling, it has been expressed by the equation given below;

$$\boldsymbol{Q} = \left(\frac{\boldsymbol{RQD}}{\boldsymbol{Jn}}\right)\left(\frac{\boldsymbol{Jr}}{\boldsymbol{Ja}}\right)\left(\frac{\boldsymbol{Jw}}{\boldsymbol{SRF}}\right) \qquad 3.3$$

This expression is in effect of a function of 3 parameters which are approximately as given below;

Block Size = RQD/J_n

Inter-block Shear Strength = Jr/ Ja

Active Stress = Jw/SRF

The value of Q can be related to the support requirement of a tunnel by considering the equivalent dimension (D_e) of the excavation and this is defined as given by the equation;

$$\boldsymbol{De} = \frac{\textbf{Excavation Span,diameter or hieight}}{\textbf{Excavation support ratio}(\boldsymbol{ESR})} \qquad 3.4$$

The ESR is a function of the operational duty of the tunnel and relates to the stability required by such an opening, and a full description is given in Table 3.3. Using the values for Q and De, Barton et al. (1974) defines a series of 38 support types designed to encompass a full range of excavation requirements and rock mass conditions (Fig.3.2). They also give empirical relationships for the bolt length, maximum unsupported span and permanent support pressure for the roof and wall of the excavation as expressed by the equations given here under;

1. **Bolt Length (m):**

$$L = \frac{2 + 0.15B}{ESR} \qquad 3.5$$

Where B is the excavation width in meter

Table 3.3: Suggested excavation support ration (ESR) according to Barton (1974; as cited in Bieniawski, 1989)

Type of Underground Excavations	ESR
A. Temporary mine opening	3.0-5.0
B. Vertical shafts	
✓ Circular section	2.5
✓ Rectangular/ square section	2.0
C. Permanent mine openings, water tunnels for hydropower (excluding high pressure penstock), pilot tunnels, drifts and headings for large openings, surge chamber	1.6
D. Storage caverns, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels	1.3
E. Power stations, major highway or railroad tunnels, civil defense chambers, portals, intersections	1.0
F. Underground nuclear power stations, railroad stations, factories	0.8

2. **Maximum Unsupported Span(m):**

$$\text{Maximum span} = 2(\text{ESR})\ \text{Q}^{0.4} \qquad 3.6$$

3. **Permanent Roof Support Pressure(kg/m²):**

$$Pr = \frac{2Q^{-1/3}}{Jr}; if\ the\ joint\ set, Jn \geq 3 \qquad 3.7$$

$$Pr = \frac{2}{3}(Jn)^{\frac{1}{2}}(Jr)^{-1}Q^{-1/3}; if\ the\ joint\ set, Jn\ < 3 \qquad 3.8$$

4. **Permanent Wall Support Pressure (kg/m^2):**

$$Pw = \frac{2}{3}(Jn)^{1/2}(Qw)^{-1/3}Jr^{-1} \qquad 3.9$$

Q_w is wall factor given according to the absolute value of Q; where $Qw = 5Q\ for\ Q > 10;\ Qw = 2.5Q\ for\ 0.1 < Q < 10;\ and\ Qw = Q\ for\ Q < 0.1$.

Bieniawski (1989) proposed an empirical relationship between the RMR and Q value, and the expression is given by the eq.3.10 below;

$$RMR = 9\ ln^{Q} + 44 \qquad 3.10$$

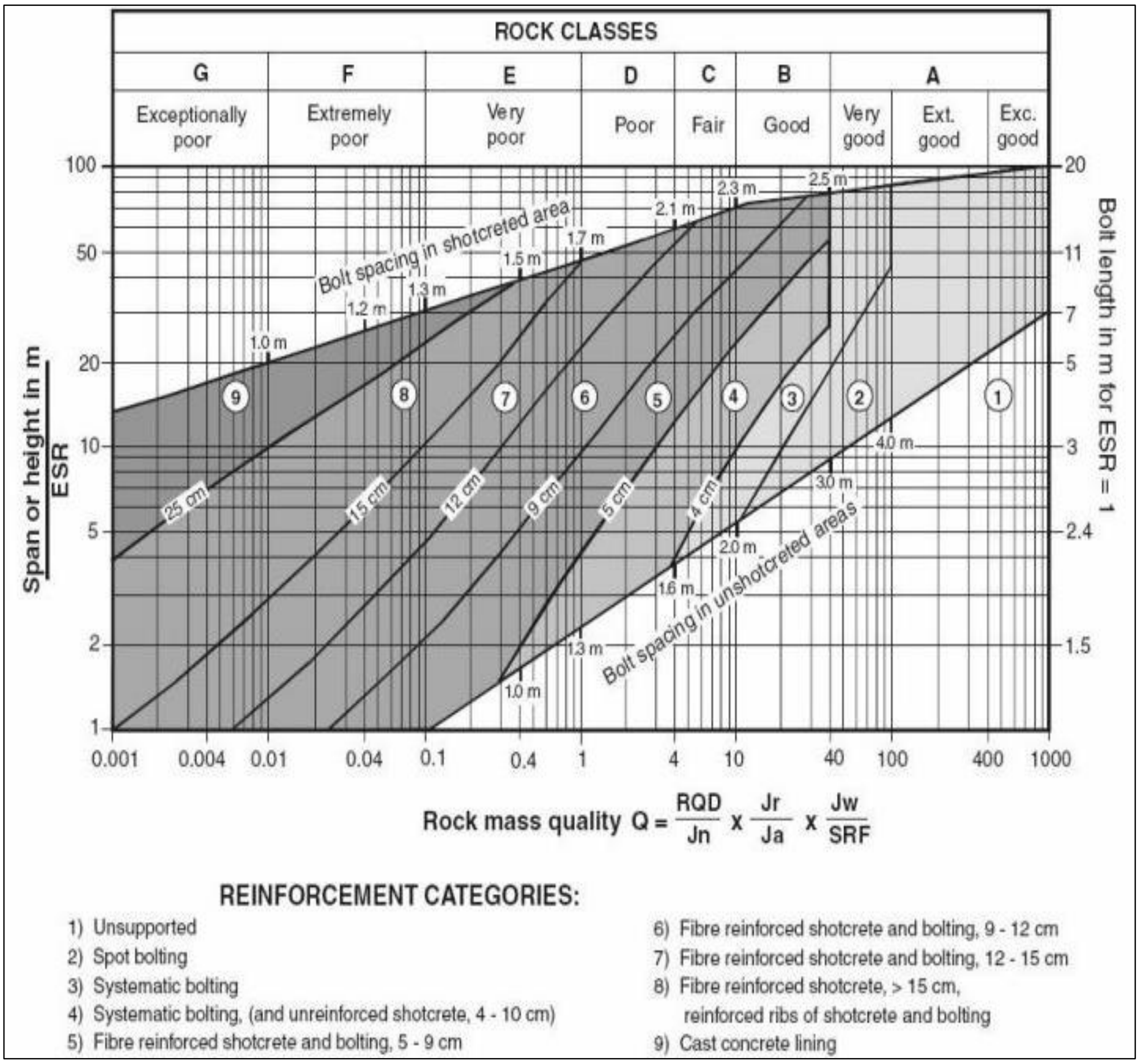


Figure 3.2: Estimated support categories based on the Q- value (Grimstad and Barton, 1993; as cited in Hoek, 2007)

Palmstrom and Brock (2006) investigated rock mass classification systems and particularly the Q-method , and showed that actually the Q- system is most applicable within a certain range of parameters as shown by the shaded area in Figure 3.3. Outside this area, supplementary calculations and methods of excavation are recommended. For poor quality rocks these systems are less effective as shown in Figure 3.8. In this lower quality rock masses the modules values (support criteria) are sensitive to small changes in the rating values.

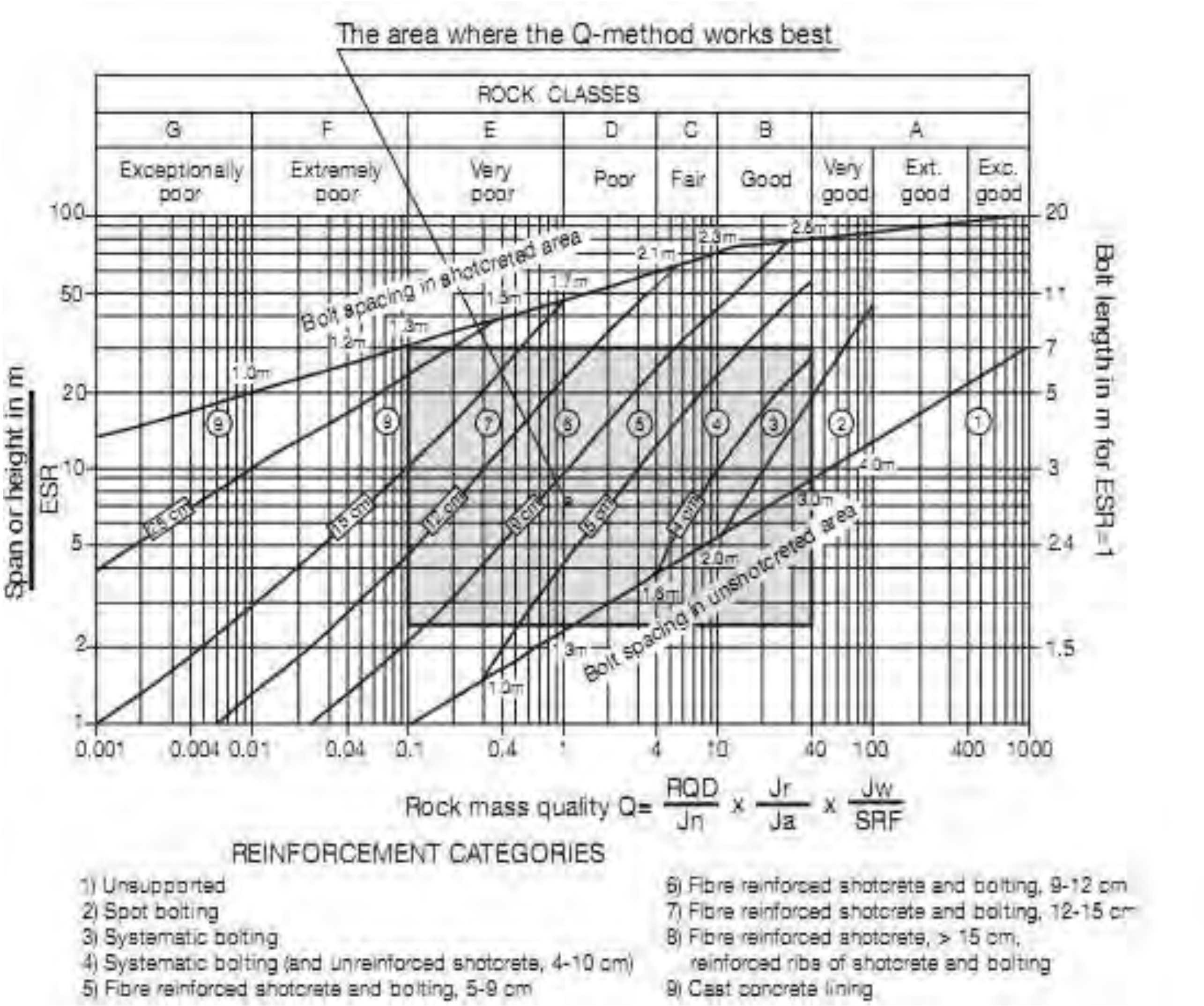


Figure 3.3: Limitations of the Q- system for rock support. Outside the shaded area supplementary evaluations should be applied (Palmstrom and Brock, 2006)

c) Determination of the Strength of Rock Masses

One of the challenges in designing underground structures is that of estimating the strength and deformation properties of the in-situ rock mass. In case of jointed rock masses, an evaluation of these properties presents formidable theoretical and experimental problems. However, such question is of fundamental importance in all major designs involving excavations in rock, it is essential that some attempt be made to estimate these strength and deformation properties and that these estimates should be as realistic and reliable as possible (Hoek et al., 1995).

Thus, the early development of intact rock failure criterion for underground excavation design process was realized by Hoek and Brown in 1980, and later modified to determine the strength and deformation properties of jointed rock masses. After several times modification by different researchers, this rock failure criterion may provide a simple empirical relationship which is sufficiently accurate for most underground, foundation and slope design processes.

In the present study, an effort is done to conduct a though tunnel site investigation and geotechnical evaluations by using the aforementioned experiences in order to give an engineering geological appraisal for rock tunnel of Zarima Dam Project. Preliminary analysis has been performed for tunnel support systems using empirical design approaches which are derived from the rock mass classification schemes, namely the Rock Mass Rating and Q-systems, and the details are presented in this research work.

Chapter 6 Conclusion and Recommendations

6.1. Conclusion

The water resource in Ethiopia ranks first in the Africa. The total theoretical irrigation potential of the country is estimated to be 3 million hectares of land. Ethiopian Government is currently focusing on developing the country's irrigation potential in medium to large scale schemes. The Tekeze River and its tributaries have been selected by Ethiopian Sugar Corporation as one of its key irrigation development areas. As such Zarima dam project is proposed across Zarima River to develop 40,000 ha land for sugar cane cultivation. The project head work comprises central asphalt core rock fill dam of 153 m height, spillway, diversion conduit and middle level outlet tunnel for the dam.

According to the design report of SGI and SC (2014), the middle outlet tunnel of Zarima dam will have excavation diameter of 5 meters and extends for 600 m long and is designed along the left abutment of Zarima dam. It is proposed to be excavated by drill and blast tunneling methods from both outlet and inlet tunnel portals. The rock overburden thickness varies from 12 m to 105 m, and the geology through which the tunnel is excavated is composed of Meta volcanic rocks of low grade metamorphic unit.

This study is then initiated to achieve a compressive engineering geological appraisal of the proposed tunnel, that includes studying the geological features and mechanical properties of rocks, identifying the potential geological problems that could affect the construction of the tunnel, engineering geological evaluation on the site investigation results and producing relevant geotechnical parameters for underground designing purposes, and finally providing the preliminary empirical design of underground excavation rock supports. To meet these research objectives, detailed geotechnical site investigations have been carried out along the tunnel route, which comprises field geological mapping at scale of 1:5000, geostructural surveying, discontinuity characterizations, rock mass classifications and rock mechanics tests.

From the geological mapping of the present study, three rock units are observed at the tunnel site, namely; Sedimentary Rock, Meta-volcanic Rock and Meta-ultramafic Rock, from top to bottom. The sedimentary rocks are separated from the underlying Meta-volcanic rocks by unconformable contact zone of variable thickness. The unconformable contact zone is marked by loose unconsolidated material, which grades to weathered basement rocks.

The geology through which the proposed tunnel will be excavated comprises mainly low grade metamorphic units of Meta-volcanic rocks underlain by the Meta-ultramafic rocks. These rocks are blanketed by variable thicknesses of colluvial deposit and extremely weathered rocks.

For the present study, in order to understand the mechanical and physical properties of the rocks to be encountered along the tunnel route, specific gravity, porosity, unit weight and uniaxial compressive strength tests were performed on representative samples of Meta volcanic rocks. Accordingly, the unit weight of the fresh rock is 2.58 gm/cm^3 and the weathered rock is 2.42 gm/cm^3. The porosity varies from 2.43 % to 16.5 % for fresh and weathered rock respectively. The uniaxial compressive strength of the rocks is ranging from 13 Mpa to 45 Mpa, and the rock classification according to intact rock strength is weak to strong rock. In addition, petrographic analysis done on selected rock samples reveal that the Meta-volcanic rock in the study area is compositionally Calcite-Epidote-Chlorite Schist.

From the results of the geotechnical site investigations carried out in the present study, a tunnel geotechnical modeling has been done to understand the engineering properties of the rock masses encounter along tunnel route. As such three major tunnel geotechnical units (Gtu1, Gtu2 and Gtu3) have been identified in the study area. These tunnel units are basically classified based on their engineering properties directly related to the degree of rock mass weathering, nature of discontinuities and intact rock strength, through which the important geotechnical parameters for the in-situ rock masses could be understood with respect to the stability of tunnel excavations. Accordingly the following geotechnical zones can be possibly foreseen for the mid-level outlet tunnel of Zarima dam;

Geotechnical Unit-1 (Gtu.1): This tunnel unit is characterized by slightly weathered and fresh rock masses with closely to widely spaced joints. The porosity and unit weight of the rocks is 2.47 % and 2.58 gm/cm^3 respectively. The intact rock strength is reached to an average of 45 Mpa with RQD value is 65 % on average.

Geotechnical Unit-2 (Gtu.2): This tunnel unit is characterized by moderately weathered and closely jointed rock masses. The porosity and unit weight is 6.27 % and 2.48 gm/cm^3 respectively. The maximum intact rock strength is reached to 29 Mpa with an average RQD value of 42 %.

Geotechnical Unit-3 (Gtu.3): It is characterized by highly weathered and fractured rock masses. The porosity and unit weight of the rock is 16.5 % and 2.42 gm/cm^3 respectively, and the intact rock strength is reached to an average of 13 Mpa.

For each tunnel geotechnical unit, RMR, GSI, Q values and Rock Mass Strength have been determined at tunnel grades. Based on the geomechanical evaluations done along the tunnel alignment, for about 600 m tunnel stretch, it is estimated that the rock classes according to Bieniawski's classification will be occurred in the following percentages;

- RMR 44 to 69, fair to good quality of rock masses; 42 %
- RMR 21 to 38, poor quality of rock masses; 30 %
- RMR 00 to 20, very poor quality of rock masses; 28 %

As it is observed from the GSI determination, the rock masses in the study area has the GSI values ranging from 21 to 71, which is categorized as very poor to good quality of rock masses. The global strength of the jointed rock masses, as determined by using Rock Lab software, is reached to minimum 1.12 Mpa and maximum 9.6 Mpa with intact rock strength varying from 13 Mpa to 45 Mpa. Besides, from the evaluations of tunnel quality index (Q-system), it is found that the rock masses encounter along the tunnel route has Q value of lowest 0.03 and highest 10.94. These Q values can be considered for selecting temporary rock support of the tunnel.

The boreholes drilled along the tunnel alignment confirm that full length of the tunnel will be excavated in dry ground water conditions. However, significant amount of rain water seepage is highly suspected during rainy season as the tunnel will pass through highly fractured/weathered rock masses particularly at portal sections. The water may reduce the shear strength of the rock masses and increase loads on the underground excavation supports, and appropriate drainage system shall be provided during excavation works.

The geological map and resistivity profiles indicate that a total of three geological structures will cross the tunnel axis. These major fracture zones are oriented NNE-SSW (shear zone) to ENE-WSW (Faults), and their orientations are fair to unfavorable with respect to the tunnel axis as they cut the tunnel excavation obliquely. Moreover, the stereographic analysis done in the present study reveals the rock joint structures found in the tunnel site show cross cutting relationships, and this can affect the stability of the tunnel excavation by inducing rock wedge failures.

The results of the rock mass classifications conducted in the study area show that the rocks along the tunnel alignment are very poor to good quality of rock masses. In addition, as the tunnel will be excavated through shallow depth, the crosscutting relationships of the joint structures may be more pronounced at the tunnel site, and then rock wedge failures are highly suspected during excavation works. Thus, by considering these situations, five types of underground excavation supports have been suggested for the proposed tunnel such as support type A, B, C, D, and E as discussed in detailed in chapter 5. These rock support categories satisfy the following tunneling situations;

- The shotcrete thickness becoming in the design is generally slightly higher than the one suggested for temporary rock supports with Q methodology, as the Q values modified to $Q^1 = 5Q$ for selecting temporary rock supports
- The design bolt length is generally substantially higher (2.5 m vs. 1.72 m) and having smaller spacing.

Therefore, it can be conclude that the proposed underground excavation rock support design is slightly more on the safe side if compared with the one suggested with the Q system for temporary rock supports.

6.2. Recommendations

Based on the results of the present study, the following recommendations have been forwarded for middle level outlet tunnel of Zarima dam.

- It needs to be emphasis that this study is based on limited amount of field data, and the results should only be used as estimates.

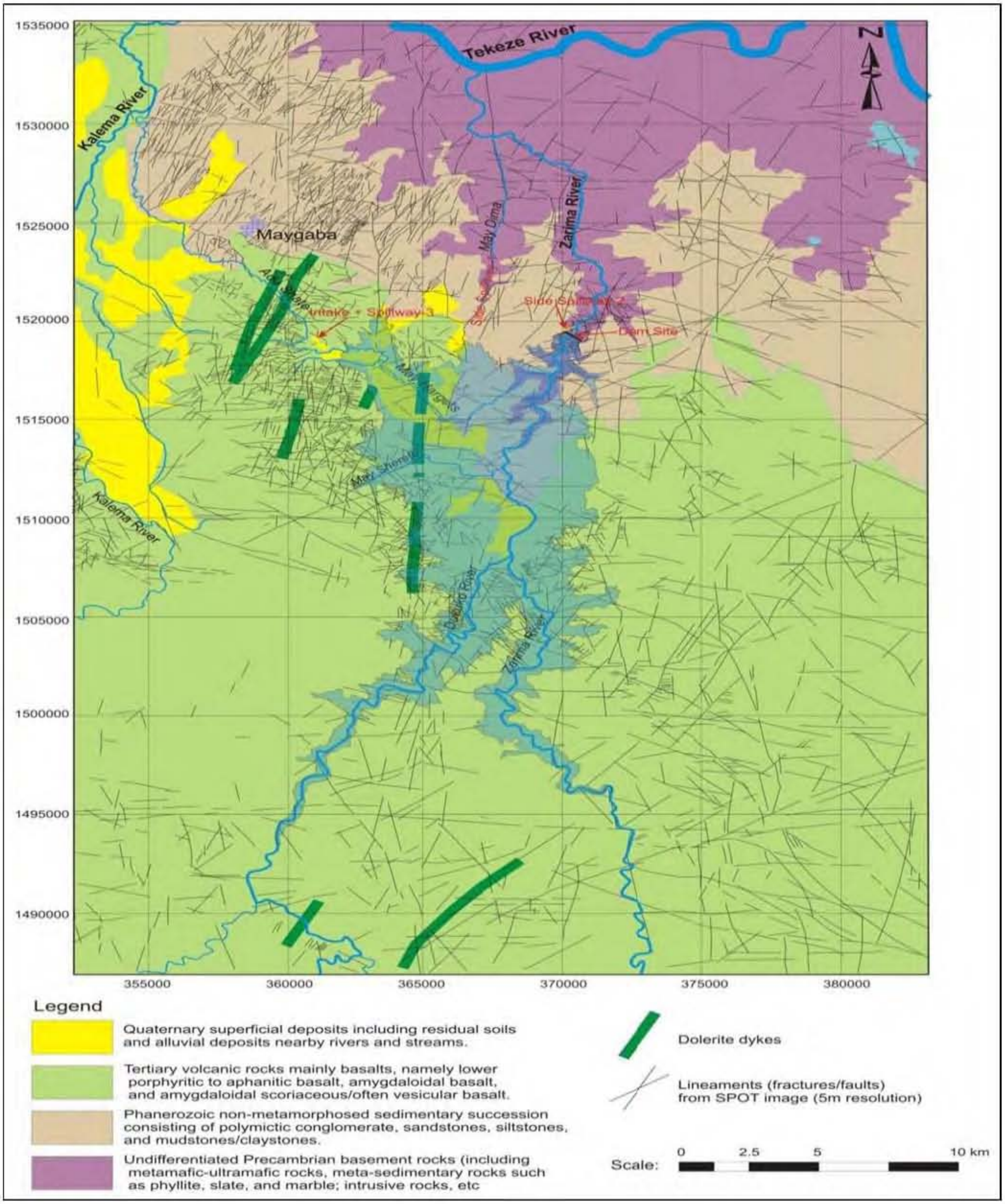
- The present study will give a great effort in documenting the geological-geotechnical conditions for evaluating the design of the project. Supplementary research is therefore recommended for a better understanding of the engineering geology of the tunnel and increased confidence and reliability of geotechnical design parameters.

- The greater frequency of bad rock classes may intercept the first and last tunnel stretch (at inlet and outlet portal sections), which will requires removal of the soft ground such as highly fractured and weathered rock masses having the rock overburden thickness below 15 m. If the rock overburden thickness is over 15 m, heavy rock support measures, like steel ribs with reinforced shotcrete, is necessary to stabilize underground excavations in the study tunnel.

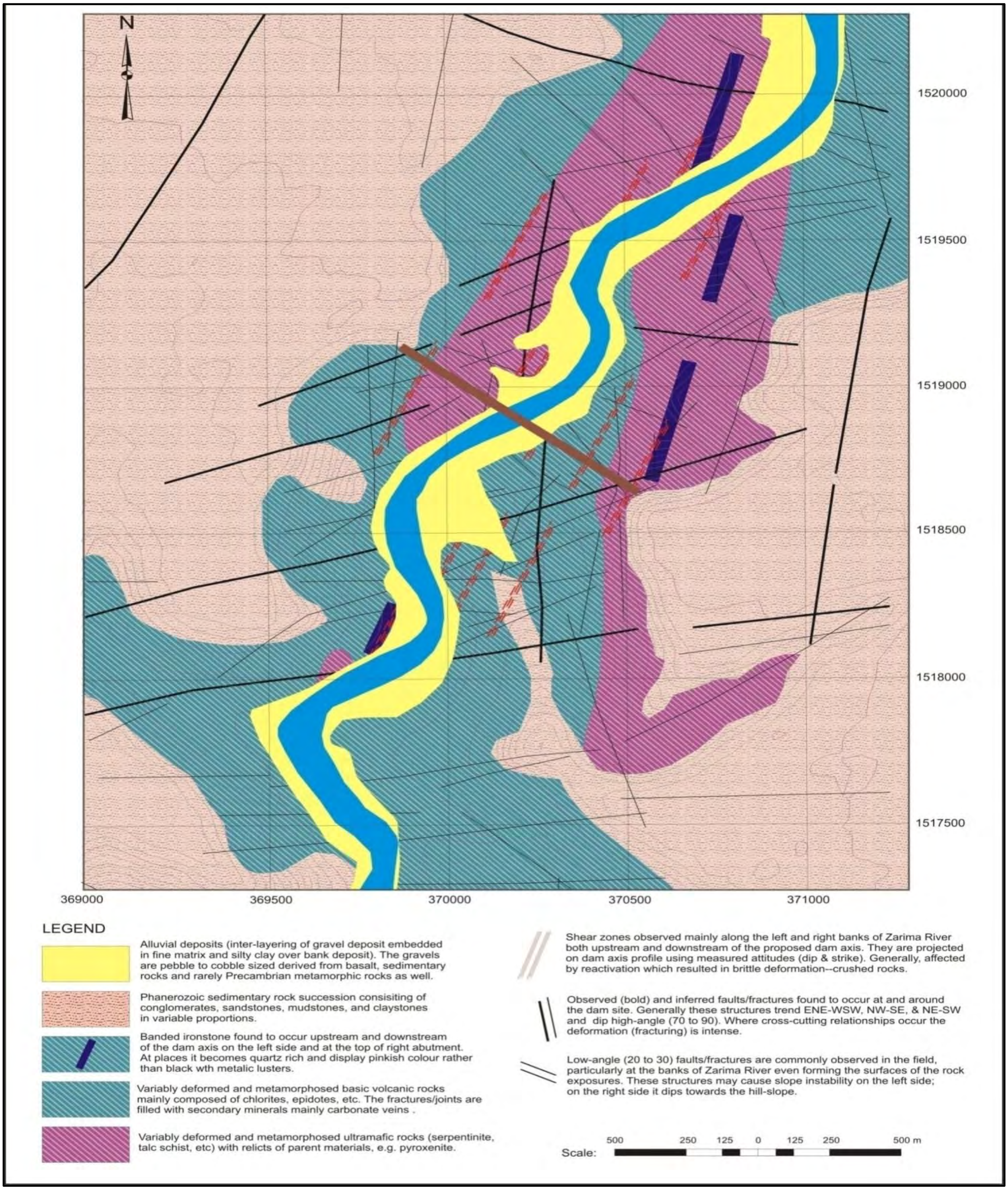
- 3-Dimensional underground geological mapping and rock mass classifications (GSI and Q-system) will be required during construction works for assessing the stability of underground excavations and the suitability of preliminary rock support analysis.

- Consolidation grouting may be required during tunnel excavation in soft ground conditions such as highly fractured, sheared and weathered rock masses of the study area.

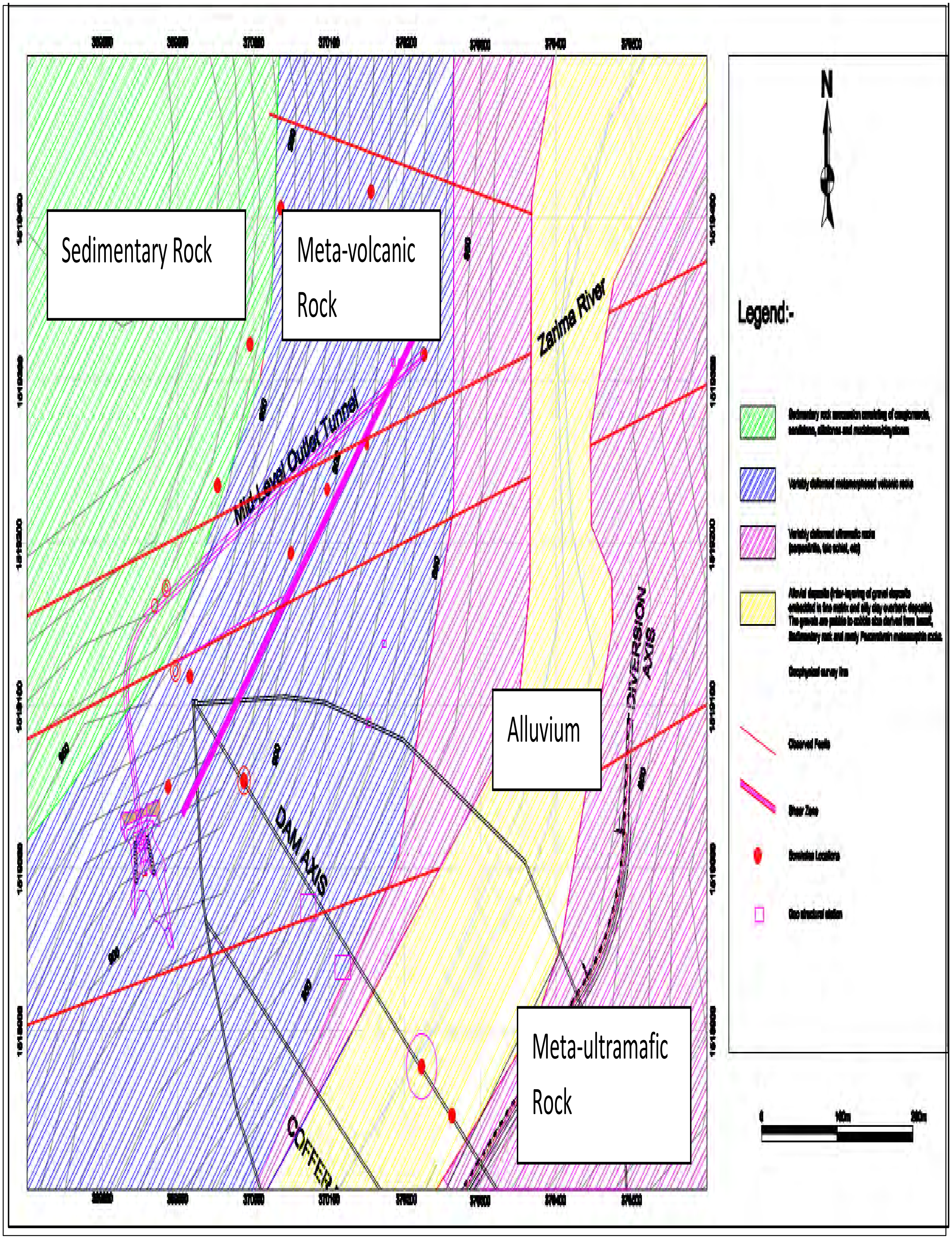
- The role of geotechnical parameters which has been assessed in the present study are as one of particular importance in respect of the support of tunnels. It is of major importance in the design of rock reinforcement systems in views of the latter needing to match and complement the rock structure as closely as possible. Emphasis requires to be placed on rock structural data obtained during site investigation stage, and should be verified and reviewed in the light of further information obtained during the construction of the tunnel. As a result the tunnel rock support design should be updated as new geotechnical data become available following new rock exposures obtained by inspection of underground excavations.



(A) General Geology of the Study Area



(B) Local Geology of the Study Area



(C) Geology of the Mid-Level Outlet Tunnel Alignment

Addis Ababa University

School of Earth Sciences

Collage of Natural and Computational Sciences

Engineering Geological Appraisal of Mid-Level Outlet Tunnel for Zarima Dam; Tigray Region, Northern Ethiopia

A Thesis

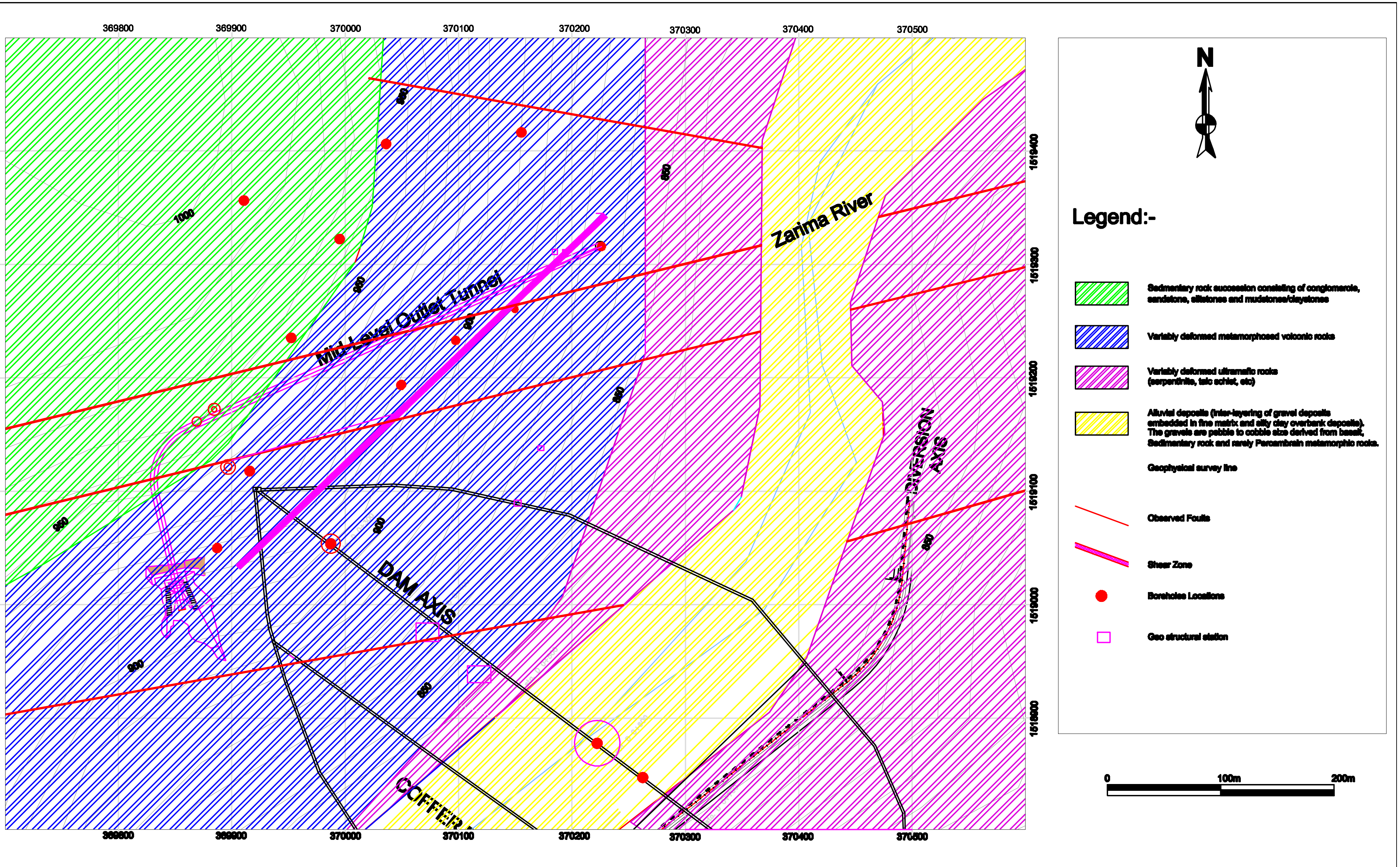
Submitted to

The School of Graduate Studies of Addis Ababa University

In Partial Fullfilment of the Requirements for the Degree of Masters in Engineering Geology

Abichu Lule

June 2016


N
Legend:-
Sedimentary rock succession consisting of conglomerate, sandstone, siltstone and mudstone/claystone
Variably deformed metamorphosed volcanic rocks
Variably deformed ultramafic rocks (serpentinite, talc schist, etc)
Alluvial deposits (inter-layering of gravel deposits embedded in fine matrix and silty clay overbank deposits). The gravels are pebble to cobble size derived from basalt, Sedimentary rock and rarely Percambrain metamorphic rocks.
Geophysical survey line
Observed Faults
Shear Zone
Boreholes Locations
Geo structural station
0
100m
200m
Mid-Level Outlet Tunnel
Zarima River
DAM AXIS
DIVERSION AXIS
COFFER
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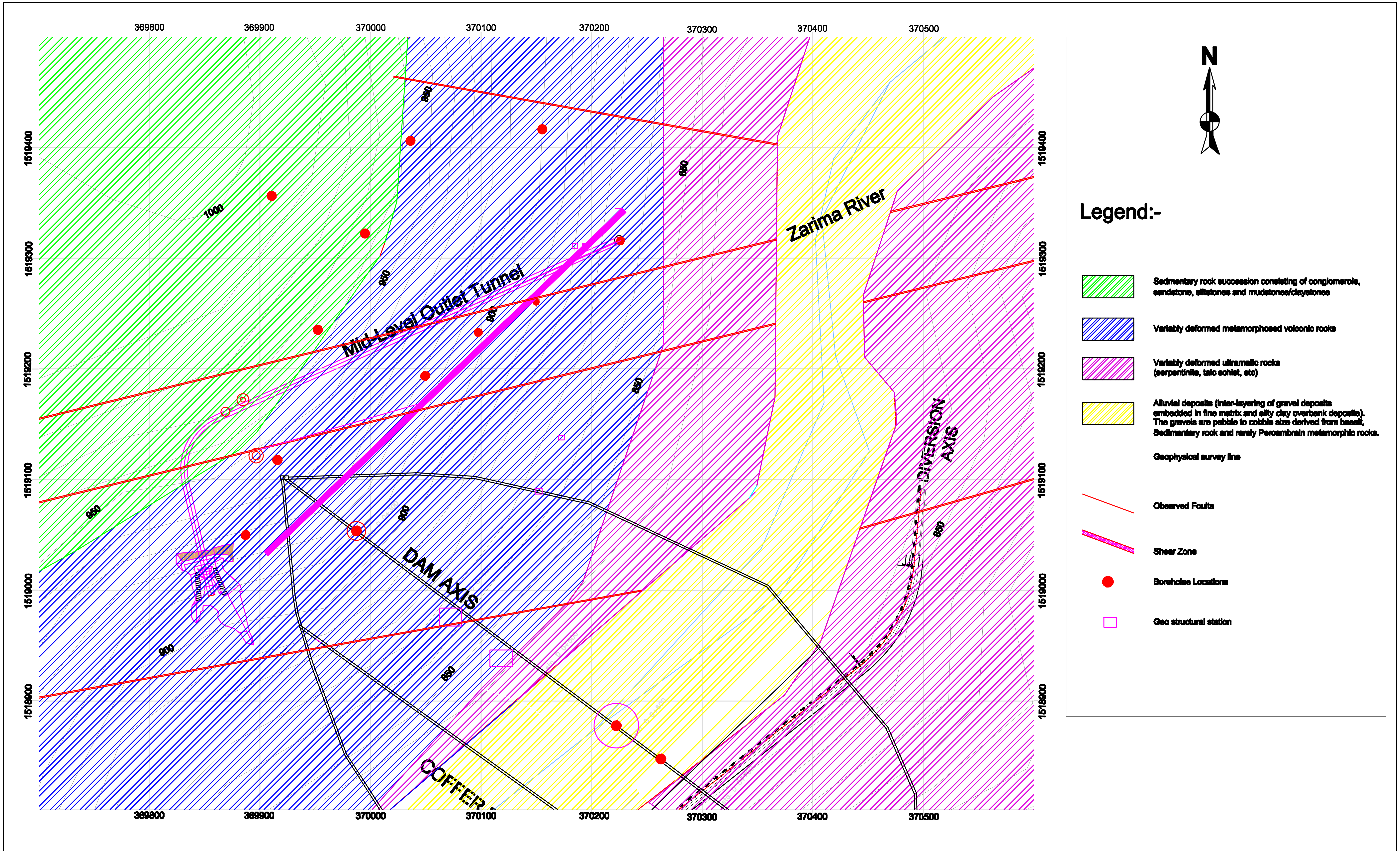

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Approved by Board of Examiners:

Dr. Balemual Atnafu ……………………………………..
Chairman

Dr. Trufat Hailemariam ………………………………………..
Advisor

Dr. Tarun Kumar Raghuvanishi ..………………………………………
Examiner

Dr. Ameha Atnafu ………………………………………
Examiner

DECLARATION

I hereby declare that the thesis entitled "*ENGINEERING GEOLOGICAL APPRAISAL OF MIDDLE LEVEL OUTLET TUNNEL FOR ZARIMA DAM, TIGRAY REGION, NORTHERN ETHIOPIA*" is my original work. It has been carried out under the supervision of Dr. Trufat Hailemariam , School of Earth Sciences, Addis Ababa University during the year 2015 as part of Master of Science Program in Engineering Geology. I further declare that this work has not been submitted to any other University or Institution for the award of any degree or diploma and all sources of materials used for the thesis have duly acknowledged.

Name: Abichu Lule

Signature: _____________

Advisor: Dr. Trufat HaileMariam

Place and Date of Submission: School of Graduate Studies, Addis Ababa University

June 2016