VALIDATION OF THE ROCK QUALITY TUNNELING INDEX, Q-SYSTEM, IN UNDERGROUND MINE TUNNELING ON A SOUTH AFRICAN PLATINUM MINE

by

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DECLARATION

All the information gathered including the literature survey and underground q-observations done in this thesis were compiled by myself. The fall of ground statistics were obtained when completing the mines code of practice and was further modified into a presentable format. All the interpretation and concluding remarks were completed by myself to substantiate the work. I thus declare that this thesis which I am submitting to the University of Pretoria for the Master's degree, represents my own work and has never been submitted by me to any other tertiary institution for any degree.

ABSTRACT

The South African mining industry has been dominated by experts on stope and tunnel support design for gold mines in the last 50 years. Little work to date has been done on the Bushveld Complex Platinum and Chrome Mines. Many questions still remain to date how to properly design support in a quasi-static environment using geological characteristics as an indicator and design tool. Many believe empirical means are best to establish design criteria for the Platinum and Chrome Mines. The question remains how to go about establishing a sound empirical approach to generate reliable design criteria.

In the platinum-mining environment poor rockmass support interaction has been associated with highly jointed and low friction rockmass structures, as well as the fall out of blocks between support units, where highly persistent vertical jointing is present.

This thesis will provide a simple approach in analysing existing critical rockmass parameters and provide information with an empirical validation method based on Barton's Rock Tunneling Quality Index, Q, for rockmass conditions found on a typical South African Platinum mine.

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TERMINOLOGY

Aperture	The perpendicular distance between adjacent rock surfaces of a discontinuity
Block size	Rock block dimensions resulting from the intersection of joint sets and resulting from spacing and orientation of the individual sets
Critical Bond length	That minimum bonded length of a particular tendon and grout combination that develops a pull-out resistance equal to that of the tensile strength of the tendon
Filling	Material that separates the adjacent rock surfaces of a discontinuity and that is usually weaker than the parent rock.
Joint	A break in the rock of a geological origin, not man made, along which there has been no visible displacement or movement.
Joint Set	A group of joints, which run parallel to each other
Joint System	If joint sets intersect they form what is called a joint system.
Persistence	The discontinuity trace length observed in an exposure Termination in solid rock or at other discontinuities reduces persistence. Describing the areal extent or size of a discontinuity within a plane
Random Joints	Joints which do not have the same orientation as the joint sets observed. They are not visible for long distances, only a couple of centimeters or perhaps meters
Rockbolt	Generic term for all types of inflexible rock reinforcement units, as well as to the process of rock reinforcement (e.g. Roofbolting)
Rock mass	In-situ rock, composed of small or large pieces of solid rock limited by discontinuities
Rockfall	Loosening or failure of rock from the rock mass

Rock reinforcement	The installation of rockbolts, cables or any other type of element in a rock mass to reinforce and mobilize the inherent strength of the rock, so that the rock becomes self-supporting. The rock reinforcement element is installed inside the rock mass, that is, it forms part of the rock mass
Roughness	The inherent surface roughness and waviness relative to the mean plane of the discontinuity
Seepage	Water flow and moisture visible in individual discontinuities or in the rock mass as a whole
Shotcrete	This is a mixture of cement, aggregate and water which is pumped pneumatically through a nozzle onto walls of an excavation to form a bonded coherent layer. It may contain admixtures, additives and fibres or a combination of these to improve tensile, flexural and shear strength, resistance to cracking
Tendon	Includes the generic "rockbolt", plus flexible forms such as "cable anchor"
Wall strength	The equivalent compression strength of the rock adjacent to the surface of a discontinuity

INTRODUCTION

The current industry regulations and guidelines call for systematic underground support that is capable of resisting 95% of all potential falls of ground as determined by statistical analysis. In the last 12 years the data obtained from fall of ground accidents (including fatalities) at Impala Platinum Mine is limited in off-reef excavations compared to the stoping horizon. The cost to support the excavations systematically to a 50kN/m² was therefore considered unacceptable. A more acceptable design criterion was required for the mine's problem. Various rockmass classification systems were suggested by numerous consultants. However the rockmass classification system is just intended for the use or application to the specific problem identified, therefore further research was necessary to ensure that a design system or rockmass classification system leading to a bolt design system is applicable to the Impala problem.

Impala Platinum Mine is situated 23km North of the town Rustenburg and covers approximately 25 km on strike from the most southern to the most northern shaft. The mine is currently mining the Merensky reef and the UG2 reef for platinum group metals and various other by products. Both the Merensky reef and the UG2 reef are part of the Bushveld Igneous Complex. The Merensky reef consists of pyroxenite and pegmatoid units and the UG2 Chromitite seam consists of chromitite and pegmatoid units. The Merensky reef overlays the UG2 Chromitite seam by 60m in the north , increasing to a 130m middling towards the south. The general strike of these orebodies is north-northwest to south-southeast. Local variations in the orebodies can lead to an east-west strike.

The average stoping width mined on the Merensky reef is 1,16m and on the UG2 reef 0,91m. Impala Platinum Ltd. mining depths ranges from 30m to 1200m, with the current mean rock breaking depth of the Merensky reef being 700m below surface and the UG2 mean rock breaking depths at 500m below surface.
Impala produces platinum, palladium, rhodium and nickel and their contribution to mine income is approximately 50%, 22%, 16% and 6% respectively. To ensure that the most current rock mechanics and strata control principles are applied for the safe and

economic design of all mine workings a centralised Rock Engineering function is employed, with it being split into a projects section and an operational section. The operational section's main activities consist of planning and design, risk assessment and strata control. The projects section main activities consist of life of mine design, large excavation stability, new mine prospects and seismic network analysis.

With mine tunneling on Impala throughout the 13 shafts it is impossible to visit each and every development end on a regular basis. Therefore the Impala strata control wing consisting of strata control officers and strata control observers which are mainly functional in the area of information gathering upon which support recommendations are generated. To ensure that these recommendations are made promptly it must be supported by a sound rockmass classification system.

The study of the stability of tunnels in rock is basically a strata control problem in the field of Rock Mechanics and assumes that the rockmass is anisotropic, heterogeneous and discontinuous in nature and that failure tends to be confined to structural discontinuities in shallow tunnels. Rational analysis of tunnel stability in materials with such properties requires that certain geological propositions which are necessary before definition of properties of the tunnel stability can be described, are adopted : (1) that structural discontinuities are detectable and their physical characteristics can be described quantitatively, (2) that within the whole mass it is possible to define smaller masses with similar jointing, (3) that a reliable model representing jointing of a rockmass can be constructed and (4) that the surface of failure will be plane or combinations of planes (Piteau, 1971).

Prerequisite to such an analysis is a qualitative and quantitative deduction of the geology, particularly of the attitude, geometry and spatial distribution of the discontinuities. Since the significant physical and mechanical properties of the mass are, for a large part, a function of the discontinuities, the basic principles on which the studies depend are therefore (1) the systems of jointing, (2) their relationship to possible failure surfaces and (3) strength parameters of the joints. There is an additional very important factor, namely water pressures in the joints. Other factors such as mineralogy, lithology and weathering, high horizontal stresses of tectonic or other origins, natural conditions of tunnels that occur in the vicinity in the same rockmass as the proposed tunnels and effects of time on reduction of strength together with the size and shape of tunnels must also be considered (Piteau, 1971).

Whether a tunnel will be stable or unstable in the same rockmass will depend on the margin by which the forces that tend to resist failure exceed those that tend to cause failure. The stability of tunnels in a stratified rockmass depends largely upon the presence of and nature of the discontinuities within the rockmass.

An underground excavation is an extremely complex structure and the only theoretical tools which the rock engineer has available to assist him in his task are a number of grossly simplified models of some of the processes which interact to control the stability of the excavation. These models can generally only be used to analyse the influence of one particular process at a time, for example, the influence of structural discontinuities or of high rock stresses upon the excavation. It is seldom possible theoretically to determine the interaction of these processes and the rock engineer is faced with the need to arrive at a number of design decisions in which his engineering judgment and practical experience must play an important role (Hoek & Brown, 1980).

Sometimes a project will be fortunate to have an experienced rock engineer on staff who has designed and supervised the construction of underground excavations in similar rock conditions to those being considered, these design decisions can be taken with some degree of confidence. Where no such experience is readily, what criteria can be used to check whether one's own decisions are reasonable ?. The answer lies in some form of classification system which enables one to relate one's own set of conditions to conditions encountered by others. Such a classification system acts as a vehicle which enables a rock engineer to relate the experience on rock conditions and support requirements gained on other sites to the conditions anticipated on his own site (Hoek & Brown, 1980).

A Rockmass classification scheme is intended to be used to classify the rockmass during feasibility and the preliminary design stages of a project (Hoek, 1998). At its simplest this may involve using the classification scheme as a checklist to ensure all relevant information has been considered. Use of a rockmass classification scheme does not (and cannot) replace some of the more elaborate design procedures.

Rockmass classification systems are still qualitative and empirical, rendering then inapplicable to all geotechnical situations. For example, a "poor" rock in a shallow tunnel in shale may need intensive support. A similar shale at greater depth may also be classified as "poor", but the in situ stress state may tend to clamp it, thereby not requiring the intensive support in the former case. Thus rockmass classification systems should be calibrated for every situation they are used in, just as they were for the situation they were developed in.

Relatively detailed information regarding in situ stresses, rock mass properties and planned excavation sequence is required at the initial stages of a project. As this information becomes available, the use of the rock mass classification schemes should be updated and used in conjunction with site-specific analyses.

Most of the multi-parameter classification schemes, like the Rock Structure Rating (Wickham et al, 1972), the Geomechanics Classification from Bieniawski (1973, 1989) and the Q-System from Barton et al (1974) were developed from civil engineering case histories in which all of the components of the engineering geological character of the rockmass were included. These schemes are directly applicable to mining, but many require alterations to suit conditions not yet encountered in civil engineering projects.

Empirical assessments of rock reinforcement and rockmass classification provide a useful supplement to any detailed analytical work. Empirical assessments can be very useful whenever adequate geotechnical data is unavailable for detailed structural analysis or whenever simplified analytical models are inapplicable (Stillborg, 1994).

The aim of this thesis is to contribute to the validation of rockmass classification methods in mining applications with specific reference to Barton's Rock Tunneling Quality Index, Q, which has been in use at Impala Mine since 1993. The classification scheme has been used for underground mine tunneling. This tunneling includes crosscuts, drives, large chambers, chairlift and conveyor decline excavations. The purpose of the thesis is to review rockmass classification systems available and in current use in the civil and mining industries and to compare these with the Q-System for their applicability and choose the best one suitable to the Impala problem. This validation of the Q-system will thus allow the reader to apply it with confidence to a similar geotechnical problem. The modifications required for the Q-System to make it suitable for application to Impala Mine are then discussed.

Chapter II

GEOLOGICAL SETTING AT IMPALA MINE

The lease area of Impala Platinum Mine lies on the western lobe of the Bushveld Complex (Figure 2.1).

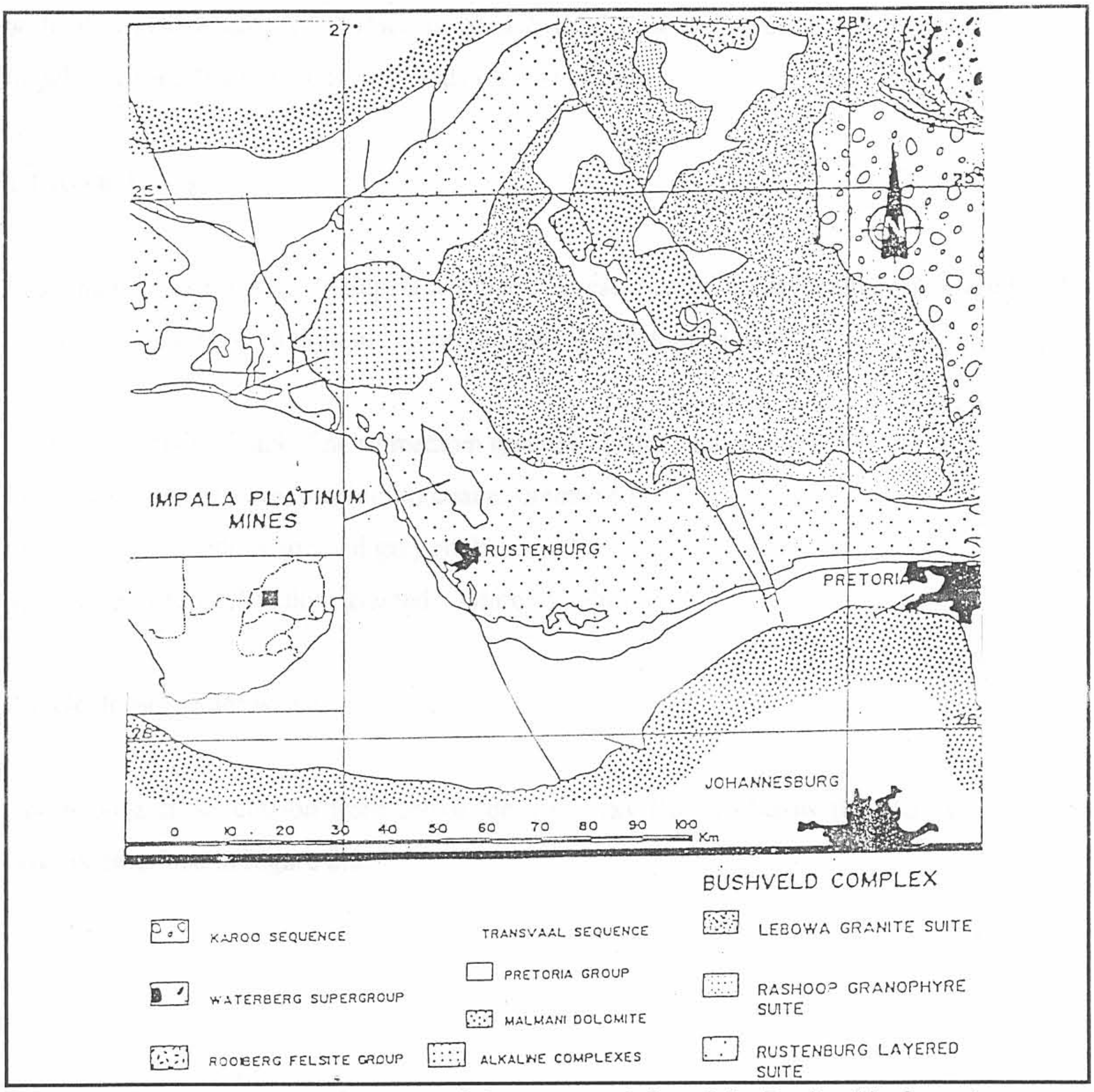


FIG. 2.1 - Locality plan - Geology of the western lobe of the Bushveld Complex showing Impala lease area

This Bushveld Complex is a large layered intrusion covering the central Transvaal. It consists of alternating layers of chromitite, pyroxenite, norite and a variety of anorthosite's which dip towards the centre at an average of 9 to 10 degrees, but this increases with depth. Strike at Impala is north-northwest to south-southeast, although locally east-west strikes can occur.

The combined lease area is 24km along strike. Two reefs, namely the Merensky and UG2, both of which outcrop on surface in places (Mellowship, 1996), are being exploited at Impala for their Platinum Group Metals (PGM) content.

2.1 Rock Types

Four main types are seen at Impala and these repeat themselves cyclically. These are listed in order from darkish to light in colour (increasing anorthosite content).

a) Chromitite is a black, fine to medium grained, tightly packed rock.
b) Pyroxenite is brown, medium to coarse-grained rock.
c) Norite is a medium grained grey rock.
d) Anorthosite is a medium grained – white to light grey rock.

2.2 Geological Succession

The geological succession from above the Merensky Reef to below the UG2 Chromitite layer is described in Figure 2.2.

Thickness (metres)	Name	Description
34,0	HW5	Mottled and Spotted Anorthosite
3-6	HW4	Large Spotted Anorthosite
5-7	HW3	Large mottled Anorthosite
1,5-3	HW2	Spotted Anorthosite Norite
2-6	HW1	Norite
2-3	Bastard Pyroxenite	*(Medium-coarse grained Pyroxenite may have thin Chromitite Layer at base)*
2-3	M3	Mottled Anorthosite
3-7	M2	Spotted Anorthosite *Norite (Characteristically layered towards top)*
0,5	M1	Norite *(Not well developed, grades into M2 and MR, Pyroxenite)*
1,0-1,5	Merensky Pyroxenite	*(Medium-coarse grained Pyroxenite)*
	Chromitite Layer	*(Pegmatoid usually has thin chromitite stringer at base then 2cm mottled Anorthosite Layer)*
0,4	FW1	Spotted Anorthosite Norite *(Maybe mottled at top)*
0,2	FW2	Cyclic *Unit (Pyroxenite-Spotted Anorthositic Norite-Mottled Anorthosite)*
3-5	FW3	Spotted Anorthositic Norite *(Often split into FW3(a) and FW3(b) by horizontal fault plane*
0,1-0,3	FW4	Mottled *Anorthoiste (Two Anorthositic Layers at base, separated by spotted anorhtositic norite)*
±1,0	FW5	Spotted Anorthositic Norite
1-3	FW6(a)	Mottled Anorthosite
1-3	FW6(b)	Large Spotted Anorthosite
1-3	FW6(c)	Mottled Anorthosite
1-3	FW6(d)	Mottled Anorthosite with Pyroxenite Boulders
		Thin Chromitite Layer with horizontal fault plane
35	FW7	Spotted Anorthosite Norite (Often greenish, chloritic partings towards top five poor ground, ±1,0m thick Olivine platy layer at top
0,8-1,2	FW8	Spotted Anorthosite
3-6	FW9	Mottled Anorthosite
3-5	FW10	Spotted Anorthositic Norite
12-15	FW11	Spotted Anorthositic Norite
10-12	FW12	Mottled Anorthosite (in places large spots) - 1cm Chromitite layer at contact
5-7	UG2 Pr	Pyroxenite with leader Chromite layers
0,7	UG2	Chromitite
0,7-1,0	UG2	Pegmatoid *(May include layers or patches of Pyroxenite)*
9-13	FW13	Spotted Anorthositic Norite *(in places Anorthosite)*

FIG. 2.2 - Impala Platinum Limited - Generalised Geological succession

A name system has been developed at Impala where the succession has been divided into distinct units with a number of marker units, with distinctive characteristics, used to facilitate this process. These units will be dealt with separately. The thickness of many of the units varies across the lease area with a general thinning occurring down dip and north (Figure 2.3).

2.2.1 Hangingwall units to the Merensky Reef

The Bastard Pyroxenite is a non mineralized pyroxenite layer lying approximately 10,0m above the Merensky Unit. Middling 3 is a whitish large mottled anorthosite of up to 3m in thickness. Middling 2 is a spotted anorthosite of approximately 3m in thickness. Middling 1 is a norite of 0,2m – 0,3m thickness. Higher hangingwall units than the Bastard Pyroxenite are rarely exposed in underground workings.

2.2.2 Merensky Reef

The Merensky reef refers to that portion of the Merensky unit and underlying footwall that is economically exploitable for PGM's. Three types of Merensky Reef can be identified depending on the Footwall unit directly underlying the reef.

The Pyroxenite reef has a basal chromitite layer (up to 3cm thick) resting directly on the footwall layers. A pegmatoid Reef has a pegmatoid below the chromitite layer that sometimes has a very thin chromitite layer at the contact with the footwall.

Because of the undulating nature of the reef and the tendency to cut through the footwall layers, locally a system has been developed to differentiate between the different reef settings. Merensky "A" reef describes the reef when resting on Footwall 1. Merensky "B" reef describes the reef when resting on Footwall 2. . Merensky "C" reef describes the reef when it has cut through Footwall 2. Deep Merensky "C" Reef describes the reef when it is resting on or below footwall 6. All of the above can be either a Pyroxenite or a Pegmatoid Reef.

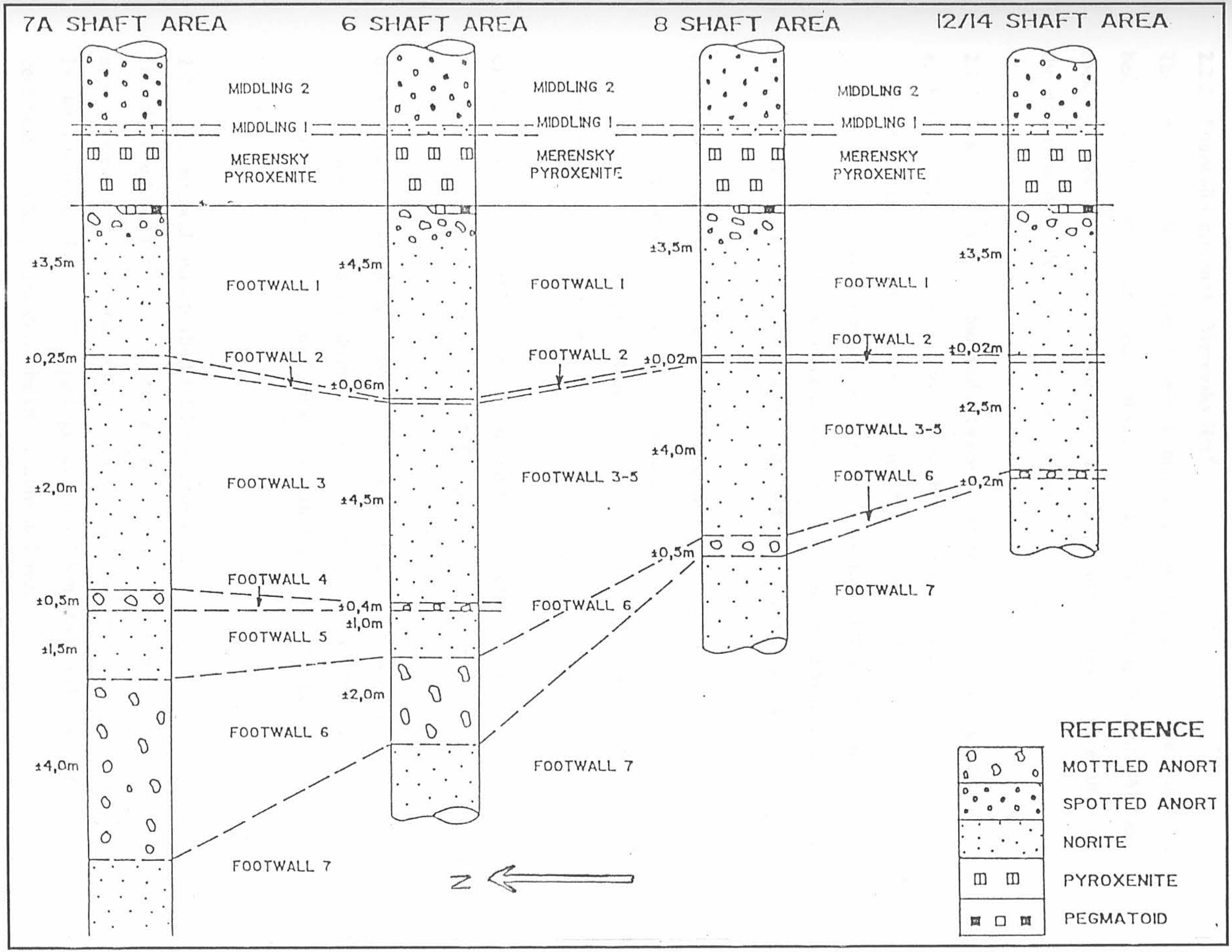


FIG. 2.3 - Section showing thinning of the footwall units at Impala

2.2.3 Footwall Units to the Merensky Reef

The footwall units to the Merensky unit are numbered from 1 to 12 with increasing depth before the UG2 unit is intersected. Footwall's 1,3,5 and 7 are all basically norite. All these rocks will look the same in hand specimens in identification problems. Marker units are therefore essential in allowing sub-division to occur.

2.2.4 Marker Units in the footwall between the Merensky and UG2 (Figure 2.3).

a) **Footwall 2** consists of three distinct layers which are always present despite varying thickness. The top layer is a pink to white anorthosite that grades downward into a layer of spotted anorthosite. The bottom layer is a very dark pyroxenite. This unit has an average thickness of 12cm but can be as little as 1cm in some areas where the spotted anorthosite portions is poorly developed.

b) **Footwall 4** is usually represented by two thin pink white anorthosite layers (2cm) separated by a zone of spotted anorthosite. This footwall is generally not developed in northern parts of Impala but a mud infilled shear is locally developed in its place.

c) **Footwall** 6 is a whitish, large mottled anorthosite with a thin chromitite layer usually associated with the top contact. Thickness can vary from 2cm to 60cm.

d) Near the top of **Footwall 7,** a very distinctive layer is usually present, in which dark greenish-black olivine and pyroxenite form bands. These bands vary from 0,2m to 1,4m in thickness and are called the Olivine Platy Norite Layers (O.P.L.'s).

2.2.5 Hangingwall units to the UG2 Chromitite Unit

Directly overlying the UG2 Chromitite Layer is the UG2 Pyroxenite. This unit is approximately 8,0m thick and contains a package of three chromitite layers called the Leader Chromitite Layers. This package averages 50cm thick and lies from a few centimeters to a few metres above the UG2 Chromitite Layer.

An erratically developed thin chromitite layer is sometimes developed between the UG2 Chromitite Layer and the leader Chromitite Layer and is called the Intermediate Chromitite Layer. Where developed, this layer can cause hangingwall parting where it is developed close to excavations.

2.2.6 UG2 Chromitite Unit

The UG2 Chromitite Layer is a well-defined 50 to 80cm (usually 60cm) thick layer with sharp contacts. Beneath the unit is a coarse-grained pegmatoid varying from 0 to 1,5m in thickness with an average of 50cm. The absence of this pegmatoid usually indicates potholing of the UG2 Chromitite layer.

2.2.7 Footwall units to the UG2 Chromitite Unit

The immediate footwall unit is Footwall 13, which is a spotted anorthosite and varies in thickness from 8,0m to 10,0m. Below this lies the UG1 unit which comprises a 6,0m to a 8,0m thick pyroxenite overlying a 1,0m thick chromitite layer called the UG1 Chromitite layer. This UG1 Chromitite Layer can split into two or more layers of up to a meter in width with lens like layers of either anorthosite or pyroxenite between them.

Beneath the UG1 unit is Footwall 16 which is an anorthositic layer containing numerous chromitite layers over the upper 2,5m These layers are irregular and vary in thickness from a few mm to several cm's.

2.3 Geological Structures

2.3.1 Potholing

Potholes occur when either reef horizon cuts through its footwall units and comes to rest on or in a lower unit than is normally the case. Several effects occur :

a) Different hangingwall or footwall units are exposed.
b) The reef dip changes.
c) An increase in joint density is usually associated with the pothole edge.
d) Parting planes in the hangingwall are moved closer to the hangingwall of the excavation. Where large-scale (deep) potholing occurs, the effect can also be noticed in drive and travellingway development.

2.3.2 Dykes

Dykes are sheet like intrusive rocks which are not parallel to the layering and have one of two possible mechanisms of intrusion. They are either forced into cracks or have created their own cracks due to pressure while in liquid form and have cooled in situ. The dykes could therefore have formed in areas where weaknesses were present prior to their intrusion and are indicators of potentially poor ground conditions while the dyke themselves may also contribute to the conditions of the ground.

Four main types of dykes are exposed in both stoping and development :

Pegmatite veins are white, coarse-grained intrusions on a centimeter scale and have dips of approximately 80 degrees. They can cause sidewall problems due to slabbing on the dyke, which is most evident in drives. They tend to be more common in the UG2 chromitite workings.

Lamprophyre dykes are medium to coarse grained with a shiny brown appearance (sparkles under cap lamp illumination) and vary in size from a few centimeters to the occasional dyke of a metre or more in thickness. Dips are normally near vertical and the trend is E-W across the lease area. These dykes are often friable and tend to deteriorate on exposure to air and water.

Dolerite dykes are dark green to black, fine to medium grained intrusions, usually several metres thick with a near vertical dip. They are blocky by nature and usually have well-developed sympathetic joint zones on either side.

*Dolerite sills are l*ocally dolerite intrusions which can also exhibit a flat dip (10-45 degrees) and are called sills. The flat dipping nature of these sills can have serious implications where these lie within 5,0m of the hangingwall of excavations and usually a restriction on mining in this region is imposed.

2.3.3 Faults

These are discontinuities in the rock along which the strata are displaced. The amount of the displacement is variable and can reach up to 150m. Two types of displacement occur, namely horizontally and vertically, sometimes a combination of the two can be observed.

The faults are usually infilled with soft material such as clays and form weak zones. The infilled material is usually more friable or likely to expand and cause parting when wet. Water and methane are sometimes associated with faulting, but water can also be introduced along this plane during washing and drilling operations. North and northwest trending faults are dominant and the dips encountered tend to dip at an average 70 to 80 degrees.

2.3.4 Joints

Joints are natural breaks in the rockmass, which may be infilled, and occur across the lease area. The density of jointing is significantly higher close to faults, dykes and pothole edges. The immediate hangingwall and footwall are broken up by joints and generally 3 joint sets can be identified although as many as 5 joint sets can occur. Joint directions can vary with dominant joint sets aligned on strike on some shafts but conversely the dominant joint set could be aligned on dip at other shafts. The mean dip angle appears to be within 15 degrees of the vertical, with a scatter of 25 degrees on either side. This general picture does not rule out the sporadic occurrence of low planar joints or sills.

There is a high incidence of low angle curve joints across the lease area which results in large falls of ground if not properly supported and early enough identified. They tend to extend into the hangingwall and can cause alteration of the surrounding rock. They tend to be hidden due to their flat dipping nature and can result in poor hangingwall conditions. While not always continuous, they can extend several meters into the hangingwall and are often difficult to identify. They are sometimes referred to as "cooling domes".

2.3.5 Replacement Pegmatoid

The most common type of replacement pegmatoid occurs as ultramafic pegmatoid. This is a shiny, black extremely coarse grained rock usually rich in magnetite. This can be confused with chromitite, but it is important to note that the magnetite occurs in irregular patches and does not form a uniform layer similar to the chromitite occurrence in the reef. In general, the replacement process seems to prefer the anorthositic rocks, but occurrences are known where the pyroxenite layers as well as part or all of the Merensky reef has been replaced.

Where the Merensky reef and/or the footwall has been replaced, but the chromitite layer is still unidentifiable, it becomes essential for mining purposes to know what type of reef has been replaced. Replacement Pegmatoid, because of its irregular and unpredictable nature, presents an awkward problem with respect to mining.

2.3.6 Dunite Bodies

Small magnetite-dunite pegmatite pipes or plugs are known to occur in the northern parts of the lease area. They are dark greenish – black colour with a fine to medium grained nature. These bodies are intrusive and displace the reef whilst also causing strike swings in the process. These bodies are often associated with replacement Pegmatoid.

2.4 Water

A feature of mining in the area is the low incidence of underground water. In the shallow parts of the mine the water inflow that does occur is connected to the surface water table.

2.5 Rock Strength

The Rock Strength of the rocks in the Bushveld Complex especially at Impala Mine vary throughout the lease area and vary through the different types of rock (see Table 2.1). The determination of the global mechanical properties of a large mass discontinuous in-situ rock remains one of the most difficult problems in the field of rock mechanics. Stress strain properties are required for use in the determination of the displacements induced around mine excavations, and overall strength properties are required (Brady & Brown, 1985).

TABLE 2.1 - The Uniaxial Compressive Strength on the Rock Strata Horizons directly above and below reef

	Wildebeestfontein North			Wildebeestfontein South			Bafokeng North			Bafokeng South		
	lowest	Highest	Average	lowest	Highest	Average	lowest	Highest	Average	lowest	Highest	Ave
asterd Merensky				62 MPa	154 MPa	106 MPa	85 MPa	118 MPa	106 MPa	100 MPa	168 MPa	142
Middling 3	68 MPa	160 MPa	110 MPa	91 MPa	123 MPa	107 MPa	135 MPa	149 MPa	145 MPa	104 MPa	199 MPa	168
Middling 2	74 MPa	110 MPa	99 MPa	103 MPa	151 MPa	129 MPa	123 MPa	166 MPa	142 MPa	97 MPa	152 MPa	120
Middling 1	NIL	NIL	NIL	92 MPa	145 MPa	120 MPa	90 MPa	108 MPa	99 MPa			
Pyroxenite	62 MPa	109 MPa	92 MPa	57 MPa	109 MPa	86 MPa	61 MPa	98MPa	76 MPa	127 MPa	148 MPa	135
Pegmatoid	43 Chrome band	136 MPa	96 MPa	30 MPa	One Only	30 MPa				51 MPa	152 MPa	87
Footwall 1				45 MPa	134 MPa	83 MPa	123 MPa	155 MPa	137 MPa	80 MPa	115 MPa	93
Footwall 2				138 MPa	One Only	138 MPa					One Sample	71
Footwall 3				72 MPa	109 MPa	96 MPa	82MPa	121 MPa	106 MPa	72 MPa	127 MPa	85
Footwall 4				112 MPa	136 MPa	126 MPa				83 MPa	184 MPa	143
Footwall 5	79 MPa	100 MPa	89 MPa	62 MPa	149 MPa	107 MPa	103 MPa	134 MPa	121 MPa			
Footwall 6	86 MPa	121 MPa	105 MPa				129 MPa	172 MPa	150 MPa	92 MPa	203 MPa	144
Footwall 7	77 MPa	114 MPa	96 MPa				115 MPa	138 MPa	126 MPa	97 MPa	134 MPa	113
Footwall 8							255 MPa	260 MPa	258 MPa			
Footwall 9										262 MPa	264 MPa	263
Footwall 10												
Footwall 11										176 MPa	246 MPa	211
Footwall 12							198 MPa	235 MPa	217 MPa	174 MPa	202 MPa	188
UG2 Pyroxenite							209 MPa	242 MPa	226 MPa		258 MPa	
UG2 Chromite							99 MPa	138 MPa	119 MPa		101 MPa	
UG2 Pegmatoid							133 MPa	209 MPa	171 MPa			
Footwall 13							169 MPa	213 MPa	191 MPa		244 MPa	
UG1 Pyroxenite							141 MPa	233 MPa	187 MPa			
UG1 Chromitite							66 MPa	141 MPa	104 MPa			
Footwall 16							94 MPa	218 MPa	156 MPa		251 MPa	

Because of the difficult of determining the overall strength of a rockmass by measurement, empirical approaches are generally used. An attempt to allow for the influence of rock quality on rock mass strength was made by Bieniawski (1976) who assigned Coulomb shear strength parameters, c and Φ, to the various rock mass classes in his geomechanical classification. The most completely developed of these empirical approaches is that introduced by Hoek and Brown (1980) who proposed the empirical rock mass strength criteria.

$$\sigma_{1S} = \sigma_3 + (m\sigma_c\sigma_3 + s\sigma_c^{\ 2})^{0.5} \tag{2.1}$$

Where σ_{1s} is the major principal stress at peak strength, σ_3 is the minor principal stress, *m and s*, are constants that depend on the properties of the rock and the extent to which it had been broken before being subjected to failure stresses, and σ_C is the uniaxial compressive strength of the intact rock material. Hoek and Brown (1980) estimated that the parameters *m and s* varied with the rock type and rock mass quality according to Table 2.2.

TABLE 2.2 - Approximate strength criteria for intact rock and jointed rockmasses (After Hoek & Brown, 1980)

Rock Quality (1)	Carbonite Rocks with well developed crystal cleavage (Dolomite, limestone and marble) (2)	Lithified argillaceous rocks, mudstones, siltstone, shale and slate) (3)	Arenaceous rocks with strong crystal cleavage (sandstone, and quartzite) (4)	Fine grained polymineralic igneous crystalline rocks, (andesite, dolerite diabse and rhyloite) (5)	Coarse grained polymineralic igneous and methamorphic rocks (amphibolite, gabbro, gneiss, granite, norite and quartz diorite) (6)
Intact rock samples – laboratory size rock specimens free from structural defects (CSIR rating 100+; NGI rating 500)	$\sigma_{1n} = \sigma_{3n} + \sqrt{7\sigma_{3n} + 1}$ $\tau_n = 0.816(\sigma_n+0.140)^{0.658}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{10\sigma_{3n} + 1}$ $\tau_n=0.918(\sigma_n+0.099)^{0.677}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{15\sigma_{3n} + 1}$ $\tau_n=1.044(\sigma_n+0.067)^{0.692}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{17\sigma_{3n} + 1}$ $\tau_n=1.086(\sigma_n+0.059)^{0.694}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{25\sigma_{3n} + 1}$ $\tau_n=1.220(\sigma_n+0.040)^{0.705}$
Very good quality rock mass – tightly interlocking undisturbed rock with unwqeathered joints spaced at 3m (CSIR rating 85; NGI rating 100)	$\sigma_{1n} = \sigma_{3n} + \sqrt{3.5\sigma_{3n}+0.1}$ $\tau_n = 0.651(\sigma_n+0.028)^{0.679}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{5\sigma_{3n} + 0.1}$ $\tau_n = 0.739(\sigma_n+0.02)^{0.692}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{7.5\sigma_{3n}+0.1}$ $\tau_n = 0.848(\sigma_n+0.013)^{0.702}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{8.5\sigma_{3n}+0.1}$ $\tau_n = 0.883(\sigma_n+0.012)^{0.705}$	$\sigma_{1n}= \sigma_{3n}+\sqrt{12.5\sigma_{3n}+0.1}$ $\tau_n = 0.998(\sigma_n+0.008)^{0.712}$
Good quality rock mass – fresh to slightly weathered rock, slightly disturbed with joonts spaced at 1-3m (CSIr rating 65; NGI rating 10)	$\sigma_{1n}=\sigma_{3n}+\sqrt{0.7\sigma_{3n}+0.004}$ $\tau_n = 0.369(\sigma_n+0.006)^{0.649}$	$\sigma_{1n}=\sigma_{3n}+\sqrt{1.0\sigma_{3n}+0.004}$ $\tau_n = 0.427(\sigma_n+0.004)^{0.683}$	$\sigma_{1n}=\sigma_{3n}+\sqrt{1.5\sigma_{3n}+0.004}$ $\tau_n = 0.501(\sigma_n+0.003)^{0.695}$	$\sigma_{1n}=\sigma_{3n}+\sqrt{1.7\sigma_{3n}+0.004}$ $\tau_n = 0.525(\sigma_n+0.002)^{0.698}$	$\sigma_{1n}=\sigma_{3n}+\sqrt{2.5\sigma_{3n}+0.004}$ $\tau_n = 0.603(\sigma_n+0.002)^{0.707}$
Fair quality rock mass – several sets of moderately weathered joints spaced ar 0,3-1m (CSIR Rating 44; NGI rating 1.0)	$\sigma_{1n}=\sigma_{3n}+\sqrt{0.14\sigma_{3n}+0.0001}$ $\tau_n = 0.198(\sigma_n+0.0007)^{0.642}$	$\sigma_{1n}=\sigma_{3n}+\sqrt{0.20\sigma_{3n}+0.0001}$ $\tau_n = 0.234(\sigma_n+0.0005)^{0.675}$	$\sigma_{1n}=\sigma_{3n}+\sqrt{0.30\sigma_{3n}+0.0001}$ $\tau_n = 0.280(\sigma_n+0.0003)^{0.688}$	$\sigma_{1n}=\sigma_{3n}+\sqrt{0.34\sigma_{3n}+0.0001}$ $\tau_n = 0.295(\sigma_n+0.0003)^{0.691}$	$\sigma_{1n}=\sigma_{3n}+\sqrt{0.50\sigma_{3n}+0.0001}$ $\tau_n = 0.346(\sigma_n+0.0002)^{0.700}$
Poor quality rock mass – numerous weathered joints spaced at 30-500mm with some gouge filling/clean waste rock (CSIR rating 23; NGI rating 0,1)	$\sigma_{1n}=\sigma_{3n}+\sqrt{0.04\sigma_{3n}+0.00001}$ $\tau_n = 0.115(\sigma_n+0.0002)^{0.646}$	$\sigma_{1n}=\sigma_{3n}+\sqrt{0.05\sigma_{3n}+0.00001}$ $\tau_n = 0.129(\sigma_n+0.0002)^{0.655}$	$\sigma_{1n}=\sigma_{3n}+\sqrt{0.08\sigma_{3n}+0.00001}$ $\tau_n = 0.162(\sigma_n+0.0001)^{0.672}$	$\sigma_{1n}=\sigma_{3n}+\sqrt{0.09\sigma_{3n}+0.00001}$ $\tau_n = 0.172(\sigma_n+0.0001)^{0.674}$	$\sigma_{1n}=\sigma_{3n}+\sqrt{0.13\sigma_{3n}+0.00001}$ $\tau_n = 0.203(\sigma_n+0.0001)^{0.684}$
Very poor quality rock mass numerous heavily weathered joints spaced less than 50mm with gouge filling/waste rock fines (CSIR rating 3; NGI rating 0.01)	$\sigma_{1n}=\sigma_{3n}+\sqrt{0.007\sigma_{3n}+0}$ $\tau_n = 0.042(\sigma_n)^{0.534}$	$\sigma_{1n}=\sigma_{3n}+\sqrt{0.010\sigma_{3n}+0}$ $\tau_n = 0.050(\sigma_n)^{0.539}$	$\sigma_{1n}=\sigma_{3n}+\sqrt{0.015\sigma_{3n}+0}$ $\tau_n = 0.061(\sigma_n)^{0.546}$	$\sigma_{1n}=\sigma_{3n}+\sqrt{0.017\sigma_{3n}+0}$ $\tau_n = 0.056(\sigma_n)^{0.548}$	$\sigma_{1n}=\sigma_{3n}+\sqrt{0.025\sigma_{3n}+0}$ $\tau_n = 0.078(\sigma_n)^{0.556}$

CHAPTER III

FALL OF GROUND STATISTICS IN TUNNELS AT IMPALA PLATINUM MINE

This analysis covers a five-year period from 1992 to 1996. In order to obtain meaningful results from the analysis, a sizable database of fall of ground accidents is required. To meet this requirement all the available reportable accident and lost time injury data were gathered and combined for the five-year period.

The following information was extracted from the accident reports for the analysis:

- Reef type
- Stope or Development.
- Depth below surface.
- Distance from face.
- Excavation size.
- Origin of the fall of ground. (Face, Hangingwall, Sidewall or Footwall.)
- Mechanism. (Buckling, Shear or Dead weight.)
- Size of fall of ground (Small, medium or large)
- Shape of fall (Block, dome, wedge or scaling).
- Dimension of fall of ground (Max. height, width, length, area, volume and weight.)
- Rock type.
- Proximity of major geological features (Faults, dykes, potholes and joint sets)
- Boundaries of the fall of ground (Joints, faults, dykes, chromitite layer).

The database was analysed looking at fall out heights. A 95% cumulative percentage cut-off limit was used, since it is accepted in the industry that the support system must be designed to prevent 95% of the falls of ground. This criterion will be adopted for analysis of the parameters pertaining to the fall of ground dimensions.

The database analysis was addressed using two approaches consisting of a quantitative statistical analysis then followed by an in depth detailed investigation of the accidents reports to extract any further useful information. The mean values calculated in the different quantitative statistical analyses are given in Table 3.1 with their respective 95% confidence limits.

The maximum dimensions were always measured with regards to the fall of ground size. For example a fall of ground with a wedge shape vertical cross-section, the maximum thickness is the measurement from the base to the apex of the wedge. The information was complied into a single database, which was later broken down into the four following databases for analysis:

⇒ ***Impala Mine***; the database containing all Impala Mine fall of ground accidents from 1992 to 1996.

⇒ ***Mine - Development***; the database containing all development fall of ground accidents on Impala Mine from 1992 to 1996.

⇒ ***Merensky - Development***; the database containing all the fall of ground accidents in Merensky Reef development from 1992 to 1996. This includes on and off reef development.

⇒ ***UG2 - Development***; the database containing all the fall of ground accidents in UG2 on and off reef development from 1992 to 1996.

TABLE 3.1 - Statistical Analysis of Falls of Ground accidents at Impala : '92-'96

Thickness – Dev (m)	
Mean	03148
Standard Error	0.058158175
Median	0.2
Mode	0.3
Standard Deviation	0.290790876
Sample Variance	0.084559333
Kurtosis	-0.12601531
Skewness	1.113088118
Range	0.88
Minimum	0.02
Maximum	0.9
Sum	7.87
Count	25
Largest (1)	0.9
Smallest (1)	0.02
Confidence Level (95%)	0.120032549

Areal Extent – Dev (m^2)	
Mean	2.0138
Standard Error	0.624213591
Median	0.98
Mode	0.06
Standard Deviation	3.121067953
Sample Variance	9.741065167
Kurtosis	7.843844574
Skewness	2.672870353
Range	13.49
Minimum	0.01
Maximum	13.5
Sum	50.345
Count	25
Largest (1)	13.5
Smallest (1)	0.01
Confidence Level (95%)	1.228313267

Length Dev (m)	
Mean	1.492
Standard Error	0.262419511
Median	1
Mode	0.3
Standard Deviation	1.312097557
Sample Variance	1.7216
Kurtosis	1.66300208
Skewness	1.23604191
Range	5.3
Minimum	0.1
Maximum	5.4
Sum	37.3
Count	25
Largest (1)	5.4
Smallest (1)	0.1
Confidence Level (95%)	0.541607141

Width – Dev (m)	
Mean	0.852
Standard Error	0.145644544
Median	0.7
Mode	0.2
Standard Deviation	0.728222722
Sample Variance	0.530308333
Kurtosis	2.463805994
Skewness	1.501189576
Range	2.9
Minimum	0.1
Maximum	3
Sum	21.3
Count	25
Largest (1)	3
Smallest (1)	0.1
Confidence Level (95%)	0.300595504

Weight Dev (Tons)	
Mean	3.108122
Standard Error	1.088052919
Median	0.873
Mode	#N/A
Standard Deviation	5.440264594
Sample Variance	29.59647885
Kurtosis	5.753835166
Skewness	2.3327567576
Range	22.31938
Minimum	0.00062
Maximum	22.32
Sum	77.70305
Count	25
Largest (1)	22.32
Smallest (1)	0.00062
Confidence Level (95%)	2.245630392

Volume – Dev (m^3)	
Mean	1.00262
Standard Error	0.350984812
Median	0.27
Mode	#N/A
Standard Deviation	1.754924062
Sample Variance	3.079758465
Kurtosis	5.753835166
Skewness	2.332767576
Range	7.1998
Minimum	0.0002
Maximum	7.2
Sum	25.0655
Count	25
Largest (1)	7.2
Smallest (1)	0.0002
Confidence Level (95%)	0.724396901

The development categories include on and off reef development. As the analysis broke the database down into reef horizons, stoping and development, a lack of data became a problem. The lack of data means low number of fatal accidents in development and no data available for ordinary falls of ground in development. Only the information describing the dimensions of the rockfalls for the period 1992 to 1996 will be analysed.

3.1 Results of the analysis of reportable and fatal fall of ground accidents from 1992 to 1996

The analysis was broken down into various categories for comparison purposes. The main purpose was to highlight the typical shape and size of falls of ground that need to be suitably supported in off-reef tunnel development. The analysis looks on a mine wide level, which will be focused on off-reef tunnel development, 34.5% of all reportable accidents occurred in development (i.e. off-reef tunnels, raises, re-raises, boxholes and travelway's) :

- 57.4% of the above occurred on the Merensky Reef Horizon.
- 41.6% of the above occurred on the UG2 Reef Horizon.
- the remaining 1% occurred during capital development projects (declines).

3.1.1 Size of falls of ground

The analysis consisted of 90 falls of ground representing 23.4% in off-reef tunnel development. This low number is due to the fall of ground dimensions not being recorded in every investigation report.

- Nearly all falls are discontinuity bounded, most commonly joints and chromitite layers
- Most falls of ground occur in the footwall of the Merensky Reef or the UG2 reef i.e. where the bulk of the mine tunnels are situated
- Falls confined to blocks, wedges or scaling are always discontinuity bounded.
- For length, width, height, weight, volume, areal and height a 95 cumulative percentage limit has been determined.

The following charts substantiate the above conclusions.

Figure 3.1 shows a cumulative percentage and histogram plot of the various thickness of reportable falls of ground accidents in mine development from 1992 to 1996. Thus 95% cumulative percentage of falls of ground thickness is 0.85m.

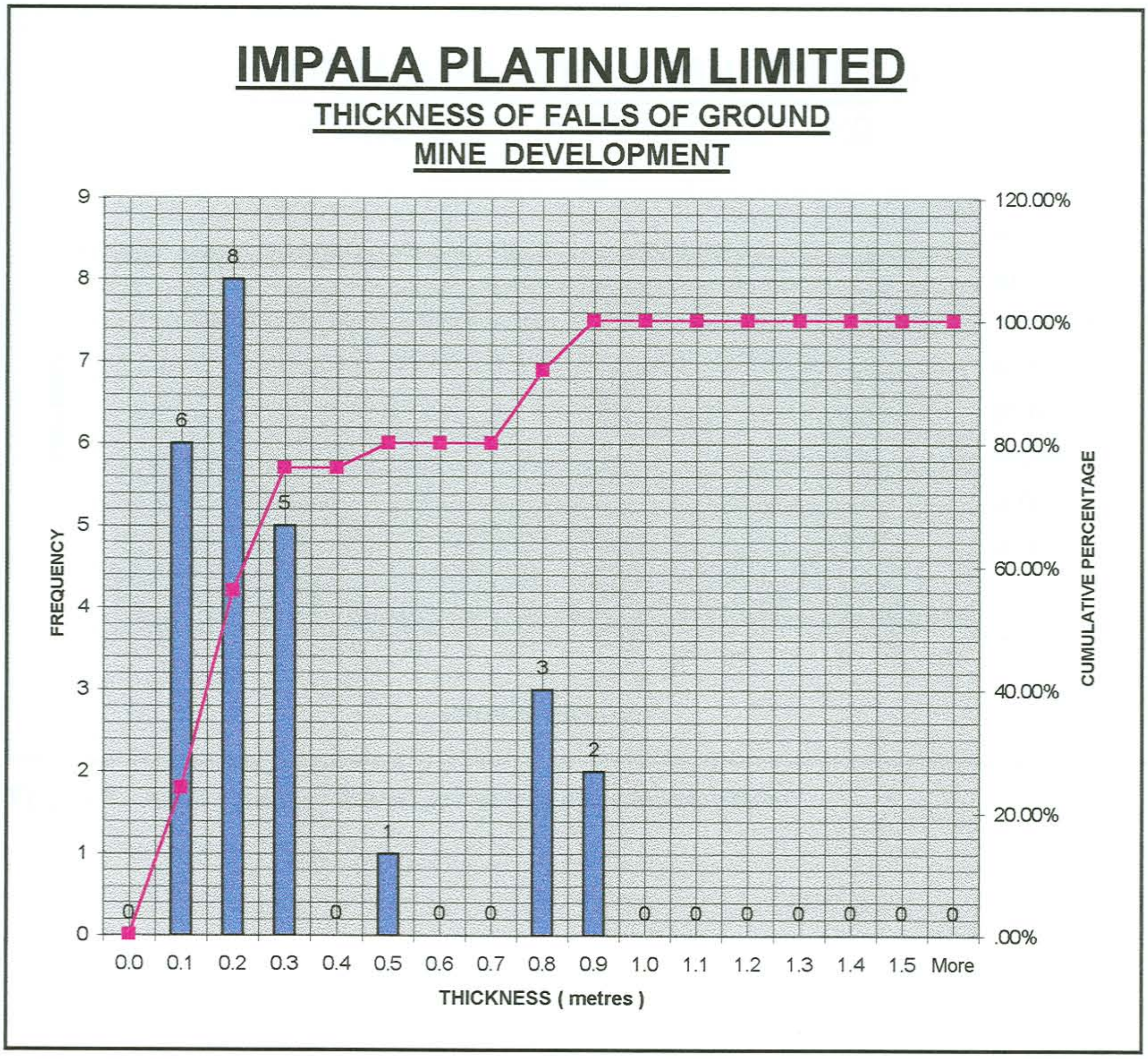


FIG. 3.1 - Fall out thickness in mine development

Figure 3.2 shows a cumulative percentage and histogram plot of the areal extent of reportable and fatal falls of ground accidents in mine development from 1992 to 1996. Thus the 95 cumulative percentage of falls of ground represent an areal extent of $9m^2$.

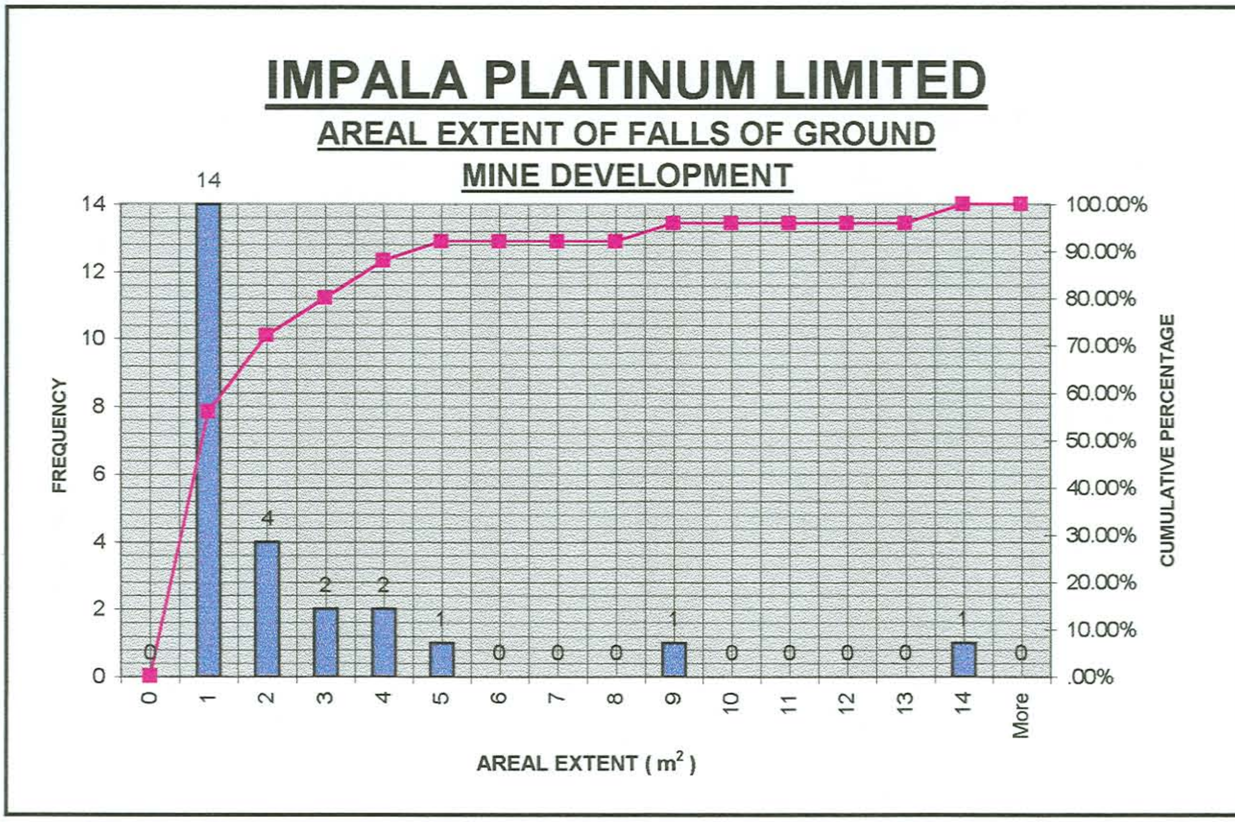


FIG. 3.2 - Areal extent of falls of ground in mine development

Figure 3.3 shows a cumulative percentage and histogram plot of the mass (kg's) of reportable falls of ground accidents in mine development from 1992 to 1996. Thus the 95 cumulative percentage of falls of ground represents a mass of 13 000 Kg.

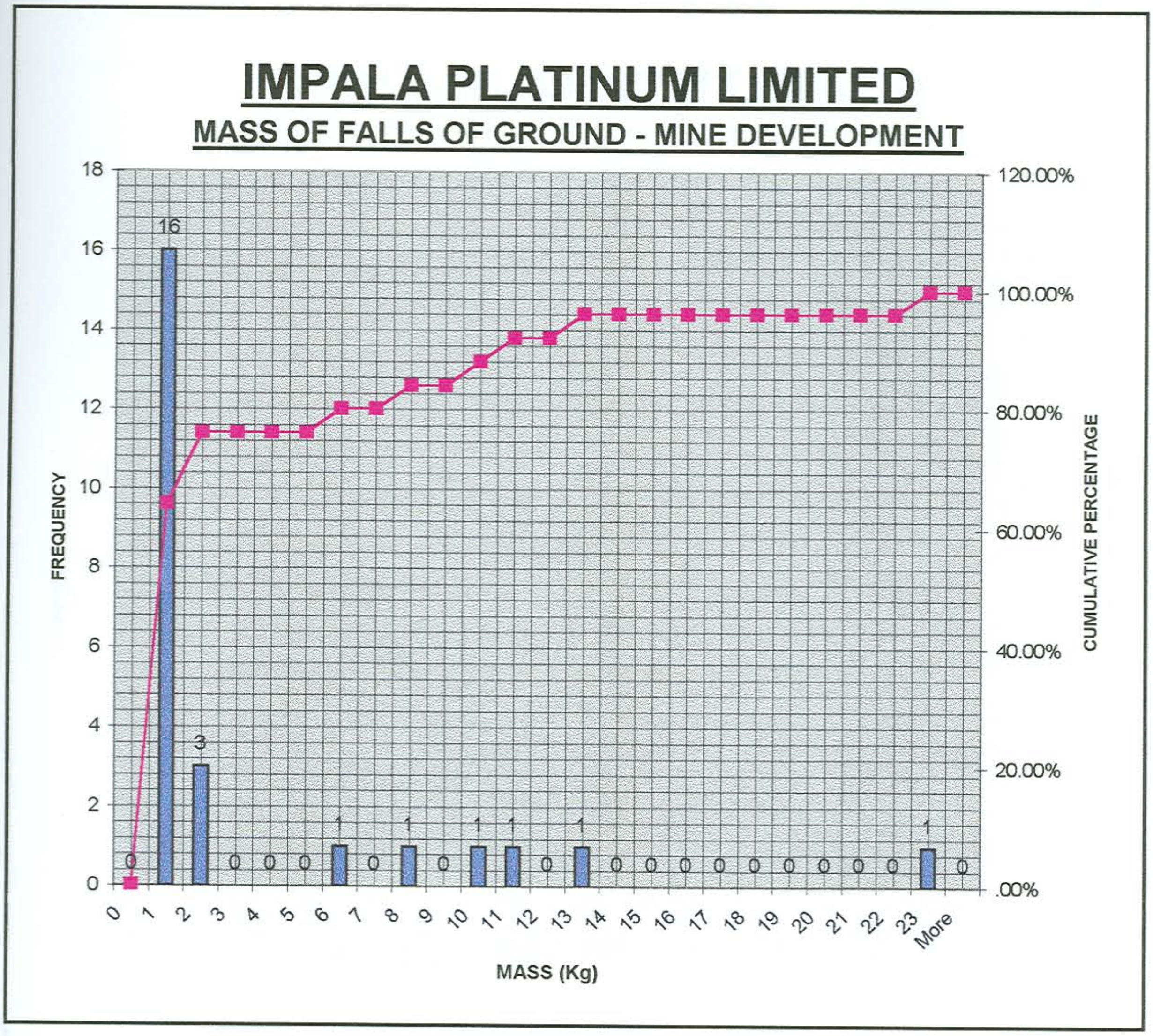


FIG. 3.3 – Mass of falls of ground in mine development

Figure 3.4 shows a cumulative percentage and histogram plot of the volume in m³ of reportable falls of ground accidents in mine development from 1992 to 1996. Thus the 95 cumulative percentage of falls of ground represents a volume of 5m³.

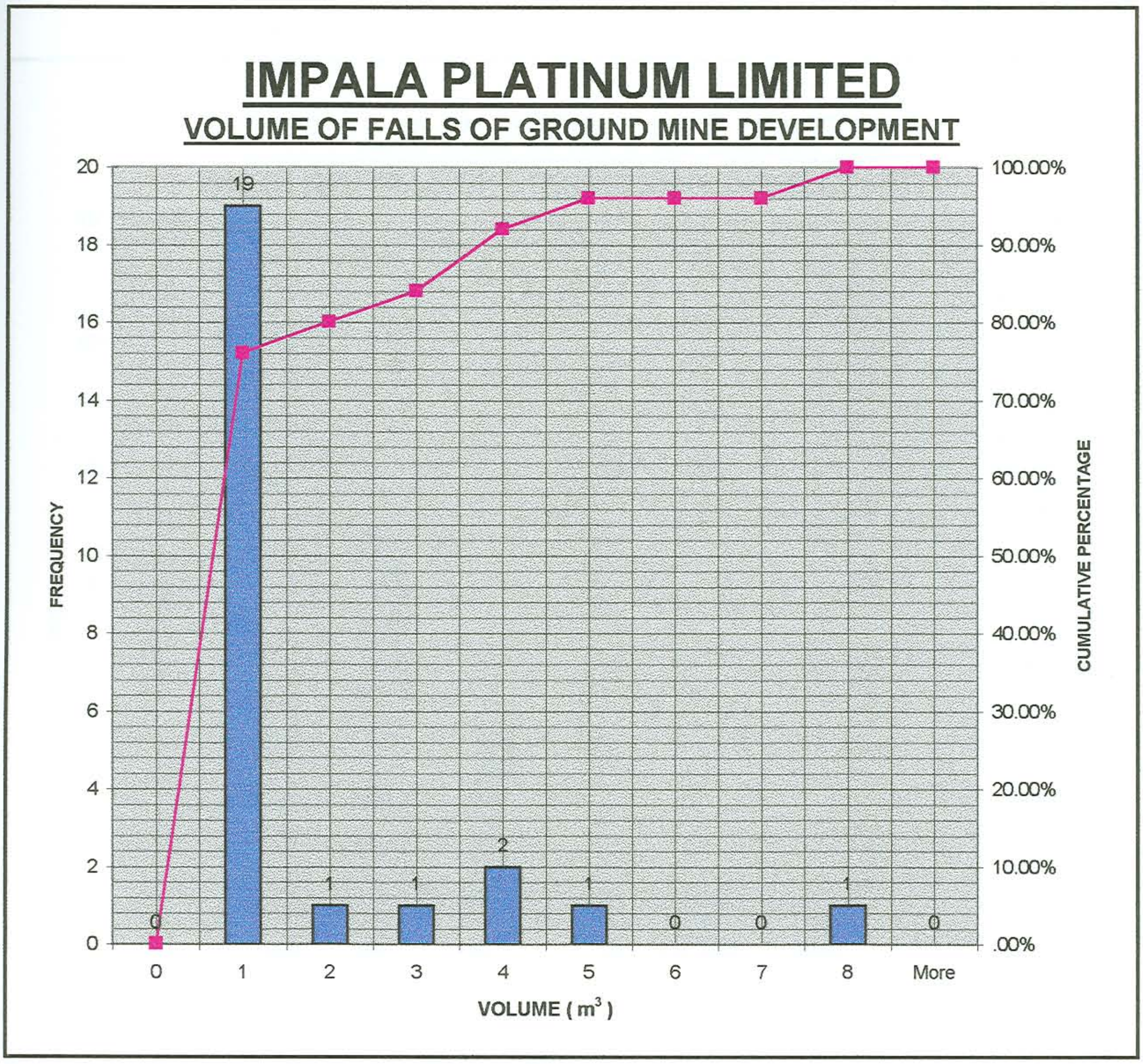


FIG. 3.4 - Volume of falls of ground in mine development

Figure 3.5 shows a cumulative percentage and histogram plot of the width (m) of reportable falls of ground accidents in mine development from 1992 to 1996. The width of falls of ground represents a measurement 90° to the long axis of a tunnel. Thus the 95 cumulative percentage of falls of ground represents a width of 2,5m.

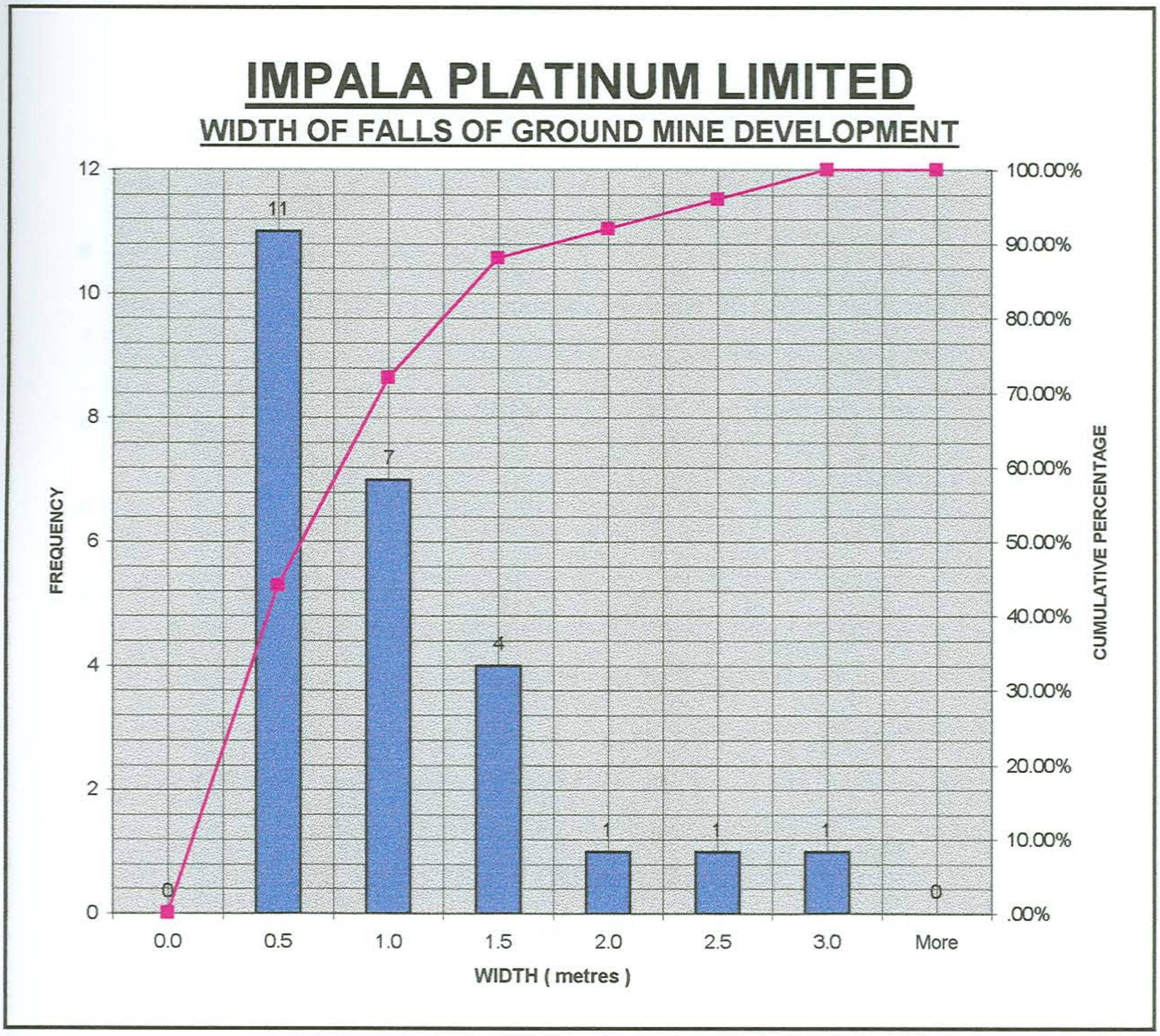


FIG. 3.5 - Width of falls of ground in mine development

Figure 3.6 shows a cumulative percentage and histogram plot of the length (m) of reportable and fatal falls of ground accidents in mine development from 1992 to 1996. The length of falls of ground represents a measurement parallel to the long axis of a tunnel. Thus the 95 cumulative percentage of falls of ground represents a length of 3,5m.

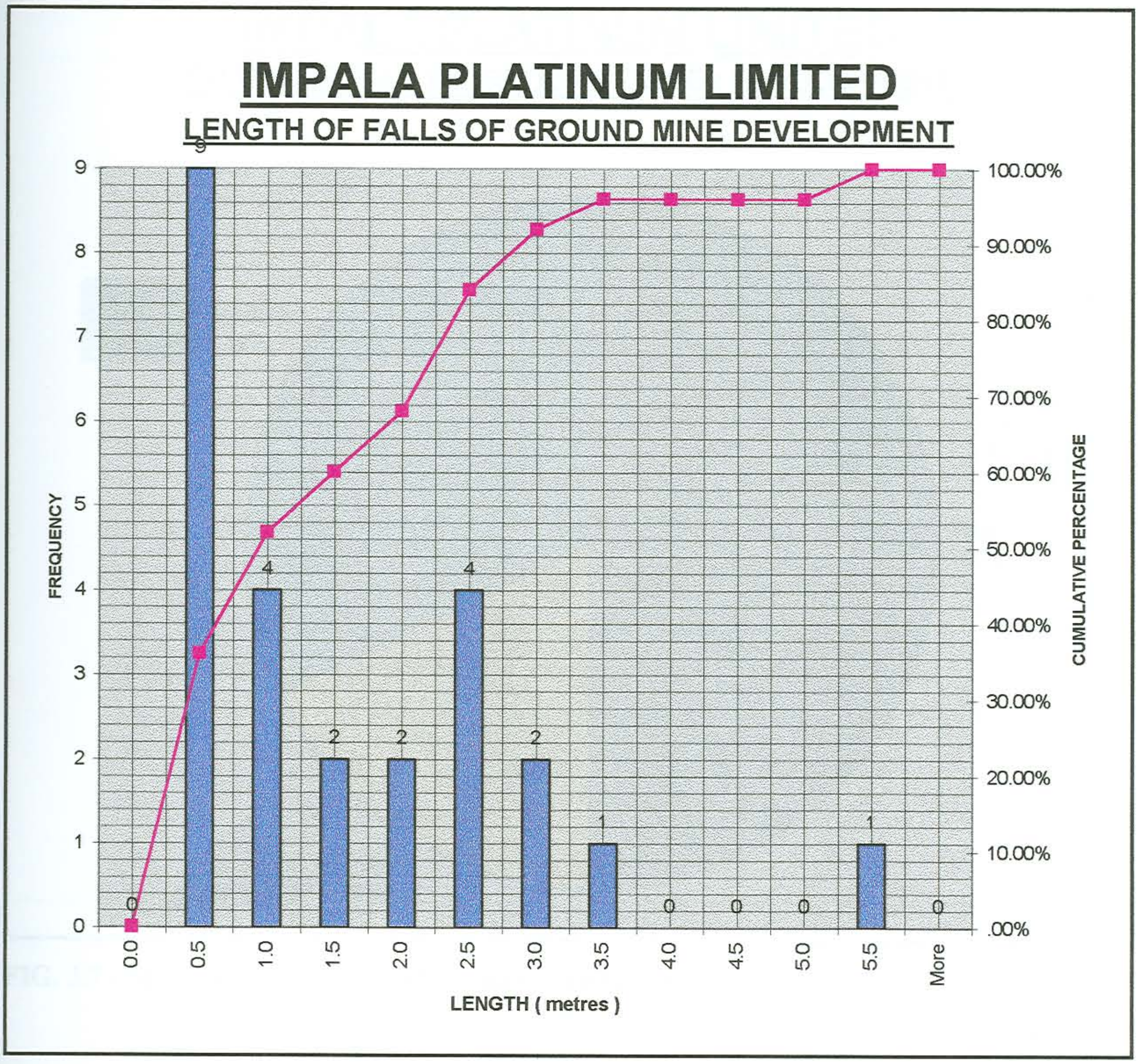


FIG. 3.6 - Length of falls of ground in mine development

Figure 3.7 shows a pie chart of the typical shapes of reportable and fatal fall of ground accidents in mine development from 1992 to 1996. Thus 50% of the falls of ground are represented by a block shape, 18% of the falls are represented by wedges and 32% by scaling.

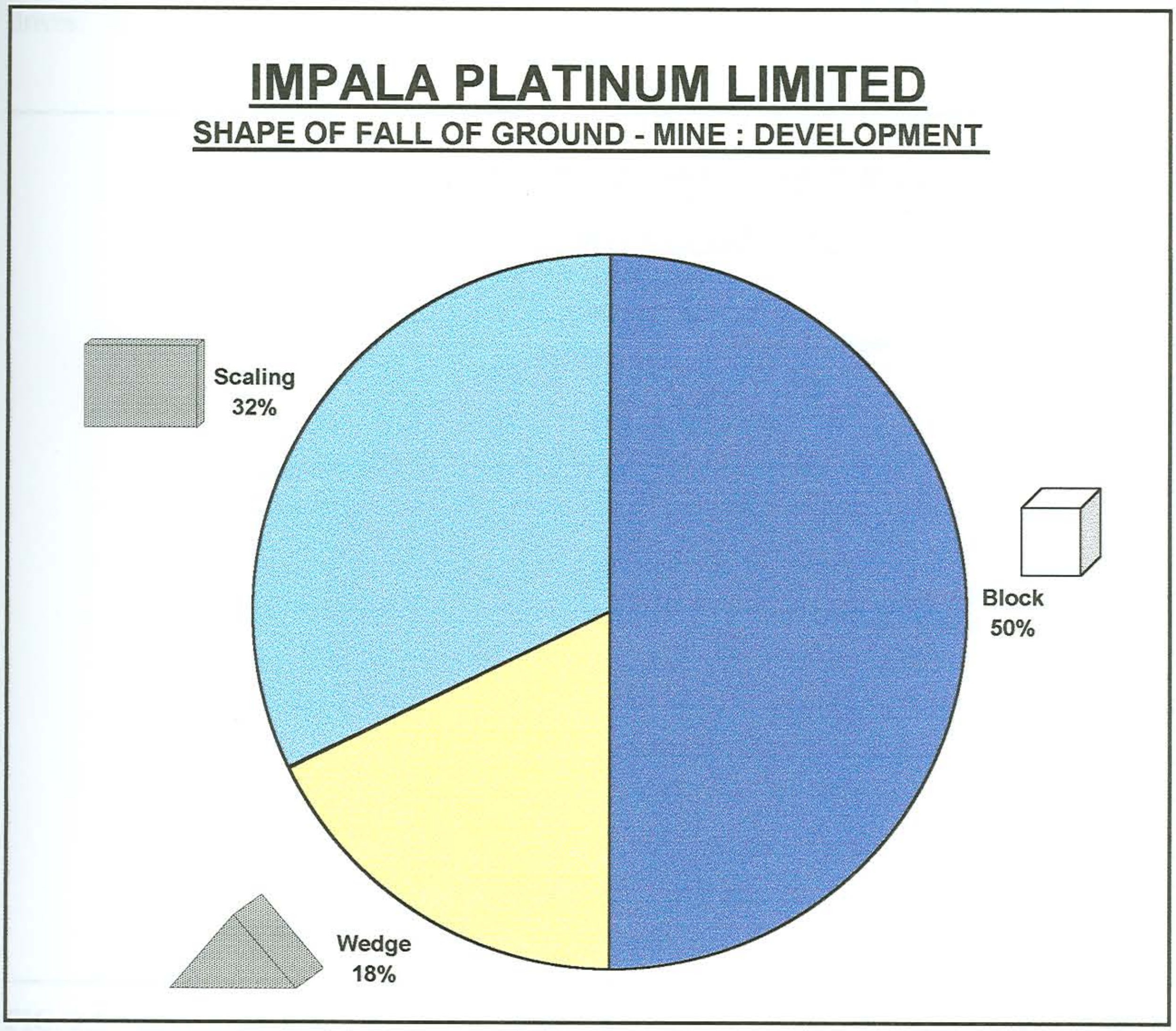


FIG. 3.7 - Shape of falls of ground in mine development

Figure 3.8 shows a pie chart of the typical rock type responsible for reportable falls of ground accidents in mine development from 1992 to 1996. Thus 34% of the falls of ground occurred from Merensky footwall in drives and 37% of the falls of ground occurred from UG2 footwall drives. The implication is that 71% of falls of ground in mine development originated in off-reef drives.

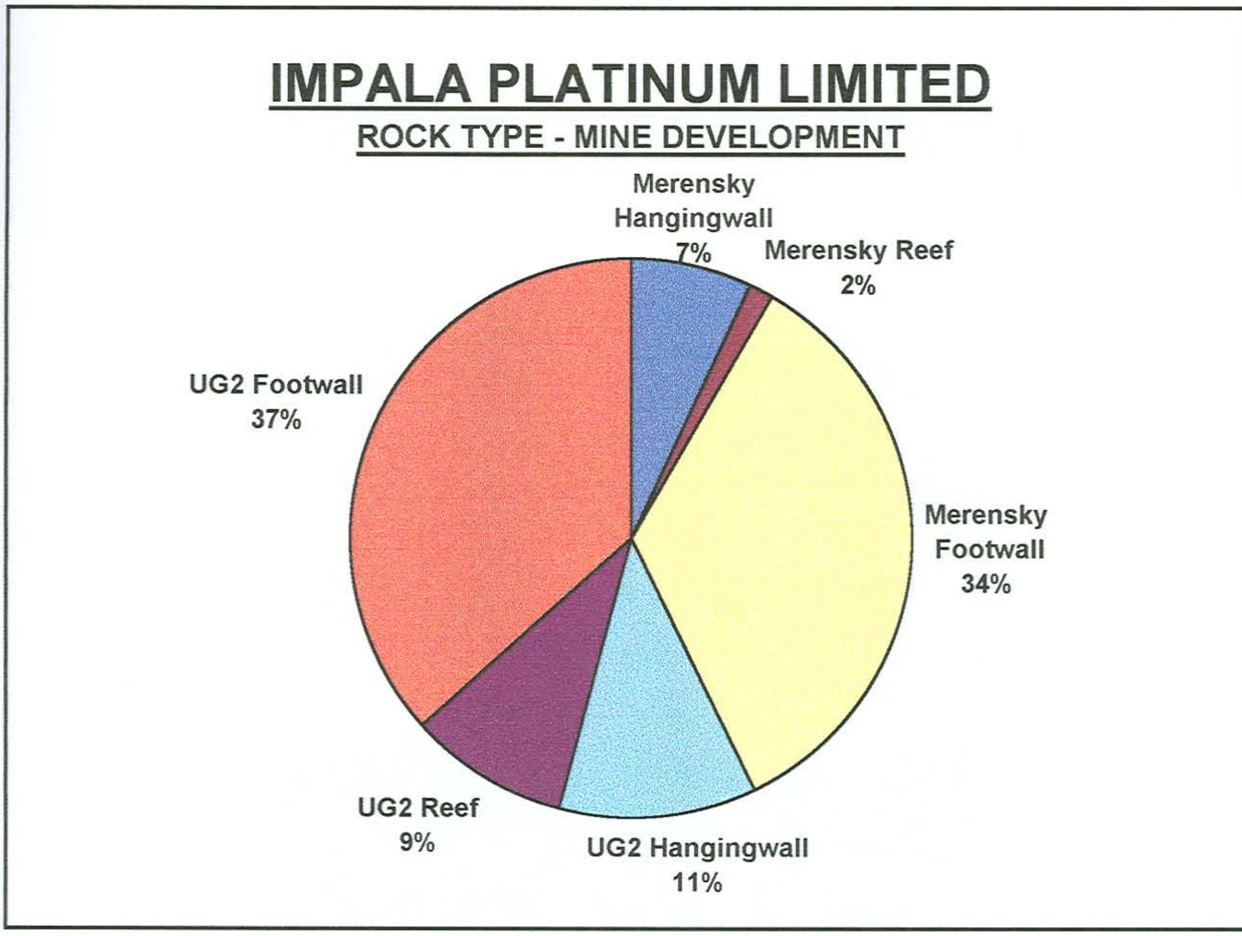


FIG. 3.8 - Rock type falls of ground in mine development

Figure 3.9 shows a pie chart of the typical boundaries responsible for reportable and fatal fall of ground accidents in mine development from 1992 to 1996. Thus 63% of the falls of ground occurred with jointing as boundaries, 27% were chromitite layers, 6% faults and 4% Dykes. Therefore the rockmass classification used at Impala must include joint analysis, as the majority of falls of ground are bounded by joints.

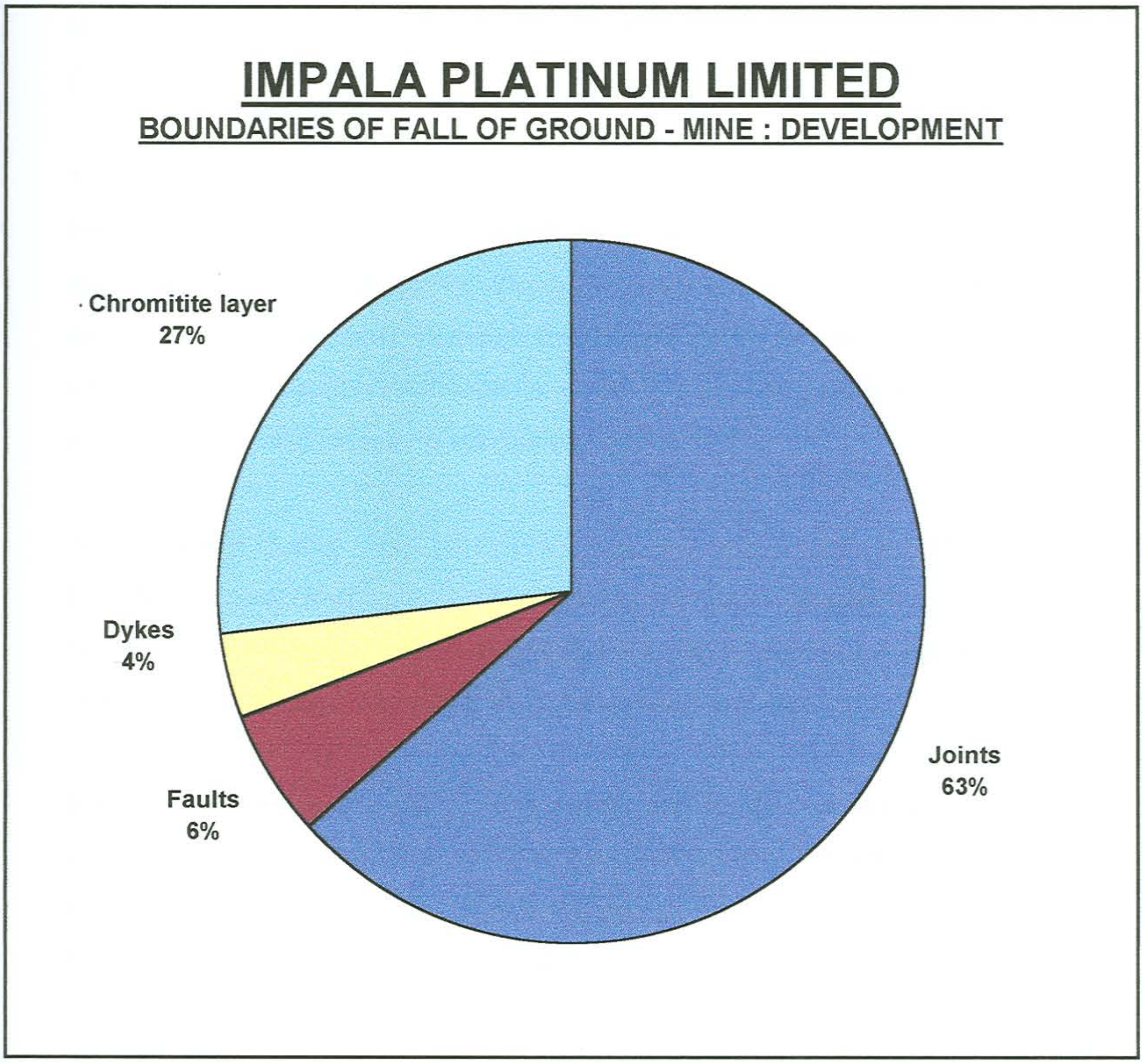


FIG. 3.9 - Boundaries of falls of ground in mine development

CHAPTER IV

ROCKMASS CLASSIFICATION SCHEMES AND GEOMETRY

In a classification system empirical relations between rock mass properties and the behaviour of the rock mass in relation to a particular engineering application, are combined to give a method of designing engineering structures in or on a rock mass (Hack, 1997).

Theoretically, a proper description or geotechnical calculation to determine the behaviour of a rock mass should include all properties in a rock mass including all spatial variations of the properties. This is unrealistic and is also not possible without disassembling the rock mass. Therefore the standard procedure is to divide a rock mass into homogeneous geotechnical units. In practice, such homogeneity is seldom found and material and discontinuity properties vary within the unit.

The Tunneling Quality Index (Q-System) proposed by Barton et al (1974) and the Rock Mass Rating (RMR) classification system proposed by Bieniawski (1973) are the two most commonly used rock mass classification systems. Both are designed to assess factors which influence the stability of an underground excavation

Both methods incorporate geological, geometric and design/engineering parameters in arriving at a quantitative value of their rock mass quality. The similarities between RMR and Q stem from the use of identical, or very similar, parameters in calculating the final rock mass quality rating. The differences between the two systems lie in the different weightings given to similar parameters and in the use of distinct parameters in one or the other scheme.

The RMR uses compressive strength directly while Q only considers strength as it relates to in situ stress in competent rock. Both schemes deal with the geology and geometry of the rock mass, but in slightly different ways. Both consider ground water, and both include some component of rock material strength.

Some estimate of orientation can be incorporated into Q using a guideline presented by Barton et al (1974) : 'the parameters Jr and Ja should relate to the surface most likely to allow failure to initiate'. The greatest difference between the two systems is the lack of a stress parameter in the RMR system.

However various other researchers in the field of rock mechanics have either proposed an alternative system or modify an existing rock mass classification scheme. The literature survey that follows concludes that all classification systems are based on a specific set of parameters or problems, and that their applicability is therefore limited.

The problems experienced by the various researchers do not necessarily advocate that his specific system can be applied generally. Below are examples of such classifications systems, checklists and pure observation ability in the field. It was necessary to conduct research into the various rockmass classification systems to confirm a suitable systems which could relate to the Impala problem. The following will assist the reader to objectively understand the complexity of the Impala problem, which lies in the quest to design the optimum support system for a typical shallow platinum hard rock off-reef tunnel.

The detailed analysis of the rockmass surrounding a tunnel developed 20 years ago, which until today has not been supported, and a tunnel which currently is being developed, which is extremely unstable due to rockmass structure rather than poor rockmass will further fine tune the focus on the problem.

4.1 Identification of keyblocks shapes and sizes

Stability problems in blocky, jointed rock are often associated with gravity falls of blocks from the hangingwall and sidewalls. Rock stresses at relatively shallow depths are generally too low to have a significant effect upon this failure process which is controlled by the three dimensional geometry of the excavation and the rock structure.

A simple identification of keyblocks in tunnels is intended to assess the long-term stability criteria for specific tunnels. It is also virtually impossible to secure a design without the necessary investigation.

4.1.1 Geological Discontinuities in rock

A discontinuity may be defined as a boundary within the rock mass which marks a change in the mass properties and thereby a change in engineering characteristics. This definition includes features such as lithological boundaries, bedding planes, joints and faults.

Joints can usually be seen on an exposed rock surface. They appear as approximately parallel or randomly orientated cracks separated by as little as several centimeters or by as much as 10m or more. One set of joints commonly forms parallel to bedding and there are usually at least two other sets in other directions (SIMRAC, 1994).

Igneous rocks, as in the case study, have irregular jointing systems with three or more sets. Rocks that have been deformed by folding often contain roughly parallel seams of sheared and crushed rock produced by interlayer slip or minor fault development. Such features are sometimes observed around the potholes of the Bushveld Complex rocks. Faults that may off-set all other crossing structures may also occur in the rock surrounding the tunnel. Thus there is a full range of weaknesses in rock masses with a statistical distribution of spacings and orientations at all scales.

The importance of weaknesses stems from the special properties that such features superimpose on rock. Basically, the rockmass becomes weaker and sometimes highly anisotropic, which create a variety of potential problems. Figure 4.1 shows how blocks might fall from the hangingwall of a tunnel due to intersecting joints.

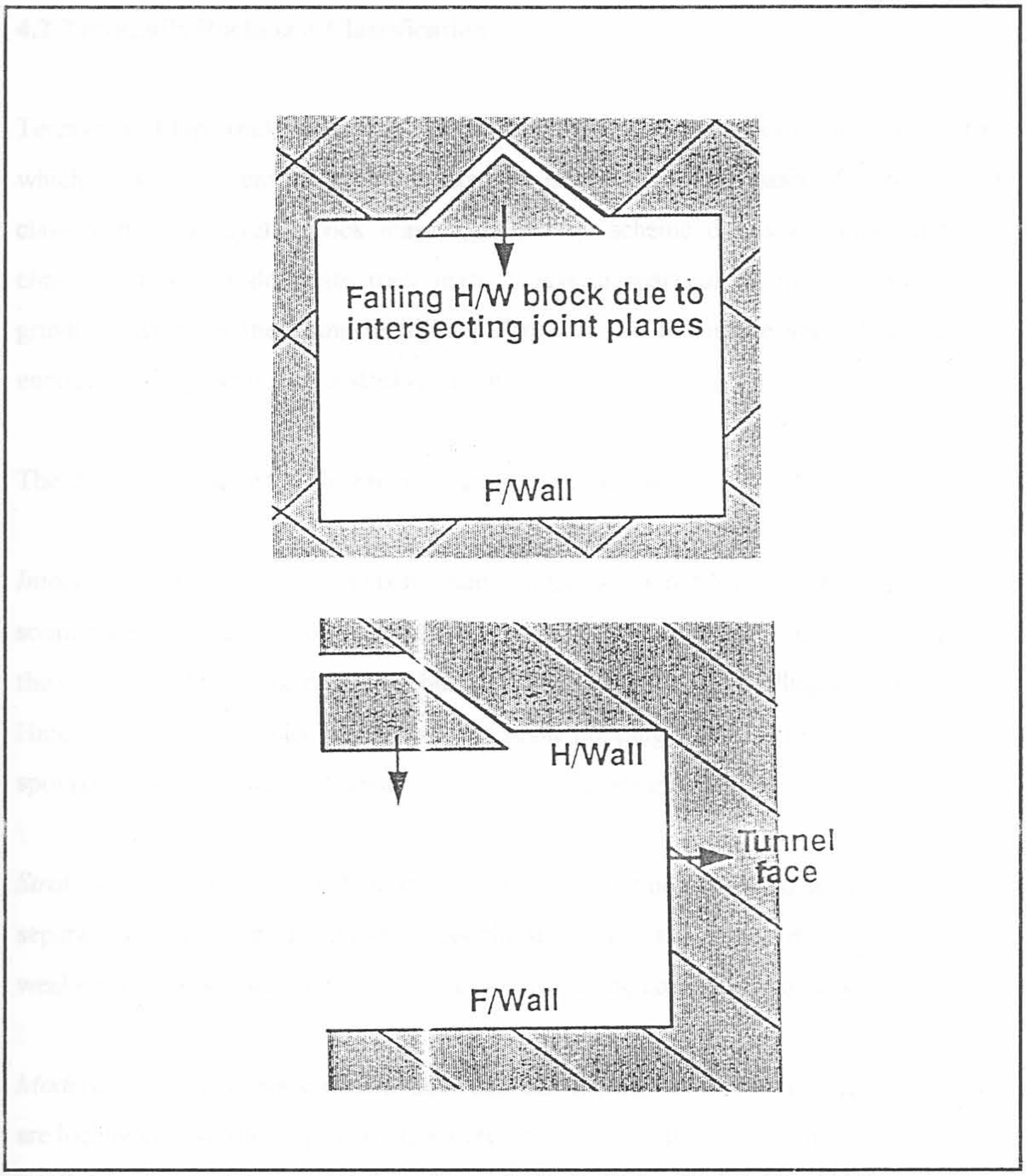


FIG. 4.1 - Potential unstable hangingwall block (After SIMRAC, 1994)

4.2 Terzaghi's Rockmass Classification

Terzaghi's (1946) rock mass classification is applied to the design of tunnel support in which rock loads, carried by steel sets, are estimated on the basis of a descriptive classification. Terzaghi's rock mass classification scheme draws attention to those characteristics that dominate rock mass behaviour, particularly in situations where gravity constitutes the dominant driving force. It can further be applied to shallow enough workings that in-situ stress is not important.

The factors considered in Terzaghi's system are as follows :-

Intact rock contains neither joints nor hair cracks. Hence if it breaks, it breaks across sound rock. On account of the damage to the rock due to blasting, spalls may drop off the roof several hours or days after blasting. This is known as a spalling condition. Hard, intact rock may also be encountered in the popping condition involving the spontaneous and violent detachment of rock slabs from the sides or roof.

Stratified rock consists of individual strata with little or no resistance against separation along the boundaries between the strata. The strata may or may not be weakened by traversed joints. In such rock the spalling condition is quite common.

Moderately jointed rock contains joints and hair cracks, but the blocks between joints are locally grown together or so intimately interlocked that vertical walls do not require lateral support.

Blocky and seamy rock consists of chemically intact or almost intact rock fragments which are entirely separated from each other and imperfectly interlocked. In such rock, vertical walls may require lateral support.

Crushed but chemically intact rock has the character of crusher run. If most or all the fragments are as small as fine sand grains and no recementation has taken place, crushed rock below the water table exhibits the properties of a water-bearing sand.

Squeezing rock slowly advances into the tunnel without perceptible volume increase. A prerequisite for squeeze is a high percentage of microscopic particles of micaceous minerals or clay minerals with a low swelling capacity.

Swelling rock advances into the tunnel chiefly on account of expansion. The capacity to swell seems to be limited those rocks that contain clay minerals such as montmorillonite, with a high swelling capacity.

Terzaghi's system is quick and easy to apply but it leaves a lot of factors out that would be important in mining. It is used to design support using sets, which are generally not used in the mining industry. The system is therefore not considered appropriate for platinum mining because of the limited detailed analysis of the jointed rockmass.

4.3 Rock Quality Designation Index (RQD)

Deere (1964) developed the RQD index to provide a quantitative estimate of rock mass quality from drill core logs. RQD is defined as the percentage of intact core pieces longer than 100mm in the total length of core. The core should be at least 50mm in diameter and should be drilled with a double barrel diamond drilling equipment. Figure 4.2 illustrates how the rock quality designation index is determined.

L = 38cm L = 17cm L = 20cm L = 15cm L = 9cm

Total L = 99cm

$$RQD = \frac{\text{Sum length of core pieces} > 10\text{cm}}{\text{Total length of core run}} \times 100\%$$

$$= 90\text{cm} / 99\text{cm} \times 100\%$$

$$= 91\%$$

FIG. 4.2 - Procedure for measurement and calculation of RQD (After Deere, 1964)

Palmstrom ***(1982)*** suggested that, when no core is available but discontinuity traces are visible in surface exposures or exploration audits, the RQD may be estimated from the number of discontinuities per unit volume.

$$RQD = 115 - 3.3 * Jv \qquad (4.1)$$

Where *Jv* is the sum of the number of joints per unit length for all joint (discontinuity) sets known as the volumetric joint count.

RQD is intended to represent the rock mass quality in situ. Thus the most important use of RQD is as a component of the RMR and Q rock mass classifications which is discussed later.

- The RQD support criteria system has limitations in areas where the joints contain thin clay fillings or weathered material. Such a case might occur in near surface rock where weathering or seepage has produced clay which reduces the frictional resistance along joint boundaries. This would result in unstable rock although the joints may be widely spaced and the RQD high.
- The RQD does not take direct account of other factors such as joint orientation which must influence the behaviour of a rock mass around an underground opening.
- It does not provide an adequate indication of the range of behaviour patterns which may be encountered when excavating underground.

4.4 Rock Structure Rating (RSR)

The RSR (Wickham et al., 1972) is a quantitative method for describing the quality of a rock mass and for selecting appropriate support on the basis of a Rock Structure Rating classification. Most of the case histories, used in the development of this system, were for relatively small tunnels supported by means of steel sets, although historically this system was the first to make reference to shotcrete support.

The RSR system is the first system to demonstrate the logic involved in developing a quasi-quantitative rock mass classification system.

The following are the parameters considered in the RSR :-

i. Parameter A, Geology : General appraisal of geological structure on the basis of (see Table 4.1) :

a) Rock type origin (igneous, metamorphic,, sedimentary).

b) Rock hardness (hard, medium, soft, decomposed).

c) Geologic structure (massive, slightly faulted/folded, moderately faulted/folded, intensely faulted/folded)

TABLE 4.1 - Rock Structure Rating - Parameter A - General Area Geology

	Basic Rock Type							
	Hard	**Medium**	**Soft**	**Decomposed**	**Geological Structure**			
Igneous	1	2	3	4				
Metamorphic	1	2	3	4				
Sedimentary	2	3	4	4	**Massive**	**Slightly Folded or Faulted**	**Moderately Folded or Faulted**	**Intensively Folded or Faulted**
Type 1					30	22	15	9
Type 2					27	20	13	8
Type 3					24	18	12	7
Type 4					19	15	10	6

ii. Parameter B, Geometry : Effect of discontinuity pattern with respect to the direction of the tunnel drive on the basis of (see Table 4.2) :

a) Joint spacing.

b) Joint orientation (strike and dip).

c) Direction of tunnel drive.

TABLE 4.2 - Rock Structure Rating - Parameter B - Joint Pattern, direction of Drive

Average Joint Spacing	Strike perpendicular to axis					Strike parallel to axis		
	Direction of drive					Direction of drive		
	Both	With Dip		Against Dip		Either Direction		
	Dip of Prominent Joints [a]					Dip of Prominent Joints		
	Flat	Dipping	Vertical	Dipping	Vertical	Flat	Dipping	Vertical
1. Very closely jointed, <2 in	9	11	13	10	12	9	9	7
2. Closely jointed, 2-6 in	13	16	19	15	17	14	14	11
3. Moderately jointed, 6-12 in	23	24	28	19	22	32	32	19
4. Moderate to blocky, 1-2 ft	30	32	36	25	28	30	38	24
5. Blocky to massive, 2-4 ft	36	38	40	33	35	36	34	28
6. Massive, >4 ft	40	43	45	37	40	40	38	34

iii) Parameter C : Effect of ground water inflow and joint condition on the basis of (see Table 4.3) :

a) Overall rock mass quality on the basis of A and B combined.

b) Joint condition (good, fair, poor).

c) Amount of water inflow (in liters per minute per 280m of tunnel).

TABLE 4.3 - Rock Structure Rating - Parameter C - Ground Water, Joint Condition

Anticipated Water inflow Gpm/1000 ft of tunnel	Sum of parameters A + B					
	13-44			45-75		
	Joint Condition [b]					
	Good	Fair	Poor	Good	Fair	Poor
None	22	18	12	25	22	18
Slight, <200 gpm	19	15	9	23	19	14
Moderate, 200-1000 gpm	15	22	7	21	16	12
Heavy, > 1000 gpm	10	8	6	18	14	10

[a] Dip: Flat: 0-20°; dipping: 20-50°; and vertical: 50-90°

[b] Joint condition: good = tight or cemented; fair = slightly weathered or altered; poor = severely weathered, altered or open

Each of the components listed above provide a numerical value of RSR :

$$RSR = A + B + C \tag{4.2}$$

The RSR classification used Imperial units. For a 7,32m diameter tunnel with a RSR value of 62, the predicted support would be 2 inches (50,8mm) of shotcrete and 1 inch (25,4mm) diameter rockbolts spaced at 5 foot (1,524m) center's (See figure 4.3).

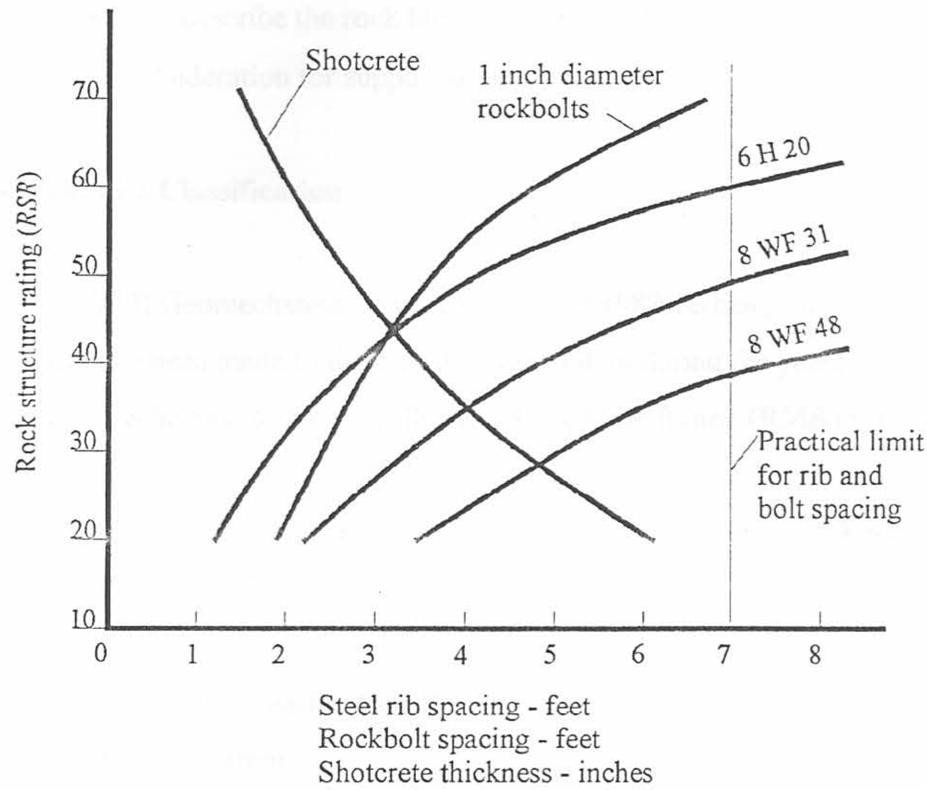


FIG. 4.3 - RSR Support estimates for 24ft (7.3m) Diameter circular tunnel (After Wickham et al, 1972)

The 62 RSR value consists of a hard metamorphic rock which is slightly folded or faulted and moderately jointed, with joints striking perpendicular to the axis of the tunnel, dipping at between 20° and 50°. A moderate water inflow of between 912 and 4560 liters per minute.

The RSR classification system is not widely in use today. However Wickham et al's work played a significant role in the development of other classification systems. This rating system however is not considered for the following reasons :-

- It mainly considers circular tunnels
- It describes jointing and orientation (geometry) however the important critical factor joint roughness is not considered
- Generally does not describe the rock block volume involved
- Steel sets as a consideration for support design

4.5 Geomechanics Classification

Bieniawski's (1989) Geomechanics classification is the 1989 version, for many changes have been made to the classification system during the years. This geomechanics classification system is called the Rock Mass Rating (RMR) system.

The following 6 parameters shown in Table 4.4 are used to classify a rock mass using the RMR system.

TABLE 4.4 - Rockmass Classification Parameters for the Rock Mass Rating (RMR) system

1. Uniaxial compressive strength of rock material.
2. Rock Quality Designation (RQD).
3. Spacing of discontinuities.
4. Condition of discontinuities.
5. Groundwater conditions.
6. Orientation of discontinuities.

In applying this classification system, the rock mass is divided into a number of structural regions and each region is classified separately. The boundaries of the structural regions usually coincide with a major structural feature such as a fault or with a change in rock type.

Guidelines for the selection of support in tunnels in rock for which the value of RMR has been determined and are given in Table 4.6 after using Table 4.5.

TABLE 4.5 - Rock Mass Rating (After Bieniawski, 1989)

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

Parameter			Range of values						
1	Strength Of Intact rock Material	Point-load Strength index	>10 MPa	4-10 MPa	2-4 Mpa	1-2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100-250 MPa	50-100 MPa	25-50 MPa	5-25 MPa	1-5 MPa	<1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core Quality RQD		90%-100%	75%-90%	50%-75%	25%-50%	<25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		>2m	0.6-2m	200-600mm	60-600mm	<60mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Un-weathered wall rock	Slightly rough surfaces Separation <1mm Slightly weathered walls	Slightly rough surfaces Separation <1mm Highly weathered walls	Slickensided surfaces Or Gouge <5mm thick Or Separation 1-5mm Continuous	Soft gouge >5mm thick or Separation > 5mm Continuous		
	Rating		30	25	20	10	0		
5	Ground Water	Inflow per 10m tunnel length (l/m)	None	<10	10-25	25-125	>125		
		(Joint water press)/(Major principal σ)	0	<0.1	0.1-0.2	0.2-0.5	>0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

Strike and dip orientations		Very Favourably	Favourably	Fair	Unfavourably	Very Unfavourably
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 – 81	80-60	60-41	40-21	<21
Class number	I	II	III	IV	V
Description	Very good rock	Good Rock	Fair Rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES

Class number	I	II	III	IV	V
Average stand up time	20 yrs for 15m span	1 year for 10m span	1 week for 5m span	10hrs for 2.5m span	30 min for 1m span
Cohesion of rock mass (kPa)	>400	300-400	200-300	100-200	<100
Friction angle of rock mass (deg)	>45	35-45	25-35	15-25	<15

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions

Discontinuity length (persistence) Rating	<1m 6	1-3m 4	3-10m 2	10-20m 1	>20m 0
Separation (aperture) Rating	None 6	<0.1mm 5	0.1-1.0mm	1-5mm 1	>5mm 0
Roughness Rating	None 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
Infilling (gouge) Rating	None 6	Hard filling < 5mm 4	Hard filling >5mm 2	Soft filling < 5mm 2	Soft filling >5mm 0
Weathering Ratings	Unweathered 6	Slightly Weathered 5	Moderately weathered 3	Highly weathered 1	Decomposed 0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELING **

Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip – Dip 45-90°	Drive with dip – Dip 20-45°	Dip 45-90°	Dip 20-45°
Very favourable	Favourable	Very favourable	Fair
Drive against dip – Dip 45-90°	Drive against dip – Dip 20-45°	Dip 0-20 – Irrespective of strike°	
Fair	Unfavourable	Fair	

* Some conditions are mutually exclusive. For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such case use A.4 directly.

** Modified after Wickham et al (1972)

TABLE 4.6 - Guidelines for excavating and support of 10m span tunnels in accordance with RMR system (After Bieniawski, 1989)

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

Cummings et al (1982) and Kendorski et al (1983) have also modified Bieniawski's RMR classification to produce the MBR (modified basic RMR) system for mining. This system was developed for block caving operations in the USA.

It involves the use of different ratings for the original parameters used to determine the value of RMR and the subsequent adjustment of the resulting MBR value to allow for blasting damage, induced stresses, structural features, distance from the cave front and size of the caving block. Support recommendations are for isolated or development drifts as well as for the final support of intersections and drifts.

The above guidelines (Table 4.5) have been published for a 10m span horseshoe shaped tunnel, constructed using drill and blast methods, in a rockmass subjected to a vertical stress <25MPa (equivalent to a depth below surface of 900m). Tables 4.5 and 4.6 have not had a major revision since 1989.

The overall Rock Mass Rating is obtained by adding the values of the ratings determined for the individual parameters. The RMR value may be adjusted for the influence of discontinuity orientation by applying the corrections.

Limitations to the system include the following :-

- Data mainly obtained from civil engineering excavations in sedimentary rocks in S.A.
- Intact rock strength cannot generally be determined with a 10m interval in a tunnel section, this is very impracticable and costly. Drill core of a tunnel section is not always available. There are large variations in rockmass strength in the Bushveld Complex which have not being clearly defined yet
- The system lacks a stress parameter
- Support considerations are limited to a 10m excavation span

4.6 Modifications to RMR for mining

Bieniawski's (1978) Rock Mass Rating (RMR) system was originally based upon case histories drawn from civil engineering. Consequently, the mining industry tended to regard the classification as somewhat conservative and several modifications have been proposed in order to make the classification more relevant to mining applications.

Laubscher (1977) modified Bieniawki's geomechanics classification on the basis of experience gained in a number of chrysotile asbestos mines in Africa. Laubscher and Taylor (1976) and Laubscher and Page (1990) have described a Modified Rock Mass Rating system for mining. This MRMR takes the basic RMR value, as defined by Bieniawski, and adjusts it to account for in situ and induced stresses, stress changes and the effects of blasting and weathering.

In using Laubscher's MRMR system it should be borne in mind that many of the case histories upon which it is based are derived from caving operations. Originally, block caving in asbestos mines in Africa formed the basis for the modifications, but subsequently, other case histories from around the world have been added to the database.

The classification, set out in Table 4.7, uses the same five classification parameters as Bieniawski's scheme but involves differences in detail. Each of the five classes is divided into subclasses, A and B, new ranges and ratings for intact rock strength (IRS in Table 4.7) are used, and the joint spacing and condition of joint parameters are evaluated differently (Brady & Brown, 1985).

The only discontinuities (joints) included in the assessment of RMR are those having trace lengths greater than one excavation diameter or 3m, and those having trace lengths of less than 3m that are intersected by other discontinuities to define blocks of rock. True spacings of the three most closely spaced joint sets present in the rock mass are used in conjunction with Figure 4.4 to obtain a joint spacing rating on a scale of 0 to 30. The way the in which the joint condition rating is influenced by a range of factors are set out in Table 4.7. Before the basic rating for the rock mass is applied, it is adjusted to take account of weathering, field and induced stresses, changes in stress due to mining operations, orientations of blocks with exposed bases and blasting effects (Brady & Brown, 1985).

Limitations to the system include the following :-

- The data is mainly obtained from civil engineering excavations in sedimentary rocks in S.A. and mainly being modified to assess block caving operation
- The intact rock strength cannot generally be determined with a 10m interval in a tunnel section, this is very impracticable and costly. Drill core of a tunnel section not always available. There are large variations in rockmass strength in the Bushveld Complex which have not being clearly defined yet

TABLE 4.7 - Modified geomechanics classification scheme (After Laubscher, 1977)

class	**1**		**2**		**3**		**4**		**5**		
rating	**100-81**		**80-61**		**60-41**		**40-21**		**20-0**		
description	**Very good**		**good**		**fair**		**poor**		**Very poor**		
subclasses	**A**	**B**	**A**	**B**	**A**	**B**	**A**	**B**	**A**	**B**	
item											
1. RQD %	100-91	90-76	75-66	65-56	55-46	45-36	35-26	25-16	15-6	5-0	
Rating	20	18	15	13	11	9	7	5	3	0	
2. IRS, MPa	141-136	135-126	125-111	110-96	95-81	80-66	65-51	50-36	35-21	20-6	5-0
Rating	10	9	8	7	6	5	4	3	2	1	0
3. Joint spacing											
Rating	30……………………………………………………………0										
4. condition of joint	45°……………………… ……………………………5°										
	30 ……………………………………………………………0										
Rating											
	Inflow per 10m Length or joint water pressure		0		25l/min		25-125l/min		125l/min		
		0			0.0-0.2		0.2-0.5		0.5		
5. Groundwater	Major principal stress Or										
description	Completely dry		Completely dry		moist only		moderate pressure		severe problems		
rating	10		10		7		4		0		

TABLE 4.8 - Assessment of joint condition - adjustments as combined percentages of total possible rating of 30 (After Laubscher, 1977)

Parameter	Description	Percentage adjustment
Joint expression (large scale)	Wavy uni-directional	90-99
	Curved	80-89
	straight	70-79
Joint expression (small scale)	striated	85-99
	smooth	60-84
	polished	50-59
Alteration zone	Softer than wall rock	70-99
	Coarse hard-sheared	90-99
	Fine hard-sheared	80-89
	Coarse soft-sheared	70-79
	Fine soft-sheared	50-69
	Gouge thickness < irregularities	35-49
	Gouge thickness > irregularities	12-23
	Flowing material > irregularities	0-11

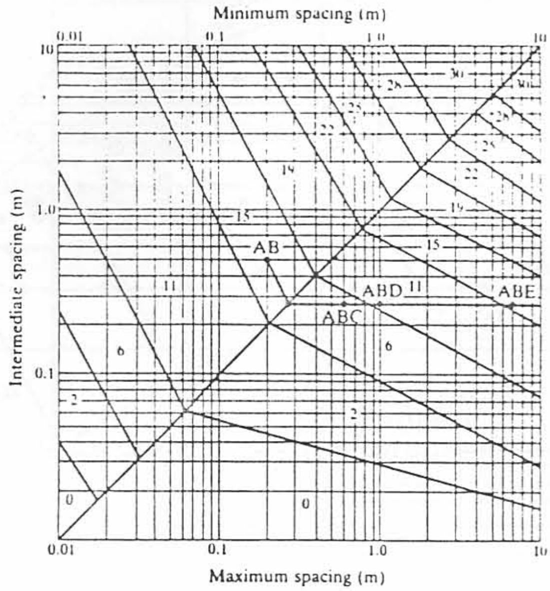


FIG. 4.4 - Joint spacing ratings for multi-joint systems (after Laubscher, 1977)

4.7 Classifications Involving Stand-up Time

Stini (1950) proposed a rock mass classification and discussed many of the adverse conditions which can be encountered in tunneling. He emphasized the importance of structural defects in the rock mass and stressed the need to avoid tunneling parallel to the strike of steeply dipping discontinuities. While both Terzaghi and Stini had discussed time-dependent instability in tunnels, it was Lauffer (1958) who proposed that the stand-up time for an unsupported span is related to the quality of the rock mass in which the span is excavated (Hoek & Brown, 1980).

In a tunnel, the unsupported span is defined as the span of the tunnel or the distance between the face and the nearest support, if this is greater than the tunnel span. The significance of the stand-up time concept is that an increase in the span of the tunnel leads to a significant reduction in the time available for the installation of support (See Figure 4.5 and 4.6).

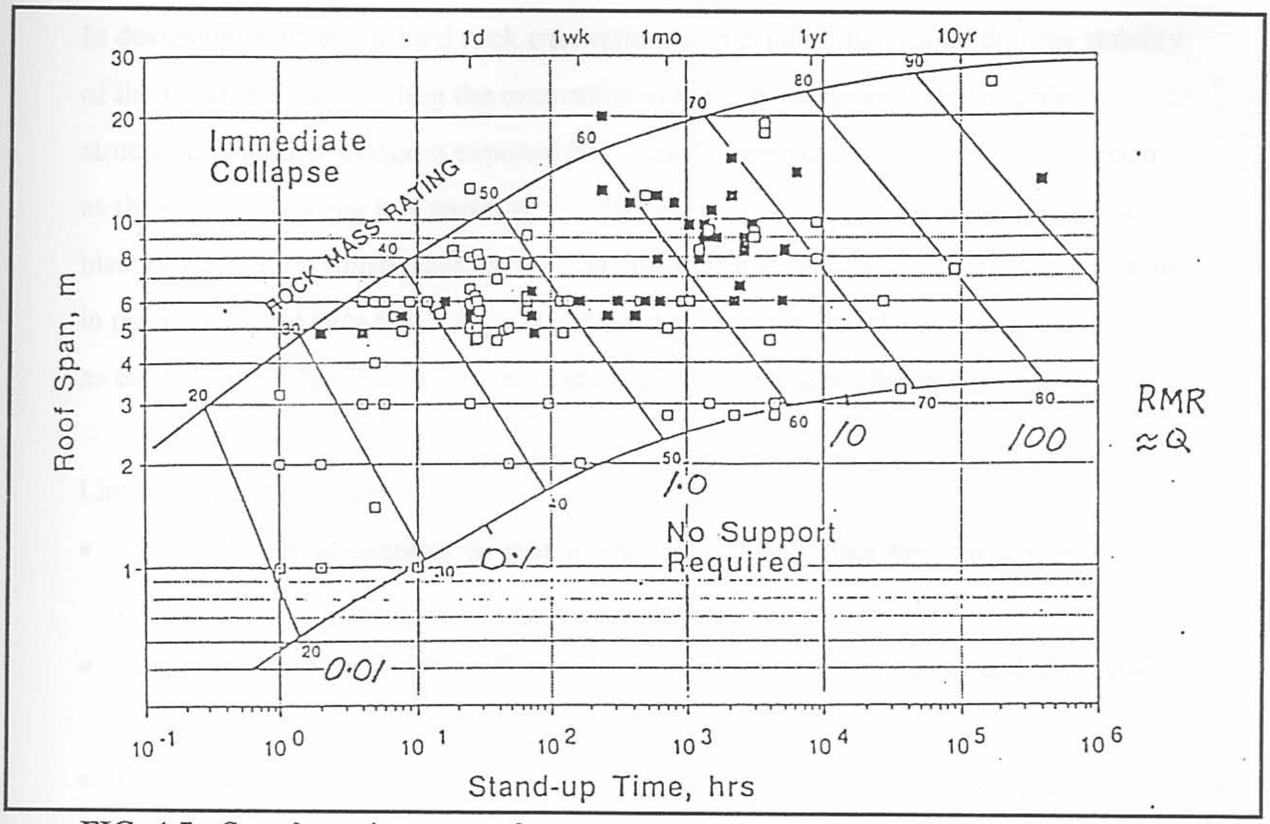


FIG. 4.5 - Stand-up time vs roof span compared to rock quality, RMR & Q value.

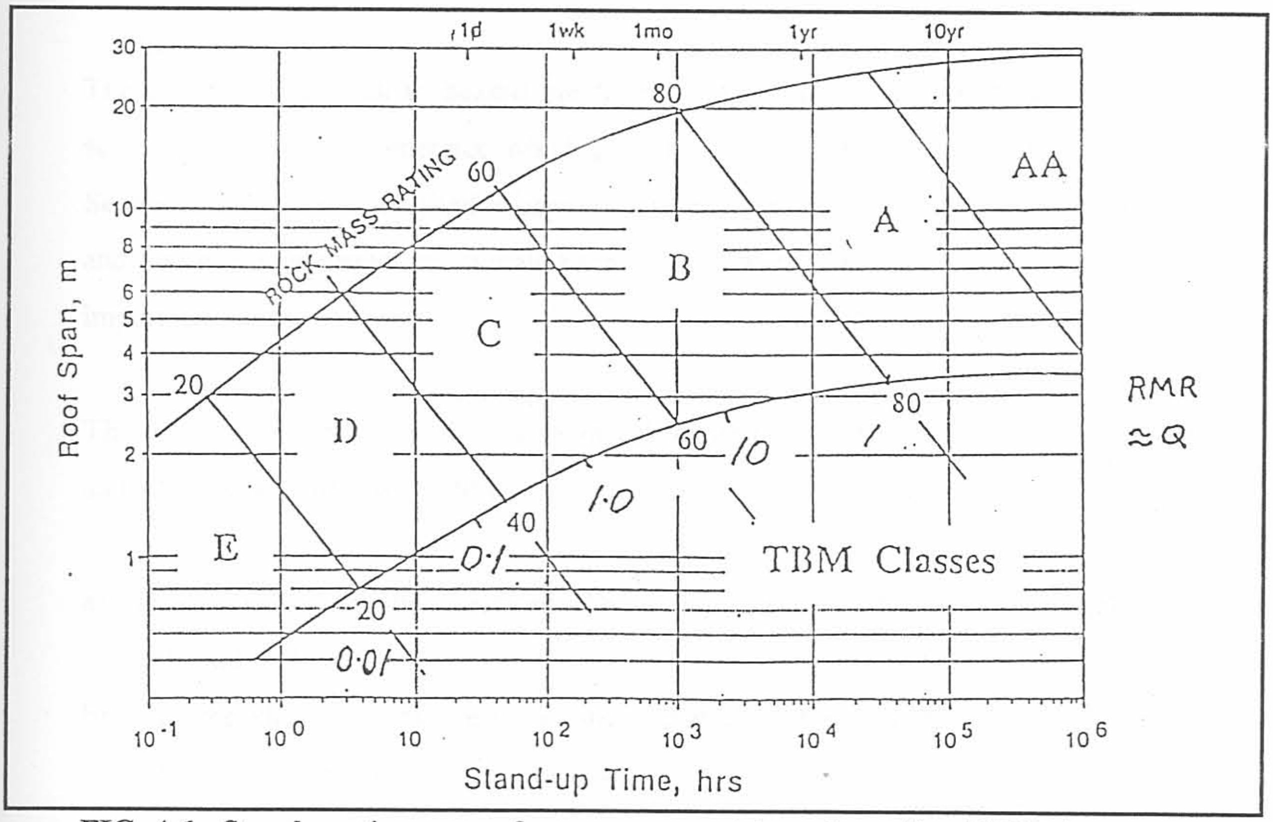


FIG. 4.6 - Stand-up time vs roof span compared to rock quality, TBM classes

In designing support for hard rock excavations it is prudent to assume that the stability of the rockmass surrounding the excavation is not time dependent. Hence, if a structurally defined wedge is exposed in the roof of an excavation, it will fall as soon as the rock supporting it is removed (i.e. barring). This can occur at the time of the blast or during the subsequent scaling operation. If it is required to keep such a wedge in place, or to enhance the margin of safety, it is essential that the support be installed as early as possible, preferably before the rock supporting the full wedge is removed.

Limitations to the system :

- The system is incomplete in that it only describes a time limit to a specific rock class.
- The above can be highly influenced by the blasting operations and the type of explosives used.
- Little consideration is given to the rockmass instability due to rock structure.

4.8 Checklist Methodology

The checklist approach to hazard identification takes place in two phases. Firstly, a series of questions is generated which pertain to a given hazard or group of hazards. Secondly, a consensus method is used to determine individual scores for each question and thereby generating an overall hazard score with which to assess the relative importance of the hazard.

There are a number of limitations to any checklist method that should be recognized and addressed (SIMRAC, 1998) :-

a) There are likely to be omissions. These should become fewer as the checklist is used and updated;
b) The checklist is insensitive to situations that are subject to change and may, after some time, contain irrelevant questions;

c) Perhaps the greatest disadvantage with a checklist is that it tends to put "blinkers" onto the user who becomes disinclined to look beyond checklist items for hazardous situations

Despite not completely satisfying all of the requirements for hazard identification, the checklist approach does provide an appropriate methodology particularly if limited rock engineering resources are available. The methodology is also useful as an initial approach to hazard identification and risk assessment.

The checklist methodology has three immediate benefits (SIMRAC, 1998) :-

i) It allows a hazard-based comparison or two or more activities to be carried out using a simple, arbitrary, but consistent, scoring method
ii) It allows an early identification and ranking of hazards that are relevant to the activity.
iii) It provides information on the nature of incidents which, potentially, can arise from hazard together with their possible cause and consequences.

4.9 Rockwall condition factor

Wiseman (1979) originally proposed an application of a system of evaluating the conditions of tunnels in a variety of conditions in all 20km of tunnel that was surveyed and analysed. The name of the classification system was given with the 1st edition of the Guide to methods of Ameliorating the Hazards of Rockfalls and Rockbursts (COMRO, 1988). The Rockwall condition factor (RCF) is given below :

$$RCF = (3\sigma_1 - \sigma_3) / F.\ \sigma_c \qquad (4.3)$$

where σ_1 and σ_3 are the major and minor principal stresses within the plane of the excavation cross section; and F is a factor to represent the down grading of σc (the uniaxial compressive strength) for the representative rock mass condition and excavation size. The formulation of the RCF is based on a simple comparison of the maximum induced tangential stress of an assumed circular excavation to the estimated rockmass strength. The empirical relationship between the rockwall conditions factor (RCF) and recommended support systems is based on extensive field studies of Witwatersrand gold mine tunnels which generally have dimensions of 3m by 3m. In general it was found that for RCF < 0.7, good conditions prevailed with minimum support requirements ; for $0.7 <$ RCF < 1.4, average conditions prevailed with moderate support systems requirements ; and for RCF > 1.4, poor ground conditions prevailed with special support requirements.

Empirical relationships have been derived between the RCF and the potentially unstable rock mass thickness for competent rock masses ($F = 1$) due to fracturing. It should be noted that this depth represents the potential unstable block height and will be less than the total depth of fracturing. These guidelines indicate that a RCF = 0.7, the anticipated thickness of unstable rock mass to be supported is approximately 0.7 x the radius of the excavation, and at RCF = 1.4 this thickness is approximately 1.2 x the radius. Under conditions of seismic loading, the increased extent of instability due to the transient dynamic stresses must be considered in the support design.

The above value ranges of the RCF criterion may not apply in Bushveld complex mines where the rock mass is igneous as opposed to the brittle quartzite's encountered in the gold mines (Jager & Ryder, 1999).

4.10 Rock Tunneling Quality Index, Q

Barton et al (1980) of the Norwegian Geotechnical Institute proposed a Tunneling Quality Index (Q) for the determination of rock mass characteristics and tunnel support requirements. The numerical value of the index Q varies on a logarithmic scale from 0.001 to a maximum of 1,000 and is defined by :

$$Q = (RQD/Jn) * (Jr/Ja) * (Jw/SRF) \qquad (4.4)$$

where	RQD	is the Rock Quality designation
	Jn	is the joint set number
	Jr	is the joint roughness number
	Ja	is the joint alteration number
	Jw	is the joint water reduction factor
	SRF	is the stress reduction factor

The category breakdown of the Q-System is shown in Table 4.9. The first quotient (RQD/Jn), representing the structure of the rockmass, is a crude measure of the block or particle size. The second quotient (Jr/Ja) represents the roughness and frictional characteristics of the joint walls or filling materials. This quotient is weighted in favor of rough, unaltered joints in direct contact. It is to be expected that such surfaces will be close to peak strength, that they will dilate strongly when sheared, and they will therefore be especially favorable to tunnel stability. When rock joints have thin clay mineral coatings and fillings, the strength is reduced significantly. The rock wall contact, after small shear displacements have occurred, may be a very important factor for preserving the excavation from ultimate failure. Where no rock wall exists, the conditions are extremely unfavourable to tunnel stability. The third quotient (Jw/SRF) consists of two stress parameters. SRF is a measure of :

a) loosening load in the case of an excavation through shear zones and clay bearing rock b) rock stress in competent rock and c) squeezing loads in plastic incompetent rocks.

The above can be seen as a total stress parameter. Jw is a measure of water pressure, which has an adverse effect on the shear strength of joints due to a reduction in effective normal stress. The above is a complicated empirical factor describing the active stress.

TABLE 4.9 - Classification of individual parameters used in the Tunneling Quality Index Q (After Barton et al 1974)

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

TABLE 4.9 - (cont'd) Classification of individual parameters used in the Tunneling Quality Index Q (After Barton et al 1974)

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

TABLE 4.9 - (cont'd) Classification of individual parameters used in the Tunneling Quality Index Q (After Barton et al 1974)

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

ADDITIONAL NOTES ON THE USE OF THESE TABLES

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

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

Thus the rock tunneling quality - Q - can now be considered to be a function of only three parameters which are crude measures of :

1. Block size (RQD/Jn)
2. Inter-block shear strength (Jr/Ja)
3. Active stress (Jw/SRF)

There are several other parameters that could be added to improve the accuracy of the classification system. One could be the joint orientation. Many case records include the necessary information on structural orientation in relation to excavation axis, but it was not found to be the important general parameter that might be expected.

Many underground observations on Impala will substantiate the above statement. In some tunnels jointing can be found to be parallel with excavation length. However the hangingwall in most cases were found to be intact. It must further be said that the characteristic of the jointing determined the integrity of the hangingwall.

The parameters Jn, Jr and Ja appear to play a more important role than orientation, because the number of joint sets determines the degree of freedom for block movement (if any) and the frictional and dilatational characteristics can vary more than the down-dip gravitational component of unfavourably orientated joints.

Figure 4.7 shows how interrelated the value of the index Q to the stability and support requirements of underground excavations are.

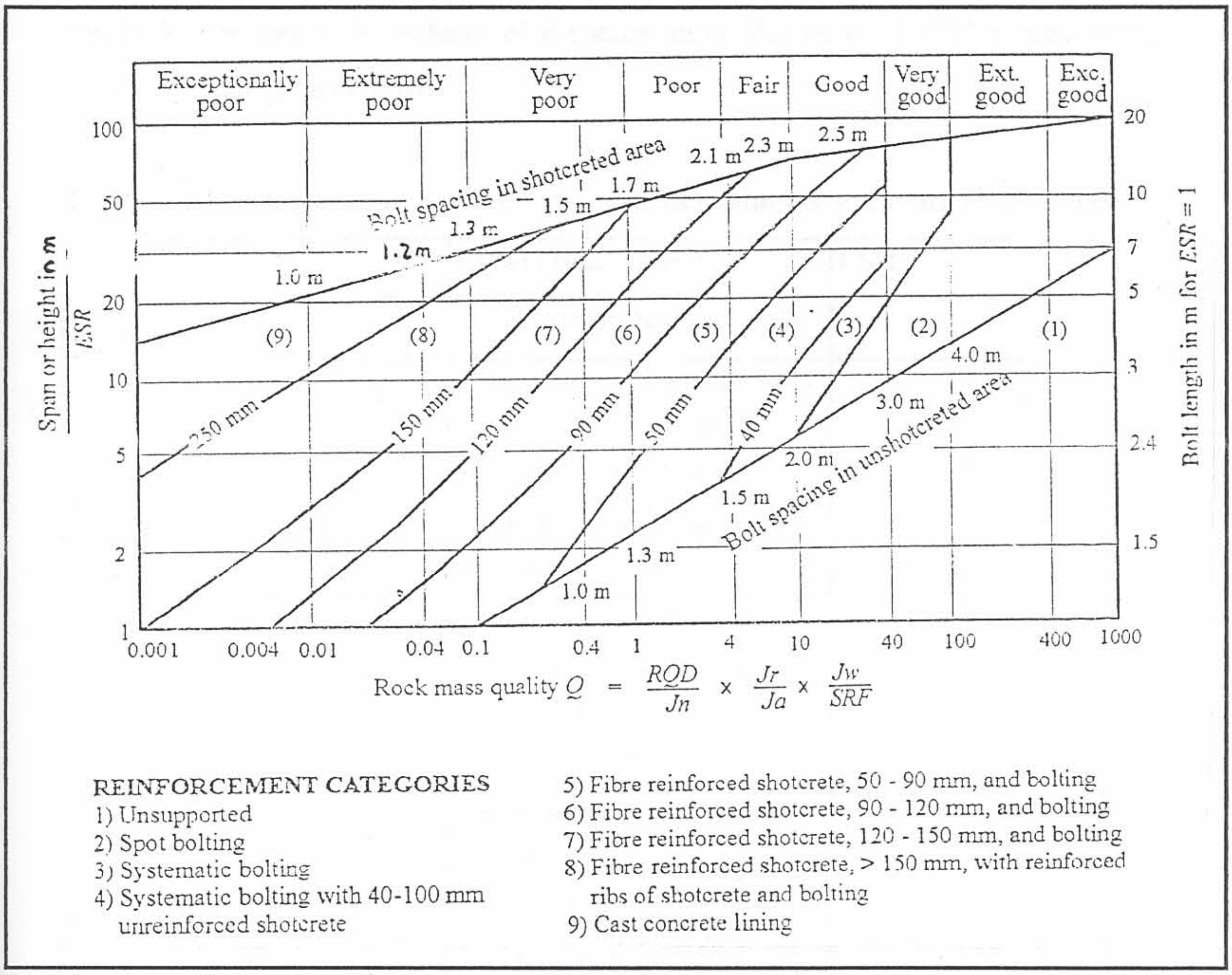


FIG. 4.7 - Estimated support categories based on the Tunneling Quality Index - Q - (After Grimstad & Barton, 1993)

Barton et al (1980) defined an additional parameter which they called the Equivalent Dimensions, De, of the excavation (Figure 18). This dimension is obtained by dividing the span, diameter or wall height of the excavation by a quantity called the Excavation Support Ratio,

Thus

$$De = \frac{\text{Excavation span, diameter or height (m)}}{\text{Excavation Support Ratio ESR}} \qquad (4.5)$$

The value of equivalent support ration (ESR) is related to the intended use of the excavation and to the degree of security that is demanded of the support system installed to maintain the stability of the excavation. Barton et al (1980) suggest the following values (Table 4.10) :

TABLE 4.10 - Equivalent Support Ratio (ESR) values to excavation category

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

The equivalent dimension, De, plotted against the value of Q, is used to define a number of support categories in a chart (Figure 4.7) published in the original paper by Barton et al (1980). This chart has recently been updated by Grimstad and Barton (1993) to reflect the increasing use of steel fibre shotcrete in underground excavation support.

Loset (1992) suggests that, for rocks with $4<Q<30$, blasting damage will result in the creation of new 'joints' with a consequent local reduction in the value of Q for the rock surrounding the excavation. He suggests that this can be accounted for by reducing the RQD value for the blast-damaged zone. Barton et al (1980) provide additional information on rockbolt length, maximum unsupported spans and roof support pressures from the excavation width B and the Excavation Support Ratio ESR:

The length (L) of rockbolts can be estimated from the excavation width B and the excavation support ratio ESR:

$$L = \frac{2+0.15B}{ESR} \qquad (4.6)$$

Where,

B = excavation width

ESR = Equivalent Support Ratio

The maximum unsupported span is given by the following relationship :

$$2 * ESR * Q^{0.4} \qquad (4.7)$$

Based upon analysis of case records, Grimstad and Barton (1993) suggest that the relationship between the value of Q and the permanent roof support pressure P_{roof} is estimated from :

$$P_{roof} = \frac{2 * \sqrt{J_n} * Q^{-1/3}}{3 * J_r} \qquad (4.8)$$

When evaluating all the rockmass classification schemes listed above the Q-System fits the profile to the Impala problem best and is a reliable and simple rockmass classification scheme. However as with many other classification schemes the Q-System relates to the civil engineering discipline. However it is opted to take the Q-system and validate it for use on Impala Mine.

CHAPTER V

CASE STUDIES

Validating the Q-system requires an unsupported excavation on the mine which in today's context is rare. The 1st case study, 10 level crosscut on No. 9-Shaft, Impala Platinum, was developed in 1981 and supported only with spot bolting in wider span sections.. The tunnel was also geologically logged in 1981.

The 2nd case study, chosen to provide a more even spread from good to poor rockmass conditions, is a conveyor decline tunnel at No. 14-Shaft Impala Platinum. This tunnel is currently being developed and needs to be supported immediately because of block fallout's soon after the blasting operations. The above is suitable for the study because a detailed log of the rockmass response to tunneling can be kept for the purposes of evaluating the Q-System.

5.1 Methodology

The Q-system is assumed to include enough information obtained from underground observations to provide a realistic assessment of the rock mass strength and hence the stability of any excavations developed in that rock mass. The Q-system parameters is given below :

$$Q = \frac{RQD}{Jn} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \qquad (5.1)$$

where, RQD is Rock Quality Designation, Jn is the joint number, Jr is the joint roughness, Ja is the joint alteration, Jw is the joint water and SRF is the stress reduction factor.

The methodology used to estimate each of the above parameters from underground observations will be discussed briefly.

5.1.1 Estimating the RQD from Scan line Measurements

The RQD for a rock mass can be calculated from scan line measurements taken underground. A scan line is defined as a line, usually a tape, set on the surface of the rock mass, and the survey consists of counting the number of joints which intersect this line along its length. The scan lines were chosen to be 10 meters long to avoid the possibility that the joint spacing is greater than the length of the scan line.

In the tunnels the tape was laid to the length of the tunnel to form the scan line. The number of joints or planes of weakness that crossed the tape in those ten metres were counted. The number of joints counted divided by the distance of 10 metres gives the number of joints per metre in that direction. This value is equal to S in equation 5.2.

For the width of the excavation (i.e. 0-5m) jointing was counted and divided by the width of the excavation in the 10m section. This value is equal to D in equation 5.2.

Observations included any falls of ground or areas where portions of the hangingwall have been exposed in the vertical direction, where low angle or horizontal joint and other discontinuities were seen. The number of joints or planes of weakness that occur in the vertical direction was also measured and divided by the height distance. The value obtained is equal to V in equation 5.2. The sum of the joint densities for the three directions are calculated using equation 5.2 and RQD calculated using equation 5.3.

$$J_D = V + S + D \tag{5.2}$$

$$RQD = 115 - 3.3 \times J_D \tag{5.3}$$

If the RQD obtained from equation 5.3 was less than 10%; the value entered into the Q-rating equation was 10. If the value obtained from the equation 5.3 was greater than 100% the value entered into the Q equation was equal 100.

5.1.2 Estimating Jn

This number is a measure of the number of joint sets observed at the site. To select the correct discontinuities as joints it was necessary to work according to the following definitions :

Aperture - is the perpendicular distance between adjacent rock surfaces of a discontinuity tendon.

Joint - is a break in the rock of a geological origin, not man made, along which there has been no visible displacement or movement.

Joint set - is a group of joints, which run parallel to each other and a joint system is made up of two or more intersecting joint sets.

Random joints - are joints which do not have the same orientation as the joint sets observed. They are not visible for long distances, only a couple of centimeters or perhaps meters.

The rock mass rating sheet (Table 4.9) was used as a guide to obtain the values entered into the Q rating equation.

5.1.3 Estimating Jr

The Joint Roughness is defined as the measure of the surface unevenness and waviness of the joint relative to its mean plane. This unevenness and waviness will influence the ability of the two surfaces to slide against each other. This has an interlocking effect that prevents the blocks from sliding. In underground observations it is important to distinguish between the unevenness, which is a small-scale feature and waviness which is a large-scale feature (see Figure 5.1). The in-situ shear test provides us with a value which can be defined as the internal angle of friction.

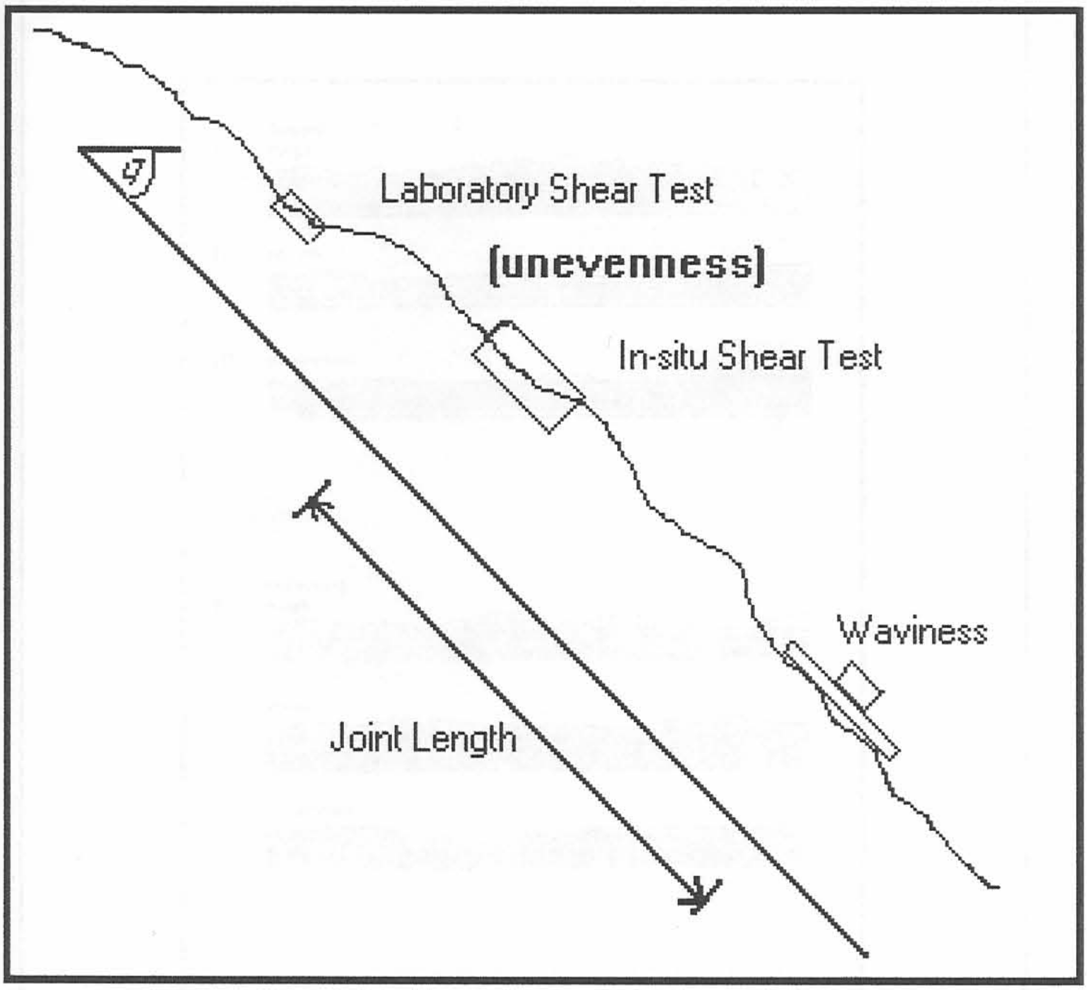


FIG. 5.1 - Different scales of roughness, small scale of laboratory shear test, medium scale of an in-situ shear test and the large scale waviness of the joint (After Barton et al, 1974)

Figure 5.1 shows the difference between small scale and large-scale roughness. This will assist the user to make a final decision about the joint roughness number. The internal angle of friction is used to determine joint alteration number in the absence of mineralogical properties.

The next step was to observe the joint roughness, which in many occasions were tightly closed and were difficult to define in the limited dimensions of the tunnel. Definitions shown in Figure 5.2 and in Table 4.9 were used as a guideline in defining Jr.

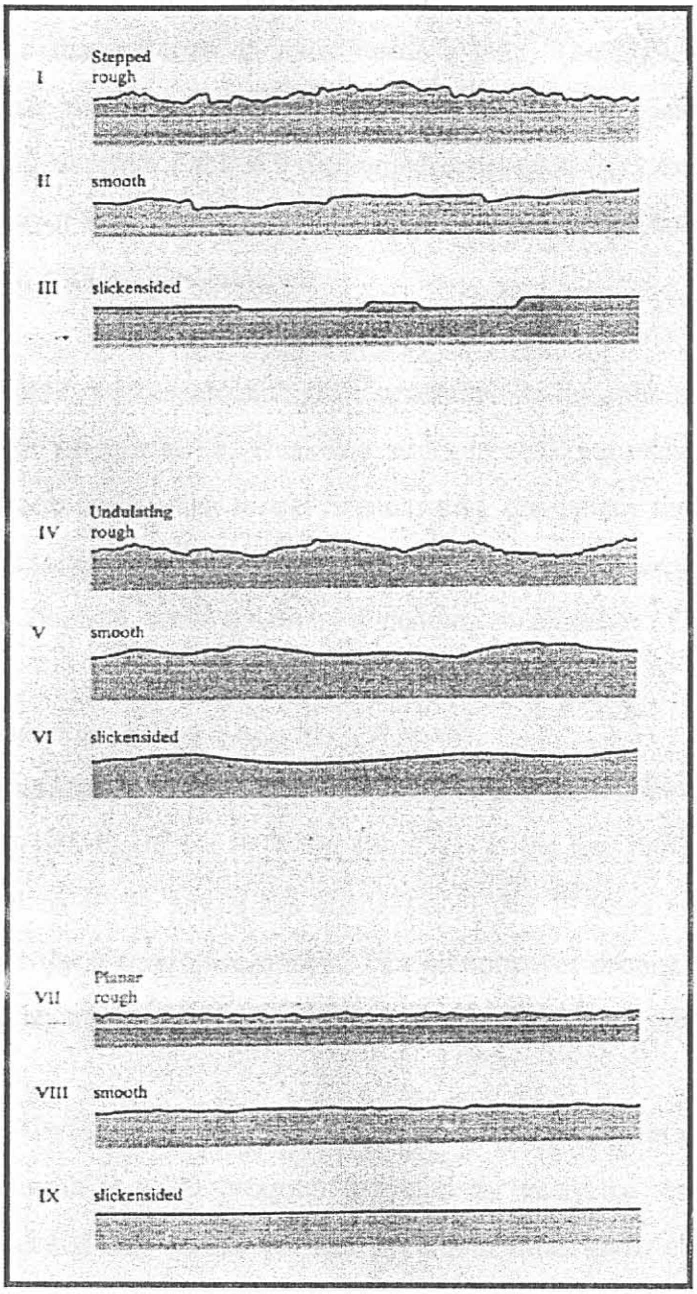


FIG. 5.2 - Profiles of different classes of joint roughness (After Barton et al, 1974)

5.1.4 Estimating Ja, Jw and SRF

Joint Alteration refers to the filling found along the joint plane. The thickness and strength of the filling determines the strength of the joint and its ability to resist slipping. The joint alteration can range from tightly closed joints with no filling to joints with fillings thicker than 3mm or zones of crushed rock. Table 4.9 was used as a guideline. Water is very critical to the stability of excavations and consequently there is an adjustment to de-rate the joint strength due to the presence of water inside a joint. The presence of water will reduce friction or cause the filling in the joint to weather, thus increasing the instability of the hangingwall and sidewall. Table 4.9 distinguishes between a dry excavation or minor inflow, medium inflow or pressure, outwash of joint fillings, large inflow or high pressure in competent rock with unfilled joints etc.

The SRF component includes geological structures in the rock mass. The SRF is divided into three major categories i.e. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated, competent rock, rock stress problems, squeezing rock, plastic flow of incompetent rock under influence of high rock pressure and swelling rock, chemical swelling activity depending on presence of water.

5.1.4.1 Numerical modeling using MINSIM W

It was further necessary to investigate the stress influence to choose the correct category in the stress reduction factor list in Table 4.9. Thus in the first case study the stress analysis was conducted to verify any stress changes that the 10 level crosscut might have been subjected to. In the second case study, 23 Level conveyor decline, which is currently being developed and no stoping had been undertaken, therefore virgin stress conditions prevail.

The program MINSIM W (CSIR, 1997) was used to conduct the stress analysis for the first case study. Minsim is a 3D program designed for analyzing stresses and displacements which are associated with tabular excavations and assumes linear elastic behaviour. It is well suited to the analysis of narrow tabular stopes as those found in the platinum and chrome mines of the Bushveld complex. MINSIM is optimized for deep level mining, but can be used for modeling shallow mines if the finite depth option is selected (Jager and Ryder, 1999).

In its basic form, MINSIM comprises two separate programs. The data input for these programs is in the form of pure ASCII text files, which can be created or edited with any ASCII text editor, such as Microsoft's notepad. However, the input data is fairly complicated, and even slight errors can lead to major problems at a later stage (Jager & Ryder, 1999).

5.1.5 Evaluating the Q-value

The Q-Value was obtained using the guidelines outlined above and in Table 4.9. This was compared to Figure's 4.7, 5.3, 5.4, 5.5 and Table A.1, A.2, A.3, A.4 and A.5 to evaluate the Q-System for underground rockmass classification application.

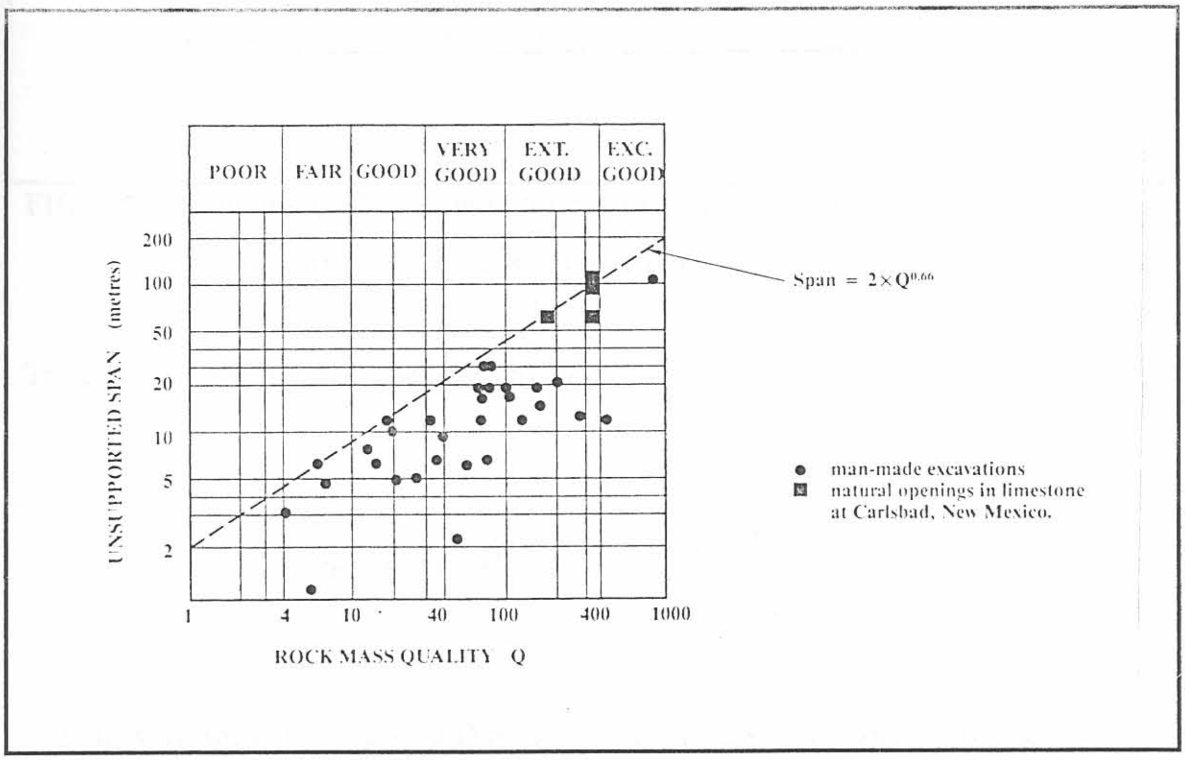


FIG. 5.3 - Man made and natural unsupported excavations in different quality rock masses (After Barton, 1976)

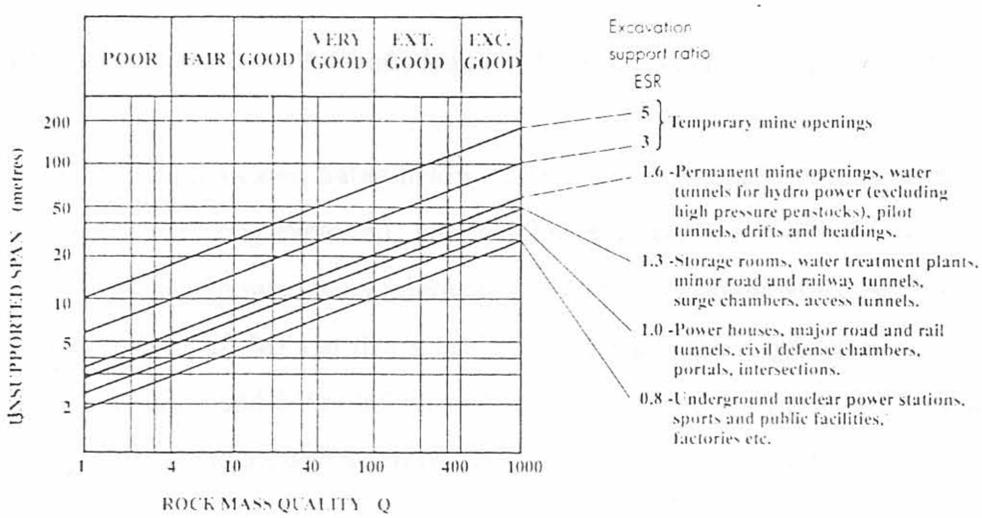


FIG. 5.4 - Recommended maximum unsupported excavation spans for different rock mass quality (Q) and ESR values (After Barton, 1976)

The equation which defines the lines plotted in figure 5.3 can be written as follows :

$$\text{Span of opening} = 2 \times ESR \times Q^{0,4} \qquad (5.4)$$

Alternatively, the critical value of Q for a given excavation span can be expressed in the form :

$$Q = (span / (2 \times ESR))^{2,5} \qquad (5.5)$$

The support guideline issued by Barton et al (1977) can be viewed in Appendix A. Support for categories 1 to 38 is listed in Table A.1 to Table A.4 . Figure 5.5 is a rough guideline of the placing of the categories used in the Q-system.

5.2 10 Level Crosscut, No. 9-Shaft, Impala Platinum (Plan 1 - Appendix E)

10 Level Crosscut is located 640m below surface at No. 9-Shaft where both the Merensky and UG2 Reefs are being exploited. The middling between the two reefs varies between 90 and 100m, with no known stress interaction to date. The initial stress state at No-9-shaft was estimated to be 20 MPa at this specific level using 9,8m/s^2 (gravitational acceleration), 3200 kg/m^3 (density) and 640m below surface to determine the virgin stress condition. The crosscut intersects hangingwall 1 through to hangingwall 5 (See Figure 2.2, p8).

The crosscut average width is 3,0m with sections that widen out to 5.2 metres in places. The only support installed in this crosscut was at these wider sections and spot bolting was conducted in one area. The tunnel was developed in 1981, over the last 20 year period the excavation has been subjected to water, ventilation and possible stress changes. The Q-system was used to obtain a Q-value for this tunnel along seventy seven 10m intervals representing the total tunnel.

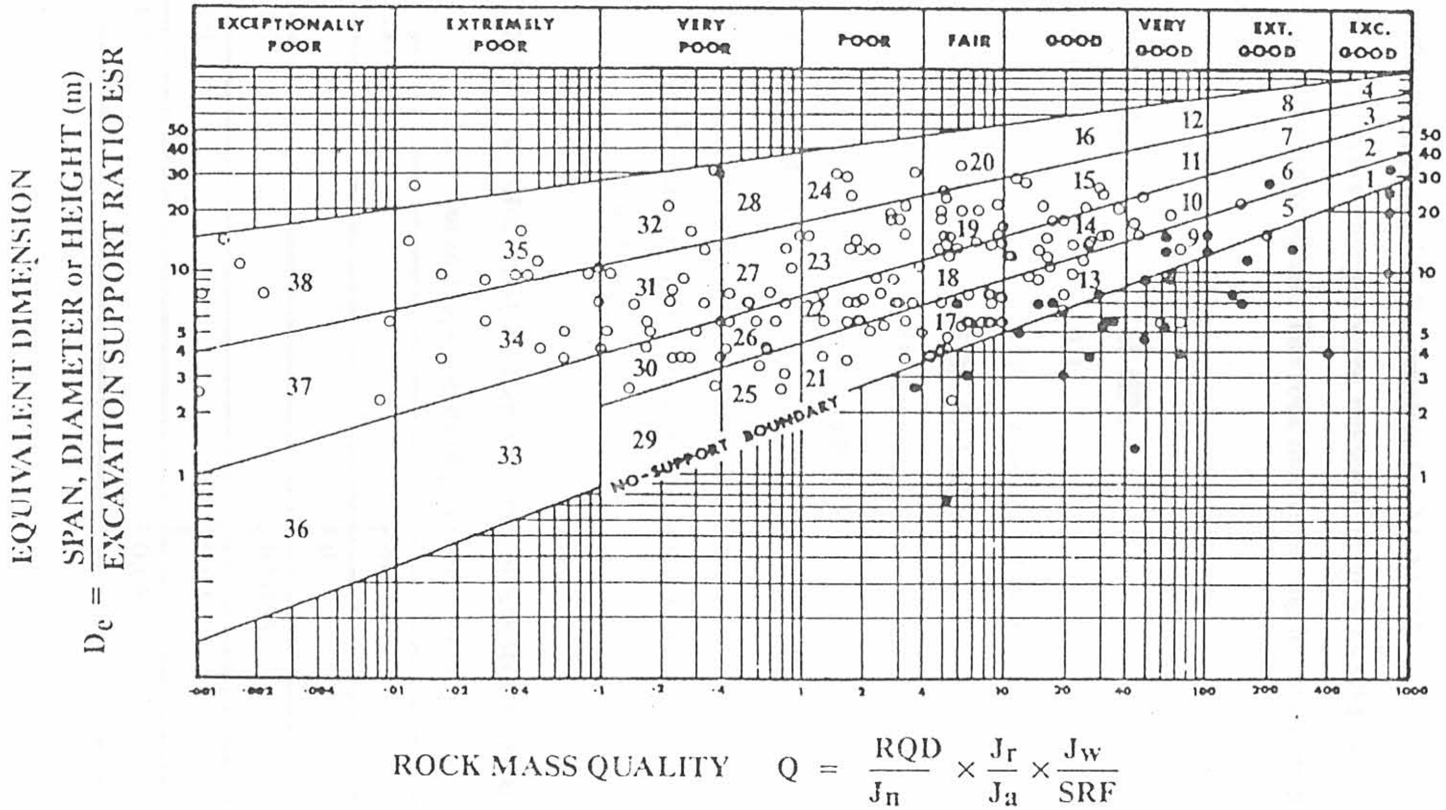


FIG. 5.5 - Recommended support for different rock mass quality (Q) and ESR values (After Barton et al, 1977)

5.2.1 MINSIM W, Computer Modeling

The MINSIM W program was used to determine the excavation stress reduction parameter.

5.2.1.1 Rock Engineering Parameters for MINSIM

The crosscut stress changes were modeled using MINSIM version 3.2 (CSIR, 1997) for Windows software program. The rock mechanics modeling parameters used are stipulated below :

Young's Modulus	-	68 GPa
Poisons Ratio	-	0.2
K-ratio	-	1.0 (Spencer, 1993)
Stoping Width	-	1,0m for both reefs
Coarse size	-	10 metres
Depth option	-	Finite depth
Backfill "Soup" Width	-	1,0m

Backfill stress / strain relationship is used to simulate the behaviour of the 3m x 6m yielding pillars shown below in Table 5.1.

TABLE 5.1 - Backfill "SOUP" Parameters - Stress/strain relationship of the 3m x 6m in stope yielding pillars (After T.J. Kotze, 1997)

Stress (MPa)	Strain
0	0
-5,65	0,0025
-14,0	0,04
-14,3	0,06

5.2.1.2 Mining Steps

Table 5.2 lists the various mining steps involved in conducting a simple stress analysis. The stress analysis we are interested in is the 1st two mining steps. These mining steps provide information regarding any stress changes that the tunnel might have been subjected to.

TABLE 5.2 - Mining steps modeled using MINSIM W

Mining Step	Action
Step 1 – 12/97	Merensky reef mined out – Merensky Crosscut Pillar left intact
Step 2 – 01/98	UG2 reef mined out with 30% Geological losses and a 70m wide crosscut left intact

5.2.1.3 Model

Six on reef windows were placed on the Merensky reef and 7 on-reef windows on the UG2 reef. For the off-reef stress analysis five vertical sheets were used to determine stresses on the various levels. These 5 off-reef windows were not used by choice but rather because the limitation inaccuracy of one window where each line would have represented 40m. A plan view of the UG2 in the model is shown below in Figure 5.6 with the window placing shown in Figure 5.7.

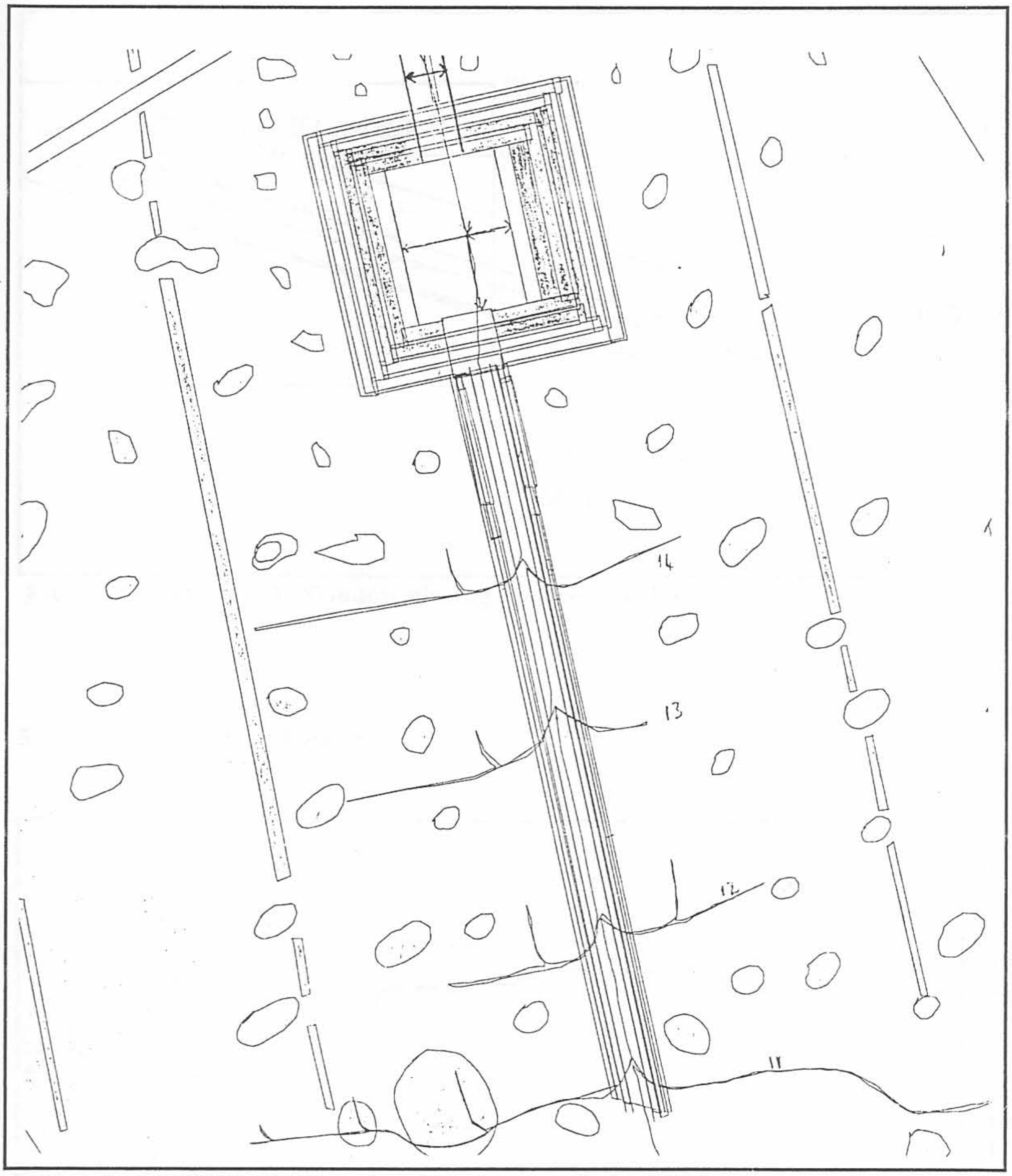


FIG. 5.6 - MINSIM W, Plan view for stress analysis - UG2

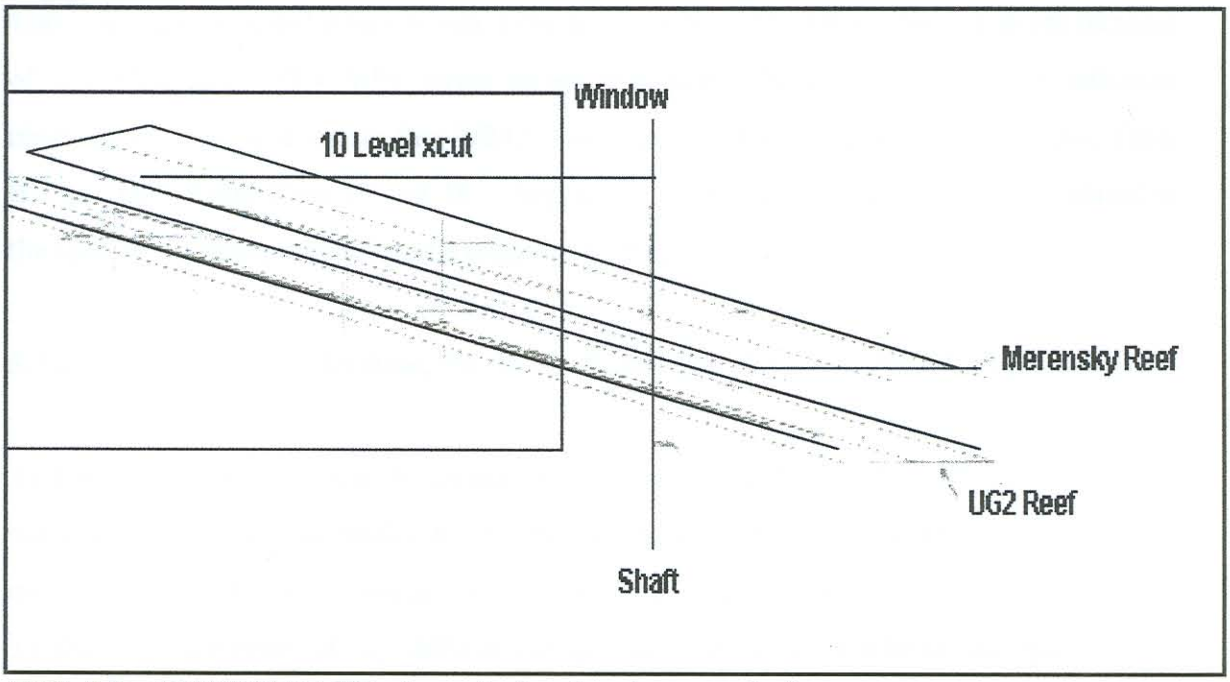


FIG. 5.7 - MINSIM W, Window placing for stress analysis

5.2.1.3 Stress Analysis Results

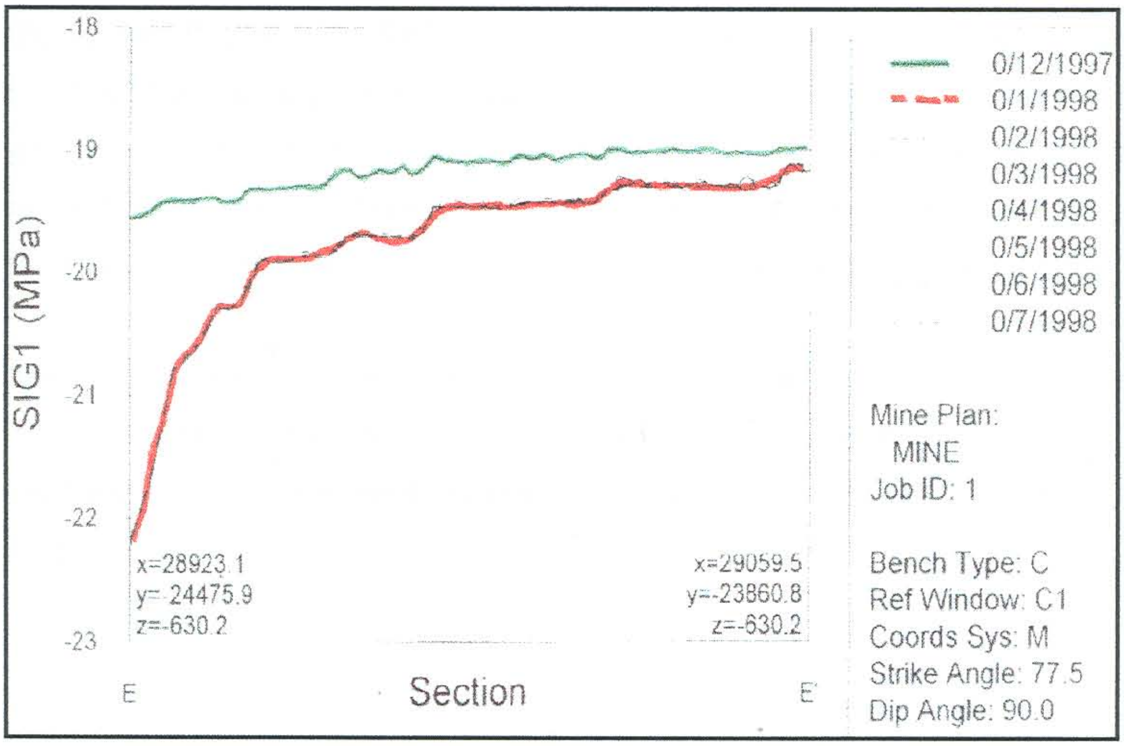


FIG. 5.8 - Section of crosscut showing stress state prior to and after mining of the two reefs

The maximum principal stress Sigma 1 reaches a high of 22,1MPa. This is a stress increase of 2,7 MPa from 19,4 MPa virgin stress condition. The only visible stress influence observed underground was at Peg W24331 where stress fracturing was observed (see Table B.1 site no. 19 and Plate 17 and 18 – Appendix C). The stress fracturing can be related to the spotted anorthosite which has a tendency to fracture when exposed.

5.3 23 Level Conveyor Decline, No. 14-shaft, Impala Platinum (Plan 2 - Appendix E)

23 Level Conveyor Decline is located 1058m below surface at No. 14-Shaft, which is currently mining the Merensky Reef. The conveyor decline extends the depth to which mining can take place, known as the "Deeps". The initial stress (virgin stress) state at No. 14-shaft is in the order of 32,3 MPa at this specific level. There will be no anticipated stress changes which will enhance the instability of the excavation. The conveyor decline is currently being developed in footwall 16 anorthosite, which can be viewed in the generalised geological succession which is shown in figure 2.2. The decline is developed with an average width of 5,6m with 30cm over break in most sections.

The support installed in the decline consists of 3,0m long, 16mm diameter, shepherd crooks which are full column grouted at a spacing of 1,0m on both dip and strike. The tunnel has given rise to a fair amount of concern with hangingwall and sidewall stability with fallout's up to 2m high occurring. Commonly these fallout's occurred almost immediately following the blast. This excavation has been subjected to water and ventilation for two months.

The Q-value rock mass classification was conducted using Table 4.9 as a guideline. 12 Stations were Q-rated at 10m intervals. The highest value was determined to be at 1.3 and the lowest value was 0.6 thereby ranging between poor and very poor as shown in Table B.1).

5.3.1 23 Level Conveyor Decline Stress Analysis

The stress analysis showed that there is little stress change in the conveyor decline. In the future it is anticipated that stress change will not influence the stability of the excavation. The virgin stress state will have to be taken as the ultimate stress state, namely 32,14 MPa. The K-Ratio for this depth on Impala is assumed to be one, which means a hydrostatic stress state (Impala Platinum Ltd., 1999).

5.4 Q-Rating information analysis of 10 level crosscut and 23 level conveyor decline

The ratings obtained (See Table B.1 and B.2) were scrutinized in detail to determine *critical Q-value parameters (*See Table B.3). The Q-Values were plotted and showed a fairly uneven spread throughout the exceptionally good to fair Q-Value classification of 10 level crosscut to a poor very poor classification of 23 level conveyor decline (see Figure 5.9).

The rockmass in the 10 level crosscut was classified as good (45%), very good (25,9%), exceptionally good (12,9%) and fair (14,9%). The one poor ground condition case (1,3%) is mainly due to a multiple shear zone which is not supported. Six of the seven fair ground condition cases are also not supported. One of these sections is a wider section and is supported with 1,8m long shepherd crooks, spaced 1m apart.

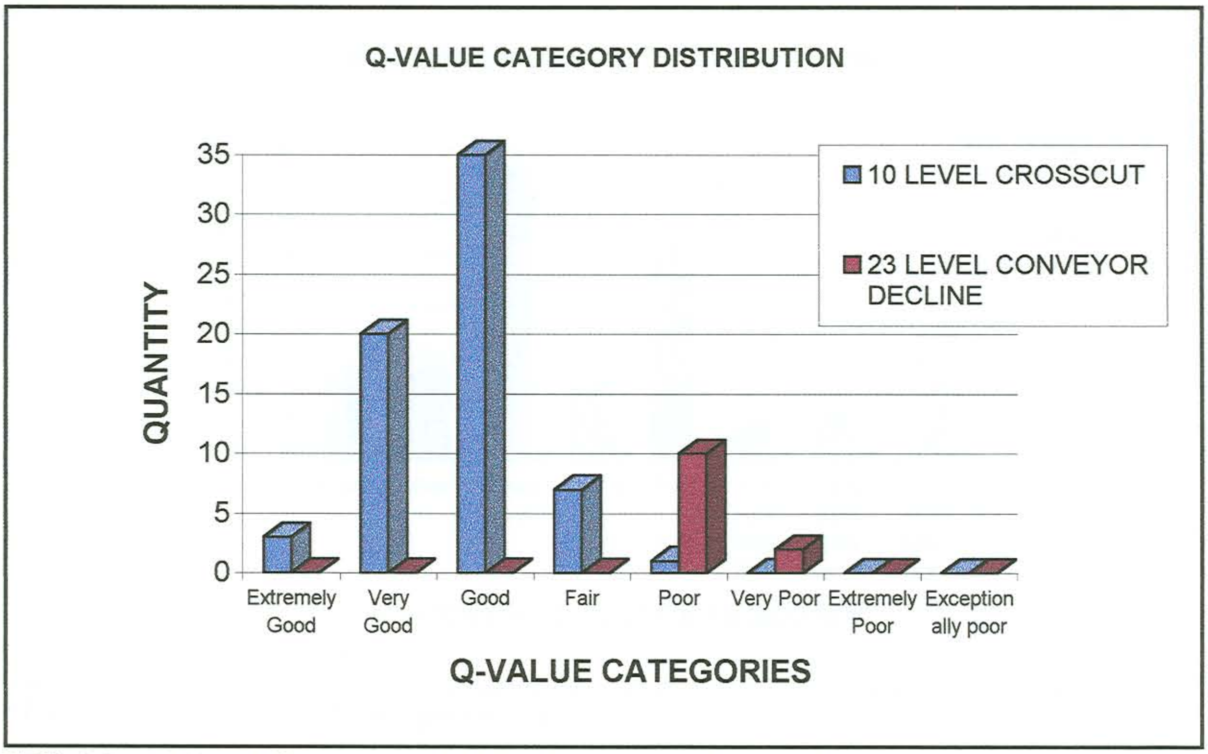


FIG. 5.9 - 10 Level Crosscut and 23 level conveyor decline Q-value distribution

The conveyor decline critical Q-value parameters mainly consist of poor (83%), very poor (17%). For this case the excavation will be compared to supported workings (See Figure 5.4, p 70). The 3,0m long shepherd crooks are spaced 1m apart on dip and strike. These parameters / categories were further broken down in the sub headings describing the typical ground condition characteristic (see Table B.4).

In Figure 5.10 it is shown that the ground condition by means of *joint number category* mainly consists of 1 joint set which represents 35% of the tunnel that was rock mass classified using the Q-system, followed by two joint sets (25,9%) and the rest of the plot tends to the massive rock mass with fewer joints presence.

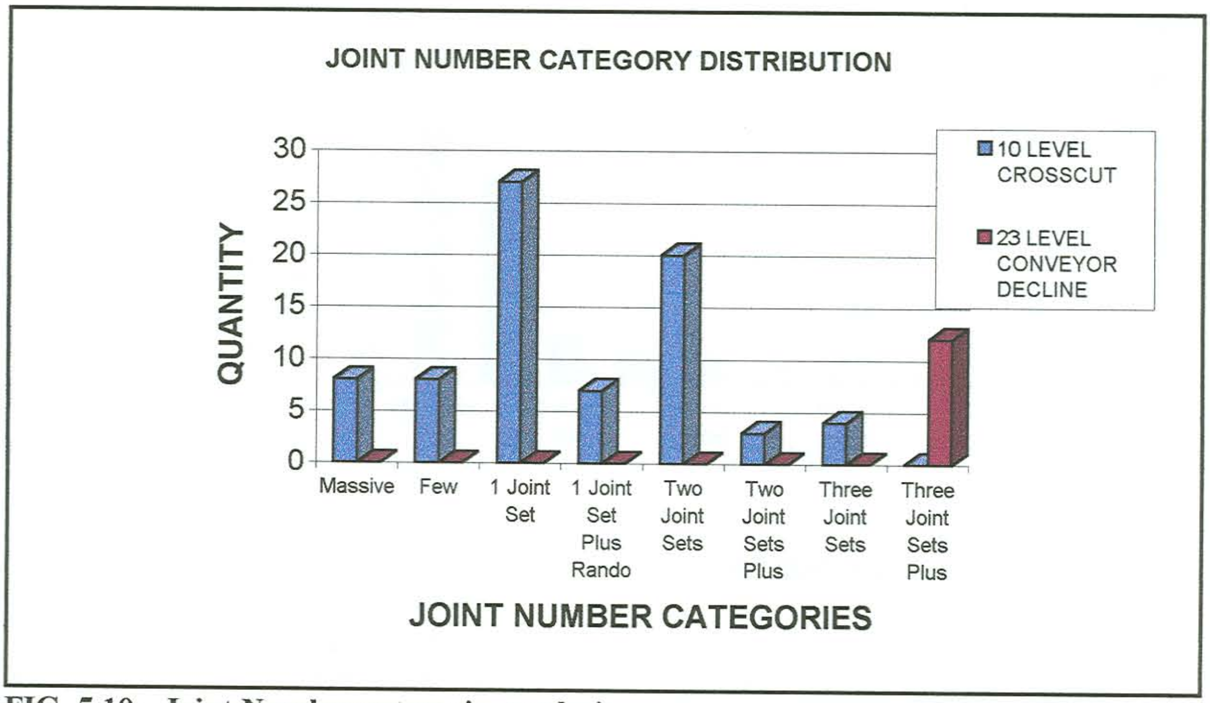


FIG. 5.10 - Joint Number categories analysis

In Figure 5.10 it is shown that the ground condition by means of joint number category in the 23 level conveyor decline mainly consist of 3 joint sets plus random (i.e. 100% of all the stations q-rated). The joint number category was analysed and plotted against the average q-value in Figure 5.11. Thus the average Q-value (1,2) obtained in the 23 level conveyor decline compare quite well with the three joint sets observed underground in the excavation.

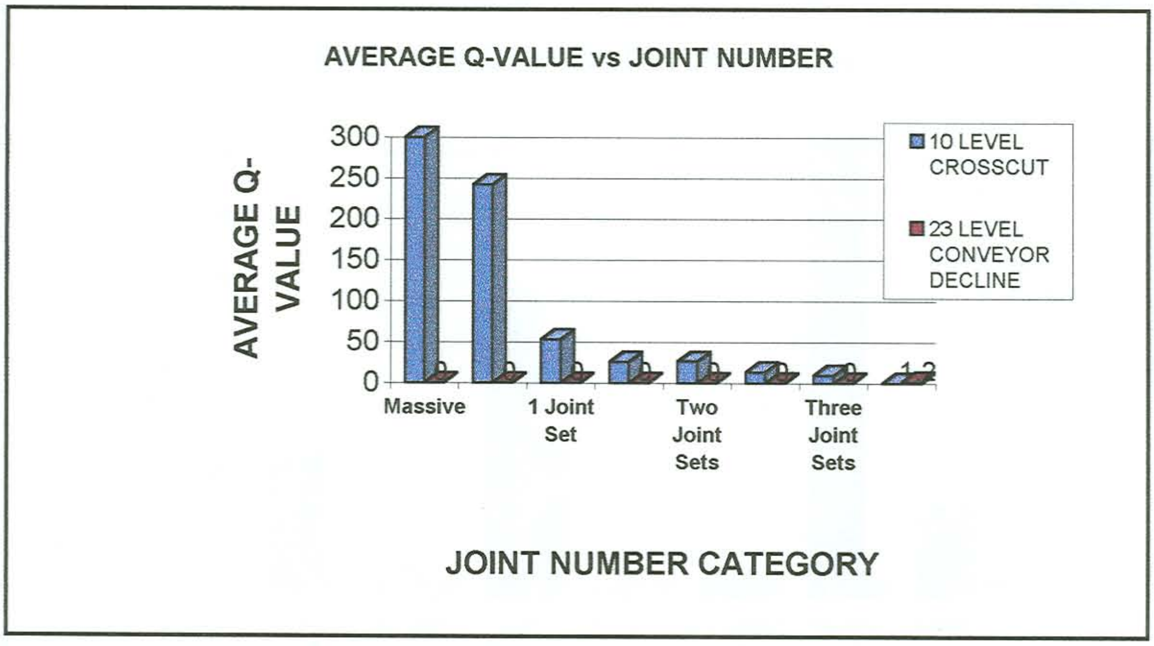


FIG. 5.11 - Joint Number categories vs Average Q-values

It is evident in Figure 5.12 that the 10 level crosscut *jointing roughness* consist mainly of rough / irregular planar (45%), rough / irregular undulating (29,8%) and joint spacing further than 3m (12,9%). This is a further indication of the very good ground conditions that was experienced in the crosscut over the last 20 years.

The spread of the joint roughness categories (Figure 5.12) in the 10 level crosscut is not so even from joint spacing >3m through to smooth planar parameters. This is also revealed in the plot against correlation with the Q-Values (see Figure 5.13). The 23 level conveyor decline jointing roughness however consists mainly of the rough / irregular undulating category (see Figure 5.12). This can be viewed in Plate 26 and 27 (Appendix D). This is slightly contradicting with the very poor ground conditions experienced in the conveyor decline soon after the blasting operation. This joint roughness is supported by the measured internal angle of friction of 35 degrees (see Plate 28 - Appendix D). There can be a slight adjustment due to size affect. The two samples were taken from the sidewall and were hand specimen size.

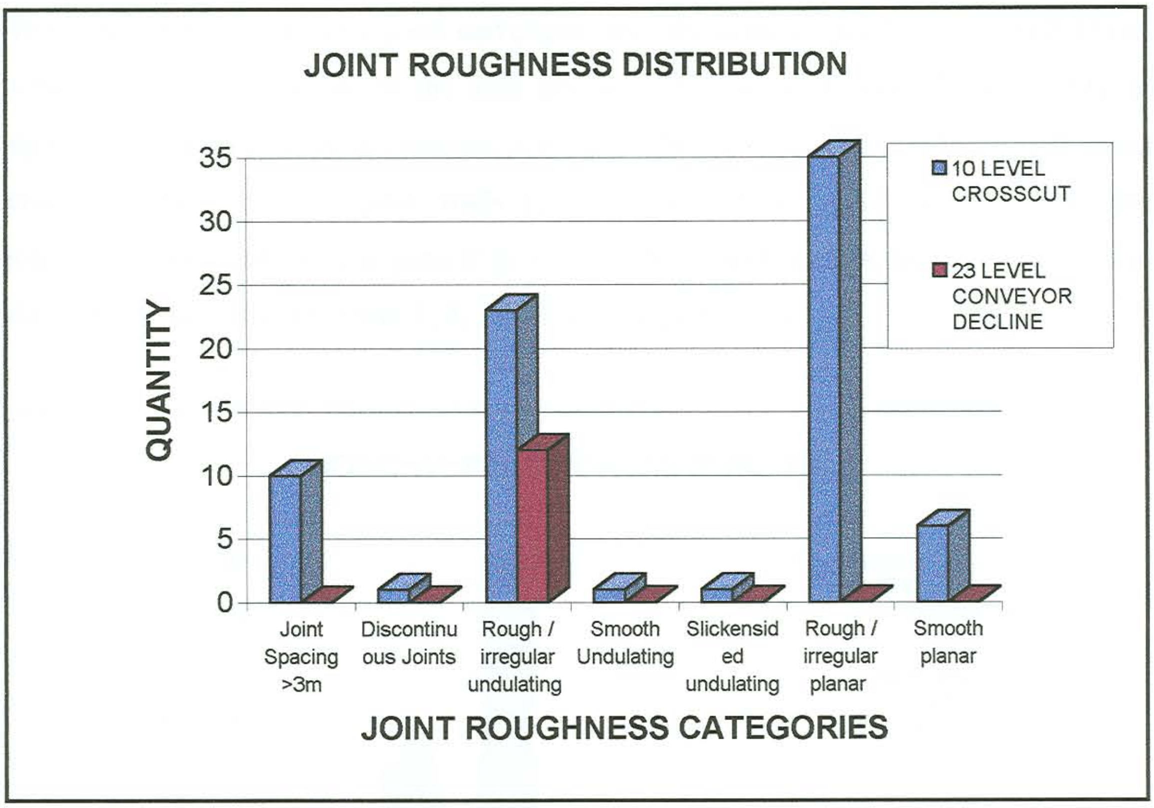


FIG. 5.12 - Joint roughness categories

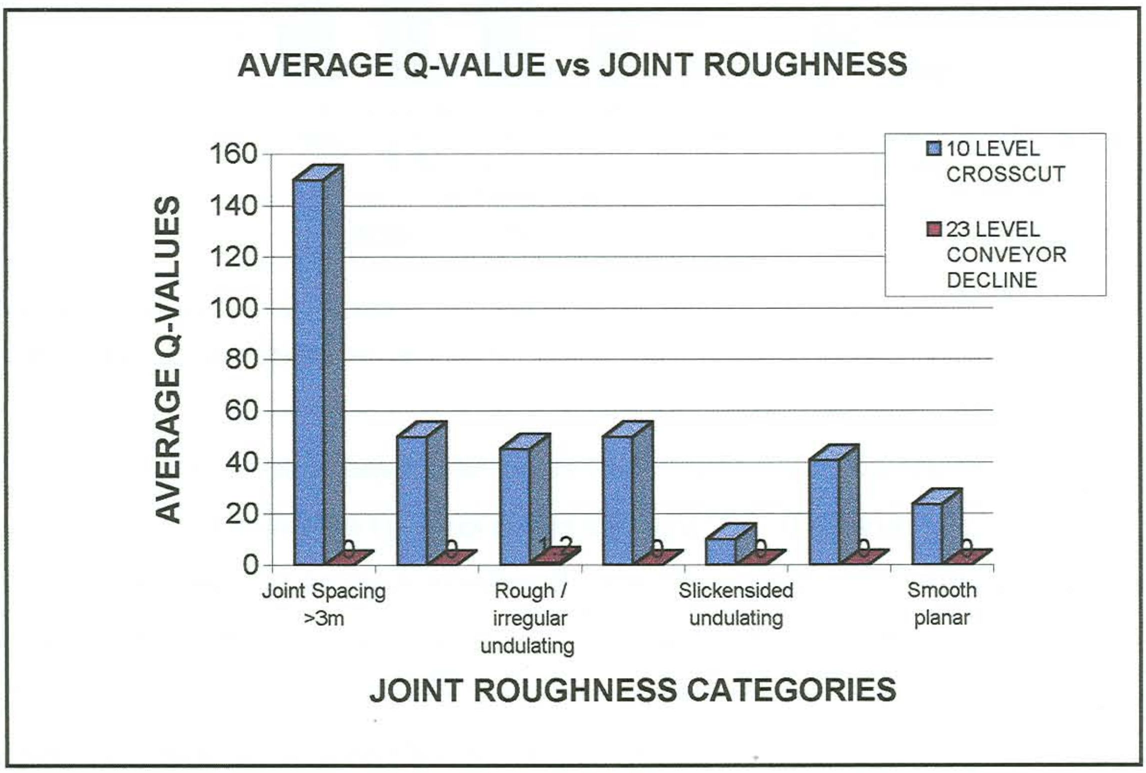


FIG. 5.13 - Joint Roughness categories vs Average Q-value

The *joint alteration* shows a good correlation with the average Q-values (Figure 5.15) with a fairly normal distribution of the joint alteration sub parameters (see Figure 5.14). The significant joint alteration parameters are the slightly altered joint wall-non soft mineral coating (52%), unaltered joint walls (28,5%) and tightly healed (12,9%). The above information substantiates the general ground condition and justifies the excavation stability throughout the years (see Plate 1, 2, 3, 7, 8,13 - Appendix C).

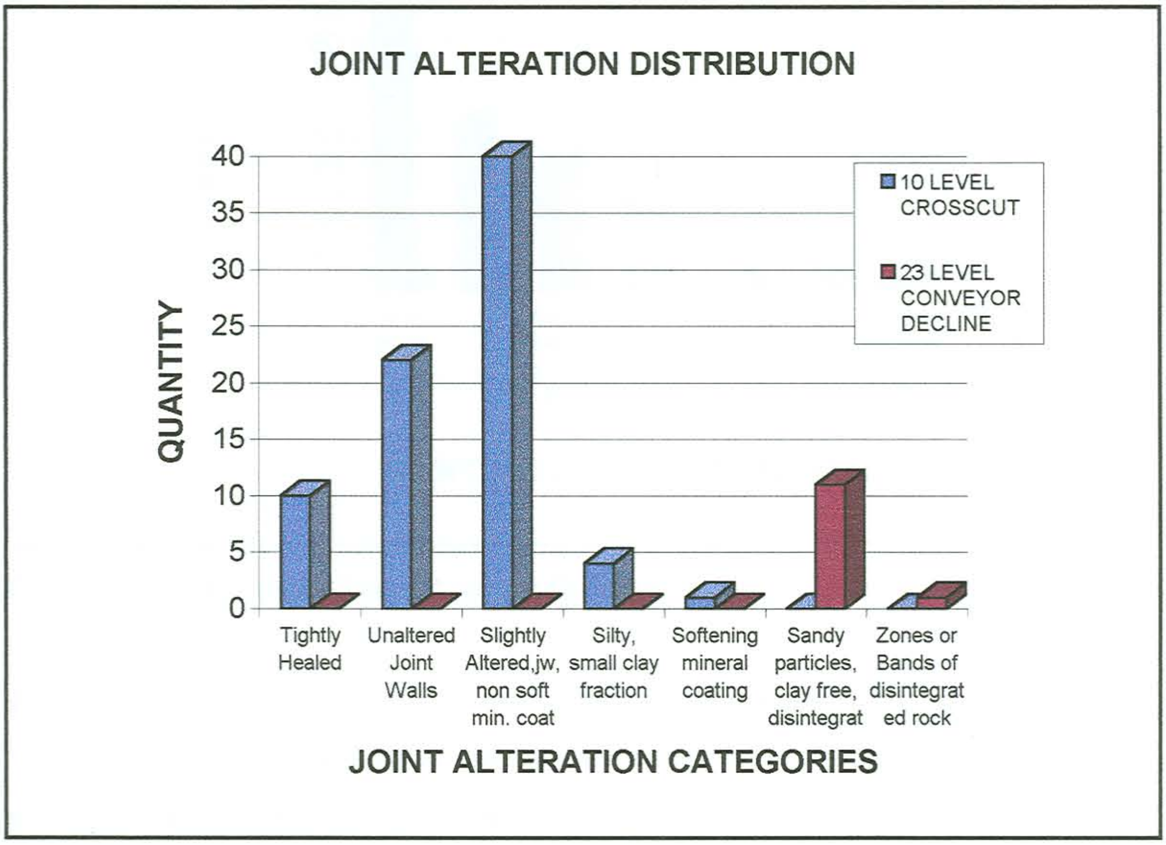


FIG. 5.14 - Joint Alteration categories

The joint alteration sub parameters in Figure 5.14 provide a skew distribution with a good correlation with the average Q-values shown in Figure 5.15. The most significant joint alteration parameter is the sandy particles, clay free, disintegrated rock (83%) and zones or bands of disintegrated rock (17%). This can be viewed in Plate 26 and 27 (Appendix D). The above information substantiates the general ground condition and justifies the excavation instability so soon after the blasting operation .

The plot in Figure 5.14 supports poor ground conditions in the 23 level conveyor decline and can be taken as a general occurrence on Impala when poor ground conditions are found which are structurally controlled and the stability of the excavation is influenced.

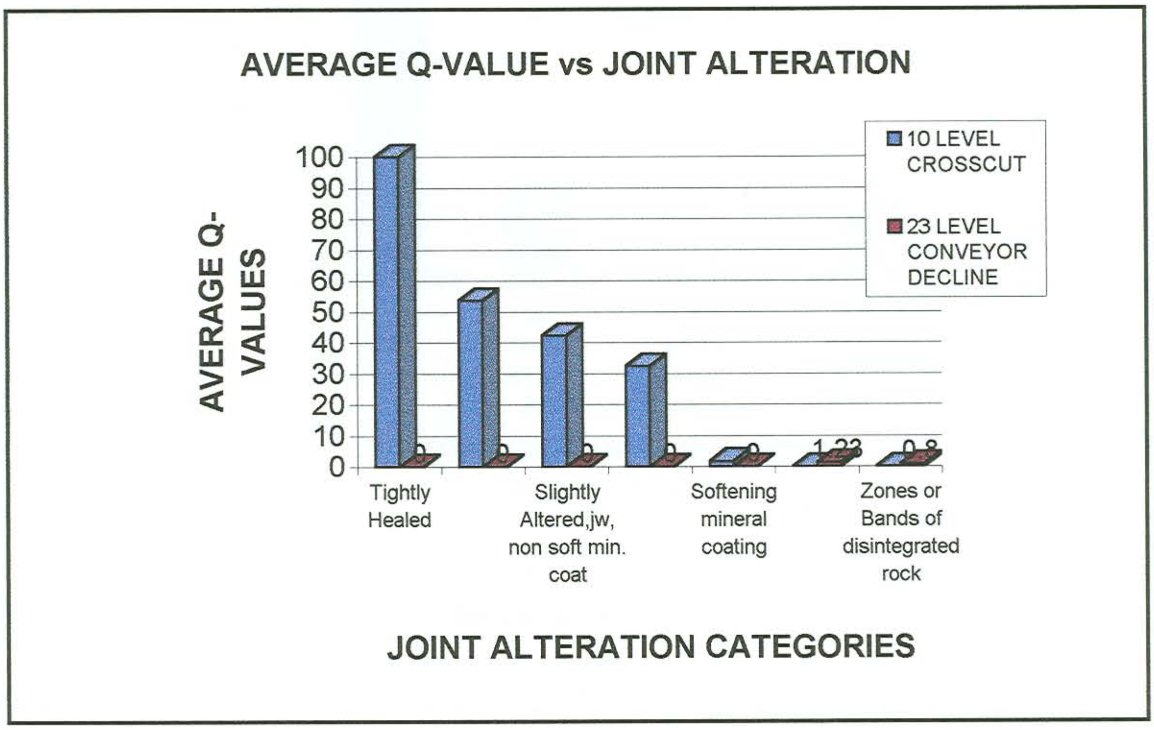


FIG. 5.15 - Joint Alteration vs Average Q-value

The last parameter critically reviewed was the stress reduction factor and is shown in Figure 5.16. A high amount of medium stress (78%) was significant of the crosscut with single shear and multiple shears with 21% and 1% respectively (Plate 7, 8,14, 15 and 16 - Appendix C). There is also a good correlation with the average Q-values (see Figure 5.17). The main and only parameter described in the 23 level conveyor decline is loose open joints, sugar cube, heavy jointed, with the emphasis on loose open joints (see Plate 23, 24, 25, 26, 26 and 27 - Appendix D).

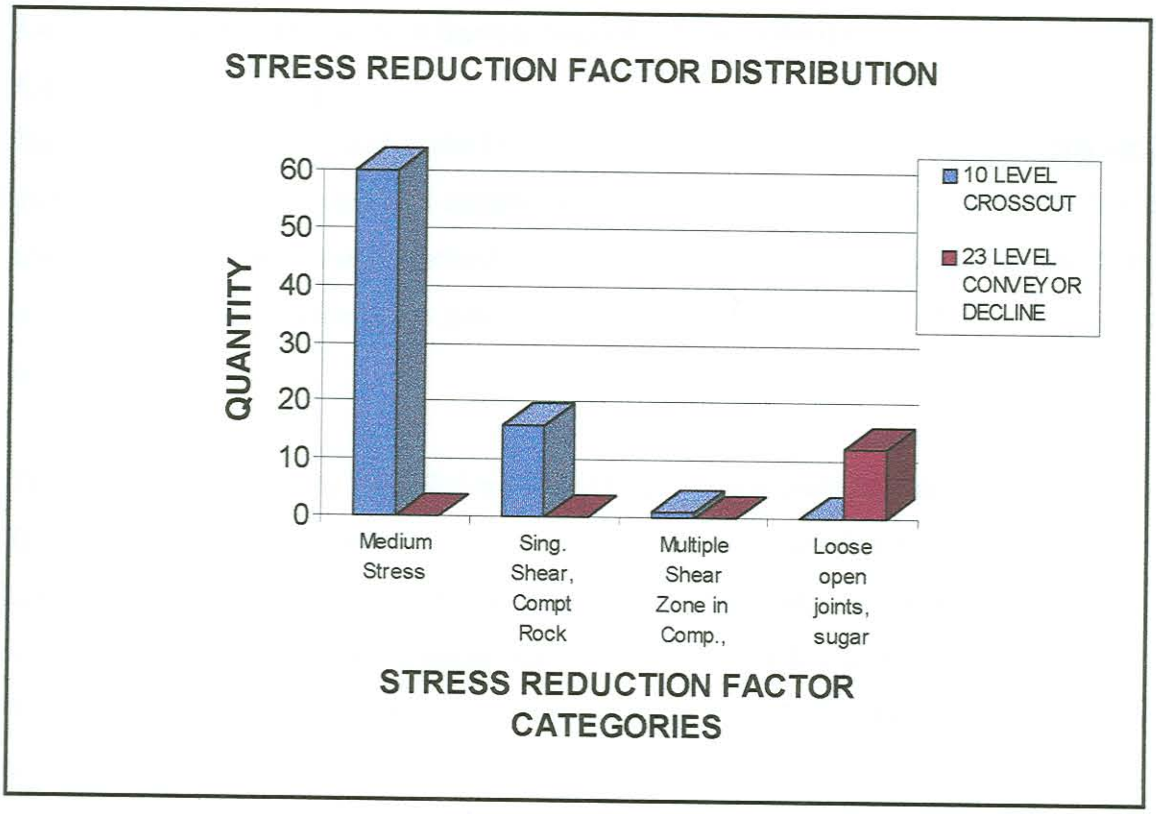


FIG. 5.16 - Stress Reduction Factor categories

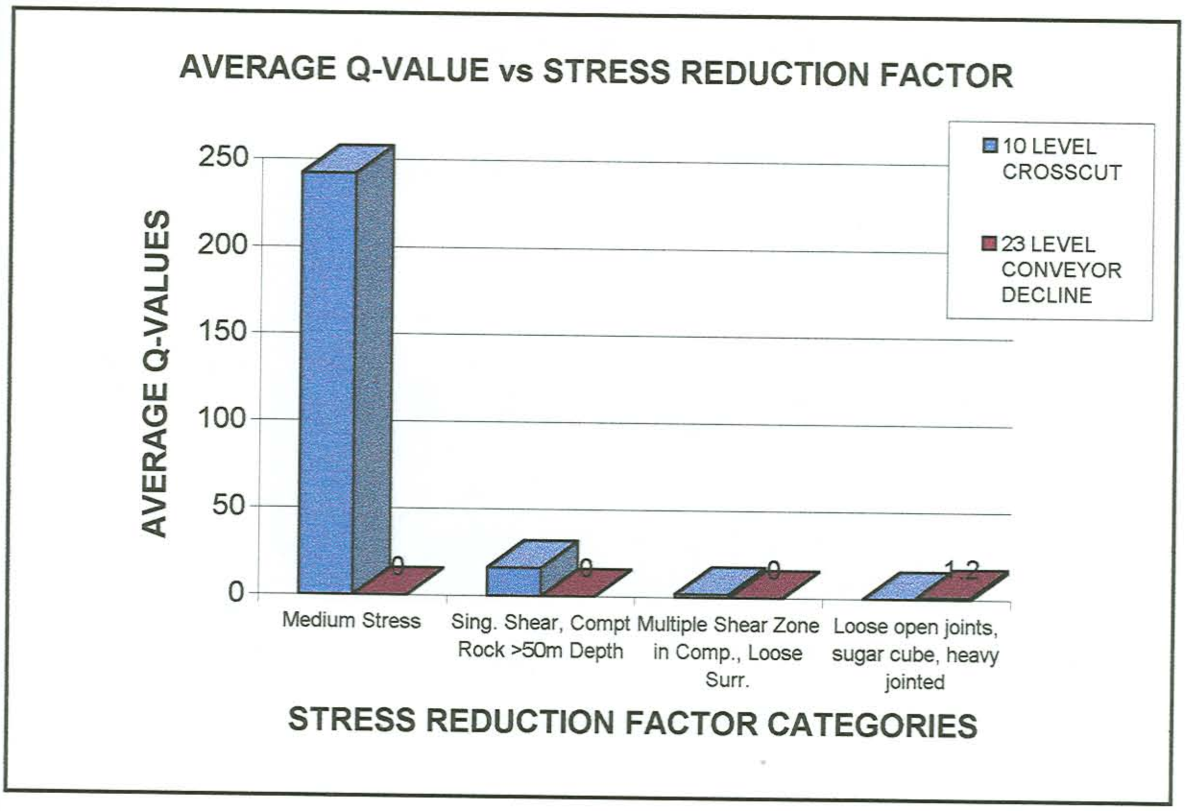


FIG. 5.17 - Stress Reduction Factor vs Average Q-value

5.4.1 Q-Rating Comparison to Barton Support Requirements

5.4.1.1 Unsupported spans

The data (Q-values) obtained in the 10 level crosscut at No. 9-Shaft provided a data set for unsupported spans that could be compare to the Barton graphs (Figure's 4.5, 4.6, 5.3, 5.4 and 5.5). The data is used to check whether the observations made satisfy the Barton criteria. Where correlation is not good, modification to the system for application at Impala would be necessary.

The calculated data set in Table B.1 was used to produce a Barton comparison - Equivalent Dimension vs Q-values Table B.3. A scatter plot was constructed to compare to the Barton unsupported line (see Figure 5.18). It must be noted that only the areas where no support was installed in the 10 level crosscut was used to produce the scatter plot (see Plate 1,2, 3,4,5,7,8, 11,12,17 and 18). An excavation support ratio of 1,6 was used for a permanent mine opening. The data provided some interesting information :

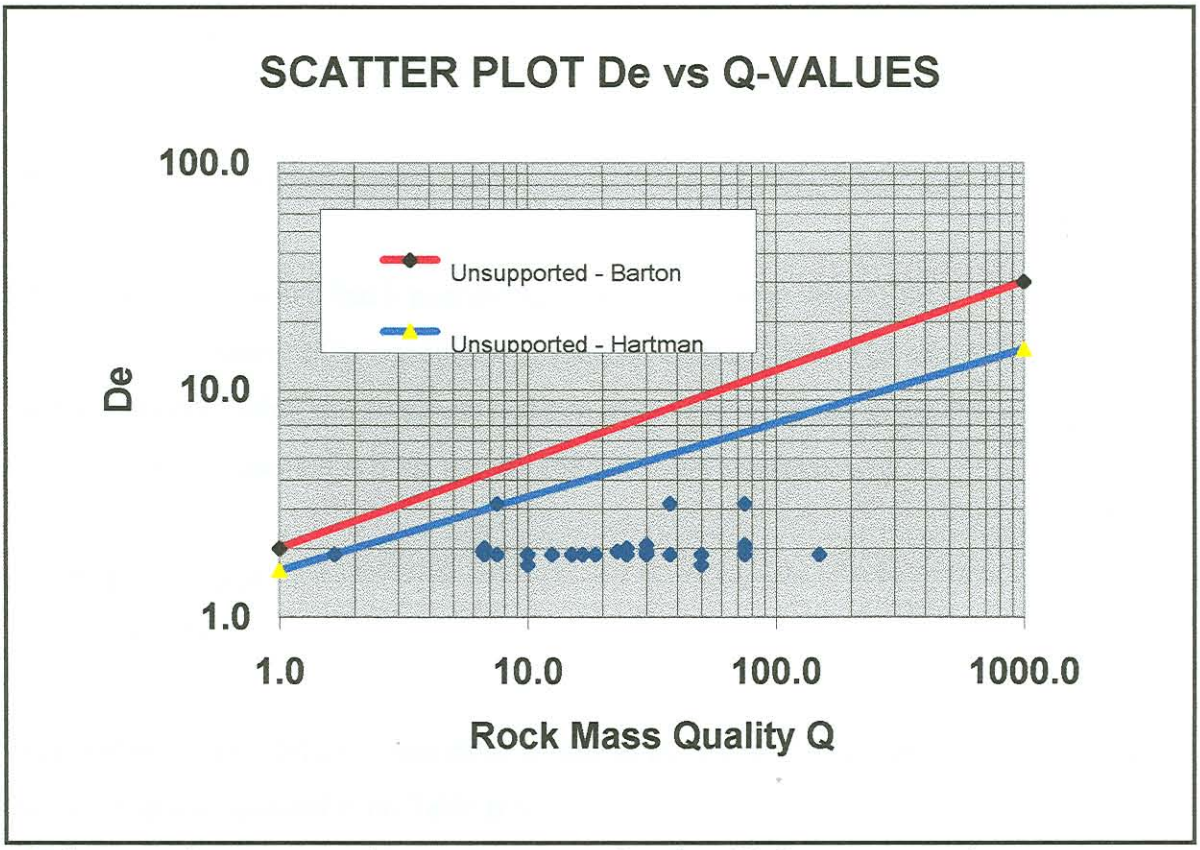


FIG. 5.18 - Scatter plot of De vs Rockmass Quality (Q) for an excavation support ratio of 1,6 (Permanent mine openings) 10 Level Crosscut

a) The scatter plot for equivalent dimension against the Q-value to a log scale was first scrutinized. All the data grouped underneath the unsupported line. According to Barton in this specific case the area should not have been supported to stabilize the excavation. This was just the case. Thus 20 years stability for a section which was not initially rated using the Barton technique as prescribed in Table 4.9 (p56-57).

b) However this recording of data necessitates the need to alter the “no support” line of Barton to an altered, more conservative, Hartman unsupported line for Impala Platinum Limited (see Figure 5.18). The calculations below support the above mentioned statement.

The Barton formula is shown below :

$$Span = 2 * ESR * Q^{0.4} \tag{5.6}$$

Where, ESR (equivalent support ratio) is equal to 1,6 for this specific case.

An altered unsupported line is postulated for equivalent span ratio (ESR) of 1,6 (i.e. permanent mine openings) for Impala Platinum Mine. The altered line was constructed using the maximum and minimum value obtained in the data set. The ESR in the equation is fixed to 1,6. The following are unknowns :

a) Q - power value
b) constant (2)

The following calculations were done to determine the altered unsupported line equation with the values obtained from Table B.8 :

Site No. 31

Q = 1,7 ; Span = 3 ; De = 1,9

and

Site No. 19

Q = 7,5 ; Span = 5 ; De = 3,1

The above values were substituted into the following equation :

$$Span = A * ESR * Q^{Y} \tag{5.7}$$

where,

A and Y are unknowns

Thus

$$A = \frac{1,875}{1,7^{Y}} \tag{5.8}$$

and

$$A = \frac{3,125}{7,5^{Y}} \tag{5.9}$$

thus, 5.8 and 5.9 can be written as,

$$\frac{3,125}{7,5^{Y}} = \frac{1,875}{1,7^{Y}} \tag{5.10}$$

$$3{,}125 * 1{,}7^{Y} = 1{,}875 * 7{,}5^{Y} \tag{5.11}$$

$$1{,}7^{Y} - 0{,}6 * 7{,}5^{Y} = 0 \tag{5.12}$$

The equation in 5.10 was resolved using the Gauss Algorithm root finding method (De la Rosa et al, 1984) :

Thus from 5.11,

$y \neq 0$ follows that $y = 0{,}3442$ (5.13)

Substitute (5.13) into (5.8)

$$A = \frac{1{,}875}{1{,}7^{(0{,}3442)}} \tag{5.14}$$

$$A = 1{,}56 \tag{5.15}$$

The above is a perfect solution following the plot in Figure 5.18 and Figure 5.19 below. Thus the Hartman modified formula can be written as follows,

$$\text{Unsupported } Span = 1{,}56 * ESR * Q^{0{,}3442} \tag{5.16}$$

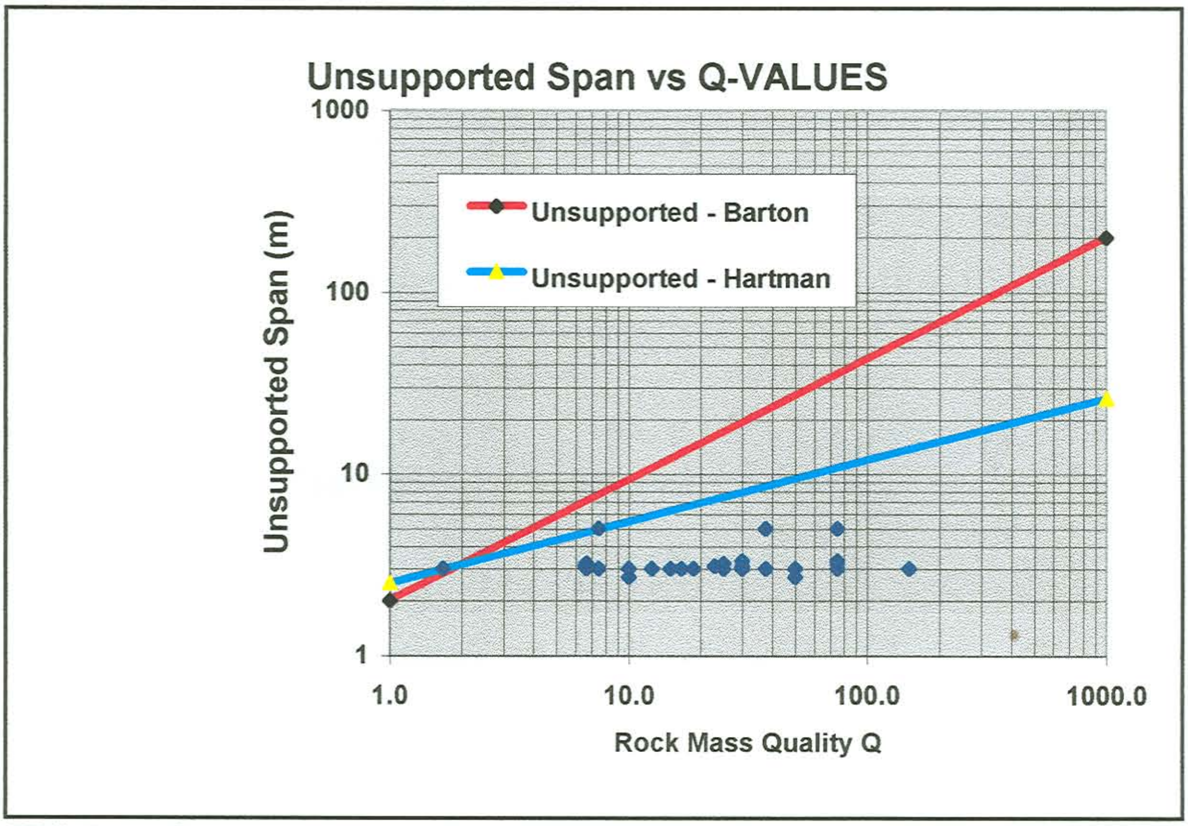


FIG. 5.19 - Scatter plot Unsupported span vs Rockmass Quality (Q)

c) The Data had to be used to compare it with Barton's - Man-made and natural, unsupported excavations see Figure 5.3 (p69). Barton's (1976) unsupported excavations case studies in different quality rock masses created the following formula

$$Span = 2xQ^{0.66} \qquad (5.17)$$

The data does not satisfy the unsupported line. Therefore a more conservative approach was adopted and the line had to be altered as with the above unsupported line (see below for calculations) :

Using the following data,

Site No. 31

Q = 1,7 ; Span = 3 ; De = 1,9

and

Site No. 19

Q = 7,5 ; Span = 5 ; De = 3,1

The above values were substituted into the following equation :

$$3 = 2 * 1{,}7^{Y} \quad (5.18)$$

$$5 = 2 * 7{,}5^{Y} \quad (5.19)$$

Thus following the Gauss algebraic step method (De la Rosa et al, 1984) equation's 5.17 and 5.18 can be written as,

$$0 = 0{,}6667 * 1{,}7^{Y} \quad (5.20)$$

$$0 = 0{,}4 * 7{,}5^{Y} \quad (5.21)$$

The above functions 5.19 and 5.20 can be stepped subtracted according to the Gauss step method,

thus $$0 = 0{,}4 * 7{,}5^{Y} - 0{,}6667 * 1.7^{Y} \quad (5.22)$$

and $$y = 0{,}3441921 \quad (5.23)$$

thus giving the following modified Hartman formula,

$$Span = 2 * Q^{0,3441921} \tag{5.24}$$

5.4.1.2 Supported spans

The Q-ratings obtained in the 23 level conveyor decline at No. 14-Shaft is a data set of 12 points with 10m intervals to compare to the Barton graphs (see Figure 5.3 & 5.4, p68, and Figure 5.5, p69). It was necessary to include the exceptionally poor to the poor range Q-values in the Barton graph. The data set was plotted on a scatter plot graph (see Figure 5.20 and Figure 5.21) using the calculated data set from Table B.9. An excavation support ratio of 1,6 was used as this is a permanent mine opening.

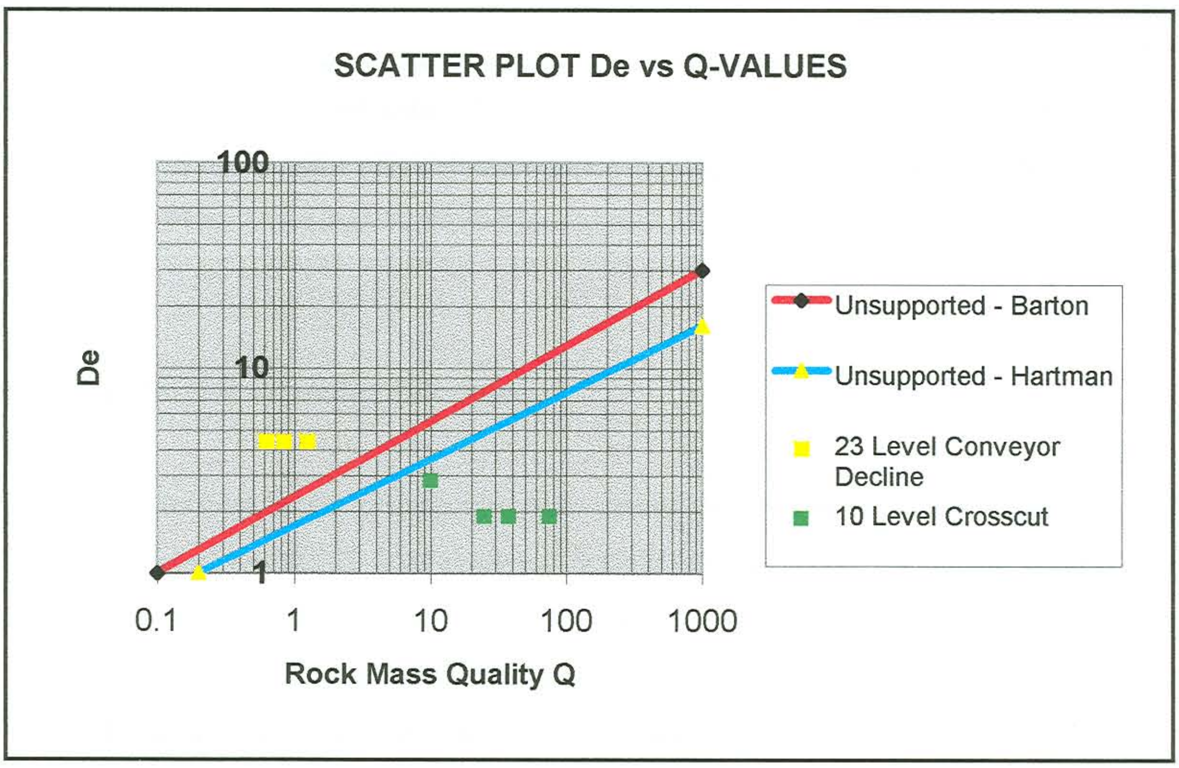


FIG. 5.20 - Scatter plot for supported data points on an Equivalent Dimension vs Rock Mass Quality : Q-Value Excavation Support Ratio of 1,6 (Permanent Mine Opening) 10 level crosscut and 23 Level Conveyor Decline

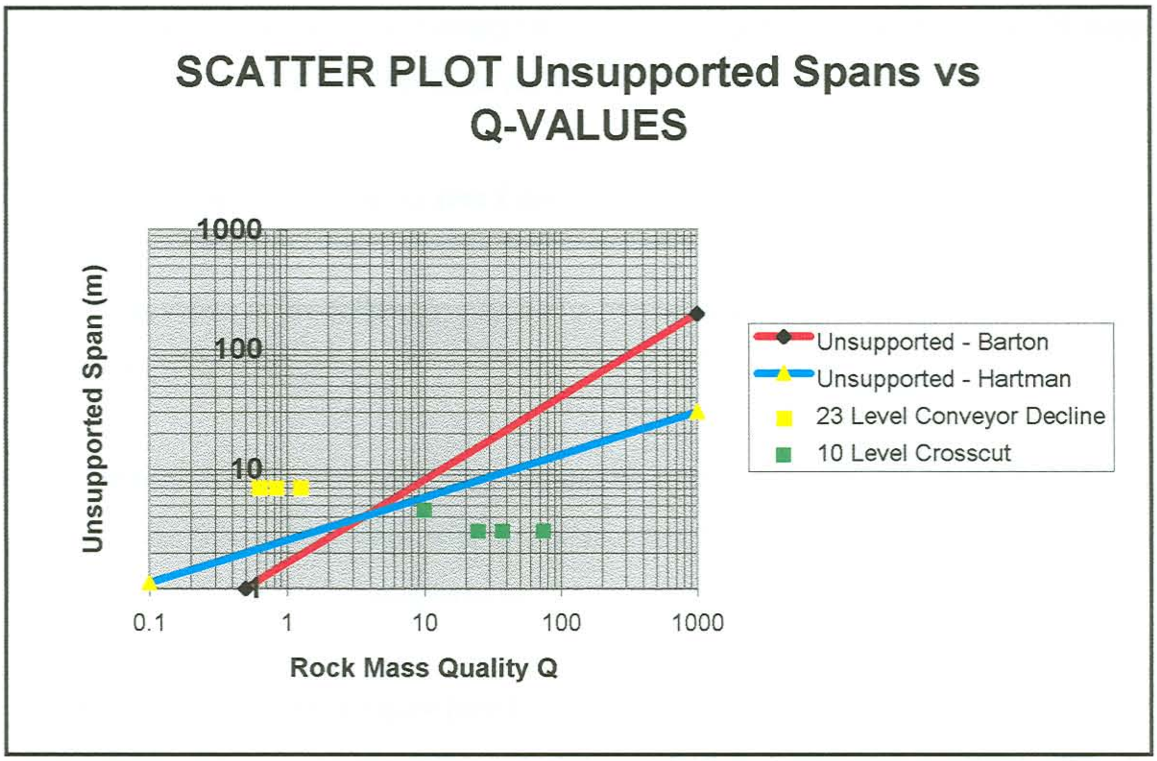


FIG. 5.21 - Scatter plot for supported data points on an unsupported span vs Rock Mass Quality : Q-Value Excavation Support Ratio of 1,6 (Permanent Mine Opening) 10 level crosscut and 23 Level Conveyor Decline

The data provided interesting information :

a) The scatter plot for the unsupported span against the Q-value to a log scale showed that all the data from the 23 level conveyor decline grouped above the Barton unsupported line and the data from 10 level crosscut group below both the Barton and Hartman unsupported line. Thus showing that the 10 level crosscut excavation support, according to Barton and Hartman, was unnecessary. However the 23 level conveyor decline data points should have been supported to stabilize the excavation, as was the case in practice. The support installed is 3m long, 16mm diameter, shepherd crooks on a 1m spacing on strike and 0,75m on dip. 50mm Fibre re-enforced shotcrete will also be added to complete the support installation.

The data set of 10 in the 23 level conveyor decline (see Table B.9) describing the poor Q-value category, according to Barton (1976), must be supported to the listed No. 21

support category and the remaining two must be supported according to No. 26 support category (see Table A.2 and Table A.3).

i) No. 21 Support Category (see Figure 5.5, p71)

The provision is the following : RQD/Jn = 12.5 and Jr/Ja = 0.75 :- Thus the type of support to be used is :

Systematic bolting - un-tensioned, grouted to a 1m spacing.
Shotcrete 25mm to 50mm with Supplementary Note I by Barton et al (1977) as a prescription (Still to be completed in 23 level conveyor decline).

ii) No. 26 Support Category (see Figure 5.5, p71)

No provision required for the following, however the following support is required :
Systematic bolting - tensioned, (expanding shell type for competent rock masses, grouted post-tensioned in very poor quality rock masses), 1m spacing. with Supplementary Note VIII, X (See Table A.5).
Shotcrete 50mm to 75mm with Supplementary Note XI (See Table A.5).

OR

Systematic bolting - un-tensioned, grouted, 1m spacing, with Supplementary Note I, IX (See Table A.5).
Shotcrete 25mm to 50mm.

b) The support bolt length according to Barton, must be 2.3m. The critical bond length for a 16mm diameter shepherd crook using cement capsule grouting with 1hour curing is 23cm. This suggests that the 3m long shepherd crook is long enough but does not have the load bearing capabilities to withstand the theoretical load it will be subjected to.

c) The spacing requirement for the bolts has to change to a 1m by 75cm spacing or alternatively change the steel parameters to a 550 MPa steel to increase the load bearing capabilities of the support tendon.

The above investigation into the Barton rockmass classification has provided the author with the necessary confidence to use the scheme. It also provided me with insight into the rock character, structure and excavation size relationship, as well as the ability of the rockmass, surrounding the excavation, to withstand the force of gravity. The above is enforced by the interpretation of the findings. Meaning that the proposed line below a rock mass quality Q-value of one will have to be critically evaluated and measured against the obtained values.

CHAPTER VI

CONCLUSIONS

In a mining environment such as Impala Platinum where stress changes on tunnel elevations are small, rockfalls from the hangingwall will in most cases be controlled by the strength of the rockmass, joints and the dimensions of tunnels. In certain instances the presence of unfavourably orientated structures could result in large block fall outs. The Barton Rock Tunneling Quality Index does not include a parameter for describing such a phenomenon, which is insignificant in relation to the other categories i.e. rock quality designation, joint number, joint roughness, joint alteration, joint water and stress reduction factor.

The fall of ground analysis in Chapter 3 was done to ensure that the fall of ground problem on Impala was structurally controlled by jointing and supported by various joint characteristics and rock strength variations. However the choice to validate the Q-system for Impala was substantiated by the fact that the main parameters of the system are involved with block size (RQD/Jn), inter-block shear strength (Jr/Ja) and active stress (Jw/SRF). The Q-system in its entirety was scrutinized and modifications were made to suit the Impala ground conditions.

A conclusion concerning rock mass rating systems in general are that they are still qualitative, site specific, and therefore must be modified to suit the particular situation. Generalisation of a single RMR system to all conditions may not always be possible, if the chosen system is sometimes inapplicable, then the engineer should turn to basic engineering design principles (SIMRAC, 1998).

The empirical approach in validating the Q-system for Impala lead to the following changes in the formula (5.4),

$$Span = 1{,}56 * ESR * Q^{0{,}3442}$$

where ESR is equivalent to 1,6 for a permanent mine opening. This was however necessary for a conservative approach on the mine where man-made and natural unsupported excavations are described in Figure 5.4 and Figure 5.5.

Barton (1976) created another equation, following numerous case studies in unsupported man-made excavations and natural openings in limestone at Carlsbad, New Mexico. This is shown below as the altered version

$$Span = 2 * Q^{0,3441921}$$

The conservative approach is mainly due to the limited amount of information presented in this thesis and that these excavations are mainly supported by shepherd crooks and will not be secondary supported by a cast concrete lining. Thus concluding the use of the Q-value Rock Quality Tunnel Index at Impala Platinum as a typical rock classification indicator with some modifications that must be taken into consideration when conducting a typical rockmass classification on Impala.

The geological structure orientation to excavation orientation however still dictates the scale of excavation instability. It was further noted that using the rockmass classification system in itself is not a goal but rather a foundation from which to determine if and what support systems are required. It is thus essential that the rock mass classification process followed a systematic approach to convert observations into workable results.

It was felt at a certain stage that the fall of ground analysis with the length (3,5m), width (2,5m), weight (13 000 kg), volume (9m^3), areal (9m^2) and height (0,9m) for which a 95 cumulative percentage limit has been determined, should be combined with the Q-system whereby the bolt lengths, spacing and ultimate strength are altered or substituted.

This would have altered the support design rock height, spacing and strength of the bolt or tendon to such an extent that it would assist in reducing overall cost to the mine. However it was decided not to combine the two fields of information, due to the limited amount of fall of ground information and the lack of information regarding typical excavation widths with fall out heights.

The joint roughness category graph produced in Figure 5.12 and the average Q-value plotted against joint roughness in Figure 5.13 provided some concern about the weighting of the joint roughness category. The system may not adequately warn the rock engineer of an impending rockfall hazard.

Therefore once support is opted for, irrespective of the rockmass strength, the question of support resistance and length of anchor become highly contentious bearing in mind the unknowns and the lack of rigid guidelines.

BIBLIOGRAPHY

BARTON, N., Lien, R. & Lunde, J. 1974. Engineering classification of rock masses for the design of tunnel support. Rock Mechanics. Vol. 6, No. 4, 189-236. New York : Pergamon.

BARTON, N., Loset, F., Lien, R. & Lunde, J. 1980. Application of the Q-system in design decisions. In subsurface space, (ed. M. Bergman) 2, 553-561. New York: Pergamon.

BIENIAWSKI, Z.T. 1978. Rock mass classification in rock engineering. In Exploration for rock engineering, proc. of the symp., (ed. Z.T. Bieniawski) 1, 97-106. Cape Town. Balkema.

BRADY, B.H.G. & BROWN, E.T. 1985. Rock Mechanics for Underground Mining. London. George Allen & Unwin.

CHAMBER OF MINES OF SOUTH AFRICA RESEARCH ORGANIZATION (COMRO). 1988. An industry Guide to methods of ameliorating the hazards of rockfalls and rockbursts. Johannesburg, S.A.

CUMMINGS, R.A., Kendorski, R.A., Bieniawski, Z.T. 1982. Caving rock mass classification and support estimation. U.S. Bureau of Mines Contract Report #J0100103. Chicago: Engineers International Inc.

DEERE, D.U. 1968. Geological considerations in Rock Mechanics in engineering practice. K.G Stagg and O.C. Zienkiewiez (eds), p1-20. London. Wiley.

DE LA ROSA, B., Doman, D., Gildenhuys, D.G., Grabe, P.J., Jooste, T. de W., Pretorius, J.G., Pretorius, L.M. and Schoeman, M.J. 1984. Introduction to Algebra. McGraw-Hill.

DE MAAR, W.J. & HOLDER, G. 1993. An overview of Rock Engineering Problems Rustenburg Platinum Mines Limited (Rustenburg Section). Proceedings of the SANGORM Symposium : Rock Engineering Problems Related to Hard Rock Mining at Shallow Intermediate Depth. Rustenburg, S.A, 1993.

DIDRIKSON, V. & ISANDER, A. 1995. Collapse of the Bolmen Water Supply Tunnel. First South African Rock Engineering Symposium. September 1997.

ESTERHUIZEN, G.S. & STREUDERS, S.B. 1997. Rockfall Hazard Evaluation using Probabilistic Keyblock Analysis. First South African Rock Engineering Symposium. September 1997.

GAVISH, E. 1977. Evaluation of empirical rock mass classification methods (Q, RMR) for tunneling in bedded rock : Lessons from the Giloh Tunnel, Jerusalem. MSc Thesis. Department of Geological and Environmental Sciences, Bengurion University of the Negev, Beer-Sheva. 180p.

GRIMSTAD, E. and BARTON, N. 1993. Updating the Q-system for NMT. Proc. int. symp. on sprayed concrete - modern use of wet mix sprayed concrete for underground support, Fagernes, (eds Kompen, Opsahl and Berg). Oslo: Norwegian Concrete Assn.

HACK, R. 1997. Rockmass strength by rock mass classification. International Institute for Aerospace Survey and Earth Science (ITC), The Netherlands. 1st South African Rock Engineering Symposium. September 1997.

HOEK, E. 1998. Course notes by Evert Hoek. Canada. Evert Hoek Consulting Engineer Inc. Canada.

HOEK, E. and BROWN, E.T. 1980. Underground Excavations in Rock. The Institution of Mining and Metallurgy. London. England.

IMPALA PLATINUM MINE LTD. 1999. Rock Engineering Code of Practice in the prevention of rockfalls and rockbursts, March, 1999. Impala Rock Engineering Department.

JAGER, A.J.& RYDER, J.A. 1999. A Handbook on Rock Engineering Practice for tabular hard rock mines. SIMRAC.

KOTZE, T.J. 1979. A report on the measurement of joints and other structural features on Impala. Group Rock Mechanics Department. Report No. G.R.M. 50/79.

KOTZE, T.J. 1987. An interim report on the Tunnel support Design aspects at Impala Platinum. Genmin Rock Engineering Department.

KOTZE, T.J. 1997. Using Backfill "Soup" parameters to simulate yielding pillars on Impala Mine. Internal Rock Engineering Report. Impala Rock Engineering Department.

LAUBSCHER, D.H. 1977. Geomechanics classification of jointed rock masses - mining applications. Trans Instn. Min. Metall. 86, A1-8.

LAUBSCHER, D.M. & PAGE, C.H.1990. The design of rock Support in high stress or weak rock environments. Proc. 92nd Can. Inst. Min.Metall. AGM, Ottawa, Paper #91.

LAUBSCHER, D.H. & TAYLOR, H.W. 1976. The importance of geomechanics classification of jointed rock masses in mining operations. In Exploration for rock engineering, (ed. Z.T. Bieniawski)1, 119-128. Cape Town. Balkema.

LOSET, F. 1992. Support needs compared at the Svartisen Road Tunnel. Tunnels and Tunneling, June.

LOUGHER, D.R. and MELLOWSHIP, P. 1993. Strata control problems associated with geological structures on Impala Platinum Mines. Proceedings of the SANGORM Symposium on Rock Engineering Problems Related to Hard Rock Mining at Shallow to Intermediate Depth, Rustenburg 1993.

MAURENBRECHER, P.M. 1997. Road cut of 1972 at waterval Boven revisited : a review of stability by stereographic methods. Faculty of Applied Geoscience, University of Technology, Delft, the Netherlands. First South African Rock Engineering Symposium. September 1997.

MELLOWSHIP, P. 1996. Impala Platinum Limited Geology - Training Module. Geology Department. Impala Platinum Limited.

CSIR (1997) : MINSIM-W - Boundary element code for calculating elastic stress and displacements. CSIR: Division of Miningtek, Johannesburg:RSA.

NAJM, K. & SADAGHIANI, M.H. 1996. Optimization of cavern excavation procedure based on the instrumentation data in the Karun 3 hydroelectric power project, Iran. First South African Rock Engineering Symposium. September 1997.

PALMSTROM, A. 1982. The volumetric joint count - a useful and simple measure of the degree of rock jointing. Proc. 4th congr. Int. Assn Engng Geol., Delhi 5, 221-228.

PITEAU, D.R. 1971. Geological Factors Significant to the Stability of Slopes Cut in Rock. De Beers Consolidated Mines. Slope Case Study.

REDELINGHUIS, A, Julyan, F.W., Steyn, B.L. & Benade, F.J.C. 1989. Quantitative Methods for Management Decision Making. Durban. Butterworth.

SIMRAC (1994). Identification of probable keyblock shapes and sizes for various known joint set and bedding combinations for drives and cross-cuts in Bushveld Complex Mines. Interim Project Report. Project. No. GAP 026, Department of Minerals and Energy, Johannesburg.

SIMRAC (1998). Hazard identification for Rock Engineering. Final Project Report. Project No. GAP 339, Department of Minerals and Energy, Johannesburg.

STACEY, T.R. 1984. Support of large excavations. SAIMM School, Rock Mechanics in Mine Management.

STILLBORG, B. 1994. Professional users Handbook for Rockbolting. Second Edition. Germany. Trans Tech Publications.

TERZAGHI, K. 1946. Rock defects and loads on tunnel supports. In Rock tunneling with steel supports, (Eds R. V. Procter and T.L. White 1, 17-99. Youngstown, OH: Commercial Shearing and Stamping Company).

TERBRUGGE, P.J. and LANGENHOVEN, E.M. 1991. Applications of Rock Engineering to Pit Slope Optimisation at De Hoek. International Society for Rock Mechanics. Symposium – Impact of Rock Engineering on Mining and Tunneling Economics, October 1991.

WICKHAM, G.E., Tiedeman, H.R. and Skinner, E.H. 1972. Support determination based on geological predictions. In Proc. North American rapid excav. tunneling conf., Chicago, (Eds. K.S. Lane and L.A. Garfield), 43-64. New York: Soc. Min. Engrs, Am. Inst. Min. Metall. Petrom Engrs.

WILKES, T.A. 1991. The Role of Rock Mass Classifications in aiding Effective Support Design in Shallow Workings at Koffiefontein Mine, South Western Free State. International Society For Rock Mechanics. Symposium – Impact of Rock Engineering on Mining and Tunneling Economics, October 1991.

APPENDIX A

TABLE A.1 - Suggested support measures for the 38 categories (After Barton et al, 1977). Support measures for Rock Masses of "Exceptional', Extremely Good', 'Very Good' and Good Quality (Q range : 1000–10)

Support category	RQD/J_n	J_r/J_a	Span/ESR	Type of support	Note
1*	–	–	–	sb (utg)	–
2*	–	–	–	sb (utg)	–
3*	–	–	–	sb (utg)	–
4*	–	–	–	sb (utg)	–
5*	–	–	–	sb (utg)	–
6*	–	–	–	sb (utg)	–
7*	–	–	–	sb (utg)	–
8*	–	–	–	sb (utg)	–
9	≥ 20	–	–	sb (utg)	–
	< 20	–	–	B (utg) 2.5 – 3 m	–
10	≥ 30	–	–	B (utg) 2 – 3 m	–
	< 30	–	–	B (utg) 1.5 – 2 m + clm	–
11*	≥ 30	–	–	B (tg) 2 – 3 m	–
	< 30	–	–	B (tg) 1.5 – 2 m + clm	–
12*	≥ 30	–	–	B (tg) 2 – 3 m	–
	< 30	–	–	B (tg) 1.5 – 2 m + clm	–

Key to Support Tables:

sb = spot bolting
B = systematic bolting
(utg) = untensioned, grouted
(tg) = tensioned, (expanding shell type for competent rock masses, grouted post-tensioned in very poor quality rock masses; see Note XI)
S = shotcrete
(mr) = mesh reinforced
clm = chain link mesh
CCA = cast concrete arch
(sr) = steel reinforced

Bolt spacings are given in metres (m). Shotcrete, or cast concrete arch thickness is given in centimetres (cm).

Support category	RQD/J_n	J_r/J_a	Span/ESR	Type of support	Note
13	≥ 10	≥ 1.5	–	sb (utg)	I
	≥ 10	< 1.5	–	B (utg) 1.5 – 2 m	I
	< 10	≥ 1.5	–	B (utg) 1.5 – 2 m	I
	< 10	< 1.5	–	B (utg) 1.5 – 2 m + S 2 – 3 cm	I
14	≥ 10	–	≥ 15 m	B (tg) 1.5 – 2 m + clm	I, II
	< 10	–	≥ 15 m	B (tg) 1.5 – 2 m + S (mr) 5 – 10 cm	I, II
	–	–	< 15 m	B (utg) 1.5 – 2 m + clm	I, III
15	> 10	–	–	B (tg) 1.5 – 2 m + clm	I, II, IV
	≤ 10	–	–	B (tg) 1.5 – 2 m + S (mr) 5 – 10 cm	I, II, IV
16* See note XII	> 15	–	–	B (tg) 1.5 – 2 m + clm	I, V, VI
	≤ 15	–	–	B (tg) 1.5 – 2 m + S (mr) 10 – 15 cm	I, V, VI

* Authors' estimates of support. Insufficient case records available for reliable estimation of support requirements.

Note: The type of support to be used in categories 1 to 8 will depend on the blasting technique. Smooth wall blasting and thorough barring-down may remove the need for support. Rough wall blasting may result in the need for single applications of shotcrete, especially where the excavation height is > 25 m. Future case records should differentiate categories 1 to 8.

TABLE A.2 - Suggested support measures for the 38 categories (After Barton et al, 1977). Support measures for Rock Masses of 'Fair' and 'Poor' Quality (Q range : 10 – 1)

Support category	Conditional factors RQD/J_n	J_r/J_a	Span/ESR	Type of support	Note
17	> 30	–	–	sb (utg)	I
	(≥ 10, ≤ 30)	–	–	B (utg) 1 – 1.5 m	I
	< 10	–	≥ 6 m	B (utg) 1 – 1.5 m + S 2 – 3 cm	I
	< 10	–	< 6 m	S 2 – 3 cm	I
18	> 5	–	≥ 10 m	B (tg) 1 – 1.5 m + clm	I, III
	> 5	–	< 10 m	B (utg) 1 – 1.5 m + clm	I
	≤ 5	–	≥ 10 m	B (tg) 1 – 1.5 m + S 2 – 3 cm	I, III
	≤ 5	–	< 10 m	B (utg) 1 – 1.5 m + S 2 – 3 cm	I
19	–	–	≥ 20 m	B (tg) 1 – 2 m + S (mr) 10 – 15 cm	I, II, IV
	–	–	< 20 m	B (tg) 1 – 1.5 m + S (mr) 5 – 10 cm	I, II

Key to Support Tables:

sb = spot bolting
B = systematic bolting
(utg) = untensioned, grouted
(tg) = tensioned, (expanding shell type for competent rock masses, grouted post-tensioned in very poor quality rock masses; see Note XI)
S = shotcrete
(mr) = mesh reinforced
clm = chain link mesh
CCA = cast concrete arch
(sr) = steel reinforced

Bolt spacings are given in metres (m). Shotcrete, or cast concrete arch thickness is given in centimetres (cm).

Support category	Conditional factors RQD/J_n	J_r/J_a	Span/ESR	Type of support	Note
20* See note XII	–	–	≥ 35 m	B (tg) 1 – 2 m + S (mr) 20 – 25 cm	I, V, VI
	–	–	< 35 m	B (tg) 1 – 2 m + S (mr) 10 – 20 cm	I, II, IV
21	≥ 12.5	≤ 0.75	–	B (utg) 1 m + S 2 – 3 cm	I
	< 12.5	≤ 0.75	–	S 2.5 – 5 cm	I
	–	> 0.75	–	B (utg) 1 m	I
22	(> 10, < 30)	> 1.0	–	B (utg) 1 m + clm	I
	≤ 10	> 1.0	–	S 2.5 – 7.5 cm	I
	< 30	≤ 1.0	–	B (utg) 1 m + S (mr) 2.5 – 5 cm	I
	≥ 30	–	–	B (utg) 1 m	I
23	–	–	≥ 15 m	B (tg) 1 – 1.5 m + S (mr) 10 – 15 cm	I, II, IV, VII
	–	–	< 15 m	B (utg) 1 – 1.5 m + S (mr) 5 – 10 cm	I
24* See note XII	–	–	≥ 30 m	B (tg) 1 – 1.5 m + S (mr) 15 – 30 cm	I, V, VI
	–	–	< 30 m	B (tg) 1 – 1.5 m + S (mr) 10 – 15 cm	I, II, IV

* Authors' estimates of support. Insufficient case records available for reliable estimation of support requirements.

TABLE A.3 - Suggested support measures for the 38 categories (After Barton et al, 1977). Support measures for Rock Masses Of 'Very Poor' Quality (Q range : 1 – 0.1)

Support category	Conditional factors RQD/J_n	J_r/J_a	Span/ESR	Type of support	Note
25	> 10	> 0.5	–	B (utg) 1 m + mr or clm	I
	> 10	> 0.5	–	B (utg) 1 m + S (mr) 5 cm	I
	–	≤ 0.5	–	B (tg) 1 m + S (mr) 5 cm	I
26	–	–	–	B (tg) 1 m + S (mr) 5 – 7.5 cm	VIII, X, XI
	–	–	–	B (utg) 1 m + S 2.5 – 5 cm	I, IX
27	–	–	≥ 12 m	B (tg) 1 m + S (mr) 7.5 – 10 cm	I, IX
	–	–	< 12 m	B (utg) 1 m + S (mr) 5 – 7.5 cm	I, IX
	–	–	> 12 m	CCA 20 – 40 cm + B (tg) 1 m	VIII, X, XI
	–	–	< 12 m	S (mr) 10 – 20 cm + B (tg) 1 m	VIII, X, XI

Key to Support Tables:

sb = spot bolting
B = systematic bolting
(utg) = untensioned, grouted
(tg) = tensioned, (expanding shell type for competent rock masses, grouted post-tensioned in very poor quality rock masses; see Note XI)
S = shotcrete
(mr) = mesh reinforced
clm = chain link mesh
CCA = cast concrete arch
(sr) = steel reinforced

Bolt spacings are given in metres (m). Shotcrete, or cast concrete arch thickness is given in centimetres (cm).

Support category	Conditional factors RQD/J_n	J_r/J_a	Span/ESR	Type of support	Note
28*	–	–	≥ 30 m	B (tg) 1 m + S (mr) 30 – 40 cm	I, IV, V, IX
See note XII	–	–	(≥ 20 m, < 30 m)	B (tg) 1 m + S (mr) 20 – 30 cm	I, II, IV, IX
	–	–	≤ 20 m	B (tg) 1 m + S (mr) 15 – 20 cm	I, II, IX
	–	–	–	CCA (sr) 30 – 100 cm + B (tg) 1 m	IV, VIII, X, XI
29*	> 5	> 0.25	–	B (utg) 1 m + S 2 – 3 cm	–
	≤ 5	> 0.25	–	B (utg) 1 m + S (mr) 5 cm	–
	–	≤ 0.25	–	B (tg) 1 m + S (mr) 5 cm	–
30	≥ 5	–	–	B (tg) 1 m + S 2.5 – 5 cm	IX
	< 5	–	–	S (mr) 5 – 7.5 cm	IX
	–	–	–	B (tg) 1 m + S (mr) 5 – 7.5 cm	VIII, X, XI
31	> 4	–	–	B (tg) 1 m + S (mr) 5 – 12.5 cm	IX
	≤ 4, ≥ 1.5	–	–	S (mr) 7.5 – 25 cm	IX
	< 1.5	–	–	CCA 20 – 40 cm + B (tg) 1 m	IX
	–	–	–	CCA (sr) 30 – 50 cm + B (tg) 1 m	VIII, X, XI
32	–	–	≥ 20 m	B (tg) 1 m + S (mr) 40 – 60 cm	II, IV, IX
See note XII	–	–	< 20 m	B (tg) 1 m + S (mr) 20 – 40 cm	III, IV, IX
	–	–	–	CCA (sr) 40 – 120 cm + B (tg) 1 m	IV, VIII, X, XI

TABLE A.4 - Suggested support measures for the 38 categories (After Barton et al, 1977). Support measures for Rock Masses of 'Extremely Poor' and 'Exceptionally Poor' Quality (Q range : 0.1 – 0.001)

Support category	Conditional factors RQD/J_n	J_r/J_a	Span/ESR	Type of support	Note
33*	≥ 2	–	–	B (tg) 1 m	IX
				+ S (mr) 2.5 – 5 cm	
	< 2	–	–	S (mr) 5 – 10 cm	IX
	–	–	–	S (mr) 7.5 – 15 cm	VIII, X
34	≥ 2	≥ 0.25	–	B (tg) 1 m	IX
				+ S (mr) 5 – 7.5 cm	
	< 2	≥ 0.25	–	S (mr) 7.5 – 15 cm	IX
	–	< 0.25	–	S (mr) 15 – 25 cm	IX
	–	–	–	CCA (sr) 20 – 60 cm	VIII, X,
				+ B (tg) 1 m	XI
35	–	–	≥ 15 m	B (tg) 1 m	II, IX
				+ S (mr) 30 – 100 cm	
	–	–	≥ 15 m	CCA (sr) 60 – 200 cm	VIII, X,
See note				+ B (tg) 1 m	XI, II
XII	–	–	< 15 m	B (tg) 1 m	IX, III
				+ S (mr) 20 – 75 cm	
	–	–	< 15 m	CCA (sr) 40 – 150 cm	VIII, X,
				+ B (tg) 1 m	XI, III
36*	–	–	–	S (mr) 10 – 20 cm	IX
	–	–	–	S (mr) 10 – 20 cm	VIII, X,
				+ B (tg) 0.5 – 1.0	XI
37	–	–	–	S (mr) 20 – 60 cm	IX
	–	–	–	S (mr) 20 – 60 cm	VIII, X,
				+ B (tg) 0.5 – 1.0	XI
38	–	–	≥ 10 m	CCA (sr) 100 – 300 cm	IX
	–	–	≥ 10 m	CCA (sr) 100 – 300 cm	VIII, X,
See note				+ B (tg) 1 m	II, XI
XIII	–	–	< 10 m	S (mr) 70 – 200 cm	IX
	–	–	< 10 m	S (mr) 70 – 200 cm	VIII, X,
				+ B (tg) 1 m	III, XI

Supplementary notes by BARTON, LIEN *and* LUNDE

I. For cases of heavy bursting or 'popping', tensioned bolts with enlarged bearing plates often used, with spacing of about 1 m (occasionally down to 0.8 m). Final support when 'popping' activity ceases.

II. Several bolt lengths often used in same excavation, i.e. 3, 5 and 7 m.

III. Several bolt lengths often used in same excavation, i.e. 2, 3 and 4 m.

IV. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 2 – 4 m.

V. Several bolt lengths often used in same excavation, i.e. 6, 8 and 10 m.

VI Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 4 – 6 m.

VII. Several of the older generation power stations in this category employ systematic or spot bolting with areas of chain link mesh, and a free span concrete arch roof (25—40 cm) as permanent support.

VIII. Cases involving swelling, for instance montmorillonite clay (with access of water). Room for expansion behind the support is used in cases of heavy swelling. Drainage measures are used where possible.

IX. Cases not involving swelling clay or squeezing rock.

X. Cases involving squeezing rock. Heavy rigid support is generally used as permanent support.

XI. According to the authors' experience, in cases of swelling or squeezing, the temporary support required before concrete (or shotcrete) arches are formed may consist of bolting (tensioned shell-expansion type) if the value of RQD/J_n is sufficiently high (i.e. > 1.5), possibly combined with shotcrete. If the rock mass is very heavily jointed or crushed (i.e. RQD/J_n < 1.5, for example a 'sugar cube' shear zone in quartaite), then the temporary support may consist of up to several applications of shotcrete. Systematic bolting (tensioned) may be added after casting the concrete, but it may not be effective when RQD/J_n < 1.5 or when a lot of clay is present, unless the bolts are grouted before tensioning. A sufficient length of anchored bolt might also be obtained using quick setting resin anchors in these extremely poor quality rock masses. Serious occurrences of swelling and/or squeezing rock may require that the concrete arches are taken right up to the face, possibly using a shield as temporary shuttering. Temporary support of the working face may also be required in these cases.

XII. For reasons of safety the multiple drift method will often be needed during excavation and supporting of roof arch. Categories 16, 20, 24, 28, 32, 35 (SPAN/ESR > 15 m only).

XIII. Multiple drift method usually needed during excavation and support of arch, walls and floor in cases of heavy squeezing. Category 38 (SPAN/ESR > 10 m only).

TABLE A.5 - Supplementary notes by Barton, Lien and Lunde (After Barton et al, 1977)

Supplementary notes by BARTON, LIEN *and* LUNDE

I. For cases of heavy bursting or 'popping', tensioned bolts with enlarged bearing plates often used, with spacing of about 1 m (occasionally down to 0.8 m). Final support when 'popping' activity ceases.

II. Several bolt lengths often used in same excavation, i.e. 3, 5 and 7 m.

III. Several bolt lengths often used in same excavation, i.e. 2, 3 and 4 m.

IV. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 2 – 4 m.

V. Several bolt lengths often used in same excavation, i.e. 6, 8 and 10 m.

VI. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 4 – 6 m.

VII. Several of the older generation power stations in this category employ systematic or spot bolting with areas of chain link mesh, and a free span concrete arch roof (25—40 cm) as permanent support.

VIII. Cases involving swelling, for instance montmorillonite clay (with access of water). Room for expansion behind the support is used in cases of heavy swelling. Drainage measures are used where possible.

IX. Cases not involving swelling clay or squeezing rock.

X. Cases involving squeezing rock. Heavy rigid support is generally used as permanent support.

XI. According to the authors' experience, in cases of swelling or squeezing, the temporary support required before concrete (or shotcrete) arches are formed may consist of bolting (tensioned shell-expansion type) if the value of RQD/J_n is sufficiently high (i.e. > 1.5), possibly combined with shotcrete. If the rock mass is very heavily jointed or crushed (i.e. RQD/J_n < 1.5, for example a 'sugar cube' shear zone in quartzite), then the temporary support may consist of up to several applications of shotcrete. Systematic bolting (tensioned) may be added after casting the concrete, but it may not be effective when RQD/J_n < 1.5 or when a lot of clay is present, unless the bolts are grouted before tensioning. A sufficient length of anchored bolt might also be obtained using quick setting resin anchors in these extremely poor quality rock masses. Serious occurrences of swelling and/or squeezing rock may require that the concrete arches are taken right up to the face, possibly using a shield as temporary shuttering. Temporary support of the working face may also be required in these cases.

XII. For reasons of safety the multiple drift method will often be needed during excavation and supporting of roof arch. Categories 16, 20, 24, 28, 32, 35 (SPAN/ESR > 15 m only).

XIII. Multiple drift method usually needed during excavation and support of arch, walls and floor in cases of heavy squeezing. Category 38 (SPAN/ESR > 10 m only).

Supplementary notes by HOEK *and* BROWN *(1980)*

A. Chainlink mesh is sometimes used to catch small pieces of rock which can become loose with time. It should be attached to the rock at intervals of between 1 and 1.5 m and short grouted pins can be used between bolts. Galvanized chainlink mesh should be used where it is intended to be permanent, e.g. in an underground powerhouse.

B. Weldmesh, consisting of steel wires set on a square pattern and welded at each intersection, should be used for the reinforcement of shotcrete since it allows easy access of the shotcrete to the rock. Chainlink mesh should never be used for this purpose since the shotcrete cannot penetrate all the spaces between the wires and air pockets are formed with consequent rusting of the wire. When choosing weldmesh, it is important that the mesh can be handled by one or two men working from the top of a high-lift vehicle and hence the mesh should not be too heavy. Typically, 4.2 mm wires set at 100 mm intervals (designated 100 x 100 x 4.2 weldmesh) are used for reinforcing shotcrete.

C. In poorer quality rock, the use of untensioned grouted dowels as recommended by BARTON, LIEN and LUNDE depends upon immediate installation of these reinforcing elements behind the face. This depends upon integrating the support drilling and installation into the drill-blastmuck cycle and many non-Scandinavian contractors are not prepared to consider this system. When it is impossible to ensure that untensioned grouted dowels are going to be installed immediately behind the face, consideration should be given to using tensioned rockbolts which can be grouted at a later stage. This ensures that support is available during the critical excavation stage.

D. Many contractors would consider that a 200 mm thick cast concrete arch is too difficult to construct because there is not enough room between the shutter and the surrounding rock to permit easy ac-

TABLE A.5 - (cont.) Supplementary notes by Barton, Lien and Lunde (After Barton et al, 1977)

cess for pouring concrete and placing vibrators. The US Army Corps of Engineers (1978) suggests 10 inches (254 mm) as a normal minimum while some contractors prefer 300 mm.

E. BARTON, LIEN and LUNDE suggest shotcrete thicknesses of up to 2 m. This would require many separate applications and many contractors would regard shotcrete thicknesses of this magnitude as both impractical and uneconomic, preferring to cast concrete arches instead. A strong argument in favour of shotcrete is that it can be placed very close to the face and hence can be used to provide early support in poor quality rock masses. Many contractors would argue that a 50 to 100 mm layer is generally sufficient for this purpose, particularly when used in conjunction with tensioned rockbolts as indicated by BARTON, LIEN and LUNDE, and that the placing of a cast concrete lining at a later stage would be a more effective way to tackle the problem. Obviously, the final choice will depend upon the unit rates for concreting and shotcreting offered by the contractor and, if shotcrete is cheaper, upon a practical demonstration by the contractor that he can actually place shotcrete to this thickness. In North America, the use of concrete or shotcrete linings of up to 2 m thick would be considered unusual and a combination of heavy steel sets and concrete would normally be used to achieve the high support pressures required in very poor ground.

Supplementary note by STILLBORG

Untensioned, grouted rockbolts are recommended in several support categories. At the time when BARTON et al. proposed their guide for support measures the friction anchored rockbolts were not yet available. The note under Table 6 in connection with BIENIAWSKI's guide for excavation and support in rock tunnels is therefore equally applicable here.

Supplementary notes by HOEK *and* BROWN *(1980)*

A. Chainlink mesh is sometimes used to catch small pieces of rock which can become loose with time. It should be attached to the rock at intervals of between 1 and 1.5 m and short grouted pins can be used between bolts. Galvanized chainlink mesh should be used where it is intended to be permanent, e.g. in an underground powerhouse.

B. Weldmesh, consisting of steel wires set on a square pattern and welded at each intersection, should be used for the reinforcement of shotcrete since it allows easy access of the shotcrete to the rock. Chainlink mesh should never be used for this purpose since the shotcrete cannot penetrate all the spaces between the wires and air pockets are formed with consequent rusting of the wire. When choosing weldmesh, it is important that the mesh can be handled by one or two men working from the top of a high-lift vehicle and hence the mesh should not be too heavy. Typically, 4.2 mm wires set at 100 mm intervals (designated 100 x 100 x 4.2 weldmesh) are used for reinforcing shotcrete.

C. In poorer quality rock, the use of untensioned grouted dowels as recommended by BARTON, LIEN and LUNDE depends upon immediate installation of these reinforcing elements behind the face. This depends upon integrating the support drilling and installation into the drill-blastmuck cycle and many non-Scandinavian contractors are not prepared to consider this system. When it is impossible to ensure that untensioned grouted dowels are going to be installed immediately behind the face, consideration should be given to using tensioned rockbolts which can be grouted at a later stage. This ensures that support is available during the critical excavation stage.

D. Many contractors would consider that a 200 mm thick cast concrete arch is too difficult to construct because there is not enough room between the shutter and the surrounding rock to permit easy ac-

APPENDIX B

E B.1 - CASE STUDY 1 - 10 LEVEL CROSSCUT NO. 9-SHAFT - IMPALA PLATINUM - 640 m BELOW SURFACE

Depth bel. surf. (m)	Exc. Height	Exc. Width	Jt Vertical	Dip	Strike	RQD	Act. RQD	Description	Jn	Description	Jr	Description	Ja	Description	Jw	Description	SRF	Description	Q-value	Description	Photos	Supp. installed & pattern/spacing - Photo No.
640	3	3	2	1	4	108.07	100	Excellent	4	Two Joint Sets	3	Rough/irregular undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	37.5	Good	Yes	Plate 19,20
640	3	3	1	0	4	109.5	100	Excellent	3	1 Joint Set Plus Random	3	Rough/irregular undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	50.0	Very Good		No Bolting
640	3	3	2	1	2	110.27	100	Excellent	4	Two Joint Sets	3	Rough/irregular undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	2.5	Sing. Shear Zone, Compt. Rock, >50m	15.0	Good		No Bolting
640	3	3	2	0	5	107.3	100	Excellent	4	Two Joint Sets	3	Rough/irregular undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	2.5	Sing. Shear Zone, Compt. Rock, >50m	15.0	Good		No Bolting
640	3	3	5	3	2	106.31	100	Excellent	9	Three Joint Sets	3	Rough/irregular undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	2.5	Sing. Shear Zone, Compt. Rock, >50m	6.7	Fair		No Bolting
640	3	3	4	5	3	105.65	100	Excellent	9	Three Joint Sets	3	Rough/irregular undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	2.5	Sing. Shear Zone, Compt. Rock, >50m	6.7	Fair	Yes	Plate 14,15,16
640	3	3	2	1	0	112.47	100	Excellent	2	One Joint Set	3	Rough/irregular undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	75.0	Very Good	Yes	No Bolting
640	3	3	1	5	2	110.05	100	Excellent	4	Two Joint Sets	3	Rough/irregular undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	37.5	Good	Yes	No Bolting
640	3	3	2	1	0	112.47	100	Excellent	2	One Joint Set	3	Rough/irregular undulating	3	Silty, small clay fraction	1	Dry Excavation	1	Medium Stress	50.0	Very Good		No Bolting
640	3	3	1	1	4	109.17	100	Excellent	2	One Joint Set	3	Rough/irregular undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	75.0	Very Good		No Bolting
640	3	3	1	4	2	110.38	100	Excellent	4	Two Joint Sets	3	Rough/irregular undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	37.5	Good		Spot Bolting
640	3	3	0	1	1	113.57	100	Excellent	4	Two Joint Sets	3	Rough/irregular undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	37.5	Good		No Bolting
640	3	3	1	2	0	113.24	100	Excellent	2	One Joint Set	3	Rough/irregular undulating	3	Silty, small clay fraction	1	Dry Excavation	1	Medium Stress	50.0	Very Good		No Bolting
640	3	3	0	0	0	115	100	Excellent	0.5	Massive	5	Joint spacing >3m	0.75	Tightly Healed	1	Dry Excavation	1	Medium Stress	1333.3	Extremely Good		No Bolting
640	3	3	0	1	0	114.67	100	Excellent	1	few	3	Rough/irregular undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	150.0	Extremely Good		No Bolting
640	3	3	1	0	6	107.3	100	Excellent	3	One Joint Set plus Random	1.5	Slickensided undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	2.5	Sing. Shear Zone, Compt. Rock, >50m	10.0	Good		No Bolting
640	3	3	2	1	3	109.17	100	Excellent	4	Two Joint Sets	4	Discontinuous Joints	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	50.0	Very Good		No Bolting
640	3	3	0	0	2	112.8	100	Excellent	2	One Joint Set	2	Smooth undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	50.0	Very Good		No Bolting
640	3	5	0	0	4	112.36	100	Excellent	2	One Joint Set	3	Rough/irregular undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	10	O1 / O2 < 0,15	7.5	Fair	Yes	Plate 17,18
640	3	5	0	0	2	113.68	100	Excellent	2	One Joint Set	3	Rough/irregular undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	75.0	Very Good	Yes	No Bolting
640	3	3	0	0	0	115	100	Excellent	1	few	5	Joint spacing >3m	0.75	Tightly Healed	1	Dry Excavation	1	Medium Stress	666.7	Extremely Good		No Bolting
640	3	3	0	1	0	114.67	100	Excellent	1	few	3	Rough/irregular undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	150.0	Extremely Good		No Bolting
640	3	3	2	1	3	109.17	100	Excellent	6	Two Joint Sets Plus Random	3	Rough/irregular undulating	2	Slighty altered jw,non-soft. Min.coat	0.66	Damp	2.5	Sing. Shear Zone, Compt. Rock, >50m	6.6	Fair		No Bolting
640	3	3	0	1	3	111.37	100	Excellent	2	One Joint Set	3	Rough/irregular undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	2.5	Sing. Shear Zone, Compt. Rock, >50m	30.0	Good		No Bolting
640	3	3	0	0	0	115	100	Excellent	0.5	Massive	5	Joint spacing >3m	0.75	Tightly Healed	1	Dry Excavation	1	Medium Stress	1333.3	Extremely Good		No Bolting
640	3	3	0	0	1	113.9	100	Excellent	2	One Joint Set	3	Rough/irregular undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	75.0	Very Good	Yes	Plate 11,12,13
640	3	3	0	1	2	112.47	100	Excellent	2	One Joint set	1	Smooth planar	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	25.0	Good	Yes	50% Spotbolting
640	3	3	0	0	2	112.8	100	Excellent	2	One Joint Set	3	Rough/irregular undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	75.0	Very Good	Yes	50% Spotbolting
640	3	3	0	0	1	113.9	100	Excellent	2	One Joint set	3	Rough/irregular undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	2.5	Sing. Shear Zone, Compt. Rock, >50m	30.0	Good		No Bolting
640	3	3	0	1	2	112.47	100	Excellent	3	One Joint Set plus Random	1	Smooth planar	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	16.7	Good		No Bolting
640	3	3	0	3	0	114.01	100	Excellent	2	One Joint Set	1	Smooth planar	4	Softening mineral coating	1	Dry Excavation	7.5	Multiple Shear Zone in competent, loose suurounding, any depth	1.7	Poor		No Bolting
640	3	3	0	0	0	115	100	Excellent	1	few	5	Joint spacing >3m	0.75	Tightly Healed	1	Dry Excavation	1	Medium Stress	666.7	Extremely Good		No Bolting
640	3	3	1	1	1	112.47	100	Excellent	1	few	1	Smooth planar	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	50.0	Good		No Bolting
640	3	3	0	2	5	108.84	100	Excellent	4	Two Joint Sets	3	Rough/irregular undulating	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	37.5	Good		No Bolting
640	3	3	1	1	2	111.37	100	Excellent	2	One Joint Set	1.5	Rough/irregular planar	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	37.5	Good		No Bolting
640	3	3	0	0	1	113.9	100	Excellent	1	few	1.5	Rough/irregular planar	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	2.5	Sing. Shear Zone, Compt. Rock, >50m	30.0	Good		No Bolting
640	3	3	0	0	2	112.8	100	Excellent	2	One Joint Set	1.5	Rough/irregular planar	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	37.5	Good		No Bolting
640	3	3	0	0	4	110.6	100	Excellent	2	One Joint Set	1.5	Rough/irregular planar	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	2.5	Sing. Shear Zone, Compt. Rock, >50m	15.0	Good		No Bolting
640	3	3	1	1	3	110.27	100	Excellent	3	One Joint Set plus Randum	1.5	Rough/irregular planar	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	25.0	Good		No Bolting
640	3	3	0	0	0	115	100	Excellent	0.5	Massive	5	Joint spacing >3m	0.75	Tightly Healed	1	Dry Excavation	1	Medium Stress	1333.3	Extremely Good		No Bolting
640	3	4,5	0	1	3	#VALUE!	100	Excellent	3	One Joint Set plus Randum	1.5	Rough/irregular planar	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	2.5	Sing. Shear Zone, Compt. Rock, >50m	10.0	Fair/Good	Yes	Shepherd Crooks, 12mm diam, 1,8m long, 1m spacing
640	3	3	0	2	0	114.34	100	Excellent	2	One Joint Set	1.5	Rough/irregular planar	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	37.5	Good	Yes	Plate 9, 10
640	3	3	0	2	5	108.84	100	Excellent	4	Two Joint Sets	1.5	Rough/irregular planar	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	2.5	Sing. Shear Zone, Compt. Rock, >50m	7.5	Fair		No Bolting
640	3	3	0	3	2	111.81	100	Excellent	4	Two Joint Sets	1.5	Rough/irregular planar	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	18.8	Good		No Bolting
640	3	3	0	2	4	109.94	100	Excellent	4	Two Joint Sets	1.5	Rough/irregular planar	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	18.8	Good		No Bolting
640	3	3	0	0	2	112.8	100	Excellent	2	One Joint set	1	Smooth planar	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	25.0	Good		No Bolting
640	3	3	0	0	1	113.9	100	Excellent	1	few	1.5	Rough/irregular planar	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	75.0	Very Good		No Bolting
640	3	3	0	0	0	115	100	Excellent	0.5	Massive	5	Joint spacing >3m	0.75	Tightly Healed	1	Dry Excavation	1	Medium Stress	1333.3	Excessively Good		No Bolting
640	3	3	0	0	0	115	100	Excellent	0.5	Massive	5	Joint spacing >3m	0.75	Tightly Healed	1	Dry Excavation	1	Medium Stress	1333.3	Excessively Good		No Bolting
640	3	3	0	2	5	108.84	100	Excellent	4	Two Joint Sets	1.5	Rough/irregular planar	2	Slighty altered jw,non-soft. Min.coat	1	Dry Excavation	1	Medium Stress	18.8	Good		No Bolting
640	3	3	2	2	0	112.14	100	Excellent	4	Two Joint Sets	1	Smooth planar	1	Unaltered joint walls	1	Dry Excavation	1	Medium Stress	25.0	Good		No Bolting
640	3	3	0	1	1	113.57	100	Excellent	2	One joint Set	1.5	Rough/irregular planar	1	Unaltered joint walls	1	Dry Excavation	2.5	Sing. Shear Zone, Compt. Rock, >50m	30.0	Good		No Bolting
640	3	3	0	4	0	113.68	100	Excellent	2	One joint Set	1.5	Rough/irregular planar	1	Unaltered joint walls	1	Dry Excavation	1	Medium Stress	75.0	Very Good	Yes	Plate 7, 8
640	3	3	0	0	6	108.4	100	Excellent	2	One joint Set	1.5	Rough/irregular planar	1	Unaltered joint walls	1	Dry Excavation	1	Medium Stress	75.0	Very Good		No Bolting
640	3	3	0	0	3	111.7	100	Excellent	2	One joint Set	1.5	Rough/irregular planar	1	Unaltered joint walls	1	Dry Excavation	2.5	Sing. Shear Zone, Compt. Rock, >50m	30.0	Good	Yes	Plate 5
640	3	3	0	0	2	112.8	100	Excellent	2	One joint Set	1.5	Rough/irregular planar	1	Unaltered joint walls	1	Dry Excavation	1	Medium Stress	75.0	Very Good		No Bolting
640	3	3	1	0	0	113.9	100	Excellent	2	One joint Set	1.5	Rough/irregular planar	1	Unaltered joint walls	1	Dry Excavation	1	Medium Stress	75.0	Very Good	Yes	Plate 6, Rockstuds, spaced 1m apart
640	3	3	0	2	3	111.04	100	Excellent	4	Two Joint Sets	1.5	Rough/irregular planar	1	Unaltered joint walls	1	Dry Excavation	1	Medium Stress	37.5	Good		No Bolting
640	3	3	0	1	0	114.67	100	Excellent	1	few	1.5	Rough/irregular planar	1	Unaltered joint walls	1	Dry Excavation	1	Medium Stress	150.0	Extremely Good		No Bolting
640	3	3	0	0	0	115	100	Excellent	0.5	Massive	5	Joint spacing >3m	0.75	Tightly Healed	1	Dry Excavation	1	Medium Stress	1333.3	Excessively Good		No Bolting
640	3	3	0	0	0	115	100	Excellent	0.5	Massive	5	Joint spacing >3m	0.75	Tightly Healed	1	Dry Excavation	1	Medium Stress	1333.3	Excessively Good		No Bolting
640	3	3	2	1	4	108.07	100	Excellent	4	Two Joint Sets	1.5	Rough/irregular planar	1	Unaltered joint walls	1	Dry Excavation	1	Medium Stress	37.5	Good		No Bolting
640	3	3	0	0	2	112.8	100	Excellent	2	One Joint Set	1.5	Rough/irregular planar	1	Unaltered joint walls	1	Dry Excavation	1	Medium Stress	75.0	Very Good		No Bolting
640	3	3	0	2	3	111.04	100	Excellent	6	Two joint sets plus random	1.5	Rough/irregular planar	1	Unaltered joint walls	1	Dry Excavation	2.5	Sing. Shear Zone, Compt. Rock, >50m	10.0	Fair/Good		No Bolting
640	3	3	0	1	2	112.47	100	Excellent	3	One Joint Set plus Random	1.5	Rough/irregular planar	3	Silty, small clay fraction	1	Dry Excavation	1	Medium Stress	16.7	Good		No Bolting
640	3	3	2	2	3	108.84	100	Excellent	9	Three Joint Sets	1.5	Rough/irregular planar	1	Unaltered joint walls	1	Dry Excavation	1	Medium Stress	16.7	Good		No Bolting
640	3	3	1	2	4	108.84	100	Excellent	4	Two Joint Sets	1.5	Rough/irregular planar	1	Unaltered joint walls	1	Dry Excavation	1	Medium Stress	37.5	Good		No Bolting
640	3	3	1	1	2	111.37	100	Excellent	3	One Joint Set plus Random	1.5	Rough/irregular planar	1	Unaltered joint walls	1	Dry Excavation	1	Medium Stress	50.0	Very Good		No Bolting
640	3	3	2	1	3	109.17	100	Excellent	4	Two Joint Sets	1.5	Rough/irregular planar	1	Unaltered joint walls	1	Dry Excavation	1	Medium Stress	37.5	Good		No Bolting
640	3	3	2	2	4	107.74	100	Excellent	9	Three Joint Sets	1.5	Rough/irregular planar	1	Unaltered joint walls	1	Dry Excavation	2.5	Sing. Shear Zone, Compt. Rock, >50m	6.7	Fair		No Bolting
640	3	3	0	2	3	111.04	100	Excellent	4	Two Joint Sets	1.5	Rough/irregular planar	1	Unaltered joint walls	1	Dry Excavation	1	Medium Stress	37.5	Good		No Bolting
640	3	3	2	1	1	111.37	100	Excellent	2	One Joint Set	1.5	Rough/irregular planar	1	Unaltered joint walls	1	Dry Excavation	1	Medium Stress	75.0	Very Good		No Bolting
640	3	3	1	4	3	109.29	100	Excellent	4	Two Joint Sets	1.5	Rough/irregular planar	1	Unaltered joint walls	1	Dry Excavation	1	Medium Stress	37.5	Good	Yes	Plate 2,3,4
640	3	3	1	2	2	111.04	100	Excellent	4	Two Joint Sets	1.5	Rough/irregular planar	3	Silty, small clay fraction	1	Dry Excavation	1	Medium Stress	12.5	Good		No Bolting
640	3	3	1	1	2	111.37	100	Excellent	2	One Joint Set	1.5	Rough/irregular planar	1	[illegible]	[illegible]	[illegible]						

B.2 - CASE STUDY 2 - 23 LEVEL CONVEYOR DECLINE NO. 14-SHAFT - IMPALA PLATINUM - 1058m BELOW SURFACE

Depth bel. surf. (m)	Exc. Height	Exc. Width	Jt Vertical	Dip	Strike	RQD	Act. RQD	Description	Jn	Description	Jr	Description	Ja	Description	Jw	Description	SRF	Description	Q-value	Description	Photos	Supp. installed & pattern/spacing
1058	9.5	7	2	16	15	101.954	100	Excellent	12	Three Joint Sets Plus Random	3	Rough/Irregular undulating	6	Zones or bands of disintergrate or crushed	1	Dry Excavation	5	Loose open joints, sugar cube, heavy jointed	0.8	Very poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
1058	9.5	7	3	16	14	102.078	100	Excellent	12	Three Joint Sets Plus Random	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintergated rock	1	Dry Excavation	5	Loose open joints, sugar cube, heavy jointed	1.3	Poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
1058	9.5	7	4	16	17	100.316	100	Excellent	12	Three Joint Sets Plus Random	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintergated rock	0.5	Damp	5	Loose open joints, sugar cube, heavy jointed	0.6	Very poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
1058	9.5	7	2	11	9	106.432	100	Excellent	12	Three Joint Sets Plus Random	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintergated rock	1	Dry Excavation	5	Loose open joints, sugar cube, heavy jointed	1.3	Poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
1058	9.5	7	3	13	10	104.954	100	Excellent	12	Three Joint Sets Plus Random	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintergated rock	1	Dry Excavation	5	Loose open joints, sugar cube, heavy jointed	1.3	Poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
1058	9.5	7	2	13	7	106.715	100	Excellent	12	Three Joint Sets Plus Random	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintergated rock	1	Dry Excavation	5	Loose open joints, sugar cube, heavy jointed	1.3	Poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
1058	9.5	7	2	15	10	104.641	100	Excellent	12	Three Joint Sets Plus Random	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintergated rock	1	Dry Excavation	5	Loose open joints, sugar cube, heavy jointed	1.3	Poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
1058	9.5	7	2	9	7	108.035	100	Excellent	12	Three Joint Sets Plus Random	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintergated rock	1	Dry Excavation	5	Loose open joints, sugar cube, heavy jointed	1.3	Poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
1058	9.5	7	2	8	8	107.894	100	Excellent	12	Three Joint Sets Plus Random	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintergated rock	1	Dry Excavation	5	Loose open joints, sugar cube, heavy jointed	1.3	Poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
1058	9.5	7	2	10	9	106.762	100	Excellent	12	Three Joint Sets Plus Random	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintergated rock	1	Dry Excavation	5	Loose open joints, sugar cube, heavy jointed	1.3	Poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
1058	9.5	7	2	10	8	107.234	100	Excellent	12	Three Joint Sets Plus Random	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintergated rock	1	Dry Excavation	5	Loose open joints, sugar cube, heavy jointed	1.3	Poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
1058	9.5	7	2	13	8	106.244	100	Excellent	12	Three Joint Sets Plus Random	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintergated rock	1	Dry Excavation	5	Loose open joints, sugar cube, heavy jointed	1.3	Poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart

TABLE B.3 : Q-VALUE CATEGORIE DISTRIBUTION

	10 LEVEL CROSSCUT	23 LEVEL CONVEYOR DECLINE
Q-VALUE CATEGORIES	QUANTITIES	QUANTITIES
Exceptionally Good	10	0
Extremely Good	3	0
Very Good	20	0
Good	35	0
Fair	7	0
Poor	1	10
Very Poor	0	2
Extremely Poor	0	0
Exceptionally poor	0	0

TABLE B.4 : JOINT NUMBER CRITICAL PARAMETERS DISTRIBUTION

	10 LEVEL CROSSCUT		23 LEVEL CONVEYOR DECLINE	
Joint Number	Quantity	Ave. Q-Value	Quantity	Ave. Q-Value
Massive	8	500	0	0
Few	8	242.3	0	0
1 Joint Set	27	52.75	0	0
1 Joint Set Plus Random	7	25.5	0	0
Two Joint Sets	20	25.9	0	0
Two Joint Sets Plus Random	3	12.9	0	0
Three Joint Sets	4	9.2	0	0
Three Joint Sets Plus Random	0	0	12	1.2

TABLE B.5 : JOINT ROUGHNESS CRITICAL PARAMETERS DISTRIBUTION

	10 LEVEL CROSSCUT		23 LEVEL CONVEYOR DECLINE	
Joint Roughness	Quantity	Ave. Q-Value	Quantity	Ave. Q-Value
Joint Spacing >3m	10	500	0	0
Discontinuous Joints	1	50	0	0
Rough / irregular undulating	23	45.18	12	1.2
Smooth Undulating	1	50	0	0
Slickensided undulating	1	10	0	0
Rough / irregular planar	35	40.76	0	0
Smooth planar	6	23.6	0	0

TABLE B.6 : JOINT ALTERATION CRITICAL PARAMETERS DISTRIBUTION

	10 LEVEL CROSSCUT		23 LEVEL CONVEYOR DECLINE	
Joint Alteration	Quantity	Ave. Q-Value	Quantity	Ave. Q-Value
Tightly Healed	10	500	0	0
Unaltered Joint Walls	22	53.58	0	0
Slightly Altered,jw, non soft min. coat	40	42.22	0	0
Silty, small clay fraction	4	32.3	0	0
Softening mineral coating	1	1.7	0	0
Sandy particles, clay free, disintegrated rock	0	0	11	1.23
Zones or Bands of disintegrated rock	0	0	1	0.8

TABLE B.7 : STRESS REDUCTION FACTOR CRITICAL PARAMETERS DISTRIBUTION

	10 LEVEL CROSSCUT		23 LEVEL CONVEYOR DECLINE	
Stress Reduction Factor	Quantities	Ave. Q-Value	Quantities	Ave. Q-Value
Medium Stress	60	265	0	0
Sing. Shear, Compt Rock >50m Depth	16	16.2	0	0
Multiple Shear Zone in Comp., Loose Surr.	1	1.7	0	0
Loose open joints, sugar cube, heavy jointed	0	0	12	1.2

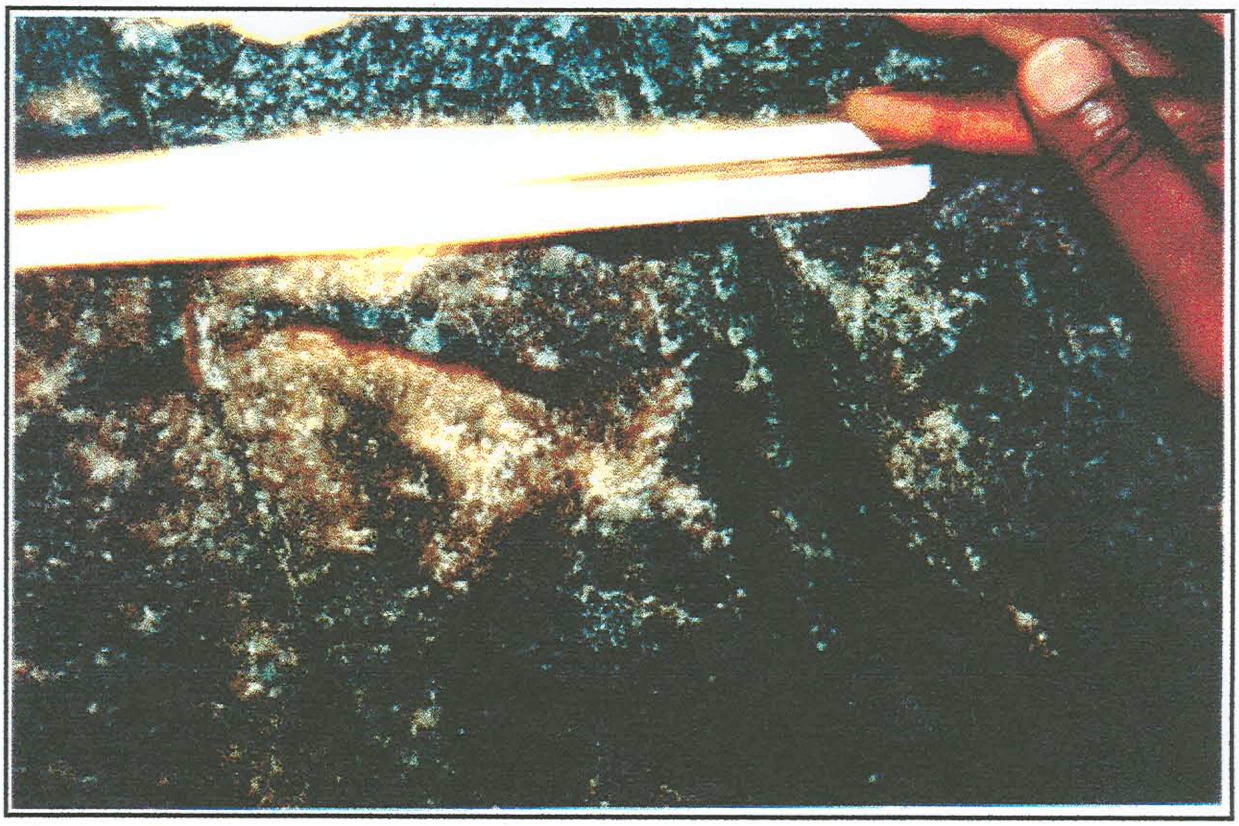
Table B.8 : Barton Comparison - Scatterplot Equivalent Dimension vs Q-values Data Base - 10 Level Crosscut

Site No.	Exc. Width	De	Q-value	Description	Photos	Supp. Installed & pattern/spacing - Photo No.
1	3	1.9	37.5	Good	Yes	Plate 19, 20
2	3	1.9	50.0	Very Good		No Bolting
3	3	1.9	15.0	Good		No Bolting
4	3	1.9	15.0	Good		No Bolting
5	3	1.9	6.7	Fair		No Bolting
6	3.2	2.0	6.7	Fair	Yes	Plate 14,15,16
7	3.2	2.0	75.0	Very Good	Yes	No Bolting
8	3	1.9	37.5	Good	Yes	No Bolting
9	3	1.9	50.0	Very Good		No Bolting
10	3	1.9	75.0	Very Good		No Bolting
12	3	1.9	37.5	Good		No Bolting
13	3	1.9	50.0	Very Good		No Bolting
15	3	1.9	150.0	Extremely Good		No Bolting
16	2.7	1.7	10.0	Good		No Bolting
17	2.7	1.7	50.0	Very Good		No Bolting
18	3	1.9	50.0	Very Good		No Bolting
19	5	3.1	7.5	Fair	Yes	Plate 17, 18
20	5	3.1	75.0	Very Good	Yes	No Bolting
22	3	1.9	150.0	Extremely Good		No Bolting
23	3.1	1.9	6.6	Fair		No Bolting
24	3	1.9	30.0	Good		No Bolting
29	3	1.9	30.0	Good		No Bolting
30	3	1.9	16.7	Good		No Bolting
31	3	1.9	1.7	Poor		No Bolting
33	3	1.9	50.0	Good		No Bolting
34	3	1.9	37.5	Good		No Bolting
35	3	1.9	37.5	Good		No Bolting
36	3	1.9	30.0	Good		No Bolting
37	3	1.9	37.5	Good		No Bolting
38	3	1.9	15.0	Good		No Bolting
39	3.2	2.0	25.0	Good		No Bolting
40	3	1.9	37.5	Good	Yes	Plate 9, 10
41	3	1.9	7.5	Fair		No Bolting
42	3	1.9	18.8	Good		No Bolting
43	3	1.9	18.8	Good		No Bolting
44	3	1.9	25.0	Good		No Bolting
45	3	1.9	75.0	Very Good		No Bolting
46	3	1.9	18.8	Good		No Bolting
47	3	1.9	25.0	Good		No Bolting
48	3.3	2.1	30.0	Good		No Bolting
49	3.3	2.1	75.0	Very Good	Yes	Plate 7, 8
50	3	1.9	75.0	Very Good		No Bolting
51	3.1	1.9	30.0	Good	Yes	Plate 6
52	3	1.9	75.0	Very Good		No Bolting
53	5	3.1	37.5	Good		No Bolting
54	3	1.9	150.0	Extremely Good		No Bolting
55	3	1.9	37.5	Good		No Bolting
56	3	1.9	75.0	Very Good		No Bolting
57	3	1.9	10.0	Fair/Good		No Bolting
58	3	1.9	16.7	Good		No Bolting
59	3	1.9	16.7	Good		No Bolting
60	3	1.9	37.5	Good		No Bolting
61	3	1.9	50.0	Very Good		No Bolting
62	3	1.9	37.5	Good		No Bolting
63	3	1.9	6.7	Fair		No Bolting
64	3	1.9	37.5	Good		No Bolting
65	3	1.9	75.0	Very Good		No Bolting
66	3	1.9	37.5	Good	Yes	Plate 2,3,4
67	3	1.9	12.5	Good		No Bolting
68	3	1.9	75.0	Very Good		No Bolting
69	3.1	1.9	22.9	Good	Yes	Plate 1

TABLE B.9 : CASE STUDY 2 - 23 LEVEL CONVEYOR DECLINE NO. 14-SHAFT - CALCULATED CONDITIONAL FACTORS - BARTON et al (1977).

Site No.	Exc. Width	De - Span/ESR	Act. RQD	Description	Jn	Description	RQD / Jn	Jr	Description	Ja	Description	Jr/Ja	Jw	Q-value	Description	Photos	Supp. Installed & pattern/spacing
1	7	4.4	100	Excellent	12	Three Joint Sets Plus Random	8.3	3	Rough/Irregular undulating	6	Zones or bands of disintergrate or crushed	0.5	1	0.8	Very poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
2	7	4.4	100	Excellent	12	Three Joint Sets Plus Random	8.3	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintegrated rock	0.75	1	1.3	Poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
3	7	4.4	100	Excellent	12	Three Joint Sets Plus Random	8.3	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintegrated rock	0.75	0.5	0.6	Very poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
4	7	4.4	100	Excellent	12	Three Joint Sets Plus Random	8.3	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintegrated rock	0.75	1	1.3	Poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
5	7	4.4	100	Excellent	12	Three Joint Sets Plus Random	8.3	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintegrated rock	0.75	1	1.3	Poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
6	7	4.4	100	Excellent	12	Three Joint Sets Plus Random	8.3	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintegrated rock	0.75	1	1.3	Poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
7	7	4.4	100	Excellent	12	Three Joint Sets Plus Random	8.3	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintegrated rock	0.75	1	1.3	Poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
8	7	4.4	100	Excellent	12	Three Joint Sets Plus Random	8.3	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintegrated rock	0.75	1	1.3	Poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
9	7	4.4	100	Excellent	12	Three Joint Sets Plus Random	8.3	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintegrated rock	0.75	1	1.3	Poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
10	7	4.4	100	Excellent	12	Three Joint Sets Plus Random	8.3	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintegrated rock	0.75	1	1.3	Poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
11	7	4.4	100	Excellent	12	Three Joint Sets Plus Random	8.3	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintegrated rock	0.75	1	1.3	Poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart
12	7	4.4	100	Excellent	12	Three Joint Sets Plus Random	8.3	3	Rough/Irregular undulating	4	Sandy particles, clay free, disintegrated rock	0.75	1	1.3	Poor	Yes	3,0m Long, 16mm Diameter Shepherd Crook, Spaced 1m apart

PLATE 1



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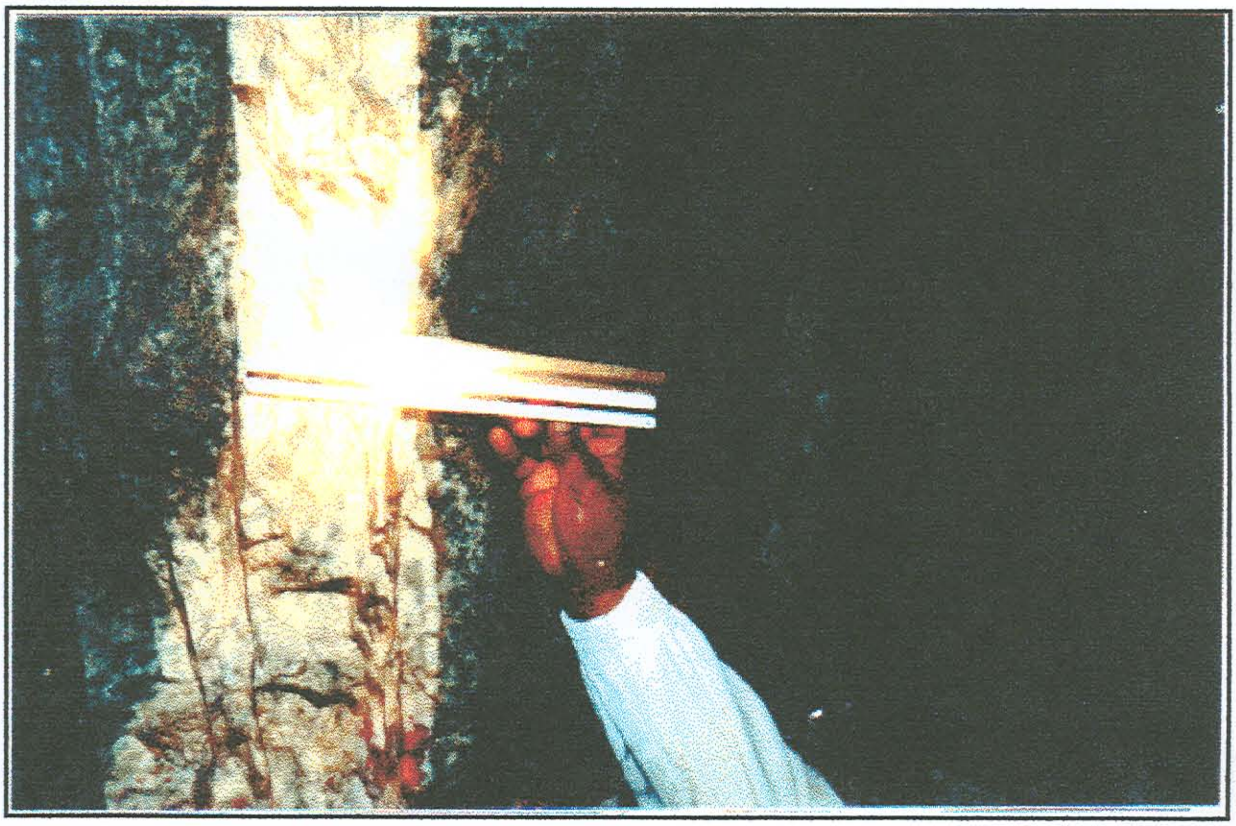

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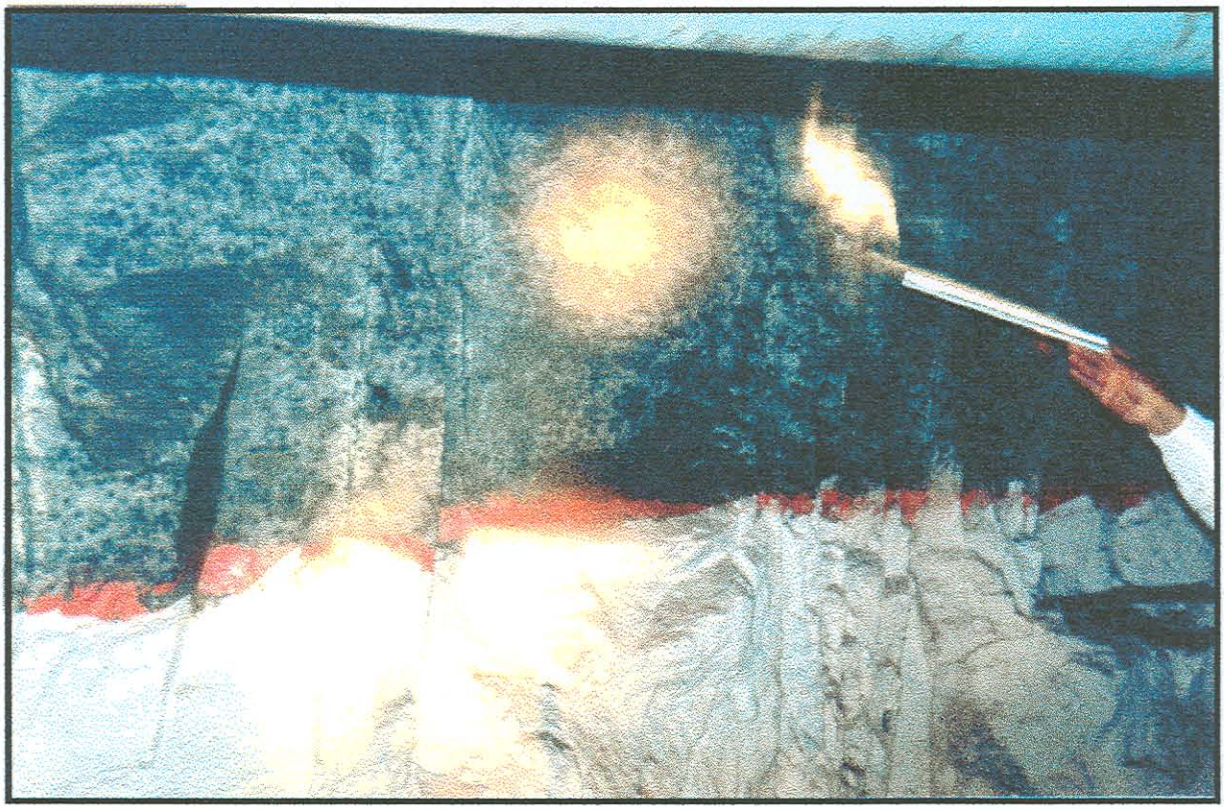


PLATE 4

PLATE 5

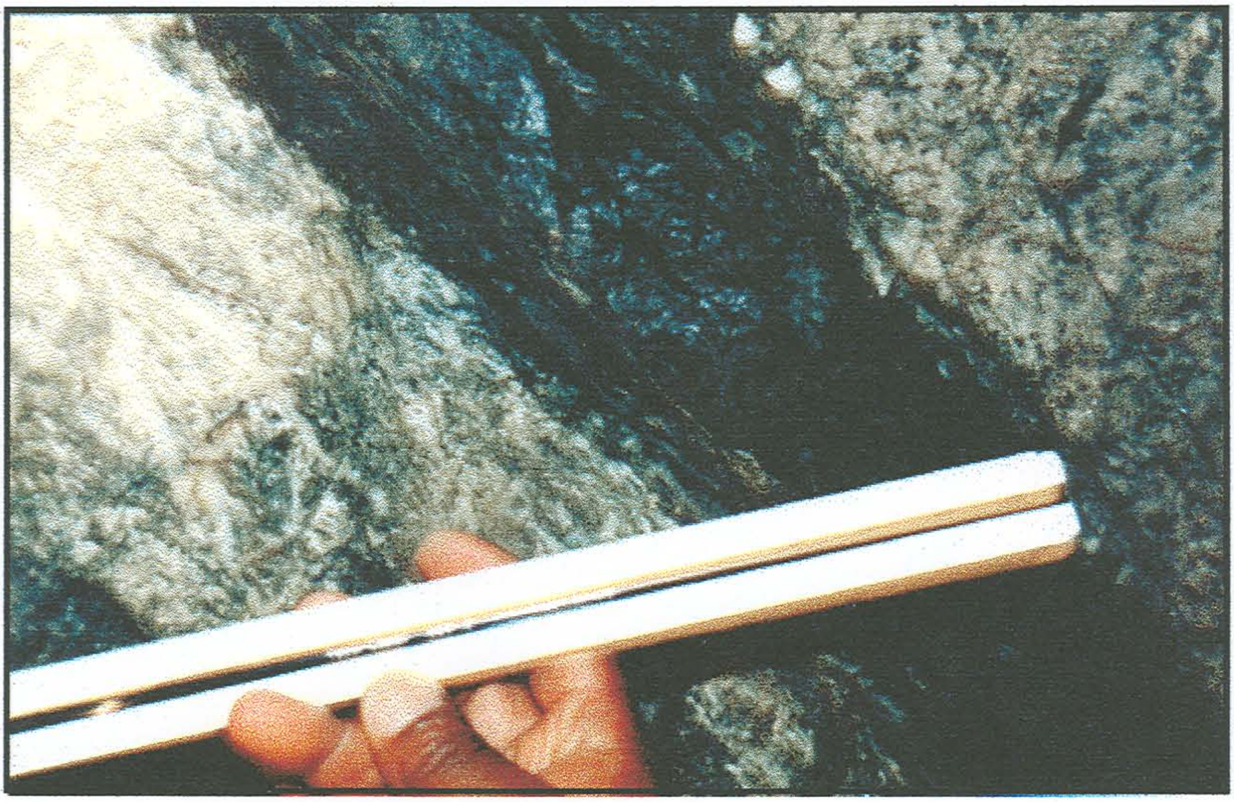
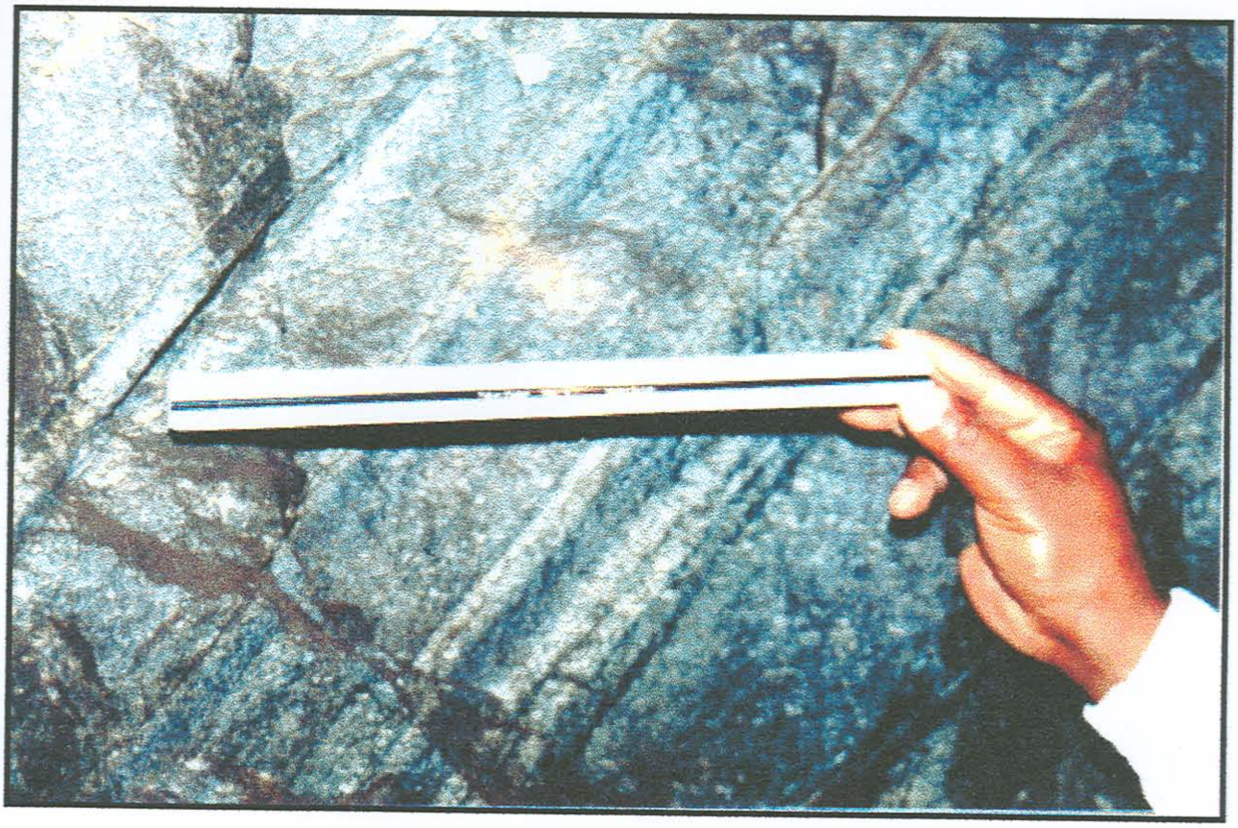


PLATE 6

PLATE 7



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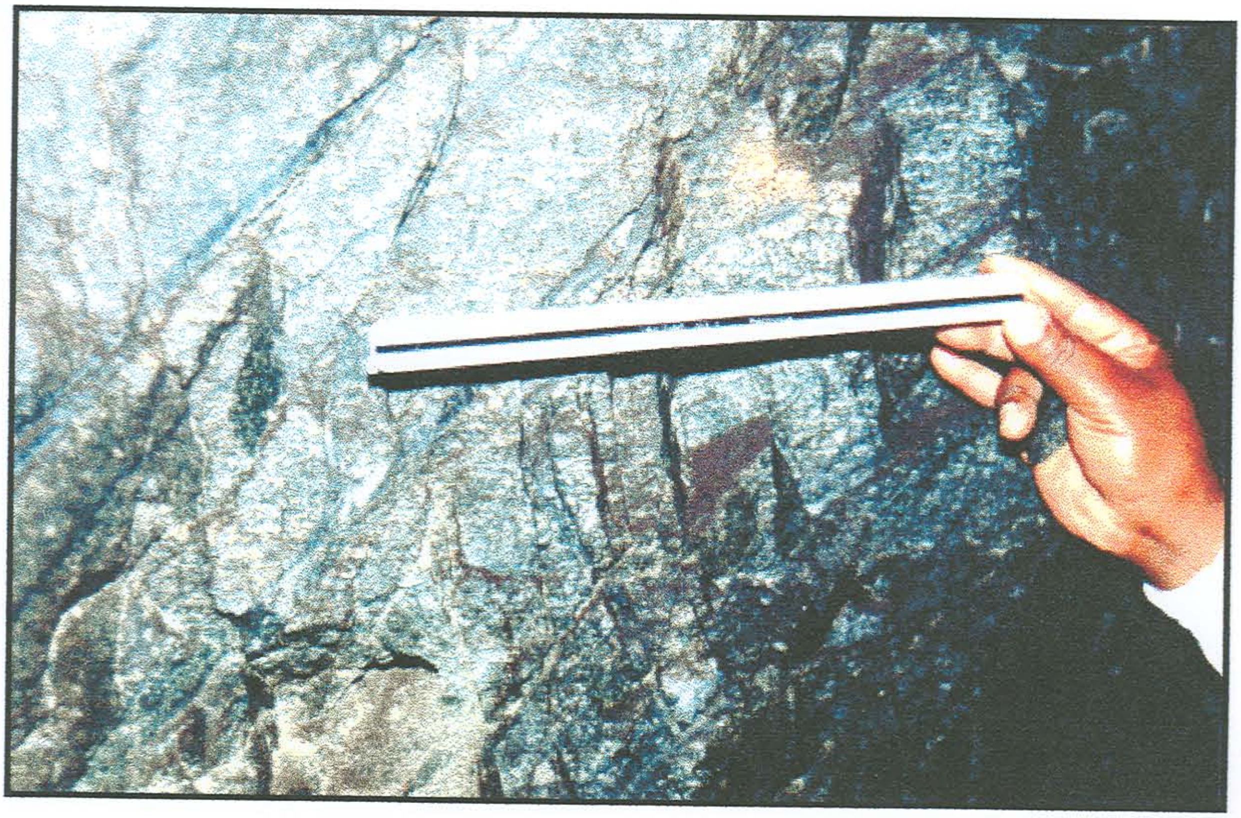

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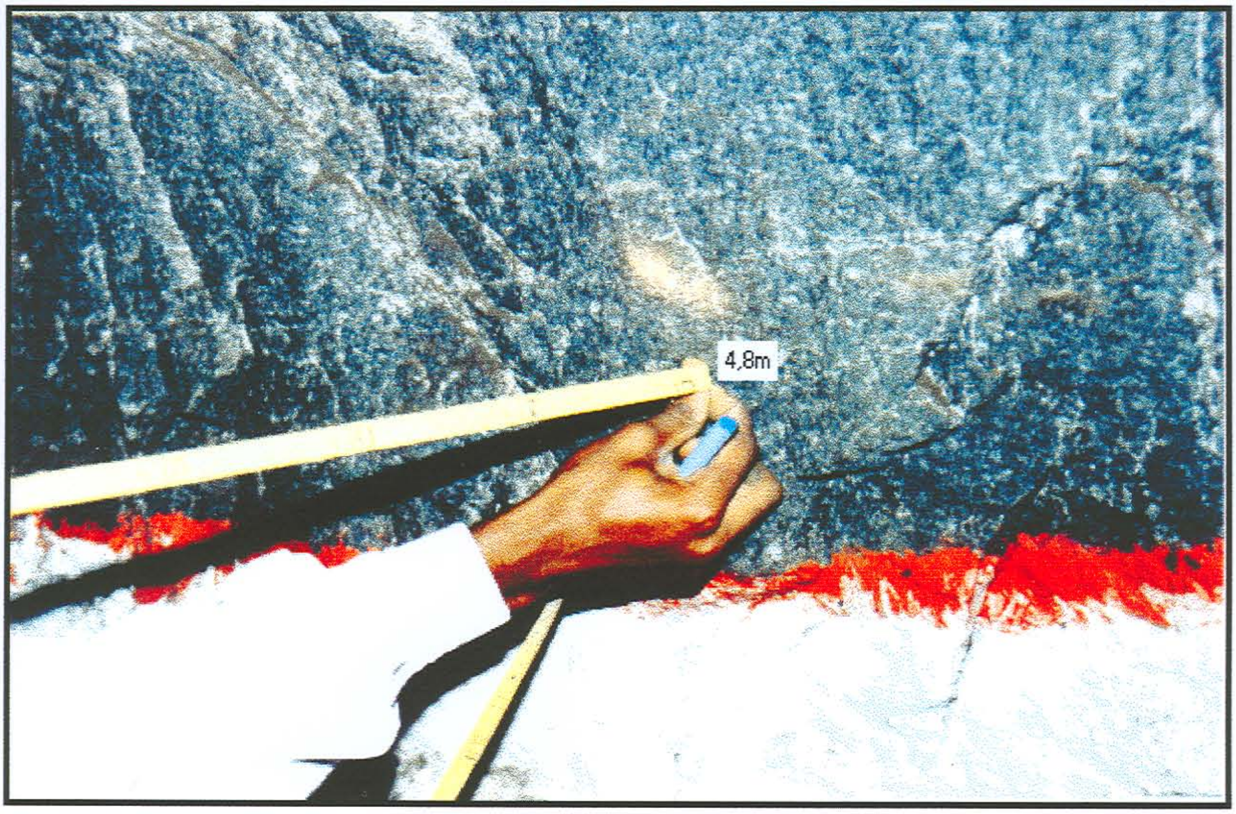

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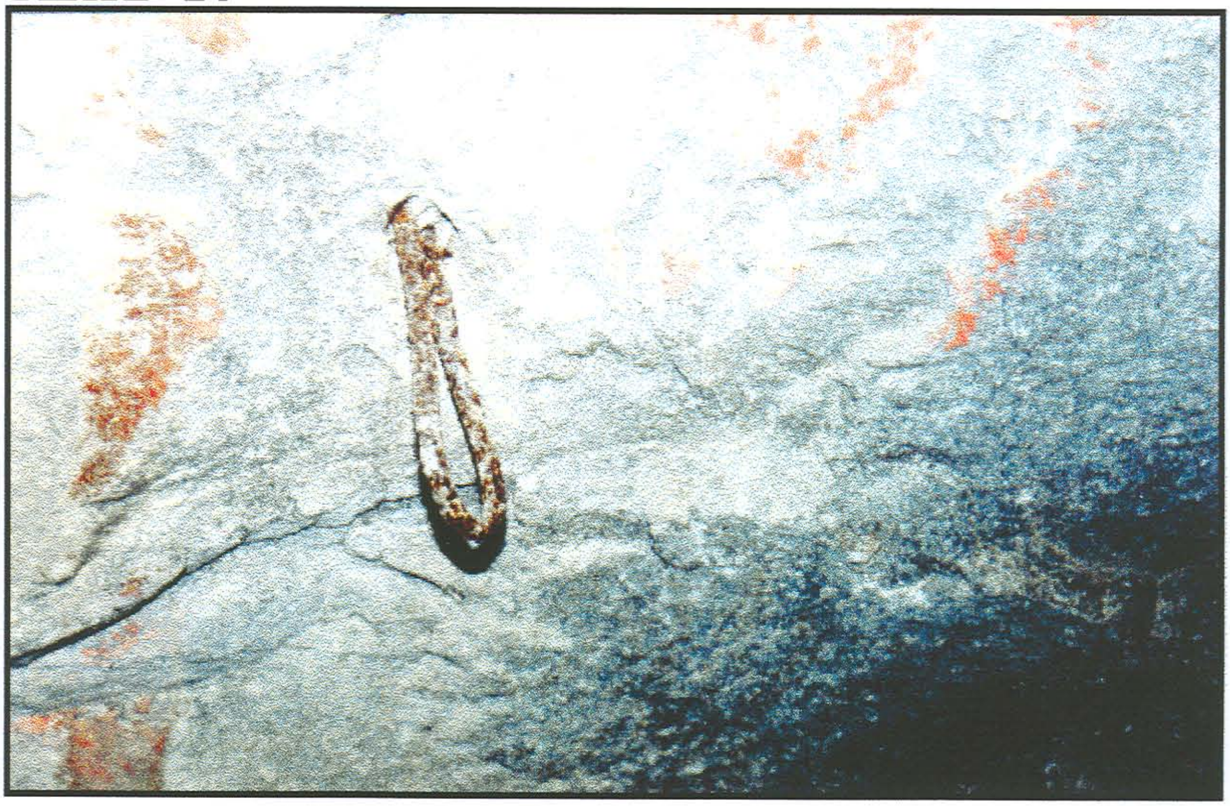

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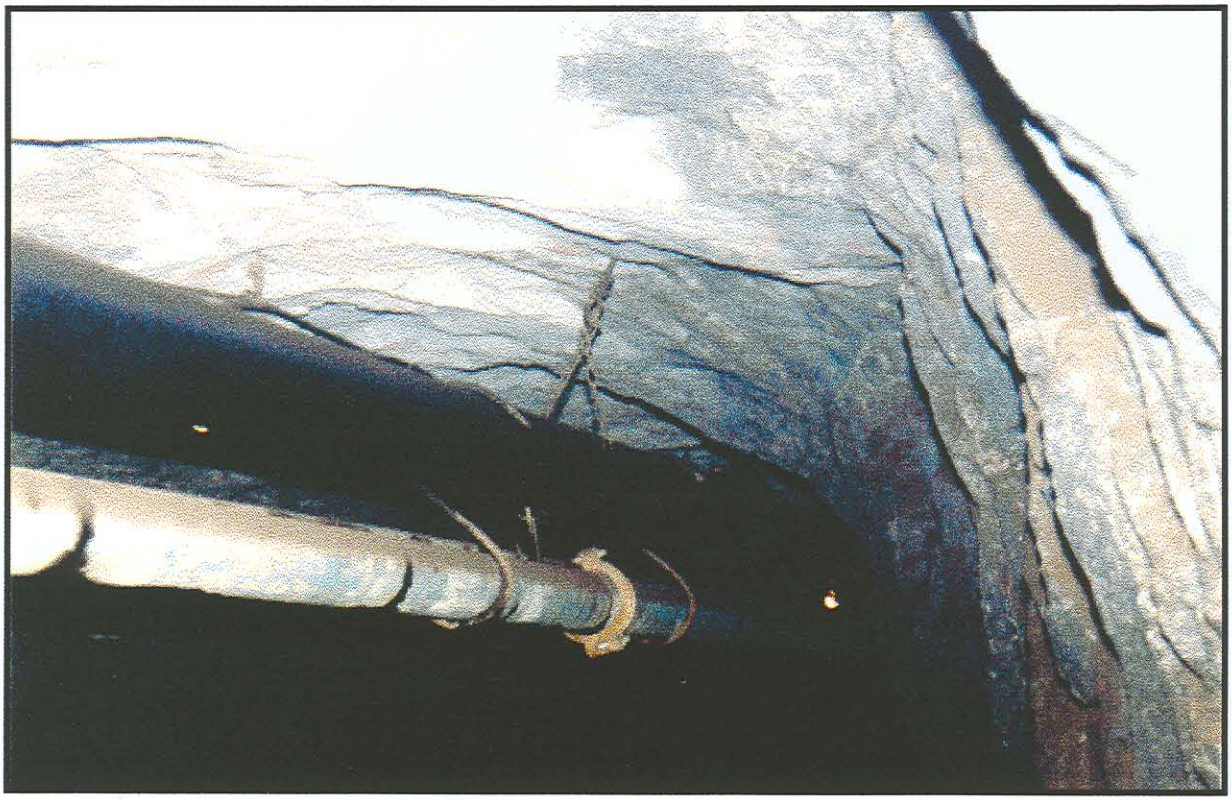
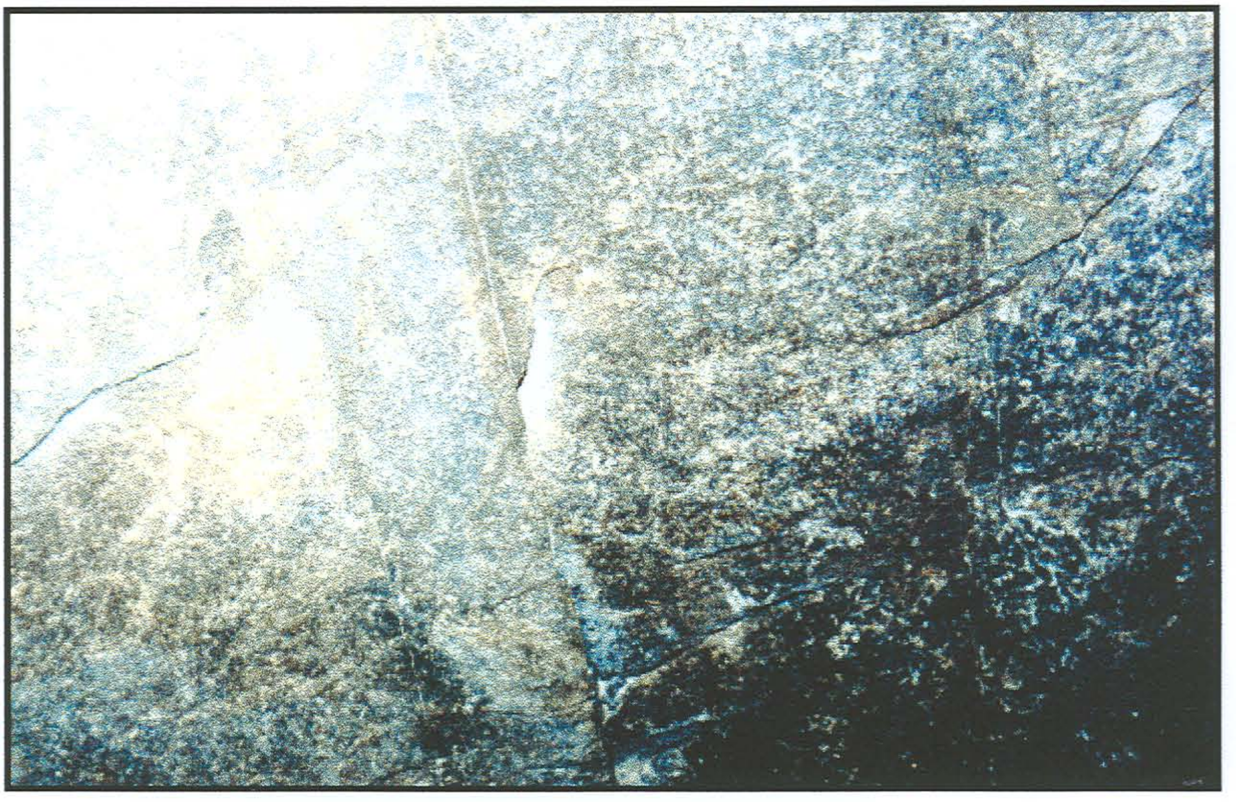


PLATE 12

PLATE 13



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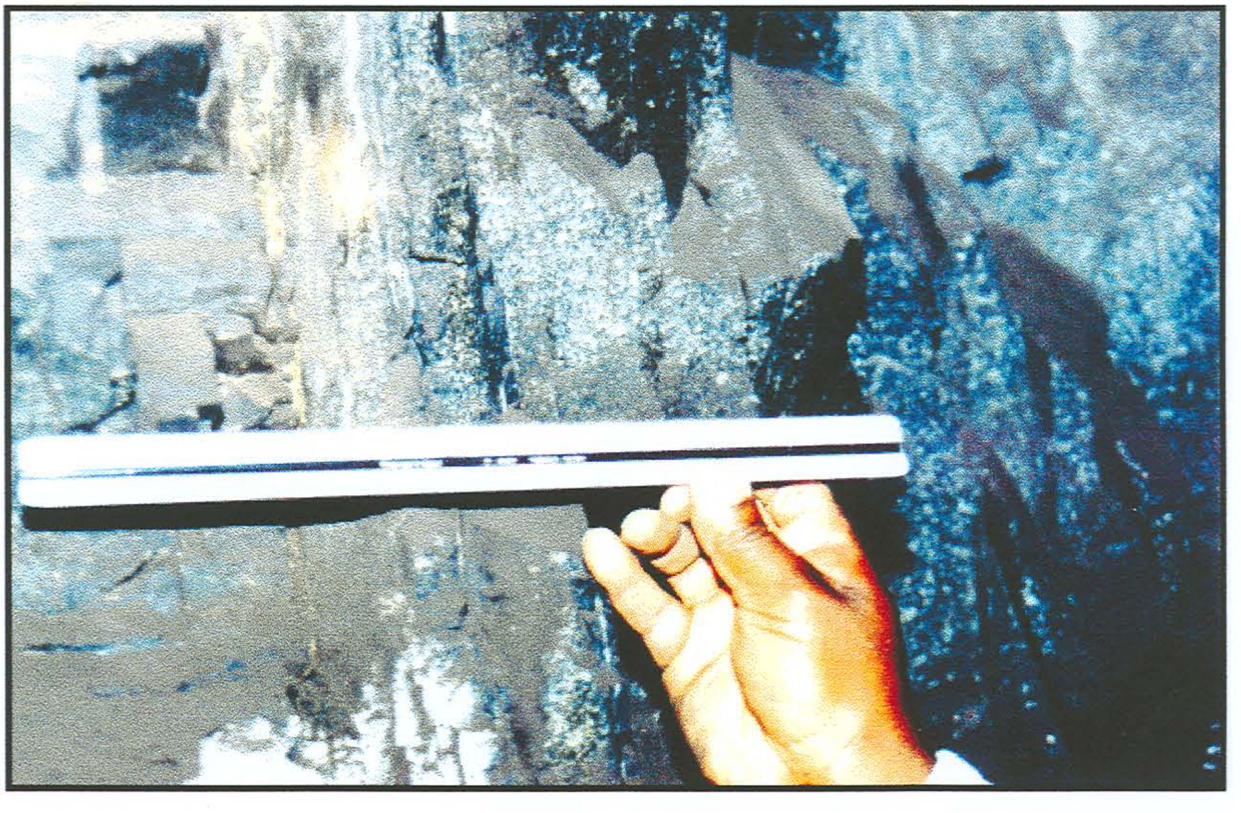

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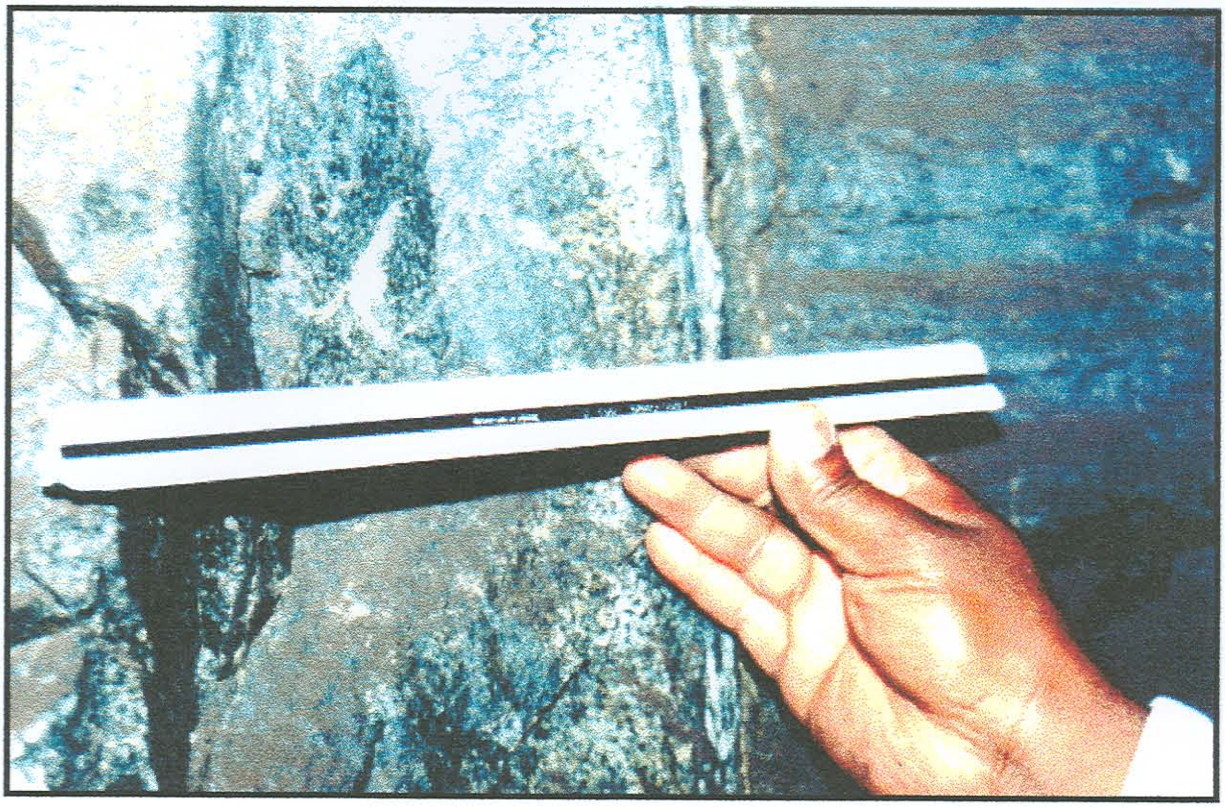
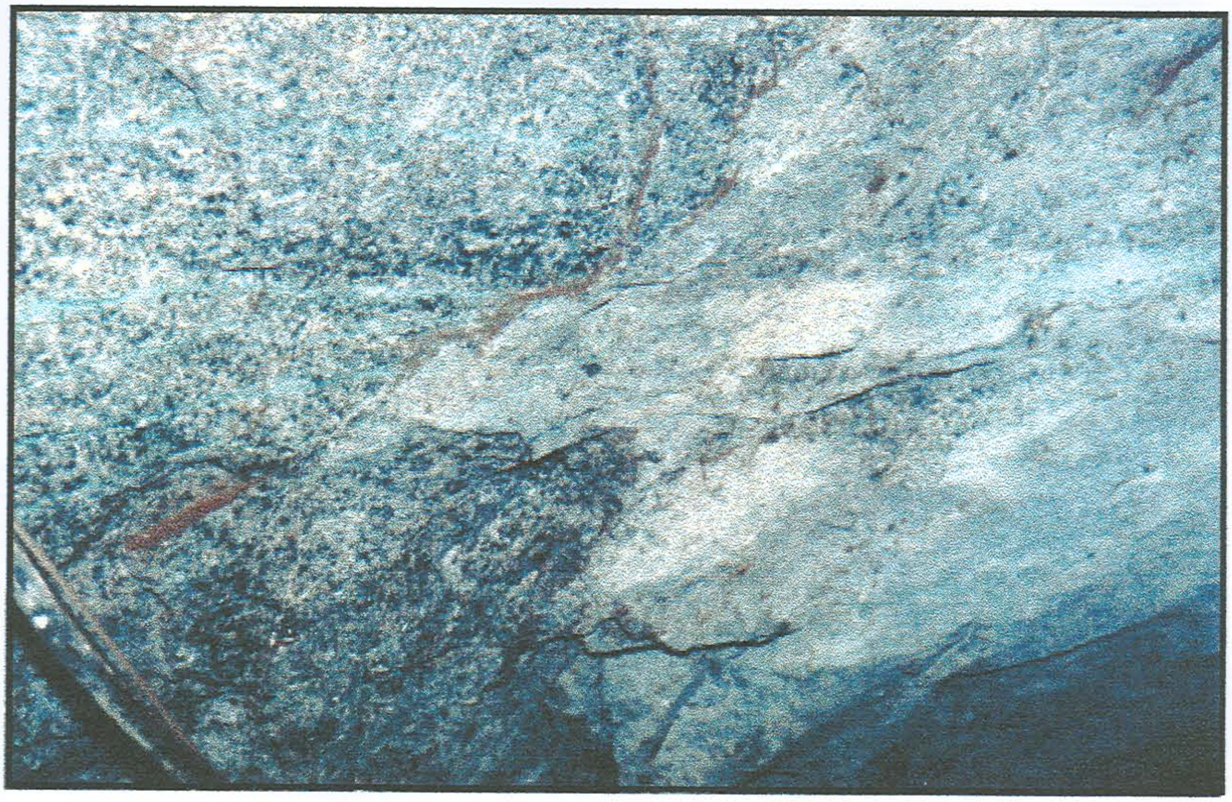


PLATE 16

PLATE 17



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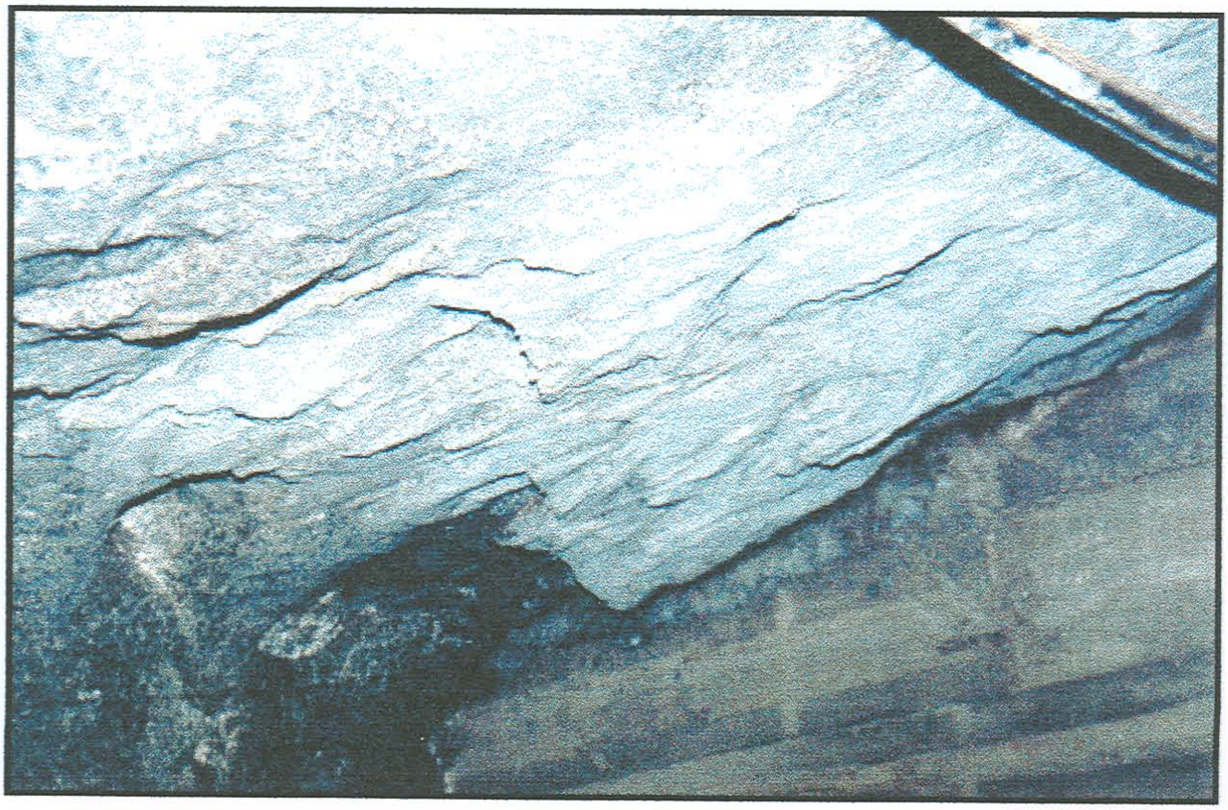

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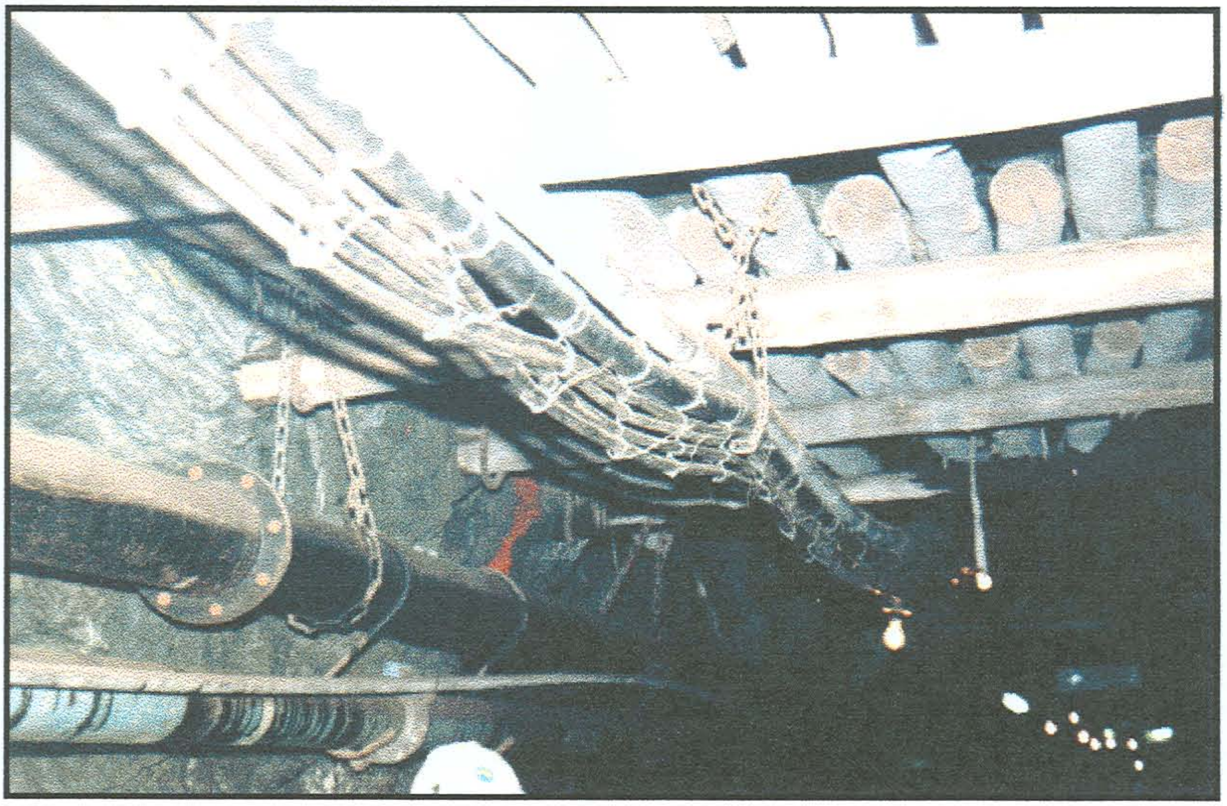


PLATE 20

APPENDIX D

PLATE 21

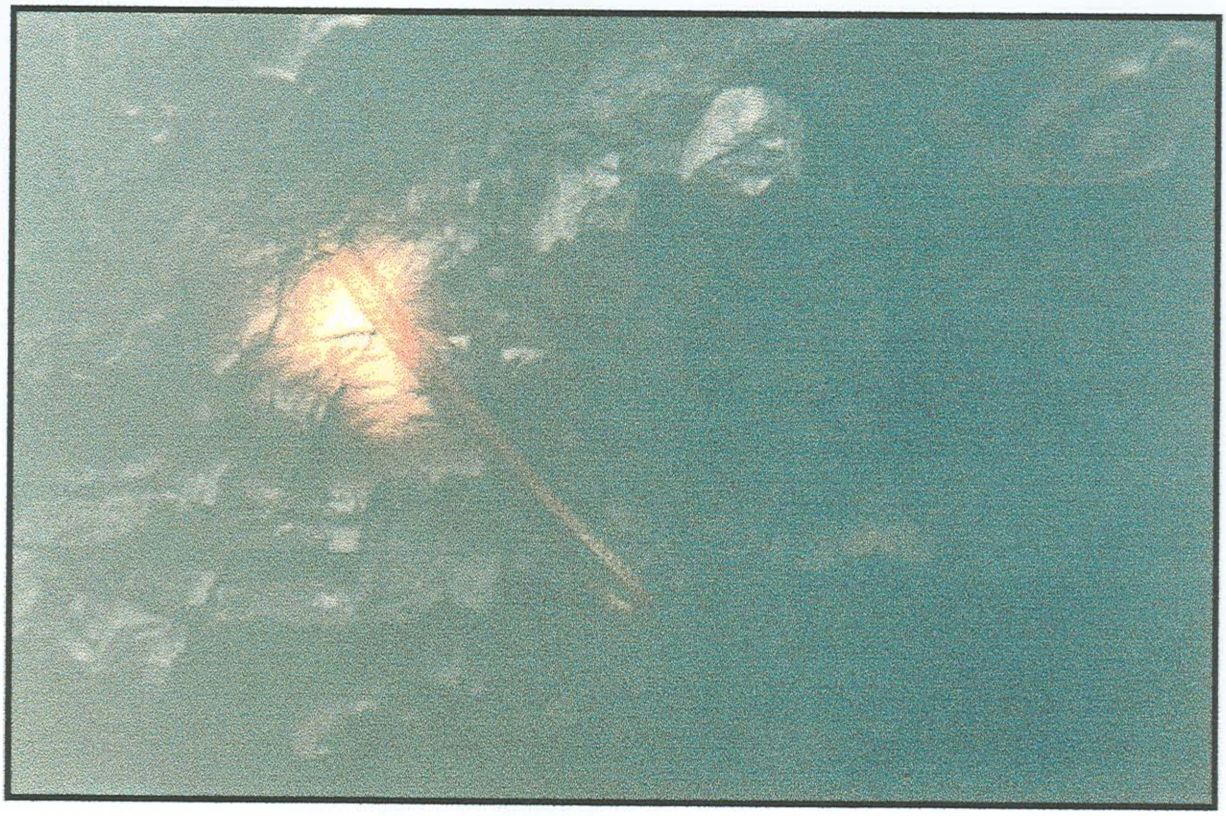
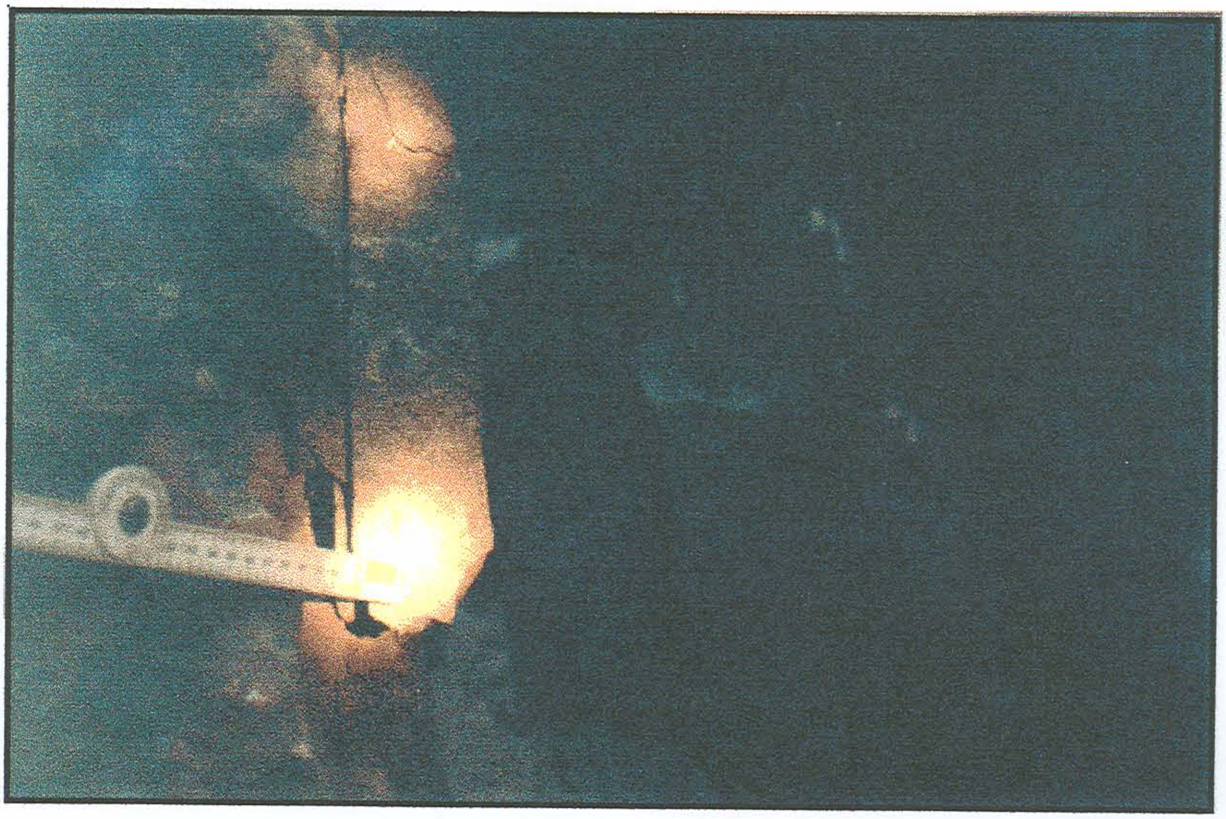


PLATE 22



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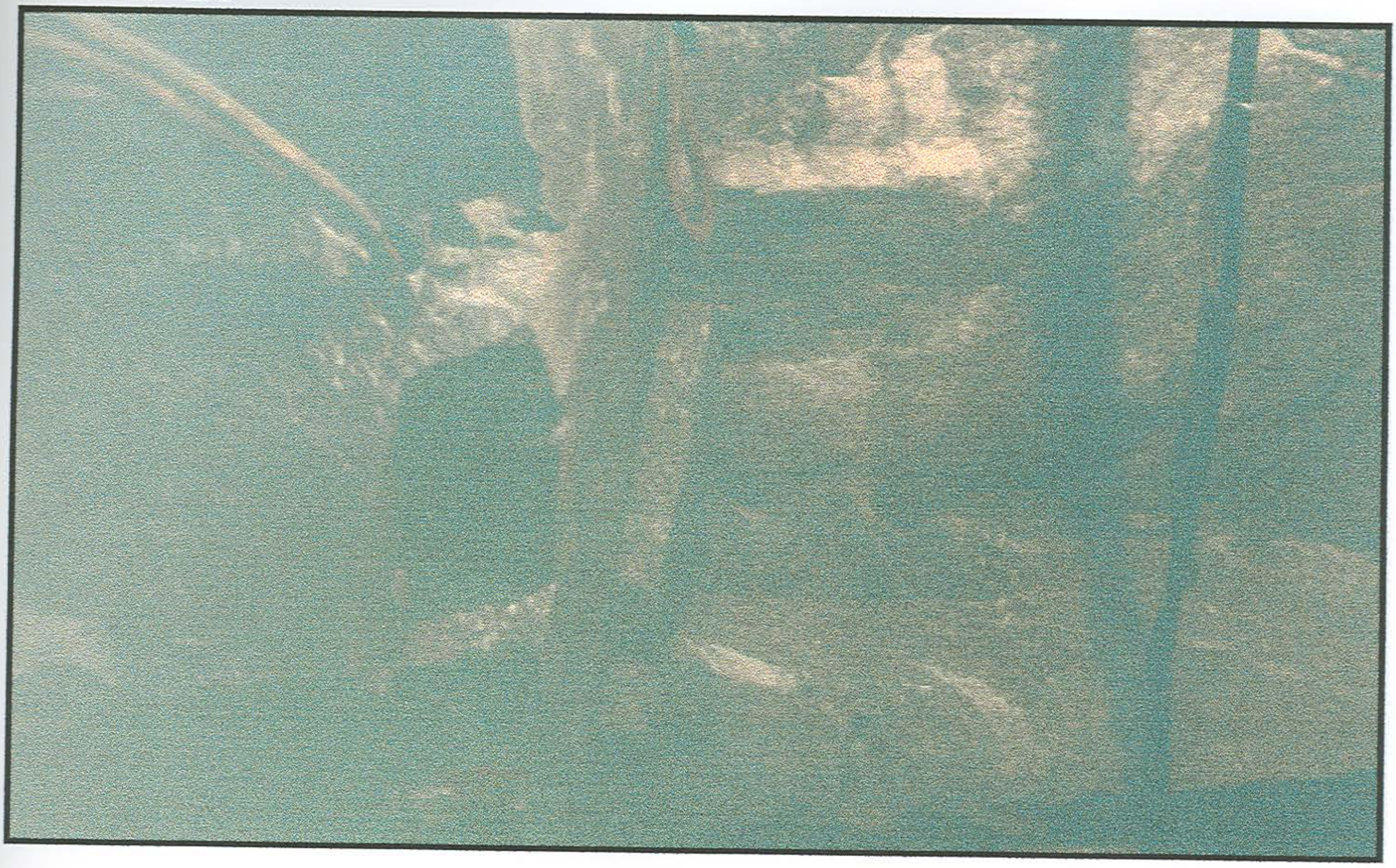

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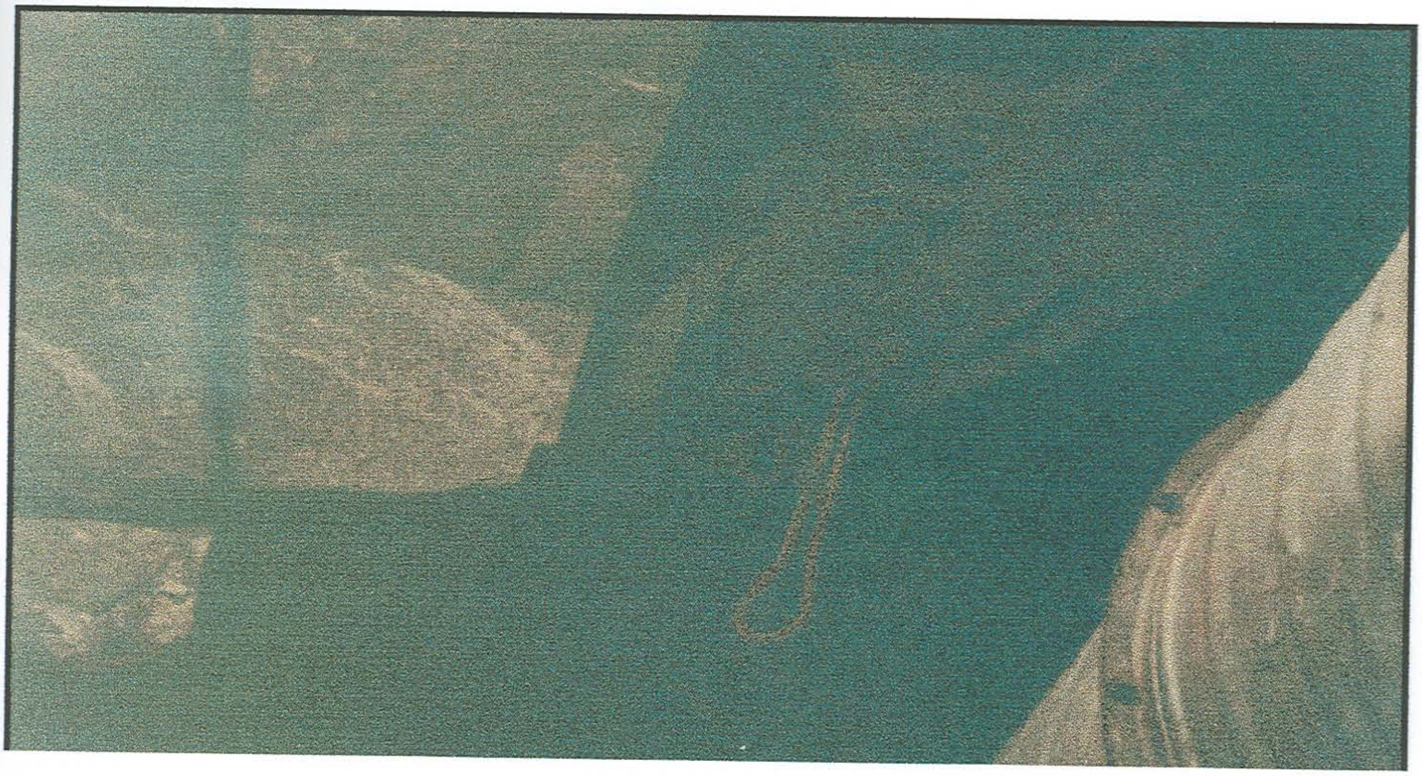

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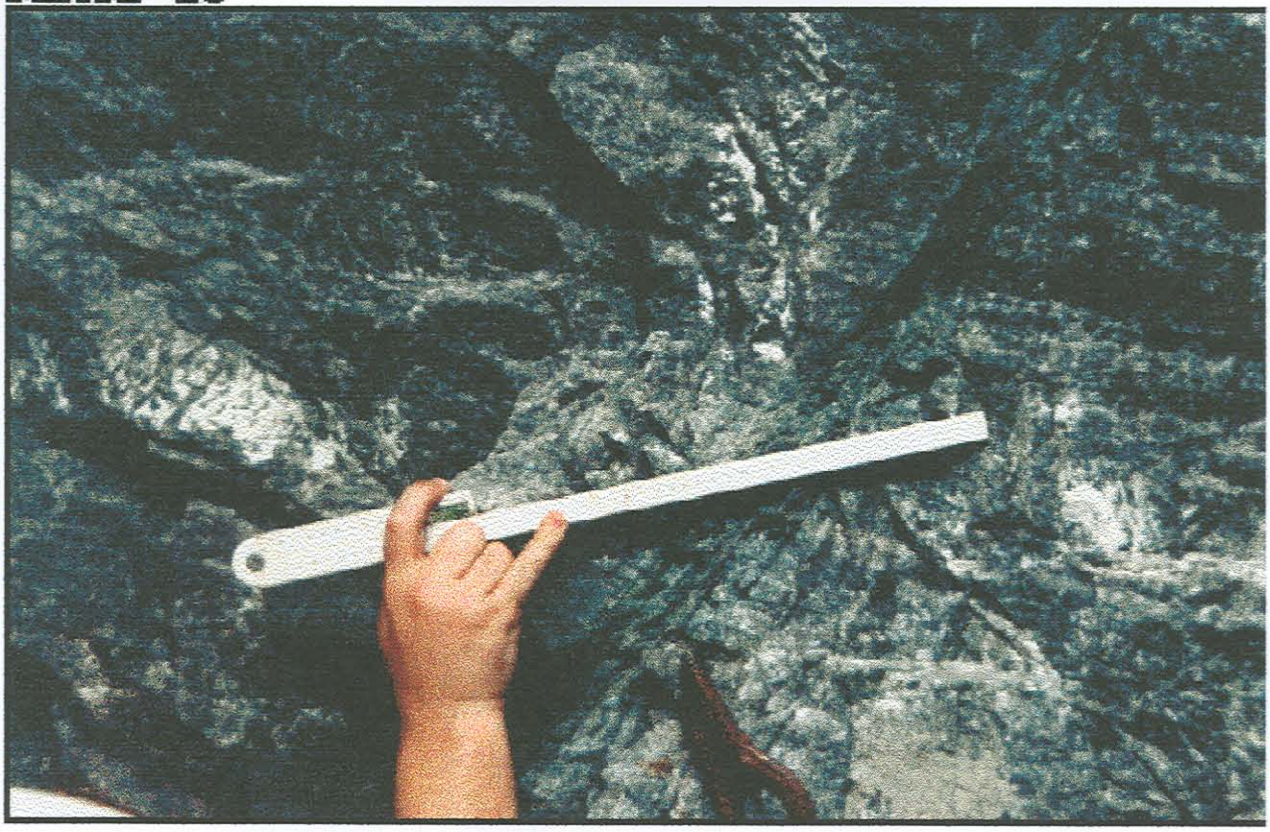


PLATE 26

PLATE 27

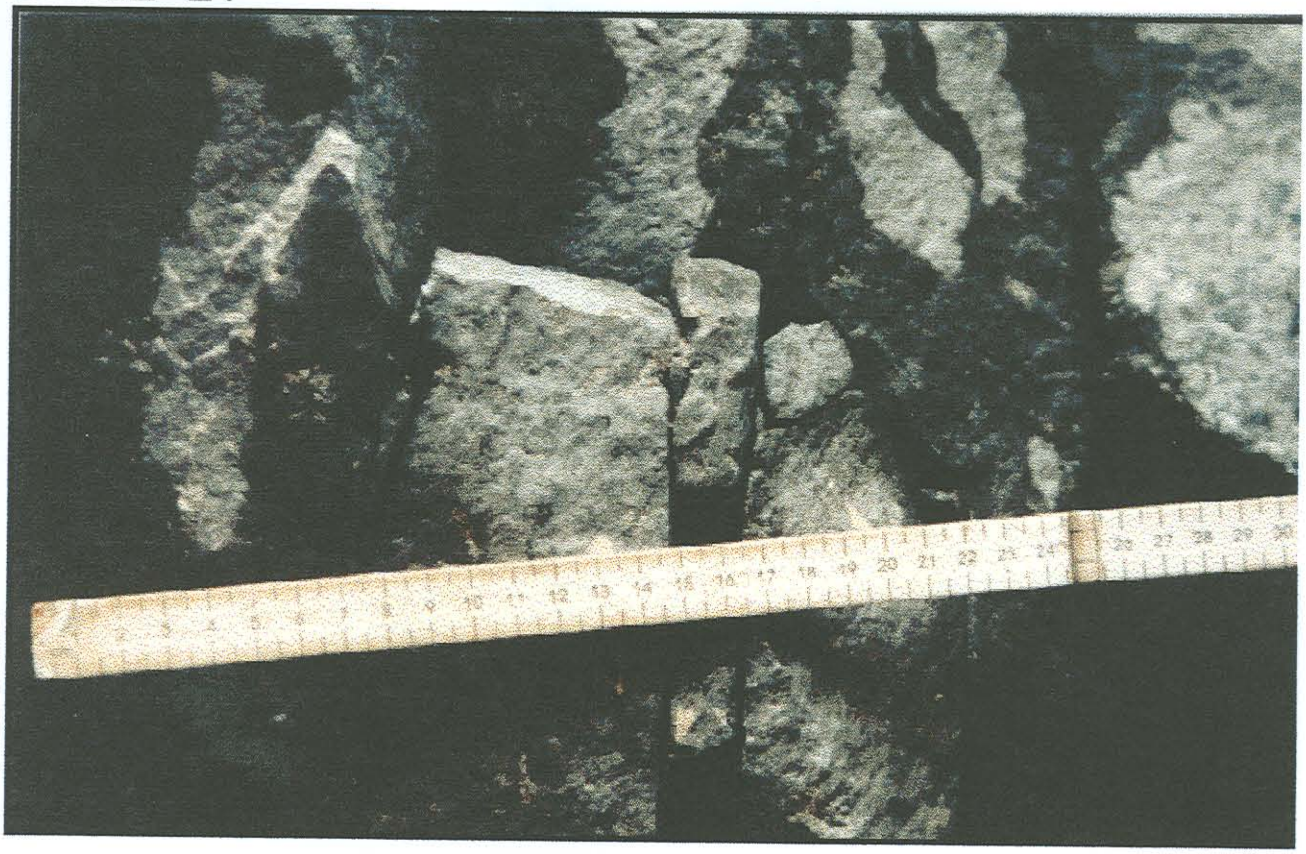
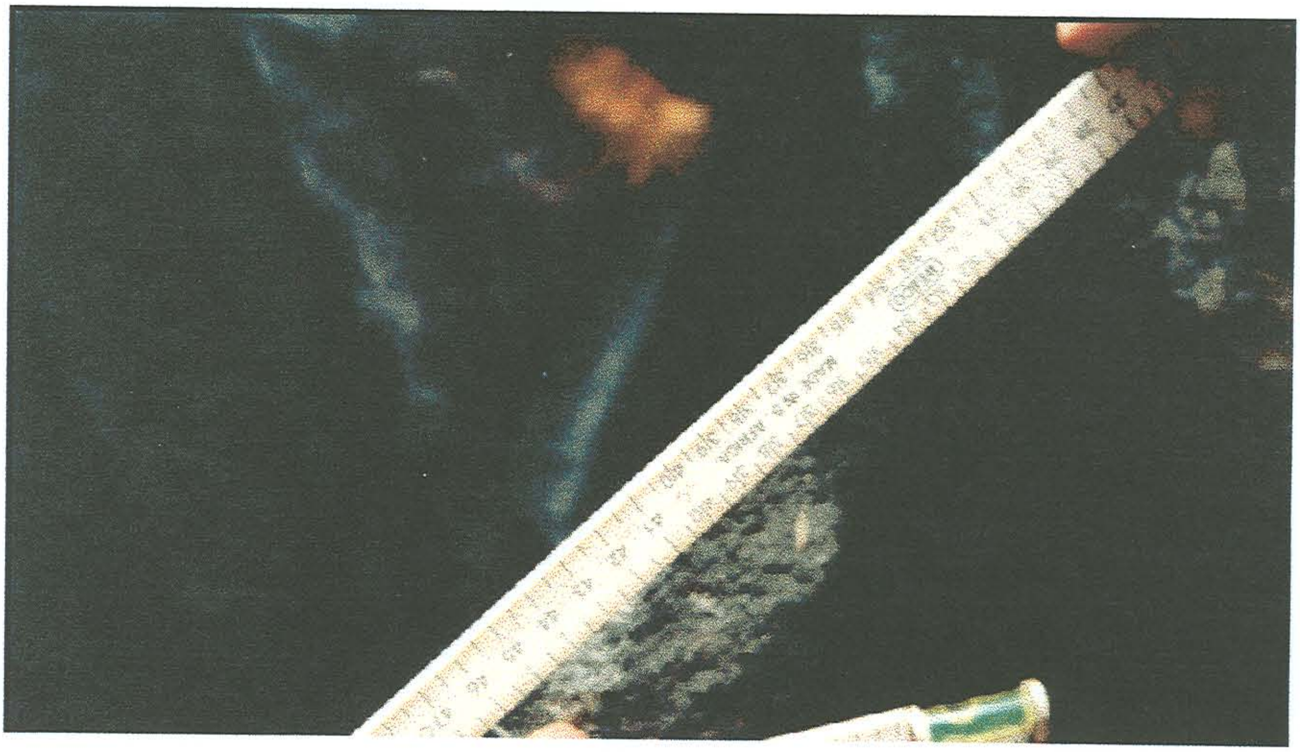
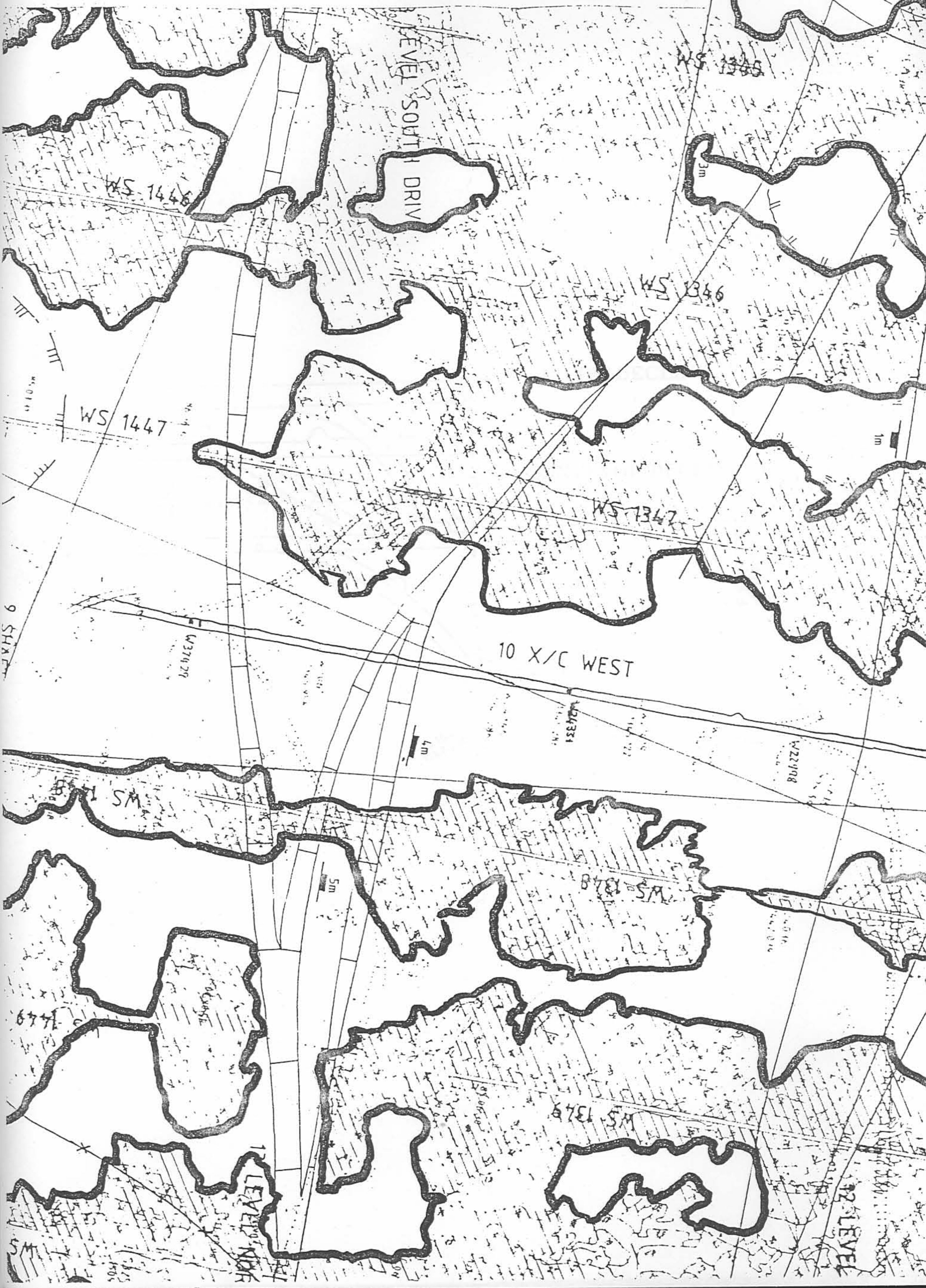


PLATE 28

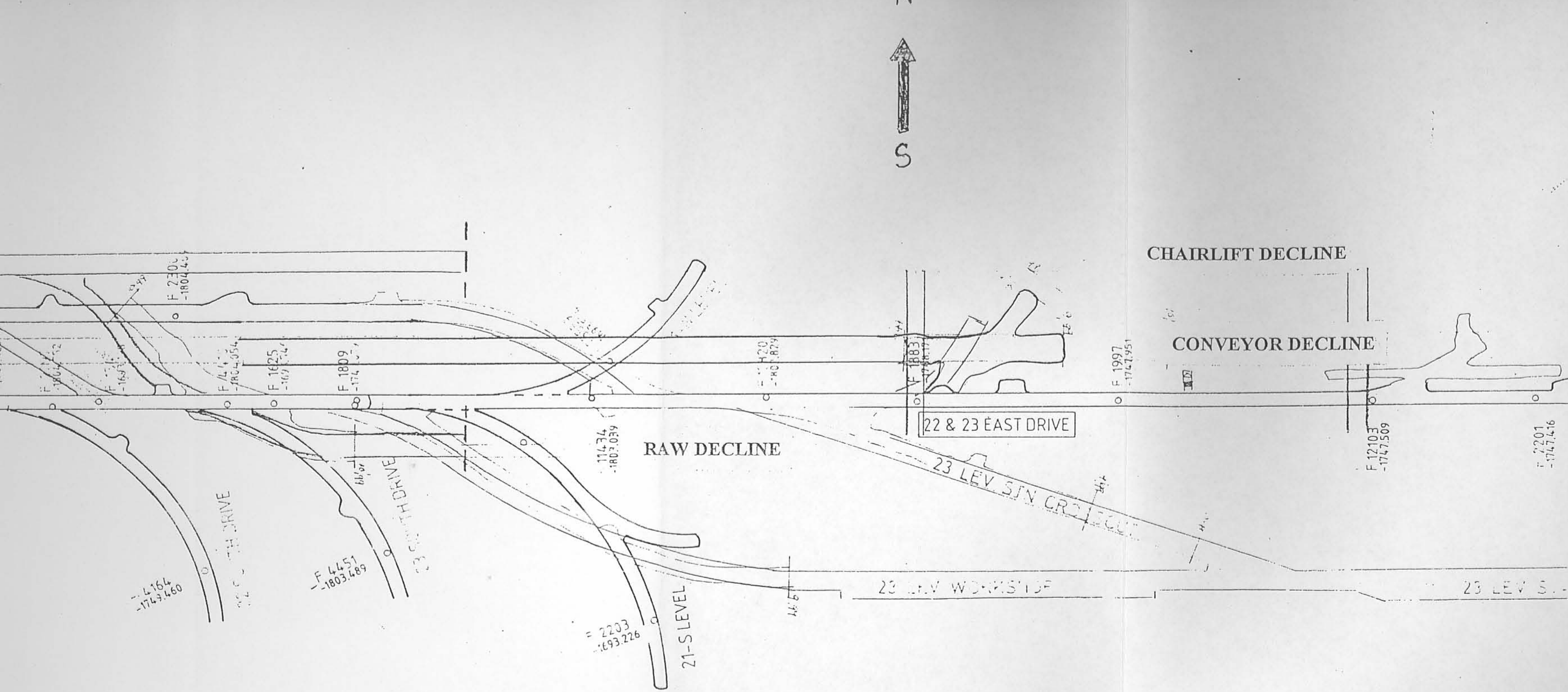


APPENDIX E

PLAN 1 - 10 LEVEL CROSSCUT – N0. 9-SHAFT
SCALE : 1 : 1000




WS 1245
LEVEL SOUTH DRIVE
WS 1145
SOUTH DRIVE
WS 1246
WS 1146
S
N
WS 1147
3m
10 X/CUT WEST
NBN 6
W18562
10 LEVEL
W20505
WS 1248
WS 1148
WS 1249
LEVEL NORTH


N
S
CHAIRLIFT DECLINE
CONVEYOR DECLINE
RAW DECLINE
22 & 23 EAST DRIVE
23 LEV STN CROSSCUT
21-S LEVEL
F 1625
F 1809
F 1997
F 4451
-1803.489
F 2203
-1693.226
F 12103
-1747.509
F 1883
F 2300
F 11434
-1747.460