EVALUATION OF STRATA CONDITIONS AND PERFORMANCE OF SUPPORT SYSTEMS OF THE EMERALD SHAFT GOLD MINE IN GAUTENG PROVINCE OF SOUTH AFRICA

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A Dissertation submitted to the Department of Earth Sciences, Faculty of Science, Engineering, and Agriculture, University of Venda, in fulfilment of the requirement for the Master of Earth Sciences in Mining and Environmental Geology.

February 2025

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DECLARATION

I, **Mphaphuli Sathiel**, declare that this dissertation is my work, except where otherwise acknowledged. It is being submitted for the Masters of Earth Sciences in Mining and Environmental Geology at the University of Venda, and it has not been submitted for the award of any degree in any other University.

Student's signature:  Date: 21/02/2025

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ACKNOWLEDGMENT

I extend my most heartfelt appreciation to the following entities and people who provided essential backing and assistance throughout this research project.

First, I offer a great praise to God who has provided His divine protection during every step of this transformative experience.

I extend my deepest gratitude to my co-supervisors Prof. F. Amponsah-Dacosta and Dr. S.E. Mhlongo who granted me the chance to conduct this research alongside their expert supervision and mentoring. Their vision together with sincerity and motivation continues to provide constant inspiration.

I am also grateful to Mr. Ben van Wyk, Director of the Emerald Shaft Gold Mine, for granting me access to conduct research at the mine site. The Rock Engineering and Mining Departments at the Emerald Shaft Gold Mine provided invaluable assistance in gathering data and samples, and I extend my heartfelt thanks to them.

To my closest friends, I express my appreciation for their encouragement and inspiration, which helped me stay motivated throughout this project.

The final acknowledgment goes to my mother Tovhowani Mphaphuli for supplying continuous moral backing and intellectual drive. The Almighty God demonstrates his riches through his blessings upon you all.

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ABSTRACT

The creation of underground excavations causes disturbances in the rock mass, leading to geotechnical uncertainty and increased risk of fall of ground or backbreak. The incompetency of the rock mass, high tensile fractures, and concentration of horizontal virgin and mining-induced stresses pose significant threats to both safety and production, highlighting the need for effective strata control measures.

This study aimed to evaluate the strata conditions and performance of support systems of the Emerald Shaft Gold Mine to ensure long-term stability. The objectives of the study included reviewing the mine design and layout, determining geological conditions and their influence on the rock strata, examining the performance of the support systems used in the raise and production stopes of the Emerald Shaft Gold Mine., and recommending appropriate strategies for strata control.

A comprehensive geotechnical investigation was conducted at the Emerald Shaft Gold Mine to characterize the rock mass and inform mine design. The study applied a multi-disciplinary approach including field profiling, geological and geotechnical mapping, field test and laboratory analysis, field observations, and analytical and numerical modelling. The investigation involved a detailed analysis of rock mechanic data, geological conditions, and raise and stope support systems to evaluate the stability of the mine.

The study revealed complex geological conditions, variable rock competency, and distinct mechanical properties. The rock mass quality was compromised by regional geological features, such as dykes and faults. The findings highlighted the importance of site-specific characterization, comprehensive support design, and targeted support measures to ensure safe and efficient mining operations. The study also identified areas prone to rockfalls, fall of ground incidents, and stope instability, emphasizing the need for robust support design and regular monitoring.

The study concluded that a tailored support design approach, considering tendon performance, ground conditions, and stress loading, is essential to ensure optimal ground stability and prevent rockfalls. The recommended support measures include the installation of grouted dowels, pre-stressed elongates, and fibre-reinforced shotcrete. Regular monitoring and maintenance of support systems are crucial to

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ensure their effectiveness and prevent accidents. The study's findings and recommendations provide valuable insights for mine design, support strategies, and safety protocols at the Emerald Shaft Gold Mine, and can be applied to similar mining operations.

This study contributed significantly to the understanding of strata conditions and support systems in Emerald Shaft Gold Mine. The research provided new insights into the complex geological conditions and variable rock competency of the mine, highlighting the importance of site-specific characterization and tailored support design. The findings of the study and recommendations offered valuable contributions to the existing knowledge on strata control and support systems, providing a framework for improving mine safety and efficiency. Specifically, this research demonstrated the effectiveness of a multi-disciplinary approach in evaluating rock mass quality and designing optimal support systems and highlighted the need for regular monitoring and maintenance to ensure the effectiveness of support measures.

Keywords: *Fall of ground, Mine design, Rock mass characterization, Stability analysis, Strata control, Support design, Geological conditions,*

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LIST OF MEASUREMENT UNITS

'	Minutes
"	Seconds
%	Percentage
kN	Kilonewton
kN/m^2	Kilonewton per Square meters
m	Meter
m^2	Square meters
m^3	Cubic meter
Mb	Rock Mass Constant
mm	Millimetre
MN/m	Millinewton per meter
MPa	Megapascal
ºC	Degree Celsius
S	Spacing
v	Poisson's ratio
w	Stoping Width
E	Young's modulus
ρ	Rock density
σ_1	Major Principal Stress
σ_3	Confined Stress
σn	Normal stress
σt	The maximum tensile stress of the beam
σv	Virgin (Vertical) stress
τ	Maximum shear stress
φ	Internal friction angle

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LIST OF SYMBOLS

A	Total area
c	Cohesion
g	Gravitational acceleration
Sp	Spacing
v	Poisson's ratio
w	Stoping Width
E	Young's modulus
ρ	Rock density
σ_1	Major Principal Stress
σ_3	Confined Stress
σn	Normal stress
σt	The maximum tensile stress of the beam
σv	Virgin (Vertical) stress
τ	Maximum shear stress
φ	Internal friction angle

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LIST OF ABBREVIATIONS

1LE	1 Level East
1LW	1 Level West
ASG	Advanced Strike Gully
COMRO	Chamber of Mines Research Organization
DMR	Department of Mineral Resources
EMRBH	Emerald Raise Borehole
ESR	Excavation Support Ratio
F/W	Footwall
FOG	Fall of Ground
FoS	Factor of Safey
GPS	Geographical Positioning System
GSI	Geological Strength Index
H/W	Hangingwall
ISRM	International Society of Rock Mechanics
MR	Main Raise
N	North
NE	Northeast
NW	Northwest
QTZ	Quartzite
RMR	Rock Mass Rating
RQD	Rock Quality Designation
S	South
SE	Southeast
SIMRAC	Safety in Mining Research Advisory Committee
SR	Support Resistance
SRF	Strength Reduction Factor
SW	Southwest
UCS	Uniaxial Compressive Strength

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CHAPTER ONE: INTRODUCTION

1.1 Background of the Study

Mining encompasses the extraction of economically valuable minerals or geological materials from the earth's surface or subsurface. In 1988, COMRO identified three mining environments namely shallow, intermediate, and deep. These environments are determined by factors like depth, stress fracturing, slope closure rate, virgin stress level, regional support, and geology. However, mining and strata conditions vary depending on the environment encountered.

The underground mine is designed in such a way that the total cost of operation, development, and infrastructure is minimized thereby maximizing the production and safety of the operation (Song *et al.*, 2013). Detailed reserve block and economic modelling are completed and delineated to maximize Net Present Value (NPV) (Little *et al.*, 2012; Song *et al.*, 2013). In this regard, appropriate mine planning and scheduling are required to generate profit. However, the underground mine workings or excavations such as stope, tunnel, and large chamber designs are greatly controlled by the ground conditions. Therefore, an entire mining environment needs to be described, and the ground control district also needs to be defined and delineated.

Ground instability causes block failure and stress failure when produced and virgin stress exceeds the rock mass's strength (Hudson, 1989). The mining-induced seismicity exacerbates and causes failure in the underground mining environment (Potvin and Hadjigeorgious, 2020). However, Hodson and Cook (1967) suggested that when making sure that structures or mine workings are stable, stresses acting on the structures must be always less than that part of the structure. In this regard, a well-designed, and optimized mine layout provides safety to the operation by controlling rockburst and rockfall hazards.

Strata condition acts as a descriptive measure indicating the severity of excavation-induced fracturing, geological discontinuities found around an underground excavation, and potential rockfalls (Güla et al., 1998). The assessment of strata conditions combines mechanical rock mass properties with geological data as well as direct empirical observations of rock masses in practice. Quantitative rock mass classification systems have also been used to categorize and characterize rock mass

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(Watson & Noble, 1997; Malda *et al*., 2021; Qazi and Singh, 2023). The rock mass classification systems in defining geotechnical areas are considered essential for the suitability of mining and civil engineering projects (Güla *et al*., 1998).

The economy, design (mine and support design), and safety of the mining projects are dependent upon the understanding and determination of the engineering properties of rock. Such rock engineering properties include all properties that are relevant to engineering applications before and after extraction. These properties provide information about the quality of the bedrock or rock strata and its suitability for construction purposes. Rock engineering properties are controlled by the geological discontinuities within the rock mass and the inherent properties of intact rock.

In dealing with the strata conditions of underground excavations, a strata control risk management tool is used to ensure adequate control of the underground mining environment thereby minimizing the likelihood of regional and local instability. An effective ground support system, span rock monitoring, control assessments, and safe design are employed as part of the strata control risk management tool, including adherence to support standards and mining layout requirements. In this regard, strata control emphasizes maintaining stability around mine openings (raise, haulages, tip or orepass, crosscuts, and stopes) where operations are currently or will be carried out, taking place. Mining, rock, and geotechnical engineers investigate and comprehend the strata load transfer behaviour, failure mode, support systems load transfer mechanisms and installation processes, as well as strata stability monitoring techniques (Ali, 2022).

Underground rock support is used worldwide to improve the stability and load-bearing capacity of underground excavations for the required life span. The geotechnical or rock engineers select the optimal ground support based on the rock mass properties and design guidelines. For many years, support systems in South Africa were designed using empirical analysis (Rock Mass Rating and Q-system). Recently, tributary area theory has been the methodology employed in South African gold, platinum, and chrome mines (Daehnk *et al*., 2001). The tributary area theory assumes that the weight of the overburden is carried by support units or pillars in proportion to the mined-out area surrounding the support units or pillars.

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In mining, active, and passive support units are installed to increase the strength of the rock mass to be self-supported (Masethe, 2015). According to Bradly and Brown (1993), active support exerts a load and resists rockwall displacement immediately after installation, and passive support develops load when the rock mass surrounding the excavation dilates or deforms. The support performance in the specific ground condition is qualified through visual inspections and quantified with in-situ testing for careful consideration. However, natural pillars (unmined ground) are left to provide regional and local stability for the underground excavation. In pillar design, the safety factor (SF) is used to determine pillar strength, performance, and average pillar stress (Masethe, 2015). During the early stages, researchers adopted an empirical formula to estimate hard rock pillar strength. With the introduction of computer systems, researchers have adopted numerical analysis to estimate pillar strength. In this regard, monitoring support units, underground pillars, and rock mass is significant in preventing potential failures.

Rock engineering has developed remarkably over decades as it progressed from basic empirical practices to modern quantitative engineering methods. The development of analytical and numerical modelling techniques through research has improved understanding of mining-related stress changes and mining strength assessment capabilities for shallow and deep mines (Jager and Ryder, 1999). The South African hard rock mining sector remains susceptible to numerous accidents as rock-related occurrences including rockfalls and rockbursts persist to make up over 50% of total incidents (Jager and Ryder, 1999). The occurrence of rockfalls and seismicity-based accidents became responsible for 30% of all mining fatalities recorded in South African mines during 2024 according to the data collected by Legodi (2024). Rockfalls alongside rockburst events remain key dangers throughout the mining sector (Netshilaphala, 2018) and thus the continuous assessment and management of strata conditions ensures personnel safety and stable mine operations.

1.2 Problem Statement

The underground excavations are created to gain access to the ore deposit (reef) and ease the recovery of the ore deposit. These excavations cause a disturbance in the rock mass or create geotechnical uncertainty which affects both the safety and production of the mine. The incompetency or poor rock mass, high tensile fractures,

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open-up geological discontinuities, concentration and distribution of horizontal virgin and mining-induced stresses in and around the excavation pose significant risk of fall of ground or backbreak. The occurrence of these events causes fatalities and hinders production through stope re-opening which might lead to closure. Specifically, the Emerald Shaft Gold Mine faces several geotechnical challenges that need to be resolved through study to ensure safety, efficiency, and productivity. These challenges include complex geological conditions and variable rock competency, weak hangingwall conditions, high K ratio, unstable wedges, frequent fall of ground incidents, and support unit failures. These issues pose significant risks to worker safety and impact ore extraction, stope performance, and overall mining operations. Although the mining technology has advanced in deep and effective mining, the poor ground conditions at Emerald Shaft Gold Mine, adversely affect the strata conditions and hence, a study is needed to come up with site specific characterization and customized support design. This will enhance the ground stability, safety and efficiency and reduce the risks of accidents, enhance productivity, and stabilize and successful mining operations in the long run.

1.3 Objectives of the Study

The main objective of the study was to evaluate the strata conditions and performance support systems of the Emerald Shaft Gold Mine, in particular the development raise and the stopes to ensure the long-term stability of the mine. The specific objectives of the study were:

- To analyse the mine design and layout of the Emerald Shaft Gold Mine.
- To determine geological conditions and the influence they have on the rock strata or rock mass.
- To evaluate the performance of the types and patterns of support systems used in the raise and production stopes of the Emerald Shaft Gold Mine.
- To recommend appropriate strategies for strata control at the mine.

1.4 Study Questions

The research is aimed at investigating and evaluating the strata conditions of the Emerald Shaft Gold Mine to provide long-term stability of underground excavations. The study will focus on answering the following questions:

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- What governs the design and layout of Emerald Shaft Gold Mine?
- What are the geological conditions and their effects on the strata conditions at the mine?
- What types of support units and systems are used to stabilize the hangingwall of the mine?
- What strategies can be used effectively to prevent the fall of ground and channel failure, and ensure the long-term stability of Emerald Shaft Gold Mine?

1.5 Justification and Significance of the Study

The extraction of rock and minerals from the surface or beneath the earth's surface, recognised as mining, contributes to the economic development of South Africa. The different extraction methods employed are determined by several factors such as geological, economic, and political factors. In underground mines or deep excavations of hard and soft rock, the mine openings are made to gain access to the reef or ore body, safely transportation, and total retrieval or recovery of the ore by stopping or caving. These affect the entire state or strata condition of the excavation. The underground excavations are considered the most dangerous and prone to fall of ground or channel failure (ground instability) and rockburst events (Jager and Ryder, 1999). Hence, groundfalls are the main source of many incidents in an underground environment. Although modern technology and recent techniques have been adopted, the failure of strata in a subterranean environment remains problematic to the safety and production of the operations.

The study focuses on the investigation and evaluation of strata and geology of the Emerald Shaft Gold Mine. This study contributes to the understanding of how the reef or mineralized body and the host rock behave mechanically. The study involves rock mass characterization, geotechnical mapping, and inspections of underground excavations. The outcomes of these aspects will ensure that the ground support design and excavation geometry are suitable for stable ground. Both internal and external stakeholders (owners, and consultants) will benefit from the study thereby reviewing the report which will specify the area of interest within the mine to assist management in updating a code of practice (COP) and review of mine standards. Geology, rock engineering, and mining departments will also benefit as the study report will be useful for current and future use. This study allows these departments to

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facilitate the exploration, design, and maintenance of underground openings. It also contributes to the employee's safety, as it promotes the long-term stability of the underground tunnels. The significance of the study is to examine, assess, and evaluate the rock strata and geological conditions of Emerald Raise to provide insights on how to manage or prevent the failure of underground strata.

1.6 Description of the Study Area

Krugersdorp, a gold mining town in the Gauteng Province of South Africa, has been a hub for underground gold mining activities in the surrounding West Rand area for decades. However, this region has experienced a history of strata failure, fall of ground, and rockburst events, resulting in fatal accidents and premature shaft closures (SIMRAC, 2000; Stacey and Gumede, 2007; and DMR, 2017). To address these strata problems, a comprehensive investigation and evaluation of the Emerald shaft strata conditions were conducted, requiring in-depth knowledge and a broad understanding of the rock mechanics in the study area. This subsection provides an overview of the study location, geography, and geology of the Emerald Shaft area, setting the context for the investigation and evaluation of the strata conditions.

1.6.1 Location of the Study Area

Emerald Shaft Gold Mine is a shallow underground gold mine located southwest side of Krugersdorp between 26º 07′27″ S and 27º 45′07″E in West Village shown in Figure 1.1. The study area is situated 35 km west of Johannesburg, on the R28 reef drives in Mogale City Municipality (West Rand District), next to Sasol garage. Emerald Shaft falls within the mining right 132 (Sithole, 2015).

1.6.2 Geography of the Study Area

The area of study is about 1550 to 1740 meters above sea level. It consists of hilly or undulating plains with occasional steep ridges. There are no major watercourses along the routes. In the western region, there are the Magaliesberg and Witwatersberg mountains. There are two main rivers in the area namely Skeerpoort and Crocodile Rivers with tributaries joining at 30º to these rivers. Heaps of mine waste dump or tailings are all over creating a new landscape in the area (Short *et al*., 2015).

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The area is comprised of distinct kinds of soil from various parental rock materials. Other soils occur, but the soils are reddish-brown to reddish and belong to the Hatton type (Orthic topsoil above mostly red rootless subsoil on rocks). They are mostly light in texture (15-25% clay) and vary in depth from less than 300mm to more than 1.2m in places. Locally, the soil is tan, mottled, and often coarse, with a basal layer of subsoil and Avalon-shaped. Occasionally flat brown soils occur just above the bedrock and belong to the mispah type (Paterson, 2008).

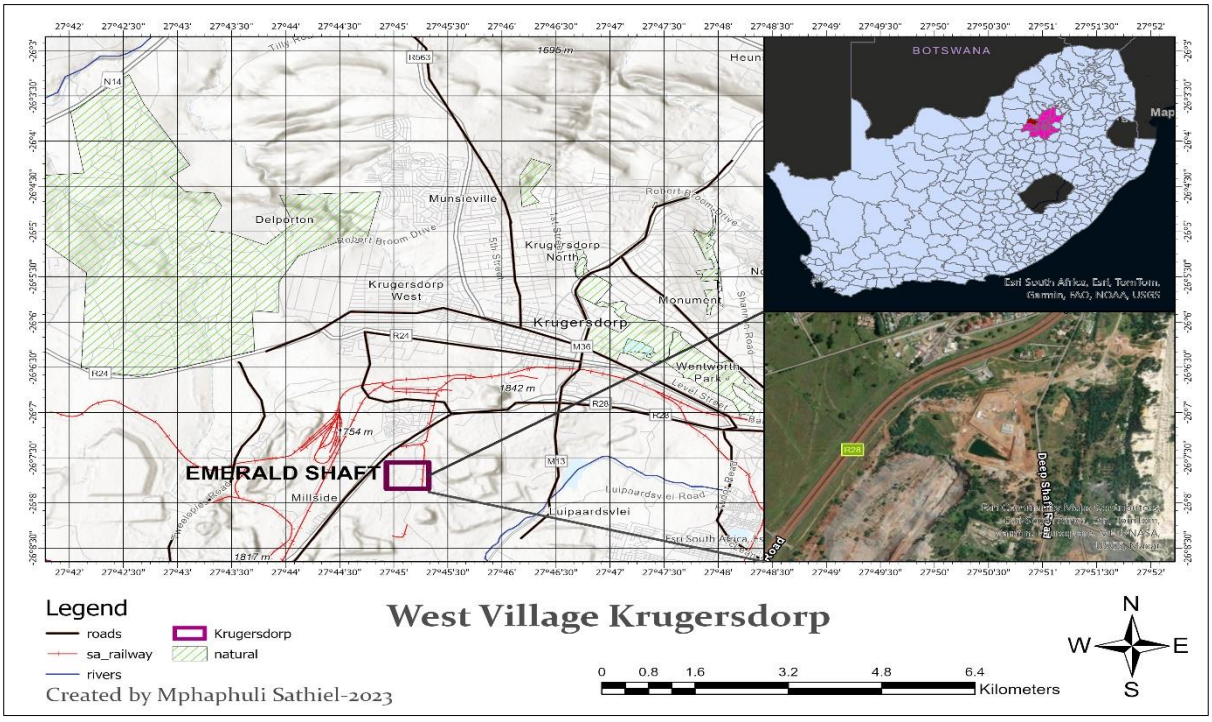


Figure 1.1: Location map of West Village Krugersdorp, Emerald Shaft.

1.6.3 Geology of the Study Area

Gauteng geology falls under the Kalahari Precambrian belt. The Kalahari craton consists of two cratons namely: the small Zimbabwe craton to the north and the large Kaapvaal craton to the south. These cratons are separated from one another by the Limpopo belt (Jacob *et al.*,2008). The Kaapvaal craton is the only area where mid-Archean (3.6-2.5 Ga) volcanic and sedimentary rocks are well preserved. This craton is made up of various deformed early Archean greenbelts intruded by tonatic gneisses and a variety of granitic plutons (Arndt, 2011).

The Emerald Shaft is geographically situated in the Witwatersrand Supergroup of the Witwatersrand basin. The West Rand group consists of dolomite, limestone, iron

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formation, conglomerate, shale, quartzite, and black reef (Au or gold) Formations illustrated in Figure 1.2. Between Randfontein and Krugersdorp there are small covers of the Venterdorp supergroup (conglomerate, shale, and quartzite) of government formation and lava of the Klipriviersberg formation (Geological Survey, 1986).

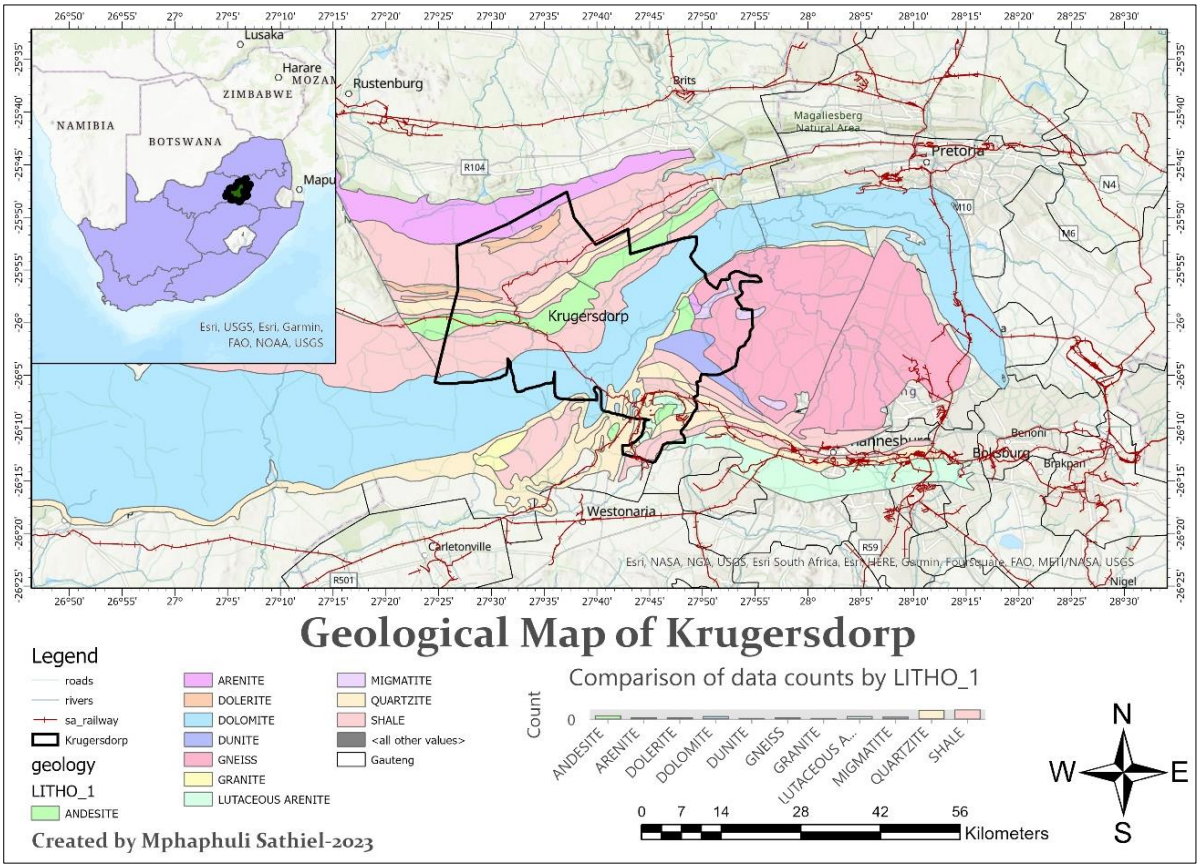


Figure 1.2: Geological sequence of Krugersdorp.

1.7 Organization of the Dissertation

The organization of the dissertation provides a clear and concise overview of the structure and content of the dissertation. It serves as a roadmap, guiding the readers through the dissertation and helping them understand the overall organization of the dissertation. This dissertation comprises six chapters and the following section provides an overview of the contents chapters.

Chapter 1 introduces the research topic, and provides background information, problem statements, objectives of the study, study questions, justification and significance of the study, and study area description. It concludes with an overview of the dissertation's structure and content.

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Chapter 2 is earmarked for literature review, and it provides an overview of the existing literature on strata control and support systems in underground mines, highlighting the geological and geotechnical factors that affect rock mass behaviour.

Chapter 3 describes the research methodology used to investigate the strata conditions and performance of support systems at the Emerald Shaft Gold Mine. Key methods used in this study toss assist in data collection include diamond drilling, geotechnical core logging, rock face mapping, and laboratory rock mass testing.

The methodology chapter sets the stage for Chapter 4: Results and Discussion, which presents the findings of the study. This chapter covers various aspects of the Emerald Shaft Gold Mine, including a review of the design and layout of the mine, orebody description, analysis of the stratigraphy, rock strength, ground conditions, and rock mass quality of the study area.

Chapter 5 focuses on determining the support units or elements of the mine and designing suitable support systems based on the determination of support requirements. This chapter used empirical and analytical approaches to design suitable support systems for the mine.

Finally, *Chapter 6:* Conclusion and recommendations summarises the main findings of the study, outlines the conclusions drawn, and provides recommendations for future research and practice. This section explores the strategic value of the study for both geotechnical engineering and mining fields.

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CHAPTER TWO: LITERATURE REVIEW

The literature overview delivers extensive analysis of strata situations within mine sites including their effect on excavations below ground and the rock masses' reactions to mining activities. The review makes use of existing studies to bridge crucial gaps while also addressing existing underground mining challenges for effective strata control strategy development. Key topics covered in this review include geological setting, site investigation methods, and support determination, providing a foundation for understanding the complex interactions between strata conditions, rock behaviour, and mining operations.

2.1 Regional Geology of Witwatersrand Basin

The Witwatersrand basin is a significant gold mining field, with 55% of recovered gold coming from auriferous sediments. The first gold-bearing conglomerates were discovered in 1886 (Pretorius, 1974). The basin is filled with sediments and volcanic deposited between 2.5 and 2.7 Ma ago and stretches from Johannesburg to the Free State province. According to Rodney et al. (2016), the combination of white quartzite (Orange Grove Quartzite), which forms the outermost stratigraphic layer of the basin, with water, led to the origin of the name Witwatersrand, meaning "White Water Ridge. This basin took the form of a yoked or overlie basin with a radius of about 350 km (Ainslie, 1981). It consists of various strata, including the Witwatersrand supergroup, Dominion group, Granitoid, and Greenstone illustrated in Figure 2.1. The Venterdorp supergroup, consisting of the Klipriviersberg group (915 m thick), and the Main-Bird group, composed of Bird amygdaloid (Table 2.1).

A full succession of the members comprising the Witwatersrand sequence is not present in the basin. Some of the groups are partially or completely missing due to erosional removal or depositional overlapping (Tankard *et al*., 2013). Table 2.1 represents the stratigraphic assemblage (composition) and thickness of all groups that are present in different localities of the basin. The basin had an early period of high crustal instability (Pretorius, 1981). Earth materials entering the basin are drawn from the Simatic lithosphere, containing limited quantities of sediments mixed with volcanism (Von Gruenewaldt et al., 1991). The intermediate or middle stages of basin development were devoid of volcanic activities, with continental and shallow-water

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sedimentation-dominated events (Els, 1995). The middle stage forms the intensive sedimentation in which the ratio of sand and shale increase, respectively (Catuneanu, 2001).

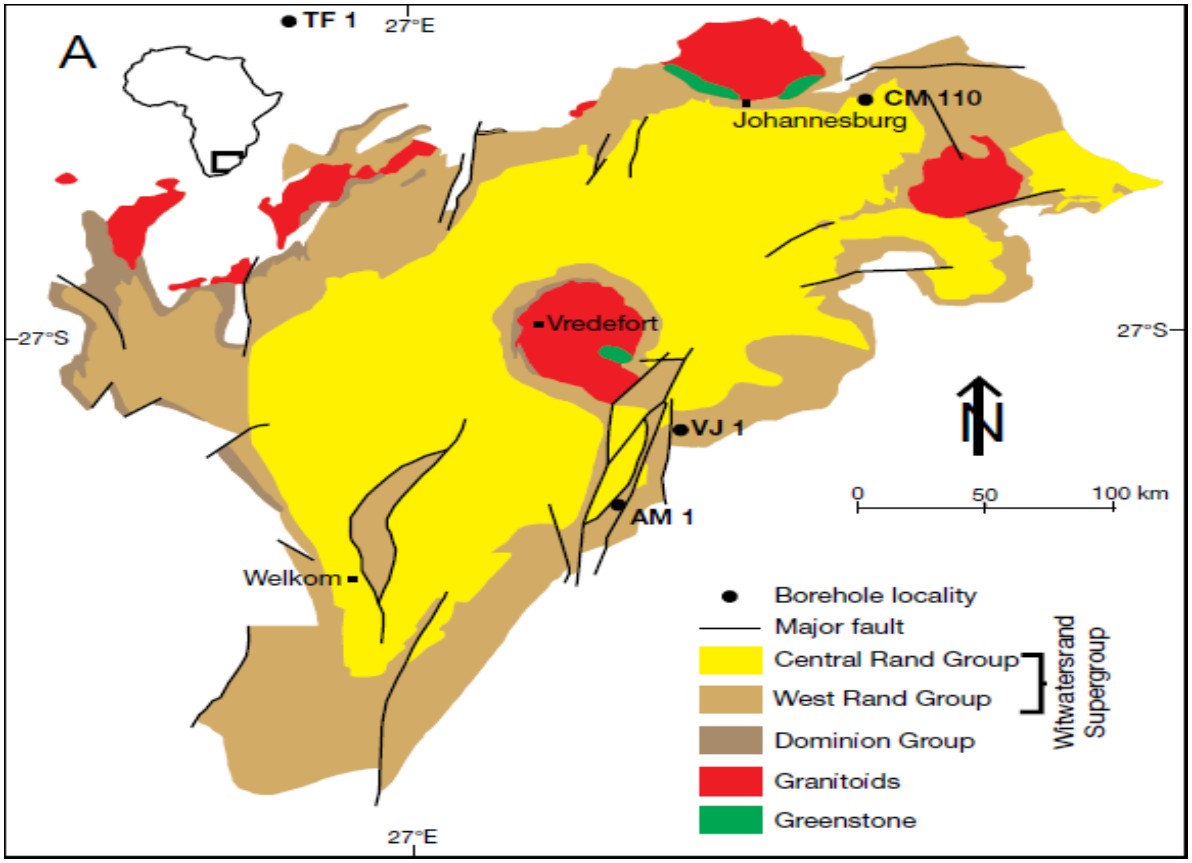


Figure 2.1: Geological map of the Witwatersrand Basin (Smith *et al*. 2012)

Table 2.1: Composite stratigraphical thickness of Witwatersrand sequence (Modified from Pretorius, 1974)

Group	Total (m)	Volcanic (m)	Quartzite (m)	Shale (m)
Klipriviersberg	3050	740	130	180
Kimberly-Elburg	1670	2	1640	30
Main-Birds	1490	0	1010	180
Jeppestown	1380	300	410	550
Government	1970	420	1240	730
Hospital Hill	1620	0	610	1010
Dominion	2720	0	60	10
		2650		
Klipriviersberg	3050	2740	130	180
Upper Witwatersrand	3160	300	2650	210
Lower Witwatersrand	4970	420	2260	2290
Dominion	2720	2650	60	10s
Witwatersrand	13900	6110	5100	2690

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The 13900m thick stratigraphic column contains argillaceous (2 960m) and arenaceous (5 100m) sediments, and volcanic (6 110m) products, with a low fine-grained percentage indicating high energy during formation, crucial for Witwatersrand mineralization (Pretorius, 1974). Viljoen (1963) and Whiteside (1970b) described Witwatersrand succession volcanic rocks, forming the Dominion group. The lower basic formation is 610m thick and is underlain by 40 m of conglomerates, shaly quartzites, and sericitic quartzites, while the upper acidic formation of 2400m thick composed of rhyolite, and a subordinate amount of andesite, tuff, and volcanic ash. The upper volcanics lack sediments. However, Burk *et.al*. (1986), and Rodey *et.al*. (2016) suggested that Witwatersrand sediments are underlaid by metamorphosed volcanic and minor classic sediments, and much of the basin is overlaid by volcano sedimentary.

2.3 Witwatersrand Supergroup

Two billion years (2.9 Ga) ago the Witwatersrand Supergroup accumulated as clastic sedimentary rocks along with a minor presence of volcanic formations. The supergroup contains two main divisions which include the upper Central Rand group and the lower West Rand group (SACS, 1980). The Central Rand group is the largest, with a thickness of 7400m (Figure 2.2), and has more gold-bearing deposits than the West Rand group with minor gold-bearing conglomerates.

2.3.1 Central Rand Group

The Central Rand group contains two separate subgroups known as Johannesburg and Turfontein. Johannesburg is composed of quartzite and conglomerates, with gold-bearing ones like the South Reef Main Reef Leader. Turfontein, the last subgroup, is made up of conglomerates and quartzite, with gold-bearing ones like the Kimberly Reef and Elsburg Reefs. The majority of Witwatersrand Basin gold deposits exist within Central Rand conglomerates (Nethengwe, 2015).

2.3.2 West Rand Group

The West Rand group is an argillaceous formation with arenaceous horizons (Lurie, 2008). The formations comprise three subgroups known as Hospital Hill, Government, and Jeppestown. Hospital Hill is the oldest subgroup, consisting of alternating quartzite

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and shale, and is 1500m thick. Banded ironstones, chemical sediments, and quartzite are misnomers. Government and Jeppestown groups have sub-greywackes, composed of quartz, chlorite, muscovite, and fine-grained clays. Jeppestown group contains intercalated quartzites, conglomerates, shales, andesites, tuff, basalt, and diamictite, and is about 1140m thick. Volcanics are typically between 30 and 100m high, and carbonic shale has been discovered from Jeppestown. Shales are made up of quartz, clay minerals, sericite, chloritoid, pyrophyllite, and chlorite.

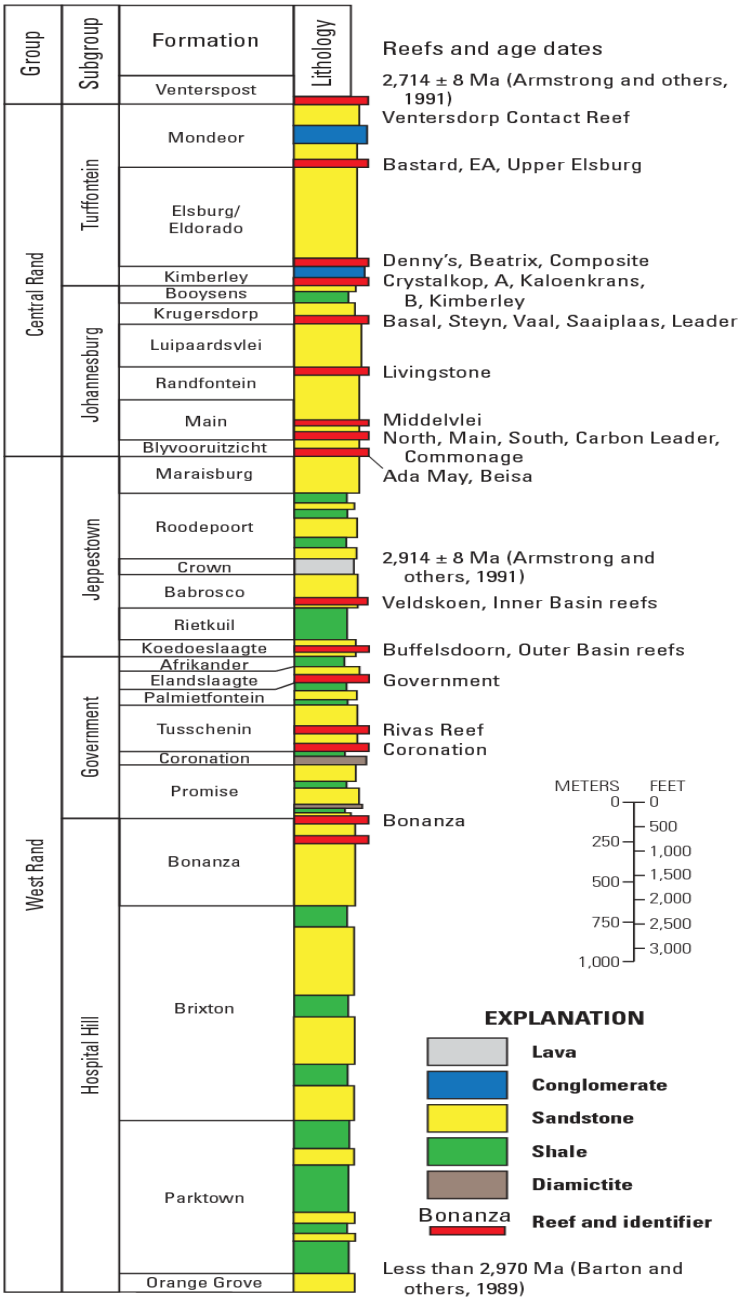


Figure 2.2: Stratigraphic column showing the position of the Witwatersrand supergroup (Ryan and Anderson, 2018)

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2.4 Goldfields within the Witwatersrand Basin

Within the Witwatersrand basin, there are nine gold-bearing goldfields where gold was mined or is currently being mined. These goldfields include the West Rand goldfields, Carletonville goldfields, Klerksdorp goldfields, Welkom/Free State goldfields, Central Rand goldfields, East Rand goldfields, South Rand Evander goldfields, Evander goldfields, and the Kroonstad goldfields. Figure 2.3 shows the location of different goldfields within the Witwatersrand basin. For this study, attention will only be given to the West Rand goldfields which fall within the area under study.

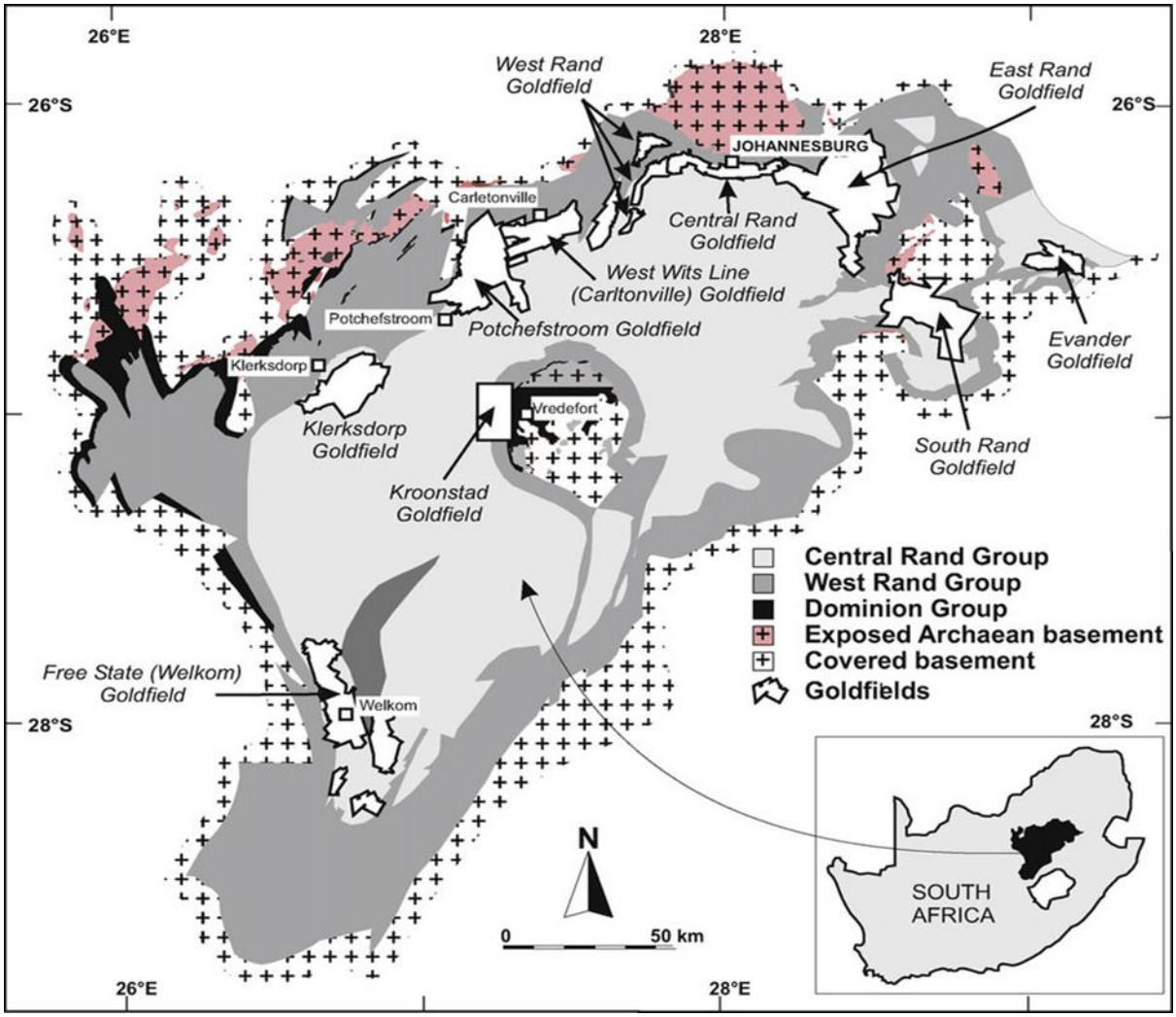


Figure 2.3: The geological map of Witwatersrand basin and its goldfields distribution (modified after Pretorius *et al.,*1986)

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2.4.1 West Rand Goldfields

A generalized stratigraphic column represents the West Rand goldfields as depicted in Figure 2.4 with its economic reef bands. The stratigraphic column starts from the oldest reef bands at the bottom to the youngest reef band at the top which happens to be the Ventersdorp Contact Reef (VCR) and is present throughout the mining lease area. The Ventersdorp Contact Reef is concentrated at the base of the Ventersdorp Supergroup, so it does not form part of the Witwatersrand Supergroup.

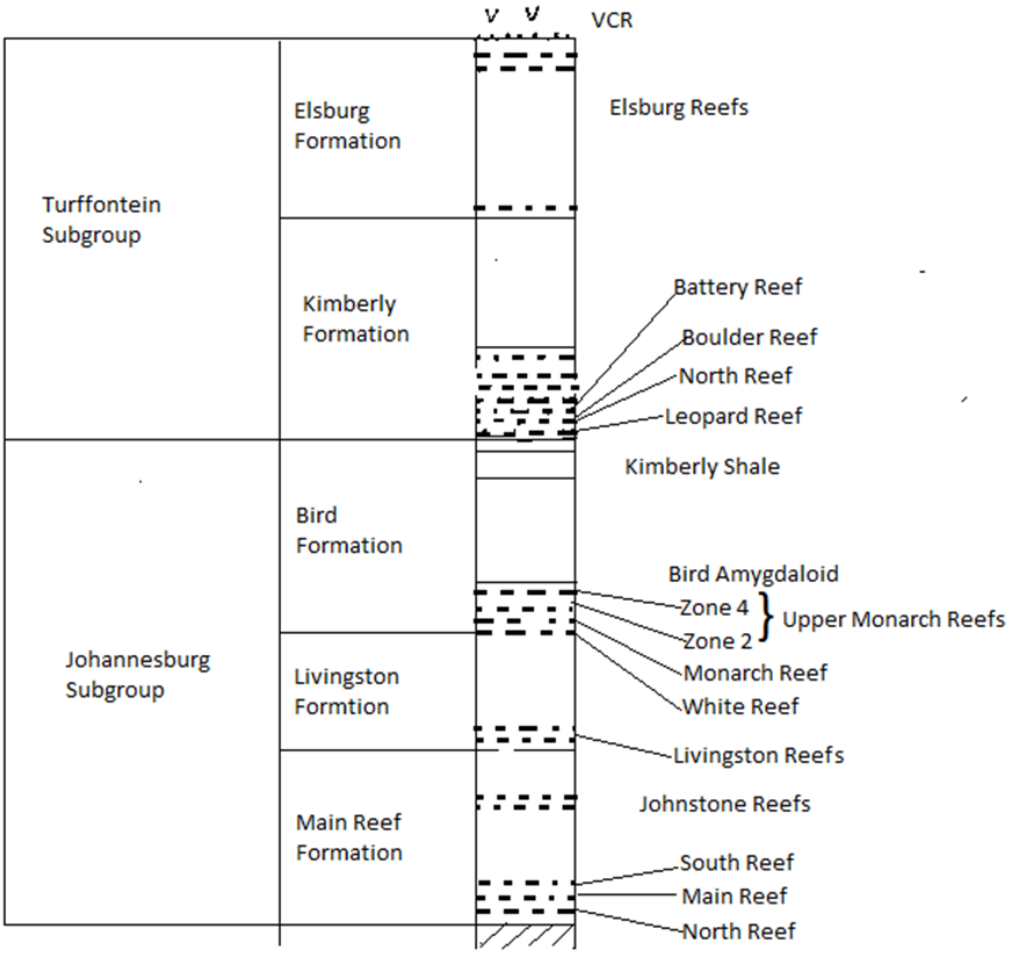


Figure 2.4: Shows a generalized stratigraphic column of the West Rand goldfield (Lurie, 2008)

According to (Lurie, 2008) a total of 20 gold-bearing conglomerates have been mined on the West Rand goldfields by different companies. These companies include West Rand Consolidated (WRCM), Randfontein Estate Gold Mine (REGM), Doornkop Mine, and Durban Roodepoort Deep (DRD). Of all these gold-bearing conglomerates; the main source of gold came from the South and Reef Main Reef. Other reef bands that

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are worth mentioning that were heavily mined and others are currently being mined include the Bird Reefs, Elsburg Reefs, Livingstone Reefs, and the Kimberly Reefs.

2.4.2 Major Structure of West Rand Goldfields

The West Rand Goldfields comprises of Witpoortjie gap and the West Rand syncline geological structures or features. The Witpoortjie gap was formed because of faulting. It is a horst block that is bounded by two normal faults. These normal faults are the Witpoortjie fault and the Roodepoort fault. West Rand goldfields are also separated by the Witpoortjie fault from central Rand goldfields. The regional structural feature on the West Rand goldfields close to Emerald Shaft Gold Mine is the West Rand Syncline which formed because of broad folding shown in Figure 2.5. It pinches out in the south-southeast direction (Anderson, 1994). The strike orientation of the reef bands changes on either side of the West Rand Syncline. The Northern limb of the West Rand Syncline strikes the East to west direction and dips South, and the Southern limb strikes Northeast.

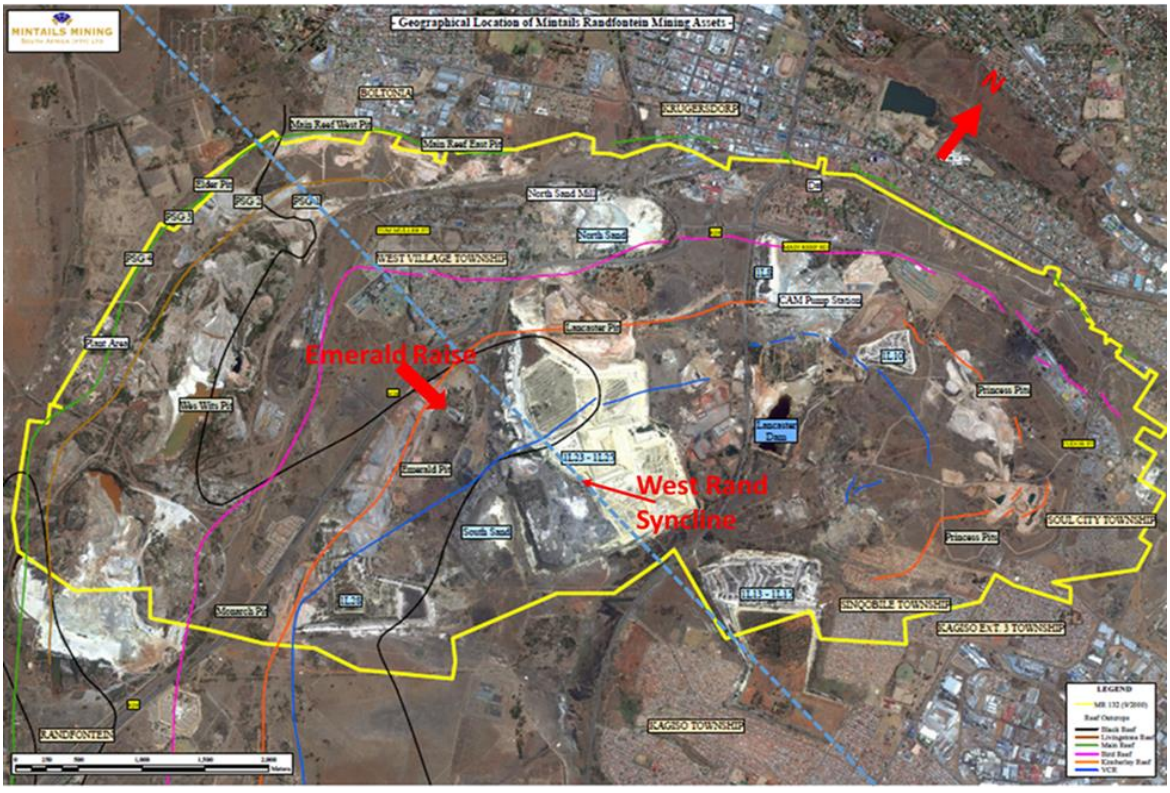


Figure 2.5: Regional structural features of the West Rand goldfield near Emerald Raise in mining right 132 (Sithole, 2015)

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2.5 Mine Design and Layout

The rock engineering study is of great importance in establishing principles (code of practice) for the design and monitoring of mining excavations. The prime objectives are to maximize stability, safety, and cost-effectiveness. The effective mining depth, local geological structures, strength of effective mining strategies at any depth, and prevailing rock mass conditions are influencing factors for global mining strategies. These also include natural factors that include the dip angle of the ore body and thick or multiple reefs which have a huge influence on the mine procedure and standard (Jager and Ryder, 1999).

The underground mine workings and structures are designed to possibly eliminate failure. However, in the deep mine excavation, the degree of failure of the rock is inevitable only if the mining operation continues. Wilson and Morc-O Ferrel (1970) suggested that the extent of rock failure matches the estimated field stress values which can be observed through underground examinations. In making sure that structures or mine workings are stable, stresses acting on the structures must be always less than that part of the structure (Hodson and Cook, 1967; Stacey, 2001; Chehreghani *at.al*, 2022). Before extracting and excavating, most of the rock is impressively sturdy in its physical composition. The mechanical continuity of the rock is interrupted by geological disturbances (dyke, sills, fracture, faults. bedding, and parting plains). The surroundings of the deep excavation contain fractured or broken rock as the result of blasting effects. This is regarded as unstable and unsafe conditions. An appropriate mine design practice eliminates those hazards.

Hodson and Cook (1967) discovered that traditional engineering design methods focus on complete failure elimination when constructing various structures. Practically, the possibility of failure is obtained by maintaining safety factors (the relationship between experimentally determined strength value and predicted structural stress (Stacey, 2001). The duo declared that the chosen criterion ensures adequate protection for structural safety regardless of above or below-ground constructions. However, deep mine excavations will limit the mining operations that exist in their present form.

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2.6 Mine Operating Environment

The underground mining environment varies depending on the locality of the orebody. These environments can be either shallow, intermediate, or deep. In this regard, COMRO (1988) defined those three mining environments, and their associated conditions presented in Table 2.2. The depth of mining plays a central role when decisions need to be made about mining methods and support strategies. However, the strata conditions and mining conditions are distinct according to the environment encountered.

Table 2.2: Mining environment and its associated conditions (Jager and Ryder,1999)

	Shallow	**Intermediate**	**Deep**
Typical depth (m)	<1000	1000-2500	>2500
Stress fracturing	Minimal or none	Moderate	High
Stope closure rate	10	10-30	>30
ERR (MJ/M^2)	<8	8-40	>40
Regional support	Essential	Optional	Essential
Rock burst hazards	Minimal	Moderate to severe	Sever
Geological influence	Strong	Moderate	Moderate
Vertical virgin stress level (MPa)	Low <25	Moderate 25-60	High >60

2.7 Influence of Geological Structures on Mining Conditions

The presence of geological structures within a rock mass creates unfavourable mining conditions. Hoek and Marinos (2007) demonstrate that rock stope and slope failures mainly depend on the presence of rock mass internal discontinuities and their interactions. Meanwhile, Pluijm and Marshak (2004) established a definition of geological structures that identifies natural rock and rock mass features that can be described by their dimensions and spatial distribution patterns. These geological structures include faults, joints, bedding planes, schistosity, and foliations. Hence, Priest (1993) divided natural discontinuities into joints, faults, fractures and fissures, bedding, and cleavage. These features have common characteristics of negligible high fluid conductivity, low shear strength, and tensile strength contrasted to the nearby rock material. Fookes and Parrish (1969), Attewell and Woodman (1971), Priest (1975), and Goodman (1976) adopted “discontinuity” to cover the whole range of mechanical defects while avoiding any interference conceding their geological origins.

A Joint can be defined as a fracture or crack in a rock mass that has little or no replacement (Price,1966). Within approximately 1km of the Earth's surface, joints can

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be discovered in all proficient rocks, in various orientations, and with sizes that range from a few mm to several 100m. The formation of joints results from several typical geological processes explained by Priest (1993) such as intrusive rock cooling producing columnar joints and tectonic activities and rock stress relaxation alongside surfaces that undergo uplift and erosion. The rock mass contains two types of joints which include systematic ones that result in parallel sets and non-systematic ones that display random orientations. Barton (1976) observed that a joint's shear strength smoothly transitions from its initial roughness-dependent shear strength to the high strength based on typical stress displayed by the joint plane. Rock engineers need to account for voids that occur from both regular and irregular joint arrangements. When underground mining exposes a joint system, loose blocks, and wedges formed through joint systems can fail to stick to their position in the hangingwall due to gravitational forces exceeding tensile strength. It is crucial to recognise, reinforce, and pass through joints appropriately when conducting underground operations. It is necessary to use a suitable and precise standard to distinguish and categorize joints properly.

A fault, according to Price (1966), is a surface in a rock mass where the material on either side exhibits significant displacement because of shear failure. The most common method for finding faults is to look for features of the rock mass, like veins or bedding, that don't line up properly when viewed from either side of the fault plane. A fault may also be indicated by the development of slickensides or the presence of fault gouge (Priest, 1993). Faults are produced because of tectonic stresses. Kersten (1969) asserts that slipping occurs when shear stresses exceed shear strength on a particular plane within the rock formation. Whitten and Brooks (1972) asserted that instead of developing as a single flat feature within a rock formation, faults typically take the form of an area made up of fracture sets that are closely spaced out and aligned in the same direction. According to Priest (1993), gouge and brecciated material commonly appears in fault zones.

According to Priest (1993), the term "bedding" describes a surface formation on a sedimentary rock brought about by changes in the size, orientation, mineral composition, or chemical characteristics of the sediment during its deposition. Bedding planes may denote a variation in either colour or texture, and they do not always imply a break or interruption in the rock layer, according to Priest (1993). Several variables

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including mineral composition, grain size distribution, and grain orientations influence bedding plane makeup (Giani, 1992).

The term "rock cleavage" describes a property of rocks in which a pattern within the material is created by the alignment of minerals and results in mechanical differences in various directions. Metamorphosis, in which flat or elongated minerals within the rocks align themselves according to the force being applied, causes the formation of fissures and uneven strength in rocks (Giani, 1992; Ghosh, 1993).

2.8 Ground Control District (GCD)

The Witwatersrand and Bushveld complexes exhibit fluctuating rock mass conditions, necessitating comprehensive geotechnical classification systems for optimized stoping environment management (Brady and Brown, 2005). Mine design methods are used for regional and local scales, determining mining methods, layout, sequence, and regional support based on estimated values of key parameters (Hoek and Brown, 1997). To ensure safe and effective operations, strategies for controlling local strata must be identified after determining regional strategies (Bieniawski, 1989).

Areas designated as geotechnical present unique rock behaviour and hazardous conditions which stem from mining operations (Hoek *et al*., 2000).). Predicting rock mass behaviour near underground excavations is challenging due to their intricate nature (Brady and Brown, 2005). Previous excavations in similar geological settings can provide insights, but a system of categorization is necessary to provide an initial understanding of the potential behaviour of rocks around the planned excavation site (Bieniawski, 1989). Understanding these factors is crucial for ensuring safety and safety in mining activities.

Typically, when there is limited knowledge about the characteristics of the rock formation, plans for subterranean excavations are devised at the outset of a project (Hoek *et al*., 2000). These design procedures rely on comparable past encounters in geological conditions. The classification system functions as a vital tool for regional correlation purposes to establish average rock mass strength (Bieniawski, 1989).

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2.9 Rock Mass Classification Systems

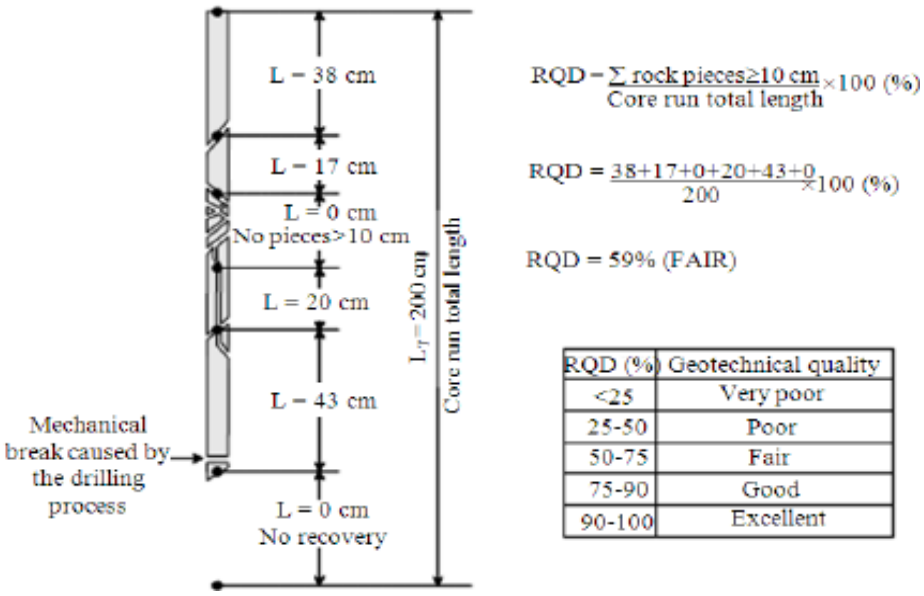
The process of rock mass classification organises rock masses into defined categories based on established relationships (Bieniawski, 1989). The rock mass classification systems employ an empirical evaluation method for rock quality assessment that forms the base for experimental geotechnical designs used extensively in rock engineering. Thus, providing the foundation for predicting rock mass behaviour and enabling comparison of experiences across different rock formations. Rock mass classification systems serve as tools during project initiation to predict rock mass quality and necessary support requirements (Rochmadi and Karnawati, 2023).

Rock mass classification systems like RQD, RMR, Q-system, GSI, and MRMR are widely used to improve site investigations by systematically identifying and quantifying input data. These systems supply rock mass-type authentication parameters through checklists followed by stability analysis that quantifies results as poor through excellent classifications. However, major drawbacks include using one classification method, ignoring analytical and observational design methods, and insufficient input data. The selection of classification systems depends on different parameters, yet researchers advise applying multiple methods when inspecting sites at the beginning of projects (Hoek, 2000).

2.9.1 Rock Quality Designation Index

Deer *et al*. (1967) introduce the concept of rock quality designation (RQD) of rock mass from the drilled core. RQD is the most fundamental form of rock classification. It is defined as the sum of the core pieces longer than 100mm in a core run expressed as a percentage of the total core run length (Deer *et al*.,1967) illustrated in Figure 2.6.

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RQD (%)	Geotechnical quality
<25	Very poor
25-50	Poor
50-75	Fair
75-90	Good
90-100	Excellent

Figure 2.6: Procedure for measurement and calculation of RQD (After Deere, 1967)

Despite the common RQD obtained from borehole core measurements, many formulas had been created, deduced, or formed using the mapping data. Palmström (1982) proposed a method for calculating the Rock Quality Designation (RQD) for clay-free rock masses. The determination of the RQD requires counting discontinuities per unit volume of outcrop and adit exposure area using this quantitative equation:

$$RQD = 115 - 3.3Jv \quad (1)$$

Jv represents the combined number of joints throughout the length of all joint sets identified as the volumetric joint count. RQD displays the in-situ rock mass quality through a measurement of the fractures present in the rock formation.

2.9.2 Rock Mass Rating

As a quantitative empirical way to assess the quality of rock mass, Bieniawski (1973) initially published the rock mass rating (RMR) system. The grades that Bieniawski assigned to many parameters have undergone major revisions. This topic of rock mass strength assessment appears in current publications alongside the original 1976 version. Rock mass rating is considered one of the best rock mass classification systems. The RMR system is used to categorize rock mass determined in the field and interpretation of core drilled data using six parameters (Bieniawski, 1989): uniaxial compressive strength (UCS) of the rock material, RQD, spacing of discontinuities, orientation of discontinuities, condition of discontinuities and groundwater conditions.

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2.9.3 Tunnelling Quality Index

Barton *et al*. (1974) established the Q-system as a method to evaluate rock mass characteristics and required tunnel support. The Q-system is based on the evaluation of numerous (more than 200) underground cases in which the value of the Q-system on an algorithmic scale from 0.001 to 1000 (Barton *et. al*, 1974). In the Q-system classification, the rock mass is classified based on the six parameters such as the RQD index, the number of joint sets (Jn), the roughness of the weakest joint (Jr), the degree of alteration or infilling along the weakest joints (Ja), the joint water reduction factor (Jw), and the stress reduction factor (SRF) (Rochmadi and Karnawati, 2023).

$$Q = \frac{RQD}{Jn} \cdot \frac{Jr}{Ja} \cdot \frac{Jw}{SRF} \quad (2)$$

These six parameters are measured from rock face mapping and core logging. The value of the Q-system is obtained using the equation 2. The rock block size RDQ/Jn combined with the inter-block shear strength Jr/Ja, and the active stress JW/SRF form a ratio system of determining a Q-value.

2.9.4 Mining Rock Mass Rating

As an extension of the Rock Mass Rating system that was suitable for mining, the Mining Rock Mass Rating (MRMR) system was initially established in 1974. Following the same principle as RMR rating values correspond to fundamental rock mass parameters based on parameter condition assessments (Laubscher, 1990). The following factors are evaluated under the MRMR: RQD, rock strength (UCS), joint spacing, condition, and water. In contrast to the RMR system, the MRMR modifies the original rating out of 100 for use in mine planning and design. These modifications are specific to the mining context. Each subsequent adjustment rating multiplies the original rating to derive the following adjustments that include weathering, orientation mining-induced stress, and blasting adjustments. Input values for the MRMR system can also be obtained from exposed rock surfaces and boreholes (Laubscher, 1990).

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2.9.5 Geological Strength Index

Apart from other rock mass classifications, the geological strength index (GSI) is the recent classification system proposed by Hoek (1995) as an alternative to RMR to determine rock mass strength. The introduction of GSI sought to overcome the weaknesses of the Rock Mass Rating (RMR) system when assessing extremely poor rock masses. Additionally, the GSI sought to avoid double-counting the impact of groundwater and joint orientations, which are already accounted for in the RMR system, as well as the joint water reduction factor (Jw) and the stress reduction factor (SRF) in the Q-system. This classification system exists as a tool in engineering rock mechanics to provide dependable information about rock masses needed for the numerical design of slopes, tunnels, and foundations within the rock environments (Hoek, 2000The Geological Strength Index (GSI) classification system is based on the concept that numerous discontinuities within a rock mass can be simplified to an operationally homogeneous and isotropic rock mass. The GSI system assumes isotropic rock mass behavior, implying that the response to loading is direction independent. Consequently, its application is limited to rock masses lacking a dominant structural orientation or gravitational instability influenced by structural features. When the failure of rock masses does not depend on anisotropy the Hoek-Brown criterion along with the GSI chart shown in Figure 2.7 can be used with proper precautions (Hoek and Marinos, 2001).

The sources of data for estimating the GSI value of the rock mass are obtained from measurements of the excavated slope, tunnel faces, outcrops, and borehole core logging (Bertuzzi *et. al*, 2016). However, the GSI value of the rock mass can best be determined through observations of exposed excavation faces, tunnel openings, and slopes that have been excavated. The GSI values range from 10 for extremely poor and 100 for intact rock, calculated using the RMR_{89} (Bieniawski, 1989), and Q-system (Barton *et. al*,1974) as:

- $GSI = RMR_{89} - 5$ for $RMR_{89} > 23$ (3)
- $GSI = 9LogQ + 44$ for $RMR_{89} < 23$ (4)

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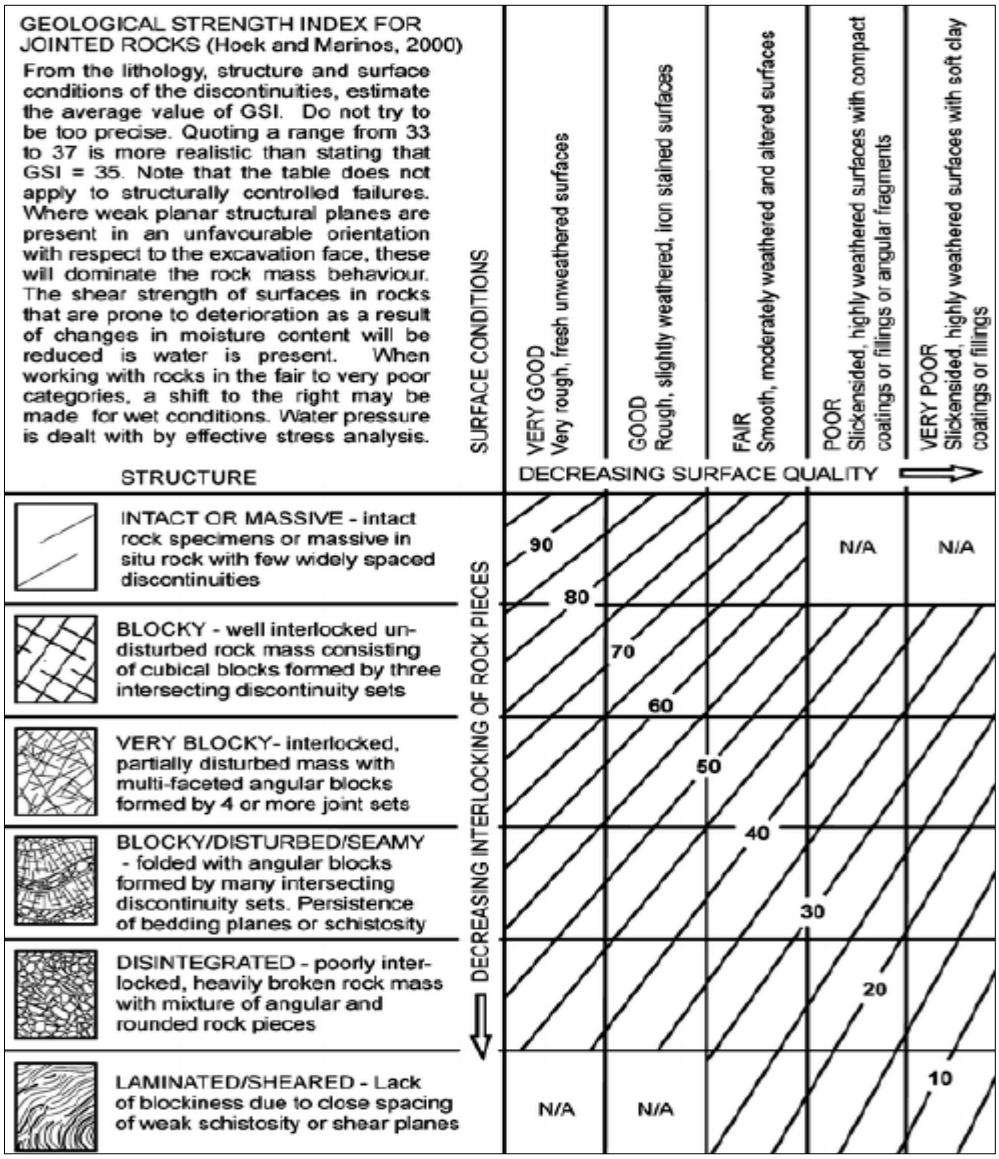


Figure 2.7: The GSI chart for estimating rock mass quality (Hoek and Marinos, 2000)

2.10 Rock Types and Rock Strength

In South African hardrock mines, hardrock orebodies contain gold, chrome, platinum, copper, diamond, and nickel. Typically, the common rock types such as igneous, metamorphic, and massive sedimentary associated with these orebodies are summarised in Table 2.3. At West Rand gold mines, igneous and low-grade metamorphosed sedimentary rock is the major host rock where gold ore is recovered.

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Table 2.3: Typical rock type in South Africa hardrock or tabular environment (Middindi Consulting, n.d.).

Igneous rock	Sedimentary rock	Metamorphic rock
Pegmatite	Argillite	Quartzite
Granite	Bended iron formation (BIF)	Schist
Pyroxenite	Coal	Gneiss
Kimberlite	Conglomerate	Slate
Andesite	Chert	Anthracite
Anorthosite	Shale	Marble
Norite	Dolomite and Limestone	Serpentinite

A wide knowledge of rock mass strength is required for the planning of both surface and subsurface excavations. According to Edelbro (2006), enhancing the predictive ability for rock mass strength and gaining a deeper understanding of rock mass failures can mitigate mining stability issues, optimize waste rock extraction, and improve both working conditions and operational costs in underground mines. The different rock kinds exhibit a wide spectrum of rock mass characteristics. This is due to the discontinuous and diverse nature of rock mass. Compositional variation can cause variations in uniaxial compressive strength (UCS) within a single rock mass. However, Stavrapoulou (1982), discovered that the heterogeneous characteristics of Witwatersrand quartzite created different mechanical properties within this rock material. The strength of quartzite increased with quartz concentration. Güler *et al*. (1998) presented a classification of UCS in complete rock that is summarised in Table 2.4.

Table 2.4: The classification of UCS of intact rock (Modified after Güler *et al*.,1998)

Category	UCS (MPa)	Description
1	<120	Soft
2	120 – 200	Medium
3	>200	Hard

The distinction between weak and strong rock types, aids in identifying areas susceptible to support units punching into the hangingwall, unwinding neaby support units, and locations requiring additional support measures such as the use of areal coverage support. It also helps to calculate the excavation's safety factor, commonly known as the factor of safety. The UCS is most important in shallow mining areas with low-stress fractures. Table 2.5 summarises the weak and strong rock types encountered in South African hardrock gold mining.

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Table 2.5: Rock competency categories for South African hardrock gold mines.

Weak rock types	**Strong rock types**
Westonaria formation lava	Alberton formation lava
Argillaceous quartzite	Siliceous quartzite
Shale	Siliceous conglomerate
Argillaceous conglomerate	

2.10.1 Rock Mass Strength Estimation

In geotechnical engineering and geomechanics, intact rock strength is one of the fundamental parameters required to forecast rock mass behaviour (Ewy, 1999). According to Hoek (1977) and Šancer et al. (2022), UCS is crucial for solving engineering difficulties such as roof support, pillar design, excavation technique, and rock mass classification. The assessment of rock mass strength in a stress environment predicts the stability of underground excavations. This is achieved by taking into account rock mass parameters such as UCS, friction angle, Poisson's ratio, and young modulus. When outlining GCD, UCS distinguishes between weak and strong rock mass zones.

Researchers have utilised a variety of approaches to assess rock mass strength. These procedures involve field and rock laboratory testing of chip and drilled core samples (Burbaum, 2023). The most often used laboratory tests to determine rock mass strength are uniaxial compression strength, triaxial compression strength, point load index, and indirect compression strength (Brazilian test) (Hoek, 2001; Hack and Huisman, 2002; Mehranpour and Kulartilate, 2016, Burbaum, 2023). Hack and Huisman (2002) measured the strength of the rock mass in the field with a Schmidt hammer (which measures the "rebound hardness" of the teste rock material) and estimated it using geological hammer blows. When laboratory testing is unavailable, Hoek (2001)) advised that basic field rock testing be performed for measuring UCS, as detailed in Table 2.6. The ability of the rock to withstand breakage under the impact, as well as the results of simple field hardness tests using a geological hammer, can estimate the strength of the rock mass. These estimations are useful for initial design work, but more accurate results should be obtained by laboratory testing for comprehensive design studies.

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Table 2.6: Field estimation of UCS of the rock mass (Modified from ISRM, 1978)

	Grade and term	Field identification	Approx. σ_c (MPa)
0	Extremely soft rock	– Dented by thumbnail.	0.25 – 1
1	Very soft rock	– Crumbles with firm hammer blows and easily peeled with a pocketknife.	1 - 5
2	soft rock	– Difficult to peel with a pocketknife, shallow indentations with firm geological hammer blows.	5 – 25
3	Medium rock	– Resists pocketknife. The sample can be fractured with a single firm geological hammer blow.	25 – 50
4	Hard rock	– Requires multiple geological hammer blows to fracture.	50 – 100
5	Very hard rock	– Needs numerous geological hammer blows to fracture.	100 – 250
6	Extremely hard rock	– Can only be chipped with a geological hammer.	> 250

2.10.2 Laboratory Rock Mass Testing

Laboratory rock testing is a quantitative method that utilises drill core samples and small chips with a diameter of 20 to 50 cm to determine the strength parameters of the rock mass. Depending on the kind of rock test to be performed and the parameters to be evaluated, there are differences in sample preparation. Uniaxial compression (point load test), triaxial compression test (confined compression), Brazilian test (indirect tension test), shear test, biaxial and multiaxial test are used to determine the uniaxial compressive strength and tensile strength of the rock mass. The Brazilian test also known as the indirect tension test, uniaxial compression test, and triaxial compression test which involves confined compression are briefly explained in this study.

2.10.3 Uniaxial Compression Test and Point Load Test

The uniaxial compressive strength (UCS) is determined in the laboratory on regular test pieces where compressive stress is applied along two axes (two directions). However, the preparation of these test bodies is often a time-consuming and costly process. In this sense, core or lump rock samples can be cheaply tested in a field or laboratory context by using the point load test (PLT), which is used in rock or

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geotechnical practice to assess rock strength indices (Rusnak & Mark, 2000). The technique (PLT) is appropriate or useful in the field since it is rapid and does not necessitate the processing of rock samples, which may have uneven shapes.

According to ISRM (1985), 50 mm should be used as the reference diameter. The determination of rock material uniaxial compressive strength in a given region requires calculating the mean strength of the rock sample through this equation:

$$\sigma_C = \sigma_{C50} \left(\frac{50}{D}\right)^{0.2} \quad (5)$$

Where σ_{c50} is the strength for rock samples with a 50mm diameter, and D is the actual diameter in mm of the rock sample being tested. The strength is ascertained through the application of field pressing and laboratory testing of large samples, yielding conversion relationships. Thus, the drawback of the PLT is that the UCS is measured indirectly from the point load index (Šancer *et al.*, 2022).

The fundamental process for testing and calculating the point load index was established in 1985 by the International Society of Rock Mechanics (ISRM). Axial, block, or lump, and diametral are the three basic types of point load testing. On rock core samples with a diameter ranging from 50 to 55 mm, the axial and diametral tests are performed. The longitudinal axis of the core is loaded in the axial test, whereas the latitudinal axis of the core as shown in Figure 2.8, is loaded in the diametral test.

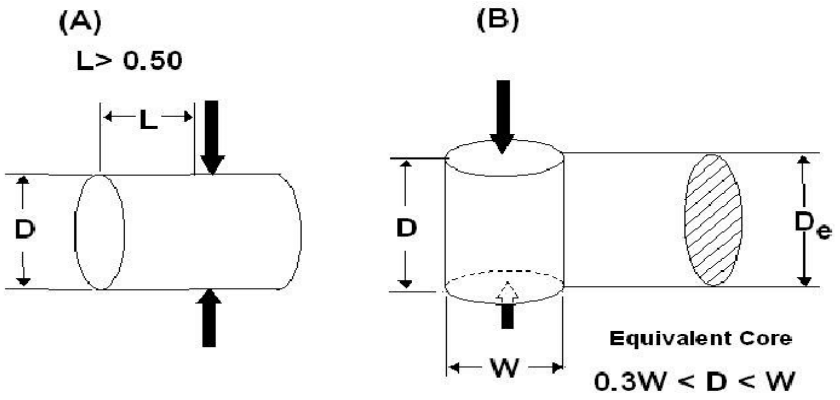


Figure 2.8: Diametral-and-axial-point-load-tests (Mark and Molinda, 2015)

The point load test yields the uncorrected point load index (Is), which is subsequently adjusted to account for a standard diameter (De) of 50mm unless the core being tested is approximately 50mm in diameter. The point load index (Is) can be calculated using

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a specific equation that considers the required load to fracture the core and its corresponding diameter.

$$I_{S(50)} = \frac{P}{De^2} \quad (6)$$

The failure load P represents the product of pressure and piston area while De stands for equivalent core diameter. The ratio of P to De^2 determines the point load index (Is). The relationship between UCS (σc) and Is depends on sample diameter as UCS (σc) = 24 x Is for D_{50} and UCS (σc) = (14 + 0.175D), Is for all other D values. According to Hoek (1977), the accuracy of the point load test (PLT) in predicting the UCS is consequently determined by the ratio of the UCS to the tensile strength. Most brittle rock types have a ratio of about 10. Whereas soft rock types such as mud and claystone, have a ratio of approximately to 5.

2.10.4 Triaxial Compression Test

Triaxial compression is described as the state of the body under the impact of compressive force or stress applied in three directions as indicated in Figure 2.9. In an underground mining environment, the rock surrounding excavations or mine workings is compressed triaxially rather than uniaxially. Triaxial compression tests are conducted to replicate the stress conditions that occur deep within the earth, far from underground excavations, allowing researchers to study the behaviour of rock under confining pressure, where σ1 > σ2 = σ3.

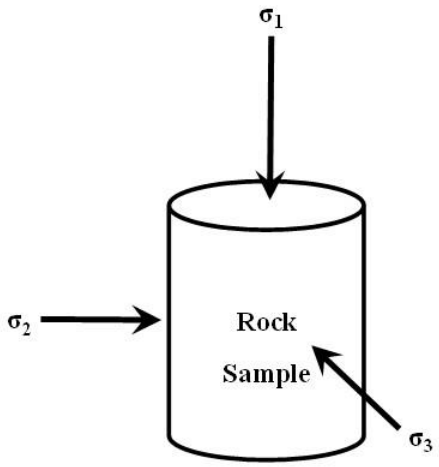


Figure 2.9: Triaxial compression of the rock sample (Fjaer *et.al*, 2021)

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In a triaxial compression test, a core sample is enclosed in a sealed pressure cell fitted with a latex jacket, as illustrated in Figure 2.10. Confining stress is applied by pressurizing the oil in the hydraulic cell, while strain is measured similarly to the UCS test. The confining stress is maintained at a constant level, and the axial stress is gradually increased until the sample fails at its peak strength. Subsequent deformation is achieved by reducing the stress.

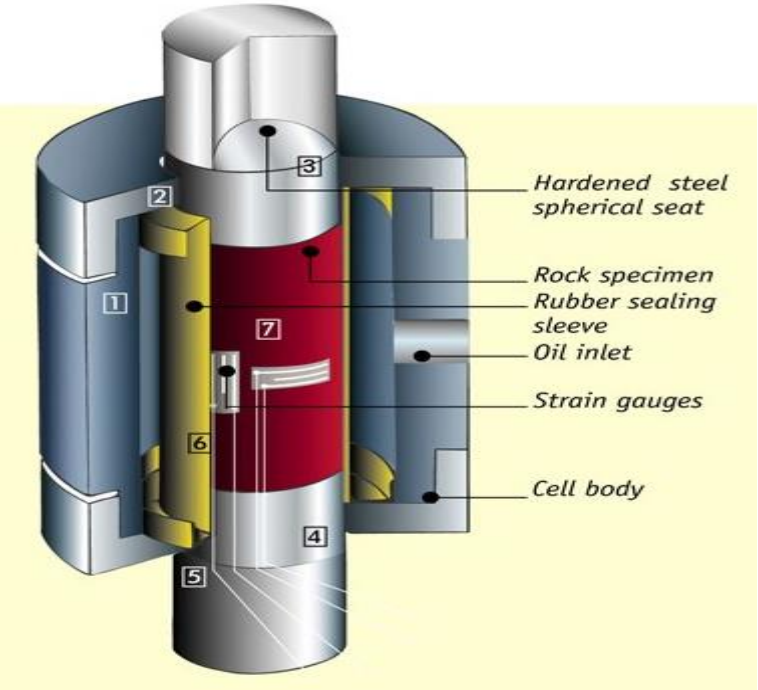


Figure 2.10: Equipment for confined tensile rock testing (adapted from Ramsey and Chester, 2004).

2.10.5 Brazilian or Diametral Compression Test (indirect tension test)

The Brazilian known as the diametral compression test was created in response to the challenges of doing direct uniaxial tension testing. The diametral compression is used to create tensile stresses in a thin disc of rock (dogbone specimen) loaded throughout its diameter illustrated in Figure 2.11.

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Figure 2.11: Brazilian tensile test configuration (Rocscience, 2023)

In the determination of the tensile strength using the Brazilian test method, a core disc sample is placed in the testing equipment, and the diameter, and thickness of the core disc varies depending on the purpose of the experiment. The core disc specimen is placed across its diameter and subjected to compressive load until it fractures (Al-Allaq, *et al*., 2021). Therefore, diametral tensile strength is calculated using the following equation:

$$\sigma_t = \frac{2P}{\pi DT} \tag{7}$$

P represents the applied load while *D* indicates core diameter, *t* indicates core thickness, and *T* represents rock specimen tensile strength.

2.10.6 Rock Strength Findings

Several researchers used rock samples of varying sizes (diameter and thickness) to evaluate the strength of the rock mass. Mehranpour and Kulartilate (2016) employed a 160mm rock core sample for the uniaxial and triaxial tests, as well as 50mm and 20mm core samples for the Brazilian test. Al-Allaq *et al.* (2021) used core samples with a diameter of 14.75mm and a thickness of 7mm to calculate the tensile strength of rock mass. However, Burbaum (2023) employed 44mm and 45mm samples, with length-to-diameter ratios of 2.00 and 2.08, respectively. It was discovered that reducing sample size increased Young's modulus, a rock sample's strength, by raising stress rates (Hoek and Brown, 1997; Potyondy and Cundall, 2004; Mehranpour and Kulartilate, 2016; Al-Allaq, 2021; and Burbaum, 2023). Table 2.7 summarises the findings of hardrock strength.

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Table 2.7: Hardrock strength subjected to compressive load

Rock type	UCS (MPa)	Reference
Conglomerate	160.0	(Edelbro, 2006)
Sandstone	80.4-210	(John,1972, and Edelbro, 2006)
Quartzite	227.0	(Komurlu, 2018, and Zhu, 2019)
Dolerite	327.0	(John,1972)

2.11 Rock Strength Failure Criterion

Several rock strength failure criterions determine the intact rock strength under the triaxial stress situation (deep excavations) by considering the three stresses such as σ1, σ2, and σ3. The most popular rock strength failure criterion utilised in this study to analyse rock strength under triaxial stress conditions are Mohr-Coulomb (linear envelope) and Hoek-brown strength failure criterion (nonlinear envelope).

The Mohr-Coulomb failure criterion was the first one known in rock mechanics. Griffiths (1990) implies that failure occurs when the shear force on a given plane approaches its shear strength. The majority of geotechnical software' are constructed in terms of the Mohr-Coulomb failure criterion, such as the Rocscience software suite (Rocslide, Rocplane, dip, phase, Roclab, and Rocdata). This criterion is defined by the following equation:

$$\tau_{Max} = C + \tan \varphi \sigma_n \quad (8)$$

The equation describes the relationship between shear strength (*τmax*) and normal stress (*σn*), incorporating two key rock properties: cohesion (*C*) and the angle of internal friction (*φ*). The Mohr-Coulomb failure criterion has several limitations. The criterion is based on laboratory sample tests only and not on true rock behaviour, and it overestimates the strength of in situ rock as a result of discontinuities. Lastly, the curvature of the confinement graph often displays a visible downward curvature.

The Hoek-Brown criterion was designed to overcome the shortcomings of the Mohr-Coulomb criterion (Hoek and Brown, 1980). This criterion was originally designed to assess the strength of a rock mass. In this regard, the Hoek-Brown criterion was developed to address shear failure in rock. Despite its original intent, the Hoek-Brown criterion has been broadly applied to various types of rock masses, including those of very poor quality, largely due to the absence of more suitable alternatives. The Hoek-Brown failure criterion is totally empirical, and Gibson (2006) endorsed it as a well-

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developed rock mass strength failure criterion that uses Hoek and Marion's (2002) geotechnical strength index (GSI) as input. The Generalised Hoek-Brown failure criterion for jointed rock masses is defined as follows:

$$\sigma_1 = \sigma_3 + \sigma_{Ci}\left(m_b \frac{\sigma_3}{\sigma_{Ci}} + S\right)^a \quad (9)$$

The σ_1 and σ_3 represent the major and minor principal stresses respectively. The constants m_b (ranging from 0.5 to 5) and s are rock-type dependent, with s equal to 1 for intact rock. The UCS of the rock sample is also represented by σ_{ci} in equation 9.

2.12 Stability and Rock Support for Underground Excavations

Support systems are an important safety component in all underground mines. Ground supports are used to increase the stability of underground excavations or openings. A stable excavation can last for a long time without failing. The stability of rock excavation is highly influenced by its quality, stress, groundwater, size, geometry, and orientation. It is further influenced by rock mass features such as unstable blocks, weak zones, and deterioration caused by water exposure. Ground instability causes bock failure and stress failure when produced and virgin stress exceeds the rock mass's strength (Hudson, 1989). These are referred to as stress and structural failure modes. Furthermore, mining-induced seismicity exacerbates and causes failure in the underground mining environment (Potvin and Hadjigeorgious, 2020).

Since Roman times, underground excavations have been stabilized by rock or ground support. In underground hard rock mining, numerous approaches are used to recover a wide range of ore deposits safely. Several innovative support systems for hard rock applications have been developed as a result of technological advancements and the experience of rock, geotechnical, and mining engineers. These include free-standing supports, pre-stressing devices, and upgraded cribs.

2.12.1 Classification of Support System

The classification of the support system is based on the method of stabilizing the rock mass around the excavation. The support classification is divided into two categories. Firstly, external support (timber prop, mechanical prop, and hydraulic prop) essentially provides surface restraint to the rock mass installed on the excavation boundary.

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Secondly, rock reinforcement (tendon support) modifies the internal behaviour of the rock mass installed within the rock mass. The support unit is further classified as either active or passive support (Masethe, 2015). Brady and Brown (1993) defined active support as a support unit that exerts a force or load and resists rockwall displacement immediately after installation, and passive support develops load when the rock mass surrounding the excavation dilates or deforms as classified in Table 2.8.

Table 2.8: Active and passive support units and the conditions they best operate in (Modified after Deanke *et.al*, 1998, Jager and Ryder, 1999).

Support units	**Static condition**	**Dynamic condition**
Active support		
End-anchored bolt.	x	
Pre-stressed pack	x	
Rapid-yielding hydraulic prop		x
Crush pillar.		x
Rock prop		x
Passive support		
Grouted ripple bar		
Shotcrete	x	x
Mesh and lacing.		
Cemented backfill.	x	
Timber sets	x	
x indicates the support unit performs well in static or dynamic conditions		

Daehnk *et al.* (2001) indicated that the instope support system typically consists of packs, props, and backfill used in underground hardrock mining such as gold, platinum, and chrome mines in South Africa to reduce the risk of fall of ground and rockburst. However, Stacey (2001b), argues that underground excavations commonly depend on wire mesh, shotcrete, and shotcrete reinforced with different fibres to mitigate the fall of ground. Hence, DMR (2017), outlined recommended support units or systems used in the stope and development end to maximize the stability of these excavations shown in Table 2.9.

Table 2.9: Instope support and development support (DMR, 2017)

Stope support	**Development support**
Packs	Roof bolts
Stick support	Packs
Elongate	Steel props
Roof bolts	Timber and steel sets
Pillars	Rope lacing and meshing
Backfill	Mechanical anchors
Mechanical props	Mechanical props

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2.12.2 Selection and Design of Support System

The identification of rock mass behaviour and potential failure modes is crucial for selecting and designing an adequate support system. According to Potvin and Hadjigeorgious (2020), support design necessitates an awareness of the interaction between the geometry of the excavation, surrounding rock mass, stress region, and the support system's strength and displacement capability. The relevance of ground support is to withstand deformation of the by stress changes and mining activities. However, there is no distinct approach to ground support design. Various approaches were used, and are still used today with minor variations, to estimate rock support requirements for support design. The evaluation of support requirements is heavily reliant on observations, personal judgment, and experience. In general, empirical, analytical, and numerical methods are most commonly employed to determine the support unit or system. These support evaluation techniques need accurate knowledge of the strength properties of unfractured rocks.

2.12.2.1 An Empirical Approach for Support Determination

Support systems in South Africa have been designed using empirical analysis, based on rock mass rating and excavation span. Rock mass classification systems are crucial for determining excavation stability and selecting support units. In mining, rock mass rating (RMR), Q-systems, and mining rock mass rating (MRMR) are used for empirical designs, without relying on performance data for successful design.

Support Determination using Rock Mass Rating (RMR)

Bieniawski (1989) proposed that the RMR value can used to determine the appropriate underground excavation support system, including rock bolts, wire mesh, steel rib, and shotcrete. The RMR classification also determines the stand-up time of the rock mass before the support system is installed shown in Figure 2.12 (Bieniawski, 1989). This is established by examining the correlation between the RMR value and the unsupported roof span, which is the distance between the excavated surface and the previously existing support system (Rochmadi and Karnawati, 2023).

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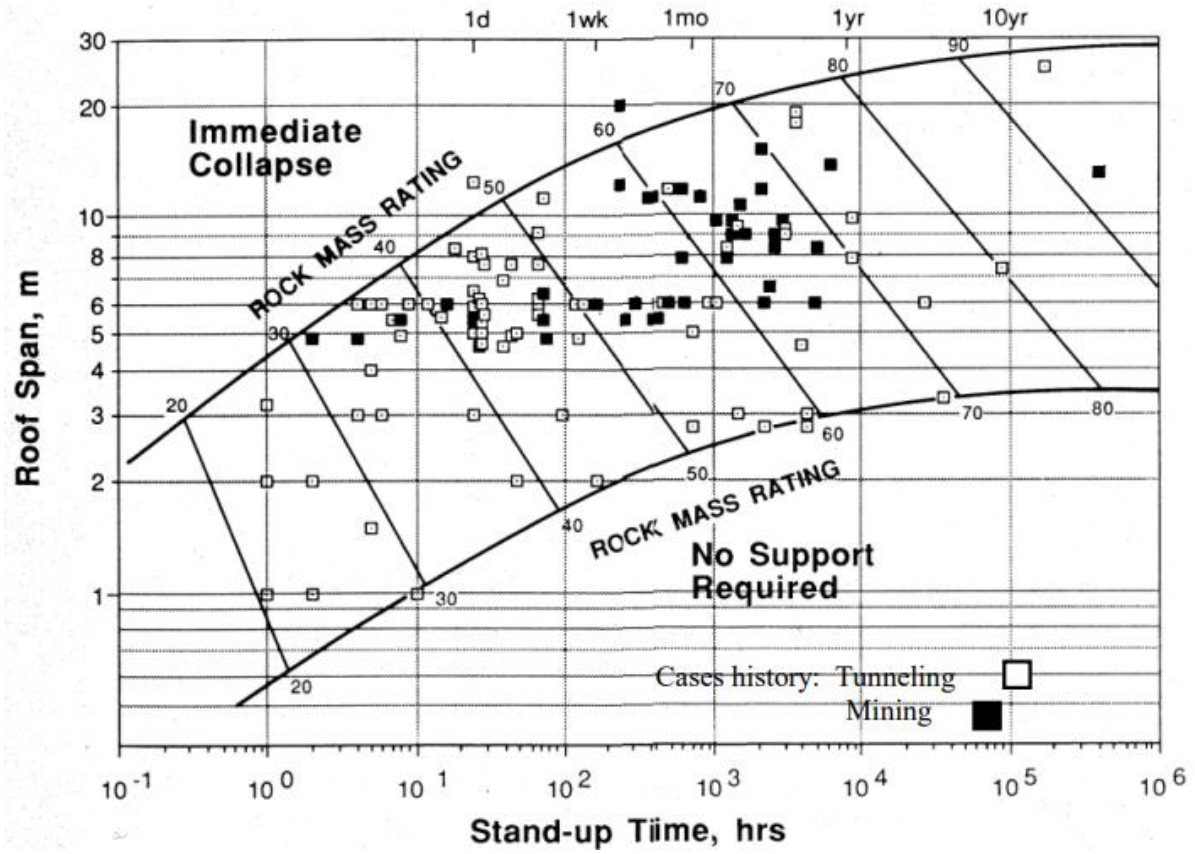


Figure 2.12: Relationship between Stand-up time, span, and RMR classification (after Bieniawski (1989)

Support Determination using Q-system

In determining the support system based on the Q-system, Barton *et al* (1974) indicated that additional geotechnical parameters such as the equivalent dimension of the excavation (De) are required. The De value is calculated by dividing the diameter, span, or height wall of the excavation by the excavation support ratio (ESR).

$$\text{De} = \frac{\text{height of the tunnel}}{\text{Excavation support ratio ESR}} \qquad (10)$$

Barton et al. (1974) recommend an ESR value of 1 for tunnels, which is based on the type of excavation. Figure 2.13 illustrates the Q-support chart based on the tunnelling quality index (Q-system classification).

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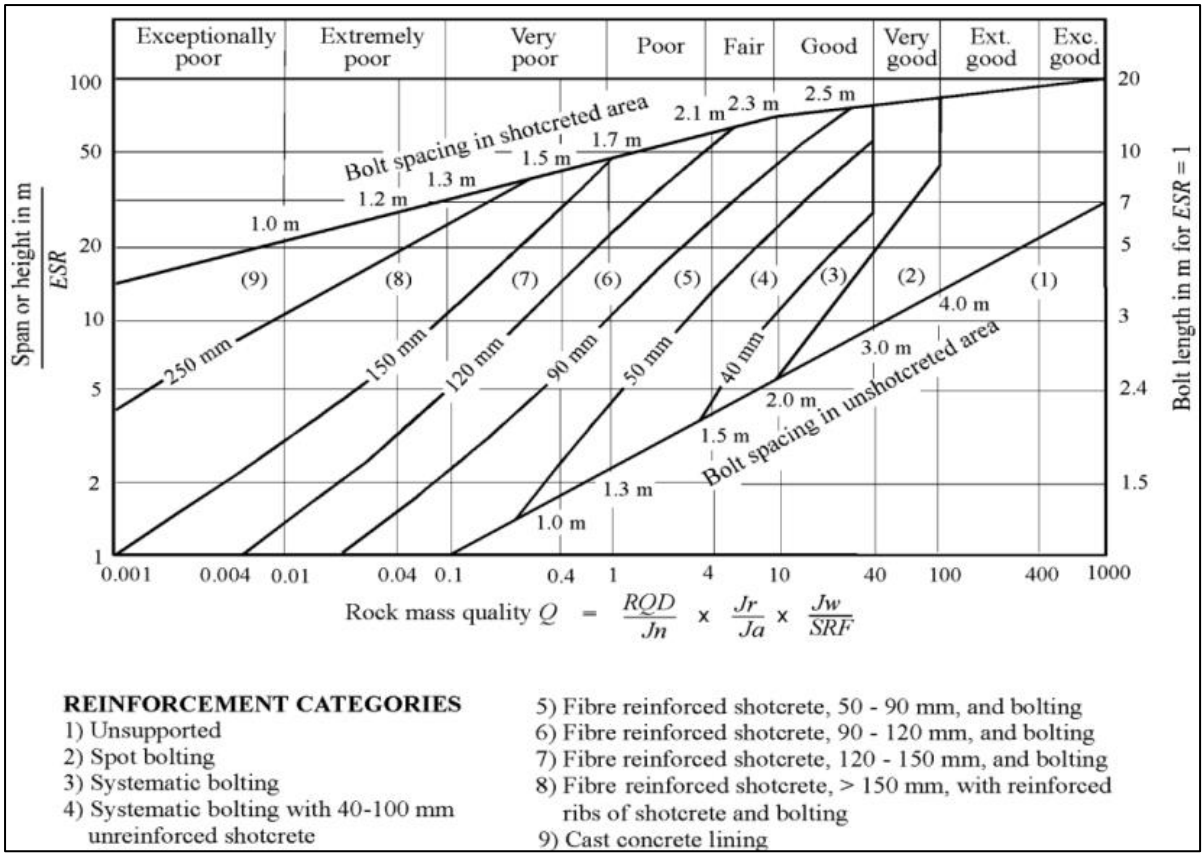


Figure 2.13: Q-system support categories (Grimstad and Barton, 1993)

According to Palmström and Broch (2006), the Q-system has limitations when applied to extremely poor or exceptionally challenging ground conditions. This is because poor ground conditions are difficult to classify and describe. A small span excavation of 2-3m Q-system can be carried out; however, in a wide span of 30-40m, the design should not rely solely on classification systems. In practice, most of the concerns mentioned by Palmström and Broch (2006) are relevant to most classification systems. Despite their shortcomings, classification systems are quite effective in the early stages of design for identifying geotechnical areas of similar rock mass and developing ground support schemes.

2.12.2.2 Analytical and numerical approach for support determination

Analytical limit equilibrium approaches need additional knowledge about the support demand and capacity to evaluate the performance of the support units. Typically, ground support demand is calculated using the overburden weight of failing wedges or ground. The deadweight is compared to the support capability. When adopting the limit equilibrium approach, comprehensive tests or assessments are undertaken to

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evaluate the load-bearing capacities of the support units. However, adopting the limit equilibrium technique for dynamic support is challenging since it is impossible to predict both the dynamic capacity and demand of the support system simultaneously.. Alternatively, numerical approaches can be used to model ground support. Non-explicit design involves comparing support capacity to the weight of a potential failed zone, while explicit modelling requires detailed data on ground support element performance.

2.12.3 Support Requirement and Performance

Jager and Ryder (1999) suggested that to adequately support the underground excavations, important factors such as potential fallout thickness (support resistance and energy absorption requirement), closure rate (stiffness and yield ability requirement of the support unit), and the block size of the unstable hangingwall (support spacing and area coverage requirement) must be considered summarised in Figure 2.14.

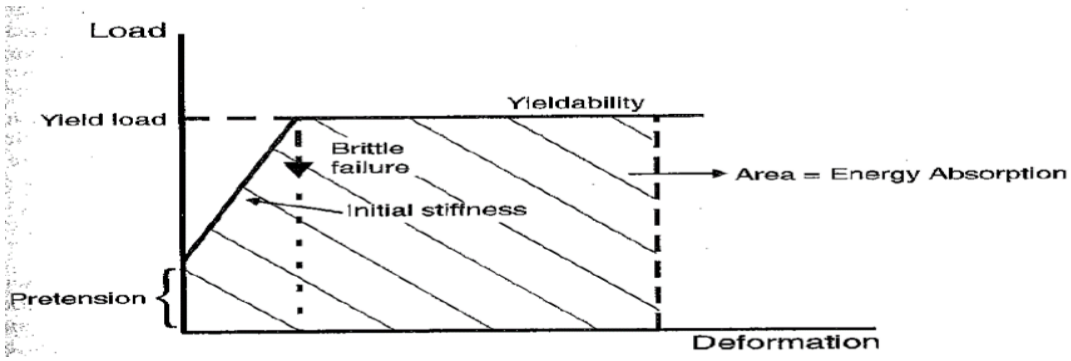


Figure 2.14: Key characteristics of the support element

The primary goal of the support system is to keep the shattered hangingwall beam intact, preventing loose rock falls for a set amount of time. Support units apply sufficient force/load to the hangingwall, directly support loose rocks, and limit bed separation. The type of the generated force/load is largely influenced by the local ground conditions. Thus, support systems are built and selected while delineating GCDs in every mining environment.

In shallow stoping conditions, support systems must provide resistance to deformation at low closure rates, particularly for both hangingwall and footwall. In contrast, deep stoping conditions demand support that rapidly exerts clamping forces on the fractured

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hangingwall, preventing the destabilization of unstable blocks (Gay and Jager, 1987; Roberts *et al*., 1993). The presence of weak strata in the footwall is considered when they allow punching of support units, yielding of instope pillars, or cause heaves with abnormally high rates of closure. To prevent loose rocks from falling into the mine workings, Jager and Robert (1986), argue that a support system must be able to supply a good area coverage. When discontinuities define unstable keyblocks with unfavourable orientations, support spacing should be reduced to be smaller than the dimensions of the smallest critical keyblock.

The stability of timber props (150-200mm diameter) and packs decreases as the height and closure amount increase (Dianke *et al.,*2001),. According to results obtained from laboratory compression tests of timber props, the average yielding strength was found to be 22.7MPa. the compressive test also indicates a peak load capacity of 260kN before the onset of buckling failure. The timber pack with a height-to-width ratio greater than 2:1 is generally considered to be unstable. Kujoana (2009) in the “quest to effective pack support” study, proved that the larger H:W (height to width ratio) greater than 2:1 can be applied with a low convergence. Therefore, the H:W of 2:1 should be given as a minimum. Hydraulic prop blast-out rates increase with longer prop lengths, specifically when the stope width is greater than 1.5m. However, Robert (1995) conducted tests on mechanical props to determine their buckling potential and energy absorption capacity as a function of increasing length. It was found that between the length of 2.5m and 2.7m props failed by buckling load below 120kN.

Wagner (1995) suggested that mechanical anchors should be deployed only when rock strata reach a uniaxial compressive strength higher than 50MPa due to the substantial contact stresses that occur at anchor ends. The mechanical anchor performance evaluation requires an analysis of the equilibrium forces operating on its system components (Windsor and Thompson 1997). According to Van der Merwe and Madden (2000), the long free span of steel tendons enables mechanical anchor bolts to elongate until they reach slip levels between 30 to 70kN. Different from resin anchors installation requires additional focus and extra time to complete it than mechanical anchors. When the bonded length remains below 0.61, the total load inside the resin stays lower than the steel material strength as determined by Mark (2000).

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2.12.3.1 Tributary Area Theory, Support Resistance, and Energy Absorption

In the past, support system design relied on established practices, experiential knowledge, and budgetary constraints. In South African platinum, gold, and chrome mines, the current support design methodology is primarily based on the tributary area theory (Daehnk *et al*., 2001). This theory assumes that the weight of the overburden is distributed among support units or pillars in proportion to the surrounding excavated area. The tributary area theory calculates the weight of the rock mass based on the reef plane and potential fall height, then distributes this weight among a fixed number of support units according to their attributed area. This area is determined by the face layout and fall-out height. However, a significant limitation of this theory is that it fails to account for the natural jointing or fracturing that typically occurs in the rock being supported.

The height of potential rock mass instability is a key parameter that influences the design of the support system. The desired strength for rockfall support systems alongside their energy absorption capacity depends primarily on the thickness of rocks expected to fall. As defined by Jager and Ryder (1999), support resistance (SR) is calculated as the force (*F*) applied by a support unit, divided by the tributary area (*A*) of the hangingwall it supports, resulting in a pressure value expressed in kN/m². For a pure rockfall control support resistance of a support unit must be sufficient to carry the deadweight strata involved. Therefore, the following equation is used to determine support resistance in a static condition which is required to be 50kN/m^2:

$$F/A = \rho g t \quad (11)$$

F represents the weight exerted on the support unit (N), *A* is a tributary area (m^2), *ρ* is the density of rock (typically assumed to be 2750 kg/m³ for quartzite, *g* is a gravitational force (9.8 -10 m/s^2), and *t* is the height of fall (m). Robert (1995) determined the fall-out thicknesses for the various reefs at a 95% frequency level using a compressive accident analysis of all rock-related fatalities of the gold mines since 1990 detailed in Table 2.10.

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Table 2.10. Fallout thickness for the various reefs at 95 % frequency level (Modified after Robert,1995, and Daehnk, 2001)

Reef type	Rockfall	
	Fall-out thickness (m)	Support resistance (kN/m^2)
Basal reef	1.8	47.7
Vaal reef	1.2	31.8
VCR	1.2	31.8
Carbon leader reef	1.0	27.7

The energy absorption (*E*) idea is based on the principle that during dynamic deformation of the hangingwall, kinetic energy is imparted to an ejected block of rock, which must be absorbed by the support units without undergoing undue deformation. In the case of the seismic and rockburst (required SR for the dynamic conditions 200kN/m^2) prone area, energy absorption criteria are calculated based on the initial hangingwall velocity of 3m/s (Daehnk *et al.*,1998). The hangingwall needs to be arrested within 0.2m (Robert, 1995). The results of underground observations, seismic measurements, rockburst back analysis, and numerical analysis simulations have indicated that the hangingwall can accelerate to a velocity of about 3m/s. The 1m/s rapid hydraulic prop has been observed to occasionally punch into the hanging and footwall during seismic loading. This indicates that the prop did not yield rapidly enough because the rockburst velocity exceeded 1m/s. Thus, the total energy absorbed by the support system is calculated using the following equation:

$$E = \frac{1}{2}mv^2 + mgh \qquad (12)$$

The energy absorption capacity of the support system can be calculated using the formula *E*, which takes into account the mass of the hangingwall (m), its initial velocity (v, assumed to be 3m/s), and its downward displacement (h, assumed to be 0.2m). Table 2.11 presents details about the required specific energy absorption levels for different reefs.

Table 2.11: Ejection thickness and energy absorption criteria for various reefs at 95% frequency level (Modified after Robert, 1995)

Reef type	Rockburst	
	Ejected thickness (m)	Energy absorption (kg/m^2)
Basal reef	2.6	45.4
Vaal reef	1.2	20.9
VCR	1.8	31.4
Carbon leader reef	2.2	38.4

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2.13 Site Investigation Procedure and Data Collection

According to Stacy and Swart (2001), ensuring excavation stability and optimal ore deposit recovery requires a phased approach to investigation, spanning from the pre-feasibility stage to full production. Various methodological approaches are employed to address strata condition issues, driven by the project's scope. Initially, a desk study is conducted to gather essential information, followed by fieldwork and laboratory testing. Researchers have successfully combined extensive desk studies, underground mapping, geological and geotechnical logging, geophysical analysis, and laboratory tests to gain comprehensive insights into the geological and strata conditions of the area under investigation (NGR, 2000; Stacy and Swart, 2001; Dupreez, 2018; Russell, 2018).

Russell (2018) conducted a study to enhance geotechnical data confidence at the Sishen Iron Ore Mine by integrating laser scanner face mapping data into the mine's geotechnical database. The researcher employed a multi-faceted approach, combining face mapping, core logging (geotechnical logging), and laboratory tests to gather comprehensive information on intact rock properties, discontinuity properties, structural orientation, and rock mass properties. Russell (2018) emphasizes that face mapping is a straightforward yet invaluable method for acquiring geotechnical and geological data, particularly when compared to other digital techniques such as remote face mapping. Recent advancements in technology have further expanded the possibilities for data collection in underground settings. According to Ryan (2020), using unmanned aerial vehicles (UAVs), and portable sensors enables the safe and efficient collection of high-quality data from subsurface or underground workings.

2.13.1 Desk Study

During the initial stage of an investigation, desk research plays a crucial role in gathering essential information. A significant amount of time is dedicated to collecting, examining, and studying pertinent background materials. These materials encompass geological and topographical maps, geotechnical and geological reports, and aerial photographs. This approach is the most cost-effective and convenient means of obtaining firsthand information. Aerial photographs, in particular, prove invaluable in detecting geological weak zones at an early stage. When viewed in stereoscope, these weaknesses become more pronounced. Geological structures such as gullies,

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depressions in the ground, and trenches become apparent due to the exaggerated scale. As noted by NGR (2000), aerial photographs provide critical information for planning and design, making them a vital component of the early investigation stage.

2.13.2 Measurements of Geological Structures

To comprehend and quantify the effect of discontinuities on rock mass behaviour, meticulous measurements are taken. These measurements are crucial for investigating geological structures, classifying rock masses, and providing essential input data for various numerical, empirical, and analytical models. These models encompass a range of applications, including stability of the rock mass, blasting, fluid flow, rock cutting, rock deformation, and the design of a support system. Two primary sampling strategies have been employed to measure structural features: logging of boreholes and examination of exposed faces through face mapping. These approaches enable researchers to gather valuable data on discontinuities, ultimately informing a deeper understanding of rock mass behaviour.

2.13.2.1 Borehole Sampling

Borehole sampling, commonly known as core logging, has been a widely used technique for many years. The primary purpose of diamond drilling is to assess the size of ore deposits and adjacent strata. However, diamond drilling also plays a crucial role in identifying rock types, mineralogy, and conducting geotechnical investigations. Furthermore, the collected rock core samples and borehole sidewalls can be subjected to mechanical and fluid flow studies (Snow, 1970). Remote visual inspection methods can also be employed to examine the borehole walls (Kamewada et al., 1990), while geophysical equipment can be utilised to explore the surrounding geology.

Rosengren (1970 and Geological Society (1970)) offer thorough instructions for logging rock cores for engineering purposes. The recommended core logging procedure involves the removal of the core protecting cover, providing the recovered core with the borehole Identification (ID) and marking the end depth, and taking a colour photograph. Subsequent steps include rock strength testing, other damaging field tests, splitting the cores for additional photographing, and reassembling and re-sealing the core for future reference.

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2.13.2.2 Mapping of the Exposed Face

Rock face mapping involves the direct collection of data from the exposed face of a rock mass, encompassing geological, structural, and geotechnical mapping. Geotechnical mapping, in particular, plays a crucial role in rock engineering and geotechnical engineering, as it provides valuable data on the condition and the quality of rock masses. Mapping data include information on rock type, weathering, the strength of the intact rock mass, weathering, and discontinuity characteristics, such as persistence, spacing, orientation, and state (Read *et al*., 2009). Structural face mapping is a specialized approach that focuses on collecting information on the discontinuities present in the exposed rock mass face. This information is essential for kinematic analysis, which predicts potential failure modes, and numerical analysis. Various techniques are employed in structural face mapping, including scanline mapping, window or cell mapping, and digital mapping using laser scanning and photogrammetry (McQuillan, 2013).

I) Manual Face Mapping

Manual face mapping involves the direct collection of geological, structural, and geotechnical data from the exposed rock face, requiring physical contact between the operator and the face. The process includes two fundamental manual methods known as scanline and window mapping techniques.

Scanline mapping is a geotechnical method used to gather data on exposed faces. This involves designating a horizontal or vertical direction, known as a scanline, along which geotechnical data is captured using a tape measure placed on the face. Scanlines can extend up to 30m, while smaller distances can be delineated using sprayed paint. The geological features intersecting the scanline are meticulously documented, including the position, orientation, persistence, and circumstances of discontinuities (Simanguson *et al*., 2004).

Window mapping is regarded as an alternative method that divides a face into cells. The location and attributes of each face are measured or mapped within the designated window cell. Within each cell, discontinuity sets are identified, and feature attributes such as orientation, spacing, and persistence are quantified (Call, 1992; and Priest, 1993).

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Advantages and Disadvantages of Manual Face Mapping

Exposed rock face measurements offer a unique advantage in that they enable the direct measurement of geological structure attributes across a vast area of rock. This approach also facilitates the observation of geological relationships between different discontinuity groups. However, there are notable drawbacks to this method. The rock face is frequently positioned far from the point of attraction and is vulnerable to deterioration from blasting, plant development, and weathering. Although competent people are required to collect these measurements, the expense associated with labour and equipment is comparatively minimal in comparison to those incurred by borehole sampling. Historically, measurements at exposed faces were conducted randomly and subjectively using geological mapping methods, yielding insufficient quantitative information for engineering design.

Manual rock face mapping is a time-consuming, labour-intensive process that requires physical contact with the rock face (Gumede, 2005). In underground mining environments, access to the rock face is often limited, and geologists or rock engineers conducting mapping are exposed to rockfall hazards. Moreover, manual face mapping cannot provide a comprehensive representation of the rock mass. It can be challenging to distinguish between natural discontinuities and blast-induced fracturing on the mapping face. Window or cell mapping techniques are also limited, as they only measure discontinuities within the designated window cell. Consequently, manual rock face mapping is subject to biases related to orientation, size, truncation, and censoring (Zhan, 2006; Wines and Lilly, 2001).

II) Digital Face Mapping

Digital mapping is a sophisticated technology that allows for the digital mapping of rock faces using cutting-edge technologies such as 3D photogrammetry (SiroVision system) and laser scanning (Maptek system) or Lidar scanning. 3D photogrammetry creates a face point cloud through the use of stereo photographs (Reid and Harrison, 2000; Little, 2006). This process involves capturing a series of overlapping photographs, which are then used to create a 3D mosaic. Laser scanning as an example of digital mapping, offers a fast and efficient means of data collection. It captures hundreds of thousands of points per second at great resolution, providing precise information about the rock face. Typically, scan points provide X, Y, and Z

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coordinates as well as intensity data. By incorporating geotags, this technique enables accurate 3D reconstruction of the rock face in space (Slob and Hack, 2007). Automated system scanners examine the orientation patterns to extract useful relational information from the mapping face. In contrast, digital photogrammetry technology creates 3D digital face representations that software uses to determine orientation measurements and length and area dimensions as well as planes of identical orientation (Maptek, 2013).

Advantages and Disadvantages of Digital Face Mapping

Digital face mapping techniques offer a faster and more efficient means of data collection compared to traditional methods. This technology accelerates data processing, reduces software system demands, and provides a higher level of accuracy for measuring discontinuities. Moreover, digital face mapping eliminates the risk of rockfall exposure for personnel, as it does not require direct physical contact with the face. However, the equipment and software applications employed in digital face mapping are costly. Additionally, the operation and data analysis requires highly skilled workers or professionals to interpret the results and draw meaningful conclusions.

2.14 Study Gaps

Literature review identifies major developments in the field of rock mechanics and strata control in the mining industry. Nonetheless, despite these changes, there are still some knowledge gap that still require future research. In particular, the Emerald Shaft Gold Mine has its own peculiarities, which demand site-specific studies to allow optimum design and layout of the mine, characterization of geological conditions, evaluation of the support system performance, and stipulate effective strata control measures.

One of the primary gaps in knowledge is the limited understanding of mine design and layout optimization for Emerald Shaft Gold Mine. Although there are general principles and guidelines (Hoek & Brown, 1980; Bieniawski, 1989), the geology and geotechnical specifics of this mine necessitate a special research to achieve safe and efficient mining operations. Furthermore, it is essential to describe the geological conditions and their influence on the behaviour of rock strata to make the right decisions about

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design and support systems of the mine. The characterisation of geological conditions at Emerald Shaft Gold Mine is however required in terms of the influence of geological structures, rock types and alteration zones, on rock mass behaviour.

To comprehend rock or strata conditions in mining environments, various techniques have been employed. The literature review reveals that geologists typically utilise descriptive methods, whereas rock engineers employ analytical or empirical methods to investigate geological structures or discontinuities. These studies are primarily conducted to inform mine design, stability, support design, and cost-effectiveness. Initially, researchers attempted to establish the Rock Quality Designation (RQD) as a single, unified rock mass classification system (Guler et al., 1998). However, this technique proved insufficient, since the complex mining environment necessitates a more detailed description and measurement of the multiple aspects that influence underground excavation stability. The inhomogeneous and anisotropic nature of rock masses, combined with the varying concentration of stress fields (including both virgin and mining-induced stresses) surrounding underground excavations, necessitates ongoing, sound rock engineering practices for assessing rock mass stability.

The evaluation of support system performance is another area where knowledge gaps exist. Although several support systems have been designed and used in diverse mining settings (Wagner, 1995; Daehnk *et al*., 2001), little has been done to evaluate the performance of the support systems in the same type of mining setting, especially in underground gold mines, such as Emerald Shaft. This gap in knowledge demonstrates the necessity of carrying out site-specific studies on the design, installation, and monitoring of support systems (Potvin & Hadjigeorgiou, 2020). Rock mass classification systems, such as RMR (Bieniawski, 1989), Q-system (Barton *et al*., 1974), and GSI (Hoek *et al*., 1995), have been widely used in various mining environments to characterize rock mass behaviour and inform support design. Nevertheless, further studies are necessary in terms of the implementation of these systems to Emerald Shaft Gold Mine, as well as limitations and possible biases.

By addressing these knowledge gaps, this study aims to fill these gaps in knowledge to contribute towards the emergence of more efficient and effective gold mining practices at Emerald Shaft Gold Mine. The study will analyse the mine design and layout, characterize geological conditions, evaluate support system performance, and

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recommend appropriate strategies for strata control. By providing a comprehensive understanding of the geological and geotechnical conditions of the mine, this study will inform mine design and support system decisions, ultimately enhancing the safety and productivity of the mine.

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CHAPTER THREE: RESEARCH METHODOLOGY

The methodology involves the established procedures for recognising and obtaining information about the selected topic (Wilkinson and Birmingham, 2003). The understanding of the strata conditions and ground conditions of the underground mining environment (at 1 level of Emerald Shaft Gold Mine) requires a clear and detailed description of the methods, procedures, or techniques adopted for data collection and analysis. In this regard, a quantitative approach design was adopted to achieve study objectives and answer study questions. The methodology is summarised in Figure 3.1.

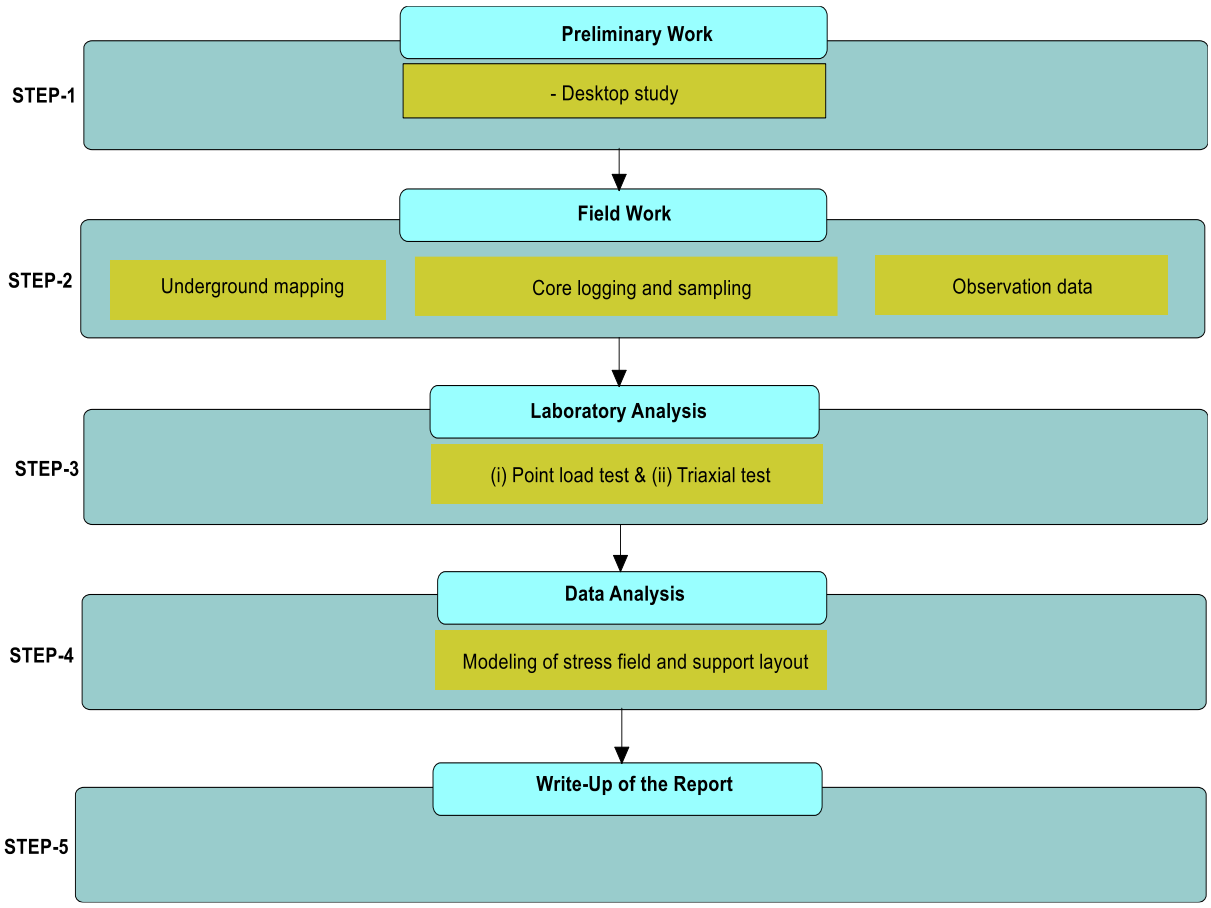


Figure 3.1: Summary of the methodology used in this study

3.1 Stages of the Study

This study was divided into three stages: preliminary study, fieldwork (field survey), and laboratory work to achieve the objectives of the study and answering the study questions. The preliminary study includes desktop studies and reconnaissance surveys. The desktop studies were conducted to gather first-hand information about

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the study and the area under study. Information gathered as part of desktop studies was from books, journals, the internet, geological, geotechnical, engineering, survey data, and reports data sources. Fieldwork or field studies were conducted to collect onsite raw data outside the laboratory or library. Fieldwork includes site observations, site measurements or mapping, and sampling. This is usually followed by laboratory work to analyse the collected samples.

3.2 Data Collection Methods

The data collection methods are study procedures used to gather or collect data for the study. In this study, primary and secondary data collection were employed. The primary data was obtained from fieldwork studies whereas secondary data was obtained from desktop studies. The types of data were divided into two models namely surface and subsurface data. Both served as a variable in creating geological models, engineering geological models, empirical references, and categorization of rock mass quality. This was done to determine the mine design, underground support system, stand-up time, and determining appropriate mining methods. Surface data was collected from surface mapping. In contrast, subsurface or underground data was collected from underground rock mapping, diamond drilling geotechnical core logging, and underground observations including discontinuity measurement (discontinuity survey) to achieve the objective of the study.

The objectives of the study were attained through a different data collection methods. The analysis of the mine design was done using desktop studies and underground observations, where surface and subsurface information, such as mine plans, layouts, and geological models, were used to determine the possible geotechnical problems and provide the recommendations on strata control. The geological conditions were defined by surface mapping, underground rock mapping, diamond drilling and geotechnical core logging, which gave much information about the rock mass and which guided the design of the mines and the support systems to be used. Also, support systems were assessed by underground observation such as discontinuity measurement and monitoring of the support system performance to determine the effectiveness of the support system and the areas of improvement. Through the incorporation of all data gathered, evidence-based recommendations on strata control were formulated, which guaranteed safe and efficient mining operations and acted as

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the input into the design of the support system and mine planning. In this sense, the subsequent sub-sections explain the procedures employed in the data collection of the study.

3.2.1 Diamond drilling and geotechnical core logging

In this study, diamond drilling together with geotechnical core logging served as the fundamental methods for geotechnical data acquisition. The geotechnical core logging procedure was specially developed to capture the mechanical and structural properties of the rock masses. The geotechnical core logging procedure allowed for the gathering of data on a variety of rock characteristics and geotechnically critical key structures, such as joint surface condition, fracture frequency, fracture orientation, and rock strength. The logging procedure delivers an extensive system for gathering complete information about the rock mass structure of the Emerald Shaft Gold Mine. The logging method enables the identification of all significant rock mass classification systems including RMR, Q-system, and GSI. The logging system provides an accurate procedure to assess rock mass and structural conditions for understanding the strata conditions of the mine.

Prior to commencing diamond drilling, the location of each intended borehole was precisely recorded using GPS technology. The coordinate system, along with any associated errors, was carefully noted. To ensure accuracy, the GPS device was powered on and allowed to acquire satellite signals for at least 30 minutes to ensure maximum signal strength. Next, the device was set to the WGS84 datum, and the projection was set to latitude and longitude with decimal degrees. The precision of the device was tested by entering known coordinates obtained from ArcGIS and Google Earth which were cross-checked against its displayed results. Any disparities were compensated for, and the procedure was repeated until the readings of the device matched the known coordinates with an acceptable margin of error. The decimal degrees were then subsequently converted to UTM-zone coordinates (Northing and Easting) to facilitate 3D modelling. An illustration of the plan of location of the boreholes is presented in Figure 3.2.

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Figure 3.2: Geological and geotechnical borehole plan showing the location of the holes drilled

A total of eight boreholes, spaced 25 to 50m apart, were drilled from the surface elevation (ranging from 1734m to 1743m) to depths of 61.8m to 84.3m. The drilling of eight boreholes followed an assessment of area geological complexity and data precision demands together with drilling expenses and location considerations. Four historical boreholes which are the drill holes constructed in previous periods for exploration or mining operations and scientific research were analysed to provide additional context and valuable insights into the subsurface geology. EMRBH2 and 4 were drilled along the same strike line as EMRBH5. Borehole EMRBH6 was collared northeast of EMRBH7.

The drilled cores were grouped and placed in trays, with each sample clearly marked for identification. A color-coding system was used to indicate the depth of the borehole (red), sample metres (white), and sample demarcation and identity (yellow). Samples were subsequently transferred to the laboratory for processing. The sampling data was captured in borehole spreadsheets for use in modelling.

3.2.2 Rock face mapping and underground chip sampling

Rock face mapping involves the direct collection of data from exposed rock mass faces, encompassing geological, structural, and geotechnical mapping. This process

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is crucial for designing underground excavations, as it provides essential information for updating geological models, assessing rock mass characteristics, and interpreting geotechnical measurements. The primary objective of geological rock mapping is to gather data on rock mass types, geological structures, and geotechnical qualities, which is vital in identifying potential hazards, such as discontinuity planes that can separate the rock mass into unstable blocks (Wanna *et al*., 2014). The rock face mapping involves assessing various parameters, including rock type and weathering grade, orientation, spacing, persistence, roughness, wall strength, aperture, type of filling, and seepage. By evaluating these parameters, rock face mapping provides critical insights into the geological and geotechnical characteristics of the rock mass, ultimately informing design decisions and ensuring the safety and stability of underground excavations.

In this study, structural and geotechnical mapping were conducted to gather data on geological structures that could impact mine design and planning, as well as the thickness of exposed reef or channels. The quality and condition of the rock mass were also assessed during geotechnical mapping. Due to the unavailability of digital mapping equipment and software, manual rock face mapping (scanline mapping) was employed.

A 30m exposed rock face at 1 level (east and west panels) of the Emerald Shaft Gold Mine was mapped. To facilitate clear visualization of geological structures and discontinuities, the area of interest was watered down before mapping. The rock face was measured to be 30m, and a measuring tape was kept in place as a scanline. When discontinuities and lithological contacts were encountered, these features were mapped using a clinometer, tape measure, and geological hammer, as shown in Figure 3.3. The data was meticulously recorded in a field notebook and on mapping sheets. The mapping process was conducted after every stage of blasting, from the centre gully of the wide raise mining at survey peg K108 to the panels. Survey pegs were used for direction and to locate the panels. Consequently, underground survey data was converted to surface level (above sea level) for consistency. The collected mapping data was stored for data processing, analysis, and geological modelling.

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Figure 3.3: Underground face mapping of geological discontinuity

Underground chip sampling requires adherence to fundamental principles to ensure accurate and reliable results. These principles include ensuring the sample area is thoroughly clean, segregating and demarcating the sample area, measuring the sample width, and chipping the sample. To prepare the sample area, it was watered down with clean water to remove contaminants. The face was then marked with a white marker, indicating the length and width of the sample illustrated in Figure 3.4. Lines marking the length of the sample were drawn perpendicular (90°) to the channel or reef waste contact, spaced 10cm apart. Chipping of the sample was achieved using a geological hammer. The chipped samples were measured and kept between 8cm and 20cm in size. Each sample was then placed into one of thirty sample bags, accompanied by a sample ticket number. The samples were subsequently sent to the laboratory for analysis.

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Figure 3.4: An Illustration of face marking before chipping the sample.

3.2.3 Underground observation

Underground observation at Emerald Shaft Gold Mine involved a thorough visual examination of the hangingwall, sidewalls, footwall, and support system. This observation was conducted at three critical stages: during the early entry examination, throughout the shift, and at the end of the shift. The early entry examination was a critical process where underground workings were inspected and declared safe before the commencement of daily mining operations. At Emerald Shaft Gold Mine, the examination area was divided into three zones. Zone 1 spanned from survey peg K100 to K108, covering a distance of 25.984m. Zone 2 commenced at the end of the centre gully, extending to the stope, and was marked by survey pegs K100 to K102 and K108 to K110, spaced 5m apart. Zone 3, considered a danger zone, was located 10m from the advancing face at the stope, where production took place, and the area was susceptible to blast impacts. The entry examination procedure involved a series of critical checks, including testing for gases, air velocity, temperature, and ventilation. Additionally, the area was watered down to facilitate a clear examination of geological structures or discontinuities.

To monitor the accumulation of gases underground, a Gas Detecting Instrument (GDI) was used. The GDI is designed to detect a range of gases, including noxious,

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poisonous, and flammable gases. To ensure comprehensive coverage, the GDI was moved in a zig-zag pattern, testing the sidewalls, hangingwall, and footwall. In areas with a stoping width exceeding 2m, an extension stick was utilised to reach the highest points of the excavation, ensuring that all areas were thoroughly tested. The readings obtained from the GDI were subsequently compared against established standards to ensure a safe working environment.

To ensure adequate ventilation supply, air velocity tests were conducted. Measurements were taken 3m from the area where ventilation airflow entered the working area. A tape measure was stretched between two points, 3m apart, and a tester at the up-dip end dispersed white powder until it reached the other tester at the end of the tape. The arrival time of the dispersed powder was recorded by stopping a watch and logging in a record book. This measurement was repeated three times to ensure accuracy. The average air velocity was then calculated from these measurements. According to safety regulations, the minimum required air velocity at a working stope is 0.25 m/s.

The underground environment is very crucial in such a way that temperature is maintained and regulated at all costs. A whirling hygrometer was used to measure the temperature underground. The mechanical rotating handle was rotated for about 30 seconds and the instrument detected wet and dry bulb temperature. To comply with the mine health safety standard, wet and dry bulb temperatures were kept within 32.5ºC and 37ºC respectively.

Watering down the hangingwall, sidewalls, and footwall exposes geological structures and loosened rocks on these walls. During the examination, exposed geological structures are assessed, and loosened rocks are addressed through the barring down process, which involves using a pinch bar to remove loosened rocks from the hanging and sidewalls. The support unit or system is also examined, and the installation of support units (both permanent and temporary) is verified. This verification process includes visual observation for damage, measurement of support spacing measurement of distance from the working face (not exceeding 1m for temporary and 3m for permanent support). A clino-ruler is also used to ensure the correct installation angle (perpendicular to the foot and hangingwall) of 90º for the support units. The

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measurements receive analysis based on established benchmarks while accounting for ground conditions modifications.

3.3 Field and Laboratory Rock Mass Testing

The strength of the rock mass were determined through both field and laboratory assessments in this study. Testing was conducted on three types of samples: in-situ rock mass, small blocks of chipped rock samples, and drilled core samples. The results of these tests were analysed to delineate the stress field acting on the rock specimen, simulating the conditions of an underground excavation. This comprehensive approach enabled a thorough understanding of the rock mass strength and its behaviour under various stress conditions.

3.3.1 Field rock mass testing

Field rock mass testing was conducted to estimate the uniaxial compressive strength (UCS) of the rock mass. The impact blow and point load tests were employed as they provided a rapid and readily available means of estimating UCS in the field. A point load test (PLT) was performed on 32 small blocks of rock samples, comprising quartzite, conglomerate, shale, and mudrock, with sizes ranging from 40mm to 60mm. The load required to fracture or break each sample was recorded, allowing for the determination of the UCS of these rock samples.

In addition, a geological hammer was used to perform an impact blow test on the surrounding rocks of the excavation, including the hangingwall, sidewalls, and faces. The number of impact blows required to fracture the rock was counted, and the results were used to estimate the UCS of the rock based on the field estimation guidelines suggested by the ISRM (1978). According to Hoek (2001), this particular method serves well for both the early design phase of excavations and scenarios without laboratory access.

3.3.2 Laboratory rock testing

Laboratory rock testing is a quantitative technique that employs drill core samples and small chips to determine the parameters of rock mass strength. The analysis combines analytical methods with numerical techniques to gain complete knowledge of rock

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mass characteristics. Comprehensive design studies require laboratory rock testing for its accurate and reliable data which supplements the quick results obtained from fieldwork equipment.

Two laboratory tests were performed to evaluate rock mass strength properties in this research. The point load test (PLT) measured the uniaxial compressive strength (UCS) of the rock mass. The triaxial (confining) test was used to evaluate the triaxial compressive strength (TCS) of the rock mass.

3.3.2.1 Core sample preparation for point load and triaxial tests

In geotechnical and rock investigations, rock core samples are prepared for laboratory testing to assess the strength and deformation properties of intact rock. Figure 3.5 depicts the preparation procedure, which includes four major steps: core collection (1), cutting (2), trimming (3), and profiling (4). The core rock samples are collected from the field and stored into the core trays which are then taken to the laboratory for analysis. A diamond saw or core cutter performs the cutting process by dividing the rock core into shorter segments. The cutting procedure strips away damaged sections from the core and then readies the specimen for upcoming operations. Trimming offers selections by eliminating irregularities together with impurities that exist on cut core specimens such as fractures or weathered areas. The representative sample results depend heavily on this step to show the state of the original rock material while minimizing test distortion due to weaknesses in the sample. The preparation for profiling includes smoothing down the ends of the core specimens into flat surfaces. A grinding or lapping machine performs this step to achieve proper alignment of the specimen during testing. Since prepared, these specimens are placed into specialized testing equipment, such as point load and triaxial testing apparatus, as illustrated in Figure 3.5. For this study, a total of 100 rock core specimens were carefully prepared, representing four distinct lithologies: quartzite, conglomerate, mudrock, and shale.

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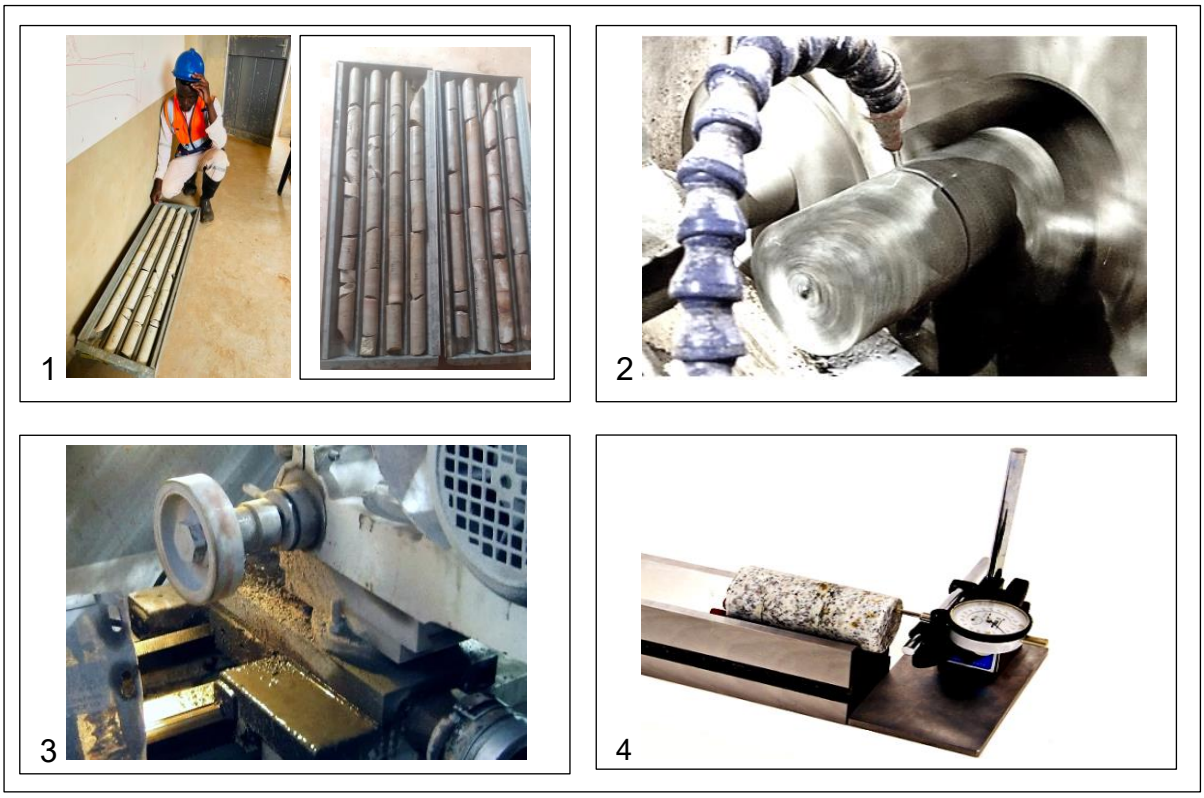


Figure 3.5: Rock core specimens preparation for point load and triaxial testing

The field-collected drilled cores with 50mm diameter underwent size reduction through cutting using a diamond-impregnated steel cutting disc mounted on a toolpost grinder. The equipment includes a water-cooling system to stop overheating. A diamond-impregnated cup-shaped grinding wheel was used to flatten the core ends through its diamond-impregnated surface. A flatness tester confirmed the flatness of the core specimens. Each specimen needed to meet International Society for Rock Mechanics (ISRM) recommendations by being trimmed to 100mm length which equals two times the core diameter of 50mm.

3.3.2.2 Point load test

Due to the time consumption and the expense of performing a uniaxial compression test, a point load test was used to determine the compressive strength of the rock specimens or samples. However, the empirical equation that is used to calculate the UCS based on the Point Load Index (Is) can create some uncertainty (incorrect data input) that can result in inaccuracy when compared to the direct measurement of UCS through uniaxial compression test results. From the four basic types of point load testing including axial, block, lump, and diametral tests, axial point load testing was

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conducted on 50 core samples, each with a diameter of 50mm and a thickness of 100mm. The test provides a direct estimation of the uniaxial compressive strength (UCS) of the core rock specimens. In addition, the anisotropy of the rock core specimen is determined with ease by using axial point load testing. Unlike diametrical point load testing which is sensitive to the anisotropy. Force loads were applied to specimens until failure occurred or breakage happened while the longitudinal axes of each core sample remained parallel to the loading apparatus. The failure load (P) was recorded, and the point load index (Is) was calculated using Equation 5. The uniaxial compressive strength (UCS) of the rock specimen (D50) was then determined using the following equation:

$$UCS\ (\sigma c) = \ 24.Is \qquad (5)$$

3.3.2.3 Triaxial (confined) test

Triaxial compression refers to the state of stress in a body subjected to compressive forces or stresses in three directions. Triaxial compression experiments simulate the behaviour of rock under confinement, away from underground openings, where the principal stresses are unequal (σ1 > σ2 = σ3). Considering the accuracy of the triaxial rock core testing and making an informed decision, Hoek and Brown (2002) suggested that at least six of the trials must be performed on the same rock type. In this study, 10 rock core specimens for each of five rock types (mudrock, shale, gold-bearing conglomerate, argillaceous, and siliceous quartzite) which amount to a total of 50 specimens, were subjected to triaxial compression testing. Each rock specimen was enclosed in a sealed pressure cell with a latex jacket and subjected to confining stress by pressurizing oil. Axial stress was then increased while maintaining constant confining stress until the specimen failed at its peak strength. Throughout the test, stress and strain measurements were continuously recorded using strain gauges.

The Mohr-Coulomb and Hoek-Brown failure criteria were the basis for evaluating and interpreting results acquired through triaxial testing. The RocData software applied these criteria to visualise and interpret the test outcomes. The Mohr-Coulomb criterion (Equation 8) creates a linear link between rock shear strength behaviour and normal stress values. The criterion describes how intact rocks behave and helps determine rock shear strength parameters like cohesion and friction angle. The Hoek-Brown

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criterion presents a non-linear failure envelope that incorporates rock inherent strength properties alongside confining stress effects (Equation 9). The criterion proves better at modelling rock mass behaviour, so engineers use it frequently in their fieldwork.

The combination of both criterions provides an extensive foundation of understanding about rock strength characteristics from triaxial testing procedures. The analysis demonstrates the shear strength properties including behaviour patterns and potential failure types for the examined rock material. The acquired information provides the essential framework to build stable underground openings and protects employees and mining equipment during operations. Figure 3.6 illustrates how Mohr-Coulomb and Hoek-Brown criterions were applied to triaxial test results to show rock strength characteristics. The setup and input parameters for Mohr-Coulomb and Hoek-Brown failure classifications is discussed in section 3.5.3.

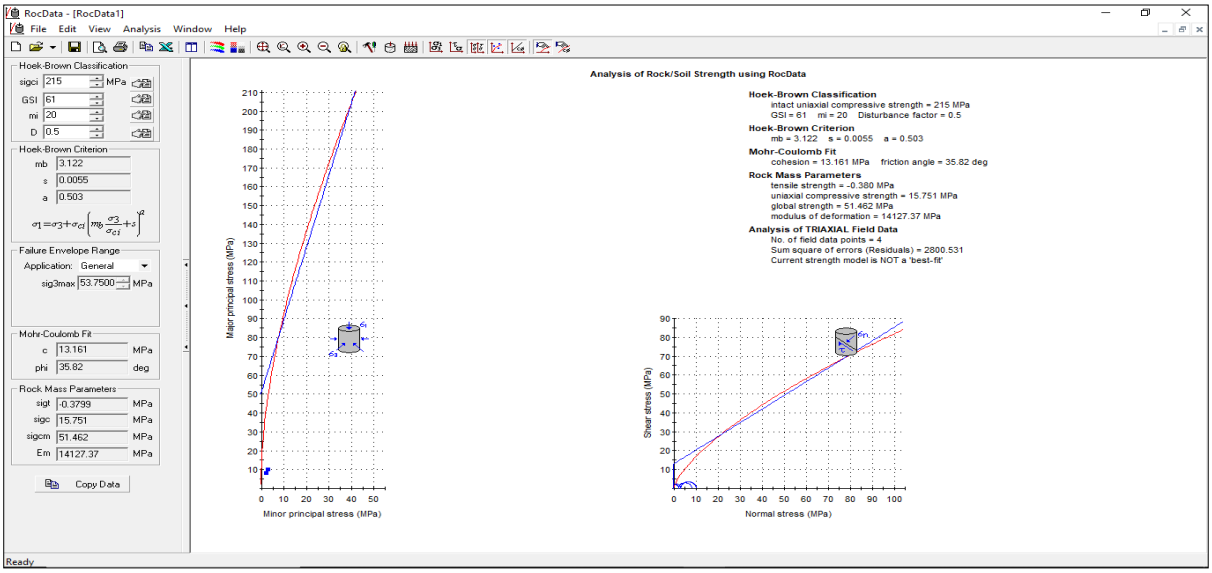


Figure 3.6: Triaxial rock strength analysis by applying Mohr-Coulomb and Hoek-Brown failure criterions in Rocdata application

3.4 Rock Mass Classification System

The rock mass classification system provides vital information about excavation ease together with the required support properties matching particular ground conditions. Underground excavation design requires the essential knowledge provided through rock mass classification work. As recommended by Hoek (2000) to use multiple rock mass classification systems, this study employs three widely used rock mass classification systems to assess the rock quality of the raise and the stopes of Emerald

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Shaft Gold Mine. The classification systems used include Rock Mass Rating (RMR), Tunnelling Quality Index (Q), and Geotechnical Strength Index (GSI). These classification systems attracted selection for their capability to offer detailed assessments of rock mass quality by incorporating geological alongside geotechnical and engineering characteristics and the outcomes can be comparable to each other for deeper insights into rock mass characteristics.

The rock mass classification in this study was determined through a thorough analysis of various data sources, including face mapping data (discontinuity survey), borehole data, and uniaxial compressive strength (UCS) test results from both field and laboratory testing. The RMR_{89} system features basic design and general usage capabilities. Furthermore, this system assigns numerical ratings to help estimate how strong and stable the rock mass will be along with its deformability. Six key parameters introduced by Bieniawski (1984) were assessed. These parameters include the uniaxial compressive strength (UCS) of the rock material, Rock Quality Designation (RQD), spacing of discontinuities, condition of discontinuities, groundwater conditions, and orientation of discontinuities.

The classification of the rock mass based on the Rock Mass Rating (RMR_{89}) system was done in a stepwise procedure to identify the quality of the rock mass following the RMR_{89} guidelines presented in Appendix III (D). The Uniaxial Compressive Strength (UCS) determined in the laboratory was assigned first, and classified according to the rock strength. Then, drill core and mapping data were used to calculate the Rock Quality Designation (RQD) and was rated according to the percentage of good rock quality. The field measurement of discontinuity spacing was recorded and rated by the distance between discontinuities. The roughness, aperture, and infilling were used to evaluate the condition of discontinuities. Observed water flow was used to assess the groundwater conditions. Lastly, the discontinuity orientation was determined in terms of dip and dip direction of the tunnel or stope and assigned to the rock mass quality table presented in Appendix III. RMR value is calculated by adding the ratings of six parameters and categorizes the rock masses into five groups (very poor rock = 0-20, poor rock = 21-40, fair rock = 41-60, good rock = 61-80, very good rock = 81-100.

In addition to RMR, the Q-system classification (Barton *et al*., 1974) was also employed and worked as a valuable tool due to its capability of combining stress-

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fracture relationships with water-soaked rock mass behaviour. In order to classify the rock mass quality under Q-system classification, a step-by-step method was used following Q-system classification of parameters and ratings guidelines presented in Appendix III (E). The process entailed an assessment of six important parameters, namely: the Rock Quality Designation (RQD) index, the number of joint sets (Jn), the roughness of the weakest joint (Jr), the alteration or infill degree along the weakest joints (Ja), the joint water reduction factor (Jw), and the stress reduction factor (SRF). The drill core and field data were used to calculate the RQD index and this index was rated according to the percentage of good rock quality. The joint sets were counted and then rated according to the observed joint sets. Joint roughness was used to assess and rate the roughness of the weakest joint. The extent of alteration or infill along the weakest joints was assessed and graded depending on the extent of alteration or infill. Joint water reduction factor was calculated according to the presence of water in joints and rated on this basis. Lastly, the stress mitigation factor was also assessed according to rock stress conditions and rated as such. The Q-value was subsequently computed using Equation 2 which enabled the rock mass quality to be classified.

Lastly, GSI was employed to assess and quantify the strength of the rock mass. The GSI received selection due to its capability of considering both geological structure and historical geological factors which affect the rock mass behaviour. When performing GSI system assessments the rock mass quality was determined by examining the combination of rock structure with rock surface conditions. The GSI values were calculated using the RMR_{89} and Q-system. The calculations were performed using two equations:

- $GSI = RMR_{89} - 5$ for $RMR_{89} > 23$ (3)

- $GSI = 9LogQ + 44$ for $RMR_{89} < 23$ (4)

3.5 Data Analysis, Presentation, and Interpretation

The data obtained by field observation and laboratory experiments were recorded, summarised and analysed thoroughly through a combination of statistical, empirical, analytical and numerical methods. The results of the analysis, field study, and

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literature reviews were stored, analysed, and presented clearly and concisely using various visual aids, including statistical tables, pie charts, stereonets, graphs, sketches, and maps. The process of data analysis is well described with details of the software, software setting and calibration, and coding in the subsection below.

3.5.1 Statistical Analysis

Statistical analysis is an application of statistical techniques in data collection, analysis, and interpretation. This study employed statistical methods to analyse the results of field measurements and laboratory tests to find trends and patterns in rock mass behaviour and the uncertainty of rock mechanics and strata control. The statistical analysis was conducted with the help of Microsoft Excel, a popular spreadsheet application, and Openpyxl 3.1.5, a Python library that enables the programmatic access to the Excel files. Various statistical methods such as descriptive statistics, and correlation analysis were used to analyse the data. Statistical parameters, including mean, standard deviation, and linear regression coefficients were calculated in Excel using formulas and functions, such as AVERAGE, STDEV, and LINEST. Data manipulation and analysis have been coded and automated with the help of Openpyxl 3.1.5 such as rock mass quality spreadsheets presented in Appendix III, which guaranteed the efficient and correct calculation of statistical parameters. The spreadsheet was coded to safeguard fixed data against deletion or misplacement by accident, and this assures data integrity. This allowed an efficient analysis, protection of data, and automatic manipulation of data, which minimised the chances of human error. The integration of the Excel and Openpyxl allowed conducting the statistical analysis effectively and correctly, which offered useful information on the data. The data were summarised and relationships between variables were identified by the results of the analysis to create predictive models.

3.5.2 Empirical Analysis and design

Rock mass classification systems such as Rock Mass Rating (RMR) system, Q-system and the GSI were used to conduct empirical analysis. Such systems are used to rate parameters of the rock mass (rock strength, joint spacing, and ground water conditions) to describe rock mass behaviour and hence support design. The method applied was the empirical rating tables and charts of the rock mass rating, such as

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RMR rating table and Q-system chart to determine the classification of the rock mass as well as estimate the stability of the rock mass. The findings offered a complete knowledge of rock mass behaviour, such as strength, deformability, and stability, and they were applied in the development and the optimization of support systems, in the assessment of stability, and in the assessment of the potential hazards. The analysis provided a methodical and consistent method of describing rock mass behaviour with the help of the empirical rating tables (Appendix III, A, B, and C) and charts to support design (Appendix III, D).

3.5.3 Analytical Analysis and Design

Analytical analysis uses mathematical and logical methodology to provide precise solutions to complicated problems. The present study employed analytical analysis to solve stress patterns around underground excavations, forecast the conditions of rock failure, and assess the stability of rock masses. Analysis special software packages such as RocData, Dips, and Unwedge were used to obtain exact solutions to rock mechanics and stability analysis problems.

RocData was used to interpret the rock properties and behaviour and estimate parameters of rock strength under Hoek-Brown and Mohr-Coulomb failure models. The analytical functionality in the software aided in the evaluation of rock mass stability, the determination of the failure potential, and the design of support to underground excavations. The quality of input data, including joint orientation and rock mass properties, is a crucial aspect in the quality of the analysis, however. Irrespective of this limitation, the results were applied to optimize supporting systems, whereby stability and safety were guaranteed.

The rock mass was analysed using Dips and Unwedge software packages because of their specific features. Dips was chosen because it can analyse joint and fault orientations and offer statistical summaries and visualisation in the form of stereonets and rose diagrams and this allowed the identification of possible failure modes. Unwedge, which can be used to evaluate the stability of excavation in a jointed rock mass and design the support system was used to recognize potentially unstable wedges and stabilize them using rock bolts. According to Potvin and Hadjigeorgiou, (2020), Unwedge assume that the potentially unstable wedges are clearly delineated

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by the dominant structures. The factor of safety is governed by the consequences of the failure mode (instability). The design factor of safety is assumed to be 2-5 due to gravity loading and 1.5-2 due to the side wall drive of unstable wedges (Potvin and Hadjigeorgiou, 2020). Despite the fact that the quality of these analyses is reliant on good input data and simplifications in their nature, the software packages gave good insight in the behaviour of rock masses and support design. . RocData, Dips, and Unwedge packages gives good insight into rock mass behaviour to assist design decisions. The subsection provide project settings and calibrations of RocData, Dips, and Unwedge packages.

3.5.3.1 RocData Project Settings and Calibration

RocData 3.0 software was used to measure the strength of the intact rock samples by using laboratory and field data. The project settings were adjusted to apply the MPa unit to the stress and the General Hoek-Brown and Mohr-Coulomb strength criteria were chosen to be analysed shown in Figure 3.7.

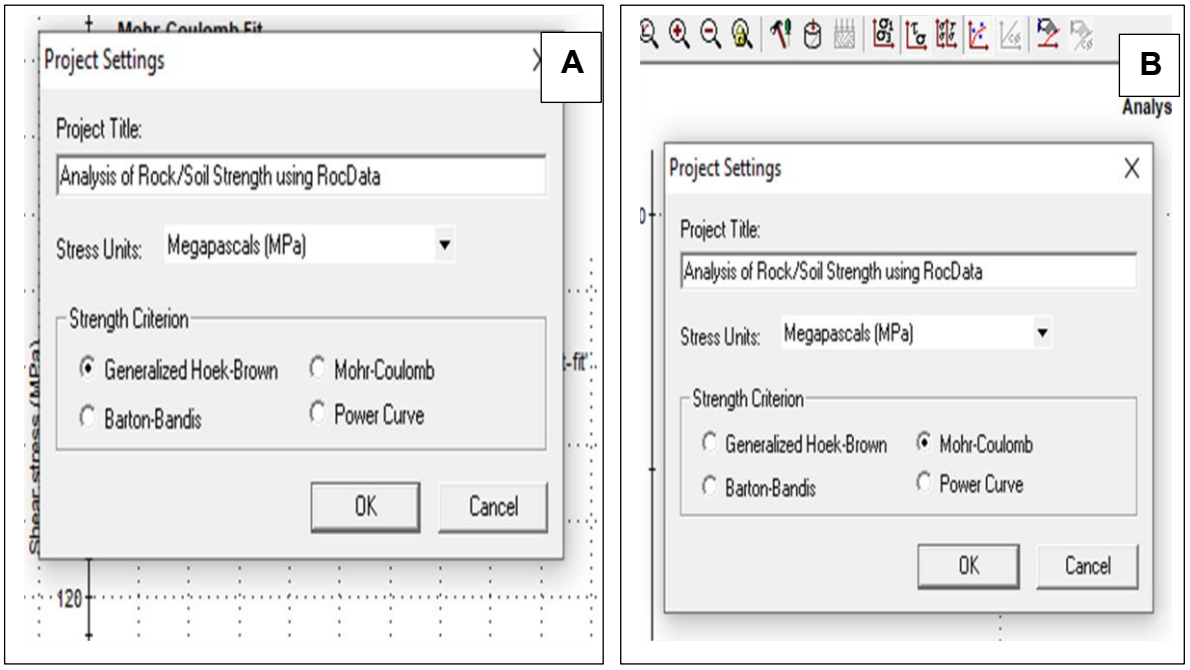


Figure 3.7: Project setting in RocData to perform rock strength analysing using Hoek-Brown (A) and Mohr Coulomb (B) strength Criterions

Laboratory data was imported into the software as an excel sheet containing ten laboratory trials or tests for each rock type illustrated in figure 3.8 (siliceous quartzite

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major and minor principal data). Three methods of curve fitting were used to analyse the data; these were Lavenberg-Marquart, Simplex and Linear Regression. The Linear Regression tool gave the best fit calibration with the minimum squares of errors (79.34 and 94.40) for Hoek-brown and Mohr-Coulomb respectively.

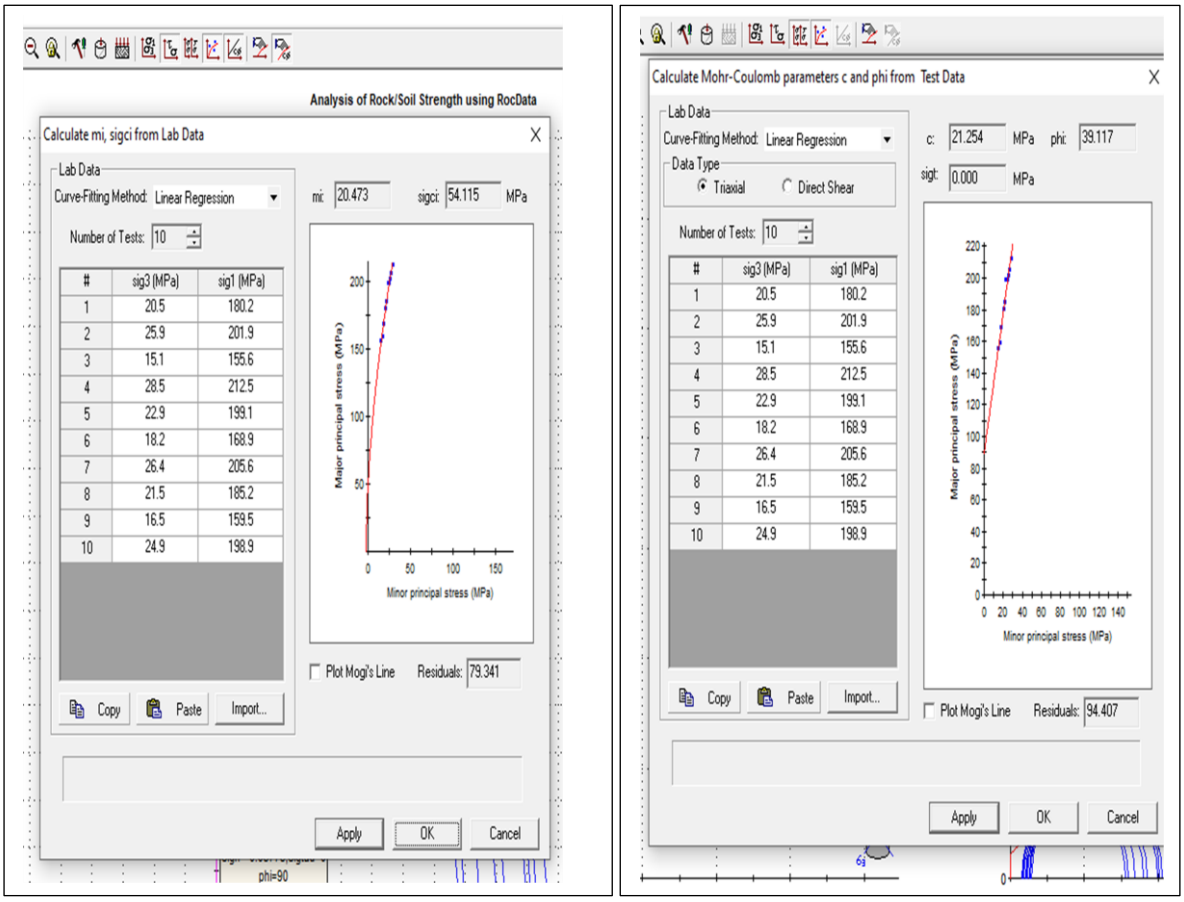


Figure 3.8: Laboratory data input with the used of the Linear Regression tool that gave the best fit calibration

Hoek-Brown classification parameters were defined as Uniaxial Compressive Strength (UCS) 120 MPa, Geological Strength Index (GSI) 55, Material Constant (Mi) 20 and Disturbance Factor (D) 0.8 shown in Figure 3.9 (Field data input parametes). The rock mass parameters were estimated using these parameters in estimating the strength and deformation properties of the rock mass. The Mohr-Coulomb categorization values were as follows; Cohesion = 13.9 MPa and Friction Angle = 42.03 degrees shown in Figure 3.10. Mohr-Coulomb analysis was subjected to the best fit Linear Regression calibration.

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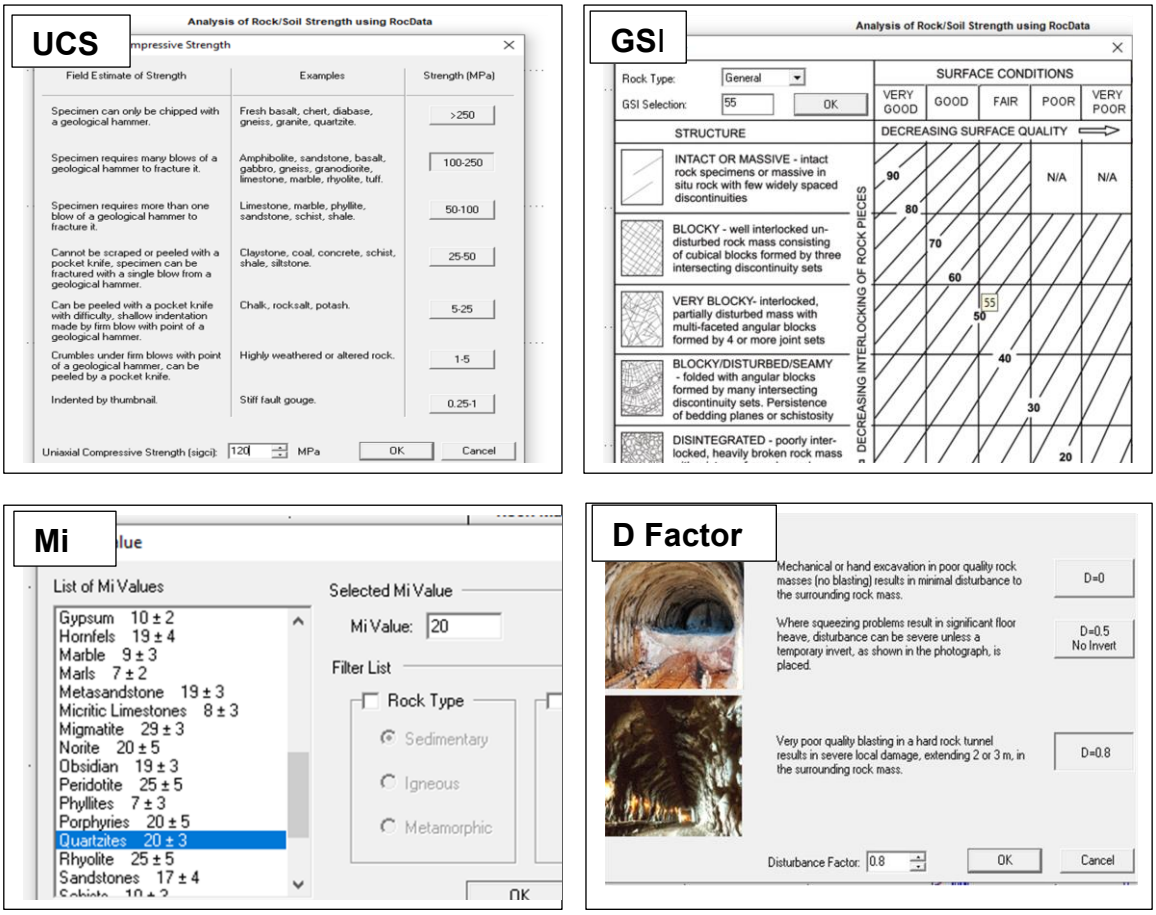


Figure 3.9: Field data input parameters for General Hoek-Brown Classification in RocData 3.0

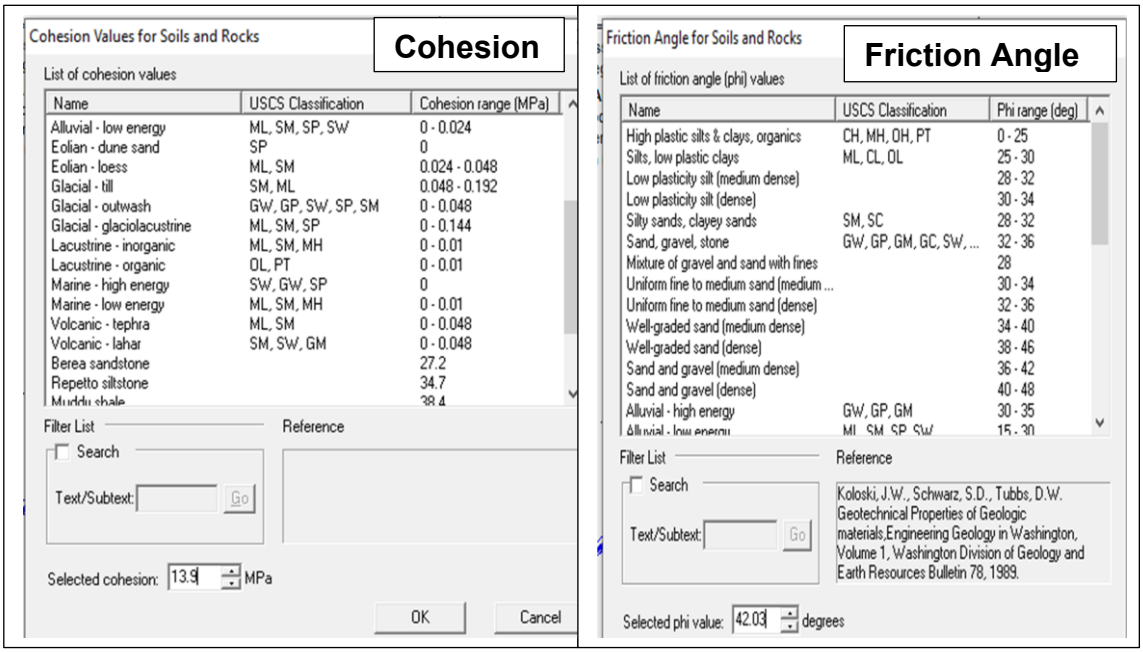


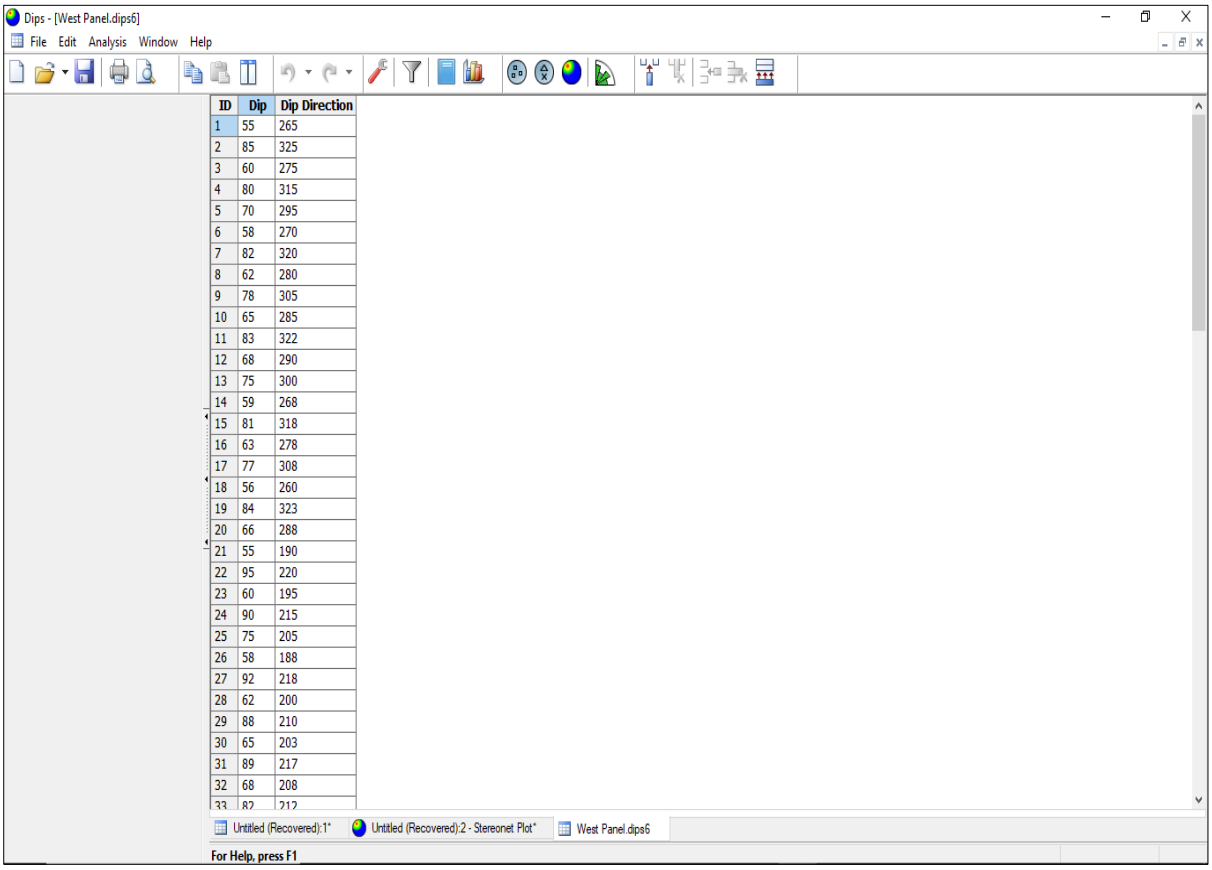
Figure 3.10: Input parameters for Mohr-Coulomb Classification

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The scope of the failure envelope was specified as a tunnel, the unit weight was taken as 0.027 MN/m 3 (which is equivalent to 2750 kg/m^3) and the depth was 750m. The strength and stability of rock mass under such conditions were estimated by both Hoek-Brown and Mohr-Coulomb criteria.

3.5.4.2 Dip Project Setting and Calibration

Analysis of the structural data of the Main Raise, East and West panels was done using the Dip 6 software presented in Appendix II. The project parameters were fixed in such a way that the global orientation format was Dip/Dip Direction with an inclination angle of 0. Figure 3.11 show the input of data into the Dip 6 software. The software read discontinuity or structural mapping data in a spreadsheet and plotted on a stereonet. This has made possible the choice of several options of analysis such as vectors, contour range, density, and wedge analysis and this has made it possible to get insights on the possible failure mechanisms and stability of the rock mass (Rocscience, 2012).



ID	Dip	Dip Direction
1	55	265
2	85	325
3	60	275
4	80	315
5	70	295
6	58	270
7	82	320
8	62	280
9	78	305
10	65	285
11	83	322
12	68	290
13	75	300
14	59	268
15	81	318
16	63	278
17	77	308
18	56	260
19	84	323
20	66	288
21	55	190
22	95	220
23	60	195
24	90	215
25	75	205
26	58	188
27	92	218
28	62	200
29	88	210
30	65	203
31	89	217
32	68	208
33	82	212

Figure 3.11: The discontinuity or structural mapping data input in Dips 6 software

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3.5.3.3 Unwedge Project Settings and Calibration

The Unwedge 3.0 was used to determine the stability of the raise, west and east panels. MPa was the unit of stress used in the project, which was named as the Stability of Raise, West and East Panels Wedge Analysis. The geometry of the raise was set as 4m of diameter and east and west panels were 3m long and 2.5m wide illustrated in Figure 3.12. The designed factor of safety was assumed to be 2 which account for both gravity and side wall drive failure modes of mechanism.

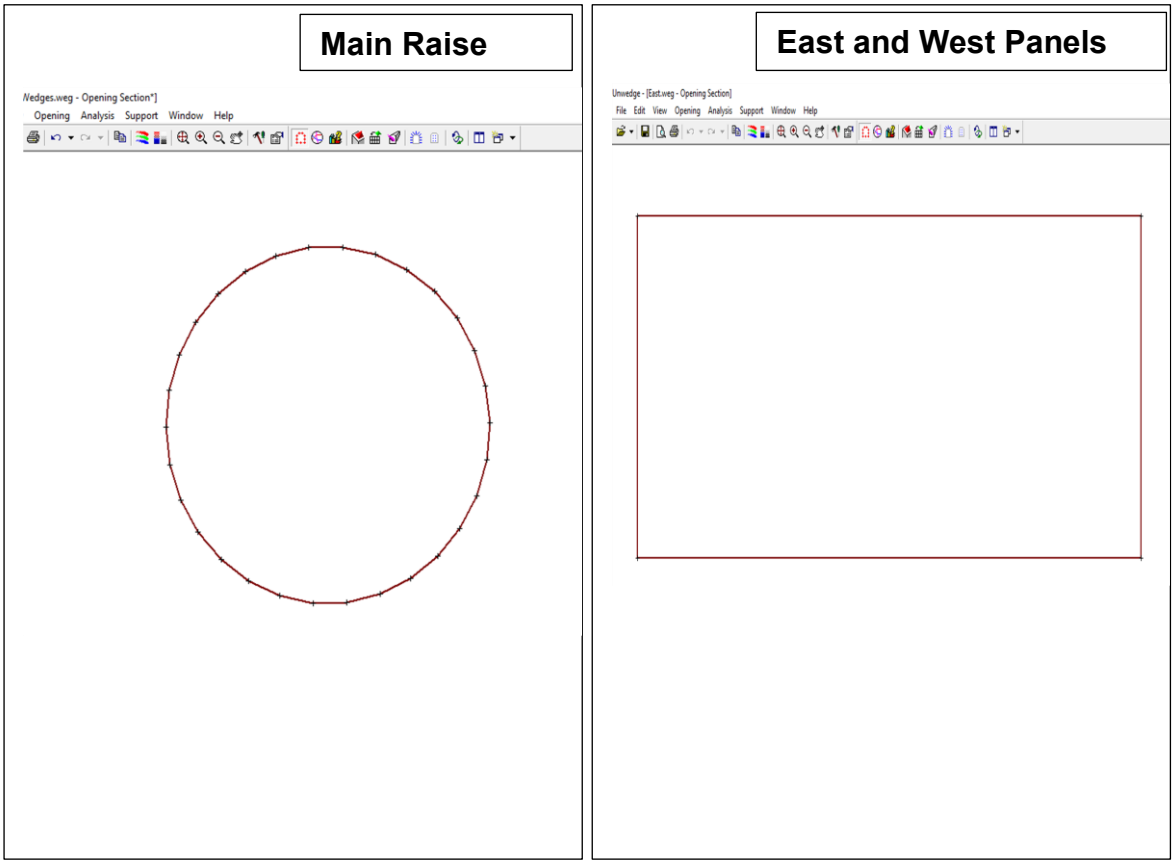


Figure 3.12: The geometry of the Main Raise, East and West panels excavations

The discontinuity data were fed into the software, which included the general parameters of excavation orientation, the design factor of safety, and the unit weight of the rock type (2750 kg/m^3). The use of the Mohr-Coulomb model also included joint orientation and properties such as cohesion, the angle of friction, and tensile strength as input parameters illustrated in Figure 3.13. These common properties through field mapping made it possible to produce wedges in the Main Raise, East and West panels.

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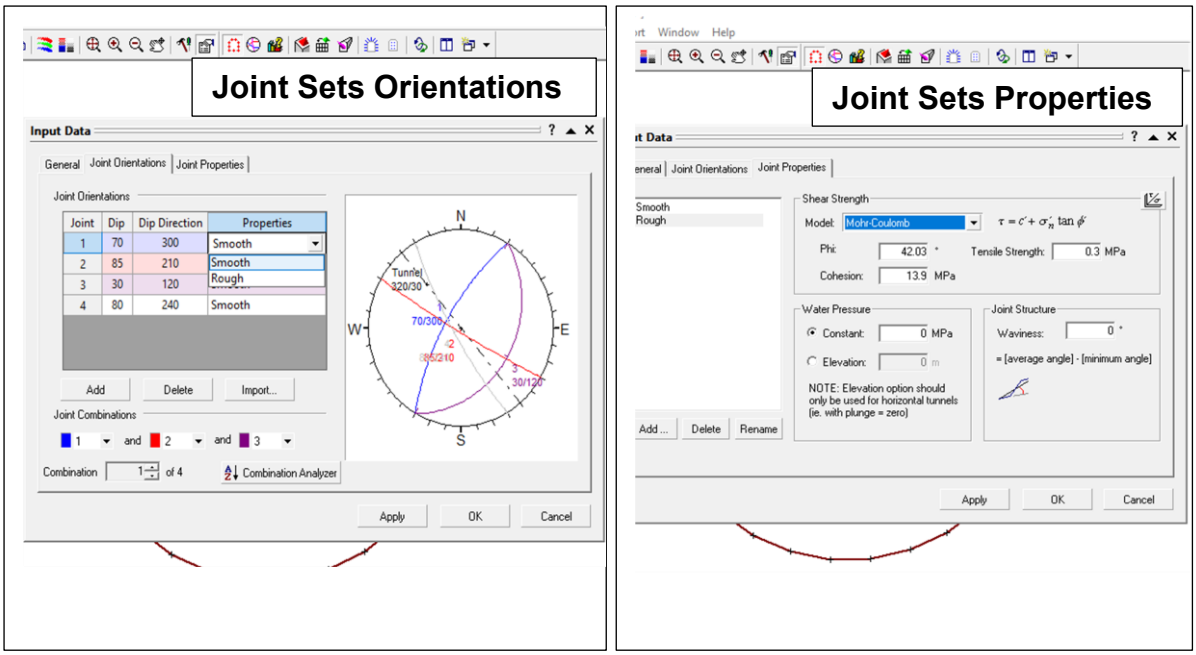


Figure 3.13: Joint Orientation, and joint Properties input data in Unwedge software

A 3D model of the visualization was created as illustrated in Figure 3.14, showing all wedges created by the intersection of sets of joints and the related factor of safety. In-situ stress conditions were determined and the field stress was defined as a function of depth. Different components of stress were examined using stress visualization options and the change in the factor of safety of the wedges noted.

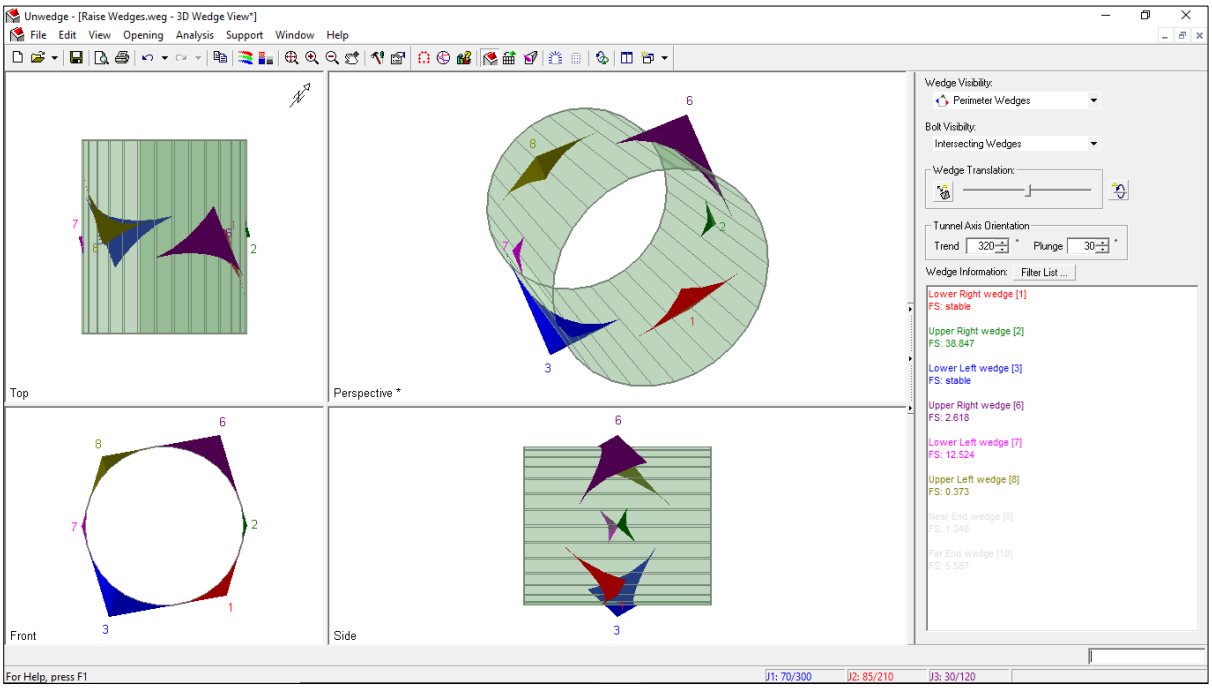


Figure 3.12: 3D Visualisation of wedges generated by the intersection of Joint Sets

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To stabilize the possible unstable wedges the apex height was considered and the type of support, such as standard roof bolts (split set) with grouted dowels and concrete lining were specified and implemented on the model. Split set bolts properties were defined as illustrated in Figure 3.15 and the change in factor of safety of the unstable wedges determined after the application of supports.

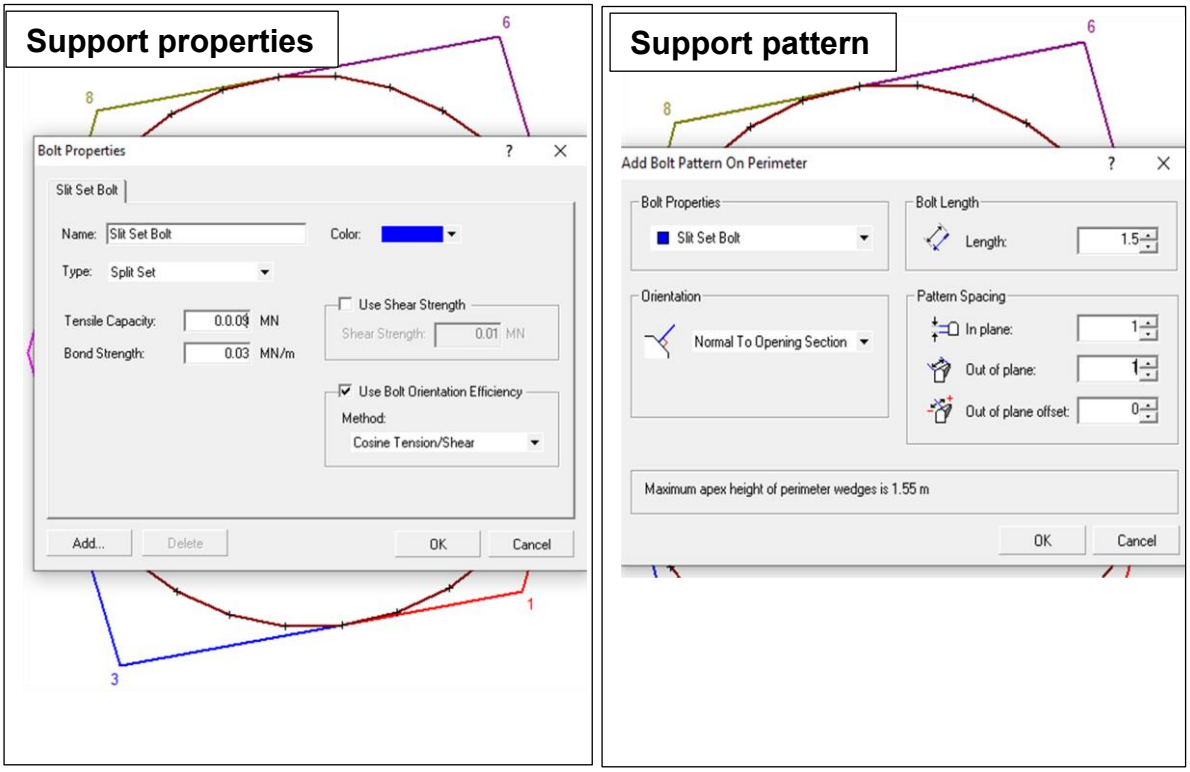


Figure 3.15: Support properties and pattern data input

3.5.4 Numerical Analysis and Design

Numerical analysis is the use of numerical techniques, examples of which are finite element or finite difference techniques to model the behaviour of rock masses and underground excavations. The work uses numerical modelling to mimic the behaviour of rock masses under different stress conditions, simulate rock mass behaviour in response to excavation and forecast underground opening stability (Rocscience, 2015). Numerical analysis is an effective approach to the study of complex problems of rock mechanics, which allows simulating a variety of situations and predicting the behaviour of the mass of the rock, which ultimately serves as a guide to the design of support and the stability and safety of the underground excavation.

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The numerical analysis was carried out using RocPhase2 which is a two-dimensional finite element software package specifically designed to the rock mechanics and the rock engineering problems. RocPhase2 was selected because of its capability to simulate the behaviour of rock masses under different stress state, model complex geological structure and determine the stability of underground openings. When calibrated, elastic estimates take an initial estimate accurately, but complex cases such as large yielding areas and interacting excavations are better estimated using elasto-plastic analysis. RocPhase2 assume geometry, stress fields and failure mechanisms to be confined to the plane of analysis. 2D models are well suited to what-if scenarios, being easy, fast, and having simple result visualisation (Chehreghani *at.al*, 2022). Nevertheless, there are limitations such as constraints of 2D analysis influences validity of analysis. In spite of these, RocPhase2 offered detailed information on rock mass performance and support designing, which makes it an effective tool in rock mechanics and engineering. The subsection provide the project settings and calibration of Rocphase2 modeller 8 for analysis and interpretation.

3.5.4.1 RocPhase2 Project Setting and Calibration

RocPhase2 Modeller 8 software was used to examine the stability of the raise at the Emerald Shaft Gold Mine. The project settings were adjusted to have a plane strain analysis type, Gaussian Elimination type of solver, metric units and stress in MPa. There were two stages outlined: Stage 1 is an in-situ stage; Stage 2 is the final stage (excavated raise).

An external boundary (Triple the excavation size) was established and a raise of 4m in diameter created. The rock materials were arranged following the stratigraphic unit of Emerald Shaft Gold Mine including mudrock, shale, and quartzite shown in Figure 3.16. The engineering properties of each of the materials, as determined in the laboratory, contained the Young's Modulus, Poisson Ratio, and the unit weight in which the data is summarised in Table 4.3 were assigned to each rock material. The materials were characterized as isotropic, elastic, and elasto-plastic types. Failure criterions were determined as Hoek-Brown and Mohr-Coulomb.

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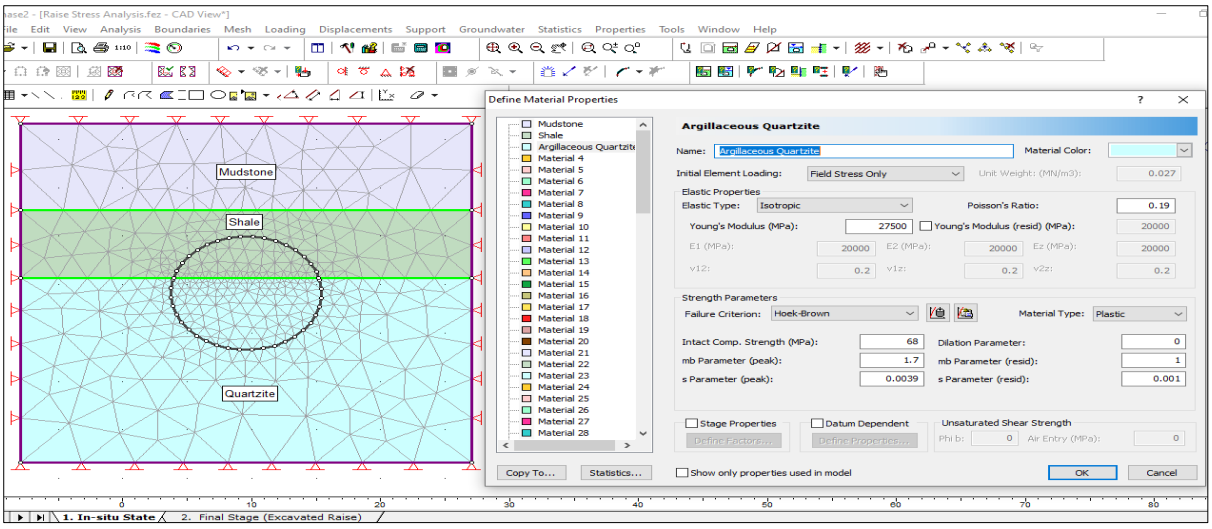


Figure 3.16: Model creation and material assignment (engineering properties of rock mass) in stage 1

The boundary mesh generated was computed to simulate fractures in the model with intensification along the excavation boundary. Field stress was added as a stress loading (σv=20.23, σh=30.35. and σ3=12.51 presented in Table 4.18) to the model. In the second stage, the raise was excavated as shown in Figure 3.17, and the model was computed and analysed. For an elastic analysis type, software compute the data in less than one minute. However, the elasto-plastic type took more than a minute hence proved to be useful in analysing the zone of potential failure by creating the finite elements shown in Figure 4.23.The analysis and interpretation in this case was based on Sigma 1 and Z (Table 4.20 and 4.21, Figure 19 and 21), stress factor (Figure 4.23), and the total displacement or deformation of the raise (Figure 4.25).

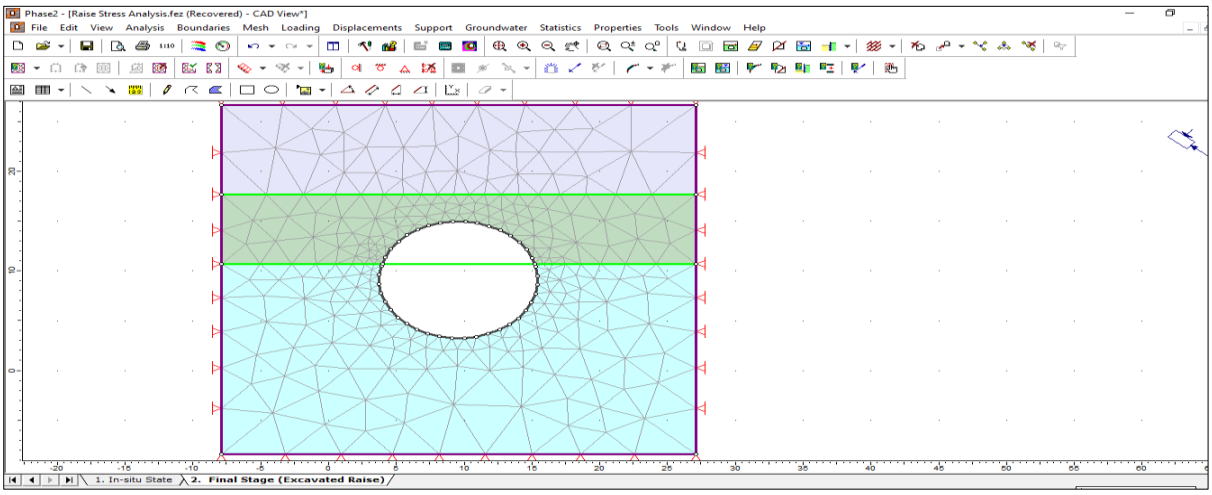


Figure 3.17: The excavation of raise in stage 2

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In order to reduce the impacts of stress and displacement, support systems were incorporated, such as full bonded bolts and concrete lining. The support and spacing properties are determined and the model was recomputed, which yielded a figure that represents the effects of the support systems shown in Figure 5.6. Figure 3.18 illustrate the addition of the support systems in to the model and the input parameters of the support in which each support parameters where the unit parameters for support unit are indicated and further discussed in Chapter 5. The purpose of this analysis was to determine whether the raise was stable and how effective the support systems were.

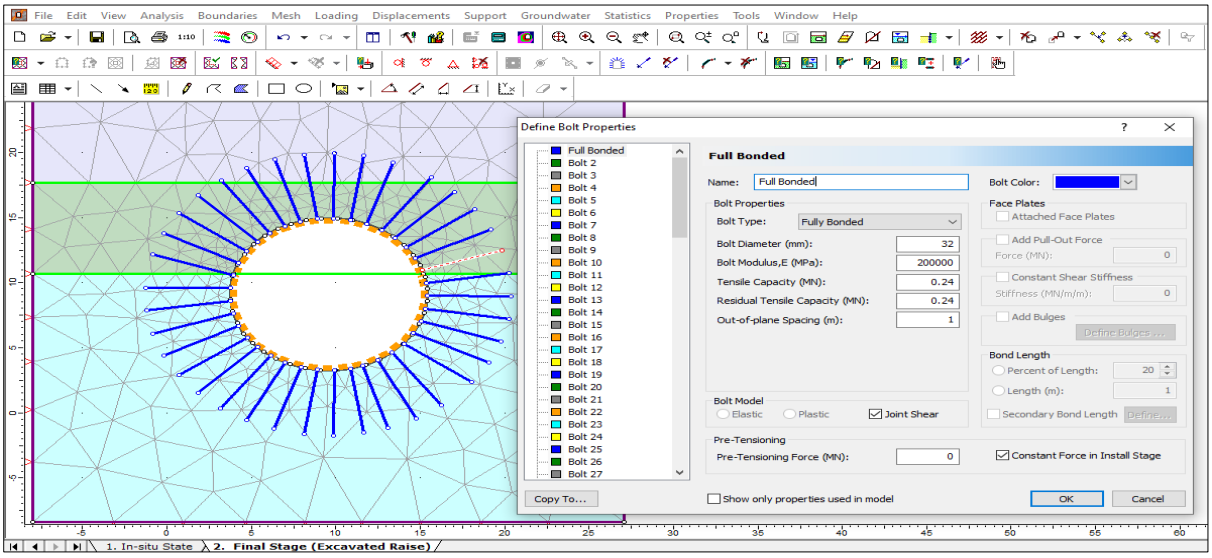


Figure 3.18: The addition of support unit or element into the model

3.5.5 Summary and Validation

The multi-dimensional analysis of the strata conditions and support systems at the Emerald Shaft Gold Mine involved a well-balanced angle, where the fusion of these three methodologies: empirical, numerical, and analytical methods was explicitly employed to meet the research objectives of assessing the mine stability and safety in order to mitigate the risk, optimizing the support systems and layout to achieve efficiency of work, and exploring the behaviour of rock masses to generate knowledge of geotechnical interest. The study established an integrated information base concerning rock mechanics and strata control, and it was well supported by cross-checking the result and utilizing the benefits of both methods, enhanced prediction accuracy and confidence, and optimal support and mine layout design.

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CHAPTER FOUR: RESULTS AND DISCUSSION

This chapter presents a comprehensive analysis and discussion of the data obtained through the various methods and procedures described in Chapter 3. Additionally, this chapter provides an overview of the mine design, and the gold recovery process implemented in the production stopes of the Emerald Shaft Gold Mine operation. The collected data was meticulously analysed and interpreted, with the results presented clearly and concisely. This chapter covers the key aspects of Emerald Shaft Gold Mine operations and design, stratigraphy, rock strength, geological composition, rock mass quality, geotechnical domain, stress environment, stability analysis, support system, and performance of the Emerald Shaft Gold Mine.

4.1 Design and Layout Review of Emerald Shaft Gold Mine Operation

The Emerald Shaft Gold Mine is a shallow underground gold mine utilizing track-bound operations and sublevel stoping methods to extract gold deposits. The mine is located within Mining Right 132 and adjoins the Emerald open pit operation. The primary focus is recovering the Kimberly reef, a gold-bearing conglomerate. Access to the mine is via the Emerald main raise, an inclined reef development featuring two levels: the 1st level (first reef drive) and 2nd level (second reef drive), as illustrated in Figure 4.1. The inclined raise at the surface collar exposes the Battery A reef, trending northwest to southeast with a 30° dip. Mining occurs in narrow to intermediate-sized breast stopes (East and West panels), with stoping widths ranging from 1.5 to 2.2 meters. These stopes are conventionally mined using drilling and blasting techniques.

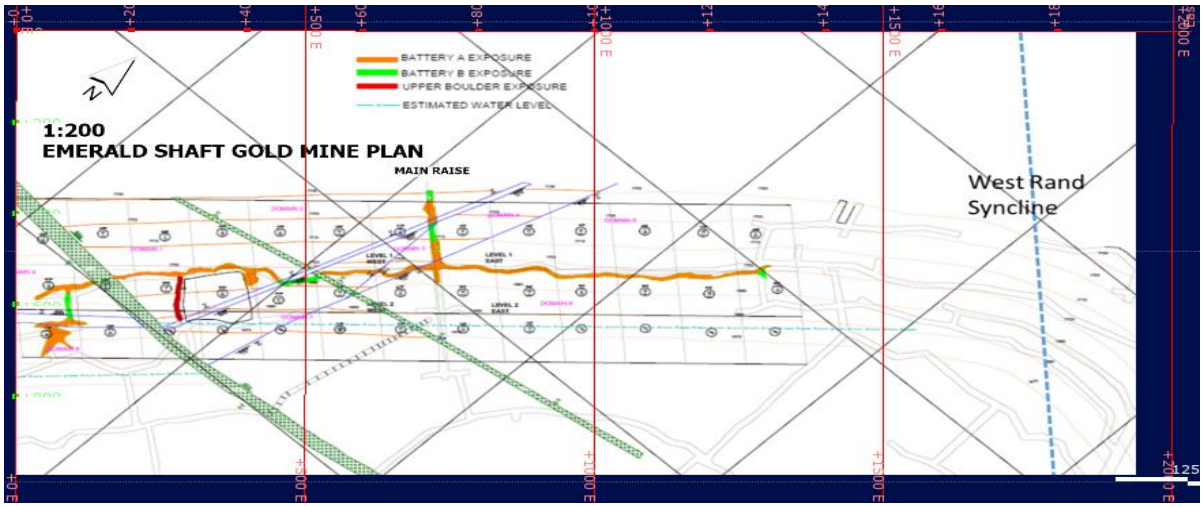


Figure 4.1: Emerald Shaft Gold Mine Plan

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4.1.1 Raising and ledging

As a result of the Battery A reef outcropping on the surface, a pencil raise was developed on the reef drive at the surface collar. The pencil raise was sequentially ledged to establish the stope panels, namely the west and east panels, and to facilitate the installation of support, as illustrated in Figure 4.2. To ensure stability and accommodating gully shoulder support, the ledging distance was carefully maintained at no more than 10m, adhering to the mine standard guidelines. This controlled ledging approach enabled the effective establishment of the stope panels and the implementation of a reliable support system.

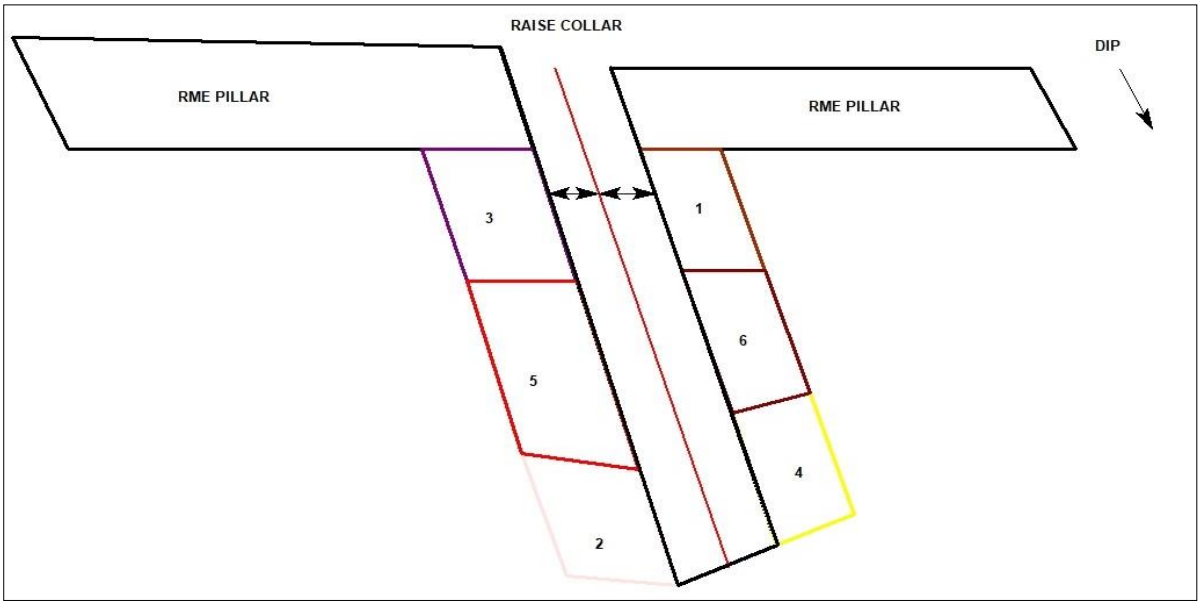


Figure 4.2: Raising and ledging of the Main Raise

4.1.2 Mining sequence and Stope layout

Following the installation of systematic support, breast stopes were established in the east and west directions, aligning with the mining direction. The sublevel stoping method, which involves breast stopes mining, is well-suited for tabular inclined orebodies. The current mining configuration employs an underhand approach, with the lead and leg of the panels. As shown in Figure 4.3, the bottom panel 2W leads, while the 1W lag behind, with an approximate distance of 5m between them. Consequently, the east panels remain in the ledging face. The mining layout is designed to facilitate regular distribution of stresses surrounding the excavated panels, providing a free

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breaking point during blasting activities. The panels advance according to the monthly production schedules, which are indicated by different colours in Figure 4.3.

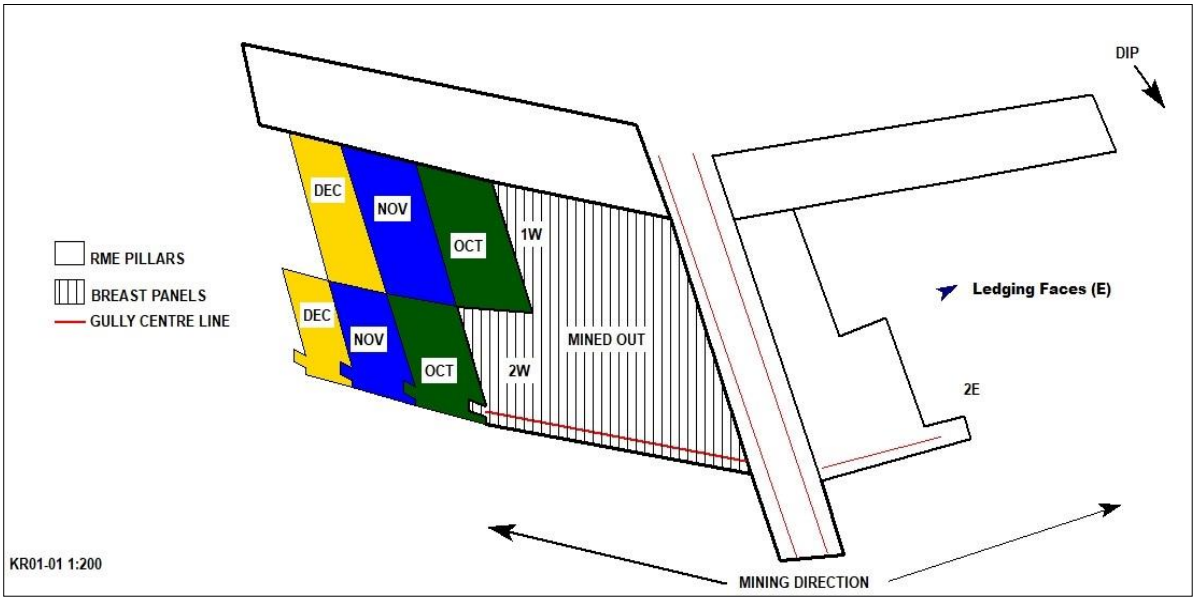


Figure 4.3: Stope layout and monthly scheduling targets

4.1.2 Gully Layout

Centre or dip, slusher, and strike gullies play a crucial role in mining operations at Emerald Shaft Gold Mine. These gullies provide a pathway for workers to access the production panels and facilitate the transportation of materials in and out of the panels. Additionally, gullies serve as emergency escape routes, enable the cleaning and removal of blasted ore and waste rock, and allow for ventilation flow into the working area. Figure 4.4 illustrates the gully layout and standard at Emerald Shaft Gold Mine. The centre gully (raise) is developed perpendicular to the strike gully and is used for personnel entry to the shaft, transportation of materials, and removal of blasted ore and waste. A traveling way was established on the east side of the raise. At the bottom of the 1-level, a strike gully was developed at an angle of 85º from the centre line to the panel face. This design facilitates efficient cleaning, ventilation flow, and personnel entry.

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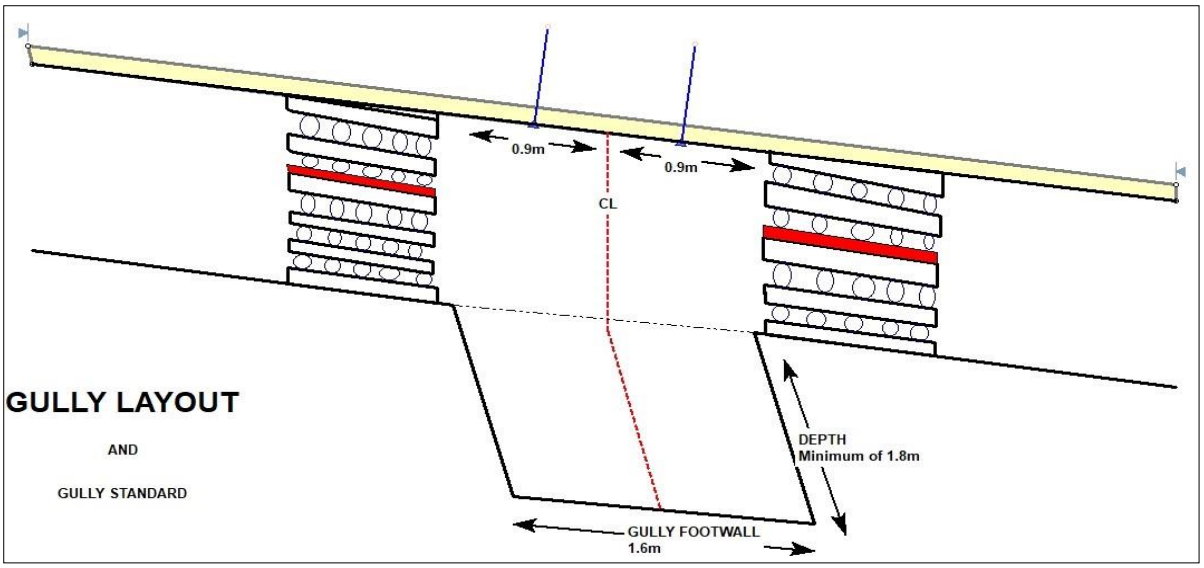


Figure 4.4: Gully layout and standard

4.2 Orebody Description

The Kimberly reefs of the Westonaria Formation are the primary gold-bearing conglomerate orebody mined at Emerald Shaft Gold Mine. According to mapping data and geological reports, these tabular (sheet-like separated from other rocks by bedding planes), inclined reefs have an average dip of 40° southeast and strike in a northeast-southwest direction. At the 1-level of Emerald, the channel width ranges from 120.4 to 200.7 cm, averaging 160.55 cm, with a corresponding average gold grade of 3.03 g/t. The intervening quartzite between reef horizons varies in thickness from 2.3 to 9 meters. The currently mined Kimberly reefs in the study area are subdivided into Battery and Boulder reefs, as illustrated in Figure 4.1.

4.2.1 Kimberly Reefs

The Battery reefs are subdivided into six conglomerate bands, designated Battery A to Battery F. However, the study area (Lease Area 132) focuses on Battery A to Battery C reef bands. These gold-bearing conglomerates exhibit varied characteristics and gold content.

4.2.1.1 Battery A Reef

Observations reveal that Battery A consists of large pebbles of oligomictic conglomerate where clasts are from single rock type, ranging from clast- to matrix-

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supported. The medium grey matrix is coarse-grained, moderately to poorly sorted, and contains both smoky and milky quartz, as well as occasional chert pebbles. This unit exhibits a fining upward sequence. Disseminated pyrite is present around pebbles throughout, with buckshot pyrite (large, rounded pyrite grains greater than 1mm) concentrated near the bottom contact. Notably, Battery A is a high-paying reef. Figure 4.5 illustrates the geological model, showcasing the 3D orebody geometry of the Battery A reef, generated from drilled hole data.

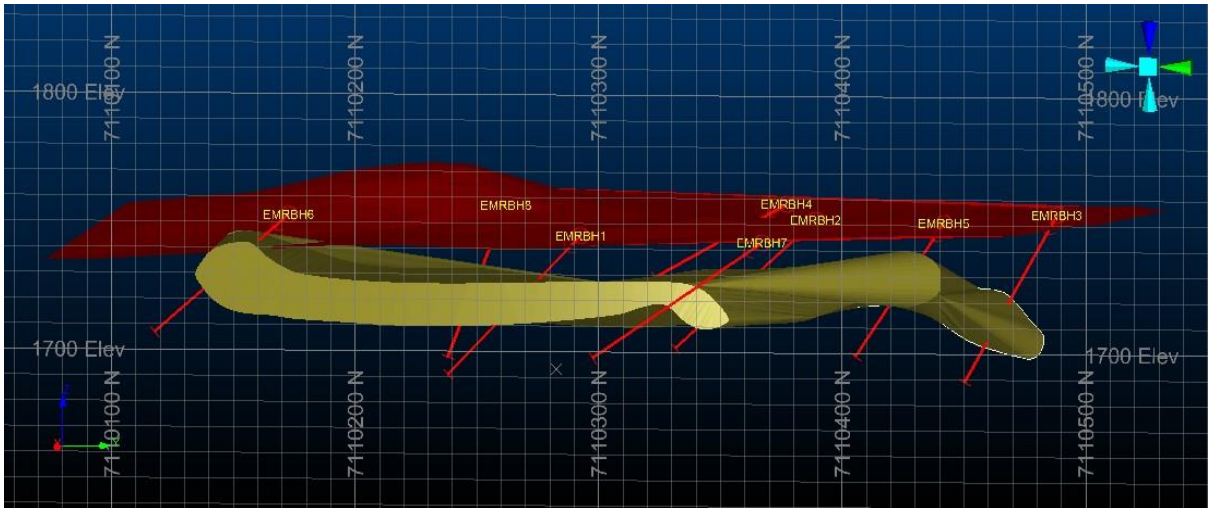


Figure 4.5: 3-Dimensional orebody geometry of a Battey A reef model

4.2.1.2 Battery B Reef

Battery B consists of a thick package of alternating cycles of polymictic conglomeritic bands where the clasts represent more than one rock type and semi-argillaceous, coarse-grained quartzite. The conglomeritic bands feature large to medium-sized pebbles, comprising both milky and smoky quartz, and angular chert clasts. Mineralization is sparse, with pyrite present in limited quantities, although disseminated pyrite occurs in some areas.

4.2.1.3 Battery C Reef

Battery C is characterized by medium to large, matrix-supported, polymictic conglomerate. The conglomerate is moderately to poorly sorted, containing smoky and milky quartz pebbles, as well as sub-rounded chert clasts. The medium grey, gritty matrix occasionally hosts disseminated pyrite, both within the matrix and surrounding the pebbles. Notably, buckshot pyrite is observed near the top contact.

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4.2.2 Boulder Reefs

The Boulder reefs host two gold-bearing conglomeratic bands: the Lower Boulder and the Upper Boulder. These bands are separated by a slightly argillaceous proto-quartzite (Medium-grained sandstone) with scattered pebbles.

The Upper Boulder consists of a very large pebble to cobble-sized, clast-supported, oligomictic conglomerate. It features rounded, closely packed pebbles and is moderately to poorly sorted. Notably, pebble sizes and package thickness decrease from Monarch to Emerald Raise.

The Lower Boulder is characterized by large, oligomictic conglomerate that is clast to matrix supported. It is poorly sorted and moderately packed, with a medium grey, coarse-grained matrix. In some areas, the Lower Boulder exhibits two reef bands separated by an internal quartzite. Mineralization typically occurs as buckshot pyrite near the bottom contact and disseminated pyrite around pebbles and within the matrix

4.3 Stratigraphy of Emerald Shaft Gold Mine

The analysis of stratigraphic profiles determines the competency of the rock mass and lithological contacts between rock strata and reef bands, pinpointing exact locations for mining. Figure 4.6 illustrates the stratigraphic unit of Emerald Shaft Gold Mine including the lithological contacts and the location of reef bands in which collar and survey data including coordinates, elevation, depth, dip, and azimuth are presented in Table 4.1. The area is made up of sedimentary and metamorphic rock types. The borehole data locally around Emerald Shaft Gold Mine was used to delineate any variability of the hangingwall and footwall rock types to the Kimberly reefs. Core logging observations from boreholes reveal variations in hangingwall and footwall composition across the Emerald Shaft Gold Mine. The hangingwall comprises mudrock, shale, and argillaceous quartzite, whereas the footwall comprises argillaceous and siliceous quartzite. The distinct hangingwall and footwall lithologies exhibit different rock mass behaviour. The identification of the rock mass as soft or hard in the hangingwall or footwall yields important information for the accurate selection of the support type and support strategies.

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The stratigraphic observations from the borehole data have revealed variations in gold reef thickness across different batteries, as shown in Figure 4.7. Battery C exhibits a relatively narrow reef thickness, ranging from 1.3 to 1.4m. In contrast, Battery A and Battery B display thicker reef thickness, ranging from 1.2 to 2.4m and 1.6 to 2.4m, respectively. The variations in reef thickness influence the design of stoping widths and support systems.

Table 4.1: The collar and survey data of the drilled boreholes of Emerald Shaft Gold Mine

Hole ID	Easting (m)	Northing (m)	Elevation (m)	Depth (m)	Dip (º)	Azimuth (º)
EMRBH1	575263	7110251	1734	76.61	45	130
EMRBH2	575178	7110335	1737	76.71	41	90
EMRBH3	575189	7110435	1740	75.40	-60	55
EMRBH4	574987	7110293	1735	61.80	28	284
EMRBH5	575221	7110394	1738	63.88	-55	55
EMRBH6	575295	7110137	1743	71.64	-18	90
EMRBH7	575363	7110342	1736	83.21	33	95
EMRBH8	575037	7110185	1736	64.08	68	270

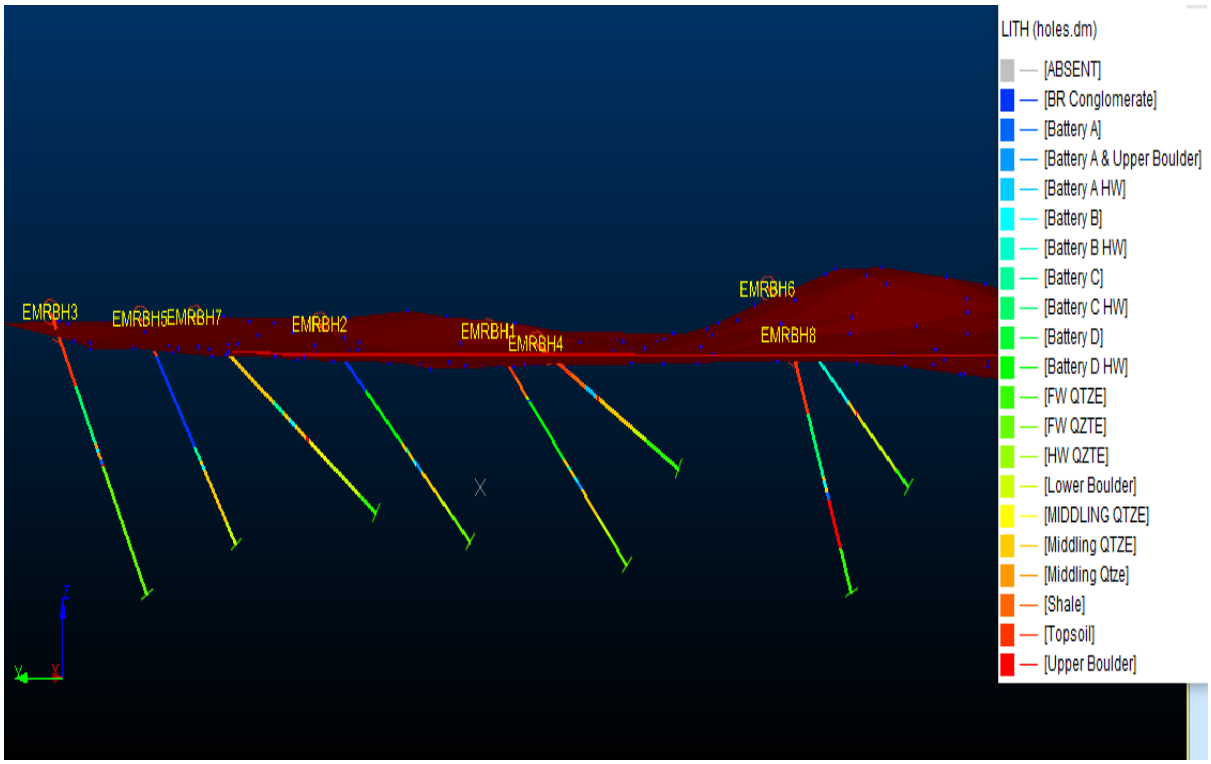


Figure 4.6: Stratigraphic unit of Emerald Shaft Gold Mine

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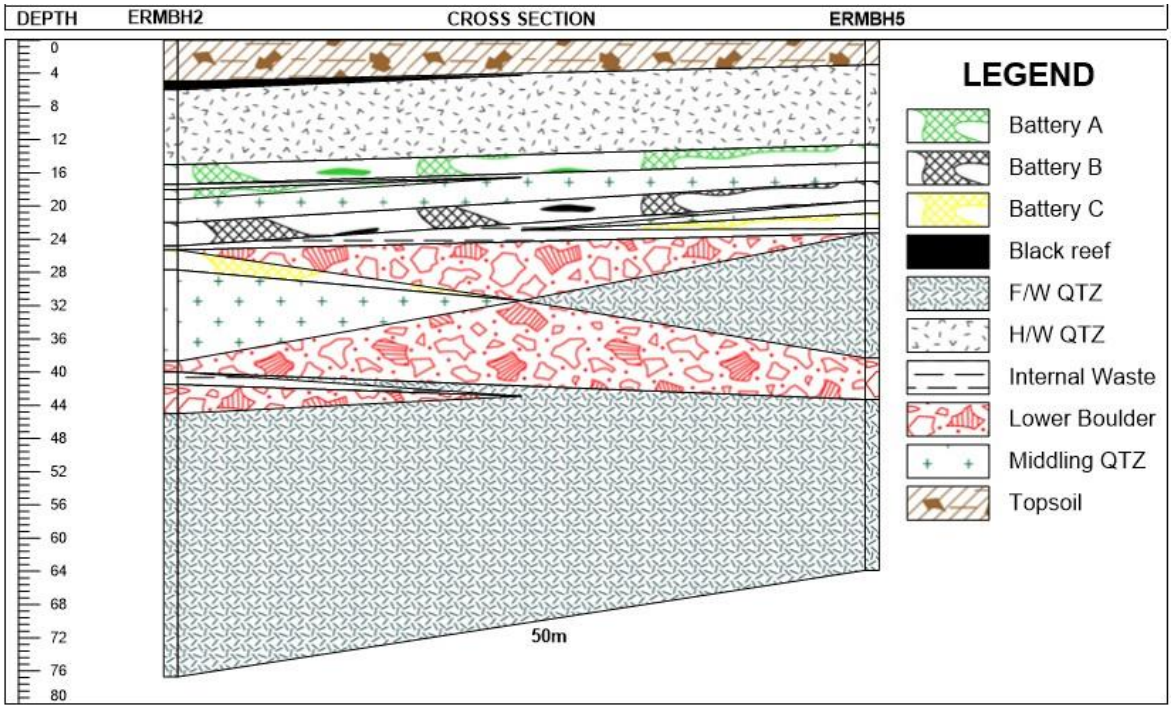


Figure 4.7: Correlation of EMRBH 2 and 5 with lithological boundaries and reef band thicknesses

4.4 Rock Strength

Rock strength plays a critical role in mining operations, significantly influencing the stability, efficiency, and safety of extraction activities. The strength of the rock affects the stability of mine excavations, with weaker rocks requiring additional support, increasing costs, and slowing production, while stronger rocks reduce the need for support, enhancing safety and efficiency.

4.4.1 Field Rock Mass Test Estimation

During field tests conducted according to the International Society for Rock Mechanics (ISRM) 1978 guidelines for field rock and soil strength identification, the following observations were made: The topsoil was indented with difficulty by a thumbnail. Mudstone could be peeled with difficulty using a pocketknife. Shale was even harder, impossible to scrape or peel with a pocketknife, but could be broken with a single strike of a geological hammer. Argillaceous quartzite samples required 6 blows to fracture, while siliceous quartzite samples required 18 blows. The estimated strength of soil and rock encountered at the Emerald Shaft Gold Mine operation is presented in Table 4.2.

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Table 4.2: Estimated UCS of soil and rock types according to ISRM (1978)

Term		Field identification	Approx. σ_c (MPa)
Lithology	**Competency**		
Topsoil	Hard clay	– Indented with difficulty by thumbnail.	> 0.50
Mudstone	Weak rock	– Peeled with difficulty by pocketknife	5-25
Shale	Medium rock	– Resists pocketknife. The sample can be fractured with a single firm geological hammer blow.	25-50
Argillaceous Quartzite	Strong rock	– Fractured with 6 hammer blows.	50-100
Siliceous Quartzite	Very strong rock	– Fractured with 18 hammer blows.	100-250

4.4.2 Laboratory Rock Mass Test Results

The assessment of the rock properties from the laboratory work enables researchers or rock/ geotechnical engineers to differentiate between the weak and strong rock masses. The determination of the engineering properties of the rock mass including UCS, cohesion, friction angle, young modulus, and poison ratio informs rock engineering design and stability analysis for mining, tunneling, and other geological applications.

The laboratory test results provide valuable insights into the rock engineering properties of the hangingwall, ore zone, and footwall rocks as shown in Table 4.3. The Unconfined Compressive Strength (UCS) values indicate that the rocks can withstand significant compressive stress before failing, with the ore zone rock exhibiting moderate UCS values (55MPa). The hangingwall rocks are relatively weak to strong, with UCS values ranging from 28.5 to 85MPa, while the footwall rocks are the strongest, with UCS values of 120MPa, consistent with findings by Tannant and Kaiser (2014).

In terms of deformation, Poisson's ratio (v) values indicate that the ore zone rocks exhibit moderate deformation, while the hangingwall rocks are more deformable to least deformable, and the footwall rocks are the least deformable. Young's modulus (E) values indicate that the ore zone rocks are relatively stiff, while the hangingwall rocks are less stiff (mudrock and shale) hence hangingwall argillaceous quartzite is very stiff, and the footwall rocks are very stiff.

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The friction angle (φ) and cohesion (c) values indicate that the ore zone rocks exhibit moderate resistance to shear and inherent strength, while the hangingwall rocks are weaker, and the footwall rocks are the strongest. The unit weight values indicate that the ore zone rocks have moderate density, while the hangingwall rocks are less dense, and the footwall rocks are very dense.

Table 4.3: Summary of laboratory results

Parameters	Hangingwall			Ore Zone	Footwall	
	Mudrock	**Shale**	**Arg. QTZ**	**Conglomerate**	**Arg. QTZ**	**Sil. QTZ**
UCS (MPa)	28.5	47.5	85.0	55.0	85.0	120.0
E (GPa)	4.9	6.5	27.5	17.5	27.5	29.8
Poisson's ratio (v)	0.29	0.24	0.19	0.26	0.19	0.18
Friction angleϕ (º)	28.75	33.75	41.25	36.25	41.25	42.03
Cohesion (MPa)	3.5	7.0	13.8	9.0	13.8	13.9
Unit weight (kg/m^3)	2000	2300	2750	2600	2750	2750

The triaxial test results provide valuable insights into the mechanical properties of the five rock types presented in Table 4.4. All rock types exhibit shear/tensile failure modes, indicating a tendency to fail along planes of weakness or fractures. The failure envelope, which represents the maximum confining pressure a rock can withstand before failing, varies significantly among the rock types. Siliceous quartzite and argillaceous quartzite exhibit the highest failure envelopes (18.3 MPa), indicating high strength and stability, similar to results reported by Villaescusa *et al*. (2013. In contrast, Mudrock and Shale display relatively low failure envelopes (3.4 MPa and 9.2 MPa, respectively). The principal stresses vary significantly between rock types, with siliceous quartzite displaying the highest stresses (185.8 MPa) and mudrock showing the lowest stresses (27.9 MPa). The stress ratios range from 10.9 (mudrock) to 8.4 (siliceous quartzite), indicating a moderate to high degree of stress anisotropy, as noted by Grimstad and Barton (1993). Appendices IA and IB display ten testing trials each for different rock types while showcasing their variance in major and minor principal stress measurements demonstrating rock mass heterogeneity. The wide range of conditions provides essential knowledge about both rock mass mechanics and their effects on mine stability and design.

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Table 4.4: Uniaxial Compressive Strength (UCS) and Failure Envelope parameters for different rock types

Rock type	σ1 (MPa)	σ2 (MPa)	σ3 (MPa)	Failure envelope (σ3 max: MPa)
Mudrock	27.9	7.4	2.6	3.4
Shale	83.4	21.9	7.3	9.2
Conglomerate (Ore)	57.6	16.3	5.6	6.5
Argillaceous quartzite	127.76	35.21	12.70	18.3
siliceous quartzite	185.8	48.6	22.0	18.3

4.5 Ground Conditions of Emerald Shaft Gold Mine

The ground conditions of shallow hardrock gold mines in the Witwatersrand Basin, such as the Emerald Shaft Gold Mine, are significantly influenced by geological discontinuities. These discontinuities are broadly categorized into structural and compositional types. Structural discontinuities, including joints, faults, and weak bedding planes, and compositional discontinuities, such as dykes and sills, play a crucial role in determining the overall rock mass behaviour and stability of excavations. At Emerald Shaft Gold Mine, the presence and characteristics of these geological structures were assessed within three distinct zones: MR, 1LE, and 1LW. These zones form three structural domains that are critical in understanding the mine's geotechnical behaviour and ensuring the stability of underground excavations. However, some areas were inaccessible, highlighted in red in Figure 4.8, due to poor conditions, closures, and Fall of Ground (FOG) incidents.

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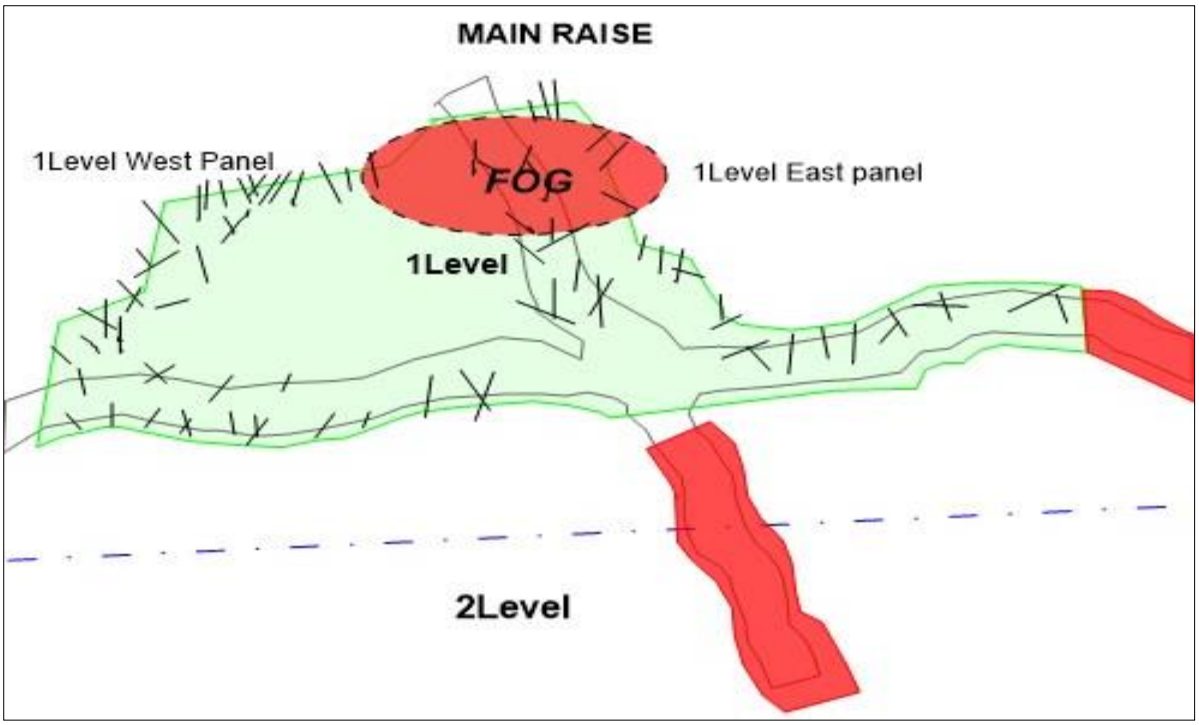


Figure 4.8: Major structures mapped in the underground decline. Red highlighted areas were inaccessible

4.5.1 Geological Characterization of the Main Raise Zone

According to the mapping and logging data, the Main Raise (MR Zone) comprises four joint sets with distinct orientations. Joint sets 2 and 4 are very steep to vertical, dipping at 85° ± 5° and 80° ± 10°, respectively. Joint set 1 has a moderate steepness, dipping at 70° ± 20°, whereas joint set 3 is shallow dipping, inclined at 30° ± 15° relative to the hangingwall. These orientation variations are crucial for understanding rock mass behaviour and potential hazards, as outlined in Table 4.5.

Table 4.5: Joint sets based on orientation in MR Zone

Joint sets	Dip	Dip range	Dip Direction	Dip range
J1	70	+/- 20	300	+/- 15
J2	85	+/- 5	210	+/- 15
J3	30	+/- 15	120	+/- 20
J4	80	+/- 10	240	+/- 30

The joint persistence at the MR Zone is approximately 5m, terminating at intersections with a wide spacing of around 8m. However, the persistence of joint sets was reduced where intersecting joints caused joint termination. The intersection of multiple joint sets (notably sets 1 and 4 and sets 2 and 4) creates blocky mining ground and unfavourable

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hangingwall conditions. Furthermore, increased fracturing and water seepage were observed at intersection areas, exacerbating potential stability issues.

In the MR Zone, joint sets located at the entrance of the main raise, extending 15 meters downward, were found to be filled with a serpentine material as illustrated in Figure 4.9. This material, comprising clay and mica minerals, is prone to dissolution upon contact with water, thereby reducing the strength and cohesion of the joints. This can lead to horizontal or vertical movement due to the slippery surface between the joints. However, from 15 meters down to the haulage entrance, no infilling materials or water presence were observed on the joints. The joint sets exhibited roughness characteristics ranging from planar to waviness.

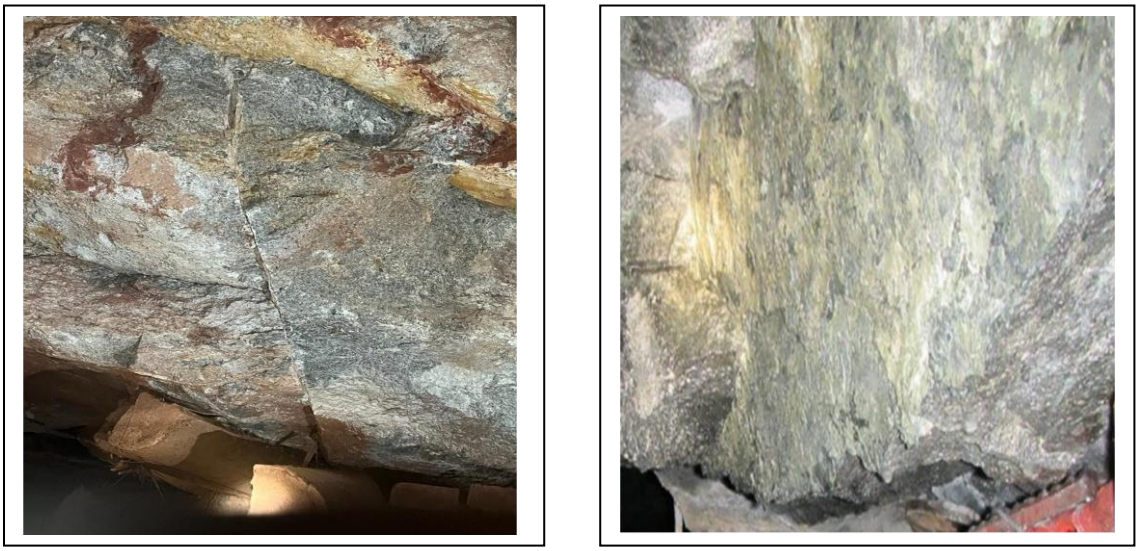


Figure 4.9: Serpentine infill on a rock plane noted by brownish to greenish colour

During the investigation period, two major normal faults were intersected along the main raise, as shown in Figure 4.1 (main plan). Faults displayed evidence of sliding shears or shear displacement. In contrast, other geological features such as dykes, sills, and folding were not intersected. The characteristics of the two faults are presented in Table 4.6. Notably, both faults have a down-throw of 2m and trend in a Southwest to Northeast direction. However, variations were observed in the dip angle and deformation of the rock mass. The rock mass near the fault plane showed higher fracture frequency and this frequency decreased steadily as the distance grew from the fault plane.

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Table 4.6: The characteristics of faults At MR Zone

Fault information	**Fault 1**	**Fault 2**
Type	Normal Fault	Normal Fault
Location	K106 from the collar of raise	K107 from the collar of raise
Orientation	SW-NE, 30º Dip	SW-NE, 45º Dip
Displacement	2m	2m
Rock deformation	Bending in adjusted rock	Bending and in adjusted rock
Mineralization (infill)	Quartz-calcite vein	Quartz-feldspar pebbles

Typically, the condition in the main raise of Emerald Shaft Gold is greatly influenced by join sets and their orientation, and the presence of normal faults. However, the collar of the raise comprised of weak unconsolidated materials and weak rock mass (approximately 6m thick) shown in Figure 4.10. The scaling at the collar of the raise and the seepage of the surface were observed such negatively affect the stability of the excavation.



Figure 4.10: Deformed Main Raise collar: Irregular shape Due to weak rock mass degradation

4.5.2 Geological Characterization of the 1LE Zone

The 1Level East (1LE) Zone, where gold-bearing conglomerate extraction is underway, features four distinct joint sets (J1-J4) with varying orientations. Joint set 1 is steep (75° ± 10°), joint set 2 is moderate (45° ± 20°), joint set 3 is shallow or sub-

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parallel to the hangingwall (35° ± 15°), and joint set 4 is moderate to steep (60° ± 10°), as summarised in Table 4.7. The predominantly sub-vertical to vertical orientation of joint sets 1, 2, and 4 relatives to the panel face poses significant stability concerns for the stope. Furthermore, observations of scaling and wedging at the panel edge highlight potential rock mass instability.

Table 4.7: Joint sets based on orientation in 1LE Zone

Joint sets	Dip	Dip range	Dip Direction	Dip range
J1	75	+/- 10	300	+/- 25
J2	45	+/- 20	50	+/- 15
J3	35	+/- 15	175	+/- 38
J4	60	+/- 10	45	+/- 15

The 1LE Zone exhibits complex jointing, with three intersecting joint sets: Joint sets 1 and 4, joint sets 2 and 4, and joint sets 2 and 3. This intersection pattern typically indicates poor ground conditions. The joint sets have a uniform spacing of 4m and an average persistence of 8m, resulting in a moderately to highly fractured rock mass. The close spacing and long persistence amplify the interaction between joint sets, particularly joint sets 1 and 4, and joint sets 2 and 4, raising significant concerns regarding rock mass stability. This complex geological structure necessitates careful monitoring and mitigation measures to ensure safe and efficient mining operations.

The compositional structure (dyke) intersected approximately 7m from the head of the 1Level 2E panel (scan line mapping) shown in Figure 4.11. The geological feature intrudes/cuts across the overlaying and underlying rock strata (shale, argillaceous quartzite, and gold-bearing conglomerate. The area affected by the dyke which is the zone of thickness is approximately 1.5m.

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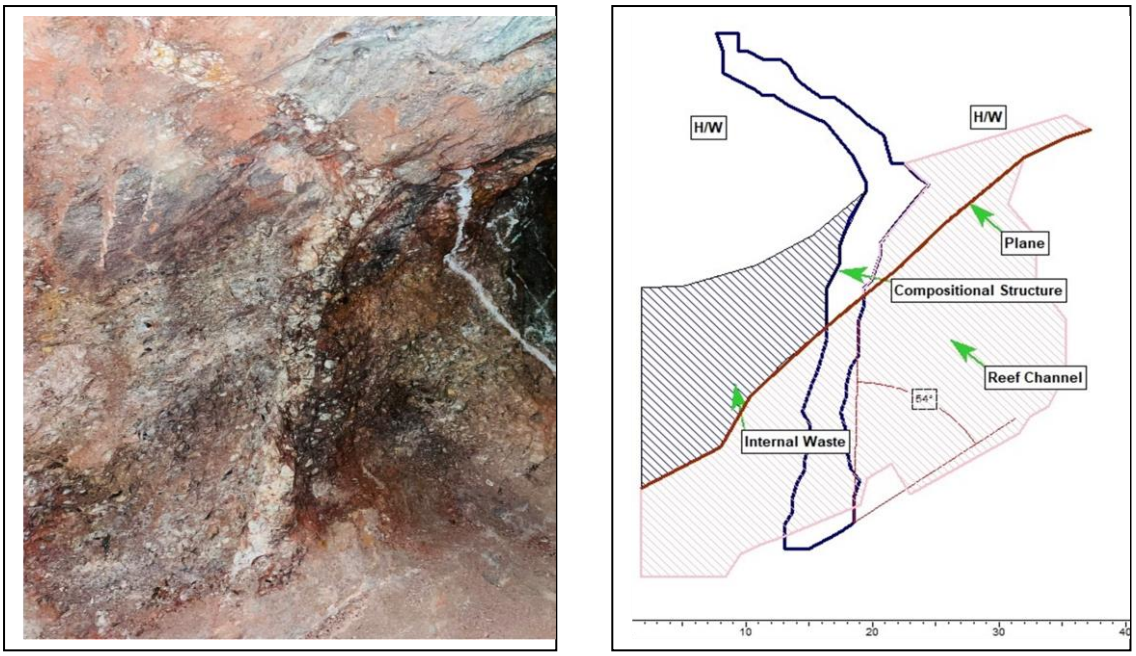


Figure 4.11: The compositional structure (dyke) Mapped at 1LE Zone

Structural mapping data summarised in Table 4.8 has identified a dyke located 7 meters from the head of the 3E panel in the 1LE zone. This quartz porphyry dyke has a moderate dip of 54° and a thickness of approximately 0.8 meters. The dyke exhibits calcite alteration, indicating potential hydrothermal activity. Its presence and orientation have implications for local rock mass stability.

Table 4.8: summary of the mapping parameter

Information	**Dyke**
Location	1LE (7m from the head of the 3E panel)
Orientation	54º Dip
Thickness	0.8m
Composition	quartz porphyry
Alteration	Calcite

4.5.3 Geological Characterization of the 1LW Zone

The 1Level West (1LW) Zone, similar to MR and 1LE Zones comprises four joint sets presented in Table 4.9. The joint sets (J1-J4) characterize the rock mass, exhibiting diverse orientations. Joint set 1 (J1) dips at 70° (±20°) in a west-northwest direction (295° ± 30°), while joint set 2 (J2) is steeply dipping at 85° (±5°) south-westward (205° ± 15°). Joint set 3 (J3) has a shallow dip of 25° (±10°) in a northwest direction (310° ±

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35°), contrasting with joint set 4 (J4), which dips at 80° (±10°) south-westward (230° ± 15°). These joint sets display varied dips and directions, indicating a complex rock mass structure at 1LW Zone.

Table 4.9: Joint sets based on orientation in 1LW Zone

Joint sets	Dip	Dip range	Dip Direction	Dip range
J1	70	+/- 20	295	+/- 30
J2	85	+/- 5	205	+/- 15
J3	25	+/- 10	310	+/- 35
J4	80	+/- 10	230	+/- 15

Potential intersections occur between J1 and J2, J1 and J4, and J2 and J4, increasing rock fragmentation. At the intersection, water seepage was observed from the hangingwall. Furthermore, clay minerals' alteration (oxidation) was observed in areas denoted by yellowish and greenish colour. The rock mass is moderately to highly fractured with a uniform spacing of 5m and an average persistence of 8m. The moderate spacing and long persistence amplify the interaction between joint sets, particularly J1 and J2, and J2 and J4. This raises concerns about rock mass stability. Lithology contact acted as a barrier or conduit for fluid flow, influencing joint persistence and rock mass behaviour.

The ground condition in the 1LW Zone is influenced by the complex geological structures, including faults and dykes, which impact regional mine stability. Two major normal faults and multiple dykes are present, with similar characteristics observed in the MR Zone. The diorite dykes trend in a northwest-southeast direction (Figure. 4.1). Notably, a reverse fault was intersected, dipping at 35° (Figure. 4.12), resulting in localized ground gain. The surrounding area (within a 3m radius) exhibits weak rock mass characteristics, manifested by overbreak during blasting operations, leading to increased stope width in the panel.

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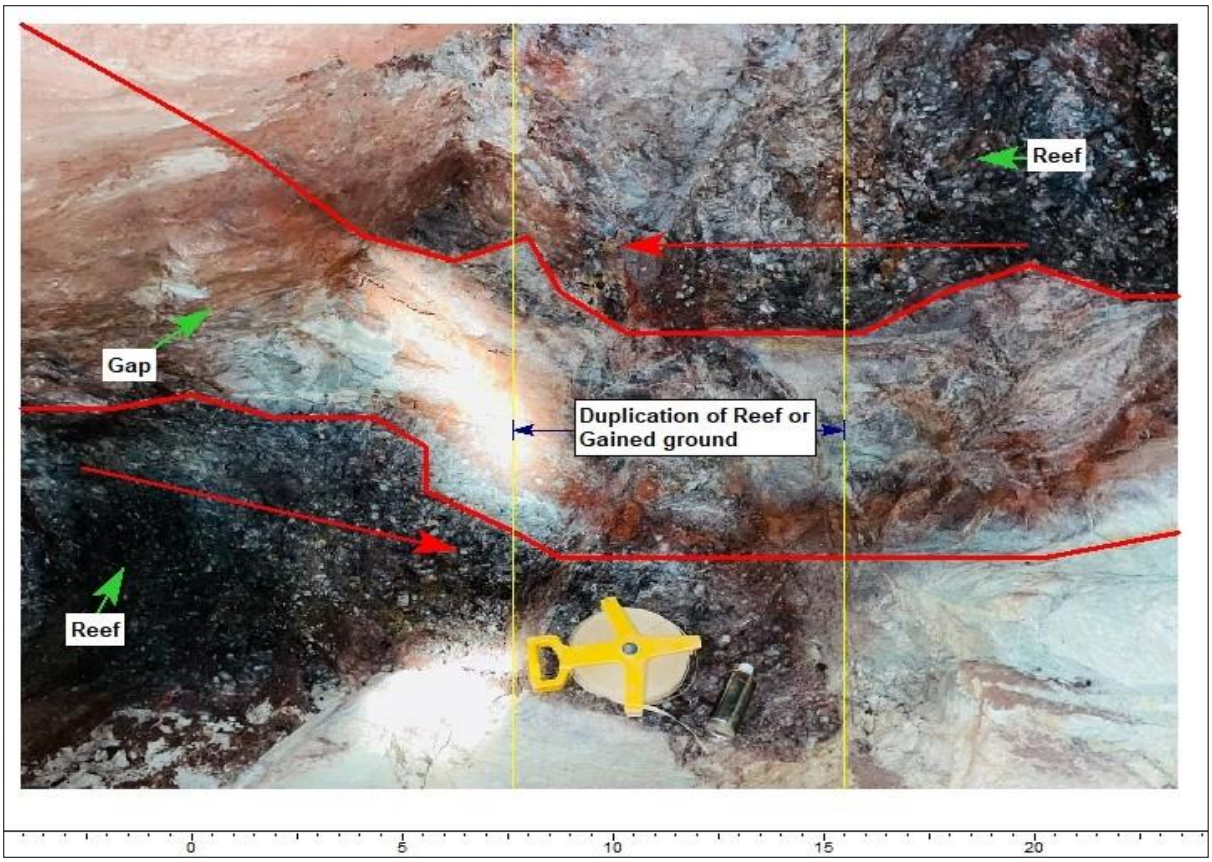


Figure 4.12: The reverse fault mapped at 2W panel

4.6 Rock Mass Quality

To evaluate the rock mass quality of the Emerald Shaft gold mine's raise and panels, empirical methods incorporating three widely recognised rock mass classification systems, as recommended by Hoek (2002), were utilised. Specifically, the study employed the Rock Mass Rating (RMR) system, Q-system, and Geological Strength Index (GSI) in which field measurements and observations are captured and presented in Appendix III. By combining these industry-standard classification systems, a thorough understanding of the rock mass quality in this section was achieved, providing valuable insights for mining operations.

4.6.1 Rock Mass Rating (RMR)

The Rock Mass Rating (RMR) system, also known as the geomechanical classification, is an empirical approach developed by Bieniawski (1976) to assess the quality of rock masses. In this study, the 1989 version of the RMR classification system was adopted to evaluate the rock mass quality in the MR, 1LE, and 1LW Zones. This

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system considers six key parameters: Uniaxial Compressive Strength (UCS) of the rock mass, Rock Quality Designation (RQD), spacing of discontinuities, condition of discontinuities, groundwater conditions, and orientation of discontinuities. These parameters provide a comprehensive framework for assessing the rock mass quality and informing engineering design decisions.

4.6.1.1 Rock Mass Rating at MR Zone

The calculated RMR value of 56 categorizes at MR Zone the rock mass as "fair" quality, indicating moderate rock conditions with structural deficiencies and weathering (Bieniawski, 1989). Although the intact rock strength is relatively high (85 MPa), the presence of discontinuities with moderate persistence, aperture, and infilling, combined with moderate weathering, compromises the overall rock mass quality summarised in Table 4.10. This is consistent with findings by Villaescusa et al. (2013), who noted that discontinuity characteristics significantly impact rock mass stability.

Table 4.10: Summary of the RMR values at MR Zone

Parameter		**Description and values**	**Rating**
Strength of intact rock (UCS)		85MPa	7
RQD (%)		70% (Fair)	13
Spacing of discontinuity		8m	20
Condition of discontinuity	Persistence Aperture Roughness Infilling Weathering	5m 4mm Slightly rough Soft filling Moderately weathered	2 1 3 2 3
Ground water condition		Damp	10
Orientation (Effects in tunnelling)		Drive against dip (45-90º), fair	-5
RMR		**Fair rock**	**56**

4.6.1.2 Rock Mass Rating at 1LE Zone

The RMR value of 60 at 1LE Zone indicates that the rock mass is of "fair" quality, suggesting moderate stability and requiring some support measures presented in Table 4.11. This classification is largely influenced by the relatively close spacing of discontinuities (4m) and smooth discontinuity surfaces, which may compromise the rock mass's overall integrity. However, the high strength of the intact rock (80 MPa) and fair RQD value (70%) provide some mitigating factors. The damp groundwater condition and unfavourable orientation for stoping also contribute to the moderate

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RMR value. Overall, the RMR of 60 suggests that careful design and construction considerations are necessary to ensure stability, particularly in stoping applications where the rock mass may be subjected to significant stress.

Table 4.11: Summary of the RMR values at 1LE Zone

Parameters		**Description and values**	**Rating**
Strength of intact rock (UCS)		80MPa	7
RQD (%)		70% (Fair)	13
Spacing of discontinuity		4m	20
Condition of discontinuity	Persistence	8m	2
	Aperture	4mm	1
	Roughness	Smooth	1
	Infilling	None	6
	Weathering	Slightly weathered	5
Ground water condition		Damp	10
Orientation (Effects in mining)		Drive against dip (45-90º), fair	-5
RMR		**Fair rock**	**60**

4.6.1.3 Rock Mass Rating at 1LW Zone

The RMR value of 55 at 1LW Zone indicates a "fair" rock mass quality Presented in Table 4.12, highlighting moderate stability and a need for targeted support measures. The reduced RMR value compared to previous assessments is primarily due to the decrease in RQD (65%) and the wet groundwater condition, which increases the potential for instability. Although the strength of the intact rock remains relatively high (78 MPa), the presence of soft filling and slightly weathered discontinuities contributes to the overall downgrade. The fair orientation for stoping provides some relief, but careful planning is still necessary to address potential rock mass failures.

Table 4.12: Summary of the RMR values at 1LW Zone

Parameters		**Description and values**	**Rating**
Strength of intact rock (UCS)		78MPa	7
RQD (%)		65% (Fair)	13
Spacing of discontinuity		5m	20
Condition of discontinuity	Persistence	8m	2
	Aperture	5mm	1
	Roughness	Slightly rough	3
	Infilling	Soft filling	2
	Weathering	Slightly weathered	5
Ground water condition		Wet	7
Orientation (Effects in mining)		Drive against dip (45-90º), fair	-5
RMR		**Fair rock**	**55**

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4.6.2 Q-system

The Q-system, also known as the Tunnelling Quality Index, was developed by Barton et al. (1974) to evaluate rock mass characteristics and design appropriate tunnel support systems. In the Q-system classification, the rock mass is classified based on the six parameters such as the RQD index, the number of joint sets (Jn), the roughness of the weakest joint (Jr), the degree of alteration or infilling along the weakest joints (Ja), the joint water reduction factor (Jw), and the stress reduction factor (SRF) (Rochmadi and Karnawati, 2023). The fraction of RDQ/Jn is a rock block size, Jr/Ja is an inter-block shear strength, and JW/SRF is an active stress. This sub-section presents and discusses the Q-system values at MR, 1LE, and 1LW Zones using equation 2.

4.6.2.1 Q-value at MR Zone

The calculated Q-value of 0.17 at the MR Zone indicates a very poor rock mass quality, suggesting significant engineering challenges. This low Q-value is primarily attributed to the presence of multiple weakness zones, reflected in the high-stress reduction factor (SRF) of 10.0, and the adverse joint characteristics, including four joint sets and softening or low-friction clay coatings presented in Table 4.13. The relatively fair Rock Quality Designation (RQD) of 70% is overshadowed by these detrimental factors, resulting in an overall poor rock mass rating. This Q-value emphasizes the need for comprehensive support designs, reinforced excavation methods, and careful monitoring to mitigate potential instability risks.

Table 4.13: Summary of the rock mass parameters in the determination of the Q-system value at MR Zone

Parameters	**Description and values**	**Ratings**
RQD (%)	Fair	70%
Joint set number (*Jn*)	Four joint sets	15
Joint roughness number (*Jr*)	Rough or irregular, planar	1.5
Joint alteration number (*Ja*)	Softening or low-friction clay coating	4.0
Joint water Reduction (*Jw*)	Dry excavation or minor inflow	1.0
Stress Reduction Factor (*SRF*)	Multiple occurrences of weakness zone	10.0
Q-system (Equation 2)	**Very poor rock mass quality**	**0.17**

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4.6.2.2 Q-value at 1LE Zone

The Q-value of 0.62 at 1LE Zone indicates a rock mass quality that falls within the "poor" to "fair" category, suggesting a moderate level of stability. This rating is influenced by the fair Rock Quality Designation (RQD) of 70% and the presence of four joint sets but is positively impacted by the smooth and unaltered joint conditions, dry excavation environment, and relatively low-stress reduction factor presented in Table 4.14. Overall, this Q-value implies that the rock mass can be excavated safely with standard support measures but may require additional reinforcement in areas with higher stress concentrations or more intense jointing.

Table 4.14: Summary of the rock mass parameters in the determination of the Q-system value at 1LE Zone

Parameters	**Description and values**	**Ratings**
RQD (%)	Fair	70%
Joint set number (*Jn*)	Four joint sets	15
Joint roughness number (*Jr*)	Smooth, planar	1.0
Joint alteration number (*Ja*)	Unaltered joint wall surface	1.0
Joint water Reduction (*Jw*)	Dry excavation or minor inflow	1.0
Stress Reduction Factor (*SRF*)	Multiple shear zone in competent rock	7.5
Q-system (Equation 2)	**Poor" to fair rock mass quality**	**0.62**

4.6.2.3 Q-value 1LW Zone

The Q-value of 0.11 at 1LW Zone indicates an extremely poor rock mass quality, suggesting high instability risks and significant engineering challenges. This very low Q-value is primarily driven by multiple weakness zones and adverse joint conditions, including softening clay coatings and medium water inflow, which overshadow the fair Rock Quality Designation of 65% presented in Table 4.15. The rock mass is highly susceptible to failure, requiring comprehensive support and reinforcement measures. Specialized designs, real-time monitoring, and adaptive management are crucial to mitigate risks. Groundwater inflow and potential rock bursts or seismicity are major concerns, emphasizing the need for rigorous geological and geotechnical analysis.

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Table 4.15: Summary of the rock mass parameters in the determination of the Q-system value at 1LW Zone

Parameters	Description and values	Ratings
RQD (%)	Fair	65%
Joint set number (*Jn*)	Four joint sets	15
Joint roughness number (*Jr*)	Rough or irregular, planar	1.5
Joint alteration number (*Ja*)	Softening or low-friction clay coating	4.0
Joint water Reduction (*Jw*)	Medium in flow or pressure	0.66
Stress Reduction Factor (*SRF*)	Multiple occurrences of weakness zone	10.0
Q-system (Equation 2)	**Extremely poor rock mass quality**	**0.11**

4.6.3 Geological Strength Index (GSI)

The Geological Strength Index (GSI) rock mass classification system is employed as an alternative classification system to RMR and Q-system to determine the rock mass quality of MR, 1LE, and 1LW Zones using equation 3 of RMR_{89} value greater than 23 (Hoek, 1985). The Geological Strength Index (GSI) values across the zones indicate varying degrees of rock fabric and structural integrity presented in Table 4.16. Notably, the 1LE zone exhibits the highest GSI value of 55, suggesting better rock quality and structural competence. In contrast, the MR and 1LW zones have lower GSI values of 51 and 50, respectively, indicating more disturbed rock conditions and potential for instability. The relatively narrow range of GSI values (50-55) suggests that the rock masses are generally characterized by blocky to disturbed conditions, with some influence from jointing and fracturing indicated in the GSI chart Hoek and Marinos, 2000 (Figure 4.13). These GSI values will play a critical role in informing support design and rock reinforcement strategies to ensure safe and efficient mining operations.

Table 4.16: Summary of the GSI values at MR, 1LE, and 1LE Zones in relation to RMR_{89} values

Zones	RMR_{89}	GSI
MR	56	51
1LE	60	55
1LW	55	50

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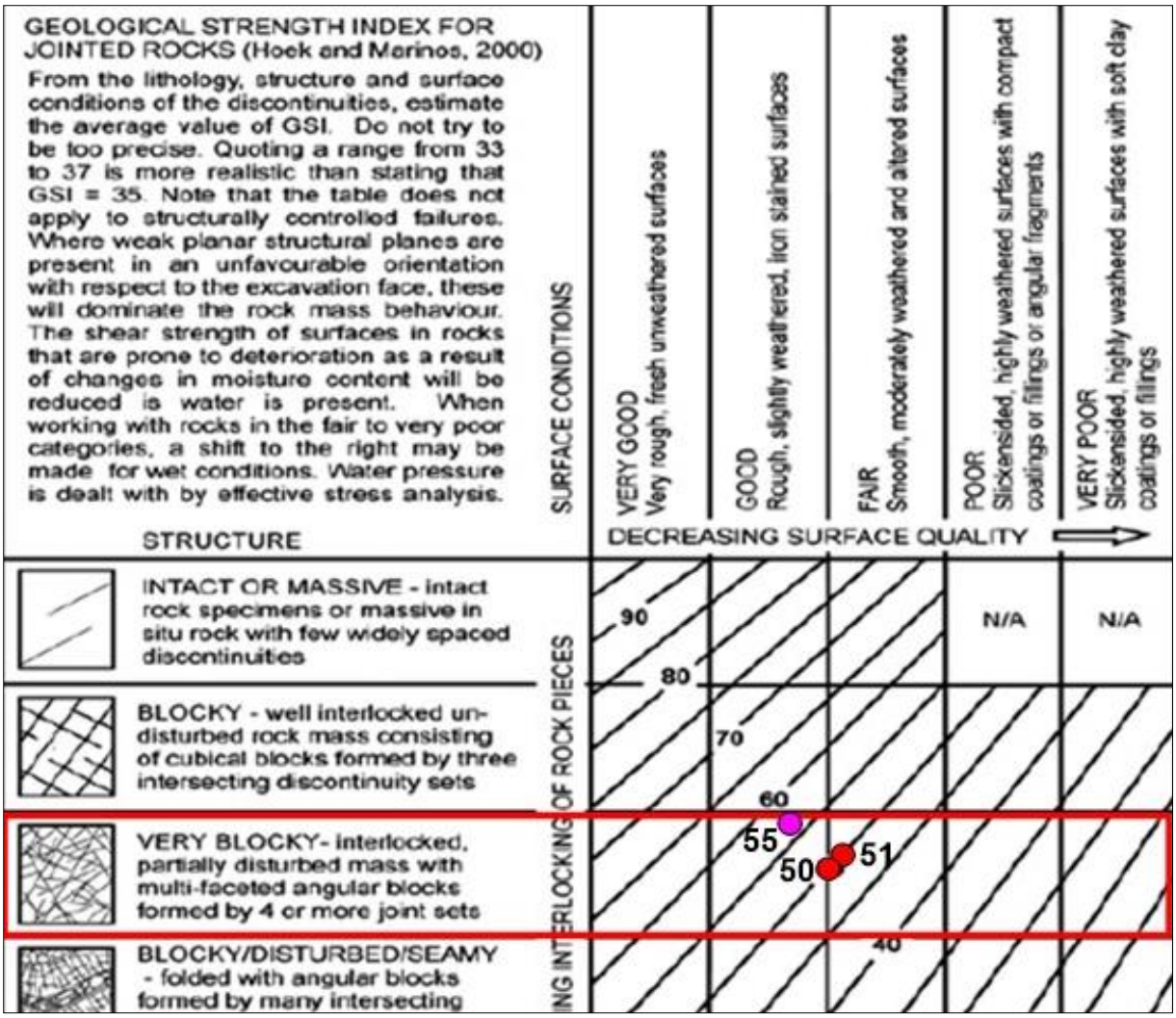


Figure 4.13: The GSI Chart summarizing the rock quality and degree of fracturing at MR, 1LE, and 1LW Zones

4.7 Geotechnical Domains

Geotechnical domains refer to areas or zones within a rock mass or soil deposit that exhibit similar geological and engineering properties, influencing the stability and behaviour of the ground (Bieniawski, 1989; Hoek & Brown, 1997; Marinos et al., 2005). The geotechnical domains identified in the study area shown in Figure 4.14, reveal distinct variations in rock mass quality, influencing the stability and extractability of ore reserves. The red domains (MR and 1LW) exhibit relatively poor rock mass quality, characterized by lower RMR and Q-values, indicating increased instability risks and support requirements presented in Table 4.17. In contrast, the pink domain (1LE) shows improved rock mass quality, with higher RMR and Q-values, suggesting more stable conditions. The Geological Strength Index (GSI) values across the domains

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also highlight differences in rock fabric and structure, with the pink domain displaying better structural integrity. These geotechnical domain variations necessitate tailored support designs and extraction strategies to ensure safe and efficient mining operations.

Table 4.17: Rock mass classification values for each geotechnical domain

Zones	Geotech Domains	RMR_{89}	Q-system	GSI
MR	Red	56	0.17	51
1LE	Pink	60	0.67	55
1LW	Red	55	0.11	50

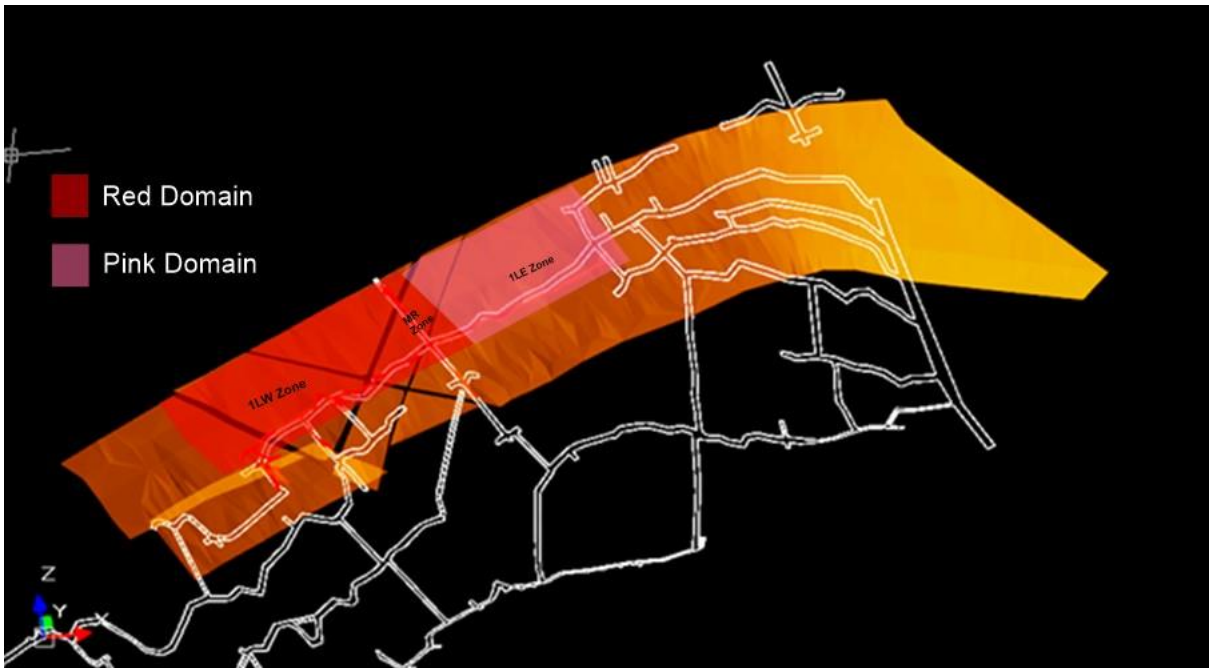


Figure 4.14: Location of geotechnical domains of Emerald Shaft Gold Mine

Regional geological features as shown in Figure 4.15, particularly dykes and faults in the 1LW zone and faults in the 1 MR Zone, significantly impact rock mass quality values, compromising the overall stability and integrity of the rock mass. These features undermine the stability and extractability of ore reserves using sub-level stoping. Dykes reduce rock mass quality, increase instability risks, and alter stress fields, potentially leading to rock bursts and water infiltration. Faults create zones of weakness, disrupt ore body continuity, and elevate seismicity risks.

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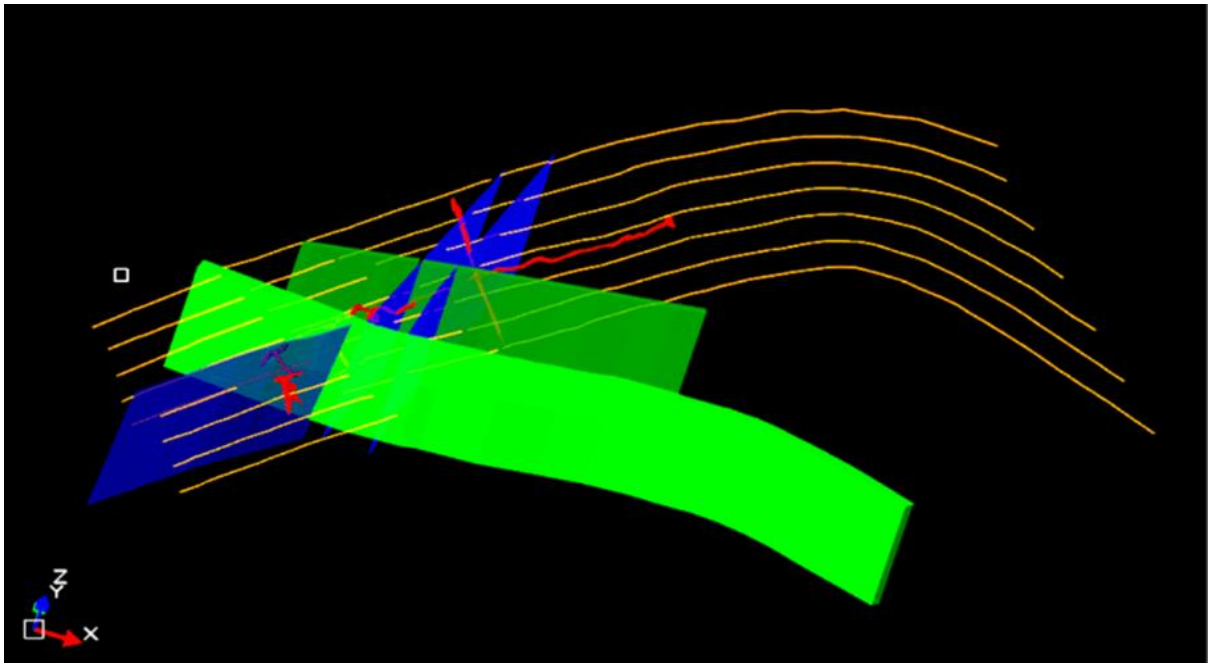


Figure 4.15: Major geological features of Emerald Shaft Gold Mine (Green = Dyke and Blue = Fault)

4.8 Stress Environment

Prior to making any excavations underground, the environment is subjected to vertical stresses caused by the weight of the overburden and the horizontal stress components. The stress environment in the region was characterized by determining the in-situ stress conditions. Analysis revealed a K-ratio of 1.5, representing the horizontal to-vertical virgin stress ratio, calculated as a function of depth (750m) and overburden weight indicating a shallow mining environment. The principal in-situ stresses exhibit a northwest-southeast (NW-SE) and Southwest-Northeast (SW-NE) orientation related to the mine grid. The horizontal stress (250°) and vertical stress (320°) orientation suggests a rotated stress field, possibly due to tectonic forces and structural complexity. Table 4.18 summarises the magnitude and orientation of the in-situ stresses determined in the quartzite rock, providing valuable insights into the regional stress regime.

Table 4.18: The magnitude and orientation of the in-situ stresses

Stress type	Magnitude (MPa)	Orientation
σ_v	20.23	320º
σ_h	30.35	250º
σ_3	12.51	140º

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4.9 Stability Analysis

The stability analysis of underground workings identifies areas requiring special attention. This comprehensive approach involves a combination of analytical and numerical methods, including the analysis of beam thickness and keyblock formations, numerical analysis of stress distributions surrounding excavations, and a review of FOG incidence records. Roof stability is influenced by multiple mechanisms, and identifying and understanding these mechanisms is crucial for ensuring roof stability. In mining environments, roof stability varies significantly due to geological differences, resulting in distinct effects in different areas (Swart and Handley, 2005).

4.9.1 Kinematic Wedge Analysis

The stability of underground excavations is significantly influenced by minor and major geological structures. Rock masses often contain fractures formed during their creation, due to temperature fluctuations, and when stresses exceed the rock's strength. To evaluate the impact of joint set orientations and persistence on wedge formations, this study utilised the Rocscience Unwedge application using the structural mapping data. The analysis incorporated a minimum safety factor of 2, aligned with hardrock mining design stability standards, to ensure reliable assessments of excavation stability.

4.9.1.1 Main Raise Wedge Analysis

The Main Raise, trending 320° and plunging 30°, is surrounded by six wedges formed by the intersection of four joint sets shown in Figure 4.16. The wedge stability analysis reveals varying levels of stability across the rock mass illustrated in Table 4.19. Without considering field stress, wedges 1 and 3 are stable, while wedge 8 is highly unstable (FoS=0.00). Wedges 2, 6, and 7 have FoS values of 38.35, 2.62, and 12.52, respectively, indicating stability, marginal stability, and conditional stability.

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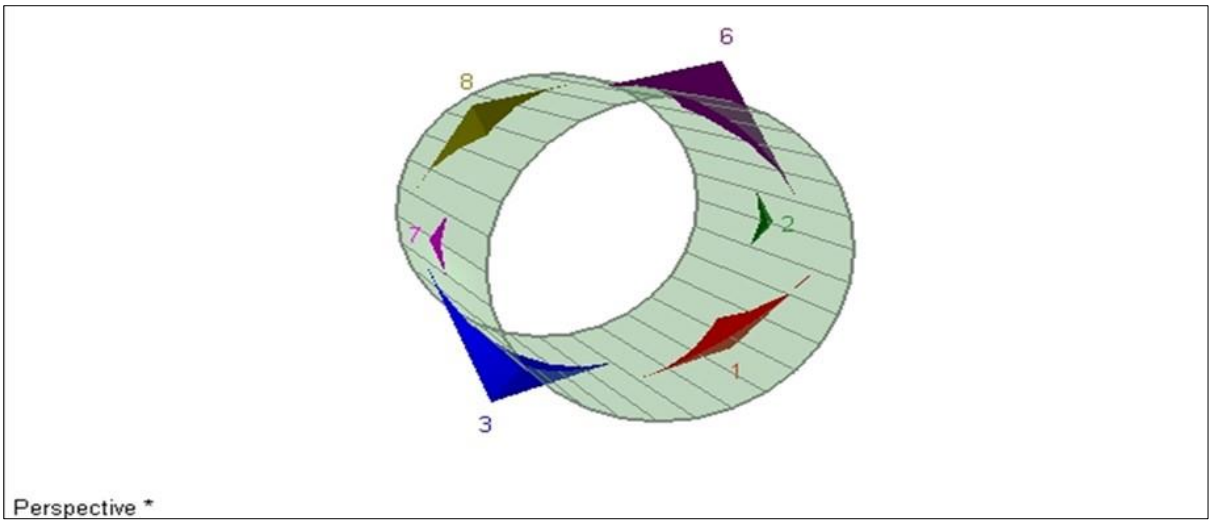


Figure 4.16: Wedges formed at the Main Raise (MR Zone)

When field stress is applied, the stability of most wedges remains unchanged, except for wedge 8, which exhibits a slight increase in FoS to 0.38, still indicating high instability although slight clamping of the wedge had occurred. Notably, the field stress does not significantly impact the overall stability of the other wedges, suggesting that the existing rock mass structure and geometry are the primary controlling factors. However, wedge 8's critical condition warrants close monitoring and potential support measures to prevent failure, while wedges 6 and 7 require regular inspection due to their relatively low FoS values.

Table 4.19: The influence of field stress on the factor of safety of the wedges formed in the Main Raise

Wedges	FoS (No stress field)	FoS (With stress field)
Lower right wedge (1)	Stable	Stable
Upper right wedge (2)	38.35	38.35
Lower left wedge (3)	Stable	Stable
Upper right wedge (6)	2.62	2.62
Lower left wedge (7)	12.52	12.52
Upper left wedge (8)	0.00	0.38

4.9.1.2 Stope Panels Wedge Analysis

The formation of unstable wedges within the 1LE and 1LW Zones' production panels significantly impacts stope performance and ore dilution. Overbreaking of panel faces results in brows, uneven hangingwalls, and poor face shape, largely due to uneven field stress distribution. As illustrated in Figure 4.17 and 4.18 respectively, wedge modelling reveals that most unstable wedges form on the hangingwall and panel face,

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highlighted in red. Notably, wedge analysis identifies unstable Wedges 8 and 4 in the 1LE and 1LW Zones, respectively, with factor of safety (FoS) values of 0.00 and 0.58. When field stresses are applied, the FoS increases to 1.38 and 1.03, respectively. To enhance stability, additional support is necessary to increase support density

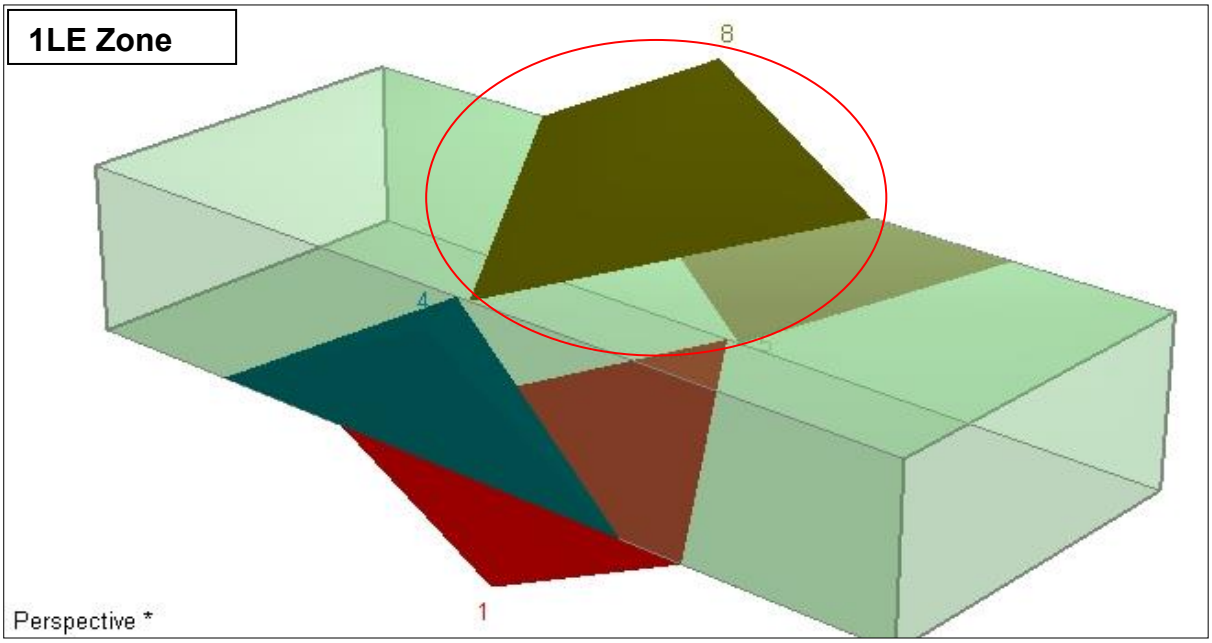


Figure 4.17: Critical wedge formation within 1LE Zone

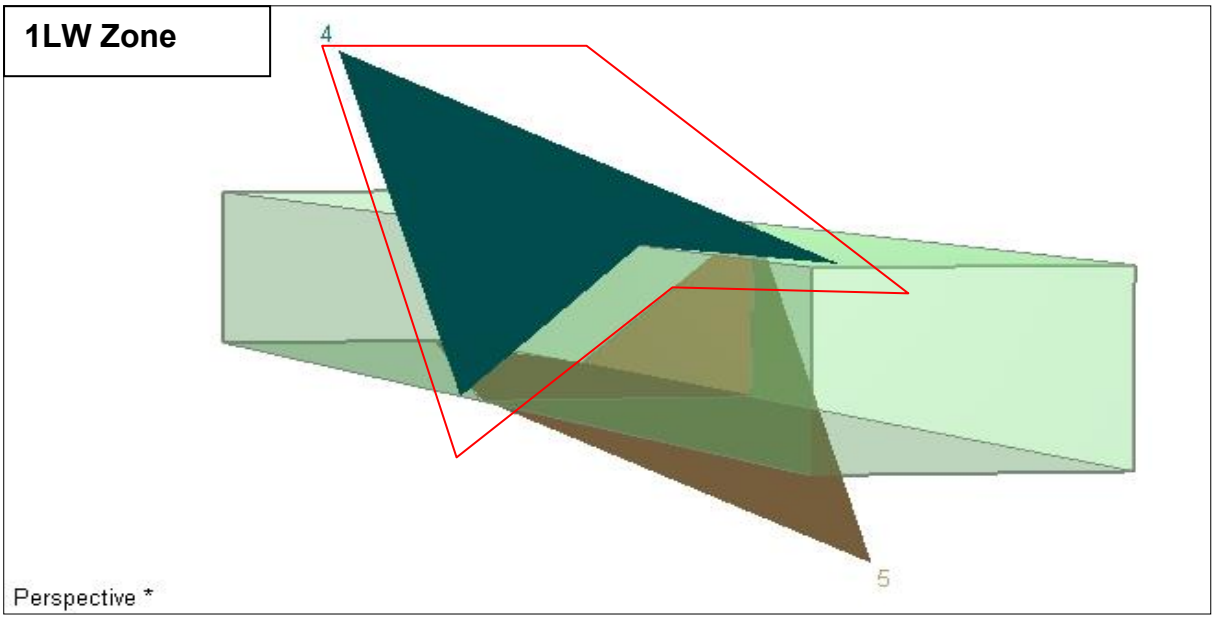


Figure 4.18: Critical wedge formation within 1LW Zone

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4.9.2 Stress Distribution Analysis

Prior to mining, in-situ stress measurements are conducted to determine the magnitude and orientation of natural virgin stress at specific depths within the mining environment. To simulate rock behaviour in response to these stresses, a stress distribution analysis was performed using Phase2 software and field stress data. The analysis utilised laboratory-derived engineering properties of the rock mass input data for elastic and elasto-plastic analyses based on Mohr-Coulomb and Hoek-Brown Failure criteria.

4.9.2.1 Sigma 1 and Z Stress Distributions

Table 4.20 presents the in-situ stress and the resultant (Field stress or absolute stress) stress as a result of developing or excavating the Main Raise. The major principal stress sigma 1 (23.23MPa) is distributed equally in the ground before mining. However, the stress changes when the raise is fully erected or excavated. As shown in Figure 4.19, in the NE-SW direction, the high stress levels are indicated by red and yellow contours.

Table 4.20: Major principal stress (Sigma 1) distribution comparison: In-situ and excavated raise conditions

In-situ state		**Final stage (Excavated Raise)**	
Distance (m)	**Sigma 1 (MPa)**	**Distance (m)**	**Sigma 1 (MPa)**
0	23.23	0	48.71
1.82	23.23	1.82	33.85
3.63	23.23	3.63	27.93
5.45	23.23	5.45	23.64
7.26	23.23	7.26	23.29
9.08	23.23	9.08	23.02
10.89	23.23	10.89	22.91
12.71	23.23	12.71	22.87
14.52	23.23	14.52	22.91
16.34	23.23	16.34	22.94

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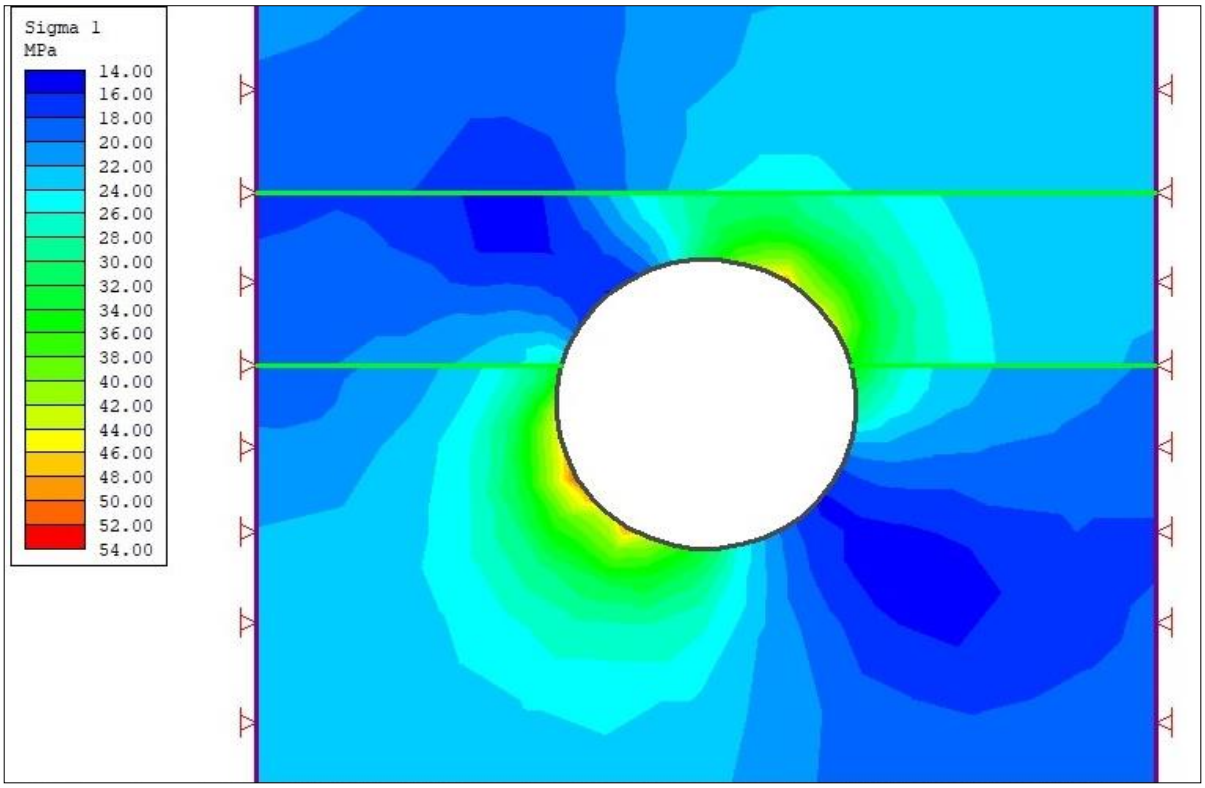


Figure 4.19: Distribution of sigma 1 stress surrounding the Main Raise

The stress distribution surrounding the raise indicates a high-stress concentration near the boundary, with Sigma 1 values decreasing rapidly from 48.71MPa at the boundary to 33.85MPa at 1.82m shown in Figure 4.20. Beyond this point, the stress decreases gradually, reaching a relatively stable value of around 23-24MPa between 5-16m. Notably, the in-situ Sigma 1 value of 23.23MPa falls within this stable range, suggesting that the raise is likely in a stable condition at distances beyond 5m.

However, the high-stress concentration near the raise boundary may require additional support or reinforcement to prevent rock mass failure. The stress relaxation zone, spanning 1.82-5m, is also prone to deformation or failure and should be monitored closely. Overall, the data suggest that the raise is experiencing significant stress redistribution due to excavation, but the far-field stress appears to be relatively stable.

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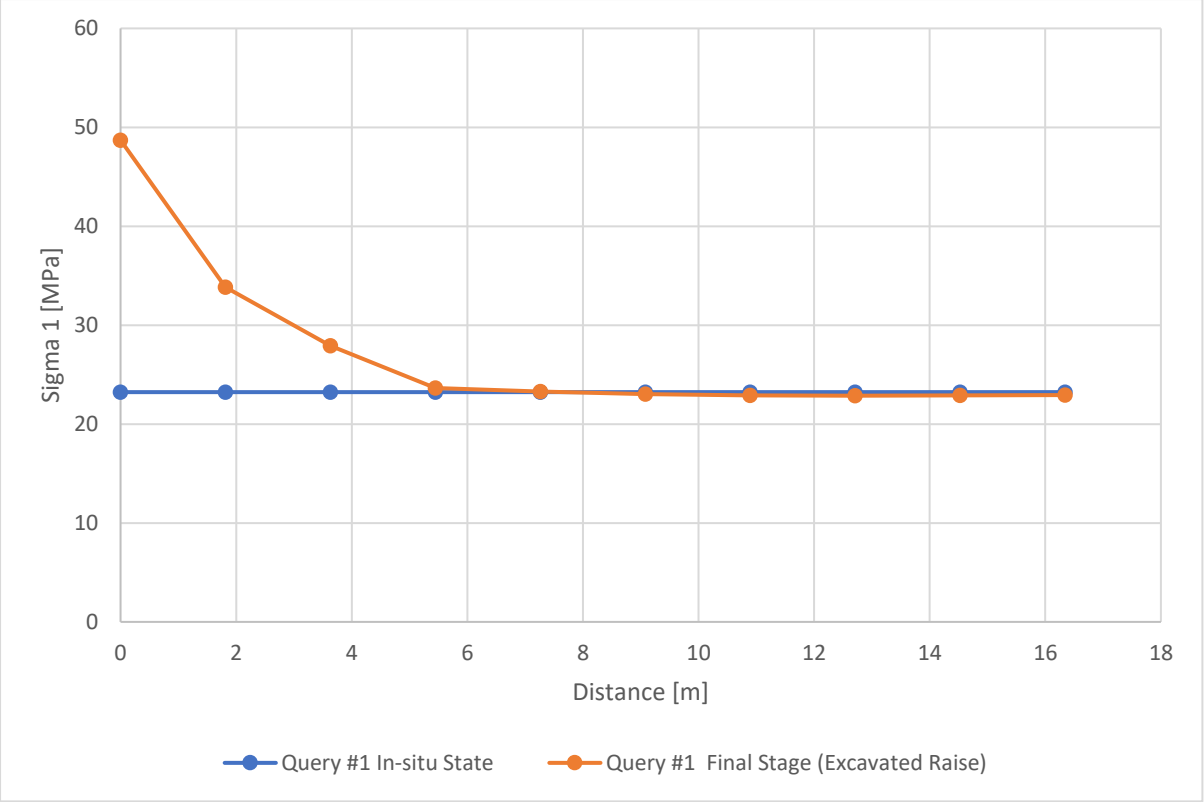


Figure 4.20: Change in Sigma 1 as moving away from the excavation

Table 4.21 presents the in-situ stress and the resultant field stress sigma Z as a result of excavating the Main Raise. vertical stress (sigma Z of 30.51MPa) is distributed equally in the ground before mining. However, the stress changes when the raise is excavated. As shown in Figure 4.21, similar to sigma 1, the stress concentration increases in the NE-SW direction.

Table: 4.21: Sigma Z distribution comparison - In-situ and excavated raise conditions

In-situ state		**Final stage (Excavated Raise)**	
Distance (m)	**Sigma Z (MPa)**	**Distance (m)**	**Sigma Z (MPa)**
0	30.51	0	34.10
1.82	30.51	1.82	31.79
3.63	30.51	3.63	30.96
5.45	30.51	5.45	30.111
7.26	30.51	7.26	30.10
9.08	30.51	9.08	30.09
10.89	30.51	10.89	30.09
12.71	30.51	12.71	30.13
14.52	30.51	14.52	30.17
16.34	30.51	16.34	30.20

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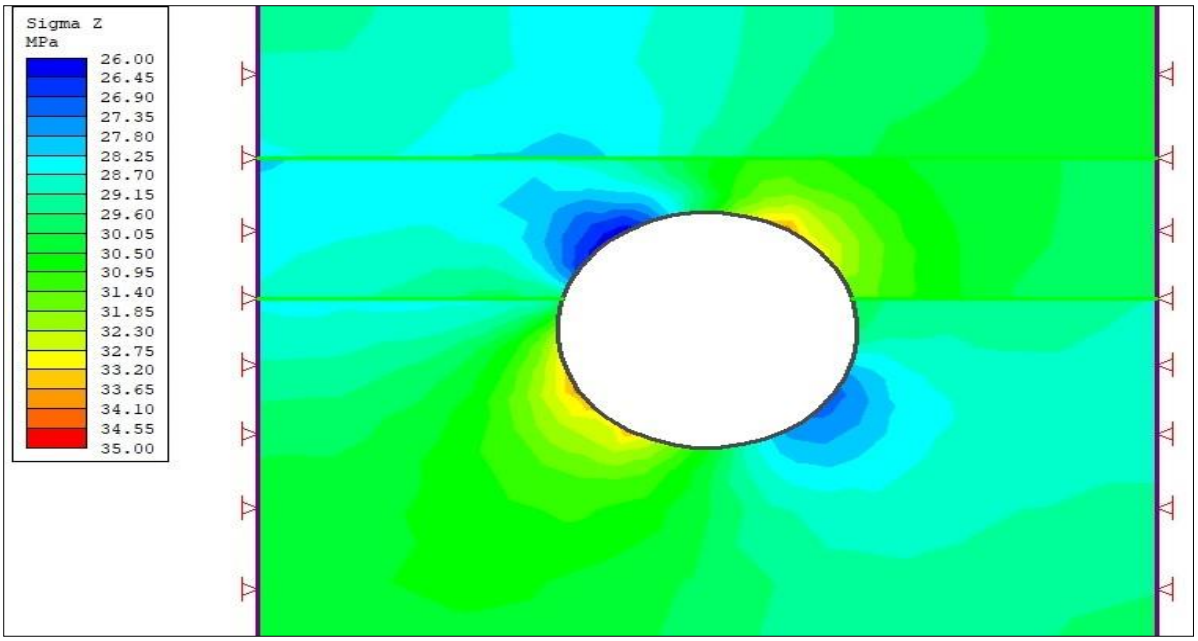


Figure 4.21: Distribution of Sigma Z surrounding the Main Raise

The stress distribution surrounding the raise for Sigma Z (vertical stress) indicates a gradual decrease from 34.10MPa at the raise boundary to 31.79MPa at 1.82m, followed by a relatively stable value fluctuating between 30.09-30.21MPa beyond 1.82m shown in Figure 4.22. Notably, the in-situ Sigma Z value of 30.51MPa falls within this stable range, confirming the raise's stability.

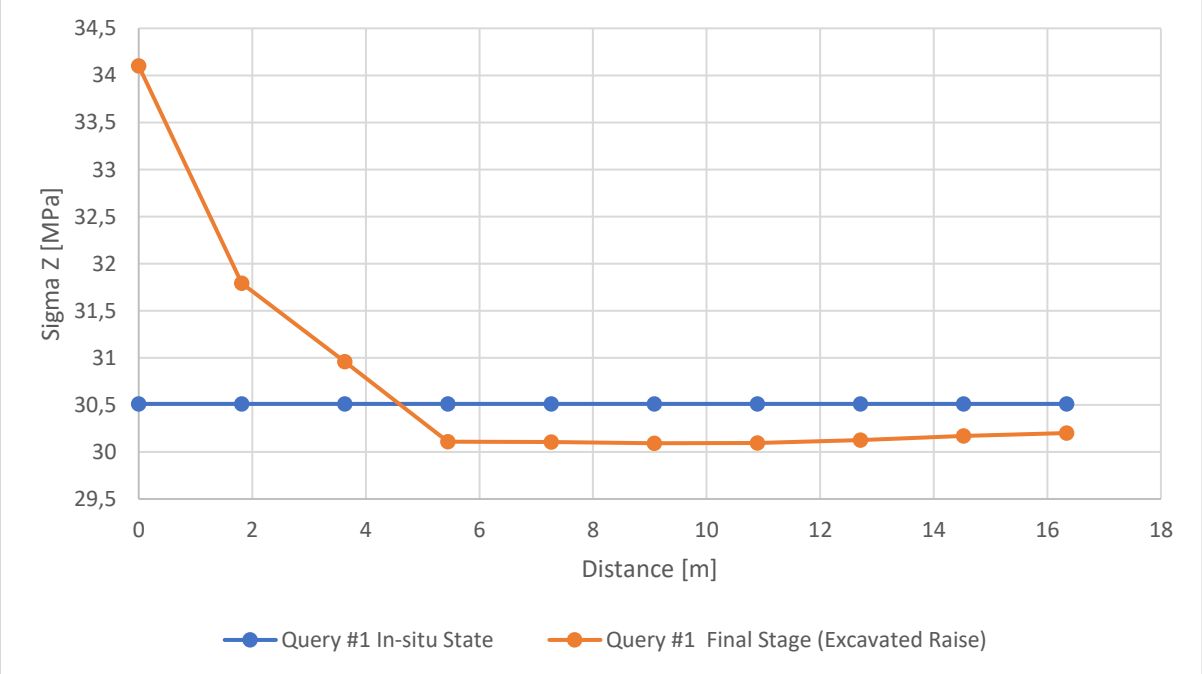


Figure 4.22: Change in Sigma Z as moving away from the excavation

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Compared to Sigma 1 (maximum principal stress), Sigma Z exhibits a more gradual decrease and stabilizes at a relatively constant value. The ratio of Sigma 1 to Sigma Z decreases with distance, indicating reduced stress anisotropy. While Sigma 1 showed high-stress concentration near the raise boundary, Sigma Z's initial decrease suggests stress relaxation due to excavation.

The stable Sigma Z values beyond 1.82m indicate uniform vertical stress in the far-field, contrasting with Sigma 1's higher stabilized value (23-24MPa) beyond 5m. This difference suggests an anisotropic stress field, with Sigma 1 as the major principal stress. Overall, the analysis highlights the importance of considering both Sigma 1 and Sigma Z in support design and rock reinforcement strategies to ensure long-term stability.

4.9.2.2 Strength Factor

The strength factor represents the ratio of available rock mass strength to generate stress, at a specific location, in this case at the rock mass surrounding the Main Raise excavation. As shown in Figure 4.23, the large zone of overstress surrounding the raise can be observed with a strength factor of 0.86. The zone of plastic yielding exhibiting shear and tensile failure around the excavation can be observed with the total number of yielded finite elements of 256. As such, all of the rock contained within this zone has a strength factor of less than 1 and will fail if left unsupported.

4.9.2.3 Total Displacement

The data from the raise excavation reveals a clear trend: the total displacement decreases as the distance from the origin (0m) increases as shown in Figure 4.24. The rate of decrease is not constant, with the displacement decreasing more rapidly at shorter distances and slowing down at greater distances. The maximum total displacement of 0.035254651m occurs at the origin and decreases by approximately 26% from 0m to 2.12m.

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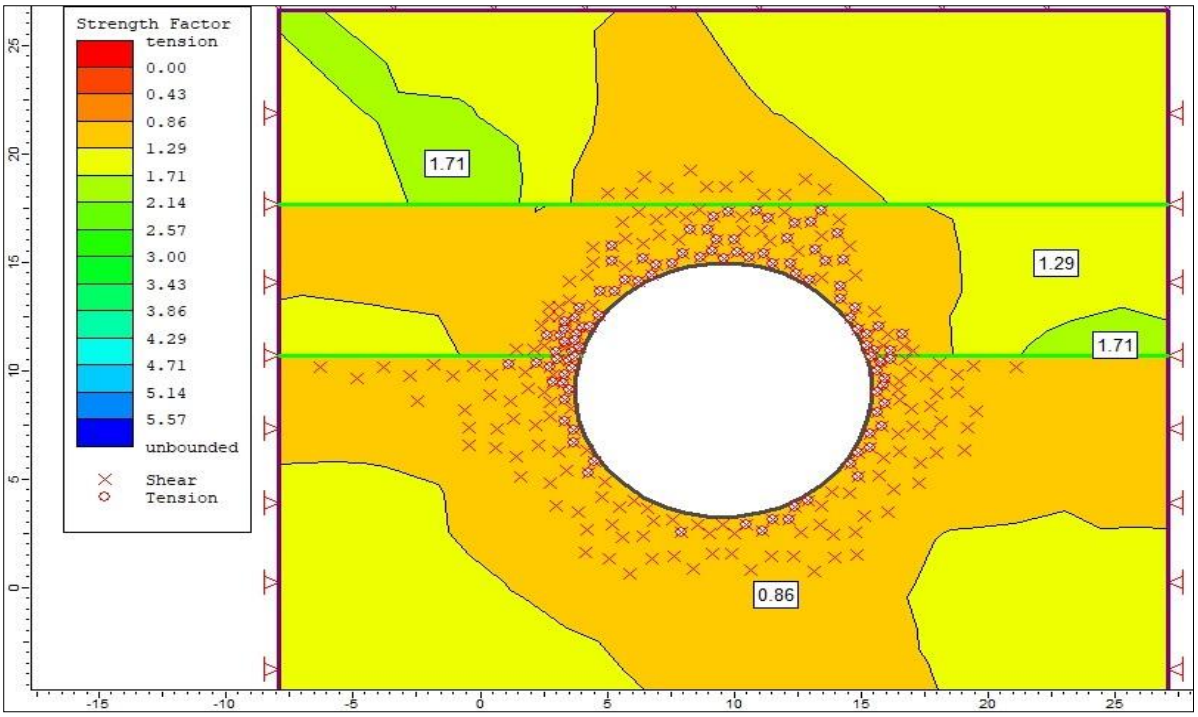


Figure 4.23: Strength factor contours and yielded elements, after plastic analysis of unsupported excavation

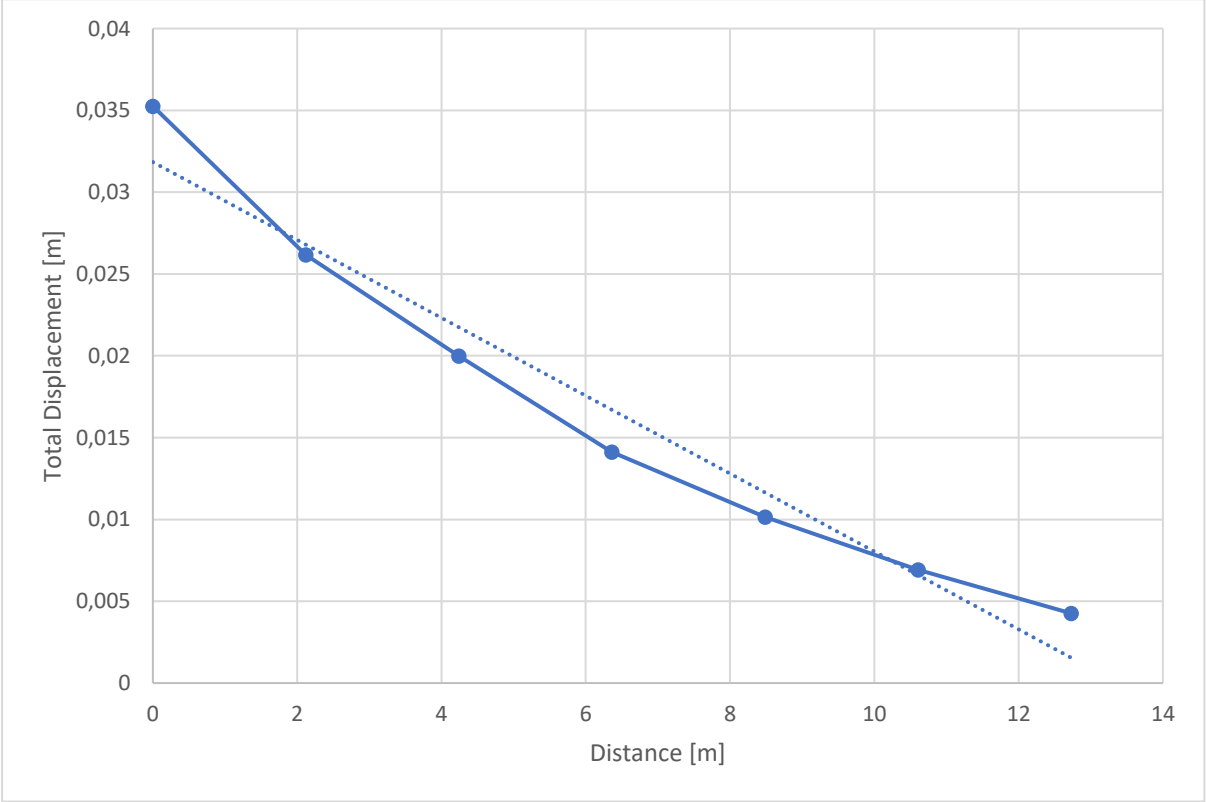


Figure 4.24: Total displacement of the rock mass surrounding the Main Raise

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As the distance increases, the rate of decrease in total displacement slows down. Beyond 6.36m, the rate of decrease becomes more gradual. At 12.73m, the total displacement is approximately 12% of the maximum displacement. These observations suggest that the maximum displacement occurs near the origin, indicating a higher risk of instability in this region. As one moves away from the origin, the decrease in total displacement may indicate a reduction in stress or deformation, eventually transitioning to a more stable region. As shown in Figure 4.25, the raise excavation is subjected to the inward deformation and footwall heave.

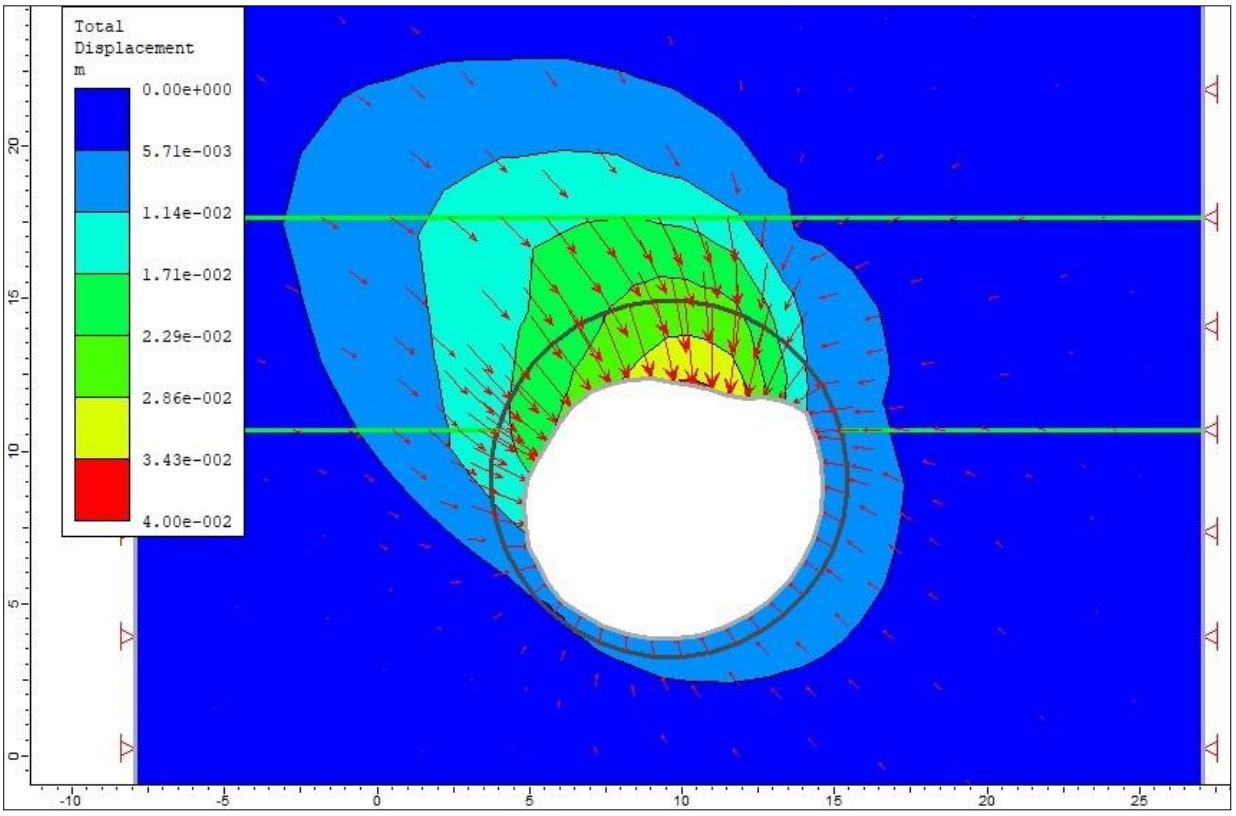


Figure 4.25: Displacement contours, vectors, and deformed boundary around Main Raise excavation

4.9.3 Fall of Ground (FOG)

Fall of ground (FOG) poses significant risks to the mining industry, affecting worker safety and production. Localized FOG incidents can be catastrophic, depending on their severity and impact. Research by Ryder and Jager (2002) and Mponya (2016) indicates that most FOG occurrences happen in the face area, where primary mining activities (drilling, blasting, and supporting) take place. Accurate recording and

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analysis of FOG events are vital for informing support design, identifying high-risk areas, and implementing targeted measures to mitigate FOG occurrences and enhance worker safety.

According to the Fall of Ground (FOG) statistics at Emerald Shaft Gold Mine, production stopes are the most prone areas for FOG occurrences. The face area accounts for the highest number of FOG incidents, at 65% shown in Figure 4.26, due to the high number of personnel and ongoing production activities such as drilling, blasting, and supporting. In contrast, the working area and back area record significantly lower FOG occurrences, at 28% and 7%, respectively. The back area's low FOG incidence is attributed to minimal personnel presence and no ongoing work activities. These statistics highlight the need for targeted support design and safety measures in high-risk areas, particularly the face area.

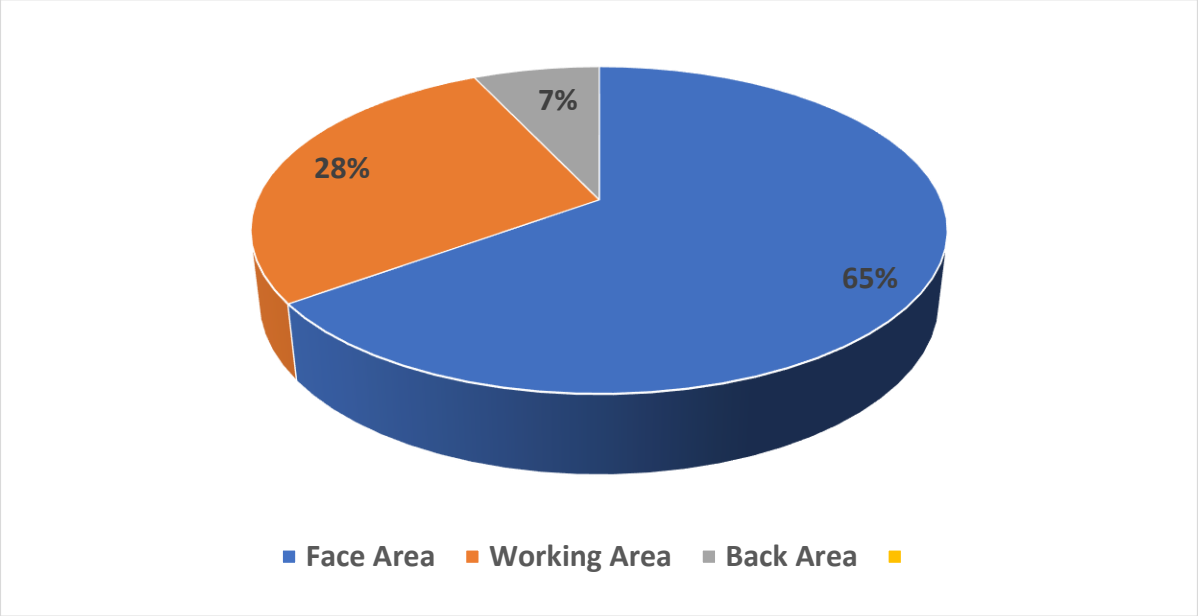


Figure 4.26: Records of FOG incidents at the production stopes

In contrast, Main Raise recorded the most FOG occurrences, with 72% of incidents happening within 15m from the raise collar. This area is characterized by a friable or poor condition hangingwall, evident from the discontinuous direction line on the hangingwall. Notably, no support units (roof bolts) were observed in this section, leaving it unsupported. With an RMR_{89} value of 56, indicating "fair ground condition," the optimum stand-up time for unsupported excavations is one week. However, it is suspected that this area exceeded the one week without support before failure

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occurred. Furthermore, water seepage into the excavation likely contributed to weakening the bond between the hanging strata. Evidence of chemical reaction (oxidation) is visible in Figure 4.27, characterized by yellowish to greenish discoloration on the hangingwall.



Figure 4.27: Evidence of FOG at the collar of the raise

Regional geological discontinuities, particularly dykes, and faults, significantly contribute to Fall of Ground (FOG) occurrences at Emerald Shaft Gold Mine. Figure 4.28 points to the location of FOG accidents due to the presence of regional geological features (dykes and faults). These structural features intersect the existing rock strata, increasing tensile stress in the rock mass. Specifically, dykes and faults account for 18% and 10% of FOG incidents, respectively. Furthermore, joints are the primary contributor to FOG in blocky ground conditions, responsible for 59% of incidents.

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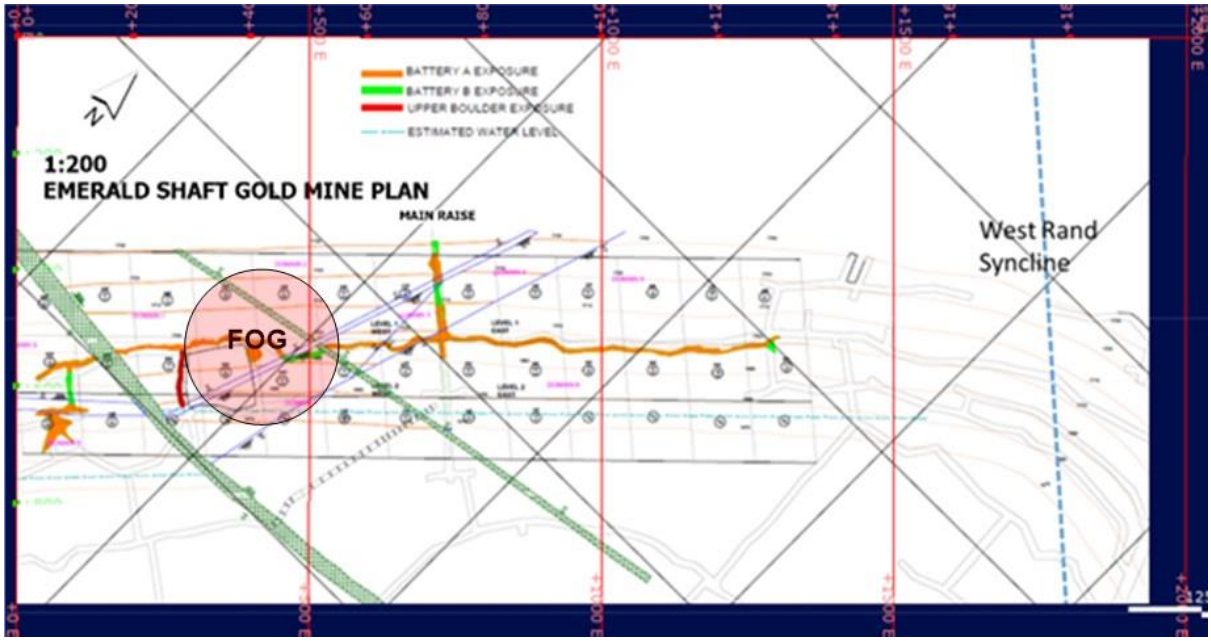


Figure 4.28: FOG Incidents: Influence of geological features (Highlighted in red)

4.10 Support System and Performance of Emerald Shaft Gold Mine

Underground excavations are supported to ensure safe working environments, combat fall of ground (FOG) hazards, minimize rockburst impacts, and control convergence and closure of mine workings. Effective support systems enable mine workers to perform their duties underground without the risk of rock mass failures and FOG-related injuries. Identifying poor ground conditions, and geologically disturbed areas, and providing adequate support are critical to promoting mine safety and sustainability.

According to Hoek and Wood (1987), the support of underground excavations, such as those in mining, involves the methods and materials used to maintain and enhance the stability and load-bearing capacity of the surrounding rock mass. This includes the support of unstable geological features, mining conditions such as brows, and the rock mass surrounding the excavations. In this study, raise, gully, and stope panels support systems at one level were investigated.

4.10.1 Main Raise Support System

Tunnel support systems are designed to ensure stability and safety in underground excavations. These systems typically integrate two key components: reinforcement elements, such as tendon supports, that increase the rock mass's inherent strength,

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and support elements, including shotcrete, wire mesh, and lacing, that contain unstable rock masses between reinforcement units (Jager and Ryder, 1999). At Emerald Shaft Gold Mine, the Main Raise support system exemplifies this approach, featuring a combination of tendons (split sets and eye bolts), concrete packs, timber sets with concrete, round timber cribbing, and timber elongates. The system is divided into two sections - the travelling way and material transportation way - with specific support units utilised in each section presented in Table 4.22.

Table 4.22 The support units utilised at the travelling and material transportation ways

Support types	**Travelling way**	**Mono-winch way**
Concrete packs	X	X
Timber sets and cribbing	X	
Tendons	X	X
Timber elongate	X	X

The Main Raise travelling way employs a robust support system, comprising pre-stressed concrete packs, timber sets and cribbing, tendon (mechanical bolts), and pre-stressed elongates, as detailed in Table 4.21. Underground inspections confirmed the effectiveness of these support systems, particularly in the centre gully shoulder, where double skin-to-skin pre-stressed concrete packs and pre-stressed timber elongates reinforce the dip side. Additionally, timber sets and cribbing successfully contain weak material (topsoil) at the entrance shown in Figure 4.29. Nonetheless, observations highlighted a concern: undercutting of timber elongates at the entrance, caused by surface water runoff during heavy rainfall, which led to the pull-out of inadequately secured timber elongates.

In the material transport way, the raise shoulders are effectively supported by skin-to-skin pre-stressed concrete packs, ensuring stability and safety. Furthermore, the hangingwall is reinforced with a combination of Split set and eye bolts, as depicted in Figure 4.30. The eye bolts play a crucial role in this area, not only providing essential support but also serving as anchorage points for rigging operations.

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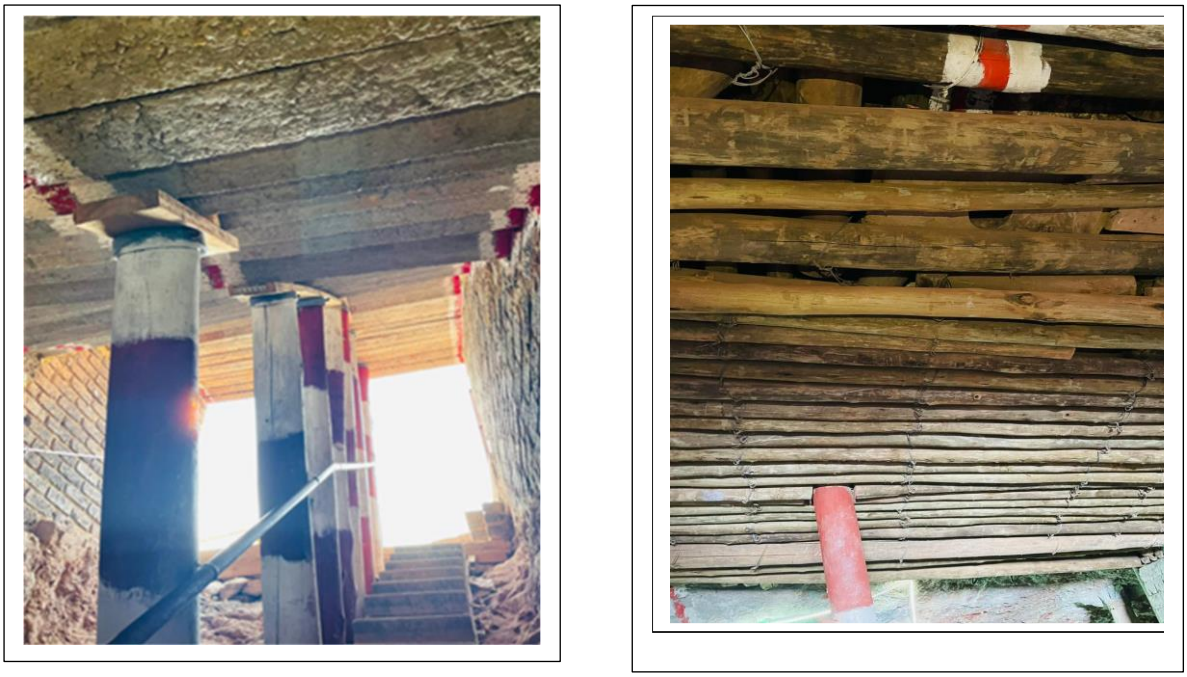


Figure 4.29: Hybrid Support System: Timber sets and concrete cribbing at Main Raise travelling way



Figure 4.30: Hangingwall Support System: Tendon Support with Split Sets and Eyebolts

Although tendon support elements offer the advantage of integrating overlying strata into a coherent beam, preventing bed separation, the unique conditions of the

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excavation – including its height, large span, and sedimentary hangingwall – diminished the effectiveness of Split sets and eye bolts. This is attributed to the low tensile strength of the roof bolts and high tensile stress (tensile dome) from the overburden rock mass. Observations revealed that the bolts loosened or lost tension in friable ground conditions, compromising stability and leading to fall of ground (FOG) incidents.

To ensure compliance with mine support standards, supplementary support systems were installed at the lower sections of the Main Raise brow (illustrated in Figure 4.31). This reinforcement consisted of pre-stressed timber elongates and Split set bolts applied to both sides of the brow, in accordance with recommendations by Thompson et al. (2012) and Villaescusa et al. (2013). The enhanced support density on the brow maintains its structural integrity, thereby mitigating the risks of injuries, fatalities, and production disruptions that could result from brow instability or failure.



Figure 4.31: Brow encountered at the lower part of the raise

4.10.2 Stope Support System

In South African underground hardrock mining, traditional stope support systems consist of packs, props, and backfill (Daehnk *et al*., 2001). However, modern mines

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have adopted a more comprehensive approach, combining in-stope bolting and netting with packs, mechanical props, and backfill to effectively support stope panels (Ferreira, 2012). At Emerald Shaft Gold Mine, the stope support system features an integrated design of support units tailored to meet the specific requirements of both working and back areas, as illustrated in Figure 4.32.

The working area, where routine mining operations occur, is divided into two sub-zones: the face area, spanning the first three metres (3m) from the face, and the working area, extending between the face and a defined distance back. Support unit failure should not occur within this area. In contrast, the back area, located beyond the working area, is restricted to worker entry due to potential instability (Stacy and Swart, 2001).

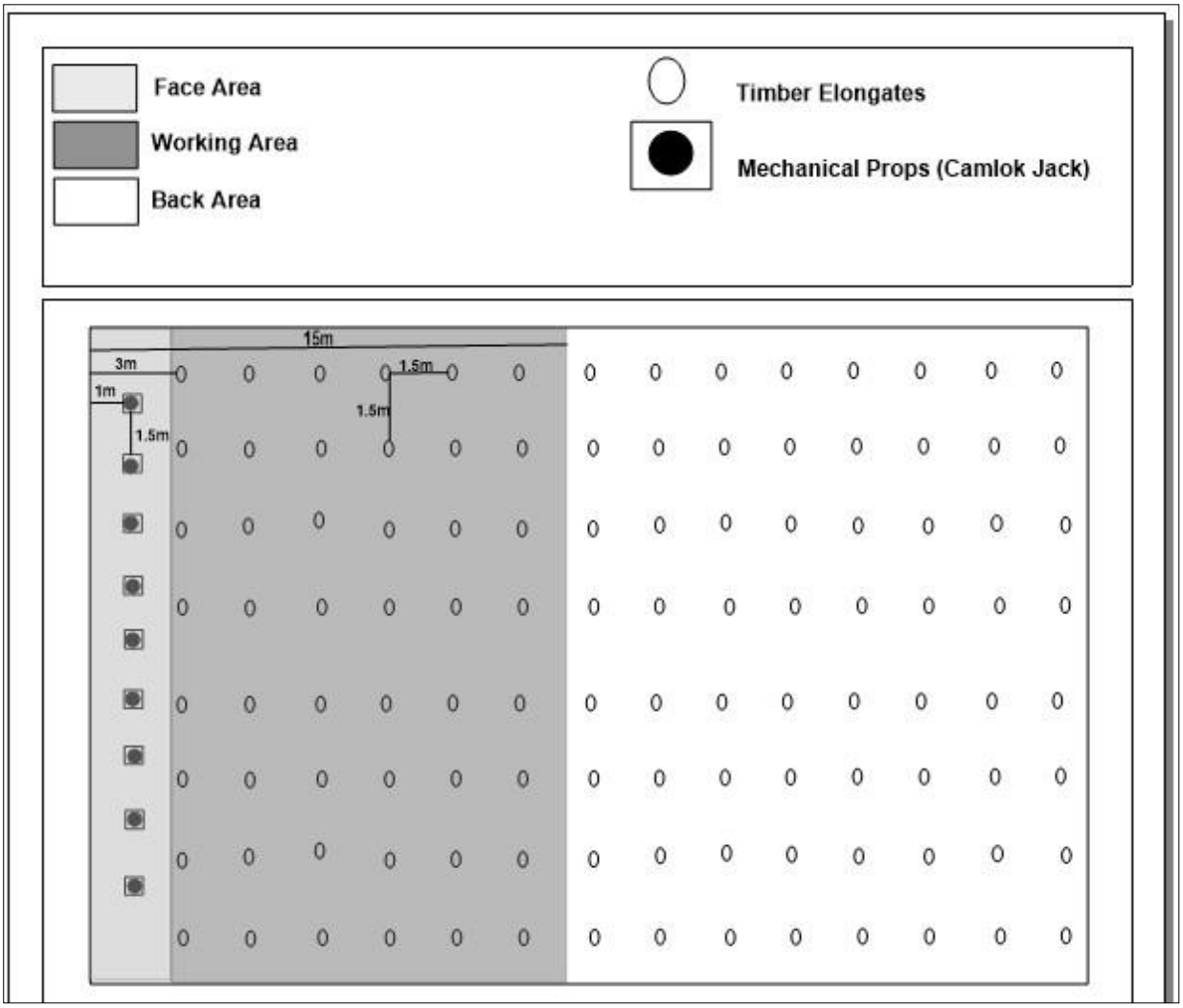


Figure 4.32: Stope support layout and standard of Emerald Shaft Gold Mine

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The face area was secured with temporary mechanical props or camlok jacks (Figure 4.33), ensuring safe working conditions and preventing fall of ground (FOG) hazards. In compliance with mine support standards, these temporary units were installed 1 m from the panel face and 1.5 m dip. To maintain a safe environment, temporary supports were installed prior to commencing work and removed after permanent units were installed, or at the end of each shift before blasting.

Although mechanical props can become frozen due to stope closure, complicating removal (Villaescusa *et al*., 2013), the implemented temporary support system effectively mitigated FOG hazards in the face area. Furthermore, the mechanical props were successfully removed remotely once permanent support was installed, minimizing risks associated with manual removal.



Figure 4.33: Face area supported by mechanical props (camlok jacks)

The working area at Emerald Shaft Gold Mine is reinforced with 150-200mm diameter pre-stressed timber elongates, installed 1.5m dip and along the strike of the panels, within 3m of the working face. This design challenges conventional concerns raised by Stacy and Swart (2001), which suggest that mine poles (timber elongates) are susceptible to blast damage when installed close to the face. To activate the support

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units, pre-stressing devices (Jackpot) are placed on top of the timber elongates and pumped with water, exerting a force into the hangingwall illustrated in Figure 4.34A. The post-blasting observations demonstrate the resilience of the pre-stressed elongates, which remain intact and functional despite multiple blasting rounds shown in Figure 4.34B.

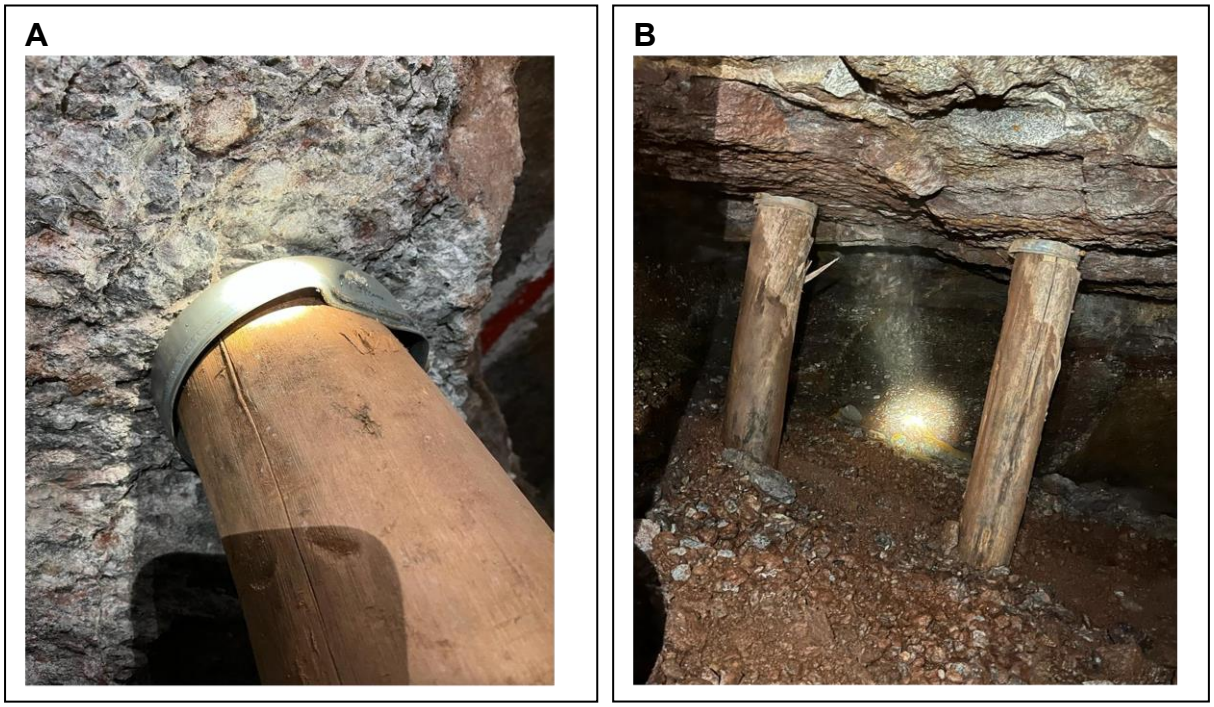


Figure 4.34: Pre-Stressed Timber Elongates: (A) With pre-stressing device and (B) intact after blasting within 3m from face

The Emerald Shaft Gold Mine's stratigraphic unit comprises weak rock types, including shale, argillaceous quartzite, and argillaceous conglomerate. As evident in Figure 4.35A, the rock mass's deadweight overwhelms the support units, resulting in punching into the hangingwall, rock deformation, and reduced effectiveness. Additionally, irregular support patterns and variations in installation angles are observed. Notably, a 110 mm diameter pre-stressed timber elongate (mine pole) experienced buckling failure due to insufficient capacity to support the overburden strata's weight, as shown in Figure 4.35B, emphasizing the need for selecting support elements with adequate strength and stiffness.

Despite this failure, the use of timber elongates provides a critical early warning system, emitting loud noises hours before backbreak occurs, indicating potential

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instability. This audible warning allows mine personnel to take proactive measures to ensure safety and maintain rock mass stability, as supported by Thompson et al. (2012), who noted that timber supports can provide valuable early warning signs of impending instability.

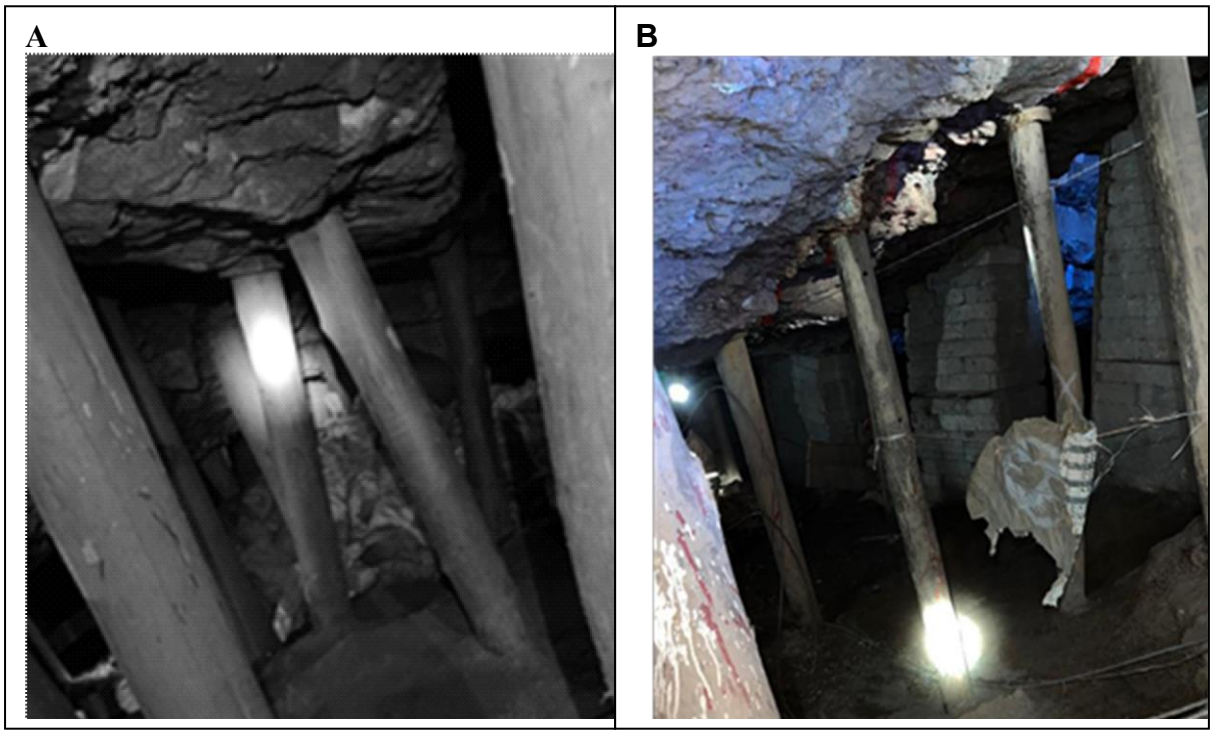


Figure 4.35: Support System Failure: (A) Support units punching into hangingwall and (B) buckling of timber elongate

In the back area, which encompasses sections already mined out as shown in Figure 4.32, pre-stressed timber elongates remain in place to provide residual support. Despite the absence of active mining operations, this area is still considered high-risk due to the potential for damage from local or regional fall of ground (FOG) events to extend into the working area. The lack of maintenance activities in the back area exacerbates these risks. As evident in Figure 4.36, the timber elongates in this area undergo significant decay, which is correlated with methane gas generation. This degradation not only undermines the support system's effectiveness but also introduces safety concerns related to methane accumulation in the explosive range.

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Figure 4.36: The decaying timber elongates in the back area of the stope panel

4.10.3 Gully Support System

Gullies are a critical component of underground gold mines, serving as a means of personnel ingress and egress to the working face of the panel. They also function as emergency escape routes, facilitate the cleaning of blasted ore, and enable ventilation flow into the working area. To ensure the stability and safety of these gullies, a support system is employed, comprising the installation of roof bolts, wire mesh or steel nets, rock props or timber elongates, and timber or concrete packs. These support elements provide necessary reinforcement to the hangingwall and gully shoulders. The type and configuration of support units used in the gullies depend on the prevailing ground conditions and the degree of fracturing. At the Emerald Shaft Gold Mine, the hangingwall of the gullies at the 1-level is supported by roof bolts (Splitsets), as shown in Figure 4.37. Additionally, the gully shoulder is reinforced with pre-stressed timber elongates and double skin-to-skin concrete packs, spaced 1m apart along the strike. According to the mine's support standard, roof bolts in the gullies are typically spaced 1m apart in both the dip and strike directions. However, observations revealed an irregular pattern of installed bolts, as illustrated in Figure 4.37A.

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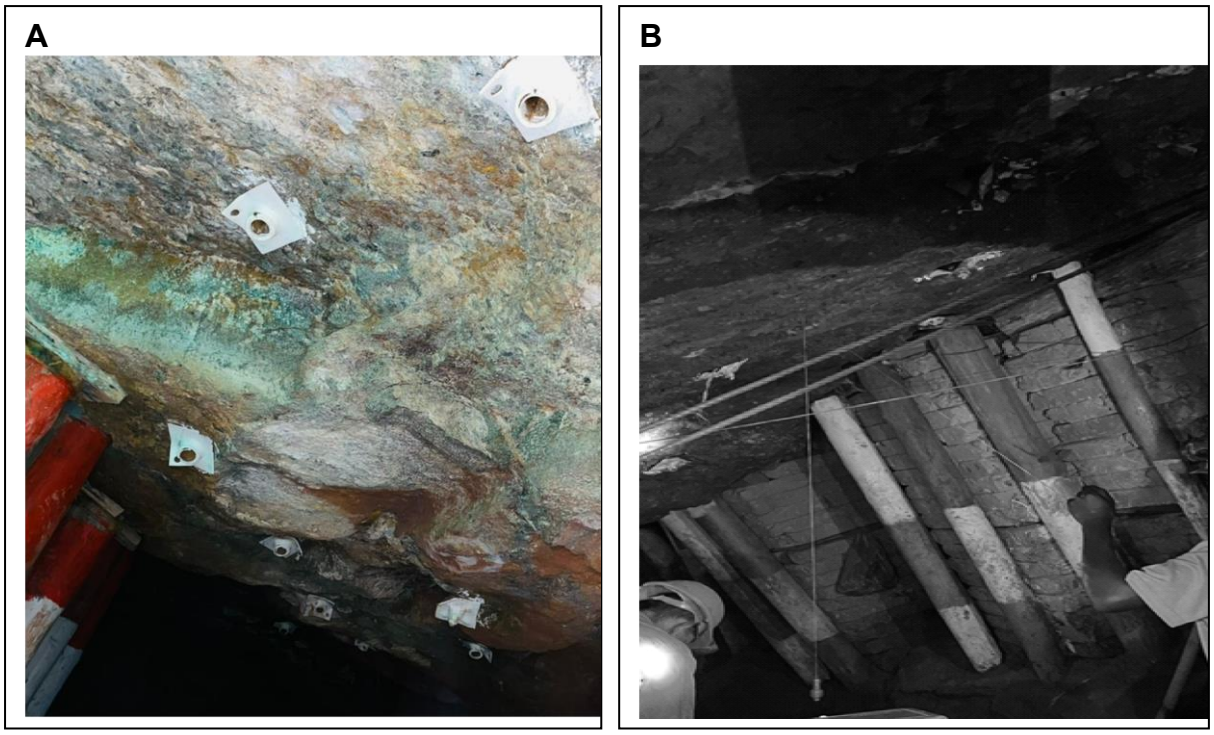


Figure 4.37: The installed support units at the strike gully of Emerald Shaft Gold Mine

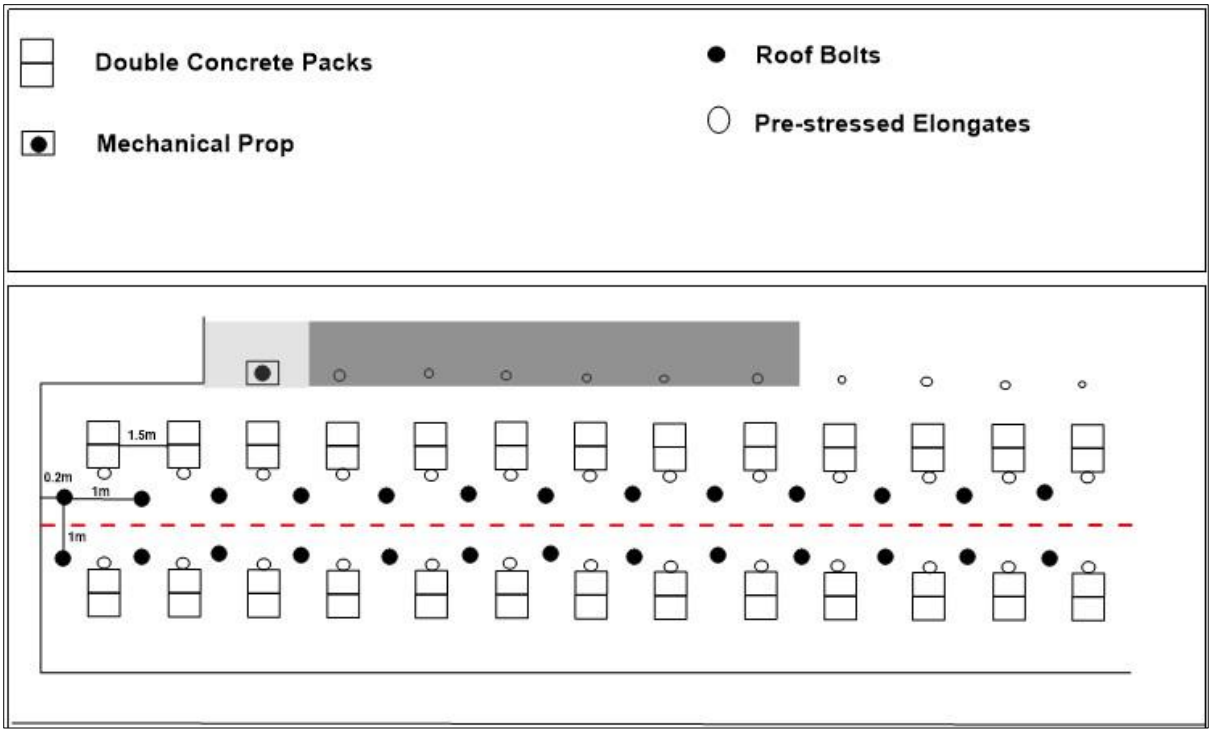


Figure 4.38: Strike gully support layout and standard with heading or ASG

The presence of a reef in the footwall at the gully has resulted in deviations from the standard width and height of the gully. Observations made during underground

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inspections revealed that the support units, comprising pre-stressed elongates and concrete packs, have punched into the gully shoulder rock mass, as shown in Figure 4.39. Furthermore, it was observed that the support units at the gully shoulder have been undermined or cut below, leaving them exposed and vulnerable to sliding if proper care is not taken. This situation necessitates immediate attention to prevent potential accidents and ensure the stability of the gully.



Figure 4.39 Pack and timber elongate punching into the week gully shoulder footwall

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CHAPTER 5: SUPPORT DETERMINATION AND DESIGN

This chapter involves determining the support units or elements and designing suitable support systems based on the determined support requirements. In this regard, the support determination is based on the results discussed in Chapter 4 which involved the assessment of the rock mass, identification of the potential hazards, and performance at MR, 1LE, and 1LW Zones. This included geological and geotechnical characterization, discontinuity mapping and analysis, stress analysis, and hazard identification. Figure 5.1 illustrates the general conceptual framework for support design procedure.

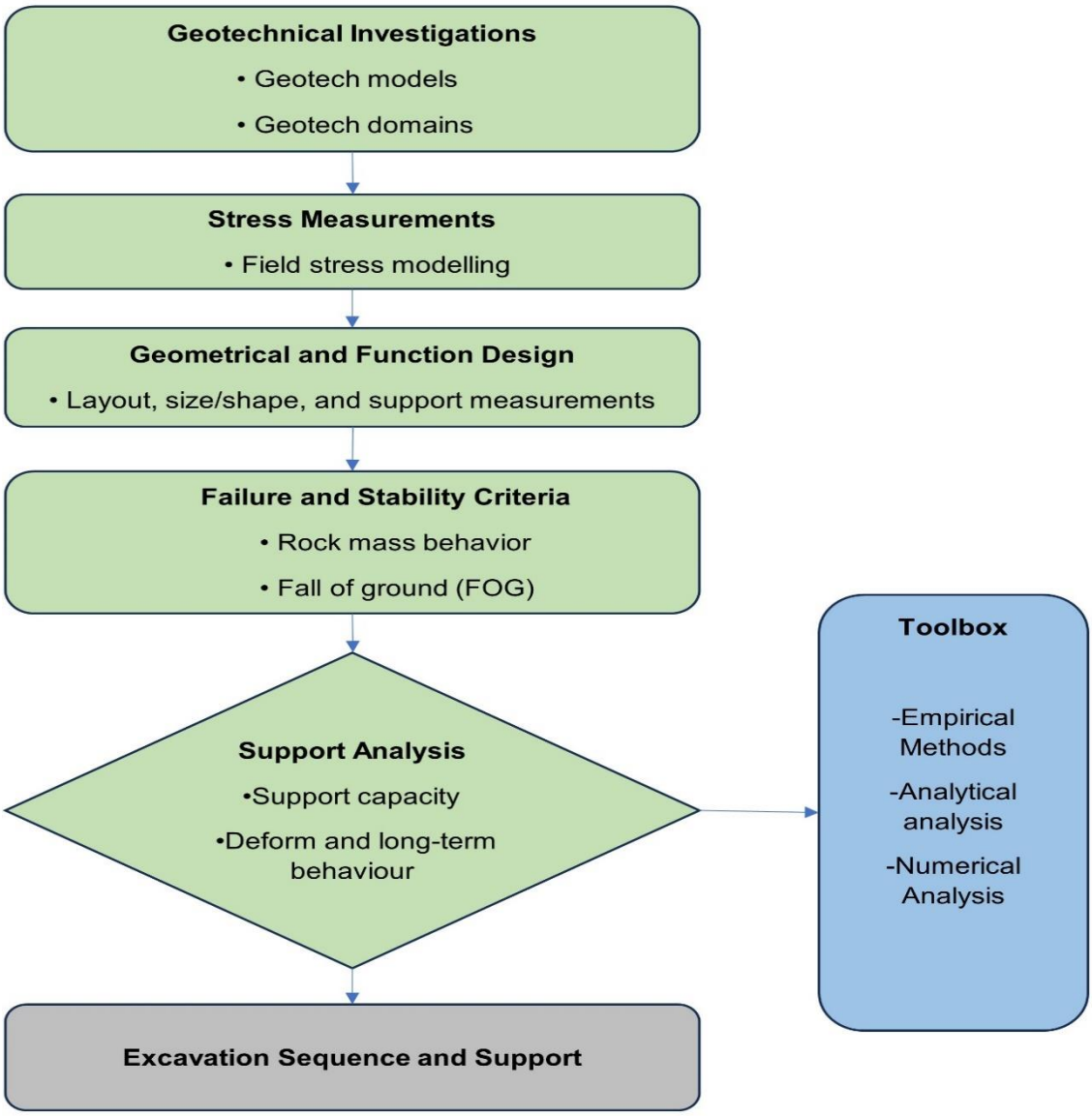


Figure 5.1: The general conceptual framework for support design procedure (Modified after Gomes, 2013)

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5.1 Main Raise Support Design

The systematic support (current support standard) at the Main Raise features 1.5m long rock bolts (shepherd crook) and friction bolts (Split Set bolts) with a capacity of 90kN, spaced 1m x 1m dip and strike (RME, 2021). However, underground support observations reveal poor performance of these bolts, with tendons succumbing to corrosion and being pulled out from the raise walls due to low shear strength and adverse rock mass conditions. Main Raise support design typically considers tendon performance, ground conditions, and stress loading, incorporating empirical and analytical approaches (Bieniawski, 1992; Choquet & Hadjigeorgious, 1993; Stille & Palmström, 2003). These approaches ensure sound support design, addressing the complex interactions between rock mass, stress, and support systems to maintain stability.

5.1.1 Empirical Ground Support Design for Main Raise

In South Africa support system has been designed using empirical analysis which generates support recommendations based on rock mass rating and excavation span. In the absence of the rock mass data, a general empirical design rule is used (Ryder and Jager, 2002). The general empirical design rule suggests that the length of the tendon must be equal to the half-span or height of excavation and spacing be equal to the half-length of the tendon support. Due to the availability of rock data, the common two rock mass classification systems were employed. The RMR_{89} and Q-system were used supplemented by MRMR for raise support design and recommendations. The rock mass finding indicates fairground (RMR_{89}=56), and poor (Q-value=0.17). According to these ratings the geotechnical domain is classified as Red Domain and requires appropriate support design.

5.1.1.1 RMR_{89} Support for Main Raise

With an RMR value of 56, the rock mass is classified as Class III. The support recommendations include 4-5m long bolts in the crown and walls, wire mesh in the crown, and fibre support (50-100mm thick in the crown and 30mm thick on the sides). Steel ribs are not necessary for this ground condition. Nonetheless, Laubscher (1990) recommends systematic grouted bolts at 1m x 1m spacing (dip and strike) for enhanced stability.

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5.1.1.2 Q-system Support for Main Raise

Based on the Q-System value of 0.17, the rock mass is classified as "very poor" to "inferior" quality, necessitating comprehensive support measures. With a span of 4m and an Excavation Support Ratio (ESR) of 1.6 of the permanent mine openings, the calculated De value of 2.5m presented in Table 5.1 suggests a high risk of rock instability. Systematic support of the pattern bolting, wire mesh, and shotcrete installation is recommended to mitigate this risk.

Table 5.1: Q-Support parameters

Parameter	**Value**
Q-System	0.17
Span	4m
ESR (Excavation Support Ration	1.6
De (Equivalent Diameter	2.5m

According to the Q-support chart (Figure 5.2), the recommended support system consists of bolts and shotcrete. Bolts with a length of 2.4m are spaced 1.3m apart in dip and strike directions (Table 5.2), providing systematic support to the excavation. Additionally, a layer of fibre-reinforced shotcrete, 50-90mm thick, is applied to prevent rockfall and provide supplemental support. This design ensures stability in the very poor rock mass conditions indicated by the Q-System value of 0.17

Table 5.2: Support design according to the Q-Support chart

Support Type	**Description**	**Spacing (m)**	**Length (m)**	**Thickness (mm)**
Bolts	Systematic support	1.3	2.4	-
Shotcrete	Supplemental support	-	-	50-90

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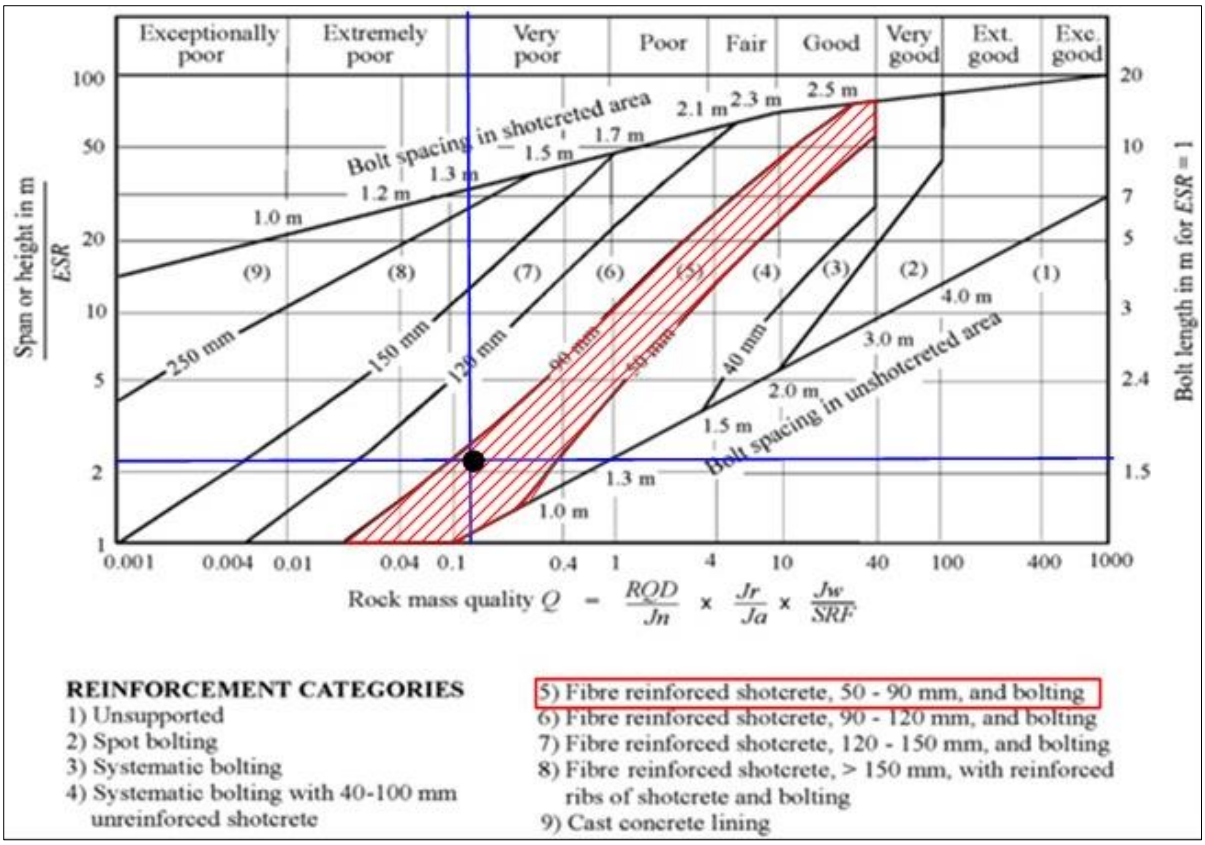


Figure 5.2: Q-system support chart recommendations

5.1.2 Analytical Ground Support Design for Main Raise

The analytical approach involves applying conceptual models to replicate the behaviour and response of rock masses. Examples of this approach include numerical and structural analysis (Swart and Hardley, 2005). In underground excavations, the most common failure mechanism is the fall of blocks through sliding or falling from the back of the excavations.

5.1.2.1 Structural (wedges or keyblocks) Analysis

Unstable keyblocks are typically formed by the intersection of two or more joint sets or when two or more joint sets dip towards each other. To prevent wedge formation, support or reinforcement systems are installed to stabilize the rock mass and prevent potential failures. The wedge analysis model at the Main Raise reveals the formation of an unstable wedge from four intersecting joint sets. The maximum apex height of the perimeter wedges is 1.55m. Figure 5.3 illustrates the support of the raise excavation using standard primary bolts, 1.5m in length, spaced 1m x 1m, with a yield

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strength capacity of 90kN. Although the factor of safety (FoS) of the wedges slightly increases, the wedge remains unstable with a FoS of 0.978, which is less than 2. The failure of the wedges and support could be attributed to insufficient bolt capacity (90 kN) and length (1.5m), which is less than the maximum apex height of the perimeter wedges (1.55m).

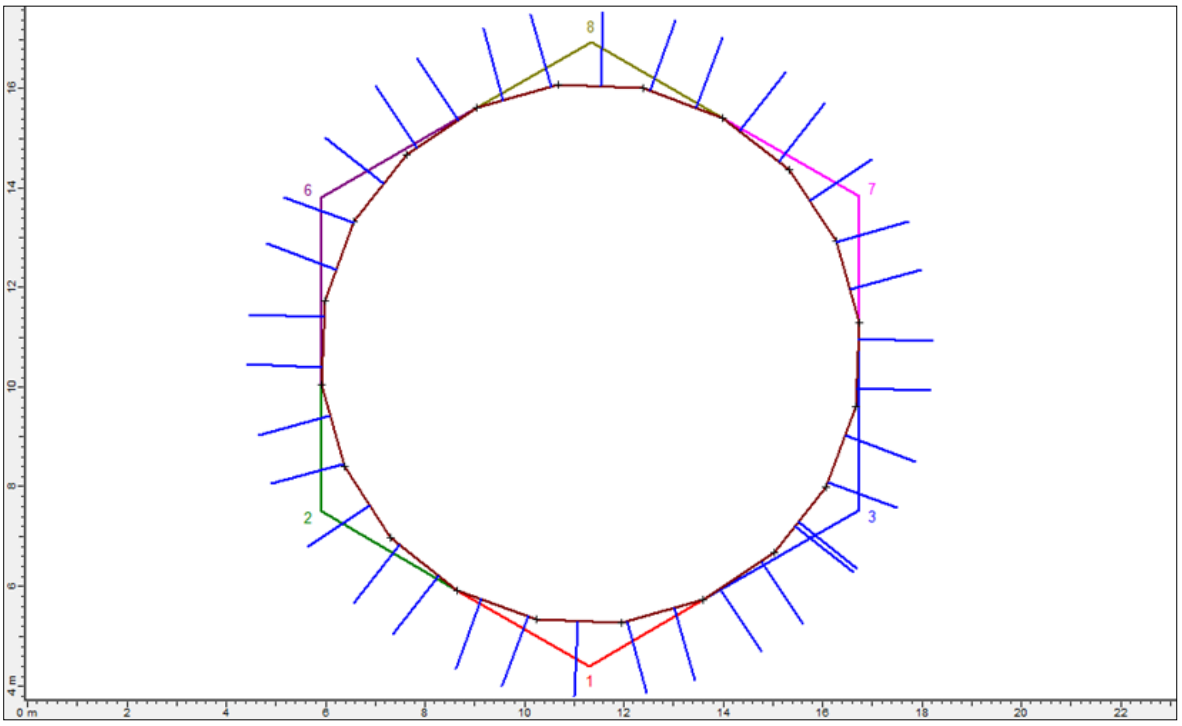


Figure 5.3: The support of the raise excavation using the standard primary roof bolts.

Considering the RMR_{89} and Q-system support recommendations, Split set bolts 3m in length (within the 2.4m to 4m range) were evaluated, along with the current standard bolt spacing of 1m x 1m and a yield capacity of 150kN. As shown in Figure 5.4, the 3m Split set bolts exceed the maximum apex height of the perimeter wedges (1.55m).The factor of safety (FoS) of the wedges increases, particularly for the crucial wedge 8, from 0.978 to 1.087. However, the new FoS of the wedge remains relatively low. Furthermore, the observed falling of small rock blocks between the support units necessitates the use of area coverage secondary support. According to the RMR89 and Q-system support recommendations, the addition of 50mm thick fibre shotcrete, with a shear strength of 2MPa and a unit weight of 2651 kg/m³, significantly enhances the stability of the Main Raise.

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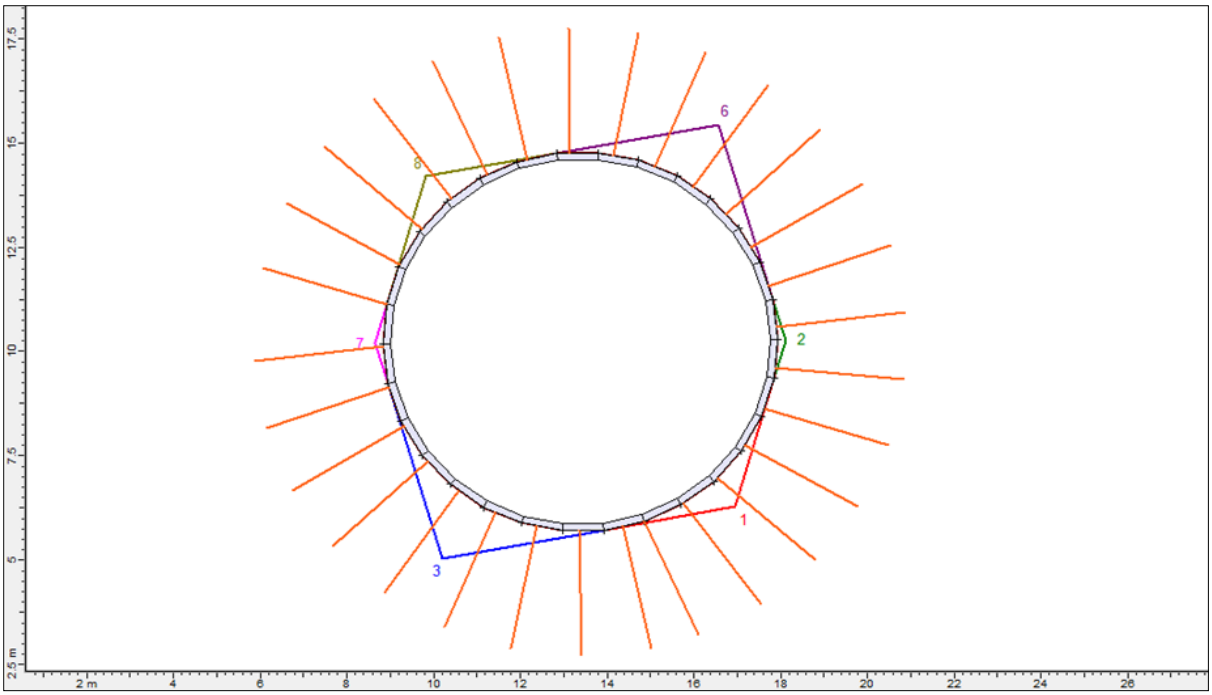


Figure 5.4: The combination of split set bolts and fibre shotcrete liner support of the Main Raise

Despite increasing the length and yield capacity of Split set bolts, these tendons, without the addition of fibre shotcrete, exhibit poor performance. Therefore, alternative bolt types should be investigated. Mechanical anchors are suitable for the ground conditions at Emerald Shaft Gold Mine, which are characterized by low stress, moderate to strong rock types, and a shallow mining environment. However, mechanical anchors have limitations. If the anchor slips or the bolt breaks, the capacity of the bolt drops to zero, and the supported rock can fall. Moreover, mechanical anchors are prone to corrosion due to oxidation when exposed to aggressive groundwater. To address these issues, grouted dowels are a viable alternative. Unlike mechanical anchors, grouted dowels remain anchored and provide support even if the dowel slips or the face plate breaks off. Additionally, the grout capsule is resistant to corrosion, making grouted dowels a more reliable option.

Figure 5.5 illustrates the interaction model between the support tendon (grouted dowels) and the wedges surrounding the raise. The grouted dowel properties include a yield strength of 489.1 MPa, yield capacity of 240 kN, radius of 32mm, and length of 2.5m. Additionally, the bolt has a plate capacity of 100 kN and a bond strength of 0.3 MN/m. The application of grouted dowels to support the rock mass and wedges formed

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on the raise space (1m x 1m in plane) demonstrates exceptional performance. This is evident from the significant increase in the factor of safety (FoS) of wedge 8, from 0.38 to 9.50, resulting in a substantial improvement in the FoS of the other wedges. Compared to Split set bolts, grouted dowels are more suitable for use with wire mesh due to their superior performance, simple installation, cost-effectiveness, and high corrosion resistance in permanent installations. The wire mesh provides excellent area coverage, keeping rock fragments in place between the grouted dowels.

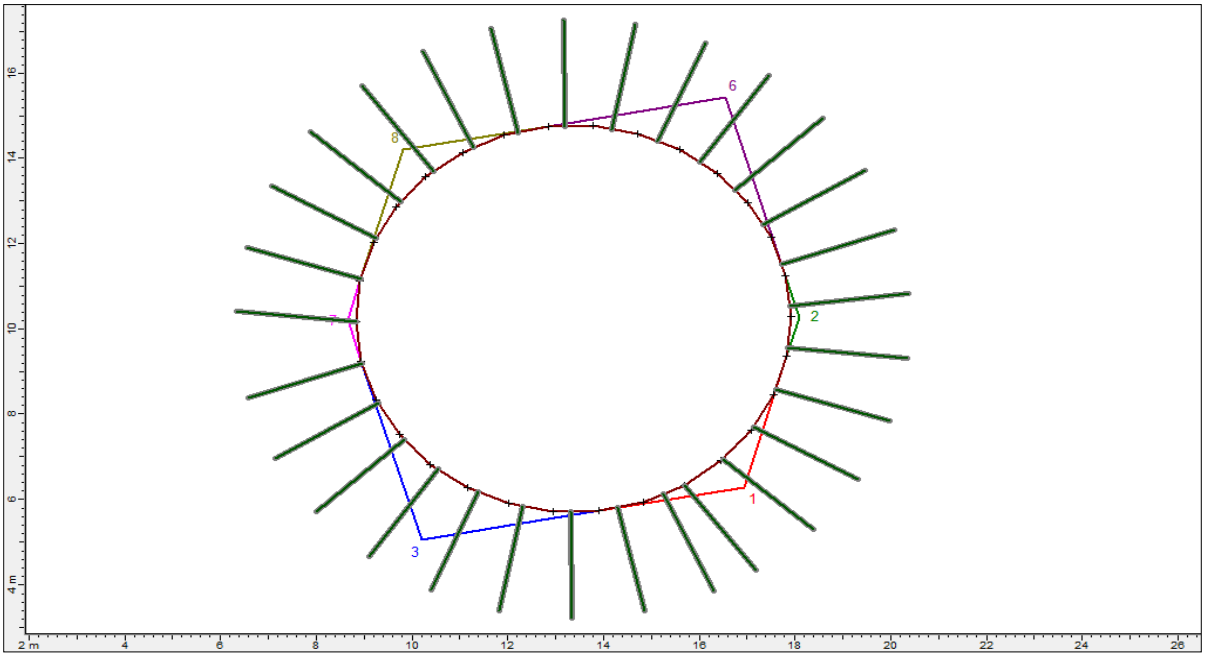


Figure 5.5: The application of grouted dowels in the Main raise.

5.1.2.2 Numerical Ground Support Evaluation for Main Raise

The analysis using RocScience Phase 2 software reveals significant improvements in ground stability with the introduction of support units. Initially, the unsupported raise exhibited unstable ground conditions with 236 finite elements and a total displacement of 0.035509m. The addition of grouted bolt support (3m long, spaced 1m, 240kN peak capacity) yielded a moderate improvement in ground stability, reducing finite elements to 211 and displacement to 0.0266729m. However, this was still insufficient, prompting the introduction of shotcrete support, which significantly enhanced ground stability, reducing finite elements to 132 and displacement to 0.009851m as shown in Figure 5.6. These highlight the importance of a comprehensive support design, incorporating multiple support methods, to achieve optimal ground stability.

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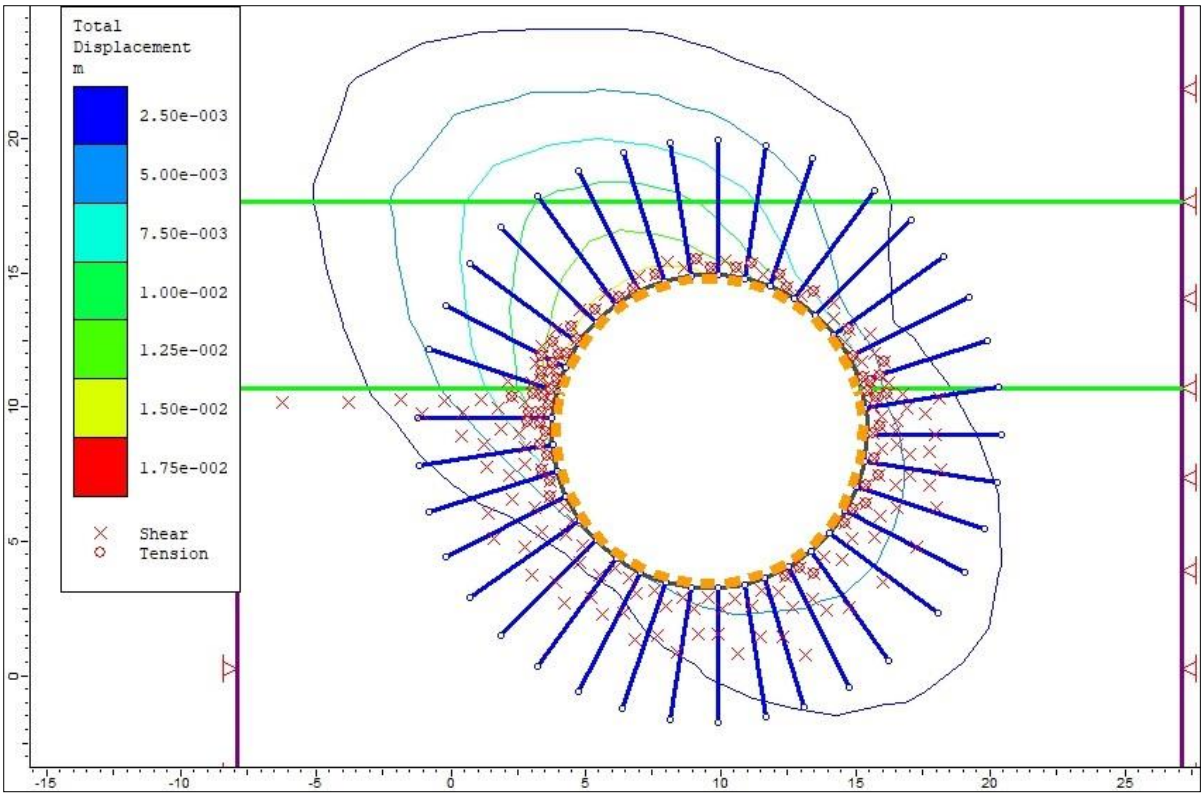


Figure 5.6: The influence of support unit on yielded finite elements and total displacement of the excavation walls

5.2 Stope Support Evaluation and Design

To mitigate the risks associated with rockfalls and rockbursts, stope support systems are widely used in the gold and platinum mining industries (Daehnke *et al*., 2001). These systems typically consist of mine poles, rock props, and packs. At Emerald Shaft Gold Mine, the narrow tabular stopes in shallow mining environments exhibit low stress, moderate to strong hangingwall rock types, and minimal closure. Consequently, stiff support units, such as pre-stressed elongates of mine poles, are employed as permanent support in the stopes. However, Fall of Ground (FOG) incidents often occur due to joint interactions and inadequate confinement. This section evaluates the stope support system based on support demand, support capacity, and the tributary area concept applied to the stope hangingwall. Figure 5.7 illustrates the conceptual stope support evaluation procedure and design.

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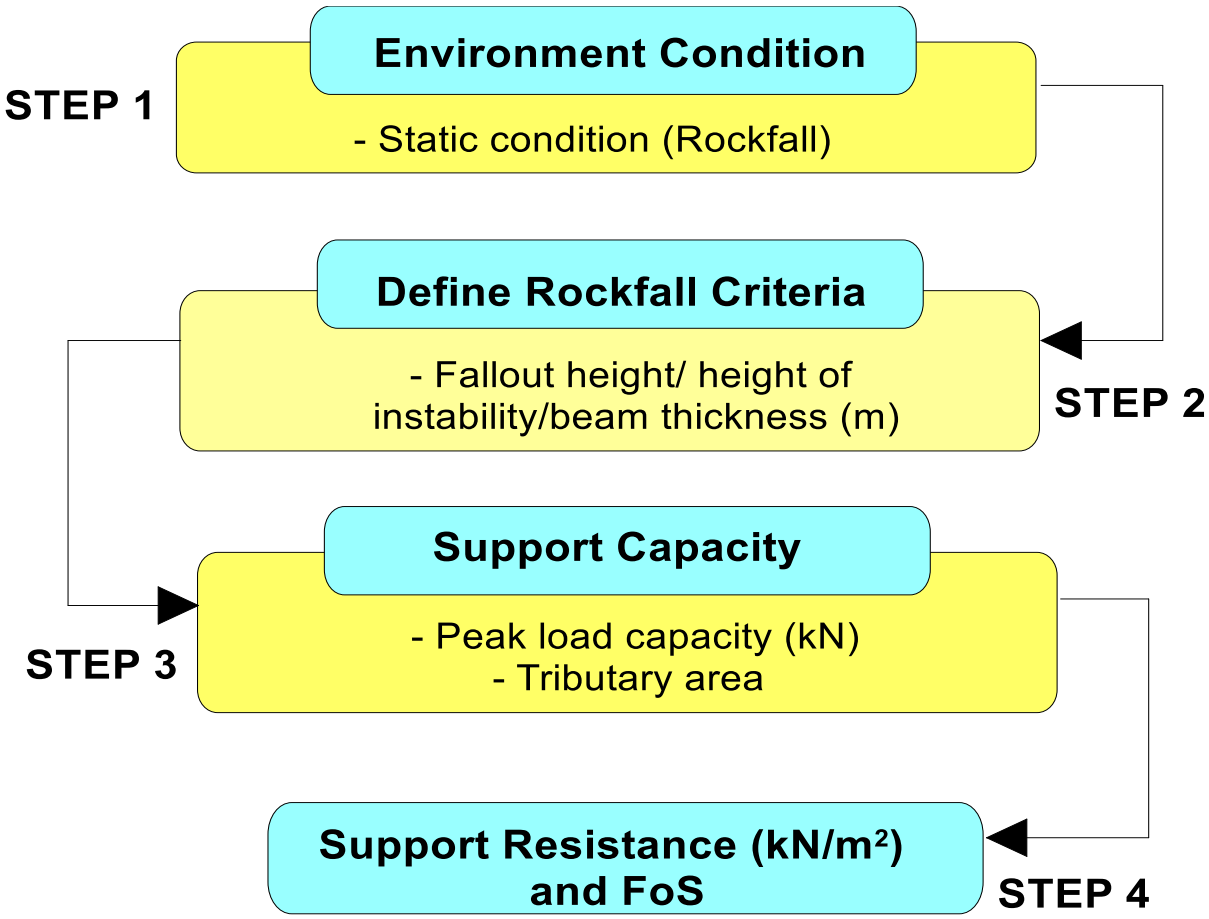


Figure 5.7: Stope support evaluation procedure

5.2.1 Stope Support Requirements and Environment Conditions

The ground conditions in the study area are susceptible to falls of ground or rockfalls, posing a significant hazard to mining operation. In contrast to deep gold mines, which are prone to both rockfalls and rockbursts, a systematic stope support system should be designed to withstand the deadweight of potential falls. According to the Witwatersrand gold mines, the accepted maximum stope support resistance is 50 kN/m² (Deahke *et al*., 1998, Jager and Ryder, 1999). However, this value may vary depending on the specific ground conditions encountered, emphasizing the need for site-specific rock mechanics data. Based on the available rock mechanic data presented in Table 5.3, the required support resistance (demand) is 24 kN/m², considering the observed beam thickness of 0.6 m. This value represents the minimum support resistance required to ensure stability and prevent rockfalls in the area.

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Table 5.3: Rockfall resistance of Kimberly reef

Type of reef	Beam thickness (m)	FOG area (m^2)	Rockfall (kN/m^2)	Closure rate (mm/m)
Kimberly	0.6	21.2	24	8

5.2.2 Rockfall Criteria

The height of potential rock mass instability (t) is a crucial parameter in stope support evaluation and design. This height determines the support demand, which is the deadweight of potential rockfall that the support units or elements must carry. According to Stacy and Swart (2001), the value of "t" should be greater than or equal to the thickness of 95% of the cumulative frequency of rockfall and rockburst events. However, due to insufficient data, the value of "t" was estimated from observations of brow thickness and height of wedge failure (FOG).

This approach has been used by rock engineers for decades to determine the height of fall from in situ observations of dislodged rock. Consequently, an estimated "t" value of 0.9m was obtained from the observations. Using the average rock density of 2750 kg/m^3 and the assumed gravitational force of 10 m/s^2, the support demand was calculated as 24.75 kN/m^2 using equation 11. Notably, this determined support demand shows little difference from the rockfall resistance of a Kimberly reef (24 kN/m^2) with the observed beam thickness of 0.6 m.

5.2.3 Support Capacity and Performance

The capacity of a support unit refers to the peak load it can carry. To assess support element capacity, the stope is divided into working and back areas. At Emerald Shaft Gold Mine, the stope support system consists of pre-stressed timber elongates (in both working and back areas) and mechanical props (Camlok jacks at the face area).

The pre-stressed elongate (150-200mm) has a peak load capacity of 260 kN, while the mechanical prop has a peak load capacity of 150 kN. However, the downgraded load capacity of the pre-stressed elongate is 182 kN, which aligns with the actual underground performance of this type of unit. Table 5.4 provides support system details including mean peak load capacity and tributary area. Meanwhile Figure illustrates tributary area coverage by temprary support and permanent support units.

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Table 5.4: Support area capacity in terms of tributary area

Support area	Support type	Tributary area (m^2)	Load capacity (kN)
Working area	150-200mm pre-stressed timber elongate	4	182
Face area	Mechanical prop	6	150

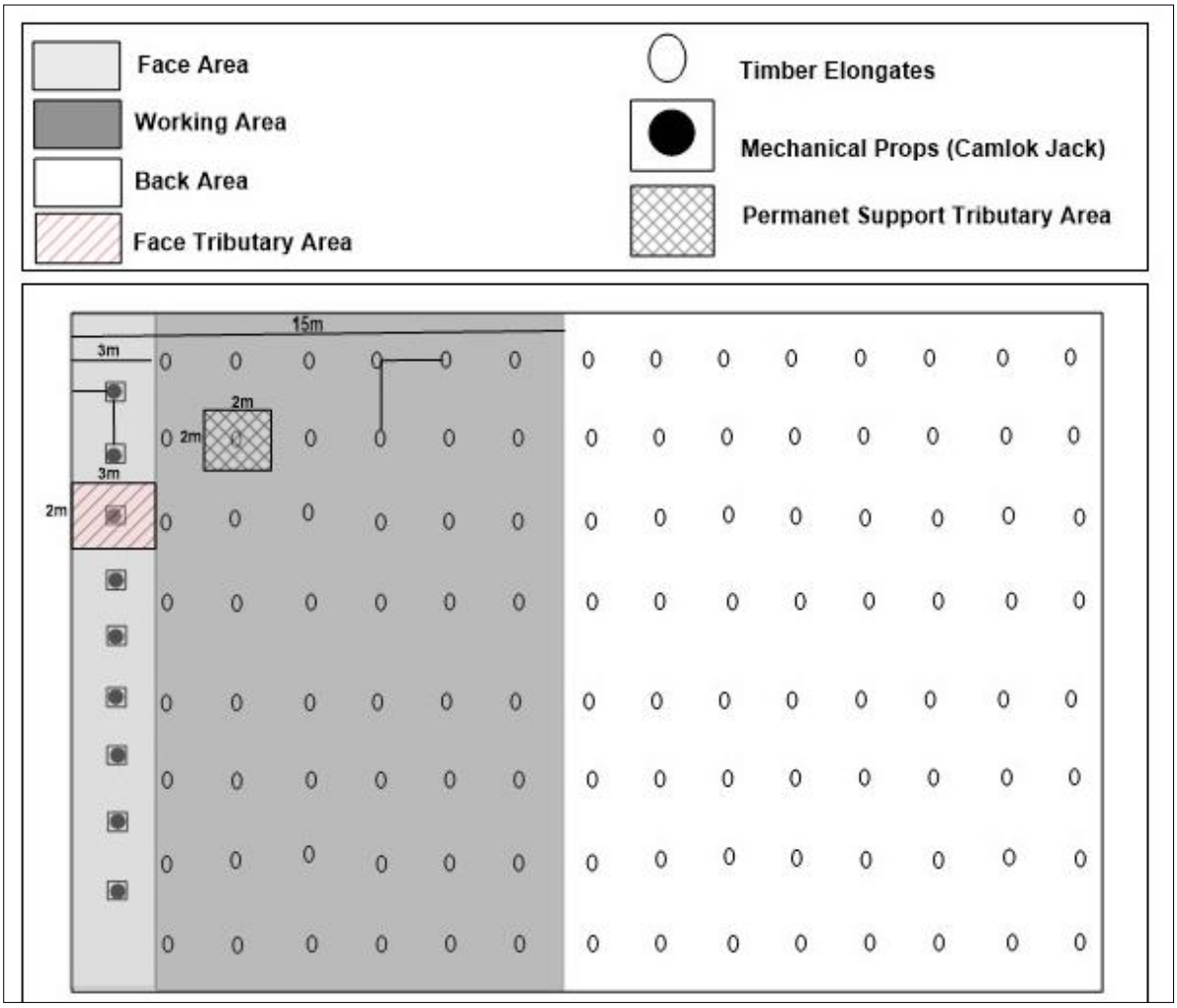


Figure 5.8: Stope support layout showing tributary area of temporary and permanent support

To mitigate the effects of stope closure, a support unit is required to cater to the rate at which stope closure occurs, which is a result of both elastic and inelastic deformations. Underground observations reveal a stope closure rate of 8 mm/m of face advance. At 3m from the face (last line of permanent support from the sweeping line), the stope closure is 24mm. To control the 120mm stope closure for the 10 m and 5 m into the working area, permanent support units are necessary. The buckling failure

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of timber elongates, which are 110 mm (slender elongates) and 2m in height, was observed. The slenderness and high stoping width are suspected to be the cause of the buckling failure, as the increase in support height reduces the stiffness of the support unit (Stacy and Swart, 2001; Daehnke *et al*., 2001). This highlights the importance of considering support design and installation in relation to the specific geological conditions and excavation geometry. Typically, the variation of the reef channels affects the stoping width and support performance, emphasizing the need for a comprehensive understanding of the geological and geotechnical conditions to inform support design and installation (Tannant and Kaiser, 2014).

5.2.3 Support Resistance and Factor of Safety

According to Jager and Ryder (1999), support resistance (SR) is a critical concept in rock engineering design, whereby a support unit or the smallest representative section of the support system is averaged over the tributary area of the hangingwall to be supported by that unit or portion of the system. This is expressed in units of kN/m², representing the average resistance provided by the support unit over the tributary area (Jager and Ryder, 1999). The support resistance of a unit should be sufficient to resist the support demand from the deadweight of the overburden rock mass. To determine the support resistance, the mean underground peak load capacity of a support unit is divided by the tributary area applicable to that unit. Table 5.5 presents the results of calculated support resistance for the working area and face area, respectively. These calculations are essential in ensuring that the selected support system can provide the necessary resistance to maintain stability and prevent rock falls or other hazards in the working area and face area (Hoek and Brown, 1980).

Table 5.5: Support resistance at working area and face area by support units

Support area	**Support type**	**Support resistance (kN/m^2)**
Working area	150-200mm pre-stressed timber elongate	45.5
Face area	Mechanical prop	25

The factor of safety (FoS) or safety factor is a critical parameter in rock engineering design, representing the ratio of the support demand to support resistance capacity. In this regard, the FoS is calculated as the support demand (overburden deadweight

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= 24.75 kN/m²) divided by the support resistance capacity. The permanent support system, comprising pre-stressed timber elongates, provides a relatively high safety factor of 1.84, indicating a satisfactory level of support resistance capacity. However, the face area support resistance does not meet the demand, with a safety factor of 1.01, which is marginally acceptable (Kaiser *et al*., 2001). To address this situation, two possible solutions can be considered: (1) increasing the temporary support density at the face to enhance the support resistance capacity, or (2) reducing the permanent support distance from the face to provide additional support resistance.

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CHAPTER 6: CONCLUSION AND RECOMMENDATIONS

This chapter provides key findings, conclusions, and recommendations from the geotechnical investigation conducted at the Emerald Shaft Gold Mine. The outcomes of the study highlight the complex geological conditions, the need for a tailored support design and extraction strategy, and the recommended support measures to ensure safe and efficient mining operations.

6.1 Summary of the Findings

A comprehensive study was conducted at the Emerald Shaft Gold Mine to evaluate the rock mass, and geological conditions, and raise and stope support systems performances to inform mine design. The key findings of the study include a detailed overview of the Emerald Shaft Gold Mine operations, characterization of the ground conditions and stress environment, stability analysis of raise and stopes at 1-level, and assessment of support performance and determination of more suitable support systems. These findings provide valuable insights into the mine's geological and geotechnical conditions, stability, and support requirements, ultimately informing optimal mine design and operational strategies.

6.1.1 Design and Layout of Emerald Shaft Gold Mine Operation

Using the sublevel stoping method, the Emerald Shaft Gold Mine was established to extract Kimberly and Boulder reefs, which vary in thickness from 1.2 to 2.5m. Prior to the establishment of production stopes, a pencil raise was developed and ledged for a maximum distance of 10m. The breast stopes were mined conventionally using an underhand configuration with leads and lags of bottom and top panels. However, variations in reef thickness (1.2 to 2.5m) resulted in reef remnants in the hangingwall and footwall, leading to variations in stoping width ranging from 1.5 to 2.2m. These variations affected support performance due to the fact that long mine poles (timber elongates) and camlok canopies were use in which the high stoping width was associated with the strength reduction of those support units. This also resulted in uneven distribution of virgin and mine-induced stresses.

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6.1.2 Ground Conditions and Stress Environment of the Mine

The study employed a multi-faceted approach, comprising stratigraphic profiling, field testing, and laboratory analysis to evaluate the strata conditions and performance of support systems of the Emerald Shaft Gold Mine to provide long-term stability of the underground excavations. The results showed variable rock competency, ranging from weak to very strong in the hangingwall and strong in the footwall, as well as distinct lithological contacts and reef thickness. The rock types exhibited a wide range of mechanical properties, including Unconfined Compressive Strength (28.5-120MPa on average), deformation, and shear resistance. The heterogeneous and anisotropic nature of the rock masses highlights the importance of site-specific characterization for optimizing mine design and support strategies.

Traditionally, shale and argillaceous lithologies are considered soft, whereas siliceous quartzite is considered hard A cut-off value of approximately 200 MPa is commonly used to distinguish between soft and hard lithologies The UCS test results for the Emerald Shaft Gold Mine indicate that the rock masses are predominantly soft, with UCS values falling below 200 MPa. However, it is noted that argillaceous and siliceous quartzite lithologies are classified as hard according to the ISRM field rock strength estimation criteria.

The study uncovered complex geological conditions, significantly influenced by structural and compositional discontinuities contributing to weak hangingwall conditions. Three distinct zones (MR, 1LE, and 1LW) were identified as structural domains, crucial for understanding the mine's geotechnical behaviour. A structural analysis of the area reveals varied orientation and density concentrations of structural domains as shown in Appendix II. Notably, joint sets were predominantly sub-vertical to vertical and sub-parallel to the rock strata, hangingwall, and face, resulting in unstable ground conditions. The study also identified distinct geotechnical domains (red (MR and 1LW) and pink (1LE) domains) with varying rock mass quality, as determined by rock mass classification systems (RMR_{89}, Q-system, and GSI). These domains significantly impact the overall mine stability and extractability of ore reserves. Regional geological features, such as dykes and faults, were found to compromise rock mass quality, stability, and integrity. The stability of excavations at

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the Emerald Shaft Gold Mine is significantly influenced by jointing, which contributed to weak hangingwall conditions.

In-situ stress conditions were determined at the Emerald Shaft Gold Mine to characterize the stress environment prior to underground excavations. The results revealed a K-ratio of 1.5, indicating a significant horizontal stress component. The principal in-situ stresses exhibit a northwest-southeast and southwest-northeast orientation, with a rotated stress field likely influenced by tectonic forces and structural complexity. These findings provide valuable insights into the regional stress regime in hardrock mining, informing mine design and excavation strategies.

The wedge stability analysis (stability of wedges formed by intersecting joints) results revealed varying levels of stability, with some wedges exhibiting high instability in both raise and production stopes with the factor of safety less than 2. The application of field stress did not significantly impact overall stability with little horizontal clamping, indicating that rock mass structure and geometry are primary controlling factors. However, unstable wedges require close monitoring and potential support measures to prevent failure. The formation of unstable wedges in production panels significantly impacts stope performance and ore dilution, emphasizing the need for additional support to enhance stability.

Typically, the horizontal virgin stress (field stress) causes a slight clamping effect on the joints or fractures in the raise and stopes. However, low virgin stress levels can lead to bed separations and backbreak collapses in shallow mines. In contrast, the horizontal clamping of fractures observed at Emerald Shaft Gold Mine is more typical of deep mines, where the increased stress levels help to maintain the stability of the rock mass.

The numerical analysis stress distribution results showed distinct differences in stress behaviour, with a high-stress concentration near the raise boundary and reduced stress anisotropy with distance. A large plastic zone with a strength factor of 0.86 was identified, indicating overstress and potential failure if left unsupported. Displacement analysis revealed a clear trend of decreasing total displacement with distance from the origin, with the maximum displacement occurring near the origin. The findings highlight the importance of considering both stress and displacement in support design and rock reinforcement strategies to ensure long-term stability.

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Fall of ground (FOG) incidents pose significant risks to the mining industry. At Emerald Shaft Gold Mine, production stopes, particularly the face area, are most prone to FOG occurrences. The Main Raise area recorded the most incidents, attributed to poor hangingwall conditions, inadequate support, and regional geological discontinuities. Dykes, faults, and joints (in blocky condition) contribute to FOG incidents by increasing tensile stress in the rock mass. The findings highlight the need for targeted support design, safety measures, and regular monitoring to mitigate FOG occurrences and enhance worker safety.

6.1.3 Type and Performance of the Support Systems used in the Mine

The Main Raise support system at Emerald Shaft Gold Mine utilises a combination of tendons, concrete packs, and timber sets to ensure stability and safety. Underground inspections confirmed the effectiveness of these support systems, particularly in reinforcing the dip side and containing weak material. However, concerns were raised regarding the undercutting of timber elongates due to surface water runoff. Additionally, the tendon support elements were found to be less effective in the Main Raise excavation due to their low tensile strength, leading to loosening or loss of tension and compromising stability, which resulted in fall of ground incidents.

The mine's stope support system features an integrated design, with support units tailored to meet specific requirements for both working and back areas. Temporary mechanical props or camlok jacks effectively mitigated fall of ground hazards in the face area, while pre-stressed timber elongates proved resilient to blast damage in the working area. However, deadweight led to support unit failures (buckling failure) and punching into the rock strata. Despite these challenges, the mine's support system has been effective in ensuring safe working conditions, and the use of timber elongates provides a critical early warning system for potential instability.

Emerald Shaft Gold Mine gullies system typically includes roof bolts, wire mesh, rock props, timber elongates, and packs. However, observations revealed irregularities, including deviating roof bolt patterns and deviations from standard gully dimensions due to the presence of a reef. Support units, such as pre-stressed elongates and concrete packs, have punched into the gully shoulder rock mass or been undermined,

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posing a significant risk to gully stability and safety. Urgent attention is necessary to address these issues and prevent potential accidents.

6.1.4 Support Determination and Design

The current support units (tendon) at the Main Raise has been found to be inadequate due to corrosion, low shear strength, and adverse rock mass conditions. A revised support design approach is necessary, considering tendon performance, ground conditions, and stress loading. Rock mass classification systems, including RMR_{89}, Q-system, and MRMR, were employed, indicating fair to poor ground conditions. The recommended support measures include the installation of long bolts, wire mesh, fibre support, and systematic grouted bolts. The Q-System value of 0.17 classifies the rock mass as "very poor" to "inferior" quality, necessitating comprehensive support measures. The recommended support system includes pattern bolting, wire mesh, shotcrete installation, and fibre-reinforced shotcrete to prevent rockfall and ensure stability.

The analytical approach in rock mechanics revealed that the primary failure mechanism at the Main Raise is the fall of blocks, with wedge analysis showing an unstable wedge formed by intersecting joint sets. Standard primary bolts were insufficient, resulting in a factor of safety (FoS) below 2. Evaluating 3m Split set bolts improved the FoS but remained relatively low. Additional secondary support, such as 50mm thick fibre shotcrete, is recommended. However, grouted dowels demonstrated exceptional performance, significantly increasing the FoS and providing reliable support. They are more suitable for use with wire mesh due to their superior performance, simplicity, cost-effectiveness, and corrosion resistance.

Analysis using RocScience Phase 2 software revealed significant improvements in ground stability at the Main Raise with the introduction of support units. Initially, the unsupported raise exhibited unstable ground conditions, but the addition of grouted bolt support moderately improved stability. However, the most notable enhancement occurred with the introduction of shotcrete support, which reduced unstable elements from 236 to 132 and total displacement from 0.035509m to 0.009851m. These results highlight the importance of a comprehensive support design incorporating multiple support methods to achieve optimal ground stability.

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The study area's ground conditions are prone to falls of ground or rockfalls, posing a significant hazard to mining operations. A systematic stope support system is necessary to withstand the deadweight of potential falls. Back analysis of collapsed dimensions revealed a required support resistance of 24 kN/m², considering a beam thickness of 0.6m for Kimberly reef. However, the support demand was calculated as 24.75 kN/m². The capacity of the support units, consisting of pre-stressed timber elongates and mechanical props were evaluated. The calculated support resistance and factor of safety (FoS) values indicate that while the permanent support system has a satisfactory FoS of 1.84, the face area support resistance is marginally acceptable with a FoS of 1.01. Increasing temporary support density or reducing permanent support distance from the face are proposed solutions to address this.

6.2 Conclusions

A comprehensive study was conducted at the Emerald Shaft Gold Mine to evaluate the rock mass, geological conditions, and raise and stopes support systems performances to ensure long-term stability. The following key conclusions were drawn from this study:

- The Emerald Shaft Gold Mine's geological conditions, rock mass quality, and stress environment pose significant challenges to mining operations. Key findings emphasize the importance of site-specific characterization, tailored support designs, and extraction strategies to ensure safe and efficient mining operations
- The hangingwall at Emerald Shaft comprised soft and heterogeneous lithologies, including mudstone, shale, and argillaceous quartzite. These lithologies contained weak intrusions and exhibited mining-induced fractures, which compromised their integrity. Deformation was observed across argillaceous layers, indicating a lack of stability. Moreover, the presence of beds and lamination within the rock mass resulted in a significant reduction in shear strength between the interfaces of adjacent lithologies, further contributing to the instability of the hangingwall.
- Inadequate support, poor hangingwall conditions, and regional geological discontinuities contribute to FOG incidents. Hence, the hangingwall instabilities at Emerald Shaft Gold Mine have been largely controlled by geological

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discontinuities, weak strata, and excavation spans rather than stress controlled. Comprehensive support measures, including pattern bolting, wire mesh, shotcrete installation, and fibre-reinforced shotcrete, are necessary to prevent rockfall and ensure stability. Ultimately, a systematic stope support system incorporating multiple support methods is essential for withstanding the deadweight of potential falls and ensuring optimal ground stability.

6.3 Study Challenges, Limitations, and Impact on the Study Findings

The study was faced by a number of challenges such as limited access to some parts of the mines, inadequate historical data and equipment which affected data collection and analysis. The manual mapping of rock face was employed because of the absence of the digital mapping equipment. Nonetheless, the study offers great information on mine stability and safety. Methodological limitation which include the limitations of the 2D modelling. However, the results of the study provide feasible ideas of enhancing mine stability and safety, and the procedures developed can be modified to be used in a different mining environment. With the recognition of such challenges and limitations, this study tries to offer a clear evaluation of what it has to offer and areas of future study.

6.4 Recommendations

Based on the findings, challenges and limitations of this comprehensive geotechnical investigation, the following recommendations are proposed to ensure the long-term stability and safety of the Emerald Shaft Gold Mine:

6.4.1 Recommendations for the Company

- When mining in friable or blocky ground, it is essential to implement comprehensive support measures to ensure stability. This includes providing timber elongate support units with headboards, reducing support spacing, and utilizing area coverage support systems, such as nets or mesh, in stopes. Additionally, applying shotcrete (fibre support) in the Main Raise is recommended to reinforce the rock mass further and prevent rockfalls, thereby maintaining a safe working environment.

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- To minimize the risk of hangingwall instability in stopes, it is recommended that the gold reef be kept closer to the hangingwall, thereby avoiding exposure of weak lithology, such as shale, which is prone to massive breakouts. This approach will help prevent over blasting and ensure a safer mining environment.
- It is recommended that when encountering solid and prominent regional geological features such as dykes and faults, mining should be conducted at an angle of 45º to these features, avoiding parallel mining. To ensure stability, support density should be increased, with support provided on both sides of the weak plane, prioritizing the weaker side. Additionally, support spacing should be reduced to a maximum of 0.5m on both sides of the weak plane, providing enhanced reinforcement and mitigating potential risks associated with mining near significant geological features.
- Implement a revised support design approach, considering tendon and pre-stressed timber elongate performances, ground conditions, and stress loading.
- To prevent the propagation and creation of stress-induced fractures, bed separation, and tensile domes, it is recommended that support be installed immediately after excavation. Furthermore, stoping widths should be minimized and stoping faces kept straight, thereby mitigating the risks associated with stress-induced instability and ensuring a safer and more stable mining environment.
- Based on RMR_{89}, Q-support systems, and support performance the installation of long bolts, wire mesh, fibre support, and systematic grouted bolts to ensure optimal ground stability is recommended.
- Conduct regular monitoring and maintenance of support systems to ensure their effectiveness and develop a comprehensive support design and extraction strategy to ensure safe and efficient mining operations.
- It is recommended that a regular training program be implemented for mine workers, focusing on best practices in mining and adherence to establish mine standards. This program will enhance workers' knowledge, skills, and compliance with safety protocols, ultimately reducing the risk of accidents and improving overall mine performance.

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6.4.2 Recommendations for Future Studies

- A detailed study is recommended to investigate the regional geological features, particularly dykes and faults that impact the stability of the mine. This study should assess the influence of these features on K-ratio values and the potential for seismicity, providing critical insights for ensuring the long-term stability and safety of the mine.
- Conduct advanced numerical modeling using software such as FLAC3D or 3DEC to simulate the behaviour of the rock mass and support systems under various loading conditions.
- Research ways to increase access to inaccessible mine spaces, including the creation of new technologies or safe data collection protocol including the use digital mapping equipment, drones or other similar technologies in order to increase data efficiency and precision.

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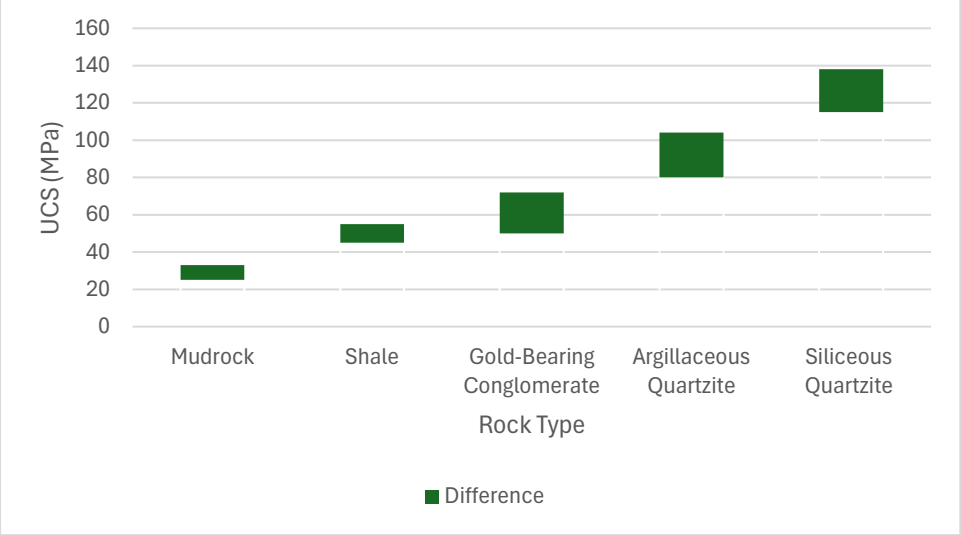
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APPENDICES

APPENDIX I: Laboratory Test Results

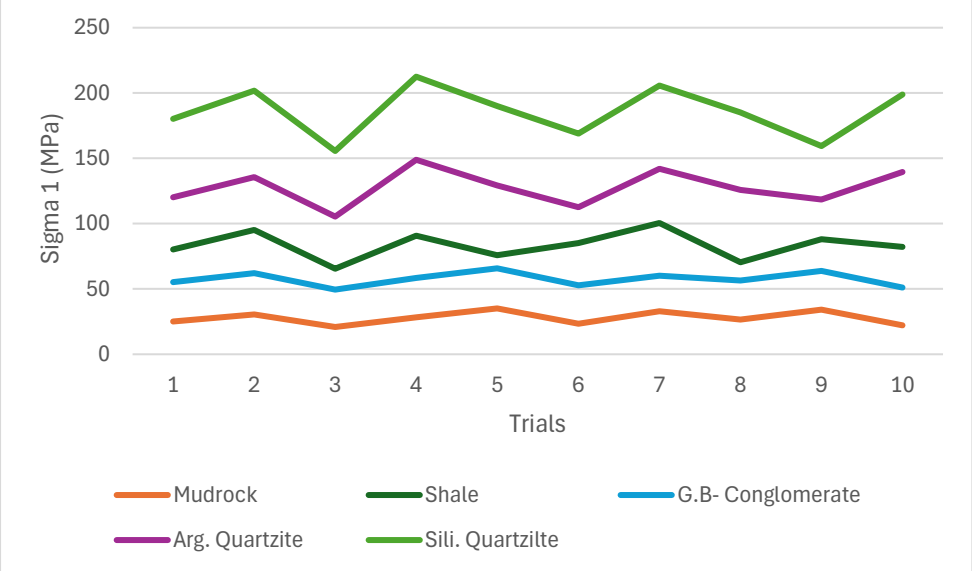
A: Minimum and Maximum UCS Range of the Rock Types

Present the minimum and maximum value (range) of the rock types highlighting the deference between these values



B: Major Principal Stress (σ_1) Results of the Rock Types

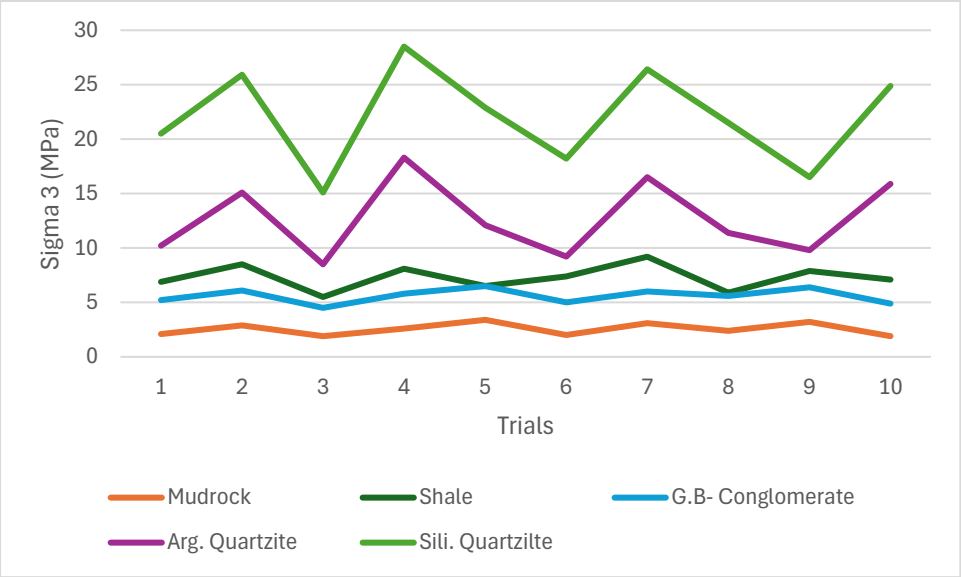
The variations in the major principal stress (σ_1) values for each rock type across the ten trials.



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C: Minor Principal Stress (σ_3 or Confining Stress) Results of the Rock Types

The variations in the minor principal stress (σ_3) values for each rock type across the ten trials.

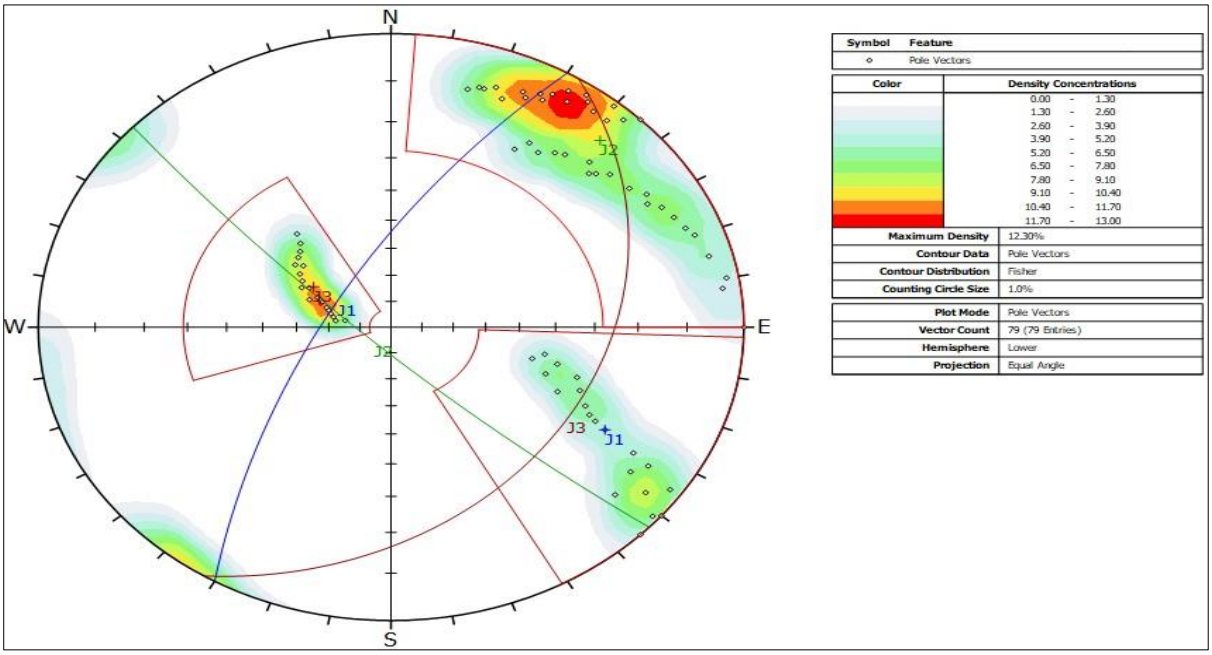


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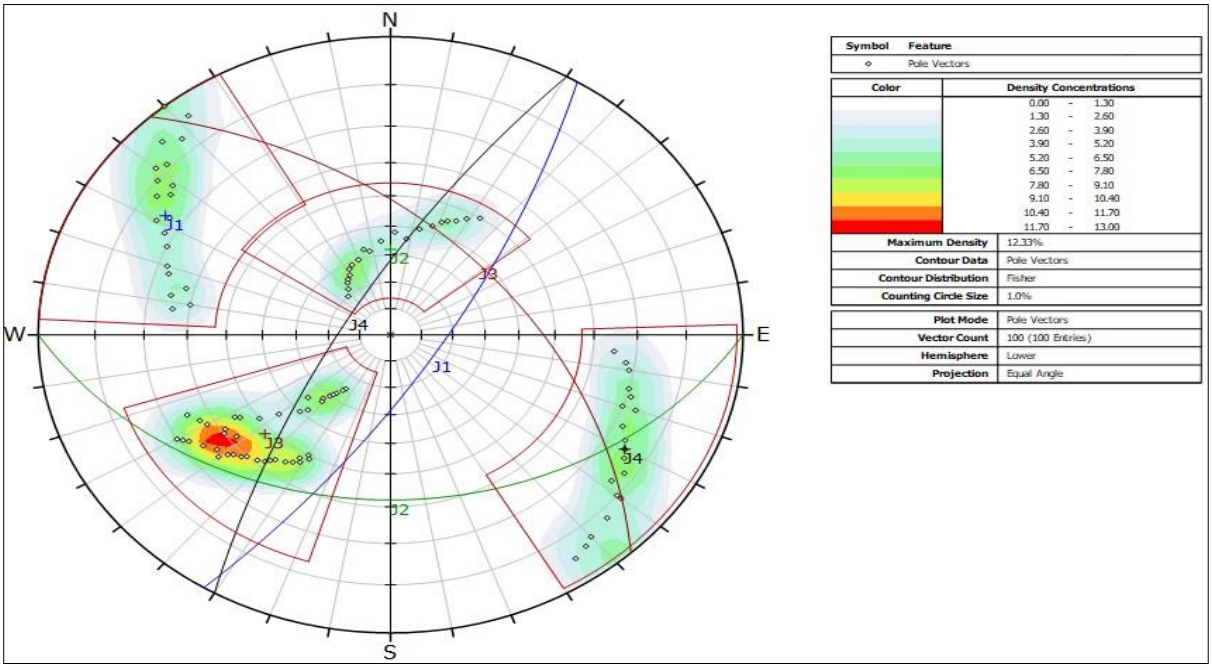
APPENDIX II: Structural Domains (Structural Mapping Data)

Appendix II present the major Joints Sets distributions by density concentration on the stereonet obtained from the structural mapping data from MR, 1LE, and 1LW Zone for structural domain of a mine.

A: MR Zone Joint Sets Distributions

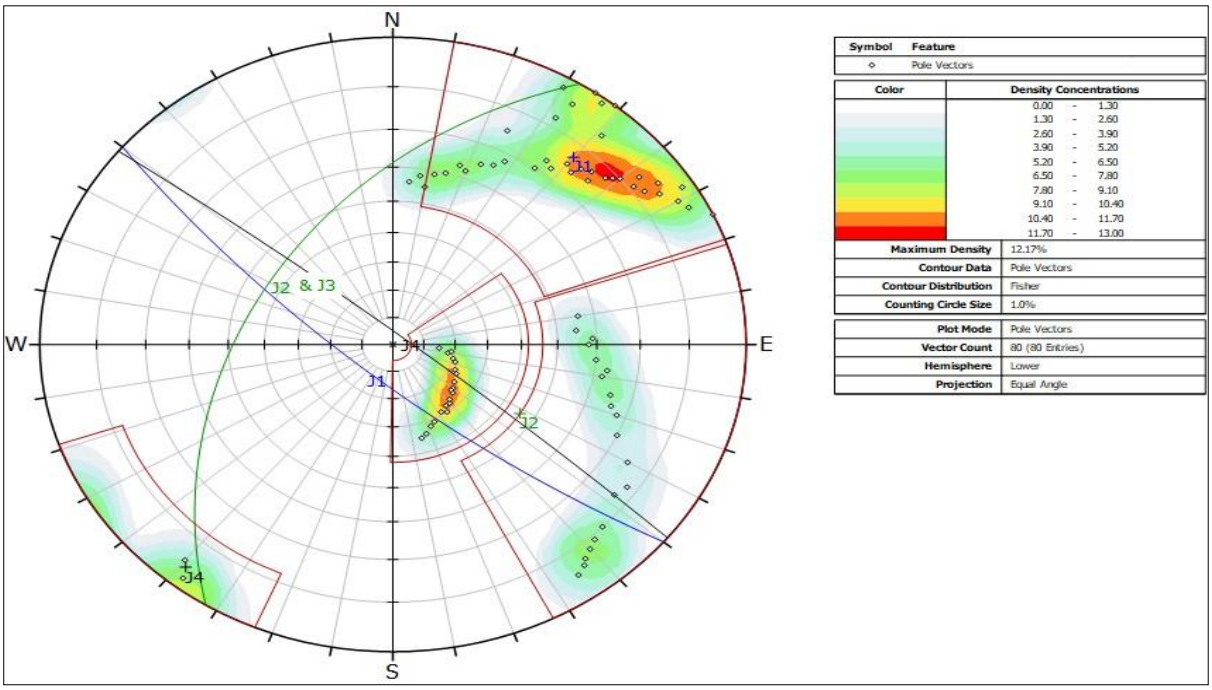


B: 1LE Zone Joint Sets Distributions



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C: 1LW Zone Joint Sets Distributions



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APPENDIX III: Rock Mass Quality (RMR and Q-System Classification)

Appendix III (A, B, and C) Present rock mass quality ratings for each zones.

A: MR Zone Rock Quality Measurements and Observations

GEO-CALCULATIONS			Project:	Master of Science			Locality:	MR Zone		Observer:	Sathiel Mphaphuli
			FIELD OBSERVATIONS and/or MEASUREMENTS				Date:	23/09/2023			
Locality	Type of locality		outcrop	cutting	in tunnel	Locality size =	15m	Orientations of main joint sets			
	Description / weathering		Moderate weathering			Photo :	-	strike (°)	dip (°)	dip direction	spacing (m)
Rock	Type of rock and description	Arg. Quartzite				Rock sample :	yes	210	75	300	8
						Clay sample :	-	120	85	210	6
	Rock weathering or alteration	fW	none / fresh	slight	moderate	high	complete	30	45	120	7
			a	b	c	d	e	150	80	240	8
	Rock strength	UCS	< 1MPa	1 - 5 MPa	5 - 25MPa	25 - 50MPa	50 - 75MPa	75 - 100MPa	100-150MPa	150 - 250MPa	> 250MPa
			a	b	c	d	e	f	g	h	i
Degree of jointing (fill in for RQD and/or Vb and/or Jv)	RQD	RQD	< 10	10 - 25	25 - 40	40 - 50	50 - 60	60 - 75	75 - 90	90 - 100	100
			a	b	c	d	e	f	g	h	i
	Block volume	Vb	< 1cm3	1 - 100cm3	0.1 - 1dm3	1 - 15dm3	15 - 125dm3	0.125- 1m3	1 - 8m3	8 - 50m3	> 50m3
			a	b	c	d	e	f	g	h	i
	Vol. joint count (joints/m3)	Jv	> 60	60 - 45	45 - 30	30 - 20	20 - 10	10 - 5	5 - 3	3 - 1	< 1
			a	b	c	d	e	f	g	h	i
	Joint spacing	Sa	> 2 m	0.6-2m	0.2 - 0.6 m	0.06 - 0.2m	< 0.06m	(This is the smallest joint spacing, often the main joint set)			
			a	b	c	d	e				
Number of joint sets (in area ca. 5m x tunnel span)		Jn	random joints only	1 joint set	1 set + random	2 joint sets	2 sets + random	3 joint sets	3 sets + random	4 joint sets	crushed
			a	b	c	d	e	f	g	h	i
Orientation of main joint set (related to tunnel)		Co	very favourable	favourable	fair	unfavourable	very unfavour.	Strike/dip (°) of main joint set → (related to tunnel)			300
			a	b	c	d	e				
Joint roughness (Jr)	Joint smoothness	js	very rough	rough	slightly rough	smooth	polished	slickensided	filled joint (seam)		(Jr = jw x js)
			a	b	c	d	e	f	g		
	Joint planarity	jw	discontinuous	strongly undul.	mod. undul.	slightly undul.	planar	filled joint (seam)			
			a	b	c	d	e	f			
Joint condition (fill in for A. or for B.)	A. Joint without filling	Ja	healed	fresh	slightly weath.	weathered	sand/silt coat.	clay coating	Direction(°) → of tunnel or cavern		320
			a	b	c	d	e	f			
	B. Joint with filling	filling < 5mm →	sand /silt	hard clay	soft clay	swelling clay	filling > 5mm →	sand /silt	hard clay	soft clay	swelling clay
			h	j	l	n		i	k	m	o
Joint type	Type of joints (joint length)	jL	crack	fissure	very short joint	short joint (1-3m)	mod. joint (3-10m)	long joint	seam (filled joint)		
			a	b	c	d	e	f	g		
	Joint separarion	e	< 0.1mm	0.1 - 1mm	1 - 5mm	5 - 25mm	Interlocking (IL) →	very tight	tight	disturbed	v. poor interlock
			a	b	c	d		a	b	c	d
Groundwater conditions		Jw	dry / above GWL	damp	wet	dripping	gushing	flowing	inburst		
			a	b	c	d	e	f	g		
Stress level or stress type		SRF	at surface	low stress	moderate stress	high stress	slight burst	mod. burst	strong burst	mild squeeze	high squeeze
			a	b	c	d	e	f	g	h	i
Weakness zone	Type of zone		complex zone	clayzone <50m	clayzone >50m	freq. shears	simple <50m	simple >50m	crushed zone		
			j	k	l	m	n	o	p		
	Orientation of zone	Coz	very favourable	favourable	fair	unfavourable	very unfavour.	Strike/dip (°) of zone →		related to N	related to tunnel
			a	b	c	d	e				
	Size (thickness) of zone	Tz	very small	small 0.3-1m	moderate 1-3m	large 3-10m	v. large >10m	Thickness (m) of zone →			2
			a	b	c	d	e				

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	Zone description	Shear zone affected by two faults

B: 1LE Zone Rock Quality Measurements and Observations

GEO-CALCULATIONS			**Project:**	*Master of Science 01*		**Locality:**	*1LE Zone*		**Observer:**	*Sathiel Mphaphuli*
			FIELD OBSERVATIONS and/or MEASUREMENTS					**Date:**	*03/11/2023*	
Locality	Type of locality		outcrop	cutting	in Stope	Locality size =	30m	**Orientations of main joint sets**		
	Description / weathering		Slightly weathered			Photo :	-	strike (°)	dip (°)	dip direction	spacing (m)
Rock	Type of rock and description		Arg. Quartzite			Rock sample :	yes	210	75	300	4
						Clay sample :	-	335	45	65	3
	Rock weathering or alteration	fW	none / fresh	slight	moderate	high	complete	85	35	175	4
			a	b	c	d	e	330	60	60	5
	Rock strength	UCS	< 1MPa	1 - 5 MPa	5 - 25MPa	25 - 50MPa	50 - 75MPa	75 - 100MPa	100-150MPa	150 - 250MPa	> 250MPa
			a	b	c	d	e	f	g	h	i
Degree of jointing (fill in for RQD and/or Vb and/or Jv)	RQD	RQD	< 10	10 - 25	25 - 40	40 - 50	50 - 60	60 - 75	75 - 90	90 - 100	100
			a	b	c	d	e	f	g	h	i
	Block volume	Vb	< 1cm3	1 - 100cm3	0.1 - 1dm3	1 - 15dm3	15 - 125dm3	0.125- 1m3	1 - 8m3	8 - 50m3	> 50m3
			a	b	c	d	e	f	g	h	i
	Vol. joint count (joints/m3)	Jv	> 60	60 - 45	45 - 30	30 - 20	20 - 10	10 - 5	5 - 3	3 - 1	< 1
			a	b	c	d	e	f	g	h	i
	Joint spacing	Sa	> 2 m	0.6-2m	0.2 - 0.6 m	0.06 - 0.2m	< 0.06m	(This is the smallest joint spacing, often the main joint set)			
			a	b	c	d	e				
Number of joint sets (in area ca. 5m x tunnel span)		Jn	random joints only	1 joint set	1 set + random	2 joint sets	2 sets + random	3 joint sets	3 sets + random	4 joint sets	crushed
			a	b	c	d	e	f	g	h	i
Orientation of main joint set (related to tunnel)		Co	very favourable	favourable	fair	unfavourable	very unfavour.	**Strike/dip (°) of main joint set →** (related to stope)			**300**
			a	**b**	c	d	e				
Joint roughness (Jr)	Joint smoothness	js	very rough	rough	slightly rough	smooth	polished	slickensided	filled joint (seam)		(Jr = jw x js)
			a	b	**c**	d	e	f	g		
	Joint planarity	jw	discontinuous	strongly undul.	mod. undul.	slightly undul.	planar	filled joint (seam)			
			a	b	c	**d**	e	f			
Joint condition (fill in for A. or for B.)	A. Joint without filling	Ja	healed	fresh	slightly weath.	weathered	sand/silt coat.	clay coating		**Direction(°) → of stope**	90E
			a	**b**	c	d	e	f			
	B. Joint with filling	filling < 5mm →	sand /silt	hard clay	soft clay	swelling clay	filling > 5mm →	sand /silt	hard clay	soft clay	swelling clay
			h	j	l	n		i	k	m	o
Joint type	Type of joints (joint length)	jL	crack	fissure	very short joint	short joint (1-3m)	mod. joint (3-10m)	long joint	seam (filled joint)		
			a	b	c	d	**e**	f	g		
	Joint separarion	e	< 0.1mm	0.1 - 1mm	1 - 5mm	5 - 25mm	**Interlocking** (IL) →	very tight	tight	disturbed	v. poor interlock
			a	**b**	**c**	d		a	**b**	c	d
Groundwater conditions		Jw	dry / above GWL	damp	wet	dripping	gushing	flowing	inburst		
			a	b	c	d	e	f	g		
Stress level or stress type		SRF	at surface	low stress	moderate stress	high stress	slight burst	mod. burst	strong burst	mild squeeze	high squeeze
			a	b	**c**	d	e	f	g	h	i
Weakness zone	Type of zone		complex zone	clayzone <50m	clayzone >50m	freq. shears	simple <50m	simple >50m	crushed zone		
			j	k	l	m	n	o	p		
	Orientation of zone	Coz	very favourable	favourable	fair	unfavourable	very unfavour.	**Strike/dip (°) of zone →**		related to N	related to stope
			a	**b**	c	d	e			54	**320**
	Size (thickness) of zone	Tz	very small	small 0.3-1m	moderate 1-3m	large 3-10m	v. large >10m	**Thickness (m) of zone →**			**1.5**
			a	b	c	d	e				
	Zone description		Dyke intrusion (Crosscutting the existing rock strata)								

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C: 1LW Zone Rock Quality Measurements and Observations

GEO-CALCULATIONS			Project:	Master of Science 01		Locality:	1LW Zone		Observer:	Sathiel Mphaphuli
			FIELD OBSERVATIONS and/or MEASUREMENTS				Date:	11/06/2024		
Locality	Type of locality		outcrop	cutting	in Stope	Locality size =	30m	**Orientations of main joint sets**		
	Description / weathering		Moderately weathered			Photo :	-	strike (°)	dip (°)	dip direction	spacing (m)
Rock	Type of rock and description	Arg. Quartzite				Rock sample :	yes	205	70	295	5
						Clay sample :	--	115	85	205	4
	Rock weathering or alteration	fW	none / fresh	slight	moderate	high	complete	220	25	310	3
			a	b	c	d	e	140	80	230	5
	Rock strength	UCS	< 1MPa	1 - 5 MPa	5 - 25MPa	25 - 50MPa	50 - 75MPa	75 - 100MPa	100-150MPa	150 - 250MPa	> 250MPa
			a	b	c	d	e	f	g	h	i
Degree of jointing (fill in for RQD and/or Vb and/or Jv)	RQD	RQD	< 10	10 - 25	25 - 40	40 - 50	50 - 60	60 - 75	75 - 90	90 - 100	100
			a	b	c	d	e	f	g	h	i
	Block volume	Vb	< 1cm3	1 - 100cm3	0.1 - 1dm3	1 - 15dm3	15 - 125dm3	0.125- 1m3	1 - 8m3	8 - 50m3	> 50m3
			a	b	c	d	e	f	g	h	i
	Vol. joint count (joints/m3)	Jv	> 60	60 - 45	45 - 30	30 - 20	20 - 10	10 - 5	5 - 3	3 - 1	< 1
			a	b	c	d	e	f	g	h	i
	Joint spacing	Sa	> 2 m	0.6-2m	0.2 - 0.6 m	0.06 - 0.2m	< 0.06m	(This is the smallest joint spacing, often the main joint set)			
			a	b	c	d	e				
Number of joint sets (in area ca. 5m x tunnel span)		Jn	random joints only	1 joint set	1 set + random	2 joint sets	2 sets + random	3 joint sets	3 sets + random	4 joint sets	crushed
			a	b	c	d	e	f	g	h	i
Orientation of main joint set (related to tunnel)		Co	very favourable	favourable	fair	unfavourable	very unfavour.	**Strike/dip (°) of main joint set →** (related to stope)			**295**
			a	**b**	c	d	e				
Joint roughness (Jr)	Joint smoothness	js	very rough	rough	slightly rough	smooth	polished	slickensided	filled joint (seam)		(Jr = jw x js)
			a	b	**c**	d	e	f	g		
	Joint planarity	jw	discontinuous	strongly undul.	mod. undul.	slightly undul.	planar	filled joint (seam)			
			a	b	c	**d**	e	f			
Joint condition (fill in for A. or for B.)	A. Joint without filling	Ja	healed	fresh	slightly weath.	weathered	sand/silt coat.	clay coating	**Direction(°) → of stope**		270W
			a	**b**	c	d	e	f			
	B. Joint with filling	filling < 5mm →	sand /silt	hard clay	soft clay	swelling clay	filling > 5mm →	sand /silt	hard clay	soft clay	swelling clay
			h	j	l	n		i	k	m	o
Joint type	Type of joints (joint length)	jL	crack	fissure	very short joint	short joint (1-3m)	mod. joint (3-10m)	long joint	seam (filled joint)		
			a	b	c	d	**e**	f	g		
	Joint separarion	e	< 0.1mm	0.1 - 1mm	1 - 5mm	5 - 25mm	**Interlocking (IL) →**	very tight	tight	disturbed	v. poor interlock
			a	**b**	**c**	d		a	**b**	c	d
Groundwater conditions		Jw	dry / above GWL	damp	wet	dripping	gushing	flowing	inburst		
			a	**b**	c	d	e	f	g		
Stress level or stress type		SRF	at surface	low stress	moderate stress	high stress	slight burst	mod. burst	strong burst	mild squeeze	high squeeze
			a	b	**c**	d	e	f	g	h	i
Weakness zone	Type of zone		complex zone	clayzone <50m	clayzone >50m	freq. shears	simple <50m	simple >50m	crushed zone		
			j	k	l	m	n	o	p		
	Orientation of zone	Coz	very favourable	favourable	fair	unfavourable	very unfavour.	**Strike/dip (°) of zone →**		related to N	related to stope
			a	**b**	**c**	d	e			310	**35**
	Size (thickness) of zone	Tz	very small	small 0.3-1m	moderate 1-3m	large 3-10m	v. large >10m	**Thickness (m) of zone →**			**3**
			a	b	c	d	e				
	Zone description	Multiple occurrences of weakness zone (dykes and faults)									

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D: Rock Mass Rating System guideline (After Bieniawski 1989)

RMR Classification: presenting the classification parameters and their ratings.

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS									
Parameter			Range of values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	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		> 2 m	0.6 - 2 . m	200 - 600 mm	60 - 200 mm	< 60 mm		
	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 < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Ground water	Inflow per 10 m 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 & mines	0	-2	-5	-10	-12		
		Foundations	0	-2	-7	-15	-25		
		Slopes	0	-5	-25	-50			
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		

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E: Classification of individual parameters (guideline) used in the Q-System (After Barton et al 1974)

DESCRIPTION	VALUE	NOTES
1. ROCK QUALITY DESIGNATION	***RQD***	
A. Very poor	0 - 25	1. Where *RQD* is reported or measured as ≤ 10 (including 0), a nominal value of 10 is used to evaluate *Q*.
B. Poor	25 - 50	
C. Fair	50 - 75	
D. Good	75 - 90	2. *RQD* intervals of 5, i.e. 100, 95, 90 etc. are sufficiently accurate.
E. Excellent	90 - 100	
2. JOINT SET NUMBER	J_n	
A. Massive, no or few joints	0.5 - 1.0	
B. One joint set	2	
C. One joint set plus random	3	
D. Two joint sets	4	
E. Two joint sets plus random	6	
F. Three joint sets	9	1. For intersections use ($3.0 \times J_n$)
G. Three joint sets plus random	12	
H. Four or more joint sets, random, heavily jointed, 'sugar cube', etc.	15	2. For portals use ($2.0 \times J_n$)
J. Crushed rock, earthlike	20	
3. JOINT ROUGHNESS NUMBER	J_r	
a. Rock wall contact		
b. Rock wall contact before 10 cm shear		
A. Discontinuous joints	4	
B. Rough and irregular, undulating	3	
C. Smooth undulating	2	
D. Slickensided undulating	1.5	1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m.
E. Rough or irregular, planar	1.5	
F. Smooth, planar	1.0	
G. Slickensided, planar	0.5	2. J_r = 0.5 can be used for planar, slickensided joints having lineations, provided that the lineations are oriented for minimum strength.
c. No rock wall contact when sheared		
H. Zones containing clay minerals thick enough to prevent rock wall contact	1.0 (nominal)	
J. Sandy, gravely or crushed zone thick enough to prevent rock wall contact	1.0 (nominal)	
4. JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)
a. Rock wall contact		
A. Tightly healed, hard, non-softening, impermeable filling	0.75	1. Values of ϕr, the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present.
B. Unaltered joint walls, surface staining only	1.0	25 - 35
C. Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	25 - 30
D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	20 - 25
E. Softening or low-friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1 - 2 mm or less in thickness)	4.0	8 - 16

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4. JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)	
b. Rock wall contact before 10 cm shear			
F. Sandy particles, clay-free, disintegrating rock etc.	4.0	25 - 30	
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous < 5 mm thick)	6.0	16 - 24	
H. Medium or low over-consolidation, softening clay mineral fillings (continuous < 5 mm thick)	8.0	12 - 16	
J. Swelling clay fillings, i.e. montmorillonite, (continuous < 5 mm thick). Values of J_a depend on percent of swelling clay-size particles, and access to water.	8.0 - 12.0	6 - 12	
c. No rock wall contact when sheared			
K. Zones or bands of disintegrated or crushed	6.0		
L. rock and clay (see G, H and J for clay	8.0		
M. conditions)	8.0 - 12.0	6 - 24	
N. Zones or bands of silty- or sandy-clay, small clay fraction, non-softening	5.0		
O. Thick continuous zones or bands of clay	10.0 - 13.0		
P. & R. (see G.H and J for clay conditions)	6.0 - 24.0		
5. JOINT WATER REDUCTION	J_w	approx. water pressure (kgf/cm^2)	
A. Dry excavation or minor inflow i.e. < 5 l/m locally	1.0	< 1.0	
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0 - 2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5 - 10.0	1. Factors C to F are crude estimates; increase J_w if drainage installed.
D. Large inflow or high pressure	0.33	2.5 - 10.0	
E. Exceptionally high inflow or pressure at blasting, decaying with time	0.2 - 0.1	> 10	2. Special problems caused by ice formation are not considered.
F. Exceptionally high inflow or pressure	0.1 - 0.05	> 10	
6. STRESS REDUCTION FACTOR		***SRF***	
a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated			
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock any depth)		10.0	1. Reduce these values of *SRF* by 25 - 50% but only if the relevant shear zones influence do not intersect the excavation
B. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth < 50 m)		5.0	
C. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth > 50 m)		2.5	
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)		7.5	
E. Single shear zone in competent rock (clay free). (depth of excavation < 50 m)		5.0	
F. Single shear zone in competent rock (clay free). (depth of excavation > 50 m)		2.5	
G. Loose open joints, heavily jointed or 'sugar cube', (any depth)		5.0	

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DESCRIPTION		VALUE		NOTES
6. STRESS REDUCTION FACTOR			***SRF***	
b. Competent rock, rock stress problems				
	σ_c/σ_1	σ_t/σ_1		2. For strongly anisotropic virgin stress field
H. Low stress, near surface	> 200	> 13	2.5	(if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c
J. Medium stress	200 - 10	13 - 0.66	1.0	to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$. When σ_1/σ_3 > 10,
K. High stress, very tight structure	10 - 5	0.66 - 0.33	0.5 - 2	reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where
(usually favourable to stability, may				σ_c = unconfined compressive strength, and
be unfavourable to wall stability)				σ_t = tensile strength (point load) and σ_1 and
L. Mild rockburst (massive rock)	5 - 2.5	0.33 - 0.16	5 - 10	σ_3 are the major and minor principal stresses.
M. Heavy rockburst (massive rock)	< 2.5	< 0.16	10 - 20	3. Few case records available where depth of
c. Squeezing rock, plastic flow of incompetent rock				crown below surface is less than span width.
under influence of high rock pressure				Suggest *SRF* increase from 2.5 to 5 for such
N. Mild squeezing rock pressure			5 - 10	cases (see H).
O. Heavy squeezing rock pressure			10 - 20	
d. Swelling rock, chemical swelling activity depending on presence of water				
P. Mild swelling rock pressure			5 - 10	
R. Heavy swelling rock pressure			10 - 15	

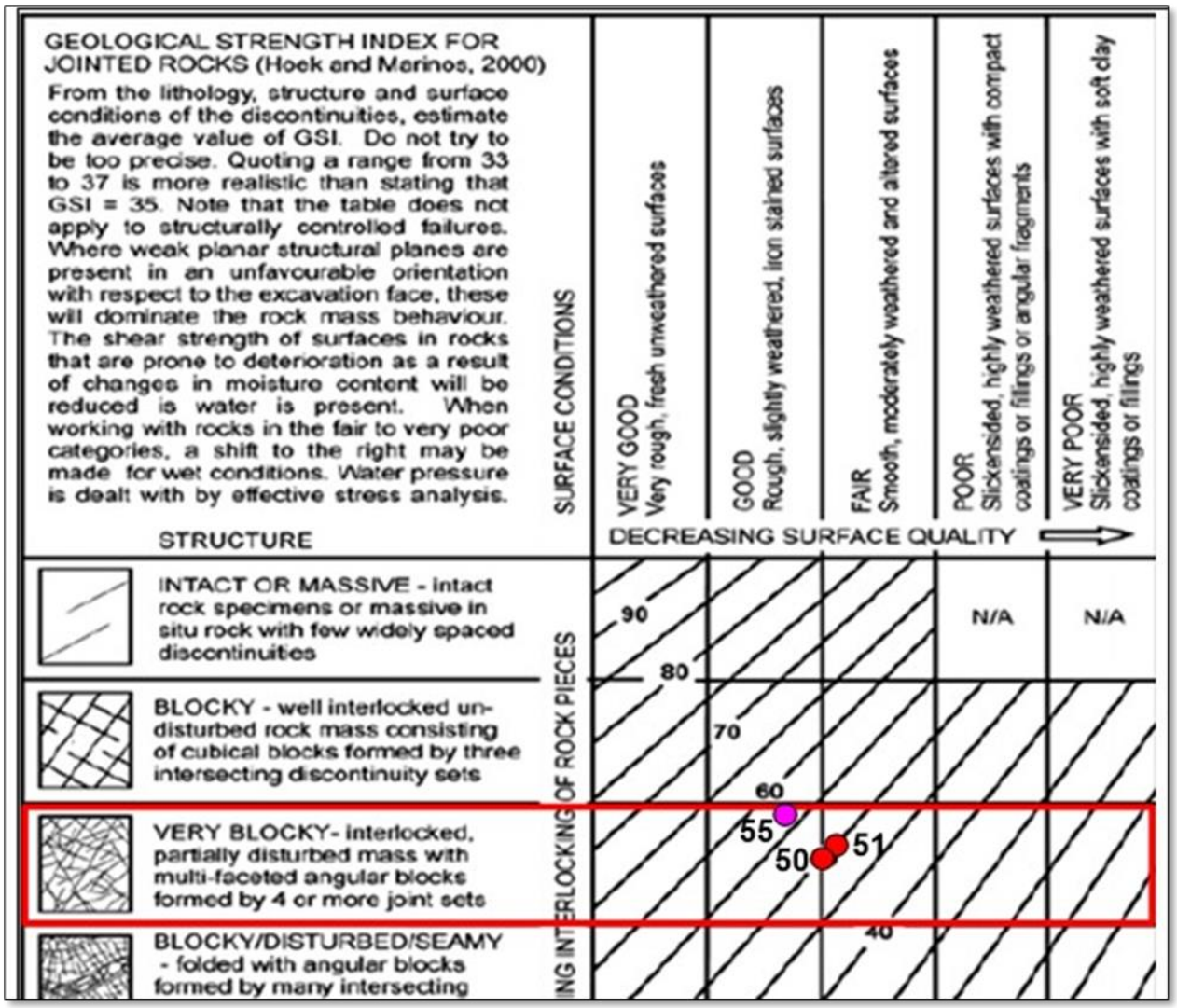
ADDITIONAL NOTES ON THE USE OF THESE TABLES

When making estimates of the rock mass Quality (*Q*), the following guidelines should be followed in addition to the notes listed in the tables:

1. When borehole core is unavailable, *RQD* can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relationship can be used to convert this number to *RQD* for the case of clay free rock masses: $RQD = 115 - 3.3\ J_v$ (approx.), where J_v = total number of joints per m^3 ($0 < RQD < 100$ for $35 > J_v > 4.5$).
2. The parameter J_n representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed, these parallel 'joints' should obviously be counted as a complete joint set. However, if there are few 'joints' visible, or if only occasional breaks in the core are due to these features, then it will be more appropriate to count them as 'random' joints when evaluating J_n.
3. The parameters J_r and J_a (representing shear strength) should be relevant to the weakest significant joint set or clay filled discontinuity in the given zone. However, if the joint set or discontinuity with the minimum value of J_r/J_a is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of J_r/J_a should be used when evaluating Q. The value of J_r/J_a should in fact relate to the surface most likely to allow failure to initiate.
4. When a rock mass contains clay, the factor *SRF* appropriate to loosening loads should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent, the strength of the intact rock may become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength. A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.
5. The compressive and tensile strengths (σ_c and σ_t) of the intact rock should be evaluated in the saturated condition if this is appropriate to the present and future in situ conditions. A very conservative estimate of the strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

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F: The GSI Chart summarizing the rock quality and degree of fracturing at MR, 1LE, and 1LW Zones



G: Rock Mass Classification Values for each Geotechnical Domain

Zones	Geotech Domains	RMR_{89}	Q-system	GSI
MR	Red	56	0.17	51
1LE	Pink	60	0.67	55
1LW	Red	55	0.11	50

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