

**Rock Mass Stability Analysis at an Underground Opening:
The Case of Lega Dembi Gold Mine, Shakiso, South
Ethiopia**



Derartu Jibro Abdukerim

A Thesis Submitted to the Department of Applied Geology

School of Applied Natural Science

Presented in Partial Fulfillment of the Requirement for the
Degree of Master's in Engineering Geology

Office of Graduate Studies

Adama Science and Technology University

November, 2022

Adama, Ethiopia

Rock Mass Stability Analysis at an Underground Opening: The Case of Lega
Dembi Gold Mine, Shakiso, South Ethiopia

Derartu Jibro

Advisor: Dr. Endalu Tadele

Co-Advisor: Dr. Mohammed Haji

A Thesis Submitted to the Department of Applied Geology,

School of Applied Natural Science

Presented in Partial Fulfillment of the Requirement for the Degree of
Master's in Engineering Geology

Office of Graduate Studies

Adama Science and Technology University.

November, 2022

Adama, Ethiopia

Declaration

I hereby declare that this Master Thesis entitled is “**Rock Mass Stability Analysis at an Underground Opening: The Case of Lega Dembi Gold Mine, Shakiso, South Ethiopia**” my original work. That is, it has not been submitted for the award of any academic degree, diploma, or certificate in any other university. All sources of materials that are used for this thesis have been duly acknowledged through citation

Name of the Student

Signature

Date

Recommendation

We, the advisors of this thesis, hereby certify that we have read the revised version of the thesis entitled “**Rock Mass Stability Analysis at an Underground Opening: The Case of Lega Dembi Gold Mine, Shakiso, South Ethiopia**” was prepared under our guidance by **Derartu Jibro** and submitted in partial fulfillment of the requirements for the degree of Masters of Science in **Engineering Geology**. Therefore, we recommend the submission of a revised version of the thesis to the department following the applicable procedures.

Major Advisor

Signature

Date

Co-advisor

Signature

Date

Approval Page

We, the advisors of the thesis entitled “**Rock Mass Stability Analysis at an Underground Opening: The Case of Lega Dembi Gold Mine, Shakiso, South Ethiopia**” and developed by Derartu Jibro, hereby certify that the recommendation and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis.

Major Advisor	Signature	Date
---------------	-----------	------

Co-Advisor	Signature	Date
------------	-----------	------

We, the undersigned, members of the Board of Examiners of the thesis by **Derartu Jibro** have read and evaluated the thesis entitled “**Rock Mass Stability Analysis at an Underground Opening: The Case of Lega Dembi Gold Mine, Shakiso, South Ethiopia**” and examined the candidate during the open defense. This is, therefore, to certify that the thesis is accepted for partial fulfillment of the requirement of the degree of Master of Science in **Engineering geology**.

Chairperson	Signature	Date
-------------	-----------	------

Internal Examiner	Signature	Date
-------------------	-----------	------

External Examiner	Signature	Date
-------------------	-----------	------

Final approval and acceptance of the thesis is contingent upon the submission of its final copy to the Office of Postgraduate Studies (OPGS) through the Department Graduate Council (DGC) and School Graduate Committee (SGC).

Department Head	Signature	Date
-----------------	-----------	------

School Dean	Signature	Date
-------------	-----------	------

Office of Postgraduate Studies, Dean	Signature	Date
--------------------------------------	-----------	------

ACKNOWLEDGEMENT

This work would not have been possible without the financial support and laboratory facilities of Adama Science and Technology University. I would like to express my special gratitude to my advisor Dr. Endalu Tadele as well as to my co-advisor Dr. Mohamed Haji who made this work possible. Their guidance and advice helped me through all stages of writing this thesis. Detailed subsurface data, geophysical and borehole data were collected from MIDROC Gold Mine Company and I want to thank the Company, the Head and General Manager of Company Dr. Dabere for providing the data and their support. I am grateful to the staff in the civil and geotechnical engineering department of Addis Ababa Science and Technology University, who permitted me to use the necessary tools in the lab. Last but not least I would like to thank my friend and families for helping me to complete this thesis.

Table of Contents

Contents	Page
ACKNOWLEDGEMENT	vi
Table of Contents	vii
List of tables	x
List of figure and illustration	xi
List of Acronyms and Abbreviations.....	xiv
ABSTRACT	xv
CHAPTER ONE.....	1
INTRODUCTION	1
1.1 Background	1
1.2 Statement of the Problem.....	2
1.3 Objective of the Study	2
1.3.1 General Objectives	2
1.3.2 Specific Objectives.....	3
1.4 Location and Accessibility.....	3
1.5 Climate and topography	4
1.6 Significant of the Study	5
CHAPTER TWO.....	6
LITERATURE REVIEW	6
2.1 Introduction.....	6
2.2 Geology of the Study Area–Review	7
2.3 What is underground instability?	7
2.4 Factors Affecting the Stability of Underground Opening.....	8
2.5 Slope Failure Mechanism	9
2.6 Shear Strength Failure Criteria of Rock.....	10
2.6.1 Mohr-Coulomb Strength Criteria	10
2.6.2 Hoek Brown Strength Criteria.....	11
2.7 Underground Opening Design Approaches	12
2.7.1 Empirical approach: Rock mass classification systems	12
2.8 Numerical Methods.....	17
2.9 Ground control systems	18
CHAPTER THREE	19
MATERIALS AND METHODS	19

3.1 General.....	19
3.2 Methodology	19
3.2.1 Review and analysis of existing data	19
3.2.2 Detailed fieldwork and sampling	20
3.2.3 Laboratory Analysis	20
3.3 Design of the tunnel using Rock mass classification method.....	28
3.4 Numerical Modelling and support system design (Rock stability Analysis).....	30
3.4.1 Finite Element Method (FEM)	30
3.4.2 Kinematic Analysis	30
3.4.3 Phase2/RS2.....	35
3.4.4 RocData/RocLab	35
3.4.5 RocSupport.....	36
CHAPTER FOUR	37
GEOLOGY	37
4.1 Regional Geology and Structural Setting	37
4.2 Tectonic and Structural Setting.....	37
4.3 Regional Context	38
4.4 Structure.....	40
4.5 Geology and Structure of the Lega Dembi area.....	40
4.6 Local geology	41
4.6.1 Introduction	41
4.6.2 Geology	42
4.6.3 Joints.....	43
4.6.4 Subsurface Geology	44
CHAPTER FIVE	45
RESULTS AND DISCUSSION.....	45
5.1 General.....	45
5.2 Engineering properties of rocks	46
5.2.1 Rock strength result.....	46
5.3 Finite element method	47
5.3.1 Tunnel information.....	47
5.3.2 Finite Element Method of Analysis for East Sakaro underground tunnel	49
5.3.3 FEM With culvert analysis.....	53
5.3.4 FEM With bolt Analysis	55
5.4 Kinematic Analysis.....	56
5.5 Unwedge Analysis.....	59

5.6 Support System.....	63
5.6.1 Effect of bolt diameter, length and spacing on the factor of safety of the tunnel	63
5.6.2 Support by Bolt and shotcrete	65
CHAPTER SIX	68
CONCLUSION AND RECOMMENDATION	68
RECOMMENDATION.....	70
REFERENCES	71
APPENDICES.....	79
Appendix 1: Rock material characterization	79
Appendix 1.1 Name of the Rocks used for Laboratory test	79
Appendix 2: Rock mass property determination	85
Appendix2.2 Rock mass property determination using Rocdata	86
Appendix 3: Borehole Data	87

List of tables

Table 2.1 Rock mass classification RMR system (Bieniawski, 1989).....	14
Table 3.1 Analytical method adopted for rock sample Parameter.....	20
Table3.2 Rock mass classification values of the rock samples collected field and core drill.....	29
Table5.1 Name of the Rocks used for Laboratory test	46
Table5.2 Table Inputs for FEM	47
Table5.3 Tunnel Geometry Properties of East Sakaro underground tunnel.....	48
Table5.4 Input for kinematic analysis 1.....	56
Table5.5 Input for kinematic analysis 2.....	58
Table 5.6 Effect of bolt diameter, length and spacing on factor of safety of the tunnel.....	64
Table5.7 Effect of bolt diameter, length and spacing with shotcrete of 10mm thickness on the factor of safety of the Tunnel	66

List of figure and illustration

Fig1.1 East Sakaro Underground Tunnel	2
Fig1.2 Location Map of the study area.....	4
Fig 3.1 Underground rock formation of the study area.....	20
Fig3.2 schimdt Hammer Test of rock samples a) Metagranodiorite=RS1, b) Amphibolite=RS2, c) Quartz mica schist=RS3, d) Graphitic Quartz mica schist=RS4.....	21
Fig3.3 schimdt Hammer Test for RS1	21
Fig3.4 schimdt Hammer Test for RS2.....	21
Fig3.5 schimdt Hammer Test for RS3.....	22
Fig3.6 schimdt Hammer Test for RS4.....	22
Fig3.7 Point Load test of Rock Samples.....	24
Fig 3.8 Point load test before.....	24
Fig3.9 Point load test after.....	24
Fig3.10 Samples prepared for Dry Density test.....	25
Fig 3.11 Measuring the Volume of rock Samples and preparing them to be oven dried for 24 hours.....	25
Fig3.12 Preparing Rock Samples to conduct Saturated Density test.....	26
Fig3.13 Conducting Saturated Density test for rock samples.....	26
Fig3.14 Rock samples obtained from Saturated Density test after being placed in water for 24 hours.....	26
Fig3.15 Preparing Rock Samples for Slake Durability Test.....	27
Fig3.16 Slake durability Test (Before).....	27
Fig3.17 Slake Durability Test for RS1.....	28
Fig3.18 Slake Durability for RS2.....	28

Fig3.19 Slake durability Test for RS3.....	28
Fig3.20 Slake Durability test for RS4.....	28
Fig3.21 Core drilling data from Sakaro underground tunnel	29
Fig3.22 Measurement of core drilling data for calculating RQD and RMR.....	29
Fig4.1 Regional geology of Shakiso	39
Fig4.2 Geological map of the study area	43
Fig 4.3 Subsurface Layers of Sakaro Uderground tunnel	44
Fig5.1 Sakaro Underground Gold Mine (Portal gate of Sakaro Underground Tunnel).....	45
Fig5.2 Tunnel Geometry of East Sakaro.....	48
Fig5.3 Tunnel shape	49
Fig5.4 Meshed Horseshoe tunnel before computing.....	49
Fig5.5 Maximum and minimum stress distribution with respect to the displacement.....	50
Fig5.6 Effective stress along Sigma 1.....	50
Fig5.7 Normal stress Vs total displacement.....	51
Fig5.8 Effective stress along Sigma 3.....	52
Fig5.9 Graph showing shear stress vs distances.....	53
Fig5.10 Effective stress along sigma1.....	53
Fig5.11 Effective stress along sigma 3.....	54
Fig5.12 Effective stress along sigma 1.....	55
Fig5.13 Effective stress along sigma 3.....	55
Fig5.14 Critical points showing planar failure for stereonet 1.....	57
Fig5.15 Critical points showing Wedge failure for stereonet 1.....	57
Fig5.16 Critical points showing planar failure for stereonet 2.....	58

Fig5.17 Critical points showing Wedge failure for stereonet 2.....	58
Fig5.18 3D view of wedge failure for analysis 1.....	59
Fig5.19 3D Multi view of wedge failure for analysis 1.....	60
Fig5.20 bolt support system for analysis 1.....	60
Fig5.21 Shotcrete support system for analysis 1.....	61
Fig5.22 3D view of wedge failure for analysis 2.....	61
Fig5.23 3D multi view for analysis 2.....	62
Fig5.24 Bolt support system for analysis 2.....	62
Fig5.25 Shotcrete support system for analysis 2.....	63
Figure 5.26 Support design by varying bolt diameter, spacing and Length.....	6
5	
Figure 5.27 Stabilization using Bolt and Shotcrete	67

List of Acronyms and Abbreviations

MIDROC- Mohammed International Development Research and Organization Companies

GSI- Geological Strength Index

RMR- Rock Mass Ratings

RQD- Rock Quality Designation

LEM- Limit Element Method

FEM- Finite Element Method

DEM- Discrete Element Method

RQD- Rock Quality Designation

RSR- Rock Structure Rating

UG- Underground Mining

SRF- Strength Reduction Factor

FOS- Factor of Safety

UCS- Uniaxial Compressive Strength

ABSTRACT

The stability of rock mass is critical in an underground excavation because it ensures the safety of the working environment and the successful ore exploration. Numerical Rocscience softwares enable rapid assessments of the support requirements for underground excavations at any stage of a project, even when geotechnical data is limited. The underground excavation frequently relies on Numerical analysis and other methods such as rock mass classification methods, which provide a quick way of assessing the rock mass quality and support requirements.

Instabilities in rock slopes are a major geological hazard that frequently causes the failure of various civil engineering structures such as open-pit mines and tunnels. The current research was conducted at the Lega Dembi underground gold mine, specifically in Shakiso, South Ethiopia. The tunnel length, width and height are 1.5km, 5m and 5.5m respectively. It used for ore transportation and frequently impacted by rock slope instabilities. As a result, the purpose of this research is to identify and analyze critical rock slope sections along this tunnel using the Finite Element Method (FEM), kinematic analysis, Rocdata, unwedge, and Rocsupport methods. The main lithologies along the tunnel line are (1) Metagranodiorite (2) Amphibolite (3) Quartz Mica Schist and (4) Graphitic Quartz Mica Schist. On the specimen obtained from core and surface outcrop the following tests are carried; dry and saturated density, slake durability and point load tests are carried out. Dry and saturated test results range from 24 to 25KN/m³ and 26-27KN/m³ respectively. Graphitic-quartz mica schist has a lower strength value than the other rocks and amphibolite is the very competent geological unit. From laboratory strength tests and other Strength tests design parameters determined from rock lab Software. Based on the geological and Tunnel geometry information, the kinematic analysis shows two dominant failures mode that are planar and wedge failures that had occurred at sakaro underground opening.

The support system also design for the access tunnel using bolt only and bolt with shotcrete. The results shows the use of a bolt with a diameter of 17mm, spacing 0.25 and 1m length and a factor of safety 2.44 which is acceptable from economical and safety point of view. This factory of safety is at which point the tunnel would be stable, and this is accomplished with numerical modelling of underground rock mass stability, including an investigation of the effectiveness of rock supports.

Key words: *underground excavation, Tunnel stability, factor of safety, RocSupport, bolt, shotcrete*

CHAPTER ONE

INTRODUCTION

1.1 Background

In engineering construction a tunnel is a structure built underground that runs through the surrounding soil or rock. The length of a tunnel is usually much greater than its width. Tunnels are designed to withstand earth forces from all directions. They are primarily employed in the mining (ore extraction), public utilities (water, sewage and fuel transportation) and transportation industries (roadways and railways). Tunnels are complex structures as they should be designed to withstand the loads imposed by different factors.

Tunneling in modern mining areas involves the construction of subterranean road, underground and railroad connections, subterranean installations and access tunnels but also water tunnels (supplying dam plants and power plants) and pipelines. Mining tunnels are necessary for accessing, extracting and transporting underground ores. However, the choice of tunneling methods and mining equipment varies depending on geologic conditions and rock and soil types.

The East Sakaro underground tunnel, which provides access to the mine area, is used to transport the ore. Its stability plays a significant role on the successful mining operations and the economy of the projects. The interactions between the rock mass and the support system depend on factors like rock mass properties, the geometry and properties of the supports, the geological condition, etc. The response of rock masses is sensitive to the loading and unloading processes (Cai *et al.*, 2016). A good understanding of the rock mass response during and after the excavation is helpful to design or modify the excavation and support procedures and to avoid the potential failures around the openings. The likely ground types must be assessed and after that, combined with the components of the tunnel geometry, the primary in situ stresses and the groundwater regime, the possible failure modes must be considered. Hence, the contribution of engineering geological information for safe and economical tunneling can be presented by a rock mass classification value (e.g. RMR, Q, GSI or others) and by Numerical methods. Moreover it requires an understanding of the tunnel failure type and different excavation and supporting sequences that could result in totally different reactions in the rock masses. Thus, these modes of failure, followed by the appropriate mechanical properties that are required for

Tunnel design, are the premise for the numerical design of the appropriate primary support measures to achieve stable tunnel conditions.



Fig1.1 East Sakaro Underground Tunnel (Derartu jibro, March 2022)

1.2 Statement of the Problem

For the stability analysis of underground openings, there are so many aspects that need to be addressed due to the complexity of the problem. The geometrical and mechanical properties of the discontinuities, the rock mass response around underground excavations which is a function of the strength of the intact rock, , the orientations and magnitudes of in situ stress, the sequence of excavation and supporting, etc. For this study the specific problem around the tunnel are (i) Lack of accurate estimation of joint /discontinuity properties (ii) Progressive failure around an underground opening subjected to high stresses (iii) Lack of predicting the possible failure. Therefore, it is very important to address the above issues related to underground opening based on geological and geomechanical data available under different scenarios of field conditions.

1.3 Objective of the Study

1.3.1 General Objectives

The main objectives of the study are the rock mass stability evaluation around an underground opening at Lega Dembi gold mine in shakiso under different filed conditions.

1.3.2 Specific Objectives

1. To assess the geology and rock mass condition of the mine site
2. To determine the mechanical properties of intact rock and discontinuity around the underground opening
3. To undertake numerical simulation of the underground opening at different geologic and rock mass condition of the site.
4. To identifying the type of failure and failure zone around the underground opening
5. To design properly and provide support so as to mitigate the tunnel failure and ensure the long term stability of the opening for the safe ore transportation.

1.4 Location and Accessibility

Two of the major mines of Ethiopia are located near Shakiso: the Lega Dembi gold and the Kenticha tantalum mines. The study area, Sakaro underground tunnel is located near the open pit gold mining site which is situated in Shakiso district, near Shakiso town in southern Ethiopia. This underground tunnel is located in the Guji Zone of Oromia Region which has a latitude and longitude of 5°44'18"N 38°51'18"E and an elevation of 1758 meters above the sea level. Access routes to most parts of the study area are very limited. However, there are few secondary routes that depart from Shakiso town road which connect with some parts of the study area. The project area is accessed by 500km South of Addis Ababa. The Study area can be reached on all accessible road using four-wheel drive vehicles.

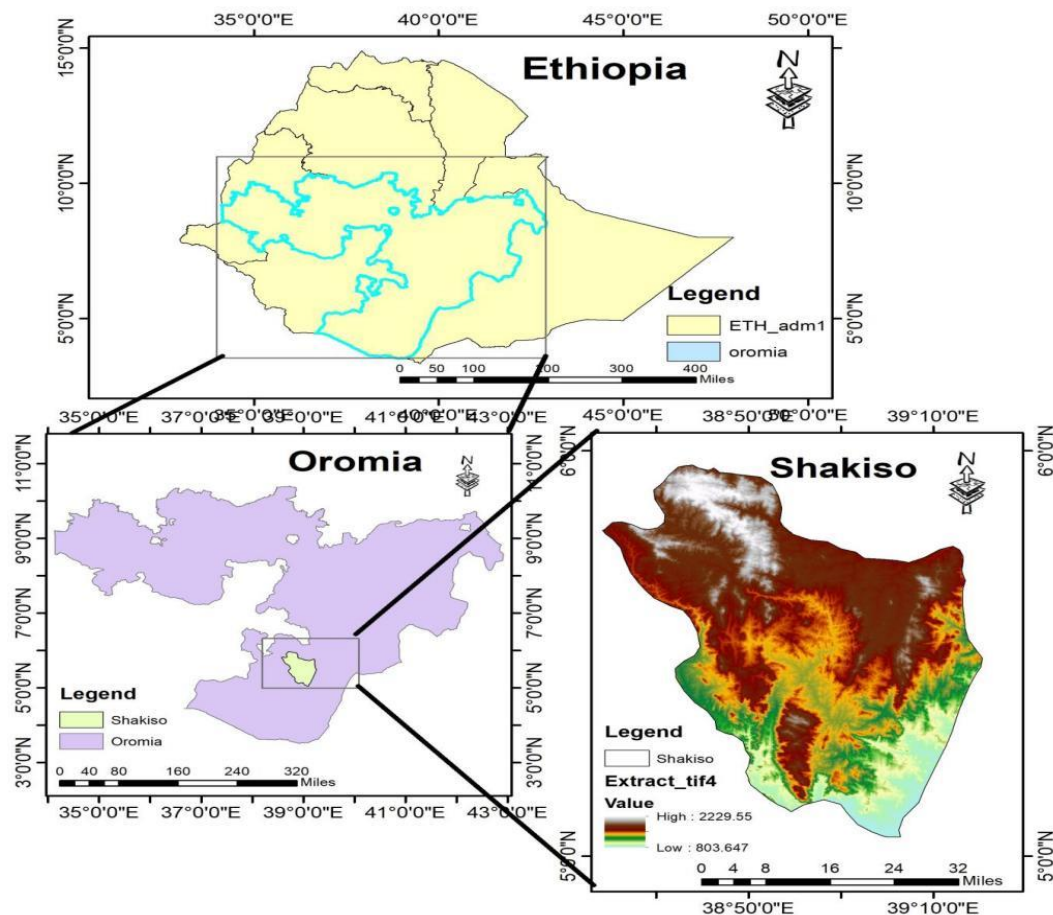


Fig1.2 Location Map of the study area (by Derartu jibro)

1.5 Climate and topography

In Shakiso, the seasons are mostly cloudy, the dry season is partly cloudy, and it is warm year round. The area has abrupt changes from the rainy season (March to May and May to mid- November) to the dry season (primarily mid-November to march). Annual precipitation ranges from 140mm over the high ground to 600mm in areas of lower elevation. The total rainfall measured at Lega Dembi for 2004 was 645mm, a significant proportion of which (135mm) fell during the month of October.

The elevation ranges from 900m to 2200m above mean sea level north of Gora village. Consequently, days are warm to hot, while night tend to be cool. Over the course of the year, the temperature typically varies from 53°F to 85°F and is rarely below 50°F or above 88°F.

The terrain is rugged with steep linear ridges and narrow valleys. Abrupt changes in elevation are related to lithology and geological structure. Metagabbro, metatonalite and metagranite generally occupy the high ground with metasedimentary rocks predominating river valleys. Faults and folds have significant impact on the drainage system geometry.

1.6 Significant of the Study

The followings are the main significant of the study:

- Investigating geological and rock mass conditions around underground opening
- Accurately modelling the complex geological condition that representing the actual site
- Suggestions for the selection of appropriate rock supports provided.
- Numerical modelling of the underground opening for the gold mine company
- A base of feature researchers working in similar projects

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

Mining is defined as the process of exploiting a valuable mineral resource naturally occurring in the earth's crust (Caro, 2007). The extraction of mineral resources which is from the earth's crust is classified into two; surface mining and underground mining (UG). In surface mining, all the extraction operation is exposed to the atmosphere while in underground mining, all the operation are conducted inside the earth's crust.

Underground mining methods are usually employed when the depth of the deposit and/or the waste to ore ratio (stripping ratio) are too great to commence a surface operation. Once the economic feasibility has been verified, the most appropriate mining methods must be selected according to the natural/geological conditions and spatial/geometric characteristics of mineral deposits (Borton, 2002). Considerations include

- Spatial/geometric characteristics of the deposit concerned i.e. about the: the shape, size, thickness, plunge and depth.
- Strength of the hanging wall, footwall and ore body. Economic value of the ore and grade distribution within the deposit.

The selection of underground mining methods is primarily based on the geological/spatial setting of the deposit. Candidate methods can therefore be chosen and ranked based on estimated operational/capital costs, production rates, availability of labors and materials/equipment's and environmental considerations. Reflecting the importance of ground support, underground mining methods are categorized in three classes on the basis of the extent of support required (Newman, 2010): unsupported, supported and caving. Unsupported methods are essentially self-supported and require no major artificial system of support to carry super-incumbent loads, instead relying on the natural competence of the walls of the openings and pillars.

The followings are considered unsupported methods: Room-and-pillar mining, shrinkage stopping, sublevel stopping etc.

Supported methods are used when production openings (stopes) are not sufficiently stable to remain open during operation but the opening is required to be held open to prevent caving or surface subsidence. In other words, the supported class is employed when the other two categories of methods, unsupported and caving, are not applicable. There are three specific methods in the supported class. Cut and fill stopping, Stull stopping, Square-set stoping. Cut-and-fill and stull stoping are intended for moderately competent rocks whereas square-set stoping is suitable for the least competent rocks. Caving methods are defined as those associated with induced, controlled, massive caving of the ore body, the overlying rock or both.

2.2 Geology of the Study Area–Review

The study area is regionally located in Adola Belt, south of Ethiopia. The detail geology of the study area was rarely known before the study was conducted by the Geological Survey of Ethiopia. The Adola Belt is characterized by two major rock associations arranged in five intercalated lithostructural domains. The first group constitutes the high grade, polydeformed and metamorphosed schists and gneisses whereas the second group consists of metavolcano-sedimentary secessions and ultrabasic rocks. The map of the Adola Belt and the surrounding areas shows the relationship between the Adola, Moyale and Bulbul Belts, the major NW-SE faults and the N-S trending shear zone. The volcano-sedimentary sequence and ultrabasic rocks which forms most part of the Adola Belt are terminated against NW-SE trending.

2.3 What is underground instability?

For stability evaluation and support design, a thorough understanding of the rock mass behavior around underground tunnels is required. It is more likely that underground instability will occur in a mining process. Blocks generated by discontinuities either fall or slide from the excavation periphery as a result of an applied force (typically gravity) or stresses induced by the excavation process as well as which causes tunnel instability complex geology and engineering activities.

The design of underground excavations in hard and jointed rock masses must take structurally controlled instability into account. Gravity falls of wedges or blocks from the

excavated roof or sliding along planes or lines of junction of planes in the roof and sidewalls can all contribute to instability.

One of the issues with ore deposit excavation in underground mines is the local disturbance which makes it unstable and results in the release of energy in the form of roof falls. A variety of factors influence the scope and severity of such events.

2.4 Factors Affecting the Stability of Underground Opening

In terms of the subsurface scenario, lithology of the area, geological structures and groundwater affects both the stress state and rock strength (water in some rocks reduces the rock strength) and in situ stress are all elements that affect the roadway or the area near the tunnel. The lateral stress system and the in situ stress field induced by the overburden strata must be considered. A range of elements, such as the joint contact area, the joint aperture, the roughness of the joint wall, the relevant properties of the filling material, and so on, can affect the mechanical properties of joints (Goodman et al., 1968).

Different forms of failure for rock slopes include plane, wedge, toppling and rock fall (; Hocking, 1976; Hoek and Bray, 1981). When tension exceeds the rock strength or when substantial differential stresses are observed, problems may emerge around the excavations (Bhasin and Grimstad, 1996; Hudson, 2001). The physical stability of rocks is affected by the conditions under which they form such as heat, temperature, compression, pressure and molecular structure. A support system must be installed following excavation to enable the rock mass support itself.

The planar and wedge mode of failure is the most common kind of failure in rock slopes. In case of plane mode of failure, the rock mass that rests on the potential failure plane is subjected to gravitational pull. The stability of a slope with a potential plane mode of failure is determined by governing factors such as slope height (h), dip of potential failure plane, tension crack, shear strength parameters of potential failure surface (Cohesion (C) and angle of friction, height of water in tension crack and horizontal earthquake acceleration (a)) (Raghuvanshi et al., 2015; Raghuvanshi et al., 2019)

2.5 Slope Failure Mechanism

In geotechnical terms, a rock slope is a surface of rock mass that is inclined. Slopes can be natural or manmade. It is important to monitor rock slopes such as their balance, shear strength and ability to withstand movement and to ensure that slope failure does not occur. There are mainly four types of failures which are described as follows (Stead, 2015).

1. Plane failure

A planar failure of rock slope occurs when a mass of rock in a slope slides down along a relatively planar failure surface. The failure surface is usually structural discontinuities such as bedding planes, faults, joints or the interface between bedrock and an overlying layer of weathered rock. The favorable conditions for plane failure are as follows:

- the dip direction of the planar discontinuity must be within ($\pm 20^\circ$) of the dip direction of the slope face.
- the dip of the planar discontinuity must be less than the dip of the slope face.
- the dip of the planar discontinuity must be greater than the angle of friction on the surface.

2. Wedge failure

Wedge failure can occur in a rock mass with two or more sets of discontinuities whose lines of intersection are approximately perpendicular to the strike of the slope and dip towards the plane of the slope.

- The trend of the line of intersection must approximate the dip direction of the slope face.
- The plunge of the line of intersection must be less than the dip of the slope face. The line of intersection under this condition should daylight on the slope.
- The plunge of the line of intersection must be greater than the angle of friction of the surface.

3. Toppling failure

Toppling failure occurs when columns of rock, formed by steeply dipping discontinuities in the rock rotate about an essentially fixed point at or near the base of the slope followed by slippage between the layers. Jointed rock mass which is closely spaced and steeply dipping discontinuity sets that dip away from the slope surface are necessary prerequisites for toppling failure.

4. Rotational failure

The failure surface in a section of rotating slips can be a circular arc or non-circular curve. Circular slips are generally associated with homogeneous soil conditions whereas non-circular slips are associated with non-homogeneous conditions. When the shape of the failure surface is altered by the presence of a neighboring stratum of significantly differing strength, translation and compound slides occur. Translational slips are most common where the next stratum is at a shallow depth below the surface and the neighboring stratum is planar and generally parallel to the slope. When the next stratum is at a larger depth, compound slips occur, with the failure surface comprising curved and planar parts. A rotating slide is the movement of material along a curved surface.

2.6 Shear Strength Failure Criteria of Rock

2.6.1 Mohr-Coulomb Strength Criteria

The Mohr-Coulomb criterion assumes that the maximum shear stress controls failure and this failure due to shear stress is dependent on the normal stress. This can be shown by drawing the Mohr's circle in terms of maximum and minimum stresses for different states of failure. The best straight line that touches these Mohr's circles is the Mohr-Coulomb failure line. As a result, the Mohr-Coulomb criterion is expressed as (Coulomb, 1972):

$$\tau = C + \sigma \tan \phi \dots\dots\dots (2.1)$$

Where τ is the shear stress, σ is the normal stress (negative in compression), c is the cohesion of the material and ϕ is the angle of friction.

Effective stress controls the shear strength of rocks and soils which is independent of whether the failure occurs under drained or undrained conditions. The Mohr-Coulomb strength envelope depicts the connection between shear strength and effective stress.

2.6.2 Hoek Brown Strength Criteria

Because representative specimens are too large for laboratory testing, estimating the mechanical behavior of closely jointed rock masses is one of the fundamental challenges in rock mechanics.

For design purposes, practitioners are frequently asked to anticipate the strength of large-scale rock masses. Fortunately, Hoek and Brown (1980) established an empirical failure criterion for intact and jointed rock masses based on the curve fitting of triaxial test data. The criterion is based on the Geological Strength Index classification system (GSI). One of the few non-linear criteria widely accepted and used by engineers to evaluate the strength of a rock mass is the Hoek–Brown criterion. (Li A, 2008). Details of the implementation of the Hoek–Brown criterion into the numerical analysis formulations can be found in Merifield (2006). In this study, the latest version of Hoek–Brown failure criterion (Hoek, 2002) is employed.

$$\sigma_1' = \sigma_3' + (mb \times \sigma_3'^a \times \sigma_{ci} + s) a \dots\dots\dots (2.2)$$

where σ_1' and σ_3' are the maximum and minimum effective principal stresses at failure, mb is the value of the Hoek-Brown constant for the rock mass, s and a are constants which depend upon the rock mass characteristics, and σ_{ci} is the uniaxial compressive strength of the intact rock pieces.

Where:

$$mb = m_i \times \exp(GSI - 10028 - 14D) \dots\dots\dots (2.3)$$

$$S = \exp(GSI - 1009 - 3D) \dots\dots\dots (2.4)$$

$$a = 12 + 16(e^{-\frac{GSI \times 15}{200}} - e^{-203}) \dots\dots\dots (2.5)$$

The GSI (Geological Strength Index) was introduced because Bieniawski's rock mass rating (RMR) system (Bieniawski, 1989) and the Q-system (Barton, 2002) were deemed to be unsuitable for poor rock masses. The GSI ranges from about 10, for extremely poor rock masses, to 100 for intact rocks. The parameter D is a factor that depends on the degree of disturbance.

The uniaxial compressive strength is obtained by setting $\sigma_3=0$,

$$\sigma_c = \sigma_{ci} \times S \times \alpha \dots\dots\dots (2.6)$$

2.7 Underground Opening Design Approaches

2.7.1 Empirical approach: Rock mass classification systems

Rock mass rating (RMR) (Stille, 2008) and Q systems (Harrison, 2000) were developed to provide tunnel support estimates through a rating of rock masses. They are internationally accepted systems and are extensively used in tunnelling. In addition, the advancement of the geotechnical software that is easier to use led to an increased requirement of data related to mass properties. This kind of data is needed as input for analysis in the numerical solutions for designing tunnels. The onset of numerical tools to handle rock-support interaction and the advancement of concepts related to ground reaction curve permitted issues to be managed well beyond the ultimate extent for application of different tunnel support classification systems such as the RMR and Q systems. Deere and Deere (1988) proposed a methodology for getting estimates of the strength of fractured rock masses based on an appraisal of the interlocking of rock pieces and the condition of the surfaces between these pieces. For such an evaluation, the Geological Strength Index (GSI) was presented. The GSI has been established over a long period (Romana, 2003) to meet the desires of practitioners and cases that were not at first realized. The application of the mass properties from the GSI values basically accepts that the rock mass behaves isotropically. However, it is not factual where there is a clear anisotropic behavior which is characterized by favored failure surface or discontinuities. The appropriate use of rock mass characterization systems notably the GSI (Barton, 2002) allowed to quantify difficult grounds for the evaluation of its geotechnical properties for the selection of the design and support parameters.

i) The RMR system

The RMR system has established many applications in several engineering projects like tunnels, foundations, mines and slopes but utmost applications are in the field of tunnelling (Bieniawski, 1992). The system originated specifically for estimating the rock tunnel support (Barton, 2008). For this purpose, to estimate the adequate tunnel support measure, the quantitative valuation of the rock-mass quality is connected with tunnel support design. The parameters which created the system were eight and then modified to six parameters. Consequently the basic RMR score is the sum of the scores of five stated parameters. It comprises the score for point load strength (R_1), RQD (R_2), joint spacing (R_3), joint condition (The discontinuity condition term is a function of discontinuity persistence, aperture, roughness, infilling and weathering) (R_4) and groundwater condition (R_5). Once adjusting RMR for joint direction with respect to the tunnel alignment and direction of excavation score (R_6). The corresponding quality is achieved using the following equation

$$\text{RMR} = R_1 + R_2 + R_3 + R_4 + R_5 + R_6 \dots\dots\dots(2.7)$$

One of the oldest but still a widely used system is RMR (Bieniawski, 1989). It was developed in South Africa for underground mining but currently widely used in civil engineering where excavation and support are determined by the RMR value which results in five different support classes. Adjustment factors and refinements are possible. Based on a combination of five parameters each parameter is expressed by a point rating.

Table 2.1 Rock mass classification RMR system (Bieniawski, 1989) A.
CLASSIFICATION PARAMETERS AND THEIR RATINGS

Parameter			Range of values						
1	Strength of intact rock material	Point-load strength index	>10MPa	4 – 10MPa	2 – 4MPa	1 – 2MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial compression strength	>250MPa	100 – 250MPa	50 – 100MPa	25 – 50MPa	5-25 MPa	1-5 MPa	<1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		>2m	0.6 – 2m	200 – 600mm	60 – 200mm	<60mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation <1mm Slightly weathered walls	Slightly rough surfaces Separation <1mm Highly weathered walls	Slickensided surfaces or gouge <5mm thick or Separation 1-5mm continuous	Soft gouge >5mm thick or Separation >5mm continuous		
	Rating		30	25	20	10	0		
5	Ground water	Inflow per 10m tunnel length (l/m)	None	<10	10 – 25	25 – 125	>125		
		(Joint water press)/ (major principal σ)	0	<0.1	0.1 – 0.2	0.2 – 0.5	>0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		
B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)									
Strike and dip orientations			Very Favourable	Favourable	Fair	Unfavourable	Very Unfavourable		
Ratings	Tunnels and mines		0	-2	-5	-10	-12		
	Foundations		0	-2	-7	-15	-25		
	Slopes		0	-5	-25	-50	-60		
C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS									
Rating			100 ← 81	80 ← 61	60 ← 41	40 ← 21	<21		
Class number			I	II	III	IV	V		
Description			Very good rock	Good rock	Fair rock	Poor rock	Very poor rock		
D. MEANING OF ROCK CLASSES									
Class number			I	II	III	IV	V		
Average stand-up time			20 yrs for 15m span	1 year for 10m span	1 week for 5m span	10 hrs for 2.5m span	30 min for 1m span		
Cohesion of rock mass (kPa)			>400	300 – 400	200 – 300	100 – 200	<100		
Friction angle of rock mass (deg)			>45	35 – 45	25 – 35	15 – 25	<15		
E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY CONDITIONS									
Discontinuity length (persistence)			<1m	1.3m	3 – 10m	10 – 20m	>20m		
Rating			6	4	2	1	0		
Separation (aperture)			None	<0.1mm	0.1 – 1.0mm	1 – 5mm	>5mm		
Rating			6	5	4	1	0		
Roughness			Very rough	Rough	Slightly rough	Smooth	Slickensided		
Rating			6	5	3	1	0		
Infilling (gouge)			None	Hard filling <5mm	Hard filling >5mm	Soft filling <5mm	Soft filling >5mm		
Rating			6	4	2	2	0		
Weathering			Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed		
Rating			6	5	3	1	0		

The addition of the points results in the RMR rating where $RMR = (RS + RQD + spacing + condition\ of\ discontinuities + groundwater) + reduction\ factor(s)$ (reduction factors for orientation, excavation damage, etc.)

RQD is used as an input parameter in RMR and Q systems (Cording and Deere, 1972; Merritt, 1972 and Deere and Deere, 1988) which is related RQD to Terzaghi's rock load factors and rock bolt requirements in tunnels. The Rock Quality Designation (RQD) was developed by Deere *et al.*, (1966) which is aimed to measure the rock quality based on core recovery by diamond drilling. The definition of RQD was the percentage of core recovered in intact pieces over 100 mm (4 in.) in length in the total length of the borehole, given by the following equation (Deere and Deere, 1988).

Nowadays, RQD is more popular to estimate the rock mass properties (Zhang and Einstein, 2004; Zhang, 2016) and used is a basic parameter in the two widely used rock mass classification systems, the rock mass rating (RMR) (Bieniawski 1976, 1989) and Q system (Barton *et al.*, 1974). Under the circumstances when RQD cannot be obtained directly from the core, some equations associated with fracture frequency were used. For example, Palmström (1982) suggested an estimation of RQD from the number of joints per unit volume for clay free rock mass is in the following manner.

$$RQD = 115 - 3.3 j_v \dots\dots\dots (2.8)$$

where j_v is the total number of joints per square meter.

ii) Q system

This classification system was established in 1974 after the RMR system based on tunnel cases. This system was independent from the RMR system but there are some parameters in common, that is, rock-quality designation (RQD) and groundwater. The system also included joint roughness, infilling of the joints and rock load which were missed in the RMR system. The plot between RQD and the tunnel span for the tunnel case records is the start of the development of support plot for the Q system. The correlation was improved between RQD and the unsupported span by including the joint set number as an additional parameter. This modified RQD will be further improved by including the joint roughness, joint alteration, water pressure and rock load. The Q-system can therefore be used as a guideline in rock support design decisions. The six parameters are applied in the following manner to compute the rock-mass quality:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \dots\dots\dots (2.9)$$

where RQD is with a minimum score of 10, J_n symbolizes the score of the joint sets number, J_r is the score for the roughness of joint surface, J_a symbolizes the score for the clay filling or alteration degree, J_w symbolizes the score of groundwater pressure effects and stress reduction factor (SRF) is the score for faulting, strength and stress ratios in hard rocks and squeezing or swelling.

This system was established on tunnelling case records for hard and jointed rock masses. It was initially developed to classify rock masses for assessing the need for supports in tunnels. The parameters rating can be based on the geological interpretations in tunnels during its construction or by core logging during exploration. Through time, the RMR and Q systems have been updated so as to increase their trustworthiness for tunnel support. Along with tunnel support, these systems are also used to calculate the rock masses' geomechanical properties, the stand-up time for the tunnel and support load (Barton, 1999, Barton, *et al.*, 2012).

Generally RQD is used in both systems as a mandatory parameter but their correlation with other jointing parameters is very difficult. The difficulty in correlation is that RQD is a one-dimensional measurement based only on core pieces longer than 0.1 m, which means that the application of RQD in rock engineering may lead to inaccuracy. These systems are misused continuously due to the limitations used and assumptions made in developing them have been ignored by users. For this reason, empirical classification systems contain a hidden critical risk. The assigned parameters are neither independent nor directly connected with rock behaviour. Therefore, the users only consider the geological characterization of the rock mass followed by ground support recommendation without reflection of the probable ground behaviour.

Rock-mass classification systems are frequently used in rock engineering and design and have gained wide attention; however, these classification systems have limitations. Nevertheless, if applied appropriately, they are valuable design tools (Palmstrom, 2006). One of the many reasons for updating the two systems is to point out their limitations. Some limitations found were inherent to the two systems, which have been proven from their use over time whereas the others were overcome with advancing the formulas and parameters further. The inherent limitations of these systems formed basis for new

classification systems. Stille and Palmstorm pointed out six necessities for a true rock mass classification system and, according to them; none of these systems fulfil these requirements (Stille and Palmström, 2003). However, Hand (1997) distinguished between the two main types of classification, that is, unsupervised and supervised classification.

Many empirical systems in rock engineering have been suggested with respect to a specific rock-mass structure and/or particular purposes. Rock mass related professionals incline to the empirical method over the theoretical and numerical methods due its simplicity (Goel, 1995). However, these empirical approaches are unable to characterize complex rock-masses. This is probably one of the causes why rock engineers have continued to develop new systems or modify and extend current ones. Each classification has its own capabilities and therefore they are used in parallel as much as possible.

Limitations of RQD: - it does not reflect fully the rock mass quality as it only considers the extent of fracturing of the rock mass and does not account for the strength of the rock or mechanical and other geometrical properties of the joints. As RQD depends on the sampling line orientation relative to preferential orientation, distribution of discontinuities but it does not accurately give exact measurement (Goel, 1995).

2.8 Numerical Methods

With the advent of advanced computer techniques the numerical methods have become an efficient and low-cost tool to solve the complex problems in rock engineering. For a practical problem, the rock mass response to construction disturbance is complicated, which is affected by various factors. Numerical modelling will capture the most relevant mechanisms of the system and provide adequate models for various engineering purposes. RocLab, Phase2 or RS2, Unwedge, Rocsupport and FEM (Finite Element Method) software are some of the most extensively used numerical methods in rock engineering for tunnel support systems. The purpose of the rock slope kinematic analysis is to discover the typologies of rock slope failures based on stereographic interpretation and to provide a foundation for empirical design. To measure the quality of the rock mass, this method has been widely utilized in rock engineering (civil and mining). RocLab is a software tool that uses the generalized Hoek-Brown failure criterion to calculate rock mass strength parameters. RocLab is a Rocscience Inc. product. It is a free piece of software. Other

geotechnical, mining, and civil engineering software solutions are available from Rocscience. It is a software tool for estimating the rock mass strength parameters based on the most recent version of the Hoek-Brown failure criterion for a rock mass (Hoek and Brown, 2002). The software also determines the rock mass's strength parameters. Phase2 mining software is an interactive program for studying and modeling soil and rock slopes. It's utilized in geotechnical, civil and mining applications to plan subterranean and surface excavations. Unwedge analysis analyzes underground tunnel excavations with intersecting discontinuities and visualizes the model in 3D, determines the stability of tetrahedral wedges formed along the tunnel perimeter and ends, and calculates their respective support requirements to achieve the design factor of safety and quickly create wedge models. The factors that affect the shear strength are water, stress field and supports. RocSupport is a simple software tool for calculating deformation in circular or near circular excavations in weak rocks as well as visualizing the tunnel's interaction with various support systems. The stress-strain connection is used in the Finite Equilibrium Method (Pinheiro, 2018).

2.9 Ground control systems

Currently, both rock bolting and shotcrete are used in underground mining; rock bolt spacing is determined only by the rock mass quality whereas shotcrete thickness is determined by the tunnel span as well as the rock mass quality. Their contribution to shotcrete thickness, however, is not always equal. The development of the back analysis for rock has been based on this principle of support (Zhang, 2004).

Large-scale in situ tests can provide a reliable approximation of joint properties while the cost should also be considerable subject. According to the change in maximum displacements around the tunnel, some researchers determined that joint shear stiffness was the most sensitive factor for their model while joint friction angle was the least. Furthermore, investigations into the discontinuities focused on the development of appropriate joint constitutive laws (Souley et al., 1997; Cui et al., 2016; Sainsbury and Sainsbury, 2017). Numerous scholars have made efforts to date to find an accurate representation or modeling of discontinuities for practical situations.

CHAPTER THREE

MATERIALS AND METHODS

3.1 General

It is essential to examine the available rock-mass behaviour information whether they are addressed by the Rock mass classification systems or Numerical systems. The two rock mass classification systems, RMR and Q, are based on jointed rock mass data. The numerical softwares, Roclab analyzes from datas that are obtained from rock mass classification and then Unwedge, FEaM and Rocsupport are analysed based on the Rocdata information in order to decide appropriate support system.

3.2 Methodology

For this research work, different methods are being integrated to obtain information on both engineering geological and rock stability conditions of the study area. The methodology that is used in this study to achieve the objectives of the research encompasses reviewing and analysing of existing data, collecting field data and sampling, laboratory testing and data processing. Then primary and secondary data were being integrated in to analyse, summarize and present the results. The approach and some applications of the aforementioned methodologies will be discussed as follows.

3.2.1 Review and analysis of existing data

Prior to the field work, preparation was being undertaken. Important field equipment was collected. This is always carried out at an early phase of the research before going to field work and involves collecting and analysis of existing information at office. From existing topographic maps and previous study in the study area, information like structure, topography, ground water condition, and lithology was being reviewed and compiled. Depending on lithological units and geological structures of study area, suitable line of a tunnel was selected to collect sample during field work.

3.2.2 Detailed fieldwork and sampling

During field survey, detail geological and engineering geological assessment of the tunnel was carried out at the site. These include detailed discontinuity surveying, field description of soils and rocks, mapping and sampling for laboratory analysis. Geologic mapping was carried out using the representative tunnel lines which intersect various lithological units and major geological structures.



Fig 3.1 Underground rock formation of the study area /Rock type: Graphitic quartz mica schist/ (Derartu jibro, March 2022)

3.2.3 Laboratory Analysis

After completion of field sampling, geotechnical laboratory tests were carried out on representative rock samples, metagranodiorite, amphibolite, quartz mica schist and graphitic quartz mica schist. The tests that were performed for rocks are point load, dry and saturated density, Schmidt hammer, and slake durability test.

Table 3.1 Analytical method adopted for rock sample Parameter

Parameters	Methods	Standard
Strength	Point load test	ASTM D 2938
Density	Dry density determination	ISRM 1978
Durability	Saturated density test	ISRM 1978
	Slake durability test	ASTM D 4644

A) Schmidt Hammer test

Assessing the compressive strength of rock using suitable co-relations between rebound index and compressive strength, assessing the uniformity of rock, assessing the quality of the rock in relation to standard requirements and assessing the quality of one element of rock in relation to another can be done with the rebound hammer method.

When the rebound hammer's plunger is forced into the rock's surface, the spring-controlled mass rebounds and the amount of rebound depends on the rock's surface hardness. The compressive strength of the rock is assumed to be proportional to the surface hardness and thus the rebound. The rebound number or rebound index is calculated using a graduated scale.

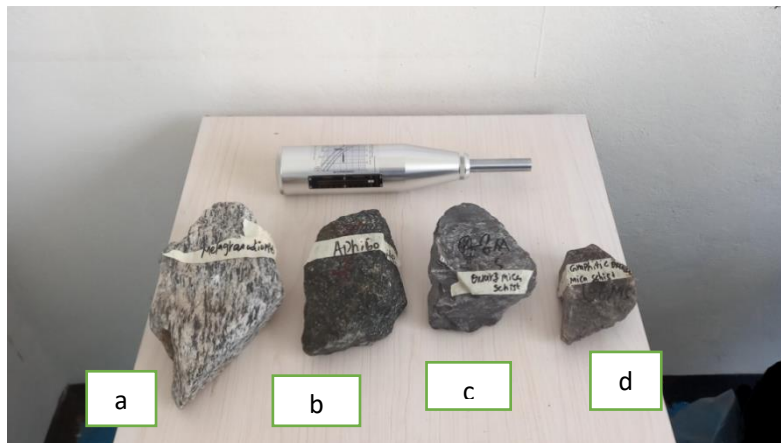


Fig3.2 schimdt Hammer Test of rock samples a) Metagranodiorite=RS1, b) Amphibolite=RS2, c) Quartz mica schist=RS3, d) Graphitic Quartz mica schist=RS4



Fig3.3 schimdt Hammer Test for RS1



Fig3.4 schimdt Hammer Test for RS2

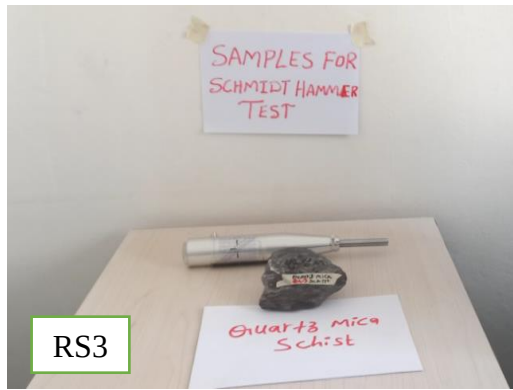


Fig3.5 schmidt Hammer Test for RS3

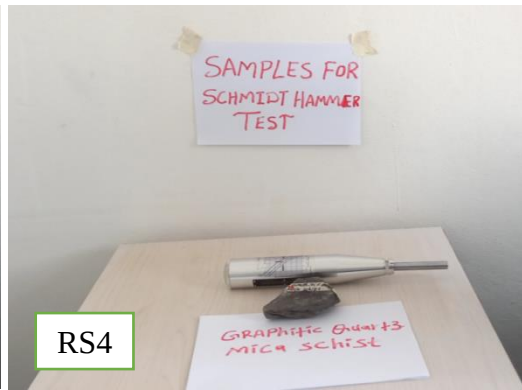


Fig3.6 schmidt Hammer Test for RS4

B) Point load strength test/Brazilian test

As a core logging index, this test has grown in popularity. The test, which was initially known as the Brazilian test and was used to assess the tensile strength of brittle rocks, is at least 20 years old. The point load test is a simple and low-cost method of determining the quantitative rock strength. The test yields a reliable assessment of the rock's uniaxial compressive strength if a clean tensile fracture runs between the loading locations. The point load test uses standardized equipment such as a pair of point loading platens mounted on a hydraulic hand-pump for field use or a loading frame for laboratory application. The load at failure, P , is recorded for the strength calculation.

The point load test is ineffective on very soft rocks like mudstone and claystone, as well as soft evaporites. When working with anisotropic rocks like slate, it is critical to make sure that loading direction is either parallel to or perpendicular to the prevalent weakness direction. One downside of the point load test, which is common to all other strength tests, is that the rock is broken, which might cause confusion when interpreting fractures in the core. Before the rock is used for point load tests, photographing of the core and determination of RQD (Rock Quality Designation) which is based on the number of intact core pieces of more than 10cm length should be performed.

A testing plan was prepared and, if necessary, a sampling plan to deliver specimens for point load testing using the protocols stated below for the specific specimen shapes (diametral, axial, block or irregular). For diametral testing, rock specimens with a length/diameter ratio larger than one are appropriate. The specimens were placed in the test

equipment and the platen was closed to establish contact along the diameter of a rock. The distance L was checked which was between the contact points and the nearest free end is at least 0.5 times the diameter of the rock. D and L were calculated and the tracks of them were kept properly. Rules that must be followed when conducting point load test;

The D diameter must be between 15-85mm

$$\checkmark \text{ Width average} = \frac{\text{width1} + \text{width2}}{2}$$

Length must be $0.5 < L$ and $IP = D/W_{av}$

The uncorrected point load strength, Is , is calculated as:

$$Is = \frac{P}{De^2}, \text{ MPa} \quad \text{Where: } P = \text{failure load, N,}$$

De = equivalent core diameter (mm), and is given by:

$De^2 = D^2$ for diametric core tests without penetration, mm^2 , or

$De^2 = \frac{4A}{\pi}$ for axial, block, and lump tests, mm^2 ;

$De^2 = \frac{4}{\pi} (W \times D)$ for other shapes

Where: $A = W \times D$ = minimum cross-sectional area

$De^2 = \frac{4}{\pi} (W \times D)$ for other shapes

$$F = \frac{De^{0.45}}{50}$$

Where: F is the size correction factor for tests near the standard 50-mm size,

Therefore; - $Is_{(50\text{mm})} = F \times Is$

$UCS = 24 \times Is_{(50\text{mm})}$

➤ Uniaxial compressive strength test



Uniaxial compressive strength, Poisson's ratio and Young's modulus are all determined using the uniaxial compressive test. The unconfined compressive test is also known as the uniaxial compressive test. In this test, rock samples are squeezed in circular cylinders parallel to their longitudinal axis. It is the simplest and oldest test for assessing the characteristics of rocks and it remains one of the most practical and useful methods determining the shear strength of rocks.



Fig3.7 Point load test of rock samples



Fig 3.8 A) Rock samples before
Conducting point load test



Fig3.9 B) Rock samples after Point load test is
done

C) Dry and saturated density test

Four methods used to determinate bulk density of rock were investigated in this research using four diverse rock types. The total mass of each rock sample was weighed before after

putting it in oven. The selected rocks were put in oven for 24hrs for dry density calculation. Saturated density test was also done in the same procedure, but the difference is the rock samples were put in water for 24 hour in order to calculate the saturated density. The density is obtained from the mass of a rock and from the final volume minus initial volume.

Volume rock sample= final volume - initial volume

$$\text{Dry density} = \frac{\text{dry mass}}{\text{volume}} \dots\dots\dots \rho = \frac{m}{v}$$

$$\text{Saturated density} = \frac{\text{saturated mass}}{\text{volume}}.$$



Fig3.10 Samples prepared for dry density test



Fig 3.11 Measuring the volume of rock samples and preparing them to be oven dried for 24 hours



Fig3.12 preparing rock samples to conduct saturated density test



Fig3.13 Conducting Saturated density test for rock samples



Fig3.14 Rock samples obtained from saturated density test after being placed in water for 24 hours

D) Slake durability test

The resistance of a rock sample to weakening and disintegration caused by a conventional drying and wetting cycle will be discussed. This slake-durability test is used to determine how resistant rocks are to weathering.

Samples are tumbled in mesh drums through a water medium and then oven-dried for two cycles in this test. The Slake Durability Index is a measure of mass loss in percent.

Durability testing is a method of determining a system's properties over time under varied load circumstances. This test is used to determine a rock sample's resistance to weakening and disintegration after being exposed to two normal cycles of drying and wetting in a slaking fluid, usually water. This testing allows us to determine the stability of transaction response times over time.

To perform this test, the sample was dried to a consistent mass in an oven set to 105°C. The sample was placed in the machine's drum and the weight of the sample plus the drum was recorded as 'A.' The cover was fit to the drum and was placed in the trough. The trough was filled 20mm below the drum axis with slaking fluid. The drum was rotated at 20 revolutions per minute for 10 minutes. Then the drum was taken out of the trough and the lid was removed. The drum and any remaining sample were dried in an oven set to 105°C. As 'B,' the weight of the drum plus the sample that was kept was added. Steps 2–5 were repeatedly done for another 10 minutes.



Fig3.15 Preparing rock samples for slake durability test



Fig3.16 Conducting slake durability test



Fig3.17 Slake durability test for RS1



Fig3.18 Slake durability test for RS2



Fig3.19 Slake durability test for RS3



Fig3.20 Slake durability test for RS4

3.3 Design of the tunnel using Rock mass classification method

The rock mass classification systems (RMR, Q and GSI systems) are empirical methods used in rock engineering projects. The methods are preferred to be used in the early stage of an underground project when limited information is available. With the development of several mature systems, they play important roles on the evaluation of rock mass conditions, the design of preliminary support system, the implementation of appropriate construction sequence and the determination of location or layout of openings. In our case RQD was calculated from core drilling data and RMR was obtained from the parameter on the field measurement.



Fig3.21 Core drilling data from Sakaro underground tunnel



Fig3.22 Mesurement of core drilling data for calculating RQD and RMR

Table3.2 Rock mass classification values of the rock samples collected field and core drill

Rock samples	Rock ID	RQD	RMR	GSI
Metagranodiorite	RS1	49	70	65
Amphibolite	RS2	58	80	75
Quartz-mica schist	RS3	54	76	71
Graphitic-quartz mica schist	RS4	53	78	73

3.4 Numerical Modelling and support system design (Rock stability Analysis)

With the advancement of advanced computer technology, the numerical method has become an efficient and low-cost tool for solving complex rock engineering problems. In practice, the rock mass response to construction disturbance is complicated, influenced by a variety of factors. Numerical modeling will capture the most important system mechanisms and provide appropriate models for various engineering purposes. In this case FEM, Phase2, Rocdata, Unwedge, and Rocsupport are the softwares that will be used as numerical methods in rock engineering.

3.4.1 Finite Element Method (FEM)

Despite the fact that FEM is based on the stress-strain relationship, push re-distributions are undoubtedly better processed. Previous research have shown that the finite element method (FEM) with shear strength reduction methodology is useful for analysing the performance together with the slope's safety factor and detecting the failure surface. The shear strength reduction technique uses FEM to reduce soil strength parameters until the soil collapses. The strength reduction factor (SRF) is the FOS (factor of safety) obtained by this approach (SRF). For further information; the shear strength parameters (cohesion and internal friction angle) are continuously reduced until the solution diverges from convergence criteria like displacement, force, or strain energy. The slope is supposed to have failed at that point. The ratio of initial and reduced strength values during failure is known as the factor of safety. The FEM approach to finding the slopes factor of safety will meet the criteria for effective computer-aided analysis.

3.4.2 Kinematic Analysis

The Kinematic analysis is a method of determining the probable modes of failure and failure zones for unfavorable jointed rock slopes. The Braton compass was used to measure the dip and dip direction of the rocks discontinuity orientation.

Kinematic analysis option in Dips software allows you to easily and quickly check for various rock slope stability failure modes on a Stereonet plot. In this case, simply input the slope orientation needed are (Slope dip and dip Direction), friction angle, and lateral limits and then choose one of the following failure modes

❖ Dips

A Dips is a program designed for the interactive analysis of orientation-based geological data. The software is kinematic analysis software that is used to determine the failure type and critical surfaces. It is used to perform the kinematic analysis of rock slopes and used to be capable of analyzing the potential for the various structurally controlled modes of rock slope failures (plane, wedge and toppling).

i) Planar Sliding

✓ Planar Sliding Assumptions

An important assumption regarding “pure” planar sliding on a single plane, is that release planes are assumed to exist (e.g., lateral joints, tension cracks or other mechanism) which allow a planar sliding failure to occur.

Such release planes are not explicitly modeled in the planar sliding kinematic analysis but what we should be aware is that some release mechanisms must exist to allow a block sliding on one plane to be removed from the slope. Planar sliding can be considered as a special case of wedge sliding where sliding takes place on only one plane and other planes act as release planes.

✓ Daylight Envelope / Slope Plane

The planar sliding **kinematic analysis** failure mode is a test for sliding on a single plane.

The key elements of the planar sliding kinematic analysis are:

- Daylighting condition for planes
- Pole friction cone (angle measured from center of stereonet)
- Lateral limits (optional)

For planar sliding, the daylighting condition can be defined either by a daylight envelope (when considering pole vectors) or by the plane itself (when considering dip vectors).

When considering pole vectors, all pole vectors that fall within the daylight envelope (of the slope plane) represent planes which can kinematically daylight from the slope (i.e., the dip vectors of these planes all point "out" from the slope plane).

When considering dip vectors, all dip vectors that fall outside the slope plane (great circle) represent planes which can kinematically daylight from the slope.

These two criteria are exactly equivalent. All poles that fall within a daylight envelope are represented by corresponding dip vectors outside the slope plane. In fact, the daylight envelope itself is defined by all possible pole vectors whose dip vectors lie exactly on the slope plane (i.e., parallel to the slope plane).

✓ **Friction Cone**

A friction cone defines the limits of frictional stability on a stereonet. It is important to note the following:

When considering pole vectors, we use a pole friction cone and the friction angle is measured from the center of the stereonet. All poles outside of the pole friction cone represent planes which dip steeper than the friction angle and can slide if kinematically possible. The cone angle for a pole friction cone = friction angle.

When considering dip vectors (or intersections), we use a plane friction cone and the friction angle is measured from the equator (perimeter) of the stereonet. All dip vectors inside the plane friction cone represent planes which dip steeper than the friction angle and can slide if kinematically possible. The cone angle for a plane friction cone = 90° - friction angle.

✓ **Lateral Limits**

The daylighting conditions described above for planar failure represent theoretical limits of all possible planes which can daylight from a slope face.

In practice it has been observed that planar failure tends to occur only if the dip direction of a plane is within a certain angular range of the slope face dip direction. Typically a value

of 20° to 30° is used based on empirical observations (Goodman 1980; Hudson and Harrison 1997).

You can define the lateral limits for planar failure in the sidebar or the kinematic analysis dialog. The lateral limits define two straight lines through the center of the stereonet, at plus/minus the specified angle from the slope dip direction. The lateral limits add a further restriction to the critical zone for planar sliding (poles or dip vectors must be within the lateral limits in order to slide).

If the lateral limits are not considered, then the entire daylight envelope will be considered kinematically valid. All poles within the daylight envelope (or all dip vectors outside the slope plane) will be considered for Planar Sliding. This represents a worst case and most conservative scenario in which all theoretically possible planes are considered for Planar Sliding.

ii) Wedge Sliding

The **wedge sliding kinematic analysis** failure mode is a test for sliding of wedges formed by the intersection of two planes.

The key elements of wedge sliding kinematic analysis are:

- Slope plane
- Intersection plotting
- Plane friction cone (angle measured from perimeter of stereonet)

Wedges can slide on two planes (along the line of intersection) or on one plane (if one plane has a more favorable orientation for sliding than the line of intersection). Wedge Sliding is based on the analysis of intersections. Dip vectors of individual planes are also considered to determine if wedges slide on two planes or one plane. Pole vectors are not used.

When two joint planes intersect, they can form a wedge which can slide out of the slope (depending on kinematic and frictional considerations). The intersection of two planes

forms a line in 3-dimensional space. The trend/plunge of the line plots as a single point on a stereonet which we refer to as an intersection.

Wedge sliding kinematic analysis is based on the analysis of intersections. If an intersection point satisfies the frictional and kinematic conditions for sliding then it represents a risk of wedge sliding

The primary critical zone for Wedge Sliding is the crescent shaped area:

- Inside the plane friction cone and
- Outside the slope plane

Sliding on Two Planes or One Plane

Depending on the wedge geometry, a wedge may slide along the line of intersection (i.e., on two planes) or on one plane. Sliding on one plane can occur when the dip vector of one of the wedge planes has a more favorable orientation for sliding than the line of intersection.

Note: Other failure modes such as rotational sliding or toppling are not considered in this analysis.

iii) Direct Toppling

The direct toppling **kinematic analysis** failure mode is a test for direct toppling as defined in Hudson and Harrison (1997).

The key elements of direct toppling kinematic analysis are:

- Two joint sets intersect such that the intersection lines dip into the slope and can form discrete toppling blocks
- A third joint set exists which act as release planes or sliding planes allowing the blocks to topple

Direct toppling can be an issue when you have near vertical intersections dipping into the slope and near horizontal base planes which undercut the blocks and form release planes.

3.4.3 Phase2/RS2

RS2 (Phase2 9.0) is a powerful 2D finite element program for soil and rock applications (RS2 = Rock and Soil 2-dimensional analysis program). RS2 can be used for a wide range of engineering projects and includes excavation design, slope stability, groundwater seepage, probabilistic analysis, consolidation and dynamic analysis capabilities.

With RS2, complex and multi-stage projects can be easily analyzed with a wide range of supporting modeling options and various material models, such as Mohr's Coulomb, Generalized Hoek-Brown and Cam-Clay. One of its major features is that finite element slope stability analysis can be performed using the shear strength reduction method. It also includes steady state, finite element groundwater seepage analysis, which is built right into the program.

In addition, Phase2 is a 2-dimensional plastic finite element program for calculating stresses and displacements around underground openings, and can be used to solve a wide range of mining and civil engineering problems involving plane strain, elastic or plastic materials, staged excavations (up to 50 stages), multiple materials, support (bolts / shotcrete), constant or gravity field stress, jointed rock, groundwater (include pore pressure analysis). The phase2 program consists of 3 program modules: model, compute and interpret.

3.4.4 RocData/RocLab

RocData is a versatile toolkit for the analysis of rock and soil strength data and the determination of strength envelopes and other physical parameters. RocData is an interactive toolkit for the analysis of rock and soil strength data. Strength envelopes and other parameters can be determined from curve-fitting of test data. RocData can be used to determine the parameters of linear and non-linear strength envelopes for rock and soil, based on the analysis of triaxial or direct shear strength data. RocData utilizes four of the most widely used strength models in geotechnical engineering - Generalized Hoek-Brown, Mohr-Coulomb, Barton-Bandis and Power Curve.

Failure envelopes are plotted in both shear-normal and principal stress space. Shear-normal stress envelopes generated by RocData can be applied to problems such as limit equilibrium analysis of slope stability while the principal stress envelopes can be used for modeling the behavior of underground excavations. RocData is a highly interactive program that allows users to easily test different strength parameters and observe how they impact a failure envelope, giving users a better understanding of the material strength.

The material strength properties determined by RocData can be used as input for numerical analysis programs such as Phase2 (finite element stress analysis and support design for excavations).

3.4.5 RocSupport

RocSupport is a quick and simple to use program for estimating the deformation of circular tunnels in weak rock and visualizing the tunnel interaction with various support systems.

The analysis method used in RocSupport is commonly referred to as "rock-support interaction" or "convergence-confinement" analysis. This analysis method is based on the concept of a "ground reaction curve" or "characteristic line," obtained from the analytical solution for a circular tunnel in an elasto-plastic rock mass under a hydrostatic stress field. Given the tunnel radius, in-situ stress conditions, rock parameters and support parameters, a ground reaction curve and a support reaction curve are calculated. The intersection of these curves determines a factor of safety for the support system. Account for long term tunnel behaviour by applying a strength reduction factor to obtain a long-term Ground Reaction Curve

CHAPTER FOUR

GEOLOGY

4.1 Regional Geology and Structural Setting

Variations in the geometry of the advancing West and East Gondwana caused a three-part tectonic division during the Ediacaran assembly of the Arabian-nubian Shield. In Ethiopia and Kenya, pure shear-dominated transpression and lateral extension produced a pervasive north-south structural trend represented by north-trending shear zones and 'greenstone belts.' The Arabian-Nubian Shield encompasses Yemen, Egypt, Sudan, Eritrea, Ethiopia, and Kenya. It is one of the world's largest swaths of Neoproterozoic crust, formed between 870 and 550 Ma. It is 3500 kilometers long and more than 1500 kilometers wide at its widest point. The dominant features are intra-oceanic arc volcanism, broad batholithic granitoid intrusions, and late molassic volcano-sedimentary sequences. Gold and/or base metal deposits can be found throughout the shield, including porphyry Cu-Au deposits (e.g., Jebel Ohier in Sudan); volcanic hosted massive sulphide (VHMS) Cu-Zn, Cu-Au, and Au deposits (e.g., Jabal Sayid in Saudi Arabia; Bisha-Hambok cluster in Eritrea; Hassai/Ariab in Sudan); and epithermal gold deposits (e.g. (e.g., Sukari in Egypt; Al Sukhaybarat, Bulghah in Saudi Arabia).

4.2 Tectonic and Structural Setting

The Arabian-Nubian Shield's current shape is the result of an early broad-scale stage of doming, a post-Jurassic extensional feature that culminated in the opening of the Red Sea Rift. A series of terranes were deformed, metamorphosed, and accreted between 850 and 550 Ma, with peak metamorphism and ubiquitous shearing occurring primarily between 650 and 500 Ma, coinciding with the Shield's final construction at 620 Ma. Historically, there have been numerous gold occurrences in Ethiopia, both primary and placer. The Adola belt produced >55 t of placer gold between 1936 and 1996. The output of illegal artisanal miners is not included in this number. In the 1950s, placer production began to fall, and by 1978, it had been drastically reduced. In 1975, artisanal miners in Sakaro and Dermi Dama discovered two hard rock gold discoveries. Following that, in the late 1970s, EMRDC (Ethiopian Mineral Resources Development Corporation) conducted an

aggressive and detailed exploration operation in the Adola area, which led to the discovery of the Lega Dembi and Sakaro main gold discoveries in 1979. The deposits are found in the South Ethiopia Terrane in the Arabian-Nubian Shield's southwestern part, specifically in the Adola granite-greenstone belt, which covers an area of around 5000 km².

The Lega Dembi deposit was first mined as an open pit by the government-owned before being sold to the Saudi-backed MIDROC Gold in 1997. Before going underground, MIDROC continued to mine in an open pit. Lega Dembi's reserves were depleted in 2018, and the mine was shut down. Mining alluvial and eluvial gold in Sakaro began in 1986, and by 1991, had produced 895.7 kg. In 2009, MIDROC Gold began mining in Sakaro as an underground operation (Girma, 1993; Billay et al., 1997; Tadesse, 1999).

4.3 Regional Context

The Adola greenstone belt is made up of two blocks of metamorphosed supracrustal rock that run north-south. Each is 5 to 15 kilometers broad, and they are 10 kilometers apart. They are the following:

✚ **The Megado volcanosedimentary belt:** in the west is made up of ultramafic and tholeiitic mafic volcanic rocks and intrusives intercalated with arkose, feldspathic quartzite, quartzite, and pelite, as well as polymictic conglomerate and greywacke. Small pod- or lens-like entities, mostly tonalitic, intrude into mafic rocks (Gilboy 1970; Chater 1971; Bisrat 1993; Ghenzebu et al., 1994; Worku and Schandelmeier 1996). The tectonic eastern edge of this belt is known as the Lega Dembi-Aflata shear zone in the region (Worku and Yifa 1989; Worku and Schandelmeier 1996). Gneissose tonalite has primary intrusive contacts with supracrustal assemblages on the western border.

✚ **The Kenticha ultramafic belt:** in the east is dominated by ultramafic rocks with subordinate amphibolites and sedimentary rocks, which are found as biotite schists and small graphitic schists and marbles (Gilboy 1970; Chater 1971). Gneissic rocks, including monotonous quartzo-feldspathic biotite gneisses with subordinate muscovite-quartz schists, staurolite-garnet-biotite schists, impure marbles, and amphibolites, separate and flank the two belts (Gilboy 1970; Chater 1971; Kozyrev et al., 1985; Ghebreab 1989; Worku and Yifa 1989). The gneissic terrane is intruded by large tonalite masses. Gneissose granites are found only in gneissic

terrain, but post-tectonic granites can be found on the margins of greenstone belts and within the gneissic terrane. Gichile and Fyson (1993) dated a tonalite intrusion to the west of the Megado band at 765 Ma, while the gneissic basement, which is associated with gneisses from the Mozambique belt, is Neoproterzoic in age (Kazmin et al., 1978; Gass 1977, and references therein; Vail 1976, 1983).

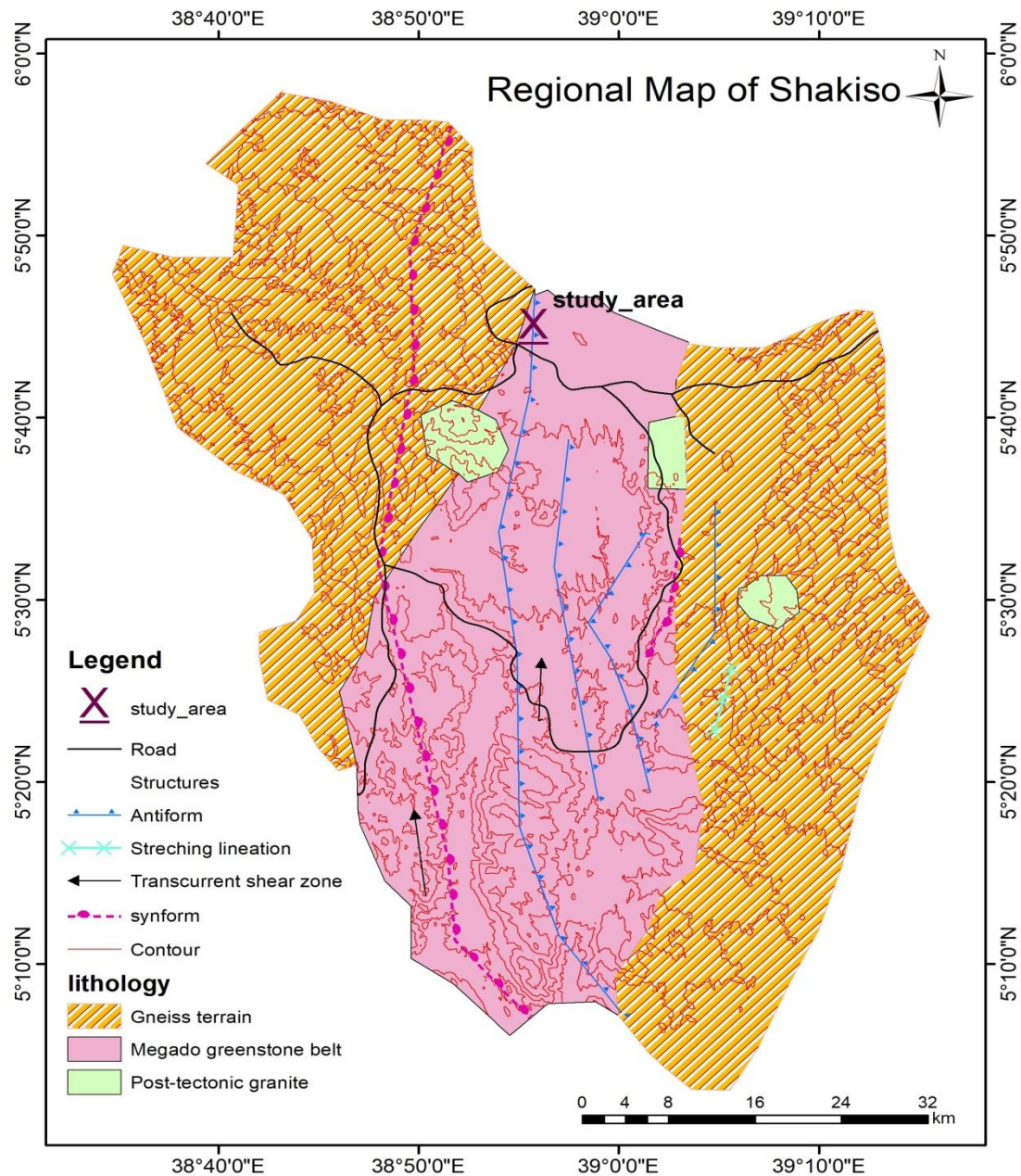


Fig4.1 Regional geology of Shakiso (by Derartu jibro; 2022)

4.4 Structure

The Lega Dembi and Sakaro deposits are both located on or near the eastern edge of the Megado volcanosedimentary belt.

Within the Megado belt and neighboring gneissic terrane, five primary deformations have been identified:

1. - an early gneissosity-forming event in the gneissic terrane, manifested as early folds in the Megado belt, and associated with a compressive thrust event;

That resulted in regional north-south trending vertical folds that constitute the region's primary structural grain and are linked to an east-west contractional event (Worku and Schandelmeier 1996);

2. - strike-slip shearing along the greenstone-gneissic domain contacts as a result of a NW-SE directed transpressional event (e.g. Beraki 1995; Worku and Schandelmeier 1996)

3. - strike-slip shearing along the contacts between the greenstone and gneissic domains as a result of a NW-SE directed transpressional event (e.g. Beraki 1995; Worku and Schandelmeier 1996)

4 - responsible for east-west trending, upright, moderate to steep easterly and/or westerly plunging folds as a result of deformation 3 transpressional shearing;

5. - late, brittle NW-SE and NE-SW to east-west trending faults which disrupt the north-south trending granite-greenstone terrane.

4.5 Geology and Structure of the Lega Dembi area

Lega Dembi's host sequence is deeply worn and poorly exposed, with a sharp westward drop. The lithostratigraphic sequence is separated into the gneissic domain in the east, which includes quartzo-feldspathic, biotite gneisses and amphibolites and the Megado belt volcanosedimentary domain in the west. Ultramafic schists and diverse meta-sedimentary lithologies make up the sequence in the local Lega Dembi area. The 40° to 70°W dipping contact between the gneissic domain and supracrustal sequence is typically used to form ultramafic talc schists with mylonitic fabrics. Although bands of talc schists are interleaved locally with the gneisses, this contact is tectonic and generally acute. The talc schist's width

varies over the length of the four interconnecting pits, which are the Upper Lega Dembi, Northern, Central, and Southern, from north to south. The talc schist is 5 m thick in the Northern pit, pinching out locally, but gradually thickens to the south, reaching a maximum thickness of 180 m to the south of the Southern pit.

The Lega Dembi deposit is located within the Megado belt, which has been distorted by a huge north-south trending fold. Although there are inadequate clues to determine whether the fold closure is a syn- or antiform, it is located south of the deposit, where the ultramafic talc schists are thickest.

The Lega Dembi deposit's western edge is formed by this huge meta-gabbro and accompanying smaller amphibolites. These mafic bodies form a north-south ridge paralleling the Megado belt's structural grain, and are intruded locally by stringer- and pod-like tonalite bodies with penetrative planar and linear fabrics, which are folded together with the amphibolites on a metre scale.

Shear sense indicators in meta-sediments show both normal and reverse dip-slip, although sinistral strike-slip is always present. Similarly, structural components along the contact between the gneissic basement and the Neoproterozoic volcano-sedimentary sequence reveal a substantial sinistral strike-slip component. This deformation is gradual, with the sinistral strike-slip shearing displacing the veins and foliation-parallel quartz veins, while the latter is transected by the north-south fabric. In the northern and southern parts of the Lega Dembi deposit, a sequence of NW-SE and east-west brittle and brittle-ductile deformation faults disrupts the mineralisation continuity.

4.6 Local geology

4.6.1 Introduction

The study area deposits are found in the South Ethiopia Terrane in the Arabian-Nubian Shield's southwestern part, specifically in the Adola granite-greenstone belt, which covers an area of around 5000 Km².

The East Sakaro project area is located 10Km south-southwest of Shakiso, 3Km southwest of Lega Dembi mine. The target consists of late-deformation, shear zone-hosted, auriferous

quartz vein system emplaced along the contact between quartz-mica schist and amphibolite and also within the schist.

The study area's lithostratigraphic sequence is separated into the gneissic domain in the east, which includes quartzo-feldspathic, biotite gneisses and amphibolites and the Megado belt volcanosedimentary domain in the west. Ultramafic schists and diverse meta-sedimentary lithologies make up the sequence in the local Lega Dembi area (Source MIDROC, East Sakaro report, 2008).

4.6.2 Geology

The lithological units underlying the East Sakaro project area comprise graphitic quartz-mica schist, amphibolite, metagranodiorite, graphitic quartzite and pegmatite. The first four rock types predominate in the area.

- ❖ **Graphitic quartz-mica schist:** covers the central portion of the prospect area and contain intercalations of amphibolite 5m to 15m width. The schists are intensely silicified in places resulting in the complete obliteration of the metamorphic fabric. The graphite content of the schists varies considerably, often over short interval, feature zones ranging from a few centimeters to one meter in width occurs within the schists.
- ❖ **The quartz-mica schists:** strike 020^0 and dip 60^0 to 75^0 west. The continuity of the schists north of ESGTH01 is well established by drill holes that the schist plunges towards north. The schists are both underlain and overlain by amphibolite.
- ❖ **The amphibolite:** are intensely silicified in places, particularly near the contact with the quartz-mica schists. They are derived from intermediate to basic, highly deformed, intrusive igneous rocks emplaced during a single magmatic event and subsequently metamorphosed to amphibolite facies. The most deformed portions of the intrusive is now represented by amphibolite schist.
- ❖ **Metagranodiorites** represent more felsic marginal components of the intrusive while its core is made up of relatively undeformed gabbroic rocks. Strong evidence of these relationships is found within borehole core.

4.6.3 Joints

A joint is a brittle-fracture surface in rocks along which little or no displacement has occurred. Present in nearly all subsurface rocks, joints extend in various directions, generally more toward the vertical than to the horizontal. In the study area, the joints were observed in graphitic quart-mica schist rocks and some near the ore body with 2 joint sets. Both joints are characterized by slightly rough surfaces with a joint spacing ranging from 0.03 to 0.06m.

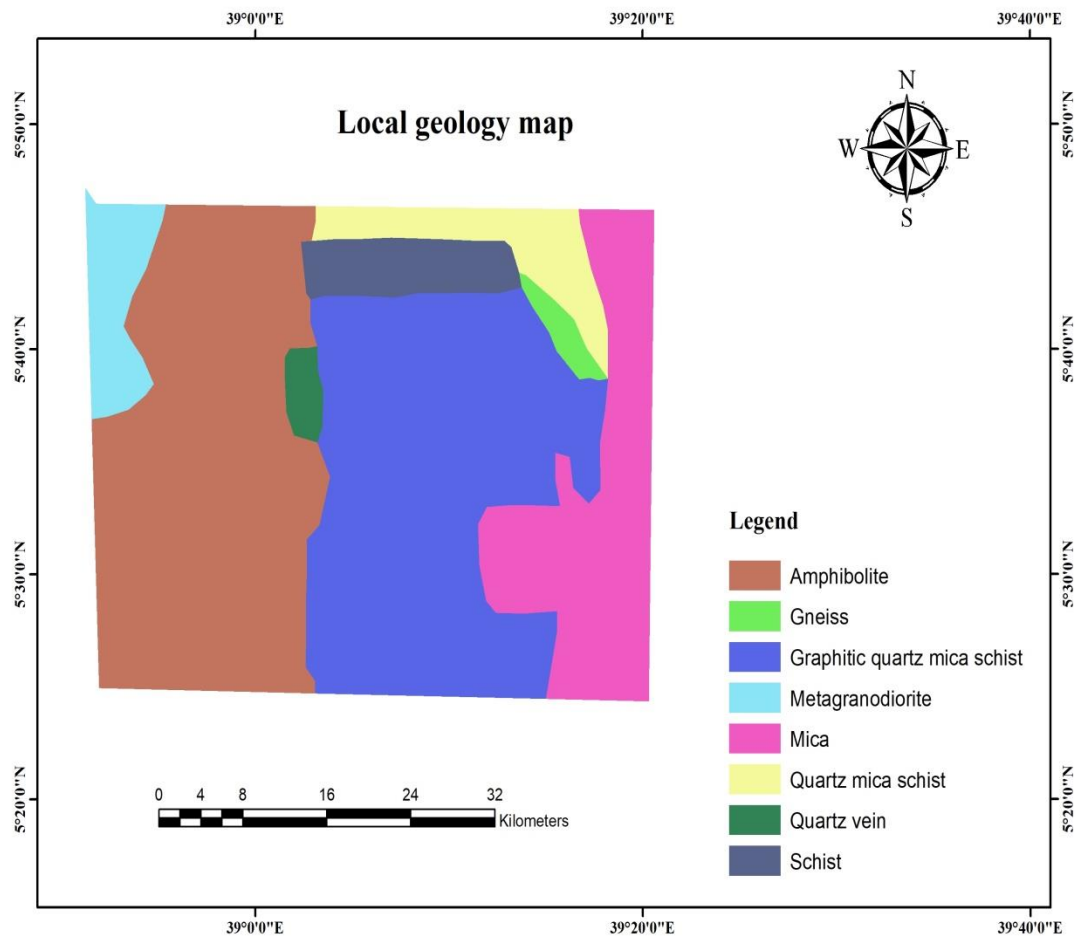


Fig4.2 Geological map of the study area (by Derartu jibro; 2022)

4.6.4 Subsurface Geology

The subsurface data was collected from MIDROC and it includes the borehole data. Five boreholes were drilled for the maximum depth of 398.34m. The profile of the borehole shows five lithologic layers. These are Top soil graphitic-quartz mica schist, ore body, quartz-mica schist and amphibolite were found respectively from top to bottom. The groundwater information in the borehole presented as dry or groundwater has not been observed in five of the borehole data.

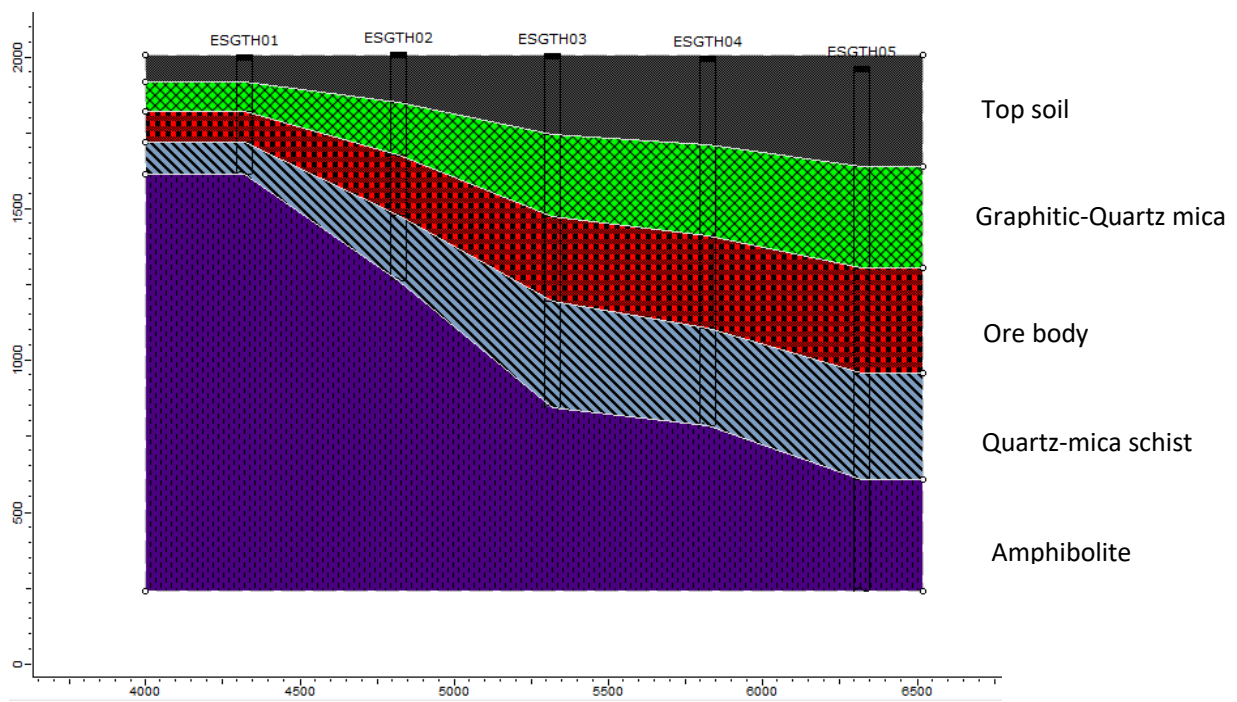


Fig4.3 Subsurface Layers of Sakaro Uderground tunnel

CHAPTER FIVE

RESULTS AND DISCSION

5.1 General

Gold is Ethiopia's main mineral export and has been mined since ancient times, primarily as alluvial or free gold. The Lega Dembi deposit is the largest gold producer in Ethiopia. It is situated in late-Precambrian metamorphosed sediments of the N-S trend. MIDROC Gold, one of the subsidiary companies administered under MIDROC Technology Group, acquired the Lega Dembi Gold mine from Ethiopian government through privatization for 172 million dollars in 1997 (Jing, 2003). Since 1998 the company has been mining gold and silver from the mine, located in the Oromia Regional State, Guji zone. This Gold mine has an installed annual production capacity of four tons of gold. It was an open pit mine but MIDROC Gold has built the first underground mine in the country and conducted various exploration activities to extend the economic life of the mine.

The underground East Sakaro Tunnel, which provides access to the mining area, will be used to transport essential ore from underground mining projects. Its stability plays an important role in undertaking successful mining operations and project economics. Interactions between rocks and support systems depend on factors such as rock properties, support geometry and properties, and geological conditions. Rock responses are sensitive to unloading and loading processes (Cai et al., 2016).



Fig5.1 Portal gate of a tunnel for Sakaro underground gold mine (Derartu jibro, March 2022)

In this study, Finite Element analysis method was performed to determine the factor of safety of the critical section the tunnel. For Numerical softwares, laboratory tests for rocks were inputted. Dry and saturated density, point load test, Schmidt hammer, and slake durability tests were used to determine the rock samples properties. The results from the finite element method and other softwares; kinematic analysis, Rocdata, Unwedge, and Rocsupport were compared with each other. Also from the joints orientation data, the area is mostly affected with planar and more dominantly wedge failures which will be discussed in the following.

5.2 Engineering properties of rocks

Table 5.1 Name of the Rocks used for Laboratory test

No	Sample ID	Name of the Rock/Lithology
1	RS1	Metagranodiorite
2	RS2	Amphibolite
3	RS3	Quartz Mica Schist
4	RS4	Graphitic Quartz Mica Schist

The tests were performed for rock samples with a critical section showing structurally or non-structurally controlled behavior. To determine the strength of the rock a point load dry and saturated density was performed moreover and slake durability tests were also performed to determine the properties of a rock sample.

5.2.1 Rock strength result

Generally in first Schmidt hammer test for all rocks the rebounding point were the same for all rocks in each case but for RS4 strength testing, the rock was not strong enough to withhold the load applied on it and it is said to be weaker than the rest of the rocks provided for the test.

In point load, dry and saturated density test cases it can be concluded that graphitic quartz-mica schist is slightly weathered rock at the top next to the top soil while the amphibolite showed high strength value compared to quartz-mica schist and metagranodiorite.

As for the slake durability test amphibolite was the strongest of all rocks to be affected by weathering. When the rocks are placed from high to low based on resistance of weathering: Amphibolite> Quartz-mica schist> Graphitic Quartz-mica schist > metagranodiorite. All the laboratory data are showed in the appendices.

5.3 Finite element method

Table 5.2 Inputs for FEM

Sample ID	Rock type	USC	GSI	Mi	Poinson ratio (μ)	Young modulus (Em)
RS4	Graphitic Quartz-mica schist	105	73	20 b/c(quartz +_20	0.4	4125Mpa

After putting the above values into Rocdata software, we got some outputs that could help as an input for the FEM analysis (Table 5.2). Since the tunnel pass through a single profile, the analysis was done for one rock slopes considering plastic material type and the initial field stress only. The factor of safety of slope in the analysis obtained through these processes, and this is decided after analyzing the stress distribution or the displacement shown from tunnel to out ward using graphs. Also Analysis was done assuming the stability of the tunnel with culvert, without culvert, with bolt, without bolt and if economically feasible with both culvert and bolt.

5.3.1 Tunnel information

East Sakaro Underground tunnel is a horseshoe type tunnel with radius of 2.5m, overall height to corners of 5.5m, and width of 5m with dry water condition. The length from the portal to the ore body is 1.5Km. Tunnel direction is S-W and the joint orientation is N-S. The profile that the tunnel pass thought out of the four types of lithology is the Graphitic quartz-mica schist. Note: the following softwares will focus on the specific profile that the tunnel passes through.

Table5.3 Tunnel Geometry Properties of East Sakaro underground tunnel

Tunnel geometry properties	Tunnel diameter (m)	5m
	Length of tunnel (km)	1.5km
	Excavation face height	5.5m
	Tunnel length /Tunnel diameter (Z/2R)	300m
	Overburden Weight	1870ton

✚ Tunnel Shape = Horseshoe

Horseshoe type tunnel shape is a combined shape of arches and circular tunnel.

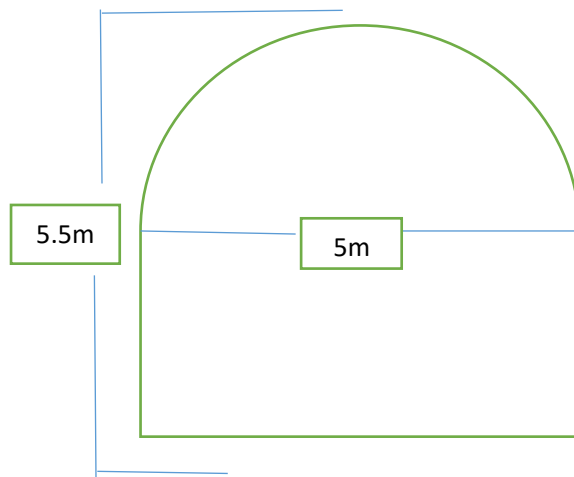


Fig5.2 Tunnel Geometry of East Sakaro

5.3.2 Finite Element Method of Analysis for East Sakaro underground tunnel

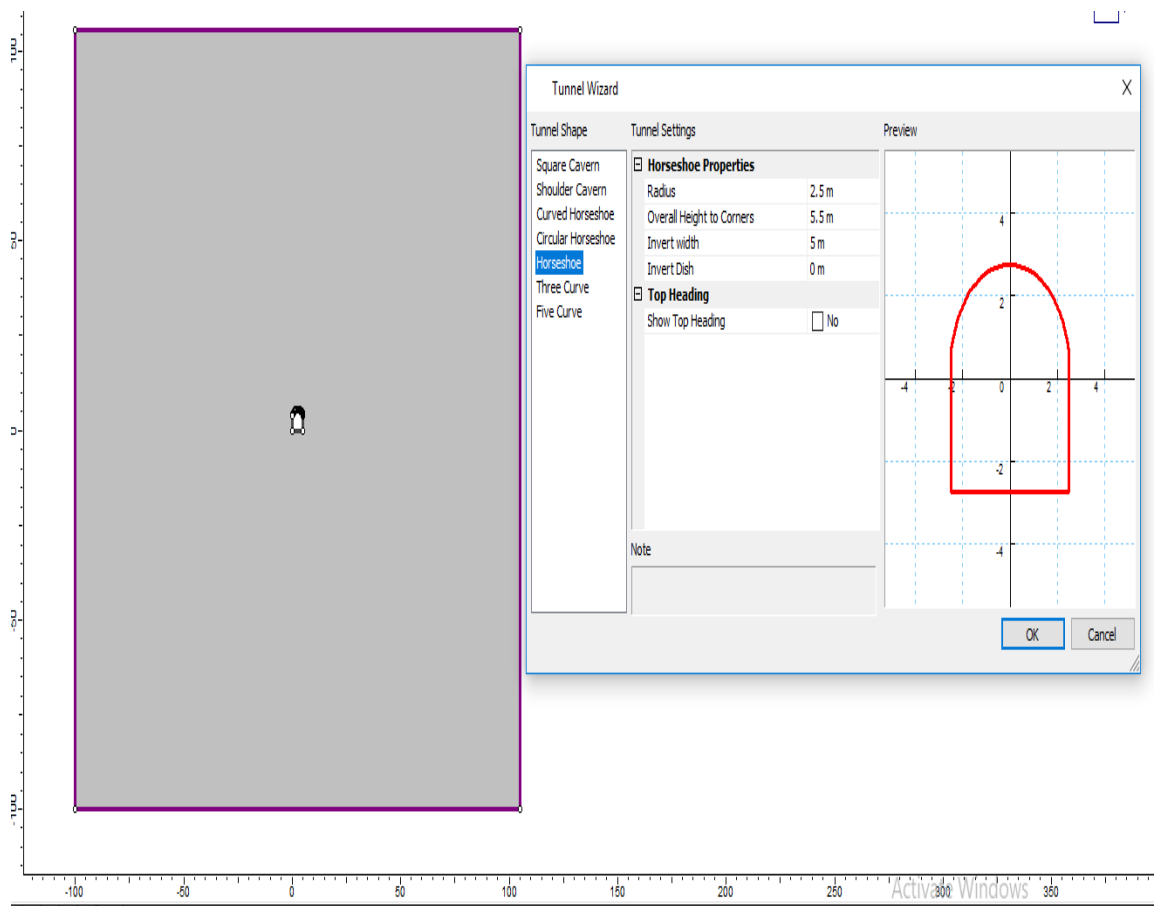


Fig5.3 Tunnel shape

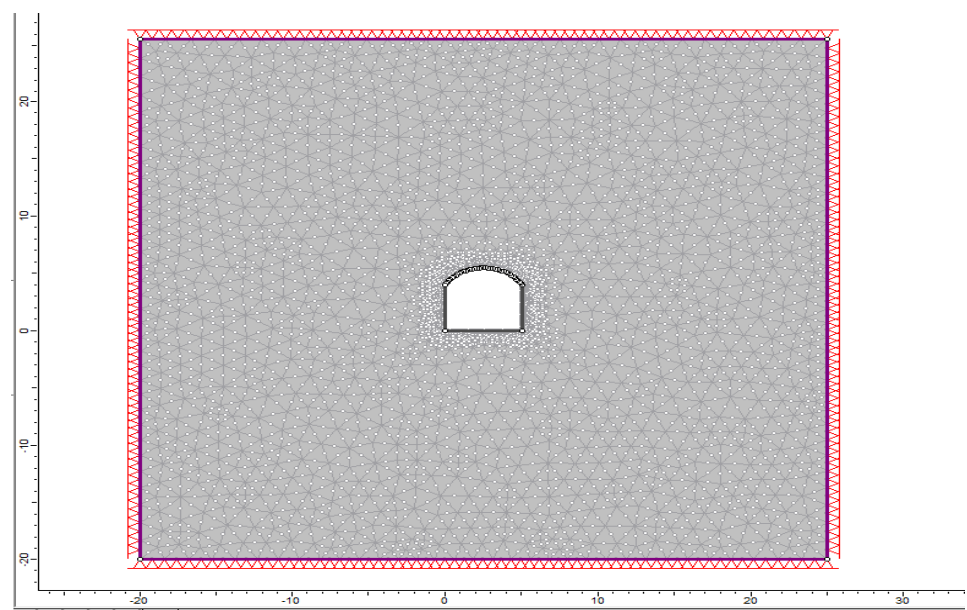
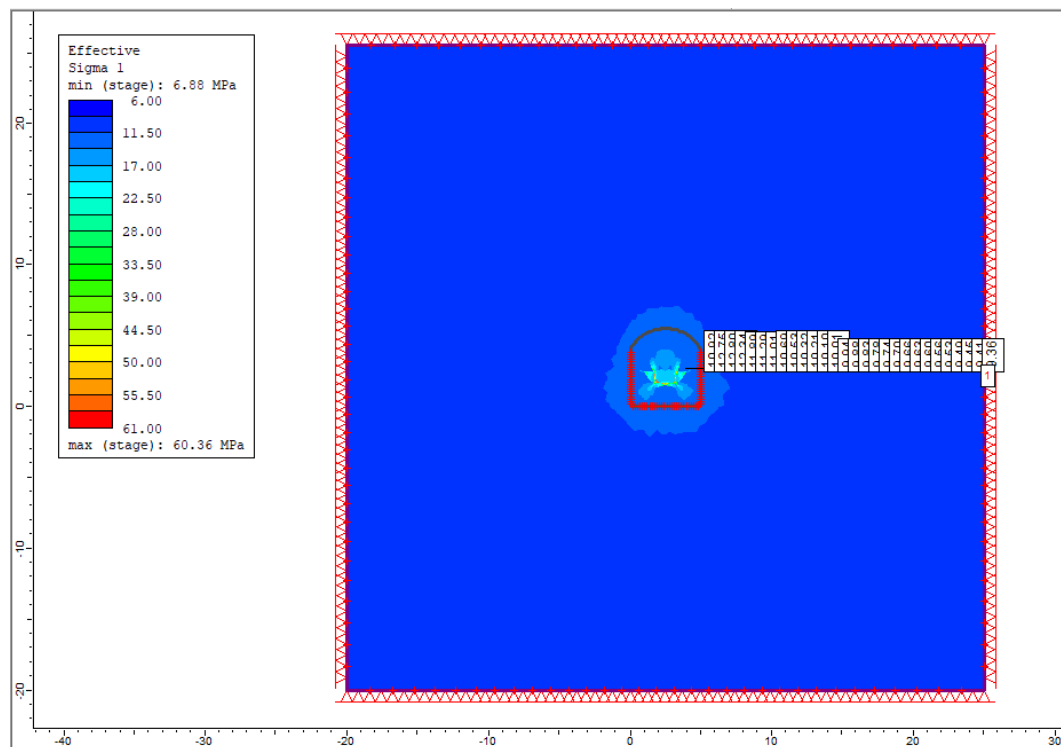
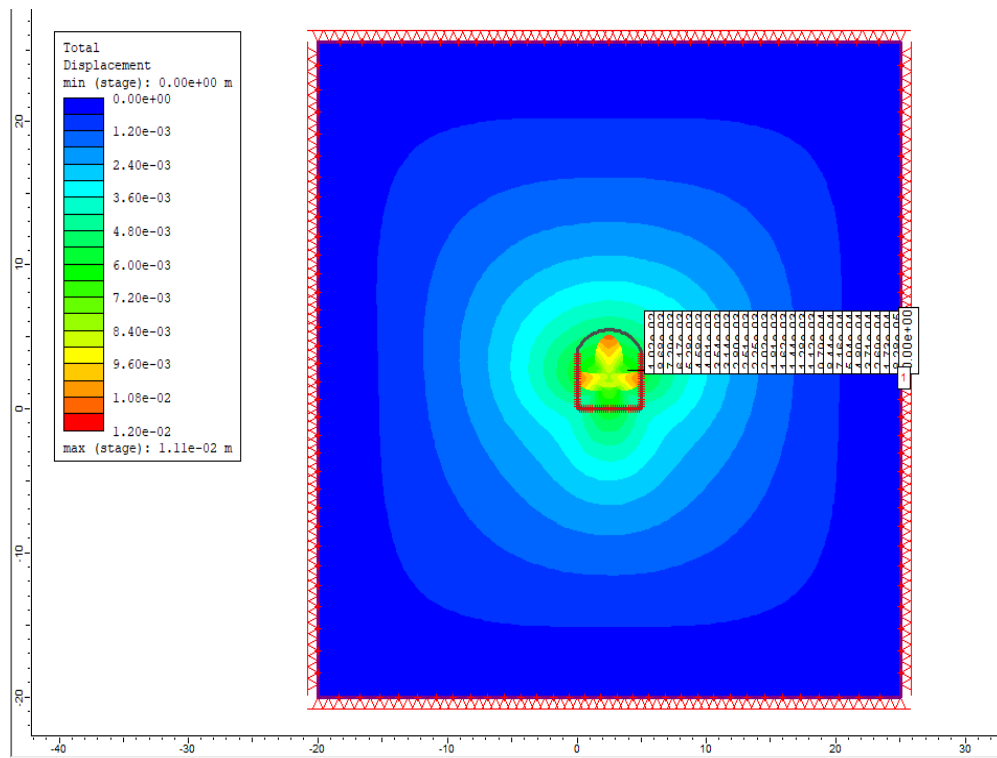


Fig5.4 Meshed Horseshoe tunnel to compute the stress distribution



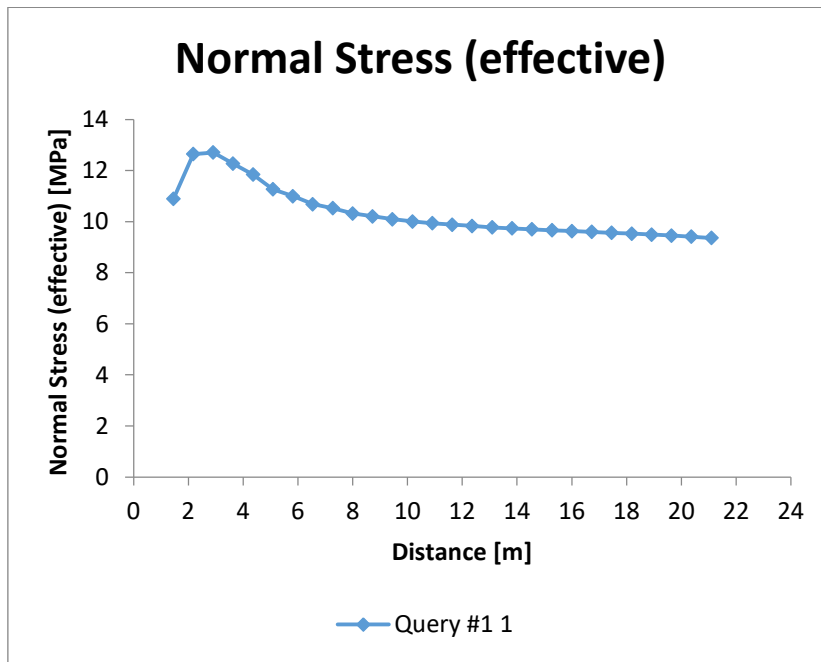


Fig5.7 Normal stress Vs total displacement 1

- ✚ Effective stress decreases as distance increases.
- ✚ The maximum effective stress is 12MPa, but the normal stress is 11Mpa when the distance is 1m.
- ✚ Furthermore, the stress increases up to 3m and then gradually decreases. This explains the tunnel corners are not stable.
- ✚ This explains why the stress decreases from the tunnel's center to the tunnel's edge.

Report 1 Generated from FEM

The following report is generated from the FEM software explaining in detail about the above figures.

- ✓ **General setting:** Single stage model, Analysis Type= Plane Strain, Solver Type= Gaussian Elimination, Units=Metric, stress as MPa, Permeability Units=meters/second, Time Units= seconds
- ✓ **Analysis option:-** Number of Load Steps=Automatic, Convergence Type=Comprehensive, Tensile Failure=Reduces Shear Strength, Joint tension= reduces joint stiffness by a factor of 0.01
- ✓ **Groundwater Analysis**

Method=Static

Mesh

Mesh type: Graded, Element type: 6 Noded triangles

- ✓ **Material Properties=GQMS (Graphitic-Quartz Mica Schist)**, Initial Element Loading= Field Stress Only, Unit Weight= 0.027 MN/m^3 , Elastic Type= Isotropic, Poisson's Ratio= 0.4, Young's Modulus= 4125 MPa, Use Residual Young's Modulus =No, Failure Criterion= Mohr-Coulomb, Material Type =Plastic, Peak Tensile Strength= 0 MPa, Peak Friction Angle =43.313 degrees, Peak Cohesion =9.075 MPa, Static Water Mode= Dry

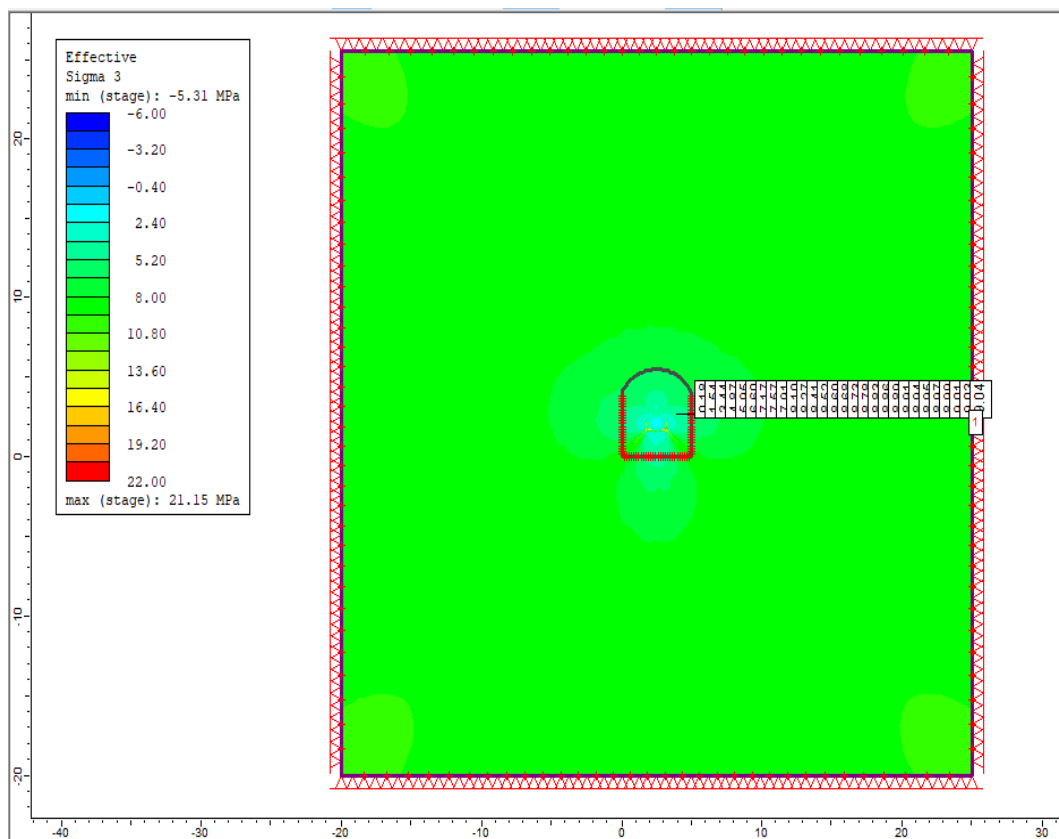


Fig5.8 Effective stress along Sigma 3 (Max and min stress are 21.15Mpa and -5.31Mpa respectively)

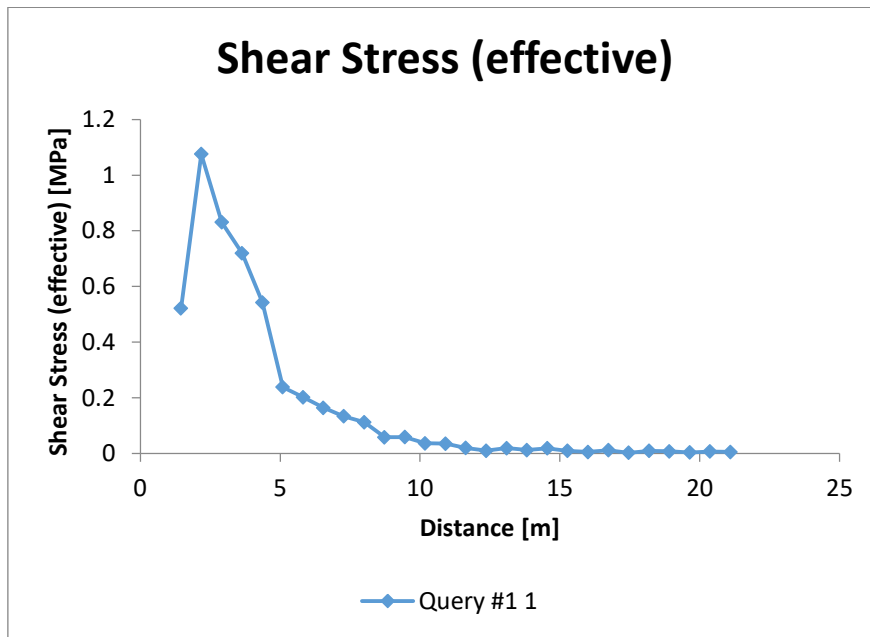


Fig5.9 Graph showing shear stress vs distances

Shear stress decreases or becomes zero and remains zero as distance increases. The maximum shear stress is between 1 and 1.2 Mpa, which explains why the stress decreases from tunnel to tunnel.

5.3.3 FEM With culvert analysis

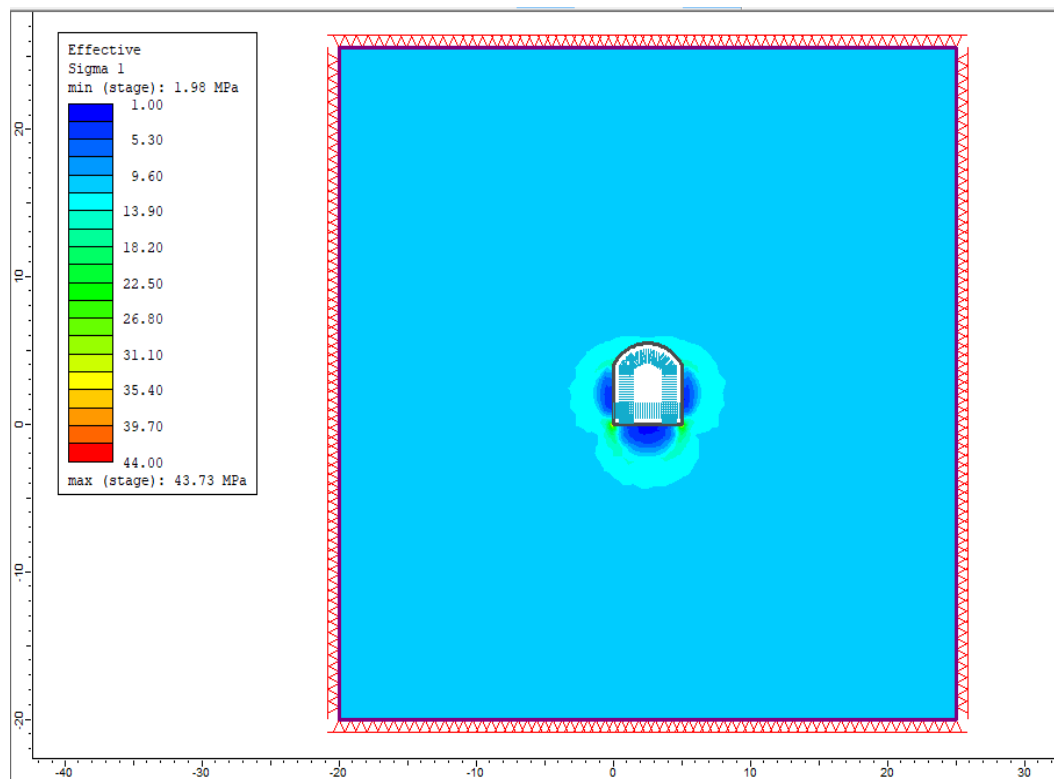


Fig5.10 Effective stress along sigma1 (Max=43.73Mpa and Min=1.98Mpa)

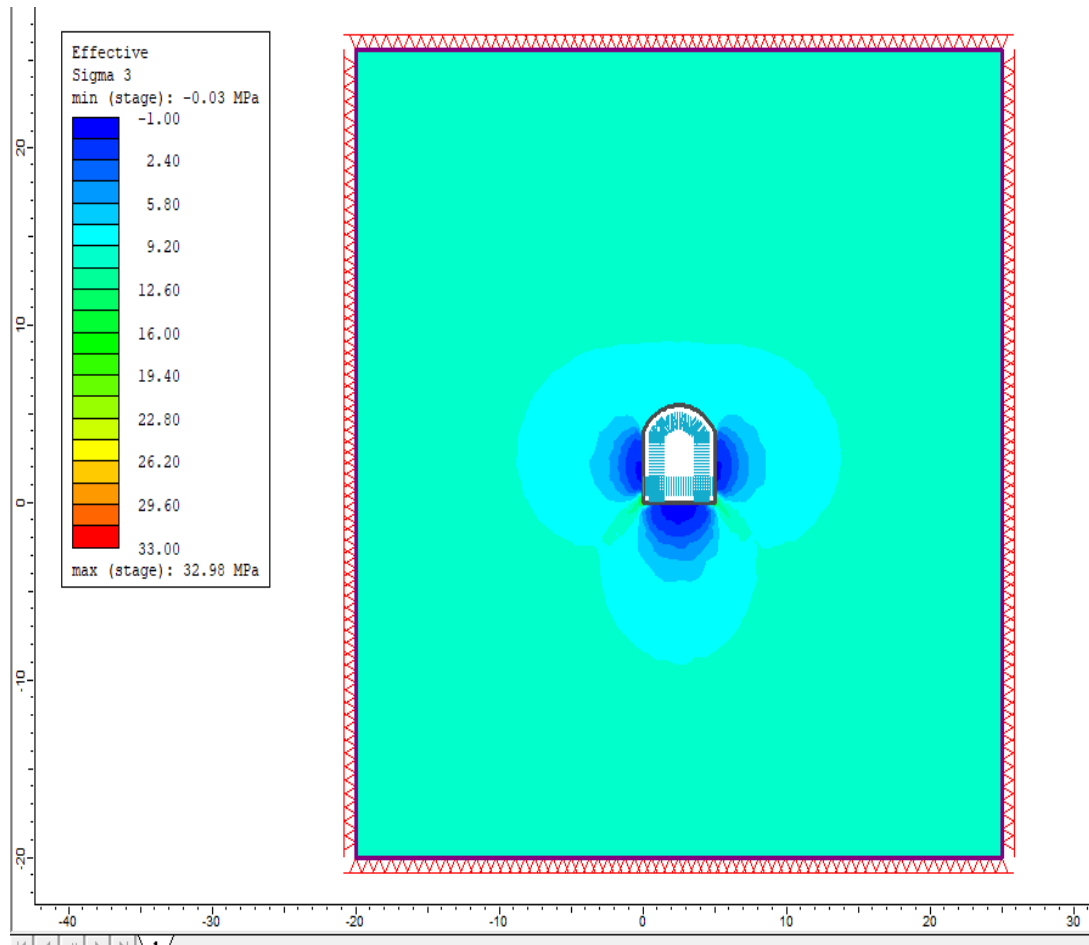


Fig5.11 Effective stress along sigma 3 (Max=32.98Mpa and Min=-0.03Mpa)

In this case after using culvert for a support there is an observation that the maximum effective stress has decreased compared to the previous figures that didn't have any support. The point here is a single support system can give impact on the stress distribution.

5.3.4 FEM With bolt Analysis

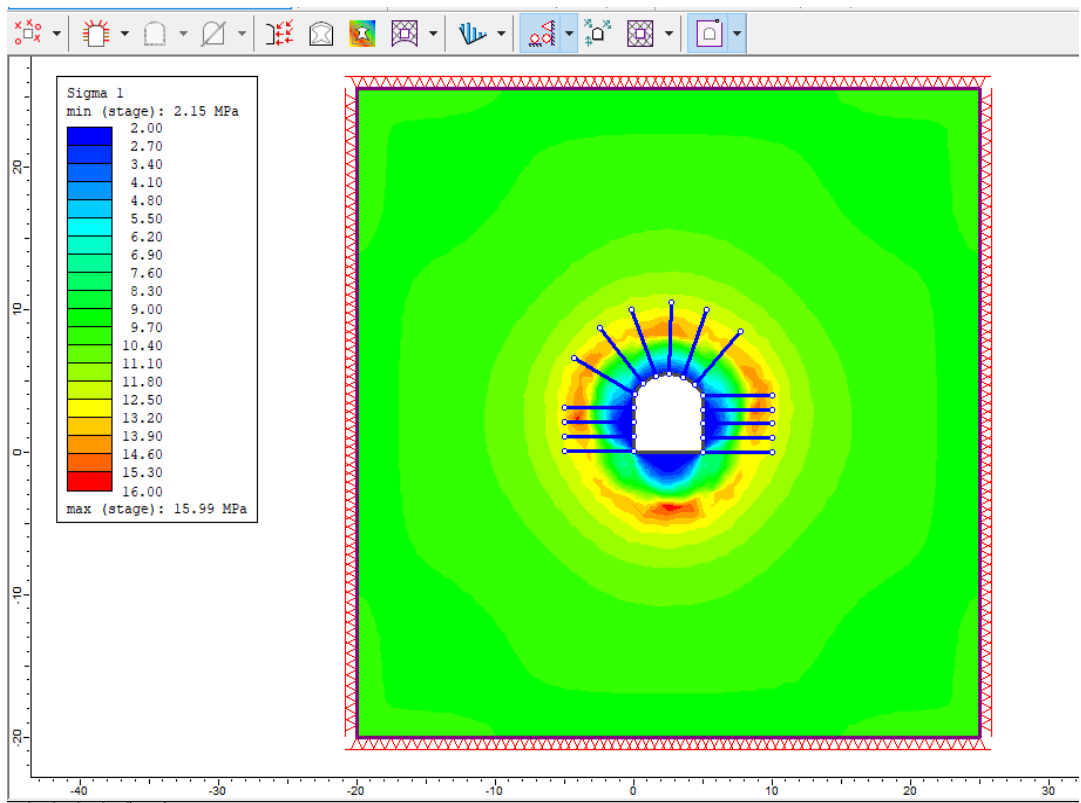


Fig5.12 effective stress along sigma 1(Max=15.99Mpa and Min=2.15Mpa)

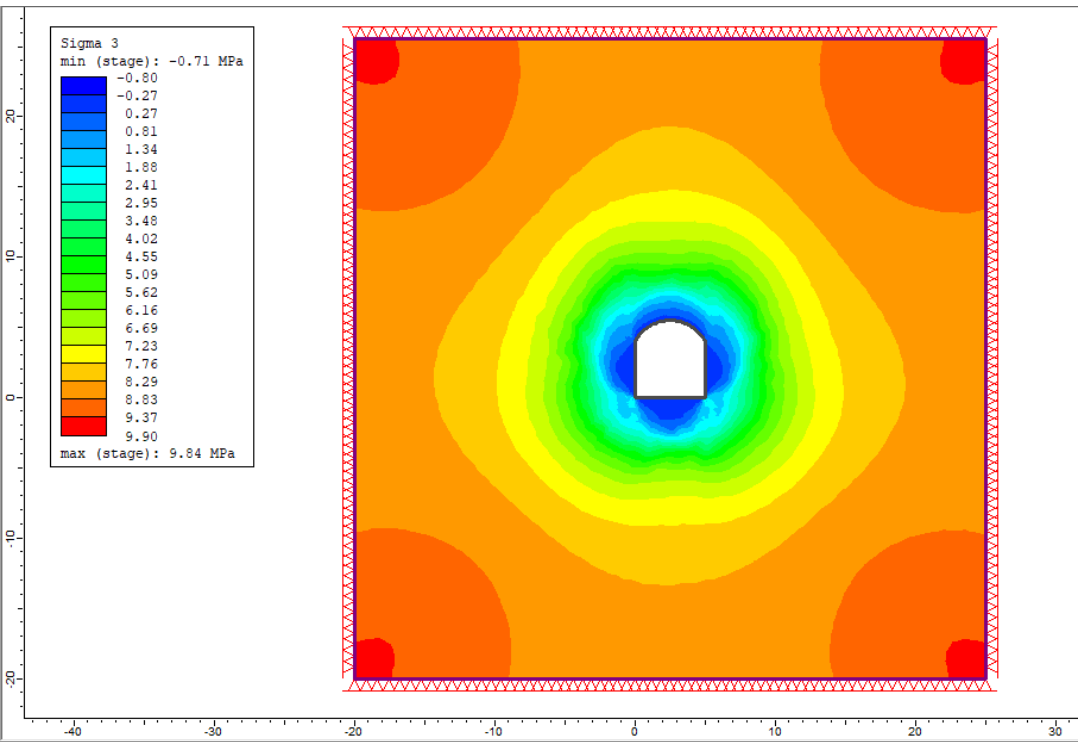


Fig5.13 Solid effective stress along sigma 3 (Max=9.84 and Min=-0.71Mpa)

Fig 5.13 shows that the stress is much lower around the corners of the tunnel because the bolt is being instilled and it minimized the stress which made the tunnel more stable. However, away from the tunnel to out ward, the stress is being higher than near the tunnel and this is due bolt is only inserted at some limited distance.

In general the above analysis showed that the maximum effective stress decreases more when using bolt and in another analysis when culvert is used. Also the above graphs representation shows as the distance goes far from the tunnel the stress distribution becomes less and lesser, this means the stress distribution is higher. In addition the analysis indicates that the stress becomes lesser near the tunnel corners when bolt is assumed as support and culvert system. Bearing this in mind, any support system would be good to use but while choosing the maximum safe point, the supporting material is related with the cost that should be considered before the final decision is made. In this case, using bolt is better considering. However during the construct of any types of support system in an underground tunnel, the cost or economical implication is very important.

5.4 Kinematic Analysis

Table5.4 Input for kinematic analysis 1

Joint number	Dip amount	Dip direction	Estimated failure type	Slope
J1	60 ⁰	210 ⁰	Planar and wedge failure	Dip amount=73
J2	60 ⁰	336 ⁰		
J3	45 ⁰	88 ⁰		Dip direction=100

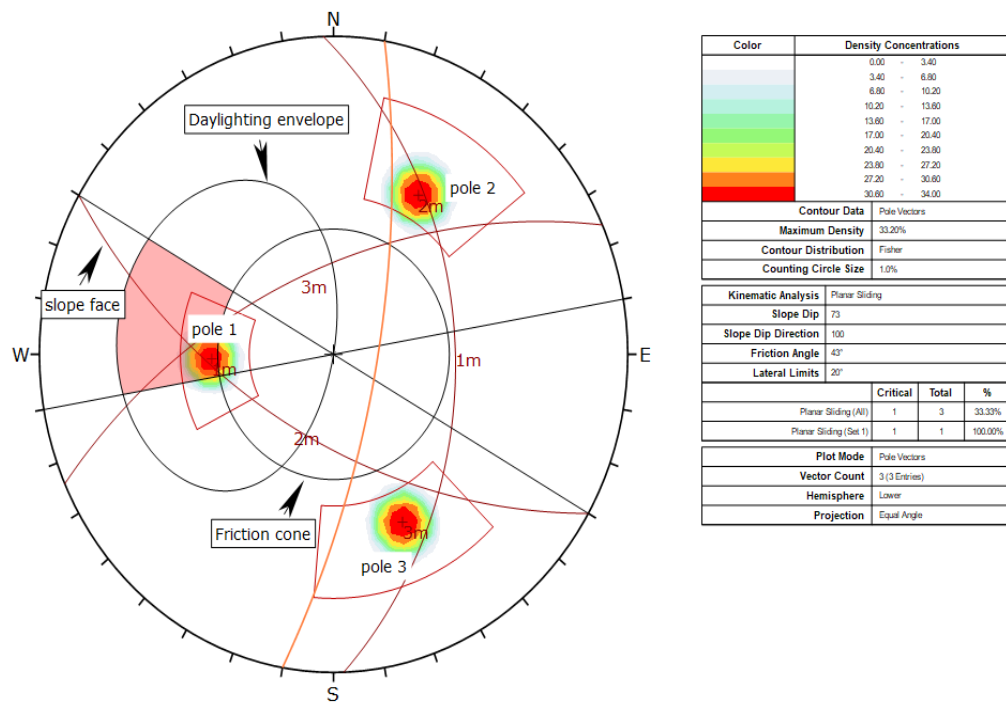


Fig5.14 Critical points showing planar failure for stereonet 1

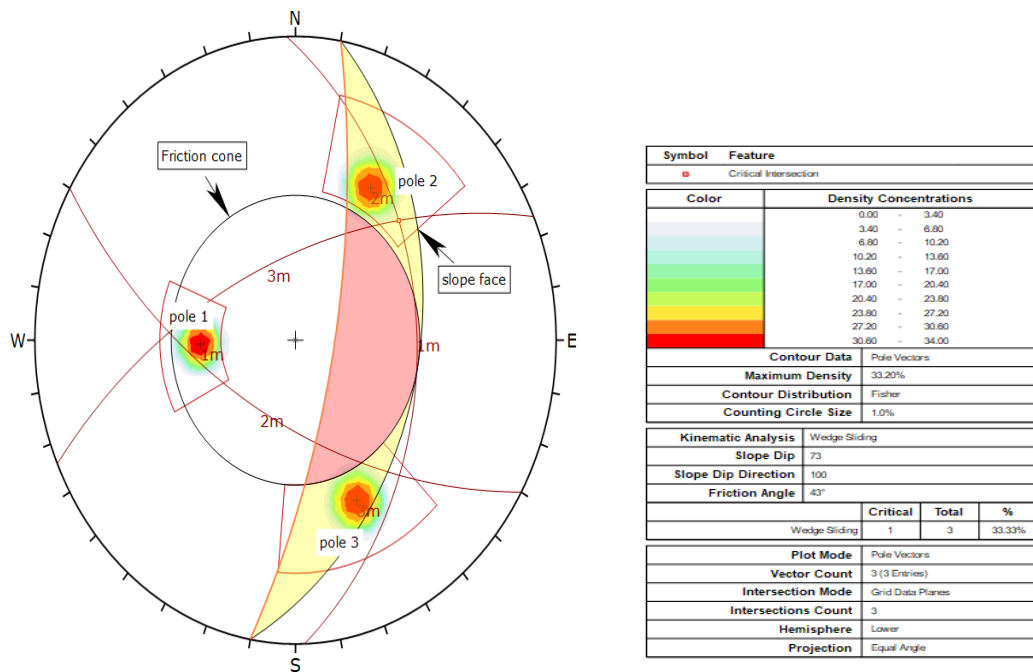


Fig5.15 Critical points showing wedge failure for stereonet 1

Table5.5 Input for kinematic analysis 2

Joint number	Dip amount	Dip direction	Estimated failure type	Slope
J1	55 ⁰	110 ⁰	Planar and wedge failure	Dip amount=73
J2	60.5 ⁰	210 ⁰		
J3	47 ⁰	300 ⁰		Dip direction=110

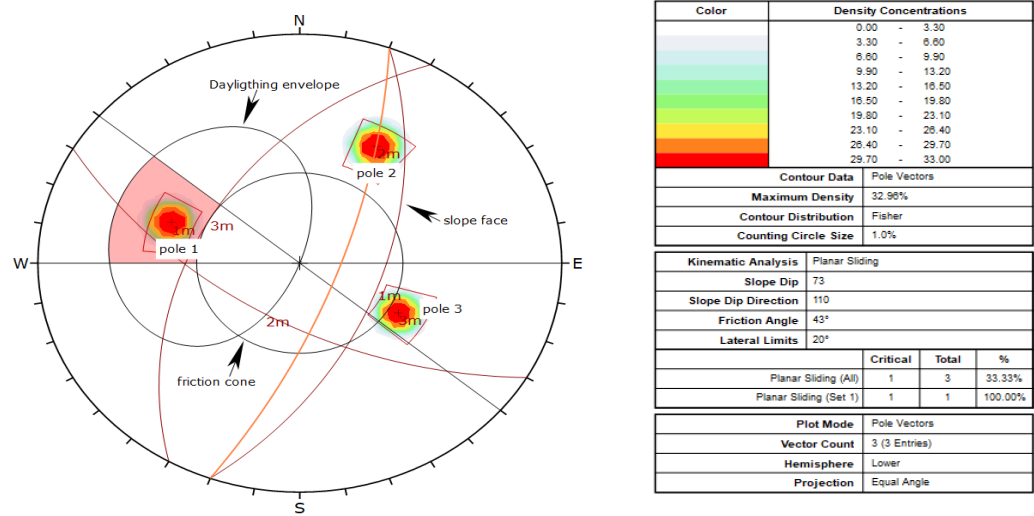


Fig5.16 Critical points showing planar failure for stereonet 2

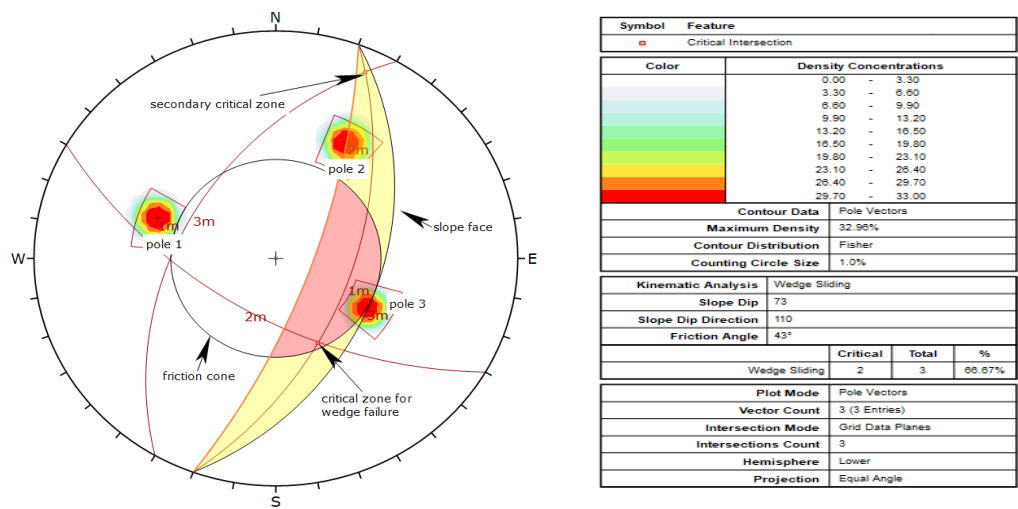


Fig5.17 Critical points showing wedge failure for stereonet 2

In this analysis on stereonet 1, 3 joint number were used J1-J3 in order to test what type of failure occurs around the point. In stereonet analysis 1 there are both planar and wedge failure: critical points for planar failure occurs only on joint set 1, for wedge failure critical point occurred at the intersection of J1 and J3 (fig 5.20)

As for the case of sterionet 2: critical points for planar failure was on joint set 1 and two wedge failures were occurred; the first was between J1 and J2 and the second one is between J1 and J3. The intersectional joints J1 and J2 lays in primary critical zone and J1 and J3 in secondary critical zone for wedge failure (fig 5.22). The area is more affected by wedge type of failure than the other types of failures.

5.5 Unwedge Analysis

In this Analysis, it should be noted that the FOS (factor of safety) of slope to be stable for mining=1.2, building=1.6, and road=1.3, respectively. Since the area is a mining area, FOS that is obtained from the software will be compared with 1.2. Therefore if the FOS excides 1.2 it said to be stable and if it's below 1.2 the conclusion would be unstable slope.

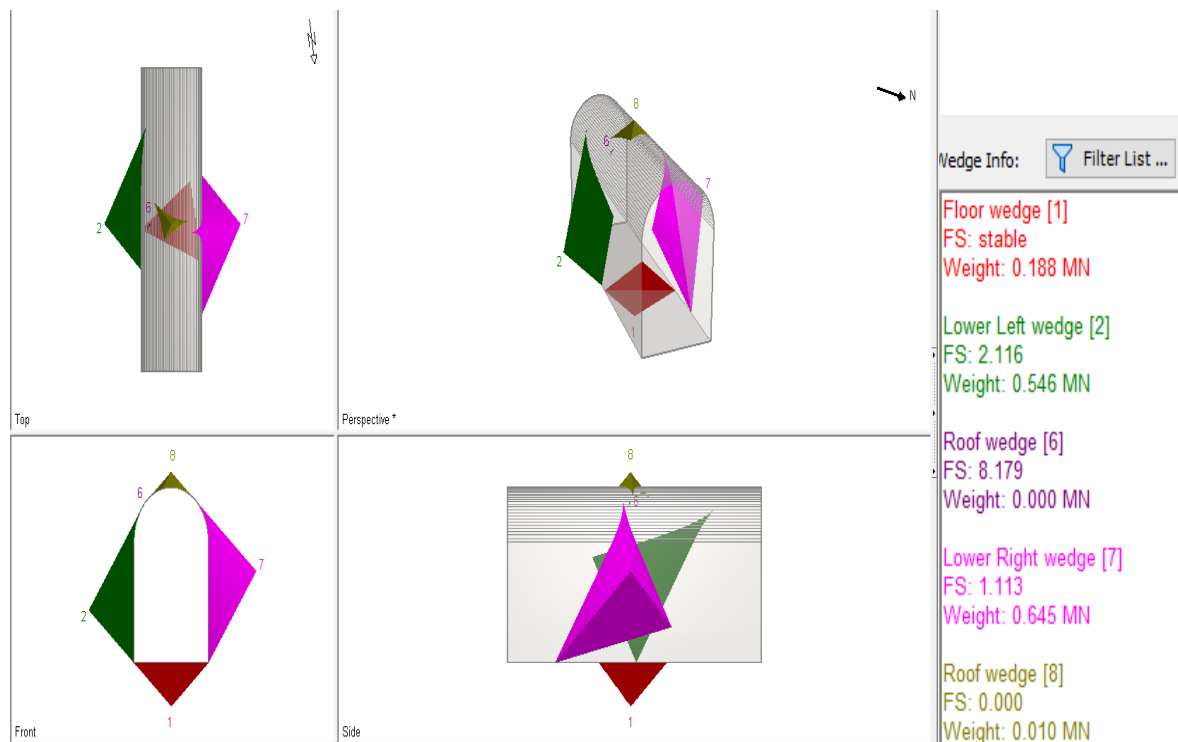


Fig5.18 3D view of wedge failure for analysis 1

According to this analysis, the tunnel is stable at the floor and lower left side and unstable at the lower right and roof of the tunnel, with factors of safety of 1.113 and 0.000, respectively. The factor of safety that is described at right hand side in small table above shows at what point the tunnel is stable and unstable. The instability is due to the possible of wedge failure occurring at these points.

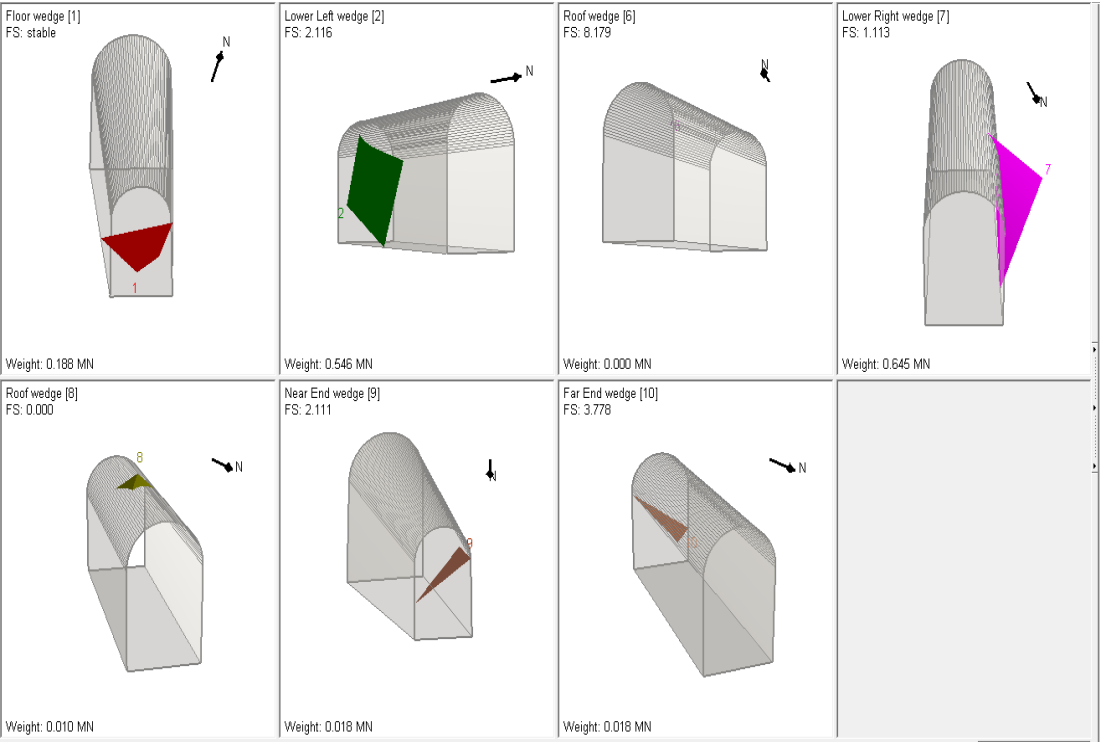


Fig5.19 3D Multi view of wedge failure for analysis 1

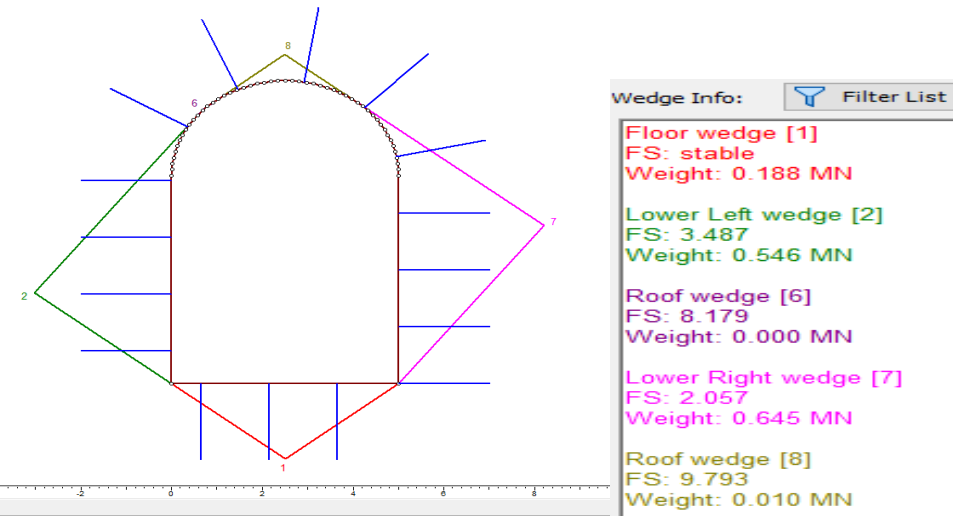


Fig5.20 bolt support system for analysis 1

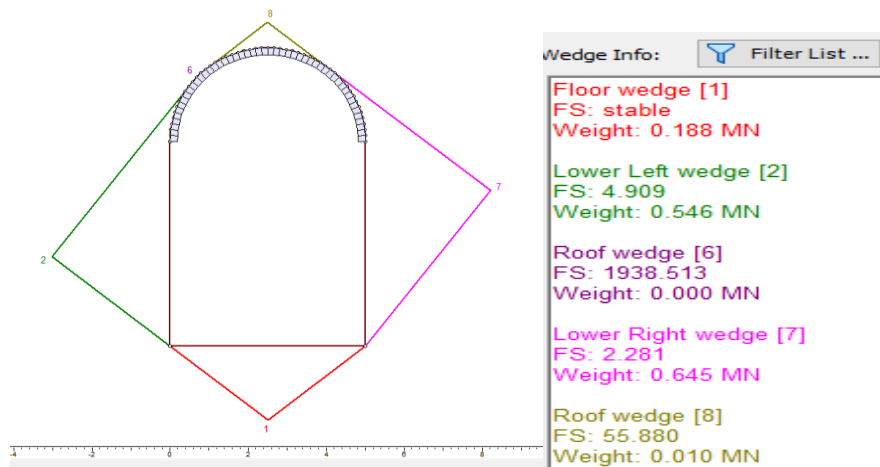


Fig5.21 Shotcrete support system for analysis 1

After the installation of support systems, bolt and shotcrete on the above figures, the factor of safety tends to increase indicating that the tunnel is safer and stable because it exceeds 1.2 FOS. Detail information is shown in each figure.

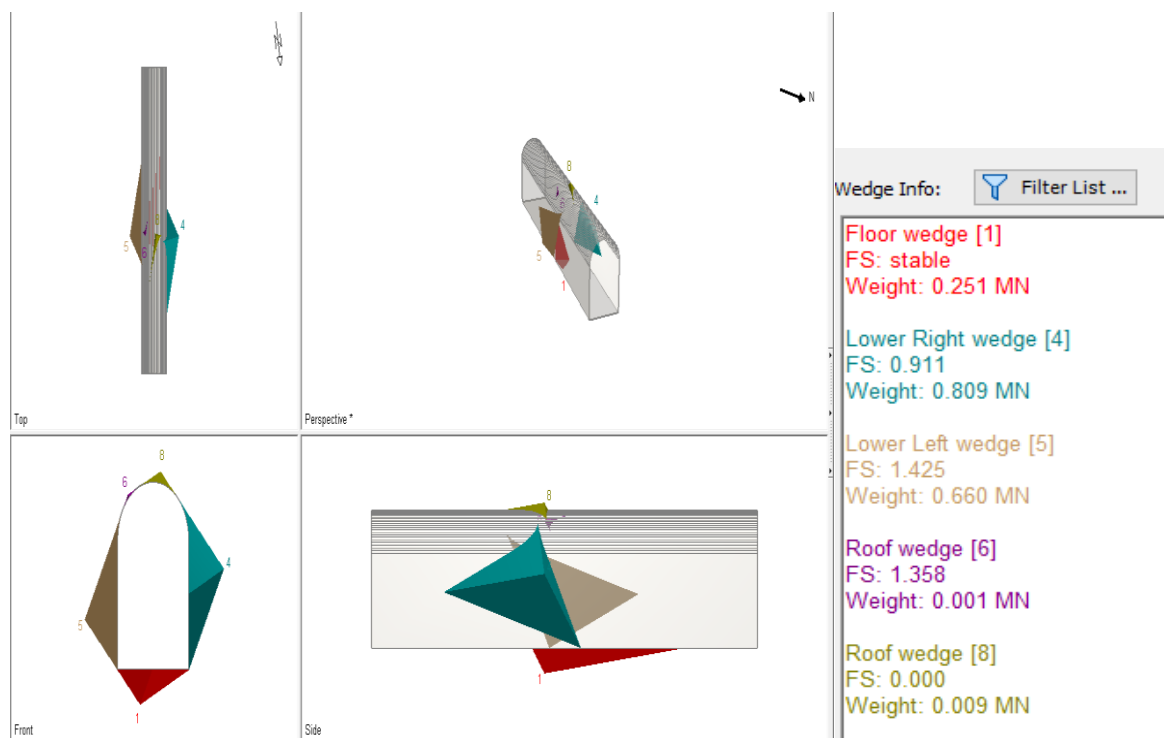


Fig5.22 3D view of wedge failure for analysis 2

Similarly with the analysis 1 the tunnel is unstable in this analysis at the lower right and roof of the tunnel, with factors of safety of 0.911 and 0.000, respectively. This is due to the

possibility of wedge failure at these points. It should be noted that the other three FS can be stated that the slope is nearly stable because the FS of slope to be stable for mining=1.2, building=1.6, and road=1.3, respectively.

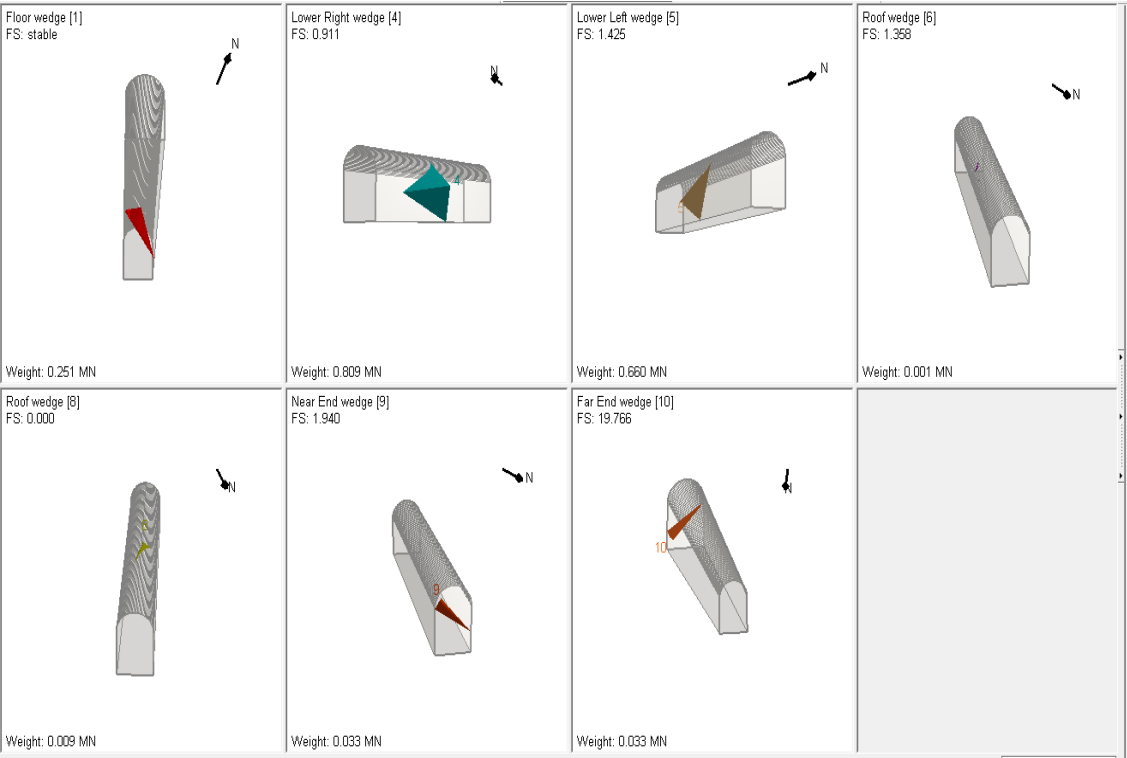


Fig5.23 3D multi view for analysis 2

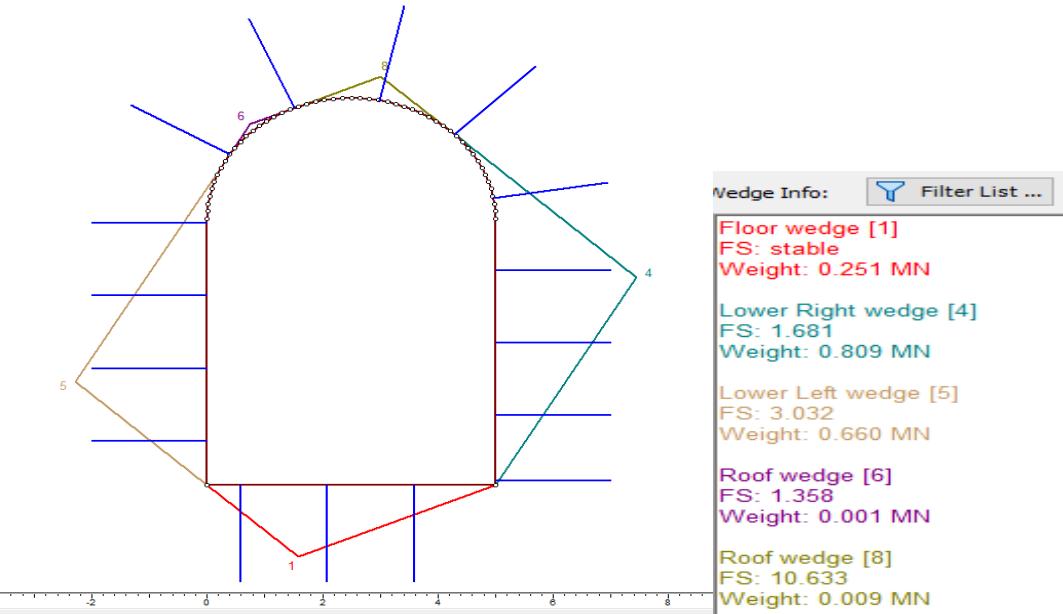


Fig5.24 Bolt support system for analysis 2

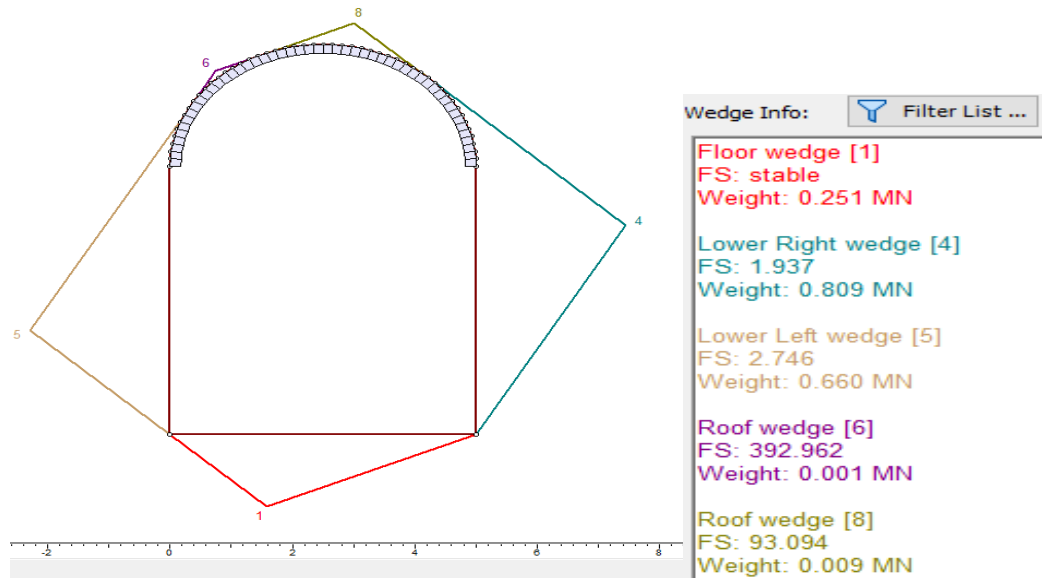


Fig5.25 Shotcrete support system for analysis 2

In general the factor of safety tends to increase after the installation of support systems, bolts, and shotcrete, indicating that the tunnel is safer and more stable because the factor of safety observed after using both support systems became higher value. In addition the floor is said to be stable by the software itself, meaning minimum or no wedge failure is expected at this point. At right hand side of the above figures detail information is shown.

5.6 Support System

Attempt is made to simulate and evaluate the stability of the slope after stabilized with bolt only and with bolt and shotcrete. The factor of safety of the critical slope section without any support was less than between 0.0 and 1.13, from Unwedge software analysis. For the slope to stable the factor of safety must equal to or greater than 1.2 because our area to consider is a mine area. The simulation result is presented in table 5.6 and 5.7.

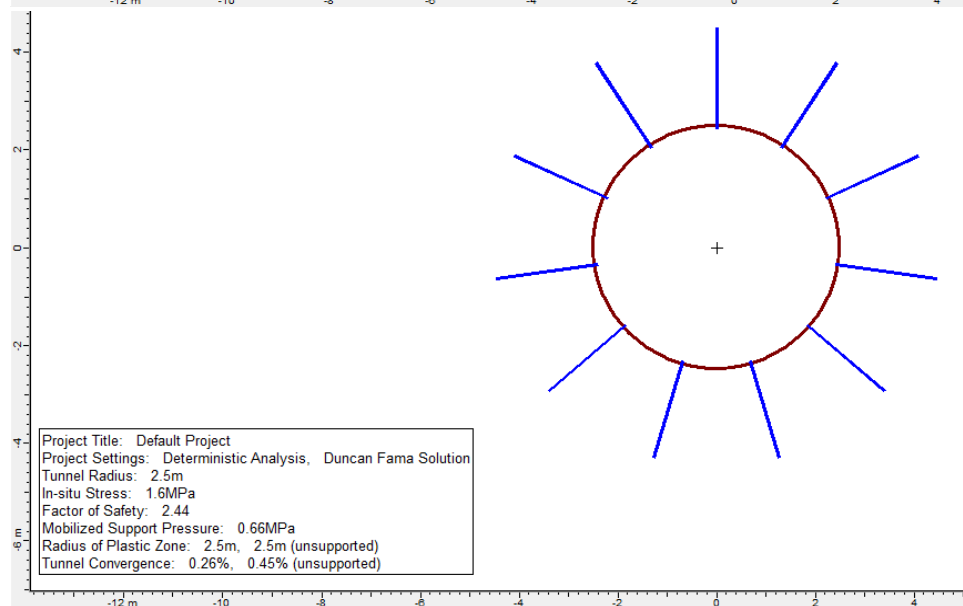
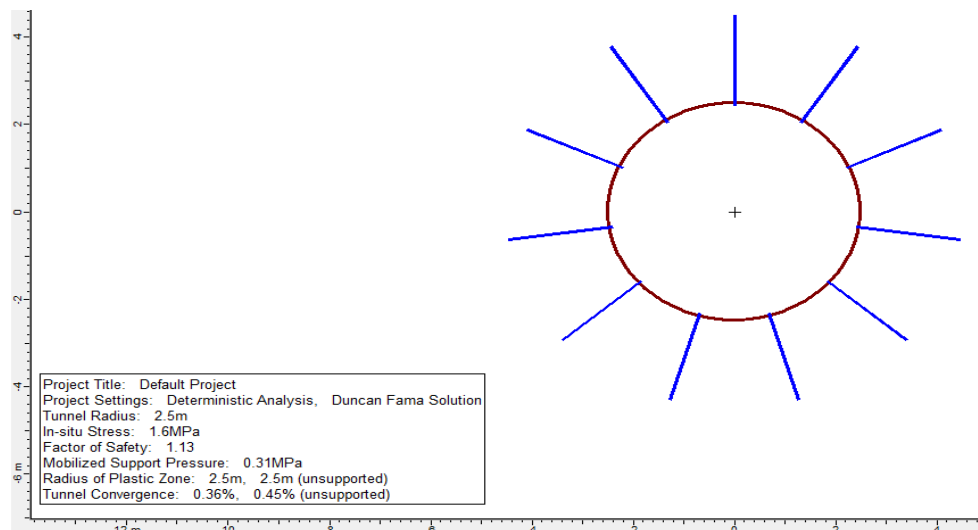
5.6.1 Effect of bolt diameter, length and spacing on the factor of safety of the tunnel

The factor of safety of the tunnel is assessed by varying the bolt diameter (34, 25, 19 and 17mm) and keeping the bolt spacing and length constant (1m). The maximum factory of safety values obtained are 1.13, 2.44, 1.37 and 1.7. From these results, bolt diameter 17mm,

spacing 0.25 and 1m length of bolt from the face of tunnel (that have a factor of safety 2.44) is recommended to stabilize the tunnel.

Table 5.6 Effect of bolt diameter, length and spacing on factor of safety of the tunnel

Bolt	Bolt Diameter	Bolt Spacing	Bolt Length	Factory of safety
Analysis 1	34	1	1	1.13
	25	1	1	1.04
	19	1	1	0.95
	17	1	1	0.86
Analysis 2	34	1	1	1.13
	25	0.75	1	1.26
	19	0.5	1	1.53
	17	0.25	1	2.44
Analysis 3	34	1	1	1.13
	25	1	2	1.37
	19	1	3	1.72
	17	1	4	1.7



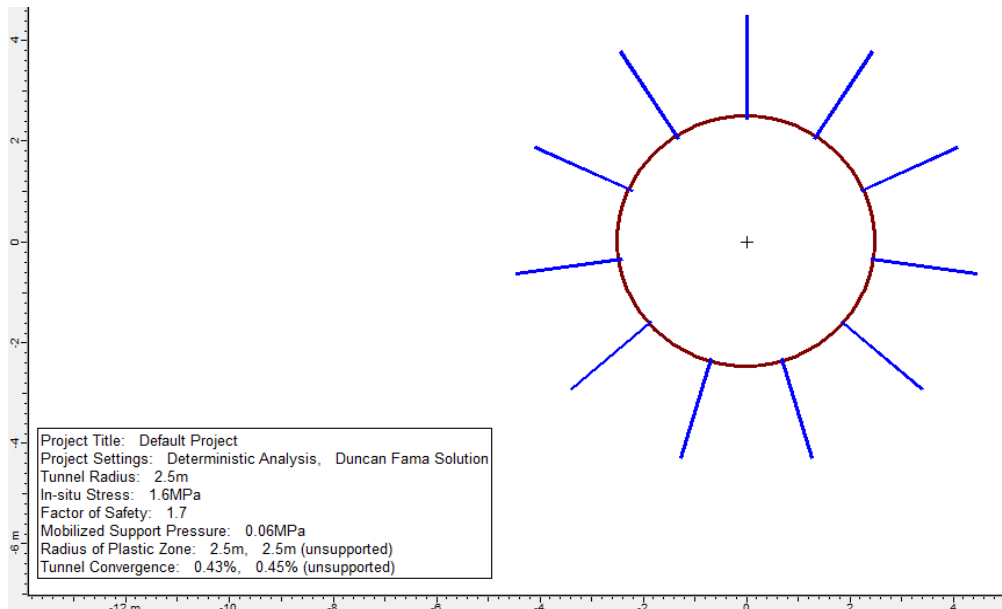
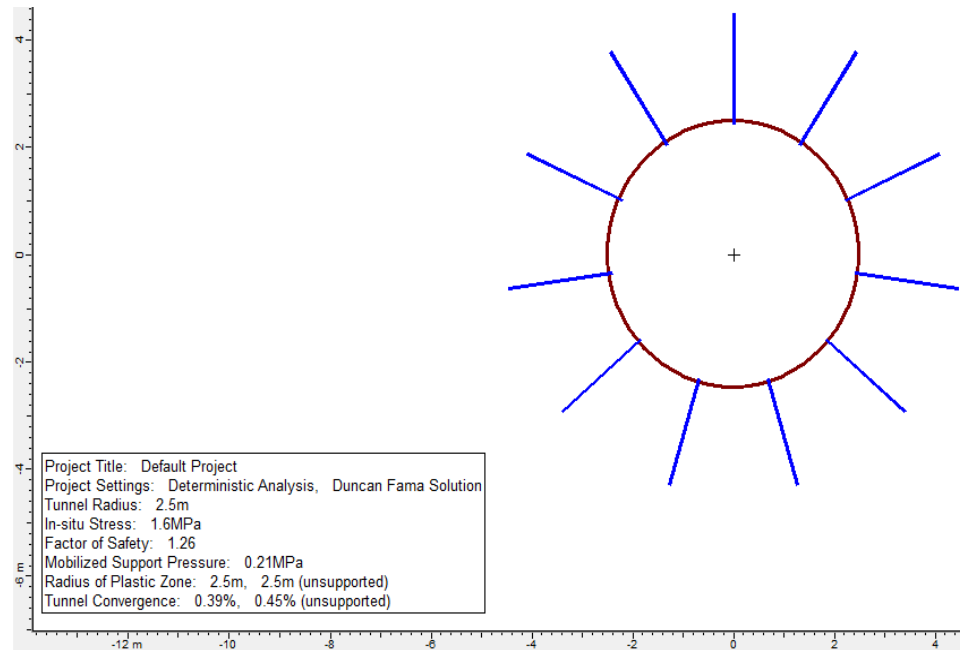


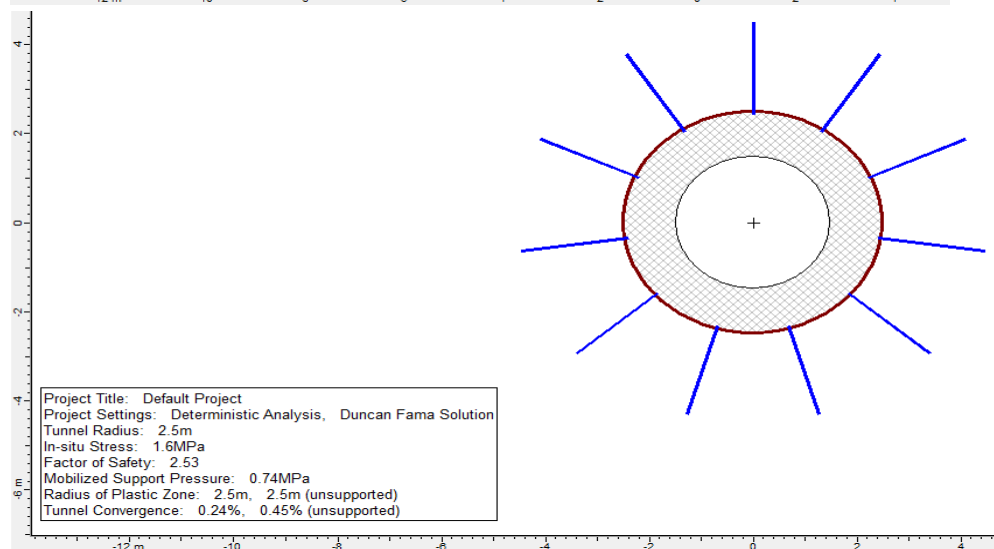
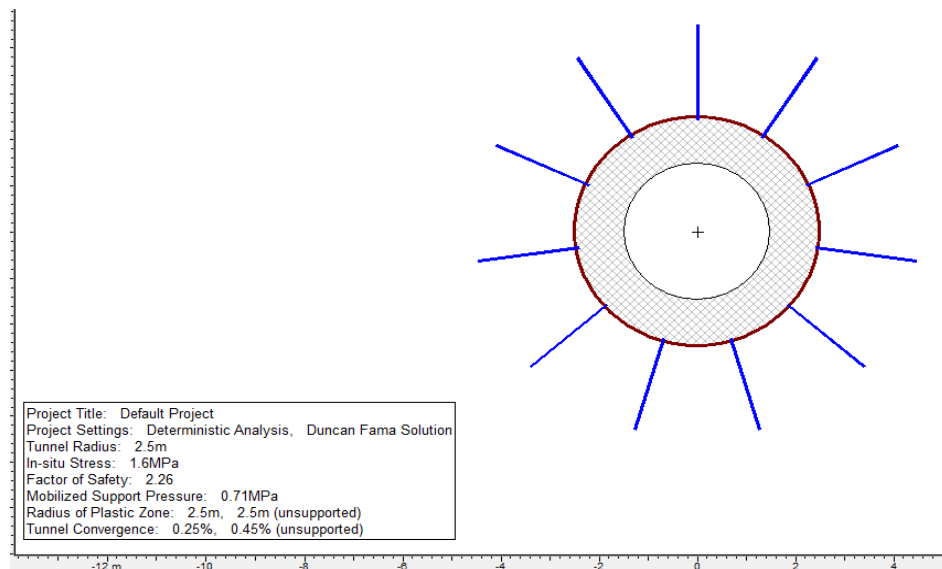
Figure 5.26 Support design by varying bolt diameter, spacing and length

5.6.2 Support by Bolt and shotcrete

The factor of safety of the tunnel can be assessed by applying different bolt diameter (34, 25, 19 and 17mm) and Shotcrete 10mm and the optimized factory of safety (2.26) obtained at 17mm bolt diameter and 1m bolt spacing and 1m bolt length. The details information is presented in table5.7 and figure5.36as follows.

Table5.7 Effect of bolt diameter, length and spacing with shotcrete of 10mm thickness on the factor of safety of the Tunnel

Bolt with schotcrete (10mm)	Bolt Diameter	Bolt Spacing	Bolt Length	Factory of safety
Analysis 1	34	1	1	2.53
	25	1	1	2.44
	19	1	1	2.35
	17	1	1	2.26
Analysis 2	34	1	1	2.53
	25	0.75	1	2.66
	19	0.5	1	2.93
	17	0.25	1	3.84
Analysis 3	34	1	1	2.53
	25	1	2	2.96
	19	1	3	3.58
	17	1	4	4.46



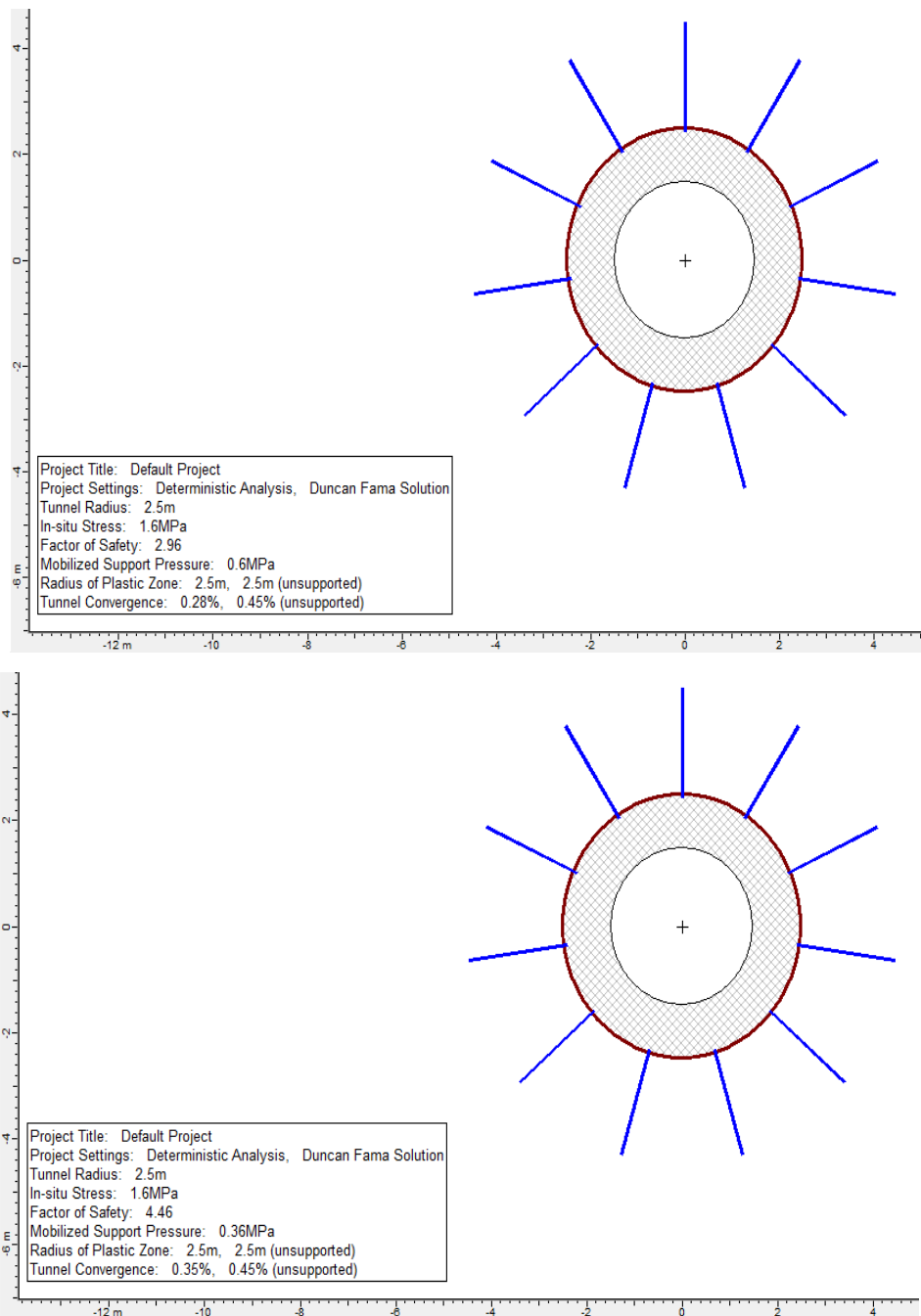


Figure 5.27 Tunnel stabilization using bolt and shotcrete

CHAPTER SIX

CONCLUSION AND RECOMMENDATION

- The local geology of the study area is categorized into four lithologic units: amphibolite, graphitic-quartz mica schist, quartz-mica schist, and metagranodiorite. The area consists of different geological discontinuities like faults, joints and folds. These geological structures are developed in different units with different orientations. The orientations of the discontinuities are vertical to sub horizontal. The tunnel is oriented in SW direction and the joint orientation observed in the study area is oriented in the North to South direction.
- The rock profile in which the tunnel is passing through is characterized by fine grained rock and slightly weathered rocks that have a possibility of failure.
- Along the tunnel sections; two critical rock slope sections were selected. The subsurface conditions of the critical slope section studied through five boreholes and classify into five lithological layers namely; (1) top soils (2) graphitic-quartz mica schist (3) ore body (4) quartz-mica schist and (5) amphibolite.
- As part of the geotechnical characterization, rock slope samples were collected from the underground excavations. The dominant rock type in the tunnel is graphitic-quartz mica schist which is dark grey in color with GSI value of 73MPa and UCS value of 105Mpa. Dry and saturated unit weights range from 26 to 27KN/m³. The Rocdata software showed the cohesion and friction angle of 9.975KN/m² and 43.3° respectively. Similarly, for the rest of the rock units observed in the critical rock slope sections their respective point load strength, dry and saturated density and slake durability were also determined.
- The critical rock slope sections were analyzed using finite element and kinematic analysis methods. Two structurally controlled rock slope failures have been identified and each of them showed a planar and a wedge failure. The factor of safety was obtained from the finite element method.
- The Unwedge analysis was used to show or display the wedge failure observed in kinematic analysis by 3D view. The software interprets the presence of a wedge failure at the roof and lower right side with low factor of safety value. Consequently, two scenarios were considered to reduce the failure and to increase

factor of safety; (1) analysis using bolt (2) shotcrete support system. As a result, the FOS increases more using shotcrete than bolt leading for a safer and stable roof. However using shotcrete over bolt for 1.5km tunnel length would cause economic crisis. Also shotcrete is not an effective for large block. Therefore installing a bolt with appropriate length and diameter at by ranging the distance of bolt insertion along the tunnel line would be better choice.

- In Rocsupport software the factor of safety of the tunnel assessed by applying different bolt diameter (34, 25, 19 and 17mm) and Shotcrete (10mm) and the optimized factory of safety (2.26) obtained at 17mm bolt diameter and 1m bolt spacing and length. At bolt diameter of 34, 25 and 17mm) the FOS 2.53, 2.96 and 2.26) respectively.
- The factor of safety of the tunnel was assessed by varying the bolt diameter (34, 25, 19 and 17mm) and keeping the bolt spacing and length constant (1m).The maximum factory of safety obtained 1.3, 2.44, 1.37 and 1.7.From these results, the bolt diameter with 17mm, spacing 0.25 and 1m length of bolt from the face of tunnel (that have a factor of safety 2.44) is recommended to stabilize the tunnel.
- In addition according to the information obtained from the mine area and the company, they have been using bolt with length of 2m and bolt diameter of 17mm for their support system, however this support system is temporary and it will not hold forever. As a solution in the above software analysis using FEM, analysis with bolt, without bolt, with culvert, without culvert and with bolt and culvert insertion, the analysis showed that the stress distribution decreases when the support is used and increase when less support system is provided.
- Generally, based on the observations from laboratory results and numerical analysis methods, we can easily state that bolt with shotcrete can minimize the instability condition and reduce further weathering of rocks. Therefore bolting with shotcrete is recommended for the safe operation of a mine and transportation of the ore.

RECOMMENDATION

Depending on site investigation, laboratory test result and numerical analysis result of the critical rock slope sections, the following are recommended:

- Advanced detail geotechnical examination, particularly on Joint introduction is required to analyse their impact on the rocks types present at the study area. Physical modelling, limited contrast Roclab, and Rocsupport strategy of analysis can be used to gain a better understanding of the displacement and calculate the safety of support systems. A thorough investigation in stabilization analysis is advised, including bolt and culvert connections, shotcrete and steel installations. A more thorough investigation of the groundwater situation is required. Techniques for rock slope stabilization are proposed for the following critical points and/or areas:

- Creating a permanent support system, such as a bolt with an appropriate and suitable bolt size, for our case 1m bolt length and 17mm bolt diameter as well as other possible support systems, Bolt with 10mm shotcrete while avoiding any water from either inside or outside the tunnel.

In addition, it would be safe and successful support if the capabilities to fund both bolt and shotcrete support systems are available.

REFERENCES

- Abdelsalam, M.G. and Stern, R.J. (1996). *Sutures and Shear Zones in the Arabian-Nubian Shield*. *Journal of African Earth Sciences*, 23, 289-310.
- Aydan, O., Ulusay, R., and Kawamoto, T. (1997). *Assessment of rock mass strength for underground excavations*. *Int. J. Rock Mech*, 1-8p.
- Barton, N (1999). *TBM performance estimation in rock using QTBM*. *Tunn. Tunn. Int.*, 30–34p.
- Barton, N. (1986), *Deformation phenomena in jointed rock*. *Int. J. Rock Mech. Min. Sci. Geomech*, 43–64p.
- Barton, N. (2002). *Some new Q-value correlations to assist in site characterization and tunnel design*. *Int. J. Rock Mech. Min. Sci.* 185–216p.
- Barton, N. (2007). *Rock mass characterization for excavations in mining and civil engineering*. In *Proceedings of the International Workshop on Rock Mass Classification in Underground Mining*, IC, Pittsburgh, PA, USA, May; 3–13p.
- Barton, N.; Bieniawski, (2008) *Z. RMR and Q-Setting Record Straight*. Available online: <http://worldcat.org/>
- Barton, N.; Lien, R.; Lunde, J (1974). *Engineering classification of rock masses for the design of tunnel support*. *Rock Mech.*, 6, 189–236
- Beniawski, Z (1990). *Tunnel design by rock mass classifications*; Army Engineer Waterways Experiment Station: Vicksburg, MS, USA.
- Beraki W (1995). *Structural geology and geochemistry of the Neoproterozoic Adobha and Adola Belts (Eritrea and Ethiopia)*. PhD thesis, University of Giessen, Germany 218 pp
- Bhasin and Grimstad, 1996 R. Bhasin, E. Grimstad. *The use of stress-strength relationships in the assessment of tunnel stability Tunnelling and Underground Space Technology*, 11 (1) (1996), pp. 93-98

- Bhasin, R.; Grimstad, E (1996). *The use of stress-strength relationships in the assessment of tunnel stability*. Tunn. Undergr. Space Technol., 11, 93–98.
- Bieniawski, Z (July 1975). Case studies: *Prediction of rock mass behavior by the geomechanics classification*. In Proceedings of the Second Australia-New Zealand Conference on Geomechanics, Brisbane, Australia, 21–25
- Bieniawski, Z. (1979) *The geomechanics classification in rock engineering applications*. In Proceedings of the 4th ISRM Congress, Montreux, Switzerland, 2–8 September.
- Bieniawski, Z. T. (1989). *Engineering rock mass classifications: a complete manual for engineers and geologists in mining, civil, and petroleum engineering*. John Wiley & Sons.
- Bieniawski, Z.T (1992). *Design Methodology in Rock Engineering: Theory, Education and Practice*; Rotterdam, The Netherlands.
- Bisrat Y (1993). *The geochemistry of the meta-basic rocks of the Adola volcano-sedimentary-ultramafic assemblages: evidence for supra-subduction (SSZ) ophiolitic sequence of Adola, Southern Ethiopia*. Rep EIGS, Addis Ababa, 18 pp
- Carlos A. Jaramillo. (2017). Impact of seismic design on tunnels in rock – Case histories. Underground Space. Volume 2, Issue 2, PP
- Caro, R; Epstein, (2007); *An integrated approach to the long-term planning process in the copper mining industry*. In Handbook of Operations Research in Natural Resources; Springer: New York, NY, USA; Volume 99, pp 595-609.
- Chater A (1971). *The geology of the Megado region of Southern Ethiopia*. Unpubl PhD Thesis Leeds University, UK 193 pp.
- Cording, E.J. and Deere, D.U. (1972). *Rock Tunnel Supports and Field Measurements*. Proceedings of 1st NARETC, Chicago, 601-622
- Cui, Z., Sheng, Q., and Leng, X. (2016). *Control effect of a large geological discontinuity on the seismic response and stability of underground rock caverns: A case study of Baihetan #1 surge chamber*. Rock Mech. Rock Eng., 49: 2099-2114p.

- de Coulomb, C. A. (1972). *Coulomb's Memoir on Statics: An Essay in the History of Civil Engineering*. Cambridge University Press.
- Deere, D.; Hendron, A.; Patton, F.; Cording, E (1966). *Design of surface and near-surface construction in rock*. In Proceedings of the 8th US Symposium on Rock Mechanics (USRMS), Minneapolis, MN, USA, 15–17p.
- Deere, D.U., and Deere, D.W. (1988). *The rock quality designation (RQD) index in practice*. Rock Classification Systems for Engineering Purposes, ASTM STP 984, Philadelphia, 91-101p.
- Fasihnikoutalab, Mohammad & Huat, Bujang & Asadi, Afshin & Daneshmand, Saman. (2012). *Numerical stability analysis of tunnel by PLAXIS*. EJGE. Vol. 17, Bund. D, PP 451-46.
- Feng Yang, Xinlei Sun, Xiangcou Zheng, Junsheng Yang.(2017). *Stability analysis of a deep buried elliptical tunnel in cohesive– frictional (c–) soils with a nonassociated flow rule*. Canadian Geotechnical Journal, Vol. 54, No. 5: pp. 736-741.
- Ghebreab W, Yohannes E, Giorgis LW (1992). *The Lega Dembi gold mine: an example of shear zone-hosted mineralization in the Adola greenstone belt, Southern Ethiopia*. J Africa Earth Sci 15: 489±500
- Ghenzebu WG, Hassen N, Yemane T (1994). *Geology of the Agremaryam area*. Ethiopian Institute of Geological Surveys, Addis Abab
- Gilboy CF (1970). *The geology of the Garibero region of Southern Ethiopia*. Unpubl PhD Thesis, Leeds University, UK, 176 p
- Goel, R. K. and Jethwa, J. L. 1995. *Prediction of support pressure using RMR classification*. Proceeding of Indian Getech Conference, Surat, India, pp 203-205.
- Goodman, R.E., Taylor, R.L., and Brekke, T.L. (1980). A model for the mechanics of jointed rock. *Journal of the Soil Mechanics and Foundations Division ASCE* 94, SM3, 637-659.

- Grimstad, E and Barton, N. 1993. *Updating of the Q-system for NMT. Int. Symposium on Sprayed Concrete - Modern use of wet mix sprayed concrete for underground support, Fagernes*. Eds: Kompen, Opsahl and Berg, Norwegian Concrete Association, Oslo, pp 44-66.
- Hand, D.J. and Henley, W.E. (1997). *Statistical Classification Methods in Consumer Credit Scoring: A Review*. Journal of Royal Statistical Society, 160, 523-541.
- Harrison, J.; Hudson, J.; Popescu, M.E (2000). *Engineering Rock Mechanics: Part 2. Illustrative Worked Examples*. Appl. Mech. Rev., 55p.
- Hoek, E. and Bray, J.W. (1981). *Rock Slope Engineering*. Revised 3rd Edition, The Institution of Mining and Metallurgy, London, 341-351.
- Hoek E, B. E. (1980). *Empirical strength criterion for rock masses*. .
- Hoek E, C.-T. C. (2002). *Hoek-Brown Failure criterion-2002 edition*. Toronto: Proceedings of the North American Rock Mechanics Symposium.
- Hoek, E.; Carranza-Torres, C (2002). Corkum, B. *Hoek-Brown failure criterion-2002 edition. Proc. NARMS-TAC*, 267–273p.
- Hoek, E.; Marinos, P.; Benissi, M (1998). *Applicability of the Geological Strength Index (GSI) classification for very weak and sheared rock masses*. The case of the Athens Schist Formation. Bull. Eng. Geol. Environ., 151–160p.
- Hudson, J. A. and Harrison, J. P. (1997), *Engineering rock Mechanics: An introduction to the principles*. Published by Elsevier Science Ltd. 444pp.
- I.-M.Lee, J.-K.Park (2000). *Stability analysis of tunnel keyblock: a case study. Tunnelling and Underground Space Technology*. Volume 15, Issue 4, pp 453-462.
- Jiading Wang, Youjiang Zeng, Yuanjun Xu & Kaiqiang Feng. (2017). *Analysis of the influence of tunnel portal section construction on slope stability*. Journal Geology, Ecology, and Landscapes. Volume 1, Issue 1, pp 56-65.

- Jiading Wang, Youjiang Zeng, Yuanjun Xu and Kaiqiang Feng.((2017)). *Analysis of the influence of tunnel portal section construction on slope stability*. Geology, ecology, and landscapes. Vol. 1, No. 1, PP 56–65.
- Jibson, R. W. (2011). *Methods for assessing the stability of slopes during earthquakes -a retrospective*. Engineering Geology, Vol.122, Issue 1-2, pp 43-50.
- Jin, Y. B., Cui, X. S., & Lu, X. F. (2000). *Tibet Manla water high spillway tunnel intake slope stability analysis and treatment*. Hydrogeology and Engineering Geology, 6, pp 39–41.
- Jing, L. (2003) *A review of techniques, advances and outstanding issues in numerical modeling for rock mechanics and rock engineering*. Int. J. Rock Mech. Min. Sci., 283-353.
- K.G.Goshtasbi & S.F.Dehghan. (2001). *Stability Analysis and Support Design of Glandroud Coal Mine Tunnel*. 1 7th International Mining Congress and Exhibition of Turkey- IMCET2001, pp265-268.
- Kazmin V Alemu S, Tilahun B (1978). *The Ethiopian basement: Stratigraphy and possible manner of evolution*. Geol Rundsch, 67: 531±546
- Kozyrev V, Girma K, Safonov J, Bekele WM, Tuliankin V, Best-ujev A, Baijapov A, Teweldemedhin T, Gurbanivich G, Kai- tukov M, Arijapov A (1985). *Regional geological and exploration work for gold and other minerals in the Adola goldfields*. Report, EMRDC, Addis Ababa.
- Kun Liu, Xinrong Liu, Zuliang Zhong and Li Yi. (2017). *Analysis on Stress and Stability of Lining in Partially-blocked Tunnel Drainage System*. Journal of Engineering Science and Technology Review, Vol 10, No 3 ,pp139-149.
- Li, A. J. (2008). *Stability charts for rock slopes based on the Hoek–Brown failure criterion*. International Journal of Rock Mechanics and Mining Sciences.
- Merifield RS, L. A. (2006). *Limit analysis solutions for the bearing capacity of rock masses using the generalised Hoek-Brown criterial*. Int J Rock Mech Min Sci .
- Ming Xiao and Chen Zhao, (2017). “*Stability Analysis of Steel Lining at Pressure Diversion Tunnel Collapse Zone during Operating Period*,” Mathematical Problems in Engineering, Volume 2017, Article ID 3280414, 14 pages.

- Newman, A.M.; Bieniawski, Z (2010). *A Modified Version of the Geomechanics Classification for Entry Design in Underground Coal Mines*. Society of Mining Engineers of AIME: Littleton, CO, USA, 23-32p.
- P. Perazzelli and G. Anagnostou.(2012). *Comparing the limit equilibrium method and the numerical stress analysis method of tunnel face stability assessment*. Geotechnical Aspects of Underground Construction in Soft Ground – Viggiani (ed). pp. 463-475.
- Palmström, A. (1982). *The volumetric joint count-a useful and simple measure of the degree of rock jointing*. In Proceedings of the Fourth International Congress, IAEG, New Delhi, 221-228p.
- Palmstrom, A.; Broch, E. (2006). *Use and misuse of rock mass classification systems with particular reference to the Q-system*. *Tunnel Underground*. Space Technol., 575–593
- Peck, W (2000). *Determining the stress reduction factor in highly stressed jointed rock*. Aust. Geomech, 7–60p.
- Raghuvanshi ,T.K., Lensa Negassa , Kala P.M. (2015). *GIS based Grid overlay method versus modeling approach: A comparative study for landslide hazard zonation (LHZ) in Meta Robi District of West Showa Zone in Ethiopia*. The Egyptian Journal of Remote Sensing and Space Sciences.18:235–250
- Romana, M.; Serón, J.B.; Montalar, E (2003). *SMR Geomechanics classification: Application, experience and validation*. In Proceedings of the 10th ISRM Congress, Sandton, South Africa, 8–12p.
- S. J. Peng et al., (2013). "Application of Fuzzy Information Optimization Technology on Analysis of Loess Landslide Stability", Applied Mechanics and Materials, Vols. 321-324, pp. 2389-2395.

- Sainsbury, B.L., and Sainsbury, D.P. (2017). *Practical Use of the Ubiquitous-Joint Constitutive Model for the Simulation of Anisotropic Rock Masses*. Rock Mech. Rock Eng., 1507- 1528p.
- Samir Maghous, José E. B. Winiawer and Vanessa F. Pasa Dutra. (2017). *Stability analysis of tunnels driven in jointed rocks by means of a homogenized limit analysis approach*. International Journal for Numerical and Analytical Methods in Geomechanics. Volume 38, Issue 18. Pages 2009–2032.
- Sen, Z.; Sadagah, B.H (2003), *Modified rock mass classification system by continuous rating*. Eng. Geol. 67, 269–280.
- Souley, M., Homand, F., and Thoraval, A. (1997). *The effect of joint constitutive laws on the modelling of an underground excavation and comparison with in situ measurements*. Int. J. Rock Mech. Min. Sci., 97-115.
- Stead, D. &. (2015). *A critical review of rock slope failure mechanisms: the importance of structural geology*. Journal of Structural Geology, 74, 1-23.
- Stille, H., and Palmström, A. (2003). *Classification as a tool in rock engineering*. Tunnelling and Underground Space Technology, 331-345
- Stille, H., and Palmström, A. (2003). *Classification as a tool in rock engineering*. Tunnelling and Underground Space Technology, 331-345.
- Stille, H.; Palmström, A. (2008) *Ground behavior and rock mass composition in underground excavations*. Underground. Space Technol., 46–64.
- Terzaghi, K. Theoretical Soil Mechanics. (1943). Published Online: 19 DEC 2007.
- Wickham, G.; Tiedemann, H.; Skinner, E.H (1975). *Support determinations based on geologic predictions: 3F, 8T, 13R*. Proceedings Retc. AIMMPE, New York, USA, V1, 1972, Int. J. Rock Mech. Min. Sci. Geomech ,43–64.

- Wickham, G.E., Tiedemann, H.R., and Skinner, E.H. (1972). *Support determinations based on geological predictions*. In Proceedings of North American Rapid Excavation and Tunneling Conference, Chicago, 43-64.
- Worku H, Schandelmeier H (1996). *Tectonic evolution of the Neoproterozoic Adola Belt of southern Ethiopia: evidence for a Wilson Cycle process and implications for oblique plate collision*. Precamb. Res 77: 179±21
- Worku H, Yifa K (1989). *The tectonic evolution of the Precambrian metamorphic rocks of the Adola belt (Southern Ethiopia) and its implication on gold mineralization*. Report, EIGS, Addis Ababa, 56 pp
- Xiao-Li Yang and Rui. (2017). *Limit Analysis of Stability of Twin Shallow Tunnels Considering Surface Settlement*. KSCE Journal of Civil Engineering.
- Xiaozhao Li and Yangsong Zhang. (2009). *Finite Element Analysis of the Stability of Tunnel Surrounding Rock with Weak Rock Layer*. Modern Applied Science. Vol.3 No. 12, pp22-27.
- Xuansheng Cheng; Xiaoyan Zhang; Wenjun Chen; Weiwei Xu. (2017). *Stability Analysis of a Cross-Sea Tunnel Structure under Seepage and a Bidirectional Earthquake*. International Journal of Geomechanics/ Volume 17, Issue 9.
- Zhang, L. (2016). *Determination and applications of rock quality designation (RQD)*. J. Rock mech. and Geotechnical Eng., 389-397.
- Zhang, L., and Einstein, H.H. (2004). *Using RQD to estimate the deformation modulus of rock masses*. Int. J. Rock Mech. Min. Sci., 337-341.

APPENDICES

Appendix 1: Rock material characterization

Appendix 1.1 Name of the Rocks used for Laboratory test

No	Sample ID	Name of the Rock/Lithology
1	RS1	Metagranodiorite
2	RS2	Amphibolite
3	RS3	Quartz Mica Schist
4	RS4	Graphitic Quartz Mica Schist

Appendix 1.2 Schmidt Hammer test results

Sample ID					
Test No	RS1	RS2	RS3	RS4	
1	40	40	26	14	14
2	42	32	26	12	12
3	38	34	20	10(broken)	12
4	34	44	22	12	14
5	40	42	26	12	12
6	42	34	22	12	12
7	36	44	28	12	12
8	36	46	28	12	14
9	30	42	22	12	14
10	32	42	22	12	14

Appendix 1.3 Results obtained from Point Load test

Sam ple ID	Len gth L(m m)	Dep th D (m m)	Upp er Wid th W1 (m m)	Low er Wid th W2 (m m)	W3	Avera ge Width Waver age (mm)	Loa d P (MP a)	Equiva lent diamet er of core sample DE^2	Point Load Index Stren gth Is (MPa)	F	Is (50) (M Pa)	UC S (M pa)
RS1 -01	22.0 7	43. 62	65.4 1	68.8 1		67.11	15.4 9	3717.7	0.042	1.0 8	4.1 5	107 .9
RS1 -02	24.0 9	44. 25	54.7 7	24.0 9		67.86	19.6 99	3813.5 6	0.052	1.0 9	5.6	135
RS2 -01	34.4 9	44. 64	66.0 5	67.8 3		66.94	26.6 57	3795	0.07	1.0 98	7.7	185 .1
RS2 -02	26.6 7	40. 59	57.2 8	69.7 4		63.51	25.8 23	3275	0.078	1.0 6	8.3	200
RS3 -01	40.6	35. 23	105. 05	70.1 8	66. 69	80.64	16.5 5	3608	0.045 8	1.0 8	4.9	118 .8
RS3 -02	28.6 2	25. 88	54.2 6	45.0 1		49.64	8.53 7	1631.3 8	0.023	0.9	4.7	113
RS4 -01	19.1 5	37. 64	68.0 1	84.3 4		76.17	8.85 4	3641.3	0.024	1.0 8	2.6 4	63.
RS4 -02	48.6 2	28. 58	71.4 2	108. 09	89. 57	89.69	15.3 95	3255.5	0.047	1.0 6	5.0 1	120
RS4 -03	20.2 2	30. 99	65.7 9	29.7 1		20.22	15.3 5	1879.3	0.08	0.9	7.3	175 .2

Appendix 1.4 Masses of the samples before conducting any saturated and/or dry density test

No	Sample ID	Mass (g)
1	RS1-01	117.0
2	RS1-02	121.4
3	RS2-01	240.9
4	RS2-02	168.5
5	RS3-01	203.0
6	RS3-02	87.3
7	RS4-01	238.6
8	RS4-02	184.8
9	RS4-03	111.7

Appendix 1.5 Dry Density test results after 24 hr oven dried

No	Sample Name	Dry Mass (g)	Volume (Vi)(ml)	Volume (Vf) (ml)	Vf-Vi	Density $P = m/v$ (g/cm ³)	Unit weight $P \cdot g$ (KN/m ³)
1	RS1-01	116.5	400	445	45	2.69	26.9
2	RS1-02	121.3	400	445	45	2.69	26.9
3	RS2-01	239.6	500	590	90	2.66	26.6
4	RS2-02	168.2	500	562	62	2.71	27.1
5	RS3-01	203.0	700	775	75	2.7	27
6	RS3-02	87.2	600	633	33	2.64	26.4
7	RS4-01	238.6	600	690	90	2.65	26.65
8	RS4-02	184.6	700	780	80	2.305	23.05
9	RS4-03	111.8	500	545	45	2.48	24.8

Appendix1.6 Saturated test after being in water for 24 hr

No	Sample Name	Sturated Mass (after 24 hr) (g)	Volume (Vi) (ml)	Volume (Vf) (ml)	Vf-Vi (ml)	Density $\rho=m/v$ (g/cm ³)	Unit weight $\rho \cdot g$ (KN/m ³)
1	RS1-01	117.5	400	445	45	2.611	26.2
2	RS1-02	122.1	400	448	48	2.54	25.4
3	RS2-01	240.9	500	590	90	2.67	26.7
4	RS2-02	168.7	500	562	62	2.72	27.2
5	RS3-01	204.9	700	778	78	2.62	26.2
6	RS3-02	87.6	600	635	35	2.5	25
7	RS4-01	239.8	600	790	90	2.64	26.4
8	RS4-02	185.8	700	770	70	2.65	26.5
9	RS4-03	112.8	500	545	45	2.5	25

Appendix1.7 Slake duirability results after the 1st and 2nd cylcle

No	Sample ID	Initial Masses (g)	Mass (g) after 1 st cycle	Mass (g) after 2 nd cycle	Final weigth after cleaning
1	RS1	491.7	490.2	487.9	487.8
2	RS2	481.3	481.0	481.0	481.0
3	RS3	480	479.3	478.3	478.1
4	RS4	515.4	514.7	513.2	512.9

Appendix 1.8 Rock Type UCS calculations (Source: MIDROC Gold Mine PLC)

MIDROC GOLD MINE PLC - EAST SAKARO PROJECT

ROCK TYPE UCS CALCULATIONS

Uniaxial Compressive Strength (UCS), Elastic Modulus (E), Poisson's Ratio (v)												
Specimen Ref No				Diameter	Length	L/D	Mass	Density	Failure	E	v	UCS
SAMPLE NO.	Interval		Hole ID	Wits No	(mm)	(mm)	(g)	(kg/m ³)	Load (kN)	(GPa)		(MPa)
	From	To										
HWUCS - 01	79.40	79.52	ESGTH 01	1-01	47.5	117.5	2.5	572	2747	268	66.38	151.24
HWUCS - 02	80.78	80.90	ESGTH 01	1-02	45.7	118.5	2.6	570	2932	253	66.51	154.24
HWUCS - 03	82.78	82.90	ESGTH 01	1-03	47.5	120.0	2.5	582	2737	346	53.27	195.25
HWUCS - 04	89.38	89.50	ESGTH 01	1-04	47.5	118.5	2.5	636	3029	448	76.51	252.81
HWUCS - 05	91.73	91.85	ESGTH 01	1-05	47.5	116.8	2.5	608	2938	358	73.46	202.03
HWUCS - 06	92.52	92.64	ESGTH 01	1-06	47.5	117.4	2.5	626	3009	580	79.19	327.30
HWUCS - 07	94.50	94.62	ESGTH 01	1-07	47.5	119.0	2.5	628	2978	428	78.18	241.53
HWUCS - 08	95.60	95.72	ESGTH 01	1-08	47.5	119.0	2.5	632	2997	522	68.90	294.57
HWUCS - 09	97.30	97.42	ESGTH 01	1-09	47.5	119.0	2.5	622	2950	160	53.35	90.29
FWUCS - 18	314.00	314.12	ESGTH 04	4-18	47.2	118.4	2.5	618	2983	304	68.74	173.74
FWUCS - 19	320.07	320.19	ESGTH 04	4-19	47.2	103.0	2.2	528	2930	302	74.31	172.37
FWUCS - 20	321.43	321.67	ESGTH 04	4-20	47.2	113.4	2.4	578	2913	218	62.82	124.59
FWUCS - 21	323.10	323.22	ESGTH 04	4-21	47.2	115.4	2.4	614	3041	438	82.63	250.32
FWUCS - 19	361.50	361.62	ESGTH 05	5-19	47.4	112.4	2.4	572	2884	348	64.74	197.21
FWUCS - 20	362.40	362.52	ESGTH 05	5-20	47.4	118.4	2.5	612	2929	517	81.53	292.98
FWUCS - 21	365.72	365.84	ESGTH 05	5-21	47.4	118.5	2.5	570	2726	396	72.02	224.41

R3 amphibolite
R3 amphibolite
R3 amphibolite
R3/4 amphibolite + pegmatitic vein
R3/4 amphibolite
R3/4 amphibolite
R3 amphibolite
R3 amphibolite
R3 amphibolite
R3 Amphibolite
R3 Amphibolite
R3 Amphibolite
R3 Amphibolite
R3 Amphibolite
R3 Amphibolite
R3 Amphibolite
R3/4 Amphibolite

Mean 202.33
Max 327.30
Min 90.29
STDEV 68.19

Uniaxial Compressive Strength (UCS), Elastic Modulus (E), Poisson's Ratio (v)															
Specimen Ref No				Diameter	Length	L/D	Mass	Density	Failure	E	v	UCS			
SAMPLE NO.	Interval	Hole ID	Wits No	(mm)	(mm)		(g)	(kg/m ³)	Load (kN)	(GPa)		(MPa)			
	From	To													
HWUCS - 01	157.50	157.62	ESGTH 02	2-01	47.5	117.0	2.5	570	2749	231	41.25	0.34	130.36	R3/4	GQMS
HWUCS - 02	159.21	159.33	ESGTH 02	2-02	47.6	117.3	2.5	562	2692	178	58.91	0.26	100.03	R3/4	GQMS
HWUCS - 03	160.52	160.64	ESGTH 02	2-03	47.6	114.0	2.4	554	2731	186	40.73	0.37	104.52	R3	GQMS
HWUCS - 04	167.27	167.39	ESGTH 02	2-04	47.6	116.2	2.4	570	2757	163	37.74	0.28	91.60	R3	GQMS
HWUCS - 05	169.27	169.39	ESGTH 02	2-05	47.6	116.2	2.4	574	2776	152	39.99	0.38	85.42	R3	GQMS
HWUCS - 06	170.83	170.95	ESGTH 02	2-06	47.6	116.4	2.4	572	2761	188	35.69	0.30	105.65	R3	GQMS
HWUCS - 07	172.07	172.19	ESGTH 02	2-07	47.6	115.0	2.4	564	2756	166	31.91	0.23	93.28	R3	GQMS
HWUCS - 08	173.90	174.02	ESGTH 02	2-08	47.6	115.3	2.4	560	2729	226	45.65	0.34	127.00	R3	GQMS
HWUCS - 09	175.16	175.28	ESGTH 02	2-09	47.6	115.2	2.4	566	2761	217	39.10	0.29	121.94	R3	GQMS + QV
HWUCS - 01	254.66	254.78	ESGTH 03	3-01	47.6	117.3	2.5	586	2807	126	51.09	0.18	70.81	R3	GQMS
HWUCS - 02	256.82	256.94	ESGTH 03	3-02	47.6	11.0	0.2	590	30141	127	48.71	0.39	71.41	R3	GQMS
HWUCS - 03	258.66	258.78	ESGTH 03	3-03	47.6	116.2	2.4	574	2776	158	56.31	0.32	88.59	R3	GQMS
HWUCS - 04	264.22	264.34	ESGTH 03	3-04	47.6	117.4	2.5	576	2757	142	58.93	0.27	80.00	R3/4	GQMS
HWUCS - 05	266.10	266.22	ESGTH 03	3-05	47.6	117.3	2.5	576	2759	107	44.33	0.40	59.88	R3	GQMS
HWUCS - 06	267.74	267.86	ESGTH 03	3-06	47.6	118.0	2.5	580	2762	40	22.50	0.45	22.59	R3	GQMS
HWUCS - 07	269.37	269.49	ESGTH 03	3-07	47.6	117.6	2.5	586	2800	101	47.60	0.24	56.94	R3	GQMS
HWUCS - 08	270.32	270.44	ESGTH 03	3-08	47.6	111.8	2.3	562	2775	120	49.80	0.38	67.34	R3	GQMS
HWUCS - 09	272.39	272.51	ESGTH 03	3-09	47.6	116.0	2.4	574	2781	174	48.03	0.35	97.78	R3/4	GQMS
HWUCS - 01	282.69	282.81	ESGTH 04	4-01	47.2	116.5	2.5	562	2757	148	32.55	0.21	84.58	R3	GQMS
HWUCS - 02	283.30	283.42	ESGTH 04	4-02	47.2	116.6	2.5	574	2766	102	34.76	0.19	58.14	R3	GQMS
HWUCS - 03	284.29	284.41	ESGTH 04	4-03	47.2	116.6	2.5	576	2776	122	51.01	0.20	69.60	R3	GQMS
HWUCS - 04	290.96	291.08	ESGTH 04	4-04	47.2	118.0	2.5	572	2770	238	50.07	0.25	136.02	R3	GQMS
HWUCS - 05	291.74	291.86	ESGTH 04	4-05	47.2	117.4	2.5	568	2765	159	43.89	0.13	90.87	R3	GQMS
HWUCS - 06	294.61	294.73	ESGTH 04	4-06	47.2	115.8	2.5	574	2833	221	84.85	0.13	126.30	R3	GQMS
HWUCS - 07	295.19	295.31	ESGTH 04	4-07	47.2	115.6	2.4	562	2778	218	44.08	0.14	124.59	R3	GQMS
HWUCS - 08	295.74	295.86	ESGTH 04	4-08	47.2	116.4	2.5	568	2789	240	51.55	0.24	137.21	R3	GQMS
HWUCS - 09	297.52	297.64	ESGTH 04	4-09	47.2	116.4	2.5	568	2789	160	51.34	0.21	91.44	R3	GQMS
HWUCS - 01	319.82	319.94	ESGTH 05	5-01	47.6	118.4	2.5	594	2819	174	43.59	0.13	97.78	R3	GQMS
HWUCS - 02	322.16	322.28	ESGTH 05	5-02	47.6	118.4	2.5	588	2791	209	51.14	0.26	117.57	R3	GQMS
HWUCS - 03	322.66	322.78	ESGTH 05	5-03	47.6	118.6	2.5	592	2805	165	51.54	0.24	92.88	R3	GQMS
HWUCS - 04	329.42	329.54	ESGTH 05	5-04	47.6	116.3	2.4	572	2764	244	45.34	0.18	136.95	R3	GQMS
HWUCS - 05	330.96	331.08	ESGTH 05	5-05	47.6	118.8	2.5	592	2800	238	43.19	0.17	133.74	R3	GQMS
HWUCS - 06	332.28	332.40	ESGTH 05	5-06	47.6	118.8	2.5	602	2848	110	50.95	0.26	61.69	R3	GQMS
HWUCS - 07	333.81	333.93	ESGTH 05	5-07	47.6	118.8	2.5	594	2810	196	51.42	0.20	110.11	R3	GQMS
HWUCS - 08	335.13	335.25	ESGTH 05	5-08	47.6	118.6	2.5	588	2786	195	39.42	0.20	109.58	R3	GQMS
HWUCS - 09	336.42	336.54	ESGTH 05	5-09	47.6	118.8	2.5	600	2838	270	46.66	0.33	151.73	R3/4	GQMS + QV
FWUCS - 13	280.86	280.98	ESGTH 03	3-13	47.4	118.4	2.5	584	2795	241	48.24	0.25	136.57	R3	GQMS
FWUCS - 14	282.91	282.53	ESGTH 03	3-14	47.4	117.3	2.5	574	2773	212	40.93	0.39	120.14	R3	GQMS + QV
FWUCS - 15	283.13	283.15	ESGTH 03	3-15	47.2	116.4	2.5	566	2779	153	41.96	0.22	87.56	R3	GQMS + QV
FWUCS - 16	284.68	284.80	ESGTH 03	3-16	47.4	117.4	2.5	570	2751	240	41.62	0.35	136.06	R3	GQMS + QV
FWUCS - 17	286.14	286.26	ESGTH 03	3-17	47.4	117.3	2.5	574	2773	188	32.38	0.38	106.54	R3	GQMS + QV
FWUCS - 18	288.10	288.22	ESGTH 03	3-18	47.2	116.5	2.5	562	2757	213	53.53	0.31	121.73	R3	GQMS + QV
FWUCS - 19	294.28	294.40	ESGTH 03	3-19	47.4	115.8	2.4	564	2760	218	37.50	0.19	123.54	R3	GQMS
FWUCS - 20	296.13	296.25	ESGTH 03	3-20	47.4	118.4	2.5	580	2776	187	43.90	0.34	105.97	R3	GQMS
FWUCS - 21	298.17	298.29	ESGTH 03	3-21	47.4	114.6	2.4	562	2779	212	41.25	0.33	120.14	R3	GQMS
FWUCS - 13	347.17	347.29	ESGTH 05	5-13	47.4	110.6	2.3	542	2777	267	49.36	0.28	151.56	R3	GQMS
FWUCS - 14	348.20	348.32	ESGTH 05	5-14	47.4	112.8	2.4	556	2793	288	42.28	0.20	163.21	R3	GQMS
FWUCS - 15	350.05	350.17	ESGTH 05	5-15	47.5	116.8	2.5	618	2986	173	45.79	0.32	97.81	R3	GQMS
FWUCS - 16	351.27	351.39	ESGTH 05	5-16	47.4	118.0	2.5	574	2757	258	46.16	0.45	146.21	R3	GQMS
FWUCS - 17	352.71	352.83	ESGTH 05	5-17	47.4	117.0	2.5	568	2751	268	62.53	0.27	151.78	R3	GQMS
FWUCS - 18	354.10	354.22	ESGTH 05	5-18	47.4	118.0	2.5	576	2766	143	n.a*	n.a	81.04	R3	GQMS
												Mean	105.02		
												Max	163.21		
												Min	22.59		
												STDEV	29.92		

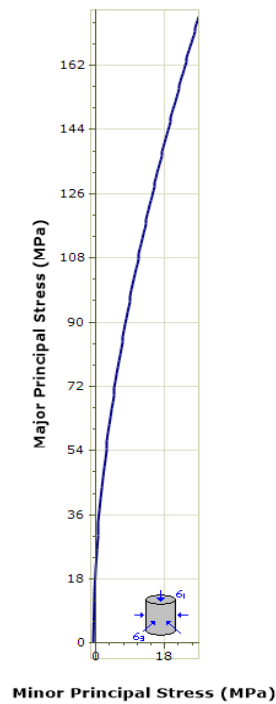
Uniaxial Compressive Strength (UCS), Elastic Modulus (E), Poisson's Ratio (v)												
Specimen Ref No				Diameter	Length	L/D	Mass	Density	Failure	E	v	UCS
SAMPLE NO.	Interval		Hole ID	Wits No	(mm)	(mm)	(g)	(kg/m ³)	Load (kN)	(GPa)		(MPa)
	From	To										
FWUCS - 13	105.17	105.29	ESGTH 01	1-13	47.5	116.6	2.5	570	2759	158	33.29	89.16
FWUCS - 14	105.90	106.02	ESGTH 01	1-14	47.5	117.8	2.5	572	2740	221	45.18	124.71
FWUCS - 15	107.20	107.32	ESGTH 01	1-15	47.5	116.8	2.5	580	2802	173	45.79	97.81
FWUCS - 16	109.39	109.51	ESGTH 01	1-16	47.5	114.6	2.4	560	2758	184	42.93	103.83
FWUCS - 17	110.25	110.37	ESGTH 01	1-17	47.5	117.0	2.5	572	2759	192	39.20	108.35
FWUCS - 18	111.60	111.72	ESGTH 01	1-18	47.5	117.0	2.5	570	2749	138	45.61	77.88
FWUCS - 19	119.42	119.54	ESGTH 01	1-19	47.5	117.3	2.5	572	2752	173	39.02	97.81
FWUCS - 20	121.07	121.19	ESGTH 01	1-20	47.5	117.5	2.5	572	2747	127	43.51	71.71
FWUCS - 13	200.40	200.52	ESGTH 02	2-13	47.5	118.2	2.5	578	2760	204	37.80	115.12
FWUCS - 14	201.66	201.78	ESGTH 02	2-14	47.5	118.1	2.5	614	2934	155	47.40	87.47
FWUCS - 15	203.03	203.15	ESGTH 02	2-15	47.5	117.4	2.5	572	2749	138	27.03	77.88
FWUCS - 16	204.36	204.48	ESGTH 02	2-16	47.5	117.4	2.5	572	2749	153	52.56	86.24
FWUCS - 17	205.78	205.90	ESGTH 02	2-17	47.5	115.3	2.4	566	2770	204	40.15	115.12
FWUCS - 18	207.42	207.54	ESGTH 02	2-18	47.5	116.5	2.5	584	2829	206	53.24	116.25
FWUCS - 19	214.49	214.61	ESGTH 02	2-19	47.5	116.0	2.4	566	2753	216	55.37	121.87
FWUCS - 20	215.80	215.92	ESGTH 02	2-20	47.5	114.5	2.4	554	2730	231	49.83	130.36
FWUCS - 21	216.80	216.98	ESGTH 02	2-21	47.5	115.4	2.4	564	2758	268	60.51	151.24
Mean											104.28	
Max											151.24	
Min											71.71	
STDEV											21.48	

Apenddx 2: Rock mass property determination

Appendix 2.1 Geological strength index for jointed rock masses (Marinos, 2000)

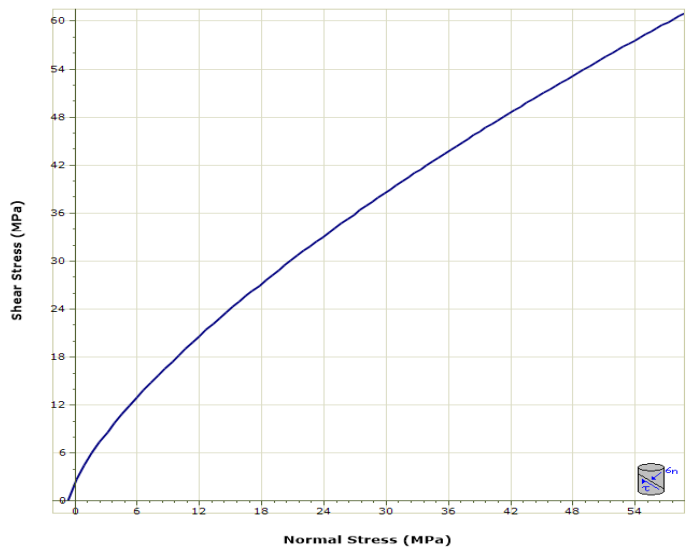
GEOLOGICAL STRENGTH INDEX FOR JOINTED ROCKS (Hoek and Marinos, 2000)		SURFACE CONDITIONS				
From the lithology, structure and surface conditions of the discontinuities, estimate the average value of GSI. Do not try to be too precise. Quoting a range from 33 to 37 is more realistic than stating that GSI = 35. Note that the table does not apply to structurally controlled failures. Where weak planar structural planes are present in an unfavourable orientation with respect to the excavation face, these will dominate the rock mass behaviour. The shear strength of surfaces in rocks that are prone to deterioration as a result of changes in moisture content will be reduced if water is present. When working with rocks in the fair to very poor categories, a shift to the right may be made for wet conditions. Water pressure is dealt with by effective stress analysis.		DECREASING SURFACE QUALITY				
STRUCTURE		SURFACE QUALITY				
INTACT OR MASSIVE - intact rock specimens or massive in situ rock with few widely spaced discontinuities		VERY GOOD Very rough, fresh unweathered surfaces	GOOD Rough, slightly weathered, iron stained surfaces	FAIR Smooth, moderately weathered and altered surfaces	POOR Stickensided, highly weathered surfaces with compact coatings or fillings or angular fragments	VERY POOR Stickensided, highly weathered surfaces with soft clay coatings or fillings
BLOCKY - well interlocked undisturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets		90	80	70	N/A	N/A
VERY BLOCKY- interlocked, partially disturbed mass with multi-faceted angular blocks formed by 4 or more joint sets		80	70	60	N/A	N/A
BLOCKY/DISTURBED/SEAMY - folded with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes or schistosity		70	60	50	40	30
DISINTEGRATED - poorly interlocked, heavily broken rock mass with mixture of angular and rounded rock pieces		60	50	40	30	20
LAMINATED/SHEARED - Lack of blockiness due to close spacing of weak schistosity or shear planes		50	40	30	20	10
		N/A	N/A	N/A	N/A	N/A

Appendix2.2 Rock mass property determination using Rocdata



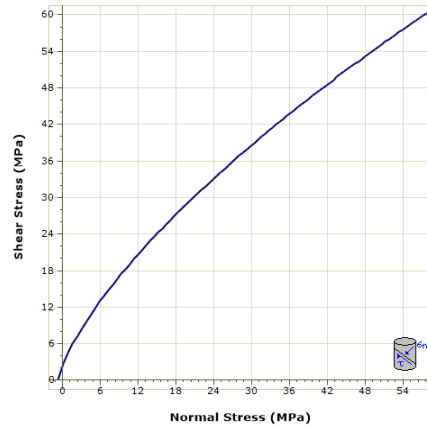
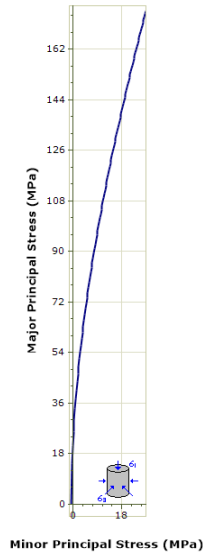
Material 1 - Principal Stress Envelope

Material 1	
Hoek Brown Classification	
intact uniaxial compressive strength	105 MPa
GSI	73
mi	20
disturbance factor	0
intact modulus	5254 MPa
Hoek Brown Criterion	
mb	7.625
s	0.05
a	0.501
Failure Envelope Range	
application	general
sig3max	26.25 MPa
Mohr Coulomb Fit	
cohesion	9.075 MPa
friction angle	43.313 deg
Rock Mass Parameters	
tensile strength	-0.686 MPa
uniaxial compressive strength	23.354 MPa
global strength	42.056 MPa
modulus of deformation	4125.833 MPa



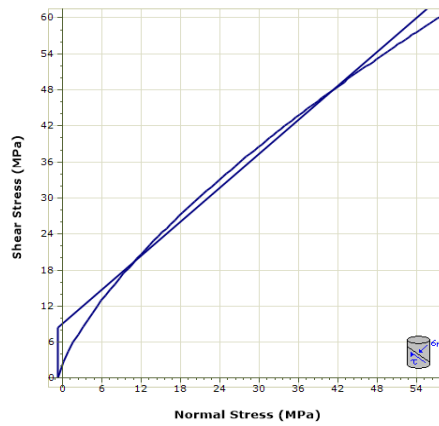
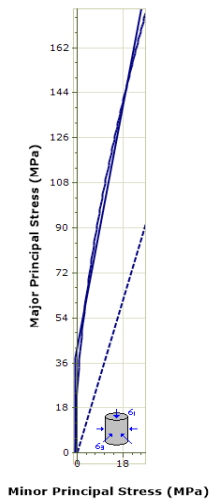
Material 1 - Shear vs. Normal Stress Envelope

Material 1	
Hoek Brown Classification	
intact uniaxial compressive strength	105 MPa
GSI	73
mi	20
disturbance factor	0
intact modulus	5254 MPa
Hoek Brown Criterion	
mb	7.625
s	0.05
a	0.501
Failure Envelope Range	
application	general
sig3max	26.25 MPa
Mohr Coulomb Fit	
cohesion	9.075 MPa
friction angle	43.313 deg
Rock Mass Parameters	
tensile strength	-0.686 MPa
uniaxial compressive strength	23.354 MPa
global strength	42.056 MPa
modulus of deformation	4125.833 MPa



Material 1	
Hoek Brown Classification	
intact uniaxial compressive strength	105 MPa
GSI	73
mi	20
disturbance factor	0
intact modulus	5254 MPa
Hoek Brown Criterion	
mb	7.625
s	0.05
a	0.501
Failure Envelope Range	
application	general
sig3max	26.25 MPa
Mohr Coulomb Fit	
cohesion	9.075 MPa
friction angle	43.313 deg
Rock Mass Parameters	
tensile strength	-0.686 MPa
uniaxial compressive strength	23.354 MPa
global strength	42.056 MPa
modulus of deformation	4125.833 MPa

— Material 1 - Principal Stress Envelope
— Material 1 - Shear vs. Normal Stress Envelope



Material 1	
Hoek Brown Classification	
intact uniaxial compressive strength	105 MPa
GSI	73
mi	20
disturbance factor	0
intact modulus	5254 MPa
Hoek Brown Criterion	
mb	7.625
s	0.05
a	0.501
Failure Envelope Range	
application	general
sig3max	26.25 MPa
Mohr Coulomb Fit	
cohesion	9.075 MPa
friction angle	43.313 deg
Rock Mass Parameters	
tensile strength	-0.686 MPa
uniaxial compressive strength	23.354 MPa
global strength	42.056 MPa
modulus of deformation	4125.833 MPa

— Material 1 - Principal Stress Envelope
— Mogi
— Material 1 - Mohr-Coulomb Envelope
— Material 1 - Shear vs. Normal Stress Envelope
— Material 1 - Mohr-Coulomb Envelope

Appendix 3: Borehole Data

Borehole

ID	Elevation	Depth
ESGTH1	2000.69	192.9
ESGTH2	2009.388	262
ESGTH3	2002.323	354.91
ESGTH4	1993.875	398.73
ESGTH5	1961.194	384.7

