CHAPTER 1

INTRODUCTION

1.1 Project Location

The Kansanshi Copper and Gold Mine (Plc), is owned by the First Quantum Minerals Limited (FQM) and ZCCM-IH in the ratios 80% and 20% respectively. Kansanshi Mine is located approximately 15 km north of Solwezi in North Western Province of Zambia and 18km south of Democratic Republic of Congo border. The Copperbelt town Chingola is approximately 180km southeast of the Mine. Figure 1.1 indicates Kansanshi Mine location.

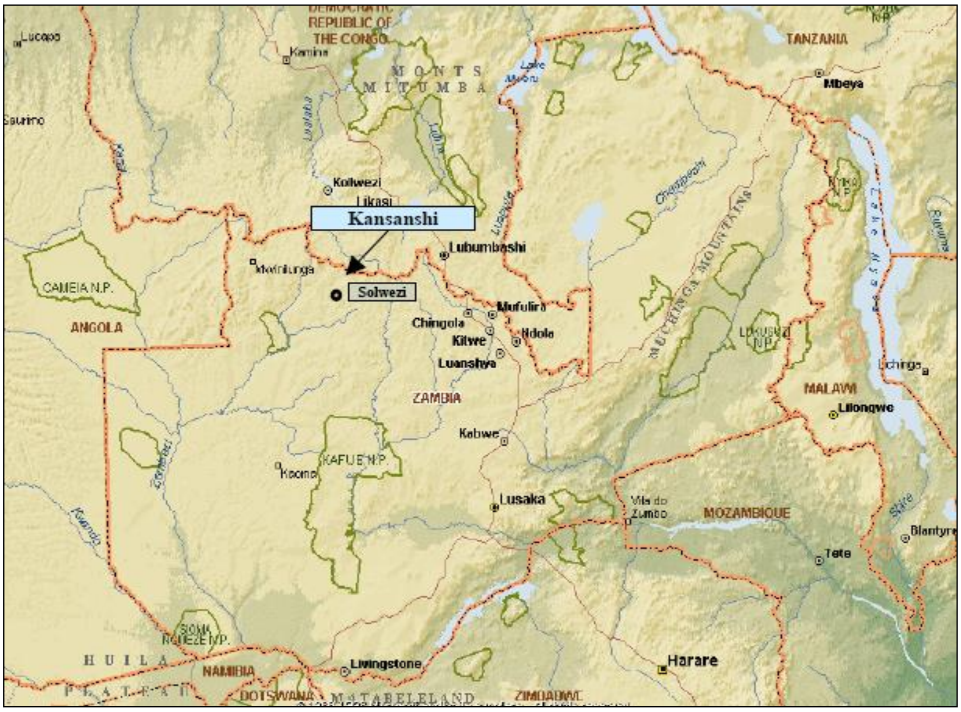

Figure 1.1 Kansanshi Mine Locations

There are currently two open pits in operation; that is the Main and North West. In the future, both pits will join to form a single large open pit. It is expected that the North of Main Pit will reach a depth of 295 metres by December 2031 and the North West Pit will reach a final depth of 160 metres.

1.2 Project Background

The Kansanshi Mine is one of the oldest known Copper and Gold deposit in Zambia, with evidence of direct copper smelting dating back to the fourth century. Since the rediscovery of these ancient workings in 1899, the deposit has been mined intermittently for recovery of high grade Copper ore.

From 1903 until 1914, Copper was recovered by underground mining of high-grade veins. Mining activities were terminated with the onset of World War one, resumed in 1927, but the operations were shut down again in 1932 due to the worldwide economic depression. In 1952, further exploration and mine development commenced, with production resuming in 1956. Rich oxide ore from upper levels of the underground mine was shipped to the Nkana Mine on the Copperbelt for direct smelting until October 1957. Sulphide concentrate were also produced on site from rich vein ore at lower mine level through a small concentrator.

In 1969, ZCCM approved development of an open pit mine at Kansanshi to treat high grade oxide ores in a leach plant. This activity continued through April 1986 when operations ceased again due to low copper prices and other economic reasons.

During 1988, ZCCM constructed a small sulphide flotation concentrator at site with a capacity of approximately 200 tonnes per day and commenced open pit mining activities. Concentrate from the facility, contained up to 15 grade/tonnage of copper, was transported to the Copperbelt for smelting. In January 1997, as part of the privatization programme for ZCCM, Cyprus Amax Minerals Company (Cyprus) entered into an agreement with ZCCM and the Government of the Republic of Zambia to secure majority ownership of surface leases and selected assets associated with the Kansanshi project.

In May 1997, Cyprus initiated geological investigations and metallurgical test work activities, aimed at developing reserves capable of supporting a major mining facility.
Cyprus became part of Phelps Dodge during the undertaking of a prefeasibility study (PFS). This PFS examined Copper production of 124 000 tonnes per annually over a 24

year life span from mining of a total of 267 metric tonnes of ore. This comprised 101 metric tonnes of mixed ores requiring flotation and leaching, and 166 metric tonnes of sulphide ores requiring flotation only.

Also following submission of the PFS, Phelps Dodge determined that the Kansanshi Project did not meet its corporate requirements and rendered the project for sale. Thereafter, First Quantum Minerals Operations Limited bought and took managements control of the Kansanshi project, initially focusing on open pit mining and higher grade, leachable ores was more appropriate.

1.3 Kansanshi Geology

The Kansanshi Deposit occurs within the northwest-trending Kansanshi antiform, which exposes rocks of the Kansanshi Mine formation in its core. The Kansanshi antiform hosts four major stratigraphic units which structurally, from top to bottom, are described as follows:

- **Upper Dolomite** – the sequence comprises dolomite and dolomite marble and can be described geologically as pale brown-grey to medium grey, fine grained, saccharoidal and iron-free dolomite;
- **Upper Pebble Schist** – a monotonous sequence of non-bedded biotitic, calcareous, garnet-bearing schist;
- **Kansanshi Mine Formation** – this formation consists of interlayered phyllite, schist, marble and calcareous schist; and
- **Lower Pebble Schist Formation** – this unit is calcareous biotite schist, locally garntiferous that contains up to 10 % of exotic clasts.

Figures 1.2 and 1.3 indicate Kansanshi stratigraphic column and geology plan view of North West and Main pit respectively.

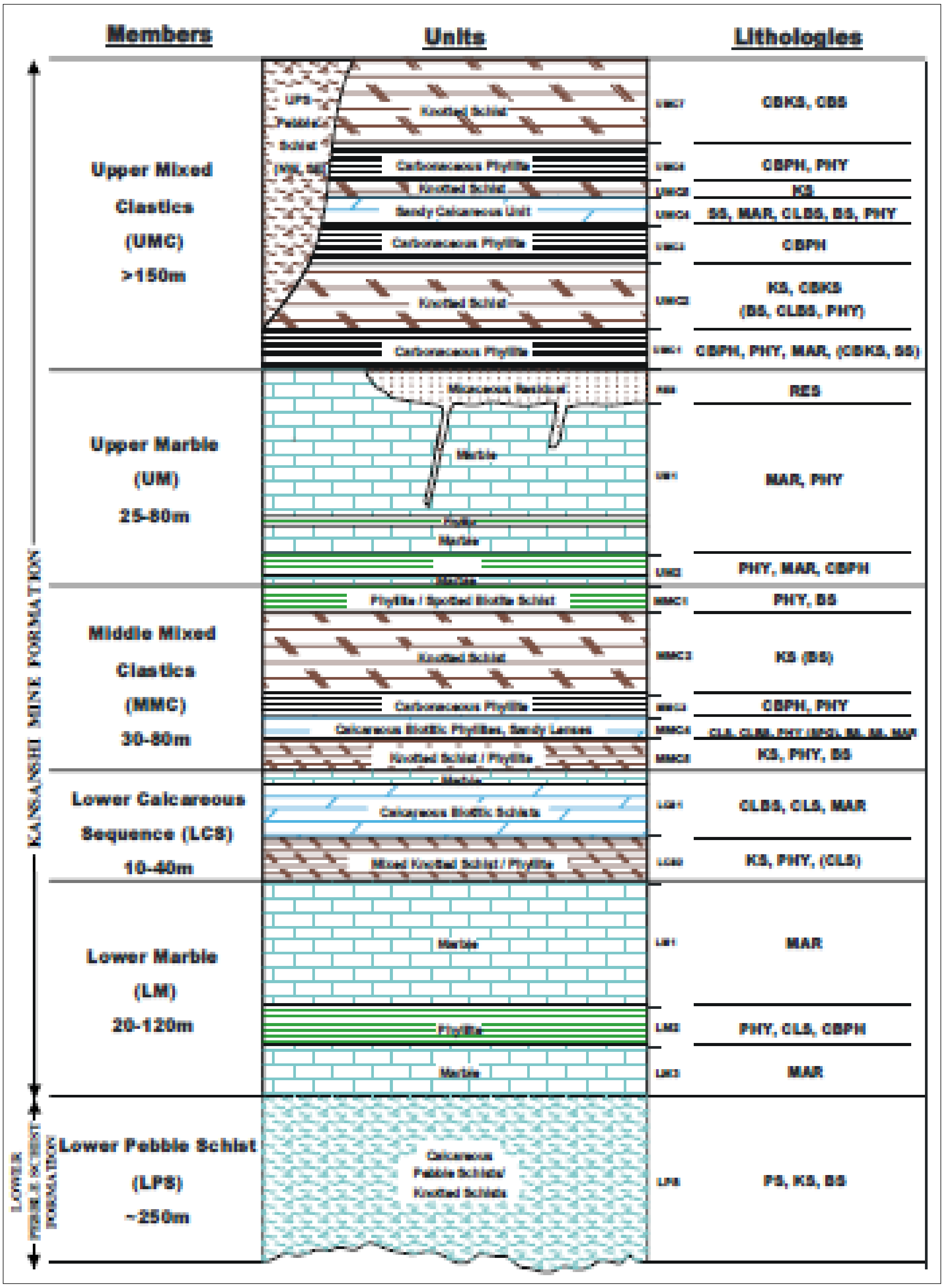

Figure 1.2 Kansanshi Stratigraphic Column (GRDMiniproc Limited, 2002)

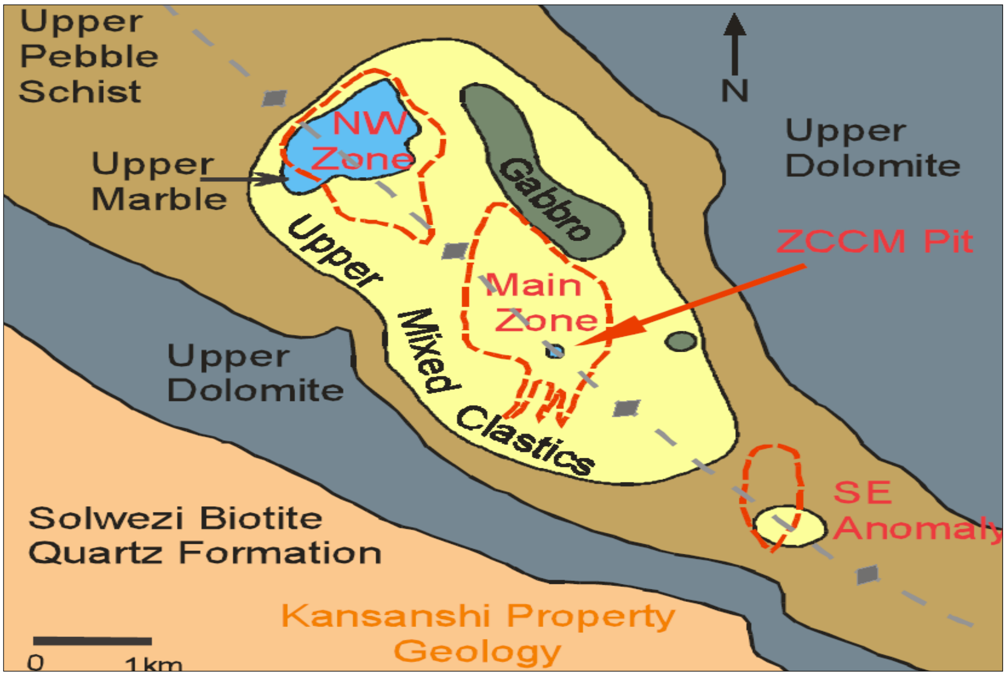

Figure 1.3 Kansanshi Geology Plan View (after GRDMiniproc Limited, 2002)

1.4 Problem Statement

The significance of this study was necessitated by the prevailing situation at Kansanshi Mine, with regards to the effects of excess water coming from both the surface water and from subsurface strata. The recharged of water from the surface and subsurface resulted in low production targets and Kansanshi management were spending huge sums of money trying to combat water and putting up remedial actions. During the wettest months the flow into main pit was 3000 to 3500 m^3/hour, and during severe storms, inrushes, breakdowns of pumps and power outages on the pumps affected production targets.

The current pumping capacity uses a combination of sumps, down-shaft submersible pumps, and in-pit bores, adding to a total capacity of 118,000 cubic metres per day. The drawdown in the water table below Main 3 was only eight metres during six months; as a result large volumes recharge the aquifers. Therefore, the decline will serve as dewatering excavation for the pit.

The decline will be developed spiral down to just above the water table and then develop tunnels approximately 4m x 4.5m parallel to the main water bearing structures.

These wide fracture zones, the 4800 Zone to the West, 5400 Zone in the Eastern pit wall, and along the Lower Marble bed, which outcrops in the floor of Main 3 pit. This study focused on rock mass characterization to determine appropriate support system. In addition, it is important to be aware of the fact that the success of the decline will help control ground and surface water.

1.5 Significance of Study

The outcomes of this research project were very vital for the successful support system, using both the Q-Systems and rockmass rating (RMR). This classification helped were to quantify the behavior of a continuum in terms of assessing material properties and thereby predicted the overall rock mass properties to determine safe and economical support design.

1.6 Research Objectives

The principal objectives of this research were to:

a) Characterize the rock mass characteristics of Kansanshi Mine Decline, and
b) Recommend support design system based on rock characterisation, which is safe, practical and cost effective system.

1.7 Research Methodology

The research methodology involved review of available reports on the mine and consultation with relevant mining personnel. Besides, a number of research methods were available and the selection of which depended on of rock mass characterisation systems. Figure 1.4 indicates summarised methodologies adopted in order to achieve the objectives:

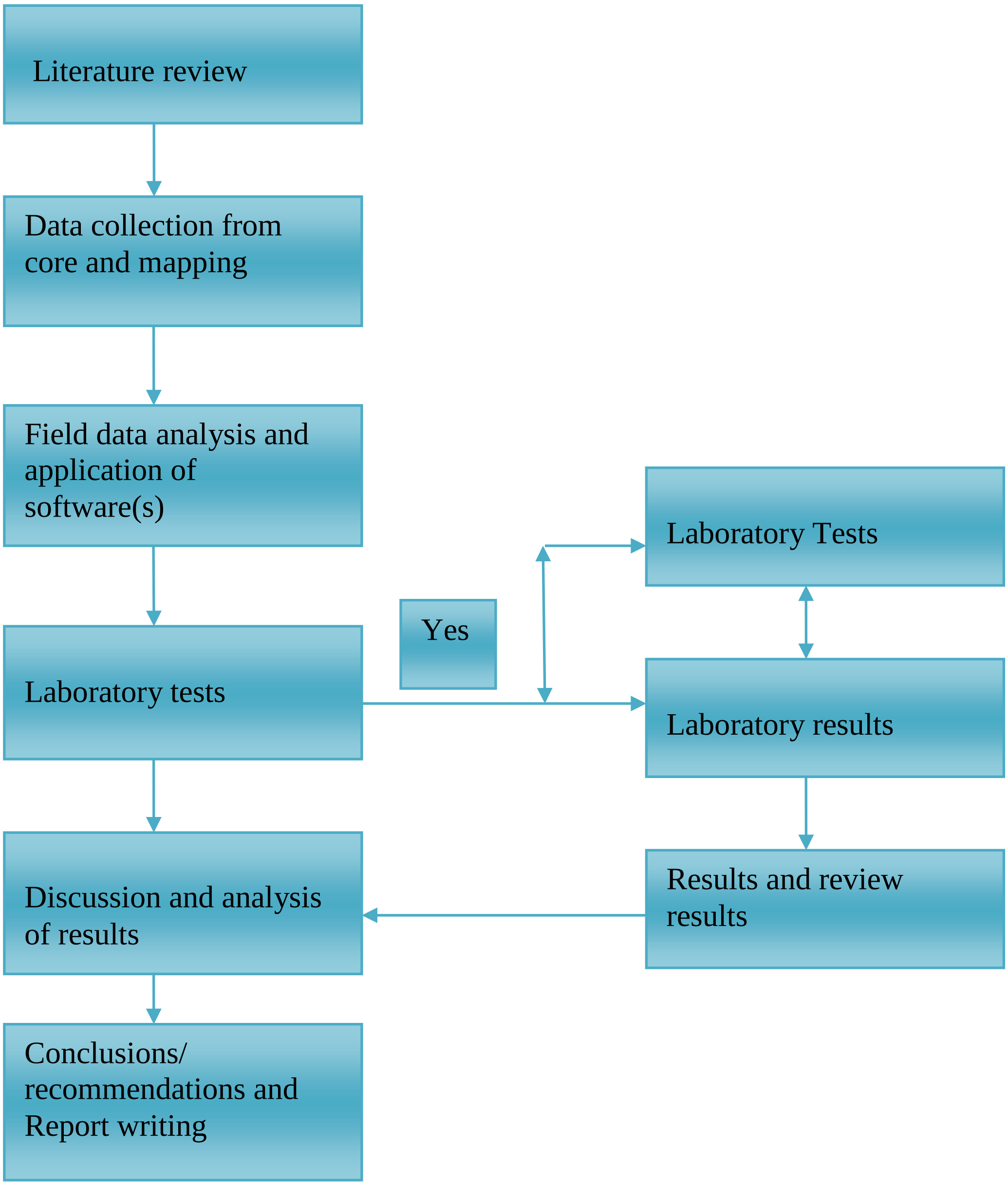

Figure 1.4 Study Methodologies Flow Sheet

CHAPTER 2

LITERATURE SURVEY

This chapter presents discussion on review of support interaction and rock mass classification systems.

2.1 Review of Support Interaction

In order to achieve the best support system design, the mechanical behaviour of the rock masses reinforced by full grouted bolts, i.e. the rock-bolts interaction, needs to be fully understood. The design methodologies for roof were classified in two categories:

- Analytical methods
- Geotechnical classifications

2.1.1 Analytical Methods

The oldest, simplest, and probably still the most widely used equation for bolt design is dead-weight suspension (Obert and Duvall, 1967).

A modified version of this design principle (Wagner, 1985) is still being used in South African collieries in the design of suspension methods. The design of roof bolt systems, based on the dead-weight principle, had to satisfy the following requirements:

- The strength of the roof bolt system, SB, has to be greater than weight, W, of the loose roof layer that has to be carried;

$$\sum_{i-1}^{n} SB > W \tag{2.1}$$

- The anchorage forces, AF, of the roof bolt system have to be greater than the weight of the loose roof layer;

$$\sum_{i-1}^{n} AF > W \tag{2.2}$$

- Usually the design incorporates a safety factor, SF;

$$\sum_{i-1}^{n} SB\iota - SF.W > and \ \sum_{i-1}^{n} AF\iota - SF.W > 0 \qquad (2.3)$$

- The number , n, of bolts/m^2 required to support a loose layer or layers of thickness, t, is given by;

$$n = SF\ (\rho gt/pf) \qquad (2.4)$$

Where:

SF = Safety Factor

ρ = Density of Suspended strata

g = Gravitation acceleration

pf = Anchorage capacity

The method is suitable for suspension bolting in low-stress environments. However, horizontal forces can greatly increase the loads applied to roof bolts (Fairhurst and Singh, 1974). Signer (1990) found that, measured loads on roof bolts are twice what would be predicted by dead-weight design.

Beam theory has also been used in South African Collieries in the design of roof bolt systems since 1980. The parameters that govern the behaviour of gravity-loaded beams with clamped ends are as follows:

- Maximum bending stresses (MPa) $\sigma_{xy} = \dfrac{\rho g L^2}{2t}$ (2.5)
- Maximum shear stress (MPa) $\sigma_{xy} = \dfrac{3\rho\rho g}{4}$ (2.6)
- Maximum deflection (m) $\sigma_{xy} = \dfrac{\rho g L^4}{32Et^2}$ (2.7)

Where:

L = roof span (width of an excavation) (m)

t = thickness of roof layer (m)

ρ = density of suspended strata, kg/m^3

g = gravitational acceleration (m/s^2)

E = Elastic Modulus (MPa)

In Australia, Frith (1998) proposed model that was based on underground measurements and divides mine roofs into two classes:

- Static roof that is self-supporting and requires minimum reinforcement, and
- Buckling roof that is thinly bedded and tends to fail layer by layer as a result of horizontal stress.

Frith (1998) proposed that the behaviour of the second type of roof can be explained by the basic structural engineering concept of the Euler bucking beam. There have been a number of trials of high-tension fully grouted bolts in Australia, and the results are reported to be positive.

Allen and Arduino (2002) determined the permanent shotcrete thickness (t) and the bolt/anchor spacing (b) from the following expressions derived from recommendations on support given by Barton (1979).

- t (Shortcrete thickness) = $11.25 \times D_e / Q^{0.43}$ (2.8)
- b (Bolt/anchor spacing) = $1.21 \times Q^{0.17}$ (2.9)

Where:

D_e = equivalent dimension

Q = Rock mass quality

Afrouz (1973) determined also bolt and anchor lengths for permanent support that depends on the dimensions of the excavations. Lengths used in the roof arch are usually related to the span, while lengths used in the walls are usually related to the height of the excavations.

The ratio of bolt length to span tends to reduce as the span increases. Accordingly, the following recommendations are given as a simple rule of thumb, to be modified as in situ conditions demand:

For roof:

Bolts $L = 2 + 0.15B/ESR$ (2.10)

Anchors $L = 0.40B/ESR$ (2.11)

For walls:

Bolts $L = 2 + 0.15B/ESR$ (2.12)

Anchors $L = 0.35H/ESR$ (2.13)

Where:

- L = length (m)
- B = width of an excavation or span (m)
- H = excavation height (m)
- ESR = excavation support ratio

2.1.1.1 Excavation Stability

The stability analyses for underground mines, is principally based on rock mass behaviour, although structural instability mechanisms need to be considered in environments where a dominant (ubiquitous) structure governs the excavation stability. The following sections of the report examine the principle design considerations and techniques for underground excavation design. Many of the design concepts are applicable to both tunnel and stope design although the origins of some design guidelines, and particularly those that are empirically based, may restrict their application to a specific excavation category.

2.1.1.2 Structural Instability

In this context structural stability considers modes of failure where the potential mechanism of failure can be reasonably well defined and evaluated by deterministic and mechanism specific means. The assessment of these failure mechanisms generally requires a relatively high level of geotechnical data and would normally only be associated with feasibility or operational phases of design.

2.1.1.3 Block Instability

In areas of good rock mass quality, where general rock mass instability is unlikely, the primary mode of failure is likely to be due to the formation of isolated wedges (blocks) in the back or sidewalls of the excavations. These blocks are either identified in situ by observation and mapping or by parametric studies of the potential for block formation and release based on analysis of the known defect orientations and persistence.

The former method of block identification usually leads to a spot bolting strategy whereas the latter is used to ensure that systematic bolting is capable of supporting the identified block instabilities. Computer based tools are available to assist in the visualisation of the block geometry and definition of support requirements. Figures 2.1 and 2.2 indicate the supported block wedge on the roof and sideward with roof bolts respectively.

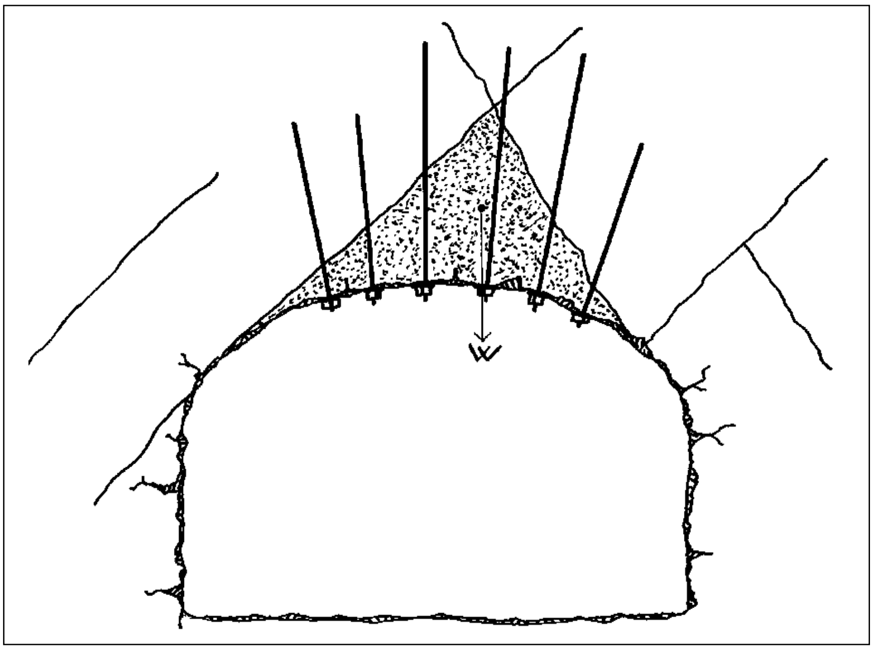

Figure 2.1 Supported block wedge (Hayes and Altounyan 1995)

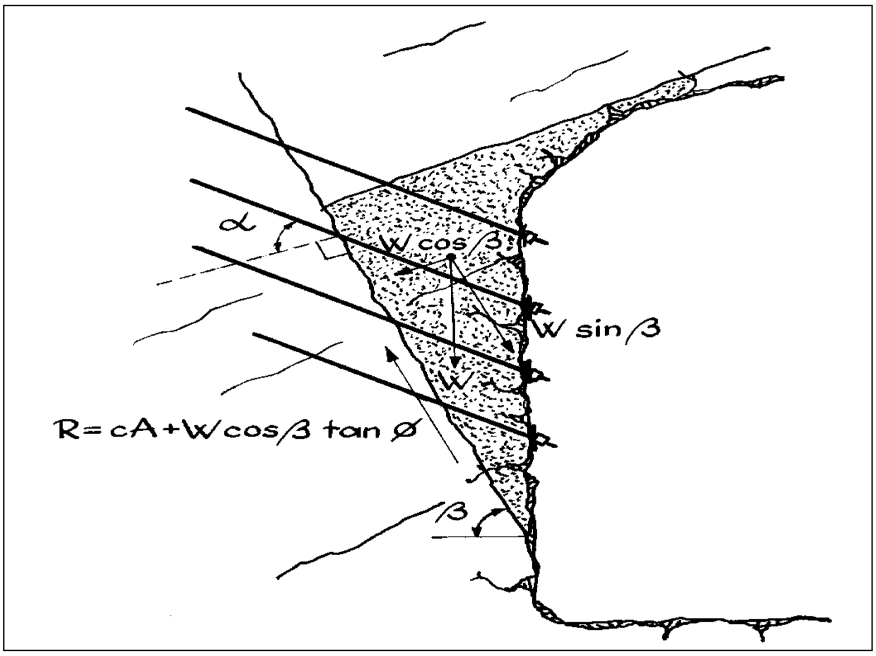

Figure 2.2 Supported side wedge (Hayes and Altounyan 1995)

The required support capacity is either based on the deadweight of the block if it will be released from the back of the excavation or to increase the shear resistance if the block instability is based on sliding from the sidewall. In both cases an understanding of the likely block volume is necessary, and in the latter case it is also necessary, in order to minimise bolting requirements, to have an understanding of the shear strength of the sliding plane. Rock bolt support must be of sufficient length to anchor the block in stable ground. This will be a function of the depth of the unstable block into the rock mass and a sufficient length to provide adequate anchorage.

2.1.1.4 Beam Buckling and Bending of Rockmass

For buckling instability we are principally concerned with identifying a dominant rock mass structure that is likely to lie parallel to an excavation rock wall. Where bedding or foliation, at a relatively small spacing relative to the excavation dimension, dominates this rock mass structure then there is a strong potential for buckling instability. Figure 2.3 indicates supported beam of stratified rock mass with roof bolts.

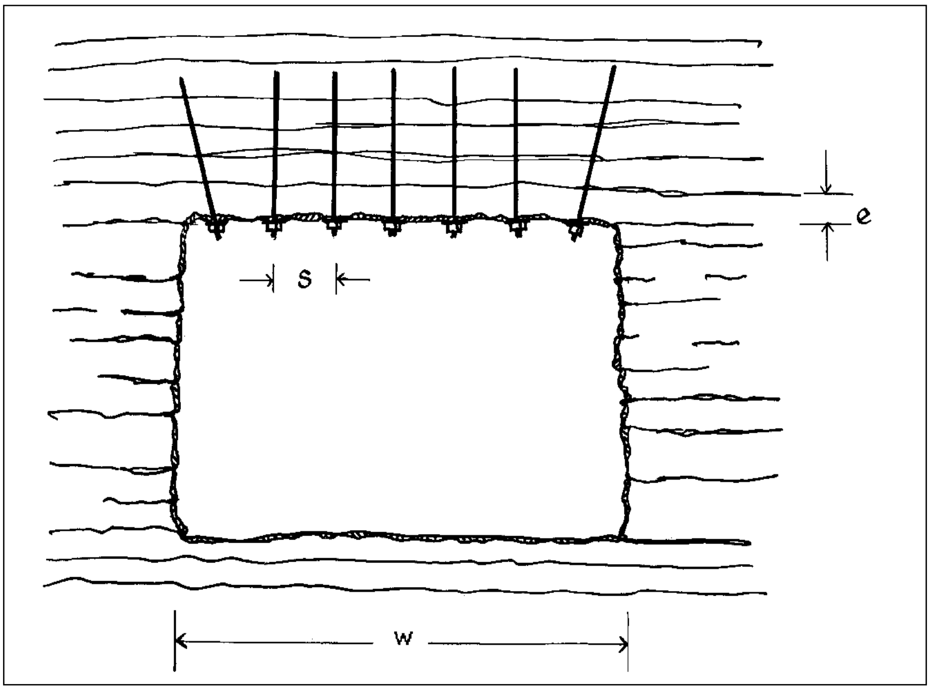

Figure 2.3 Supported beam of stratified rock mass (Hayes and Altounyan 1995)

The method is applicable where the slabs created by the dominant rock mass structure can be represented by the intact material properties, no cross structures exist and the out of plane dimension of the slab is significantly larger than the one in-plane length. The load applied to the stratified rockmass, is due to the induced stress in the immediate boundary of an excavation.

2.1.1.5 Gravity Induced Bending

The potential for gravity induced bending can be assessed by the application of a Voussoir beam approach. This approach is applicable where the beam is broken by sub-perpendicular cross joints, such that the beam has no tensile strength and the end abutments of the span are stable (rigid). The principal input parameters are the beam geometry, rock mass modulus, rock UCS and density. A useful rule for stability monitoring, derived from this approach, is that the limit of stability is reached at a mid-span deflection of approximately 10% of the beam thickness, although at low UCS values this is reduced due to the early onset of rock failure in the beam. Figure 2.4 indicates an estimation of beam stability theory developed by Hayes and Altounyan 1995.

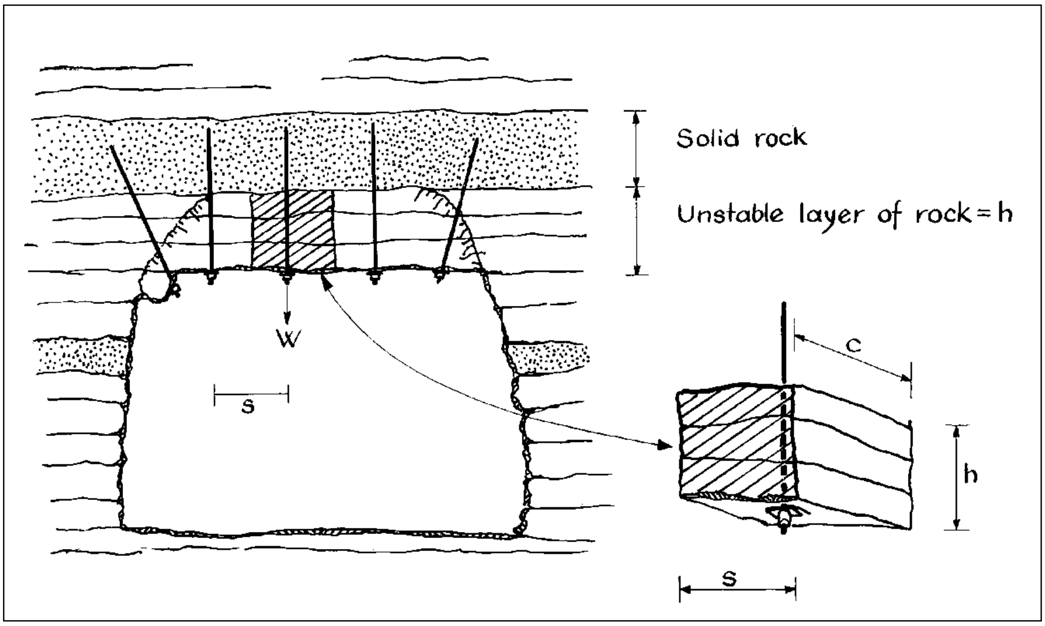

Figure 2.4 Supported beam of solid rock from unstable layer underneath (Hayes and Altounyan 1995)

To achieve excavation stability the design strategy for both the above instability mechanisms is to add reinforcing bolts at a suitable spacing and to a depth that creates, or anchors a stable "reinforced" beam. Due to the assumptions inherent in the above beam analyse, it is recommended that a suitably high Factor of Safety is applied to the design, of the order from 1.5 to 2.0.

2.1.1.6 Rockmass Instability of Excavation

The creation of an excavation results in the formation of an unstable tensile zone in the immediate roof, above which a natural arch is formed where the rock mass loading condition is primarily compressive. The extent of the unstable zone will be a function of the rock mass structure, loading environment, span of the excavation and stability of the sidewalls. The application of un-tensioned, grouted support units within this environment is to pin the unstable rock mass to the competent natural arch as presented in Figure 2.5.

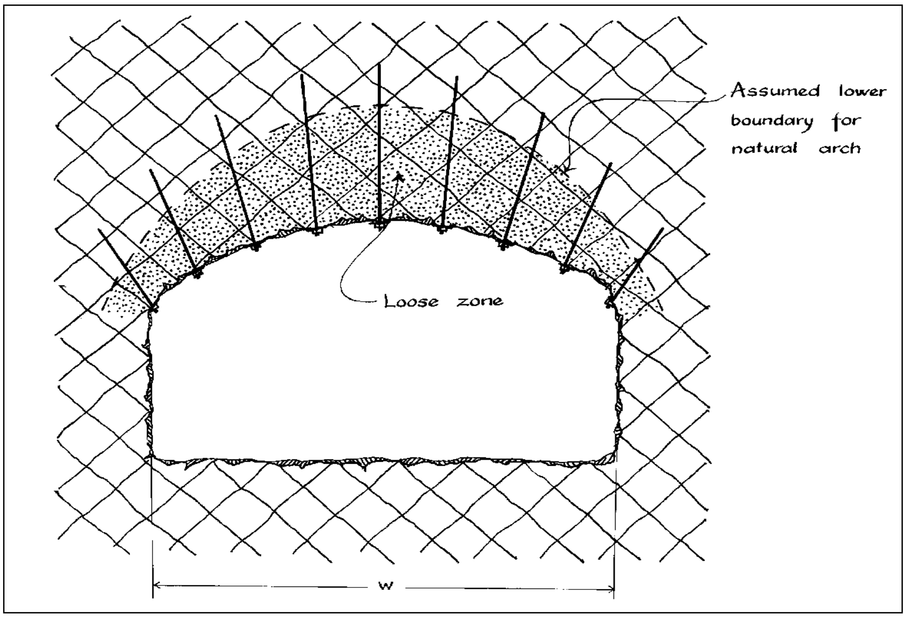

Figure 2.5 Supported block and lose rockmass underneath (Hayes and Altounyan 1995)

The illustration in Figure 2.5 indicates that, the length of the rock bolts must be sufficient to anchor into competent rock mass. Stillborg (1994) also suggested an empirical estimation of the required rock bolt length as presented below.

$$L = 1.40 + 0.184w \qquad (2.14)$$

Where:

w = the span of the excavation (metres) (Stillborg 1994)

L = the required rock bolt length (metres)

The technique above, and the use of un-tensioned grouted rock bolts, is generally more applicable to competent, moderately jointed rock mass structures, which result in the lower boundary of the natural arch being closer to the boundary of the excavation. In high stress environments, where significant fracturing of the rock mass may occur, or under low stress conditions but within more highly discontinuous rock mass structures, the depth of instability may exceed the practical depth of rock bolt anchorage. Under these conditions excavation stability may be achieved by the creation of reinforced beam or arch structures within the discontinuous rock mass.

The guidelines for support system design in order to create a reinforced rock mass arch within a highly jointed rock mass are:

Bolt length = $1.60 + \sqrt{(1.0 + 0.012.w^2)}$ (Stillborg 1994) (2.15)

Bolt spacing = 3 x joint spacing (e) (Stillborg 1994) (2.16)

Bolt tension = 0.5 - 0.8 x capacity of bolt (Stillborg 1994) (2.17)

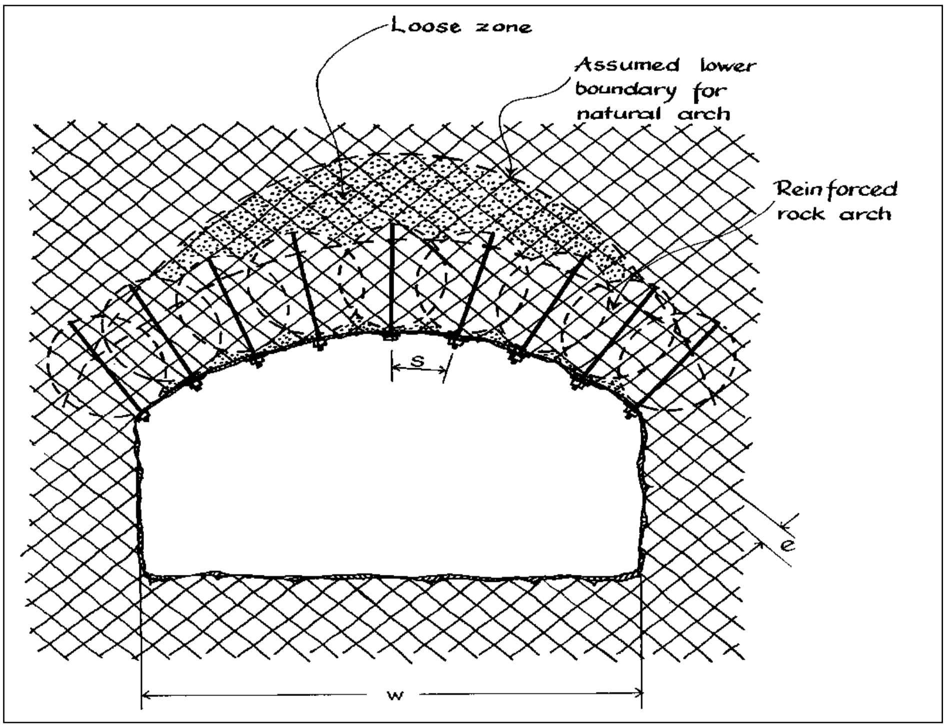

Figure 2.6 Rock bolting pattern for an excavation in jointed rock (after Stillborg 1994)

The creation of reinforced rock mass steel arch, within highly discontinuous rock mass structures (Stillborg 1994), is considered that the concept of the creation of a structurally competent reinforced rock mass arch is the basis of most of the empirical rock bolt system design methods.

Due to the complexities within the rock mass structure, or the lack of detailed understanding of the rock mass at early phase of design, the assessment of excavation stability based on specific structural analysis may not be possible. Under these conditions it is convenient to utilise empirical design methodologies based on analysis of the geotechnical environment. The analysis of the geotechnical environment, and thus design confidence, will be a function of the design phase of the project and the availability of suitable geotechnical data or information.

The majority of empirical design methodologies are based on experience gained within shallow, low stress, civil engineering applications. They thus take limited consideration of a mining environment where quasi-static or dynamic stress change may be an issue.

However, they do offer a basis for an initial guideline of stability and appropriate support systems.

Simple design rules based on an evaluation of case studies were developed by the US Army Corps of Engineers (1980). These rules have found widespread application, particularly as an initial estimate of support requirements, and are applied as the basis for rock bolt spacing in the design of rock bolt support systems within many mines.

Other design rules for rock bolting within jointed rock mass structures, with tight joint conditions, were put forward by Farmer and Shelton (1980). These are similar to those given by the US Army Corps of Engineers (1980), but consideration is also given to the number of joint sets and their orientation relative to the excavation and rock bolt installation as presented in Table 2.1.

Table 2.1 Tunnel support guidelines (after US Army Corps of Engineers, 1980).

Parameter	**Empirical Rule**
Minimum length	Greatest of:
	a) 2 x bolt spacing
	b) 3 x thickness of critical and potentially unstable rock
	Blocks
	c) For elements above the springline*
	spans < 6 m : 0.5 x span
	spans between 18 m and 30 m ; 0.25 x span
	spans between 6 m and 18 m : interpolate 3 - 4.5 m
	d) For elements below the springline*
	height < 18 m : as c) above
	height > 18 m : 0.2 x height
Minimum spacing	Least of:
	a) 0.5 x bolt length
	b) 1.5 x width of critical and potentially unstable blocks
	c) 2.0 m (greater spacing difficult to attach fabric support)
Minimum spacing	0.9 to 1.2 m
Minimum average confining	Greatest of
Pressure	a) Above springline
	either pressure = vertical rock load of 0.2 x opening width
	or 40 kN/m^2
	b) Below springline
	either pressure = vertical rock load of 0.1 x opening height
	or 40 kN/m^2
	c) Intersections: 2 x confining pressure determined above

* this is typically interpreted as the point of deviation from vertical in the upper sidewall.

Table 2.2, indicates the empirical guideline for design of rock bolt system in exaction less 15m span.

Table 2.2 Empirical guidelines for design of rock bolt system in excavations <15 m span (after Farmer and Shelton, 1980)

Number of discontinuity sets	**Rock bolt design**	**Comments**
≤ 2 inclined at 0 - 45° to horizontal	L = 0.3 B s = 0.5L (depending on thickness and strength of strata). Install bolts perpendicular to lamination where possible with wire mesh to prevent flaking	The purpose of bolting is to create a load carrying beam over the span. Fully bonded bolts create greater discontinuity shear stiffness. Tensioned bolts should be used in weak rock, sub-horizontal tensioned bolts where vertical discontinuities occur.
≤ 2 inclined at 45 - 90° to horizontal	For side bolts: L > h.sinψ (if installed perpendicular to discontinuity): L > h.tanψ (if installed horizontally). Where h is the distance of installation from the point of daylight of laminations in the sidewall that defines the largest unstable wedge, and ψ the inclination of laminations from vertical.	Roof bolting as above. Side bolts designed to prevent sliding along planar discontinuities. Spacing should be such that anchorage capacity is greater than sliding or toppling weight. Bolts should be tensioned to prevent sliding.
≤ 3 with clean, tight interfaces	L = 2s s = 3-4 x block dimension. Install bolts perpendicular to excavation periphery with wire mesh to prevent flaking.	Bolts should be installed quickly after excavation to prevent loosening and retain tangential stresses. Pre-stresses should be applied to create a zone of radial confinement. Sidewall bolting where toe of wedge daylights in sidewall.

These simple design rules allow initial estimations of support requirements but give limited or no consideration of the rock mass characteristics and as such will tend to be conservative in nature. A more detailed empirical design methodology can be based on a rock mass classification system to indicate the selection of a support system as described in the following section. The most widely used classification systems are the Q system (Barton, Lien and Lunde 1974) and the Geomechanics classification, or Rockmass rating (RMR) system (Bieniawski 1989), which also forms the basis of the Modified (or Mining) Rock Mass Rating (MRMR) system (Laubscher, Taylor 1976).

2.2 Review of Existing Classification Systems

Decline support requirements was analysed using worldwide accepted empirical guidelines for support selection. Besides, a more detailed empirical design methodology was used based on a rockmass classification system to indicate the selection of a support system. The most widely used classification systems are the Q-system (Barton, Lien and Lunde 1974) and the Geomechanics classification, or Rockmass rating (RMR) system (Bieniawski1989), which also forms the basis of the Modified (or Mining) Rock Mass Rating (MRMR) system (Laubscher and Taylor 1976).

Both methods incorporate geological, geometric and design or 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 systems lie in the different weightings given to similar parameters and in the use of distinct parameters in one or the other scheme.

Rockmass Rating (RMR) uses compressive strength directly while Q-system 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 groundwater, and both include some component of rock material strength. Some estimate of orientation can be incorporated into Q using a guideline presented by Barton (1994); '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.

When using either of these methods, two approaches can be taken. One is to evaluate the rock mass specifically for the parameters included in the classification methods; the other is to accurately characterise the rock mass and then attribute parameter ratings at a later time. The latter method is recommended since it gives a full and complete description of the rock mass which can easily be translated into either classification index. If rating values alone had been recorded during mapping, it would be almost impossible to carry out verification studies.

For the purpose of this report, two classification methods were used, the Q System developed by Barton (1994) and the Geomechanics Classification System developed by Bieniawski (1989).

2.2.1 Q-System

The Q System classification is based on three aspects:

- Rock block size (RQD/J_n)
- Joint shear strength (J_r /J_a)
- Confining stress (J_w/SRF)

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

All selected values for the above six parameters, based on observed or estimated conditions are substituted into the equation to obtain the value of the rock quality index (Q).

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \qquad (2.15)$$

The Q system does not take the rock material strength into account explicitly, although it is implicitly included in arriving at the SRF assessment. The orientation of joints is also not taken into account since it is considered that the number of joint sets, and hence the potential freedom of movement for rock blocks is more important. The range in values of Q is from 0.001 for extremely poor rock to 1000 for excellent rock. The core represented a source of geotechnical information, and structurally controlled stability situations. The following basic parameters were required from geotechnical core logging:

- Depth, Description (Rock Type),
- Core recovery, RQD - rock quality designation,
- Strength of intact rock (Rock Hardness),
- Weathering/alteration, Fracture/discontinuity type, Fracture frequency,
- Dip angle of structure with respect to core axis and dip direction in case of oriented core, and
- Discontinuity condition.

The logged core was divided into separate geotechnical formations or design zones, within which the core displayed similar geotechnical characteristics, and within which the rock mass was expected to perform uniformly in an excavation.

(a) Rock Quality Designation (RQD)

RQD is defined as the ratio of the cumulative length core size more than 100mm in length in a drill run to the total length of the drill run.

$$\mathrm{RQD\%} = \frac{\text{Total length of core} \geq 100\text{mm x} 100}{\text{Length of core run}} \tag{2.16}$$

Engineering judgement must be exercised for poorly orientated boreholes. For example holes parallel to bedding in a sedimentary deposit may indicate very high values of RQD, whereas holes across bedding in the same rock may indicate much lower RQD's.

Palmström (1982) suggested that, when no core is available but discontinuity traces are visible in surface exposures or exploration tunnels, the RQD may be estimated from the number of discontinuities per unit volume. The suggested relationship for clay-free rock masses is: RQD = 115-3.3Jv. Where; Jv is the sum of the number of joints per unit length for all joint (discontinuity) sets known as the volumetric joint count. For Jv < or =1.4, RQD is equal to 100.

RQD is a directionally dependent parameter and its value may change significantly, depending upon the borehole orientation. The use of the volumetric joint count can be quite useful in reducing this directional dependence. In this method of study, bench face count of fracture frequency to come up with Jv was used to determine RQD using Palmström's relationship. Blast induced fractures were not included when estimating Jv. The correct procedures for measuring core pieces and the calculation of RQD is given in Figure 2.7.

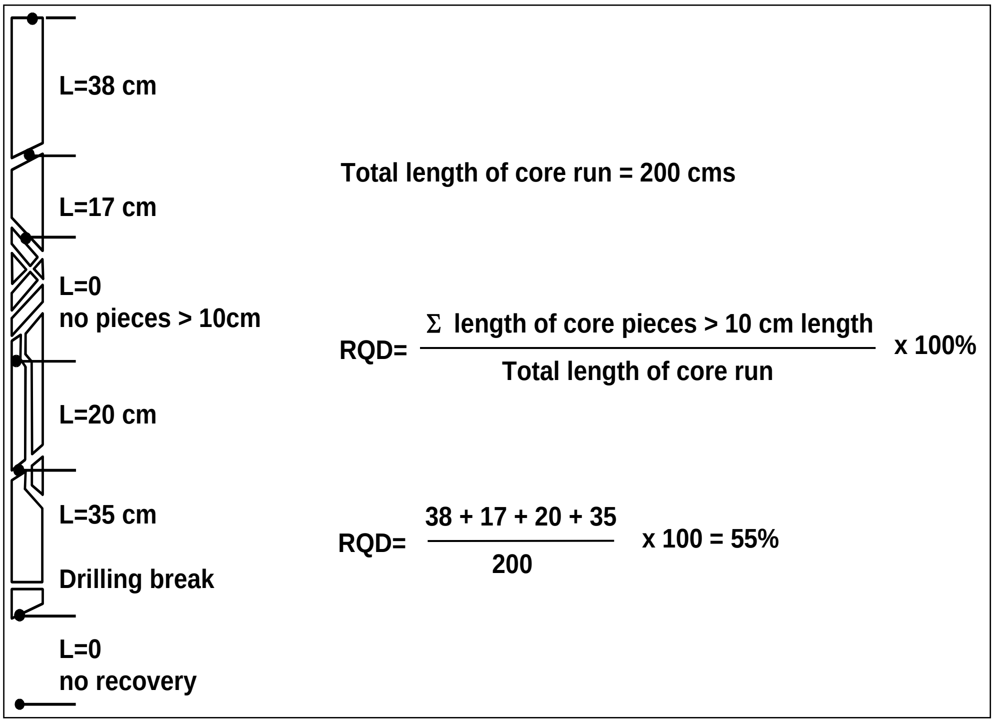

Figure 2.7 Procedure for measuring and calculating of RQD (after Deere, 1989).

Note:

- Where RQD is reported or measured as less than 10, a nominal value of 10 is used to evaluate Q.

The number of defect sets, over a defined logging length (drill core or mapping), which should be roughly representative of the likely excavation dimension; the number of defect sets should be noted. Although this number can also be defined from orientation data, it may be the case that not all sets are present within a given rock mass volume at any one time. The number of defect sets within a given rock mass volume broadly defines the likely block shape.

(b) Joint Set Number – Jn

The joint set ratings are presented in Table 2.3.

Table 2.3 Joint Set Number (after Barton 1994)

Number of Joint Sets	Joint Set No. J_n
Intact, no or few joints	0.5 —1.0
One joint set	2
One joint set plus random joints	3
Two joint sets	4
Two joint sets plus random joints	6
Three joint sets	9
Three joint sets plus random joints	12
Four or more joint sets, random, heavily jointed, sugar cube, etc.	15
Crushed rock, earth like	20

Notes:

- For intersections use $J_n = 3\ J_n$
- For portals use $J_n = 2\ J$

(c) Joint Roughness Number - J_r

Distinction is made between the large scale nature of planes and the small scale roughness as well as between continuous and discontinuous joints. Table 2.4 presents joint roughness number (J_r) ratings after Barton 1994.

Table 2.4 Joint Roughness Number (after Barton 1994)

Description of Joint Surface Roughness	Discontinuous	Undulating	Planar
Rough	4.0	3.0	1.5
Smooth	3.0	2.0	1.0
Slickensided	2.0	1.5	0.5
Planes containing gouge thick enough to prevent rockwall contact	1.5	1.0	1.0

Note:

- Add 1.0 to *Jr* if the mean spacing of the relevant joint set is greater than 3m.

(d) Joint Alteration Number - J_a

The joint alteration number takes into account the weathering of, or coating on, joint surfaces and the thickness and nature of any gouge infill present in the joints. This parameter will determine the shear strength of the rock mass as well as its deformability and potential to squeeze or swell. Table 2.5 presents joint alterations ratings or values.

Table 2.5 Joint Alteration Number (after Barton 1994)

Description of Gouge	Joint Alteration Number J_a for Joint Separation (mm)		
	<1.0[1]	**1.0-5.0**[2]	**>5.0**[3]
Tightly healed, hard, non-softening impermeable rock mineral filling	0.75	-	-
Unaltered joint walls, surface staining only	1.0	-	-
Slightly altered, non-softening, non-cohesive rock mineral or crushed rock filling	2.0	4.0	6.0
Non-softening, slightly clayey non-cohesive filling	3.0	6.0	10.0
Non-softening strongly over-consolidated clay mineral filling, with or without crushed rock	3.0	6.0	10.0
Softening or low friction clay mineral coatings and small quantities of swelling clays	4.0	8.0	13.0
Softening moderately over-consolidated clay mineral filling, with or without crushed rock	4.0	8.0[4]	13.0
Shattered or micro-shattered (swelling) clay gouge, with or without crushed rock	5.0	10.0[4]	18.0

Notes:

- Joint walls effectively in contact.
- Joint walls come into contact before 100mm shear.
- Joint walls do not come into contact at all upon shear.

(e) Joint Water Reduction Factor - Jw

The joint water reduction factor allows for the water pressure on the joint walls, as well as the potential for the outwash and softening of joint gouge. Table 2.6 indicates joint water reduction factor.

Table 2.6 Joint Water Reduction Factor (after Barton 1994)

Condition of Groundwater	**Head of water (m)**	**Joint Water Reduction Factor J_w**
Dry excavation or minor inflow 5 litre/minute locally	<10	1.0
Medium inflow, occasional outwash of joint/fissure fillings	10 – 25	0.66
Large inflow in competent ground with unfilled joints/fissures	25-100	0.5
Large inflow with considerable outwash of joint/fissure fillings	25-100	0.33
Exceptionally high inflow upon excavation, decaying with time	>100	0.2-0.1
Exceptionally high inflow continuing without noticeable decay	>100	0.1-0.05

Notes:

- Last three categories are crude estimates. Increase J_w if drainage measures are installed.
- Special problems caused by ice formation are not considered.

(f) Stress Reduction Factor – SRF

SRF values for weakness zones and competent rock are dealt with separately below

a) Weakness zones intersecting excavation which may cause loosening of rock mass when tunnel is excavated. Table 2.7 presents stress reduction factors for weak zones.

Table 2.7 Stress Reduction Factor for weakness zones (after Barton 1994)

Description	SRF Value
Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10
Single weakness zones containing clay or chemically disintegrated rock (depth of excavation < 50m)	5
Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)	2.5
Single shear zones in competent rock (clay-free), loose surrounding rock (any depth)	7.5
Single shear zones in competent rock (clay-free) (depth of excavation < 50m)	5.0
Single shear zones in competent rock (clay-free) (depth of excavation > 50m)	2.5
Loose open joints, heavily jointed or "sugar-cube" etc (any depth)	5.0

Note:

- Reduce these values of SRF by 25-50% if the relevant shear zones only influence, but do not intersect the excavation.

b) Competent rock, rock stress problems. Table 2.8 indicates the stress reduction factor for competent rock and rock stress problems.

Table 2.8 Stress Reduction Factor for competent rock and rock stress problems (after Barton 1994)

Description	**UCS /σ_1**	**σ_t / σ_1**	**SRF Value**
Low stress, near-surface	>200	>13	2.5
Medium stress	200-10	13-0.66	1.0
High stress, very tight structure (usually favourable to stability, may be unfavourable for wall stability)	10-5	0.66-0.33	0.5-2
Mild rock burst (massive rock)	5-2.5	0.33-0.16	5-10
Heavy rock burst (massive rock)	<2.5	<0.16	10-20

2.2.2 Geomechanics Classification System (RMR)

The Geomechanics Classification System derives a Rock Mass Rating (RMR), obtained by summing five parameter values and adjusting this total by taking into account the joint orientations. The parameters included in the system are:

- Rock material strength (UCS),
- RQD,
- Joint spacing,
- Joint roughness and separation, and
- Groundwater.

The descriptions and corresponding ratings for these parameters and the joint orientation adjustment are given in Appendix A. The RMR value can range between zero and 100, and a relationship has been found between RMR and Q as follows (Bieniawski, 1989):

$$RMR = 9 \ln Q + 44. \text{ Therefore, } Q = e^{((RMR - 44)/9)} \tag{2.17}$$

2.3 Geological Strength Index (GSI)

GSI (geological Strength Index) is used to for estimating the rock mass strength based on geological descriptions of the rock mass structure and defect surface conditions. Figures 2.8 and 2.9 indicate geological strength index for blocky jointed rocks and metamorphic rocks (after Hoek, E. 2000).

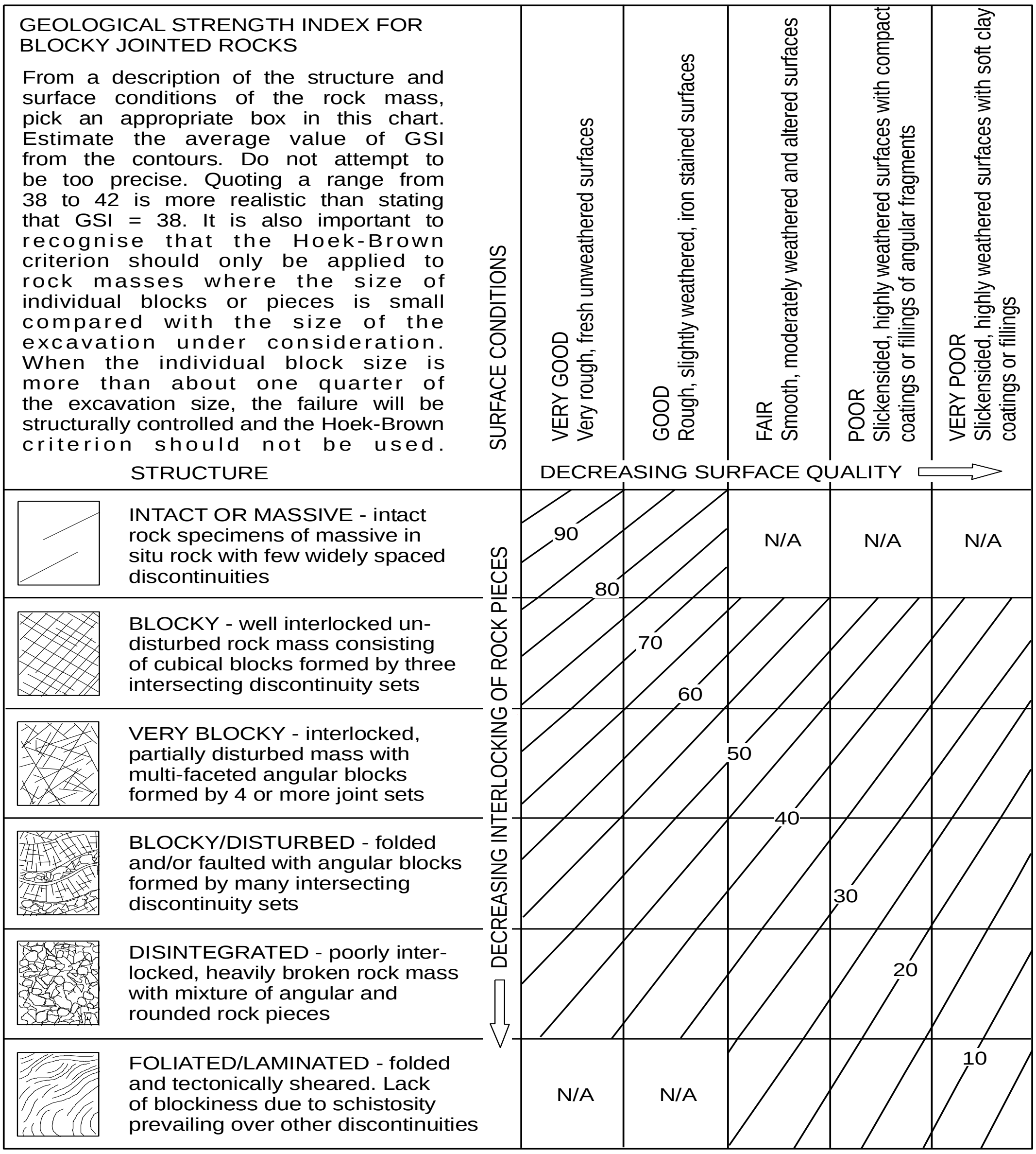

Figure 2.8 GSI for Blocky Jointed Rocks (after Hoek, E. 2000)

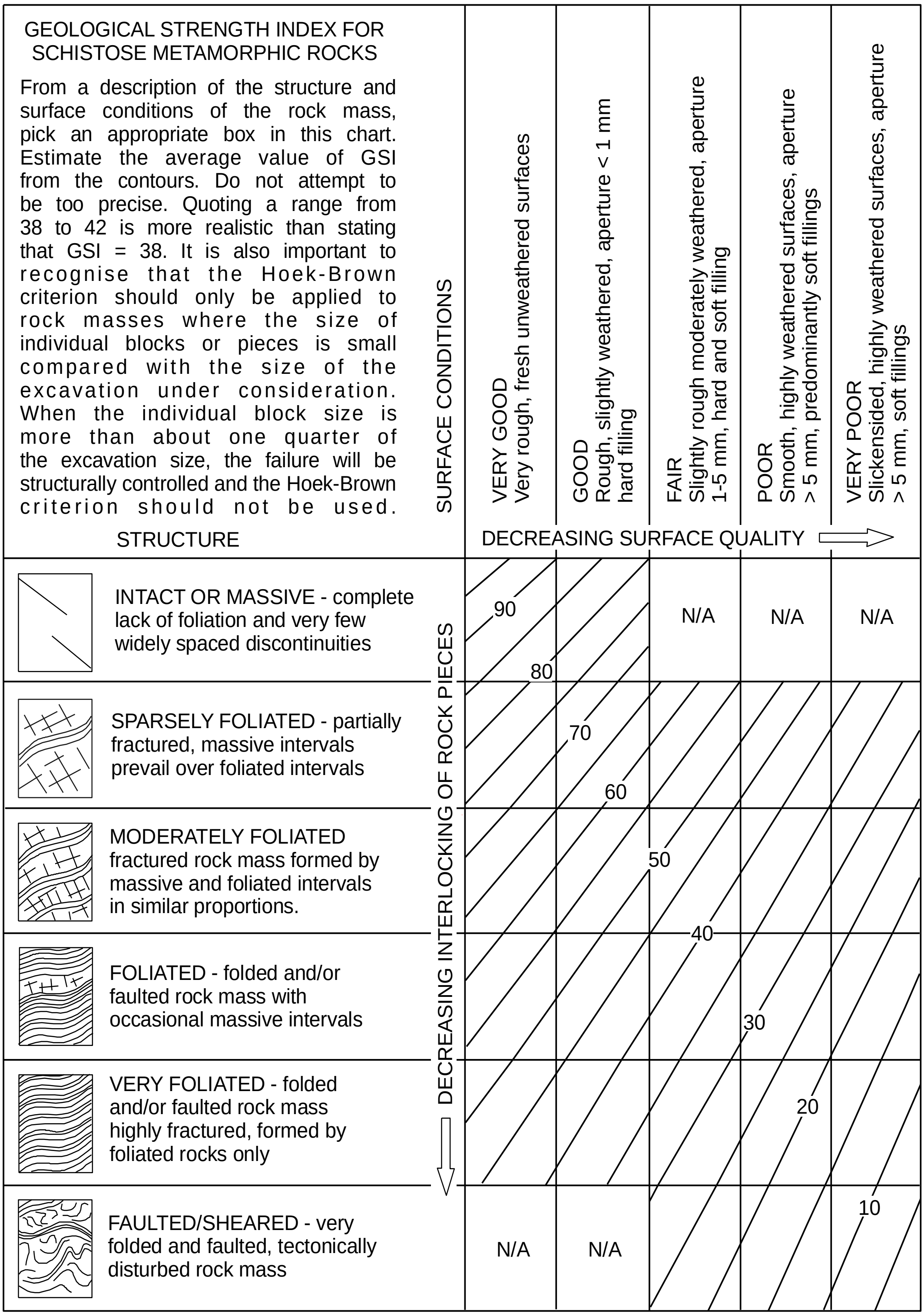

Figure 2.9 GSI -Metamorphic Rocks (after Hoek, E. 2000)

For better quality rock masses GSI can be estimated directly from RMR_{76} (after Bieniawski, 1976) with the groundwater rating set to 10 (dry) and the adjustment for joint orientation set to 0 (very favourable). The RMR_{76} value is difficult to estimate for very poor quality rock masses ($RMR_{76} < 18$); hence the GSI charts above should be used directly. For RMR_{89} (after Bieniawsk in 1989) GSI = RMR_{89}-5. Where RMR_{89} has groundwater rating set to 15 and the adjustment for joint orientation set to 0. As above, the RMR_{89} value is difficult to estimate for very poor quality rock masses ($RMR_{89} <$ 23); hence the GSI charts above should again be used directly.

2.4 Mining Rock Mass Rating (MRMR)

The MRMR was developed a series of support guidelines for tunnels. The philosophy behind Laubscher's method of support selection is that it is a function of both the original rock mass state (as provided by RMR) and the influence of mining (as provided by MRMR). The empirical database and classification of support requirements within the MRMR system have a greater basis in mass mining environments. Table 2.9 indicates Laubscher's guidelines for a general mining environment.

Table 2.9 Guidelines for a General Mining Environment (after Laubscher 1993)

Adjusted	**Original rock mass ratings, RMR**									
ratings	90 - 100	80 - 90	70 - 80	60 - 70	50 - 60	40 - 50	30 - 40	20 - 30	10 - 20	0 - 10
70 - 100										
60 – 70	A	a								
50 – 60	B	b	a	a						
40 - 50	B	b	b	b	b	c				
30 - 40	R	r	c	c	c	d	d			
20 - 30				d	e	f	f	c + l		
10 - 20						f/p	h + f/p	h + f/l	h + f/l	
0 - 10							h + f/p	f/p	t	T

Where:

Rock reinforcement

a) Local bolting at joint intersections
b) Bolts at 1m spacing
c) B and straps and mesh if rock is finely jointed
d) B and mesh / steelfibre reinforced shotcrete, bolts as lateral restraint
e) D and straps in contact with shotcreted
f) E and cable bolts as reinforcing and lateral restraint
g) F and pinning
h) Spilling
i) Grouting

Rigid lining

j) Timber
k) Rigid steel sets
l) Massive concrete
m) K and concrete
n) Structurally reinforced concrete

Yielding lining, repair technique, high deformation

o) Yielding steel arches
p) Yielding steel arches set in concrete or shotcrete

Fill

q) Fill

Spalling control

r) Bolts and rope-laced mesh

Rock replacement

s) Rock replaced by stronger material
t) Development avoided if possible

Supplementary notes:

Bolt length may be estimated from:

$$L = 1.0m + (0.33 \times S \times f) \qquad \text{(Laubscher, 1993)} \qquad (2.18)$$

Where:

S = span of an excavation

f = (if the RMR is 0 to 20, then f = 1.3; RMR 21 to 30, f = 1.2: RMR 31 to 40, f = 1.15; RMR 41 to 50, f = 1.1; RMR 51 to 60, f = 1.05, RMR > 61, f = 1.0)

The constant of 1.0m accounts for anchorage depth in excess of the depth of instability, surface roughness and protruding bolt length.

2.5 Types of Rockmass Reinforcement and Support

The primary objective in geotechnical design is to maximise the inherent rock mass strength by appropriate excavation geometry and size. This will minimise the need for artificial rock mass reinforcement or support. Where it becomes necessary to apply reinforcement or support this must be selected to be compatible with the anticipated behaviour of the rock mass to ensure maximum effectiveness.

The term rock mass reinforcement is applied to rock bolts, which are inserted into the rock mass to principally reinforce the shear strength of the rock mass defects. The term support is applied to systems that act on the rock wall surface and are external to the rock mass. However, in practice these terms may be somewhat interchangeable, particularly the use of the term support to encompass both rock bolts and surface support systems.

In general terms the key characteristics of reinforcement and support systems are defined as:

- Active or passive support, where active support provides immediate resistance to rock mass deformation and passive support requires rock mass deformation to generate resistance,

- The stiffness is the rate at which load is generated in the reinforcement or support unit with rock mass deformation,
- Strength or yield load is the peak load capacity of the system. It is usually applied to either the tensile or compressive capacity of the reinforcement or support but is equally applicable to the capacity of the system to accommodate shear deformation,
- Yield is the ability of the reinforcement or support to deform. It is usually applied to the amount of post peak deformation, and
- Energy absorption is a measure of the ability to accommodate dynamic deformation due to seismic loading. It is estimated from the area under the load deformation curve of the reinforcement or support system.

The design parameters for the reinforcement and support system of bolt length, spacing and recommended shotcrete thickness have been addressed in Appendix B.

In the design of a system comprising reinforcement and surface support components it is important to ensure compatibility between both the behavioural characteristics of the various components of the system and the rock mass, as the system will fail at its weakest link, making the remaining capacity redundant or over designed.

Where appropriate, the likely variability in the performance of the reinforcement or support system must also be considered in the assessment of the design capacity, in order to achieve a given level of design reliability.

In general, it is desirable to minimise the amount of rock mass deformation (apart from the initial 'elastic' deformation) as this minimises relaxation of the rock mass structure with a resulting loss in the overall rock mass strength. This in turn will lead to increased demand on the reinforcement and support systems to maintain excavation stability. This rock mass behaviour is illustrated by the concept of the ground reaction curve in Figure 2.10.

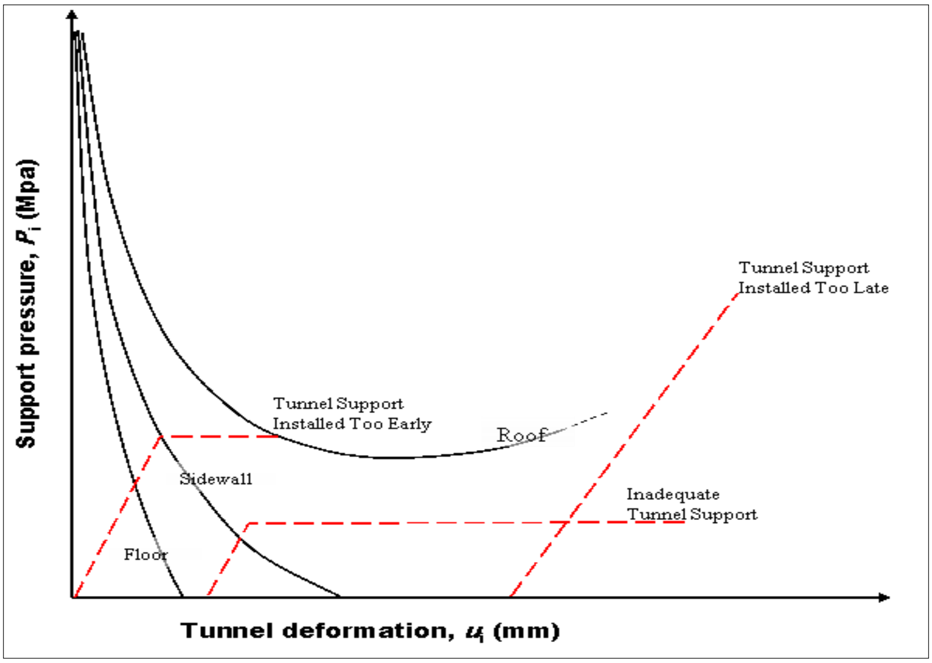

Figure 2.10 Ground reaction curve for rock mass reinforcement and support systems (after Laubscher 1993)

2.5.1 Support and Reinforcement Units

There are usually one or more types of support elements and support systems which will outperform others with regards to site specific ground conditions, working environment. The objective of this section was to attempt to outline the available support and reinforcing elements and the methods for determining a suitable support system as presented in Table 2.12.

Each of these support elements has specific properties and load displacement characteristics which will provide one or more of the three primary support functions, i.e. to reinforce the rockmass, to retain broken rock and to securely hold rock, loose rock or to tie back the retaining elements. Typical support elements are outlined in Table 2.10.

The load displacement performance of support elements should be considered in relation to rock mass deformation. High expected rock mass deformations need to be matched to high displacement limits. Table 2.11 presents load displacement parameters of support elements.

Table 2.10 Characteristic of Typical Support Elements (CAMIRO 1995)

Support Characteristic	Support Function		
	Reinforcing	**Retaining**	**Holding**
Stiff	Grouted rebar	Shotcrete	Grouted rebar
Soft	-	Mesh	Long mech. Bolt
Strong	Cable bolt	Reinforced shotcrete	Cable bolt
Weak	Thin rebar	No. 9 gauge mesh	Split set
Brittle	Grouted rebar	Plain shotcrete	Grouted rebar
Yielding	Cone bolt	Chain link mesh	Yielding swellex

Table 2.11 Load Displacement Parameters of Support Elements (CAMIRO 1995)

Description	Peak Load (kN)	Displacement Limit (mm)	Energy Absorption (kJ)
19mm resin grouted rebar	120-170	10-30	1-4
16 mm cable bolt	160-240	20-40	2-6
16mm 2m mechanical bolt	70-120	20-50	2-4
16mm 4m debonded cable bolt	160-240	30-50	4-8
16mm grouted smooth bar	70-120	50-100	4-10
split set bolt	50-100	80-200	5-15
Yielding Swellex	80-90	100-150	8-12
Yielding Super Swellex	180-190	100-150	18-25
6mm cone bolt	90-140	100-200	10-25
No. 6 gauge weld mesh	24-28	125-200	2-4/m2
No. 4 gauge weld mesh	34-42	150-225	3-6/m2
No. 9 gauge chain link mesh	32-38	350-450	3-10/m2
shotcrete and weld mesh	2xmesh	mesh	3-5xmesh

Table 2.12 Examples of Support Types

Type	Examples and Variations	Advantages	Disadvantages
Mechanical Rockbolts	Different mechanical shells available. Forged or nut type heads.	Immediate active support	Shells only perform within a limited range of hole diameter. Susceptible to corrosion.
Friction Stabilizers	Split Sets, mild steel, galvanised, Stainless steel	Immediate support and reinforcing	Hole diameter critical to effectiveness. Susceptible to corrosion
Swellex	Mild steel, coated, stainless steel, and yielding	Immediate support and reinforcing. Initial small diameter can facilitate installation in very poor ground and long bolts from small excavations	Hole diameter critical to effectiveness. Relies on use of pump. Susceptible to corrosion
Hollow Groutable Bolts	Ingersoll Rand, Stelpipe	Immediate active support followed by longer term full column support.	Installation through mesh with jumbos can be awkward. Grouting can be difficult (especially with Stelpipes).
Combination Bolts	CT-Bolts	Immediate active support followed by longer term full column support.	Installation can be difficult through mesh with jumbos. Grouting can also be difficult.
Rebar	Mild steel, high tensile steel, continuous threading, resin or cement grout	High tensile capacity	Only effective once grout sets
Smooth Bar	Resin or cement grout	Capable of limited yield with cement grout	Only effective once grout sets

Type	Examples and Variations	Advantages	Disadvantages
De-stranded Hoist Rope	De-greased or not	Strong, yielding reinforcing	Labour intensive manufacture
Cone Bolts	Strata Control Systems, installed with cement grout and plate	Wax coated smooth bar, cone designed to yield through grout	Only performs with plate attached
Yielding Cable Bolts	Freyssinet	Mechanical, steel on steel mechanism	Relatively expensive
Cable Bolts	Plain strand, bulbed, de-bonded, multi-strand, mechanical, grouted, tensioned	Relatively cheap, strong reinforcing. Lengths cut to design requirements on site.	Requires specialist grouting equipment. Can corrode from centre of cable
Hoist Ropes	Locked coil, stranded, solid, hemp core	Very strong, yieldability dependent on extent of degreasing and rope core	Labour intensive installation, supply limited
Straps	Flat, W, mesh, cable	Used with or without mesh	Only useful for limited range of block size
Plates and Washers	Flat, domed, mesh, large/small	Domed plates provide limited initial yield capability and angled installation	

Type	Examples and Variations	Advantages	Disadvantages
Timber Sets	Round poles, cut timber		Strength dependent on timber type, diameter and installation relative to load. Unsuitable for trackless working areas
Steel Sets	Leg bracing, Timber shielding, timber lagging, expanding foam filling	Visible performance. Very strong capacities available.	Time consuming installation. Relatively expensive.
Type	**Examples and Variations**	**Advantages**	**Disadvantages**
Steel Arches	Yielding	Visible performance, high load bearing capacities available	Limited range for excavation size variation
Concrete Lining	Pre-formed (e.g. ARMCO), cast in-situ	Pre-determined strengths	Bulky to move and relatively difficult to install
Shotcrete	With/without mesh	Can be pre-mixed and delivered where required. Reference 'Design of Shotcrete Support' section in 'Underground Excavations in Hard Rock'.	Susceptible to failure in high deformations. Quality governed by operator skills

Type	Examples and Variations	Advantages	Disadvantages
Fibrecrete	Variation in fibre performance	Remains relatively intact following relatively high deformation	Relatively expensive
Polymers	Everbond, Mineguard	Thin, rapid setting	Relatively unproven, some are toxic.
Mesh	Diamond, Welded, Expanded	Varying strengths, yieldable, plain, galvanized, stainless steel	Can be time consuming to install
Lacing	Various patterns, installed in combination with mesh and bolts (with eyes/shepherd crooks)	Strong, suitable for large scale deformation and seismic areas	Labour intensive.
Cable Trusses	Various designs, fully grouted, mechanical shells, yielding	Strong surface support when used in combination with mesh	Requires extensive blocking /lagging when rock surface is concave
Cable Slings	Cables installed with split sets or Swellex	Can be installed without specialised equipment	Limited application
Timber Props	Poles, Cut timber	No specialised equipment required	Labour intensive

Type	Examples and Variations	Advantages	Disadvantages
Mechanical Props	Acrow Props, Camlok Props Screw, jacking, yielding, with/without headboards or load spreaders	Pre-set loads, yieldable, re-usable	Unsuitable for mechanised mining
Hydraulic Props	100kN, 200kN, 400kN, 1600kN, varying yield capacities (closure and velocities)	Pre-set loads, re-usable	Only suitable for narrow excavations. Capital intensive
Timber packs	Matpacks, combined elongate/slab packs	Suitable for large scale deformations	Labour intensive. Shrink with time. Fire risk unless treated. Only for narrow excavations
Grout packs	Cement/backfill grout	Can be tailored and designed to suit load/deformation requirements	Labour intensive plus requires a grout supply system
Composite Packs	Brick and timber	Can be designed to suit requirements	Labour Intensive
Cement Grout	Low Heat, pumped (pre-or post), cement capsules	Far cheaper than resin	Quality affected by water: cement ratio, quality of water and installation
Resin grout	Slow, medium, fast	Effective. Can be installed fast at end of hole, slow to collar, allowing full column grout with tensioning.	Expensive, diameter of capsule, bolt and hole must be matched. Requires special storage facilities.

2.6 Defect Shear Strength

An estimation of the shear strength of structural discontinuities or joints is used in the assessment of structural sliding in open pit walls or block stability in underground openings. The shear strength of a defect is typically expressed in terms of a Mohr-Coulomb relationship of friction and cohesion. These are selected for the normal stress environment that is appropriate for the analysis. These shear strength parameters can be determined directly, at a sample scale, from laboratory shear box testing, or alternatively estimated from defect parameters derived from logging.

Defect shear strength, which is based on geotechnical logging parameters, can be estimated using Barton's non-linear failure criterion. Barton's Criterion is empirical and is defined as follows:

$$\tau = \sigma_n \mathrm{Tan}\,(\phi_{basic} + \mathrm{JRC.Log}_{10}\,(\mathrm{JCS}/\sigma_n)) \qquad (2.19)$$

Where;

τ	=	defect shear strength (MPa)
σ_n	=	normal stress acting on the defect surface (MPa)
ϕ_{basic}	=	defect basic friction angle (degrees)
JRC	=	Joint Roughness Coefficient (dimensionless)
JCS	=	Joint wall compressive strength (MPa)

Barton's criterion provides a non-linear relationship between the shear strength of, and the normal stress acting on the defect, based on the following considerations:

- JRC is based on the defect conditions obtained from core logging. This should be downgraded, where appropriate, to reflect the larger scale surface profile and possibility of blast damage,
- JCS is typically the UCS estimated from field logging codes or laboratory testing of the material representative of the joint wall,
- Basic Friction Angle is based on estimates,
- Normal stresses that act on the defects in the vicinity of open pit and underground excavation surfaces are typically low, and are usually based on a simple geometrical assessment of the loading due to, the dip of the defect, its proximity to the excavation surface and the stresses in the vicinity of the excavation.

The resulting, equivalent, Mohr-Coulomb defect shear strength parameters of friction and cohesion are estimated from a line tangential to Barton non-linear relationship over the range of normal stress of interest as indicated in Figure 2.11.

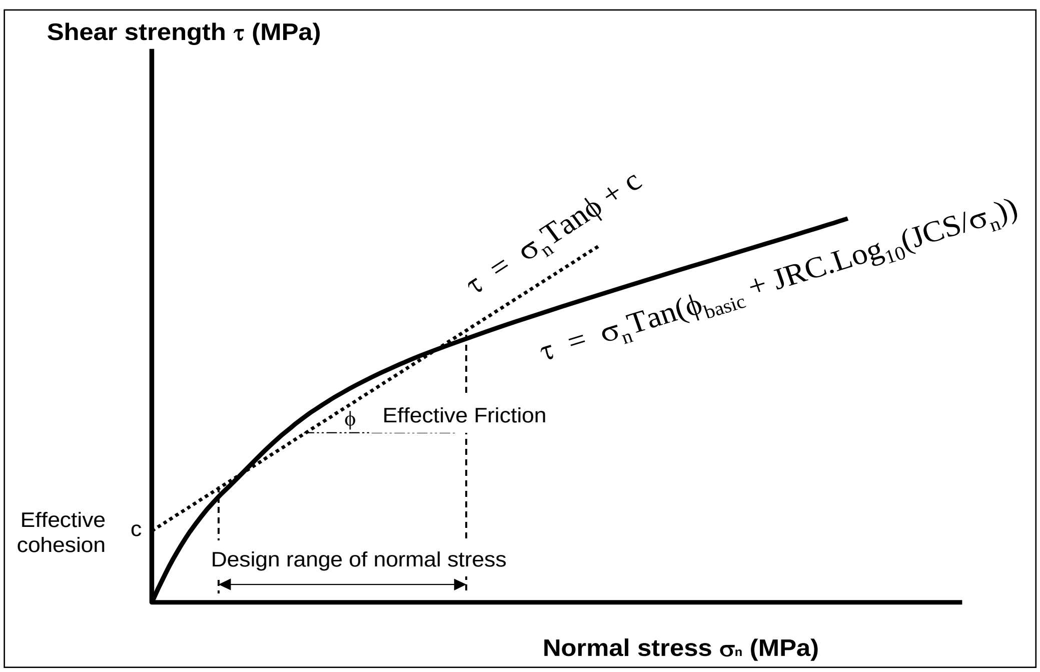

Figure 2.11 Line Tangential / Barton non-linear relationship (after Barton 1994)

For small rock mass volumes close to the excavation surface (low normal stress) it is often conservatively assumed, for purpose of stability assessments, that the cohesive component of the shear strength is zero.

2.6.1 Rockmass Strength and Deformation

For the assessment of overall rock mass stability in open pit slopes or the competency of the rock mass around underground excavations an estimate of the strength and deformation characteristics of the global rock mass is necessary. These characteristics are associated with the large scale behaviour of the rock mass and as such it is often not practical to establish these properties from laboratory testing.

The empirical relationships based on rock mass classifications have been derived to aid in these design study. The equations presented in the following sections are readily solved with available software packages, and are presented here to provide a background understanding to these systems.

2.6.1.1 Rockmass Strength

Hoek and Brown (1993) proposed a failure criterion for jointed rock masses based on an assessment of the interlocking rock blocks and the condition of the surfaces between these blocks. The most general form of the Hoek-Brown criterion for jointed rock masses is;

$$\sigma_1' = \sigma_3' + \sigma_c \{m_b (\sigma_3'/\sigma_c) + s\}^{a} \qquad (2.20)$$

Where:

m_b (a reduced value of m_i for the rock mass) s and a, are constants dependent on the rock mass classification.

m_b is a function representing (1) the angle of internal friction and, (2) Particle interlocking.

s is a function representing - Inter-particle tensile strength and Particle interlocking.

σ_c is the uniaxial compressive strength.

σ_1' and σ_3' are the major and minor principle stresses at failure.

m_b, s and a are difficult to calculate and are generally estimated based on rock mass rating methods as follows;

$$m_b = m_i.\exp\left[(GSI-100)/(28-14D)\right]. \quad (2.21)$$

$$s = \exp\left[(GSI-100)/(9-3D)\right] \text{ and} \quad (2.22)$$

$$a = {}^{1}/_{2} + {}^{1}/_{6}\left(e^{-GSI/15} - e^{-20/3}\right) \quad (2.23)$$

Where:

D = disturbance of the rock mass, which varies from 0 for undisturbed in situ rock mass to 1 for very disturbed rock masses; and mi for intact rock, by rock group after Hoek and Brown, 1993 as shown in Table 2.13.

GSI = Geological Strength Index.

Table 2.13 Values of the constant mi, for intact rock (after Hoek and Brown 1993)

Rock Type	Class	Group	Texture			
			Coarse	**Medium**	**Fine**	**Very Fine**
SEDIMENTARY	Clastic		Conglomerates (21 ± 3) Breccias (19 ± 5)	Sandstones 17 ± 4	Siltstones 7 ± 2 Greywackes (18 ± 4)	Claystones (4 ± 2) Shales (6 ± 2) Marls (7 ± 2)
	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestone (10 ± 2)	Micritic Limestones (9 ± 2)	Dolomites (9 ± 3)
		Evaporites		Gypsum 8 ± 2	Anhydrite 12 ± 2	
		Organic				Chalk 7 ± 2
METAMORPHIC	Nonfoliated		Marble 9 ± 3	Hornfels (19 ± 4) Metasandstone (19 ± 3)	Quartzites 20 ± 3	
	Slightly foliated		Migmatite (29 ± 3)	Amphibolites (26 ± 6)		
	Foliated*		Gneiss 28 ± 5	Schists 12 ± 3	Phyllites (7 ± 3)	Slates 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3	Diorite 25 ± 5		
			Granodiorite (29 ± 3)			
		Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3) Basalt (25 ± 5)	Obsidian (19 ± 3)
		Pyroclastic	Agglomerate (19 ± 3)	Breccia (19 ± 5)	Tuff (13 ± 5)	

Note that values in parenthesis are estimates.
* These values are for intact rock specimens tested normal to bedding or foliation. The value of m_i will be significantly different if failure occurs along a weakness plane.

The uniaxial compressive strength of the rock mass may be estimated by setting σ_3 to zero in the general form of the Hoek-Brown equation. The tensile strength of the rock mass is estimated from the relationship:

$$\sigma_t = s.\sigma_{ci}/m_b \qquad (2.23)$$

The Hoek-Brown failure criterion produces a non-linear relationship between σ_1 and σ_3, whereas many analysis techniques are based on estimates of rock mass friction and cohesion derived from the linear Mohr-Coulomb failure criterion. Equivalent friction angle and cohesive strength are estimated by the fitting of an average linear relationship to the Hoek-Brown criterion over the σ_3 range $\sigma_t < \sigma_3 < \sigma_{3max}$, where σ_{3max} is typically estimated from excavation depth and stress, or slope height, for the global rock mass strength (discussed below). For general design purposes it is useful to use a 'global rock mass strength' (σ_{cm}). This is estimated for the stress range $\sigma_t < \sigma_3 < 0.25.\sigma_{ci}$ and is expressed as:

$$\sigma_{cm} = \sigma_{ci}\,(m_b + 4s - a\,(m_b - 8s))\,(m_b/4 + s)^{a-1} / (2.\,(1+a)(2+a)) \qquad (2.24)$$

These strength parameters are usually calculated by software packages and either reported or used directly in numerical analyses. An approximate estimate of the rock mass strength is given by:

$$\sigma_{cm} = 0.019\sigma_{ci}.e^{0.05GSI} \qquad (2.25)$$

Design charts are also available, which present the relationships between GSI and m_i and/or σ_{ci} to estimate the rock mass friction angle, cohesive strength, global rock mass strength and deformation modulus for tunnel depths greater than 30m. The MRMR classification system has also been developed to provide an estimate of the rock mass strength. This is expressed either as the rock mass strength (RMS), which can be considered as equivalent to the previously discussed global rock mass strength, or in terms of the design rock mass strength (DRMS), which is equivalent to the rock mass UCS.

The RMS is defined as the intact rock strength (IRS), downgrade to 80% of its value for scale effect, and then downgraded for the influence of defects. The RMS rating is thus given by:

$$RMS = IRS \times 0.8 \times (RMR\ rating - IRS\ rating)/80 \qquad (2.26)$$

The DRMS is the RMS adjusted for weathering, orientation and excavation method and is given by:

$$DRMS = RMS \times weathering\ adj.\ (\%) \times orientation\ adj.\ (\%) \times excavation\ adj\ (\%) \qquad (2.27)$$

2.6.3 Rockmass Deformation

Generally, it is impractical to determine the site-specific rock mass modulus. Hence empirical estimation methods are again generally used. The following is a relationship that is typically applied;

$$E_m = (1-{}^{D}/_{2})\ \sqrt{(\sigma_{ci}/100)}.10^{(RMR-10)/40} \text{ for } \sigma_{ci} < 100\text{; or}$$

$$E_m = (1-{}^{D}/_{2})\ 10^{\ (RMR-10)/40} \text{ for } \sigma_{ci} >= 100 \qquad (2.28)$$

Where;

σcm = Uniaxial compressive strength for the rockmass (MPa)
σ_{ci} = Uniaxial compressive strength for intact rock (MPa)
E_m = Rock mass deformation modulus (GPa)
RMR = Rock mass rating
D = Disturbance factor

From the above relationship it should be noted that the principal factors in determining the rock mass modulus are the rock mass classification and estimation of the degree of disturbance.

It may also be necessary to estimate a Poisson's Ratio of the rock mass. This will be dependent on the intact rock properties and the frequency and characteristics of the defects within the rock mass. In practice only a very rough estimate can be made and would typically vary from 0.2 for very good quality rock masses to 0.3 for very poor quality rock masses.

2.7 Estimation of Support Pressure Utilizing Rockmass Quality (Q)

The support pressure capacity of tensioned or grouted bolts (P) is equal to the yield capacity of one bolt (Y, if adequately anchored) divided by the square of the spacing (S), expressed as follows:

$$P=Y/S^2 \qquad (2.29)$$

Where:

- P = support pressure capacity, kPa.
- Y = yield capacity of one bolt, kN.
- S = spacing between bolts, m.

An empirical equation to relate permanent support pressure and Q is given as follows:

$$P_{roof} = (66.7\ (Jn)^{0.5}/Jr) \times Q^{0.3} \qquad (2.30)$$

$$P_{wall} = (66.7\ (Jn)^{0.5}/Jr) \times Q_w^{0.3} \qquad (2.31)$$

Where:

- P_{roof} = permanent roof support pressure, (kPa)
- P_{wall} = permanent wall support pressure, (kPa)
- Jn = Joint set number.
- Jr = Joint roughness number.
- Q = Rock mass quality.

- Q_w = Wall factor (= 1.0Q up to 5Q)

2.8 Stacey and Page Modified Q for Raise Boring

Stacey and Page (1986) developed a rock mass classification system for large bored shafts and raises based on a modification of Q (Rock mass quality). In summary, Q is estimated conventionally and is then multiplied by a number of factors reflecting:

- The intrinsic stability of the sub-vertical, curved sidewall,
- The orientation of defects with respect to sidewall and crown, and
- The potential for progressive weathering of exposed and unsupported raise walls during and service life.

These adjustments are given in Tables 2.14 and 2.15, respectively. A wall adjustment is applied:

$$Q_{wall} = 2.5 \text{ x } Q \qquad \text{when } Q > 1 \text{ and} \qquad (2.32)$$

$$Q_{wall} = Q \qquad \text{when } Q < 1. \qquad (2.33)$$

Table 2.14 Adjustments for Joint Orientations

Face Orientation Adjustment				Wall Orientation Adjustment			
No of flat dipping (0^o-30^o) major joint sets	**1**	**2**	**3**	**No of steep dipping (60^0-90^0) major joint sets**	**1**	**2**	**3**
Adjustment	0.85	0.75	0.60	Adjustment	0.85	0.75	0.60

Table 2.15 Adjustments for Weathering

Weathering Description	Slight	Moderate	Severe
Adjustment	0.9	0.75	0.5

2.9 Monitoring of Loads and Pressures on Support

This is used to measure movements and pressures on support systems during construction and compared with predictions, to give warning of adverse or unpredicted behavior, and to allow timely remedial action. Based on the measurements, the support and excavation methods can be adjusted for economy or to enhance stability.

Besides retaining walls, tunnel liners, anchors, and rockbolts only develop their prescribed working pressures and loads as the rock starts to move against the retaining system. Attempts at predicting these pressures and loads can be unreliable, so monitoring is often specified to check and confirm the actual pressures that are developed.

2.9.1 Hydraulic Pressure Cells

The pneumatic/hydraulic type of pressure cell consists of a flat jack connected to a pneumatic or hydraulic transducer which in turn is connected by twin flexible plastic tubes to a terminal panel. Various sizes and shapes are available, from small rectangular cells for installation in a thin concrete tunnel lining to large circular flat jacks intended for installation in earth embankments and foundations.

The flat jack consists of two thin sheets of metal, welded around the perimeter and filled with oil or mercury to form a hydraulic pillow. It is connected by a short length of metal tube to the hydraulic transducer. Care is needed to avoid air pockets and stress raisers, so that the ground pressure perpendicular to the flat jack is equal to the hydraulic pressure in the jack.

This pressure is measured in the same way as described for the pneumatic piezometer, except that for higher pressures and over greater lengths of transmission tube, a mixture of oil and kerosene is used as the measuring fluid in place of air or nitrogen.

2.9.2 Extensometers

These instruments are used for monitoring the displacement between an anchor at the base (bottom) of the borehole (normally assumed to be stable) and the collar position. They are constructed of either wire or rods although a recent development has been the use of magnetic anchors (Sonic Probe). Normally a hand held meter or a data logger will read the extensometer although a useful development of the extensometer is the Tell Tale. This simple extensometer has a highly visual readout (of a single anchor) at the collar of the hole, usually colour coded to indicate zones of safe to unsafe displacement, which enables operators to easily assess any change in rock mass conditions.

2.9.3 Crack meters

These are typically manual devices, although they can be more sophisticated electronic instruments similar to surface extensometers. These instruments measure the displacement (normal and/or shear) across a defined joint or fault.

2.9.4 Electric Pressure Cells

These consist of two circular plates of steel, each with a central portion of reduced thickness that form a flexible diaphragm whose deflection is measured with electric resistance strain gauges or vibrating wires. In the vibrating-wire type of cell, metal pillars are mounted on the internal face at the circle of maximum angular deflection. External pressure causes a small angular rotation of the pillars, between which a vibrating wire is stretched under tension. The two halves are bolted together and waterproofed. Electric leads connect the cell to remote readout.

2.9.5 Load Cells

Load cells (also known as force transducers or dynamometers) are used to monitor the tension in cable anchors and rockbolts, and the loads in steel ribs, columns, and arches.

2.9.6 Convergence Indicators

The convergence indicator is made between two or more fixed points around the excavation perimeter; repeat monitoring provides rates of convergence, changes in rate and total convergence values. The advantages of this indicator are that, the instrument is simple, inexpensive and robust in an underground environment. The sensitivity is less than 0.1mm over a 10m span.

Figure 2.12 shows the high precision mechanical convergence measuring system.

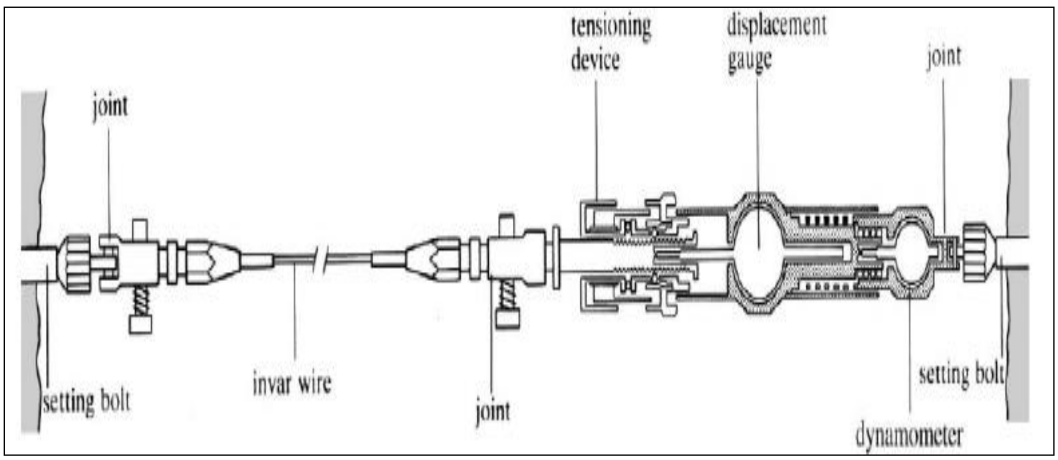

Figure 2.12 Distometer, a high precision mechanical convergence measuring system (after Brady and Brown 1993)

In addition to the above systems less routine monitoring may also include:

- **Stress Monitoring**

  This is usually measured as isolated stress measurement programmes but can be used to ‘continuously’ measure stress change. These are fairly specialised installations

- **Support/Reinforcement Monitoring**

  These instruments may include load cells installed to monitor the load developed in support systems or between the support and the rock face. In operations employing bulk slurry backfilling it may be routine to use this type of instrumentation to monitor the pressure between the backfill and the retaining structures. Reinforcement units such as cable bolts may be specially manufactured to contain instrumentation to monitor strain and load development within the cable, e.g. the SMART cable.

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CHAPTER 3

RESEARCH METHODOLOGY

This chapter discusses various methodologies that were undertaken in order to achieve the objectives. Collection of sufficient and relevant geotechnical data included:

1. Site investigation,
2. Desktop study,
3. Geophysical surveys,
4. Geotechnical drilling and logging of core,
5. Mapping of bench faces,
6. Software application for design purpose, and
7. Laboratory testing of rock samples.

3.1 Site Investigations

The purpose of initial site investigation for this study was to establish the feasibility of the project, which involved understanding of geology, hydrologeology and rock mass structures. The following information was vital in the understanding of the rock mass;

- Rock type,
- Depth and character of overburden,
- Major geological discontinuities such as joints and faults,
- Groundwater conditions, and
- Potential problems, such as weak ground or swelling rock.

3.2 Desktop Study

A desk study involved a detailed review of all available literature and previous reports. In addition, verbal discussions with local consultants and employees at various technical levels were helpful to provide vital related data to the study.

Several groundwater assessments were carried out in recent years, with two of the studies included developing and calibration of groundwater models. However, the following were the number of consultants that were engaged to investigate groundwater.

3.2.1. Water Management Consultants (WMC) (1998)

WMC's were engaged by a previous owner, Cyprus Amax Kansanshi Plc., to carry out pre-feasibility hydrogeology investigations and evaluate dewatering requirements for the mine. This report is not available, but is summarized in other reports. WMC also carried out data compilation from historic sources, exploratory drilling, field testing and analysis, water sampling and computer modelling.

A significant aspect of the WMC study was the completion of the first numerical groundwater model for the mine area. Transient calibration of the model was possible using the 1978 to 1982 pumping from the Main South Shaft. The hydrogeology data from this study as well as their specific conclusions regarding dewatering requirements were not present in the available report.

3.2.2 Groundwater Services (2002)

Groundwater Services, conducted a study beginning in 2001 to assess the available hydro- geological data and to evaluate the likely mine dewatering requirements for the proposed large open cut mine operations at Main Pit and North West Pit. Groundwater Services reviewed the work carried out by Water Mine Consultants and developed an updated numerical model for the mine area. The study was based on existing information, with no drilling or other significant field work undertaken.

Besides no indication of how the abstraction should be distributed or where boreholes should be sited was attempted due to limitations of available data and the associated uncertainty of the model. The total quantum of groundwater abstraction required for the Main Pit (MP) increases from 13,000 to100,000 m^3/d. Recommendations included drilling of test boreholes and piezometers to more accurately determine the local groundwater piezometry as well as aquifer hydraulic parameters. It was also recommended that the Main South Shaft be used in conjunction with new dewatering boreholes as part of the total scheme.

3.2.3 SRK Consulting (2004)

SRK Consulting completed a brief review of the Main South Shaft pumping, water level records and developed the outline of a programmed work for the development of a de-watering system for the mining operations.

3.3 Drilling and Logging of Borehole Cores

The purpose of geotechnical drilling was to confirm the geological interpretation, determine the quality and characteristics of the rock through geotechnical logging of the core. The rock mass was classified into geotechnical domains based on the relative strength parameters and discontinuity characteristics in the various geological formations. The logging exercise emphasised on the following basic parameters:

- Depth,
- Description (Rock Type),
- Core recovery,
- RQD,
- Strength of intact rock (UCS),
- Weathering or alteration,
- Fracture or discontinuity type,
- Fracture frequency,
- Dip angle of structure with respect to core axis and dip direction in case of

oriented core, and

- Discontinuity condition.

The format for describing the geotechnical characteristics of a rock mass was based on Barton's Q-system and rock mass rating (RMR).

3.4 Window Mapping

The mapping comprised geotechnical inspection of exposed excavations. The purposes of the mapping were to:

- Characterise structural orientation and defect conditions, and
- Locate major structures such as faults.

The geotechnical mapping defect data collected is presented in Appendix F. The mapping method employed was a combination of window mapping and scan line mapping to determine RMR. The defect included:

- Estimated field strength (UCS),
- Defect orientation,
- Lihtology type,
- Defect length and continuity,
- Defect shape,
- Joint surface condition,
- Rock quality designation (RQD),
- Degree of Weathering, and
- Groundwater water conditions.

In total, 195 individual structures were mapped. Stereoplots of the data were included in Appendix G.

3.5 Computer Aided Designing

In this research the structural data collected was analyzed using the Rocscience software such as Dips, Unwedge and numerical modelling (Rocscience 2011). Though, there are a number of software packages available to determine structural orientation, wedge sizes, support requirements and factors of safety.

3.5.1 Dips

Orientations of discontinuities were analysed using Dips software. The data was analysed using spherical projection techniques to determine the kinematic failure of structural orientation.

This software utilised the inputs from the structural joint analysis program Dips. The geometry of an excavation was entered into the program either manually, or via a dxf, and various support options were assessed.

3.5.3 Phase2

Geotechnical simulation package such as Phase2 was used to assess the applicability and response of support to changing stress environments. These analyses will generally require a relatively long time to conduct and the accuracy of the results will be controlled by the rock mass properties. Rock mass properties at the periphery of excavations are not easily determined and hence the response of the rock mass alone to stress changes is difficult, even without consideration of support.

The programs can indicate the relative difference in deformation expected for unsupported and/or unreinforced excavations as opposed to reinforced and/or supported excavations.

A comparison between numerical and limit equilibrium methods are presented in Table 3.1.

Table 3.1 Comparison of numerical and limit equilibrium analysis methods (Itasca 2003)

Analysis result	Numerical solution	Limit equilibrium
Equilibrium	Satisfied every where	Satisfied only for specific objects, such as slices
Stresses	Computed everywhere using field equations	Computed approximately on certain surfaces
Deformation failure	Part of the solution yield condition satisfied everywhere; slide surfaces develop ‘’ automatically’’ as conditions dictate.	Not considered. Failure allowed only on certain predefined surfaces; no check on yield condition elsewhere.
Kinematics	Satisfies kinematic constraints.	A single kinematic condition is specified according to the particular geologic conditions.

CHAPTER 4

DATA COLLECTION

A number of research methodologies were used during data collection and the presentation is as discussed in following section.

4.1 Laboratory Testing of Rock Samples

Undisturbed block samples from the saprolite unit were collected. These samples were sent to South Africa and submitted to Civilab in Johannesburg. Geotechnical testing of the samples included the following:

- Foundation and indicator testing,
- Consolidated drained triaxial testing, and
- Soaked, consolidated drained triaxial testing.

The saproilite was characterised by a deep weathered profile ranging between extremely and highly weathered, and is controlled by both the parent rock and major structural features. Relic discontinuity structures, such as bedding/foliation are retained within the weathered fabric, with surface infilling and smooth/planar shape discontinuity profile.

The intact strength is generally less than 15MPa and frequently less than 5MPa. According to the British Soil Classification System for engineering purposes, the Kansanshi saprolites classify as an ML soil and can be described as a silt. The Plasticity Index and Liquid Limit is non-plastic to slightly plastic. The results of the triaxial and shear box testing are summarized in Table 4.1.

Table 4.1 Summary of Shear Strength Testing Results

Sample Number	Test Type	Cohesion (kPa)	Friction Angle
KAN S6	Consolidated Drained Triaxial	13	32
KAN S1	Consolidated Drained Test Shear Box	22	23
KAN S4	Consolidated Drained Test Shear Box	87	21

Where:

KAN S (Number) is the sample ID

Furthermore, standard testing on core included specific rock properties such as unixial compressive strength (UCS), and direct shear strength of rock defects. Table 4.2 presents the summary of laboratory results parameters.

Table 4.2 Kansanshi Geotechnical Units (Golders Pit Slope Review Report 2009)

Parameters	Mean	Lower Bound	Upper Bound	Distribution
Saprolite				
Internal Friction Angle, φ_i, degree	30	27	33	Normal
Cohesion, c, kPa	9.47	1	19	Normal
Unit Weight, kM/ m^3	19	17	21	Normal
Clastics (Knotted schist, Phylites, Carbonaceous Phylites)				
UCS, MPa	82	18	146	Normal
GSI	53	1	88	Normal
mi	8	6	10	Normal
Unit Weight, kM/ m^3	26			
Marble				
UCS, MPa	86	72	116	Normal
GSI	61	6	90	Normal
mi	9	-	-	
Unit Weight, kM/ m^3	26	-	-	

The saprolite unit had been preliminary classified, according to the British Soil Classification System for engineering purposes, as a SM-SC soil, which is either well or gap graded, with less than 35 % of silt or clay content. Effective stress conditions had been considered in the saprolite analyses. Fully saturated and completely drained conditions were analysed for, demonstrating the sensitivity of the saprolite slopes to changes in pore pressure. Fully saturated slopes were simulated using a pore pressure ratio of 0.11.

Drained conditions were assumed to have a pore pressure of zero. In clastics and marble units, fully saturated slopes were simulated using a pore pressure ratio of 0.16. Drained conditions were assumed to have a pore pressure of zero.

4.2 Core Logging and Recovery

The core was divided into separate geotechnical zones, within which the core displayed similar characteristics. The following characteristics are presented in for each unit in a summarised Table 4.2.

- Rock type, weathering and alteration,
- Rock quality designation (RQD),
- Geological strength index (GSI), and
- Q – System parameters.

Core recovery was expressed as the length of recovered core or compared to the drilled length and expressed as a percentage. Appendix D indicates the summarised rock quality designation (RQD) and core covered for borehole DCDD05A (DCDD – Borehole ID and 05 – ID number).

Figure 4.1 Core recovered for borehole DCDD05

The geological strength index (GSI) was calculated from Bieniawski's RMR given by the following equation (4.1) according to Hoek (2000).

$$GSI = RMR_{89} - 5 \qquad 4.1$$

Where:

RMR_{89} is Rock Mass Rating based on Bieniawski's (1989)

Table 4.2 Summarised of Geotechnical Core Logged Data

(a) Knotted Schist	RQD (%)	RMR (%)	GSI	Q_RATING
Average	10.85	57.76	52.76	9.8994
maximum	20	88	83	75.733
minimum	3	31	26	0
Standard Deviation (%)	6.874	19.2	19.2	11.289
(c) Marble	**RQD (%)**	**RMR (%)**	**GSI**	**Q_RATING**
Average	14.62	67.04	64.73	33.72
maximum	20	92	87	213.33
minimum	3	39	34	0.015
Standard Deviation (%)	6.118	13.5	13.48	57.989
(e) Phylites	**RQD (%)**	**RMR (%)**	**GSI**	**Q_RATING**
Average	6.804	48.52	44.17	1.159
maximum	20	78	73	8.85
minimum	3	32	27	0.0025
Standard Deviation (%)	5.969	12.97	13.58	2.2582
(g) Carbonaceous Phylites	**RQD (%)**	**RMR (%)**	**GSI**	**Q_RATING**
Average	5.727	45.81	41.72	1.6312
maximum	20	78	73	29.2
minimum	3	33.1	28.1	0.0013
Standard Deviation (%)	5.526	10.56	11.86	5.0429

(b) Residual Marble	RQD (%)	RMR	GSI	Q_RATING
Average	4.73	41.24	36.2	0.3405
maximum	20	73.1	68.1	5.6
minimum	3	34.1	29.1	0.0013
Standard Deviation (%)	4.34	6.862	6.86	1.0089
(d) Calcareous Biotite Schist	**RQD (%)**	**RMR**	**GSI**	**Q_RATING**
Average	10.9	54.78	49.8	1.63
maximum	17	75	70	3.36
minimum	3	39	34	0.02
Standard Deviation (%)	6.42	11.59	11.6	1.5309
(f) Biotite Schist	**RQD (%)**	**RMR**	**GSI**	**Q_RATING**
Average	9.56	58.4	53.4	6.5422
maximum	20	77	72	25.867
minimum	3	37.1	32.1	0.0075
Standard Deviation (%)	6.01	14.31	14.3	7.7663
(h) Pebble Schist	**RQD (%)**	**RMR**	**GSI**	**Q_RATING**
Average	17.4	75.44	70.4	0
maximum	20	78	73	0
minimum	13	71	66	0
Standard Deviation (%)	2.88	2.877	2.88	0

4.3 Hydrogeological Conceptual Model

A review of several hydrogeological data obtained suggested that the aquifers are developed in the fractured hales, phyllites and schists and the Middle and Upper Mixed Clastic members of the Kansanshi Mine Formation. The Marble is massive and widely fractured with evidence of karst and cavity formation only in shallow zones primarily above the water table.

The rock types encountered at Kansanshi have low primary permeability and the rock fabric itself inhibits flow. The main flow is still through a secondary permeability mainly veins, fractures in the clastic rocks and associated cavities in the marble. There is a good hydraulic continuity between the various formations through sub-vertical fractures and weathering that crosses geological boundaries. The integration of hydrogeological and hydrochemical data was necessary to establish a preliminary conceptual model for Kansanshi mine.

4.3.1 Hydrostratigraphic Units and Hydraulic Properties

The major hydrostratigraphic units at Kansanshi Mine are:

- **Weathered shallow aquifer**

  The shallow aquifer is unconfined and exhibits primary porosity characteristics due to the weathered sandy and clay matrix. Fracture porosity is significant at the base of the aquifer due to fractured saprolitic bedrock. The weathered sand and clay matrix and the fractured regolith/sap rock form a single hydrostratigraphic unit. From borehole drilling results, in many places, the base of the aquifer is well represented by fresh massive upper mixed clasts phyllites and schists.

  The depth of the shallow weathered aquifer is from 40m to 60m, with water levels less than 30m. Head differences between the shallow aquifer and deeper aquifers, was shown with borehole MDW09. Water strike within the shallow weathered aquifer was encountered at 18m to 21m. A rest water level of 10m was measured, with a blowout yield of 0.2m^3/hr. Water strikes in the deeper aquifer were encountered from 120m within cavities in the upper marble and calcite vein. Rest water level for the deeper aquifer was 109m, with a blowout yield of $3m^3$/hr. Between the two aquifers, fresh solid upper mixed clasts, phyllite, knotted schist and biotite schist were encountered, completely separating the two units.

The major structures, such as the 4800m and 5400m zones, the depth of weathering are in excess of 100m and can reach 200m. Along these zones, there is hydraulic continuity among all the various hydrostratigraphic units.

- **Upper Mixed Clasts (UMC)**

Most water strikes within the upper mixed clasts and the middle mixed clasts are associated with calcite-quartz veins and highly fractured. This aquifer is characterised by fracture porosity and ground water potential is governed by the amount of fracture connectivity.

The aquifer exhibits confined conditions with transmissivity from $30m^2$/day to 350 m^2/day. Aquifer storage coefficients or storativity derived from test pumping data, ranged from $4x10^{-4}$ to $5x10^{-3}$, indicating confined conditions. However, there is no major difference in aquifer hydraulic characteristics between the upper mixed clasts and the middle mixed clasts.

Note: Storage coefficient or storativity (S) is defined as volume of water released from storage per unit surface area of the aquifer per unit decline in the component of hydraulic head normal to that surface.

- **Upper Marble (UM), Middle Mixed Clasts (MMC) and Lower Calcareous Unit (LCU)**

To identify the aquifers tested, no water strike records were available from the boreholes test pumped by WRC. However from some of the boreholes used to construct a geological cross-section striking NS from the NW pit to the Main pit, water strikes were encountered in the UMC, the UM and the MMC. Although some boreholes struck water in the UM from the NW pit dewatering borehole drilling, aquifer hydraulic characteristics of the UM are available from only one borehole NWDW04, as the others have not yet been tested. The transmissivity, calculated from the data, was $33m^2$/day from constant pumping testing (CRT) and $74m^2$/day from recovery data. Aquifer storage coefficients calculated from two observation boreholes NWDW03 and NWDW05 was $2x10^{-4}$. Though, the data was often distorted by response to abstraction from the NW pit sump. The results from test pumping indicated there is hydraulic continuity between the UMC, UM and MMC via sub-vertical fractures and quartz-calcite veins and fractures.

- **Lower Marble (LM)**

Preliminary long-term test pumping data from the main pit dewatering boreholes indicated a highly permeable and high storage in the Lower Marble aquifer. Groundwater is stored in cavities and weathered zones in the marble. In addition to these cavities being groundwater storage and permeable zones, permeability is further enhanced by sub-vertical to vertical veins and structures, creating a lattice type aquifer with connection to the upper aquifers. The transmissivity obtained from test pumping was $500m^2$/day to $2650m^2$/day, with aquifer storativity of 0.014 to 0.034.

The high groundwater potential of the LM is supported by that. Despite a total abstraction of 40 $000m^3$/day to 50 $000m^3$/day for all the pumping nodes from the main pit, in five months, only a maximum of 10m of drawdown was observed from boreholes in the main pit. This gives a maximum drawdown rate in the pressure head of 2m/month (0.6m/day).

- **Lower Pebble Schist (LPS)**

The pebble schist forms the lower boundary of the LM aquifer. This unit is >100m thick and could be an impermeable layer between the LM and the Roan dolomite. The Roan dolomite is a well-known, very high yielding aquifer. No test data is available for this zone.

To date two main conduits for groundwater flow were identified, the 4800 zone and 5400 zone. These are wide (100m scale) zones of deep weathering associated with high densities of North-South trending veins. In places the weathering extends through the Lower Marble into the Lower Pebble Schist. On a regional scale the main groundwater flow direction was from the North (Congo-Zambezi water shed) to the South, following the topography. Appendix C indicates the main components of the conceptual model; including groundwater elevation contours, the deeply weathered zones that act as groundwater conduits, and the interpreted fault network.

4.4 Decline Conceptual Design

The concept of the decline is to create a portal in the existing pit wall, spiral down to just above the water table and then develop tunnels approximately 4m x 4.5m parallel to the main water bearing structures, the 4800 and 5400 zone. Tunnels are to be developed to the north and south and vertical holes will be drilled from the main tunnel to intercept the main water bearing structures.

The vertical 1.2m diameter holes will be drilled to intersect the conduits, through which the extracted groundwater may be pumped. Figure 4.2 indicates decline conceptual design and an estimated target depth of the decline is approximately 35m below the final pit bottom.

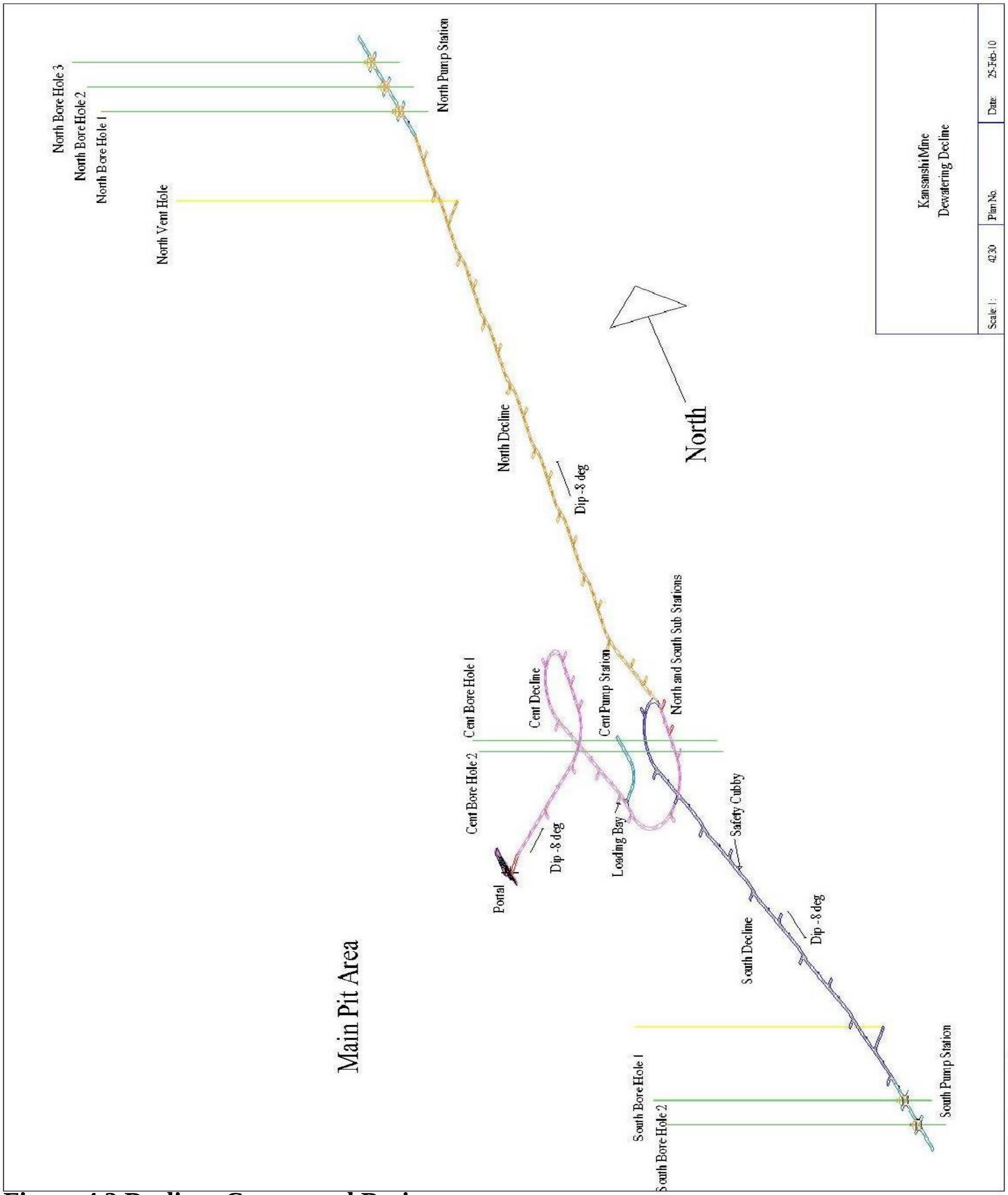

Figure 4.2 Declines Conceptual Design

4.5 Kansanshi Rainfall Data

Kansanshi project is located within a region having typical wet-dry climate. Solwezi experiences seasonal variation and maximum short deviation precipitation rates. Figure 4.3 shows 3 years of rainfall data for Kansanshi mine. The rainfall is recorded at five rainfall gauges at respectively; the open pit, mine plant, Kansanshi weather station, tailings storage facility and the guest house. From 2009 to 2011, yearly rainfall ranged from 1285mm to 1452mm. Estimated long-term average yearly rainfall is 1400mm, with daily rainfall as high as 90mm.

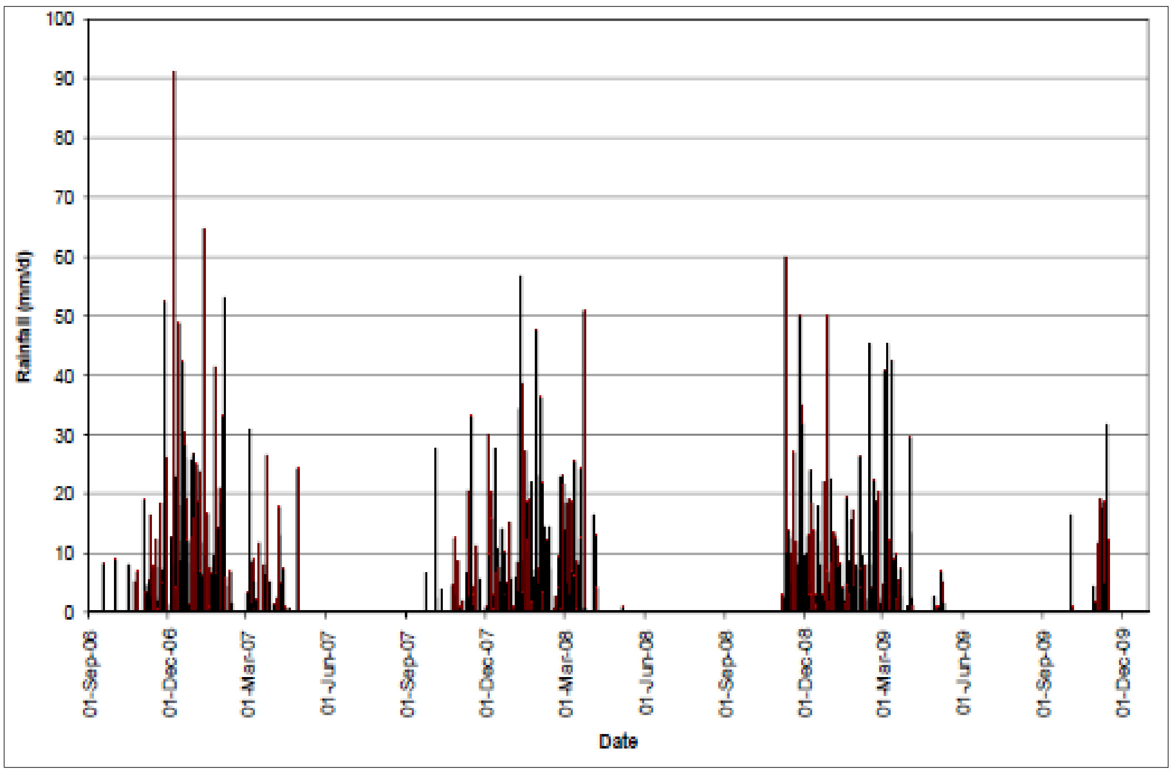

Figure 4.3 Average Annual Rainfall Data

Chapter 5

DATA ANALYSIS AND DISCUSSION OF RESULTS

5.1 Geotechnical Bench Mapping Results

The average results obtained from window mapping are presented in Table 5.1 and structural mapping data is presented in Appendix F and stereographic projection in the Appendix G.

Table 5.1 Summary of RMR and Q-rating from window mapping

Ratings	RMR (%)	Q-System
Average	40	0.86
Maximum	54	3.75
Minimum	27	0.05
Standard Deviation (%)	5.4	0.73

The results in Table 5.1 shows, rock mass ratings (RMR) were in the range approximately 40 to 60. Thus the rock is described as fair to good ground that requires 3m threaded rock bolts, spaced 1.5m to 2m in the crown and walls with diamond wire mesh, and 50mm to 100mm fibre reinforced shortcrete in the crown and 30mm in the sides as described in Table 5.2.

5.1.1 RMR and Q-System Relationship

A correlation between the RMR and Q-Systems was made using the following equation:

$$Q = e^{((RMR-44)/9)} \qquad (5.1)$$

Therefore, using the average RMR (40%), the Q-system value predicted by the equation (5.1) is 0.6. The window mapping provided the Q-System value of 0.86. This was considered a good correlation and concluded that the quality of geotechnical mapping was consistent between the two classification systems.

Table 5.2 Excavation Technique and Support Selection Based On RMR System (Bieniawski, 1989)

<table>
<tr><th colspan="5">Shape: horseshoe; width: 10 m; vertical stress: below 25 MPa; construction: drill and blast</th></tr>
<tr><td rowspan="2">Rock mass class</td><td rowspan="2">Excavation</td><td colspan="3">Support</td></tr>
<tr><td>Rock bolts (20 mm dia. fully bonded)</td><td>Shotcrete</td><td>Steel sets</td></tr>
<tr><td>Very good rock
I
RMR:81 - 100</td><td>Full face
3 m advance</td><td colspan="3">Generally no support required
except for occasional spot bolting</td></tr>
<tr><td>Good rock
II
RMR: 61 - 80</td><td>Full face
1.0 - 1.5 m advance
Complete support 20 m from face</td><td>Locally bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh</td><td>50 mm in crown where required</td><td>None</td></tr>
<tr><td>Fair rock
III
RMR: 41 - 60</td><td>Top heading and bench
1.5 - 3 m advance in top heading. Commence support after each blast. Complete support 10 m from face</td><td>Systematic bolts 4 m long spaced 1.5 m - 2 m in crown and walls with wire mesh in crown.</td><td>50 - 100 mm in crown and 30 mm in sides</td><td>None</td></tr>
<tr><td>Poor rock
IV
RMR: 21 - 40</td><td>Top heading and bench
1 - 1.5 m advance in top heading. Install support concurrently with excavation 10 m from face</td><td>Systematic bolts 4 - 5 m long, spaced 1 - 1.5 m in crown and walls with wire mesh</td><td>100 - 150 mm in crown and 100 mm in sides</td><td>Light to medium ribs spaced 1.5 m where required.</td></tr>
<tr><td>Very poor rock
V
RMR: < 20</td><td>Multiple drifts. 0.5 - 1.5 m advance in top heading. Install support concurrently with excavation, Shotcrete as soon as possible after blasting</td><td>Systematic bolts 5 - 6 m long spaced 1 - 1.5 m in crown and walls with wire mesh. Bolt invert</td><td>150 - 200 mm in crown, 150 mm in sides and 50 mm on face</td><td>Medium to heavy ribs spaced 0.75 m with steel lagging and fore poling if required. Close invert.</td></tr>
</table>

5.2 Structural Discontinuity Orientation

The discontinuity data was obtained from inclined drilling at decline location in the Main Pit. This structural data collected was assessed using the Rocscience software Dips (2011). Three major joints sets were identified based on significant structural data concentrations for both the weighted and unweighted data sets. The contour plots with the major planes overlaid are shown in Figures 5.1 and 5.2. The major planes were summarised in Table 5.3 for both weighted and unweighted data sets.

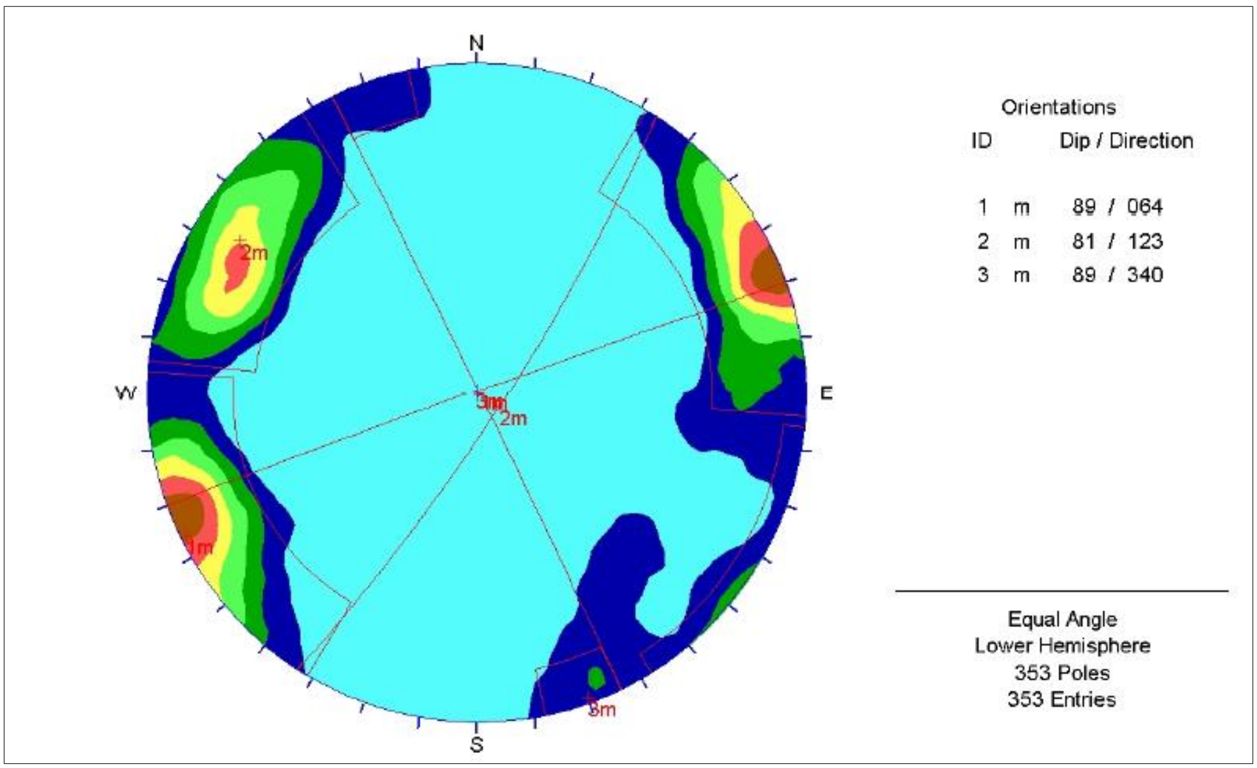

Figure 5.1 Weighted contour plot showing major joint sets and bedding

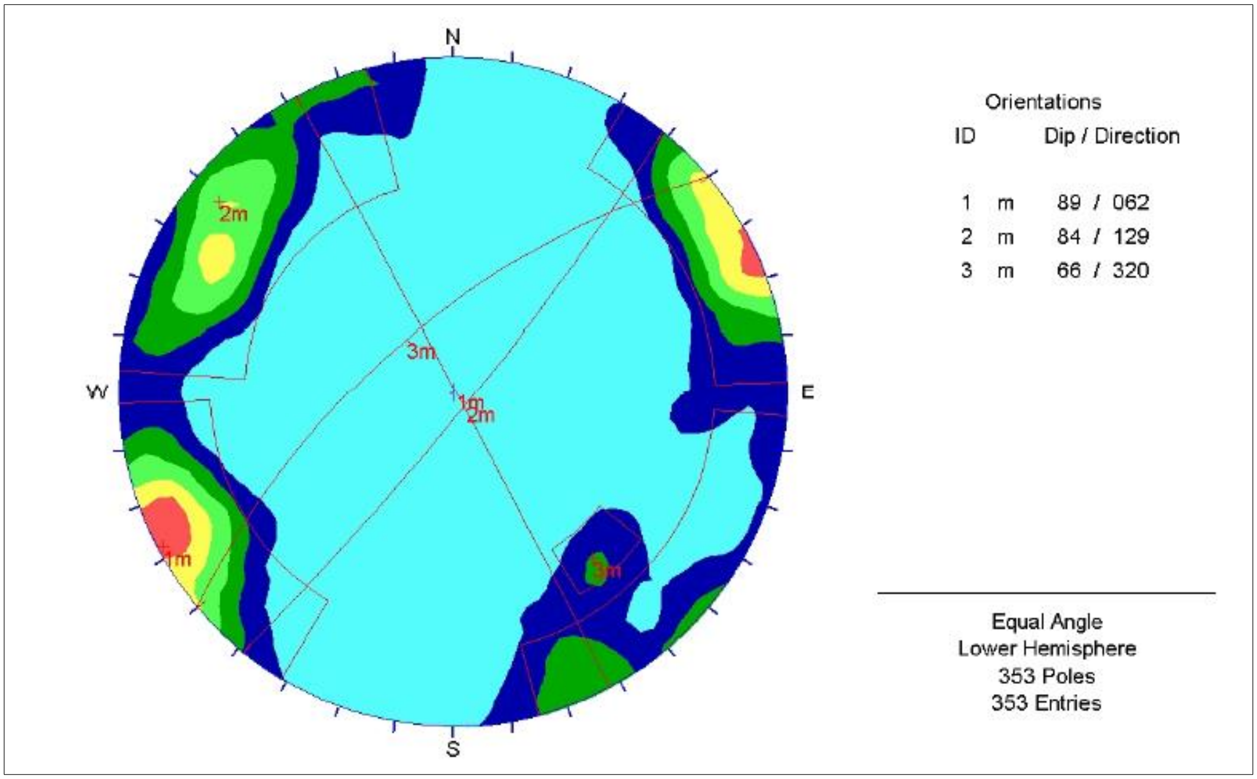

Figure 5.2 Unweighted contour plots showing major joint sets and bedding

Table 5.3 Summary of Joint Sets Identified for Kansanshi Main Pit

Weighted					Unweighted				
Joint Set ID	Joint set	Dip	Dip Direction	Type	Joint Set ID	Joint set	Dip	Dip Direction	Type
1m	J1	89	062	Joint	1m	J1	88	062	Joint
2m	J2	81	122	Joint	2m	J2	81	123	Joint
3m	B1	5	172	Bedding	3m	B1	5	172	Bedding
4m	J3	68	321	Joint	4m	J3	71	324	Joint

5.2.1 Shear Strength of Discontinuities

This non-linear failure criterion was used as it is concepts of joint roughness and wall strength for the shear strength of discontinuities in a rock mass. The Barton-Bandis criterion defines the shear strength of a discontinuity as defined in equation 2.19. The input parameters used to model shear strength are summarised in Table 5.4.

Table 5.4 Parameters for Discontinuity Shear Strength

Geotechnical Domain	Basic Friction Angle (Degrees)	JRC			JCS (MPa)		
		Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound
Clastics	27	3	10	6	82	18	146
Marble	31	3	14	8	72	116	86

The discontinuity shear strengths for the discontinuities are shown in Table 5.5.

Table 5.5 Shear Strength Parameters for Discontinuity Surfaces

Geotechnical Domain	Friction Angle (Degrees)			Cohesion (MPa)		
	Mean	Lower Bound	Upper Bound	Mean	Lower Bound	Upper Bound
Clastics	38	31	41	0.0311	0.0116	0.073
Marble	36	57	46	0.0600	0.0132	0.2272

5.2.2 Kansanshi Design Sectors

Kinematic analysis was carried out using Dips for average wall directions specific to the design sectors. The structurally controlled failures that could potentially develop in the rock slopes were identified. The three modes of failure examined that included planar, wedge and toppling failure. Table 5.4 presents the approximate wall direction for each design sector.

Table 5.6 Design Sectors

	Design Sector 1	Design Sector 2	Design Sector 3	Design Sector 4
Wall Direction (degrees)	**204,152,100**	**270,230**	**320**	**40,65,90**

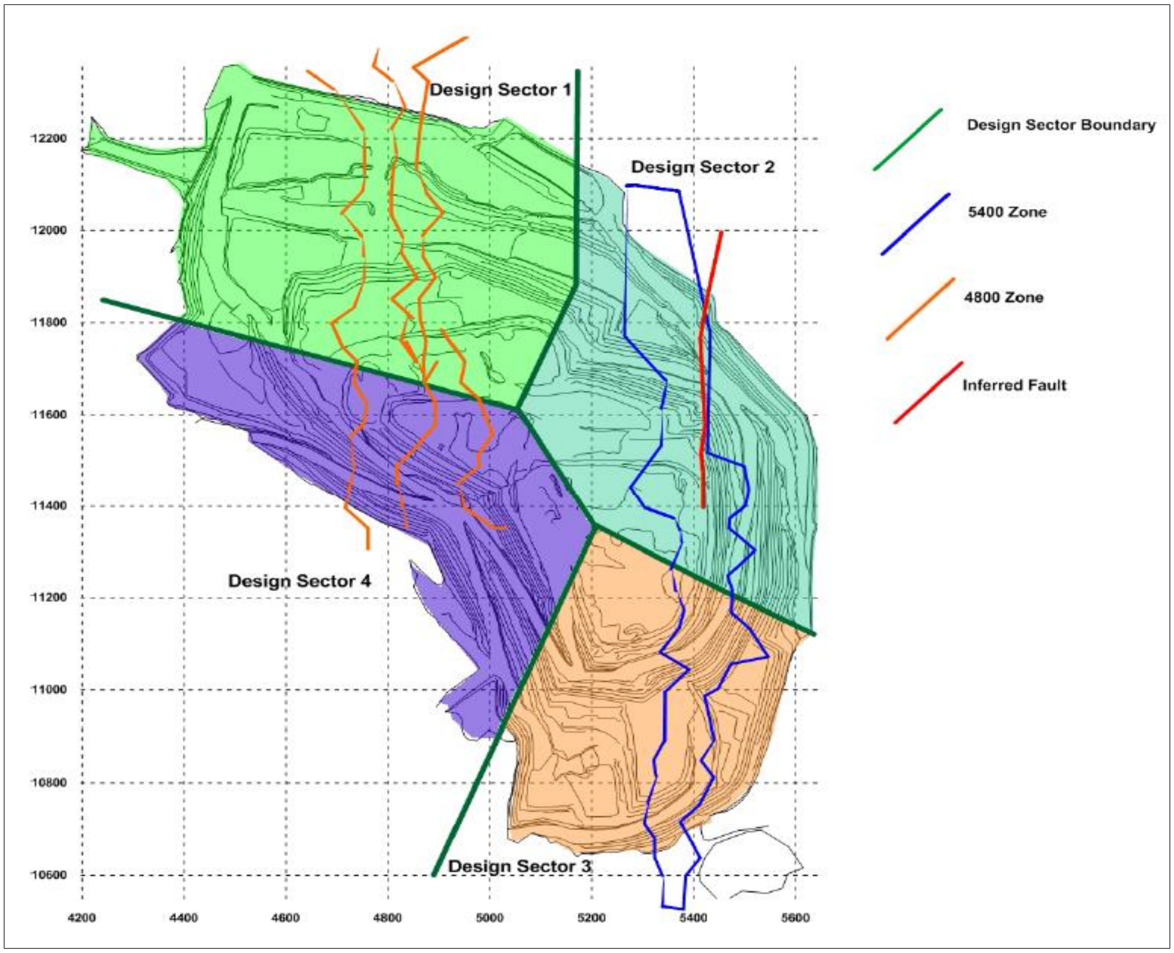

Figure 5.3 Kansanshi Main Pit Design Sectors

The findings from each design sectors were:

- **Design Sector 1**

  Kinematic analysis was carried out with average wall directions of 152, 204, and 100 degrees. Planar failure is expected to occur on discontinuity sets J3 and on J2. Wedge failure is mainly due to intersections between discontinuity sets J1 & J2 and J1 & J3. A bench face angle of 85 degrees suggests both planar and wedge failure to be problematic for all wall directions considered, particularly a wall direction of 100 degrees as the factors of safety is less than 1 (F.O.S <1). Toppling failure is associated predominantly with discontinuity set J1 and J2.

- **Design Sector 2**

  Kinematic analysis was carried out with average wall directions of 270 and 230 degrees. Planar failure is expected to occur mainly on discontinuity set J2. Wedge failure is mainly due to intersections between discontinuity sets J1 & J2 and J2 & J3. The factors of safety for the wedge and planar failures analysed for this sector are greater than a FOS of 1.2, thus suggesting stability. However, toppling failure associated with discontinuity set J1, J2 and J3 is expected for all wall directions and slope angles considered.

- **Design Sector 3**

  Kinematic analysis was carried out with average wall direction of 320 degrees. Planar failure is not of concern in this design sector. Wedge failure is mainly due to intersections between discontinuity sets J2 & J3. The factors of safety for these wedge failures analysed for this sector are greater than a F.O.S of 1.2, suggesting the wedges formed in the rock slope are stable. However, the wedge failure volumes are of great significance therefore lower bench face angles should be used. Toppling failure in this sector is associated predominantly with discontinuity set J1.

- **Design Sector 4**

  Kinematic analysis was carried out with average wall directions of 90, 40, and 65 degrees. Planar failure is expected to occur on discontinuity set J3 for all wall directions and bench face angles considered. Wedge failure is mainly due to intersections between discontinuity sets J1 & J3 and J2 & J3. A bench face angle of 85 degrees suggests both planar and wedge failure to be problematic for certain wall directions as indicated by unstable safety factors of less than 1. Toppling failure is associated predominantly with discontinuity set J1 and J2.

5.3 Support Requirements Based on Q-system

The equivalent Dimension, D_e, of the excavation was obtained by dividing the span, diameter or wall height of the excavation by a quantity called the Excavation Support Ratio, ESR. Hence:

$$D_e = \frac{\text{Excavation Span, Diameter or Height (m)}}{\text{Excavation Support Ratio (ESR)}} \qquad (5.2)$$

The value of ESR is related to the intended use of the excavation and to the degree of security which is demanded of the support system installed to maintain the stability of the excavation. Barton et al (1974) suggest the following values:

Excavation Category	**ESR**
A. Temporary mine openings	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.	1.0
E. Underground nuclear power stations, railway stations, sports and public facilities, factories.	0.8

The equivalent dimension (D_e) was calculated as:

$$D_e = 4/1.6 = \mathbf{2.5m}$$

Where:

Width of an excavation or span = 4m

Excavation support ratio (ESR) = 1.6

The equivalent dimension, D_e, plotted against the value of Q, was used to define a number of support categories in a chart published in the original paper by Barton et al (1974). This chart has recently been updated by Grimstad and Barton (1993) to reflect the increasing use of steel fibre reinforced shotcrete in underground excavation support. Figure 5.5 shows the estimated support based on Q index.

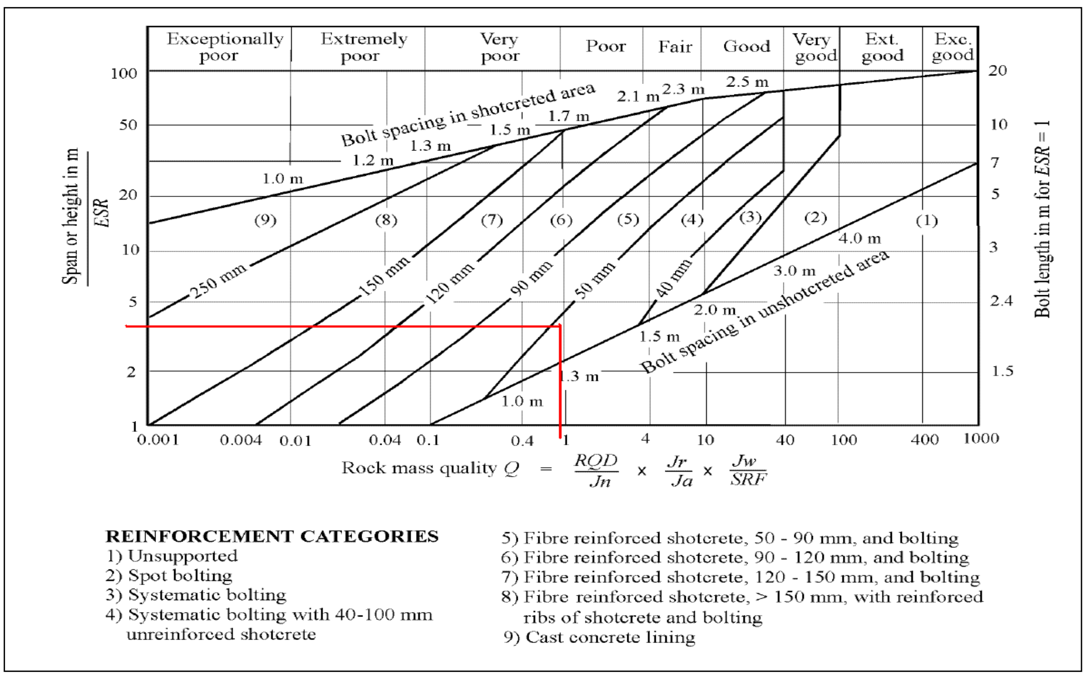

Figure 5.5 Estimated support categories based on the tunnelling quality index Q (after Grimstad & Barton, 1993)

Using Figure 5.5 the average Q-rating falls within category four (4) for systematic bolting with 40-100mm un-reinforced shotcrete. Bolt spacing is between 1 and 1.3m apart and length is approximately 1.6m.

Barton et al (1974) provided additional information on rockbolt length, maximum unsupported spans and roof support pressures to supplement the support recommendations published in the original (1974) paper. The bolt length can be worked out using the following equation:

$$L = 2 + 0.15B/ESR \qquad (5.3)$$

Where:

L = bolt length (m)
B = width of an excavation or span (m)
ESR = equivalent support ratio.

The minimum length of the bolts with reference to equation 5.3 is:

$$L = 2 + (0.15 \times 4)/1.6$$
$$L = \mathbf{1.625m}$$

The calculated minimum bolt length correlates very well with the finding of bolt length in Figure 5.5.

5.3.1 Maximum Unsupported Span

The Maximum unsupported span was worked out using Figure 5.6 and compared using the following equation:

Max span (unsupported) = $\mathbf{2(ESR)\ Q^{0.4} = 2(1.60) \times 0.86^{0.4} = 4m.}$ Therefore, the maximum unsupported span is **4m** to be used in slightly weathered formations. Figure 5.6 indicates the safety factor equal one, using the RMR of 40 and unsupported span of 4 metres.

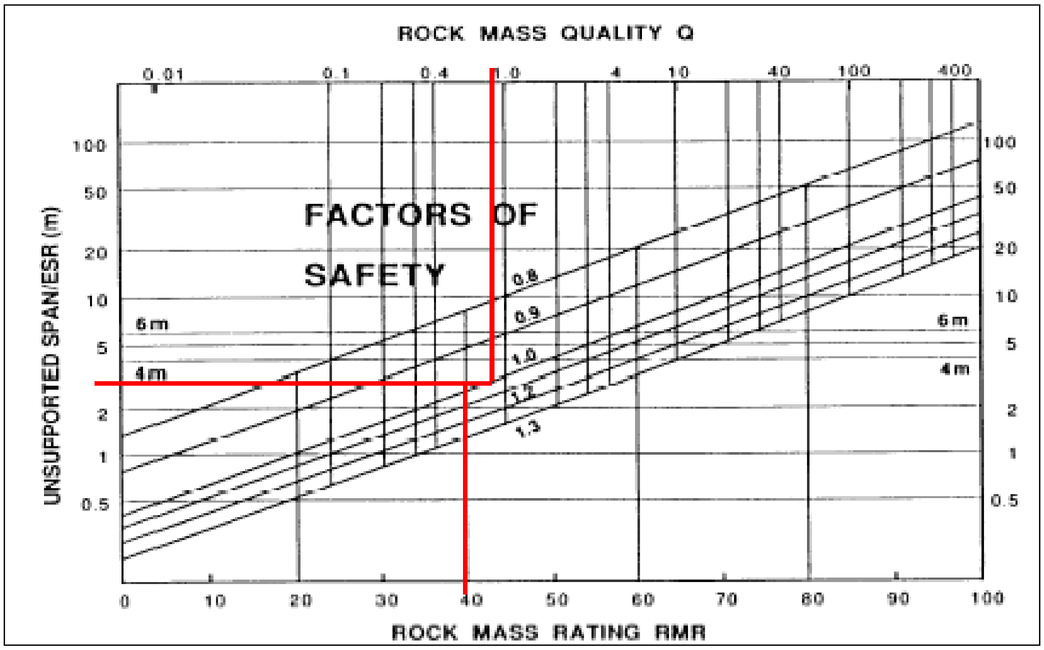

Figure 5.6 Estimated factors of safety as a function of rock mass conditions and unsupported spans (Houghton and Stacey, 1980)

5.4. Portal Highwall Support

The cable anchors were drilled vertically down from the bench above and into the high wall. These were 20 metres long, pre-stressed 40 tonne bearing, and double grouted, and spaced out at five metres along the bench. After installing the cable anchors, the bottom 5.0 metres were grouted first, then when the grout was set the cable anchor was pre-tensioned to 10 tonnes. This was then followed by grouting of the whole length of the cable anchor. A fence and a line of gabions were established on the upper bench to catch any loose rock and minimise water erosion.

Figures 5.7, 5.8 and 5.9 indicate the installation of pre-stressed double grouted cable anchors, section view of support system around the portal and the front view of the portal.

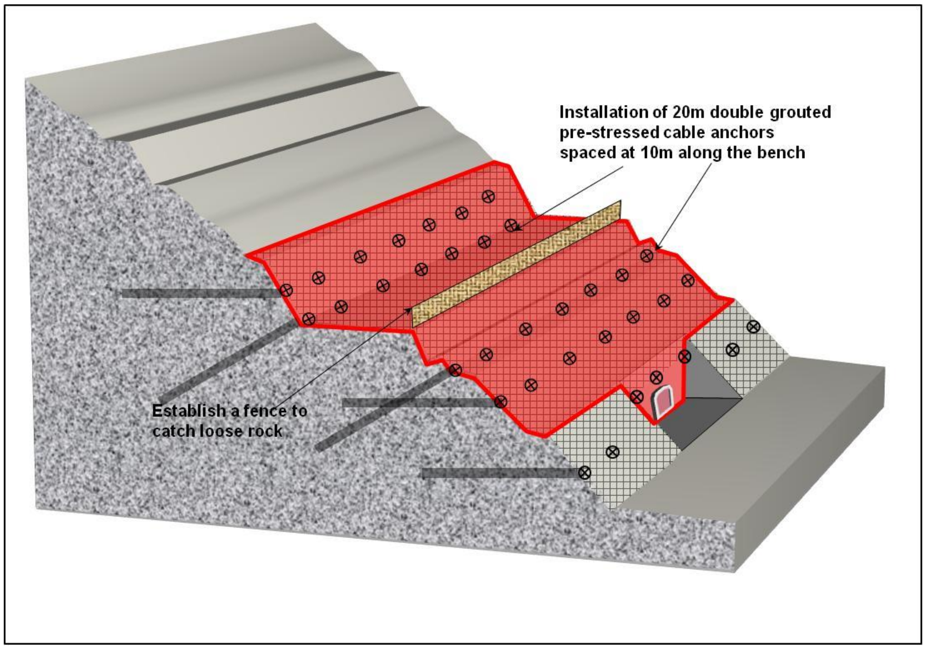

Figure 5.7 Installation of pre-stressed double grouted cable anchors

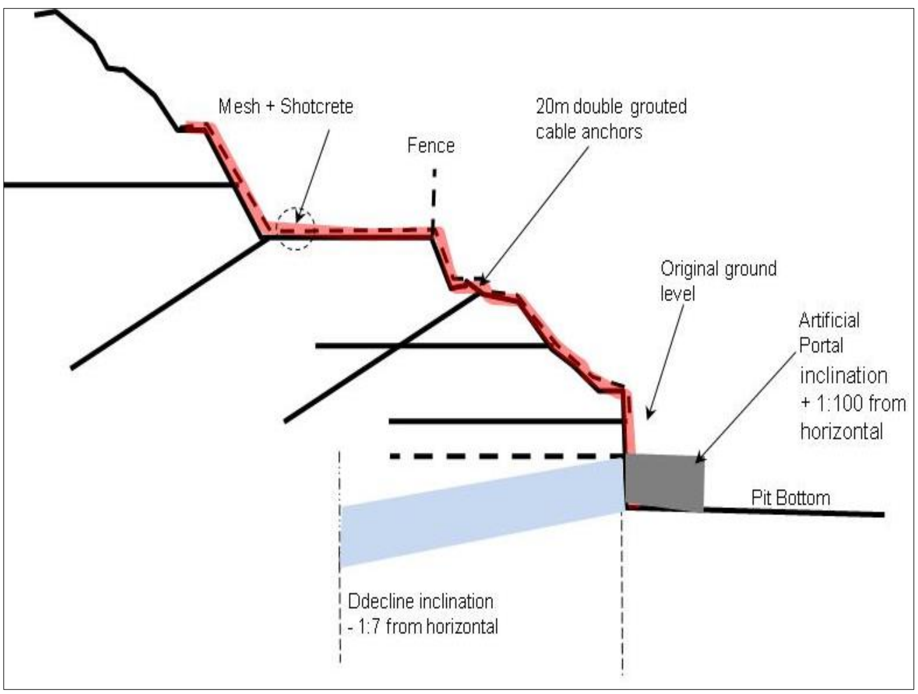

Figure 5.8 Section view for support around the portal

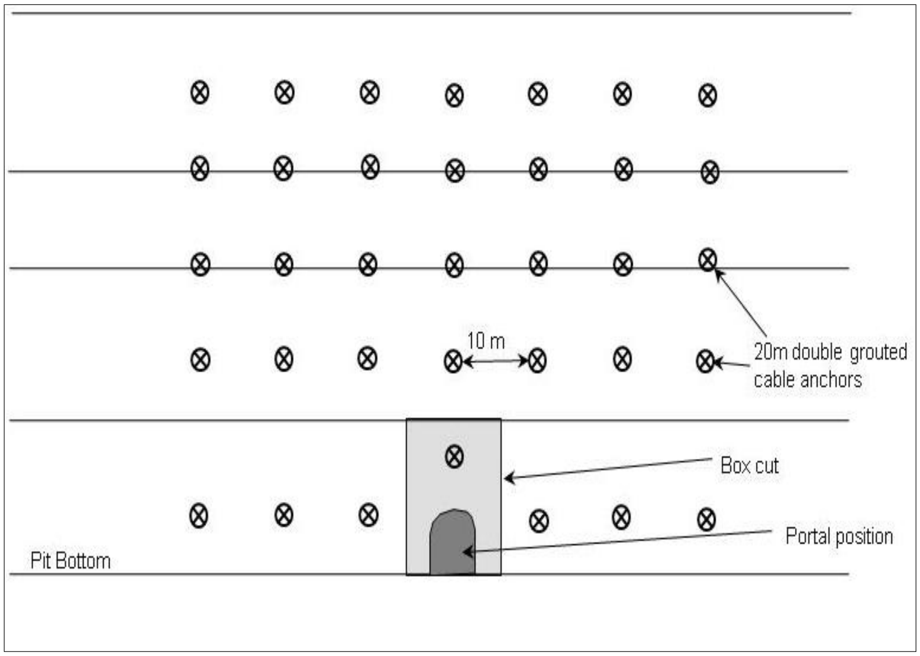

Figure 5.9 Front view (looking towards the portal) supports around the portal

Once the high wall was fully supported, the first five metres of the portal opening was excavated and supported with roof bolting and wire mesh. Arches were installed from the inside out towards the opening, and continue out beyond the face for a distance of five metres to form the artificial portal. This was to protect the portal from falling rocks and water running down the face.

The reinforced steel bars (16 mm diameter) were installed above and on the sides of the arches and spaced 0.3 metres apart. Box shuttering was installed on the inside of the arches to enable concrete to be poured into the opening between the excavation and the arches. Figures 5.10 and 5.11 indicate the steel arches installed at the portal and 3m rock bolts installed over shortcrete.

Figure 10 Installed Steel arches at the portal entrance

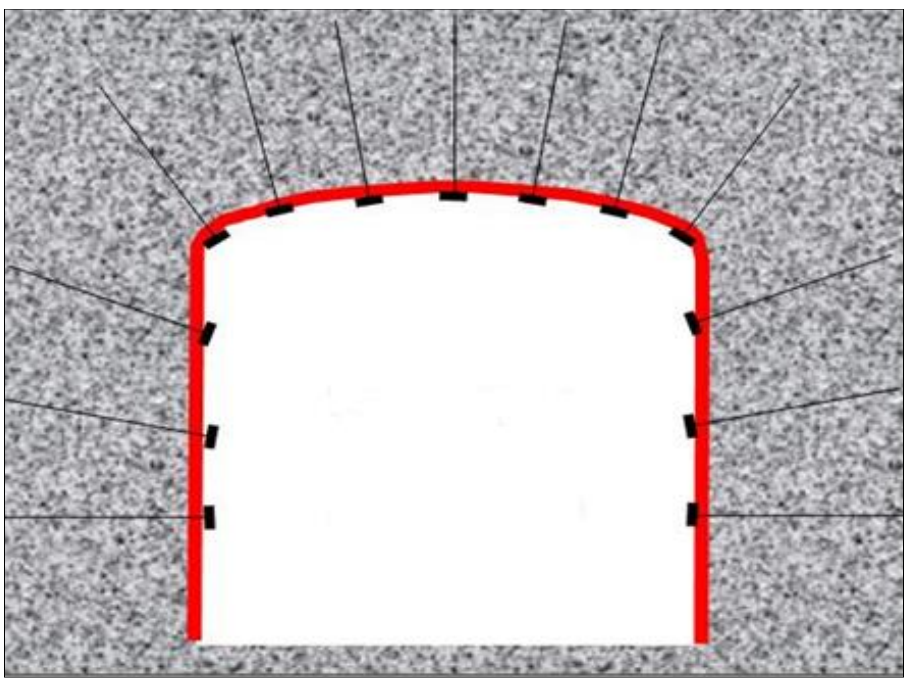

Figure 5.11 Installed 3m Rock Bolts

Chapter 6

CONCLUSIONS AND RECOMMEDATIONS

6.1 Conclusions

The ultimate aim of this study was to characterise the rockmass into domains based on the litho-stratigraphic unit and four major geotechnical domains were identified. However, the following were the major geotechnical domains identified:

- **Deeply weathered saprolite domain**

  This deeply weathered profile is controlled by both the parent rock and major structural features. Relic discontinuity structures, such as bedding and foliation were retained within the weathered fabric, with surface infilling and smooth-planar shape discontinuity profile. The intact strength is generally less than 15MPa and frequently less than 5MPa as field estimate. Field mapping revealed this material is transitional between rock and soil. As well as exhibiting low “intact” strength, in less weathered portions with the geological strength index (GSI) values of less than 25.

- **Marble**

  The marble is usually a fresh to a slightly weathered rock mass, with widely spaced discontinuities (more massive when compared to the other fresh rock geotechnical domains). Average intact rock strength is approximately 130MPa, with a standard deviation of 40MPa. The mean RQD values are at 83 % and with a standard deviation of 20%. Discontinuity surfaces are generally rough and planar with some surface staining.

- **Phyllite**

  The fresh Phyllite has an estimated intact rock strength of approximately 148MPa, with a standard deviation of 63MPa. RQD values are high with an average of 89% and joint surfaces were found to be slightly rough and planar.

- **Knotted Schist**

  The fresh Knotted Schist has an estimated intact rock strength of approximately 119MPa, with a standard deviation of 66MPa. RQD values are high with an average of 77%.

- Using the average RMR (40%), the Q-system value predicted by the equation was 0.6 and the window mapping provided the Q-System value of 0.86. This was considered a good correlation and concluded that the quality of geotechnical mapping was consistent between the two classification systems.

- The results in Table 5.1 shows, rock mass ratings (RMR) were in the range approximately 40 to 60. Thus the rock mass is described as fair to good ground. However, requires 3m threaded rock bolts, spaced 1.5m to 2m in the crown and walls with diamond wire mesh, and 50mm to 100mm fibre reinforced shortcrete in the crown and 30mm in the sides with reference to Table 5.2.

- The main groundwater conduits identified to date are the sub vertical 4800 and 5400 zones. These represent the best targets for dewatering and flow groundwater flow is from the North to the South and the lower marble is the main aquifer of concern.

6.2 Recommendations

The following were the main recommendations:

- It is strongly recommended that ongoing rock mass rating should be carried out during the development or the progress of the decline and relative adjustments to the support requirements should be made based on the principles set in this report.
- Additional geotechnical information needs to be gathered from the 5400 zone due to a high level of data uncertainty in terms of the thickness of the aquifer. Besides the quantitative influence of slips, joints and other geological discontinuities should be well understood by using geophysical techniques that are more accurate in determining or predicting thickness of the layers or discontinuities.
- Monitoring Programme should be formalised for the high wall above the decline, considering substantial height and poor rock mass quality of the saplorate formation. This would ideally include automated prism monitoring, Slope stability radar and time domain reflectometers to detect sub-surface shearing within rock mass.
- The effect of tensioning, non-tensioning on support performance could not be established in this study and it is suggested that a new testing procedure should be developed for testing the performance of tensioned bolts or rook bolts.
- In the highly watery excavation, active-passive support should be applied (e.g. grouted dowels) and shortcreting should be applied to avoid the instability of lose rocks. In addition wire mesh should be also being applied as per the ground condition.
- The most important need at this stage is to improve or train the capabilities of the supervisors and workers below supervisorial level. Improving the capabilities is important to achieve desired specifications, for instance, thickness of the lining on the roof and sidewalls.

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Appendices

Appendix A: Rock mass rating system (after Bieniawski, 1989)

Parameter			Ranges 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 compressive strength	>250 MPa	100-250 MPa	50-100 MPa	25-50MPa	5-25 MPa	1-5 MPa	<1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core quality *RQD*		90%-100%	75%-90%	50%-75%	25%-50%	<25%		
	Rating		20	17	13	8	3		
3	Spacing of joints		>2m	0.6-2m	200-600mm	60-200mm	<60mm		
	Rating		20	15	10	8	5		
4	Condition of joints		Very rough surface Not continuous No separation 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	Groundwater	Inflow per 10m Tunnel length (l/min)	None	10	10-25	25-125	>125		
		Joint water press-ure/major principal stress	0	0.0 – 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		
6	Strike and dip orientations of joints*		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable		
	Rating		0	-2	-5	-10	-12		

*   The Effect of Joint Strike and Dip Orientation in Tunnelling

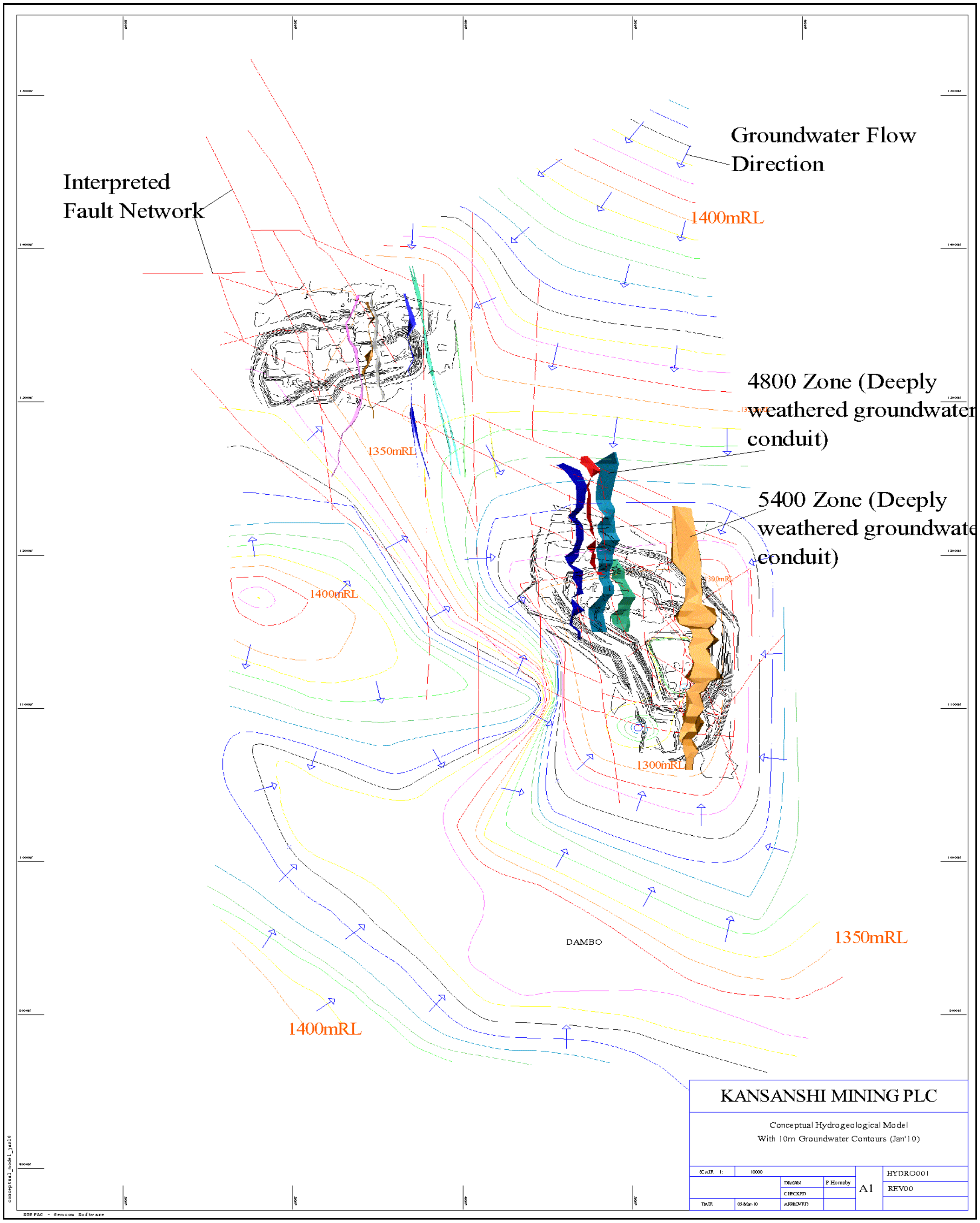
Strike perpendicular to excavation axis				Strike parallel to excavation axis		Dip 0° -20° irrespective of strike
Drive with dip		Drive against dip				
Dip 45° - 90°	Dip 20° - 45°	Dip 45° - 90°	Dip 20° - 45°	Dip 45° - 90°	Dip 20° - 45°	
Very favourable	Favourable	Fair	Unfavourable	Very unfavourable	Fair	Fair

Appendix B: Recommended shotcrete applications in underground mining, for different rock mass conditions (after Hoek and Bawden 1995)

Rock Mass Description	Rock Mass Behaviour	Support Requirements	Shotcrete Application
Massive metamorphic or igneous rock. Low stress conditions.	No spalling, slabbing or failure.	None	None
Massive sedimentary rock. Low stress conditions.	Surfaces of some shales, siltstones, or claystones may slake as a result of moisture content change.	Sealing surface to prevent slaking.	Apply 25 mm thickness of plain shotcrete to permanent surfaces as soon as possible after excavation. Repair shotcrete damage due to blasting.
Massive rock with single wide fault or shear zone.	Fault gouge may be weak and erodible and may cause stability problems in adjacent jointed rock.	Provision of support and surface sealing in vicinity of weak fault of shear zone.	Remove weak material to a depth equal to width of fault or shear zone and grout rebar into adjacent sound rock. Weldmesh can be used if required to provide temporary rockfall support. Fill void with plain shotcrete. Extend steel fibre reinforced shotcrete laterally for at least width of gouge zone.
Massive metamorphic or igneous rock. High stress conditions.	Surface slabbing, spalling and possible rockburst damage.	Retention of broken rock and control of rock mass dilation.	Apply 50 mm shotcrete over weldmesh anchored behind bold faceplates, or apply 50 mm of steel fibre reinforced shotcrete on rock and install rockbolts with faceplates; then apply second 25 mm shotcrete layer. Extend shotcrete application down sidewalls where required.
Massive sedimentary rock. High stress conditions.	Surface slabbing, spalling and possible squeezing in shales and soft rocks.	Retention of broken rock and control of squeezing.	Apply 75 mm layer of fibre reinforced shotcrete directly on clean rock. Rockbolts or dowels are also needed for additional support.
Metamorphic or igneous rock with a few widely spaced joints. Low stress conditions.	Potential for wedges or blocks to fall or slide due to gravity loading.	Provision of support in addition to that available from rockbolts or cables.	Apply 50 mm of steel fibre reinforced shotcrete to rock surfaces on which joint traces are exposed.
Sedimentary rock with a few widely spaced bedding planes and joints. Low stress conditions.	Potential for wedges or blocks to fall or slide due to gravity loading. Bedding plane exposures may deteriorate in time.	Provision of support in addition to that available from rockbolts or cables. Sealing of weak bedding	Apply 50 mm of steel fibre reinforced shotcrete on rock surface on which discontinuity traces are exposed, with particular attention to bedding plane traces.

		plane exposures.	
Jointed metamorphic or igneous rock. High stress conditions.	Combined structural and stress controlled failures around opening boundary.	Retention of broken rock and control of rock mass dilation.	Apply 75 mm plain shotcrete over weldmesh anchored behind bolt faceplates or apply 75 mm of steel fibre reinforced shotcrete on rock, install rockbolts with faceplates and then apply second 25 mm shotcrete layer. Thicker shotcrete layer may be required at high stress concentrations.
Bedded and jointed weak sedimentary rock. High stress conditions.	Slabbing, spalling and possibly squeezing.	Control of rock mass failure and squeezing.	Apply 75 mm of steel fibre reinforced shotcrete to clean rock surfaces as soon as possible, install rockbolts, with faceplates, through shotcrete, apply second 75 mm shotcrete layer.
Highly jointed metamorphic or igneous rock. Low stress conditions.	Ravelling of small wedges and blocks defined by intersecting joints.	Prevention of progressive ravelling.	Apply 50 mm of steel fibre reinforced shotcrete on clean rock surface in roof of excavation. Rockbolts or dowels may be needed for additional support for large blocks.
Highly jointed and bedded sedimentary rock. Low stress conditions.	Bed separation in wide span excavations and ravelling of bedding traces in inclined faces.	Control of bed separation and ravelling.	Rockbolts or dowels required to control bed separation. Apply 75 mm of fibre reinforced shotcrete to bedding plane traces before bolting.
Heavily jointed igneous or metamorphic rock, conglomerates or cemented rockfill. High stress conditions.	Squeezing and 'plastic' flow of rocks mass around opening.	Control of rock mass failure and dilation.	Apply 100 mm of steel fibre reinforced shotcrete as soon as possible and install rockbolts, with faceplates, through shotcrete. Apply additional 50 mm of shotcrete if required. Extend support down sidewalls if necessary.
Heavily jointed sedimentary rock with clay coated surfaces. High stress conditions.	Squeezing and 'plastic' flow of rock mass around opening. Clay rich rocks may swell.	Control of rock mass failure and dilation.	Apply 50 mm of steel fibre reinforced shotcrete as soon as possible, install lattice girders or light steel sets, with invert struts where required, then move steel fibre reinforced shotcrete to cover sets or girders. Forepoling or spiling may be required to stabilize face ahead of excavation. Gaps may be left in final shotcrete to allow for movement resulting from squeezing or swelling. Gap should be closed once opening is stable.
Mild rockburst conditions in massive rock subjected to high stress conditions.	Spalling, slabbing and mild rockbursts.	Retention of broken rock and control of failure propagation.	Apply 50 to 100 mm of shotcrete over mesh or cable lacing which is firmly attached to the rock surface by means of yielding rockbolts or cablebolts.

APPENDIX C: Kansanshi Mine Conceptual Hydrogeological Model



Appendix D: Summarised Rock Quality Designation (RQD) and Recoveries

BHID	FROM	TO	LENGTH	RECOVERY	REC %	RQD	RQD %	LONGEST	CORE SIZE
DCDD05	0.00	2.60	2.60	1.10	42%				HQ
DCDD05	2.60	3.60	1.00	0.70	70%				HQ
DCDD05	3.60	4.60	1.00	0.94	94%				HQ
DCDD05	4.60	5.60	1.00	1.00	100%				HQ
DCDD05	5.60	6.60	1.00	0.86	86%				HQ
DCDD05	6.60	7.60	1.00	0.85	85%				HQ
DCDD05	7.60	8.60	1.00	0.78	78%				HQ
DCDD05	8.60	10.10	1.50	1.38	92%				HQ
DCDD05	10.10	11.60	1.50	1.03	69%				HQ
DCDD05	11.60	13.10	1.50	0.98	65%				HQ
DCDD05	13.10	14.60	1.50	1.10	73%				HQ
DCDD05	14.60	16.10	1.50	1.20	80%				HQ
DCDD05	16.10	17.60	1.50	1.30	87%				HQ
DCDD05	17.60	19.10	1.50	1.07	71%				HQ
DCDD05	19.10	20.60	1.50	1.40	93%	0.29	19%	0.29	HQ
DCDD05	20.60	22.10	1.50	1.50	100%	0.65	43%	0.35	HQ
DCDD05	22.10	23.60	1.50	1.06	71%				HQ
DCDD05	23.60	25.10	1.50	1.25	83%	0.99	66%	0.53	HQ
DCDD05	25.10	26.60	1.50	0.86	57%	0.15	10%	0.15	HQ
DCDD05	26.60	28.10	1.50	1.40	93%	0.40	27%	0.23	HQ
DCDD05	28.10	29.60	1.50	1.20	80%	0.59	39%	0.59	HQ
DCDD05	29.60	31.10	1.50	1.24	83%	0.63	42%	0.20	HQ
DCDD05	31.10	32.60	1.50	1.20	80%	0.10	7%	0.10	HQ
DCDD05	32.60	34.10	1.50	1.04	69%	0.54	36%	0.24	HQ
DCDD05	34.10	35.60	1.50	1.34	89%	0.61	41%	0.23	HQ
DCDD05	35.60	38.60	3.00	2.51	84%	2.29	76%	0.64	HQ
DCDD05	38.60	41.60	3.00	0.83	28%		0%		HQ
DCDD05	41.60	44.60	3.00	2.45	82%	1.88	63%	0.72	HQ
DCDD05	44.60	47.60	3.00	2.93	98%	2.90	97%	0.95	HQ
DCDD05	47.60	50.60	3.00	2.50	83%	1.57	52%	0.42	HQ
DCDD05	50.60	53.60	3.00	2.98	99%	2.81	94%	0.88	HQ

BHID	FROM	TO	LENGTH	RECOVERY	REC %	RQD	RQD %	LONGEST	CORE SIZE
DCDD05	53.60	56.60	3.00	2.38	79%	0.95	32%	0.95	HQ
DCDD05	56.60	59.60	3.00	3.00	100%				HQ
DCDD05	59.60	62.60	3.00	2.20	73%				HQ
DCDD05	62.60	65.60	3.00	1.93	64%				HQ
DCDD05	65.60	68.60	3.00	2.91	97%	2.08	69%	0.68	HQ
DCDD05	68.60	71.60	3.00	3.00	100%	2.90	97%	0.94	HQ
DCDD05	71.60	74.60	3.00	3.00	100%	1.80	60%	0.67	HQ
DCDD05	74.60	77.60	3.00	2.75	92%	1.52	51%	0.93	HQ
DCDD05	77.60	80.60	3.00	3.00	100%	2.82	94%	0.92	HQ
DCDD05	80.60	83.60	3.00	2.95	98%	2.43	81%	0.87	HQ
DCDD05	83.60	86.60	3.00	3.00	100%	1.91	64%	0.50	HQ
DCDD05	86.60	89.60	3.00	2.20	73%	1.00	33%	0.53	HQ
DCDD05	89.60	92.60	3.00	2.77	92%	0.82	27%	0.36	HQ
DCDD05	92.60	95.60	3.00	1.70	57%	1.06	35%	0.92	HQ
DCDD05	95.60	98.60	3.00	1.60	53%	0.47	16%	0.47	HQ
DCDD05	98.60	101.60	3.00	2.65	88%				HQ
DCDD05	101.60	103.10	1.50	1.04	69%				HQ
DCDD05	103.10	103.60	0.50	0.44	88%				HQ
DCDD05	103.60	104.60	1.00	0.66	66%				HQ
DCDD05	104.60	105.60	1.00	0.56	56%				HQ
DCDD05	105.60	106.60	1.00	0.10	10%				HQ
DCDD05	106.60	107.60	1.00	0.40	40%				HQ
DCDD05	107.60	108.60	1.00	0.70	70%				HQ
DCDD05	108.60	110.60	2.00	1.19	60%				HQ
DCDD05	110.60	112.60	2.00	0.38	19%				HQ
DCDD05	112.60	113.10	0.50	0.40	80%		0%		HQ
DCDD05	113.10	113.60	0.50	0.48	96%		0%		HQ
DCDD05	113.60	114.60	1.00	1.00	100%		0%		HQ
DCDD05	114.60	115.60	1.00	1.00	100%		0%		HQ
DCDD05	115.60	116.60	1.00	1.00	100%		0%		HQ
DCDD05	116.60	117.10	0.50	0.50	100%		0%		HQ
DCDD05	117.10	117.60	0.50	0.44	88%		0%		HQ
DCDD05	117.60	119.60	2.00	1.85	93%		0%		HQ
DCDD05	119.60	120.60	1.00	1.00	100%		0%		HQ

BHID	FROM	TO	LENGTH	RECOVERY	REC %	RQD	RQD %	LONGEST	CORE SIZE
DCDD05	120.60	121.60	1.00	1.00	100%		0%		HQ
DCDD05	121.60	122.60	1.00	0.91	91%		0%		HQ
DCDD05	122.60	123.60	1.00	0.98	98%	0.91	91%	0.41	HQ
DCDD05	123.60	124.60	1.00	1.00	100%	0.15	15%	0.15	HQ
DCDD05	124.60	125.60	1.00	1.00	100%				HQ
DCDD05	125.60	126.10	0.50	0.50	100%				HQ
DCDD05	126.10	126.60	0.50	0.48	96%				HQ
DCDD05	126.60	128.60	3.00	1.38	46%	0.39	13%	0.26	HQ
DCDD05	128.60	129.60	1.00	0.50	50%				HQ
DCDD05	129.60	130.60	1.00	0.50	50%				HQ
DCDD05	130.60	131.60	1.00	0.40	40%				HQ
DCDD05	131.60	132.60	1.00	1.00	100%				HQ
DCDD05	132.60	133.60	1.00	1.00	100%				HQ
DCDD05	133.60	134.60	1.00	0.85	85%				HQ
DCDD05	134.60	135.10	0.50	0.20	40%				HQ
DCDD05	135.10	135.60	0.50	CAVITY					HQ
DCDD05	135.60	136.10	0.50	0.36	72%				HQ
DCDD05	136.10	136.60	0.50	CAVITY					HQ
DCDD05	136.60	137.10	0.50	0.45	90%				HQ
DCDD05	137.10	137.60	0.50	0.50	100%				HQ
DCDD05	137.60	138.60	1.00	1.00	100%				HQ
DCDD05	138.60	139.10	0.50	0.50	100%				HQ
DCDD05	139.10	140.10	1.00	0.70	70%				HQ
DCDD05	140.10	140.60	0.50	0.19	38%				HQ
DCDD05	140.60	141.60	1.00	0.70	70%				HQ
DCDD05	141.60	142.60	1.00	0.62	62%				HQ
DCDD05	142.60	143.60	1.00	0.95	95%	0.62	62%	0.28	HQ
DCDD05	143.60	144.70	1.10	1.10	100%	0.43	39%	0.27	HQ
DCDD05	144.70	145.60	0.90	0.90	100%	0.65	72%	0.30	HQ
DCDD05	145.60	146.60	1.00	0.89	89%	0.68	68%	0.31	HQ
DCDD05	146.60	149.60	3.00	2.82	94%	2.07	69%	0.49	HQ
DCDD05	149.60	151.40	1.80	1.80	100%	1.75	97%	0.71	HQ
DCDD05	151.40	152.60	1.20	0.70	58%	0.12	10%	0.12	HQ
DCDD05	152.60	153.00	0.40	0.38	95%				HQ

BHID	FROM	TO	LENGTH	RECOVERY	REC %	RQD	RQD %	LONGEST	CORE SIZE
DCDD05	153.00	155.60	2.60	1.75	67%	0.85	33%	0.33	NQ
DCDD05	155.60	158.60	3.00	3.00	100%	2.45	82%	0.46	NQ
DCDD05	158.60	161.60	3.00	2.98	99%	2.49	83%	0.60	NQ
DCDD05	161.60	164.60	3.00	3.00	100%	2.10	70%	0.45	NQ
DCDD05	164.60	167.60	3.00	3.00	100%	2.60	87%	0.56	NQ
DCDD05	167.60	170.60	3.00	2.90	97%	2.36	79%	0.87	NQ
DCDD05	170.60	173.60	3.00	3.00	100%	2.50	83%	0.84	NQ
DCDD05	173.60	176.60	3.00	3.00	100%	2.77	92%	0.52	NQ
DCDD05	176.60	179.60	3.00	2.79	93%	2.58	86%	0.57	NQ
DCDD05	179.60	182.60	3.00	2.96	99%	2.75	92%	0.85	NQ
DCDD05	182.60	185.60	3.00	3.00	100%	2.90	97%	0.96	NQ
DCDD05	185.60	188.60	3.00	3.00	100%	2.65	88%	0.87	NQ
DCDD05	188.60	191.60	3.00	3.00	100%	2.00	67%	0.33	NQ
DCDD05	191.60	192.54	0.94	0.94	100%	0.40	43%	0.20	NQ
DCDD05	192.54	194.04	1.50	CAVITY					NQ
DCDD05	194.04	194.60	0.56	0.52	93%				NQ
DCDD05	194.60	196.60	2.00	1.33	67%	0.34	17%	0.14	NQ
DCDD05	196.60	197.15	0.55	0.53	96%	0.14	25%	0.14	NQ
DCDD05	197.15	197.60	0.45	CAVITY					NQ
DCDD05	197.60	200.60	3.00	2.90	97%	2.26	75%	0.37	NQ
DCDD05	200.60	203.60	3.00	2.66	89%	2.32	77%	0.71	NQ
DCDD05	203.60	206.60	3.00	2.73	91%	2.26	75%	0.50	NQ
DCDD05	206.60	209.60	3.00	3.00	100%	2.79	93%	0.36	NQ
DCDD05	209.60	212.60	3.00	3.00	100%	2.92	97%	0.97	NQ
DCDD05	212.60	215.60	3.00	2.99	100%	2.63	88%	0.81	NQ
DCDD05	215.60	218.60	3.00	3.00	100%	2.48	83%	0.74	NQ
DCDD05	218.60	221.60	3.00	3.00	100%	2.40	80%	0.67	NQ
DCDD05	221.60	224.60	3.00	3.00	100%	2.62	87%	0.53	NQ
DCDD05	224.60	227.60	3.00	3.00	100%	3.00	100%	0.95	NQ
DCDD05	227.60	230.60	3.00	3.00	100%	2.64	88%	1.00	NQ
DCDD05	230.60	233.60	3.00	2.96	99%	2.06	69%	0.66	NQ
DCDD05	233.60	236.60	3.00	3.00	100%	2.10	70%	0.74	NQ
DCDD05	236.60	236.60	3.00	3.00	100%	2.66	89%	0.68	NQ
DCDD05	236.60	242.60	3.00	3.00	100%	2.65	88%	0.64	NQ

BHID	FROM	TO	LENGTH	RECOVERY	REC %	RQD	RQD %	LONGEST	CORE SIZE
DCDD05	242.60	244.10	1.50	1.39	93%	0.90	60%	0.40	NQ
DCDD05	244.10	245.60	1.50	1.49	99%	1.19	79%	0.47	NQ
DCDD05	245.60	247.10	1.50	1.46	97%	1.32	88%	0.44	NQ
DCDD05	247.10	248.60	1.50	1.38	92%	1.15	77%	0.61	NQ
DCDD05	248.60	250.60	2.00	1.94	97%	1.53	77%	0.90	NQ

Note:

DCDD 05 = borehole Id and borehole number

Appendix E: Core Logging Data

Borehole	Drilling Interval		Total m	Rock Code	Weathering	Rock STR	RQD %	RMR		Bieniawski's RMR
	From	To							GSI	Q_RATING
KRDD21	303.5	306.5	3	PS	FR	7	17	75	70	
KRDD21	300.5	303.5	3	PS	FR	7	17	75	70	
KRDD21	297.5	300.5	3	PS	FR	7	13	71	66	
KRDD21	294.5	297.5	3	PS	FR	7	20	78	73	
KRDD21	291.5	294.5	3	PS	FR	7	20	78	73	
KRDD21	288.5	291.5	3	PS	FR	7	17	75	70	
KRDD21	285.5	288.5	3	PS	FR	7	20	78	73	
KRDD21	273.5	276.5	3	PS	FR	7	20	78	73	
KRDD21	270.5	273.5	3	PS	FR	7	13	71	66	
DCDD005	203.5	206.5	3	MAR	FR	12	20	90	85	103.47
DCDD005	34.1	37.1	3	MAR	FR	12	20	81	76	1.68
DCDD005	242.5	245.5	3	MAR	FR	12	20	92	87	106.67
DCDD005	215.5	218.5	3	MAR	FR	12	20	90	85	106.67
DCDD005	239.5	242.5	3	MAR	FR	12	20	85	80	15
DCDD005	236.5	239.5	3	MAR	FR	12	20	92	87	200.53
DCDD005	233.5	236.5	3	MAR	FR	12	20	90	85	101.33
DCDD005	230.5	233.5	3	MAR	FR	12	20	90	85	105.6
DCDD005	227.5	230.5	3	MAR	FR	12	20	90	85	60.8
DCDD005	224.5	227.5	3	MAR	FR	12	20	90	85	105.6
DCDD005	221.5	224.5	3	MAR	FR	12	20	78	73	9.6

Borehole	Drilling Interval		Total m	Rock Code	Weathering	Rock STR	RQD %	RMR		Bieniawski's RMR
	From	To							GSI	Q_RATING
DCDD005	218.5	221.5	3	MAR	FR	12	20	83	78	13.65
DCDD005	209.5	212.5	3	MAR	FR	12	20	90	85	106.67
DCDD005	206.5	209.5	3	MAR	FR	12	20	90	85	103.47
DCDD005	200.5	203.5	3	MAR	FR	7	20	85	80	103.47
DCDD005	197.5	200.5	3	MAR	FR	7	20	85	80	104.53
DCDD005	194.5	197.5	3	MAR	FR	7	20	84	79	4.8
DCDD005	191.5	194.5	3	MAR	FR	7	13	63	58	2.88
DCDD005	189.1	191.5	2.4	MAR	FR	7	8	57	52	9.3
DCDD005	188.6	189.1	0.5	MAR	FR	7	3	47	42	0.15
DCDD005	185.6	188.6	3	MAR	FR	7	17	68	63	10.267
DCDD005	182.6	185.6	3	MAR	FR	7	20	74	69	22.2
DCDD005	212.5	215.5	3	MAR	FR	12	20	90	85	104.53
DCDD005	15.03	16.1	1.07	MAR	FR	12	20	85	80	0.31
DCDD005	37.1	38.6	1.5	MAR	FR	7	20	71	66	6.4
DCDD005	31.1	34.1	3	MAR	FR	4	13	55	50	1.9667
DCDD005	28.1	31.1	3	MAR	FR	7	17	62	57	7.8
DCDD005	26.6	28.1	1.5	MAR	FR	7	13	53	48	0.8667
DCDD005	25.1	26.6	1.5	MAR	FR	7	13	53	48	0.5
DCDD005	22.1	25.1	3	MAR	FR	7	17	57	52	78.933
DCDD005	21.44	22.1	0.66	MAR	FR	12	17	75	70	0.2533
DCDD005	16.1	19	2.9	MAR	FR	12	20	82	77	96
DCDD005	44.6	47.6	3	MAR	FR	7	8	49	44	1.36
DCDD005	245.5	250.5	5	MAR	FR	12	20	92	87	202.67
DCDD005A	179.5	182.5	3	MAR	FR	7	20	72	82	204.8
DCDD005A	74.85	77	2.15	MAR	FR	7	20	60.1	55.1	2.7
DCDD005A	59.6	60.5	0.9	MAR	FR	7	20	71	66	0.15
DCDD005A	71.6	74	2.4	MAR	FR	7	13	69	64	10.2
DCDD005A	74.6	74.85	0.25	MAR	SW	4	3	45	40	0.54
DCDD005A	42	44.6	2.6	MAR	HW	2	8	45.1	40.1	0.1033
DCDD005A	185.5	188.5	3	MAR	FR	7	20	55	65	6.6

Borehole	Drilling Interval		Total m	Rock Code	Weathering	Rock STR	RQD %	RMR		Bieniawski's RMR
	From	To							GSI	Q_RATING
DCDD005A	77.6	80.6	3	MAR	FR	7	8	60	55	1.12
DCDD005A	86.6	87.6	1	MAR	SW	4	3	47	42	0.015
DCDD005A	87.6	88.6	1	MAR	SW	4	3	47	42	0.015
DCDD005A	88.6	89.6	1	MAR	SW	4	3	47	42	0.015
DCDD005A	89.6	92.6	3	MAR	FR	7	8	66	61	19.8
DCDD005A	92.6	95.6	3	MAR	FR	7	3	51	46	12.9
DCDD005A	95.6	95.96	0.36	MAR	FR	7	3	51	46	2.16
DCDD005A	74	74.6	0.6	MAR	SW	4	3	45	40	0.765
DCDD005A	25.1	28.1	3	MAR	FR	7	8	46	41	0.3867
DCDD005A	20.5	22	1.5	MAR	FR	12	13	76	71	8.1
DCDD005A	22	22.6	0.6	MAR	FR	12	13	76	71	5.4
DCDD005A	24.85	25.1	0.25	MAR	FR	7	8	61	56	4.35
DCDD005A	80.6	83.6	3	MAR	SW	4	3	52	47	0.63
DCDD005A	28.1	31.1	3	MAR	FR	7	8	47	42	0.52
DCDD005A	31.5	33.2	1.7	MAR	FR	7	3	39	34	0.12
DCDD005A	35.5	37.1	1.6	MAR	FR	7	8	66	61	4.4
DCDD005A	37.1	38.6	1.5	MAR	FR	7	3	43.1	38.1	0.6
DCDD005A	38.6	40.1	1.5	MAR	HW	1	3	39.1	34.1	0.08
DCDD005A	40.1	40.6	0.5	MAR	HW	1	13	49.1	44.1	0.2
DCDD005A	40.6	41.6	1	MAR	FR	7	20	80	75	16
DCDD005A	41.6	42	0.4	MAR	FR	7	13	68	63	14.667
DCDD005A	206.8	209.8	3	MAR	FR	7	20	59	69	6.4
DCDD005A	167.5	170.5	3	MAR	FR	7	20	64	74	13.95
DCDD005A	170.5	173.5	3	MAR	FR	7	20	65	75	213.33
DCDD005A	173.5	176.5	3	MAR	FR	7	17	62	72	170.67
DCDD005A	176.5	179.5	3	MAR	FR	7	20	63	73	6.3333
DCDD005A	182.5	185.5	3	MAR	FR	7	20	62	72	39.6
DCDD005A	188.5	188.8	0.3	MAR	FR	7	13	56	66	155.73
DCDD005A	188.8	191.8	3	MAR	FR	7	20	63	73	13.65
DCDD005A	194.8	194.8	0	MAR	FR	7	13	57	67	10.2

Borehole	Drilling Interval		Total m	Rock Code	Weathering	Rock STR	RQD %	RMR		Bieniawski's RMR
	From	To							GSI	Q_RATING
DCDD005A	77	77.6	0.6	MAR	MW	4	3	39.1	34.1	0.54
DCDD005A	164.5	167.5	3	MAR	FR	7	17	62	72	13.2
DCDD005A	203.8	206.8	3	MAR	FR	7	20	65	75	14.85
DCDD005A	191.8	194.8	3	MAR	FR	7	20	63	73	15
DCDD005A	209.8	212.8	3	MAR	FR	7	20	65	75	14.55
DCDD005A	212.8	215.8	3	MAR	FR	7	20	55	65	3.72
DCDD005A	218.8	221.8	3	MAR	FR	7	20	55	65	2.84
DCDD005A	221.8	224.8	3	MAR	FR	7	20	55	65	3.36
DCDD005A	224.8	227.8	3	MAR	FR	7	20	55	65	3.72
DCDD005A	227.8	230.8	3	MAR	FR	7	20	55	65	3.8
DCDD005A	230.8	233.8	3	MAR	FR	7	20	55	65	19.4
DCDD005A	233.8	236.8	3	MAR	FR	7	20	55	65	3.68
DCDD005A	200.8	203.8	3	MAR	FR	7	20	65	75	14.1
DCDD005A	161.5	164.5	3	MAR	FR	7	20	67	77	15
DCDD005A	114	116.6	2.6	MAR	FR	7	17	75	70	8.55
DCDD005A	116.6	119.6	3	MAR	FR	7	20	82	77	27.9
DCDD005A	124.6	125.6	1	MAR	FR	7	17	77	72	28.8
DCDD005A	125.6	128.6	3	MAR	FR	7	20	80	75	15
DCDD005A	128.6	131.6	3	MAR	FR	7	20	79	74	3.64
DCDD005A	131.6	134.6	3	MAR	FR	7	20	79	74	200.53
DCDD005A	134.6	137.6	3	MAR	FR	7	20	78	73	15
DCDD005A	152.5	155.5	3	MAR	FR	7	13	55	65	5.025
DCDD005A	137.6	140.6	3	MAR	FR	7	17	82	77	147.2
DCDD005A	155.5	158.5	3	MAR	FR	7	17	59	69	5.8667
DCDD005A	149.5	152.5	3	MAR	FR	7	20	64	74	198.4
DCDD005A	146.6	149.5	2.9	MAR	FR	7	17	67	77	177.07
DCDD005A	143.6	144.2	0.6	MAR	FR	7	20	78	73	28.5
DCDD005A	145.4	146.6	1.2	MAR	FR	7	20	64	74	198.4
DCDD005A	158.5	161.5	3	MAR	FR	7	20	60	70	192
DCDD005A	142.1	143.6	1.5	MAR	FR	7	20	75.1	70.1	11.55

Borehole	Drilling Interval		Total m	Rock Code	Weathering	Rock STR	RQD %	RMR		Bieniawski's RMR
	From	To							GSI	Q_RATING
DCDD005A	83.6	86.6	3	MARBLE	SW	4	3	52	47	0.03
DCDD005A	197.8	200.8	3	MARBLE	FR	7	20	65	75	14.55
DCDD005B	23.6	25.1	1.5	MARBLE	FR	12	13	79	74	6.6
DCDD005B	26.6	28.1	1.5	MARBLE	FR	12	8	64	59	1.35
DCDD005B	25.1	26.6	1.5	MARBLE	FR	12	3	53.1	48.1	0.0667
DCDD005B	20.24	21.4	1.16	MARBLE	MW	7	3	56	51	0.05
DCDD005B	28.1	29.6	1.5	MARBLE	FR	12	8	61.1	56.1	1.95
DCDD005B	139.1	140.1	1	MARBLE	FR	12	3	56	51	0.05
DCDD005B	77.6	80.6	3	MARBLE	FR	12	20	84	79	9.4
DCDD005B	95.6	96.3	0.7	MARBLE	FR	12	3	59	54	3.2
DCDD005B	92.6	95.6	3	MARBLE	FR	12	8	67	62	7
DCDD005B	89.6	92.6	3	MARBLE	FR	12	8	71	66	1.8
DCDD005B	86.6	89.6	3	MARBLE	FR	12	8	71	66	2.2
DCDD005B	83.6	86.6	3	MARBLE	FR	12	13	76	71	6.4
DCDD005B	80.6	83.6	3	MARBLE	FR	12	17	80	75	8.1
DCDD005B	73	77.6	4.6	MARBLE	FR	12	13	79	74	5.1
DCDD005B	71.6	72.8	1.2	MARBLE	FR	12	13	72	67	6
DCDD005B	68.6	71.6	3	MARBLE	FR	12	20	84	79	9.7
DCDD005B	47.6	50.6	3	MARBLE	FR	12	13	75	70	5.2
DCDD005B	31.1	32.6	1.5	MARBLE	FR	7	3	51.1	46.1	0.0333
DCDD005B	32.6	34.1	1.5	MARBLE	FR	7	8	56.1	51.1	1.2
DCDD005B	34.1	35.6	1.5	MARBLE	FR	7	8	56.1	51.1	2.7333
DCDD005B	35.6	38.57	2.97	MARBLE	FR	12	17	80	75	7.6
DCDD005B	66.3	68.6	2.3	MARBLE	FR	12	13	77	72	6.9
DCDD005B	44.6	47.6	3	MARBLE	FR	12	20	83	78	9.7
DCDD005B	50.6	53.6	3	MARBLE	FR	12	20	83	78	12.533
DCDD005B	53.6	54.7	1.1	MARBLE	FR	12	20	86	81	9.4
DCDD005B	41.82	44.6	2.78	MARBLE	FR	12	13	77	72	6.3
DCDD005B	29.6	31.1	1.5	MARBLE	FR	12	8	61.1	56.1	0.112
KRDD21	207.5	210.5	3	MARBLE	FR	7	17	77	72	

Borehole	Drilling Interval		Total m	Rock Code	Weathering	Rock STR	RQD %	RMR		Bieniawski's RMR
	From	To							GSI	Q_RATING
KRDD21	192.45	195.5	3.05	MARBLE	FR	7	13	64	59	
KRDD21	210.5	213.5	3	MARBLE	FR	7	20	78	73	
KRDD21	201.5	204.5	3	MARBLE	FR	7	20	78	73	
KRDD21	198.5	201.5	3	MARBLE	FR	12	17	85	80	
KRDD21	213.5	216.5	3	MARBLE	FR	7	17	75	70	
KRDD21	195.5	198.5	3	MARBLE	FR	12	20	88	83	
KRDD21	204.5	207.5	3	MARBLE	FR	7	20	78	73	
KRDD21	189.5	192.45	2.95	MARBLE	FR	7	17	68	63	
KRDD21	186	189.5	3.5	MARBLE	FR	7	13	64	59	
KRDD21	216.5	219.5	3	MARBLE	FR	7	13	69	64	
KRDD21	246.5	249.5	3	MARBLE	FR	7	17	70	65	
KRDD21	268	270.5	2.5	MARBLE	FR	4	8	58	53	
KRDD21	267.5	268	0.5	MARBLE	FR	7	8	68	63	
KRDD21	261.5	264.5	3	MARBLE	FR	7	17	75	70	
KRDD21	258.5	261.5	3	MARBLE	FR	7	17	70	65	
KRDD21	255.5	258.5	3	MARBLE	FR	7	17	73	68	
KRDD21	264.5	267.5	3	MARBLE	FR	7	17	73	68	
KRDD21	249.5	252.5	3	MARBLE	FR	7	17	73	68	
KRDD21	219.5	222.5	3	MARBLE	FR	7	17	70	65	
KRDD21	244.8	246.5	1.7	MARBLE	FR	7	20	70	65	
KRDD21	242.6	243.5	0.9	MARBLE	MW	2	3	39	34	
KRDD21	237.5	240.5	3	MARBLE	SW	4	13	52	47	
KRDD21	234.5	237.5	3	MARBLE	FR	7	17	67	62	
KRDD21	231.5	234.5	3	MARBLE	SW	4	13	63	58	
KRDD21	228.5	231.5	3	MARBLE	FR	7	20	68	63	
KRDD21	225.5	228.5	3	MARBLE	FR	7	17	70	65	
KRDD21	222.5	225.5	3	MARBLE	FR	4	17	67	62	
KRDD21	252.5	255.5	3	MARBLE	FR	7	17	73	68	
KRDD21	243.5	243.8	0.3	MARBLE	SW	4	3	41	36	
KRDD21	109	111.9	2.9	MARBLE	FR	7	17	69	64	

Borehole	Drilling Interval From	To	Total m	Rock Code	Weathering	Rock STR	RQD %	RMR	GSI	Bieniawski's RMR Q_RATING
KRDD21	90.5	91	0.5	MARBLE	FR	7	8	59	54	
KRDD21	106	109	3	MARBLE	FR	7	13	66	61	
KRDD21	103	106	3	MARBLE	FR	7	20	75	70	
KRDD21	100	103	3	MARBLE	FR	7	13	66	61	
KRDD21	97	100	3	MARBLE	FR	7	20	78	73	
KRDD21	240.5	242.6	2.1	MARBLE	MW	4	13	46	41	
KRDD21	91	94	3	MARBLE	FR	7	20	78	73	
KRDD21	88	89.5	1.5	MARBLE	FR	4	3	51	46	
KRDD21	112	115	3	MARBLE	HW	2	3	40	35	
KRDD21	94	97	3	MARBLE	FR	7	20	78	73	
KRDD21	120.8	121	0.2	MARBLE	SW	4	3	48	43	
KRDD21	118	119.9	1.9	MARBLE	FR	4	13	58	53	
DCDD005	155.6	158.6	3	RES	HW	1	3	42	37	0.025
DCDD005	164.6	167.6	3	RES	HW	1	3	39	34	0.0067
DCDD005	167.6	168.7	1.1	RES	HW	2	3	40	35	0.0067
DCDD005	20.6	21.44	0.84	RES	HW	1	3	44	39	0.0033
DCDD005	13.1	15.03	1.93	RES	HW	1	3	38	33	0.0033
DCDD005	11.6	13.1	1.5	RES	HW	1	3	38	33	0.0033
DCDD005	10.1	11.6	1.5	RES	HW	1	3	38	33	0.0033
DCDD005	7.1	10.1	3	RES	HW	1	3	38	33	0.0033
DCDD005	6.3	7.1	0.8	RES	HW	0	3	38	33	0.0367
DCDD005	19	20.6	1.6	RES	HW	1	3	44	39	0.05
DCDD005A	50.6	53.6	3	RES	HW	2	3	38.1	33.1	0.0033
DCDD005A	53.6	56.6	3	RES	HW	1	3	37.1	32.1	0.0033
DCDD005A	56.6	59.6	3	RES	HW	1	3	34.1	29.1	0.0033
DCDD005A	60.5	62.5	2	RES	HW	1	3	35.1	30.1	0.1033
DCDD005A	62.6	65.6	3	RES	HW	1	3	41	36	0.02
DCDD005A	44.6	47.6	3	RES	HW	2	3	42.1	37.1	0.0033
DCDD005A	13.1	16.1	3	RES	HW	1	3	41	36	0.0044
DCDD005A	19.1	20.5	1.4	RES	HW	1	3	37.1	32.1	0.0033

Borehole	Drilling Interval		Total m	Rock Code	Weathering	Rock STR	RQD %	RMR		Bieniawski's RMR
	From	To							GSI	Q_RATING
DCDD005A	47.6	50.6	3	RES	HW	2	3	40.1	35.1	0.12
DCDD005A	31.1	31.5	0.4	RES	HW	1	3	34.1	29.1	0.06
DCDD005A	33.2	34.1	0.9	RES	HW	1	3	34.1	29.1	0.06
DCDD005A	34.1	35.5	1.4	RES	HW	2	8	47.1	42.1	0.1956
DCDD005A	22.6	24.85	2.25	RES	HW	1	8	42.1	37.1	0.0967
DCDD005A	113.6	114	0.4	RES	HW	2	3	37.1	32.1	0.01
DCDD005A	119.6	121.6	2	RES	FR	7	20	73.1	68.1	5.6
DCDD005B	16.1	17.6	1.5	RES	EW	1	3	40	35	0.005
DCDD005B	17.6	19.1	1.5	RES	EW	0	3	41	36	0.005
DCDD005B	19.1	20.24	1.14	RES	SW	1	3	37.1	32.1	0.01
DCDD005B	21.4	23.6	2.2	RES	EW	0	8	48.1	43.1	2.15
DCDD005B	10.1	12.6	2.5	RES	EW	1	3	46	41	0.0025
DCDD005B	8	10.1	2.1	RES	EW	0	3	37.1	32.1	0.0013
DCDD005B	62.6	65.6	3	RES	SW	1	3	37.1	32.1	0.03
DCDD005B	96.3	98.6	2.3	RES	SW	7	3	47	42	0.5333
DCDD005B	38.57	41.82	3.25	RES	MW	1	3	43.1	38.1	0.02
DCDD005B	65.6	66.3	0.7	RES	SW	1	13	47.1	42.1	2.07
DCDD005B	54.7	56.6	1.9	RES	SW	1	20	54.1	49.1	1.88
DCDD005B	56.6	59.6	3	RES	SW	1	3	37.1	32.1	0.02
DCDD005B	59.6	62.6	3	RES	SW	1	3	37.1	32.1	0.03
DCDD005B	72.8	73	0.2	RES	SW	0	13	48.1	43.1	0.405
DCDD005A	110.6	113.6	3	RES	MW	2	3	37.1	32.1	0.03
DCDD005	128.6	131.6	3	CBPH	FR	7	20	70	65	3.28
DCDD005	119.6	122.6	3	CBPH	FR	7	20	78	73	29.2
DCDD005	116.6	119.6	3	CBPH	FR	7	20	75	70	13.95
DCDD005	113.6	116.6	3	CBPH	FR	7	20	78	73	3.44
DCDD005	131.6	134.6	3	CBPH	FR	7	17	70	65	2.08
DCDD005	4.1	6.3	2.2	CBPH	HW	0	3	37	32	0.0033
DCDD005	2.6	4.1	1.5	CBPH	HW	1	3	38	33	0.0033
DCDD005	1.1	2.6	1.5	CBPH	HW	1	3	38	33	0.0033

Borehole	Drilling Interval From	To	Total m	Rock Code	Weathering	Rock STR	RQD %	RMR	GSI	Bieniawski's RMR Q_RATING
DCDD005	0	1.1	1.1	CBPH	HW	1	3	40	35	0.0033
DCDD005A	242.8	245.8	3	CBPH	FR	7	13	51	61	2.92
DCDD005A	239.8	242.8	3	CBPH	FR	7	20	58	68	6.4
DCDD005A	1.1	4.1	3	CBPH	HW	1	3	34.1	29.1	0.0033
DCDD005A	4.1	7.1	3	CBPH	HW	1	3	34.1	29.1	0.0044
DCDD005A	7.1	10.1	3	CBPH	HW	1	3	38	33	0.0033
DCDD005A	10.1	13.1	3	CBPH	HW	1	3	41	36	0.0044
DCDD005A	16.1	19.1	3	CBPH	HW	1	3	37.1	32.1	0.0044
DCDD005A	215.8	218.8	3	CBPH	FR	7	17	52	62	3.36
DCDD005A	236.8	239.8	3	CBPH	FR	7	20	57	67	14.7
DCDD005B	14.6	16.1	1.5	CBPH	SW	1	3	44	39	0.0013
DCDD005B	13.1	14.6	1.5	CBPH	SW	1	3	45	40	0.01
DCDD005B	3.6	4.6	1	CBPH	SW	1	3	38.1	33.1	0.0013
DCDD005B	12.6	13.1	0.5	CBPH	SW	1	3	45	40	0.01
DCDD005B	7.6	8	0.4	CBPH	SW	0	3	43	38	0.0025
DCDD005B	6.6	7.6	1	CBPH	SW	1	3	38.1	33.1	0.0013
DCDD005B	4.6	5.6	1	CBPH	SW	1	3	39.1	34.1	0.0033
DCDD005B	2.6	3.6	1	CBPH	SW	1	3	41.1	36.1	0.005
DCDD005B	0	2.6	2.6	CBPH	SW	1	3	45	40	0.0067
DCDD005B	5.6	6.6	1	CBPH	SW	1	3	41.1	36.1	0.0025
DCDD005B	101.6	103.1	1.5	CBPH	HW	1	3	33.1	28.1	0.005
DCDD005B	98.6	101.6	3	CBPH	SW	1	3	35.1	30.1	0.0133
DCDD005B	103.6	104.6	1	CBPH	HW	1	3	33.1	28.1	0.005
DCDD005B	104.6	105.6	1	CBPH	HW	0	3	35.1	30.1	0.005
DCDD005B	118.14	119.6	1.46	CBPH	FR	4	3	50	45	0.0333
DCDD005B	103.1	103.6	0.5	CBPH	HW	1	3	33.1	28.1	0.005
DCDD005B	113.1	113.8	0.7	CBPH	SW	4	3	50	45	0.0133
DCDD005B	105.6	106.6	1	CBPH	SW	1	3	36.1	31.1	0.02
DCDD005B	106.6	107.6	1	CBPH	SW	2	3	40.1	35.1	0.005
DCDD005B	107.6	108.6	1	CBPH	SW	2	3	42	37	0.005

Borehole	Drilling Interval		Total m	Rock Code	Weathering	Rock STR	RQD %	RMR		Bieniawski's RMR
	From	To							GSI	Q_RATING
DCDD005B	108.6	109.4	0.8	CBPH	SW	2	3	47	42	0.02
DCDD005B	109.4	110.6	1.2	CBPH	MW	4	3	48	43	0.05
DCDD005B	120.6	121.6	1	CBPH	FR	4	3	50	45	0.0167
DCDD005B	112.6	113.1	0.5	CBPH	SW	4	3	50	45	0.0133
DCDD005B	119.6	120.6	1	CBPH	FR	4	3	50	45	0.0333
DCDD005B	113.8	114.6	0.8	CBPH	FR	4	3	50	45	0.0133
DCDD005B	114.6	115.6	1	CBPH	FR	4	3	50	45	0.0333
DCDD005B	115.6	116.6	1	CBPH	FR	4	3	50	45	0.0333
DCDD005B	116.6	117.1	0.5	CBPH	FR	4	3	50	45	0.0333
DCDD005B	117.1	118.14	1.04	CBPH	FR	4	3	50	45	0.0333
DCDD005B	110.6	112.6	2	CBPH	SW	4	3	49	44	0.1333
KRDD21	55	58	3	CBPH	SW	7	3	51	46	
KRDD21	25	28	3	CBPH	HW	2	3	39	34	
KRDD21	61	64	3	CBPH	HW	2	3	37	32	
KRDD21	58	59.6	1.6	CBPH	SW	4	3	43	38	
KRDD21	52	55	3	CBPH	FR	7	8	56	51	
KRDD21	40	42.8	2.8	CBPH	FR	7	13	58	53	
KRDD21	37	40	3	CBPH	FR	7	8	53	48	
KRDD21	34	37	3	CBPH	MW	4	8	49	44	
KRDD21	31	34	3	CBPH	MW	4	8	49	44	
KRDD21	28	31	3	CBPH	HW	2	8	42	37	
KRDD21	59.6	61	1.4	CBPH	HW	2	3	37	32	
KRDD21	89.5	90.5	1	CBPH	HW	2	8	39	34	
KRDD21	85	88	3	CBPH	HW	2	3	37	32	
KRDD21	21.5	23	1.5	CBPH	HW	2	3	39	34	
KRDD21	23	24	1	CBPH	HW	2	3	39	34	
KRDD21	24	24.5	0.5	CBPH	HW	2	3	39	34	
KRDD21	24.5	25	0.5	CBPH	HW	2	3	39	34	
DCDD005	134.6	137.6	3	PHY	FR	7	17	70	65	2.68
DCDD005	122.6	125.6	3	PHY	FR	7	20	73	68	5.9

Borehole	Drilling Interval		Total m	Rock Code	Weathering	Rock STR	RQD %	RMR		Bieniawski's RMR
	From	To							GSI	Q_RATING
DCDD005	41.6	44.6	3	PHY	FR	7	8	59	54	0.8
DCDD005	38.6	41.6	3	PHY	FR	7	8	58	53	6.15
DCDD005	47.6	50.6	3	PHY	MW	4	13	52.1	47.1	0.14
DCDD005	53.6	56.6	3	PHY	SW	4	8	57	52	0.6
DCDD005A	248.8	250.8	2	PHY	FR	7	13	52	62	8.85
DCDD005A	245.8	248.8	3	PHY	FR	7	17	55	65	3.56
DCDD005A	98.6	101.6	3	PHY	HW	2	3	35.1	30.1	0.0267
DCDD005A	65.6	68.6	3	PHY	HW	2	3	40.1	35.1	0.0044
DCDD005A	101.6	104.6	3	PHY	HW	2	3	38.1	33.1	0.0133
DCDD005B	141.6	142.6	1	PHY	FR	7	3	50	45	0.1333
DCDD005B	138.6	139.1	0.5	PHY	FR	7	3	46	41	0.0667
DCDD005B	137.6	138.07	0.47	PHY	SW	7	3	47	42	0.0067
DCDD005B	137.1	137.6	0.5	PHY	SW	7	3	47	42	0.0133
DCDD005B	136.6	137.1	0.5	PHY	SW	7	3	53	48	0.0133
DCDD005B	133.6	134.6	1	PHY	MW	2	3	36	31	0.0067
DCDD005B	132.6	133.6	1	PHY	MW	2	3	36	31	0.0333
DCDD005B	131.6	132.6	1	PHY	MW	0	3	39.1	34.1	0.015
DCDD005B	130.6	131.6	1	PHY	MW	1	3	38	33	0.015
DCDD005B	129.6	130.6	1	PHY	MW	1	3	40.1	35.1	0.0075
DCDD005B	128.6	129.6	1	PHY	SW	1	3	35.1	30.1	0.0025
DCDD005B	127.6	128.6	1	PHY	SW	1	3	47	42	1.3
DCDD005B	126.6	127.6	1	PHY	SW	4	3	41.1	36.1	0.3467
DCDD005B	125.6	126.6	1	PHY	SW	4	3	38.1	33.1	0.0533
DCDD005B	124.6	125.6	1	PHY	SW	4	3	48	43	0.0444
DCDD005B	123.6	124.6	1	PHY	FR	4	3	50	45	0.5
DCDD005B	122.6	123.6	1	PHY	FR	4	20	67	62	4.55
DCDD005B	121.6	122.6	1	PHY	FR	4	3	50	45	0.0222
DCDD005B	134.6	135.1	0.5	PHY	MW	2	3	49	44	0.0333
KRDD21	180.5	183.5	3	PHY	SW	4	8	55	50	
KRDD21	153.5	154	0.5	PHY	HW	1	3	33	28	

Borehole	Drilling Interval		Total m	Rock Code	Weathering	Rock STR	RQD %	RMR		Bieniawski's RMR
	From	To							GSI	Q_RATING
KRDD21	157	158	1	PHY	HW	2	3	32	27	
KRDD21	159	160	1	PHY	HW	2	3	32	27	
KRDD21	154	154.7	0.7	PHY	HW	1	3	33	28	
KRDD21	49	52	3	PHY	SW	4	8	53	48	
KRDD21	46	49	3	PHY	MW	4	3	44	39	
KRDD21	43	46	3	PHY	SW	7	3	53	48	
KRDD21	42.8	43	0.2	PHY	FR	7	13	60	55	
KRDD21	115	118	3	PHY	FR	4	13	59	54	
KRDD21	276.5	279.5	3	PHY	FR	7	17	75	70	
KRDD21	282.5	285.5	3	PHY	FR	7	20	78	73	
KRDD21	279.5	282.5	3	PHY	FR	7	20	78	73	
DCDD005A	95.96	98.6	2.64	PHY	HW	2	3	35.1	30.1	0.04
KRDD21	156.5	157	0.5	PHY	HW	1	3	33	28	
KRDD21	158	159	1	PHY	HW	2	3	32	27	
DCDD005	125.6	128.6	3	KS	FR	7	20	70	65	1.58
DCDD005	140.6	143.6	3	KS	FR	7	20	65.1	60.1	4
DCDD005	137.6	140.6	3	KS	FR	7	17	65.1	60.1	6
DCDD005	110.6	113.6	3	KS	SW	7	17	65	60	12.6
DCDD005	107.6	110.6	3	KS	SW	7	17	67	62	4.6667
DCDD005	179.6	182.6	3	KS	FR	7	20	78	73	7.8
DCDD005	104.6	107.6	3	KS	SW	7	17	69	64	
DCDD005	74.6	77.6	3	KS	SW	7	17	64	59	3.44
DCDD005	71.6	73.1	1.5	KS	SW	4	3	42.1	37.1	0.25
DCDD005	102.6	104.6	2	KS	HW	2	3	38	33	4.2
DCDD005	101.6	102.6	1	KS	SW	7	20	65.1	60.1	1.185
DCDD005	98.6	101.6	3	KS	SW	7	20	84	79	7.5
DCDD005	95.6	98.6	3	KS	SW	7	20	72	67	4.2
DCDD005	92.6	95.6	3	KS	SW	7	20	59.1	54.1	3.2333
DCDD005	89.6	92.6	3	KS	SW	7	20	69	64	75.733
DCDD005	86.6	89.6	3	KS	SW	7	3	57	52	0.64

Borehole	Drilling Interval		Total m	Rock Code	Weathering	Rock STR	RQD %	RMR		Bieniawski's RMR
	From	To							GSI	Q_RATING
DCDD005	83.6	86.6	3	KS	SW	7	3	55	50	0
DCDD005	80.6	83.6	3	KS	SW	7	20	74	69	2.8
DCDD005	77.6	80.6	3	KS	SW	7	20	72	67	3.85
DCDD005	73.1	74.6	1.5	KS	SW	4	8	49.1	44.1	0.55
DCDD005	50.6	53.6	3	KS	SW	4	3	52	47	0.88
DCDD005	62.6	64.1	1.5	KS	HW	2	3	36.1	31.1	0.01
DCDD005	64.1	65.6	1.5	KS	MW	4	3	39.1	34.1	0.01
DCDD005	65.6	67.1	1.5	KS	SW	4	13	54.1	49.1	0.52
DCDD005	67.1	68.6	1.5	KS	SW	4	17	60.1	55.1	1.05
DCDD005B	248.5	250.6	2.1	KS	FR	12	17	84	79	7.7
DCDD005B	170.6	173.6	3	KS	FR	12	17	76	71	16.6
DCDD005B	167.6	170.6	3	KS	FR	12	17	76	71	15.8
DCDD005B	166.8	167.6	0.8	KS	FR	12	17	76	71	23.2
DCDD005B	164.6	166.8	2.2	KS	FR	12	17	76	71	23.2
DCDD005B	161.6	164.6	3	KS	FR	12	13	75	70	14
DCDD005B	158.94	161.6	2.66	KS	FR	12	17	79	74	16.6
DCDD005B	179.6	182.6	3	KS	FR	12	20	88	83	18.4
DCDD005B	209.6	212.6	3	KS	FR	12	17	80	75	25.867
DCDD005B	247.1	248.5	1.4	KS	FR	12	17	84	79	7.7
DCDD005B	245.6	247.1	1.5	KS	FR	12	17	84	79	8.8
DCDD005B	244.1	245.6	1.5	KS	FR	12	17	84	79	7.9
DCDD005B	242.6	244.1	1.5	KS	FR	12	13	80	75	6
DCDD005B	239.6	242.6	3	KS	FR	12	17	84	79	8.8
DCDD005B	236.6	239.6	3	KS	FR	12	17	84	79	8.9
DCDD005B	233.6	236.6	3	KS	FR	12	13	80	75	7
DCDD005B	230.6	233.6	3	KS	FR	12	13	80	75	6.9
DCDD005B	227.6	230.6	3	KS	FR	12	17	82	77	11.733
DCDD005B	224.6	227.6	3	KS	FR	12	20	85	80	13.333
DCDD005B	221.6	224.6	3	KS	FR	12	17	77	72	8.7
DCDD005B	218.6	221.6	3	KS	FR	12	17	77	72	8

Borehole	Drilling Interval		Total m	Rock Code	Weathering	Rock STR	RQD %	RMR		Bieniawski's RMR
	From	To							GSI	Q_RATING
DCDD005B	173.6	176.6	3	KS	FR	12	17	76	71	18.4
DCDD005B	182.6	185.6	3	KS	FR	12	20	87	82	19.4
DCDD005B	185.6	188.7	3.1	KS	FR	12	17	84	79	23.467
DCDD005B	188.7	191.6	2.9	KS	FR	12	13	70	65	6.7
DCDD005B	191.6	192.54	0.94	KS	SW	12	8	64	59	4.3
DCDD005B	215.6	218.6	3	KS	FR	12	17	79	74	8.3
DCDD005B	198.82	200.6	1.78	KS	FR	12	17	73	68	5
DCDD005B	200.6	203.6	3	KS	FR	12	17	83	78	15.4
DCDD005B	203.6	206.6	3	KS	FR	12	13	76	71	7.5
DCDD005B	206.6	209.6	3	KS	FR	12	20	83	78	9.3
DCDD005B	176.6	179.6	3	KS	FR	12	17	85	80	22.933
DCDD005B	212.6	215.6	3	KS	FR	12	17	79	74	11.733
KRDD21	160.9	162	1.1	KS	EW	0	3	31	26	
KRDD21	162	163	1	KS	HW	1	3	33	28	
KRDD21	163	163.1	0.1	KS	HW	1	3	33	28	
KRDD21	168.5	169	0.5	KS	HW	1	3	33	28	
KRDD21	169	169.1	0.1	KS	HW	1	3	33	28	
KRDD21	169.1	170	0.9	KS	EW	0	3	31	26	
KRDD21	170	171.5	1.5	KS	HW	1	3	33	28	
KRDD21	171.5	171.9	0.4	KS	HW	1	3	33	28	
KRDD21	149.6	151	1.4	KS	HW	1	3	33	28	
KRDD21	132	133	1	KS	MW	4	13	50	45	
KRDD21	133	136	3	KS	MW	4	13	51	46	
KRDD21	136	139	3	KS	MW	4	8	46	41	
KRDD21	139	142	3	KS	MW	4	8	46	41	
KRDD21	142	143.4	1.4	KS	MW	4	3	41	36	
KRDD21	145	146	1	KS	MW	4	3	36	31	
KRDD21	152.7	153	0.3	KS	HW	1	3	33	28	
KRDD21	153	153.5	0.5	KS	HW	2	3	34	29	
KRDD21	131.5	132	0.5	KS	MW	4	3	38	33	

Borehole	Drilling Interval From	To	Total m	Rock Code	Weathering	Rock STR	RQD %	RMR	GSI	Bieniawski's RMR Q_RATING
KRDD21	148.4	148.6	0.2	KS	HW	2	3	34	29	
KRDD21	81.4	82	0.6	KS	HW	2	13	44	39	
KRDD21	79	81.4	2.4	KS	SW	7	13	63	58	
KRDD21	76	79	3	KS	SW	7	13	63	58	
KRDD21	73	76	3	KS	SW	7	13	63	58	
KRDD21	70	73	3	KS	FR	7	13	63	58	
KRDD21	67	70	3	KS	SW	7	13	61	56	
KRDD21	64	67	3	KS	SW	4	8	49	44	
KRDD21	84	84.7	0.7	KS	HW	1	3	36	31	
KRDD21	84.7	85	0.3	KS	HW	1	3	36	31	
KRDD21	124.5	125.5	1	KS	MW	4	3	38	33	
KRDD21	124	124.5	0.5	KS	MW	4	3	38	33	
KRDD21	122.5	124	1.5	KS	MW	4	3	38	33	
KRDD21	121.8	122.5	0.7	KS	MW	4	3	38	33	
KRDD21	82	84	2	KS	HW	2	3	37	32	
KRDD21	111.9	112	0.1	KS	HW	2	17	48	43	
KRDD21	8	9.5	1.5	KS	HW	1	3	37	32	
KRDD21	6.5	8	1.5	KS	HW	1	3	37	32	
KRDD21	3.5	5	1.5	KS	HW	1	3	37	32	
KRDD21	20	21.5	1.5	KS	HW	2	3	39	34	
KRDD21	2	3.5	1.5	KS	HW	1	3	37	32	
KRDD21	11	12.5	1.5	KS	HW	1	3	37	32	
KRDD21	18.5	20	1.5	KS	HW	2	3	39	34	
KRDD21	17	18.5	1.5	KS	HW	2	3	39	34	
KRDD21	15.5	17	1.5	KS	HW	1	3	37	32	
KRDD21	151	151.8	0.8	KS	HW	2	3	34	29	
KRDD21	183.5	186	2.5	KS	SW	4	8	49	44	
KRDD21	130.5	131.5	1	KS	MW	4	3	38	33	
KRDD21	5	6.5	1.5	KS	HW	1	3	37	32	
DCDD005	149.6	152.6	3	CLBS	SW	7	13	64	59	3.2

Borehole	Drilling Interval		Total m	Rock Code	Weathering	Rock STR	RQD %	RMR		Bieniawski's RMR
	From	To							GSI	Q_RATING
DCDD005	161.6	164.6	3	CLBS	HW	1	3	39	34	0.02
DCDD005	168.7	170.6	1.9	CLBS	HW	7	3	47.1	42.1	0.0533
DCDD005	158.6	161.6	3	CLBS	SW	4	3	39.1	34.1	0.1867
DCDD005	152.6	155.6	3	CLBS	FR	7	17	62.1	57.1	2.68
DCDD005	170.6	173.6	3	CLBS	SW	7	13	53.1	48.1	0.62
DCDD005	173.6	176.6	3	CLBS	FR	7	17	69	64	2.92
DCDD005	176.6	179.6	3	CLBS	FR	7	17	75	70	3.36
KRDD21	177.5	180.5	3	CLBS	FR	4	17	58	53	
KRDD21	174.5	177.5	3	CLBS	FR	4	3	44	39	
KRDD21	173	173.5	0.5	CLBS	SW	4	17	58	53	
KRDD21	173.5	174.5	1	CLBS	FR	4	8	49	44	
DCDD005	70.1	71.6	1.5	BS	MW	2	3	37.1	32.1	0.0075
DCDD005	68.6	70.1	1.5	BS	MW	2	3	37.1	32.1	0.0075
DCDD005A	104.6	107.6	3	BS	HW	2	3	40.1	35.1	0.0133
DCDD005A	107.6	110.6	3	BS	HW	2	3	40.1	35.1	0.0133
DCDD005B	158.6	158.94	0.34	BS	FR	12	17	76	71	16.6
DCDD005B	144.7	145.6	0.9	BS	FR	7	13	64	59	7.2
DCDD005B	155.6	158.6	3	BS	FR	12	17	74	69	16.4
DCDD005B	149.6	151.66	2.06	BS	FR	12	20	77	72	25.867
DCDD005B	147.2	149.6	2.4	BS	FR	12	13	70	65	13.8
DCDD005B	154.55	155.6	1.05	BS	FR	12	8	65	60	6.6
DCDD005B	145.6	146.5	0.9	BS	FR	7	13	64	59	4.5333
DCDD005B	143.6	144.7	1.1	BS	FR	7	8	59	54	3.9
DCDD005B	142.6	143.6	1	BS	FR	7	13	64	59	2.5833
DCDD005B	140.6	141.6	1	BS	FR	7	3	53	48	0.2
DCDD005B	135.6	136.1	0.5	BS	FR	4	3	45	40	0.05
DCDD005B	146.5	147.2	0.7	BS	FR	12	13	69	64	6.9

Borehole	Drilling Interval		Total m	Rock Code	Weathering	Rock STR	RQD %	RMR		Bieniawski's RMR
	From	To							GSI	Q_RATING
DCDD005	143.6	146.6	3	VNQ	SW	7	17	62.1	57.1	0.96
DCDD005	146.6	149.6	3	VNQ	SW	4	8	50.1	45.1	1.17
DCDD005	56.6	59.6	3	VNQ	MW	4	3	45	40	0.08
DCDD005	59.6	61.1	1.5	VNQ	MW	4	3	45	40	0.0133
DCDD005A	68.6	71.6	3	BXC	HW	2	3	40.1	35.1	0.0233
DCDD005B	153	154.55	1.55	VNQ	FR	12	8	60	55	2.2
DCDD005B	152.6	153	0.4	VNQ	FR	12	3	55	50	0.0444
DCDD005B	151.66	152.6	0.94	VNQ	FR	12	3	56	51	0.1185
DCDD005B	140.1	140.6	0.5	VNQ	FR	12	3	57	52	0.1333
DCDD005B	138.07	138.6	0.53	VNQ	FR	12	3	48	43	0.0222
DCDD005B	194.04	197.15	3.11	VNQ	MW	12	3	50.1	45.1	0.04
DCDD005B	197.6	198.82	1.22	VNC	FR	12	13	58	53	3.00
KRDD21	129	130	1	VNQ	MW	4	3	40	35	
KRDD21	126.5	127	0.5	VNQ	EW	0	3	37	32	
KRDD21	167.5	167.8	0.3	VNQ	HW	1	3	33	28	
KRDD21	160	160.9	0.9	VNQ	EW	0	3	31	26	
KRDD21	165.8	166	0.2	VNQ	FR	4	3	36	31	
KRDD21	166	166.1	0.1	VNQ	EW	0	3	31	26	
KRDD21	166.9	167	0.1	VNQ	EW	0	3	31	26	
KRDD21	143.4	145	1.6	VNQ	MW	4	3	38	33	

Borehole	Drilling Interval		Total (m)	Rock Code	Weathering	Rock STR	RQD %	RMR		Bieniawski's RMR
	From	To							GSI	Q_Rating
DCDD005	149.6	152.6	3	CLBS	SW	7	13	64	59	3.2
DCDD005	161.6	164.6	3	CLBS	HW	1	3	39	34	0.02
DCDD005	168.7	170.6	1.9	CLBS	HW	7	3	47.1	42.1	0.053333
DCDD005	158.6	161.6	3	CLBS	SW	4	3	39.1	34.1	0.186667
DCDD005	152.6	155.6	3	CLBS	FR	7	17	62.1	57.1	2.68
DCDD005	170.6	173.6	3	CLBS	SW	7	13	53.1	48.1	0.62
DCDD005	173.6	176.6	3	CLBS	FR	7	17	69	64	2.92
DCDD005	176.6	179.6	3	CLBS	FR	7	17	75	70	3.36
KRDD21	177.5	180.5	3	CLBS	FR	4	17	58	53	
KRDD21	174.5	177.5	3	CLBS	FR	4	3	44	39	
KRDD21	173	173.5	0.5	CLBS	SW	4	17	58	53	
KRDD21	173.5	174.5	1	CLBS	FR	4	8	49	44	

Where;

Weathering Code

EW 100% Extreme weathered, including soil
SW Strongly weathered (>50%)
MW Partly weathered (<50%)
WW weathered coatings and fractures
FR Fresh rock

Lithological Code

MAR-Marble, PS- Pebble Schist, RES -Residual Marble
VN- Vein, CV – Cavity, SAP – Saprolite,
CBS- Carbonaceous Schist, CBKS – Carbonaceous Knotted Schist, BS-Biotite Schist, KS – Knotted Schist,
CLS – Calcareous Schist

Appendix F Structural Mapping Data

A	B	C	D	E	F	G	H	I	J	K	L	M	O	S	W	Y	AA	AC	AE	AH	AJ	AL	AN
MAIN PIT														A2	A4= E1+E2+E3+E4+E5					A5	B	ieniawki'	ieniawki
Line	From	To	y	x	z	Date mappe	Rock Cod	DISCONT INUI	DIP	DIP DIR gri	DIP DIR uta	Weath.	Ert. Rock Stren	RQ	ERSIST	Apert	Rough	INFILL	ll Wea	Ground	Jt Ori	RM	GS
M3_EAST_ctrl5_to_ctrl6	0.00	0.30	11249.74	5494.438	1357.637	11-12-2009	CBPH	foliation	10	122	132	HW	R1	<25	10-20M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	34	29
M3_EAST_ctrl5_to_ctrl6	0.00	0.30	11249.74	5494.438	1357.637	11-12-2009	CBPH	joint	65	194	204	HW	R1	<25	1-3M	0.1-1.0MM	SMOOTH	SOFT; <5MM	HIGHLY	DRY	FAIR	41	36
M3_EAST_ctrl5_to_ctrl6	3.00	3.00	11247.43	5493.829	1357.141	11-12-2009	CBPH	foliation	15	233	243	HW	R1	<25	10-20M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	34	29
M3_EAST_ctrl5_to_ctrl6	3.70	3.70	11246.97	5493.707	1357.042	11-12-2009	CBPH	vein	85	32	042	HW	R2	<25	1-3M	>5MM	VROUGH	HARD; >5MM	HIGHLY	DRY	FAIR	48	43
M3_EAST_ctrl5_to_ctrl6	6.00	6.00	11244.66	5493.098	1356.547	11-12-2009	CBPH	foliation	20	327	337	HW	R1	<25	10-20M	1-5MM	SLROUGH	SOFT; <5MM	HIGHLY	DRY	FAIR	27	22
M3_EAST_ctrl5_to_ctrl6	9.30	9.30	11241.43	5492.245	1355.853	11-12-2009	CBPH	joint	37	74	084	HW	R1	<25	3-10M	0.1-1.0MM	SMOOTH	NONE	HIGHLY	DRY	FAIR	48	43
M3_EAST_ctrl5_to_ctrl6	6.00	6.00	11244.66	5493.098	1356.547	11-12-2009	CBPH	shear	80	342	352	MW	R3	25-50	3-10M	>5MM	VROUGH	SOFT; >5MM	MODERATE	DRY	FAIR	53	48
M3_EAST_ctrl5_to_ctrl6	7.00	7.00	11243.74	5492.854	1356.348	11-12-2009	CBPH	joint	85	65	075	MW	R3	25-50	<1M	1-5MM	SLROUGH	HARD; <5MM	MODERATE	DRY	FAIR	49	44
M3_EAST_ctrl5_to_ctrl6	7.80	7.80	11242.82	5492.611	1356.15	11-12-2009	CBPH	joint	79	151	161	MW	R3	25-50	1-3M	1-5MM	SLROUGH	SOFT; <5MM	MODERATE	DRY	FAIR	45	40
M3_EAST_ctrl5_to_ctrl6	9.00	9.00	11241.89	5492.367	1355.952	11-12-2009	CBPH	joint	77	66	076	MW	R3	25-50	1-3M	>5MM	ROUGH	HARD; >5MM	MODERATE	DRY	FAIR	46	41
M3_EAST_ctrl5_to_ctrl6	9.00	9.00	11241.89	5492.367	1355.952	11-12-2009	CBPH	foliation	15	309	319	MW	R3	25-50	10-20M	NONE	SMOOTH	NONE	MODERATE	DRY	FAIR	44	39
M3_EAST_ctrl5_to_ctrl6	9.30	9.30	11241.43	5492.245	1355.853	11-12-2009	CBPH	joint	87	144	154	MW	R3	25-50	<1M	>5MM	ROUGH	HARD; >5MM	MODERATE	DRY	FAIR	46	41
M3_EAST_ctrl5_to_ctrl6	12.90	12.90	11238.2	5491.392	1355.159	11-12-2009	CBPH	foliation	10	194	204	EW	R0	<25	3-10M	NONE	SMOOTH	NONE	DECOMPOSEI	DRY	FAIR	33	28
M3_EAST_ctrl5_to_ctrl6	14.80	14.80	11236.36	5490.905	1354.763	11-12-2009	CBPH	shear	75	180	190	HW	R1	<25	1-3M	0.1-1.0MM	SMOOTH	SOFT; <5MM	HIGHLY	DRY	FAIR	36	31
M3_EAST_ctrl5_to_ctrl6	16.50	16.50	11234.97	5490.54	1354.465	11-12-2009	CBPH	joint	77	54	064	HW	R1	<25	1-3M	0.1-1.0MM	ROUGH	NONE	HIGHLY	DRY	FAIR	44	39
M3_EAST_ctrl5_to_ctrl4	0.00	1.50	11251.16	5494.683	1356.5	12-12-2009	CBPH	shear	6	137	147	HW	R1	<25	>20M	0.1-1.0MM	SMOOTH	SOFT; <5MM	HIGHLY	DRY	FAIR	42	37
M3_EAST_ctrl5_to_ctrl4	0.50	2.00	11251.63	5494.745	1356.521	12-12-2009	CBPH	vein	6	50	060	HW	R1	<25	3-10M	1-5MM	ROUGH	SOFT; <5MM	HIGHLY	DRY	FAIR	30	25
M3_EAST_ctrl5_to_ctrl4	0.50	2.00	11251.63	5494.745	1356.521	12-12-2009	CBPH	foliation	8	157	167	HW	R1	<25	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl5_to_ctrl4	2.00	2.00	11252.11	5494.807	1356.542	12-12-2009	CBPH	joint	85	318	328	HW	R1	<25	3-10M	0.1-1.0MM	SLROUGH	SOFT; <5MM	HIGHLY	DRY	FAIR	41	36
M3_EAST_ctrl5_to_ctrl4	2.00	2.00	11252.11	5494.807	1356.542	12-12-2009	CBPH	vein	70	212	222	HW	R1	<25	<1M	1-5MM	SMOOTH	SOFT; <5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl5_to_ctrl4	6.00	7.00	11256.42	5495.362	1356.73	12-12-2009	CBPH	foliation	12	142	152	HW	R1	<25	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl5_to_ctrl4	6.00	6.50	11255.94	5495.301	1356.709	12-12-2009	CBPH	joint	74	340	350	HW	R1	<25	1-3M	1-5MM	ROUGH	SOFT; <5MM	HIGHLY	DRY	FAIR	37	32
M3_EAST_ctrl5_to_ctrl4	6.50	7.00	11256.9	5495.424	1356.751	12-12-2009	CBPH	joint	84	232	242	MW	R2	<25	1-3M	0.1-1.0MM	SMOOTH	SOFT; <5MM	MODERATE	DRY	FAIR	37	32
M3_EAST_ctrl5_to_ctrl4	7.00	7.50	11257.38	5495.486	1356.772	12-12-2009	CBPH	joint	85	280	290	MW	R2	<25	3-10M	0.1-1.0MM	ROUGH	HARD; <5MM	MODERATE	DRY	FAIR	41	36
M3_EAST_ctrl5_to_ctrl4	7.50	7.50	11257.38	5495.486	1356.772	12-12-2009	CBPH	foliation	16	155	165	MW	R2	25-50	3-10M	NONE	SMOOTH	NONE	MODERATE	DRY	FAIR	43	38
M3_EAST_ctrl5_to_ctrl4	8.50	8.50	11258.33	5495.609	1356.814	12-12-2009	CBPH	joint	82	235	245	MW	R2	25-50	3-10M	0.1-1.0MM	ROUGH	HARD; <5MM	MODERATE	DRY	FAIR	48	43
M3_EAST_ctrl5_to_ctrl4	9.00	9.00	11258.81	5495.671	1356.835	12-12-2009	CBPH	joint	64	336	346	MW	R2	25-50	1-3M	0.1-1.0MM	ROUGH	HARD; <5MM	MODERATE	DRY	FAIR	50	45
M3_EAST_ctrl5_to_ctrl4	10.00	10.50	11259.77	5495.794	1356.877	12-12-2009	CBPH	joint	88	54	064	MW	R1	<25	1-3M	0.1-1.0MM	SLROUGH	SOFT; <5MM	MODERATE	DRY	FAIR	40	35
M3_EAST_ctrl5_to_ctrl4	10.00	10.50	11259.77	5495.794	1356.877	12-12-2009	CBPH	joint	81	315	325	MW	R1	<25	3-10M	0.1-1.0MM	ROUGH	SOFT; <5MM	MODERATE	DRY	FAIR	40	35
M3_EAST_ctrl5_to_ctrl4	10.00	10.00	11259.77	5495.794	1356.877	12-12-2009	CBPH	foliation	7	90	100	MW	R1	<25	3-10M	NONE	SMOOTH	NONE	MODERATE	DRY	FAIR	37	32
M3_EAST_ctrl5_to_ctrl4	11.50	11.50	11261.2	5495.979	1356.94	12-12-2009	CBPH	joint	76	226	236	MW	R1	<25	1-3M	0.1-1.0MM	ROUGH	SOFT; <5MM	MODERATE	DRY	FAIR	52	47
M3_EAST_ctrl5_to_ctrl4	11.50	11.50	11261.2	5495.979	1356.94	12-12-2009	CBPH	joint	88	225	235	MW	R1	<25	3-10M	1-5MM	ROUGH	SOFT; <5MM	MODERATE	DRY	FAIR	42	37

MAIN PIT														A2	A4= E1+E2+E3+E4+E5					A5	B	Bieniawski's	Bieniawski's
Line	From	To	y	x	z	Date mappe	Rock Cod	DISCONT INUI	DIP	DIP DIR gri	DIP DIR utm	Weath.	Est. Rock Stren Inde	RQI	ERSIST	Apert	Rough	INFILL	ll Wea	Ground	Jt Ori	RM	GS
M3_EAST_ctrl5_to_ctrl4	11.50	11.50	11261.2	5495.979	1356.94	12-12-2009	CBPH	joint	76	226	236	MW	R1	<25	1-3M	0.1-1.0MM	ROUGH	SOFT;<5MM	MODERATE	DRY	FAIR	52	47
M3_EAST_ctrl5_to_ctrl4	11.50	11.50	11261.2	5495.979	1356.94	12-12-2009	CBPH	joint	88	225	235	MW	R1	<25	3-10M	1-5MM	ROUGH	SOFT;<5MM	MODERATE	DRY	FAIR	42	37
M3_EAST_ctrl5_to_ctrl4	pt4	pt4	11261.68	5496.041	1356.961	12-12-2009	CBPH	joint	84	55	065	MW	R1	<25	3-10M	0.1-1.0MM	SLROUGH	SOFT;<5MM	MODERATE	DRY	FAIR	38	33
M3_EAST_ctrl5_to_ctrl4	pt4	pt4	11261.68	5496.041	1356.961	12-12-2009	CBPH	foliation	12	45	055	MW	R1	<25	3-10M	NONE	SMOOTH	NONE	MODERATE	DRY	FAIR	37	32
M3_EAST_ctrl5_to_ctrl4	7.30	8.00	11257.38	5495.486	1356.772	12-12-2009	CBPH	shear	12	126	136	HW	R1	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	36	31
M3_EAST_ctrl5_to_ctrl6	0.00	0.00	11250.2	5494.56	1356.458	12-12-2009	CBPH	foliation	15	125	135												
M3_EAST_ctrl5_to_ctrl6	1.00	1.00	11233.08	5490.147	1354.276	12-12-2009	CBPH	foliation	22	200	210												
M3_EAST_ctrl5_to_ctrl6	3.00	3.00	11231.13	5489.85	1354.294	12-12-2009	CBPH	foliation	9	250	260												
M3_EAST_ctrl5_to_ctrl6	4.50	4.50	11229.67	5489.627	1354.307	12-12-2009	CBPH	foliation	10	335	345												
M3_EAST_ctrl5_to_ctrl6	5.50	5.50	11228.7	5489.478	1354.316	12-12-2009	CBPH	foliation	12	327	337												
M3_EAST_ctrl5_to_ctrl6	7.00	7.00	11227.24	5489.255	1354.329	12-12-2009	CBPH	foliation	20	225	235												
M3_EAST_ctrl5_to_ctrl6	8.50	8.50	11225.79	5489.032	1354.343	12-12-2009	CBPH	foliation	9	286	296												
M3_EAST_ctrl5_to_ctrl6	9.50	9.50	11224.81	5488.883	1354.352	12-12-2009	CBPH	foliation	13	252	262												
M3_EAST_ctrl4_to_ctrl3	0.00	pt4	11261.68	5496.041	1356.961	14-12-2009	CBPH	shear	11	117	127	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl4_to_ctrl3	0.00	pt4	11261.68	5496.041	1356.961	14-12-2009	CBPH	foliation	18	105	115	MW	R1	<25	3-10M	NONE	SMOOTH	NONE	MODERATE	DRY	FAIR	37	32
M3_EAST_ctrl4_to_ctrl3	0.50	1.50	11262.67	5496.125	1356.992	14-12-2009	CBPH	joint	88	224	234	MW	R1	<25	3-10M	0.1-1.0MM	ROUGH	SOFT;<5MM	MODERATE	DRY	FAIR	40	35
M3_EAST_ctrl4_to_ctrl3	0.50	1.50	11262.67	5496.125	1356.992	14-12-2009	CBPH	joint	83	125	135	MW	R1	<25	1-3M	0.1-1.0MM	ROUGH	SOFT;<5MM	MODERATE	DRY	FAIR	42	37
M3_EAST_ctrl4_to_ctrl3	3.00	3.00	11264.65	5496.292	1357.053	14-12-2009	CBPH	foliation	15	104	114	MW	R1	<25	3-10M	NONE	SMOOTH	NONE	MODERATE	DRY	FAIR	37	32
M3_EAST_ctrl4_to_ctrl3	3.00	3.50	11265.14	5496.333	1357.069	14-12-2009	CBPH	joint	86	307	317	MW	R1	<25	3-10M	1-5MM	ROUGH	HARD;<5MM	MODERATE	DRY	FAIR	39	34
M3_EAST_ctrl4_to_ctrl3	3.00	3.50	11265.14	5496.333	1357.069	14-12-2009	CBPH	joint	80	222	232	MW	R1	<25	1-3M	0.1-1.0MM	ROUGH	SOFT;<5MM	MODERATE	DRY	FAIR	42	37
M3_EAST_ctrl4_to_ctrl3	3.00	4.00	11265.14	5496.333	1357.069	14-12-2009	CBPH	shear	8	105	115	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl4_to_ctrl3	5.50	6.50	11267.62	5496.542	1357.146	14-12-2009	CBPH	foliation	11	110	120	MW	R1	<25	3-10M	NONE	SMOOTH	NONE	MODERATE	DRY	FAIR	37	32
M3_EAST_ctrl4_to_ctrl3	5.50	6.50	11267.62	5496.542	1357.146	14-12-2009	CBPH	joint	87	236	246	MW	R1	<25	3-10M	1-5MM	ROUGH	HARD;<5MM	MODERATE	DRY	FAIR	39	34
M3_EAST_ctrl4_to_ctrl3	5.50	6.50	11267.62	5496.542	1357.146	14-12-2009	CBPH	joint	86	312	322	MW	R1	<25	3-10M	0.1-1.0MM	ROUGH	HARD;<5MM	MODERATE	DRY	FAIR	42	37
M3_EAST_ctrl4_to_ctrl3	5.50	6.50	11267.62	5496.542	1357.146	14-12-2009	CBPH	shear	20	160	170	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl4_to_ctrl3	10.00	10.50	11271.57	5496.876	1357.269	14-12-2009	CBPH	foliation	20	224	234	HW	R0	<25	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	34	29
M3_EAST_ctrl4_to_ctrl3	10.00	10.50	11271.57	5496.876	1357.269	14-12-2009	CBPH	shear	47	164	174	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl4_to_ctrl3	pt3	pt3	11273.55	5497.043	1357.331	14-12-2009	CBPH	foliation	10	241	251	HW	R0	<25	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	34	29
M3_EAST_ctrl3_to_ctrl2	0.00	1.50	11274.51	5497.082	1357.422	14-12-2009	CBPH	foliation	5	18	028	HW	R1	<25	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl3_to_ctrl2	2.50	3.50	11276.44	5497.16	1357.603	14-12-2009	CBPH	joint	85	60	070	HW	R1	<25	3-10M	1-5MM	ROUGH	SOFT;<5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl3_to_ctrl2	3.50	3.50	11276.92	5497.179	1357.648	14-12-2009	CBPH	foliation	7	132	142	HW	R1	<25	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl3_to_ctrl2	4.00	4.50	11277.4	5497.199	1357.693	14-12-2009	CBPH	joint	60	149	153	HW	R1	<25	1-3M	0.1-1.0MM	ROUGH	SOFT;<5MM	HIGHLY	DRY	FAIR	40	35
M3_EAST_ctrl3_to_ctrl2	5.50	6.50	11279.32	5497.277	1357.874	14-12-2009	CBPH	shear	20	115	125	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30

MAIN PIT

														A2	A4= E1+E2+E3+E4+E5					A5	B	Bieniawski'	Bieniawski's
Line	From	To	y	x	z	Date mappe	Rock Cod	DISCONTINUIT	DIP	DIP DIR gri	DIP DIR utm	Weath.	Ert. Rock Stren	RQ	ERSIST	Apert	Rough	INFILL	ll Weat	Ground	Jt Ori	RM	GS
M3_EAST_ctrl3_to_ctrl2	6.00	6.50	11279.32	5497.277	1357.874	14-12-2009	CBPH	joint	88	344	354	HW	R1	25-50	1-3M	1-5MM	ROUGH	HARD;<5MM	HIGHLY	DRY	FAIR	44	39
M3_EAST_ctrl3_to_ctrl2	6.00	6.50	11279.32	5497.277	1357.874	14-12-2009	CBPH	joint	77	48	058	HW	R1	25-50	1-3M	1-5MM	ROUGH	HARD;<5MM	HIGHLY	DRY	FAIR	44	39
M3_EAST_ctrl3_to_ctrl2	6.00	6.50	11279.32	5497.277	1357.874	14-12-2009	CBPH	foliation	16	88	098	HW	R1	25-50	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	40	35
M3_EAST_ctrl3_to_ctrl2	7.50	8.00	11280.77	5497.335	1358.01	14-12-2009	CBPH	vein	88	42	052	HW	R1	25-50	3-10M	>5MM	VROUGH	SOFT;>5MM	HIGHLY	DRY	FAIR	38	33
M3_EAST_ctrl3_to_ctrl2	7.50	8.00	11280.77	5497.335	1358.01	14-12-2009	CBPH	joint	85	142	152	HW	R1	25-50	3-10M	1-5MM	ROUGH	HARD;<5MM	HIGHLY	DRY	FAIR	42	37
M3_EAST_ctrl3_to_ctrl2	8.00	8.00	11281.25	5497.355	1358.055	14-12-2009	CBPH	shear	17	150	160	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl3_to_ctrl2	8.00	8.00	11281.25	5497.355	1358.055	14-12-2009	CBPH	foliation	15	165	175	HW	R1	25-50	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	40	35
M3_EAST_ctrl3_to_ctrl2	9.00	10.00	11282.69	5497.413	1358.191	14-12-2009	CBPH	joint	90	332	342	HW	R1	25-50	1-3M	0.1-1.0MM	ROUGH	HARD;<5MM	HIGHLY	DRY	FAIR	47	42
M3_EAST_ctrl3_to_ctrl2	9.00	10.00	11282.69	5497.413	1358.191	14-12-2009	CBPH	joint	89	230	240	HW	R1	25-50	1-3M	0.1-1.0MM	ROUGH	HARD;<5MM	HIGHLY	DRY	FAIR	47	42
M3_EAST_ctrl3_to_ctrl2	9.50	9.50	11282.69	5497.413	1358.191	14-12-2009	CBPH	shear	19	156	166	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl3_to_ctrl2	9.50	9.50	11282.69	5497.413	1358.191	14-12-2009	CBPH	foliation	14	152	162	HW	R1	25-50	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	40	35
M3_EAST_ctrl3_to_ctrl2	10.50	11.00	11283.65	5497.452	1358.282	14-12-2009	CBPH	vein	88	40	050	HW	R1	25-50	1-3M	>5MM	VROUGH	HARD;>5MM	HIGHLY	DRY	FAIR	40	35
M3_EAST_ctrl3_to_ctrl2	10.50	11.50	11283.65	5497.452	1358.282	14-12-2009	CBPH	joint	83	140	150	HW	R1	25-50	3-10M	0.1-1.0MM	ROUGH	HARD;<5MM	HIGHLY	DRY	FAIR	45	40
M3_EAST_ctrl3_to_ctrl2	pt2	pt2	11285.58	5497.53	1358.463	14-12-2009	CBPH	shear	14	184	194	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl2_to_ctrl1	0.00	1.00	11286.06	5497.663	1358.477	14-12-2009	CBPH	shear	25	167	177	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl2_to_ctrl1	0.00	1.00	11286.06	5497.663	1358.477	14-12-2009	CBPH	joint	83	23	033	HW	R0	<25	1-3M	0.1-1.0MM	ROUGH	SOFT;<5MM	HIGHLY	DRY	FAIR	39	34
M3_EAST_ctrl2_to_ctrl1	2.00	2.50	11287.5	5498.061	1358.519	14-12-2009	CBPH	shear	27	162	172	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl2_to_ctrl1	4.00	4.50	11289.42	5498.592	1358.575	14-12-2009	CBPH	shear	18	118	128	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl2_to_ctrl1	2.50	>20	11288.46	5498.326	1358.547	14-12-2009	COBX					HW	R0	<25							FAIR	-5	-10
M3_EAST_ctrl6_to_ctrl7	0.00	0.50	11234.05	5490.296	1354.267	14-12-2009	RES	joint	73	70	080	HW	R0	<25	1-3M	0.1-1.0MM	ROUGH	HARD;<5MM	HIGHLY	DRY	FAIR	39	34
M3_EAST_ctrl6_to_ctrl7	1.00	1.00	11233.08	5490.147	1354.276	14-12-2009	RES	foliation	27	175	185	HW	R0	<25	<1M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	38	33
M3_EAST_ctrl6_to_ctrl7	2.00	2.00	11232.1	5489.999	1354.285	14-12-2009	RES	foliation	27	22	032	HW	R0	<25	<1M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	38	33
M3_EAST_ctrl6_to_ctrl7	3.00	3.00	11231.13	5489.85	1354.294	14-12-2009	RES	foliation	17	35	045	HW	R0	<25	<1M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	38	33
M3_EAST_ctrl6_to_ctrl7	4.00	4.00	11230.16	5489.701	1354.303	14-12-2009	RES	foliation	42	218	228	HW	R0	<25	<1M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	38	33
M3_EAST_ctrl7_to_ctrl8	2.00	2.00	11218.52	5487.896	1354.387	14-12-2009	RES	foliation	20	198	208	HW	R0	<25	<1M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	38	33
M3_EAST_ctrl7_to_ctrl8	3.50	3.50	11217.07	5487.658	1354.383	14-12-2009	RES	foliation	50	253	263	HW	R0	<25	<1M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	38	33
M3_EAST_ctrl7_to_ctrl8	6.00	6.00	11214.67	5487.261	1354.377	14-12-2009	RES	foliation	84	14	024	HW	R0	<25	<1M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	38	33
M3_EAST_ctrl8_to_ctrl9	11.50	12.00	11194.3	5482.452	1354.295	14-12-2009	RES	shear	24	208	218	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl8_to_ctrl9	13.50	14.00	11192.43	5481.891	1354.285	14-12-2009	RES	shear	20	204	214	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl8_to_ctrl9	14.00	14.00	11191.96	5481.751	1354.282	14-12-2009	RES	joint	82	65	075	HW	R0	<25	1-3M	0.1-1.0MM	ROUGH	SOFT;<5MM	HIGHLY	DRY	FAIR	39	34
M3_EAST_ctrl8_to_ctrl9	14.00	14.50	11191.49	5481.611	1354.28	14-12-2009	RES	joint	88	342	352	HW	R0	<25	1-3M	0.1-1.0MM	ROUGH	SOFT;<5MM	HIGHLY	DRY	FAIR	39	34
M3_EAST_ctrl9_to_ctrl10	0.00	0.00	11189.62	5481.051	1354.27	14-12-2009	CBPH	foliation	30	200	210	HW	R0	<25	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	34	29
M3_EAST_ctrl9_to_ctrl10	0.00	0.00	11189.62	5481.051	1354.27	14-12-2009	CBPH	vein	88	60	070	HW	R0	<25	1-3M	>5MM	ROUGH	SOFT;>5MM	HIGHLY	DRY	FAIR	33	28

MAIN PIT														A2	A4= E1+E2+E3+E4+E5					A5	B	Bieniawski'	Bieniawski's
Line	From	To	y	x	z	Date mappe	Rock Cod	DISCONT INUI	DIP	DIP DIR gri	DIP DIR utm	Weath.	Est. Rock Stren	RQI	ERSIST	Apert	Rough	INFILL	ll Wea	Ground	Jt Ori	RM	GS
M3_EAST_ctrl9_to_ctrl10	0.00	0.00	11189.62	5481.051	1354.27	14-12-2009	CBPH	shear	15	200	210	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl9_to_ctrl10	1.00	1.50	11188.77	5480.58	1354.104	14-12-2009	CBPH	shear	16	188	198	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl9_to_ctrl10	1.00	1.50	11188.77	5480.58	1354.104	14-12-2009	CBPH	foliation	14	223	233	HW	R0	<25	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	34	29
M3_EAST_ctrl9_to_ctrl10	1.00	1.50	11188.77	5480.58	1354.104	14-12-2009	CBPH	joint	79	158	168	HW	R0	<25	3-10M	1-5MM	ROUGH	SOFT;<5MM	HIGHLY	DRY	FAIR	34	29
M3_EAST_ctrl9_to_ctrl10	1.00	1.50	11188.77	5480.58	1354.104	14-12-2009	CBPH	joint	82	77	087	HW	R0	<25	3-10M	1-5MM	ROUGH	SOFT;<5MM	HIGHLY	DRY	FAIR	39	34
M3_EAST_ctrl9_to_ctrl10	0.00	1.50	11189.2	5480.816	1354.187	14-12-2009	CBPH	joint	84	144	154	HW	R0	<25	3-10M	1-5MM	ROUGH	SOFT;<5MM	HIGHLY	DRY	FAIR	39	34
M3_EAST_ctrl9_to_ctrl10	2.50	3.00	11187.49	5479.874	1353.855	14-12-2009	CBPH	vein	76	54	064	HW	R1	<25	1-3M	>5MM	ROUGH	SOFT;>5MM	HIGHLY	DRY	FAIR	44	39
M3_EAST_ctrl9_to_ctrl10	2.50	3.00	11187.49	5479.874	1353.855	14-12-2009	CBPH	foliation	11	182	192	HW	R1	<25	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl9_to_ctrl10	2.50	3.00	11187.49	5479.874	1353.855	14-12-2009	CBPH	joint	88	273	283	HW	R1	<25	1-3M	0.1-1.0MM	ROUGH	SOFT;<5MM	HIGHLY	DRY	FAIR	38	33
M3_EAST_ctrl9_to_ctrl10	2.50	3.00	11187.49	5479.874	1353.855	14-12-2009	CBPH	joint	86	130	140	HW	R1	<25	3-10M	1-5MM	ROUGH	SOFT;<5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl9_to_ctrl10	3.00	6.00	11186.21	5479.168	1353.606	14-12-2009	CBPH	joint	85	135	145	HW	R1	<25	3-10M	1-5MM	SL ROUGH	HARD;<5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl9_to_ctrl10	4.50	5.00	11185.79	5478.932	1353.523	14-12-2009	CBPH	joint	84	50	060	HW	R1	<25	3-10M	0.1-1.0MM	SL ROUGH	HARD;<5MM	HIGHLY	DRY	FAIR	36	31
M3_EAST_ctrl9_to_ctrl10	5.00	5.00	11185.36	5478.697	1353.44	14-12-2009	CBPH	foliation	11	215	225	HW	R1	<25	<1M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	39	34
M3_EAST_ctrl9_to_ctrl10	4.50	5.00	11185.36	5478.697	1353.44	14-12-2009	CBPH	vein	86	0	010	HW	R1	<25	1-3M	>5MM	ROUGH	SOFT;>5MM	HIGHLY	DRY	FAIR	32	27
M3_EAST_ctrl9_to_ctrl10	6.50	7.50	11184.08	5477.931	1353.191	14-12-2009	CBPH	foliation	5	240	250	MW	R2	25-50	3-10M	NONE	SMOOTH	NONE	MODERATE	DRY	FAIR	43	38
M3_EAST_ctrl9_to_ctrl10	6.50	7.50	11184.08	5477.931	1353.191	14-12-2009	CBPH	joint	75	170	180	MW	R2	25-50	<1M	NONE	SL ROUGH	NONE	MODERATE	DRY	FAIR	54	49
M3_EAST_ctrl9_to_ctrl10	6.50	7.50	11184.08	5477.931	1353.191	14-12-2009	CBPH	joint	90	50	060	MW	R2	25-50	3-10M	NONE	SL ROUGH	NONE	MODERATE	DRY	FAIR	48	43
M3_EAST_ctrl9_to_ctrl10	6.50	7.50	11184.08	5477.931	1353.191	14-12-2009	CBPH	joint	68	188	198	MW	R2	25-50	3-10M	NONE	SL ROUGH	NONE	MODERATE	DRY	FAIR	48	43
M3_EAST_ctrl9_to_ctrl10	9.00	9.50	11181.95	5476.813	1352.776	14-12-2009	CBPH	foliation	3	274	284	MW	R2	25-50	3-10M	NONE	SMOOTH	NONE	MODERATE	DRY	FAIR	43	38
M3_EAST_ctrl9_to_ctrl10	9.00	9.50	11181.95	5476.813	1352.776	14-12-2009	CBPH	joint	82	134	144	MW	R2	25-50	3-10M	0.1-1.0MM	ROUGH	HARD;<5MM	MODERATE	DRY	FAIR	46	41
M3_EAST_ctrl9_to_ctrl10	9.00	9.50	11181.95	5476.813	1352.776	14-12-2009	CBPH	joint	86	72	082	MW	R2	25-50	3-10M	0.1-1.0MM	ROUGH	HARD;<5MM	MODERATE	DRY	FAIR	46	41
M3_EAST_ctrl9_to_ctrl10	10.00	10.50	11181.09	5476.343	1352.61	15-12-2009	CBPH	foliation	8	323	333	MW	R1	25-50	3-10M	NONE	SMOOTH	NONE	MODERATE	DRY	FAIR	42	37
M3_EAST_ctrl9_to_ctrl10	10.00	10.50	11181.09	5476.343	1352.61	15-12-2009	CBPH	vein	86	35	045	MW	R1	25-50	1-3M	1-5MM	ROUGH	HARD;<5MM	MODERATE	DRY	FAIR	51	46
M3_EAST_ctrl9_to_ctrl10	10.00	10.50	11181.09	5476.343	1352.61	15-12-2009	CBPH	joint	84	72	082	MW	R1	25-50	3-10M	0.1-1.0MM	SL ROUGH	SOFT;<5MM	MODERATE	DRY	FAIR	41	36
M3_EAST_ctrl9_to_ctrl10	10.00	10.50	11181.09	5476.343	1352.61	15-12-2009	CBPH	joint	74	130	140	MW	R1	25-50	3-10M	0.1-1.0MM	ROUGH	SOFT;<5MM	MODERATE	DRY	FAIR	43	38
M3_EAST_ctrl9_to_ctrl10	10.00	10.50	11181.09	5476.343	1352.61	15-12-2009	CBPH	shear	15	147	157	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl9_to_ctrl10	11.00	11.50	11180.24	5475.872	1352.444	15-12-2009	CBPH	foliation	5	268	278	MW	R1	25-50	3-10M	NONE	SMOOTH	NONE	MODERATE	DRY	FAIR	42	37
M3_EAST_ctrl9_to_ctrl10	11.00	11.50	11180.24	5475.872	1352.444	15-12-2009	CBPH	joint	77	65	075	MW	R1	25-50	1-3M	0.1-1.0MM	SL ROUGH	SOFT;<5MM	MODERATE	DRY	FAIR	45	40
M3_EAST_ctrl9_to_ctrl10	11.00	11.50	11180.24	5475.872	1352.444	15-12-2009	CBPH	joint	84	128	138	MW	R1	25-50	3-10M	0.1-1.0MM	SL ROUGH	HARD;<5MM	MODERATE	DRY	FAIR	43	38
M3_EAST_ctrl9_to_ctrl10	11.00	11.50	11180.24	5475.872	1352.444	15-12-2009	CBPH	shear	11	61	071	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl9_to_ctrl10	12.00	12.50	11179.39	5475.401	1352.278	15-12-2009	CBPH	vein	76	210	220	MW	R1	25-50	3-10M	>5MM	ROUGH	SOFT;>5MM	MODERATE	DRY	FAIR	49	44
M3_EAST_ctrl9_to_ctrl10	12.00	12.50	11179.39	5475.401	1352.278	15-12-2009	CBPH	foliation	6	292	302	MW	R1	25-50	3-10M	NONE	SMOOTH	NONE	MODERATE	DRY	FAIR	42	37
M3_EAST_ctrl9_to_ctrl10	12.00	12.50	11179.39	5475.401	1352.278	15-12-2009	CBPH	joint	84	237	247	MW	R1	25-50	1-3M	0.1-1.0MM	ROUGH	HARD;<5MM	MODERATE	DRY	FAIR	49	44

MAIN PIT														A2	A4= E1+E2+E3+E4+E5					A5	B	Bieniawski'	Bieniawski's
Line	From	To	y	x	z	Date mappe	Rock Cod	DISCONTINUI	DIP	DIP DIR gri	DIP DIR utr	Weath.	Est. Rock Stren	RQ	ERSIST	Apert	Rough	INFILL	ll Wea	Ground	Jt Ori	RM	GS
M3_EAST_ctrl9_to_ctrl10	12.00	12.50	11179.39	5475.401	1352.278	15-12-2009	CBPH	joint	70	142	152	MW	R1	25-50	3-10M	0.1-1.0MM	ROUGH	HARD;<5MM	MODERATE	DRY	FAIR	45	40
M3_EAST_ctrl9_to_ctrl10	12.00	12.50	11179.39	5475.401	1352.278	15-12-2009	CBPH	shear	14	65	075	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl9_to_ctrl10	13.00	13.00	11178.53	5474.93	1352.112	15-12-2009	CBPH	foliation	11	355	005	MW	R1	25-50	3-10M	NONE	SMOOTH	NONE	MODERATE	DRY	FAIR	42	37
M3_EAST_ctrl9_to_ctrl10	13.00	13.00	11178.53	5474.93	1352.112	15-12-2009	CBPH	joint	73	234	244	MW	R1	25-50	1-3M	0.1-1.0MM	SLROUGH	HARD;<5MM	MODERATE	DRY	FAIR	45	40
M3_EAST_ctrl9_to_ctrl10	13.00	13.00	11178.53	5474.93	1352.112	15-12-2009	CBPH	joint	75	137	147	MW	R1	25-50	3-10M	0.1-1.0MM	SLROUGH	HARD;<5MM	MODERATE	DRY	FAIR	43	38
M3_EAST_ctrl9_to_ctrl10	13.50	13.50	11178.11	5474.695	1352.029	15-12-2009	CBPH	foliation	20	28	038	MW	R1	25-50	3-10M	NONE	SMOOTH	NONE	MODERATE	DRY	FAIR	42	37
M3_EAST_ctrl9_to_ctrl10	13.50	13.50	11178.11	5474.695	1352.029	15-12-2009	CBPH	shear	30	13	023	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl9_to_ctrl10	14.00	14.50	11177.68	5474.459	1351.946	15-12-2009	CBPH	foliation	21	118	128	MW	R1	25-50	3-10M	NONE	SMOOTH	NONE	MODERATE	DRY	FAIR	42	37
M3_EAST_ctrl9_to_ctrl10	14.00	14.50	11177.68	5474.459	1351.946	15-12-2009	CBPH	joint	84	242	252	MW	R1	25-50	1-3M	0.1-1.0MM	SLROUGH	SOFT;<5MM	MODERATE	DRY	FAIR	43	38
M3_EAST_ctrl9_to_ctrl10	14.00	14.50	11177.68	5474.459	1351.946	15-12-2009	CBPH	joint	73	150	160	MW	R1	25-50	1-3M	0.1-1.0MM	SLROUGH	SOFT;<5MM	MODERATE	DRY	FAIR	43	38
M3_EAST_ctrl9_to_ctrl10	14.00	14.50	11177.68	5474.459	1351.946	15-12-2009	CBPH	shear	8	293	303	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl9_to_ctrl10	14.00	14.50	11177.26	5474.224	1351.863	15-12-2009	CBPH	joint	79	248	258	MW	R1	<25	3-10M	0.1-1.0MM	ROUGH	HARD;<5MM	MODERATE	DRY	FAIR	40	35
M3_EAST_ctrl9_to_ctrl10	14.00	14.50	11177.26	5474.224	1351.863	15-12-2009	CBPH	joint	70	165	175	MW	R1	<25	3-10M	0.1-1.0MM	ROUGH	HARD;<5MM	MODERATE	DRY	FAIR	40	35
M3_EAST_ctrl9_to_ctrl10	14.00	14.50	11177.26	5474.224	1351.863	15-12-2009	CBPH	foliation	10	78	088	MW	R1	<25	3-10M	NONE	SMOOTH	NONE	MODERATE	DRY	FAIR	37	32
M3_EAST_ctrl9_to_ctrl10	15.50	16.00	11175.98	5473.518	1351.614	15-12-2009	CBPH	foliation	9	306	316	MW	R1	25-50	3-10M	NONE	SMOOTH	NONE	MODERATE	DRY	FAIR	42	37
M3_EAST_ctrl9_to_ctrl10	15.50	16.00	11175.98	5473.518	1351.614	15-12-2009	CBPH	vein	82	242	252	MW	R1	25-50	1-3M	>5MM	ROUGH	SOFT;>5MM	MODERATE	DRY	FAIR	46	41
M3_EAST_ctrl9_to_ctrl10	15.50	16.00	11175.98	5473.518	1351.614	15-12-2009	CBPH	joint	81	262	272	MW	R1	25-50	<1M	0.1-1.0MM	SLROUGH	HARD;<5MM	MODERATE	DRY	FAIR	47	42
M3_EAST_ctrl9_to_ctrl10	15.50	16.00	11175.98	5473.518	1351.614	15-12-2009	CBPH	joint	80	138	148	MW	R1	25-50	<1M	0.1-1.0MM	SLROUGH	HARD;<5MM	MODERATE	DRY	FAIR	47	42
M3_EAST_ctrl9_to_ctrl10	15.50	16.00	11175.98	5473.518	1351.614	15-12-2009	CBPH	thrust	69	58	068	HW	R0	<25	10-20M	1-5MM	SMOOTH	SOFT;<5MM	HIGHLY	DRY	FAIR	27	22
M3_EAST_ctrl9_to_ctrl10	15.50	16.00	11175.98	5473.518	1351.614	15-12-2009	CBPH	shear	26	88	098	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl9_to_ctrl10	15.50	16.00	11175.98	5473.518	1351.614	15-12-2009	CBPH	foliation	2	35	045	MW	R1	<25	3-10M	NONE	SMOOTH	NONE	MODERATE	DRY	FAIR	37	32
M3_EAST_ctrl9_to_ctrl10	17.00	17.50	11175.12	5473.047	1351.448	15-12-2009	CBPH	thrust	32	34	044	HW	R0	<25	3-10M	1-5MM	SMOOTH	SOFT;<5MM	HIGHLY	DRY	FAIR	28	23
M3_EAST_ctrl9_to_ctrl10	17.00	17.50	11175.12	5473.047	1351.448	15-12-2009	CBPH	shear	38	95	105	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl9_to_ctrl10	17.00	17.50	11175.12	5473.047	1351.448	15-12-2009	CBPH	foliation					R0	<25	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	34	29
M3_EAST_ctrl9_to_ctrl10	17.00	17.50	11175.12	5473.047	1351.448	15-12-2009	CBPH	foliation	35	38	048	HW	R0	<25	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	34	29
M3_EAST_ctrl10_to_ctrl11	pg10	6.00	11171.23	5470.977	1350.938	15-12-2009	CBPH	shear	15	15	025	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl10_to_ctrl11	5.00	9.00	11167.3	5468.946	1350.618	15-12-2009	CBPH	shear	11	273	283	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl10_to_ctrl11	8.00	15.00	11163.37	5466.915	1350.298	15-12-2009	CBPH	shear	5	254	264	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl10_to_ctrl11	12.00	15.00	11161.62	5466.012	1350.156	15-12-2009	CBPH	foliation	12	232	242	HW	R0	<25	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	34	29
M3_EAST_ctrl10_to_ctrl11	12.00	15.00	11161.62	5466.012	1350.156	15-12-2009	CBPH	foliation	35	245	255	HW	R0	<25	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	34	29
M3_EAST_ctrl10_to_ctrl11	12.00	15.00	11161.62	5466.012	1350.156	15-12-2009	CBPH	shear	10	248	258	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl10_to_ctrl11	12.00	15.00	11161.62	5466.012	1350.156	15-12-2009	CBPH	foliation	26	195	205	HW	R1	25-50	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	40	35
M3_EAST_ctrl11_to_ctrl12	11.00	pg12	11144.85	5456.617	1348.703	15-12-2009	CBPH	shear	26	232	242	EW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	DECOMPOSEI	DRY	FAIR	34	29

MAIN PIT														A2	A4= E1+E2+E3+E4+E5					A5	B	Bieniawski'	Bieniawski'
Line	From	To	y	x	z	Date mappe	Rock Cod	DISCONTINUI	DIP	DIP DIR gri	DIP DIR utm	Weath.	Est. Rock Stren	RQ	ERSIST	Apert	Rough	INFILL	ll Weat	Ground	Jt Ori	RM	GS
M3_EAST_ctrl12_to_ctrl13	pg12	1.00	11144	5456.137	1348.662	15-12-2009	CBPH	shear	38	260	270	EW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	DECOMPOSED	DRY	FAIR	34	29
M3_EAST_ctrl12_to_ctrl13	pg12	1.00	11144	5456.137	1348.662	15-12-2009	CBPH	foliation	42	235	245	HW	R0	<25	3-10M	NONE	SMOOTH	NONE	DECOMPOSED	DRY	FAIR	33	28
M3_EAST_ctrl12_to_ctrl13	1.50	2.00	11143.14	5455.679	1348.655	15-12-2009	CBPH	shear	20	218	228	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	DECOMPOSED	DRY	FAIR	34	29
M3_EAST_ctrl12_to_ctrl13	1.50	2.00	11143.14	5455.679	1348.655	15-12-2009	CBPH	foliation	62	255	265	HW	R0	<25	3-10M	NONE	SMOOTH	NONE	DECOMPOSED	DRY	FAIR	33	28
M3_EAST_ctrl12_to_ctrl13	2.50	3.00	11142.28	5455.222	1348.648	15-12-2009	CBPH	shear	7	218	228	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl12_to_ctrl13	2.50	3.00	11142.28	5455.222	1348.648	15-12-2009	COBX	foliation	55	58	068	HW	R0	<25	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	34	29
M3_EAST_ctrl12_to_ctrl13	3.50	4.00	11141.43	5454.764	1348.641	15-12-2009	CBPH	shear	4	170	180	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl12_to_ctrl13	4.50	5.00	11140.57	5454.306	1348.634	15-12-2009	CBPH	shear	15	92	102	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl12_to_ctrl13	4.50	5.00	11140.57	5454.306	1348.634	15-12-2009	CBPH	joint	84	50	060	HW	R1	25-50	1-3M	0.1-1.0MM	ROUGH	HARD;<5MM	HIGHLY	DRY	FAIR	47	42
M3_EAST_ctrl12_to_ctrl13	4.50	5.00	11140.57	5454.306	1348.634	15-12-2009	CBPH	foliation	9	178	188	HW	R1	25-50	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	40	35
M3_EAST_ctrl12_to_ctrl13	6.00	6.50	11139.28	5453.62	1348.624	15-12-2009	CBPH	shear	13	123	133	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl12_to_ctrl13	6.00	6.50	11139.28	5453.62	1348.624	15-12-2009	CBPH	joint	89	45	055	HW	R1	25-50	1-3M	0.1-1.0MM	ROUGH	HARD;<5MM	HIGHLY	DRY	FAIR	47	42
M3_EAST_ctrl12_to_ctrl13	6.00	6.50	11139.28	5453.62	1348.624	15-12-2009	CBPH	foliation	5	142	152	HW	R1	25-50	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	40	35
M3_EAST_ctrl12_to_ctrl13	7.50	8.00	11137.99	5452.933	1348.614	15-12-2009	CBPH	shear	24	148	158	MW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	MODERATE	DRY	FAIR	37	32
M3_EAST_ctrl12_to_ctrl13	7.50	8.00	11137.99	5452.933	1348.614	15-12-2009	CBPH	foliation	6	105	115	MW	R1	25-50	3-10M	NONE	SMOOTH	NONE	MODERATE	DRY	FAIR	42	37
M3_EAST_ctrl12_to_ctrl13	7.50	8.00	11137.99	5452.933	1348.614	15-12-2009	CBPH	joint	82	55	065	MW	R1	25-50	1-3M	0.1-1.0MM	ROUGH	HARD;<5MM	MODERATE	DRY	FAIR	49	44
M3_EAST_ctrl12_to_ctrl13	7.50	8.00	11137.99	5452.933	1348.614	15-12-2009	CBPH	joint	86	304	314	MW	R1	25-50	3-10M	0.1-1.0MM	ROUGH	HARD;<5MM	MODERATE	DRY	FAIR	45	40
M3_EAST_ctrl12_to_ctrl13	5.00	8.00	11138.85	5453.391	1348.621	15-12-2009	CBPH	joint	84	122	132	MW	R1	25-50	3-10M	0.1-1.0MM	ROUGH	HARD;<5MM	MODERATE	DRY	FAIR	45	40
M3_EAST_ctrl12_to_ctrl13	9.00	10.00	11136.28	5452.018	1348.6	15-12-2009	CBPH	foliation	3	147	157	MW	R1	25-50	3-10M	NONE	SMOOTH	NONE	MODERATE	DRY	FAIR	42	37
M3_EAST_ctrl12_to_ctrl13	9.00	10.00	11136.28	5452.018	1348.6	15-12-2009	CBPH	joint	46	278	288	MW	R1	25-50	3-10M	0.1-1.0MM	ROUGH	HARD;<5MM	MODERATE	DRY	FAIR	52	47
M3_EAST_ctrl12_to_ctrl13	9.00	10.00	11136.28	5452.018	1348.6	15-12-2009	CBPH	joint	87	306	316	MW	R1	25-50	3-10M	0.1-1.0MM	ROUGH	HARD;<5MM	MODERATE	DRY	FAIR	47	42
M3_EAST_ctrl12_to_ctrl13	9.00	10.00	11136.28	5452.018	1348.6	15-12-2009	CBPH	joint	85	222	232	MW	R1	25-50	3-10M	0.1-1.0MM	ROUGH	HARD;<5MM	MODERATE	DRY	FAIR	47	42
M3_EAST_ctrl12_to_ctrl13	11.00	11.50	11134.99	5451.331	1348.59	15-12-2009	CBPH	foliation	5	118	128	MW	R1	25-50	3-10M	NONE	SMOOTH	NONE	MODERATE	DRY	FAIR	42	37
M3_EAST_ctrl12_to_ctrl13	11.00	11.50	11134.99	5451.331	1348.59	15-12-2009	CBPH	joint	62	267	277	MW	R1	25-50	3-10M	0.1-1.0MM	ROUGH	HARD;<5MM	MODERATE	DRY	FAIR	52	47
M3_EAST_ctrl12_to_ctrl13	11.00	11.50	11134.99	5451.331	1348.59	15-12-2009	CBPH	joint	84	232	242	MW	R1	25-50	3-10M	0.1-1.0MM	ROUGH	HARD;<5MM	MODERATE	DRY	FAIR	47	42
M3_EAST_ctrl12_to_ctrl13	11.00	11.50	11134.99	5451.331	1348.59	15-12-2009	CBPH	joint	79	7	017	MW	R1	25-50	3-10M	0.1-1.0MM	ROUGH	HARD;<5MM	MODERATE	DRY	FAIR	47	42
M3_EAST_ctrl12_to_ctrl13	11.00	11.50	11134.99	5451.331	1348.59	15-12-2009	CBPH	shear	20	158	168	HW	R0	<25	>20M	>5MM	SMOOTH	SOFT;>5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl12_to_ctrl13	11.00	11.50	11134.99	5451.331	1348.59	15-12-2009	CBPH	thrust	63	60	070	HW	R0	<25	<1M	0.1-1.0MM	SMOOTH	SOFT;<5MM	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl12_to_ctrl13	13.50	15.00	11132.42	5449.958	1348.569	15-12-2009	CBPH	foliation	15	103	113	HW	R1	<25	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	35	30
M3_EAST_ctrl12_to_ctrl13	13.50	15.00	11132.42	5449.958	1348.569	15-12-2009	CBPH	joint	80	242	252	HW	R1	<25	3-10M	0.1-1.0MM	ROUGH	HARD;<5MM	HIGHLY	DRY	FAIR	38	33
M3_EAST_ctrl12_to_ctrl13	13.50	15.00	11132.42	5449.958	1348.569	15-12-2009	CBPH	vein	80	208	218	HW	R1	<25	3-10M	>5MM	ROUGH	SOFT;>5MM	HIGHLY	DRY	FAIR	42	37
M3_EAST_ctrl12_to_ctrl13	13.50	15.00	11132.42	5449.958	1348.569	15-12-2009	CBPH	joint	83	322	332	HW	R1	<25	3-10M	0.1-1.0MM	ROUGH	HARD;<5MM	HIGHLY	DRY	FAIR	38	33
M3_EAST_ctrl12_to_ctrl13	PG13	PG13	11129.84	5448.585	1348.549	15-12-2009	CBPH	foliation	6	192	202	HW	R1	25-50	3-10M	NONE	SMOOTH	NONE	HIGHLY	DRY	FAIR	40	35

Appendix G Stereographic Projection of Mapped Data