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Fall of ground management in shallow underground hard rock mining: a case study in a Chrome mine

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A research report submitted to the Faculty of Engineering and the Built Environment, University of the Witwatersrand, Johannesburg, in partial fulfilment of the requirements for the degree of Master of Science in Engineering.

Johannesburg, 2018

DECLARATION

I declare that this research report is my own unaided work. It is being submitted for the Degree of Master in Science in Engineering to the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination to any other University.

…….

Netshilaphala Vhutali Carol

…….day of …….. (Year) …….

At ……

ABSTRACT

Rock fall related hazards will forever be present in the mining industry, therefore it is considered important to have an active fall of ground (FOG) management technique that will help combat such incidents. The Chrome mine has experienced numerous falls of ground incidents in the previous years. This was a major concern, as there was already a ground control and monitoring system in place. The current fall of ground management technique includes visual observations where panels are rated (Panel Risk Rating) based on their geological complexity and compliance to support standards. In addition, monitoring such as Ground Penetrating Radar (GPR), Borehole camera inspections and FOG lights are used. With such defined ground control measures put in place, rock falls continue to occur.

The main aim of this study was to identify loopholes within the current ground control system in order to know how it can be improved to be more efficient. This was achieved by looking at the current FOGs management technique used at the mine together with historical fall of ground database so as to identify the main causes of FOGs. It was found that most falls of ground were due to intersection of joints and flat dipping structures such as domes. Main challenges identified within the current system include:

- Inadequate early entry examination (prominent hazards identified during PRR relate to early entry examination);
- GPR scanning is only conducted on a monthly basis, 10m from the current face to the back area, in strike direction only; and
- FOG lights trigger easily due to blast vibrations.

The above listed challenges can be addressed as follows, to help improve the current system to combat FOGs at the mine.

- By ensuring that early entry examination is conducted adequately by competent and skilled personnel;
- GPR scanning can be conducted in both dip and strike direction, after every 3 blasts (for an average advance per blast of 3m) instead of once a month to avoid leaving out some areas not scanned (i.e. Portions between the area scanned in the previous month and 10m from the current face area);
- Rocktales can replace FOG lights for continuous ground monitoring.

Having identified the main causes of FOGs, it was noticed that the current ground control and monitoring system did not consider the impact of joint as there was no documented joints data other than that presented in the Mine's Code of Practice. Underground joint mapping was conducted using window mapping to obtain joint data. Furthermore, two methods i.e. brow thickness data and historical fall thickness were used to estimate the overall fallout thickness. Probabilistic approach was used to evaluate rock falls in the North and South sections of the mine. This approach also allows the evaluation of different support systems.

Fallout thickness outcome from both approaches confirmed the estimate from joint data analysis in J-Block. Key block size distribution indicated that about 80% of key blocks formed are $1m^3$ in both sections. Meanwhile fallout thickness evaluations showed that the overall fallout thickness at the Chrome mine is 0.95m. Rock fall probability evaluations indicate that small blocks tend to fall between supports whereas larger blocks fail the support system through rotation. Support failure may occur if the support resistance is smaller than the required resistance. Support layout analysis show that spacing, length and capacity of support units have an effect on stability. These aspects are critical for support design of jointed rock mass. Furthermore, to prevent small blocks from falling between support units, areal coverage (wire mesh, Oslo straps, shotcrete etc.) is suggested in heavily jointed rock masses.

This study has demonstrated that falls of ground management strategies are broad. The more the techniques are applied in a FOG management system, the better the outcome. Moreover, the study showed that the use of joint data can improve the current fall of ground management system at the mine, through its ability to evaluate the stability of excavations. As a result, awareness of the expected key blocks in a particular excavation is achieved, which helps in support design. Therefore, when continuous ground monitoring devices are installed, it will be known that a certain block size has moved. Thus, correct measures can be put in place to minimise rock fall related hazards. The same approach can be utilised while focusing on a single panel rather than the whole mine or section, which is strategic for stability enhancement.

ACKNOWLEDGEMENTS

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CHAPTER 1: INTRODUCTION

Fall of ground is one of the major hazards in underground mine operations. Generally, fall of ground is gravity induced due to the interaction of geological discontinuities such as joints, faults, domes, and other planes of weakness within the rock mass. The frequency of fall of ground is a concern in most mines as it may result in fatalities, injuries and damage to equipment. It is therefore important to have a sound falls of ground (FOGs) management plan in a mine in order to minimise FOGs occurrence. This research study is based on a Chrome mine located in Limpopo province, in the Eastern Bushveld Complex. Illustrated in Figure 1.1, is the geology and Chrome mines located in the Bushveld Complex.

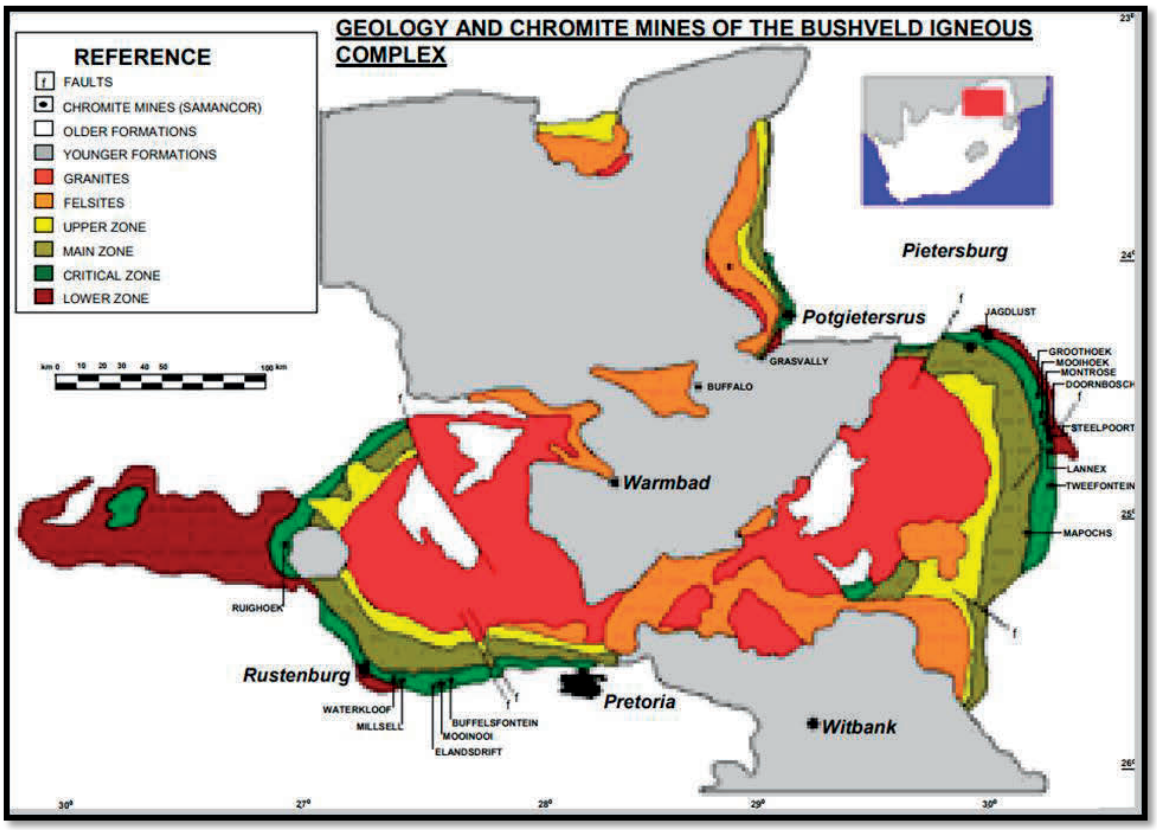


Figure 1.1: Geology and Chrome mines in the Bushveld Complex *(Internal Mine Document, 2013)*

There are two aspects in mining that cannot be changed. These are the rock mass in which the ore body is located, and the occurrence (shape, size and orientation) and grade of the orebody

(Yilmaz, 2015). Ryder & Jager (2002) described a rock as a complex engineering material of which its behaviour is influenced by numerous factors. In geotechnical engineering work, inconsistency between predictions of the rock behaviour when loaded as opposed to the performance of underground rock support, can take place. Mine designs based on these predictions tend to be traditional, whereas at times they are actually not safe (Ryder & Jager, 2002). The geology in the eastern BC consists of significant faults, dykes, joints, domes and potholes. These geological features influence the behaviour of the rock mass thereof. Fall of ground (FOG) has been one of the major causes of injuries, fatalities and accidents in underground mining. FOG can occur immediately after blasting has taken place or after ground support has been installed, where rocks can dislodge in-between support units. In other cases, FOGs are due to support failure. It is, therefore, vital to have in place measures to manage falls of ground and unplanned ground movement in a mine.

1.1. Problem statement

Rock fall related hazards have been a concern in the mining industry. The mining method employed at the Chrome mine is mechanised bord and pillar, and the rock mass is characterised mainly by joints, faults and flat dipping discontinuities such as domes. Interaction of these structures form unstable blocks with potential to fall under the influence of gravity. Although regional and local support in the form of pillars and tendons respectively are put in place, rock falls are still experienced. Injuries and accidents related to rock falls are being recorded timeously. It was for this reason that the Rock Engineering Department implemented a ground control and monitoring system as a way of managing falls of ground. The system includes visual observations (Panel Risk Rating) and the use of Ground Penetrating Radar (GPR) scanning, Borehole camera and Fall of Ground (FOG) light for continuous monitoring of the hanging wall strata. However, even after the implementation of ground control measures, falls of ground continue to occur. Several cases of FOG have been recorded in the past eight years. This project study intends to investigate loopholes within the existing ground control and monitoring system at the mine through evaluation of the current ground monitoring strategies.

1.2. Research background and motivation

The potential for rock falls to occur in underground mining environment is highly dependent on the location and presence of geological discontinuities such as joints, faults, fractures and other weak planes in the rock mass. An interaction between these structures may result in the formation of a wedge or blocks of rock, with potential to induce fall of ground (FOG). Implementation of sound fall of ground management strategy in such an environment is necessary to help minimise and control rock related hazards. At the studied Chrome mine, there is lack of sound systematic approach to help stabilise underground mine excavations.

Without sound strategic planning and design in place, the risks associated with FOG increase. Although solid ground is left unmined (pillar) as a way of stabilising the underground excavations, together with other support units, the probability of rock failure remains. According to the Mining Industry Occupational Health & Safety statistics, from 1994 to 2014, rock fall related hazards accounted up to 30% of injuries and fatalities in South African mines (Anon, nd). The zero harm milestones in the South African mining sector can only be achieved through active fall of ground management systems, appropriate design layouts and adequate support. Fall of ground management helps to monitor the effectiveness of support design and ground control measures in place. Improvement in safety to achieve zero harm in the mining industry motivates this research.

1.3. Research objectives

The main objective for conducting this study is to come up with a sound systematic approach to help minimise and control falls of ground at the Chrome mine. Specific objectives include:

- To understand the input parameters required for mining layout and support design to minimise rock fall risks at the Chrome mine.
- Determining the effectiveness of ground monitoring equipment.
- Determine the mode of rock failure.
- Ensuring support unit length is adequate to clamp the average fallout thickness at the mine.

1.4. Research methodology

The methodology adopted for this research study includes:

- Review of the area of study (mining environment, mining layout and support design);
- Review and analysis of fall of ground database from previous years;
- Review of fall of ground management system currently used;
- Collecting brow thickness data for fallout thickness analysis;
- Reviewing the characterisation and measurement of rock mass jointing;
- Underground joint mapping; and
- Use of joint mapping results in the prediction of unstable blocks of rock.

A review of the current falls of ground management system was conducted to identify loopholes and challenges with the aim of improving the system to help combat FOGs at the mine. Underground joint mapping was conducted in the North and South sections of the mine. Expected outcome from the above mentioned exercise is the development of joint characterisation database, which include joint orientation, spacing and length. The joint data was analysed in order to incorporate numerical modelling into the fall of ground management system. A commercially available rock engineering software program called J-Block was used in this regard for probabilistic analysis. A stability evaluation was carried out for two mining orientations (panels mining on strike and up dip holing) in order to identify potential unstable blocks.

1.5. Content of the report

Chapter 2 details the literature and practical examples around the concept of falls of ground in shallow hard rock mining environments. A discussion of the current fall of ground management strategy employed at the mine is given in Chapter 3. Analyses of fall of ground database and fallout thickness are presented in Chapter 4. Chapter 5 focuses on characteristics of a jointed rock mass and how joint data can be collected underground. Underground joint mapping procedures and results from the Chrome mine are presented in Chapter 6, and in Chapter 7, these results are analysed using the probabilistic approach. Discussion on relevant improvements that could help combat FOGs at the mine, are given in Chapter 8. The research study recommendations and conclusions are provided in Chapter 9.

CHAPTER 2: REVIEW OF SHALLOW HARD ROCK MINING FOGs

2.1. Introduction

The term fall of ground (FOG) is used to classify incidents related to unexpected rock mass movement or the uncontrolled release of rock in excavations due to gravity, pressure, or rockburst. Occurrence of planes of weakness in the hanging wall strata is a major factor of excavation stability in shallow underground mining environments. This is because the interaction of these planes may result in unstable blocks of rocks with potential to fall under the influence of gravity. This Chapter aims at providing literature and practical examples around the concept of falls of ground in shallow hard rock mining environments. Firstly, a brief history on FOGs in the mining sector is given in section 2.2. In sections 2.3 and 2.4, a review of the geological background of the study area and the behaviour of rock masses at shallow mining depths is presented respectively. The influence of mining layout and support design on stability is also explained in section 2.5. Conclusions are given in the last section of this chapter.

2.2. Falls of ground in the mining sector

Mining is considered the most dangerous industry in the world. This is because mining has ever present hazards than any industry due to the potential for falling rock. Historical analysis by Leger (1991) show that fatalities in the mining industry were mostly attributed to rock falls from the early stages of mining in South Africa. Rock fall accounts for most fatalities in the mining industry (Figure 2.1). Although the number of fatalities has reduced over the years, fall of ground remains the main cause of fatalities in the mining industry (Anon, 2017).

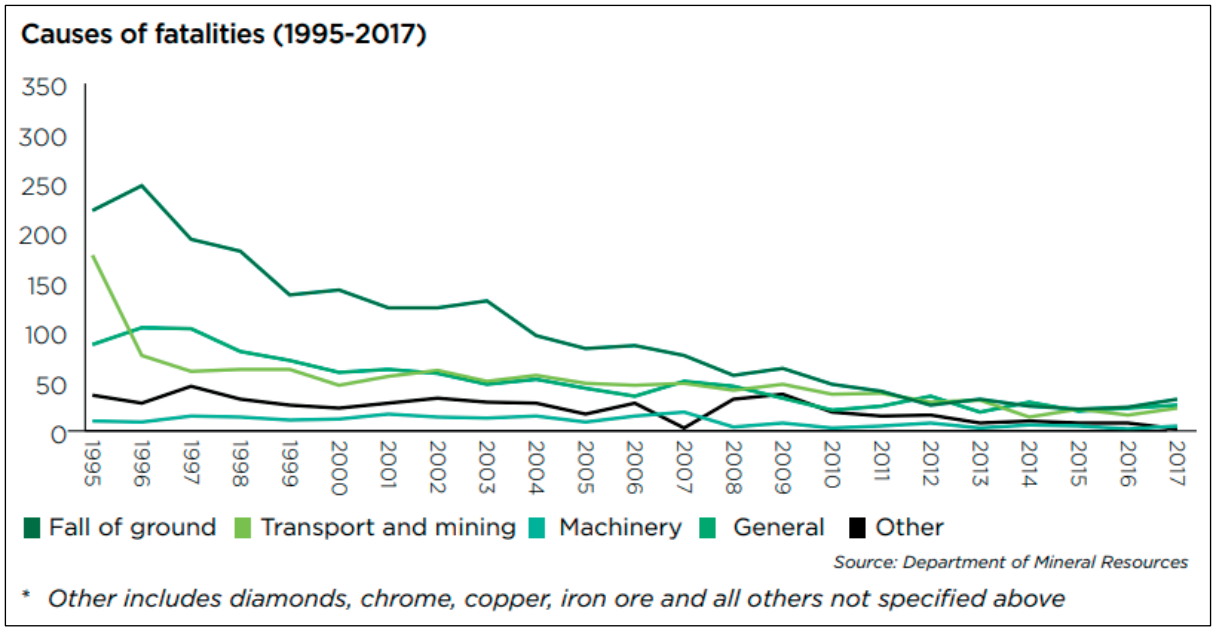


Figure 2.1: Causes of fatality in the mining industry *(Anon, 2017)*

To date, fall of ground incidents account for most fatalities and injuries in the South African mining industry. In 2017, there was a rising concern on the number of accidents relating to rockbursts and falls of ground (Anon, 2017). Although rock related accidents are more serious in some commodities than others within the same industry, elimination and control of these accidents remains fundamental for the South African mining sector to reach its zero harm milestones.

Most rock fall related accidents, according to the South African Mining Industry Occupational Safety and Health (Anon, nd); happen at the face where most of the work is done. After blast, the rock in the excavation is unstable because it is adjusting to new stresses since it has just been exposed by the blast and not yet supported with primary support (Anon, nd). In other cases, rocks dislodge from the hanging wall due to failure of support, where the 'capacity' of the support units does not meet the 'demand'. It is important to understand strategies that can be used to combat rock fall. These strategies can be understood better with the knowledge of what causes the falls of ground in underground mining environment.

The South African mining industry has experienced a noticeable increase in rockbursts in the year 2017 (Anon, 2017) .Generally, deep mines are prone to rockburst and falls of ground, which are both rock fall related hazard. The geological location and depth of mining influence to some degree the factors that contribute to unstable blocks of ground, which have

potential to fall under the influence of gravity. Since this research study is based on a specific Chrome mine at shallow depth, it is considered appropriate to reiterate briefly the geological setting of the study area.

2.3. Geological background of the study area

The Palaeoproterozoic Bushveld Complex in South Africa is located in the central northeast portion of the Kaapvaal Craton, and it is said to be the largest intrusive complex because of its economic mineral resources in quantities unparalleled elsewhere in the world (Klemm et al., 1982; Schouwstra & Kinloch, 2000; Lenhardt & Eriksson, 2012; Von Gruenewaldt et al., 1985 and Kinnaird, 2005). These mineral resources include platinum group elements (PGEs) and chromitite (Cawthorn, 2015). The Bushveld Complex (BC) comprises of four limbs spreading across four provinces in South Africa (Limpopo, Mpumalanga, Gauteng and North West); namely the eastern, western, southern and northern limbs (Roelofse & Ashwal, 2012). The eastern and western limbs are more exposed as compared to the northern and southern limbs (Kinnaird, 2005). Figure 2.2 shows the geological distribution of the Bushveld Complex.

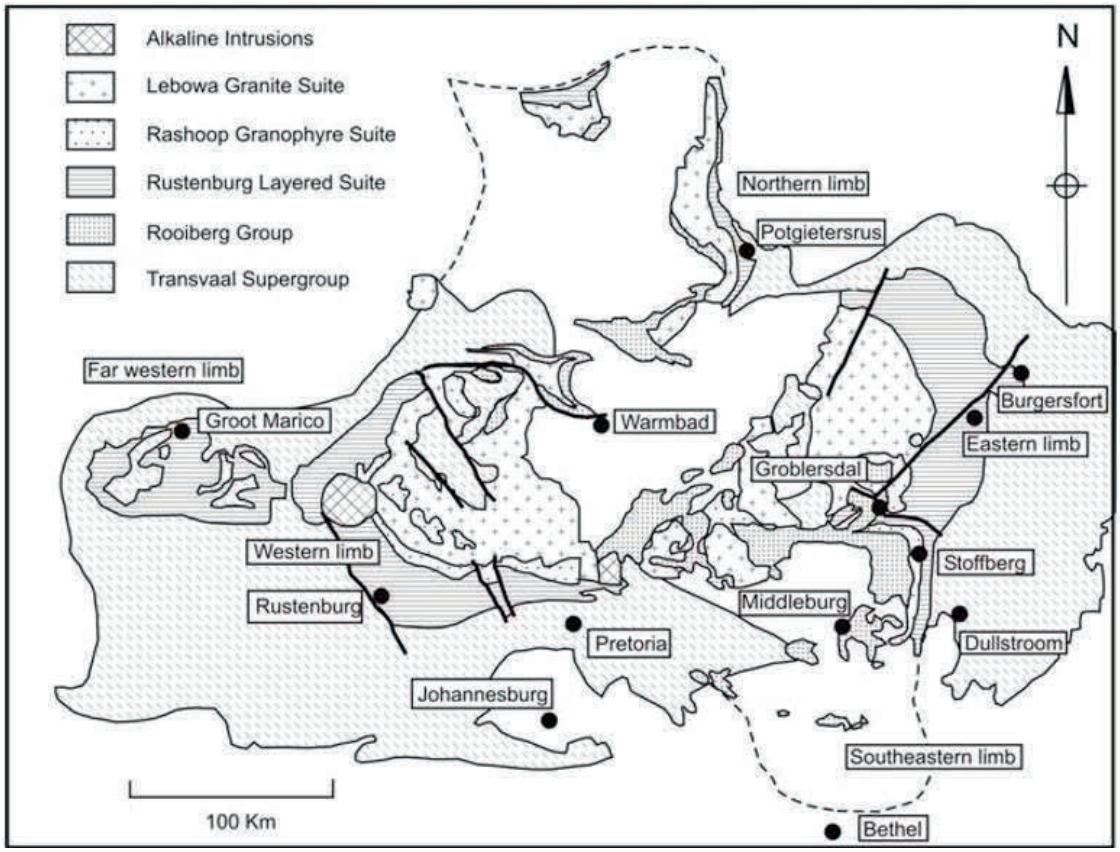


Figure 2.2: Geological distribution of the Bushveld Complex *(Kinnaird, 2005)*

Amongst others, the Rustenburg Layered Suite of the BC stratigraphically comprises of a Marginal zone, Lower zone, Critical zone, Main zone and Upper zone. The Upper Critical zone contains both the Merenskey reef and the UG2 reef (PGEs) (Kruger , 1990; Uken , 1998; Eales & Cawthorn, 1996). The Critical zone found in both the eastern and western limbs hosts the largest stratified chromitie reserves in the world (Kinnaird, 2005).

The BC is subdivided into groups based on their formation, and dominant rock types as shown in Table 2.1. Different rock types will behave differently under the same stress conditions because of their formation and mineral composition. Engineering properties of rocks (strength, weathering, hardness and durability) are influenced by the formation and mineral composition of the rock. The rock texture and degree of porosity will also have an impact on the strength of the rock (Ryder & Jager, 2002).

Table 2.1: Subdivisions of the Bushveld Complex with dominant rock types *(Nong, 2010)*

Lebowa granite Suite	Nebo, Makhutso, klipkloof, Bobbejaankop and Verana granites
Rashoop Granophyre suite	Stavoren and Diepkloof granophyres, Rooikop porphyritic granite, Zwartbank pseudogranophyre
Rustenburg Layered Suite	**Upper Zone:** Subzone C (diorite), Subzone B (gabbronorite), Subzone A (gabbronorite, norite) **Main zone:** Upper subzone (gabronotite), Lowe subzone (gabbroorite, norite) **Critical Zone:** Upper subzone (norite, arnothosite, pyroxenite), Lowe subzone (pyroxenite) **Lowe zone:** Upper pyroxenite aubzone (norite), Harzburgite subzone, Lowe pyroxenite subzone **Marginal Zone:** Norite
Rooiberg group	Schrikkloof formation (flow banded rhyolite), Kwaggasnek formation (massive rhyolite), Damwal formation (dacite, rhyolite), Dullstroom formation (basaltic andesite)

The Critical zone is sub divided into the Lower, Middle and Upper Groups. The Lower Group (LG) consists of seven (7) layers which are held within the pyroxenite rock (Maier, et al., 2012). Chromitite at the area of study is produced from the Lower Group (LG), where the LG6 and LG6A reefs are mined. Illustrated in Figure 2.3 is a simplified lithology at the Chrome mine where stoping height ranges between 2.1m to 2.3m. This is because the production blast includes LG6A (0.32m), the middling which is the pyroxenite (0.65m) and the LG6 (1.10m) chromitite layer. Depending on the presence of geological features, the stoping height varies from one location to another (Internal Mine Document, 2013).

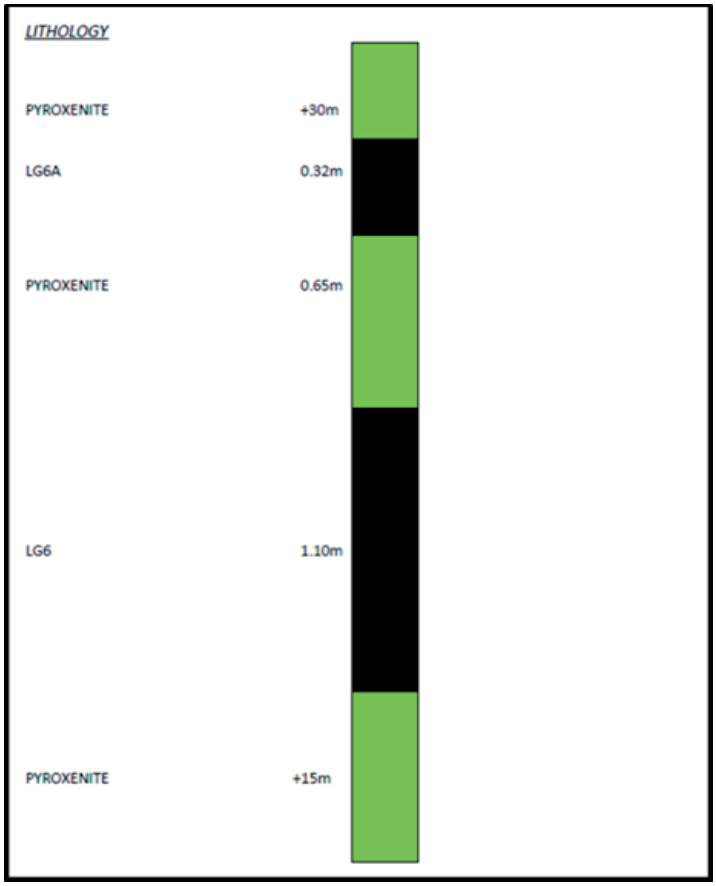


Figure 2.3: Simplified lithology at the Chrome mine *(Internal Mine Document, 2013)*

Out-crops are visible in the Dwars River location in the Upper Critical zone as depicted in Figure 2.4. Alternating layers of chromitite and anorthosite are visible in the image. This type of formation appears to be the hanging wall strata in some mines in the eastern BC. The occurrences of chromitite stringers in the immediate hanging wall strata pose a high risk of FOG. These stringers create beams of varying thickness and there is less cohesion between the pyroxenite and chromitite stringers hence the FOG hazard. Pegmatoidal lenses are also common in this area. Whenever identified above the LG6A reef band, it is advisable to remove it with production blast to combat the FOG risk thereof.



Figure 2.4: Alternating layers of chromitites and anorthosites at Dwars River Heritage Site

The eastern Bushveld Complex is characterised by geological complexities such as faults, potholes and intrusions creating dykes and sills (Uken , 1998; Robertson, 1977; and Roberts & Clark-Moster, 2010). In addition, geological discontinuities (fractures, joints, and parting planes) related to rock mechanics are also present in the area (Nong, 2010). As a result, the continuity and consistency in width of chrome bearing layers is disturbed by the occurrence of geological structures. The behaviour of rock mass around mine excavations is highly dependent on the presence of geological structures, pre-existing stresses and mining induced stresses (Quaye & Guler, 1998; Adoko et al., 2016). Therefore, the knowledge of geological structures plays a vital role in the overall design of a mine.

At the Chrome mine, a summary of geological structures encountered is compiled and results presented as shown in Figures 2.5 and 2.6, on a monthly basis. It can be seen from the pie charts that the most prominent geological structures intersected are joints (normal joints and flat dipping joints). Although in some parts of the mine there are no major geological features intersected, faults, domes and continuous horizontal planes (detected by GPR scans and Borehole camera) are intersected in some areas. The above-described approach used at the Chrome mine gives insight of potential causes of falls of ground and can be used to help

determine the possible modes of failure. It is therefore important to have geological background knowledge in order to understand the behaviour of a rock mass in a particular location. Rock mass behaviour in shallow mining environments is dealt with in the next section.

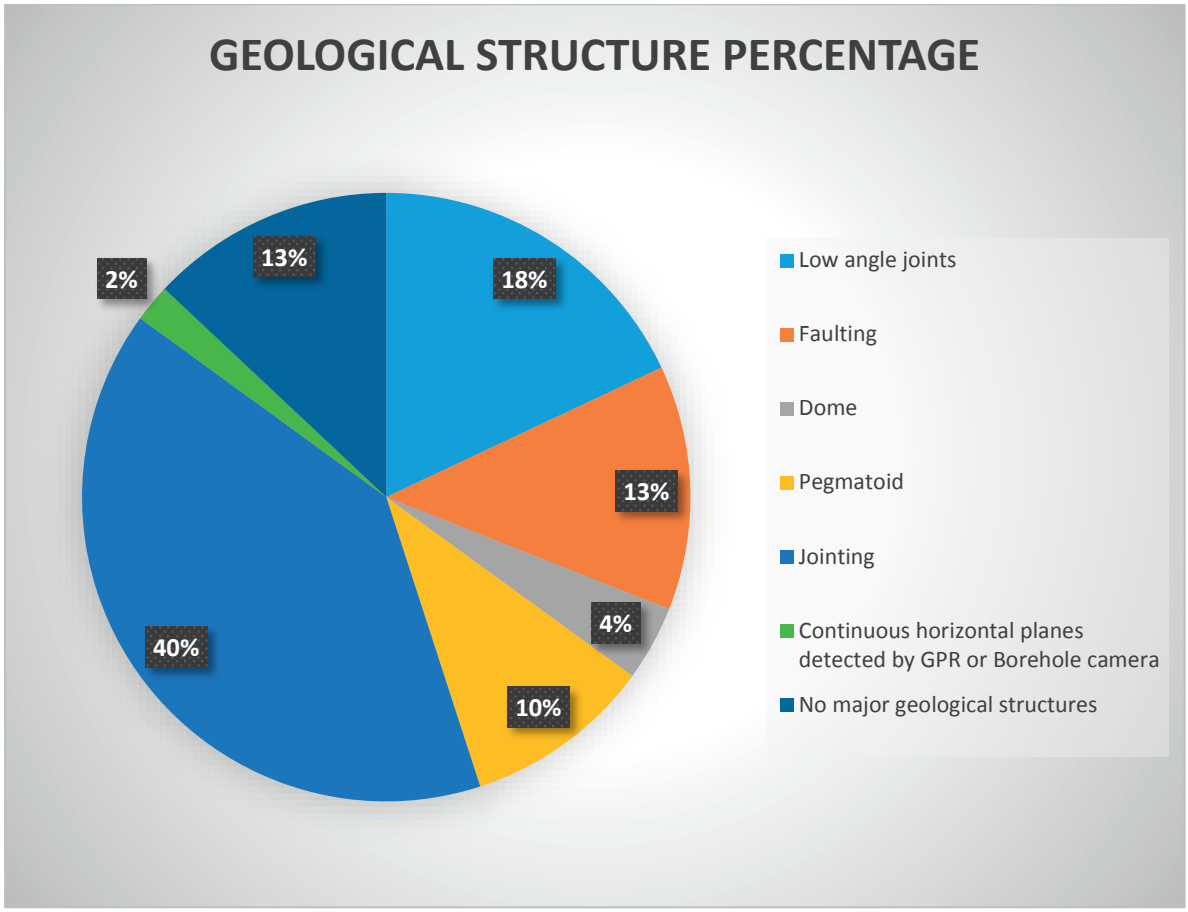


Figure 2.5: Percentages of geological structure occurrences in February 2018

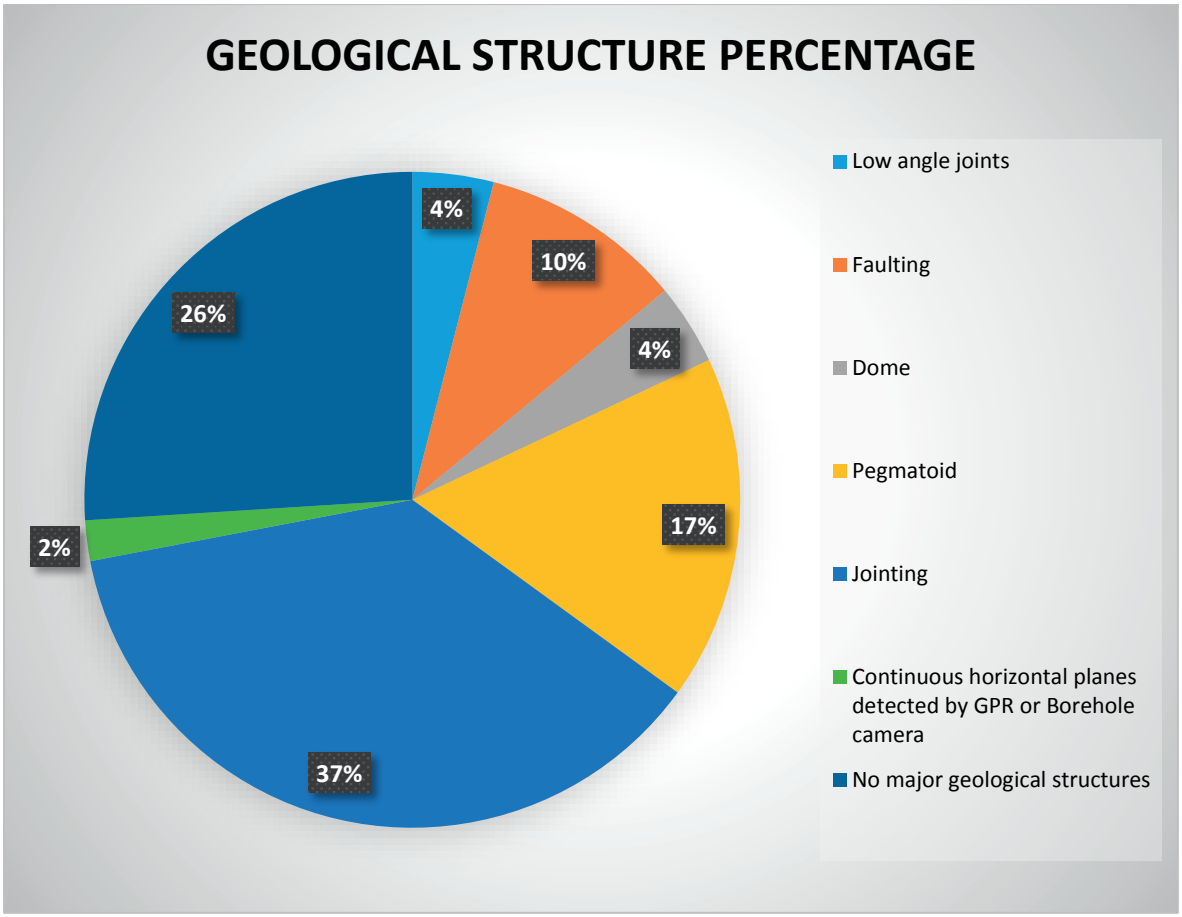


Figure 2.6: Percentages of geological structure occurrences in April 2018

2.4. Rock mass behaviour in shallow underground hard rock mining environment

Rock mass behaviour varies in different environments because there are factors which govern the overall behaviour of a rock mass. Jager & Ryder (1999) described these factors which include:

- Geological weaknesses: Rock masses are nonhomogeneous and therefore comprise of geological weaknesses such as parting planes, joint sets, dykes, and faults (minor and major). These geological structures permits further fracturing and weakening of the rock mass (ISRM, 1978). A high concentration of geological structures in a rock mass may lead to rock fall problems in mining. In addition, major faults tend to trigger seismic events (Jager & Ryder, 1999).

- Rock stresses: Stress levels tend to increase with increasing depth below surface. Prior to any mining, pre-existing stress known as 'primitive' or virgin stress, exist.

The virgin vertical stress component ($q_{v)}$ is defined by the overburden weight ($q_v = \rho gh$). The virgin horizontal stress component (q_h) is defined by the K-ratio ($K=q_h/q_v$). According to Jager & Ryder (1999) the K-ratio decreases with depth, i.e. at shallow depths K>1 and at greater depths K,0.5. Apart from virgin stresses, rock mass behaviour is also influenced by mining induced stresses. Virgin and mining induced stresses combine to give 'resultant' or 'absolute' stress which is redistributed around an excavation. Redistributuion of stresses gives a state of stress known as field stress around the rock mass surrounding the excavation (Jager & Ryder, 1999).

- Rock strength and friction properties: The strength and friction properties of a rock form part of the engineering and geotechnical data required in the design of excavations. Various rock types have different strength and friction properties. These are obtained from conducting laboratory tests such as uniaxial compressive stress test, and triaxial tests on cylindrical samples of rocks. The stress strain behaviour of a rock is governed by two factors i.e. Elastic modulus (E) and Poison's ratio (υ). When a rock is subjected to load until failure, its behaviour and manner of failure relates to the above-mentioned properties (Ryder & Jager, 2002).

- Rock types: Rock types vary based on the way they were formed. Three major rock types are Igneous, Sedimentary and Metamorphic rocks. Igneous rocks form by cooling down of magma. Sedimentary rocks are formed through sediment solidification whereas metamorphic rocks form through metamorphism of either igneous or sedimentary rocks. According to Jager & Ryder (1999), host rocks in South African mines are usually competent.

Mining is classified as shallow where the depth below surface does not exceed 1000m (Ryder & Jager, 2002). Local geological structures and dominant rock mass conditions associated with the area affect the mining strategies employed at any depth. As outlined in section 2.3, the area of study is characterised by joints, faults domes and horizontal planes. Ground conditions in the periphery of these geological features are usually blocky. Therefore the quality of rock mass in the proximity of major geological structures is considered a potential fall of ground hazard, and requires to be supported adequately and monitored. An example of

fall of ground due to intersection of multiple prominent joints is shown in Figure 2.7. The joints were observed to have soapy infilling material known as seperntinite.



Figure 2.7: Fall of ground as a result of multiple parting planes with sepentinite infilling *(Internal Mine Document, 2011)*

The intersection of near vertical joints and low angled joints may result in the formation of wedges and unstable blocks, which have potential to fail if not adequately supported. Low angle joints and domes also pose FOG hazards. Examples of these types of failure are shown in Figure 2.8. According to (Qamata & Steenkamp, nd) the identification of geological structures underground is a challenge to most production personnel. The use of technology to assist in the identification of geological structures in the immediate hanging wall is crucial for decision making in addressing potential fall of ground areas (Qamata & Steenkamp, nd).



Figure 2.8: Wedge failure (Qamata & Steenkamp, nd)

Potential rock failure at shallow depths can be defined by relatively widely-spaced opened up geological discontinuities or tensile fractures, which take form of a beam in the hanging wall strata, causing beam failure. Failure can also be in the form of key blocks from interacting geological structures that dropout, which is sometimes followed by large scale hanging wall unravelling or stope back-breaks on a large scale (Jager & Ryder, 1999). The mode of failure is sometimes rapid and violent or slow. The thickness of failure also varies depending on factors triggering that particular failure. Rock falls might be in a small scale, but yet still regarded hazardous.

Jager & Ryder (1999) state that problems experienced in shallow mines are particularly their own. Nevertheless, some of these can be experienced in intermediate or deep mining environments with multi-reef stress-relieve conditions for similar reasons. Stress levels are related to the depth below surface because as depths increases, stress levels are also expected to increase. Thus shallow mining environments are associated with low stresses and minimal fracturing. The presence of fracturing in deep level mining generates horizontal diagonal stresses which tend to lock the hanging wall into a self-supporting beam.

According to (Ozbay et al., 1995), shallow hard rock mining environments are usually associated with joints and bedding planes that weaken the hanging wall strata. The hanging wall rock mass at this depth is characterised by well-defined discontinuities subjected to deadweight tension (Ozbay et al., 1995). This is one of the problems associated with hard rock shallow mining environments, where an extended tensile zone in the hanging wall promotes bed separation, creating a potential for massive collapse. The collapse can extend to

surface if the mining depth is very shallow (closer to surface). If the critical span of the excavation is exceeded, the hanging wall strata become susceptible to back-breaks. With this problem addressed, shallow mines require robust and stiff support systems which react rapidly at little convergence (Ozbay et al., 1995). It is for this reason that shallow mining is prone to potential falls of ground as compared with other mining environments.

In shallow hard rock mining environments (depths less than 1000m), there are relatively high horizontal virgin stresses than vertical stresses, however Nyungu & Stacey (2014) state that failure can also occur at low stress conditions in some mines in the Bushveld complex. With time, stress induced spalling on underground excavation walls become visible. An example of this rock behaviour is illustrated in Figure 2.9. According to the time-dependent study conducted on BC rocks by Nyungu & Stacey (2014), the results show that the magnitude of tensile strength is about 5% of their UCS magnitude.



Figure 2.9: Spalling in a haulage tunnel in a platinum mine located in the western Limb of the BC *(Nyungu & Stacey, 2014)*

Shallow mining is classified to be less than 1000m below surface. At intermediate depth (1000m to 2250m) rock blocks are well clamped, whereas at greater depths (>2250m), very high stress levels are experienced which induce failure of intact rock material or failure along joints and rockbursts (Ryder & Jager, 2002). When a mining induced seismic event radiates intense shock wave to cause visible damage to an excavation, it is referred to as rockburst (Heunis, 1980). Generally, it is expected that the number and magnitude of seismic events increase with increasing depth below surface (Ledwaba et al., 2012). According to Cai (2016), rockburts occur under two conditions. Firstly the rock mass must be able to store large amounts of energy and poses a "strong bumping-prone characteristic when damaged" and secondly, mining area geological conditions must be such that they favour the formation of high stress concentration to accumulate huge energy (Cai, 2016). This suggests that rockbursts are not only prone to deep level mining but can be experienced in shallow mining environments that poses the above-mentioned conditions. Depth less than 1000m is classified as low seismic zone in the eastern BC because only natural tectonic seismicity is known to occur in the area. This is because most prominent tectonic features, which are; the Steelpoort fault, Dwars river fault and the Klipriver fault in the eastern BC.

Problems associated with rock mass behaviour at shallow mining environments prove that excavations at shallow depths in general, require more robust support contrary to deep level mining environments. Jager & Ryder (1999) recommend shallow mining strategies aimed at controlling the extended tensile zones in the hanging wall on a regional basis in order to prevent local rock falls within working areas. The knowledge of K-ratios can assist in mine layout and support designs, particularly in areas with relatively high horizontal virgin stresses. Likewise, the knowledge of the presence of geological structures is critical when designing layout and support systems (which is presented in the following section).

2.5. Mining layout and support design

Appropriate mining layout and support design is influenced by the knowledge and understanding of the rock mass behaviour in a particular environment as indicated in the previous section. This section focuses on the mining layout and support designs employed in similar mining environments like at the Chrome mine. There are numerous attributes considered when selecting the type of mining method, these include:

- The depth, size, shape and orientation of the ore body;

- Physical characteristics of both the orebody and the host rock;
- Grade distribution and consistency of the orebody width;
- Presence of geological structures and hydrology.

In addition, the analysis of pre-existing mining stress field and stress distribution because of mining activities, rock quality, strength and the nature of geological discontinuities is deemed essential for support design of mining excavations. The presence of geological structures within the identified economic mineral resource of interest turns to disrupt and influence the type of mining method employed to extract that particular ore body. Furthermore, the size, shape and orientation of the excavation will depend on the geological structures present within the orebody.

Most shallow underground mines in the eastern Bushveld Complex use conventional breast mining or mechanised bord and pillar mining method depending on the above-mentioned factors. Mines with fairly dipping ore body geometries generally prefer using mechanised bord and pillar mining method. The Chrome mine uses trackless mechanised mining method (TMMM) in the form of bord and pillar. A simulation configuration of the mine is shown in Figure 2.10. The ore body dips at an average dip angle of 10^0 westwards striking North-South direction. There is consistency in the width of the LG6 and LG6A chrome layers hence bord and pillar was a suitable mining method for this ore deposit.

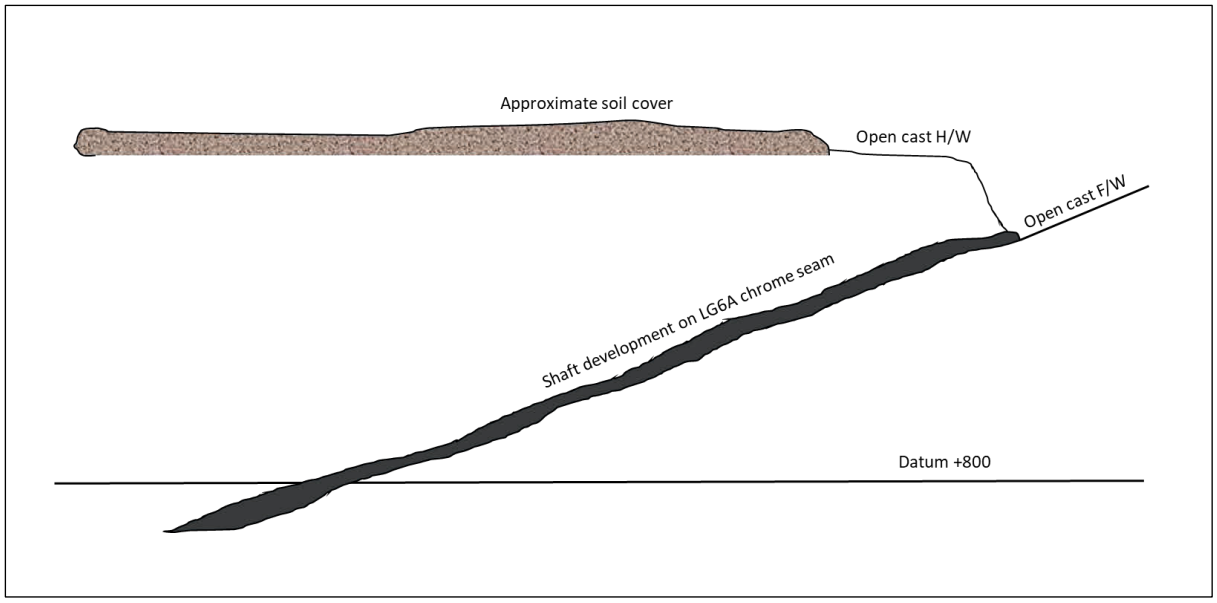


Figure 2.10: Flat dipping ore body at shallow depth *(Internal Mine Document, 2013)*

Swart et al. (2005) indicated that when designing stope panels, very few mines use a systematic design procedure where rock mass characterization, estimation of rock mass properties, appropriate stability analysis and potential rock failure modes are considered. But usually, rock engineering design processes are neglected and excavation spans are decided either by equipment size and/or previous experiences in similar conditions. The consequence of such designs may be uncontrolled ground movements and massive collapses for shallow mining environments.

In bord and pillar mining, part of ground rock is left unmined in the form of pillars for support purposes. Usually pillars in shallow and intermediate depths are used for roof support. At these depths, large tensile zones may exist in the immediate hanging wall as outlined in the previous section (2.4). Therefore, robust stiff support in the form of pillars is used for regional support to minimise rock fall related hazards. The process of mine design is usually based on empirical methods and practical experience, which does not qualify as engineering design based on a discussion by Bieniawski (1998). The design of stable pillars in underground hard rock mining, particularly for bord and pillar layouts as stated by Swart et al. (2005), is based on the factor of safety (FOS) design criterion. Factor of safety relates to the ratio of pillar strength to pillar stress expressed as follows:

$$FOS = \frac{Pillar\ strength}{Pillar\ stress}$$

$Pillar\ strength = K\frac{W_{eff}^{\propto}}{H^{\beta}}$ And $Pillar\ stress = \frac{\rho g h}{1-e}$

Where:

K = strength of the pillar rock mass material;

$W_{eff} = 4\frac{A_p}{R}$ = effective pillar width;

A_p = the plan area of the pillar;

R = the pillar perimeter;

H = pillar height;

ρ = density of the rock mass;

g = gravitational acceleration;

h = depth;

e = extraction ratio;

α and β are numerical constants.

The pillar strength formula expressed above is the only empirical formula that was tested against sufficient field data (Swart et al., 2005). A back-analysis by Salamon and Munro (Salamon, 1967) was carried out in coal mining for square pillars and the constant values for K, α and β were 7.2 MPa, 0.46 and 0.66, respectively. However, for hard rock mining, only a few back-analyses have been conducted (Swart et al., 2005). Results from back-analysis by (Hadley & Grant, 1972) showed constant values of K, α and β to be equivalent to 133 MPa, 0.50 and 0.75 respectively for competent rib pillars. Therefore, the pillar strength formula for hard rock mining can be expressed as follows:

$$Pillar\ strength = K\frac{W_{eff}^{0.5}}{H^{0.75}} \quad (MPa)$$

Pillar sizes increase with increasing depths below surface, and this equation applies for width to height ratios less than 4.5. Where the width to height ratio exceeds 4.5, the pillar strength is calculated as follows:

$$Pillar\ strength = K\frac{2.5}{V^{0.07}}\left\{0.13\left[\left(\frac{R}{4.5}\right)^{4.5} - 1\right] + 1\right\} \quad (MPa)$$

Where:

$R = \frac{W_{eff}}{H}$ And $V = W_{eff}^2.H$

It is clear from the relationships given in the formulas that pillar strength will depend on the rock mass strength of the pillar rock type. The size and shape of the pillar relates to the height and width of the pillar. In addition, the overall pillar strength will also be determined by the amount of geological structures orientated unfavourably in the area. Large pillars lead to low extraction ratio and small pillars may lead to overall mine instability. Therefore pillar strength need to be determined with relevance to the load acting on the pillar (York et al., 1998). Pillars are said to be the main support component in shallow mines. This is because of large tensile stresses present in the hanging wall and geological weaknesses present in the hanging wall rock mass (Ozbay et al., 1995).

A fully mechanised platinum bord and pillar mine in the eastern Bushveld complex was designed based on (Hadley & Grant, 1972) formula, with pillar sizes increasing with depth (Esterhuizen, 2014). The same approach was used to design pillars at the Chrome mine, where regular bord and pillar pattern is employed (Figure 2.11). Bord width ranges between 6m and 12m subject to depth below surface and presence of geological disturbances. Pillar sizes increase as depth of mining increases.

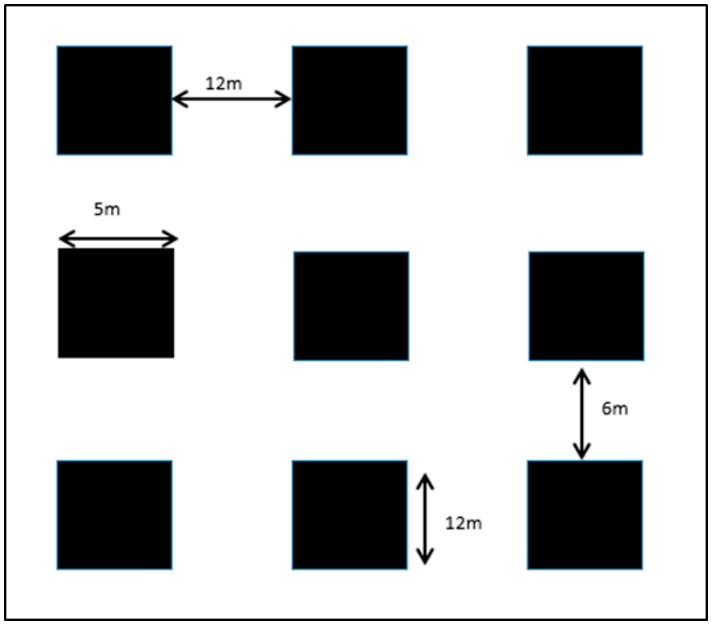


Figure 2.11: Schematic diagram of a regular pillar layout

The larger the excavation, the more the geological structure are exposed, which in turn might influence the stability of mine excavations and the support system required thereof. As a way of improving excavation stability, it is always advisable to mine moving away from geological structures (Jager & Ryder, 1999). At times, poor ground conditions may necessitate a different mining layout as shown in Figure 2.12. This example is from a Zimbabwean platinum room and pillar mine where there is intense faulting and jointing in the rock mass resulting in poor ground conditions. To enhance stability a different layout was designed for that specific ground condition. In addition, a new support system was also designed for this area to help improve safety and production, as the previous support system was not adequate for such poor ground conditions. The new system includes the use of longer roof bolts (2.1m) spaced at 1.2m x 1.2m whereas the previous support system had 1.8m long roof bolts spaced at 1m x1m (Chikande & Zvarivadza, 2016).

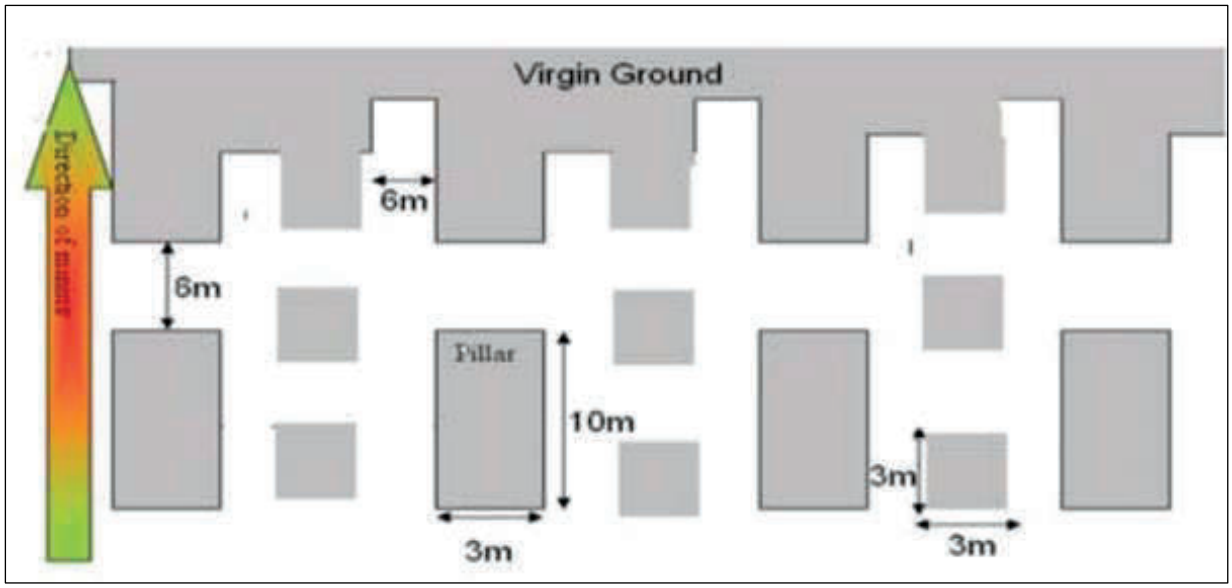


Figure 2.12: Mining layout in poor ground conditions at a Zimbabwean platinum room and pillar mine *(Chikande & Zvarivadza, 2016)*

The mining method used in the study area already provides regional support by means of pillars. Underground excavations are supported for three main reasons which are; to ensure safety of the working places, to prevent falls of ground by preventing key blocks from falling and lastly to control the movement of large blocks or fragments in the periphery of the excavation.

Owing to that, local support is deemed essential. Generally, support design for underground excavation is summarised into two aspects, both measured in (kN/m^2):

- Demand: load generated by the hanging wall (to be supported by one support unit)
- Capacity: the resistance or load generated by one support unit over a specific tributary area.

If the demand exceeds the capacity, then failure will occur. The aim is to stabilise the hanging wall by providing sufficient support resistance to avoid failure (i.e. capacity must be greater than the demand). As in stope support (Figure 2.13), the Chrome mine uses 110 kN, 1.5m long x 18mm diameter mechanical end anchored roof bolts, used with 10cm square dog-ear face plate. To ensure stability, these support elements are to be installed orientated at 70^0 to 90^0, at maximum spacing of 2m on both dip and strike direction. To ensure that the required support resistance is achieved, the roof bolts are tensioned immediately after installation. In

addition, tensioning of roof bolts is also recommended after blast. In cases where there is a need to support the hanging wall beam to a length beyond that supported by primary support, secondary support in the form of 4.5m long x18mm diameter pre-tensioned (150 kN), full column grouted (25t) cable anchors are installed. Cable anchor spacing is also 2m on both dip and strike direction.

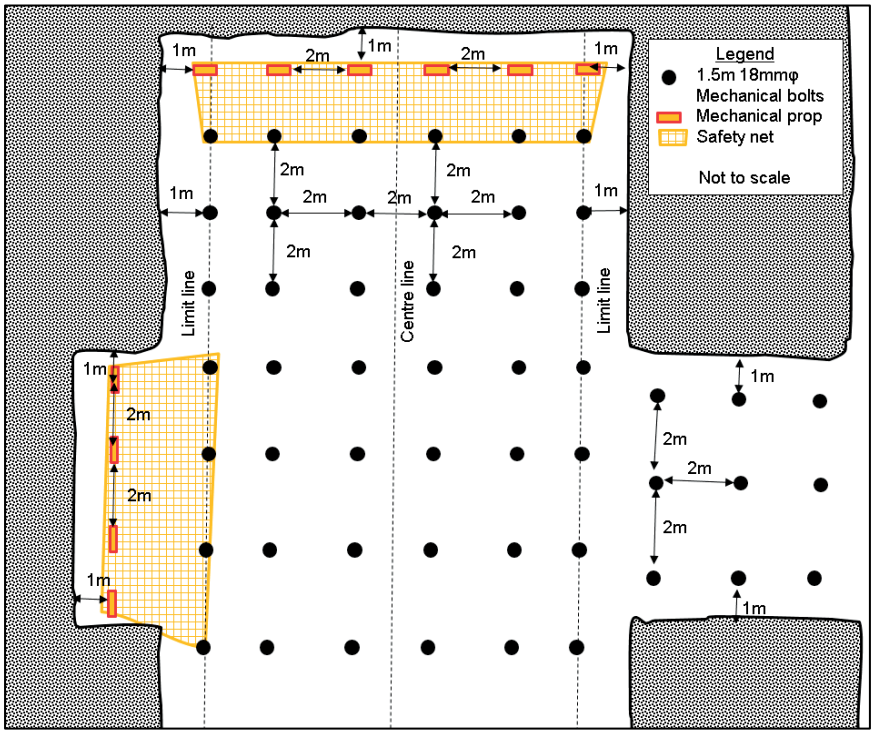


Figure 2.13: Local support layout *(Internal Mine Document (b), 2017)*

Fall of ground can happen as soon as the face has been exposed during early entry examination or after support installation, where blocks of rocks dislodge in-between support. Most rock related hazard occurs at the face area where most of the work is done. Therefore, proper early entry examination should be conducted at all times, ensuring that temporary support is installed to mine standard. Primary support should be installed as soon as possible to minimise instability of the hanging wall and sidewall after the face has been exposed. In poor ground conditions, secondary support is required to prevent hanging wall and sidewall deterioration (Esterhuizen, 2014). It is important to know the average fall out thickness because support design must aim at supporting 100% of the fall out thickness.

2.6. Summary and conclusions

This chapter has described shallow hard rock mining environment by reviewing geological background and rock mass behaviour expected in shallow mines. Furthermore, stability analysis was conducted by means of reviewing mining layout and support designs employed in shallow hard rock mining environments similar to the area of study. Reviews conducted in this chapter show that the behaviour of rock mass differs from location to location. Instability in shallow mining is usually due to the presence and orientation of geological structures. The ultimate failure mode in such cases can be beam, wedge, and block failure. Rock mass classification cannot be used to analyse the above-mentioned failure modes. However, rock mass classification is still a fundamental aspect in the process of designing stable excavations.

Although some areas may show similarities, it is important to understand the geological background for the specific area for design purposes. In addition, empirical analyses are recommended because the design of rock structures generated by mining activities is a concern in rock engineering. It can therefore be concluded that rock engineering designs have a major implications when it comes to safe recovery of mineral resources. To minimise the potential of instability, emphasis should be put in identifying the location of possible failure planes and the most likely mode of failure and appropriate techniques of analysis should be carried out. This means that there should be measures in place to manage stability and prevent unwanted ground movement. The next chapter details fall of ground management strategies used at the Chrome mine.

CHAPTER 3: FALLS OF GROUND MANAGEMENT STRATEGY AT THE AREA OF STUDY

3.1. Introduction

In mining, rock fall related hazards are forever present. The fact that fall of ground management strategies have been put in place means that fall of ground is a major concern. Fall of ground management is an important aspect in mining; hence, it is critical for a mine to implement a strategy that will help combat rock fall related hazards. This chapter outlines the fall of ground management strategies used at the Chrome mine. As explained in the previous chapter, rock mass behaves differently in different locations. Since the area of study is characterised mainly by joints, faults, horizontal planes and flat dipping structures such as dome, the fall of ground management strategy used at the mine was influenced by the presence of these structures.

Practising safe mining and installation of suitable support improves excavation stability (Chikande & Zvarivadza, 2016). However, excavation stability is not only limited to safe mining practises and support installation. Additional emphasis should be put on observations and monitoring of the proposed mining and support layouts. An adequate engineering design includes monitoring, which is an extremely important aspect of the design process. Monitoring will help to observe if the designed structure is behaving expectedly or not. Moreover, the role of monitoring is to check if the design criteria used are appropriate. It can be carried out by means of direct visuals or sophisticated monitoring instrumentation (Stacey, 2004). Continuous ground monitoring is essential in the mining industry, because as the excavations advance, stresses are redistributed around mine excavations, and therefore mining personnel are exposed to hazardous environments. Various mechanisms can be used for continuous monitoring depending on factors such as cost, availability and effectiveness of the monitoring system. As mentioned above, FOG management strategies include the use of ground monitoring equipment. It is therefore considered appropriate to understand what ground monitoring is all about and its relevance in the context of combating falls of ground.

3.1.1. Objectives of ground monitoring

In rock engineering, rock monitoring is conducted for four main reasons which are to (OHMS, nd):

- Record natural values and variations in geotechnical parameters prior the start of a project;
- Ensure safety as mining progresses by giving warning of excessive rock deformation;
- Validate assumptions used during the design phase of a project; and
- Confirm the support and layout performance in a specific environment.

The role of monitoring in design is to be able to check whether the designed structure behaviour mechanism is as expected and whether the appropriate design criteria was used (Stacey, 2004). Therefore, monitoring plays a very important part in design.

Ground monitoring is not only restricted to underground mining operations, but is also applicable in surface mining. However, for this study the focus is on underground mining. In this particular mine, ground is monitored in order to locate the potential areas susceptible to fall of ground. Ground monitoring also helps control and give warnings to personnel working underground, thus giving them time to evacuate and barricade unsafe working areas, or make the area safe by trying means to stabilise the ground, if possible.

3.1.2. Categories of instrumentation

Ground monitoring instruments vary and mines use different instrumentation depending on their main objective and costs of the equipment. There are optical, hydraulic, mechanical and electrical instruments that can be used to monitor ground in mining. The choice of instrument will depend also on the parameter to be monitored. Parameters that can be directly measured underground using different monitoring instrumentations may include:

- Rock fracture
- Movement across weakness planes (joint, fracture, or fault)
- Convergence or relative displacements of boundary points in an excavation
- Displacement of rock from the excavation periphery
- Stress or load on support units

Each parameter has its own distinct monitoring instrument. In this study, the objective is to monitor rock fracture and the displacement or movement of rocks from the periphery of the excavations as it relates to rock fall hazards.

3.1.3. Ground monitoring systems and accuracy

Different instruments measure different parameters. It is therefore important to identify and understand the main objectives so as to know which monitoring instrument to choose in order to solve the problem. Most of these monitoring instruments are man-operated thus; accuracy, error and sensitivity are aspects that need to be accounted for, in the obtained results.

The area of study has adopted a ground monitoring system consisting of equipment namely; Ground penetrating radar (GPR), Borehole camera and fall of ground (FOG) light. Each of these is dealt with in sections 3.2, 3.3 and 3.4 respectively. Apart from monitoring equipment, visual observations are conducted on a monthly basis and panels are rated according to their level of rock fall related risks. The use of both visualisation and ground monitoring equipment broadens up the understanding and interpretation of results obtained from ground monitoring equipment.

3.2. Ground Penetrating Radar (GPR) Scanning

The use of radio waves to image sub-surfaces in high resolution was conceptualised in the 1920s. By then, radio echo sounders were used in Polar Regions to profile ice thickness (Singh, 2007). With time, instrumentation has evolved. Ground penetrating radar (GPR) is a geophysical instrument used in both surface and underground operations to detect magnetic horizontal and sub-horizontal geological structures. GPR scanning is a geophysical method that operates with radar pulse for subsurface imaging. The method is non-destructive as it uses electromagnetic radiation to detect and reflect signal from the subsurface being scanned. GPR is capable of detecting high-resolution records of continuous subsurface structures in a short period, up to a depth of 20m (Buynevich, nd). The detected and reflected structures can be in the form of voids, change in material or fractures and cracks. The reflections appear at boundaries of different electric constants. Principles that apply in reflection seismology are similar to those of GPR scanning, where high frequency radio waves are transmitted through the ground. According to Singh (2007) the only difference is the source of energy, which is electromagnetic in this case, and acoustic energy in reflection seismology.

Impulse GPR systems nowadays use short burst of electromagnetic energy. From a transmitter, the energy is transmitted with frequencies ranging between 10MHz to over 1GHz. These values however depend upon the transmitting antenna (Singh, 2007). Ground penetration in GPR scanning is limited based on the electrical conductivity of the ground, power emitted by the radar, and the transmitted centre frequency. Electromagnetic energy is more dissipated into heat; as a result, signal strength is reduced. Therefore, conductivity decreases with increasing depths. Better resolutions are obtained when using higher frequencies. Nevertheless, as far as depth is concerned, higher frequencies do not penetrate as far as lower frequencies. According to Apel & Dezelic (2005), there is a difference when scanning a wet rock mass versus a dry rock mass. Dry rock masses allow deeper penetration but the shortcoming is when interpreting results, it is easy to omit smaller voids or clay layers (Apel & Dezelic, 2005). When determining hanging wall stability, GPR scanning is considered the best tool for mapping features in the hanging wall strata (Apel & Dezelic, 2005). This technique is also considered to be a quicker way of determining geological structures ahead of mining (Vogt et al., 2005).

Treloar (2003) indicated that the use of GPR technology in mining was initially introduced in deep gold mining for fracture mapping. A trial of this technology was conducted in a shallow mining environment characterised by dome structures and the results were a success. Further trial by Intergraded Seismic Systems International, Geophysics (Pty) Ltd. (ISSI, 2001) showed that the orientation and extent into the hanging wall of detected dome structures can be determined (Treloar, 2003). The use of GPR technology has now been adopted in most shallow mining environments in the South African mining industry.

The Chrome mine uses the SIR 3000 system comprising of 40QMHRZ antenna. After data is collected underground, desktop processing is done using RADAN software. For the mine, the hanging wall target range is between 1m and 7m into the immediate hanging wall. This is because above 8m into the hanging wall, the ground is considered ‘self-supported’ in this environment (Internal Mine Document (b), 2016). A GPR with higher frequency would have been considered to obtain better details, but the shortfall is, lower hanging wall targets would have been reached as stated previously.

Applications of Ground Penetrating Radar

The application of Ground Penetrating Radar (GPR) technology has become useful in the mining industry. A stated, previously, the method is not limited to underground operations, but can be used on surface as well. In this case, the focus is on underground operations as the area of study is an underground mine. Borman (2006) stated that GPR scanning is very useful when it comes to estimating the positions of geological discontinuities such as potholes and faults. It is however still considered an estimate not an absolute measurement, thus good to be used for solid rock scanning rather than open spaces.

Information about the immediate hanging wall strata is obtained by scanning of the hanging wall using the GPR. From the scan, geological discontinuities within the immediate hanging wall can be identified. The process of application involves three stages, which are:

- GPR scanning conducted underground - Data is displayed and stored in the GPR computer before it is downloaded for processing
- Data processing – Process of removing unwanted 'noise' from the data collected during the scanning process. Highlighting useful information
- Data interpretation – Processed data is interpreted, analysed and distributed in the form of a report.

The distance to be scanned is measured at 10m from the face backwards, and the nearest peg should reference it. Three scans per panel are collected i.e. up-dip centre and down-dip to acquire more data on that particular panel (Figure 3.1). It is critical to follow recommended procedures for operating the equipment as stipulated in the mine's standard operating procedure (Internal Mine Document (b), 2016). Failure to do so may result in inadequate data collection, which will affect the interpretation thereof. Safety of personnel operating the machine is important. Correct personal protective equipment should be worn at all times. It is advisable for the person scanning not to look into the hanging wall so as to prevent disruption of a persistent and continuous scan.

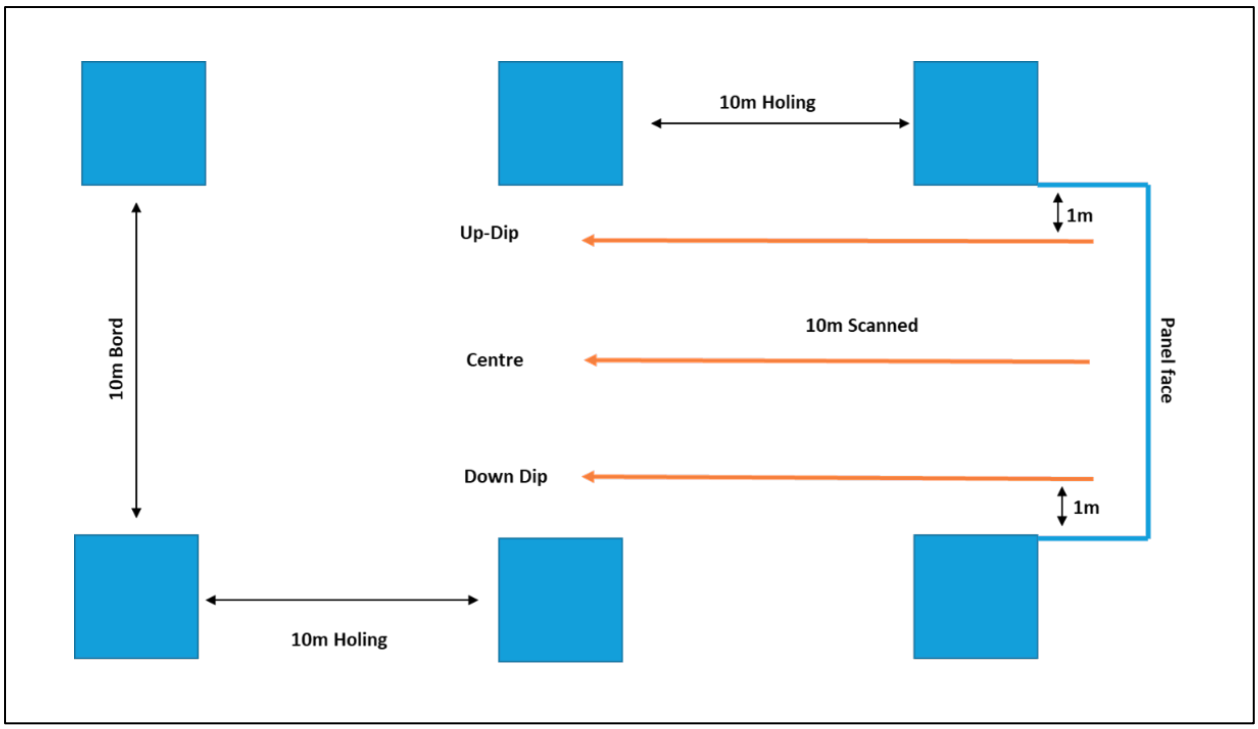


Figure 3.1: Schematic diagram of a panel to be scanned using GPR

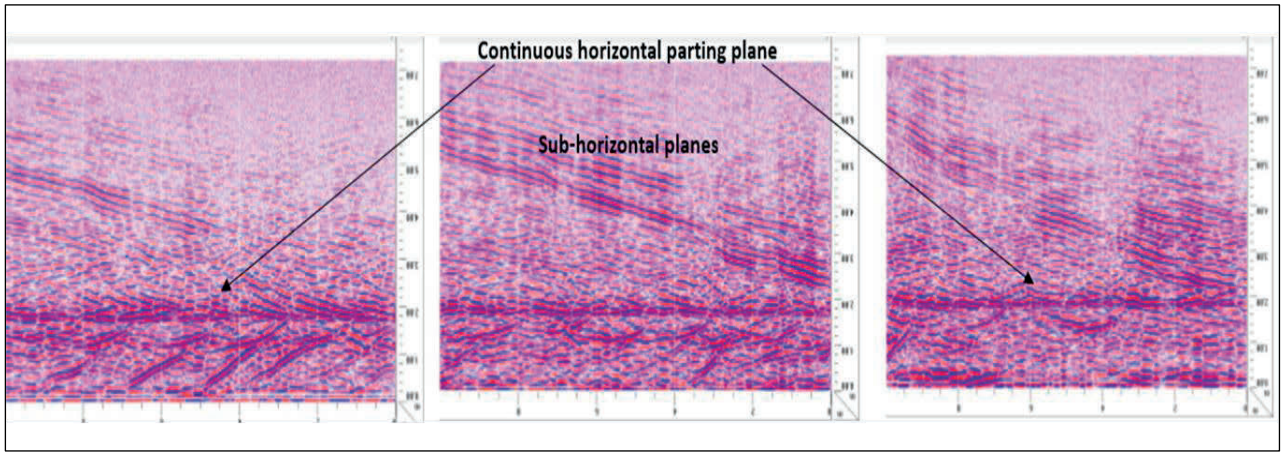


Figure 3.2: Processed GPR scan image

An example of a processed GPR scan is presented in Figure 3.2. The scan estimated continuous horizontal parting planes from the face, 10m back at about 2m into the immediate hanging wall in all the threes scans (up-dip, centre and down-dip). Such structures cannot be identified through visual observations. In this case, primary support, which is 1.5m long, is

insufficient to support the structures detected. To stabilise the ground, 4m long cable anchors are recommended. More GPR scans are presented in APPENDIX E.1 and APPENDIX E.2.

Pros and cons of Ground Penetrating Rader

Conducting ground penetrating radar (GPR) surveys timeously is critical as it assists in the identification and subsequent treatment of any hazardous ground conditions that may result from the surveyed ground. Therefore, it promotes safer working environment for all underground personnel. Results during scanning process can be interpreted and immediate decision regarding workplace stability can be taken when structures identified require immediate attention. Generally, GPR gives real-time data acquisition since the SVGA monitor displays the hanging wall as scanning progresses. Its ability to detect flat dipping joints and domes allows for pro-active measures to be taken in terms of improving stability. For example, leaving in-situ pillars and revising mining strategies where major flat dipping structures are identified. In addition, the technique can determine the amount of reef displacement into the hanging wall in cases where there are major up throw faults.



Figure 3.3: GPR scanning *(Internal Mine Document (b), 2016)*

The GPR machine is powered by a rechargeable battery. It is therefore advisable to take at least two batteries underground to minimise uncertainties. The process of scanning requires at least two individuals, one holding the antenna against the hanging wall and the other controlling the SVGA monitor GPR (Figure 3.3). It is therefore inconvenient when one of the operators is absent. Data collected is stored in a computer system, and it is therefore critical to save data and ensure the system is virus free to avoid losing data. Many are times where panels are either full of water or broken ore at the time of visit. As a result, one or none scans are taken in such panels. The panels can only be scanned once the water has been drained or broken ore has been cleaned out. This is challenging as little or no data is collected. Operating the GPR machine requires training and the process of scanning is labour intensive as the machine is heavy to carry around.

GPR scanning only detects horizontal and sub-horizontal structures when scanning on the hanging wall. As mentioned before, results acquired from this technique are only estimates. To confirm these estimated structures within the hanging wall, the Chrome mine uses borehole camera inspections. This technique is detailed in the next section.

3.3. Borehole Camera Inspection

The use of Borehole camera in mining related investigation is broad. It is regarded as one of the most valuable tools in the mining industry as it allows visualisation of structures within the hanging wall. Visualisation in the form of videos and images of drilled boreholes, shafts, and open sinkholes and any vertically accessible subsurface excavations can be obtained when using borehole camera (Anon, 2013). Various models of borehole cameras are available in the market worldwide (Figure 3.4). Borehole cameras consist of a screen, where visualisation takes place and offsets that are connected to the camera. On the end of the first offset is a camera, which comes with its own light source.

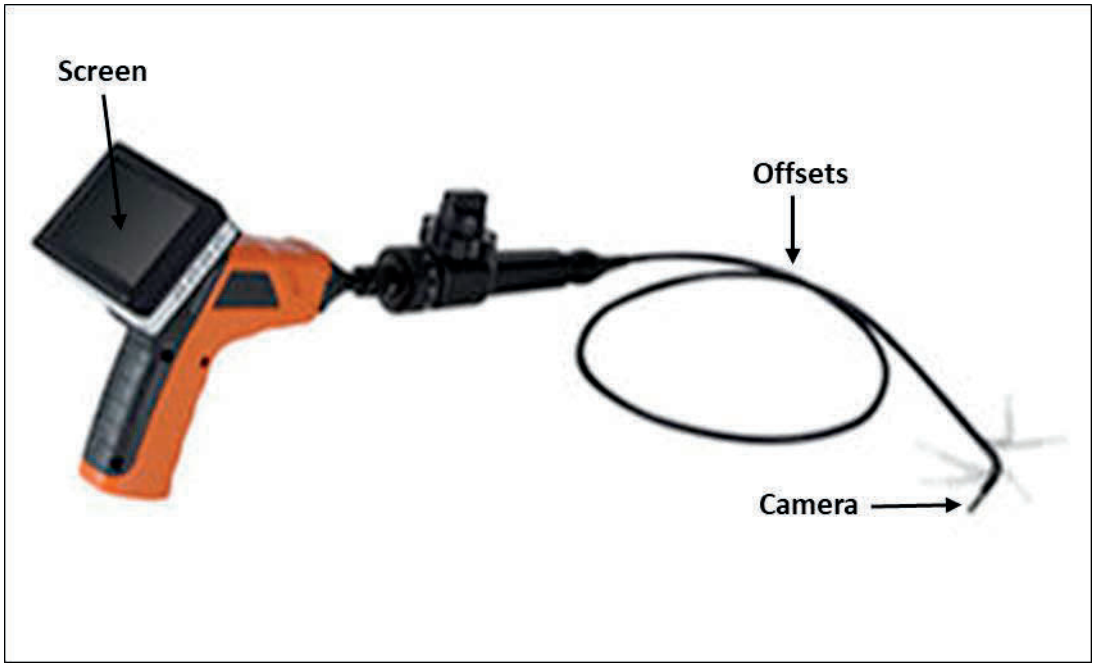


Figure 3.4: Borehole camera *(Anon, nd)*

Since the camera does not give the depth, a tape is usually attached to the offsets in order to know the location within the hanging wall, where geological structures are observed. An initiative set out by Two Rivers Platinum to actively monitor and control ground conditions on a daily basis involved the use of Borehole cameras. Since 2005, the results have shown that the rate and frequency of FOG occurrences has drastically dropped (Esterhuizen, 2014).

Applications of Borehole Camera

In mining, borehole cameras can be used for different reasons. These may include but are not limited to stratigraphic logging, fracture logging, ground water information, and many more (Hawkins & Evans, 2004). In this particular mine, the equipment is used for fracture logging where location and condition of geological structures is investigated. Information concerning the frequency of occurrence of structures within the immediate hanging wall, and the degree of infilling of these structures is recovered.

Borehole camera inspection is carried out in panels where geological structures are suspected or were detected by GPR (Internal Mine Document (a), 2015). When borehole camera inspection is recommended to confirm the structures, a drilling note is issued for either 2m or 4m long holes to be drilled in the hanging wall for the inspection to be conducted. The depth

of borehole is determined by the location at which the structures were detected in the hanging wall. During the process of inspecting, the recording unit is inserted with the borehole camera for record keeping. The information is saved in the form of a video and images. However, at the Chrome mine, when this technique is practised, only images are saved.

The position and length of drilled boreholes is dependent on results from the GPR scans. Usually three boreholes are requested for borehole camera inspection, if continuous horizontal structures are detected by the scans (up-dip, centre, and down-dip). Results are interpreted and recommendations are given based on the severity of the structures. Recommendations may be installation of secondary support and/or installation of FOG light for continuous monitoring of the hanging wall strata.

Interpretation of Borehole Camera scans

Borehole camera inspections are important as they provide quick visuals of what is happening within the hanging wall strata. Examples are shown in Figures 3.5, 3.6, and 3.7 Continuous horizontal planes appear to be in a form of a full circle whereas near horizontal planes are in a 'U' shape on only one side of the borehole (Figure 3.7). A continuous horizontal plane of weakness was identified with infilling material approximately 5mm thick (Figure 3.5). These structures were identified at about 2.0 m into the immediate hanging wall, as detected by the GPR scans. In Figure 3.6 two continuous sub-horizontal weakness planes are shown with visible infilling material. The infilling material at this area is usually seperntinite, it is smooth and soapy. Without the use of borehole camera, the presence of infilling material would remain unknown. Thus, the Borehole camera inspection is critical for this ground monitoring system.

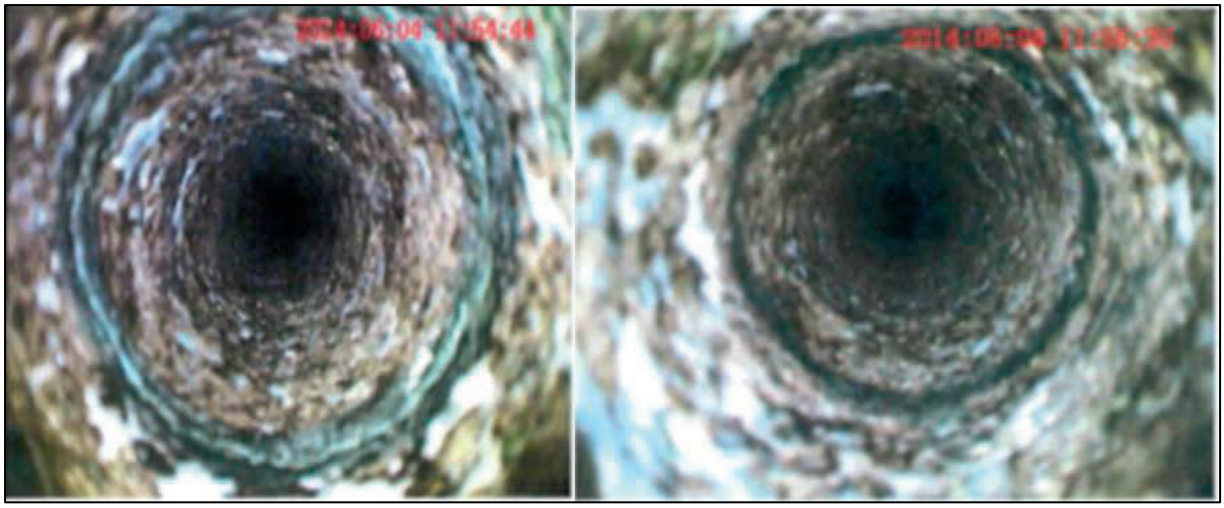


Figure 3.5: Continuous horizontal parting planes *(Internal Mine Document (b), 2014)*

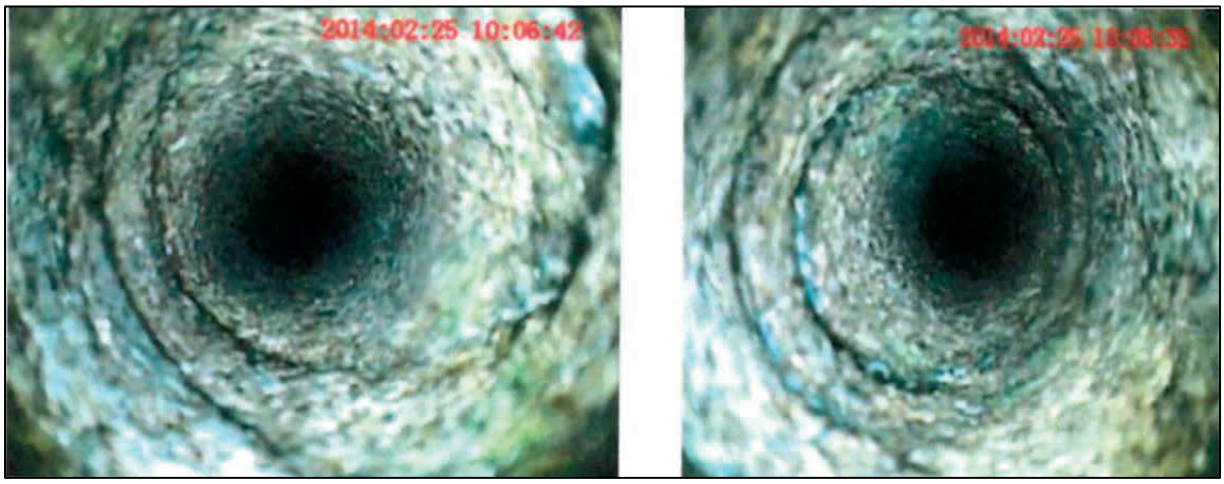


Figure 3.6: Two continuous sub-horizontal weakness planes with visible infilling material *(Internal Mine Document (a), 2014)*



Figure 3.7: Near vertical joint observed at 2.2m in the hanging wall *(Internal Mine Document (a), 2014)*

The equipment is potable and powered by rechargeable batteries (household supply). Offsets are bendable and sometimes are stuck in the hole. For this reason, the borehole diameter should be at least 20mm to 25mm bigger than the camera diameter. After confirming structures detected by GPR, using Borehole camera inspection, FOG light is installed for continuous monitoring of the detected structures. The next section details how FOG lights operate.

3.4. Fall of Ground (FOG) Light

Fall of ground lights are ground-monitoring instruments designed to detect and warn personnel underground of separation or movement within the rock mass above, thus helping to prevent losses of lives and provide safe working conditions. FOG light is designed to give warning by flashing highly visible light emitting diode (LED) indicator. Green LED identifies a correctly installed FOG light in its monitoring state. To warn personnel of unsafe ground movement, the instrument flashes red LED. FOG lights come in different lengths and movement size detections. The mine uses 2m and 4m trigger lengths that detect movement of 2.5mm. A typical example of a FOG light is shown in Figure 3.8.



Figure 3.8: Fog light *(Anon, nd)*

Applications of FOG light

FOG lights are installed in areas where geological structures were detected by GPR scans and confirmed by Borehole camera inspections. Depending on the depth at which the structures are located, different lengths of FOG lights are utilised. A systematic approach is used when installing this monitoring equipment. The process is simple and straightforward, nonetheless, caution is recommended (Figure 3.9).



Figure 3.9: FOG light installation procedure *(Anon, nd)*

Green light indicates that the working place is safe as shown in Figures 3.10 and 3.11, whereas the red light indicates separation within the hanging wall strata. In addition to geologically disturbed ground, FOG lights are also installed in each refuge bay and waiting

place at the mine. The location of each FOG light is referenced to the nearest survey peg (Internal Mine Document (b), 2015).

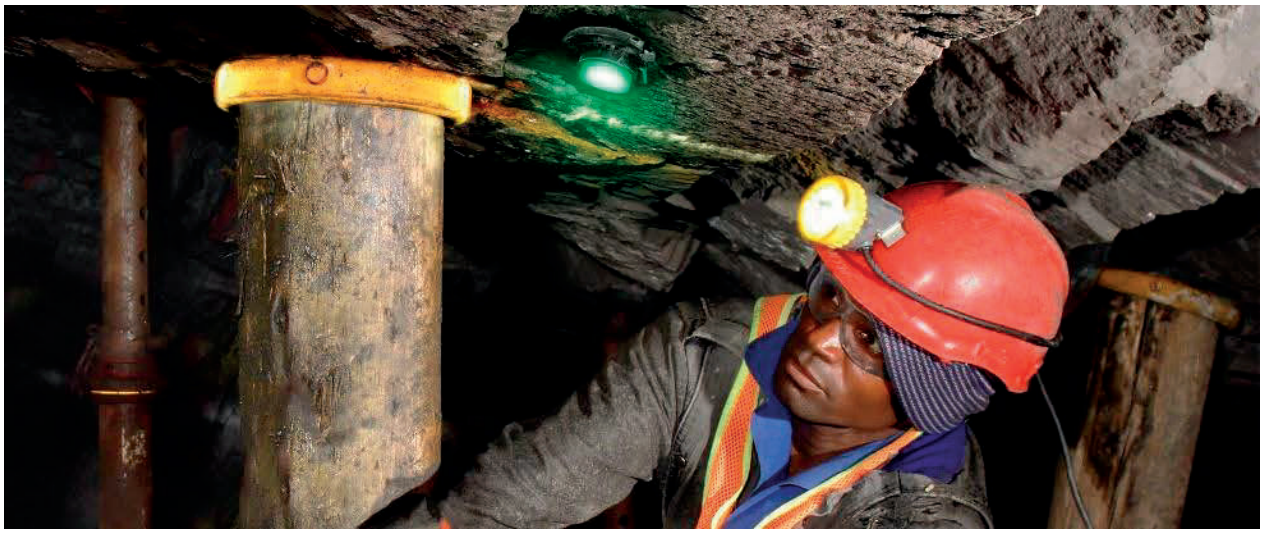


Figure 3.10: FOG light in monitoring condition (Anon, nd)



Figure 3.11: FOG light in monitoring condition at the Chrome mine

Pros and cons of FOG lights

The use of FOG lights for ground monitoring has become common in most underground mines lately. Advantages of FOG lights as stipulated by the manufacture (Anon, nd) include:

- "Simple installation;
- Highly visible LED;
- Available with single or dual indication;
- Once triggered cannot be reset;
- Battery life is approximately 12 months; and
- Comes in various lengths and displacement settings"

There have been cases where FOG lights flashed red immediately after installation or due to vibrations from blasts. In some cases, FOG lights are damaged by moving equipment such as LHDs during the cleaning process. These FOG lights are however replaced, but such incidents disturb the monitoring system. An alarming disadvantage is that unless random checks are conducted to all installed FOG lights around the mine, it will not be known if they are all still in monitoring condition or not. FOG lights are intended for continuous ground monitoring and should therefore serve the purpose. There is no ideal circumference that is specified for these FOG lights. The area monitored by a single FOG Light remains unknown unless the expected key block size is identified. This leaves FOG lights not serving their designed purpose at this mine.

3.5. Panel risk ratings

Panel risk ratings are conducted by means of visual observations. The purpose of this exercise is to rate all working panels based on their geological structures, ground conditions and compliance to support installation. This procedure was adopted after fall of ground analyses by the Mine Health and Safety Inspectorate and the role of rock engineering to combat fall of ground incidents from occurring (Internal Mine Document (a), 2016). It was noted that early entry examinations were conducted inadequately if conducted at all. Moreover, lack of knowledge and appropriate skill relating to identifying geological structures in the working place were contributing to fall of ground incidents. Panel risk ratings were introduced to make awareness to all underground employees about the start-up of high-risk panels and how to eliminate them (Internal Mine Document (a), 2016).

Rock engineering personnel conduct this procedure as they are trained and better equipped in this field. The panel risk rating exercise involves classifying working panels using a checklist (APPENDIX A) in terms of their percentage compliance, judging by the adverse geological structures, general ground condition, mining practices and support installation. There are three categories, which are:

- Low risk – Compliance percentage is equal or greater than 80%
- Medium risk – Compliance percentage ranges between 79% -60%
- High risk – Compliance percentage is less than 60%

Low risk	Medium risk	High risk

Visual observations are recorded using an underground panel risk rating (PRR) checklist. After completing the underground checklist, an electronic rating spreadsheet is completed using the rating matrix shown in Figure 3.12. Underground PRR checklist and electronic rating spreadsheet templates are provided in APPENDIX A.

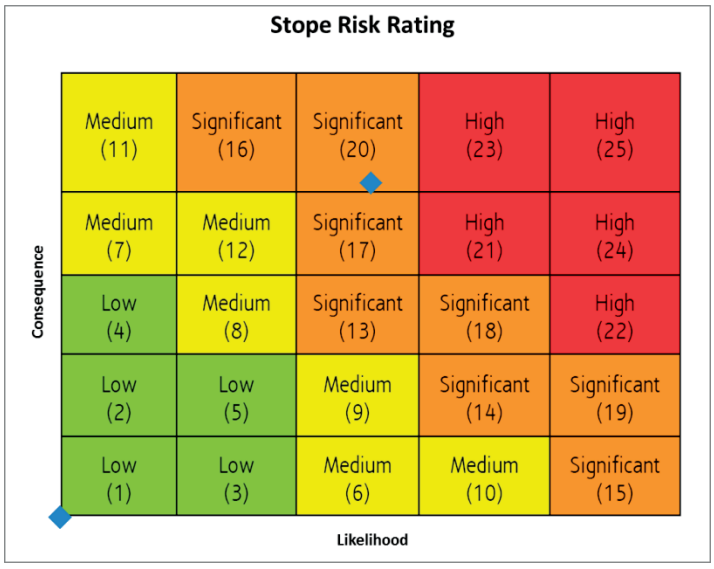


Figure 3.12: Rating matrix

Overall monthly results are presented in a graphical report format. High-risk panels are usually permanently barricaded off with effective barricade or re-established by either up-dipping or down-dipping at the next holing. An example of monthly PRR results from both

North and South sections show most panels rated at medium risk compliance percentage (Figures 3.13 and 3.14).

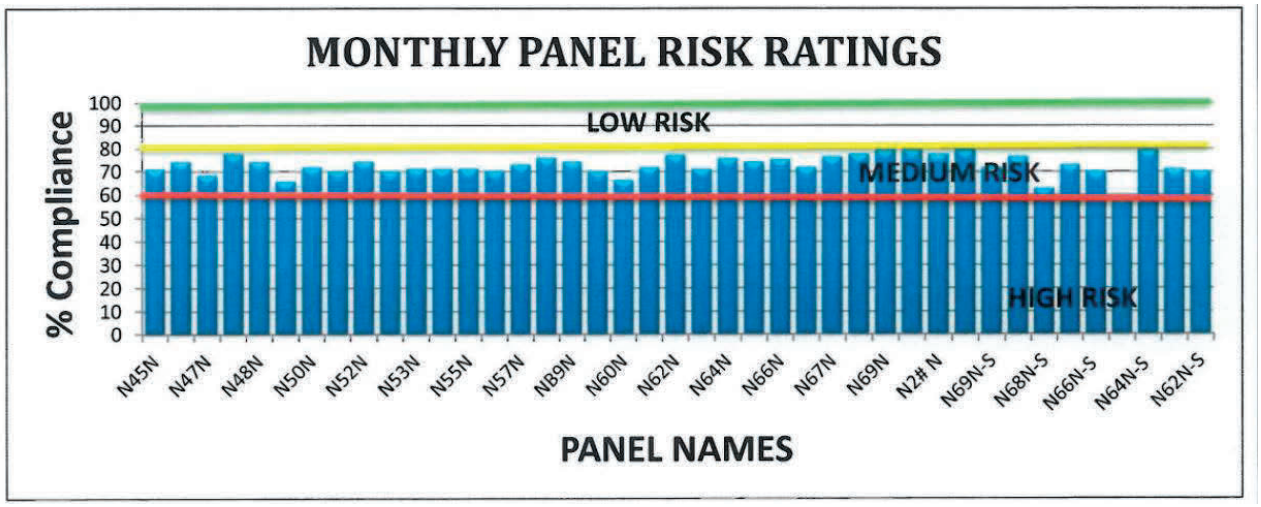


Figure 3.13: North Section Monthly Panel Risk Ratings (April 2018)

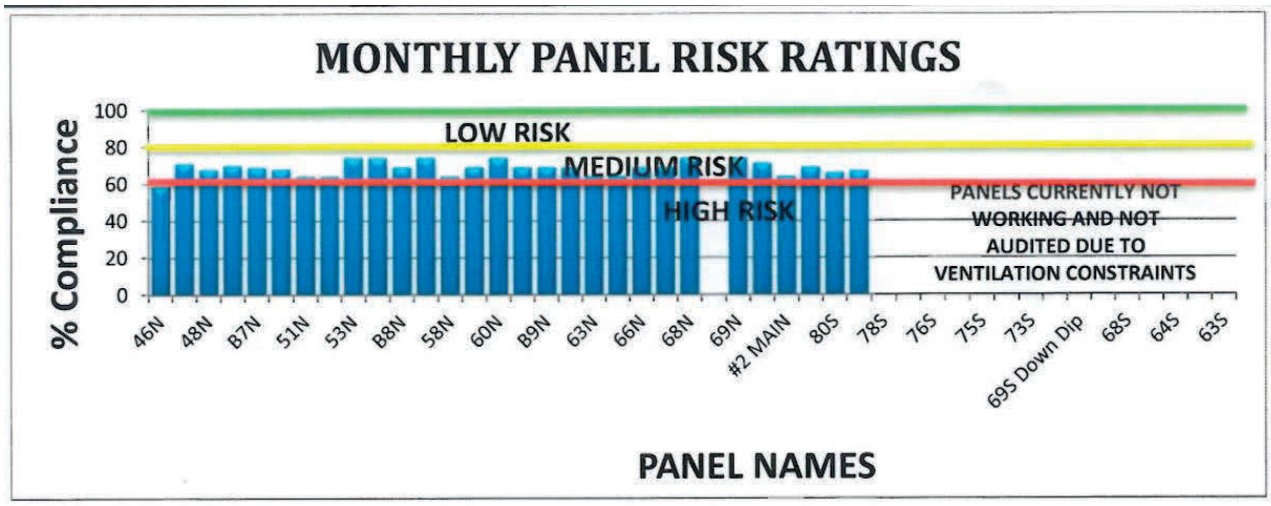


Figure 3.14: South Section Monthly Panel Risk Ratings (April 2018)

3.6. Discussion and conclusions

With such a defined ground monitoring system currently used by the mine, there should be loopholes within, leading to ground falls. Bearing in mind that rock falls result from unstable blocks that are defined by the interaction of both geological structures and stress induced

fractures, the design process described by Stacey (2006) was followed to identify loopholes within the system. If the main objective defined in Step 1 (Problem statement) of the wheel of design (Figure 3.15) is to be able to predict falls of ground, and yet they continue to occur unexpectedly, it means that the objective is not met. When analysing the current ground control system (Step 5) it was found that there are uncertainties in relation to the area or size of blocks monitored by the current monitoring system. Therefore, alternatives need to be considered. It may be necessary to carry out a completely new monitoring design or find ways, which can be used to improve the current system to be able to meet the main objective it was designed for in the first place.

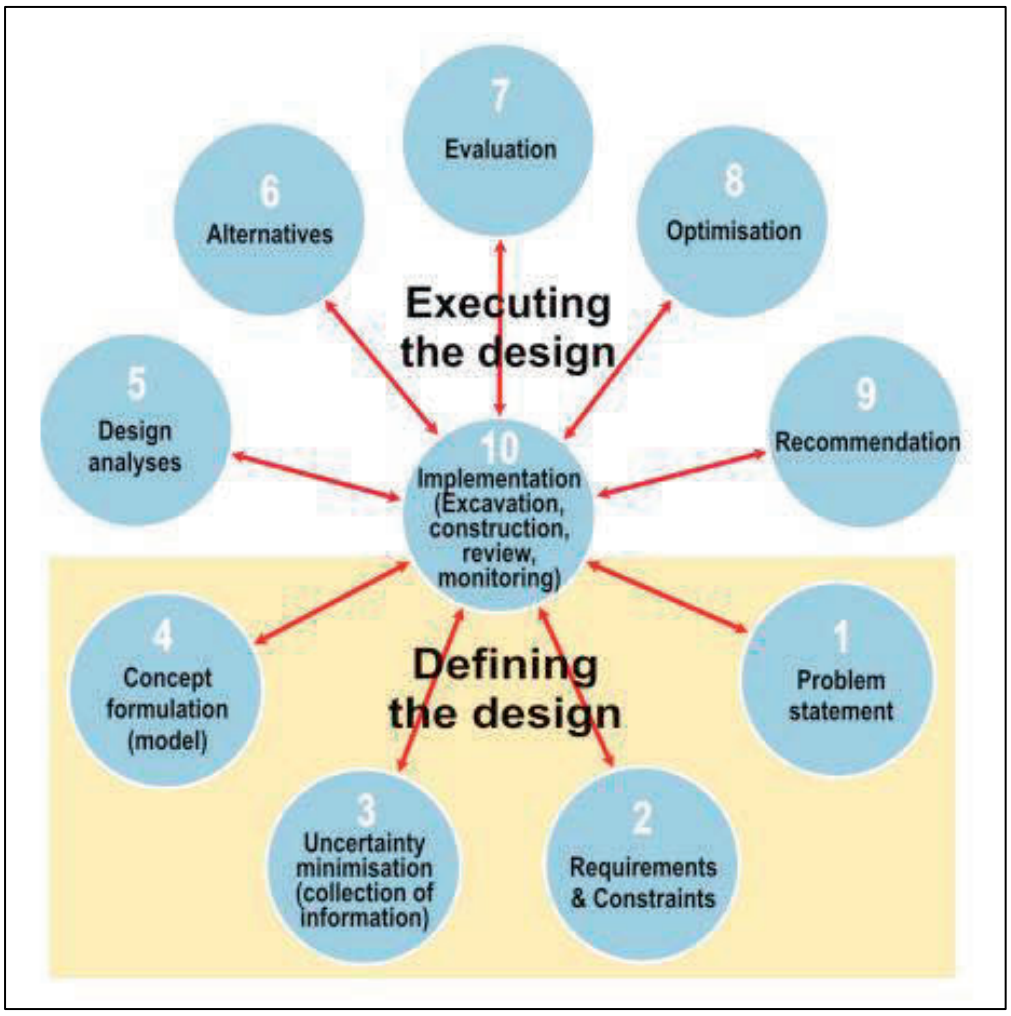


Figure 3.15: Engineering design process wheel *(Stacey, 2006)*

Three possibilities for failure in this case may be that rocks are falling where there is completely no support installed; rocks are falling in between support units; or rocks are falling due to failed installed support system (insufficient support). Stacey & Gumede (2007) refer to the three cases as related to inadequate design of the support system. Often, rock fall accidents are pointed to poor quality control and personnel in the mines not adhering to mine procedures and standards relating to support installation. This might be correct, however not always the case. At times, the main cause of continuous rock falls corresponds with the design. The next chapter reviews historical falls of ground in order to have an understanding on the main causes of falls of ground at the mine even after there is an active fall of ground management system in place.

CHAPTER 4: EVALUATION OF FALLS OF GROUND HISTORY AT THE MINE

4.1. Introduction

Excavation stability is influenced by numerous factors such as geological structures, rock type, and rock strength and stress distributions. In shallow mining, geological structures have a significant influence on the stability of an excavation in a particular rock mass. Rock properties vary and their variability differs from area to area and in different scenarios. This chapter looks at historical falls of ground with the aim of understanding the major contributors to instability in this particular mine. This is achieved by looking at the fall of ground database from the past eight years. Furthermore, fallout thickness evaluations are conducted to ensure stability. The fallout thickness estimation is aimed at ensuring that support units currently employed cover 100% of the fallout thickness (i.e. Support unit length is longer than the average fallout thickness) throughout the mine. The mine consists of two main sections and each section is dealt with separately.

4.2. Fall of ground history at the Chrome mine

Over a long term, statistics shows that the rate of fatalities in the mining industry has dropped. In contrast, short-term analyses of individual mines, particularly the Chrome mine under study shows an increase in fall of ground incidences. Historical data shows that the frequency of falls of ground is increasing with depth below surface as mining progresses (Figure 4.1).

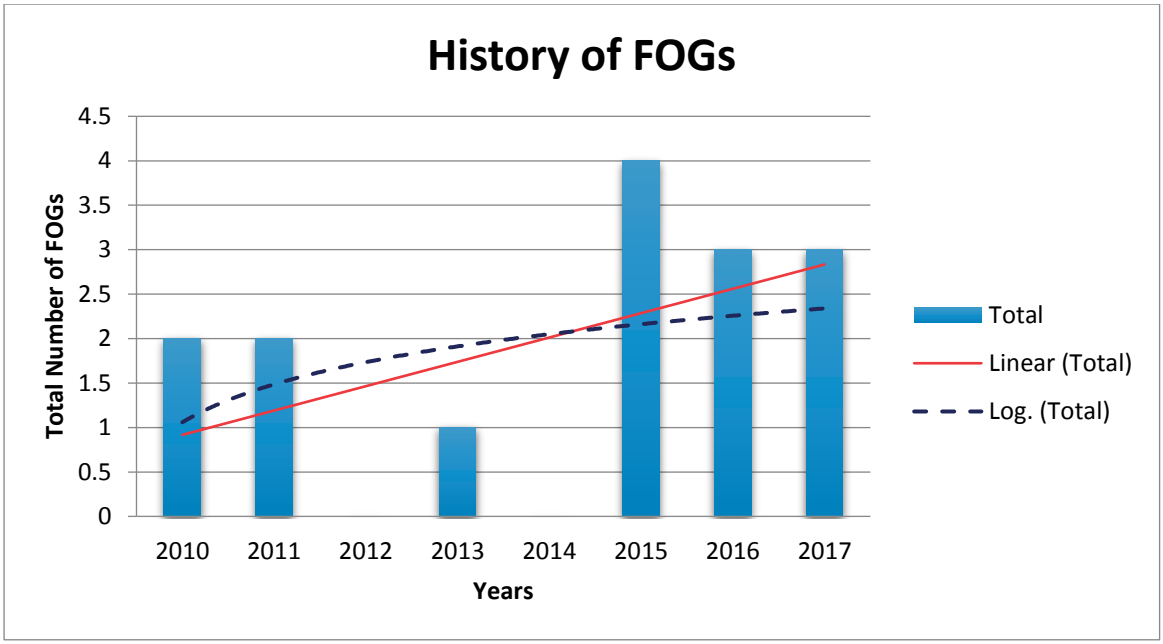


Figure 4.1: Historical record of fall of ground incidences per year

In 2010 through 2011, two incidents were recorded per year. This improved in 2012 where no incident occurred because of complying with mine standards and less geologically disturbed ground conditions. The same outcome was evident in 2014; this proves that it is possible to attain zero falls of ground throughout the mine in a year. A drastic increase occurred in 2015 where four incidents were experienced. It was a major concern as the Rock Engineering Department had already implemented the ground control and monitoring system at the time. The implementation of such a system would be expected to minimise the occurrences of rock falls in the mine. However, that is not the case as it can be seen in Figure 4.1 that six more incidents were recorded in two years (2016 to 2017). A detailed fall of ground database is presented in APPENDIX B.

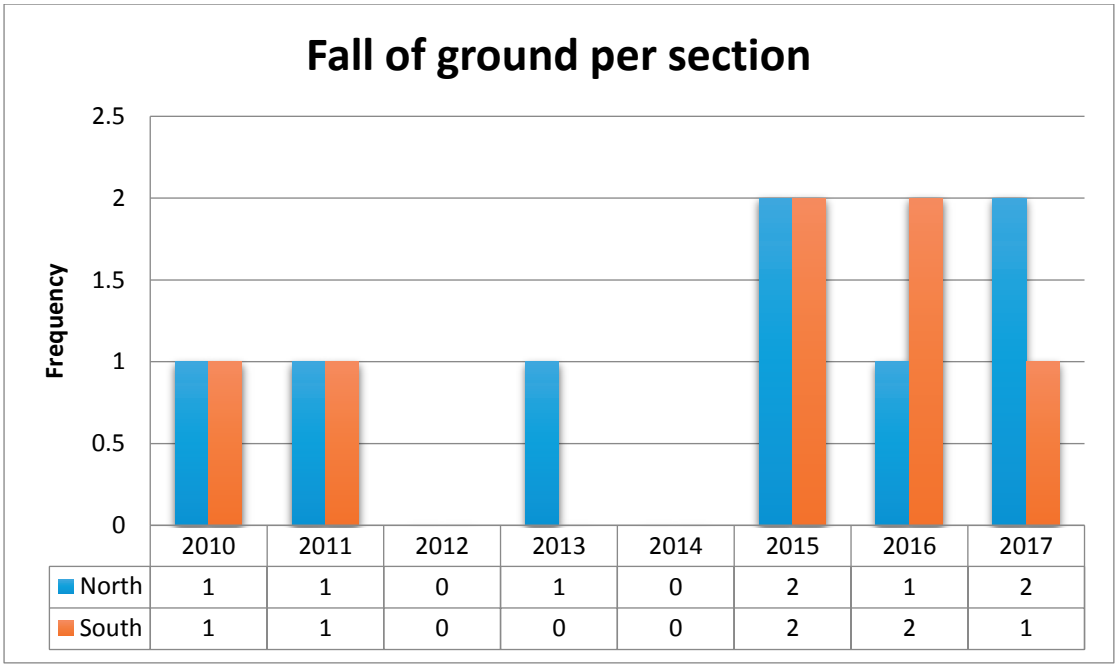


Figure 4.2: FOG per section

A conclusion can be drawn based on the historical data that there are uncertainties within the current ground control and monitoring system. In order to identify these uncertainties, it is important to separate the two main sections within the mine. A summary of fall of ground history per section is shown in Figure 4.2. The North and South section accounts for 53.3% and 46.7% of FOGs respectively. Although the difference is small, it is still considered important to deal with each section individually. The separation is motivated by the distance between the two sections. The average distance between the two sections currently is approximately 800m (Figure 4.3). North shafts are currently developed towards the northwest direction. As a result, the distance between the two sections will increase. The two sections are therefore considered separately to account for variability of rock properties.

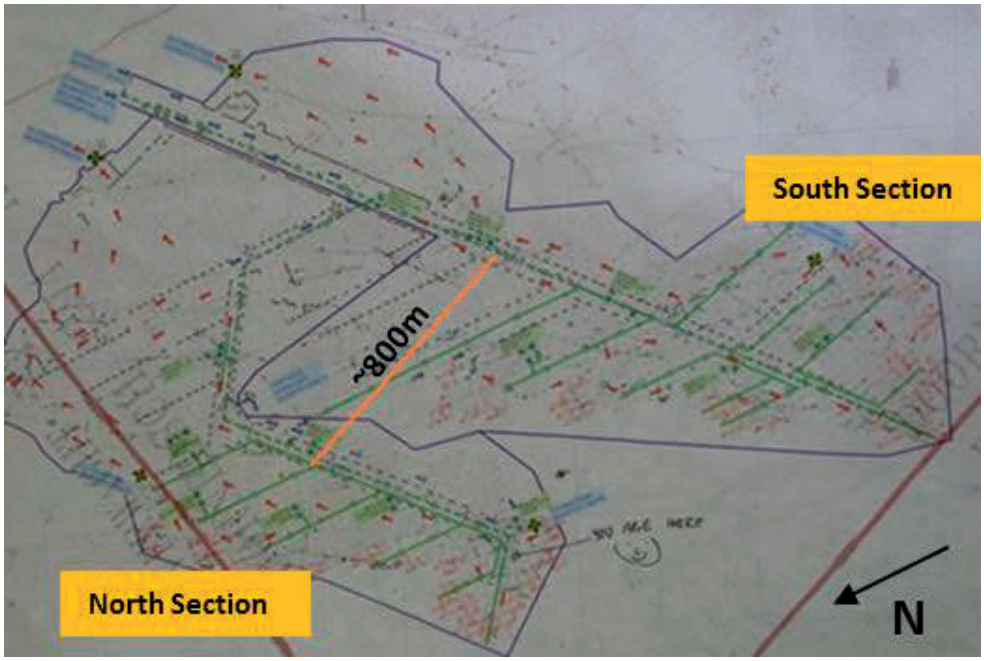


Figure 4.3: Mine layout showing the average distance between two sections

4.2.1. Causes of falls of ground

The knowledge and understanding of aspects that triggered falls of ground at this mine is important. This knowledge will assist in terms of measures that need to be put in place to avoid repetition of unwanted ground movement. In shallow mining environments, falls of ground occur because of interacting geological discontinuities such as faults, joints, domes, and parting planes. Their interaction result in unstable blocks of rocks, which tend to fall under the influence of gravity. For such a mining environment, the dominant failure mechanism is called a wedge failure (Figure 4.4). An unstable block is created due to the intersecting of a near vertical joint together with a low angle join in the hanging wall strata. There is a potential for the block to fail. Failure in this case is due to insufficient support. The support unit length is not adequate for such a scenario since it is not long enough to clamp the joint.

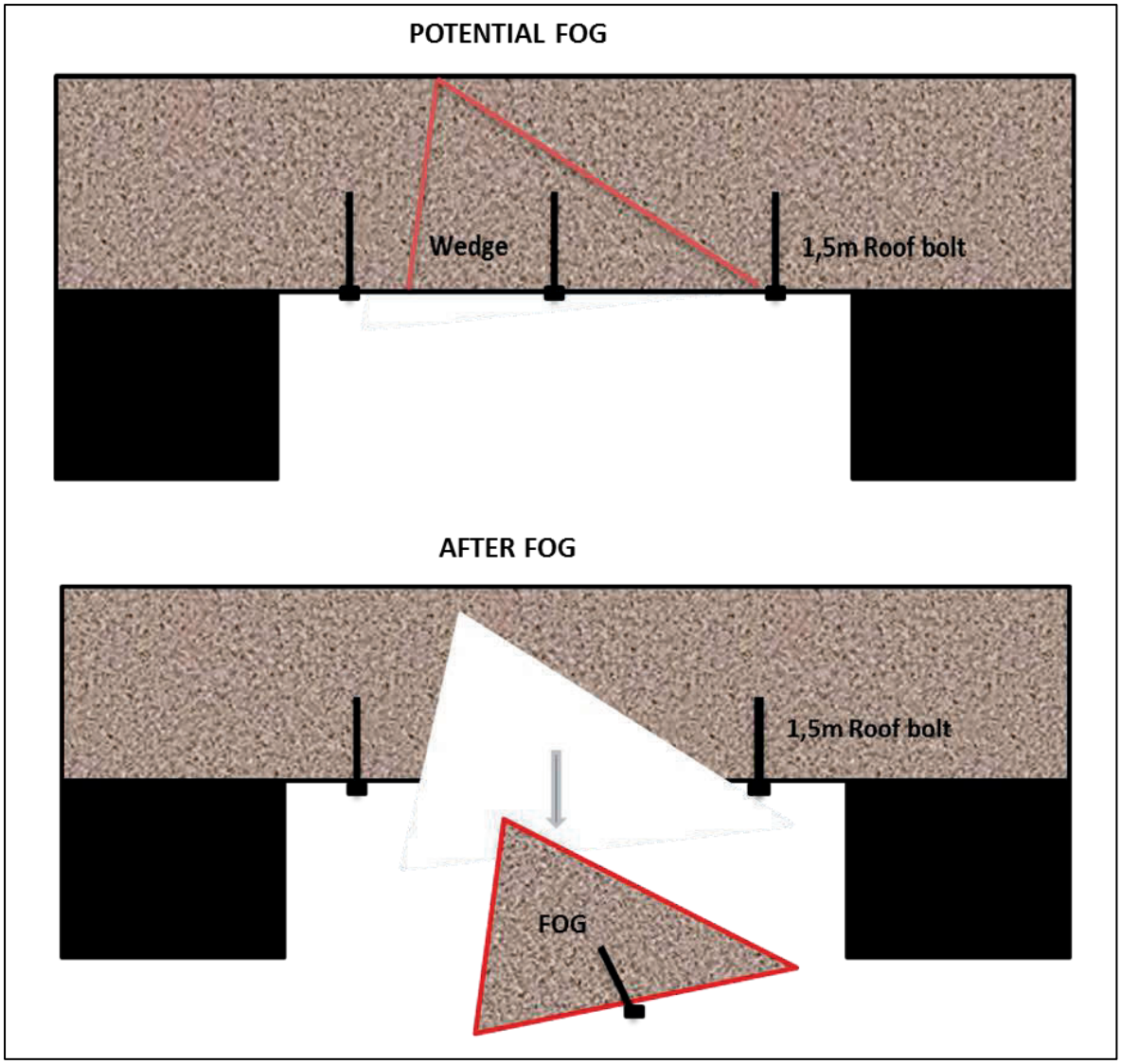


Figure 4.4: Schematic diagram of wedge failure mechanism

A similar failure mechanism can take place in-between support units, were the wedge is either smaller or support unit spacing is large. Similar scenarios lead to falls of ground in both the North and South sections of the Chrome mine. An example of a wedge failure mechanism from the hanging wall strata in the face area is shown in Figure 4.5 and Figure 4.6.



Figure 4.5: Wedge failure at the face area (Internal Mine Document, 2018)

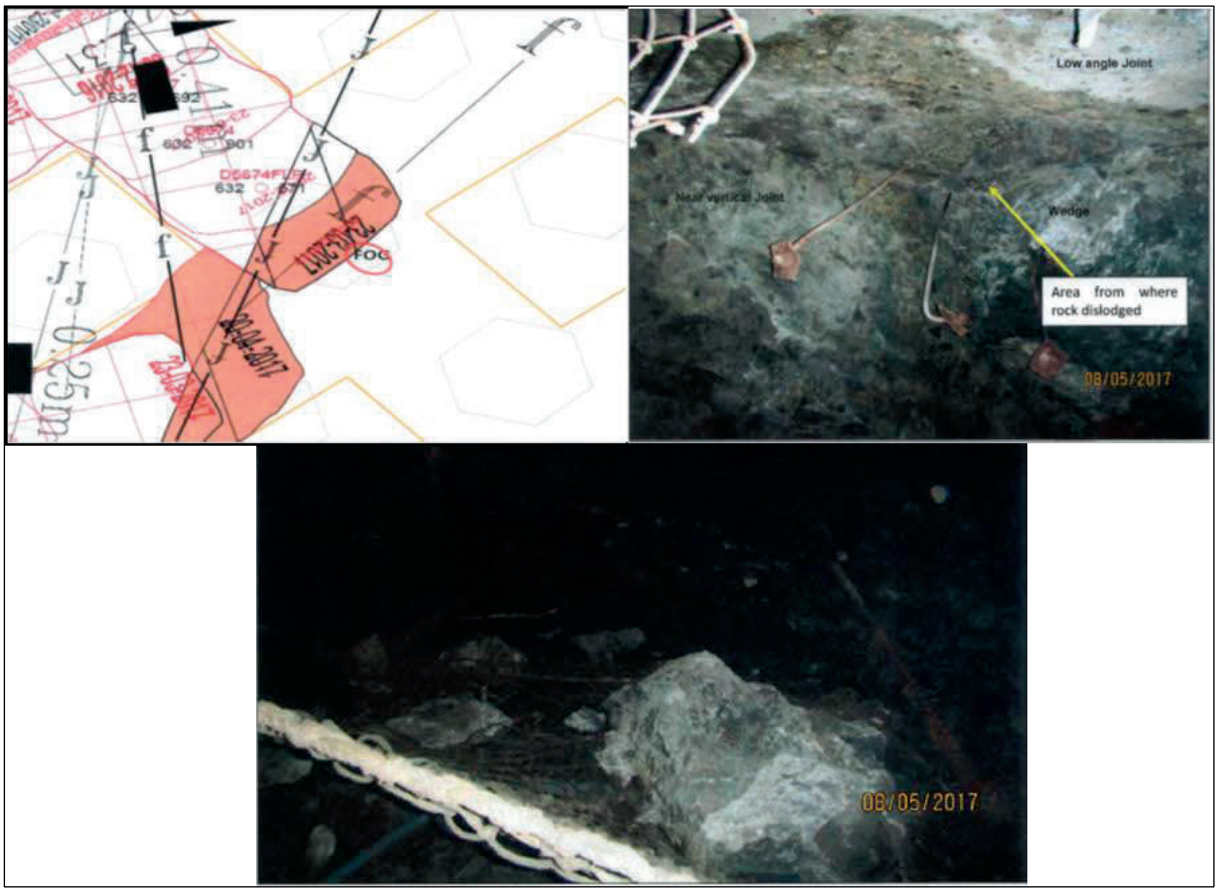


Figure 4.6: FOG following intersection of geological structures (Internal Mine Document (a), 2017)

Various geological structures contribute towards rock falls in the Chrome mine. As previously mentioned, the most common mechanism of failure is wedge failure. The wedge can form on either the hanging wall, sidewall or face. Attributing to the historical data on falls of ground, 73% of rock falls dislodged from the hanging wall (Figure 4.7). In mining, work is carried out in the face area. As a result, most incidents occur in the face area. Activities carried out at the face include drilling (support and production holes), charging, loading and support installation. 60% of FOG incidents occurred when drilling was in progress. Only 23% of the incidents occurred in panel back areas. Usually back areas are abandoned in terms of risk assessments. Attention is given to face areas. The mine operates three shifts per day. FOG incidents have been recorded for morning shift, afternoon shift and in-between shifts as shown in Figure 4.7. Morning shifts account for 73% of falls of ground in the past 8 years.

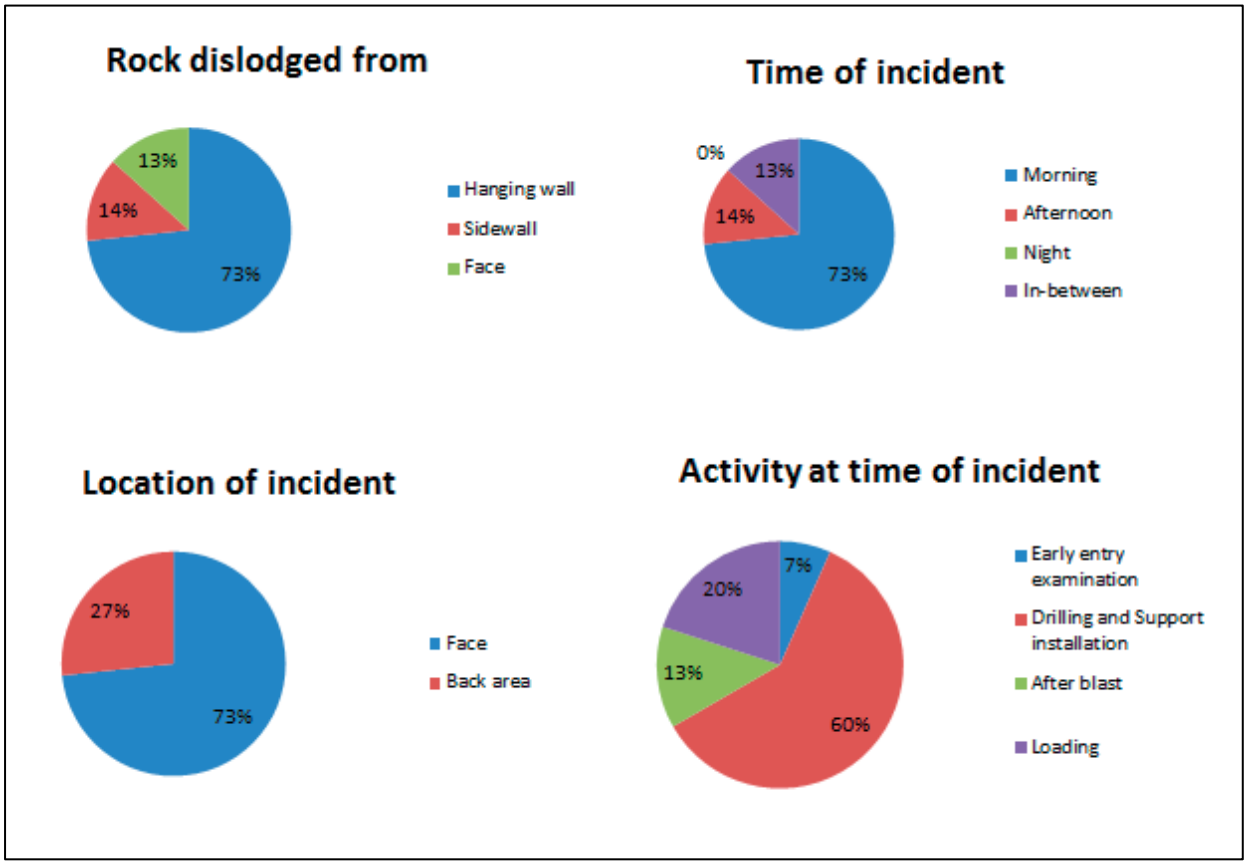


Figure 4.7: Area of rock dislodge, location, time and activity of incident

4.2.2. Compliance with mine standards

Failure to comply with mining standards increases the risk of rock fall related hazards. In some cases, there are no major geological structures involved. For example, during support installation in the face area, a rock fell on the safety net and rolled over. Failure to install the safety net with enough S-hooks resulted in a rock fall incident. This could have been avoided if enough S-hooks were installed to ensure that the safety net is tight and not sagging. Early entry examination and continuous barring are important factors that can limit rock related hazards of this kind. Approximately 35% of fall of ground incidents in this mine were a result of failure to comply with support standards recommended by the mine. Compliance was addressed in detail in Chapter 3 Section 3.5, where risk ratting are conducted to check whether working panels are complying with mine standards as a way of minimising possibilities of FOGs.

Following a study by Swart (2005), Table 4.1 summarises rock engineering practices that can be used to improve stability of underground excavations.

Table 4.1: Rock engineering practices to ensure stability

Rock Engineering practices	Chrome mine
FOG associated with geological structures	✓
FOG database (incidents investigated, recorded and analysed)	✓
Rock mass classification carried out	x
Panels designed based on possible failure mechanisms	✓
Adequate rock mass characterisation and database	x
Monitoring carried out (FOG lights)	✓

The use of appropriate rock engineering practices in design is recommended. The Chrome mine follows most of the above listed practices. As previously mentioned, most falls of ground are associated with geological structures, therefore it is considered important to use a systematic engineering approach in designs to enhance the stability and minimise falls of ground. Prior to any support designs, it is critical to have an insight of the expected fall out thickness. Evaluations of the mine's fall out thickness is discussed in the next section.

4.3. Fallout thickness case studies

The code of practice (COP) to combat rock fall and rockburst accidents in tabular metalliferous mines requires that support be designed to cater for the fall out thickness that corresponds to 95% cumulative probability of occurrence. A histogram of previous rock falls measured can be plotted to obtain the average fallout thickness. Jager & Ryder (1999) estimated fallout thickness using historical data from falls of ground as shown in Figure 4.8.

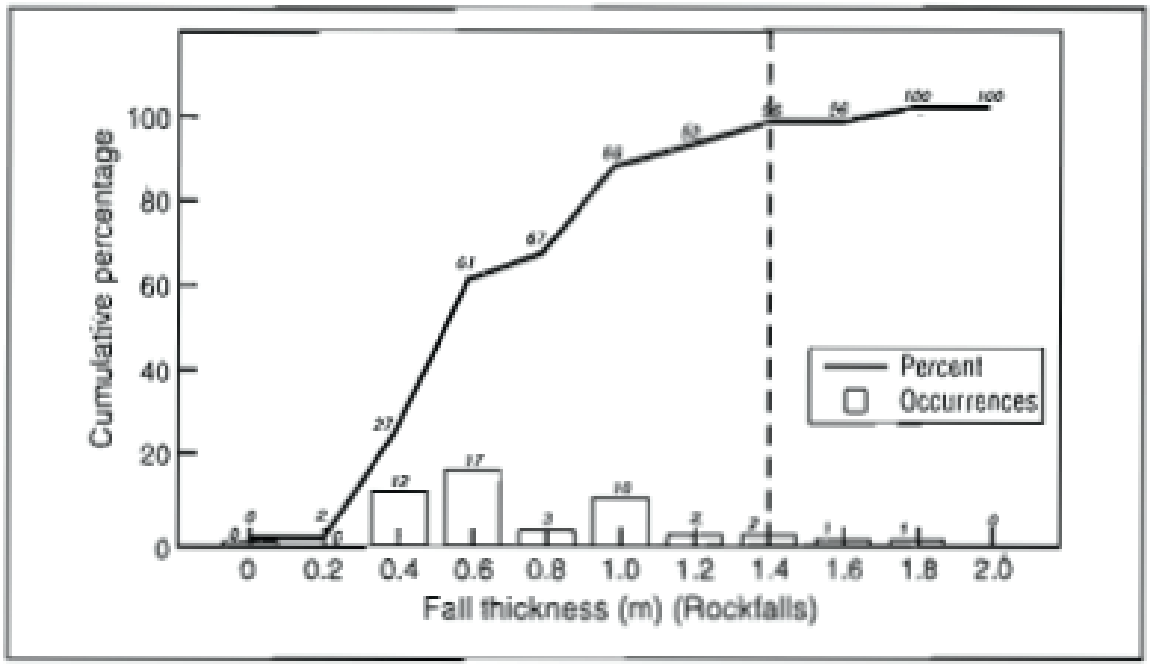


Figure 4.8: Cumulative rock fall thickness for a particular ground control district (Jager & Ryder, 1999)

Two approaches were used to determine fall out thickness. Firstly, fall out thickness from previous FOGs was used and secondly, brow thicknesses where measured for the same exercise. Table 4.2 summarises fall out thickness estimation based on historical FOG incidents at the mine.

Table 4.2: Fallout thickness from historical FOG incidents

Thickness range (cm)	Frequency	Cumulative frequency
0	0	0%
01-20	5	33%
21-40	2	47%
41-60	6	87%
61-70	0	87%
71-80	1	93%
81-90	0	93%
91-100	1	100%

Based on the historical data from the Chrome mine, the average fallout thicknesses at 95% cumulative frequency was found to be between 0.9m and 1.0m i.e. 0.95m (Figure 4.9). The fallout thickness in this case is based on historical FOG thicknesses for both sections combined.

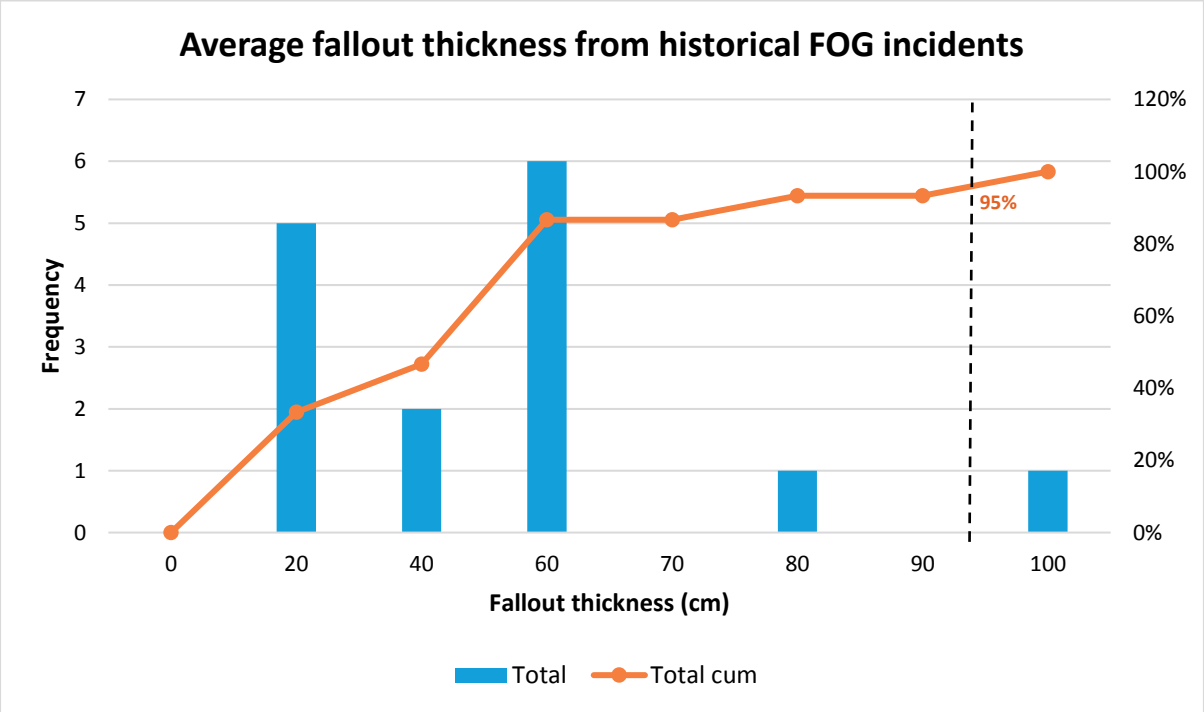


Figure 4.9: Average fallout thickness estimated from historical FOG data

Brow thickness data was collected for each section. This method was conducted to confirm whether the fallout thickness from historical data would correlate with that obtained using

brow thickness method. For this method, brow thicknesses were measured in both the North and South section. Detailed data collected is provided in APPENDIX C: where C.1 and C.2 represent North and South section respectively. Brows were measured in active panels at the time of visit. In sections where no mining was taking place, data was not collected.

Currently, the fallout thickness stipulated in the Code of practise of the mine is 0.7m. Support units are however 1.5m long and where essential, secondary support deemed 4m in length is used. The last update on fallout thickness was conducted 5 years back. Two methods are used in this study to determine whether the support units are still sufficient to support 100% of the current fallout thickness.

4.3.1. North section data analysis

A total of 112 brows were measured in the North section. Table 4.3 shows the North section thickness range and frequencies. This data was used to estimate the fallout thickness using the cumulative frequency as shown in Figure 4.10.

Table 4.3: Fallout thickness estimation from brow data (North section)

Thickness Range (cm)	Frequency	Frequency (%)	Cumulative frequency (%)
0 - 10	0	0%	0%
11 - 20	5	4%	4%
21 - 30	13	12%	16%
31 - 40	26	23%	39%
41- 50	26	23%	63%
51 - 60	17	15%	78%
61 - 70	12	11%	88%
71 - 80	3	3%	91%
81 - 90	2	2%	93%
91 - 100	7	6%	99%
100 and more	1	1%	100%

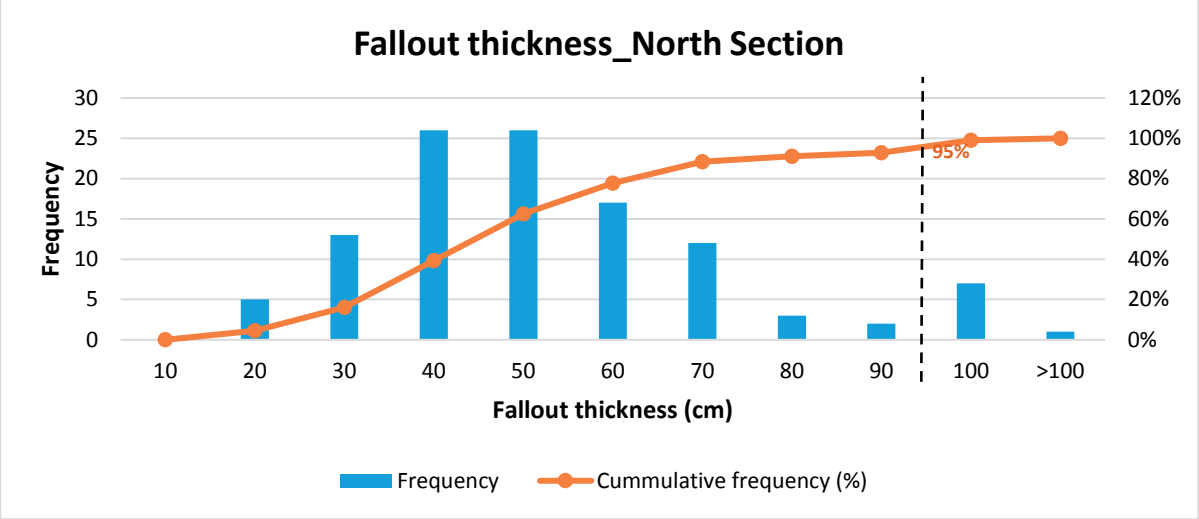


Figure 4.10: Fallout thickness (North section)

Based on the 95% cumulative frequency by (Jager & Ryder, 1999), average fallout thickness in the North section is within the 90-100 cm range.

4.3.2. South Section data analysis

In the South section, a total of 111 brows were measured. The same method stated in subsection 4.3.1 was also used to estimate the fallout thickness in the South section using data provided in Table 4.4.

Table 4.4: Fallout thickness estimation from brow data (South section)

Thickness range (cm)	Frequency	Frequency (%)	Cumulative frequency (%)
0 - 10	1	1%	1%
11 - 20	6	5%	6%
21 - 30	25	23%	29%
31 - 40	25	23%	51%
41- 50	24	22%	73%
51 - 60	9	8%	81%
61 - 70	7	6%	87%
71 - 80	6	5%	93%
81 - 90	1	1%	94%
91 - 100	5	5%	98%
100 and more	2	2%	100%

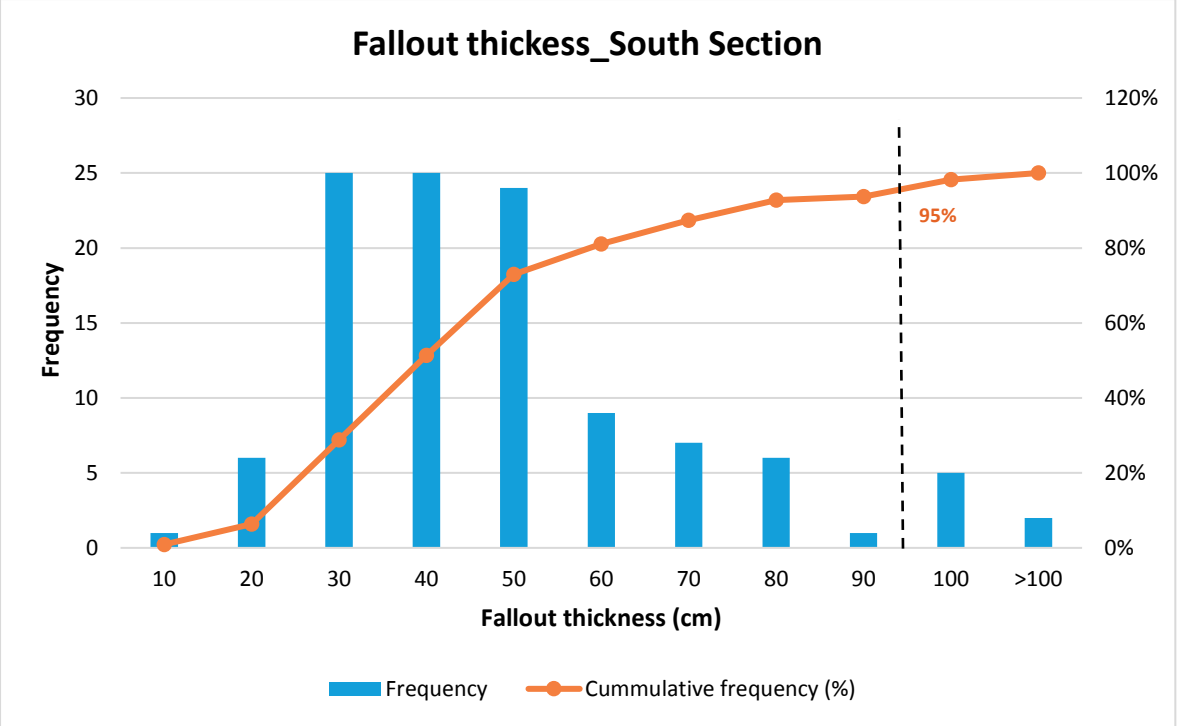


Figure 4.11: Fallout thickness (South section)

Based on the 95% cumulative frequency by (Jager & Ryder, 1999), average fallout thickness in the South section is within the 90-100cm range (Figure 4.11). Fallout thickness obtained in both sections is within the 90-100cm thickness range. Five years ago, the average fallout thickness was 0.7m. Results show that currently the average fall out thickness is between 0.9m and 1.0m i.e. 0.95m. The difference in the two average fallout thicknesses, prove that ground conditions are not always the same throughout the mine. The current ground condition is characterised by many geological discontinuities as opposed to five years ago. Hence, the current fallout thickness is approximately 1.0m

Owing to the support capacity limits of support unit, the support elements spacing that will cater for the fallout thickness are determined. The support spacing is then compared to that of the weakness planes to ensure the support spacing is not excessive. Results show that in both sections, primary support (110kN, 1.5m long roof bolts) is sufficient to clamp 100% of the fall out thickness. The current support pattern of 2m spacing with a factor of safety of 1.29 is still acceptable in this condition.

4.4. Summary and conclusions

This chapter reviewed historical falls of ground and their main triggering factors. It was found that most FOGs were caused by geological structures. Inability to comply with support standards also affects the stability of mining excavations. Fallout thickness evaluations were also conducted using two approaches, namely historical data (FOG database) and brow thickness data. Both of these methods show that the average fallout thickness for both sections is 0.95m. Although results from the two methods are similar, caution should be exercised with roof bolts installation angle. If the roof bolt installation angle is between 70 and 90 degrees, then the 1.5m long roof bolts can still be sufficient to support an average fallout thickness of 0.95m. As soon as the angle of installation gets shallow, the support unit length becomes insufficient to support 100% of the estimated fallout thickness. Roof bolt installation angle is still a challenge at the Chrome mine as it was indicated in Section 3.5, that panels are rated medium risk due to this factor.

Rock fall related hazards should be avoided as they pose unsafe working conditions. Based on this study, synthetic approach can be used in conjunction with actual mine fall of ground database. This gives rise to a larger database with more accurate parameters that can be used in further designs. In conclusion, the overall FOG database together with the current FOG management strategy shows that joint properties are not taken into account in the system as much as they affect stability. It has been identified that there are uncertainties in terms of the block size monitored by the system, the evaluation of probability of rock falls can be carried out by means of joint data measurement and analysis (Gumede, 2006). As a result, the block size monitored can be estimated from joint data (Gumede & Stacey, 2007). Chapter 5 gives a review on joint characteristics that influence FOG and how they can be measured underground in order to use joint data for predicting potential rock falls.

CHAPTER 5: CHARACTERISATION AND MEASUREMENT OF ROCK MASS JOINTING

5.1. Introduction

Different minerals combine to form different rock types. A large volume of rocks is called a rock mass. Naturally, a rock mass is not a single solid block of rock. A rock mass is made up of different rock minerals together with geological disturbances such as joints, faults, parting planes and other weakness planes. Palmström (2001) summarises the building blocks of a rock mass as shown in Figure 5.1

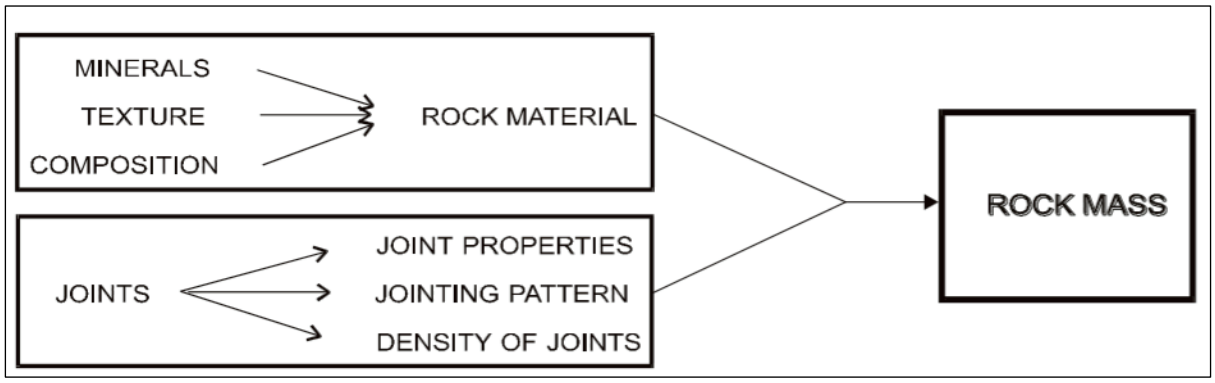


Figure 5.1: Building blocks of a rock mass *(Palmström, 2001)*

Rock masses are considered to be weaker than intact rocks as they consist of discontinuities (ISRM, 1978). These discontinuities form individual blocks within the rock mass, thus weakening the rock mass strength as compared to intact rock strength. A summary on how intact rock differs from a rock mass as a result of discontinuities is presented in Figure 5.2 (Hoek et al., 1998). Engineering excavations that are practiced in a rock mass require the knowledge of features that characterise that particular rock mass. For this study, the most common and prominent geological discontinuities are joints. The presence of joints in a rock mass therefore influences the excavation and support design. It is mandatory for rock engineering personnel to design stope support in South African mines as part of their duty allying the code of practice to combat rock fall and rockburst accidents in tabular metalliferous mines. These support design methodologies are supposed to be motivated and documented. The design procedure used at this particular mine however does not take into account the actual sizes of blocks or wedges that might be formed in the immediate hanging

wall. Although empirical models of determining fallout thickness in a statistical basis are used, the lateral dimensions in this case are ignored.

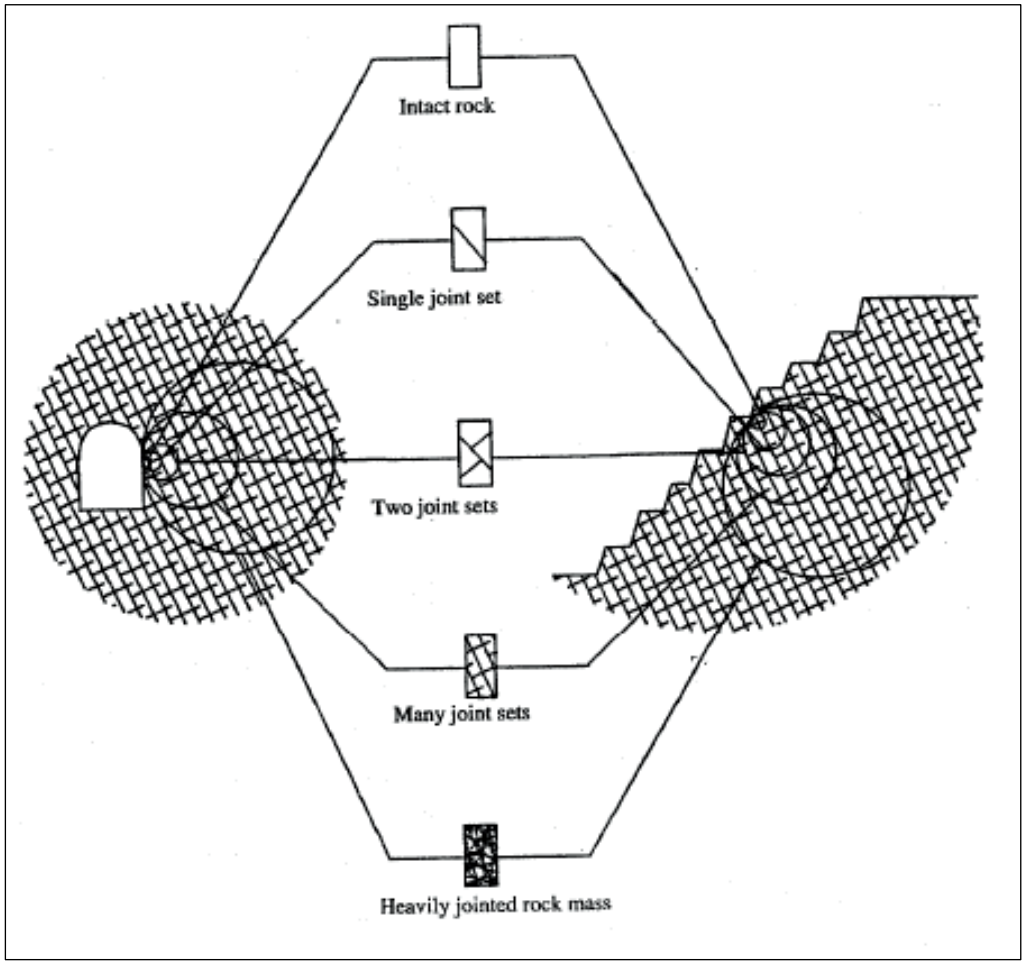


Figure 5.2: Difference between intact rock and rock mass *(Hoek et al., 1998)*

The interaction of geological discontinuities (joints, bedding planes, fractures and faults) form unstable blocks of rocks in mining excavations. It has been identified that there is a general lack of information on the characteristics of these structures, particularly the geometrical properties. To remedy this condition to some extent, this Chapter provides literature relating to joint formation mechanisms, joint characteristics and ways of measuring joint characteristics underground. This is completed with the interest in understanding how joint data can be used in the prediction of rock falls following a study by Gumede and Stacey (2007).

5.2. Joint formation mechanism

Joints are natural occurring fractures in a rock, which form discontinuities in a rock mass. It is believed that joints were formed during the rock formation ages. Several suggestions on joint formation mechanisms have been provided by researchers (Leith , 1923; Dershowitz, 1984; Hobbs et al., 1976; Ivanova, 1998; Suppe, 1985; Price, 1966). Although various suggestions were put forward, there is a common hypothesis that the formation may have been due to a single or combination of these mechanisms. The joint formation mechanisms are classified into three categories. First mechanism relates to the earth stresses (earthquakes). Secondly, joints are said to have formed during the cooling down of rocks. During the cooling process, the rock experiences different temperatures which tend to influence the expansion and contraction rates. Lastly, it is believed that stress changes due to overburden loading and unloading also gave rise to jointing in rock masses.

A general conclusion can be made that jointing in a rock mass is due to rock cooling, earthquakes, folding, and contraction of sediments, compressive and tensile stresses as a result of regional compression. Shear, tensile and compressive stresses due to these factors give rise to joints when the stresses exceed the strength of the rock mass. The rock mass tends to fail along weakness planes thus creating joints. Based on the rock material and severity of these stresses, joints formed tend to vary. Price (1966) stated that although it is generalised that joints form under tensile stress they can also form under compressive stress.

Ivanova (1998) classified geological disturbances occurring within a rock mass into two main groups namely; tensile features and shear features. Tensile features are mainly joints, opening in planes that are either parallel to maximum or intermediate principal stresses and/or orthogonal to minimum principal stress. On the other hand, shear features are faults forming in intermediate principal stress at an angle less than 45 degrees in the direction of maximum principal stress. Faults and joints differ. Their difference can be seen visually. Faults displace the two sides relative to one another parallel to itself whereas a joint does not show any lateral displacement along its plane. In most cases a rock mass consist of more joints than faults.

Some joints may have infilling material due to the flow of different minerals. The infill material is heavily dependent on the location and depth below surface. Joints at the study area have infilling martial called seperntinite. It is due to inflow of groundwater. This is mainly because the mining environment is of shallow depth. Sepentinite is soapy in texture and it

tends to weaken the joint contact. The soapy material makes it easy for sliding to take place along that particular plane. A group of joints with similar geotechnical properties is referred to as a joint set. Joint sets are usually a result of stresses acting on a rock mass in a near parallel manor over time. Rock falls underground generally involve the interaction on a joint set with other geological structures. Joint characterisation is therefore an important aspect in mining.

5.3. Review of joint characteristics

In general terms, the size of a rock mass is large such that it is difficult to measure its mechanical properties. Therefore information relating to joint properties of a particular rock mass is obtained through site observations and drilled core. As already stated previously, joints are geological discontinuities in a rock mass that separates the rock into distinct blocks without any visible lateral displacement. They are normally aligned sub-vertically or traverse bedding planes. Joints are also weakness planes and therefore have a degree of influence in excavation stability, either directly or indirectly so. Weakness planes in a rock mass are characterised by various properties which influence the strength of the rock mass thereof.

In order to understand how a jointed rock mass behaves, it is important to have a comprehensive description of factors that characterise it. These factors include joint shape, size, orientation, spacing, length, and strength. Joint shape and size are difficult to measure in two-dimension. The most important factors that describe rock joints according to (Priest & Samaniego, 1983) are orientation, spacing, length and shear strength of the joint. However, in this study the interest is in the three geometrical properties namely: length, spacing and orientation. Predominantly, the joint properties of interest in a project are influenced by the nature of the problem and its objectives. As this project is based on falls of ground and identifying the potential unstable key blocks, the joint properties of interest where selected based on their ability to define a block of rock in a rock mass. The three joint properties of interest are discussed individually below.

5.3.1. Joint orientation

Joint orientation is the attitude of joint in space. It is described by the angles of dip and dip direction. Joints are orientated in 3D but can be represented in a 2D space using stereonets. Dip angle is the maximum inclination angle of a plane measured between that plane and

horizontal plane, whereas dip direction is measured clockwise in the horizontal plane from north line of the dip. Figure 5.3 shows the relationship between dip, dip direction and strike (ISRM, 1978). Strike is always perpendicular to the dip direction. Strike is an azimuth of a horizontal line in the plane of an inclined feature (joint) in a rock mass. According to (Gumede & Stacey, 2007), joint orientation is one of the most important properties because it effectively neutralises the effects of other properties when orientated favourably.

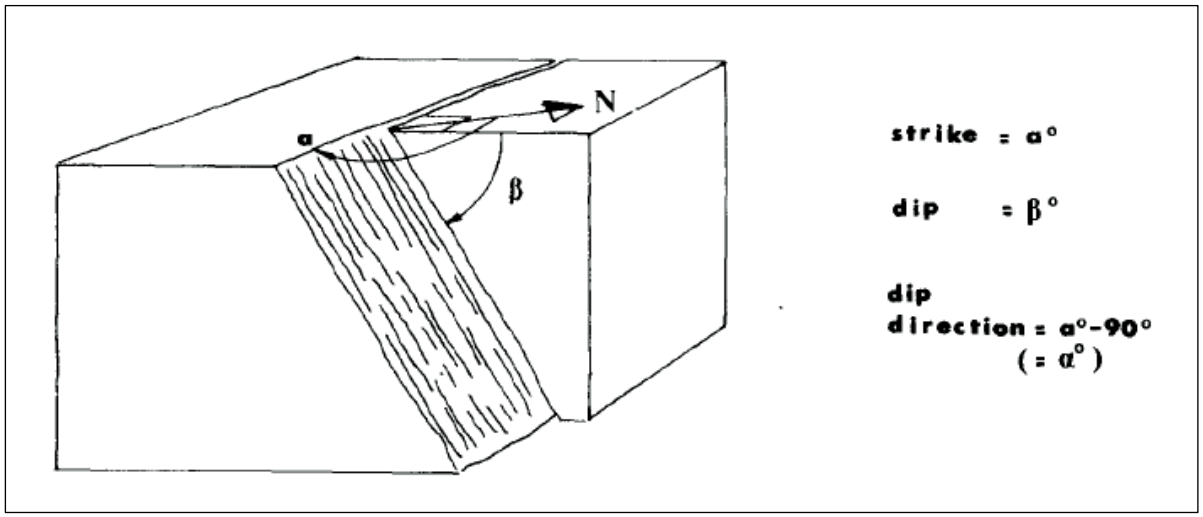


Figure 5.3: Relationship between dip, dip direction and strike *(ISRM, 1978)***.**

Joints may occur individually or in a group. A group of joints clustered in statistically preferred directions is referred to as a joint set. In contrast, individual joints are usually referred to as random joints (Gumede & Stacey, 2007). Gumede (2006) further explained that in most cases random joints are ignored in analysis unless they contribute a significant proportion to the total number of joints.

Research on joint orientation distributions has been conducted by several researchers who came up to different conclusions. Conclusions from researchers such as (Baecher et al., 1977; Stacey & Bartlett, 1990; Gumede, 2006; Grady, 1983; Barton, 1976) showed that joint orientation follow a normal distribution. Joint orientation of different joint sets determines the shapes of blocks of rocks formed within a rock mass. In relation to joint orientation, the excavation orientation will influence the stability thereof. The failure mode is also determined by the joint orientation properties.

5.3.2. Joint length

Lines that can be visually identified in a rock mass are joint traces. Each joint trace extends to a certain length which specifies the joint length in a rock mass. Joint lengths are known to define the probable length of failure of a weakness plane. The length is measured from one end of the trace to the other. Measurements can be taken underground or on surface where the rock mass is exposed. Usually, only a small portion of the joined rock mass is exposed. It is therefore difficult to measure the exact trace of the joint. Due to this constrain, only the exposed portion is measured and the overall length estimated statistically (Gumede & Stacey, 2007).

As a result, joint length measurements are said to be biased since they are difficult to measure accurately. Research by (Baecher et al., 1977; Bridges, 1975; Barton, 1976; Kulatilake et al., 1995; Robertson, 1977; Zhang , 2004) that has been conducted previously shows that the frequency distributions of joint lengths tend to follow a log-normal or negative exponential distribution. A suggestion on correcting the biased joint length measurements was made by (Kulatilake & Wu, 1984).

Joint length has an influence on stability. A rock mass with discontinuous joints is considered to be more stable than a rock mass with continuous joints. The more continuous or the longer the joint is, the weaker the rock mass becomes because failure occurs along the plane of weakness.

5.3.3. Joint spacing

Joint spacing refers to jointing intensity of given rock mass. It is the perpendicular distance measured in-between individual joints. Joint spacing can be measured per unit distance normal to the orientation of the joint set (Nezomba, 2012). According to (Terzaghi, 1965) joint spacing error is encountered when measuring. This is due to sampling lines cutting the joint set at angles less than 90^o. As a result, joint spacing is overestimated. Figure 5.4 shows the correct way to measure joint spacing. To remedy this error, an equation was developed, expressed as follows (Nezomba, 2012):

$$d = \ell \sin(\theta)$$

Where: ℓ is the distance between successive joints of the same set

d is the true spacing distance

θ is the angle between the scanline and the mean orientation of the joint set

The angle θ is obtained accurately if the scan line tape angle is known. In cases where there is no scanline tape, joint spacing can be measured perpendicularly between adjacent joints.

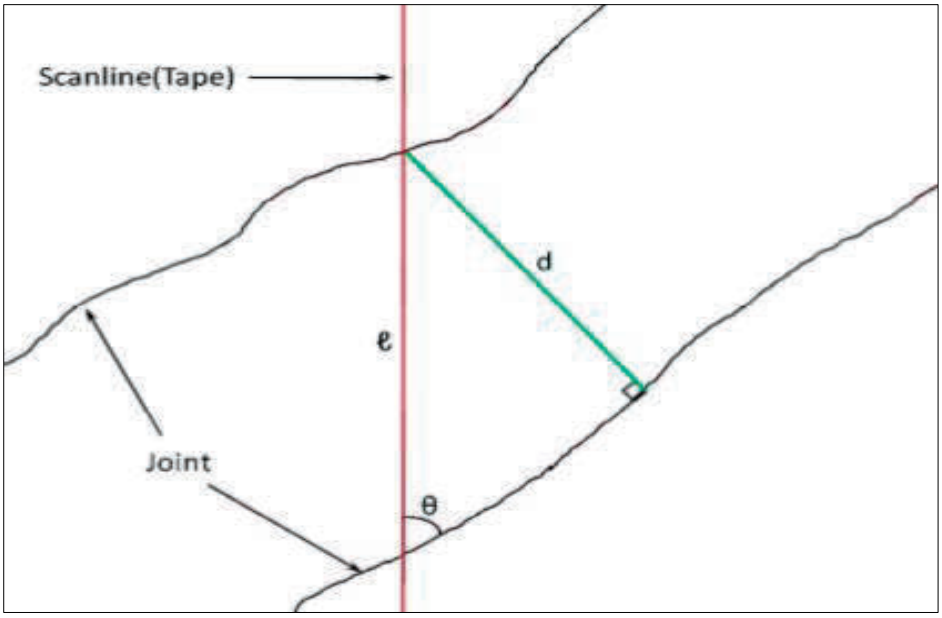


Figure 5.4: Correct measurement of joint spacing *(Nezomba, 2012)*

The more closely spaced the joints are, the weaker the rock mass. This means that there is a large number of joints within a unit area in a rock mass. Joint spacing defines the possible sizes of blocks of rocks that can be formed in a rock mass. In addition this property also detects whether rock mass analysis should be carried out using soil mechanics principle or rock mechanics principle, as the two principles differ in their analysis (Gumede, 2006).

Previous researches on joint spacing show that in general, joint spacing values are positively skewed. Their approximation can be achieved by either negative exponentials or log normal functions (Gumede & Stacey, 2007). A number of researchers (Call et al., 1976; Priest & Hudson, 1981; Wallis & King, 1981; and Kulatilake et al., 1995) suggested that joint spacing values are well demonstrated using negative exponential distributions. On the other hand, some (Steffen et al., 1975; Bridges, 1975; Barton & Choubey, 1977 and Kulatilake et al., 1995) concluded that lognormal distribution is best applicable for joint spacing values.

Gumede & Stacey (2007) concluded that there is a little difference between the two distributions in practical.

In conclusion, the discussed joint properties are important in underground mining. Without the understanding of joint orientation, length and spacing the potential of rock falls cannot be determined in a given excavation. Unless these joint properties are known, the size, shape and possible rock failure mechanisms will remain unknown.

5.4. Joint mapping techniques

Underground joint data can be recovered using numerous techniques. For this study, only underground joint mapping techniques are discussed. The most common techniques are spot mapping, scan line mapping and cell or area mapping. Orientated drill core can also be interpreted to obtain joint data. All these methods are explained in this section.

5.4.1. Core logging

Core logging can be conducted in core recovered from orientated drilled core and interpreted. This technique is an indirect scanline mapping of unexposed ground as joint orientation and spacing can be obtained from the recovered core. Although joint orientation and spacing can be interpreted, the joint length is unknown. This is one of the main disadvantages of core logging. In addition, core can be lost and in highly fractured ground, it is difficult to trace the joint orientation (Gumede, 2006).

In underground mine practices; diamond drilling is usually conducted in order to recover information from unmined ground. Figure 5.5 shows an example of core recovered from underground diamond drilling. Diamond drilling helps attain direct information about structures ahead of mining. The information includes; lithology, geological structures, hydrology and physical and chemical properties of rocks (Wonik & Olea , 2007). Drilled core can also be used in rock mass classification by means of different rock mass rating systems after (Deer, 1963; Coates, 1964; Barton et al., 1974; Bieniawski, 1974; Terzaghi, 1946 and Deer et al., 1967).

Rock mass systems require parameters such as discontinuity spacing, orientation, roughness, degree of alteration, infilling material, ground water content, and rock strength. Although

some of these systems require many parameters than others, they all allocate a number to each parameter in order to rank the rock when determining its quality (Ege, 1849).

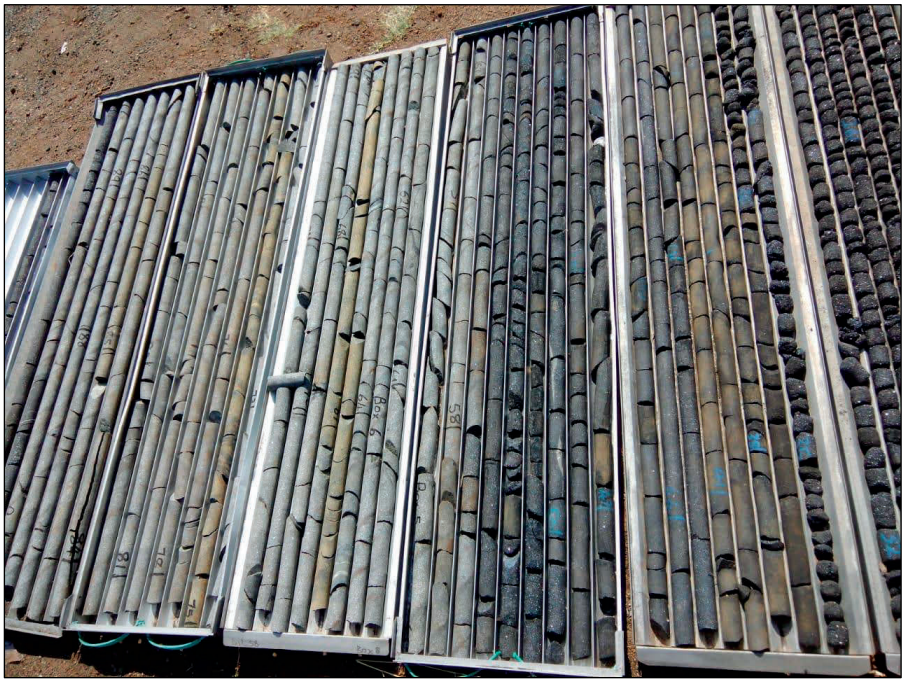


Figure 5.5: Orientated core from diamond drilling

5.4.2. Spot mapping

According to (Brown, 2007) this techniques only involves mapping of joints that are selected to be more important than others. The shortfall that comes with this technique is that other joints are ignored. The level of confidence when using this technique to estimate fall of ground probabilities will be very low. Ignored joint data considered not important also influences the stability of excavations, because they also determine rock block formation (Brown, 2007).

5.4.3. Scanline mapping

Scanline mapping is the most common technique used in underground mining. This technique requires a measuring tape to be extended over a distance on the surface of a rock mass. The tape can be pinned on both ends in order to keep it in place for the duration of mapping (Brady & Brown, 2004). All joints intersecting the tape are mapped (Figure 5.6).

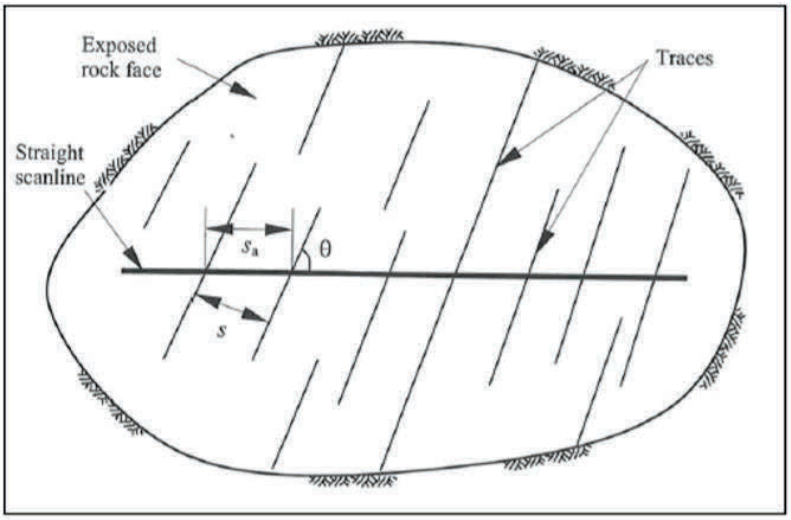


Figure 5.6: Scanline mapping *(Zhang , 2004)*

5.4.4. Window mapping

Window mapping is sometimes referred to as cell or area mapping. This is because of the nature of mapping. The techniques involves a systematic manor of dividing the area to be mapped (face, hanging wall, or side wall) into zones of equal size. Depending on the mine, the zone shapes will vary. Some may be rectangular while some are square. According to (Nicholas & Sims, 2000), these zones are usually square dimensions. Visual observations are carried out in each window, and joint properties (orientation, spacing, length, etc.) are recorded. An example of a rectangular window is shown in Figure 5.7.

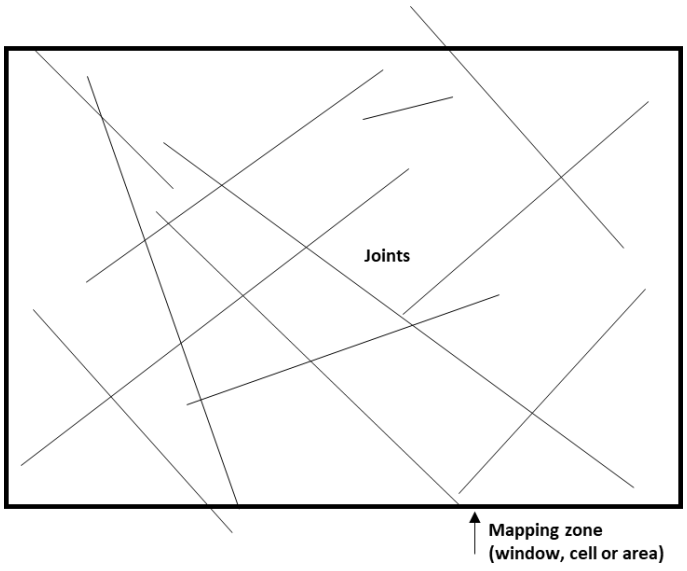


Figure 5.7: Schematic diagram of window mapping zone

Each of these methods has advantages and disadvantages. The mapping methods only focus on joints or weak planes that are exposed, and therefore the structural behaviour behind the exposed surface remains unknown. A summary of advantages and disadvantages of scanline mapping and window mapping is given in Table 5.1 based on previous research (Priest & Hudson, 1981; Gumede, 2006; Nezomba, 2012; Baecher et al., 1977; Mathis, 1987; Landmark & Villaescusa, 1992; Brady & Brown, 2004).

Table 5.1: Advantages and disadvantages of scanline mapping and window mapping

Scanline mapping	**Window mapping**
Advantages	***Advantages***
Data is easy to process and analyse	Reduces sampling biases for orientation and size
The approach is systematic thus easy to control	Considers scattered joints, therefore provides better knowledge of variability in joint properties
Accurate mapping technique	Mapping is quicker, thus more measurements can be taken
Disadvantages	***Disadvantages***
Unreliable information for mean joint orientation less than 20^{o}	Lengths beyond window mapping zone are ignored
Mapping takes more time	
Scattered joints are ignored	

Both scanline and window mapping techniques were considered. However due to limiting factors such as time, access, lighting and exposure, window mapping technique was used for joint data collection in this study.

5.5. Summary and Conclusions

Rock materials are building blocks of a rock mass. Rock masses are not continuous due to geological disturbances. The discontinuities within a rock mass weaken the strength of that rock mass as it has been discussed. The most dominant discontinuities in rock masses were

found to be joints. Joint formation mechanisms influence the joint characteristics of a given joint. The most important joint properties that contribute to underground rock falls are joint orientation, spacing and length as discussed. Joint orientation will determine the possible failure mechanism, Joint spacing will determine the possible size of block formed and joint length will determine the possible shape and failure plane. These properties can be attained by using various underground mapping techniques.

A systematic measurement of joint characteristics conducted by Gumede (2006) was proposed for this study. Jointing geometries (orientation, spacing and lengths) were collected and interpreted with the aim of predicting the probability of rock falls in the mine together with the probability of failure in supported panels. The next chapter details how joint data was collected underground in order to complete the above-mentioned exercise.

CHAPTER 6: UNDERGROUND JOINT MAPPING

6.1. Introduction

Important joint properties and mapping techniques were discussed in the previous chapter. This chapter demonstrates how two of the mapping techniques were used to obtain joint data. Joint mapping was carried out in both the North and South sections of the mine. A brief description of geological discontinuities at the study area is given in the first section 6.2. Furthermore, procedures followed when collecting joint data are provided in section 6.3. In section 6.4 joint mapping results for each section are given. A summary and conclusion on statistical results relating to mapped joint is provided in section 6.5.

6.2. Discontinuity characteristics at the study area

The current underground workings are between 400m to 500m below surface. LG6 and LG6A chrome seams dip at an average dip angle of 10 degrees striking north-south direction. The middling waste (pyroxinite) ranges between 0.8 and 1.0m in thickness, separating the LG6A chrome seem from the LG6A seam above it. The same middling rock type extends more than 10m above the LG6A seam. Pyroxinite is the dominant hanging wall strata. As mentioned previously, the geological setting of the Chrome mine is dominated by joints, faults and domes. Historical fall of ground data presented in chapter 4 showed that most fall of ground incidents involved jointing. Concerning joint mapping, no systematic joint mapping has been done at the mine. The only documented joint data available is summarized in Table 6.1 (Internal Mine Document, 2013).

Table 6.1: General joint set summary

Joint set No	Joint set	Orientation	Formation mechanism
J1	Intermediate	Steeply dipping and Sub-parallel to dip the direction	Primary fracture associated with magma cooling.
J2	Minor	Flat dipping (less than 50degrees) and parallel to bedding planes	Associated with J3 as oblique trending joints
J3	Major	Steeply dipping and Sub-parallel to strike	Last event associated with faulting. It cuts across other joints.

Examples of near-vertical joint, flat dipping joint, horizontal plane and fault at the mine are shown in Figures 6.1, 6.2, 6.3 and 6.4 respectively. The hanging wall instability is subjective to minor joint sets and structures such as domes due to their flat dipping nature. When east-west striking joint sets combine with major joint sets, they tend to cause most of the hanging wall instability problems at the Chrome mine. As falls of ground are often associated with failure along an existing weak plane in a rock mass, the risk of falls of ground occurring at the Chrome mine exists when a jointed rock mass is exposed. The exposed joints were mapped. The mapping technique used is explained in the next section.

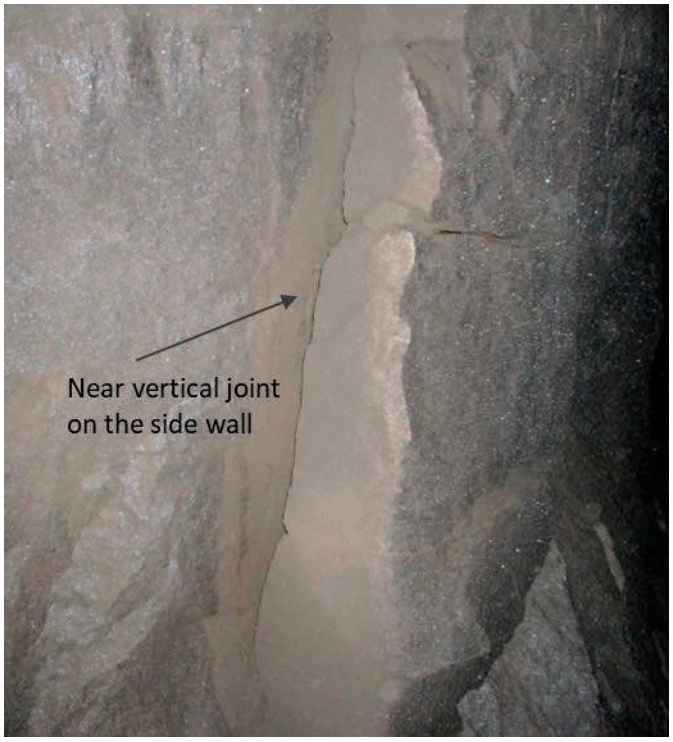


Figure 6.1: Near vertical joint on side wall (LG6 seam)



Figure 6.2: Flat dipping joint on hanging hall



Figure 6.3: Horizontal parting planes in the hanging wall forming brows



Figure 6.4: Up throw fault displacing the LG6A reef onto the hanging wall

6.3. Underground mapping

Joint mapping was carried out in the north and south sections of the Chrome mine using window mapping and scan line mapping techniques. However, most joint mapping in the working panels was carried out using window mapping. This is mainly because of limiting factors such as time and accessibility to exposed joints. Procedure followed for each technique is provided below.

Scanline mapping

A 30m long measuring tape was suspended straight between two points in a working panel mining southwards (Figure 6.5). The tape was looped over roof bolts in order to keep it in place avoiding sagging. After the tape was successfully put in place, all joints intersecting the tape were mapped using the template shown in Table 6.2. The mapping process involved measuring and recording the following:

- Distance along the tape where discontinuities are intersecting the tape
- Type of discontinuity
- Dip angle and dip direction of discontinuity
- Spacing and trace lengths of discontinuities

As stated previously, scanline mapping technique was least used to collect joint data. This technique was only used in panel 74S back area and the data is given in APPENDIX D.3. This technique was specifically chosen for this panel because there are five FOG lights in the area within 4m and yet not many joint traces were visible.

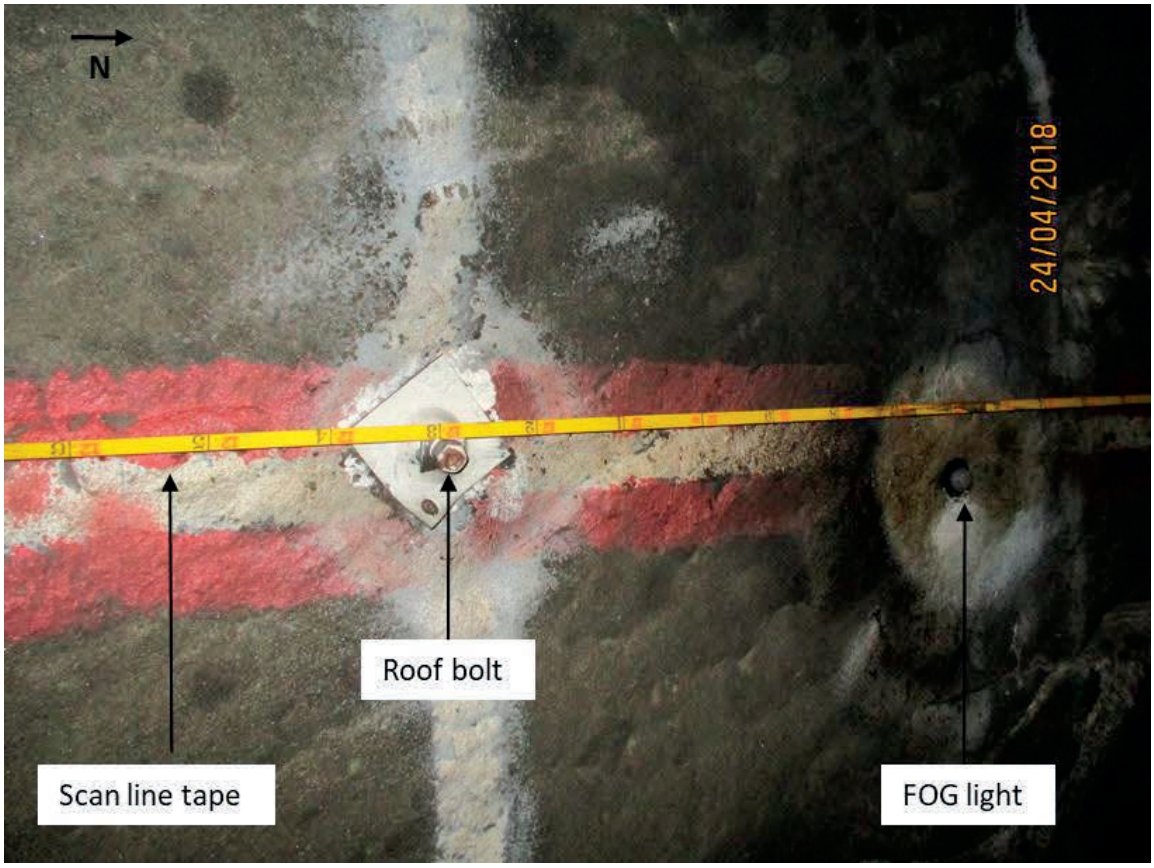


Figure 6.5: Scan line tape aligned to the centreline of the panel

Table 6.2: Scanline mapping data sheet template

SECTION:_____ PANEL:_____ TAPE LENGTH:_____											
SCANLINE TAPE ORIENTATION:_____ PANEL ORIENTATION: _____											
Offset (m)	Geological structure	Dip (deg)	Dip Direction (deg)	Range (deg)	Spacing (m)			Joint Trace Length (m)			
					Min	Max	Average	Min	Max	Average	

Window mapping

Window mapping technique was conducted in both sections of the mine. The technique is fast, thus more panels were mapped using this technique. The mining layout consists of 10m panels and 12m x12m pillars (Figure 6.6). Panels are mined northwards and southwards with mining height of 2.1m. The mapping zone in this case was rectangular in shape. The data sheet template used to record mapped data is shown Table 6.3. After defining the mapping window, discontinuities observed within the window were mapped. The following was measured and recorded:

- Type of discontinuity
- Dip angle and dip direction of the discontinuity
- Spacing and trace length of discontinuities

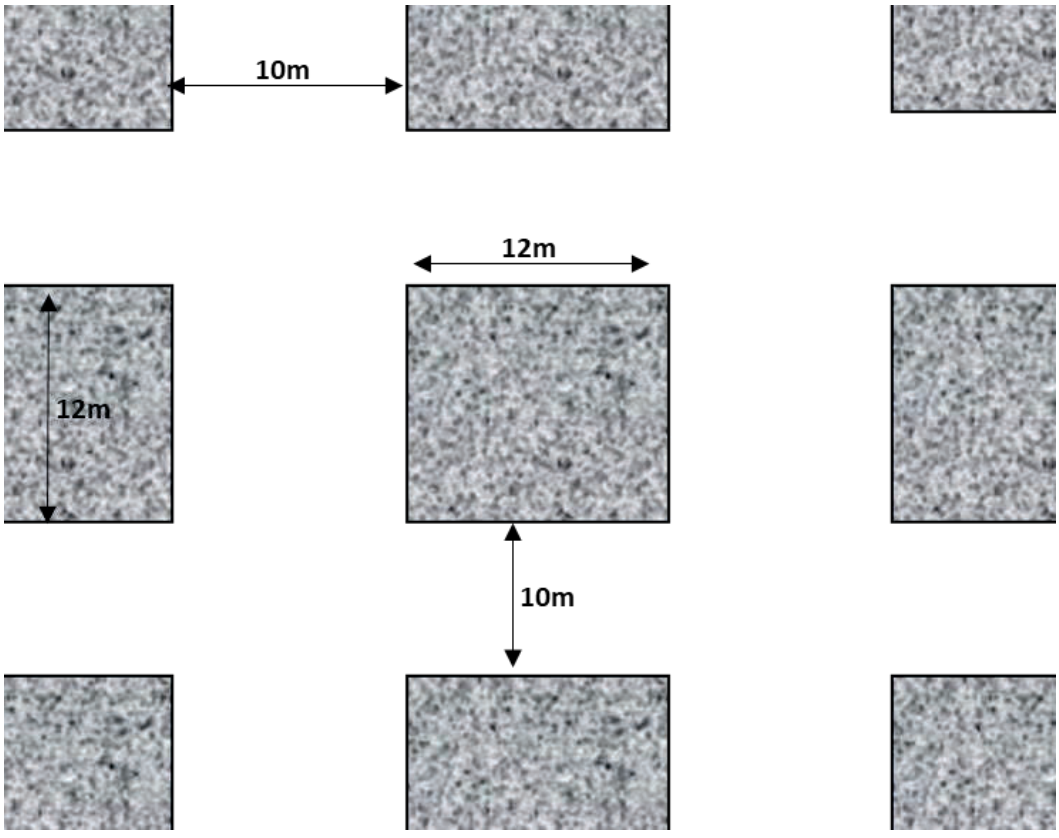


Figure 6.6: Mine working layout

Table 6.3: Window mapping data sheet template

JOINT MAPPING DATA SHEET							
Section	Panel name	Geological structure	Dip (deg)	Dip Direction (deg)	Average spacing (m)	Average trace length (m)	Comment

Joint mapping data collected using window mapping from both sections is provided in APPENDIX D.1 and APPENDIX D.2 respectively. GPR scans for mapped area are also provided in APPENDIX E.

6.4. Joint data analysis

In order to analyse the collected joint data, the first step was to group joints according to their orientation. This was achieved by using the DIPS 7.0 Rocscience software. Dips software is designed to interactively analyse geological discontinuities data orientation. The software also helps to visualise and analyse structural data in 2D using the same technique as that applied in manual stereonets (Rocscience, 2002). Joint orientation analysis is presented in sub-section 6.4.1. The second step was to analyse the joint length and joint spacing data. The results are detailed in sub-sections 6.4.2 and 6.4.3, respectively.

6.4.1. Joint orientation

Joint orientation is defined by the dip and dip direction angle. This joint property is regarded as one of the most important parameter since it neutralises the effect of other properties when oriented favourably (Gumede & Stacey, 2007). A graphic illustration of joint orientation results in the North section is given in Figure 6.7. Three major joint sets are identified. The dominant joint set (J1) is orientated at $67^0/107^0$, joint set 2 (J2) orientated at $83^0/23^0$ and

lastly (J3) flat dipping at $54^0/266^0$. Joint orientation results from the South section also show three joint sets (Figure 6.8). The set orientation for J1, J2 and J3 are $61^0/280^0$; $85^0/25^0$; and $47^0/114^0$ respectively. Although three joint sets were identified in each section, there is discrepancy in the dip and dip direction of these sets. These results support the general joint set data given in Table 6.1, showing that there are three main joint sets at the Chrome mine. Two of which are steeply dipping and one flat dipping.

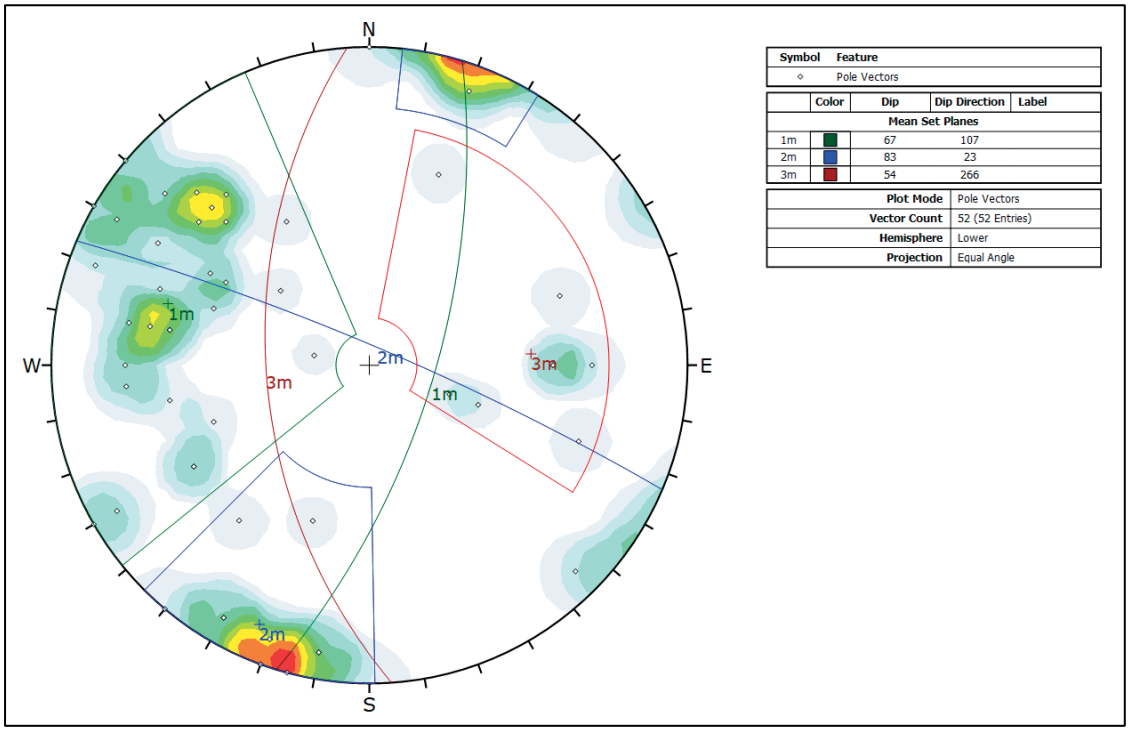


Figure 6.7: Overall joint orientation in the North Section

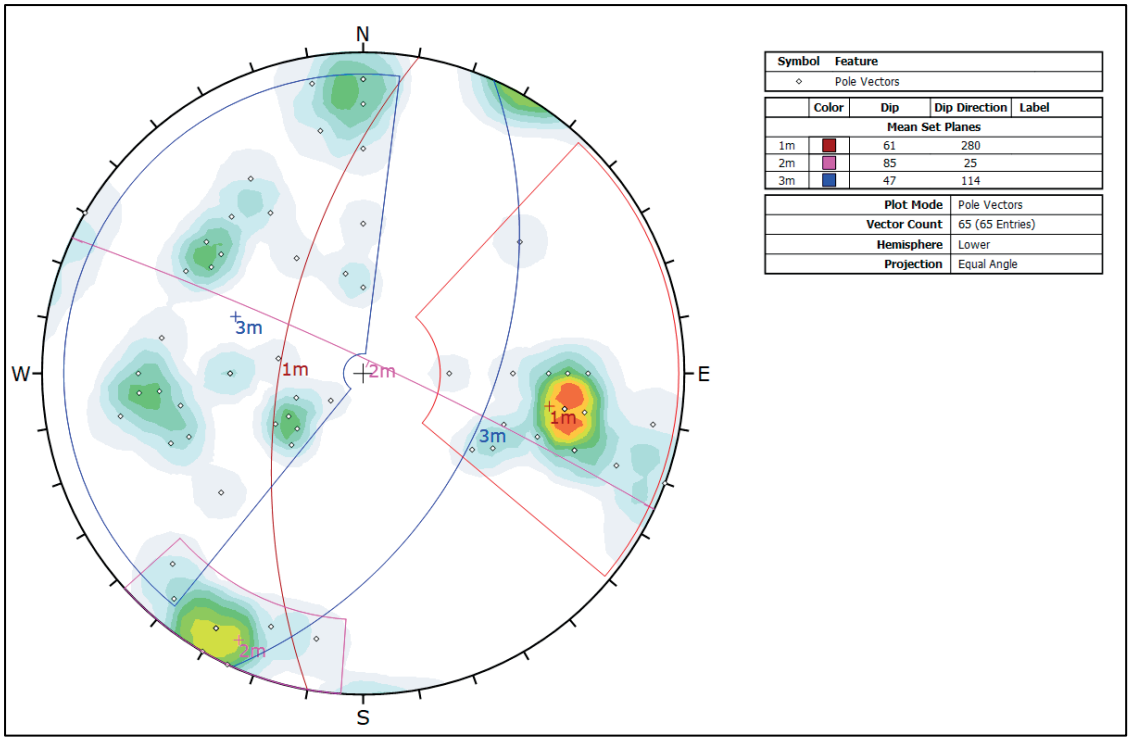


Figure 6.8: Overall joint orientation in the South Section

Overall joint orientation in Panel 74S (Figure 6.11) back area where scanline mapping technique was used to collect data shows that there are two dominant joint sets as it appears in Figure 6.9. Both sets are steeply dipping ($87^0/26^0$ and $75^0/165^0$). These structures alone do not pose FOG hazard. Visual observations alone would lead to a conclusion that the hanging wall in this panel is stable. Contrary to that, GPR scans conducted in the same area show that there is a continuous parting sub-horizontal plane between 2m to 3m in the hanging wall (Figure 6.10). It can be seen that the steeply dipping joints were not picked up by the scan, due to its incapacity to detect near-vertical structures. After confirming the existence of these structures using borehole camera inspection, FOG lights (4m trigger length) spaced not more than 5m apart, were installed to monitor the hanging wall stability.

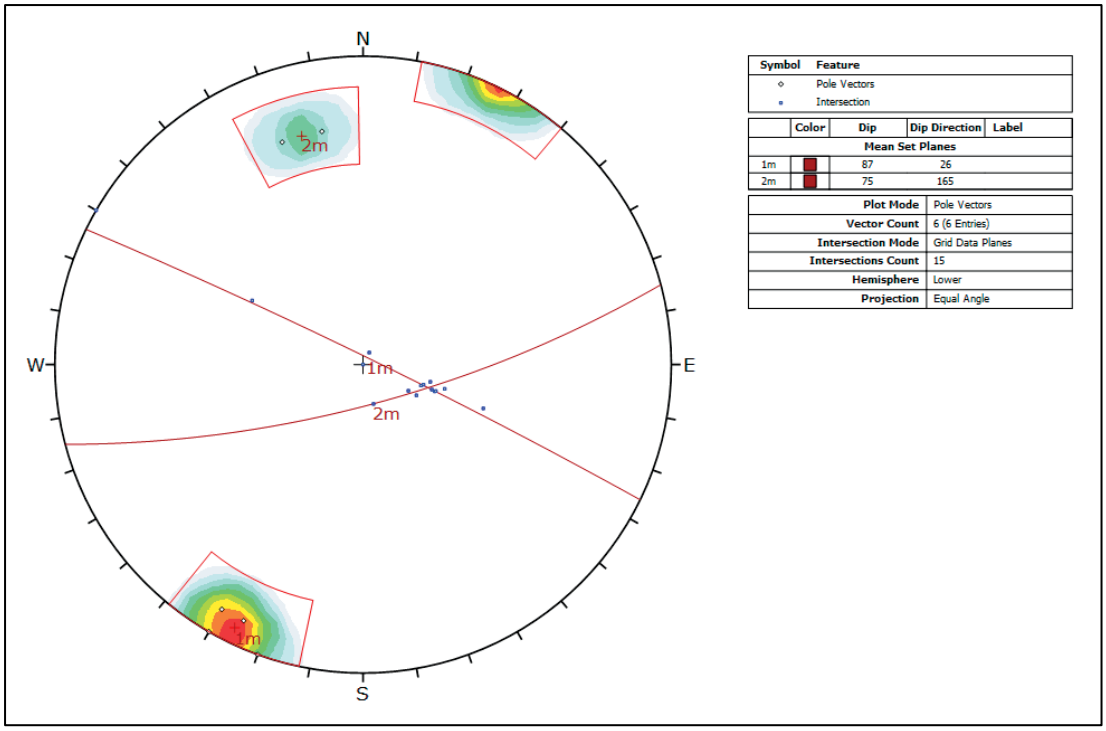


Figure 6.9: Overall joint orientation in Panel 74S back area

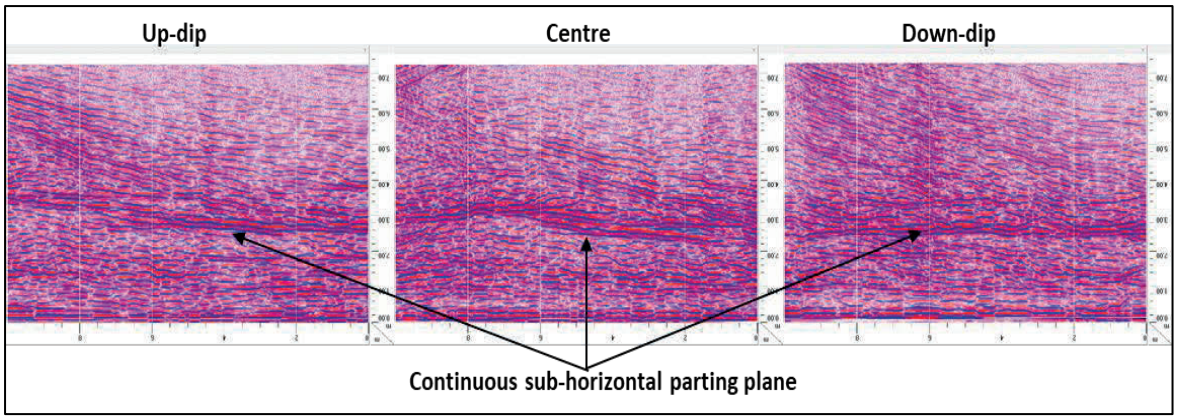


Figure 6.10: GPR scan at Panel 74S back area



Figure 6.11: FOG lights installed at Panel 74S back area

At times, visual observations may determine an excavation as stable while it is actually unstable as proved by GPR scans from Panel 74S (Figure 6.11). In contrast, GPR scans may detect that there are no major structures in the hanging wall whereas joint mapping proves otherwise. This may be due to the fact that GPR scanning only detects flat dipping structures. Therefore, it is advisable to utilise joint mapping techniques, coupled with visual observations and GPR scanning to ensure excavation stability.

6.4.2. Joint length

Joint lengths are known to define the possible failure plane in a rock mass and block size formed. The challenge with measuring joint length is that only the exposed length can be measured. It is therefore difficult to attain the actual joint length, thus approximations are recorded. Research work by Call et al. (1976) suggests that assessing joint lengths, minimum joint lengths are more defined as compared to maximum lengths since this joint property has exposure length limitations. Furthermore, a study by Bye & Bell (2001) suggests that prominent joints are generally continuous laterally. Microsoft excel was used to analyse joint trace length data for joint sets determined from Dips. An example of joint length data distribution is shown in Table 6.4. Joint trace data fits both lognormal and negative exponential distributions as shown in Figure 6.12.

Table 6.4: South Section Joint set 3 trace length cumulative frequency

Joint trace length limit (m)	Trace length (m)	Frequency	Cumulative frequency	% Cumulative frequency
0-2	2	1	1	100
2-4	4	2	3	100
4-8	8	13	16	99
8-12	12	15	31	95
12-16	16	4	35	91
16-20	20	2	37	90
>20	>20	0	37	89

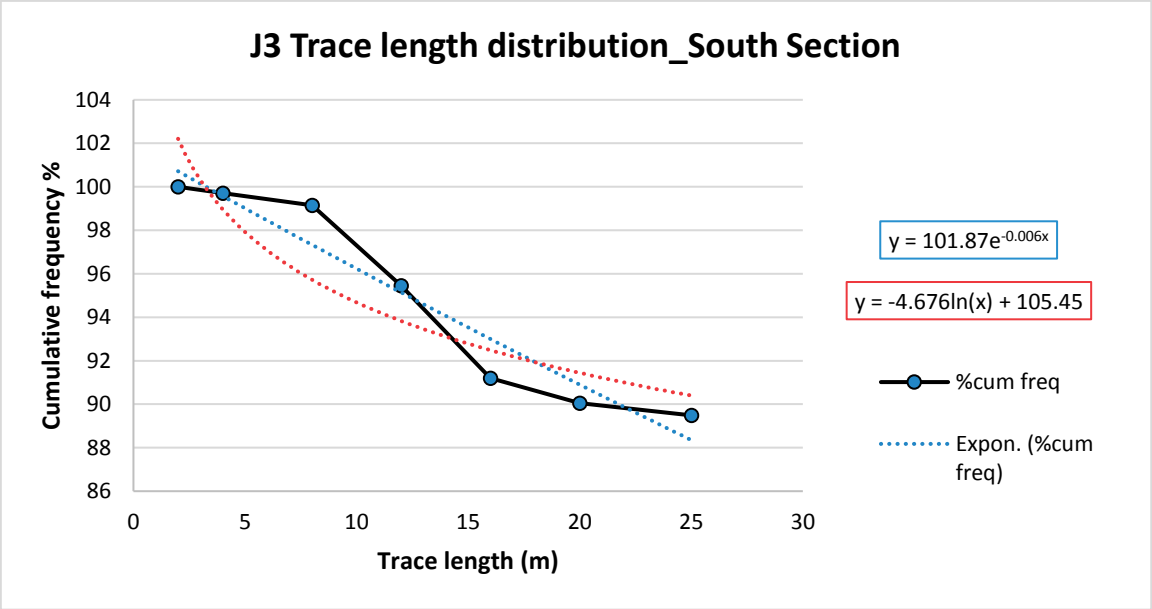


Figure 6.12: J3 trace length in the South Section showing lognormal distribution

6.4.3. Joint spacing

Joint spacing was measured underground as the perpendicular distance between joints in the same set as explained in sub-section 5.3.3. Since Dips was used to determine dominant joint sets, joint spacing data from a particular joint set defined in Dips was used to determine the mean joint spacing for that set. This was completed through excel data analysis. Examples of probability of joint spacing distribution are shown in Table 6.5 and Table 6.6. Joint spacing

data can be represented by both negative exponential distribution and lognormal distributions as seen in Figure 6.13 and Figure 6.14.

Table 6.5: North section Joint set 2 cumulative frequencies

Joint Spacing limit (m)	Joint Spacing (m)	Frequency	Cumulative frequency	% Cumulative frequency
0-1.5	1.5	1	1	100
1.5-2	2	4	5	96
2-3	3	4	9	80
3-4	4	1	10	63
>4	>4	0	10	59

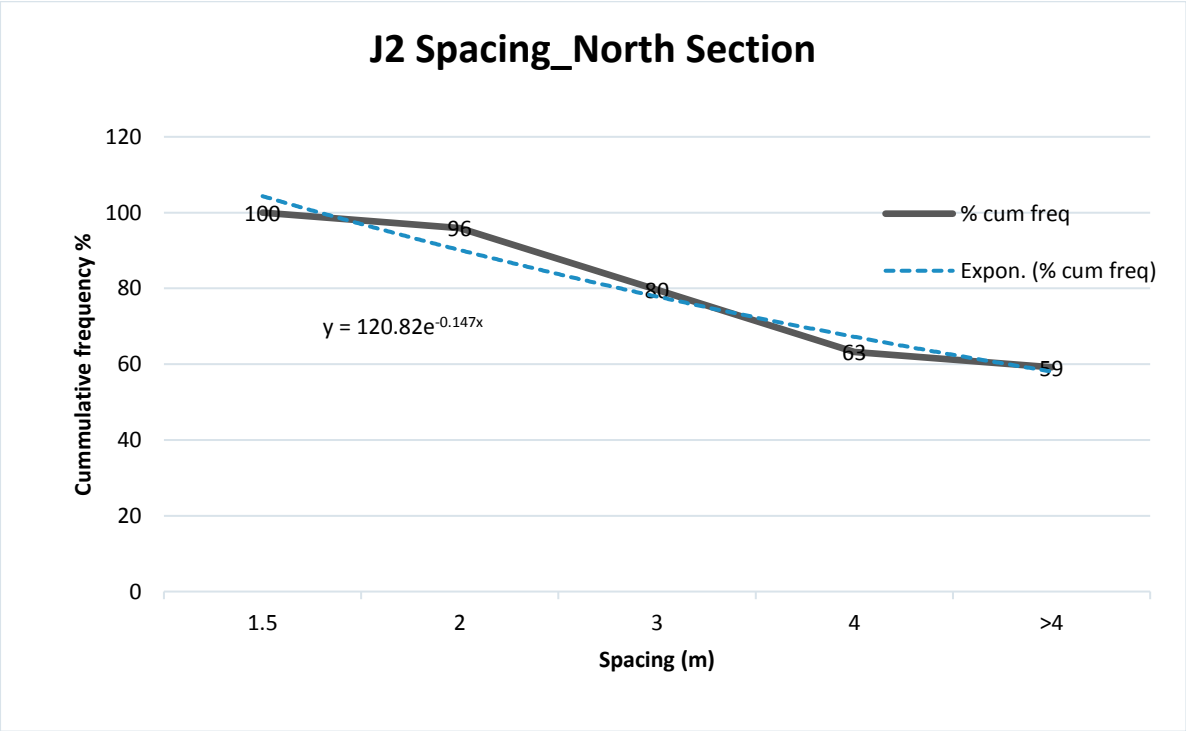


Figure 6.13: North section J2 spacing showing lognormal distribution

Table 6.6: South section Joint set 2 cumulative frequency

Joint spacing limit (m)	Joint Spacing (m)	Frequency	Cumulative frequency	% Cumulative frequency
0-1	1	4	4	100
1-2	2	9	13	95
2-3	3	13	26	84
3-4	4	1	27	68
4-5	5	5	32	67
>5	>5	0	32	61

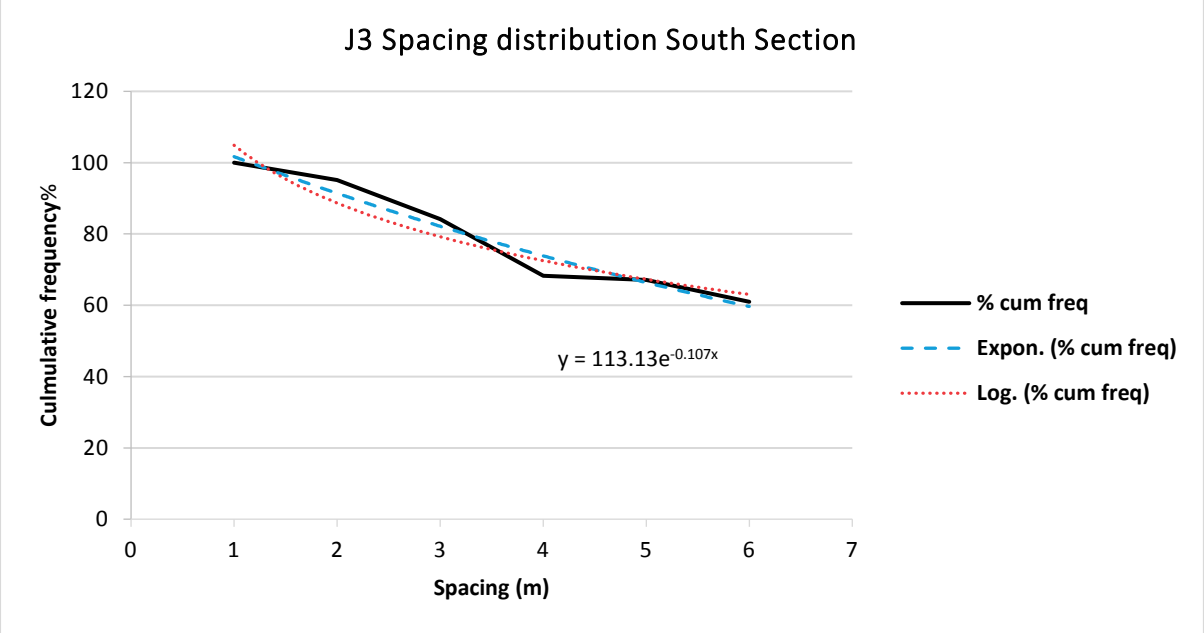


Figure 6.14: South Section J3 spacing showing negative exponential distribution

6.5. Summary and conclusions

After identifying the main geological discontinuities within the mine, joint mapping was conducted in order to gather more information relating to geometric properties of these discontinuities as they have potential to cause falls of ground. This chapter explained how underground joint data mapping was carried out using scanline mapping and window mapping. The data from both sections was processed and analysed, comparing the two sections. Bases on joint data analysis above, it can be concluded that both joint length and joint spacing data can be distributed in two ways i.e. lognormal distribution and negative

exponential distribution as outlined in preceding chapter. Overall, joint data analysis results are summarized in Table 6.7. The summarized data can now be used to predict rock fall probability together with the evaluation of support. The next chapter deals specifically with this aspect.

Table 6.7: Overall joint data for the two sections

Section	Joint set No	Dip (deg)	Dip Direction (deg)	Range (deg)	Spacing (m)			Length (m)		
					Mean	Min	Max	Mean	Min	Max
North	J1	67	107	10	2.5	0.2	6	13	5	20
	J2	83	23	10	2.5	1.5	4	16	10	20
	J3	54	266	10	2.4	1	4	14	5	22
South	J1	61	280	10	2.9	0.8	7	8.3	2	5
	J2	85	25	10	3	1	5	10	1.5	20
	J3	47	114	10	2.6	0.5	5	9.5	2	20

CHAPTER 7: USING JOINT DATA TO PREDICT ROCK FALL PROBABILITY

7.1. Introduction

Excavation stability is influenced by the presence of geological structures. Preceding Chapters outlined geological structures that characterise the study area, where underground mapping was conducted to collect joint properties influencing excavation stability. This chapter aims at using the collected joint data to evaluate rock fall probability. Statistical data is required in order to interpret results in a probabilistic manner. For the evaluation of rock fall probability in a jointed rock mass, usually computer programmes are used to simulate key blocks of rock with potential to fall under the influence of gravity. These blocks are tested against support, to see if they will remain stable or fail and/or if the support resistance is adequate to stabilise the blocks. The process of simulating blocks is repeated multiple times to obtain statistical results (Esterhuizen & Streuders, 1998). This approach can be considered where there is a need to improve support design for jointed rock mass.

Section 7.2 explains the approach whereby joint data is used to model probabilistic key blocks in an excavation. The approach explained in Section 7.2 is used to evaluate rock fall probability for the North and South sections, results are presented in Section 7.3. Summary and conclusions are provided at the end of the Chapter.

7.2. Jointed rock mass stability analysis

Underground mining is prone to rock fall hazards. Unless measures are put in place, fall of ground incidents will always be experienced. There are various ways in which support design for jointed rock mass is approached. These approaches according to (Dunn et al., 2008) can be ‘rules of thumb’, rock mass classification systems i.e. the Q system by (Barton et al., 1974) or Rock Mass Rating by (Bieniawski, 1973) or conventional wedge analysis from (Goodman & Shi, 1985). The approach used will depend on the nature of discontinuities in a rock mass as it determines the size and shape of key blocks formed where there is interaction. Support systems are designed to stabilise as many key blocks as possible (Esterhuizen & Streuders, 1998).

Probabilistic methods are mostly preferred lately, as they require fewer assumptions in support designs because the method estimates potential key block sizes and their frequency as

part of support design using joint data (Dunn et al., 2008). According to Gumede (2006), jointed rock mass stability analyses are categorised into three models, which are:

- Numerical model (differential and integral);
- Analytical model (stereonets and closed form solutions); and
- Probabilistic model (stereonets and block theory)

Probabilistic modelling was found to be suitable for stability analysis in this study since joint data analysis in the previous chapter was demonstrated to follow a statistical distribution. There are numerous modelling software, which utilise probability principles to model excavation stability; J-Block is one of them as explained below:

J-Block is a well-known rock engineering software designed to evaluate the probability of potential rock falls in mining excavations. Underground mapping of individual stress fractures and joints is not completely practical. Owing to this limitation, the software simulates blocks in the hanging wall formed by jointing. The simulation is completed by means of statistical methods in order to identify potential unstable key blocks (Esterhuizen, 2003). After key blocks have been identified, they are placed at random positions within a predefined excavation with and without support. This is practiced to comprehend possible failure modes and for support system evaluation. Depending on the block size and weight, failure may occur between support units. However, in cases where the block is located over support units, and its weight greater than the support strength, failure will occur.

Results from J-Block are presented in the form of histograms where the probabilities of failure of different key block sizes are shown. In addition, the software also gives results showing the density of falls in a particular area, referred to as 'area of interest'. These results can be used to optimise support spacing as it indicates the area prone to high frequency of rock falls. The same results can also be used to identify hazardous areas linked to the support system in that excavation (Esterhuizen, 2003). J-Block only simulates one surface at a time, thus corners are not simulated as they are made of more than one surface (Grenon & Hadjigeorgiou, 2003).

7.3. Rock fall probability evaluation at the Chrome mine

Multiple block analysis was conducted using joint data results summarised in Table 6.7. For J-Block stability analysis in both sections, the following parameters were used:

- *Excavation geometry*: rectangular excavation where 10m face x 22m (centre-to-centre) back area defined the area of interest.
- *Stress*: At 500m below surface, virgin vertical stress is 16MPa whereas the corresponding horizontal stress is 24MPa assuming K ratio of 1.5 (density of pyroxenite = 3200kg/m^3)
- *Support*: Current support is 1.5m long roof bolts, with 110kN capacity. Support pattern is 2m x 2m with 1m offsets.

Multiple block analysis using input data (Figure 7.1 and Figure 7.2) for both sections was run for 10000 key block simulations. Results obtained from the simulations include key block size distribution, distribution of failure probabilities, failure and stability modes. In addition, area and support layout analysis were also carried out.

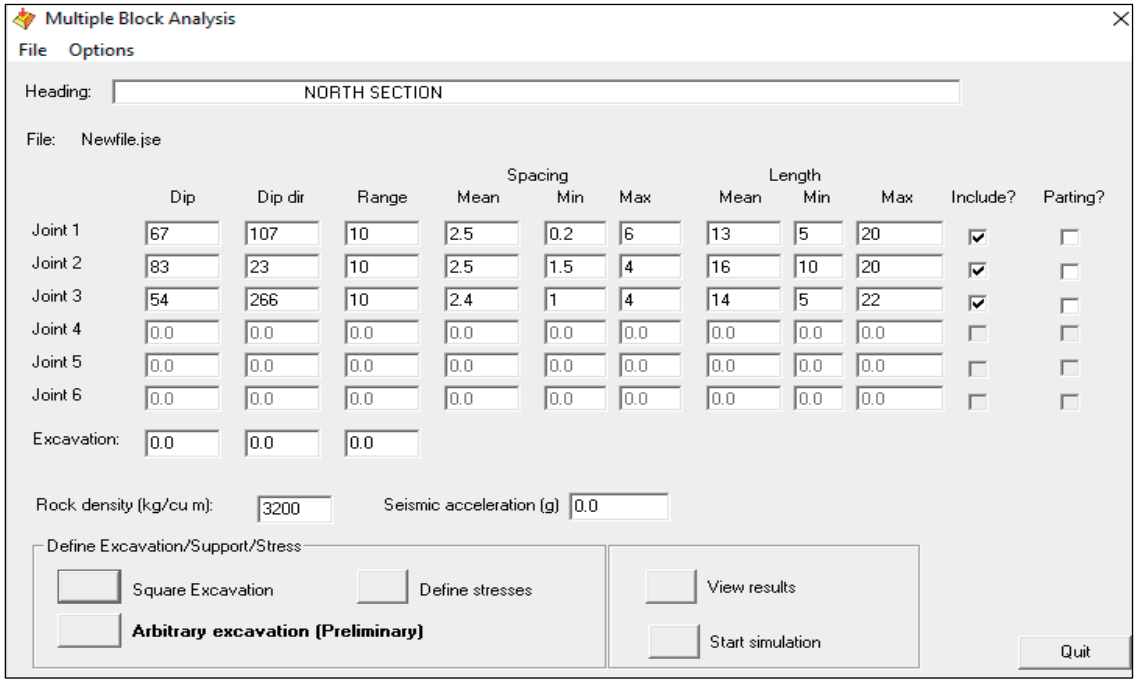


Figure 7.1: North section data input window in J-Block

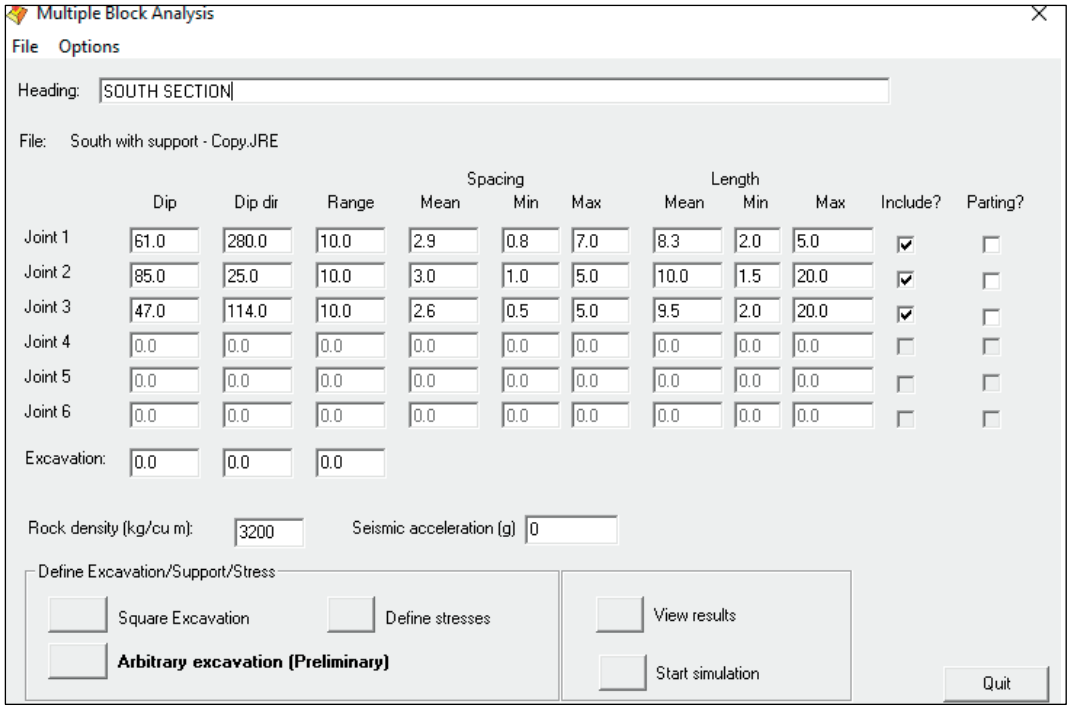


Figure 7.2: South section data input window in J-Block

7.3.1. Key block size distribution

Simulation of joint sets in the North section show that 61% of key blocks formed are less than one cubic-meter. The key block size distribution probability ranges between 1% and 9% as the block size increases from $2m^3$ to $15m^3$. The probability of $1m^3$ blocks failing in-between support is 6.3%. For blocks greater than $1m^3$ the probability decreases, but the probability that support will fail increases (Figure 7.3). Probability that support will fail as block size increases ranges between 1% and 12%.

As shown in Figure 7.4, key block size distribution in the South section show that 72% of blocks formed are less than $1m^3$. As the block size increases, the key block size distribution also decreases. The probability of $1m^3$ blocks failing in-between support is 77%. As the block size increases, the probability of blocks failing between supports decreases. However, the probability of blocks greater than $1m^3$ failing due to support failure increases (6% to 18%).

Failure probabilities in the North section are less than those in the South. This is because of varying joint set orientation. Although both sections have three joint sets, the south joint sets are more flat than those of North. This means that joint geometries in the south section have potential to form more key bocks.

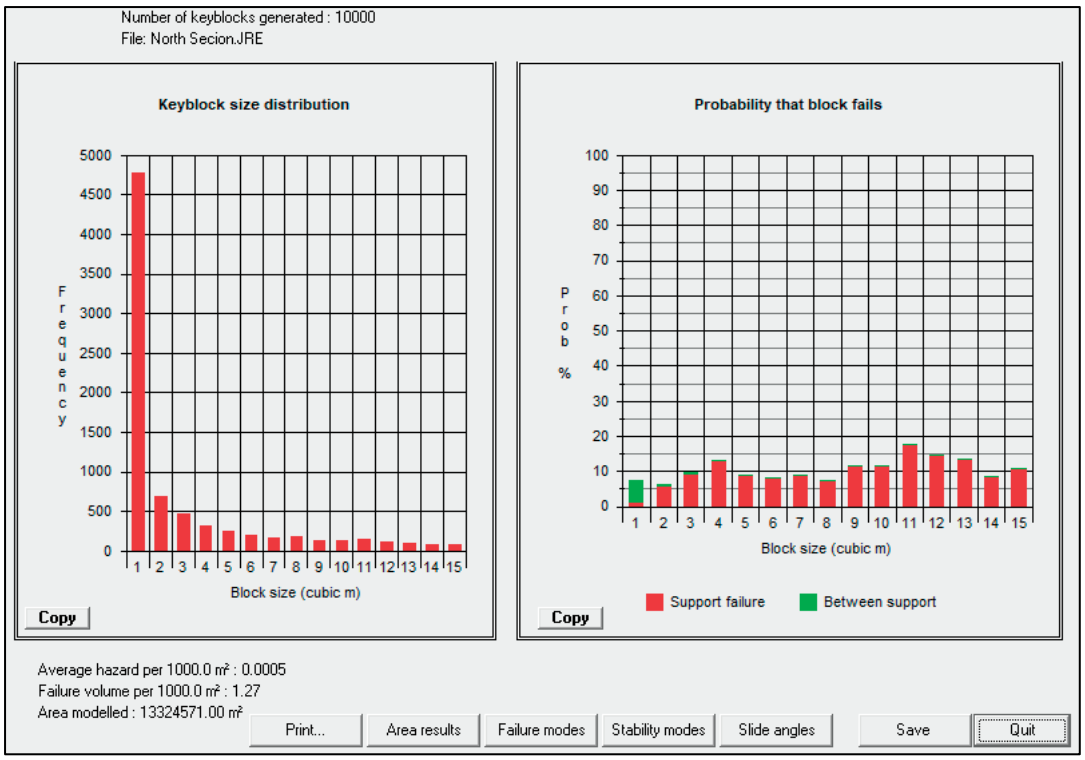


Figure 7.3: North section key block size distribution

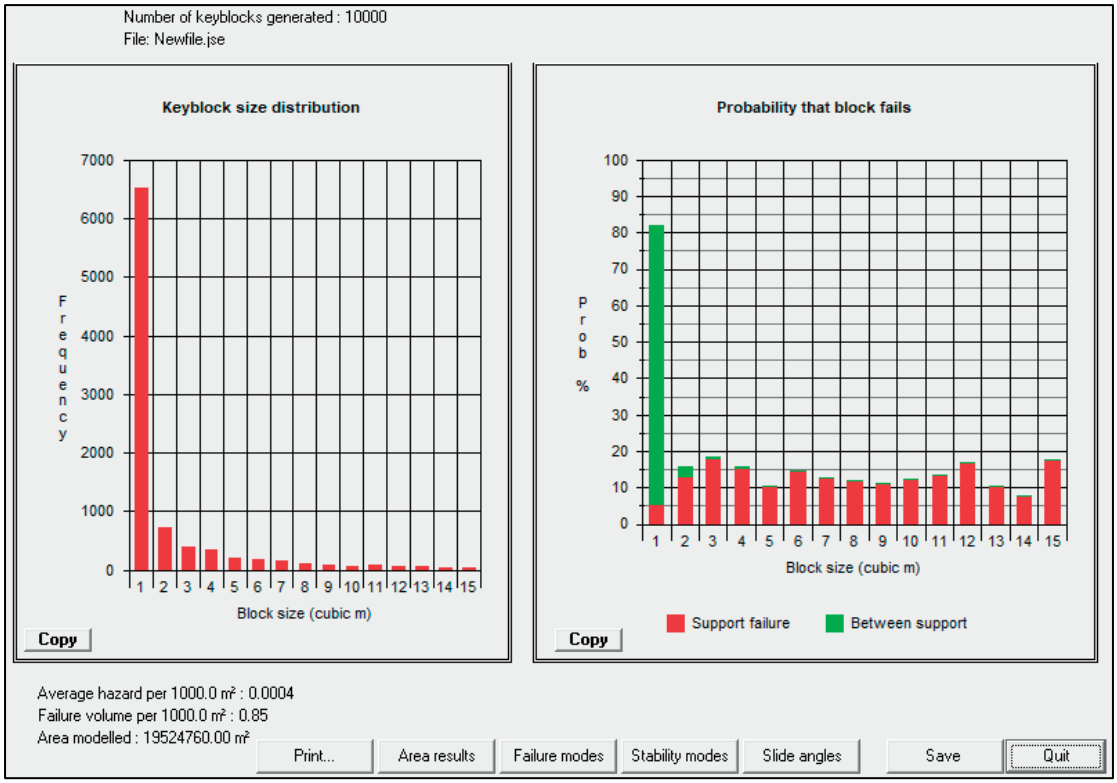


Figure 7.4: South section key block size distribution

Area outcome distributions shows that most block failure are localised in the centre of excavation (Figure 7.5 and Figure 7.6). This may not completely be the case as J-Block simulates one surface at a time. Since edges (blue) consist of more than one surface, corners were not considered in the simulation. J-Block outcome show that there is 8.7% and 26.6% fall percentage for North and South sections respectively. Based on the fall frequency distribution, the South section (198 blocks) is most likely to experience rock falls when compared to the North (124 blocks).

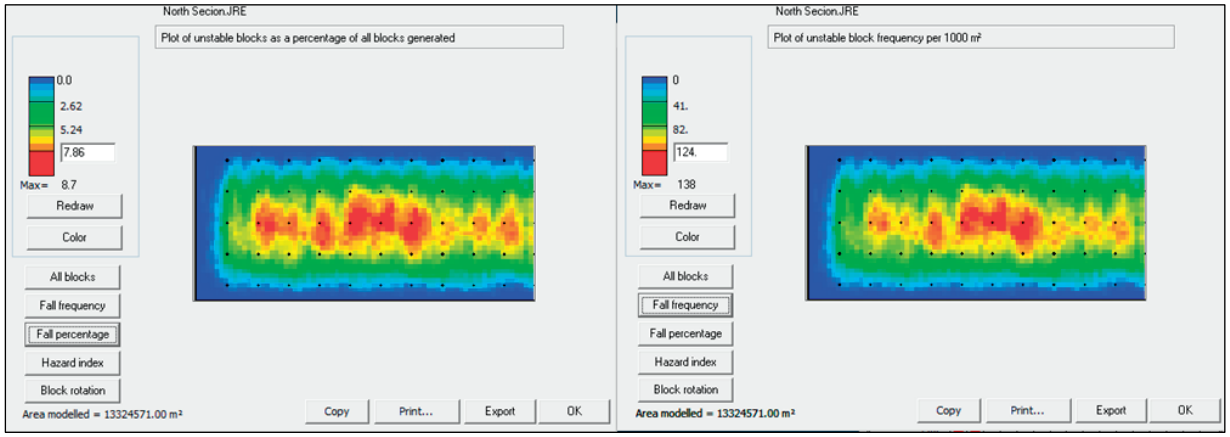


Figure 7.5: North section area modelled

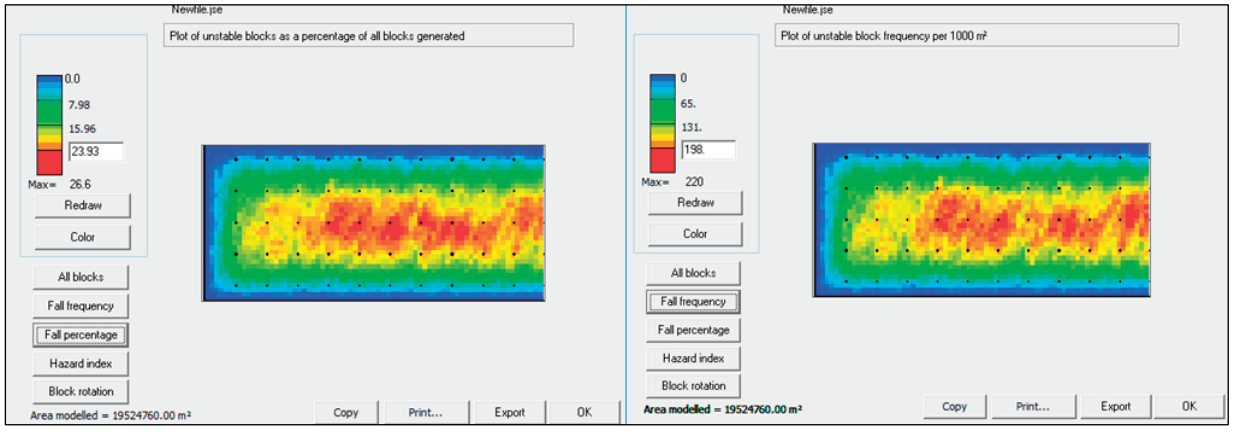


Figure 7.6: South section area modelled

7.3.2. Block failure modes and stability modes

Rock failure modes detected by J-Block include single pane, double plane, drop out and rotation. In the North section, there is 93% probability of $1m^3$ key blocks to fail by drop out. This is because small blocks are likely to fall between support units. As key block size increases, rotation failure mode probabilities also increase (53% to 100%) as shown in Figure 7.7. The South section is more likely to experience various failure modes, supporting higher failure probabilities given in sub-section 7.3.1. Drop out failure probability is high (80%) for $1m^3$ block sizes, and decreasing with increasing key block size (47% to 7%). As shown in Figure 7.8, single plane failure mode is more likely to occur due to sliding along 50^0 dipping planes. On the other hand, sliding planes for key blocks in the North section are steeper, ranging between 60^0 and 80^0 (Figure 7.9). The larger the block size, the more likely is to fail by rotation (0% to 59%).

Stability mode results (Figure 7.10) from both sections are similar. All block sizes are stabilised by friction as opposed to support. However, there is 48.95% and 57.48% that tendons are too short for North and South sections respectively.

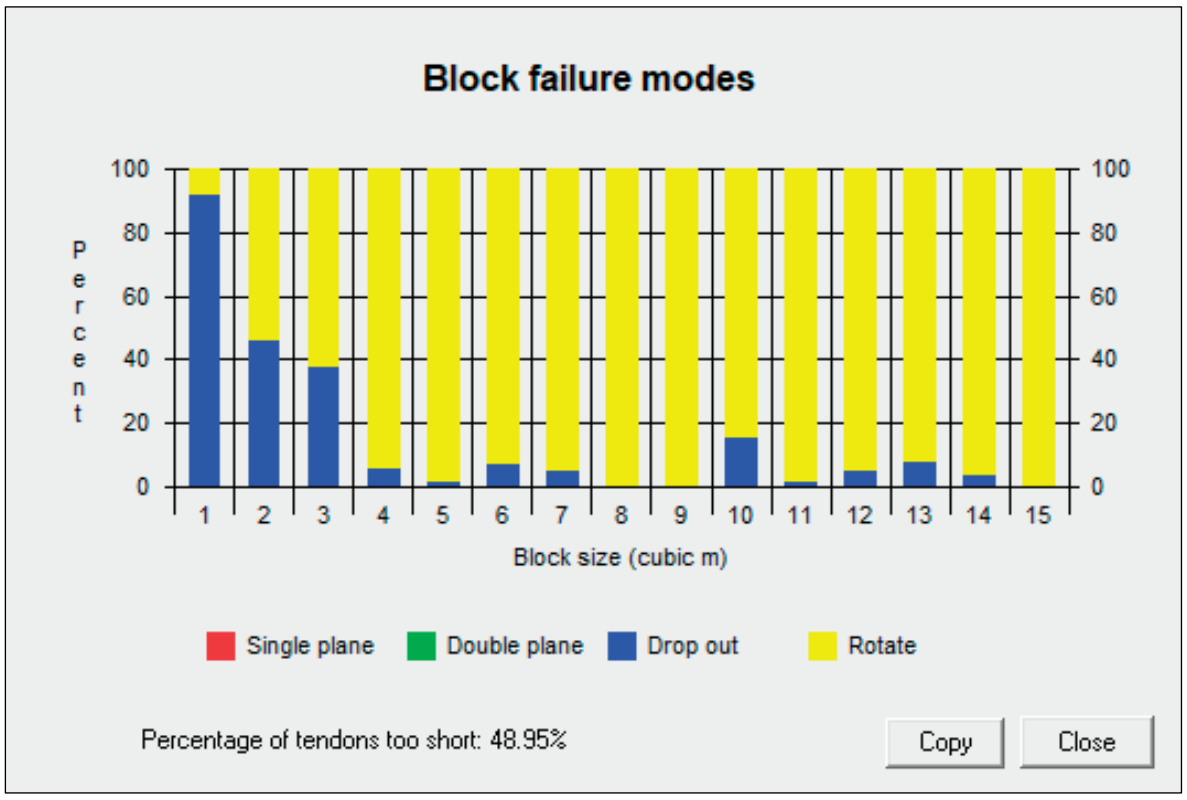


Figure 7.7: North section block failure modes

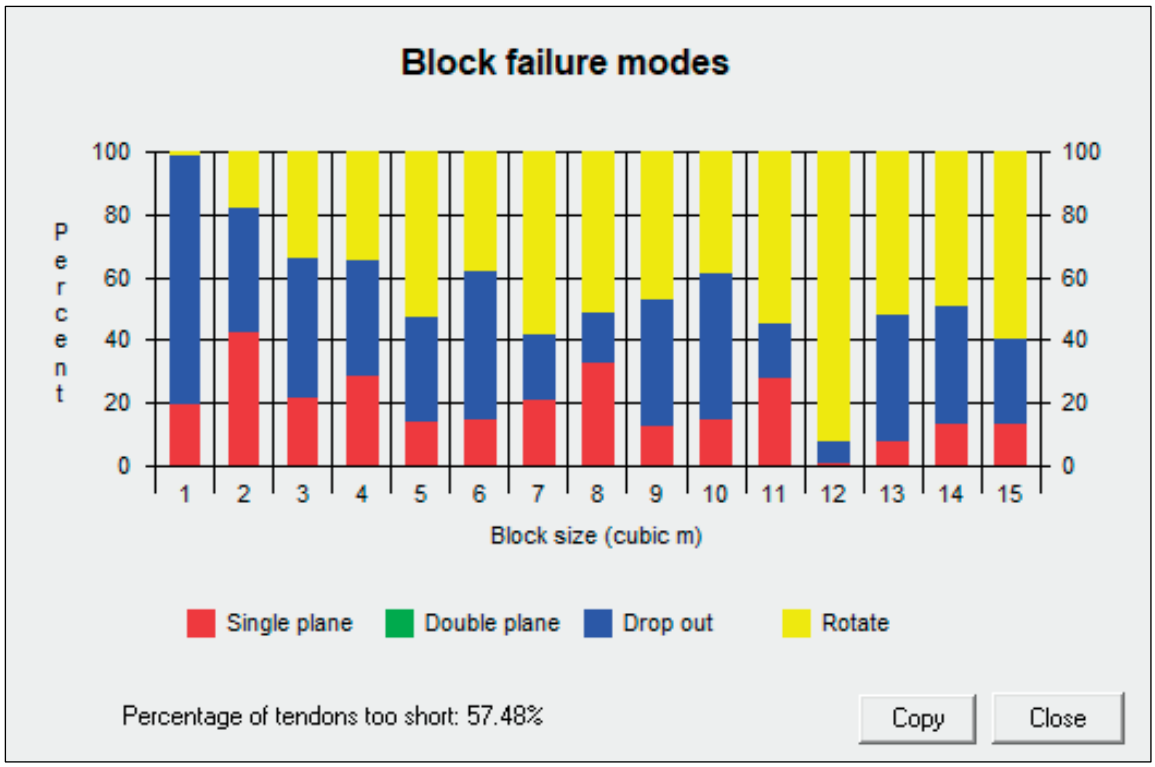


Figure 7.8: South section block failure modes

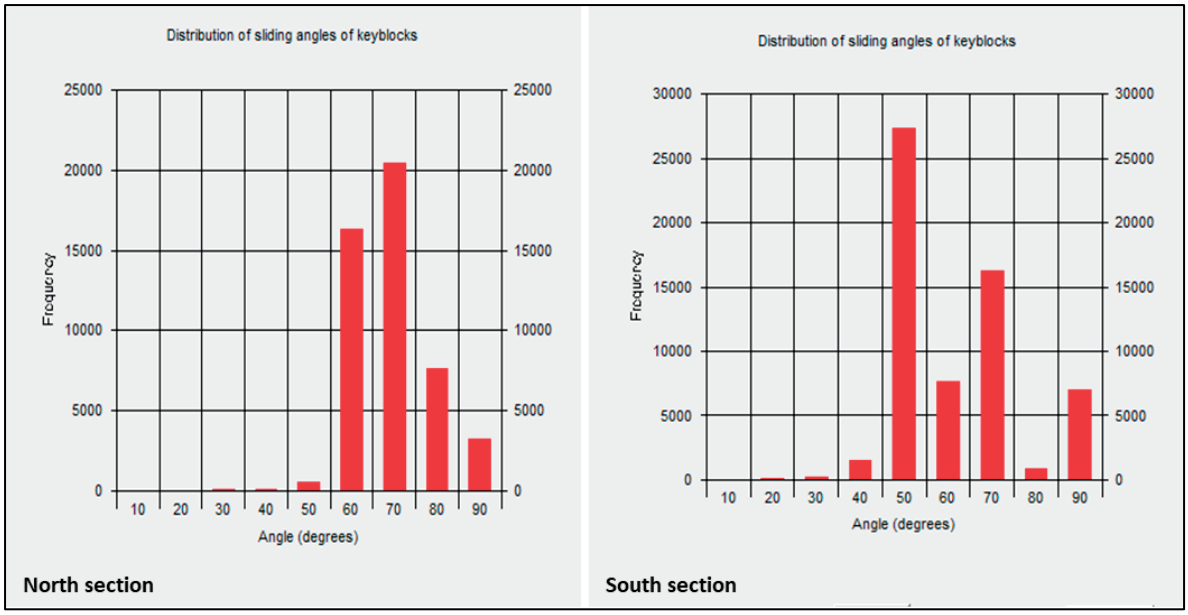


Figure 7.9: Distribution of sliding angles of key blocks

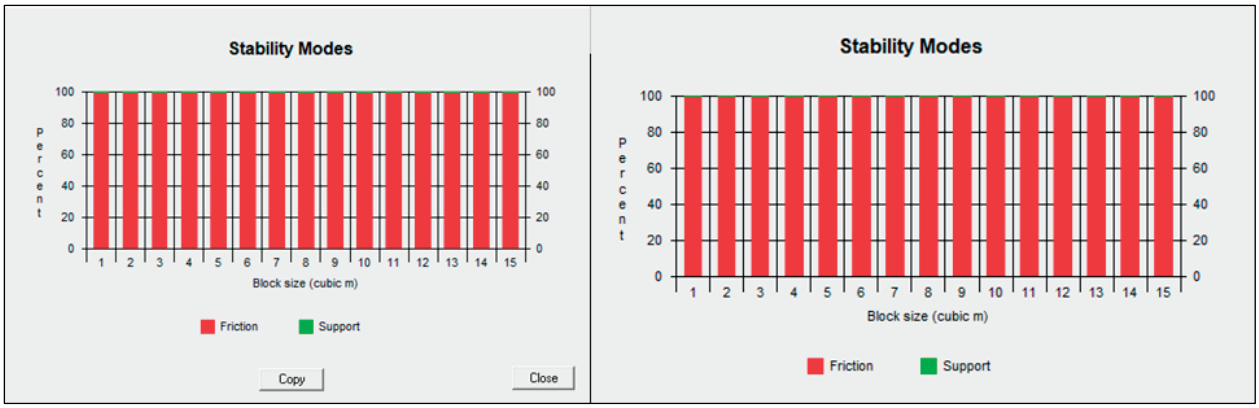


Figure 7.10: North and South section stability modes

7.3.3. Rock fall probability analysis for up-dip holing

Both North and South input data were rerun to simulate rock fall probabilities when mining up-dip holing since this is a bord and pillar operation. Results are presented in APPENDIX F. The outcome for both sections shows minimal variability in terms of probability of key block size distribution and probability that blocks fail. $1m^3$ blocks are more likely to fail in-between support units. Block size increases with the probability that support will fail. Comparing the two sections, probability of rock failing is greater in the South section. Larger blocks result in support failure, and are associated with rotation failure mode.

Area results indicate high fall percentages at the centre of excavation, 11.6% for North and 28.1% for South. Relating the two mining orientations, up-dip is most likely to experience higher fall frequencies. When mining up-dip, block failure mode results indicate that the probability that support tendons are short is slightly lower than when mining on strike. Overall, block failure modes experienced in each section when mining up-dip versus mining on strike shows little or no difference.

7.4. Support layout analysis

Installed support improves safety and stability in excavations. Support layout and capacity will influence excavation stability. If the weight of generated key blocks exceeds the support capacity, failure is deemed to take place. Therefore, support systems employed must be adequate enough to stabilise the excavation. Analyses conducted in preceding sub-sections

showed that failure is most likely to occur in-between support units for $1m^3$ key blocks. This means that support spacing significantly affects stability.

Table 7.1 summarises the effect of support spacing on $1m^3$ block size failure probability. The same support units (non-grouted roof bolts 1.5m long @ 110kN) were evaluated over three different support spacing layouts. Decreasing support spacing decreases the probability that small key blocks will fall between supports. Nevertheless, reducing support spacing also increases support failure probability. Support failure may occur if the support resistance is smaller than the required resistance. To remedy the probability of support units being too short as explained in sub-section 7.3.2, models were also run with varying support lengths and capacity keeping a constant spacing of 2m x 2m Table 7.2. Based on these results, it can be concluded that the probability of support failure decreases with increasing support capacity for the same support length and spacing (Figure 7.11).

Table 7.1: Effect of support spacing on probability of failure

Support spacing (m)	2.0 x 2.0		1.5 x 1.5		1.0 x 1.0	
Failure type and Block size (m3)	% Prob of Support failure	% Prob of fall between support	% Prob of Support failure	% Prob of fall between support	% Prob of Support failure	% Prob of fall between support
1	1.3	6.3	1.5	4.9	2.1	4.1
2	5.9	0.3	8.4	0.1	2.8	0.0
3	9.5	0.2	12.5	0.0	5.2	0.0
4	13.1	0.0	13.6	0.0	9.6	0.0
5	8.9	0.0	18.6	0.0	6.8	0.0
6	8.2	0.0	23.2	0.0	19.4	0.0
7	8.8	0.0	19.5	0.0	17.9	0.0
8	7.4	0.0	23.4	0.0	26.7	0.0
9	11.7	0.0	18.6	0.0	13.1	0.0
10	11.6	0.0	25.0	0.0	30.7	0.0
11	17.7	0.0	18.3	0.0	38.1	0.0
12	14.8	0.0	27.8	0.0	30.0	0.0
13	13.7	0.0	8.8	0.0	22.3	0.0
14	8.7	0.0	23.0	0.0	25.5	0.0
15	11.0	0.0	21.6	0.0	28.8	0.0

Table 7.2: Effect of support length and support capacity on stability

Effect of Support length and Capacity on 2m x 2m constant spacing									
Capacity (KN)	110			150			200		
Support length (m)	1.5	2	2.5	1.5	2	2.5	1.5	2	2.5
Block size (m3)	% prob of support failure			% prob of support failure			% prob of support failure		
1	1.3	1.1	1.2	1.1	1.2	1.3	1.1	1.2	1.0
2	5.9	6.2	6.3	5.4	5.3	6.1	3.4	4.7	5.0
3	9.5	10.7	9.3	8.7	10.4	10.0	5.8	8.3	8.0
4	13.1	12.3	8.7	10.4	11.2	12.5	7.7	7.3	8.8
5	8.9	11.0	8.7	11.9	10.2	6.4	11.2	8.1	9.7
6	8.2	9.2	15.8	9.8	7.7	7.6	10.6	11.0	7.8
7	8.8	14.3	11.7	11.9	18.6	15.2	10.9	7.6	13.1
8	7.4	10.5	16.4	12.4	13.1	12.3	12.9	6.0	13.5
9	11.7	10.9	16.6	11.8	11.8	10.6	9.8	13.3	11.1
10	11.6	13.6	23.1	15.4	16.3	18.3	8.9	16.2	8.0
11	17.7	12.0	17.0	8.5	14.6	13.9	13.8	12.2	12.6
12	14.8	6.9	10.5	9.4	4.5	9.0	6.8	7.2	10.0
13	13.7	18.2	12.9	9.6	15.1	11.6	11.9	11.7	22.2
14	8.7	14.1	24.3	9.2	12.2	16.2	14.4	11.4	14.1
15	11.0	12.8	16.8	9.6	10.8	8.0	19.3	7.4	8.9

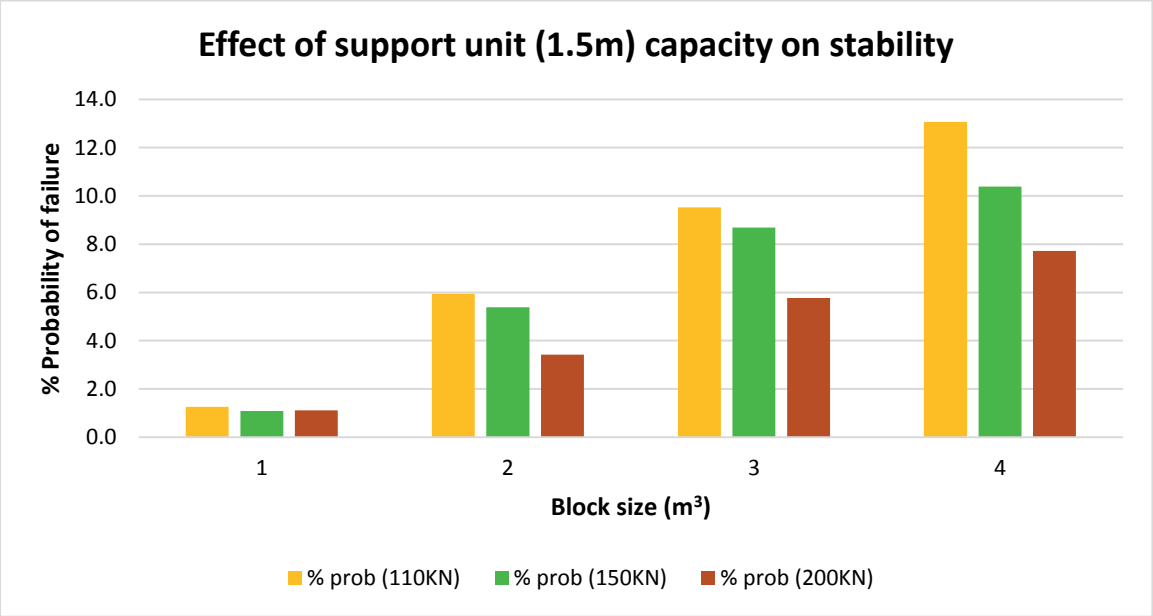


Figure 7.11: Effect of support unit capacity on stability

Introducing areal support in the South section reduced probability of small blocks falling between supports. The effect of areal support on stability is summarised in Table 7.3.

Table 7.3: Effect of areal support on stability

AREAL SUPPORT (500 kN)								
Width (m) x Length (m)	2 x 2		1.5 x 1.5		1 x 1		0.5 x 0.5	
Block sizes (m3)	% prob Support failure	% prob fall between supports	% prob Support failure	% prob fall between supports	% prob Support failure	% prob fall between supports	% prob Support failure	% prob fall between supports
1	0.1	11.9	0.1	5.7	0.0	4.1	0.0	1.7
2	1.1	0.1	0.7	0.0	0.4	0.0	0.1	0.0
3	1.5	0.0	0.6	0.0	0.6	0.0	0.1	0.0
4	1.8	0.0	1.5	0.0	0.9	0.0	0.5	0.0
5	2.2	0.0	1.9	0.0	1.0	0.0	0.2	0.0
6	1.6	0.0	1.4	0.0	1.4	0.0	0.3	0.0
7	3.1	0.0	1.8	0.0	1.6	0.0	0.6	0.0
8	2.1	0.0	1.2	0.0	1.5	0.0	0.0	0.0
9	2.4	0.0	2.8	0.0	0.8	0.0	0.7	0.0
10	3.5	0.0	1.5	0.0	1.2	0.0	0.5	0.0
11	3.0	0.0	2.7	0.0	0.0	0.0	0.0	0.0
12	2.8	0.0	5.6	0.0	0.0	0.0	0.0	0.0
13	6.4	0.0	3.4	0.0	0.0	0.0	0.0	0.0
14	5.0	0.0	2.0	0.0	0.6	0.0	0.6	0.0
15	1.1	0.0	3.8	0.0	0.7	0.0	0.0	0.0

7.5. Summary and conclusions

The use of joint data to predict rock fall probability has been demonstrated using J-Block software in this chapter. Probability of key block size formed confirms with the overall fallout thickness evaluated in section 4.3, whereby 0.95m was found to be the average fallout thickness throughout the mine. Rock fall probabilistic analysis carried out in this chapter indicate that about 80% of all key blocks formed are $1m^3$ in size. Results show that larger blocks are more likely to fail by means of rotation. Furthermore, to prevent small blocks from falling between support units, areal coverage (wire mesh, Oslo straps, shotcrete etc.) is suggested in heavily jointed rock masses. In conclusion, probabilistic analysis can be used to evaluate probability of rock falls, and support design for stability enhancement. The next chapter discussed relative improvements that can be used to help combat FOGs at the Chrome mine.

CHAPTER 8: DISCUSSION ON RELEVANT IMPROVEMENTS TO CAMBAT FOGs AT THE MINE

8.1. Introduction

Unexpected rock mass movement may lead to uncontrolled release of rocks that can result into injuries and fatalities. Unfortunately, the natural environment in which the mine is located in cannot be changed (Yilmaz, 2015). However, rock engineering is there to ensure that mining is carried out profitably and safely. The stability of underground excavations is a key concern in this field. As a result, focus is mainly on stability enhancement by means of excavation support designs and monitoring of ground conditions. Mining practices have an influence in the behaviour of rock masses. This is because whenever an excavation is constructed in a particular rock mass, stresses are redistributed around the newly created excavation. Owing to these changes, risk assessments and close monitoring of the exposed rock mass are considered to be essential (Walke & Yerpude, 2015).

This chapter addresses relevant improvement in the current falls of ground management system at the Chrome mine. Since the main objective for conducting this study was to come up with a sound systematic approach to help minimise and control falls of ground at the Chrome mine, this chapter is deemed essential. Section 8.2 discusses the main challenges presented by the current falls of ground management system and how they can be improved. Each aspect of the system is looked at individually in this section. A summary and conclusions of the relevant improvements to combat FOGs at the Chrome mine are given at the end of the chapter.

8.2. Main challenges and how they can be improved

Falls of ground management systems are designed to maintain safe working environment by enhancing stability. At the Chrome mine, the falls of ground management system comprises of visual observations and the use of ground monitoring equipment. These equipment include ground penetrating radar (GPR), borehole camera and fall of ground light (FOG Light). A detailed discussion on each of these aspects is presented in chapter 3 (falls of ground management strategy at the area of study) of this report. A concern arose after falls of ground continued to occur despite the system been put in place. The main challenges, which are

identified within each aspect of the current FOG management system, are discussed in the following subsections.

8.2.1. Visual observations

Visual observations are conducted by means of panel risk ratings (PRR) wherein panels are rated based on various factors with potential to induce falls of ground. Rock engineering personnel conduct panel risk rating procedures as they are trained and better equipped in this field. The exercise involves classifying working panels in terms of their percentage compliance, judging by the adverse geological structures, general ground condition, mining practices and support installation. Nevertheless, this study showed that there is difficulty in identifying and supporting geological structures respectively, encountered by mining personnel. In addition, early entry examinations are conducted inadequately. This means that there is lack of knowledge and appropriate skill in terms of making the work area safe. Hence the PRRs comprises of hazards and risks that can be avoided if early entry examination is conducted efficiently.

Prominent hazards identified during panel risk ratings are:

- Intersection of flat dipping joints and dome structures;
- Prominent joints and faults resulting in blocky ground conditions;
- Loose blocks of rocks in proximity to geological structures;
- Primary support (camp-lock jack and safety nets) not installed adequately;
- Chrome left on the hanging wall;
- Pegmatoid identified on the hanging wall posing FOG hazard as there is less cohesion between pegmatoid and pyroxenite;
- Geological structures not demarcated and supported to mine standard;
- Damaged, missing and or protruding roof bolts;
- Roof bolts not correctly tensioned since load indicator had not collapsed (curled);
- Roof bolts installed at shallow angles (less than 70 degrees); and
- Sockets identified in the hanging wall beam (damage from excessive drilling and blasting).

The above listed hazards can be addressed through:

- Demarcation and adequate support installation of identified geological structures;
- Continuous barring of loose blocks of rocks along geological structures;
- Adequate installation of primary support (camp-lock jacks fitted with a remote release sling, and nets tightly fastened with s-hooks)
- Ensuring no chrome is left on the hanging wall by either barring down or blasting it using short holes;
- Blast off pegmatoid with production blast, where identified above the LG6A reef band;
- Ensure support units are installed to mine standard (correct spacing in both dip and strike direction);
- Ensure support unit installation angle is between 70 and 90 degrees (use starter-jumper to drill support holes);
- Ensuring support units are correctly tensioned with initial installation;
- Damaged and protruding roof bolts must be replaced and cut off; and
- Avoid drilling and blasting the hanging wall beam to avoid sockets.

It is important that these hazards be avoided by conducting adequate early entry examination by qualified and competent individuals. These hazards remain until the rock engineering personnel come to conduct panel risk rating. During this time, work is carried out in the panels. It means that there is a high possibility that mining individuals will get injured since these hazards may result in FOG. Therefore, it is critical that adequate early entry examinations are conducted prior any work being carried out in the panels. Continuous barring (mid-shift barring) is also critical, as there can be loose rock along prominent geological structures. Focus should not only be put on the face area but the back areas as well since blast vibrations may loosen up rock blocks in the back areas also. Since rock engineering personnel only conduct PRRs monthly in each panel, it is vital that mining personnel are able to identify and mitigate minor rock related hazards. In cases where major hazards are identified, rock engineering personnel should be notified.

8.2.2. Ground condition monitoring

The current FOG management system comprises of three monitoring equipment namely; Ground Penetrating Radar (GPR), Borehole camera and fall of ground (FOG) light. The application of each monitoring equipment was discussed in Chapter 3. The fact that FOGs continued occurring at the mine is a concern since these equipment are available to assist detect any potential unstable blocks that may cause FOGs. Conducting this study assisted in identifying challenges within the current system that might be triggering factors of FOGs at the mine. Discussed below are challenges relating to each monitoring equipment which might have been missed in the system, hence FOGs. Ways in which these challenges can be improved are also presented.

<u>Ground Penetrating Radar (GPR)</u>

Ground penetrating radar (GPR) scanning is conducted on a monthly basis in each panel. Three scans (up-dip, centre, and down-dip) are taken from the face 10m to the back area in the direction of strike. GPR penetrates up to 7m into the immediate hanging wall (Figure 8.1). Any horizontal and sub-horizontal structure detected within the hanging wall strata is displayed in the GPR scan. This practice is critical as it gives details of structures within the immediate hanging wall that cannot be visualized by a naked eye unless exposed.

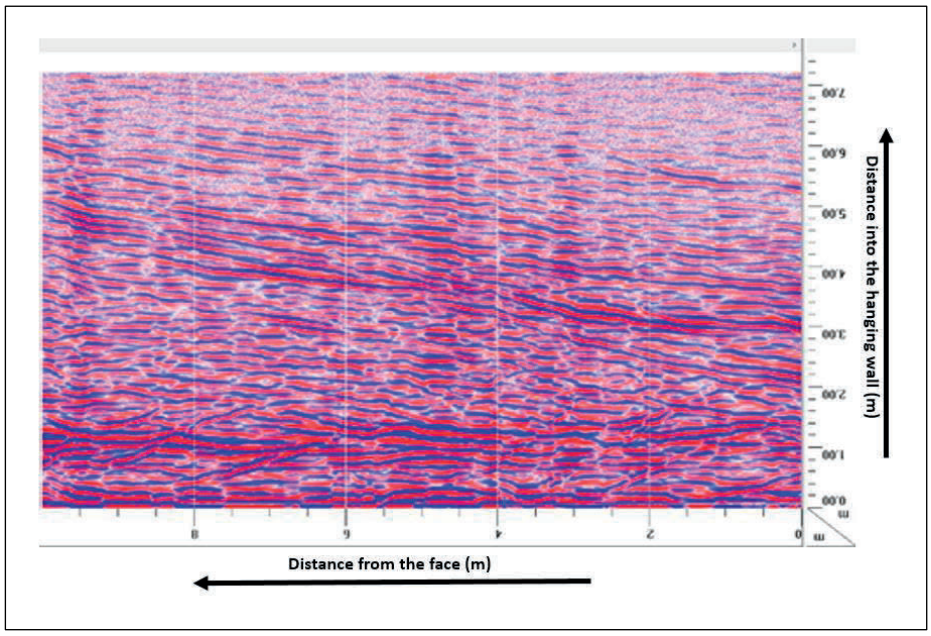


Figure 8.1: GPR scan image showing distance scanned and penetration depth

GPR scanning is done once a month in each panel. This means that only 10m from the current face to the back area is scanned. If the average advance per blast at the mine is 3m, and a panel is blasted over 3 times in one month, then there are portions in the panel (between the area scanned the previous month and 10m from the current face area), which are not scanned at all. To remedy this possibility, GPR scans can be conducted after every three blasts (Figure 8.2). In addition, more data acquisition can be achieved through scanning in both strike and dip direction (Figure 8.3). The current scan length target is 10m. Since panels are 10m wide, the process can still be conducted in dip direction. Therefore, any structures detected by the GPR can easily be located in the panel, using GPR scans obtained from both strike and dip direction.

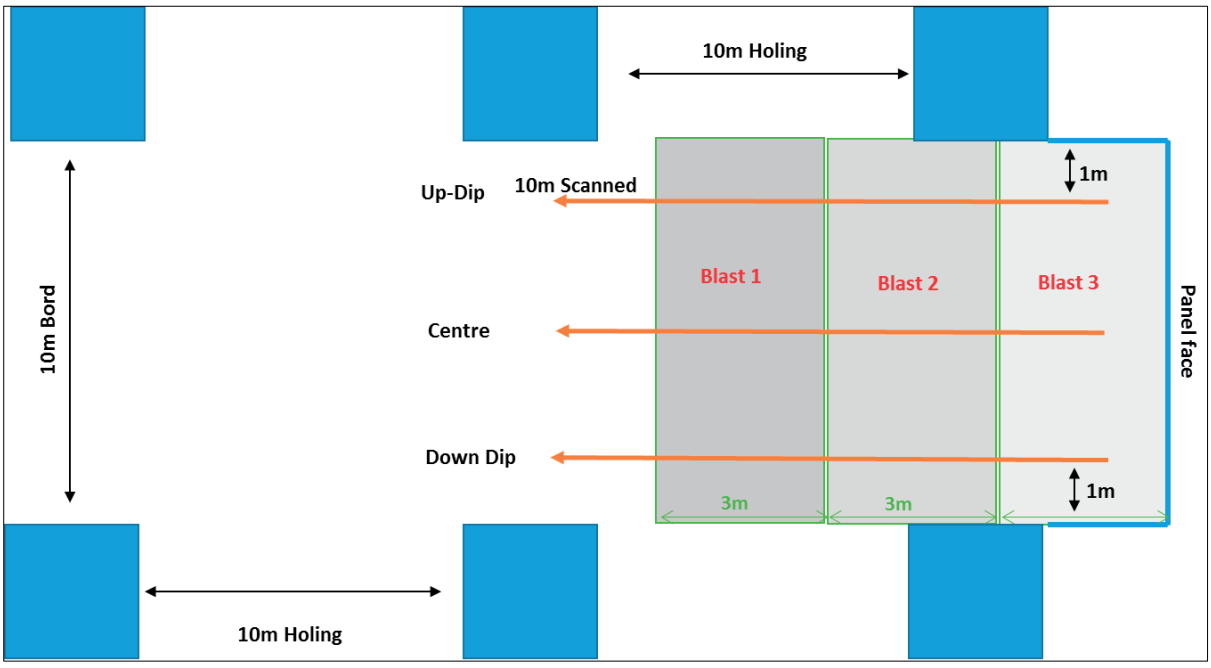


Figure 8.2: GPR scanning plan for a panel after three blasts

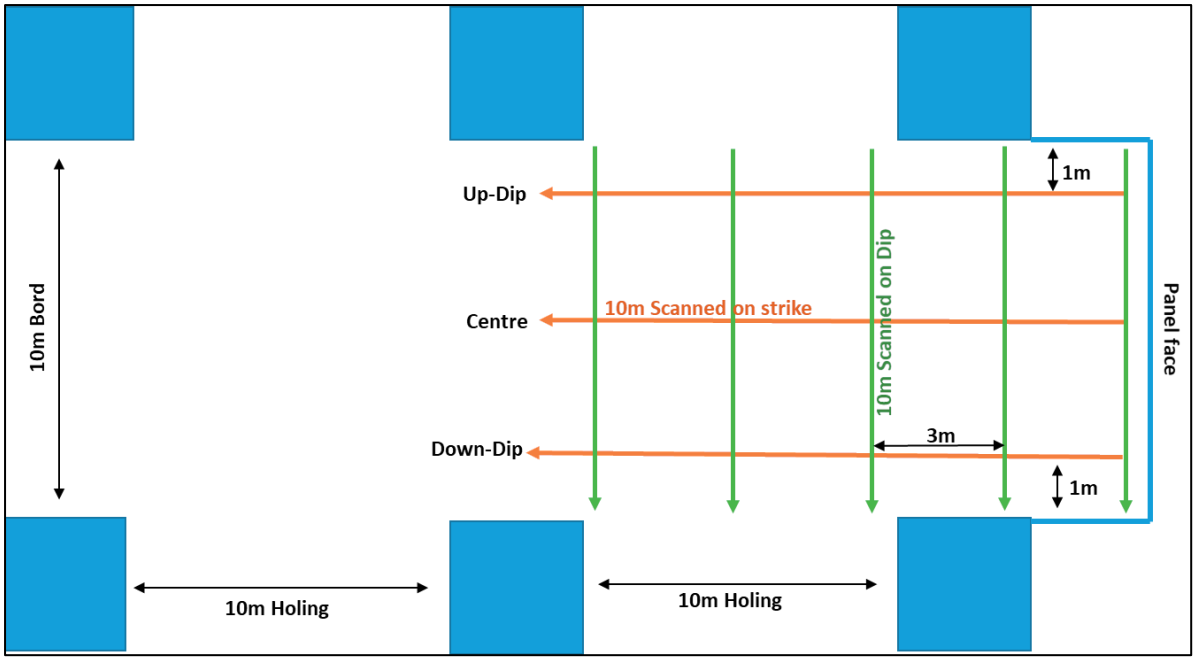


Figure 8.3: GPR scan layout for both dip and strike direction

Borehole camera inspection

Borehole camera inspections are only done when structures are detected from GPR scanning. This practice provides quick visuals of the exact structure detected within the hanging wall. As explained in Chapter 3 (section 3.3), borehole camera results show the characteristics such as infilling material, of the detected structure. In most cases, three boreholes are drilled (up-dip, centre, and down-dip) when GPR scan detects a structure in the hanging wall. The proposed layout in Figure 8.3 will be useful, as it will direct the exact position in the panel where the structure is detected.

Borehole camera observations are underutilised in the system because they are only recommended where GPR scans detect a structure. The practice is quick and simple and from it, useful information is obtained. Therefore, this practice can be applied even when no structures were detected by GPR. GPR only detects horizontal and sub-horizontal structures. However, borehole camera inspections can display both flat and steeply dipping structures. A unique fall of ground prevention strategy implemented at Two Rivers Platinum mine by (Esterhuizen, 2014), showed that borehole camera observations coupled with FOG investigation can be used to minimise falls of ground in local ground conditions. The same approach can be implemented at the Chrome mine where similar ground conditions are experienced (Figure 8.4).

Instead of only drilling three holes (up-dip, centre, and down-dip) for borehole camera inspection, camera holes can be drilled at an interval of 5m. This way it will be easy to identify change in ground conditions as mining advances. Camera holes should be drilled at 4m length so that secondary support recommendations can be easily issued where necessary.

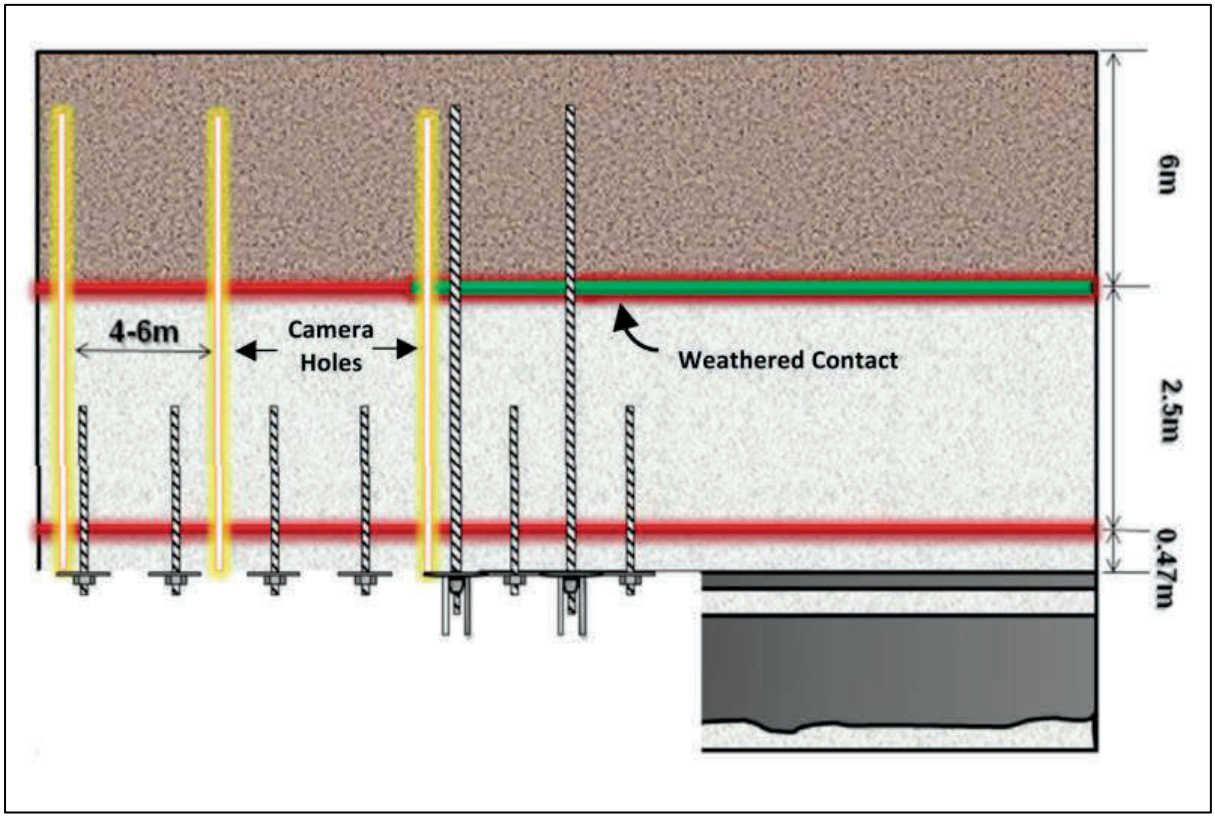


Figure 8.4: Adjusted support strategy based on local ground conditions *(Esterhuizen, 2014)*

Fall of ground (FOG) light

Fall of ground light is a monitoring instrument used to give warning of any strata movement and potential ground fall. Warning signals are given by means of visual light emitting diode (LED). When the FOG light is flashing green LED, it is in its monitoring condition. This means that the working area is safe. As soon as movement is detected (2.5mm in this case), the LED flashes red. At the Chrome mine, FOG lights are installed in boreholes where borehole camera inspection was conducted to confirm the structures detected by GPR as discussed in Chapter 3 (section 3.4). Depending on the depth at which a structure was located, a 2m or 4m long FOG lights are installed.

FOG lights are used for continuous monitoring of the hanging wall strata. The instrument is too sensitive that if not handled with care (or overturned while installing), it flashes red right after installation. In addition, the sensitiveness is also seen after blasting where the FOG light flashes red due to blast vibrations. When this happens, these FOG lights are usually replaced. Furthermore, during the cleaning process, LHDs sometimes damage the FOG light and it has to be replaced. These actions disturb the whole monitoring system, as there is no consistency in ground monitoring if the equipment is damaged. It is unfortunate that rock engineering personnel have to do a random check throughout the mine in areas where FOG lights have been installed, to see if they are still in their monitoring state because there is no centralised system showing FOG lights installed throughout the mine. This is one of the major challenges that the mine experiences with FOG lights.

The most alarming shortfall with FOG lights is that there is no specified monitoring circumference or area provided for FOG lights. The manufacturer only specifies the trigger length and movement detector length, which is 2m or 4m and 2.5mm respectively. This means that the mine has to identify the possible or expected block size that they want to monitor when installing FOG lights underground. However, the current system does not specify any blocks but installs FOG lights any way. There are various ground monitoring equipment in the market, which serve the same purpose as FOG lights. A comparison was conducted between FOG lights and Rocktales as specified by the manufactures (Anon, nd) and (QUALIROCK (a), 2016) respectively. Results are presented in Table 8.1.

Table 8.1: Comparison of benefits between Rocktale and FOG light (*QUALIROCK (a), 2016***;** *Anon, nd***)**

Rocktale	**Fog light**
Two LED lights (blue and red)	Two LED lights (Green and Red)
Two triggering anchors	One triggering anchor (1.5mm to 6.5mm)
Piston snaps out when triggered	Efficient
Highly visible	Easily visible
Durable battery power	12 months plus battery life
Length customisable and colour-coded branding according to length	Various lengths available up to 6m
Easy and fast installation	Simple to install
Can be used closer to blasting areas	Tamperproof
Very low priced to increased coverage	Cost effective
24/7 monitoring	Continuous monitoring
Fail safe devise	
Suitable in low seam and stope heights	
Mine vehicle-friendly in low areas	
Dust and water proof	
Mechanical device, no electronics	
Helps in decision making on type of support to be installed	

Rocktale is a continuous ground-monitoring instrument manufactured by QUALIROCK. It is designed to give warning of hanging wall instability in underground operations. Rocktales have colour-coded branding according to their lengths (Blue- less than 2m; Red- 2m to 3m; and Green- more than 3m). The device is equipped with two LED lights (blue and red) and two triggering anchors. Each triggering anchor is assigned to an LED light where blue LED light flashed due to movement in the lower roof and red from the upper roof as shown in Figure 8.5. Triggering anchor range can be as little as 1mm to 3mm for bottom anchors and the upper trigger anchor can be pre-set to a maximum length of 2m (measured from the bottom of the hanging wall) depending on the stratigraphy and lithology of the mine (QUALIROCK (a), 2016). The bottom trigger usually targets blocky ground conditions, key

bocks and low angled joints. If movement is detected in the lower strata, the blue LED will start flashing. This means that the lower triggering anchor moved along the lower roof in downward direction. As a result, the inner piston with blue reflective tape snaps downwards, triggering the blue LED light to flash making it visible and identifiable that there has been movement in the lower strata. The same concept applies when movement is detected in the upper strata. In contrast, the outer piston moves downward along the upper roof causing the red LED light to flash (QUALIROCK (a), 2016).

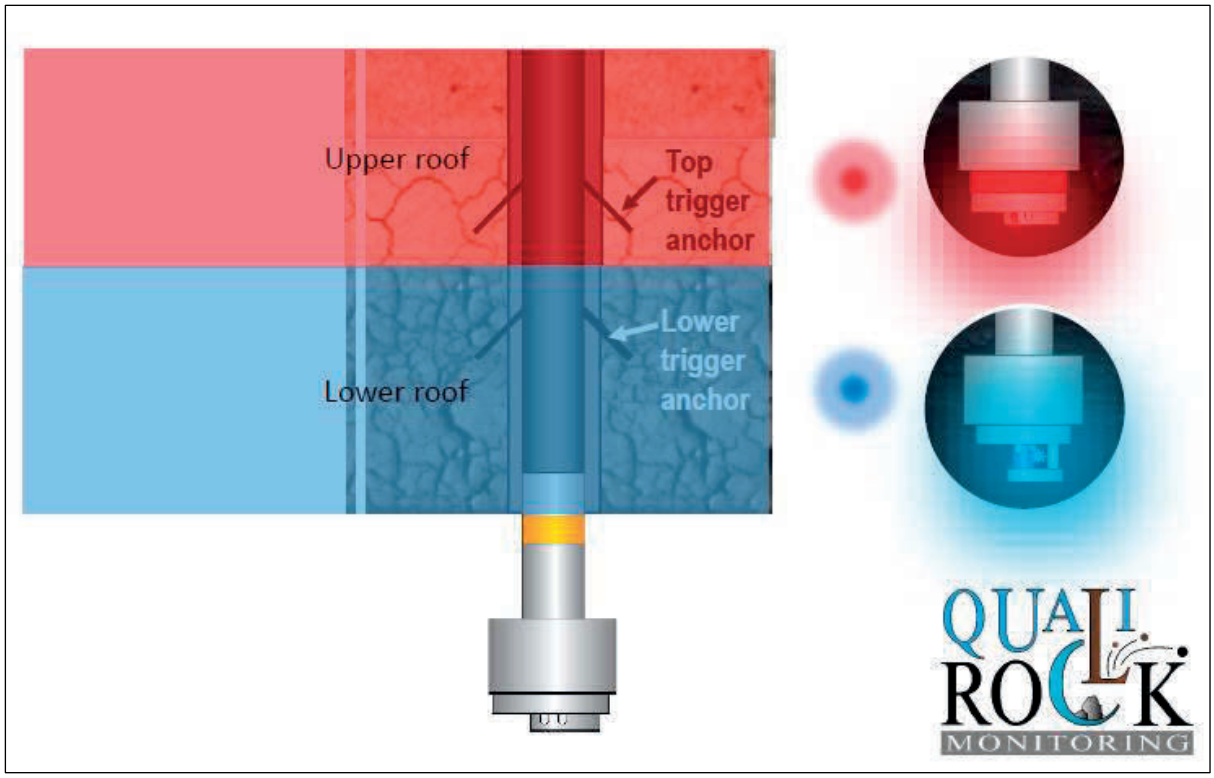


Figure 8.5: Rocktale installed in the hanging wall *(QUALIROCK (b), 2016)*

The Rocktale is more advantageous as it monitors movement of two bedded strata. Since GPR detects sub-horizontal and horizontal parting planes or structures that are confirmed after, using borehole camera inspection, Rocktale would be an appropriate monitoring equipment to use in order to know how much of each strata has moved if any movement occurs. Whereas with FOG lights, once it triggers, it remains flashing red. This means that it is not easy to tell if there has been further movement after the first triggering movement. The device also has various target length as FOG lights do. An exceptional benefit with Rocktale

is that they help with support decision-making as it is clear from the LED lights which strata has moved. Therefore, it will be easy to recommend the length of support needed in that particular area. Moreover, Rocktales are more resistive to blast vibrations when compared to FOG lights. Blasting will only affect the bottom-triggering anchor if the device is installed closer to the face. However, according to the manufacture (QUALIROCK (b), 2016) the following can be done to prevent triggering from blasting:

- Tighten the Rocktale triggering anchor to make triggering difficult;
- Install the Rocktalel but only remove the clip after the blast; or
- Do not install the Rocktale prior the blast (install after the blast)

Further research is recommended for continuous ground monitoring equipment to be modified into a centralised monitoring system. This will help trace all areas that are under continuous ground monitoring throughout the mine.

8.2.3. Support system design

Ground support systems play an important role in ground stabilisation. The main challenge identified through conducting this study is that the current FOG management system focusses more on the monitoring aspect rather than support design for highly jointed ground conditions. Chapter 6 and 7 showed how joint data can be used in prediction of falls of ground. It can be concluded that this is the main loophole within the current system. The exercise carried out in Chapter 6 and 7 showed that joint data is critical for support design purposes. At times, visual observations may determine an excavation as stable while it is actually unstable, in contrast, GPR scans may detect that there are no major structures in the hanging wall whereas joint mapping proves otherwise. This may be due to the fact that GPR scanning only detects flat dipping structures. Therefore, it is advisable to utilise joint mapping techniques, coupled with visual observations and GPR scanning to ensure excavation stability.

FOG lights are installed in the working places; however, the current system does not clarify the extent of the block monitored with each FOG light. Neither is there an ideal circumference specified by the manufacture that these FOG lights monitor. To remedy this challenge, joint data can be used for probabilistic analysis to estimate the block sizes to be expected for a defined ground condition. Probabilistic analysis carried out in Chapter 7

showed that about 80% of all key blocks formed are $1m^3$ in size, which conforms to the average fallout thickness at the mine (0.95m), obtained from both back analysis and brow data thickness. This means that the support system must incorporate areal coverage for highly jointed rock masses to prevent falls of ground. It is highly recommended that the mine introduce probabilistic analysis in the FOG management system. This will assist in combating FOGs at the mine since support designs will be based on local geological data.

8.3. Summary and conclusions

Falls of ground are the main contributors of fatalities and accidents in underground mining. Rock engineering practices are there to ensure that mining is carried out safely and profitably. Falls of ground are induced by either natural or human factors. Natural factors include geological structures, whereas human factors are poor examination of ground conditions, inadequate drilling and blasting practices, poor support installation and inadequate barring procedures. The above listed factors were found to be the main triggering factors of FOGs at the mine. It is therefore important that competent individuals who are trained and skilled for the job, conduct all these activities. Analysis conducted on the current fall of ground management system at the mine shows that there are prominent challenges within the system that are contributing to falls of ground. These challenges include:

- Inadequate early entry examination (prominent hazards identified during PRR relate to early entry examination);
- GPR scanning is only conducted on a monthly basis, 10m from the current face to the back area, in strike direction; and
- FOG lights trigger easily due to blast vibrations.

The above listed challenges can be addressed as follows, to help improve the current system and combat FOGs at the mine.

- Ensure early entry examination is conducted adequately by competent and skilled personnel;
- GPR scanning can be conducted after every 3 blasts in both dip and strike direction;
- Rocktales can replace FOG lights for continuous ground monitoring.

In conclusion, underground risk ratings and working area audits coupled with continuous ground monitoring are deemed essential to help combat falls of ground. The mine must introduce the use of joint data to determine the probability of rock falls. This approach will assist in support design for various ground conditions. Further research on continuous ground monitoring devices is considered vital for monitoring to be centralised and traceable.

CHAPTER 9: CONCLUSIONS AND RECOMMENDATIONS

Fall of ground is one of the major hazards in underground mining. Fatalities and injuries in the mining industry are largely due to rock related hazards. Unfortunately, the rock mass in which the ore body is located, its occurrence (shape, size and orientation) and grade cannot be changed (Yilmaz, 2015). However, mining practices and support layouts can be designed to minimise rock related hazards. These designs should be based on rock mass behaviour at that particular environment. Generally, in shallow hard rock mining environments, fall of ground is gravity induced due to the interaction of geological discontinuities such as joints, faults, domes, and other planes of weakness within the rock mass.

The Chrome mine has experienced numerous fall of ground incidents in the previous years. This was a major concern, as there was already a ground control and monitoring system in place. The current fall of ground management technique includes visual observations (Panel Risk Rating), where panels are rated based on their geological complexity and compliance to support standards. In addition, monitoring such as Ground Penetrating Radar (GPR), Borehole camera inspections and FOG light are used. With such a defined fall of ground management system, rock fall related incidents occur unexpectedly. The main objective for conducting this study was to come up with a sound systematic approach to help minimise and control falls of ground at the Chrome mine. This was achieved by looking at the current FOGs management technique used at the mine together with historical fall of ground database so as to identify the main causes of FOGs. It was found that most falls of ground were due to intersection of joints and flat dipping structures such as domes. In addition, challenges identified within the current system are:

- Inadequate early entry examination (prominent hazards identified during PRR relate to early entry examination);
- GPR scanning is only conducted on a monthly basis, 10m from the current face to the back area, in strike direction; and
- FOG lights trigger easily due to blast vibrations.

The following can be practiced in order to help combat FOGs at the mine:

- Ensure early entry examination is conducted adequately by competent and skilled personnel;

- GPR scanning can be conducted in both dip and strike direction, after every 3 blasts (for an average advance per blast of 3m) instead of once a month to avoid leaving out some areas not scanned (i.e. Portions between the area scanned in the previous month and 10m from the current face area);
- Rocktales can replace FOG lights for continuous ground monitoring.

After identifying the main causes of FOGs and analysing the current FOGs management system, it was noticed that the system did not consider the impact of joint, as there was no documented joint data other than that presented in the Mine's Code of Practice. A review on joint characteristics and measurement of joint properties underground was conducted since there was no joint data present. Reviews show that joint formation mechanisms influence the joint characteristics of a given joint and the most important joint properties that contribute to underground rock falls are joint orientation, spacing and length. Joint orientation determines the possible failure mechanism, Joint spacing determines the possible size of block formed and joint length the possible shape and failure plane. These properties were attained using window mapping technique. Other techniques include scanline mapping and core logging.

The joint data was analysed in order to incorporate numerical modelling into the fall of ground management system. A commercially available rock engineering software program called J-Block was used in this regard for probabilistic analysis. A stability evaluation was carried out for two mining orientations (panels mining on strike and up-dip holing) in order to identify potential unstable blocks. Probability of key block size formed confirms with the overall fallout thickness evaluated using brow thickness data and historical fallout thickness from FOG incidents, whereby 0.95m was found to be the average fallout thickness throughout the mine. Rock fall probabilistic analyses indicate that about 80% of all key blocks formed are $1m^3$ in size. Results show that larger blocks are more likely to fail by means of rotation. Support system evaluation is deemed essential for excavation stability. Large key blocks require support units with capacity greater than their weight. Likewise, to prevent small blocks from falling between support units, areal coverage (wire mesh, Oslo straps, shotcrete etc.) is suggested in heavily jointed rock masses.

In conclusion, probabilistic analysis can be used to evaluate probability of rock falls, and support design for stability enhancement. This study showed that joint data can be used to improve the current fall of ground management system at the mine though its ability to evaluate the stability of excavations. As a result, the knowledge of the expected key blocks in

a particular excavation can be achieved. Therefore, the size of blocks monitored by FOG will be known, thus correct measures can be put in place to minimise rock related hazards. The same approach can be utilised while focusing on a single panel rather than the whole mine or section, which is strategic for stability enhancement.

The following were the major observations from the study:

- Probability of FOG increase with increasing geological structures present in a specific excavation. Orientation and size of excavation as opposed to orientation and size of geological structures is a critical aspect for support design in underground mine operations, regardless of the depth below surface.
- FOG is a concern to the mining industry. The fact that ground control and monitoring systems are put in place supports that rock related hazards are a major concern in this industry.
- The current fall of ground management system focuses mostly on monitoring, therefore emphasis needs to be put in support design of jointed rock mass.
- Owing to exposure of geological structures on a daily basis in the mine, probabilistic approach is deemed essential for excavation stability analysis and support design in jointed rock masses.

The following recommendations arose from the study:

- Consider each section separately as rock properties are not always the same. Although the variability may be minimal at times, it is advisable to consider the North and South section separately in this case. This will also help with identifying any sudden changes in either section.
- In areas where there are prominent geological structures, joint data information should be collected for probabilistic analysis. Implement rock fall hazard evaluation using probabilistic key block analysis.
- Use diamond drilled core to attain joint data for rock fall hazard analyses of the ground ahead.
- Introduce Rocktale within the system to replace FOG lights, since Rocktales are more beneficial when compared to FOG lights.

- Ensure continuous ground monitoring devices are installed to monitor a possible unstable block that has been confirmed through probabilistic analysis.
- Further research can be conducted in relation to continuous ground monitoring devices, to introduce a centralised system that is controlled in the control room. To avoid having to go back and forth to check whether the device is still in its monitoring condition, a centralised monitoring system is deemed essential.

REFFERENCES

Adoko, A., Phumaphi, P. & Zvarivadza, T., 2016. *Quantifying Rock Mass Behavior around Underground Excavations.* Texas, USA, American Rock Mechanics Association.

Anon, 2013. *Introduction and Use of the Borehole Camera in Mining-Related Investigations,* U.S: Office of Surface Mining Reclamation and Enforcement.

Anon, 2017. *Chamber of Mines of South Africa. Safety in Mining Fact Sheet.* [Online]
Available at: http://www.chamberofmines.org.za/component/jdownloads/send/22-special-features/214-fact-sheet-safety-in-mining-2016
[Accessed 18 04 06].

Anon, nd. [Online]
Available at: http://www.globalsources.com/si/AS/TVBTECH-Shenzhen/6008848305937/pdtl/Video-borescope-camera/1130290584.htm
[Accessed 18 08 2017].

Anon, nd. *Mining Industry Occupational Safety & health.* [Online]
Available at: http://www.mosh.co.za/falls-of-ground/leading-practices/reasons
[Accessed 13 08 2017].

Anon, nd. *New Concept Mining.* [Online]
Available at: http://www.ncm.co.za/foglight.html
[Accessed 19 08 2017].

Apel, D. B. & Dezelic, V., 2005. Evaluation of high frequency ground penetrating radar (GPR) in mapping strata of dolomite and limestone rocks for ripping technique. *International Journal of Surface Mining, Reclamation and Environment,* 19(4), pp. 260-275.

Baecher, G. B., Einstein , H. H. & Lanney, N. A., 1977. *Statistical description of rock properties and sampling.* Colorado, Proceedings 18th US Symposium on Rock Mechanics, June 22-24.

Barton, N., 1976. The shear strength of rock and rock joints. *International Journal of Rock Mechanics and Mining Science and Geomechanical Abstracts,* 13(09), pp. 1-24.

Barton, N. & Choubey, V., 1977. The shear strength of joints in theory and in practice. *Rock Mech,* Volume 10, pp. 1-65.

Barton, N., Lien, F. & Lunde , J., 1974. Engineering classification of rock masses for the design of tunnel support:. *Rock Mechanics,* 06(04), pp. 189-236.

Bieniawski, Z. T., 1973. Engineering classification of rock masses. *Trans. S. African Institute Civil Eng,* 15(12), pp. 335-344.

Bieniawski, Z. T., 1974. *Geomechanics classification of rock masses and its application in tunneling.* Colorado, International Society for Rock Mechanics, pp. 27-32.

Bieniawski, Z. T., 1998. Towards a creative design process in mining. *Mining Engineering,* Volume 40, pp. 1040-1044.

Borman, D., 2006. Underground Mine Profiling. *International Platinum Conference 'Platinum Surges Ahead.The Southern African Institute of Mining and Metallurgy.,* pp. 165-168.

Brady, B. H. & Brown, E. T., 2004. *Rock Mechanics for Underground Mining.* Third Edition ed. s.l.:Springerlink.

Bridges, M. C., 1975. *Presentation of fracture data for rock mechanics.* New Zealand, Brisbane, pp. 144-148.

Brown, E. T., 2007. *Block Caving Geomechanics*, .. 2nd Edition ed. Brisbane: Julius Kruttschnitt Mineral Research Centre, The University of Queensland.

Buynevich, I. V., nd. *High-Resolution Subsurface Imaging of Coastal Barriers,* US: USGS Woods Hole Science Center / Coastal and Marine Geology.

Bye, A. R. & Bell, F. G., 2001. Stability assessment and slope design at Sandsloot open pit, South Africa. *International Journal of Rock Mechanics and Mining Sciences ,* Volume 38, pp. 449-466.

Cai, M., 2016. Prediction and prevention of rockburst in metal mines – A case study of Sanshandao gold mine. *Journal of Rock Mechanics and Geotechnical Engineering,* 8(2), pp. 204-211.

Call, R. B., Savelv, J. & Nicholas, D. E., 1976. Estimation of joint set characteristics from surface mapping data. *Proc. 17th U.S. Symposium on Rock Mechanics,* pp. 2B2-1-9.

Cawthorn, R. G., 2015. The Bushveld Complex, South Africa. In: B. Charlier, O. Namur, R. Latypov & C. Tegner, eds. *Layered Intrusions.* Dordrecht: Springer, pp. 517-587.

Chikande, T. & Zvarivadza, T., 2016. Review of support systems used in poor ground conditions in platinum room and pillar mining: a Zimbabwean case study. *The Southern African Institute of Mining and Metallurg*, 116(04), pp. 332-332.

Coates, D. F., 1964. Classification of rocks for rock mechanics. *International Journal of Rock Mechanics and Mining Sciences,* 01(03), pp. 421-429.

Deer, D. U., 1963. Technical description of rock cores for engineering purposes. *Rock Mechanics and Engineering geology,* 01(01), pp. 16-22.

Deer, D. U. et al., 1967. *Design of surface and near-surface construction in rock, in Fairhurst, Charles, ed., Failure and breakage of rock.* New York, American Institute of Mining , Metallurgy and Petroleum Engineers, pp. 237-302.

Dershowitz, W. S., 1984. *Rock Joint Systems, Ph.D. Thesis*, Massachusetts Institute of Technology: Cambridge, Massachusetts.

Dunn, M. J., Earl, P. J. & Watson , J., 2008. Support design using probabilistic keyblock methods, in Proceedings 6th International Symposium on Ground Support in mining and civil engineering applications. Volume 30, pp. 623-636.

Eales, H. V. & Cawthorn, R. G., 1996. The Bushveld Complex. *Developments in Petrology*, Volume 15, pp. 181-229.

Ege, J. R., 1849. *Core Index, A Numerical Core-Logging Procedure for Estimating Rock Quality.* U.S.Geological Survey Circular 954 ed. U.S: U.S. Geological Survey.

Esterhuizen, A., 2014. Unique fall-of-ground prevention strategy implemented at Two Rivers Platinum Mine. *The Journal of The Southern African Institute of Mining and Metallurgy,* 114(10), pp. 785-790.

Esterhuizen, G. S., 2003. *JBlock Uses's Manual,* Pretoria: s.n.

Esterhuizen, G. S. & Streuders, S. B., 1998. Rockfall hazard evaluation using probabilistic keyblock analysis. *The Journal of The South African Institute of Mining and Metallurgy,* 98(02), pp. 59-64.

Goodman, R. E. & Shi, G., 1985. *Block theory and its application in rock engineering,* New Jersey: Prince Hall.

Grady, L. A., 1983. *The Application of Geostatistical Techniques to the Analysis of Joint Data. MSC Thesis,* Johannesburg: University of Witwatersrand.

Grenon, M. & Hadjigeorgiou, J., 2003. Open stoping using 3D joint networks. *Rock Mech. Rock Engng,* 36(3), pp. 183-208.

Gumede, H., 2006. *Development of data sets on joint characteristics and consideration of associated instability for a typical South African gold mine. MScEng Dissertation,* Johannesburg: University of the Witwatersrand.

Gumede, H. & Stacey, T. R., 2007. Measurement of typical joint characteristics in South African Gold mines and the use of these characteristics in the prediction of rockfalls. *Journal of the South African Institute of Mining and Metallurgy,* 107(5), pp. 335-344.

Hadley, D. G. F. & Grant, F., 1972. *Stope and pillar design for the Elliot Lake Uranium Mines,* s.l.: CIM Bulletin.

Hawkins, J. W. & Evans, R. S., 2004. *Uses of the Borehole Camera in Hydrologic Investigations Related to Coal Mining.* Morgantown, WV, ASMR, pp. 847-859.

Heunis, R., 1980. The development of rock-burst control strategies for South African gold mines. *The Journal of The South African Institute of Mining and Metallurgy,* 80(04), pp. 139-150.

Hobbs, B. E., Means, W. D. & Williams, P. F., 1976. *An outline of Structural Geology*, New York: Wiley.

Hoek, E., Kaiser, P. K. & Bawden, W. F., 1998. *Support of underground excavations in hard rock.* Netherlands: Balkema.

Internal Mine Document (a), 2014. *Borehole camera monitoring report,* S.A: Chrome Mine.

Internal Mine Document (a), 2015. *Borehole Camera Standard Operating Procedure,* S.A: Chrome Mine.

Internal Mine Document (a), 2016. *Workplace Risk Rating Standard Operating Procedure,* S.A: Chrome mine.

Internal Mine Document (a), 2017. *Investigation into fall of ground incident,* S.A: Chrome mine.

Internal Mine Document (b), 2014. *Borehole camera monitoring report,* S.A: Chrome Mine.

Internal Mine Document (b), 2015. *Underground Ground Monitoring Instrumentation Standard Operating procedure,* S.A: Chrome Mine.

Internal Mine Document (b), 2016. *Ground penetrating Radar (GPR) Standard Operating Procedure,* S.A: Chrome mine.

Internal Mine Document (b), 2017. *Standard Operating Procedure, Underground Primary Support.,* S.A: Chrome Mine.

Internal Mine Document, 2011. *Investigation into fall of ground,* S.A: npublished mine document.

Internal Mine Document, 2013. *Mandatory code of practice to combat rockfall and rock burst accidents in tabular metalliferous mines,* Limpopo: Unpublished Mine Document.

Internal Mine Document, 2018. *Investigation into fall of ground incident,* SA: Chrome Mine.

ISRM, 1978. Suggested methods for the quantitative description of discontinuities in rock masses. *International Journal of Rock Mechanics and Mining Sciences and Geomechanical,* 15(6), pp. 319-368.

ISSI, 2001. *Application of the Rock Radar in the Bushveld Igneous Complex,* s.l.: ISSI Publication.

Ivanova, V. M., 1998. *Geologic and Stochastic Modelling of Fracture Systems in Rocks,* PhD Thesis: Massachusetts Institute of Technology.

Jager, A. J. & Ryder, J. A., 1999. *A Handbook on Rock Engineering Practice for Tubular Hard.* Johannesburg: The Safety in Mines Research Advisory Committee (SIMRAC).

Kinnaird, J. A., 2005. *The Bushveld Large Igneous Province.* [Online]
Available at: http://www.largeigneousprovinces.org/sites/default/files/BushveldLIP.pdf

Klemm, D. D. et al., 1982. The Formation of Chromite and Titanomagnetite Deposits Within the Bushveld Igneous Complex. In: A. G. e. a. O. Genesis, ed. Berlin, Heidelberg: Special Publication of the Society for Geology Applied to Mineral Deposits, vol 2, pp. 351-370.

Kruger , F. J., 1990. The stratigraphy of the Bushveld Complex: a reappraisal and the relocation of the Main Zone boundaries. *S.Afr.J .Geo,* 93(2), pp. 376-381.

Kulatilake, P. H. S. W. et al., 1995. *Discontinuity network modelling of the rockmass around a tunnel close to the proposed permanent shiplock area of the three gorges dam site in China.* Daemen and Schutz, Balkema, pp. 807-812.

Kulatilake, P. H. S. & Wu, T. H., 1984. Estimation of mean trace length of discontinuities. *Rock Mechanics and Rock Engineering,* 17(4), pp. 215-232.

Landmark, G. L. & Villaescusa, E., 1992. Geotechnical mapping of Mount Isa Mines. , *Proc. Western Australia Conference on Mining and Geomechanics, Szwedzicki, Baird and Little (eds), Western Australian School of Mines,* pp. 329-333.

Ledwaba, L. S., Scheepers, J., Durrheim, R. J. & Spottiswoode, M., 2012. Rockburst Damage Mechanism at Impala Platinum Mine. *The Southern African Institute of Mining and Metallurgy*, pp. 367-386.

Leger, J. P., 1991. Trends and causes of fatalities in South African mines. *Safety Science,* Volume 14, pp. 169-185.

Leith , C. K., 1923. *Structural Geology,* New York: Holt.

Lenhardt, N. & Eriksson, P. G., 2012. Volcanism of the Palaeoproterozoic Bushveld Large Igneous Province: the RooibergGroup, Kaapvaal Craton. *Precambrian Res 214–215*, pp. 82-94.

Maier, W. D., Barnes, S. J. & Groves, D. I., 2012. The Bushveld Complex, South Africa: formation of platinum–palladium, chrome- and vanadium-rich layers via hydrodynamic sorting of a mobilized cumulate slurry in a large, relatively slowly cooling, subsiding magma chamber. *Mineralium Deposita.*

Mathis, J. I., 1987. Discontinuity mapping- A comparision between line and area mapping. *Proc. 6th Int.Cong. Int. Soc. Rock Mech,* Volume 2, pp. 1111-1114.

Nezomba, E., 2012. *Use of joint trace data to evaluate stability of mining excavations, and validation against underground observations. MSc Eng Dissertation,* Johannesburg: University of the Witwatersrand.

Nicholas, D. E. & Sims, D. B., 2000. Collecting and using geological data for slope design. *International conference on slope stability in surface mining, Hustrulid, McCarter and Van Zyl (eds), SME, USA,* pp. 11-21.

Nong, S., 2010. *Review of Successes and Failures of Mechanisation Trials on Platinum Mines of the Bushveld Complex. Masters of Science in Engineering* , Johannesburg: University of the Witwatersrand.

Nyungu, D. & Stacey, T. R., 2014. Time-dependent tensile strengths of Bushveld Complex rocks and implications for rock failure around mining excavations. *The Journal of The Southern African Institute of Mining and Metallurgy,* 114(10), pp. 765-772.

OHMS, nd. *Strata Control Preparation Classes- Course Notes.* Potchefstroom: Open House Management Solutions (Pty) Ltd .

Ozbay, M. U., Ryder, J. A. & Jager, A. J., 1995. The design of pillar systems as practised in shallow hard-rock tabular mines in South Africa. *The Journal of The South African Institute of Mining and Metallurgy,* 95(01), pp. 7-12.

Palmström, A., 2001. Measurement and characterization of rock mass jointing. In: V. M. Sharma & K. R. Saxena , eds. *In-Situ Characterization of Rocks.* Norway: Balkema Publishers, pp. 1-40.

Price, N. J., 1966. *Fault and joint development in brittle and semi-brittle rock,* London: Pergamon Press.

Priest, S. D. & Hudson, J. A., 1981. Estimation of discontinuity spacing and trace length using scan line surveys. *Int. J. Rock Mech. Min. Sci and Geomech. Abstr,* Volume 18, pp. 183-197.

Priest, S. D. & Samaniego, J. A., 1983. *A model for the analysis of discontinuity characteristics in two dimensions.* Melbourne, In Proceedings. 5th International Society for Rock Mechanics Congress.

Qamata, X. & Steenkamp, C. J., nd. *Negotiating Complex Geological Structures By Applying New Mining Strategies,* s.l.: AMMSA.

QUALIROCK (a), 2016. *Rocktale broucher. Smart and effective rock strata monitoring,* Bronkhorspruit: Qualirock monitoring.

QUALIROCK (b), 2016. *The effect of blasting on the RockTale*, Bronkhorspruit: Qualirock monitoring.

Quaye , G. B. & Guler, G., 1998. The importance of both geological structures and mining induced stress fractures on the hangingwall stability in a deep level gold mine. *The Journal of The South African Institute of Mining and Metallurgy,* 98(03), pp. 157-162.

Roberts, M. K. C. & Clark-Moster, V., 2010. *Is there some commonality between the geological structures in the Bushveld Complex and the Great Dyke?.* South Africa, Platinum in transition 'Boom or Bust', pp. 149-156.

Robertson, A. M., 1977. *The determination of the stability of slopes in jointed rock with particular reference to the determination of strength parameters and mechanics of failure, Ph.D. Thesis,* Johannesburg: University of Witwatersrand.

Rocscience, 2002. *Dips User's Guide*, s.l.: Rocscience.

Roelofse, F. & Ashwal, L. D., 2012. The Lower Main Zone in the Northern Limb of the Bushveld Complexça41·3 kmThick Sequence of Intruded and Variably Contaminated Crystal Mushes. *Journal of petrology,* 53(7), pp. 1449-1476.

Ryder, J. A. & Jager, A. J., 2002. *Rock mechanics for tabular hard rock mines.* s.l.:The Safety in Mines Research Advisory Committee (SIMRAC).

Salamon, M. G., 1967. Method of designing bord-and-pillar workings. *Journal of the South African Institute of Mining and Metallurgy,* Volume 68, pp. 68-78.

Schouwstra , R. P. & Kinloch, E. D., 2000. A Short Geological Review of the Bushveld Complex. *Platinum Metals Rev,* 44(1), pp. 33-39.

Singh, N. G., 2007. Ground penetrating radar (GPR) 'Mineral base profiling and orebody optimization'. *The 6th International Heavy Minerals Conference 'Back to Basics', The Southern African Institute of Mining and Metallurgy*, pp. 179-182.

Stacey, T. R., 2004. The link between the design process in rock engineering and the code of practice to combat rock fall and rockburst accidents. *The Journal of The South African Institute of Mining and Metallurgy,* 104(01), pp. 29-34.

Stacey, T. R., 2006. *Design - a Strategic Issue.* Perth, Australian Centre for Geomechanics, pp. 13, Section 4.

Stacey, T. R. & Bartlett, P. J., 1990. *Probabilistic evaluation of orepass stability and support, Static and Dynamic Considerations in Rock Engineering,* A A: Balkema.

Stacey, T. R. & Gumede, H., 2007. Evaluation of risk of rock fall accidents in gold mine stopes based on measured joint data.. *The Journal of the South African Institute of Mining and Metallurgy.*, 107(5), pp. 345-350.

Steffen, O. K., Kerrich, J. E. & Jennings, J. E., 1975. Recent developments in interpretation of data from joint surveys in rock masses. *Proc. 6th Regional Conference on Soil Mechanics and Foundation engineering, Durban,* Volume 2, pp. 17-26.

Suppe, J., 1985. *Principles of structural geology,* USA: Prentice-Hall.

Swart, A. H., 2005. *Investigation of factors governing the stability of stope panels in hard rock mines in order to define a suitable design methodology for shallow mining operations. Master of Engineering (Mining) Dissertation,* Pretoria: University of Pretoria.

Swart, A. H., Human , J. L. & Harvey, F., 2005. Rock engineering challenges. *The Southern African Institute of Mining and Metallurgy,* 105(02), pp. 103-106.

Terzaghi, K. C., 1946. Introduction to tunnel geology; Rock defects and loads on tunnel supports. In: R. V. Proctor & T. L. White, eds. *Rock tunneling with steel supports.* Youngstown, Ohio: The Commercial Shearing & Stamping Co, pp. 15-99.

Terzaghi, R., 1965. Sources of error in joint surveys. *Geotechnique,* 5(3), pp. 287-304.

Treloar, M., 2003. *The Use of Ground Penetrating Radar (GPR) to Delineate Adverse Geological Structures at Samancor – Western Chrome Mines,* WCM: Samancor Chrome Mines.

Uken , R., 1998. *The geology and structure ofthe Bushveld Complex metamorphic aureole in the Olifants River area,* Durban, South Africa: Department ofGeology and AppliedGeology University ofNatal.

Vogt, D., van Schoor, M. & du Pisani, P., 2005. The application of radar techniques for in-mine feature mapping in the Bushveld Complex of South Africa. *The Journal of The South African Institute of Mining and Metallurgy ,* 105(06), pp. 391-400.

Von Gruenewaldt, G., Sharpe, M. R. & Hatton, C., 1985. The Bushveld Complex: Introduction and Review. In: *Economic Geology and the Bulletin of the society of economic geologists.* Pretoria: Society of Economic Geologists, pp. 803-812.

Walke, D. V. & Yerpude, R. R., 2015. Significance of Strata Monitoring Instruments in Roof Fall Risk Assessment of an Underground Coal Mine. *International Journal of Scientific and Research Publications,* 5(6), pp. 1-9.

Wallis, P. F. & King, M. S., 1981. Discontinuity spacings in a crystalline rock. *Int. J. Rock Mech. and Min. Sci,* Volume 17, pp. 63-66.

Wonik, T. & Olea , R. A., 2007. Borehole Logging. In: *Environmental Geology.* Berlin, Heidelberg: Springer, pp. 431-474.

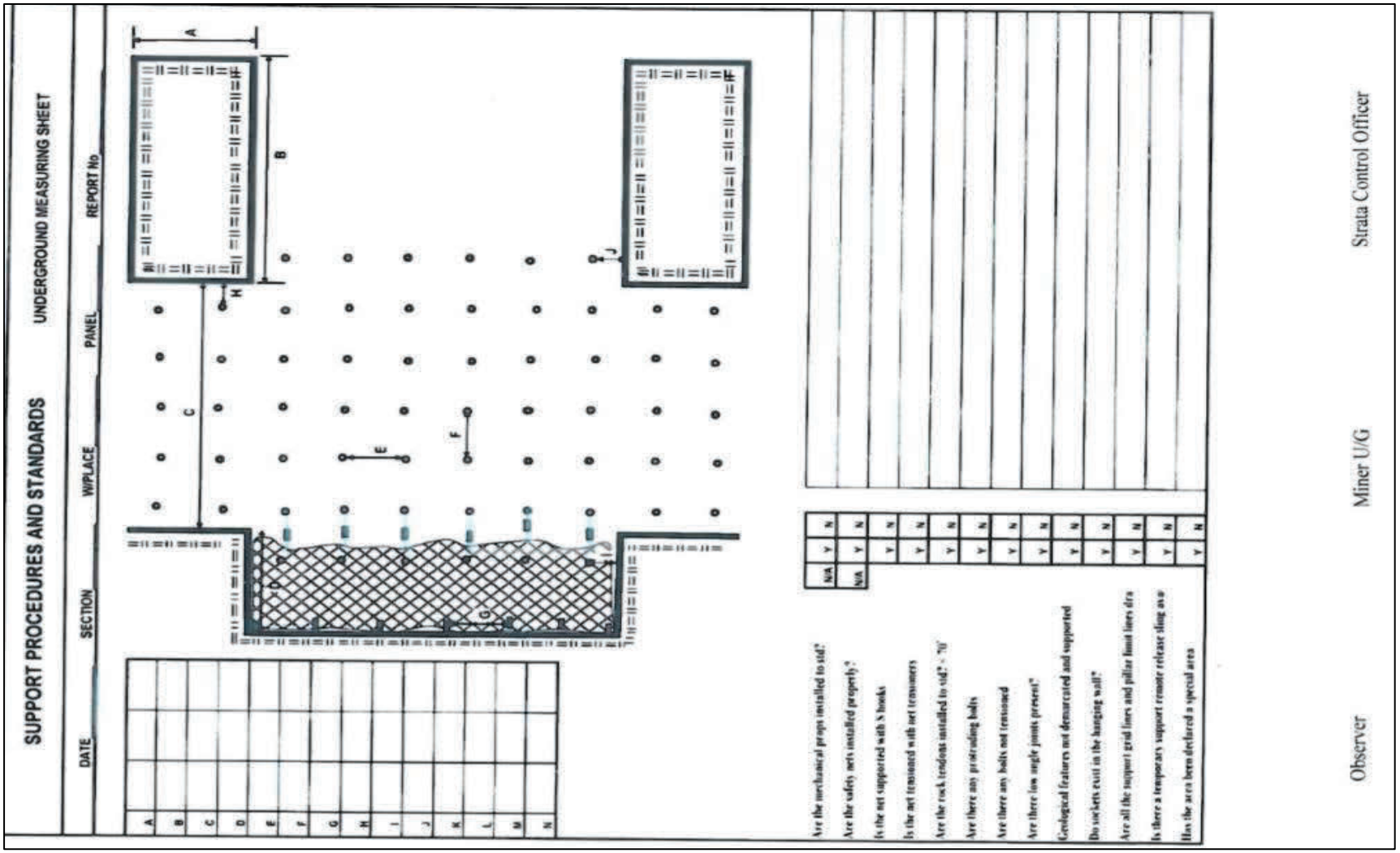
Yilmaz, H., 2015. *Rock Engineering. MINN 4010-Course Notes.* Johannesburg: Wits- School of Mining Engineering.

York, G. et al., 1998. *Develop guidelines for the pillar systems for shallow and intermediate depth, tabular hard rock mines and provide a methodology for assessing hangingwall stability and support requirements for the panels between pillars.* [Online]
Available at: http://www.mhsc.org.za/sites/default/files/GAP334REPORT.pdf
[Accessed 19 08 2017].

Zhang , L., 2004. *Drilled Shafts in Rock: Analysis and Design,* A A: Balkema Publishers.

APPENDIX A: PANEL RISK RATING

A.1. Panel Risk Rating Underground Checklist template



A.2. Electronic rating spreadsheet

	Mine		**Monthly Planning**					
	Panel Name							
	Date of compilation							
	Mine Overseer		**Panel Risk Rating**					
	Sheet completed by							
	Sheet checked by							
Group	**Parameter**	**Measurement**	**Standard**	**Actual**	**Comments**	**Likelihood**	**Consequence**	**Risk**
Temporary support	Are the mechanical props installed to standard	Y/N	Y	Y		1	1	1
	Is the safety net installed to standard	Y/N	Y	Y		1	1	1
	Is the safety net supported with S hooks	Y/N	Y	y		1	1	1
	Is the net tensioned with net tensioners	Y/N	Y	Y		1	1	1
	Is there a temporary support release sling available	Y/N	Y	Y		1	1	1
								5
Primary support	Are The rock tendons installed to standard (< 70 °)	Y/N	Y	Y		1	1	1
	Are there any protruding bolts	Y/N	N	N		1	1	1
	Are there any bolts not tensioned	Y/N	N	N		1	1	1
	Have the support grid lines been drawn in	Y/N	Y	Y		1	1	1
	Has the primary support tendons been installed to STD (spacing)	Y/N	Y	Y		1	1	1
	Primary support distance to face (comment activity on face)	m	1	0.9		1	1	1
	Primary support distance to side walls	m	1	0.9		1	1	1
								7
Geology	Any major geological structure present?	Dyke/Fault/Pothole	N	N		1	1	1
	Are all the geological features demarcated	acceptable/not acceptable	Acceptable	Acceptable		1	1	1
	Are all the geological features supported to standard	Y/N	Y	Y		1	1	1
	Are there sockets in the hanging wall	Y/N	N	N		1	1	1
	Are there any flat dipping features (<60° from Horizontal)	Y/N	N	N		1	1	1
	Is there water from the hanging wall	Y/N	N	N		1	1	1
								6
Pillar Cutting Compliance	Is the pillar the correct width	Y/N	Y	Y		1	1	1
	Is the pillar the correct length	Y/N	Y	Y		1	1	1
	Stoping Width (plan and actual)	Y/N	2.2	2.1		1	1	1
	Face Shape	acceptable/not acceptable	Acceptable	Acceptable		1	1	1
	Panel span (skin to skin)	m	10	10		1	1	1
	Leads and Lags General in the section	acceptable/not acceptable	Acceptable	Acceptable		1	1	1
	Mining direction	breast, up dip, down dip	N/A	Down dip		1	1	1
	Distance to holing	acceptable/not acceptable	N/A	Acceptable		1	1	1
								8
Calculations	Support Density	%	100	100		1	1	1
	Support Compliance	%	100	100		1	1	1
	Q rating	Unit	4	5		1	1	1
								3
							Total	**29**

Percentage compliance	**100.00%**	**Mine Overseer's Comments**	
Comments			

Recommendations	**Completed**	**Date**
Previous Recommendations		
New recommendations:		

Observer **Shift Supervisor**

Strata Control Officer

Mine Overseer

APPENDIX B: FALL OF GROUND DATABASE

Year	Time	Section	Activity	Area	Dislodged from	Fallout thickness (cm)	Geological structures
2010	MS	N	DSI	F	HW	0-20	70 cm upthrow fault
	MS	S	DSI	BA	HW	21-40	N/A
2011	MS	N	DSI	F	HW	0-20	Multiple joints
	AS	S	EEE	F	SW	0-20	NA
2012	*	*	*	*		*	*
2013	AS	N	L	BA	HW	41-60	N/A
2014	*	*	*	*		*	*
2015	MS	N	L	F	HW	21-40	Prominent joints
	MS	S	L	BA	HW	41-60	Chrome on H/W
	MS	S	DSI	BA	HW	0-20	N/A
	MS	N	DSI	F	HW	0-20	Prominent joints
2016	MS	S	DSI	F	F	0-31	Dome structure
	IBS	N	AB	F	HW	90-100	Prominent and low angle joints
	IBS	S	AB	F	HW	0-60	Dome structure and near vertical joints
2017	MS	S	DSI	F	HW	30	Low angle joint
	MS	N	DSI	F	F	40	Prominent joints with infilling material
	MS	N	DSI	F	SW	70	Low angle joint and dome

LEGEND			
MS - Morning Shift	N - North Section	DSI - Drilling and/or Support installation	
AS - Afternoon Shift	S - South Section	L - Loading	HW - Hanging wall
IBS - In-between Shifts	EEE - Early Entry Exam	AB - After blast	SW - Sidewall
* - No incident		BA- Back Area	F - Face

APPENDIX C: BROW THICKNESS DATA

C.1. North Section Brow thickness data

Panel	Peg	Distance (m)	Thickness (cm)	Dome/ low angle joint
N46N	N6017	3.3m	50	N
	N6017	5.9m	80	N
	N5751	-8.5m	35	N
NB7N	N5731	3m	60	N
	N5724	3m	40	N
	N5724	5m	40	Y
	N5874	2m	20	N
	N6015	8M	50	N
N48N	N5730	-1.2m	42	N
	N5730	7.7m	80	N
	N5848	5.4m	40	Y
	N6045	13.7m	47	N
	N6045	-4.3m	50	N
	N6001	10.3m	40	N
	N6001	3.8m	60	N
	N6001	-3.8m	30	N
	N6001	-13.2m	50	Y
	N6001	-12.1m	55	N
	N6001	-9.5m	48	N
	N5849	-8.5m	60	N
	N5849	-6.2m	20	N
	N6029	-3.8m	40	N
	N6047	-18.1m	35	Y
	N6047	-4.4m	40	N
	N6047	10.4m	50	N
	N6047	14.2m	37	N
	N5418	1m	40	N
	N5743	1.4m	50	N
NB8N	N5841	-3.6M	70	N
	N5841	-2M	30	N
	N6021	-19.5M	80	Y
	N6021	-2m	40	Y
	N6021	8m	30	N
	N6021	-3M	30	N
N53N	N6020	9.5m	60	N
	N6020	7.7m	30	Y
	N5853	12.1m	25	N
	N5853	6.3m	45	N
	N5786	6.5m	40	N

N54N	N5740	2m	30	N
	N5740	3.9m	50	N
	N6028	17.5m	100	N
	N6028	2.1m	45	N
	N5844	8.9m	50	N
	N5722	1.4m	60	N
	N5722	7.5m	95	N
N56N	N5843	3.2m	70	N
	N5771	1m	100	N
	N5878	10.7m	40	Y
	N5878	3m	60	N
N57N	N6041	3.2m	70	N
	N6041	-1.6m	20	N
	N6041	12.7m	48	N
	N5748	8.6m	70	N
N58N	N6040	6.5m	40	N
	N6040	12.3m	60	N
	N5780	24.4m	58	N
NB9N	N5778	-3.6m	20	N
	N5868	3m	15	N
	N5868	8.9m	100	N
	N6010	-5m	70	Y
	N6010	10m	85	N
	N6010	10m	30	N
	N6010	-4m	50	Y
N59N	N5496	8.1m	47	N
	N5496	6.2m	61	N
	N6019	1.1m	42	N
	N6019	5m	61	N
	N6019	3.7m	66	N
	N6019	4.4m	65	N
N60N	N6009	15.9m	35	N
	N6009	11.5m	40	Y
	N6009	7.6m	40	N
	N6009	6.5m	30	N
N61N	N6035	15.4m	67	Y
	N6005	8.0m	31	N
	N6005	3.5m	35	N
N63N	N5762	18m	39	N
	N6039	20m	86	N
N64N	N5729	22.6m	100	N
	N5729	11.6m	50	N
	N5729	5.5m	40	N
	N5729	13.9m	50	Y
	N6004	6.6m	45	N

	N6026	8.6m	30	N
N65N	N5775	12.4M	100	N
	N5775	19.7M	100	N
	N5776	14.2M	50	N
	N5776	7.2M	30	N
N66N	N6048	2.1m	45	N
	N6048	1.7m	55	N
NB10N	N5851	10m	50	N
1 Shaft	N5761	14.3m	58	N
	N5761	9.6m	35	N
	N5775	4m	34	Y
2 Shaft	N5760	6m	60	N
	N5490	0.2m	57	N
	N5490	13.9	70	N
3 Shaft	N5471	-1.3m	57	N
	N5724	-9.5m	52	N
	N5724	8.3m	43	Y
	N5874	18m	39	N
N67N	N6022	5.4m	30	N
	N6022	17.7m	67	N
	N6023	33.3m	30	N
N66N	Damaged	35m	35	N
N65N	N6005	6.7m	120	Y
	N6005	21m	55	N
N64N	N5795	2.7m	35	N
	N5795	2.4m	35	N
	N5795	7.9m	45	N
N63N	N6012	9m	55	N

C.2. South Section Brow thickness data

Panel	Peg	Distance (m)	Thickness (cm)	Dome/ low angle joint
B7N	S5865	11.8m	35	N
	S5211	8.6m	70	N
	Damaged	2.3m	100	N
	S5516	5.9m	69	Y
58N	S5815	3.1m	50	N
	S5867	11.7m	70	N
	S5867	8.5m	50	N
	S5867	6.9m	42	N
	S5867	9.4m	60	N
56N	S5904	3.2m	93	N
	S5904	18.6m	20	N
B8N	S5290	-12.2m	75	N
	S5518	3.8m	60	Y
	S5933	2.1m	35	N
	S5933	6.1m	50	N
	S5563	2.3m	43	N
55N	S5369	10.2m	19	N
	S5369	2.8m	45	N
	S5584	8.2m	48	N
	S5298	-3.6m	67	N
54N	S5826	27.3m	35	N
	S5672	5.4m	41	N
	S5672	5.7m	39	N
	S5672	4.9m	30	N
53N	S5044	16.1m	110	Y
	S5044	22.2m	70	N
	S5694	-8.3m	30	N
52N	S5927	5.4m	50	N
	S5927	1.5m	42	N
	S5927	-10.5m	75	N
50N	S5696	2.4m	50	N
	S5696	20.2m	35	N
61N	S5581	9.0m	40	N
	S5530	5.1m	30	N
	S5919	3.4m	30	N
60N	S5935	-17.7m	23	N
	S5905	-4.6m	40	Y
	S5892	5.1m	42	N
	S5856	0.5m	48	N
	S5387	-2.2m	36	N
59N	S5292	-6.1m	50	N

	S5292	0.5m	30	N
	S5292	6.3m	35	N
	S5906	-3.2m	48	N
	S5891	5.3m	80	N
B9N	Damaged	2.7m	40	N
	Damaged	12.6m	40	N
62N	S5662	9	20	N
	S5934	-51.5m	22	N
	S5934	-48.2m	23	N
	S5579	-7.3m	1	N
	S5428	-10.7m	50	N
63N	S5939	7.8m	50	N
	S5939	0.6m	30	N
64N	S5561	14.1m	110	N
	S5561	7.3m	70	N
65N	S5667	31.2m	35	N
	S5667	52.1m	25	N
66N	S5822	10.6m	38	N
	S5822	12.8m	25	N
67N	S5823	29.2m	26	N
	5823	33.7m	20	N
1 Shaft	S5868	6m	70	Y
	S5868	9m	60	N
	S5868	3m	40	N
	S5868	8m	30	N
	S5868	23m	30	N
	S5909	20m	20	N
2 Shaft	S5897	8m	25	N
	S5897	8	15	N
	S5832	7.9m	100	Y
3 Shaft	S5829	8m	40	N
	S5829	10m	40	N
	S5829	13m	25	N
78 S	S5925	7.8m	60	N
	S5925	3.9m	50	N
	S5925	8m	45	N
	S5925	10.5m	30	N
	S5925	16.8m	40	N
	S5925	39.9m	60	N
77S	Damaged	9.7m	50	N
	Damaged	15.7m	35	N
	S5936	5.9M	30	N
	S5936	9.6M	40	N
	S5936	11.9M	30	N
	S5936	3.8M	35	N

	S5936	20.2M	60	N
	S5895	7.5M	30	N
	S5901	5.4M	40	N
	S5937	3.1M	30	N
75S	S5924	7.2M	90	N
	S5924	20.3M	50	N
	S5924	21.9M	100	N
	S5811	0.5	30	N
	S5810	4.8M	30	Y
	S5810	17.8M	40	N
	S5810	35M	80	N
72S	S5819	31.2M	100	N
71S	S5815	1.3M	80	N
	S5838	0.4M	60	N
70S	Damaged	2M	40	N
	S5860	6.9	30	Y
	S5899	11.8M	50	N
	S5899	21.5M	55	N
B10S	S5898	2.1m	80	N
	S5898	6.6m	30	N
	S5898	4.9m	50	N
	S5820	11.9m	40	N
	S5820	4.2m	35	Y
	S5589	8.8m	45	N
	S5638	14.4m	55	N

APPENDIX D: JOINT MAPPING DATA

D.1. North joint mapping data

NORTH SECTION: JOINT MAPPING DATA							
Section	**Panel name**	**Joint set No**	**Dip (deg)**	**Dip Direction (deg)**	**Average spacing**	**Average trace length**	**Comment**
N BELT 7N	N45N	Fault	75	135	-	12	0.2m throw
		J	85	60	1	15	Blocky ground due to intersection of joints with Sepentinite infilling
		J	60	270	2.5	20	
	N46N	Fault	70	135	5	13	0.1m throw
		J	90	40	2	20	
		J	65	250	1.5	5	
	N47N	J	85	120	6	15	Sepentinite infilling
		J	40	290	3	12	
		J	65	40	4	20	
	N48N	J	85	30	2	12	
		J	70	110	3	15	Sepentinite infilling
	N49N	J	90	60	3	12	

		J	55	150	1.5	10	Supported with 1.5m long roof bolts
	N50N	Fault	70	140	0.2	7	0.2m throw
		J	85	10	2	15	
	N51N	Fault	65	100	-	20	Minor fault
N BELT 8N	N52N	J	85	20	2.5	15	
		J	30	290	1.5	10	
	NB8N	J	90	15	2.5	10	Blocky ground
		J	55	120	3	8	
		J	70	270	4	22	Sepentinite infilling
	N53N	J	90	180	2	10	
		Fault	65	60	-	20	0.7m throw
	N54N	Fault	55	20	-	20	0.2m up trow
		J	90	130	2.5	20	
		J	70	290	1	15	
	N55N	Fault	60	120	-	18	0.6m upthrow
	N56N	J	85	200	3	18	
		J	65	80	1	15	Sepentinite infilling
	N57N	J	90	20	3	15	
		J	55	70	3.5	5	Sepentinite infilling
		J	65	200	2.5	20	Blocky ground

N BELT 9N	N58N	Faults	70	100	2	15	Minor faults (0.1m and 0.3m throw)
	NB9N	Fault	80	130	-	20	0.8m throw
	N59N	Fault	75	120	-	15	0.6m throw
	N60N	J	85	315	3	10	
		J	20	100	1.5	6	0.4m to 1.0m over break
		J	65	60	2	12	Panel span reduced to 8m
	N61N	Faults	65	135	3	20	0.48m and 0.3m throw
	N62N	Faults	70	130	4	9	LG6A displaced into the hanging wall
		J	85	30	1	15	
		J	60	270	3	7	Sepentinite infilling
	N63N	J	55	110	5	9	
N BELT 10N	N64N	J	90	120	1.5	9	
	NB10N	J	85	110	1.5	6	Chrome and pegmatoid (0.2m)lenses on the hanging wall
	N70N	J	75	85	2	10	Chome left on the hanging wall
NORTH SHAFTS	Shaft 1	J	75	90	1.5	9	Sepentinite infilling
	Shaft 3	J	85	10	2	18	
	N70N-S	J	65	100	6	20	Sepentinite infilling. Chrome left on the hanging wall
	N69N-S	J	90	20	1.5	15	Sepentinite infilling, protruding roof bolts

		J	75	100	2	12	
	N68N-S	J	40	130	1	10	Hanging wall overbreak (0.9m -1.3m). panel side reduced to 8m

D.2. South joint mapping data

SOUTH SECTION: JOINT MAPPING DATA							
Section	**Panel name**	**Geological structure**	**Dip (deg)**	**Dip Direction (deg)**	**Average spacing (m)**	**Average trace length (m)**	**Comment**
BELT 7N	46N	J	35	45	4	6	0.7m hanging wall over break. Protruding roof bolts along the joint
		Fault	65	85	-	12	
	48N	J	65	280	2	13	Sepentinite infilling
		J	60	270	1.5	8	
		J	70	290	2	10	
	50N	J	65	270	1.5	2	Sepentinite infilling. Chrome left on the hanging wall
BELT 8N	52N	J	85	180	1	9	
		J	30	50	5	15	Panel span reduced to 8m
		J	70	280	3	6	
BELT 9N	58N	Fault	70	290	-	7	
		J	60	125	3	10	Sepentinite infilling
		J	25	70	5	6	Dome structure

	59N	J	65	230	1.5	9	Sepentinite infilling
	62N	J	15	50	2.5	9	
BELT 10N	63N	Fault	70	150	-	9	0.5m throw
		J	85	30	3	15	
		J	35	60	1	2	Wedge formed
		J	65	280	7	5	
	64N	Fault	65	140	-	15	
		J	45	90	3	20	Wedge formed
		J	60	290	4	12	
		J	80	20	5	7.5	
	66N	J	70	85	2.5	10	Sepentinite infilling
		J	45	150	5	8	
		J	85	280	3.5	7	
	67N	Fault	65	130	-	5	
		J	70	180	1.5	12	Blocky ground
		J	45	90	0.5	4	0.5m over break
		J	85	40	2.5	8	
	68N	J	80	290	4	8	
		J	50	180	2.5	12	Chrome and pegmatoid on the hanging

							wall
		J	65	70	1	15	
South shafts	Shaft 2	J	80	45	2.5	12	
		J	65	280	4	6	Chrome left on hanging wall
	Shaft 3	J	80	10	3	1.5	
		J	65	100	2.5	12	
		J	50	290	0.8	3	Chrome left on the hanging wall
	81S	J	90	120	2.5	10	
		J	70	90	2	20	
		J	60	130	1.5	5	
	80S	J	75	80	2.5	10	Sepentinite infilling
		J	85	170	2	15	
	79S	J	90	25	3	20	
		J	30	100	4.5	4	Chrome on the hanging wall
		J	70	270	3	12	
BELT11S	78S	J	85	30	3.5	10	
		J	35	170	2	5	Dome structure
		J	45	305	4	15	
		J	60	80	2.5	9	

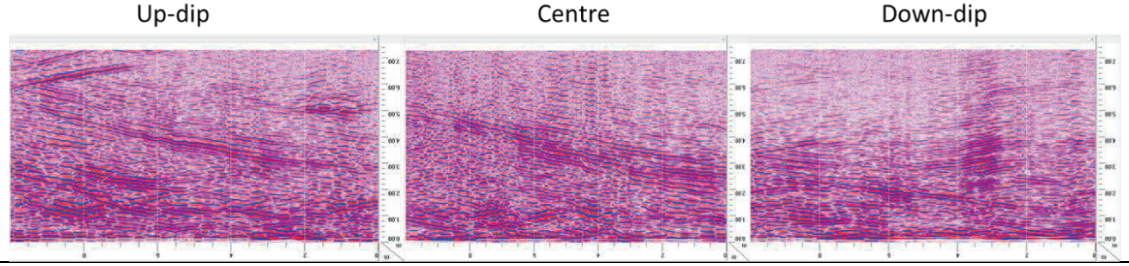
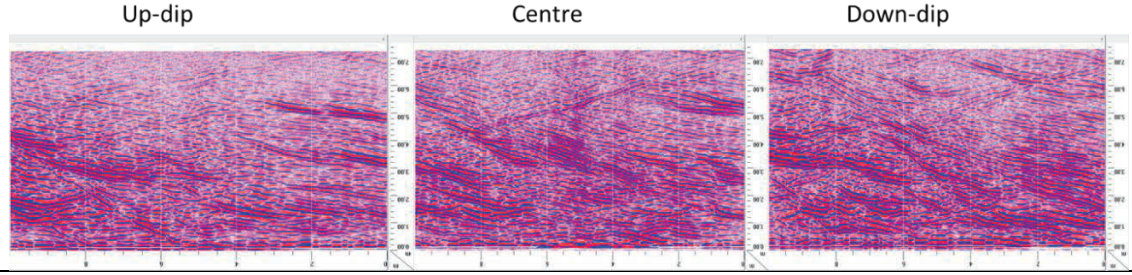
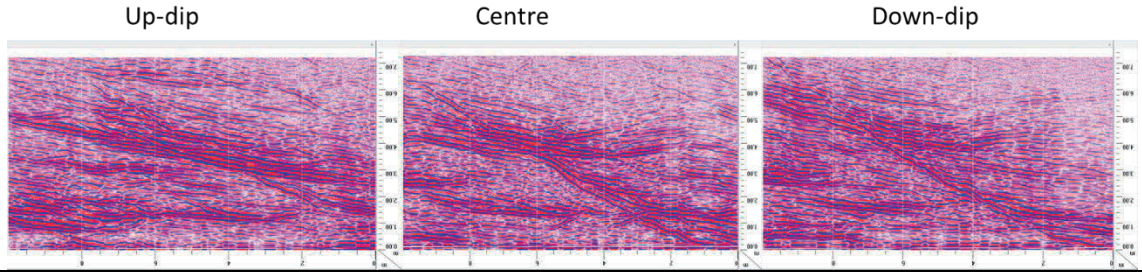
	77S	Fault	65	120	-	10	
		J	80	180	1.5	12	
		J	30	60	2	7	Dome structure
		J	65	280	1	10	
	76S	J	90	290	2.5	10	
		J	30	180	1.5	10	Dome structure
		J	60	70	2	7	
	B11S	J	85	180	3	5	Sepentinite infilling
		J	50	300	2.5	9	Blocky ground
		J	60	50	5	6	
	75S	J	90	30	2	9	Sepentinite infilling
		J	50	270	3	6	Chrome left on the hanging wall
		J	75	170	2.5	8	
	74S	J	85	30	1	6	
		J	30	270	3	10	0.3m hanging wall over break
		J	60	150	2.5	8	Sepentinite infilling

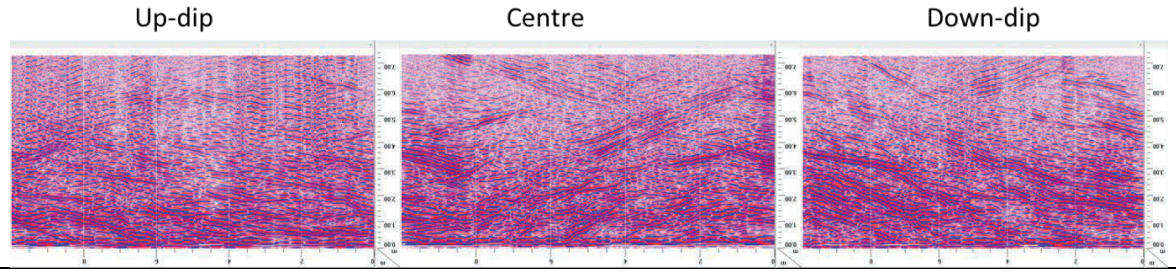
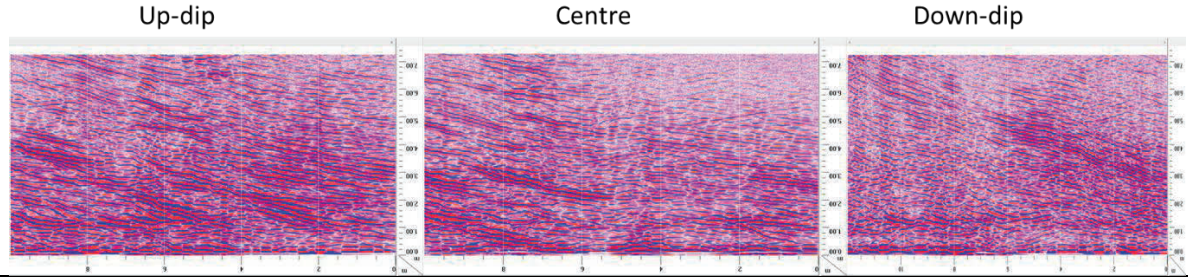
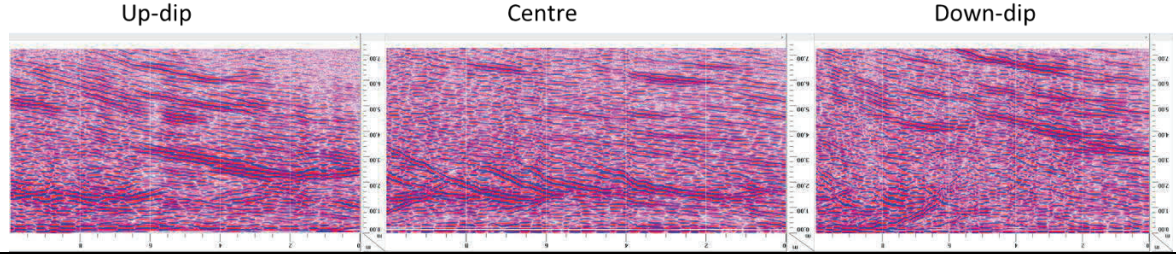
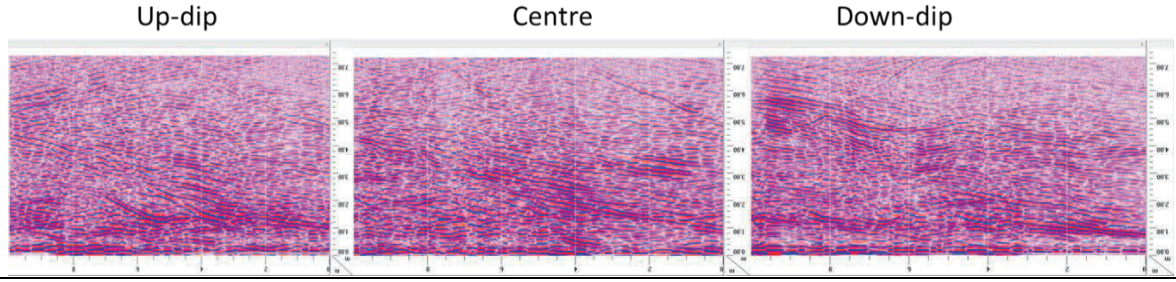
D.3. Scanline mapping joint data (74S back area)

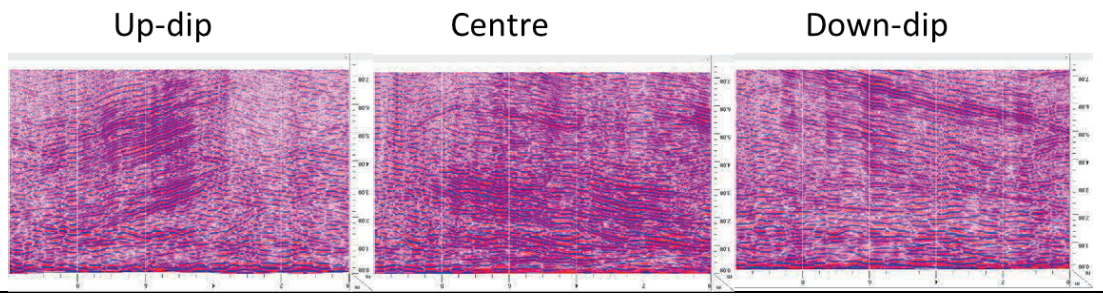
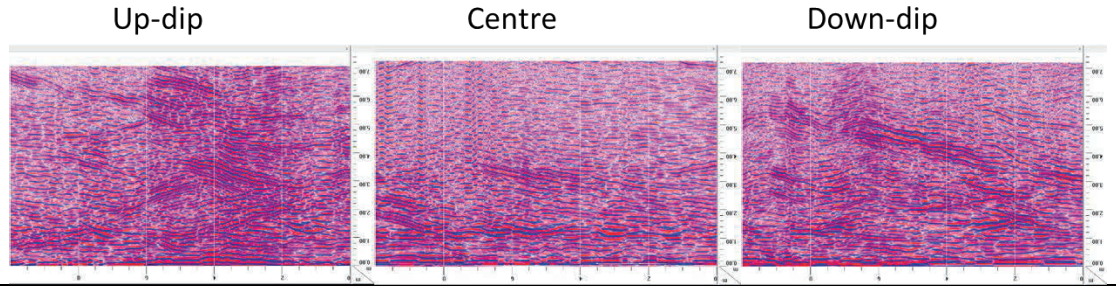
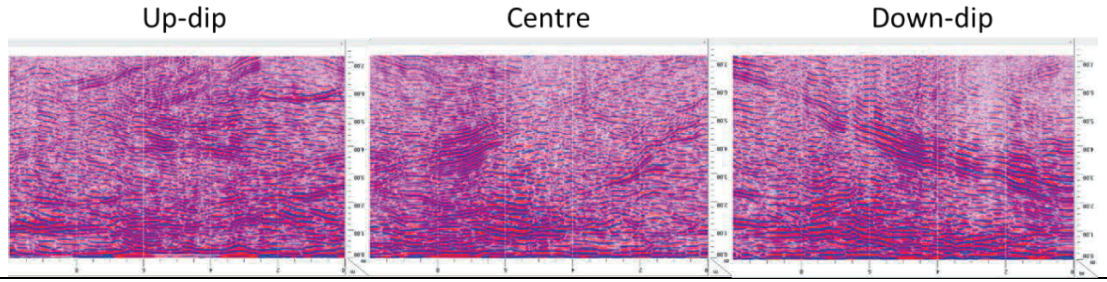
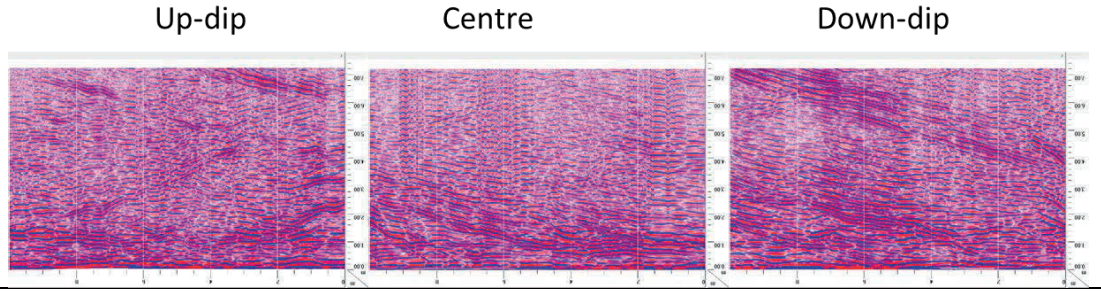
SECTION: SOUTH PANEL: 74S Back Area TAPE LENGTH: 30m										
SCANLINE TAPE ORIENTATION: NORTH/SOUTH (0/180) PANEL ORIENTATION: NORTH/SOUTH (0/180)										
Offset (m)	Geological structure	Dip (deg)	Dip Direction (deg)	Range (deg)	Spacing (m)			Joint Trace Length (m)		
					Min	Max	Average	Min	Max	Average
1.4	J	85	30	10	-	-	-	-	-	0.7
3.9	J	75	170	10	-	-	-	-	-	0.5
6.1	J	90	30	10	-	-	-	-	-	1.5
11.2	J	75	160	10	-	-	0.6	-	-	4.5
16.0	J	90	20	10	-		0.2	-	-	8.7
19.6	J	85	25	10	-	-	0.8	-	-	1.0

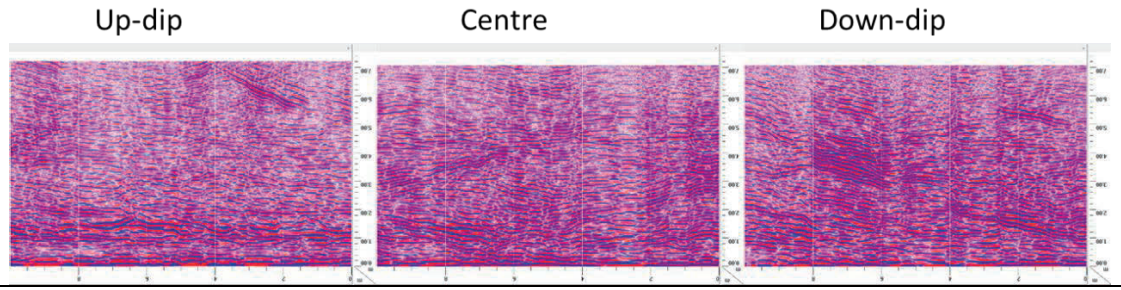
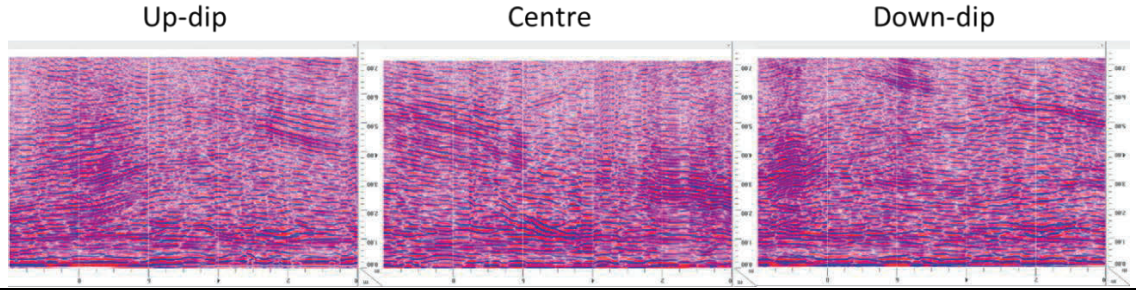
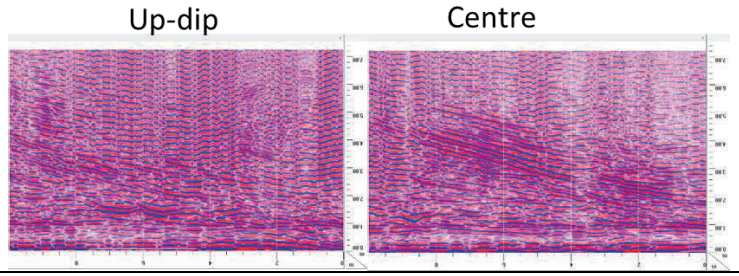
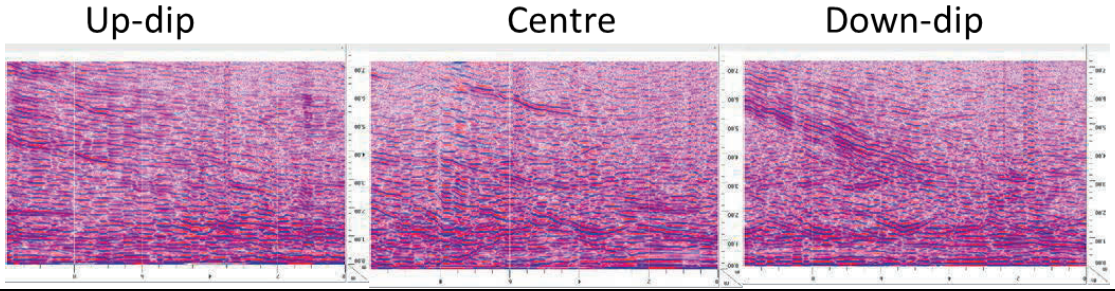
APPENDIX E: GPR SCANS

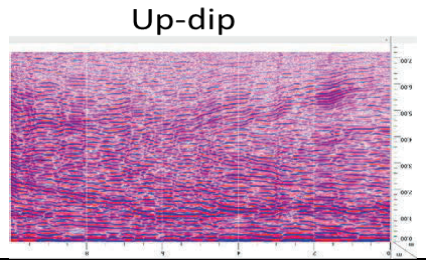
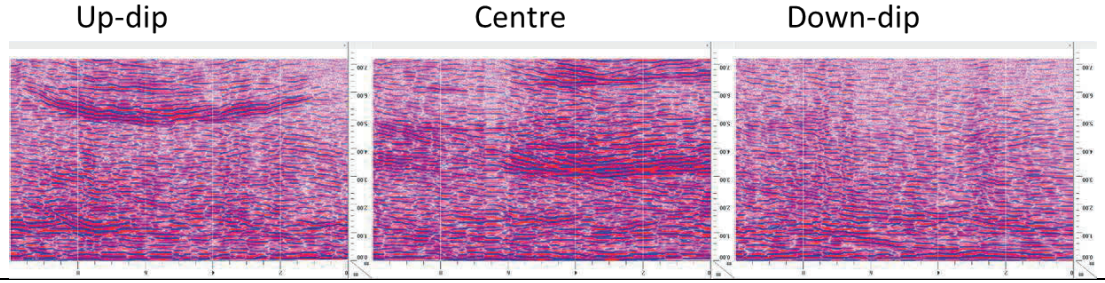
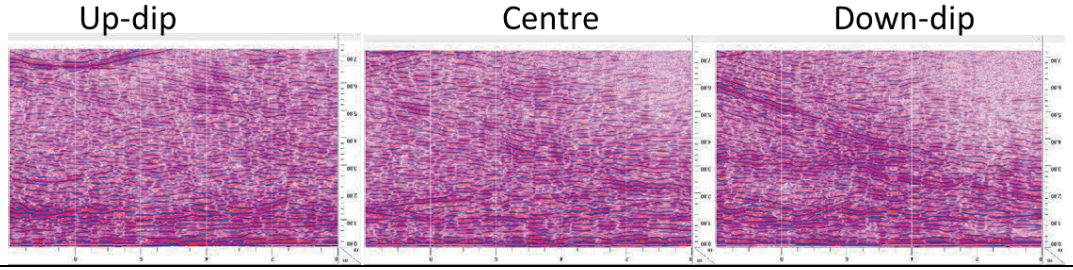
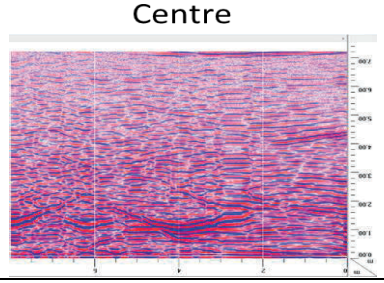
E.1. North Section GPR scans

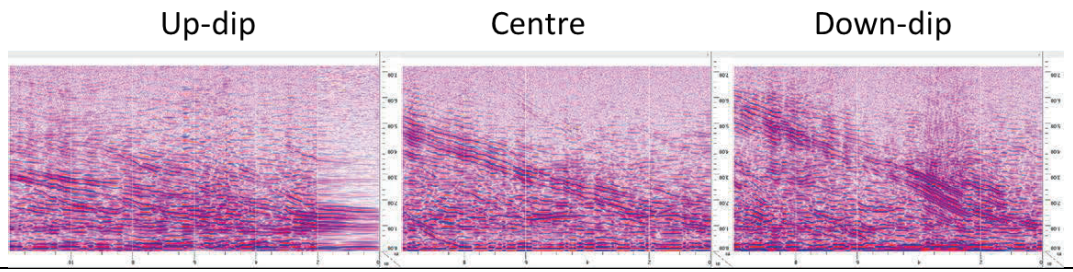
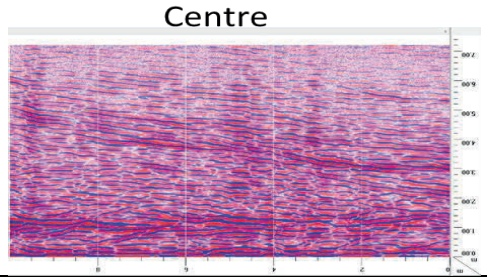
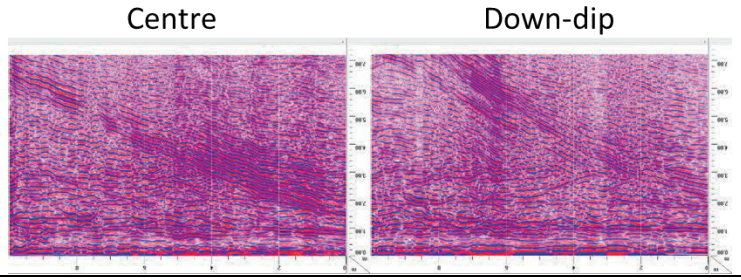
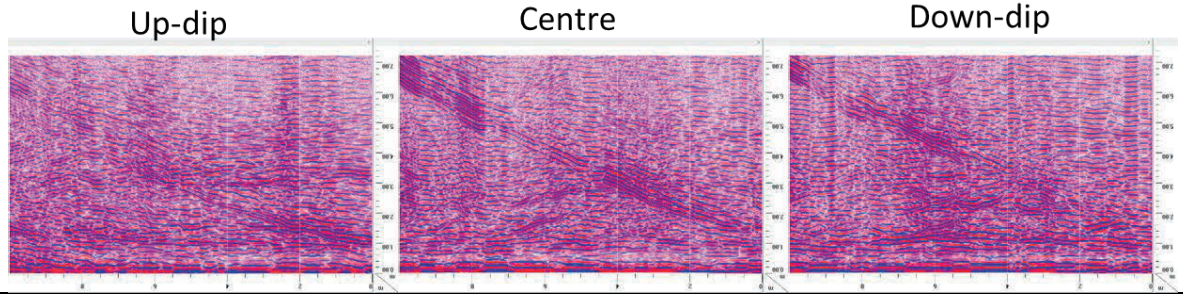
PANEL	COMMENT	GPR SCAN
N45N	Continuous and non- continuous flat dipping structures between 3m and 5m into the hanging wall.	
N47N	Non- continuous flat dipping structures in all three scans.	
NB7N	Continuous flap dipping planes. The plane depth increases towards the panel back area.	

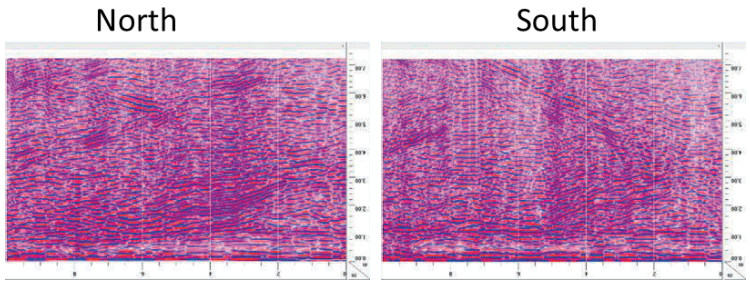
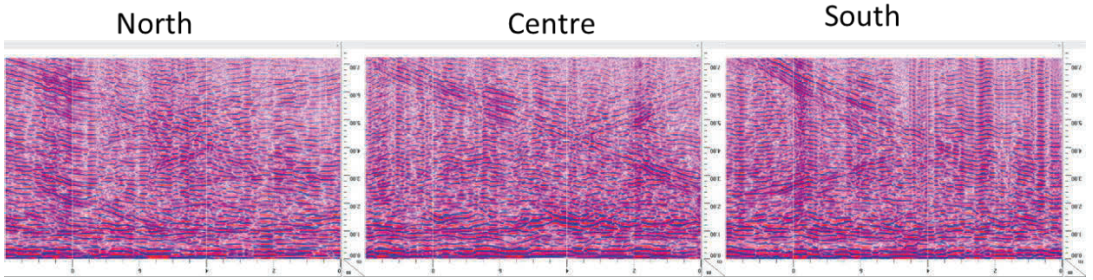
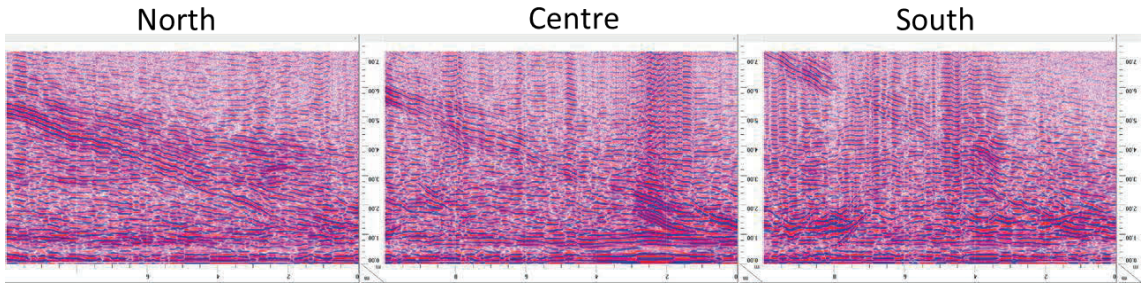
N48N	Discontinuous planes in the hanging wall throughout the panel.	
N50N	Non- continuous flat dipping planes in all scans.	
N51N	Continuous and non- continuous flat dipping planes in all scans.	
N52N	Non- continuous flat dipping structures	

NB8N	Non-continuous planes in the hanging wall	
N53N	Discontinuous planes in the hanging wall throughout the panel.	
N54N	Non- continuous flat dipping structures	
N55N	Non- continuous flat dipping planes at 4m -7m in the down-dip scan. A non- continuous structure between 0 and 2m in the hanging wall in both up-dip and center scans.	

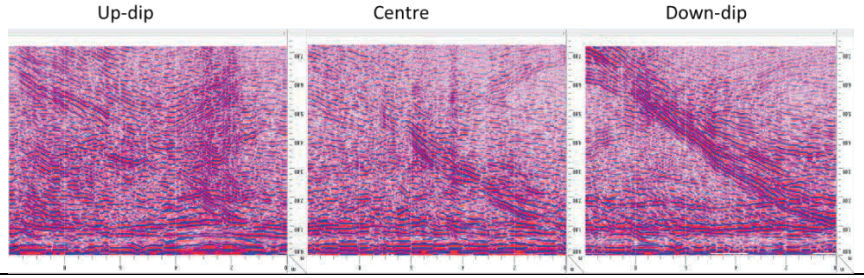
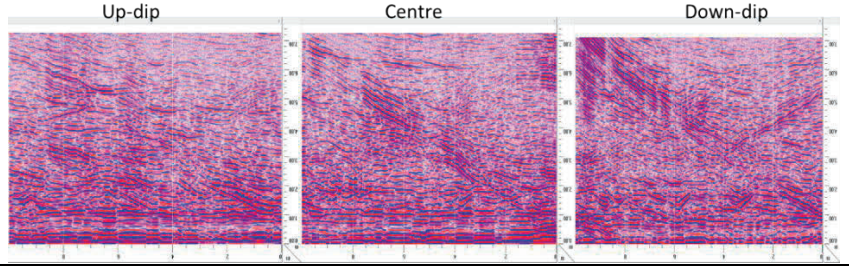
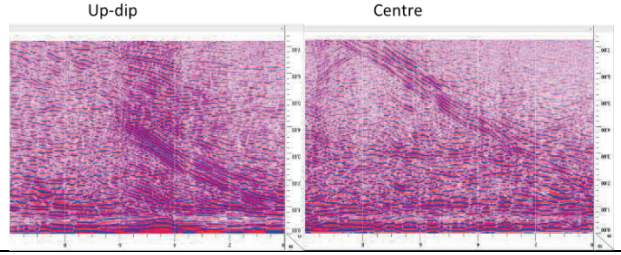
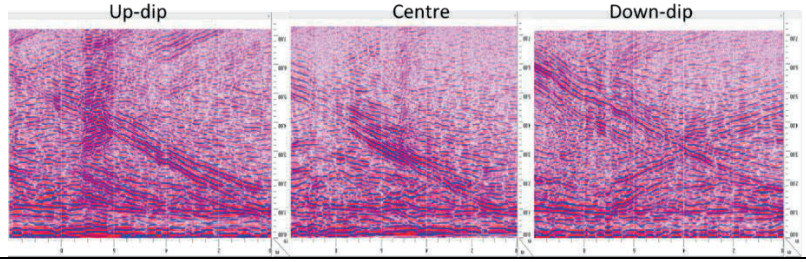
N57N	Discontinuous planes in the hanging wall throughout the panel. No major structures.	
N58N	Discontinuous planes in the hanging wall throughout the panel. No major structures.	
NB9N	No major structures. Discontinuous planes in the hanging wall throughout the panel.	
N59N	No major structures detected.	

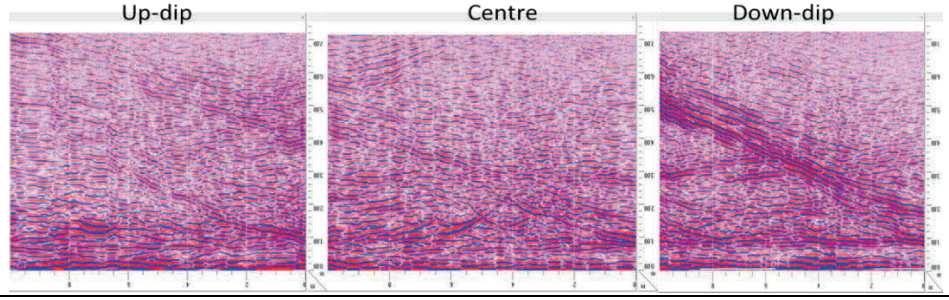
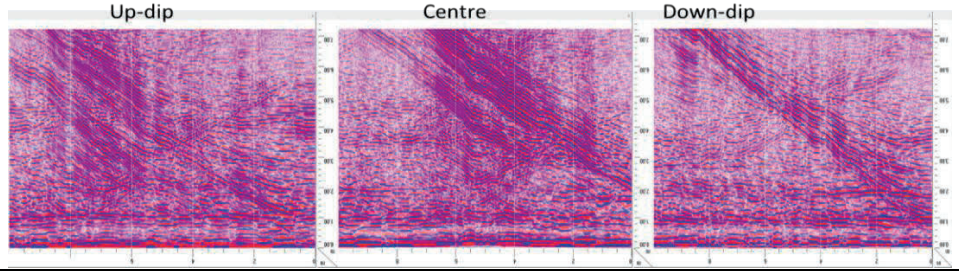
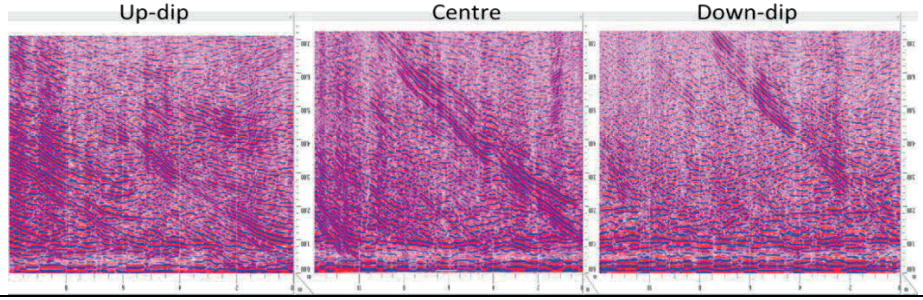
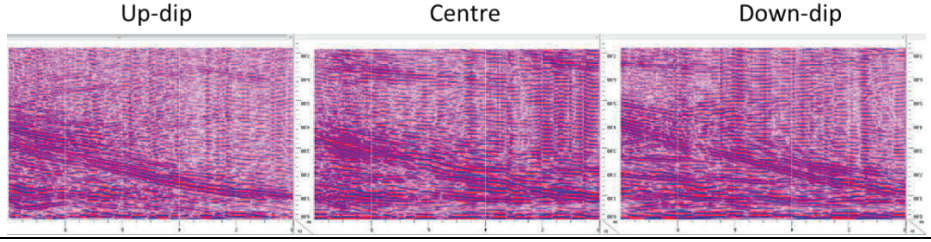
N60N	No prominent structures detected.	
N61N	Horizontal structures detected between 4m and 6m into the hanging wall in the up-dip and center potion of the panel.	
N62N	No major structures detected in the center and down dip scans. The up dip scan shows a similar flat dipping structure detected in the adjacent panel (N61N) at 7m. The structure is considered self-supported.	
N63N	Non- continuous horizontal structure detected between 1m and 2m in the immediate hanging wall.	

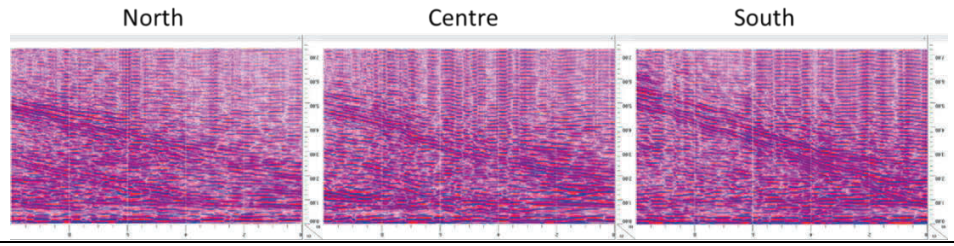
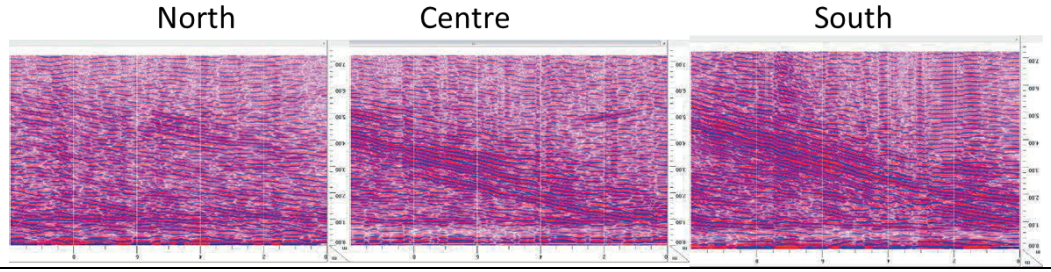
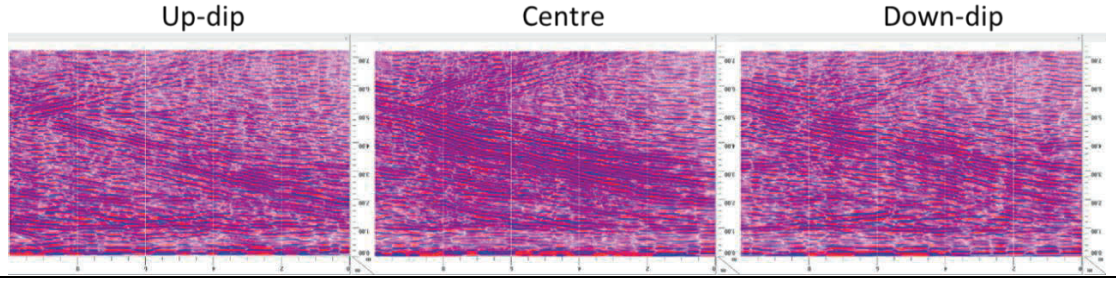
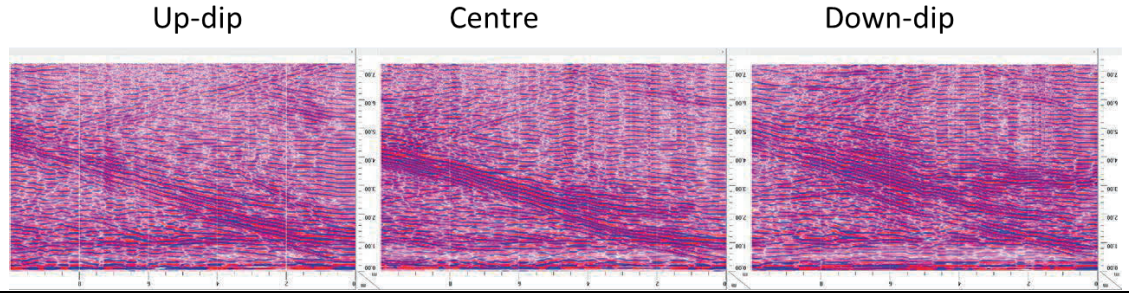
6N4N	Continuous flat dipping structure observed in all three scans.	
N65N	Continuous horizontal planes detected at 1.5m and discontinuous flat dipping planes between 3m and 5m.	
NB10N	Continuous flat dipping structure observed in the up-dip and center scans.	
N70N	Continuous flat dipping structure observed in both down-dip and center scans.	

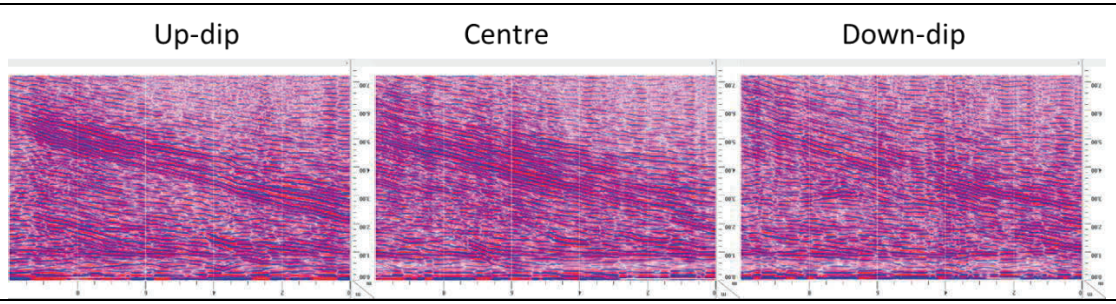
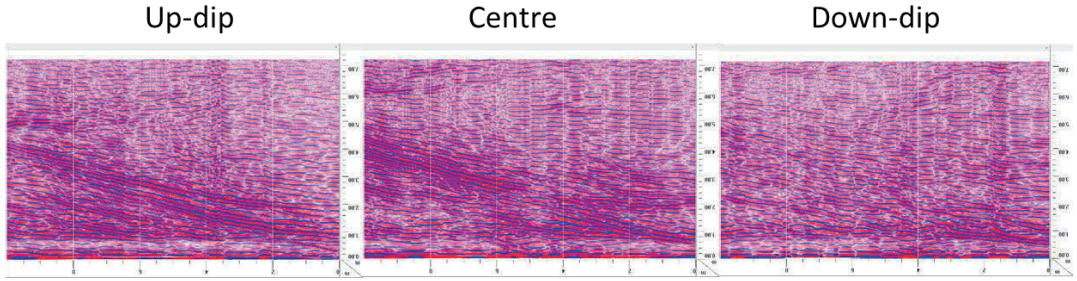
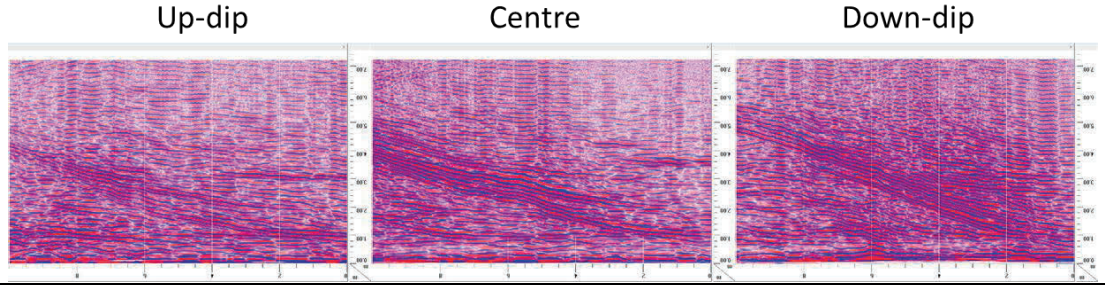
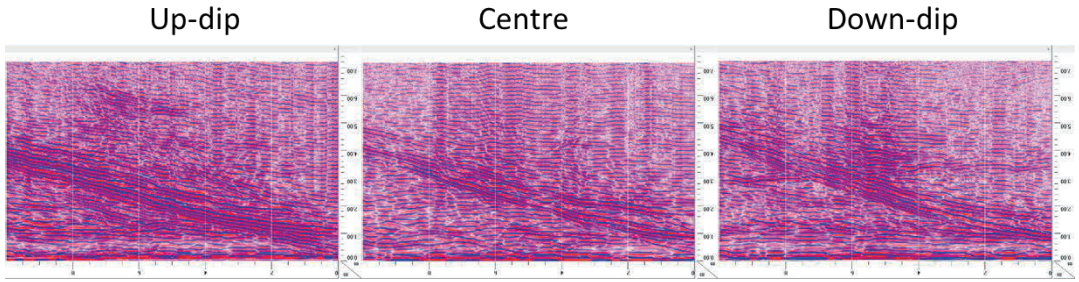
Shaft 1N	No prominent structures were detected.	
Shaft 2N	Non- continuous flat dipping structure observed between 3.5m and 7m.	
Shaft 3N	Non- continuous flat dipping structure observed in all three scans.	

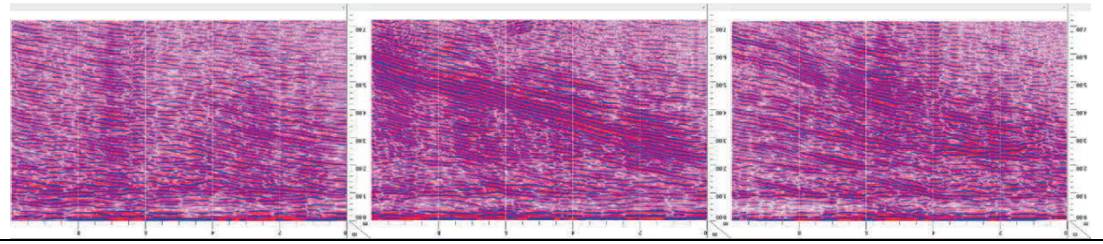
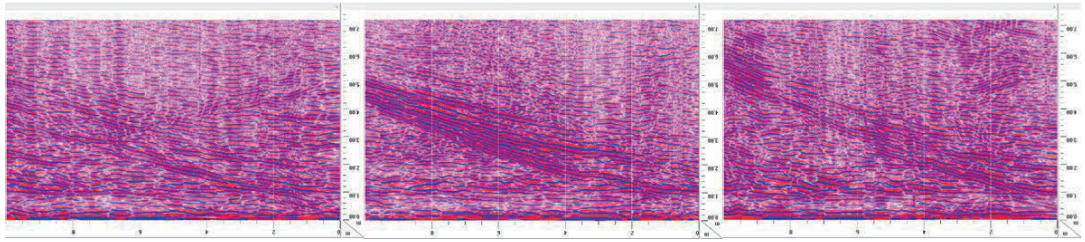
E.2. South Section GPR scans

PANEL	COMMENT	GPR SCAN
46N	Continuous flat dipping structure observed in the down dip scan. Centre and up-dip show non-continuity.	
47N	Continuous flat dipping structure prominent in the down dip scan.	
48N	Continuous flat dipping structure observed in both up-dip and center scans.	
49N	Continuous flat dipping structure observed in all three scans.	

B7N	Continuous flat dipping structure observed in the down-dip scan	
50N	Continuous flat dipping structure observed in the center scan.	
51N	Continuous flat dipping structure observed in the center scan.	
70N	Continuous flat dipping structure observed in the center scan.	

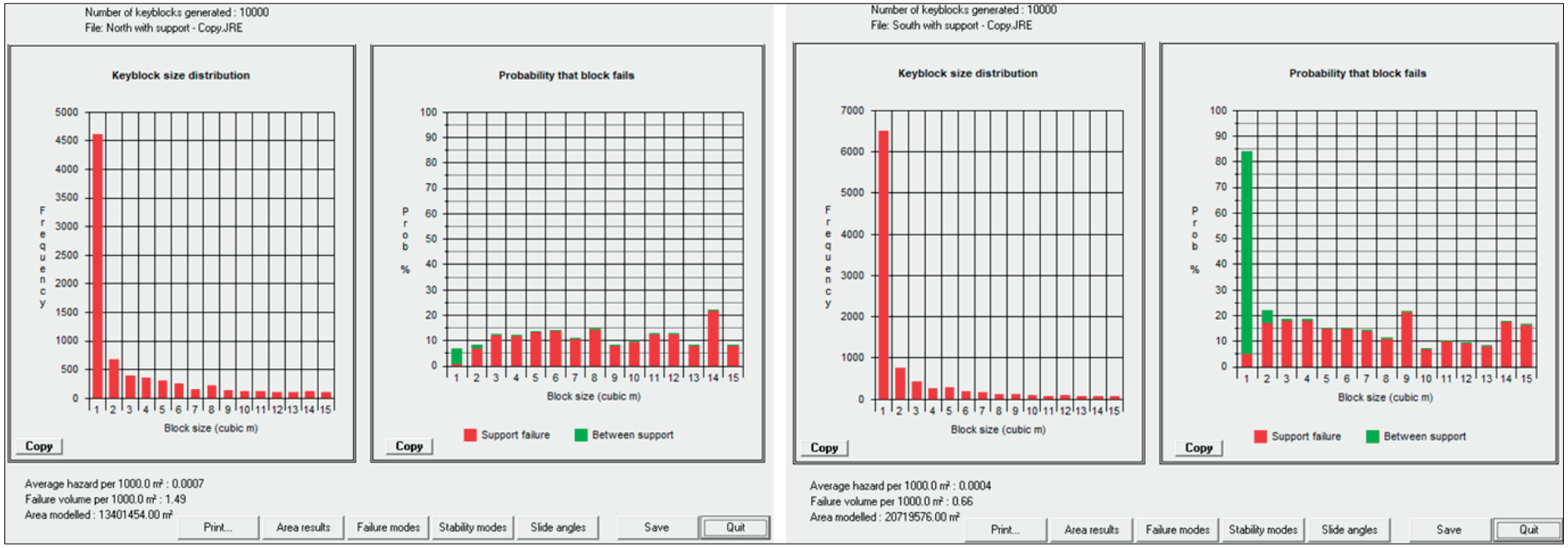
Shaft 1	Continuous flat dipping structure in all three scans.	
Shaft 2	Continuous flat dipping structure in the south and center scans	
81S	Continuous flat dipping structure in all three scans, but prominent in the center potion of the panel.	
80S	Continuous flat dipping structure in all three scans.	

79S	Continuous flat dipping structure in all three scans.	
77S	Continuous flat dipping structure in the up-dip and center scans	
76S	Continuous flat dipping structure in all three scans.	
B11S	Continuous flat dipping structure in all three scans.	

75S	Continuous flat dipping structure observed in the center scan.	Up-dip Centre Down-dip 
74S	Continuous flat dipping structure observed in the center scan.	Up-dip Centre Down-dip 
73S to 63S		NO SCANS DUE TO VENTILATION CONSTRAINS

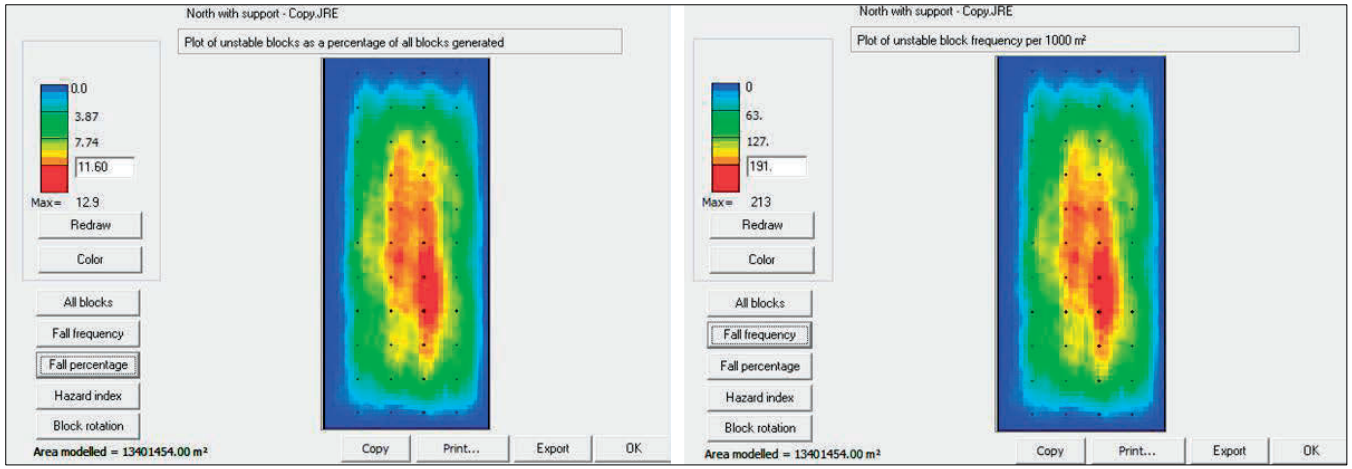
APPENDIX F: J-BLOCK ANALYSIS FOR UP-DIP HOLING

i. Key block distribution and Probability that blocks fall (North-left, and South –right)

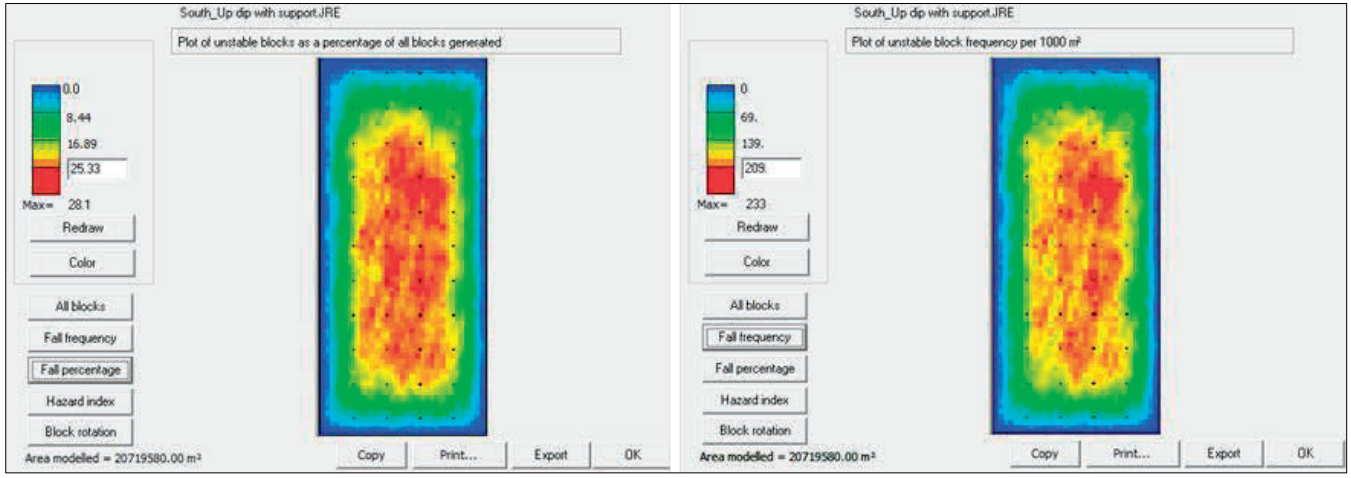


ii. Fall percenyage distribution and fall frequency

- North section

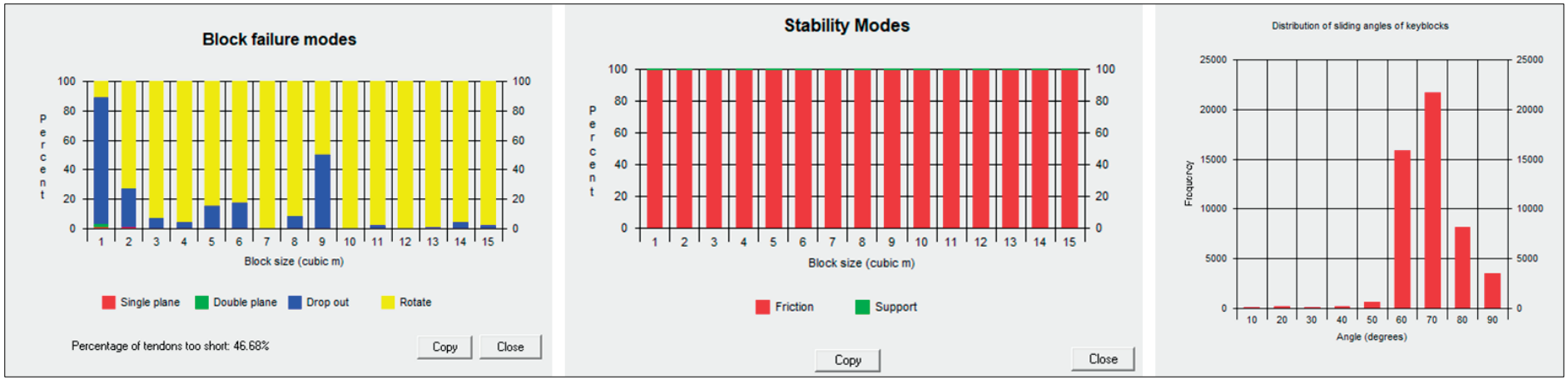


- South section



iii. Block failure mode, Stability modes and distribution of sliding angles of key blocks

- North section



- South section

