CASE STUDY: THE GEOTECHNICAL INVESTIGATION AND SUPPORT DESIGN OF A VERTICAL SHAFT WITH FOCUS ON ROCK MASS CLASSIFICATION

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

Johannesburg, 2015

CANDIDATE'S DECLARATION

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

Denisha Sewnun

6 July 2015

ABSTRACT

This research report focuses on the geotechnical analysis and support design for the construction of the Platreef vertical production shaft 2 for the proposed Platreef Mine. The geotechnical investigation includes the characterisation and analysis of the quality of the subsurface conditions observed in the shaft 2 borehole (GT017) to the final depth of drilling. The nature and quality of the sub-surface ground conditions is derived from data collected from the geotechnical borehole log as well as from laboratory testing results. The analysis of the quality of the subsurface conditions focuses on three rock mass classification systems, viz. Laubscher's (1990) Mining Rock Mass Rating Classification, Hoek's (2013) quantification of GSI and Barton et al's (1974) Norwegian Geotechnical Institute's Q-System. The suitability of sinking the vertical production shaft in the designated position is addressed, together with design recommendations and a risk assessment pertaining to the support of the shaft. As classic rock mass classification systems are utilised in this study, experience is added to the current knowledge base regarding the use of these systems, allowing a reference point for solutions to similar situations. Adjustments that may be made to these systems have also been demonstrated, providing further insight on the appropriate use of these approaches.

To my husband, Yashved Sewnun

For your everlasting love and support…

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1 INTRODUCTION

In 2013, Platreef Resources requested the geotechnical assessment of a vertical borehole for the design and construction of the Platreef vertical production shaft 2 for the Platreef project. The Platreef project is an underground mining project focused on the development of a viable underground mining operation to extract what is known as the Flatreef deposit. The Flatreef deposit is a shallow dipping tabular orebody (approximately 24m thick on average) which contains platinum group elements. This Platreef project is located in the Limpopo Province of South Africa and is currently in feasibility stage (Figure 1-1).

Figure 1-1: Platreef Project Location (Platreef PEA, 2013)

The objective of this research project is to undertake a geotechnical investigation for the design and construction of the vertical shaft to a depth of 1100 m, the depth to which the shaft will be sunk. The geotechnical investigation includes the characterisation and analysis of the quality of the rock mass conditions observed in the Shaft 2 borehole (GT017) to the final depth of drilling. The nature and quality of the sub-surface ground conditions is derived from data collected from the geotechnical borehole log as well as from laboratory testing results. The suitability of sinking the vertical production shaft in the designated position is addressed, together with design recommendations and a risk assessment pertaining to the rock support of the shaft.

With the aim of investigating and classifying the quality of the rock mass, use is made of three rock mass classification systems, viz. Laubscher's (1990) Mining Rock Mass Rating Classification, Barton et al's (1974) Norwegian Geotechnical Institute Q-System and Hoek's (2013) quantification of GSI. The use of the Q-System facilitates the classification of the rock mass in terms of quality and allows for the determination of design support recommendations for the shaft. Laubscher's (1990) rock mass rating values and Hoek's (2013) GSI values also provide a classification of the rock mass aimed to verify and validate the Q-values achieved.

This research topic was chosen because it allows the opportunity to provide Platreef Resources with in depth information and knowledge of their rock mass conditions in the vicinity of the Shaft 2 borehole. As Barton's Q, Laubscher's MRMR and Hoek's GSI approaches are utilised in the study, strength and experience is added to the current knowledge base regarding the use of these systems, thus allowing a reference point for solutions to similar situations. In addition to the rock mass classification systems used, typical adjustments that can be made to each system have also been demonstrated to provide further insight on the use of these approaches.

1.1 Research Objectives

Research objectives include the following:

- To characterise the quality of the in-situ ground conditions to the drilled depth of 1100.00 m.
- To provide geological data and information on the sub-surface geotechnical conditions over the full depth of the vertical shaft.
- To classify the rock along the length of the borehole in terms of Laubscher's (1990) Mine Rock Mass Rating (MRMR) system, Barton's (1974) Norwegian Q-System and Hoek's (2013) quantification of GSI.
- To identify adverse geotechnical sub-surface conditions that could affect the stability of the vertical shaft by conducting a structural analysis using the Rocscience software DIPS and UNWEDGE and undertaking a risk assessment using the software package JBLOCK.
- To provide support recommendations over the full depth of the vertical shaft based on a 10 m shaft diameter with a 300 mm concrete lining.
- To demonstrate the use and the adjustments that may be made to classic rock mass classification systems to provide insight on the application of these systems.

1.2 Structure of the Research Report

Chapter 1 of the research report provides an introduction to the research topic, which includes research objectives and why the research was carried out.

Chapter 2 details a literature review of classic rock mass classification systems, where each system and its requirements for application are described and discussed.

In Chapter 3 the geotechnical investigation for Shaft 2 is presented. Investigational and laboratory results are included here.

Chapter 4 presents the rock mass classification approaches utilised with respect to Barton's Q, Laubscher's MRMR and Hoek's quantification of the GSI system.

In chapter 5 a structural analysis to the proposed depth of the shaft is presented.

In chapter 6 support design recommendations for the shaft are presented based on the results of Barton's Q system

Chapter 7 includes a risk assessment using the software package JBLOCK.

In Chapter 8, conclusions and recommendations derived by the author are offered and discussed.

All research report references are listed as Section 9.

2 LITERATURE REVIEW

Having presented a brief introduction to the research report topic in chapter 1, a critical literature review of the philosophy, implementation and evolution of quantitative rock mass classification systems is now discussed.

“When the number of geological weakness planes becomes impractical to consider individually, the rock mass must be treated as a pseudo continuum” (Stacey, 2013). The most effective way of handling such a rock mass is to employ rock mass classification. Rock mass classification methods are commonly used today as a useful means of investigating rock mass stability, support requirements in underground openings, rock mass deformability and rock mass strength. Rock mass classification systems thus form a critical component for empirical design and are widely used today (Bieniawski, 1989). Rock mass classification systems have proved their use as a powerful tool in rock engineering applications and have been highly beneficial since their introduction into the industry (Stacey 2001).

While rock mass classification systems can be utilised with apparent ease and simplicity, it must be kept in mind that inaccurate results may be produced if these systems are not utilised with caution. Experience and sound engineering judgement are thus critical to the success in the use of these systems. Nine classifications are described below which illustrate the evolution of rock mass classification systems over time.

2.1 Terzaghi’s Rock Load Classification

In 1946 Terzaghi published what is considered the earliest reference on the use of a rock mass classification approach for the evaluation of rock support in tunnels. This system was based on the recommendation of steel arch support systems in tunnels (Terzaghi, 1946). While the approach was widely utilised in the USA for over 25 years, it is not suitable to modern tunnels that apply shotcrete and install rock bolts as a form of support (Bieniawski, 1973). Furthermore, the method is considered qualitative or semi quantitative as the interpretation of the rock condition is based on the opinion of the user. For these reasons this system is rarely used today.

2.2 Lauffer’s Stand Up Time Classification System

Lauffer (1958) proposed that the stand-up time for an unsupported span relates to the condition of the rock mass in which the span is excavated. In tunnelling operations, the unsupported span is defined as “the span of the tunnel or the distance between the face and the nearest support, if this is greater than the tunnel span” (Hoek, 1995).

What is important to consider in the stand-up time concept is that the increase in span of the tunnel leads to a reduction in the time for support installation. For example, “a pilot tunnel in a fair rock mass may be successfully constructed with minimal support, while a larger span tunnel in the same rock mass may not be stable without the immediate installation of substantial support” (Hoek, 1995).

The Lauffer classification was a notable step forward in tunnelling. This is since this system introduced the concept of an active unsupported rock span as well as corresponding stand up time, which are significant parameters for the determination of the type and amount of support in tunnels. The disadvantage of the classification system is that the two parameters are difficult to establish and thus much is demanded of practical experience (Bieniawski, 1973).

Over the years Lauffer's classification has been modified by various authors, notably Pacher et al (1974), and now forms part of a general tunnelling approach known as the New Austrian Tunnelling Method (Golser, 1976).

"The New Austrian Tunnelling Method includes a number of techniques for safe tunnelling in rock conditions in which the stand-up time is limited before failure occurs" (Golser, 1976). These techniques consider the use of smaller headings and benching or multiple drifts resulting in a reinforced ring inside which the bulk of the tunnel may be excavated. These techniques may be used in both soft rock and in intensely jointed rock. Caution should be employed when applying these techniques in hard rock where different failure mechanisms occur (Hoek, 1995).

2.3 Rock Quality Designation

Rock Quality Designation (RQD) was developed as an index of rock quality initially utilised in a design and construction project in 1964. Following this project, research continued over several years at the University of Illinois. The RQD concept was later brought to the attention of the engineering and geology profession with the paper by Deere and colleagues (Deere et al, 1967). In 1968 a chapter by Deere was introduced to an international audience leading to the acceptance of RQD and its use in many countries (Deere and Deere, 1988).

2.3.1 The Developmental Period

In 1964 at the Nevada test site for underground nuclear testing, it became evident that the area comprised granite of a poorer quality compared to that of an alternate site (Deere and Deere, 1989). Although this was the case, detailed logs by qualified personnel did not readily reveal the difference in quality of these granites. Visual indictors in the core that illustrated poor rock conditions included numerous small core pieces depicting weathered joints and sheared surfaces, rock fragments and core pieces of evidently altered rock. By contrast, the alternate site portrayed core that was of a hard, nearly unweathered nature, with core pieces of a much greater length (Deere and Deere, 1989).

In order to illustrate the poorer quality rock at the Nevada site, it was decided to opt for a "modified core recovery" approach in which only intact pieces of core of 10 cm or greater in length were counted. In this way, the quality of rock was downgraded by excluding core pieces not of the requisite length as well as unrecovered pieces (Deere and Deere, 1989). Following this idea, large scale boring logs were presented to management where the "modified core recovery" was plotted with depth with the following intervals:

- Values greater than 95 percent (later changed to 90 percent) was coloured blue and deemed as excellent quality rock.
- The 95 to 75 percent interval was coloured green and designated as good quality.
- The 75 to 50 percent interval was coloured orange and designated as fair quality.
- The 50 to 25 percent and 25 to 0 percent intervals were coloured red and deemed poor to very poor quality rock, respectively (Deere and Deere, 1989).

The name Rock Quality Designation was applied to the overall procedure (Deere and Deere, 1989).

An example of the correct way in which to measure RQD is presented in Figure 2-1.

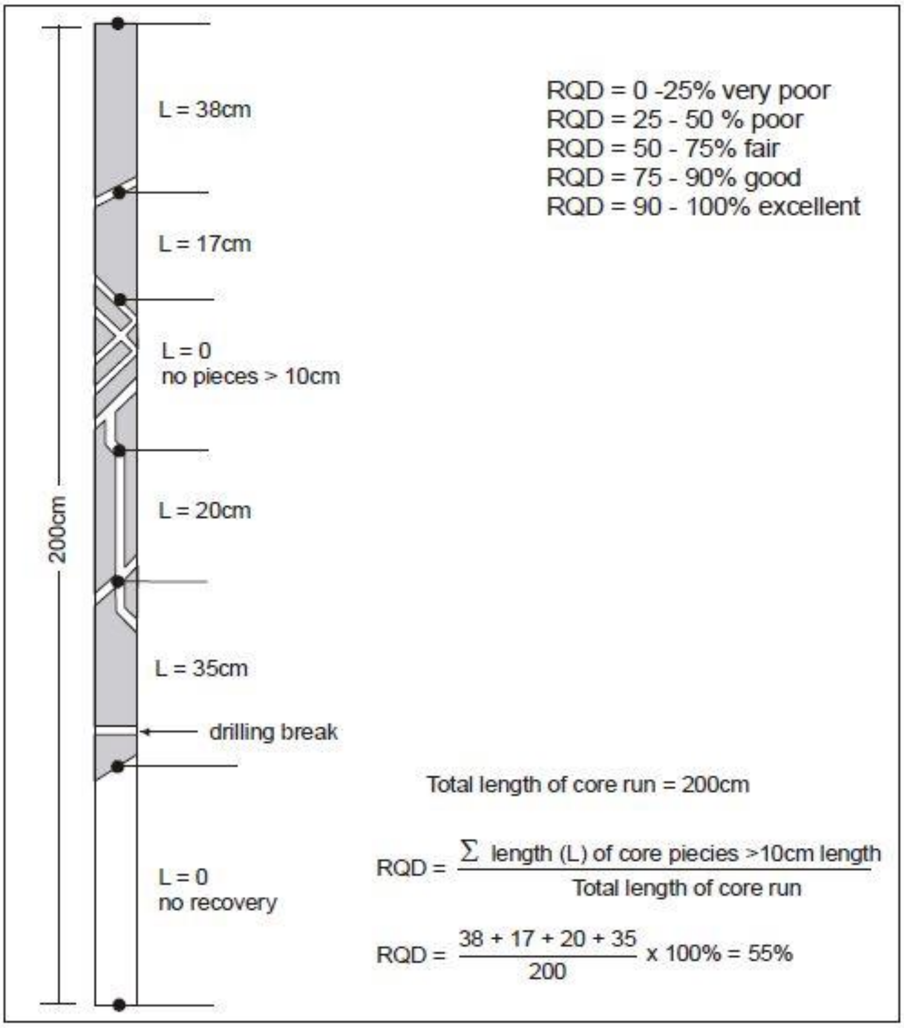


Figure 2-1: RQD determination (after Deere, 1989)

2.3.2 Recommended Techniques for RQD Logging

While the methodology to determine RQD is very simple and straight forward, this approach comes with various recommended techniques (Deere and Deere, 1989). It is important for users of the RQD system to keep these techniques in mind when applying this method. As with many classification approaches, the equation for determination of the final output is well recognised, however, recommended techniques and additional notes for application are sometimes forgotten or unheard of. This is possibly due to more recent publications of RQD not reflecting these finer details explicitly.

In order to successfully use the RQD system, the following practices should be applied:

- Core diameters for RQD logging should be of NQ (45mm) or HQ (63.5mm) size. Where weak or foliated rocks exist, larger size core should be utilised. Generally, the smaller BQ and BWX sizes should not be considered, and if used, should be identified with a disclaimer.
- Length measurements of the core should be along the centreline (Figure 2-2). Where mechanical breaks of the core occur due to handling or otherwise, the core must be fitted together as counted as a continuous piece.
- Ideally the length of each core run should be 1.5 m in length; however where the core depicts a good quality a 3 m run may be used. Where core is laminated, soft, comprise unfavourable joint or bedding orientations, is highly jointed etc., shorter run lengths should be considered (0.75 m to 1.5 m).

- RQD intervals should be subdivided within a core run when zones of clearly different rock quality are recognised.
- Fresh and slightly weathered rock should be used in the RQD count, while an asterisk should be placed next to the RQD value where moderately weathered rock exists. Highly weathered rock, completely weathered rock and residual soil should not be included.
- Drilling supervision and prompt logging in the field by a qualified geologist or geotechnical engineer is recommended (Deere and Deere, 1989).

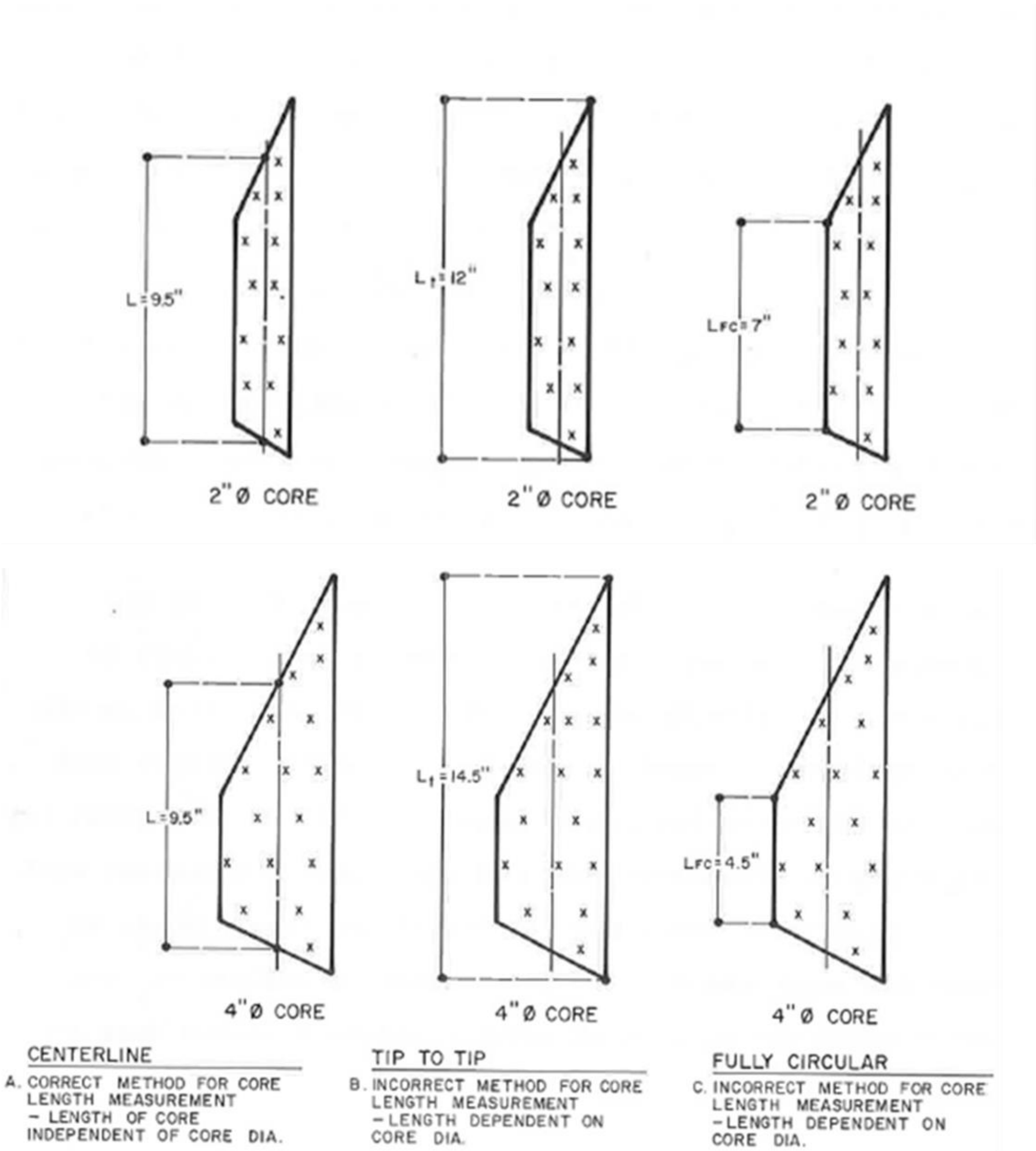


Figure 2-2: RQD measurement (Deere and Deere, 1989)

2.3.3 Limitations of Deere's RQD Approach

RQD is accompanied by various limitations. Some of these limitations are outlined below:

- As with all one dimensional measurements, RQD is directional (Palmstrom, 2005). RQD is therefore sensitive to the orientation of the borehole. This is clearly demonstrated in Figure 4-1. Figure 4-2 shows three extreme cases, where RQD is either equal to 0 or 100 depending on the orientation of the borehole, even though the RQD method is being applied to the exact same joint sets with the exact same spacing.

- Inaccurate results may be produced if RQD is determined weeks after the core has been drilled. This is because the core can undergo transportation damages, drying, disintegration, stress relief, etc. in this time.
- Inaccurate results may occur if the core is not drilled to the correct diameter or is not logged along the centre line by a competent geotechnical engineer or engineering geologist.
- RQD cannot be used in isolation to establish the quality of the rock mass. It should thus be applied within any relevant rock mass classification system to provide a more accurate account on the rock quality in a given area.
- Overall it is evident that RQD is a widely accepted system that is used extensively today. RQD allows greatly for the "red flagging" of zones with poor quality rock. This is likely the most attractive quality of this approach. While the RQD system should be not be used in isolation for rock mass classification, RQD does provide as an excellent input parameter for other popular rock mass classification systems (Eg. Bieniawski's 1973 Rock Mass Rating system and Barton's 1974 Q system).

2.4 Rock Structure Rating

The Rock Structure Rating (RSR) classification system is a quantitative classification system developed by Wickam et al (1972) that quantifies the quality of a rock mass. Furthermore, it may be used to select the appropriate support based on the RSR achieved.

"It was found that most case histories used in the development of this system were for relatively small tunnels supported by means of steel sets, although historically this system was the first to make reference to shotcrete support" (Hoek, 1995). A further significance of the RSR system is that it introduced the notion of using rating components to attain at a numerical value of RSR.

The Rock Structure Rating (RSR) is defined by the equation:

$$\boldsymbol{RSR = A + B + C}$$

Where: A = the geology parameter,

B = the geometry parameter, and

C = the effect of groundwater inflow and joint condition

The geology parameter (A) accounts for geological features such as:

- Rock origin,
- Rock hardness, and
- Rock fabric.

The geometry parameter (B) accounts for the effect of the discontinuity pattern based on the direction of a tunnel, considering:

- Joint spacing,
- Joints orientation, and
- Direction of tunnel advance.

The effect of groundwater seepage and joint condition (parameter C) is taken into account based on:

- Rock mass quality as derived from the combination of parameters A and B,
- The joint condition, and
- The amount of water inflow into a tunnel.

The parameter rating values are determined using tables developed by Wickham et al. (1972), to arrive at a calculated RSR value out of a maximum of 100. The tables used to evaluate the parameters are presented from Table 2-1 to Table 2-3.

Table 2-1: RSR Parameter A (after Wickam et al, 1972)

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

Table 2-2: RSR Parameter B (after Wickam et al, 1972)

	Strike perpendicular to Axis					Strike parallel to Axis		
	Direction to Drive					Direction of Drive		
	Both	With Dip		Against Dip		Either Direction		
	Dip of Prominent Joints					Dip of Prominent Joints		
Average Joint Spacing	Flat	Dipping	Vertical	Dipping	Vertical	Flat	Dipping	Vertical
Very Closely Jointed < 2 in.	9	11	13	10	12	9	9	7
Closely Jointed 2–6 in.	13	16	19	15	17	14	14	11
Moderately Jointed 6–12 in.	23	24	28	19	22	23	23	19
Moderate to Blocky 1–2 ft	30	32	36	25	28	30	28	24
Blocky to Massive 2–4 ft	36	38	40	33	35	36	34	28
Massive > 4 ft	40	43	45	37	40	40	38	34

Table 2-3: RSR Parameter C (after Wickam et al, 1972)

	Sum of Parameters A+B					
	13 - 44			45 - 75		
	Joint Condition					
	Good	Fair	Poor	Good	Fair	Poor
None	22	18	12	25	22	18
Slight <200 gpm	19	15	9	23	19	14
Moderate 200-1000 gpm	15	11	7	21	16	12
Heavy >1000 gpm	10	8	6	18	14	10

2.5 Bieniawski's Rock Mass Rating System

In 1973, Bieniawski presented the Geomechanics classification system, which is better known as the Rock Mass Rating (RMR) classification system in the attempt to meet both practical applications and effective communication between the engineer and the geologist or the designer and the contractor. According to Bieniawski, a rock mass classifcation system should:

- Group a rock mass into classes of similar behaviour.
- Provide a foundation for understanding the rock mass characteristics.

- Assist in the planning and design of structures in rock by acquisition of the quantitative data required for the solution of real engineering problems.
- Provide a common understanding to enhance communication among all persons concerned with a geomechanics problem (Bieniawski, 1973).

It was further suggested that these aims should be fulfilled by ensuring that the adapted classification system is simple and meaningful and is based on measurable parameters which can be determined both quickly and cheaply in the field (Bieniawski, 1973).

2.5.1 Bieniawski 1973

The 1973 Geomechanics classification system includes the following parameters:

- RQD
- Weathering
- Unixial compressive strength of intact rock
- Joint spacing
- Joint orientation
- Joint separation
- Joints continuity, and
- Groundwater

Each parameter is discussed briefly below.

RQD

RQD was chosen in this system as it was considered particularly useful for classifying rock masses for the selection of tunnel support systems.

Weathering and Alteration

Weathering and Alteration are significant factors in the behaviour of a rock mass. For this purpose the following weathering classification should be employed (Bieniawski, 1973):

Unweathered:	No evidence of weathering. Rock is fresh, and bright crystals are evident. There are only a few discontinuities which may display light staining
Slightly weathered:	Weathering is developed on open discontinuity surfaces and extends up to 10mm into the rock mass. Discontinuity surfaces are discoloured.
Moderately weathered:	Discolouration penetrates through the larger part of the rock mass. The rock material is not friable (except in the case of poorly cemented sedimentary rocks). Discontinuities are stained and/or contain a filling comprising altered material.
Highly weathered:	Weathering encompasses the rock mass. Rock material is partly friable, has no lustre and is discoloured with the exception of quartz.
Completely weathered:	Rock is completely discoloured, decomposed and in a friable condition. Only fragments of the rock structure is preserved (Bieniawski, 1973).

Uniaxial Compressive Strength

The subject of strength classification is a fairly controversial topic with varying opinions. Bieniawski (1973) is of the opinion that the strength classification as proposed by Deere (1966) is both convenient and realistic for use in the field of rock mechanics. Table 2-4 thus presents a modified version of Deere's classification, adapted for the purpose of conforming to the SI metric system of units. This classification is considered realistic and practical in subdivisions, and is widely recognised throughout the world.

Table 2-4: Strength Classification of Intact Rock (Bieniawski, 1973)

Description	Uniaxial Compressive Strength (MPa)	Rock Types
Very Low Strength	1 - 25	Chalk, Rocksalt
Low Strength	25 - 50	Coal, Siltstone, Schist
Medium Strength	50 - 100	Sandstone, Slate, Shale
High Strength	100 - 200	Marble, Granite, Gneiss
Very High Strength	> 200	Quartzite, Dolerite, Gabbro, Basalt

Criticism has been directed toward the strength classification of the intact rock table which highlights that no subdivisions are given below 25 MPa. However, "strength values of less than 25 MPa do not contribute to the overall mobility of the rock mass, and thus by not introducing a subdivision below 25 MPa, a conservative approach is purposefully adopted" (Bieniawski, 1973).

The strength of intact rock may be determined either by conducting Uniaxial Compressive Strength tests or Point Load tests. Note that the strength of a rock material should not be confused with hardness. Strength should therefore not be measured using a pen knife or geological hammer. In rock mechanics, hardness may be measured using a Schmidt hammer, the product of which may be multiplied by the unit weight of the rock, resulting in a correlation to uniaxial compressive strength (Bieniawski, 1973). This relationship is not convenient to use and is often avoided.

Spacing of Joints

Joint spacing is of great significance in the evaluation of a rock mass. This is since the presence of joints affects rock mass strength, whereby the smaller the spacing, the lower the rock mass strength, and vice versa. In this regard the classification by Deere (1968) is the most widely used and is thus recommended (Table 2-5).

Table 2-5: Classification for Joint Spacing (Bieniawski, 1973)

Description	Joint Spacing	Rock Mass Grading
Very Wide	> 3 m	Solid
Wide	1 – 3 m	Massive
Moderately Close	0.3 – 1 m	Blocky/Seamy
Close	50 – 300 m	Fractured
Very Close	< 50 mm	Crushed and Shattered

Strike and Dip Orientations

The combined effect of strike and dip orientations play an important role in slope stability and in tunnels. Complex classification systems that take strike and dip experience into account may be developed, however practical experience indicates that a qualitative assessment is sufficient for most practical situations (Bieniawski, 1973). This simply involves assessing if the dip and strike conditions of a rock mass are favourable or unfavourable for the proposed excavation.

Joint Separation

Joint separation is often not considered in rock mass classification systems, however this parameter is found to be of importance to the quantitative assessment of a rock mass and is therefore included in Bieniawski's (1973) rock mass classification system.

Joint Continuity

The continuity of joints is significant since the greater the extent of the joint, the lower the strength of the rock mass. Joints may or may not contain gouge infill. If gouge is present within joints, the type and thickness of the gouge should be recorded.

Groundwater

Groundwater is known to have a significant impact on jointed rock masses (Bieniawski, 1973). Where groundwater exists, the strength of the rock mass is reduced. Rock mass classification systems should therefore account for the influence of ground water.

It is evident that all parameters described above contribute to the overall quality of a rock mass (Table 2-6). However, as all parameters do not contribute equally in this regard, Bieniawski assigned a rating to each parameter using a weighted numerical value (Table 2-7). The final rock mass rating value is then determined by the sum of the weighted values determined for each parameter.

Table 2-6: Geomechanics classification of jointed rock masses (Bieniawski, 1973)

Item	Class and description	1	2	3	4	5
		Very good	Good	Fair	Poor	Very Poor
1	RQD (%)	90 - 100	75 - 90	50 - 75	25 - 50	< 25
2	Weathering	Unweathered	Slightly Weathered	Moderately Weathered	Highly Weathered	Completely Weathered
3	Intact Rock Strength (MPa)	> 200	100 - 200	50 - 100	25 - 50	< 25
4	Joint Spacing	> 3 m	1 - 3 m	0.3 – 1 m	50 – 300 m	< 50 mm
5	Joint Separation	< 0.1 mm	< 0.1 mm	0.1 – 1 mm	1 – 5 mm	> 5 mm
6	Joint Continuity	Not continuous	Not continuous	Continuous without gouge	Continuous with gouge	Continuous with gouge
7	Groundwater inflow (per 10 m adit)	None	None	Slight < 25 litres/min	Moderate 25 – 50 litres/min	Heavy > 125 litres/min
8	Strike and Dip Orientation	Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
9	Stand-up Time (Tunnels)	10 years	6 months	1 week	5 hours	10 min
10	Active Unsupported span (Tunnels)	5 m	4 m	3 m	1.5 m	0.5 m

Table 2-7: Individual Ratings for classification Parameters (after Bieniawski, 1973)

Item	Parameter		Class				
			1	2	3	4	5
1	RQD		16	14	12	7	3
2	Weathering		9	7	5	3	1
3	Intact Rock Strength		10	5	2	1	0
4	Joint Spacing		30	25	20	10	5
5	Joint Separation		5	5	4	3	1
6	Joint Continuity		5	5	3	0	0
7	Ground water		10	10	8	5	2
8	Dip and Strike Orientations	Tunnels	15	13	10	5	3
		Foundations	15	13	10	0	- 10
Total Rating			100 - 90	90 - 70	70 - 50	50 - 25	< 25
Class of Rock			Very Good	Good	Fair	Poor	Very Poor

2.5.2 Modifications to the RMR System

Following the development of the rock mass rating system, Bieniawski further refined this classification approach over the years as more case histories became available (Bieniawski, 1979). The major modifications made to the system over the years are as follows:

Bieniawski 1974:

- A parameter for joint condition was implemented;
- A strike and dip parameter was added and the strike and dip parameter for tunnels was removed;
- The weathering, joint separation and joint continuity parameters were removed; and
- The weighting for the RQD parameter was increased from 16 to 20 (Bieniawski, 1974).

Bieniawski 1975:

- The initial joint condition parameter rating was increased from 15 to 30.
- The rock strength parameter rating was increased from 10 to 15.
- The strike and dip parameter ratings were modified from a range of 3 to 15 to a range of 0 to 12 (Bieniawski, 1975).

Bieniawski 1976:

- The joint condition parameter rating was altered to 25.
- The concept of rock mass classes was introduced, where each class was sub-divided into intervals of 20 (Bieniawski, 1976).

Bieniawski 1979:

In 1979 Bieniawki highlighted that the RMR was established as a useful tool for assessing rock mass conditions in the field on engineering projects, and emphasised that greater effort should be made to use the system for mining applications (Bieniawski, 1979).

Bieniawski 1989:

- The joint discontinuity spacing parameter was modified to give a maximum rating of 20.
- The ground water parameter rating was increased to 15.

- The joint condition parameter was increased back to 30 from 25.
- The conditions of discontinuities were further quantified.
- The assessment of sub-horizontal joints was revised from “unfavourable” to “fair” (Bieniawski, 1989).

The 1989 RMR system is presented in Table 2-8 and Table 2-9.

Table 2-8: The 1989 RMR Classification System (Bieniawski, 1989)

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS									
Parameter			Range of Values						
1	Strength of intact Rock Material	Point Load Strength Index	>10 MPa	4-10 MPa	2-4 MPa	1-2MPa	For this low range- uniaxial compressive test is preferred		
		Uniaxial Compressive Strength	>250 MPa	100-250 MPa	50-100 MPa	25-50 MPa	5-25 MPa	1-5 MPa	<1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core Quality RQD		90%-100%	75%-90%	50%-75%	25%-50%	<25%		
	Rating		20	17	13	8	3		
3	Spacing of Discontinuities		>2m	0.6m-2m	200-600mm	60-200mm	<60mm		
	Rating		20	15	10	8	5		
4	Condition of Discontinuities (See E)		Very rough surfaces; Not continuous; No separation; Unweathered wall rock	Slightly rough surfaces; Separation <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 / 10m tunnel length (l/m)	None	<10	10-25	25-125	>125		
		(Joint water press.) / (Major principal stress)	0	<0.1	0.1-0.2	0.2-0.5	>0.5		
		General Conditions	Completely Dry	Damp	Wet	Dripping	Flowing		
	Ratings		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)						
Strike And Dip Orientations		Very Favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels and Mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-

Table 2-9: 1989 RMR Classification System Cont. (Bieniawski, 1989)

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS					
Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	<21
Class Number	I	II	III	IV	V
Description	Very Good Rock	Good Rock	Fair Rock	Poor Rock	Very Poor Rock
D. MEANING OF ROCK CLASSES					
Class Number	I	II	III	IV	V
Average Stand-Up Time	20yrs for 15m span	1yr for 10m span	1 week for 5m span	10hrs for 2.5m span	30min for 1m span
Cohesion of Rock Mass (kPa)	>400	300-400	200-300	100-200	<100
Friction Angle of Rock Mass (°)	>45	35-45	25-35	15-25	<15
E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY CONDITIONS					
Discontinuity Length (Persistence)	<1m	1-3m	3-10m	10-20m	>20m
Rating	6	4	2	1	0
Separation (Aperture)	None	<0.1mm	0.1-1.0mm	1-5mm	>5mm
Rating	6	5	4	1	0
Roughness	Very Rough	Rough	Slightly Rough	Smooth	Slickensided
Rating	6	5	3	1	O
Infilling (Gouge)	None	Hard Filing <5mm	Hard Filling >5mm	Soft Filling <5mm	Soft Filling >5mm
Rating	6	4	2	2	0
Weathering	Unweathered	Slightly Weathered	Moderately Weathered	Highly Weathered	Decomposed
Rating	6	5	3	1	0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELING			
Strike Perpendicular to Tunnel Axis		Strike Parallel to Tunnel Axis	
Drive with Dip: Dip 45°- 90°	Drive with Dip: Dip 20°- 45°	Dip 45°- 90°	Dip 20°- 45°
Very Favourable	Favourable	Very Unfavourable	Fair
Drive against Dip: Dip 45°- 90°	Drive against Dip: Dip 20°- 45	Dip 0°- 20°: Irrespective of Strike	
Fair	Unfavourable	Fair	

2.5.3 Strengths and Limitations of the System

The strengths and limitations of the RMR system are outlined as follows:

- The RMR system is straight forward and easy to use.
- The method may be used in several applications such as coal mining, hard rock mining, slope stability, foundation stability and tunnelling.
- A shortcoming of the system is that the output is rather conservative, which can lead to over design in terms of support recommendations. This may be overcome by monitoring rock mass behaviour during construction and fine-tuning rock mass classification outputs to align with the local conditions (Bieniawski, 1989).
- The RMR system only includes stresses up to 25MPa. The system thus does not include stress problems in tunnelling (Palmstrom, 2009).

2.6 The Norwegian Geotechnical Institute's Q-System

Use will be made of the Norwegian Geotechnical Institute's Q system to facilitate the derivation of Q values for each geotechnical interval of the Platreef Resources borehole GT017. The Q-system was originally based on the study of roughly 200 tunnel cases which shows that there is a meaningful relationship between the amount and type of permanent support and rock mass quality, Q, with regard to tunnel stability (Barton et al, 1974). The original Q-system database is presented in Table 2-10 below.

Table 2-10: Summary of original Q-System Database (after Hutchinson and Diederichs, 1996)

Excavation Type	No. of Case Histories
Temporary mine openings	2
Permanent mine openings, low pressure water tunnels, pilot tunnels, drifts and headings for large openings	83
Storage caverns, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels	25
Power stations, major road and railway tunnels, civil defence chamber portals, intersections	79
Underground nuclear power stations, railway stations, sports and public facilities, factories	2

Using six classification parameters, Q values are attained by substitution of parameter rating values into the following equation:

$$Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF}$$

Where: *RQD* is the rock quality designation

Jn is the joint set number

Jr is the joint roughness number

Ja is the joint alteration number

Jw is the joint water reduction factor

SRF is the stress reduction factor

Each quotient in the Q equation is based on a specific aspect. These can be described as follows:

RQD/Jn: Refers to the overall structure of the rock mass, and is considered as a crude measure of block size

Jr/Ja: Refers to the joint condition and thus represents an approximation of shear strength

Jw/SRF: Jw is a measure of water pressure which has an adverse effect on the shear strength of joints. “SRF is a measure of loosening load in the case of excavation through shear zones and clay bearing rock, rock stress in sound rock and squeezing loads in plastic unstable rock” (Barton et al, 1974). The quotient Jw/SRF is thus a complex factor which relates to the “active stresses in the rock” (Barton et al, 1974).

“The rock mass descriptions and ratings for each of the six parameters result in Q values that range from 0.001 to 1000, which encompass rock mass qualities from heavy squeezing rock to unjointed rock” (Barton et al, 1974).

Table 2-11: Q-System Classification

Q Index Value	Rock Mass Class
0.0001-0.01	Exceptionally Poor
0.01-0.1	Extremely Poor
0.1-1	Very Poor
1-4	Poor
4-10	Fair
10-40	Good
40-100	Very Good
100-400	Extremely Good
400-1000	Exceptionally Good

It must be kept in mind that while the Q system is well suited to determine rock mass quality, it does not take into account rock mass strength explicitly, nor does it include joint orientation information. To account for this, it is considered implicitly in the derivation of a stress reduction factor (SRF):

$$\boldsymbol{SRF = UCS/\sigma 1}$$

Where: *UCS* is the unconfined Compressive Strength

$\sigma 1$ is the major principal stress

2.6.1 Development of the Q-System

From the development of the Q-System adjustments and improvements have been published and the original Q-System is now based on more than 1000 case histories (Table 2-12). In 2002, Barton published a technical paper entitled “Some New Q-Value Correlations to Assist in Site Characterisation and Tunnel Design”, which introduced a number of changes to the respective Q-System parameters. The amended Q-value parameters are presented from Table 2-13 to Table 2-16.

Table 2-12: Developments to the Q-System (Palmstrom and Broch, 2006)

Year	Development	Author(s) and title of paper
1974	The Q-system is introduced	Barton*, Lien*, and Lunde*: Engineering classification of rock masses for the design of tunnel support.
1977	Estimate of rock support in tunnel walls Estimate of temporary support	Barton*, Lien*, and Lunde*: Estimation of support requirements for underground excavation.
1980	Q system for estimate of input parameters to the Hoek-Brown failure criterion for rock masses	Hoek and Brown: Underground excavations in rock.
1988	New, simplified rock support chart	Grimstad* and Barton*: Design and methods of rock support.
1990	Rock support of small weakness zones	Löset*: Using the Q-system for support estimates of small weakness zones and for temporary support (in Norwegian)
1991	Estimate of Q values from refraction seismic velocities	Barton*: Geotechnical design.
1992	The application of the Q-system in the NMT ("Norwegian method of tunnelling")	Barton* et al.: Norwegian method of tunnelling.
1992	Estimate of squeezing using Q values	Bhawani Singh et al.: Correlation between observed support pressure and rock mass quality.
1993	Updating the Q-system with: – adjustment of the SRF values – application of new rock support methods – Q estimated from seismic velocities – estimate of deformation modulus for rock masses – adjustment for narrow weakness zones	Grimstad* and Barton*: Updating of the Q-system for NMT.
1995	Introduction of Qc with application of compressive strength	Barton*: The influence of joint properties in modelling jointed rock masses.
1997	Q-system applied during excavation	Löset*: Practical application of the Q-system
1999	Q_{TBM} is introduced	Barton*: TBM performance estimation in rock using Q_{TBM}.
2001	The Q-system is applied for estimating the effect of grouting	Barton* et al.: Strengthening the case for grouting.
2002	Further development of the Q-system	Barton*: Some new Q-value correlations to assist in site characterization and tunnel design.
* = NGI people		

Table 2-13: Joint Set Number (Barton, 2002)

Joint set number		J_n
A	Massive, no or few joints	0.5–1
B	One joint set	2
C	One joint set plus random joints	3
D	Two joint sets	4
E	Two joint sets plus random joints	6
F	Three joint sets	9
G	Three joint sets plus random joints	12
H	Four or more joint sets, random, heavily jointed, 'sugar-cube', etc.	15
J	Crushed rock, earthlike	20

Notes: (i) For tunnel intersections, use $(3.0 \times J_n)$. (ii) For portals use $(2.0 \times J_n)$.

Table 2-14: Joint Roughness Number (Barton, 2002)

Joint roughness number		J_r
(a) *Rock-wall contact, and (b) rock-wall contact before 10 cm sear*		
A	Discontinuous joints	4
B	Rough or irregular, undulating	3
C	Smooth, undulating	2
D	Slickensided, undulating	1.5
E	Rough or irregular, planar	1.5
F	Smooth, planar	1.0
G	Slickensided, planar	0.5
(b) *No rock-wall contact when sheared*		
H	Zone containing clay minerals thick enough to prevent rock-wall contact.	1.0
J	Sandy, gravely or crushed zone thick enough to prevent rock-wall contact	1.0

Notes: (i) Descriptions refer to small-scale features and intermediate scale features, in that order. (ii) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m. (iii) $J_r = 0.5$ can be used for planar, slickensided joints having lineations, provided the lineations are oriented for minimum strength. (iv) J_r and J_a classification is applied to the joint set or discontinuity that is least favourable for stability both from the point of view of orientation and shear resistance, τ(where $\tau \approx \sigma_n \tan^{-1} (J_r/J_a)$.

Table 2-15: Joint Alteration Number (Barton, 2002)

Joint alteration number		ϕ_r approx. (deg)	J_a
(a) *Rock-wall contact (no mineral fillings, only coatings)*			
A	Tightly healed, hard, non-softening, impermeable filling, i.e., quartz or epidote	—	0.75
B	Unaltered joint walls, surface staining only	25–35	1.0
C	Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	25–30	2.0
D	Silty- or sandy-clay coatings, small clay fraction (non-softening)	20–25	3.0
E	Softening or low friction clay mineral coatings, i.e., kaolinite or mica. Also chlorite, talc, gypsum, graphite, etc., and small quantities of swelling clays	8–16	4.0
(b) *Rock-wall contact before 10 cm shear (thin mineral fillings)*			
F	Sandy particles, clay-free disintegrated rock, etc.	25–30	4.0
G	Strongly over-consolidated non-softening clay mineral fillings (continuous, but <5 mm thickness)	16–24	6.0
H	Medium or low over-consolidation, softening, clay mineral fillings (continuous, but <5 mm thickness)	12–16	8.0
J	Swelling-clay fillings, i.e., montmorillonite (continuous, but <5 mm thickness). Value of J_a depends on per cent of swelling clay-size particles, and access to water, etc.	6–12	8–12
(c) *No rock-wall contact when sheared (thick mineral fillings)*			
KLM	Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)	6–24	6, 8, or 8–12
N	Zones or bands of silty- or sandy-clay, small clay fraction (non-softening)	—	5.0
OPR	Thick, continuous zones or bands of clay (see G, H, J for description of clay condition)	6–24	10, 13, or 13–20

Table 2-16: Stress Reduction Factor (Barton, 2002)

	Stress reduction factor	SRF		
(a)	*Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated*			
A	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10		
B	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation $\leqslant 50$ m)	5		
C	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation > 50 m)	2.5		
D	Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)	7.5		
E	Single shear zones in competent rock (clay-free), (depth of excavation $\leqslant 50$ m)	5.0		
F	Single shear zones in competent rock (clay-free), (depth of excavation > 50 m)	2.5		
G	Loose, open joints, heavily jointed or 'sugar cube', etc. (any depth)	5.0		
		σ_c/σ_1	σ_θ/σ_c	SRF
(b)	*Competent rock, rock stress problems*			
H	Low stress, near surface, open joints	> 200	< 0.01	2.5
J	Medium stress, favourable stress condition	200–10	0.01–0.3	1
K	High stress, very tight structure. Usually favourable to stability, may be unfavourable for wall stability	10–5	0.3–0.4	0.5–2
L	Moderate slabbing after > 1 h in massive rock	5–3	0.5–0.65	5–50
M	Slabbing and rock burst after a few minutes in massive rock	3–2	0.65–1	50–200
N	Heavy rock burst (strain-burst) and immediate dynamic deformations in massive rock	< 2	> 1	200–400
		σ_θ/σ_c	SRF	
(c)	*Squeezing rock: plastic flow of incompetent rock under the influence of high rock pressure*			
O	Mild squeezing rock pressure	1–5	5–10	
P	Heavy squeezing rock pressure	> 5	10–20	
		SRF		
(d)	*Swelling rock: chemical swelling activity depending on presence of water*			
R	Mild swelling rock pressure	5–10		
S	Heavy swelling rock pressure	10–15		

Notes: (i) Reduce these values of SRF by 25–50% if the relevant shear zones only influence but do not intersect the excavation. This will also be relevant for characterisation. (ii) For strongly anisotropic virgin stress field (if measured): When $5 \leqslant \sigma_1/\sigma_3 \leqslant 10$, reduce σ_c to $0.75\sigma_c$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c to $0.5\sigma_c$, where σ_c is the unconfined compression strength, σ_1 and σ_3 are the major and minor principal stresses, and σ_θ the maximum tangential stress (estimated from elastic theory). (iii) Few case records available where depth of crown below surface is less than span width, suggest an SRF increase from 2.5 to 5 for such cases (see H). (iv) Cases L, M, and N are usually most relevant for support design of deep tunnel excavations in hard massive rock masses, with RQD/J_n ratios from about 50–200. (v) For general characterisation of rock masses distant from excavation influences, the use of SRF = 5, 2.5, 1.0, and 0.5 is recommended as depth increases from say 0–5, 5–25, 25–250 to > 250 m. This will help to adjust Q for some of the effective stress effects, in combination with appropriate characterisation values of J_w. Correlations with depth-dependent static deformation modulus and seismic velocity will then follow the practice used when these were developed. (vi) Cases of squeezing rock may occur for depth $H > 350Q^{1/3}$ according to Singh [34]. Rock mass compression strength can be estimated from $\mathrm{SIGMA_{cm}} \approx 5\gamma Q_c^{1/3}$ (MPa) where γ is the rock density in t/m^3, and $Q_c = Q \times \sigma_c/100$, Barton [29].

Table 2-17: Joint Water Reduction Factor (Barton, 2002)

	Joint water reduction factor	Approx. water pres. (kg/cm^2)	J_w
A	Dry excavations or minor inflow, i.e., < 5 l/min locally	< 1	1.0
B	Medium inflow or pressure, occasional outwash of joint fillings	1–2.5	0.66
C	Large inflow or high pressure in competent rock with unfilled joints	2.5–10	0.5
D	Large inflow or high pressure, considerable outwash of joint fillings	2.5–10	0.33
E	Exceptionally high inflow or water pressure at blasting, decaying with time	> 10	0.2–0.1
F	Exceptionally high inflow or water pressure continuing without noticeable decay	> 10	0.1–0.05

Notes: (i) Factors C to F are crude estimates. Increase J_w if drainage measures are installed. (ii) Special problems caused by ice formation are not considered. (iii) For general characterisation of rock masses distant from excavation influences, the use of J_w = 1.0, 0.66, 0.5, 0.33, etc. as depth increases from say 0–5, 5–25, 25–250 to > 250 m is recommended, assuming that RQD/J_n is low enough (e.g. 0.5–25) for good hydraulic connectivity. This will help to adjust Q for some of the effective stress and water softening effects, in combination with appropriate characterisation values of SRF. Correlations with depth-dependent static deformation modulus and seismic velocity will then follow the practice used when these were developed.

2.6.2 Support Recommendations

Estimates of support requirements are required at three stages of a project: feasibility studies, detailed planning, and during excavation (Barton, et al 1980).

Support requirement estimation is essential as this allows the estimation of support costs in the early stages of a project. A practical way in which to estimate support was thus created in the development of the Q-Support chart (Barton, 2002).

The method of quantifying the quality of a rock mass to obtain a Q-value was developed using several case records until a relationship was achieved between Q, the excavation dimension and actual support installed (Barton et al 1974). Using these three variables Barton's Q-support chart (1974) was created from which users can determine what support is required in a given excavation based on a Q-value, the excavation span and the excavation support ratio (ESR).

In recent years, despite the number of new case records, no serious changes have been made to the initial Q parameter ratings. Only three strength/stress SRF ratings have been increased to accommodate rock masses under extremely high stress. This was done so that the appropriate quantities of systematic bolting and reinforced shotcrete can be received when plotted on the support chart (Barton 2002). Barton's Q-support chart is presented in Figure 2-3.

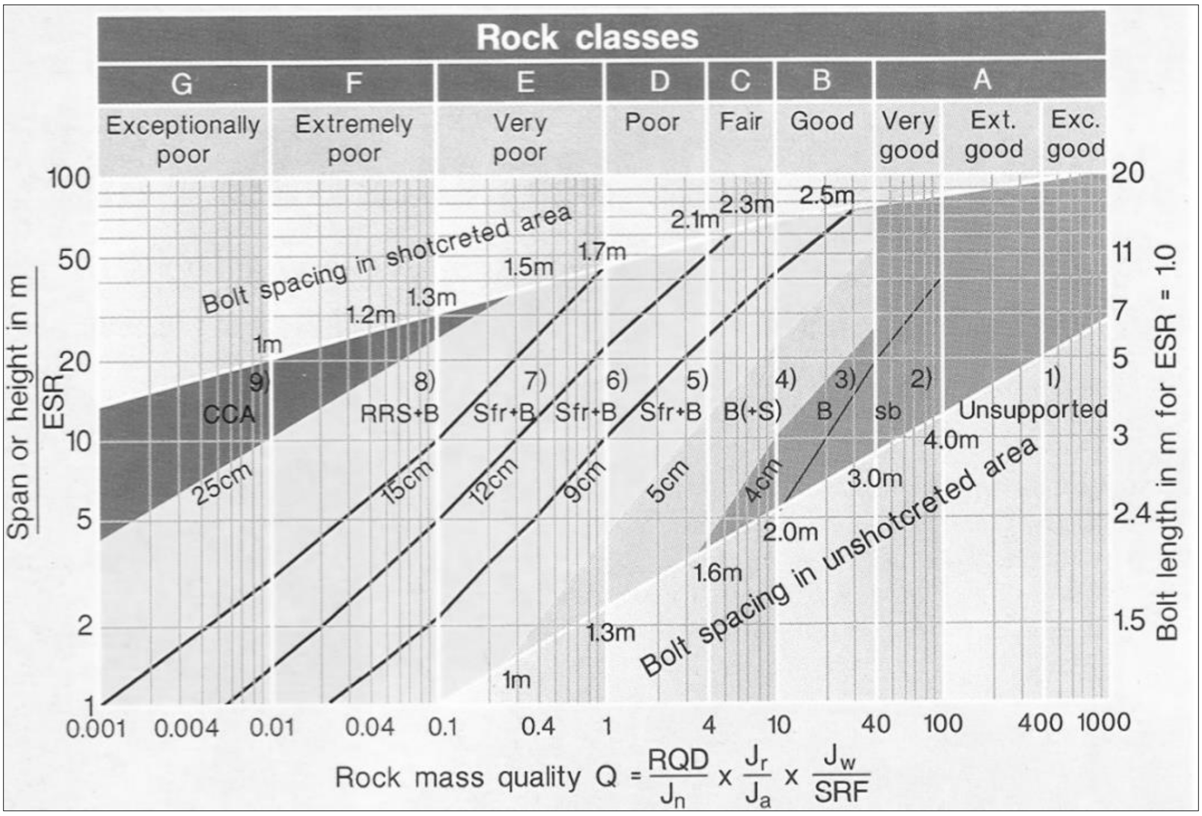


Figure 2-3: Updated Q-support chart (Barton, 2002)

2.6.3 Comparison to RMR

Correlations between the RMR and the Q-system have been proposed by many researchers. Perhaps the most popular correlation is that which was proposed by Bieniawski (1976) which was based on 117 case studies (Goel et al, 1996). These cases covered a wide range of RMR and Q values resulting in the following correlation:

$$RMR = 9 \ln Q + 44$$

Following this correlation, Barton (1995) also derived a relationship between the RMR and Q:

$$RMR = 15logQ + 50$$

While the correlations mentioned above are the most popular, a number of authors have also derived similar correlations for specific applications. A summary of correlations reflecting the determination of RMR from Q is presented in Table 2-18.

Table 2-18: Q and RMR System Correlations (after Milne et al, 1989)

Correlation	Source	Application
RMR = 13.5 logQ + 43	New Zealand	Tunnels
RMR = 12.5 logQ + 55.2	Spain	Tunnels
RMR = 5 lnQ + 60.8	South Africa	Tunnels
RMR = 43.89 – 9.9 lnQ	Spain	Soft Rock Mining
RMR = 10.5 ln Q + 41.8	Spain	Soft Rock Mining
RMR = 12.11 log Q + 50.81	Canada	Hard Rock Mining
RMR = 8.7 ln Q + 38	Canada	Tunnels, Sedimentary Rock
RMR = 10 ln Q + 39	Canada	Hard Rock Mining

2.6.4 Benefits and Limitations of the Q-System

Some of the benefits and limitations of the Q-system is outlined below:

- The use of the Q-system can be of great benefit during the feasibility and preliminary design stages of a project, when little detailed information on the rock mass and its stress and hydrologic environment is available. At its simplest, it may therefore be used as a check list to ensure that all relevant information has been considered. In a more comprehensive way, this classification system may be used to build up a depiction of the composition and characteristics of a rock mass, so as to provide initial estimates of support requirements for tunnels (Palmstrom and Broch, 2006).

- It is important to recognise that the use of a rock mass classification system like the *Q*-system does not replace detailed design procedures. However, the use of design procedures requires access to relatively detailed information on in situ stresses, rock mass properties and planned excavation process or sequence, none of which is generally available at an early stage in the project. When this information becomes available, the use of the rock mass classification schemes should thus be updated and used in conjunction with site specific analyses (Palmstrom and Broch, 2006).

- The *Q*-system is best applied in jointed rock masses where instability is caused by block falls. For other forms of ground behaviour in tunnels the suitability of the Q-System is limited (Table 2-19).

- The Q-System works best in the area highlighted in Figure 2-4. For conditions outside this area it is recommended that other tools are employed in the rock engineering process.

- The Q-system is best suited for use in the planning stages of a project, and is less useful for the recommendation of rock support during construction (Palmstrom and Broch, 2006).

- It must be kept in mind that the design correlations published in the numerous papers on the Q and RMR must be used very cautiously. This is mostly because if used in geological environments that differ significantly than those comprising the original case studies inaccurate results may be produced (Pells and Bertuzzi, 2007).

Table 2-19: Suitability of the Q-System (Palmstrom and Broch, 2006)

Types of ground behaviour	Suitability *
Stable	2
Fall of block(s) or fragment(s)	1 - 2
Cave-in	3
Running ground	4
Buckling	3
Slabbing, spalling	2
Rock burst	3 - 4
Squeezing ground	3
Ravelling from slaking or friability	4
Swelling ground	3
Flowing ground	4
Water ingress	4
** 1 Suitable; 2 Fair; 3 Poor; 4 Not applicable*	

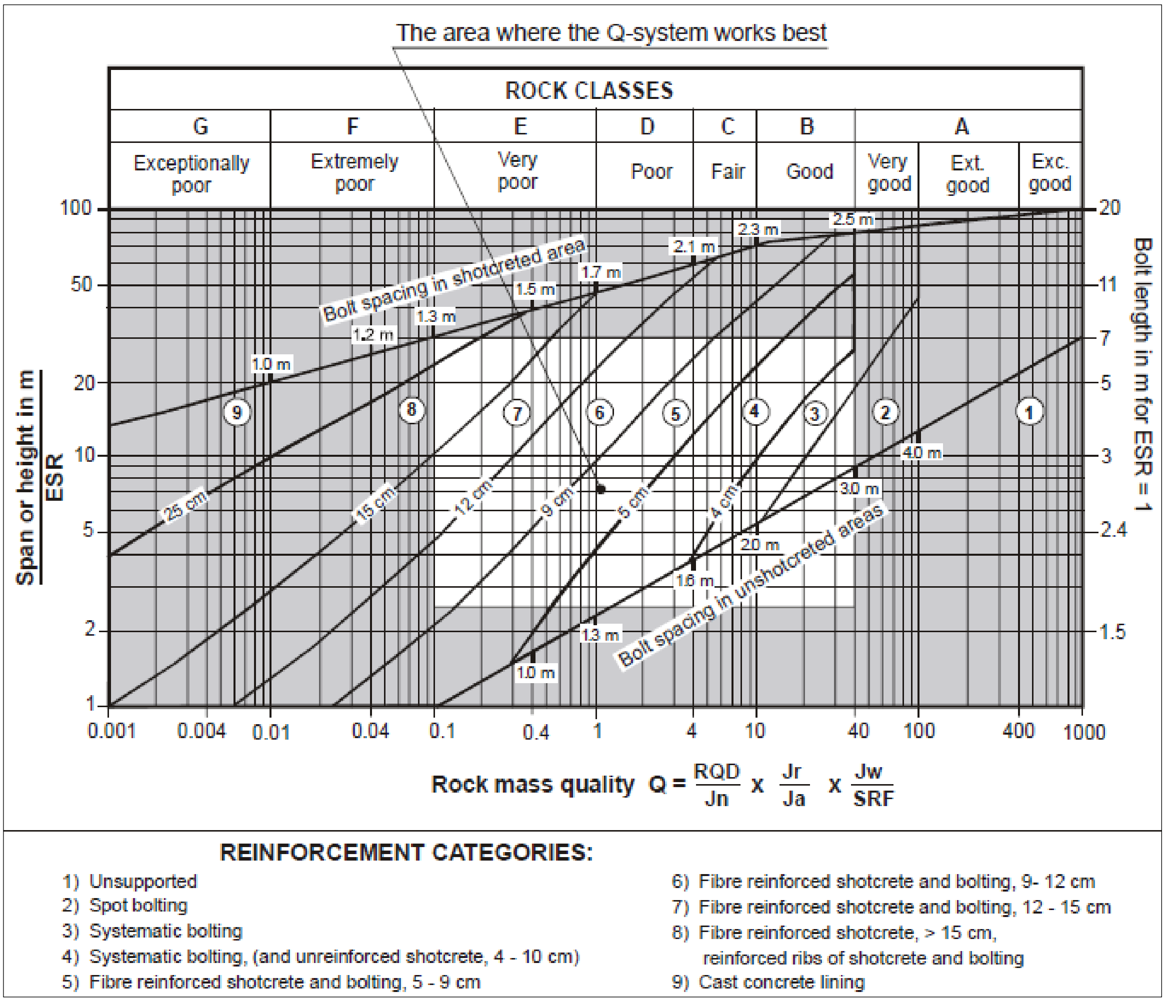


Figure 2-4: Limitations of the Q-Support Chart (Palmstrom and Broch, 2006)

Overall it may be concluded that the Q-system may be successfully applied in jointed rock masses, for the classification of stability and for providing support estimates for tunnels and rock caverns, preferably during the planning stages of a project (Palmstrom and Broch, 2006).

2.7 Laubscher's Mining Rock Mass Rating Classification System (MRMR)

The MRMR system is a classification system for use in jointed rock masses (Jakubec and Laubscher, 2000), and has been widely used in mining operations for over 30 years. This system is a means of identifying engineering parameters that are required for safe design and efficient mining (Laubscher and Taylor, 1976).

The MRMR classification approach by Laubscher (1990) is derived from the rock mass rating (RMR) classification system developed by Bieniawski (Bieniawski, 1973) and was initially published in 1975. This system considers the same parameters as the RMR system, however combines groundwater and joint condition. The parameters utilised in this classification system are as follows:

- Intact rock strength (UCS)
- Rock Quality Designation (RQD)
- Joint spacing
- Groundwater, and
- Joint Condition.

Through the allocation of rating values, within a specific range, each parameter is evaluated separately, resulting in an RMR value ranging between zero and 100. Rating values for each of these parameters is presented in Table 2-20, Table 2-21 and Figure 2-5.

Table 2-20: Mining Rock Mass Classification (after Laubscher, 1990)

<table>
<tr><th colspan="2">Parameter</th><th colspan="11">Range of Values</th></tr>
<tr><td rowspan="2">1</td><td>RQD</td><td colspan="2">100-97</td><td>96-84</td><td>83-71</td><td>70-56</td><td>55-44</td><td>43-31</td><td>30-17</td><td>16-4</td><td colspan="2">3-0</td></tr>
<tr><td>Rating
(= RQD x 15/100)</td><td colspan="2">15</td><td>14</td><td>12</td><td>10</td><td>8</td><td>6</td><td>4</td><td>2</td><td colspan="2">0</td></tr>
<tr><td rowspan="2">2</td><td>UCS (MPa)</td><td>185</td><td>184-165</td><td>184-165</td><td>164-145</td><td>144-125</td><td>124-105</td><td>104-85</td><td>84-65</td><td>44-25</td><td>24-5</td><td>4-0</td></tr>
<tr><td>Rating</td><td>20</td><td>18</td><td>16</td><td>14</td><td>12</td><td>10</td><td>8</td><td>6</td><td>4</td><td>2</td><td>0</td></tr>
<tr><td rowspan="2">3</td><td>Joint Spacing</td><td colspan="11">Refer to Figure 2-5</td></tr>
<tr><td>Rating</td><td colspan="6">25</td><td colspan="5">0</td></tr>
<tr><td rowspan="2">4</td><td>Joint Condition Including Groundwater</td><td colspan="11">Refer to Table 2-21</td></tr>
<tr><td>Rating</td><td colspan="6">40</td><td colspan="5">0</td></tr>
</table>

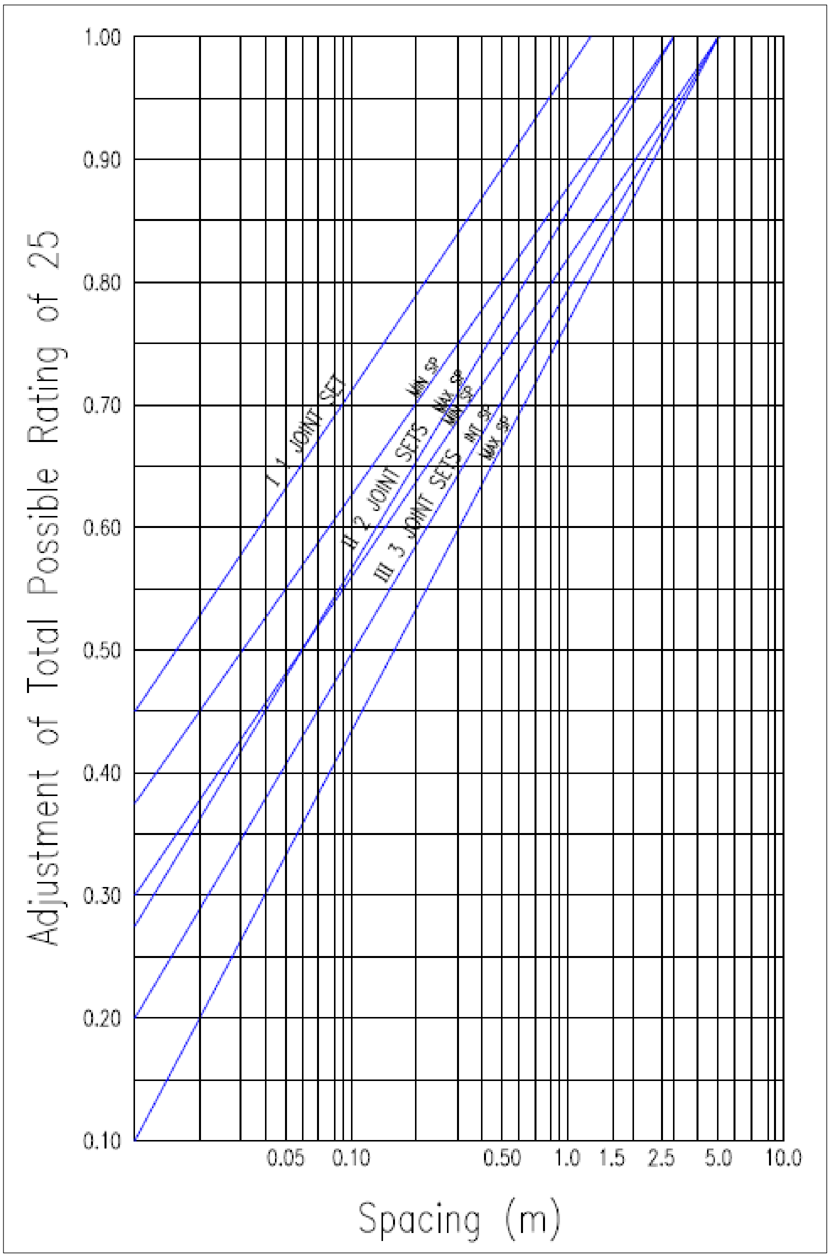


Figure 2-5: Graph for Joint Spacing Rating (Laubscher, 1990)

Once a RMR value is obtained a MRMR value can be determined by applying the appropriate adjustments for mining conditions. Adjustments considered include weathering, joint orientation, mining induced stresses and blasting damage. A weathering adjustment is appropriate for rock types which are susceptible to deterioration over time (Laubscher, 1990). The orientations of joints affect excavation stability and are therefore accounted for by an adjustment based on the orientation of the joints with respect to the vertical axis of the block. Stresses can impact the stability of an excavation based on the excavations shape, the depth at which it is excavated etc. It is thus important to apply a mining induced stress adjustment. Positive adjustments are applied for good confinement, while negative adjustments are applied for poor confinement (Laubscher, 1990).

Table 2-21: Groundwater and Joint Condition Rating (Laubscher, 1990)

Parameter	Description		Dry Condition	Wet Conditions		
				Moist	Moderate pressure 25-125 1/mm	Severe Pressure >125 1/min
A Joint Expression (large scale irregularities)	Wavy	Multi-Directional	100	100	100	95
		Uni-Directional	95 90	95 90	90 85	80 75
	Curved		89 80	85 75	80 70	70 60
	Straight		79 70	74 65	60	40
B Joint Expression (small scale irregularities or roughness	Very rough		100	100	95	90
	Striated or rough		99 85	99 85	80	70
	Smooth		84 60	80 55	60	50
	Polished		59 50	50 40	30	20
C Joint Wall Alteration Zone	Stronger than wall rock		100	100	100	100
	No alteration		100	100	100	100
	Weaker than wall rock		75	70	65	60
D Joint Filling	No fill – surface staining only		100	100	100	100
	Non softening and sheared material (clay or talc free	Coarse Sheared	95	90	70	50
		Medium Sheared	90	85	65	45
		Fine Sheared	85	80	60	40
	Soft sheared material (eg talc)	Coarse Sheared	70	65	40	20
		Medium Sheared	65	60	35	15
		Fine Sheared	60	55	30	10
	Gouge thickness <amplitude of irregularity		40	30	10	
	Gouge thickness <amplitude of irregularity		20	10	Flowing material 5	

Following the development of the MRMR system modifications and improvements were made to system in 2000 (Laubscher and Jakubec, 2000). These modifications are presented in Figure 2-6.

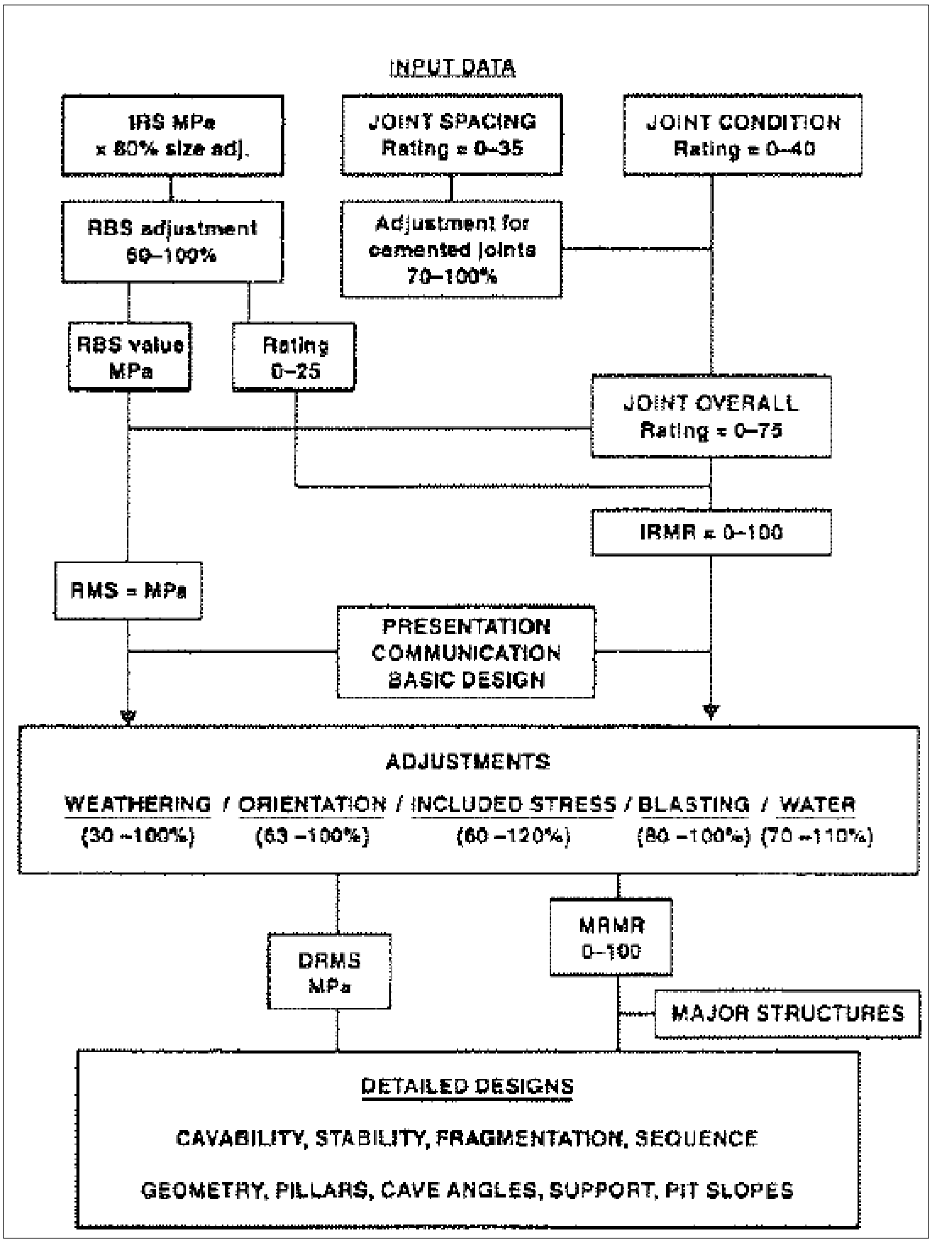


Figure 2-6: Modifications to the MRMR System (Jakubec and Laubscher, 2000)

Following the use of the MRMR system for mining applications, this system was further identified for use in cave mining, as the MRMR system provides the necessary data for an empirical definition in terms of evaluating the hydraulic radius (Laubscher, 1994).

When the MRMR system is properly applied, the results are highly acceptable for use. However, as rock masses do not conform to an ideal pattern, engineering judgement is also often required to make the use of this system a success (Laubscher and Jakubec, 2000).

2.8 Geological Strength Index (GSI)

The use of the GSI chart (Figure 2-7) is based on “descriptive categories of rock mass structure and discontinuity surface conditions” (Hoek, 1994). The original Geological Strength Index (GSI) chart was developed for qualified and experienced geologists or engineering geologists and has been found to work successfully when these qualified practitioners are available (Hoek et al, 2013). However, there are many instances where data collection is carried out by persons who are less comfortable with these qualitative descriptions.

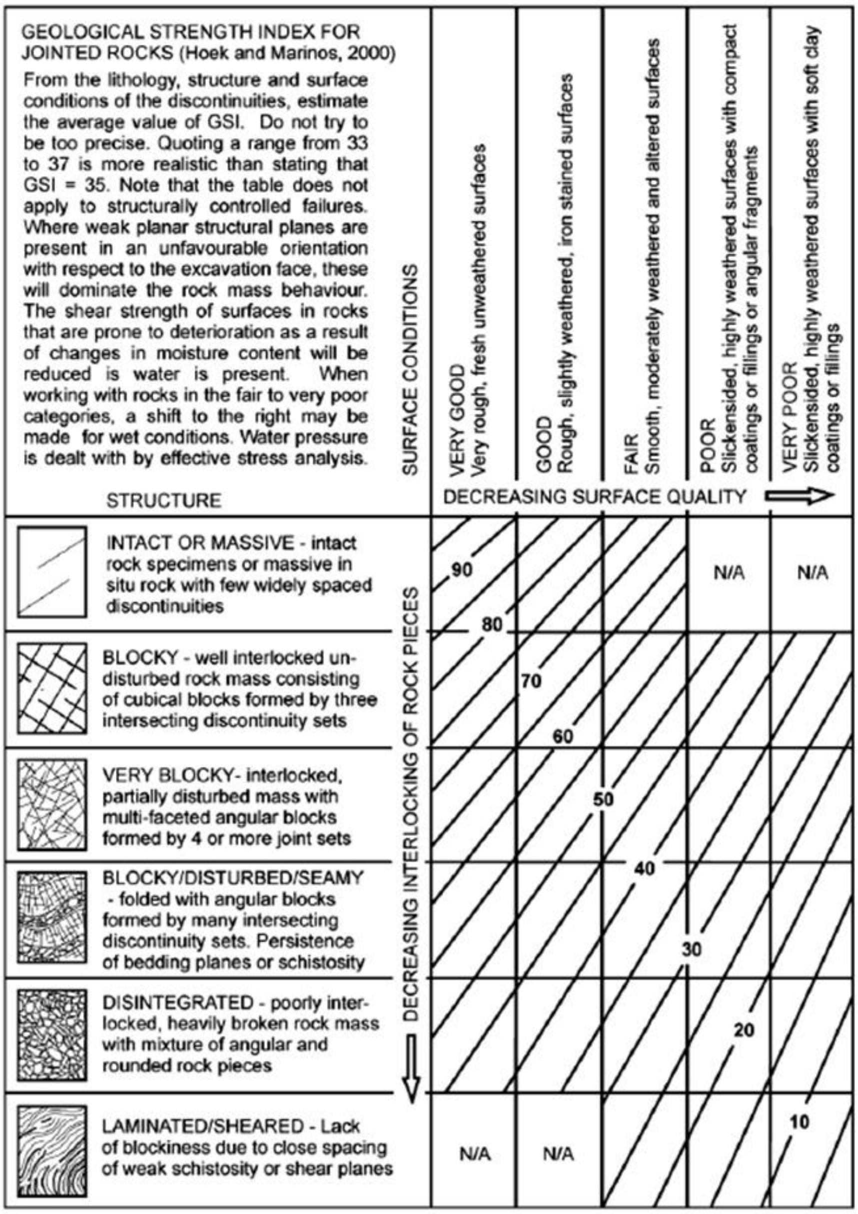


Figure 2-7: Orginal GSI chart (Hoek, 1994)

To this end, the issue of quantifying GSI has been developed. The original GSI chart published by Hoek and Marinos (2000) has thus been modified with the addition of 2 scales, Scale A and Scale B (Figure 2-8). "Scale A has been added for representation of the 5 divisions of surface quality with a range of 45 points, defined by the approximate intersection of the GSI = 45 line on the axis. Scale B represents the 5 divisions of the block interlocking scale with a range of 40 points in the zone in which quantification is applied" (Hoek et al, 2013).

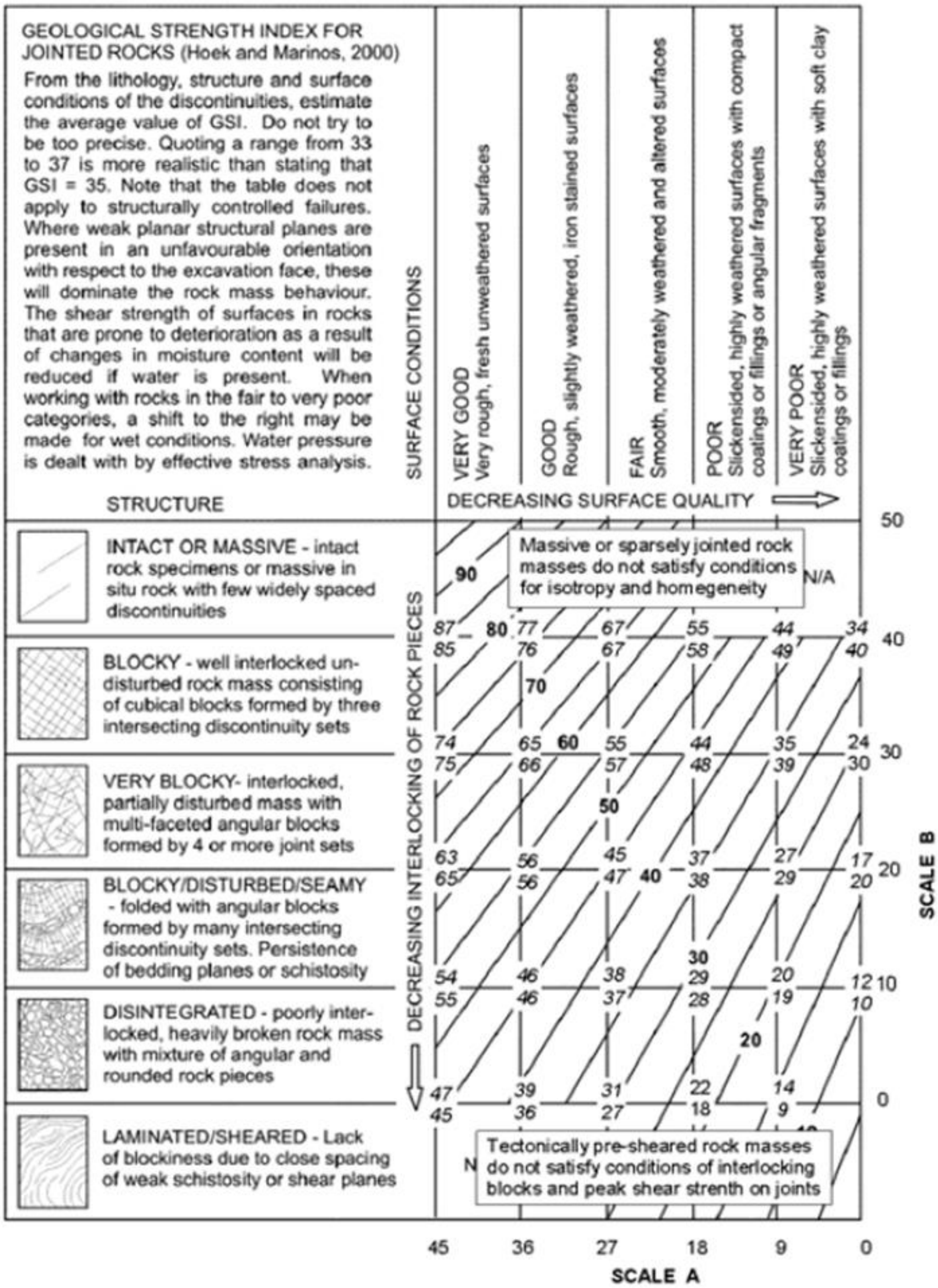


Figure 2-8: Original GSI chart with the addition of Scale A and Scale B (Hoek et al, 2013)

Further to the addition of scales, it was observed that the original GSI lines were hand drawn, and are thus neither parallel nor equally spaced (Hoek et al, 2013). A correction to the original lines has thus also been made, and is presented in Figure 2-9.

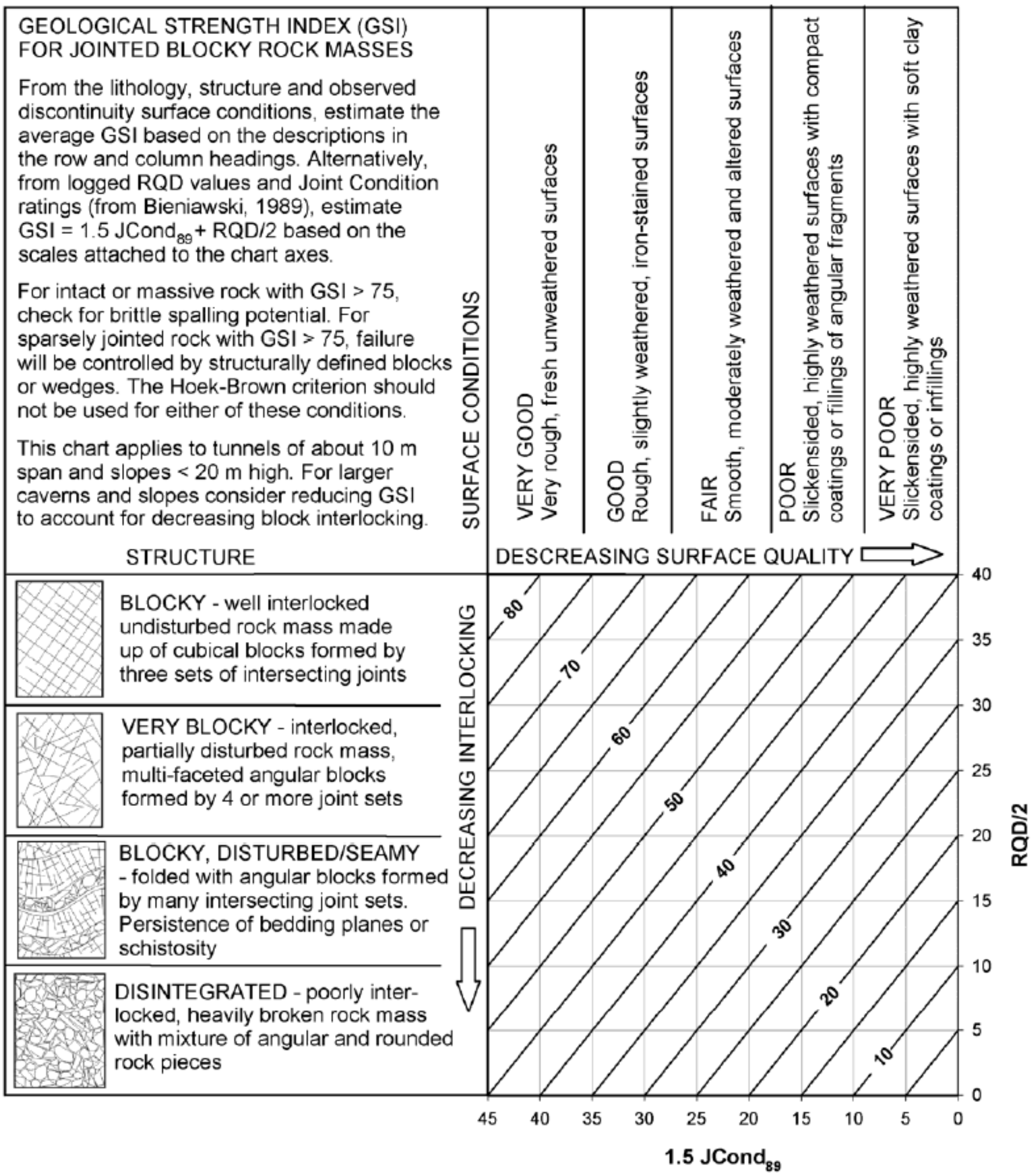


Figure 2-9: Updated GSI Chart (Hoek et al, 2013)

It must be kept in mind that the correction of the GSI lines and the addition of the scales do not change the original chart's function in terms of estimating GSI from field observations (Hoek et al, 2013). Instead, the updated chart serves as both a qualitative and quantitative tool for rock mass classification.

While the updated GSI chart provides a quantitative approach to this rock mass classification system, the following conditions and limitations apply:

- It is assumed that the rock mass is homogeneous and isotropic. Distinct discontinuity sets, which are closely spaced (relative to the proposed excavation size) should therefore be present (Figure 2-10).
- For intact massive or very sparsely jointed rock, the GSI chart should not be utilised. This is because there are an insufficient number of joints sets in massive rock to satisfy the conditions of homogeneity and isotropy. To avoid the use of the chart in this case, the upper row of the updated chart has been removed (Hoek, 2013).

- The GSI system does not directly take into account the effects of water and stresses in the rock mass.
- "The GSI system does not provide a direct correlation between rock mass quality and GSI value, however, it is suggested that GSI can be related to RMR by GSI = RMR – 5, for a rock mass of good quality" (Hoek, 1997).
- As the lower row of the GSI chart represents previously sheared, transported or heavily altered materials, the conditions described above do not apply. This row has thus also been removed from the updated GSI chart (Hoek et al, 2013).

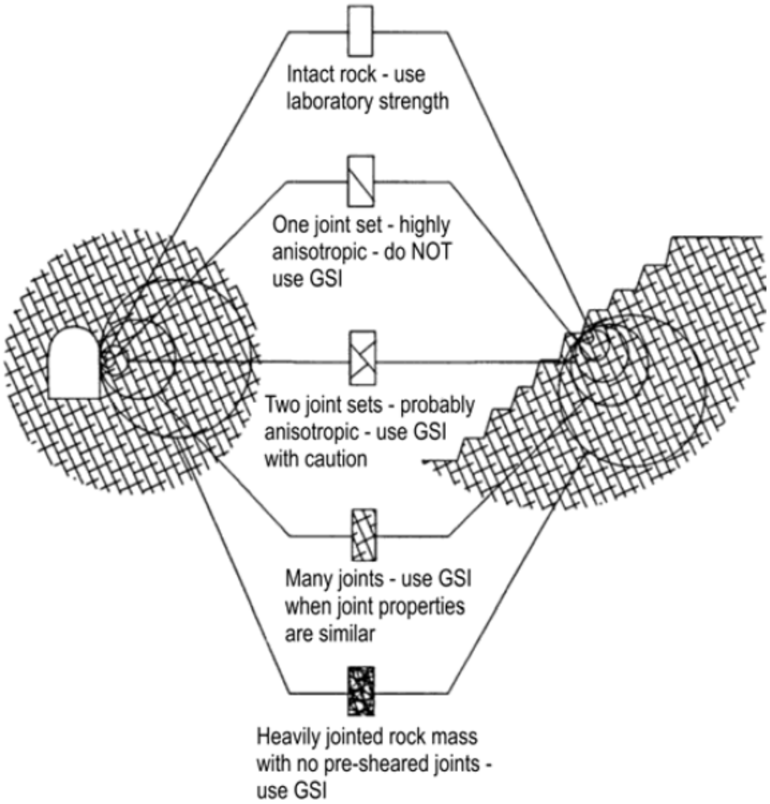


Figure 2-10: Limitations of GSI depending on scale (Hoek et al, 2013)

GSI in terms of RQD and Joint Condition

Upon investigation it was found that Scale A of the updated GSI chart relates to the Joint Condition ($JCond_{89}$) rating defined by Bieniawski (1989), and that Scale B corresponds well to RQD as defined by Deere (1963). Figure 2-9 illustrates that Scale A is defined by 1.5 multiplied by $JCond_{89}$ while Scale B is defined as RQD/2. The value of GSI is thus given by the sum of these two terms resulting in the following relationship:

$$\boldsymbol{GSI = 1.5JCond_{89} + RQD/2}$$

In situations where $JCond_{89}$ data is not available, two alternative parameters for the surface quality axis may be utilised. The first is the $JCond_{76}$ version of the $JCond_{89}$ parameter, which can be used as a direct replacement of $JCond_{89}$:

$$\boldsymbol{GSI = 2JCond_{76} + RQD/2}$$

The second is the quotient Jr/Ja which correlates to $JCond_{89}$ providing an acceptable approximation for engineering applications:

$$GSI = \frac{52Jr/Ja}{1 + Jr/Ja} + RQD/2$$

Overall it may be concluded that the quantification of GSI using the above mentioned relationships is acceptable for the characterisation of jointed rock masses (Hoek et al, 2013).

2.9 Palmstrom's correlation between Volumetric Joint Count and RQD

Joints intersecting a rock mass divide the rock into blocks of several sizes, ranging from crushed rock to 1 cm^3 blocks to blocks that are massive and are several m^3 in size. These sizes are the result of joint spacing, number of joint sets, and the length and persistence of the joints present (Palmstrom, 2005). Block size plays an important role in the behaviour of a rock mass. This parameter is thus utilised in many rock mass classification systems applied today, including:

- The ratio between RQD and Jn (factor for number of joint sets) in the Q system.
- RQD and joint spacing (S) in the RMR system.
- Block volume (Vb) in the RMi (Rock Mass Index) system.
- GSI (Geological Strength Index) system.

Block size is also utilised in certain numerical modelling methods as an input.

2.9.1 Types of Block Size Measurements

Even though joints form complex 3-dimensional shapes in the crust, measurements are predominantly made on 2-dimensional surfaces and along 1-dimensional borehole lines or scanlines (Palmstrom, 2005). Only a limited proportion of the joints are therefore correctly measured in a location. The method to be used for measuring block size depends on local conditions on site and the stage of a project. For example in the planning or exploration stage, where the rock is hidden by soil, rotary core, shafts, adits or geophysical measurements may be used to determine block size. In the construction stage, where rock mass conditions can be easily observed in a tunnel, mine or cutting, more accurate measurements are possible. Table 2-22 illustrates the main methods used to measure block size.

Table 2-22: Methods for measuring block size (Palmstrom, 2005)

Rock Surfaces	Drill Cores	Refraction Seismic
Block Volume (Vb)	Rock Quality Designation (RQD)	Sound Velocity of rock masses
Volumetric Joint Count (Jv)	Fracture Frequency	
Joint Spacing (S)	Joint Intercept	
Weighted Joint Density (wJd)	Weighted Joint Density (wJd)	
Rock Quality Designation (RQD)	Block Volume (Vb)	

Note that it is important to select the method yielding representative recordings (Palmstrom, 2005).

2.9.2 Volumetric Joint Count

Volumetric joint count (Jv) was developed by Palmstrom in 1974. Jv is a 3-dimensional measurement for the density of joints. It is thus well suited to rock masses where well-defined joint sets occur. Jv may be defined as the number of joints intersecting a volume of 1 m^3. The following equation may be utilised to determine Jv:

$$Jv = \frac{1}{S1} + \frac{1}{S2} + \frac{1}{S3} + \frac{1}{Sn}$$

As random joints are not considered in joint sets, they are not accounted for in the equation above. Palmstrom (1982) has thus presented a correction where a spacing of 5 m per random joint may be applied where random joints exist:

$$Jv = \frac{1}{S1} + \frac{1}{S2} + \frac{1}{S3} + \frac{1}{Sn} + \frac{Nr}{5\sqrt{A}}$$

Where: *Nr* is the number of random joints in in the actual location

A is the area of the rock mass in m^2

Random joints should be accounted for where observed as these joints represent a part of the total number of discontinuities present in a rock mass (Palmstrom, 2005). Furthermore, they may significantly influence the overall behaviour of the rock mass.

The classification of Jv is presented in Table 2-23.

Table 2-23: Classification of Jv (slightly modified after Palmstrom, 2005)

Jv	<1	1 - 3	3 - 10	10 - 30	30 – 60	> 60
Classification	very low	low	moderate	high	very high	crushed

2.9.3 Rock Quality Designation using Jv

RQD was initially developed to offer a quantitative estimate of rock mass quality from drill core. Since RQD is one dimensional, based only on intact core greater than 10 cm, it is difficult to correlate RQD to other measurements of jointing (Palmstrom, 2005). Palmstrom (1974) made the first attempt to correlate RQD, when the volumetric joint count was presented. The correlation between RQD and Jv is as follows:

$$RQD = 115 - 3.3Jv$$

Where: *RQD* = 0 for Jv > 35

RQD = 100 for Jv < 4.5

More recently, Palmstrom (2005) extended his research of the RDQ correlation with Jv with the inclusion of computer generated blocks of varying shapes and sizes. From the research conducted, Palmstrom (2005) suggested the following modification to his equation:

$$RQD = 110 - 2.5Jv$$

This new relationship was found to provide somewhat better results compared to the initial RQD = 115 -3.3Jv relationship.

2.10 Literature Review Findings

Based on the literature review conducted, the following findings are observed:

The object of a classification system is to assign a value to a rock mass as opposed to a vague descriptive term. Practical experience has indicated that rock mass classification systems thus work well. It must be kept in mind that each classification system should be used as a guide, and that each case should be examined in detail (Laubscher, 1977). It is also important to note that the success and accuracy of a classification system depends greatly on the quality of the data collected and sampling of the area that is being investigated.

The use of more than one rock mass classification system is recommended as this allows for validation and confirmation of estimates made. This in turn increases confidence in the results achieved.

In order to gain the most value from a rock mass classification system, the user should be aware of the limitations of each system, and the system should be used by practitioners with practical experience (Palmstrom, 2008).

It is essential to note that use of the general classification design approach in the Q-System and all other classification systems is contrary to detailed engineering design processes. This is since there are no applied mechanics calculations of stress or displacement, no computations, or information, as to loads, strains and stresses in the support elements (shotcrete, rockbolts and sets), and therefore nothing against which to compare field-monitoring data. The position of the classification design approach in relation to modern limit state design is thus unknown and unknowable (Pells and Bertuzzi, 2007).

The use of classification systems can be of great benefit during the feasibility and preliminary design stages of a project, when little detailed information on the rock mass and its stress and hydrologic environment is available (Pamstrom and Broch, 2006).

Based on the research findings a geotechnical investigation and support design was completed using Barton Q, Laubscher's MRMR and the quantification of GSI, as these rock mass classification systems are considered the most appropriate for use for this particular case study. More than one system was chosen to validate and confirm results. Chapter 3 that follows, details the geotechnical investigation. This is followed by the rock mass classification undertaken which is presented in chapter 4.

3 GEOTECHNICAL INVESTIGATION

In 2013, Platreef Resources requested that a geotechnical assessment of a vertical borehole for the design and construction of the Platreef vertical production shaft 2 be conducted. The suitability of sinking the vertical production shaft in the designated position to a depth of 1100 m is addressed, together with design recommendations pertaining to the support of the shaft.

Upon investigation of the geology of the northern limb, it was found that this area comprises one of the world's largest primary economic deposits of Platinum Group Elements. The regional and local geological setting of the Platreef project area is described in some detail below.

3.1 Regional Geological Setting

The northern limb, commonly known as the Platreef, forms part of the 2.0 Ga Bushveld Complex, which is located in the north eastern part of South Africa. The mafic to ultramafic portion of the complex, referred to as the Rustenburg Layered Suite, intrudes the Paleoproterozoic sediments of the Transvaal Supergroup and Archean granite basement in the northern part of the Kaapvaal Craton (Cawthorn, 1996). The Rustenburg Layered Suite is divided into five zones known as the: Marginal Zone Norite, Lower Zone Pyroxenite and Dunite, Critical Zone Chromite-Pyroxenite-Norite Cyclic Units, Main Zone Gabbronorite and Upper Zone Anorthosite, Gabbro and Magnetite (Cawthorn, 1996). These zones are illustrated in Figure 3-1.

The Ysterburg-Planknek and Zebediela Faults play an important role in the regional geology of the northern limb of the Bushveld Complex as these are second-order, northwest trending extensional structures within the Transvaal Supergroup (Figure 3-1). It is thought that these faults could be due to reactivated structures formed in the Archean Basement during the Murchison Orogeny. Overall the Ysterburg-Planknek and Zebediela faults compartmentalise the northern limb and disrupt the magmatic stratigraphy in the area (Platreef Resources Internal Memorandum, 2012).

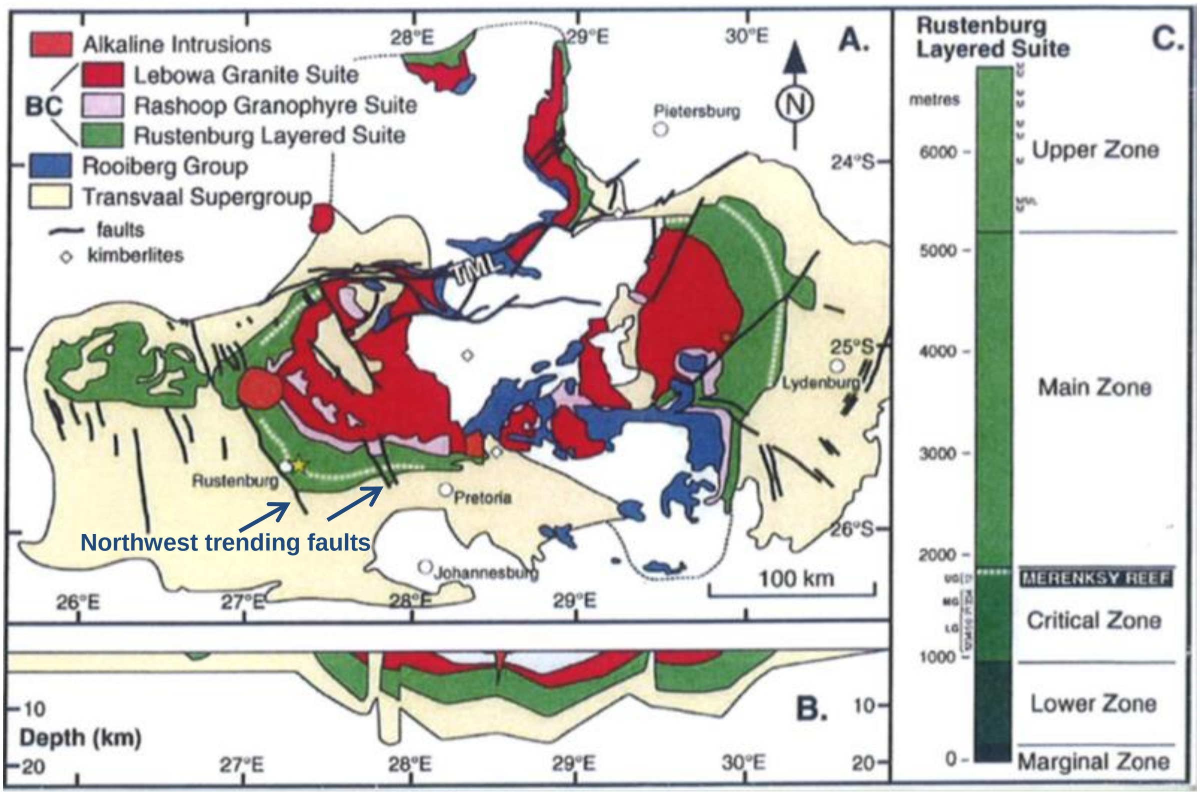


Figure 3-1: Regional Geological Setting (after Scoates and Freidman, 2008)

3.2 Platreef Site Geology

The Platreef comprises a complex zone of igneous and hybrid lithological units that lie at the base of the Northern Limb of the Bushveld Complex, in contact with metasedimentary and granitic floor rock types. "It is found that the variability of lithology and thickness along strike is attributed to underlying structures and assimilation with the country rocks". (Platreef Resources Internal Memorandum, 2012). Lithological units encountered on Platreef include pyroxenites, gabbronorites, norites, harzburgites, peridotites and serpentinites. Granite veins are also very common (AMC Consultants, 2013).

The major cyclic units that are present within the project area (Figure 3-2) compare with the Upper Critical Zone (UCZ) rock sequence as described for the main Bushveld Complex. These units are described as follows:

- The Turfspruit Cyclic Unit (TCU): This unit comprises Merensky anorthosite and pyroxenite and hosts the Bushveld's principal mineralized reefs. The TCU is continuous across most parts of the Project area. (Platreef Resources Internal Memorandum, 2012).
- Norite Cycles (NC1 and NC2): These are cyclic units found adjacent to the TCU and include the Pseudo Reef, and UG2. (Platreef Resources Internal Memorandum, 2012).
- T2 Upper and T2 Lower: Within the TCU unit, high-grade platinum group elements, nickel and copper mineralization is hosted within a pegmatoidal, mafic to ultramafic sequence, which is bound by chromitite stringers and coarse-grained sulphides known as T2 (Platreef Resources Internal Memorandum, 2012). The T2 pegmatoid is subdivided into an upper pyroxenitic unit (T2 Upper) and a lower olivine-bearing pyroxenitic or harzburgitic unit (T2 Lower).
- T1: Overlying the T2 unit is a barren non-pegmatoidal feldspathic pyroxenite unit of variable thickness, termed T1. A second mineralized zone, known as the T1m, is found near the top of the T1 unit. This zone comprises disseminated, medium- to coarse-grained sulphides (Platreef PEA, 2013).

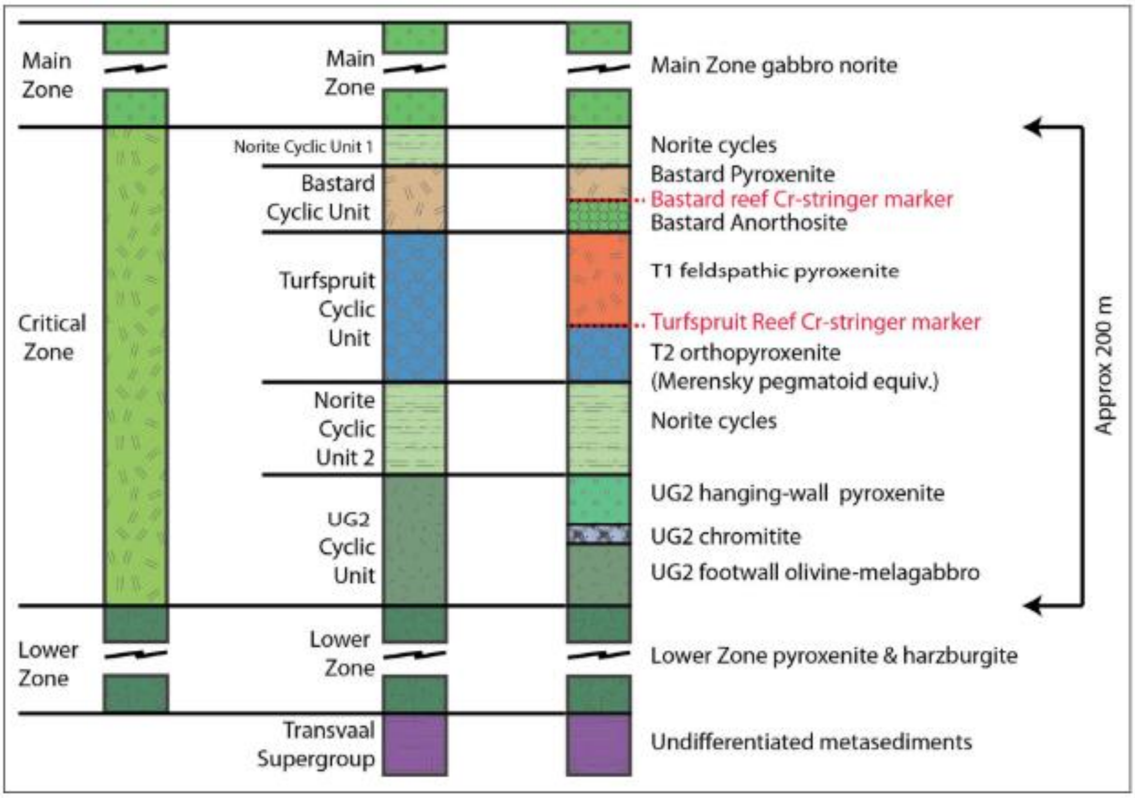


Figure 3-2: Local Stratigraphy of the Platreef Project Area (Ivanhoe Mines, 2015)

3.3 Geotechnical Borehole GT017

A vertical borehole (GT017) was drilled at the centre of the proposed shaft position in order to determine the nature, quality and properties of the rock mass to a depth of 1100 m. The maximum allowable deflection of the geotechnical borehole was set at 2° to ensure that the drilling remained as vertical as possible over its full length, such that the data collected was representative of the sub-surface ground conditions at the position of the proposed shaft.

The position of borehole GT017 (shaft 2) is depicted in Figure 3-3 (SRK Report No.450790, 2013). Note that boreholes GT007, GT012, GT013, GT014 and GT016 in Figure 3-3 represent boreholes drilled for the Platreef Pre-Feasibility Study (PFS) for Platreef Mine (SRK Report No. 458213v1, 2014).

During the drilling process the drill rods became lodged at the bottom of the drill hole. The drill hole was thus deflected away from these rods (at 922m). The depths of the core recovered with respect to this deflection are as follows:

GT017: 0.00 m – 922.70 m

GT017D1: 895.00 m – 1100.07 m

The deflection of the borehole GT017 is presented from Figure 3-4 to Figure 3-6.

From the deviation graphs of GT017 it is observed that the borehole deflected by a total of 7 m in the north-south direction and 16 m in the east-west direction. Considering the shaft diameter of 10 m, the author is of the opinion that this deviation is not critical. The data from the borehole is thus reasonably representative of the rock mass within the vicinity of shaft 2.

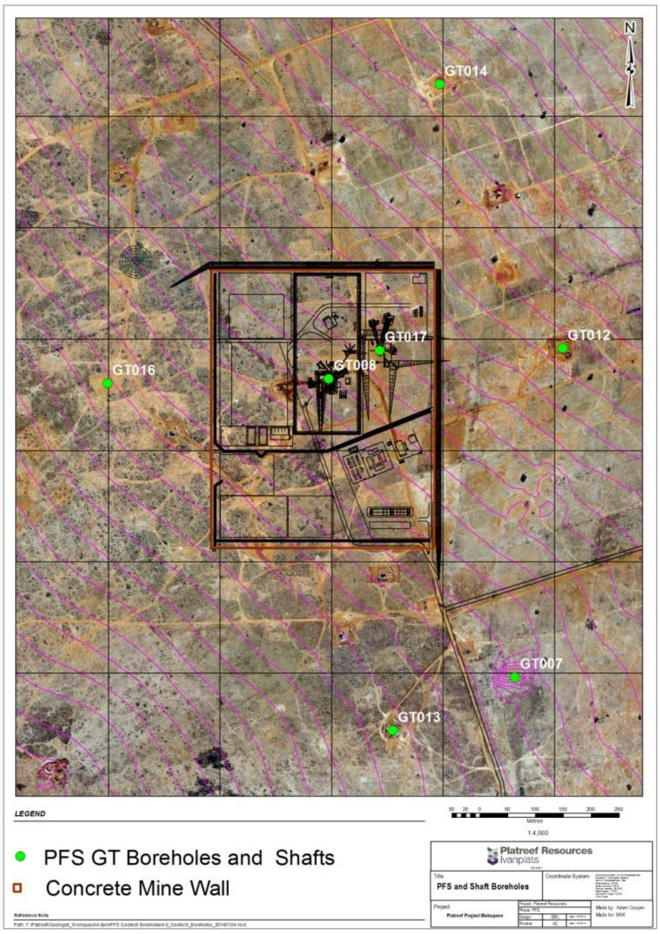


Figure 3-3: Location of GT017

Figure 3-4: Deviation of GT017 from planned position (plan view)

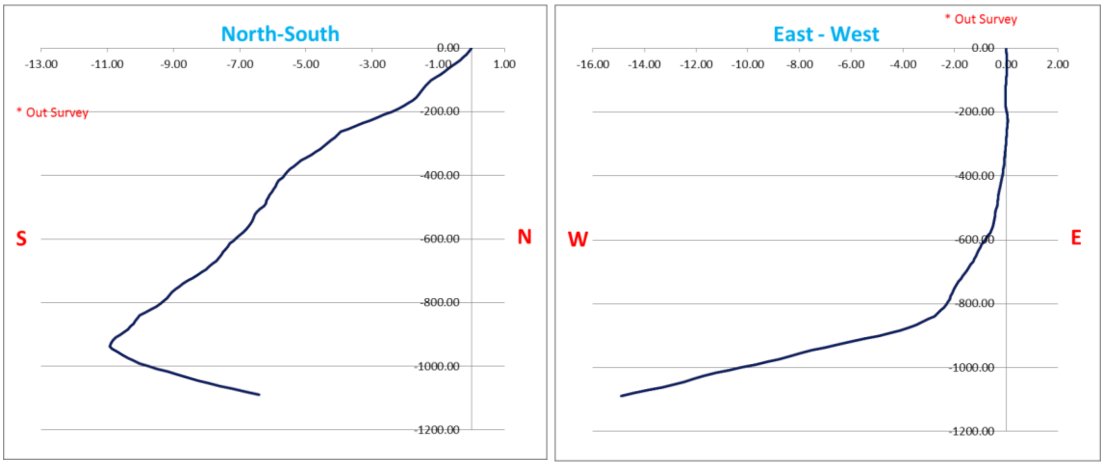


Figure 3-5: North-South and East-West deviation of GT017 (cross section)

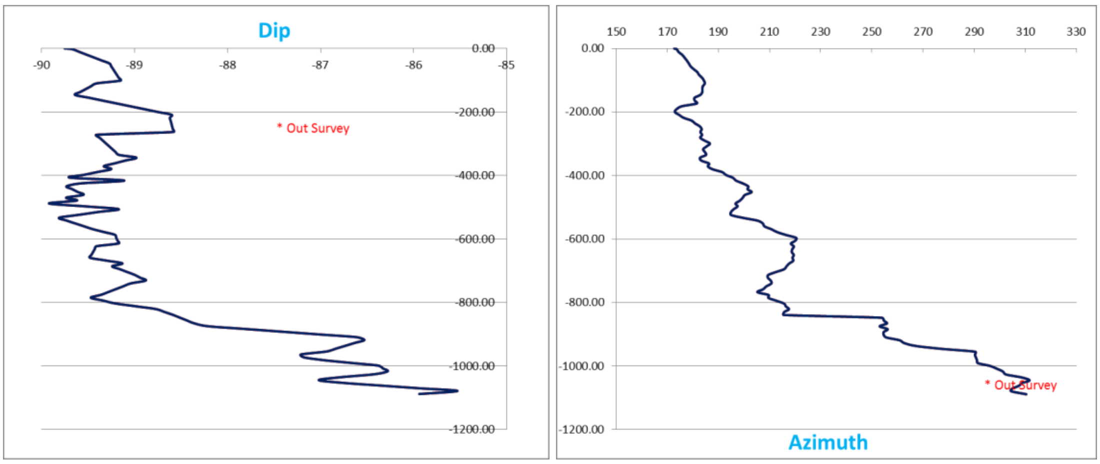


Figure 3-6: Dip and Azimuth deviation of GT017 (cross section)

3.4 Geotechnical Logging

The core recovered from geotechnical borehole GT017 was geotechnically logged by Platreef representatives, according to recommended methodologies, in order to identify discrete geotechnical zones, i.e. zones of rock expected to behave uniformly when exposed within the mining environment. Geotechnical data was collected with respect to the following parameters:

- The extent and distribution of geotechnical zones;
- The rock types present along the depth of the hole;
- RQD;
- The quality of matrix / rock mass defects, i.e. faults, shear zones, intense fracturing and zones of deformable material;
- The quantity of solid rock versus weak rock;
- The Intact Rock Strength / hardness (IRS);
- The degree and nature of rock weathering;
- The total number / density / frequency of structures (FF);
- The condition of structures, i.e. roughness profile, wall alteration and infilling.

As core of the Platreef site was well known from previous investigations (logging of shaft 1 core and QAQC site visit for 65 boreholes for the Platreef PFS study), a QA/QC of the geotechnical logs for shaft 2 was conducted using the borehole photographs. From the logs it was observed that in some cases the length of matrix recorded did not coincide with core photographs. To this end the author has adjusted the length of matrix records to 0.00 m where matrix was not evident. An example of this is illustrated in Figure 3-7, where a length of 0.14 m of matrix was recorded for an interval from 573.22 m to 573.42 m, although no matrix is evident.

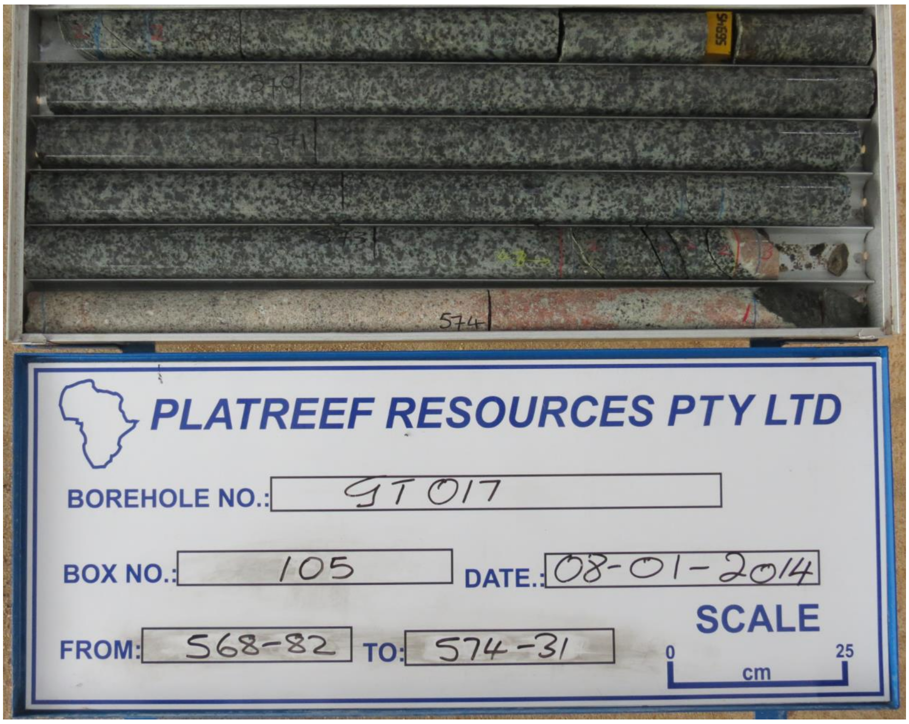


Figure 3-7: Borehole photograph illustrating no evidence of matrix

3.5 Lithological Units

Rock types intersected along the shaft 2 borehole are as follows:

- Gabbronorite (GN)
- Norite (N)
- Granite Veins (GRV)
- Anorthosite (AN)
- Feldspathic Pyroxenite (FPX)
- Hornfels (HF)
- Mafic Pegmatite Veins (MPV)
- Orthopyroxenite (OPX)

Soil and scree (SS) from natural ground level to a depth of 7.00 m constitutes the residual overburden unit for GT017 (Figure 3-8). Below the overburden, gabbronorite (GN) is the dominant rock type; from a depth of 7.00 m to 753.83 m. Intercalated with the gabbronorite are veins of potassium-rich granite (GRV), approximately 0.10 m to 5.00 m thick (Figure 3-9). Together the overburden and GN layer constitute the upper critical zone (UCZ) of the hangingwall (section 3.2).

Underlying the gabbronorite, there is a gradational contact with anorthosite (AN). The anorthosite is intercalated with norite (N) to make up the first norite cycle (NC1) layer from 753.83 m to 787.97 m.

Underlying NC1, feldspathic pyroxenite (FPX) is present from a depth of 787.97 m to 828.60 m. This makes up the T1 layer of the reef zone (Figure 3-10), which is the feldspathic pyroxenite unit of Turfpruit Cyclic Unit (TCU). Orthopyroxenite (OPX) and FPX are present from 828.60 m to 831.65 m to make up the T2U layer of the reef zone, which is the pyroxenite unit of the TCU (section 3.2). Sulphides are well developed in T1 and T2U, and together make up the reef zone for GT017.

Norite (N) underlies the reef zone and constitutes the dominant rock type at depth (Figure 3-11). Norite occurs from a depth of 831.65 m to the end of hole at a depth of 1100.07 m. Within the norite are layers of hornfels (HF) that range in thickness from 0.45 m to 6.41 m. Mafic pegmatitic veins (MPV) and GRV < 1 m thick are also present within the norite layer. Overall the norite layer constitutes the footwall and the second norite cycle (NC2) layer of the stratigraphy.

Table 3-1: Major Lithological Units identified in GT017

Depth (m)		Lithological Unit	Mining Position	Stratigraphy	Thickness (m)
From	To				
0.00	7.00	SS	Overburden	MZ	7.00
7.00	753.83	GN	Hangingwall	MZ	746.83
753.83	787.79	N/AN	Hangingwall	NC	33.96
787.79	828.60	FPX	Reef Zone	T1	40.81
828.60	831.65	OPX/FPX	Reef Zone	T2U	3.05
831.65	1100.07	N	Footwall	NC2	268.42

The lithological distribution of shaft 2 is presented in Table 3-2.

Table 3-2: Lithological distribution of shaft 2

Lithology	GN	N	GRV	AN	FPX	HF	MPV	OPX
Percentage	61 %	25 %	6 %	3 %	3 %	1 %	0.2 %	0.1 %

Note: 0.7 % of the borehole comprises soil and scree

A stratigraphic column indicating the lithologies encountered in GT017 is presented in Figure 3-12.

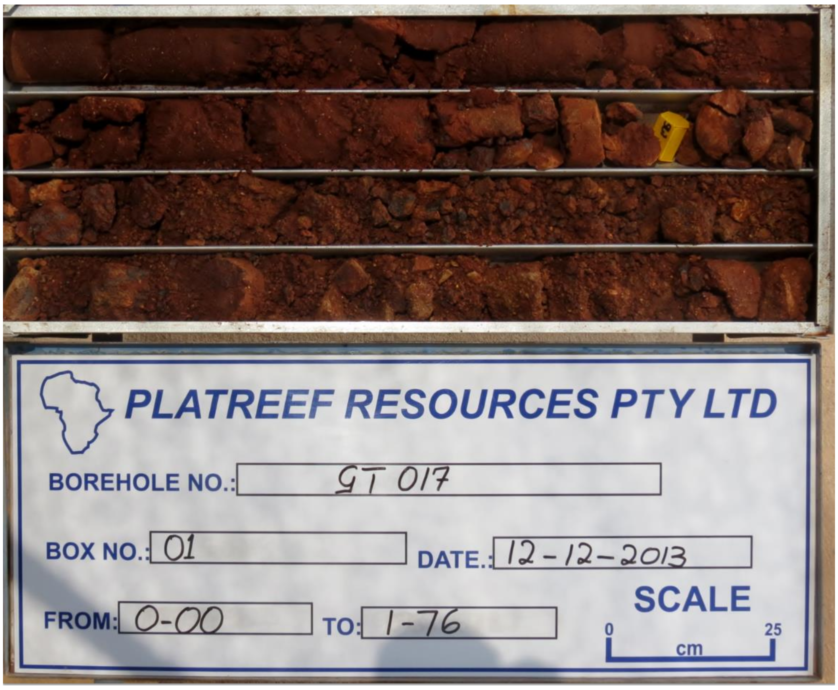


Figure 3-8: GT017 soil and scree

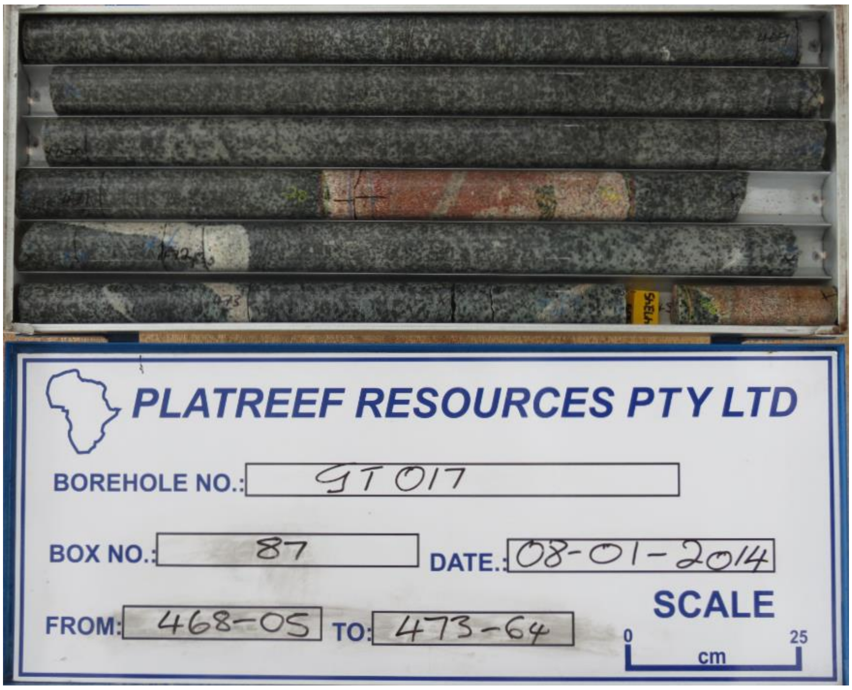


Figure 3-9: Hangingwall gabbronorite intersected with granite veins

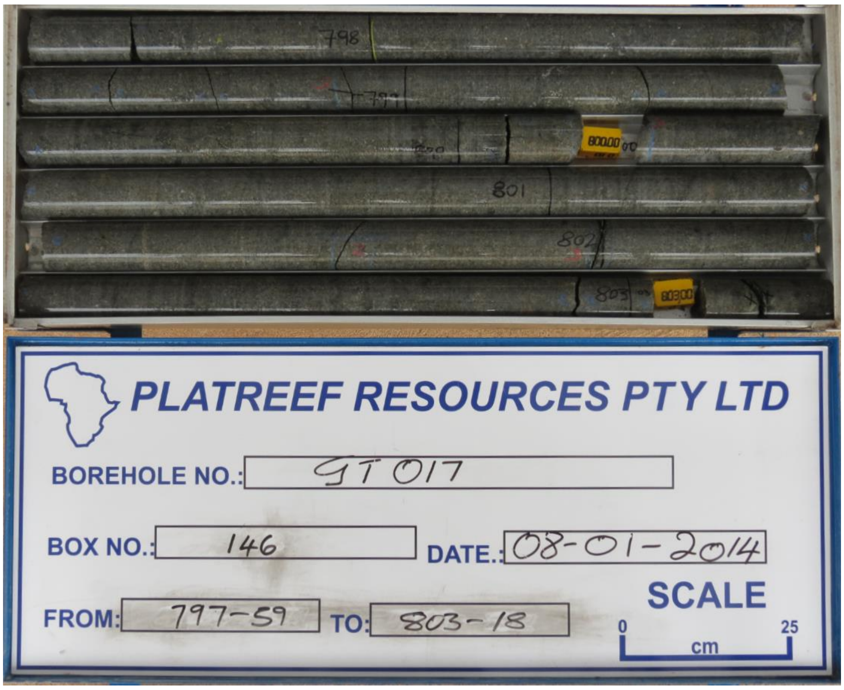


Figure 3-10: Reef feldspathic pyroxenite

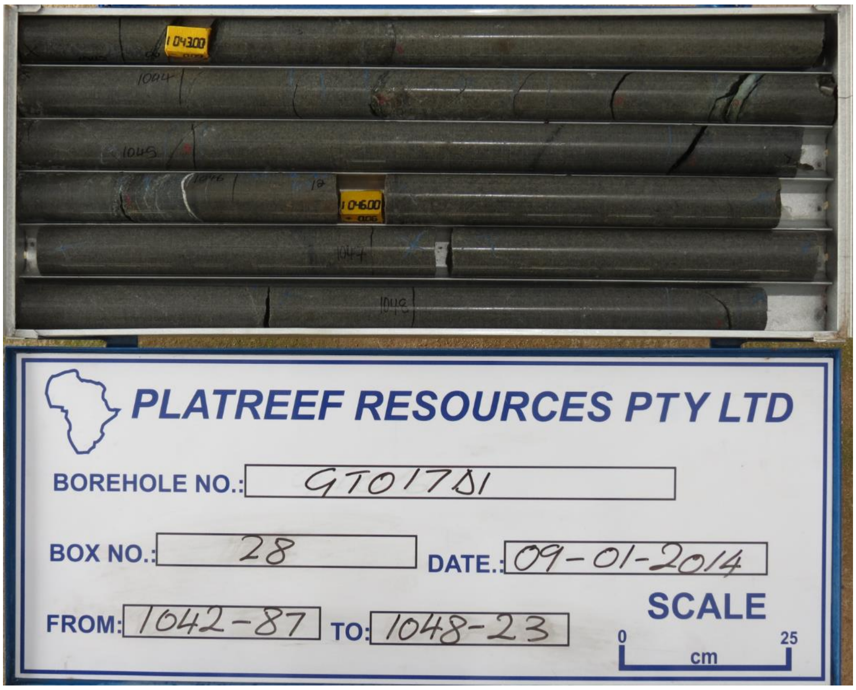


Figure 3-11: Footwall norite

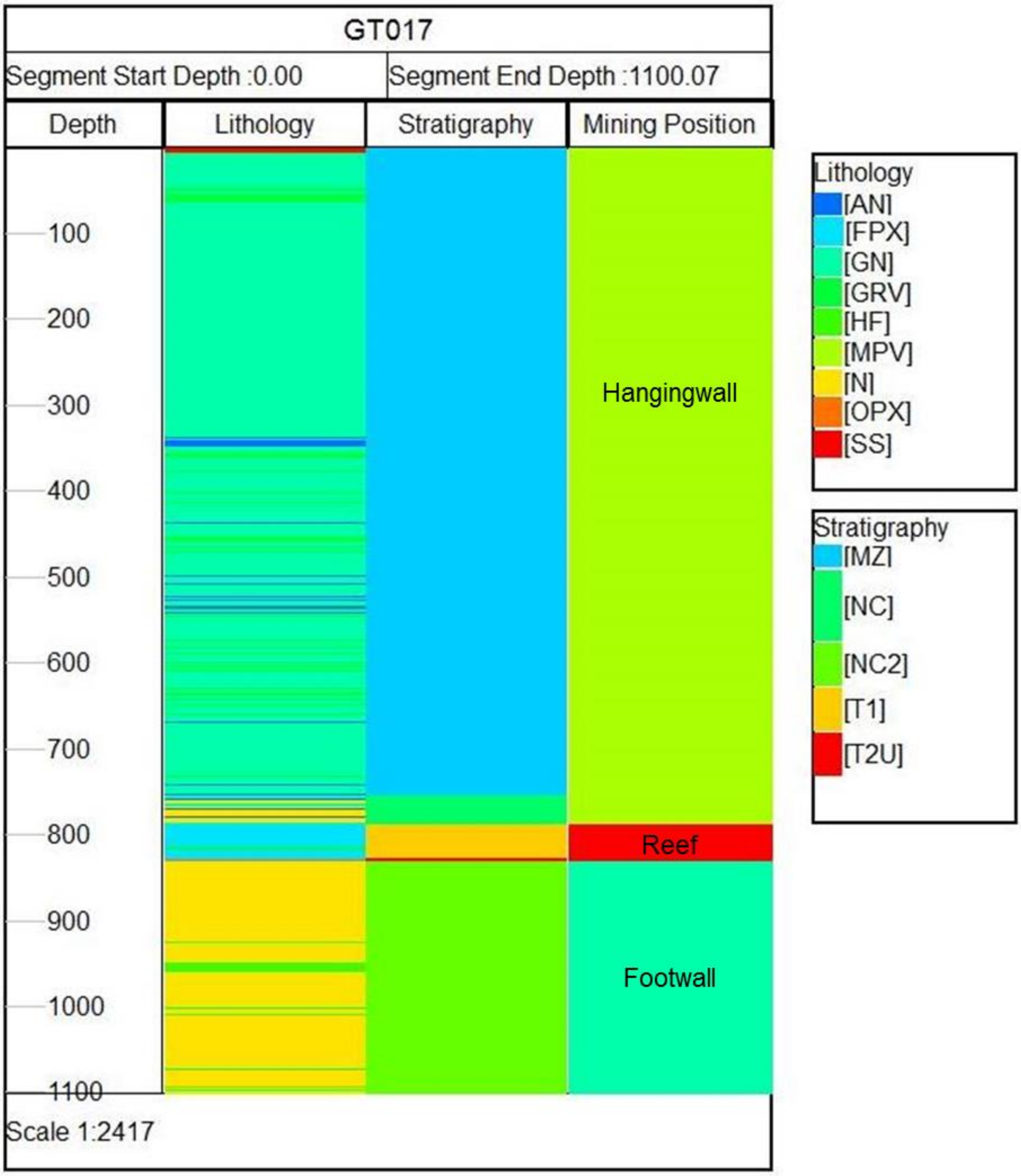


Figure 3-12: GT017 Stratigraphic Column

3.6 Laboratory Testing

Laboratory tests were conducted on representative rock specimens from GT017 to obtain an indication of the intact rock strengths of the lithological units for rock mass classification purposes. The laboratory testing programme requested comprised of the following geomechanical tests:

- UCM Uniaxial Compressive Strength with Young's Modulus and Poisson's Ratio;
- UCS Uniaxial Compressive Strength;
- TCS Triaxial Compressive Strength;
- BIT Indirect Tensile Strength Test (Brazilian Method).

UCM and BIT tests were conducted on the major occurring rock units that were intersected by borehole GT017; while UCS tests were performed on minor occurring rock units identified (Table 3-3). TCS tests were conducted on GN, FPX and N, as these are the dominant units in the hangingwall, orebody and footwall respectively. All testing was conducted by Rocklab in Pretoria, South Africa.

Table 3-3: Laboratory Testing Programme

Test	No. of tests for Major Rock Types				
	GN	N	GRV	AN	FPX
UCM	6	6	6	6	6
BIT	3	3	3	3	3
TCS	9	9	-	-	9
	No. of tests for Minor Rock Types				
	HF		MPV		
UCS	6		6		

As test specimens that failed along discontinuities were excluded from the analysis, the full suite of laboratory results for GT017 could not be utilised. In order to improve the laboratory analysis, successful test results from GT017 were supplemented with relevant test results from nearby borehole GT008 (Figure 3-3) so as to achieve a full suite of test results that incorporated all lithological units intersected in GT017.

The total number and type of geomechanical tests chosen for the laboratory analysis is presented in Table 3-4. All test results are listed in Appendix A.

Table 3-4: Number of laboratory tests used in analysis from GT017 and GT008

	GT017			GT008			Total no. of tests		
	BIT	UCM/ UCS	TCS	BIT	UCM	TCS	BIT	UCM/ UCS	TCS
GN	3	5	8	-	-	-	**3**	**5**	**8**
GRV	3	5	-	-	-	10	**3**	**5**	**10**
AN	3	6	-	-	3	11	**3**	**6**	**11**
N	3	1	6	-	3	-	**3**	**4**	**6**
FPX	3	4	8	-	-	-	**3**	**4**	**8**
HF	-	5	-	3	-	4	**3**	**5**	**4**
MPV	-	1	-	-	-	-	**-**	**1**	**-**

From the available test results, intact rock strength was estimated by fitting a Hoek-Brown failure envelope to the data for each rock type. This approach was adopted as it provides a simplified linear correlation between the UCS, BIT and TCS test results. In this method the Hoek-Brown failure envelope is represented as a straight line where the Y axis intercept represents the intact rock strength squared (σci2) (Hoek, 2012). The intact rock strength for each unit is thus the square root of the y-intercept. The individual Hoek-Brown failure envelopes for each rock unit are presented from Figure 3-13 to Figure 3-17.

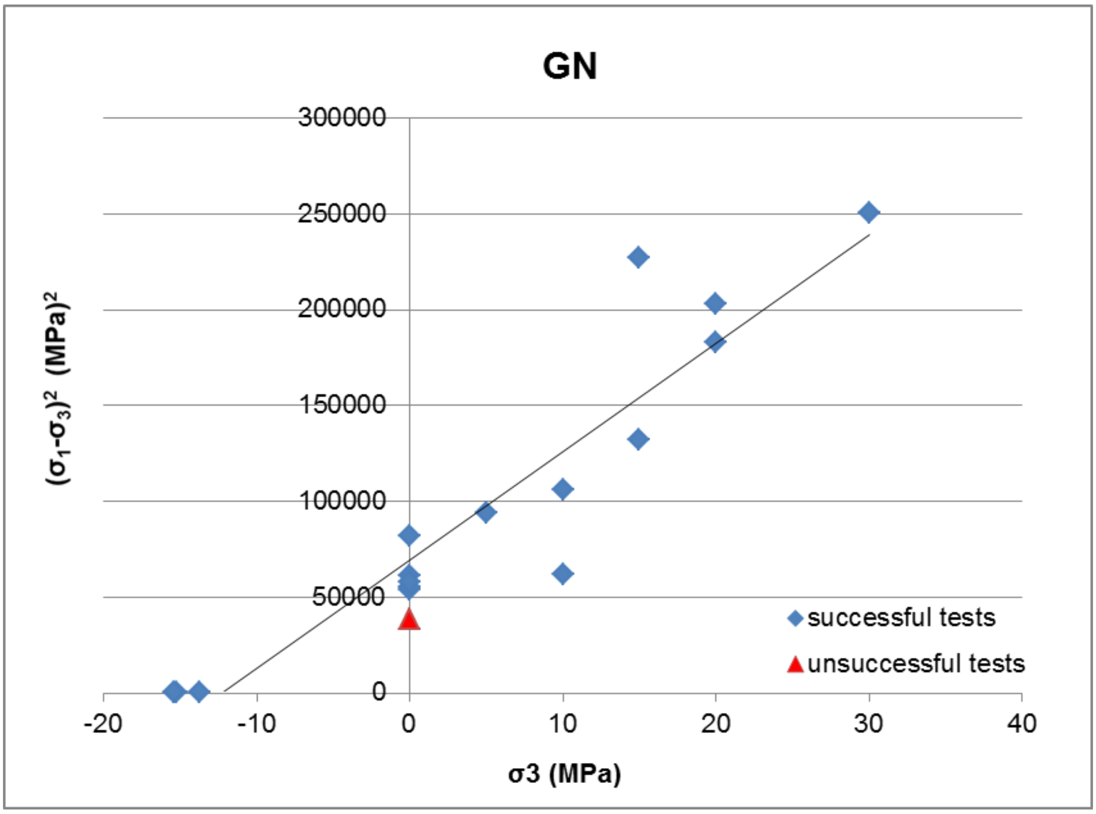


Figure 3-13: GN test results

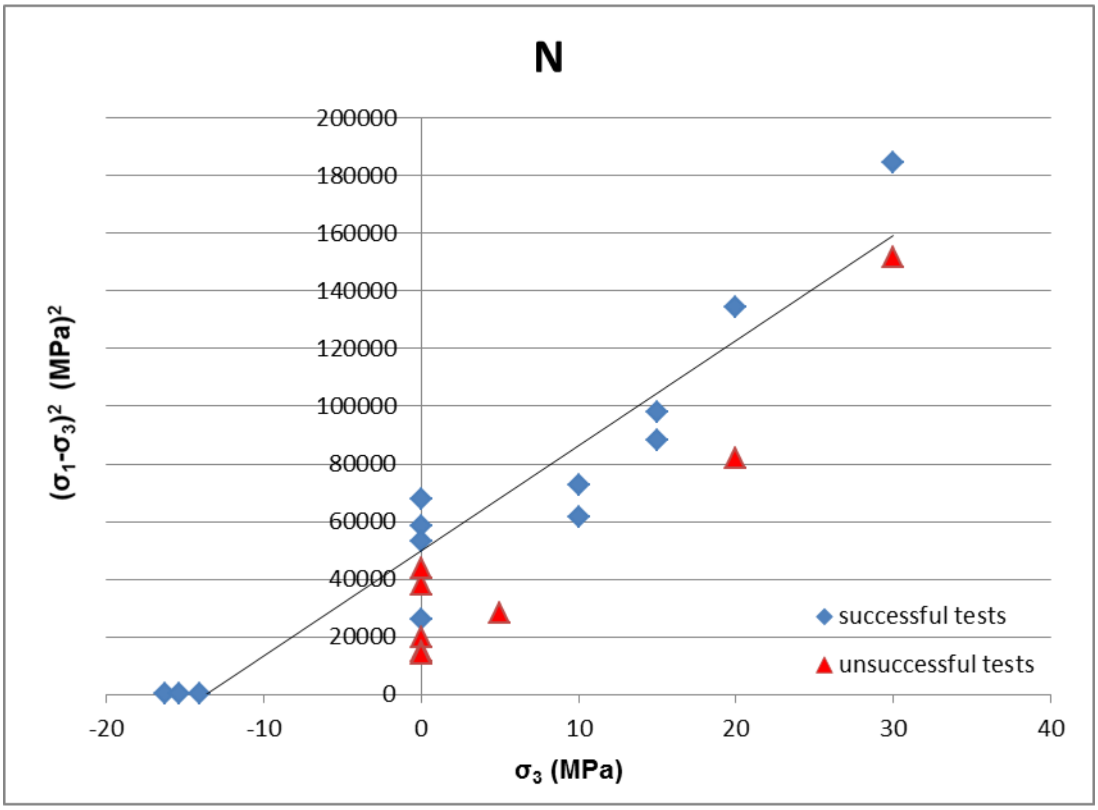


Figure 3-14: N test results

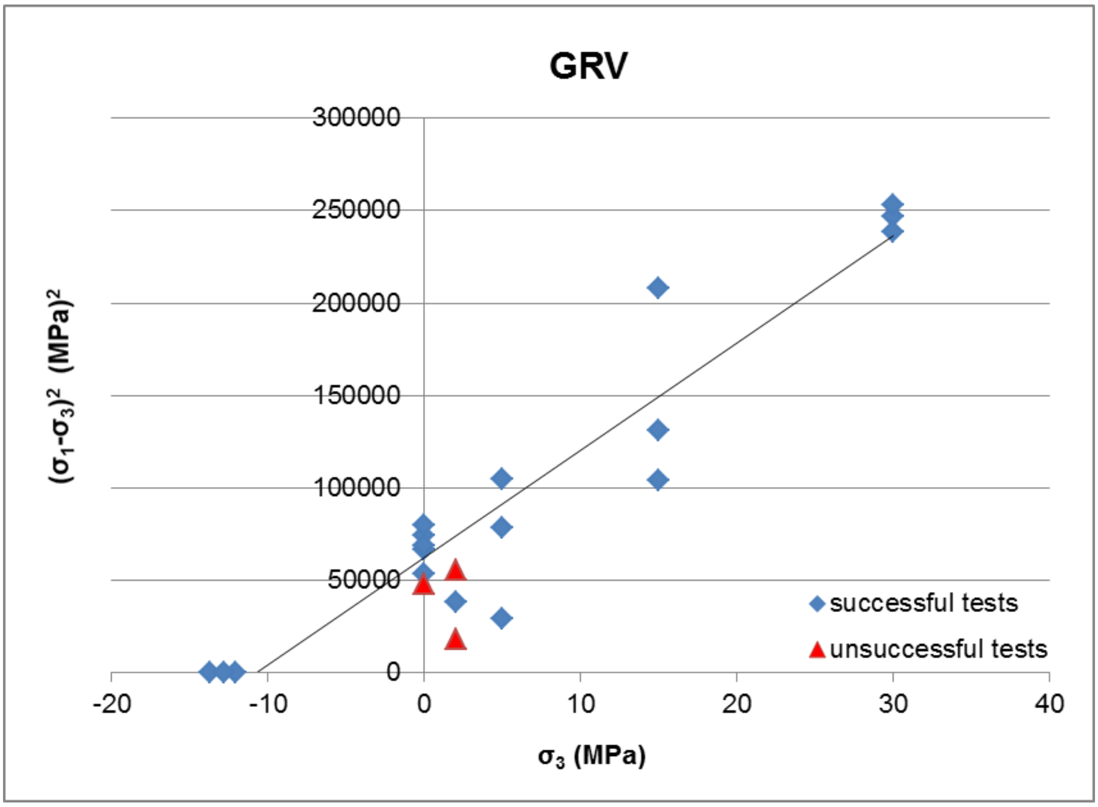


Figure 3-15: GRV test results

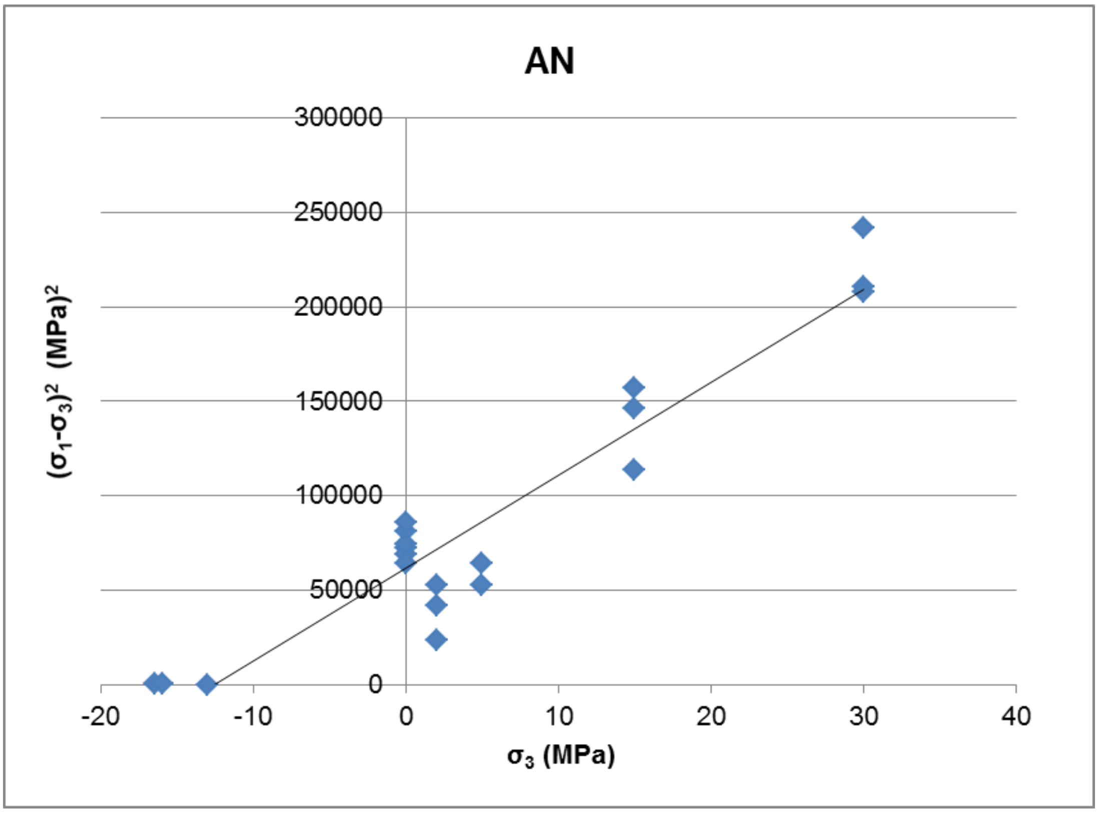


Figure 3-16: AN test results

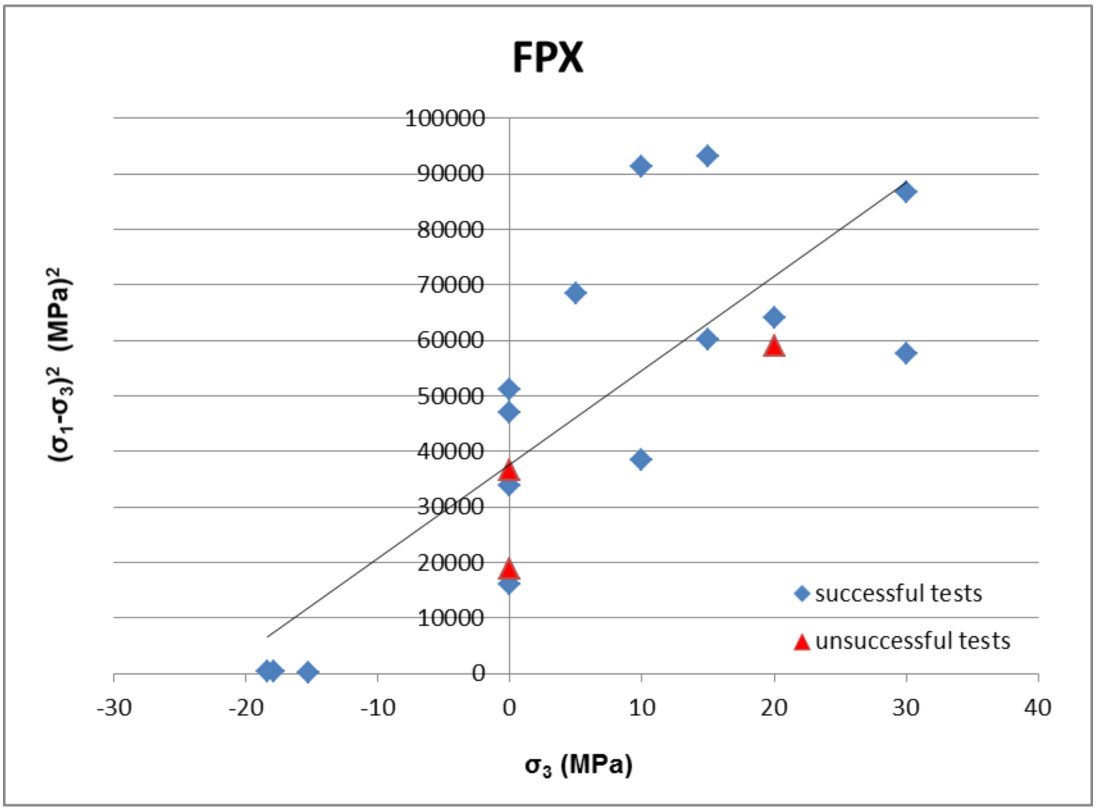


Figure 3-17: FPX test results

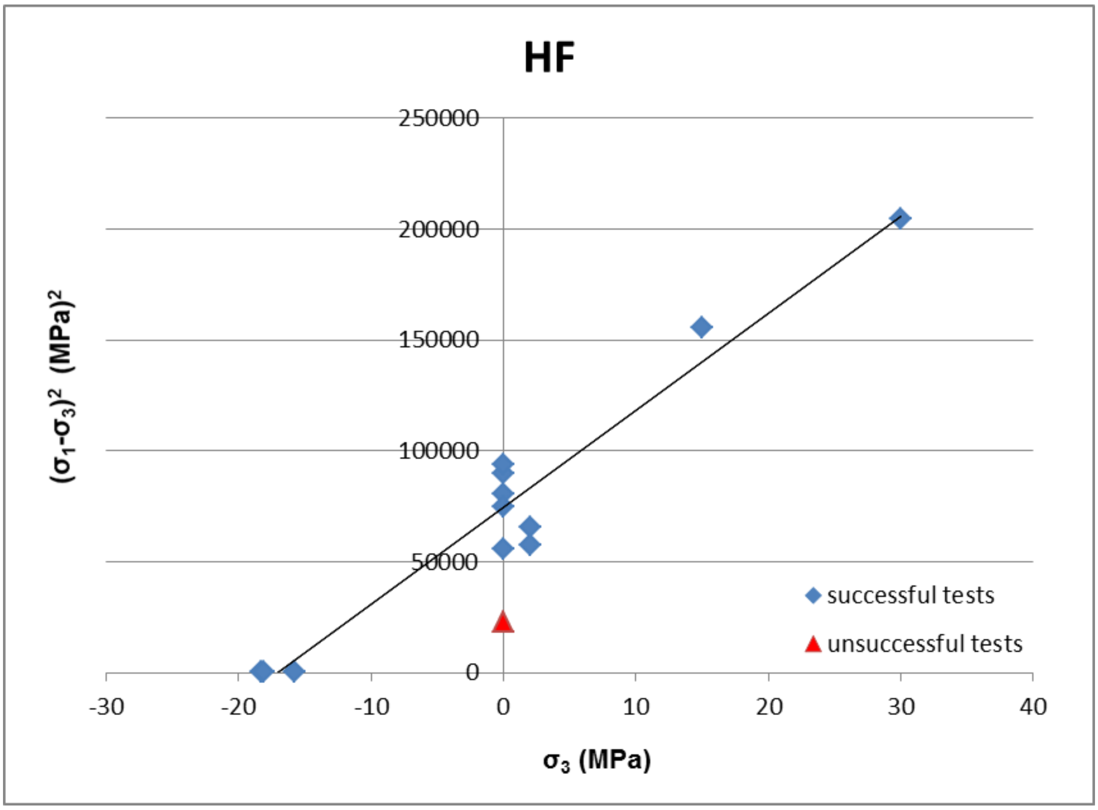


Figure 3-18: HF test results

As there was only one successful UCS test result produced for MPV, with no other test results available, a Hoek-brown failure envelope could not be constructed for this rock type. MPV only occurs as a total thickness of 1.82 m throughout the borehole (0.1 %). This is not a concern as the intact rock strength that was achieved in the successful UCS test (132 MPa) was utilised for rock mass classification purposes. A summary of the rock strengths determined for all other rock units for use in the classification of the rock mass are presented in Table 3-5. Overall these strengths are of a typical range for the Platreef area.

Table 3-5: Intact Rock Strength

	GN	N	GRV	AN	FPX	HF
Min UCS	232	161	231	253	127	236
Intercept	69547	49900	62147	62055	37829	74331
Mean UCS	**264**	**223**	**249**	**249**	**194**	**273**
Max UCS	286	260	282	285	226	306
Std. Dev	22	43	19	14	45	27
No. samples	16	13	18	20	15	12

3.7 Geotechnical Investigation Conclusions

From this chapter the following may be concluded:

- Geotechnical logging was undertaken to identify the geotechnical sub-surface conditions.
- Rock Units identified from the geotechnical logging include gabbronorite, norite, granite veins, anorthosite, feldspathic pyroxenite, hornfels, mafic pegmatite veins, orthopyroxenite.

- The lithologies identified within the vicinity of shaft 2 range in intact rock strength from 194 MPa to 273 MPa for the various lithologies. These strengths are essential as they form inputs into the rock mass classification, which is described in the next chapter.
- The geotechnical logs and the laboratory test results are essential for the rock mass classification of the rock within the vicinity of shaft 2.

Chapter 4 details the classification of the rock mass using Barton's Q-system, Laubscher's MRMR system and Hoek's GSI system.

4 ROCK MASS CLASSIFICATION

To classify the quality of the rock mass, use was made of three rock mass classification systems, viz. Laubscher's Mining Rock Mass Rating, Barton's Q and the quantification of GSI. The Q-System was adopted for the determination of support class recommendations for shaft 2 (section 5.2). Laubscher's RMR values were determined for the verification and validation of the Barton Q values derived for the various lithological units comprising the rock mass. The Geological Strength Index (GSI) was determined along the length of the borehole for the purposes of comparison of rock mass quality.

4.1 The Norwegian Geotechnical Institute's Q-System

Use was made of the Norwegian Geotechnical Institute's Q-System to determine the rock mass quality in the vicinity of shaft 2. These values were also required to provide recommendations for rock support of the shaft.

Rock Quality Designation (RQD)

As the conventional method of determining RQD is based purely on core pieces greater than 100mm, it is encumbered with several limitations. For example, "RQD = 0 where the joint spacing is 10cm or less, while RQD = 100 where the distance is 11cm or more" (Palmstrom, 2005). This is expressed in the sketch presented as Figure 4-1. Furthermore, it is found that an orientation bias exists. Figure 4-2 illustrates three extreme examples, "where RQD has values 0 and 100 for the same type and degree of jointing only due to the direction of the borehole" (Palmstrom, 2005).

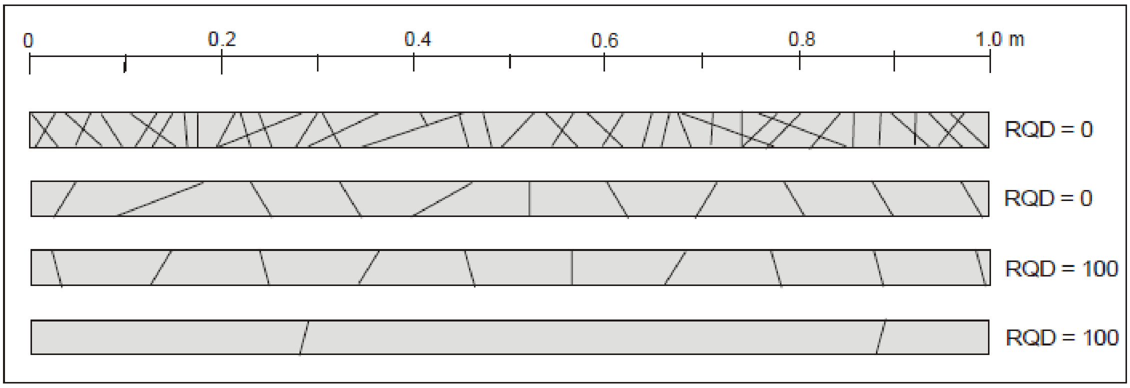


Figure 4-1: Limitations of the conventional method of determining RQD (Palmstrom, 2005)

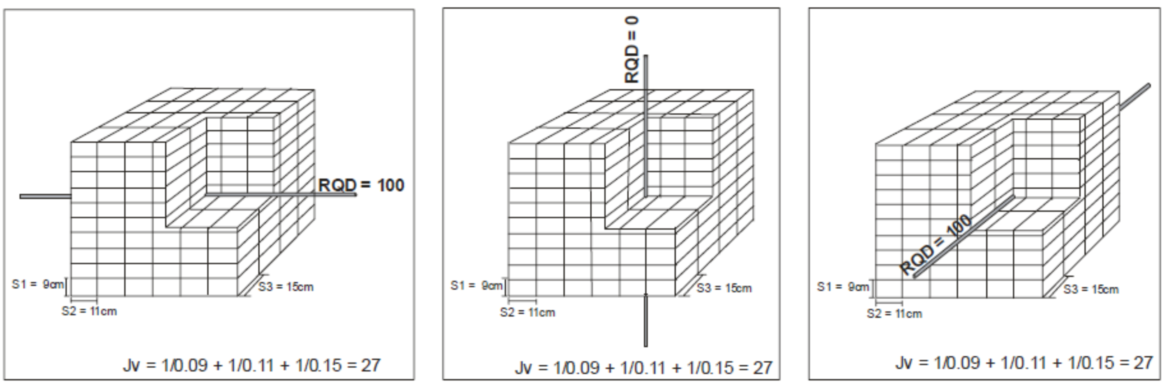


Figure 4-2: Borehole orientation bias (Palmstrom, 2005)

Due to these limitations, RQD for each geotechnical interval was calculated using a volumetric joint count through the application of Palmstrom's (1974) equation:

$$RQD = 115 - 3.3Jv$$

Where*: Jv* is the joint volume per m^3

$$Jv = \frac{FF}{m}\,(0° - 30°) + \frac{FF}{m}\,(30° - 60°) + \frac{FF}{m}\,(60° - 90°)$$

Where: *FF/m (0°-30°*) is the fracture frequency/m of alpha angles between 0-30° (Laubscher, 1990)

FF/m (30°-60°) is the fracture frequency/m of alpha angles between 30-60°

FF/m (60°-90°) is the fracture frequency/m of alpha angles between 60-90°

By use of the 3-dimensional volumetric joint count (Jv), a much better characterisation of the block size and thus the RQD is established (Palmstrom, 2005).

As Palmstroms approach to determine RQD does not take core loss and the length of matrix into account (Figure 4-3), a correction factor was included into the Palmstrom equation which reduces the RQD in intervals where core loss and matrix is present:

$$RQD = \frac{(Core\ Recovery - Length\ of\ Matrix)}{Interval\ Length} * 115 - 3.3Jv$$

A summary of the RQD values determined using Palmstrom's method along the depth of the borehole are presented in Figure 4-4.

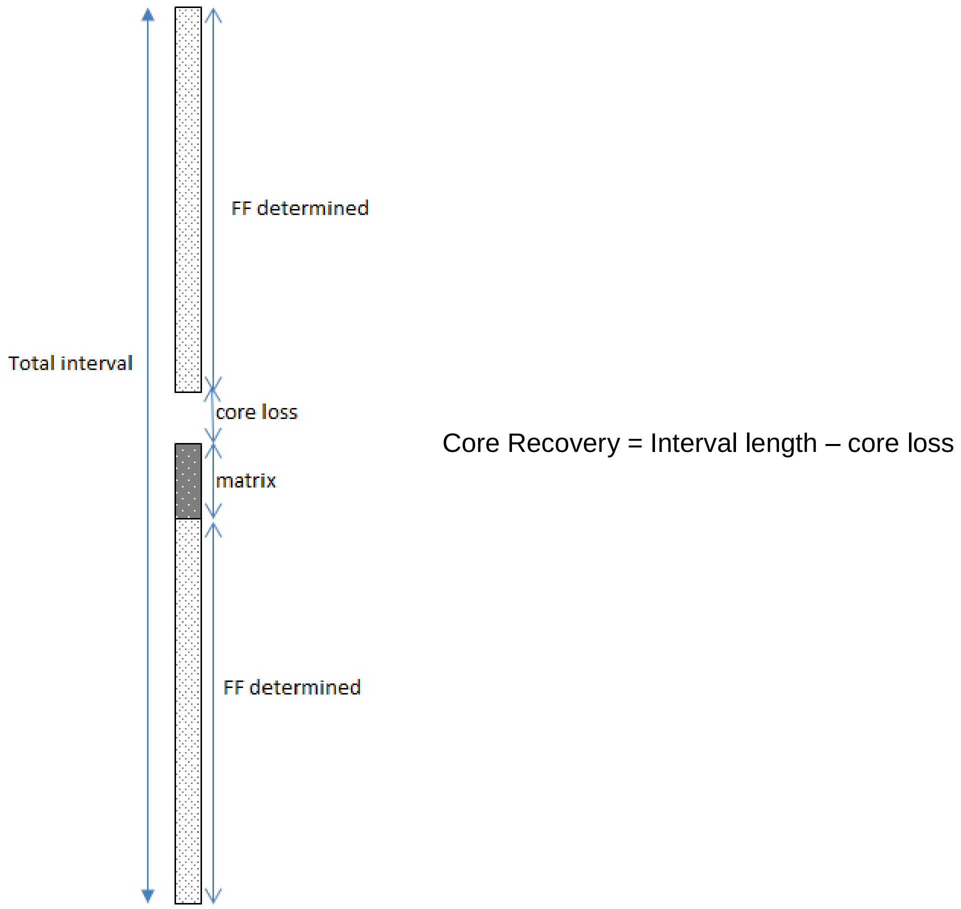


Figure 4-3: Geotechnical interval illustrating core loss and presence of matrix

From the RQD results calculated it was observed that the majority of low RQD values achieved were due to very small geotechnical intervals that were logged (<1 m).

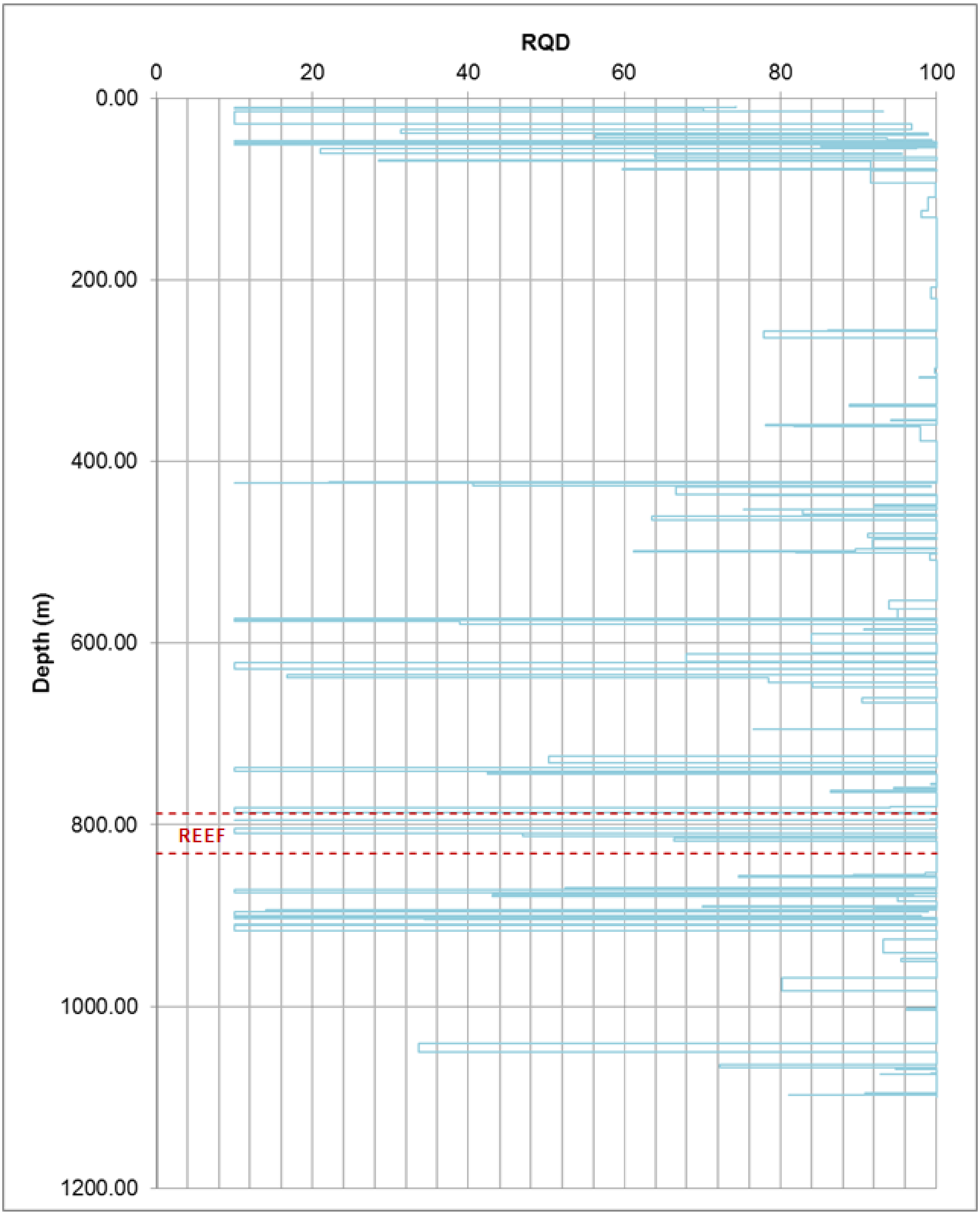


Figure 4-4: GT017 RQD Results

Jn, Jr, Ja and Jw Parameters

A summary of the values determined for Jn, Jr and Ja is presented from Table 4-1 to Table 4-3. Jr/Ja along the depth of the borehole is presented in Figure 4-6.

Table 4-1: Jn Results

Jn Factor	1	2	4	9
Number	114	129	78	38
Total	359	359	359	359
%	32%	36%	22%	11%

Table 4-2: Jr Results

Jr factor	0.5	1	1.5	2	3
Number	4	185	104	45	21
Total	359	359	359	359	359
%	1%	52%	29%	13%	6%

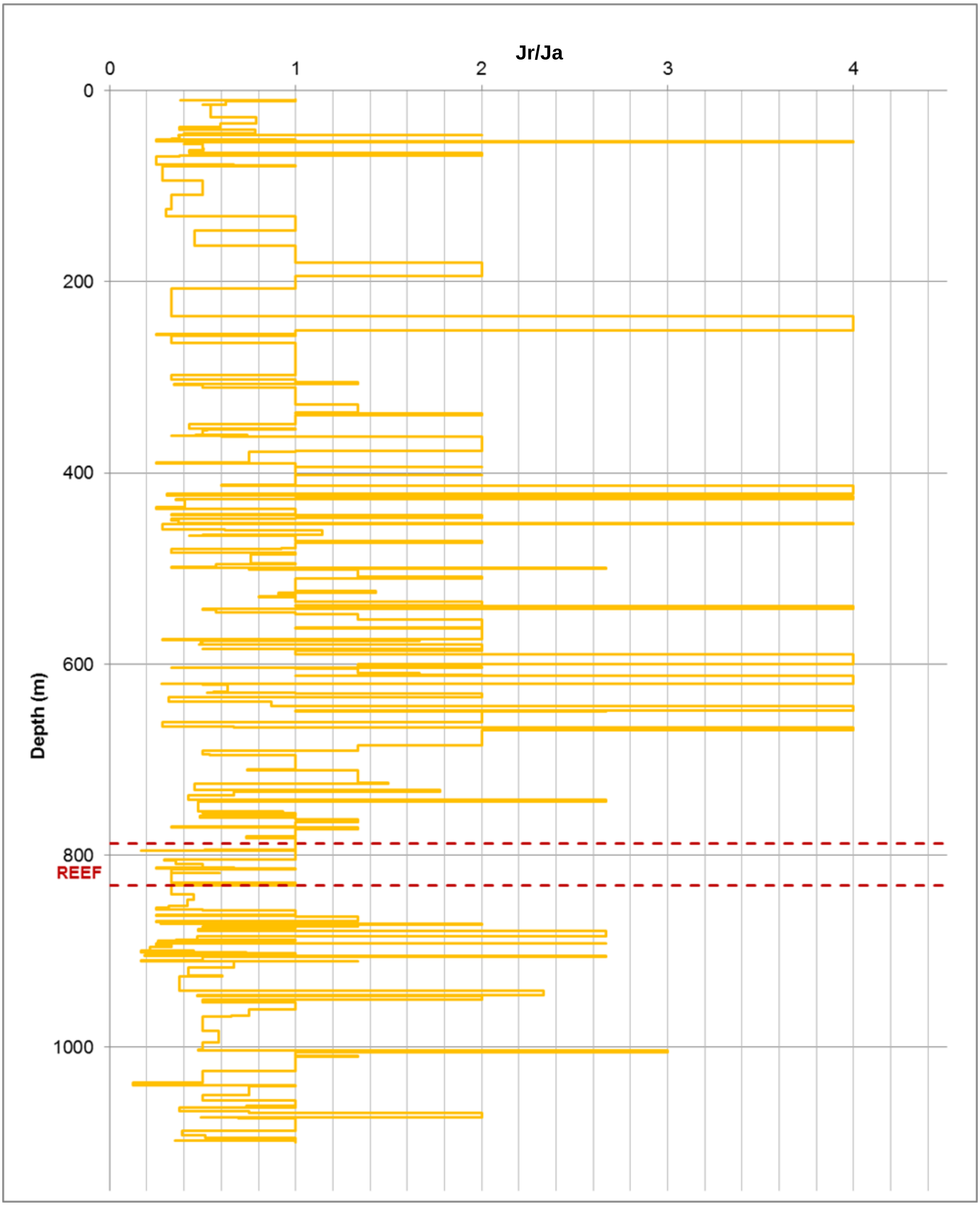
Table 4-3: Ja Results

Ja Factor	0.75	1	2	2.5	3	3.5	4	6.5
Number	66	116	16	15	55	37	53	1
Total	359	359	359	359	359	359	359	359
%	18%	32%	4%	4%	15%	10%	15%	0.3%

An example of a weak joint is presented in Figure 4-5. As the joint is both a slickensided joint and contains chlorite infill it would have a Jr rating of 0.5 and a Ja rating of 4.

Figure 4-5: Slickensided undulating joint surface

Figure 4-6: GT017 Jr/Ja Results



Jw and SRF values were determined using Barton's (2002) tables. Jw values were chosen based on the presence of groundwater that was noted in the GT017 geotechnical log. From the log it was observed that there is ground water present near surface (<50 m deep). A summary of the Jw factors determined is presented in Table 4-4.

Table 4-4: Jw Results

Jw Factor	**0.66**	**1**
Description	Wet	Damp/Dry
Number	5	354
Total	359	359
%	1%	99%

SRF Parameter

SRF values were chosen for each geotechnical interval to facilitate the determination of Q values along the length of the borehole. This was done by employing the use of Barton's (2002) SRF tables. For geotechnical intervals that comprised of competent rock, the SRF factor chosen was based on the value resulting from the following relationship:

$$\boldsymbol{UCS/\delta 1}$$

Where: *UCS* is the mean intact strength of the rock unit

δ1 is the vertical principal stress

UCS values were obtained from the mean intact rock strengths derived from laboratory testing for the various rock types that intersected the shaft 2 borehole (Table 3-5).

Vertical principal stress was determined based assuming a k ratio of 1 using the following equation:

$$\boldsymbol{\delta 1 = \rho g h}$$

Where: ρ is the rock density

g is the gravitational constant (9.81 m/s^2)

h is the depth

As horizontal stresses are of a greater importance when dealing with a shaft, the k ratio was increased to a value of 1.5 to consider the effect this would have on the Q-values. It was found that the increase in k-ratio increases the horizontal stresses and thus lowers the Q-values; however as the rock units are strong (from 194MPa to 2644MPa) this does not result in a change in Q-class.

For geotechnical intervals that contain weaknesses (ie. matrix) the SRF value assigned was based on Barton's (2002) SRF table for weakness zones. To ensure that weaknesses were well taken into account, a more conservative approach was adopted by assigning a higher SRF value to matrix present that is greater than 10 cm (SRF value of 5.0) and 50 cm (SRF value of 7.5) in length.

A summary of the SRF values determined for GT017 is presented in Table 4-5.

Table 4-5: SRF Results

SRF Factor	**1**	**1.25**	**2.5**	**5**	**7.5**
Number	221	102	21	13	2
Total	359	359	359	359	359
%	62%	28%	6%	4%	1%

The values determined for SRF along the depth of the GT017 is presented in Figure 4-7.

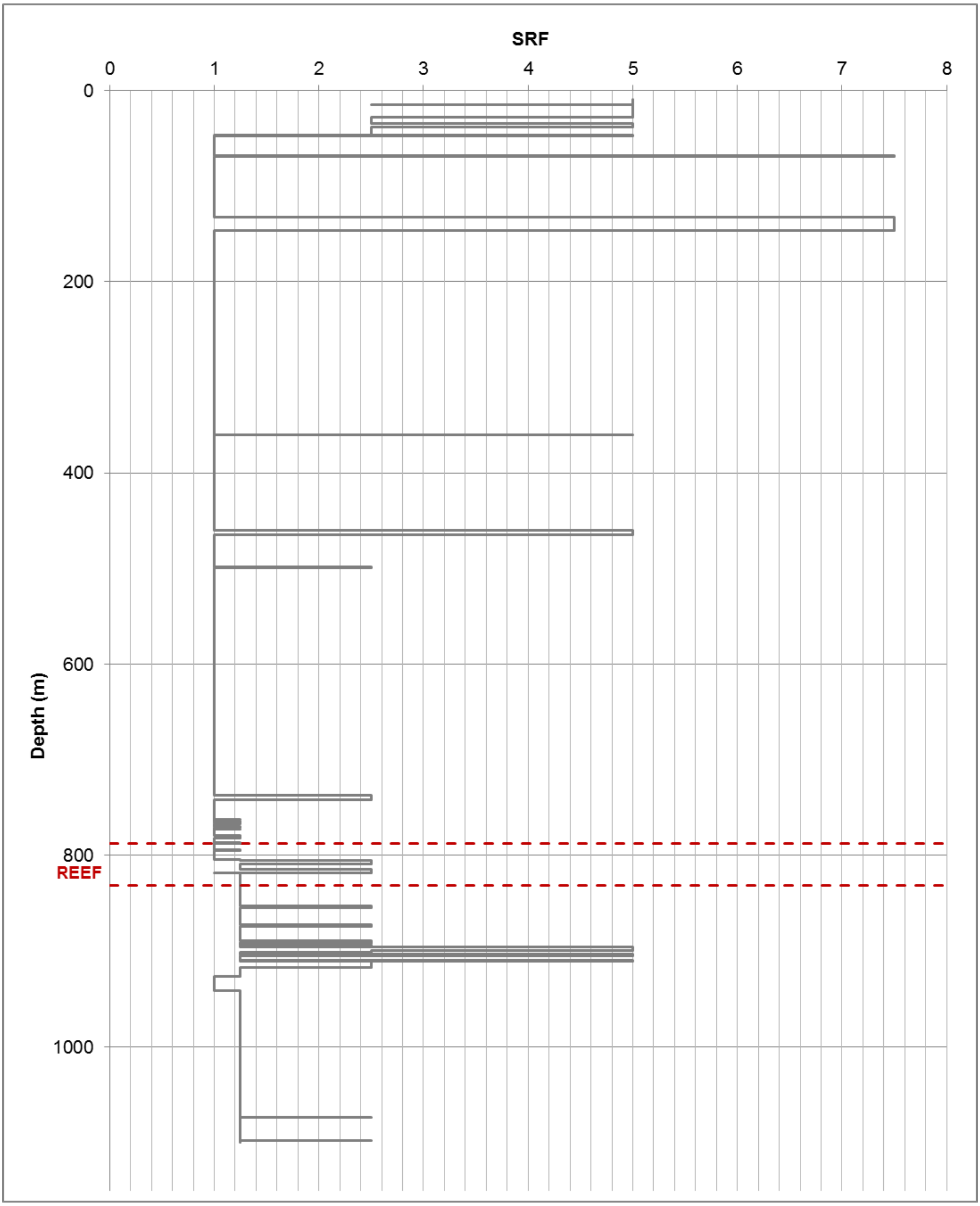


Figure 4-7: GT017 SRF Results

Adjustment to Q_{wall}

Given that the Q-System is primarily concerned with the stability of tunnel roofs, an adjustment factor was applied to the resultant Q-values in order to account for the sidewalls of the vertical shaft. The following adjustment (Barton et al, 1974) was thus applied to the Q values:

$$\boldsymbol{Qwall = 5Q} \quad \text{Where: } Q > 10$$

$$\boldsymbol{Qwall = 2.5Q} \quad \text{Where: } 0.1 < Q < 10$$

$$\boldsymbol{Qwall = Q} \quad \text{Where: } Q < 0.1$$

Qwall values determined for GT017 are presented in Figure 4-8.

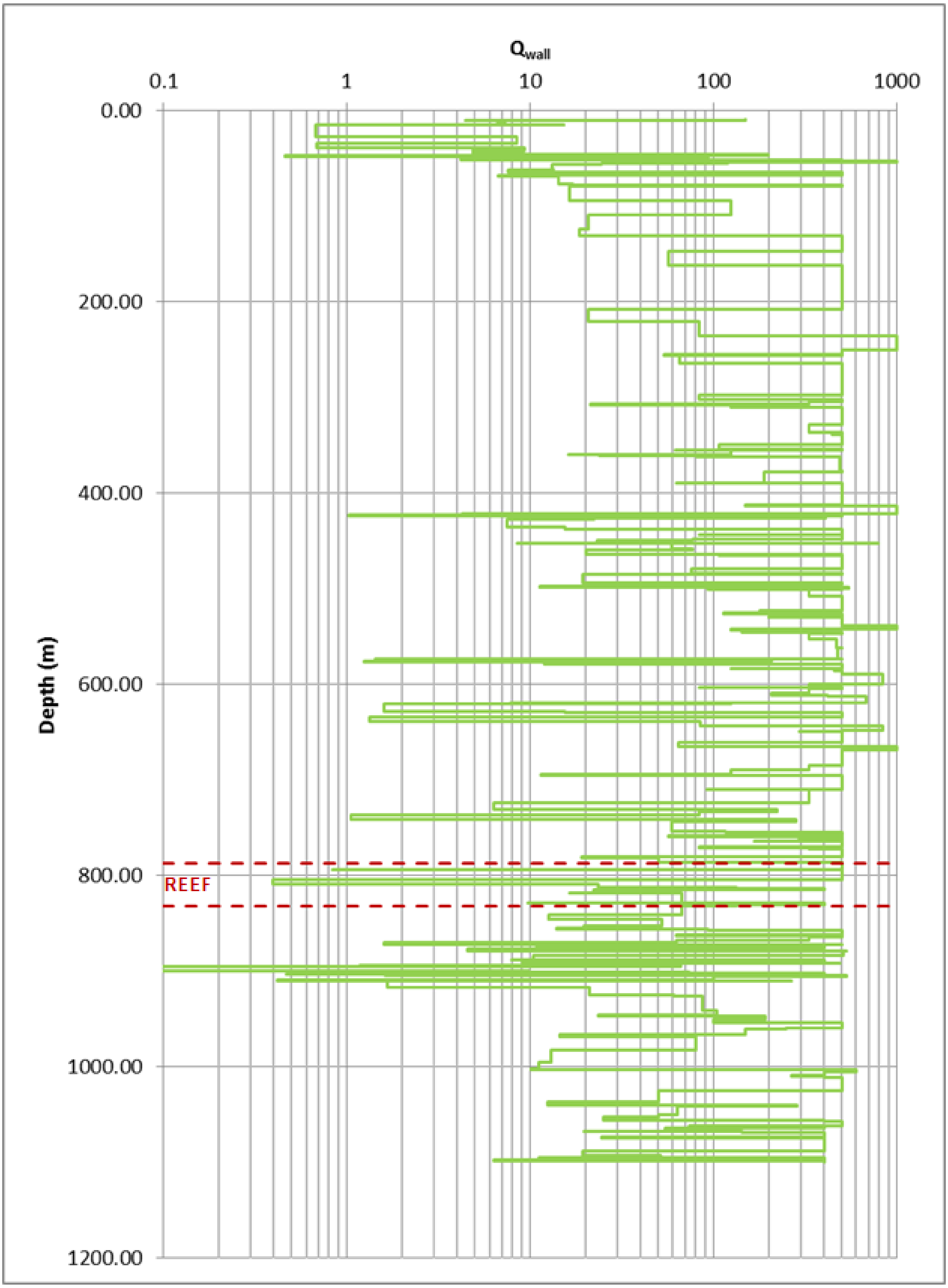


Figure 4-8: GT017 Q_{wall} Results

Considering the Norwegian Geotechnical Institute's Q system, with the Q_{wall} adjustment, the quality of the rock mass is classified as "Good" to "Exceptionally Good" (91 % of the borehole). There are small zones which are classified as “Fair” (7 % of the borehole), while 2 % of the borehole is classified as “Very Poor” to “Poor”.

4.2 Laubscher’s (1990) Rock Mass Rating Classification

Laubscher’s (1990) Mining Rock Mass Rating Classification System evaluates discrete geotechnical domains based on strength (Intact Rock Strength, IRS), fracture frequency, joint condition and weathering characteristics. Each of the resultant domains is evaluated separately, through the allocation of rating values, within a specific range, for each parameter.

Fracture Frequency

Fracture frequency (FF/m) is defined as the number of fractures intersecting an interval divided by the length of that interval. The fracture frequency for GT017 was measured for joints in sets between 0 to 30 degrees, 30 to 60 degrees and 60 to 90 degrees to the core axis. As the FF/m does not recognise core recovery, the FF/m should be increased where core loss is encountered (Laubscher, 1990). Furthermore, FF/m should be increased where matrix is present. In order to account for core loss and matrix, an adjustment was applied for GT017 where the FF/m was divided by the solid core recovery and multiplied by 100.

As the GT017 borehole does not intersect all joint sets at 90 degrees, apparent spacing (and apparent FF/m) per set is recorded rather than the true spacing. To determine the true FF/m for each set the following equations were applied (as per Laubscher, 1990):

$$\boldsymbol{True\frac{FF}{m}\ (0° - 30°) = \frac{FF}{m}(0° - 30°)/Sin15°}$$

$$\boldsymbol{True\frac{FF}{m}\ (30° - 60°) = \frac{FF}{m}(30° - 60°)/Sin45°}$$

$$\boldsymbol{True\frac{FF}{m}\ (60° - 90°) = \frac{FF}{m}(60° - 90°)/Sin75°}$$

The FF/m with depth for GT017 is presented in Figure 4-9. From the fracture frequencies determined, it was observed that many of the very high FF/m values calculated (>40) are due to small geotechnical intervals that were logged (<1 m) along the borehole. Overall, there is a larger amount of fracturing present in the footwall compared to the hangingwall.

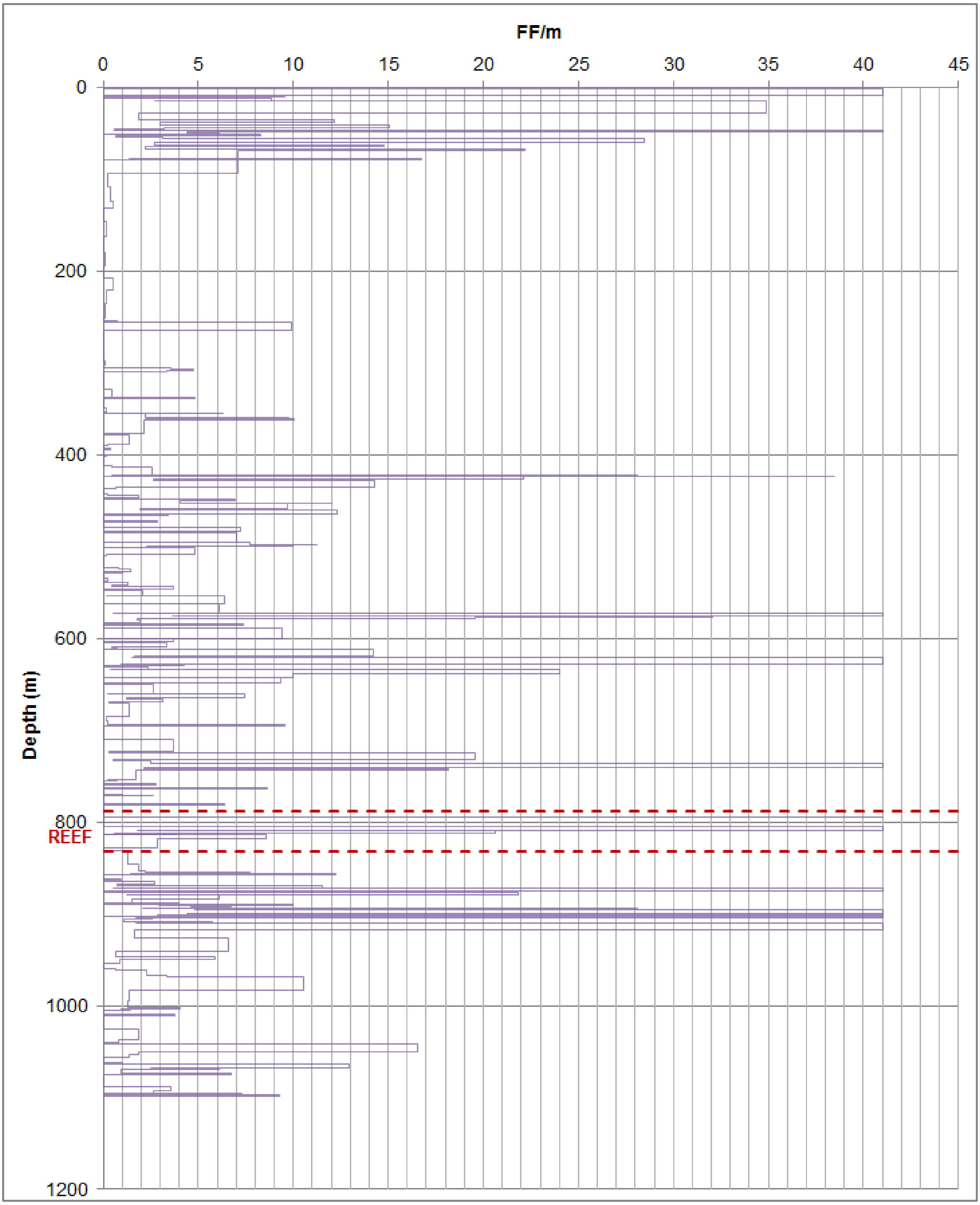


Figure 4-9: GT017 Fracture Frequency Results

RMR Results

The RMR values determined along the depth of the borehole are presented in Figure 4-10.

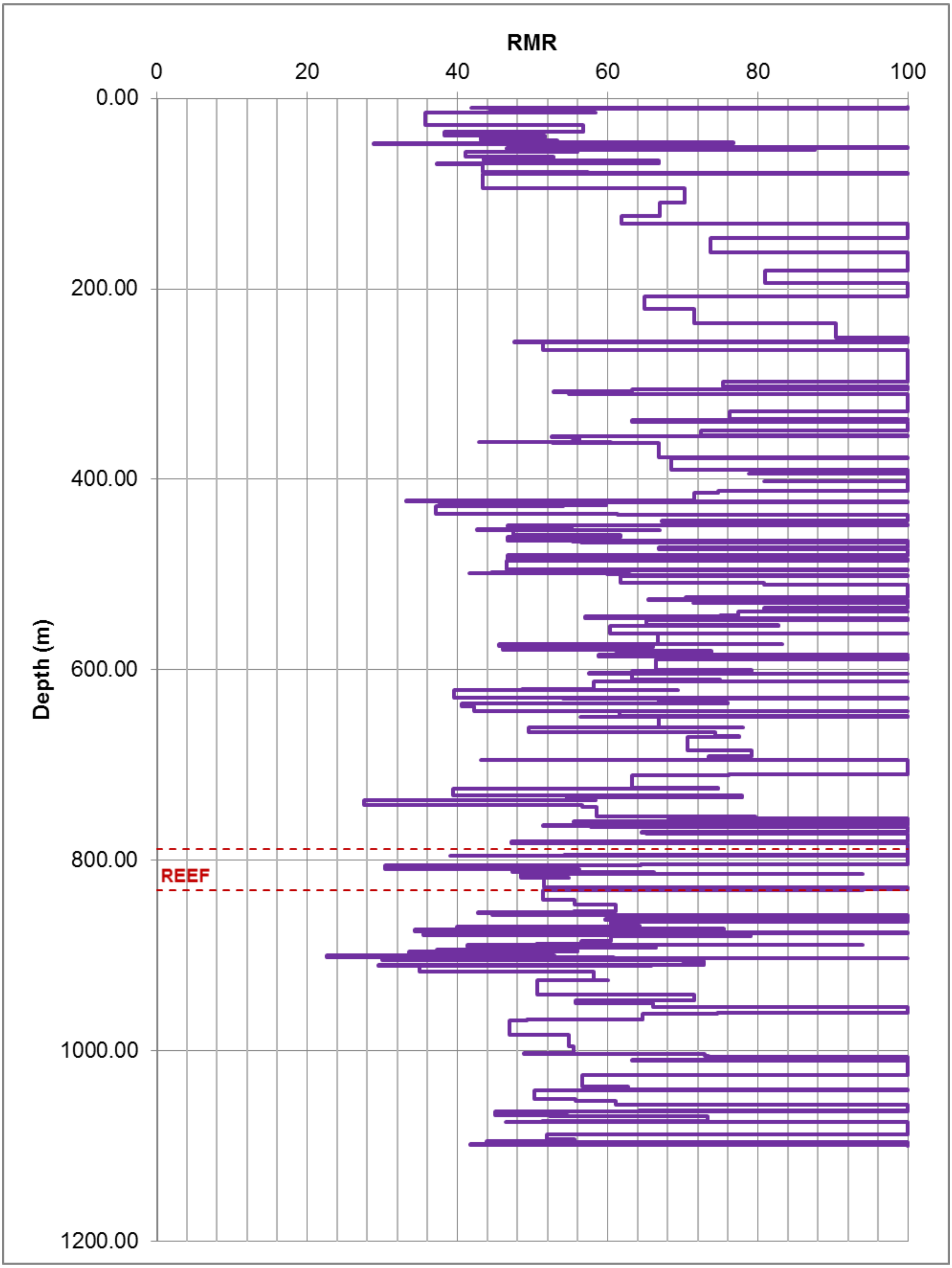


Figure 4-10: GT017 RMR Results

From the RMR results determined the rock mass quality in the vicinity of shaft 2 may be classified as "good" rock.

4.3 Geological Strength Index (GSI)

GSI values for the hangingwall, orebody and footwall zones were calculated to determine rock mass properties for comparison. The following expression was used to calculate GSI (Hoek, 2012) from the raw Q input parameters:

$$\boldsymbol{GSI = \frac{52\frac{Jr}{Ja}}{1+\frac{Jr}{Ja}} + RQD/2}$$

This was calculated for each geotechnical interval. The GSI values determined along the length of the borehole is presented in Figure 4-11.

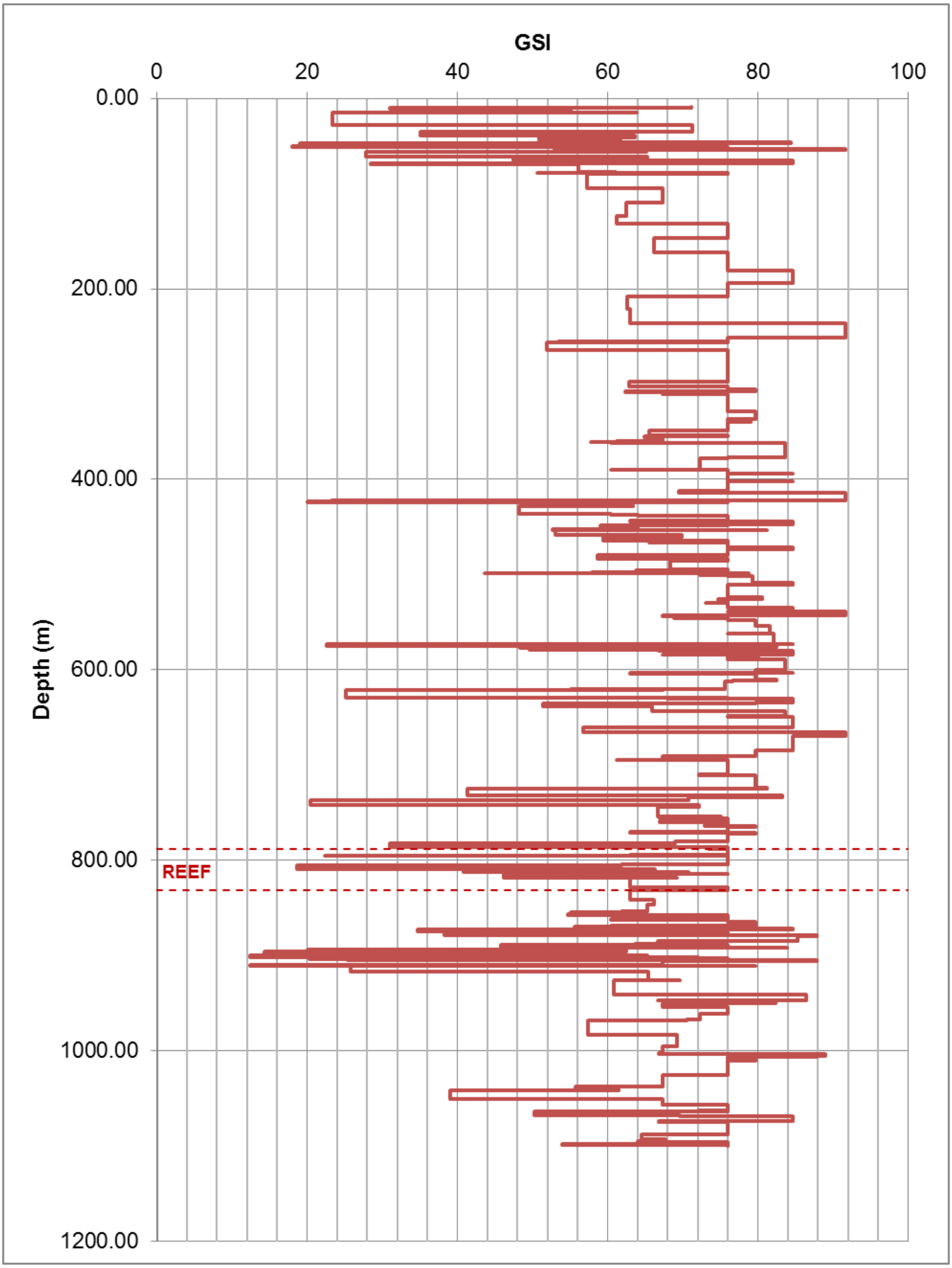


Figure 4-11: GT017 GSI Results

4.4 Comparison of Laubscher's RMR, Barton's RMR from Q_{wall} and the GSI System

As Q_{wall} and Q-values are expressed on a log scale, these values are challenging to compare to Laubscher's RMR and Hoek's GSI values. Q_{wall} and Q-values were thus converted to Barton's Rock Mass Rating (RMR) for the purpose of statistical analysis. The relationship used for the conversion from Q/Q_{wall} to RMR is as follows:

$$\boldsymbol{RMR = 15 logQ + 44}$$

$$\boldsymbol{RMR = 15 logQwall + 44}$$

The comparison of the mean values for Laubscher's RMR, Q, Q_{wall} and GSI for the hangingwall, orebody and footwall is presented in Figure 4-12. From the comparison it is evident that the results are similar, being within 10 points. Overall it is observed that the rock mass quality is higher in the hangingwall compared to that of the orebody and footwall. Furthermore it is evident that the RMR values determined from Q_{wall} are higher than that of Q, GSi and Laubscher's RMR values. This is expected due to the adjustments made to Q for the vertical walls of the shaft. Taking into account each rock mass classification system, together the rock mass may be classified as "good" rock.

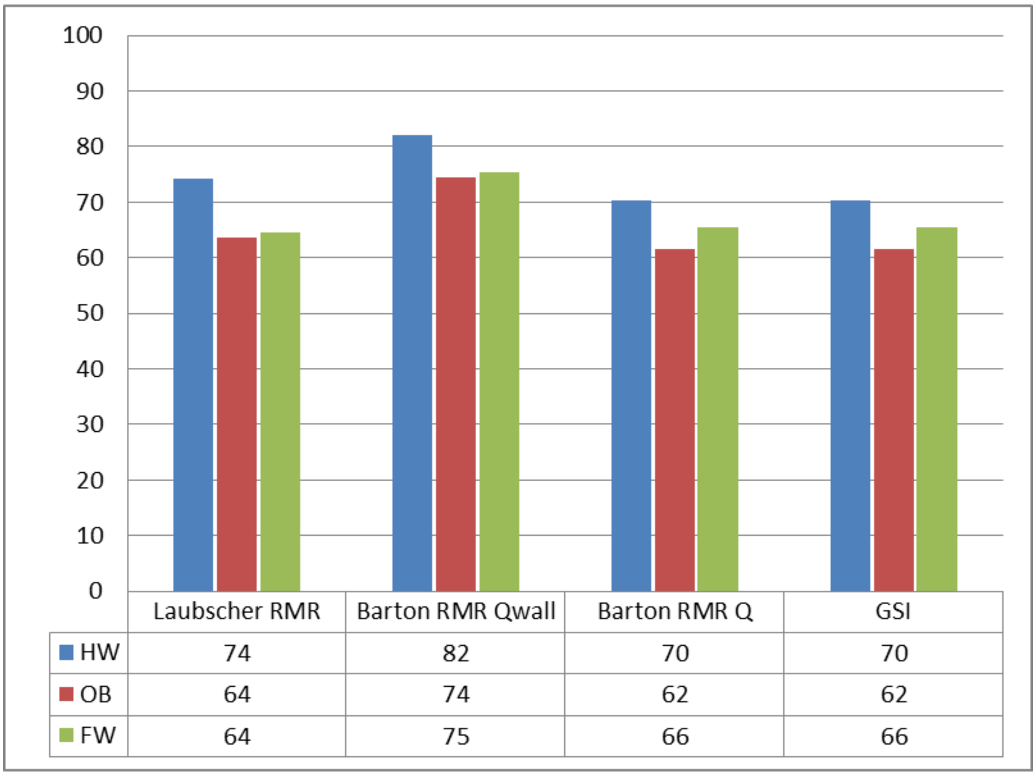


Figure 4-12: Comparison of Rock Mass Classification results

From this chapter use was made of three rock mass classification systems to determine the quality of the rock mass at Platreef Resources within the vicinity of shaft 2. From the comparison of the results from each system it was observed that the systems compare well, and that overall the rock mass may be described as "good". Following this chapter, a structural analysis is presented in chapter 5, which highlights the joint sets and their orientations within the rock mass, and the potential modes of failure that may occur.

5 STRUCTURAL ANALYSIS

A structural analysis is undertaken in this chapter, which highlights the joint sets and their orientations within the rock mass, and the potential modes of failure that may occur in the shaft.

Based on the joints identified from geotechnical logging, and wireline data received from Platreef Resources, a structural analysis was carried out for borehole GT017. Joints along the depth of the borehole are presented in Figure 5-1, where set 1, set 2 and set 3 refer to joints with alpha angles between 0 to 30°, 30 to 60° and 60 to 90° respectively. As set 1 occurs parallel to sub-parallel to the vertical orientation of the borehole, this set was not very apparent along most of the hole. A correction for the orientation of the sets was thus applied to achieve a more accurate distribution of the joints along the borehole.

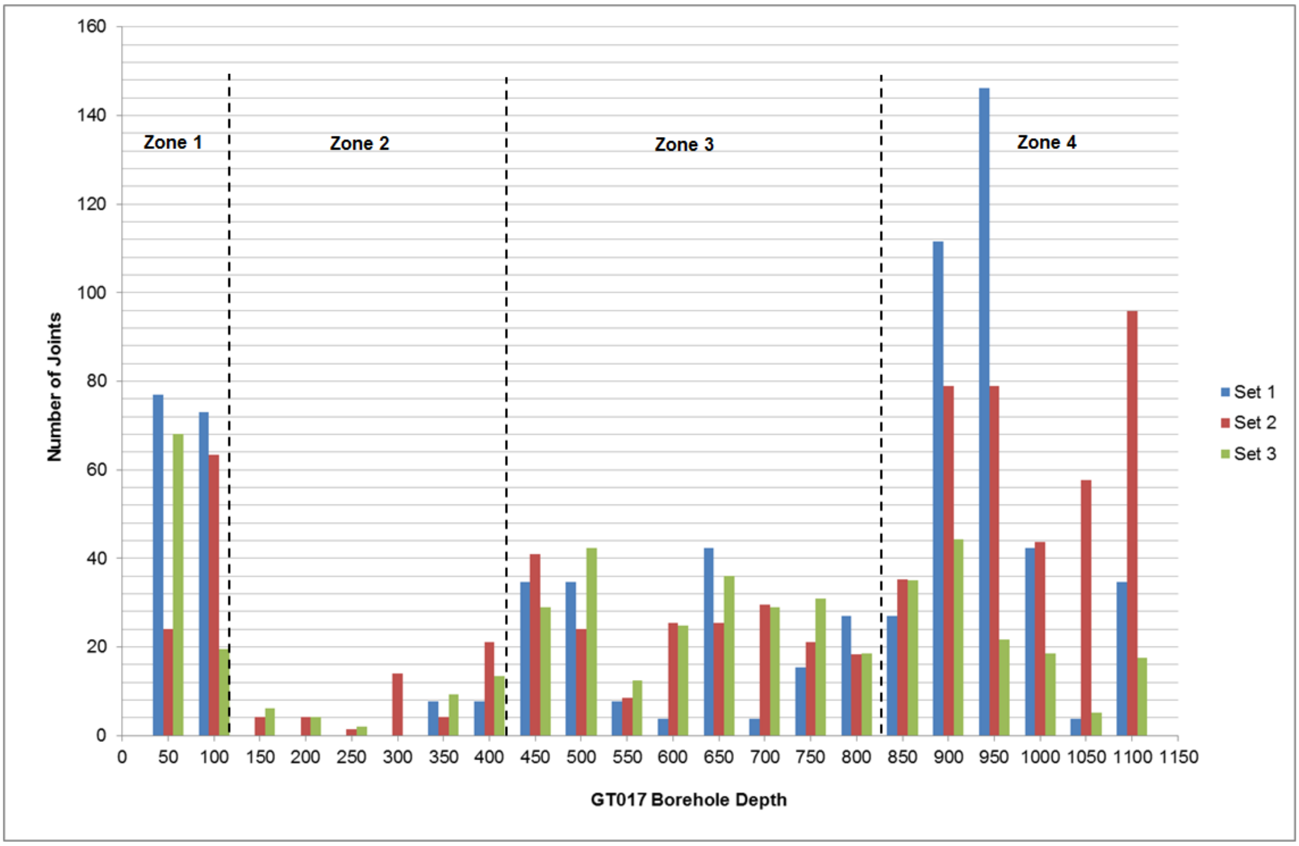


Figure 5-1: Joint data analysis

From the distribution of joint data present in GT017, it was observed that the concentration of jointing along the depth of the borehole varies. The data was thus divided into 4 structural zones based on joint concentration. A high concentration of joints is present near surface (zone 1) , which is followed by a low concentration in jointing from 100 m in depth to approximately 400 m in depth (zone 2). Jointing increases from 400 m to 831 m (zone 3), with the highest concentration in jointing present in the footwall (zone 4).

In terms of the distribution of joint sets, it was observed that set 1 and set 2 are most prominent in zone 4. Set 1 to set 3 are distributed somewhat evenly over zones 1 and 3, with set 1 less apparent is zone 2.

Once the depth along the borehole was divided into the various structural zones, ATV data received from Platreef was used in the Rocscience software DIPS to produce stereographic projections for the analysis of joint sets. The stereographic projections created for each zone are presented from Figure 5-2 to Figure 5-5.

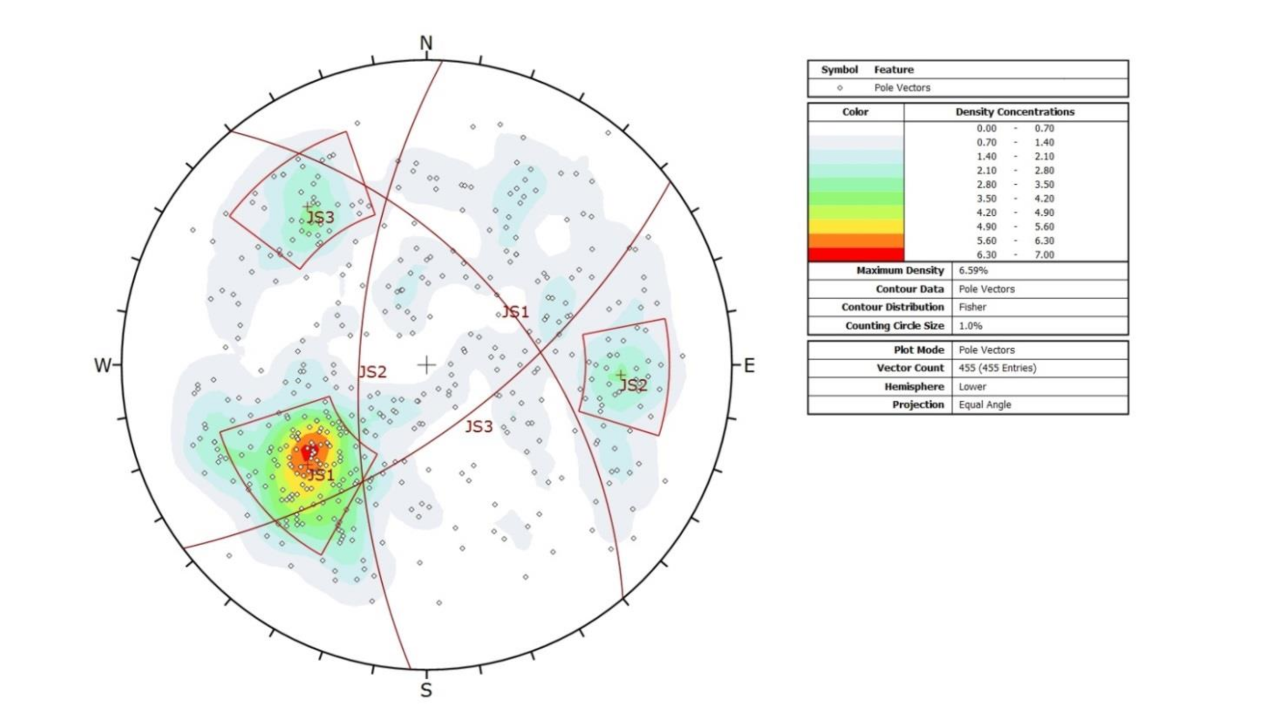


Figure 5-2: Structural Zone 1

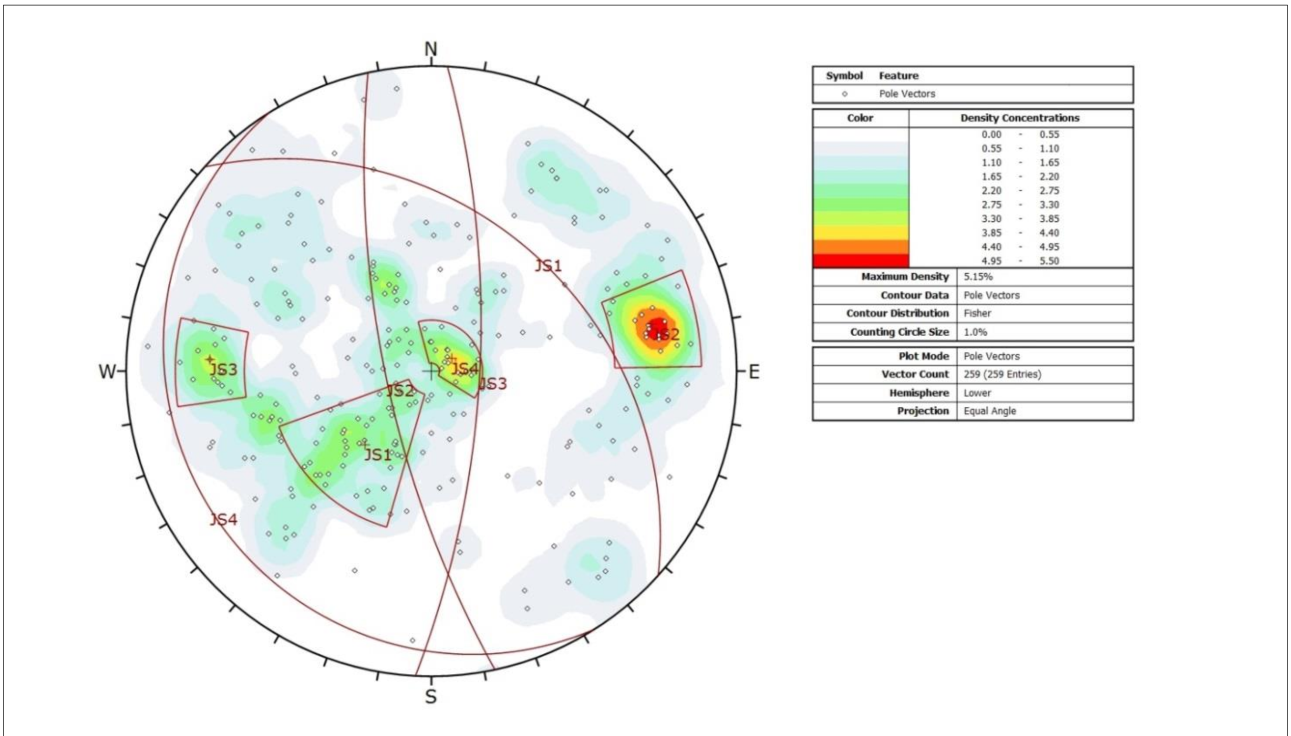


Figure 5-3: Structural Zone 2

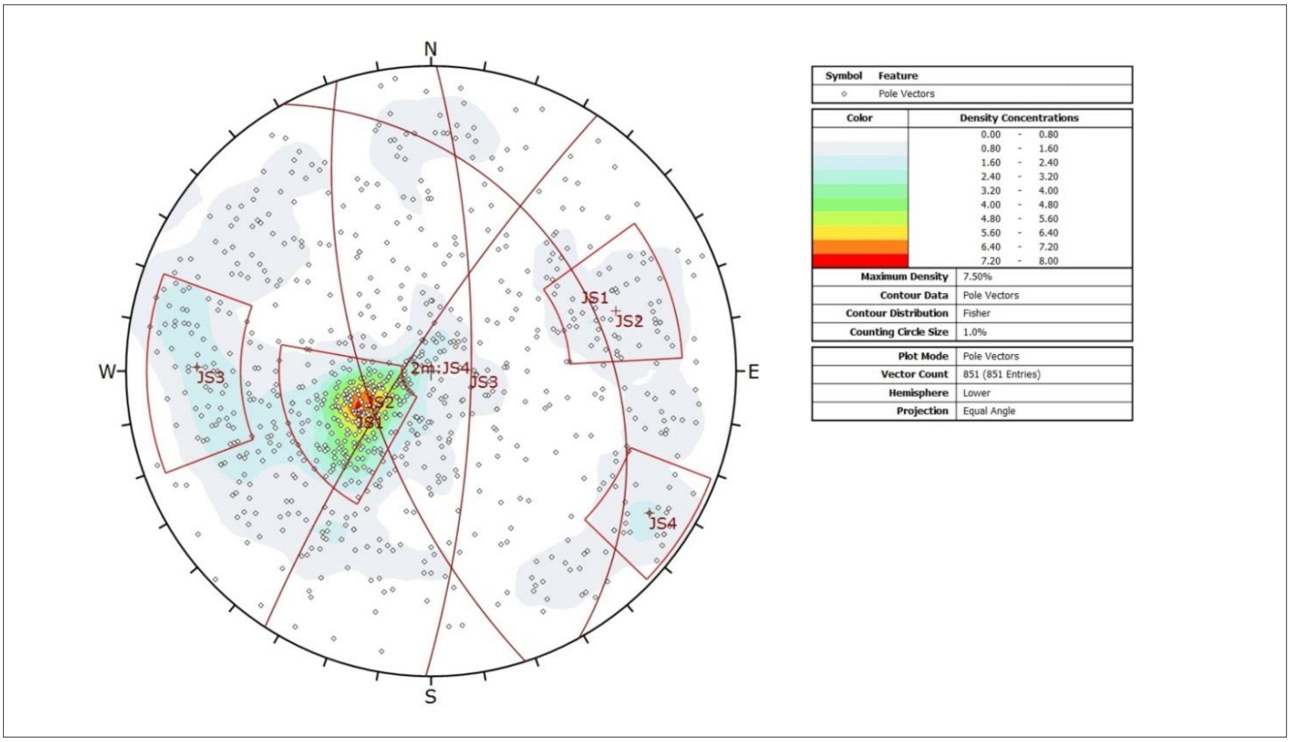


Figure 5-4: Structural Zone 3

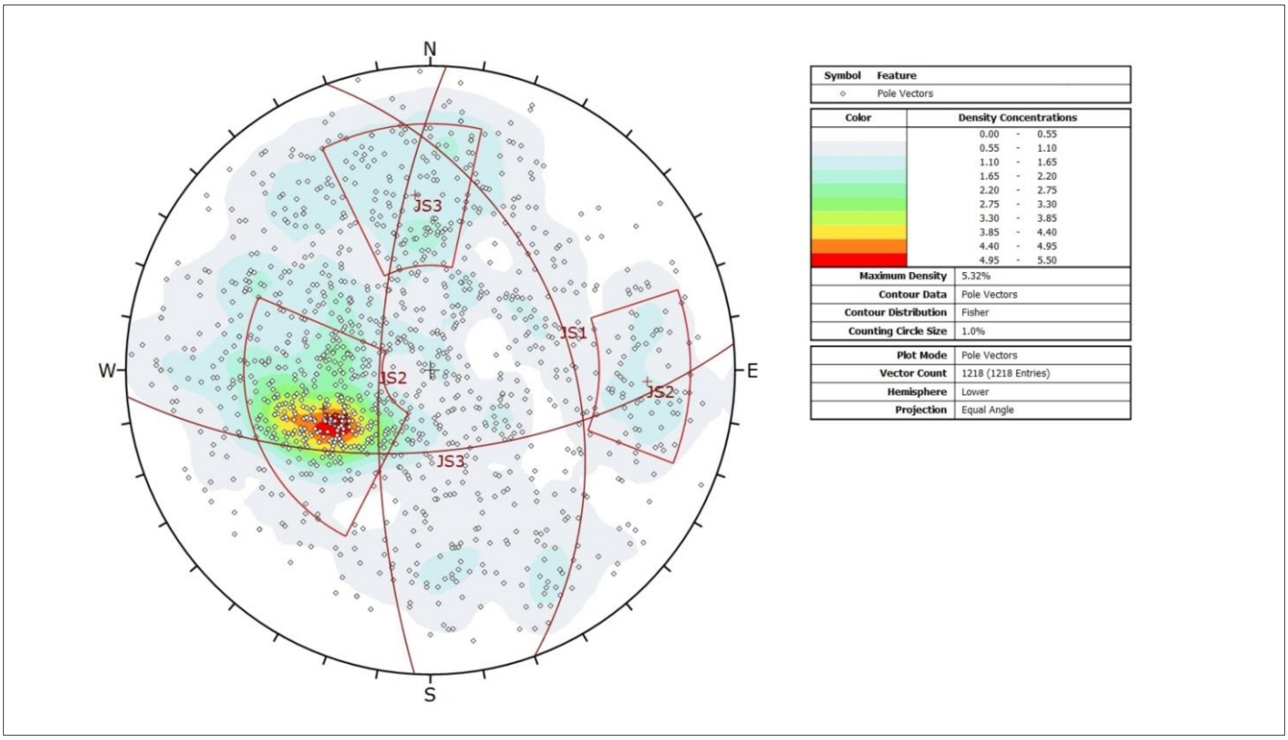


Figure 5-5: Structural Zone 4

From the stereographic projections it is evident that while joint sets can be identified from the concentration of the joints present; there is also scatter in the data which is not unexpected when dealing with structural orientations obtained from drilling.

Joint sets identified for each structural domain are presented in Table 5-1.

Table 5-1: GT017 Joint Orientations

	Mean Joint Orientations (Dip/Dip Direction)			
Zone	**Joint Set 1**	**Joint Set 2**	**Joint Set 3**	**Joint Set 4**
1	54/050	65/273	66/143	NA
2	36/042	73/258	72/093	09/238
3	31/063	65/252	75/091	81/303
4	41/070	71/273	60/175	NA

True joint spacing has been calculated for each joint set identified (Table 5-2). This was determined using the apparent spacing of the joints, the mean dip of each joint set and the angle of intersection of each borehole (Figure 5-6).

Table 5-2: True Joint Spacing Summary

	Joint Set 1			Joint Set 2			Joint Set 3			Joint Set 4		
Zone	**min**	**mean**	**max**	**min**	**mean**	**max**	**min**	**mean**	**max**	**min**	**mean**	**max**
1	0	**0.2**	5.2	0	**0.34**	3.54	0.01	**0.51**	2.73	NA	NA	NA
2	0.01	**4.51**	44.11	0	**3.41**	16.44	0	**3.46**	33.52	0.19	**21.06**	76.39
3	0.01	**1.62**	25.23	0.22	**7.03**	46.46	0.01	**2.08**	14.34	0	**0.94**	6.84
4	0	**0.54**	6.25	0.01	**0.63**	4.78	0.02	**1.99**	14.81	NA	NA	NA

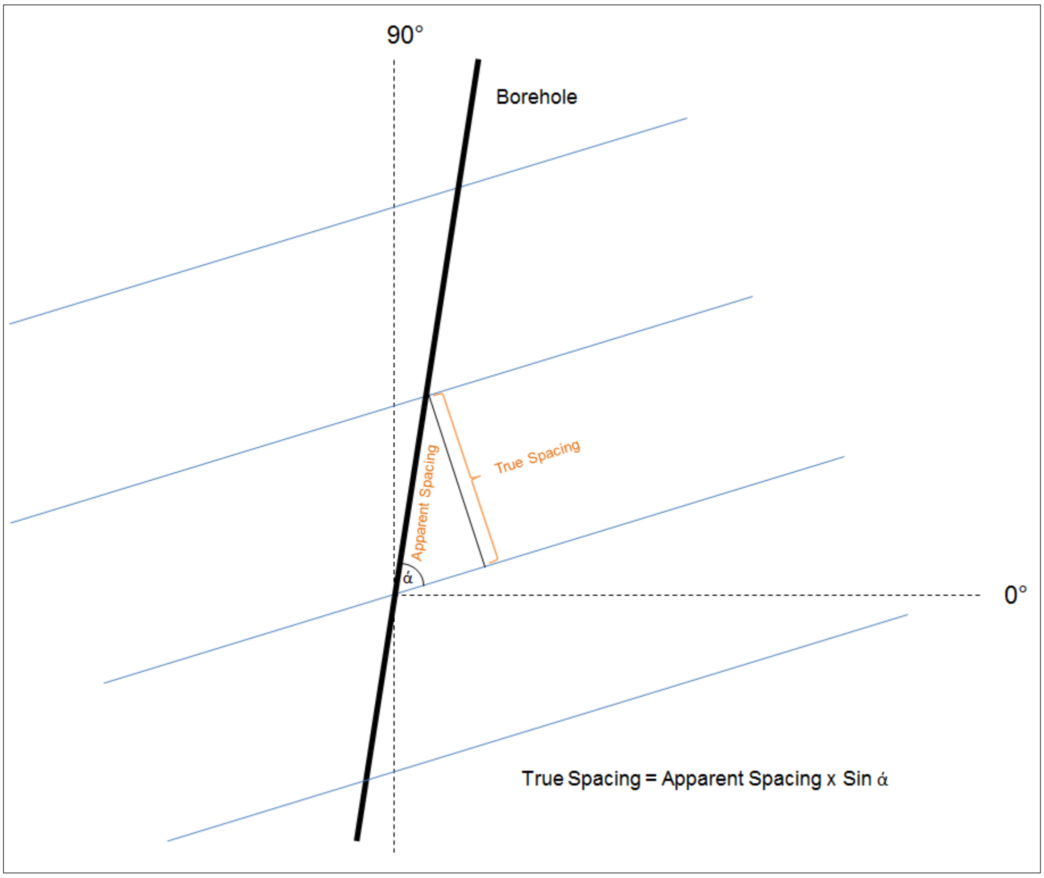


Figure 5-6: True Joint Spacing

As additional joint statistics are required for the JBlock analysis, the standard deviation for joint orientations and the friction angle per structural zone was determined for GT017 (Table 5-3 and Table 5-4). Friction angles were calculated using Barton's (2002) equation:

$$\varphi = \mathbf{arctan}\left(\frac{\boldsymbol{Jr}}{\boldsymbol{Ja}}\right)$$

Where: φ s the friction angle

Jr is the joint roughness

Ja is the joint alteration

"The dual reinforcement of the 'cohesive component' and 'frictional component' concept considers that the ratio Jr/Ja closely resembles the dilatant or contractile coefficient of friction for joints and filled discontinuities" (Barton, 2002). Barton's equation was thus chosen to determine friction angle as filled discontinuities were identified in the geotechnical log.

Table 5-3: Joint Set Standard Deviations

Zone	Joint Set 1		Joint Set 2		Joint Set 3		Joint Set 4	
	Dip	Dip Direction	Dip	Dip Direction	Dip	Dip Direction	Dip	Dip Direction
	Std. dev.	Std. dev.	Std. dev.	Std. dev.	Std. dev.	Std. dev.	Std. dev.	Std. dev.
1	9	11	7	9	6	7	NA	NA
2	13	15	4	38	5	5	4	6
3	11	16	5	5	9	8	6	12
4	11	21	12	11	7	11	NA	NA

Table 5-4: Friction Angle Summary

Zone	Mean Friction angle	Standard Deviation
1	30	13
2	38	13
3	39	13
4	33	14

5.1 Unwedge Analysis

The RocScience tool UNWEDGE was applied for Zones 1 to 4 to evaluate the maximum wedge size that can be created in a 10.0 m diameter shaft, with the associated factor of safety. A factor of safety (FoS) less than 1.0 implies an unstable wedge susceptible to failure (sliding into the shaft). The analysis was based on a worst case scenario where no support is used in the shaft.

Figure 5-7 illustrates the possible wedge geometry and orientation for six possible wedges for Zone 1. Table 5-5 presents the results for the lowest factors of safety.

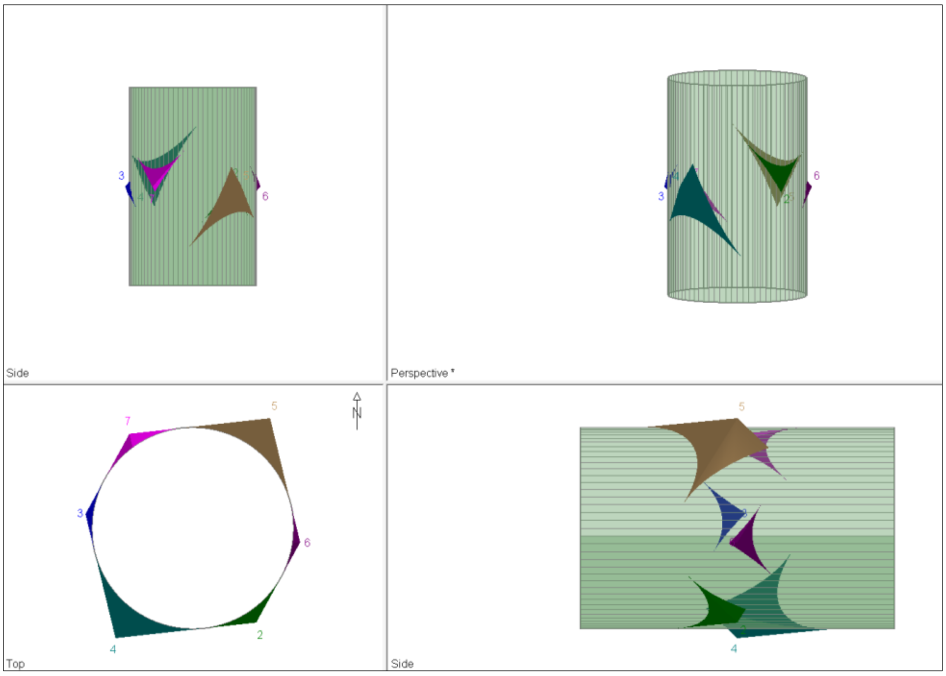


Figure 5-7: Possible Wedge Geometry - Zone 1

Table 5-5: Unwedge Analysis - Worst Case Results

Zone	Wedge ID	Factor of Safety	Volume (m^3)	Weight	Apex Height (m)	Resisting Force (MN)	Driving Force (MN)
Zone 1	**South West**	**6.6**	4.51	0.14	1.66	0.72	0.11
Zone 2	**South**	**7.1**	186	5.58	12.36	29.32	2.13
Zone 3	**North West**	**11.5**	277	8.30	17.22	39.10	3.40
Zone 4	**North**	**9.1**	2.71	0.08	1.38	0.64	0.07

From the analysis it can be concluded that the factor of safety does not drop below 1 for Zones 1 to 4 and that it is unlikely that the shaft will encounter any major wedge failure.

5.2 Kinematic Analysis

To determine the probability of wedge failure in the shaft a kinematic analysis was undertaken for Zones 1 to 4 of the shaft. The possibility of wedge failure was undertaken for the north, south, east and west sides of the shaft and are presented in Table 5-6.

Table 5-6: Probability of Wedge Failure

Zone	North	South	East	West
Zone 1	27	32	29	35
Zone 2	22	22	21	30
Zone 3	19	16	14	26
Zone 4	20	18	12	31

From the analysis undertaken it was observed that Zone 1 has the highest probability of failure, with failure most likely to occur on the western side of the shaft (Figure 5-8).

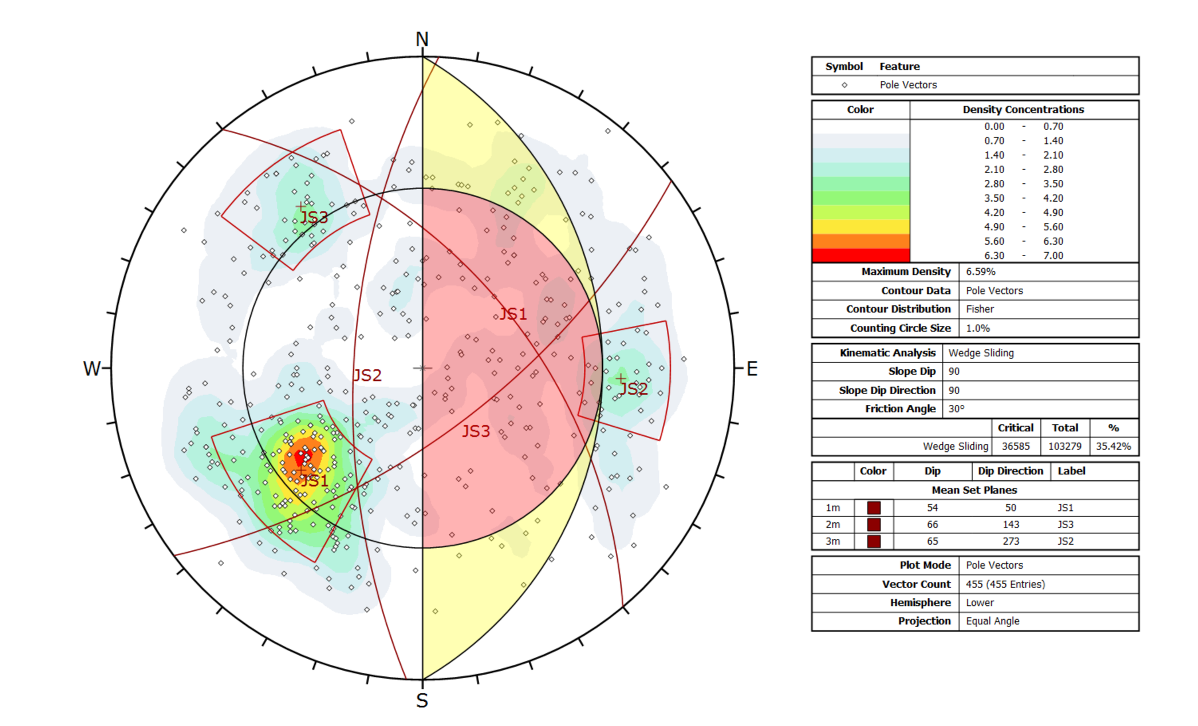


Figure 5-8: Possibility of Wedge Failure in Zone 1

From the structural analysis undertaken it may be concluded that the rock mass contains 4 structural zones:

- Zone 1 (0-100m)
- Zone 2 (100-400m)
- Zone 3 (400-831m)
- Zone 4 (831-EOH)

Each structural zone has a varying degree of jointing, with different joint sets. Overall there are three to four joint sets identified in the rock mass which could lead to wedge instability. The following chapter takes this into consideration along with the rock mass classification determined for the rock mass, to provide rock support recommendations for the vertical shaft.

6 SHAFT SUPPORT RECOMMENDATIONS

The design of rock support for the shaft was based on Barton's Q support chart (2002) as illustrated in **Error! Reference source not found.**. This chart is a robust design tool developed based on case histories of tunnels, shafts and caverns collated over the years and accommodates advances in support technology. The support recommendations obtained from the chart are based on different combinations of rock quality (Q values) and Equivalent Span, which is the ratio of the span to the Excavation Support Ratio (ESR).

The ESR is a factor used by Barton to account for different degrees of allowable instability based on excavation service life and usage. Following Barton's ESR recommendations for underground openings (1974), ESR=2.5 was utilised for the primary support of the shaft, which gives a ratio of 2.4 considering a shaft diameter of 10 m. An ESR of 1.2 can be taken for the final support of the shaft.

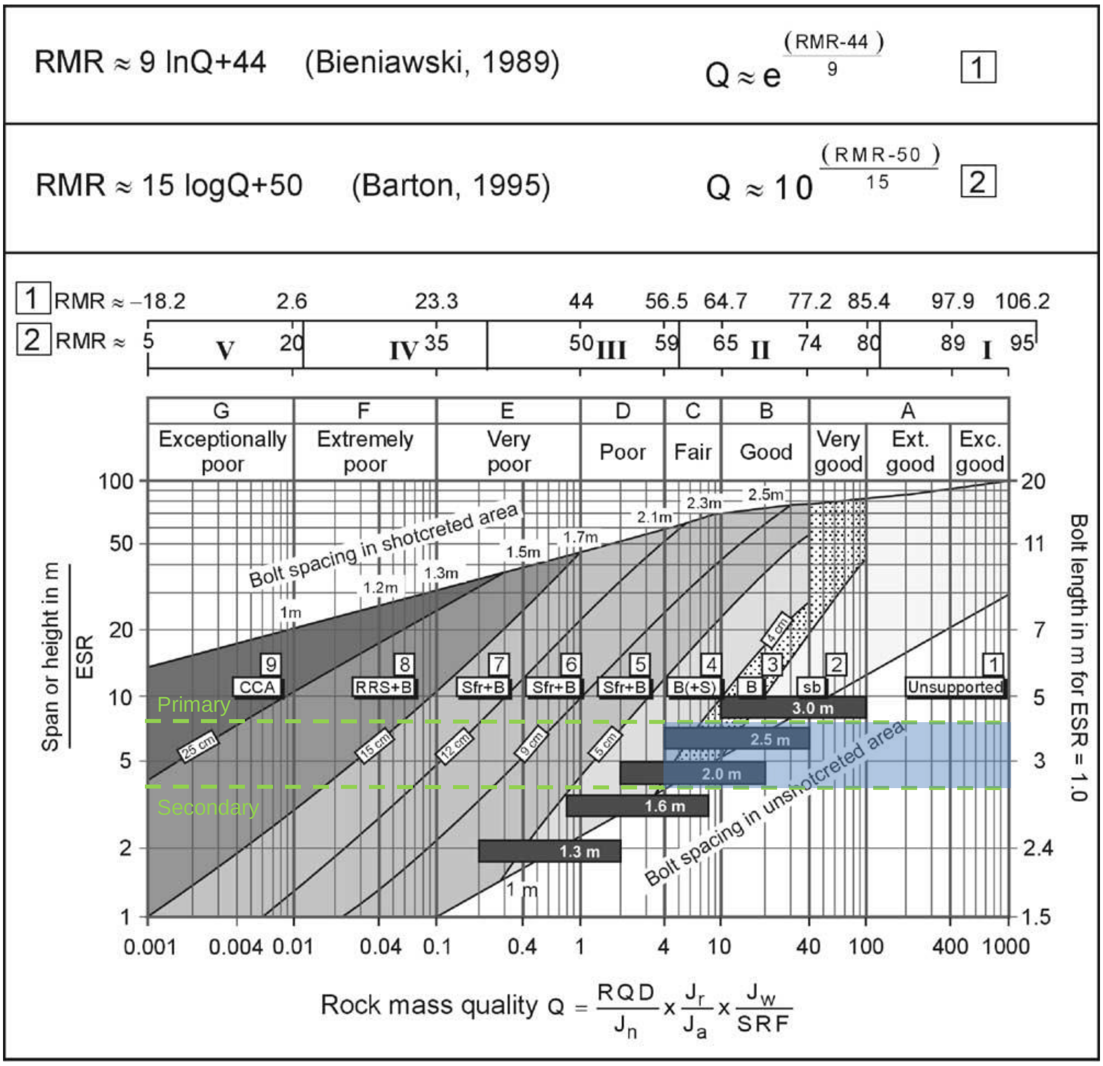


Figure 6-1: Q Support Chart (Barton, 2002)

The blue shaded zone in **Error! Reference source not found.** illustrates the range in Q_{wall} values for 98% of the rock mass. Based on this range no support is required for the majority of the shaft. A conservative approach was however applied which includes some level of support through the shaft. This was aimed to protect equipment and personnel working at the shaft bottom.

With this aim in mind it was determined that the primary support, which is installed prior to casting the concrete lining, should comprise blast resistant mesh and tendons, as a minimum. This is such that any spalling, or loose blocks, may be contained by the mesh. The secondary support will comprise a 300 mm concrete lining.

Primary support recommendations are presented in Table 6-1.

Table 6-1: GT017 Support Recommendations

Depth (m)		Primary Support	Secondary Support
0.00	7.00	Excavate box cut , or maintain reinforced concrete lining within 3.0 m of shaft bottom, soil nails can be used.	Collar designed for static and dynamic loads, 300 mm reinforced concrete lining
7.00	1100.07	1.8 m split sets, 1.5 m spacing, 4.5 - 8.2 tons, 46 mm outer diameter of tube (SS-46 model) Mesh, 80 mm aperture, 5 mm gauge	300 mm concrete lining
Weak Intervals >1m		2.4 m long resin bars, 1.5 m spacing	300 mm concrete lining
Poor Ground (Faults, Wedges)		6 m long cable anchors, 25 ton	300 mm concrete lining

It is to be noted that a pre-sink will be required in the weathered material, and to construct the shaft collar.

It is also recommended that cable anchors are installed where major faults and large wedges are identified. These should, therefore, be available on site and used on an *ad-hoc* basis if necessary.

Where weak intervals > 1 m thick exist, it is recommended that 2.4 m long resin bars are utilised. Weak rock <1 m^3 is likely to be caught by the mesh. A summary of the weak intervals (Q_{wall}< 4) identified for GT017 is presented in Table 6-2.

It can be concluded from this chapter that while no support is required for the majority of the shaft, some level of support has been recommended in the form of split sets and mesh to protect personnel working in the shaft. Chapter 7 details a risk assessment using the computer software JBlock to determine the failure potential of the shaft considering the rock support determined.

Table 6-2: GT017 weakness zones

From	To	Interval Length (m)	Geology	Mining Position	Matrix (m)	RQD (Palmstrom)	Q_{wall}	RMR (L90)	GSI
9.18	10.30	1.12	GN	HW	0.15	70	2.2	42	49
10.30	10.35	0.05	GN	HW	0.05	10	3.3	100	31
10.35	11.45	1.10	GN	HW	0.10	70	3.6	44	55
14.69	15.02	0.33	GN	HW	0.05	10	0.7	36	23
27.88	35.06	7.18	GN	HW	0.70	31	0.7	38	35
46.30	47.00	0.70	GN	HW	0.13	10	0.5	29	19
65.00	68.00	3.00	GN	HW	0.78	29	0.9	37	28
355.62	359.83	4.21	GRV	HW	0.29	78	3.2	55	61
423.40	423.50	0.10	GN	HW	0.00	10	2.6	38	20
620.91	621.18	0.27	GN	HW	0.00	10	4.0	40	25
736.84	736.96	0.12	GN	HW	0.05	10	1.1	28	20
804.58	804.75	0.17	FPX	REEF	0.07	10	0.4	30	19
893.40	893.73	0.33	N	FW	0.12	14	1.2	37	20
894.58	895.40	0.82	N	FW	0.26	10	0.0	34	14
899.66	899.76	0.10	N	FW	0.08	10	0.8	23	12
902.26	903.02	0.76	N	FW	0.34	10	0.1	31	20
903.62	904.05	0.43	N	FW	0.20	34	0.8	30	25
909.25	909.68	0.43	N	FW	0.30	10	0.1	29	12
910.49	910.63	0.14	N	FW	0.10	10	1.7	35	26
1095.46	1097.77	2.31	N	FW	0.09	81	3.2	42	54

Note: Values in red denote interval lengths >1 m

7 RISK ASSESSMENT

"The stability of sidewalls in vertical and inclined shafts is partly determined by wedges of rock which form by the intersection of joints or faults in the walls of the shaft" (Esterhuizen, 1990). These wedges fail by either sliding or falling into the excavation. The number of wedges which may be encountered and their probability of failure is information which is essential to support design in shafts (Esterhuizen, 1990). With this in mind, a risk assessment was conducted to verify the support design, taking into account the structures within the rock mass and the size and shape of the excavation to be constructed.

The risk assessment (which considered a probability of failure approach) was carried out using the computer software JBlock. JBlock is designed to create and analyse geometric blocks or wedges known as keyblocks. This is based on data collected in the form of joint orientations, trace lengths, joint conditions and friction angles (Joughin et al, 2012). JBlock then places these blocks in relation to the excavation created and tests the stability of each block (Joughin et al, 2012). The program has the capacity to simulate a large number of keyblocks as a function of joint set characteristics from which to derive a statistical failure distribution. The software can then be utilised to evaluate the influence of various support systems and support layouts (Dunn, 2010).

One of the key limitations of the JBlock application with respect to shaft analysis is that a closed, circular excavation geometry cannot be simulated directly. JBlock is currently limited to planar surfaces. In order to overcome this limitation, the failure potential of keyblocks has thus been simulated for the north, south, west and east sides of the shaft per structural zone to gain an overall appreciation of the failure potential around the shaft. A planar side wall is used to represent each direction as indicated by the red lines in Figure 7-1. The width of the side walls was determined using basic trigonometry for the assumed geometry, which is considered representative. For a segment with a 90° angle in 10 m diameter circle, the chord is 7.07 m, which was taken as the width of each sidewall.

Joint input parameters used to create the keyblock geometries are taken from the joint set and friction angle data for each structural zone (chapter 5, Table 5-1, Table 5-2, Table 5-3 and Table 5-4).

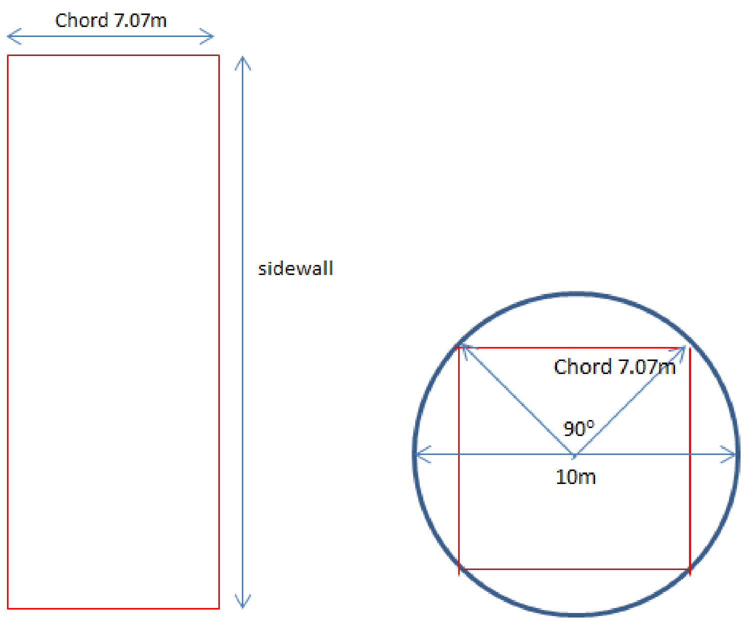


Figure 7-1: JBlock simulation side walls

The first step is to simulate keyblocks, based on the shaft orientation and joint set characteristics. An example of the simulated keyblock size distribution is presented in Figure 7-2.

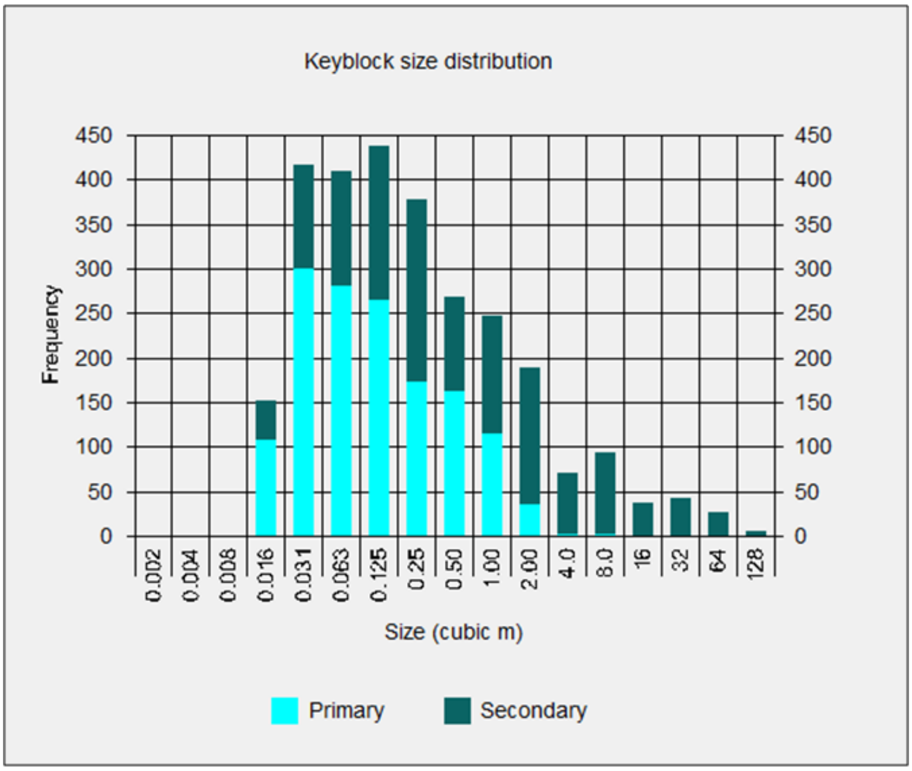


Figure 7-2: Typical keyblock size distribution

The second step is to simulate rock falls. A keyblock limit equilibrium analysis is performed on each keyblock, which are placed in random positions within the walls of the shaft. During the rock fall simulation the exposed surface area is recorded, which allows the number of simulated rock falls to be normalised.

A failure analysis of the possible keyblocks was evaluated for each structural zone in the context of four scenarios:

- North facing - primary support
- South facing - primary support
- West facing - primary support
- East facing – primary support

A summary of the input parameters used in JBlock are presented in Table 7-1.

Table 7-1: JBlock input parameters

General		Excavation	
No. of keyblocks	100 000	Length (m)	30
Density	3000 kg/m^3	Width (m)	7.07
External load	10 kPa	Blast adv. (m)	2
Aspect Ratio	1		
Support		**Hazard Analysis**	
Split Sets	4.5 ton, 1.8 m length, 1.5 m x 1.5 m spacing	Min. no. of mining step simulations	1000
Mesh	Not simulated	Min. no. of times to test each block	5
Concrete Lining	Not simulated	Percentage blocks released at face	75%
		Dist. for 99 % of blocks to be released	40 m

A summary of all simulations run in JBlock is presented from Table 7-2 to Table 7-5.

Note that concrete lining and mesh runs were not conducted for the analysis since it was found that when simulated these supports overcompensate in terms of their strength when applied to a vertical excavation in the software.

From the results simulated, the true height of failed blocks was corrected by subtracting the simulated height of each failed wedge by the segment height (Figure 7-3).

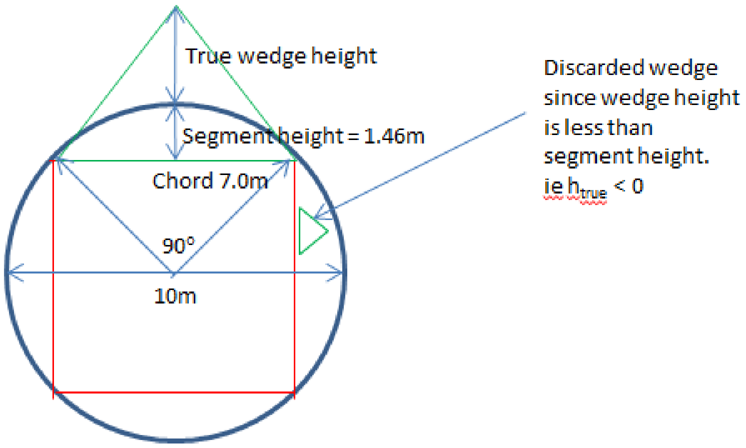


Figure 7-3: True wedge height illustration

Using these results, equivalent overbreak per domain was calculated by dividing the fallout volume by the total area exposed. This value was divided by the diameter of the excavation to determine the percentage overbreak. Overbreak determined for the north, south, east and west walls were combined to provide the total overbreak per domain.

Table 7-2: Structural Zone 1 JBlock Results

Zone 1				
	North	**South**	**East**	**West**
Total Area Exposed (m^2)	20004	19127	24968	24643
Total number of keyblocks tested	660	1160	175	730
Total number of failed keyblocks	38	2	0	394
Percentage keyblocks failed	**5.76**	**0.17**	**0.00**	**53.97**
Fallout volume (m3)	4.9	2.4	0.0	417.0
Overbreak (m)	0.0002	0.0001	0.0000	0.0169
Overbreak (%)	0.002%	0.001%	0.000%	0.169%
Total Overbreak (%)	**0.17%**			

Table 7-3: Structural Zone 2 JBlock Results

Zone 2				
	North	**South**	**East**	**West**
Total Area Exposed (m^2)	1221663	1232922	2401133	2378952
Total number of keyblocks tested	73774	45826	7808	57431
Total number of failed keyblocks	1401	192	5	126
Percentage keyblocks failed	**1.90**	**0.42**	**0.06**	**0.22**
Fallout volume (m3)	4284.0	592.0	0.3	437.0
Overbreak (m)	0.0035	0.0005	0.0000	0.0002
Overbreak (%)	0.0351%	0.0048%	0.0000%	0.0018%
Total Overbreak (%)	**0.04%**			

Table 7-4: Structural Zone 3 JBlock Results

Zone 3				
	North	**South**	**East**	**West**
Total Area Exposed (m^2)	230406	234169	514241	289045
Total number of keyblocks tested	12611	13267	2355	1065
Total number of failed keyblocks	356	78	2	48
Percentage keyblocks failed	**2.82**	**0.59**	**0.08**	**4.51**
Fallout volume (m3)	1155	99	1.5	23
Overbreak (m)	0.0050	0.0004	0.0000	0.0001
Overbreak (%)	0.0501%	0.0042%	0.0000%	0.0008%
Total Overbreak (%)	**0.06%**			

Table 7-5: Structural Zone 4 JBlock Results

Zone 4				
	North	**South**	**East**	**West**
Total Area Exposed (m^2)	130812	131235	216993	218819
Total number of keyblocks tested	10569	10231	11087	9925
Total number of failed keyblocks	107	4	12	1548
Percentage keyblocks failed	**1.01**	**0.04**	**0.11**	**15.60**
Fallout volume (m^3)	43	11	31	1724
Overbreak (m)	0.0003	0.0001	0.0001	0.0079
Overbreak (%)	0.0033%	0.0008%	0.0014%	0.0788%
Total Overbreak (%)	**0.08%**			

The simulated number of rockfalls was also determined for a range of block sizes from 0.1 m^3 to 30 m^3. This was normalised to rock falls per 100 m advance using the total area exposed and the shaft diameter. This is presented for each structural domain in Figure 7-4. This provides an indication of the frequency of occurrence of rockfalls.

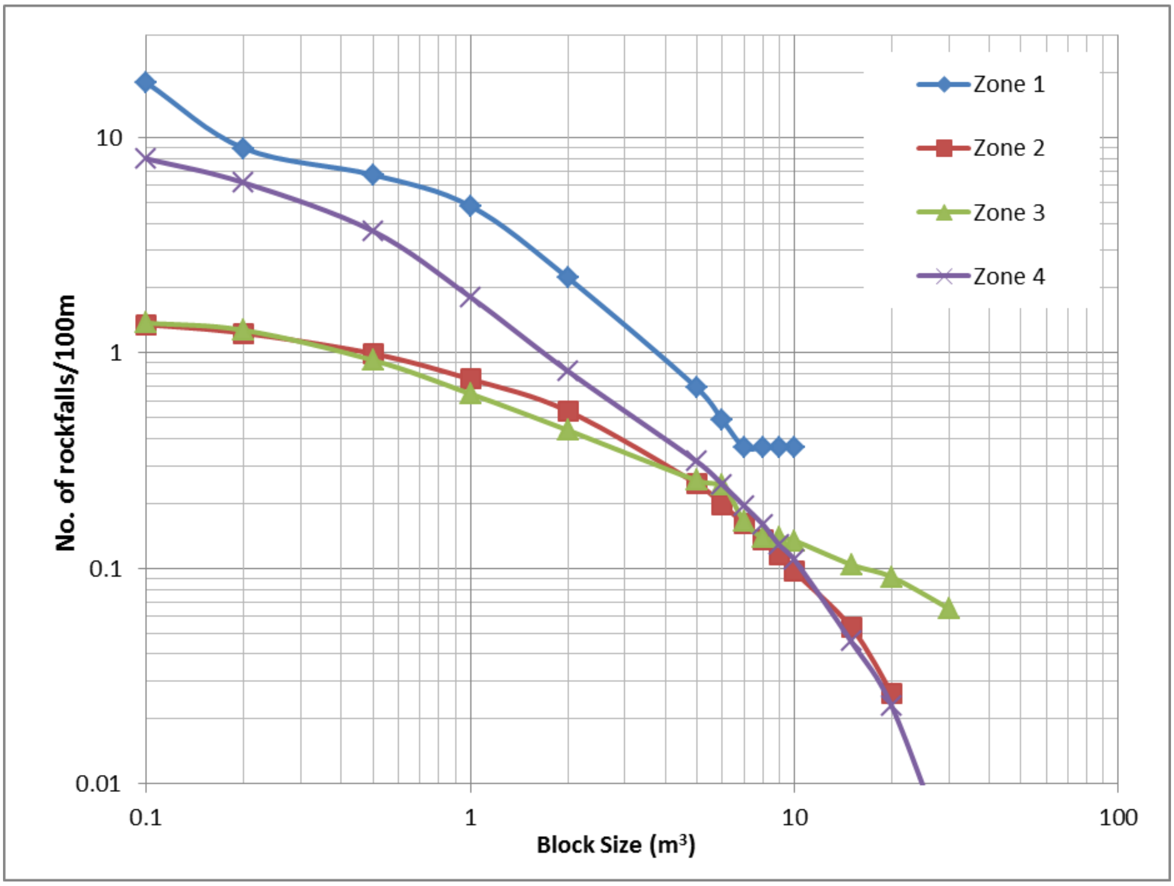


Figure 7-4: Number of rockfalls per 100 m advance for each structural zone

From the results simulated it was observed that structural zones 2 and 3 produced less than 2 rockfalls per 100 m of advance, with the majority of failed blocks ranging between 0.1 m^3 to 1 m^3.

Structural zone 1 produced the highest number of rockfalls per 100 m of advance. This is due the smaller spacing's between joints sets compared to the other structural zones.

While a number of blocks has failed it is important to note that the simulations were run without the application of wired mesh. A number of failed blocks will thus be retained with the application of the recommended 80 mm aperture wired mesh, which will reduce the hazard to equipment and personnel working at the bottom of the shaft.

While larger rockfalls are infrequent, it must be noted that these falls could occur. This analysis thus highlights the need to identify potential unstable blocks and install additional support during sinking. It is therefore important that the cable anchors recommended are available on site such that these may be used if necessary.

From this chapter it may be concluded that the percentage overbreak from geological structures and number of rockfalls per 100 m of advance is low, and is unlikely to influence the overall project. Based on the use of the JBlock software it is evident that the program allows for the identification of high risk areas in an underground excavation, where the size and frequency of unstable blocks can be determined. Further development of the software should focus on the use of the existing keyblock theory to evaluate circular shafts (Esterhuizen, 1990). Chapter 8 outlines the conclusions and recommendations determined for the geotechnical investigation and the findings from the use of the rock mass classifications utilised in the study.

8 CONCLUSIONS AND RECOMMENDATIONS

A geotechnical investigation has been completed for shaft 2 which is to be excavated to a depth of approximately 1100 m. In carrying out the investigation, the sub-surface ground conditions from the dedicated shaft borehole (GT017) were determined.

The outcome of the geotechnical investigation of GT017 indicates the following:

- There is a higher degree of jointing in the footwall (832m to end of hole) and orebody (788m to 832m) compared to the hangingwall (surface to 788m).
- The intact uniaxial compressive strength of the units range from 194 MPa to 264 MPa (determined from laboratory test results).
- There are small weak zones within the hangingwall, orebody and footwall (< 1m). These are sheared and altered. These weakness zones will have a structural effect on the shaft excavation and have been taken into consideration in the support recommendations.
- Considering the Norwegian Geotechnical Institute's Q system, with the Qwall adjustment, the quality of the rock mass is classified as "Good" to "Exceptionally Good" (91 % of the borehole). There are small zones which are classified as "Fair" (7 % of the borehole), while 2 % of the borehole is classified as "Very Poor" to "Poor".
- From the comparison of the mean values for Laubscher's RMR, Qwall and GSI for the hangingwall, orebody and footwall, it is concluded that the different classification system results compare well, being within 10 point range.
- The RMR values determined from Qwall are higher than those of Laubscher's RMR values. This is expected due to the adjustments made to Q for the vertical walls of the shaft.
- Overall the rock mass quality is higher in the hangingwall compared to that of the orebody and footwall.
- From the JBlock risk assessment conducted it was concluded that the percentage overbreak from geological structures and number of rockfalls per 100 m of advance is low, and is unlikely to influence the overall project. However, it must be noted that while unlikely, large wedges could occur. This highlights the need to identify potential unstable blocks and install additional support where necessary during sinking.

From the interpretation of the geotechnical borehole log, it can be concluded that the rock mass is of a suitable quality for the sinking of shaft 2 to a depth of approximately 1100 m.

Preliminary support for the Platreef vertical shaft has been recommended along the length of the borehole, which is presented in Table 6-1.

From the rock mass classification systems utilised, the following is concluded:

- The object of a classification system is to assign a value to a rock mass as opposed to a vague descriptive term. Practical experience has indicated that rock mass classification systems thus work well, provided that there is a common understanding of the practical aspects relating to the individual rating system. It must also be kept in mind that each classification system should be used as a guide, and that each case should be examined in detail with engineering judgement in mind (Laubscher, 1977).
- The success and accuracy of the results of a classification system depends greatly on the reliability of the data available and the sampling of the area being investigated.

- The use of more than one rock mass classification system is recommended as this allows for validation and confirmation of estimates made. This in turn increases confidence in the results achieved.
- In order to gain the most value from a rock mass classification system, it is recommended that these systems are used by practitioners with practical experience, and that each user is aware of the limitations of each system (Palmstrom, 2008).
- It is essential to note that use of classification systems is contrary to detailed engineering design processes. This is since “there are no applied mechanics calculations of stress or displacement, no computations, or information, as to loads, strains and stresses in the support elements (shotcrete, rockbolts and sets), and therefore nothing against which to compare field-monitoring data. The position of the classification design approach in relation to modern limit state design is thus unknown and unknowable” (Pells and Bertuzzi, 2007).
- The use of classification systems can be of great benefit during the planning stages of a project, when little detailed information on the rock mass and its stress and hydrologic environment is available (Pamstrom and Broch, 2006).
- The Q system with the adaptation of Qwall provides an appropriate system for the determination of rock mass quality and rock support for a shaft analysis.
- While the Q system with the application of the Qwall adjustment is the appropriate rock mass classification system to analyse the rock quality for shaft sinking, the use of other classification systems allows for comparison and validation of what is determined by the primary classification system utilised.
- Taking core loss and the presence of matrix into account for the FF/m and Palmstrom’s RQD equation allows the ability to downgrade these values and their respective RMR and Q values for intervals where weakness zones exist.
- While use of Barton’s support chart provides adequate support recommendations, these should be used as a guideline and should not replace good engineering judgement. Additional support requirements should be employed where deemed necessary.

From the geotechnical investigation undertaken and the assessment of the rock mass classification systems utilised, it is believed that the research report has met the objectives outlined in chapter 1.

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Appendices

Appendix A: Laboratory Test Results

BH ID	Sample Number	From (m)	To (m)	Rock Type	UCS (MPa)	Failure Code
GT017	GTT 081	814.4	314.67	**GRV**	272	YA
GT017	GTT 082	815.5	815.81	**GRV**	219	6B
GT017	GTT 083	816.4	817.02	**GRV**	258	YA
GT017	GTT 084	816.4	817.02	**GRV**	282	YA
GT017	GTT 085	817.4	818.06	**GRV**	262	YA
GT017	GTT 086	817.7	817.89	**GRV**	231	YA
GT017	GTT 090	780	780.8	**AN**	285	YA
GT017	GTT 091	780	780.8	**AN**	269	YA
GT017	GTT 095	780	780.8	**AN**	263	YA
GT017	GTT 096	780.9	781.13	**AN**	293	YA
GT017	GTT 097	781.1	781.32	**AN**	273	YA
GT017	GTT 098	781.3	781.55	**AN**	253	YA
GT017	GTT 101	782.1	782.82	**N**	195	5B
GT017	GTT 102	782.1	782.82	**N**	141	2B
GT017	GTT 109	785.6	786.19	**N**	122	4B
GT017	GTT 110	785.6	786.19	**N**	210	6B
GT017	GTT 115	785.4	786.63	**N**	118	3B
GT017	GTT 116	779.1	779.39	**N**	161	YA
GT008	N UCM2	882.5	882.84	**N**	242	YA
GT008	N UCM3	927.9	928.3	**N**	260	YA
GT008	N UCM4	1008	1008.88	**N**	231	YA
GT017	GTT 118	819.5	819.95	**FPX**	184	YA
GT017	GTT 119	819.5	819.95	**FPX**	226	YA
GT017	GTT 125	821.2	821.58	**FPX**	217	YA
GT017	GTT 126	891.9	922.26	**FPX**	191	3B
GT017	GTT 132	827.1	827.93	**FPX**	127	XA
GT017	GTT 133	827.1	827.93	**FPX**	137	4B
GT017	GTT 147	368	368.68	**GN**	232	YA
GT017	GTT 148	368	368.68	**GN**	236	YA
GT017	GTT 156	618.6	619.07	**GN**	197	2B
GT017	GTT 157	743.9	744.72	**GN**	286	YA
GT017	GTT 162	749.4	750.33	**GN**	248	YA
GT017	GTT 163	749.4	750.33	**GN**	241	YA
GT017	GTT 135	953.8	954.22	**HF**	283.7	YA
GT017	GTT 136	954.2	954.68	**HF**	150.9	2B
GT017	GTT 137	955.2	955.6	**HF**	299.4	YA
GT017	GTT 138	955.2	955.6	**HF**	305.9	YA
GT017	GTT 139	957.8	958.16	**HF**	273.3	YA
GT017	GTT 140	959.6	960.12	**HF**	236.3	YA
GT017	GTT 141	829.3	829.84	**MPV**	82.4	6B
GT017	GTT 142	829.3	829.84	**MPV**	131.6	XB
GT012	GTT 143	908.5	908.82	**MPV**	84.3	6B
GT012	GTT 144	908.5	908.82	**MPV**	104.2	3B
GT017	GTT 145	830.9	831.18	**MPV**	115.1	0B
GT017	GTT 146	831.4	831.56	**MPV**	130.3	4B

BH ID	Sample Number	Depth (m)		Rock Type	Confining Pressure sigma 3 (MPa)	TCS sigma 1 (MPa)	Failure Code
		From	To				
GT017	GTT 099	781.55	781.9	**N**	5	173	2B
GT017	GTT 103	782.135	782.815	**N**	10	258	XA
GT017	GTT 104	783.08	783.585	**N**	15	328	XA
GT017	GTT 105	783.08	783.585	**N**	20	386	XA
GT017	GTT 107	783.585	783.785	**N**	30	460	XA
GT017	GTT 108	783.795	784.14	**N**	10	280	XA
GT017	GTT 112	784.14	784.73	**N**	15	312	XA
GT017	GTT 113	784.14	784.73	**N**	20	306	3B
GT017	GTT 114	784.14	784.73	**N**	30	419	2B
GT017	GTT 121	819.945	820.66	**FPX**	5	267	XA
GT017	GTT 122	819.945	820.66	**FPX**	10	312	XA
GT017	GTT 123	819.945	820.66	**FPX**	15	320	XA
GT017	GTT 127	823.79	824.105	**FPX**	20	263	3B
GT017	GTT 128	824.97	825.25	**FPX**	30	324	XA
GT017	GTT 129	825.49	826.22	**FPX**	10	206	XA
GT017	GTT 130	825.49	826.22	**FPX**	15	260	XA
GT017	GTT 131	825.49	826.22	**FPX**	20	273	XA
GT017	GTT 134	827.045	827.93	**FPX**	30	270	XA
GT017	GTT 149	367.95	368.68	**GN**	5	312	XA
GT017	GTT 151	429.26	429.78	**GN**	10	336	XA
GT017	GTT 152	429.26	429.78	**GN**	15	378	XA
GT017	GTT 153	525.905	526.48	**GN**	20	448	XA
GT017	GTT 154	525.905	526.48	**GN**	30	531	XA
GT017	GTT 155	618.57	619.07	**GN**	10	259	XA
GT017	GTT 158	743.87	744.72	**GN**	15	492	XA
GT017	GTT 159	743.87	744.72	**GN**	20	471	XA
GT017	GTT 161	749.375	750.33	**GN**	specimen could not be prepared		
GT008	KGTCS1	142.05	142.67	**GRV**	2	136	3B
GT008	KGTCS1	142.05	142.67	**GRV**	5	177	XA
GT008	KGTCS1	142.05	142.67	**GRV**	15	377	XA
GT008	KGTCS1	142.05	142.67	**GRV**	30	527	XA
GT008	KGTCS3	267.42	267.97	**GRV**	2	197	XA
GT008	KGTCS3	267.42	267.97	**GRV**	5	285	XA
GT008	KGTCS3	267.42	267.97	**GRV**	15	338	XA
GT008	KGTCS3	267.42	267.97	**GRV**	30	533	XA
GT008	KGTCS5	671.49	672.46	**GRV**	2	238	2B
GT008	KGTCS5	671.49	672.46	**GRV**	5	329	XA
GT008	KGTCS5	671.49	672.46	**GRV**	15	471	XA
GT008	KGTCS5	671.49	672.46	**GRV**	30	519	XA
GT008	ANTCS1	570.8	571.75	**AN**	2	156	XA
GT008	ANTCS1	570.8	571.75	**AN**	5	235	XA
GT008	ANTCS1	570.8	571.75	**AN**	15	352	XA
GT008	ANTCS1	570.8	571.75	**AN**	30	521	XA

BH ID	Sample Number	Depth (m)	Depth To (m)	Rock Type	Confining Pressure sigma 3 (MPa)	TCS sigma 1 (MPa)	Failure Code
GT008	ANTCS2	609.21	610.17	**AN**	2	232	XA
GT008	ANTCS2	609.21	610.17	**AN**	5	259	XA
GT008	ANTCS2	609.21	610.17	**AN**	15	411	XA
GT008	ANTCS2	609.21	610.17	**AN**	30	489	XA
GT008	ANTCS3	648.05	648.97	**AN**	2	207	XA
GT008	ANTCS3	648.05	648.97	**AN**	5	291	2B
GT008	ANTCS3	648.05	648.97	**AN**	15	397	XA
GT008	ANTCS3	648.05	648.97	**AN**	30	486	XA
GT008	HFTCS1	939.25	940.18	**HF**	2	258	XA
GT008	HFTCS1	939.25	940.18	**HF**	5	156	2B
GT008	HFTCS1	939.25	940.18	**HF**	15	409	XA
GT008	HFTCS1	939.25	940.18	**HF**	30	482	XA
GT008	HFTCS2	1106.39	1108.16	**HF**	2	242	YA
GT008	HFTCS2	1106.39	1108.16	**HF**	5	91	3B
GT008	HFTCS2	1106.39	1108.16	**HF**	15	291	4B
GT008	HFTCS2	1106.39	1108.16	**HF**	30	184	4B
GT008	HFTCS3	1131.52	1132.37	**HF**	2	316	0B
GT008	HFTCS3	1131.52	1132.37	**HF**	5	199	2B
GT008	HFTCS3	1131.52	1132.37	**HF**	15	310	4B
GT008	HFTCS3	1131.52	1132.37	**HF**	30	481	0B

*Please note results presented in red failed on discontinuities

BH ID	Sample No.	Depth (m) From	Depth (m) To	Rock Type	Brazilian Tensile Strength MPa
GT017	UTB-087	814.67	814.83	**GRV**	14
GT017	UTB-088	814.67	814.83	**GRV**	12
GT017	UTB-089	814.67	814.83	**GRV**	13
GT017	UTB-092	780.04	780.8	**AN**	17
GT017	UTB-093	780.04	780.8	**AN**	16
GT017	UTB-094	780.04	780.8	**AN**	13
GT017	GTT 100	782.14	782.82	**N**	16
GT017	GTT 106	783.08	783.59	**N**	14
GT017	GTT 111	785.63	786.19	**N**	15
GT017	GTT 117	819.45	819.95	**FPX**	15
GT017	GTT 120	819.95	820.66	**FPX**	18
GT017	GTT 124	821.2	821.58	**FPX**	18
GT017	GTT 150	367.95	368.68	**GN**	15
GT017	GTT 160	743.87	744.72	**GN**	14
GT017	GTT 164	749.38	750.33	**GN**	15
GT008	HF UCM1	934.65	935.33	**HF**	16
GT008	HF UCM3	1131.21	1131.52	**HF**	18
GT008	HF UTB2	1108.65	1108.83	**HF**	18

Appendix B: GT017 Rock Mass Classification

From (m)	To (m)	Interval Length (m)	Geology	UCS	Q	Q_{wall}	RMR (L90)	RMR Barton From Qwall	RMR Barton From Q	GSI
0.00	1.54	1.54	SS	NA	NA	NA	NA	NA	NA	NA
1.54	7.00	5.46	SS	NA	NA	NA	NA	NA	NA	NA
7.00	9.18	2.18	CAL	NA	NA	NA	NA	NA	NA	NA
9.18	10.30	1.12	GN	264	0.9	2.2	42	55	49	49
10.30	10.35	0.05	GN	232	1.3	3.3	100	58	52	31
10.35	11.45	1.10	GN	264	1.4	3.6	44	58	52	55
11.45	14.69	3.24	GN	264	6.2	15.4	58	68	62	64
14.69	15.02	0.33	GN	264	0.3	0.7	36	48	42	23
15.02	27.88	12.86	GN	264	3.4	8.5	57	64	58	71
27.88	35.06	7.18	GN	264	0.3	0.7	38	48	42	35
35.06	38.80	3.74	GN	264	3.7	9.3	52	65	59	64
38.80	40.96	2.16	GN	264	2.0	4.9	43	60	54	51
40.96	44.46	3.50	GN	264	3.7	9.4	53	65	59	62
44.46	46.30	1.84	GN	264	99.5	497.3	77	90	80	84
46.30	47.00	0.70	GN	232	0.2	0.5	29	45	39	19
47.00	47.88	0.88	GN	232	18.7	93.7	53	80	69	64
47.88	50.05	2.17	GRV	249	1.7	4.2	54	59	53	18
50.05	51.42	1.37	GRV	249	100.0	500.0	100	90	80	76
51.42	51.77	0.35	GN	264	10.6	53.2	47	76	65	53
51.77	53.26	1.49	GN	264	200.0	1000.0	88	95	85	92
53.26	54.19	0.93	GN	264	23.9	119.6	56	81	71	65
54.19	55.73	1.54	GN	264	9.7	24.4	44	71	65	64
55.73	56.00	0.27	GRV	249	5.2	13.1	41	67	61	28
56.00	60.76	4.76	GRV	249	5.4	13.4	53	67	61	65
60.76	62.46	1.70	GRV	249	3.0	7.6	44	63	57	48
62.46	65.00	2.54	GRV	249	100.0	500.0	67	90	80	85
65.00	68.00	3.00	GN	232	0.4	0.9	37	49	43	28
68.00	68.88	0.88	GN	264	4.2	10.4	38	65	59	44
68.88	69.62	0.74	GN	264	5.7	14.3	43	67	61	56
69.62	77.66	8.04	GN	264	6.8	17.1	57	68	63	61
77.66	77.89	0.23	GN	264	10.0	24.9	51	71	65	51
77.89	78.77	0.88	GN	264	100.0	500.0	100	90	80	76
78.77	79.80	1.03	GN	264	6.5	16.3	43	68	62	57
79.80	94.00	14.20	GN	264	25.0	124.9	70	81	71	67
94.00	109.00	15.00	GN	264	8.2	20.6	67	70	64	62
109.00	124.00	15.00	GN	264	7.5	18.7	62	69	63	61
124.00	131.32	7.32	GN	264	100.0	500.0	100	90	80	76
131.32	132.74	1.42	GN	264	13.3	66.7	100	77	67	76
132.74	147.00	14.26	GN	264	11.4	56.8	74	76	66	66
147.00	162.00	15.00	GN	264	100.0	500.0	100	90	80	76
162.00	168.59	6.59	GN	264	100.0	500.0	100	90	80	76
168.59	180.53	11.94	GN	264	100.0	500.0	81	90	80	85
180.53	194.00	13.47	GN	264	100.0	500.0	100	90	80	76
194.00	207.60	13.60	GN	264	8.3	20.7	65	70	64	63
207.60	221.00	13.40	GN	264	16.7	83.3	72	79	68	63

From (m)	To (m)	Interval Length (m)	Geology	UCS	Q	Q_{wall}	RMR (L90)	RMR Barton From Qwall	RMR Barton From Q	GSI
221.00	236.00	15.00	GN	264	200.0	1000.0	90	95	85	92
236.00	251.00	15.00	GN	264	100.0	500.0	100	90	80	76
251.00	255.00	4.00	GN	264	12.5	62.5	64	77	66	60
255.00	255.40	0.40	GN	232	10.8	53.8	48	76	65	53
255.40	255.86	0.46	GN	264	100.0	500.0	100	90	80	76
255.86	256.61	0.75	GN	264	13.0	64.9	51	77	67	52
256.61	264.20	7.59	GN	232	100.0	500.0	100	90	80	76
264.20	264.51	0.31	AN	249	100.0	500.0	100	90	80	76
264.51	279.51	15.00	GN	264	100.0	500.0	100	90	80	76
279.51	285.44	5.93	GN	264	100.0	500.0	100	90	80	76
285.44	286.00	0.56	AN	249	100.0	500.0	100	90	80	76
286.00	297.75	11.75	GN	264	16.6	83.1	75	79	68	63
297.75	302.67	4.92	GN	264	100.0	500.0	100	90	80	76
302.67	304.86	2.19	GN	264	100.0	500.0	100	90	80	76
304.86	305.15	0.29	GN	264	66.7	333.3	63	88	77	80
305.15	307.08	1.93	GN	264	8.5	21.3	53	70	64	62
307.08	308.23	1.15	GN	264	25.0	125.0	55	81	71	67
308.23	310.69	2.46	GN	264	100.0	500.0	100	90	80	76
310.69	315.32	4.63	GN	264	100.0	500.0	100	90	80	76
315.32	315.38	0.06	GN	264	100.0	500.0	100	90	80	76
315.38	320.00	4.62	GN	264	100.0	500.0	100	90	80	76
320.00	328.42	8.42	GN	264	66.7	333.3	76	88	77	80
328.42	337.11	8.69	GN	264	100.0	500.0	100	90	80	76
337.11	337.59	0.48	GRV	249	88.8	444.2	63	90	79	79
337.59	339.22	1.63	AN	249	100.0	500.0	100	90	80	76
339.22	339.45	0.23	GRV	249	100.0	500.0	100	90	80	76
339.45	342.07	2.62	GN	264	100.0	500.0	100	90	80	76
342.07	349.23	7.16	AN	249	21.4	107.1	72	80	70	66
349.23	353.85	4.62	GN	264	100.0	500.0	100	90	80	76
353.85	354.68	0.83	GRV	249	12.3	61.6	53	77	66	65
354.68	355.62	0.94	GN	264	25.0	125.0	56	81	71	67
355.62	359.83	4.21	GRV	249	1.3	3.2	55	58	52	61
359.83	360.33	0.50	GN	264	9.6	23.9	43	71	65	58
360.33	361.30	0.97	GRV	249	16.2	80.8	60	79	68	61
361.30	361.58	0.28	GN	264	24.5	122.7	53	81	71	60
361.58	362.07	0.49	GRV	249	98.0	489.8	67	90	80	84
362.07	377.16	15.09	GN	264	100.0	500.0	100	90	80	76
377.16	378.18	1.02	GRV	249	37.5	187.5	69	84	74	72
378.18	389.50	11.32	GN	264	12.5	62.5	70	77	66	60
389.50	389.70	0.20	GN	264	100.0	500.0	100	90	80	76
389.70	393.54	3.84	GN	264	100.0	500.0	79	90	80	85
393.54	394.00	0.46	GRV	249	100.0	500.0	100	90	80	76
394.00	401.98	7.98	GN	264	100.0	500.0	81	90	80	85
401.98	402.39	0.41	GRV	249	100.0	500.0	100	90	80	76
402.39	405.44	3.05	GN	264	100.0	500.0	100	90	80	76

From (m)	To (m)	Interval Length (m)	Geology	UCS	Q	Q_{wall}	RMR (L90)	RMR Barton From Qwall	RMR Barton From Q	GSI
405.44	412.14	6.70	GN	264	30.0	150.0	75	83	72	70
412.14	413.65	1.51	GRV	249	200.0	1000.0	72	95	85	92
413.65	422.00	8.35	GN	264	8.0	19.9	67	69	64	63
422.00	422.16	0.16	GN	264	1.7	4.3	33	59	53	23
422.16	423.40	1.24	GN	264	100.0	500.0	100	90	80	76
423.40	423.50	0.10	GN	264	1.0	2.6	38	56	50	20
423.50	423.89	0.39	GN	264	100.0	500.0	100	90	80	76
423.89	424.18	0.29	GN	264	81.3	406.5	60	89	79	62
424.18	427.00	2.82	GN	264	8.9	22.2	54	70	64	63
427.00	428.00	1.00	GN	264	3.0	7.5	37	63	57	48
428.00	436.05	8.05	GN	264	6.2	15.6	61	68	62	60
436.05	437.56	1.51	AN	249	76.2	380.8	100	89	78	64
437.56	437.77	0.21	GRV	249	100.0	500.0	100	90	80	76
437.77	438.96	1.19	AN	249	100.0	500.0	100	90	80	76
438.96	439.05	0.09	GRV	249	100.0	500.0	100	90	80	76
439.05	443.45	4.40	GN	264	16.7	83.3	67	79	68	63
443.45	444.00	0.55	GRV	249	100.0	500.0	68	90	80	85
444.00	447.23	3.23	GN	264	100.0	500.0	100	90	80	76
447.23	447.97	0.74	GN	264	15.3	76.7	47	78	68	59
447.97	449.52	1.55	GN	264	9.3	23.1	55	70	64	64
449.52	452.78	3.26	GN	264	3.4	8.5	43	64	58	53
452.78	452.97	0.19	GRV	249	158.4	791.9	67	93	83	81
452.97	453.26	0.29	GN	264	11.8	59.2	48	77	66	53
453.26	458.81	5.55	GRV	249	15.5	77.4	62	78	68	70
458.81	460.17	1.36	GN	264	1.6	4.0	47	59	53	59
460.17	464.52	4.35	GN	232	100.0	500.0	100	90	80	76
464.52	464.91	0.39	GRV	249	25.0	125.0	55	81	71	67
464.91	465.32	0.41	GN	264	21.4	107.1	56	80	70	66
465.32	466.05	0.73	GRV	249	100.0	500.0	100	90	80	76
466.05	471.28	5.23	GN	264	100.0	500.0	100	90	80	76
471.28	471.64	0.36	GRV	249	100.0	500.0	67	90	80	85
471.64	473.45	1.81	GN	232	100.0	500.0	100	90	80	76
473.45	473.64	0.19	GRV	249	100.0	500.0	100	90	80	76
473.64	479.05	5.41	GN	264	23.1	115.4	69	81	70	75
479.05	479.91	0.86	GN	264	15.2	76.1	47	78	68	59
479.91	483.44	3.53	GN	264	100.0	500.0	100	90	80	76
483.44	485.47	2.03	GN	264	7.7	19.3	47	69	63	68
485.47	494.74	9.27	GN	264	100.0	500.0	100	90	80	76
494.74	495.74	1.00	AN	249	25.6	128.0	49	82	71	64
495.74	497.88	2.14	GN	264	1.8	4.5	45	60	54	58
497.88	498.58	0.70	AN	249	4.1	10.2	42	65	59	44
498.58	498.89	0.31	GRV	249	109.4	547.2	60	91	81	79
498.89	499.97	1.08	AN	249	18.7	93.7	63	80	69	72
499.97	500.74	0.77	GN	264	100.0	500.0	100	90	80	76
500.74	501.17	0.43	GRV	249	66.1	330.6	62	88	77	79

From (m)	To (m)	Interval Length (m)	Geology	UCS	Q	Q_{wall}	RMR (L90)	RMR Barton From Qwall	RMR Barton From Q	GSI
501.17	508.36	7.19	GN	264	100.0	500.0	81	90	80	85
508.36	510.59	2.23	AN	249	100.0	500.0	100	90	80	76
510.59	523.57	12.98	GN	264	35.7	178.6	70	84	73	81
523.57	525.28	1.71	AN	249	22.7	113.6	65	81	70	75
525.28	526.89	1.61	GN	264	100.0	500.0	100	90	80	76
526.89	527.18	0.29	AN	249	100.0	500.0	100	90	80	76
527.18	527.43	0.25	GRV	249	100.0	500.0	100	90	80	76
527.43	528.11	0.68	AN	249	100.0	500.0	100	90	80	76
528.11	529.48	1.37	GN	264	40.0	200.0	71	85	74	73
529.48	529.67	0.19	GRV	249	100.0	500.0	100	90	80	76
529.67	534.44	4.77	GN	264	100.0	500.0	81	90	80	85
534.44	538.37	3.93	AN	249	100.0	500.0	100	90	80	76
538.37	539.17	0.80	GRV	249	200.0	1000.0	77	95	85	92
539.17	542.47	3.30	GN	264	25.0	125.0	75	81	71	67
542.47	543.50	1.03	AN	249	28.6	142.9	57	82	72	69
543.50	546.11	2.61	GN	264	100.0	500.0	100	90	80	76
546.11	547.47	1.36	GRV	249	66.7	333.3	65	88	77	80
547.47	553.20	5.73	GN	264	66.7	333.3	83	88	77	80
553.20	553.42	0.22	GRV	249	93.9	469.4	60	90	80	82
553.42	562.14	8.72	GN	264	100.0	500.0	100	90	80	76
562.14	562.31	0.17	GRV	249	95.0	474.9	67	90	80	82
562.31	573.22	10.91	GN	264	100.0	500.0	83	90	80	85
573.22	573.42	0.20	GN	264	3.1	7.9	46	63	57	23
573.42	575.05	1.63	GRV	249	41.7	208.3	66	85	74	83
575.05	575.53	0.48	GN	264	7.8	19.4	49	69	63	48
575.53	576.58	1.05	GN	264	8.0	19.9	46	69	64	50
576.58	579.12	2.54	GN	264	12.0	60.0	61	77	66	67
579.12	579.84	0.72	GRV	249	100.0	500.0	74	90	80	85
579.84	583.62	3.78	GN	264	25.0	125.0	80	81	71	67
583.62	583.75	0.13	GRV	249	100.0	500.0	100	90	80	76
583.75	584.02	0.27	GN	264	100.0	500.0	100	90	80	76
584.02	584.16	0.14	GRV	249	90.7	453.5	59	90	79	80
584.16	586.09	1.93	GN	264	100.0	500.0	100	90	80	76
586.09	586.29	0.20	GRV	249	100.0	500.0	100	90	80	76
586.29	589.30	3.01	GN	264	100.0	500.0	100	90	80	76
589.30	589.45	0.15	GRV	249	168.0	840.1	66	94	83	84
589.45	600.06	10.61	GN	264	66.7	333.3	79	88	77	80
600.06	601.21	1.15	GRV	249	66.7	333.3	63	88	77	80
601.21	603.11	1.90	GN	264	100.0	500.0	73	90	80	85
603.11	603.40	0.29	GRV	249	16.7	83.3	58	79	68	63
603.40	603.97	0.57	GN	232	100.0	500.0	100	90	80	76
603.97	604.58	0.61	GRV	249	66.7	333.3	63	88	77	80
604.58	609.67	5.09	GN	264	41.7	208.3	75	85	74	83
609.67	611.35	1.68	GRV	249	83.9	419.6	74	89	79	77
611.35	612.08	0.73	GN	264	100.0	500.0	100	90	80	76

From (m)	To (m)	Interval Length (m)	Geology	UCS	Q	Q_{wall}	RMR (L90)	RMR Barton From Qwall	RMR Barton From Q	GSI
612.08	612.35	0.27	GRV	249	136.0	679.9	58	92	82	76
612.35	620.09	7.74	GN	264	3.1	7.8	56	63	57	61
620.09	620.23	0.14	GRV	249	15.3	76.7	49	78	68	55
620.23	620.91	0.68	GN	264	25.0	125.0	69	81	71	67
620.91	621.18	0.27	GN	264	1.6	4.0	40	59	53	25
621.18	628.82	7.64	GN	264	6.2	15.5	65	68	62	69
628.82	629.44	0.62	GRV	231	100.0	500.0	100	90	80	76
629.44	630.01	0.57	GN	264	13.2	65.8	54	77	67	68
630.01	630.11	0.10	GRV	249	100.0	500.0	100	90	80	76
630.11	630.23	0.12	GN	264	100.0	500.0	100	90	80	76
630.23	630.67	0.44	GRV	249	100.0	500.0	67	90	80	85
630.67	634.42	3.75	GN	264	66.7	333.3	76	88	77	80
634.42	634.76	0.34	GN	264	6.2	15.4	41	68	62	51
634.76	638.68	3.92	GRV	249	18.1	90.7	42	79	69	66
638.68	643.47	4.79	GN	264	100.0	500.0	100	90	80	76
643.47	643.58	0.11	GRV	249	168.1	840.7	62	94	83	84
643.58	648.68	5.10	GN	232	100.0	500.0	100	90	80	76
648.68	648.98	0.30	GRV	249	58.8	293.9	56	87	77	82
648.98	649.16	0.18	AN	249	100.0	500.0	100	90	80	76
649.16	649.55	0.39	GRV	249	100.0	500.0	67	90	80	85
649.55	660.35	10.80	GN	264	66.7	333.3	78	88	77	80
660.35	660.54	0.19	GRV	249	12.9	64.7	50	77	67	57
660.54	665.54	5.00	GN	264	16.7	83.3	62	79	68	71
665.54	665.87	0.33	GRV	249	200.0	1000.0	74	95	85	92
665.87	669.19	3.32	GN	264	100.0	500.0	78	90	80	85
669.19	669.94	0.75	AN	249	100.0	500.0	71	90	80	85
669.94	684.94	15.00	GN	264	66.7	333.3	79	88	77	80
684.94	690.20	5.26	GN	264	25.0	125.0	74	81	71	67
690.20	694.50	4.30	GN	264	5.1	12.9	43	67	61	61
694.50	694.85	0.35	GRV	249	100.0	500.0	100	90	80	76
694.85	710.04	15.19	GN	264	18.5	92.6	76	79	69	72
710.04	710.60	0.56	GRV	249	66.7	333.3	63	88	77	80
710.60	723.86	13.26	GN	264	37.5	187.5	75	84	74	81
723.86	724.54	0.68	GN	264	2.5	6.4	39	62	56	41
724.54	731.54	7.00	GN	264	44.4	222.2	78	85	75	83
731.54	733.63	2.09	GRV	231	16.7	83.3	55	79	68	71
733.63	736.84	3.21	GN	264	16.5	82.3	58	79	68	70
736.84	736.96	0.12	GN	264	0.4	1.1	28	50	44	20
736.96	741.44	4.48	GN	264	4.4	11.1	56	66	60	66
741.44	741.88	0.44	AN	249	91.5	457.4	57	90	79	72
741.88	743.91	2.03	AN	249	11.8	59.2	59	77	66	67
743.91	753.83	9.92	GN	264	23.3	116.7	80	81	71	75
753.83	755.19	1.36	AN	249	24.8	124.1	68	81	71	67
755.19	755.71	0.52	N	223	100.0	500.0	100	90	80	76
755.71	758.78	3.07	GN	264	12.1	60.5	55	77	66	67

From (m)	To (m)	Interval Length (m)	Geology	UCS	Q	Q_{wall}	RMR (L90)	RMR Barton From Qwall	RMR Barton From Q	GSI
758.78	759.93	1.15	AN	249	100.0	500.0	100	90	80	76
759.93	761.81	1.88	N	223	80.0	400.0	100	89	79	76
761.81	762.70	0.89	AN	249	57.7	288.3	51	87	76	73
762.70	764.43	1.73	N	223	26.7	133.3	58	82	71	80
764.43	764.83	0.40	GN	264	100.0	500.0	100	90	80	76
764.83	765.15	0.32	AN	249	100.0	500.0	100	90	80	76
765.15	765.52	0.37	GN	264	100.0	500.0	100	90	80	76
765.52	766.06	0.54	AN	249	100.0	500.0	100	90	80	76
766.06	766.33	0.27	N	223	80.0	400.0	100	89	79	76
766.33	767.00	0.67	AN	249	100.0	500.0	100	90	80	76
767.00	767.36	0.36	GN	264	100.0	500.0	100	90	80	76
767.36	767.99	0.63	AN	249	100.0	500.0	100	90	80	76
767.99	770.03	2.04	N	223	13.3	66.7	65	77	67	63
770.03	770.82	0.79	AN	249	66.7	333.3	65	88	77	80
770.82	772.11	1.29	N	223	80.0	400.0	100	89	79	76
772.11	772.43	0.32	AN	249	100.0	500.0	100	90	80	76
772.43	778.89	6.46	N	223	80.0	400.0	100	89	79	76
778.89	780.04	1.15	N	223	6.1	15.3	47	68	62	69
780.04	781.68	1.64	AN	249	10.0	50.0	100	75	65	31
781.68	786.30	4.62	N	223	80.0	400.0	100	89	79	76
786.30	787.50	1.20	AN	249	95.0	475.0	100	90	80	73
787.50	787.79	0.29	GN	264	100.0	500.0	100	90	80	76
787.79	794.20	6.41	FPX	194	4.5	11.3	54	66	60	68
794.20	794.45	0.25	AN	249	16.7	83.3	57	79	68	63
794.45	794.61	0.16	FPX	194	2.0	5.0	39	60	55	22
794.61	794.70	0.09	AN	249	100.0	500.0	100	90	80	76
794.70	804.58	9.88	FPX	194	5.9	14.7	64	68	62	62
804.58	804.75	0.17	FPX	194	0.2	0.4	30	44	38	19
804.75	809.07	4.32	FPX	194	4.1	10.2	56	65	59	66
809.07	809.22	0.15	FPX	194	9.4	23.5	47	71	65	41
809.22	812.63	3.41	FPX	194	26.7	133.3	66	82	71	71
812.63	813.00	0.37	FPX	194	10.0	25.0	54	71	65	60
813.00	813.96	0.96	FPX	127	80.0	400.0	94	89	79	76
813.96	814.43	0.47	FPX	194	4.4	11.1	49	66	60	46
814.43	818.21	3.78	GRV	249	6.6	16.4	55	68	62	69
818.21	818.57	0.36	FPX	127	13.3	66.7	52	77	67	63
818.57	828.60	10.03	FPX	194	3.9	9.7	53	65	59	66
828.60	828.74	0.14	OPX	194	80.0	400.0	100	89	79	76
828.74	829.37	0.63	FPX	194	80.0	400.0	100	89	79	76
829.37	830.02	0.65	MPV	132	80.0	400.0	94	89	79	76
830.02	830.84	0.82	OPX	194	26.7	133.3	59	82	71	71
830.84	831.65	0.81	MPV	132	13.3	66.7	51	77	67	63
831.65	840.84	9.19	N	223	4.0	10.1	56	65	59	66
840.84	846.04	5.20	N	223	8.3	20.8	61	70	64	65
846.04	853.10	7.06	N	223	3.1	7.8	56	63	57	62

From (m)	To (m)	Interval Length (m)	Geology	UCS	Q	Q_{wall}	RMR (L90)	RMR Barton From Qwall	RMR Barton From Q	GSI
853.10	854.77	1.67	N	223	4.5	11.2	43	66	60	55
854.77	855.74	0.97	N	223	10.0	25.0	59	71	65	60
855.74	856.37	0.63	N	223	14.9	74.7	45	78	68	55
856.37	857.79	1.42	N	223	80.0	400.0	100	89	79	76
857.79	861.85	4.06	N	223	10.0	25.0	60	71	65	60
861.85	862.78	0.93	N	223	80.0	400.0	100	89	79	76
862.78	864.21	1.43	N	223	53.3	266.7	60	86	76	80
864.21	868.20	3.99	N	223	10.0	25.0	64	71	65	60
868.20	869.80	1.60	N	223	2.2	5.4	40	61	55	56
869.80	871.85	2.05	N	223	80.0	400.0	76	89	79	85
871.85	872.03	0.18	N	223	2.7	6.7	34	62	56	35
872.03	874.57	2.54	N	223	4.3	10.9	50	66	60	66
874.57	875.09	0.52	N	223	80.0	400.0	100	89	79	76
875.09	875.37	0.28	N	223	80.0	400.0	100	89	79	76
875.37	876.27	0.90	N	223	12.9	64.7	53	77	67	69
876.27	876.67	0.40	N	223	1.8	4.5	35	60	54	38
876.67	878.93	2.26	N	223	106.7	533.3	79	91	80	88
878.93	879.10	0.17	MPV	132	101.3	506.6	61	91	80	85
879.10	884.22	5.12	N	223	4.2	10.5	56	65	59	67
884.22	887.91	3.69	N	223	3.2	7.9	50	63	58	64
887.91	888.10	0.19	MPV	132	80.0	400.0	94	89	79	76
888.10	889.05	0.95	N	161	1.8	4.6	41	60	54	46
889.05	890.68	1.63	N	223	10.5	52.6	55	76	65	68
890.68	891.16	0.48	N	223	13.3	66.7	61	77	67	63
891.16	891.82	0.66	N	223	49.2	245.9	67	86	75	84
891.82	892.03	0.21	N	223	9.3	23.2	49	70	65	57
892.03	893.40	1.37	N	223	10.0	25.0	56	71	65	60
893.40	893.73	0.33	N	161	0.5	1.2	37	51	45	20
893.73	894.58	0.85	N	223	13.2	66.0	56	77	67	62
894.58	895.40	0.82	N	223	0.0	0.0	34	30	30	14
895.40	899.66	4.26	N	223	2.0	4.9	53	60	54	65
899.66	899.76	0.10	N	161	0.3	0.8	23	49	43	12
899.76	901.62	1.86	N	223	14.7	73.7	61	78	68	72
901.62	902.26	0.64	N	223	80.0	400.0	100	89	79	76
902.26	903.02	0.76	N	161	0.1	0.1	31	35	35	20
903.02	903.62	0.60	N	223	20.0	100.0	57	80	70	67
903.62	904.05	0.43	N	161	0.3	0.8	30	49	43	25
904.05	904.60	0.55	N	223	106.7	533.3	70	91	80	88
904.60	905.90	1.30	N	223	20.0	100.0	73	80	70	67
905.90	909.25	3.35	N	161	2.1	5.2	39	61	55	58
909.25	909.68	0.43	N	161	0.1	0.1	29	34	34	12
909.68	910.49	0.81	N	223	53.3	266.7	66	86	76	80
910.49	910.63	0.14	N	223	0.7	1.7	35	53	47	26
910.63	917.00	6.37	N	223	8.4	21.1	58	70	64	65
917.00	925.54	8.54	N	223	12.1	60.5	60	77	66	70

From (m)	To (m)	Interval Length (m)	Geology	UCS	Q	Q_{wall}	RMR (L90)	RMR Barton From Qwall	RMR Barton From Q	GSI
925.54	926.18	0.64	HF	273	17.5	87.4	51	79	69	61
926.18	941.05	14.87	N	223	20.7	103.7	72	80	70	86
941.05	946.39	5.34	N	223	9.4	23.5	60	71	65	67
946.39	947.04	0.65	N	223	38.2	190.9	56	84	74	82
947.04	950.26	3.22	N	223	20.0	100.0	66	80	70	67
950.26	953.64	3.38	HF	273	80.0	400.0	100	89	79	76
953.64	960.05	6.41	HF	273	40.0	200.0	75	85	74	76
960.05	960.67	0.62	N	223	30.0	150.0	65	83	72	72
960.67	967.11	6.44	N	223	5.8	14.5	49	67	61	71
967.11	968.31	1.20	N	223	16.0	80.1	47	79	68	57
968.31	983	14.69	N	223	5.2	13.1	55	67	61	69
983.00	995.1	12.10	N	223	4.4	11.1	56	66	60	67
995.10	1002.52	7.42	N	223	4.2	10.6	49	65	59	67
1002.52	1003.6	1.08	GRV	249	120.0	600.0	73	92	81	89
1003.60	1005.6	2.00	N	223	106.7	533.3	73	91	80	88
1005.60	1005.78	0.18	GRV	249	80.0	400.0	100	89	79	76
1005.78	1008.42	2.64	N	223	80.0	400.0	100	89	79	76
1008.42	1009.17	0.75	GRV	249	53.3	266.7	63	86	76	80
1009.17	1010.24	1.07	N	223	80.0	400.0	100	89	79	76
1010.24	1011	0.76	HF	273	80.0	400.0	100	89	79	76
1011.00	1025.03	14.03	N	223	10.0	50.0	57	75	65	67
1025.03	1037.08	12.05	N	223	5.0	12.5	63	66	60	56
1037.08	1040.53	3.45	N	223	56.8	284.1	100	87	76	62
1040.53	1041.14	0.61	HF	273	10.1	50.4	50	76	65	39
1041.14	1050.89	9.75	N	223	10.0	50.0	56	75	65	67
1050.89	1052.63	1.74	N	223	10.0	25.0	61	71	65	67
1052.63	1056.07	3.44	N	223	80.0	400.0	100	89	79	76
1056.07	1056.81	0.74	HF	273	80.0	400.0	100	89	79	76
1056.81	1062.07	5.26	N	223	14.7	73.7	64	78	68	72
1062.07	1062.62	0.55	N	223	80.0	400.0	100	89	79	76
1062.62	1063.18	0.56	N	223	80.0	400.0	100	89	79	76
1063.18	1063.94	0.76	N	223	10.8	54.1	45	76	66	50
1063.94	1067.15	3.21	N	223	7.8	19.6	55	69	63	65
1067.15	1067.38	0.23	N	223	28.4	142.2	52	82	72	70
1067.38	1068.83	1.45	N	223	80.0	400.0	73	89	79	85
1068.83	1073.53	4.70	N	223	4.9	12.2	51	66	60	67
1073.53	1074.2	0.67	GRV	249	12.9	64.3	46	77	67	68
1074.20	1074.95	0.75	N	223	80.0	400.0	100	89	79	76
1074.95	1088.1	13.15	N	223	7.8	19.4	52	69	63	65
1088.10	1092.8	4.70	N	223	10.3	51.4	56	76	65	68
1092.80	1095.1	2.30	HF	273	4.5	11.2	44	66	60	64
1095.10	1095.29	0.19	N	223	80.0	400.0	100	89	79	76
1095.29	1095.46	0.17	HF	273	80.0	400.0	100	89	79	76
1095.46	1097.77	2.31	N	223	1.3	3.2	42	57	52	54
1097.77	1098.22	0.45	HF	273	80.0	400.0	100	89	79	76

From (m)	To (m)	Interval Length (m)	Geology	UCS	Q	Q_{wall}	RMR (L90)	RMR Barton From Qwall	RMR Barton From Q	GSI
1098.22	1100.07	1.85	N	223	80.0	400.0	100	89	79	76