THE APPLICATION OF THE Q TUNNELLING QUALITY INDEX TO ROCK MASS ASSESSSMENT AT IMPALA PLATINUM MINE

BY

W. HARTMAN*

AND

M.F.HANDLEY†

October 2001

*Senior Rock Mechanics Engineer, Douglas Partners, 96 Hermitage Road, West Ryde, New South Wales 2114, Australia (formerly Senior Rock Engineering Officer Projects, Impala Platinum Limited) e-mail: wouterhartman@aol.com

†Associate Professor of Rock Mechanics, Department of Mining Engineering, University of Pretoria, Pretoria 0002. Tel.: 012-420-3181, Fax: 012-362-5304, e-mail: mhandley@postino.up.ac.za

SYNOPSIS

Publications on hard rock tunnel support design in the South African mining industry have originated primarily from the gold mines during the last 50 years. Little has been published to date on the platinum and chrome mines of the Bushveld Complex. This paper covers the search for a suitable design methodology for off-reef tunnels at Impala Platinum Mine, situated on the Western Lobe of the Bushveld Complex. The fall of ground accident statistics for off-reef excavations of the mine are presented and the available rock mass quality evaluation systems are reviewed. The Q Tunnelling Quality Index (or Q-Index) is selected because it assesses the important driving factors behind falls of ground at Impala. Two off-reef excavations are evaluated using the Q-Index, and it is shown that minor modifications are required for implementation at Impala. It will take some time before support design throughout the mine is based on the outcome of proper geotechnical investigations based on the Q-Index. Widespread implementation of such support designs should help to solve the fall of ground problem in off-reef excavations while at the same time reducing support costs.

1 INTRODUCTION

The current mining industry regulations and guidelines for South Africa call for systematic underground support that is capable of resisting 95% of all potential falls of ground as determined by statistical analysis. Data obtained from all fall of ground injuries at Impala Platinum Mine for the last 12 years show that most falls occurred on the reef horizon, with a smaller proportion in off-reef excavations. A proper engineering approach to support design is already in place for stoping, but support in off-reef excavations is still designed using the 33 kN/m^2 -support resistance criterion without a prior geotechnical investigation. This practice suffers from the disadvantage that geotechnical conditions, which affect the potential for rockfalls, are ignored. Hence, some off-reef excavations are likely to be over-supported while others are under-supported.

Consultants to the mine suggested several rockmass classification systems to take account of geotechnical conditions in support design, but it soon became apparent that every available rockmass classification system has its origins in an application to a specific problem, therefore limiting its generality, and potential applicability on the mine. It became apparent during this period that in order to ensure safe and cost-effective support in off-reef excavations at Impala Platinum, a suitable rockmass classification system would have to be found.

This paper describes the approach used to select and implement the Q-Tunnelling Quality Index[1] (or Q-Index) in the off-reef excavation support design process at Impala Platinum. First, an accident analysis is presented in which the nature of the

fall of ground problem in off-reef development is defined. Then, currently available rockmass evaluation systems are described to highlighting the strengths and weaknesses of each. This forms the background to the choice of the Q-Index, and provides a clear indication why it is suitable for Impala. To confirm its applicability, the Q-Index is applied to two tunnels in different geotechnical conditions. The first case study covers a tunnel, which has stood virtually unsupported in a very good quality rockmass for twenty years, while the second covers an incline shaft presently being developed in a fair to poor quality rockmass. The methodology developed to assess these tunnels is described and the support designs appropriate for these tunnels is presented.

The case studies show that the Q Tunnelling Quality Index predicts larger stable unsupported spans than those observed by experience on the mine. The results were used to adjust the relationship between the equivalent dimension D_e and the rock mass quality index Q (Barton et al.[1]). Although all rock mass classification systems strive for simplicity while at the same time addressing the complexity and diversity of natural rock masses, the adjustment to the Q-Index for Impala suggests that a general system applicable to all situations still does not exist. It appears from this study and the experience of others, for example Laubscher and Jakubec[2], that all rock mass classification schemes need some adjustment to suit local site conditions, and that the user should apply proper engineering judgment when doing so. A simple and reliable rockmass evaluation system based on the Q Tunnelling Quality Index together with a support design procedure for off-reef development is now in place on Impala Platinum Mine.

2 MINE LOCATION AND GEOLOGICAL SETTING

Impala Platinum Mine is situated 23km north of Rustenburg on the western edge of the western lobe of the Bushveld Complex. The mining area extends approximately 25 km from the south to the north (see Figure 1). The mine exploits the Merensky Reef and the UG2 Reef for platinum group metals and several other by-products.

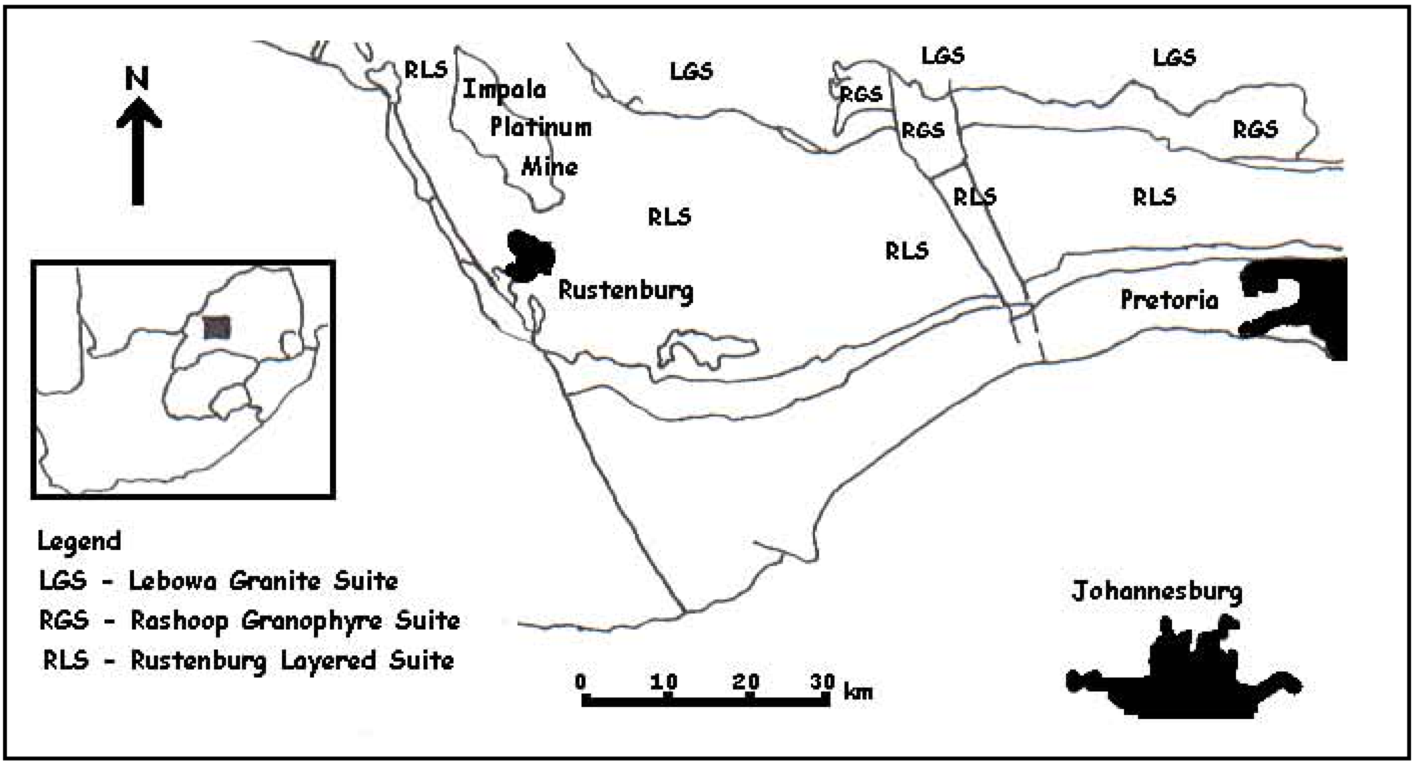

Figure 1: Location of Impala Platinum Mine

The Bushveld Complex is a layered igneous intrusion consisting of mafic rocks such as chromitite, pyroxenite, norite, gabbro, gabbronorite, harzburgite, and anorthosite in the Lower, Critical, and Main zones[3]. The Upper Zone, which forms the cap of the layered intrusion, consists of gabbro, anorthosite, and gabbronorite towards its base, with diorites at the top. The various Bushveld Granites succeeded these mafic intrusions. There are seven overlapping mafic intrusions, which are shaped like

inverted cones spreading out laterally toward the surface. Each can be likened to an inverted wine glass. The layers around the edge of each intrusion (equivalent to the wine glass base) dip towards the centre at an average of 9 to 10 degrees. The dip increases towards the centre where the vertical feeder is situated. The Bushveld Complex is large, stretching 370 km east west across North West Province, northern Gauteng, and Mpumalanga.

Table I: Generalised Geological Succession for Impala Platinum Mine with Average Unit Thickness, Unit Names and Rock Types

Average Thickness (m)	Unit	Rock Type
34	HW5	Mottled and spotted Anorthosite
3-6	HW4	Spotted Anorthosite
5-7	HW3	Mottled Anorthosite
1.5-3	HW2	Spotted Anorthositic Norite
2-6	HW1	Norite
2-3	Bastard Pyroxenite	Pyroxenite, Coarse Grained
2-3	M3	Mottled Anorthosite
3-7	M2	Spotted Anorthositic Norite
0.5	M1	Norite
1-1.5	Merensky Pyroxenite	Medium to Coarse grain Pyroxenite
0-2	Merensky Reef	Chromitite Layer – Pegmatoid
0.4	FW1	Spotted Anorthositic Norite
0.2	FW2	Cyclic Unit – Pyroxenite
3-5	FW3	Spotted Anorthositic Norite
0.1-0.3	FW4	Mottled Anorthosite
1-3	FW5	Spotted Anorthositic Norite
1-3	FW6	Cyclic Unit - Spotted and Mottled Anorthosite
1-3	FW7	Spotted Anorthositic Norite
0.8-1.2	FW8	Spotted Anorthosite
3-6	FW9	Mottled Anorthosite
3-5	FW10	Spotted Anorthositic Norite
12-15	FW11	Spotted Anorthosite
10-12	FW12	Mottled Anorthosite
5-7		UG2 Pyroxenite with Leader Chromitite Stringers
0,7	UG2 Reef	Chromitite
10-12	FW	UG2 Pegmatoid
5-7	FW13	Spotted Anorthositic Norite
6-8	FW14	Pyroxenite
1-3	UG1 (FW15)	Chromitite with anorthosite and pyroxenite lenses
>100	FW16	Anorthosite with chrome stringers

The most important platinum group element carriers in the Bushveld Complex are the Merensky Reef and the UG2 Reef, which occur parallel to the major layering as distinct layers usually less than 2 metres thick. The Merensky Reef lies at the base of the Main Zone, and consists of pyroxenite and pegmatoid units, while the UG2 Chromitite Seam is located between 20 to 300 metres below the Merensky Reef in the Critical Zone. Both carry economic concentrations of the platinum group elements, namely platinum, palladium, ruthenium, rhodium, iridium, and osmium as well as the base metals nickel, copper and cobalt, and finally gold, in minor concentrations. At Impala, the vertical separation between the two reefs increases from 60 metres in the north to 130 metres in the south. The general strike of both orebodies is north-northwest to south-southeast, dipping gently towards the east at about 10 degrees. Both reefs outcrop at surface along the western boundary of the Impala lease[4]. The geological succession for the Impala Platinum Mine is summarised in Table I.

3 THE PROBLEM

The off-reef development at Impala Mine is placed in the footwall of the Merensky and UG2 reefs, in rock masses that vary in quality from very poor to extremely good. Until recently, all support designs were based upon the 33 kN/m^2 resistance criterion that was applied universally without regard to the rock mass quality. This criterion had been determined from the 95th percentile on a cumulative fall of ground thickness curve based on fall of ground statistics on the mine.

Universal use of a single criterion such as this means that some excavations may be under-supported while others are over-supported. In addition to this, fall of ground

accidents continue to occur in off-reef development. It is therefore clear that a more scientific approach based on well-established rock engineering principles is necessary. First, an analysis of fall of ground accidents at Impala is presented, followed by an assessment of different rockmass evaluation schemes. This forms the background to the choice of the Q-Index, which is then applied to mine conditions in an underground study described in Section 4.2.

3.1 Fall of Ground Analysis

The fall of ground data used in the analysis covers a five-year period from 1992 to 1996, to ensure statistically meaningful results from a sufficiently large database. The data has been strictly limited to falls of ground in off-reef development to remove any bias that may arise if in-stope data were included. The following information was extracted from the accident reports for the analysis:

- reef type
- stope or off-reef development
- depth below surface
- distance from face
- excavation size
- origin of the fall of ground: face, hangingwall, sidewall, or footwall
- mechanism of fall of ground: buckling, shear, or dead weight
- size of fall of ground: small, medium, or large
- shape of fall block: dome, wedge, or scaling
- dimensions of fall of ground: height, width, length, area, volume, and weight
- rock type
- proximity of major geological features: faults, dykes, potholes or joints

boundaries of the fall of ground: joints, faults, dykes, or partings between layers such as a chromitite seam

The database has been visualised by preparing a number of cumulative frequency plots and pie charts, which appear in Figures 2 to 5 (Appendix I) and Figures 6 to 10 respectively (Appendix I). The 95th percentile dimensions, obtained from Figures 2 to 5 have been collected into Table II for easy reference.

Table II: List of 95th Percentile Variables for Falls of Ground in Off-reef Tunnels

95th Percentile Variable	Dimension
Thickness (m)	0.9
Area (m^2)	9.0
Width (m)	2.5
Length (m)	3.5
Volume (derived from above variables, m^3)	8.0
Mass (derived from above variables, tons)	25.0

The pie charts demonstrate aspects of falls of ground in off-reef tunnels e.g. location, shape, origin, rock type and boundaries that cannot be represented in cumulative frequency plots. These show that the majority of falls of ground are joint-bounded blocks originating in the hangingwall of the excavation. None of the statistics presented above suggest an insoluble problem. Instead, they suggest that appropriately designed local support would have been sufficient to prevent every occurrence listed.

3.2 Description of Rockmass Classification Schemes

A number of rockmass classification schemes have been applied in the field, and as far as is known, none of the authors have claimed that their classification can be applied generally. This survey shows that most classifications were designed to address a specific set of problems, thereby limiting their range of applicability.

3.2.1 Terzaghi's rock mass classification

Terzaghi's[5] 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. The scheme classifies rock as intact, stratified, moderately jointed, blocky and seamy, crushed (but chemically intact), squeezing and swelling. Where one or more of these characteristics dominate, Terzaghi proposed a particular steel set design. His classification scheme was applied to railroad tunnels in the Alps, and has been widely used in North America ever since the paper was published. It is generally not applicable to hard rock mines because steel sets are expensive and seldom used, but could still be applied to shallow workings in weathered rock where in-situ stress is not important.

3.2.2 Rock Quality Designation (RQD)

Deere[6] developed the RQD index to provide a quantitative estimate of rockmass quality from drill core logs. The RQD is defined as the sum of the lengths of intact core pieces longer than 100mm expressed as a percentage of the total length of core. The core should be at least 50mm in diameter and should be drilled with double barrel diamond drilling equipment. The RQD can be misleading in rockmasses where discontinuities in the rock are widely spaced and contain either infilling or weathered

material[7]. Such a situation may result in a blocky, unstable rockmass, despite a high measured RQD. The RQD is thus unsuitable as a rockmass evaluator on its own, but has proven to be valuable as a component of more sophisticated rockmass rating schemes (see Sections 3.2.4. and 3.2.9).

3.2.3 Rock Structure Rating (RSR)

The RSR[8] is a quantitative method for describing the quality of a rock mass and for selecting appropriate support based on the classification. Most of the case histories, used in the development of this classification, were for relatively small, shallow tunnels supported by means of steel sets. The RSR was the first to refer to shotcrete as a means of support. It is unsuitable for hard rock mines because it confines itself to shallow tunnels and steel sets or shotcrete support.

3.2.4 Stini and Lauffer classifications

Stini[9] proposed a rock mass classification for tunnels and discussed many of the adverse conditions, which can be encountered in tunneling. The original work is in German; therefore, it attracted little attention in the English-speaking world[10]. He emphasized the importance of structural defects in the rock mass and stressed the need to avoid tunnelling parallel to the strike of steeply dipping discontinuities. While both Terzaghi and Stini had discussed time-dependent instability in tunnels, it was Lauffer[11] 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.

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 stand-up time, i.e. the time available for the installation of support. Knowledge of stand-up time is useful in designing mining-support cycles for advancing tunnels, but provides no guide to a permanent support system. This classification may suffice as a part of a support design process for an underground mine, but would require further development for it to be universally applicable.

3.2.5 Checklist methodology for hazard identification in tunnels

The checklist approach was developed to assess the risk of falls of ground in a mine tunnel[12]. The hazard identification takes place in two phases: the user answers a series of questions which pertain to a given hazard or group of hazards and then, a ranking method is used to determine scores for each answer, thereby generating an overall hazard score with which to assess the relative importance of the hazard[12]. The disadvantage of this system is that the user tends to focus only on the points addressed by the questions, and if they do not highlight a particular hazard, it is likely to remain unnoticed. This method does not propose any support for a given hazard, but acts merely to alert the rock engineering practitioner to a certain group of hazards it is designed to identify. It is therefore unsuitable as a basis for objective support designs in mine tunnels.

3.2.6 Rockwall Condition Factor

The first classification scheme to be designed for use in deep level mine tunnels is that of Wiseman[13], who proposed evaluating the conditions of tunnels using in situ stress, and the uniaxial compressive strength of intact rock. Ryder[14] added the empirical rock condition factor *F* to Wiseman's[13] equation, and coined the name Rockwall Condition Factor (RCF) in 1987. The best available description of the RCF may be found in

COMRO[14], pages 88 and 89. 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. It is given by:

$$RCF = \frac{3\sigma_1 - \sigma_3}{F\sigma_c} \quad (1)$$

In equation (1), the major and minor principal stresses within the plane of the tunnel cross-section are σ_1 and σ_3 respectively, F is Ryder's[14] dimensionless empirical rock mass condition factor, and σ_C is the uniaxial compressive strength of the intact rock material.

Wiseman[13] logged some 20 kilometres of underground tunnels in the South African gold mining industry, and related the results obtained to the support systems installed in the tunnels. It therefore represents the first attempt to objectively determine support required given certain rockmass and stress conditions for underground mines. Although it is fairly widely used in the gold mines after Ryder's[14] addition of a rockmass condition factor, it is considered applicable to tunnels that are affected by both jointing and mining induced fractures, and is therefore not entirely appropriate to evaluate the potential for discontinuity-bounded falls of ground, which are commonest at Impala Platinum Mine.

3.2.7 The CSIR Geomechanics Classification for Jointed Rockmasses

Bieniawski[16] proposed a rockmass evaluation system that addresses some of the limitations outlined above by providing a more comprehensive description of the rockmass from which a quantitative result is obtained. This classification system is called the CSIR Geomechanics Classification for Jointed Rock Masses, or the Rock Mass Rating (RMR). It classifies a rock mass using the following six parameters:

uniaxial compressive strength of the rock material

Rock Quality Designation (RQD)

spacing of discontinuities

condition of discontinuities

orientation of discontinuities

groundwater conditions

When applying this classification system, the rock mass should be divided into structural regions for separate classification. Structural features such as faults or dykes, or changes in rock type should define the boundaries of a structural region.

The RMR has been applied to civil engineering projects at or near surface where the in situ stresses are generally not of any concern. This is the first scheme to consider geotechnical conditions in detail, but it is still generally unsuitable for underground mines because it omits the in situ stress from the classification.

3.2.8 ***Laubscher's Mining Modification to Bieniawski's RMR***

In 1973, DH Laubscher met with ZT Bieniawski to discuss the RMR, which was then being developed at the CSIR[17]. At the meeting, it became clear that the RMR was much better than that under development in Zimbabwe by Heslop and Laubscher[18]. Although Bieniawski's scheme was better, it did not possess the flexibility to adjust to different mining situations, and this led to the development of the Modified Rock Mass Classification for Jointed Rock Masses[17] based on Bieniawski's RMR. 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[19, 20, 21]. The modified classification scheme uses the RMR obtained from Bieniawski's scheme, and adjusts it to account for weathering, joint orientation, induced stress, blasting, and water according to the following equation:

$$MRMR = RMR \times adjustment\ factors \tag{2}$$

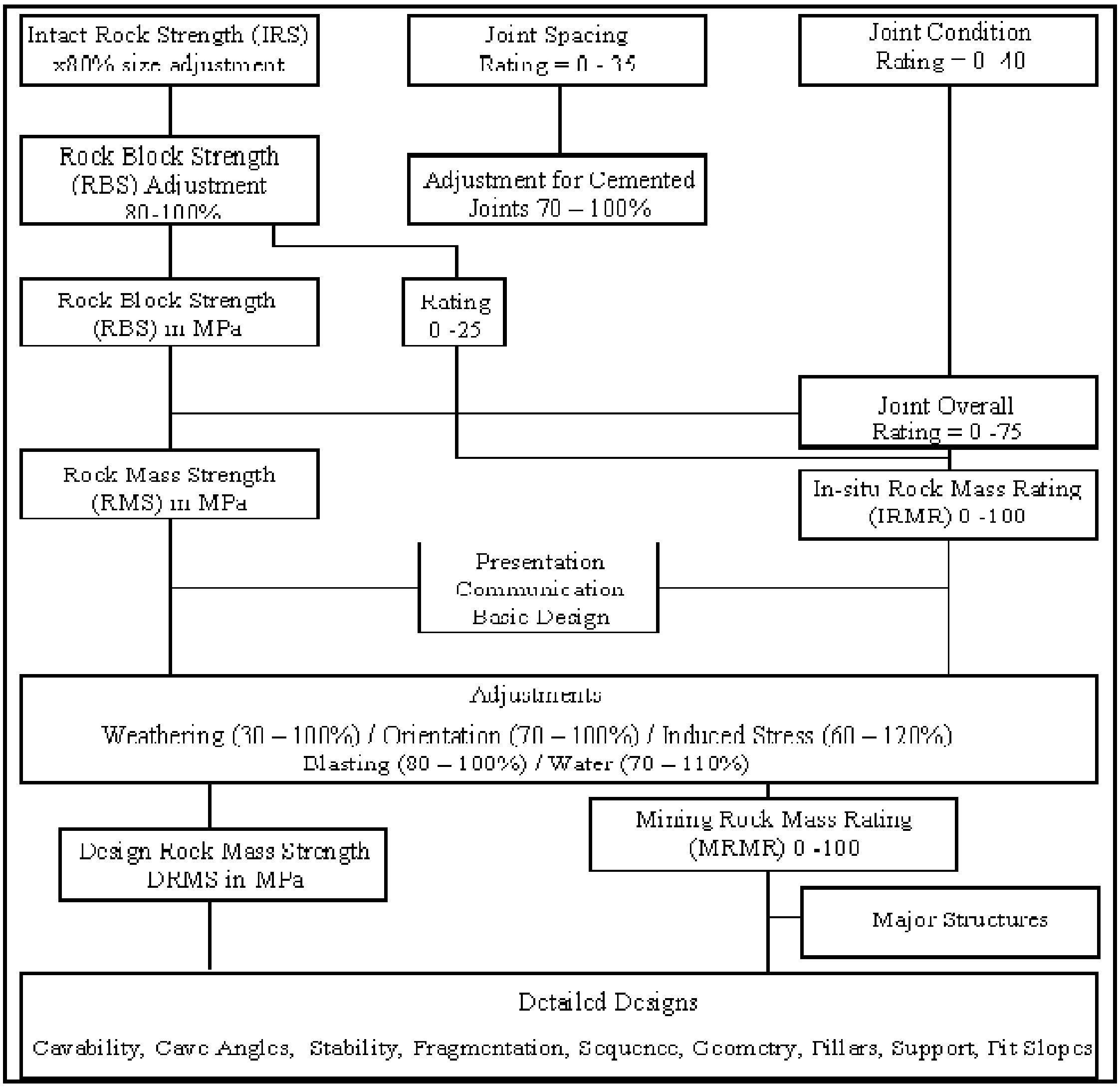

Figure 11: Flow Sheet of the Mining Rock Mass Rating (MRMR) Procedure with Recent Modifications (after Laubscher and Jakobec[17])

The classification has grown in complexity to the current procedure shown as a flowchart in Figure 11. This classification scheme is fine for design and detailed

design, but its complexity of the scheme as a whole makes it too tedious to apply to the support design procedure on a day-to-day basis. Furthermore, there is a considerable amount of subjectivity in the choice of adjustments for water, blasting, weathering, and so on. Jakubec and Laubscher[22] discuss this issue in their introduction and to repeat in their own words "…there is a growing concern in the mining community about their appropriateness and usefulness as a mine design tool. Some of the concerns are based on misunderstandings and misuse of the classification systems. It must be understood that a classification system can give the guidelines, but the geologist or engineer must interpret the finer details. The most important pitfall to avoid is the belief that the method is a rigorous analysis." These concerns are applicable to every scheme that is available: engineering judgment is of paramount importance when assessing rockmass conditions. The apparent complexity of the MRMR system makes it difficult to use underground daily. The authors decided to choose a scheme that could adequately address the variability of geotechnical conditions underground at Impala Platinum Mine, while at the same time being simple enough to make objective rockmass assessments daily.

3.2.9 Rock Tunneling Quality Index, Q

Barton et al.[1] 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:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \tag{3}$$

Where RQD is the Rock Quality designation

J_n is the joint set number

J_r is the joint roughness number

J_a is the joint alteration number

J_w is the joint water reduction factor

SRF is the stress reduction factor

The first quotient (RQD/J_n), represents the discontinuous structure of the rockmass, and can be considered a crude measure of the block or particle size. The second quotient (J_r/J_a) represents the roughness and frictional characteristics of the discontinuity surfaces and filling materials. This quotient is weighted in favor of rough, unaltered joints with opposing surfaces in direct contact with each other. When rock joints have thin clay mineral coatings and fillings, the strength of the rockmass as a whole is reduced significantly, which is taken into account by the second quotient. The third quotient (J_w/SRF) consists of two stress parameters. The SRF is a measure of loosening load in the case of an excavation through shear zones and clay bearing rock; rock stress in competent rock; and squeezing loads in plastic incompetent rocks. Together with J_w, which is a measure of water pressure in joints, the quotient forms a complex empirical stress factor describing the active stresses in a tunnel.

In summary, the rock tunneling quality Q is a measure of:

block size (RQD/J_n)

inter-block shear strength (J_r/J_a)

active stress (J_w/SRF).

These three factors are important in assessing mine tunnel stability, and together with the support guidelines provided by the system, provide a simple yet flexible scheme to take geotechnical factors into account in tunnel support designs.

3.3 Choice of Rockmass Rating System Applicable to Impala Platinum Mine

The tunneling quality index Q proposed by Barton et al[1] and the MRMR classification scheme developed and modified by Laubscher[17] are probably the two most commonly used rock mass classifications in mining rock mechanics. Both are designed to assess factors, which influence the stability of underground excavations. The similarities between the MRMR and the Q-Index 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 MRMR uses the uniaxial compressive strength of intact rock directly while the Q-Index 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 consider in-situ stress, although the MRMR approaches stress more quantitatively than the Q-Index. Like the MRMR, the Q-Index has evolved over the years; for example, it has been extended to estimate tunnel boring machine performance in rockmasses[23]. Unlike the MRMR, which has been generalised for all mining, the Q-Index has remained focused on tunneling.

Since both systems consider the factors necessary to assess underground tunnel stability, neither supercedes the other as a suitable system for application to off-reef

tunnels at Impala Platinum Mine. The choice of the Q Tunneling Quality Index was therefore made for the following reasons:

- its simplicity as a measure of block size, inter-block shear strength, and active stress, all of which are critical factors in the fall of ground problem, as determined from the fall of ground analysis;
- the Q-Index is easy to use on a daily basis underground;
- the simple relationship provided in the scheme to decide whether support is required or not.

The Q-Index has since been applied in two case studies at Impala Platinum Mine.

4 CASE STUDIES

Two excavations were chosen for the case studies, namely 10 Level Crosscut West and 23 Level Conveyor Decline. The first excavation is in a fair to extremely good rockmass, the second in a poor to very poor rockmass. The range of the two combined covers all conditions likely to be encountered at Impala Platinum Mine.

The 10 Level Crosscut is located 640 metres below surface at No. 9-Shaft where both the Merensky and UG2 Reefs are being exploited. It lies above both reefs at the shaft, crosscutting down through the strata in a westerly direction until it intersects the Merensky Reef 120 m west of the shaft, and the UG2 Reef about 640 m west of the shaft. Mining has been restricted from taking place either directly above or below the crosscut, which has limited mining induced stress changes in the crosscut itself to small values. The vertical virgin stress along the entire length of the crosscut is estimated to be approximately 20 MPa, and mining induced stress changes remain

insignificant. The crosscut intersects the entire stratigraphic succession given in Table I.

The average width of the crosscut is 3 m with sections that widen out to 5.2 metres in places. Support was installed in the crosscut at the wider sections and spot bolts were installed in one other area. The tunnel was developed in 1981, and has remained open without any significant falls of ground along its entire length ever since. The Q-Index was estimated for 10 m intervals over a distance of 770 metres in this tunnel.

The 23 Level Conveyor Decline is located 1058 metres below surface at No. 14-Shaft, which is currently mining the Merensky Reef. The vertical virgin stress at No. 14-shaft is approximately 32.3 MPa at 23 Level. The decline is currently being developed in Footwall 16 anorthosite below the UG2 Reef, at an average width of 5.6 metres span and an average 30 centimetres overbreak.

The support installed in the decline consists of full column grouted, 3 metres long, 16 millimetre diameter shepherd crooks placed one metre apart on dip and strike. The rock mass shows hangingwall and sidewall instability, with falls of ground up to two metres high taking place almost immediately after the blast.

4.1 Methodology Used to Obtain the Q Tunneling Quality Index

The Q-Index is used to assess the stability of an excavation, and to provide guidelines for the excavation support. The methodology used to estimate the input parameters for the Q-Index is discussed briefly below.

4.1.1 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. Three mutually perpendicular scan lines were set up in the tunnel, the first parallel to the axis of the tunnel, the second spanning the tunnel width at the midpoint of the axial scan line, and the third vertical, also located at the midpoint of the axial scan line. The scans were repeated every ten metres along the axis of the tunnel. The scan line length along the axis of the tunnel was chosen to be ten metres in order to capture joint sets with spacing of up to ten metres.

Joint densities (number of joints per linear metre) were found by counting the number of joints intersecting the scan line and dividing by its length. In this way a linear joint density for the tunnel axis direction *D*, for its span *S*, and its height, *V* were obtained and inserted into equations (4) and (5) below:

$$J_D = D + S + V \tag{4}$$

$$RQD = 115 - 3.3 J_D \tag{5}$$

If the RQD obtained from Equation (5) is less than 10%, then a value of 10% is entered into Equation (2). For values greater than 100% obtained using Equation (5), a value of 100% is used for the RQD in equation (2).

4.1.2 Estimating the other variables

Estimates for J_n, J_r, J_a, J_w, and the SRF are obtained as described by Barton et al.[1].

Where the excavation is expected to be subject to a stress change, a stress analysis is carried out using MINSIM-W[24] to help estimate a stress - rock strength relationship.

4.2 Q-Index Analysis of 10 Level Crosscut and 23 Level Conveyor Decline

A total of 89 tunneling quality index measurements were made, 77 in 10 Level Crosscut and 12 in 23 Level Conveyor Decline. This represents 890 metres of tunnel covered using this method. Figure 12 provides statistics of the spread of Q-Index values obtained in the two excavations.

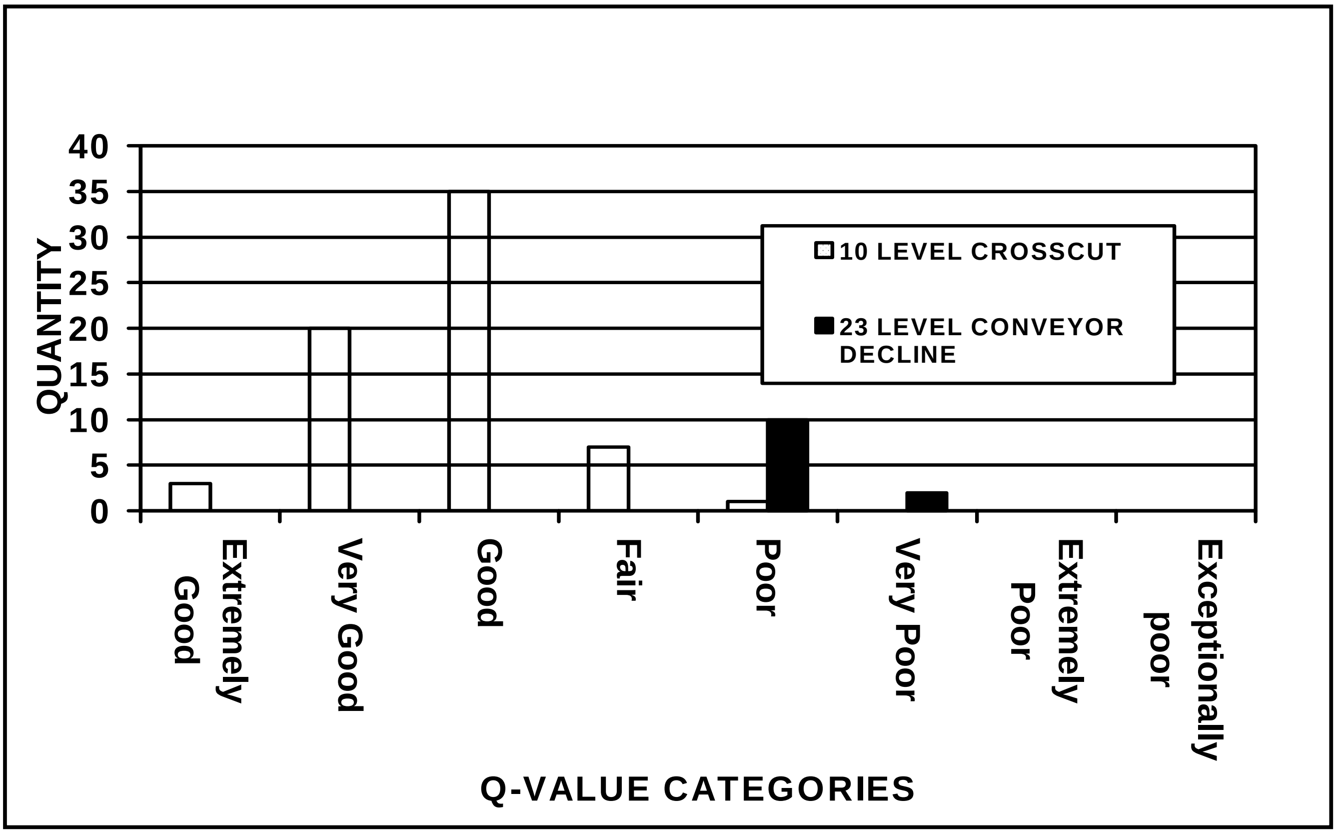

Figure 12: Distribution of Q-Index Values Obtained from 10 Level Crosscut and 23 Level Conveyor Decline

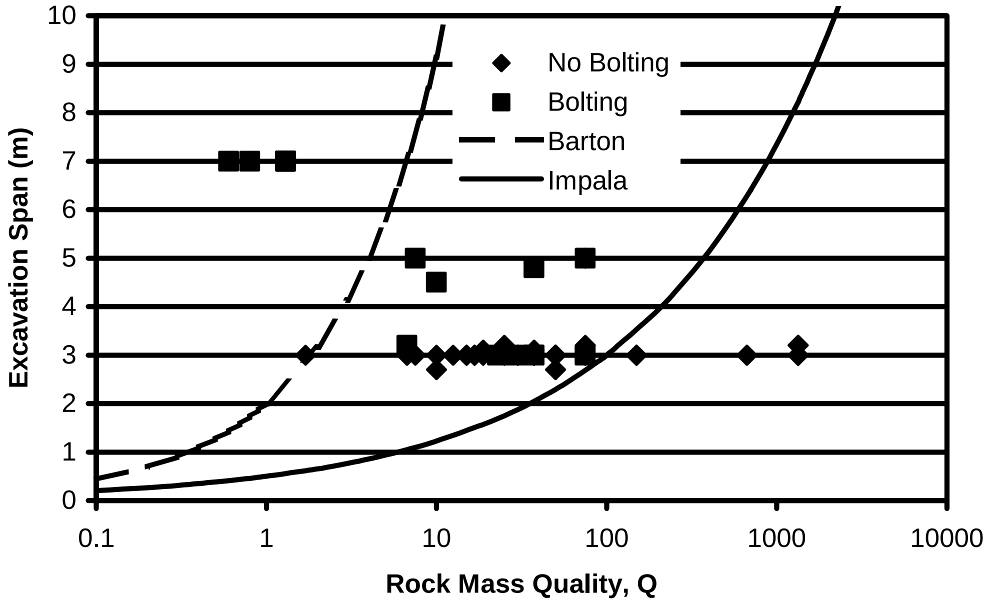

Figure 13: Scatter plot of Equivalent Dimension vs. Q-Index for an Excavation Support Ratio of 1.6 (Permanent Mine Openings)

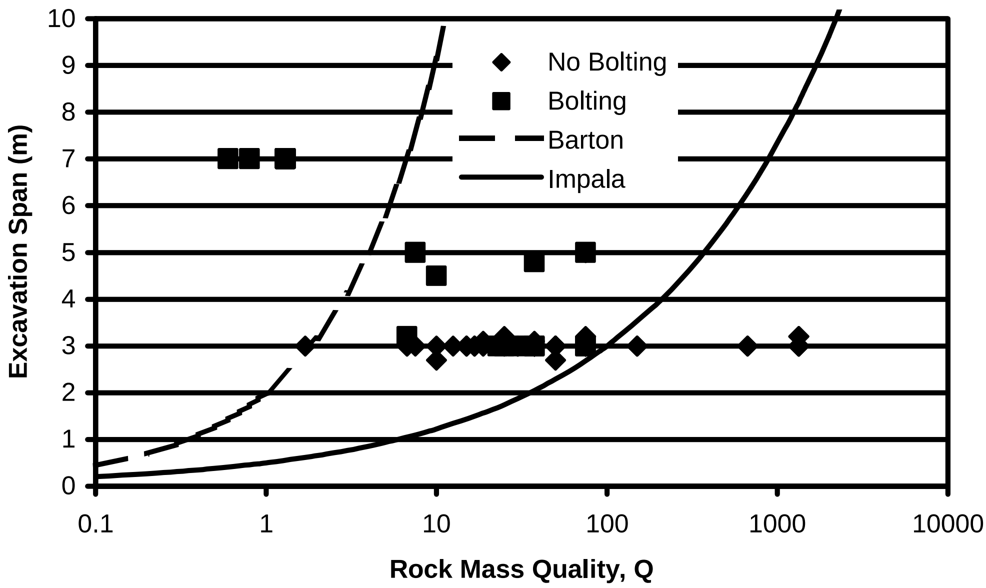

Figure 14: Scatter Plot of Excavation Span vs. Q-Index

Figures 13 and 14 are log-linear scatter plots of Equivalent Dimension (D_e) versus Q-Index and Excavation Span versus Q-Index respectively. Two populations of points appear in the plots, the diamond symbols for unsupported sections, and the squares for supported sections. The type of support is ignored in these plots, so that excavations with support ranging from spot bolts to heavy support using several types of support elements are considered supported. In both plots, Barton's[1] "no support" line straddles both unsupported and supported excavation data, which suggests that the current decision line position is not suitable for Impala. If this line were used as a guideline, there would be occasions when no support would be installed in an excavation that needs support, which could lead to a fall of ground. The "no support" line only straddled data from 10 Level Crosscut, which is located in a good to very good rockmass. All data for the 23 Level Conveyor Decline plotted well to the left of the "no support" line, since this excavation yielded relatively low Q-Index values.

Barton's "no support" line is given by the following formula for the Equivalent Dimension – Q-Index relationship:

$$D_e = 2Q^{0.4} \tag{6}$$

D_e is the Equivalent Dimension, defined by excavation height or excavation span divided by the Excavation Support Ratio, which is given by Barton[1] as 1.6 for permanent mine openings. In considering the relationship for excavation span, Barton[24] considered both natural and man-made excavations, from which he deduced that:

$$Span = 2Q^{0.66} \tag{7}$$

In order to estimate where the "no support" line should plot for Impala data, it is assumed that conditions demanded support wherever it was installed, and that unsupported sections really do not need support. Since geotechnical parameters have not until recently been taken into account in the decision to support or not, it is also assumed that the relevant supervisor decided where support was necessary on the strength of a visual check. This decision was probably based on a fall of ground, either shortly after the development blast, or sometime later in the excavation's life.

There is no evidence to suggest that the relationship should not be the power law already given by Barton[1], hence this will be retained in the following general form:

$$Span = a \times ESR \times Q^{b} \tag{8}$$

Where it remains to determine *a* and *b* to suit the data obtained for Impala, assuming in all cases that ESR = 1.6 for permanent mine openings. Two points on the "no support" line need to be defined in order to determine the parameters for the relationship. The smallest dimension measured in the fall of ground data is 0.5 m (see Figures 3, 4, and 5). It is therefore assumed that a fall of ground this size could occur in an excavation with a span of 0.5 m. Secondly, it appears from Figures 13 and 14 that all excavations with Q-Index values of 1.0 or less require support. The first point for the "no support" line in Figure 14 would thus be (1.0, 0.5). It also appears from the data in Figures 13 and 14 that excavations with Q-Index values of 100 or more never need support. Since this was only ever observed in 10 Level Crosscut, a span of 3 m is assumed, and the second point is thus (100.0, 3.0).

Using the two points to solve for *a* and *b* in equation (8) and assuming the ESR = 1.6, we obtain:

$$D_{E} = 0.3125 Q^{0.3891} \tag{9}$$

$$Span = 0.3125 \times ESR \times Q^{0.3891} \tag{10}$$

These lines appear in Figures 13 and 14, and it can be seen that they separate the unsupported data from the supported data more effectively than Barton's[1,24] lines. The result of the changed support lines is that tunnels of smaller span will require support than that required by the original analyses of Barton[1] and Barton[24]. The reasons for this smaller span may lie in the hardness and brittleness of the rock material, together with the fact that it is blasted. If the tunnels were mechanically excavated, it is expected that larger spans would be stable in the same conditions. This indicates the site-specific nature of the Q Tunnelling Quality Index, and how it needs to be modified for the environment in which it is being used.

4.3 Bolt length design for permanent mine openings

The Q-Index analysis first provides a guideline on whether the tunnel needs support or not, using the revised "no support" line. If support is required, then the support design must consider the bolt type in use, the 95th Percentile fallout height, and fall of ground dimensions. An example design, which results in a bolt length of 1.2 metres spaced 1 m apart, appears in Appendix 1. Note that this design should not be applied everywhere on the mine. Rather, geotechnical conditions at the site should be reckoned into the support design. A comparable design using Barton's[1] formula follows in Appendix 1. There is an opportunity to modify this formula as well for conditions at Impala Platinum Mine.

The work in implementing the use of geotechnical parameters in tunnel support design is still not complete. Geotechnical mapping should be carried out far more widely on the mine, and should include excavations with larger spans than those measured in this paper. This will help to fix the “no support” line with better confidence. Short-life excavations should also be included in the mapping. With this increased database it will be possible to determine a set of excavation support designs for all geotechnical conditions and all excavation spans at Impala, so that support design could be simplified to a reference to a standard support table for the mine based on the Q-Index. With time it will be possible to refine the support designs to increase their cost-effectiveness, while at the same time eliminating falls of ground in off-reef excavations.

5 CONCLUSION

Support designs for off-reef excavations at Impala previously met the 33-kN/m^2 support resistance criterion without any consideration of the geotechnical conditions. This approach is no longer acceptable because it does not prevent fall of ground accidents. There are also many instances where such support may not be necessary.

A fall of ground accident analysis for off-reef tunnels at Impala revealed that:

- 95% of all falls of ground were 0.9 metres thick or less;
- nearly all were controlled by discontinuities in the rockmass;
- large falls of ground are rare;
- conventional support designs already in use could have prevented all falls of ground if the circumstances leading to them had been identified beforehand.

The Q Tunnelling Quality Index is sufficiently flexible to identify the potential fall of ground hazard if it is properly used at Impala, and it can be used as a guideline to more cost-effective support. Since testing the viability of the Q-Index as a rockmass classification scheme underground, it has shown that:

- the Q-Index addresses all the critical parameters that control falls of ground in tunnels at Impala;
- it is easy to apply underground, and geotechnical data gathering using the system is quick;
- Barton's[1] "no support" line needs to be modified for Impala;
- the changes to the parameters for the "no support" line represent a first estimate, which should be refined as more geotechnical data is gathered;
- routine mapping of the tunnels should reveal all potential fall of ground hazards;
- Impala Platinum Mine should be able to take remedial action against potential falls of ground before they occur.

Implementing effective support designs based on geotechnical investigations should help solve the fall of ground problem in off-reef excavations, while underground support designs for off-reef excavations could be simplified once a comprehensive geotechnical database has been built up.

6 ACKNOWLEDGEMENTS

The authors would like to thank the management of Impala Platinum Limited for permission to publish this paper. A special mention in this regard is expressed to P.J. Anderson and N.F. Fernandes for providing the opportunity to gather information as part of this MSc thesis.

7 REFERENCES

1. Barton N, Lien R, and Lunde J (1974): Engineering classification of rock masses for the design of tunnel support. *Rock Mechanics*, Vol. 6, No. 4, 1974, pp. 189-236.
2. Jakubec J and Laubscher DH (2000): The MRMR Rock Mass Rating Classification System in Mining Practice. *Massmin 2000,* Brisbane, Queensland, 29 October – 2 November 2000, pp. 413-421.
3. Vermaak CF (1995): *The Platinum Group Metals: A Global Perspective*. Mintek, Randburg, South Africa, 1995.
4. Mellowship P (1996): *Impala Platinum Limited Geology - Training Module.* Geology Department Impala Platinum Limited, 1996.
5. Terzaghi K (1946): Rock defects and loads on tunnel supports. In: *Rock Tunneling with Steel Supports*, Eds. R. V. Procter and T.L. White Vol. 1, pp. 17-99. Youngstown, OH: Commercial Shearing and Stamping Company, 1946.
6. Deere DU (1964): Technical description of rock cores for engineering purposes. *Rock Mechanics and Engineering Geology* Vol. 1 No. 1, pp. 17-22, 1964.

7. Merritt AH (1972): Geologic prediction for underground excavations. *Procs. First North American Rapid Excavation and Tunnelling Conference,* AIME, New York, 1972, pp. 115-132.

8. Wickham GE, Tiedeman HR, and Skinner EH (1972): Support determination based on geological predictions. In Proc. *North American Rapid Excavation and Tunneling Conference*, Chicago, 1972, pp. 43-64. New York: Soc. Min. Engrs, Am. Inst. Min. Metall. Petrom. Engrs.

9. Stini I (1950): *Tunnelbaugeologie*. Vienna: Springer Verlag, 1950, 366p.

10. Hoek E and Brown ET (1980): *Underground Excavations in Rock*. The Institution of Mining and Metallurgy, London, England, 1980.

11. Lauffer H (1958): Gebirgsklassifiziering fur den spollenbau. *Geologie en Bauwesen,* Vol. 24, No. 1, 1958, pp. 46-51.

12. SIMRAC (1998): *Hazard identification for Rock Engineering*. Final Project Report, Project GAP 339, Department of Minerals and Energy, Johannesburg, 1998.

13. Wiseman N (1979): *A Study of the Factors Affecting the Design and Support of Gold Mine Tunnels*. Research Report No. 50/78, Chamber of Mines Research Organization, Johannesburg, 1979.

14. Ryder JA (2001): Personal communication by telephone, 2nd November 2001.

15. COMRO (1988): *An Industry Guide to Methods of Ameliorating the Hazards of Rockfalls and Rockbursts*. Research Organization of the Chamber of Mines of South Africa (COMRO), Johannesburg, 1988.

16. Bieniawski ZT (1978): Rock mass classification in rock engineering. In *Proceedings of the Symposium on Exploration for Rock Engineering,* (ed. Z.T. Bieniawski) Vol. 1, pp. 97-106. Johannesburg, 1978.

17. Laubscher DH and Jakubec J (2000): The MRMR Rock Mass Classification for Jointed Rock Masses. Chapter 57, pp 475-481, Society of Mining Engineers, AIME, 2000.

18. Heslop TG (1973): Internal company report.

19. Laubscher DH (1977): Geomechanics classification of jointed rock masses-mining applications. *Trans Instn. Min. Metall.* 1977, Vol. 86, pp. A1-8.

20. Laubscher DH and Taylor HW (1978): The importance of geomechanics classification of jointed rock masses in mining operations. In: *Proceedings of the Symposium on Exploration for Rock Engineering* (ed. Z.T. Bieniawski) Vol. 1, pp. 119-128. Cape Town. Balkema, 1978.

21. Laubscher DM and Page CH (1990): The design of rock Support in high stress or weak rock environments. *Proc. 92nd Can. Inst. Min. Metall. AGM*, Ottawa, Paper No. 91, 1990.

22. Jakubec J and Laubscher DH (2000): The MRMR Rock Mass Rating System in Mining Practice. Proceedings of *MassMin 2000*, Brisbane, Queensland, Australia 29 October – 2 November 2000, pp.413-421.

23. Barton N (2000): *TBM Tunnelling in Jointed and Faulted Rock*. Rotterdam: AA Balkema, 173p.

24. Barton N (1976): Recent experiences with the Q-system of tunnel support design. *Proceedings of the Symposium on Exploration for Rock Engineering*, Johannesburg, Vol. 1, pp. 107-117.

APPENDIX I

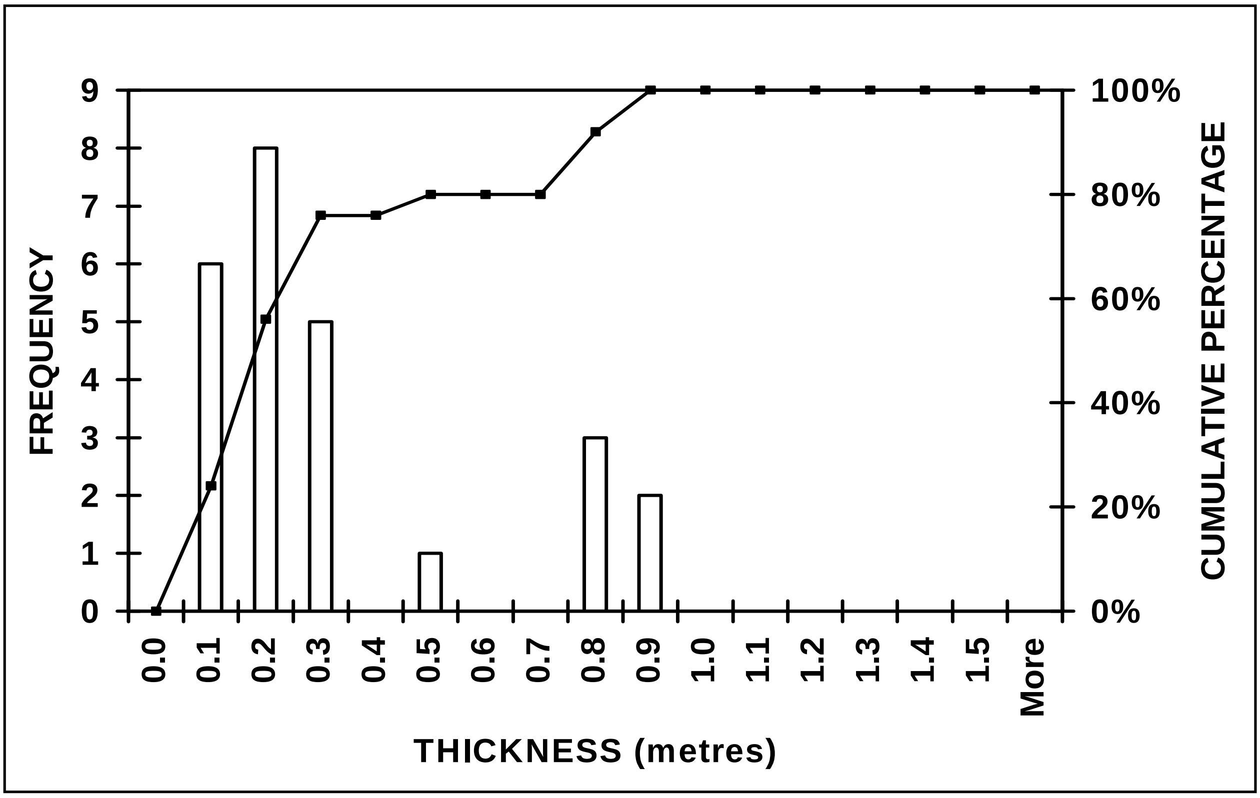

Figure 2: Cumulative Frequency Plot of Maximum Thickness of Falls of Ground

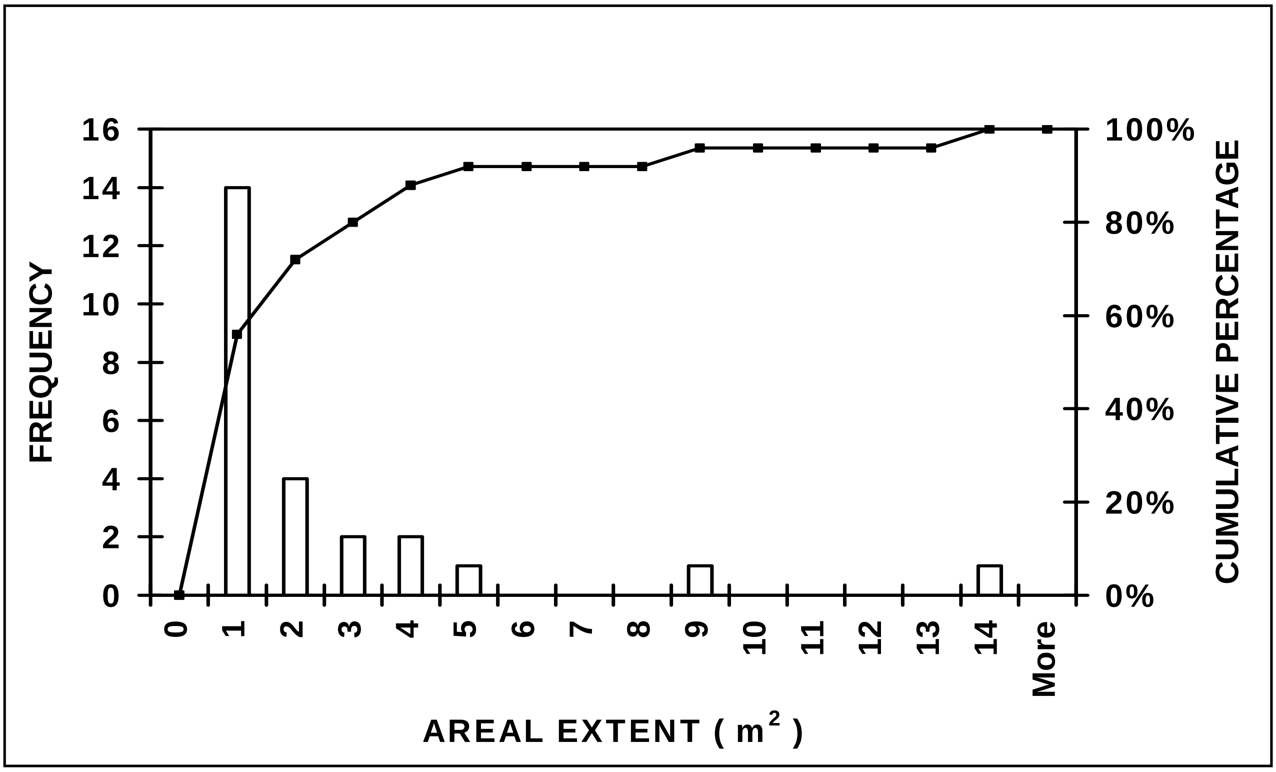

Figure 3: Cumulative Frequency Plot of Areal extent of Falls of Ground

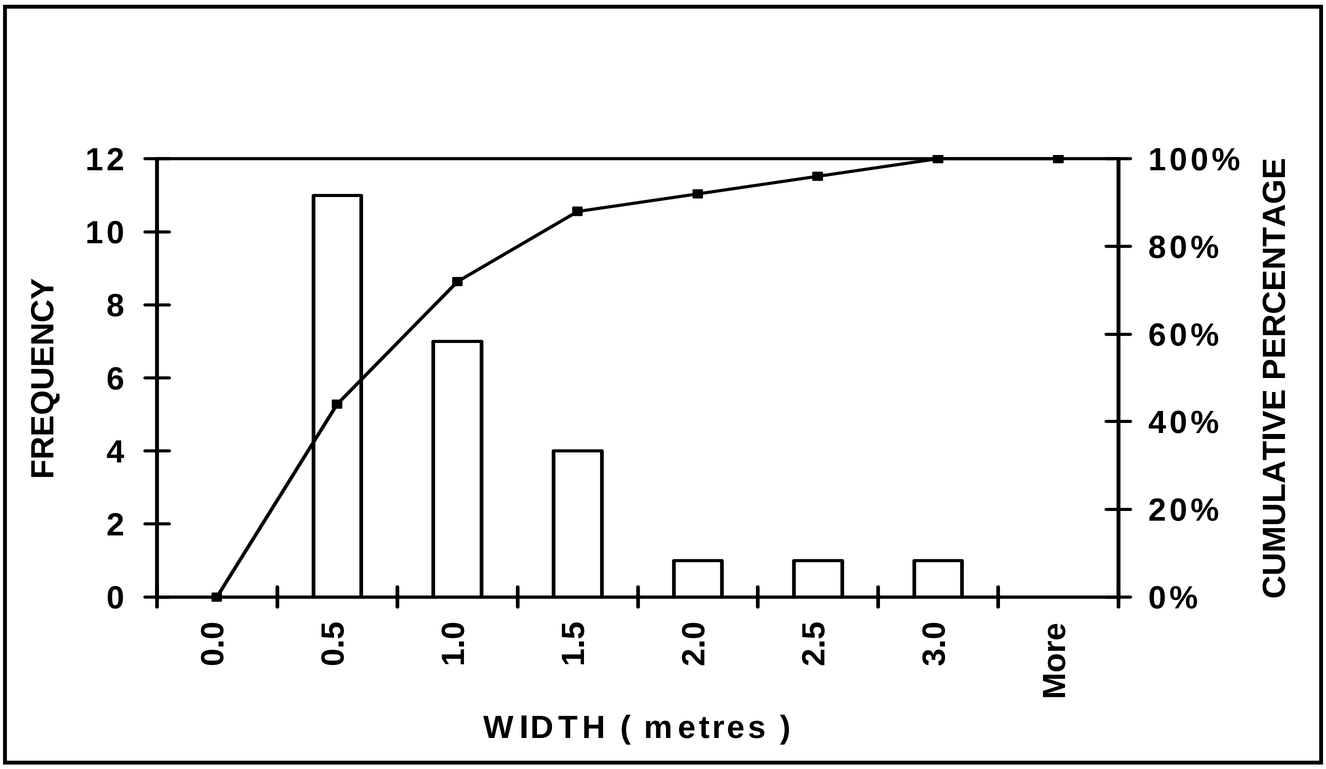

Figure 4: Cumulative Frequency Plot of Width of Falls of Ground

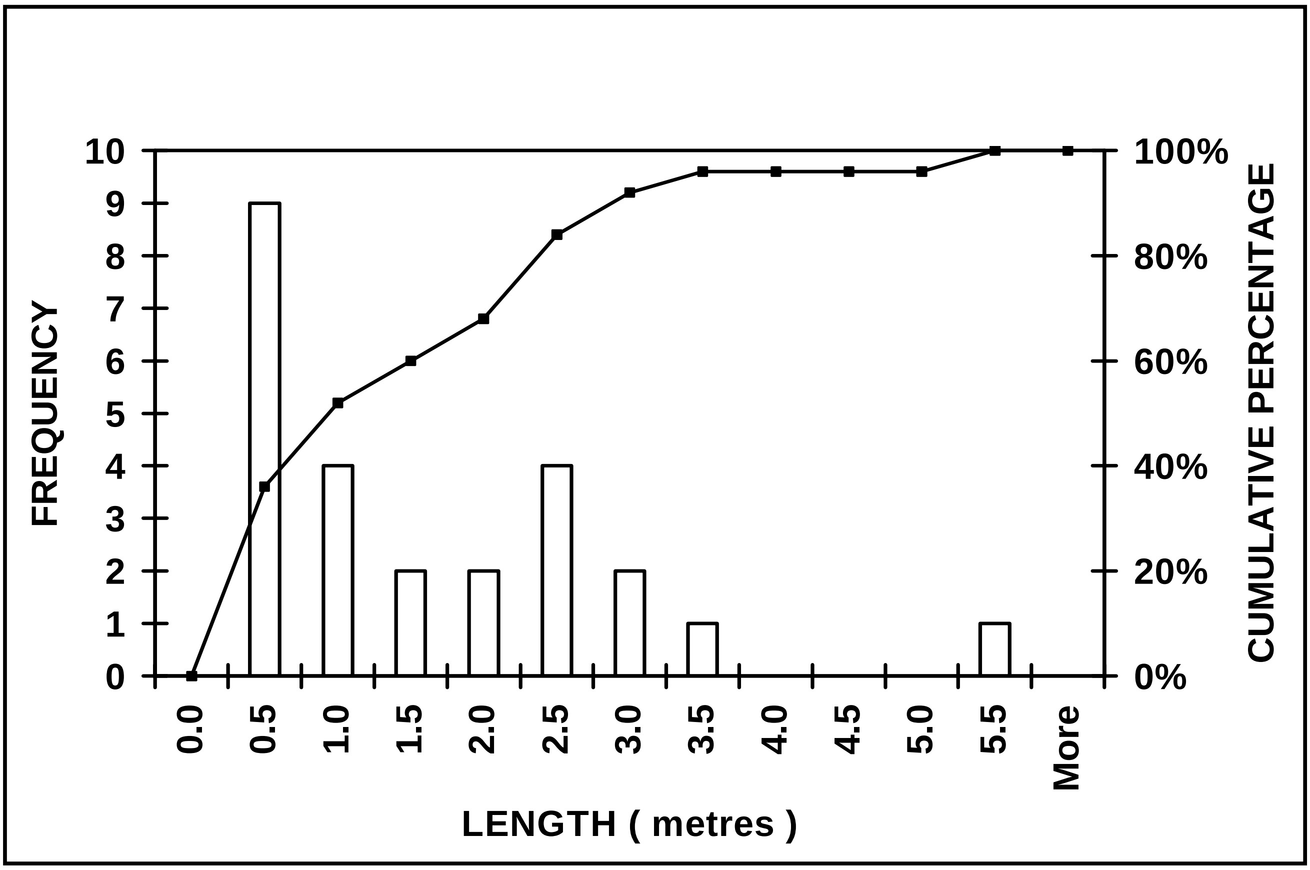

Figure 5: Cumulative Frequency Plot of Length of Falls of Ground

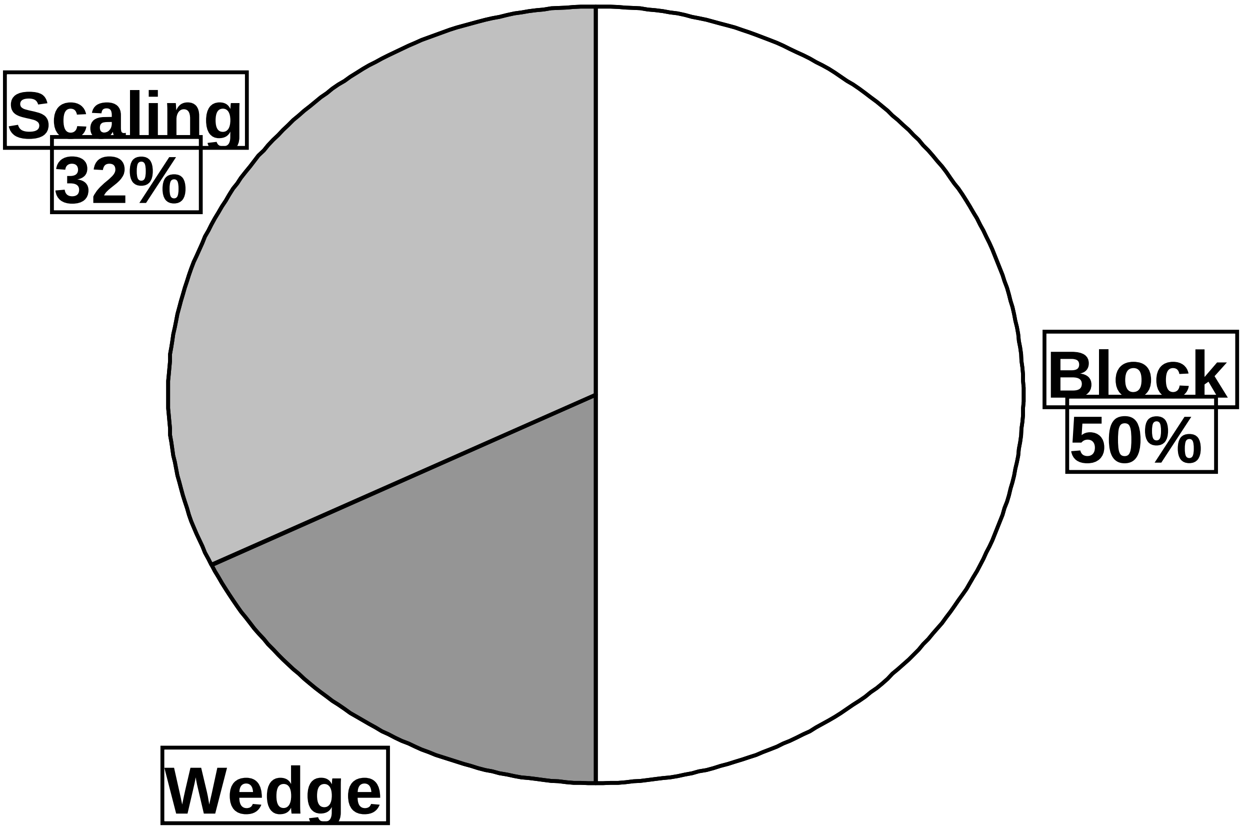

Figure 6: Shape of Falls of Ground

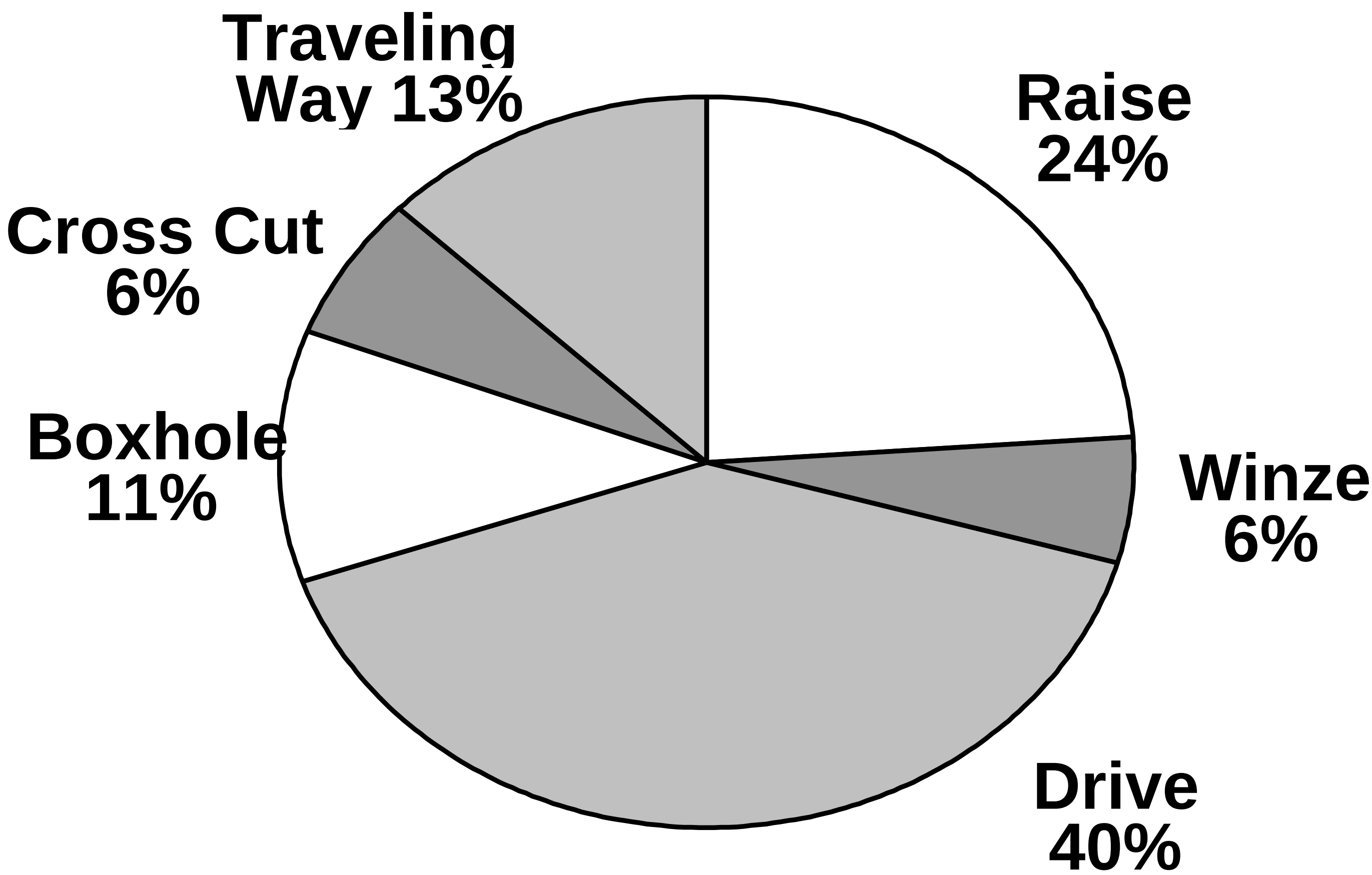

Figure 7: Location of Falls of Ground

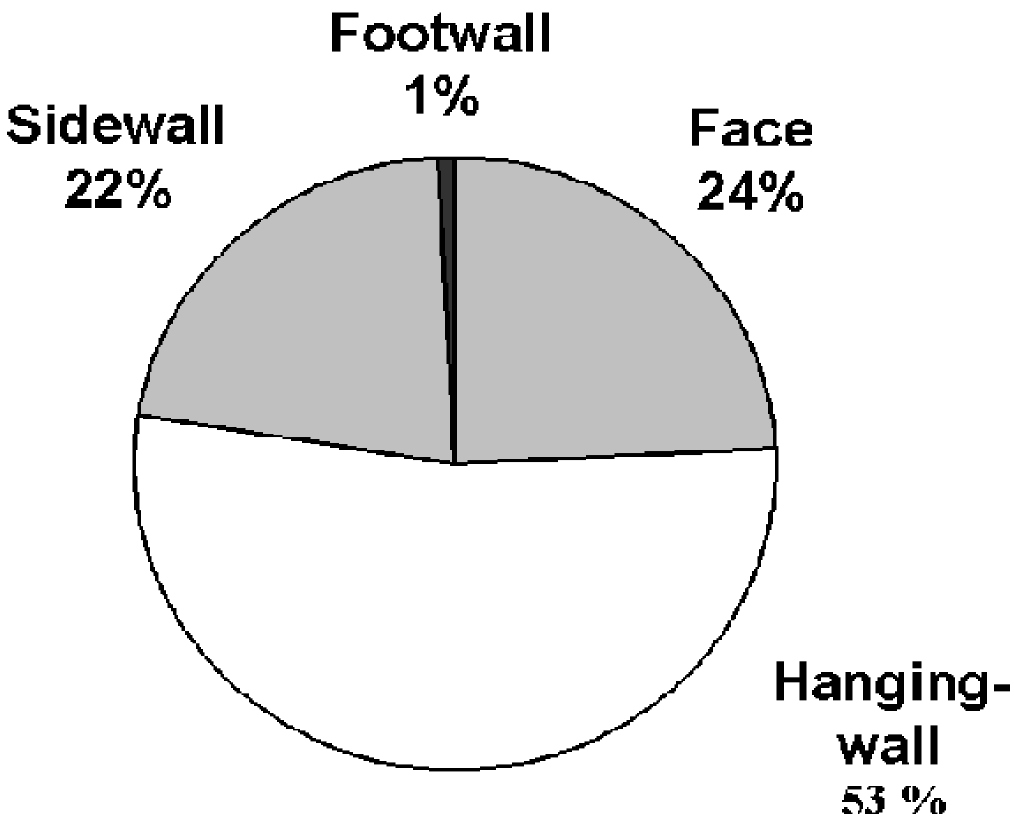

Figure 8: Origin of Falls of Ground

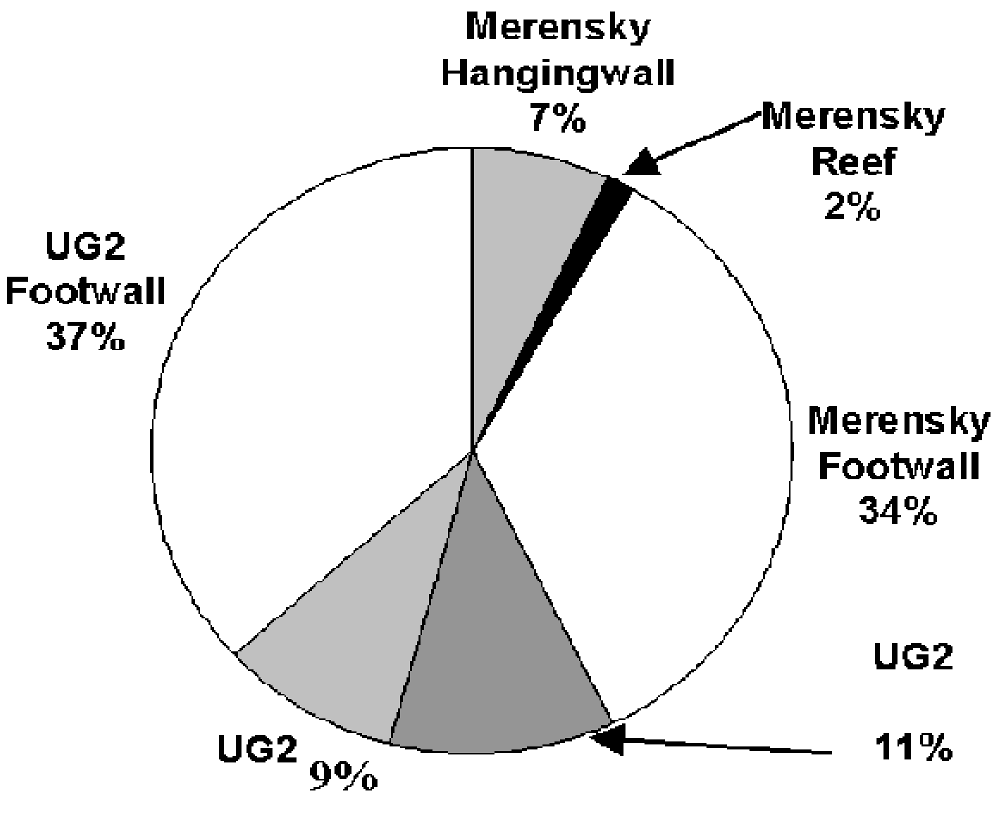

Figure 9: Rock Type Involved in Falls of Ground

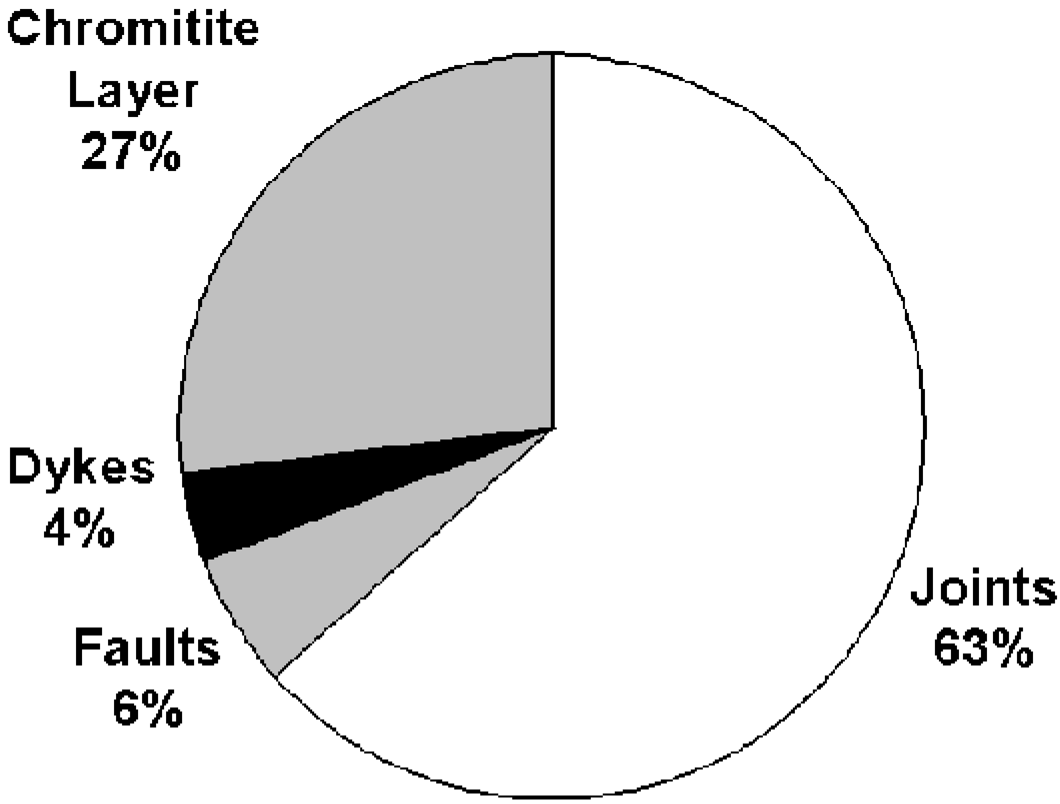

Figure 10: Boundaries of Falls of Ground

APPENDIX II

CALCULATION OF REQUIRED BOLT LENGTH FOR 3 METRE SPAN TUNNELS

Fall of ground data suggest that 56% of the falls are 1 m^2 in area or less, 44% of falls are 0.5 m or less wide, and 36% are 0.5 m or less long. To prevent these from occurring, a maximum bolt spacing of 1 by 1 m is necessary. This does not mean that bolt spacings everywhere on the mine should be 1 x 1 metre. Geotechnical mapping using the Q-Index will be far more effective in deducing maximum bolt spacings than fall of ground accident data, and should be used for this purpose wherever possible.

Each bolt must suspend a potential fall of ground. Hence, it should be bonded to solid ground above the 95th percentile potential fall-out height of 0.9 m. Assume that

grouted rebars will be used. If the 95th percentile fall of ground thickness is 0.9 m, then the rebar must be long enough to suspend 0.9 m of rock. The volume of rock to be supported by each rebar is thus 0.9 m^3, amounting to a deadweight of 27.4 kN, if the rock density is assumed 3000 kg/m^3. The strength of a grout bond with the support element is given by:

$$bond \ \ strength = \frac{pull - out \ \ force}{\pi \times rebar \ \ diameter \times bond \ \ length} \quad (11)$$

Underground pull tests have revealed an average 4 MPa bond strength for cement grouts that have cured for 1 hour. Rearranging equation (12):

$$bond \ \ length = \frac{pull - out \ \ force}{\pi \times rebar \ \ diameter \times bond \ \ strength} \quad (12)$$

Assuming a 16 mm rebar diameter, a bond length of 0.14 m is required. Using a factor of safety of 2.0, the minimum bond length should be 0.28 m. The rebar length should thus be 0.9 + 0.28 = 1.18 metres, say 1.2 metres.

Comparing this with a typical bolt design from Barton et al.[1]:

$$L = \frac{2.0 + 0.15B}{ESR} \quad (13)$$

Where L is bolt length, B is the excavation width, and the ESR is the Excavation Support Ratio - a value related to the intended use of the excavation and the degree of security. For permanent mine openings assume the ESR = 1.6. For Impala, a

permanent 3.0 m wide excavation will thus require a bolt 1.53 m long. This design length is some 20% longer than the design using the fall of ground data and measured pull-out forces. With time, there will be sufficient geotechnical data to change this formula to suit conditions at Impala Platinum Mine.